dam world 2025

Fifth International
Dam World Conference

Preserving life through dams

Proceedings of the Fifth International DAM WORLD Conference

Volume 2

edited by:
Carlos Pina
Eliane Portela
Laura Caldeira

LABORATÓRIO NACIONAL
DE ENGENHARIA CIVIL

PORTUGAL • LISBON • LNEC

SPONSORS

SILVER

BRONZE

LABORATÓRIO NACIONAL
DE ENGENHARIA CIVIL

dam world 2025

PORTUGAL • LISBON • LNEC

Proceedings of the
Fifth International DAM WORLD Conference
Preserving life through dams

Volume 2

edited by:
Carlos Pina, Eliane Portela, Laura Caldeira

Legal Notice
The scientific quality and content of the papers published herein are of the entire responsibility of their authors. Therefore, the publisher accepts no responsibility or liability whatsoever with regard to the information contained in this publication.
The total or partial reproduction, and any representation of the substantial or partial content of the present publication, by any process or means whatsoever, including copy or transfer, without the express consent of the publisher, is prohibited and constitutes an infringement sanctioned by the Law.

Copyright © LABORATÓRIO NACIONAL DE ENGENHARIA CIVIL, I. P.
Divisão de Divulgação Científica e Técnica
AV DO BRASIL 101 • 1700-066 LISBOA
e-e: livraria@lnec.pt
www.lnec.pt

Publisher: LNEC

Collection: National and international Meetings

Series: RNI 118

1st. edition: 2025 (pdf)

Descriptors: Dams safety / Dams behaviour / Concrete dam / Embankment dam / International congress
Descritores: Segurança de barragens / Comportamento de barragens / Barragem de betão / Barragem de aterro / / Congresso internacional

CDU 627.82(063)(100)
ISBN 978-972-49-2332-1
DOI 10.34638/084d-e697

Preface

The DAM WORLD Conference Series serves as a leading forum for sharing the latest scientific and technical advancements, as well as discussing the key challenges and solutions facing the dam engineering sector.

The inaugural DAM WORLD Conference took place in Brazil in 2012. The subsequent events, held inPortugal in 2015 and Brazil in 2018, were both highly successful. The Fourth International DAM WORLD Conference, originally scheduled as an in-person event in Lisbon in 2020, was instead held as an online event – the DAM WORLD Webinar – due to the global pandemic.

The National Laboratory for Civil Engineering (LNEC) is honoured to host the Fifth International DAM WORLD Conference. Continuing the tradition of its predecessors, this event will provide a platform for delegates from around the world to exchange valuable knowledge and expertise. The theme of DW2025 is "Preserving Life Through Dams," and we are committed to exceeding your expectations. Canada will be the invited country, with distinguished experts and representatives sharing their insights and advancements in dam engineering.

In addition to technical sessions, the conference will feature keynote lectures and pre-conference oneday courses, all designed to enhance professional practice.

The proceedings of the Fifth International DAM WORLD Conference will be published in two volumes. All included papers have undergone a rigorous peer-review process, and only those that were accepted will be presented at the conference and included in the official proceedings.

The Conference Organising Committee would like to extend their appreciation to all the authors for their contributions and dedication in preparing and submitting their papers. We also wish to express our sincere gratitude to the members of the Scientific Committee for their thorough and thoughtful review of the papers featured in these proceedings.

Laura Caldeira
Conference Chair

Organisation

Conference Chair
Laura Caldeira, Portugal

Conference Co-Chairs
António Batista, Portugal
João Bilé Serra, Portugal
José Marques Filho, Brazil
Miguel Augusto Zydan Sória, Brazil
Teresa Viseu, Portugal
Túlio Bittencourt, Brazil

Organising Committee
Andrea Brito, Portugal
Carlos Serra, Portugal
Eliane Portela, Portugal
José Lucas Marques, Brazil
José Rocha Afonso, Portugal
Sílvia Amaral, Portugal

Scientific Committee

President

Carlos Pina, Portugal

Ahmed Chraibi, Morocco
Alberto Marulanda, Colombia
Alberto Sayão, Brazil
Ali Noorzad, Iran
Andy Small, Canada
Anthony Burnett, Canada
António Bettencourt Ribeiro, Portugal
António Pinheiro, Portugal
António Tavares de Castro, Portugal
Bettina Geisseler, Germany
Caius Priscu, Canada
Carlos Fava, Argentina
Carlos Granell, Spain
Claudio Fattor, Argentina
Daniele Cazzuffi, Italy
David Rivera Pantoja, Spain
Devendra Kumar Sharma, India
Diego Antônio Fonseca Balbi, Brazil
Domingos Silva Matos, Portugal
Enrique Cifres, Spain
Etienne Grimal, France
Feng Jin, China
Fernanda Rocha, Portugal
Francesco Federico, Italy
Gensheng Zhao, China
Gerald Zenz, Austria
Giovanni De Cesare, Switzerland
Giovanni Ruggeri, Italy
Gonçalo Tavares, Portugal
Grégory Coubard, France
Humberto Marengo-Mogollon, Mexico
Ignacio Escuder, Spain
Isabelle Levesque, Canada
Jean-Pierre Tournier, Canada
João Cunha, Portugal
João Marcelino, Portugal
João Portugal, Portugal
Jorge Saldanha Matos, Portugal
Jorge Vasquez, Portugal
José Falcão de Melo, Portugal
José Machado do Vale, Portugal
José Mateus de Brito, Portugal
José Vieira de Lemos, Portugal
Louis Hattingh, South Africa
Luca Pagano, Italy
Luis de la Torre Abietar, Spain
Luís Lamas, Portugal
Luiz Paniago Neves, Brazil
Lurdes Pimenta, Portugal
Marc Smith, Canada
Marcelo Giulian Marques, Brazil
Marco André Argenta, Brazil
Margaret Trias, Canada
Paulo Castro, Portugal
Ricardo Pimentel, Portugal
Robin Charlwood, USA
Rogério Menescal, Brazil
Sandra Soares Frazão, Belgium
Stefan Felder, Australia
Teresa Fusaro, Brazil
Tetsuya Ishida, Japan
Tony Bennett, Canada
Travis Bowser, Canada

Conference Themes

Main Themes

T1 Concrete and Masonry Dams
T2 Embankment Dams and Levees
T3 Hydraulics for Dams
T4 Tailings Dams
T5 Environmental and Social Issues
T6 Regulation and Governance
T7 Dams and Climate Change

Topics

ST1 Design and Modelling
ST2 Design Innovation and New Materials
ST3 Geology and Dam Foundation
ST4 Underground Works
ST5 Hydrology and Appurtenant Works
ST6 Seismic Analysis
ST7 Instrumentation and Monitoring
ST8 Dam Safety Evaluation
ST9 Construction and Rehabilitation
ST10 Operation and Maintenance
ST11 Deterioration
ST12 Decommissioning and Dam Removal
ST13 Dam Management Systems and New Technologies
ST14 River and Integrated Reservoir Management
ST15 Risk Assessment
ST16 Dam Failure and Emergency Action Plans (EAP)
ST17 Sedimentation in Reservoirs
ST18 Dams and Climate Change Adaptation
ST19 Hydro, Solar and Wind Energy Associated to Dams

Index

Volume 1

Keynotes

Wadi Derna Disaster (Lybia Sept. 2023) 1
Lino, Michel; Chraibi, Ahmed Fouad

Water Scarcity, Climate Change Adaptation, and the Role of Dams and Reservoirs 3
Oliveira, Rodrigo

Deterioration of Dams Affected by Concrete Swelling. The Role of Observation and Modelling in Safety Control and Rehabilitation 27
Lopes Batista, António

Geomechanical Behaviour of Tailings Storage Facilities 65
Świdziński, Waldemar

Concrete Dams and Earthquake: Numerical Analysis Techniques and Practical Advices for Engineers 67
Robbe, Emmanuel

The Failure Modes of Flood Protection Levees, Consequences on Management and Engineering 99
Tourment, Rémy

Special Session Canada

Simplified Analysis of Selected ASR PFMS 117
Curtis, Dan

Canadian Dam Association and Dams in Canada 119
Smith, Marc; Trias, Margaret

Failure Modes: a Canadian Perspective, Case Study on Internal Erosion 121
Smith, Marc

Tailings Dam Safety - Developments in Canada 123
Al-Mamun, Mohammad; Burnett, Anthony

Dam Safety Management Systems 125
Bennett, Tony

Public Safety Around Dams: Practical Guidance for Meeting Owner Responsibilities in Canada 127
Bennett, Tony

Panel Discussion: Challenges in Developing Dam Safety Guidelines 129
Priscu, Caius

Tailings Dam Safety- Developments in Canada 131
Burnett, Anthony

T1 Concrete and Masonry Dams

The Frost and Leaching Deterioration of Dams in Altitude: Case Studies 133
Paglia, Christian; Antonietti, Samuel

Rock-Filled Concrete (RFC) Arch Dams With or Without Contraction Joints 147
Jin, Feng; Zhang, Quanyi

Rehabilitation Study of Temengor and Chenderoh Dam for Their Safety Enhancement and Life Extension 157
Siskos, Ioannis; Du, Chongjiang

Dam FPO Geomembrane After 14 Years Exposure 169
Favre, Pierre; Eckl, Martin

The Strength at the Base of an Old Concrete Dam, Based on the Spanish Experience With Large-Scale In Situ Shear Tests 181
Asanza Izquierdo, Enrique; Laina Gómez, Carlos; Díez Torres, Juan Antonio; Escudero Merino, Diego; Antón Sempere, José

Structural Solution for the Rehabilitation of Spillway Piers Subjected to Corrosion of the Prestressing Cables in the Trunnion Girder 201
Dalcin, Marcos; Takase, Kauê; Generoso, Bianca

Design and Construction of Hardfill Cofferdam on River Alluvium Deposits 213
Du, Chongjiang; Beck, Sebastian; Jung, Uysung

Pathology Mapping in Concrete Dams through Digital Photogrammetry and Image Processing 223
Roque, Dora; Henriques, Maria João

Safety Verification by 3D Dynamic Analysis of a 100-Year-Old Masonry Arch–Gravity Concrete Dam 233
Iwata, Naoki; Kiyota, Ryoji; Kawasaki, Hideaki; Kurihara, Masaharu

Continuous Ambient Vibration Measurements: Experimental Set Up and First Results on an Arch Dam 249
Steck, Barthelemy; Robbe, Emmanuel; Humbert, Nicolas

Preventive Maintenance with ROV and Sonar Underwater Inspections: a Strategic Approach to Reducing Downtime and Costs in Hydraulic Tunnels Failures 269
Iannotta, Fabio; Malak, David

Automated Defect Change Detection and Quantification for Large Gravity Dams and Spillways – a Pathway to Deterioration Modelling 287
Fowler, Joshua; Aparicio, Javier

A Study on the Influence of the Reservoir Water Level on the Seismic Performance of a Large Double Curvature Arch Dam 303
Oliveira, Sérgio; Ramos, Rafael; Alegre, André; Proença, Jorge; Mendes, Paulo; Silva, João; Serra, Carlos

From Sensor Data to Actionable Information with KMIDam. Seismic Event Reporting and Long-Term Structural Health Monitoring of Dams 323
Alegre, André; Skolnik, Derek; Ciudad-Real, Mauricio; Franke, Mathias; El Idrissi, Mohamed

Dynamic Behaviour of a Concrete Dam: Development of Statistical and Machine Learning Models for Interpretation of Monitoring Data 343
Sejas, Andres Mauricio Guzman; Pereira, Sérgio; Mata, Juan; Cunha, Álvaro

How Effective is Monitoring Concrete Dams with GNSS in 3D Mode? 357
Lima, José Nuno

Analysis of the Dynamic Characteristics of Trapezoidal CSG Dams Using Seismic Records 377
Yasuda, Nario; Cao, Zengyan; Kozaki, Yuya

Creep Tests in Daivões Dam 389
Serra, Carlos; João, Silva

Local and Global Parameter Sensitivity for Concrete Hydraulic Structures Under Seismic Loading Considering Structure and Material Variation 407
de AlmeidaTorres Filho, Rodrigo José; Segura, Rocio; Paultre, Patrick

GIS-Based Database for Improved Dam Management in Cabo Verde 427
Correia e Silva, Carlos Filipe; Tavares, Emanuel; Delgado, Adlisa; Fernandes, João Nuno; Dias, Ivo Figueiredo

Relations Between Seismic Input Motion, Dam Pattern, and Seismically Induced Sliding of Gravity Dams in European Context 437
Labbé, Pierre

Analysis and Interpretation of the Structural Behaviour of Daivões Dam During the First Filling of the Reservoir 455
Serra, Carlos; Mata, Juan

Contributions to the Analysis of the Exothermic-Mechanical Phenomenon in Mass Concrete 475
Mamede Pereira Gomes, Flávio

Evaluation Method for Safety of Concrete Slab Stress Based on Multi-axial Strength Criterion 503
Mi, Zhankuan; Wei, Kuangmin; Wang, Fang

Adaptive Warning Thresholds in Dam Safety Using Kernel Density Estimation 523
Silva Cancino, Nathalia; Irazabal, Joaquín; Salazar, Fernando; Mata, Juan

Rehabilitation of Sordo Dam 535
Costa, André; Cordeiro, José; Vale, José Machado do; Nicolau, Luís; Gonçalves, Sofia

Influence of Fine Materials Addition on the Coefficient of Thermal Expansion of Repair Mortars 571
Goulart Gomes de Oliveira, Rafaela; Reginato, Luan; Funchal de Faria, Étore; Luiz Willrich, Fábio; Parapinski dos Santos, Ana

Development of Improvements in Flow Meter Devices for Dam Monitoring 581
Antônio Sucapuca Aracayo, Luis; João Arenhardt, Bruno; Miranda Rojas, Ricardo; Liska Darli, Diego; Reginato, Luan

Hydrological Risk of Floods and Collapse due to Overtopping of the Ribeira de Principal Dam on Santiago Island in Cape Verde 595
Fernandes, Manuel; Silva, Francisco

Deterioration Due to Simultaneous Expansive Reactions in Dam Concrete: Evaluations on Prototypes 613
Funahashi Júnior, Eduardo Issamu; Pagan Hasparyk, Nicole; Gambale, Patrícia; Possan, Edna; Chapira Kuperman, Selmo

Yes, There Appear to be Options to Stop or Control ASR in Some Dams! 635
Charlwood, Robin; Godart, Bruno; Isgor, Burkan; Grimal, Etienne

A 3D Point Cloud Model of Santa Luzia Dam and Its Contribution for Improving the Finite Element Mesh 657
Henriques, Maria; Dias, Ivo; Roque, Dora; Gomes, Alexandre

Applications of GNSS Technology in Dam Monitoring: a Systematic Literature Review 673
Moreira, Hanna Barreto de Araújo Falcão; Quental Coutinho, Roberto; Silva Alves, Jailson; Aragão Júnior, Wilson Ramos

Dam Safety: How a Good Contractual Design and a Thorough Project Execution Management Can Contribute to it 687
Geisseler, Bettina

Caculo Cabaça Hydropower Project - Evaluation of the Susceptibility of Aggregates for Participation in Expansive Alkali-silica Reaction 695
Custódio, João; Bettencourt Ribeiro, António; Chico, Augusto; Costa, Cristina; Mateus, Gonçalo; Liu, Wen

River Diversion of the Caculo Cabaça Hydropower Project 719
Costa, Maria Cristina; Mateus, Gonçalo; Amador, António; Morgado, António; Chico, Augusto; Zebin, Xiong; Wen, Liu

LNEC Support to the Design Adaptation of Alto Tâmega Dam Due to the Rock Slide on the Right Bank Slope 739
Muralha, José; Lamas, Luís; Batista, António Lopes; Mata, Juan; Serra, Carlos

First Infilling of the Underground Structures of the Gouvães Hydroeletric Scheme - Monitoring and Observed Behaviour 759
Pereira, Renato; Muralha, José; Lamas, Luís

Assessment of Concrete Dam Foundation Discharges for Different Reservoir Levels Following a Hydromechanical Discrete Model 775
Braga Farinha, Maria Luísa; Monteiro Azevedo, Nuno; Oliveira, Sérgio

Stability Analysis of Concrete Dam Foundations Following a Hybrid Discrete Element/Finite Element Approach 791
Monteiro Azevedo, Nuno; Braga Farinha, Maria; Oliveira, Sérgio

Implementation of 3D Thermal Models in the Construction of the Alto Tamega Dam 811
Capinha, Luis; Chacón LLorente, Rafael; Siguenza Ibañez, Alicia; de la Torre Abietar, Luis; Vinagre Alba, Emilio José

T2 Embankment Dams and Levees

Prediction Models for the Hydraulic Behavior of Irapé Dam 833
Marcelino, João; Divino, Paula Luciana; Balbi, Diego

Notes on the Application of New Zealand's 2022 NSHM in Slope Stability 845
Dirikolu, Tarik; Zabolotnii, Elena; Sivathayalan, Siva; Read, John; Venter, Julian

Tropical Lateritic and Saprolitic Soils – The Brazilian Experience 863
Freitas Jr, Manoel; Pastore, Eraldo

Micro Seismic Resonance (MSR) - A Helpful and Complimentary Tool for Mapping Seepage Through, Beneath and Around Dams, Embankments & Levees 883
K. Hughes, Andy; Blanchard, Ryan

The Use of Leakage Detection Systems to Improve Reservoir Safety and Direct Remedial Works 901
Hughes, Andy

Selection of the Critical State Parameter M for Deformation Analysis Under Generalized Stress Paths 915
Murali, Aravind; Bangera, Vinayak; Zabolotnii, Elena; Sivathayalan, Siva

Advanced Geomembrane Systems for Dams and Reservoirs of New Pumped Hydro Storage Schemes 929
Vaschetti, Gabriella; Jackson, A.; Blanco, Alejandro

Experimental Investigation of Sediment Propagation in case of Homogeneous Earth-fill Dam Break by Piping 945
Büyüker, Zehra; Taşkaya, Ebru; Bombar, Gökçen; Tayfur, Gökmen

Embankment Dam Monitoring Using Electrical Resistivity Tomography for Abnormal Seepage Analysis 965
Abdulamit, Altan; Pisculidis, Tudor; Obreja, Razvan

Integration and Challenges in the Slope Health Monitoring System (SHMS) 985
Carneiro, Samuel; Guimarães, Filipe; Penha, Iggor; Faria, Silvia; Thimotio, Flavio; Souza, Camila; Pereira, Vinicius

Codegua, a Compendium of the Chilean Experience in CFRD Dams 999
San Martin, Luis; Materon, Bayardo

Development of an Integrated System for Risk Management and Decision-Making: an FMEA/FMECA and PFMA-Based Approach 1019
Etchegaray Mendes, Daniel

Simplified Analytical Solutions of Free Surface Migration Within Originally Dry, Coarse Grain Sized, Embankment Dam Materials 1039
Federico, Francesco; Cesali, Chiara

Advanced Limit Equilibrium 2D Analyses to Assess Global Sliding Safety of Embankment Dams Under Seismic Actions 1059
Federico, Francesco; Cesali, Chiara; Caushi, Herald

In-situ Estimation of Breach Outflow Hydrographs From Fluvial Dike Failures: a Methodology Integrating Real-time Monitoring and Physical Modelling 1077
Jónatas, Ricardo; Amaral, Sílvia; Aleixo, Rui; Bilé Serra, João; Ferreira, Rui

Lime Treatment for Hydraulic Structures: Benefits, Challenges, and Long-Term Performance 1101
Bertola, Federica; Nicaise, Sylvie; Saussaye, Lucile; Verelst, Kristof; van Hemert, Henk

Brief Insights: In-situ Permeability Test in Weathered and Poorly-Graded Rockfill 1117
Laina-Gómez, Carlos; Escudero-Merino, Diego; Asanza-Izquierdo, Enrique; Díez Torres, Juan Antonio

The Design and Construction of 71 Ndúe Dam – Angola 1129
Costa, Rui; Sousa Cruz, Jorge; A. Henriques, Nuno

Main Failure Modes in Embankment Dams, Mappable with Drone Survey 1145
Leite, Daniel; Marcelino, João; Guerra, Nuno; Manso, João; Sardinha, Ricardo

Construction of an Asphalt Core Dam in a Seismic Zone 1163
Graça, Newton; Moreira, Jose; Carleto, Bruno; Pantoja, Tarcisio

Geological and Geotechnical Studies in Dams Design. Foundation, Reservoir and Construction Materials 1181
Roque, Magda

Application of a Cost-Effective In-Place Inclinometer System on a Zoned Earth Dam 1197
Palma, João; Correia dos Santos, Ricardo; Marcelino, João; Marques, Nuno

Setting an Earthquake Magnitude Threshold and Evaluating its Impact on a Rockfill Dam 1219
Pandey, Binaya Raj; Zenz, Gerald

Occurrence of Canaliculi and Treatments in the Foundations of the Belo Monte Hydroelectric Complex During Construction and Operation 1233
Braga Boaventura, Marcelo; Pimenta de Ávila, Joaquim; Abrahão, Ricardo Antonio; Charone Junior, Wady; Lellis dos Santos, Wagner; Diogo Krajewski, Erlene; Carvalho Lima, Leandro

Developing a Remote Sensing Tool for Reservoir Volume Evaluation 1249
Pimenta, João; Fernandes, João; Azevedo, Alberto

Optimizing Seismic Intensity Measures for Fragility Analysis of Earth Dams: a Comprehensive Evaluation Incorporating Functionality Metrics 1259
Segura, Rocio; Veylon, Guillaume

Geostatistical Spatial Prediction Models of Resurgence and Canaliculus in the Dikes Surrounding the Intermediate Reservoir of the Belo Monte Hydroelectric Plant 1277
Silva Junior, Edivaldo; Fernandes, Débora; Januário, Adriana; Rodrigues, Rodrigo; Reginato, Luan; Concepcion, Candy; Krajewski, Erlene; Boaventura, Marcelo; Lima, Leandro

The Effects of Hypothetical Ochre Clogging of Drainage System on Slope Stability of the Itumbiara Dam, Brazil 1291
Mendonça, Marcos Barreto; Cavalcanti, Maria do Carmo Reis; Machado, Gabriela Wolguemuth

Let's Talk About FMEA 1303
Fusaro, Teresa Cristina; Vaz de Melo, Alexandre; Etchegaray Mendes, Daniel

Quxue Dam 2D FEM Analysis - Asphalt Core Displacements Comparison with Similar Structures 1321
Cavalcanti, Maria do Carmo; Barreto de Mendonça, Marcos; Silva Drumond, Fernanda

Numerical Simulation of Dam Behavior During the Reservoir Filing Until Reaching Steady-State Conditions 1337
da Conceição Rodriguez Moreira, Pablo Henrique; Cavalcanti, Maria do Carmo; Barreto de Mendonça, Marcos

Arching Effect Induced by a Concrete Gallery Inside Tullaroop Dam - Three-dimensional Finite Element Analysis 1357
Nogueira Rodrigues, Gabriel; Cavalcanti, Maria do Carmo

InSAR Monitoring and FEM Modeling of the Vadomojón Earthfill Dam (Córdoba, Spain) 1373
Esteban Mateos, Elías; González Tejada, Ignacio; Lamas-Fernández, Francisco; Ruiz-Armenteros, Antonio; Marchamalo-Sacristán, Miguel; Martínez-Marín, Rubén; Sancho, Candela; Fernández-Landa, Alfredo; Granados, Alfredo; Granados, Isabel; Jorda, Luis

A CFD Account of the Mean Flow and Turbulence over Breached Embankment Dam 1387
L Ferreira, Rui M; Martins, Nuno M; Alvarez, Teresa; Viseu, Teresa

Critical Aspects of Using Vibrating Wire Piezometers in Geotechnical Monitoring of Dams: How to Meet Basic Operating Principles of the Devices and Avoid Common Errors that Compromise Monitoring 1405
Bastos, Wellerson; Pimenta Filho, Marcos; de Almeida, Luiza; Ferreira, Marina; Santos Junior, Mauro

Reinforcement of the Foundation Waterproofing of the Alvorninha Dam 1427
Romeiro, Manuel; Candeias Portugal, João

Drone Survey of an Embankment Dam To Detect Surface Changes 1441
Henriques, Maria; Silva, Hugo

A Detailed Safety and Risk Assessment of Pequenos Libombos Dam 1459
Falcao de Melo, Jose; Sardinha, Ricardo; Pimenta, Lurdes; Duarte, Alexandre; M. Freitas, Rui; Juízo, Dinis; Martins, Amiro; Sengo, D.; Timba, J.; Mabunda, Luís

A Detailed Safety and Risk Assessment of Massingir Dam 1479
Sardinha, Ricardo; Falcao de Melo, Jose; Pimenta, Lurdes; Duarte, Alexandre; M. Freitas, Rui; Juízo, Dinis; Martins, Amiro; Miguel, F.; Cuna, I.; Mabunda, Luís

Design of Raghai Embankment Dam in a Complex Geological and Geotechnical Framework (Tunisia) 1501
Rebelo, Virgílio; Antunes, Joana; Morgado, António; El Ghali, Abdessalem; Yahia, Ali; Pinto, Sónia; Naguindás, Febin; Khlifi, Rabii

Calucuve Dam – Geotechnical Conditions and Design 1525
Sardinha, Ricardo; Badry, Hesham; Pimenta, Lurdes; Abreu, Rui; Leite, Daniel; Custódio, Hugo; Naguindás, Febin

Design, Supervision, and Technical Assistance to Construction of IRCL Southern Expansion 1545
Antunes, Joana; Repsold, Lucas; Almeida, Lúcia; Sardinha, Ricardo; Oliveira, Mafalda

The Use of Sentinel-1 Radar Images Associated With the Insar Technique to Monitor Surface Displacements of a Dam in Brazil 1567
Viana, Deivid Reis; Mendonça, Marcos Barreto; Silva, Wagner Barreto; Rocha, Carlos Henrique Oliveira

Evaluation of a Geophysical Study Involving Electrical Resistivity and Spontaneous Potential Tests for Piping Identification on the Whater Dam on Mato Grosso do Sul - Brazil 1581
Mestnik, Yuri; Ribeiro, Andre Batistela

Slope Stabilization on the Left Bank Upstream of the Caniçada Dam – Considerations on Constraints, Quality and Maintenance of Ground-anchored Structures 1599
Rodrigues de Carvalho, Mariana; Escaleira Esteves, Carlos; Caldeira, Laura

Volume 2

T3 Hydraulics for Dams

Development of Innovative Siphon Spillway for Existing Dams 1623
Kawasaki, Hideaki

Climate Shift Impacts on Hydrological Patterns in Mining Areas Located in Northern Brazil 1641
Costa, Denis; Rodrigues, Francisco; Ferrari, Darym; Oliveira, Keyla; Portes, Andrea; Yokozawa, Stephanie; Sol, Fernanda; Gomide, Paulo

Investigation of the Breach Shape Effects on the Flood Propagation in the Case of the Distorted Physical Model of Ürkmez Dam 1655
Arkış, Tanıl; Bombar, Gökçen; Güney, Mehmet Şükrü; Tayfur, Gökmen

A Methodology to Determine the Flow Curves for a River Barrage in Potentially Submerged Conditions 1673
Zuccalà, David; Gatti, Daria; Scuppa, Isabella; Cirillo, Carlo

Freeboard and Spillway Capacity Progressive Reductions for the Lago Verde Rockfill Dam, Due to the Crest Settlements After Several Decades of Operation 1693
Cesali, Chiara; Federico, Francesco; Maestri, Marina

Extraordinary Maintenance of San Mauro's Dam (Italy): Replacement of the Main Tainter Gates and Restoration of the Safety Conditions of the Structure 1709
Munari, Stefano; Vallese, Gaetano; Bianco, Lorenzo

Analysis of the Influence of Manning's Coefficient in Dam Failure Simulations Using Remote Sensing Data .. 1721
Salgado, Sérgio; Carvalho, Elsa; Viseu, Maria Teresa; de Oliveira, Othon Fialho

Ontime Modeling of the 14 de Julho Partial Dam-Break in South of Brazil 1733
Moecke, Giorgia; De Lara, Pedro; Diniz, Felipe

Development of the Combined Fish Pass of Fratel's Dam using Numerical and Physical Modelling 1745
Falcao de Melo, Jose; Sassetti Mendes, Lourenço; Muralha, António; Pinto, Pedro; Oliveira, João Miguel; Carvalho, Liliana Benites; Almeida, Pedro Raposo; Quintella, Bernardo; Alexandre, Carlos M.; Rato, Ana Sofia

Spillway Modeling: Comparing RNG and LES Turbulence Models 1777
Quaresma, Ana; Pinheiro, António

Overall Hydraulic Model Test of Caculo Cabaça Hydropower Project 1793
Haiyan, Yang; Liqun, Wang; Amador, António; Costa, Maria Cristina; Chico, Augusto; Wen, Liu

Strengthening Key Water Infrastructure in Mozambique: Risk Analysis of Massingir Dam Downstream Valley 1813
Amaral, Sílvia; Rocha, Fernanda; Viseu, Teresa; Juízo, Dinis; Saimone, Francisco; Pimenta, Lurdes; Miguel, Fernando; Cuna, Ivan; Mabunda, Luis

Strengthening Key Water Infrastructure in Mozambique: Risk Analysis of Pequenos Libombos Dam Downstream Valley 1827
Amaral, Sílvia; Rocha, Fernanda; Viseu, Teresa; Juízo, Dinis; Saimone, Francisco; Pimenta, Lurdes; Sengo, Delário; Timba, Jaime; Mabunda, Luis

T4 Tailings Dams

Optimizing Tailings Dam Safety: Integrating Human Factors and Evolving Hazards 1841
Oboni, Franco; Oboni, Cesar

Containment of Concrete Dams Compacted with Roller Using Iron Aggregate 1859
Pereira, Frank; Carmo, João; Andrade, Walton

Mining Dam's Classification & the Brazilian Practice 1889
Rangel Bittencourt, Lucas; Zic, Kresimir

Decommissioning of Tailings Dams: Case Study and Critical Analysis of the Decommissioning Stages of the Samambaia Dam 1911
Pires, Ian Henrique; Romanini, Augusto

Influence of Soil and Climate Variability on Earth Dam Design in Brazil 1929
Campos, Darym Junior Ferrari de; Sol, Fernanda Rocha Reis; Veras, José Dias Diego; Costa, Denis Duda; Ferreira, Stephanie Yumi Yokozawa; Gomide, Paulo Cesar Riquelme

Emergency Action Plan and the Precautionary Principle in Protecting Lives 1947
Sant'Ana, Marcelo; Velloso, Verônica; Motta Velloso, Márcia; Lopes, Daniela

ITRB Experience at Vale Mineração Tailings Storage Facilities 1957
Eleoterio, Marcos; Costa, Filipe; Marques, Eduardo

Comparative Study of 2D and 3D Slope Stability Analyses Applied to a Tailings Dam 1969
de Melo, Rayssa; Carneiro, Ananda; Almeida, Eduarda; Ferraz, Roberto; Marques, Eduardo

Evaluation of Slope Stability in Tailings Dam Construction and Decommissioning Stages Using 3D Limit Equilibrium and Strength Reduction Factor: a Comparative Analysis 1983
Barbosa, Jaime; Dantas, Marcelle; Carvalho, Wesley; Norena, Nilthson

From 2-D to 3-D Limit Equilibrium Analyses on Tailings Dam Stability: Assumptions, Benefits and Uncertainties 2003
Ferreira, Gonçalo

Evaluating 3D Effects on Tailings Dam Stability: a Comparative Analysis Using FS_{3D}/FS_{2D} Ratios 2023
Conceição, Raul; Ferreira, Gonçalo; Camões Lourenço, João; Lopes, Henrique

Technical and Economic Feasibility of Using Iron Mining Waste as Borrow Material for Dam Decharacterization 2045
Pereira, Frank; Tacco, Rafaela; Ribeiro, Adriana; Mazon, Alessandra; Carmo, João Bosco; Guerra, Felipe; Cavalcante, André; Torres, Vidal; Cristiane, Sebastião; Leal, Ana

Geotechnical Risk Assessment of Tailing Storage Facilities - Lessons Learned 2065
Esfehani, Morteza; Figueiredo, Luis Melges de; Carnavale, Thiago de Souza

Sensitivity Analysis of a Tailing Dam Stability Properties Using Sobol Indices and Sensitivity Factors 2099
Rodrigues Leal, Gabriela; Torrico Siacara, Adrian

Dam Raising Using a Reinforced Soil Containment Structure 2113
Ribeiro Paulino, Gustavo; Vilela Wanderley Neto, Ronaldo; Tozetti Franco, Ciro Gabriel; Tozetti Franco, João Emílio; Fernando Fanaya Dluhosch, André; Margarete Magalhães de Oliveira Cadavez, Camyla; Eduardo Pimentel, Victor

Geotechnics 4.0: Use of Big Data and Artificial Intelligence in Geotechcnical Investigations at Dams in Mining 2125
Biondo, Marieli; Piton, Rafael; Costa, Viviane

Evaluation of the Impact of Changing the Tailings Disposal Method in the Reservoir of a Rock Soil Dam 2143
Cruz, Mateus; Franco, Ciro; Flores, Katherine

Historical Cases of Liquefaction in Dams: Lessons Learned 2157
Lima Ferreira, Renata; Muniz de Faria, Thiago; Pereira, Luana

Rescue Systems: the Challenge of Performing Geotechnical Surveys in Tailings Dams at an Emergency Level 2173
Santos, Fernando; Biondo, Marieli; Sousa, Eduardo

A Mathematical Way to Claim Our Experience: Following the Bayesian Approach 2191
Viegas, Jeniffer; Gallardo, António

Technological Challenges in Tailings Dams in the Brazilian Amazon 2213
Pino Véliz, Pedro; Kong Diaz, Patricia

Tailings Dams Engineering: Amazon vs. Non-Amazon Challenges 2241
Pino Véliz, Pedro; Kong Diaz, Patricia

Construction of the Southern Expansion of the Cerro do Lobo Waste Facility 2249
Manso, Joao; Brito, Andrea; Barroso, Madalena; Marcelino, João; Galrão, Rui; Oliveira, Mafalda

Southern Expansion of the Cerro do Lobo Waste Facility. Mechanical Characterization of Soil-Rockfill Mixtures 2261
Brito, Andrea; Manso, João; Marcelino, João; Galrão, Rui; Oliveira, Mafalda

Applications of InSAR Technology in Dam Monitoring: a Systematic Literature Review 2273
Silva Alves, Jailson; Quental Coutinho, Roberto; M. Ruiz-Armenteros, Antonio; Barreto de Araújo Falcão Moreira, Hanna; Ramos Aragão Júnior, Wilson

Automated Geotechnical Monitoring Project of a Brazilian Mining Tailings Dams 2285
Silva, André; Gonçalves, Rachel; Orsseti Dias, Carlos; Meireles, Henrille

Safety Evaluation of an Upstream Raised Red Mud Tailings Dam 2303
Bottaro, Lucas; Lopes, Isabel; Tavares, Goncalo; Garcia, Marta

L-Band SAR Derived Soil Moisture Maps for the Monitoring of Earthen Dams in Scandinavia 2319
Hinton, Mark; Lynch, Jonathan; Rabinovitch, Yonatan

T5 Environmental and Social Issues

A GIS-Grid Nonpoint Pollution and Self-Purification Model for Hydropower Reservoirs 2331
Ribas, José Roberto; Fariña, Maria Elena Arce; Severo, Juliana C Ribas; Garcia, Andrés Suárez

Decharacterization Programs for Dams in Brazil: Solutions and Challenges 2345
Schaefer Santos, Felipe; Sayuri Tanabe Nishijima, Paula; Coelho de Campos, Gisleine; Shoji Iyomasa, Wilson

Sustainable Reservoir Operation: Balancing Power Generation and Downstream Environmental Maintenance - The Case of the Cahora Bassa Dam 2367
Guale, Rosaque; Carvalho, Elsa; Alves, Elsa

Rethinking Economic Risk for Amazonian Tailings Dams 2385
Pino Véliz, Pedro; Kong, P.

Enhancing Tailings Dam Classification for GISTM and ESG Conformance Using Generative AI 2401
Bachu, Lakshmin

T6 Regulation and Governance

The Development of Reservoir Legislation in the UK – Past, Present & Future – A Critical Review and Recommendations 2417
Hughes, Andy

Evaluation of Dam Safety Practices in Mozambique Through Risk Assessment 2439
Mabunda, Luis António; Escuder-Bueno, Ignacio; Morales-Torres, Adrián; Mancini, Davide; Romero-Duque, Sergio Andrés; Silva, Marco

Proposal to Obtain a Correlation of Concrete Properties from Standardized Radiological Testing 2455
Moreira do Carmo, João Bosco; Meira Belo, Luiz Claudio; Soares Lameiras, Fernando; Marques Filho, José

Emergency Action Protocol Implemented by Inea at the Açude da Concórdia Dam 2463
Dias, Fernanda Spitz; Amaral, Daniele Pereira Batista; Alves da Silva, Milena; Bielschowsky, Cauê; Pereira, Thiago Henrique Zampieri

The Challenges of the Brazilian Electric Energy Regulatory Agency on Mitigate Effects from 2024 Rio Grande do Sul Floods 2481
Leite, Sérgio; da Silva, Ludimila; Gomes, Luiz Rogério

The Role of the Brazilian Electric Energy Agency in Dam Safety - A Review of Recent Years and Prospects for the Future 2507
Leite, Sérgio; da Silva, Ludimila; Mendes, Marcelo

Developments of Interest for Dam Industry Regarding Seismic Action in the Eurocode 8 of Next Generation 2533
Labbé, Pierre

Opportunities and Challenges in Applying BIM Methodologies Within the Portuguese Dam Safety Framework 2551
Serra, Carlos; Silva, João; Carvalho, Jorge

Dona Francisca HPP Crisis Management: Extreme Flood Event Response 2569
Moecke, Giorgia; de Lara, Pedro Guilherme; Fonseca, Maria Eduarda

GISTM Requirements Conformance and the Influence of the Key Roles for the Tailings Governance Framework: a Statistic Approach 2583
Borges, Marina; Menezes, Danielle; Delgado, Bruno; Mota, Hernani

General Criteria for Implementation of Dam Safety Management System 2593
Sieber, Luiz

Eletrobras' Experience in Implementing Emergency Action Plans for Dams 2609
Fernandes, Roberta Fonseca Santos; Peplau, Guilherme Rocha; Azevedo, Lizandra Prado de; Deotti, Leonardo de Oliveira Guerra; Vianna, Vinicius Ferreira

Dam Safety KPI at Eletrobras: Development and Application of Tactical Key Performance Indicadors 2625
Fernandes, Roberta Fonseca Santos; Deotti, Leonardo de Oliveira Guerra; Vianna, Vinicius Ferreira; Peplau, Guilherme Rocha; Azevedo, Lizandra Prado de

T7 Dams and Climate Change

Planning the Semi-arid Region: Land Use Capability Allied to Underground Dams as a Measure for Mitigating Deterioration 2643
Costa e Silva, Danilo

21st Century Climate Challenges and Debris Disasters: Assessing Legacy Dams Amid Record Climate Events 2657
McCully, Jared; Meeks, Paul; Rollins, Paul

Impact of Climatic Drift on Arch Dams Behaviour 2675
Suchier, Louis; Robbe, Emmanuel; Simon, Alexandre; Coubard, Grégory

Increasing Reservoir Storage Capacity by Using the Fusegate® System in the Brazilian State of Bahia 2693
De Simone, Cesare; Del Rey, Franck

Climate Resilience in Reservoirs: Recommendations from Chile 2713
Garrido, Carlos; San Martin, Luis

Urban Water Dams Decommissioning Using Nature-based Solutions: the Case of Monjolinho Dam, in Brazil 2733
Povinelli, Silvia Claudia Semensato; Vasconcelos, Anaí Floriano; Mattos, Sergio Henrique Vannucchi Leme de; Rother, Débora Cristina; Gonçalves, Luciana Márcia

State-of-the-Art Review on Advancements Dam Monitoring Technologies 2751
Silva, André; Levesque, Christiane

Diversifying Electricity Sources in Water-Stressed Regions: a Multi-Criteria Approach for the São Francisco River Basin 2769
Barbosa Souza Júnior, Clécio

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DEVELOPMENT OF INNOVATIVE SIPHON SPILLWAY FOR EXISTING DAMS

Hideaki Kawasaki[1]

Japan dam
engineering centre

ABSTRACT

Recently, deficiencies in the spillway discharge capacity of existing dams have become an important global topic. In contrast, improvements in spillway discharge capacity have been delayed owing to the high cost and difficulty of construction, to hold spillway functions. In this situation, an innovative siphon spillway was developed for the disaster restoration of a small-scale earthen dam, named the Utagaura Dam (hereinafter, U Dam), completed in 1960, with a height of 15 m for the water supply owned by Sasebo City (hereinafter, S City), Nagasaki Prefecture.

This dam body overflowed because of heavy rainfall in August 2019, and its downstream surface was severely washed out. Due to insufficient discharge capacity causing the overflow, both the normal full water level and the inflow design flood water level (HWL) were lowered to meet current safety criteria. In addition, the discharge capacity was increased several times based on recent rain data. However, it is quite difficult to satisfy the above requirements using conventional methods.

Basic designs to improve the U Dam spillway revealed that the siphon spillway was superior to other types of facilities, such as open channels, conduit gates, and flood tunnels. After the first design of the new siphon spillway, the authors conducted the first hydraulic model experiment; however, when approaching HWL, negative pressures around the summit of the siphon pipe critically exceeded the allowable limit of -7.0 mH_2O.

Thus, the authors made extensive improvements, such as enlarging the inflow cross-sectional area to reduce negative pressures and narrowing the cross-sectional area toward the outlet to enable stable discharges. Eventually, in the second hydraulic model experiment using this innovatively changed design, the authors confirmed that the measured negative pressures satisfied the allowable limit with a sufficient margin, and the inflow design flood could be discharged at the HWL.

The construction works of U Dam siphon spillway is to start soon, and this Innovative siphon spillway is expected to spread to embankment dams and concrete dams economically and effectively. Furthermore, the author and a study team have continued to study an other-type siphon spillway for flood control use.

Keywords: siphon spillway, existing dam, discharge capacity, negative pressure, hydraulic model experiment

[1] kawasaki@npo-fuudo.net, Japan

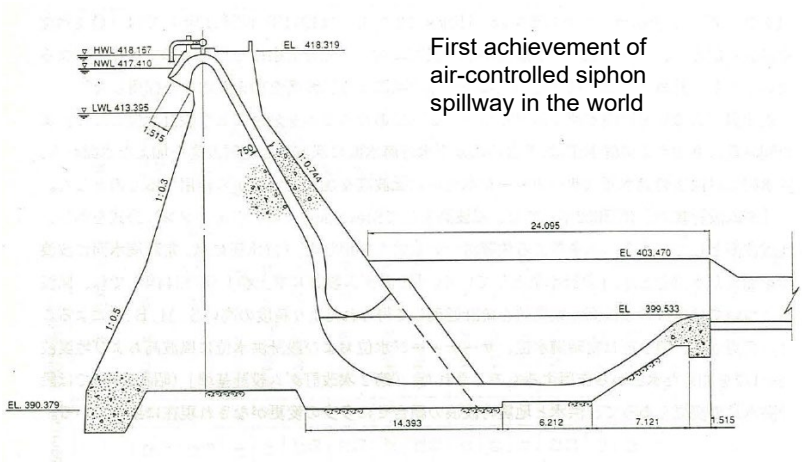
1. HISTORICAL SIPHON SPILLWAY

The history of siphons dates back to ancient times. In ancient Rome, they were used to pump water from reservoirs to aqueducts. However, the theorisation of the siphon principle was new when vacuum pumps were developed in the 17th century, and their use in daily necessities and industrial products began at the end of the 19th century. Since the 20th century, the siphon principle has been widely used in dams.

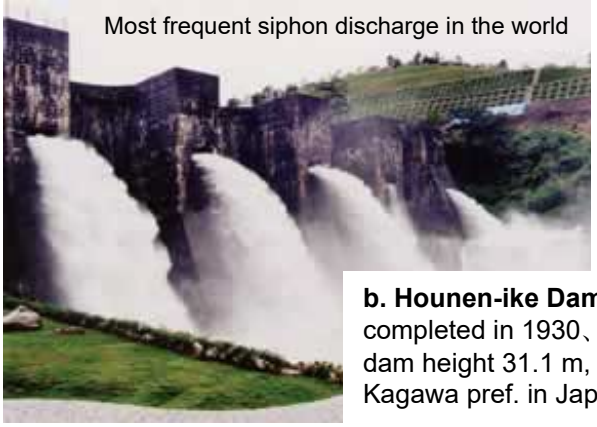
In the 1920s, there were some dams in Europe and the USA with standard siphon spillways that installed an air vent at the top of the siphon to reduce air bubbles and stabilise the flow inside the pipe. In addition, air control methods that involved mixing with air to reduce negative pressure and create stable flow conditions were developed.

The first air-controlled siphon was the Japanese Dam (Fig. 1a), which was developed to allow floodwater to pass through. In Japan, an ingenious siphon spillway was constructed using only stone and concrete (Fig. 1b); this dam still has the highest frequency of flood discharge by siphons. Japan was once the leading siphon spill country.

Siphon spillways have been widely used worldwide since the 1930s, and the U.S. Bureau of



a. Furukawa Dam: completed in 1929, dam height 23.7m, Hokkaido in Japan



b. Hounen-ike Dam: completed in 1930、dam height 31.1 m, Kagawa pref. in Japan

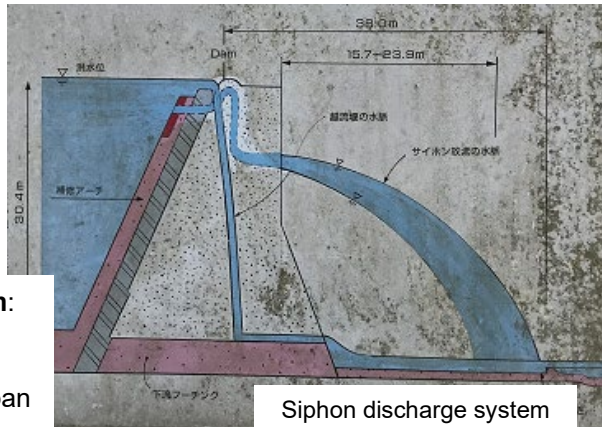
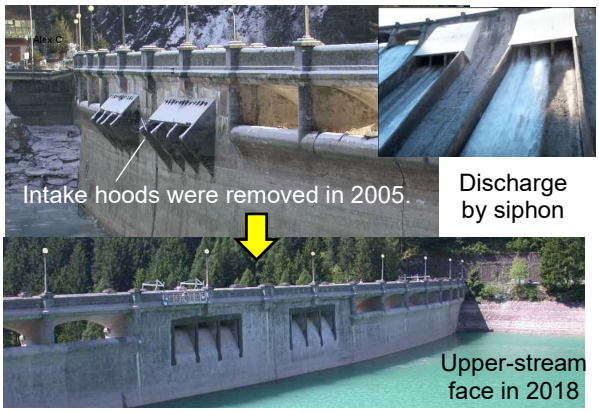


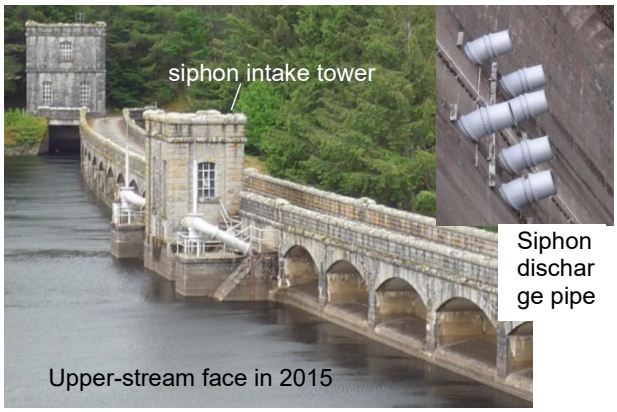
Fig. 1 – Siphon spillways in Japan: siphon still active for 95 years

Reclamation recommended a minimum negative pressure at atmospheric pressure x 0.7 (-7.2 mH_2O) to prevent cavitation damage based on experimental data [2].

Fig. 2a shows a free-overflow dam with a steel hood for removing the siphon, and Fig. 2b shows a dam with a unique siphon steel pipe. Fig. 3a shows the largest siphon with an exhaust system installed, and Fig. 3b shows a large siphon based on detailed experiments. These siphon spillways are noteworthy because of their state-of-the-art technologies.

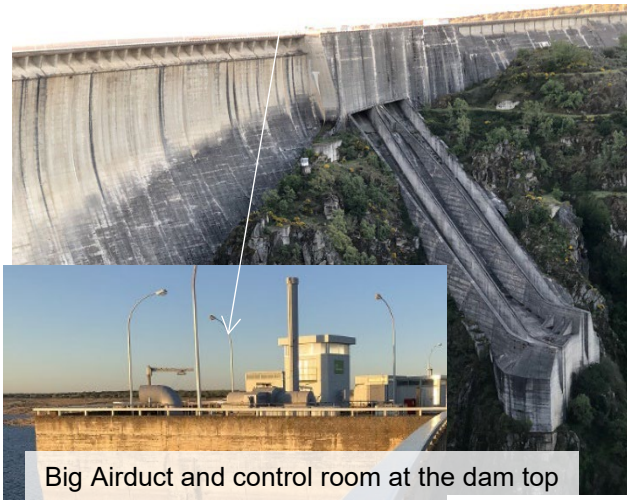


a. Santa Catrine Dam: completed in 1932, dam height 55 m, North in Italy

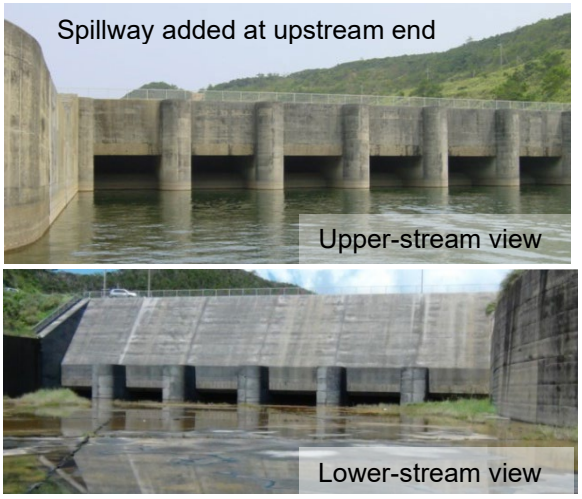


b. Laggan Dam: completed in 1934, dam height 48 m, Scotland in UK, Siphon still active

Fig. 2 – Siphon spillways in Europe in1930's



a. Almendra Dam: completed in 1970, dam height 202m, Northwest in Spain



b. Fukuchi Dam: spillway completed in 1983, dam height 92m, Okinawa pref. in Japan

Fig. 3 – Big-scale siphon spillways after 1970's

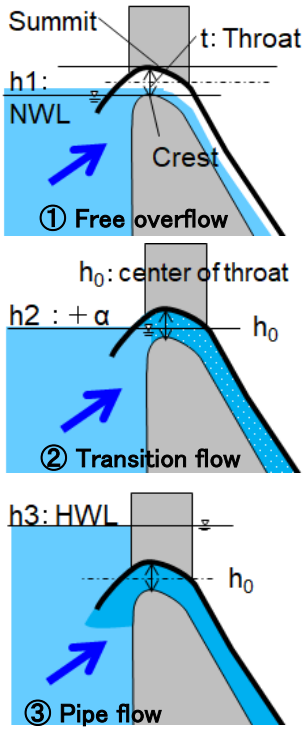
2. MECHANISM OF SIPHON SPILLWAY AND NORMAL DESIGN

Siphon spillways have the advantage of being automatic and economical during emergencies. Normally, the water level can be automatically maintained at overflow crest by free overflow. When the water level rises during a flood, the flow status changes to a pipe flow through a transition flow, reaching flow velocities several times faster than a normal free overflow. Fig. 4a shows a typical flow status that changes from free overflow to pipe flow with water level.

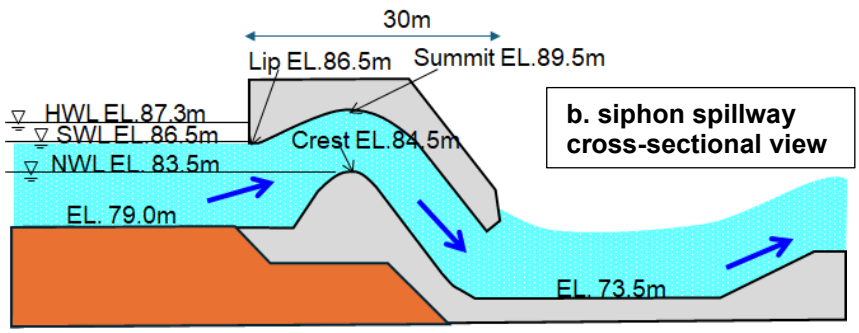
The Fukuchi Dam (rockfill dam with a dam height of 92 m, completed in 1973) is a suitable example of a siphon spillway design. From 1980 to 1983, a siphon spillway was built at the upstream end of the reservoir for emergency discharge during floods.

Fig. 4b shows a cross-section of the siphon spillway, which has a more theoretical siphon design based on hydraulic model experiments, such as its rational arch shape, elevations of the crest and summit, lower velocity at the inflow section, and underwater discharge to eliminate air intake from the outlet. Its features are as follows: “no operation required during floods, no daily maintenance, direct discharge via chute to the sea immediately, big-scale summit section (7.0 m wide x 5.0 m high x 6 mouths), and big-scale maximum discharge volume of 900 m^3/s”.

Fig. 4c shows the results of the hydraulic model experiment. Its free overflow status is kept until EL.86.5 m (SWL; highest water level in floods), and when the water level exceeds this level, the flow status changes to transition flow with a rapid increase in velocity. Furthermore, when it exceeds EL.87.3 m (HWL; inflow design flood water level), the flow status changes to pipe flow. Its hydraulic feature is that the transition flow zone prone to turbulence is only 0.8 m high. Moreover, because the throat height was large at 5 m and the summit was 2.2 m higher than the HWL, the maximum flow velocity was maintained below 5 m/s to reduce negative pressures. The U Dam discussed in this study was subjected to hydraulic analysis using Fukuchi Dam as a model of the siphon spillway.



a. Flow status



c. H-Q relation by the hydraulic model experiment

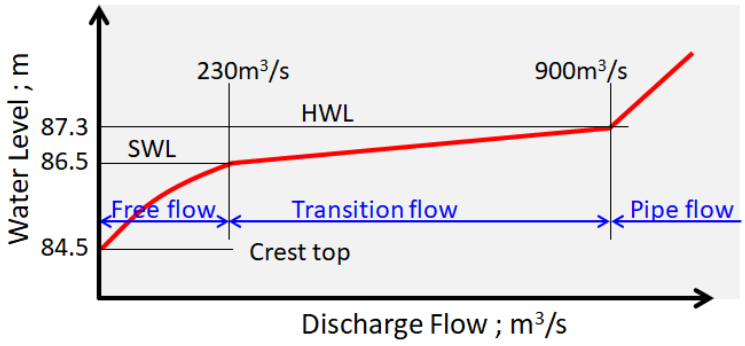


Fig. 4 – Flow status and siphon spillway of Fukuchi Dam

3. EARTHEN DAM DISASTER AND ADOPTION OF SIPHON SPILLWAY

3.1. Earthen dam overflow disaster due to insufficient spillway discharge capacity

U Dam is a small-scale earthen dam with a height of 15 m for the water supply of S City. However, the embankment overflowed owing to heavy rainfall in August 2019 and experienced a downstream surface washout. As shown in Fig. 5, the downstream face of the U Dam was eroded by overflow owing to a deficiency in flood discharge capacity. Thereafter, the restoration of the eroded area was initiated, and embankment works were completed through national expenditure in 2021. However, due to insufficient spillway discharge capacity causing the overflow, the discharge capacity was increased to 44 m^3/s in accordance with the current safety standards, by lowering the normal water level through S city expenditure. As this was a strict requirement, S City consulted the author about countermeasures.

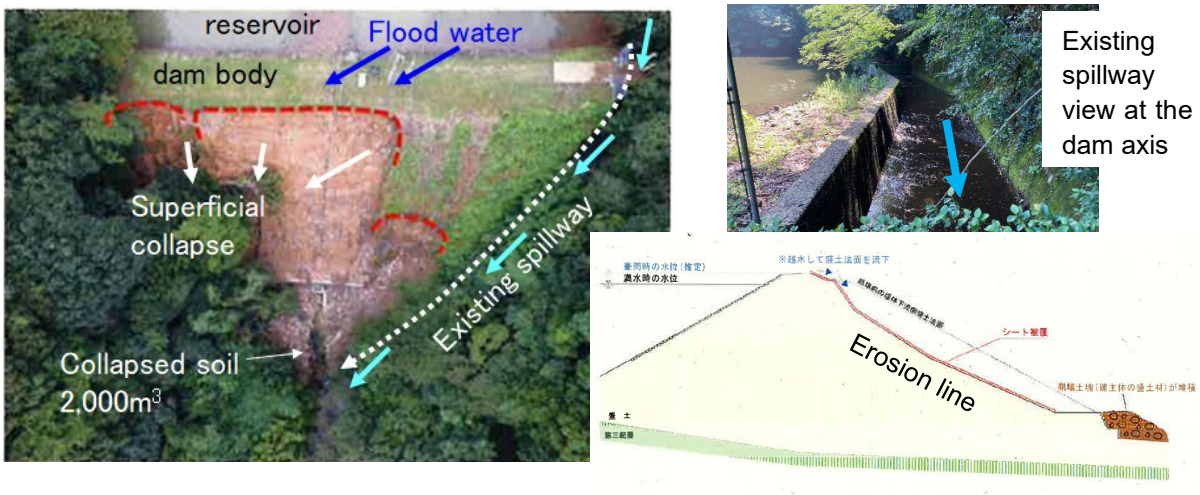


Fig. 5 – Disaster situation of U Dam

3.2. How to increase discharge capacity with realistic means and economical costs

Initially, an open channel for U Dam was considered to expand the existing spillway by cutting the left bank mountain largely like shown in Fig. 6a, however, this plan required significantly to increase the construction costs by the large-scale slope cutting and excavation of dam body.

Secondarily, a new siphon spillway with a compact size shown in Fig. 6b was considered. This newly proposed siphon spillway enabled to reduce the construction cost greatly remaining the existing spillway. The discharge capacity of the siphon was set at 40 m^3/s, and combined with that of the existing spillway, it was set at 44 m^3/s.

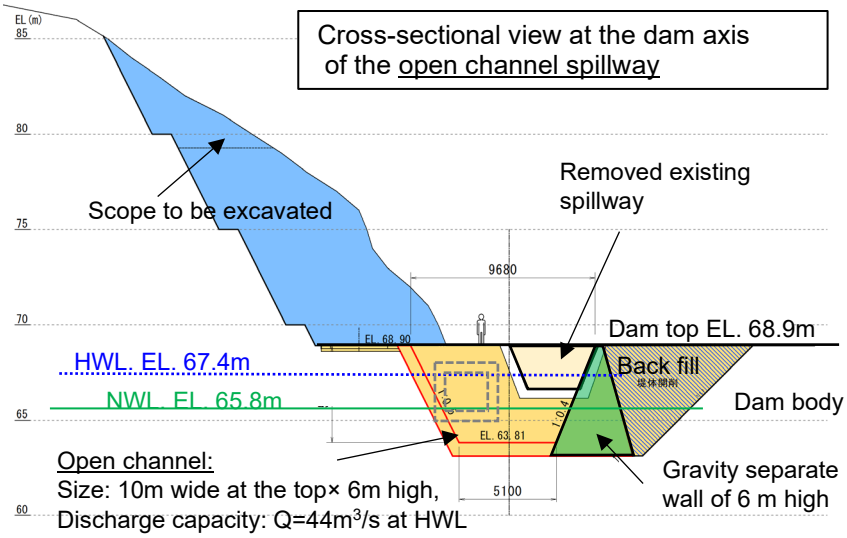
Once the basic designs were almost complete, the open-channel and siphon plans were compared. Consequently, it was confirmed that the siphon spillway is superior to the open channel, as shown in Fig. 6c. Thus, this new siphon spillway was adopted for U Dam, and the first design involved setting all cross-sections of the siphon to be identical (refer to Fig. 7).

a. Case of the open channel spillway
(after the basic design)

Discharge capacity of Existing spillway:

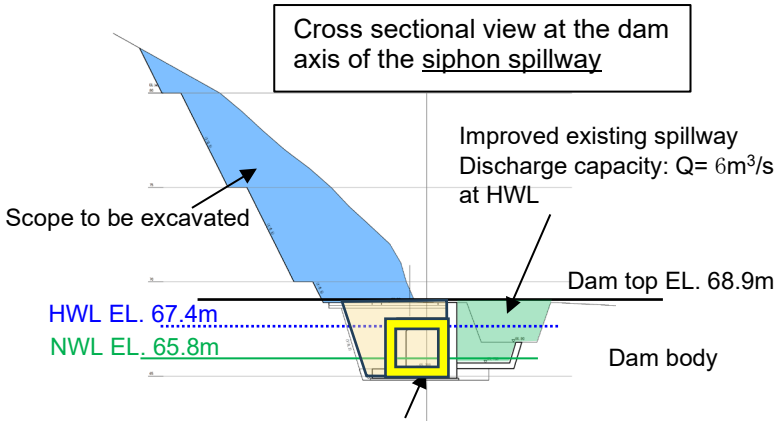
・5m³/s at HWL (inflow design flood water level)
・12m³/s at the dam top

・Inflow design flood is 44 m³/s



b Case of the siphon spillway

(after the first design)



Siphon spillway
Box Inside size b2.2m× h2.2m,
Discharge capacity: Q= 40 m³/s at HWL

c. 3D image of open channel and siphon spillway

Red means the open channel.

Blue means the siphon spillway.

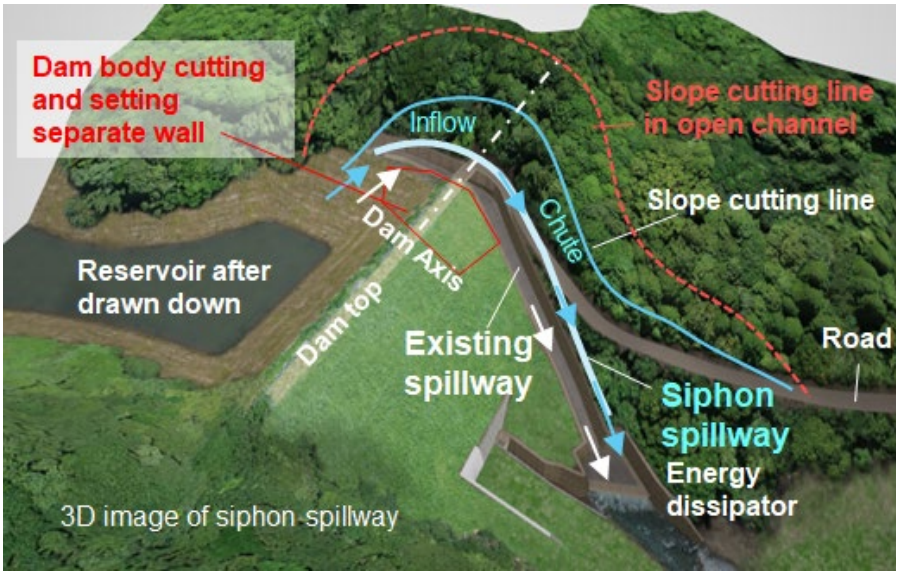


Fig. 6 – Comparison between open channel spillway and siphon spillway

4. FIRST HYDRAULIC MODEL EXPERIMENTS AND IMPROVEMENTS

4.1. Experimental planning

To confirm the discharge capacity and negative pressure, a hydraulic model experiment was conducted as follows: The specifications are presented in Table 1.

a. Discharge volume was calculated from the pumped circulation volume.

b. Inside pressures were measured using piezometers mounted on the bottle in the siphon. The allowable negative pressure to avoid cavitation was set to "-7 mH_2O".

Table 1 Specifications of the spillway model

Item	Real Size	Model	Remarks
Scale	1/1	1/16	K_L
Inflow design flood (m^3/s)	44.0	0.0430	$Q=K_L^{5/2}=44.0/1024=0.0430\ m^3/s$
Spillway extension(m)	74.3	4.64	$L=74.3/16=4.64$ m

Fig. 7 shows the longitudinal cross-section of the hydraulic model, and it should be noted that the negative pressure is likely to increase along the gentle gradient downstream of the summit. In addition, the summit position was lowered slightly downstream of the dam axis, as shown in Fig. 8, to avoid interference with the existing spillway. The cross-section of the siphon pipe was constant from the intake to the outlet (height 1.5 m, width 2.4 m, flow velocity: 10 m/s).

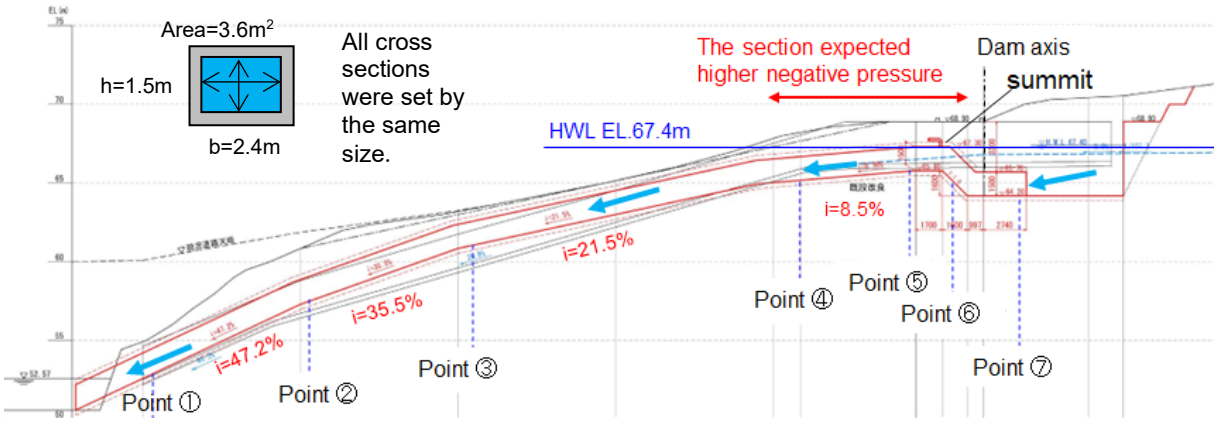


Fig. 7– Longitudinal cross-section of hydraulic model experiment

Fig. 8 – Inflow area in hydraulic model experiment

4.2. First experimental results

The results of the first experiment, shown in Fig. 9, are as follows.

a. Discharge capacity: It was confirmed that more than the inflow design flood was discharged at HWL of EL.67.4 m (=Inflow design flood water level). →Satisfied

b. Negative pressure: When near HWL, negative pressures in the summit and its downstream exceeded the allowable negative pressure of -7.0 mH_2O. →No Good
When the water level was above HWL, negative pressures increased, reaching -12 mH_2O at point ⑥ (summit), -9 m at point ⑤, -6.5 m at point ④ (gradient change), and -6.0 m at point ③.

c. Stability of inflow: When approaching the HWL, air sucking, and vibration often occur owing to insufficient water intake. →No Good

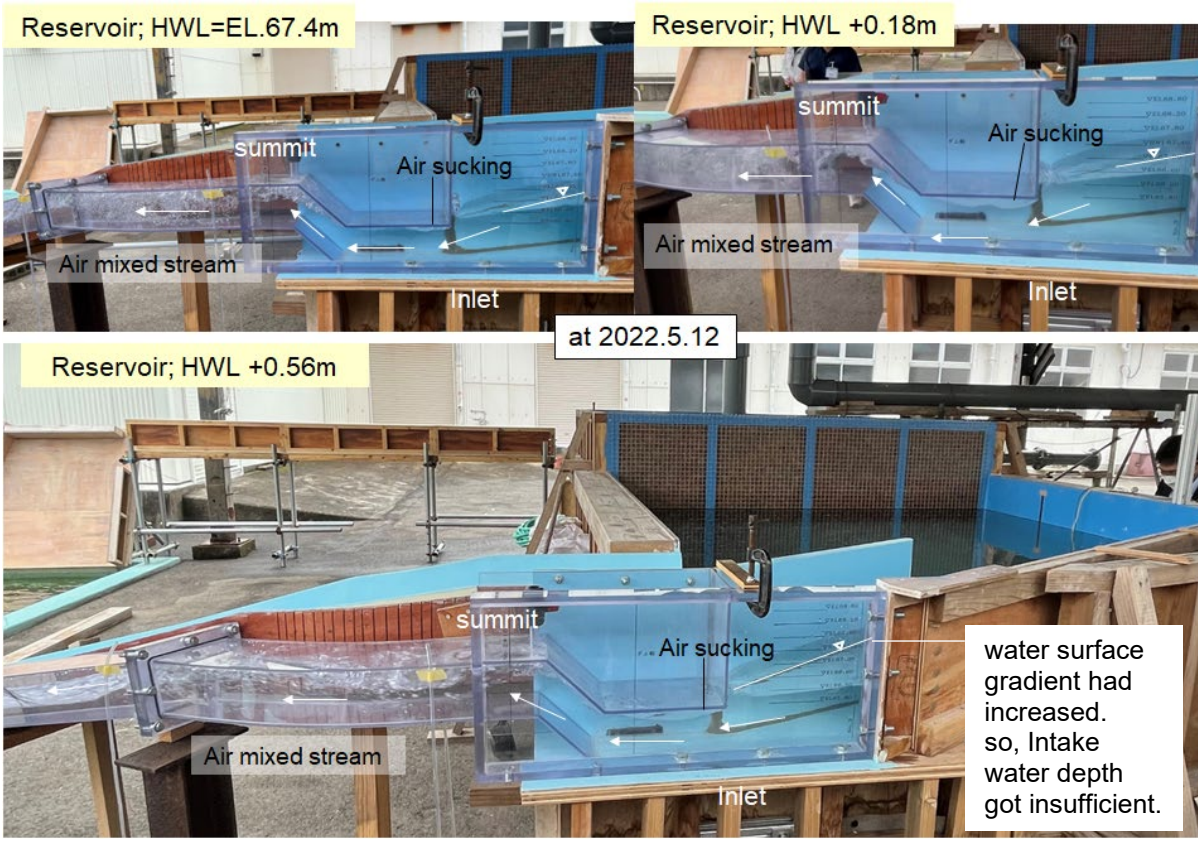


Fig. 9 – Status of siphon spillway in the first experiment

4.3. Causes of the excessive negative pressures

Pressure measurements were performed at several additional points to confirm the cause of the excessive negative pressures. Subsequently, it was measured that a negative pressure of -11mH_2O had occurred at point ⑦ (the inlet), which caused a significant increase in negative pressures downstream.

This was presumed to be due to insufficient inflow from the reservoir. The water surface gradient increased as the flow velocity increased, thereby lowering the water level of the inlet and causing air to suck from the inlet. To solve this problem, the velocity at the inlet was decreased to supply a sufficient flow.

In addition, the high negative pressure at points ④ and ③ on the downstream side was considered due to separated flows occurring at the gradient change point.

4.4. Improvements in the siphon spillway design

The following improvements to decrease negative pressures were made, as shown in Fig.10.

a. The summit was transferred 15 m upstream to secure the stable intake water.
b. The inflow section was expanded to double to decrease the velocity.
c. The outlet was narrowed to decrease the inflow velocity and stable the discharge.
d. All corners inside were rounded to decrease the loss.

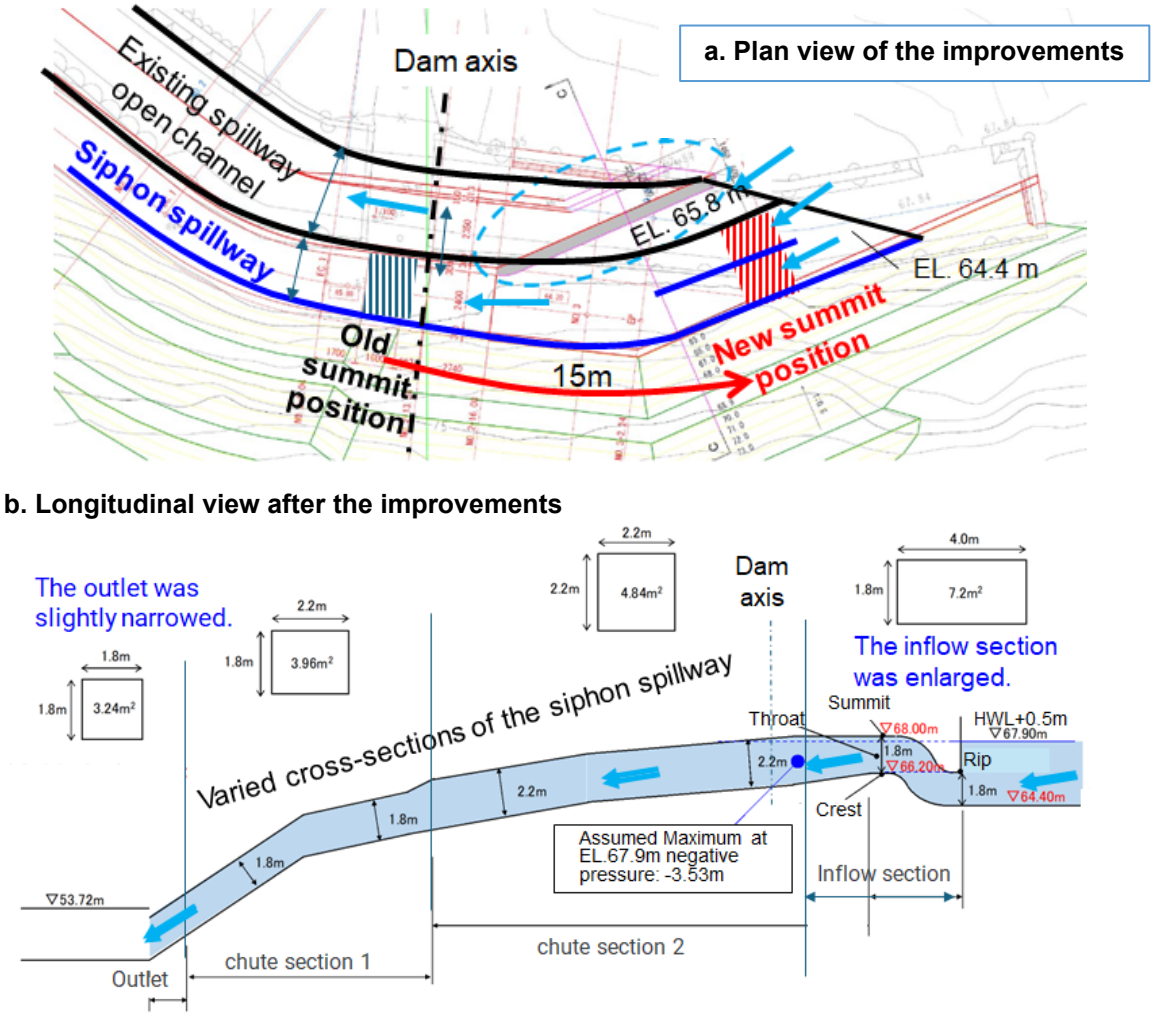


Fig.10 – Improvements in siphon spillway shape

5. SECOND HYDRAULIC MODEL EXPERIMENT AND RESULTS

5.1. Experimental planning

After significantly improving the siphon spillway design, the experimental waterway was rebuilt and a second hydraulic model experiment was conducted in October 2022.

The spillway model specifications are the same as those listed in Table 1, and the measurement methods match those used in the first experiment.

5.2. Situation of the second experiment

The second experiment is shown in Fig. 11. The flow status is determined as follows:

a. At HWL -0.1m (EL.67.3 m, top of Fig.11): When the water level reached approximately 60% of the throat height, the negative pressure became -4 mH_2O, and although there was some air mixing, there was almost no vortex generation and no separated flow. At this time, the flow status was considered to have reached the boundary between the transitional flow and pipe flow.

b. At the HWL (EL.67.4 m, centre of Fig.11), the above situation continued. As the water level increased, the flow gradually stabilised.

c. At HWL +0.5 m (EL.67.9 m, bottom of Fig.11), there was no air mixing over the entire length of the pipe, and there was a quiet, full-pipe flow.

5.3. Judgement of the second experiment results

Regarding the second set of experimental results, issues with the discharge capacity, excessive negative pressure, and flow stability were resolved, as shown below.

a. Discharge capacity: It was confirmed that more than the inflow design flood 44 m^3/s at HWL=EL.67.4 m was discharged, along with the existing spillway.

b. Negative pressure: The minimum negative pressure was -4 mH_2O, which was larger than the allowable limit of -7 mH_2O, with a significant margin at HWL-0.1m=EL.67.3 m. Over the HWL, the negative pressures improved gradually with an increase in water pressure.

c. Stability of inflow: Air sucking was almost eliminated, and the flow became stable, exceeding HWL=EL.67.4 m.

d. Energy dissipation: After the water was discharged from the outlet, some rotating flows occurred in the stilling basin, causing the water to rise. However, the flow velocity of the water overflowing from the subdam was maintained below 10 m/s.

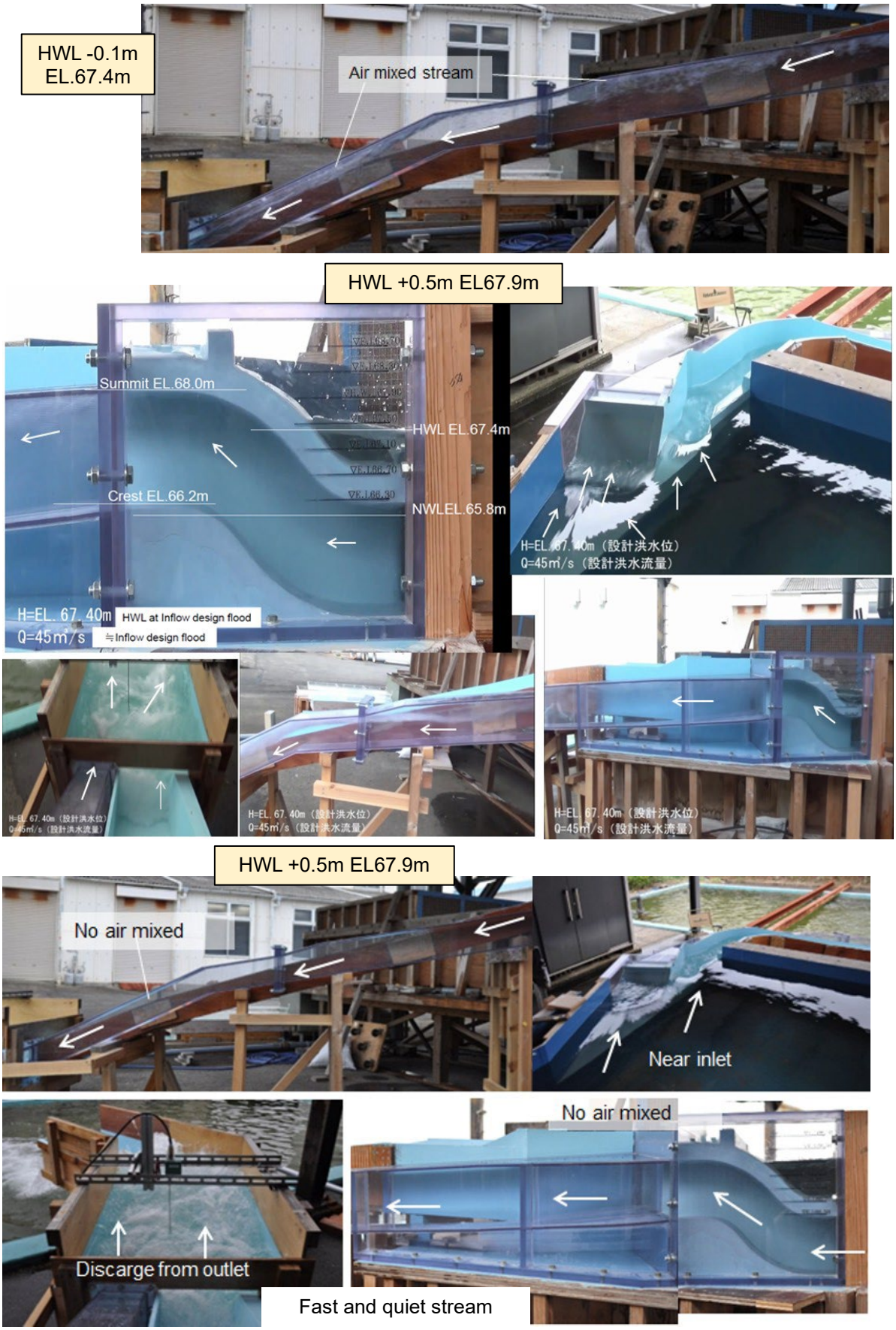


Fig.11 –Status of siphon spillway in the second experiment, Qevqdgt"4244

5.4. Results and Considerations on negative pressures and velocities

The results and considerations for negative pressures and velocities in the second experiment are shown in Fig.12.

a. The negative pressure at point ⑤ at the inlet was significantly improved to -1.62 mH_2O. This effect is due to the reduction in the flow velocity caused by the expansion of the cross-sectional area.

b. The negative pressure of -3.92 mH_2O at point ④ was bigger than the value of point ⑤ because the flow loss increased with the height increase of the cross-section.

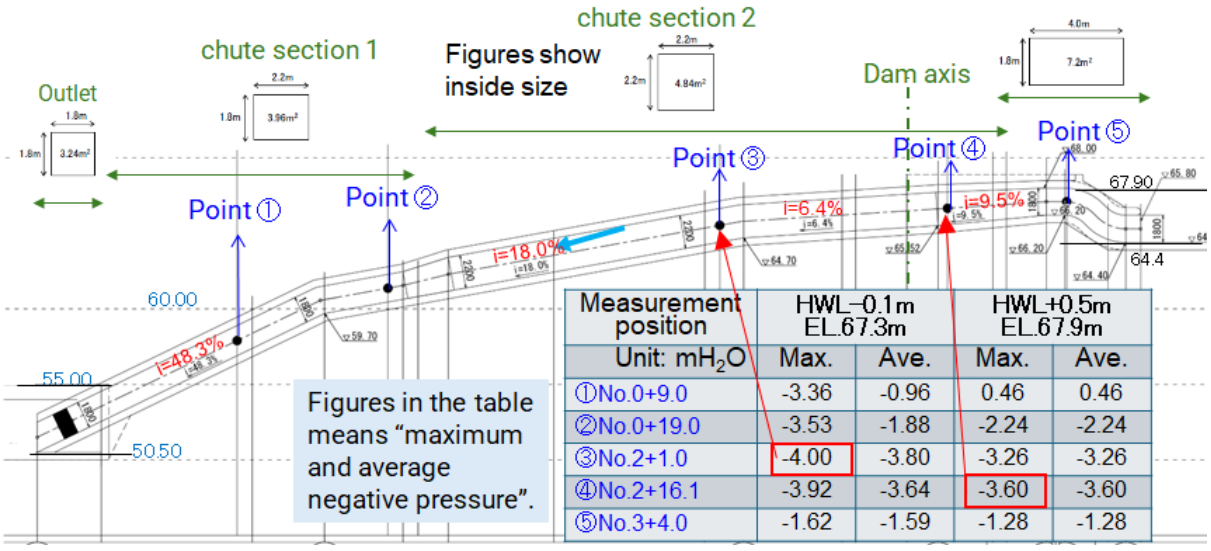
c. The minimum negative pressure of -4.00 mH_2O occurred at point ③, which is presumed to be due to separated flow at the point of gradient change.

d. The siphon spillway discharge, which was the revised measured value, was 41.9 m^3/s at HWL.

e. The velocity at HWL at point ⑤ was 5.82 m/s (=Q/A, Q=41.9 m^3/s, A=7.20 m^2;cross-sectional area in inflow). Although the velocity in the inflow section was lower than that in the normal orifice flow, it was higher than that in the normal free overflow.

f. The outlet flow velocity at HWL-0.1m was 12.9 m/s (=Q/A, Q=41.9 m^3/s, A=3.24 m^2:cross-sectional area in the outlet), which is fast as same as the normal orifice flow.

g. The flow coefficient was 0.78 (=outlet flow velocity /√(2gh), h: effective head 13.75 m). Though this value of 0.78 is considerably higher than the normal range of 0.6 ~ 0.7, it is believed to be a result of enhancing full pipe flow by narrowing the outlet, which increases flow stability and reduces losses, such as separated flow loss.



Measurement position	HWL−0.1m EL.67.3m		HWL+0.5m EL.67.9m	
Unit: mH_2O	Max.	Ave.	Max.	Ave.
①No.0+9.0	-3.36	-0.96	0.46	0.46
②No.0+19.0	-3.53	-1.88	-2.24	-2.24
③No.2+1.0	-4.00	-3.80	-3.26	-3.26
④No.2+16.1	-3.92	-3.64	-3.60	-3.60
⑤No.3+4.0	-1.62	-1.59	-1.28	-1.28

Fig.12 – Measurement points and results of inside pressures

5.5. Comparison of negative pressures between measured and calculated values

As shown in Fig.13, the calculated value of negative pressure generally coincides with the measured value in the hydraulic model experiment.

In a conventional siphon spillway, the minimum negative pressure occurs at the summit, such as point ⑤. However, as this spillway has a long and gentle slope between the inflow and the chute section, the calculated minimum negative pressure of -4.26 mH_2O occurs at point ④. Conversely, the measured value was -3.92 mH_2O, almost coinciding with the calculated values.

The flow loss is larger than that of a conventional siphon spillway, with the cumulative loss coefficient being 0.744 at point ⑤, which consists of an entrance loss of 0.150, two sites curvature loss of 0.532, a friction loss of 0.60, etc. In addition, this value is 1.321 at point ④, which is added by a widening loss of 0.178, a friction loss of 0.395, etc.

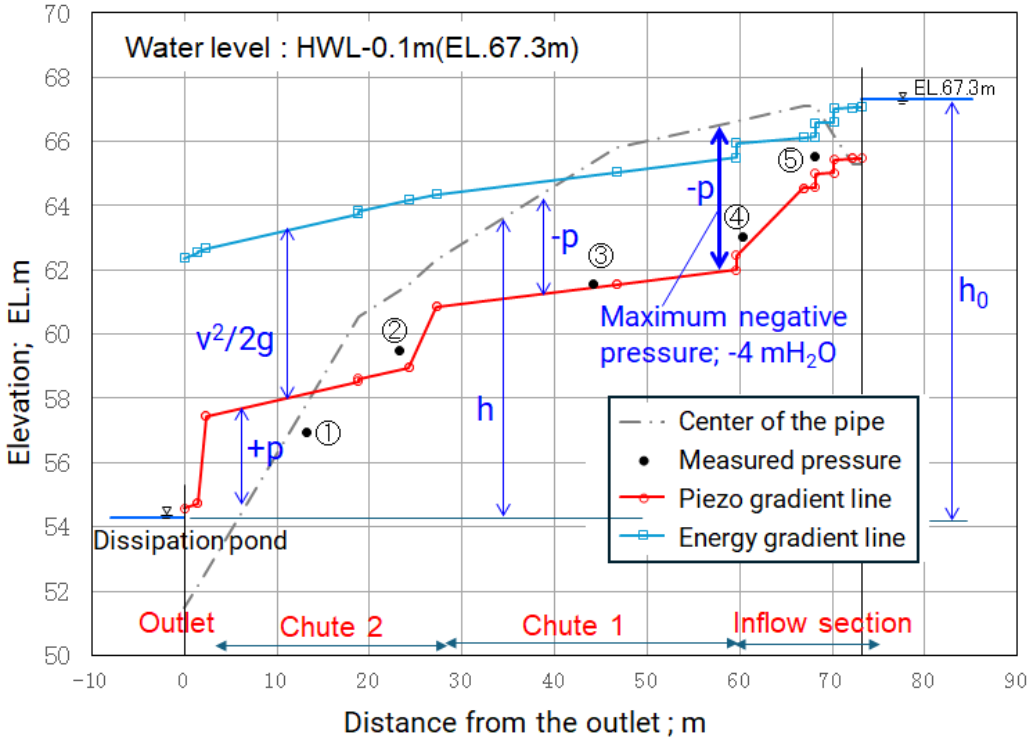


Fig.13 – Comparison of negative pressures between measured and calculated values

☆The energy gradient line: h_0- loss= h+ $v^2/2g$+ p
It was calculated by Bernoulli's theorem:

★The piezo gradient line: p=h_0-h- loss- $v^2/2g$

h: head (m), p: pressure (mH_2O), v: velocity (m/s),
loss: $\Sigma \xi \times (v^2/2g)$: Cumulative loss head (mH_2O) from reservoir water level to any point, ξ : loss rate

5.6. Relation between water level and flood discharge

The relationship between water level and flood discharge in the second experiment is shown in Fig.14. Consequently, the discharge at HWL EL.67.4 m satisfied the inflow design flood of 44 m^3/s or more, and the transition of "free overflow > transition flow > pipe flow" was observed, similar to the H-Q of Fukuchi Dam shown in Fig. 4. Furthermore, the inflow design flood water level (HWL) was almost at the boundary between the transition and pipe flows. After exceeding the HWL, the flood discharge increased steadily without rapid increase of water level.

The flood discharge was conducted separately for each of the three cases. As shown in Fig.14, at EL.67.26 m, the measured discharge was 5 m^3/s for Case A of the existing spillway alone, 28 m^3/s for Case B of the siphon spillway alone, and 44 m^3/s for Case C of the composite flow. Here, the discharge flow of Case C is more than that of together Case A＋B, and the cause seems to be disturbance of case B. Accordingly, the H-Q curve of Case B was revised by subtracting Case A from that of Case C. The authentic flow was the composite flow of Case C, and the design of the U-dam spillway was made using the H-Q curve of Case C.

The flow coefficient ($V/\sqrt{(2gh)}$) to 44 m^3/s was 0.79, using the discharge velocity of 13.6 m/s at the outlet (3.24 m^2) and the effective head (h) of 15m. Though the value of 0.79 was rather high compared to the conventional values, this value is variable to the outlet area.

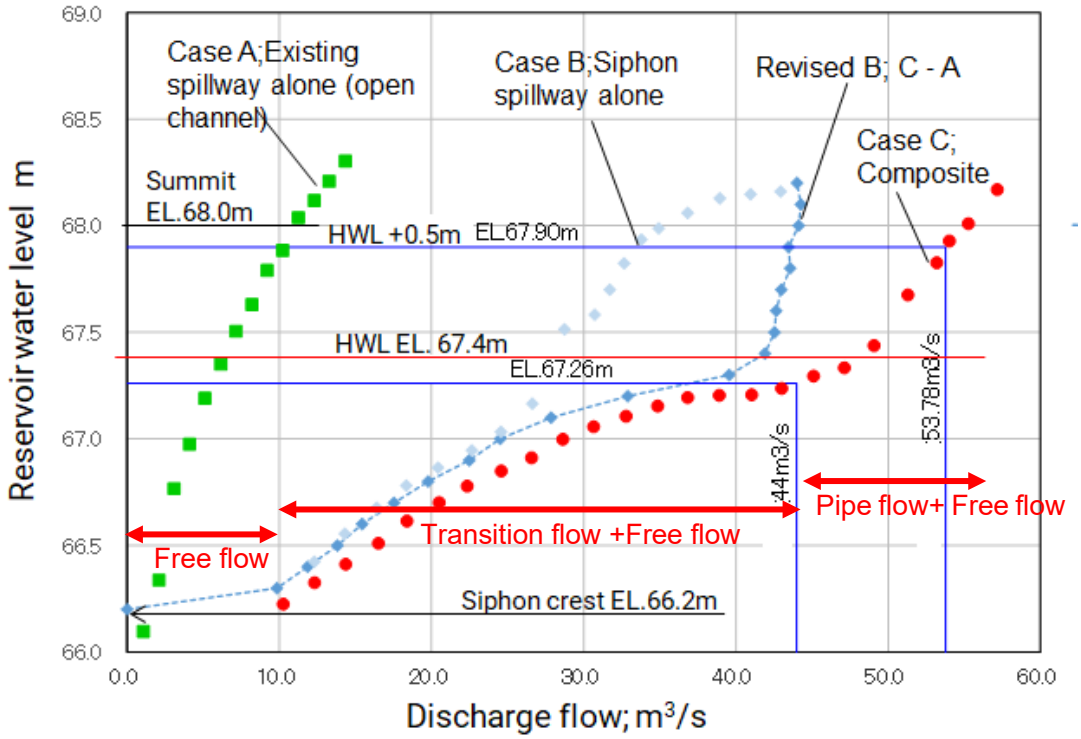


Fig.14 – Relation between water level and flood discharge (H-Q curve)

5.7. Final spillway design

After various values were confirmed through the hydraulic model test, the final design shown in Fig.15 and the construction plan for the U dam siphon spillway were prepared until 2023. Subsequently, explanations of this new technology were provided to relevant national and prefectural agencies, and structural safety and downstream protection were reviewed.

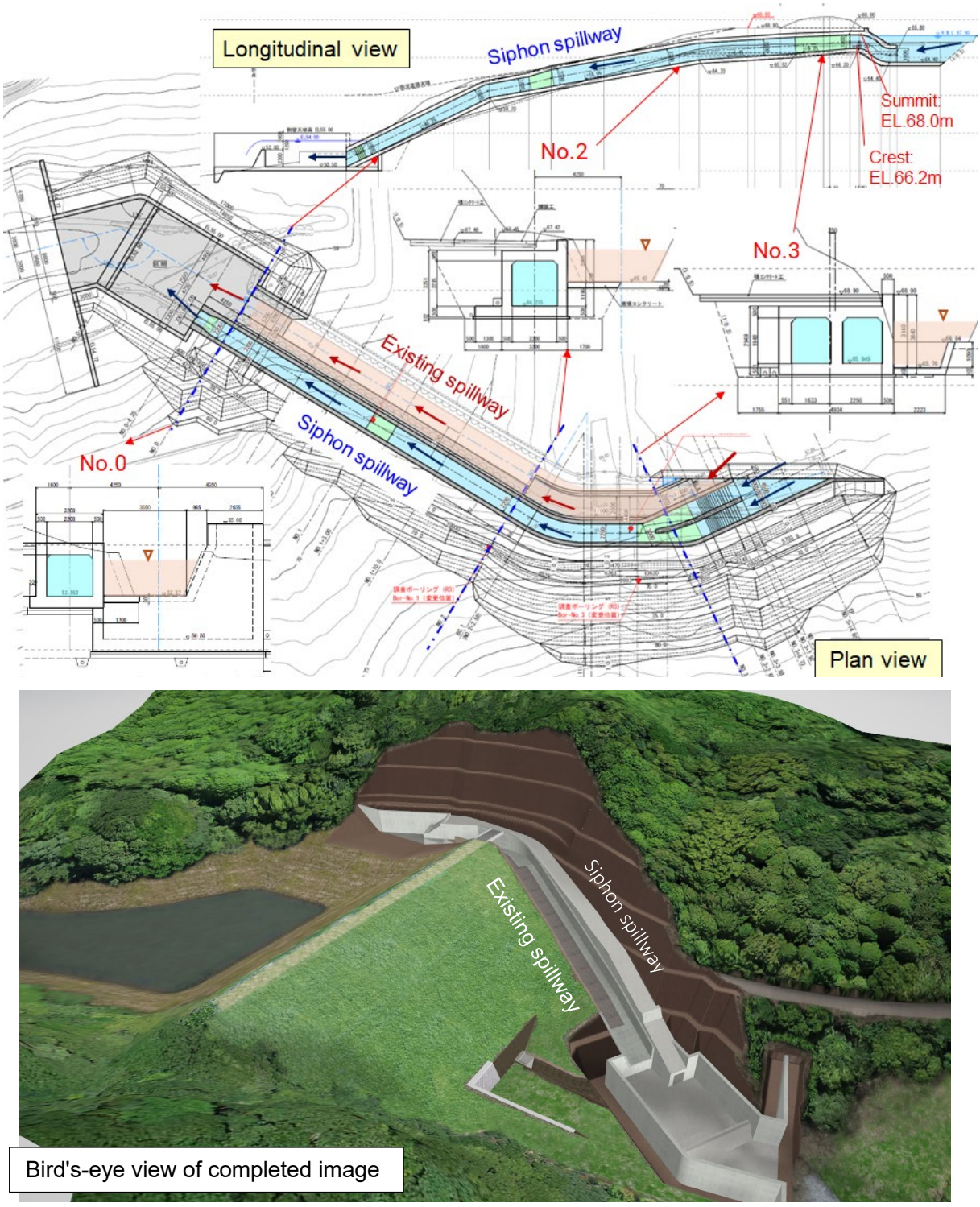


Fig.15 – Final spillway design view of U Dam and completion image

6. CONCLUSIONS

6.1. Conclusions in U Dam siphon spillway design

This study is based on the research and development of a new siphon spillway targeting the U dam, which is a small earthen dam.

a. First hydraulic model experiments revealed that the discharge capacity was slightly insufficient compared to the inflow design flood of 44 m^3/s, and the negative pressure in the pipe exceeded critically the allowable limit -7 mH_2O.

b. To improve the negative pressure, the hydraulic model of the siphon spillway was redesigned by transferring the inlet closer to the reservoir and increasing the cross-sectional area of the inflow section.

c. The second hydraulic model experiment revealed that the measured negative pressures were bigger than the allowable limit with enough margin, and they were generally consistent with the piezo gradient line by calculation. In addition, a target flow of 44 m^3/s at the HWL was discharged with increasing its flood discharge steadily without rapid increase of water level. It implies the combination of siphon spillway and existing spillway was very effective.

d. After the above confirmation, a detailed design of the spillway of the U dam was conducted, and the construction works are expected to start in 2025 fiscal year.

6.2. Innovative features not found in previous siphon spillway

This siphon spillway is functionally a conventional emergency-use spillway, but has the following features not found in previous siphon spillways.

a. Long and gentle slope: The U Dam was the first dam to arrange a siphon spillway with a long and gentle slope downstream. Its shape overlaps with the image of the “Siphon Chain Model”, which is recently disseminated.

b. Change of cross-sectional area: There are some changes in the cross-sectional area to cope with the negative pressure. In particular, the inflow section is large.

c. Controlled negative pressure: Although more negative pressures occurred from the summit to the downstream gradient change point, negative pressures were controlled to avoid exceeding allowable values by lowering the inflow velocity.

d. Higher velocity at outlet: Because the siphon makes effective use of the water heads between the inlet and outlet, the outlet velocity is higher than that of conventional siphons.

e. Less air sucking: Normal siphon spillways are prone to air sucking when flow conditions are disturbed. This new spillway enabled a significant reduction in air sucking by narrowing the outlet, lowering the inflow velocity, and discharging underwater.

6.3. Cost advantages of the new siphon spillway

The following are cost advantages to use in embankment dams.

a. Downsizing of spillway: Because the inflow velocity can be increased several times compared to the free-overflow type, the necessary cross-sectional area can be significantly reduced. This implies a great cost reduction, including excavation work.

b. Smaller intake water depth: Because the intake water depth of the siphon is smaller, the spillway construction cost can be effectively reduced by reducing the excavation depth and slope-cutting area.

c. More suitable layout: Because this new siphon spillway can be slender and compact, it is possible to adopt a topographically economical alignment, such as sharing the entrance and outlet areas with the adjacent existing spillway efficiently.

d. More effective energy dissipating: By extending the spillway pipe downstream, it was easier to place the siphon outlet underwater. Thus, the cost of downstream facilities can be reduced by effectively decreasing water energy.

e. Easier construction: Generally, because this siphon spillway is constructed using the existing spillway with minimal diversion or cofferdams, it not only reduces costs but also drastically shortens the construction period.

ACKNOWLEDGEMENTS

I would like to express my gratitude to Sasebo City and West Japan Engineering Consultants, Inc., for various cooperations.

REFERENCES

[1] Kawasaki, H. (2024). Discharge Innovation by the New-type Siphon Spillway. Annual Lectures, Japan Society of Dam Engineers

[2] U.S. Reclamation Manual, Chapter 12

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

CLIMATE SHIFT IMPACTS ON HYDROLOGICAL PATTERNS IN MINING AREAS LOCATED IN NORTHERN BRAZIL

Denis Costa[1] Progen S.A	**Francisco Rodrigues**[2] Progen S.A	**Darym Ferrari**[3] Progen S.A	**Keyla Oliveira**[4] Vale.S.A
Andrea Portes[5] Vale.S.A	**Stephanie Yokozawa**[6] Vale.S.A	**Fernanda Sol**[7] Vale.S.A	**Paulo Gomide**[8] Vale.S.A

ABSTRACT

The Northern region of Brazil, which 80% of its area is covered by the Amazon Forest, is one of the most vulnerable parts of the world to the effects of climate changes and variability and consequently impacts on local hydrological regime leading to severe droughts or extreme floods. In this region, average annual precipitation may reach 2000 to 3000 mm easily and the rainy season is mostly associated with the southward displacement of the Intertropical Convergence Zone (ITCZ). On this turn, is well known that global climate oscillation (e.g., El Niño Southern Oscillation-ENSO, and anomalies on the Atlantic Ocean Sea Surface Temperature) plays a main role in the position of the ITCZ, directly impacting the amount of rain that's fall in this region. It is in this area, especially in the state of Pará, where is located largest iron ore mining areas of Brazil such as Serra dos Carajás, including their Tailings Storage Facilities (TSF) to store waste material and control water supply and consequently downstream floods. Therefore, this study focuses to investigate possible connections between spatiotemporal hydrological variabilities, in areas where iron ore mining is placed, and large-scale climate phenomena such as ENSO and Atlantic SST anomalies. Multivariate statistical techniques (e.g., wavelets analysis) will be applied as an attempt to capture signals of changes in the climate and consequently shifts on local hydrological pattern by analysing differences in the time-frequency of accumulated precipitation comparing the signal spectrum of wavelets before the 2000's and after 2000's to 2024. The results will be based on a qualitative model to contribute as a step toward the development of long-term hydrological forecasting over mining areas located in the Northern region of Brazil to improve water management in the TSF's mainly related to the occurrence of extreme events of precipitation which may represent risks related to uncontrolled flood waves on these structures.

Keywords: Tailing Storage Facilities, Hydrological extreme events, Climate shift, Amazon region.

[1]denisdudacosta@gmail.com, Brazil

1. INTRODUCTION

The Northern region of Brazil, particularly the Amazon biome, is one of the most critical areas on Earth vulnerable to climate change and variability in local precipitation regimes. The Amazon's climate plays a pivotal role in driving atmospheric processes, influencing weather patterns both locally and across the continent [1]. As a result, natural climate variability and anthropogenic changes are highly sensitive and have profound effects on precipitation and hydrological patterns. The local precipitation regime, especially in the eastern Amazon, is strongly influenced by global climate phenomena, including the Intertropical Convergence Zone (ITCZ), the Atlantic Meridional Mode (AMM), and the El Niño-Southern Oscillation (ENSO) [2].

The ITCZ, a region of converging trade winds near the equator, is a primary driver of precipitation in the Amazon basin. Seasonal shifts in the ITCZ's position determine the onset and intensity of the rainy season, particularly in eastern Amazonia. Variability in the ITCZ's movement can lead to prolonged droughts or excessive rainfall, both of which significantly impact the region's ecosystems, local livelihoods, and economic activities. The AMM, characterized by variations in sea surface temperatures (SSTs) in the tropical Atlantic, further influences the ITCZ's position and, consequently, regional rainfall patterns [3]. Positive AMM phases, marked by warmer SSTs in the northern tropical Atlantic, are associated with reduced rainfall in the southern Amazon, while negative phases often enhance precipitation.

ENSO, one of the most extensively studied climate phenomena, has profound effects on the Amazon's climate. El Niño events, characterized by anomalously warm SSTs in the equatorial Pacific, typically induce drought conditions in the Amazon. Conversely, La Niña events, marked by cooler SSTs, are often associated with increased rainfall in the region. The interaction between ENSO and Atlantic SSTs can either amplify or mitigate these effects, underscoring the complexity of climate dynamics in Northern South America [1, 4].

Changes in precipitation patterns driven by these climate phenomena have significant implications for the region. These include water shortages caused by droughts, flooding, increased soil erosion, and direct impacts on mining operations and their Tailings Storage Facilities (TSFs). Such shifts in climate can profoundly affect the region's economic stability and security. Recent studies suggest that the frequency and intensity of extreme climate events in the Amazon may be increasing due to global warming, raising concerns about long-term changes in hydrological patterns [5]. Therefore, understanding the role of climate phenomena in modulating precipitation variability is crucial for predicting and mitigating the impacts of climate change.

The primary objective of this study is to investigate the influence of these climate phenomena on precipitation patterns using multivariate statistical techniques. By analyzing differences in the time-frequency of precipitation during the rainy season in the eastern Amazon, this study aims to identify signals of climate change and shifts in local hydrological patterns. The findings from this analysis could provide a foundation for developing long-term forecasting models to improve water management strategies, particularly for TSF water balance and flood control in mining areas located in Northern Brazil.

2. MATERIALS AND METHODS

2.1. Study Area

The study area considered in this research is the Itacaiúnas River Basin, located in Northern Brazil, specifically in the southern part of the state of Pará. This basin encompasses the eastern region of the Brazilian Amazon (5°00’00”S to 8°00’00”S latitude and 48°00’00”W to 52°00’00”W longitude). Covering an area of 41,732 km² [6], the Itacaiúnas Basin is one of the main tributaries of the Tocantins-Araguaia River (Figure 1).

The primary economic activity in this region is mineral exploration, particularly iron ore mining in the Serra dos Carajás, which hosts some of the largest iron ore mining operations in Brazil. Additionally, the area contains a significant number of Tailings Storage Facilities (TSFs). These structures are designed to manage waste materials and control water supply and flooding within the mining sites.

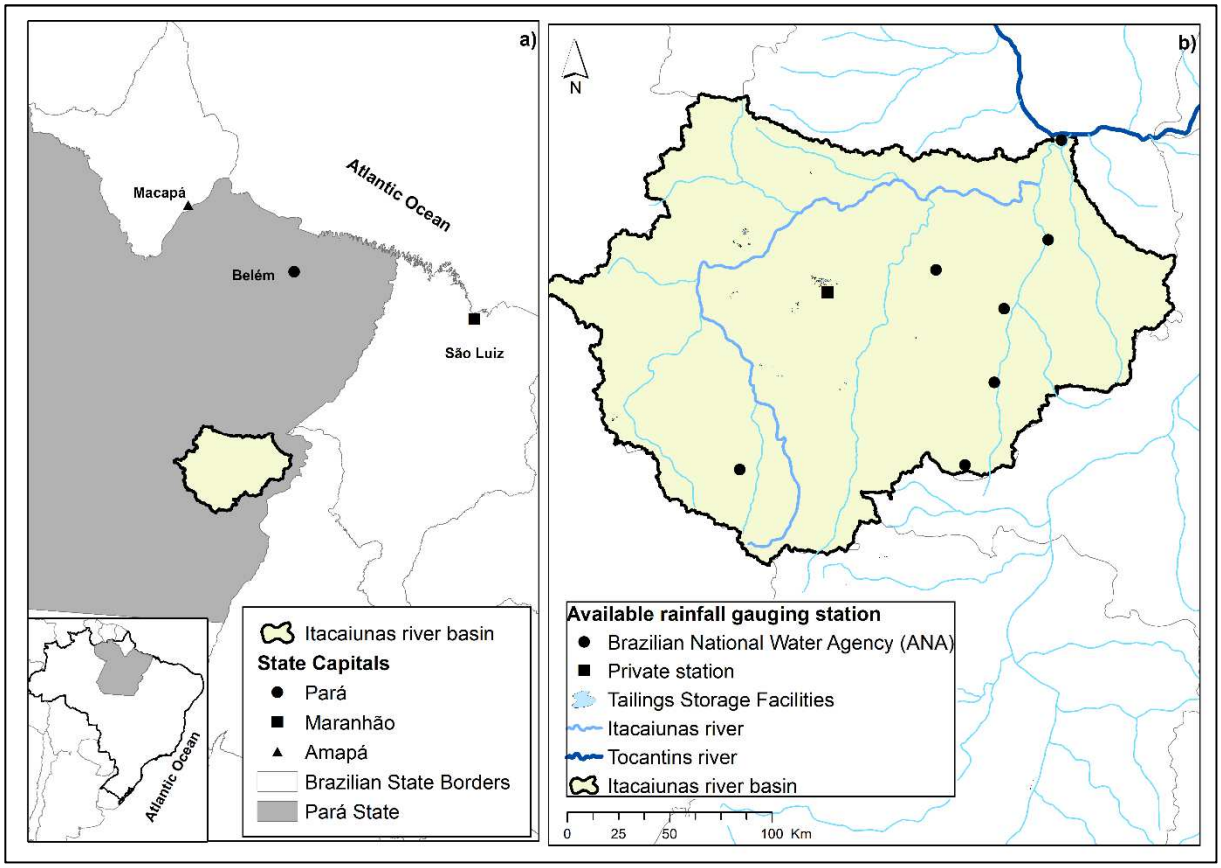


Figure 1. Location map: (a) the overview location of the Itacaiunas basin in Brazil, and (b) the study area and available rainfall stations.

A defining characteristic of tropical regions is the division of the precipitation regime into distinct rainy and dry seasons. In the Itacaiúnas Basin, the rainy season occurs from November to April, with January, February, and March being the wettest months. During this period, rainfall accounts for approximately 50% of the average annual precipitation. These characteristics align with the climatic conditions of the zonally oriented band of the Amazon, as described by [4]. The study further highlights that the Amazon region can be divided into three major homogeneous areas based on rainfall regimes. Figure 2 illustrates the variations and average conditions of the private gauging station, which is highly representative of the precipitation regime in the Itacaiúnas Basin.

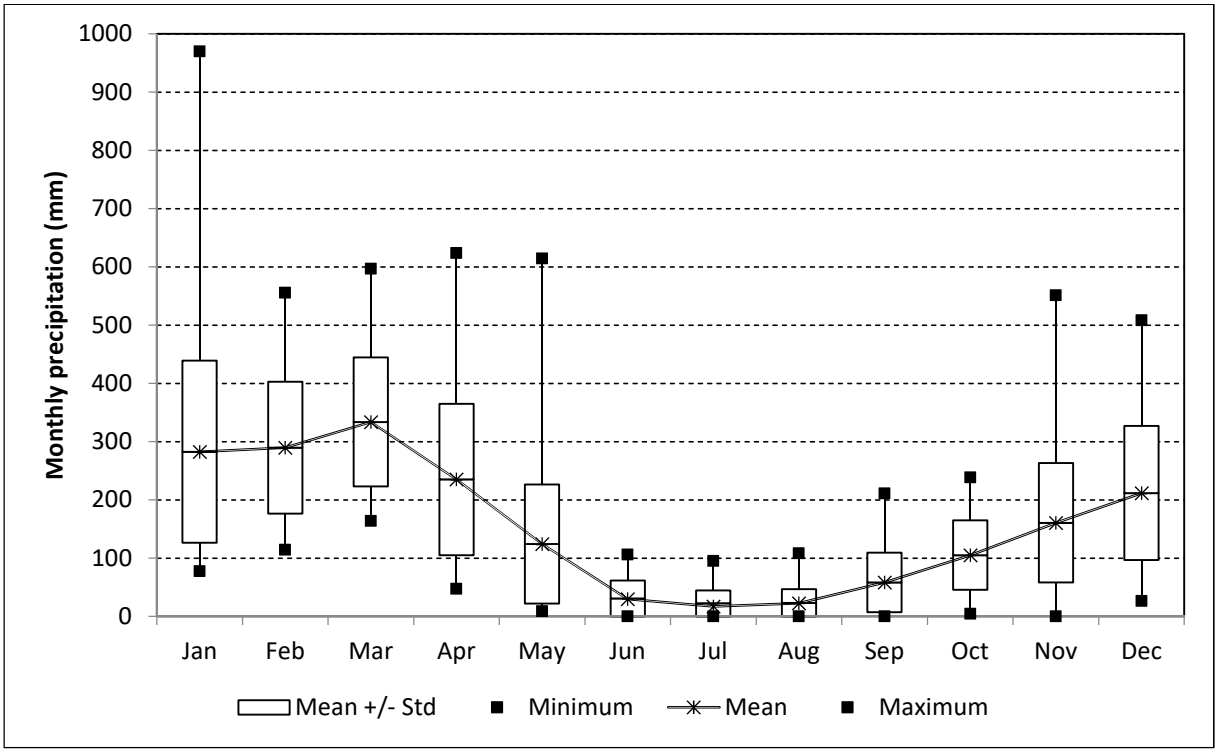


Figure 2. Monthly average precipitation considering Private station with the degree of dispersion.

2.2. Data Collection

The data collection process assessed the temporal and spatial distribution of rainfall stations within the study area. Compared to other regions of Brazil, the Amazon region exhibits a significant gap in the spatial coverage of monitoring stations. Thus, by applying a criterion of a minimum 30-year time series, it was possible to filter and identify only the 8 rainfall stations listed in Table 1.

Table 1. Available rainfall stations.

Station Code	Lat	Long	Source	Period
549002	-5.3656	-49.125	ANA	1973-2014
549004	-5.9347	-49.677	ANA	1982 - 2023
549007	-5.8031	-49.183	ANA	1988 - 2023
649000	-6.4278	-49.42	ANA	1984 - 2023
649001	-6.7947	-49.549	ANA	1984 - 2023
649002	-6.1053	-49.378	ANA	1985 - 2023
650001	-6.8153	-50.539	ANA	1984 - 2023
Private	-6.7153	-50.439	Private	1982 - 2024

The time series of rainfall data consist of daily precipitation records from eight gauging stations located within the study area. These time series primarily cover the period from 1984 to 2023,

with data sourced from the Brazilian National Water Agency (ANA) for seven stations and one station owned by a private company (referred to here as the Private station).

Due to the limited number of available stations and their uneven spatial distribution across the watershed (Figure 1 (b)), this study assumes homogeneous rainfall characteristics [2]. The Private station, which is the closest rainfall station to Serra dos Carajás and the surrounding Tailings Storage Facilities (TSFs), is used to represent the rainfall conditions of the eastern Amazon region and the Itacaiúnas Basin.

The daily precipitation time series from the Private station were first aggregated into trimestral accumulations, focusing on the period from January to March (P-JFM), which corresponds to the rainiest season in the study area. Subsequently, the time series were divided into two distinct periods—1982 to 2022 and 2003 to 2024—to validate potential anomalies caused by long-term climate variability and possible shifts in the hydrological seasonality of the region.

To investigate the influence of global climate phenomena on local hydrological variability, two climate indices were utilized: the El Niño-Southern Oscillation (ENSO), represented by the Southern Oscillation Index (SOI), and the Atlantic Meridional Mode (AMM) [7, 8]. Monthly time series for both indices were obtained from the National Center for Atmospheric Research (NCAR) (http://www.cgd.ucar.edu/cas/catalog/climind/) and the National Oceanic and Atmospheric Administration (NOAA) (http://www.esrl.noaa.gov/psd/data/climateindices/list/).

2.3. Multivariate Analysis

To evaluate the temporal variability between the January-March precipitation (P-JFM) and the climate indices, a time-frequency analysis was conducted using wavelet techniques, as applied in previous studies [3, 9]. The analysis considered lagged climate indices up to four months prior to the rainy season to capture potential precursor signals.

Wavelet analysis is a more robust method for analyzing non-stationary time series compared to traditional techniques such as Fourier analysis [10]. The continuous wavelet transform (CWT) enables the examination of time series across different scales of periodicity, identifying dominant modes of variability and their statistical significance over time [11, 12]. The CWT of a time series X can be defined as:

$$W_{X,\Psi}(s,t) = (X(t) * \Psi_s(t)), \quad (1)$$

where t is the time and Ψ_s is the mother wavelet at the scale s. The Morlet mother wavelet is widely used in geophysical signal analysis [12,13], and its equation is defined as:

$$\Psi_s(\eta) = \pi^{-1/4} e^{i\omega_s \eta} e^{-\eta^2/2}, \quad (2)$$

where, η and ωs are time and frequency respectively, both non-dimensional. Cross wavelet analysis (XWT) was also employed to assess the covariance between two variables. This method identifies periods of high common power in the time-frequency domain between the January-March precipitation (P-JFM) and the climate indices [10, 12]. By applying XWT, the analysis enhances the understanding of temporal cycles in the precipitation time series that are linked to global climate phenomena. The XWT spectrum is defined by:

$$W_{XY}(s,t) = W_X(s,t)\, W_Y^*(s,t), \tag{3}$$

where * denotes the complex conjugate of $W_Y(s,t)$.

All calculations were performed using the MatLab® software suite. The output maps were produced in ArcGIS® 10.2. The CWT was computed using the software provided by Torrence and Compo [12] (http://atoc.colorado.edu/research/wavelets/), while the Cross Wavelets was computed using a software package provided by Jevrejeva et al. [10], at the URL: http://www.pol.ac.uk/home/research/waveletcoherence/.

3. RESULTS AND DISCUSSIONS

3.1. Variability pattern of P-JFM precipitation

The wavelet analysis of the precipitation time series from the Private station is illustrated in Figure 3. Due to the length of the time series, the continuous wavelet transform (CWT) was analyzed over its entire duration rather than being split into two periods (i.e., 1982–2000 and 2000–2024) to ensure the capture of low-frequency signals. The CWT reveals high power with statistical significance in high-frequency bands (2–4 years) throughout most of the time series, suggesting interannual variability driven by persistent climate phenomena in the region. In contrast, the power spectrum for low-frequency bands (8–16 years) is weaker compared to high frequencies and only becomes more significant after the 2000s. This shift may indicate changes in the periodicity pattern of precipitation variability during the rainy season (P-JFM) in the Itacaiúnas Basin at a multiannual scale.

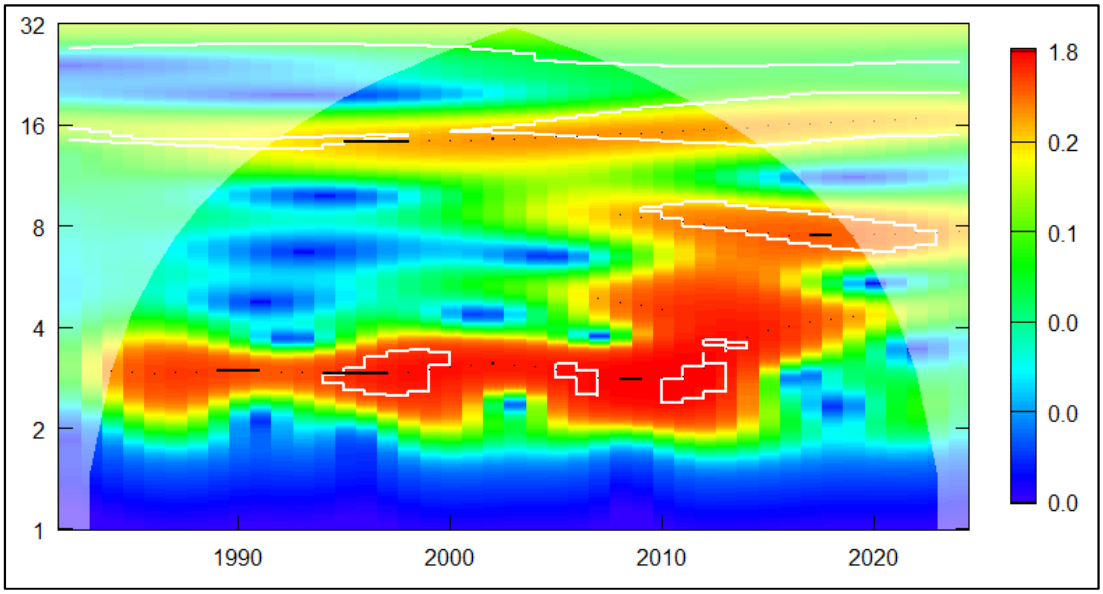


Figure 3. CWT from precipitation data. The white contours highlight the 95% confidence level. X-axis represents the time series in years, Y-axis the oscillation period (years), and bar colours the power spectrum.

3.2. Climate Oscillation influences P-JFM variability

To investigate the influence of climate phenomena on the periodicity patterns of local precipitation in the Itacaiúnas Basin, a cross wavelet transform (XWT) analysis was conducted between the Atlantic Meridional Mode (AMM) and Southern Oscillation Index (SOI) climate indices and the January-March precipitation (P-JFM). The XWT revealed significant high common power with a one-month lag between AMM and P-JFM (Figure 4), particularly for frequencies around a 16-year period across the time series. Additionally, significant power spectra were observed for frequencies between 2–4 years and 8 years, though these were predominantly evident only after the 2000s.

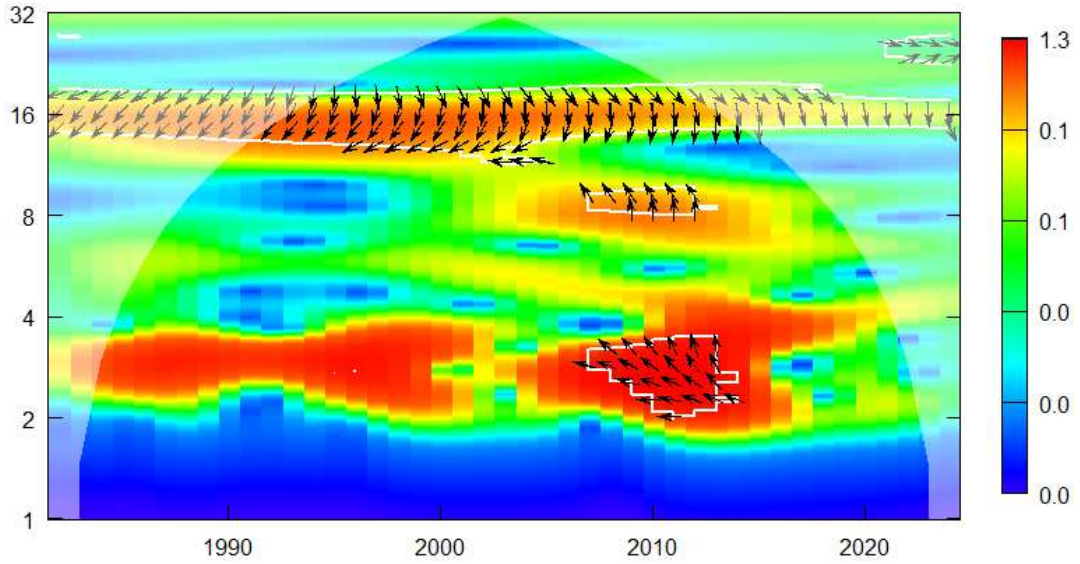


Figure 4. Cross Wavelet Transform between AMM climate index in December and P-JFM. The thick withe contours represent 95% of confidence level.

For the Southern Oscillation Index (SOI), a similar pattern to that observed between the Atlantic Meridional Mode (AMM) and precipitation is evident (Figure 5), particularly for low-frequency bands (2–4 years) after the 2000s. High-frequency signals around a 16-year period were also identified, underscoring the influence of SOI on the local rainy season. However, the results suggest that this influence is weaker compared to that of AMM over the same time span.

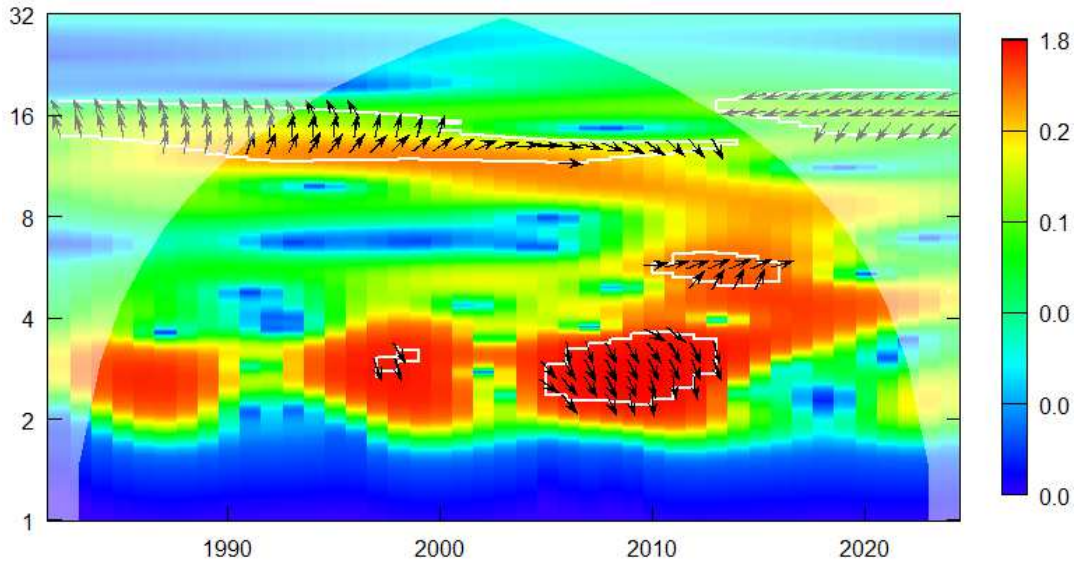


Figure 5. Same as Figure 4 for SOI climate index in October and P-JFM.

In general, the results demonstrate that the variability patterns in January-March precipitation (P-JFM), as identified through continuous wavelet transform (CWT) analysis, align closely with the cross wavelet transform (XWT) results for the Atlantic Meridional Mode (AMM) and Southern Oscillation Index (SOI) climate indices in December and October, respectively. This suggests that AMM and SOI act as modulators of both long- and short-term variability in the

local rainy season within the Itacaiúnas Basin. Thus, above- or below-normal precipitation conditions in the region can be directly linked to the state of these climate indices, with a lag of 1–3 months relative to the rainy period of the region.

As highlighted by [3], the interannual and multiannual variability patterns of AMM are key indicators of shifts in the position of the Intertropical Convergence Zone (ITCZ) over the tropical Atlantic. The ITCZ, in turn, is widely recognized as the primary driver of precipitation variability in the eastern Amazon during the rainy season [1]. The findings of this study corroborate this relationship, showing a one-month lagged correlation between AMM and P-JFM. Notably, AMM exhibited a statistically significant short-term connection only after the 2000s, while its long-term influence on wetter or drier conditions during the rainy season persisted throughout the entire time series. This indicates that AMM is a reliable predictor of rainy conditions in the study area and a valuable parameter for monitoring long-term hydrological shifts associated with Atlantic climate oscillations.

The influence of the El Niño-Southern Oscillation (ENSO), represented by the SOI index, on P-JFM in the Itacaiúnas Basin is also evident, consistent with previous studies [4, 14]. The SOI conditions in October (i.e., a three-month lag relative to the local rainy season) appear to directly impact wetter or drier conditions during the rainy season, with increased statistical significance after the 2000s. The three-month lagged influence of ENSO, compared to the one-month lag of AMM, suggests that SOI's primary role is mediated through teleconnections. In other words, conditions in the Pacific Ocean influence the Atlantic dipole mode, which in turn affects precipitation patterns in the eastern Amazon [14].

Although the results indicate that AMM and SOI have had a stronger influence on local precipitation in the Itacaiúnas Basin after the 2000s, the relatively short time series available for the study area limits the ability to draw definitive conclusions about significant shifts in local hydrological patterns, particularly for high-frequency, short-term influences. Nevertheless, the persistent long-term influence of both climate indices on precipitation is evident, with no abnormal trends observed when comparing the phases of these indices with P-JFM. This suggests that AMM and SOI are reliable predictors of the rainy season in the region and can serve as parameters for monitoring shifts in the hydrological patterns of the Itacaiúnas Basin. However, it is important to note that these comparisons are based solely on the rainiest period of the year. Isolated extreme events, which were not investigated in this study, may exhibit distinct anomalies and could provide further insights into changes on local climate and in hydrological patterns when analyzed in terms of their frequency, intensity, duration, and magnitude.

4. CONCLUSIONS

The variability patterns of precipitation during the rainy season in the eastern Amazon, specifically within the Itacaiúnas River Basin, were investigated to assess potential shifts in local hydrological patterns associated with temporal oscillations of climate phenomena such as the El Niño-Southern Oscillation (ENSO) and the Atlantic dipole mode, using multivariate statistical analysis.

Wavelet analysis of precipitation data from the Private station (1982–2024) revealed distinct variability patterns. High-frequency signals (2–4 years) exhibited statistically significant cycles throughout the entire series, indicating that interannual variability is likely linked to persistent climatic phenomena. After 2000, low-frequency signals (8–16 years) emerged, suggesting multiannual influences on rainy season precipitation (P-JFM).

Cross-wavelet analysis demonstrated that the Atlantic Meridional Mode (AMM) and the Southern Oscillation Index (SOI) modulate both short- and long-term rainfall variability. AMM exhibited stronger correlations, particularly at 16-year frequencies with a one-month lag, while SOI reinforced its influence on local precipitation patterns for both short- and long-term variability with a three-month lag. Both indices significantly influenced precipitation variability after the 2000s at low frequencies, with ENSO impacting precipitation through teleconnections that link Pacific and Atlantic conditions, thereby modulating the signal of AMM.

Although the influence of AMM and SOI appears more pronounced after 2000, the relatively short time series limits the ability to draw definitive conclusions about shifts in hydrological patterns at high frequencies. Isolated precipitation events were not analyzed but could provide additional insights into potential changes in precipitation patterns and the occurrence of extreme events in the region. Nevertheless, the results clearly demonstrate the influence of Atlantic and Pacific Ocean conditions on long-term variability, particularly at a 16-year scale, indicating that AMM and SOI are reliable predictors of rainy season conditions in the Itacaiúnas Basin.

Understanding the effects of changes in climate phenomena, as represented by the Atlantic Meridional Mode (AMM) and Southern Oscillation Index (SOI) signals, and their impacts on shifts in the local hydrological regime, can significantly improve the forecasting of rainy season variability driven by global climate phenomena. This knowledge can support the development of advanced forecasting systems for water resources management, particularly in relation to Tailings Storage Facilities (TSFs) in mining sites within the basin. Such systems can enhance planning and preparedness for the rainy season, ensuring more effective management of mining operations and water resources.

REFERENCES

[1] Uvo, C. B., & Nobre, P. (2002). Climate variability in the Amazon basin. Hydrological Sciences Journal, 47(4), 595-606.

[2] Yoon, J. H., & Zeng, N. (2010). An Atlantic influence on Amazon rainfall. Climate Dynamics, 34(2-3), 249-264.

[3] Costa, D. D., Uvo, C. B., Rolim Da Paz, A., De Oliveira Carvalho, F., & Fragoso Jr, C. R. (2018). Long-term relationships between climate oscillation and basin-scale hydrological variability during rainy season in eastern Northeast Brazil. *Hydrological Sciences Journal*, *63*(11), 1636–1652. https://doi.org/10.1080/02626667.2018.1523614

[4] Uvo, Cíntia Bertacchi. 1998. *Influence of Sea Surface Temperature on Rainfall and Runoff in Northeastern South America : Analysis and Modeling*. Report / Department of Water Resources Engineering, Lund Institute of Technology, Lund University, Sweden: 1021. Univ.https://search.ebscohost.com/login.aspx?direct=true&AuthType=ip,uid&db=cat07147a&AN=lub.1081302&site=eds-live&scope=siteManso, J. (2017). Predicting the behaviour of rockfill embankments, PhD Thesis, IST.

[5] Marengo, J. A., et al. (2018). Changes in climate and land use over the Amazon region: Current and future variability and trends. Frontiers in Earth Science, 6, 228.

[6] Silva, R. C. F., Pimentel, M. A. D. S., & Araújo, A. N. (2022). Caracterização Morfométrica e Geomorfológica da Bacia Hidrográfica do Rio Itacaiunas (BHRI), Amazônia Oriental, Brasil. *Revista Brasileira De Geografia Física*, *15*(3), 1556–1563. https://doi.org/10.26848/rbgf.v15.3.p1556-1563.

[7] Trenberth, K. E. The Definition of El Niño. Bull. Am. Meteorol. Soc. 1997, 78, 2771–2777, doi:10.1175/1520-0477(1997)078.

[8] Chiang, J. C. H.; Vimont, D. J. Analogous Pacific and Atlantic meridional modes of tropical atmosphere-ocean variability. J. Clim. 2004, 17, 4143–4158, doi:10.1175/JCLI4953.1.

[9] Costa, D. D.; da Silva Pereira, T. A.; Fragoso, C. R.; Madani, K.; Uvo, C. B. Understanding Drought Dynamics during Dry Season in Eastern Northeast Brazil. Front. Earth Sci. 2016, 4, doi:10.3389/feart.2016.00069.

[10] Jevrejeva, S.; Moore, J.C.; Grinsted, A. Influence of the Arctic Oscillation and El Niño-Southern Oscillation (ENSO) on ice conditions in the Baltic Sea: The wavelet approach. J. Geophys. Res. 2003, 108, 1–11, doi:10.1029/2003JD003417.

[11] Grinsted, A.; Moore, J. C.; Jevrejeva, S. Application of the cross wavelet transform and wavelet coherence to geophysical time series. Nonlinear Process. Geophys. 2004, 11, 561–566, doi:10.5194/npg-11-561-2004.

[12] Torrence, C.; Compo, G. P. A Practical Guide to Wavelet Analysis. Bull. Am. Meteorol. Soc. 1998, 79, 61–78, doi:10.1175/1520-0477(1998).

[13] Liu, Y.; Brown, J.; Demargne, J.; Seo, D. J. A wavelet-based approach to assessing timing errors in hydrologic predictions. J. Hydrol. 2011, 397, 210–224, doi:10.1016/j.jhydrol.2010.11.040.

[14] Tedeschi, R. G.; Grimm, A. M.; Cavalcanti, I. F. A. Influence of Central and East ENSO on precipitation and its extreme events in South America during austral autumn and winter. Int. J. Climatol. 2016, 36, 4797–4814, doi:10.1002/joc.4670.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

INVESTIGATION OF THE BREACH SHAPE EFFECTS ON THE FLOOD PROPAGATION IN THE CASE OF THE DISTORTED PHYSICAL MODEL OF ÜRKMEZ DAM

Tanıl Arkış[1]
Tecnimont, Bursa, Turkiye

Gökçen Bombar[2]
Department of Civil Engineering, İKÇU, İzmir, Turkiye

Mehmet Şükrü Güney[3]
Department of Civil Engineering, İEU, İzmir, Turkiye

Gökmen Tayfur[4]
Department of Civil Engineering, İYTE, İzmir, Turkey

ABSTRACT

Flood propagation due to trapezoidal and triangular breaches is studied by means of a distorted physical model of Ürkmez Dam, built in open area of Hydraulics Laboratory within Dokuz Eylül University. The physical model with a horizontal scale of 1/150 and vertical scale of 1/30 contains the reservoir, the dam and the dam downstream valley until the sea. The geometric parameters of the model are determined by using the available maps. Accordingly, the model of the dam of 2.84 m long and 1.07 m high has a reservoir of about 12 m³. The downstream area, corresponding to Ürkmez village, is nearly 300 m². The residences, highway and Ürkmez creek are also represented in the physical model. The breaching problem is investigated for partial failures simulated by trapezoidal and triangular breaches in the dam body. The temporal water depths are measured by using e+ WATER L sensors and the velocities are determined by means of UVP transducers. The variations of the water depths and local velocities with time are also studied numerically by means of the software FLOW3D. The numerical analyses demonstrated a close correlation in depth-averaged velocities and water depths between the two turbulence model scenarios. The discrepancies observed between experimental and numerical results are likely attributable to the specific mesh sizes and roughness values selected. The numerical results are obtained by using RNG and k - ε turbulence models. Thus, obtained water depth and velocity values are compared with those obtained from the experiments performed in the scope of the project TÜBİTAK (Scientific and Technological Research Council of Türkiye) 110M240. The numerical results were found to be close to each other for depth average velocities and water depths, in both turbulence model cases. Future investigations should consider employing a variety of mesh dimensions and roughness values to refine numerical analyses and facilitate more comprehensive comparisons between experimental and numerical outcomes.

Keywords: Distorted physical model, Dam failure, Flood propagation, Trapezoidal breach, Triangular breach

[1]arkistanil@hotmail.com, Türkiye

1. INTRODUCTION

A review of the literature reveals the presence of both experimental and numerical studies. Several investigations compare laboratory experimental results with numerical modeling outputs. For instance, Zhang et al. (2016) [1] examined the flow and sediment transport resulting from dam breaks under various bed conditions using a two-dimensional numerical model. Hooshyaripor et al. (2017) [2] conducted dam break experiments with sudden release gates inclined at 30° - 90°, employing different hydrographs, and presented two empirical relationships applicable for estimating peak discharge and maximum water levels at any point downstream. Kamra et al. (2018) [3] performed experimental and numerical studies in two and three dimensions for scenarios where the dam break flow is directed towards a vertical wall. They employed Reynolds-averaged Navier-Stokes equations for their numerical simulations. Upon reviewing the existing literature, a notable gap was identified concerning research on the development of precise numerical models for distorted physical models under various constraints and parameters. Consequently, this study was conducted to address this identified deficiency.

The physical model was designed and built in the scope of the research project TÜBİTAK 110M240 titled "*Physical and numerical investigation of dam break flood waves - Application to real dams in GIS environment*" according to the Froude similarity law since the gravitational force was dominant. Within the scope of this project, the primary objective is to solve and compare experimental results on a numerical model for the first time. The horizontal and vertical scales of the model were selected such that it can be built and operated conveniently and still be large and big enough to measure flow depths and velocities with sufficient accuracy. According to the available space in the open area of the Hydraulics Laboratory within Dokuz Eylül University, the horizontal and vertical scales were selected as ℓ_{xr}: 1/150 and ℓ_{zr}: 1/30, respectively. Thus, the distortion coefficient is m= ℓ_{zr} / ℓ_{xr} = 5 and the following scales may be defined [4]:

The slope scale:

$$S_r = \ell_{zr}/\ell_{xr} = m \tag{1}$$

The cross-sectional area scale:

$$A_r = \ell_{xr}\ \ell_{zr} \tag{2}$$

Volume scales:

$$V_r = {\ell_{xr}}^2 \ell_{zr} \tag{3}$$

As the time is a characteristic parameter, in addition to the Froude number the Strouhal number also must be taken into consideration:

$$St = \frac{vt}{\ell} \tag{4}$$

where v, t and ℓ denote the velocity, the time and the length, respectively. The Froude number is formed by a typical velocity which is in the x direction of the flow [5]. Therefore, the Froude velocity scale is interpreted as

$$v_r = v_{xr} = \sqrt{\ell_{zr}} = 1/5.48 \tag{5}$$

which gives for the time scale of the distorted model of

$$t_r = \frac{\ell_{xr}}{\sqrt{\ell_{zr}}} = \frac{1}{m}\sqrt{\ell_{zr}} = 1/27.4 \tag{6}$$

The Ürkmez Dam, located on the Ürkmez Stream in İzmir, Türkiye, was constructed between 1985 and 1989 for the purposes of irrigation and flood control. The geometric characteristics of Ürkmez Dam (prototype) and its distorted physical model are given in Table 1. Ürkmez Dam and upstream part are shown in Fig. 1.

Table 1 – The geometric characteristics of the prototype and physical model.

Characteristics	Prototype	Physical model
Crest length (m)	426	2.84
Crest width (m)	12	0.08
Dam height from its base (m)	32	1.07
Lake volume at minimum water level (m^3)	375 000	0.556
Lake volume at maximum water level (m^3)	8 625 000	12.778
Lake volume at normal water level (m^3)	7 950 000	11.778
Lake active volume (m^3)	7 575 000	11.222

Fig. 1 – The general view of Ürkmez Dam and upstream part

An area of about 300 m^2 was reserved to the construction of the distorted physical model. The construction area was arranged and concrete was poured after the placement of the reinforcing bars. The cross-sections concerning the dam lake were drawn by using the related maps. The sections were manufactured using metal profiles. The downstream region was constructed in a similar manner and the surface was lined with concrete. The downstream part of the physical model was completed after placing the highway, the Ürkmez creek, and the residences.

The dam bodies were designed and manufactured from steel. The dam bodies were designed such that they had a fixed and a movable part. The movable part was trapezoidal in the first dam body (Fig. 2) and triangular in the second dam body (Fig. 3).

Fig. 2 – Dam body with trapezoidal moving part

Fig. 3 – Dam body with triangular moving part

2. MEASUREMENT DEVICES AND METHODOLOGY

e+ WATER L® [6] level sensors shown in Fig. 4 measured the flow depths. These level sensors were distributed over the lake and downstream part of the Ürkmez dam. Level measurement values have been automatically compensated to adjust to the alterations in air pressure and water density, which are influenced by temperature changes, hence providing automatic adjustment of water levels within the model in response to any fluctuations in water density or temperature. The velocities were measured by means of UVP (ultrasonic velocity profiler) transducers shown in Fig. 5. Fig. 6 shows locations of the level meters (S1, S2, …, S11) and UVP transducers (U1, U2, …, U5) [7]

Fig. 4 – e+ WATER L level sensors

Fig. 5 – UVP transducers

Fig. 6 – Location of the level meters and UVP transducers at the downstream of the dam

2.1. Flood Hydrographs

The partial dam break was simulated by lifting the trapezoid or triangular shaped part of the dam body. The time-dependent water levels in the lake were recorded by means of the level meter S1. The so measured values are converted to the volume by using the previously determined water depth - volume curve of the model reservoir, shown in Fig. 7.

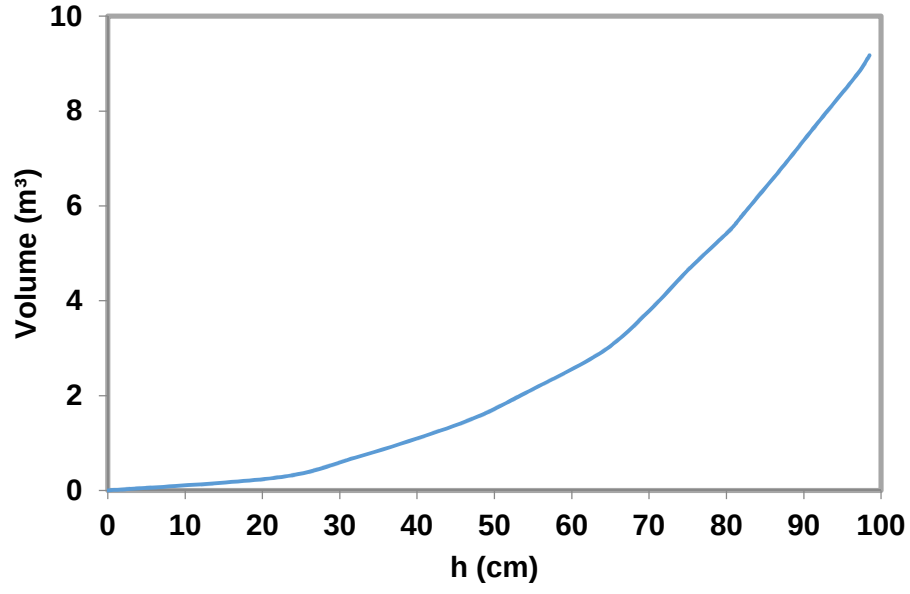


Fig. 7 – The volume-water depth curve

Time-dependent water depth values allowed the determination of the changes in the reservoir volume and the discharge values of the hydrograph were obtained by dividing the volume differences to the elapsed time intervals. The so obtained hydrographs corresponding to trapezoidal and triangular breach cases are given in Fig. 8 and Fig. 9, respectively.

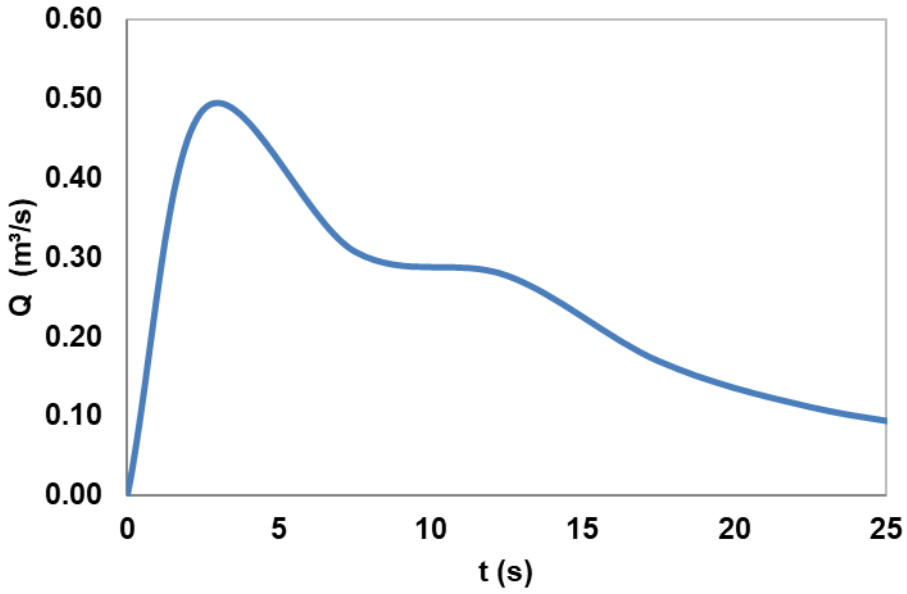


Fig. 8 – Flood hydrograph in the case of the trapezoidal breach

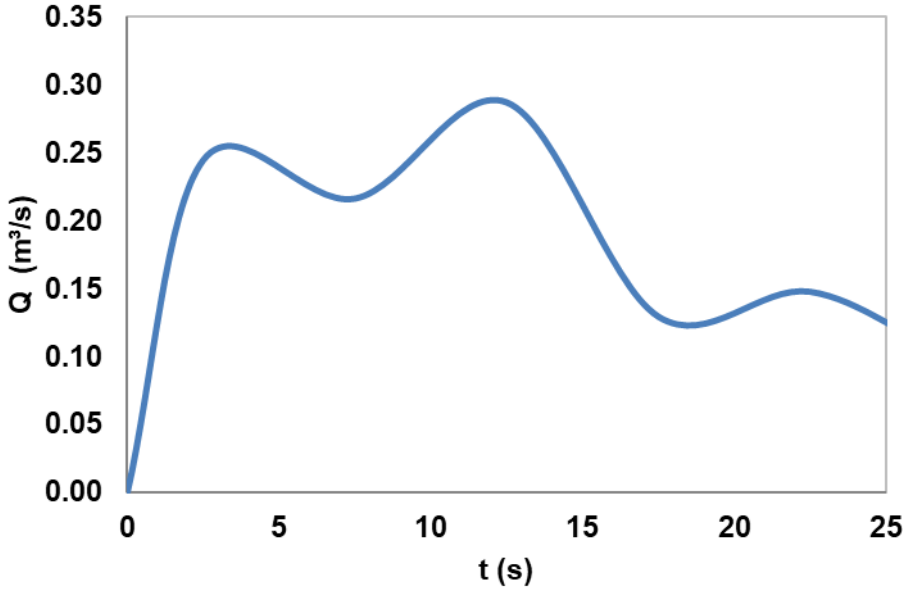


Fig. 9– Flood hydrograph in the case of the triangular breach

The peak discharge value is found to be greater in the trapezoidal breach case but the time to reach the peak values was nearly the same in both cases.

2.2. Flood Propagation

The propagation of the flood wave resulting from the partial dam breaks was recorded by means of high precision cameras. The regions reached by the flood wave at time 2 s, 8 s and 14 s for trapezoidal vs triangular breach cases are presented in Fig. 10 a.& b., Fig. 11 a.& b. and Fig. 12 a.& b. respectively.

Fig. 10 – The region where the flood wave reached at time 2 s in the a. trapezoidal and b. triangular breach cases

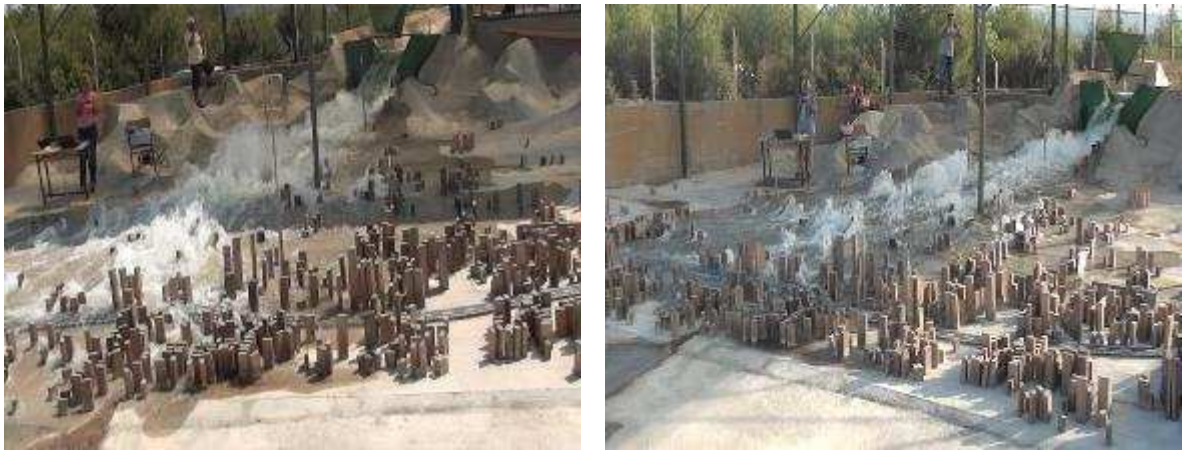


Fig. 11 – The region where the flood wave reached at time 8 s in the a. trapezoidal and b. triangular breach cases

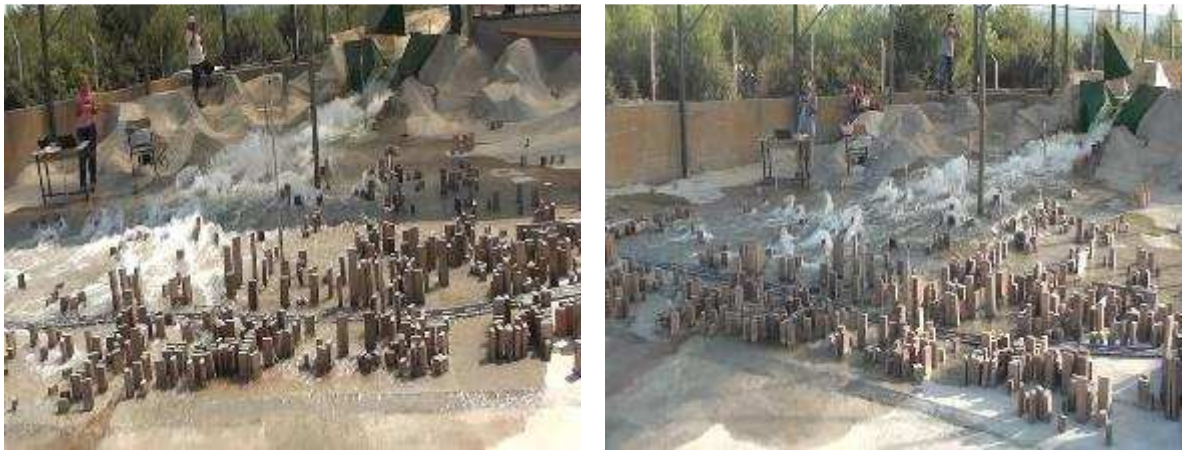


Fig. 12 – The region where the flood wave reached at time 14 s in the a. trapezoidal and b. triangular breach cases

3. NUMERICAL ANALYSIS

Numerical study was performed by means of the software FLOW-3D. The physical model geometric properties and the experimental conditions were reflected in the numerical analyses, The presented results were obtained by taking volume elements with dimensions 4 cm by 4 cm by 1 cm. The Renormalized Group (RNG) and k-ε turbulence models were applied. The ground roughness values were simulated by taking into consideration also the houses, roads and concrete surfaces on the model. Before starting the simulation, “Render” menu was activated to obtain the configuration of the physical model [8]. The screen shots corresponding to trapezoidal and triangular breach cases are given in Fig. 13 and Fig. 14, respectively.

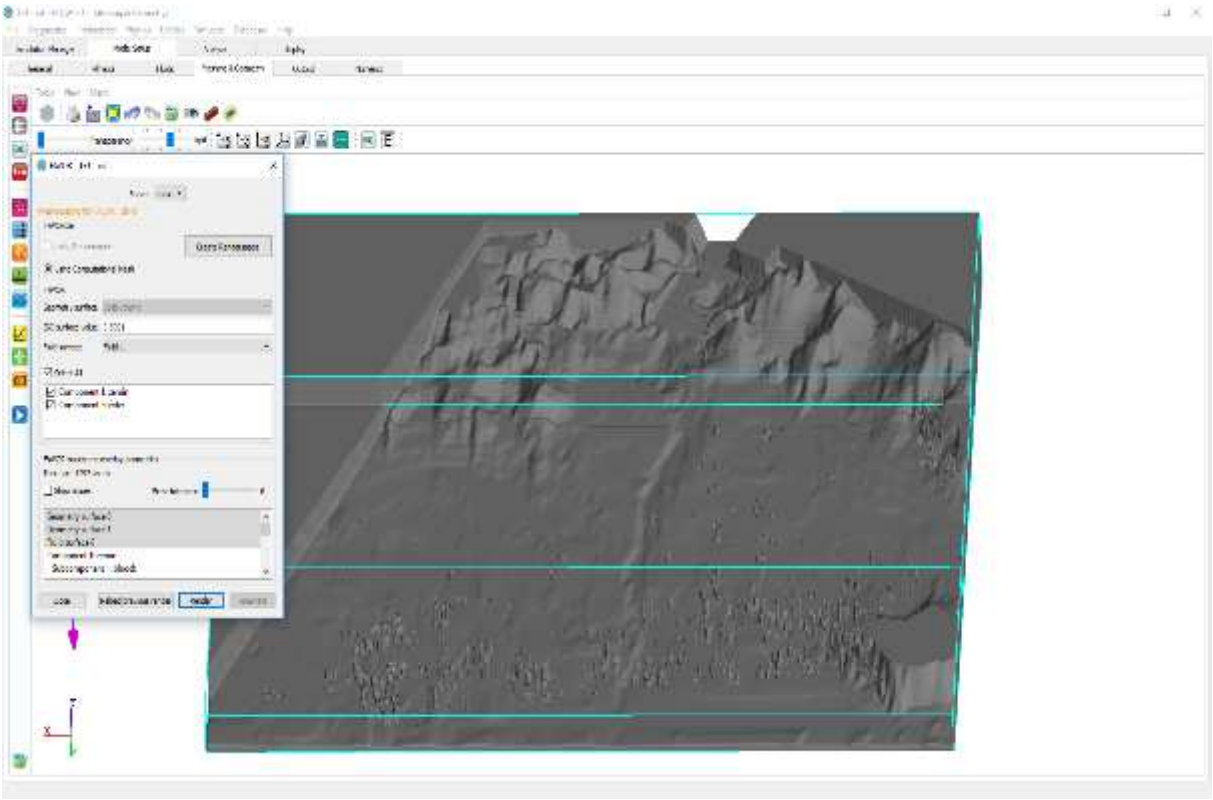


Fig. 13 – Screen shot in the case of the trapezoidal breach simulation

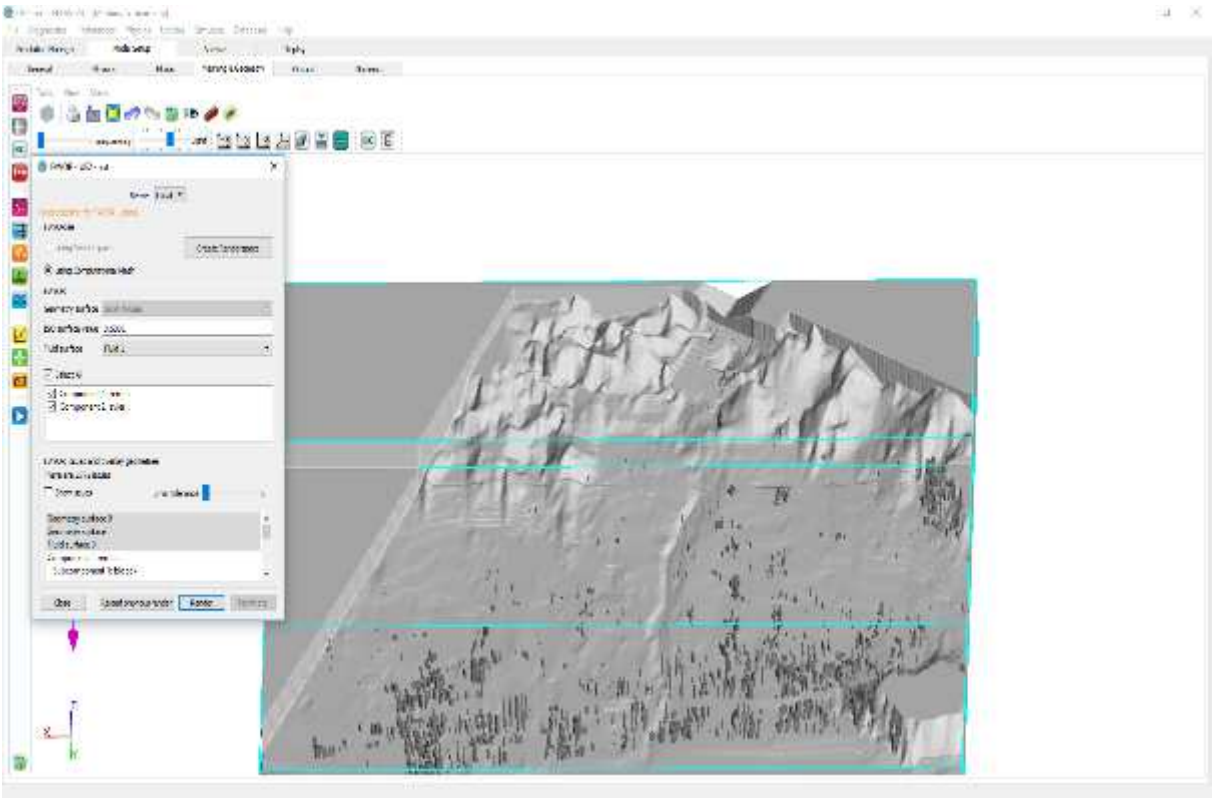


Fig. 14– Screen shot in the case of the triangular breach simulation

4. COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

4.1. Temporal Water Depths

Times at which the flood wave reached some locations are given in Table 2 for trapezoidal breach and in Table 3 for triangular breach.

Table 2 – Times (s) at which the flood wave reached some locations for trapezoidal breach

Location	Numerical (k-ε)	Numerical (RNG)	Experimental (Trapezoidal)
S2	3	3	2
S3	2	2	2
S4	9	7	12
S5	7	9	6
S6	3	5	4

Table 3 – Times (s) at which the flood wave reached some locations for triangular breach

Location	Numerical (k-ε)	Numerical (RNG)	Experimental (Triangular)
S2	3	3	4
S3	4	2	2
S4	9	7	15
S5	9	9	7
S6	5	5	4

As seen in Tables 2 and 3, the numerical results obtained from the two different turbulence models were found to be close to each other. The compatibility between the numerical and experimental results are within an acceptable range, considering the complexity of the flow and the limitations of the models. The tables compare numerical results from k-ε and RNG turbulence models with experimental data from Trapezoidal and Triangular breaches. The models show good agreement with experimental data at some locations. However, notable discrepancies exist, particularly at location S4, potentially due to experimental breach differences, measurement errors, or model limitations. The RNG model demonstrated better results than the k-ε model in some cases. Given the complexity of turbulent flows, the observed deviations are within an acceptable range.

As an illustrative example, the numerical and experimental time-dependent water depths corresponding to the location S3 are presented in Fig.15 and Fig.16 for the trapezoidal and triangular breach cases, respectively.

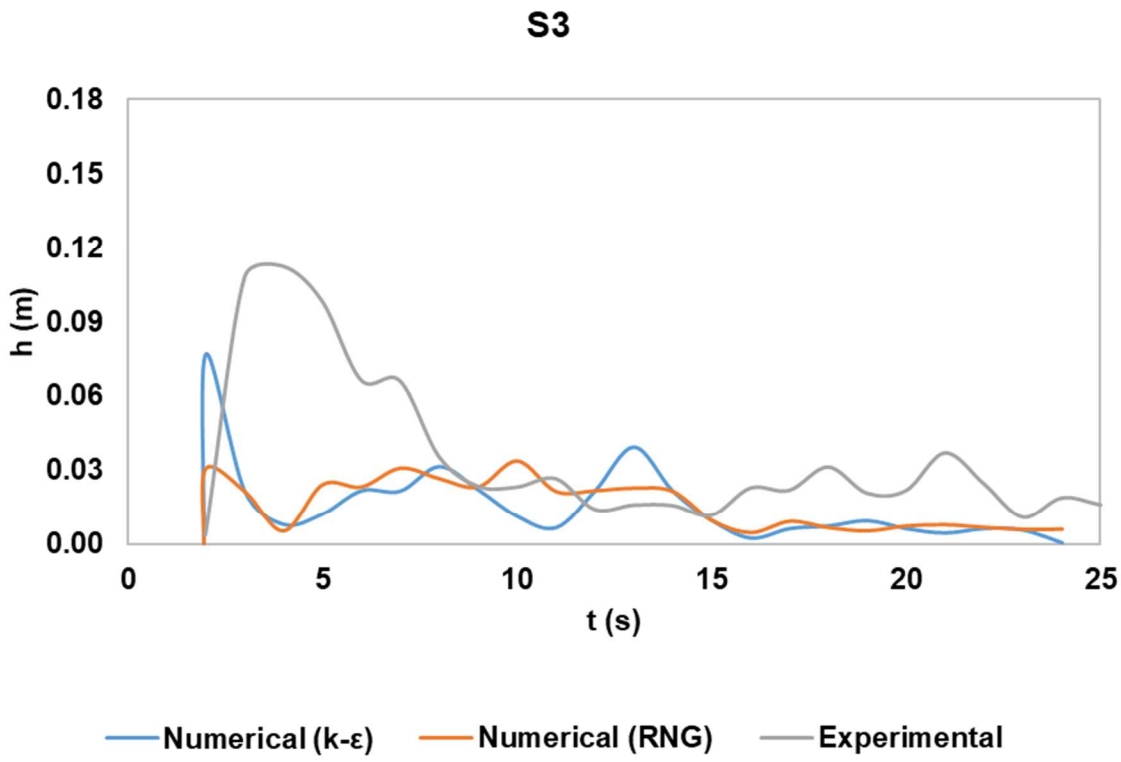


Fig. 15–Experimental and numerical water depths obtained at the location S3 in the trapezoidal breach case

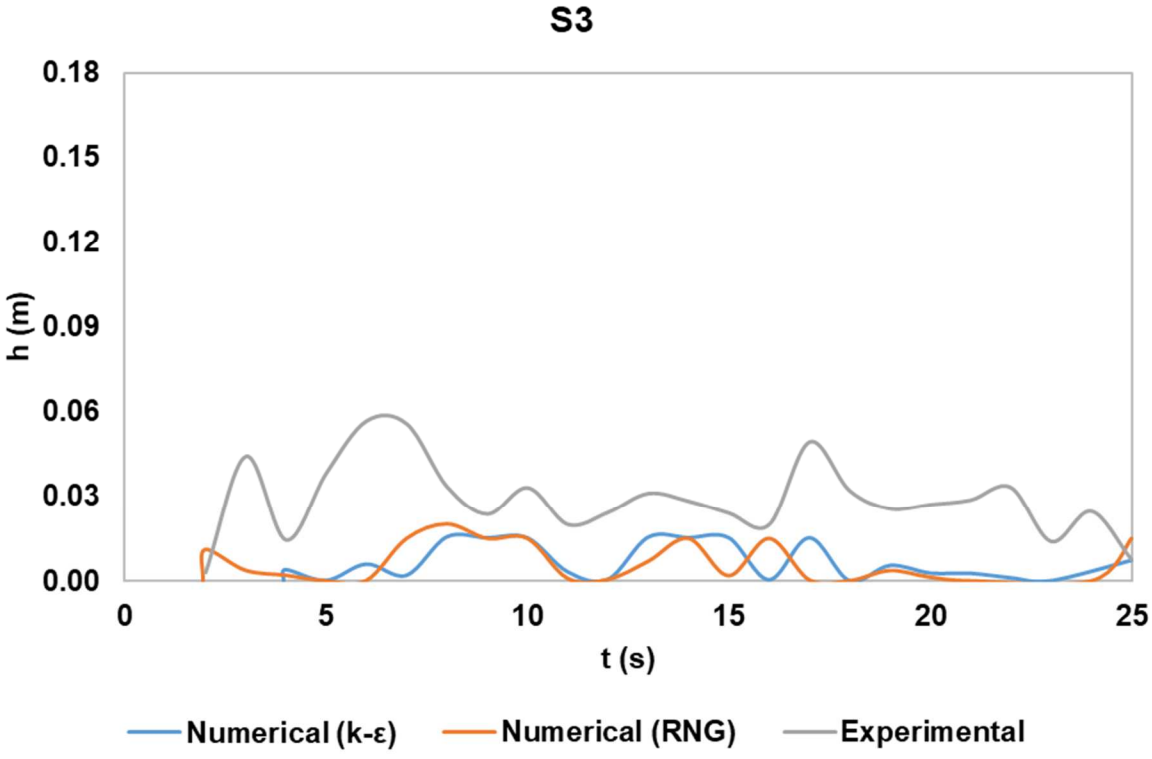


Fig. 16– Experimental and numerical water depths obtained at the location S3 in the triangular breach case

Numerical analyses were performed by using two different turbulence models. During the experiments, the maximum depths in the region close to the dam, in the trapezoidal and triangular breach cases, were recorded as 10.7 cm and 5.7 cm, respectively. In nature (prototype), these values correspond to 3.21 m and 1.71 m, respectively. In Flow 3D simulations, these maximum depth values were calculated as 7.7 cm, 3.4 cm (k-ε) and 3.1 cm, 1.5 cm (RNG).

In general, the numerical water depths were found to be smaller than those obtained from the experiments. The numerical results obtained from the two different turbulence models were found to be close to each other. This limitation is recognized, and authors believe it stems from the inherent challenges in simulating complex transient flows. However, ongoing advancements in numerical methodologies, such as adaptive mesh refinement and improved turbulence modelling, hold promise for enhancing predictive accuracy in upcoming researches.

4.2. Temporal Depth Average Velocities

The components of the numerical velocities having the same direction that of the UVP transducers U1 and U2 were determined by using common trigonometric expressions and then they were compared with the experimental findings. Considering the directional characteristics of wave propagation, UVP measurements indicated negative velocity vectors at certain spatial locationsAs an illustrative example, the computed temporal average velocity values at the location S4 (U2), are given in Fig. 17 and Fig. 18 for the trapezoidal and triangular breach cases, respectively and together with experimental ones. A notable observation was that the left coast experienced more severe flooding in the trapezoidal breach scenario compared to the triangular breach. Additionally, the numerical results for both depth-averaged velocities and water depths were quite similar between the two turbulence models used. In experimental

In the trapezoidal breach case, the measured highest velocity was about 2.7 m/s (which implies 14.8 m/s in the prototype). In the triangular breach case, the measured highest velocity was about 2.3 m/s (which implies 12.6 m/s in nature) and such velocities are capable to create serious damages in terms of life and property in certain regions of Ürkmez village. The left coast was affected more from the flood wave in the case of the trapezoidal breach compared to triangular breach.

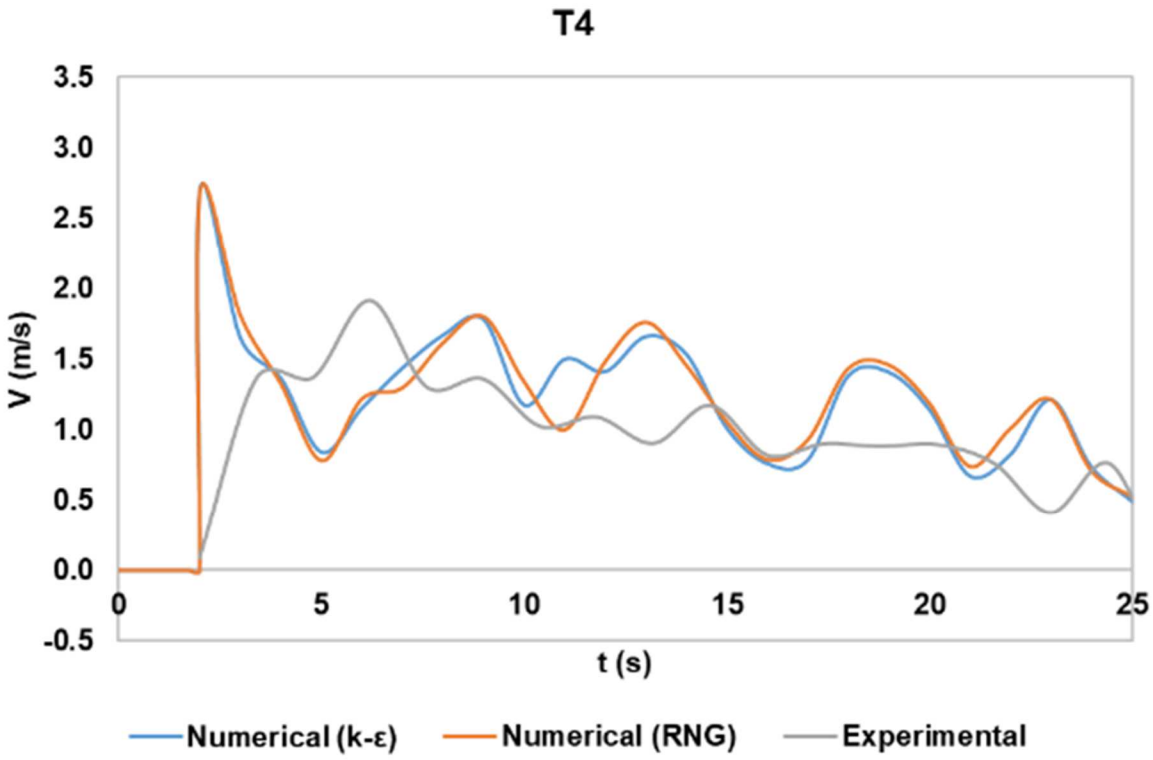


Fig. 17 – Experimental and numerical depth average velocities at the location S4 in the trapezoidal breach case

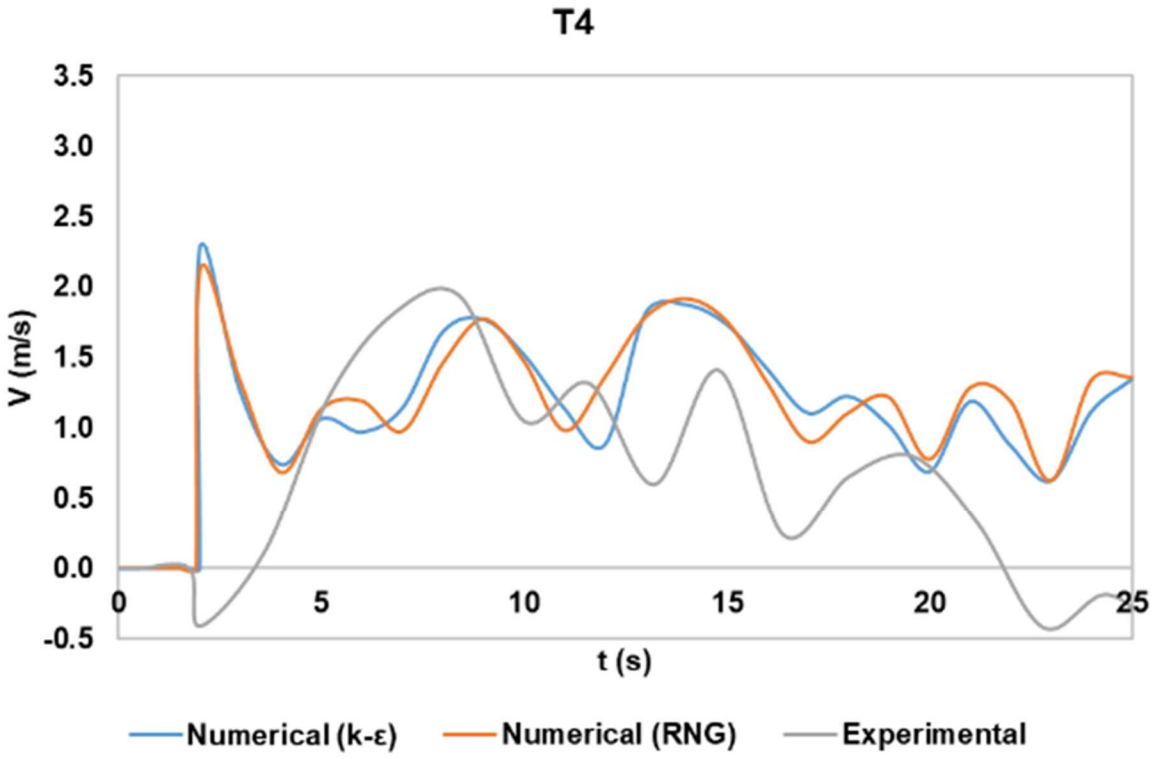


Fig. 18 – Experimental and numerical depth average velocities at the location S4 in the triangular breach case

5. CONCLUSIONS

A distorted physical model of Ürkmez Dam was designed and built to study the flood propagation due to dam break resulting from trapezoid or triangle shaped breaches. The temporal water depths and velocities were measured by means of elaborated devices. The wave propagation was recorded by using high precision cameras.

The numerical results were found to be close to each other for depth average velocities and water depths, in both turbulence model cases. Some discrepancies observed between the experimental findings and numerical results stem probably from the selected mesh sizes and roughness values. For this reason, it will be useful to perform numerical analyses by using different mesh dimensions and various roughness values, it is also intended to realize some more comprehensive comparisons between experimental and numerical results.

6. ACKNOWLEDGEMENTS

The authors thank TÜBİTAK (The Scientific and Technological Research Council of Türkiye) for the financial support provided through the project 110M240. They also thank Prof. Dr. Turhan ACATAY for his very valuable advises and technician İsa Üstündağ for his contributions during the construction of the physical model.

REFERENCES

[1] Zhang, M., XU Y., Qiao Y., Jiang H., Zhang Z. and Zhang G. (2016). Numerical simulation of flow and bed morphology in the case of dam break floods with vegetation effect. Journal of Hydrodynamics, 28(1), 23–32.

[2] Hooshyaripor, F., Tahershamsi, A. and Razi, S. (2017). Dam break flood wave under different reservoir’s capacities and lengths. Sadhana – Indian Academy of Science, 42, 1557–1569.

[3] Kamra, M., Mohd, N., Liu, C., Sueyoshi, M. and Hu, C. (2018). Numerical and experimental investigation of three-dimensionality in the dam-break flow against a vertical wall. Journal of Hydrodynamics, 30, 682–693.

[4] Güney, M.Ş., Tayfur, G., Arkış, T., Bombar, G., (2015). Experimental and numerical investigation of flood propagation due to trapezoidal breach in the distorted physical model of Ürkmez dam, Dam Engineering, vol 25, issue 4, pp. 171- 187.

[5] Yalın, M.S., (1971). Theory of hydraulic models. The Macmillan Press Ltd, London.

[6] www.eijkelkamp.com (2011). e+ WATER L® from Eijkelkamp Agrisearch Equipment.

[7] Güney, M.Ş., Tayfur, G., Bombar, G., Elçi, S., (2014). Distorted physical model to study sudden partial dam break flows in an urban area. ASCE, Journal of Hydraulic Engineering, DOI: 10.1061/HY.1943-7900.0000926.

[8] Flow-3D, Help, Ver. 11.2, Flow Science Inc.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

A METHODOLOGY TO DETERMINETHE FLOW CURVES FOR A RIVER BARRAGE IN POTENTIALLY SUBMERGED CONDITIONS

David Zuccalà[1]
ISMES S.p.A. CESI Group

Daria Gatti[2]
ISMES S.p.A. CESI Group

Isabella Scuppa[3]
Enel Green Power S.p.A.

Carlo Cirillo[4]
Enel Green Power S.p.A.

ABSTRACT

In order to provide a useful and simple tool for predicting flow rates in transit during floods through river barrages, a methodology has been developed identifying the flow curves (under free surface conditions or through sluice gates, with free or submerged outflow), with various degrees of opening of the gates. The simplest formulations cannot always be applied because the highest discharge values could occur in drowned conditions. In formulas describing submerged flow, the discharge is a function of both upstream and –usually not known– downstream water levels. The approach described in the paper aims to overcome this difficulty by estimating the downstream discharge-water level relationship by means of a numerical model applied to a riverbed stretch downstream the barrage.

The methodology has been implemented in a software tool called «EFFLUSSI», configurated for some EGP river barrages.

Keywords: outflow diagrams, dam outlets, river barrages, software tool, submerged flow.

[1]david.zuccala@ismes.it , Italy
[2]daria.gatti@ismes.it, Italy.
[3]Isabella.scuppa@enel.com, Italy
[4]carlo.cirillo@enel.com , Italy
Marco Belotti, marco.belotti@ismes.it, ISMES S.p.A. CESI Group, Italy

1. INTRODUCTION

River barrages produce an increase of the upstream water level contained in the riverbanks, with the main purpose to stabilise the reservoir and to facilitate water diversion. River barrages usually retain a small-sized reservoir.

Large river barrages (in Italy those with a height greater than 15 metres and/or a reservoir volume greater than 10^6 m³) belong to the class of so-called *large dams*. Many barrages, which are excluded from the above-mentioned category, are, on the other hand, included in the group of the *minor* or *small dams*.

For instance, barrages submitted to the Italian Presidential Decree Law [10] belong to this class (i.e. those whose height is between 10 and 15 metres and/or whose reservoir volume is between 10^5 and 10^6 m³). In general Italian large dams are submitted to Ministerial Circular [11], whereas minor dams are submitted to regional laws, for instance [12]. Under operating conditions, but especially under flooding conditions, it becomes relevant to know the discharge flowing through the barrage for the hydraulic safety of the upstream and especially downstream territories.

To obtain this discharge from the reservoir level measured by a hydrometer, the barrage outlets must be characterised by their outflow laws for various degrees of opening of the gates, varying the transiting discharge too. This is in common practice achieved by applying the classical *laws of foronomy*.

The limitations of these formulations become non-negligible during floodings characterised by high water levels upstream and downstream the barrage, since water may flow in a submerged condition.

In these situations, if it is assumed downstream the barrage the formation of a short stretch of rapid current and just up ahead a hydraulic jump, in a slightly-sloped riverbed (the most frequent case) there is the risk of discharge overestimation. In fact, the hydraulic jump in this case appears against the gate and completely submerged, until it disappears completely as the discharge increases.

In submerged outflow formulations, the transit discharge depends on both the water levels upstream and downstream the gates (and, of course, gate opening). The downstream water level is often uncertain or unknown because it is hard to measure, due to the oscillatory phenomena in time and space associated to floodings.

The approach suggested below has been conceived to solve this problem. It exploits analytical expressions drawn from literature which take into account the case of submerged flow. They are associated with a 1D or 2D numerical model applied to a suitable river stretch situated

immediately downstream of the river barrage. This makes it possible to create a *Q-h* relation (discharge-water level diagram) allowing to compute the flowing discharge despite the dependence on the downstream water level.

This methodology has been incorporated into the *EFFLUSSI* software developed by ISMES (a CESI Company), which, once arranged for a given barrage, can quickly calculate, without requiring complex inputs, in the various cases of free flow and under sluice gate flow and non-drowned and drowned conditions, the discharge passing through the barrage for various degrees of gate opening. Examples are provided, related to two Enel Green Power's barrages (located in Abruzzo region, in Central Italy).

2. DISCHARGE LAWS IMPLEMENTED FOR WEIR OUTFLOW

2.1. Main sections

For free outflow over weirs, the classic formula for the broad crested weir is used, which uniquely links discharge to hydraulic head:

$$Q = \mu_{stram}\, b\, H^{3/2} \sqrt{2g} \tag{1}$$

(symbols defined in Table 2).

In case the downstream water level increases up to submerge the outflowing stream, a jump appears immediately downstream the weir. This ensures the presence of a short stretch of rapid current just downstream the section where the critical state k is established, and precisely this rapid current guarantees that the downstream level, if not excessively high, does not produce any influence on the current. The discharge is therefore a one-to-one function of the hydraulic head.

Given a discharge through the weir, there is a downstream level, called the *borderline submergence*, above which the effects of submerged condition begin to appear and break the uniqueness of the relationship between discharge and head. Under such conditions, the jump tends to decrease as the downstream level rises, until it eventually disappears.

According to *Villemonte's formula* (2), used in the present methodology, the discharge begins to decrease when the downstream water level exceeds the sill Z_s:

$$Q = \mu_{stram}\, b\, H^{3/2}\ \sqrt{2g} \left[1 - \left(\frac{h}{H}\right)^{3/2}\right]^{0.385} \tag{2}$$

In the case of drowned weir, the discharge decreases, compared to the case of passing through the critical state, by a factor K_s dependent on the ratio of downstream and upstream water depth. The discharge reduction is represented by the so-called submergence:

$$K_s = \left[1 - \left(\frac{h}{H}\right)^{3/2}\right]^{0.385} \quad (3)$$

The exponent 3/2 applies to outflow through rectangular shape outlets. The typical dependence of the discharge reduction coefficient from the downstream water level is shown in Fig. 1.

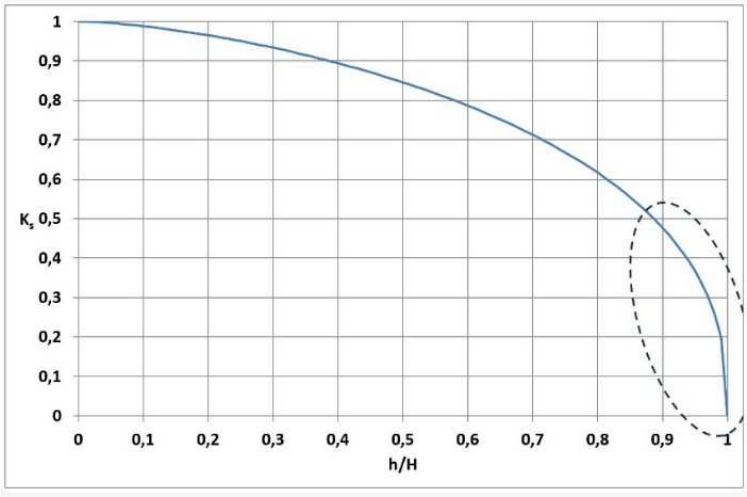


Fig. 1 – Discharge reduction coefficient for a drowned weir

It is evident from this graph that as the downstream water level approaches the upstream one, i.e. under strongly drowned conditions with ratios of downstream to upstream water depth greater than 0.9 (dashed area in the plot), small variations in the h/H ratio give rise to large variations in discharge, amplifying the uncertainties in the levels and making the application of the formula unreliable.

3. DISCHARGE LAWS IMPLEMENTED FOR FLOW UNDER A SLUICE GATE

A sluice gate causes a contraction of the fluid section immediately after the gate itself.

In a *weak slope riverbed*, under hydraulically *free flow* conditions, a sluice gate perturbs the upstream current by generating a backwater profile. It perturbs also a small stretch of the downstream current by generating a rapid current.

The extension of the backwater profile is finite and depends only on the riverbed slope: the lower the slope, the further upstream the effect extends. Downstream of the sluice gate there is a hydraulic jump (located where the total upstream hydrodynamic pressure of the current coincides with the downstream one), i.e. a discontinuity in the streamline profile.

For the free flow under sluice gates, the well-known general formula for large rectangular openings can be applied [2, p. 436]:

$$Q = \frac{2}{3}\mu_{batt}\, b\, \sqrt{2g}\left[H^{3/2} - (H-a)^{3/2}\right] \quad (4)$$

that joins without discontinuities to (1) concerning the non-submerged weir.

μ_{batt} (given by the product of a contraction coefficient C_c = 0.61 and a velocity coefficient C_V = 0.98) represents the outflow coefficient and is assumed to be 0.60.

Other discharge laws such as $Q = \mu_{batt}\, ab\sqrt{2g(H - C_c a)}$ [2, p. 118] specific for bottom outlets predict discharge with minor differences from equation (4).

Submerged flow under sluice gate can be formally represented by the classical expression:

$$Q = C_d ab\sqrt{2gH} \tag{5}$$

where, however, the discharge coefficient, instead of being constant, is:

$$\mathrm{C_d} = 0.611\left(\frac{H-a}{H+15a}\right)^{0.072}(H-h)^{0.7}\left\{0.32\left[0.81h\left(\frac{h}{a}\right)^{0.72} - H\right]^{0.7} + (H-h)^{0.7}\right\}^{-1} \tag{6a}$$

With some simple steps, this expression can be rewritten to better emphasise the dependence from upstream and downstream water depths in dimensionless form:

$$C_d = \frac{0.611\left(\frac{\frac{H}{a}-1}{\frac{H}{a}+15}\right)^{0.072}}{1+0.32\left[\frac{0.81\left(\frac{h}{a}\right)^{1.72} - \frac{H}{a}}{\frac{H}{a}-\frac{h}{a}}\right]^{0.7}} \tag{6b}$$

Submerged flow occurs if both of the following conditions are satisfied:

- $h > a$ (outlet opening submerged by downstream water level, as well as by the upstream one)
- $\frac{H}{a} < 0.81\left(\frac{h}{a}\right)^{1.72}$ i.e. $H < 0.81\, h\left(\frac{h}{a}\right)^{0.72}$ (Fig. 2) (7)

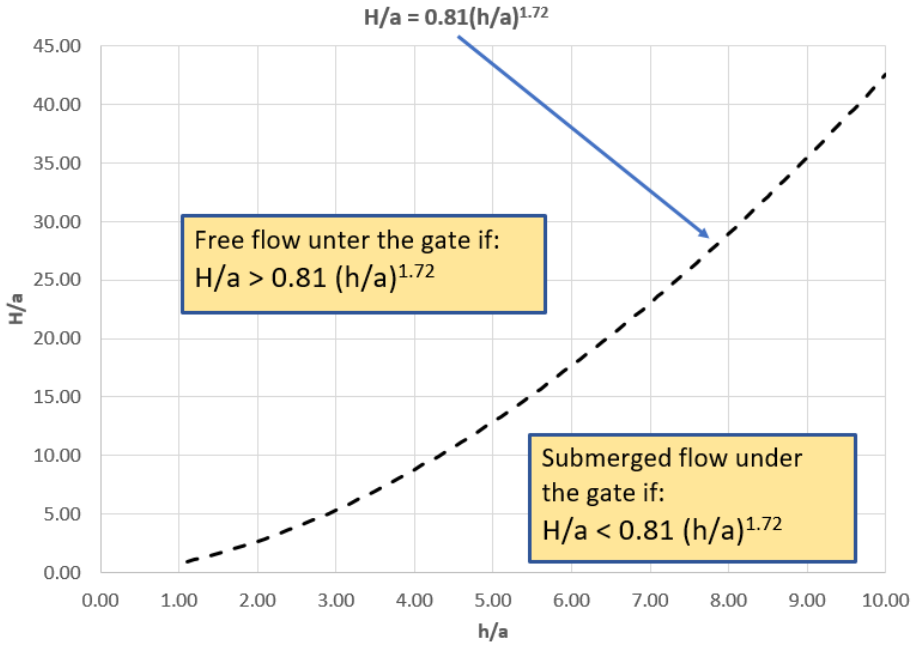


Fig. 2 – The dashed curve divides submerged and non-submerged flow domains

Fig. 3 outlines the dependence of C_d on upstream level H and downstream level h, both in the case of free-flow and of various degrees of submergence, represented by the family of curves associated with the dimensionless parameter h/a increasing with the importance of drowning.

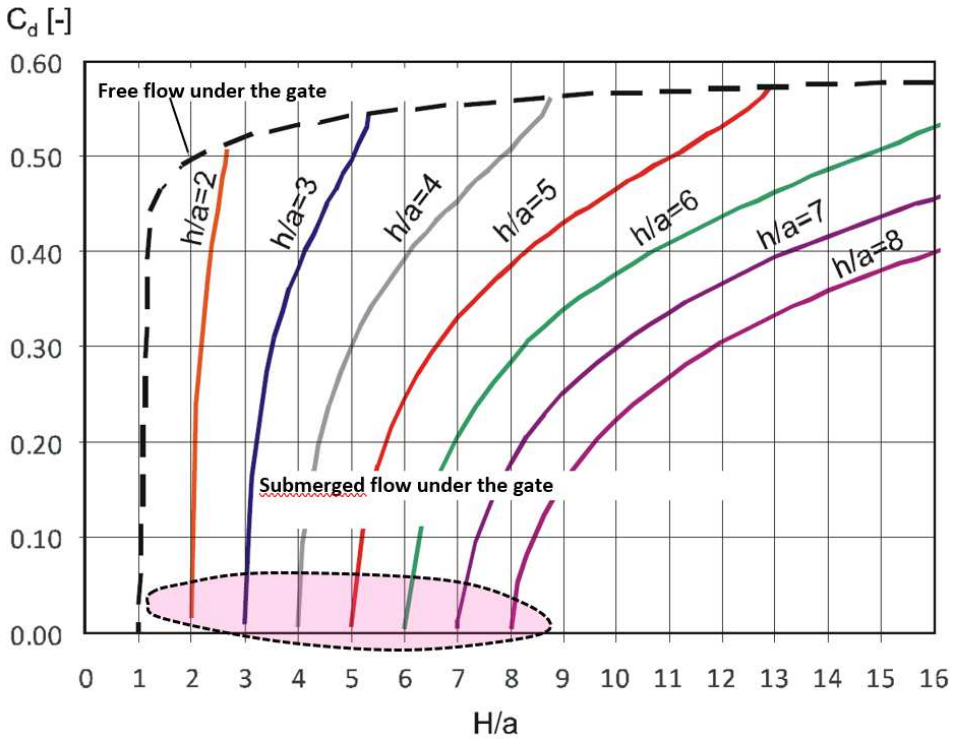


Fig. 3 – Flow under a sluice gate: C_d coefficient for various submerged conditions [1, 4]

As already observed for the case of the weir (Fig. 1), Fig. 3 shows that also in the case of outflow under a sluice gate, as the downstream level approaches the upstream one, i.e. under strongly submerged conditions (regions highlighted in pink in the graphs) characterised by ratios between downstream and upstream water depth greater than 0.9, small variations in levels correspond to large variations in discharge (through the variation of K_s in the case of weir and of C_d in the case of flow under sluice-gate).

Therefore, situations with small upstream-downstream water level differences, i.e. $H \approx h$, which corresponding to the lower segment of the curves where the slopes are very steep, are affected by strong indeterminacy.

From a strictly mathematical point of view, the closer H and h (upstream and downstream level respectively) are, the more C_d (and K_s) and consequently Q decrease until they cancel out. We note that **under such extreme conditions, discharge evaluations are not reliable**.

Above formulas are grouped below in Table 1 together with the criteria for applying each one to the cases under consideration.

Table 1 – Discharge formulas in the different cases considered

Free outflow over a weir if $H < a$ and $Z_{wv} \leq Z_s$

$$Q = \mu_{stram}\, b\, H^{3/2} \sqrt{2g} \qquad \mu_{stram} = 0.385 \qquad (1)$$

Drowned (submerged) weir if $H < a$ and $Z_{wv} > Z_s$

$$Q = \mu_{stram}\, b\, H^{3/2} \sqrt{2g} \left[1 - \left(\frac{h}{H}\right)^{3/2}\right]^{0.385} \qquad (2)$$

Free flow under a sluice gate if $H \geq a > h$
or if or if $H \geq a$ $h \geq a$ and $\frac{H}{a} \geq 0.81 \left(\frac{h}{a}\right)^{1.72}$

$$Q = \mu_{stram}\, b\, H^{3/2} \sqrt{2g} \left[1 - \left(\frac{h}{H}\right)^{3/2}\right]^{0.385} \qquad (4)$$

Submerged flow under a sluice gate if $H > h \geq a$ and $\frac{H}{a} < 0.81 \left(\frac{h}{a}\right)^{1.72}$

$$Q = C_d ab\sqrt{2gH} \qquad (5)$$

$$C_d = \frac{0.611\left(\frac{\frac{H}{a}-1}{\frac{H}{a}+15}\right)^{0.072}}{1+0.32\left[\frac{0.81\left(\frac{h}{a}\right)^{1.72}-\frac{H}{a}}{\frac{H}{a}-\frac{h}{a}}\right]^{0.7}} \qquad (6b)$$

Table 2 – Symbology

g	[m/s^2]	Gravitational acceleration
Q_{tot}	[m^3/s]	Total discharge through the outlets
Q_i	[m^3/s]	Discharge through i-th outlet
n	[ ]	No of outlets
b	[m]	Outlet width
b_{lat}	[m]	Lateral spillway width
a	[m]	Gate opening
a_{max}	[m]	Maximum gate opening
Z_s	[m a.s.l.]	Sill of the outlets
$Z_{s\,lat}$	[m a.s.l.]	Sill of lateral spillway
Z_{wm}	[m a.s.l.]	Reservoir water level
Z_{wv}	[m a.s.l.]	Downstream water level
H	[m]	Upstream head ≈ Upstream water depth (related to sill) = $Z_{wm} - Z_s$
h	[m]	Downstream head ≈ Downstream water depth (related to sill) = = $Z_{wv} - Z_s$
m_{stram}	[ ]	Weir free flow coefficient = 0.385
m_{batt}	[ ]	Flow under a sluice gate coefficient = 0.60
C_d	[ ]	Discharge reduction coefficient for submerged flow under a gate
K_s	[ ]	Submergence factor
k	[m]	Critical water depth
t	[m]	Sill height over riverbed

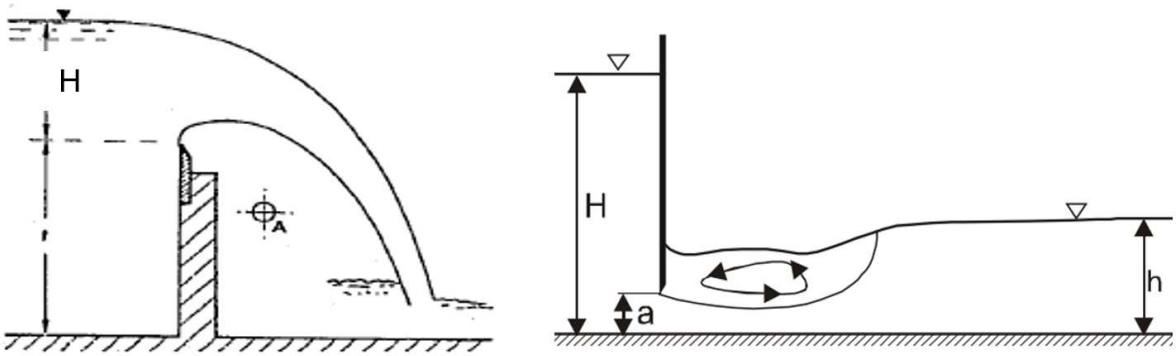


Fig.4 – Free flow over a weir (left [2]) and flow under a sluice gate (right [3])

4. CHARACTERISATION OF THE OUTLETS OF A RIVER BARRAGE

4.1. Discharge laws

The expressions in Table 1 can be rewritten in the general form:

$$H = H(Q_i, a_i, h) \tag{8}$$

These formulas are used by neglecting the arrival upstream velocity compared to the hydrostatic head in the reservoir.

In the following, the outflow laws are applied to both the case of plane and radial gates, although more specific, even if more complex, expressions exist for the latter.

Regarding the hypotheses taken, it is assumed that the discharge transits in a controlled manner entirely through the outlets, without bypassing or even overtopping the barrage as could happen in extreme situations, i.e. that the total discharge Q_{tot} corresponds to the sum of the discharge Q_i passing through the various gates.

The downstream water height h, impactful in submerged conditions, depends on the total discharge Q_{tot} along the river (assumed to be equal to the sum of the discharges through all the sluices). Therefore, the outflow discharge law of an individual sluice cannot disregard the total discharge passing through the barrage.

Given the opening ai of the i-th sluice gate, (8) can be interpreted as a family of $H_h(Q_i)$ curves parameterised by the downstream level h. This level is generally not measured near the river barrage, and in any case its measurement is difficult even in laboratory due to the oscillations and phenomena such as undulating jumps [3].

In order to overcome this problem that would leave the outflow laws undetermined, i.e. to suppress one degree of freedom from eq. (8), it has been decided to estimate the relationship between Q_{tot} and h by applying a numerical model, one or two-dimensional, with various constant discharge values, to a river stretch where the barrage be the upstream boundary.

The dependence on the downstream water level can thus be overcome by solving the following system:

$$\begin{cases} H = H(Q_{tot}, h, a) \\ h = h(Q_{tot}) \end{cases} \tag{9}$$

where the special case of all gates having the same opening has been considered:

$a_i = a, i = 1, 2, \ldots n$.

The total outflow curves of the river barrage $H = H\,(Q_{tot}, a)$ can then be computed. They are also valid under potential submerged conditions, for various degrees of sluice gate opening. The solutions can be found numerically, as it is generally not possible to explicit analytically the variables of interest.

Given the opening of the gates, as the discharge increases, the reservoir level follows a typical trend exemplified in Fig. 5: the outflow law is composed of parts of curves corresponding to different regimes. For each discharge value the curve that provides the highest reservoir level is selected.

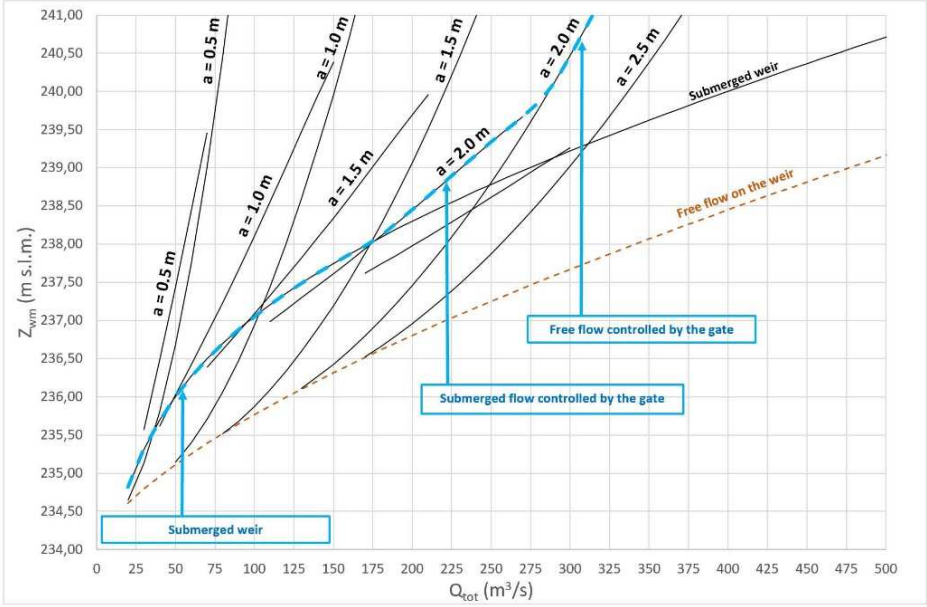


Fig. 5 – Example of outflow law obtained by combining stretches of curves associated with different regimes

4.2. Estimation of the total discharge through the barrage, given the opening of each gate and the upstream water level

Given the opening ai of each gate and the measured upstream water depth H, Q_{tot} can be founded by solving the following system:

$$\begin{cases} Q_i = Q_i(H, h, a_i) \\ h = h(Q_{tot}) \\ Q_{tot} = \sum_i Q_i \end{cases} \quad (10)$$

The first equation depends on the type of hydraulic regime through each sluice and can be one of the four formulas shown in Table 1.

Also this system must be solved numerically by a recursive method, proceeding with the following steps:

1. For each gate, by comparing H and a_i, the algorithm determines whether the gate is completely over the free surface ($H < a_i$) or it can control the flow ($H > a_i$).
2. A first-attempt value of Q_{tot}, (taken for example from a list of increasing discharge values) is chosen.
3. h is calculated as a function of Q_{tot} .

4. For each gate the algorithm determines whether the outflow is non-submerged or submerged, applying the criteria set out in Table 1.
5. In the case of a flow controlled by a gate, if Villemonte's formula (2) [4] returns a higher flood level, the latter is considered valid (this is the case of a sluice gate which is slightly immersed in the current and does not limit the outflow due to the prevalence of drowning already present in free water surface conditions).
6. For each gate the appropriate formula for the identified flow regime is entered into (10) and the expected discharge for each sluice gate Q_i is calculated.
7. The Q_i 's are summed and the algorithm checks if their sum coincides with the initially assumed total flow Q_{tot} .
8. If yes, the system is solved, otherwise a new attempt is made, modifying Q_{tot} and starting again from point (3). The attempts follow one after the other until the solution of the system (10) is found, thus determining Q_{tot}.

5. ESTIMATION OF DISCHARGE THROUGH RIVER BARRAGES

A software called *EFFLUSSI* has been developed, in Windows environment, using the libraries and resources of the ".NET Framework 4.5.X" distributed by Microsoft. This computation tool can be used on PCs running Windows 10 or higher. It is distributed as an executable file and does not require installation (ISMES - CESI Company, 2023).

The *EFFLUSSI* code solves the system (10), determining Q_{tot} by trial and error, given the reservoir level and the opening of each gate. To prioritise simplicity of use, the geometric parameters are preloaded in the software, as well as the downstream rating curve for the barrages of interest. Once the user has chosen the barrage, he must enter the measured reservoir level and the opening of each gate. After launching the calculation, the result appears, as shown in Fig. 6.

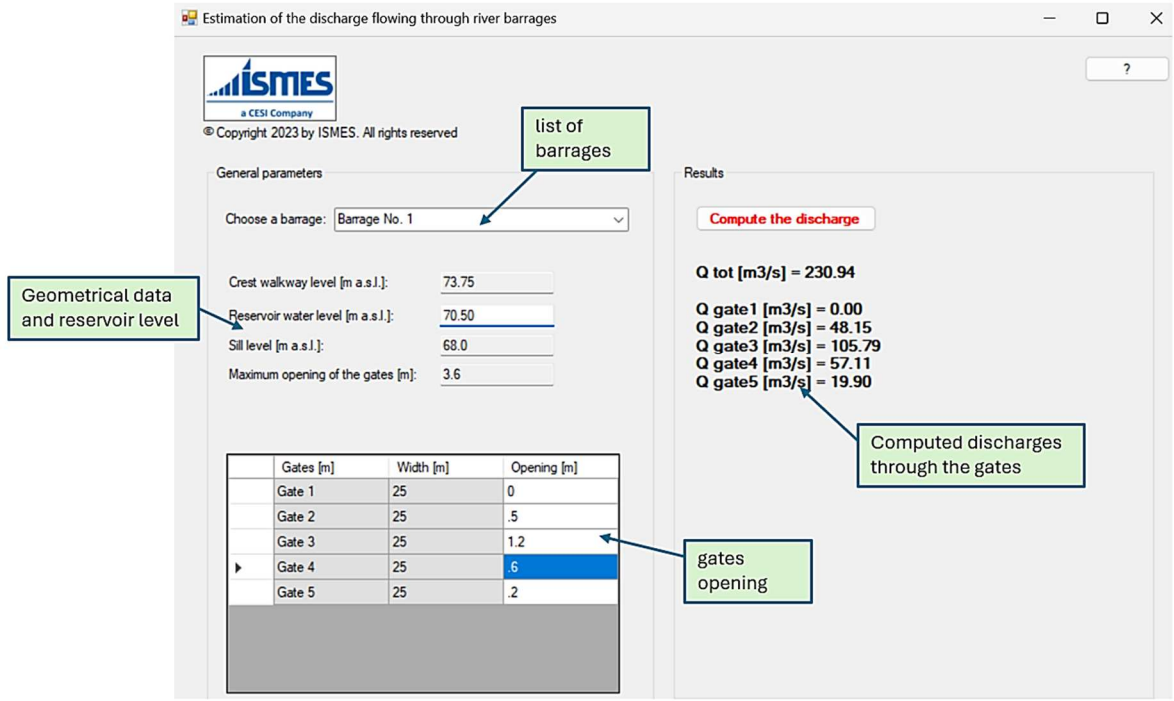


Fig. 6– *EFFLUSSI* software tool: calculation example

The outflow laws determined for two barrages as examples are shown below.

5.1. Application to river barrage No. 1

This is a concrete river barrage consisting of 5 overflow sills regulated by 25 m large radial gates. The crest walkway has an elevation of 75.00 m a.s.l.

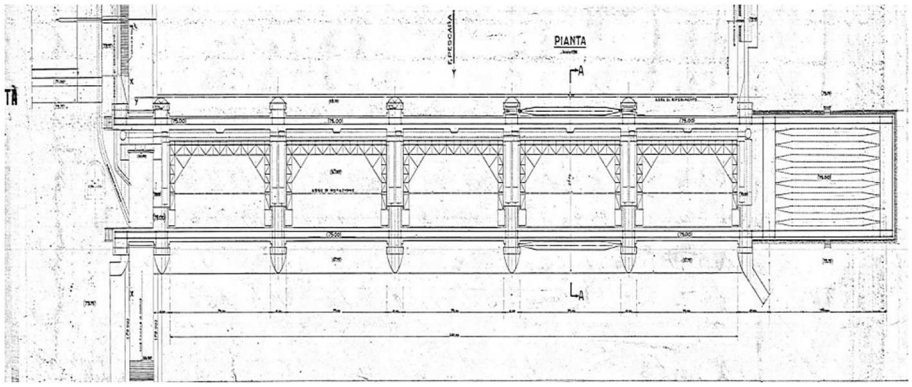


Fig. 7 – Layout of barrage No. 1

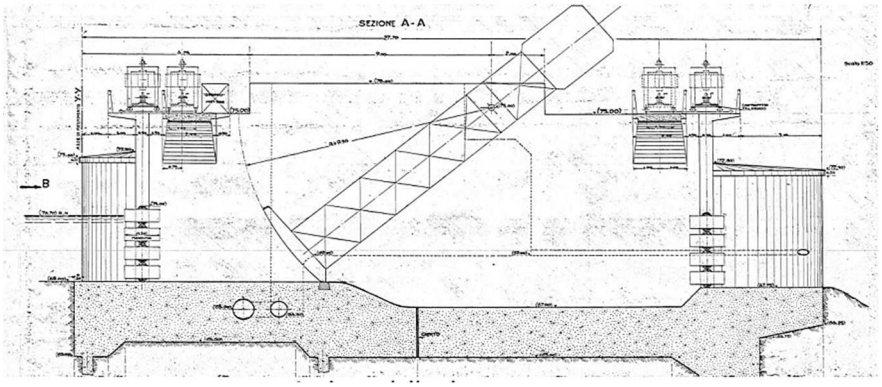


Fig. 8 – Longitudinal cross-section of barrage No. 1

Fig. 9 – Side view of barrage No. 1

The characteristic parameters for the purposes of the subsequent calculations are given in the following table.

Table 3 – Geometric-hydraulic parameters of barrage No. 1

n	$= 5$
b	$= 25\ m$
Z_s	$= 68.00\ m\ a.s.l.$
a_{max}	$= 3.60\ m$
μ_{stram}	$= 0.385$
μ_{batt}	$= 0.58$

The downstream flow conditions $h=h(Q_{tot})$ have been evaluated by means of a two-dimensional numerical model applied to an appropriate reach close to the barrage. HEC-RAS 6.2 code has been applied, in this case taking the digital terrain model from LIDAR data provided by the National Italian Geoportal (Ministry of the Environment).

The two-dimensional hydrodynamic model, starting from an appropriate representation of the terrain morphology of the territory in two dimensions, makes it possible to simulate the flooding in the two plane directions. Hydraulic quantities are averaged out the vertical axis, providing water level and velocity vector at each point of the calculation domain.

Fig. 10 shows the first reach of the domain with the computational mesh and the water depth distribution of the water depth for the 200-years return period. The mesh has a higher resolution along the centerline riverbed. Outside the banks, however, it adequately reflects the terrain morphology.

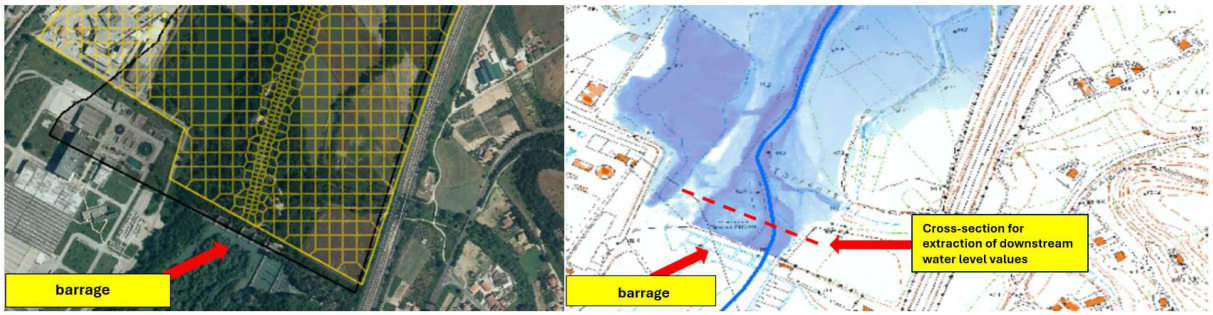


Fig. 10 – Initial part of the modelled river stretch downstream barrage No. 1

The rating curve along the dotted line, obtained by means of simulations associated with different constant values of discharge, is shown in Fig. 11.

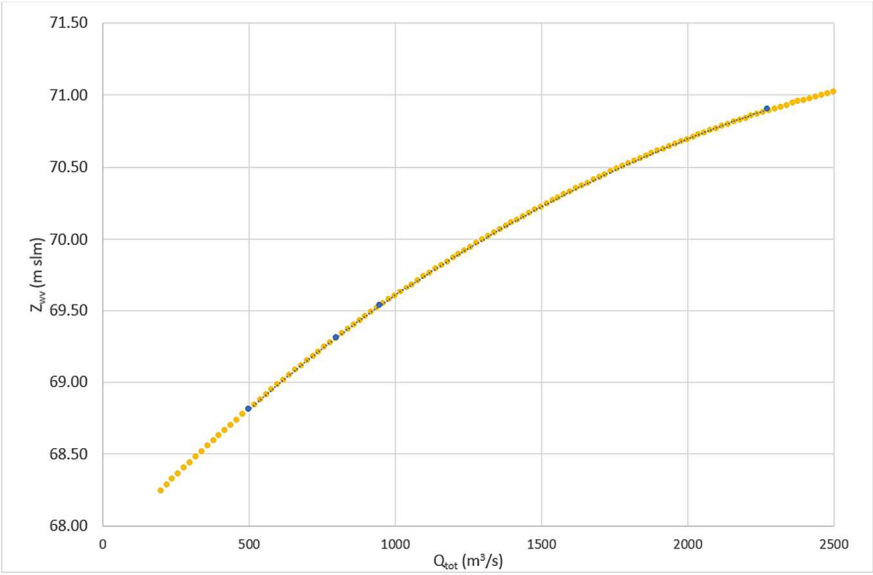


Fig. 11 – Downstream rating curve: average water level vs. total discharge (barrage No. 1)

Fig. 12 shows the discharge laws through the barrage for different degrees of opening of the gates, assuming that such opening is the same for each gate. The curves were obtained by combining the laws corresponding to the different possible regimes, outlined in Fig. 5.

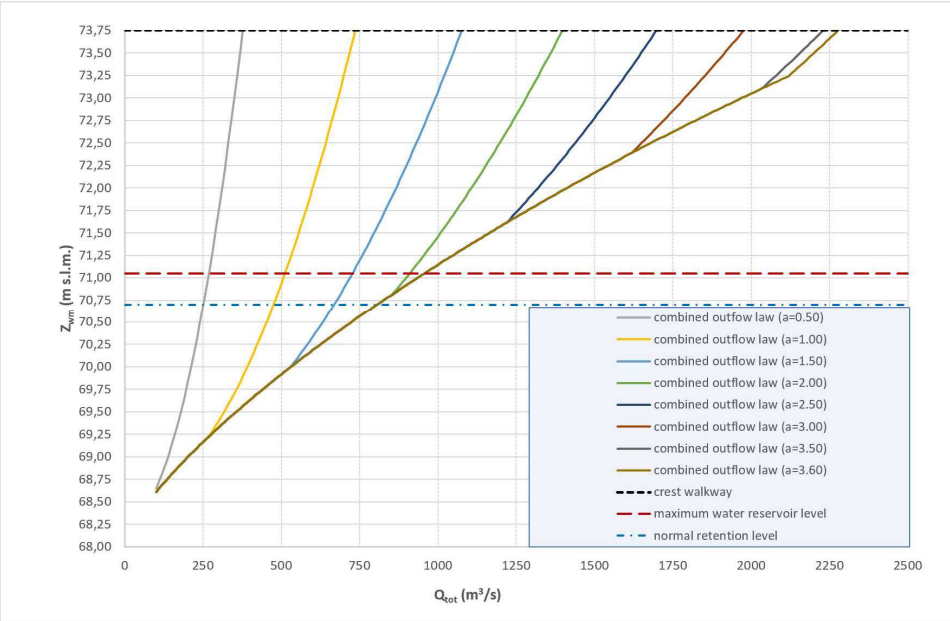


Fig. 12 – Discharge laws through barrage No. 1 for different gate openings (same opening for each gate)

If Q_{tot} is greater than 80 m^3/s, the free outflow takes place under submerged conditions (according with (2)), as the downstream level exceeds the sill level (68.00 m a.s.l. - see Fig. 11).

The gate-controlled flow, in contrast, is generally not drowned.

5.2. Application to river barrage No. 2

This is a concrete river barrage consisting of 3 overflow sills regulated with 10 m large sluice gates and 4 weirs near the left bank with a total extension, without considering the intermediate piers, of 59 m. The crest walkway is located at an elevation of 539.50 m a.s.l.

Table 4 – Geometric-hydraulic parameters of barrage No. 2

n	$= 3$
b	$= 10\ m$
Z_s	$= 532.50\ m\ a.s.l.$
a_{max}	$= 3.00\ m$
b_{lat}	$= 59.00\ m$
μ_{stram}	$= 0.385$
μ_{batt}	$= 0.58$

Fig. 13 – River barrage No. 2: downstream view

The downstream outflow conditions, represented by the rating curve $h=h(Q_{tot})$ (Fig. 15), have been evaluated using a two-dimensional numerical model (Fig. 14), similarly to the previous case.

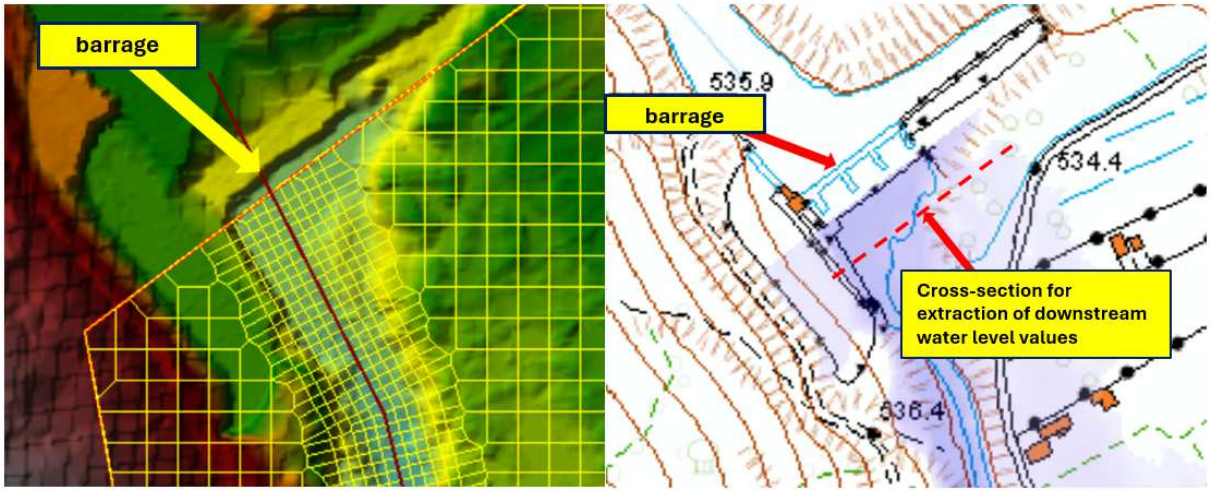


Fig. 14 – Initial part of the modelled river stretch downstream barrage No. 2

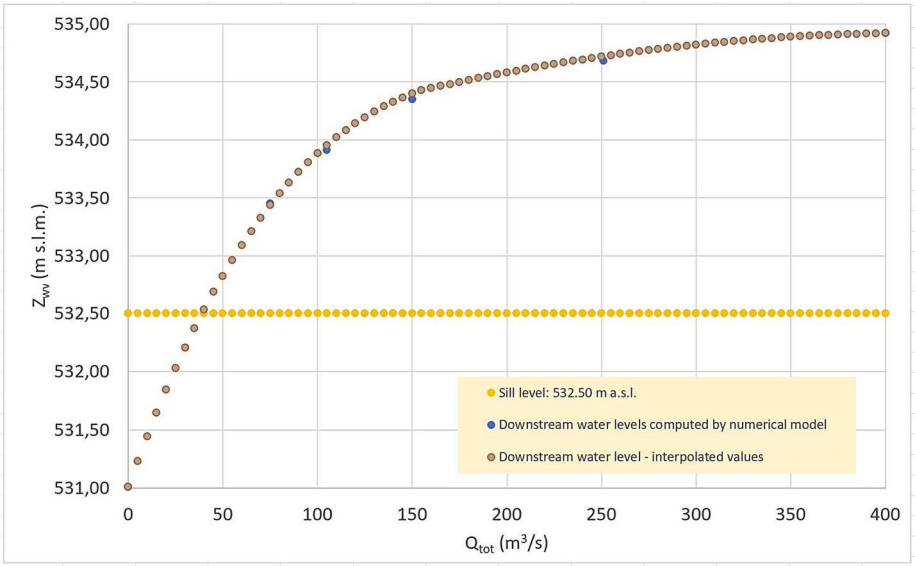


Fig. 15 – Downstream rating curve: average water level vs. total discharge (barrage No. 2)

Fig. 16 shows the discharge laws through the barrage for different degrees of opening of the gates, assuming that such opening is the same for each gate. Also in this case the curves have been obtained by combining the laws corresponding to the different possible regimes.

As the total discharge rises, the course of the curves change when the reservoir level exceeds 535.30 m a.s.l. because the lateral spillway starts working.

For opening greater than 2.0 m, the curve corresponding to the opening of 2.0 m remains valid.

If the total discharge Q_{tot} is greater than 40 m^3/s, the free flow through the barrage occurs under submerged conditions. On the other hand, the gate-controlled flow is not drowned except for small opening values of the gates. Free flow over the lateral spillway is never submerged.

For such reasons the curves have been plotted for gate opening up to 2.0 m. Beyond this value, free-outflow conditions prevail because the sluice gates do not control the upstream level, also thanks to the contribution of the lateral spillway.

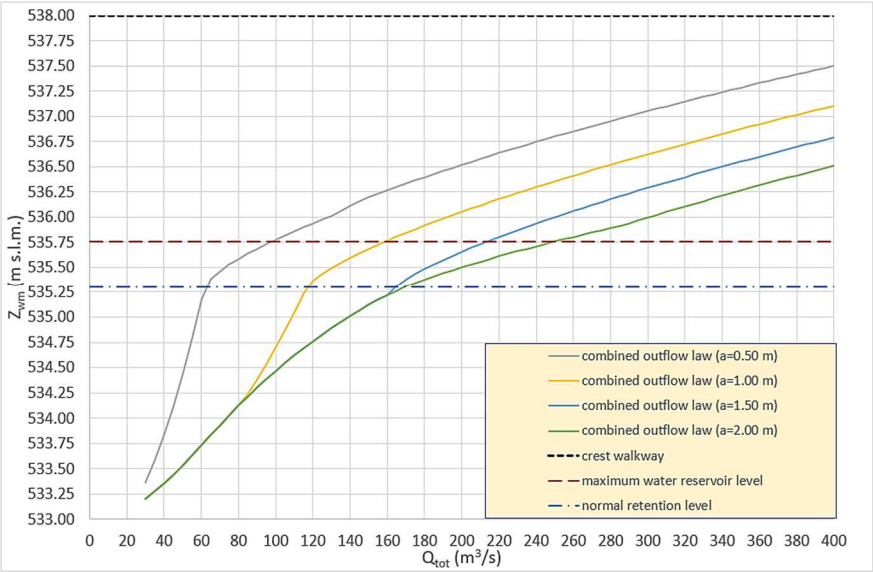


Fig. 16 – Discharge laws through barrage No. 2 for different gate openings (same opening for each gate)

At a glance, the procedure outlined in § 4.1 allows to characterize a potentially drowned barrage by computing its discharge laws, whereas the procedure of § 4.2 computes the flowing discharge in given conditions of upstream water levels and openings of the gates. In both cases downstream water levels are estimated by a hydraulic numerical model.

This methodology can be in principle implemented using analytical formulations other than those listed in Tab. 1 (for instance formulas relative to flow under radial gates).

6. CONCLUSIONS

The problem of characterising the discharge capacity of river barrages, related to the determination of the discharge magnitude during floods, even under potentially submerged conditions, has been addressed with established literature formulations. In the case of submerged condition, however, the mathematical expressions include the downstream water level in addition to the other usual variables (upstream level, discharge and opening of gates).

The developed methodology makes it possible to overcome the unknown downstream level by using a numerical model to construct a rating curve along the riverbed downstream a given barrage. This relationship is then put into system with the formulas describing the outflow laws, either to characterise the outlets or to estimate the discharge flowing along the river, given the reservoir level and the opening of each gate. For this last estimate, an easy-to-use software application has been developed, which can be supplied after the necessary configuration to implement the discharge laws of a specific plant.

The procedure needs only the geometry of the barrage and the downstream river morphology as input, but the hydraulic parameters could of course be calibrated in case any data achieved during with floods of known discharge be available.

REFERENCES

[1] Blevins, R. D. (1984). Applied fluid dynamics handbook. Krieger Pub. (2003).

[2] Citrini, D., Noseda G. (1987). Idraulica. Casa Editrice Ambrosiana.

[3] Kubrak, E. et al. (2020). Flow Measurements Using a Sluice gate. Analysis of Applicability. Water, 12, 819. https://www.mdpi.com/2073-4441/12/3/819

[4] Villemonte, J.R. (25/12/1947). Submerged-Weir Discharge Studies. Engineering News-Record. https://www.fs.usda.gov/biology/nsaec/fishxing/fplibrary/Villemonte_1947_Submerged_weir_discharge_studies.pdf

Italian regulatory references

[5] Decreto del Presidente della Repubblica 1° novembre 1959, n. 1363. Approvazione del regolamento per la compilazione dei progetti, la costruzione e l'esercizio delle dighe di ritenuta.

[6] Circolare P.C.M. 13 dicembre 1995, n. DSTN/2/22806. Disposizioni attuative e integrative in materia di dighe.

[7] Legge Regionale Abruzzo 27/06/2013, n. 18. Disciplina degli sbarramenti di ritenuta e degli invasi idrici di competenza regionale.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

FREEBOARD AND SPILLWAY CAPACITY PROGRESSIVE REDUCTIONS FOR THE LAGO VERDE ROCKFILL DAM, DUE TO THE CREST SETTLEMENTS AFTER SEVERAL DECADES OF OPERATION

Chiara Cesali[1]
University of Rome Tor Vergata, Italy

Francesco Federico[2]
University of Rome Tor Vergata, Italy

Marina Maestri
Alperia SpA
Bozen, Italy

ABSTRACT

The Lago Verde dumped rockfill dam, in northern Italy, show (vertical and horizontal) displacements of the crest (recorded and cumulated during more than 50 years of operation), due to comminution and breakage mechanical processes of composing large sized blocks, coupled with the corresponding re-arrangement of voids as well as to the drawdown-filling cycles of the reservoir. Currently, the maximum vertical and horizontal displacements amount to 0.47 m and 0.24 m (towards downstream), respectively. The current rate of the crest settlement is 6-7 mm/year.

The crest settlements induce the reduction of the freeboard and the possible corresponding reduction of the spillway and other outlet works discharges, if a lowering of the maximum water elevation be imposed, in order to maintain the original freeboard (i.e. 1,9 m).

To evaluate the correct evolution of the hydraulic safety factor of the dam, related in turn to the maximum level of the reservoir and to the efficiency of the outlets, a detailed hydrological study of the catchment underlying the dam was developed. In particular, through the geomorphological method, the reference 1000-year ARI (Average Recurrence Interval) peak flow discharge was determined, by taking into account the melting of the glaciers and, parametrically, several values of the flow velocities along the slopes and the mountain streams.

Thus, the hydraulic balance (based on continuity equation) between the incoming catchment flow discharges (1000-year ARI) and the spillway and bottom outlet discharges was carried out, also by taking into account the storage volume of the reservoir, and under different possible working (opening) conditions of the bottom outlet.

For each simulated scenario, the current and expected hydraulic safety of dam - based on the minimum freeboard between the dam crest level (at the current and expected elevations related to the evolution of settlements) and the reservoir water level - was verified.

Definitely, there would remain a minimum effective hydraulic freeboard equal to +1.97 m at the current date, or +1.47 m (in the absence of interventions) in 70 years, assuming an additional crest settlement of 50 cm (7 mm/year). Correspondingly, the design hydraulic freeboard, 1.9 m, at the time of construction of the dam would remain guaranteed for another 10 years, approximately.

Keywords: Crest settlement, freeboard, spillway capacity, hydrological analyses, hydraulic balancing of water reservoir.

[1] cesali@ing.uniroma2.it, Italy
[2] fdrfnc@gmail.com, Italy

1. INTRODUCTION

The Lago Verde dumped rockfill dam, in Northen Italy, show (vertical and horizontal) displacements of the crest (measured and cumulated during more than 50 years of operation), due to comminution and breakage mechanical processes of composing large sized blocks, coupled with the corresponding slow re-arrangement of voids, in turn induced by own weight as well as the drawdown-filling cycles of the reservoir ([1]).

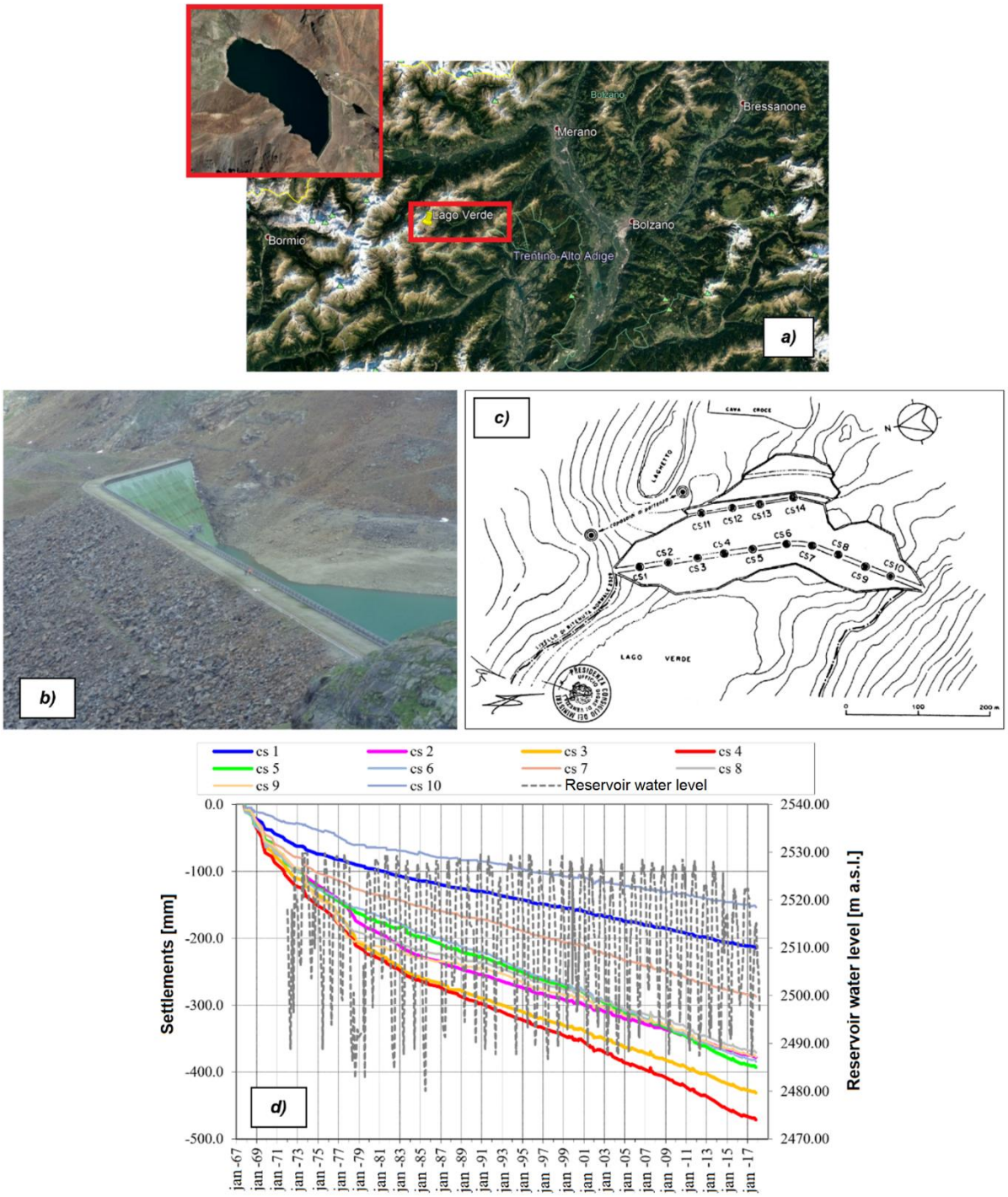


Fig. 1 – Lago Verde Dam: a) geographic setting; b) photo of the crest of the dam; c) location of landmarks (vertical displacements); d) measures of settlements in 50 years.

These movements have been continuously recorded since the late 60s, through fourteen landmarks (vertical displacements) and fourteen collimation targets (horizontal displacements). Currently, the maximum vertical and horizontal displacements amount to 0.47 m (see Fig. 1, for “cs 4” landmark) and 0.24 m (towards downstream), respectively.

The lowering of the crest level, due to the progressive settlements, has led to the need to recover the minimum design freeboard, i.e. 1,90 m as per F.C.E.M. (operating and maintenance conditions document of the dam).

Specifically, the original elevation of the dam crest was 2.531,7 m a.s.l.; the maximum reservoir water level is 2.529,8 m a.s.l. and the spillway elevation is 2.529,0 m a.s.l..

The spillway flow discharge (Q_s) may be estimated through the following relationship:

$$Q_s = \mu l h^{3/2} \sqrt{2g} \tag{1}$$

being $\mu = 0{,}47$; l = 45 m (spillway length); h = water depth above the spillway.

Instead, the bottom outlet flow discharge (Q_b) may be evaluated through the following formula:

$$Q_b = \sqrt{\frac{(h_m - h_c)}{\frac{1}{2\,gS^2C_v^2} + \frac{4^{10/3} \cdot n^2 \cdot L_1}{\pi D^{16/3}}}} \tag{2}$$

being L_1 = 80,26 m (hydraulic tunnel length, up to contracted cross-section); D = 1,9 m (hydraulic tunnel diameter); S = 0÷1,17 m^2 contracted cross-section area; C_v = 0,9 (contraction coefficient); n = 0,013 s/m$^{1/3}$ (Manning coefficient); $(h_m - h_c)$ = difference in elevation (in [m]) between the reservoir water level and the centreline of the contracted section (see Fig. 2).

Under the initial design condition previously introduced, the maximum spillway flow discharge is 67 m^3/s (i.e., h = 0.8 m in eq. (1)) and the maximum bottom outlet flow discharge is 29 m^3/s, overall about 96 m^3/s.

The current (minimum) elevation of the dam crest is 2.531,23 m a.s.l. (by considering the measured maximum settlement at “*cs 4*” landmark, 0.47 m). By imposing the minimum design freeboard (1,90 m), the current maximum reservoir water level should be 2.529,33 m a.s.l..

Under this current situation, the spillway flow discharge is about 18 m^3/s (i.e. h = 0.33 m in eq. (1)); the maximum bottom outlet flow discharge remains substantially equal to 29 m^3/s, thus the maximum flow discharge is about 47 m^3/s (about half of the original one).

Hydrological analyses carried out over the years by the dam manager (through the application of the well-known, simple, rational method) provided the following values for the ARI (average recurrence interval) 1000 years peak flow discharge: 45÷55 m^3/s.

Thus, the minimum freeboard would still be guaranteed today.

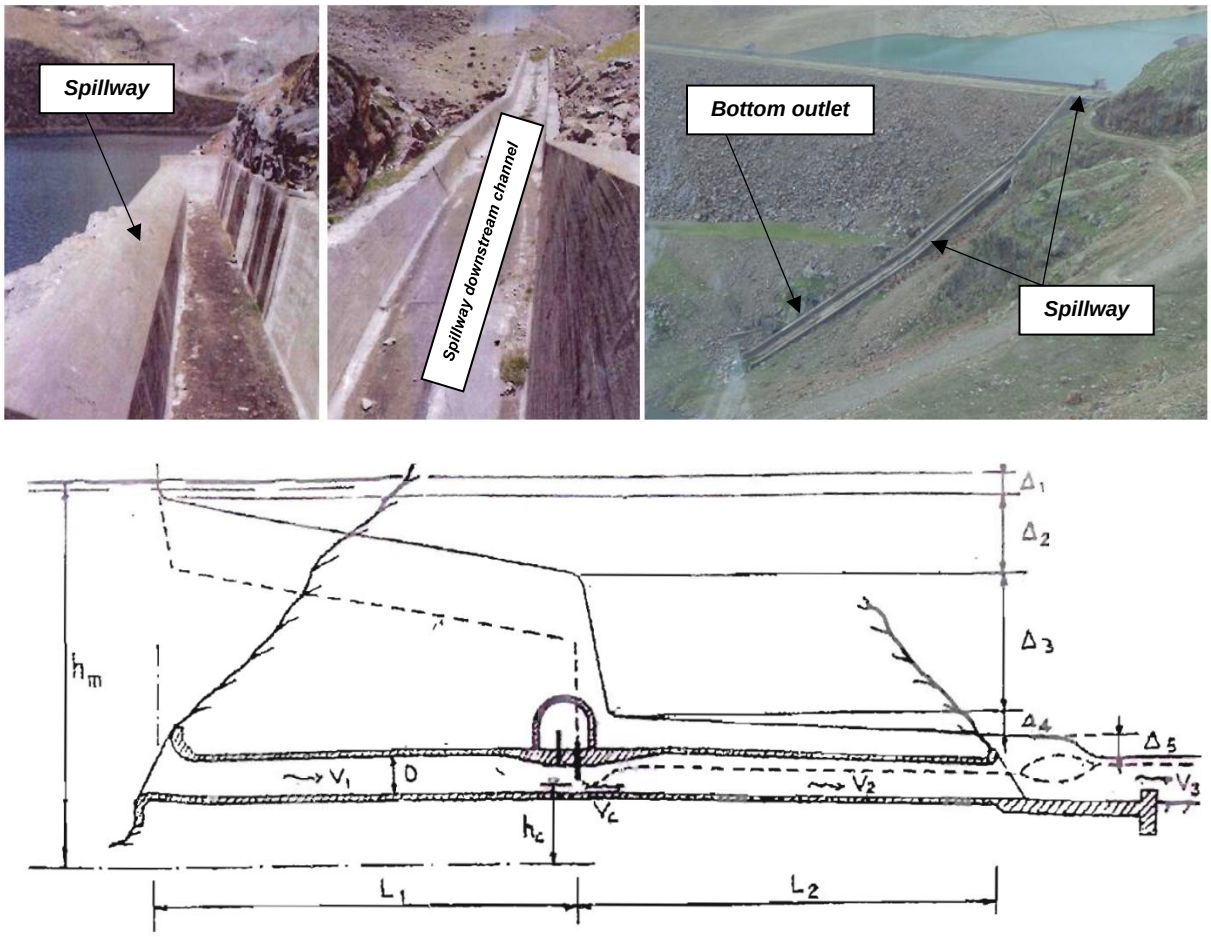


Fig. 2 – Lago Verde Dam: photos of spillway and bottom outlet (above) and scheme of the bottom outlet (below).

Although the current situation is still considered acceptable (due to the size and altitude of the catchment basin, as well as the flow rates involved), however, in consideration of the persistence of the measured settlements and the recently issued Italian technical standards and laws for dams (which today - compared to the past - would require a higher minimum freeboard), it is considered appropriate to develop and keep available and updated, also in light of the expected progress of the aforementioned settlements (6-7 mm/year, [1]), detailed (and more advanced than the rational method) hydrological/hydraulic studies as well as specific technical proposals aimed at keeping under control and improving the hydraulic safety of the dam.

To this purpose, as well as to define any necessary (active or passive) countermeasures, a detailed approach, also by taking into account the lamination effect coupled to the reservoir (dynamic hydraulic balance), based on the *Geomorphological Instantaneous Unit Hydrograph (GIUH)* method for determining the design flood hydrographs, is proposed.

The flow chart of this procedure is shown in the following figure.

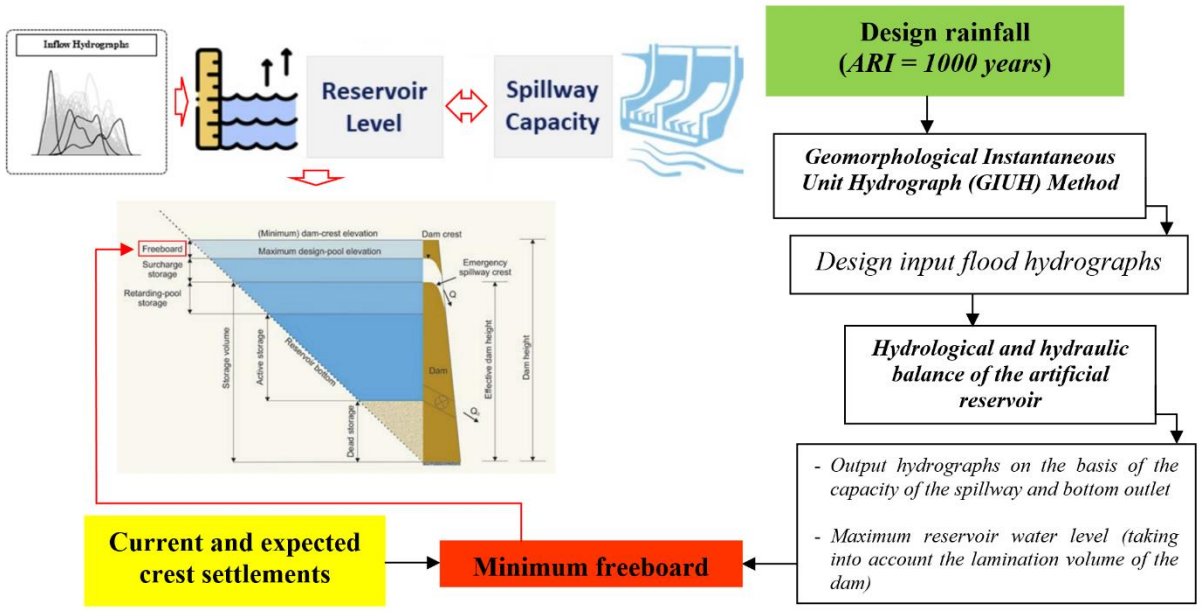


Fig. 3 – Detailed approach to evaluate the minimum freeboard: flow chart.

2. HYDROLOGICAL ANALYSES

As previously introduced, the input flood hydrographs pertaining to the catchment under the Lago Verde Dam (i.e. inlet to the reservoir) were determined using the *Geomorphological Instantaneous Unit Hydrograph (GIUH)* method.

This method follows the standard setting of linear inflow-outflow models, based on time-invariant transfer functions [2]. Its application, based on the morphology of the surface hydrographic network, has represented a great advance in the search for simplified techniques for the representation of the hydrological response, compared to the adoption of purely conceptual limited models, such as those implemented in most of the numerical codes available today.

A geomorphological model offers the opportunity to take into account in a detailed, complete and effective way the influence of the morphometric characteristics of the catchment on the concentration of the outflows, or on the hydrological response of the catchment itself.

Thus, this method depends on morphological parameters such as flow velocity of the slopes and flow velocity in the hydrographic network, which must therefore be chosen appropriately and reasonably.

In this study, for the implementation of the *GIUH* method, the AdBtoolbox software (AIDRO tool) was applied; for more details see https://gn.mase.gov.it/portale/en/adb-toolbox.

First of all, the catchment of Lago Verde Dam was delimited (on the basis of DTM provided by http://geokatalog.buergernetz.bz.it/geokatalog/#!) and the relative morphometric characteristics were determined, as shown in the following figure.

A (km²)	6,34
H_{max} (m a.s.l.)	3.442,0
H_{min} (m a.s.l.)	2.516,0
H_{med} (m a.s.l.)	2.918,9
L_s (km)	2,0
i_{bas} (m/m)	0,66
I_{str} (m/m)	0,167

(A = catchment area; H_{max} = catchment maximum elevation; H_{min} = catchment minimum elevation; H_{med}= catchment average elevation; L_s = main stream length; i_{bas} = catchment average slope; i_{str} = main stream average slope)

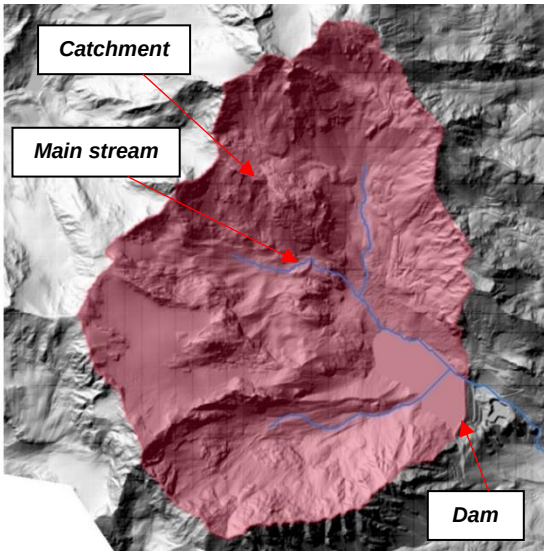


Fig. 4 – Lago Verde Dam: catchment and related morphometric characteristics.

Afterwards, the rainfall data measured at the nearby "*Ultimo Fontana Bianca*" rain gauge, in the period 1961-2023 (i.e. 58 observations), for the durations 1, 3, 6, 12, 24 hours, were collected and statistically analysed through the *log-normal distribution* (chosen among others following adaptation tests), in order to define the design *Depth-Duration-Frequency* (DDF) curves, according to the relationship: $h_r = a \cdot t^n$ (h_r = rainfall depth; a *[mm/h^n]* and n, typical parameter of DDF). The values of coefficients a and n, for different ARIs, are below shown.

Table 1 – Lago Verde Dam: design DDF curves, for different *ARIs*.

ARI (years)	*a [mm/h^n]*	*n [ ]*
10	20.92	0.49
30	25.15	0.48
50	26.72	0.48
100	29.13	0.47
500	34.69	0.46
1000	37.10	0.46

In the AdBtoolbox software, the following input parameters must be defined and assigned:

- *Curve Number* (*CN,* dimensionless parameter indicating the runoff response characteristic of a catchment; CN = 89 for the catchment under examination)
- "threshold" (of catchment area, default value = 0.003 km^2)
- "hyetogram shape/method" (*Alternate Blocks* method was chosen because it allows to better simulate the measured rainfall data)
- "peak advance" (default value = 0.5)
- "rainfall data step" (default value = 5 mm)
- "AMC" (Antecedent soil Moisture Conditions)
- V_v = runoff/slope flow velocity
- V_c = runoff/stream flow velocity

In order to calibrate the hydrological model developed in AIDRO tool, specifically the parameters *AMC,* V_v, V_c, the rainfall records of the August 1987 event, only for which the peak flow discharge measure - 12 m^3/s - is available, were recovered.

As shown in Fig. 5, the rainfall data measured during the August 1987 event can be associated with an ARI 10 years for durations of 1÷3 hours (closer to the characteristic/critical ones of the catchment under examination), with an ARI 30 years for durations of approximately 6 hours and with an ARI 50 years for durations of 12÷24 hours.

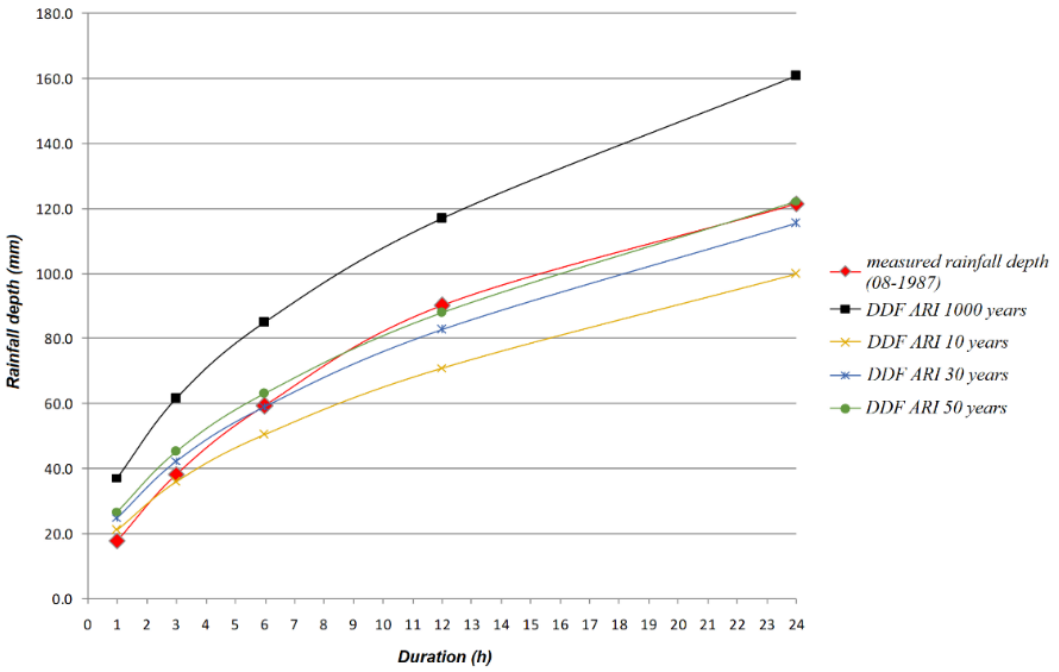


Fig. 5 – Lago Verde Dam: comparison between the values of rainfall depth measured during the August 1987 event and the DDF curves.

On the basis of these measured rainfall data, the associated hyetogram (with 15 minutes discretization), to be imposed to the developed hydrological model, was defined (Fig. 6).

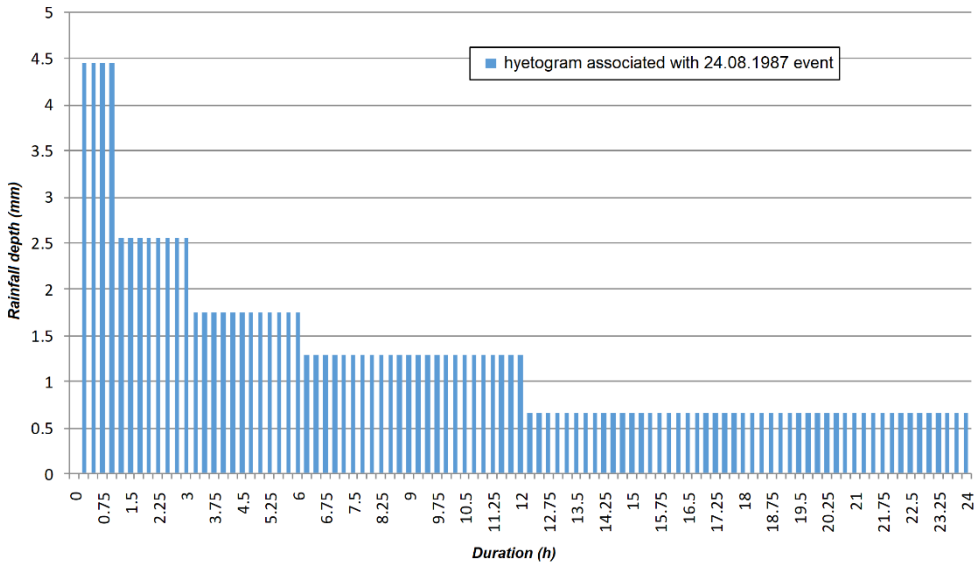


Fig. 6 – Lago Verde Dam: hyetogram associated with the August 1987 event.

Fig. 7 shows the final result of the calibration process of the developed hydrological model, i.e. the resulting flood hydrograph, with a peak flow discharge of 12.5 m^3/s, consistently with the measured/recorded value (about 12 m^3/s).

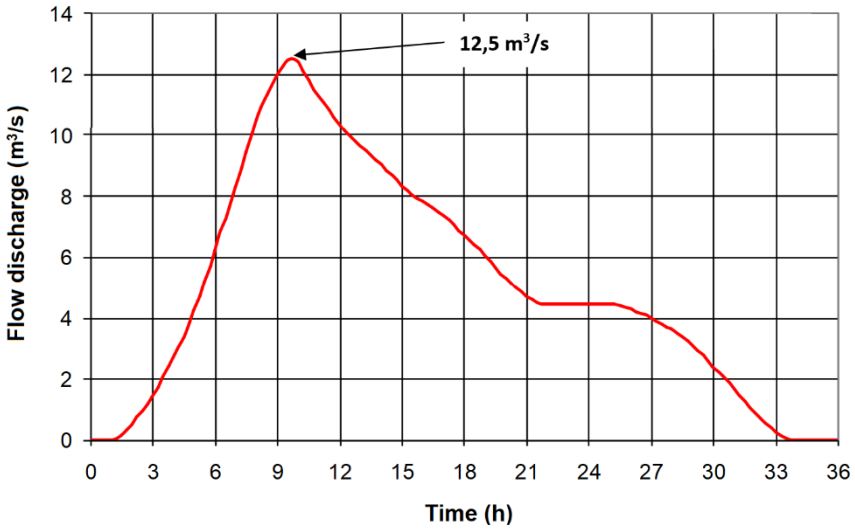


Fig. 7 – Lago Verde Dam: flow hydrograph associated with the August 1987 event, provided by the developed (calibrated) hydrological model.

This result was obtained by applying the following parameters: $V_c = V_v = 0.1$ m/s and "AMC" = 2 (hence CN = 89), that indicate, at least according to the simulated event, a low influence of the outflows along the branches/streams of the catchment in the formation of the flood, therefore mainly due to the phenomenon of runoff of the inflows along the slopes of the basin.

It is worth observing that the simulated event (the only one available) is associated with ARIs that are considerably shorter than the reference/design ones (i.e. ARI 1000 years), therefore it could be considered not very significant for the purposes of this study.

For these reasons, for the parameters V_v and V_c, typical values from technical literature are also analysed and considered. Specifically, V_v may be assumed values between 0.1 ÷ 0.3 m/s ([3]). V_c may be evaluated through the following relationship: $V_c = 5{,}83\, i_{bas}{}^{0{,}275}$ ([4]), from which values up to 5 m/s can be obtained for the catchment under examination.

Based on the characteristics of the catchment, the precautionary value AMC = 3 (from which CN = 96,5) is also assumed.

Through the developed calibrated hydrological model and on the basis of the previous considerations, by parametrically assigning the values of the parameters V_v and V_c, the following, different, reference flow hydrographs (ARI 1000 years) are obtained:

- *Hydrograph 1 – V_v= 0.3 m/s, V_c= 3 m/s;*
- *Hydrograph 2 – V_v= 0.2 m/s, V_c= 2 m/s;*
- *Hydrograph 3 – V_v= 0.1 m/s, V_c= 1 m/s.*

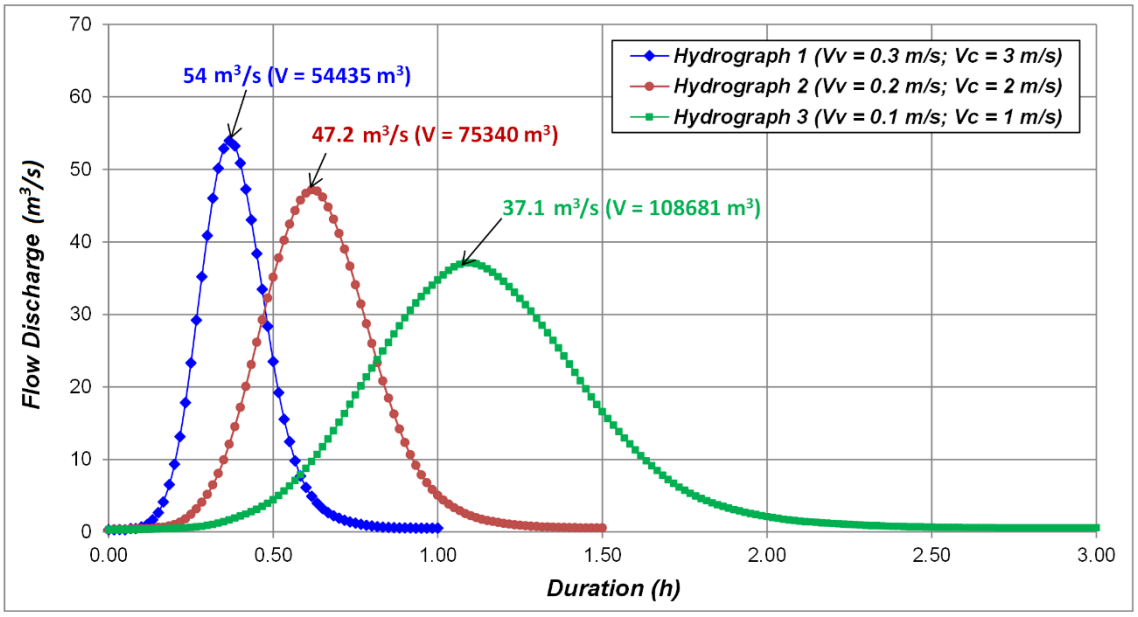


Fig. 8 – Lago Verde Dam: input flow hydrograph (ARI 1000 year).

According to this assumption, several "*hydrological behaviours*" of the catchment under examination are considered. V_c values higher than those associated with hydrograph 1, although possible, are not considered because, as shown in the next chapter, they provide extremely small changes in the freeboard and, therefore, not sufficiently cautious.

As V_c and V_v decrease, the changes in the freeboard become progressively more appreciable. Specifically, the values of V_c and V_v associated with the hydrograph 3 can be considered as the lower limit of effective admissibility, with reference to the calculations and values from the technical literature above: as shown in the following hydraulic assessment, the maximum changes in freeboard correspond to them.

Additionally, due to the location of the catchment and the Lago Verde dam, the contribution in peak flow discharge deriving from the melting of the glaciers was also considered. Specifically, this contribution has been evaluated by the linear kinematic method, i.e., as a triangular flood hydrograph (added to those previously derived), with peak flow discharge (Q_c) equal to:

$$Q_c = \frac{M \cdot A}{3{,}6} \tag{3}$$

being M (= melting rate in mm/h) equal to (e.g. [5]):

$$M = A_{\%}\left(\frac{i \cdot T_a}{80} t_c + \frac{K_s \cdot T_a}{24}\right) t_c \tag{4}$$

being $A_{\%}$ = percentage of snow-covered basin surface area ($A_{\%}$=*1*); i = rainfall intensity (mm/h), $i = a \cdot {t_c}^{n-1}$, with a and n parameters of the DDF curve for ARI 1000 years; T_a = daily mean temperature (T_a=5.1°C); K_s = melting factor (K_s=6.64 mm/(°C·day)); t_c = time/duration corresponding to the peak of the reference flood hydrograph.

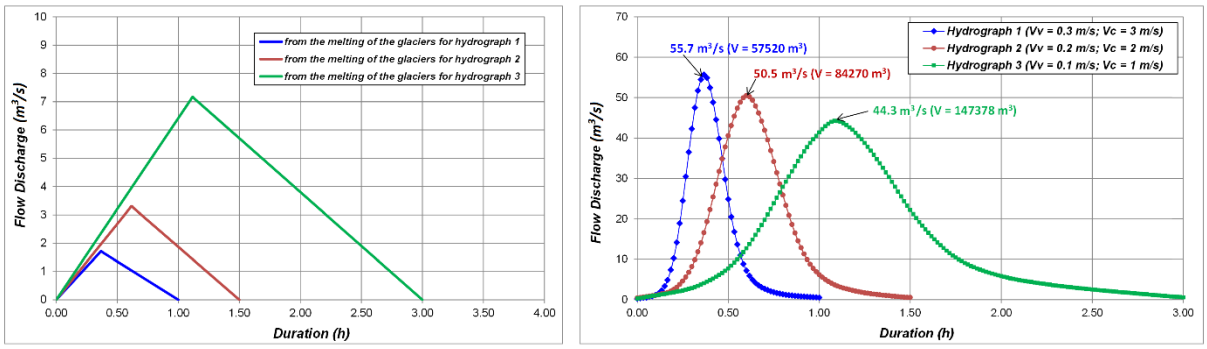


Fig. 9 – Lago Verde Dam: contribution deriving from the melting of glaciers (left) and input flow hydrograph (ARI 1000 year) under the melting of glaciers condition (right).

3. HYDRAULIC ASSESSMENT

The hydraulic balance of an artificial reservoir can be described through the following governing equation [6]:

$$Q_i - Q_u = \frac{dV_i}{dt} \tag{5}$$

being Q_i = flow rate entering the reservoir; Q_u = flow discharge exiting the reservoir; V_i = volume of the artificial reservoir. By discretizing the previous equation through the finite difference method, adopting simple first-order approximations and central difference schemes, the following algebraic expression can be obtained:

$$\frac{{Q_i}^t + {Q_i}^{t-1}}{2} - \frac{{Q_u}^t + {Q_u}^{t-1}}{2} = \frac{\Delta V_i}{\Delta t} = \frac{{V_i}^t - {V_i}^{t-1}}{\Delta t} \tag{6}$$

where the superscript *t* and *t-1* indicate respectively the values at the end and at the beginning of the generic (assigned) time interval Δt.

The flow discharge Q_u can be expressed as a function of the increase in the water level (*h*) in the reservoir or of the water depth over the spillway, which in turn depends on the (*surcharge storage*) volume of the reservoir V_i; specifically, for the Lago Verde artificial reservoir:

$$h(V_i) = \frac{0{,}8}{1{,}98 \cdot 10^5} \cdot V_i \tag{7}$$

Thus,

$$Q_u = Q_{u,s} + Q_{u,f} = Q_{u,s}(V_i) + Q_{u,f} \tag{8}$$

being $Q_{u,f}$ = flow rate from the bottom outlet and $Q_{u,s}$ = flow rate from the spillway (according to the previous eq. (1)):

$$Q_{u,s}(V_i) = \mu L_s [h(V_i)]^{\frac{3}{2}} \sqrt{2g} = \mu L_s \sqrt{2g} \left[\frac{0{,}8}{1{,}98 \cdot 10^5} \cdot V_i\right]^{\frac{3}{2}} \tag{9}$$

Thus, the eq. (5) is numerically solved (through an algorithm in MATLAB language).

As previously observed, for the water level in the reservoir ranged between 2.529,80 m a.s.l. (the maximum reservoir water level) and 2.529,0 m a.s.l. (the spillway elevation) which identify the so-called "*surcharge storage volume*", the discharge flow rate can be considered constant, equal to approximately 29 m^3/s (= $Q_{u,f}$). To this purpose, the hydraulic analyses are carried according to the following different operating conditions for the bottom outlet:

- Scenario A: "*closed bottom outlet*";
- Scenario B: "*partially opened bottom outlet*" ($Q_{u,f} = 14{,}5\ m^3/s$), according to a controlled opening operation over 30 minutes;
- Scenario C: "*fully opened bottom outlet*" ($Q_{u,f} = 29\ m^3/s$), according to a rapid opening operation.

However, as per F.C.E.M., the flow discharge from the bottom outlet must not exceed the input flow discharge into the reservoir during the increasing phase of hydrograph.

The residual freeboard is therefore evaluated for each of the scenarios as defined above, for the various input hydrographs previously determined.

3.1. Main results and discussion

In the following tables, the obtained main results are shown ($Q_{i,\,max}$ = maximum inflow peak discharge; V = hydrograph volume; $Q_{u,max}$ = maximum outflow discharge; $Q_{u,\,s,\,max}$ = maximum outflow discharge from spillway; $Q_{u,\,f,\,max}$ = maximum outflow discharge from bottom outlet; $V_{i,max}$ = maximum *surcharge storage* volume in the reservoir), alternatively under presence and absence of melting of glaciers conditions (*(*)* the residual freeboard is evaluated with respect to the current minimum dam crest elevation).

Table 2 – Main results (no melting of glaciers).

Case	Input hydrograph			Outflow hydrograph				$V_{i,max}$ [m³]	*Maximum water level* [m s.l.m.]	*Residual freeboard* [m] *(*)*
	$Q_{i,max}$ [m³/s]	V [m³]	*Duration* [h]	*Scenario*	$Q_{u,max}$ [m³/s]	$Q_{u,\,s,\,max}$ [m³/s]	$Q_{u,\,f,\,max}$ [m³/s]			
Hydrograph 1	54,0	54.435,0	1,0	A	6,5	6,5	0,0	41.770,0	2.529,17	2,06
				B	18,7	4,2	14,5	31.365,0	2.529,13	2,10
				C	30,78	1,78	29,0	17.622,0	2.529,07	2,16
Hydrograph 2	47,2	75.340,0	1,5	A	10,0	10,0	0,0	55.580,0	2.529,23	2,00
				B	18,9	4,4	14,5	32.290,0	2.529,14	2,09
				C	31,52	2,52	29,0	22.239,7	2.529,09	2,14
Hydrograph 3	37,1	108.681,0	3,0	A	12,5	12,5	0,0	64.740,0	2.529,26	**1,97**
				B	23,55	9,05	14,5	52.134,2	2529,22	2,01
				C	32,51	3,51	29,0	27.744,5	2529,12	2,11

Table 3 – Main results (with melting of glaciers).

Case	Input hydrograph			Outflow hydrograph				$V_{i,max}$ [m³]	*Maximum water level* [m s.l.m.]	*Residual freeboard [m] (*)*
	$Q_{i,max}$ [m³/s]	V [m³]	*Duration* [h]	*Scenario*	$Q_{u,max}$ [m³/s]	$Q_{u,\,s,\,max}$ [m³/s]	$Q_{u,\,f,\,max}$ [m³/s]			
Hydrograph 1	55,7	57.520,0	1,0	A	7,0	7,0	0,0	43.730,0	2.529,18	2,05
				B	19,1	4,6	14,5	33.120,0	2.529,13	2,10
				C	31,04	2,04	29,0	19.278,5	2.529,08	2,15
Hydrograph 2	50,5	84.270,0	1,5	A	11,4	11,4	0,0	60.720,0	2.529,25	1,98
				B	19,9	5,4	14,5	37.000,0	2.529,15	2,08
				C	32,35	3,35	29,0	26.911,3	2.529,11	2,12
Hydrograph 3	44,3	147.378,0	3,0	A	17,3	17,3	0,0	80.150,0	2.529,32	**1,91**
				B	27,00	12,50	14,5	64.636,8	2529,26	1,97
				C	34,17	5,17	29,0	35.879,9	2529,15	2,08

As previously specified, the lowest residual freeboard values are obtained for the hydrograph 3 which, although affected by a lower peak flow discharge than the remaining considered hydrographs, defines a greater water volume.

It is also reasonable to exclude the scenario A (unusable bottom outlet) which is unlikely/far from occurring as the dam is continuously subjected to maintenance and under high surveillance.

By way of example, in the following figures, the evolution of inflow and outflow discharges, *surcharge storage* volume in the reservoir, as well as the water depth over the spillway and the corresponding freeboard, according to scenario C and hydrograph 1 (no melting of glaciers), and scenario B and hydrograph 3 (with melting of glaciers), is shown.

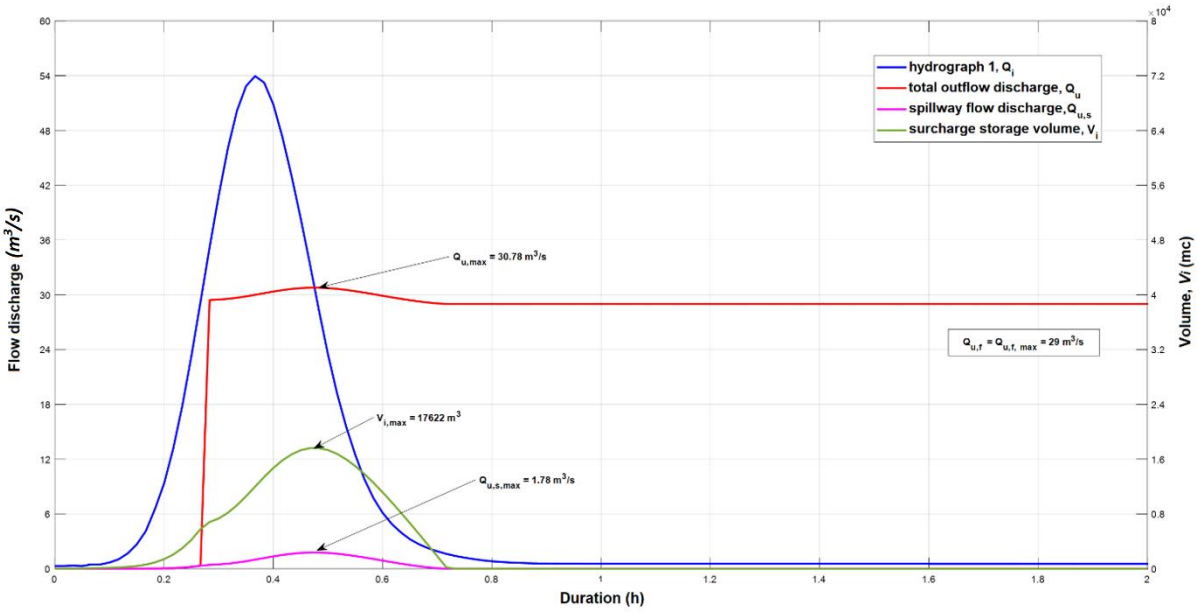


Fig. 10 – Lago Verde Dam: hydraulic balance, evolution of inflow and outflow discharges (hydrograph 1, scenario C, no melting of glaciers).

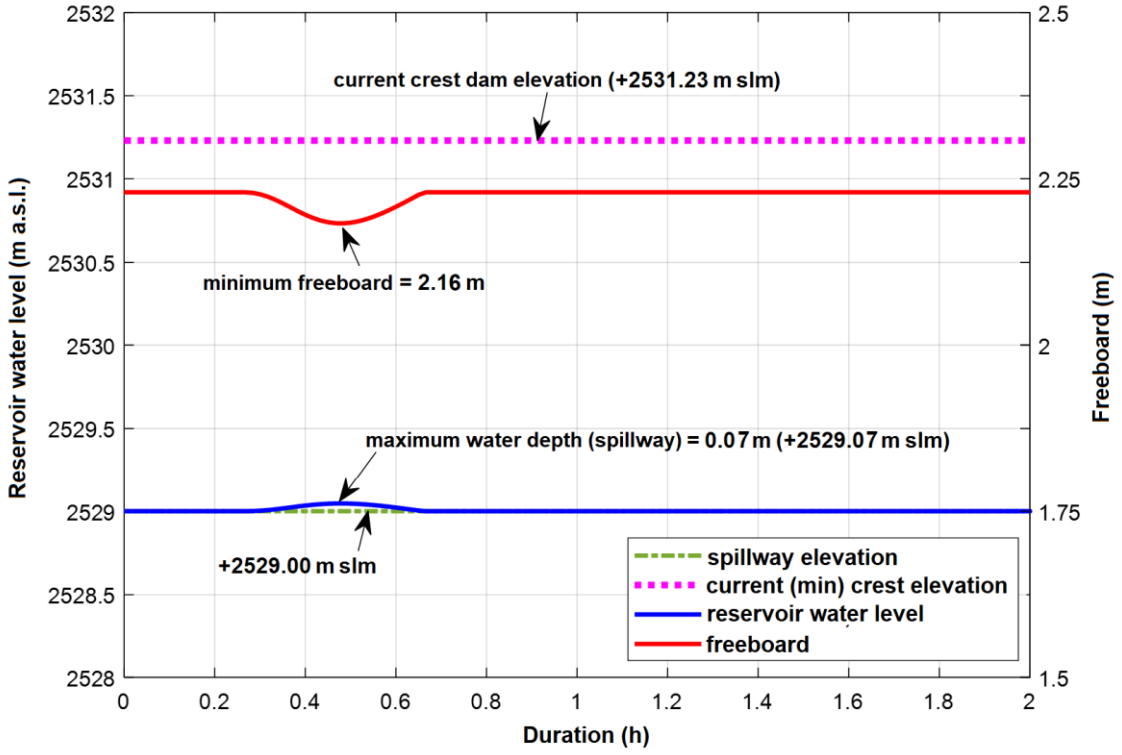


Fig. 11 – Lago Verde Dam: hydraulic balance, evolution of reservoir water level, freeboard, water depth over spillway (hydrograph 1, scenario C, no melting of glaciers).

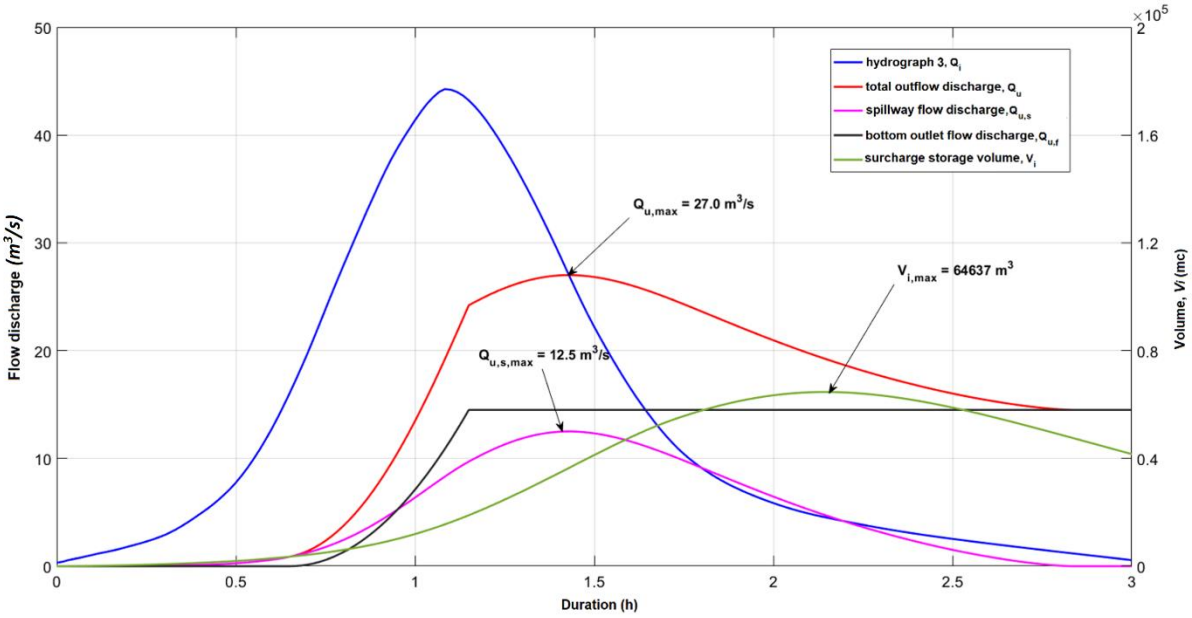


Fig. 12 – Lago Verde Dam: hydraulic balance, evolution of inflow and outflow discharges (hydrograph 3, scenario B, with melting of glaciers).

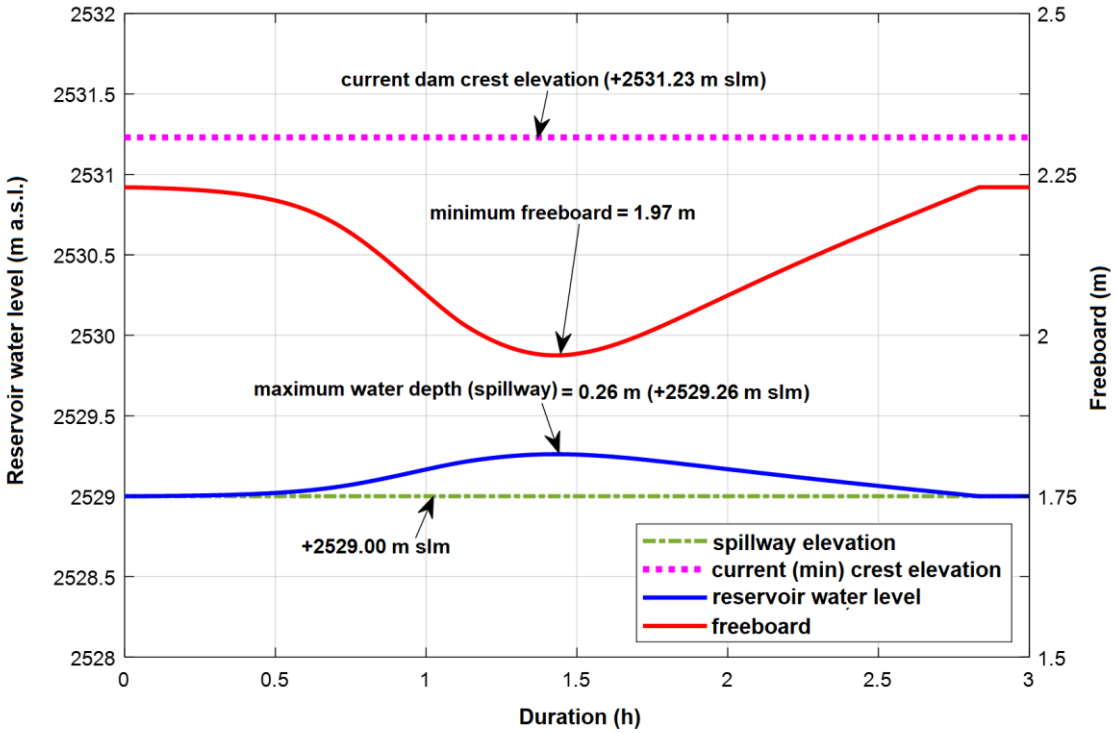


Fig. 13 – Lago Verde Dam: hydraulic balance, evolution of reservoir water level, freeboard, water depth over spillway (hydrograph 3, scenario B, with melting of glaciers).

Ultimately, with reference to ARI 1000 years peak flow discharges and the numerical simulations performed, a minimum effective freeboard (difference between the crest level and the reservoir level) equal to +1.97 m at the current situation (i.e., scenario B, hydrograph 3), or +1.47 m (in the absence of countermeasures) in 2087, by assuming a further settlement of the dam crest of 50 cm at "*cs4*" landmark, taking into account the current evolution of the settlements (6-7 mm/year, [1]), would remain. Correspondingly, the "original" design hydraulic freeboard, 1.9 m (F.C.E.M.), at the time of construction of the dam would remain ensured for another 10 years, approximately.

4. CONCLUSIONS

The analyses and corresponding results previously described allowed to highlight the following aspects:

- peak flow discharge values (ARI 1000 years), i.e. about 40÷60 m^3/s, deriving from rainfall and snowfall (simultaneously), are comparable with those ones obtained in previous studies (by the dam manager);
- the input peak flow discharge to the reservoir does not represent the only variable to be considered in the evaluation of the outflows from the dam's discharge devices (spillway and bottom outlet), and therefore in the verification of the minimum available freeboard; also the outflow volume, i.e. the volume defined by the flood hydrograph, plays a role that is not negligible;
- the artificial reservoir plays a key role with its lamination/ surcharge storage capacity;
- the "original" design hydraulic freeboard, 1.9 m (F.C.E.M.), would remain ensured for another 10 years, approximately;
- after this period, specifically 2087, in order to comply with the original minimum freeboard value +1.90 m, the maximum reservoir water level, in the absence of any structural intervention (e.g. on spillway), should be set at +2,528.83 m a.s.l. (= +2,530.73 m a.s.l. – 1.90 m, where +2,530.73 m a.s.l. corresponds to the "cs4" crest elevation, expected in 70 years, by considering the current settlement rate, 6-7 mm/year), i.e. 17 cm below the current elevation of the spillway, and then the water level should therefore first rise by 17 cm to start the spillway discharge, temporarily reducing the freeboard to +1.73 m;
- in the alternative hypothesis of a failure of the bottom outlet, and of not implementing any structural intervention (and therefore of not complying with the original minimum freeboard), the temporary reduction of the freeboard would be set at the maximum value of 49 cm (freeboard = +1.41 m, see scenario A, hydrograph 3), always with

reference to ARI 1000 years flow discharge, in 2087 (in approximately 70 years), guaranteeing in any case the impossibility of phenomena of uncontrolled release of water from the artificial reservoir associated with overflowing from the crest.

REFERENCES

[1] Federico, F., Cesali, C., Cacciotti, M., Maestri, M. (2020). Evolution of crest displacements of the Lago Verde rockfill dam during fifty years of operations. 4th International Dam World Conference, Lisbon, Portugal, 2020.

[2] Mesa, O., Mifflin, E. (1986). On the relative role of hillslope and network geometry in hydrologic response. In: Scale Problems in Hydrology, V Gupta, I Rodríguez-Iturbe, E Wood (eds), Springer: Berlin; 1–17.

[3] Gregoretti, C., Degetto, M. (2012). *Debris Flow Modeling Tool: Reference Manual*. University of Padova, TeSAF Department.

[4] Di Lazzaro, M. (2009). Regional analysis of storm hydrographs in the Rescaled Width Function framework. Journal of Hydrology, 373(3):352-365.

[5] Misumi, R., et al. (2014). Empirical Relationships for Estimating Liquid Water Fraction of Melting Snowflakes. JOURNAL OF APPLIED METEOROLOGY AND CLIMATOLOGY, vol. 53, pp. 2232-2245.

[6] Brath, A. (2019). Flood risk mitigation downstream of dams. Lamination programs: state of implementation, critical aspects and perspectives. L'Acqua n.6/2019 (in italian).

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EXTRAORDINARY MAINTENANCE OF SAN MAURO'S DAM (ITALY): REPLACEMENT OF THE MAIN TAINTER GATES AND RESTORATION OF THE SAFETY CONDITIONS OF THE STRUCTURE

Stefano Munari[1]
Enel Green Power, Italy

Gaetano Vallese[2]
Enel Green Power, Italy

Lorenzo Bianco[3]
Bianco s.r.l., Italy

ABSTRACT

The San Mauro dam is located on the Po River near Turin and is made up of six 25 m tainter gates and one 12.375 m. On the right of the main structure a canal of approximately 13 km brings the waters towards the Cimena power plant with a power of 22.8 MW.

Following a specialist inspection carried out on the dam in 2020, serious damage to the steel trusses constituting the original gates was highlighted, such as to assign the work a value of CHS (Component Health Status) very low.

It was therefore decided to replace five of the seven (the oldest) gates during the years 2024-2026. It was also restored the system for lifting and moving the stoplogs, dismantled in the 2000s, in order to facilitate the work of replacing the gates.

The article describes in particular the construction site campaigns regarding the restoration of the stoplogs movement system and the replacement of the first tainter gate.

Keywords: Tainter gate, site construction, stoplog, maintenance

[1] stefano.munari@enel.com, Italy
[2] gaetano.vallese@enel.com, Italy
[3] tecnico@biancosrl.net, Italy

1. DESCRIPTION OF THE SAN MAURO'S DAM

The Cimena hydroelectric power plant, located in the province of Turin, was built in the years 1946-1949 by the enterprise Società Idroelettrica Piemonte (SIP) for the exploitation of the waters flowing in the upper course of the Po river for hydroelectric purposes and has a total power equal to 22.8 MW. In particular, the Cimena power plant uses water coming from a diversion weir located in San Mauro Torinese. The intake structure on the Po River is in the territory of the Municipality of San Mauro (TO), immediately near the city of Turin.

The dam is made up of 6 tainter gates 25 m wide each with a dam crest at 199.20 m above sea level and a normal depth water of 2.80 m. There is also another tainter gate opening 12.375 m wide with a crest at 198.50 and a normal depth water of 3.50 m with the aim of removing silt from the dam.

The dam had an important modification in 2002, in fact the number of 25 m gates were increased from 4 to 6 following the flood of 2000 which significantly affected the town of San Mauro. The two new tainter gates were built into the left side.

The intake, sand removal and regulation gates of the headrace channel are located on the right side, next to the dam.

The channel, which runs partly outdoors and partly in tunnel, for a total length of approximately 13 km, is designed for a maximum flow rate of 110 m^3/s and conveys the water to the loading basin of the Cimena power plant.

Fig. 1 – View of the San Mauro's dam from the right side

2. INSPECTION OF THE STRUCTURE AND DEFINITION OF THE COMPONENT HEALTH STATUS

Following an inspection by the staff of the operation unit in 2020, severe damage to the truss structure of the steel components was found on the original gates (total 5 gates). The seals, especially those at the bottom of the gates, were also severely compromised. Due to the situation, it was not considered possible to maintain the safe operation of the weir, especially in the event of particularly intense flood events, which could cause serious damage to the gates and, consequently, downstream.

At the same time as the inspection, a CHS (Component Health Status) value of 1 (the lowest) was assigned within the company database on components; this database allows, through an annual update of these values, to efficiently plan the activities of maintenance.

To solve the problem, a project order was opened which, following further specialist inspections, identified to proceed with the substitution of the entire 5 gates, preliminarily restoring the stoplogs and the stoplog lifter system in order to execute the replacement.

Fig. 2 – Damage on the truss structure on a tainter gate of San Mauro

3. CHOICE OF THE INTERVENTION AND AUTHORIZATION PROCESS

Once the project has been financially approved, it was necessary to evaluate the temporary works to be carried out to make the construction site operational. Until the 2000s, the weir had a stoplog system with a length of 12.375 to be installed with a use of a concrete block positioned at the middle of the gate length. The installation activity of the stoplogs on the 25 m gates had to be carried out by divers and was very complicated (Fig. 3).

The supply of an alternative stoplog lifter system was however adopted due to the impossibility of realizing a temporary dam in front of the weir which would avoids long out of service of the Cimena power plant.

Furthermore, the execution of the intervention was planned in a period of low hydraulicity of the Po river, which for this case coincides with the winter period (Fig. 4).

In fact, the ratio between the average maximum annual flow expected at the weir and the average maximum for the winter period is equal to 3.65; in this way it was possible to design the intervention considering an expected flow rate at the dam reduced by this value compared to the 200-year flow defined by the flood risk management plan and equal to 4300 m^3/s [1].

This method made it possible to define an expected flow rate during the execution of the construction site activity equal to 1181 m^3/s, with this flow it was evaluated the upstream water level with a gate out of service.

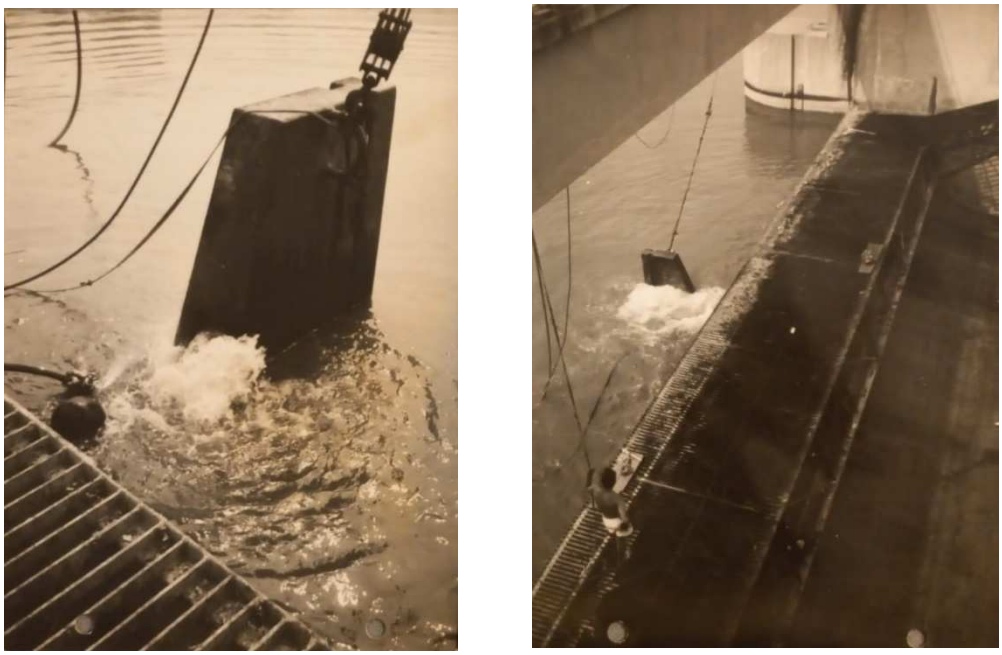


Fig. 3 – Ancient stoplog system for the 25 m gates of San Mauro

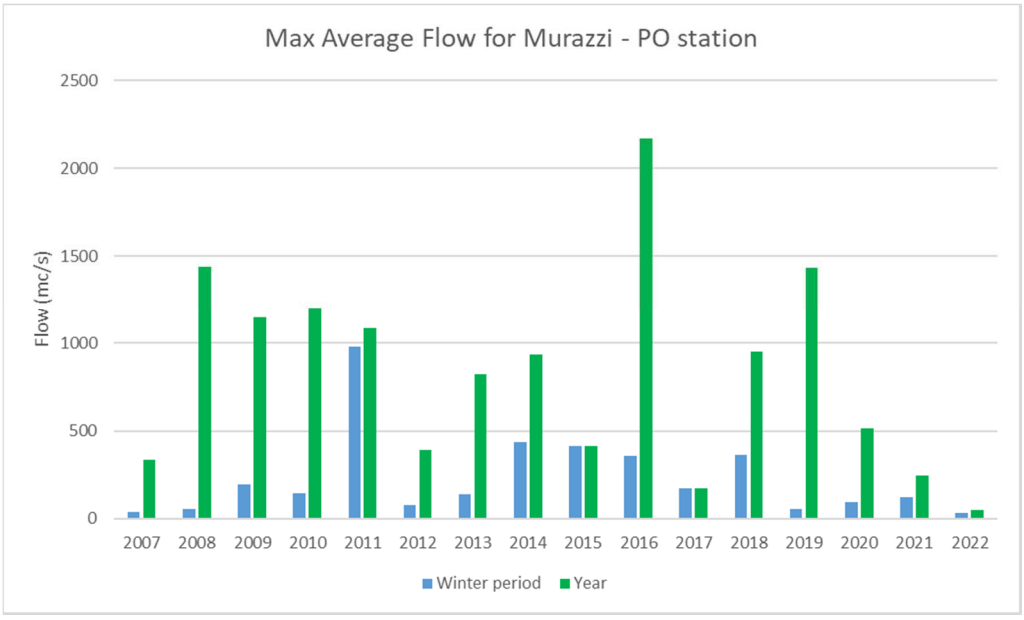


Fig. 4 – Average maximum flow for Murazzi station on Po river, upstream the weir of San Mauro

To issue the hydraulic authorization for the installation of the construction site by the Po Basin Authority, a two-dimensional hydraulic model was carried out [2], using HEC-RAS vers. 6.3, in order to estimate the water levels expected with a gate out of service due to the presence of the stoplog. The model had already been calibrated using roughness coefficients adopted in 2020 during the regularization of the hydraulic dam by the Piedmont region.

The upstream level reached with a gate out of service was equal to 202.34 m so the height of the stoplog was set at 202.40 m.

Subsequently a timetable of activities detailed below was defined:

- April 2023 – December 2023: installation of the new stoplog lifter system
- January 2024 – May 2024: supply of the stoplogs and replacement of the 12.375 m gate
- December 2024 – March 2025: replacement of two 25 m gates
- December 2025 – March 2026: replacement of two 25 m gates

4. NEW STOPLOG LIFTER SYSTEM INSTALLATION

Starting from April 2023, the installation of the new stoplog lifter system with a load capacity of 30 t began. The new structure was created by installing multi-directional scaffolding along the entire weir to allow the installation of the anchor beam and the related guide hoists.

The new system involved the installation of a HEB320 galvanized steel beam anchored to the concrete beam of the crest. The stoplog lifter system ends with a steel shelter and an underground storage pit.

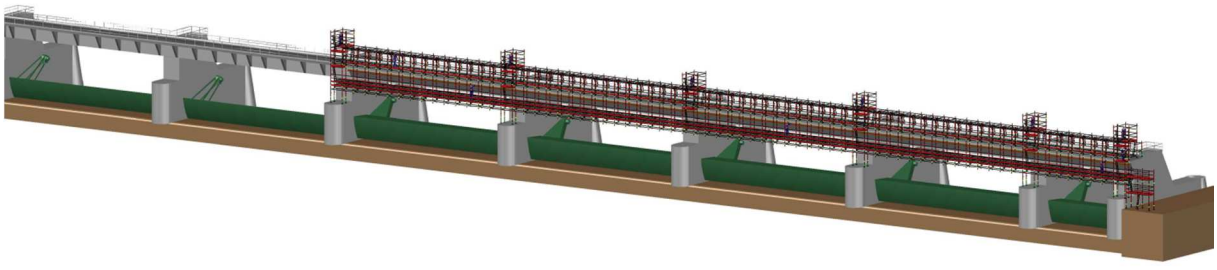


Fig. 5 – Multidirectional scaffolding on the San Mauro weir

The installation of the new system was completed in December 2023 with the execution of a static load test at 110% of the nominal capacity (33 t) which had a positive outcome. Furthermore, an operational test of the new crane was carried out along the entire track, having a total length of 125 m.

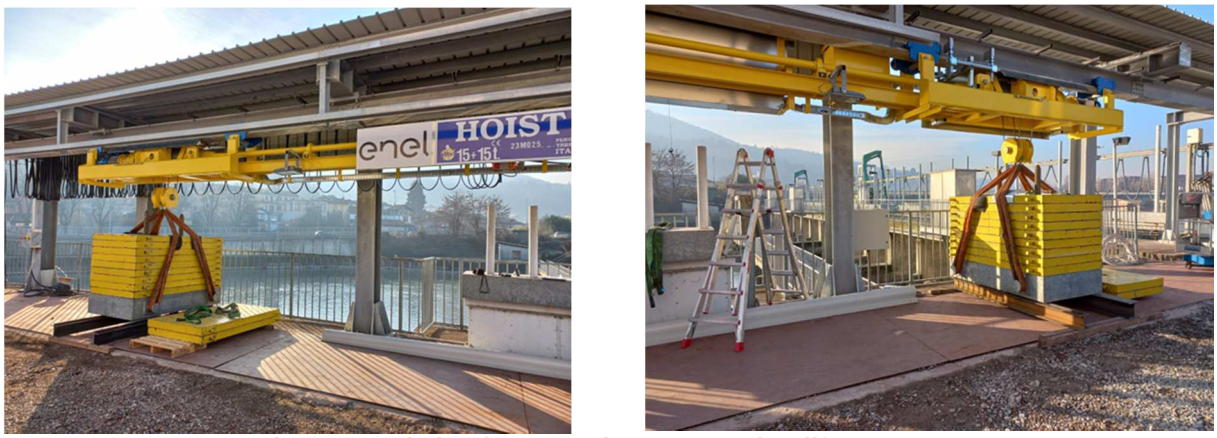


Fig. 6 – Static load test on the new stoplog lifter system

5. REPLACEMENT OF THE FIRST TAINTER GATE

Immediately after the test of the new lifter system it started the site construction for the substitution of the first tainter gate by the Bianco enterprise. The work began in December 2023 and ended in June 2024 with those steps:

- Provisional reinforcement of the access bridge to the weir (necessary to allow the transit of the mobile crane)
- Installation of the scaffolding access downstream of the gate
- Positioning of the new stoplogs and demolition of the old gate and the sills (on sides and on the bottom)
- Construction of the new anchor blocks for the hydraulic pistons
- Transport and installation of the new gate using a mobile crane
- Hydraulic test of the new gate

5.1. Reinforcement of the access bridge

To allow the transit of a mobile crane with a total weight of 60 t it was necessary to reinforce the access bridge of the dam: in fact, the bridge has a load capacity of only 26 t.

The replacement of the bridge was immediately discarded because in addition to access to the dam it also allows access to a municipal sports center, therefore a steel reinforcement of the existing concrete structure was implemented.

Fig. 7 – Reinforcement of the access bridge

5.2. Demolition of the old gate

In order to allow the demolition of the old gate a stoplogs system was positioned in front of the opening. This system it was chosen to avoid losing hydroelectric production from the Cimena power plant which has a total power of 22 MW.

The stoplogs were designed in three different sections to allow them to pass along the crane track due to limited vertical space available: every section of the stoplog system has a height of 1.30 m compared with the available space between the bottom and the lower edge of the track crane equal to 1.40 m.

After the positioning of stoplogs it began the complete removal of the old gate and the bottom and lateral sills; this operation lasted approximately two weeks: the concrete demolitions concerned only the fixed sills, avoiding the removal of the sector gate pintles embedded in the concrete piles of the weir.

In fact, after having assessed the discrete state of these elements, it was decided, in agreement with the construction company, to reuse them, avoiding demolitions which would have been difficult.

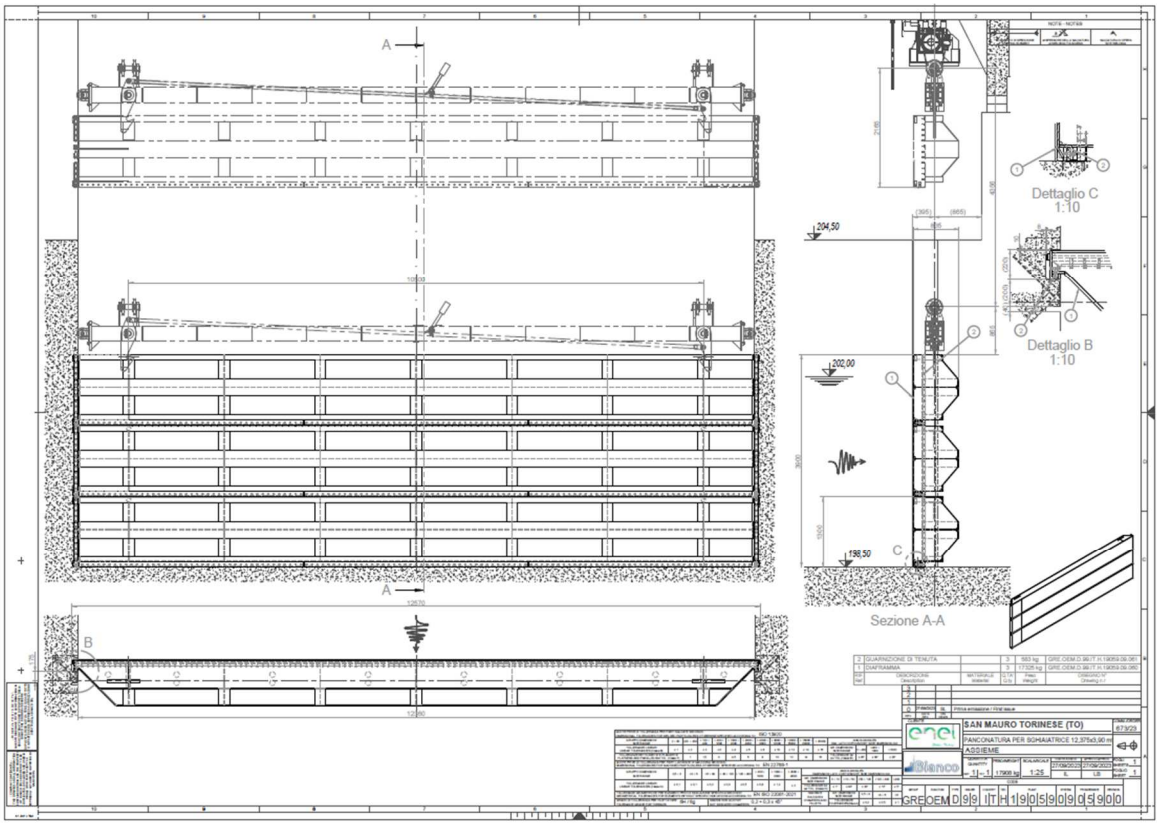


Fig. 8 – Project of the stoplog and demolition of the old gate

5.3. Installation of the new gate

The installation of the new gate also involved the modification of the lifting system, which went from the chain scheme to the one with a hydraulic piston in order to guarantee easier execution of maintenance activities, as this system does not have normally wet components. The activity lasted about two months and two new blocks of concrete anchoring of the pistons were built.

Another problem to be faced was linked to the transport of the gate to the installation site, in fact the length of over 12 meters and the overall weight of approximately 21 t [3] made this activity complex due to the very limited spaces.

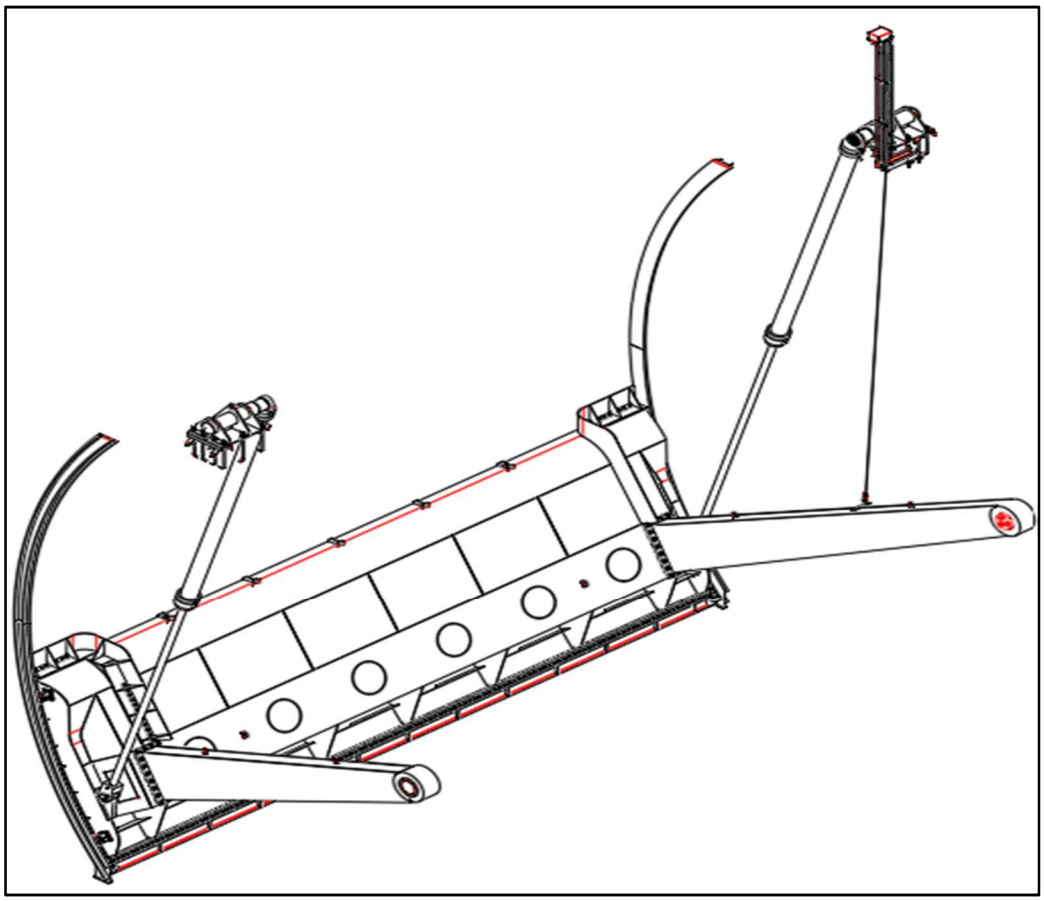


Fig. 9 – Project drawing of the new gate

5.4. Hydraulic test of the gate

After the end of the installation of the gate was performed a hydraulic test in order to verify the behaviour of the new structure.

The test has been constituted by these controls:

- Visual inspection of the control equipment in the control room.
- Checks, controls and tests indicated in the data survey tables.
- Reading, during movements of the gate, of some values such as:
    - operating pressures at the various points of the hydraulic circuit.
    - voltages and currents of the electrical system.
    - operating times.
- Calibration of equipment whose measured values differ from the nominal ones indicated in the data survey tables.
- Checking the correct functionality of position transmitter and receiver devices, etc.
- Possible calibration and fine-tuning of the same.
- Checking the regularity of movement of mechanical parts.

- Checking the state of conservation of protective coatings such as paint, galvanized parts, etc.
- Checking the oil level in the tank and checking the hydraulic seals.

The test was performed on June 20th, 2024, with a positive outcome: the maximum value of oil pressure during the opening of the gate was 70 bar (nominal level < 105 bar).

Fig. 10 – Project drawing of the new gate

6. CONCLUSIONS

After successfully completing the first year of work, the four remaining gates will be replaced over the next two winter seasons.

The experience gained during the first season will help us avoid some of the issues encountered on-site, particularly those related to placing the stoplogs in water undoubtedly one of the most critical activities.

To mitigate risks during the lowering of the stoplogs, it was decided to position them by reducing the upstream water level to its minimum, thereby taking the weir out of service during this activity (three days of out of service for every gate installation).

REFERENCES

[1] AIPO (2016). Piano di gestione del rischio alluvioni, Parma.

[2] Cotroneo. A (2020). HEC-RAS Moto Vario: Manuale sull'utilizzo del software HEC-RAS, Reggio Calabria.

[3] Erbisti, P.C.F. (2017). Design of Hydraulic Gates, CRC Press.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

ANALYSIS OF THE INFLUENCE OF MANNING'S COEFFICIENT IN DAM FAILURE FLOOD SIMULATIONS USING REMOTE SENSING DATA

Sérgio Ricardo Toledo Salgado
Faculty of Engineering of the University of Porto (FEUP),

Elsa Maria Carvalho[2]
Faculty of Engineering of the University of Porto (FEUP),

Maria Teresa Viseu[3]
National Laboratory of Civil Engineering

Othon Fialho de Oliveira[4]
National Water and Sanitation Agency

ABSTRACT

Dam failures pose significant risks to both life and infrastructure, making precise flood simulations essential to mitigate their impacts. This study investigates the influence of Manning's roughness coefficient on dam failure flood simulations, utilizing three freely accessible high-resolution Land Use and Land Cover (LULC) datasets: MapBiomas, ESRI Annual Land Cover, and Dynamic World, all processed via Google Earth Engine. The analysis encompasses two dams located in the semi-arid region of Pernambuco, Brazil, using the HEC-RAS 6.5 model to explore variations in key hydraulic parameters such as peak flow, flood propagation time, and inundated areas. Additionally, the study emphasizes the importance of accurately defining the Self-Rescue Zone (SRZ), a critical emergency response area defined within 30 minutes of flood wave propagation, typically as required by several dam safety legislations. Differences in SRZ dimensions were analysed based on the LULC datasets, revealing significant variations linked to Manning's coefficient. The results highlight the potential of LULC data in failure simulations, especially in data-deprived regions. However, they reinforce the substantial impact of LULC datasets on hydraulic modelling results, which may influence emergency planning.

Keywords: LULC datasets, Manning's coefficient, dam failure, LULC datasets, Self-Rescue Zone

.

[1]up202001034@up.pt, Brazil

1. INTRODUCTION

Dam failures are technological disasters that can be catastrophic, posing serious threats to life and infrastructure. Accurate prediction of flood scenarios is crucial to mitigate these risks, and numerical models, especially two-dimensional (2D) simulations, have become indispensable tools. However, the calibration of roughness coefficients, particularly Manning's coefficient, remains a significant challenge due to the scarcity of historical data on extreme events [1]. This limitation is even more pronounced in dam failure studies, where the lack of comparable historical events significantly hinders the calibration process [2, 3].

Remote sensing data, such as Land Use and Land Cover (LULC) datasets, offer a promising alternative by providing detailed information on land use and surface cover. This study utilizes three high-resolution and freely accessible LULC datasets: MapBiomas [4], ESRI Annual Land Cover[5], and Dynamic World [6], all obtained through the Google Earth Engine platform (GEE).

Additionally, the study emphasizes the importance of accurately defining the Self-Rescue Zone (SRZ), a crucial region for emergency evacuations during the critical first minutes following a dam failure, as outlined in the legislation of Brazil and Portugal [7, 8]. The definition of the SRZ largely depends on the propagation characteristics of flood waves, which are influenced by the topography of downstream valleys and Manning's coefficient.

Two case studies were undertaken, comprising two dams with different sizes, both located in the semi-arid region of Pernambuco, Brazil. The HEC-RAS 6.5 model was used for these simulations, which focused on understanding the impact of varying Manning coefficients. The findings of this study aim to advance hydraulic modelling practices, strengthen emergency planning, and improve the accuracy of attenuation forecasts.

2. METHODOLOGY

2.1. Case Study Overview

This case study focuses on two dams situated in Brazil's semi-arid region, specifically in the state of Pernambuco. The area is defined by its semi-arid climate, which is characterised by sparse and irregular rainfall. Furthermore, the rivers downstream of the dams are intermittent, with flow occurring only during periods of rainfall. The geographic location of the two dams selected for the case study is presented in figure 1.

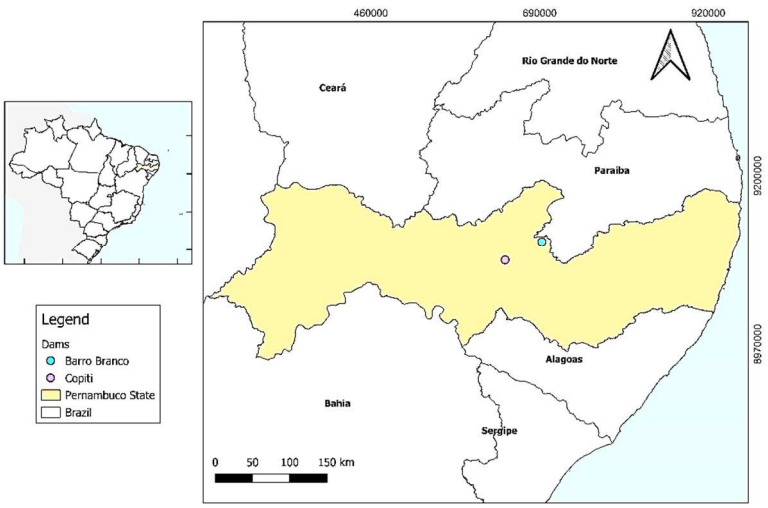


Fig. 1: Case Studies Localization Map

Table 1 presents information regarding the dams and, in addition, their classification according to the ICOLD criterion. This classification is determined by the values of the maximum height of the dam (H) and the maximum accumulated volume (V) [9].

Table 1 – Dams selected for the case study

Dam	Height (m)	Reservoir volume (hm³)	$H^2\sqrt{V}$	Size
Copiti	17.22	6.33	746.05	Large
Barro Branco	12.56	0.27	81.97	Small

The Pernambuco region has a digital terrain model ranging from 0.5 to 1 meter, the PE3D model [10]. This accurate representation is used in urban planning, environmental management and water resource studies. [11].

2.2. Land Use and Land Cover

This study employed three high-resolution LULC datasets: Dynamic World (DW), MapBiomas 10m Collection BETA (MB), and ESRI 10m Annual Land Cover (ERSI). These datasets were chosen for their high spatial resolution and comprehensive temporal coverage, leveraging Sentinel-2 imagery to map land use and land cover at a 10-meter resolution.

MapBiomas is a dataset tailored to the Brazilian region, offering detailed annual maps for the period of 2016–2022 [4]. In contrast, ESRI provides global data from 2017–2023, generated through deep learning techniques applied to billions of labeled pixels [5]. Dynamic World stands out for its near-real-time updates (every 2–5 days), providing probabilistic labels for nine land cover classes, enhanced by cloud and shadow removal techniques [6].

A script was constructed on the Google Earth Engine platform to enable the retrieval and downloading of LULC data corresponding to the region of interest for each case study. Figure 2 presents an overview of the process for obtaining the datasets.

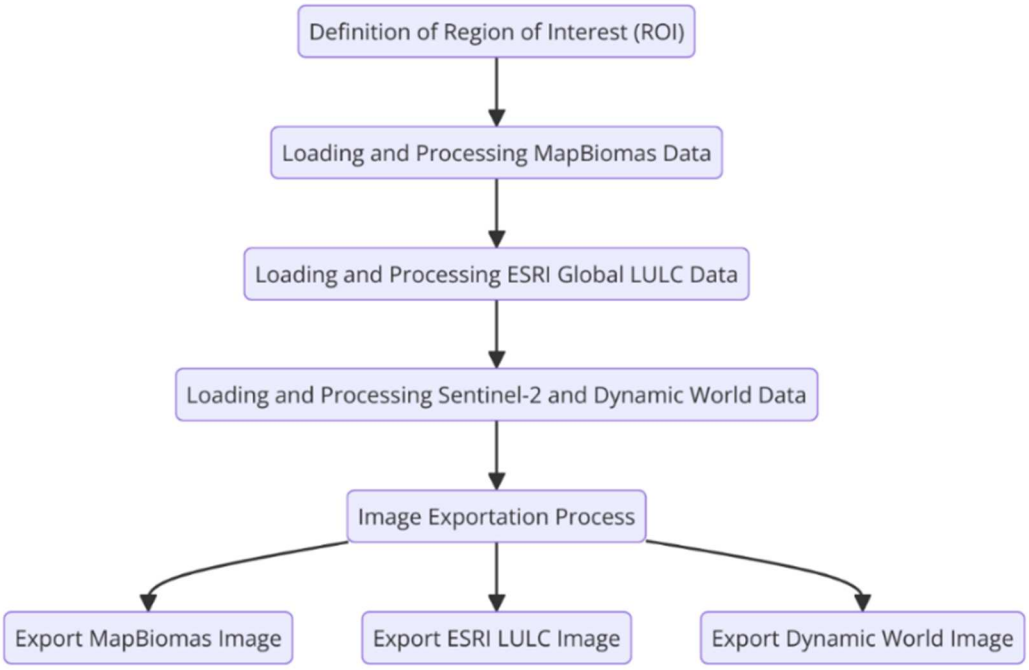


Fig. 2- Flowchart of processing Land Use and Land Cover (LULC) data in Google Earth Engine.

For the study, the 2022 MapBiomas and 2023 ESRI 10m Annual Land Cover datasets were used. Meanwhile, for Dynamic World, due to its cloud-related limitations, images from 11/2023 for Copiti valley and 07/2023 for Barro Branco valley were utilized.

The LULC data were employed as the foundation for determining Manning's roughness coefficient values, with consideration of the reference values presented in [12]. However, differences in the interpretation and classification of land cover classes, as well as for the selection of Manning value ranges, have been observed to result in varying Manning coefficients, even when analysing the same area.

In the study areas, Manning's roughness coefficient was predominantly observed to range between 0.04 and 0.08 for the MapBiomas dataset, around 0.08 for the Dynamic World dataset, and approximately 0.04 for the ESRI 10m Annual Land Cover dataset. Figure 3 illustrates these conclusions with the example of the Copiti dam.

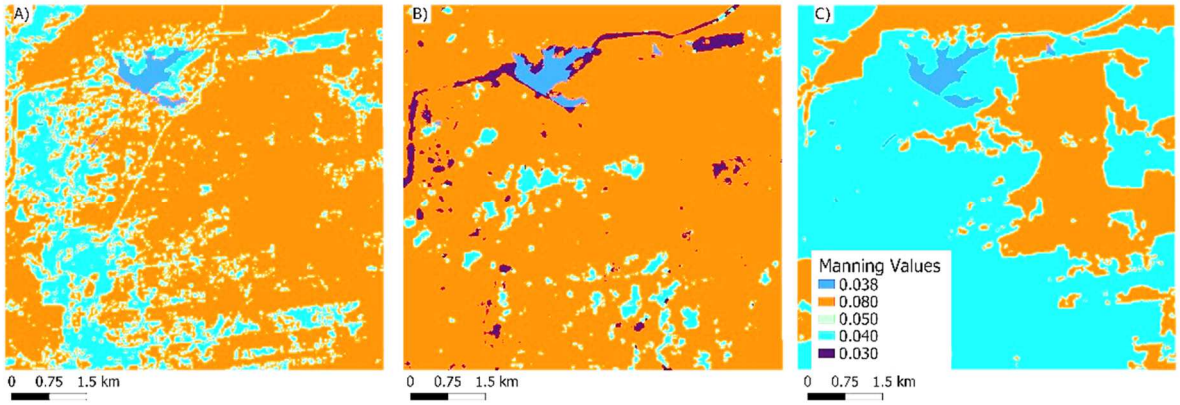


Fig. 3: Maps of Manning's roughness coefficients for Copiti Dam areas: A) MapBiomas 10m Collection BETA, B) Dynamic World, and C) ESRI 10m Annual Land Cover.

2.3. Dam Break Analysis

In the HEC-RAS model, the two-dimensional unsteady flow is solved using the mass conservation equation and the Saint-Venant momentum conservation equations along the x and y axes [13].

HEC-RAS 2D permits the specification of a default Manning's value for the entire 2D flow area and allows the designation of a Manning roughness coefficient via a Land Cover layer in RAS Mapper.

Each dam was evaluated under the assumption of a piping failure scenario, and the breach parameters were calculated using Froehlich's equations (1995), which are incorporated into HEC-RAS. The formulas used are as follows [14]:

$$t_f = 0{,}00254 V_w^{0{,}53} h_b^{-0{,}9} \quad (1)$$

$$B_{ave} = 0{,}1803 k_0 V_w^{0{,}32} h_b^{0{,}19} \quad (2)$$

where $\boldsymbol{B_{ave}}$ is the average breach width (m), $\boldsymbol{t_f}$ is the breach formation time (hours), $\boldsymbol{k_0}$ is a constant (1,4-overtopping or 1,0-Piping), V_w is the volume of water in the reservoir (m³) and h_b is height of water above the breach (m).

A simulation was conducted under identical conditions and consistent parameters for each dataset. For each LULC dataset, a dam-break simulation was performed to evaluate how different LULC configurations affect critical hydraulic parameters. The analysis focused on key hydraulic variables, including peak flow ($\boldsymbol{Q_P}$), time to peak ($\boldsymbol{t_P}$), depth ($\boldsymbol{h_P}$), and flood area.

Flood propagation time is an important variable for planning emergency response, ensuring timely alerting of vulnerable populations and defining necessary actions. In fact, the estimated distance covered by the flood within 30 minutes defines the Self-Rescue Zone (SRZ), a

reference standard in countries like Portugal and Brazil, where rapid individual action is required.

Accordingly, the variations in distances travelled within a 30-minute period were analysed for the dams in the case study, with a specific focus on the influence of different LULC datasets.

3. RESULTS AND DISCUSSION

3.1. Flood propagation

Figure4 illustrate the attenuation of flood discharge along the downstream valley for the Barro Branco and Copiti dams. The propagation of the hydrograph is related to the size of the dam, as simulations with Barro Branco show a rapid attenuation compared to Copiti, regardless of the LULC dataset used.

Simulations employing Manning's coefficients derived from the ESRI LULC dataset, noted for its lower roughness values, consistently produced the highest peak flows. In contrast, those using the Dynamic World LULC dataset, characterized by higher roughness, yielded significantly lower peak flows. Meanwhile, simulations based on the MapBiomas data presented results that were intermediate when compared to the aforementioned datasets, highlighting the varying impacts of land use and cover characteristics on hydrological responses.

For the Barro Branco dam, the flow rates estimated within a 5 km distance were 289.1 m^3/s, 254.8 m^3/s, and 206.8 m^3/s, respectively, for simulations using the ESRI, MapBiomas, and Dynamic World datasets. These results indicate a 13.5% difference between MapBiomas and ESRI and a -19.0% difference between MapBiomas and Dynamic World simulations. At a 20 km, the difference between MapBiomas and ESRI is 12.7%, while the discrepancy between MapBiomas and Dynamic World increases to -25.1%.

For the Copiti Dam, the flow rates at a 5 km distance are approximately 1757.8 m^3/s, 1705.4 m^3/s, and 1617.64 m^3/s for simulations using the ESRI, MapBiomas, and Dynamic World datasets, respectively. These results indicate a difference of 3.1% between MapBiomas and ESRI, and -5.1% between MapBiomas and Dynamic World.

At a 20 km distance, the simulation using ESRI shows a peak flow 11.5% higher, while the peak flow estimated using Dynamic World shows a difference of -31.1% compared to MapBiomas.

This variation can be explained by the rupture hydrograph, which was influenced by the size of the dams, resulting in lower flows for Barro Branco compared to Copiti. Additionally, the

different distribution patterns of Manning's coefficient (n) values also played an important role in this explanation.

Regarding Manning's coefficient, the average value in Dynamic World was approximately 0.08 in both analyzed regions. However, the mean values of n, considering the flooded area, were 0.06 in MapBiomas and 0.05 in ESRI 10m Annual for Barro Branco, while for Copiti, these values were 0.05 in MapBiomas and 0.04 in ESRI 10m Annual. Therefore, the distribution of Manning's coefficients in MapBiomas and ESRI 10m Annual in Copiti resulted in lower resistance to hydrograph propagation compared to Barro Branco. On the other hand, the Dynamic World simulations showed practically the same Manning's coefficient values in both areas.

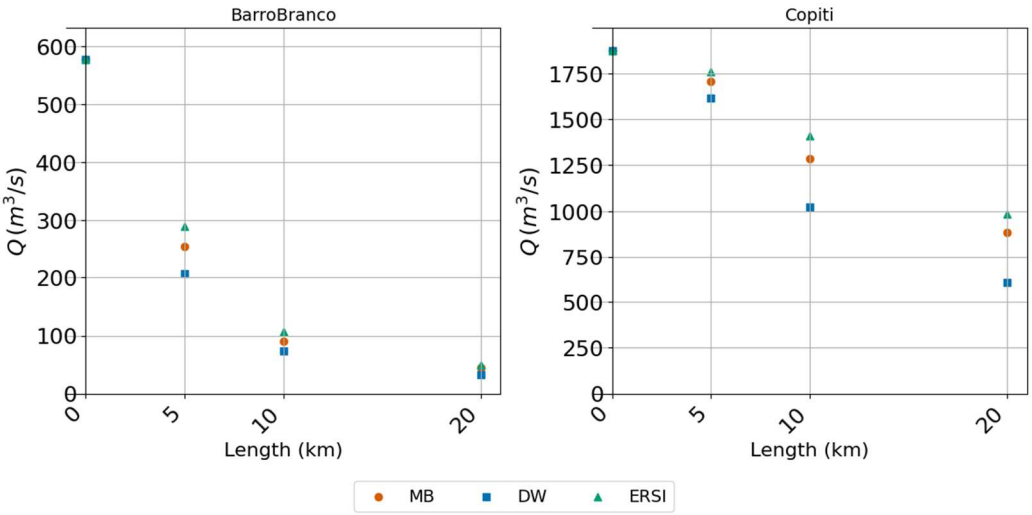


Fig. 4- Propagation of hydrographs at different distances and LULC data sets for the Barro Branco and Copiti dams - (MapBiomas (MB), Dynamic World (DW) and ESRI 10m Annual Land Cover (ERSI)

Figures 5 and 6 present the stage hydrographs of the dams under analysis and allow the evaluation of Manning's coefficient influence on the simulation results in terms of wave propagation time and depth.

The propagation of the flood wave was observed to occur at varying velocity in the simulations conducted using the three different datasets. The propagation was found to be slower in the simulations utilising the Dynamic World dataset, while the simulations employing the ESRI dataset indicated a faster propagation of the wave. The MapBiomas dataset produced intermediate results, falling between those observed for Dynamic World and ESRI.

The impact of Manning's values is evident in the comprehensive analysis conducted. The simulations utilising the Dynamic World set, which represents terrain with heightened resistance, demonstrate the lowest flow rates but record-high depth values, for instance, at 5 km in both dams. Additionally, they exhibit the second-highest value at 10 km in both dams.

However, the simulations with the ESRI dataset showed shallower depths, with the highest value occurring only at 20 km downstream of the Barro Branco dam. At this point, the distribution of Manning's values across the analyzed LULCs was very similar. The simulations using the MapBiomas dataset resulted in intermediate depth values at 5 km for both dams and at 20 km for the Barro Branco dam, but recorded the greatest depth at 10 km for both dams.

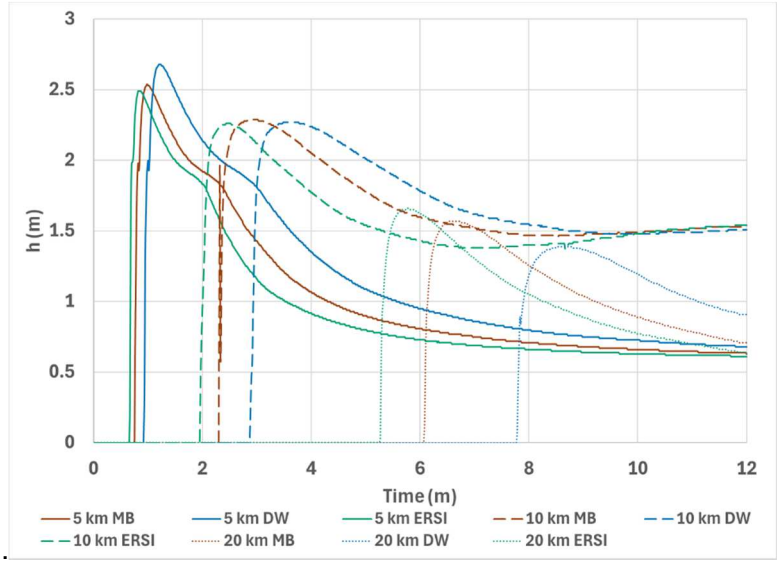


Fig. 5- Barro Branco dam stage hydrograph

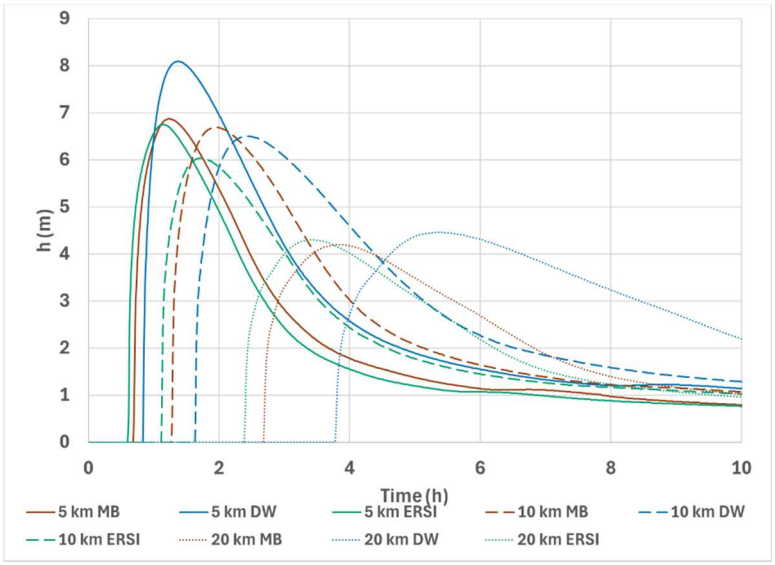


Fig. 6- Copiti dam stage hydrograph

The results indicate that flood propagation time could be a more critical factor than water depth when assessing the impact of roughness estimates in hydrodynamic modeling. While water depth is influenced by both flow rate and channel roughness, variations in roughness can significantly affect the extent of flooded areas.

These variations are primarily driven by differences in soil resistance, with simulations based on the Dynamic World dataset—which represents higher resistance—producing the largest flooded areas, followed by MapBiomas and ESRI, as shown in Table 2.

For the Barro Branco dam, the flooded area estimated by Dynamic World was 360.93 ha, which is 1.11% larger than the area simulated using MapBiomas and 1.61% larger than the area simulated using the ESRI dataset.

Similarly, for the Copiti dam, the flooded area estimated by Dynamic World was 885.27 ha, representing a 7.76% increase compared to the MapBiomas estimate and a 10.47% increase compared to the ESRI estimate.

Table 2 – Flooded Areas for Barro Branco and Copiti Dams Based on Different LULC Datasets

LULC Dataset	Barro Branco	Copiti
	Flood Area (ha)	Flood Area (ha)
Mapbiomas	356.93	816.57
Dynamic World	360.93	885.27
ERSI	355.13	792.58

3.2. Self-Rescue Zone

The analysis of simulations with different LULC datasets revealed significant differences in SRZ estimates for the dams, figure 7. The ESRI 10m Annual dataset consistently presented the highest SRZ values for both dams. On the other hand, the Dynamic World dataset produced the lowest values for both dams.

The MapBiomas dataset presented intermediate SRZ values for both dams, reflecting an intermediate response in terms of distance compared to the other LULC datasets. Figure 6 highlights the variability in SRZ distances for the Barro Branco and Copiti dams as a function of the different LULC datasets analyzed.

The differences in SRZ estimates among the LULC datasets for the Barro Branco dam are significant. The Dynamic World predicts a 16.4% reduction in the extent of the saturated zone (SRZ) compared to MapBiomas. On the other hand, the ESRI dataset shows the largest SRZ extent, with an increase of 10.4% compared to MapBiomas.

For the Copiti dam, the differences among the datasets are also notable. Dynamic World predicts the smallest SRZ extent, with an 18.3% reduction compared to MapBiomas. In contrast, the ESRI dataset estimates the largest SRZ extent, representing a 16.6% increase compared to MapBiomas.

The Copiti dam exhibited lower SRZ values across all datasets compared to the Barro Branco dam. This disparity can be attributed to the breach characteristics of the dams. Specifically, dams with faster failures, characterized by shorter breach formation times, result in a quicker onset of hydrograph propagation.

The discrepancy between the calculated distances for the SRZ illustrates the significance of establishing manning coefficient values and formulating an effective emergency plan. In the Dynamic World simulation, when manning values are higher and propagation is slower, the SRZ may appear to be undersized. However, simulations with faster propagation result in a greater distance from the SRZ, which, in theory, may enhance safety but may necessitate a considerable investment in rapid evacuation strategies.

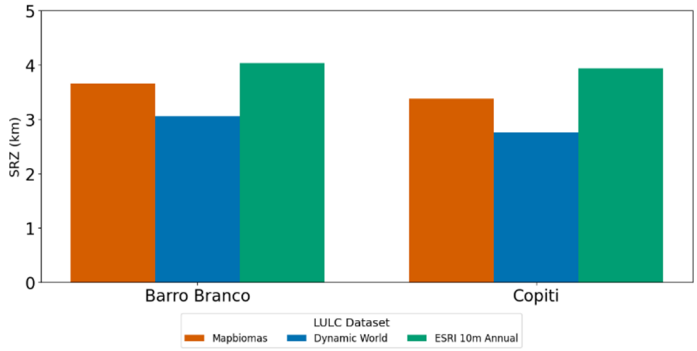


Fig. 7 – SRZ Estimates for dams using different LULC dataset

4. CONCLUSIONS

The results demonstrate that the employment of LULC datasets can serve as a valuable informational resource for the determination of Manning's coefficient in the hydrodynamic modeling of dam failures, particularly in regions where data is scarce. However, the differences observed between the LULC data indicate that they exert a considerable influence on estimates of critical hydraulic parameters, including peak flow, flood wave propagation time, flooded areas, and the delineation of the SRZ.

The analyses showed that the estimated flood areas varied significantly between the different LULC datasets, highlighting the direct effect of terrain roughness on modelling results. Simulations based on the Dynamic World set with higher Manning coefficients resulted in larger flood areas, potentially increasing the magnitude of impacts and the need for planning and mitigation.

However, the ESRI set, despite estimating higher flows due to the lower roughness coefficients, produced more restricted flood areas which may underestimate the real impact in certain scenarios.

In the case of the SRZ, higher Manning's coefficients, such as those observed in Dynamic World, resulted in smaller SRZ extensions, which could lead to under sizing of this area and compromise the effectiveness of emergency response. In contrast, the ESRI set resulted in larger SRZ extensions, which may increase safety, but could lead to higher costs in implementing evacuation strategies. These results reinforce the need to balance safety and operational costs in emergency planning. The choice of the LULC dataset also had an impact on the depth of the flows. Regions with greater roughness showed higher depths, even in low flow scenarios, showing that channel roughness plays a crucial role in flow dynamics. The use of LULC datasets from MapBiomas showed intermediate results in terms of flood areas, peak flows and SRZ delineation. This characteristic is directly related to the influence of the assumed Manning values derived from the detailed regional soil classification. These results may indicate a preference for regional LULC data, such as MapBiomas, whenever available. In this sense, the definition of the Manning coefficient is not merely a technical data point for models; it constitutes an indispensable element for public safety. Subsequent studies should prioritize refining LULC databases and their influence on downstream damage, including the population at risk, the affected infrastructure, and additional considerations.

REFERENCES

[1] V. Merwade, F. Olivera, M. Arabi, and S. Edleman, "Uncertainty in Flood Inundation Mapping: Current Issues and Future Directions," Journal of Hydrologic Engineering, vol. 13, no. 7, pp. 608-620, 2008, doi: doi:10.1061/(ASCE)1084-0699(2008)13:7(608).

[2] A. Bharath, A. V. Shivapur, C. G. Hiremath, and R. Maddamsetty, "Dam break analysis using HEC-RAS and HEC-GeoRAS: A case study of Hidkal dam, Karnataka state, India," Environmental Challenges, vol. 5, p. 100401, 2021/12/01/ 2021, doi: https://doi.org/10.1016/j.envc.2021.100401.

[3] I. M. Almeida, H. A. Santos, O. de Vasconcelos Costa, and V. B. Graciano, "Uncertainty reduction in flood areas by probabilistic analyses of land use/cover in models of two-dimensional hydrodynamic model of dam-break," Stochastic Environmental Research and Risk Assessment, vol. 38, no. 4, pp. 1335-1350, 2024/04/01 2024, doi: 10.1007/s00477-023-02635-6.

[4] MapBiomas Project. "Beta Collection of the Annual Coverage and Land Use Maps Series of the Brazil using Sentinel 2 images." https://brasil.mapbiomas.org/mapbiomas-cobertura-10m/ (accessed 01/07/2024, 2024).

[5] K. Karra, C. Kontgis, Z. Statman-Weil, J. C. Mazzariello, M. Mathis, and S. P. Brumby, "Global land use / land cover with Sentinel 2 and deep learning," in 2021 IEEE International Geoscience and Remote Sensing Symposium IGARSS, 11-16 July 2021 2021, pp. 4704-4707, doi: 10.1109/IGARSS47720.2021.9553499.

[6] C. F. Brown et al., "Dynamic World, Near real-time global 10 m land use land cover mapping," Scientific Data, vol. 9, no. 1, p. 251, 2022/06/09 2022, doi: 10.1038/s41597-022-01307-4.

[7] BRASIL, "Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens destinadas à acumulação de água para quaisquer usos, à disposição final ou temporária de rejeitos e à acumulação de resíduos industriais, cria o Sistema Nacional de Informações sobre Segurança de Barragens e altera a redação do art. 35 da Lei no 9.433, de 8 de janeiro de 1997, e do art. 4º da Lei no 9.984, de 17 de julho de 2000.," Lei nº 12.334, vol. 12.334, 2010.

[8] PORTUGAL, "Decreto-lei nº. 21, de 28 de março de 2018. Altera o regulamento de segurança de barragens e aprova o regulamento de pequenas barragens.," 1ª serie, 2018. [Online]. Available: https://dre.pt/dre/detalhe/decreto-lei/21-2018-114937037.

[9] ICOLD, "Bulletin 157: Small dams: design, surveillance and rehabilitation," in "Bulletin 157," International Committee on Large Dams, Paris, 2011.

[10] PERNAMBUCO. "Pernambuco tridimensional." www.pe3d.pe.gov.br (accessed.

[11] ICOLD, "Small dams: design, surveillance and rehabilitation," ed: International Committee on Large Dams, 2011, p. 149.

[12] T. V. Chow, Open-channel Hydraulics. McGraw-Hill, 1959.

[13] USACE, "HEC-RAS Hydraulic Reference Manual. March 2023.," United States Army Corps of Engineers 2023.

[14] D. C. Froehlich, "Embankment dam breach parameters revisited," International Water Resources Engineering Conference - Proceedings, vol. 1, 01/01 1995.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

ONTIME MODELING OF THE 14 DE JULHO PARTIAL DAM-BREAK IN SOUTH OF BRAZIL

Giorgia Moecke[1]
Fractal Engenharia

Pedro Guilherme de Lara[2]
Fractal Engenharia

Felipe Diniz[3]
Fractal Engenharia

ABSTRACT

The partial breach of the 14 de Julho Dam, located on the Taquari-Antas River in southern Brazil, occurred during the extreme hydrological event of May 2024 in Rio Grande do Sul. At the time of the event, the Taquari-Antas River section, which spans about 250 km along the entire Ceran cascade, was equipped with a calibrated and fast-executing one-dimensional HEC-RAS model, used for modeling as part of the Emergency Action Plan (EAP) for the structures. When the breach at the crest of the 14 de Julho Dam was detected, the model was used to reproduce the event in real-time, estimating and assessing potential impacts downstream, such as arrival time, peak outflow, maximum water level, and the maximum incremental height of the dam-break wave. The real-time information generated was crucial for protecting the population and guiding civil defense actions. After the event, the results generated by the model were compared with data from downstream fluviometric stations, where the maximum differences between the modeled and measured incremental heights were up to 0,35 m, confirming the good accuracy of the modeling. This article discusses the model's conditions and highlights the importance of having operational hydrodynamic models for extreme situations like those experienced in Rio Grande do Sul in May 2024.

Keywords: Dam-break, Flood, Breach, EAP.

[1] giorgia@fractaleng.com.br, Brazil
[2] pedro@fractaleng.com.br, Brazil
[3] felipe.diniz@fractaleng.com.br, Brazil

1. INTRODUCTION

The south region of Brazil suffered from extreme rainfall events the lasted from April to May of 2024. The successive and intense rain was followed by severe flooding that caused cities to be evacuated and put some dams in high alert.

Among the impacted dams, there is the 14 de Julho Dam operated and owned by Ceran and located on the Taquari-Antas River, a 250 km long cascade in the state of Rio Grande do Sul. Along with a free surface spillway, the 14 de Julho Dam possesses an auxiliary spillway controlled by two gates. Due to the failure of both remote and local operation of the auxiliary spillway - caused by the loss of access to the dam during the flood - the reservoir water level began to rise. As a result, the dam was nearly overtopped and ended up suffering a partial dam-break at the crest of the free surface spillway, on May 2nd.

When the breach was confirmed, Ceran got in contact with Fractal Engineering company, who was already conducting studies for the entire cascade, to assess in real-time the potential damage of the dam breach even though the real breach parameters were still unknown. Thanks to a calibrated and fast-executing one-dimensional HEC-RAS model, used for modeling as part of the Emergency Action Plan (EAP), valuable and ontime informations were provided for the public defense organizations and the entrepreneur in order to accesses the potential damage and safeguard the population downstream.

Below it is presented the experience of Rio Grande do Sul in May 2024 with a real-time dam break modelling and how crucial it can be in decision making.

2. 14 DE JULHO DAM

The 14 de Julho Dam is located on the Taquari-Antas River, a 250 km long cascade in the state of Rio Grande do Sul in south of Brazil. It is owned and operated by Ceran company and is capable of generating a total of 100 MW of energy.

The dam is a RCC type with a maximum height of 33 m (El. 111,85 m) and it is 362 m wide. The reservoir has a stored volume of 58,40 hm^3 at regular operative level. The flood control is guaranteed by two spillways, one being free surface and the other gate controlled. The free surface spillway is 247 m wide and the controlled spillway is composed of two gates, each 16 m wide and 23,68 high. Together the spillways combine to a total discharge capacity of 18.114,00 m^3/s, equivalent to an 10.000 years return period flood.

The figure below illustrates the dam layout.

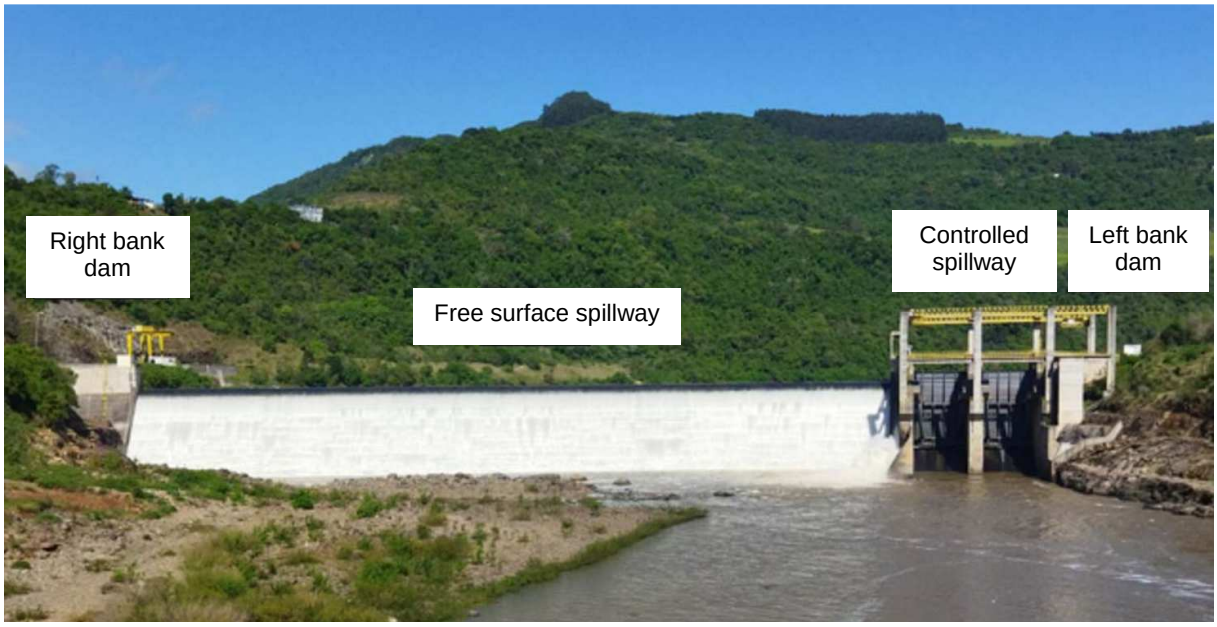


Fig. 1 –14 de Julho Dam.

3. PARTIAL BREACH EVENT

Between April 30 and May 2, 2024, the Taquari-Antas River basin, along with much of the state of Rio Grande do Sul, experienced the impacts of a historic, large-scale flood. At the 14 de Julho Dam, the situation worsened following the failure of the auxiliary spillway gate activation system, which remained partially open during the flood's peak. As a result, the reservoir level exceeded its Full Reservoir Level (FRL).

The combination of elevated water levels and potential structural deficiencies—currently under investigation by an independent team of consultants—led to the partial failure of the dam. This failure resulted in the formation of a breach in the remaining section of the main free-surface spillway, as reported by Ceran on May 2, 2024.

According to information provided by Ceran, the breach initiated at 13:32 on May 2, 2024, in the section where the spillway connects to the right overbank. The breach, measuring approximately 200 meters in width and 8 meters in depth, fully developed by 13:40 on the same day, completing the opening process in approximately 8 minutes.

At the moment of failure, the reservoir water level was at Elevation 111.61 meters, with an outflow of approximately 14,000 m^3/s.

Figure 2 illustrates the partial breach immediately after the flood event. Figure 3 presents the variation in water levels within the reservoir and downstream of the 14 de Julho Dam throughout the event. Figure 4 displays the inflow hydrograph at the dam.

Fig. 2 – Partial breach of 14 de Julho Dam.

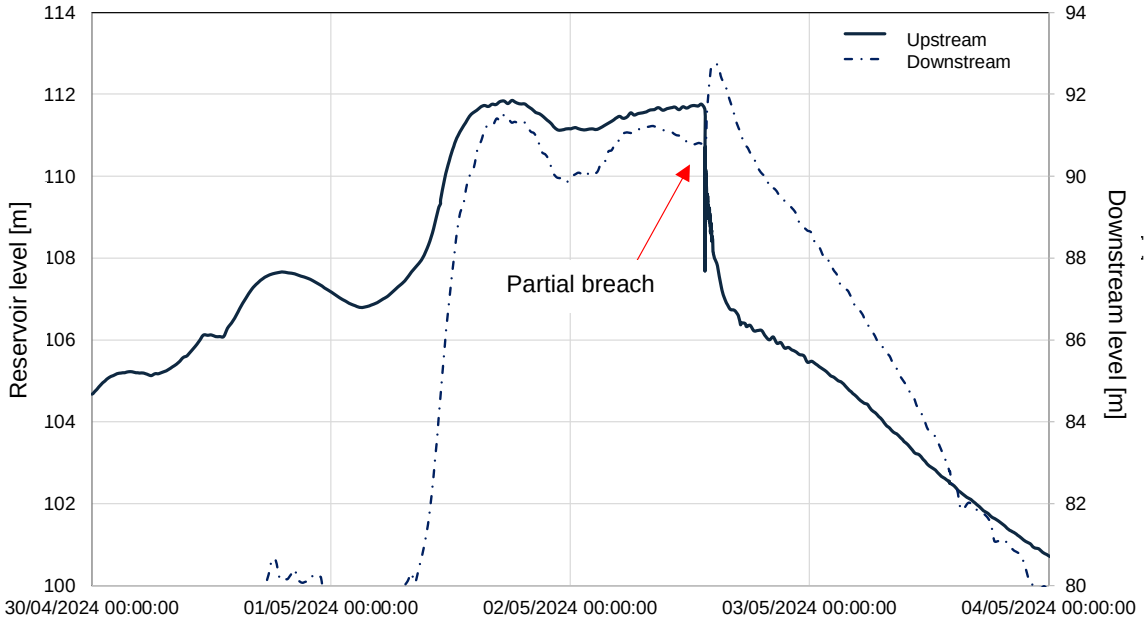


Fig. 3 – Water levels in the reservoir (upstream) and downstream of the 14 de Julho Dam during the event of May 2024.

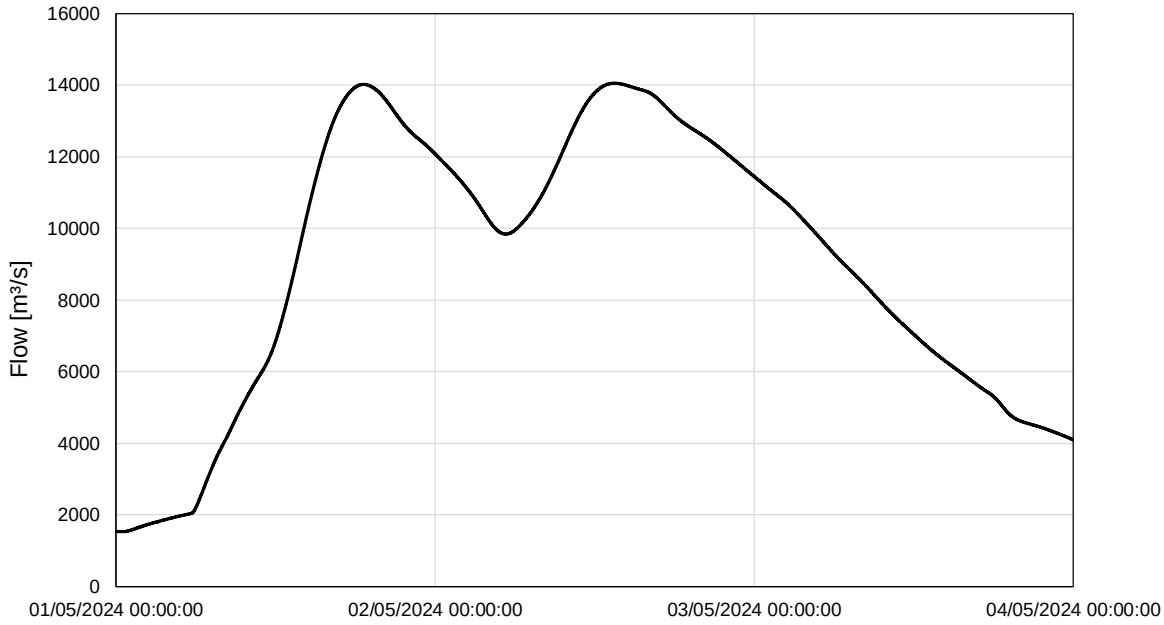


Fig. 4 – Inflow at 14 de Julho Dam during the event of May 2024.

After the partial breach of the dam, the water level in the region of the Power House (around 10 km downstream of the dam) rose by 1,95 m, with the recession starting around 4 h after the beginning of the event.

The failure was also recorded by two fluviometric stations located downstream of the dam, one located in the city of Santa Tereza (34 km downstream), Fig. 5, and another located in the city of Muçum (60 km downstream), Fig. 6.

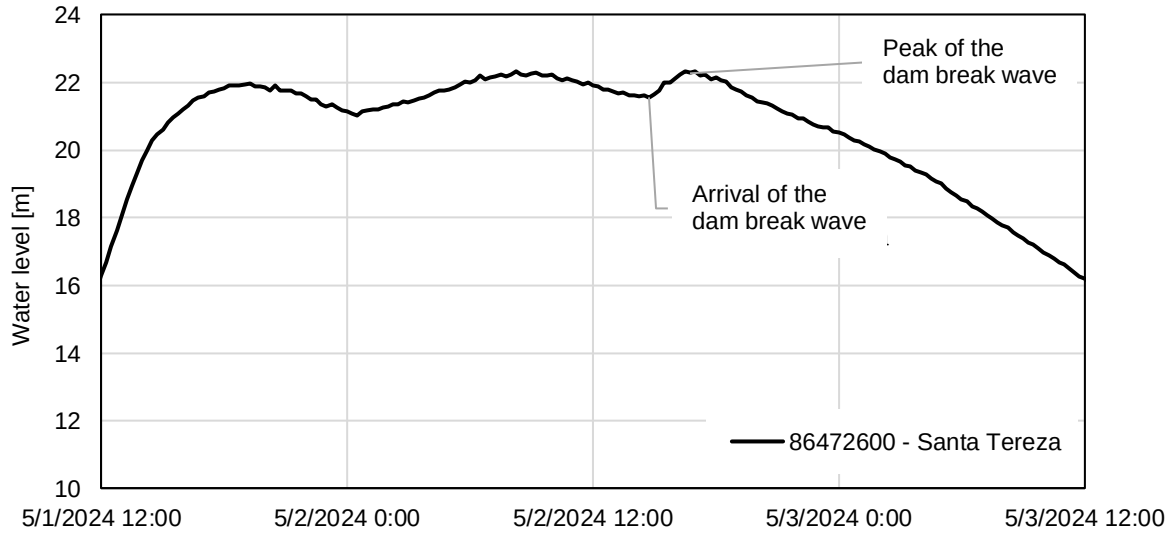


Fig. 5 – Water level measured in the fluviometric station of Santa Tereza city during the dam break event.

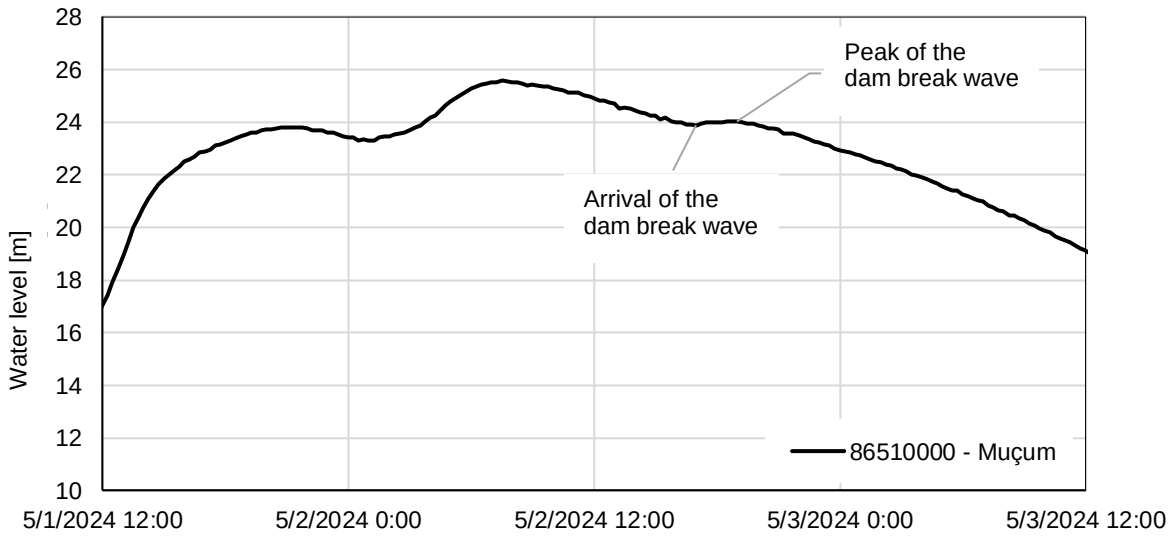


Fig. 6 – Water level measured in the fluviometric station of Muçum city during the dam break event.

Based on the data from the stations, the impact was most significant in Santa Tereza which recorded an incremental height of around 0,75 m in relation to the water level prior to the dam failure. Also, the dam break wave arriving at 14h45 (01h10 after the breach). In Muçum, the incremental height was around 0,15 m, with the dam break wave arriving around 17h (03h20 after the breach). At both fluviometric stations, the event lasted around 4 hours.

3.1. PARTIAL BREACH PARAMETERS

Based in the information available during the event, and reavaluated right after, the Table 1 summarizes the main breach parameters used in the event modelling that it is presented in the next section.

Table 1 – Partial breach parameters of 14 de Julho Dam

Position	Crest of free surface spillway
Failure mechanism	Slide / Partial collapse
Shape	Rectangular (200 m wide and 8 m high)
Development time	8 min
Reservoir water level at the breaching time	111,61 m (0,24 m below dam's crest)

4. DAM-BREAK MODELING

The hydraulic model was developed in HEC-RAS [1] and consists of approximately 250 km of the Taquari-Antas river, from the Castro Alves Dam, at the top of the cascade and also owned and operated by Ceran, to Taquari city near the confluence with the Jacuí river. Right from the 14 de Julho Dam to the end of the model, the distance is around 178 km. The model is a 1D model with a total of 1.577 cross sections, spaced at an average of 162 m from one another. All simulations were conducted under unsteady flow conditions. Due to the extensive length of the modeled reach, a one-dimensional (1D) approach was selected to optimize computational efficiency and reduce simulation time. A two-dimensional (2D) approach would not have generated results quickly enough to support emergency response efforts, such as those required during the May 2024 flood events, and would not necessarily have provided greater accuracy than a 1D model. Additionally, the Taquari-Antas River is predominantly steep and fast-flowing, with limited overbank flooding, except near its confluence with the Jacuí River. These characteristics confirm that a 1D model is appropriate for representing the river's hydraulic behavior.

The employed Digital Terrain Model (DTM) has a precision of 1:5.000, pixel resolution of 2x2 m and average altimetric error of 0,48 m. This DTM is 140 km long and it goes from the Castro Alves Dam to Muçum city located 60 km downstream from 14 de Julho Dam. To complete the remaining 110 km from Muçum city to Taquari city, at the end of the model, it was used a MERIT (Multi Error Removed Improved Terrain) obtained by Google Earth Engine.

The Manning's roughness coefficients were adopted according to bibliographic recommendations for the morphology of the region. The coefficients were defined as 0,100 for

the flood plains and between 0,045 and 0,075 for the riverbed. All adopted values are in line with Chow (1959) [2].

A calibration procedure was carried out to check the model's consistency in representing the flow processes along the Taquari-Antas River. To do this, the flood records of SEPT23 and NOV23 at the Powerhouse and dam were used. With the calibration, the average altimetric error for the spillway was 0,13 m and for the Powerhouse region it was 0,26 m.

Fig. 7 shows the profile of the Taquari-Antas River along the model's region, with an average slope of 0,080%. Fig. 8 illustrates the model's design.

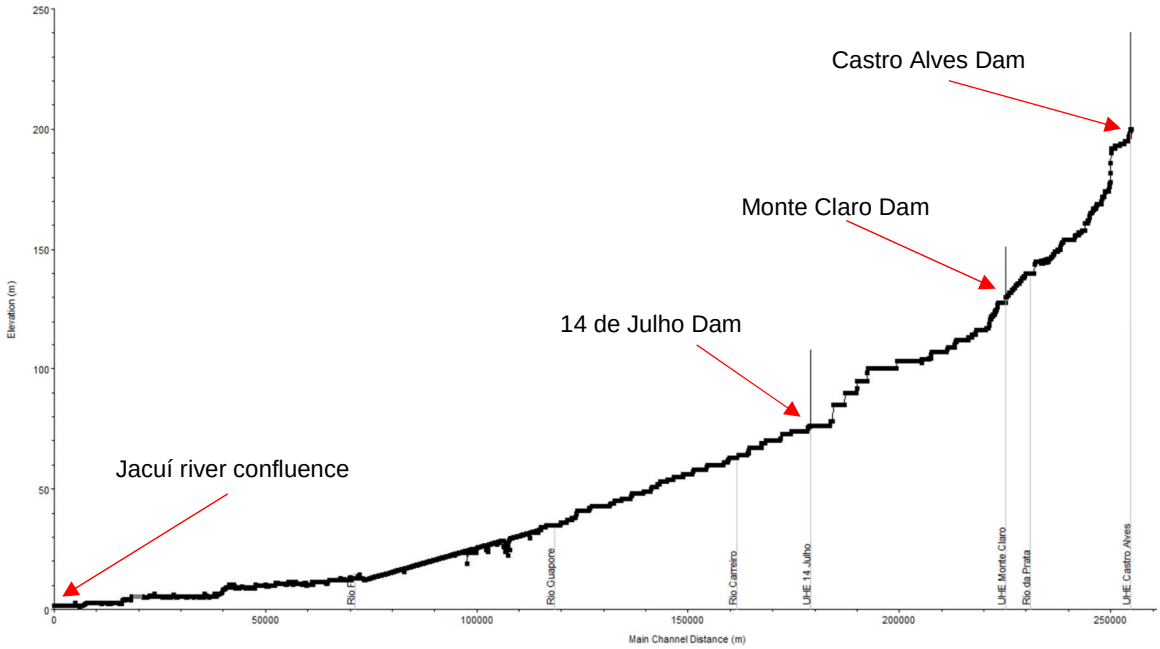


Fig. 7 – Profile of the Taquari-Antas River along the model's region.

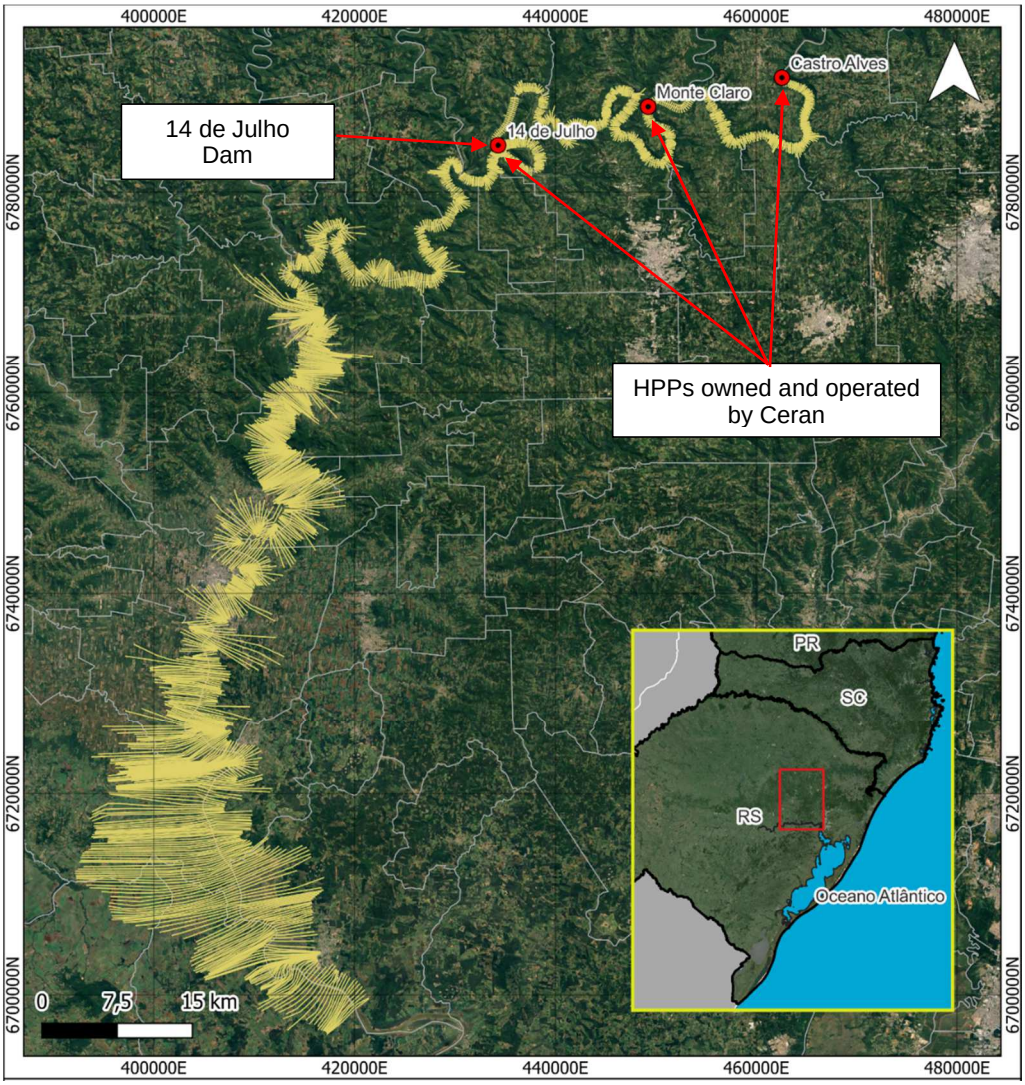


Fig. 8 – HEC-RAS 1D model design with its cross sections.

5. RESULTS

The following figures illustrate the behavior of the dam break wave along the downstream valley of 14 de Julho Dam for the event that occurred in May 2024. A hydrograph and a curve of the incremental height of the dam break wave are presented for each section of interest, each section representing the distance in meters measured from the dam. The base scenario for comparison and measurement of the incremental height was the extreme flood at the time of the event, but without the dam failure. This condition was modelled with this purpose.

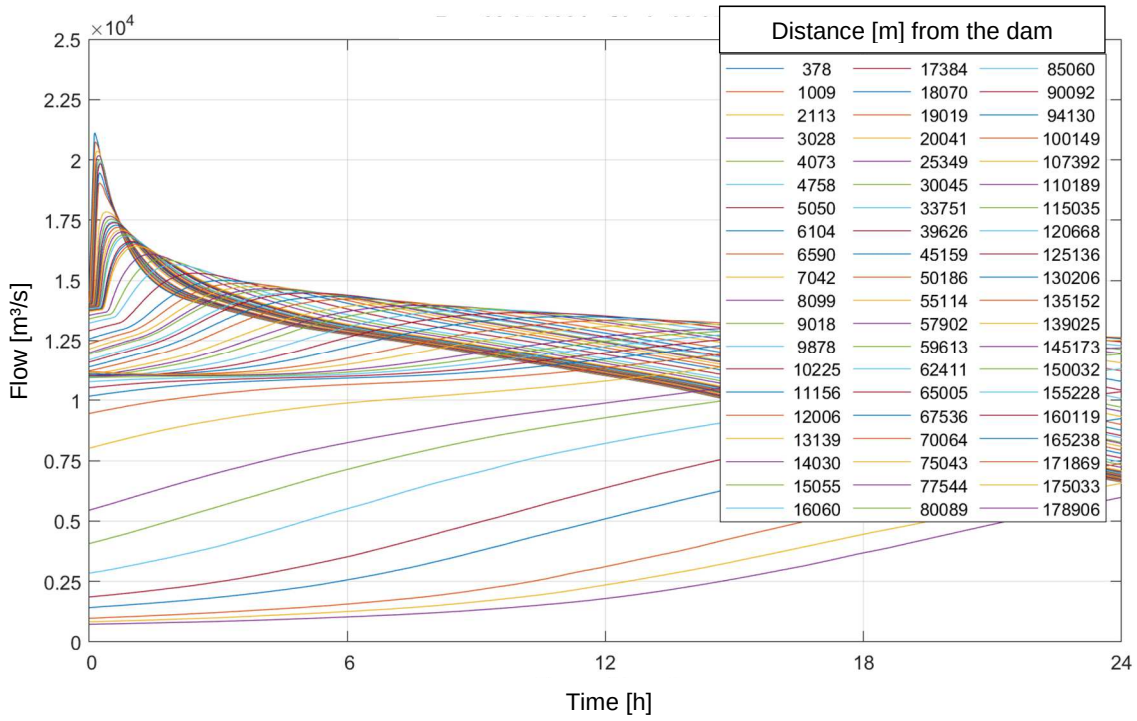


Fig. 9 – Hydrograph of the dam break wave along the downstream valley.

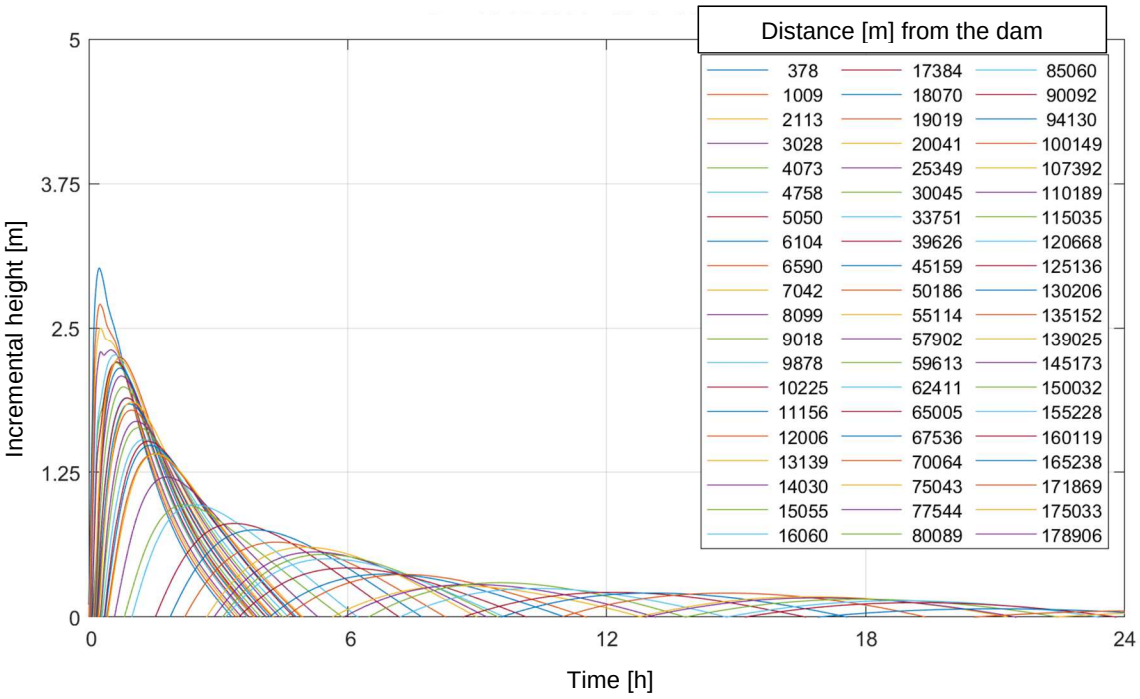


Fig. 10 – Incremental height of the dam break wave along the downstream valley.

According to the results, the incremental height of the dam break wave, difference between the water level with and without the dam failure, was around 3 m in the closest regions to the dam. In the region of the Powerhouse (around 10 km downstream of the dam), the incremental height was around 1,89 m, with an arrival time of 12 minutes. The dam break wave advanced 20 km in 30 min, reaching an incremental height of 1,40 m.

At the fluviometric stations in Santa Tereza city and Muçum city, the incremental heights were around 0,97 m and 0,50 m, respectively. The differences between the model's results and the fluviometric stations in terms of the wave's incremental effect were around 0,22 m for Santa Tereza and 0,35 m for Muçum. These differences are within the accuracy range of the DTMs and can be justified by the terrain variation, as well as the inaccuracy of the actual outflow from 14 de Julho Dam during the event. The wave arrival times at the fluviometric stations were consistent with those observed during the event, with 1 hour for Santa Tereza and 3,5 hours for Muçum.

At the end of the model, 178 km from 14 de Julho Dam and 15 km downstream from Taquari city, close to the Jacuí river confluence, the dam break wave arrived in around 29 h with an incremental height of 0,06 m – illustrating the complete dampening of the event at the end of the model.

6. CONCLUSIONS

Although the reservoir’s volume of 14 de Julho Dam can be considered large and, in a conventional total failure event, could generate a significant change in the Taquari-Antas River's fluviometric regime, the partial failure that occurred on May 2th of 2024 did not have such significant impacts along the downstream valley. This statement is also based on the information measured at the fluviometric stations of Taquari city and Muçum city, as well as the hydraulic model’s response of the event carried out for this study. Especially considering that prior to the partial failure, the downstream valley was already flooded due the heavy rainfall and extreme flows and the population was already evacuating the region.

The results presented here indicate that the model successfully captured the partial breach of the 14 de Julho Dam, providing a reliable basis for more detailed analyses to assess the potential damage associated with this failure event.

Furthermore, having a representative and computationally efficient hydraulic model is essential for delivering critical real-time information, enabling public defense agencies to respond more effectively and enhance population safety during emergency situations.

REFERENCES

[1] U. S. ARMY CORPS OF ENGINEERS. Hydrologic Engineering Center. Training Documents – Using HEC-RAS for Dam Break Studies. TD-39, USCE, August 2014.

[2] CHOW, V.T. (1959) – “Open-channel hydraulics”, New York, McGraw-Hill, 680 p.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DEVELOPMENT OF THE COMBINED FISH PASS OF FRATEL'S DAM USING NUMERICAL AND PHYSICAL MODELLING

José F. Melo
Laboratório Nacional de Engenharia Civil

Lourenço Sassetti Mendes[1]
Laboratório Nacional de Engenharia Civil

António Muralha
Laboratório Nacional de Engenharia Civil

Pedro Pinto
EDP

João Miguel Oliveira
EDP

Liliana Benites Carvalho
EDP

Pedro Raposo Almeida
MARE/Universidade de Évora

Bernardo Quintella
MARE/Universidade de Lisboa, Faculdade de Ciências

Carlos M. Alexandre
MARE/ Universidade de Évora

Ana Sofia Rato
MARE/Universidade de Évora

ABSTRACT

In 2025, EDP, Portugal's leading energy production and distribution company, will carry out intervention at the Fratel Hydroelectric Scheme on the Tagus River, to improve the power production efficiency and to provide migratory fish access to upstream habitats. The tailrace channel will be reprofiled, and a new fish pass system was designed, which is the focus of this study. The project aims to enable fish migration past the powerhouse afterbay sill, even during low water levels, and reach the future fish lock.

The fish pass system comprises three key elements: a 230-metre-long fish-dedicated channel, excavated along the tailrace, a four-pool fishway, and a bypass channel with irregular baffle walls.

A comprehensive study conducted at the National Civil Engineering Laboratory of Portugal (LNEC) employed computational fluid dynamics (CFD) and physical modelling to optimise the system proposed by EDP. Velocity fields and flow depths across varying operational scenarios were evaluated. A multidisciplinary team, comprising experts in hydraulic engineering, computational and physical modelling, and ecological science from LNEC, EDP, and MARE/University of Évora, worked in close collaboration, recommending design optimizations.

The large domain dimensions would imply an unfeasibly large physical model. Similarly, a single CFD model to accurately reproduce all geometries would be computationally prohibitive due to the range of time and length scales involved. Hence, the combination of CFD and physical modelling was essential for detailed testing of several design iterations, minimising physical trials, and significantly reducing costs.

CFD enabled the modelling of the whole domain. The fish migration conditions between the tailrace confluence with the Tagus River and the afterbay sill were analysed in detail using a volume-of-fluid model, whereas the pool fishway was modelled indirectly. Concurrently, a 1:10 scale physical model focused on the afterbay, the pool fishway, and the bypass channel. This facilitated detailed insights regarding fish migration past the afterbay sill and the fish lock discharge attraction potential, further refining the design.

The project's success was driven by the integrated CFD and physical modelling approach and the interdisciplinary effort, demonstrating the critical role of collaboration in advancing eco-friendly hydraulic infrastructures.

Keywords: Dam, Eco-hydraulic, Fish pass system, Computational modelling, Physical modelling.

[1] lmendes@lnec.pt, Portugal

1. INTRODUCTION

1.1. Background and Context

Hydroelectric dams have a critical role in the production and storage of renewable energy, contributing to the reduction of greenhouse gas emissions [1]. However, these infrastructures pose negative ecological impacts on riverine ecosystems, including habitat fragmentation due to barriers to fish migration, altered flow regimes and dissolved gases and nutrients concentrations [2]. Moreover, the artificialization of river stretches in the tailrace channels, and hydropeaking operations — characterized by sudden discharge variations to meet energy demands — alter river hydromorphology and disrupt fish behaviour [3,4,5].

Effective fish pass systems are essential to restore connectivity and promote biodiversity conservation, mitigating the ecological disruptions. However, the design and operation of these systems often face significant engineering challenges due to site-specific hydraulic and biological constraints. River systems require innovative approaches to address the unique hydrological and ecological dynamics posed by these constraints. Emerging research highlights the complexities of fish migration, including alternative migration patterns influenced by environmental cues. Moreover, invasive species, which compete for resources and disrupt native habitats, further complicate ecological restoration efforts, emphasizing the value of holistic approaches to ecological restoration [6].

Addressing these compounded impacts demands multidisciplinary and collaborative efforts to develop site-specific methodologies, that combine advanced technological and ecological knowledge [7]. After monitoring and evaluating, each case-study may also provide valuable insights for similar ecosystems worldwide.

1.2. Relevance of the Study

The Fratel Hydroelectric Scheme on the Tagus River is paradigmatic. Operational since 1974, the dam has since been part of the Portuguese hydropower system [8]. However, it disrupted ecological connectivity in this important river system, impacting migratory fish species reliant on upstream habitats for critical life stages [5]. In 2025, EDP, the main energy production and distribution company in Portugal, will undertake an intervention, involving the reprofiling of the tailrace channel and the development of a new fish pass system, to improve the power production efficiency and, simultaneously, to provide better conditions for migratory fish to access upstream habitat.

This intervention aligns with global strategies to balance hydropower development with ecological preservation. It will be a significant step to enhance connectivity to upstream habitats, hence, reconnecting fragmented ecosystems along the Tagus River [6]. The fish pass

system is designed to enable migratory fish to swim past the powerhouse afterbay sill, even during low water levels, reaching the entrance of a future fish lock.

The fish pass system comprises three elements. A 230-metre-long channel, excavated along the tailrace reprofiled channel, enables migration during low water levels, and provides resting areas. A four-pool fishway and a bypass channel with irregular baffle walls allow fish movement past the afterbay sill, reaching the fish lock entrance.

A comprehensive study conducted at the National Civil Engineering Laboratory of Portugal (LNEC) employed computational fluid dynamics (CFD) and physical modelling to optimise the system proposed by EDP. Velocity fields and flow depths across a range of operational scenarios were evaluated. A multidisciplinary team, comprising experts in hydraulic engineering, computational and physical modelling, and ecological science from LNEC, EDP, and MARE/University of Évora, worked in close collaboration, analysing the results, and recommending design optimizations.

1.3. Methodological Approach

In the past, complex hydraulic structures, such as fish pass systems, were solely evaluated using physical modelling, implying significant building and operation costs. Despite the scale effects and experimental limitations, this method delivers measurements of flow in critical areas.

Over the last 20 years, computational fluid dynamics (CFD) of hydraulic structures proliferated among researchers and practising engineers due to technological advances. CFD allows testing designs in a relatively short time, with significant cost savings. However, CFD models may have limitations in reproducing the most complex flows, or demand exorbitant computational resources, especially in the presence of aeration [9,10,11]. Studies highlight the effectiveness of CFD in evaluating fishways, particularly for analysing flow structures and improving hydraulic conditions [12].

An integrated approach of CFD and physical models may mitigate shortcomings of both and even potentiate each other, significantly reducing costs and promoting more design optimisation. Additionally, CFD simulations provide critical pre-analysis insights for the initial physical model setup, minimising physical trials, and improving efficiency. Hence, this combination is the most advanced and robust methodology for designing efficient and sustainable fish pass systems.

1.4. Importance of Interdisciplinary Collaboration

This project exemplifies the importance of interdisciplinary collaboration, involving experts in hydraulic engineering, computational and physical modelling, and ecological sciences.

Collaborative efforts ensure the fish pass system addresses both technical and ecological requirements of target location and respective fish species, integrating insights from regional and international case studies.

1.5. Structure of the Paper

This paper is structured as follows: Section 2 enumerates the objectives of the study, Section 3 presents the fish pass system description, and Section 4 outlines the methodology. Section 5 presents design stages and their assessments. Section 6 discusses the methods, and the workflow applied. The conclusions in Section 7 summarize key findings and recommendations for future applications.

2. OBJECTIVES

The primary objective is to validate and optimise the hydraulic performance of a new fish pass system, designed to enable migratory fish to access the future fish lock entrance, even during low water levels.

The secondary objective is to refine the methodology for integrating computational fluid dynamics (CFD) and physical modelling to evaluate large and complex hydraulic structures.

3. DESCRIPTION OF THE FISH PASS SYSTEM

3.1. Fratel Dam

The Fratel Hydroelectric Scheme (HS), operational since 1974, is the first on the Portuguese section of the Tagus River. It is located 32 km downstream of Spain's Cedillo HS and 21 km upstream of the Belver HS (Fig. 1). The reservoir has a total storage capacity of 93 hm^3 and an active storage capacity of 21 hm^3. The Belver reservoir is considered to extend upstream to the Fratel tailrace confluence with the Tagus River, significantly influencing the design and performance of the fish pass system. Table 1 summarises the characteristic reservoir levels of both reservoirs.

Fig. 1 – Location of the Fratel Hydroelectric Scheme and other cross-river structures on the Tagus River (Google Earth)

Table 1 – Reservoir characteristics levels for the Fratel and Belver Hydroelectric Schemes

Hydroelectric Scheme	Full Storage Level (m)	Minimum Operating Level (m)	Maximum Flood Level (m)
Fratel	74.00	71.00	76.00
Belver	46.15	44.00	47.15

The Fratel Hydroelectric Scheme consists of an overflow-dam on the right bank and separated by the central-wall, the hydraulic circuit of the generating units, the powerhouse, and the substation on the left bank (Fig. 2).

The spillway dam is a gravity-type concrete structure with a maximum height of 48 m and a crest length of 160 m extending to the central-wall. The dam crest is at elevation (78.00 m). The flood spillway has a maximum discharge capacity of 16,500 m³/s, and comprises six identical bays, each 18.75 m wide, with the crest at elevation (57.00 m). Energy dissipation is achieved through hydraulic jump dissipation basins.

Fig. 2 – Overview of the Fratel Hydroelectric Scheme from downstream [8]

The powerhouse is equipped with three generating units featuring vertical-axis Kaplan turbines, with an installed capacity of 132 MW and a nominal flow rate of 250 m³/s per unit. The static head ranges from 18 m to 29 m, and the turbine draft tubes' crest is at elevation (34.00). Immediately downstream, the afterbay consists of an ascending concrete ramp with approximately 60 m length and 50 m wide. The terminal section is a sill at the elevation (45.70), which includes six piers designed for the installation of cofferdams. Each pier has a cross-section 1.0 m wide, and the total clear span is 43.26 m (Fig. 3).

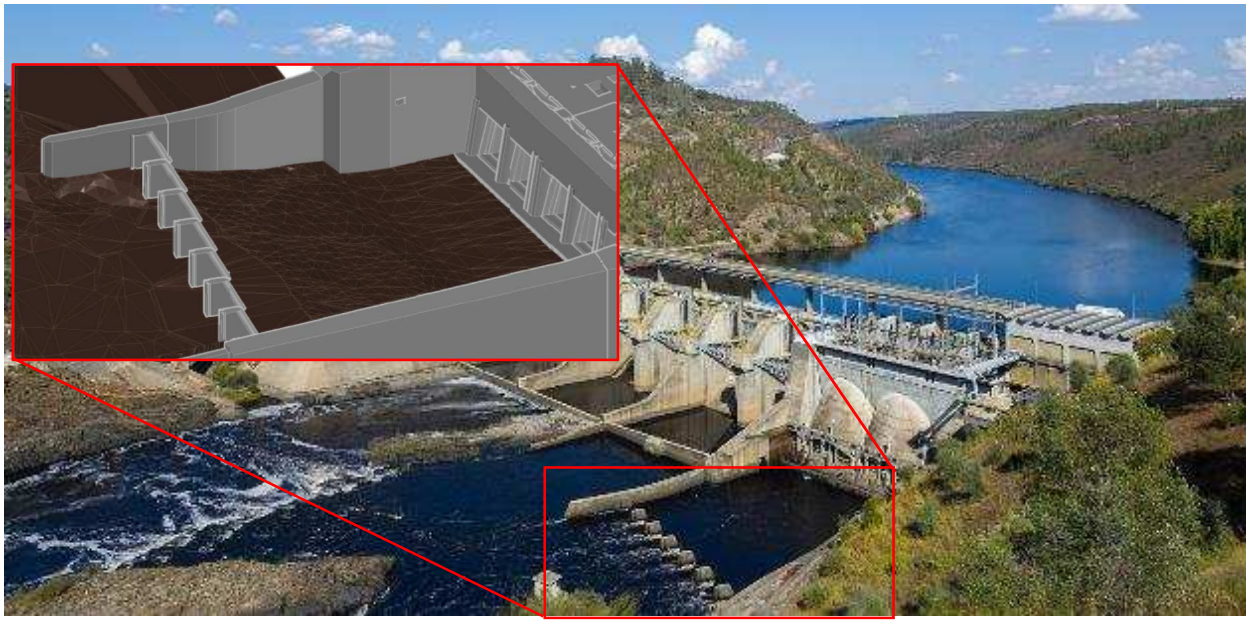


Fig. 3 – Central-wall, tailrace structure, including submerged area scheme (Source - File: CP 2240 Barragem de Fratel.jpg — https://en.wikipedia.org)

A fish lock entrance is positioned on the right side of the central-wall, in the afterbay (Fig. 4). The bottom of the entrance is 0.93 m higher than the afterbay sill, with the crest at elevation (45.70). Although the mechanical equipment was not installed during the dam's construction, operational assessments indicated that water levels suitable for the lock's operation would occur only 6% of the time annually [13].

Downstream of the afterbay, there is a rock-excavated channel with an irregular surface and an average elevation of (45.50), with 240 m long and 60 m wide. This channel follows the left bank, discharging the turbine outlets to the main riverbed. In the downstream extremity, there is a rock formation with an average crest elevation of (46.00), which functions as a ramp-weir with a length varying from 10 to 20 meters.

Fig. 4 – Fish lock entrance: view from the tailrace structure of the generating units [8]

3.2. Intervention motivation and scope

In 2025, EDP will initiate an intervention at the Fratel Hydroelectric Scheme on the Tagus River to improve power production efficiency and river connectivity, enabling fish access to upstream habitats. The operational head was increased 0.7 m by reprofiling the tailrace channel, which was widened, and its bottom lowered and levelled to (44.00 m). Priorly, with the average level at (45.50), it was occasionally dry, preventing fish from accessing the area downstream of the afterbay sill.

Subsequently, EDP analysed the impact of the reprofiling on facilitating migratory fish movement between the Belver reservoir and the Fratel dam, aiming to ensure access to the

entrance of the future fish lock in the afterbay, even during low water levels [13]. This led to the design of a new fish pass system, which is the focus of this study.

The design of the fish pass system is guided by the following key considerations:

1. The fish pass system will not function during periods of operation of the hydroelectric plant or flood spillways' discharge. In these scenarios, the maximum flow at the turbine outlets is approximately 10 m^3/s, due to leakage, which is significantly lower than the discharge during turbine operation (200 m^3/s to 600 m^3/s). The fish lock contributes with an additional flow of 1 m^3/s.
2. The level of the Belver reservoir, which extends to the Fratel tailrace channel, ranges between the minimum elevation (44.00 m) and the maximum elevation (46.15 m) and is a key factor in defining the solutions. The most frequent elevation is (45.00 m).
3. The afterbay sill crest, at elevation (45.70 m), creates an obstacle to fish movement. Lowering the sill is not feasible due to the minimum submergence requirements of the turbine units. A preliminary study suggested installing a fish pass device to resolve this issue, facilitating connectivity between the tailrace channel and the afterbay.
4. The channel bed of the reprofiled tailrace channel is at elevation (44.00 m), occasionally posing a limitation to fish movement, due to the lack of water depth. Therefore, constructing a dedicated fish channel excavated into the channel bed was considered to ensure connectivity, even during low water levels.
5. A monitoring campaign conducted in 2022 and 2023 identified the main target fish species to be considered in this study, concluding that native migratory species are mainly present in tributaries of the Tagus with lotic characteristics and downstream of Belver. In the case of the European eel, it was also found upstream of Belver HS. It was also concluded that there is a significant increase in the predominance of non-indigenous species in the most upstream stretches of the studied area.
6. Hydraulic performance criteria include velocities below 1.0 m/s throughout most of the fish pass system, with occasional maximums of up to 1.5 m/s along paths shorter than 2 m.
7. The installation of an efficient fish lock mechanism in the Fratel HS is critical for the success of the new fish pass system, providing effective access to upstream habitats necessary for spawning and other life stages, supporting the sustainability of aquatic ecosystems.

3.3. Initial design

The initial design of the new fish pass system is described below, and the design iterations are presented and analysed in Section 5 — Results (Fig 5).

The reprofiling of the tailwater channel involved lowering the average bed elevation from (45.50) to (44.00) and widening the bottom to 40 m, which allowed for an increase in the operational head by 0.7 m.

Additionally, the initial solution for the new fish pass system consists of four key elements, arranged from downstream to upstream, following the fish migration path:

- **230-Metre-Long Fish-Dedicated Channel**
    - A fish-dedicated channel, 230 metres in length, with a trapezoidal cross-section featuring an upper width of 2.5 m and a lower width of 1.5 m, and a bed elevation at (43.00 m), excavated along the left side of the tailwater channel.
    - This channel ensures hydraulic continuity along the tailwater channel, connecting the Belver reservoir to the downstream side of the afterbay sill, even when the reservoir is near its minimum value (44.00 m).
- **Four-Pool Fishway**
    - A four-pool fishway, constructed in concrete, is 18.1 metres long, 3.9 metres wide, and 4.6 metres high. It is located on the right side of the afterbay sill, adjacent to the central wall that separates the powerhouse from the spillway. The top elevation of the external walls and internal baffles is (46.50 m).
    - Each pool measures 4.0 m in length, 2.7 m in width, and 1.25 m in height, with a bottom slope of 5%, providing a resting area for fish. The vertical slots between pools are 0.4 m wide.
    - Optimal operating conditions are achieved with a discharge of 1 m^3/s and a total drop of 1.25 m, distributed across the three vertical slots between pools and the two slots at the entry and exit. Each drop has a height of 0.25 m.
    - This fishway enables fish to overcome the maximum flow drop of 1.25 m between the tailwater channel (44.00 m) and the afterbay (45.25 m).
- **Afterbay Sill Bypass Channel**
    - Located on the right side of afterbay sill, adjacent to the pool fishway, features a rectangular cross-section with a base width of 2.5 m and a height of 0.7 m, resulting in a crest at level (45.00). It starts in the sill crest and develops along the fishway (Fig. 6).

- Reduces the afterbay minimum level to approximately (45.25) and generates a fish attraction flow next to the fishway, and into the 230-meter-long fish-dedicated channel, using the turbines leaks discharge.
- Designed to operate effectively within the water levels of the afterbay, ranging from (45.00 m) to (46.15 m).
- Creates an alternative path for fish movement past the afterbay sill when water levels on both sides of the sill are similar. Under these conditions, the bypass channel ensures favourable flow velocities, compensating for the low discharge from the pool fishway.

- **Fish lock entrance**
  - Located in the afterbay, at the central wall, with a rectangular cross-section measuring 1.2 x 1.0 m (W x H) and a sill at elevation (45.25 m), recessed by 1.4 m.
  - Designed to allow fish access to the fish lock across all afterbay water levels, which range from (45.00 m) to (46.15 m).

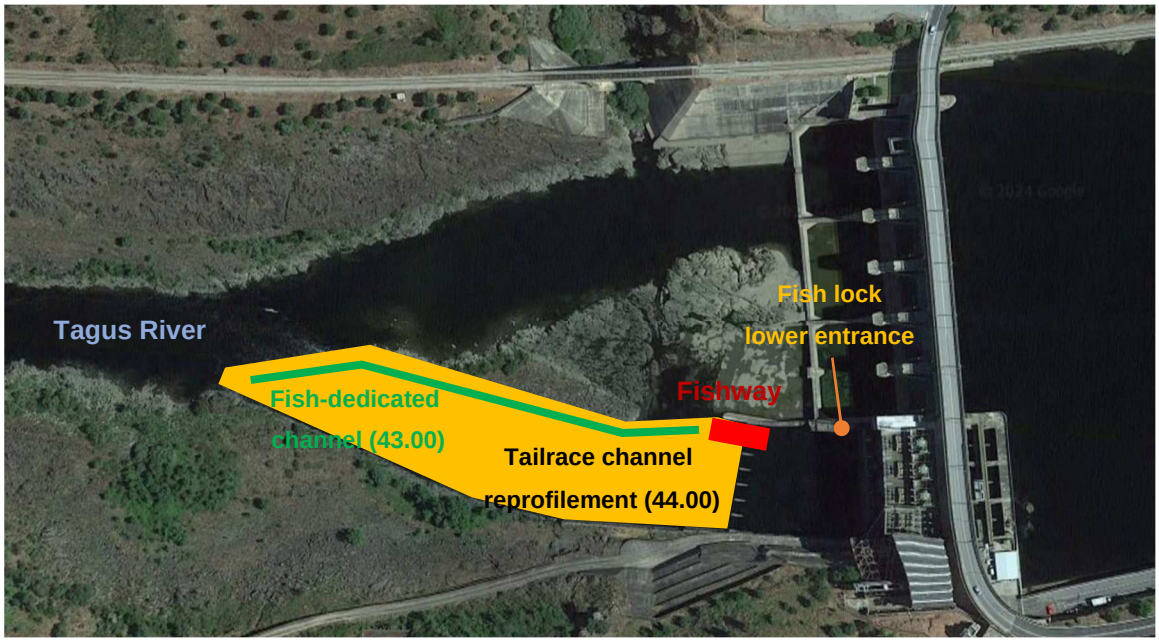


Fig. 5 – Initial design of the new fish pass system: top view (Source: Google Earth)

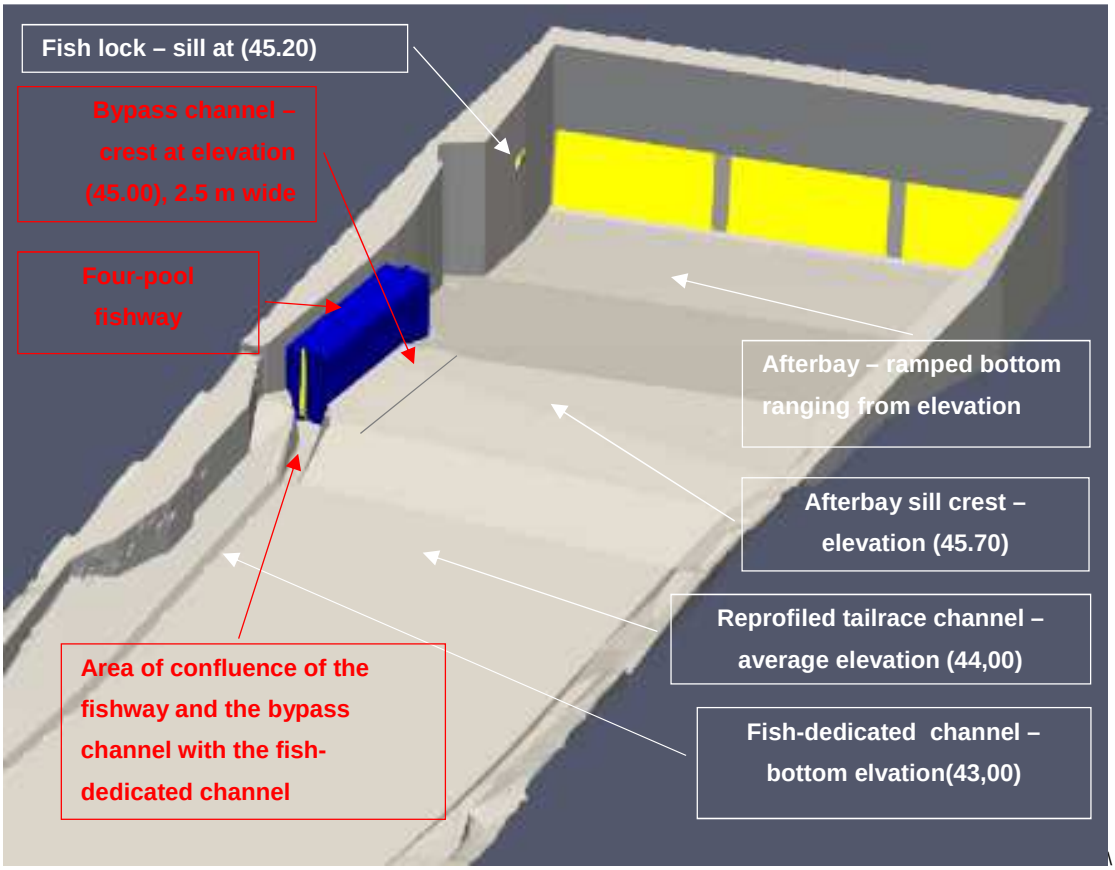


Fig. 6 – Initial design of the new fish pass system: 3D scheme of the afterbay

The downstream water level is dictated by Belver, ranging from elevations (44.00 m) to (46.15 m), while the afterbay is constrained by the sill's crest elevation (45.00 m).

Considering two paths for discharging flows from turbine leakage and fish lock operation past the afterbay sill (fishway and bypass channel), the design aimed to ensure suitable conditions for fish movement without significantly altering the afterbay water surface elevation. At low and intermediate Belver reservoir levels, fish are expected to use the fishway, as the bypass channel velocities are too high. However, for high levels in Belver, the fishway velocities decrease significantly, while the bypass channel velocities become more moderate, turning the bypass channel more effective by providing stronger fish attraction flows.

4. METHODOLOGY

4.1. Initial considerations

The hydraulic study of the new fish pass system at the Fratel Hydroelectric Scheme involves reproducing flow conditions between the afterbay and the tailrace confluence with the main channel of the Tagus River, covering an area of approximately 355 m by 90 m (Fig. 7).

Additionally, upstream and downstream sections of the confluence are included to define flow conditions accurately, such as water surface levels, resulting in a total domain length of 450 m.

Fig. 7 – The new fish pass system: area of interest. Source: Google Earth [13]

The large domain dimensions and low flow rates (4 to 10 m^3/s), resulting in shallow flow depths (less than 1 m), would require an impractically large physical model, as detailed in Section 4.3. Similarly, developing a single computational model to accurately capture all geometries would be computationally prohibitive due to the wide range of time and length scales involved.

In 2018, EDP conducted a preliminary study on the reprofiling of the tailrace channel using two-dimensional modelling. However, at the present study, the geometries involved — primarily the fishway, the turbine dissipation basin, the lowered sill and its transition to the secondary channel, the secondary channel, and the confluence — indicated the importance of capturing the three-dimensional nature of the flow in these areas. Consequently, a three-dimensional numerical model was selected.

To address these challenges, the study adopted a dual approach combining CFD and physical modelling. This integration was essential for detailed testing of multiple design iterations, minimising the need for physical trials, and significantly reducing costs.

A multidisciplinary team comprising experts in hydraulic engineering, computational and physical modelling, and ecological science from LNEC, EDP, and MARE/University of Évora, worked in close collaboration. For each design iteration, the performance was analysed, identifying potential issues, and suggesting optimizations, until a good overall system performance was achieved.

Next, both modelling techniques are described following the order application, where the fish pass system analysis started from the global to the detailed analysis, i.e. from CFD to physical model.

4.2. Computational Modelling

Computational fluid dynamics (CFD) simulations were conducted to globally model the new fish pass system, considering the entire domain between the tailrace confluence with the Tagus River and the afterbay. Key elements were accurately reproduced, including the reprofiled channel and its confluence with the Tagus River, the 230-metre-long fish-dedicated channel, the afterbay with the three turbine circuit outlets, and the fish lock entrance flow. The pool fishway, however, was modelled indirectly.

The primary objective was to evaluate the hydraulic performance of the following elements:

- Tailrace channel
- 230-metre-long fish-dedicated channel
- Bypass channel at the afterbay sill

CFD provided critical pre-analysis insights and expedited design optimisation, before the construction of physical model, minimising physical trials.

Numerical modelling was performed using OpenFOAM v2012, a state-of-the-art open-source computational fluid dynamics (CFD) software (OpenCFD, 2020). The interFoam solver, designed for two immiscible and incompressible fluids (air and water), was employed due to the predominantly free-surface flow with negligible air entrainment. The solver uses the finite volume method to solve the Navier-Stokes equations, applying the URANS (Unsteady Reynolds-Averaged Navier-Stokes) approach with the realizable k-epsilon turbulence model. The free surface is captured using the VOF (Volume of Fluid) technique. Further details can be found in Mendes (2021).

OpenFOAM, and interFoam, have been successfully applied at LNEC to various types of free-surface flows, such as the spillways of the Fierza Dam in Albania [15], as well as in several scientific research projects on air-water turbulent flows [9,10,11].

The computational grids were primarily orthogonal, consisting of cubic elements, with few exceptions at the boundaries. The total number of cells was approximately 10 million, with a size of 0.25 × 0.25 × 0.125 m^3 (X, Y, and Z axes) for cells occupied by water. The vertical direction, associated with gravitational acceleration, was aligned with the Z-axis.

The following boundary conditions were applied:

- **Fish lock**: Uniformly distributed discharge and aligned with the internal geometry of the existing structure.
- **Turbine outlets**: Leakage discharge for each unit, uniformly distributed and perpendicular to the outlet section face.
- **Fishway**: Only inflow and outflow, uniformly distributed along the depth and perpendicular to the inlet and outlet faces of the fishway sections.
- **Concrete walls**: Equivalent sand-grain roughness height of 0.5 cm.
- **Terrain next to the fishway**: Including the turbine ramp and transition zone from the tailrace sill to the reprofiled channel, with an equivalent sand-grain roughness height of 5.0 cm.
- **Reprofiled terrain and Tagus River**: Equivalent sand-grain roughness height of 10.0 cm.
- **Downstream**: outflow/inflow adjustments according to the defined water surface level.
- **Top boundary**: volume continuity condition regulated by air inflow/outflow.

The following numerical schemes were used: Euler and local Euler (Local Time Stepping) for the temporal derivative; Gauss linear gradient, with and without multidirectional limiter, for the spatial gradient; Gauss upwind and Gauss linearUpwind for the velocity divergence; and Gauss upwind and Gauss limitedLinear for the turbulence fields.

In each simulation, it was monitored: total kinetic energy, turbulent kinetic energy, inlet and outlet flow rates, maximum velocity, maximum pressure, and water levels at eight locations Simulations were run for the duration required to ensure steady-state conditions, particularly regarding the stabilization of water surface levels in the afterbay and at the confluence, as well as the outlet flow rate at the downstream boundary. The average computation time was eight days, using 64 processors. Simulations were carried out using resources from LNEC, integrated into the high-performance computing centre of INCD (National Distributed Computing Infrastructure).

4.3. Physical Modelling

The large domain of interest and the multiple hydraulic parameters requiring accurate similarity reproduction and measurements, such as very low velocities and flow depths in the reprofiled channel, would necessitate a physical model with a very low geometric scale factor (ideally

less than 5). This would result in an occupied area exceeding 70 m by 18 m, making it excessively costly and disproportionate to the new fish pass system construction cost.

The hydraulic phenomena are governed by gravitational forces, requiring adherence to Froude similarity (Fr) to establish dimensional relationships between the reduced model and the prototype, correlating gravity and inertial forces. For turbulent flows, the geometric scale must ensure Reynolds numbers (Re) of at least 10^4 in the model to maintain similarity with the prototype. Although a geometric scale of L_r=7.4 or lower was ideal, space constraints in the experimental laboratory limited the scale to L_r=10, resulting in a model Reynolds number (Re_m) of 0.63 ×10^4, which is considered sufficiently close to the target $Re_m = 10^4$. Economic feasibility and space limitations were also factored into the scale definition.

Hence, a 1:10 scale physical model was developed to focus on the hydraulic behaviour of fish migration past the afterbay sill and the fish lock discharge attraction potential, allowing detailed flow analysis and further design refinements. It adequately reproduces the draft structures of the three turbine units, the afterbay, the fish lock discharge, the fishway, the bypass channel at the sill, and the initial segments of the reprofiled tailrace channel and the fish-dedicated channel (Fig 8).

The scaling relationships used in the study for the relevant quantities are as follows:

- Lengths (geometric scale): L_r=10
- Areas: $A_r = 10^2$
- Velocities: $Vr = 10^{1/2}$
- Flow rates: $Q_r = 10^{5/2}$
- Times: $t_r = 10^{1/2}$

The model occupies a laboratory area of 20 meters by 6 meters, with a height of 2.0 meters, and allowed for precise measurements of velocity fields, flow depths, and turbulence patterns.

Non-traditional construction methods, including CNC machining, 3D printing, and light and low-cost materials such as PVC sheets and EPS blocks, were utilised for the model's components. These components were assembled into a masonry structure consisting of two large reservoirs and a connecting platform (Fig 9).

The physical model includes four flow sources, replicating discharges from the three turbines and the fish lock, each regulated by installed valves for precise control. Downstream levels are controlled in a reservoir using exit valves.

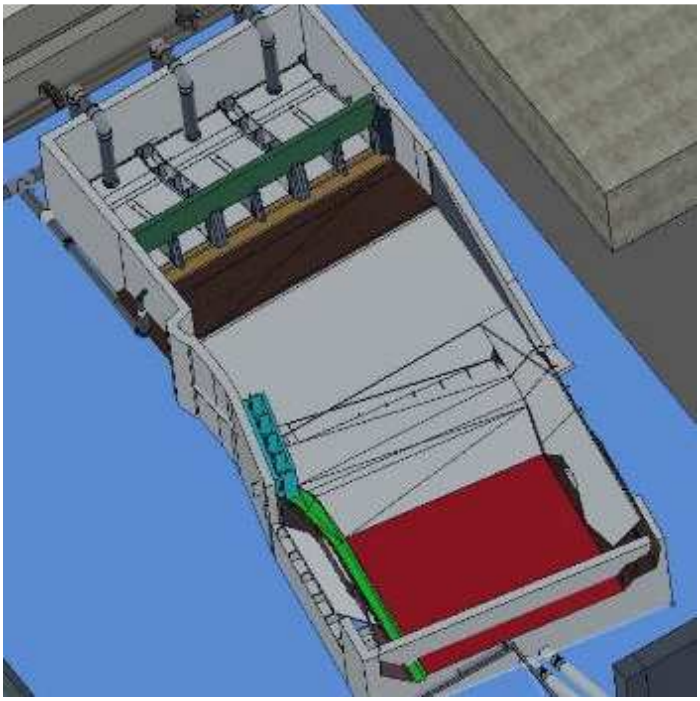


Fig. 8 – Physical model: 3D CAD scheme

Fig. 9 – Physical model construction.

The fish lock discharge system allows for adjusting the elevation of the rectangular entrance opening (1.0 m high by 1.4 m wide in the prototype) between (45.00 m) and (45.50 m) by replacing components manufactured using 3D printing. In the initial design, the new sill elevation for the fish lock entrance is set at (45.25 m).

The fishway, consisting of four successive pools separated by vertical slots, was mostly constructed using PVC sheets cut with a CNC machine. The more complex geometries of the vertical slots were produced in PLA via 3D printing (black elements in Fig. 8), as well as the turbine outlet piers.

Anticipating the need for future modifications in the bypass channel and 230 meters-long dedicated fish channel areas, these sections were constructed using expanded polystyrene (EPS) coated with a fibreglass-reinforced mortar (Figures 4.2 and 4.3 — grey surface).

Fig. 10 – Physical model construction: fishway and bypass channel

The measurements during the tests included flow rates, water levels, velocities, and flow direction. Flow rates were measured using electromagnetic and ultrasonic flow meters, while water levels were recorded with vernier point gauge in critical locations and graduated rulers near walls. Surface velocities were analysed using video processing with Matlab-PIVlab, and flow direction was qualitatively observed using dye tracers at the surface and wool threads near the bottom and within the flow.

4.4. Scenarios

The hydraulic performance of the new fish pass system was assessed for each design iteration, under multiple operating scenarios defined by varying water levels in the Belver dam reservoir, and turbine outlets. The criteria include **Hydraulic Continuity**, **Velocity Fields, and Flow Depths.**

The scenarios analysed comprise combinations of the following factors:

- **Geometry variations**, including the existing site and design iterations of the new fish pass system:
    - Before intervention
    - Initial design
    - Design A
    - Design B
- **Inflow discharges**:
    - Turbine outlet discharges: 0 m^3/s, 3×1.0 m^3/s
    - Fish lock discharge: 1.0 m^3/s
- **Water levels** at the confluence of the tailrace channel and the main riverbed of the Tagus River, corresponding to three reservoir levels at Belver, selected based on non-exceedance probabilities for January to July, as determined by EDP:
    - Minimum level of (44.00 m), with a 1% non-exceedance probability
    - Intermediate level of (45.00 m), with a 46% non-exceedance probability
    - Maximum level of (46.15 m), with a 97.2% non-exceedance probability
- **Boundary conditions:**
    - Upstream: Defined by the imposed discharges from each turbine unit and the fish lock.
    - Downstream:
        - **CFD modelling**: Defined by the water level in the Belver reservoir.
        - **Physical modelling**: Defined by the water level estimated from numerical modelling at the downstream section of the physical model.

In the CFD calculations, the fishway was indirectly modelled by not replacing the pools with a discharge sink at the entrance and a discharge source at the exit, with a flow depth according to the surrounding water levels. The discharge values were set to 0.9 m^3/s, 0.6 m^3/s, and 0.0 m^3/s for confluence levels of (44.00 m), (45.00 m), and (46.15 m), respectively.

5. RESULTS – CFD MODELLING

5.1. Initial considerations

Due to the large number of scenarios analysed, only the main results and the respective plots of velocity fields, and water surface levels of the final iteration are presented.

5.2. Before intervention

CFD simulations began with an analysis of the existing site. Additionally, a qualitative comparison was made between the flow from preliminary tests and a photograph of the site under similar flow conditions, confirming higher velocities and some air entrainment on the right side of the confluence (Fig. 11).

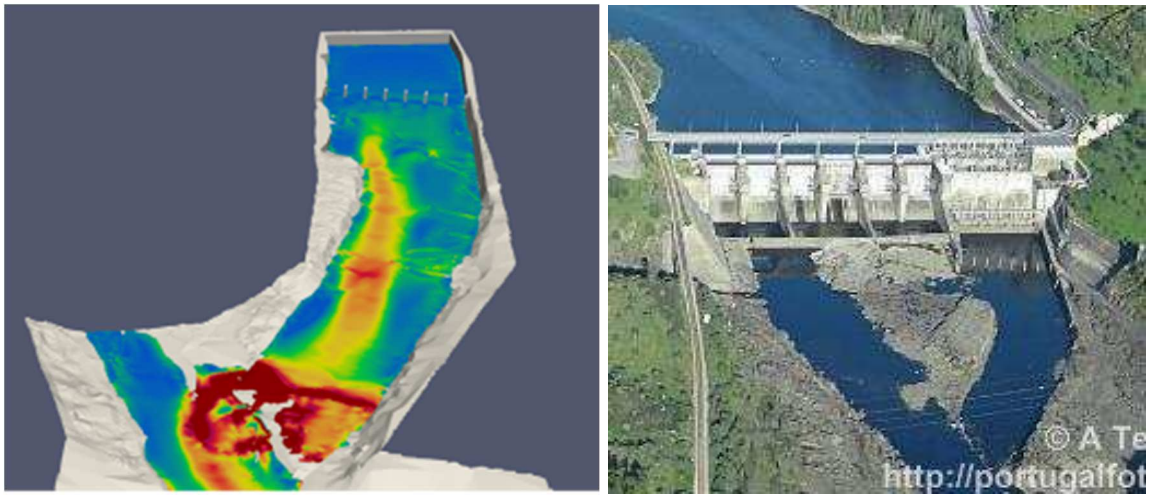
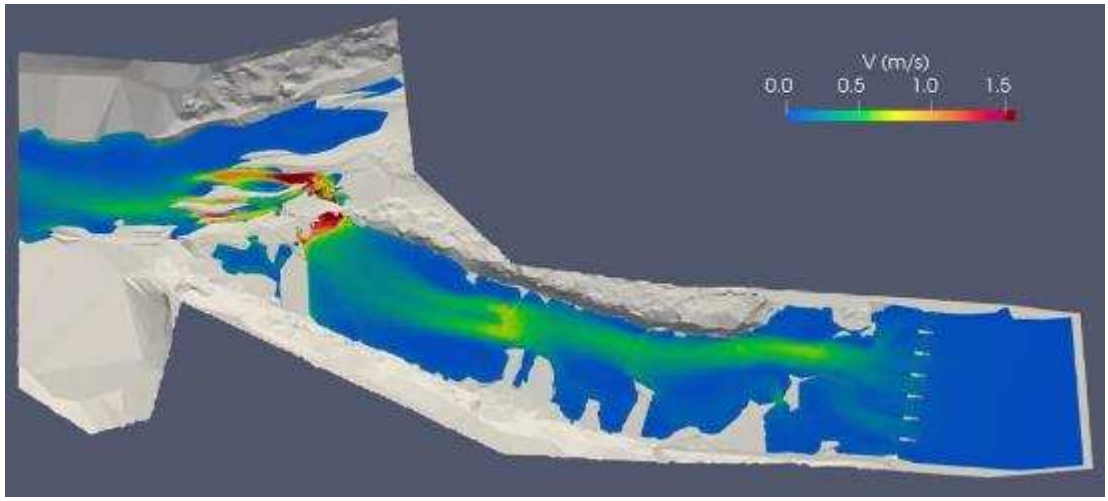


Fig. 11 – CFD model before intervention. Preliminary test: surface velocities and comparison with photograph. Leakage flow of 10.0 m³/s, confluence level (44.00 m). (source: https://portugalfotografiaaerea.blogspot.com/2013/06/barragem-de-fratel.html)

The simulations demonstrated similar hydraulic behaviour for confluence levels of (44.00 m) and (45.00 m) (Fig. 12, where the downstream section of the tailrace channel acts as a "control section", imposing subcritical flow upstream and creating localised and significant water drops. At the (46.15 m) level, the flow remains subcritical throughout the domain, controlled by the confluence level. the flow in the basin is identical for all three levels.

a)



b)

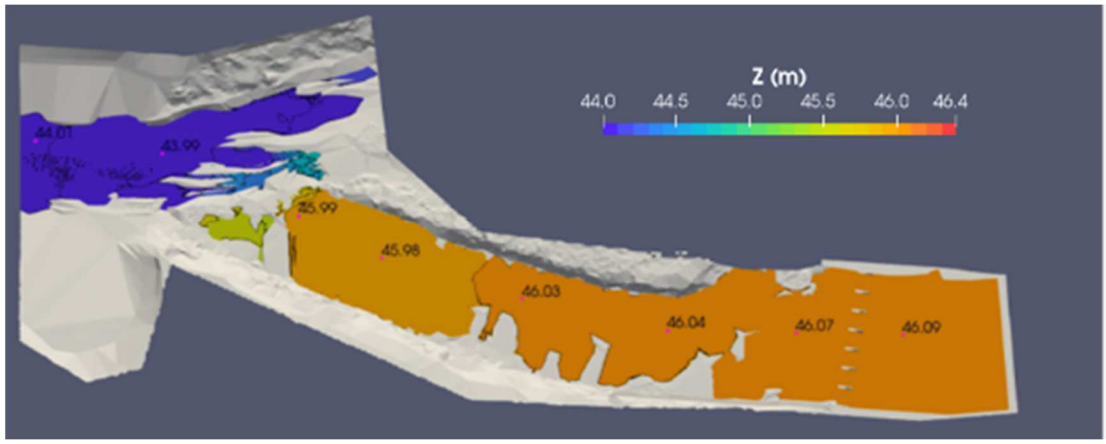


Fig. 12 – CFD model before intervention. Confluence level (44.00): a) Surface velocity field, b) water surface level.

5.3. Design iterations

Initial design

Preliminary tests based on the proposed design revealed distinct behaviours for the two analysed levels, (44.00 m) and (46.15 m), particularly regarding flow in the reprofiled channel.

At a confluence level of (44.00 m), the flow along the reprofiled channel has a shallow depth of approximately 0.2 m and velocities around 1.0 m/s. The afterbay sill acts as a control section, imposing subcritical flow in the basin. At a confluence level of (46.15 m), the flow along the reprofiled channel has a depth of approximately 2.0 m and velocities below 0.2 m/s. The flow is subcritical throughout the domain, governed by the confluence level. Similar to before the intervention, the variation in free surface elevation across the studied domain is minimal, approximately 0.15 m from the confluence to the tailrace basin.

Design iteration A

This iteration incorporates improvements to the initial design, including the creation of a resting area downstream of the fishway and four resting zones along the secondary channel.

The analyses revealed that the resting zones along the secondary channel did not achieve the desired flow effects and required widening. The geometric definition of the reprofiling was also addressed, noting that excavation would involve blasting and ripping, resulting in a somewhat irregular geometry. Consequently, it was decided to add macro-roughness elements to the numerical model geometry in the excavated areas, instead of bottoms.

Design iteration B

The final stage of CFD modelling included the following design optimisations:

- Modifications to bypass channel, increasing its width from 2.5 m to 8.0 m.
- Changes to the bypass channel geometry in plan view.
- Extension of the resting zone downstream of the fishway.
- Introduction and refinement of the five resting zones in the fish-dedicated channel, with dimensions of 8.0 m in length, 2.0 m in width, and spaced 30.0 m apart.

The velocities and water surface plots for the three analysed levels are presented in Fig. 13 and Fig. 14.

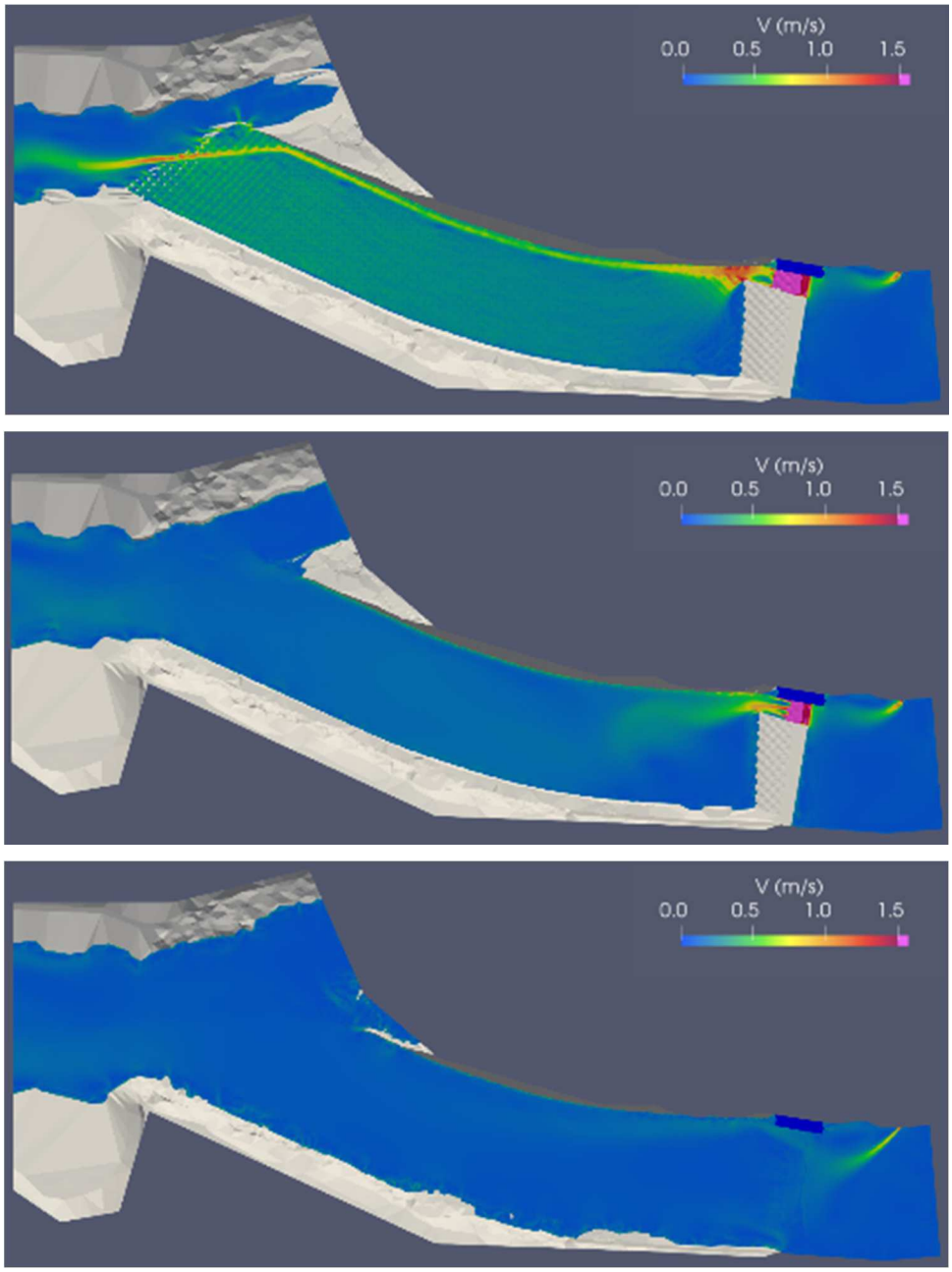
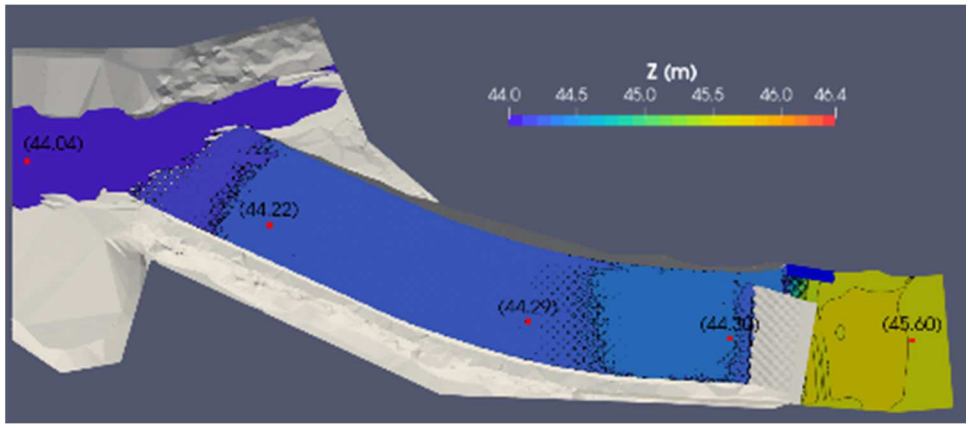
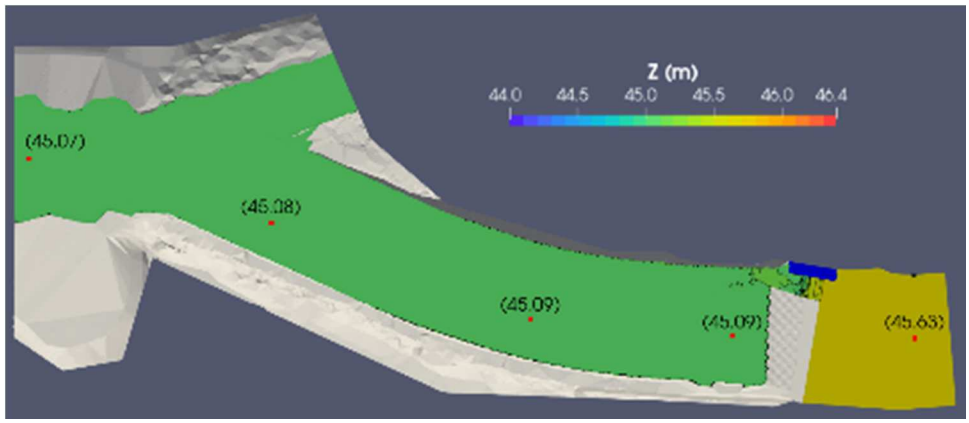


Fig. 13 – CFD final design: surface velocities. Confluence level: a) (44,00), b) (45,00) e c) (46,15)

a)



b)



c)

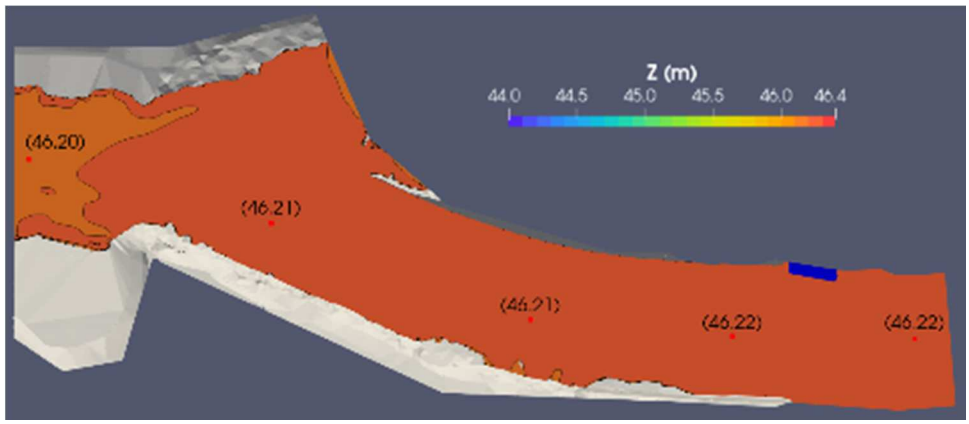


Fig. 14 – CFD final design: water surface level. Confluence levels: a) (44,00), b) (45,00) e c) (46,15)

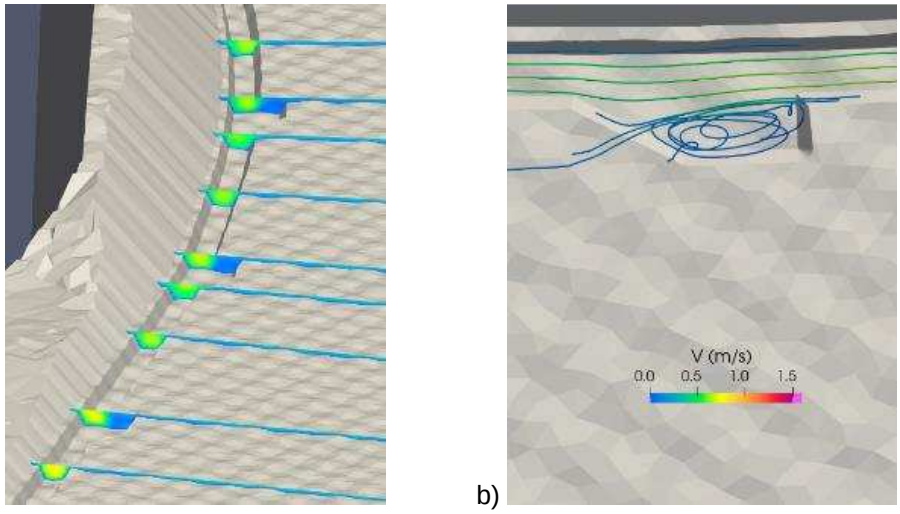


Fig. 15 – CFD final design: resting zones in the fish-dedicated channel: (a) velocity profiles in cross-sections, (b) streamlines. Confluence level (44.00 m).

The main iteration of the final design are presents in Fig. 16.

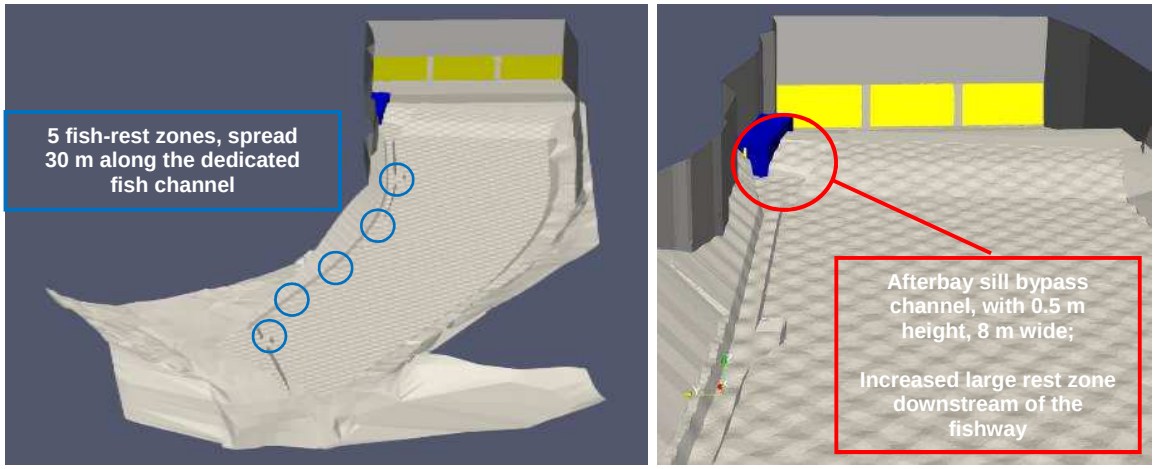


Fig. 16 – CFD final design: optimisations.

For all three scenarios, hydraulic continuity is maintained with depths not less than 1.0 m. Velocities exceed 1.0 m/s only near the lowered sill of the turbine basin, and the resting zones function as intended. Fish passage is possible via the fishway at all levels and via the lowered sill, particularly for confluence levels above (45.50 m), where the ramp becomes submerged, reducing velocities to below 1.0 m/s.

The reprofiling of the tailrace channel significantly improves hydraulic continuity, especially for Belver reservoir levels below (45.50 m), mainly due to the secondary channel, which ensures continuity at lower levels. This intervention homogenizes velocity distribution at the confluence

to below 1.0 m/s and eliminates water drops observed in before the intervention across all Belver reservoir levels.

5.4. CFD Benefits

The numerical simulations defined the configuration of the afterbay sill bypass channel and its transition to the fish-dedicated channel. They also determined the outflow curves for the downstream boundary of the physical model for the reproduced scenarios.

Numerical modelling contributed to improving the design, enabling the physical model to incorporate the proposed changes from the beginning of the trials.

The CFD modelling was based on the following assumptions:

- Simplified validation and calibration due to a lack of field data for the tailrace channel at the analysed flows and levels.
- Roughness geometry from reprofiling was simulated using custom algorithms, introducing regular patterns to approximate the random irregularities of rock excavation.
- The domain's large extent, from the tailrace to the confluence, limited resolution (maximum vertical resolution of 12.5 cm). Localised numerical modelling, including the fishway and lowered sill, was replaced by data from the physical model.
- The fishway interior was not numerically modelled; only inflow and outflow were considered, based on confluence levels.

6. RESULTS - PHYSICAL MODELLING

6.1. Initial considerations

In the physical model at scale 1:10, the fishway and turbine tailrace structures were fully reproduced.

The following program was established for the physical model trials:

- Preliminary tests to verify the general operation of the model and equipment.
- General characterisation of upstream water levels, including the vicinity and interior of the fishway and the lowered sill area.

- Velocity field characterisation in areas of interest, specially resulting of the fish loch discharge flow.

The physical model tests provided a general characterisation of flow conditions in the reproduced area, analysing three reservoir levels at Belver, along with additional intermediate levels. Detailed tests were not conducted for the (46.00 m) level, as the Belver reservoir extends to the central wall of the Fratel Hydroelectric Scheme in this scenario.

Visual inspection of flow conditions was complemented by qualitative tests using dye injected at key points to better understand flow patterns, velocity distributions, and turbulence levels, (Fig 17). Particle Image Velocimetry (PIV) technology was used to measure mean and surface velocity fields at the fish lock outlet, the fishway entry and exit zones, the fishway pools, the lowered sill area, and the downstream ramp.

6.2. Physical modelling trials

To keep the article concise, only selected results are presented next. The physical model tests started with the fish lock entrance sill at elevation (45.25 m). However, it was concluded that the lower sill elevation of (45.00 m) provides better pattern and intensity, hence, performance for the range of water levels in the turbine tailrace basin intended to ensure the fish lock's operation.

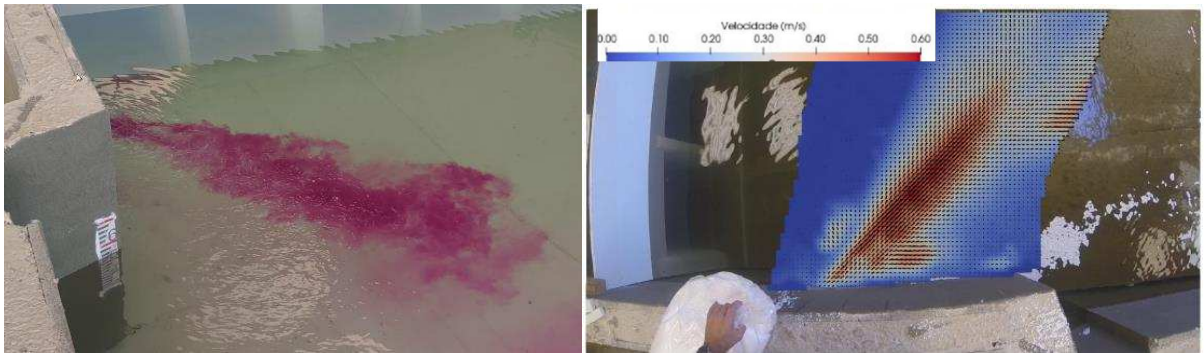


Fig. 17 – Physical model. Level (45.00). Horizontal jet from the fish lock into the turbine tailrace basin. Dye plume and surface PIV.

The pool fishway performed according to the design considerations (Fig. 17). The fishway and the lowering of the afterbay sill bypass channel ensure both hydraulic continuity for fish migration and adequate discharge conditions for flows from the fish lock and turbine leakage. Notably, the fishway is crucial for low to intermediate downstream water levels, maintaining flow depths above 1.0 m and velocities below 1.0 m/s.

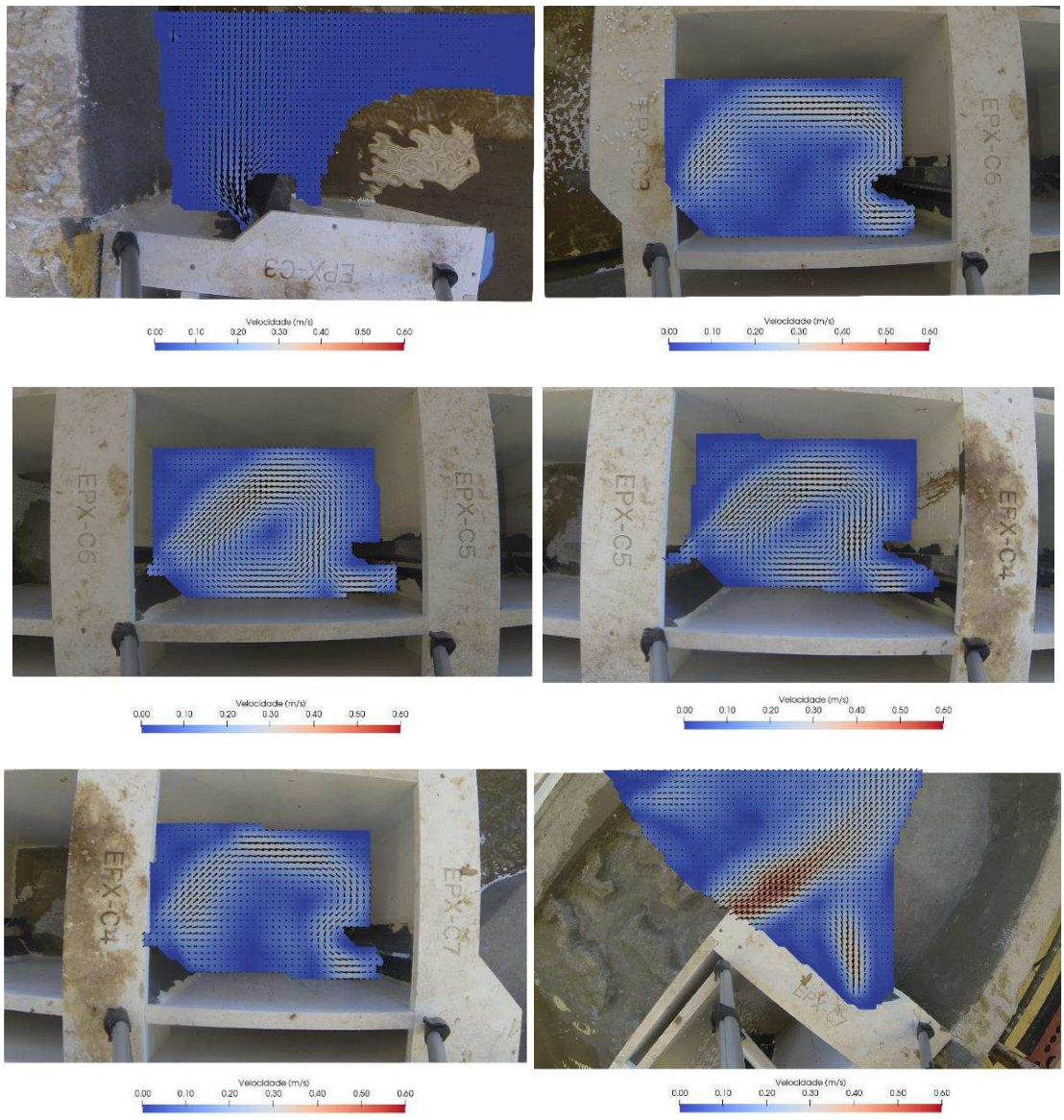


Fig. 18 – Physical model. Level (44.00). Pool Fishway. Time-average superficial velocities. Sorted by: entrance, pools 1 to 4, exit.

Maximum velocities on the ramp reach approximately 4.0 m/s, but these are not uniform across the ramp width, with values around 1.0 m/s or lower on the right half. At the confluence of the ramp and the secondary channel, the maximum observed velocity is about 2.9 m/s. However, higher velocities are concentrated on the left side of the ramp and confluence, while much of the analysed area exhibits velocities below 1.0 m/s.

Fig. 19 – Physical model. Level (45.00): Lowered sill and connecting ramp to the secondary channel. Dye plume released upstream of the flow control sill.

Despite the results of the CFD modelling, an enhancement to design iteration B was proposed: incorporating macroroughness elements on the bypass channel downstream of the lowered sill. These elements would act as obstacles to the fast flow occurring on the ramp at intermediate or low water levels in the Belver reservoir, creating flow conditions more similar to those observed in nature. Due to limitations in mesh resolution in this area of the numerical model, the effect of introducing macroroughness was studied exclusively in the physical model.

Following the hydraulic characterisation of Design iteration B from the numerical model studies, additional tests examined the hydraulic performance with macroroughness elements on the bypass channel ramp between the lowered sill and the large resting area downstream of the fishway. Two geometries for these obstacles were tested, both provided by EDP. The first iteration was based on general guidelines from MARE/UE biologists, while the second iteration refined hydraulic performance observed in the first.

Fig. 20 – Physical model. Level (45.00). Bypass channel ramp: a) flat bottom; b) macroroughness.

The addition of macroroughness elements on the bypass channel ramp downstream of the sill crest, which is lowered from the current elevation (45.70 m) to (45.20 m), improves hydraulic performance, particularly for the most frequent operating conditions corresponding to Belver reservoir levels near elevation (45.00 m).

Overall, for the most demanding and less frequent scenario of Belver's, the minimum storage level (44.00 m), and the more frequent scenario (45.00 m), the velocity fields and water depths are according to design criteria. The optimal hydraulic performance of key components was attained, particularly the fishway interior and vicinities, as well as the downstream rest area in the confluence with the 230-meters-long dedicated fish channel. These structures ensure effective flow along the two preferred paths, through the fishway and along the ramp downstream of the lowered turbine tailrace sill, providing migratory fish access to the afterbay, and consequently to the fish lock entrance.

7. DISCUSSION

The results from both numerical and physical modelling provided a comprehensive a hydraulic characterization, including velocity fields and flow depths, under various operating scenarios dictated by the water levels in the Belver reservoir, downstream of Fratel HS. The analysis

focused on the hydraulic performance of the fish pass and ensuring hydraulic continuity between the powerhouse afterbay and the tailrace.

CFD provided critical pre-analysis insights and expedited design optimisation, before the construction of the physical model, bypassing several design iterations, and minimising physical trials, further improving efficiency. Moreover, it set up an advanced design to be further developed, with focus on localized issues. The physical modelling allowed a more detailed analysis in more complex parts of the domain.

The study underscores the value of integrating numerical and physical modelling in hydraulic engineering projects. The collaborative approach ensured that the fish pass system met ecological objectives and target fish species requirements, while adhering to hydraulic and structural constraints. This methodology provides a blueprint for designing eco-friendly dams that balance energy production with environmental stewardship.

By leveraging an advanced combined modelling approach and interdisciplinary collaboration, the project achieved:

- Enhanced ecological connectivity for migratory fish.
- The integration of environmental and ecological considerations into dam, and respective operation, design.
- An efficient modelling methodology regarding time, costs, allowing multiple design optimisations in a short period of time, including a development of the solution during the phases of conception and construction of the physical model
- Insights into the practical application of numerical and physical modelling in complex hydraulic systems.

Nevertheless, posterior CFD model validation, using the physical model data, could be performed. Effectively, CFD simulations of the localized domain, both on laboratory scale (1:10) and prototype scale, could share insights on the ability of CFD replace some physical trials, and advance the capacity of extrapolation of CFD results to the prototype.

8. CONCLUSIONS

A comprehensive study conducted at the National Civil Engineering Laboratory of Portugal (LNEC) employed computational fluid dynamics (CFD) and physical modelling to optimise the new fish pass system of Fratel Hydroelectric Scheme. Velocity fields and flow depths across varying operational scenarios were evaluated.

The final design was considered adequate to allow migratory fish to move between the river and the powerhouse afterbay sill, even during low water levels, and reach the entrance of a future fish lock, which is expected to enable access to upstream habitats. The key elements

are a 230-metre-long fish-dedicated channel, dug along the tailrace, with five resting areas, a four-pool fishway, and a bypass channel with irregular baffle walls.

The combination of CFD and physical modelling proved essential to deal with the large domain and the range of spatial and time scales, avoiding an unfeasible large physical model, and exorbitant CFD calculations. This integrated approach significantly reduced costs and expedited design optimisation, bypassing several design iterations, and minimising physical trials. Additionally, CFD simulations provided critical pre-analysis insights for the initial physical model setup, further improving efficiency. The results from both methods provided a comprehensive a hydraulic characterization, including velocity fields and flow depths, under various operating scenarios.

The project's success was driven by the advanced modelling and the collaboration of specialists in hydraulic engineering, computational modelling, and ecological science from EDP, LNEC, and MARE/University of Évora. This interdisciplinary effort ensured that ecological and hydraulic objectives were seamlessly integrated, demonstrating the critical role of collaboration in advancing eco-friendly infrastructure.

ACKNOWLEDGEMENTS

The authors acknowledge EDP, LNEC, and MARE/University of Évora for their contributions. Special thanks are given to the multidisciplinary team, whose expertise and collaboration were essential.

REFERENCES

[1] World Commission on Dams (2000). Dams and Development: A New Framework for Decision-Making.

[2] Poff, N. L.; Hart, D. D. (2002). How dams vary and why it matters for the emerging science of dam removal. BioScience, 52(8), 659-668.

[3] Oliveira, I. C., Alexandre, C. M., Quintella, B. R., & Almeida, P. R. (2020). Impact of flow regulation for hydroelectric production on the movement patterns, growth, and condition of a potamodromous fish species. Ecohydrology, 13, e2250. https://doi.org/10.1002/eco.2250.

[4] Rato, A. S., Alexandre, C. M., Almeida, P. R., Costa, J. L., Quintella, B. R. (2021). Effects of hydropeaking on the behaviour, fine-scale movements and habitat selection of an Iberian cyprinid fish. River Research and Applications, 37(10), 1365–1375. https://doi.org/10.1002/rra.3848.

[5] Godinho, F. N., Alexandre, C. M., Costa, M. J., Oliveira, J. M., & Almeida, P. R. (2023). Hydropeaking impact assessment for Iberian cyprinids and leuciscids: An adaptation of the hydropeaking tool method. River Research and Applications, 39, 340–348. https://doi.org/10.1002/rra.3943.

[6] Almeida, P. R., Mateus, C. S., Alexandre, C. M., Pedro, S., Boavida-Portugal, J., Belo, A. F., Pereira, E., Silva, S., Oliveira, I., & Quintella, B. R. (2023). The decline of the ecosystem services generated by anadromous fish in the Iberian Peninsula. Hydrobiologia, 850, 2927–2961. https://doi.org/10.1007/s10750-023-05179-6.

[7] Alexandre, C. M., Quintella, B. R., Ovidio, M., Boavida, I., Costa, M. J., Palstra, A. P., Pedroso de Lima, R. L., de Lima, M. I. P., de Lima, J. L. M. P., & Almeida, P. R. (2023). Technologies for the study of hydropeaking impacts on fish populations: Applications, advantages, outcomes, and future developments. River Research and Applications, 39(3), 538-553. https://doi.org/10.1002/rra.4039.

[8] EDP (2018). Aproveitamento hidroelétrico de Fratel. Reperfilamento do canal de restituição e do leito do rio. Estudos hidráulicos. EDP Gestão da Produção de Energia, S.A, PRT-2018-00590, November.

[9] Muralha, A., Melo, J. F., & Ramos, H. M. (2020). Assessment of CFD Solvers and Turbulent Models for Water Free Jets in Spillways. Fluids, 5(3), 104. https://doi.org/10.3390/fluids5030104.

[10] Mendes, L. S., Lara, J. L., & Viseu, M. T. (2021b). Do the Volume-of-Fluid and the Two-Phase Euler Compete for Modeling a Spillway Aerator? Water, 13(21), 3092. https://doi.org/10.3390/w13213092.

[11] Mendes, L. S., (2022). Computational Fluid Dynamics of Aerated Flows in Spillways Chutes. PhD Thesis. Universidad de Cantabria, Santander.

[12] Quaresma, A.L.; Pinheiro, A.N. (2021). "Modelling of Pool-Type Fishways Flows: Efficiency and Scale Effects Assessment." Water, 13(6), 851.

[13] EDP (2021). Aproveitamento hidroelétrico de Fratel. Reperfilamento do canal de restituição. Esclarecimentos adicionais. EDP Gestão da Produção de Energia, S.A, December.

[14] OpenCFD (2020). OpenFOAM v2012. url: www.openfoam.com.

[15] Melo, J. F., Kacurri, E., Mendes L. S., Topore A. (2019) Modelling of Fierza Dam spillway operation and downstream effects on river bed and slopes, Dam Engineering, vol. 29, no. 2.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

SPILLWAY MODELING: COMPARING RNG AND LES TURBULENCE MODELS

Ana L. Quaresma[1]
CERIS, Instituto Superior Técnico, Universidade de Lisboa,

António N. Pinheiro[2]
CERIS, Instituto Superior Técnico, Universidade de Lisboa,

ABSTRACT

Computational fluid dynamics (CFD) studies have significantly increased in recent years due to their versatility, accuracy and advances in computer technology. CFD is nowadays a key tool for studying complex flow physics, with engineers and researchers increasingly using these models to enhance the design of hydraulic structures and verify their performance. This approach can reduce or even eliminate the need for physical models, saving time and costs. Despite this growth, few studies have compared the capabilities of different turbulence models to accurately simulate the complex flow dynamics of hydraulic structures, such as spillways. Among these models, Reynolds averaged Navier-Stokes (RANS) models are the most widely used, whereas studies using large eddy simulation (LES) models remain limited.

This study utilises a CFD model (FLOW-3D HYDRO®) to analyse the performance of two different turbulence models, the Renormalized-group (RNG) and the LES, in simulating flow over an ogee-crested WES-type spillway. The case study is the Lagdo dam spillway. The Lagdo multipurpose dam is located in Garoua Cameroon, on the Bénoué River, the Niger River main tributary. The Lagdo spillway features an ogee crest, WES type, divided into two spans controlled by radial gates, with a stilling basin as the outlet structure. The flow is diverted from the reservoir to the spillway and the hydropower intake through an approach channel, approximately, 380 m long, on the left bank.

The performance of both turbulence models is compared with the design discharge rating curve and reference values from the literature considering the approach flow conditions. The time taken to obtain the results was also assessed.

Both turbulence models showed similar performance, with comparable computation times. However, the LES model provides a more detailed characterization and analysis of fluctuating flow hydrodynamics, and coherent structures, compared to the RNG model. Based on the findings of this study, LES appears to be a viable alternative for practical applications in the design of hydraulic structures, such as spillways.

Keywords: Spillway, Computational Fluid Dynamics (CFD), Turbulence model, Re-Normalized Group (RNG), Large eddy simulation (LES).

[1]analopesquaresma@tecnico.ulisboa.pt, Portugal
[2]antonio.pinheiro@tecnico.ulisboa.pt, Portugal

1. INTRODUCTION

Spillways are essential hydraulic structures that play a critical role in the operational safety and functionality of dams by enabling the controlled release of water during high inflow conditions, mitigating flood risks, and safeguarding downstream environments and infrastructure. Their design and maintenance are paramount to improving dam and public safety. Extensive research of spillway crest shapes, pressures, and coefficients has provided empirical data that supports the design of these structures [1–3].

The ogee crest spillway is one of the most extensively studied and widely used designs for spillways worldwide, due to its favourable hydraulic properties [3,4]. Its profile minimises the formation of sub-atmospheric pressures on the spillway surface while maximising discharge efficiency [3,4]. Although the ogee crest shape and its flow characteristics are well understood, deviations from standard design parameters – such as altered approach conditions, slight modifications to the crest shape, or construction variances – can impact flow properties [3,4]. These deviations often require assessment to determine whether the deviation could adversely affect the spillway performance [4].

In the past, spillway design and assessment would mainly rely on physical modelling, interpolation/extrapolation of empirical knowledge, and prototype observation [1–6]. In recent years, Computational Fluid Dynamics (CFD) has increasingly been applied in hydraulic studies due to its versatility and accuracy in analysing flow hydrodynamics [4–12], offering the potential to reduce or even eliminate the need for physical modelling, thereby saving time and lowering costs in the design of hydraulic structures [5,6,13].

Most studies to date have relied on Reynolds averaged Navier-Stokes (RANS) turbulence models, particularly the standard k-ε model and the renormalized group k-ε (RNG) model [4–7,13,14]. However, RANS models are limited in their ability to capture complex recirculating or highly swirled flows, frequently underestimating secondary circulation and eddy effects [11,15,16]. The large eddy simulation (LES) turbulence model overcomes these limitations by directly simulating the large-scale eddies of the flow while modelling only the influence of small-scale turbulence on large-scale motion [11,17,18]. The LES turbulence model significantly improves accuracy in capturing complex flow physics and enables the analysis of instantaneous flow fields and coherent structures, capabilities that are not achievable with RANS turbulence models [11,17,18]. However, the application of LES in practical hydraulic engineering remains limited, as it typically demands significantly longer run times compared to RANS models [11,13,14,18].

Given the above, the primary aim of this study was to evaluate the performance of two turbulence models, RNG and LES, in accurately simulating the flow field over an ogee crest, WES-type spillway, and to discuss and compare both turbulence model results. The models were calibrated and compared against theoretical discharge equations and the design rating curve (based on classical empirical formulae), considering the approach flow conditions to the spillway. Additionally, the computational time required to achieve the results for both models was compared.

2. MATERIALS AND METHODS

2.1. Case study

This study assessed the performance of the RNG and LES turbulence models in accurately predicting the flow hydrodynamics of the Lagdo Dam spillway, located in Garoua, Cameroon, on the Bénoué River, the main tributary of the Niger River. The Lagdo dam's gated spillway features an ogee crest, of the WES-type, divided into two spans, each 12.00 m wide, controlled by radial gates (Fig. 1). The crest is at an elevation of 201.68 m. The outlet structure is a stilling basin. The reservoir's flow is diverted to the spillway and the hydropower intake through an approach channel approximately 380 m long, located on the left bank. The full supply level (FSL) of the reservoir is 216.0 m, with a maximum reservoir level (MRL) of 218.00 m. For this study, the spillway's performance was analysed considering the gates fully opened.

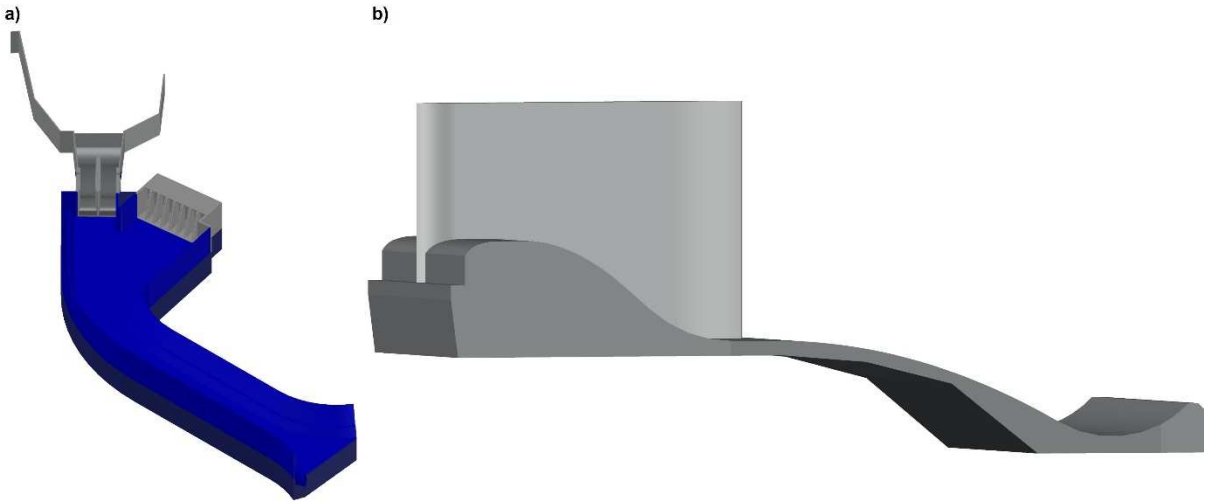


Fig. 1 – Lagdo Dam Spillway: a) Upstream side view showing the approach channel in blue and the spillway and the hydropower plant intake in grey; and b) Spillway side view detail

2.2. Numerical model

The numerical modelling of the flow field was performed using FLOW-3D HYDRO® v.2023R2, a general-purpose CFD software designed for simulating multi-physics flow problems, heat

transfer and solidification. It employs the Finite-Volume method to solve the governing equations of fluid motion on a Cartesian staggered mesh grid. FLOW-3D HYDRO® uses the FAVOR™ (Fractional Area/Volume Obstacle Representation) method to represent obstacles through fractional areas and volumes within a fixed orthogonal mesh grid [19]. Additionally, it applies the volume of fluid (VOF) method to accurately capture the free surface dynamics [20]. Average values of flow parameters (e.g. flow depth, velocity, pressure) are computed at discrete time intervals for each cell.

The geometry of the Lagdo Dam spillway was developed in AutoCAD 3D (Fig. 1) and imported into FLOW-3D HYDRO® as stereolithography (STL) files. Auxiliary blocks were considered to exclude areas irrelevant to flow hydrodynamics, thereby reducing computation time. The computational domain was discretised using multi-block mesh grids to optimise the mesh while accurately representing the geometry. Different computational domains and mesh grids were tested to ensure mesh-independent simulations and minimise the time required to achieve a time-independent solution.

A computational domain with six mesh blocks encompassing the approach channel, the HPP intake, the spillway, the outlet structure and the stilling basin, hereafter named Computational Domain 1 (CD1), was used (Fig. 2.a)). All blocks featured uniform cell sizes: the most upstream block, covering nearly the entire approach channel, had a cell size of 1.00 m. Three downstream blocks, with a cell size of 0.50 m, contained the downstream part of the approach channel, the HPP intake, and the spillway approach area. Two additional blocks, with a cell size of 0.25 m encompassed the spillway, the outlet structure and the stilling basin. CD1 contained 7 405 943 active fluid cells for the FSL test case.

Since the free flow discharge coefficient over a spillway crest is not influenced by downstream conditions [1], a smaller computational domain, referred to as Computational Domain 2 (CD2), was created. CD2 excluded the outlet structure and stilling basin mesh block being composed of five blocks encompassing the approach channel, the HPP intake, and the spillway ogee crest (Fig. 2.b)). This domain reduced the active cells by more than half, thus reducing the computation time required to obtain statistically stationary solutions and time-averaged results. CD2 had 3 021 455 active fluid cells for the FSL test case.

To evaluate the influence of mesh resolution on the results, a refined mesh was tested for CD2. This refinement halved the cell size for the spillway, spillway approach area, and HPP intake (doubling the number of cells in each direction), increasing the total number of active fluid cells to 17 126 740 for the FSL test case.

The acceleration of gravity was set in the negative z-direction. The initial pressure was set to the atmospheric pressure. The upstream boundary at the reservoir was specified as a pressure boundary condition, determined by the reservoir level being simulated. The downstream

boundary at the stilling basin (CD1) and spillway (CD2) outlets was specified as a continuative boundary (zero normal derivatives at the boundary for all quantities) representing a smooth continuation of the flow through the boundary. The bottom boundary was modelled as a wall no-slip boundary, while the side walls, top boundary and the boundaries between mesh blocks were modelled as symmetry boundary conditions (zero value for normal velocity, zero gradients for other quantities).

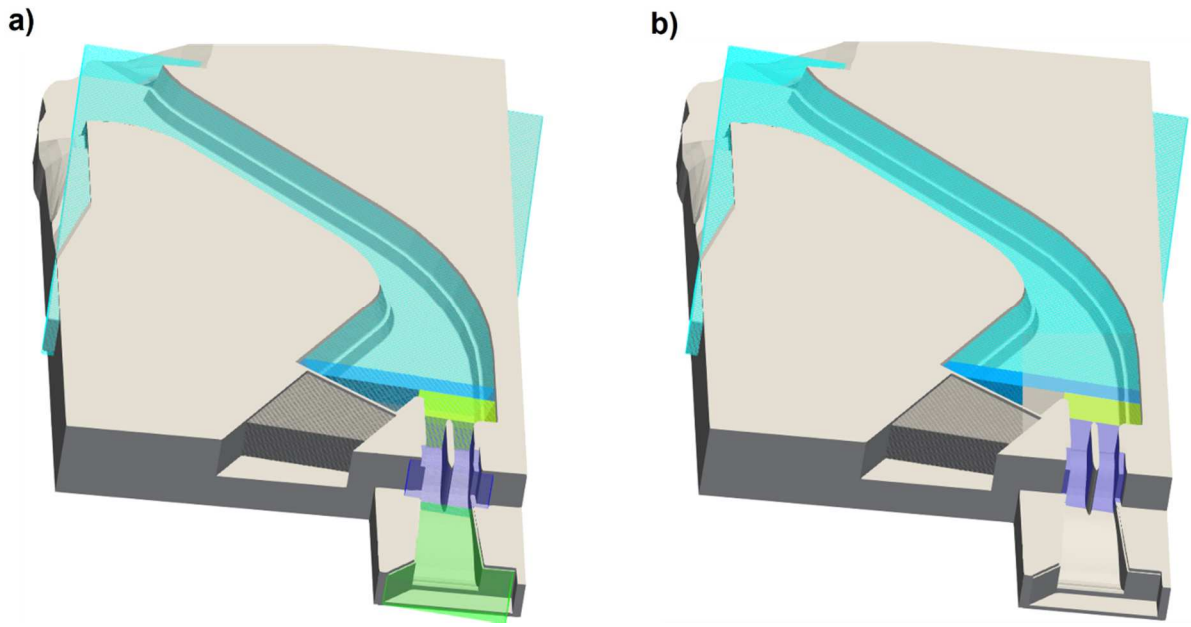


Fig. 2 – Lagdo Dam spillway numerical model computational domain (CD) and mesh blocks (Mb): a) CD1; and b) CD2. The light blue Mb encompasses the approach channel, the medium blue Mb encompasses the downstream part of the approach channel, the dark grey Mb contains the HPP intake area, the light green Mb contains the spillway approach area, the dark blue Mb encompasses the spillway, and the dark green Mb contains the outlet structure and the stilling basin. This last Mb only exists in CD1

The generalized minimum residual (GMRES) implicit solver, the default solver in FLOW-3D HYDRO®, was employed [21]. GMRES is a non-stationary iterative method, with iteration-dependent coefficients [22]. The effects of roughness were analysed by testing a range of Manning surface roughness coefficients, *n*, corresponding to the possible values of the materials.

The second-order monotonicity preserving method and the default VOF method were used. These choices were guided by results from a previous study in which various model parameters were assessed and both quantitative and qualitative comparisons between experimental and numerical results were conducted for a spillway flow controlled by an ogee crest and two radial gates [12].

Flux surface baffles were used to monitor flow parameters at specific locations, including the approach channel (up- and downstream), the spillway entrance and outlet, each spillway span, and the stilling basin (up- and downstream). Simulations were conducted over a sufficient number of time steps to achieve a statistically stationary solution and obtain converged time-averaged values. The models stabilized on average after 300 s, and the simulations were extended to 400 s to compute the time-averaged values (Fig. 3).

This study investigated the effects of the turbulence model choice on the solution by comparing the RNG, and LES models.

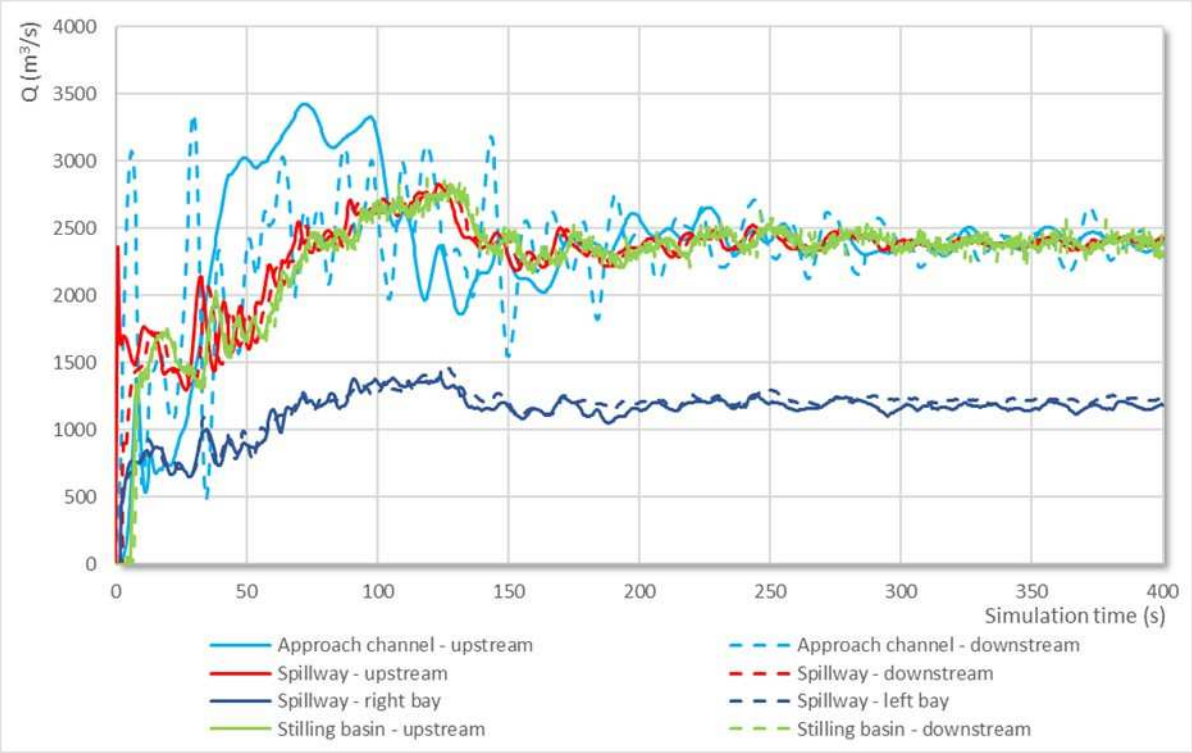


Fig. 3 – Example of discharge simulation results using the Renormalized group (RNG) model for the full supply level (FSL) test case in computational domain 1 (CD1)

2.2.1 Numerical model verification and validation

To assess the influence of the computational domain and mesh resolution on the numerical model results, comparisons were made between two computational domains, CD1, and CD2, as well as between two mesh resolutions: a coarser mesh (CM) and a refined mesh (RM).

To evaluate the influence of the computational domain, two test cases were conducted, one, for the FSL and another for the MRL. A restart simulation of the FSL test case was used to examine the impact of the mesh resolution. To validate the numerical model, a comparison was made between the numerical results and the design discharges, based on classical empirical formulae.

Table 1 summarizes the numerical model verification and validation results. The differences from the design discharges ranged within -1.7% to 0.7%, while variations between different computational domains, or mesh resolutions ranged from 0.1% to 0.4%. These differences are acceptable for the study's objectives. Consequently, the findings presented in the following section were generated using CD2 with the coarser mesh, significantly reducing the time required to complete the simulations.

Table 1 – Numerical model verification and validation results

Turbulence model	Test case	Mesh	Reservoir level (m)	Q_{design} (m^3/s)	Q_{CFD} (m^3/s)	Difference Q_{design} (%)	Difference Q_{CFD} (%)
RNG	RNG.CD1.CM.FSL	coarser	216.0 (FSL)	2 431	2 397	-1.4	-
	RNG.CD2.CM.FSL				2 399	-1.3	0.1
	RNG.CD1.CM.MRL	coarser	218.0 (MRL)	2 958	2 972	0.5	-
	RNG.CD2.CM.MRL				2 980	0.7	0.3
	RNG.CD2.RM.FSL	refined	216.0 (FSL)	2 431	2 389	-1.7	-
	RNG.CD2.CM.FSL	coarser			2 399	-1.3	0.4
LES	LES.CD1.CM.FSL	coarser	216.0 (FSL)	2 431	2 397	-1.4	-
	LES.CD2.CM.FSL				2 400	-1.3	0.1
	LES.CD1.CM.MRL	coarser	218.0 (MRL)	2 958	2 971	0.4	-
	LES.CD2.CM.MRL				2 976	0.6	0.2
	LES.CD2.RM.FSL	refined	216.0 (FSL)	2 431	2 394	-1.5	-
	LES.CD2.CM.FSL	coarser			2 397	-1.4	0.1

Difference Q_{design} indicates the differences between the numerical results and the design discharge.
Difference Q_{CFD} indicates the differences between the computational domains or mesh resolutions.

3. RESULTS AND DISCUSSION

To evaluate the performance of the RNG and LES models, a comparison was made with the design discharge rating curve (based on classical empirical formulae) and uncontrolled ogee-crested spillways flow data from previous studies [1,3,4].

Figure 4 compares the design discharge rating curve with the numerical model results. Both turbulence models show strong agreement with the design discharges, with differences of less than 1% for reservoir levels between the FSL and MRL. The results from the RNG and LES models differ by less than 0.5%.

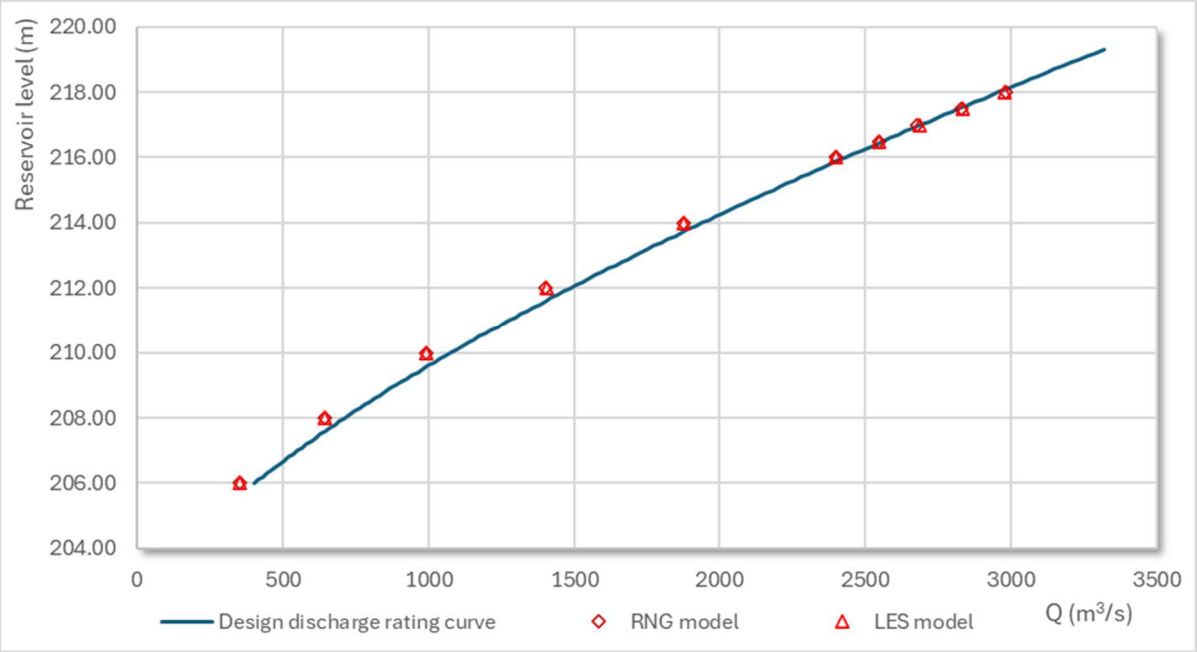


Fig. 4 – Discharge rating curve comparison

Figure 5 displays the relative difference in discharge, using the design discharge as a reference. The relative differences increased significantly at lower headwater levels, with numerical model results underestimating the discharge flow rate for reservoir levels below the FSL. Savage and Jonhson [4] when studying a standard ogee-crested spillway also found increased relative differences as headwater levels reduced.

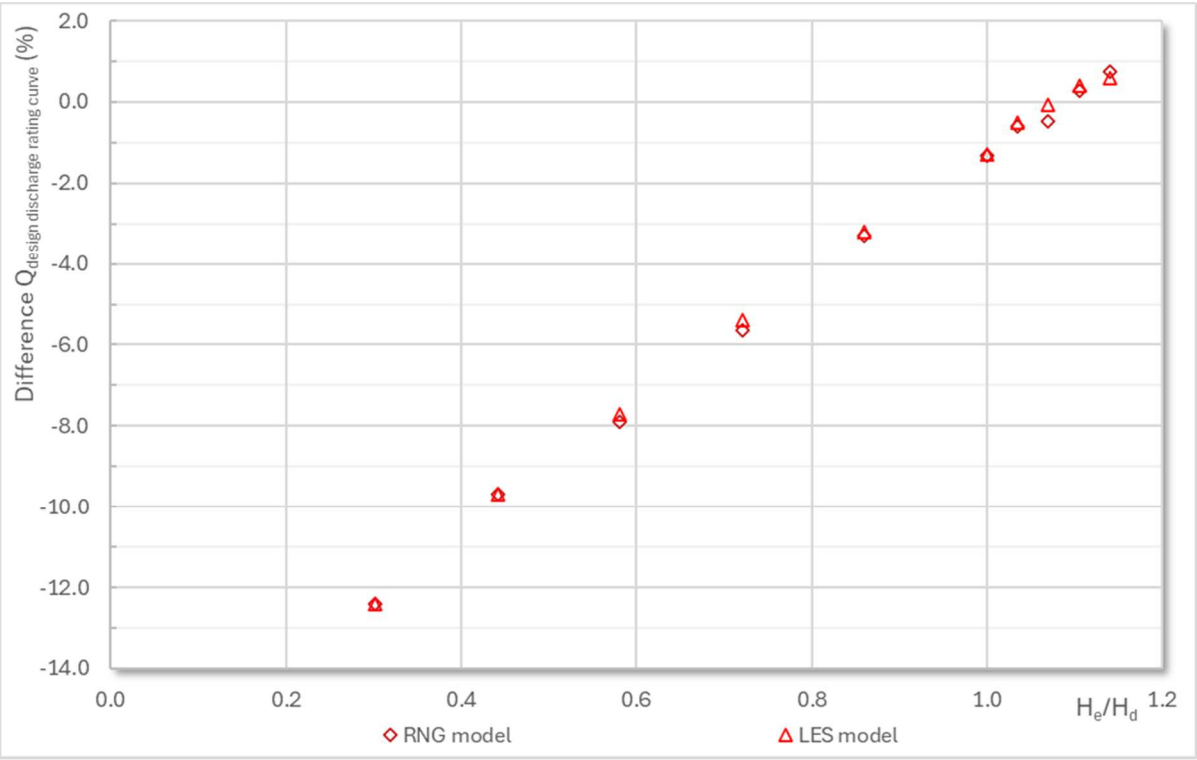


Fig. 5 – Relative percent difference in discharge using the design discharge as a basis. H_e is the effective head upstream above the crest and H_d is the design head.

Figure 6 presents a comparison of the normalized discharge simulations with results from previous studies [1,3,4]. There is a strong agreement between the numerical model results and previous studies. The design discharge rating curve shows higher values compared to all other studies and to the numerical model results for reservoir levels below the FSL. USACE [1] notes that due to possible scale effects, discharges were not measured below $H_e/H_d = 0.4$. However, prototype experience has shown that spillway crests at very low heads exhibit discharge characteristics similar to those of a broad-crested weir [1]. Additionally, as the value of P/H_d decreases (in this case study, 0.41, where P is the spillway height), the approach velocity increases, and the vertical contraction diminishes causing flow control to shift upstream. This results in a loss of efficiency, with the discharge coefficient decreasing and approaching that of a broad-crested weir [1,3].

Chanel and Doering [5] also observed an increase in the differences between 3D CFD RNG numerical model results and physical model discharges. This may help explain the larger differences observed in this study when comparing the numerical model results to the design discharge rating curve at lower headwater levels.

An advantage of using CFD is that this tool allows the analysis of the flow hydrodynamics and its main characteristics.

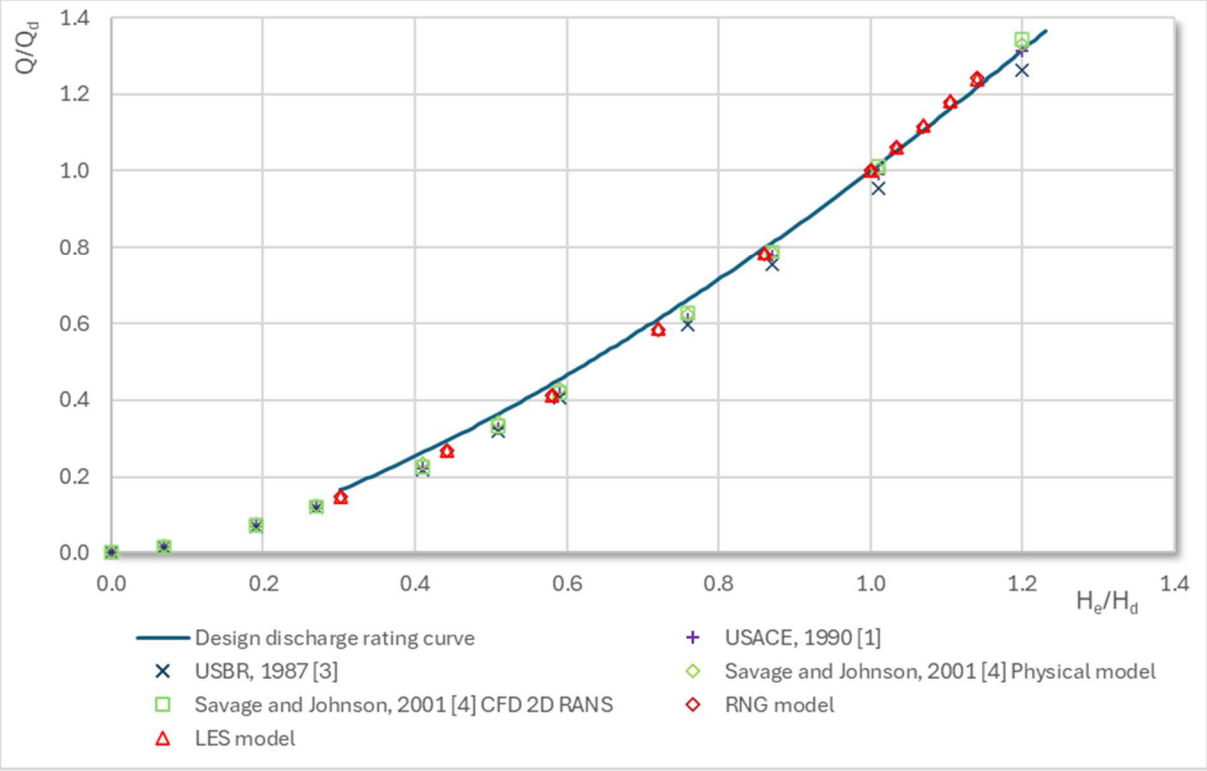


Fig. 6 – Normalized discharge comparison. Q_d is the design flow rate (based on classical empirical formulae).

Figure 7 presents the time-averaged velocity magnitude obtained with the RNG and LES turbulence models. Overall RNG and LES show a good agreement in mean velocity magnitude and flow patterns, showing the asymmetrical flow approach to the spillway, with complex flow and turbulence arising from the approach channel configuration and the HPP water intake (in the present study the HPP was considered stopped with no hydroelectricity production in all test cases). Slight differences in the swirl patterns of RNG and LES model results can be observed in Figure 7.

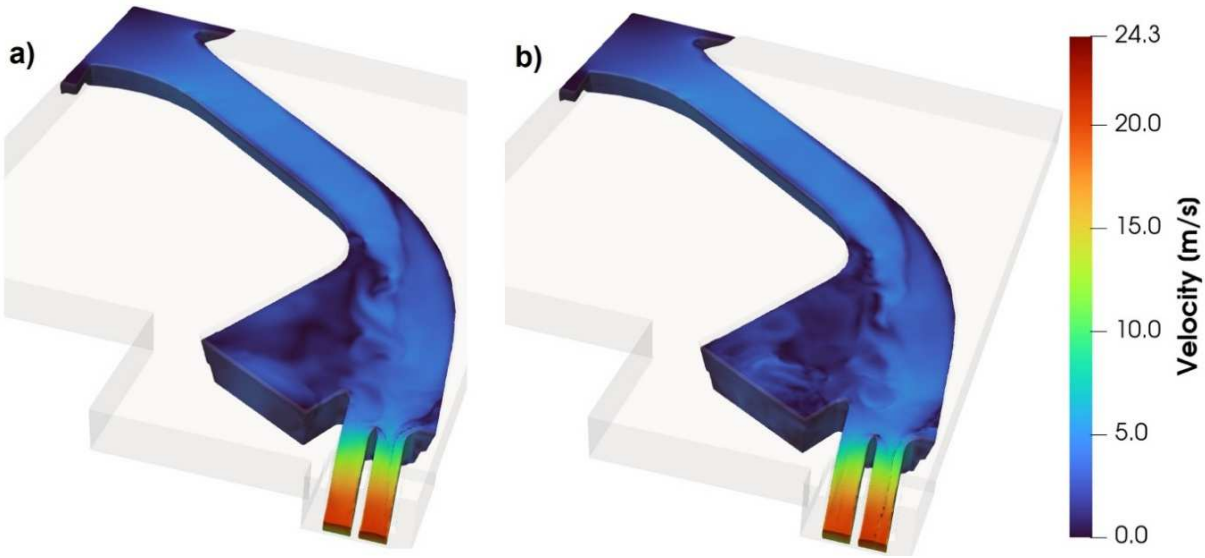


Fig. 7 – Time-averaged velocity magnitude: a) RNG; and b) LES

The unsteady nature of this flow field can be analysed with the LES model. A time series of velocity magnitude obtained with the LES turbulence model is illustrated in Fig. 8. In this figure, the evolution of vortices and recirculation regions and their influence on the spillway approach conditions can be seen.

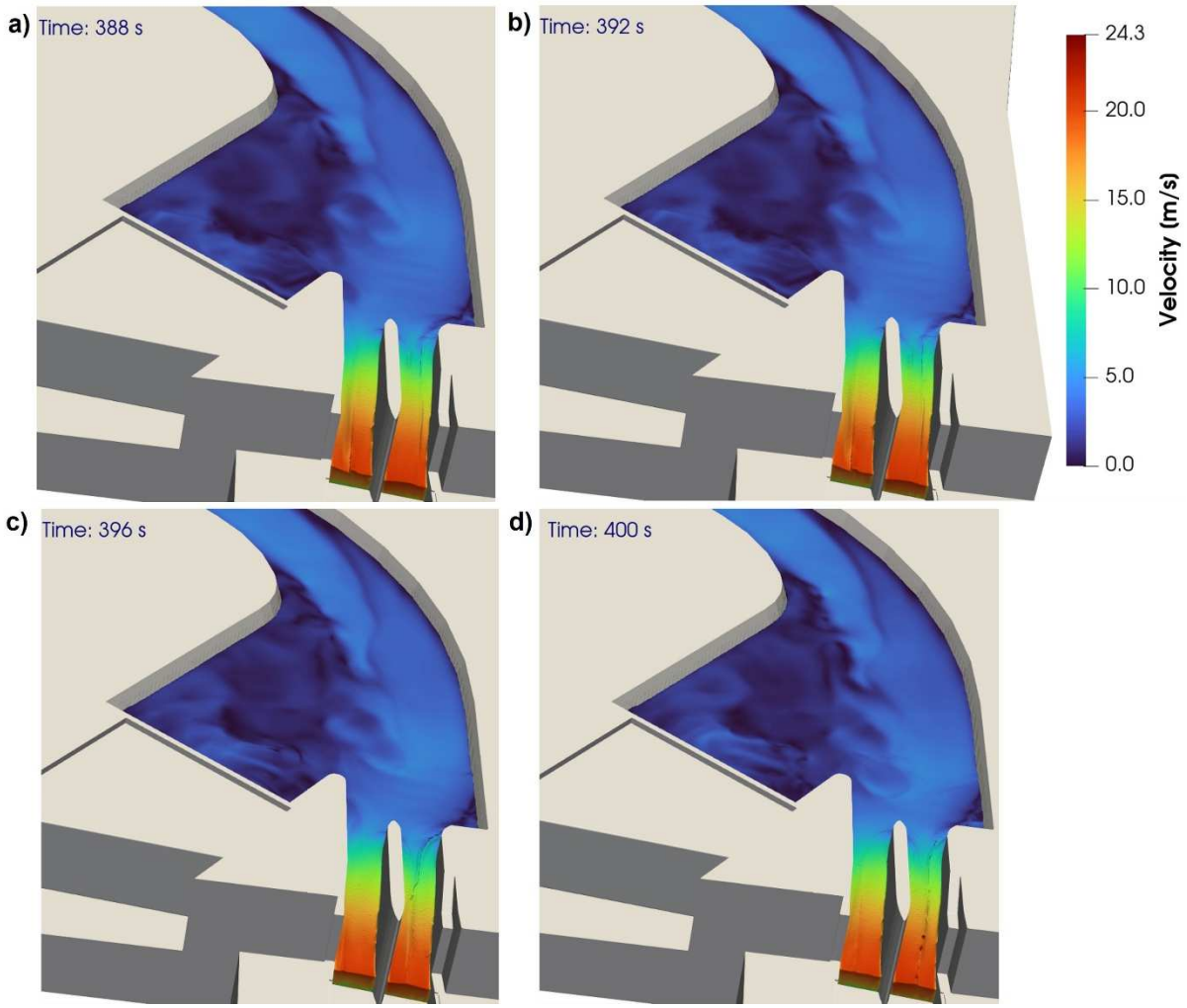


Fig. 8 – Time series of velocity magnitude obtained with LES model

Figures 7 and 8 show possible flow separation along the pier and guide walls of the spillway. The analysis of the inlet structure is important to ensure favourable approach conditions to avoid reduced discharge capacity. The time-averaged velocity magnitude in the spillway entrance for both turbulence models is shown in Figure 9. This figure confirms the flow separation along the pier and guide walls. The flows obtained in the numerical model are slightly smaller in the right bay than in the left bay. This difference is slightly larger for the RNG model than for the LES model, -4.5% compared with -4.1%. The flow is 1172 m^3/s, or 1175m^3/s in the right gate compared to 1227 m^3/s, or 1226 m^3/s in the left gate, for the RNG and LES turbulence models, respectively.

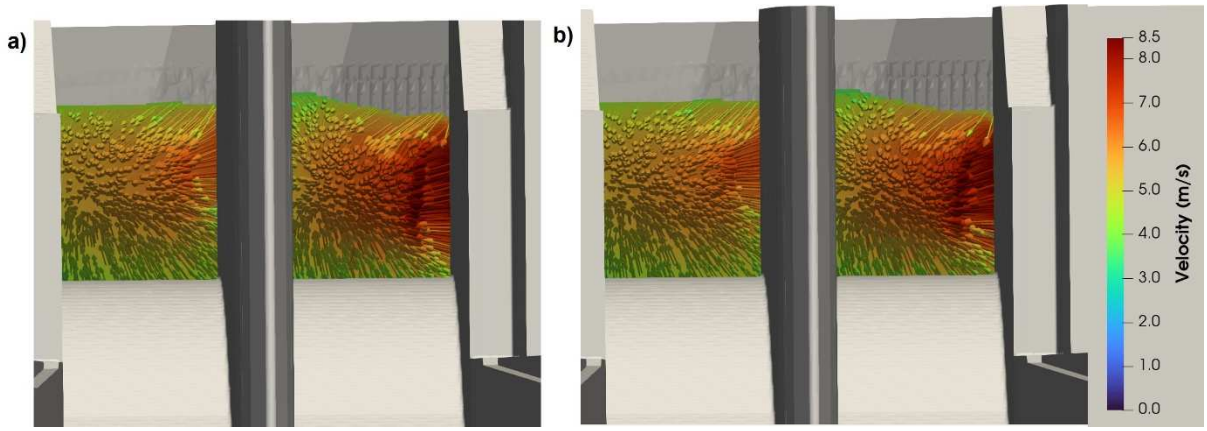


Fig. 9 – Time-averaged velocity magnitude showing velocity vectors scaled and colored by velocity magnitude in a cross-section in the spillway entrance: a) RNG; and b) LES

A pressure analysis is an important piece of spillway design and assessment, supporting structural design. Figure 10 presents the time-averaged pressure obtained with the RNG and LES turbulence models in the spillway right bay mid-span. Both turbulence models show a good agreement in pressure magnitudes and patterns.

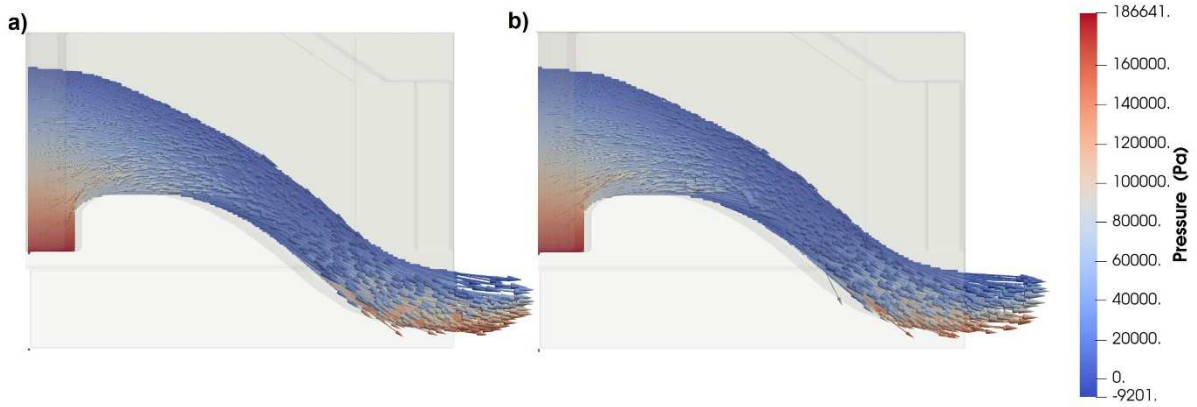


Fig. 10 – Time-averaged pressure showing velocity vectors scaled by velocity magnitude and colored by pressure in the spillway right bay mid-span: a) RNG; and b) LES

A time series of pressure in a cross-section in the spillway entrance obtained with the LES turbulence model is illustrated in Fig. 11.

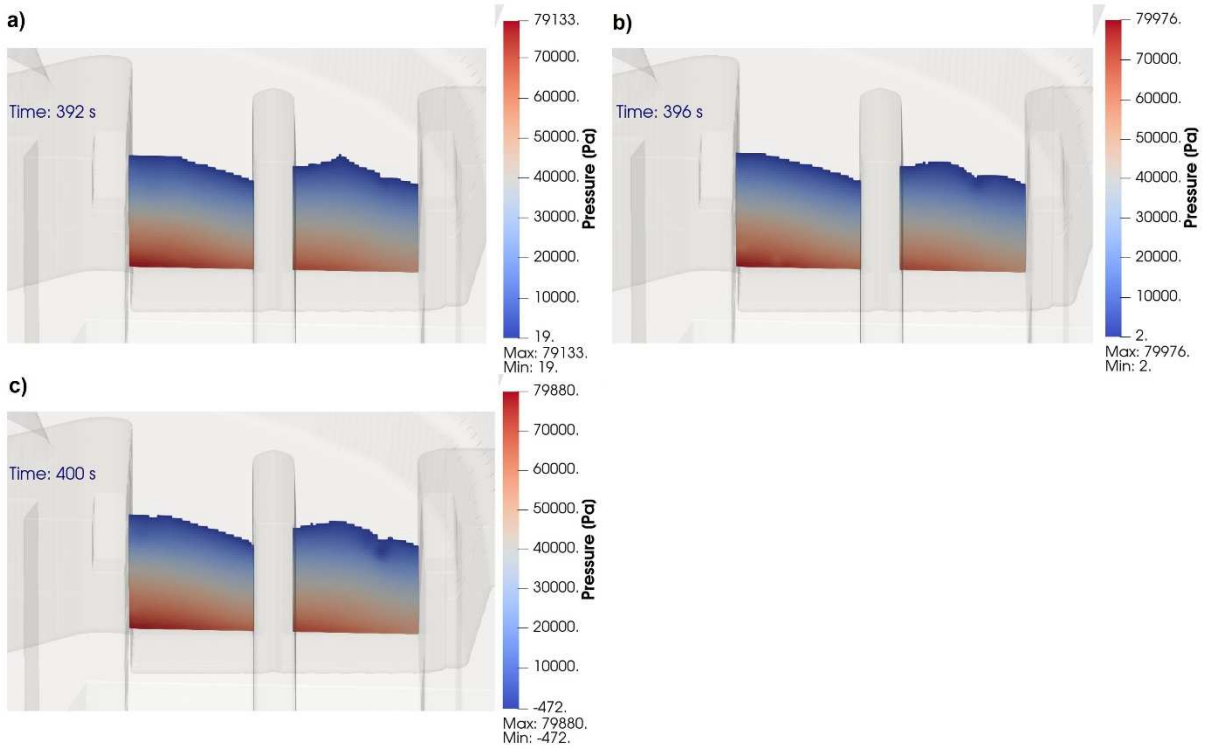


Fig. 11 – Time series of pressure in a cross-section in the spillway entrance obtained with LES model

Figure 12 presents the turbulence intensity obtained with the RNG and LES turbulence models in the spillway right bay mid-span. A good agreement was found for both turbulence models.

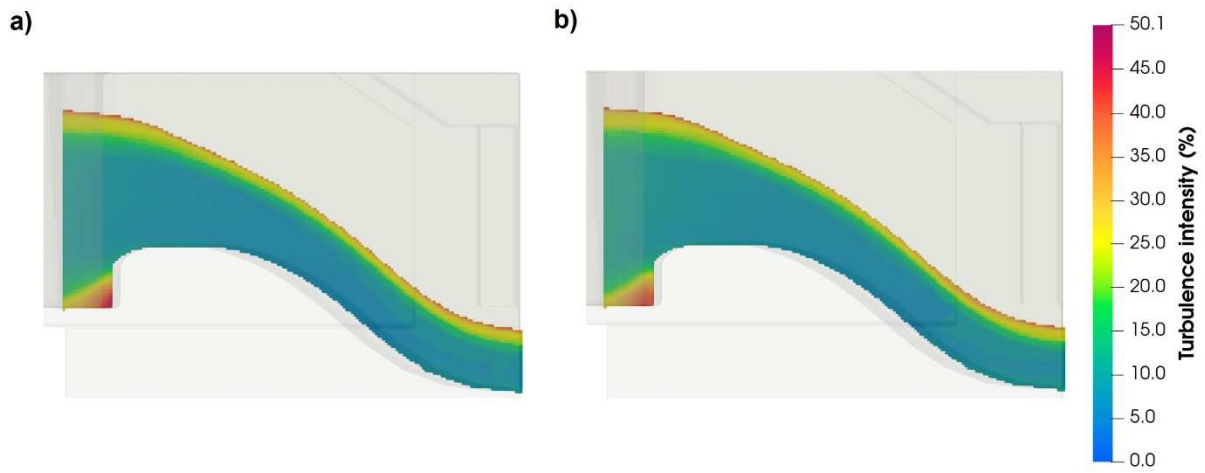


Fig. 12 – Turbulence intensity in the spillway right bay mid-span: a) RNG; and b) LES

Table 2 shows the computation details for both turbulence models. The computational time required is similar for both models, with LES simulations being slightly faster, contradicting the

general perception that the LES models require longer run times. However, LES models are typically more computationally demanding than RANS models because the former usually require finer grids to capture large eddies and the associated transient phenomena, which can lead to longer simulation times and higher computational resource requirements. In the present case, the numerical grid was the same, which allowed for a consistent basis of comparison without mesh density influencing the evaluation of turbulence model performance. Also, since FLOW-3D HYDRO® v.2023R2 uses orthogonal mesh grids, the coarsest cell sizes used were the largest possible to accurately represent the spillway crest shape. It was observed in the performed simulations that generally, the RNG model would need more iterations in each time step to attain a solution, than the LES model, which may explain the slightly longer run times. The need for more iterations to attain a solution might be related to the turbulent diffusion but this needs to be further investigated.

Table 2 – Computation details for both turbulence models

Reservoir level (m)	Number of fluid active cells	Time (h) (per 100 s of simulation time)	
		RNG	LES
206.0	1 698 830	4.7	4.3
208.0	1 954 897	5.8	5.4
210.0	2 214 947	6.9	6.4
212.0	2 479 779	8.2	7.4
214.0	2 748 824	9.6	8.9
216.0	3 021 455	10.5	9.8
216.5	3 021 455	11.0	10.0
217.0	3 159 031	11.4	10.4
217.5	3 159 031	12.2	11.1
218.0	3 301 451	12.8	12.0
Average	**2 675 970**	**9.3**	**8.6**

Processor: Intel® Xeon® Silver 4114 CPU @2.20 GHz (2 processors) x 16 cores

4. CONCLUSIONS

The main objective of this study was to compare the performance of RNG and LES turbulence models in spillway modelling. Both models demonstrated similar performance, with strong agreement to reference values from the literature. RNG and LES also showed strong agreement on flow patterns, velocity magnitudes, pressure and turbulence intensity.

The computation times for both models were comparable, with the LES model running slightly faster.

However, only the LES model can predict the instantaneous properties of the flow field. For detailed analysis of spillway performance and structural safety, as well as for design purposes, LES models offer distinct advantages, such as the ability to analyse the fluctuating flow field and crest pressures. Based on the results of this study, the LES turbulence model has proven to be a viable alternative for the design and analysis of spillways.

ACKNOWLEDGMENTS

The authors acknowledge the support of the Coordinator of the Bénoué Valley Investment Development Project (VIVA-BÉNOUÉ), representing the Minister for the Economy and Town and Country Planning (MINEPAT) for allowing the use of the Lagdo Dam spillway data as a case study. Additionally, the authors extend their gratitude to Aqualogus – Engenharia e Ambiente, Lda. for their support and for producing the AUTOCAD 3D model of the spillway. The authors are also grateful for the Foundation for Science and Technology's support through funding UIDB/04625/2020 from the research unit CERIS (DOI: 10.54499/UIDB/04625/2020). Ana L. Quaresma also acknowledges the support through funding by the Foundation for Science and Technology under the program Individual Call to Scientific Employment Stimulus (CEEC) (https://doi.org/10.54499/2022.08169.CEECIND/CP1713/CT0012, accessed on the 11th of November 2024).

REFERENCES

[1] USACE *Hydraulic Design Spillways. Engineering Design*; Engineering Manual (EM) 1110-2-1603; U.S. Army Corp of Engineers (USACE). Dept. of the Army: Washington, D.C., 1990;

{2] USACE *Hydraulic Design Criteria*; Waterways Experiment Station; U. S. Army Corps of Engineers, Waterways Experiment Station: Vicksburg, MS, 1977; Vol. I;

[3] USBR *Design of Small Dams*; Water Resources Technical Publication; 3th Edition.; United States Department of the Interior Bureau of Reclamation, 1987;

[4] Savage, B.M.; Johnson, M.C. Flow over Ogee Spillway: Physical and Numerical Model Case Study. *J. Hydraul. Eng.* **2001**, *127*, 640–649, doi:10.1061/(ASCE)0733-9429(2001)127:8(640).

[5] Chanel, P.G.; Doering, J.C. Assessment of Spillway Modeling Using Computational Fluid Dynamics. *Can. J. Civ. Eng.* **2008**, *35*, 1481–1485, doi:10.1139/L08-094.

[6] Dargahi, B. Flow Characteristics of Bottom Outlets with Moving Gates. *Journal of Hydraulic Research* **2010**, *48*, 476–482, doi:10.1080/00221686.2010.507001.

[7] Bombardelli, F.A.; Meireles, I.; Matos, J. Laboratory Measurements and Multi-Block Numerical Simulations of the Mean Flow and Turbulence in the Non-Aerated Skimming

Flow Region of Steep Stepped Spillways. *Environ Fluid Mech* **2011**, *11*, 263–288, doi:10.1007/s10652-010-9188-6.

[8] Constantinescu, G.; Kashyap, S.; Tokyay, T.; Rennie, C.D.; Townsend, R.D. Hydrodynamic Processes and Sediment Erosion Mechanisms in an Open Channel Bend of Strong Curvature with Deformed Bathymetry. *JGR Earth Surface* **2013**, *118*, 480–496, doi:10.1002/jgrf.20042.

[9] Quaresma, A.L.; Romão, F.; Branco, P.; Ferreira, M.T.; Pinheiro, A.N. Multi Slot versus Single Slot Pool-Type Fishways: A Modelling Approach to Compare Hydrodynamics. *Ecological Engineering* **2018**, *122*, 197–206, doi:10.1016/j.ecoleng.2018.08.006.

[10] Quaresma, A.; Pinheiro, A. Modelling of Pool-Type Fishways Flows: Efficiency and Scale Effects Assessment. *Water* **2021**, *13*, 851, doi:10.3390/w13060851.

[11] Rodi, W. Turbulence Modeling and Simulation in Hydraulics: A Historical Review. *J. Hydraul. Eng.* **2017**, *143*, 03117001, doi:10.1061/(ASCE)HY.1943-7900.0001288.

[12] Silva, M.R.; Couto, L.T.; Pinheiro, A.N. COMPLEMENTARY SPILLWAY OF SALAMONDE DAM. PHYSICAL AND 3D NUMERICAL MODELLING.; Porto, portugal, 2014.

[13] Burnham, J. Modeling Dams with Computational Fluid Dynamics: Past Success and New Directions.; 2011; pp. 855–893.

[14] Johnson, M.C.; Savage, B.M. Physical and Numerical Comparison of Flow over Ogee Spillway in the Presence of Tailwater. *J. Hydraul. Eng.* **2006**, *132*, 1353–1357, doi:10.1061/(ASCE)0733-9429(2006)132:12(1353).

[15] Sharif, M.A.R.; Wong, Y.K.E. Evaluation of the Performance of Three Turbulence Closure Models in the Prediction of Confined Swirling Flows. *Computers & Fluids* **1995**, *24*.

[16] Quaresma, A.L.; Romão, F.; Pinheiro, A.N. A Comparative Assessment of Reynolds Averaged Navier–Stokes and Large-Eddy Simulation Models: Choosing the Best for Pool-Type Fishway Flow Simulations. *Water* **2025**, *17*, 686, doi:10.3390/w17050686.

[17] Sotiropoulos, F. Hydraulics in the Era of Exponentially Growing Computing Power. *Journal of Hydraulic Research* **2015**, *53*, 547–560, doi:10.1080/00221686.2015.1119210.

[18] Stoesser, T. Large-Eddy Simulation in Hydraulics: Quo Vadis? *Journal of Hydraulic Research* **2014**, *52*, 441–452, doi:10.1080/00221686.2014.944227.

[19] Hirt, C.; Sicilian, J. A Porosity Technique for the Definition of Obstacles in Rectangular Cell Meshes. In Proceedings of the Proc. 4th Int. Conf. Numerical Ship hydrodynamics; Washington DC, 1985; p. 19 p.

[20] Hirt, C.W.; Nichols, B.D. Volume of Fluid (VOF) Method for the Dynamics of Free Boundaries. *Journal of Computational Physics* **1981**, *39*, 201–225, doi:10.1016/0021-9991(81)90145-5.

[21] Yao, G.F. Development of New Pressure-Velocity Solvers in FLOW-3D, Flow Science Technical Note, FSI-04-TN68 2004.

[22] Barrett, R.; Berry, M.; Chan, T.F.; Demmel, J.; Donato, J.M.; Dongarra, J.; Eijkhout, V.; Pozo, R.; Romine, C. Templates for the Solution of Linear Systems: Building Blocks for Iterative Methods. **1994**.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

OVERALL HYDRAULIC MODEL TEST OF CACULO CABAÇA HYDROPOWER PROJECT

Yang Haiyan[1]	Wang Liqun[2]	António Amador[3]	Cristina Costa[4]	Augusto Chico[5]	Liu Wen[6]
BIDR	BIDR	COBA	COBA	GAMEK	CGGC

ABSTRACT

The main dam of Caculo Cabaça hydropower project (HPP) is located in Kuanza river, the maximum dam height is 116 m with crest elevation at (633.00), and a crest length of 526 m. The dam is divided into right bank non-overflow section, spillway, bottom outlets, water intake section of ecological powerhouse and left bank non-overflow section.

The hydraulic model test of the appurtenant structures (spillway, bottom outlets and ecological powerhouse intake and tailrace canal) of the main dam was conducted to support the hydraulic design during construction design phase.

In the experimental study, the overall physical model of the main dam with a scale of 1:60 is established. The tests investigated the discharge capacity of the spillway and bottom outlets, the flow characteristics at the spillway, bottom outlets and waterway of the ecological powerhouse for different operation modes and under different flow conditions. The tests included measurements of the average pressure and velocity distribution along the hydraulic structures, measurements of water surface profiles and jet trajectories in the spillway, measurements of the flow characteristics at the water intake and tailrace canal of the ecological powerhouse and evaluation of the water levels and velocity distributions in the downstream riverbed. The tests were first conducted with fixed riverbed and then with mobile riverbed to assess the energy dissipation and scour downstream the dam.

The hydraulic model tests allowed to confirm that the discharge capacity of the spillway and bottom outlets meets the design requirements. The shapes of the hydraulic structures (spillway and bottom outlets) were adjusted and optimized based on the tests results. Finally, based on the tests results, spillway and bottom outlets operation rules and scour protection measures are suggested for the safe operation of the Caculo Cabaça hydropower project.

Keywords: Hydraulic model, Spillway, Bottom outlet, Water Intake Ecological Powerhouse.

[1] yang_hy@bidr.com.cn, China
[2] wang_lq@bidr.com.cn, China
[3] a.amador@cobagroup.com, Portugal
[4] c.costa@cobagroup.com, Portugal
[5] augusto.chico@gamek.gov.ao, Angola
[6] liuwen@cggcintl.com, China

1 PROJECT OVERVIEW

The main dam of Caculo Cabaça Hydropower Project (HPP) is arranged in the main riverbed, it is divided into right bank non-overflow section, spillway, bottom outlets, water intake section of ecological powerhouse and left bank non-overflow section.

The spillway is 102.00 m-long with five openings, each measuring 12.40 m (width) by 17.50 m (height). The spillway crest is at an elevation of (612.50). Upstream, the overflow section has a 1:1 slope (H:V), followed by a WES curve, then a 0.8:1 slope (H:V). The flip bucket is at elevation (550.00) with a 25° angle to dissipate energy [1].

The bottom outlets section is 34 m-long, with two 6.00 m diameter sections. The intake dimensions are 6.00 m-wide and 7.50 m-high, while the outlet is 5.00 m-wide and 5.10 m-high. The intake is at elevation (545.00) equipped with a bulkhead gate, and the outlet is at elevation (550.00) equipped with a radial gate. A curved flip bucket is positioned downstream of the radial gate to channel water to the riverbed centre. These hydraulic structures primarily serve to empty the reservoir and regulate downstream flow in case of powerhouse unavailability.

The intake of the ecological flow powerhouse is a dam-type intake. The bottom slab of the water inlet is at elevation (612.00). The water flows into the powerhouse through the penstock placed on the downstream face of the dam. The structures of the intake include a trash rack, inlet section, gate section, transition section, and penstock section. A flat-plate trash rack is positioned in front of the intake, and a grab-type cleaning machine is used for maintenance as needed.

The angle between the trash rack and the bottom slab of the water inlet is 80°. Downstream of the trash rack are a flat-bottom bell mouth, a bulkhead gate, an emergency gate, and a square to round transition section.

The hydraulic model tests of the dam's appurtenant structures aimed to:

- verify hydraulic operation and suggest alternative designs for better technical solutions
- evaluate discharge capacity, water surface profile, and flow regimes of water release structures
- assess spillway aeration conditions
- ensure smooth hydraulic pressure distribution along spillway and bottom outlet surfaces, avoiding dangerous negative pressures
- observe energy dissipation flow regimes and downstream scour under various conditions, recommending protective measures

2 MODEL DESIGN

2.1 Model Scale

Given the large flood discharge and reservoir area, and considering the test site and flow conditions, a physical model with a geometric scale of α=60 was selected to ensure the accuracy of observation parameters [2]. The model is based on gravity similarity criteria, maintaining the same Froude Number as the prototype, since gravity is the main force in water flow.

2.2 Model Scope

The physical model should encompass an area that replicates a flow regime similar to the prototype conditions. The height of the model must be determined by considering factors such as the highest water level, freeboard, flow gauge, erosion depth, and other relevant parameters. The primary structures to be modelled include the dam, water release structure, intake of the ecological powerhouse, downstream energy dissipator, and other permanent structures. Please refer to Fig. 1 for the model layout and Fig. 2 for photos of the physical model [2].

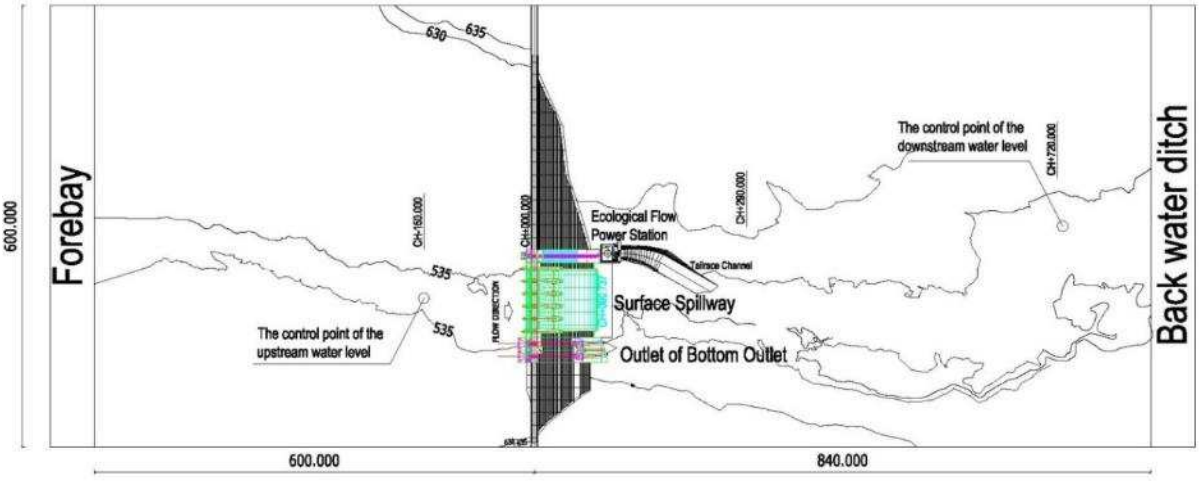


Fig. 1 - Model Layout Diagram

(1) Spillway inlet (view from upstream)

(2) Spillway intermediate pier

(3) Inlet of bottom outlet

(4) Intake of ecological powerhouse

(5) Model (view from downstream)

Fig. 2 - Photos of the physical model

2.3 Model Fabrication

The main structures of the model include the dam, water release structure, and intake for the ecological powerhouse. The spillway, piers, bottom outlet, and intake are made of plexiglass, facilitating installation and flow observation. The plexiglass has a roughness of 0.009 and a fabrication precision of ±0.2 mm, meeting the required standards. The reservoir area and river channel topography are controlled using coordinates and elevation based on a 1:500 scale topographic map and fabricated with cement mortar plaster.

2.4 Measuring Method and Accuracy

The model uses a circulated water supply system, featuring a storage pool, pumps, water level tower, and backwater channel. The system's capacity meets the project test requirements. See Fig. 3 for layout details.

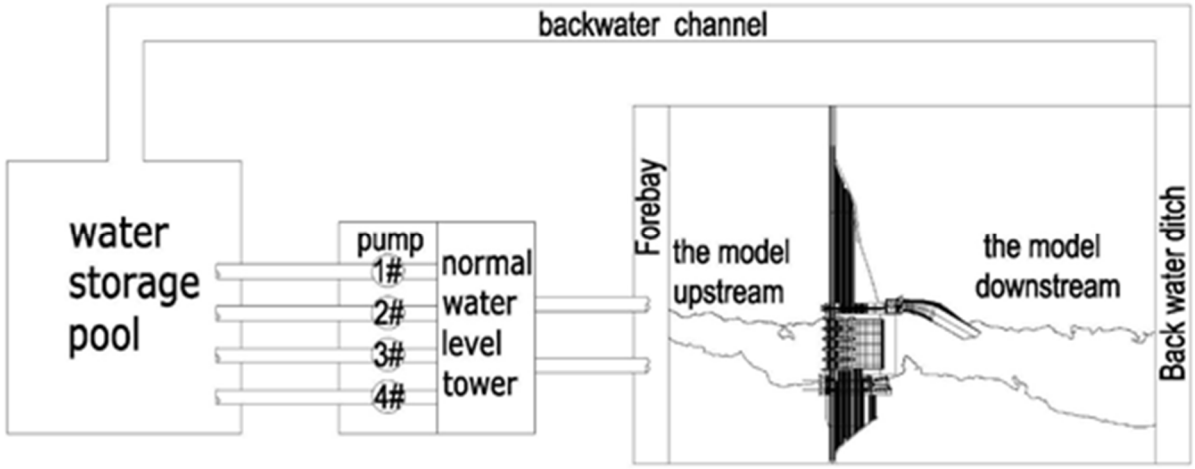


Fig. 3 - Circulated water supply system

An electromagnetic flow meter with ±0.5% precision measures flow. Water level is measured with a fixed needle (±0.1 mm precision), and water surface profile with a movable needle (±0.3 mm precision). Flow velocity is measured with a photoelectric propeller current meter (±2% precision). Time-average pressure is gauged with a piezometer (±0.5 mm precision), and air inlet with a pitot tube (±5% precision). All instruments are calibrated.

3 TESTING RESULTS OF FIXED BED

3.1 Discharge Capacity

3.1.1 Discharge capacity of spillway

When the upstream water level is at the check water level (632.0), the measured discharge flow of the spillway is 11 719.5 m3/s, which is 1.55% lower than the design calculation value of 11 904.0 m3/s. Conversely, when the upstream water level is at the full supply water level

(630.0), the measured discharge flow of the spillway is 9 900.8 m3/s, which is 0.38% higher than the design calculation value of 9 863.0 m3/s. The upstream rating curve of the spillway is shown in Fig. 4.

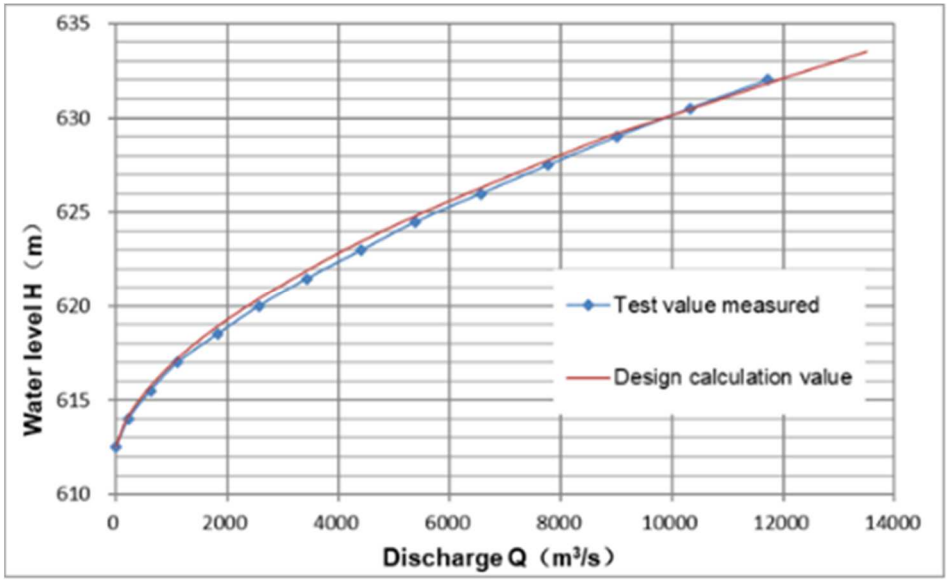


Fig. 4 - Upstream rating curve of spillway - 5 gates fully open

3.1.2 Discharge capacity of bottom outlet

When the upstream water level is at elevation (632.0), the bottom outlet discharge is 1 847.5 m^3/s, 10.37% higher than the design value of 1 673.97 m^3/s. For reservoir water level at (630.0), the discharge is 1825.5 m^3/s, 10.41% above the design value of 1 653.4 m^3/s. This shows that the bottom outlet design exceeds discharge requirements with significant redundancy. Fig. 5 illustrates the upstream rating curve.

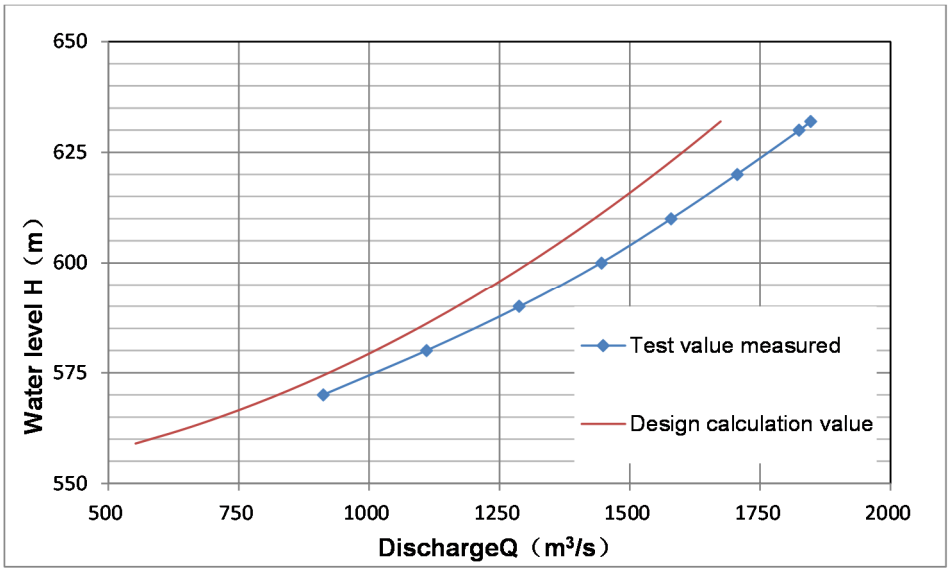


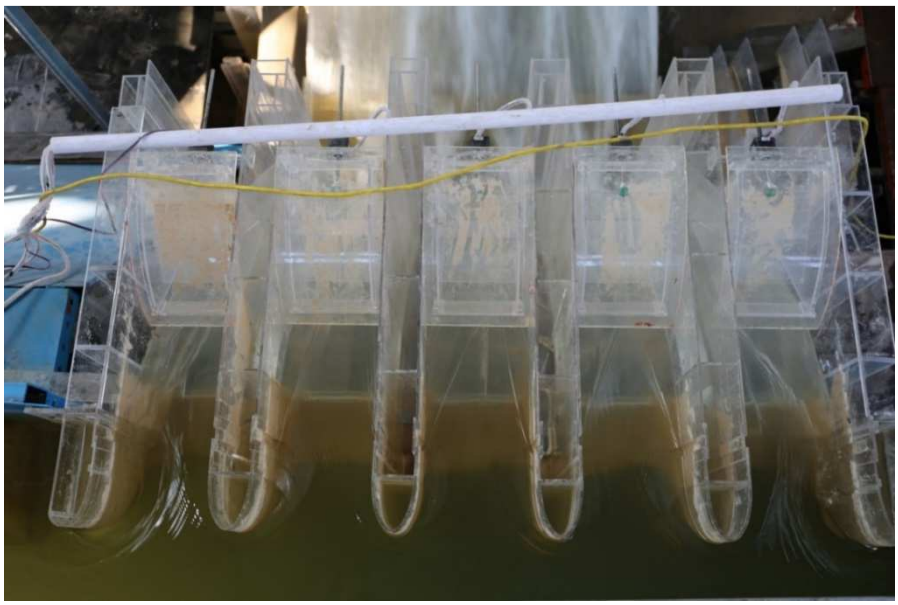
Fig. 5 - Upstream rating curve of bottom outlet - 2 gates fully open

3.2 Hydraulic Elements of Spillway

3.2.1 The Flow Regime

The water surface in front of the spillway crest remains generally stable under all conditions. As the gate opening increases, there is a slight fluctuation in the water in front of the gate. The water flows from each gate spread laterally on the spillway chute, and the water nappe formed after passing through the flip bucket falls into the downstream riverbed. Fig. 6 illustrates the flow regime under design flood conditions.

Specific tests were performed for the evaluation of the spillway aerator performance that led to some changes in the layout of this element.



(1) Flow regime of inlet

(2) Flow regime at downstream river channel

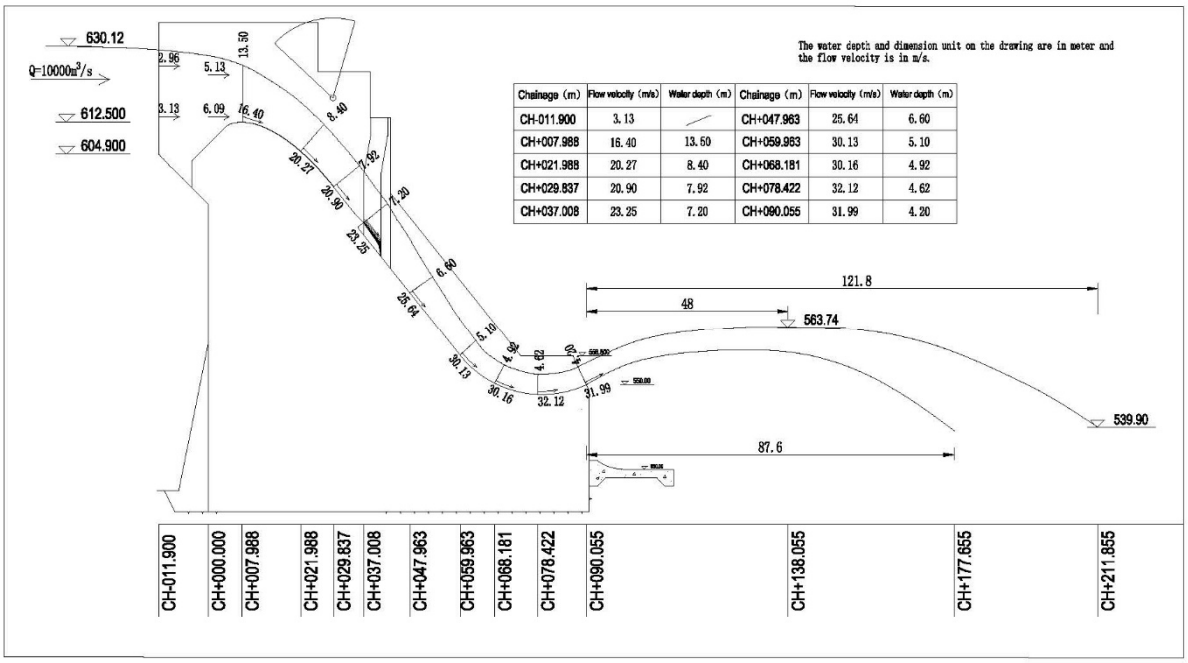
(3) Side view

(4) Flow regime on weir surface

Fig. 6 - Flow regime of design flood condition (Q=10 000 m³/s)

3.2.2 The water surface profile and distribution of flow velocity

The water surface elevation of the spillway is consistently lower than the radial gate trunnion and below the sidewall height. Fig. 7 shows the flow velocity and water surface profile under design flood conditions.



Chainage (m)	Flow velocity (m/s)	Water depth (m)	Chainage (m)	Flow velocity (m/s)	Water depth (m)
CH-011.900	3.13		CH+047.963	25.64	6.60
CH+007.988	16.40	13.50	CH+059.963	30.13	5.10
CH+021.988	20.27	8.40	CH+068.181	30.16	4.92
CH+029.837	20.90	7.92	CH+078.422	32.12	4.62
CH+037.008	23.25	7.20	CH+090.055	31.99	4.20

Fig. 7 - Distribution of flow velocity and water surface profile of spillway (Q=10 000 m³/s (design flood), H=630.12 m, all gates of spillway fully open and bottom outlet closed)

3.2.3 Time-average pressures

During the test, the time-average pressures of the spillway surfaces were measured. Fig. 8 shows the pressure distribution. The time-average pressure along the spillway's midline is

similar for all discharge conditions. The flip bucket (CH0+059.963 - CH0+083.735) has high pressure. There is no negative pressure on the spillway crest surface, intermediate pier, or abutment pier. The pressure tap 1-7 (chainage CH+047.963) is situated within the cavity formed by the jet deflection downstream of the aerator (refer to section 3.2.6). Consequently, no water pressures were recorded at this tap.

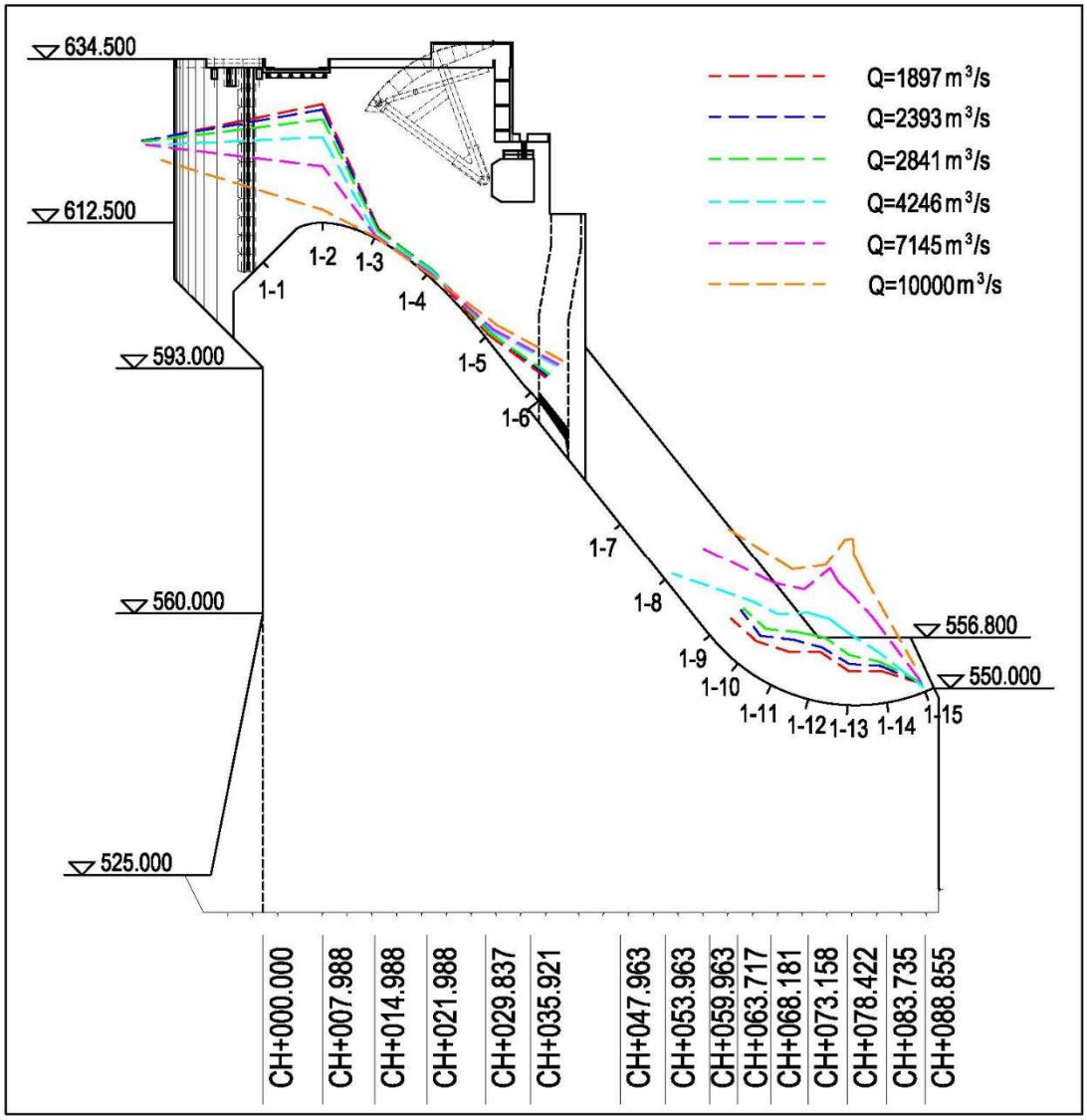


Fig. 8 - Time-average Pressure Distribution of Spillway Surface

3.2.4 Water nappe

Under all circumstances, multiple water waves are observed on the surface following the nappe discharge from the flip bucket. The left side of the water nappe impacts the tailrace canal of the ecological powerhouse, resulting in water splashes and significant erosion on the bank slope of the tailrace canal. Fig. **9** illustrates the water nappe for the condition Q=1 897 m^3/s.

Fig. 9 - Water nappe of condition Q=1 897 m³/s

3.2.5 Velocity distribution of downstream channel

During operation under various conditions, after the water nappe enters the downstream riverbed, the water surface fluctuates significantly within a certain range, and the mainstream shifts to the left side of the downstream river channel. The downstream channel velocity and water surface elevations at Q=800 m³/s are shown in Fig. 10.

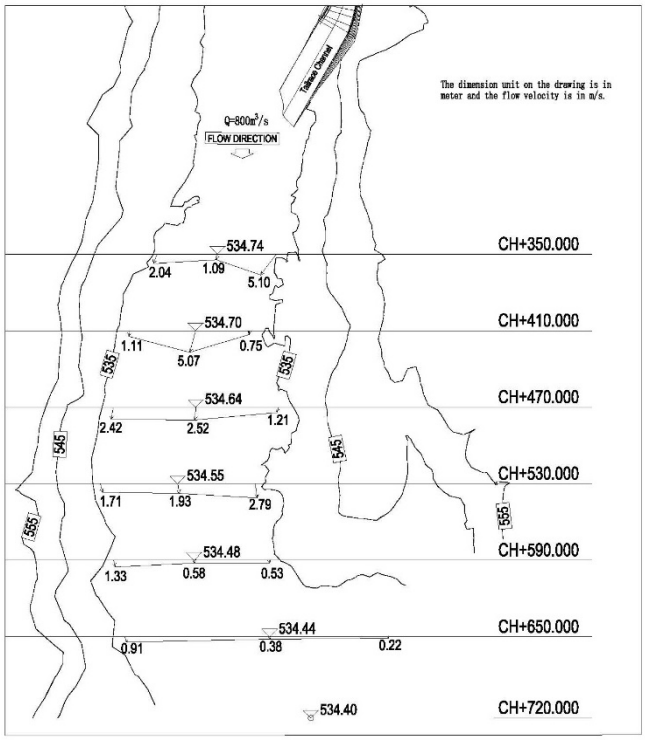


Fig. 10 - Distribution of flow velocity and water surface profile of downstream river channel at Q=800 m³/s

3.2.6 **Spillway aerators**

The Basic Design scheme of spillway aeration is as follows: ramp angle 4.86°, ramp height 0.20 m, offset height 1.533 m; the air vent is rectangular, 4.0 m-long and 1.8 m wide. Fig. 11 shows the layout of the basic design scheme [1].

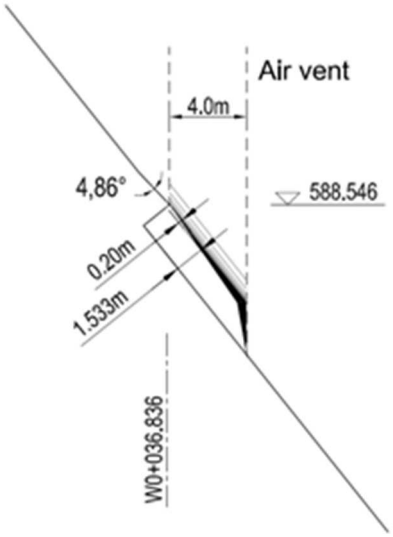


Fig. 11 - Layout of aerator in the Basic Design (unit: m)

The hydraulic calculation revealed that the Basic Design had an excessive offset height, leading to a long jet length and poor downstream flow. The optimized aerator scheme recommends a ramp height of 0.22 m and an offset height of 0.35 m. The groove size should match the air vent dimensions of 4.0 m (length) × 1.6m (width). Fig. 12 shows the layout of the optimized scheme.

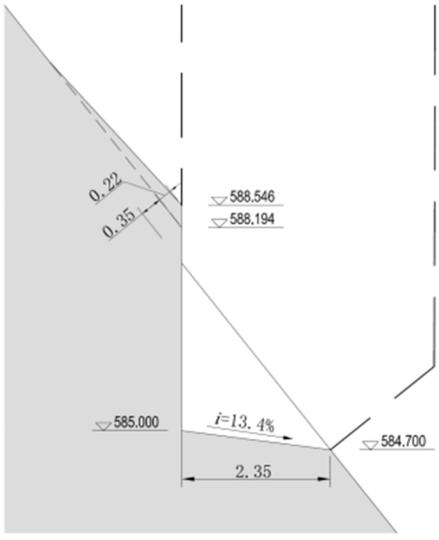


Fig. 12 - Layout of aerator for optimized scheme (unit: m)

The flow regime at the aerator under various operating conditions was observed in the hydraulic model. A stable cavity, with a length ranging from 24.3 m to 35.2 m, was formed,

being smaller at both ends and larger in the middle. The jet impact occurred upstream of the starting position of the flip bucket, with the jet direction forming an angle with the chute. The test results for the aerator were generally consistent with the hydraulic calculations. Fig. **13** illustrates the flow regime at the aerator.

Fig. 13 - Flow Regime at the Spillway Aerator (design flood)

3.3 Hydraulic Elements of Bottom Outlet

3.3.1 The water regime and surface profile

The water surface in front of the bottom outlet inlet remains stable under all conditions, with no adverse flow observed. Irregular water wings form due to the expansion of the outlet sidewalls downstream the radial gate. With the reservoir water level at elevation (632) upstream, the highest water wings range from elevation (550.10) to (550.22), which is 2.78 m lower than the radial gate trunnion elevation of (553) at chainage WS0+073.748. The outlet's leftward deflection creates a superelevation on the right side of the flip bucket. With the reservoir water level at elevation (632) upstream, the highest water surface ranges from (550.51) to (558.04), 1.96 m lower than the (560) outlet sidewall elevation. The flow regime for H=632m is shown in Fig. 14.

A special hydraulic model of the bottom outlet exit zone was built at scale 1:25 with the objective of evaluating the performance of the bottom outlet aerator, hydraulic characteristics in the curved flip bucket (velocities and water levels) as well as the cavitation risk in this zone of the bottom outlet [3].

(1) Flow regime of outlet (top view)

(2) Flow regime of water nappe

Fig. 14 - Outlet flow regime of bottom outlet (H=632m, 2 gates fully open)

3.3.2 Time-average pressures

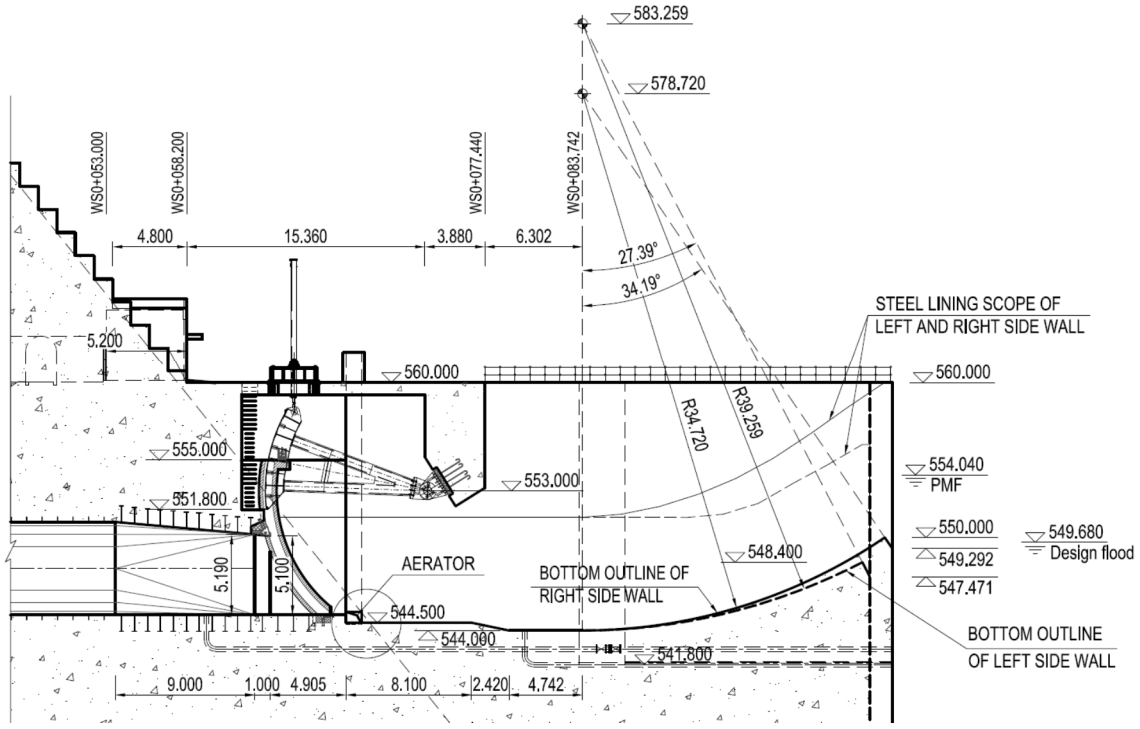
The pressure distribution indicates that the lowest pressure occurs at the top of the bottom outlet, followed by the side pressures, with the highest pressure observed at the bottom. Under all flow conditions examined, there is no negative pressure along the bottom outlet.

3.3.3 Water nappe

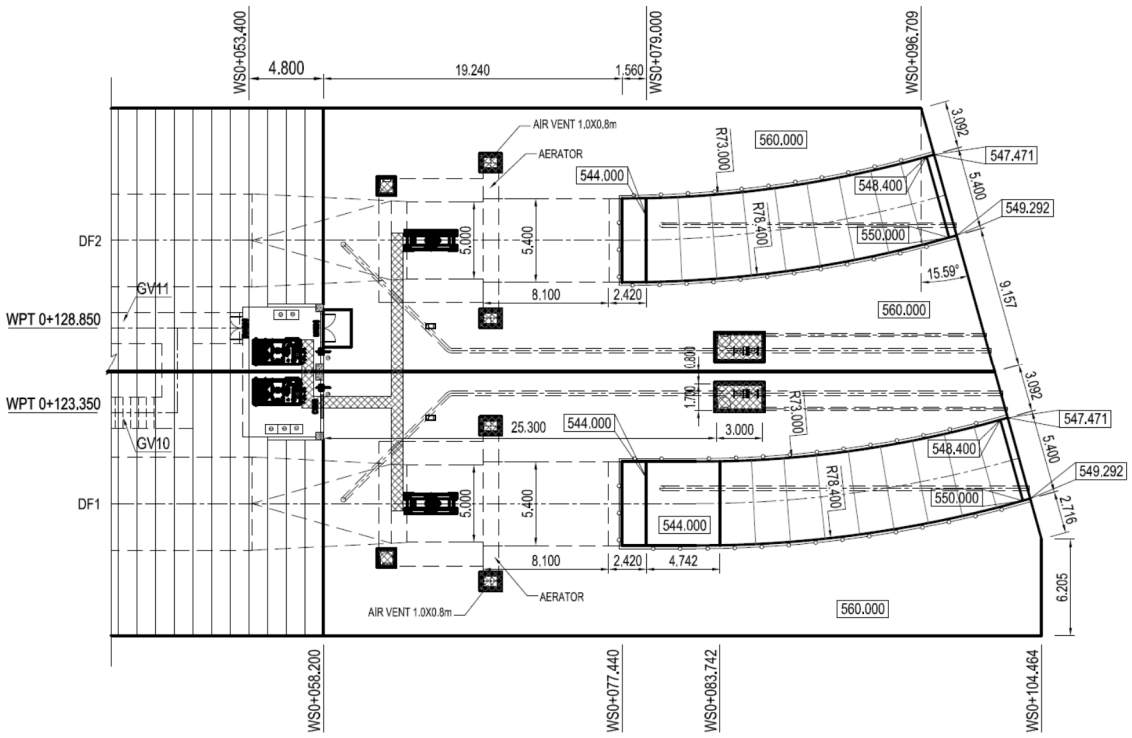
The water flow from the bottom outlet passes through a bending flip bucket section, forming an asymmetrical jet that is low on the left and high on the right side. After mixing with air, the flow spreads out vertically and horizontally. The distance from the centreline of the right-side bottom outlet to where the water nappe impacts in the riverbed is about 18 m to 22 m. The water nappe subsequently flows into the river channel near the diversion tunnel outlet.

3.3.4 Test results on aerators and jet deflectors

The average flow velocity downstream of the radial gate is nearly 30 m/s. An aerator is used to protect against cavitation damage. The recommended aerator has a drop sill and sudden expansion, each with a size of 0.5 m. To prevent backflow, a 1:4.84 slope is set downstream. The curved flip bucket at the outlet aims to direct the flow toward the riverbed centre and avoid scouring on the right bank. The recommended aerators and jet deflectors are shown in Fig. **15**.



(1) Section



(2) Plan

Fig. 15 - The recommended aerators and jet deflectors

The test results indicated that typical bottom and lateral cavities were formed under all flow conditions without backwater submerging the air vents (1.0m×0.8m). It was concluded that both the bottom cavity and lateral cavities could provide protection for the boundaries of the bottom outlet structure.

The water depths downstream of the service gate were measured. The maximum water depth along the left side wall was about 6.65 m, which was lower than the height of the side wall. On the right sidewall, the maximum depth was 9.05 m, also lower than the corresponding sidewall. The water surface remained below the trunnion elevation for all tested flow conditions, with a minimum distance of 3.02 m between the flow surface and the trunnion. The height of the sidewalls met the hydraulic requirements.

Vacuum hydraulic tests were conducted to determine if cavitation inception occurs in the bottom outlet. In vacuum tank hydraulic model tests, two methods are used to determine if cavitation has occurred: (1) Observing the presence of cavitation bubbles. (2) Comparing the sound pressure level in the high-frequency band of flow noise (typically >20kHz) at a similitude vacuum degree (ηm) with that of background flow noise (0.85ηm), specifically analyzing the sound pressure level increment (ΔSPL = SPL – SPLbg) in this frequency range. According to China's hydropower industry standards, a ΔSPL value between 5.0dB and 7.0dB serves as a criterion; a ΔSPL < 5.0dB indicates no cavitation, whereas a ΔSPL between 5.0dB and 7.0dB signifies incipient cavitation. Furthermore, a ΔSPL greater than 7.0dB suggests that cavitation

is likely formed. During vacuum tank tests, it was observed that the flow patterns around the inlet of the bottom outlet remained stable, with no typical swirl motion or cavitation flow detected. The sound pressure level in the high-frequency band of the flow noise at similitude vacuum degree (ηm) was compared to that of the background flow noise (0.85 ηm). No cavitation flow was detected in the bottom outlet when the service gate was fully opened, and no cavitation noises were recorded, as the corresponding maximum values of the sound pressure level increment (ΔSPL) at the given vacuum degree were all less than 5.0 dB. Therefore, it was concluded that no cavitation was formed.

3.4 Independent Operation of Ecological Powerhouse

When the ecological powerhouse operates independently, the water surface at the intake remains stable. Across various sections of the tailrace canal, water surface fluctuations are minimal, and flow velocity distribution is uniform. With an upstream water level at elevation (632), the vertical flow velocity in front of the trash rack ranges from 0.15 m/s to 1.08 m/s, and the flow velocity in the tailrace canal varies from 0.39 m/s to 2.45 m/s.

4 TESTING RESULTS OF MOBILE BED

4.1 Mobile rock bed range and characteristics

Scope of mobile rock bed: The rock bed extends approximately 360 m downstream of the main river channel's apron. The foundation of the riverbed is weakly weathered gneiss with catallactic zone, the bedrock erosion resistance velocity $\boldsymbol{V_k}$ selected is 8 m/s [4]. The erosion causes the rock with fractures to blocks, and the diameter of the blocks is calculated according to the formula $\boldsymbol{V_k = (5 \sim 7)d^{1/2}}$, [4] and the model's erosion resistance sphere size should range from 2.2 to 4.3 cm (with an average size of 3.25 cm). For the experiment, gravel with an average sphere diameter of 3.25 cm is selected as the bedrock erosion resistance material. Fig. 16 illustrates the arrangement of the mobile bed in the model.

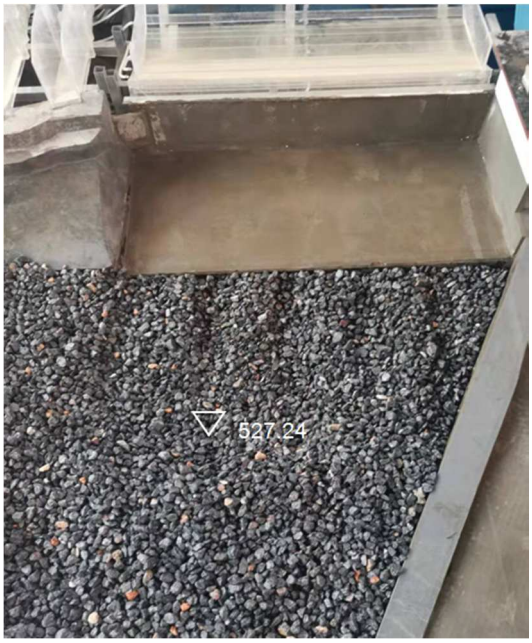
Fig. 16 - Mobile Bed Test Model

4.2 Scouring Pattern

Refer to Fig. 17 **- Mobile bed scouring of various flow conditions**

7 for the preliminary scouring pattern of the mobile bed under various flow conditions. Table 1 presents the maximum scour depth of the riverbed and the elevation of the deepest point under these different flow conditions.

The mobile bed tests are still ongoing, and the results here presented are preliminary. In fact, the interference of the spillway jet with the tailrace canal of the ecological powerplant needs to be better understood and the mobile riverbed needs to be extended to this zone of the model to assess the protective measures that have to be implemented.



(1) Q=800 m^3/s

(2) Q=2841 m^3/s (10-year flood)

(3) Q=7 145 m^3/s (10 000-year flood)

(4) Q=10 000 m^3/s (design flood)

Fig. 17 - Mobile bed scouring of various flow conditions

Table 1 – The maximum scour depth of riverbed

Cases	Discharge (m^3/s)	Chainage of the maximum scour depth of riverbed	The elevation of the deepest point(m)
1	Q=800 m^3/s	CH+165.89	527.24
2	Q=2 841 m^3/s	CH+174.05	513.98
3	Q=7 145 m^3/s	CH+241.05	496.40
4	Q=10 000 m^3/s	CH+248.89	490.28

5 CONCLUSIONS AND SUGGESTIONS

(1) The test results confirm that the design of the spillway and bottom outlet, including the discharge capacity, trunnion elevation and sidewall height, is reasonable.

(2) It is recommended to prioritize the use of the spillway for flood discharge, ensuring that all spillway gates are operated with equal openings. The maximum gate opening should be less than 10 m. For the design flood (10 000 m^3/s) it is recommended that all gates are fully opened, and bottom outlet gates are closed. The contribution of the bottom outlet for flood discharge should start by the left side gate.

(3) During spillway discharge under various flow conditions, the jet landing point on the left side impacts the tailrace canal of the ecological powerhouse, causing erosion to the platform and bank slope. The tests with mobile bed model are still ongoing to better assess the protective measures that need to be implemented.

(4) The design of the dam's associated structures was validated through hydraulic model tests. Adjustments and optimizations were made to the spillway aerator, bottom outlet aerator, and jet deflectors to ensure their efficient hydraulic performance.

REFERENCES

[1] COBA, “Detailed Design for the Tender of the Caculo Cabaça Hydropower Project”, Lisbon, 2015.

[2] BIDR/CSPDR, “Overall Hydraulic Model Test for Main Dam”, document AHCC-PE-CC-BAR-HID-NT-001-EE-06, Tianjin, 2024.

[3] BIDR/CSPDR, “Special Hydraulic Model Test for the Bottom outlet”, document AHCC-PE-CC-BAR-HID-NT-002-EE-02, Tianjin, 2024.

[4] China Water&Power Press, “Handbook of Hydropower Engineering Geology”, Beijing, 2011.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

STRENGTHENING KEY WATER INFRASTRUCTURE IN MOZAMBIQUE: RISK ANALYSIS OF MASSINGIR DAM DOWNSTREAM VALLEY

Sílvia Amaral[1] LNEC, Portugal
Fernanda Rocha LNEC, Portugal
Teresa Viseu LNEC, Portugal
Dinis Juízo SALOMON, Mozambique
Francisco Saimone SALOMON
Lurdes Pimenta COBA, Portugal
Fernando Miguel ARA, Sul, Mozambique
Ivan Cuna ARA, Sul, Mozambique
Luís Mabunda DNGRH,Mozambique

ABSTRACT

The Massingir Dam, the second largest in Mozambique, is a 4,600-meter-long and 47-meter-high earthfill dam. Located on the Elephants River, a tributary of the Limpopo River, it primarily supports irrigation in the Lower Limpopo Valley. The study of the propagation of the flood wave was developed along 85 km, from the section of Massingir dam until the confluence of the Elephants and Limpopo rivers. Within this length, the river expounds a well-defined valley, that broadens significantly over the final 50 km before its confluence, where the floodplain is used for agriculture.

The risk analysis of the downstream valley of Massingir Dam focused on evaluating the flow propagation along the Elephants River under four flood scenarios: the maximum discharge capacity of the main spillway and three distinct dam failure scenarios. The failure scenarios generate catastrophic flood waves marked by exceptionally high-water levels, velocities, and flow rates, posing severe risks to the surrounding areas. The study provides a detailed comparative analysis of the results, emphasizing the maximum wave heights and the extent of flooding across different scenarios. Additionally, it assesses the primary impacts on affected populations, economic assets, infrastructures and the environment.

By modelling these scenarios, the study aims to provide a comprehensive understanding of the risks associated with the dam's failure and support the development of effective mitigation strategies.

Keywords: Massingir Dam; Risk Analysis; Dam Failure; Downstream Valley; Inundation Maps.

[1]samaral@lnec.pt, Portugal

1. INTRODUCTION

The Massingir Dam, managed by ARA-Sul, is the second-largest dam in Mozambique. It is situated on the Elephants River, a tributary of the Limpopo River, in Gaza Province (Fig. 1). The dam serves multiple purposes, including irrigating 90,000 hectares, flood control, hydroelectric power generation with a capacity of 30 MW, and providing water supply to the town of Massingir. The catchment area at the dam site spans 67,540 km², accounting for 16.55% of the total Limpopo River basin area of 408,250 km².

The reservoir has a maximum storage capacity of approximately 2,800 hm³ and covers 138 km² at Normal Water Level (NWL). The original crest level was (130) but was raised to (131), during the rehabilitation (2006-2008).

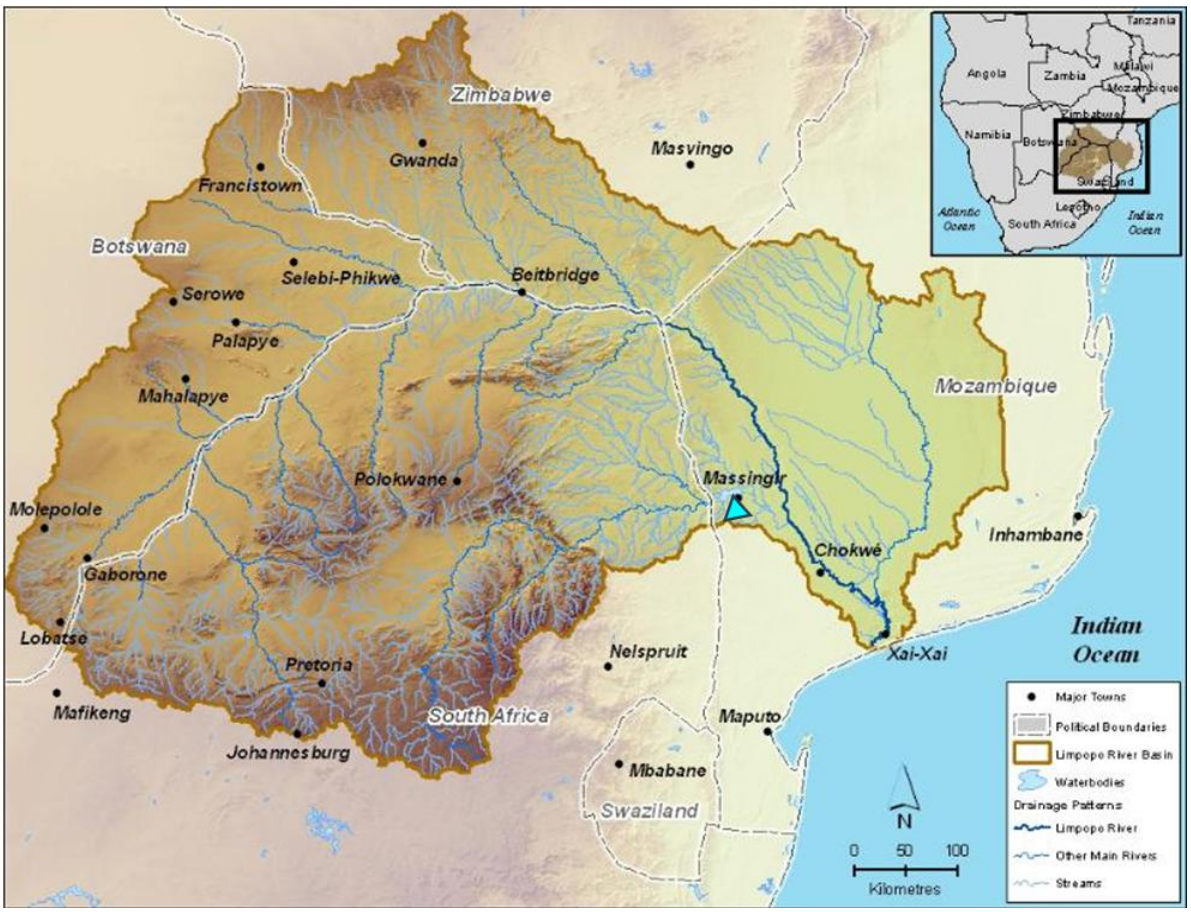


Fig. 1. Location of the Massingir dam in the Limpopo River Basin.

This zoned earth-fill dam has a crest length of 4,596 m and a height of 46 m (Fig. 2). The main spillway, designed for 10,000 m³/s, features an ogee control structure with six 18 m bays with radial gates, a 475 m concrete chute, and a hydraulic jump stilling basin. The auxiliary spillway on the upper left bank has a 151.9 m crest with 14 Hydroplus fuse gates (10.85 m each). The bottom outlet and water intake system, located on the lower right bank, include an intake tower, a gallery housing conduit, and a downstream stilling basin ([1], [2]).

Fig. 2. Massingir dam (Google Earth view).

The purpose of this work is to present the results of the flood study caused by considering different scenarios: i) the maximum discharge capacity of the main spillway (10 000 m^3/s) and ii) three different failures of the dam. All scenarios result in catastrophic flood waves, characterized by extremely high-water levels, velocities, and flow rates. The study included a thorough evaluation of the flood waves characteristics along the river, combined with insights in what concerns major impacts.

2. DOWNSTREAM VALLEY

a) General

The studied section of the Elephants River spans approximately 85 km, from the Massingir Dam to its confluence with the Limpopo River (Fig. 3). The Elephants River, roughly half the length of the Limpopo River, descends steeply from an elevation of 1,500 m at its source in South Africa to 80 m at the Massingir Dam. Downstream from the dam to the Limpopo confluence, the river flows through a well-defined valley that gradually widens in the final kilometres before merging with the Limpopo River ([3] and [4]).

b) Housing

The Massingir district is sparsely populated, with minimal human settlement in the downstream valley of the dam. The left bank of the river below the Massingir Dam is predominantly occupied by the Limpopo National Park, established in 2001. The park is home to a rich variety of wildlife, including elephants, rhinos, and buffaloes, as well as a diverse range of species.

Ecotourism is a growing focus in the region, with significant investments being made to develop this sector.

The modal type of housing in the district is traditional, with earth walls and a zinc sheet roof, without access to drinking water or electricity. On the left bank, there are several villages, namely Chibotane/Machanguene and Muôno and, further downstream, Barrio, Mazengue and Macaringue, the latter close to the confluence with the Limpopo River. Other villages are located on the right bank of the river, notably Chinhangane and, along the road connecting Macarretane to Massingir, the settlements of Boazi, Bathine, and Josefo can also be found.

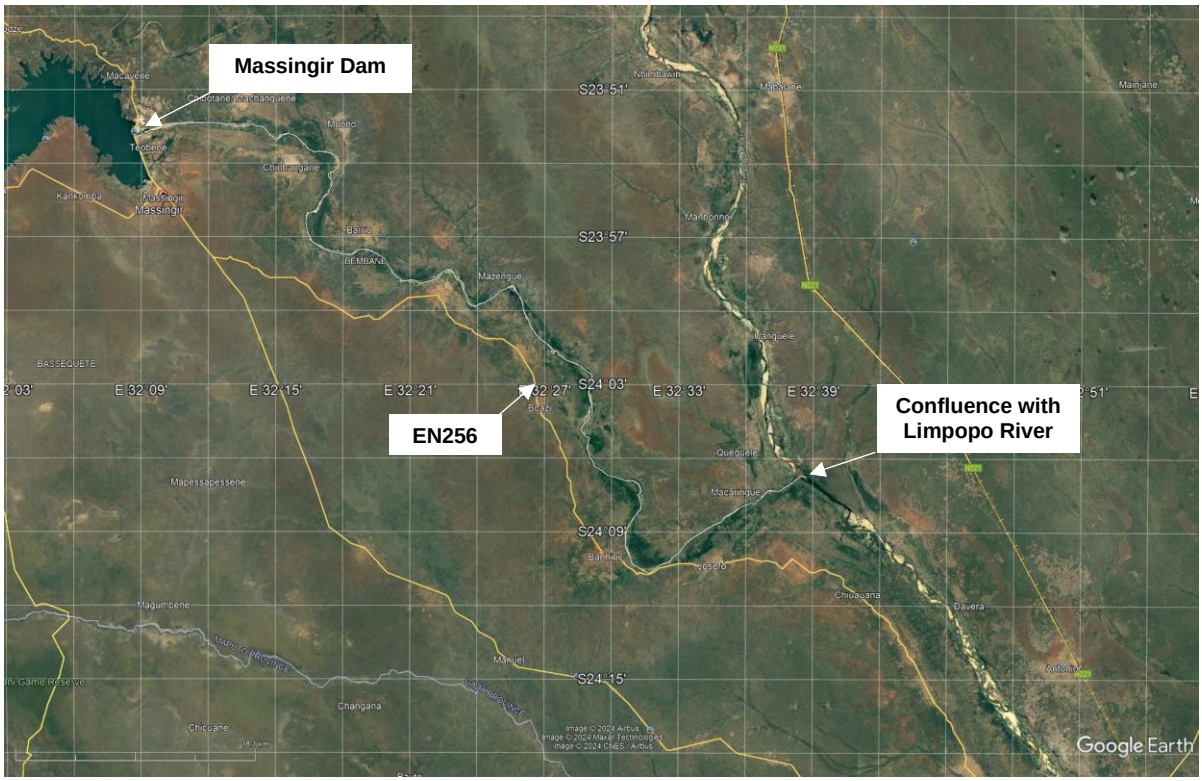


Fig. 3. Elephants River from dam cross-section up to the confluence with the Limpopo River.

c) Economic activities

The Elephants River's valley is predominantly composed of alluvial and sandy plains interspersed with stretches of savannah, providing considerable potential for community-based agricultural development. Despite the fertility of these plains and the region's suitability for irrigated agriculture, agricultural activity remains relatively limited and population settlements downstream of the Massingir Dam are mainly concentrated along the river's floodplains, following the distribution of wetlands.

On both banks of the Elephants River, a strip of family-run farms extends downstream from the dam. These agricultural areas, interspersed with patches of savannah, primarily cultivate crops such as corn, beans, and vegetables. On the right bank, the topography offers larger alluvial areas, which are suitable for more expansive agricultural activities.

This region features a mix of traditional farming and more intensive agricultural operations. For example, the Bembane agricultural area is one of the notable traditional farming zones. Additionally, intensive production farms, such as the Massingir Citrus farm (located near km 2) and the Banhine farms (located between km 60 and km 70), rely on irrigation systems like center-pivot sprinklers to support their crops.

These agricultural areas highlight the region's significant potential for both traditional and commercial farming, emphasizing the importance of water resources and irrigation systems in sustaining local agricultural activities.

d) Communication routes and critical infrastructures

On the right bank, the road number EN 256, from Macarretane to Massingir, was identified, which follows the river along a relatively long stretch between Boazi and Banhine. It is worth noting that the dam's crest allows access between the riverbanks, connecting Massingir Velho to the dam and to EN 256.

On both banks, there are pathsthat provide access from the aforementioned villages to the Elephants River, along which there are areas for cultivating maize, beans, and vegetables.

Besides the dam, the main critical infrastructures are those associated with agriculture, due to the economic value they represent for the local population, including the water abstraction infrastructure for the pivots in the Banhine farm, the pivots themselves and the irrigation channel that carries water from the Elephants River to the reservoirs that feed the pivots.

3. METHODOLOGY AND DATA

3.1. Failure Scenarios and spillway maximum discharge capacity

The classic mechanisms of failure for earth dams include overtopping, foundation failure and internal erosion of the embankment, foundation or dam abutments.

In overtopping failures, the flow over the structure causes deep cuts into the crest and erosion of the downstream slope material. These processes lead to the formation of a weak zone, which can result in a local collapse of the dam, and the deep cuts may extend down to the base of the embankment. Regardless of the zone of formation, this weak spot will initially tend to have a triangular shape. Once the less resistant material of the dam is eroded, and if high flood levels persist, erosion continues laterally, leading to a final trapezoidal shape. The extent of this lateral erosion depends on the reservoir's size and the volume of water stored. In the case of large reservoirs, like the Massingir Dam, the embankment erosion process will continue as long as water remains in the reservoir, potentially leading to a massive failure of the embankment.

Failures caused by internal erosion are not uncommon in earthfill dams and may, in some cases, be detected in time to be controlled. In this type of failure, soil particles from the embankment are carried away by water flowing through the dam's body, forming one or more internal channels. As these channels enlarge due to the flow, the discharge and erosive power of the flow increase. This progressive process generally leads to the channel(s) becoming so large that the overlying materials collapse, forming a single open channel.

There are many criteria and empirical rules for defining the possible scenarios involving failures in embankment dams. The failure of the Massingir Dam due to internal erosion was considered. Froehlich (1995) equations ([5]) were employed to determinate the breach parameters associated with a worst-case scenario to evaluate the maximum potential flood impacts in the downstream valley of the Elephants River, based on the reservoir volume (2844x106 m^3) and in the dam heigh (46 m). These equations are commonly utilized to estimate the critical parameters associated with a breach in a dam. The formulas utilized are as follows:

$$t_f = 0{,}00254 V_w^{0{,}53} h_b^{-0{,}9} \quad (1)$$

$$B_{bottom} = 0{,}1803 k_0 V_w^{0{,}32} h_b^{0{,}19} \quad (2)$$

In this worst-case scenario, the failure led to a trapezoidal breach with the following dimensions: B_{bottom}=550 m, B_{Top}=742 m and H_{Breach}=46 m. The breach formation time, estimated using the empirical formula (1), was approximately 8 hours. Based on these characteristics, the maximum outflow at the dam's cross-section was estimated at 150,000 m^3/s.

Given the significant uncertainty in defining the breach dimensions, which directly affect the hydrodynamic characteristics of the dam-break flood wave - particularly water depths and flow velocities - a sensitivity analysis was performed to assess the influence of these variations on the results.

The analysis considered smaller breaches, leading to varying maximum outflows at the dam's cross-section for different failure scenarios:

- Scenario 1 - Worst case Scenario – Q_{max}=150 000m^3/s
- Scenario 2 - Intermediate Impact Scenario – Q_{max}=75 000m^3/s
- Scenario 3 - Minimum Impact scenario – Q_{max}=25 000m^3/s

Additionally, an extra scenario (Scenario 0) was considered, corresponding to the maximum discharge capacity of the main spillway – Q_{max}=10 000m^3/s.

3.2. Hydraulic simulations and digital terrain model

The simulations were conducted using the Hec-Ras numerical model (https://www.hec.usace.army.mil/software/hec-ras/), where the one-dimensional unsteady flow is solved using the mass conservation equation and the momentum conservation equation of Saint-Venant along the x direction (USAGE, 2023 – [6]):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l \quad (3)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (QV)}{\partial x} + gA\left(\frac{\partial z_s}{\partial x} + S_f\right) = 0 \quad (4)$$

In these equations, A represents the total flow area (m²); t is the time (s), Q is the discharge (m³/s), q_t are the lateral inflows per unit length, x is the distance (m), Z is the depth (m), and S_f is the friction force, traditionally obtained using the Manning-Strickler equations.

The flood was modeled along an 85-kilometer stretch of the river, with the following boundaries:

- **Upstream boundary**: Massingir Dam cross-section (km 0.0)
- **Downstream boundary**: Confluence of the Elephants and Limpopo Rivers (km 85.0).

For the flood propagation analysis, the Elephants River downstream of the Massingir Dam was characterized using cross-sections obtained from the Shuttle Radar Topography Mission (SRTM) dataset, a global Digital Elevation Model (DEM) with a 30-meter resolution. It is important to note that the primary limitation in the flood propagation analysis lies in this topographic data used, which has a relatively low level of detail.

In fact, although the SRTM DTM provides valuable topographic information, its 30-meter resolution may not capture fine-scale variations in the terrain, which are critical for accurately modeling flood propagation and its impacts. For instance, smaller topographic features such as localized depressions, small embankments, or subtle changes in slope may not be fully represented, potentially leading to a less precise prediction of water flow behavior and flood dynamics.

4. RESULTS

4.1. Flood characteristics

The results for all scenarios are presented in Fig. 4, which shows the evolution of maximum water elevations along the downstream valley of the Massingir Dam. Table 1 summarizes the flow characteristics derived from simulations of both the spillway's maximum capacity and the three failure scenarios.

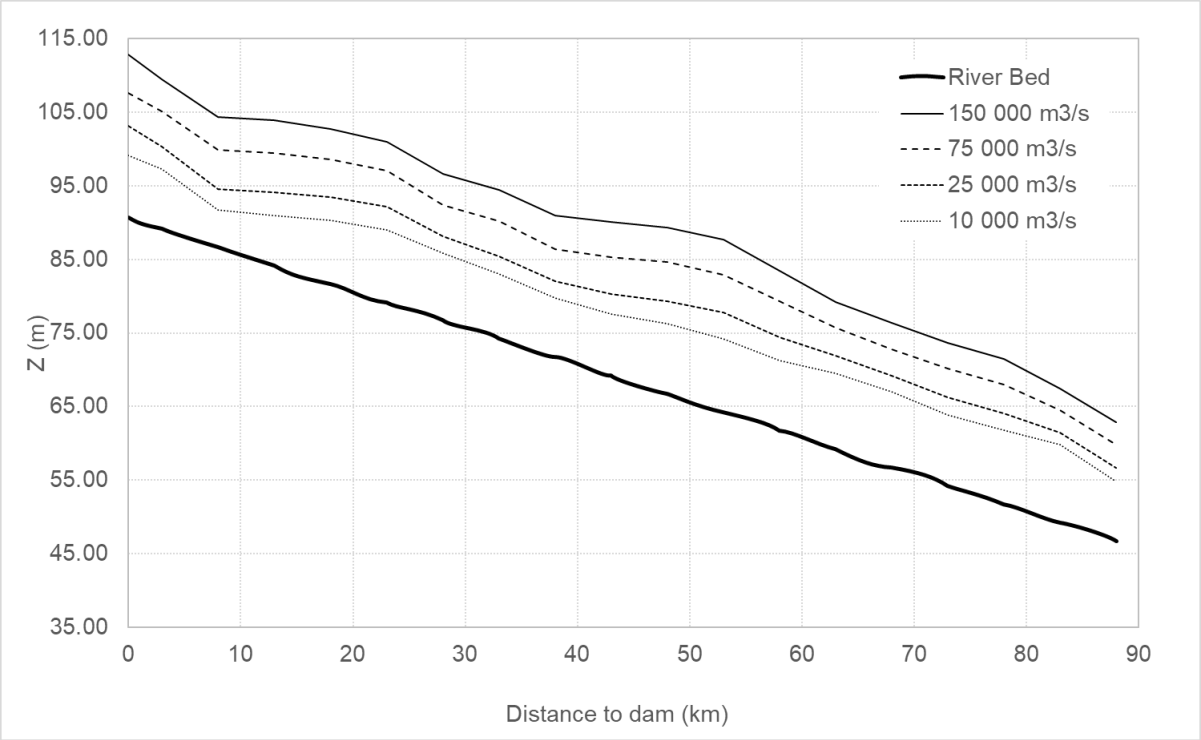


Fig. 4. Massingir Dam failure Scenarios and maximum discharge of the spillway. Comparison of maximum water levels as a function of the distance to the dam.

For the main spillway's discharge capacity of 10,000 m³/s, the maximum water elevations (Z_{water}) along the studied stretch ranged from (99) downstream of the dam to (54.8) at the confluence with the Limpopo River, with flow velocities varying from 2.1 m/s near the dam to 0.97 m/s at the confluence. The water depth (h_{water}) reached approximately 8.4 m just downstream of the dam and 8.0 m at the confluence with the Limpopo River.

The **maximum water elevations** along the studied stretch vary as follows:

- **Downstream of the dam**: (112.9) for the Worst-Case Scenario (150,000 m³/s), (107.6) for the Intermediate Impact Scenario (75,000 m³/s), and (103.1) for the Minimum Impact Scenario (25,000 m³/s).
- **At the confluence with the Limpopo River**: (62.9) for the Worst-Case Scenario, (59.9) for the Intermediate Impact Scenario, and (56.7) for the Minimum Impact Scenario.

The corresponding **flow velocities** along the studied stretch vary as follows:

- **Downstream of the dam**: 6.9 m/s for the Worst-Case Scenario, 6.3 m/s m for the Intermediate Impact Scenario, and 3.1 m/s for the Minimum Impact Scenario.
- **At the confluence with the Limpopo River**: 2.2 m/s for the Worst-Case Scenario, 1.7 m/s m for the Intermediate Impact Scenario, and 1.2 m/s for the Minimum Impact Scenario.

Table 1 – Maximum discharge capacity of the main spillway (10 000m³/s) and dam-break flood characteristics in the downstream valley of Massingir dam

		150 000 m³/s			75 000 m³/s			25 000 m³/s			10 000 m³/s		
Distance to dam	Z_{bed} (m)	Z_{water} (m)	h_{water} (m)	v_{max} (m/s)	Z_{water} (m)	h_{water} (m)	v_{max} (m/s)	Z_{water} (m)	h_{water} (m)	v_{max} (m/s)	Z_{water} (m)	h_{water} (m)	v_{max} (m/s)
0	90,70	112,87	22,17	6,93	107,61	16,91	6,29	103,14	12,44	3,09	99,14	8,44	2,06
3	89,20	109,44	20,24	6,43	105,10	15,90	4,88	100,30	11,10	2,82	97,26	8,06	1,85
8	86,70	104,33	17,63	2,72	99,92	13,22	1,96	94,62	7,92	1,34	91,71	5,01	1,26
13	84,20	103,89	19,69	2,02	99,52	15,32	1,46	94,15	9,95	0,98	90,97	6,77	0,84
18	81,70	102,70	21,00	3,06	98,63	16,93	2,26	93,49	11,79	1,36	90,34	8,64	1,02
23	79,20	101,01	21,81	2,64	97,10	17,90	2,19	92,21	13,01	1,53	89,04	9,84	1,25
28	76,70	96,60	19,90	2,84	92,38	15,68	2,45	88,11	11,41	1,98	85,89	9,19	1,63
33	74,20	94,46	20,26	3,04	90,17	15,97	2,44	85,40	11,20	1,74	82,99	8,79	1,30
38	71,70	91,01	19,31	2,02	86,43	14,73	1,74	82,04	10,34	1,39	79,70	8,00	1,09
43	69,20	90,10	20,90	1,65	85,35	16,15	1,31	80,25	11,05	1,00	77,57	8,37	0,84
48	66,70	89,33	22,63	1,72	84,60	17,90	1,33	79,32	12,62	0,94	76,26	9,56	0,78
53	64,20	87,73	23,53	3,09	82,95	18,75	2,82	77,84	13,64	2,02	74,21	10,01	1,91
58	61,70	83,40	21,70	3,64	79,32	17,62	2,89	74,45	12,75	2,00	71,22	9,52	1,44
63	59,20	79,17	19,97	2,46	75,77	16,57	1,94	71,89	12,69	1,32	69,47	10,27	0,93
68	56,70	76,32	19,62	2,68	72,73	16,03	2,16	69,13	12,43	1,51	66,97	10,27	1,19
73	54,20	73,66	19,46	2,04	70,11	15,91	1,62	66,20	12,00	1,22	63,87	9,67	0,99
78	51,70	71,42	19,72	2,55	68,01	16,31	1,97	64,04	12,34	1,39	61,77	10,07	1,07
83	49,20	67,42	18,22	3,19	64,53	15,33	2,38	61,47	12,27	1,55	59,77	10,57	1,13
88	46,70	62,91	16,21	2,17	59,86	13,16	1,73	56,67	9,97	1,22	54,80	8,10	0,97

4.2. Consequences in the downstream valley

Fig. 5 presents the inundation map for the maximum discharge capacity of the main spillway, while Fig. 6 illustrates the potential flooding area in the event of a Massingir Dam failure under the Worst-Case Scenario.

The flooded area is clearly larger in the Worst-Case Scenario, although the differences may not be immediately apparent. This is mainly due to the relatively narrow downstream valley, essentially in the first kilometres of the propagation, which limits the lateral extent of flooding. However, significant differences are observed in the submersion depth. In more severe failure scenarios, water depth increases substantially, emphasizing the greater impact of these events. The flood depth is notably higher in the Worst-Case Scenario, indicating that while the horizontal spread of flooding remains similar, the vertical inundation is far more pronounced in scenarios with higher discharge rates.

In fact, water depths reach approximately the following average values:

- **20 m** for the Worst-Case Scenario (150,000 m³/s)
- **16 m** for the Intermediate Impact Scenario (75,000 m³/s)
- **12 m** for the Minimum Impact Scenario (25,000 m³/s)
- **9 m** for the main spillway discharge capacity (10,000 m³/s).

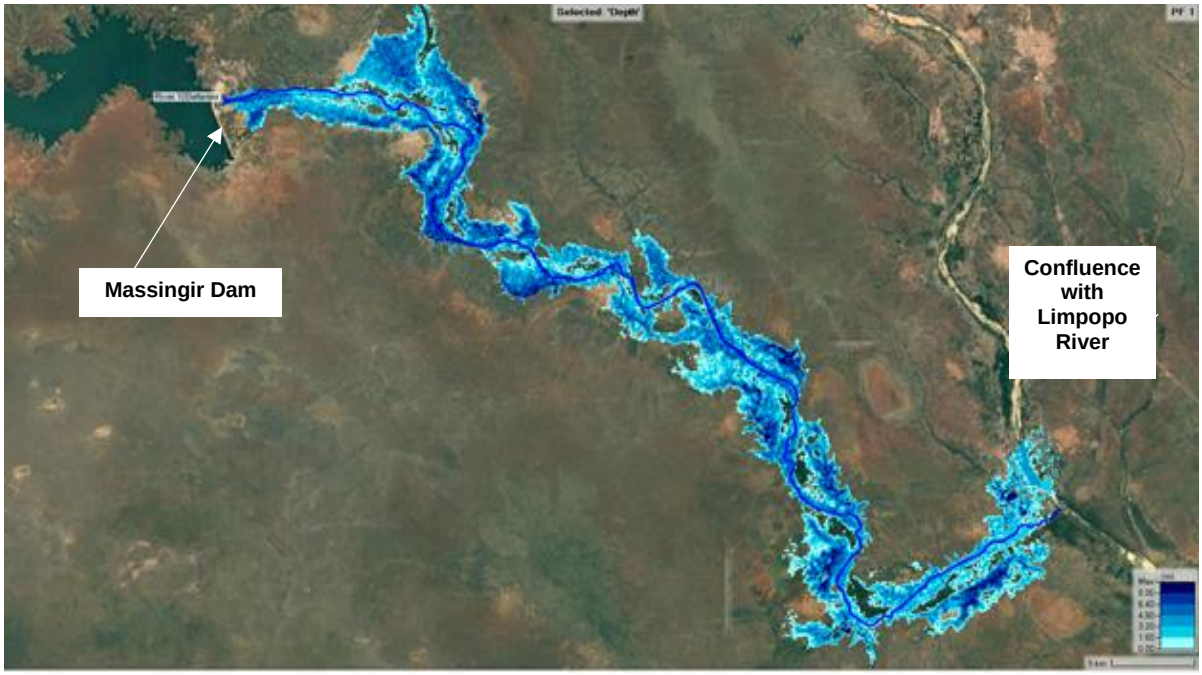


Fig. 5. Massingir maximum discharge capacity of the main spillway. Inundation map

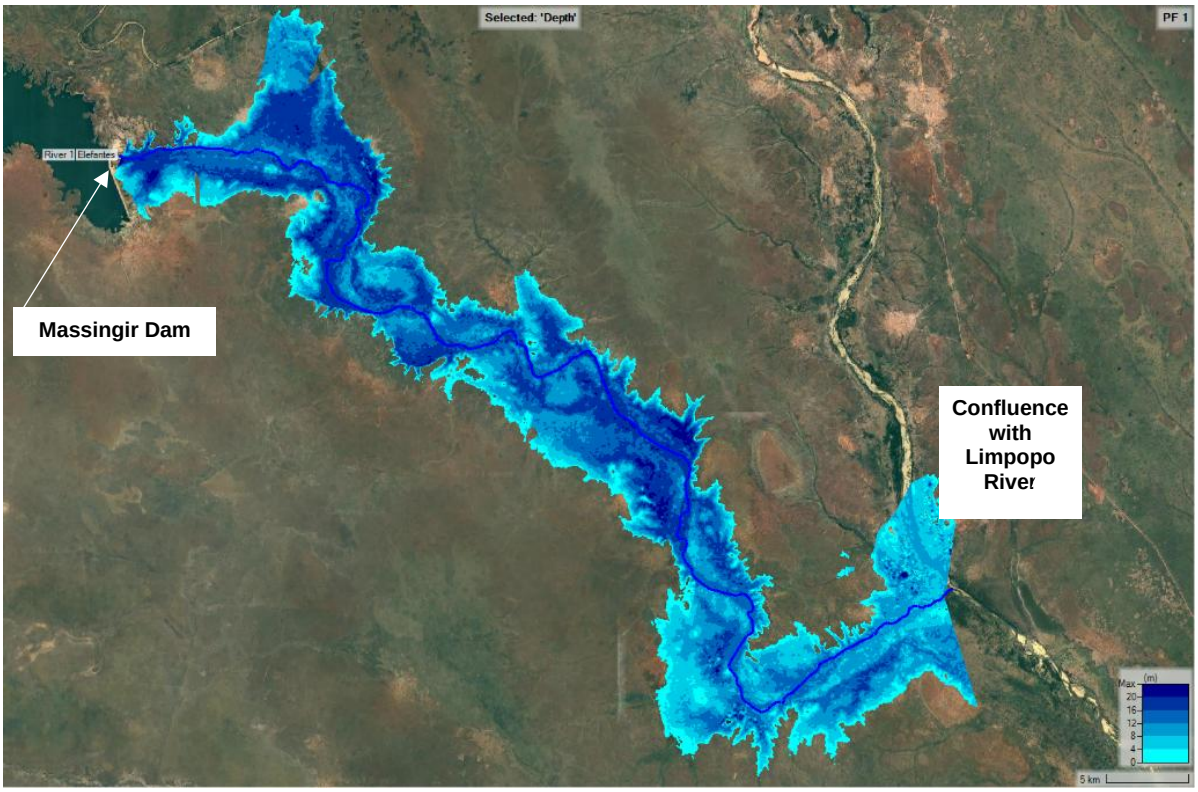


Fig. 6. Potentially flooded area by Massingir dam failure scenario, between the dam and the Elephants and Limpopo rivers confluence.

In terms of human occupation, the valley is sparsely populated and the housing structures in the area are basic, lacking extensive services or infrastructure. However, as stated, agriculture plays a crucial role in the region as it supports the local economy and livelihoods.

A high discharge corresponding to the spillway maximum discharges would have immediate consequences for water intake pumps and agricultural land directly downstream. On the left bank of the initial stretch, the settlements of Chibotane/Machanguene and Muôno are partially flooded, as well as the agricultural areas in the vicinity of the river.

The left bank tributary of the Elephants River, along with the open valley between both rivers, plays a crucial role by providing substantial damping to the flood wave. Even though Bembane agricultural area would be totally flodded, as well as Banhine water abstraction infrastructures and part of intensive agricultural.

For the failures scenarios, although the dam breach scenarios at Massingir exhibit significant differences in breach formation processes, breach dimensions, and resulting outflow hydrographs - leading to fluctuations in water depth and flow velocity in the downstream valley - the overall assessment of flood consequences remains relatively consistent. This similarity is primarily attributed to the minimal valley occupation within the first 10 km downstream of the dam, where the most significant water level and velocities differences occur, leading to a less pronounced impact. Additionally, the expanded topography in the final cross-section of the downstream valley helps maintain more comparable water depth and flow velocity values, ensuring that the overall flood impact remains consistent across the failure scenarios.

Nevertheless, should the structural integrity of the Massingir Dam be compromised, the effects would cascade far beyond the spillway maximum discharges. Additionally, the Limpopo National Park, which lies adjacent to the river, would be heavily impacted, with wildlife habitats inundated, potentially causing damages to the park’s ecosystem. For dam failure scenarios the agricultural areas in the vicinity of the Elephants River, either intensive or family run farms, would be affected by the wave flood, as well as the existing nearby settlements, affecting the livelihood of people.

As stated, there are no significant critical infrastructures within the stretch of river between the dam and the confluence of the Elephants and Limpopo rivers. In fact, the most critical assets are those associated with irrigation systems, particularly the water abstraction infrastructure for the center-pivot irrigation systems at Banhine Farm. These include the pivots themselves and the irrigation channel that transports water from the Elephants River to the reservoirs that supply the pivots.

Beyond the immediate vicinity, catastrophic flooding would affect areas downstream of the confluence of the Elephants and Limpopo rivers. Major impacts would occur affecting regions like Chókwè District and beyond. Excessive flow into the Machiane Canal could result in flooding within the city of Chókwè. Additionally, a significant increase in the Limpopo River's flow would impact all areas connected to the Limpopo Basin, including nearby villages and the city of Xai-Xai.

5. CONCLUSIONS

The main spillway has a discharge capacity of 10,000 m^3/s, resulting in an average water depth of approximately 9 m. In the event of failure scenarios, the water depths vary considerably, with average heights of 12 m for the Minimum Impact Scenario and 20 m for the Worst-Case Scenario.

Flow velocities also exhibited notable differences across the scenarios, with the most severe breach scenario not only resulting in the deepest inundation but also the highest velocities. These results emphasize the critical role of breach dimensions in determining the magnitude of flood impacts. The sensitivity analysis highlights the range of potential outcomes, providing valuable insights for risk assessment and flood management planning.

However, despite the significant differences observed across the various dam breach scenarios at Massingir, particularly those arising from the breach formation process and the corresponding outflow hydrographs, the overall assessment of the flood consequences remains relatively consistent due to the sparse valley occupation within the first 10 km downstream of the dam, where the largest water level differences occur, resulting in a less pronounced impact. Furthermore, the expanded topography in the final cross-section of the downstream valley plays a crucial role by maintaining more comparable water depth and flow velocity values. As a result, the overall flood impact remains largely uniform across the different failure scenarios.

In conclusion, while breach dimensions and resulting flood characteristics exhibit variability, the overall flood impact along the downstream stretch of the Massingir Dam remains relatively consistent. This insight is valuable for informing flood management practices and preparing for potential breach events.

It is also important to note that the primary limitation in this analysis lies in the topographic data used, specifically SRTM DTM, which has a relatively low level of detail. This limitation becomes particularly important when analyzing flood events in areas with complex terrain or fine-scale features that could significantly influence the flood's extent and intensity.

In scenarios where the topography plays a crucial role in shaping the flood path, such as areas with complex river networks, or floodplains with varied elevation, the lack of finer detail in the SRTM DTM could result in inaccuracies in the flood model's predictions. These inaccuracies may affect key flood parameters, including water depth, flow velocities, and the spatial extent of the inundation.

To improve the precision of flood modeling and more accurately assess the potential impacts of dam breach scenarios, it would be beneficial to use higher-resolution topographic data. This could include LiDAR (Light Detection and Ranging) data or high-resolution digital elevation models (DEMs) obtained through ground surveys or advanced satellite-based technologies. Such data would provide a more accurate representation of the landscape, enabling more detailed simulations of flood behavior and improving the reliability of the resulting flood impact assessments.

REFERENCES

[1] COBA, LNEC AND SALOMON (2024), “Inception Report”, Project” Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams”.

[2] COBA, LNEC and SALOMON (2024), “Inspection Report. Massingir Dam”, Product 1b of the Project “Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams”.

[3] COBA, LNEC and SALOMON (2024), “Technical Report. Massingir Dam”, Product 2b of the Project “Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams”.

[4] COBA, LNEC and SALOMON (2024), “Safety Assessment Report. Massingir Dam”, Product 3b of the Project” Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams”.

[5] Froelich D.C. (1995), “Embankment dam breach parameters revisited," International Water Resources Engineering Conference - Proceedings, vol. 1.

[6] USACE (2023), "HEC-RAS Hydraulic Reference Manual. March 2023.," United States Army Corps of Engineers.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

STRENGTHENING KEY WATER INFRASTRUCTURE IN MOZAMBIQUE: RISK ANALYSIS OF PEQUENOS LIBOMBOS DAM DOWNSTREAM VALLEY

Sílvia Amaral[1]	Fernanda Rocha	Teresa Viseu	Dinis Juízo	Francisco Saimone
LNEC	LNEC	LNEC	SALOMON	SALOMON

Lurdes Pimenta	Delário Sengo	Jaime Timba	Luís Mabunda
COBA	ARA, Sul	ARA, Sul	DNGRH,

ABSTRACT

The Pequenos Libombos Dam, a 1,640-meter-long and 48-meter-high mixed zoned earthfill structure, with a 100-meter-long central gravity concrete zone, is located on the Umbelúzi River in Mozambique. The study on flood wave propagation was conducted over a 42.5 km river stretch, from the dam section until the Espírito Santo Estuary, at the mouth of Maputo Bay. The topographic configuration of the valley downstream of the dam reveals a broad, open landscape with a narrow riverbed that is considerably smaller in scale compared to the expansive floodplain. In its final stretch, the river becomes highly meandering, with a wide floodplain shaped by tidal influences.

Although of reduced likelihood, the failure of a dam causes a significant and sudden increase in the flow in the downstream sections of the river, forming a flood wave that propagates at high speeds, causing a rapid rise in the river's level and flooding of its banks.

The risk analysis of the downstream valley of Pequenos Libombos Dam focused on evaluating the flow propagation along the Umbelúzi River considering two scenarios: i) the maximum discharge capacity of the spillway (6 800 m3/s) and ii) a large failure of the dam. Both scenarios result in catastrophic flood waves, characterized by extremely high-water levels, velocities, and flow rates. The study included a thorough evaluation of the flood waves characteristics along the river, combined with insights in what concerns major impacts.

By modelling these scenarios, the study aims to provide a comprehensive understanding of the risks associated with the dam's accidents and support the development of effective mitigation strategies.

Keywords: Pequenos Libombos Dam; Risk Analysis; Dam Failure; Downstream Valley; Inundation Maps.

[1]samaral@lnec.pt, Portugal

1. INTRODUCTION

The Pequenos Libombos dam is located on the Umbelúzi River, in the Maputo province. Umbelúzi is an international river, born in Eswatini, and only its last 77 km are in Mozambique. The Pequenos Libombos dam is located at 34.5 km from the border and at 42.5 km from the initial section of Espirito Santo estuary (Fig. 1). The catchment area at the dam section is 3 900 km^2, corresponding to 71% of the total Umbelúzi catchment (5 460 km^2). The reservoir has a storage capacity of 393 hm^3. The completion year was 1986 and the first filling occurred between 1986 and 1989 ([2] and [3]).

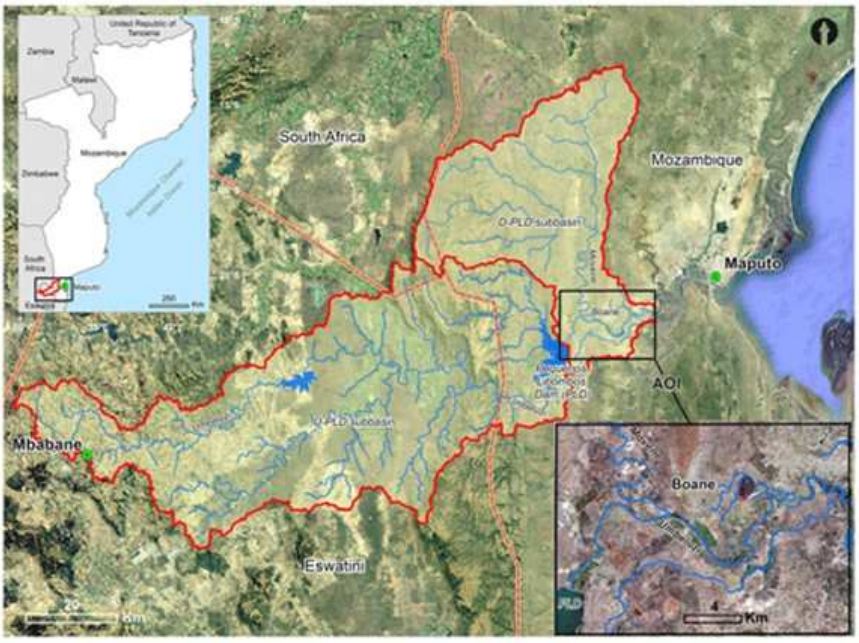


Fig. 1: Location of Pequenos Libombos dam in the Umbelúzi river basin (Cea *et al.,* 2024 [1])

The dam serves multiple purposes, as the public water supply of Maputo, Matola and Boane (main purpose), irrigation (around 3 000 ha), industrial supply and flood control. It is also equipped with a small hydroelectric power generation unit (1.7 MW), currently suspended due to a long duration drought that shortens the water available for other uses (interrupted by a flood in January 2022). The dam is managed by ARA-Sul.

The Pequenos Libombos is a mixed zoned earthfill dam located at the right and left abutments, while a concrete dam occupies the central valley, housing the appurtenant structures. The earthfill dam at the right and left abutments features a zoned cross-section, with a central core made of clayey materials and a chimney filter protecting the downstream face of the core.

The dam has a total crest length of 1 640 m, including a 100 m at the concrete central section Its maximum height is 48 m at the concrete section and 46 m at the embankment section. The crest level is 51.5 m. The appurtenant works include spillway, bottom outlets and water intake concentrated in the concrete dam section (Fig. 2).

Fig. 2: Pequenos Libombos dam (Google earth view)

The purpose of this work is to present the results of the flood study caused by considering two different scenarios: i) the maximum discharge capacity of the spillway (6 800 m^3/s) and ii) a large failure of the dam. Both scenarios result in catastrophic flood waves, characterized by extremely high-water levels, velocities and flow rates. The study included a thorough evaluation of the flood waves characteristics along the river, combined with insights in what concerns major impacts.

2. DOWNSTREAM VALLEY

The studied stretch of the Umbelúzi river River extends from the Pequenos Libombos Dam to the confluence with the Tembe River, covering approximately 42.5 km. The topographic configuration of the downstream valley varies significantly between the dam and the estuary, particularly in the width of the riverbed and the size of the riverbanks, defining three distinct reaches ([3]):

- Pequenos Libombos - Boane (14.5 km): small width of the larger riverbed, with the left bank being more embedded;
- Boane – Maputo WTP (from 14.5 to 20.5 km): wider riverbed, with flat banks providing flooding of a vast areas;
- Maputo Water Treatment Plant (WTP) - Confluence with Tembe (from 20.5 to 42.5 km): banks gradually widen over the final kilometres, into flood plain, very meandered and subject to the action of the tide.

2.1. Housing and economic activities

The main localities with the potential to be affected by the flood wave from a dam failure are (Fig. 3): Mafuiane (along the EN2 and the access road to the dam), Congoana; municipality of Boane and Belo Horizonte on the left bank and Massaca, Boane and the village of the WTP on the right bank. Boane and Belo Horizonte are the largest populational localities.

Fig. 3: Main localities on the Umbelúzi valley

The uncontrolled occupation of the downstream area of the Pequenos Libombos Dam is a significant concern. With the municipalization of Boane, there is growing pressure from both residents and political leaders to develop the floodplain along the Umbelúzi River. In addition to expanding farmland, residential construction is extending all the way to the river's mouth. The Massaca area, situated on the right bank of the Umbelúzi River, is also expanding towards the river, despite being designated solely for agricultural use. The left bank of the Umbelúzi River, within the town of Boane, is almost entirely occupied by housing that does not comply with safety distance regulations. The same happens in Belo Horizonte. Fig. 4 provides an overview of this situation. Beyond the Boane drift, there are still several agricultural fields in the riverbed that would be at risk in the event of a dam failure.

Agriculture is the basis of the economy of the Umbelúzi valley, mainly along the banks of the river, which has highly fertile soils, with good agricultural and livestock potential, which are exploited by private and family farmers for the cultivation of cereals, vegetables, legumes, tubers and fruits.

Fig. 4: Examples of soli occupation of the Umbelúzi downstream valley

Regarding the territorial occupation of the valley downstream, it was possible to see that there are two distinct zones. In the first one, located between the dam and the Maputo WTP (≅21.5 km), the valley has a considerable irrigated area. The agricultural landscape intersects with residential areas (Fig. 5). Then, between the Maputo WTP and the river mouth (≅ 21 km), the river meanders and the valley widen even further, forming a large floodplain with extensive marshy areas on the right bank (Fig. 6). In the area adjacent to the waterline the population density is low, mainly on the right bank. In the final stretch there are buildings, in an area that appears to belong to the border with the Matola River basin.

Fig. 5: Soil occupation between: a) the dam and Massaca; b) Massaca and the WTP

Fig. 6: Soil occupation between the WTP and the confluence with the Tembe River

There are other economic activities in the Umbelúzi valley, such as the extraction of inert materials in the Mafuiane region or agriculture and hotel units in Massaca. In the municipality of Boane there are several services and production units, including the Boane Agrarian Institute, an important entity to the region.

Upstream from Belo Horizonte, we can mention the Alberto Augusto Teixeira Agricultural Plant and downstream the Golfinho salt flats. These are just examples of economic activities located near the Umbelúzi River that could be temporarily disrupted by flood waves resulting from a potential dam failure, thereby affecting the livelihoods of local populations.

2.2. Critical infrastructures: Communication routes (roads and railways) and others

Several roads can be possibly affected by the inundation caused by a potential dam failure. The valley is crossed by National Road nº 2 (EN2), which gives access to Eswatini, also allowing direct connection to the cities of Maputo and Matola. Downstream from Pequenos Limbombos, the EN2 follows the Umbelúzi River along the left bank. In Boane, the road N200 diverts from EN2 to the right bank, crossing the river. From N200 derives the R400, which follows the river along the right bank, until the dam then turns south. There is also a set of secondary and tertiary paths, most of which are dirt roads, with access to local farms and residential areas.

The railway line that connects Boane to Goba ensures the transport of cargo and passengers, also providing a connection to Swaziland. The municipality of Boane is served by one of the most important railway stations on the Goba Railway.

There are several bridges over the river Umbelúzi between the dam and the river mouth.

The Water Treatment Plant that supplies Maputo (Maputo WTP), located on the right bank, approximately at km 21.5 downstream of the dam, is critical for the city of Maputo. Besides being potentially affected by the physical impact of the flood wave, can be compromised by the sediments transported.

3. METHODOLOGY AND DATA

3.1. Failure Scenarios and spillway maximum discharge capacity

The risk analysis of the Pequenos Libombos Dam focused on evaluating the propagation along the dam downstream regions of the Umbelúzi River considering two scenarios ([4] and [5]).

- **Scenario of occurrence of the maximum discharge capacity of the spillway:** corresponds to the event of inflows to the reservoir that require the total capacity of the spillway – 6800 m^3/s;
- **Dam failure Scenario**: a large and slow failure of the Pequenos Libombos dam embankment due to overtopping, resulting in a breach of approximately 80% of the cross-sectional area, defined by the dam's reference axis.

In the case of Pequenos Libombos dam, if a breach occurs, it will presumably happen in the earthfill section and should be slow and partial. However, a quicker failure of the concrete section of the dam cannot be ruled out, which could more intensively carry away the material from the dam's embankment.

The classic mechanisms of failure of earth dams include overtopping, foundation failure and internal erosion of the embankment, foundation or dam abutments.

In overtopping failures, the flow over the structure causes deep cuts into the crest and erosion of the downstream slope material. These processes lead to the formation of a weak zone, which can result in a local collapse of the dam, and the deep cuts may extend down to the base of the embankment. Regardless of the zone of formation, this weak spot will initially tend to have a triangular shape. Once the less resistant material of the dam is eroded, and if high flood levels persist, erosion continues laterally, leading to a final trapezoidal shape. The extent of this lateral erosion depends on the reservoir's size and on the volume of stored water.

Failures caused by internal erosion are not uncommon in earthfill dams and may, in some cases, be detected in time to be controlled. In this type of failure, soil particles from the embankment are carried away by water flowing through the dam's body, forming one or more internal channels. As these channels enlarge due to the flow, the discharge and erosive power of the flow increase. This progressive process generally leads to the channel(s) becoming so large that the overlying materials collapse, forming a single open channel.

There are many criteria and empirical rules for defining the possible scenarios involving failures in embankment dams. The Pequenos Libombos overtopping failure scenario was considered as the worst-case scenario to evaluate the maximum potential flood impacts in the downstream valley of the Umbelúzi River. It was therefore assumed that, at the beginning of the formation of the breach, the water level in the Pequenos Libombos reservoir was at the crest level (reservoir volume equal to 580×10^6 m^3). Empirical equations based on the reservoir volume and in the dam height (h=48 m) were employed to estimate the breach parameters associated with the failure scenario to evaluate the impacts in the downstream valley of the Umbelúzi River. The breach formation time, (t_f), was estimated using the Froehlich formula ([6]):

$$t_f = 0{,}00254 V_w^{0{,}53} h_b^{-0{,}9} \quad (1)$$

This formula is commonly used and conducted to a time of failure of approximately 3.5 hours. Based on these characteristics, the maximum outflow at the dam's cross-section was estimated as 73 500 m³/s.

3.2. Hydraulic simulations and digital terrain model

The simulations were conducted using the HEC-RAS numerical model (https://www.hec.usace.army.mil/software/hec-ras/), where the one-dimensional unsteady flow is solved using the mass conservation equation and the momentum conservation equation of Saint-Venant along the x direction (USAGE, 2023 – [7]).

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l \quad (2)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (QV)}{\partial x} + gA\left(\frac{\partial z_s}{\partial x} + S_f\right) = 0 \quad (3)$$

In these equations, A represents the total flow area (m²); t is the time (s), Q is the discharge (m³/s), q_t are the lateral inflows per unit length, x is the distance (m), Z is the depth (m), and S_f is the friction force, traditionally obtained using the Manning-Strickler equations.

The flood was modelled along a 42.5 km stretch of the river, with the following boundaries:

- **Upstream boundary**: Pequenos Libombos cross-section (km 0.0)
- **Downstream boundary**: Confluence of the Umbelúzi River with the Tembe River, that is, the initial section of Espírito Santo estuary (km 42.5).

For the flood propagation analysis, the Umbelúzi River downstream of the Pequenos Libombos Dam was characterized using cross-sections obtained from the Shuttle Radar Topography Mission (SRTM) dataset, a global Digital Elevation Model (DEM) with a 30-meter resolution.

4. RESULTS

4.1. Flood characteristics

Scenario of occurrence of the maximum discharge capacity of the spillway *(event of inflows to the reservoir that require the total capacity of the spillway – 6 800 m³/s)*

For a spillway discharge capacity of 6 800 m³/s, the modelling results lead to (Table 1a):

- maximum water levels (Z_{max}) along the studied stretch range from 31.8 m, right downstream of the dam, to 3.5 m, at the confluence with the Tembe River;
- flood wave (h_{max}) reaches a height of approximately 17 m, just downstream of the dam; at the cross section of the confluence with the Tembe River, the river level rises by about 7.8 m;
- maximum flow velocities vary between 2.7 m/s, close to the dam, and 0.27 m/s, at the confluence.

Table 1: Results for: a) flood design scenario Q=6 800 m³/s and b) Dam failure Q=73 500 m³/s

$Q_{disharged}$ 6800 m³/s	Distance to dam (km)	Z_{bed} (m)	Z_{max} (m)	h_{max} (m)	v_{max}flow (m/s)
Barragem	0,00	15	31,77	16,77	2,66
1 km	1,00	14,23	30,92	16,69	1,97
2 km	2,04	13,4	29,72	16,32	3,39
8 km	3,00	12,83	26,68	13,85	3,07
13 km	4,08	12,03	26,47	14,44	2,06
18 km	4,98	11,4	23,26	11,86	4,51
22 km	5,98	10,6	21,99	11,39	2,31
27 km	6,97	9,83	19,61	9,78	3,4
33 km	8,00	8,68	19,19	10,51	1,14
34 km	8,98	8,49	19,03	10,54	1,46
38 km	9,99	7,61	18,67	11,06	1,81
11 km	11,00	7	16,57	9,57	3,14
12 km	12,02	6,33	16,15	9,82	1,74
Boane bridge - 13.1 km	13,09	5,55	14,57	9,02	2,56
14 km	14,00	5,21	12,86	7,65	3,15
15 km	15,01	4,82	12,51	7,69	0,71
16 km	16,01	5,96	12,42	6,46	0,83
17 km	17,02	4,07	11,87	7,8	0,95
18 km	18,02	3,76	11,77	8,01	1,14
19 km	19,02	3,36	11,67	8,31	0,49
20 km	20,00	3,15	11,6	8,45	0,52
21 km	21,00	2,94	11,04	8,1	1,98
22 km	21,99	2,62	10,84	8,22	1,09
23 km	22,99	2,24	10,74	8,5	0,42
24 km	24,00	2,01	10,55	8,54	0,74
25 km	24,98	1,79	10,1	8,31	0,86
26 km	26,00	1,57	9,83	8,26	0,7
27 km	26,98	1,38	9,57	8,19	0,83
28 km	28,00	1,18	9,22	8,04	0,93
29 km	29,02	0,98	8,85	7,87	0,9
30 km	30,02	0,62	8,49	7,87	0,94
31 km	31,01	0,24	8,16	7,92	0,85
32 km	32,01	-0,15	7,92	8,07	0,73
33 km	33,00	-0,52	7,73	8,25	0,68
34 km	34,01	-0,88	7,51	8,39	0,72
35 km	35,01	-1,24	7,23	8,47	0,73
36 km	36,02	-1,6	6,93	8,53	0,72
37 km	36,99	-1,95	6,66	8,61	0,67
38 km	38,02	-2,37	6,12	8,49	0,89
39 km	39,02	-2,78	5,48	8,26	0,91
40 km	40,01	-3,18	4,65	7,83	0,97
41 km	40,99	-3,57	3,79	7,36	1,19
42 km	41,99	-3,94	3,52	7,46	0,48
43 km	42,95	-4,3	3,5	7,8	0,27

$Q_{disharged}$ 73 500 m³/s	Distance to dam (km)	Z_{bed} (m)	Z_{max} (m)	h_{max} (m)	v_{max}flow (m/s)
Barragem	0,00	15	42,79	27,79	8,8
1 km	1,00	14,23	42,88	28,65	5,08
2 km	2,04	13,4	42,17	28,77	5,44
8 km	3,00	12,83	42,2	29,37	3,12
13 km	4,08	12,03	41,93	29,9	2,83
18 km	4,98	11,4	36,1	24,7	5,14
22 km	5,98	10,6	32,07	21,47	4,7
27 km	6,97	9,83	29,84	20,01	5,89
33 km	8,00	8,68	29,47	20,79	2,61
34 km	8,98	8,49	29,3	20,81	2,63
38 km	9,99	7,61	28,69	21,08	2,92
11 km	11,00	7	28,04	21,04	4
12 km	12,02	6,33	27,47	21,14	3,78
Boane bridge - 13.1 km	13,09	5,55	22,23	16,68	3,61
14 km	14,00	5,21	20,12	14,91	4,92
15 km	15,01	4,82	19,56	14,74	2,15
16 km	16,01	5,96	19,35	13,39	1,83
17 km	17,02	4,07	18,78	14,71	2,94
18 km	18,02	3,76	18,74	14,98	1,78
19 km	19,02	3,36	18,59	15,23	1,29
20 km	20,00	3,15	18,45	15,3	1,5
21 km	21,00	2,94	18,18	15,24	2,23
22 km	21,99	2,62	18,01	15,39	1,86
23 km	22,99	2,24	17,83	15,59	1,18
24 km	24,00	2,01	17,61	15,6	1,46
25 km	24,98	1,79	17,3	15,51	1,7
26 km	26,00	1,57	16,98	15,41	1,79
27 km	26,98	1,38	16,62	15,24	1,9
28 km	28,00	1,18	16,16	14,98	2
29 km	29,02	0,98	15,67	14,69	2,05
30 km	30,02	0,62	15,2	14,58	1,96
31 km	31,01	0,24	14,67	14,43	1,9
32 km	32,01	-0,15	14,25	14,4	1,68
33 km	33,00	-0,52	13,76	14,28	1,58
34 km	34,01	-0,88	13,27	14,15	1,74
35 km	35,01	-1,24	12,7	13,94	1,79
36 km	36,02	-1,6	12,06	13,66	1,83
37 km	36,99	-1,95	11,43	13,38	1,85
38 km	38,02	-2,37	10,57	12,94	2,02
39 km	39,02	-2,78	9,83	12,61	2
40 km	40,01	-3,18	8,99	12,17	2,04
41 km	40,99	-3,57	8,23	11,8	2,1
42 km	41,99	-3,94	6,68	10,62	2,62
43 km	42,95	-4,3	5,92	10,22	1,99

Dam failure Scenario *(event leading to a maximum discharge of 75 000 m³/s)*

Due to valley topography, there is a large flow damping along the 42.5 km of the Umbelúzi river along the section. Thus, close to the confluence with the Tembe River (P22), the maximum flow is 23 971 m³/s, which corresponds to a total damping of 67.4%; the modelling results lead to (Table 1b):

- maximum water levels along the studied stretch range from 42.8 m, downstream of the dam, to a significantly lower level of 5.9 m, at the confluence with the Tembe River;
- the flood wave reaches a height of approximately 28 m, just after the dam; at the cross section of the confluence with the Tembe River, the river level rises by about 10.2 m;
- corresponding flow velocities vary between 8.8 m/s, close to the dam, and 2.0 m/s, at the confluence.

4.2. Consequences in the downstream valley

Fig. 7 and Fig. 8 show respectively the inundations maps for both scenarios: maximum spillway (design flood of 6 800 m^3/s) and failure due overtopping (flood of 73 500 m^3/s).

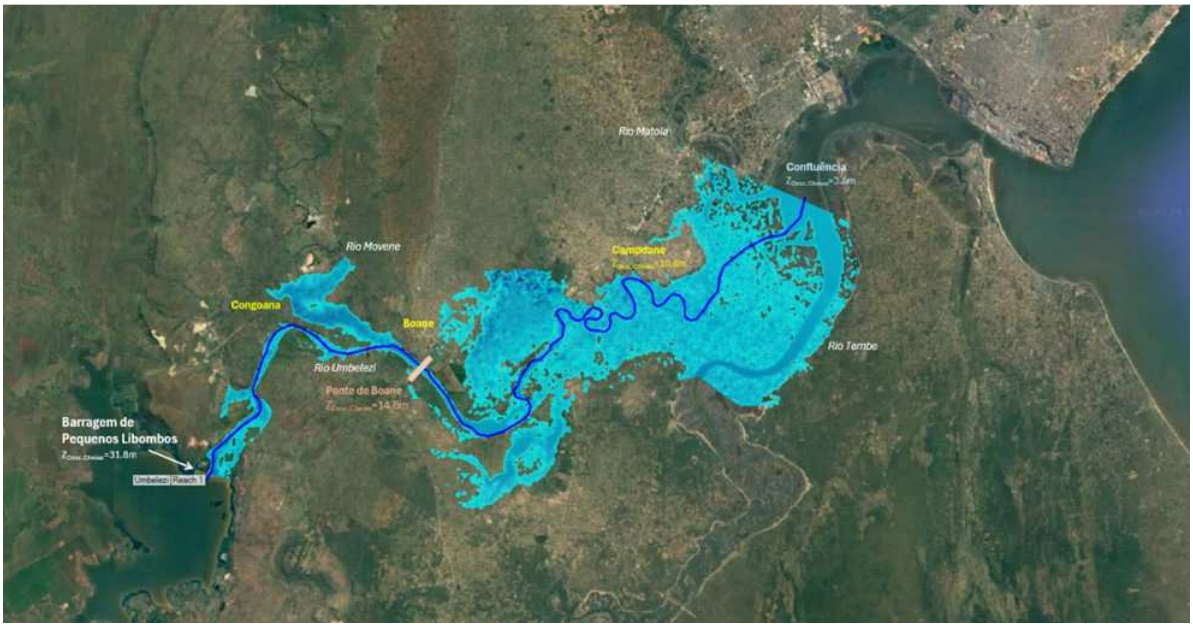


Fig. 7: Inundation map for maximum spillway (Q=6 800 m^3/s)

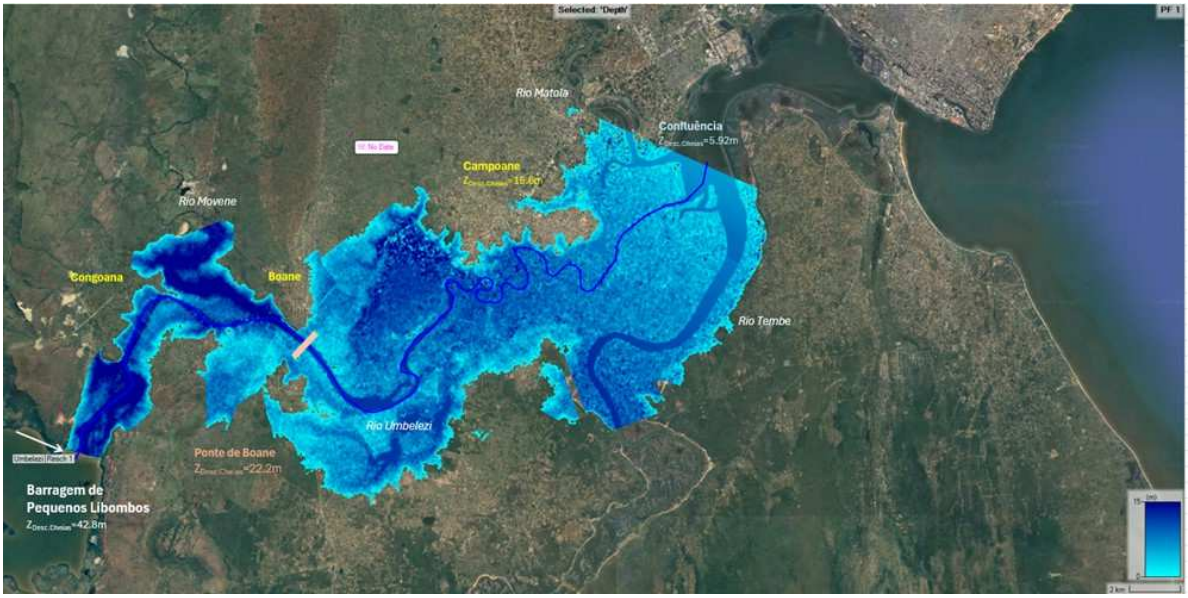


Fig. 8: Inundation map for overtopping dam failure (Q=73 500 m^3/s)

Along with depths and velocities reached (Table 1), the results obtained clearly demonstrate the destructive nature of the flood waves. According to some authors, stretches of rivers where $U.h \geq 1$ m^2/s occurs (where U is the speed and h is the height of the flow) can be considered dangerous to people and property.

Comparing the results obtained in the two scenarios, the overtopping one leads, as would be expected, to the most serious consequences. The river is subject to higher water heights, speeds and flows in this scenario. Obviously, the biggest differences between the scenarios occur in the initial propagation phase. Thus, in the dam section, the maximum value of the flood wave is 32% higher in the case of the scenario of total failure. This difference softens as the waves propagate downstream: in the Boane section, due to the large flood plains, the maximum value of the flood wave remains higher, in the case of total failure, but only by 16 %, and, in the section of the confluence of the Umbelúzi and Tembe, this difference is only 1.3 %. In this last section the difference in water heights between the two scenarios is only 10 cm.

Impact on housing and economic activities

The majority of the localities in the vicinity of the Umbelúzi river, as well as many agricultural units, are partially flooded in the worst case scenario (Fig. 9), with higher depths and velocities than for the maximum spillway scenario, as shown in a zoom of the flooded areas in the upstream area of Boane (Fig. 10).This will affect the safety of local population and their livelihood.

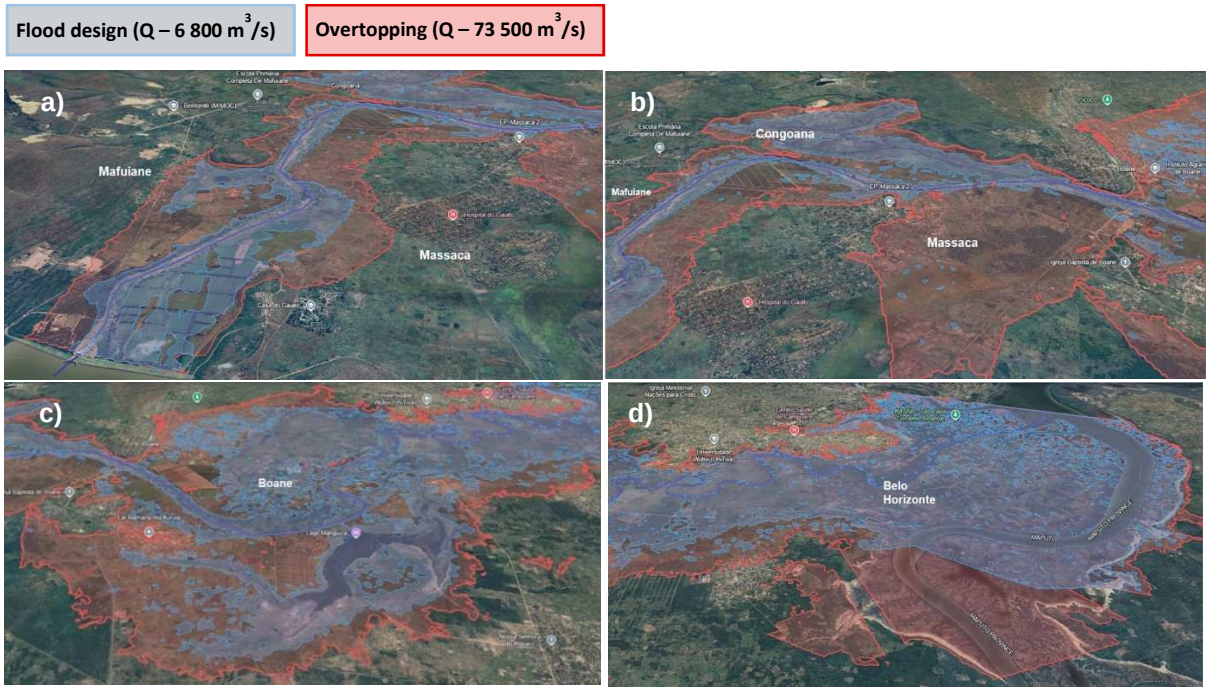


Fig. 9: Impact of inundation waves on housing and economic activities: a) Dam – Mafuiane; b) Mafuiane – upstream Boane; c) Boane – WTP and d) Belo Horizonte – Confluence of Umbelúzi and Tembe rivers

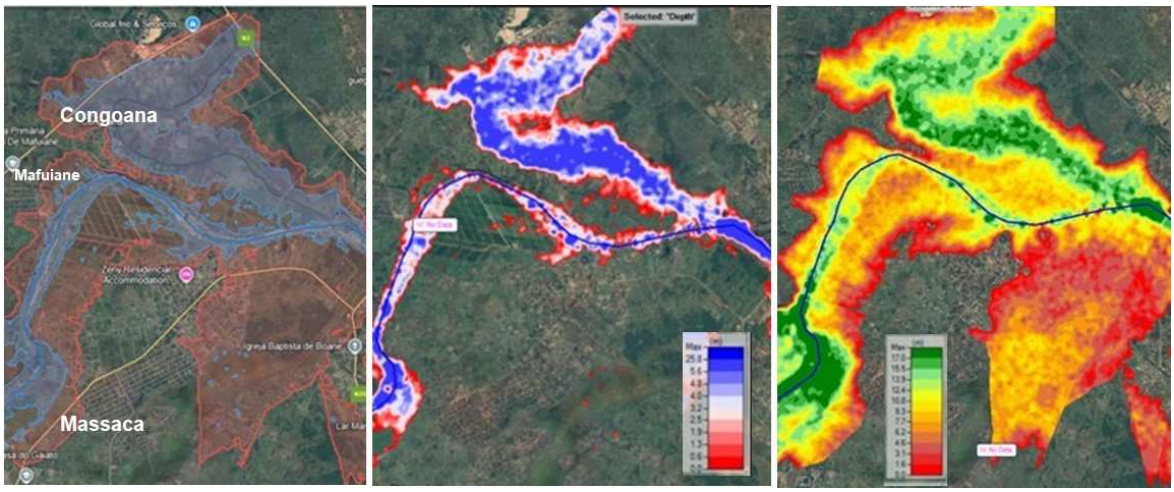


Fig. 10: Inundation map and water depth in Mafuine–upstream Boane reach of Umbelúzi river

Critical infrastructures: Communication routes (roads and railways) and others

Several roads can be affected by the inundation caused by a potential dam failure (Fig. 11).

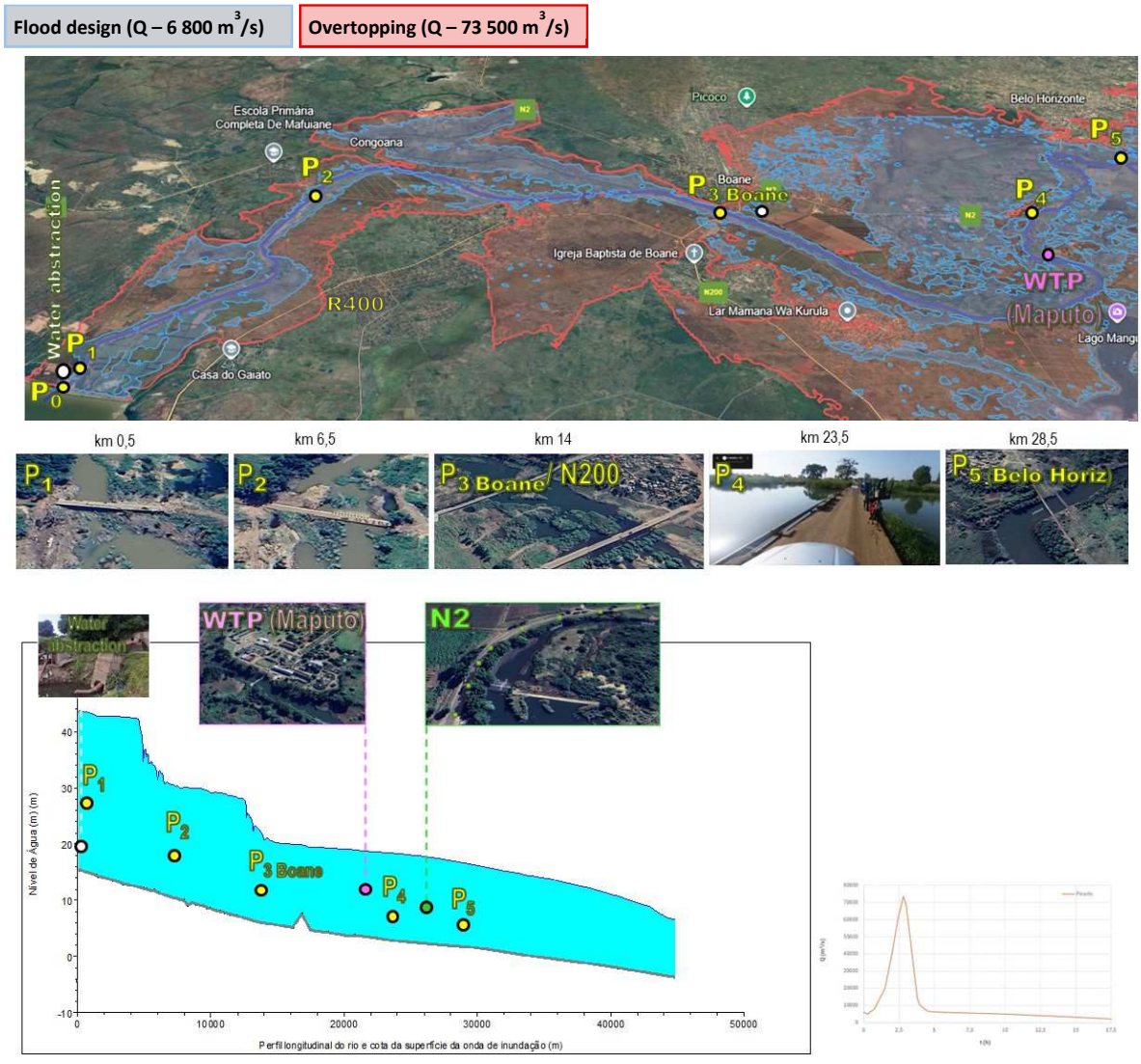


Fig. 11: Impact of inundation waves on critical infrastructures

Downstream from Pequenos Limbombos, the EN2 follows the Umbelúzi River along the left bank. This road, that gives access to Eswatini and direct connection to the cities of Maputo and Matola, is affected in several points. The road N200, diverting from EN2 in Boane and crossing the river to the right bank, is also impacted, as well as R400 and the railway line that connects Boane to Goba. There is also a set of secondary and tertiary paths, most of which are dirt roads, with access to local farms and residential areas.

Not only these roads and railways have stretches that can be flooded, as all the bridges over the river Umbelúzi between the dam and the river mouth are submerged (Fig. 11).

5. CONCLUSIONS

Although of reduced likelihood, the failure of a dam causes a significant and sudden increase of the flow in the downstream section of the river, forming a flood wave that propagates downstream at high speeds, causing a rapid rise in the river's level and flooding of its banks.

The risk analysis of the downstream valley of Pequenos Libombos Dam focused on evaluating the flood propagation along the Umbelúzi River considering two scenarios: maximum spillway discharge and a large failure due to overtopping. The latter scenario results in catastrophic flood waves, characterized by extremely high-water levels, velocities and flow rates. The study included a thorough evaluation of dam break wave simulations along the river, combined with insights from inspection visits and an assessment of the associated environmental and population impacts.

In conclusion, it could be stated that the flood waves caused by the dam failure are strongly damped in the downstream section, for both scenarios under analysis, with the damping being more pronounced for dam failure. As a result, the differences between the flood waves associated with the maximum spillway scenario and the total failure scenario decrease progressively along the Umbelúzi River.

This similarity in the magnitude of flood waves no longer occurs when the comparison is made in terms of flooded area. In fact, the delimitation of the floodable areas (Fig. 7 and 8), allows to conclude that the differences are very large for the two scenarios. Thus, although the water height values are similar, the topography of the valley, which is very open, means that small rises in level lead to the submersion of large areas.

By modelling these scenarios, the study aims to provide a comprehensive understanding of the risks associated with the dam’s failure and support the development of effective mitigation strategies.

REFERENCES

[1] CEA, L., Álvarez, M. and Puertas, J. (2024), “Using integrated hydrological–hydraulic modelling and global data sources to analyse the February 2023 floods in the Umbeluzi Catchment (Mozambique)", Hazards Earth Syst. Sci., 24, 225–243, 2024, https://doi.org/10.5194/nhess-24-225-2024.

[2] COBA, LNEC and SALOMON (2024), “Inception Report”. Project "Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams".

[3] COBA, LNEC and SALOMON (2024), “Inspection Report. Pequenos Libombos Dam”, Product 1a of the Project "Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams".

[4] COBA, LNEC and SALOMON (2024), “Technical Report. Pequenos Libombos Dam”, Product 2a of the Project "Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams".

[5] COBA, LNEC and SALOMON (2024), “Safety Assessment Report. Pequenos Libombos dam”, Product 3a of the Project "Consulting Services for Dam Safety Assessment of Pequenos Libombos and Massingir Dams".

[6] FROELICH D.C. (1995), “Embankment dam breach parameters revisited", International Water Resources Engineering Conference - Proceedings, vol. 1.

[7] USACE (2023), "HEC-RAS Hydraulic Reference Manual. March 2023", United States Army Corps of Engineers.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

OPTIMIZING TAILINGS DAM SAFETY: INTEGRATING HUMAN FACTORS AND EVOLVING HAZARDS

Franco Oboni[1]
SRK Consulting

Cesar Oboni[2]
SRK Consulting

ABSTRACT

The mining industry faces significant challenges in managing both inactive and active tailings storage facilities (TSFs), which vary widely in size, environmental parameters, and maintenance issues. Traditional risk assessment methods for TSFs often rely on incomplete and inconsistent historical data, leading to potential misclassifications and a false sense of security. This paper introduces a comprehensive quantitative risk assessment model for TSFs that addresses these limitations by incorporating human factors and evolving risks due to maintenance, operations, and climate change. The model leverages a probabilistic causality analysis to evaluate the failure processes of tailings dams, emphasizing the importance of human factors over natural events in determining failure probabilities. By analyzing a global portfolio of dams, the model provides a robust framework for assessing and mitigating risks, offering practical insights into the effectiveness of various mitigation strategies. Comparative analyses of three similar dams—categorized as “The Good, Bad, and Ugly”—demonstrate the model's capability to differentiate risk levels based on care and understanding in design, construction, and maintenance. The paper also explores the potential for preparing scripts to study mitigative alternatives, such as buttress construction or less significant ones, and discusses the application of ALARP (As Low As Reasonably Practicable) analyses in optimizing risk management. This work aims to enhance the reliability and safety of TSFs by providing a more accurate and holistic approach to risk assessment.

Keywords: Tailings Storage Facilities (TSFs), Quantitative Risk Assessment, Probabilistic Causality Analysis, Human Factors, ALARP (As Low As Reasonably Practicable)

[1]foboni@srk.com, Canada
[2]coboni@srk.com, Canada

1. INTRODUCTION

The Mining industry deals with thousands of inactive/closed heritage storage facilities and a significantly smaller number of modern active/inactive ones. The size of the facilities is extremely variable, both in terms of volume, height, length and shapes without even mentioning major “environmental” parameters such as geology, hydrogeology, seismicity, climate, topography and land use (especially downstream). In addition maintenance issues, in particular but not limited to water management subsystems further complicate any attempt to define a “simple” taxonomy of TSFs.

Despite numerous efforts by several institutions and authors [1], [2], [3], [4], [5], and [6] it appears that:

- data are incomplete and present very significant gaps,
- records do not deliver enough details to elicit proper taxonomies,
- causality/failure mode classification are inhomogeneous and doubtful to say the least with significant percentages of “unknown” cases.

As a result, when extracting information from these databases there still is a clear hazard of reaching fuzzy results due to the mixing of different populations of dams and past accidents.

On the risk assessment front, practitioners, including the authors of this paper, have dealt with single dams using various approaches going from qualitative to semi-quantitative, semi-empirical and quantitative [7]. Many of these approaches deal separately for each failure mode (stability, internal erosion, seismic, etc.) and adopt mono-dimensional consequences evaluations, which are misleading. Indeed, the accidents consequences are the sum of dimensions such as harm to people, environmental damages, business interruption, direct and indirect losses, third parties losses, etc. and neither the worst of one of these dimensions, nor merely harm to people as some guidelines suggest. In 2014 a MSc Thesis at UBC [8] linked various failure modes using a well-known reliability formula known as the series-system equation, which assumes the various failure modes were totally independent from each other, and each failure mode occurrence could be fatal to the structure. Following the same idea the US Federal Energy Regulatory Commission [9] states that failure modes should be combined by summation of their probabilities, an approximation of the series-system equation result only valid if the probabilities are all very small (NB: if that is not the case the sum could become greater than one, which is against the basic axiom of probabilities).

Only in the last years the idea of portfolio risk assessment has been pushed forward [10]. In this paper we describe how we built a general model for TSF risk assessment which allowed

to develop a “world model” covering to date more than hundred tailings dams of various types, age and built, which apparently mimics the behavior of the world-wide tailings dams portfolio.

2. DRIVING CONCEPTS

Linking specific types of failures to a potential failure mode or cause taxonomy is challenging because only a few failures might fit into a category of that taxonomy. For example, as demonstrated in [11], the "Failure distribution by cause" shows a small sample size. Similarly, the "Failure distribution by dam height" also has a limited sample size. When we try to analyze correlations, such as "Failure distribution by cause" given a certain dam height, the sample size becomes so small that one ends up with either a single case or no case at all.

From a risk point of view, i.e. when consequences of failures are included, risk is constantly evolving in both directions: probabilities (quality of design, evolving standard of care, lack of maintenance, operations, climate change, etc.) and consequences (demographic pressure, land use, public opinion, legal, etc.).

2.1. Databases may be misleading and give a false sense of security

The above always made us uneasy about blindly relying on historical data, but one should take advantage of extant data, after careful considerations of the limitations stated earlier.

Furthermore the goal of the approach was to provide simple, deconvoluted probabilities estimates of single “basic events/situations” and not “failure mode” range, to avoid confusing the users. The approach also had to stay as close as possible to “standard engineering” understanding of the very convoluted issues that may arise in dams quantitative risk assessment, as it will be discussed in the following sections.

2.2. Past different than future

In addition, it was understood that using databases without any checks would likely lead to neglect three important points, namely that:

- the quality of dams, especially the most modern “big” ones has improved over time,
- over time ancillary water management facilities may have deteriorated, as maintenance and design deviations may have developed,
- climate change will make the statement “past different from future” even more true than it is today [12].

2.3. Human factors do not show up in databases

Furthermore, it must be considered that traditional databases were not prepared with a "human factor" focus, whereas, based on numerous examples of catastrophic failures we were inclined to think that these were paramount to explain why a world portfolio of dams generally designed with the same FoS generated enough failures to stir public opinion (and regulatory/ political) outcry**.** That is why we started looking at the human factors first, followed by all the other hazards, root causes and elements/sub elements discussions.

3. FAILURE CHAIN PROCESS DRIVEN BY HUMAN CAUSES

Dams failure processes, which included "soft" root causes were studied [13]. A systemic approach of the "failure chain process" of tailings dams using a probabilistic causality analysis based on publicly available incident and accidents data from the last 60 years was presented after considering valid the particular use of the database made as it looked at very specific aspects of the failure process. Citing directly [13]: "The predictive model, geared toward filling the gap between common practice and "path to zero failures" requirements (as "requested" in the aftermath of major failures), accommodates data-mining analytics. The model "constructs" the probability of failure of a dam which is consistent with factual historical world-data. The causality of various factors entering in the dam's service life can then be individually discussed with a sensitivity analysis." Then the 2016 paper [13] showed where and how mitigative actions can benefit the most with a practical example. Attention was focused on Common Cause Failure (CCF) in operations, risk assessment, peer reviewing and inspections of tailings dams during the service life of dams. Investigations, Design and Construction extended to Management, Monitoring & Water Balance control of the dam (shortened to e-IDC in [13]) were analyzed with a probabilistic causality analysis based on publicly available incident and accidents data from the last hundred years. It was noted that a significant number of risk studies do not start with the considered tailings system definition, its functional analysis, and they oftentimes confuse hazards, risks and consequences [14] leading to misleading results. For instance it was noted: "it is for example rather common to see "insufficient FoS" considered as a hazard (or a risk), whereas such deficiencies are generally the result of deliberate human choices (excessive audacity, errors and omissions, insufficient efforts)".

The 2016 paper [13] therefore took a rather extreme, but logical line of thinking, stating that dams' failures find, in the vast majority, their root-causes in human choices and not in natural events. At the center of this reasoning there is the concept of credibility threshold. Many industries consider the limit of credibility at around 1/100,000 to 1/1,000,000 (10^{-5} to 10^{-6}) [15], [16], [17], [18] so it can be stated that any event above that limit is not an "Act of God" (or, following modern times buzz-words a "Black Swan") and should be therefore foreseen/

planned for. It was also noted that, reportedly, most dam failures have occurred for other causes than “Black Swan” natural events, but again for “chains” of gradual deviances, which become “normalized” over time, stemming from investigations, design, construction, management and long-term monitoring.

For the e-IDC four different functions (Geotechnical Investigations, Engineering, Construction, Management (including Water balance), Monitoring) constituting the chain of elements responsible for success/failure of a dam were identified. A “causalities network” was built as shown in Figure 1 using the same concept of a Bayesian network.

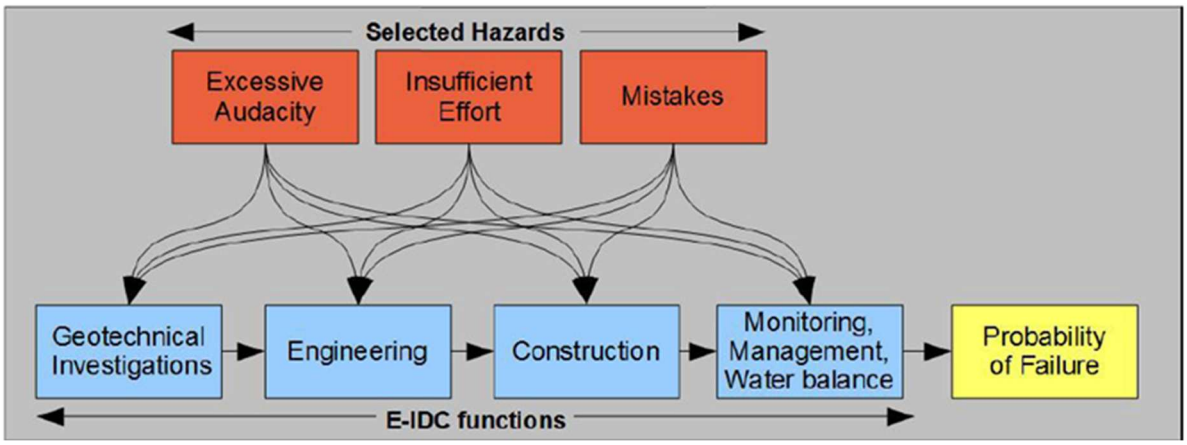


Fig. 1 – Functional scheme of the e-IDC with the hazards selected (Oboni, Oboni, 2016)

Various hazards (root causes) are lurking on each element of Figure 1 such as, in the specific approach adopted in that paper: Insufficient effort, Mistakes, Excessive Audacity, etc. leading to a probability of failure PoF for each element evaluated using a reliability model. Of course, the list of selected hazards should be discussed project by project. Failure modes previously identified in the literature, for instance ICOLD [19]) and from a 1910-2009 compilation [11] due to the selected hazards were selected as:

- Slope Instability,
- Earthquake and Mine Subsidence,
- Overtopping,
- Foundation,
- Seepage and Structural.

In Table 1, the rater wide difference in the percentage split of causality is due to the databases poor quality, requiring the study to proceed with a range rather than a single value to include uncertainties.

Table 1 –Attributed Causality Scenario stemming from project inception based on ICOLD and world-wide 1910-2009 data. NB: data had to be reorganized to comply with the selected taxonomy [13].

	ICOLD		World-wide 1910-2009 data	
Attributed Causality Scenario	Recorded failures	Failures in %	Recorded failures	Failures in %
Poor investigation	9	13	12	13
Engineering error & omission, excessive audacity	44	63	32	35
Poor Construction	17	24	48	52
Total	70	100	92	100

Two possible types of mitigations M1, M2 were then considered [13] , for the sake of an example. Their implementation was assumed to occur during the e-IDC development:

- M1: Engineering performance can be enhanced with an independent peer review (including sensible risk based decision-making procedures and risk assessment from project inception). Engineering and Peer Review become a parallel subsystem, possibly fraught by Common Cause Failure (CCF).

- M2: Monitoring, Management, Water balance function can be enhanced with Inspections paired to sensible risk based decision-making procedures and risk assessment from project inception.

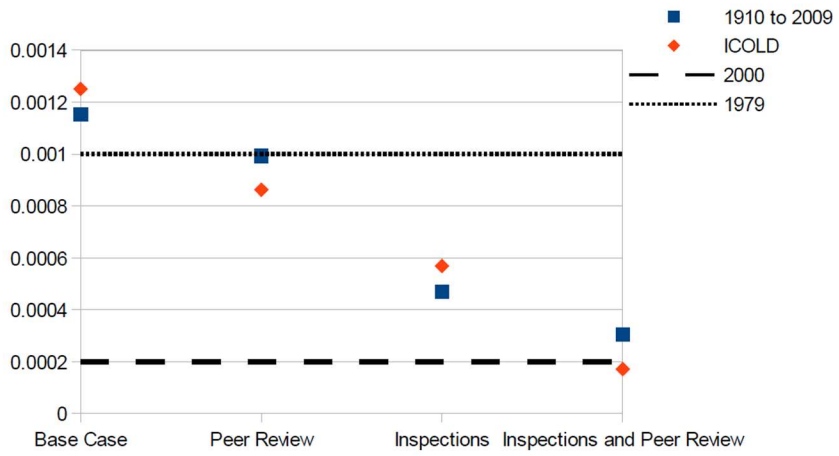


Fig. 2 – Results of the annual probability of failures derived from the model for mitigation level A,B,C,D including attributed causalities (Table 2, 3) derived from factual data (Fig. 2, 3). NB: numbering of Tables and Figures refer to the original paper [13].

Again CCF has nefarious potential on this additional parallel subsystem. Adopting a very simplified approach to CCF it was possible to assume that insufficient rigor, complacency,

conflict of interest, common excessive audacious approach in M1, M2 could reduce the expected positive result of any mitigation to nil. Four levels of mitigation were studied.

As it can be seen from Figure 2 the paper concluded that a dam sporting a probability of failure PoF of 10^{-3} could see it reduced to $2*10^{-4}$ by simply adding rigorous and unbiased peer-review and detailed inspections with swift follow-up without any changes to its geomechanical FoS or geomechanically derived PoF. Obviously this result was and is generic and should be integrated with all the dam-specific details of a candidate dam to be risk assessed: the devil is indeed always in the details!

Based on the above and more experiences made in risk analyses, a number of features aiming at describing the “devils” were introduced in the methodology. A summary is delivered in the next section.

4. ROOT CAUSES, UNDERLYING HAZARDS, MEASURES, PARENT-OF SUMMARY

A list of root causes (leftmost two columns) that appeared to cover the set of world-wide dams studied to date was introduced. The list is of course subject to changes in the future but represents in our eyes the minimum set that is absolutely necessary to deliver an image of the risks generated by a tailings dam.

As it can be seen in table 2, there are many hazards that cover the entire system, and many common “parents” within the system. Indeed governance, human errors and other “soft issues” have ubiquitous effects on a dam system and become “parents” to other hazards within the considered system. These constitute the Key Performance Indicators of the dam (KPIs).

Table 2 – Summary of Root Causes, Deficiencies, and Causalities in Tailings Dam Failures

Hazard & Deficiencies	Measurable as	Applies to...	Parent to...	Participates to...
Poor quality/documentation	Probability	Foundation, dam body	Foundation	All stability failures: ESA, USA, seismic, residual/liquefaction
Poor quality, excessive audacity	Probability	Foundation, dam body, dam crown	Dam body	ESA, USA, seismic, residual/ liquefaction, internal erosion
Poor geotechnical, hydrological understanding	FoSs probabilities	Foundation, water table, dam materials	Dam body	PoF for ESA, USA, seismic, residual/ liquefaction
Poor quality/documentation (Construction)	Probability	All dam system elements	Dam body, d/s face	All stability failures, erosion of D/S face

Poor quality/deficient analyses	Probability	All dam system elements	Dam body	All stability failures: ESA, USA, seismic, residual/ liquefaction
Poor quality/documentation /operations	Probability	All dam system elements	Dam body, water manage-ment	All stability failures: ESA, USA, seismic, residual/ liquefaction
Supernatant pond (engineering deficiencies)	Anomalies	Dam body, dam crown	Dam body	Overtopping failure, oversteepening
River at toe (hydraulics/climate change)	Erosion	Dam body	Dam body	“Wet” stability failure
Weir/spillway (operational deficiencies)	Mainte-nance	Dam d/s slope	D/s face	Overtopping failure
Penstock/gallery (operational deficiencies)	Mainte-nance	Dam internal structure	Dam body	Overtopping, internal erosion
Diversion ditches (operational deficiencies)	Mainte-nance	Dam crown	D/s face	Overtopping failure, catastrophic
Pipeline at crest (operational deficiencies)	Mainte-nance	Dam crown	Dam d/s face	Erosion, stability failure
Active/Inactive deposition (human error)	Aggrava-tion	Dam body	Dam body	Static liqu./ residual failure
Heavy equipment vibrations (human error)	Vibrations	Dam body, dam toe	Dam body	Static liqu./ residual failure
Blasting operations (human error)	Vibrations	Dam body, dam toe	Dam body	Static liqu./ residual failure
Exceedance of piezometric levels (human error)	Piezo-metric	Dam body, dam toe	Dam body	Static liqu./residual failure
Pre-existing weak layer (geology/geomor-phology)	Pmin-Pmax	Foundation	Dam body	All stability failures
Pre-existing landslide (geology/geomor-phology)	Pmin-Pmax	Foundation	Dam body	All stability failures

Footnotes:

1. ESA: Effective stress Stability Analysis
2. USA: Undrained Stability Analysis
3. Pmin Pmax: Probability min., max.4. d/s: Downstream
4. FoS: Factor of Safety

5. EVALUATION WORKFLOW

The methodology's compact algorithm is a quantitative semi-empirical approach.

The various hazards, elements constituting the homogeneous dam system to be evaluated are combined following the logic described in table 2, organized in such a way to reduce redundant calculations and to avoid double counting. This leads to results that have been shown to match, to date, the world-wide portfolio behavior. Human factors and governance issues are of course built-in together with other paramount details.

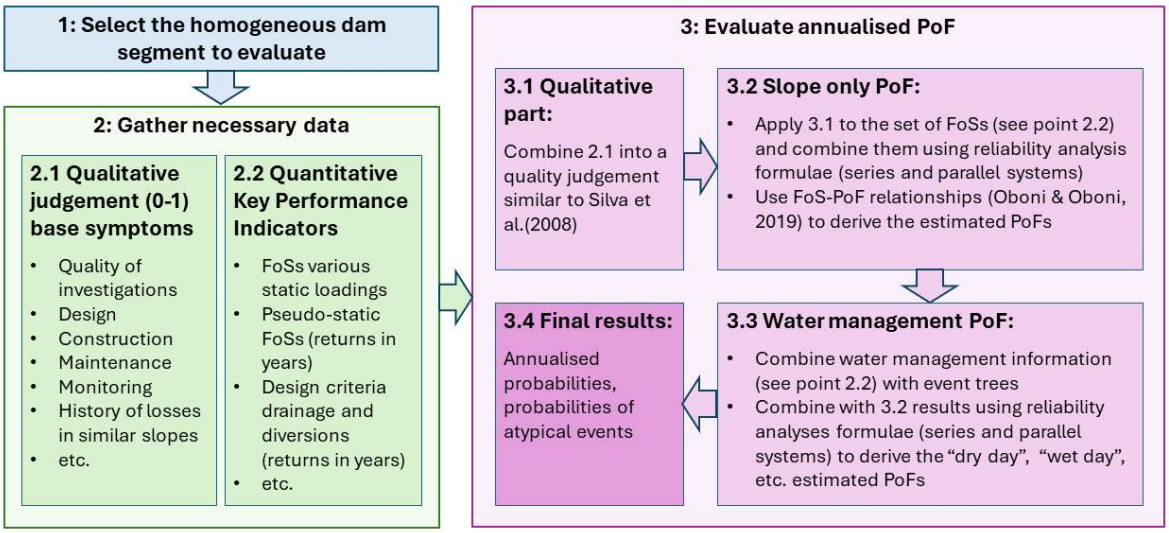


Fig. 3 –. Systemic probability evaluation workflow

As we will see later, this workflow makes it possible to establish pre-set values of families of underlying hazards to deliver a priori orders of magnitude of dams probabilities of failure, guide new users as shown in the example of "The Good, Bad and Ugly" dam below.

Also, as discussed later it is possible to prepare "scripts" guiding the formulation of what-if scenarios for mitigations such as for instance buttresses, as shown in the last section of this paper.

6. COMPARATIVE ANALYSIS OF THREE SIMILAR DAMS

6.1. Dams description: The Good, Bad and Ugly

In this section we focus on the comparative analysis of three active, centerline dams, bearing a main tailings distribution line, designed with the same FoS stability criteria:

- FoS drained (ESA): 1.4-1.6
- FoS undrained (USA): 1.3-1.4
- FoS pseudostatic : 1.2 for return =10,000 and 1.3 for return= 2,475

The three dams are under the same climatic conditions, with no water course at the toe. They have berms on the downstream face to control face erosion and are built with selected materials, well-engineered rockfill/earth fill, and a clay core with drainage blanket.

Table 3 – Example of the good, the bad, the ugly, to show the impact on the results using the methodology described in Figure 3.

The good	The bad	The ugly
Top of the line care for investigation, design, testing, construction, water balance return of anomalies 200yrs, excellent analysis. Excellent water management: weir (1:10000), diversion ditches (1:1000), no penstock, Pipeline at crest well protected (1:1000). Conventional monitoring and maintenance, inspections and reviews. No deviances from original design, no defects. Water levels under control. No liquefaction suspicions, no pre-existing slopes movements.	Average care/understanding for investigation, design, testing, construction, water balance is fair, same return (200yrs), Fair water management: weir (1:10000), diversion ditches (1:1000), no penstock, Pipeline at crest fairly protected (1:1000) All KPIs ok, also from the understanding point of view, but not excellent: Conventional monitoring and maintenance, inspections and reviews. No deviances from original design, no defects. Water levels under control. No liquefaction suspicions, no pre-existing slopes movements.	Poor care/poor understanding for investigation, design, testing, construction, water balance is poor, same return (200yrs), Poor water management: weir (1:10000), diversion ditches (1:1000), no penstock, Pipeline at crest poorly protected (1:1000) All KPIs below standards/poorly understood. Conventional monitoring and maintenance, inspections and reviews. No deviances from original design, no defects. Water levels under control. No liquefaction suspicions, no pre-existing slopes movements.

6.2. Comparative results (Good→Bad→Ugly)

As shown in Figure 4, the “good” dam lies at the bottom of the world-wide benchmark range, the “bad” dam at the top of the same range and finally the “ugly” is above the benchmark upper bound [20].

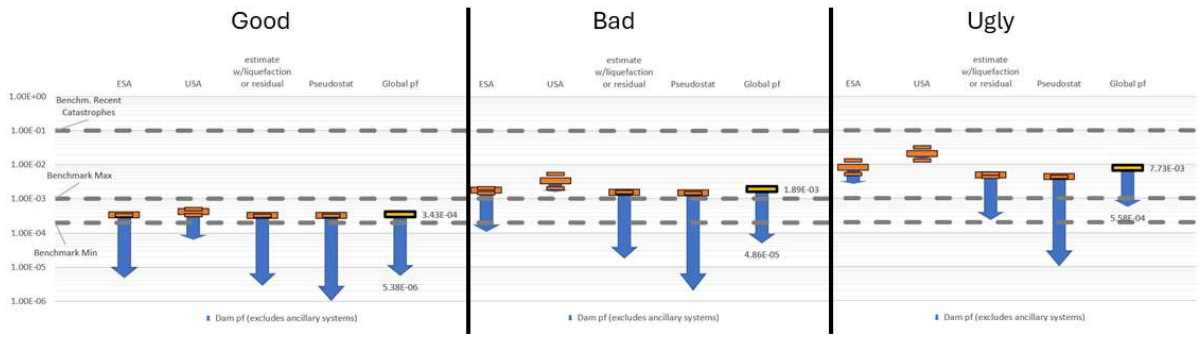


Fig. 4 – Estimated annualized probabilities of failure

It is interesting to compare the composite scenarios (Table 4, sunny day, extreme rain, etc.) among the three dams. as well as the relative causalities (Table 5). Note that the causalities are relative values and always add up to 100% by definition.

Table 4 – The good, bad and ugly failure mode and annualized PoF.

	Annualized PoF		
Failure Mode"	The good	The bad	The ugly
Sunny day	4.70E-06	4.86E-06	5.58E-04
WET day no AWTM failure	1.74E-05	4.07E-04	6.43E-03
Extreme rain event	3.65E-04	2.26E-03	1.00E-02
Quake<MCE	3.18E-04	1.45E-03	4.36E-03
MCE quake	3.18E-04	1.45E-03	4.36E-03
Liq stat/res	3.22E-04	1.53E-03	4.91E-03

Table 5 –Main causalities per dam.

Area	Themes/ comments	Good	Bad	Ugly
Investigations	Sampling; insitu testing; on site vane; soil classification, lab testing (shear, triaxial and oedometers)	24%	28%	28%
Design	Boreholes depth, density, trenches; cross section material; cross section type; berms & erosion control; project depth of analysis & detail; as built plans, alterations plans; stability analyses reasonableness	51%	42%	42%
Construction	Construction supervision; divergence from plans; known errors and omissions.	5%	5%	10%
Oper. Mon. Inspec., maint. & repairs	Deformations; pore pressure; operations and maintenance; inspections and review; repaired defects/damages	20%	20%	20%

In the good dam the "design" causality leads with 51%, Investigation 24%, Construction 5% and Inspections, monitoring and maintenance are at 20%. This last value does not change for the bad and the ugly, because the other factors variations are more significant in the specific case. Indeed, in the Bad and the Ugly dams investigation grows to 28%, while design reduces to 42% (because there are many factors of ignorance that impact the other key performance indicators). Hence Investigations and construction increase respectively to 28% and 10%.

7. PREPARING SCRIPTS TO STUDY MITIGATIVE ALTERNATIVES

7.1. Example for an existing inactive dam undergoing a possible buttress construction.

In this section we examine which data would change if a buttress were built at a dam, leading to the possibility of preparing a script for buttress mitigation.

Indeed, a number of dam's data will be better due to the studies to undertake to design the buttress and the enhanced stability conditions. Operations, maintenance, monitoring etc. should also improve.

For instance it can be assumed that the buttress would be based on enhanced geological and geotechnical understanding, selected materials, new stability analyses, deformations analyses and would be monitored under a well-designed inspections schedule and oversight by a competent EoR and independent review board.

This exercise has been conducted in various real-life cases and, as shown in the next sections has led to the evaluation of the ALARP level in cooperation with the EoR and the owner.

7.2. Other possible scripts

There are no limitations to the scripts that could be written to facilitate new users and quick scenario building for single dams, and even portfolios.

It appears that a good approach could be to develop three identical scripts per scenario, offering a good-bad-ugly set of alternatives to pick from.

The user would then:

a) Decide is the dam belongs to the "good-bad-ugly" quality category to start with

b) Insert the "as is" set of factors of safety delivered by the engineers for the dam.

c) Run as many scripts as needed to test alternative mitigations on the dam, including alternatives that may provoke a "jump" of quality category.

7.3. ALARP analyses

Several examples of ALARP (As Low as Reasonably Practicable) quantitative analyses have been published to date [21] including a development [22] that looks at the evolution of ALARP over time. Indeed, defining the ALARP should not be seen as a static process, as it has to encompass various time horizons along the life of a project/facility. Short term ALARP mitigation level may indeed be very different from the long term one due to different uncertainties, changing system conditions and external evolution, such as for example land use around the considered facility which will require adaptation.

The term ALARP is present in various guidelines, such as the Canadian Dam Association (CDA), the Mining Association of Canada (MAC), and the Australian National Committee on Large Dams (ANCOLD), as well as in GISTM/ICMM. In some older literature ALARP was defined as a probability level linked to harm to people; then it was defined as the trade-off equilibrium between risk mitigation costs and risk reduction. This has unfortunately introduced confusion among users and even regulators, oftentimes stemming precisely from the evolution of the definition (see Figures 5, 6 for an example). Let's note that ALARP should not be confused with tolerance thresholds. Indeed both the tolerability of the risk generated by a dam as well as its mitigation level reaching ALARP should be demonstrated.

In its most modern definition ALARP is reached when it is possible to demonstrate disproportion between mitigative cost and the risk-benefit gained.

Being able to demonstrate ALARP status, is paramount to various stakeholders such as owners, Engineer of Record (EoR), insurers, lenders, regulators and finally the public.

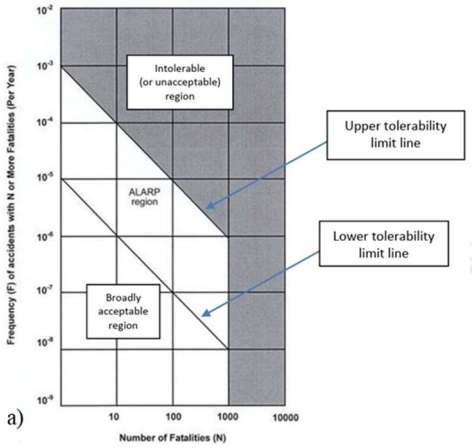


Fig. 5 –. Generic harm to people tolerance also called f-N curve. This curve features an obsolete definitions of ALARP.

Fig. 6 –. ICMM 2021's Concept of assessing benefit of mitigation effort to residual risks with ALARP at trade-off point, called LMALARP [22].

To evaluate the ALARP level of a structure mitigation measures proposed by the EoR with input from the TSF's owner and his technical Team must be used. Each proposed mitigation measure must be cost evaluated by the engineers/EoR, and then their effect on the dam system risks must be analyzed. This process requires constant interaction between the EoR, the technical Team and the risk assessment team following the steps below:

- The EoR proposes mitigation measures.
- The risk analysis team possibly makes other mitigation proposals.
- The mitigations are discussed and checked for constructability and possible sequence (say minimum 3 alternatives, max 6).
- The risk analysis team generates the PoFs of the mitigation alternatives.

Using the results of the previous step, the EoR and the technical team will then rank the mitigation measures by increasing risk mitigation effectiveness and cost, allowing to evaluate the minimum ALARP mitigation level (see Figure 6), i.e. the level that corresponds to the "equilibrium" between risk reduction and mitigative cost.

Figure 7 displays the ranking of various composite mitigation actions sorted by decreasing probability of failure of a real-life dam. Once the results of Figure 7 are available, the risk analysis team builds point by point (that is 3 to 6, depending on how many alternatives were selected) the risk vs. mitigative cost curves and evaluates the minimum ALARP, that is the trade-off point of Figure 6. Figure 8 displays an example of Risk reduction vs. CAPEX graph for the real-life dam discussed in Figure 7. In this example the minimum ALARP is therefore located at the completion of the "erosion protection" mitigation task and the final ALARP will be located to the right of that level.

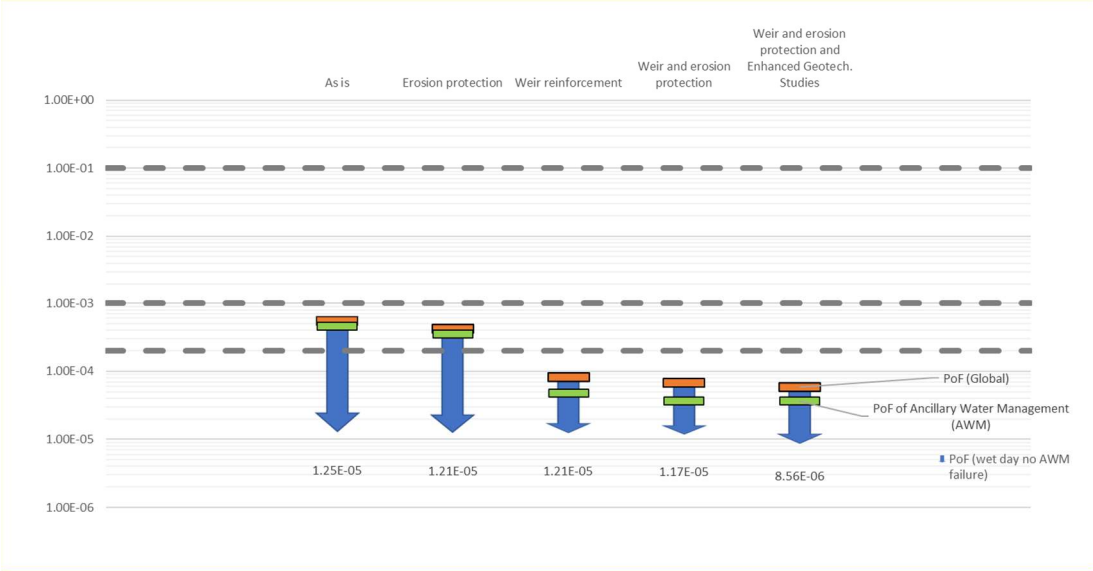


Fig. 7 –. Reduction of the PoF as a function of the chain of mitigations starting from "as is" state.

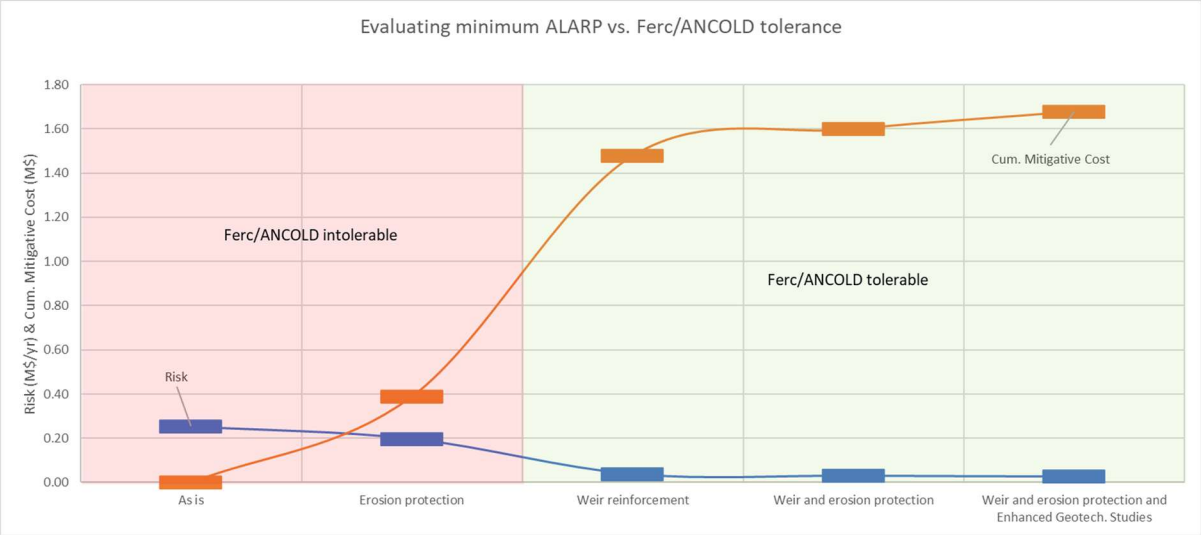


Fig. 8 –. Finding the "minimum ALARP" (intersection of the risk reduction (blue curve) vs. mitigative investment (orange curve). The pink area corresponds in this example to mitigation levels that are still intolerable from the harm to people point of view, i.e. FERC/ANCOLD intolerable.

It is paramount to remember that, as it is expected that all mitigations may have side-effects, these should be carefully studied before the final decisions are made.

The ALARP process, as described above, is iterative and demands a nuanced evaluation that balances risk reduction and constructability with cost considerations. By systematically analyzing each mitigation's effectiveness and cost impact, the process ensures that the selected level of mitigation is both economically viable and aligned with safety objectives. As already mentioned, this approach allows stakeholders—owners, engineers, and regulatory bodies alike—to make informed, responsible decisions regarding dam safety. The model's flexibility in adapting to new mitigation measures and evolving risk thresholds underscores its value in TSF management, providing a structured yet adaptable framework that upholds safety as system conditions change. The resulting ALARP assessment, when conducted transparently and in collaboration with the EoR and risk analysis teams, strengthens the decision-making process and facilitates an approach that is both justifiable and resilient over time.

8. CONCLUSIONS

This paper presents a quantitative approach to tailings storage facilities risk assessment that integrates evolving hazards and human factors, bridging gaps left by traditional methodologies. The probabilistic causality model described in this paper quantitatively demonstrates the central role of human factors, underscoring how design choices, maintenance practices, and operational decisions influence risk in some cases more significantly than natural events alone. The comparative analysis of three dam cases – "Good," "Bad," and "Ugly" – highlights the

robustness of the model in differentiating risk levels and in guiding practical mitigations. Indeed, the model effectively demonstrates its capacity to differentiate hazard and hence risk levels based on real-world operational variables.

By enabling tailored risk assessment scripts and scenario planning, this approach offers a replicable framework that can support TSF owners, engineers, and stakeholders in pursuing a risk-informed strategy toward enhancing dam safety and resilience. The model not only contributes to a more nuanced understanding of TSF risks but also aids decision-making, offering pathways toward achieving safety standards that align with public, regulatory, and environmental expectations.

The implementation of ALARP concepts using the presented model further enhances its utility, offering a structured approach to evaluate and balance risk mitigation efforts against practical constraints and costs. This adaptability is a key contribution, as it enables decision-makers to apply the model in diverse scenarios, from new project planning to the assessment of existing facilities. The capability to perform quick scenario planning is essential for identifying effective risk mitigation strategies, particularly as TSF operations face increased scrutiny and evolving regulatory demands.

Beyond immediate applications, the model paves the way for future improvements in TSF safety, emphasizing the importance of continuous monitoring and adaptation as conditions change over time. This dynamic approach to risk assessment aligns with the growing need for sustainable practices and resilient infrastructure, particularly as climate change and demographic shifts alter the environmental and social context in which TSFs operate. By integrating this advanced risk model, practitioners can achieve a comprehensive understanding of risk factors and proactively manage uncertainties, thereby supporting TSF safety goals that are responsive to both public and environmental concerns.

This work not only provides a more reliable method for assessing TSF risks but also lays a foundation for further research into the role of human factors and complex causality in industrial safety. By advancing these insights, the paper contributes to the development of TSF risk management practices that better meet the expectations of regulators, insurers, and communities, ultimately fostering a path toward safer and more sustainable dam operations globally.

REFERENCES

[1] U.S. Committee on Large Dams, Committee on Tailings Dams (USCOLD) (1994) Tailings Dam Incidents. Denver: U.S Committee on Large Dams.

[2] UNEP, E., 1996. safety incidents concerning tailings dams at mines: results of a survey for the years 1980–1996 by Mining Journal Research Services, Report prepared for United Nations Environment Programme. Industry and Environment, Paris.

[3] WISE Uranium Project, 2019 WISE Uranium Project

[4] Rana, N.M., Ghahramani, N., Evans, S.G., Small, A., Skermer, N., McDougall, S. and Take, W.A., 2022. Global magnitude-frequency statistics of the failures and impacts of large water-retention dams and mine tailings impoundments. Earth-Science Reviews, 232, p.104144.

[5] Bowker, L.N. and Chambers, D.M., 2017. In the dark shadow of the supercycle tailings failure risk & public liability reach all-time highs. Environments, 4(4), p.75.

[6] CSP2- Tailings Dam Failures, http://www.csp2.org/tsf-failures-from-1915

[7] Oboni, F. and Oboni, C., 2020. Tailings Dam Management for the Twenty-First Century. Springer International Publishing.

[8] Taguchi, G., 2014, Fault tree analysis of Slurry and Dewatered Tailings Management- a framework, Master's Thesis, UBC, Vancouver.

[9] FERC, Federal Energy Regulatory Commission, 2016, Engineering guidelines for the evaluation of hydropower projects chapter 18 – level 2 risk analysis.

[10] Oboni, C., Oboni, F., 2022, Optimizing mitigation of tailings dams portfolios, TMW 2022, Denver.

[11] Azam, S. and Li, Q., 2010. Tailings dam failures: a review of the last one hundred years. Geotechnical news, 28(4), pp.50-54.

[12] Gloor, M., Halter, G., Oboni C., Oboni, F., 2022, Impact of climate change projections on tailings dams survivability, CIM 2022, Vancouver, Canada.

[13] Oboni, F., Oboni, C., 2016, A systemic look at tailings dams failure process, Tailings and Mine Waste 2016, Keystone, Colorado, USA

[14] Oboni, F., Caldwell, J., Oboni, C., 2016, Ten Rules For Preparing Sensible Risk Assessments, Risk and Resilience Mining Solutions.

[15] Comar, C., 1987, "Risk: A pragmatic de minimis approach." In De Minimis Risk, edited by C. Whipple, pp. xiiixiv. New York: Plenum Press,

[16] Wilson, A.C. & Crouch., E., 1982, Risk/Benefit Analysis; Ballinger Publishing Company, Massachusetts.. pp. 92-93;

[17] Renshaw, F. M., 1990, A Major Accident Prevention Program, Plant/Operations Progress 9, no. 3 (July), 194-197

[18] Molina, T., 1990. CSNI Workshop on PSA Applications and Limitations (No. NUREG/CP-0115; SAND-90-2797; CONF-9009346-). US Nuclear Regulatory Commission (NRC), Washington, DC (United States); Sandia National Lab.(SNL-NM), Albuquerque, NM (United States).

[19] ICOLD, U., 2001. Tailings dams–risk of dangerous occurrences, lessons learnt from practical experiences (bulletin 121). Commission Internationale des Grands Barrages, Paris, 155.

[20] Oboni, C., Oboni F., 2013, Factual and Foreseeable Reliability of Tailings Dams and Nuclear Reactors -a Societal Acceptability Perspective, Tailings and Mine Waste 2013, Banff, AB , November 6 to 9.

[21] Oboni, F., Oboni, C., 2022a, A case study on ALARP optimization, Tailings 2022, Santiago de Chile.

[22] Cueto, I., Ledesma, O., Oboni, F., 2024, Incorporating Lifespan Considerations into ALARP Decision-Making for TSFs: A Novel Methodology, TMW 2024, Vancouver.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

CONTAINMENT OF CONCRETE DAMS COMPACTED WITH ROLLER USING IRON AGGREGATE

Frank Pereira[1]
Vale S.A.

João B.M.Carmo[2]
Vale S.A.

Walton P. Andrade[3]
Engeconsol Ltda

ABSTRACT

The motivation for this paper arose from the need to present to the technical community the care taken with the execution of the Downstream Containment Structures, in Portuguese *Estruturas de Contenção a Jusante* - ECJ, built by VALE S.A. after the accidents that recently occurred in the upstream dams in the state of Minas Gerais. The execution of these works in Roller Compacted Concrete - RCC, due to the emergency context of the construction period, was a milestone for Brazilian engineering as well as a great challenge. The option of building in RCC enabled the rapid implementation of safe walls that after a few months it was possible to mitigate the imminent risk of dam ruptures upstream, protecting life in general. Two structures were built in the iron quadrangle region in Minas Gerais referred to here as ECJ 1 and ECJ 2. The production of RCC in the concrete plants of the two works totalled more than 700,000 m³. Concrete volumes were released with national records in daily and monthly RCC production. In this context, it is worth highlighting the Technological Control applied in these works, the analysis of the results of fresh and hardened concrete tests, as well as the coring drylling campaign and post-construction investigations. The evaluation of the results will be presented at the end to guarantee compliance with the criteria established to fulfill the specified requirements.

Keywords: Dams, Concrete, Containment, Tailings.

[1]frank.pereira@vale.com, Brazil
[2]jboscomc70@hotmail.com, Brazil
[3]wpacelli@gmail.com, Brazil

1. INTRODUCTION

The containment structures built in RCC were planned in such a way as to prioritize the selection of the most appropriate constituent materials for each situation, which was essential to obtain concrete that met the specified properties. Verifying the durability of the structures, especially proving the inhibition of expansive reactions verified in the Alkali Aggregate Reaction (RAA) tests, was essential in the test program. The works had to be completed in record time to ensure the protection of the cities located downstream of the dams at a critical stability level. In this sense, the first choice was to use CP III cement with a blast furnace slag content of 50%. This definition was based on studies carried out in several similar works and technological studies carried out by Furnas Centrais Elétricas S.A. in its concrete laboratory in Goiânia-Brazil. Cements CPIII-40 RS cement was predominantly used, and both ECJs. The use of aggregates from mining waste was beneficial due to the high density of these materials in relation to the gneiss rock aggregates available in the construction region. This decision was also strategic because it provided gains in RCC densities 20% higher than conventional aggregate concrete, directly reflecting the reduction of the volume of the structure, reduction of the construction time and consequently an environmentally correct action by reusing industrial waste. In ECJ 1, the Concentration Plant Granulate – GIC, from Vale's Mine, a waste with low iron content but high density, of the order of 3500 kg/m^3, was applied. In ECJ 2, the Compact Itabirite provided was applied. Both tailing aggregates had a maximum size aggregate dimension of 50 mm and were mostly used with two granulometric ranges, a coarse fraction retained in the 12 mm mesh and another fine fraction, with a percentage of material passing through the 200 sieve (0.075 mm) sufficient to ensure adequate packing of the RCC. The conventional concrete was manufactured with gneiss rock aggregates with a Maximum Size Aggregate - MSA of 19 mm and artificial sand, and in ECJ 2 it was also possible to apply pumped concrete with heavy aggregates. The GIC, as well as all aggregates of gneiss origin, were transported by rail to the construction site. The high Los Angeles abrasion restrictions for coarse aggregates for dam mass concrete are not suitable and are restricted to rockfill structures exposed to the elements or hydraulic surfaces exposed to high velocities. This proves that in the granulometry of the RCC there may be small acceptable variations to the theoretical limits of the theoretical Bolomey curve. This mass balance must be studied so that there is no segregation of the coarse fraction in the mass. The fine content will compensate for this small granulometric mismatch, which, when corrected by the Fineness Module method, will maintain the unitary amount of water in the mix design and the mixture can be applied without difficulty. The high Los Angeles abrasion restrictions for coarse aggregates are limited to rockfill structures exposed to the elements or hydraulic surfaces exposed to high speeds. The durability effect of aggregates of hematite origin, with FeO_2 contents such as those in GIC,

of around 35%, does not compromise the useful life of the concrete, as evidenced by the durability tests performed. To date, 5 years after the construction of the ECJ, no problems with the durability of the RCC have been observed in these works. The carbonation normally observed in concrete structures due to eflorescence may present a reddish coloration in concretes with hematite, but only causing an aesthetic effect. The use of hematite in its natural state, without heat treatments, would not promote the phenomenon of mass oxidation, since the RCC used will not be subjected to high temperatures capable of altering the natural effect of the oxidation phenomenon and compromising durability.

2. MATERIALS

Portland Cement

Table 1 presents the results of characterization tests of the main cements used in the works.

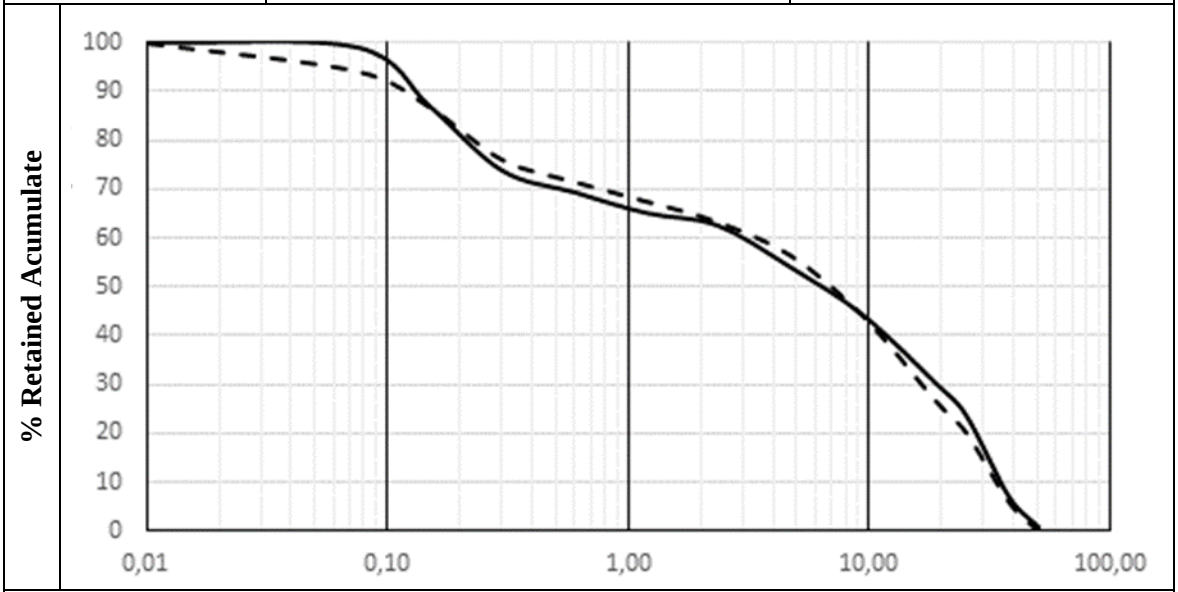
Table 1 – Characterization tests of the cements used.

Properties			ECJ 1	ECJ2	Limits NBR 5735/91
Specific Mass (g/cm^3)			2.95	2.97	-
Fineness	Residue sieve # 200 (%)		0.44	0.50	< 8.0
	Residue sieve # 325 (%)		2.1	1.8	-
	Specific area (m^2/kg)		4094	3960	-
Setting Time (min)	Start		152	148	> 60
	End		207	210	< 720
Consistency Water (%)			30.44	30.89	-
Autoclave expansion (%)			-	-	< 0.8
Compression strenght (MPa)	3 day		23.31	22.6	> 10
	7 day		36.04	35.5	> 20
	28 day		49.79	47.6	> 32
	90 day		-	-	-
Chemical components	Loss to fire (%)		2.61	4.21	< 4.5
	Insoluble residue (%)		0.43	0.86	< 1.5
	Sulfor trioxide (SO_3) (%)		2.09	1.77	< 4.0
	Magnesium oxide (MgO) (%)		4.54	3.76	-
	Silicon Dioxide (SiO_2) (%)		29.87	25.83	-
	Iron oxide (Fe_2O_3) (%)		2.04	2.50	-
	Aluminum oxide (Al_2O_3) (%)		8.31	7.60	-
	Calcium oxide (CaO) (%)		51.82	49.56	-
	Free calcium oxide (CaO) (%)		-	0.77	-
	Total alkali	Sodium oxide Na_2O (%)	-	0.11	-
		Potassium oxide K_2O (%)	-	0.41	-
		Alkaline equivalente Na_2O (%)	-	0.34	-

The variations in the aggregate granulometry were ajusted using one mix design program developed by the Contractor (*Procodos* aplicative) for concrete, respecting the limits of

theoretical curves. In the case of RCC, the Bolomey curve was applied. All tests performed with artificial sand and coarse aggregates met the limits established for their application. The RCC granulometry was also performed before and after the mixture passed through the PugMill of the RCC Plant. There was no significant loss of larger grains because care was taken in the mixing process using the blades, sand lining at the base of the mixer, and inversion of the last row of blades, so that there was no influence on the composition of the RCC, as shown in Figure 1.

Before PugMill			After PugMill	
Sieve mm	% Retained	% Retained Acumulate	% Retainde	% Retained Acumulate
50.80	0.00	0.00	0.64	0.64
38.10	6.11	6.11	6.35	6.99
25.40	14.21	20.32	16.51	23.50
19.05	6.30	26.62	6.56	30.06
9.53	17.31	43.93	13.94	44.00
4.75	12.56	56.48	9.88	53.88
2.36	6.88	63.36	8.58	62.46
1.18	4.29	67.65	2.63	65.09
0.60	3.93	71.58	4.20	69.29
0.30	4.56	76.14	4.44	73.73
0.15	10.74	86.88	13.22	86.95
0.075	7.37	94.25	12.05	99.00
MF	4.99		4.92	



Before PuggMill -------- After PuggMill

Figure 1 – Data from the RCC granulometry tests before and after the PugMill mixer – ECJ 1.

Figure 2 below shows the images of the special aggregates used in the concrete.

ECJ 1 - Agreggate GIC	ECJ 2 - Itabirito Compact

Figure 2 – Industrial waste aggregates used in the construction of ECJs.

Alkali/Aggregate Reactivity

The Alkali Aggregate Reactivity – AAR, was researched for all aggregates applied in the work in accordance with ABNT NBR 15777, proved that the aggregates were innocuous.

Definition of RCC Mix Design

The mix design of the RCC were carried out before the start of the construction. The Solocap Laboratory, specialized in road works, was adapted to meet the RCC tests. Due to the emergency need, it was necessary to do some adjustment of the laboratory facilities to carry out the RCC tests in the fresh and hardened state.

The mix design of the concretes was carried out following the Fineness Modulus Method. This method was suggested by Consultant Roy W. Carlson during the Itaipu Dam construction and adopted at the Furnas Centrais Elétricas Concrete Laboratory with some adjustment, in the midle of 1970s and widely applied on the main hydroelectric constructions in Brazil and later in several projects abroad. The Brazilian RCC standard tests published by ABNT in 2015 presented some characteristics intrinsic to the technology developed in Brazil.

It is worth highlighting the DMA – Device for determining the specific mass of RCC, developed at the Furnas Concrete Laboratory, by technicians Guilherme Leroy and Valdir Paulino de Oliveira and engineers Moacir Alexandre Souza de Andrade and Walton Pacelli de Andrade, taking advantage of a device presented in Designation 11, Figure 218. Apparatus for Direct Measurement of Surface, Moisture, Absorption, and Specific gravity of Aggregate PX-D-33523 (1988), available in the Concrete Manual A Water Resources Technical Publication Eighth Edition revised by the US Bureau of Reclamation. In ECJ 1, the RCC.6 mix design was the most used; in ECJ 2, the predominant mix was RCC4. The mix most applied in the works are presented below in Table 2

Table 2 RCC mixes most applied in the works

ECJ			ECJ 1	ECJ 2
Mix			RCC 6	RCC 4
fck	MPa		8.0	
Age of control	day		90	
Fineness modulus	-		5.44	4.71
Cement CP III (NBR)	(kg/ m³)		100	100
Water			140	155
Artificial sand			560	1279
Grave 2			2152	1443
Chemical admixture			1	1
Water / Cement W/C	-		1.40	1.55
Properties of Fresh and Hardened Concrete	Theoretical density (kg/m³)		3012	2.958
	Cannon Time (s)	Average	12.4	11.6
		S. Deviation	2.4	2.0
	Vêbê Density (kg/m³)	Average	2398	2.909
		S. Deviation	115	24,42
	DMA Density (kg/m³)	Average	2985	2.991
		S. Deviation	139	21,69
	Concrete temperature (°C)	Average	23.1	24.1
		S. Deviation	4.0	3.6
	Hardened RCC Density (kg/m³)	Average	2835	2.888
		S. Deviation	46	42
	Compressive Strength - Average (MPa)	3 day	2.1	2.1
		7 day	3.4	4.3
		28 day	6.1	9.6
		90 day	9.5	12.1
	Samples number (n)	90 day	118	163
Student Coeficient "t"			0.842	0.842
Variation Coeficient (%)			22.1	17.1
Fck real at the end of work (MPa)			8.0	9.9

3. QUALITY CONTROL

Figure 3.10-a presents the statistical data of the quality control, including the tests on fresh concrete and, in Table 3.10-b, on hardened concrete, for all RCC mixes. The tests on the properties of fresh concrete were carried out in accordance with ABNT Standards. The three parts of the Brazilian RCC standard are summarized below:

NBR 16312.1-Terminology – deals with technical terms, definitions of equipment used to characterize the properties of RCC in the fresh and hardened state, as well as field applications.

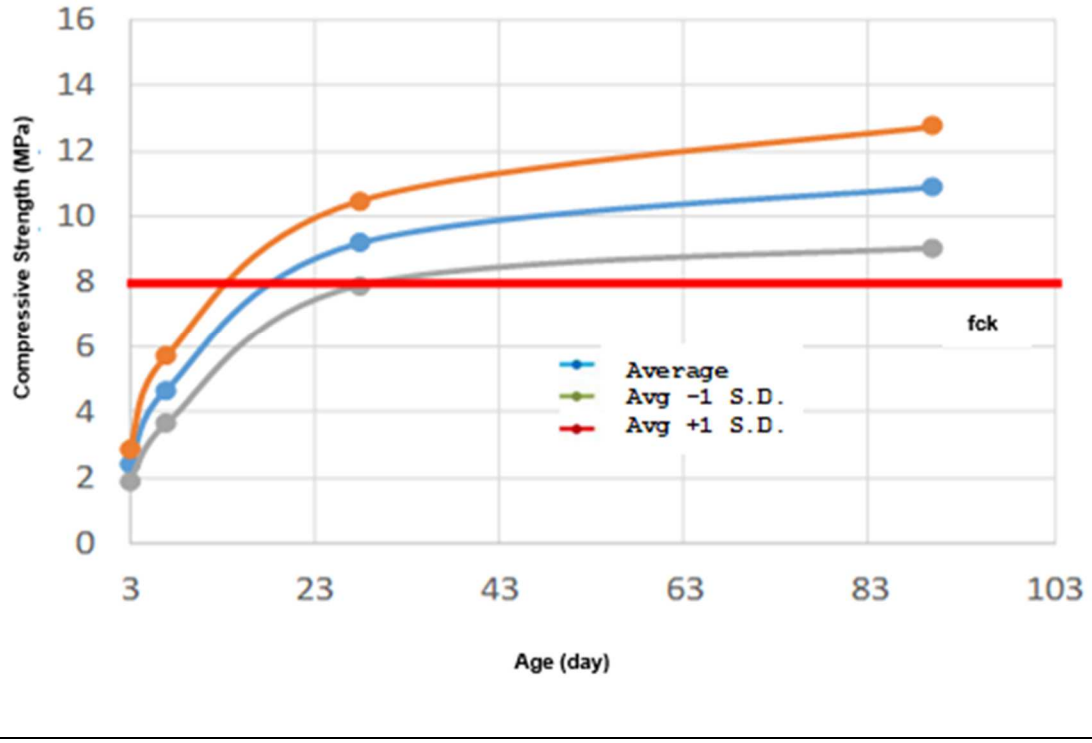
NBR 16312.2-Laboratory Preparation – Presents the laboratory equipment with its main characteristics, molding of test specimens on a vibrating table.

NBR 16312.3 - Laboratory Tests - Fresh Concrete - In addition to the DMA equipment, the other equipment used, and their respective characteristics are presented, as well as expected

values for mixtures with maximum compactness, correlating with compaction performed in the field with equipment with similar energy.

Results of the Strength Tests of the RCC Mix

The graphs of the evolution of the compressive strength at the ages tested, from the statistical analysis of the test results, are illustrated in Figures 4 and 5, referring to the mixes applied in the respective works.



RCC 4

Figure 4 – ECJ 1

Statistical Dates					
Mix	Age (day)	Samples number (n)	Compressive Strength – fck real (*) (MPa)	Standard Deviation (Mpa)	Variation Coeficient (%)
RCC.4	3	483	2	0.62	31
	7	483	4	1.12	32
	28	483	7	2.20	27
	90	479	11	2.89	24
Applied Volume: 205,231.63 m³					

(*) fck real is the recalculated fcj value (compressive strength) as a statistical parameter.

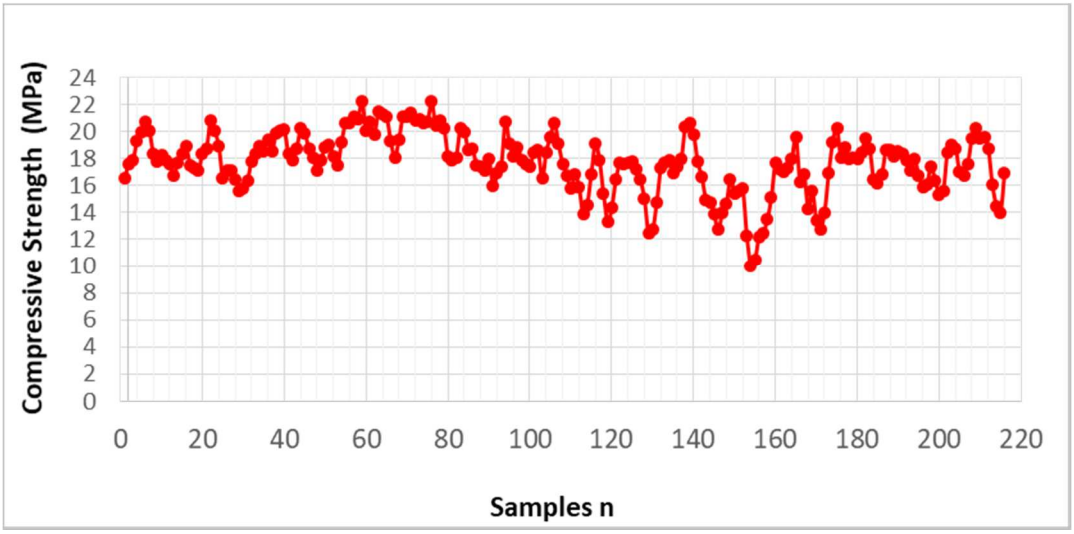


Figure 5 – Statistical dates RCC4 – Most Mix applied – 90 days – fcj - ECJ 2

In ECJ 2, the following results were obtained in the field with the RCC in the fresh state, presented in Figures 6 to 9.

Drill hole	Block	RCC Mix	Cement Content (kg/m³)	Project Fck (MPa)	Project Density 1° Phase (kg/m³)	Nuclear Densimeter Density (kg/m³)			
						Average	Std. Deviation	Coefficient of variation (%)	Number of Tests
AMCD 301 J	3	RCC.3 e RCC.4	100	8	> 2850	3031	57,75	0,02	52
AMCD 301 H	3	RCC.2, RCC.3 e RCC.4	100	8	> 2850	2990	47,20	0,02	55
AMCD 502	5	RCC.4	100	8	> 2850	2987	20,17	0,01	28
AMCD 502 B	5	RCC.4	100	8	> 2850	2987	20,17	0,01	28
AMCD 1001	10	RCC.6	90	6	> 2850	2993	23,80	0,01	20
AMCD 1001	10	RCC.5.1	125	12	> 2850	3028	23,08	0,01	46
AMCD 1001A	10	RCC.5.1	125	12	> 2850	3028	23,08	0,01	46
AMCD 1101	11	RCC.5.1	125	12	> 2850	3045	39,58	0,01	163
AMCD 1101 A	11	RCC.5.1	125	12	> 2850	3045	39,58	0,01	163
AMCD 1501	15	CCR.4	100	8	> 2850	2985	17,70	0,01	15

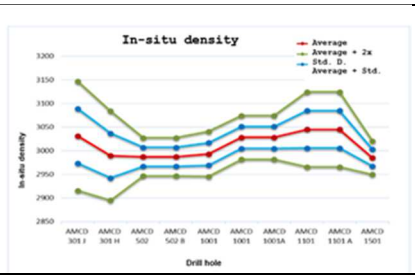


Figure 6 – Density Tests with Nuclear Densimeter

Drill hole	Block	RCC Mix	Cement Content (kg/m³)	Project Fck (MPa)	Project Density 1° Phase (kg/m³)	Nuclear Densimeter Density(kg/m³)			
						Average	Std. Deviation	Coefficient of variation (%)	Number of Tests
AMCD 301 J	3	RCC.3 e RCC.4	100	8	> 2850	3040	30,49	0,01	14
AMCD 301 H	3	RCC.2, RCC.3 e RCC.4	100	8	> 2850	3009	30,49	0,01	15
AMCD 502	5	RCC.4	100	8	> 2850	3024	15,78	0,01	17
AMCD 502 B	5	RCC.4	100	8	> 2850	3024	15,78	0,01	17
AMCD 1001	10	RCC.6	90	6	> 2850	3020	10,46	0,00	2
AMCD 1001	10	RCC.5.1	125	12	> 2850	3029	12,80	0,00	13
AMCD 1001A	10	RCC.5.1	125	12	> 2850	3029	12,80	0,00	13
AMCD 1101	11	RCC.5.1	125	12	> 2850	3026	32,45	0,01	29
AMCD 1101 A	11	RCC.5.1	125	12	> 2850	3026	32,45	0,01	29
AMCD 1501	15	CCR.4	100	8	> 2850	3037	13,88	0,01	15

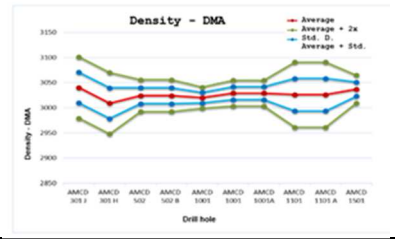


Figure 7 – Density Tests with DMA

Drill hole	Block	RCC Mix	Cement Content (kg/m³)	Project Fck (MPa)	Project Density 1° Phase (kg/m³)	Nuclear Densimeter Density (kg/m³)			
						Average	Std. Deviation	Coefficient of variation (%)	Number of Tests
AMCD 301 J	3	RCC.3 e RCC.4	100	8	> 2850	2958	20,00	0,01	14
AMCD 301 H	3	RCC.2, RCC.3 e RCC.4	100	8	> 2850	2927	19,50	0,01	15
AMCD 502	5	RCC.4	100	8	> 2850	2942	18,00	0,01	17
AMCD 502 B	5	RCC.4	100	8	> 2850	2942	18,00	0,01	17
AMCD 1001	10	RCC.6	90	6	> 2850	2936	15,30	0,01	2
AMCD 1001	10	RCC.5.1	125	12	> 2850	2945	12,00	0	13
AMCD 1001A	10	RCC.5.1	125	12	> 2850	2945	12,00	0	13
AMCD 1101	11	RCC.5.1	125	12	> 2850	2940	28,80	0,01	29
AMCD 1101 A	11	RCC.5.1	125	12	> 2850	2940	28,79	0,01	29
AMCD 1501	15	CCR.4	100	8	> 2850	2940	16,00	0,01	15

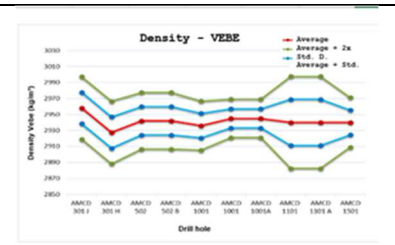


Figure 8 - Density Tests with Vêbê

Drill hole	Block	RCC Mix	Cement Content (kg/m³)	Project Fck (MPa)	Project Density 1° Phase (kg/m³)	Nuclear Densimeter Density (kg/m³)			
						Average	Std. Deviation	Coefficient of variation (%)	Number of Tests
AMCD 301 J	3	RCC.3 e RCC.4	100	8	> 2850	10	1,00	0,06	14
AMCD 301 H	3	RCC.2, RCC.3 e RCC.4	100	8	> 2850	11	0,60	0,06	15
AMCD 502	5	RCC.4	100	8	> 2850	11	1,00	0,06	17
AMCD 502 B	5	RCC.4	100	8	> 2850	11	1,00	0,06	17
AMCD 1001	10	RCC.6	90	6	> 2850	10	0,40	0,04	2
AMCD 1001	10	RCC.5.1	125	12	> 2850	11	0,92	0,08	13
AMCD 1001A	10	RCC.5.1	125	12	> 2850	11	0,92	0,08	13
AMCD 1101	11	RCC.5.1	125	12	> 2850	11	1,00	0,09	29
AMCD 1101 A	11	RCC.5.1	125	12	> 2850	11	0,95	0,09	29
AMCD 1501	15	CCR.4	100	8	> 2850	11	1,00	0,07	15

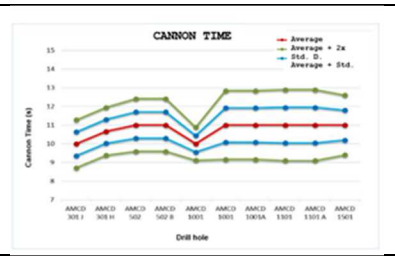


Figure 9 – Tests of Cannon Time

To compare the results of the density tests, the records made by (1) DMA (2) Nuclear Density Meter (3) Vêbê (4) Core Density were used and recorded in Table 3.

Table 3 – ECJ 2 Comparative Results

Statistical Data	Vêbê (kg/m³)	DMA (kg/m³)	Nuclear Densimeter (kg/m³)	Cores (kg/m³)
Averege	2.942	3.024	2.990	2.964
Standard Deviation (Max)	29	32	59	24

The densities of the concretes sampled in situ with a nuclear densitometer and the densities tested in the fresh state in the laboratory remained above the specified limit of 2,850 kg/m^3 for the first phase of the structure. The cannon time also remained within the range of the Technical Specification ET-1800HH-X-00002, with average values of around 12 seconds.

RCC CORE DRILLING – ECJ 1

The core drilling campaign was scheduled to verify the quality of the RCC applied and to correlate the properties obtained during the construction of the work with those obtained in the laboratory. This activity was systematically monitored by VALE and carried out by the company GEONORTE de Goiás. The extraction was carried out with equipment consisting of a double barrel, 1 m high, with 6” diameter cores. The hydraulic probe used was the Maquesonda 500. The position of the boreholes is shown in Figure 10.

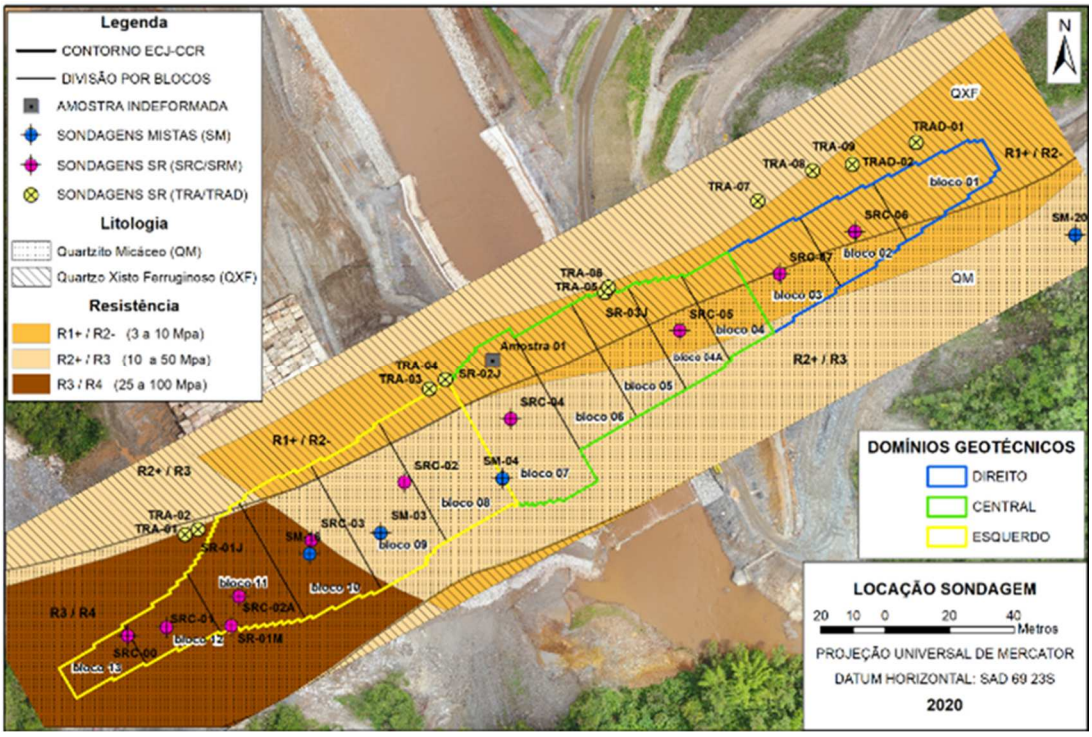


Figure 10 - Identification of Drill Holes – Original Designer

This activity was planned to verify the expected compliance with the properties of the hardened concrete in the structure itself.

General Appearance

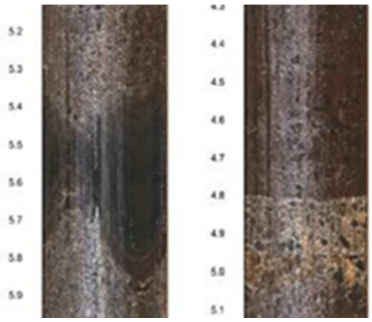
Figure 11 shows some photographs of the cores extracted from the Dam after 90 days, which occurred from February 2020.

Figure 11 – Cores extracted in ECJ 1

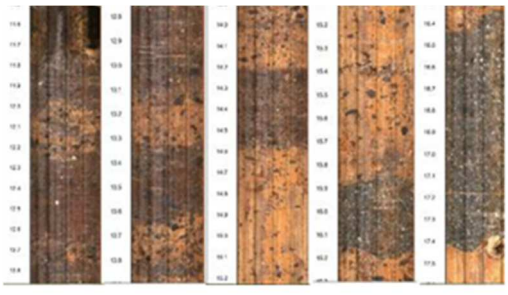
During the core drilling, television footage was taken of the inside of the holes, one of which is shown in Figure 12, hole SRC-01, contact of the RCC/ CC Conventional Concrete/Foundation Rock – not to scale, which demonstrates that the walls of the cores extracted and filmed by the company GEOSOL, when it was possible to verify the uniformity of the completely intact and closed walls, being described with insignificant voids of millimetric order.

a) Detail of the contact between the CC and the foundation rock, not to scale.

b) Detail of the contact between the CC and the foundation rock, not to scale.



c) Detail of the contact between the RCC, CC and the foundation rock, not to scale.



d) Detail of the RCC, CC and the foundation rock.

Figure 12 - Photos provided by GEOSOL by Alessandro Guimarães.

Test Results

The RCC cores were taken to the SOLOCAP Laboratory, where they were prepared and identified for testing of Specific Gravity, Absorption, Permeability, Compressive Strength, Modulus of Elasticity and for determining Cohesion and Friction Angle between a treated and untreated horizontal joint, normally obtained respectively with and without insertion of bonding mortar, with a placement interval of less than 24 hours.

Summary of RCC Core Test Results ECJ 1

In assessing the results of the tests performed on the cores, the results of the tests reported in the Technological Control Report performed for the RCC in the fresh and hardened state were considered, considering the RCC.6 mix applied in the periods corresponding to the elevations of the extracted cores. The recommended limit for density was presented in Technical Specification ET 1850DD-C-00001 Revision C, dated 08/30/2019, and was 2700 kg/m3.

The RCC cores were taken to the Solocap Laboratory, where they were prepared and identified for the Specific Gravity, Absorption, Permeability, Compressive Strength, Modulus of Elasticity tests and for the determination of Cohesion and Friction Angle between a treated and untreated horizontal joint, normally obtained respectively with and without insertion of bonding mortar, with a casting interval of less than 24 hours.

NBR 7680 – Item 6.1.7 allows corrections of 10% when extraction is done 50 cm below the top surface of the concrete of the structural component.

ACI 214-4R-03 - A correction factor of 1.06 is permitted on the average value due to damage during extraction.

Concrete Society Technical Report No. 11 - This document, prepared in 1976, allows the application of correction factors due to Excess of Voidage.

The following Table 4 presents a summary of the results of the core tests. These results were not subject to corrections of the values obtained in the tests presented.

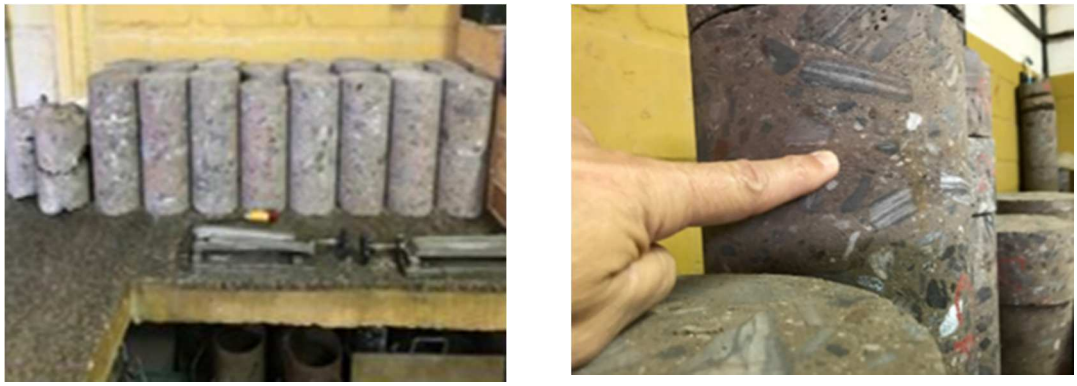
Table 4 – Results of the averages of the tests performed with the core tests – ECJ 1.

Hole		SRC-01	SRC-02A	SRC-05	SRC-06	SRC-07
Bloc		12	11	4	2	3
Density (kg/m^3)		3.094	3.148	2.953	2.924	3.043
Water absorption (%)		5.2	4.8	5.3	5.8	4.2
Compression Strength (MPa)		11.9	11.4	10.9	10.7	11.5
Splitting Tensile Strength – Brazilian Test (MPa)		1.6	1.4	1.4	1.6	1.4
Modulus of elasticity (GPa)		13.1	14.3	19.6	14.2	8.0
Permeability coeficient 10^-11 (m/s)		6.82	9.79	8.35	9.13	7.39
Direct shear	Laboratory	(*)	(**) SOLOCAP		(**) PUC -RJ	(***) SOLOCAP
	Cohesion (MPa)	0.200	0.899		0.690	0.614
	Friction angle (°)	45.0	45.3		48.4	70.1
Triaxial test Geocontrole laboratory (****)	Cohesion (MPa)	4.0				
	Friction angle (°)	51.3				

(*) Treated joint of RCC – SOLOCAP equipment; (**) Treated joint of RCC - Equipment normalized; (***) RCC No treated joint (****) Triaxial test joint RCC and Conventional Concrete.

The RCC cores were tested at the SOLOCAP Laboratory in Belo Horizonte to verify the untreated horizontal joints as well as those treated with bonding mortar. Figure 14 shows the

RCC joints and the good general appearance of the cores, very cohesive joints, comparable to similar joints.



a. Cores being prepared for testing. b. RCC joints without mortar.

Figure 14 – Cores prepared for direct shear tests on joints.

After checking the results of the samples molded in the laboratory and the contact samples collected in situ by means of rotary drilling, it was possible to establish a correlation between the results of two accredited laboratories using standardized equipment and procedures. The shear and friction angle results were correlated with the tests that proved the good adhesion between the layers and the bonding capacity between the compacted strips to be satisfactory, see Figure 15.

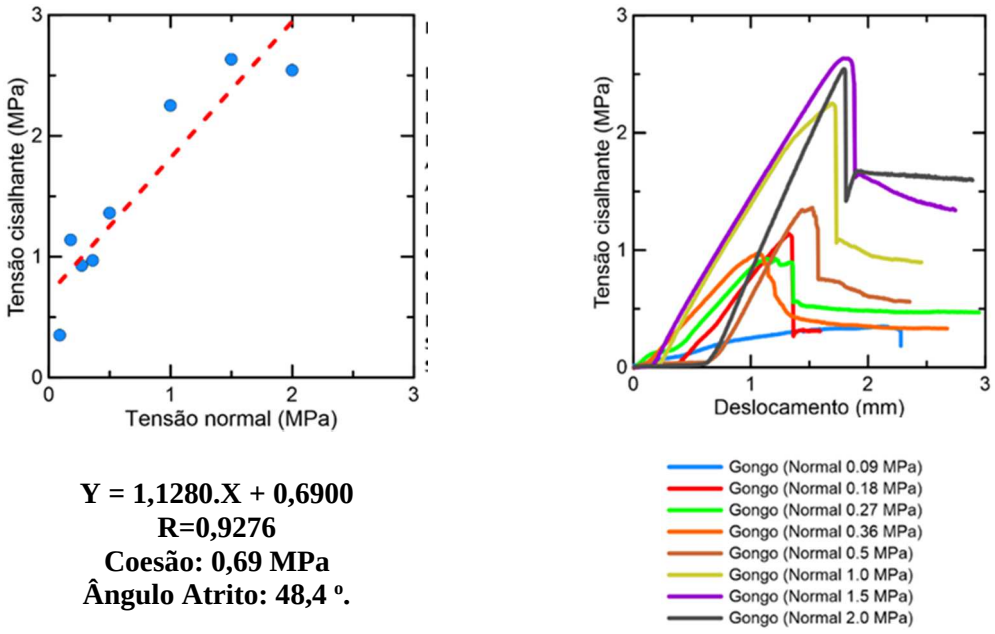


Figure 15 – Direct shear in untreated joints – PUC – RJ – ECJ 1 - Source: (CARMO 2023).

Table 5 presents the results obtained for the samples tested at the ECJ 1 construction site in the PUC-RJ laboratory, with satisfactory results and within the specified parameters, being considered adequate for the stability of the structure.

Table 5 – Data from the tests obtained on the ECJ 1 samples

Local	Normal strength (MPa)	Area (m^2)	Shear stress (MPa)	Friction angle (º)	Cohesion (MPa)
ECJ 1	0.09	0.00531	0.35	48.4	0.69
	0.18	0.00540	1.14		
	0.27	0.00552	0.93		
	0.36	0.00545	0.97		
	0.50	0.00529	1.36		
	1.00	0.00523	2.25		
	1.50	0.00522	2.63		
	2.00	0.00528	2.54		

Source: PUC – RJ – Laboratory.

Figure 16 shows the results of the tests performed at the SOLOCAP laboratory with samples of untreated joints, demonstrating good adhesion between the untreated joints with mortar up to 24 hours after the subsequent layer was placed. The results of the tests performed at the SOLOCAP laboratory, using equipment developed by this laboratory, were reduced by Professor José Marques Filho to an adjusted cohesion of 0.20 MPa and a friction angle of 45.0°. Although these values were reduced because they were performed on non-standardized equipment, they presented values considered satisfactory.

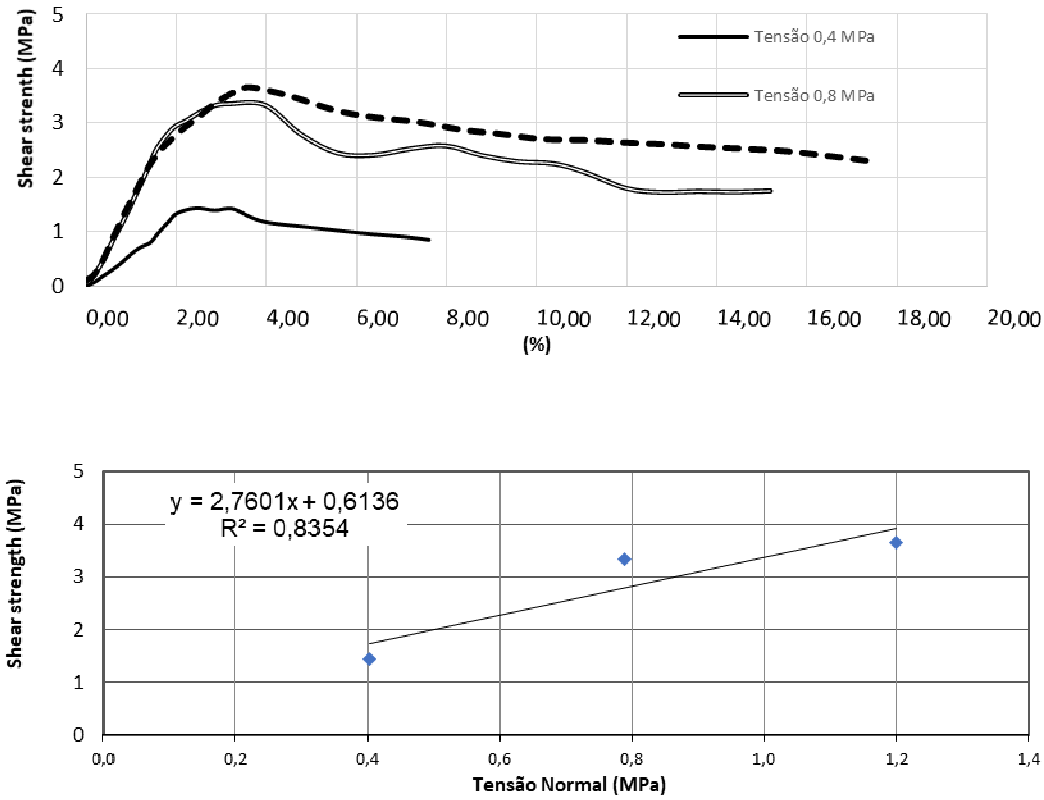
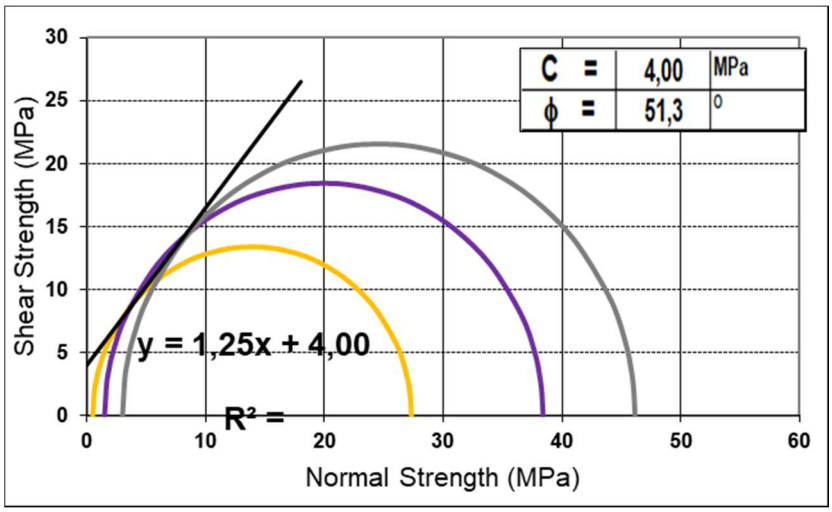
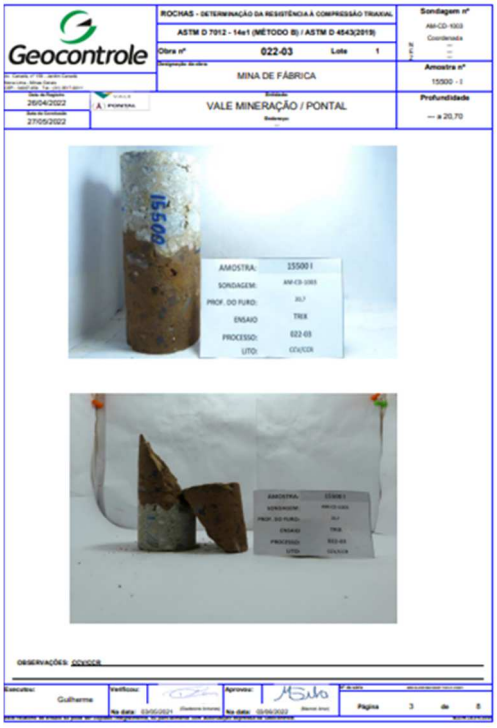


Figure 16 – RCC ECJ 1 -. Direct shear in untreated joints after 24 hours of laying. Source: SOLOCAP laboratory.

A complementary test was also performed with the RCC core extracted from the ECJ 2 construction site, covering a region between the contact between the Conventional Concrete and the RCC composed of compact itabirite aggregate. The tests were performed at the GEOCONTROLE laboratory, and the results are presented in Figure 18.



a. Core prepared for test.



b. Joint beetween RCC/ Conventional Concrete without mortar.

Figure 18 – Triaxial RCC/CC test - Equipment GEOCONTROLE laboratory and recording of a sample from ECJ2.

RCC Control in Fresh State

The specific gravity measured by VêBê and DMA shows oscillations resulting from the variation in the composition of the aggregates and the difference between the incorporated air contents, with the VêBê results being influenced by the occluded air, and the DMA value corresponding to the theoretical density.

The oscillation of the incorporated air content was from 0.55% to 2.45%. Proof that the mixes were well adjusted can be seen in Table 6.

Table 6 - Correlation of Theoretical and Measured Air in the Cores of ECJ 1

Hole	% Theorical trapped air in cores (*)	% Theorical air DMA x Vêbê (**)		
SRC-01	1.5	Lote 1	Lote 2	Lote 3
SRC-02 A	2.8	2.4	2.1	1.6
SRC-05	2.6			
SRC-06	1.4			
SRC-07	1.4			
Average	1.9	2.0		

(*) % Theoretical Air = (DMA Density - Core Density) / DMA Density) x 100

(**) % Theoretical Air = (DMA Density - Vêbê Density) / DMA Density) x 100

This regularity of the results obtained for the densities was greatly facilitated by the corrections frequently performed by the laboratory in the unit water content, adjusted and verified by the DMA device.

Compressive Strength

The minimum required average strength specified for the RCC was 8.0 MPa, at the age of 90 days. The results of the simple axial compression resistance tests obtained in the Technological Control attest to the fulfillment of the requirements specified by the Technical Specification, see Figure 18, corroborate the adequate technological control applied in the construction and the obtaining of normal values for the RCC, consistent with the adopted cement content of 100 kg/m^3.

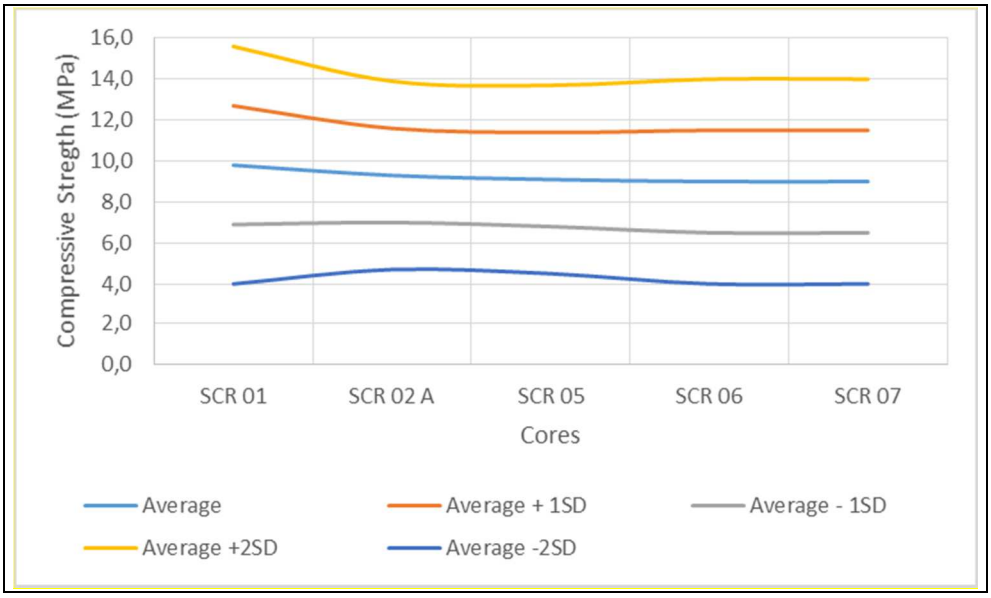


Figure 18 - Summary of Statistical Data from the Results of the RCC-6 Cores.

Splitting Tensile Strength – Brazilian Test

The Splitting Tensile Strength were consistent with the compressive strengths, proving the correlation with the technical literature and other works. The results are presented in Figure 20.

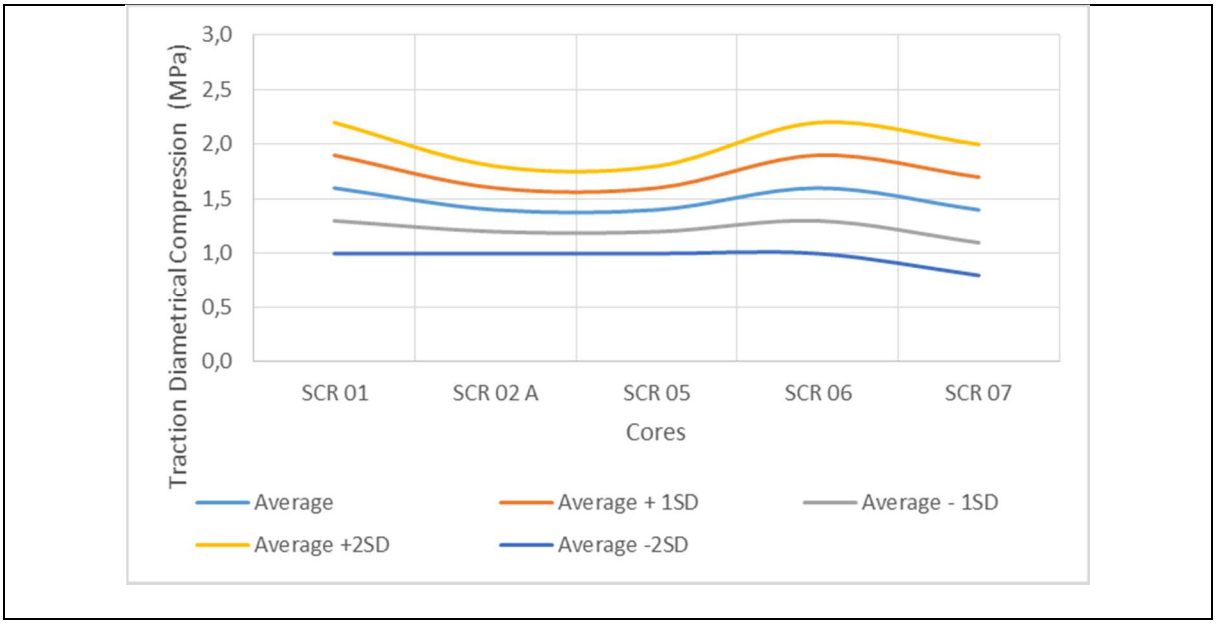


Figure 20 – Splitting Tensile Strength – Brazilian Test - Results of the Cores

Modulus of Elasticity

The Modulus of Elasticity obtained corresponds to values within the range considered consistent for RCC for this level of cement consumption, directly influenced by the aggregate

used. The use of hematite in RCC was considered unprecedented in Brazil, and no mention of this specific application was found in the international bibliography. The use of hematite as aggregate in the RCC in ECJ 1 proved to be adequate and timely for the purpose of obtaining a safe structure also in terms of elastic properties, notably in relation to the Modulus of Elasticity, as shown in Figure 21.

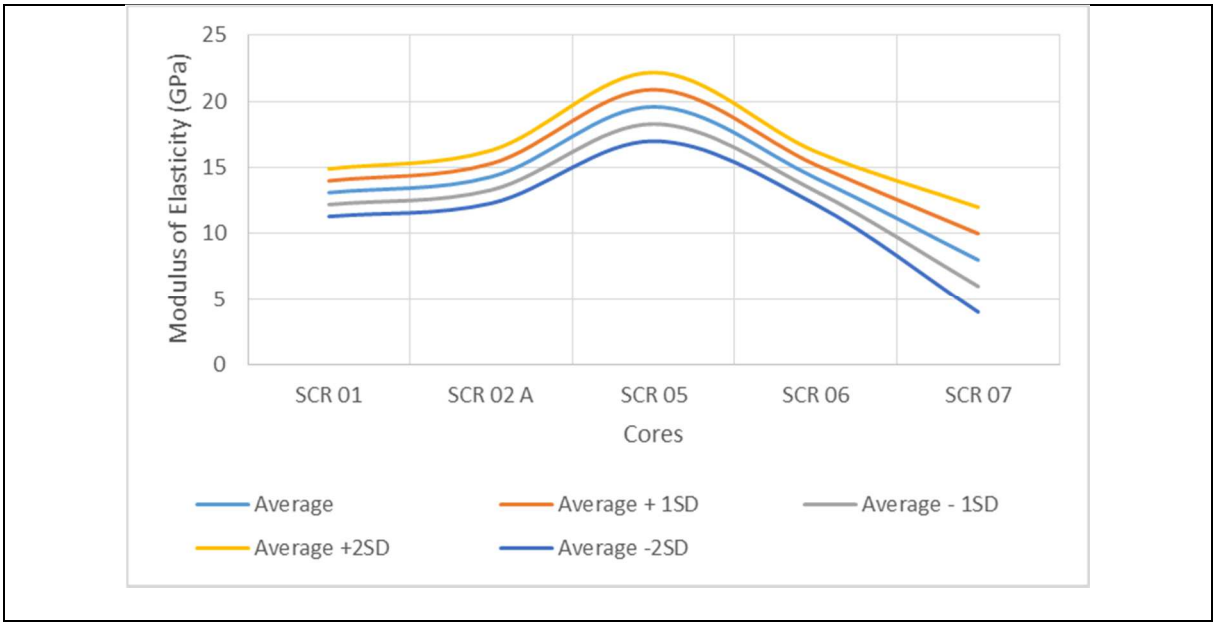


Figure 21 – Results of the Modulus of Elasticity of the Cores

ECJ 2 CORES

In order to verify the quality of the concrete applied and the execution of the structure, a core drilling campaign was carried out during the construction of the work. The drilling service was carried out by the company GEONORTE and the extraction was carried out by means of a rotary probe equipped with a double rotating barrel, with an external diameter of 8” and 1.5 m in height, with recovery of cores of 6” in diameter. The position of the drilling holes is shown in Figure 22. The drilling campaign was determined by Vale Technical Specification ET-1800HH-B-00001, as well as the tests to be carried out on the extracted cores. The extraction locations of the holes were determined in mutual agreement with VALE according to executive availability during the construction of the structure.

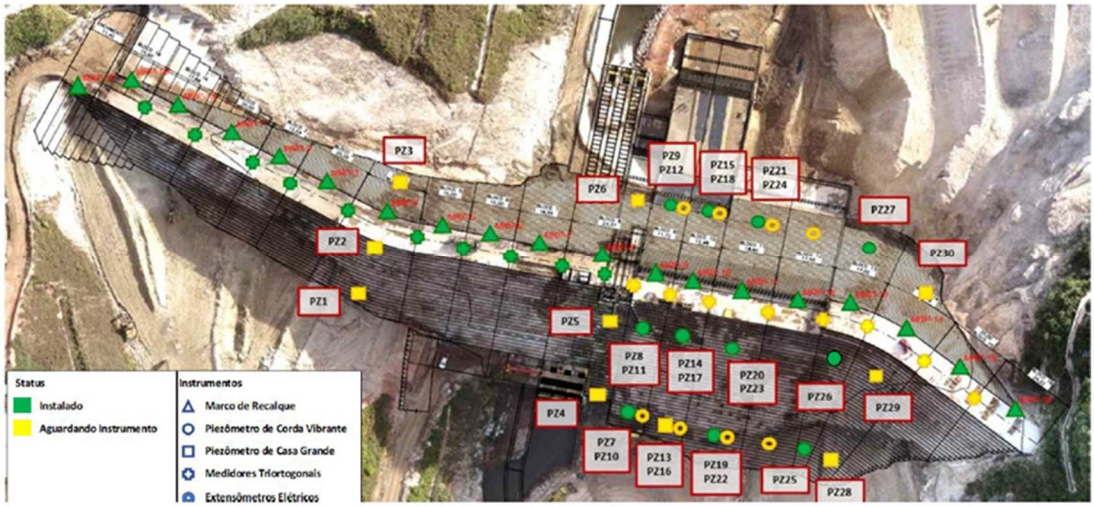


Figure 22 – Location of the holes

Nine drilling holes were drilled in Blocks 3, 5, 10, 11 and 15 of the Containment Structure. The tests carried out with the cores made it possible to analyze and compare the results obtained during the execution of the work by Technological Control. The drilling services were performed by the company GEONORTE using a March 700 Probe, with an 8” Barrel Holes applying the following products: Gelplus/Bentonite. The Figure 23 shows some images of the GEOCONTROLE probes

Figure 23 – Geonorte Drilling Equipment at ECJ 2.

Table 7 presents a summary of the results obtained from the cores at ECJ 2 and Table 8 presents all the results obtained.

Table 7 – Statistics of the Core Tests at ECJ 2

Brazilian Standard	Test	Average	Standard Deviation	Samples n
NBR 9778	Density (kg/m³)	2.959	0.024	13
	Water absorption (%)	6.7	0.8	13
NBR 7220	Splitting Tensile Strength – Brazilian Test (MPa)	1.7	0.3	17
NBR 7580	Compression Strength (MPa)	12.9	3.2	31
NBR 8522	Modulus Elasticity (GPa)	16.1	3.6	38
NBR 10786	Coeficient of Permeability (m/s)	7,57E-11	1,64E-11	12

Density control was performed by laboratory measurements using the Vêbê consistometer cylinder and by DMA. Density in the field was determined using a nuclear densimeter. Density records using the nuclear densimeter were performed systematically according to the frequency determined by Quality Assurance and reported in the Monthly Reports issued by laboratory construction consortium CMM. The analysis of the results showed a lot of uniformity in the density measured by the nuclear densimeter in the two phases of the construction of the Retaining Wall.

Table 8 is a summary of the average values of the density tests obtained by different test procedures (1) Nuclear Densimeter (2) DMA according to the NBR-ABNT 16312.3 standard - (3) Specific mass tests of cores and carried out by different entities: (1) Patrol - Densimeter (2) Laboratory CMM construction consortium (3) Solocap. These results prove the uniformity and convergence obtained both in the construction phase, in the field (densimeter) and in the laboratory (DMA), and in core extraction.

Table 8 – Correlation of RCC Density and Workability tests in ECJ 2.

BLOC	HOLE	RCC MIX	DENSITY (kg/m³)			CANNON TIME (s)	WATER ABSORPTION (%)
			NUCLEAR DENSIMETER	DMA	CORES		
03	AMCD 301 J	RCC.3	3019	3052	2980	10	7.1
		RCC.4	3044	3029	-	10	-
	AMCD 301 H	RCC.2	2906	2946	2933	12	6.0
		RCC.3	3019	3052	3004	10	5.6
		RCC.4	3044	3029	2983	10	6.4
05	AMCD 502	RCC.4	2987	3024	2946	11	6.2
	AMCD 502 B	RCC.4	2987	3024	2956	11	6.6
10	AMCD 1001	RCC.5.1	3028	3029	-	11	-
		RCC.6	2993	3020	-	10	-
	AMCD 1001 A	RCC.5.1	3028	2945	2924	11	6.5
11	AMCD 1101	RCC.5.1	3045	3026	-	11	-
	AMCD 1101 A	RCC.5.1	3045	3026	2955	11	7.5
15	AMCD 1501	RCC.4	2985	3037	-	11	-

Compressive Strength – ECJ 2

Since the characteristic strength specified for RCC is 8.0 MPa in the mixes of RCC.2, RCC.3 and RCC.4, 12 MPa in the mix RCC.5.1 and 6 MPa for RCC.6 at ages of 90 days, the results of the simple axial compression strength tests obtained in the technological control attest to

the fulfillment of the requested strength requirements, consistent with the cement consumption adopted in the mixes. Table 9 present the summary of the properties obtained in the cores, confirming the fulfillment of the values requested in the project for the control ages.

Table 9 – Resume of results Hardend RCC - ECJ 2.

Statistical Dates	NBR 7580 Compressive Strength (MPa)	NBR 7220 Splitting Tensile Strength – Brazilian Test (MPa)	NBR 8522 Modulus of Elasticity (GPa)
Samples n	31	17.0	38
Average	12.9	1.7	16.1
Standard Deviation	3.2	0.3	3.6

The analysis of these results shows that there is a considerable regularity, proven by the small amplitude of the values obtained in all the cores representing the structure's RCC.

The Modulus of Elasticity obtained corresponds to values within the range considered coherent for RCC for this level of cement consumption, directly influenced by the aggregate used.

Permeability

Table 10 presents all the results of the permeability tests on the ECJ 2 cores.

Table 10 – Summary of Permeability Tests – ECJ 2

Site	Number Test	Permeability Coef. E^ -9 (cm/s)
AMCD 301 H	348-901	9.38
AMCD 301 H	351 - 348	9.48
AMCD 502	684-2 / 683 / 686	8.49
AMCD 502 B	686	8.49
AMCD 502 B	687-3 / 686	5.53
AMCD 502	686	5.53
AMCD 502	683 / 687-1 / 686	7.75
AMCD 502 B	686	5.78
AMCD 502	684-4 / 687-1 / 697-2	5.78
AMCD 1001 A	694	6.54
AMCD 1501	701 / 699	8.91

The values obtained through the core tests show that the RCC is practically impermeable.

Cohesion and Friction Angle

According to José Marques Filho's comment: “Even though it was performed at the interface where there was no treatment with bonding mortar, the direct shear test presented a relevant result of apparent cohesion equal to 0.65 MPa and a friction angle of 70°, which, as had already been commented and was expected, are values much higher than those of the foundation.”

Complementary Investigations with heavy RCC composed of iron ore tailings

In addition to these advantages, studies on the attenuation of gamma radiation presented by CARMO 2023 indicated that this RCC, because it is heavier, can be considered suitable for containment and shielding work in massive structures, emergency shielding structures in radiological accidents, or for treating the foundations of nuclear power plants. In this study, the methodology used to conduct shielding tests on concrete samples in the CDTN laboratories in Belo Horizonte proved to be adequate for the objectives that were defined and could be adopted in the future as a standard for future research and even be considered for a standardized test. The average absorbed dose measured on the radiochromic films positioned behind for RCC and conventional concrete samples. Dose reduction of 54% was observed for RCC. Figure 24 shows the dose distribution observed on the radiochromic films.

Concrete	Average specific mass (kg/m3)	Average absorbed dosis (Gy)	Absorbed dose reduction (%)
RCC with GIC	2905	15.696	54

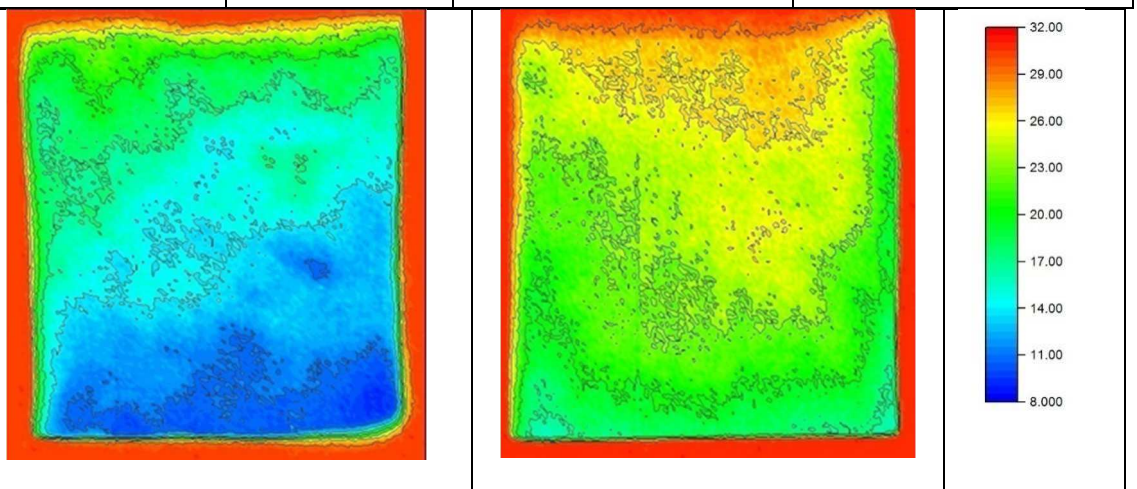


Figure 24 – Comparison of the shielding of gamma radiation of samples of RCC (left) and conventional concrete (right). The blue color means the lower dose measured at the radiochromic film and the red color the higher dose

Figure 25 – Detail of the exposure of concrete blocks to the source of Cobalt 60 in the CDTN Laboratory – Belo Horizonte – Brazil.

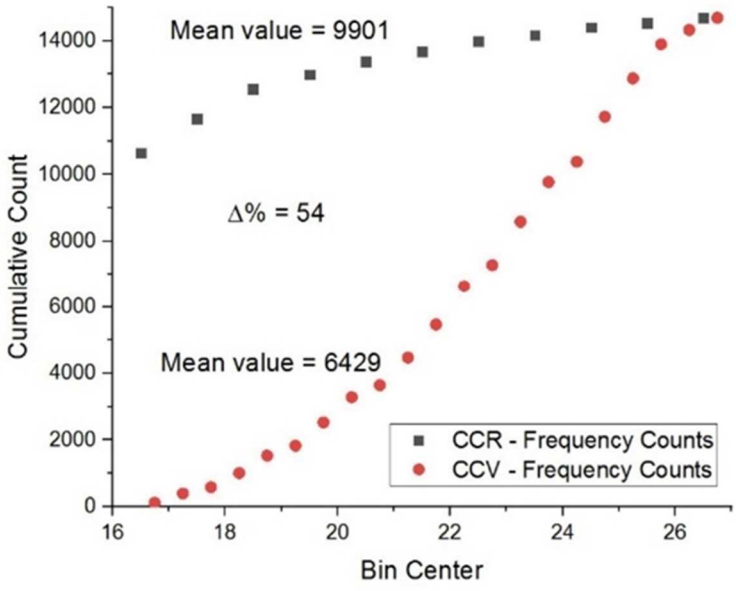


Figure 26 - One of OriginLab's statistical functions was used and this graph was obtained. Averaging the frequencies of occurrence of the data, the result of the difference between the absorbed doses of RCC in relation to conventional concrete was 54%.

4. CONCLUSIONS

Properties of RCC

In summary, the cores drylling extracted from these containment systems presented test results at 90 days on average as shown in Table 11.

Table 11 – Summary of RCC results in ECJ 1 and ECJ 2

Density RCC	3000 kg/m^3
Compressive Strength FCK	10 MPa
Water Absorption	5%
Adiabatic Temperature Rise	11 ^{0}C
Creep	175 x 10^-6/MPa
Modulus of Elasticity	13 GPa
Splitting Tensile Strength – Brazilian Test	1.6 MPa
Coeficient of Permeability	8 x 10^-11 m/s
Shear in the RCC joints	Cohesion 0,8 MPa e Friction Angle 44^0

These results demonstrated that the use of this RCC was suitable for the construction of the containment walls of ECJ 1 and ECJ 2. The constructions were fast and efficient, with an average of 70,000 m^3/month of concrete pouring and compaction, and no problems were detected due to the use of hematite tailings aggregate. The use of hematite tailings proved suitable for application in other RCC works with similar purposes in massive concrete structures, especially in emergency works.

All results of the materials used in the construction for the manufacture of RCC (cement, aggregates, chemical additives and water) were statistically controlled and met the limits established by the current standards for their application in ECJ1 and ECJ2 constructions. The results of these constituent materials were compatible with those obtained in other RCC constructions. According to ABNT NBR 10004:2004 (ABNT, 2004), GIC is classified as non-hazardous, Class II, inert waste, presenting concentration levels below the limit values specified for critical elements, such as total chromium (<0.05 mg/l), zinc (<5 mg/l) and lead (<0.01 mg/l).

The benefits of building Downstream Containment Structures like those VALE has been building in the Iron Quadrangle region to prevent new dam failure tragedies upstream are immeasurable. After the rains of January 2022 in the Minas Gerais region, the containment dams built by VALE fulfilled their role of protecting the cities downstream. There was a large volume of rain, with flows close to the maximum design levels, with a large amount of water in the regions of Minas Gerais-Brazil, but there were no records of accidents involving the rupture of dams owned by VALE. The good interaction during construction between VALE, together with the designers WALM and DF+ and CMM, resulted in the construction of the ECJ 1 and ECJ 2 projects with a construction strategy that was appropriate and consistent with the needs of this type of project. Cementitious material

The decision to use CP III type cement with 50% blast furnace slag in its composition in ECJ 1, in addition to inhibiting the AAR – Alkali Aggregate Reaction, and having a low heat of hydration, contributed significantly to temperature control by eliminating cracks of thermal origin.

Aggregates

The use of Concentration Plant Granules (GIC) from the Água Limpa Mine (AL) was the major difference from the RCC used in ECJ 1. However, this decision was only made after laboratory tests were carried out with the possibility of using all the fines passing through the 12 mm sieve. The solution found was to use an air-entraining additive in the RCC. The use of the air-entraining active ingredient solved this problem in this case, enabling the reduction and control of the unit water in the RCC mix design. The air-entraining additive content of 1% in relation to the weight of cement was able to not compromise the density of the RCC without compromising the specified resistance.

Technological Control of RCC

The use of hematite waste in RCC was considered unprecedented, with no evidence of applications of this magnitude in dam works and similar, and no mention of this specific application being observed in the international bibliography. This use of hematite as an aggregate in RCC in ECJ 1 proved to be adequate and timely for the purpose of obtaining a safe structure in terms of stability, mechanical, elastic and permeability properties. Well-designed and planned technological control in a project is of fundamental importance for the success of the undertaking. In this sense, research into new technologies and executive methodologies should be encouraged and motivated in projects and in academia, to improve this interesting technology such as RCC. The coherence of the results obtained in the tests during the construction of the Downstream Containment Structure of ECJ 1 and ECJ 2, with the cores extracted after the completion of the works, compared with the technological control tests as well as the initial tests obtained in the preliminary mix design is evident.

5. ACKNOWLEDGMENTS

VALE S.A. for making every effort to construct these emergency works as a matter of urgency, mobilizing the necessary resources to build the ECJ 1 and ECJ 2 Containment Structures.

To all the companies that participated in these works and to the professionals who, with a spirit of collaboration and determination, focused on achieving a single objective that brought greater safety and guaranteed quality to the works executed under strict construction control.

REFERENCES

[1] Consultant Report Walton Pacelli de Andrade - ECS-005-2020-R0 – Technological Control of Roller Compacted Concrete Applied in Gongo Soco.

[2] Report RL-1850DD-X-17625 Rev. A. Detailed Design Utilities / Sul Superior Dam RCC Dam Structure Stability Analysis – Foundation of ECJ Second Subsoil Investigation – Technical Report.

[3] Report Issued by GEOCONTOLE – Rocks – Determination of Triaxial Compressive Strength – ASTM D 2664 – 95 (Work No.: 10817/Drilling No.: TRA-01/Lot: 1/Sample No.: 12838.

[4] PUC – RJ Report - RCC – Studies in Laboratory - Preliminary Results for Gongo Soco – Dam Restraint - RCC – Studies in Results Laboratory Project – Dec 01-20. Determination of shear strength of concrete – rock interfaces Reference: DEC 01-20 Client: Minas Mais Consortium Final Report - Prof. Eurípedes, do Amaral Vargas Jr, Bruno Pinheiro Pires, Victor Nogueira Lima.

[5] Consolidated Report of Technological Control - Technological Control of Concrete and Geotechnics Gongo Soco (Contract No.: 5500060296/ REL-GS-CT- 0003/2020/ Date: 04/06/2020)

[6] Technical Report - Decommissioning of the Upper South Dam – S4195-F4 - Detailed utility project – Upper South Dam Downstream Containment Structure – RCC - Assessment of the use of cohesion in the concrete contact with the foundation.

[7] Report by José Marques Filho - 0GH_20200519RH - ECJ-RCC DAM – Comments on the RIZZO Report entitled “Review of Laboratory Data and Stability Analysis RCC Backup Dam Gongo Soco Mine”

[8] Report by José Marques Filho – Comments on the Report “Review of Laboratory Data and Stability Analysis RCC Backup Dam Gongo Soco Mine” – 05/25/2020.

[9] Report Issued by CMM - REL-GS-CT-003/2020, issued on 04/06/2020.

[10] Report REL-MF-CT-0013-2021 - Technological Control covering the period from July 2019 to August 2021, in the execution of the Roller Compacted Concrete (RCC) Containment structure of the Fábrica Mine, for the decommissioning of the Forquilha I, II, III, IV and Grupo Dams.

[11] Report Issued by Walton Pacelli de Andrade - Gongo Soco Work - Vale - CMM GS-CT-003 2020 R0 April 2020 Designer DF+ and Vale Retaining Wall Downstream Forquilhas Comments on Core Drilling Itabirito-MG ECS-022-2021-R0 08/17/2021 09/28/2021.

[12] Doctoral Thesis CARMO, J. B. M., Evaluation of Roller Compacted Concrete – RCC – Composed with Hematite Waste for Massive Structures – National Nuclear Energy Commission – Nuclear Technology Development Center – Postgraduate Program in Radiation Science and Technology, Minerals and Materials – Belo Horizonte – BRAZIL – 2023.

[13] PEREIRA, F.; CARMO, J.B.M.; ANDRADE, W. P. - Use of Roller Compacted Concrete - RCC for Containment of Tailings Dams - COBRAMSEG – Camburiu – BRAZIL - 2024.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

MINING DAM'S CLASSIFICATION & THE BRAZILIAN PRACTICE

Lucas R. Bittencourt[1]
Worley

Kresimir Zic2
Worley

ABSTRACT

This article examines particular aspects of Brazilian mine dam safety practices and compares them with international standards.

The main aspects revolve into the Dam Consequence Classification (DCC). Influenced by relevant dam guidelines and standards, key components include the Dam Break Study (DBS) and associated flood maps, enabling the definition of impacts from dam failure (e.g., incremental Population at Risk – PAR).

The paper compares Brazilian practices with international standards, highlighting stringent, state-driven regulations influenced by the devastating failures of Fundão and Brumadinho tailings storage facilities (TSF). Brazil's *Dano Potencial Associado* (DPA - Associated Potential Damage) system, with its three categories (High, Medium, Low), contrasts with more nuanced international classifications, often resulting in dams of different sizes and impact levels being classified under the highest category, *High*. Moreover, case studies highlight the importance of distinguishing between Total and Incremental impacts—where Total impact accounts for both dam break and natural flood effects, while Incremental impact isolates only the dam break contribution. Failing to make this distinction often results in the over-classification of dams.

Additionally, the article evaluates the industry focus on Consequence Classification as way of communicating dams, presenting Brazilian practices of Stability Condition Declaration (DCE) and Risk Category (CRI) classifications, which enhance transparency and proactive safety management. However, it is important to emphasize that their effectiveness depends on proper integration with consequence analysis.

This discussion aims to bridge gaps between Brazilian practices and international standards, advocating for a more integrated, risk-informed approach to dam safety management.

Keywords: Dam Consequence Classification, Risk Analysis, Brazil, Mining Dams, Tailings Storage Facilities

[1] lucas.bittencourt@worley.com , Australia
[2] kresho.zic@worley.com, Australia

1. INTRODUCTION

Mining dams, integral components in mineral processing, play a pivotal role in waste management and water storage. Internationally, dam classifications have historically emphasized potential downstream impacts, focusing not on the dam's physical attributes but on the broader economic, environmental, and human consequences of potential failures. Entities like the Australian National Committee on Large Dams (ANCOLD) [1], the Canadian Dam [2], the International Commission on Large Dams (ICOLD) [3], and the Global Industry Standard on Tailings Management (GISTM) [4] spearhead this approach. While Brazil has structured its dam safety policies largely at the state level, influenced significantly by past catastrophic failures, it sometimes diverges from these international benchmarks. This write-up delves into the nuances of Dam Consequence Classifications (DCC) in mining dams, contrasting Brazil's practices with global standards.

Additionally, the dam industry’s exclusive focus on the Consequences Classification and advocates for a more comprehensive integration of risk conditions or a shift towards a risk analysis approach in the way dam are categorized and communicated for a holistic safety assurance. In Brazil, Stability Condition Declaration (DCE) and Risk Category (CRI) classifications are widely used to enhance transparency and proactive safety management. While these classification systems provide a structured approach, their effectiveness is inherently tied to how well they integrate with consequence analysis.

2. TOPIC BACKGROUND

2.1 Dam Consequence Classification: International Best Practices

Dams are typically classified by their potential incremental impacts in the event of failure, commonly referred to as the Consequence Category. Notably, the DCC does not pertain to the dam's physical attributes but centres on the possible repercussions a dam failure might inflict upon downstream regions. These consequences can include economic losses, environmental damage, and risks to human life, commonly measured as Population at Risk (PAR) and Potential Loss of Life (PLL).

Tailings dams, pivotal to mining operations, stand out due to their significant failure breach ramifications. To comprehend and regulate the associated risks and to establish a foundation for design, classification systems come into play.

The CDA’s classification methodology [2], categorizes mining dams into five distinct consequence classes ranging from Low to Extreme (refer to Table 1). GISTM (refer to Table 2) also provides similar classification model; ANCOLD on the other hand, adopts a slightly

nuanced approach, proposing seven consequence categories, ranging from "Very Low" to "Extreme" (refer to Table 3).

Typically, the lowest consequence category is associated with small dams and/or those in remote areas where consequences of a failure would be minor. Whereas the Extreme dams are associated with large dams within populated areas, i.e., failure is expected to have severe consequences, including potential loss of lives (e.g., > 100).

Table 1 - CDA Dam Consequence Classification (Modified from [2])

Dam Class	Population at Risk	Incremental Losses		
		Loss of Life	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss, no long-term loss	Low economic losses, limited infrastructure
Significant	Temporary only	Unspecified	Loss of marginal habitat, restoration highly possible	Losses to recreational facilities and seasonal workplaces
High	Permanent	10 or fewer	Significant habitat loss, restoration possible	High losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Major habitat loss, restoration impractical	Very high losses affecting key infrastructure or services
Extreme	Permanent	More than 100	Critical habitat loss, restoration possible	Extreme losses to critical infrastructure or services

Table 2 - GISTM recommended Consequence Classification Matrix (modified from [4])

Dam Failure Conseq. Class	Incremental Losses				
	Potential Population at Risk	Potential Loss of Life	Environmental Impact	Health, Social and Cultural	Economic Loss
Low	None	None expected	Minimal short-term loss	No measurable effect on human health, minimal disruption	< US$1M, minimal disruption
Significant	1-10	Unspecified	No significant habitat loss	Low likelihood of health effects, some disruption	< US$10M, moderate disruption
High	10-100	Possible (1-10)	Significant habitat loss, moderate toxicity	Potential short-term health effects, 500-1,000 people affected	< US$100M, infrastructure impact, moderate relocation
Very High	100-1,000	Likely (10-100)	Major habitat loss, severe environmental impacts	Long-term health effects, significant cultural loss, 1,000 affected	< US$1B, major infrastructure impact, significant relocation
Extreme	>1,000	Many (>100)	Catastrophic habitat loss, long-term damage	Severe long-term health effects, major cultural loss, 5,000 affected	> US$1B, critical infrastructure, extensive relocation

Table 3 - ANCOLD Consequence Categories (modified from [1])

Incremental Potential Loss of Life (PLL)	Population at Risk (PAR)	Severity of Damage and Loss			
		Minor	Medium	Major	Catastrophic
< 0.1	<1	Very low	Low	Significant	High C
≥0.1 to <1	≥1 to <10	Significant (Note 2)	Significant (Note 2)	High C	High B
≥1 to <5	≥10 to <100	High C	High C	High B	High A
≥5 to <50	≥100 to <1,000	(Note 1)	High B	High A	Extreme
≥50	≥1,000	(Note 1)	(Note 1)	Extreme	Extreme

- Note 1: With PAR in excess of 100, it is unlikely that damage will be minor. Similarly with PAR in excess of 1000 it is unlikely that damage will be classified as medium.
- Note 2: Change to High C where there is potential of one or more lives being lost

2.1.1 Importance of Dam Consequence Assessment

Dam Consequence assessment and Classification is an essential part of dam design and risk management. Objectives of this classification include:

- Inform Design loading criteria selection based on classification and Risk Based Approach
- Prioritizing dam safety measures
- Guide Operation surveillance, and maintenance practices for dams (e.g., frequency requirements)
- Ensure the appropriate allocation of resources for dam safety
- Facilitate communication with stakeholders about potential risks (i.e., transparency with outcome of failure [5])
- Enable Risk Analysis assessment, guiding tolerable societal risk levels (Refer to Section 2.2)

Dam consequence assessment varies in its level of complexity and thoroughness. At its simplest, a desktop study can be employed, which involves a preliminary analysis based on available data. However, for a more comprehensive understanding, detailed assessments are recommended which will typically include a Dam Break Study (DBS).

2.1.2 Dam Break Study

A DBS is the industry state-of-the-art technique for determining the possible ramifications of a hypothetical dam failure downstream. This study typically encompasses inundation maps, flow properties, and flood onset timings.

A detailed assessment of a dam's potential impacts relies on accurate flood mapping. This procedure enables a precise comprehension of zones vulnerable to subsequent inundations. A DBS simulates breach scenarios, crafting flood maps that illustrate the extent, depth, and velocity of the resultant water flow. This vital analysis, deploying hydrological and hydraulic

modelling, ensures accurate impact evaluations on downstream communities, infrastructure, and the environment, forming the foundation for enlightened decision-making across dam design, construction, and governance.

Although the DBS will not encompass a risk assessment, it often integrates a probability element of dam failure. A Failure Mode and Effect Analysis (FMEA) is usually necessitated to discern the credible failure modes (i.e., a failure mode that can be justified based on engineering basis).

Once the failure modes have been identified the dam break is analyzed, as a minimum, for two cases:

- Sunny Day: reflects the dam failure with no rainfall within the dam's catchment (sunny conditions), coinciding with only moderate flows within the river system downstream of the dam. The dam break induced flood wave, once generated at the dam, propagates and attenuates further downstream, as an addition to concurrent local river flows.
- Rainy Day: reflects the dam failure with the Design (extreme) rainfall within the dam's catchment (local rainy conditions), concurrent with regional rainfall producing flows within the river system downstream of the dam. Whilst design rainfall is often the Probable Maximum Precipitation (PMP), it is a common practice to apply 1/100 Annual Exceedance Probability (AEP) for the concurrent regional rainfall.

Additional challenges and simulation alternatives can be associated with possible cascading failures of nearby structures, amplifying the failure flood waves of a single structure.

The dam classification will ultimately be based on the credible failure mode and failure case that would result in the worse incremental consequences of failure associated with breach of the dam.

In addition to DCC, the DBS and associated flood maps are the key element for Emergency Preparedness and Response Plan (EPRP).

2.1.3 Definition of Incremental Impact

Regional hydrological assessment aims to establish the flow conditions in the river system downstream of the considered dam. Given that the hypothetical dam break flood discharges into the river network just downstream of the dam, detailed knowledge of the magnitude of the natural flood inflow and concurrent release of the reservoir at the assumed time of dam failure is critical.

At failure, the combined natural flood and dam breach flow is the Total dam break impact, while the Incremental Impact isolates the additional effects of the dam breach beyond natural

flooding. This Incremental Impact determines the dam's consequence classification. The Total dam break inundation extent is used for EPRP (i.e., protection of downstream community).

2.1.4 Flood Hazard and Definition of PAR & PLL

Defining PAR is commonly associated with the flood hazard. Flood Hazard Maps showing the limits of the vulnerability classes are commonly used to support the assessment of potential dam breach hazards and PAR for DCC. An example of this is the method proposed by the Water Research Laboratory of the University of New South Wales [6], where the vulnerability risk is defined as a function of flow velocity (V) versus inundation depth (D). The vulnerability class, thresholds, and descriptions are shown graphically in Fig. 1.

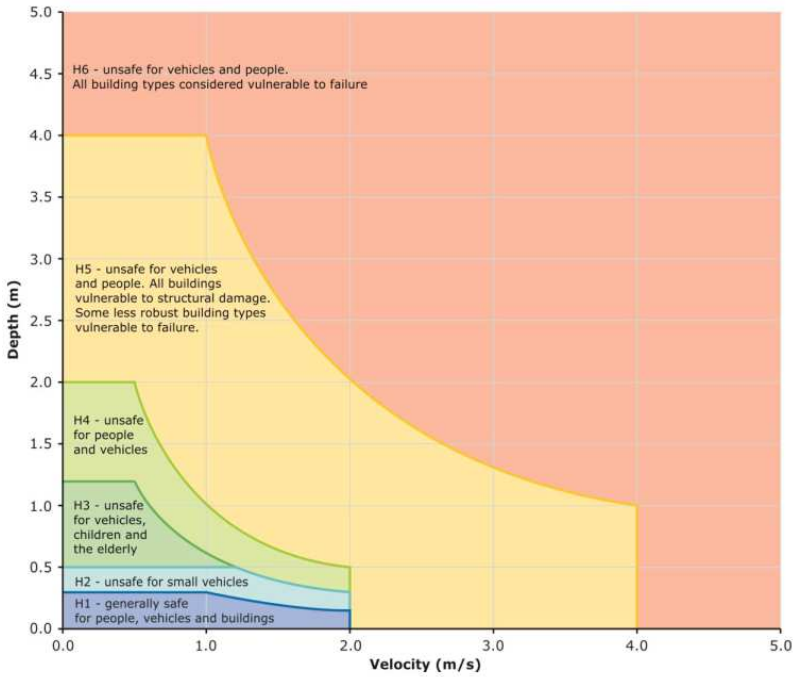


Fig. 1 - Vulnerability risk matrix [6]

ANCOLD [1] emphasizes the importance of addressing uncertainties in consequence assessment, particularly in estimating PAR in regions of concentrated populations and differentiating the Total PAR from Incremental PAR (referred to as *Dambreak* PAR). In a Rainy day flood, as per ANCOLD [1], PAR exposed to pre-*Dambreak* flood waters in areas within the D*V > 0.6 m2/s (Depth vs. Velocity), equivalent to Class H2 (refer to Fig. 1), should be discounted from *Dambreak* PAR. It is assumed this population would already have left the flood zone.

In a Sunny Day Dam break event, it is reasonable assumed that only in-bank flows would exist before the dam break, making the Sunny Day Dam break PAR effectively the Total PAR. In a Rainy Day Dam break event, there may be extensive floodplain inundation prior to the dam break. In this case, Total PAR includes those already affected by natural flooding plus those

impacted by the dam break (Incremental PAR). Only Incremental PAR is considered for consequence classification. Fig. 2 provides a simplified view of the PAR in each scenario.

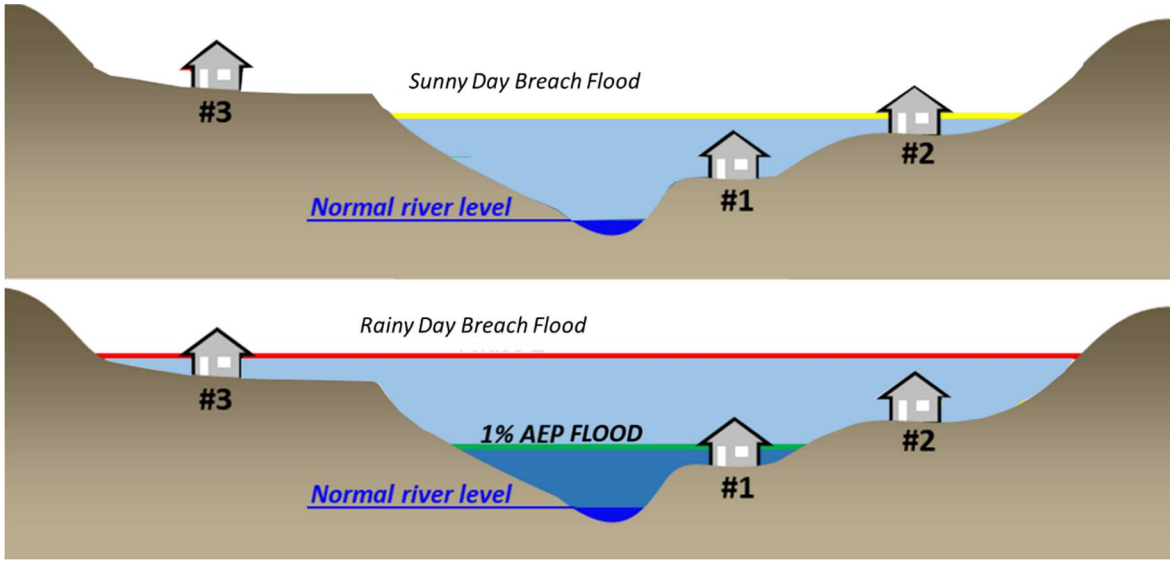


Fig. 2 - Schematic cross-section for PAR estimation.

In the schematic cross-section, the Sunny Day flood includes houses #1 and #2 as PAR. The Rainy day flood includes houses #1, #2 and #3 for Total PAR, but only houses #2 and #3 in the incremental PAR, as house #1 is already within the vulnerability risk of 1% Annual Exceedance Probability (AEP) event (green line).

Similarly, CDA [7] defines incremental losses as damages from a dam failure event minus those from the same event without failure, stressing that dam owners are responsible only for additional failure-related damages.

Estimating PLL is more complex. Graham [8] developed a loss estimation method incorporated into RCEM [9] and ANCOLD [1], using fatality rates based on flood severity, warning time, and public awareness. RCEM applies a graphical method to determine fatality rates based on D × V and warning time.

2.2 Dam Risk: Likelihood of Failure & Consequence of Failure

Dam Risk can be described as the product of the consequence and likelihood of failure. The likelihood is influenced by the dam's structural condition (including foundation), potential natural events like floods or earthquakes, and human factors. Concurrently, consequences refer to the outcomes of a dam failure.

Together, failure probability and consequences provide a holistic view of a dam's overall Risk Profile. In the dam industry, risk management are fundamentally influenced by these two key factors. This approach is a widely accepted standard in the sector and is commonly referred to as *Pragmatic Risk-Based Approach*.

Risk management often involves the use of Failure Annual Probability x Number of Fatalities plots, also known as F-N plots, as illustrated in Fig. 3. The methods align with hazardous industry standards and the principle of reducing risks to levels that are As Low As Reasonably Practicable (ALARP) [2]. The ALARP principle aims not to eliminate all risks but to minimize them to an acceptable societal level after implementing practical and cost-effective safety measures, since absolute safety cannot be guaranteed. Publications such as ICOLD [10] provides a detailed insight on the concept of ALARP.

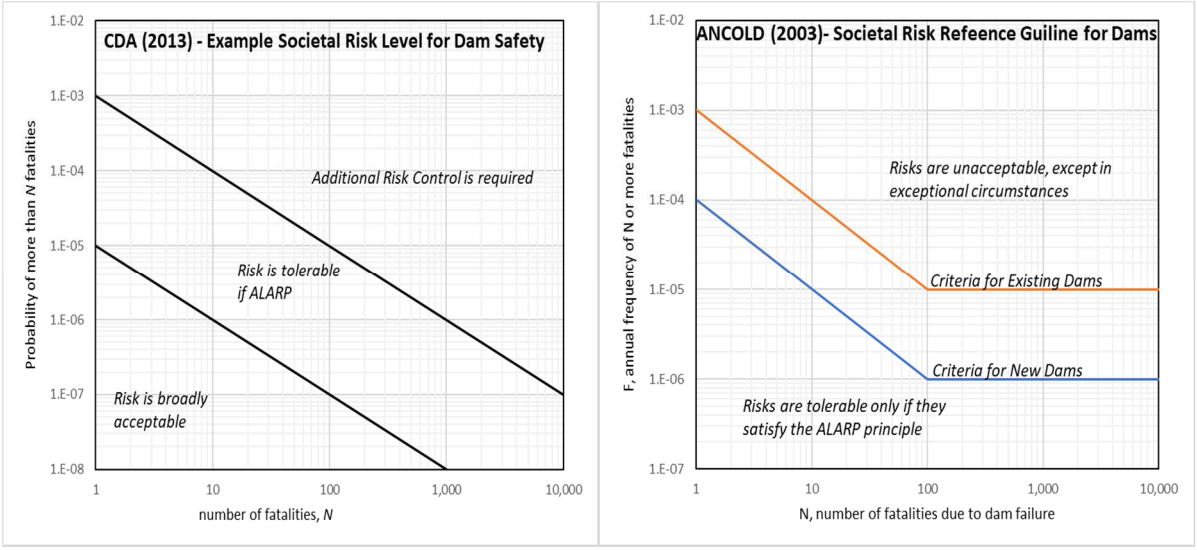


Fig. 3 - CDA and ANCOLD examples for societal risk levels acceptance guidelines for dam failures (CDA [2] left figure, ANCOLD [11] right figure)

A dam's Risk Profile is not typically simplified into categories like Consequence assessments due to several factors. Risk assessments often require site-specific knowledge and involve complex evaluations. Additionally, risks evolve over time due to factors such as aging, structural modifications, and changing climatic conditions, making rigid categorization impractical.

While a Risk classification is not a common practice internationally, dam safety protocols are typically required by most guidelines and dam safety regulations which will assess the specific risk profile of their dams, often through detailed inspections, monitoring, structural assessments, and risk analysis. However, this is usually a more dynamic, ongoing process and might not be as publicly codified as consequence classifications.

3. BRAZILIAN PRACTICE

3.1 Background

In Brazil, mining dam safety is predominantly influenced by the state’s dam safety policies, particularly in the wake of the devastating failures of the Fundão and Brumadinho (also referred

as Feijão) tailings dams. Reacting to these events, state regulatory bodies significantly tightened dam regulations, with a primary focus on ensuring public safety and emphasizing emergency procedures. New upstream dam was banned in the country (Federal Law Nº 14.066, 20 September 2020 [12]), and it was in this context that the requirements and scrutiny related dam safety became significant more robust. Currently, Brazil has one of the most restrictive Mining Dam Safety Legislation in the world.

This state-driven approach has significantly influenced dam engineering practices. While many of the local mining companies are member of ICMM and have committed to the GISTM principles, the prevailing practice remains embedded in state policies. In contrast, countries such as Australia and Canada derive much of their dam safety ethos from independent expertise organizations like ANCOLD and CDA respectively.

3.2 Dam Consequence Classification – Associated Potential Damage (DPA)

Brazil's mining dam regulation is primarily governed by ANM (*Agência Nacional de Mineração* - National Mining Agency). The ANM employs a consequence classification system known as DPA (*Dano Potencial Associado* - Associated Potential Damage), which sorts dam consequences into three categories: High, Medium, and Low [13].

The core methodology for evaluating consequences is comparable to those of CDA and ANCOLD, based on risk to human life, environmental and social impact. Refer to Table 4.

Table 4 - Details of the DPA classification – (Modified from TABLE 5 ANM [13])

Total Reservoir Volume - (a)	**Existence of population downstream - (b)**	**Environmental impact - (c)**	**Socioeconomic impact- (d)**
Very Small < = 500,000 m³ (1)	NON-EXISTENT - (0)	INSIGNIFICANT - (0)	NON - (0)
Small 500,000 to 5 million m³ (2)	UNCOMMON - (3)	LITTLE SIGNIFICANT - (2)	LOW - (1)
Average 5 million to 25 million m³ (1)	FREQUENT - (5)	SIGNIFICANT - (6)	MEDIUM - (1)
Large 25 million to 50 million m³ (4)	EXISTING - (10)	SIGNIFICANT - (8)	HIGH - (5)
Very Large > = 50million m³ (5)		VERY SIGNIFICANT AGGRAVATED - (10)	
DPA = ∑ (a up to d)			
Classification Categories	Associated Potential Damage		DPA
	High		>=13
	Medium		7<DPA<13
	Low		<=7
Classification Matrix:			

An evident distinction between these classifications is the number of categories: 3 (three) for the ANM, as opposed to 5 (five) for CDA, and 7 (seven) for ANCOLD. This discrepancy results in limited partitioning of dams, grouping large dams with extreme consequences together with smaller, lower risk dams.

To illustrate this, Fig. 4 below presents a consequence classification for 12 dams as per ANM standards (data sourced from ANM web system of January 2024 [14]), contrasted with their classification based on the CDA approach (classified by the authors). To maintain site anonymity, the dams will be referred to as Dam 1 through Dam 12 in this article.

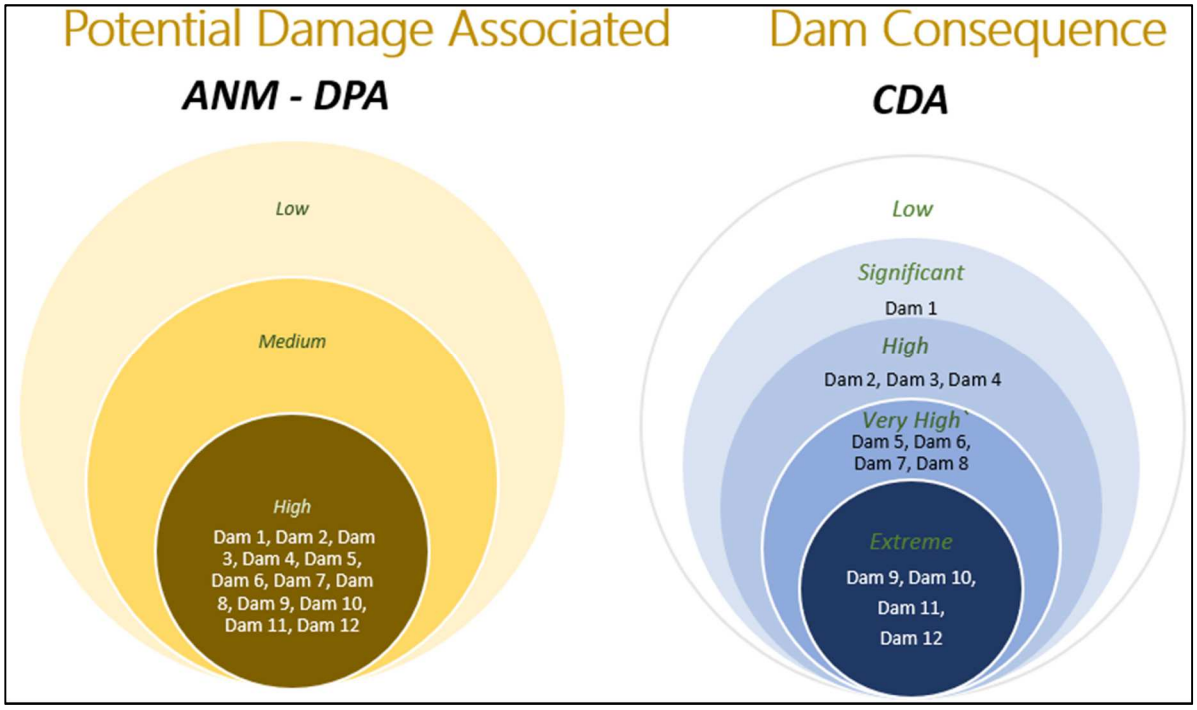


Fig. 4 - Example of dam classification based upon ANM-DPA [14] and CDA methods

Notably, all 12 dams are categorized as 'High DPA' under the ANM system. Yet, when evaluated using the CDA criteria, they are spread into four categories. The dams display notable differences in relevance, size, and consequence. For instance, Dam 12 boasts a reservoir of roughly 60 Mm^3 and has an associated PLL exceeding 100 in hypothetical dam breach scenarios. In contrast, Dam 1 reservoir is just about 50,000 m^3, with a PLL, due to a hypothetical dam breach, less than 1.

Highlighting these pronounced differences in DCC is pivotal for a suite of reasons, including resource allocation, safety prioritization, among others, as discussed in section 2.1.1.

Besides the evident class count disparity, it is noteworthy that over-classification due to the failure to distinguish between total and incremental impacts poses another challenge, which is explored further in the following chapter.

3.2.1 Uncertainty in Consequence Classification

Another primary difference between Brazilian practice based on local regulation and those of ANCOLD and CDA is the concept of Incremental Impact (refer to section 2.1.3). While the latter are explicit that dam failure consequences classification must be evaluated based on incremental failure impact, this principle is absent in the regulation which governs mining dam procedures, compiled in the so-called Resolution 95 [13]. As a result, the responsibility for distinguishing between total and incremental impacts falls on the consultant conducting the consequence assessment. If potential impacts arising from natural environmental conditions are not properly separated, they can, and often do, get incorrectly integrated into the dam's DPA classification.

It is noteworthy the terminologies of PLL or PAR, widely used in international dam guidelines, are not prevalent in the Brazilian Mining Dam Safety regulations. Instead, the more commonly used term is the Self-Rescue Zone (ZAS – *Zone de Alto Salvamento*), which has strong influence on the way the mining industry deals and with the impact of dam failure on downstream population. ANM's Resolution 95 [13] defines ZAS as:

> *LI - Self-rescue Zone (ZAS): section of the valley downstream of the dam in which it is considered that warning notices to the population are the responsibility of the entrepreneur, as there is no time sufficient for intervention by the competent authorities in emergency situations, and adopt the greatest of the following distances for its delimitation: the distance that corresponds to a time flood wave arrival time equal to 30 (thirty) minutes or 10 km (ten kilometres);*

Whilst definition of PAR and PLL, as shown in section 2.1.4, emphasizes the physical flood characteristics (depth and velocity) to determine risk, linking vulnerability to physical flood characteristics, the definition of ZAS focuses on emergency response feasibility, defining risk based on the time and distance for effective intervention.

If the breach occurs, the ZAS region must be protected, as it may be impacted by both the dam breach and the concurrent river flow. The total impacted region is catalogued (personal, environmental and social), but typically, based on the author's considerable experience with local dam safety documentation (particularly Dam Breaks and PAEBMs[1]), the incremental impact is not separated from the total impact. Misinterpretation have been observed, frequently leading to an overestimation in DCC as observed in many dam safety review reports.

[1] , The corresponding version of the Emergency Preparedness and Response Plan (EPRP) in mining Dam in Brazil is the PAEBM (*Plano de Ação de Emergencia de Barragens de Mineração* – Mining Dams Emergency Action Plan), which all dams of High DPA must issue.

The misinterpretation is more often the case for when assessing a hypothetical failure scenario in Rainy Day of small dams located near populated region. It is common in Brazil that cities are located close to the riverbanks, with population being permanently exposed to natural floods (refer to Fig. 5). Whilst this population must be accounted for dam failure emergency procedure, following common industry standards it would not play the role in the DCC, that shall be based on the incremental impact of the dam failure above the natural floods (refer to section 2.1.4).

Fig. 5 - Aerial photograph of Rio Piracicaba during January 2022 Floods – Population affected by natural river flood (credits – [15])

Some dam break consultants in Brazil limit the flood map extent to the point where the flood wave height is at least 0.15 m above the natural river height. While this approach helps avoid overestimating the incremental impact, it does not fully address the inclusion of PAR already affected by pre-existing flooding (refer to section 2.1.4). This misunderstanding can also lead consultants to limit their assessment to only the rainy-day flood map for both the EPRP and consequence assessment. While the former is reasonable as it represents the largest area, the latter is not always appropriate.

A more effective approach to avoid overestimating dam consequences due to pre-existing floods, as recommended by the New South Wales Dam Safety Committee e [16], is to assess both total and pre-dam break PAR figures in consequence evaluations.

3.2.2 Case studies

To illustrate the potential shortfalls of not appropriately consider the incremental impact to DCC, two case studies will be presented.

These cases refer to mining dams in the state of Minas Gerais - Brazil. For both cases, DBS were conducted under two scenarios: Sunny Day and Rainy Day dam failure.

In order to maintain anonymity, the dams will simply be named as “The Dam” for the purpose of this article. However, the considered case exemplifies many situations the authors have seen with the local facilities and documentation.

CASE 1

Case 1 objective is to compare Total impact and Incremental impact against Sunny and Rainy day failure scenarios. Table 5 and Table 6 respectively summarises the dam’s main characteristic and outcome of impact of the dam breach, based on available DBS.

Table 5 - The Dam - Case 1 Summary

Component	Value/Description
Purpose	Sediment Dam
Total Storage Volume (Water and solids)	990,000 m^3

Table 6 - Impact of Dam Break – The Dam – Case 1

Impact	Sunny Day Failure	Rainy Day Failure
Total PAR	272	1,040
Incremental PAR	272	316
Incremental PLL	14	3
Dam Classification **[2]**	*Very High* due to PLL of 14 (Sunny Day Scenario)	

As noted, the total PAR of Rainy Day scenario is significantly higher than the Sunny Day (1,040 vs 272). However, If the dam was to fail in an actual “Rainy Day” (i.e., local river condition in 1/100 AEP flood event), the affected region would already be partially flooded. Of the 1,040 Total PAR, 724 are from areas already impacted by pre-existing flooding.

When analysing only the incremental impact, it is noted that for PLL, Sunny Day Failure is the highest. This is because the incremental PAR from the Sunny Day is exposed to more severe flood hazards (incremental wave depth and velocity) and/or less awareness than that of the incremental PAR from Rainy Day Failure (refer to section 2.1.4).

In terms of DCC, the Dam is classified as *Very High* Consequence based on CDA guidelines, due to the PLL (>10) of the Sunny Day Failure. Based on DPA system, using ANM [13] methodology, the dam is to be classified as *High* DPA due to population within self-rescue zone > 1.

For emergency procedures, the PAEBM must be prepared based on the Rainy Day flood map as it must protect all the potential impacted region and population.

CASE 2

The objective of Case 2 is to demonstrate, how a lack of proper consideration for both total and incremental impacts can mislead DCC assessments. Table 7 presents The Dam - Case 2 summary. Table 8 presents a comparison from the reported impact of DBS based on CDA [2] and Resolution 95 [13] methods.

Table 7 - The Dam – Case 2 Summary

Component	Value/Description
Purpose	Sediment Dam
Total Storage Volume (Water and solids)	70,377 m^3

Table 8 - Impact of Dam Break - The Dam – Case 2

Impact	Dam Break Assessment Outcome	
	Method: ANM [13] (1)	Method: CDA [2] (2)
Population within the ZAS	1,610 (based on Rainy Day flood map)	Not Applicable
Incremental PAR	Not Reported	- Sunny Day scenario: Temporary only - Rainy Day scenario: 0
Incremental PLL	Not Reported	- Sunny Day scenario: 0 - Rainy Day scenario: 0
Dam Classification	DPA [13]: *HIGH* (Source: SIGBM, ANM website [17])	CDA Classification [2]: *Significant* (due to temporary population at risk, at Sunny Day failure)

Note:

(1) Total impact is presented within PAEBM is based on specific DBS for the Rainy day scenario, based on criteria presented in [13].

(2) Incremental impact was assessed for the Sunny and Rainy day breach following CDA [2]: guidelines based on DBS carried out by authors.

It is to be noted, this case study examines two distinct DBS, each potentially characterized by varying inputs and parameters, which can lead to different results of flood maps. This write-up does not intend to compare these studies in terms of flood map accuracy.

Upon analysing the numerical magnitude and reviewing the DBS results conducted by the authors, it becomes apparent that the reported population within the ZAS (1,610), is a result of the total impact, in which it is likely that the totality or near totality of the reported population, are affected in the first place by the natural 1/100 AEP concurrent river flood. Thus, while reported total impact (or population within ZAS) might be high, the incremental impact can be negligible. Hence, assuming this number as representative of the incremental PAR, can be a significant flaw in terms of DCC.

Outcomes

From these examples, it is evident that the *Delta*, i.e., incremental impact, tends to be generally lower in a Rainy Day scenario than in a Sunny Day scenario for dams of small

reservoirs and concurrent downstream flows. This occurs because Dam break flood wave tends to result in less of an impact (e.g., river depth increase) while the concurrent river flows are high as the Rainy Day (i.e., 1/100 AEP flow event), compared to Sunny Day when the river flow are much lower.

This is shown in the Fig. 6, comparing The Dam – Case 2 Dam break impact in the downstream river, for flood concurrent flow events of 1/2 AEP (Sunny Day), 1/100 AEP and 1/1,000 AEP. Note that the river level increase as a result of the dam breach for the Sunny Day (0.7m), is larger than various alternatives of the Rainy Day dam failure.

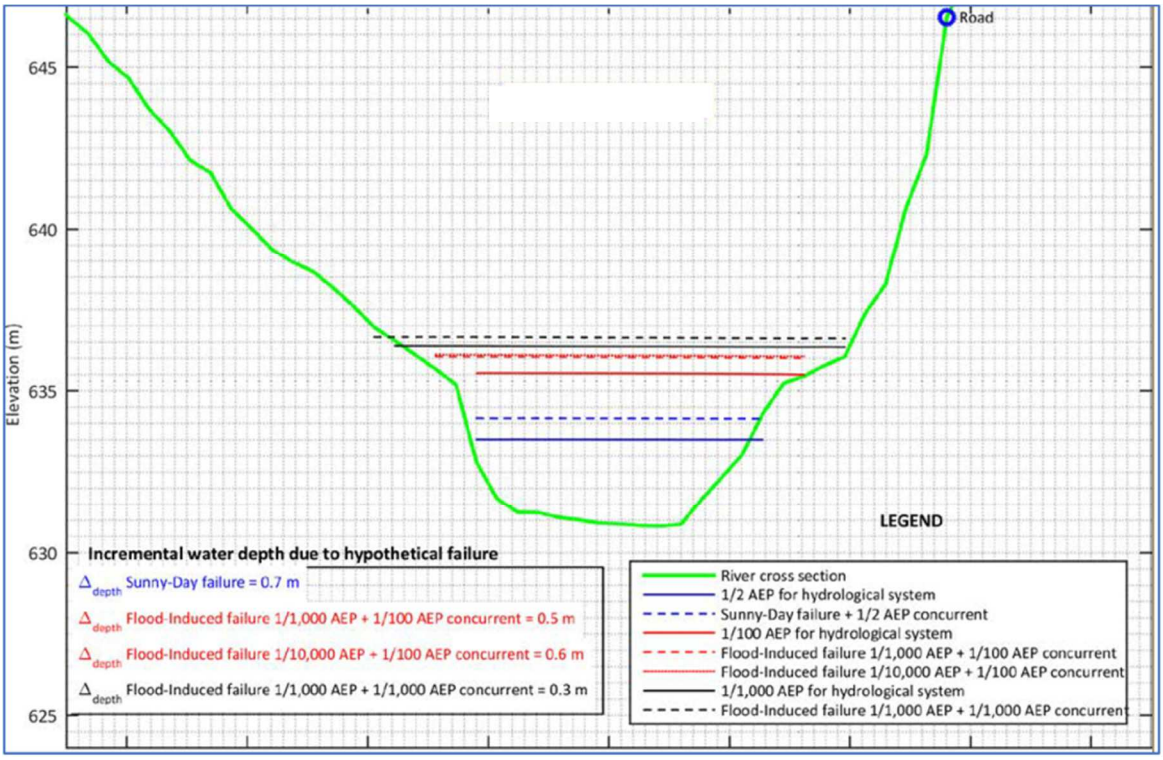


Fig. 6 - Water levels associated with different dam failure and concurrent river flow conditions) The Dam – Case 2

3.2.3 Conclusions

- There are evident inconsistencies in the approach used by the dam safety practitioners in Brazil to undertake DCC. Primarily this is due to lack of clarity in distinguishing between total and incremental dam impacts and in properly differentiating the population within ZAS from PAR and PLL.
- Sunny Day population estimate within the ZAS are typically quite close to the PAR estimates. For PLL estimation, assessment based on industry standard must be performed.

- For Rainy Day simulation, the population within the ZAS typically are similar to the Total PAR, and there is no direct relationship with Incremental PAR.
- The PAEBM documents are appropriate tool for emergency preparedness, and for this purpose the largest flood map should be utilized, not necessarily the flood map resulting from the worst incremental impact.
- However, to properly classify the Dam Consequence, only the incremental impact of the dam breach should be accounted for, to account for the risk potentially generated by the considered dam.
- Considering dam failure impacts alongside natural floods can lead to overestimate DCC. This is particularly true for small reservoir dams located upstream of populated region, as the impact is dominated by the adverse impact of the natural flood, with only a limited contribution of the dam break.
- The three-tier DPA consequence system results in facilities with genuinely extreme consequences being classified in the same category (i.e., High DPA) as dams with limited downstream impact.
- Enhancing Brazilian dam legislation or standards with more detailed guidance on DCC, incorporating incremental impact concept, could improve overall dam governance and risk management.

3.3 Brazilian Practice – Risk Classification

A unique characteristic of the Brazilian dam legislation is the requirement of classifying or defining the risk condition of the dam. The current practice in Brazil, as per Art. 5 of Regulation 95 [13] requires dam owners to provide:

- DCE (Portuguese acronym – Declaracao de Condicao de Estabilitdade) – Stability Condition Declaration
    - The DCE is a document in which the dam's technical responsible affirms the structure's safety conditions. Typically, it must be submitted periodically twice a year at every Regular Site Inspection.
    - The DCE is linked, but not restricted to, minimum design safety criteria/requirement. E.g., Dam geotechnical slope stability Factor of Safety (FoS) of minimum 1.5, and minimum inflow design flood (IDF) capacity.
    - If a dam structure is assessed as unstable for any possible failure mode, or not meeting minimum dam safety criteria as defined within Resolution 95, a negative

DCE statement shall be issued and the restrictions and constrains are to follow as per Resolution 95.

- CRI (Portuguese acronym - Categoria de Risco) - Risk Category
    - CRI is a classification that is required from dam owners or technical responsible persons, cantered on the actual condition of the dam in terms of its safety and structural soundness. It is separated in 3 (three) categories: low, medium and high.
    - To develop the classification category, a set of points is awarded based on technical characteristic of the dam, conservation status of the dam (i.e., dam's performance), and dam safety documentation compliance.
    - Dams with negative DCE will also be classified as High CRI, irrespective of the scores described above.
    - High CRI classification will result in penalties and restrictions for dam operator.

While the DCE statement is typically associated with design conditions (e.g., IDF, slope FoS), CRI will typically be associated with the dam's performance.

All Dams under the PNSB (*Politica Nacional de Seguranca de Barragens* – Dam Safety National Policy) are required to submit a DCE and have a CRI classification. According to Resolution 95 [13] this includes Dams of medium or high DPA.

Remarkably, all dams within the PNSB have publicly disclosed information available on the open-access platform SGBM (*Sistema de Gestão de Barragem de Mineração* - Mining Dam Management System). This allows public access to comprehensive data on mining dams in Brazil, including the DCE status, DPA, and CRI classifications (Fig. 7).

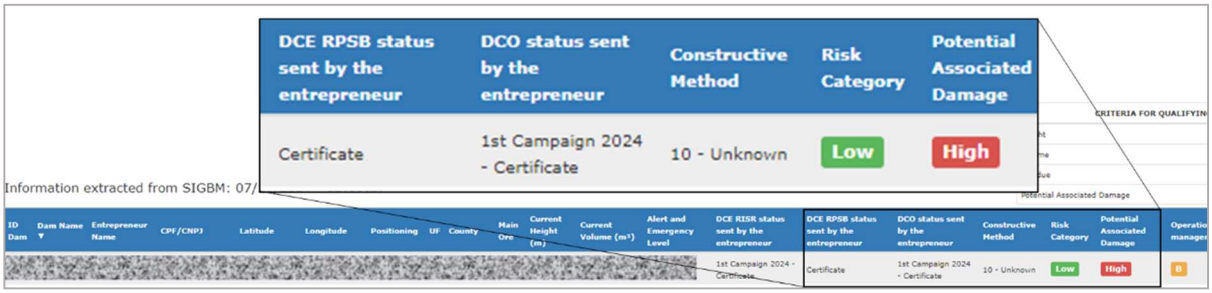


Fig. 7- Extraction from [17] - public disclosure of dam classifications and safety status information.

These classifications are a proactive manner to verify if the dam is in safe condition, and also results in transparency of Dam owners and their consultants towards the regulators and other stakeholders. This regulatory framework and experience gained from Brazilian dam regulation could serve as benchmark for international dam safety practices.

3.3.1 Risk Classification – International Practice

GISTM Principle 15 mandates companies to transparently disclose and furnish details about a Tailings Storage Facilities (TSF), promoting public accountability. Likewise, it's important to consider how dams are communicated to the public to reduce subjective perceptions of their risk. Simply labelling a dam as 'extreme consequence' can cause undue fear, even for well-constructed dams with negligible failure likelihood. Conversely, a 'low consequence' dam might be overlooked, despite a high likelihood of failure.

Consider the Airbus A380, the world's largest commercial airplane, typically carrying around 500 passengers. In the event of a catastrophic accident, the consequences would be extreme. In contrast, a common two-seat light aircraft, would result in far fewer casualties in a similar situation. Yet, if surveyed, most people would likely feel safer flying in an Airbus A380 due to its reputation for safety, despite the higher potential fatalities (i.e., consequence).

However, this perception doesn't extend to dams. If asked, people would likely prefer to live downstream of a low-consequence dam rather than an extreme-consequence dam. This is despite the fact, conceptually, extreme dams are built to much higher safety standards than a low consequence dam.

Providing additional information about Dam performance would benefits various stakeholders and potentially impacted community. The Brazilian approach of implementing DCE status and CRI classification marks a constructive development in this regard, despite some limitations discussed in section 3.3.1. Ideally, dams should be "labelled" by its *Risk Profile* encompassed by Risk Analysis assessment as prescribed e.g., by ANCOLD [18], CDA [2], and ICOLD [10]. Organizations managing dams should strive to incorporate this into regular safety audits and periodic assessments for enhanced dam safety.

3.3.2 Caveats of DCE and CRI Classification

While consequence classification serves a distinct purpose, independent of failure likelihood (refer to Section 2.1.1), the reverse is not always true. Assessing risk likelihood without considering failure consequences can be misleading. For example, a dam with a 0.1% AEP failure probability may be deemed acceptable if its failure impact is negligible. However, the same probability would be unacceptable for a dam with a PLL exceeding 100 (refer to Fig. 3).

Both DCE status and CRI classification are used for decision-making, yet they do not fully account for this concept:

- **DCE Status:** For hydraulic safety, Design rainfall / flows are typically defined based on probabilistic analysis, aligning the required AEP with its consequence category, hence, this issue does not apply. However, geotechnical stability assessments, which typically rely on

deterministic analysis with a fixed minimum FoS does not directly factor in consequences. E.g., a dam of low consequence may have received a negative DCE due to a FoS of 1.4. On the other side, a dam of extreme consequence with a FoS of 1.5 will be granted a positive DCE and will be widely appreciated as “safe”. However, assuming both dams were assessed based on the same geotechnical standards, a Risk Analysis assessment can possibly indicate the low consequence dam with FoS=1.4 to be under a broadly acceptable Societal Risk Level condition, whilst the extreme dam with FoS=1.5 may be deemed unacceptable risk condition (refer to Fig. 3). This reflects a broader geotechnical debate over deterministic and probabilistic approaches—rather than a Brazil-specific dam practice.

- **CRI Classification:** Similar challenges exist, as this classification (Low, Medium, High) is determined by a standardized scoring system, independent of consequence category. It should therefore be interpreted with caution, as higher-consequence dams require stricter risk tolerances.

4. CONCLUSION

The comparison between Brazilian mining dam regulations and international best practices reveals differences, particularly in dam classification methodologies.

For Dam Consequence Classification, International standards emphasize incremental impacts and provide detailed classification systems, facilitating risk management and resource allocation. Brazilian mining dam regulations, employ a simplified three-category classification system under the DPA framework. This often leads to lack of differentiation between dams of varying failure-impact and sizes. The different approach in defining impact to people (ZAS vs PAR/PLL) and lack of emphasis on incremental impacts, further exacerbates over-classification due to miss interpretation by practitioners. The case studies provided illuminate the importance of distinguishing between total and incremental impacts in dam failure scenarios, a distinction that is often blurred in current practices. This clarity is essential for accurate risk assessments and informed decision-making. The authors advocate incorporating specific Dam Consequence guidelines into Brazilian dam regulations to promote best practices and reduce misinterpretation.

On the positive side, the Brazilian approach of implementing DCE status and CRI classification marks a constructive development, offering a proactive measure on transparency and disclosure of dam safety condition. However, its effectiveness can be reduced by the absence of integration with consequence analysis.

Ideally, dams should be labelled by its Risk Profile, encompassing both the consequences and the likelihood of failure, through periodic risk analysis. Dam organizations and regulators should strive to incorporate this into regular safety audits and periodic assessments for enhanced dam safety.

5. REFERENCES

[1] ANCOLD, Guidelines on the Consequences Categories For Dams - October 2012, Australian National Committee on Large Dams Inorporated: www.ancold.org.au, 2012.

[2] CDA, DAM SAFETY GUIDELINES - 2007 (2013 Edition), Canadian Dam Association (CDA), 2013.

[3] ICOLD, Dam safety management: Operational phase of dams. Bulletin 154. International Commission on Large Dams., 2016.

[4] ICMM, UNEP & PRI, Global Industry Standard on Tailings Management. International Council on Mining and Metals. Available at: https://globaltailingsreview.org/global-industry-standard/, 2020.

[5] ICOLD, ICOLD Tailings Dam Safety Bulletin - November 18, 2021 - Committee L – Tailings Dams and Waste Lagoons (Presentation), Available: https://www.barrages-cfbr.eu/IMG/pdf/icold_workshop.tailings_dam_safety_bulletin.v1_2_-_may_2022.pdf, 2021.

[6] WRL, “Flood Hazard - WRL Technical Report 2014/07,” 2014.

[7] CDA, Technical Bulletin: Inundation, Consequences, and Classification for Dam Safety, Canadian Dam Association, 2007.

[8] W. J. Graham, A Procedure for Estimating Loss of Life Caused by Dam Failure, DSO-99-06, Denver, Colorado: U.S. Department of the Interior, Bureau of Reclamation, 1999.

[9] USBR, RCEM – Reclamation Consequence - Interim - Dam Failure and Flood Event Case History Compilation, U.S. Department of the Interior Bureau of Reclamation , 2015.

[10] ICOLD, Risk Assessement in Dam Safety Management - A reconnaissance of benefits, methods and current applications - Bulletin 130, International Committee on Large Dams, 2005.

[11] ANCOLD, Guidelines on risk assessment - October 2003, Hobart, Tasmania, Australia. : Australian Committee on Large Dams (ANCOLD), 2003.

[12] GOV.BR, LEI Nº 14.066, DE 30 DE SETEMBRO DE 2020, https://www.planalto.gov.br/ccivil_03/_ato2019-2022/2020/lei/l14066.htm, 2020.

[13] ANM, RESOLUÇÃO ANM Nº 95, DE 07 DE FEVEREIRO DE 2022. Com alterações da Resolução ANM nº 130/2023 e Resolução ANM nº 175/2024, ANM (Agência Nacional de Mineração), 2022.

[14] ANM, https://app.anm.gov.br/SIGBM/Publico/GerenciarPublico, Agência Nacional de Mineração - GOV.BR, 2024.

[15] Jornal Estado de Minas Gerais, Chuva deixa casas submersas em Rio Piracicaba; veja imagens aéreas - Bruno Luis Barros - 09/01/2022, Estado De Minas Gerais website – https://www.em.com.br/app/noticia/gerais/2022/01/09/interna_gerais,1336482/chuva-deixa-casas-submersas-em-rio-piracicaba-veja-imagens-aereas.shtml, 2022.

[16] NSW Dams Safety Comittee, DSC3A - CONSEQUENCE CATEGORIES FOR DAMS - June 2010 (Updated November 2015), Available: https://www.damsafety.nsw.gov.au/__data/assets/pdf_file/0004/408676/DSC3A-DSC-Consequence-Categories-for-Dams.pdf, 2015.

[17] ANM, SIGBM - Sistema Integrado de Gestão de Barragens de Mineração [https://app.anm.gov.br/Sigbm/publico]. 29/11/2023, 2023.

[18] ANCOLD, Guidelines on risk assessment. Australian National Committee on Large, 2022.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DECOMMISSIONING OF TAILINGS DAMS: CASE STUDY AND CRITICAL ANALYSIS OF THE DECOMMISSIONING STAGES OF THE SAMAMBAIA DAM

Ian Henrique Pires[1]
Mineração Usiminas S.A

Augusto Romanini[2]
Mineração Usiminas S.A

ABSTRACT

The two accidents involving tailings dams in Brazil, constructed using the upstream method for mining waste disposal, led to the prohibition of operating these structures and the requirement to decommission them. Resolution ANM No. 95 of 2022 defines the decommissioning of mining dams as structures that no longer possess the characteristics or functions of a dam, while still fulfilling the following minimum steps: cessation of tailings disposal operations, hydrological and hydrogeological control, stabilization, and monitoring. However, there is no specific methodology or evaluation framework provided for assessing the effectiveness of closure. This topic is relatively underexplored in the literature. In this paper, we present a case study of the decommissioning process for the Samambaia 0 (Zero) Tailings Dam owned by Mineração Usiminas S.A., which does not involve the removal of tailings. We evaluate all the steps recommended by Brazilian legislation and verify the chronological order of decommissioning, from conceptual to detailed design, planning, construction, and risk management. The Samambaia 0 (Zero) Dam case study highlights that ensuring effective decommissioning requires going beyond formal compliance with legal requirements. Following a logical order and respecting the uniqueness of the structure are essential for ensuring the dam's safety throughout its lifespan. The descomissing of a tailings dam aims to reduce potential risks in the event of a breach.

Keywords: decommissioning, tailings dam; dam's safety, case study.

[1] ianhpires@gmail.com, Brazil .
[2] augusto.romanini@gmail.com, , Brazil

1. INTRODUCTION

Due to the significant mining dam failures in Brazil in 2014 and 2019, the safety of mining dams has become a topic of great relevance, highlighting the vulnerability in the management of these structures. Dams, essential for activities such as water supply, energy generation, mining, and agriculture, pose significant risks in case of rupture. Consequently, a legal framework was established to ensure the safe operation of mining dams with reduced socio-environmental impact.

In the context of this new regulation, tailings dams built using the upstream method have been gradually decommissioned and closed, as this type of structure has shown greater vulnerability to geotechnical failures and adverse environmental impacts.

The objective of this paper is to analyze the steps preceding the closure of the Samambaia 0 (Zero) Tailings Dam. To this end, the influence of recent disaster impacts on the operation of this structure is evaluated. Additionally, the development of the project, the construction schedule, and the adopted risk management are discussed.

The Samambaia 0 Dam, located in the municipality of Itatiaiuçu, Minas Gerais, was designed in 2010 with the purpose of storing fine and superfine wet tailings generated during the concentration of iron ore by Mineração Usiminas S.A. (MUSA). Its initial construction used compacted clay-silt soils. Figure 1 shows the dam section.

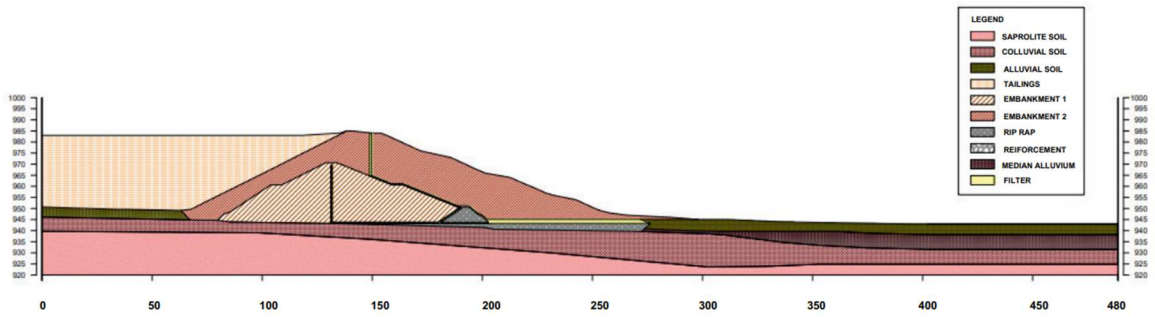


Fig. 1 – Section Map of Samambaia 0 (Zero) Dam

In 2015, to meet the demand for tailings production from mining activities, the dam was raised using the downstream method. This raising included the expansion of the internal drainage system (with a drainage blanket and vertical filter) and foundation treatment. The final configuration of the dam, after the raising, featured an 8-meter wide crest, 46 meters in height, and 536 meters in length. Table 1 presents the main technical characteristics of the structure.

Table 1 – General Characteristics of the Structure

Purpose:	Storage of tailings
Height:	46 meters
Crest Length:	536 meters
Current Reservoir Volume:	8.708.062 m³
Section Type:	Homogeneous
Current Reservoir Area:	65 hectares
Overall Slope Inclination:	2.5H:1V (approximately 21.0 to 22.0 degrees)

The structure ceased operations at the end of 2021, meaning it no longer receives mining tailings. It is currently used for the recirculation of rainwater, sediment containment, and flood control in the surface drainage area of the basin. Figure 2 presents a panoramic view of the dam embankment.

Fig. 2 - Panoramic View of Samambaia 0 (Zero) Dam before descomissing

The closure of the structure complies with Article 54 of Resolution No. 95/2022 of the National Mining Agency (ANM) [1], which establishes guidelines for tailings dams regardless of the construction method. According to this regulation, if there is a population within the flood zone, one of the following measures must be adopted: resettling the affected population, implementing actions to ensure the effective stability of the structure, or proceeding with its decommissioning.

In this context, the company opted for the decommissioning of the dam as part of the mine closure process and to mitigate the socio-environmental impacts associated with the structure.

2. METHODOLOGY

2.1. Timeline, Social Context, and Decommissioning Project

This paper presents the activities preceding the closure works of the Samambaia 0 (Zero) Dam, with the aim of exploring the technical, social, and regulatory aspects that have shaped the process. It presents the social context in which the dam is situated, the historical and regional impacts of its construction, and its operation throughout the life cycle of the structure.

As well, the paper discusses how the collapse of dams in Brazil influenced the formulation of stricter legislation, resulting in a more restrictive and demanding regulatory framework. These changes directly impacted the closure process of the Samambaia 0 (Zero) Dam, as its operational discontinuity and the end of its useful life required specific adjustments to ensure compliance with the new regulations. Thus, closure has become not only a technical necessity but also a fundamental regulatory requirement to mitigate risks and ensure the stability of the structure in accordance with current standards.

The conceptual, basic, and feasibility level designs are also presented, highlighting how they have been improved to become more viable and executable, focusing on meeting the requirements of current legislation through new geotechnical investigations. Furthermore, it will be demonstrated how these improvements have ensured greater predictability of the structure's behavior and enabled the development of an adequate plan, respecting the specificities of the dam.

2.2. Planning and Construction Schedule

Section 3.2 covers the construction stages of the project that integrate the closure process of the Samambaia 0 (Zero) Dam. Each stage be described according to the construction sequencing of the activities, with an emphasis on the main interventions and methodologies employed.

Additionally, the macro schedule of the project is presented, which organizes the activities over a two-year timeline, with completion expected by June 2025. The schedule was developed based on climatic seasonality and the applicable legal requirements for the structure.

2.3. Risk Management

Risk management applied to dam safety comprises the identification, assessment, mitigation, and monitoring of associated risks, considering the potential impacts on communities, the environment, and urban infrastructure. To this end, technical standards are employed, such as those established by the National Dam Safety Policy (PNSB), which regulate periodic inspections, preventive maintenance, and contingency plans to minimize damage in the event of incidents.

In the context of dam closure works, identifying new risks becomes essential, as the structure will be subject to new interventions. In accordance with Resolution ANM No. 95/2022 [1], the Tailings Dam Risk Management Process (PGRBM) must be updated before each structural intervention. For risk assessment, evaluation, and control, the FMEA (Failure Mode and Effects Analysis) methodology was adopted.

The FMEA methodology begins with the decomposition of the system into elements, defining their functions and identifying possible failure modes, such as the loss of a specific functionality. Although FMEA, by itself, does not require a quantitative assessment of the probability of failure, each failure mode is analyzed concerning its effects (local and global consequences) and its underlying causes, allowing classification based on severity, occurrence, and detection indices.

In the closure of the Samambaia 0 (Zero) Dam, the FMEA considered factors such as human health and safety, environmental, and economic impacts. For each identified failure, qualitative indices of severity and occurrence were assigned, allowing the definition of priority corrective actions. Thus, risk mitigation occurs systematically, ensuring that interventions align with safety and regulatory compliance criteria.

The flowchart presented in Figure 3 summarizes the risk management methodology.

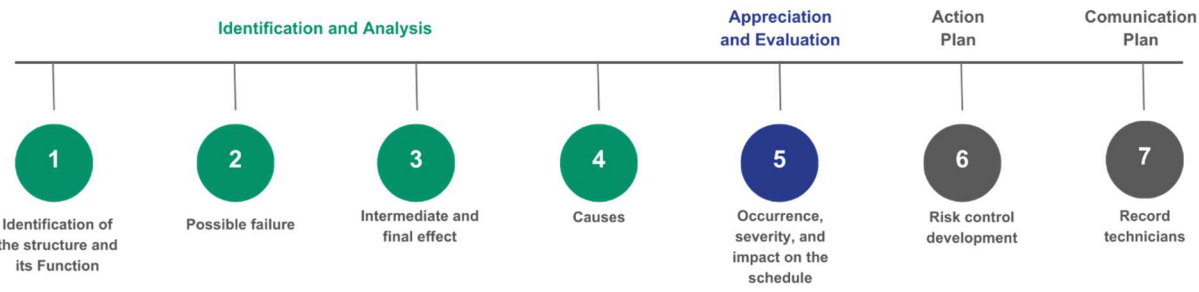


Fig. 3 - Risk Management Flowchart

In this same scope, the methodology was adapted to incorporate the impact on the schedule in the risk analysis. For this, it was necessary to evaluate the planning of activities, as well as the impacts and risks to the functionality of the structure related to possible delays and unforeseen events, considering the legal deadline to comply with Article 54 of Resolution ANM 95/2022 [1].

3. RESULTS AND DISCUSSIONS

3.1. Timeline, Social Context, and Decommissioning Project

Marais et al. (2024) [2] state that analyzing the choices made over time is crucial for understanding how disasters occur. Equally important is the context in which these decisions are made. In this regard, Figure 4 presents a timeline of the key stages in the lifecycle of the Samambaia 0 (Zero) Dam, associated with failures and influenced by Brazil's regulatory milestones on dam safety.

The significant environmental and social impacts caused by the Brumadinho dam failure (MG) influenced the decision not to raise the Samambaia 0 (Zero) Dam, considering uncertainties related to possible changes in Brazilian legislation and the public and media pressure on mining companies.

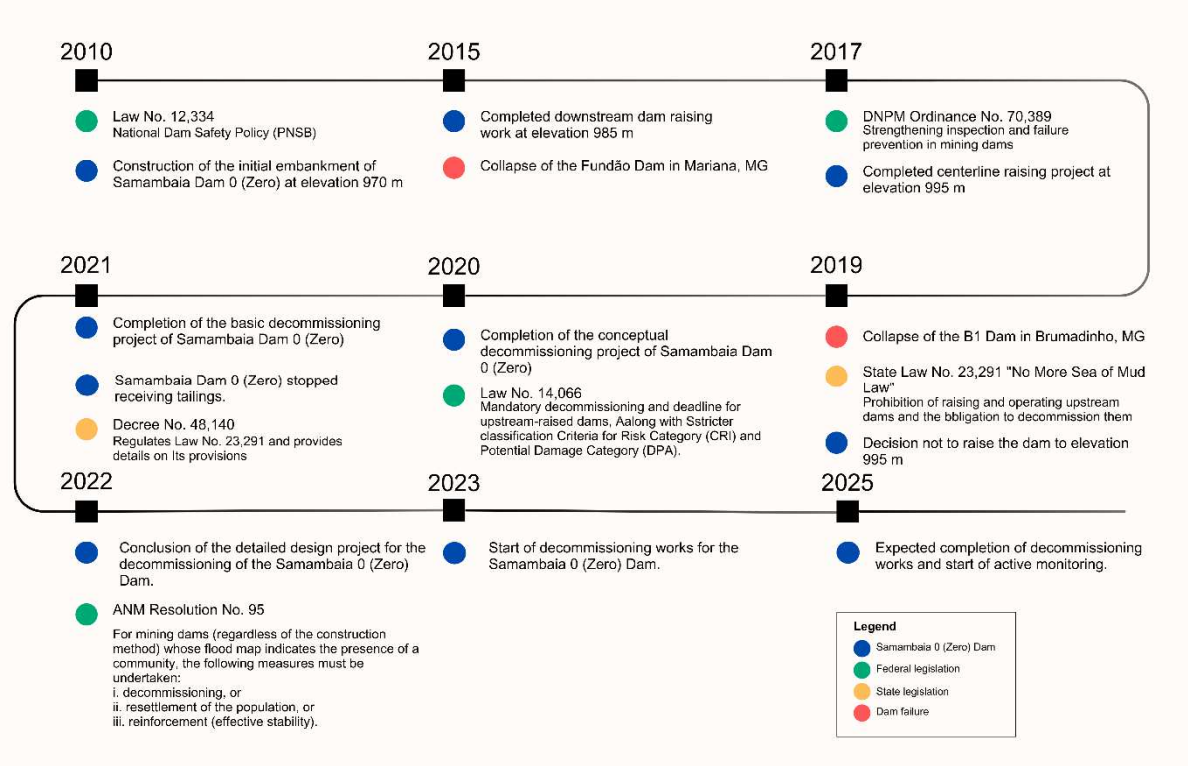


Fig. 4 - Summary Timeline of the Lifecycle of Samambaia 0 (Zero) Dam

3.1.1. Regulatory Framework: The National Dam Safety Policy

The primary regulatory framework is Law No. 12.334/2010 [3], which establishes the National Dam Safety Policy (PNSB). This legislation defines the criteria and responsibilities for the construction, operation, maintenance, and decommissioning of dams. It also creates the National Dam Safety Information System (SNISB).

3.1.2. Evolução Legislativa Pós-Desastres

The tragedies at the Fundão Dam in Mariana, Minas Gerais (2015) and the B1 Dam in Brumadinho, Minas Gerais (2019) triggered significant changes in Brazilian legislation and regulations regarding the safety of mining dams.

The DNPM Ordinance No. 70.389/2017 [4], issued by the former National Department of Mineral Production (later replaced by the National Mining Agency - ANM), complements the National Dam Safety Policy (PNSB) by establishing specific guidelines for the safety management of mining dams. The objective of the ordinance is to strengthen oversight and establish clear guidelines to prevent structural and operational failures in mining dams.

The main milestone in the state legislation of Minas Gerais regarding dams was the State Law No. 23.291/2019 [5], known as the "Never Again Mud Sea Law," which was published one month after the Brumadinho disaster. The state legislation established strict rules for the management and oversight of tailings dams. Among the main elements of the legislation are:

- Prohibition of constructing new dams using the upstream method;
- Mandatory decommissioning of dams built using the upstream method, with defined deadlines;
- Stricter environmental licensing requirements for projects involving tailings dams;
- Requirement for Emergency Action Plans (PAE) and periodic simulations.

The Brumadinho dam collapse created significant social pressure for stricter regulations and increased oversight of dams at the federal level. In this context, Law No. 14.066/2020 [6] amended the National Dam Safety Policy (PNSB) by prohibiting the operation, construction, and raising of dams using the upstream method (associated with the failed structures). The legislation regulates decommissioning as a method for closing structures of this magnitude.

Decree No. 48.140 [7], dated February 25, 2021, published by the government of Minas Gerais, regulates the State Policy for Mining Dam Safety (State Law No. 23.291/2019 [5]) and details the provisions with technical guidelines for its application. The decree reinforces norms for the management, monitoring, and inspection of mining dams. It requires the detailed registration of dams, the development of the Emergency Action Plan for Mining Dams (PAEBM), and the Tailings Dam Risk Management Plan (PGRBM). It also updates the technical criteria for risk classification and transparency measures to be taken with affected communities.

3.1.3 Steps for the decommissioning of Dams according to Resolution ANM No. 95/2022 [1]

The Resolution ANM No. 95/2022 [1], in addition to establishing normative acts for the safety of mining dams, specifies the steps for the decommissioning of tailings dams, as follows:

- Cessation of tailings disposal operations: Ending operations with the removal of associated infrastructures such as spigots and pipelines, except those intended to ensure the safety of the structure.

- Hydrological and Hydrogeological Control: Implementing effective measures to reduce or eliminate the influx of surface and groundwater into the reservoir, as well as the controlled reduction of the water table within the reservoir.
- Stabilization: Executing measures to ensure the long-term physical and chemical stability of the structures remaining on-site.
- Monitoring: Observing the structure for a minimum period of 2 (two) years after the completion of decommissioning works, aiming to ensure the effectiveness of the stabilization measures.

3.1.4. Designs

The development of the conceptual, basic, and executive projects required support to meet the initial design. Each stage of the project involved new geotechnical investigations and new adaptations to the designs to ensure the predictability of geotechnical behavior and compliance with the current regulations for a structure of this size. Table 2 elucidates the stages of geotechnical investigation carried out in association with each stage of the project.

Table 2 – Investigação geotécnica realizada por ano e projeto

Investigation Type	Year	Project Description
20 percussion boreholes 34 hand-auger drillings	2009 - 2010	Executive project of the initial embankment
9 mixed drillings	2011	Instrumentation plan
5 mixed drillings 13 percussion drillings 17 hand-auger drillings	2016	Raising design
5 CPTu drillings	2020	Basic decommissioning design
7 lines of electrical resistivity and spontaneous potential	2020	Basic decommissioning design
12 mixed drillings 11 percussion drillings 3 inspection pits for undisturbed sample collection	2020 - 2021	Executive decommissioning design
4 SPT drillings and 1 inspection pit for undisturbed sample collection	2023	Design Review

The design of the decommissioning project aims to meet the concept defined by legal instruments, with the post-decommissioning structure classified as a landfill with local stabilization through revegetation and without the accumulation or retention of any material. The decommissioning works do not include the removal of tailings.

The development of the dam ‘decommissioning project considered the risk of recent legislative changes, which presents both positive and negative aspects. On one hand, the legislation incorporates best practices from the first decommissioning and technical improvements in emissions after 2021, promoting greater safety and efficiency in the process. On the other

hand, the use of devices that are environmentally compatible and integrated, such as gabions and riprap channels, still faces technical challenges, such as the ability to meet speed and flow requirements. This situation raises questions about the technical feasibility of these devices and the possibility of using other materials, such as concrete, which, although not as easily integrated into the environment, can offer a robust solution.

Currently, the tailings deposited in the Samambaia 0 (Zero) Dam cannot be utilized. Their reuse and stacking would require environmental licensing, resulting in new environmental impacts. Thus, the complete removal of the deposited tailings becomes unfeasible. It is expected that, with the sealing of the structure, the tailings will progressively densify.

3.1.4.1. Conceptual Design

The basic design, Figure 5, envisioned the construction of downstream reinforcement in the form of a single riprap slope, the sealing of the reservoir, the installation of small sumps in the natural terrain, and the creation of a perimeter channel encircling the entire reservoir in riprap.

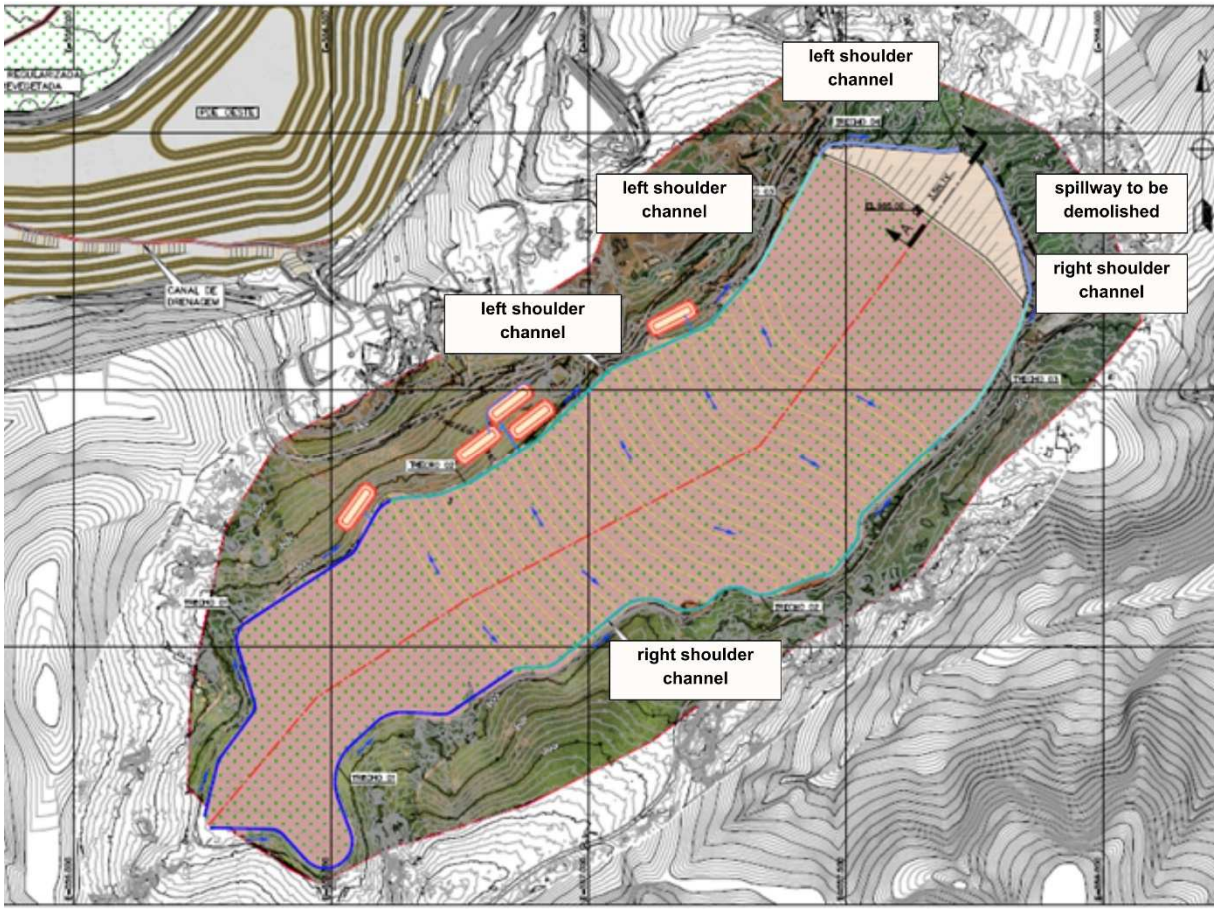


Fig. 5 - Conceptual decommissioning Plan

3.1.4.2. Basic Design

The basic decommissioning design, Figure 6, differs from the conceptual design, especially regarding hydrological and hydraulic studies. It was identified that a larger volume was needed

to absorb the surface drainage of the sub-basin. Thus, the project began to include the implementation of sumps interconnected by channels, both within the reservoir and in the natural terrain, along with all the surface drainage devices in riprap.

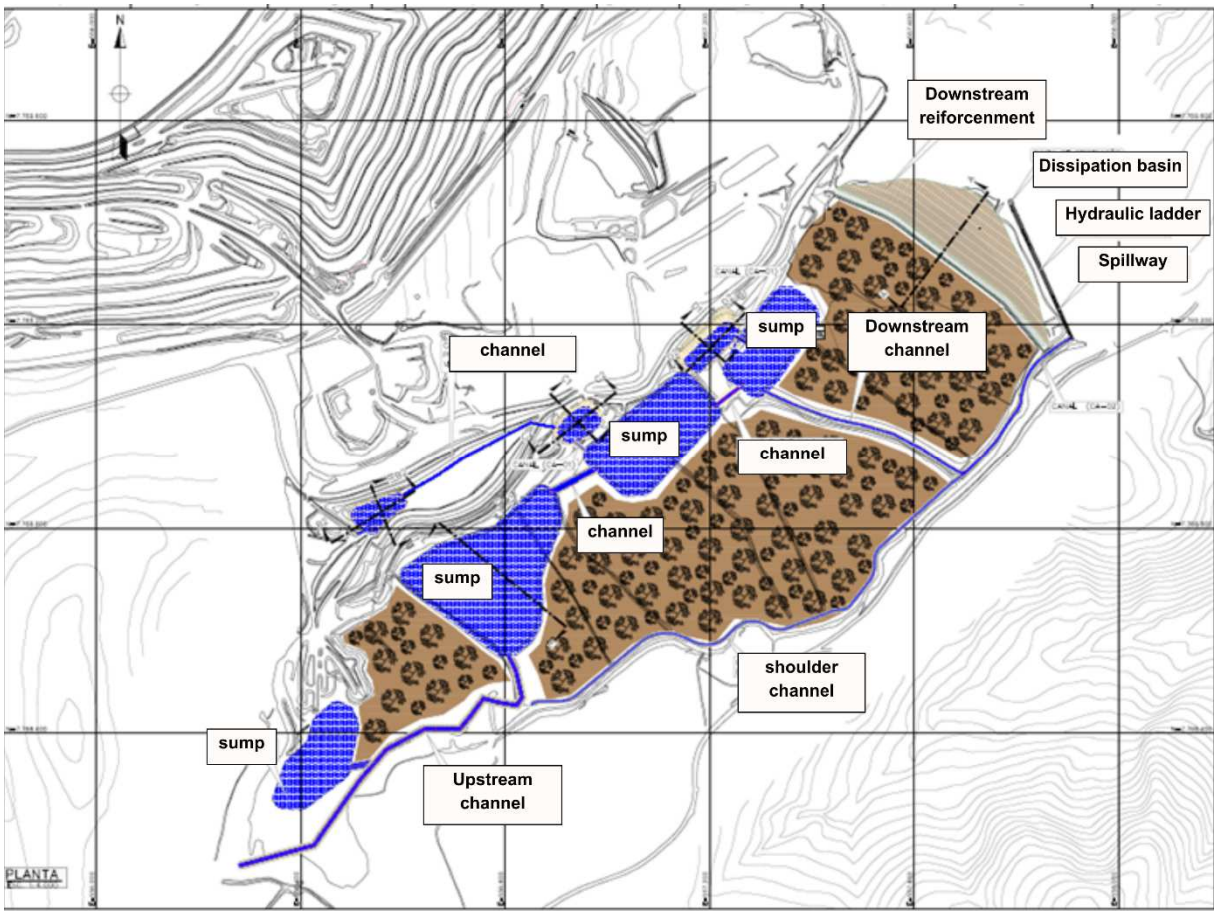


Fig. 6 - Basic Closure Plan

3.1.4.3 Feasibility level design

The executive project, supported by additional investigations, presents a downstream reinforcement in a terraced form, with a smoother geometry integrated into the valley. The soundings conducted in the foundation of the reinforcement identified the presence of a layer of soil with low strength and high compressibility, which required the inclusion of foundation treatment, previously not planned. This treatment involves the removal of this soil and its replacement with riprap. Figure 7 presents the section of Samambaia 0 (Zero) Dam, after decommissioning

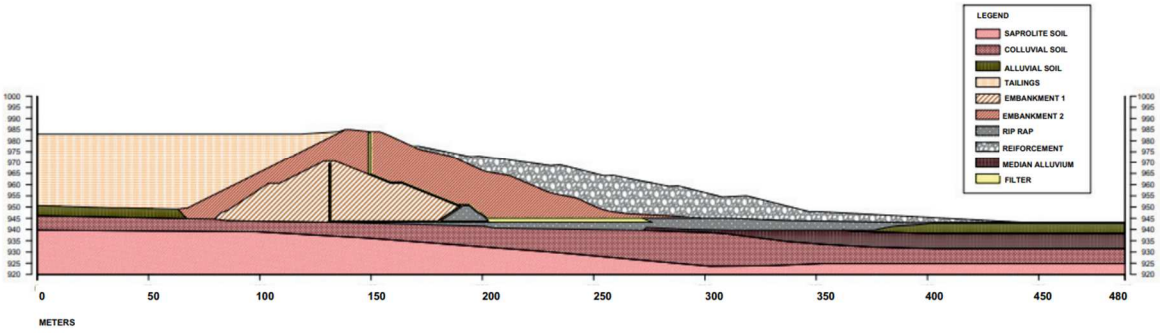


Fig. 7 - Section Map of Samambaia 0 (Zero) Dam, after decommissioning

Regarding hydrological and hydrogeological control, the executive project also includes the implementation of sumps along the reservoir's edges, intended to capture and subsequently channel surface drainage. However, these structures, as well as the connecting channels, are lined with HDPE geomembrane to prevent water infiltration; and the other channels are made of concrete. For these structures, there is a maintenance and desilting plan after decommissioning. Figure 8 presents the implemented executive decommissioning project.

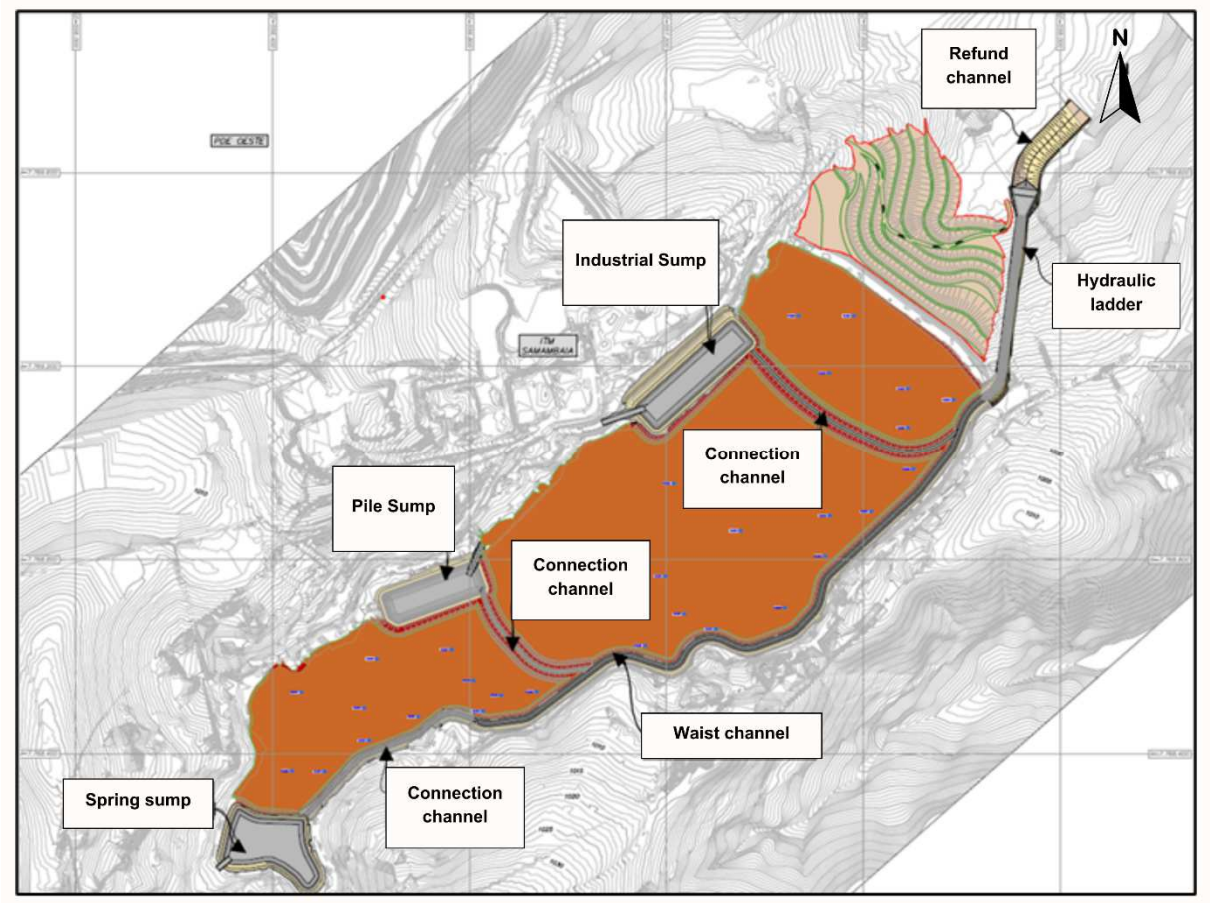


Fig. 8 - Detailed decommissioning Plan

After the completion of the decommissioning, revegetation will be carried out across the entire area, including the reservoir, the reinforcement slopes, and the surrounding areas. Additionally, new geotechnical instruments will be installed in the reservoir and the embankment, complementing the existing instruments to ensure active monitoring of the structure after decommissioning and to confirm the effectiveness of the interventions undertaken.

3.2. Planning and Construction Schedule

The construction planning for the Samambaia 0 (Zero) Dam was divided into three stages, each with critical activities. Furthermore, activities were classified based on their execution period: those to be carried out exclusively during the dry season to ensure the operational safety of the involved personnel, and those that can be executed during the rainy season. Within these stages, some activities can be performed simultaneously, meaning there is no mandatory dependency between them.

3.2.1. Construction Sequencing

Stage 1

The first stage involves the construction of a new spillway. This will include vegetation removal, excavation, leveling, and in-situ concreting with reinforced concrete, as well as the application of rockfill in the dissipation basin.

During this phase, sumps and channels will be constructed for storing and transporting rainwater, spring water, and industrial water. The implementation of the sumps will involve steps such as the excavation of tailings, leveling of the trench bottom, and lining with compacted clay material, geomembrane, and rockfill.

The reservoir of the structure will not serve as a flood attenuation feature. Instead, it will be leveled and sealed with a layer of approximately 100 centimeters of clay soil. Surface drainage from the reservoir will be directed to the perimeter channel on the right margin of the reservoir.

Stage 2

The second stage of the project involves the removal of soft and organic soil identified in significant quantities downstream of the dam. This process is considered critical due to operational challenges and the approximately 5-meter thickness of low-strength soil identified during geotechnical investigations. Removal will be carried out through mechanized excavation, followed by replacing the removed material with rockfill and a transition layer made of sand and gravel. These interventions aim to improve the foundation's load-bearing capacity

for the embankment to be constructed, as well as to mitigate deformations in the structural system.

Subsequently, adjustments will be made to the internal drainage system directed downstream of the reinforcement. This process includes removing the protection of the toe drain in rockfill and extending the sand transition layer to connect it with the rockfill of the treated foundation.

Stage 3

The final stage focuses on reinforcing the dam embankment using sterile material, predominantly composed of blocks, in a smoothed slope with a gradient of 4H:1V and a height of 40 meters. After this stage, a new surface drainage system will be implemented on the reinforcement, including berm gutters and peripheral channels.

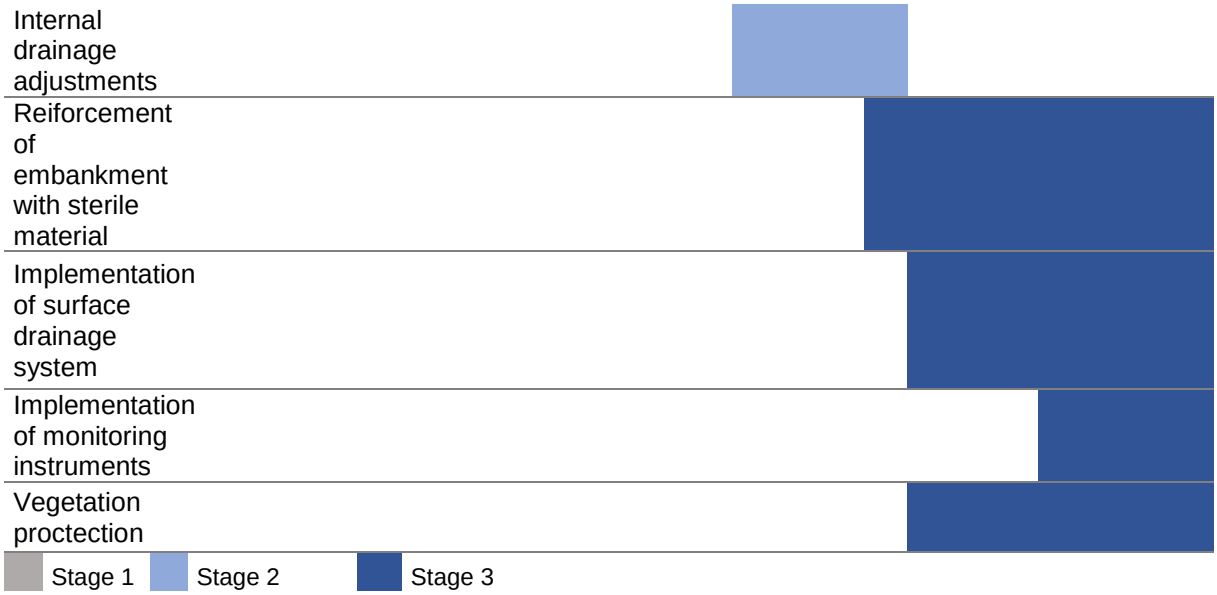
This stage is characterized by the completion of reservoir sealing and the beginning of vegetation protection. As well, the installation of new monitoring instruments is planned to ensure active and passive monitoring of the structure, particularly due to the non-removal of tailings.

3.2.2. Schedule

To complement the construction sequencing discussed in the previous subsection, Table 3 presents the schedule of the main construction activities by stage. The construction work is expected to last two years, with completion estimated for mid-2025, at which point active monitoring of the structure will begin. This monitoring will have a minimum duration of two years, as stipulated by ANM Resolution No. 95/2022 [1].

Table 3 – General Project Schedule

Stage	jun/23	jul/23	agu/23	set/23	oct/23	nov/23	dec/23	jan/24	feb/24	mar/24	apr/24	mai/24	jun/24	jul/24	agu/24	set/24	oct/24	nov/24	Dec/24	jan/25	feb/25	mar/25	jun/25
Construction of new spillway																							
Reservoir Sealing																							
Construction of Sumps and channels																							
Removal of soft and organic soil																							
Foundation reinforcement with rockfill																							



3.3. Risk Management

After selecting the risk analysis methodology and evaluating the complete history of the structure, according to Brito et al. (2024) [8], the process moves to the assessment stage, which includes the identification and evaluation of risks. It is essential to fully understand these risks, their causes, consequences, and the probability of their occurrence. Additionally, defining the acceptability and tolerability of the risks is a critical step.

The classification of risk levels was determined in accordance with ANM Resolution No. 95/2022 [1], as follows: acceptable, ALARP ("as low as reasonably practicable"), and unacceptable (or intolerable). Risks are quantified by the product of the occurrence index and severity index and subsequently classified as shown in Table 4.

Table 4 – Risk Classification

Classification	Scoring
Acceptable	From 1 to 5
ALARP	From 6 to 12.5
Unacceptable	From 12.5 to 25

The schedule impact assessment (IC) was incorporated into the risk management applied to the three stages of the dam decommissioning project. This inclusion was necessary due to the dynamic nature of risk management for a decommissioning structure, requiring an evaluation of activity planning and the impacts and risks to the structure's functionality associated with potential delays and unforeseen events.

The IC assessment aims to establish criteria for project activities and reevaluate risks in case of changes to the sequence of activities. To this end, IC scores ranging from 1 to 5 were assigned, as shown in Table 5.

Table 5 – Schedule Impact Classification

Score	Impact Level	Description
1	No Impact	The failure occurrence would not cause delays in the project schedule or compromise the structure's functionality.
2	Low Impact	The failure occurrence would cause minor delays, affecting only a single activity or having minimal impact on functionality.
3	Medium	The failure occurrence would cause moderate delays, potentially affecting subsequent activities or the structure's functionality.
4	High Impact	The failure occurrence would cause significant delays, affecting multiple stages of the decommissioning project.
5	Very High Impact	The failure occurrence would delay the project into critical periods (e.g., rainy season) or result in missed legal deadlines.

Table 6 presents the final risk classification associated with the schedule impact.

Table 6 – Final Risk Classification Associated with Schedule Impact

IC / Risk	1	2	3	4	5	Final Risk Classification
From 1 to 5						Regular
From 6 to 12.5						Important
From 12.5 to 25						Very important

Table 7 illustrates the application of risk management for one of the project stages.

Table 7 – Example of Risk Management Application

Phase	Loading and Transportation of Sterile Material
Objective	Disposal of material at the designated fill site
Possible Failure	Local mass movement
Intermediate Effect	Increased acting stresses
Final Effect	Instability
Cause	Overload due to vehicle movement
Severity Index	Medium (1.5 to 2.5)
Occurrence Index	Remote (2)
Risk Classification	Acceptable (3.4).
Schedule Impact	5
Final Risk Classification	Very Important.
Control (Occurrence)	- Topographic marking by a specialized team. - Visual inspection by an inspection technician. - Strict adherence to the executive project.

	- Ensure only necessary equipment and vehicles are present during the activity. - Technical oversight of the work, recording all occurrences in the RDO (Daily Work Report). - Team training.
Control (Schedule Impact)	- Define weekly milestones to monitor project progress and identify potential delays. - Weekly weather forecasting and adjusting the plan according to weather conditions. - Ensure machinery, workforce, and disposal areas are available.

Figure 8, in turn, provides a summary of the results obtained from the risk assessment of the Samambaia 0 (Zero) Dam. Each project stage entails unique risks associated with the works. Although no unacceptable risks were identified, Stage 3 stands out in risk management as it has the highest number of risks (11) and the most risks classified as ALARP (5).

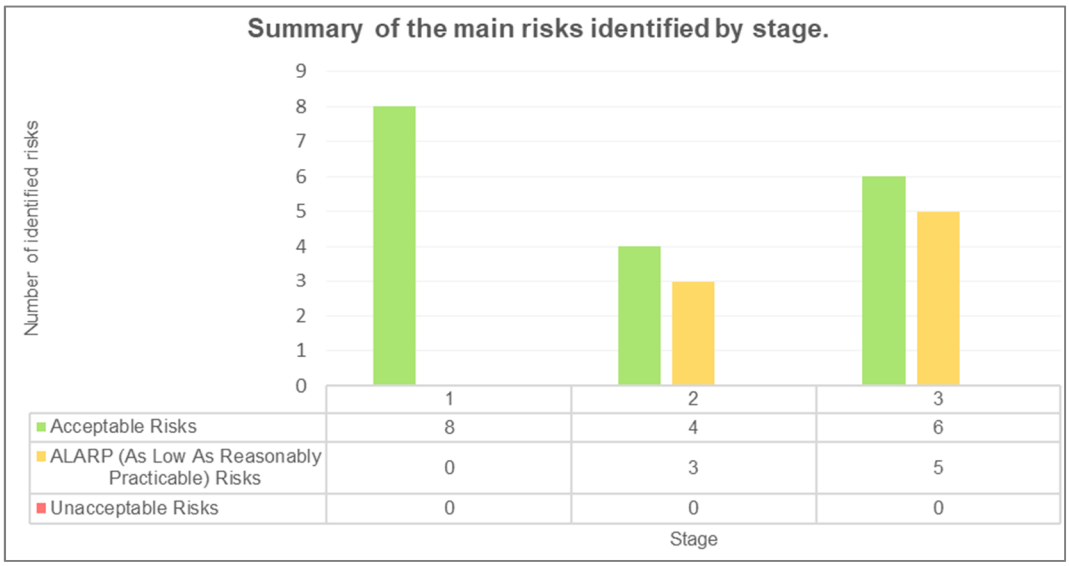


Fig. 9 - Summary of the main risks identified by stage

4. CONCLUSIONS

The relevance of strengthening Brazilian legislation focused on dam safety, in response to collapses such as those in Mariana and Brumadinho, highlighted the need for entrepreneurs to adopt stricter, more meticulous measures in the management of these structures.

The decommissioning of the Samambaia 0 (Zero) Dam reinforces the importance of respecting the specificities of each structure and adopting practices based on detailed technical studies and efficient risk management. Moreover, legislative improvements are necessary to better detail the methodologies for evaluating the effectiveness of the works

and defining the future use of decommissioning structures, promoting greater safety and sustainability in the mining sector.

The execution of the planning and schedule for the works on the Samambaia 0 (Zero) Dam is crucial to ensure safety and compliance with Resolution ANM 95/2022 [1]. Divided into three stages, the planning covers everything from the implementation of the new spillway and drainage systems to the removal of organic soil and the stabilization of the embankment with sterile material. The classification of activities into dry and rainy periods aims to ensure the operational safety of workers and the quality of the work.

The application of risk analysis allows for the evaluation of potential risks and their implications on the schedule. The incorporation of schedule impact assessment reinforces the need to meet the legal deadline for the closure of the structure. Although most risks were classified as ALARP, Stage 3 of the planning requires special attention due to the higher number of identified risks.

With the completion of the works scheduled for 2025 and subsequent active monitoring, this study is expected to serve as a reference for future interventions in tailings dams in Brazil and other similar contexts.

REFERENCES

[1] Brasil. (2022). Resolução ANM nº 95, de 7 de fevereiro de 2022. Estabelece diretrizes para a segurança de barragens de mineração e descaracterização de barragens de rejeitos. Diário Oficial da União, Brasília, DF, 8 fev. 2022. Available at: https://www.in.gov.br/en/web/dou/-/resolucao-anm-n-95-de-7-de-fevereiro-de-2022-379581106.

[2] Marais, L., Kemp, D., van der Watt, P., Matebesi, S., Cloete, J., Harris, J., Ang Li Ern, M., & Owen, J. R. (2024). The catastrophic failure of the Jagersfontein tailings dam: An industrial disaster 150 years in the making. *International Journal of Disaster Risk Reduction*, *109*, 104585. Available online 29 May 2024. DOI: https://doi.org/10.1016/j.ijdrr.2024.104585.

[3] Brasil. (2010). Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens. Diário Oficial da União, Brasília, DF, 21 set. 2010. Available at: http://www.planalto.gov.br/ccivil_03/_ato2007-2010/2010/lei/l12334.htm.

[4] Brasil. (2017). Portaria DNPM nº 70.389, de 17 de maio de 2017. Estabelece normas para a segurança de barragens de mineração. Diário Oficial da União, Brasília, DF, 18 maio 2017. Available at: https://www.gov.br/anm/pt-br.

[5] Minas Gerais. (2019). Lei Estadual nº 23.291, de 25 de fevereiro de 2019. Dispõe sobre a Política Estadual de Segurança de Barragens e dá outras providências. Diário Oficial do Estado de Minas Gerais, Belo Horizonte, MG, 26 fev. 2019. Available at: https://www.almg.gov.br.

[6] Brasil. (2020). Lei nº 14.066, de 30 de setembro de 2020. Altera a Lei nº 12.334, de 20 de setembro de 2010, para dispor sobre a Política Nacional de Segurança de Barragens e outras providências. Diário Oficial da União, Brasília, DF, 1 out. 2020. Available at: http://www.planalto.gov.br/ccivil_03/_ato2019-2022/2020/lei/L14066.htm.

[7] Minas Gerais. (2021). Decreto nº 48.140, de 25 de fevereiro de 2021. Regulamenta a Política Estadual de Segurança de Barragens de Mineração. Diário Oficial do Estado de Minas Gerais, Belo Horizonte, MG, 26 fev. 2021. Available at: https://www.almg.gov.br.

[8] Brito, E. S., Arantes, A. L. M., Rigatto, I. B., Pires, I. H., & Romanini, A. (2024). Development of a Risk Management Process for the Decommissioning of a Mining Dam. In *XXI Brazilian Congress of Soil Mechanics and Geotechnical Engineering*, *X Brazilian Rock Mechanics Symposium*, *X Brazilian Symposium of Young Geotechnical Engineers*, Balneário Camboriú, SC, September 24–27, 2024. Proceedings [...]. Balneário Camboriú, SC, Brazil: Brazilian Society for Soil Mechanics and Geotechnical Engineering.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

INFLUENCE OF SOIL AND CLIMATE VARIABILITY ON EARTH DAM DESIGN IN BRAZIL

Darym J. F. Campos[1]
Progen S. A.

Fernanda R. Sol[2]
Vale S. A.

José D. Veras[3]
Progen S. A.

Denis D. Costa[4]
Progen S. A.

Stephanie Y. Y. Ferreira[5]
Vale S. A.

Paulo C. R. Gomide[6]
Vale S. A.

ABSTRACT

Brazil's vast territorial expanse reflects its great diversity of soils and climates. This variability presents a complex issue due to the uncertainties stemming from limited knowledge of general soil and climate conditions, which ultimately impact engineering projects. Different soil permeabilities and bearing capacities, as well as intense rainfall or drought periods, can compromise the integrity of a structure. Additionally, it is common in practical cases to encounter misinterpretations of collected data due to human factors or inappropriate data collection methods, resulting in unreliable information about the subsoil. Therefore, it is crucial to conduct detailed geotechnical tests and climate studies to maximize the safety of the construction and the reliability of the information. The objective of this research is to analyze and discuss the interpretation of results from testing campaigns aimed at the design of an earth dam for tailings containment, considering investigations conducted in different periods and years, with climate as a significant aspect. To contribute to the findings, numerical simulations were employed in a theoretical model of a downstream-raised dam, providing insights that will serve as a foundation for designers, leading to more reliable projects.

Keywords: Soil Variability, Earth Dam, Numerical Simulations, Geotechnical Interpretation

[1]darymjunior@gmail.com, Brazil

1. INTRODUCTION

Well-dimensioned geotechnical investigations of the site where an engineering project is to be developed, such as the structure of an earth dam, for example, are of utmost importance for understanding the characteristics and geotechnical parameters of local soils, especially those related to soil resistance and deformability, leading to a greater probability of optimizing geotechnical projects, as well as for possible subsequent projects in the same area. Therefore, having a robust database provides conditions for the elaboration of construction planning within the safety parameters required by current regulations and technical quality of the project.

The soils of Belo Horizonte, Minas Gerais, a city located in Brazil, present great variability, both in occurrence and properties. According to [1], geotechnical uncertainties are represented by this natural variability of the materials that make up the soil and by measurement errors during the tests carried out.

Regarding uncertainties arising from surveys and sample collections, [2] highlight the possibility of errors due to the equipment used in the tests, the operational procedures adopted and/or the dispersion of data due to the lack of training of the technical staff, which can add an empirical burden to the results. Another possible factor causing uncertainty would be the use or performance of investigations in periods prior to excavations and/or soil embankments, as is the case with dams, as well as the use of results prepared by different companies that have adopted different methodologies.

The objective of this article is to analyze and discuss the interpretation of test results in an earth dam project, in addition to the impact on the safety of the structure. To this end, data from three test campaigns from different years were interpolated and equalized, in addition to numerical simulations using Slide3 software from Rocscience®. The reflections presented in this research will serve as a basis for designers, making projects more reliable.

2. GEOLOGICAL-GEOTECHNICAL AND CLIMATIC FEATURE IN EARTHWORKS

2.1. Tropical weathering profile

Brazil, being a tropical country, has a high incidence of natural weathering, which can be divided into two types of weathering: chemical and physical, that is, water precipitation, temperature, and wind speed, which cause changes in rocks, generating residual soils, and in tropical regions where the incidence of these weatherings is higher, they generate tropical soils [3].

The properties of tropical soils and predictions based on drilling and spot tests do not always reflect the behavior and performance of a foundation or embankment due to the factors

mentioned in the introduction. In the unsaturated weathering layer, the behavior of the soil changes to a greater or lesser extent with the variation in humidity throughout the year, thus interfering with its competence. The saturated layer, normally considered in engineering practice, tends to be in the domain of less weathered soils, requiring special attention to the expansibility and structure of the soil, since both can impact the competence and the execution of an investigation campaign.

In general terms, the authors [4] and [5] mention that tropical soil profiles are divided into three zones: lateritic soils (deeply weathered), saprolitic soils (slightly weathered and transitional) and saprolites (slightly weathered rocks).

Lateritic soils do not have a significant link with the parent rock, and only primary minerals resistant to weathering, such as quartz, remain present. These soils are usually found in an unsaturated state and, depending on the subsurface geomorphology, climate, and precipitation and drainage conditions, they may remain in a saturated state in areas where water accumulates during a certain period of the year. On the other hand, saprolitic soils maintain the structure of the parent rock. The structural aspect when dealing with clayey and silty soils is particularly relevant, since the resistance depends on the orientation of the particles, thus requiring evaluations when conducting sampling surveys, as is the case with mixed surveys. It is worth noting that the zone corresponding to transition soils is generally not very thick and composed of soils with heterogeneous properties and behaviors, since there are points that are very weathered and others where there is still little, representing saprolitic soils.

Finally, another relevant feature of the weathering profile concerns spatial and temporal influences, intrinsically linked to the distribution of weathering agents in each location, encompassing factors such as human action, precipitation, moisture, climate and even the presence of vegetation. [6] emphasize that variables such as these can present great variation within the same geographic region, generating a spatial diversity of soil types, weathering levels and conditions and soil properties.

2.2. Saturation state, moisture and climate

Tropical regions, such as practically all of Brazil, are areas where the atmosphere is characterized by high temperatures and significant humidity levels. In addition, they have well-defined seasons marked by a dry period followed by a rainy period, in which case precipitation is the main factor that determines the local climate [7].

However, due to its territorial extension, Brazil is influenced by different atmospheric circulation patterns, which causes distinct regional climate conditions and strong climate variability that directly impact the spatial and temporal conditions of precipitation distribution, soil moisture, evapotranspiration etc. In the Iron Quadrangle region, an important area of mining interest in

Brazil, with approximately 700 km² located in the central part of the state of Minas Gerais [8], the rainy periods occur under the influence of summer monsoons while the winter is characteristically dry with no significant volumes of rain expected.

To characterize the climate of the area, evapotranspiration and precipitation data were selected from hydrometeorological stations provided by the National Institute of Meteorology (INMET). Station 83587, located in Belo Horizonte, was used to represent the standard average conditions of the Iron Quadrangle region. The historical series covers the period from 1968 to 2020, and the average monthly values of evapotranspiration and precipitation can be seen in Fig. 1 and Fig. 2 respectively.

The region has an average annual precipitation volume of 1600 mm and potential evapotranspiration of 1300 mm. However, it is possible to verify that the period of greatest rainfall in the area occurs from October to March (Fig. 2), with 62% of the average annual volume concentrated between the months of November to January, concomitantly this period has the lowest evapotranspiration rates. On the other hand, the driest months, from June to August, represent only 1% of the average annual precipitation and consequently high evapotranspiration rates.

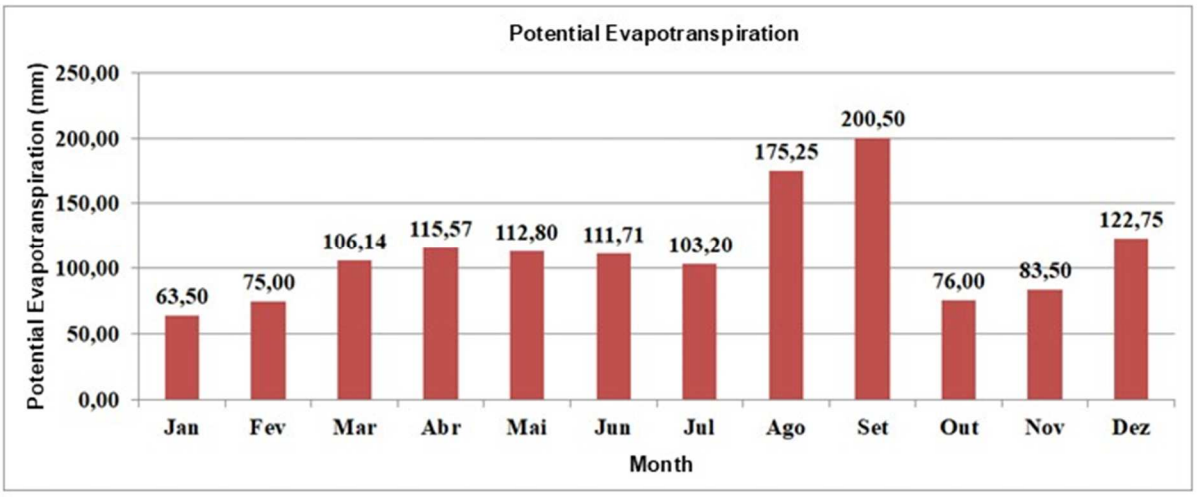


Fig. 1 - Average Potential Evapotranspiration – 83587 Station (INMET).

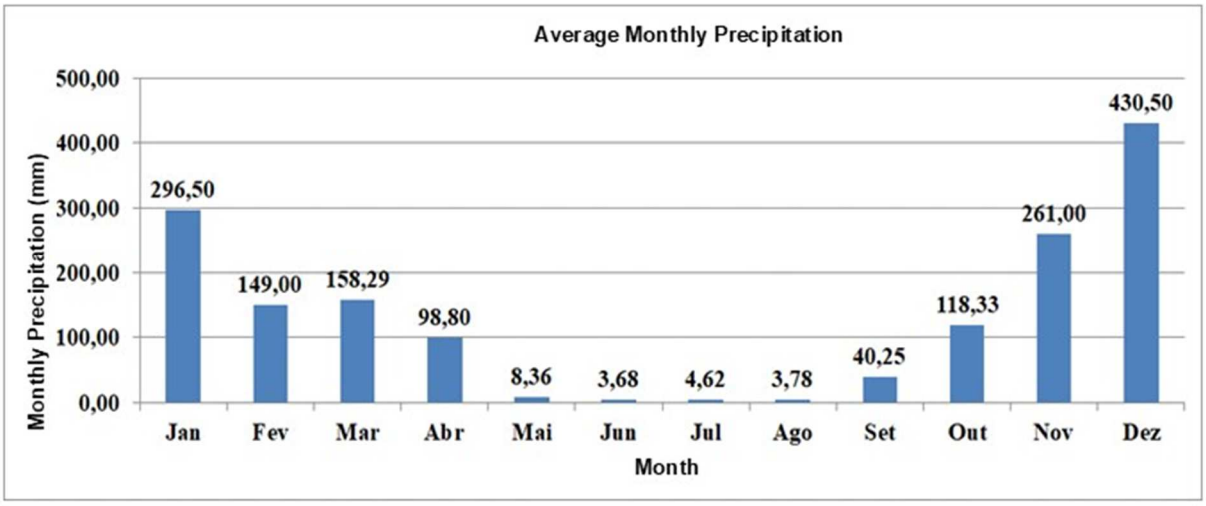


Fig. 2 - Average Monthly Precipitation – 83587 Station (INMET).

Thus, the existence of a direct soil-atmosphere interaction is evident, where the water balance in the atmosphere and precipitation conditions have a direct role in soil moisture properties, as well as in saturated/unsaturated soil conditions following a well-defined annual and seasonal variability according to the region's rainfall regime [9].

However, it is worth highlighting that such variations in saturated/unsaturated soil conditions are significant mainly in the most superficial layers that interact directly with soil-atmosphere conditions [10], that is, in the direct action of the effect of precipitation and evaporation on this portion, influencing the amount of water present in the soil.

Therefore, it is necessary to understand how the effects of climate variability conditions occur on the mechanical and hydraulic properties of soils, their impacts on the sizing of engineering works such as earth dams, and mainly on the interpretation of geotechnical investigations based on tracking that considers the periods (dry and rainy) of sample collection for testing.

2.3. Geotechnical investigations

Any civil engineering project requires a minimum knowledge of soil mechanics. In order to design, build and maintain earth dams, it is necessary to know the surface and subsurface of the environment in which the structure is to be built.

Based on this concept, geological-geotechnical studies are important to understand the various layers of the subsoil and determine their mechanical properties [11]. Geotechnical investigation and its constant technological evolution emerge in this scenario as a fundamental piece for the investigation of the foundation of tailings dam projects, aiming at an effective design and with guaranteed stability of such work, since the rupture of tailings containment can cause immeasurable social, economic and environmental damage.

Geotechnical investigation methods consist of procedures that aim to decipher the main characteristics of the soil in terms of physical, chemical and biological parameters and can be carried out in the field and in the laboratory. Field methods related to subsoil investigations can be classified into direct and indirect investigation methods.

Direct methods consist of any set of destructive operations designed to directly observe the soil or obtain samples during drilling. They can be manual and mechanical (inspection wells, trenches, drilling – auger, percussion, rotary and mixed). While indirect methods are defined as non-destructive investigation methods (geophysical studies).

According to [12], in situ tests are advantageous for evaluating the natural condition of the material. However, the aim is not to replace laboratory tests or surveys, but rather to complement them to increase the accuracy of predictive models.

Earth dams are works that aim to contain fluids (water or waste), so the main search is for materials that have low permeability characteristics, that is, clayey or silty soils, but it must also be noted that the materials must have good stability and deformability characteristics [13]. It is essential to analyze the resistance, deformability and permeability of the foundation material on which the dam will be built.

Laboratory tests must be carried out to obtain the resistance parameters of the foundation materials and the materials from the borrow pits that will be used in the dam body. It is important to note that the grain size is an important characteristic and determines the behavior of its deposition.

All tests performed in the laboratory are conducted using procedures established in technical standards. This reflects a standardization in the execution and ensures greater reliability. Although some stages of the execution of the tests (manual procedure) are exposed to some associated error, mainly in the characterization tests, another important issue that can interfere in the execution of the tests and the results obtained are the handling of the material during sampling, the period in which the sampling was performed and the transportation to the laboratory.

As a rule, the extraction of a sample (especially an undeformed one) requires special care, which should include its protection and transportation to the laboratory. For different materials, we may use different samplers, depending on which is most suitable for the resistance of the ground. In addition, techniques for preserving the integrity of the sample are standardized and must be strictly followed to ensure greater reliability in maintaining the condition of the sample and its geotechnical characteristics.

During investigations, different periods (rainy and dry) can contribute positively or negatively when it comes to material sampling. The moisture content of a sample is reflected in its

mechanical behavior during sampling. Furthermore, during laboratory tests, some materials may present different geotechnical parameters for each climatic period to which they were exposed at the time of sampling.

Therefore, the crossing of data obtained in the field and in the laboratory should promote the corroboration of the results between them, establishing the geotechnical conditions for different climatic periods and ensuring greater effectiveness in predicting the behavior of the structure.

3. METHODS

The methodology of this article follows a chronological order to facilitate understanding. The survey of geotechnical investigation campaigns, described in the following item, was used as a database, which served as a basis for the development of the article.

With the test results in hand, interpolations of the stratigraphic data obtained for the entire useful terrain were performed, aiming at a better understanding of its complexity through detailed analyses. Furthermore, the data were equalized according to the results of the laboratory tests for the pertinent soil layers and field conditions over time, bringing discussions and reflections for the understanding of soil behavior and its variability.

Once the soil profile and resistance parameters to be used in the earth dam design for tailings containment were defined, numerical simulations were carried out to assess the safety of the structure.

4. GEOTECHNICAL INVESTIGATION CARRY OUT

Geotechnical investigations mainly include mixed drilling and laboratory tests. For this earth dam for tailings containment, three holes were considered for the section analyzed to define the layers, as shown in Fig. 3, and to define the resistance parameters used in the stability analyses, five samples of each material from the campaigns of each year were used.

4.1. Investigation campaigns

When analyzing the stratigraphic profiles of the different campaigns over time, a great variability between the results can be observed, despite having been carried out in nearby locations. This difference is due, on the one hand, to the variation in the tactile-visual typification of the samples collected by the responsible company and, on the other hand, to the period in which the tests were carried out and the samples collected, mainly in the subsurface layers.

Fig. 3. shows the geotechnical section of the soil profile generated from the interpolations, where the layers of residual soil, saprolite and phyllite (parent rock) can be identified, as well

as how the dam embankment and its internal drainage system were designed. The dam was designed in compacted soil embankment, using the conventional method, and is 61 meters high, with 10m high slopes, 5m wide berms and a slope between berms of 2H:1V.

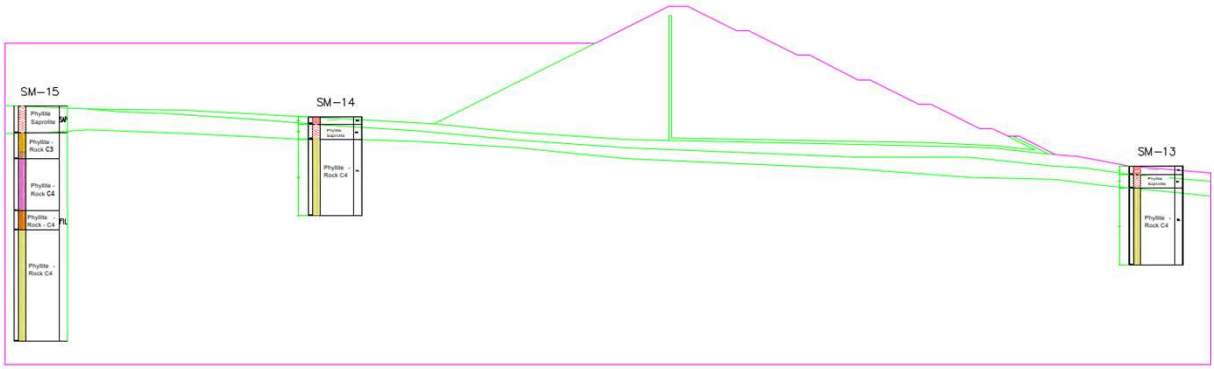


Fig. 3 - Geological section according to borehole.

The proposed modification for the compatibility of the stratigraphic layers was established based on the understanding of the results of the three drilling campaigns and the interpolation methods, involving four fundamental steps, listed below: 1) The predominant soil types were observed: Phyllite Residual Soil, Phyllite Saprolite and Phyllite; 2) The main secondary characteristic of each profile was interpreted; 3) The listed data were related to the SPT values (logs) and how they varied with depth, indicating which layers would represent the same mechanical behavior; 4) They were compatible through interpolation.

The corrections were used to simplify the characteristics in a coherent manner. With the compatibility applied, the soil/rock types and mechanical behavior were equalized between the three drilling campaigns, allowing implementation in the software. It is worth noting that mechanical behavior directly impacts the stability analysis of dams.

4.2. Laboratory tests

The samples obtained from these investigation holes were sent to a soil laboratory that was responsible for carrying out all the geotechnical characterization of the materials through characterization tests and special tests.

The geotechnical characterization of these samples included characterization tests and special tests, where the tests followed all their regulations between their preparation and the execution of the respective tests. For the characterization tests, natural moisture, complete granulometry (sieving and sedimentation), Atterberg limits and real specific mass of the grains were performed.

Among the results obtained in the characterization tests, we can highlight that the studied materials have a predominantly silty granulometry with variants of sandy silt, clayey-sandy silt, in addition to silty clay. These classifications presented give us natural moisture values

between 6% and 41% and the plasticity results reflect samples with low to medium plasticity and compressibility (ML or CL).

Special undrained consolidated triaxial tests were also performed. Each sample was tested on four specimens, where the confining stresses performed were 200 kPa, 400 kPa, 800 kPa and 1600 kPa, totaling 20 specimens.

Among the results obtained, we highlight below the resistance parameters of the triaxial tests, which were used for stability simulations that will be presented in the following chapter.

It is important to highlight that, for this 2011 campaign, the surveys and sampling were carried out during a period of high precipitation, which immediately alters the saturation conditions of the materials and may reflect divergences in the results expected for the laboratory tests.

Table 1 – Shear strength parameters - Campaign 2011.

		Phyllite Residual Soil	Phyllite Saprolite	Phyllite – Rock
	Color			
AM-01-F1	γ (kN/m³)	19	19	20
	c' (kPa)	8	15	46
	ϕ' (º)	24	26	28
AM-02-F1	γ (kN/m³)	19	19	21
	c' (kPa)	10	20	51
	ϕ' (º)	23	28	27
AM-03-F1	γ (kN/m³)	19	20	19
	c' (kPa)	7	23	51
	ϕ' (º)	24	30	29
AM-04-F1	γ (kN/m³)	18	18	21
	c' (kPa)	7	20	50
	ϕ' (º)	24	28	28
AM-05-F1	γ (kN/m³)	20	19	19
	c' (kPa)	8	22	52
	ϕ' (º)	25	28	28
AVERAGE	**γ (kN/m³)**	**19**	**19**	**20**
	c' (kPa)	**8**	**20**	**50**
	ϕ' (º)	**24**	**28**	**28**

It is important to highlight that, for this 2019 campaign, the surveys and sampling were carried out during a period of low precipitation, which impacts the results differently from the previous campaign.

Table 2 - Shear strength parameters - Campaign 2019.

		Phyllite Saprolite	Phyllite – Rock
	Color		
AM-01-F1	γ (kN/m³)	17,6	20
	c' (kPa)	23,5	69
	ϕ' (º)	24	27
AM-02-F1	γ (kN/m³)	18	25
	c' (kPa)	21	68
	ϕ' (º)	25	27
AM-03-F1	γ (kN/m³)	17,4	18
	c' (kPa)	19,6	70
	ϕ' (º)	30	28
AM-04-F1	γ (kN/m³)	18	21
	c' (kPa)	26	71
	ϕ' (º)	27	28
AM-05-F1	γ (kN/m³)	17	26
	c' (kPa)	27,4	72
	ϕ' (º)	24	30
AVERAGE	**γ (kN/m³)**	**17,6**	**22**
	c' (kPa)	**23,5**	**70**
	ϕ' (º)	**26**	**28**

The results of the most recent campaign in 2024 are presented below, the sampling of which was also carried out during a period of low rainfall, which impacts the test results.

Table 3 - Shear strength parameters - Campaign 2024.

		Phyllite Saprolite	Phyllite – Rock
	Color		
AM-01-F1	γ (kN/m³)	18	20
	c' (kPa)	21	59
	ϕ' (º)	31	33
AM-02-F1	γ (kN/m³)	19	20
	c' (kPa)	20	60
	ϕ' (º)	29	33
AM-03-F1	γ (kN/m³)	18	20

	c' (kPa)	22	58
	ϕ' (º)	28	32
AM-04-F1	γ (kN/m³)	19	20
	c' (kPa)	18	63
	ϕ' (º)	28	34
AM-05-F1	γ (kN/m³)	21	20
	c' (kPa)	19	60
	ϕ' (º)	29	33
AVERAGE	**γ (kN/m³)**	**19**	**20**
	c' (kPa)	**20**	**60**
	ϕ' (º)	**29**	**33**

Table 4 presents a compilation of the parameters obtained through the results of laboratory tests carried out in 2011, 2019 and 2024. It is worth noting that the parameters of the soil embankment, as well as the filter and tailings, remained the same for the analyses.

Table 4 - Soil investigation – Campaigns 2011, 2019 and 2024.

Material		2011			2019			2024		
		γ (kN/m³)	c' (kPa)	ϕ' (º)	γ (kN/m³)	c' (kPa)	ϕ' (º)	γ (kN/m³)	c' (kPa)	ϕ' (º)
Phyllite Residual Soil		19	8	24	-	-	-	-	-	-
Phyllite Saprolite		19	20	28	17.6	23.5	26	19	20	29
Phyllite – Rock		20	50	28	22	70	28	20	60	33
Soil embankment		22	10	35	22	10	35	22	10	35
Filter		19	5	35	19	5	35	19	5	35
Tailling		22	1	32	22	1	32	22	1	32

5. NUMERICAL SIMULATIONS

The methodology used for the numerical simulations of stability analysis was the limit equilibrium method, which consists of the subdivision into slices and the force and moment equilibrium equation [14].

The numerical simulations were performed using the Slide 6.0® software and were based on the Mohr-Coulomb (MC) rupture criterion, by the limit equilibrium methods proposed by Bishop, Morgenstern-Price and Spencer, always considering the lowest FS value among the three methods. It was considered that the materials are isotropic and homogeneous, and non-

circular search surfaces (Cuckoo Search) were adopted to search for the critical rupture surface.

As presented in the previous chapter, the analyzed section was defined based on the geotechnical holes drilled in the region, while the resistance parameters were defined based on the average of the test results from the three years of investigation.

5.1. Constitutive model used

The constitutive model used for this article was the MC model, which considers the soil as an elasto-plastic and isotropic material, being a model widely used for its simplicity and reliability. However, it is a model that has difficulty considering anisotropic materials [15].

Thus, there are other constitutive models that could be used that have different criteria than the MC model, such as Drucker-Prager, Linear Mogi, Modified Lade-Duncan, Modified Wiebols and Cook and Unified strength theory. However, even with several improvements, each of the models has its limitations. Thus, in stability analyses carried out in practice, the most common due to its familiarity, the MC model is still the most used for soils, and it is the one that will be considered for this article [15].

5.2. Numerical simulations results

The results of the stability analyses are presented below, considering the geological-geotechnical section defined by means of the drilling holes, using the average resistance parameters found in each of the years of investigation and using the MC constitutive model and the Slide software.

Figures Fig. 4, Fig. 5 and Fig. 6 present the results of the stability analyses considering the resistance parameters of the 2011, 2019 and 2024 campaigns, respectively.

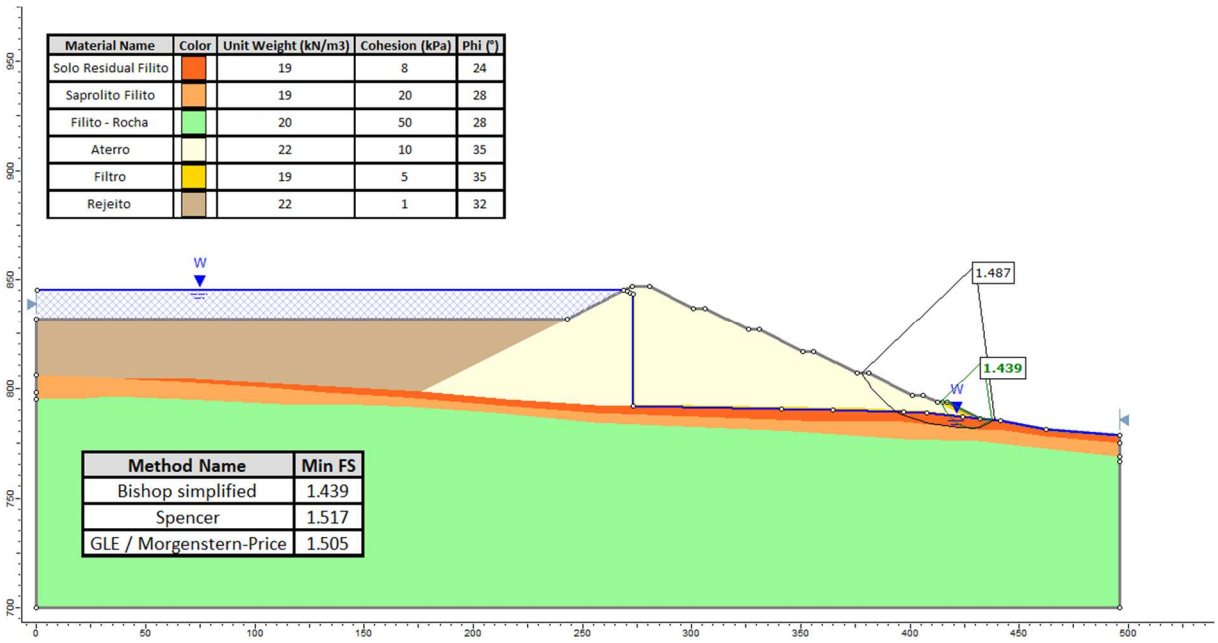


Fig. 4 - Stability analysis with parameters of 2011 campaign.

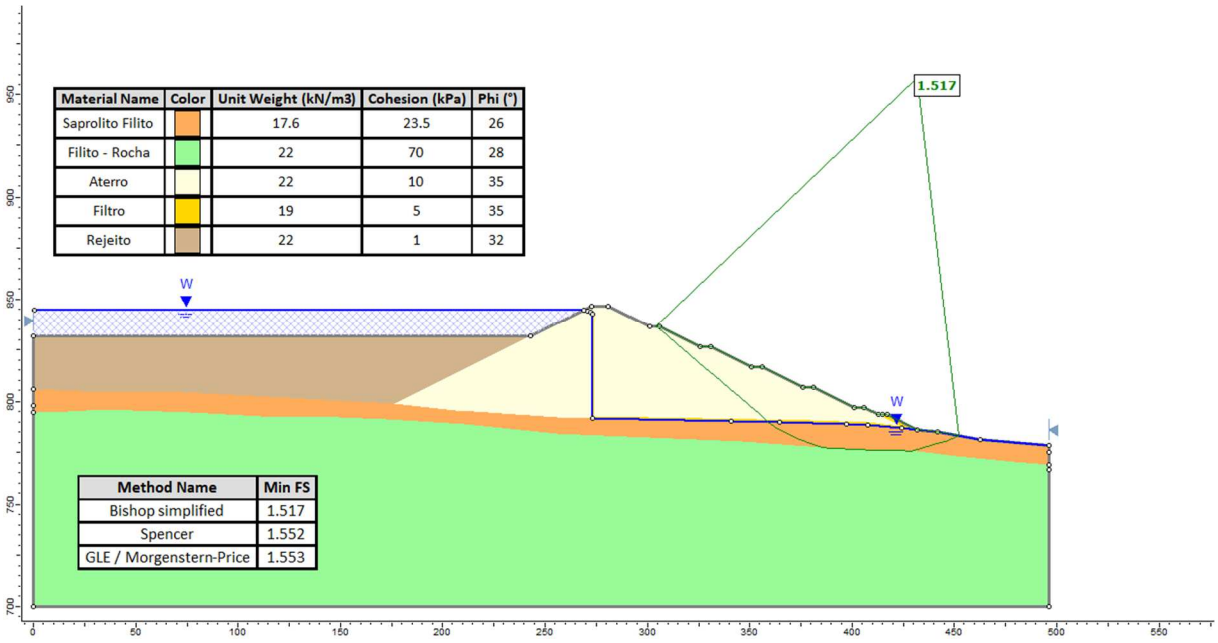


Fig. 5 - Stability analysis with parameters of 2019 campaign.

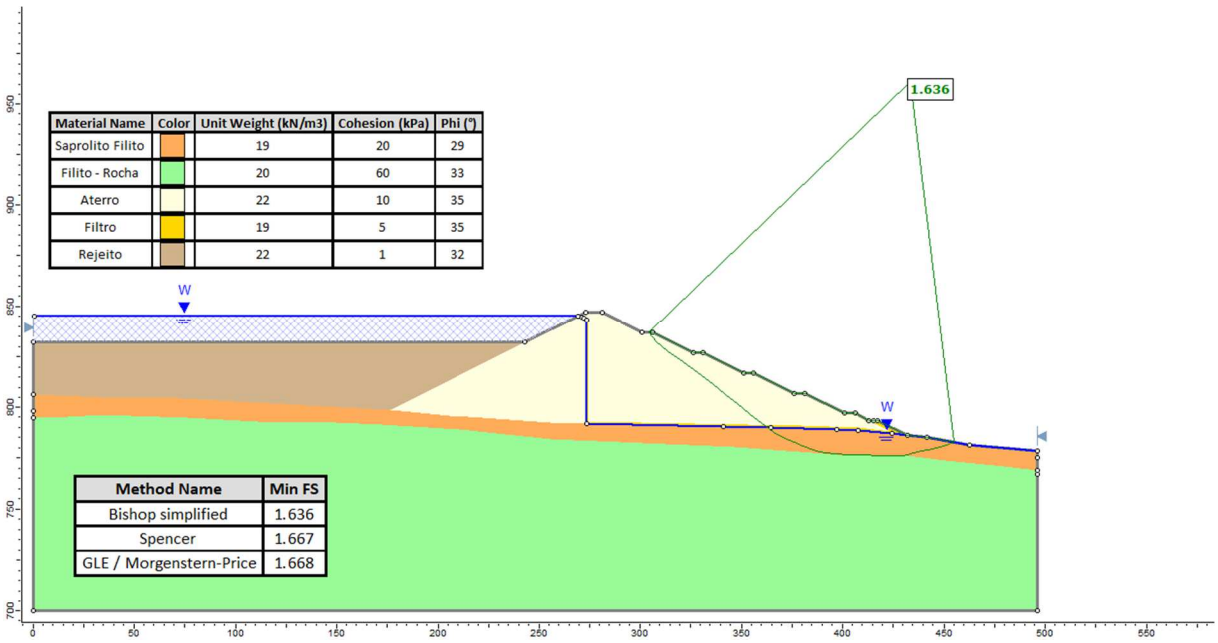


Fig. 6 - Stability analysis with parameters of 2024 campaign.

6. ANALYSIS OF RESULTS

The interpolation and equalization analyses of the test results used to define the resistance parameters will be presented below, in addition to analyzing the results of the safety factors and presenting some reflections due to the results found.

6.1. Interpolation/Equalization of Test Results

An important aspect already mentioned by [16] and [17] is related to the fact that even when geotechnical investigations are carried out close to or in exactly the same location, the results may be different simply due to variations in the testing techniques used. As a premise for the article, the same company was used to execute the 3 campaigns presented. Thus, despite the changes in the teams involved over time, it is understood that this would be a way to equalize the operational standard and be able to compare the results more fairly.

Inconsistencies were observed when comparing the results of the investigations, even when carried out in extremely close locations and during the same seasons of the year, as was the case in the 2019 and 2024 campaigns. However, it is worth highlighting that the 2011 campaign was carried out during the rainy season, which was evident when evaluating the results with lower values of the resistance parameters in the subsurface.

From a climatic point of view and according to [18], the most significant changes in the degree of saturation occur predominantly in the initial 3-4 meters of the soil profile, where the variation in soil moisture can reach about 35% throughout the year. For deeper strata, variations in soil moisture between the wettest and driest periods reach a maximum of 8%. Other studies, [19]

and [20], have shown that surface layers present greater variations in moisture throughout the year. In contrast, in lower layers, which are at greater depths, the soil is not significantly influenced by precipitation directly.

As a solution to reverse the data compatibility situation, when modeling the stratigraphic layers, some classifications previously established in the field were corrected, creating a single profile for the three campaigns that made physical sense, as shown in Fig. 3. Minimizing the influences on the results could, for example, be achieved with training for operators and automation of the test, initiatives that are gradually being implemented in the country.

6.2. Safety factors

When applying the resistance parameters of the campaigns in the geological section, the values summarized in Table 5 were identified.

Table 5 – Safety Factors

Campaign	2011	2019	2024
FS	1,49	1,52	1,64

Thus, it is possible to see that by not considering the most superficial material with lower parameters than the others, the FS increased, showing that possibly with the influence of weathering, the FS decreased.

Another important point to highlight is that the rupture circle did not reach the rock layer, due to its higher resistance parameter than the residual soil and saprolite layers. This fact shows that the data from these layers must be analyzed with caution, since they will direct the FS.

For these analyses, results from five test samples were used for each year. However, due to the difficulty of collecting an undisturbed sample, due to its heterogeneity, or due to the sample saturation condition, it is often not possible to take even one undisturbed sample, causing a lack of data reliability.

Furthermore, if there were a more extensive number of tests, it would be possible to carry out statistical stability analyses, which could help to better understand the structure's weaknesses and give greater reliability to the safety condition.

6.3. Reflections for dam projects

It is observed that the technological advances demonstrated in the practice of dam engineering are more related to the executive quality and improvement of equipment than to a broader understanding of the phenomena present during its execution.

The designs of these structures are fed by geotechnical investigations, in which a large part of the results end up being treated intuitively or according to the experience acquired by the designer and/or the executor, adding an empirical load to the result.

Thus, the degree of performance of executive processes and the degree of confidence in the capacity of masses with greater heterogeneity are limited. In this sense, it would be of great value to invest in resources to better understand the executive parameters, quantitative analyses of their variables and their results. Following this reasoning, [21] states that if the criterion used is based on the measurement of an instrument, this perception loses some of its importance in the data acquisition phase, producing more precise results and, therefore, classifications with a lower degree of empiricism and personality.

7. CONCLUSIONS

It was found that due to the timing of the tests and changes in the teams involved, there was a mismatch between the information and verifications of the samples generated by the three drilling campaigns, increasing the difficulty in interpretation and the uncertainties when compiling the stratigraphic data.

Given the points mentioned in this article, which can also be observed on several occasions in practical cases of dam engineering, the need to think and interpret the results based on reflection is highlighted as a way of contextualizing, understanding and adopting effective solutions for each case. In this article, to better utilize the research results generated, we opted for adaptations to the data based on a technical basis, leading to greater equalization or homogenization of data.

In summary, this article has shown that a dam project must reflect on the entire work in an integrated manner, seeking to understand how the stratigraphy, the N_{SPT}, the resistance parameters of the materials and the quality of execution of the executing company must be integrated for the analyses of behavior and competence on site to guarantee future projects and reliable and safe solutions.

Considering the heterogeneity of the foundation material and its possible changes, it would be enriching to carry out more extensive research and testing campaigns, analyzing more layers and carrying out several laboratory tests on each of the foundation materials, so that in this way, it is possible to find values for the most reliable resistance parameters.

Furthermore, if it were possible to have a greater number of results, it would be possible to apply probabilistic analyses of stability analyses, which could demonstrate the safety of the structure in a more realistic way.

REFERENCES

[1] Reinert, J. (2011). Estudo do comportamento das fundações profundas de obra industrial no litoral fluminense usando abordagem probabilística, Master Thesis, UFPR.

[2] Wang, Y.; Cao, Z.; Li, D. (2016). Probabilistic Approaches for Geotechnical Site Characterization and Slope Stability Analysis (2016).

[3] Sales, M. M.; de Carvalho, J. C.; Mascarenha, M. M. A. (2017). Erosão em borda de reservatório. A influência de aspectos geomorfológicos, de cobertura do solo e climáticos no surgimento e evolução de processos erosivos no entorno de reservatórios 8 (2017): 171-192.

[4] Cardoso, F. B. F. (2002). Propriedades e comportamento mecânico de solos do Planalto Central brasileiro, PhD Thesis, UnB.

[5] Camapum de Carvalho, J.; Gitirana Jr., G. P. N. (2021). Unsaturated soils in the context of tropical soils. *In*: Soils and Rocks, 44(3).

[6] Palmieri, F.; Larach, J. O. I. (1996). Pedologia e Geomorfologia. Geomorfologia e Meio Ambiente 2 (1996): 59-89.

[7] Bertacchi Uvo, C. (1998). Influence of Sea Surface Temperature on Rainfall and Runof in Northeastern South America: Analysis and Modeling, PhD Thesis, Lund University.

[8] Dorr II, J. N. V. (1969). Physiographic, Stratigraphic, And Structural Development of The Quadrilatero Ferrifero, Minas Gerais, Brasil. USGS Professional Paper, 641-A, 110p.

[9] Santos, C. H. B. (2023). Estudo numérico do efeito da interação solo-atmosfera sobre a variação sazonal da sucção em clima tropical, Master Thesis, UNESP.

[10] Samat, S. (2016). Thermomechanical modelling of ground response under environmental actions, PhD Thesis, UPC.

[11] Maragon, M. (2004). Tópicos em Geotecnia e Obras de Terra. Material didático ou institucional, UFJF.

[12] Knappett, J. A.; Craig, R. F. (2014). Mecânica dos Solos (2014).

[13] Cruz, P. T. (2004) 100 barragens brasileiras: casos históricos, materiais de construção e projetos (2004).

[14] Gerscovich, D. M. S. (2016) Estabilidade de taludes (2004).

[15] Xie, H.; Lu, J.; Li, C.; Li, M.; Gao, M. (2022) Experimental study on the mechanical and failure behaviors of deep rock subjected to true triaxial stress: A review (2022).

[16] Uzielli, M.; Vannucchi, G.; Phoon, K. K. (2005). Random field characterisation of stress-normalised cone penetration testing parameters (2005): 3-20.

[17] Ferrari de Campos, D. J. (2018). Energia de execução de estacas hélice contínua como ferramenta de avaliação da competência do terreno, Master Thesis, UnB.

[18] Camapum de Carvalho, J.; Mello, A. C.; Santos Neto, P. M.; Silva, C. M. (1995). Considerations on bearing tests on piles in collapsible soils with flooding. Meeting on unsaturated soils 2 (1995): 159-172.

[19] Mota, N. M. B. (2003). Advanced Field Trials on Unsaturated Porous Clay of Brasília: Interpretation and Application in Foundation Projects, PhD Thesis, UnB.

[20] Lessa, M. A. S. (2005). Influence of Pluviometric Indices on Tropical Soil Moisture as an Indicator of Suction Applied to Foundations, Master Thesis, UnB.

[21] De Mio. G. (2015). Condicionantes geológicos na intepretação de ensaios de piezocone para identificação estratigráfica na investigação geotécnica e ambiental, PhD Thesis, USP.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EMERGENCY ACTION PLAN AND THE PRECAUTIONARY PRINCIPLE IN PROTECTING LIVES

Marcelo Sant'Ana[1]
Fluminense Federal University

Verônica Velloso[2]
Fluminense Federal University

Márcia Velloso[3]
Fluminense Federal University

Daniela Lopes[4]
Fluminense Federal University

ABSTRACT

This article reviews the action plan for people with disabilities and training in the Secondary Safety Zone (ZSS). It addresses emergency levels, contacts involved, the description of the population with mobility difficulties or comorbidities, logistics and supplies, a map of the affected area, the location of potentially affected buildings, impacted assets, emergency notification flow, means of risk communication, and patient transport. The plan also considers human resources and response time. The main issue highlighted is the lack of specific legislation that protects populations in ZSSs. This prevents Civil Protection and Defense agencies (CPD) and others responsible for dam safety from planning and executing effective evacuation plans to save lives and reduce damage. It is proposed to develop safety measures that empower municipal PDEC bodies and the local community, promoting shared management of dam risks. The active and equal participation of entrepreneurs, security agencies and the population is crucial. The safety of populations in rescue zones becomes necessary and urgent, since managing dam risks requires an intersectoral and integrated approach, involving different spheres of government and civil society. The objective is to create a model of collective development that allows a positive coexistence with risk, respecting the socioeconomic and environmental context of the municipality. This article is based on the precautionary principle and the proposition of viable solutions. It is noteworthy that science and technological discoveries must be balanced with life, seeking to guarantee comfort and safety through a permanent action plan. This plan must include social mobilization and mental health protection, aiming to prevent or mitigate the cons of dam collapses, considering the complexity and economic, social and political specificities of populations close to dams.

Keyword: Emergency action plan (EAP), review of legislation, protection of lives, precautionary principle, dam failure.

[1] mmsantanbh@gmail.com, Brasil
[2] vmpvelloso@id.uff.br, Brasil
[3] marciavelloso@id.uff.br. Brasil
[4] daniela.lopesdmd@gmail.com, Brasil

1. INTRODUCTION

The main challenge identified is the lack of specific legislation aimed at protecting populations in ZSSs. This gap makes it difficult for Civil Defense and Protection agencies (PDEC) and other agencies responsible for dam safety to develop and implement efficient evacuation plans, compromising the preservation of lives and the mitigation of losses. In this context, this study aims to explore integrated solutions for dam risk management, with an emphasis on the protection of vulnerable populations.

This article focuses on reviewing action plans for People with Disabilities (PwD) and training in the Secondary Rescue Zone (ZSS), in addition to proposing an integrated approach for emergency management in dams. Through an interdisciplinary analysis, we seek to promote solutions that integrate social, environmental and technological aspects, respecting the precautionary principle and considering the specificities of the impacted communities.

2. LITERATURE REVIEW

2.1 Impacts of Dam Breaks

Dam failures are technological disasters with devastating impacts on the environment and affected communities. Studies indicate that the main damages include contamination of surface and groundwater, loss of biodiversity, destruction of infrastructure, loss of assets, and forced displacement of populations [1], [2]. In addition, the social and psychological implications for the affected populations are profound, including trauma, loss of livelihoods, and damage to mental health [3], [4].

According to Piciullo [5], the most frequent causes of dam failures include design errors, structural failures, lack of adequate maintenance, and extreme weather events. These factors are aggravated by the lack of rigorous monitoring and specific legislation that guarantees the safety of dams and the protection of vulnerable communities.

2.2 Risk Management and Rescue Zones

Dam risk management requires an integrated approach involving different stakeholders, including developers, security agencies, civil defense, and the local population itself. The literature highlights the importance of planning and implementing emergency plans, focusing on rescue zones, which are priority areas for evacuation and life protection [6].

Recent research indicates that Secondary Rescue Zones (ZSS) present specific challenges, such as the need to identify vulnerable populations, such as PwD, the elderly, and people with comorbidities. In addition, evacuation logistics, provision of supplies, and effective communication during emergencies are essential aspects that need to be addressed in robust action plans [7].

2.3 Legislation and Public Policies

The lack of specific legislation to protect populations in ZSS is one of the main obstacles to effective dam risk management. Although there are general regulations, such as the National Dam Safety Policy (PNSB), these are often insufficient to meet the specific needs of local communities regarding ZSS [8]. Studies suggest that a more integrated approach, involving different spheres of government and civil society, is essential to overcome these challenges.

2.4 Success Stories in Other Countries

The challenges faced in Brazil in relation to ZSS management can be analyzed in light of successful international cases:

2.4.1 **Canada:** Canada has implemented a robust monitoring system for dams, which includes continuous assessment of the condition of structures and conducting evacuation drills. The Canadian government requires mining companies to develop Emergency Action Plans that include safety zones, ensuring effective evacuation of at-risk communities [9].

2.4.2 **Sweden:** Sweden has strict dam safety legislation, which requires the creation of safety zones and detailed evacuation plans. Local authorities conduct regular drills with the public to ensure that everyone is aware of emergency procedures. This model has been shown to be effective in reducing response times during critical situations [10].

2.4.3 **Australia:** In Australia, some dam-affected regions have implemented programs that involve the community in developing Emergency Action Plans. This includes specific training for vulnerable groups, such as people with disabilities, ensuring that everyone has a clear evacuation plan and support during emergencies [11], [12].

2.4.4 **United States:** In the United States, the Army Corps of Engineers has developed detailed protocols for evacuation in areas adjacent to dams. These protocols include regular risk assessments and drills with the local population, which have contributed to a faster and more effective response in emergency situations. [13], [14], [15], [16], [17], [18].

2.4.5 **Norway:** Norway has excelled in creating tailored plans for vulnerable populations in areas near dams. This includes specific training for people with reduced mobility and the development of adapted evacuation routes, ensuring that all citizens are prepared for a possible emergency [19].

These international examples demonstrate the effectiveness of policies and practices that could be adapted to the Brazilian context to improve risk management in ZSS.

3. METHODOLOGY

This study adopted a qualitative and interdisciplinary approach, based on literature review, analysis of case studies and consultation with experts with the aim of identifying gaps and good practices in risk management.

In addition, official documents were analyzed, such as emergency plans, maps of risk areas and legal regulations.

4. CASE STUDY

4.1 Challenges and Advances in Dam Technology in Brazil

Technological progress has brought significant advances to society, covering areas such as electricity generation, mineral wealth production and water storage. However, these advances have also brought complex challenges, especially related to environmental and social impacts, in addition to the risks associated with critical infrastructure, such as dams. A study by Vale in Minas Gerais highlights the historical contradictions in the company's business systems that may have contributed to environmental disasters, such as the collapse of the Brumadinho dam [20].

Concerns about dam failures have become a central issue for local communities, experts, and policymakers due to their potentially devastating consequences. Cases such as the 2015 Fundão dam failure in Mariana and the 2019 Feijão dam failure in Brumadinho, both in the state of Minas Gerais, Brazil, have starkly demonstrated the environmental, economic, and social damage that can result from failures in this infrastructure. Analysis of the disasters reveals that the lack of adequate regulation and the absence of effective risk management were critical factors that contributed to the tragedies [21], [22], [23].

Effective implementation of standards and regulations is essential to ensure the safety of populations living in Secondary Rescue Zones (ZSS), especially people with comorbidities and

mobility difficulties. The lack of clear national legislation can result in preventable disasters. Research indicates that Brazilian authorities should adopt a more rigorous approach to ZSS, ensuring that all at-risk communities have access to adequate evacuation plans and the resources necessary to face emergencies [23], [24].

Growing concerns about the safety of mining tailings dams have intensified following the disasters in Mariana and Brumadinho. These events not only resulted in significant human losses, but also highlighted the urgent need for more effective planning in areas adjacent to dams. The Secondary Rescue Zone (ZSS) emerges as a crucial element in Action and Emergency Plans, as many communities living in these areas do not receive the same level of protection as those located in Self-Rescue Zones (ZAS) [20], [22].

The ZSS is defined as an area that is vulnerable to the risks associated with dam failures, but that does not have the same rigorous safety measures as the ZAS. The lack of specific regulations for the ZSS results in a gap in the protection of the populations living in these regions. It is imperative that these communities have access to adequate evacuation plans and the necessary resources to ensure their safety [24].

Furthermore, the ZSS concept must include considerations regarding the diversity of the local population, including people with disabilities and those with mobility difficulties. The absence of a clear protocol for the evacuation of these populations makes them even more vulnerable in critical situations. Therefore, it is essential to explore the importance of ZSS within the Action and Emergency Plans, proposing practical solutions that ensure the effective protection of communities at risk [23], [25].

4.2 Main Challenges in Implementing Training for the Population in Secondary Rescue Zones

4.2.1 Lack of Specific Legislation

The lack of clear regulations addressing ZSS makes it difficult to create adequate training and evacuation protocols. Without legal guidelines, companies and government agencies may not feel compelled to invest in training or appropriate safety measures for these areas.

4.2.2. Lack of awareness among the population about the risks

Many communities living in ZSS are not aware of the risks associated with being near dams. Lack of information can lead to apathy or lack of preparedness among the population, resulting in inadequate response during emergencies. Effective training should include education on the specific risks and actions required in the event of a dam failure.

4.2.3. Population Diversity

ZSSs often house a diverse population, including people with disabilities, the elderly, and families with young children. This diversity requires differentiated approaches to training that consider the specific needs of each group. A lack of resources and adapted strategies can result in exclusion during evacuation processes.

4.2.4. Limited Resources

Implementing training programs requires significant investments in human and material resources. Often, security agencies do not have sufficient budgets to conduct regular and comprehensive training, which compromises community preparedness.

4.2.5. Complex Logistics

Effective evacuation in emergency situations depends on adequate logistics, including well-defined and accessible escape routes. A lack of logistical planning can make it difficult to move people quickly and safely, especially those with reduced mobility or who require assistance.

4.2.6. Lack of Practical Simulations

Theoretical training without practical simulations may not adequately prepare the population for real-world situations. Practical exercises are essential to ensure that residents know how to act during an emergency, but these exercises are often not carried out as frequently as they should be.

4.2.7. Inefficient Communication

Communication about risks and evacuation procedures must be clear and accessible. Inappropriate use of available means (such as sirens or electronic messages) can lead to confusion during an emergency, making it difficult for the population to understand instructions.

5. FINAL CONSIDERATIONS

5.1 Results

The results of this study indicate that there are several deficiencies in emergency plans for dams in Brazil regarding the protection of vulnerable populations in the ZSS. It was identified that most ZSS lack:

1. **Detailed mapping of vulnerable populations:** Many communities do not have adequate records of PwD, the elderly or people with comorbidities.
2. **Evacuation infrastructure:** There is a lack of accessible means of transport and adequate shelter points.

3. **Capacity building and training:** The population and local bodies do not receive regular training for emergencies.
4. **Effective communication:** Warning systems often do not reach everyone in risk areas.

On the other hand, good practices were also identified, such as the use of technologies for real-time monitoring of dams and community mobilization in some regions.

5.2 Discussion

The findings of this study reinforce the need for a more integrated and participatory approach to dam risk management. The literature indicates that the precautionary principle should guide actions, ensuring that measures are taken before disasters occur (Reference). In this sense, the involvement of local communities is crucial to the success of any emergency plan.

These solutions must be accompanied by public policies that promote transparency and information sharing among entrepreneurs, government agencies and local communities. These solutions necessarily require legislation that provides the ZSS with the same care that exists with the ZAS.

6 REFERENCES

[1] Islam, K., & Murakami, S. (2021). Global -scale impact analysis of mine tailings dam failures: 1915–2020. Global Environmental Change, 70, 102361.

[2] Garcia, FF, Camilo Cotrim, CF, Caramori, SS, Bailão, EFLC, Nabout, JC, de Farias Neves Gitirana Junior, G., & Almeida, LM (2024). Mine tailings dams ' failures: Serious environmental impacts, remote solutions. Environment, Development and Sustainability, 1-23.

[3] Santos, MADL, Sol NAA, & Modena CM (2020). Territory and deterritorialization: Social suffering due to environmental disaster resulting from the rupture of mining dams. Saúde em Debate, 44(spe2), 262-271.

[4] Noal, DDS, Rabelo, IVM, & Chachamovich, E. (2019). The impact on the mental health of those affected after the collapse of the Vale dam. Public Health Notebooks, 35(5).

[5] Piciullo, L., Storrøsten, E.B., Liu, Z., Nadim, F., & Lacasse, S. (2022). A new look at the statistics of tailings dam failures. Engineering Geology, 303, 106657.

[6] Sant'Ana, MM, dos Santos, ALB, Von Krüger, PG, Velloso, MMP, Momose, AH, & Mello, F. (2023). Creation of an evacuation protocol for bedridden and mobility impaired people in mining tailings dam disasters. PROCEEDINGS HOME notebook.

[7] Sant'Ana, MM (2023). A case study of the emergency plan for bedridden people – Honório Bicalho – Nova Lima – MG in 2020 (Master's Dissertation). Postgraduate Program in Defense and Civil Security, Institute of Geosciences, Fluminense Federal University.

[8] Minas Gerais. (2024). GMG Resolution No. 83, April 16.

[9] Canadian Dam Association (CDA). (2013). CDA gives safety guidelines (Revised 2013).

[10] SOU 2012:46 Dammsäkerhet - Tydliga rule och tillsyn [Dam safety - Clear rules and supervision]. Stockholm: Fritzes.

[11] ANCOLD. (2019). Guidelines on tailings dams planning, design, construction, operation and closure.

[12] New South Wales Government Dams Safety Committee (NSW DSC). (2006). Risk management policy framework for dam safety. New South Wales, Australia.

[13] FEMA. (2023). Federal guidelines for dam safety. US Department of Homeland Security Federal Emergency Management Agency.

[14] FEMA. (2015). Federal guidelines for dam safety risk management. Federal Emergency Management Agency.

[15] US Army Corps of Engineers (USACE). (2021). Warning and mobilization curve generator workbook.

[16] US Army Corps of Engineers (USACE). (2020). HEC- LifeSim life loss estimation: Technical reference manual version 2.0.

[17] US Army Corps of Engineers (USACE). (2014). Engineering and design: Safety of dams – policy and procedures (Dept of the Army ER 1110-2-1156).

[18] US Bureau of Reclamation (USBoR). (2003). Guidelines for achieving public protection in dam safety decision making.

[19] Lacasse, S. (2022). Handbook risk assessment and risk management for dams. Statkraft.

[20] Botelho, MR, Vilela, RADG, Beltran, SL, Almeida, IMD, & Querol, MAP (2023). The insecurity of Vale dams in Minas Gerais: Application of activity theory to disaster analysis. Health and Society, 32(2).

[21] Lopes, LMN (2016). The collapse of the Mariana dam and its socio-environmental impacts. Sinapse Múltipla, 5(1), 1-14.

[22] Cabral, RN (2022). Risk and disaster management for the safety of mining tailings dams: The case of Brumadinho (Master's Dissertation). Technology Center, Department of Production Engineering, State University of Maringá.

[23] Barbosa, OS, Filgueira, HJA, & da Silva, TC (2023). Investigation into the causes of the rupture of mining tailings dams: Case study of the Fundão Dam in Mariana, Minas Gerais, using the Constellation Analysis technique. Brazilian Journal of Physical Geography, 16(04), 2047-2068.

[24] Fontenelle, AS (2007). Methodological proposal for risk assessment in dams in the Brazilian Northeast - Case study: Dams in the State of Ceará (Doctoral Thesis). Federal University of Ceará, Technology Center.

[25] Patrocínio, FRO, Bizawu, SK, & Silva, IM (2024). Mining tailings dams: Challenges and advances from the perspective of sustainability. Pensar Acadêmico, 22(2), 153-172.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

ITRB EXPERIENCE AT VALE MINERAÇÃO TAILINGS STORAGE FACILITIES

Marcos Eleoterio[1]
VALE

Filipe Costa[2]
WSP Canada

Eduardo Marques[3]
UFV

ABSTRACT

The Independent Tailings Review Board (ITRB) serves a vital function in the risk management of Tailings Storage Facilities (TSF), and the outcomes of their activities ought to be registered and communicated with main stakeholders. The Global Industry Standard on Tailings Management (GISTM), according to Requirement 8.7, mandates the establishment of an ITRB for facilities classified with very high or extreme consequences. Composed by independent experts, the ITRB conducts technical reviews and offers recommendations to ensure the integrity of these structures. Due to the ITRB's importance for the Tailings Management System, it is necessary to establish a governance that aims to ensure that the ITRB recommendations reach the company's operational and strategic levels. Furthermore, the Board's recommendations should, by default, be considered critical, and action plans must be established to address all of them. Major mining companies with multiple operations in different regions and countries face additional difficulty communicating ITRB recommendations through the various levels of the company. Vale has many active and inactive tailings storage facilities over its iron ore operations in Brazil, and most of them are classified, according to GISTM consequence classification criteria, as Extreme or Very High. These aspects bring additional challenges to Vale Tailings' governance and require a robust model to ensure the effectiveness of the ITRBs' work. This paper aims to show the lessons learned from Vale's experience implementing the ITRB for its TSFs and dams in iron ore operations and legacy mines for the last four years. It focuses on the ITRB meetings process, tracking the ITRB's recommendations, and the governance model, detailing its workflow and interactions with stakeholders.

Keywords: ITRB; Governance; Tailings Disposal Facilities; GISTM.

[1] marcos.eleoterio@vale.com, Brazil
[2] Filipe.SouzaCosta@wsp.com, Canada
[3] emarques@ufv.br. Brazil

1. INTRODUCTION

The Independent Technical Review Board (ITRB) serves a vital function in the safety management of dams and Tailings Storage Facilities (TSF), and it has been long used, but its practice has been limited until recently [1]. The Global Industry Standard on Tailings Management (GISTM), according to its Requirement 8.7, mandates the establishment of an ITRB for facilities classified as Very High or Extreme consequence. Composed by independent experts, the ITRB conducts technical reviews and offers recommendations to ensure the application of best practices on the management of these structures. Therefore, the purpose of the ITRB is to provide high-level guidance on the development of design, construction, maintenance, operation and decommissioning of dams and TSFs. The recent increase in the use of ITRBs by several mining companies is proof that the industry has recognized it as an important tool in the evaluation of TSF life cycle, focusing on its safety.

The GISTM recommends the establishment of ITRBs as an essential mechanism for governance and risk management, conducting periodic reviews of dam conditions and providing detailed recommendations for improvements and risk mitigation. "Effective implementation of ITRB recommendations is vital for preventing catastrophic failures and protecting communities and the environment" [2]. Even companies not fully compliant with GISTM consider using an ITRB as best practice [3, 4 and 5]. As stated by [6] "The demand for qualified professionals to serve on ITRBs has increased in recent years as the practice has become more widespread among responsible mining companies".

On this context, the purpose of the present work is to present the experience gathered by Vale based on a 4-year period of ITRB use on its Brazilian mines, including legacy operations. The process of implementing an Independent Technical Review Board (ITRB) is also highlighted, detailing its workflow and interactions with stakeholders. Focusing on the ITRB's workflow, the article highlights the sequence of technical activities, which includes periodic reviews, risk assessments, and improvement recommendations, as well as interactions with stakeholders such as mining and design companies, regulatory authorities, and local communities. This holistic approach has enabled a deeper understanding of the role and effectiveness of the ITRB as a one of the measures to ensure the safety and integrity of tailings facilities, demonstrating how its practices contribute to a more effective risk management in mining operation and, consequently, to the protection of the environment and affected communities.

2. METHODOLOGY

Figure 1 presents, as an example, the detailed flow of the process for implementing the ITRB at Vale, highlighting each step.

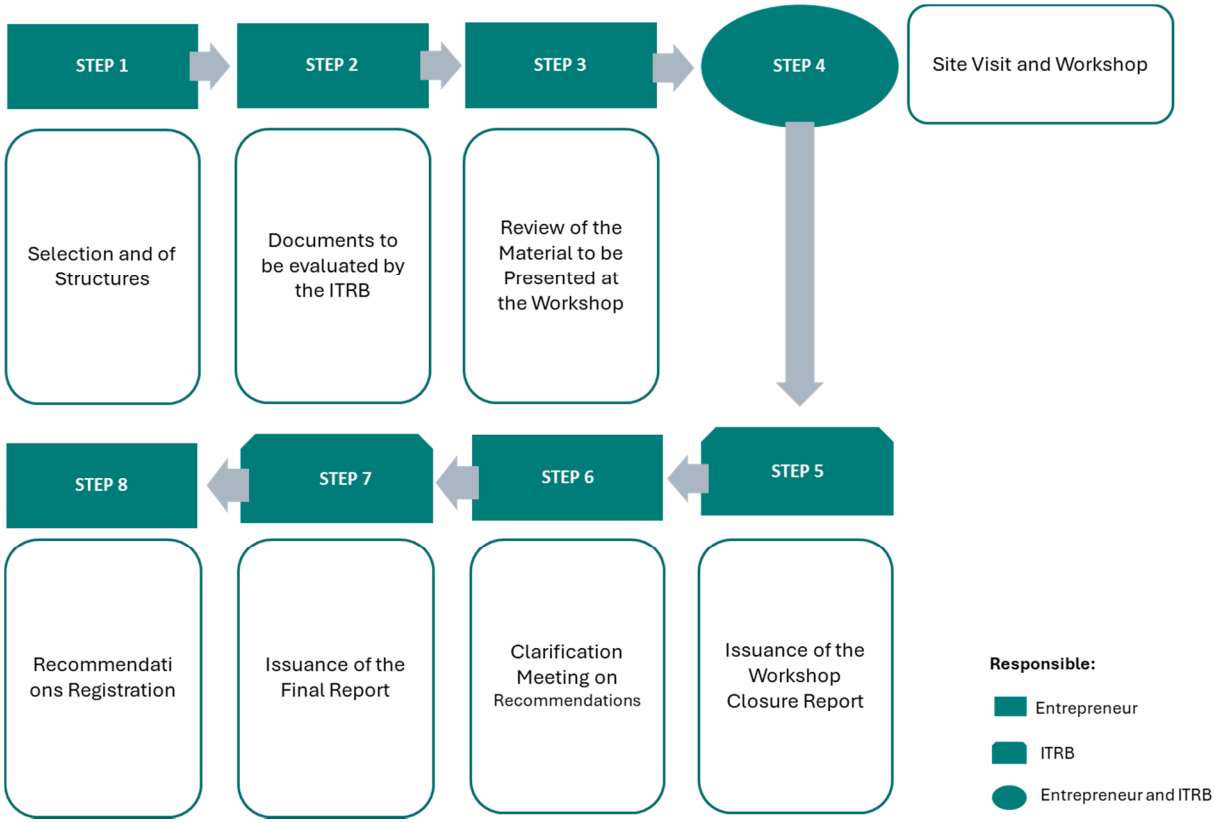


Figure 1 – ITRB Implementation Flow.

2.1. Step 1 – Selection and of facilities

ITRB Workshops are held twice a year at each site. The frequency may vary depending on the complexity of the Tailings Storage Facility, considering factors such as performance history, identified anomalies, and any significant changes in the structure or its operational behavior.

The initial step of the process involves selecting the structures to be addressed by the ITRB. This selection is guided by a rigorous consequence classification criterion, as defined in Table 1 of the Global Industry Standard on Tailings Management (GISTM). This matrix offers the classification of dams based on the potential impact of their failure, considering critical aspects such as risk to human life, environment, and property [2].

The GISTM Consequence Classification Matrix is a widely recognized framework that uses dam breach studies to assess the consequences considering five categories. This methodology considers multiple aspects, resulting in a classification that allows for the prioritization of dams based on the severity of their expected incremental consequences in the event of failure.

2.2. Step 2 – Documents to be evaluated by the ITRB

For the in person ITRB workshop to take place, it is necessary that the documents related to the structures be addressed by the ITRB be sent in advance. This timeframe ensures that the ITRB consultants receive the necessary information with adequate time to understand the operational, technical, and management aspects and the evolution after the previous meeting. It is recommended that versions in different languages be issued by the same person or, at the very least, approved by them, to ensure accuracy and consistency.

For Vale's ITRB, the following documents should be provided:

a) Dam Safety Inspection (DSI) from the Previous Cycle

A report containing data on the periodic inspection of structures, which should include:

- ✓ Date and description of the inspections performed;
- ✓ Identification of potential issues and anomalies; and
- ✓ Recommended corrective actions.

b) Dam Safety Review (DSR)

The most recent report with updated information on the integrity and performance of the dams, including:

- ✓ Detailed analysis of the current conditions of the dams;
- ✓ Performance and safety indicators; and
- ✓ Maintenance measures and improvements.

c) Monthly Reports from the EoR

At least the 3 last monthly reports prepared by the engineers responsible for operations (Engineer of Record – EoR), presenting an analysis of the conditions and activities of the structures, such as:

- ✓ Monitoring data;
- ✓ Maintenance and inspection reports; and
- ✓ Record of significant events and their respective actions.

d) External Audit Reports

Reports conducted by external entities, focusing on the most relevant aspects for the safety of the structures. They should include:

- ✓ Compliance assessments with standards and regulations;
- ✓ Identification of non-conformities and risks; and
- ✓ Recommendations for improvements and corrective actions.

e) Hazard Identification and Risk Assessment (HIRA) Report

The HIRA report, which identifies and evaluates the risks associated with the structures, which includes:

- ✓ Identification of potential hazards;

- ✓ Risk assessment associated with each hazard; and
- ✓ Recommended mitigation measures.

f) Dam Break and Consequence Classification of the Structure (GISTM)

Dam break analysis and consequence classification, according to the GISTM. This report should cover:

- ✓ Modeling of dam break scenarios;
- ✓ Assessment of potential impacts in terms of human lives, environment, and property; and
- ✓ Consequence classification, according to GISTM guidelines.

g) Decommissioning Design:

Relevant documents related to the decommissioning design, including:

- ✓ Geotechnical Design Report: Detailed geotechnical assessment of the project;
- ✓ Construction Report: Summary of the activities carried out and the results achieved; and
- ✓ Design Review Report: Review of the project design and evaluation of its compliance with technical and safety standards.

h) Design Basis Report (DBR)

Design Basis Report, which establishes the foundations and requirements for the design and construction of the structure. It includes:

- ✓ Project Objectives: Description of the project's goals and objectives;
- ✓ Input Data: Information used on project development, such as geological and hydrological data;
- ✓ Project Requirements: Specifications and criteria that the project must meet;
- ✓ Design Methodology: Approaches and methods used on project development;
- ✓ Risk Analysis: Assessment of associated risks and proposed mitigation measures;
- ✓ Design and Configuration: Detailed design of the structure; and
- ✓ Operational Requirements: Conditions and parameters that the structure must meet during its operational life.

2.3. Step 3 – Review of the Material to be Presented at the Workshop

The review of materials and presentations aims to check and validate the accuracy, clarity, and relevance of the information to be presented at the workshop for the ITRB. This step ensures that all documents and presentations are complete, correct, and aligned with the scope. This is essential to ensure that the workshop is productive and effective, meeting the proposed objectives and providing a detailed and accurate analysis of the structures discussed.

2.4. Step 4 – Site Visit and Workshop

The site visit is a crucial step that provides an important aspect of the ITRB work, as it provides a real and updated visual and effective contact to the site. It allows for a deeper and clearer understanding of all aspects of the structure or design, facilitating detailed discussion and a more precise approach to the topics during the workshop. Direct observation and data collection on-site help ensure that the information discussed and analyzed is relevant and accurate, contributing to a more robust technical presentation aligned with the observed reality.

To ensure a standardized and consistent technical presentation, the following is the proposed order for each content:

1. Agenda - Present the structure of the meeting or presentation, including the topics to be covered and the estimated time for each section.
2. Safety Share - Share relevant safety information, focusing on critical points and recommendations, presented only in the morning at the start of the activities.
3. Follow-up on ITRB Recommendations - Review the status of recommendations made in previous ITRB cycles, including implemented actions, pending items, and achieved results.
4. History/Overview of the Structure - Provide a summary of the history and evolution of the structure, including context and development over time.
5. Performance Evaluation of the Structure During the Period - Present geotechnical performance of the structure, based on the EoR Monthly Report, detailing the observed performance during the period.
6. Key Results from DSI or DSR.
    - ✓ Stability Analysis: Assess the stability of the structure, including any identified issues and their implications.
    - ✓ Design of Hydraulic Features: Review aspects related to the capacity of the dam to attenuate and convey the inflow design flood.
    - ✓ Recommendations: Present recommendations derived from the stability and hydraulic safety analysis, with suggestions for corrective actions or improvements.
7. Mine Waste Management and Water Balance Plan (when applicable);
   - Discuss the plan for tailings/sediments management and water balance, detailing the procedures and measures taken to ensure compliance and efficiency.
8. For Structures Under Construction or Adaptation/Decharacterization (Closure) Projects:

- ✓ Basic Characteristics of the Structure: Provide fundamental information about the structure, as per the original design;
- ✓ Project History – Timeline: Detail the project timeline, from planning to the current stage.
- ✓ Design Criteria and Assumptions: Specify the design criteria and assumptions adopted for the construction or adaptation.
- ✓ Design Review Recommendations: Present the recommendations resulting from the design review, if available.
- ✓ Geological-Geotechnical Investigations: Describe the investigation campaign, the results obtained, and their interpretation.
- ✓ Hydrogeological Model (when applicable): Show the hydrogeological model used, highlighting its implications for the project.
- ✓ Stress-Strain Model (when applicable): Provide details about the stress and strain model applied, if relevant.
- ✓ Instrumentation for the Construction Phase: Detail the instrumentation used during the construction phase, including objectives and expected results.
- ✓ Current Project Status: Report on the current progress of the project, including completed stages and upcoming phases.

2.5. Step 5 – Issuance of the Workshop Closure Report

At the end of the meeting a Closure Report based on all information and discussions provided is issued. Any additional information or document is also requested. After an agreed term, usually up to 20 days, a draft report is issued by the ITRB.

After the issuance of the draft report, a maximum of fifteen days is suggested for Vale's teams to review the document and provide detailed comments. This feedback is crucial for the preparation of the final report, which will be adjusted based on the observations and suggestions received.

The deadline for issuing the final report will be agreed upon with the ITRB Coordination, with an additional suggested period of fifteen days for completing this stage.

2.6. Step 6 – Clarification Meeting on Recommendations

In some cases, an additional meeting may be necessary, preferably online, to address specific issues related to the ITRB's recommendations. This meeting will serve to discuss and clarify points that require more details or explanations before the issuing of the final report. Conducting this step is crucial to ensure that all questions are fully understood by all

stakeholders and resolved, and that the final report accurately reflects the recommendations and observations made during the process.

2.7. Step 7 – Issuance of the Final Report

After resolving all questions and the necessary clarifications, the ITRB will proceed with issuing the final version of the report. This document will include all recommendations and detailed guidelines, incorporating the information and adjustments discussed during the process. The final version will reflect a comprehensive and accurate view of the ITRB's conclusions and suggestions, as agreed upon with the stakeholders.

2.8. Step 8 – Recommendations Management

After issuing the final version of the report, Vale will be responsible for recording the recommendations on its official management system. Additionally, the company must develop and input action plans to address the recommendations or provide justifications for any non-compliance or partial compliance. These action plans and justifications must be formally communicated to the ITRB.

To facilitate tracking and supervision of the recommendations, it is suggested to create a dashboard. This dashboard will provide leadership with a clear and accessible view of the recommendations and their handling, helping to monitor progress, review status, and ensure that necessary actions are implemented efficiently.

3. CASE – EXAMPLE OF THE CONTRIBUTION OF THE ITRB FOR THE TSF SAFETY MANAGEMENT

A case study with an example of the importance of the ITRB recommendations into TST safety is presented below.

The inflow hydrograph of a tailings dam, corresponding to the probable maximum discharge, was conservatively determined by rainfall-runoff transformation as recommended by [12], based on the probable maximum precipitation. The difference between the probable maximum inflow discharge, attenuated in the reservoir, and the discharge through the spillway system provides a minimum freeboard equal to 1.01 m. The spillway system consists of a 17.90 m wide weir with a crest at El. 662.94 m (resulting in a which defines a comfortable normal freeboard of 5.10 m), featuring a steep slope after the ogee. The steep slope resulting from raising the normal maximum water level in the reservoir (Figure 3). The chute has variable slopes ranging from 1.30% to 31.53%, with 2 m high side walls and a left turn before the stilling basin. In its upstream section, the chute shows signs of deterioration with surface wear due to spalling in the concrete structure, which must be repaired (Figure 4).

Figure 2 - Spillway warhead and gutter in its upstream section

Figure 3 - Surface deterioration of the spillway channel.

Based on the site visit and the information provided, this deterioration was identified by the EoR and presented to the ITRB , which provided the following recommendations:

- ✓ It is recommended, to assess the current risk, to calculate the discharge at which the current right-side wall would be overtopped and determine the recurrence interval of this discharge. This information will define the annual risk of an incident occurring on the spillway chute under the current conditions;

- ✓ It is recommended to take advantage of the current situation, in which the reservoir's water level is depleted, to repair the spillway's ogee from the existing surface spalling in the concrete.

The detailed analysis of the risks associated with the structure and the specific guidelines for repairs and improvements allowed for the identification of potential vulnerabilities and the adoption of corrective measures. The attendance of these recommendations was performed before the subsequent cycle. Figure 5 and Figure 6 show some pictures of the remediation works. The implementation of these recommendations directly contributed to risk reduction and the maintenance of the dam's integrity, reinforcing the commitment to safety and the sustainable operation of the structure.

Figure 4 Spillway repair works.

Figure 5 Progress of raising the side wall of the spillway chute.

4. CONCLUSIONS

The implementation of ITRB for tailings storage facilities has proved to be an essential step in enhancing these facilities' governance and safety management. It provides an additional and important aspect of the governance system for the mining companies that have adopted the board.

Two important aspects of these boards are that they should be composed of experts with practical experience on tailings disposal facilities and that at least one expert from each of the most important disciplines involved in the problem should be on the board. For some specific problems an external expert could be consulted to provide additional input for the board.

The ITRB group composition should comprise a balance between members' expertise and behavior. Members that do not have a collaborative behavior or those who create a less favorable environment for discussion should be avoided.

ACKNOWLEDGEMENTS

The authors would like to thanks VALE and Breno Castilho, General Manager of Geotechnical Governance and Compliance , from Vale.

REFERENCES

[1] Gerritsen, T; Johndrow, T.; Ridlen, P.W.; Winckler, C. & Yenne, L (2023). Leveraging Knowledge and Experience of a Well-Formed Independent Tailings Review Board to Enhance Tailings Facility Safety. Symposium "Management for Safe Dams" - 91st Annual ICOLD Meeting – Gothenburg 13-14 June 2023.

[2] GISTM (2020). Global Industry Standard on Tailings Management. Available at https://globaltailingsreview.org/global-industry-standard/. 40 p.

[3] Morgenstern, N. (2018). Geotechnical Risk, Regulation, and Public Policy – 6th de Mello Lecture. In Soils and Rocks, Sao Paulo. 41(2), 107-129.

[4] International Council on Mining and Metals - ICMM (2021). Tailings Management Good Practice Guide. Available at https://www.icmm.com/en-gb/guidance/innovation/2021/tailings-management-good-practice.

[5] The Mining Association of Canada - MAC (2021). Reference on adherence to the GISTM and the need for hiring an ITRB for facilities with Very High or Extreme consequences.

[6] Morgenstern, N. (2011). Improving the safety of mine waste impoundments. In *Proceedings of the 14th International Conference on Tailings and Mine Waste Conference, Vail, Colorado, USA.* CRC Press pp 3-10.

[7] Burnett A & McKenna G. (2022). Celebrating 50 years: the Syncrude Geotechnical Review Board. CDA Canadian Dam Association Conference, St Johns. Newfoundland, October 2022. 20 pp

[8] McKenna, G. (1998). Celebrating 25 Years Syncrude's Geotechnical Review Board, In *Geotechnical News*, No. 16, September, pp 34-41.

[9] McKenna, G. (2022). Building and Operating a Successful Geotechnical Review Board. Canadian Dam Association Annual Meeting, September 2022.

[10] Davidson, C. (2019). The Role of a Review Board in Creating Resilient Design. In *Proceedings of the 2019 ANCOLD (Australian National Committee on Large Dams) Conference.* ANCOLD. Hobart, Australia.

[11] Robertson, A., & Caldwell, J. (2015). Tailings Facility Review and Evaluation Standards. In *Proceedings Tailings and Mine Waste 2015 Conference, University of British Columbia, Vancouver*. Available at https://tailingsandminewaste.com/past-tmw-conferences/..

[12] Pinheiro, M. C. (2011) Diretrizes para elaboração de estudos hidrológicos e dimensionamentos hidráulicos em obras de mineração. Porto Alegre: ABRH, 2011. 308 p (In Portuguese).

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

COMPARATIVE STUDY OF 2D AND 3D SLOPE STABILITY ANALYSES APPLIED TO A TAILINGS DAM

Rayssa P. de Melo[1]
Federal University of Viçosa

Ananda Carneiro[2]
Federal University of Viçosa

Eduarda Almeida[3]
Federal University of Viçosa

Roberto Ferraz[4]
Federal University of Viçosa

Eduardo Marques[5]
Federal University of Viçosa

ABSTRACT

The 2D stability analysis is the most common stability evaluation approach used for dams in Brazil, as it must be addressed in accordance with Brazilian legislation, although is a simpler and more conservative way to obtain the safety factor. However, the two-dimensional model does not consider all the variables and boundary conditions that are involved in some real cases. The 3D model, on the other hand, can provide a geometry closer to reality, since it considers the topography and 3D aspects of the site of the dam. To check these differences between both options, this study has analyzed two and three-dimensional models of an iron ore tailing dam. Bishop, Spencer, and Morgenstern-Price methods were used to determine the Safety Factors and the 2D stability analysis, was carried out for 3 sections (center of the dam, left abutment and right abutment). The 3D analysis presented a result less conservative in relation to the abutments, and a more conservative result in relation to the center section, the latter being contemplated by the three-dimensional rupture wedge. The result agrees with previous expectations, since the center section of the dam usually presents more critical conditions, and its 2D analysis was more conservative than the 3D.

Keywords: Iron Ore, Tailings, Stability analysis

[1] rayssapedrosademelo@gmail.com, Brazil;
[2] ananda.carneiro@ufv.br, Brazil;
[3] eduardaao97@gmail.com, Brazil;
[4] rlferraz@ufv.br, Brazil;
[5] emarques@ufv.br, Brazil.

1. INTRODUCTION

Two-dimensional (2D) slope stability analysis is widely used in Brazil due to legal requirements, its simplicity and greater availability in the market. Despite its popularity, this method has limitations as it does not account for all variables and boundary conditions of the structure, especially those in which the shape of the valley is very steep and/or curved. In contrast, three-dimensional (3D) analyses offer a more realistic representation by incorporating greater complexity and detail into the geotechnical model (Bretas, 2020).

In this context, the present study aims to compare 2D and 3D analyses applied to the main embankment of an iron tailings dam located in northern Brazil. For this purpose, Slide2 and Slide3 softwares, from Rocscience, were utilized, allowing for an evaluation of the differences in factor of safety (FoS) obtained by both models. The analyses employed Bishop, Spencer, and Morgenstern-Price methods.

The 2D analyses considered three main sections (central section, left abutment, and right abutment). The 2D analyses resulted in a factor of safety (FoS) of approximately 1.6 for the central section and 1.8 for the abutments, with consistent results across the three methods. On the other hand, the 3D analysis resulted in an FoS of approximately 1.7, proving to be less conservative for the abutments but more conservative for the central section. These results align with the expectation that the central section of the dam exhibits more critical conditions, while the 2D analysis tends to overestimate the FoS due to its simplifications.

This study contributes to understanding the practical and theoretical differences between two-dimensional and three-dimensional models, emphasizing the importance of selecting the appropriate analysis method when evaluating dam safety.

2. LITERATURE REVIEW

2.1. Tailings Dam Construction Methods

Tailings dams play a critical role in mining, providing a means of disposing of the waste generated during mineral processing. According to Cardozo et al. (2016), the most common construction methods are:

- **Downstream Method**: Raises are constructed downstream of the initial dike, offering greater safety due to the stepped construction and efficient drainage. It exhibits low susceptibility to liquefaction and strong resistance to dynamic loads.

- **Upstream Method**: Involves raises on previously deposited tailings, which act as the foundation. Although it requires fewer resources, this method is more vulnerable due to low shear strength and high susceptibility to liquefaction.
- **Centerline Method**: Combines features of the previous methods, with raises constructed along a vertical axis, mitigating some disadvantages of the upstream method.

According to Cardozo (2024), unlike the downstream method, the center line and upstream methods - which is also the case for the dam analyzed in this study - involve partial rises over previously deposited tailings. While these methods reduce earthmoving costs due to the smaller volume of material required, they introduce complexities in construction control and in the installation and management of drainage and filter systems, which are critical for the structure's stability.

So, since most tailings dams in Brazil use the upstream raising method, and they have more complexities involved on its construction and control (Cardozo, 2024), it is crucial to understand its behavior.

2.2. History and Safety of Dams in Brazil

Mining in Brazil dates back to the 17th century, but the first tailings dams emerged in 1930 (Ávila et al., 2021). Over the decades, severe incidents like those in Mariana (2015) and Brumadinho (2019) exposed weaknesses in construction methods and safety management. These events spurred regulatory developments, such as Brazilian Law No. 12,334/2010 and ANM Resolution No. 13/2019, which mandate the decommissioning of upstream-raised dams by 2027.

Technological advancements have also enhanced monitoring and stability analysis capabilities, highlighting the importance of computational modeling in predicting critical scenarios and mitigating risks associated with these structures' operation.

2.3. Slope Stability Analysis

Slope stability analysis is essential for tailings dams and is commonly conducted using limit equilibrium methods, which divide the mass into slices and apply static equilibrium equations.

- **Factor of Safety (FoS)**: According to Fellenius (1922), the FOS is defined as:

$$FS = \frac{Available\ shear\ resistance}{Acting\ shear\ stresses} \quad (1)$$

For circular surfaces, it can be expressed as:

$$FS = \frac{Available\ moment\ resistance}{Acting\ moments} \quad (2)$$

- **Simplified Bishop Method**: Assumes circular surfaces and neglects shear forces between slices. The FoS is iteratively calculated using the equation (Bishop, 1960):

$$FoS = \frac{1}{\Sigma W sen\alpha}\ .\Sigma\left\{[c'b + tan\phi'.(W - u.b)].\frac{sec\alpha}{1 + \frac{tan\phi'.\ tan\alpha}{FS}}\right\} \quad (3)$$

- **Spencer Method**: Considers both force and moment equilibrium, providing a more rigorous analysis. It is based on total forces acting on the slices, with the assumption of parallel forces (Spencer, 1967).
- **Morgenstern and Price Method**: Assumes that the inclination, θ\theta, of the resultant force between slices varies according to a specific function along the failure surface. This differs from other methods, which maintain a constant relationship between forces for static determinacy (Morgenstern and Price, 1965). Thus:

$$T = \lambda\, f(x) E \quad (4)$$

or

$$tg\Theta = \frac{T}{E} = \lambda\, f(x) \quad (5)$$

Where λ lambda is a scalar parameter determined during the FoS calculation, and f(x) is an arbitrary function chosen based on the variation of forces between slices.

2.4. Computational Modeling and 2D and 3D Analyses

The Slide2 and Slide3 softwares (Rocscience) enable two-dimensional and three-dimensional stability analyses, respectively.

Two-dimensional (2D) limit equilibrium methods are based on a plane strain condition, assuming the failure surface is infinitely wide and that three-dimensional (3D) effects are negligible compared to the acting and resisting forces (Duncan, 1992). However, the safety factor in 3D analyses (FS3) is generally higher than in 2D analyses (FS2). This occurs because

FS2 is calculated for the most critical 2D section and disregards shear resistances along the lateral portions of the sliding mass, whereas FS3 encompasses a larger region.

According to Bretas (2020), 3D analyses tend to be more realistic and provide a better understanding of the problem, whether through failure mechanisms or the safety factor itself. The author also affirms that it is expected that 3D analyses present higher factors of safety (FoS) values, as 2D analyses do not account for lateral confinement effects on the failure surface.

However, Bretas (2020) statement appears to be true when comparing 3D models to specific 2D sections, and the validity of this relationship depends on the geometry and whether it is a critical region or not. According to Duncan (1996), studies on three-dimensional stability problems can be categorized into three main types of problems: slopes with curved or cornered geometries; slopes subjected to localized loads at the crest; and slopes where the potential failure surface is confined by physical boundaries.

Regarding the importance of considering the geometric conformation of the terrain and structure, results obtained by Kelesoglu (2015) reveal that concave slopes exhibit safety factors (FoS) approximately 5% to 25% higher than those of straight slopes, with FoS increasing as the concavity curvature becomes more pronounced. However, for convex slopes, curvature did not significantly affect stability determination.

Also, Qiu et al. (2006) affirmed that a three-dimensional (3D) analysis must be considered: when slope geometry and the failure surface vary significantly in the lateral direction; or the material properties are highly anisotropic; or the slope is locally loaded. Despite the variety of 3D methods proposed in the technical field, a practical and computationally efficient 3D model for stability analysis is still required.

Considering this, the most common cases for applying 3D analyses involve slopes with complex topography, foundation materials with different resistances than those composing the structure, and unusual pore pressure conditions. The presence of one or more complexity factors necessitates understanding the three-dimensional nature of the structure to calculate the safety factor (Arellano, 2001). Bromhead (2004) highlights the importance of 3D analysis in cases where combinations of soil properties and local geology lead to more critical failure mechanisms. According to Bromhead (2004), in topographies that produce highly variable cross-sections, 3D analysis is expected to better represent reality compared to 2D analysis.

In general, 2D analyses are suitable for slope design, as they provide a conservative estimate of the safety factor. However, 3D analyses are more commonly applied to slopes with complex topographies, materials exhibiting significantly different shear resistances compared to foundation materials, and/or complex pore pressure conditions. The 3D model allows spatial

variation of these effects to be incorporated into safety factor calculations (Arellano and Stark, 2001).

3. METHODS

3.1. Strength Parameters Definition

The geotechnical parameters were provided by the dam auditing company. According to the company, these parameters were calculated based on an SPT drilling campaign conducted along the main dam's embankment to evaluate the geological and geotechnical conditions of the site. Based on the obtained NSPT values, parameters were determined using correlations found in the literature, such as those by Shioi and Fukui (1982), Godoy (1983), Teixeira and Godoy (1996), Hatanaka and Uchida (1996), Chen (2004), and Schnaid (2009). The mean values within the range defined by the mean ± standard deviation were adopted.

Table 1 presents the soil parameters provided. In addition to the soil types, massif, and tailings, data are included for the material comprising the dam's filter (drainage mat and vertical drain).

Table 1 – Material Parameters

Material	Specific Weight (kN/m²)	Cohesion (kPa)	Friction Angle (º)
Massif	18	13	30
Tailing	17	0	30
Filter	20	0	33
Residual Soil	19	34	32
Saprolite	20	36	37

3.2. Seismic Activity Parameters

The analysis conducted was pseudo-static. Without specific studies for the dam area, the horizontal acceleration was based on the values defined by NBR 15421. The region where the structure is located falls within Zone 0, with seismic acceleration equivalent to 0.025 times the acceleration due to gravity (g). For vertical acceleration, half the horizontal acceleration was adopted, i.e., 0.0125g.

3.3. Stability Analysis

To perform slope stability analyses of the embankment, the topographic base and drainage system were modelled in a three-dimensional geometry using Slide3. Following this, three two-dimensional sections were generated and exported to Slide2 software for 2D analyses.

The modeling process began with the topographic base. Contour lines, provided by the mining company, were used to generate the terrain surface and the underlying soil volume. Boreholes were then allocated to enable the program to divide the embankment into soil layers, representing the layering observed in the field. Data from seven boreholes located along the main embankment were utilized, as shown in Figure 1.

Figure 1 - Boreholes location

Subsequently, the dam's plan projection and its cross-section were used to create lines following this projection, with ZZ-axis elevations (altitudes) defined to spatially position them at the same height as the real dam surface relative to the terrain. A similar method was used to model the vertical drain. From the surfaces, the volumes beneath them were generated, and material properties were assigned to each volume for instance, "Massif" properties for the embankment volume.

Finally, the piezometric levels obtained from the boreholes were not used, as the interpolation of these data yielded unsatisfactory results. The piezometric surface did not present a reasonable result, as the triangulation of water level points was anticipated to intersect the drainage system, regardless of the section. Due to the limited number of points, the program produced an irregular piezometric surface. Instead, a piezometric surface was arbitrarily defined based on terrain contour lines, aiming to represent a more realistic water flow scenario for the proposed study. This design scenario corresponds to a condition in which the drainage system functions as expected, with water entering the vertical drain and following the drainage mat to the downstream slope's toe.

Figure 2 provides perspectives of the dam embankment obtained from the 3D model built in Slide3.

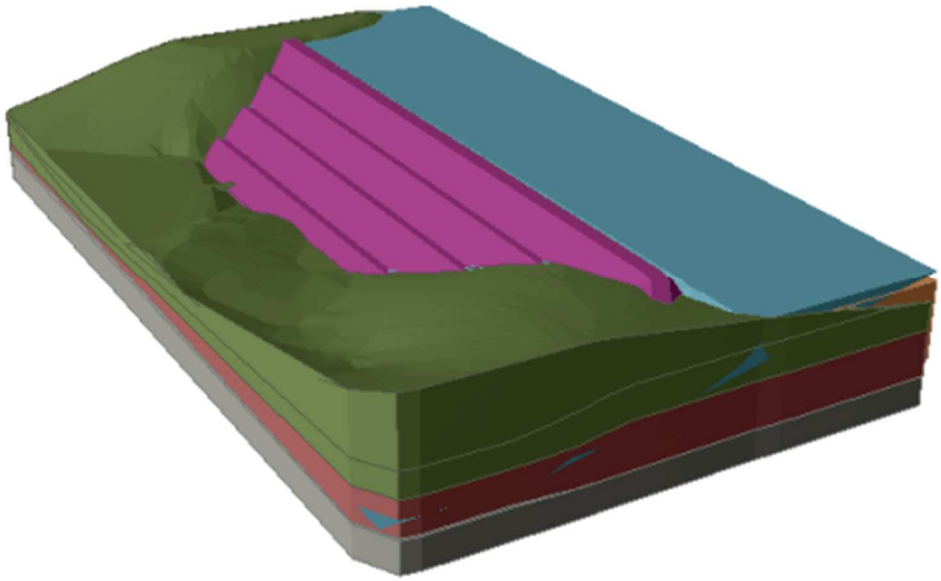


Figure 2 - 3D Model

After modeling, slope stability analyses were conducted. The failure surfaces search method chosen was the ellipsoidal one in the three-dimensional analysis. For the 2D analyses, the Cuckoo Search method was employed.

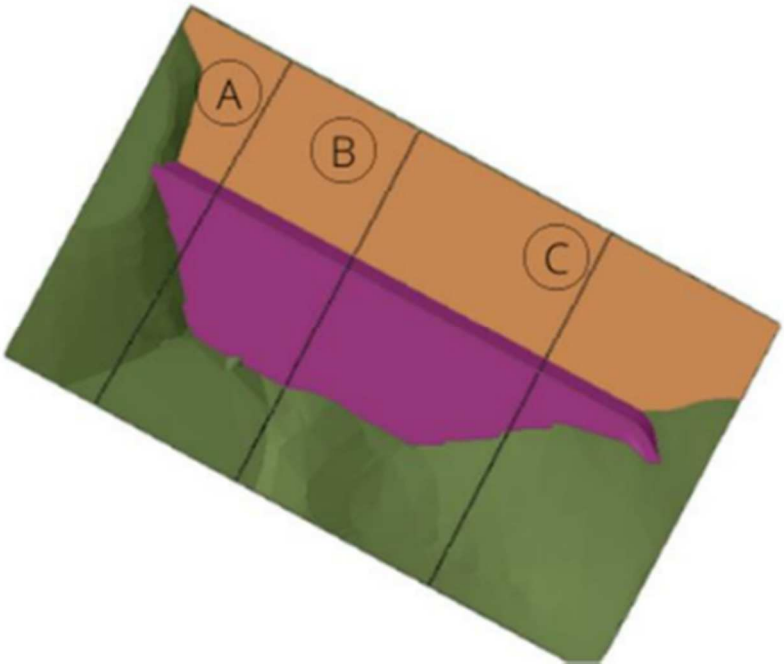


Figure 3 - Location of cross sections exported to Slide2

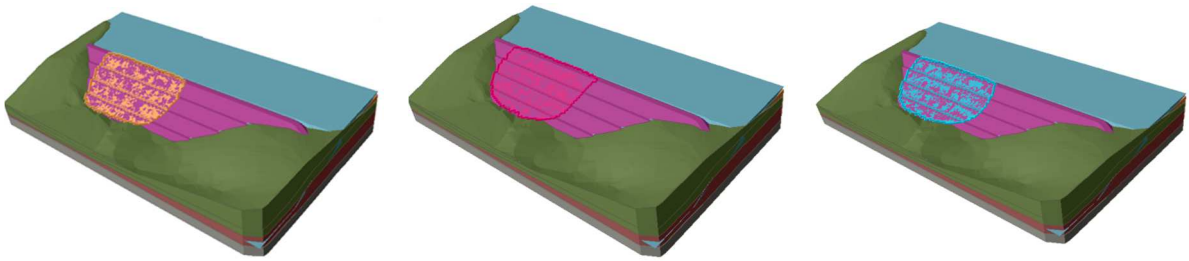
Figure 3 shows the location of the three sections, A (right abutment), B (crest) and C (left abutment), exported to Slide2.

4. RESULTS AND DISCUSSIONS

For this comparative study, results from three-dimensional and two-dimensional analyses were used, considering normal water levels, i.e., without water above the horizontal drainage material. In the 3D case, results with ellipsoidal failure surfaces were considered. **Table *2*** presents the factor of safety values for the 3D and 2D analyses. Also, Figure 4 shows the critical wedge in the 3D model by the three methods used: Bishop, Spencer and Morgenstern-Price, while Figure 6 shows the critical wedge in the 2D model for the 3 sections.

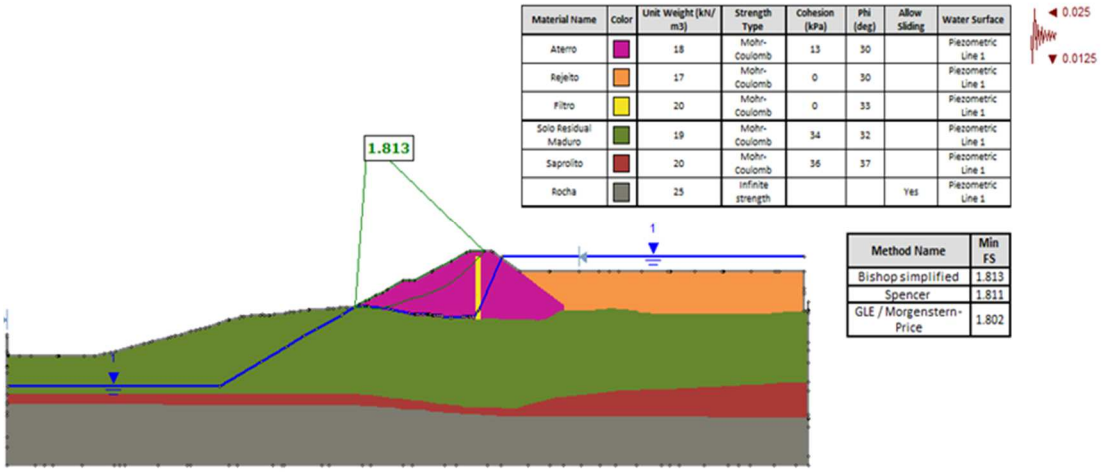
Table 2 - Factor of safety values for the 3D and 2D analyses

Analysis Type	Dam section	Bishop method	Spencer method	Morgenstern-Price method
3D analysis	-	1,683	1,708	1,704
2D analysis	A (right abutment)	1,813	1,811	1,802
	B (central)	1,658	1,647	1,636
	A (left abutment)	1,849	1,840	1,836



(a) (b) (c)

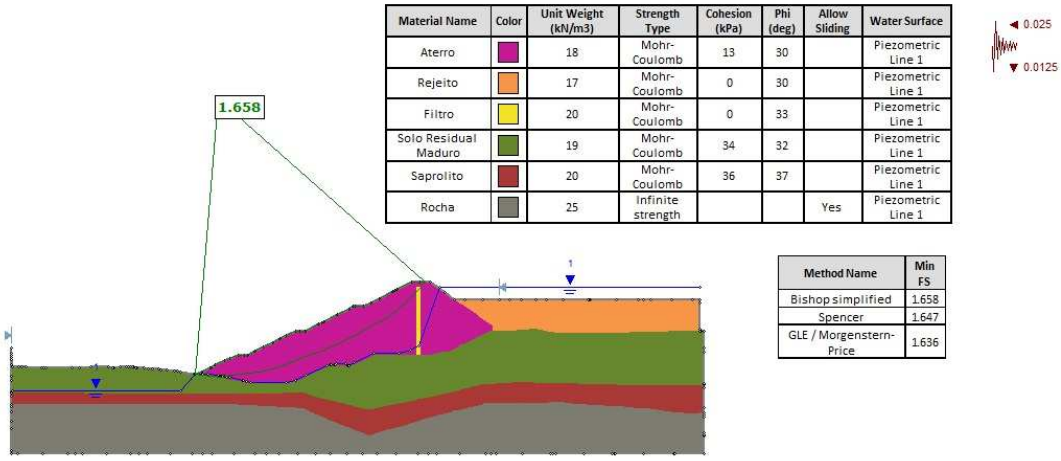
Figure 4 - Wedge of rupture in the 3D model by the (a) Spencer (b) Bishop and (c) Morgenstern-Price methods.



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	Water Surface
Aterro		18	Mohr-Coulomb	13	30		Piezometric Line 1
Rejeito		17	Mohr-Coulomb	0	30		Piezometric Line 1
Filtro		20	Mohr-Coulomb	0	33		Piezometric Line 1
Solo Residual Maduro		19	Mohr-Coulomb	34	32		Piezometric Line 1
Saprolito		20	Mohr-Coulomb	36	37		Piezometric Line 1
Rocha		25	Infinite strength			Yes	Piezometric Line 1

Method Name	Min FS
Bishop simplified	1.813
Spencer	1.811
GLE / Morgenstern-Price	1.802

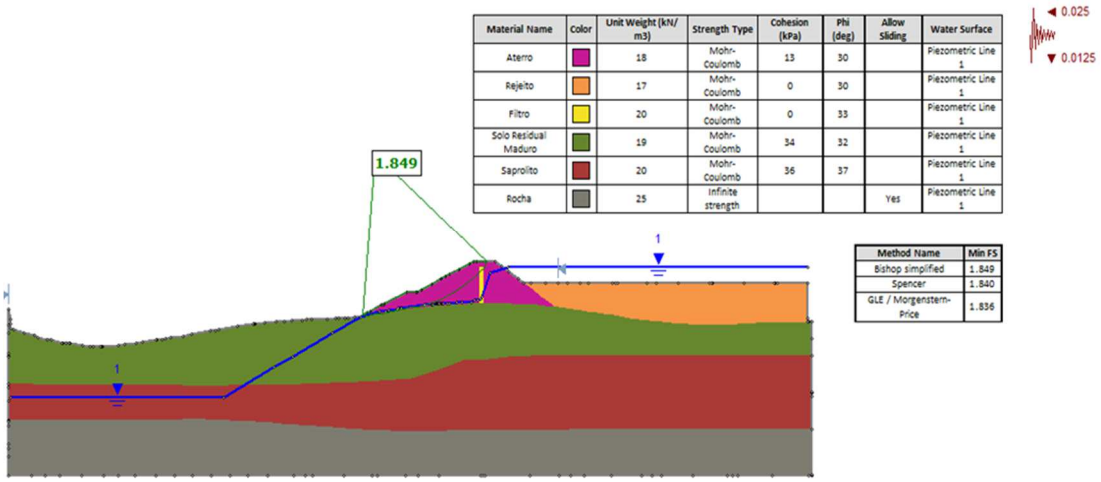
Figure 5 - Wedge of rupture in the 2D model on section A



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	Water Surface
Aterro		18	Mohr-Coulomb	13	30		Piezometric Line 1
Rejeito		17	Mohr-Coulomb	0	30		Piezometric Line 1
Filtro		20	Mohr-Coulomb	0	33		Piezometric Line 1
Solo Residual Maduro		19	Mohr-Coulomb	34	32		Piezometric Line 1
Saprolito		20	Mohr-Coulomb	36	37		Piezometric Line 1
Rocha		25	Infinite strength			Yes	Piezometric Line 1

Method Name	Min FS
Bishop simplified	1.658
Spencer	1.647
GLE / Morgenstern-Price	1.636

Figure 6 - Wedge of rupture in the 2D model on section B



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	Water Surface
Aterro		18	Mohr-Coulomb	13	30		Piezometric Line 1
Rejeito		17	Mohr-Coulomb	0	30		Piezometric Line 1
Filtro		20	Mohr-Coulomb	0	33		Piezometric Line 1
Solo Residual Maduro		19	Mohr-Coulomb	34	32		Piezometric Line 1
Saprolito		20	Mohr-Coulomb	36	37		Piezometric Line 1
Rocha		25	Infinite strength			Yes	Piezometric Line 1

Method Name	Min FS
Bishop simplified	1.849
Spencer	1.840
GLE / Morgenstern-Price	1.836

Figure 7 - Wedge of rupture in the 2D model on section C

It can be observed that the 3D analyses produced higher factors of safety compared to the 2D analyses, taking the most critical section from the 2D analyses (Section B) as a reference. Furthermore, the 3D analysis identified the most critical region as lying between the right abutment (Section A) and the central portion (Section B).

Thus, in terms of locating the most critical region of the embankment, the results of the 3D and 2D analyses were consistent. However, other 2D analyses (right and left abutments) produced higher factors of safety than the 3D analysis, with differences ranging from 5% to 8% between the factors of safety of the two models.

The results align with the behavior reported in the literature, as outlined in Section 2, exemplified by Duncan (1992). According to Duncan, factor of safety values from three-dimensional analyses are generally higher than those from two-dimensional analyses. Duncan attributes this to the fact that 2D analyses typically consider only the most critical cross-sections, neglecting shear resistance along the sides of the sliding mass, whereas 3D analyses encompass a larger region.

For the sections studied in this work, the observed differences can be explained by the fact that the sections for the 2D analyses were defined prior to the definition of the 3D critical wedge, based on areas of greater interest. Nevertheless, all results indicated that the most critical region of the embankment is located on its right side, encompassing the 2D sections A and B.

It may be mentioned that the 3D model is not entirely perfect, as variance exists in the geomechanical properties of the structure. Soil is a heterogeneous material, both in its extension and depth, and the model was generated based on 1D data from the boreholes. Thus, approximations and interpolations were made, which do not necessarily reflect the actual structure.

Furthermore, differences between 2D and 3D models are expected, with 3D models being more suitable for slopes with complex topography, materials with significantly different shear strengths compared to foundation materials, and/or complex pore pressure conditions, as discussed in Section 2.5. For the case studied here, the maximum difference of 8% between the analyses corroborates the characteristics of the structure, which does not exhibit complex topography or significant differences between materials, making the 2D analysis satisfactory for this case.

5. CONCLUSIONS

The results indicated that three-dimensional (3D) models produced more conservative FoS values than those obtained in abutment regions from two-dimensional (2D) analysis. This is reasonable since the abutments were not within the critical failure zone. In contrast, in the central region (affected by the three-dimensional failure wedge) FoS values were less conservative in 3D models than in 2D ones. This occurs because 3D analysis evaluates the critical region by considering the entire structure, reducing the likelihood of obtaining an artificially high FoS due to the selection of a less critical cross-section. Additionally, in our case, the 2D sections near the abutments were composed predominantly of foundation materials, which were more competent, while the volume of the dam itself, where the most critical parameters were present, was smaller. This explains why the dam's geometry and size near the abutments resulted in a lower failure risk in these areas.

Thus, the 3D analysis of the structure leads to higher FoS values compared to 2D analysis in cases where geometry plays a significant role, such as when there is substantial upstream slope curvature or a well-defined valley. In our study, we had a case of a dam smoothly embedded in a valley. While this effect was present, and we indeed observed variations in FoS values, our structure featured concavity but was not a highly complex case. As a result, the differences between 3D and 2D analyses were noticeable (around 8%) but still relatively moderate.

These results reinforce the enhanced ability of 3D models to better incorporate boundary conditions and provide a more realistic analysis, although they require greater effort in model development and data processing. On the other hand, in cases where the topography is relatively simple, hydraulic flows are not complex, and material properties are more homogeneous and isotropic, 2D analysis can still provide satisfactory results. However, when dealing with more complex topographies, intricate flow conditions, or anisotropic materials, 3D analysis becomes essential for accurately capturing the behavior of the structure.

REFERENCES

[1] ABNT – Associação brasileira de normas técnicas. NBR 15421:2006 – Projetos de estruturas resistentes a sismos – Procedimento. Rio de Janeiro - RJ, 2006a.

[2] Arellano, D.; Stark, T., Importance of tree-dimensional slope stability analyses in pratice. In: Slope stability 2000: proceedings of sessions of geo-denver 2000, August 2000, Denver-Colorado: The Geo-institute of the american society of civil engineering, p. 18-32.

[3] Ávila, J. P.; Sawaya, M.; Sayão, A. S. F.; Ferreira, L. A. (2021). Segurança De Barragens De Rejeitos No Brasil: Avaliação Dos Acidentes Recentes. Impactum Journals, Coimbra, Geotecnia, n.152, p. 435-464, jul. 2021.

[4] Bishop, A. W.; Morgenstern, Norbert. Stability Coefficients for Earth Slopes. Géotechnique, [S.L.], v. 10, n. 4, p. 129-153, dez. 1960. Thomas Telford Ltd. http://dx.doi.org/10.1680/geot.1960.10.4.129.

[5] Brasil. Lei n. 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens destinadas à acumulação de água para quaisquer usos, à disposição final ou temporária de rejeitos e à acumulação de resíduos industriais.

[6] Bretas, T. C. (2020) Retroanálise Probabilística Tridimensional Por Equilíbrio-Limite De Rupturas De Talude Em Belo Horizonte/Mg. 175 f. Dissertação (Mestrado) - Curso de Programa de Pós-Graduação em Engenharia Civil, Engenharia Civil, Universidade Federal de Ouro Preto, Ouro Preto, 2020.

[7] Bromhead, E. N. & Martin, P. L. Three-dimensional limit equilibrium analysis of the Taren landslide. In Advances in Geotechnical Engineering (Skempton Conference), Vol. 2, 2004. pp. 789–802. London: Thomas Telford.

[8] Cardozo, F. A. C. (2024) Comparação de Métodos Construtivos Para Barragens de Rejeito: Análise dos alteamentos à montante e à jusante. **Geotecnologias**: análises, técnicas e aplicações em pesquisa - Volume 2, Porto Alegre, v. 2, p. 44-59. Editora Científica Digital. http://dx.doi.org/10.37885/240616856.

[9] Cardozo, F. A. C. et al. (2016). Métodos Construtivos de Barragens de Rejeito: Crítica Ao Método de Montante. In: Congresso Brasileiro de Geologia, 48, 2016, Porto Alegre - RS. Anais...São Paulo - SP: Sociedade Brasileira de Geologia. < http://sbgeo.org.br >. Data de acesso: 23/10/2016.

[10] Duncan, J.M. (1992), "State-of-the-Art: Static Stability and Deformation Analysis," Proceedings of Stability and Performance of Slopes and Embankments-ll, ASCE, Vol. 1, pp. 222-266.

[11] Kelesoglu, M. K. The evaluation of three-dimensional effects on slope stability by the strength reduction method. KSCE Journal of Civil Engineering, v. 20, n. 1, p. 229-242, 2015.

[12] Morgenstern, N. R.; Price, V. E. The Analysis of the Stability of General Slip Surfaces. Géotechnique, [S.L.], v. 15, n. 1, p. 79-93, mar. 1965. Thomas Telford Ltd. http://dx.doi.org/10.1680/geot.1965.15.1.79.

[13] Qiu, C.; Esaki, T.; Xie, M.; Mitani, Y.; Wang, C. Spatio-temporal estimation of shallow landslide hazard triggered by rainfall using a three-dimensional model. Environmental Geology, v. 52, n. 8, p. 1569-1579, 2006

[14] Spencer, E. A Method of analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces. Géotechnique, [S.L.], v. 17, n. 1, p. 11-26, mar. 1967. Thomas Telford Ltd. http://dx.doi.org/10.1680/geot.1967.17.1.11.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EVALUATION OF SLOPE STABILITY IN TAILINGS DAM CONSTRUCTION AND DECOMMISSIONING STAGES USING 3D LIMIT EQUILIBRIUM AND STRENGTH REDUCTION FACTOR: A COMPARATIVE ANALYSIS

Jaime Barbosa[1]
BVP Geotecnia e Hidrotecnia

Marcelle Dantas[2]
BVP Geotecnia e Hidrotecnia

Wesley Carvalho[3]
Vale

Nilthson Noreña[4]
Vale

ABSTRACT

In recent years, the safety of tailings dams has become a critical issue within the mining industry, driven by several high-profile catastrophic failures that have caused substantial environmental damage and loss of life. As the industry continues to address these challenges, there is an increasing demand for more effective evaluation and management techniques to prevent such disasters. Two major methodologies have been evaluated and compared in the characterization of the dam: 3D Limit Equilibrium Method (LEM) and Strength Reduction Factor (SRF). Slope limit equilibrium analysis has been widely used in the geotechnical industry for many years, and today, stress and strain studies further enhance the reliability of these structures.

The 3D Limit Equilibrium (LE) methodology applies equilibrium principles to analyze the stability of tailings dams by discretizing the dam into a series of slices and evaluating forces and moments. This method is valuable for its simplicity and its direct calculation of safety factors. It is particularly effective for routine stability assessments of relatively simple geometries, but it may oversimplify the complex interactions within heterogeneous materials.

Strength Reduction Factor (SRF) Methodology employs Finite Element Analysis (FEA) to simulate the progressive failure of a tailings dam by systematically reducing the shear strength parameters until collapse is predicted. This approach provides a more detailed and realistic representation of failure mechanisms, capturing the influence of complex soil properties and boundary conditions. However, SRF is computationally intensive and requires sophisticated modeling and interpretation.

The main purpose of this study is to compare the 3D Limit Equilibrium (LE) and SRF methods in evaluating the slope adjacent to the dam during both the construction and decommissioning phases. The construction phase is modeled step-by-step, providing a reasonable approximation in the absence of historical construction data. The study focuses on defining the current state of the dam, with particular emphasis on the initial effective stresses. Furthermore, the study also addressed the process of removing the dam's tailings, which involves a planned unloading over a 6-year period, with removal rates of approximately 12.7 meters per year. This phase is critical as it may present new challenges in the slope behavior and the stresses involved, requiring a detailed evaluation to ensure stability during decommissioning.

Keywords: 3D Slope Stability Analysis, Limit Equilibrium Method Strength Reduction Factor, Finite Element Method, Tailings Dams.

[1]Jaime.barbosa@bvp.eng.br, Brazil
[2]marcelle.dantas@bvp.eng.br, Brazil
[3]wesley.soares.carvalho@vale.com, Brazil
[4]nilthson.norena@vale.com, Brazil.

1. INTRODUCTION

In geotechnical studies, determining the factor of safety of a slope against failure is very important, especially in the processes of decommissioning a dam. Both the stability of the slope and the stability of the tailings during removal processes are evaluated. Various slope failures around the world cause significant economic and environmental losses and can lead to the loss of human lives in extreme circumstances. Remediation or stabilization designs for the slope are essential to avoid such failures during tailings removal. Limit equilibrium analysis of slopes has been common in the geotechnical industry for many years. The 2D approach is conservative because 3D geometric influences are not considered in the 2D analysis. The conservatism associated with a 2D analysis has been seen as an additional safety factor [1, 3, 5, 7, 8, 9, 12, 14, 15, 16, 17, 20, 21, 22]. Recent software tools allow for enhanced 3D slope analysis through limit equilibrium techniques and the SRF parameter reduction method [19].

The limit equilibrium method is easy to use and computationally fast to execute and interpret. The main limitations associated with the limit equilibrium method include: a) most slope failures are three-dimensional volumes, b) finding the critical sliding surface remains difficult, c) it is challenging to handle more advanced and real-world stress conditions, such as Ko conditions, in a numerical model and d) the behavior of each material during the decommissioning process is time-dependent and partially drained.

3D slope failure analysis is required when the failure mass and the slope geometry change laterally, when soil/rock properties are anisotropic and heterogeneous, and when these local unloading or loading conditions are applied to the slope [2, 18]. This paper presents two different methodologies available for performing 3D slope stability checks during the construction and focus main in the decommissioning stages of a tailings dam: the Finite Element Method and the Limit Equilibrium Method [2, 4, 13].

1.1. Limit equilibrium methods (LEM) for 3D slope Stability analysis

Three-dimensional (3D) Limit Equilibrium Method (LEM) slope stability analysis is a natural extension of the two-dimensional (2D) LEM, widely employed for complex geotechnical stability problems. Research in this field has been extensive, with significant contributions [2, 4, 13]. The transition from 2D to 3D methodologies builds upon the well-established slicing techniques used in 2D analysis, evolving into 3D column-based models. This adaptation maintains key advantages, including the capacity to model intricate geometries, heterogeneous soil profiles, variable pore-water pressure conditions, and reinforcement systems.

A fundamental metric in slope stability assessment is the Factor of Safety (FOS), defined as the ratio of resisting to mobilizing forces. For a slope to be considered stable, FOS must exceed 1.0.

In practice, 3D slope stability analysis subdivides the failure mass into discrete columns to evaluate forces acting on individual elements. This methodology accounts for variations in geometry and loading profiles, offering a robust and detailed representation of slope stability, particularly in cases where 2D formulations are inadequate. The advancement of 3D LEMs continues to enhance the precision and applicability of geotechnical stability analyses [2].

1.2. 3D Finite Element Method (FEM) – SRF for 3D slope Stability analysis

The Finite Element Method (FEM) is a widely used approach in geotechnical engineering for modeling domains with known material properties under external loads to determine displacements, stress fields, and strain distributions. In slope stability analysis, FEM-based solutions have been extensively explored, with significant advancements made by researchers [6, 10]. Compared to the traditional Limit Equilibrium Method (LEM), FEM-based slope stability analysis offers several advantages, including the ability to model naturally occurring instability mechanisms without assuming predefined failure surface shapes (e.g., spheroid, ellipsoid, or log-spiral).

A key FEM-based approach for slope stability is the Strength Reduction Factor (SRF) or Shear Strength Reduction (SSR) method. This method gradually reduces the soil's shear strength parameters - cohesion (c`) and internal friction angle (φ`)—until the slope reaches failure conditions. The Factor of Safety (FOS) in SRF is defined as the ratio of the original shear strength to the reduced shear strength parameters at the point of failure, often expressed mathematically as:

$$c'_f = \frac{c'}{SRF} \quad and \quad \varphi'_f = \tan^{-1}\left(\frac{\varphi'}{SRF}\right) \tag{1}$$

In practice, the Strength Reduction Factor (SRF) is systematically increased during FEM simulations, weakening the soil properties until the model fails to converge to equilibrium conditions. This non-convergence is interpreted as the onset of slope failure, indicating the FOS. Unlike LEM, which requires assumptions about the failure surface shape, the FEM-SSR method allows for a more natural depiction of slope instability through stress-induced failure mechanisms. This results in additional insights, such as deformation patterns, stress distributions, and strain fields, which are not available through LEM formulations [11].

2. Model Implementation and Case Study Analysis

2.1. Geotechnical Parameters and Material Properties for 3D Modeling

In the absence of tests to determine the deformability parameters of certain materials, the deformability moduli were defined using the correlations shown in the equations below, and the results are presented in Tables 1 and 2.

$$E = \beta . N_{70} \quad (2)$$

$$\beta = k . \alpha . Pa\ (MPa) \quad (3)$$

Table 1 – Geotechnical Parameters for k and α

Material	k (Schnaid and Odebrecht, 2014)	α (Trofimenkov, 1974)	β (MPa)
Sand	6,0	3,0	1,82
Silty sand	5,3	3,6	1,93
Silty clayey sand	5,3	4,0	2,15
Clayey silty sand	5,3	4,4	2,36
Clayey sand	5,3	4,6	2,47
Sandy silt	4,8	4,4	2,14
Sandy clayey silt	3,8	4,6	1,77
Clayey silt	3,0	5,6	1,70
Sandy clay	4,8	5,8	2,82
Sandy silty clay	3,8	5,6	2,16
Silty sandy clay	3,8	6,0	2,31
Silty clay	2,5	6,4	1,62
Clay	2,5	7,0	1,77

Table 2 – Geotechnical Parameters Adopted in the 3D Modeling – Mohr-Coulomb Model

Material	γ (kN/m³)	c (kPa)	φ (°)	E (MPa)	v	k_v (m/s)	k_h (m/s)
DOMINIO III	23	0	32	16	0.2	5.60×10^{-5}	5.60×10^{-5}
(TA) Terraço Colúvio Aluvionar	21	1	24	25	0.2	3.00×10^{-6}	3.00×10^{-6}
(CL) Colúvio laterítico	21	10.7	28.2	40	0.3	6.82×10^{-7}	6.82×10^{-7}
CM (Conglomerado Laterizado)	22	14.7	41.9	80	0.3	3.76×10^{-6}	3.76×10^{-6}

CG (Canga ou laterita)	20	27	36.2	67	0.3	6.10×10^{-6}	6.10×10^{-6}
SP-XT (Saprolito Xisto)	19	39.1	39.7	20	0.3	1.00×10^{-7}	1.00×10^{-7}
SS-MB (Solo saprolítico de rocha metabásica)	19	13	31.8	40	0.3	1.00×10^{-6}	1.00×10^{-6}
SP-MB (Saprolito de rocha metabásica)	21	37.1	38.7	65	0.3	9.00×10^{-7}	9.00×10^{-5}
SS-FL (Solo saprolítico de filito)	20	12.1	27.9	50	0.3	7.00×10^{-5}	7.00×10^{-5}
SP-FL (Saprolito Filito)	20	34	39	70	0.3	1.00×10^{-7}	1.00×10^{-7}
SS-IF (Solo saprolítico de itabirito indef.)	22	24.2	34	40	0.3	9.00×10^{-6}	9.00×10^{-6}
SP-IF (Saprolito de itabirito indef.)	23.8	41.1	39.7	80	0.3	9.00×10^{-5}	9.00×10^{-5}
SS-QZ (Solo saprolítico de quartzito)	22	7	33	45	0.3	1.00×10^{-2}	1.00×10^{-2}
SP-QZ (Saprolito Quartzito)	22	21	36	30	0.3	9.00×10^{-7}	9.00×10^{-7}
SS-IM (Saprolítico de Itabirito Manganesífero)	20	26	36.1	28	0.3	7.53×10^{-5}	7.53×10^{-5}
SP-IM (Saprolito de Itabirito Manganesífero)	18	28	38	50	0.3	1.00×10^{-6}	1.00×10^{-6}
Metadolomito – Rocha	28	30	40	120	0.3	—	—

Fig. 1b shows the longitudinal section cut of the 3D model, along with the materials of the section under analysis, as indicated in Table 2.

It is worth noting that, during all removal stages, the water level followed the decommissioning, remaining below the surface. These studies consider the most critical situation in the use of the water level (Fig. 1a).

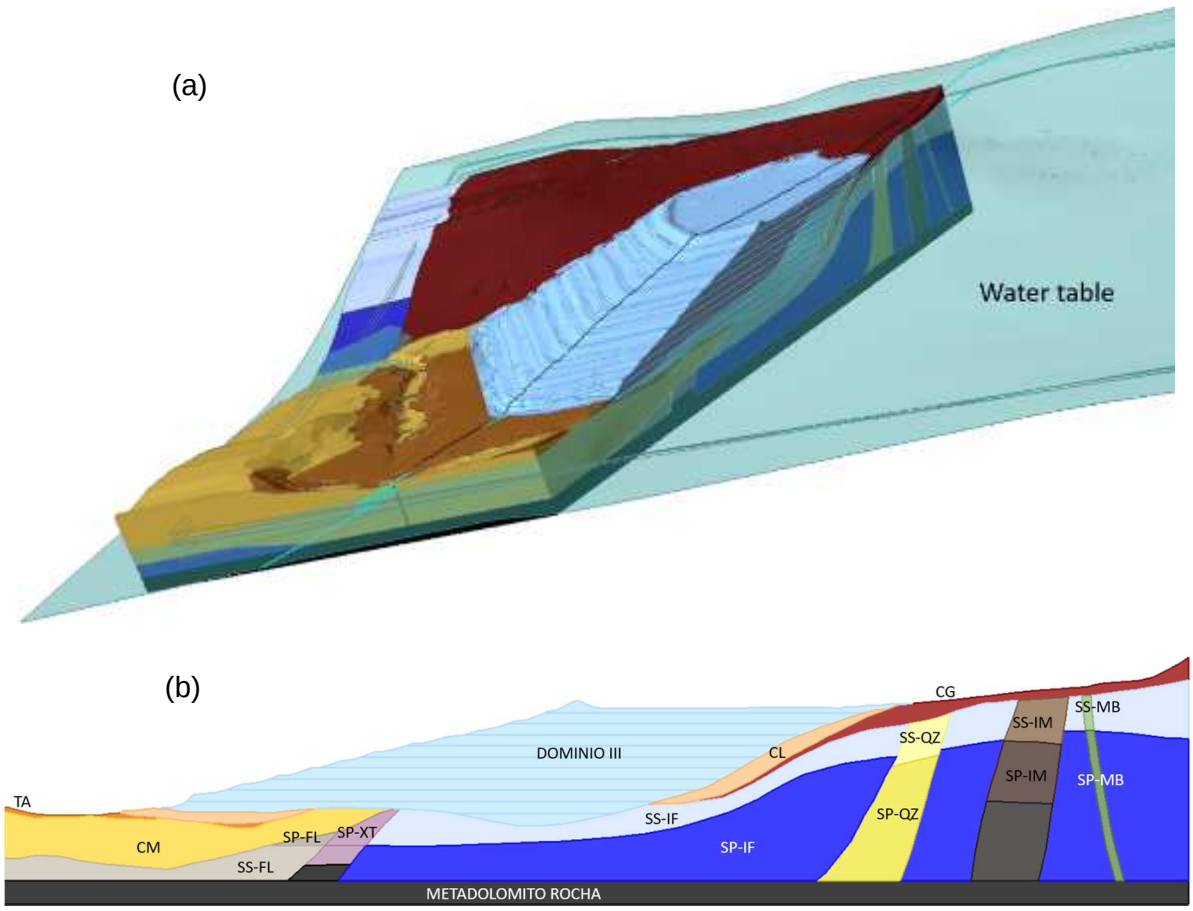


Fig. 1 – 3D Modeling Dam Construction: (a) Materials and Water table, (b) cross section with materials

2.2. 3D Numerical Modeling: Construction and Decommissioning Process Calibration

The 3D model was initially created in Leapfrog software and then imported into the RS3 program. To obtain the current stresses in the slope and the dam, a numerical simulation of the dam construction process over 9 years was performed.

Given the lack of detailed information regarding the evolution of the construction stages, 11 raising stages, each lasting one and a half years, were considered, which corresponds to approximately 12.7 meters per one year.

Table 3 presents the evolution of the construction process included in the numerical analyses. It is important to note that this process is similar to the one used for the decommissioning stages.

Table 3 – Simulation of the Construction and decommissioning Process: Stages Included

Stage Construction	Time (days) Construction	Stage Decommissioning	Time (days) Decommissioning
1st	547,5	10th	365
3rd	1095,0	8th	730

5th	1642,5	6th	1095
7th	2190,0	4th	1460
9th	2737,5	2nd	1825
11th	3285+8760	Decommissioning	2190

After the completion of the construction stages, a period of 24 years was introduced to represent the current state (state 11th), during which the excess pore pressure generated during construction is expected to fully dissipate.

The analyses assumed the piezometric lines defined by calibration in the limit equilibrium stability analyses. During the calibration, the material permeabilities were adjusted based on the existing instrumentation, always considering the limit permeability values obtained from tests.

To assess the stresses and deformations predicted in the 3D model, the model considered the final configuration of the structure. The three-dimensional finite element mesh, shown in Fig. 3, consists of 1,779,835 elements. Fig. 2 presents different views of the model and materials. In the development of the model, the total head were applied to establish the flow conditions. Fig. 4 and Fig. 5 show the distribution of the total head and the pressure head at the end of the dam construction, respectively.

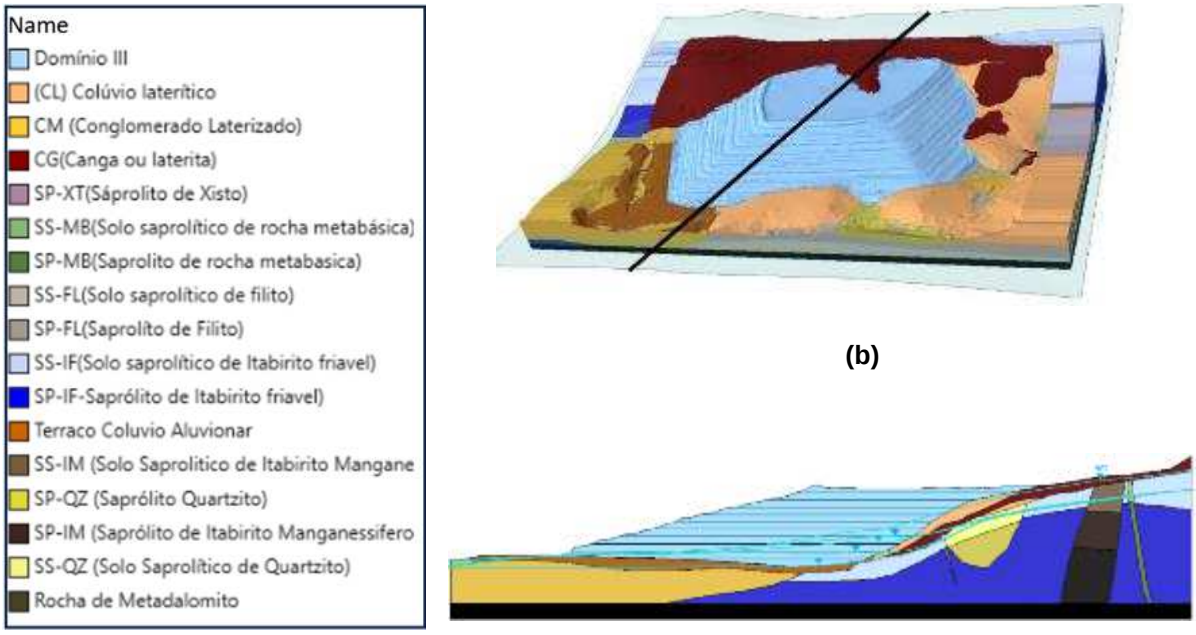


Fig. 2 – 3D Model Views in RS3: (a) Materials, (b) 3D Perspective (c) Elevation View (YZ)

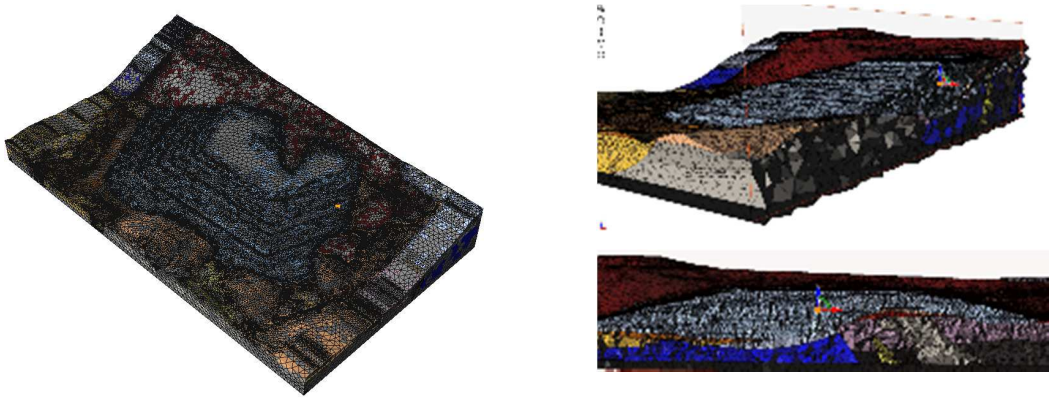
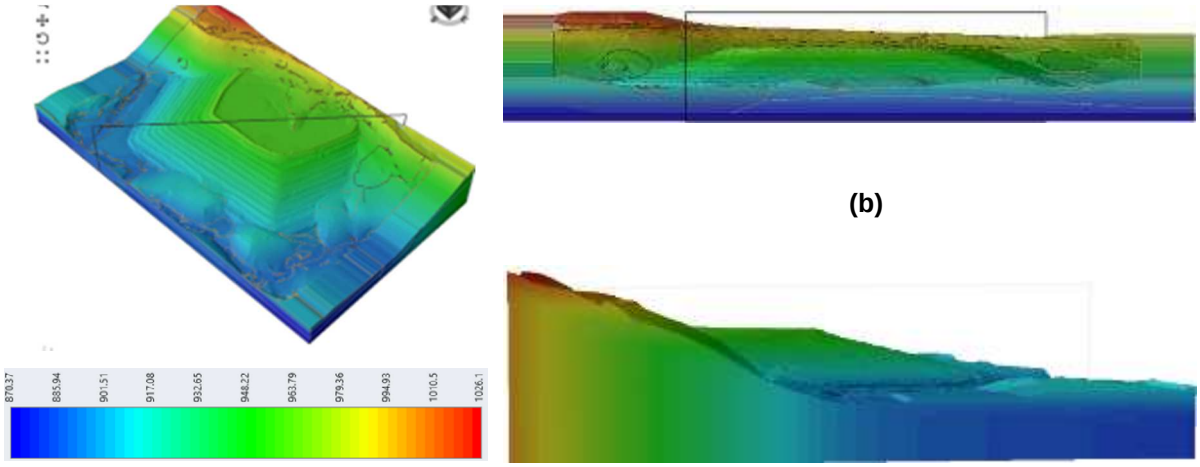
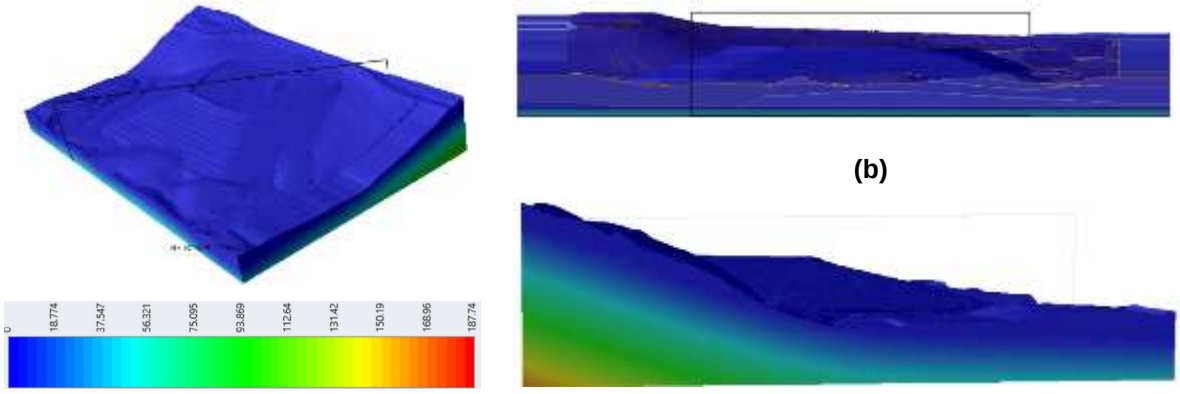


Fig. 3 – Finite Element Mesh Generated in RS3



(a) **(b)** **(c)**

Fig. 4 – Distribution total head in RS3: (a) 3D Perspective, (b) Plan View (XZ), (c) Elevation View (YZ)



(a) **(b)** **(c)**

Fig. 5 – Distribution pressure head in RS3: (a) 3D Perspective, (b) Plan View (XZ), (c) Elevation View (YZ)

Fig. 6 illustrates the 11 construction stages of the dam, detailing its progression to its current state.

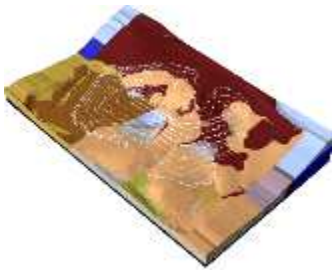
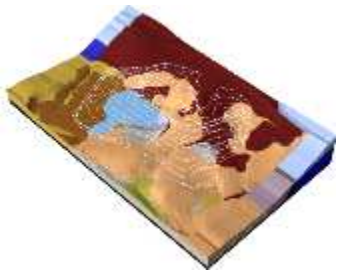
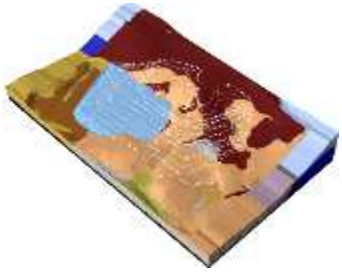
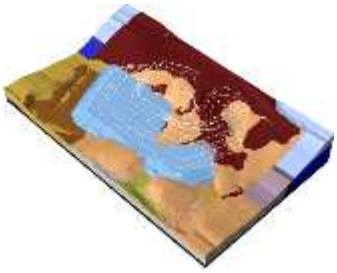
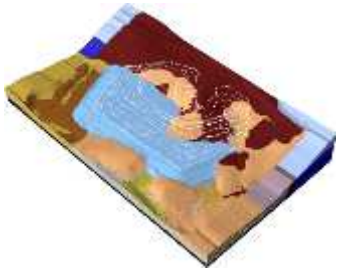
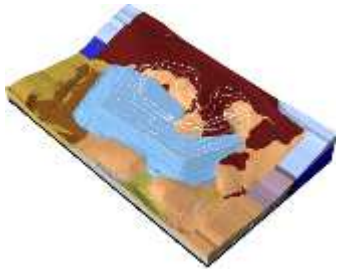
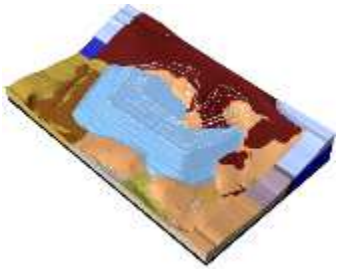
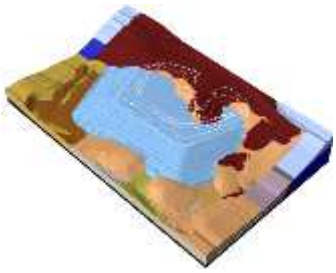
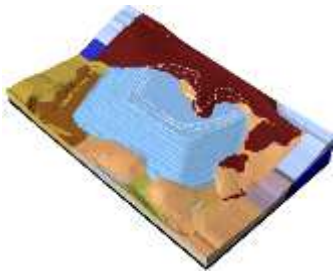
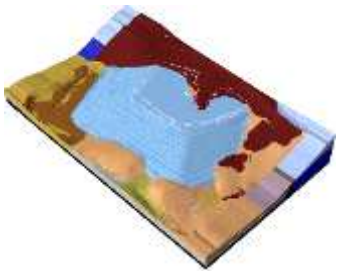
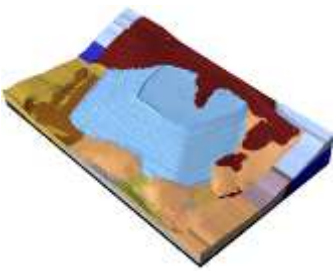
		
(a) INITIAL STAGE	**(b) 1st STAGE**	**(c) 2nd STAGE**
		
(d) 3rd STAGE	**(e) 4th STAGE**	**(f) 5th STAGE**
		
(g) 6th STAGE	**(h) 7th STAGE**	**(i) 8th STAGE**
		
(j) 9th STAGE	**(k) 10th STAGE**	**(l) 11th STAGE**

Fig. 6 – Construction Stages Analyzed with RS3

The decommissioning stages followed the 10 stages presented in Table 3, which were the same as those considered in the construction process due to the absence of detailed construction records for the dam.

Fig. 7 illustrates the decommissioning stages considered in the analyses.

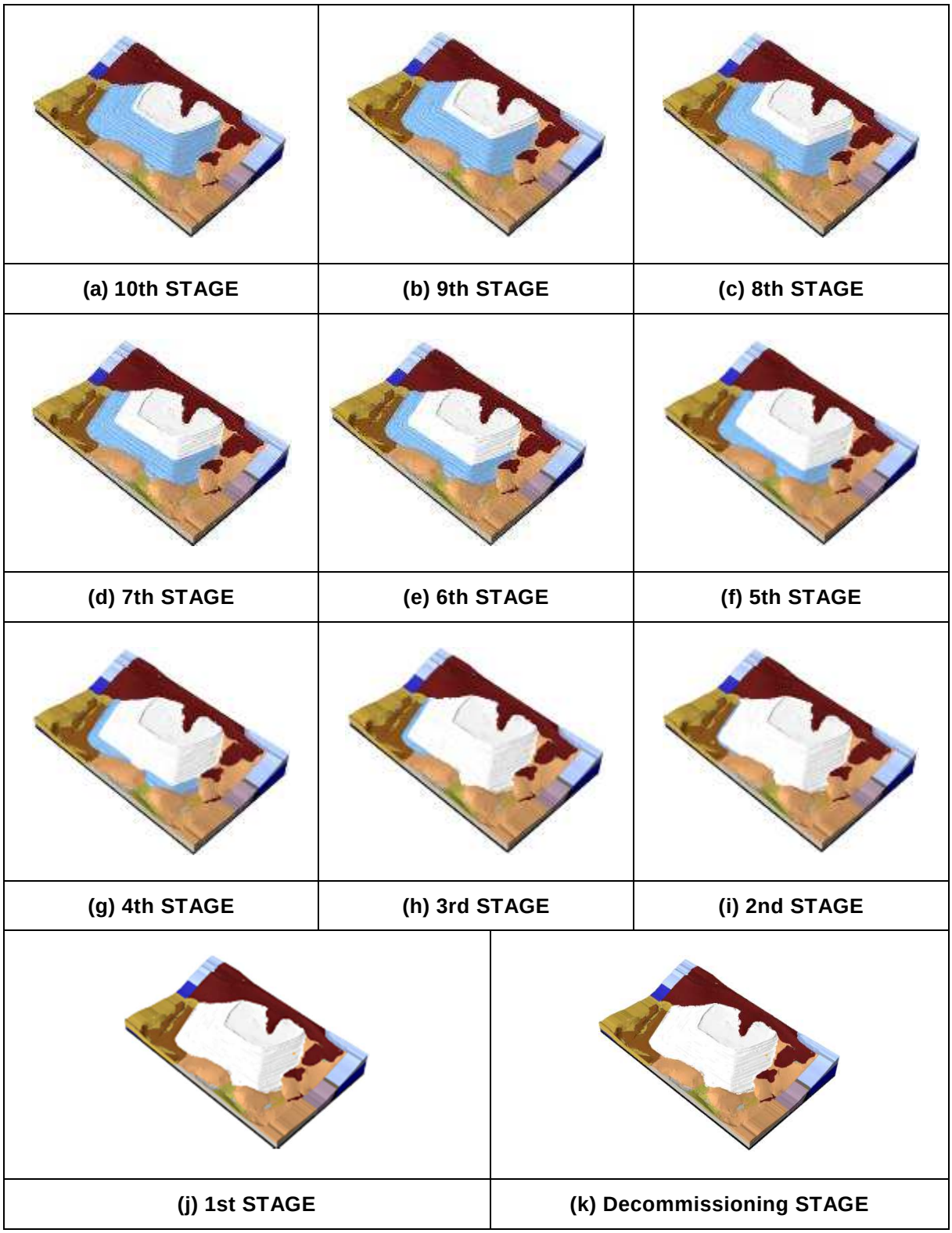


Fig. 7 – Decommissioning Stages Analyzed with RS3

Fig. 8 to 10 show the effective horizontal stresses (σ'xx and σ'yy) and vertical stresses (σ'zz) predicted at the end of the dam construction. The obtained values are within the expected range, considering the height of the dam.

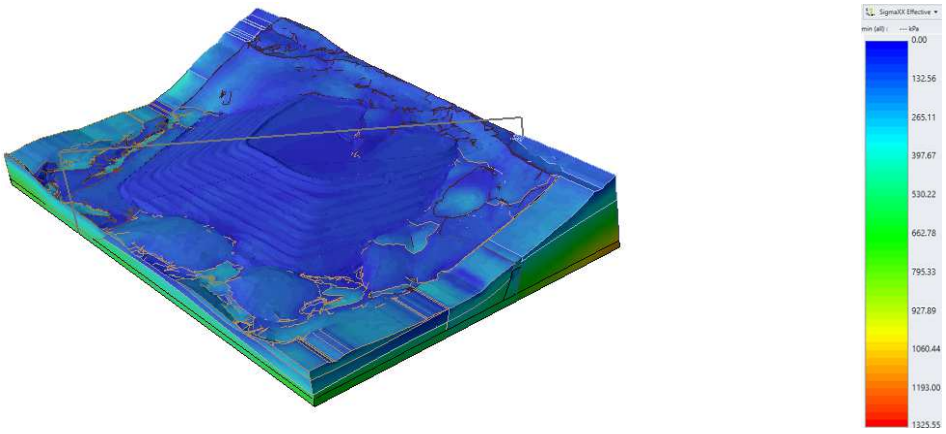


Fig. 8 – Analysis of Effective Horizontal Stresses σ'_{xx} using RS3

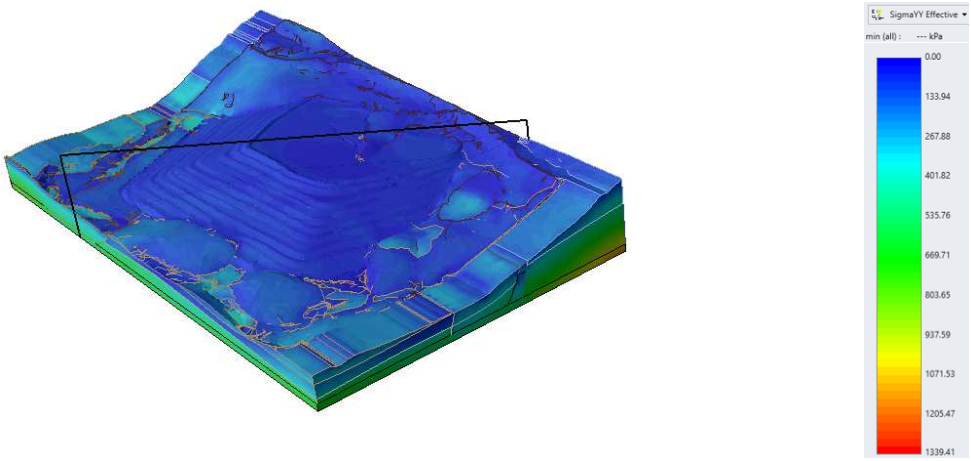


Fig. 9 – Analysis of Effective Horizontal Stresses σ'_{yy} using RS3

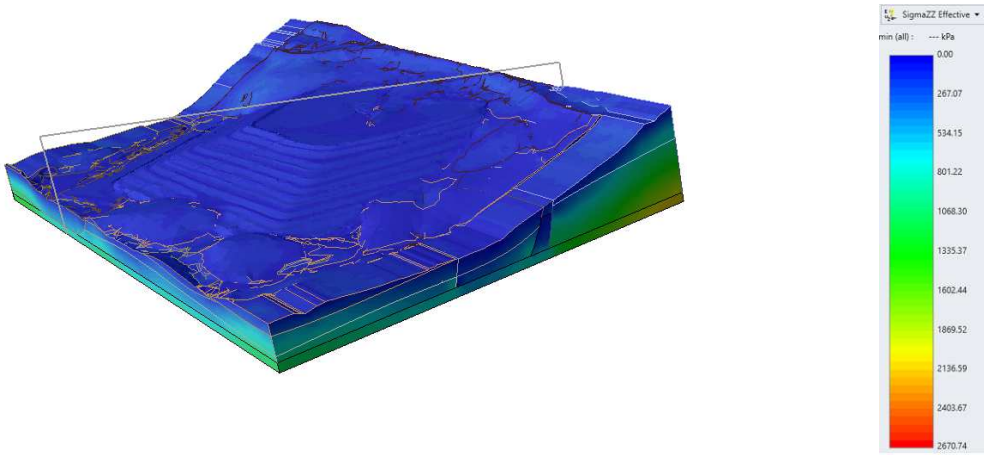
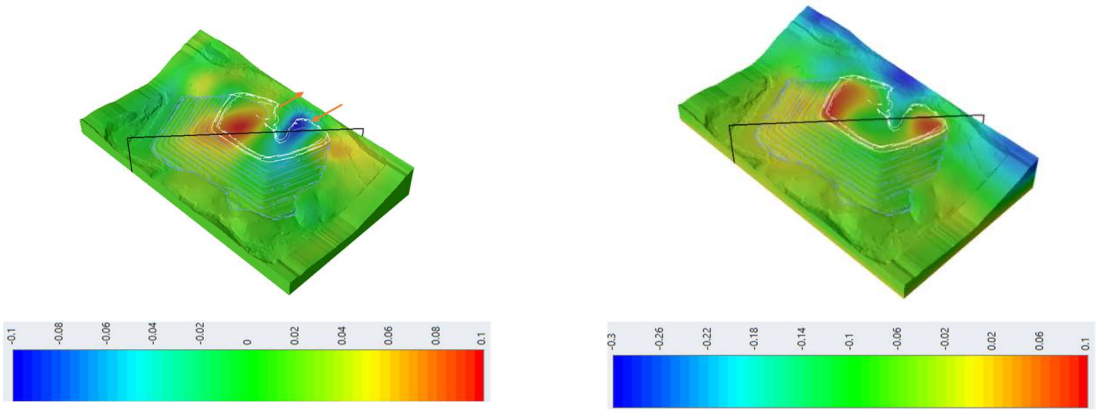


Fig. 10 – Analysis of Effective Vertical Stresses σ'_{zz} using RS3

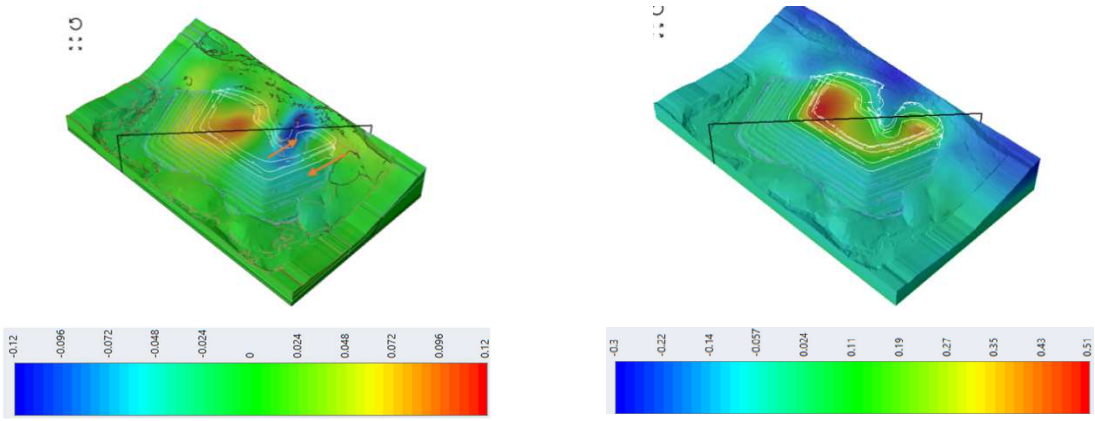
3. RESULTS: THREE-DIMENSIONAL MODELING ANALYSIS

Fig. 11 to 16 present the predicted horizontal and vertical displacements for the different decommissioning stages. The results indicate that, as excavation progresses, horizontal displacements develop on the slope, directed towards the dam, while displacements on the dam occur in the opposite direction, as highlighted by the arrows in the figures. In the initial stage, maximum horizontal displacements of approximately 10.0 cm are predicted for the slope (Fig. 11), increasing to 22 cm by the end of the decommissioning process. Additionally, the vertical displacements are observed throughout the decommissioning stages. Notably, positive displacements (upward) are consistently observed in the dam, driven by stress relief caused by the excavations.



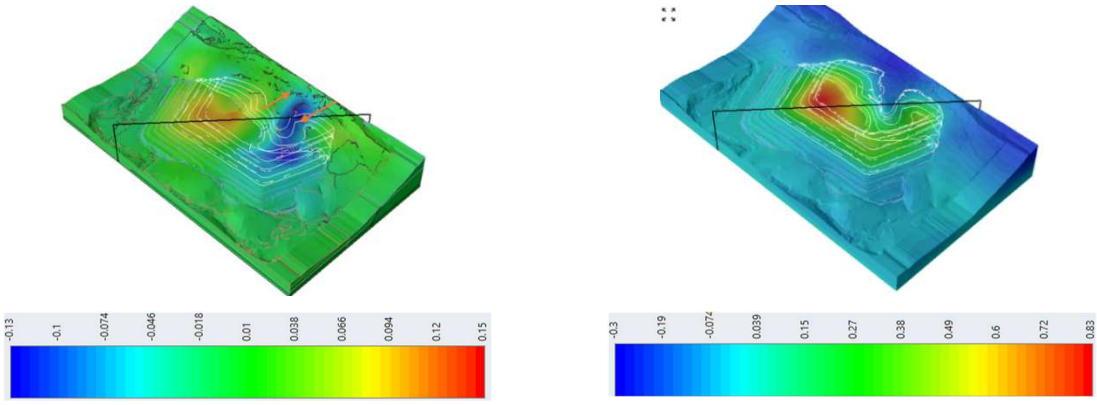
(a) **(b)**

Fig. 11 – Predicted Horizontal [m] (a) and Vertical [m] (b) Displacements in the 10th Decommissioning Stage with RS3



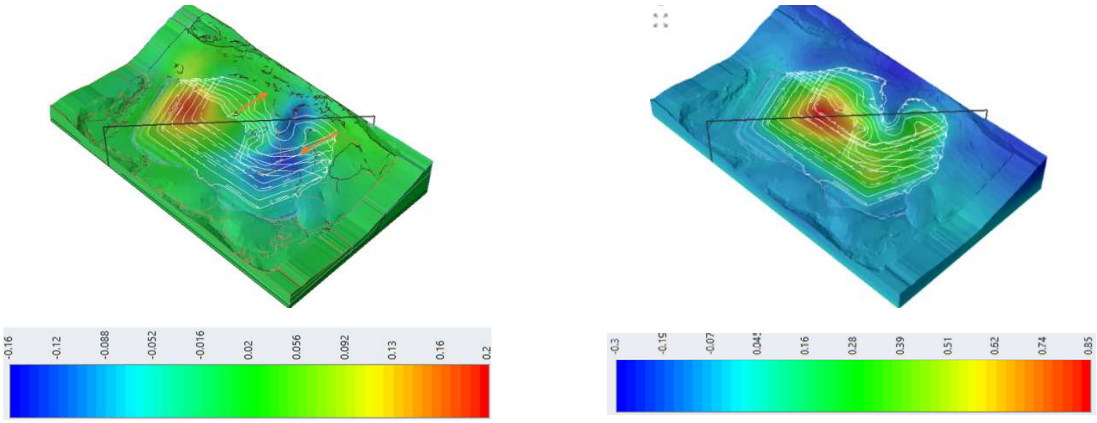
(a) **(b)**

Fig. 12 – Predicted Horizontal [m] (a) and Vertical [m] (b) Displacements in 8th Decommissioning Stage with RS3



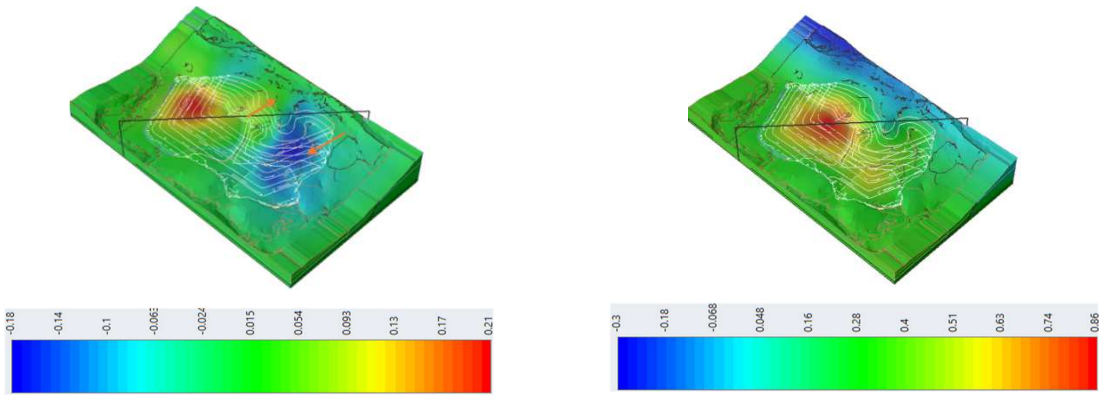
(a) **(b)**

Fig. 13 – Predicted Horizontal [m] (a) and Vertical [m] (b) Displacements in 6th Decommissioning Stage with RS3



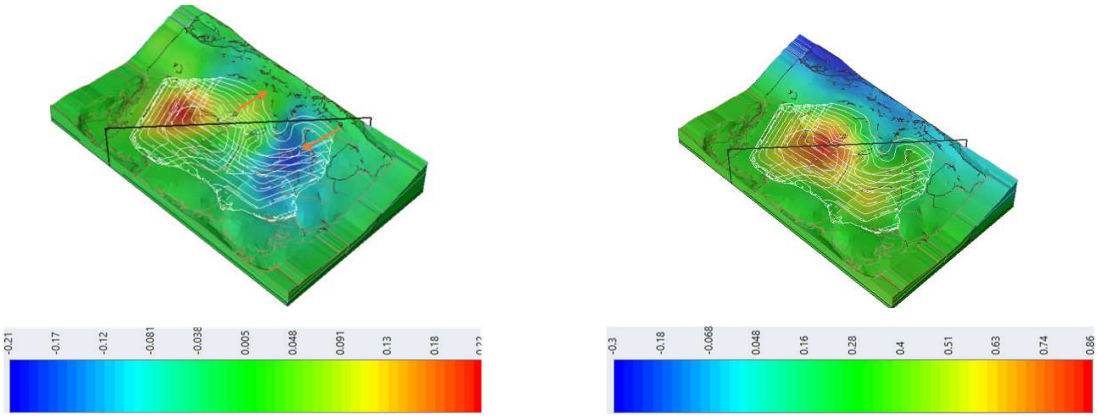
(a) **(b)**

Fig. 14 – Predicted Horizontal [m] (a) and Vertical [m] (b) Displacements in the 4th Decommissioning Stage with RS3



(a) **(b)**

Fig. 15 – Predicted Horizontal [m] (a) and Vertical [m] (b) Displacements in the 2nd Decommissioning Stage with RS3



(a) **(b)**

Fig. 16 – Predicted Horizontal [m] (a) and Vertical [m] (b) displacements: End of the Decommissioning Process with RS3

The RS3 program determines the safety factor using the SRF - Strength Reduction Factor [19]. This method involves progressively reducing the strength parameters by dividing them by increasing reduction factors until failure occurs. The safety factor is then considered as the lowest reduction factor SRF that causes failure. In the context of numerical simulations using the finite element method, the lack of convergence of the problem is considered an indication of failure. This method has the advantage of not requiring assumptions regarding the shape of the failure surface or the stress field. In the case of the Mohr-Coulomb model, the reduced parameters are the effective cohesion (c') and the tangent of the effective friction angle (tan φ'). The evaluation of the existence of a failure mechanism is performed by observing the field of maximum shear deformations.

The admissible Safety Factors are meant to address uncertainties related to the stages of construction and decommissioning, as outlined by technical standards. For natural slopes and/or slopes resulting from cuts and embankments, the Brazilian Association of Technical Standards (ABNT) establishes guidelines in ABNT NBR 11682 (ABNT, 2009) for the recommended safety factor.

When applying the Limit Equilibrium Method, acceptable safety factors (FS) range from 1.5 to 1.15 for two-dimensional modeling, depending on the level of safety required for the site. It is important to note that there are no specific laws or standards defining minimum safety factors for three-dimensional stability analyses. For this case study, FS results were considered according to the 2D standard.

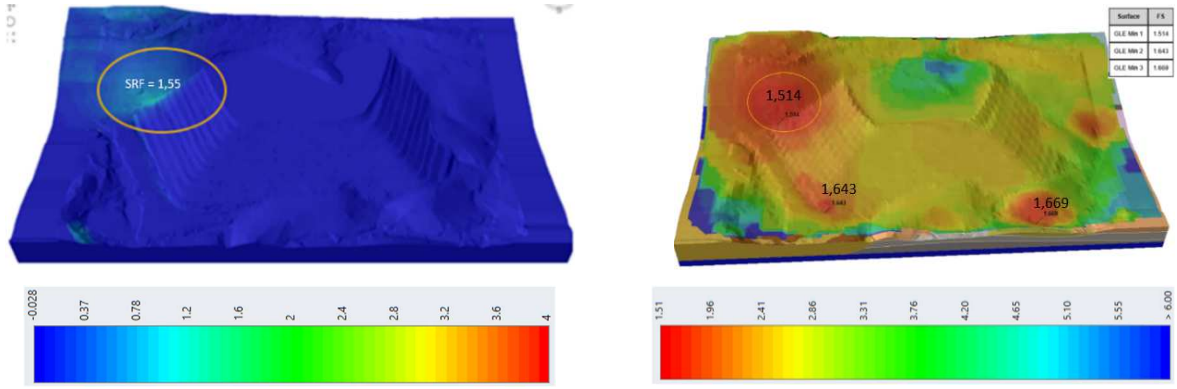
An important aspect when comparing two-dimensional (2D) and three-dimensional (3D) analyses is that traditional 2D stability analyses do not take into account the confinement of the soil mass, influenced by the lateral friction of the adjacent soil. This limitation may

result in an underestimation of the slope's load-bearing capacity. It is observed that the safety factors obtained in the 2D analysis are lower than those in the 3D analyses, as previously justified.

For this study, the GLE Morgenstern-Price methodology was adopted because it is more rigorous and better represents the complexity of the slope. However, the finite element method does not follow a specific standard. One advantage of the FEM-SRF technique is that it does not require the direction of the slip to be specified beforehand.

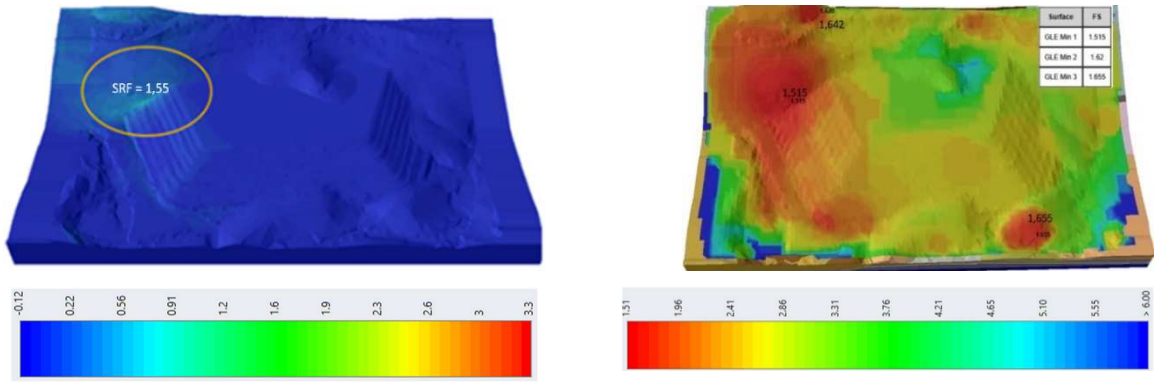
The results and the location of the slip surface from both methodologies demonstrated strong consistency throughout the various decommissioning stages.

Additionally, in Stage 2 of decommissioning (Fig. 21), values below the recommended standard 2D were observed, indicating the need for stabilization systems to ensure an increase in the Safety Factor for the next phases of material removal.



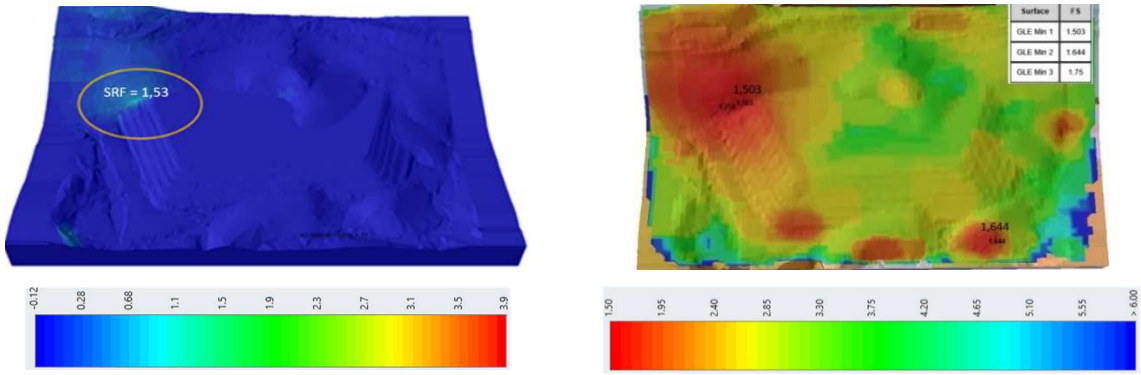
(a) **(b)**

Fig. 17 – Stability Analysis in the 11th Stage end construction: SRF = 1.55 and LEM = 1.51 using (a) Maximum Shear Strain [m] and (b) Surface Safety Map



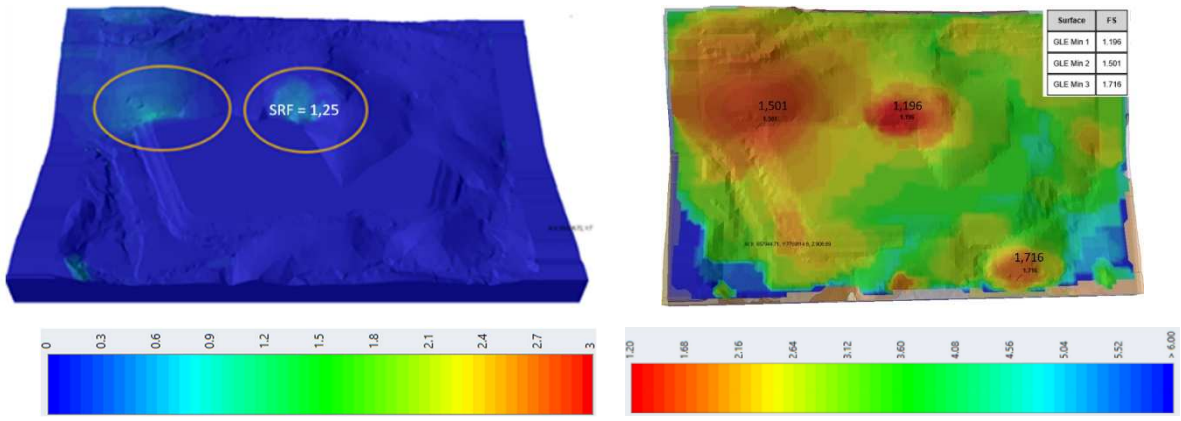
(a) **(b)**

Fig. 18 – Stability Analysis in the 8th Stage of Decommissioning: SRF = 1.55 and LEM = 1.51 using (a) Maximum Shear Strain [m] and (b) Surface Safety Map



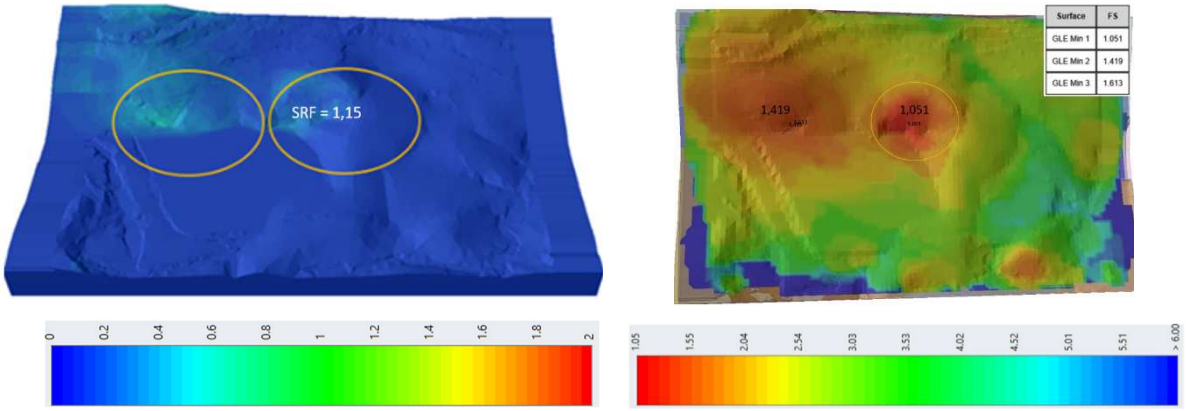
(a) **(b)**

Fig. 19 – Stability Analysis in the 6th Stage of Decommissioning: SRF = 1.53 and LEM = 1.50 using (a) Maximum Shear Strain [m] and (b) Surface Safety Map



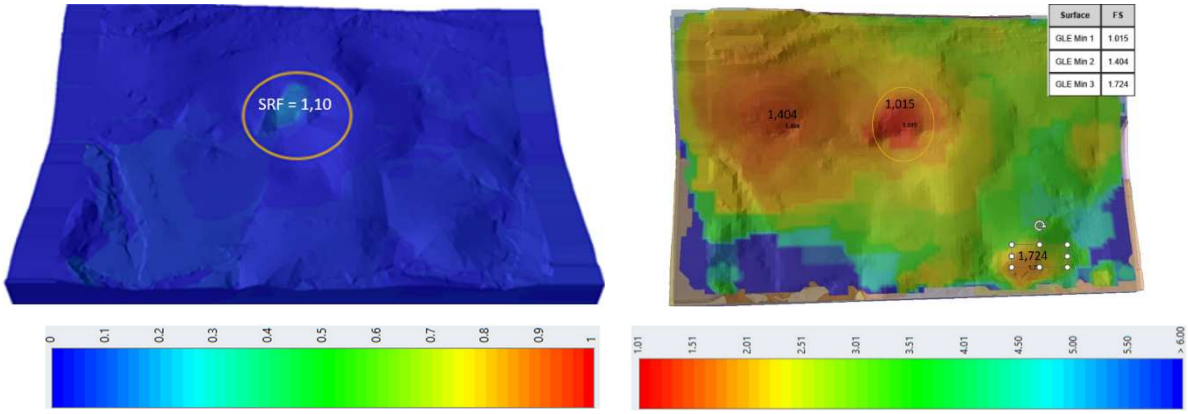
(a) **(b)**

Fig. 20 – Stability Analysis in the 4th Stage of Decommissioning: SRF = 1.25 and LEM = 1.20 using (a) Maximum Shear Strain [m] and (b) Surface Safety Map



(a) **(b)**

Fig. 21 – Stability Analysis in the 2nd Stage of Decommissioning: SRF = 1.15 and LEM = 1.05 using (a) Maximum Shear Strain [m] and (b) Surface Safety Map



(a) **(b)**

Fig. 22 – Stability Analysis in the Final Stage of Decommissioning: SRF=1.10 and LEM=1.02 using (a) Maximum Shear Strain [m] and (b) Limit Equilibrium Method (LEM)

To compare the results from both methods, a graph of the Factor of Safety (FoS) against the excavation stages was plotted, as shown in Fig. 23. This graph compiles the results of the SRF and LEM analyses, allowing for a direct comparison of the safety factors obtained from both approaches throughout the different stages of excavation.

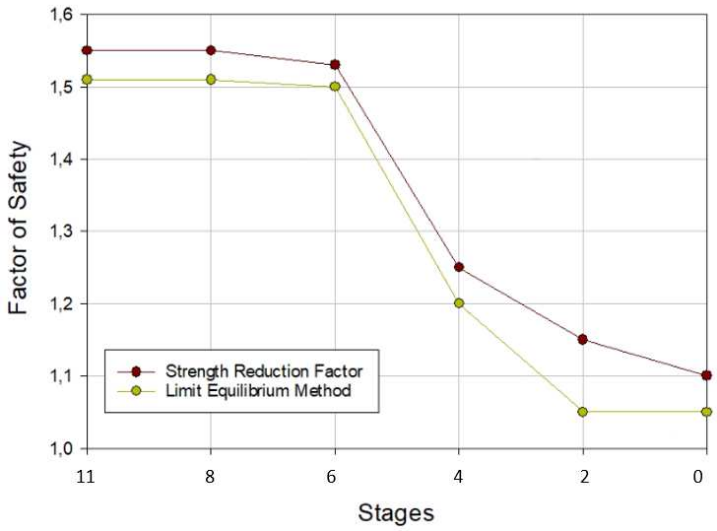


Fig. 23 – Plot of stages vs Factor of Safety for slope 3D

4. CONCLUSIONS

The objective of the 3D analyses of the Stress vs. Strain behavior was to predict the displacements and assess the stability and vulnerability of the slope due to the planned unloading for 6 years of removal.

The results showed a good agreement in the slope wedges observed using the limit equilibrium and strength reduction factor methods during the dam decommissioning stages. However, a difference in the safety factor was observed, which can be attributed to the method used in the

analysis. The difference between the two methodologies ranges from 0.04 to 0.1, as shown in Figure 23. This occurs due to the difference in how the methods assess the problem, as the limit equilibrium method is based on the slice method, while the finite element method relies on maximum shear deformations.

The strength reduction factor based on finite element methods (SRF) has proven to be an extremely efficient tool for dealing with complex 3D slope stability problems. It not only provides the minimum Factor of Safety (FS) with sufficient precision but also allows for the description of the deformed mesh of the slope on the verge of failure, including shear deformation, plasticization points, and the stress states associated with each stage of the process. In this way, SRF offers a detailed analysis of the slope's behavior evolution, providing crucial information for understanding failure conditions and contributing to more informed decision-making in the design and monitoring of geotechnical engineering projects.

In the final stages, a decrease in the safety factor is observed. This occurs because the dam was providing stabilization to the slope, and after the material from the dam was removed, the slope experienced a reduction in the safety factor. Considering that the most critical stages in the stress and strain analysis (SRF) and limit equilibrium studies are Stage 2 and the final stage, the evaluation of the safety factor using both methodologies complements the 3D studies. This approach provides a more detailed and comprehensive understanding of the system's behavior. Such an integrated method significantly contributes to the implementation of stabilization or remediation works, enabling a more accurate risk assessment and the adoption of appropriate measures to ensure the safety of structures and minimize environmental impacts.

REFERENCES

[1] Abramson, L. W., Lee, T. S., Sharma, S., & Boyce, G. M. (2002). *Slope stability and stabilization methods* (2nd ed.). New York, NY: John Wiley & Sons.

[2] Anagnosti P (1969) Three-dimensional stability of fill dams. In: *Proceedings of 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico, pp 275–280.

[3] Bishop, A. W. (1955). The use of the slip circle in the stability analysis of slopes. *Geotechnique*, 5(1), 7–17. https://doi.org/10.1680/geot.1955.5.1.7

[4] Chen RH, Chameaut J (1983) Three-dimensional limit equilibrium analysis of slopes. *Geotechnique*, 33:31–40.

[5] Chowdhury, R., Flentje, P., & Bhattacharya, G. (2010). *Geotechnical slope analysis*. Boca Raton, FL: CRC Press.

[6] Cheng, Y. M., Liu, H. T., Wei, W. B., & Au, S. K. (2005). *Location of critical three-dimensional non-spherical failure surface by NURBS functions and ellipsoid with applications to highway slopes*. Computers and Geotechnics, 32(6), 387–399.

[7] Das, B. M. (2010). *Principles of geotechnical engineering* (7th ed.). Stamford, CT: Cengage Learning.

[8] Duncan, J. M., & Wright, S. G. (2005). *Soil strength and slope stability*. Hoboken, NJ: John Wiley & Sons.

[9] Fredlund, D. G., & Krahn, J. (1977). Comparison of slope stability methods of analysis. *Canadian Geotechnical Journal*, 14(3), 429–439. https://doi.org/10.1139/t77-045.

[10] Griffiths, D. V., & Lane, P. A. (1999). *Slope stability analysis by finite elements*. Géotechnique, 49(3), 387–403.

[11] Griffiths, D. V., & Marquez, R. M. (2007). *Three-dimensional slope stability analysis by elastoplastic finite elements*. Geotechnique, 57(6), 537–546.

[12] Hoek, E., & Bray, J. W. (1981). *Rock slope engineering*. London, UK: Institution of Mining and Metallurgy.

[13] Hungr O (1987) An extension of Bishop's simplified method of slope stability analysis to three dimensions. *Geotechnique*, 37(1):113–117.

[14] Janbu, N. (1954). Stability analysis of slopes with dimensionless parameters. *Harvard Soil Mechanics Series* (No. 46). Cambridge, MA: Harvard University Press.

[15] Kumar, S., Choudhury, S. S., & Burman, A. (2022). Recent advances in 3D slope stability analysis: a detailed review. *ResearchGate*. DOI: 10.1007/s40808-022-01597-y

[16] Lu, H., Fredlund, M. D., & Fredlund D. D (2014). Comparison Between 3D Limit Equilibrium and Shear Strength Reduction Methodologies. *Geotechnical Special Publication*. DOI: 10.1061/9780784413272.303.

[17] Mongenstern NR, Price VE (1965). The analysis of stability of general slide surfaces. *Geotechnique*, 15:79–93.

[18] Rocha, M. (1971). A method of integral sampling of rock masses. *Rock Mechanics and Rock Engineering*, 3.1 (1971): 1-12.

[19] Rocscience. (2024). *RS3 User's Manual*.

[20] Spencer E (1967). A method of analysis for stability of embankment using parallel inters slice force. *Geotechnique*, 17:11–26.

[21] Terzaghi, K., Peck, R. B., & Mesri, G. (1996). *Soil mechanics in engineering practice* (3rd ed.). New York, NY: John Wiley & Sons.

[22] Whitman, R. V., & Bailey, W. A. (1967). Use of computers for slope stability analysis. *Journal of the Soil Mechanics and Foundations Division*, 93(4), 475–498.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

FROM 2D TO 3D TAILINGS DAM STABILITY ANALYSES: ASSUMPTIONS, BENEFITS AND UNCERTAINTIES

Gonçalo Ferreira[1]
TPF Consultores

ABSTRACT

The minimum factors of safety (FS) established in international norms and guidelines for deterministic stability analysis of tailing dams are based on a century-long practice with two-dimensional (2D) analysis. Most slope stability analyses neglect the three-dimensional (3D) effects of the problem without significant consequences. However, these effects depend strongly on the geometry of the problem, namely its boundary conditions, the variability of the *in situ* conditions, and the uncertainties associated with it. The transition from 2D to 3D analysis encompasses both advantages and disadvantages: while 3D analyses allow for more representative geometries, they involve more complexity and uncertainty in the model's parameters due to spatial variability. Moreover, for most of the technical community, experience with 3D limit equilibrium analyses is still recent.

In some cases, only a 3D analysis will reflect a representative geometry of the sliding mass, hence a more realistic margin of safety. However, it is well established that there is an increase of the FS just by the fact that a 3D analysis is conducted, i.e. the FS_{3D} to FS_{2D} ratio ($R_{3D/2D}$) is consistently higher than one, due to the 3D (end) effects that are neglected in a 2D analysis. These benefits of a 3D analysis increased the interest on and the use of it. However, no prescription of minimum FS_{3D} exist, thus the already established regulatory FS, based on a 2D experience, are still being used for 3D analysis. This is particularly relevant for regular dam safety reviews.

This paper discusses the benefits and uncertainties of 3D analysis, comparing the differences between the basic assumptions of these and 2D analysis. It also discusses the prescribed FS by international guidelines for stability analysis of tailing dams and the rationale behind these guidelines. Finally, it presents a simple risk-based approach for the definition of minimum FS_{3D} analysis considering the current state of the art proposals on the existing literature.

Keywords: tailings dam, 3D analysis, factor of safety, limit equilibrium method.

[1]goncalo.ferreira@tpf.pt, Portugal

1. INTRODUCTION

The Canadian Dam Association (CDA) states that the application of Factors of Safety (FS), as quantitative measures of safety in Limit Equilibrium (LE) methods in slope stability analysis, stems from a *traditional standard-based approach*, which is *deterministic* in essence [1]. Despite its shortcomings, namely the fact that does not consider the uncertainties related with the overall characterization of the structure and its materials, this deterministic approach, with decades of experience, has generally been successful.

LE methods are the most commonly used for routine slope stability analysis, as failures along critical slip surfaces in dams [2], insofar they are simpler and consume less time [3]. An alternative to LE analyses would be a stress-strain path analysis, which would allow for stress-strain compatibility and stress redistribution along the failure plane. While the latter method allows for a more representative definition of the *in situ* stress distribution and failure path, it implies the use of more complex software and constitutive models. Moreover, a drawback of stress-strain path analysis is that they do not give a direct indication of the *margin of safety* from failure, whereas the global FS of a LE analysis, even if simplified, portrays one [4].

In his 1996 overview paper about the state-of-the-art of slope stability analysis [5], Duncan refers to the proliferation and affordable costs of computers, allowing engineers convenience and computing power, which revolutionized (two dimensional, 2D) slope stability analysis, namely due to the implementation of advanced methods that satisfy all conditions of equilibrium and the possibility to quickly analyse a large number of slip surfaces. The revolution continued its path expanding into three-dimensional (3D) LE analysis. 3D slope stability analyses have been used since the 60's of the previous century [5], with a big theoretical development through the 70s and 80s, but only recently, 3D modelling gain momentum as a geotechnical tool to evaluate slope stability. As expected [6], due to the 3D modelling advantages (or the limitations of the 2D analysis), software development and the increase of computer power, 3D slope stability analysis became easier and faster to do and thus more routinely used in slope's safety evaluation (e.g. [7] and [8]).

Tailings dams have a higher rate of failure compared to water dams, which depends highly on the fact that they evolve throughout their life cycle, are seen as a passive, expensive infrastructure, and may rely strongly on tailings for stability [9] [10]. After the environmental and social impact of recent failure of several tailing dams, due to the liquefaction of saturated loose tailings, efforts have been made by the international mining community to improve the regulation of the design and safety review of mining dams (ICOLD [2] and ICMM [11]). Still, no regulatory prescriptions exist for 3D LE analysis.

Based on theoretic, technical and normative literature, this paper discusses some of the assumptions used in, and uncertainties inherent to, 2D LE analysis and slope stability methods, namely in the context of tailings dam design and safety review, as well as its transposition to the extra dimension of a 3D analysis. The discussion starts with the assumptions inherent to LE procedures, namely the method of slices, and its evolution to 3D space. It then follows to the assumptions on the definition of FS as an attempt to quantify a margin of safety in slope stability analysis. After, an overview of tailings dam stability analysis, in particular existent norms and guidelines, is presented. Finally, the paper presents an approach to estimate minimum FS for 3D LE analyses, based on current state of the art.

2. DAM STABILITY ANALYSIS

2.1. Limit equilibrium method

Traditionally, slope stability analyses are performed following LE methods, i.e., methods that are concerned with the failure of a system when the applied loads overcome the soil's ultimate resistance. Terzaghi (1943) defines a *slide* (also called a *slip*) as a "failure [that] occurs by shear along a well-defined surface", involving "a downward and outward movement of a slice of earth" [12]. Stability problems are concerned with the LE of a mass of ideal soil, prior to ultimate failure by plastic flow. Thus, it assumes that the ideal soil has an ultimate yield point, ignoring its stress-strain path. In fact, a major limitation of limit equilibrium methods is that they only satisfy static equilibrium, neglecting strain and displacement compatibility [4].

Duncan *et al.* (2014) distinguish between two different approaches to satisfy static equilibrium following a LE analysis [13]: the *single free body* approach, where the equilibrium is satisfied for the whole sliding mass (e.g. infinite slope), and the *method of slices*, where the sliding mass is divided in several vertical slices (e.g. Bishop and Janbu's procedures, Morgenstern-Price, Spencer). Since every procedure, whatever the approach, is statically indeterminate, *assumptions* must be made to achieve equilibrium. Different procedures have different assumptions. One of the simplest procedures is the *infinite slope*, which considers an infinite slope in length and the sliding to occur in a parallel plane to the slope, satisfying static equilibrium for a rectangular block whose side forces are equal in magnitude, parallel and in opposite directions, thus cancelling each other (Fig. 1a).

The most commonly used LE approach for slope stability analysis though is the *method of slices* (Fig. 1b). According to Terzaghi *et al.* (1996), the method of slices was initially used to deal with problems where the slope's surface was not a straight line and the same sliding surface cut different materials [14]. Following this method, the sliding mass is divided in several vertical slices, each one of them acted by a set of base and inter-slice forces. The parameters

involved are the soil's strength, the stress (either total or effective) on the failure plane, internal pore pressures, external water pressures and (eventually) external loads [5]. Depending on the procedure adopted, forces and/or momentum equilibrium must be satisfied.

The first proposed procedure was the Ordinary Method of Slices (OMS), which neglects the inter-slice forces and satisfies only moment equilibrium. Bishop's procedure considers the inter-slice forces horizontal satisfying also vertical forces equilibrium. Janbu's procedure while also considering horizontal inter-slice forces, only satisfies force equilibrium. Some procedures satisfy both force and momentum equilibrium, such as Morgenstern-Price and Spencer's, with the latter considering that the inter-slice forces are parallel to each other and the former a function between the shear and the normal inter-slice forces. A good and exhaustive overview of the several procedures of the method of slices is given by Duncan *et al.* (2014) [13].

Static equilibrium being satisfied, the ratio between the shear strength of the soil and the shear stresses along the base is calculated and the resultant FS is compared with a minimum (normally regulatory) FS. Following Duncan & Wright (1980), all limit equilibrium methods share the same definition of FS, assume the most critical uncertainty is the evaluation of the shear strength, assume the soil does not have a brittle behaviour (therefore the same shear strength can be mobilized along the slip surface over a wide range of strains), and use a set of equations of equilibrium with explicit assumptions [15]. The main difference between different LE procedures rests on the assumptions and equations for static equilibrium used, the shape of surface used, and the inclusion of inter-slice forces as well as the assumed relationship between these forces [3], the latter's influence depending on the type of sliding movement [4]. Several authors [15] [16] concluded that for the same loading conditions, strength parameters and geometry, a stability analysis performed with any limit equilibrium method will give approximately and correctly the same result (i.e., FS), within an acceptable margin of error (5 to 10%), thus stating that the main uncertainty of a stability analysis following this type of method lies in an adequate choice of strength parameters.

To overcome the limitations of 2D analysis for specific problems and to evaluate the 3D (end) effects, several authors presented different 3D LE procedures, each with different assumptions, to evaluate the 3D stability of slopes. Anagnosti (1969) [17] evaluated the stability of earth and rock-fill dams placed in narrow valleys or canyons using what can be considered a 3D extension of Morgenstern-Price's procedure. Baligh & Azzouz (1975) [18] followed an extension of the OMS and used a computer software (STAB3D) to evaluate the 3D effects of vertical cut and angled slopes, for different geometric lateral ends of the slip surface (vertical, conical or ellipsoidal) of cohesive soils. Hovland (1977) [19] developed a general 3D slope stability method to evaluate the end effects of a 3D sliding mass, for any type of soil's strength response and different sliding surfaces geometries, where the sliding mass is

divided in several orthogonal columns, as an extension of the 2D method of slices, nowadays named as the *method of columns* (Fig. 1c). Hovland neglected the inter-slice forces, using what can be considered an extension of Bishop's procedure. All these authors concluded that the FS obtained in 3D analysis (FS_{3D}) could be up to 50% higher than the 2D one (FS_{2D}).

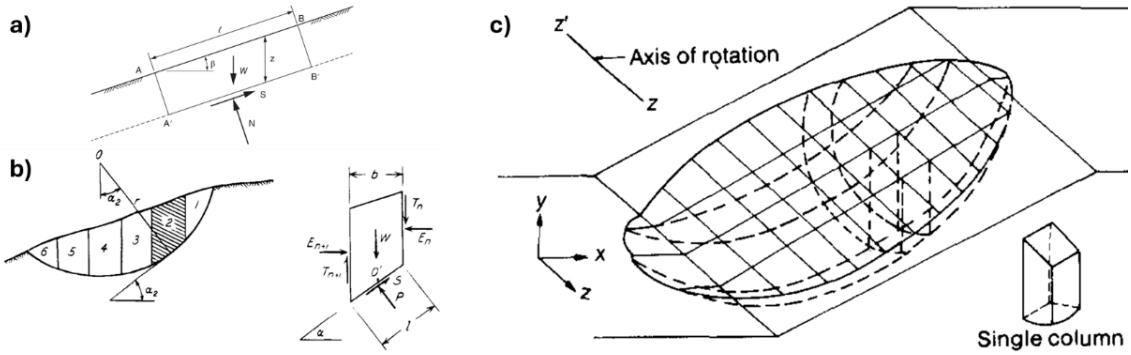


Fig. 1 – Limit equilibrium methods: a) infinite slope [13]; b) slices [14]; c) columns [20]

Chen & Chameau (1983) adopted Hovland's method [20], following what is considered a 3D extension of Spencer's procedure [6] [21], where the inter-slice forces have the same inclination and its shear component is parallel to the base of the column. Their study concludes that the end effects (3D effects) decrease with the increase of the ratio with/height (W/H) of the failure surface, as well as with the increase of the slope's angle, and increase with smaller lengths of the failure surface. The authors also conclude that in some cases the FS ratio between 3D and 2D ($R_{3D/2D}$) can go below 1,0. However, Cavounidis (1987) proved mathematically that FS_{3D} is always greater than the FS_{2D}. In fact, *ceteribus paribus*, a 3D analysis considering a very large width (big enough to be considered infinite) is virtually the same as a 2D analysis. Any proof to the contrary implies that there either was a mistake in the choice of the critical slip surface or simplified assumptions that neglect important aspects of the problem were used [21]. As Duncan's (1980) concludes that any comparative FS_{2D} analysis must be performed with the minimum FS [15], Cavounidis states that any meaningful comparison between a 3D and a 2D analysis must consider the minimum FS obtained in each of them, knowing that the 3D critical slip surface may not contain the critical 2D one [21]. The author also states that the shape (e.g. circular, elliptical, hyperbolic) of the 3D slip surface influences the minimum FS_{3D}.

This is also shown by Gens *et al.* (1988), while studying the end effects of different 3D surface ends (plane or curved, in different shapes) on a rotational slide in a homogeneous, isotropic, purely cohesive slope [22], following a similar approach to Baligh & Azzouz [18]. Confirming Cavounidis' proof, Gens *et al.* show that the FS_{3D} is always higher than the FS_{2D}, and that the effect of different end shapes in the FS converge to $R_{3D/2D}$ of 1,0 for very wide failures (W/H > 20, Fig. 3a). The authors then show that the FS_{2D} is more conservative than a FS_{3D}, with FS_{3D}

values up to 70% higher than FS_{2D}, which is particularly important when estimating shear resistance from back analysis.

Stark & Eid (1998) reiterate the importance of 3D analysis in back calculating the mobilized shear strength from a real slope failure [23]. Using a commercial 3D slope stability software which follows the method of columns, the authors focus on the analysis of translational failures (Fig. 2b), showing that the 3D effects are greater in this type of failures *vis-à-vis* rotational ones (Fig. 2a). This is because, typically, translational failures tend to slip through a weak layer at the base, thus presenting a greater difference between the mobilized shear resistance along the base and the one along the back scarp and sides of the sliding mass (Fig. 2c).

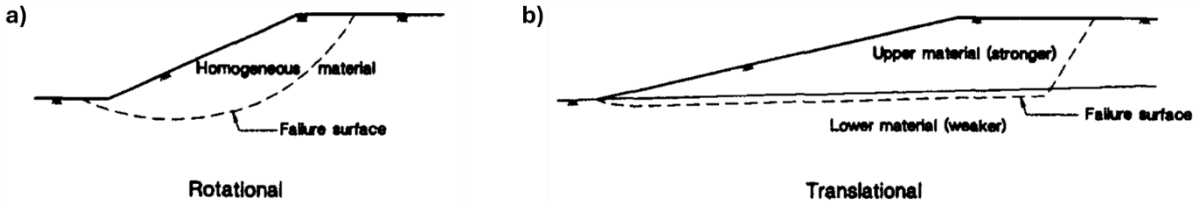


Fig. 2 – Failure modes for slopes [23]: a) rotational; b) translational

With the aim of creating practical abacus for the assessment of the need of a 3D slope stability analysis, Arellano & Stark (2000) present a parametric study of the effect of the slope angle on the $R_{3D/2D}$, using the model of Fig. 2c, varying H and W and assuming the at-rest earth pressure coefficient (K_0) [25]. The results of the study show that: $R_{3D/2D}$ is always greater than 1,0; the flatter the slope the greater the $R_{3D/2D}$ and; $R_{3D/2D}$ decreases with decreasing W/H due to the lower influence of the shearing resistance along the sides of the sliding mass. Akhtar & Stark (2017) [26] updated Arellano & Stark parametric using an earth pressure coefficient in-between K_0 and the active (K_a), finding lower $R_{3D/2D}$ (Fig. 3b).

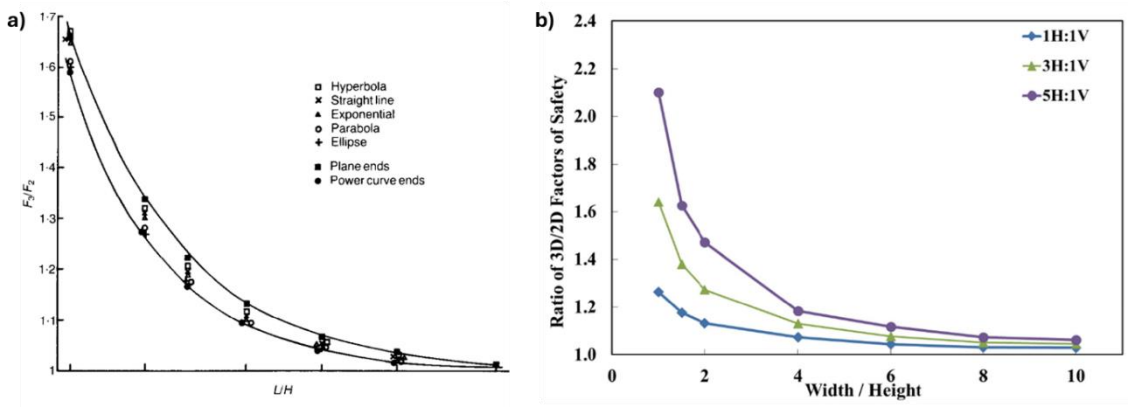


Fig. 3 – End effects (mobilized shear resistance along the vertical walls of the sliding mass): a) for different end shapes [22]; b) for different slope angles [25]

From the analysis of previous 3D LE research, Akhtar (2011) concludes that most of the assumptions used in 3D LE procedures do not represent field conditions (sliding mass

geometry, soil's type and strength, pore water pressure), with only a minor part of the studies considering field cases histories in order to validate their assumptions, and a smaller portion going beyond the cases that fit their assumptions [3]. Based on field cases and scientific literature, Akhtar defines three types of sliding: *rotational* (Fig. 2a), typical in homogeneous soils and approximately with ellipsoidal shape; *translational* slides (Fig. 2b), dominated by a weak layer at the base of the sliding mass, and; compound slides, a combination of both. The first two do not necessarily involve internal deformations, whereas the latter does.

Nowadays, it is possible to run a 3D slope stability analysis using a commercial software (e.g. Slide3 from *Rocscience* or SLOPE3D from *GeoStudio*), following different procedures of the method of columns (e.g. Bishop's, Janbu's, Spencer's, Morgestern-Price). This type of software has automatized optimization algorithms to search for the critical surface and allow for big and complex 3D geometries, to evaluate the sliding in any direction within the model, and to consider different materials and porepressure distributions.

2.2. A margin of safety

Within the ultimate limit state theoretical framework, Meyerhof (1995) defines a FS as the ratio between the *ultimate resistance of the earth structure* and the *applied loads or load effects* on the structure, in order to "ensure freedom from danger, loss, or unacceptable risks" [27]. Bromhead (1992) defines the FS as an *index of relative stability*, taken as the ratio between the *actual strength available* and the *mobilized shear strength* (i.e., the mobilized strength relative to a certain maximum resistance of the soil), with the latter being equal to the *destabilizing forces* [28]. Duncan (1996) calls it a *factor on shear strength*, using Lowe's definition of FS, who describes it as "the factor by which the shear strength of the soil would have to be divided to bring the slope into a state of barely equilibrium", considering it logical insofar the biggest uncertainty in the analysis arises from the estimation of the shear strength [5]. This FS, assumed to be the same along the slip surface, represents an average quantitative indication of slope stability, that involves a certain degree of uncertainty [13].

However, the ICMM (2021) refers that a FS is commonly misunderstood as a unique measure of safety that when increased reduces the likelihood of failure [29]. For that reason, and although the intrinsic material variations both *in situ* and in laboratory, FS are rarely reported followed by the uncertainty limitations of the analysis. Moreover, the CDA warns that a FS is only "physically meaningful with respect to given design assumptions and equations" [1].

Taking all the above into consideration, one can generally define the FS as a *margin of safety* against the *limit strength capacity of a soil or a structure* when acted upon by a set of *destabilizing actions and loads*, which allows to accommodate *uncertainties* of the analysis and *consequences* of the possible failure. FS from stability analyses are only as reliable as the

data used to define the problem, and minimum (required) FS must, case by case, be defined based on *experience*, considering the *uncertainties* and the *consequence of failure* for each design scenario [5] [13] [1].

Meyerhof (1995) stated that the first time a concept of a FS in stability analysis was used was in the 18th century, in the design of a retaining wall, and only in the beginning of the 20th century a FS of 1,5 was used for slope stability [27]. This opened the door for a practice around the world – the use of a global FS – still ongoing, especially in dam's design and safety reviews. At the time Meyerhof was writing his paper, still following Terzaghi and Peck mid-20th century proposal, the global FS for slope stabilities varied typically between 1,3 to 1,5 for earthworks and 1,5 to 2,0 for earth retaining structures. The presented ranges presupposed a level of *risk* and *uncertainty*, insofar the lower limits were usually adopted for normal loads and service conditions, and higher limits adopted when monitoring data, field tests and similar case studies were available or for temporary works [27].

Duncan (1996) defends a similar idea by stating that the criteria for an acceptable FS should take two factors into considerations [5]: (1) the *degree of uncertainty* of the conditions and material strengths in the analysis; (2) the *potential consequences of failure*. When these two factors are small, smaller FS could be accepted, whereas when one of these factors is large, then the FS should be increased. Duncan goes further, stating that it is unacceptable to have large uncertainties and large consequences, no matter the FS used in the analysis [5].

The European practice, following the Eurocode philosophy [30], is to apply partial coefficients to soil parameters, actions and resistances (an example of that is the Portuguese Dam regulation [31]), differentiating limit state design between ultimate (near-rupture) and service (allowable deformations) limit states, as well as types of designs (e.g. structural, geotechnical, equilibrium). Bond & Harris (2008) [32] explain that this philosophy takes distance from the traditional global FS, for the latter although ensuring some degree of reliability in the design, does not provide an adequate control of all the uncertainties involved in the calculations. Albeit, international norms and guidelines for dam design and safety review, particularly for tailings dams, still use traditional global FS as the deterministic parameter to assess the dam's stability safety (e.g. ICOLD [2], ANCOLD [33], CDA [34], ABNT [35]). Moreover, the definition of the minimum (required) FS for stability analysis in these norms and guidelines develops historically from the experience with 2D analysis [2]. Naturally, these FS, established to guarantee a safety margin in a limit equilibrium scenario, are intrinsically associated with the methods used to analyse the structure's stability, the latter developed for 2D geometries. The regulatory FS were then established for comparison with FS estimated from 2D analysis [36].

That said, and as already stated, a single 2D stability analysis, i.e., an analysis of one only cross-section, is only representative of the tridimensionality of a dam in the case of a wide dam

(meaning that its longitudinal dimension is significantly bigger than its height) with a homogeneous cross-section throughout, where the end effects have negligible impact on the stability of the structure, and there is little or none variability of the geology and of the resistance of the materials throughout the embankment and its foundation and reservoir. These cases rarely happen in reality. Even though single 2D analysis are not representative of the tridimensionality of the problem, they are usually considered sufficiently conservative as long as they follow a set of assumptions, the most important being that the chosen cross-section is the critical one. Often the critical cross-section is taken as the “maximum” one, i.e., the one with the biggest area (normally with the biggest height of the embankment). However, it is not always the case that the “maximum” cross-section is the critical or the most representative one, making its choice to be an arbitrary process that relies on subjective interpretation [37].

In an attempt to overcome the limitations of a single 2D analysis and take into consideration three-dimensional effects, Lambe & Whitman (1969) suggested that the FS could be estimated for at least three parallel cross sections in a slope and a *Weighted Safety Factor* could be computed using the weight of the sliding mass as a weighting factor [16]. Based on the previous method, Loer *et al.* (2004) developed the *Resistance-Weighted procedure*, where what the authors called a *quasi-three-dimensional* FS is computed by weighting the FS obtained in different cross-sections based on the total equilibrium shear force along each pair of slip surfaces [37]. Even though in some cases this method can give reasonable results, it is not a real 3D procedure for it neglects the forces between the cross-sections, which makes it unwarranted for failures with steep sides (e.g. translational ones) [3].

Nowadays, with the implementation of 3D LE methods in commercial software, it is possible and faster to routinely run 3D slope stability analysis. Furthermore, it is common knowledge that 3D analysis lead to higher FS compared to 2D (from 10% up to 100% higher [22] [24] [25] [26]). These factors lead to an increase of interest in 3D analysis. In Brazil, for example, 3D stability analyses are being used in dam safety evaluations to supplement the traditional 2D analysis. However, there are still no guidelines for regulatory FS for 3D LE analysis ($FS_{3D,min}$).

Akhtar & Stark (2017) conclude that the FS_{3D} should not be compared to 2D minimum (regulatory) FS ($FS_{2D,min}$) due to the additional shear resistance generated along the sides of the sliding surface: the recommendation, to achieve the same level of risk of a 2D LE procedure, is that a greater regulatory FS should be used in 3D analysis [26]. Stark & Ruffing (2017) propose a method to compute $FS_{3D,min}$ for comparison to the FS calculated from 3D analysis [36]. The method uses the $R_{3D/2D}$–W/H curves obtained by Akhtar & Stark (Fig. 3b) to estimate the extra margin of safety that will add to the $FS_{2D,min}$ in order to define the $FS_{3D,min}$, for each design scenario. Three approaches (equations) are proposed by the authors, where

the $FS_{2D,min}$ and the $R_{3D/2D}$ (function of W/H) are used to estimate $FS_{3D,min}$ depending on the level of risk (or conservatism) one wants to accept for the structure (eq. 1 to 3).

$$FS_{3D,min} = FS_{2D,min} \times R_{3D/2D} \text{ (more conservative)} \quad (1)$$

$$FS_{3D,min} = FS_{2D,min} + R_{3D/2D} \quad (2)$$

$$FS_{3D,min} = FS_{2D,min} + \frac{R_{3D/2D}-1}{R_{3D/2D}} \text{ (less conservative)} \quad (3)$$

Each approach would then be used depending on the design situation, taking into consideration the inherent risk associated with soil strength uncertainty, threat to life and failure consequences (see Table 2). It is important to state that Stark & Ruffing proposal comes from a decades-long study of translational slides, thus caution must be taken when applying it to rotational slides. It is nonetheless a risk-based rational method that follows the state-of-the-art of international guidelines such as the ICCM [11], presenting similar results than the previous 3D stability analysis research on rotational slides ([19[18] to [22]). Moreover, the authors recommend that a $R_{3D/2D}$–W/H curve should be defined for each structure. It is important to state that despite the focus of Akhtar, Stark and Ruffing is on trinational slides, there is 50-years long research on the 3D effects on slope stability analysis that shows similar results to the ones of these authors for rotational slides ([18] to [22]).

2.3. Tailings dams

For the design and safety evaluations of tailings dams, the ICMM (2020) requires the application of “design criteria, such as factors of safety for slope stability (…), that consider estimated operational properties of materials and expected performance of design elements” [11]. It also considers the dam risk category, based on the consequence of failure, to define design actions, thus working within a *risk-based framework*. Deterministic slope stability evaluation begun to be a practice in tailings dam design in the 1970s, based on water dams design practice [29]. Nowadays there exist several international guidelines and norms that give orientations about the margin of safety engineers should adopt in different design situations of LE slope stability analyses.

ICOLD’s Bulletin 139 (B139) state that a FS of 1,5 is generally accepted when the maximum deviatoric stress shear resistance is used in the analysis. However, it also states that a higher FS must be used for sand loose tailings, when their liquefaction can lead to a storage facility failure (e.g. upstream raised dams), and for long term scenarios where the probability of an extreme event is higher [38]. ICOLD’s B194 defines an FS of 1,5 for tailings dams under construction or being raised, adding that this is only an initial target and that higher values could be adopted when high uncertainties in geologic conditions, material properties, pore pressures, presence of sensitive and/or strain softening soils, or potential for changes to soil properties or loading conditions over time [2]. In line with the aforementioned authors, ICOLD’s

B139 suggests that the uncertainty added by the shearing resistance along the ends of the sliding mass in 3D analysis may be considered, by increasing the minimum FS for 3D analysis, and that 3D analysis should not be used “in an effort to increase the FOS obtained from that determined from the 2-D analysis”. Finally, B194 prescribes an increase on target FS for different cases and conditions, namely depending on the propensity for the materials to present post-peak strength loss (i.e., brittle behaviour) or when there is a considerable uncertainty regarding strength characterization and/or pore pressure conditions.

Two well-established international guidelines for tailings dams are the CDA and the Australian National Committee on Large Dams (ANCOLD). The CDA state that in the FS deterministic approach the uncertainties are considered by assuming conservative values for loads, resistances and the minimum prescribed FS [1]. The adequateness of these FS depends thus on the conservatism bestowed to the definition of the problem, namely its assumptions. The CDA mining dam guidelines (2014) reiterates that it is the role of the designer to establish proper FS for each mining dam depending on the load conditions [34]. ANCOLD (2012) warns for the possible need to consider non-circular failure surfaces and the need to critically assess the adequacy of the critical slip surface obtained in the analysis, insofar shallow, non-impactful slip surfaces may be caught by the search algorithm if no criteria is used to avoid it [33]. It also warns for the fact that some situations, usually related with assumptions regarding shear strength and pore pressures, can lead to an overestimation of the FS, one of them being the potential of liquefaction in upstream raised tailings dams. It is important to state that, likewise ICMM, both ANCOLD (2012) and CDA (2019) consider the dam failure consequence category to define the design flood and earthquake loading, which means that, at least for some design cases, the consequence category is indirectly considered in the LE analysis [33] [34].

Another example of a tailings dam normative is the Brazilian norm for design of tailings dam (NBR 13028:2024). This norm starts prescribing that minimum FS should consider the loading conditions, namely drained or undrained, of each material considered in the stability analysis, although, in a later paragraph, states that the prescribed FS must be obtained independent of the type of analysis and the loading conditions [35]. Moreover, it considers that when undrained strength parameters are adopted, the minimum FS to be used should be defined by the designer, following “engineering good practices”.

Table 1 presents a summary of the prescribed FS by the aforementioned international norms and guidelines, for several typical design scenarios. Most of the presented FS are deterministic and, in general, sole function of the design scenario, hence taking only into consideration different load condition. Some are more or less rigid on the prescription. For example, the CDA consider that lower FS may be acceptable for existing structures as long as a proper monitoring system shows an adequate performance [1].

Even though it is directed towards water dams, it is interesting to mention the prescriptions of Fell *et al.* (2015) for new dams with high consequence of failure [39]. These authors present FS for different design cases, defined based on the assumptions that the parameters used in the analysis are reasonably conservative and there is a good monitoring system. Besides, the authors consider that, on the one hand, higher FS should be adopted when higher uncertainties exist, namely when undrained shear strength can be mobilized, weak layers or contractive soils exist, and there is limited investigation and testing. On the other hand, lower FS can be used for existing, well monitored dams (Table 1).

Apart from Fell *et al.*, the cases presented on Table 1 do not consider the level of *uncertainties* nor the *consequence of failure* of the dam, at least not directly, therefore are outdated in relation to a *risk-based* driven framework. Some authors have been discussing the engineering practice in tailings dam design, particularly the definition of the minimum FS for LE slope stability analysis under the mentioned framework. Herza & Phillps (2017) defend that the adopted FS must portray the reliability of the dam design and account for the parameters' variability and uncertainties, so they must be adjusted for these two variables [10], proposing a gradual increase (7 to 12%, around ΔFS = +0,1) of the FS with the increase of risk.

Table 1 – Proposed FS for different design scenarios

Design scenario	ANCOLD (2012)	ABNT 13028	CDA (2019)[(1)]	ICOLD (B194)[(2)]	Fell *et al.* (2015)[(3)(4)]	Duncan *et al.* (2014)[(3)]
End of construction	-	1,3 (c)	> 1,3 (f)	1,5 (h)	1,3	1,3
Long-term and/or drained	1,5	1,5 (d)	1,5	1,5 (i)	1,5	1,5
Short-term and/or undrained	1,3 or 1,5 (a)	1,3 (e)	-	1,1 or 1,3-1,5 (j)	-	-
Reservoir drawdown	-	1,1	1,2-1,3	- (k)	1,3	1,0-1,2
Seismic (pseudo-static)	-	1,1	1,0	-	-	-
Post-seismic / -liquefaction	1,0-1,2 (b)	-	1,2	1,1 (l)	-	-

[(1)] CDA (2013) states that lower FS can be acceptable for existing structures with demonstrated performance; [(2)] B194 states that higher FS can be considered to account for uncertainties over time, uncertainties in in situ conditions, presence of weak soils or potential for changes to soil properties; [(3)] For water dams; [(4)] One can admit -0,1 for well monitored and well performing existing dams and +0,1 to +0,5 to account for uncertainty in material strength (e.g. strain-softening, contractive materials), risk of progressive failure and lack of knowledge about the dam and materials. (a) depending on the potential loss of containment; (b) Related with the confidence on estimated residual shear strength and should consider the ultimate (drained or undrained, residual or liquefied) of the soil; (c) Not applicable to intermediary raising stages of the dam, only the final one; (d) Normal operation; (e) Operation with extreme flux net; (f) Applicable also during construction. Depends on risk assessment during construction; (g) Considering liquefied resistance for tailings; (h) 1,3 can be used for the starter dike, before water and tailings are impounded; (i) One only FS for "static conditions"; (j) 1,1 for static analysis using large-strain residual undrained strength; using peak undrained strength, 1,3 or higher with no or minimum strain-softening and 1,5 with high strain-softening; (k) The designer should specify a suitable FS, accounting for frequency and magnitude of the drawdown and the consequences of failure; (l) Using "post-liquefaction" and/or residual shear strength.

A more recent proposal, which tries to reconciliate the simple deterministic approach and the risk-based framework, already taking into consideration the several existent norms and guidelines for tailing dams, is presented by Schnaid *et al.* (2020) [40]. The authors show that despite the increased attention towards previously neglected issues (e.g., post-peak strength, consequence of failure of the design, etc), the mining community still follows a deterministic approach. Based on its bibliographic review, for a normal operation situation, a FS of 1,3 is

only endorsed by some countries, whereas a value of 1,5 is generally accepted. However, the former can be justified for short-term, low consequence structures and a higher value then 1,5 may be considered for high consequence dams. A FS greater than 1,5 is also justified when there are potentially liquefiable soils in the dam, due to the great uncertainty related with the characterization of its strength and behaviour, a prominent issue in upstream raised dams. With this in mind, Schnaid *et al.* suggest a comprehensible classification which correlates three uncertainty levels, based on the ICOLD criteria (negligible, intermediate and high/critical), with three failure consequence categories, based on the ICMM criteria (relevant, high and catastrophic) [40]. The proposed FS hold for both static and undrained strength, new or existent structures, and any design situation whether short- or long-term. An important note is that the study did not cover seismic stability analysis.

Table 2 presents some proposals of FS for different uncertainty levels and consequence categories from different bibliographic sources. To construct this table the description of the uncertainty levels and consequence category terms were uniformized across the sources, since some of them presented different terms. Even though not specifically for tailings dams, Duncan *et al.* (2014) [13] present a similar proposal, therefore it was included in the table. In Table 2 one can see that, in general, a minimum value of 1,5 must be adopted for tailing dams slope stability, with only a few cases related with low consequence of failure going below that.

Table 2 – Proposed FS for different uncertainty levels and consequence categories

Level of uncertainty	Consequence category	Duncan *et al.* (2014)[(1)]	Herza & Phillips (2017)	Schnaid *et al.* (2020)[(2)]	Stark & Ruffing (2017)[(1)(3)]	
		2D	2D	2D	2D	3D[(4)]
Small	Low	1,25	1,3	1,5	1,3	1,4 (eq.3)
Small	Significant	-	1,5	1,5	1,3-1,5	1,5-1,7 (eq.2)
Small	High	1,5	1,5	1,5	1,5	1,7 (eq.1)
Medium	Low	-	1,4	1,5	-	-
Medium	Significant	-	1,5	1,5	-	-
Medium	High	-	1,7	1,5	-	-
Large	Low	1,5	1,5	1,5	1,5	1,7 (eq.2)
Large	Significant	-	1,6	1,7	1,5-2,0	1,7-2,3 (eq.1)
Large	High	2,0	(a)	1,8	2,0	2,3 (eq.1)

[(1)] FS for slopes in general; [(2)] Level of uncertainties based on ICOLD, corresponding to "negligible", "intermediate" and "high or critical". Consequence categories based on ICMM, corresponding to "Relevant", "High" and "Catastrophic"; [(3)] Consequences categories are evaluated separately by threat to human lives and downstream damage. Values showed are based on the, so they might change depending on the geometry and geology of the problem; [(4)] These FS are not prescriptive, but rather proposals based on minimum FS, following the proposed equations; (a) Level of uncertainties should be reduced prior to design.

As discussed previously, although a 3D LE analysis might be more representative of the problem at hand, it also encompasses a higher degree of uncertainty, with several authors suggesting that different minimum FS must be used for 3D analysis. Agreeing with that, Stark & Ruffing (2017) proposed a risk-based approach similar to the abovementioned authors, but

only with two extreme levels of uncertainty, adapted for 3D analysis [36]. The suggested $FS_{3D,min}$, which are based on the $R_{3D/2D}$ therefore might change with the geometry and geology of the problem, are 8 to 15% higher than the minimum FS_{2D} (Table 2). This approach, though, does not respect tailings dams *per se*, but rather slopes in general. Until today there are no official prescriptions for minimum FS to adopt in 3D dam stability analysis.

3. AN APPROACH FOR THE DEFINITION OF MINIMUM FS_{3D}

Duncan *et al.* (2014) [13] and Fredlund (2014) [24] recognized that there was an increased interest in, and sometimes misuse of, 3D LE methods, insofar they lead to higher FS, neglecting that the engineering practice is based on 2D analysis, which continues to be the case. On the other hand, from a safety review point of view, taking into consideration the international normative state of the art for tailings dams [33] [34] [35], the prescribed FS associated with LE analysis is here to stay, both for its practical simplicity and the possibility to estimate the dam's margin of safety in a certain scenario. Furthermore, with the available commercial software it is easier to perform 3D equilibrium analysis and, in the absence of a FS_{3D} prescription, there is he risk that 3D analysis results will be compared with regulatory FS_{2D}, hence regulatory FS_{3D} are warranted.

Herewith, a simple approach to define minimum FS_{3D} is put forward, one that can be used as a starting point for the discussions around this topic. This approach combines several proposals for the establishment of minimum FS_{2D} for tailings dams following a risk-based approach, as well as a methodology to convert those into minimum FS_{3D}. The proposed approach takes the following steps (summarized in Fig. 4, Table 3 and Table 4):

1) Select prescribed FS from several or a specific international norms (e.g. ANCOLD, CDA) for each mandatory design scenario, taking them as standard minimum FS_{2D};
2) Folow Herza & Phillips (2017) [10] and Schnaid *et al.* (2020) [40] proposals to adapt the above mentioned FS_{2D} into a risk-based approach, using the general categories presented in Table 2 for uncertainty and consequence levels, as follows:
    a. For the lowest class (small uncertainty and low consequence), one can reduce the prescribed (regulatory) FS_{2D} by –0,1, following Fell *et al.* (2015) [39]. This reduction must only be used for well instrumented and characterized existing dams or new dams with a robust, well-founded, data-based design and the minimum FS_{2D} cannot be below 1,0;
    b. For small uncertainty levels but significant or high consequence categories, the prescribed FS_{2D} can be adopted;
    c. For the following levels, a stepped increase of $\Delta FS = +0,1$, similar to Herza & Philips proposal, can be adopted. It is proposed that the lower consequence

levels of each uncertainty category take the previous uncertainty level as reference and that the highest risk category (large uncertainty and high consequence) is increased by +0,2 (see Table 3 notes).

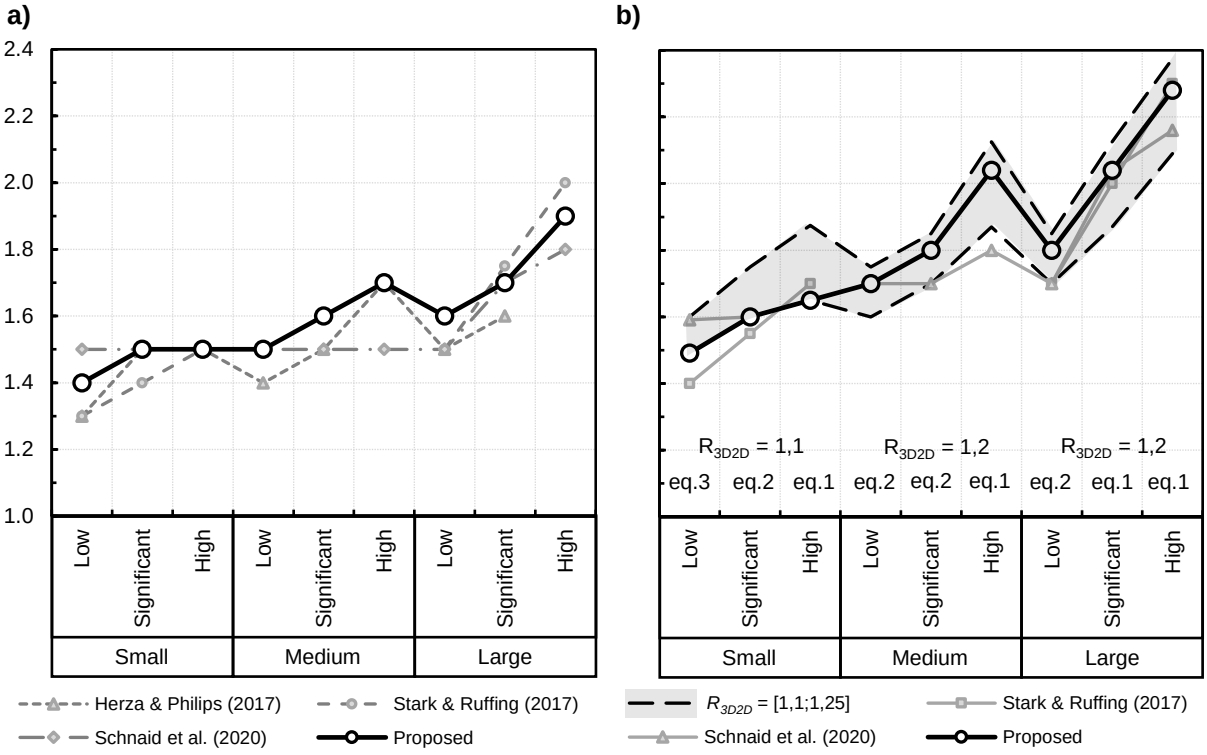


Fig. 4 – Risk-based minimum FS for long-term and/or drained conditions: a) FS_{2D}; b) FS_{3D}

3) Use Stark & Ruffing (2017) [36] equations (1 to 3) to transform the established FS_{2D} into FS_{3D}. These equations encompass different degrees of conservativism, which are suitable for the proposed risk-based approach. They depend, though, on the R_{3D2D} parameter, for which there is no standard formula or all-encompassing value to be adopted. Below are some guidelines on what to be adopted:
    a. Stark & Ruffing proposed to use the least conservative equation (eq.3) only in cases with small uncertainty and low consequence, whereas the most conservative one (eq.1) anytime the consequence category is high. For the rest of the cases, eq.2 might be suitable, with the exception of large uncertainty and significant consequence where eq.1 is also warranted (see Table 4 notes);
    b. The R_{3D2D} parameter describes the general gain of resistance by moving from a 3D to a 2D LE stability analysis, therefore it is case-dependent. However, much has been concluded from the existing 3D-effects research ([18] to [22]). Based on real slope failures, Akhtar (2011) [3] states that, typically, rotational slides have a ratio of depth-to-length of failure (H/L) between 0,15 and 0,33 and a W/L ratio between 0,8 to 1,2. From that, one can estimate that the W/H ratio is typically comprised between 2,4 and 8,0. For W/H ratios between 2 and 3,

studies show that R_{3D2D} can reach values up to 1,3. Moreover, R_{3D2D} = 1,0 is only achieved for a virtually infinite slope, thus, considering that in tailings dams the unstable mass is always finite, one can consider that R_{3D2D} is typically ≥ 1,1.

Table 3 – Minimum FS_{2D} following a risk-based approach

Design scenario	FS_{2D} (Level of Uncertainty / Consequence category)								
	Small			Significant			Large		
	Low (1)(2)	Med. (1)	High (1)	Low (3)	Med. (4)	High (4)	Low (3)	Med. (4)	High (5)
End of construction	1,2	**1,3**	**1,3**	1,3	1,4	1,5	1,4	1,5	1,7
Long-term and/or drained	1,4	**1,5**	**1,5**	1,5	1,6	1,7	1,6	1,7	1,9
Short-term and/or undrained	1,2	**1,3**	**1,3**	1,3	1,4	1,5	1,4	1,5	1,7
Reservoir drawdown	1,2	**1,2**	**1,2**	1,2	1,3	1,4	1,3	1,4	1,6
Seismic (pseudo-static)	1,0	**1,1**	**1,1**	1,1	1,2	1,3	1,2	1,3	1,5
Post-seismic / -liquefaction(6)	1,0	**1,1**	**1,1**	1,1	1,2	1,3	1,2	1,3	1,5

(1) Based on the several international norms (ANCOLD, CDA, ABNT, ICOLD); (2) –0,1 due to low levels of uncertainty and consequence, following Fell *et al.* (2015). These values should only be used for well instrumented and characterized existing dams or new dams with a robust, well-founded, data-based design; (3) Equivalent consequence category for the lower level of uncertainty +0,1 (8 to 10% more); (4) Previous level +0,1 (7 to 10% more); (5) Previous level +0,2 (12 to 15% more); (6) It is suggested that post-liquefaction LE analysis should only be used as a screening analysis and should be accompanied by stress-strain analysis to take into account the stress and resistance distribution as well as the strain-effects in the structure.

Table 4 – Minimum FS_{3D} following a risk-based approach

Design scenario	FS_{2D} (3)	FS_{3D} (Level of Uncertainty / Consequence category)								
		Small(1)			Significant(2)			Large(2)		
		Low (4)	Med. (5)	High (5)	Low (5)	Med. (5)	High (6)	Low (5)	Med. (6)	High (6)
End of construction	**1,3**	1,3	1,4	1,4	1,5	1,6	1,8	1,6	1,8	2,0
Long-term and/or drained	**1,5**	1,5	1,6	1,6	1,7	1,8	2,0	1,8	2,0	2,3
Short-term and/or undrained	**1,3**	1,3	1,4	1,4	1,5	1,6	1,8	1,6	1,8	2,0
Reservoir drawdown	**1,2**	1,2	1,3	1,3	1,4	1,5	1,7	1,5	1,7	1,9
Seismic (pseudo-static)	**1,1**	1,1	1,2	1,2	1,3	1,4	1,6	1,4	1,6	1,8
Post-seismic / -liquefaction(7)	**1,1**	1,1	1,2	1,2	1,3	1,4	1,6	1,4	1,6	1,8

(1) R_{3D2D} = 1,1; (2) R_{3D2D} = 1,2; (3) Based on the several international norms (ANCOLD, CDA, ABNT, ICOLD); (4) Using eq.3; (5) Using eq.2; (6) Using eq.1; (7) It is suggested that post-liquefaction LE analysis should only be used as a screening analysis and should be accompanied by stress-strain analysis to take into account the stress and resistance distribution as well as the strain-effects in the structure.

Fig. 4, Table 3 and Table 4 present an application of the proposed approach, comparing the minimum FS with the ones proposed by the referenced authors. In this case, the prescribed minimum FS_{2D} for each design scenario were defined (Step 1) taking into consideration all the existing international norms (ANCOLD, ABNT, CDA, ICOLD). Next, the minimum FS_{2D} for each risk category (Step 2) was defined as explained above (see notes of Table 3 and Fig. 4a). Finally, to define the minimum FS_{3D} (Step 3), besides the use of Stark & Ruffing equations as explained above (see Table 4 notes), R_{3D2D} = 1,1 was adopted for small consequence levels and R_{3D2D} = 1,2 for the two other consequence levels (Fig. 4b). Fig. 4 presents the comparison

for the long-term and/or drained condition, where one can observe that the proposed minimum FS_{2D} (a) and FS_{3D} (b) are in accordance with the literature proposals. Table 3 and Table 4 show the estimated minimum FS for all design scenarios and risk categories.

There are two important points to take into consideration regarding the use of this approach. First, Schnaid *et al.* [40] already made an equivalence of the proposed risk categories with ICOLD's risk and ICMM's consequence categories. Hence, this can be easily applied to certain geographies, such as Brazil and Canada, based on their legislation [34] [35], which already take into account those international guidelines and good-practices. Second, even though Stark & Ruffing [36] research is focused on translational slides, there is a 50-years long research on the 3D effects on slope stability analysis ([18] to [22]) that support similar findings for rotational slides, namely in what concerns the R_{3D2D} parameter. Cautious is needed when using Stark & Ruffing curves not just for rotational slides, but also for more three-dimensionally constrained structures, such as dams in narrow valleys. However, all 3D-effects studies point to a general FS increase in a 3D LE stability analysis, thus to a R_{3D2D} higher than 1,0. Hence, Stark & Ruffing equations, which only take into account this parameter, can be used universally to encompass different degrees of conservatism when adapting minimum FS_{2D} into FS_{3D}.

Finally, the presented FS are not prescriptive. In fact, the proposed approach can be adapted using specific regulatory FS and/or different weights both for the risk classes and the R_{3D2D} parameter, depending on how conservative one needs to be, i.e., on how big of a margin of safety one wants to impose. Indeed, the methodology can be used in a flexible way, where certain variables can be defined case-by-case and only certain steps are considered.

4. CONCLUSIONS

The routinely use of 3D slope stability analysis for dam safety evaluations has been growing over time due to the increase of computer power, adapted methodologies and the benefits the extra dimension brings into the analysis, namely the better representativity of the problem and the increase of FS. However, despite its benefits, a 3D slope stability analysis must take into consideration all the uncertainties related with the problem, especially the one inherent to the extra dimension. This is even more important when dealing with tailings dams where the level of uncertainties, due to the variability of the mechanical and strength behaviour of the tailings, can be large. In this paper the several sources of uncertainty in a 3D stability analysis were discussed, from the methods of analysis used to the absence of regulatory FS_{3D}, focusing on tailings dam safety evaluation. The literature review suggests that:

- A 3D slope stability analysis is more representative of the geometry of the problem, especially in cases with great three-dimensional constraints (e.g. narrow valleys), leading to higher FS since it considers the resistance along the surface lateral walls;

- The margin of safety in an LE analysis (FS) to be adopted in each design scenario is case-dependent, function of the uncertainties of the problem, namely the material's shear resistance, and the consequence of failure of the structure;
- The minimum (regulatory) FS_{3D} must be higher than the minimum FS prescribed in the existing international norms and guidelines, insofar the latter were established based on a long-time experience with 2D analysis.

Currently, no guidelines or normative documents prescribing, or help defining minimum FS_{3D}, exist. To the extent that LE analysis still is – and seems it will be, at least in the near future – the method routinely used for dam stability analysis, the definition of minimum FS_{3D} is an important step forward for dam design and safety evaluation, even more for tailings dams considering the large consequences of failure associated with them.

Several authors propose comprehensive frameworks to establish FS within a risk-based approach for tailings dams and an effort is being made to propose methods to define minimum (regulatory) FS_{3D}, based on the $R_{3D/2D}$. These proposals were followed to establish a simple risk-based approach to define minimum FS_{3D}, divided in three steps: 1) definition/adoption of standard minimum FS_{2D} for each design scenario; 2) transformation of those FS_{2D} for different risk categories; 3) transposition of the risk-based FS_{2D} into FS_{3D}. The approach is flexible enough to admit the use of different parameters, which can be adapted in a case-to-case basis, considering both the uncertainties around the structure and the consequences of its rupture, following not just international standards and guidelines, but also already proposed risk-based approaches for tailings dams. The proposal presented in this paper can be a good starting point for the necessary discussions around this important topic.

REFERENCES

[1] CDA (2013). Dam Safety Guidelines 2007 (2013 Edition).

[2] ICOLD (2022). Tailings Dam Safety. Bulletin No. 194, Version 1.0 (Final Draft).

[3] Akhtar, K. (2011). Three Dimensional Slope Stability Analyses For Natural And Manmade Slopes, PhD Thesis, University of Illinois.

[4] Krahn, J. (2003). The 2001 R.M. Hardy Lecture: The limits of limit equilibrium analyses. Canadian Geotechnical Journal 40: 643-660.

[5] Duncan, J. M. (1996). State of the Art: Limit Equilibrium and Finite-Element Analysis of Slopes. Journal of Geotechnical Engineering 122(7): 577-596.

[6] Chen, Z. Y., Yin, J.H., Wang, Y.J. (2006). The Three-Dimensional Slope Stability Analysis: Recent Advances and a Forward Look. Advances in Earth Structures: Research to Practice: 1-42.

[7] McQuillan, A., Bar, N., Yacoub, T. (2022). On the comparison of 2D and 3D stability analyses of an anisotropic slope. The Evolution of Geotech - 25 Years of Innovation – Hammah et al. (Eds).

[8] Pedrosa, P. T., Velloso, R. Q., Sieira, A. C. C. F. (2023). Limit equilibrium analysis on two- and three-dimensional models. Geotecnia 158: 3-23 (in Portuguese).

[9] Roche, C. *et al.* (2017). Mine Tailings Storage: Safety Is No Accident. A UNEP Rapid Response Assessment. United Nations Environment Programme and GRID-Arendal, Nairobi and Arendal.

[10] Hezra, J.,Phillips, J. (2017) Design of dams for mining industry. Proceedings 85th Annual Meeting of International Commission on Large Dams, ICOLD, Prague.

[11] ICMM (2020). International Council of Mining & Metals Global Tailings Standard.

[12] Terzaghi, K. (1943). Theoretical Soil Mechanics. John Wiley & Sons, Inc., New York.

[13] Duncan, J. M., Wright, S. G., Brandon, T. L. (2014). Soil Strength and Slope Stability (2nd Edition). John Wiley & Sons, New Jersey.

[14] Terzaghi, K., Peck, R. B., Mesri, G. (1996). Soil Mechanics in Engineering Practice (third Edition). John Wiley & Sons, Inc., New York.

[15] Duncan, J. M., Wright, S. G. (1980). The accuracy of limit equilibrium methods of slope stability analysis. Engineering Geology 16: 5-17.

[16] Lambe, T. W., and Whitman, R. V. (1969). Soil mechanics. John Wiley & Sons, New York.

[17] Anagnosti, P. (1969). Three-dimensional stability of fill dams. Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 2: 275-280.

[18] Baligh, M. M., Azzouz, A. S. (1975). End effects on stability of cohesive slopes. Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, GT 11: 1105-1117.

[19] Hovland, H. J. (1977). Three-dimensional slope stability analysis method. Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, GT 9: 971-986.

[20] Chen, R. H., Chameau, J.-L. (1983). Three-dimensional limit equilibrium analysis of slopes. Géotechnique 32(1): 31-40.

[21] Cavounidis, S. (1987). On the ratio of factors of safety in slope stability analyses. Géotechnique 37(2): 207-210.

[22] Gens, A., Hutchinson, J. N., Cavounidis, S. (1988). Three-dimensional analysis of slides in cohesive soils. Géotechnique 38(1): 1-23.

[23] Stark, T. D., Eid, H. T. (1998). Performance of Three-dimensional Slope Stability Methods in Practice. Journal of Geotechnical and Geoenvironmental Engineering 124(11): 1049-1060.

[24] Fredlund, M. D. (2014). Is There a Movement Toward Three-Dimensional Slope Stability Analyses? GeoStrata Magazine 18(4): 22-15.

[25] Arellano, D., Stark, T. D. (2000). Importance of three-dimensional slope stability analysis in practice. Geo-Denver 2000 (Denver, Colorado, USA): 18-32.

[26] Akhtar, K., Stark, T.D. (2017). Importance of Side Resistance in a 3D Stability Analysis. Geotechnical Frontiers 2017 278: 285-293.

[27] Meyerhof, G. G. (1995). Development of geotechnical limit state design. Canadian Geotechnical Journal 32: 128-136.

[28] Bromhead, E. N. (1992). The Stability of Slopes. Taylor & Francis, London.

[29] ICMM (2021). International Council of Mining & Metals Tailings Management – Good practice guide.

[30] EN 1997-1:2004 + AC:2009. Eurocode 7 – Geotechnical design Part 1: General rules. CEN.

[31] APA (2018). Documentos Técnicos de Apoio ao Regulamento de Segurança de Barragens (in Portuguese).

[32] Bond, A., Harris A. (2008). Decoding Eurocode 7. Taylor & Francis, London.

[33] ANCOLD (2012). Guidelines on Tailings Dams - Planning, Design, Construction, Operation and Closure.

[34] CDA (2019). Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams (2019 Edition).

[35] ABNT NBR 13028:2024. Mining – Preparation and presentation of design of tailings, sediments and/or water dams – Requirements.

[36] Stark, T. D., Ruffing, D. G. (2017). Selecting Minimum Factors of Safety for 3D Slope Stability Analyses. Geo-Risk 2017: 259-266.

[37] Loehr, J. E., McCoy, B. F., Wright, G. S. (2004). Quasi-Three-Dimensional Slope Stability Analysis Method for General Sliding Bodies. Journal of Geotechnical and Geoenvironmental Engineering 130(6): 551-560.

[38] ICOLD (2011). Improving Tailings Dam Safety. Critical Aspects of Management, Design, Operation and Closure. Bulletin No. 139.

[39] Fell, R., MacGregor, P., Stapledon, D., Bell, G., Foster, M. (2015). Geotechnical Engineering of Dams (2nd edition). CRC Press, London, UK.

[40] Schnaid, F., De Mello, L. G. F. S., Dzialoszynski, B.S. (2020). Guidelines and recommendations on minimum factors of safety for slope stability of tailings dams. Soils and Rocks 43(3): 369-395.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EVALUATING 3D EFFECTS ON TAILINGS DAM STABILITY: A COMPARATIVE ANALYSIS USING FS_{3D}/FS_{2D} RATIOS

Raul Conceição[1]
TPF Consultores

Gonçalo Ferreira[2]
TPF Consultores

João Camões Lourenço[3]
TPF Consultores;
University of Coimbra

Henrique Lopes[4]
TPF Consultores

ABSTRACT

Tailings dams are unique structures due to the materials they store and the construction methods employed, often resulting in complex three-dimensional (3D) problems. Most current slope stability analyses neglect the 3D effects without significant consequences. However, certain conditions, such as the shape of the valley where the dam is founded, the spatial variability of the tailings' resistance, and the presence of internal dykes may render the 2D simplification inadequate. 3D modelling is essential in such cases to accurately simulate the sliding mass and involved resistance. Additionally, it is well-established that the Factor of Safety (FS) obtained from a 3D limit equilibrium analysis is typically higher than that from an equivalent 2D analysis.

This paper presents a parametric study of a 3D limit equilibrium analysis by simulating a tailings dam raised by the upstream method. The study evaluates the effects of the dam's height and valley shape on overall stability. Using a straightforward and practical methodology based on current state-of-the-art techniques, specifically the FS_{3D} to FS_{2D} ratio ($R_{3D/2D}$), the results of the 3D and 2D analyses are compared.

Keywords: tailings dam, three-dimensional analysis, factor of safety, limit equilibrium method

[1]raul.conceicao@tpf.pt, Portugal; [2]goncalo.ferreira@tpf.pt, Portugal; [3]joao.lourenco@tpf.pt, Portugal; [4]henrique.lopes@tpf.pt, Portugal;

1. INTRODUCTION

Three-dimensional (3D) slope stability analysis has been used since the 1960s [1] to assess the effect of the sliding masses' tridimensionality and compare the results to those obtained in two-dimensional (2D) analyses. Since then, several different Limit Equilibrium (LE) procedures have been proposed, and different parametric studies have been performed to understand how the shape of the sliding mass and the different strength types of materials influenced the 3D response.

Despite more than 60 years of research, 3D LE slope stability analyses have only recently become more routinely used. This is a consequence of the fast-paced technological advance, namely processing power and the diffusion of commercial software for the purpose (e.g. Rocscience's Slide3 [2] and Geostudio's SLOPE3D [3]). Besides, 3D analyses are known to provide a higher Factor of Safety (FS), increasing interest in this type of analysis [4].

This is particularly relevant in dam safety assessment, especially for tailings dams, where the stability condition of the dam must be frequently evaluated. Moreover, the stability evaluations of this type of dams are still normatively associated with global FS (e.g. ANCOLD [5] and CDA [6]), which is computed via LE models.

Nonetheless, slope instability is always 3D, with a finite slip surface conditioned by the surrounding topographic, hydraulic and geologic features. In some cases, only a 3D analysis represents the problem at hand [7]. To introduce 3D LE analysis in a routine evaluation of dam stability, it is important to understand the 3D effects produced by the third dimension of the problem and the limitations of the currently available software.

Following the state-of-the-art research in 3D LE analysis, this paper presents a parametric study of the 3D valley effects in an upstream raised fully saturated tailings dam. Several geometric parameters are studied to account for the valley's confinement, and the results of the 3D analyses are compared with the 2D ones to understand the effect of each individual parameter.

2. SLOPE STABILITY ANALYSIS

2.1. 2D slope stability analysis

Slope stability analysis consists of the analysis of the slide of a mass along a well-defined slip surface. Based on the study of historic slope failure cases (e.g. [8]), three types of landslides can be defined [9]: i) rotational (Fig. 1a), typical in homogeneous soils and approximately with ellipsoidal shape; ii) translational slides (Fig. 1b), dominated by a weak layer or interface at the

base of the sliding mass and a back scarp [10]; iii) compound slides, a combination of both. The first two do not necessarily involve internal deformations, whereas the latter does.

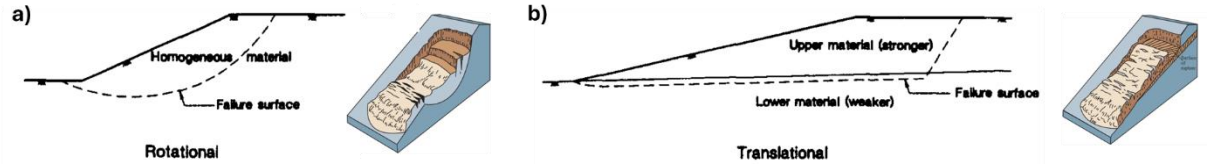


Fig. 1 – Failure modes for slopes (adapted from [10] and [8]): a) rotational; b) translational.

Slope stability is commonly analysed using LE methods, which evaluate static equilibrium without considering stress-strain paths or strain compatibility [11] [12]. The method of slices is the most widely used LE technique, dividing the sliding mass into vertical slices (Fig. 2a) subjected to gravitational, normal, and shear forces. The Factor of Safety (FS) is calculated as the ratio between the available shear strength and the applied shear loads along the slip surface [11].

Different methods (e.g., Bishop's, Janbu's, Morgenstern-Price, Spencer) vary based on equilibrium assumptions. Methods like Morgenstern-Price (1965) [14] and Spencer (1967) [15] satisfy both force and moment equilibrium. Despite methodological differences, studies [16] [17] show that all LE methods yield similar FS values (5-10% margin) when using consistent inputs, highlighting the uncertainty of soil strength.

2D analyses assume plane-strain conditions, neglecting the inherently 3D nature of slope failures. This simplification considers infinite geometry perpendicular to the sliding direction and ignores end effects, though 2D results are often conservative if the critical cross-section is selected—typically the section with the lowest FS. However, selecting this section can be subjective and is often replaced by the "maximum cross-section" (where the slope is highest), which may not be fully representative [18].

To address 2D limitations, simplified procedures estimate a 3D FS (FS_{3D}). One method calculates a weighted average FS across at least three parallel cross-sections, weighted by the sliding mass of each section [17]. Another approach accounts for the mobilised shear strength between sections, incorporating influence widths [18]. While these methods approximate FS3D, they remain limited as they neglect inter-cross-section forces and 3D end effects—critical resisting components of the slip surface.

2.2. 3D slope stability analysis

Besides most landslides having typical 3D features, thus making a 3D model a better geometrical representation of the real problem, 3D LE analyses also better simulate the behaviour of the structure, namely the spatial variability of the geologic and geotechnical characteristics of the problem [19]. Hence, this type of analysis can be better suited for complex

topographic and parametric conditions [10], for heterogeneous soils and for 3D constraints (e.g. narrow valleys) [20]. There are practical cases where 3D analyses are the most adequate, namely for curved slopes, when the 3D effects are too great for being neglected and for back-calculated strength-estimation [7]. The latter point is particularly important, for it is well-known that 3D stability analyses lead to higher FS than 2D ones [21] [20] [4] [22], as high as two times more [23]. Therefore, it might be anti-conservative to back-calculate soil strength using 2D analysis.

Anagnosti (1969) [1] was interested in evaluating the stability of earth and rock-fill dams in narrow valleys or canyons. For that purpose, the author developed a method to estimate the FS_{3D}, which can be considered a 3D extension of Morgenstern-Price's procedure, thus considering the inter-slice forces and ensuring equilibrium of both forces and moments. This procedure proposes dividing the sliding mass into 3D slices parallel to the alignment of the dam's crest. The author concluded that the FS_{3D} could be up to 50% higher than the FS_{2D}.

To evaluate the end effects of a 3D sliding mass in cohesive soils, Baligh & Azzouz (1975) [24] extend the circular arc shear failure method, where the slipping mass is considered a rigid body that rotates along the same axis, to the 3D space, considering a finite surface, instead the traditional infinitely long cylinder. The Ordinary Method of Slices (OMS) was adopted, thus neglecting the inter-slice forces. Using computer software for 3D analysis (STAB3D), the authors evaluate the 3D effects of vertical cut and angled slopes for different geometric lateral ends of the slip surface (vertical, conical or ellipsoidal). The author concludes that the 3D end effects can be as high as 40%, especially for small width (W) to height (H) ratios (W/H). The end effects for W/H bigger than four tend to be less pronounced. Gens *et al.* (1988) [20] followed a similar procedure to evaluate 3D effects in homogeneous, purely cohesive and saturated soils, showing that the FS_{3D} is always higher than the FS_{2D} (up to 70% higher) and that the effect of different end shapes converge to a 3D/2D ratio ($R3_{D/2D}$) of 1,0 for very wide failures (W/H > 20). Gens *et al.* conclude that 2D analyses are generally conservative unless a 2D back-calculated strength parameter is used. In the latter case, a 3D analysis is suggested.

Hovland (1977) [21] developed a general 3D slope stability method to evaluate the end effects of a 3D sliding mass for any soil's strength response and different sliding surface geometries, where the sliding mass is divided into several orthogonal columns, as an extension of the 2D method of slices, which was lately named as the *method of columns* (Fig. 2b). In his turn, Hovland neglected the inter-slice forces, using what can be considered an extension of Bishop's procedure. Chen & Chameau (1983) adopted Hovland's method but assumed that the inter-slice forces always acted with the same inclination throughout the whole failure mass in the direction of the movement.

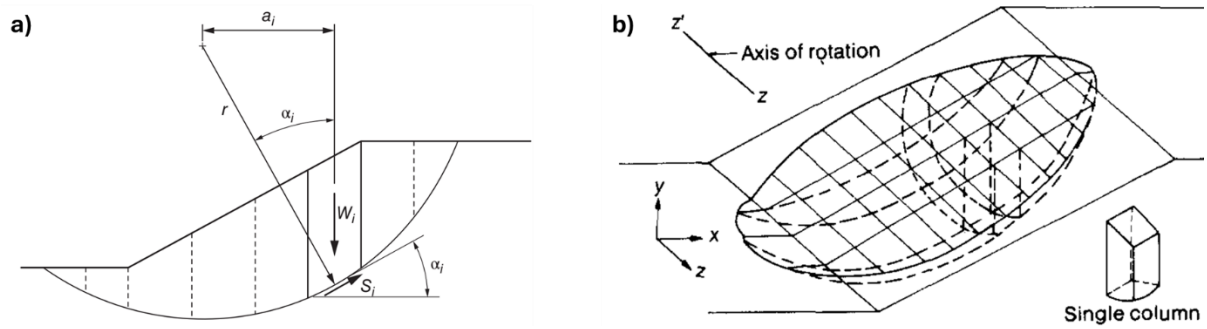


Fig. 2 – Limit equilibrium methods: a) 2D method of slices [7]; c) 3D method of columns [25].

Both Hovland and Chen & Chameau concluded that, in general, the FS_{3D} is greater than the FS_{2D} (up to 60% higher), with the latter also concluding that the $R3_{D/2D}$ approached unity for large W/H ratios. However, for cohesionless soils, in some cases, the $R3_{D/2D}$ can be smaller than 1,0. Cavounidis (1987) [26] refuted this conclusion, proving mathematically that the FS_{3D} is always greater than FS_{2D}: any proof of the contrary either implies a wrong choice of the critical slip surface in one or either of the methods or simplified assumptions neglecting essential aspects of the problem. Cavounidis also warns that the 2D critical slip surface may not be within the 3D critical slip surface.

While the authors mentioned above were concerned with rotational failures, Stark & Eid (1988) [10] used commercial software for 3D stability analysis (CLARA) to assess the 3D effects of translational failures, namely the side resistance of the lateral walls. For this purpose, the authors used a closed-form 3D mode of the translational failure (Fig. 3), where it was considered that the base was in contact with the mass with a weak layer in the foundation and that horizontal earth pressure was acting along the lateral walls of the mass. The FS_{3D} was evaluated with and without the contribution of the side resistance and compared with the FS_{2D} for the most representative cross-sections (Fig. 3a).

Stark & Eid's 3D analysis followed the method of columns to evaluate the static equilibrium and used Bishop and Janbu's (uncorrected) procedure. It was concluded that the side resistance plays an important role in the overall stability of the failure and that even with the minimum amount of side resistance, translational failures present the greatest $R_{3D/2D}$, vis-à-vis rotational failures. This is because of the weight of the quasi-vertical lateral walls of translational failures in the sum of mobilised shear resistance, which is neglected in a 2D analysis.

Arellano & Stark (2000) [27] used the abovementioned model to do a parametric study of the 3D effects in translational failures. The study aimed to create a practical abacus to assess the need for a 3D slope stability analysis to account for these effects. By varying the height and the width of the model, these authors evaluate the $R_{3D/2D}$ for different W/H and slope angles (Fig. 4a). Arellano & Stark followed Janbu's simplified procedure and assumed the at-rest

pressure coefficient (K_0) for the calculation of the side resistance. Akhtar & Stark (2017) [22] updated Arellano & Stark's parametric study considering an earth pressure coefficient in-between K_0 and the active (K_a), finding lower values of $R_{3D/2D}$ (Fig. 4b). Notwithstanding, both studies show that the 3D effects are high for more constrained slides ($R_{3D/2D}$ as high as 2,0 for W/H lower than two) and somehow negligible for W/H greater than six (Fig. 4). They also show that the effects are greater for gentle slopes (i.e., shallower slip surfaces).

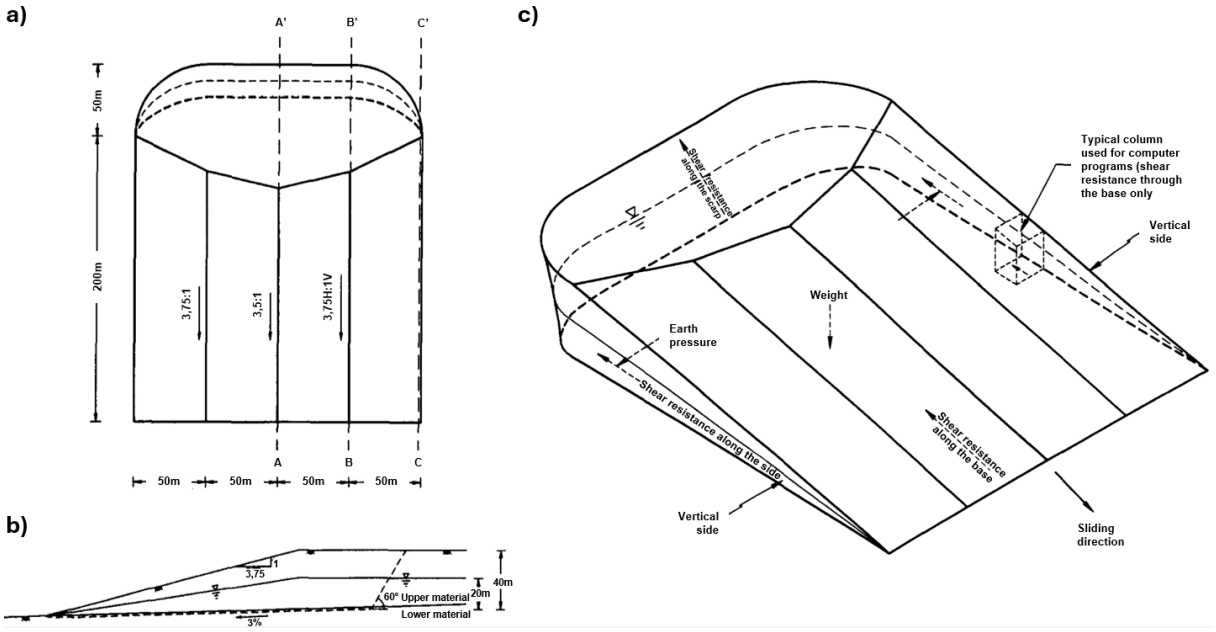


Fig. 3 – Stark & Eid's model [10]: a) plan view; b) representative cross-section; c) 3D view.

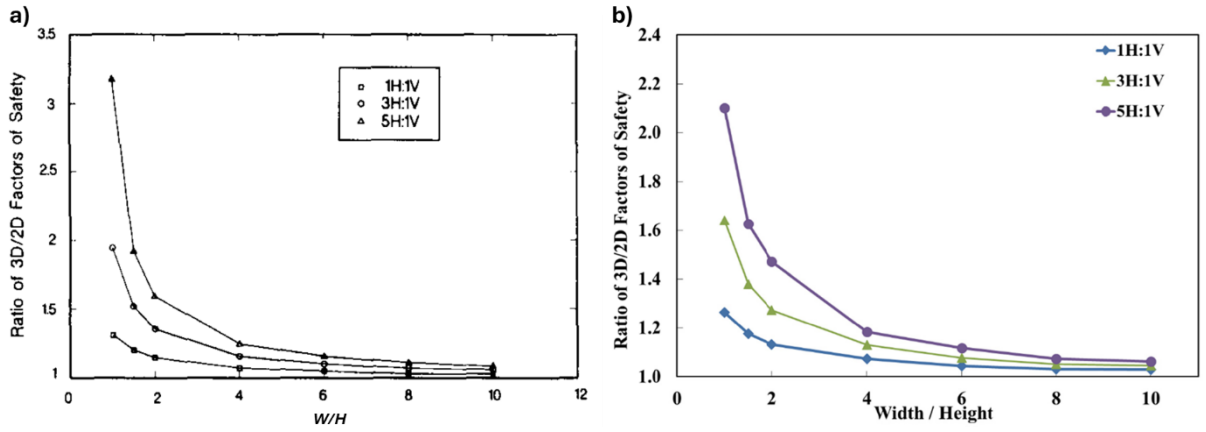


Fig. 4 – End effects for different slope angles: a) Arellano & Stark [27]; b) Akhtar & Stark [22].

2.3. Evaluating 3D effects

It has already been shown that 3D stability analyses lead to (sometimes significantly) higher FS than 2D analyses, especially for low W/H ratios. However, there are still no guidelines for regulatory FS for 3D LE analysis ($FS_{3D,min}$). In fact, Akhtar & Stark state that the FS_{3D} should

not be compared to 2D minimum (regulatory) FS ($FS_{2D,min}$) precisely due to the additional shear resistance generated along the sides of the sliding surface, thus recommending that a greater regulatory FS for 3D analysis should be used [22]. For that reason, Stark & Ruffing (2017) [23] propose a case-by-case methodology for computing a $FS_{3D,min}$, based on the work started by Stark & Eid and developed by the aforementioned authors.

The proposed methodology uses the Akhtar & Stark $R_{3D/2D}$–W/H curves (Fig. 4b) as a starting point to estimate the extra margin of safety to add to the $FS_{2D,min}$ based on the geometry of the problem (W/H and slope angle) and the $R_{3D/2D}$. If the user's problem is similar to the one analysed by the authors, the curve can be used directly, however if the geometrical problem is different it is suggested to create a characteristic curve for the structure being analysed. The authors then present three different equations to estimate a $FS_{3D,min}$, all functions of the $FS_{2D,min}$ for the design case at hand and the $R_{3D/2D}$ of the structure for the W/H of the critical slip mass.

Stark & Ruffing's methodology comes from a decades-long study that is focused on translational slides, hence caution must be taken when applying it to rotational ones. Nonetheless, the logic behind it and the procedure can be adapted for other types of sliding masses as long as the characteristic $R_{3D/2D}$–W/H curve can be adequately estimated. Moreover, the methodology can be adapted to estimate the gain coming from the three-dimensionality of the problem itself, i.e., it can be used to separate what is simply a matter of passing from a 2D analysis to a 3D one (end effects) and the gain coming from the actual geometrical constraints of the system, such as narrow and/or non-linear valley, internal structures, strength and pore pressure variability, etc.

Using a commercial software, which follows the methods of slices (*Slide2*) and columns (*Slide3*), respectively, McQuillan *et al.* (2022) compared the results of 2D and 3D analysis of an anisotropic slope of a highly bedded open pit mine [28]. The study presents a parametric evaluation of the strength model of the materials and some input features of the software, such as the slip surface search method algorithms (Cuckoo's search and Particle Swarm), the number of (initial) nests and the minimum slope depth. To quantify the difference between the 2D and the 3D analysis, the authors used several 3D extruded models of a representative 2D cross-section, each with a different width (50, 100, 200 and 400 m), showing that a 3D analysis consistently produces a higher FS and that the shorter the width of the extruded model the higher the FS, with a FS_{3D} similar to a FS_{2D} for the longer widths. Other important conclusions of the McQuillan *et al.* study are that the borders of the model influence the limits of the slipping surface, with the latter avoiding the formers, and that the default settings of the software may lead to erroneous results, failing to find the critical slip surface. In fact, even though existing slip surface search algorithms can give sufficiently good results from a practical point of view, they are not guaranteed to find the unique critical surface [29].

3. PARAMETRIC ANALYSIS: EVALUATION OF THE SHAPE OF THE VALLEY ON THE OVERALL STABILITY OF A DAM

3.1. Purpose

This analysis aims to evaluate the influence of the valley's shape where the dam is founded on a rotational slide scenario in an upstream raised tailings dam. The study was developed using commercial software (*Slide*, from *Rocscience*), which comprises several optimised search algorithms to find the critical slip surface and different 3D LE procedures, including the ones that satisfy both forces and momentum equilibrium and allow for different slip surface shapes. In fact, the literature review shows that the evaluation of the 3D effects in slope stability analysis has been mainly concerned with pre-defined closed-form shapes of the slip surface [1] [21] [24] [25] [20] [10] and more simple LE procedures [21] [25] [22]. It also shows that the state-of-the-art is concerned with translational landslides [10] [27] [22] [23].

The carried-out analysis evaluates not just the width but the shape of the geometric constraint of the dam, considering both trapezoidal and triangular shape valleys, through a parametric evaluation where both the slope of the abutments (β) and the width of the thalweg (L_M) varies. Contrary to the studies with pre-defined closed-form slip surfaces, in the present analysis, only the valley's and the embankment's geometry is defined, allowing the software to search for the critical slip surface.

3.2. Geometric models used

The models of the upstream tailings dam comprise two materials: dykes and tailings. The foundation and the abutments, defined only by plane surfaces, were considered infinitely resistant. The starter dyke is 10 m high, whereas the upstream dikes are 11 m (1,0 m overlap). All dykes were modelled considering 3 m width crest and slopes of 1V:2H upstream and 1V:2.5H downstream. The top of the tailing’s reservoir was 1 m below the dam’s crest elevation.

The direction of the thalweg is perpendicular to the dam axis, horizontal at the base of the starter dyke and sloped upwards with a ratio of 1V:10H in the upstream direction. The water level applied to the models was positioned at the top of the tailings, in the reservoir, diving downstream 5 meters before the last dyke, with the same angle as the rising dyke’s downstream slope, as shown in Fig. 5. The modelling considered a total dam’s heights (H_M) of 30, 50 and 70 m.

Taking into account the third dimension of the model, two more variables were introduced: abutments slope angle with the horizontal (β) and thalweg width (L_M). With these two variables and the development of the thalweg (defined previously), it was possible to create different

surfaces representing different valleys. β varies between 30º, 45º and 60º, and L_M varies between 0, 30 and 70 m.

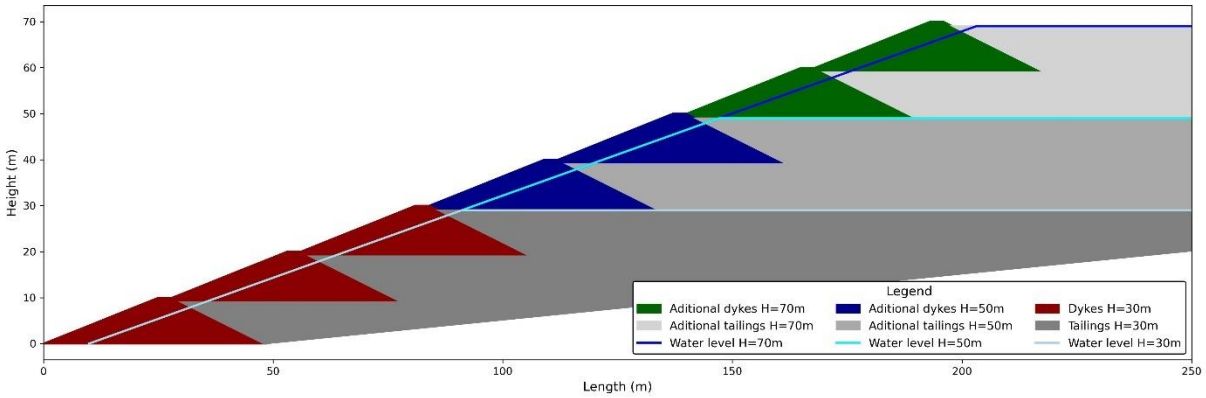


Fig. 5 – Representative 2D cross-section of the dam models (geometry and water level).

The fitting of the dam in the valley increases with the increase of β and the reduction of LM. Following the possible combinations for the two variables, 9 different surfaces were created. Those surfaces combined with the 3 heights of dams generated a total of 27 models. The surfaces (abutments and water level) and solids (dikes and tailings) were created using Autodesk Civil 3D software. The final models were made in Slide3 (Fig. 6).

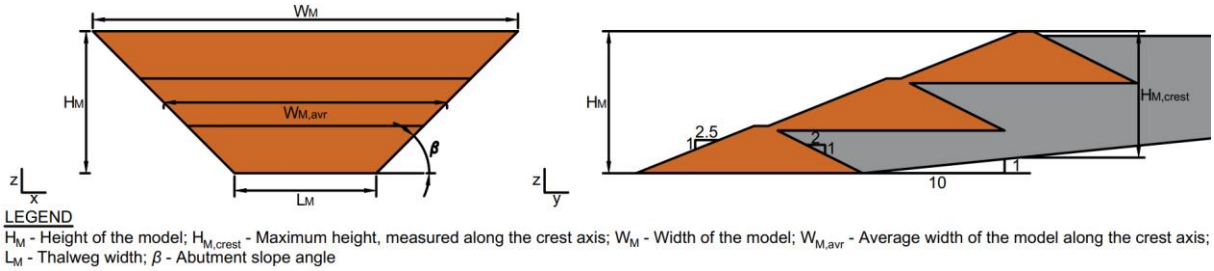


Fig. 6 – Scheme of the 3D models used in the analysis with main dimensions.

The behaviour considered for the dykes was drained, both over and under the water level. In the tailings, it was considered a drained condition above the water level and an undrained condition below the water level. For both materials, constant specific weights were adopted. The Mohr-Coulomb failure criteria defined the drained shear strength, represented by the friction angle (ϕ') and cohesion (c'). The undrained shear strength was defined through the undrained shear strength ratio (s_u/σ'_v), following the SHANSEP model. Table 1 presents the summary of the geotechnical parameters.

Slope stability analyses were conducted in 2D and 3D LE models. For the 2D models, the central cross-section of the 3D models was adopted. Due to the models' symmetry, three cross sections were analysed, one for each dam height (see Fig. 5). The Global Limit Equilibrium (GLE)/Morgenstern-Price was used in both 2D and 3D analyses. In 2D analyses, the critical surfaces were searched using the Auto Refine method, with circular type slip surfaces, without

optimisation. A filter for a minimum depth of 10 m was applied to avoid small surfaces and ensure that the surfaces occurred below the dykes, being this way critical for the global stability of the dam.

Table 1 – Geotechnical parameters

Description	Colour	γ (kN/m^3)	s_u/σ'_v	ϕ' (°)	c' (kPa)
Dykes		21	-	35	0
Tailings		27	0,25	30	0

In 3D models, the search method used was the Intelligent Search. This method starts by generating preliminary surfaces that will allow the creation of a surface safety map with regions of interest. Based on that map, the algorithm will define clusters in close regions with similar safety factors, on which the search will be focused until convergence in each one is found using particle swarm optimisation. As entry parameters, one can define the refinement of the search, which is composed by the number of preliminary surfaces, the number of clusters and the number of safety factors capped, being the last limited to the maximum number of clusters and independent from computer calculation effort. In the case study, moderate refinement was chosen, defined by 3 500 preliminary surfaces and 8 clusters. Like in the 2D analysis, no optimisation was used, and a minimum depth of 10 m filter was applied to all models.

3.3. Methodology

To evaluate the 3D effects, we followed *grosso modo* Stark & Ruffing's proposed methodology [23]. However, these authors focused on translational slides and used only the geometry of the slip surface as a model (Fig. 3). Thus, the slip surface was pre-determined, and no influence of the surrounding geometry was considered. In other words, the authors, in line with the previous 3D effects research, were only concerned with the geometry of the slip surface.

The geometry of the sliding mass stems from the confinement of the whole structure around it. Hence, an attempt was made to understand the influence of the geometry of the model (H_M, $W_{M,\,avr}$), i.e., the valley's geometry, in the slipping mass geometry (H_{SM}, W_{SM}, V_{SM}). The main difference in this study is thus the consideration of the valley (model) dimensions. The following steps evaluated the 3D effects:

1. Compute the lowest FS_{3D} (i.e., the FS for the critical 3D surface) with a 3D LE analysis;
2. Compute the lowest FS_{2D} (i.e., the FS for the critical 2D surface) with a 2D LE analysis;
3. Measure the W and the H of the slipping mass (W_{SM}, H_{SM}), as well as its volume (V);
4. Calculate the $W_{M,avr}/H_M$, W_{SM}/H_M, H_{SM}/H_M and V/H_M ratios;
5. Calculate the $R_{3D/2D}$ (= FS_{3D}/FS_{2D});
6. Define the dam's *characteristic* curve as follows:

a. Use the cross-section that leads to the lowest FS_{2D} and extrude it to different widths ($W_{M,e} = W_{M,avr}$) in a range of 0,5 to 10 times H_M;
b. For each extruded model, compute the $FS_{3D,e}$, $W_{SM,e}$, $H_{SM,e}$.
c. Define different characteristic curves with $W_{H,e}/H_M$ in each axis and $FS_{3D,e}/FS_{2D}$, $W_{M,e}/H_M$ and $H_{M,e}/H_M$ (FS_{2D} and H_M are constants).

7. Compare the $R_{3D/2D}$ of the 3D models with the $R_{3D/2D}$ obtained from the curve.

3.4. Results

Table 1 summarises the results of all 27 models used in the analysis, and In general, comparing the 3 dams' models in 3D, it is possible to notice that one of them is always stable (H = 30 m), and the other two are rarely stable (H = 50 m and H = 70 m) – by stable, one means that the FS_{3D} is above unit. Also, it is possible to notice that the factor of safety increases when the L_M reduces and β increases. This is expected insofar as the dam is more confined in the valley when L_M reduces and β increases; thus, the abutments contribute more significantly to the stability.

Table 2 summarises the extruded 3D models used to define the dams' characteristic curves.

Table 1 – Summary of the 3D models' input parameters and main results

Model	H_M (m)	β (°)	L_M (m)	$H_{M,crest}$ (m)(1)	$W_{M,avr}$ (m)(2)	FS_{2D}	FS_{3D}	$R_{3D/2D}$	W_{SM} (m)	H_{SM} (m)	Vol. (m^3)
1	30	30	0	26,55	46	0,85	1,35	1,59	34	24	16 965
2		30	30	26,55	76		1,08	1,27	69	26	54 163
3		30	70	26,55	116		1,05	1,24	104	24	73 349
4		45	0	26,55	27		1,50	1,76	24	24	8 792
5		45	30	26,55	57		1,13	1,32	59	26	38 652
6		45	70	26,55	97		1,05	1,24	99	25	73 415
7		60	0	26,55	15		1,75	2,06	25	13	3 978
8		60	30	26,55	45		1,20	1,41	44	26	24 545
9		60	70	26,55	85		1,06	1,25	78	26	58 254
10	50	30	0	40,95	71	0,65	0,91	1,41	70	36	81 859
11		30	30	40,95	101		0,83	1,28	79	46	130 437
12		30	70	40,95	141		0,78	1,20	127	45	243 478
13		45	0	40,95	41		1,15	1,78	61	25	40 742
14		45	30	40,95	71		0,89	1,38	82	37	99 054
15		45	70	40,95	111		0,78	1,21	97	46	174 704
16		60	0	40,95	24		1,31	2,03	35	25	21 521
17		60	30	40,95	54		0,97	1,50	56	36	61 125
18		60	70	40,95	94		0,83	1,29	78	46	125 866
19	70	30	0	55,35	96	0,60	0,80	1,33	120	47	199 795
20		30	30	55,35	126		0,75	1,24	126	55	299 050
21		30	70	55,35	166		0,69	1,15	133	67	421 189
22		45	0	55,35	55		0,90	1,49	74	36	86 425
23		45	30	55,35	85		0,80	1,33	98	45	178 440
24		45	70	55,35	125		0,73	1,21	114	55	283 945

Model	H_M (m)	β (°)	L_M (m)	$H_{M,crest}$ (m)(1)	$W_{M,avr}$ (m)(2)	FS_{2D}	FS_{3D}	$R_{3D/2D}$	W_{SM} (m)	H_{SM} (m)	Vol. (m^3)
25		60	0	55,35	32		1,06	1,76	44	36	54 697
26		60	30	55,35	62		0,86	1,42	68	45	113 312
27		60	70	55,35	102		0,75	1,24	100	54	232 708

Subscripts legend: SM – Slipping mass; M – Model
(1) Maximum height, measured along the crest axis; (2) Average model width along the crest axis.

In general, comparing the 3 dams' models in 3D, it is possible to notice that one of them is always stable (H = 30 m), and the other two are rarely stable (H = 50 m and H = 70 m) – by stable, one means that the FS_{3D} is above unit. Also, it is possible to notice that the factor of safety increases when the L_M reduces and β increases. This is expected insofar as the dam is more confined in the valley when L_M reduces and β increases; thus, the abutments contribute more significantly to the stability.

Table 2 – Summary of the extruded models' parameters and main results

$H_{M,e}$ (m)	$W_{M,e}$ (m)	$FS_{2D,e}$	$FS_{3D,e}$	$R_{3D/2D}$	W_{SM} (m)	H_{SM} (m)	L_{SM} (m)	W_{SM}/H_{SM}	H_{SM}/H_M	W_{SM}/W_M
30	20	0,85	1,59	1,87	19	23	52	0,8	0,77	0,95
	50		1,14	1,34	46	26	67	1,7	0,88	0,92
	75		1,05	1,23	72	25	67	2,9	0,84	0,96
	100		1,01	1,18	84	26	72	3,2	0,88	0,84
	150		0,95	1,12	134	25	67	5,4	0,83	0,89
	200		0,92	1,08	185	26	74	7,1	0,87	0,93
	500		0,90	1,06	426	25	71	16,8	0,85	0,85
	750		0,89	1,04	480	27	76	17,5	0,91	0,64
50	50	0,65	0,94	1,46	46	44	133	1,1	0,87	0,92
	75		0,82	1,28	65	46	121	1,4	0,92	0,87
	100		0,79	1,23	91	46	128	2,0	0,92	0,91
	150		0,74	1,14	135	46	130	2,9	0,92	0,90
	200		0,72	1,11	194	44	132	4,4	0,88	0,97
	500		0,68	1,05	394	45	142	8,8	0,90	0,79
	750		0,67	1,04	537	46	135	11,7	0,92	0,72
70	50	0,60	0,92	1,53	44	56	153	0,8	0,80	0,88
	75		0,77	1,28	66	55	143	1,2	0,79	0,88
	100		0,73	1,21	95	65	183	1,5	0,92	0,95
	150		0,69	1,15	135	65	182	2,1	0,92	0,90
	200		0,66	1,09	174	66	180	2,6	0,95	0,87
	500		0,63	1,04	457	67	210	6,8	0,96	0,91
	750		0,63	1,04	513	67	195	7,7	0,96	0,68

Subscripts legend: M – Model; SM – Slipping mass; M,e – Extruded model

Fig. 7 presents the dam's characteristic curve and compares the $R_{3D/2D}$ ratios obtained between the 3D models and the extruded models for each dam height. Similarly to previous studies, one can see that for low $W_{M,avr}/H_M$ ratios (< 2), the difference between 2D and 3D can be significant, whereas, for high ratios, FS_{3D} tends to the FS_{2D} value. Also, the shorter the dam is, the bigger the $R_{3D/2D}$ of the extruded models is.

Intuitively, if the $R_{3D/2D}$ of the 3D model is convergent with the $R_{3D/2D}$ of the extruded one, for the same W/H ratio, one expects that the 3D effects stem mainly from the slip surface side walls resistance. The expectation assumes that the slip surface is approximately the same in both cases. However, this may not be the case in every instance, insofar as the W/H ratio can be the same for different W and H combinations. Moreover, different 3D slip surfaces with the same W/H ratio can present substantially different geometries, with some presenting quasi-vertical side walls and others being more circular in shape, thus with less steep walls.

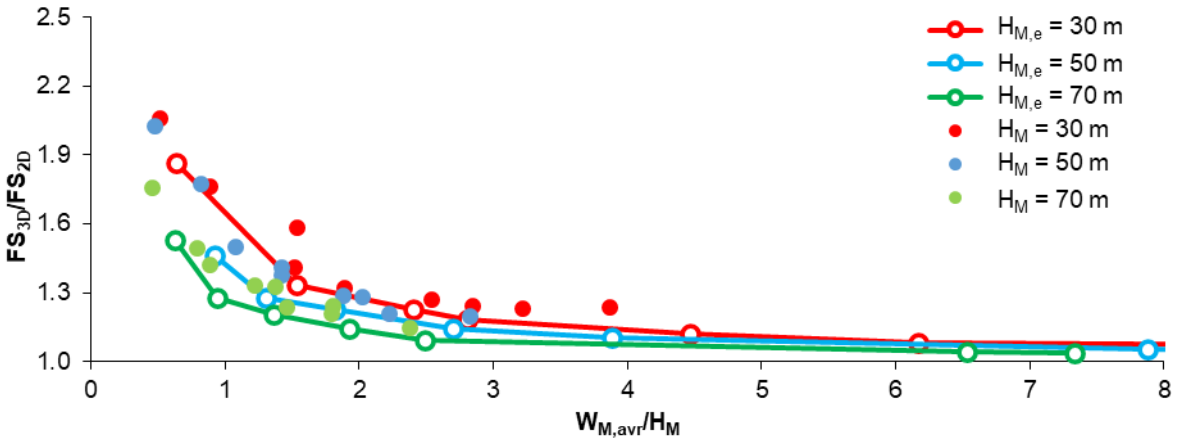


Fig. 7 – Comparison between the 3D effects of the 3D models and the extruded ones.

It is important to notice that the difference between the extruded models and the 3D models is the geometry of the abutments of the valley, with the former being vertical and the latter having a certain slope (β). Notwithstanding, when considering the average width of the model ($W_{M,avr}$), the non-extruded 3D models show very similar results to the characteristic curves (extruded models). This shows that the shape of the valley, as the confinement factor of the slipping mass, seems to be adequately evaluated by the model's $W_{M,avr}/H_M$ ratio.

Fig. 8 compares the 3D sliding surfaces to the respective 2D ones along the same cross-section. It is important to mention that since the 3D sliding surfaces are represented only by cutting the 3D mass along the same plane as the 2D cross-section, this comparison might not be accurate in the cases where the direction of the 3D sliding mass differs significantly from the central (2D) cross-section. This is particularly relevant in more complex 3D geometries. In the case of the present study, it was observed that the 3D sliding direction does not match perfectly with the 2D cross-section despite the 3D model's symmetry. This is because a 3D sliding mass with a sliding direction close enough to, but slightly skewed from, the central cross-section leads to a slightly more critical slip surface than the 2D one. Nonetheless, since the difference between the slightly skewed cross-section and the central (2D) one was negligible in the present case, the 2D cross-section of each dam's height was adopted for all corresponding 3D models.

In the 30 m high dams (Fig. 8a), it is possible to notice that the thalweg width rules the surface average depth. Model 2 is an exception, with the critical surface obtained being deeper and longer than the other surfaces with the same abutment angle. Relatively to maximum depth, almost all surfaces are similar, exiting above the starter dyke, except Model 7, which is more superficial than the others. For all the models but the ones with $L_M = 0$, there is a fairly good agreement between the 3D and the 2D slip surfaces. One can also see that the more confined the dam is in the valley ($L_M = 0$), the less representative the 2D slip surface is.

In the 50 m high dams (Fig. 8b), the thalweg width still rules the surface depth. In these models, one can observe 3 distinct exiting zones of the slip surfaces: the wider the valley, the deeper the exit zone. There is an adequate agreement between the 3D and the 2D slip surfaces, although less than in the 30 m high models. However, the agreement is better for models with wider thalweg widths.

In the 70 m high dam models (Fig. 8c), it is possible to notice a diversity of surfaces, being as deeper as the abutment angle is lower and the thalweg width is higher (a general trend in all models). In this case, only the model with a wider valley has a similar slip surface shape as the 2D one, which is the one that is closer to an infinitely long valley. It is important to state that the shape type of the slip surface may influence the agreement in shape between the 2D and the 3D models. In fact, besides the extra dimension, the 3D slip surfaces are more ellipsoidal in shape *vis-à-vis* the circular 2D ones.

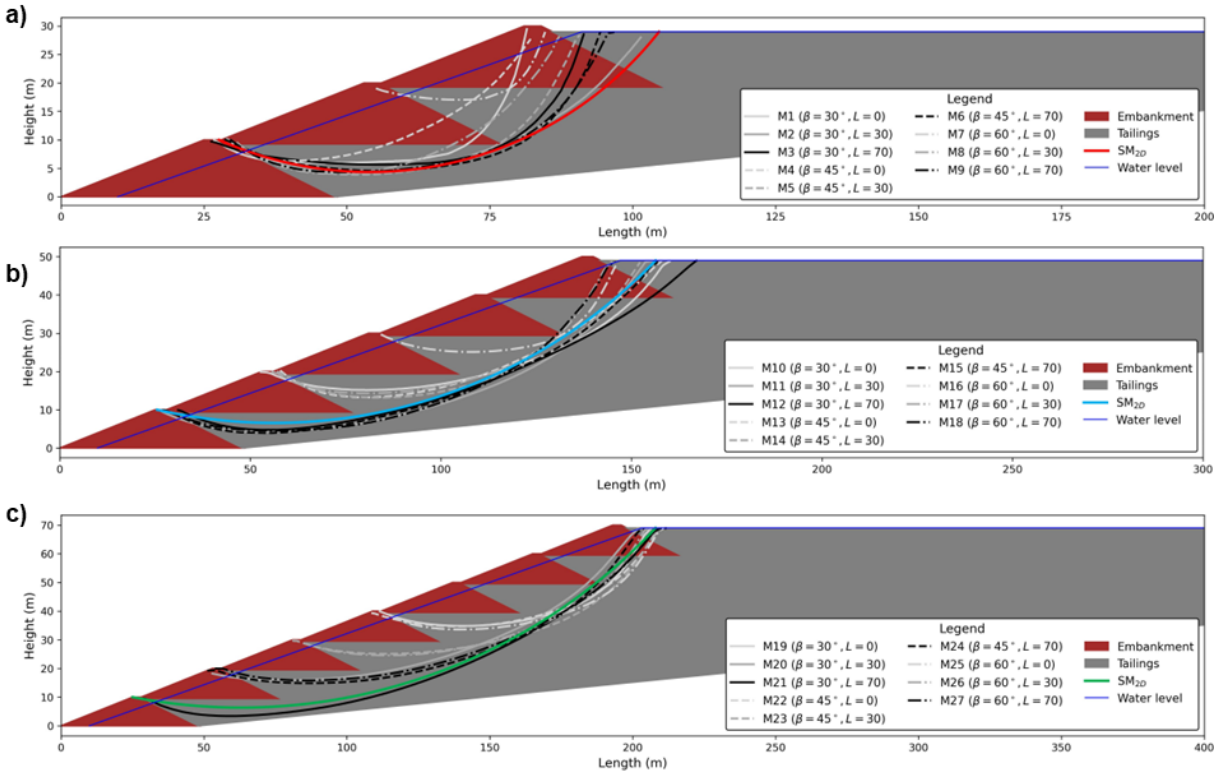


Fig. 8 – Comparison of the 2D and 3D slip surfaces along the same cross-section: a) H_M = 30 m; b) H_M = 50 m; c) H_M = 70 m.

For a more detailed analysis, it is important to discriminate the difference between the models, evaluating not only the factor of safety ratios but also other aspects such as the height of the sliding mass (H_{SM}), the width of the sliding mass (W_{SM}), and the volume of the sliding mass (V). Those variables were normalised relatively to the model's height (H_M) in the case of the volume normalised to H_M^3. Fig. 9 presents the relation between these variables and $W_{M,avr}/H_M$.

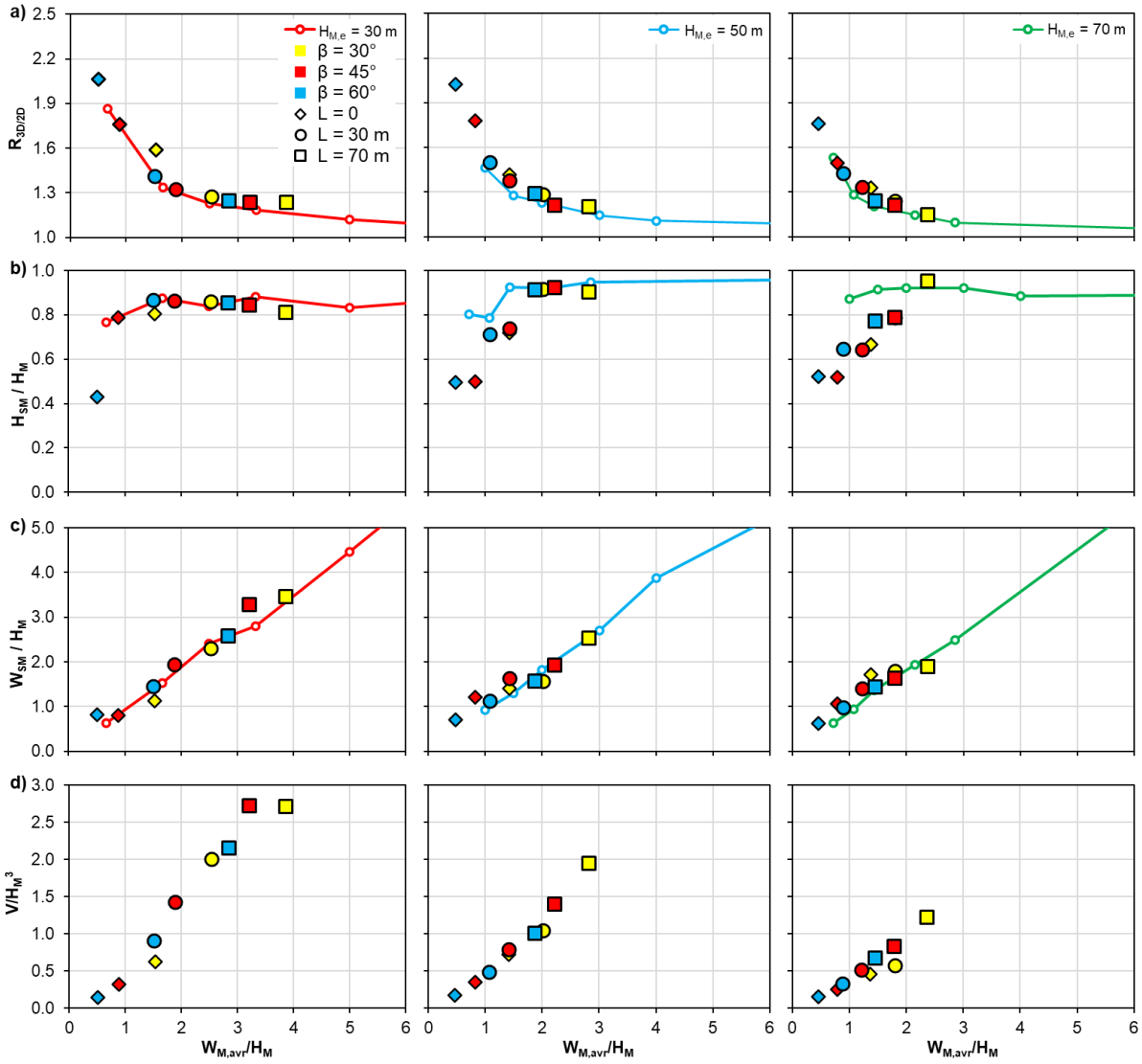


Fig. 9 – Comparison between main geometric dimensions with the $W_{M,avr}/H_M$ ratio: a) $R_{3D/2D}$; b) H_{SM}/H_M; c) W_{SM}/H_M; d) V/H_M^3.

In Fig. 9, it is possible to detect some general trends:

- The FS_{3D}/FS_{2D} ratios decrease with the decrease in β and the increase in L_M and correlate inversely with the $W_{M,avr}/H_M$ ratio, tending to 1,0 for $W_{M,avr}/H_M > 6$ (Fig. 9a);
- The sliding mass's height (H_{SM}) depends on the model's height (H_M). It becomes also dependent on $W_{M,avr}/H_M$ as taller the dam is, mainly via L_M. It is possible to see a horizontal asymptote in the H = 30 m model, starting approximately in $W_{M,avr}/H_M = 1$, and another in the H = 50 m model, starting in $W_{M,avr}/H_M = 2$. This could mean that the higher the model is, the higher as to be the $W_{M,avr}$ in order to stabilise the H_{SM} – this is verified in two of the models and could also be true for the third one if more points, with higher ratios $W_{M,avr}/H_M$, were tested.
- The W_{SM} correlates well with the $W_{M,avr}/H_M$ (Fig. 9c), in all the studied dam heights, with a positive correlation with both L_M and β variation;

- As expected, the volume of the sliding mass (V) increases when the $W_{M,avr}/H_M$ ratio increases being the increase rate higher as higher the model is (Fig. 9d);

Fig. 10 shows the correlation coefficients between the aforementioned variables for the whole data set, showing strong correlations ($R^2 > 0,7$). For the available dataset, i.e., for the set of models and parameters used, the correlation function of the slip surface's normalized width and volume is linear, whereas the correlation with $R_{3D/2D}$ and the normalized height of the slip surface follows a power function with an asymptote towards 1,0.

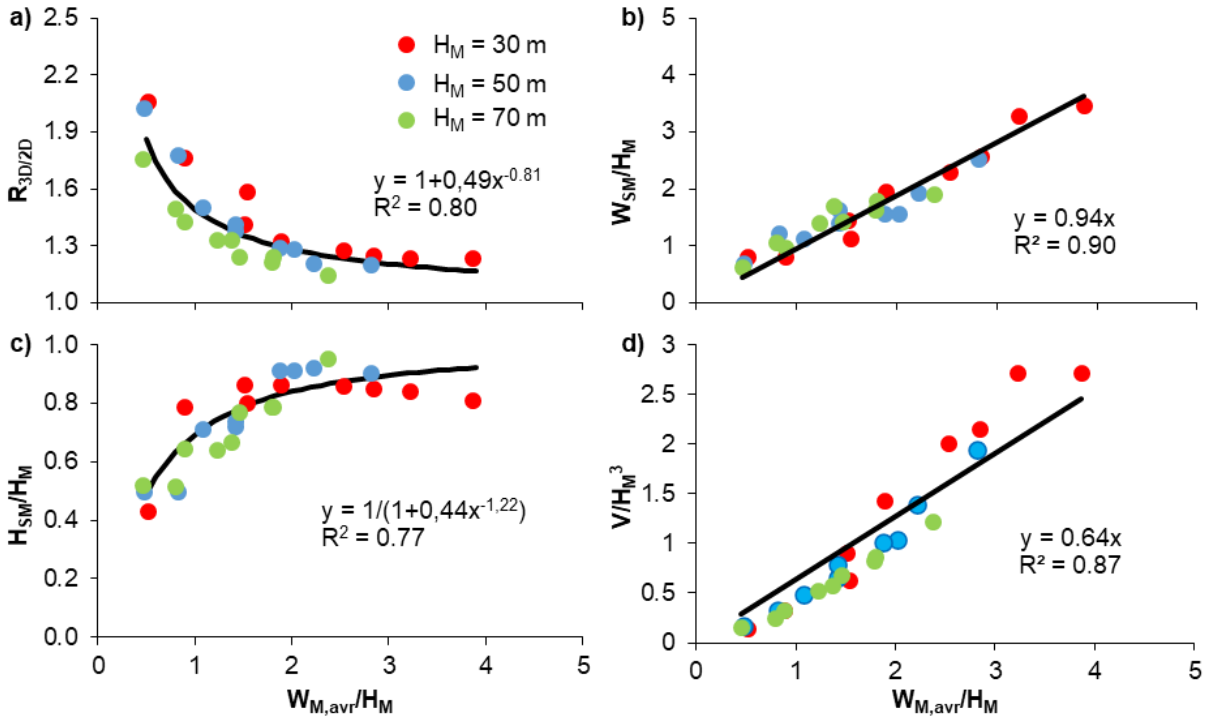


Fig. 10 –3D effects correlations with $W_{M,avr}/H_M$: a) $R_{3D/2D}$; b) W_{SM}/H_M; c) H_{SM}/H_M; d) V/H_M^3.

3.5. Limitations

The limitations found can be divided into two types: methodology and software. In the methodological approach, the following limitations were identified:

- The Stark & Ruffing methodology analyses homogeneous materials, which simplifies the extruded models created and the slip surfaces obtained. The models considered in this study comprise two different materials with distinctive load responses (drained and undrained) and with contrasting resistances – tailings are weak when compared to the dykes. This introduces complexity in the results, both in terms of FS and sliding mass dimensions. Furthermore, Stark & Ruffing worked only with the slip surface geometry, whereas in this study, we adopt a complete structure, relying on a commercial software to find the critical slip surface (the limitations of the software are discussed below);

- The 2D cross-section adopted both to compute the FS_{2D} and to build the extruded models was always the central one due to the symmetry of the models. However, even in symmetric models, the lowest FS_{2D} is sometimes found in a cross-section slightly rotated from the central one. Nonetheless, the difference between the central cross-section and the one that led to the lowest FS was very low, both concerning its geometry and the resultant FS;
- The results are only valid for the model and set of parameters used. Furthermore, this methodology is likely used for a homogeneous model along the direction perpendicular to the crest axis. Nonetheless, more geometric variables (e.g., the slope of the embankment, the slope of the thalweg, the direction of the thalweg in relation to the dam's axis, etc.) can be analysed to increase the understanding of the 3D effects in such a structure.

The software limitations encountered were:

- The intelligent search method algorithm adopted uses different shapes for the slip surfaces based on a spline function instead of allowing one to choose a specific one. This can lead to different geometries between 3D and 2D models along the same plane;
- The authors also warn that in parametric studies, the software may lead to non-linear results due to the influence of the input parameters of the slip surface search algorithm or the boundaries of the model (which restrict the length of the slip surface, insofar as it avoids those boundaries).

4. CONCLUSIONS

This paper presents a parametric evaluation of the 3D effects of the height and the valley's shape on the overall stability of a tailings dam raised by the upstream method, following state-of-the-art methodologies. Compared to the existing research in this area, this study takes a step forward by evaluating the 3D effects of the model using commercial software (*Slide*) and considering not just the slip surface geometry but also the geometry of the whole structure. This is done by varying different geometrical features of the valley and allowing the software to search the critical slip surface by means of an optimised search algorithm.

The analysis results are only valid for the geometries, water level, geotechnical parameters and load conditions considered in this study. The main results were the following:

- The 3D safety factors were always greater than the respective 2D, with $R_{3D/2D}$ ranging from 1,15 to 2,06. Similarly to the previous studies, the $R_{3D/2D}$ is greater for $W_{M,avr}/H_M$ smaller than 2 and tends to 1,0 (i.e. $FS_{3D} = FS_{2D}$) for $W_{M,avr}/H_M$ higher than 6;

- In some cases, the 3D shape of the slip surfaces along the 2D cross-section is similar to the 2D one, namely for wider valleys (>> W_M). This happens because models with wider valleys approximate the assumption of an infinitely long perpendicular dimension underlying 2D LE analysis;
- From the variables tested, the most influential to the safety of the dam is the thalweg width (L_M), followed by the abutment slope angle (β), both of which are directly associated with the confinement given by the valley;
- The height of the dam (H_M) significantly impacts the 3D stability of upstream raised fully saturated tailings dams under undrained conditions, shown by the generally stable 30 m high dam and the generally unstable higher ones;
- The shape of the valley, as the confinement factor of the slipping mass, seems to be adequately evaluated by the $W_{M,avr}/H_M$ ratio of the model. This is shown both by the approximation of the results of the 3D models and the extruded ones and by the correlation coefficients presented in Fig. 10. This is valid for the set of models and parameters used in the present analysis.

REFERENCES

[1] Anagnosti, P. (1969). Three-dimensional stability of fill dams. Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 2: 275-280.

[2] Rocscience (n/d). Slide3: 3D Limit Equilibrium Slope Stability. Theory Manuals.

[3] GEO-SLOPE International, Ltd (2021). GeoStudio. Stability Modelling with GeoStudio. Calgary, AB, Canada.

[4] Fredlund, M. D. (2014). Is There a Movement Toward Three-Dimensional Slope Stability Analyses? GeoStrata Magazine 18(4): 22-15.

[5] ANCOLD (2012). Guidelines on Tailings Dams - Planning, Design, Construction, Operation and Closure.

[6] CDA (2019). Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams (2019 Edition).

[7] Duncan, J. M., Wright, S. G., Brandon, T. L. (2014). Soil Strength and Slope Stability (2nd Edition). John Wiley & Sons, New Jersey.

[8] USGS (2004). Landslide types and processes. Fact Sheet 2004-3072. https://pubs.usgs.gov/fs/2004/3072/pdf/fs2004-3072.pdf (Accessed, Nov 2, 2010).

[9] Akhtar, K. (2011). Three-Dimensional Slope Stability Analyses For Natural And Manmade Slopes, PhD Thesis, University of Illinois.

[10] Stark, T. D., Eid, H. T. (1998). Performance of Three-dimensional Slope Stability Methods in Practice. Journal of Geotechnical and Geoenvironmental Engineering 124(11): 1049-1060.

[11] Terzaghi, K. (1943). Theoretical Soil Mechanics. John Wiley & Sons, Inc., New York.

[12] Krahn, J. (2003). The 2001 R.M. Hardy Lecture: The limits of limit equilibrium analyses. Canadian Geotechnical Journal 40: 643-660.

[13] Meyerhof, G. G. (1995). Development of geotechnical limit state design. Canadian Geotechnical Journal 32: 128-136.

[14] Morgenstern, N. R.,Price, V. E. (1965). The analysis of the stability of general slip surfaces. Géotechnique, 15(1): 79-93.

[15] Spencer, E. (1967). A method of analysis of the stability of embankments assuming parallel inter-slice forces. Géotechnique, 17(1): 11-26.

[16] Duncan, J. M., Wright, S. G. (1980). The accuracy of limit equilibrium methods of slope stability analysis. Engineering Geology 16: 5-17.

[17] Lambe, T. W., and Whitman, R. V. (1969). Soil mechanics. John Wiley & Sons, New York.

[18] Loehr, J. E., McCoy, B. F., Wright, G. S. (2004). Quasi-Three-Dimensional Slope Stability Analysis Method for General Sliding Bodies. Journal of Geotechnical and Geoenvironmental Engineering 130(6): 551-560.

[19] Chen, Z. Y., Yin, J.H., Wang, Y.J. (2006). The Three-Dimensional Slope Stability Analysis: Recent Advances and a Forward Look. Advances in Earth Structures: Research to Practice: 1-42.

[20] Gens, A., Hutchinson, J. N., Cavounidis, S. (1988). Three-dimensional analysis of slides in cohesive soils. Géotechnique 38(1): 1-23.

[21] Hovland, H. J. (1977). Three-dimensional slope stability analysis method. Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, GT 9: 971-986.

[22] Akhtar, K., Stark, T.D. (2017). Importance of Side Resistance in a 3D Stability Analysis. Geotechnical Frontiers 2017 278: 285-293.

[23] Stark, T. D., Ruffing, D. G. (2017). Selecting Minimum Factors of Safety for 3D Slope Stability Analyses. Geo-Risk 2017: 259-266.

[24] Baligh, M. M., Azzouz, A. S. (1975). End effects on stability of cohesive slopes. Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, GT 11: 1105-1117.

[25] Chen, R. H., Chameau, J.-L. (1983). Three-dimensional limit equilibrium analysis of slopes. Géotechnique 32(1): 31-40.

[26] Cavounidis, S. (1987). On the ratio of factors of safety in slope stability analyses. Géotechnique 37(2): 207-210.

[27] Arellano, D., Stark, T. D. (2000). Importance of three-dimensional slope stability analysis in practice. Geo-Denver 2000 (Denver, Colorado, USA): 18-32.

[28] McQuillan, A., Bar, N., Yacoub, T. (2022). On the comparison of 2D and 3D stability analyses of an anisotropic slope. The Evolution of Geotech - 25 Years of Innovation – Hammah et al. (Eds).

[29] Kalatehjari, R., Arefnia, A., Rashid, A. S., Ali, N., Hajihassani, M. (2015). Determination of three-dimensional shape of failure in soil slopes. Canadian Geotechnical Journal, 52(9): 1283–1301.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

TECHNICAL AND ECONOMIC FEASIBILITY OF USING IRON MINING WASTE AS BORROW MATERIAL FOR DAM DECHARACTERIZATION

Pereira, Frank[1]
Vale S/A

Tacco, Rafaela
Vale S/A (*In memorian)*

Ribeiro, Adriana
Vale S/A

Carmo, João Bosco
Vale S/A

Mazon, Alessandra
Vale S/A

Guerra, Felipe
Vale S/A

Cavalcante, André
Instituto Tecnológico Vale

Torres, Vidal
Instituto Tecnológico Vale

Sebastião, Cristiane
Vale S/A

Leal, Ana
Vale S/A

ABSTRACT

The application of materials derived from mining and beneficiation processes in engineering projects has gained substantial recognition, especially in mining operations, due to the large volumes of material moved and the limited areas available for borrow or disposal nearby. This study examines the technical and environmental feasibility of using iron mining waste as borrow material in dam decharacterization. It presents the physical, chemical, and geotechnical characterization of the waste, addressing critical properties such as leachability, compaction, permeability, and shear strength, which determine its behavior under imposed loads – essential aspects to ensure the structural and environmental integrity of dams post-decharacterization. The methodology also includes a detailed analysis of the environmental implications of waste utilization, with an emphasis on the potential reduction in storage area demand and the possibility of adding value to this material. The technical assessment is conducted in compliance with Conama Resolutions No. 430 and No. 357 and NBR 10004, taking into account waste classification and contamination potential. In addition to environmental considerations, the study includes a comparative economic analysis between the use of mining waste and conventional materials, examining acquisition, processing, transportation, and application costs. The objective is to develop methodologies that enable the safe and efficient use of these materials, fostering more sustainable and economically advantageous engineering practices. The results aim to provide practical guidelines for the incorporation of mining waste in engineering solutions, promoting sustainability in mining and aligning with circular economy policies. This project intends to stimulate interest from the scientific community and industry in economically viable and environmentally responsible solutions, reflecting the growing importance of sustainability and environmental preservation in contemporary business models.

Keywords: Mining tailings, Dam Decharacterization, Technical and economic feasibility, Sustainability in engineering, Circular economy.

[1]frank.pereira@vale.com, Brazil

1. INTRODUCTION

Waste disposal is one of the greatest environmental challenges faced by the iron mining industry, primarily due to the large volume of material generated and the scarcity of suitable and safe areas for storage. This challenge is particularly relevant in contexts where the construction of new dam structures is suspended, and conventional waste disposal solutions encounter operational and environmental limitations.

In this context, the reuse of waste for structural reinforcement of dams emerges as a sustainable alternative, aligned with the principles of the circular economy. By promoting the reutilization of these materials, it is possible not only to reduce the demand for disposal areas but also to lessen the environmental impact associated with mining processes. Furthermore, the use of waste as borrow material can meet ESG requirements, enhancing societal trust in the safety and sustainability of such practices.

This study analyzes the technical and economic feasibility of using iron mining waste as reinforcement material for dam decharacterization. The research includes a detailed characterization of the waste, covering geotechnical testing and evaluation of environmental parameters in compliance with regulatory standards such as ABNT NBR 10.004:2004 and CONAMA Resolution 357/2005. Finally, the study explores the economic potential and environmental benefits of this approach, proposing a solution that promotes more sustainable engineering practices.

2. METHODOLOGY

The methodology used in this study followed an integrated approach, combining geotechnical, environmental, and economic analyses to assess the feasibility of using dredged waste as reinforcement material for the main mass of a dam located in Ouro Preto, Minas Gerais. Initially, representative samples of dredged waste, sterile material from MA PDE and MC PDE, located near the structure, were collected. These samples were dried and prepared for laboratory testing, adjusting to the optimal moisture required for geotechnical and chemical testing.

To characterize the geotechnical properties of the waste and evaluate its potential as a reinforcement material, a series of tests were conducted. Grain size analyses determined particle distribution, classifying the material as granular or fine soil, an important factor for understanding its compactability and structural stability. Compaction tests (Proctor) were conducted to define the maximum dry density and optimum moisture content, aiming to optimize the mass compaction. Oedometer consolidation tests were performed to assess

compressibility and susceptibility to settlement under loads, providing parameters such as t_{90} and compression coefficients. Triaxial compression tests (CID and CIU), in both drained and undrained conditions, supplied shear strength parameters and allowed the analysis of material behavior under different confining stresses. Lastly, the specific gravity of grains was determined to help define the equilibrium conditions of the mass.

The environmental assessment considered compliance of the waste with Brazilian standards and CONAMA Resolutions, focusing on aspects of metal and contaminant solubilization and leaching. Tests were performed according to [1] and [2] standards to identify the mobility of heavy metals and other contaminants, ensuring that the materials presented no risk of environmental contamination. The waste was classified according to hazardousness and chemical stability per ABNT standards, while compliance with [3] was assessed to ensure that the use of waste for reinforcement would not compromise the quality of nearby water bodies, particularly regarding iron and manganese concentrations.

Various alternatives for reinforcing the main mass of the dam were then developed and analyzed, varying the type and proportion of materials used (dredged waste, sterile material from MA PDE, sterile material from MC PDE, and silty-sandy soil). Alternative 13 was selected as the most viable, requiring the least amount of rockfill while providing the best balance between economic feasibility and structural stability.

The stability analysis of section D-D' of the main mass was conducted using the Spencer, Sarma, and Morgenstern-Price methods, which consider both drained and undrained conditions. Minimum safety factors for each operational condition, including pseudo-static analysis, were obtained and compared with the required values to ensure the suitability of the waste as reinforcement material.

Finally, the economic feasibility of the proposal was evaluated by comparing the extraction, transportation, preparation, and disposal costs of the waste with alternative materials. This analysis included dredging and handling costs of the dredged waste to achieve optimal moisture, along with a cost-benefit analysis from an ESG perspective. This approach highlighted the potential of using waste to reduce environmental impact while generating corporate and social value.

3. RESULTS AND DISCUSSION

To determine the geotechnical characteristics of the waste intended for reinforcing the main mass, Table 1 summarizes the tests conducted with the selected samples. A detailed description of the tests is provided in the following subsections.

Table 1 – Samples Used in Testing

Tests	Sample	Norm
Grain Size Analysis	AM5422-177	[4]
Normal Proctor Compaction	AM5422-177	[5]
	AM5422-178	[5]
	AM5422-179	[5]
Void Ratio	AM5422-177	[6] [7]
Specific Gravity	AM5422-177	[8]
CIDsat	AM5422-177	[9]
CIUsat	AM5422-177	[10]
Natural Moisture Content	AM5422-177	[11]

3.1. Grain Size Analysis

The grain size analysis of sample AM5422-177, shown in Figure 1, indicates that the waste is predominantly granular, with the following composition: 1.0% gravel, 87.0% sand, 10.2% silt, and 1.8% clay.

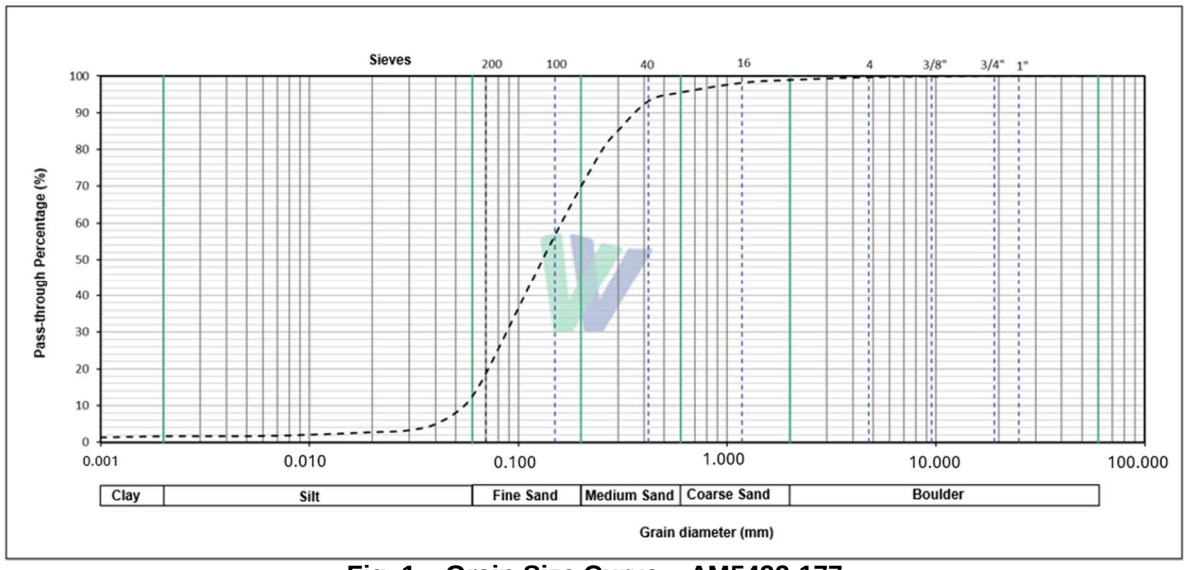


Fig. 1 – Grain Size Curve – AM5422-177

The results indicate that the waste has a predominantly sandy-silty composition and can be classified as granular soil. The predominance of sand and silt fractions suggests the material may have good compactability and drainability, favorable aspects for its use as structural reinforcement material in the main mass. Additionally, the low clay content helps reduce the

potential for expansion and shrinkage, minimizing the risk of undesirable deformations over time.

3.2. Specific Gravity and Moisture Content Tests

The specific gravity test results for sample AM5422-177 are shown in Table 2.

Table 2 – Specific Gravity Test Results

Sample	Specific Gravity (g/cm³)	Specific Weight (kN/m³)
AM5422-177	3.583	35.14

Furthermore, a test to determine the natural moisture content of sample AM5422-177 was conducted, presented in Table 3.

Table 3 – Moisture Content – AM5422-177

Moisture Content (%)	
AM5422-177	
Maximum Limit	5.3
Minimum Limit	4.8
Average	5.04

3.3. Triaxial Compression Tests

To determine the shear strength parameters of the waste, drained (CID) and undrained (CIU) triaxial compression tests were conducted, both in saturated conditions, using samples prepared from the deformed sample AM5422-177_S1, with an 85% degree of compaction.

3.4. Saturated Drained Triaxial Compression Test (CIDsat)

In the saturated drained triaxial compression test (CIDsat), the specimens were subjected to three consolidation pressures, as follows: 1 specimen at 150 kPa; 2 specimens at 300 kPa; 2 specimens at 600 kPa.

Figure 5 presents the effective stress path, while Figure 6 illustrates the stress-strain behavior of the samples.

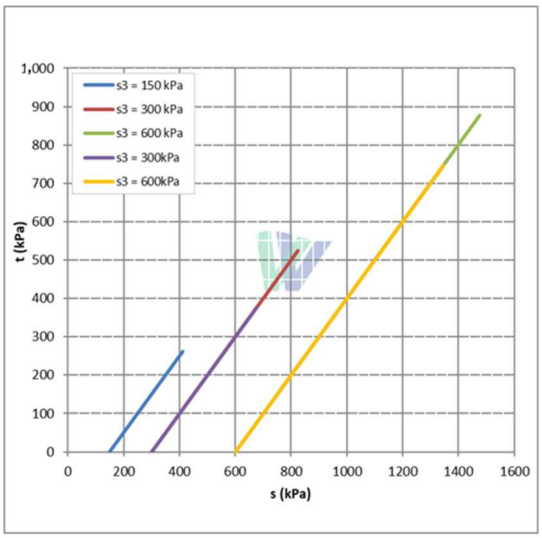


Fig. 5 – Effective Stress Path

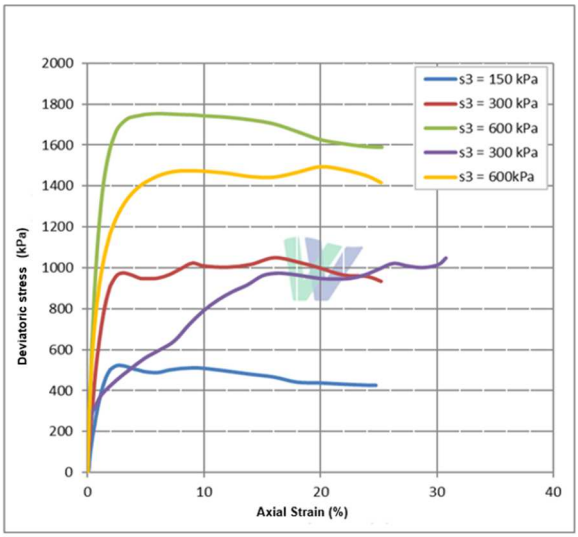


Fig. 6 – Stress-Strain Graph

The data obtained were analyzed through linear regression, using two distinct methodologies: maximum obliquity (Figure 7) and maximum deviatoric stress (Figure 8).

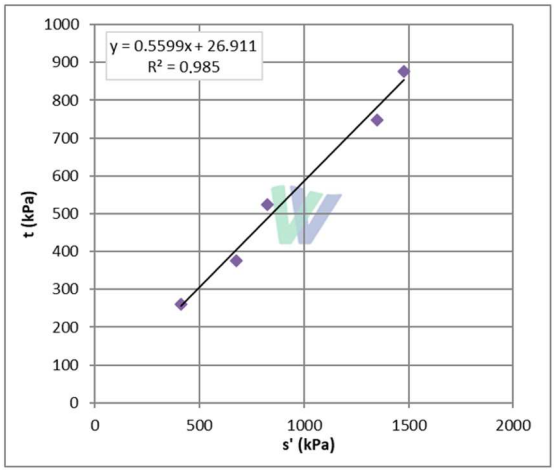


Fig. 7 – Linear Regression – Maximum Obliquity

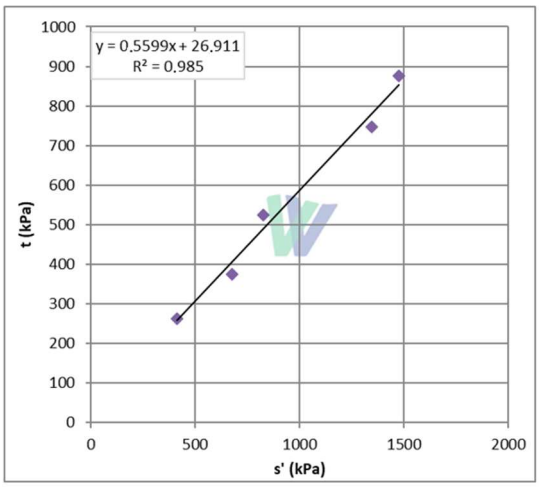


Fig. 8 – Linear Regression – Maximum Deviatoric Stress

Due to the material's contractive behavior, indicating a tendency toward liquefaction, the shear strength parameters were determined based on the saturated undrained triaxial test (CIUsat), discussed in the next section.

3.5. Saturated Undrained Triaxial Compression Test (CIUsat)

For the saturated undrained triaxial compression test (CIUsat), the specimens were molded with an 85% compaction degree relative to the Normal Proctor compaction test. The samples were then subjected to five different confining stresses: 50kPa, 100 kPa, 200 kPa, 400 kPa and 800 kPa.

Additionally, the samples were deformed up to an axial strain of 25%.

To analyze the material's behavior, stress-strain graphs (Figure 9) and effective stress paths (Figure 10) were generated.

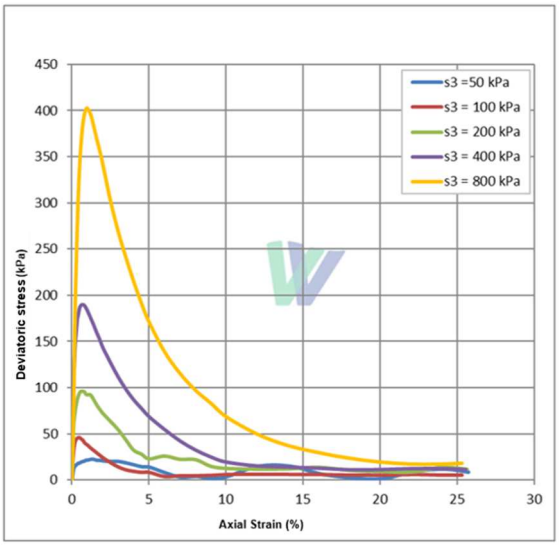


Fig. 9 – Stress-Strain Graph

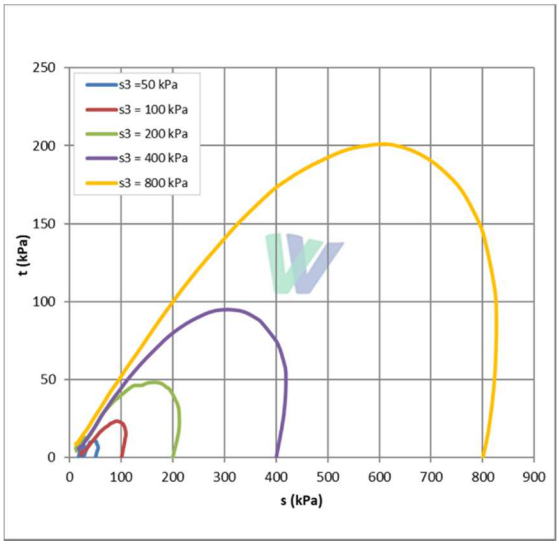


Fig. 10 – Effective Stress Path

In the stress-strain graph (Figure 9), it is observed that the material exhibits a peak shear strength, followed by a sharp drop as axial strain increases, eventually reaching a steady-state condition. This behavior is typical of materials prone to liquefaction.

According to [12], the combined analysis of the graphs (Figures 9 and 10) suggests that the waste, being characterized as granular soil, shows effective stress paths with contractive behavior, converging toward the graph origin. This behavior is identified as material liquefaction, as described by [13], [14], and [15].

Additionally, as noted by [16], granular soils can exhibit three typical responses to triaxial loading, as illustrated in Figure 11. In the case of the waste studied, the predominant response is type “1,” associated with flow failures, reinforcing the material's propensity for liquefaction.

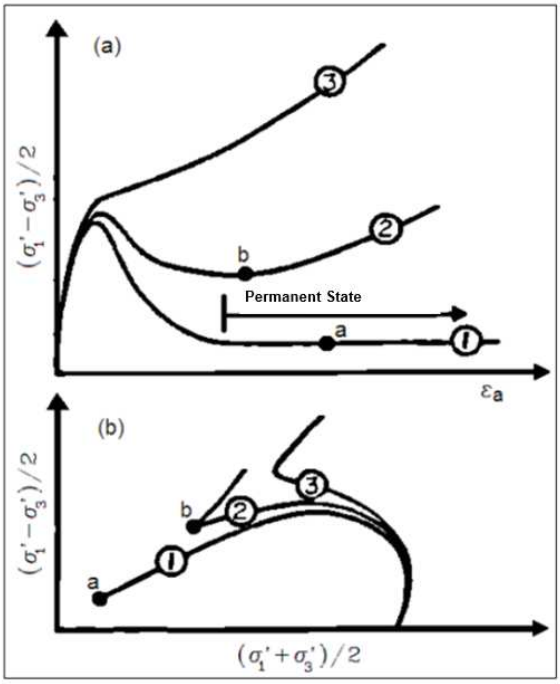


Fig. 11 – Typical Granular Soil Behaviors under Undrained Loading

The photos of the samples post-testing, shown in Figure 12, highlight the large deformations and bulging observed, indicating the material's contractive behavior under varied confining stresses.

σ'_3 = 50 kPa **σ'_3 =100 kPa** **σ'_3 =200 kPa** **σ'_3 =400 kPa** **σ'_3 =800 kPa**

Fig. 12 – Perspective of Tested Samples

Additionally, an evaluation of the material based on the behavior of Parameter A relative to the axial strains of the samples was conducted, as shown in Figure 13.

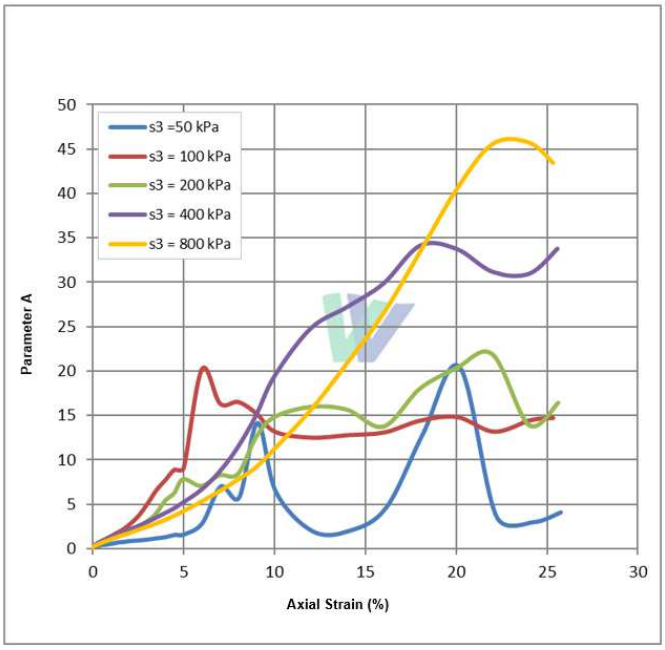


Fig. 13 – Parameter A Behavior

Parameter A is determined by the equation:

$$A = \frac{\Delta U}{\Delta \sigma} \quad (1)$$

where:

- ΔU represents the change in pore pressure; and
- $\Delta\sigma$ is the difference between stresses σ'_3 and σ'_1.

The resulting value of Parameter A can vary from “negative infinity” to “positive infinity,” as shown in Figure 14, where graph (a) represents the MIT method and graph (b) represents the Cambridge methodology. Typically, Parameter A values range from -1 to +1.

Values trending towards negative infinity indicate a dilative behavior of the material, whereas values trending towards positive infinity suggest contractive behavior. In Figure 13, it is observed that A values exceed +1 significantly, reaching approximately 45, supporting the condition of liquefaction for the waste studied.

For tailing materials, shear strength parameters are typically defined based on the critical state of the samples. However, as the tested samples showed liquefaction, the residual stresses appeared similar across samples. Additionally, due to the loose nature of granular soils in the test specimens, attempts to define critical state limits for each confining stress resulted in widely scattered points, making it impossible to plot a trend line, as shown in Figure 15. This challenge is also due to the limited number of tests conducted.

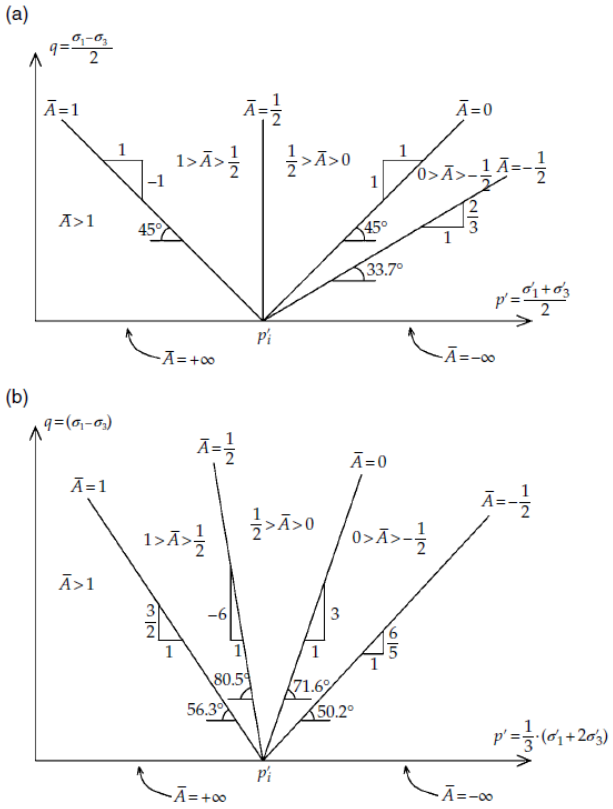


Fig. 14 – Parameter A Variation Graph

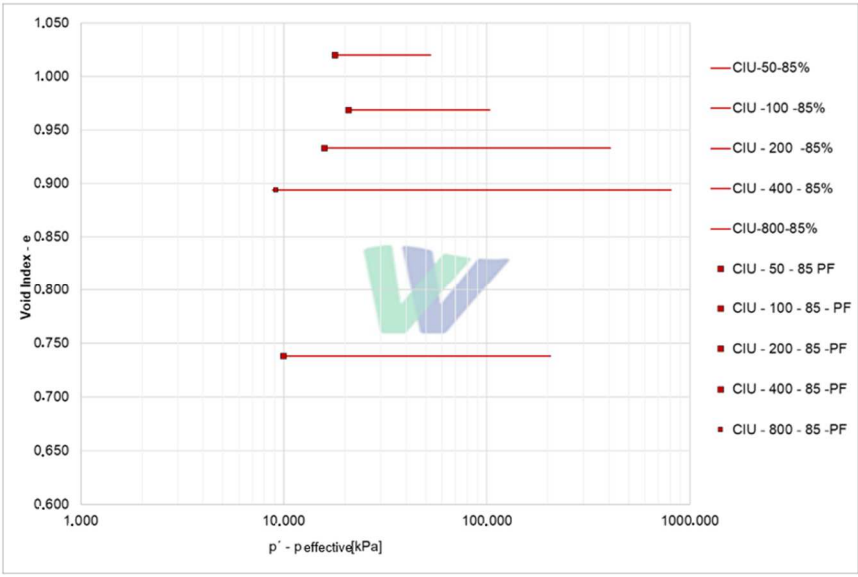


Fig. 15 – Critical State Limit

Based on the above observations, the strength parameters (cohesion and friction angle) were obtained through linear regression, using the maximum deviatoric stress method (Figure 16). Given that the waste is characterized as granular soil, the cohesion intercept was set at the origin of the graph.

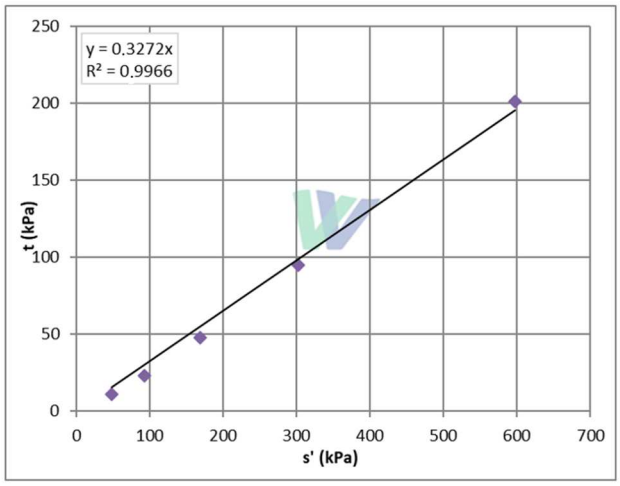


Fig. 16 – Linear Regression – Maximum Deviatoric Stress

The specific weight of the waste, presented in Table 4, was calculated as the average of the initial specific weight values for each specimen, measured before the triaxial tests.

Table 4 – Specific Weight of Waste Specimens

Triaxial Text	Sample	Specimen	Specific Weight (kN/m³)	Average Specific Weight (kN/m³)
CIU	AM5422-177	CP-01	17.65	17.6
		CP-02	17.60	
		CP-03	17.63	
		CP-04	17.58	
		CP-05	17.69	

Based on the study, the geotechnical parameters of the waste were defined and are presented in Table 5.

Table 5 – Geotechnical Parameters of the Waste

Material	Specific Weight (kN/m³)	Strength Parameters	
		c' (kPa)	ϕ' (°)
Tailing	18.0	0	19

3.6. Study of Alternatives

Based on the geometry outlined in the executive design using MA PDE sterile material (first study), new alternatives (Alternatives A to H) were developed for reinforcing the Main Mass, considering the use of dredged waste, sterile material from MA PDE and/or MC PDE, and silty-sandy soil. Table 6 summarizes the studied alternatives as potential borrow material options for reinforcing the main body of the dam in question.

Among the alternatives analyzed, Alternative “E” was selected for a detailed stability analysis, as it required the smallest volume of rockfill, supporting economic feasibility.

Table 6 – Alternatives for Reinforcing the Main Mass

	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E	Alternative F	Alternative G	Alternative H
Material	Waste Rock and Rockfill	Tailing, Waste Rock and Rockfill	Tailing, Waste Rock and Rockfill	Tailing, Waste Rock and Rockfill	Tailing, Waste Rock and Rockfill	Waste Rock and Rockfill	Waste Rock and Rockfill	Silty Sand Soil and Rockfill
Minimum Berm Width (m)	18.25	10.0	20.0	14.0	6.0	7.0	5.0	5.0
Maximum Slope Height (m)	16.95	5.0	7.0	7.0	7.0	7.0	7.0	7.0
Slope Inclination	1V:3.2H	1V:3H	1V:3H	1V:3H	1V:3H / 1V:4H	1V:3.2H	1V:3.2H / 1V:4H	1V:3.2H
Top Elavation (m)	≈803.73	≈801.65	≈802.82	≈801.78	≈803.73	≈803.73	≈803.73	≈803.73
Fill Volume – Waste Rock (m³)	1,164,649	340,747.37	409,269.71	359,325.61	354,057.46	902,114.72	1,037,088.8	244,521
Fill Volume - Tailing (m³)	-	918,571.44	773,644.04	640,269.36	755,753.77	-	-	-
Fill Volume – Rockfill (m³)	380,158.12	472,110.35	472,110.35	285,766.21	150,286.34	56,793.50	91,961.75	56,793.5
Fill Volume – Silt (m³)	-	-	-	-	-	-	-	657,590
Total Reinforcemnt Volume (m³)	1,544,807	1,731,429.1	1,655,024.1	1,285,361.1	1,260,097.5	958,908.22	1,129,050.6	958,908

3.7. Section Analyzed

The alternatives were developed based on an analysis of a representative section of the Main Mass (Section D-D’), corresponding to the section with the greatest height, as illustrated in Figure 17.

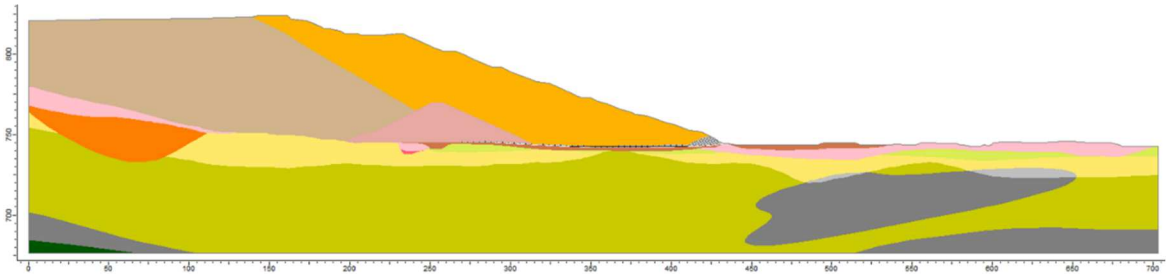


Fig. 17 – Main Mass – Section D-D'

3.8. Geotechnical Parameters

The geotechnical parameters used in the stability analyses are listed in Table 7.

Table 7 – Geotechnical Parameters Adopted in Stability Analyses

Material	Color	Y (kN/m³)	c' (kPa)	ϕ' (°)	Su_{pico}/σ'_{v0}	Su_{liq}/σ'_{v0}
Underflow		21	0	36	-	-
Undrained Underflow		21	-	-	0.29	-
Overflow		23	0	33	-	-
Undrained Overflow		23	-	-	0.20	-
Residual Overflow		23	-	-	0.06	-
Starter Dike Fill		21.5	25	27	-	-
Access Fill		21	7	30	-	-
Drain		20	0	35	-	-
Lateritic Soil + Colluvium + Alluvium		19.8	0	27	0.40	-
Quartizite SRM		21.7	9	31	-	-
Quartizite SRJ		21	13	31	-	-
Quartizite SAP		22	43	35	-	-
Itabirite Mn SRJ		22	10	31	-	-
Itabirite Mn SAP		22	22	35	-	-
Tertiary Deposit (Ochre)		22	11	25	-	-
Rockfill		22	Shear/Normal Function		-	-
Dredged Tailing Reinforcement		18	0	19	-	-
MA PDE Waste Rock Reinforcement		20	0	27	-	-
MC PDE		20	15	27	-	-
Silty Sand Soil		20	0	32	-	-

Figures 18 and 19 show the stability analysis results for Alternative E of Section D-D' of the Main Mass, using the Spencer, Sarma, and Morgenstern & Price methods.

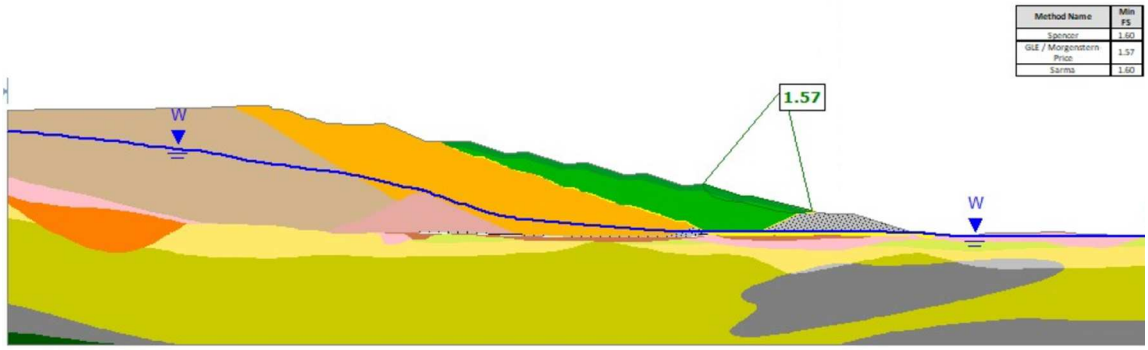


Fig. 18 – Section D-D' – Alternative E – Drained Condition

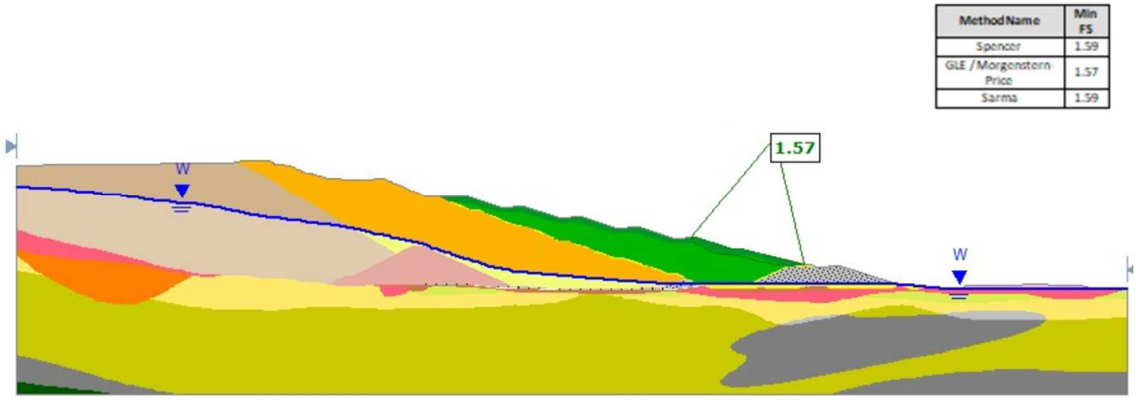


Fig. 19 – Section D-D' – Alternative E – Undrained Peak Condition

Table 8 presents the factors of safety (FS) obtained in the analyses, indicating that the values for the Main Mass are satisfactory for the proposed Alternative E.

Table 8 – Results obtained for Section D-D' of the Main Massif

Alternative	Analysis	Minimum FS	FS Obtained		
			Spencer	GLE/Morgenstern-Price	Sarma
Alternative E	Drained	1.50	1.60	1.57	1.60
	Undrained – Peak	1.50	1.59	1.57	1.59
	Undrained - Residual	1.20	1.43	1.44	1.43
	Pseudo-static	1.10	1.11	1.10	1.11

3.9. Environmental Requirements

For Alternative "E, the study considered the use of dredged tailings as reinforcement material for the main mass of the dam, encapsulated with waste rock from either the MA PDE or MC

PDE deposits (both located near the dam in Ouro Preto/MG). The environmental parameters of these materials were evaluated in compliance with standards [1], [2], [17], and [18] for waste classification and chemical stability, as well as [19] for characterization, as shown in table 09.

Table 9 – Mineral Composition.

Minerals	Sample QEMSCAN Code	MC PDE Waste Rock GTE8OA	MA PDE Waste Rock GTE8PA	Dradged Tailing GTE8QA
	% by mass			
Silicates	Quartz	24.40	8.19	52.01
	Feldspars[1]	0.01	4.75	0.04
	Anphiboles[2]	0.02	1.17	0.02
	Chlorite	0.02	16.43	0.02
	Biotite	0.14	1.85	0.04
	Muscovite	0.04	0.07	0.10
	Kaolinite	0.52	1.24	0.57
	Talc	0.02	8.86	0.05
	Titanite	0.00	0.41	0.00
	Text goeth-caul[3]	0.67	0.73	0.98
Oxides	Iron Oxides	68.73	51.98	45.88
	Manganeses Oxides	5.28	0.71	0.19
	Titanium Oxides	0.01	1.01	0.03
	Gibbsite	0.10	0.01	0.03
	Anatase	0.00	0.25	0.00
Other	Apatite	0.00	0.68	0.01
	Dolomite	0.00	1.21	0.00
	Ankerite	0.00	0.26	0.01
	Other Phases[4]	0.02	0.19	0.04

1 Plagioclase series (albite)
2 Calcium amphiboles (actinolite)
3 Goethite intergrown with kaolinite on a micrometric scale
4 Includes chalcopyrite, calcite, zircon, crandallite, and others

Solubilization and total leaching tests considered the mobilization of all metals present in the solid matrix to the water used in the analysis. Based on the values obtained, the percentage of metal mobilization was calculated (Table 10).

Table 10 – Solubilization and Leaching Classification

	Dredged Tailings		MA PDE		MC PDE	
	Iron	Manganese	Iron	Manganese	Iron	Manganese
Total % (g/100g)	21.93	0.05	32.72	0.3	45.52	1.34
Solubilized (mg/L)	0.1	2.6	0.21	2.6	0.1	0.02
Leached (mg/L)	0.1	2.08	0.1	7.64	0.1	0.02
Total Solubilization (mg/L)	54,825	130	81,800	760	113,800	3,350
Mobilization (%)	0.00018	2	0.00026	0.34211	0.00009	0.0006
Total Leaching (mg/L)	10965	26	16360	152	22760	670
Mobilization (%)	0.00091	8.00000	0.00061	5.02632	0.00044	0.00299

CONAMA Resolution 357/2005 sets guidelines for classifying and establishing quality standards for surface water bodies and defines acceptable parameters for effluent discharge, as shown in Table 11.

Table 11 – Acceptance Parameters According to CONAMA 357

Maximum Permissible Values	Class 1	Class 2	Class 3
Dissolved Iron (mg/L)	0.3	0.3	5
Dissoved Manganese (mg/L	0.1	0.1	0.5

Following the evaluation of environmental parameters and material classification, based on Brazilian environmental legislation criteria, the following findings were determined:

i. The samples showed a high iron content (>20%) with low mobility of this element.
ii. Material from the MC PDE deposit exhibited lower solubilization and leaching rates, along with low manganese mobility, making it a viable option for reinforcement. However, the iron concentration in this sample (44.25%) indicates potential for reprocessing and commercial use.

iii. The dredged tailings showed lower leaching rates compared to material from MA PDE and very low manganese concentration (0.052%), making it more suitable for structural applications. However, stability studies for manganese are recommended to minimize its mobilization.

iv. Mineralogical analysis identified a high concentration of iron oxides and several mineral phases with low iron contents associated with magnesium and silica.

v. Manganese is primarily present in oxide form, with very low concentration in the dredged waste sample.

3.10. Economic Feasibility

In the specific context of the decharacterization project for a dam located in Ouro Preto, Minas Gerais, the economic feasibility of using dredged tailings as reinforcement material for the main structure was evaluated. Although this alternative presents a higher cost compared to waste rock available in nearby stockpiles, the tailings are stored within the dam itself, necessitating additional dredging operations, handling, storage management, and a drying process to achieve the optimal moisture level before placement in the fill area.

When the analysis is expanded to include ESG (Environmental, Social, and Governance) requirements, the use of tailings becomes economically viable by adding value to the company. This approach considers challenges related to tailings disposal in mining processes, material reuse, and the importance of strengthening societal trust in the safe use of tailings as an alternative for structural reinforcement.

4. CONCLUSIONS

This study demonstrated the technical and economic feasibility of using iron mining tailings as borrow material in dam decharacterization. The results indicate that these materials comply with current environmental regulations and possess the necessary geotechnical properties to ensure the stability and safety of modified structures. The adoption of these materials can significantly reduce the environmental impact of iron mining, aligning with circular economy principles and promoting more sustainable engineering practices.

However, the study has limitations, including the need for long-term evaluations to investigate the durability and behavior of tailings under different conditions. Future research should focus on optimizing treatment and application processes for these materials, exploring emerging technologies to maximize their value. We encourage the scientific and industrial communities to collaborate on initiatives that further explore these possibilities, aiming to promote

economically viable and environmentally sound solutions that benefit both the mining sector and society as a whole.

5. HOMAGE

Rafaela Tacco (*in memorian*):
The pain of saying goodbye is difficult, but I choose to honor your memory forever. If it weren't for your determination and perseverance, we wouldn't have been able to finish and deliver this work!
On behalf of all of us, thank you for your support, companionship and friendship.
"You brought the time forward.
Your hand went crazy, making our hours crazy."
Carlos Drummond de Andrade

REFERENCES

[1] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 10.004:2004 Resíduos sólidos – Classificação.

[2] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 10.005:2004 Procedimento para obtenção de extrato lixiviado de resíduos sólidos.

[3] Resolução Conselho Nacional do Meio Ambiente, 357:2005 Dispõe sobre a classificação dos corpos de água e diretrizes ambientais para o seu enquadramento,

[4] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 7181:2016 Versão corrigida 2:2108, Solo Análise Granulométrica.

[5] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 7182:2016, Solo Ensaio de Compactação.

[6] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 16840:2020 Versão corrigida: 2021, Solo Determinação Do Índice de Vazios Máximo de Solos Não Coesivos.

[7] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 16843:2020, Solo Determinação do Índice de Vazios Mínimo de Solos Não Coesivos.

[8] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 6458:2016 Versão corrigida 2:2017, Grãos de Pedregulho Retidos Na Peneira de Abertura 4,8 MM (Determinação Da Massa Específica, Da Massa Específica Aparente e Da Absorção de Água.

[9] ASTM D7181-2011, Triaxial Compression Test - CD

[10] ASTM D4767/2011, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.

[11] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira, 16097:2012 Determinação do teor de umidade.

[12] Freire Neto, João Pimenta (2009). Estudo da liquefação estática em rejeitos e aplicação de metodologia de análise de estabilidade.

[13] Castro, G. (1969) Liquefaction of Sand. Ph.D. Thesis, Division of Engineering and Applied Physics, Harvard University, Cambridge.

[14] Casagrande, A. (1975) Liquefaction and Cyclic Deformation of Sands, a Critical Review. Proceedings of 5th Pan American Conference Soil Mechanic, Foundation Engineering, Buenos Aires, 80-133.

[15] Seed, H. B. (1979). Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. Journal of the geotechnical engineering division, 105(2), 201-255.

[16] Sriskandakumar, S. (2004). Cyclic loading response of Fraser River sand for validation of numerical models simulating centrifuge tests (Doctoral dissertation, University of British Columbia).

[17] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira 10.006:2004, Procedimento para obtenção de extrato solubilizado de resíduos sólidos.

[18] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira 10.007:2004, Amostragem de resíduos sólidos.

[19] Associação Brasileira de Normas Técnicas, Norma Técnica Brasileira 6502:2022, Solos e rochas – Terminologia.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

GEOTECHNICAL RISK ASSESSMENT OF TAILINGS STORAGE FACILITIES - LESSONS LEARNED

Morteza Esfehani[1]
WSP Canada

Luis Melges de Figueiredo
WSP Brazil

Thiago de Souza Carnavale
WSP Brazil

ABSTRACT

The tailings dam failures jeopardized the public's trust in the owners' capacity and authorities' efficiency in managing safely the tailings storage facilities. Revisiting the design approach, building a standard framework and thorough verification were required to enhance transparency and trust.

In order to obtain reliable assessment of the tailings dam safety, independent technical audits, with recognized expertise, were required to evaluate the sites' real conditions regarding the structures' stability and to identify measures to remediate and/or reinforce the facilities at risk.

WSP was then invited by mining companies to provide engineering services for independent technical audits of the tailings and water storage facilities around the world. WSP was engaged as well to undertake a Hazard Identification and Risk Analysis (HIRA) at several Water and Tailings Storage Facilities. The objectives in developing a HIRA process were: meet or exceed industry best practices; minimize harm with zero catastrophic failures of dams; develop a list of Materially Unwanted Events (MUEs); and develop Critical Controls. The HIRA process was carried out in accordance with operator internal procedures, and industry best practices which included: review of site-specific information; Failure Modes and Effects Analysis (FMEA); risk assessment for identified failure modes; identification of controls; and selection of critical controls.

The paper will provide the geotechnical aspects and lessons learned through the execution of the above-mentioned tailings facilities risk assessments around the world.

Keywords: Risk Assessment, Tailings Management, Failure Mode, Prevention, Mitigation.

[1]morteza.esfehani@wsp.com, Canada

1. INTRODUCTION

The third element of the International Council on Mining and Metals (ICMM) Tailings Governance Framework position statement is “Risk Management”, which includes risk identification, an appropriate control regime and the verification of control performance considering all phases of Tailings Storage Facilities (TSFs) [18]. TSFs are critical components in the mining industry, responsible for the containment and storage of waste materials generated from mining processes. These facilities are often large, engineered structures, designed to store tailings, which can include various waste materials such as finely ground rock, water, and chemicals used in the extraction of valuable minerals. The management of TSFs is crucial because improper design, construction, or maintenance can lead to catastrophic failures, resulting in environmental disasters [13], loss of life, and significant financial repercussions. A comprehensive geotechnical risk assessment is essential to ensure the stability and safety [9] of TSFs over their operational lifespan and post-closure. Moreover, the geotechnical risk management is one of the key aspects of performance-based design that focuses on achieving specific performance outcomes throughout the lifecycle of the facility.

This paper provides an overview of geotechnical risk assessment for tailings storage facilities, emphasizing key factors that influence their stability, methods for risk identification, and strategies to mitigate potential hazards. It also addresses the need for continuous monitoring and improvement of risk management practices in the context of evolving mining operations and climate change impacts.

2. BENEFICIAL OF HAZARD IDENTIFICATION AND RISK EVALUATION IN TAILINGS MANAGEMENT

Hazard identification and risk evaluation are essential in tailings management [1, 8] because they enable proactive measures that protect human health, the environment, and company assets. By systematically assessing and addressing hazards and risks, mining operations can avoid life losses and costly incidents improving the operational safety. Here are the key benefits of these processes as illustrated in fig. 1:

Fig. 1 – Key benefits of Risk Assessment

Prevention of Tailings Dam Failures

Proactive Hazard Control: By identifying hazards in advance such as high water pressure, weak embankments, or seepage pathways, operators can implement preventive measures that directly reduce the likelihood of dam failures.

Targeted Risk Reduction: With a clear understanding of specific risks, tailings facilities can focus resources on the most vulnerable areas, reinforcing structures, adjusting water management strategies, and making design improvements that prevent failures.

Improved Safety for People and Environment

Reduced Impact on Local Communities: Effective hazard identification and risk assessments minimize the chances of catastrophic failures that can affect nearby communities, preventing accidents that could result in loss of life or health risks.

Environmental Protection: Proactively managing hazards and risks reduces the likelihood of tailings spills or seepage that could contaminate water resources, soil, and ecosystems, preserving biodiversity and preventing long-term ecological harm [4, 13].

Enhanced Operational Efficiency

Optimized Resource Allocation: Identifying and evaluating risks allow companies to focus resources (personnel, equipment, and capital) where they are most needed, ensuring efficient and effective use of assets for stability and safety [9].

Reduced Operational Interruptions: Regular risk evaluations can help identify early indicators of potential issues, allowing operators to address them before they escalate and cause costly operational downtime.

Cost Savings from Reduced Incidents and Liabilities

Lowered Costs of Incident Management: Preventing a failure is typically much more cost-effective than dealing with the consequences. Hazard identification and risk assessments help avoid expenses related to spill cleanup, legal penalties, and environmental remediation [13].

Insurance and Liability Reduction: Demonstrating proactive risk management can reduce liability and insurance premiums, as insurers and regulatory bodies view thorough hazard identification and risk mitigation favourably.

Improved Compliance with Regulations and Standards

Meeting Regulatory Requirements: Many regulatory agencies require ongoing risk assessments and monitoring in tailings facilities. Hazard and risk evaluations help ensure compliance with these requirements, avoiding potential fines, shutdowns, or legal action.

Alignment with Industry Standards: Adhering to recognized tailings management standards (e.g., Global Industry Standard on Tailings Management [19]) builds credibility and demonstrates a commitment to responsible mining practices.

Enhanced Community and Stakeholder Trust

Transparency in Risk Management: Engaging in hazard identification and risk assessments and sharing these findings with stakeholders improve transparency, building trust with local communities, investors, and regulatory bodies.

Stakeholder Confidence: Clear, proactive risk management demonstrates that a mining company prioritizes safety and environmental protection, which can improve community relations and support long-term social license to operate [4,13].

Supporting Emergency Preparedness and Response Planning

Effective Emergency Action Plans: Understanding hazards and assessing risks allows for tailored EAPs, ensuring that response plans are specific to the most likely and impactful incidents. This preparation helps prevent loss and reduces response time in an emergency.

Enhanced Crisis Management: With a detailed risk profile, operators are better equipped to predict potential emergencies, allowing them to develop, practice, and improve emergency response scenarios for a more effective response.

Continuous Improvement in Tailings Management

Data-Driven Improvements: Regular hazard and risk evaluations provide valuable data that helps improve future designs and operational practices, contributing to ongoing enhancements in tailings management.

Adaptive Management Practices: Frequent assessments enable operators to respond to changes in external conditions, like climate impacts or increased production, adapting practices to continuously meet evolving risks.

3. OVERVIEW AND COMPARISON OF HAZARDS AND RISKS IN TAILINGS MANAGEMENT

In tailings management, hazard and risk have distinct meanings, though they are related concepts.

3.1. Definition

Hazard is any potential source of harm or adverse effect. In tailings management, a hazard refers to something inherent within the tailings facility that could potentially lead to a failure or environmental incident, such as structural instability, high water pressure, or toxic materials within the tailings.

Risk is the likelihood of the hazard causing harm or damage, combined with the consequences if it occurs. Risk considers both the probability of a failure event (such as a dam breach) and the impact it would have on people, the environment, and assets.

3.2. Focus

Hazard focuses on what could go wrong based on the conditions within the tailings facility or surrounding environment. For example, seepage within a tailings dam is a hazard because it has the potential to weaken the structure.

Risk focuses on how likely it is that something will go wrong and how severe the consequences would be. In the abovementioned example, if seepage is observed, the risk assessment would consider the probability of dam failure and the downstream impacts on communities and ecosystems.

3.3. Control Measures

Measures are taken to reduce or eliminate the hazardous scenarios. For example, installing drainage systems controls the hazard of excessive water in the tailings dam.

Risk control involves measures to minimize either the likelihood or consequence of the hazard causing harm. In tailings management, this could include monitoring systems, emergency preparedness plans, and community evacuation protocols.

3.4. Relationship

A hazardous scenario in tailings management exists independently of its risk level. The same hazard (e.g., high water levels) may represent different levels of risk depending on factors like location (fig. 2), weather conditions, dam construction, and nearby communities.

Risk combines the presence of a hazard with the probability of occurrence and severity of its effects. Risk = ∫(Hazard, Likelihood, Consequence).

Fig. 2 – Example of very high consequence of failure due to proximity of population (Source: Google Earth, March 30, 2024)

4. RECENT MAJOR TAILINGS DAM FAILURES

Historical incidents [2, 3, 10, 11, 14] underscore the importance of rigorous safety standards, risk assessments, and management practices to prevent future tailings dam failures. Each of these cases has led to changes in regulations, technology, and industry standards to improve tailings management worldwide.

The Table 1 provide a summary based on widely reported historical data and publicly documented incidents of tailings dam failures from several reputable sources. For a thorough

review and validation of each incident, including specific technical details and in-depth studies, here are some key sources often referenced for information on tailings dam failures:

- ICOLD (International Commission on Large Dams) [5]- Maintains a database on dam incidents and failures, including technical analyses and reports.
- WISE Uranium Project [10]- A comprehensive database on tailings dam failures, which documents incidents, causes, and environmental impacts worldwide.
- United Nations Environment Programme (UNEP) - Publishes reports on tailings management and provides data on significant tailings dam incidents.
- Global Industry Standard on Tailings Management (GISTM) - Developed by the ICMM, UNEP, and PRI, often references case studies in support of best practices.

These sources have helped establish detailed records and analyses that inform industry standards, regulations, and research in tailings dam management. Here's a table summarizing some of the most notable tailings dam failures in recent history, including information about their locations, causes, and consequences. This table provides a snapshot of incidents that have shaped modern tailings management practices.

Table 1 – Summary of recent dam failures

Year	Location	Facility	Cause of Failure	Consequences
1971	Buffalo Creek, USA	Buffalo Creek Dam	Overtopping due to heavy rainfall	Flooding of communities, 125 deaths, environmental contamination
1985	Stava, Italy	Stava Dam	Structural instability and poor design	268 deaths, major environmental damage, contamination of local waterways
1994	Merriespruit, South Africa	Merriespruit Tailings Dam	Overtopping and embankment collapse	17 deaths, extensive flooding and environmental contamination
1998	Aznalcóllar, Spain	Los Frailes Tailings Dam	Foundation failure due to weak substrate	Extensive contamination of Guadiamar River, 4.5 million cubic meters of tailings released
2000	Baia Mare, Romania	Baia Mare Dam	Overtopping and wall breach	Major cyanide spill into the Danube River, extensive fish kills, cross-border environmental damage in Eastern Europe
2008	Kingston, USA	TVA Kingston Fossil Plant	Wall collapse due to structural failure	4.2 million cubic meters of coal ash released, damage to homes, contamination of nearby rivers
2010	Kolontár, Hungary	Ajka Alumina Plant Dam	Structural failure, embankment collapse	10 deaths, severe environmental and health impacts, contamination of rivers and surrounding land with toxic sludge
2014	Mount Polley, Canada	Mount Polley Tailings Dam	Foundation failure	Release of 24 million cubic meters of tailings into local water bodies, major environmental and fish habitat damage

Year	Location	Facility	Cause of Failure	Consequences
2015	Mariana, Brazil	Fundão Dam	Liquefaction triggered by poor drainage	19 deaths, destruction of communities, severe environmental damage to Rio Doce, largest mining disaster in Brazil
2019	Brumadinho, Brazil	Córrego do Feijão Dam	Liquefaction due to structural failure	270 deaths, catastrophic flooding of tailings, significant environmental and community impacts
2020	Gonghe, China	Yichun Luming Dam	Heavy rainfall-induced failure	Significant environmental contamination, major release of tailings into the river

5. CHARACTERISTICS OF TAILINGS STORAGE FACILITIES

TSFs are typically designed as either embankment dams or impoundments, where the tailings are deposited in a controlled manner [3]. The stability of these structures is paramount due to the large volumes of material they must contain, sometimes under adverse environmental conditions [13].

Several factors contribute to the geotechnical risks in TSFs:

- Foundation Conditions: The strength and stability of the foundation materials (soil or rock) are critical in supporting the weight of the tailings and the embankment structure.
- Tailings Properties: The physical and chemical characteristics of the tailings, including particle size distribution, water content, and the presence of contaminants, can impact the mechanical resistance, consolidation and settlement behaviour.
- Construction Methods: TSFs may be constructed using various methods, including upstream, downstream, and centerline construction. Each method has its own inherent risks and benefits in terms of stability.
- Seepage and Drainage: Inadequate drainage systems can lead to water pressure buildup within the tailings, reducing the effective stress and increasing the risk of liquefaction [7].
- Seismic Activity: TSFs located in seismically active regions are vulnerable to earthquakes, which can trigger failures through dynamic loading and liquefaction of tailings.
- Extreme Weather Events: Climate change is exacerbating the frequency and intensity of extreme weather events, such as heavy rainfall and flooding, which can overwhelm the capacity of TSFs to safely contain tailings.

6. KEY COMPONENTS OF TAILINGS DAMS

A tailings dam has several key components that ensure it effectively contains and manages the waste materials (tailings) produced by mining operations. Each component is designed to contribute to the dam's stability, safety, and environmental management. The primary components of a tailings dam include:

- Embankment or Dam Wall: The main structure that forms the physical barrier to contain the tailings. It is often constructed using materials from mine waste, soil, or rock, and it is built in various ways (e.g., upstream, downstream or centerline) depending on specific site conditions and the type of tailings.
- Starter Dam: The initial or base dam constructed before tailings deposition begins. It supports the tailings and often serves as the foundation for future dam lifts or raises.
- Tailings Pond: The primary storage area within the dam where the slurry (a mixture of fine solid tailings and water) is deposited. As tailings settle, the water is either recycled or drained from the pond.
- Decant System: A system that removes water from the tailings pond, either through gravity decant pipes, siphons, or decant towers. This system is crucial for maintaining low water levels within the pond, which helps reduce pore pressure in the tailings and improve stability.
- Spillway or Overflow Structure: An outlet or channel designed to control the flow of excess water from the tailings dam, preventing overtopping during heavy rainfall or flood events. This structure helps to manage stormwater and avoid unintended discharge of tailings.
- Seepage Control System: Includes various structures designed to minimize water leakage from the dam. Components may include an impermeable liner at the base, filter drains, and cut-off walls that prevent seepage from weakening the dam foundation and leading to potential failure.
- Drainage System: A series of drainage pipes or gravel layers within and around the embankment designed to control pore water pressure by allowing water to drain freely. This helps maintain the structural integrity of the dam.
- Freeboard: The vertical distance between the maximum tailings level and the top of the embankment. Maintaining adequate freeboard is crucial to preventing overtopping and ensuring sufficient capacity for extreme weather events.

These components, when properly designed, constructed, and maintained, work together to ensure the dam's stability, prevent environmental contamination, and protect nearby communities from potential dam failures.

7. RISKS ASSOCIATED WITH TAILINGS DAMS COMPONENTS

Each component of a tailings dam carries specific risks, as failures or malfunctions in any of these areas can compromise the entire structure. Managing these risks requires comprehensive design, rigorous maintenance, ongoing monitoring, and emergency preparedness planning to safeguard against potential failures and environmental impacts. Here's an overview of the risks associated with each component:

Embankment or Dam Wall

Risk of Instability: Improper design, construction techniques or poor-quality materials (fig. 4) can compromise the embankment's stability, potentially leading to slope failures.

Seepage-Related Failures: Inadequate seepage control within the embankment (fig. 3 and 5) can cause internal erosion or piping, resulting in a sudden breach.

Fig. 3 – Example of erosional seepage at the toe of a tailings dam

Liquefaction Risk: Tailings within the dam can become unstable and liquefy under seismic loading or if saturated, increasing the risk of collapse [7].

Starter Dam Foundation Issues: A poorly designed or constructed starter dam foundation (fig. 4) may fail to provide a stable base for the entire structure, particularly when additional layers or "lifts" are added over time.

Fig. 4 – Use of poor-quality and degradable materials in a tailings dam foundation

Settlement and Compression: If the starter dam settles or compresses unevenly, it can create differential movement, compromising the structure's overall stability.

Fig. 5 – Examples of erosional process at downstream embankment of tailings dam

Tailings Pond

Water Management Failures: Inadequate drainage or excessive water levels can increase pore water pressure in the tailings, raising the risk of liquefaction [7].

Environmental Contamination: Failure to manage or contain water within the tailings pond can lead to contamination of nearby surface and groundwater systems, especially if harmful chemicals are present.

Decant System

Blockages or Malfunctions: Clogging of decant pipes or failure of decant towers can prevent proper drainage, leading to increased water pressure and instability within the dam.

Overloading: If the decant system is not sized correctly, it may be unable to handle sudden water influxes (e.g., from heavy rainfall), increasing the risk of overtopping or internal pressure build-up.

Spillway or Overflow Structure

Inadequate Capacity: Spillways designed with insufficient capacity may fail to handle extreme rainfall or flood events, leading to overtopping and erosion of the dam's embankment (fig. 6).

Blockages: Debris or sediment buildup can block the spillway, causing water to back up within the tailings pond and heightening the risk of overtopping (fig. 6b).

a) b)

Fig. 6 – Examples of spillway deficiencies: a) Sign of slab damage with potential risk of seepage; b) Lack of engineering design for increase of spillway capacity.

Seepage Control System

Seepage and Erosion: Failure to properly manage seepage (fig. 7) can result in internal erosion or piping, which may lead to a sudden collapse of the dam.

Liner Damage or Degradation: Damaged liners or cut-off walls may allow water to seep through the dam foundation, compromising its stability and creating pathways for contamination.

Fig. 7 – Example of erosional seepage at the toe of a tailings dam

Drainage System

Clogging or Failure: Clogged drainage pipes or mattress layers prevent water from draining properly, increasing pore water pressure and reducing shear strength in the embankment, leading to instability.

Insufficient Drainage Capacity: If the drainage system cannot handle the volume of water present (fig. 8), it may lead to water saturation and increase the risk of slope failure.

a)

b)

c)

Fig. 8 – Examples of surface drainage issues: a) Reduced capacity by sediments; b) Obstruction by backfill materials; c) Lack of engineering design.

Freeboard

Insufficient Freeboard: If freeboard is inadequate, there's a heightened risk of overtopping during heavy rainfall or flooding, potentially leading to erosion and structural failure.

Loss of Freeboard Over Time: Ongoing deposition of tailings or changes in water levels within the pond can reduce freeboard, increasing the risk of overtopping.

8. CREDIBLE FAILURE MODE IN TAILINGS MANAGEMENT

A credible failure mode is a plausible sequence of events or mechanisms that could lead to the failure of a TSF, based on the specific engineering, geotechnical, operational, and environmental conditions of the facility.

It's identified through analysis, field inspections, and historical data, and considers realistic environmental and operational scenarios (e.g., extreme weather, seismic activity, or operational issues) rather than hypothetical or highly improbable events.

8.1. Characteristics of a Credible Failure Mode

Plausibility: It is based on known conditions, such as the structural properties of the dam, tailings material properties, climate conditions, and other site-specific risk factors.

Cause and Mechanism: Each failure mode has a defined cause and failure mechanism, such as seepage-induced piping, liquefaction, embankment slope instability, overtopping, or erosion.

Consequences: The failure mode includes a realistic assessment of potential impacts, including the volume of tailings that could be released, the area affected, and potential downstream effects.

8.2. Examples of Credible Failure Modes

Liquefaction: In cases where tailings are loose and water-saturated, especially during seismic events, liquefaction can reduce the structural integrity of the tailings and embankments, leading to a sudden collapse [7].

Seepage and Piping: Water movement through the dam can erode fine particles, forming internal channels (piping) that weaken the structure and eventually cause an embankment breach (Figures 9, 10 and 11).

Fig. 9 – Uncontrolled seepage at the toe of a tailings dam.

Fig. 10 – Sinkhole caused by an internal erosion of a tailings dam.

Fig. 11 – Uncontrolled runout and seepage at the toe of a tailings dam.

Slope Instability: Instability in the embankment's slopes, often due to poor compaction, weak materials, or excessive loading, can result in gradual deformation (fig. 12) or sudden collapse.

Fig. 12 – Crack as an instability sign and unreliable remediation work.

Overtopping: Excessive water from rainfall or mismanaged water levels can lead to overtopping, causing erosion or structural damage to the dam.

8.3. Role in Risk Management

Identifying credible failure modes is essential for risk assessment and failure prevention. By recognizing these plausible failure pathways, tailings managers can implement specific preventive and mitigative measures tailored to address each risk scenario.

Failure Modes and Effects Analysis (FMEA) [12] and Probabilistic Risk Assessment (PRA) are commonly used to analyse credible failure modes, estimate their likelihood, and evaluate potential impacts.

8.4. Application in Safety and Emergency Planning

Understanding credible failure modes allows for the development of tailored emergency action plans (EAPs) and targeted monitoring systems to detect early warning signs of each failure type.

For instance, if liquefaction is identified as a credible failure mode, enhanced seismic monitoring and emergency stabilization procedures may be implemented.

9. COMMON MISUNDERSTANDING OF A CREDIBLE FAILURE MODE

In tailings risk management, the most common misunderstanding of a credible failure mode is often the assumption that any low-probability event should be excluded as "not credible" simply because it seems unlikely. This misunderstanding can lead to a lack of preparation for significant failure scenarios and result in critical risks being overlooked. Here are some specifics on why this misconception is problematic:

9.1. Confusing Low Probability with Impossibility

A failure mode can be credible and have low probability at the same time. Credibility does not imply high likelihood; rather, it means the failure mode is realistically (physically) possible given the site's characteristics, materials, operational practices, and environmental factors.

Focusing only on the most likely events can lead to a disregard for rare but potentially catastrophic events, such as seismic-induced liquefaction or extreme rainfall-induced overtopping.

9.2. Underestimating Complex or Interacting Failure Modes

Some failure modes are complex or involve cascading factors, such as the combination of internal erosion (piping) with external events like heavy rainfall. Overlooking interactions between multiple risk factors can result in an incomplete understanding of credible failure scenarios.

9.3. Overreliance on Past Performance

Another misconception is the belief that if a tailings facility has performed well in the past, it won't experience failure in the future. Past performance does not guarantee future safety, especially in cases where site conditions, climate impacts, or operational practices have evolved. Each facility should be regularly re-evaluated, as new credible failure modes can emerge with changing conditions.

9.4. Overlooking "Unknown Unknowns"

The idea that all failure modes must be apparent or obvious can lead to an underestimation of rare or poorly understood mechanisms. For example, unexpected internal erosion or unforeseen liquefaction events have led to serious incidents even in seemingly well-maintained facilities. Recognizing the potential for unknown risks, particularly in older dams, is essential.

9.5. Assuming Mitigative Measures Fully Eliminate Risk

While mitigative measures (like drainage improvements or slope reinforcements) are essential, they do not completely eliminate risk. Some assume that once these measures are in place, a failure mode is no longer credible. However, credible failure modes remain as potential pathways that require ongoing monitoring and adaptive management.

9.6. Failure to Consider the Impact of Rare Extreme Events

Some operators exclude rare but realistic events, like extreme floods or seismic events, under the assumption that these are beyond credible planning. However, tailings facilities should be

designed with these rare extreme events in mind, especially as climate change increases the frequency and severity of such conditions.

10.INITIAL EVENTS, FAILURE MODES, AND UNWANTED EVENTS CONCEPTS

In tailings risk management, initiating events, failure modes, and unwanted events are interrelated concepts that describe the sequence leading to a potential incident within a tailings storage facility (TSF). Understanding their relationships is key to identifying, assessing, and managing risks.

10.1. Definitions and Relationships

Initiating Event: An initiating event is an external or internal trigger that sets off a sequence leading towards a failure mode (figures 13 and 14). It could be an environmental factor, operational error, or unforeseen event that challenges the stability of the tailings dam or TSF. Examples include extreme rainfall, seismic activity, operational mistakes (e.g., improper tailings deposition), or equipment failure (e.g., drainage system blockage).

Fig. 13 – Examples of initial event: Failure of drainage channel at downstream surface of a tailings dam embankment.

Fig. 14 – Example of initial event: Inefficiency of surface drainage and erosional process.

Failure Mode: A failure mode describes the specific way in which a structure or system could fail due to the initiating event. It is the mechanism through which the TSF's integrity is compromised, leading to structural or functional failure. Examples include overtopping, internal erosion or piping, liquefaction, and slope instability. Each failure mode defines how the structure might break down in response to the initiating event.

Unwanted Event: The unwanted event is the outcome or consequence of a failure mode occurring. It's the actual incident that poses risks to people, property, or the environment, often manifesting as a breach or release of tailings material. Examples include tailings release into downstream rivers, flooding of communities, or contamination of groundwater and soil.

10.2. Relationship Between Initiating Event, Failure Mode, and Unwanted Event

These three elements form a causal sequence in which each element triggers or enables the next, often modeled as: Initiating Event ➔ Failure Mode ➔ Unwanted Event.

Step 1: Initiating Event. An initiating event, like extreme rainfall, creates pressure on the dam structure by raising water levels within the TSF. The event is the starting point for a potential failure sequence.

Step 2: Failure Mode. Due to the initiating event, a failure mode (e.g., overtopping) occurs. Here, overtopping is the mechanism by which the dam's capacity is overwhelmed, compromising its structural integrity.

Step 3: Unwanted Event. The overtopping causes a breach, leading to the unwanted event (a tailings release) that contaminates nearby ecosystems and potentially impacts communities.

The example scenario illustrating the sequence is:

- Initiating Event: Heavy, prolonged rainfall exceeds the design storm capacity of a TSF.
- Failure Mode: Overtopping occurs due to insufficient freeboard or drainage capacity.
- Unwanted Event: The embankment erodes and breaches, releasing tailings and contaminated water downstream.

10.3. Importance of Understanding the Sequence in Tailings Risk Management

Risk Assessment and Prevention: By identifying likely initiating events, operators can analyse credible failure modes and take preventive measures (e.g., raising the freeboard) to prevent the sequence from leading to an unwanted event.

Monitoring and Early Detection: Each step in the sequence provides opportunities for monitoring. For instance, rainfall can be measured (initiating event), water levels monitored

(failure mode indicators), and alarm systems implemented for breach detection (unwanted event).

Emergency Preparedness: Understanding potential unwanted events allows for targeted emergency action plans. If a tailings release is the unwanted event, resources can be allocated to contain and mitigate its impact.

11. QUALITATIVE AND QUANTITATIVE RISK EVALUATION

In tailings risk management, qualitative and quantitative risk evaluation methods are used to assess the risks associated with tailings storage facilities (TSFs). Both approaches play crucial roles in identifying, prioritizing, and managing potential hazards, but they differ in terms of data requirements, detail, and complexity.

11.1. Qualitative Risk Evaluation

Qualitative risk evaluation is a descriptive approach that assesses risks based on the likelihood of potential hazards and the severity of their consequences. It relies on expert judgment, historical data, and predefined risk scales.

11.2. Characteristics

Use of Risk Matrices: Commonly employs risk matrices to classify risks as high, medium, or low based on likelihood and consequence ratings (e.g., a 3x3 or 5x5 matrix).

Simple Scales: Likelihood and consequences are typically described using scales such as “rare,” “possible,” “likely” (for likelihood) and “minor,” “moderate,” “severe” (for consequences).

Data Requirements: Less data-intensive and does not require precise numerical inputs, making it suitable for early-stage assessments or when data is limited.

11.3. Advantages

Ease of Use: Quick, cost-effective, and accessible, as it relies primarily on expert judgment and established guidelines.

Identification of Key Risks: Useful for prioritizing the most significant risks, especially for facilities with limited historical data or when quantitative analysis is not feasible.

Ideal for Initial Screening: Suitable for the initial phases of risk assessment to identify which risks need more detailed analysis.

11.4. Limitations

Subjectivity: Relies heavily on expert judgment, which may lead to variability in risk ratings.

Lack of Precision: Does not provide specific probability or consequence values, limiting its precision.

Not Suitable for Detailed Analysis: Limited in evaluating complex interactions and cumulative risks across a large tailings facility.

Examples of qualitative tools are:

- Failure Modes and Effects Analysis (FMEA): Evaluates potential failure modes and ranks them based on severity, occurrence, and detectability [12].
- Bow-tie Analysis: Maps out the causes, preventive measures, and potential consequences of specific failure scenarios.

11.5. Quantitative Risk Evaluation

Quantitative risk evaluation uses numerical data and statistical methods to calculate specific probabilities of failure and expected consequences, leading to more precise risk assessments.

11.6. Characteristics

Detailed Data Analysis: Uses site-specific data, historical failure rates, and probabilistic models to assess the likelihood and consequence of risks.

Numeric Outcomes: Provides measurable values for both probability and consequence, such as the annual probability of failure and estimated volume of tailings release.

Advanced Statistical Methods: Incorporates techniques like Probabilistic Risk Assessment (PRA), Monte Carlo simulations, and fault tree analysis.

11.7. Advantages

Precision and Reliability: Quantifies risk, which allows for specific comparisons and cost-benefit analyses.

Data-Driven Decision-Making: Enables informed, data-based decisions on risk mitigation, emergency preparedness, and resource allocation.

Supports Regulatory Compliance: Many regulatory frameworks and guidelines require quantitative risk assessments, especially for high-risk facilities.

11.8. Limitations

Data Intensive: Requires a large amount of high-quality data, including environmental factors, tailings characteristics, and facility history.

Complex and Costly: Requires specialized knowledge, software, and potentially significant financial resources.

May Not Capture Uncertainties Well: While quantitative methods are precise, they may miss unknown risks or factors if not modelled accurately.

11.9. Examples of Quantitative Tools

Probabilistic Risk Assessment (PRA): Calculates the likelihood of different failure modes and the expected consequence, often using historical and statistical data.

Event Tree Analysis (ETA): Examines potential outcomes of initiating events and the probabilities of each pathway to failure.

Monte Carlo Simulation: Uses repeated random sampling to model uncertain variables and their impact on tailings failure probability and consequences.

11.10. Applications in Tailings Risk Management

Both qualitative and quantitative methods are integral to a comprehensive risk management strategy for TSFs:

Qualitative Assessments: Useful in early stages to identify critical hazards and prioritize areas for detailed analysis.

Quantitative Assessments: Appropriate for detailed risk evaluations, especially in high-risk facilities or complex scenarios requiring precise data.

Combining both approaches provides a balanced risk assessment framework, starting with qualitative assessments to screen for major hazards and progressing to quantitative methods for high-priority risks and regulatory compliance. This layered approach helps ensure both comprehensive hazard identification and precise, data-backed decision-making in tailings management.

12. CASE STUDY: QUANTITATIVE RISK ASSESSMENT OF MOUNT POLLEY TAILINGS STORAGE FACILITY

The Mount Polley tailings storage facility in British Columbia, Canada, experienced a significant failure in 2014, releasing approximately 24 million cubic meters of tailings and water into Polley Lake and surrounding areas (fig. 15). Following this failure, an in-depth investigation and quantitative risk assessment (QRA) was conducted to understand the causes and prevent future incidents [6].

Fig. 15 – Mount Polley tailings dam storage facility breach (Source: Morgenstein et al., 2016a).

The evaluation of the Mount Polley TSF was extensive, involving probabilistic risk assessment methods to assess the likelihood of different failure mechanisms and the consequences associated with each. Key components of the assessment included:

- Data Collection: Detailed historical data on facility design, construction methods, geotechnical parameters, and operational records were collected.
- Failure Mode and Effects Analysis (FMEA): The analysis identified critical failure modes, including overtopping, seismic-induced liquefaction, and piping. Each failure mode was assigned a probability based on historical performance and tailings dam industry standards [12].
- Probabilistic Analysis Using Event Tree and Fault Tree Models: Event Tree Analysis (ETA) was used to map out potential sequences of events starting from various initiating events (e.g., heavy rainfall, seismic activity) to possible outcomes, estimating the probability of each pathway leading to failure. Fault Tree Analysis (FTA) was used to break down the mechanisms that could contribute to failure at each stage. This provided a quantitative estimate of each failure mode's probability based on various contributing factors.
- Consequence Assessment: The QRA included consequence modelling to evaluate the potential impact of each failure mode. The analysis quantified potential tailings release volumes, environmental impacts, and downstream effects on ecosystems and communities.
- Monte Carlo Simulation: To account for uncertainties in geotechnical parameters and environmental conditions, Monte Carlo simulations were conducted. This approach

provided a range of possible outcomes and a probabilistic distribution of failure scenarios, helping to quantify the potential variability in risk.

The quantitative risk evaluation revealed vulnerabilities in the TSF, particularly related to the embankment's stability under seismic and rainfall events. This probabilistic analysis emphasized that the original design underestimated certain risks, which contributed to the failure. Key insights included:

- High Probability of Liquefaction: Under certain conditions, seismic events had a higher-than-acceptable probability of triggering liquefaction within the tailings structure.
- Significant Consequences of Overtopping: The model showed that overtopping could lead to severe environmental impacts, warranting improved water management practices.

13. PREVENTIVE AND MITIGATIVE MEASURES

13.1. Distinction of preventive and mitigative measures

The main difference between preventive and mitigative measures in tailings risk management lies in their timing, purpose, and approach to risk.

Timing and Objective

Preventive Measures are applied before any issues arise to avoid or reduce the likelihood of potential risks. These are proactive strategies aimed at maintaining stability and preventing failures by addressing risk factors early on.

Mitigative Measures are implemented after an issue or risk has been identified or has started to develop. The goal is to control or minimize the impact of the risk to prevent it from escalating into a full-blown failure. Mitigative actions are reactive, focusing on damage control and impact reduction.

Approach to Risk

Preventive Measures focus on eliminating or minimizing known risk factors through careful planning, design, and operation. Examples include robust engineering designs and construction, effective water management, and training for personnel to reduce human error.

Mitigative Measures focus on containing or responding to risks that have already manifested or are actively posing a threat. These actions may include emergency response plans, rapid water level reduction, or reinforcement of weakened embankments.

Risk Management Strategy

Preventive Measures are part of a proactive risk management strategy to avoid hazardous scenarios by addressing underlying causes.

Mitigative Measures are part of a reactive risk management strategy that aims to contain, respond to, or reduce consequences from a hazardous scenario that has already emerged.

In essence, preventive measures aim to stop risks from developing, while mitigative measures aim to control the consequences when risks are already present or actively threatening the stability of a tailings storage facility. Together, they create a comprehensive approach to managing tailings risks effectively.

13.2. Preventive measures in tailings risk management

Preventive measures in tailings risk management aim to reduce the likelihood of failures, minimize environmental impact, and ensure the safety of communities and ecosystems nearby. Here are the key preventive strategies commonly used in tailing storage facilities (TSFs):

- Improved Design and Construction Standards
  - Site-Specific Design: Tailings facilities should be designed with consideration for site-specific factors such as soil composition, climate, seismicity, and hydrology. This tailored approach improves stability and resilience.
  - Choosing Appropriate Construction Methods: Selecting the right construction method (e.g., upstream, downstream, or centerline) based on site conditions and materials available can significantly reduce risks. For example, downstream construction is generally more stable than upstream in seismically active regions.
  - Seepage and Drainage Controls: Installing proper drainage systems (e.g., underdrains, toe drains, and filter layers) helps prevent water buildup in the tailings, reducing pore water pressure and minimizing the risk of liquefaction and slope instability.
- Regular Monitoring and Surveillance
  - Geotechnical Instrumentation: Installing piezometers, inclinometers, and settlement markers in and around the TSF allows for real-time monitoring of pore pressure, ground movement, and settlement. These provide early warnings of potential instability.
  - Remote Sensing and Drones: Remote sensing technologies, including drones and satellite imagery, can detect subtle changes in the dam’s surface, such as settlement or erosion. This technology provides high-frequency monitoring without the need for physical presence.
  - Water Quality Monitoring: Regular monitoring of water quality in nearby surface and groundwater systems helps detect contamination early, allowing for rapid intervention.

- Routine Inspections and Maintenance
  - Frequent Inspections: Regular inspections by qualified geotechnical engineers identify minor issues, such as cracks or erosion, before they become serious problems.
  - Clearing Blockages: Keeping spillways, decant systems, and drainage channels clear of debris and sediment ensures they function effectively during heavy rainfall or rapid tailings deposition.
  - Addressing Vegetation Growth: Uncontrolled vegetation can obstruct inspection and damage drainage components. Regular removal or trimming of vegetation is essential.
- Water Management
  - Water Level Control: Maintaining low water levels in the tailings pond minimizes pore pressure, reducing the risk of liquefaction and enhancing dam stability.
  - Efficient Decant Systems: Properly designed decant towers, pipes, and siphons allow for the safe removal of excess water, particularly after heavy rains.
  - Spillway Design for Extreme Events: Spillways should be designed to handle extreme rainfall and flood events. Adequate freeboard (space between water level and the top of the dam) is crucial to avoid overtopping.
- Drainage and Seepage Control
  - Installing Cut-Off Walls and Liners: Cut-off walls or impermeable liners at the base of the dam can help contain seepage and prevent groundwater contamination, as well as reduce the risk of internal erosion or piping failures.
  - Improving Drainage Channels: Expanding or upgrading drainage channels within and around the TSF helps remove excess water and reduces pore pressure, preventing instability caused by water buildup.
  - Filter Zones for Seepage: Installing filter zones within the embankment helps prevent internal erosion by allowing controlled seepage while capturing fine particles that could weaken the dam’s structure.
- Regular Stability Assessments
  - Slope Stability Analysis: Performing stability analyses at regular intervals helps assess the factor of safety under current conditions and identify any evolving risks.
  - Seismic Stability Analysis: In seismically active areas, frequent seismic stability assessments ensure the TSF can withstand expected earthquake loading without compromising stability.
- Structural Modifications for Increased Capacity and Stability
  - Dam Raise Optimization: If raising the dam to increase capacity, using downstream or centerline construction methods improves structural stability, especially compared to upstream methods.

  - Strengthening Foundation and Substrate: Adding layers of gravel or reinforced materials beneath the dam's foundation can improve load-bearing capacity and reduce the risk of foundation settlement or failure.
- Employee Training and Awareness
  - Emergency Response Drills: Regular emergency drills prepare staff to respond effectively in case of a dam breach or other emergencies, minimizing risks to life and property.
  - Training on Monitoring Equipment: Staff should be trained to use and interpret monitoring equipment and data effectively, enabling early detection and response to signs of instability.
  - Tailings Management Protocols: Educating employees on tailings management procedures fosters a safety-first culture, reducing the likelihood of operational errors that may lead to instability.
- Environmental and Social Risk Management
  - Community Engagement: Engaging with local communities and keeping them informed about TSF risks and safety measures builds trust and prepares them for potential emergencies.
  - Environmental Protection Measures: Implementing barriers or liners, especially for facilities containing hazardous tailings, minimizes the risk of groundwater or surface water contamination.
  - Biodiversity Monitoring: Ongoing assessments of local biodiversity can provide insights into potential ecological impacts, leading to proactive environmental management strategies.
- Tailings Reprocessing and Reduction
  - Tailings Reprocessing: Extracting residual valuable minerals from tailings reduces the volume of waste, minimizing storage risks and freeing up space in the TSF.
  - Dry Stacking: Using dry stacking methods, which involve dewatering the tailings and storing them in dry stacks, is a safer alternative in arid regions. Dry-stacked tailings are more stable than wet tailings, particularly in seismic or high rainfall zones.
- Comprehensive Risk Assessments and Audits
  - Probabilistic Risk Assessments (PRA): PRA methods quantify the likelihood of different failure scenarios, enabling better-informed decision-making and mitigation planning.
  - Failure Mode and Effects Analysis: FMEA helps to identify possible failure modes, evaluate their effects, and prioritize preventive actions based on risk level.
  - Independent Audits: Periodic reviews by third-party experts provide an unbiased assessment of TSF conditions, identifying overlooked risks and ensuring compliance with best practices.

- Third-Party Audits and Continuous Improvement Programs
  - Independent Risk Audits: Engaging third-party experts for independent reviews provides an unbiased assessment of the TSF's condition and highlights risks that may not be apparent internally, supporting ongoing risk management and improvements.
  - Reviewing and Updating Risk Management Protocols: Conducting regular assessments of risk management practices, especially after minor incidents or environmental changes, ensures that the TSF operates under the latest safety standards and industry practices.
- Post-Closure Management [4]
  - Post-Closure Monitoring: Even after a TSF is decommissioned, long-term monitoring of stability and seepage is essential to detect potential issues, as residual tailings can still pose risks.
  - Revegetation and Erosion Control: Covering the TSF with vegetation helps stabilize the surface and reduces erosion, which is essential for long-term environmental stability.
  - Groundwater Monitoring: Ensuring that nearby groundwater remains uncontaminated post-closure protects both local ecosystems and community water supplies.

13.3. Emergency responses and mitigative measures in tailings risk management

Mitigative measures in tailings risk management focus on addressing and reducing the consequences of identified risks and preventing escalation if issues arise. These actions are critical for managing potential hazards within tailing storage facilities (TSFs) and ensuring that any signs of instability are effectively handled before they lead to failures. Here are the primary mitigative strategies used in tailing risk management:

- Enhanced Monitoring and Surveillance
  - Real-Time Monitoring Systems: Installing advanced monitoring systems, including piezometers, inclinometers, and remote sensing tools, enables continuous observation of dam conditions. Real-time data allows for early detection of changes in pore pressure, seepage rates, and structural deformations, which are critical indicators of instability.
  - Drone and Satellite Monitoring: Using drones and satellites to survey the dam's surface can help identify subtle shifts, cracks, or signs of erosion, enabling proactive response before significant structural damage occurs.
- Rapid Water Level Management
  - De-Watering Tailings: Reducing the water content within tailings can help decrease pore water pressure, which stabilizes the structure and mitigates the risk of liquefaction or internal erosion.

  - Emergency Water Removal Systems: Installing siphon systems or pumping systems to quickly reduce water levels during heavy rainfall or seepage can prevent overtopping and maintain safe water levels within the tailings pond.
  - Spillway Enhancements: Upgrading spillways and emergency overflow structures can handle extreme weather events or sudden water inflows, reducing the risk of dam overtopping.
- Structural Reinforcement and Stabilization
  - Strengthening the Embankment: Placing additional material, such as rock fill or geotextiles, on the dam embankment can help improve stability and reduce seepage, particularly in areas with weak foundation conditions or high pore pressures.
  - Slope Reinforcement: Adding riprap or using vegetation on slopes can reduce erosion and provide additional stability, especially during periods of high rainfall or seismic events.
  - Seismic Reinforcement: In seismic-prone areas, reinforcing the embankment with engineered structures like stone columns or reinforced earth can reduce the risk of liquefaction and increase resistance to earthquake loading.
- Emergency Response and Contingency Planning
  - Developing Detailed Emergency Action Plans (EAPs): EAPs outline specific protocols and response actions in the event of a dam failure or other emergencies, including communication channels, evacuation routes, and responsibilities.
  - Community Evacuation Drills: Working with local communities to develop and practice evacuation drills ensures a rapid and organized response, minimizing potential harm to residents near the TSF.
  - Early Warning Systems (EWS): Automated alarms that activate based on critical thresholds, such as high water pressure or rapid embankment movement, allow for immediate action, enabling quick evacuation or intervention if risks escalate.
- Post-Failure Containment and Environmental Remediation Plans
  - Downstream Containment Structures: In regions with downstream populations or ecosystems, constructing secondary containment berms or dams can capture tailings in the event of a failure, mitigating environmental and human impacts.
  - Environmental Cleanup and Restoration [13]: Preparing resources and equipment for rapid deployment can help contain tailings spills and minimize contamination in nearby water bodies and soils.
  - Groundwater and Surface Water Treatment: Post-spill water treatment facilities should be available to mitigate the effects of contamination and restore water quality if tailings seep into local water systems.

13.4. Controls versus critical controls

In tailings risk management, another common misunderstanding of a credible failure mode is often the assumption that controls and critical controls are the same. This misunderstanding is also responsible to a lack of preparation for significant failure scenarios and result in critical risks being overlooked.

Risk management for tailings facilities involves recognizing potential failure modes and their consequences, and implementing appropriate controls to mitigate these risks. Ensuring the effectiveness of these controls through continuous monitoring, internal reporting, and verification activities is crucial. Qualified and experienced experts must be involved in the development and review of the controls' effectiveness.

A control exists for preventing a potential undesirable event or mitigating its consequences. The absence or failure of such a control would significantly elevate the risk, but if other controls are in place the top event can be avoided or still mitigated.

According to ICMM, control that is essential for preventing a potential undesirable event or mitigating its consequences is termed a critical control. The absence or failure of such a control would significantly elevate the risk, even if other controls are in place.

14.CONCLUSIONS

Tailings Storage Facilities are integral to modern mining operations, but they also represent significant geotechnical risks. Through comprehensive geotechnical risk assessments, mining companies can identify and mitigate potential hazardous scenarios, ensuring the long-term stability of TSFs and minimizing the risk of catastrophic failures. As the industry evolves and environmental challenges such as climate change intensify, it is essential to continuously improve design standards, monitoring practices, and risk management strategies to safeguard both human and environmental health.

A geotechnical risk assessment of a TSF requires identifying and evaluating the various risk factors that could contribute to failure. These factors can be broadly categorized as internal and external risks.

Hazard identification and risk evaluation in tailings management are instrumental in preventing failures, safeguarding human life and the environment, and enhancing the economic sustainability of mining operations. These processes support safe and responsible management, aligning with global standards and protecting company reputation while minimizing risks.

A hazard is the source of potential harm, while risk is the likelihood and consequence of that harm occurring in the context of the tailing storage facility. Managing tailing facilities effectively

involves both identifying hazardous scenarios and assessing and mitigating risks to prevent failures and mitigate consequences.

A credible failure mode in tailings risk management is a realistic failure pathway that considers specific conditions and mechanisms that could compromise a TSF. Identifying and analysing these failure modes enables effective risk mitigation and helps prevent catastrophic failures.

The misunderstandings can lead to a lack of preparation for significant failure scenarios and result in critical risks being overlooked. Addressing these misunderstandings requires a more holistic, site-specific approach that continuously updates risk assessments, considers low-probability events with high impact, and recognizes the importance of ongoing vigilance and adaptability in tailings management practices.

The relationship between initiating events, failure modes, and unwanted events forms a chain of cause and effect in tailings risk management. Addressing each link helps to prevent or mitigate tailings facility incidents, enhancing the safety and sustainability of tailings management practices.

The failure cases study provides a clearer understanding of the probability and impact of tailings dam failure, leading to more robust management practices and regulatory guidance across the mining industry. It highlights the importance of integrating risk assessments to enhance safety and inform design changes. Tailings management guidelines, such as the Global Industry Standard on Tailings Management (GISTM), emphasized the need for risk assessments as part of TSF design and operation.

By implementing the preventive measures, mining companies can improve the safety, stability, and environmental compatibility of tailings storage facilities, helping mitigate the risks of catastrophic failures and long-term environmental damage. Implementing the mitigative measures can significantly reduce the likelihood and consequence of failures in TSFs. When combined with preventive strategies, these approaches create a comprehensive tailings risk management framework that emphasizes safety, stability, and environmental protection.

Various geotechnical instruments such as piezometers, inclinometers, and settlement gauges are installed within the tailings dam to monitor internal water pressure, slope movement, and structural stability. Real-time monitoring allows for early detection of potential issues. Faulty or insufficient monitoring equipment can lead to a lack of accurate, real-time data, making it difficult to identify early warning signs of instability. In cases where monitoring data is not continuously analysed, critical signs of instability may go unnoticed, reducing the time available to take corrective action.

REFERENCES

[1] Morrison, Kimberly Finke. (2022). Tailings Management Handbook - A Life-Cycle Approach. Society for Mining, Metallurgy, and Exploration (SME).

[2] Azam, S., & Li, Q. (2010). Tailings dam failures: A review of the last one hundred years. Geotechnical News, 28(4), 50-53.

[3] Blight, G. E. (2010). Geotechnical engineering for mine waste storage facilities. CRC Press.

[4] Chambers, D. M., & Higman, B. (2011). Long-term risks of tailings dam failure. Environmental Earth Sciences, 62(2), 305-315.

[5] ICOLD (International Commission on Large Dams). (2016). Guidelines on Tailings Dams: Risk Management. Bulletin 181. https://www.icold-cigb.org/GB/bulletins.asp

[6] Morgenstern, N. R., Vick, S. G., & Van Zyl, D. (2016a). Report on Mount Polley Tailings Storage Facility Breach. Independent Expert Engineering Investigation and Review Panel. https://www.mountpolleyreviewpanel.ca

[7] Robertson, P. K. (2010). Evaluation of the flow liquefaction and liquefied strength of tailings. Journal of Geotechnical and Geoenvironmental Engineering, 136(2), 261-270.

[8] Vick, S. G. (1983). Planning, Design, and Analysis of Tailings Dams. Wiley.

[9] World Bank Group. (2019). Mine Tailings Storage: Safety is No Accident. Mining Policy Framework.

[10] WISE Uranium Project. (2021). Chronology of major tailings dam failures. Retrieved from http://www.wise-uranium.org

[11] Lyu, Z., Chai, J., Xu, Z., Qin, Y., & Cao, J. (2019). A comprehensive review on reasons for tailings dam failures based on case history. Advances in Civil Engineering, 2019.

[12] Schafer, H.L.; Beier, N.A.; Macciotta, R. A Failure Modes and Effects Analysis Framework for Assessing Geotechnical Risks of Tailings Dam Closure. Minerals 2021, 11, 1234.

[13] Kossoff, D., Dubbin, W. E., Alfredsson, M., Edwards, S. J., Macklin, M. G., & Hudson-Edwards, K. A. (2014). Mine tailings dams: Characteristics, failure, environmental impacts, and remediation. Applied Geochemistry, 51, 229-245.

[14] Williams, David John. Lessons from Tailings Dam Failures - Where to Go from Here? Minerals 2021, 11, 853. https://doi.org/10.3390/ min11080853

[15] Williams, D. J. (2001). Geotechnical risk assessment for mine waste storage facilities. Proceedings of the International Symposium on Geotechnical Risk and Safety in Practice.

[16] Yu, S., & Li, G. (2013). Risk evaluation of tailings dam failure in China. Environmental Earth Sciences, 68(6), 1615-1625.

[17] Öztürk, N., & Hassanizadeh, S. M. (2018). A geotechnical risk assessment methodology for tailings storage facilities: A systematic approach to assessing and managing risks. Journal of Risk Research, 21(8), 958-975.

[18] ICMM (International Council on Mining & Metals). Tailings Governance Framework – Position Statement. London, 2020.

[19] GISTM (Global Industry Standard on Tailings Management). International Council on Mining and Metals (ICMM), United Nations Environment Programme (UNEP) and Principles for Responsible Investment (PRI). 2020.

[20] Morgenstern, N.R., Vick, S.G., Viotti, C.B. & Watts, B.D. (2016b). Report on the Immediate Causes of the Failure of the Fundão Dam. Fundão Tailings Dam Review Panel.

[21] Robertson, P.K., Melo, L., Williams, D.J. & Wilson, G.W. (2019) Report of the Expert Panel on the Technical Causes of the Failure of Feijão Dam.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

SENSITIVITY ANALYSIS OF A TAILING DAM STABILITY PROPERTIES USING SOBOL INDICES AND SENSITIVITY FACTORS

Gabriela R. Leal[1]
Department of Structural and Geotechnical Engineering, School of Engineering, University of São Paulo
TPF Engenharia Ltda

Adrian T. Siacara[2]
Department of Structural and Geotechnical Engineering, School of Engineering, University of São Paulo

ABSTRACT

Mining dams are increasingly being studied today, especially following recent accidents in Brazil. The stability of these dams can be influenced by various factors, including climatic effects, reservoir water levels, and variability in the geotechnical parameters of the soils composing the dam, among others. These factors can lead to variations in dam behavior, and deterministic studies, sometimes, may not adequately represent reality. Therefore, reliability studies can reduce model uncertainties, yielding results that enhance the safety of the structure. This article aims to study a mining dam located near Belo Horizonte, Minas Gerais, Brazil. Given that the model contains numerous varying parameters, the computational demand is high, thereby increasing the study's costs. Consequently, a sensitivity analysis will be conducted to identify the variables with the greatest influence on the results. The variables studied will include cohesion, friction angle, and specific weight of the embankment, residual soil and saprolite that compose the dam. This study will be conducted in a steady state analysis with the assistance of SEEP/W and SLOPE/W software from Geostudio, coupled with Matlab for the development of the sensitivity analysis. In Sobol indices, the method takes into account all uncertainties, to define the most important variables in this model, and it uses a Monte Carlo estimator of first order to estimate its indices. Three scenarios will be developed, with three different coefficients of variability, so the results for each of these scenarios will be presented. Additionally, reliability analyses are conducted using the First-Order Reliability Method (FORM). A key outcome of FORM analysis is the determination of the sensitivity factors at the design point, which highlight the most influential parameters in the reliability analysis. Finally, the results are compared between the Sobol indices and the sensitivity cosines derived from the FORM method.

Keywords: Sensitivity, Reliability, Tailings, dams.

[1]gabriela.rleal@usp.br, Brazil
[2] adrian.torrico@usp.br, Brazil

1. INTRODUCTION

Mining dams are of great importance to society, and their stability analysis plays a significant role in current studies, especially in countries like the USA, Chile, Brazil, and China, which together account for nearly half of the mining dam accidents worldwide between 1980 and 2020. In Brazil, mining represents about 4% of the Gross Domestic Product (GPD), according to Ipea [1] and there are around 936 mining dams registered in the SIGBM (Integrated Dam Safety Management System), until March 2024 [2].

The most studied and well-understood failure modes by the geotechnical community include slope instability, liquefaction, overtopping, piping, among others. This article will focus on slope instability, which occurs mainly due to landslides driven by gravitational forces.

Slope stability analysis is one of the most uncertain topics in geotechnics, with uncertainties tied to geological anomalies, the spatial variability of soil properties, scarce representative data, variable environmental conditions, approximations in geotechnical models, and human errors [3].

Thus, deterministic and probabilistic methods can be employed for these analyses. In Brazil, deterministic methods are the most commonly used, mainly because they are prescribed by Brazilian norms (NBR 13028) [4]. These methods do not explicitly account for uncertainties, relying instead on conservative parameters and models to address them. However, this approach is not always optimal for stability studies. Therefore, probabilistic methods, which incorporate statistical data analyses, allow for the quantification and integration of uncertainties into the model's development. Nevertheless, these methods can be highly expensive and time consuming.

To reduce computational costs, sensitivity analysis serves as an effective tool to quantify the contribution rate of each model variable to the uncertainty of the model output [5], thereby minimizing the number of model evaluations as much as possible.

In this article, sensitivity analysis will be conducted for a mining dam using the global Sobol method with Kucherenko sensitivity indices, and Sensitivity factors with First-Order Reliability Method (FORM). The analysis will consider the strength parameters of the embankment, residual soil, and saprolite as variables, evaluated across three scenarios with different coefficients of variation for these parameters. The results of this study will be presented for the three scenarios, offering a comparative perspective between the sensitivity and reliability analyses.

2. PROBLEM SETTING

2.1. Hydrological and geotechnical parameters

In this study, analyses of flow and stability will be conducted. For the flow analysis, the SEEP/W software, part of the GeoStudio suite, will be employed to model two-dimensional flow. The software utilizes the partial differential equation described below for flow modeling:

$$(\theta_w \beta_w + \beta)\frac{\partial u_w}{\partial_t} + m_w \frac{\partial (u_a + u_w)}{\partial_t} = \frac{\partial}{\partial_y}\left[\left(\frac{K_w}{\rho_w g}\right)\frac{\partial u_w}{\partial_y} + K_w \frac{\partial_y}{\partial_y}\right] \quad (1)$$

Where:

- K_w is the hydraulic conductivity;
- u_w is the pore-water pressure;
- θ_w is the volumetric water content;
- β is the soil compressibility;
- β_w is the water compressibility;
- m_w represents the slope of the water content function.

All materials were considered saturated, except for the silty-clay embankment and filter. For these materials, unsaturated soil parameters were defined.

To describe the retention curve for these materials, the Van Genuchten model [6] was adopted, as presented by Siacara et al. [7] and Castro et al. [8], and is defined by equation (2) below. Through these parameters, the hydraulic conductivity function (K) for unsaturated soils is also determined, as shown in equation (3):

$$\theta = \theta_r + \frac{\theta_s - \theta_r}{[1 + (a\prime\varphi)^{n_v}]^m} \quad (2)$$

$$K_w(\varphi) = K_{sat}\frac{\left\{1 - (a'\varphi)^{n-1}\left[1 + (a'\varphi)^n\right]^{-m}\right\}^2}{[1 + (a'\varphi)^n]^{\frac{m}{2}}} \quad (3)$$

Where:

- θ is the volumetric water content;
- θ_r is the residual water content;
- θ_s is the saturated water content;
- φ representes suction;
- n_v, m, and α' are empirical parameters used to fit the retention curve.

The stability analysis will be conducted using SLOPE/W, which is also part of the GeoStudio suite. The factor of safety will be calculated for each iteration of the sensitivity analysis and the probability of failure will be calculated for each step of the FORM analysis.

The general shear strength criteria used was proposed by Mohr-Coulomb and is ruled by linear equation 4 and has two independent stress state variables [9].

$$\tau = c' + (\sigma - u_a)tan\phi' + (u_a - u_w)tan\phi'' \quad (4)$$

Where c' is the effective cohesion, ϕ' is the friction angle, $(\sigma - u_a)$ is the normal stress on the plane of failure at failure, σ is the total normal stress, u_a is the air pore-air pressure, u_w is the pore-water pressure, $(u_a - u_w)$ is the matric suction and ϕ'' is the angle of the contribution to the shearing resistance related to the matric suction.

For the saturated soils, the same criteria are used but factor of the matric suction equals to zero.

2.2. Sensitivity analysis

The Kucherenko sensitivity analysis method is a technique used to understand how uncertainties in input parameters influence the output results. It defines sensitivity indices using the direct variance decomposition of the output function $F(X)$. In this study, the output function corresponds to the calculation of the factor of safety [10].

The variance Y of the equation $F(X)$ can be decomposed using equation 5 below:

$$Var(Y) = \mathrm{Var}\left[\mathbb{E}[Y|X_v]\right] + \mathbb{E}\left[Var[Y|X_v]\right] \quad (5)$$

Where:

- $\mathrm{Var}\left[\mathbb{E}[Y|X_v]\right]$ is the contribution of variable v in isolation;
- $\mathbb{E}\left[Var[Y|X_v]\right]$ is the contribution of interactions among input variables;
- $Var(Y)$ is the total variance of the model outputs.

From the variance decomposition of $F(X)$, sensitivity indices are calculated and divided into two groups: the first-order sensitivity coefficient and the total sensitivity coefficient [11].

The first-order sensitivity index (equation 6) measures the isolated effect of a variable, without considering interactions among parameters. On the other hand, the total sensitivity index (equation 7) measures the overall effect of an input variable, including its interactions with other variables.

$$S_v = \frac{\mathrm{Var}\left[\mathbb{E}[Y|X_v=x_v]\right]}{Var(Y)} \quad (6)$$

$$S_w^T = \frac{\mathbb{E}\left[Var[Y|X_v=x_v]\right]}{Var(Y)} \quad (7)$$

2.3. Reliability analysis

To quantify the failure probability of the model, a probabilistic analysis was conducted using the FORM method.

FORM (First Order Reliability Method) is a technique employed to efficiently estimate the failure probability of a system. The method simplifies the problem by approximating the limit state function linearly and transforming the variables into a standardized probabilistic space.

In slope stability problems, the performance function can be written as:

$$g(X) = g(X_1, X_2, \dots, X_n) = FS(X_1, X_2, \dots, X_n) - 1 \quad (8)$$

Where FS is the factor of safety of the stability of the model. The Morgenstern-Price stability analysis method is employed herein to evaluate *FS*. The critical surface is calculated for each iteration of X_i in the search of the design point [12].

This approach aims to determine the system's failure probability, expressed in equation 9 as:

$$P_f = P(g(X) \leq 0) \quad (9)$$

The method involves the following steps [13]:

- Definition of the limit state: The limit state is represented by the function $g(X)$, where $\boldsymbol{X}$ is the vector of random variables associated with the problem. In this study, these variables correspond to the material resistance parameters.
- Transformation to a standardized space: The random variables undergo an isoprobabilistic transformation from the physical space to the standard normal space U, facilitating the search for the design point.
- Identification of the design point: The design point is interpreted as the most probable point of failure and is the closest point to the origin in the standard normal space. This is typically achieved using iterative numerical techniques, such as gradient-based search methods. The result is the Hasofer-Lind reliability index, which is the distance from the origin to the design point [14]. Consequently, the failure probability equation is reformulated in equation 10 as:

$$P_f = \Phi(-\beta_{HL}) \quad (10)$$

Where Φ denotes the cumulative density function of the standard normal distribution, and β_{HL} represents the Hasofer-Lind reliability index.

In addition to the probability of failure, the FORM method provides a byproduct called sensitivity factors α_i^2, calculated using equation 11 below [15]:

$$\alpha = \{\alpha_i\}_{i=1,\dots,n}^{t} = \frac{\nabla g_y(y_i^*)}{||\nabla g_y(y_i^*)||} \quad (11)$$

Where ∇g_y is the gradient of the limit state function related to the random variables. Since α is a unit vector, the sensitivity factor α_i^2 indicates the relative contribution of each random variable X to the model's probability of failure.

3. METHODOLOGY

3.1. Model definition

The dam analyzed is a mining dam with the geometry presented in Fig. 1. It consists of a 40-meter-high embankment made of clayey silt material, an internal drainage system composed of a filter and a drainage blanket of the same material, a 15-meter-thick layer of residual soil, and a saprolite foundation. The reservoir is filled with tailings, and the water level at an elevation of 80 meters. The slope is 2H:1V above the berm and 1.73H:1V bellow.

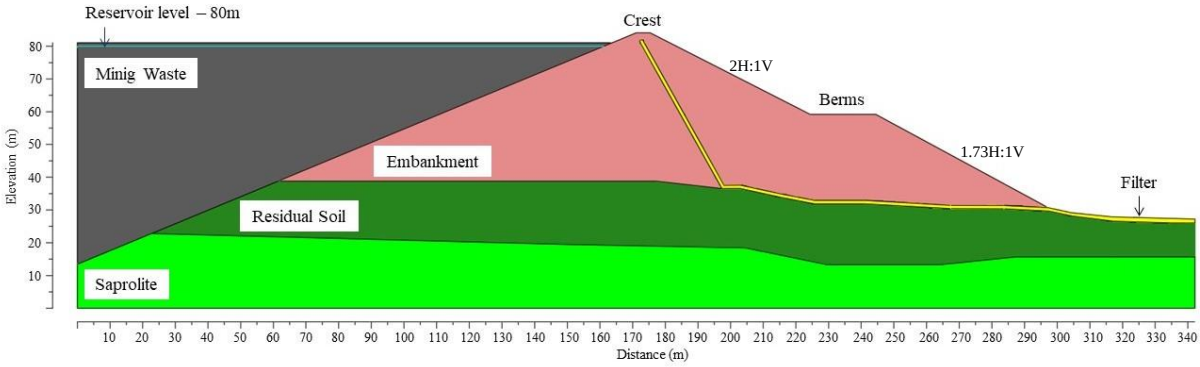


Fig. 1 – Dam geometry.

Table 1 presents the hydraulic parameters used in the model for the retention curve and hydraulic conductivity function. The Van Genuchten parameters were estimated based on the suggestions by Gonçalves [16].

Table 1 – Hydrological parameters

Material	$K_{isotropic}$ (m/s)	α (kPa^{-1})	n_v	θ_s	θ_r
Mining Waste	1x10^{-5}	-	-	-	-
Silty Clay Embankment	1x10^{-6}	0.300	1.481	0.578	0.269
Filter	1x10^{-3}	0.38	2.391	0.426	0.034
Foundation – Residual Soil	2x10^{-6}	-	-	-	-
Foundation – Saprolite	1x10^{-6}	-	-	-	-

All strength parameters were defined as average values obtained from the literature, based on data for similar soil types, and are presented in table 2.

Table 2 – Resistance parameters – Mean Values

Material Name	γ (kN/m³)	c' (kPa)	φ' (°)
Mining Waste	23	0	35
Silty Clay Embankment	23	17	30
Filter	17	0	35
Foundation – Residual Soil	18	15	30
Foundation – Saprolite	19	60	30

In this paper, nine variables were considered in the sensitivity and probabilistic analyses. These include the effective cohesion, friction angle, and unit weight of the silty-clay embankment, residual saprolitic soil, and saprolite.

Additionally, three scenarios were analyzed, in which the coefficient of variation of these input variables varied according to the values presented in Table 3 below.

Table 3 – Input parameters

Variables	COV_{low} (%)	COV_{medium} (%)	COV_{high} (%)
Effective cohesion (c')	10	30	50
Friction angle (φ)	10	20	30
Unit Weight (γ)	3	7	11

The coefficient of variation (COV) is a statistical metric that describes how much a parameter varies in relation to its average value. In probabilistic analyses, it helps define statistical distributions, which play a key role in estimating the likelihood of dam failure. When a parameter has a high COV, it means there is greater uncertainty in its value, which can significantly influence the overall stability of the dam. All variables were characterized using a lognormal distribution.

4. RESULTS

Three scenarios were carried out for the sensitivity analysis of the resistance parameters and for the reliability analysis. The sensitivity results will be presented in values of the percentage influence of each parameter on the model, and for the reliability analysis, the reliability indices and probability of failure for each scenario will be analyzed.

4.1. Sensitivity analysis using Kucherenko indices

Fig. 2 shows the results of the first-order Kucherenko indices. It can be seen how important the friction angles of the embankment and the residual soil are for the dam's safety factor, regardless of the other variables. The specific weight parameters of all the soils have minimal influence on the model, as do the cohesion and friction angle of the altered rock (saprolite).

It is worth noting the correlation between the change in the coefficient of variation and the variation in the indices found for the friction angles of the embankment and residual soil. For each scenario, as the coefficient of variation increases, there is an increase in the influence of the friction angle of the embankment and a reduction in the influence of the friction angle of the residual soil. These two parameters reach a similar influence value for the cases of medium and high coefficients of variation.

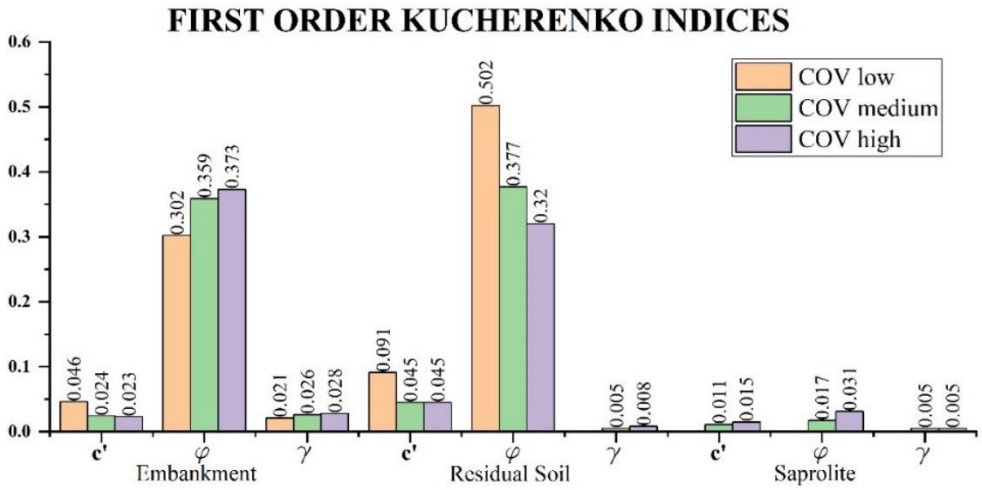


Fig. 2 – First Order Indices - Results

Fig. 3 shows the results of the total Kucherenko indices. As expected, the behavior of the variables is similar, but in this case the influence of the friction angles of the embankment and the foundation are closer. This slight adjustment can be explained by the fact that it is an index that considers the relationship between the variables.

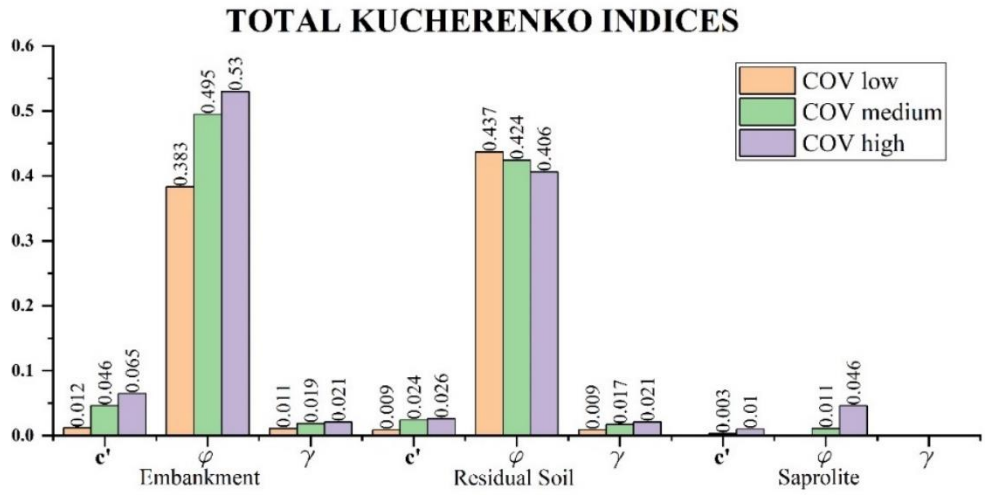


Fig. 3 – Total Indices - Results

4.2. Reliability analysis

Fig. 4 shows the results of the probability of failure and reliability index for the three scenarios.

It shows that as the coefficient of variation increases, the reliability index drops and the probability of failure increases, as expected. This effect can also be seen in Fig. 5 below. In the low coefficient of variation scenario, the convergence to the design point, i.e. to the point of greatest probability of failure, requires more iterations, while in the scenarios with higher coefficients of variation, this design point is reached with fewer iterations.

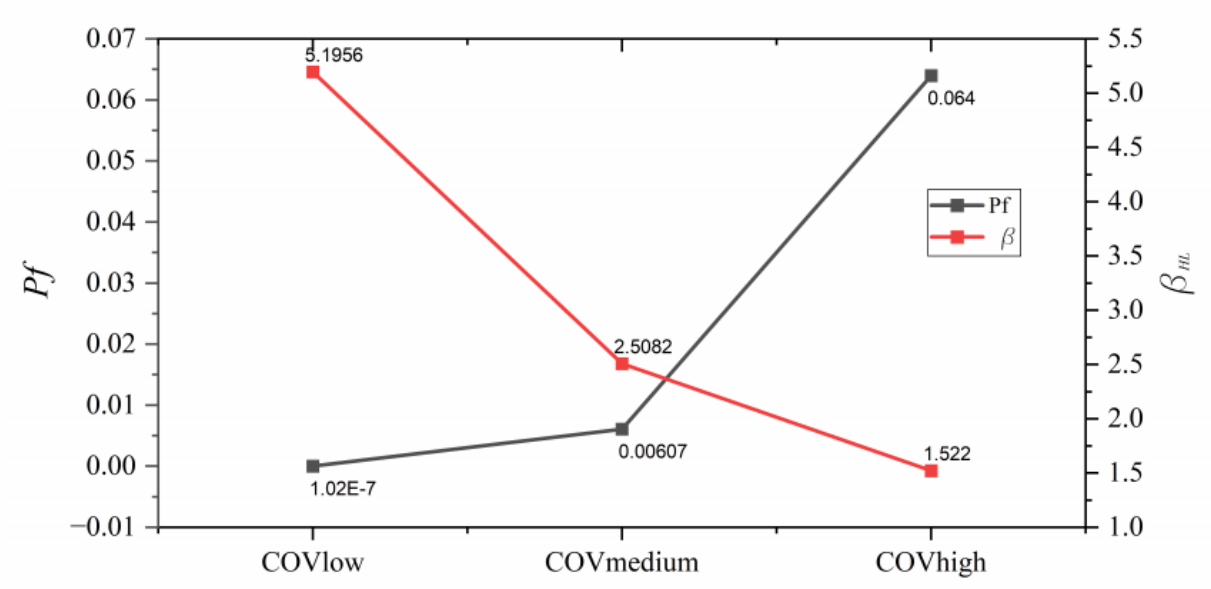


Fig. 4 – Probability of failure x Reliability index – Results

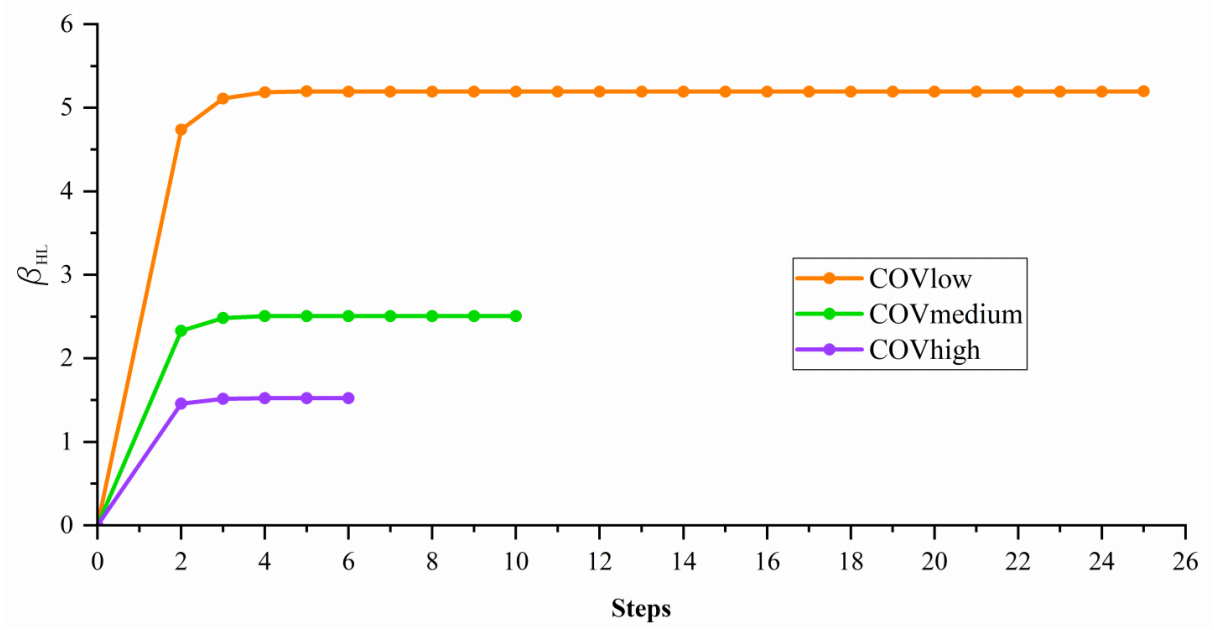


Fig. 5 – Convergence of the reliability index – Results

For comparison reasons, a deterministic analysis was performed for the model with the average values of the resistance parameters. Fig.6 shows the factor of safety found, and the failure surface encompasses the embankment and the residual soil.

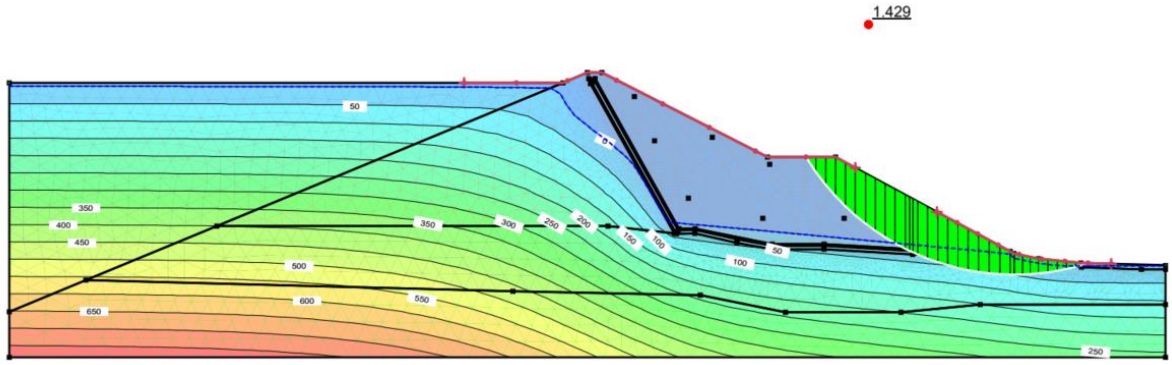


Fig. 6 – Deterministic results in terms of FS.

We can see the difference between the failure wedge of the deterministic analysis and the failure wedge of the design point in Fig. 6 and Fig. 7 respectively. The failure wedge presented by the design point, for the scenario of a medium variation coefficient, is a deeper wedge and consequently contributes more residual soil. Thus, knowing that FORM analysis is a convergence method up to the design point, we can justify why the friction angle of this lithotype has a greater influence on this sensitivity analysis. The reliability index for this scenario is 2.508.

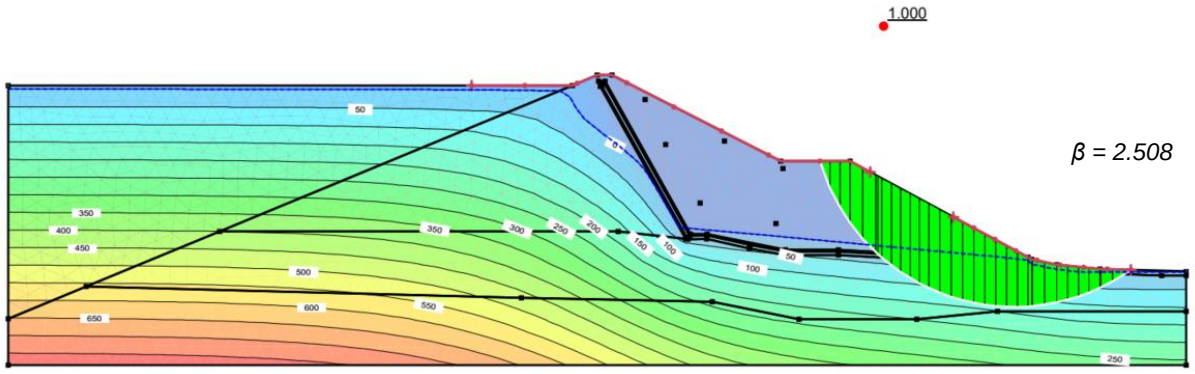


Fig. 7 – Probabilistic results in terms of Beta for the COVmedium

4.3. Sensitivity analysis using Sensitivity factors

Fig. 8 shows the sensitivity index values obtained by calculating the FORM method. In this method we can see that the friction angle of the residual soil is the most influential variable, with a value of approximately 0.8. It is worth noting that the FORM method differs from Kucherenko's sensitivity method in that Kucherenko's method performs random analysis iterations of safety factors, while the FORM method analyzes the model's probability of failure. In addition, the FORM method consists of converging until the design point is found, i.e. the analysis iterations are always converging towards failure wedges with safety factors close to 1.

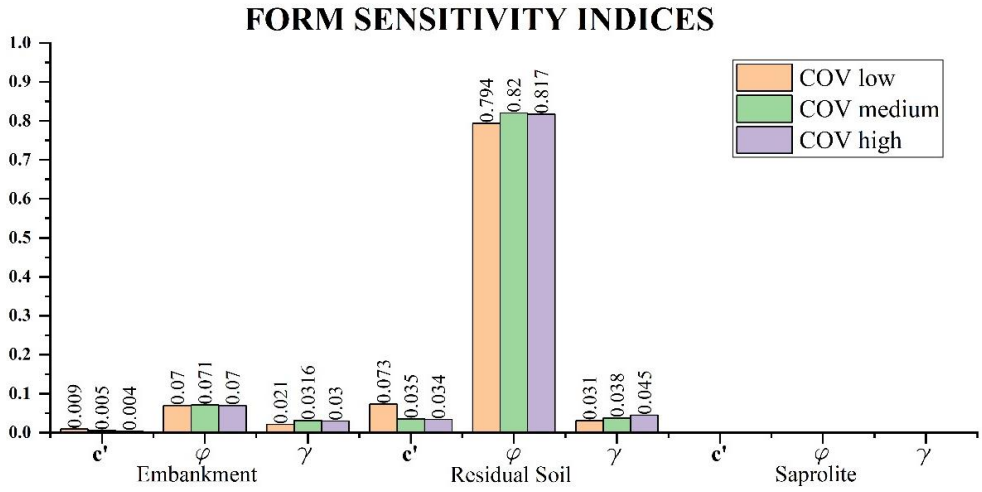


Fig. 8 – Sensitivity Factors – Results

4.4. Comparison in terms of sensitivity analysis

When analysing the results, we notice significant differences between the outcomes of the two methods. These differences can be explained by the distinct ways each method processes iterations and evaluates parameter influence. The FORM is based on the model's reliability and focuses on identifying the most likely point of failure. As a result, its sensitivity indices

measure local sensitivity, meaning they evaluate how small changes in the parameters affect the limit state around the most probable failure point. The main characteristics of FORM can be summarized as:

- Sensitivity indices show which variables have the most influence on the probability of failure at the most probable failure point;
- The indices are calculated only at the design point, so they do not reflect how the system behaves in other regions of the input space;
- Interactions between variables are not explicitly represented, focusing solely on localized effects.

On the other hand, the Kucherenko method uses a global sensitivity approach. It evaluates the influence of parameters across the entire input space, taking into account both their individual effects and combined interactions. This makes the Kucherenko method more general, as it identifies the most important parameters affecting the system's overall behaviour. However, this broader analysis requires more computational resources. The main characteristics of the Kucherenko method are:

- Sensitivity indices reflect the global influence of each variable on the system's overall behavior, considering the entire input space (unlike FORM);
- The method takes into account the variability of all parameters within their probability distributions, capturing both individual contributions and interactions among variables.

So, it is seen that these methods are complementary, FORM is ideal for localized reliability analysis while Kucherenko is better for understanding the global impact of the parameters.

5. CONCLUSIONS

Sensitivity analysis is important for quantifying the uncertainty of parameters and assessing which ones are most critical to the stability of a dam. In this study, the friction angle (φ') was the most important, both in the embankment and in the residual soil, using the Kucherenko method, and in the residual soil using the sensitivity analysis carried out by FORM. This study makes it possible not only to reduce computational costs, as it is possible to consider parameters such as the specific weight of all the lithotypes and the strength parameters of the saprolite as constant or with low variability. But it also allows efforts and resources to be concentrated on more detailed geotechnical studies and investigations of the most influential parameters.

It was also possible to observe through this study that as the coefficient of variability increases, the dispersion of the analysis results increases, thus increasing the possibility of critical states

occurring. As a result, there was an increase in the probability of failure, showing the importance of a good analysis of the parameters and considering the heterogeneity of soil behaviour.

REFERENCES

[1] Leão, R.; Rabelo, R. (2023). A Extensão da cadeia produtiva da economia mineral no PIB brasileiro, Ipea (2023), 88.

[2] Agência Nacional de Mineração – ANM (2024). Sistema Integrado de Gestão de Barragens de Mineração (SIGBM), ANM (2024).

[3] El-Ramly, H.; Morgenstern, N.R.; Cruden, D.M. (2002). Probabilistic slope stability analysis for practice, Canadian Geotechnical Journal (2002), v. 39, n.3: 665 – 683.

[4] ABNT NBR 13028:2017. Mining — Preparation and presentation of design of tailings, sediments and/or water dams — Requirements.

[5] Xu, B.; Wang, S. (2023). Sensitivity analysis of factors affecting gravity dam anti-sliding stability along a foundation surface using Sobol method. Water Science and Engineering (2023), 16(4): 399 – 407.

[6] Van Genuchten, M. (1980). A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils. Soil Science Society of America journal (1980).

[7] Siacara, A. T.; Napa-Garcia, G. F.; Beck, A.; Futai, M. (2022). Reliability analysis of an earth dam in operating conditions using direct coupling. SN Applied Sciences (2022).

[8] Castro, G.; Siacara, A.; Nardelli, A.; Lenzi, L.; Futai, M. (2022). Numerical Analysis of Infiltration in One-Dimensional Unsaturated Soil–Geotextile Column. Geotechnical and Geological Engineering (2022), 4121-4144.

[9] Fredlund, D.; Morgenstern, N.; Widger, R. (1978). The shear strength of unsaturated soils. Canadian Geotechnical Journal (1978), volume 15, number 3.

[10] Marelli, S.; Lamas, C.; Konakli, K.; Mylonas, C.; Wiederkehr, P.; Sudret, B. (2024). UQLab user manual – Sensitivity analysis (2024), Report UQLab-V2.1-106.

[11] Xu, B.; Wang, S.; Xia, H.; Zhu, Z.; Chen, X. (2023). A global sensitivity analysis method for safety influencing factors of RCC dams based on ISSA-ELM-Sobol. Structures (2023), 288-302.

[12] Siacara, A. T.; Napa-Garcia, G. F.; Beck, A.; Futai, M. (2024). Reliability Analysis of an Earth Dam Under Rainfall Effects. International Journal of Geosynthetics and Ground Engineering (2024), volume 10, article 59.

[13] Marelli, S.; Schöbi, R.; Sudret, B. (2024). UQLab user manual – Structural reliability (Rare event estimation), Report UQLab-V2.1-107.

[14] Guo, X.; Dias, D. (2020). Kriging based reliability and sensitivity analysis – Application to the stability of an earth dam, Computer and Geotechnics 120 (2020).

[15] Siacara, A. T.; Beck, A.; Futai, M. (2020). Reliability analysis of rapid drawdown of an earth dam using direct coupling. Computer and Geotechnics 118 (2020).

[16] Golçalves, F. (2012). Estudo das curvas de retenção e condutividade hidráulica estimadas pela equação de Van Genuchten (1980) para diferentes tipos de solo, master's dissertation, Federal University of Espírito Santo.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DAM RAISING USING A REINFORCED SOIL CONTAINMENT STRUCTURE

Gustavo R. Paulino[1] JF Brasil Engenharia
Ronaldo V. W. Neto[2] JF Brasil Engenharia
Ciro G. T. Franco[3] JF Brasil Engenharia
João E. T. Franco[4] JF Brasil Engenharia
André F. F. Dluhosch[5] Geo Soluções
Camyla M. M. O. Cadavez[6] Geo Soluções
Victor E. Pimentel[7] Geo Soluções

ABSTRACT

This paper presents information about a dam raising project to meet the requirements set forth in Resolutions 95 and 130 of the National Mining Agency (ANM), which establish guidelines for the safety and stability of mining dams in Brazil.

Among the requirements contained in these resolutions, some key points stand out, such as the detailed evaluation of the geotechnical conditions and potential risks of the structure, as well as the analysis related to the dam's freeboard. The importance of a Dam Safety Plan that prescribes actions for the proper management of the structure throughout all phases of its life cycle is also emphasized.

To comply with these guidelines - especially those related to the dam's freeboard - a reinforced soil containment structure was designed for the dam raising. The solution adopted involves a structure with an "L"-shaped metallic front wall, with each element being 50 centimeters in height, a mesh opening of 5 centimeters by 10 centimeters, special corrosion treatment, and soil reinforcement elements composed of polyester geogrids, which were best suited to the available fill material. The choice of this containment system was based on its ability to provide a robust and durable structure and on the high tensile strength of the geogrids used in the project, which significantly contributed to stability and minimized deformations over time. Aspects related to the agility and ease of execution of the structures were also considered in selecting the containment system.

Throughout the project, safety factor analyses were conducted under various loading conditions, and the geotechnical properties of the soils present in the foundation and the dam body were studied. Potential overloads were considered, accounting for both static and seismic loads, to ensure the structural integrity of the structure.

In this paper, key points for the project development will be highlighted, such as the variables considered, the construction and design aspects, as well as the stability analyses of the containment structure and the dam in an overall context.

Keywords: dam, raising, reinforced soil containment, heightening, resolutions, guidelines.

[1]gustavo.paulino@jfbr.com.br, Brazil
[2]ronaldo.vilela@jfbr.com.br, Brazil
[3]ciro.franco@jfbr.com.br, Brazil
[4]joao.franco@jfbr.com.br, Brazil
[5]andre.fanaya@geosolucoes.com, Brazil
[6]camyla.oliveira@geosolucoes.com, Brazil
[7]victor.pimentel@geosolucoes.com, Brazil

1. INTRODUCTION

This article discusses the development of a technical feasibility study for the application of a reinforced soil containment structure for raising the crest of a tailings dam, focusing on the analyses carried out to verify the internal, external and global stability of the structure.

Reinforced soil retaining structures have stood out for being efficient solutions, combining agility in execution and reduced costs, as well as allowing the use of different types of front wall, which makes them versatile solutions for various applications such as structures for road, rail, residential and mining works. In addition, the use of geosynthetic reinforcements, such as geogrids, significantly reduces costs, making the solution more economically viable.

The study in question was motivated by an ANM (National Mining Agency) resolution, which consolidates normative acts for the safety of mining dams in Brazil. In this body of regulations, the requirement for a minimum freeboard height of 1.00 m stands out. In this case, in order to meet this requirement within the pre-established deadlines, it became necessary to look for alternatives for the feasibility of raising the dam. In this context, the reinforced soil containment structure proved to be the most promising alternative.

The feasibility of the structure was based on an analysis of other solutions commonly used in mining. In addition to making the structure quicker to build, the reinforced soil containment allows the crest to be raised over a small width, while remaining stable thanks to the reinforcements used. It ensures that the reservoir is watertight under transient conditions of extreme flooding. And because of the checks and the way in which the structures are designed and executed, they reduce the risk of tipping, floating and sliding.

Detailed stability analyses were carried out to guarantee the effectiveness and efficiency of the proposed solution, in which the boundary conditions to which the structure will be subjected were observed. These analyses included the evaluation of geotechnical parameters based on field and laboratory tests, the required height of the containment structure, reduction factors to guarantee the integrity of the structure throughout its useful life, among others. In addition, brazilian regulatory aspects and specific requirements were taken into account to ensure compliance with the country's technical standards.

2. REINFORCED SOIL RETAINING STRUCTURE

Reinforced soil retaining structures are systems commonly designed for slope stabilization, using reinforcements that improve the soils resistance parameters. The reinforcement used in these structures gives the soil the capacity to resist tensile forces, resulting in an increase in its shear strength. This technique basically consists of three elements: the soil, the reinforcement used and the front wall.

The interaction between these three components of the solution is important to ensure the proper performance and safety of the retaining structure. In this case, tensions are transferred from the soil to the reinforcement structures (geogrids) which, in turn, while promoting greater resistance of the soil to shear forces, result in greater resistance of the structure. The front wall, on the other hand, provides the system with additional stability, preventing the soil from sliding and guaranteeing the integrity of the reinforced soil system.

Currently, there are several types of reinforced soil retaining structures, which vary according to the type of front wall, the reinforcement used and the backfill material used in the solution. Given the wide range of alternatives available, for this project a solution was selected consisting of a wire mesh front wall, where the wall is filled with gravel and the structure is reinforced with geosynthetics using geogrids.

2.1. Metal Screen Front Wall

The front wall adopted in this project consists of an electrowelded metal mesh, folded into an "L" shape, made up of CA-50 and CA-60 steel bars with a diameter of 6.3 mm and openings of 0.10 m x 0.06 m on its vertical face and 0.10 m x 0.10 m on its horizontal face. In addition, the front wall is also made up of equidistant metal bars (strutus) to give the system greater rigidity. The Fig. 1 shows the facing element adopted as a solution for the retaining structure. As for additional protection, the elements are hotdip galvanized and have corrosion protection in accordance with Brazilian standard ABNT NBR 6323 [1].

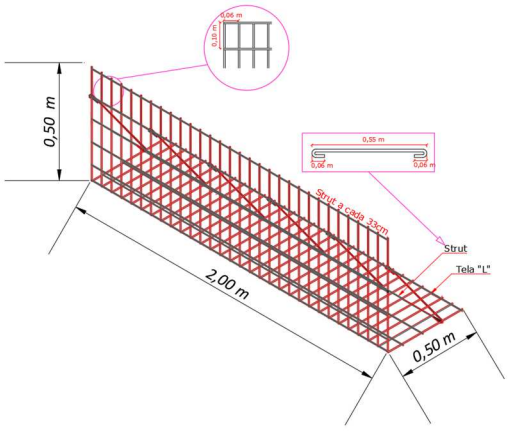


Fig. 1 - Facing element for the retaining structure

The choice of this walling was based on technical criteria, particularly its agility in assembling the containment structure. The mesh is supplied ready for installation, which simplifies the process, requiring only the insertion of the struts to maintain the proper inclination of the face. In addition, the solution allows for mechanized filling and compaction close to the face, eliminating the manual positioning and arrangement of the filling material. This method not only speeds up execution, but also guarantees a uniform, high-quality finish, making the system both efficient and practical in construction terms.

2.2. Geosynthetic reinforcement with Geogrids

According to the Brazilian standard ABNT NBR ISO 10318-1 [2], geosynthetics perform six functions: Separation, Filtration, Drainage, Reinforcement, Protection and Waterproofing. To perform the reinforcement function, woven geotextiles and geogrids can be used, both materials being characterized by properties that guarantee the soil tensile strength.

The main difference between these materials for the reinforcement function is the interaction between the reinforcement and the soil that makes up the embankment. In the case of woven geotextiles, their mobilization of forces, as indicated by Ehrlich and Becker (2009) [3], occurs in only one direction. The Fig. 2 illustrates the interaction between the soil and the woven geotextile.

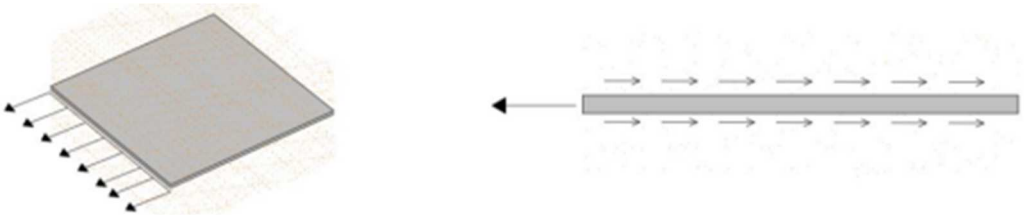


Fig. 2 - Soil and woven geotextile interaction

In the case of structures using geogrids for soil reinforcement, as mentioned by Ehlich and Becker (2009) [3], the interaction between geogrid and soil occurs differently due to the regular openings present in the material. These openings allow the soil to penetrate the voids in the geogrid, resulting in a resistance generated by friction along the longitudinal and transverse members of the geogrid (Fig. 2). This iteration mechanism contributes to greater reinforcement efficiency by better distributing the stresses in the soil and geogrid.

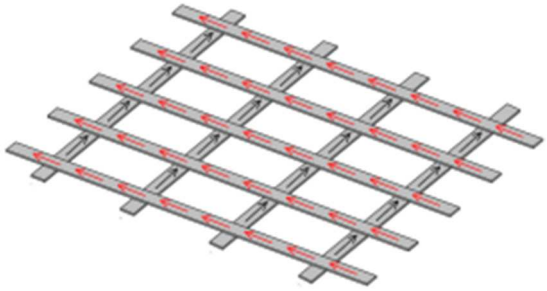


Fig. 3 - Mobilization of Efforts in the Geogrid

In this way, as well as providing greater tensile strength, the embedding of the backfill material in the geogrid provides the containment system with greater resistance to the pulling out of the reinforcement, contributing significantly to the stabilization of the structure.

Taking these factors into account, high-performance reinforcements made up of high-tenacity polyester (PET) yarns were considered for the solution in question. The yarns that make up the geogrids adopted have a polymer coating, thus providing the material with protection against UV radiation and preventing chemical and biological degradation.

3. DESIGN ASSUMPTIONS

3.1. Overloading and Seismic Action

For the stability analyses, the overloads acting on the structure to be designed were first determined. In the case of vehicular traffic, a uniformly distributed overload of 20 kN/m² was considered.

To assess the magnitude of the earthquake in the pseudo-static analyses, loads with a horizontal acceleration of 0.09 g and 0.05 g were considered, based on a local seismic study, following the recommendations of the Brazilian standard ABNT NBR 13.028 [4] and the Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams (Canadian Dam Association, 2019) [5]. The analyses are considering the Maximum Credible Earthquake according to the criteria established by the latter reference.

3.2. Soil Characterization

The soil parameters were determined based on available field and laboratory tests, allowing for the characterization of the geotechnical parameters and the geological model associated with the dam under study. The parameters were determined considering drained and undrained loading conditions, as shown in Table 1. For undrained conditions, the residual and

colluvium soils are analyzed based on the vertical stress ratio. The materials in the calculation section can be seen in Fig. 4.

Table 1 - Soil data undrained condition

Material	Dry Specific Weight	Saturated Specific Weight	Cohesion	Friction Angle	Strength Type	Vertical Stress Ratio
Landfill 1	20	21	10	28	Mohr-Coulomb	-
Landfill 2	19	20	0	25	Mohr-Coulomb	-
Sand	18	19	0	30	Mohr-Coulomb	-
Gravel	21	-	0	34	Mohr-Coulomb	-
Residual Soil	18	19	10	32	Mohr-Coulomb/Vertical Stress Ratio	0,2
Colluvium	17	18	13	24	Vertical Stress Ratio	0,29
Sediment	16	17	0	10	Mohr-Coulomb	-
Saprolite	20	21	10	35	Mohr-Coulomb	-

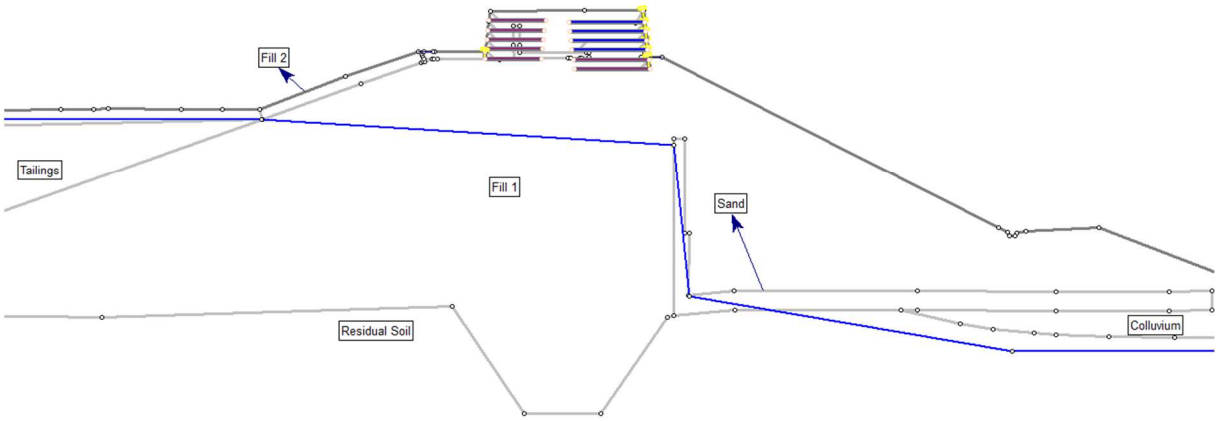


Fig. 4 - Materials in the calculation section

3.3. Stability Analyses Performed and Calculation Methodology

In order to verify the safety of the retaining structure, external, internal and global stability analyses were carried out, following the recommendations of the Brazilian standard ABNT NBR 16.920-1 - Walls and Slopes in Reinforced Soils Part 1: Reinforced Soils in Fill [6].

The external analyses represent the assessments that must be made to verify the safety of the structure in relation to overturning, sliding of the base and verification of the bearing capacity of the foundation. The internal analyses relate to the reinforcement used in the containment structure, dividing the structure into an active and a resistant zone. These analyses check the

pull-out resistance of the reinforcements and their ability to withstand stresses and prevent internal ruptures.

For the mentioned calculations, the GeoQore plugin was used, developed to assist in the correct design of reinforced soil retaining structures, was used to carry out the calculations for these analyses. The software uses the Coulomb and Rankine methods to determine the thrusts acting on the structure, as well as adopting the limit equilibrium method to calculate the safety factor of the necessary checks.

The global analysis of the structure was carried out in order to identify and evaluate potential rupture surfaces. This analysis considers both the rupture surfaces that intersect the reinforcements and those that do not, with the aim of identifying the critical rupture surface that could compromise the stability of the structure.

For the global stability analysis calculations, methodologies based on the Morh-Coulomb, Simplified Bishop, Morgenstern-Priece and Sarma calculation criteria were considered, covering the search for potential rupture surfaces, both circular and non-circular. The software Slide was used to determine these critical surfaces and evaluate the safety factors associated with them.

4. RESULTS AND DISCUSSIONS

With regard to the external and internal stability analyses, the structure has a constant geometry along the axis of the dam, there are no significant variations in its height or other geometric configuration. In this way, calculations were carried out considering a typical critical section, since the soil parameters and structural conditions of the containment remain the same throughout the length of the structure. The analysis was conducted using the GeoQore plugin, which allows for a more detailed assessment of the acting forces.

All the checks carried out, both external and internal, showed safety factor values that meet the requirements established by the Brazilian standard ABNT NBR 16.920-1. The analyses show that the designed structure has an adequate level of safety in relation to the instability mechanisms assessed, as can be seen in Fig. 5.

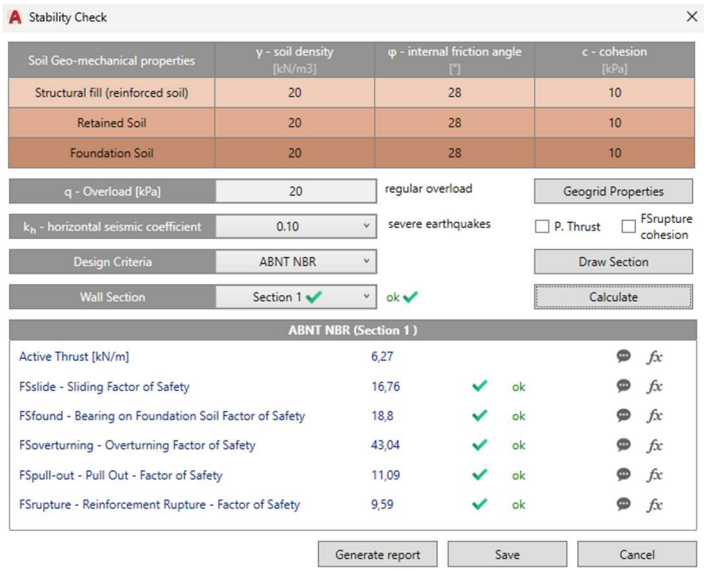


Fig. 5 - Results of the internal and external stability analyses

Once the external and internal analyses had been completed, calculations were made for the global analysis of the structure, taking into account the situations mentioned above. The aim of the global analysis was to verify the stability of the structure as a whole, including potential rupture wedges that could compromise the integrity of the structure. The geological profile of three sections was determined in order to carry out the calculations, as well as the geometry of the containment structure.

Bishop, Morgenstern-Priece and Sarma methods were used to verify potential global instability, enabling a detailed assessment of the conditions of the sections studied.

As a result, the structure was found to meet the global stability criteria established by the Brazilian standard ABNT NBR 16.920-1, as show in Fig. 6. Together with the internal and external analyses, it can be seen that the solution adopted has safety factors that are in line with the established values, ensuring the integrity of the solution adopted.

For these results, geogrids with characteristic strengths of 50 and 200 kN/m were used. These values were duly adjusted to calculation values, which take into account reduction factors in relation to material creep, possible installation damage that may occur and also chemical or environmental damage.

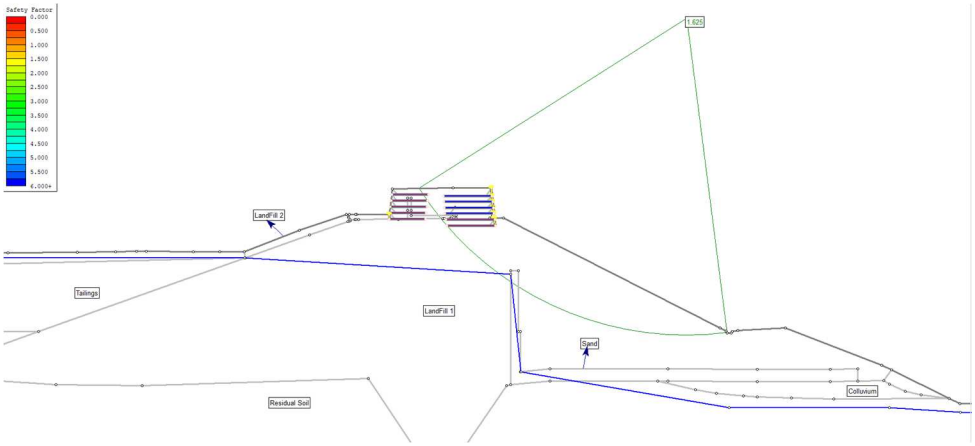


Fig. 6 - Result of the global stability analysis

The Fig. 7 shows in detail the cross-section of the reinforced soil retaining structure solution after the stability analyses.

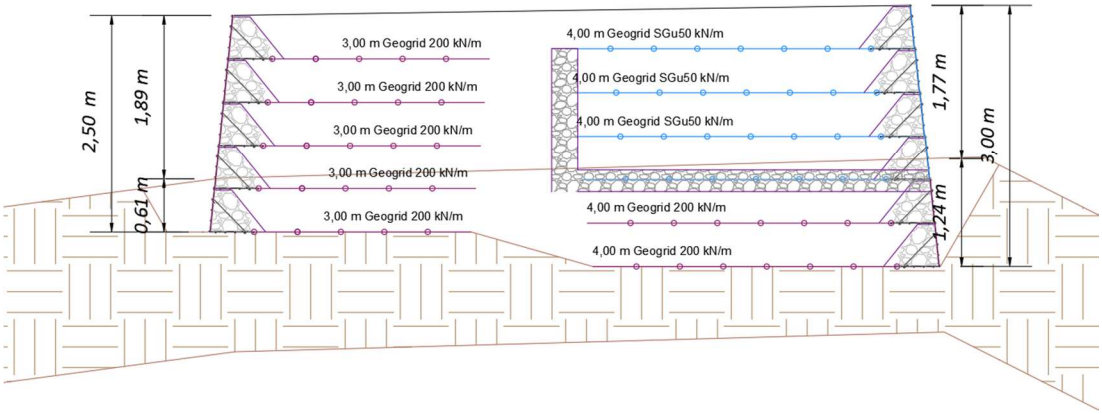


Fig. 7 - Cross-section of the Reinforced Soil Retaining Structure

5. CONCLUSIONS

Based on the information gathered, the feasibility of the solution adopted for raising the dam crest was observed. In addition to meeting all the regulatory requirements, resulting in high safety factor values compatible with the project requirements, the reinforced soil retaining structure stands out as an efficient alternative for the application in question.

Reinforced soil retaining walls have advantages over other traditional solutions. Among these advantages, it is possible to highlight the rapid execution of the work together with the lower cost of implementing the system, the elevation of the crest with a small width, the watertightness for transient flood situations and the reduced risk associated with tipping, floating and sliding. For the implementation of the containment system, when compared to more widespread containment solutions, using geogrids as reinforcement elements, in addition to providing tensile strength and stability to the system as a whole, allows the use of local soils as backfill material, which minimizes the need to transport materials. Thus, in addition to optimizing the schedule and assembly of the work, there is a reduction in costs associated with moving earth and disposing of materials.

In addition, the solution used is also sustainable, since the materials that make up the structure can be reused. A specific document has been created up containing guidelines for reusing the material in new interventions that may be necessary. The cyclical thinking behind the design of the structure and its reuse not only minimizes environmental impacts, but also reinforces the commitment to sustainability by promoting the circular economy. In this way, the proposed solution meets not only technical and economic requirements, but also current demands for sustainable practices.

Therefore, the solution adopted, which combines the use of a metal mesh front parameter with geogrid reinforcements, was the most suitable for the situation in question.

REFERENCES

[1] ABNT NBR 6323. Hot dip galvanized on steel and cast iron product - Specification;

[2] ABNT NBR ISO 10.318-1. Geosynthetics Part 1: Terms and definitions;

[3] Ehrlich and Becker (2009). Reinforced Soil Walls and Slopes: Projects and Execution. São Paulo: Oficina de Textos, 2009;

[4] ABNT NBR 13.028 - Mining – Elaboration and presentation of design of tailings, sediments and water dams – Requirements;

[5] Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams. Canadian Dam Association, 2019;

[6] ABNT NBR 16.920-1. Walls and slopes in reinforced soils Part 1: Reinforced soils in-fills.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

GEOTECHNICS 4.0: USE OF BIG DATA AND ARTIFICIAL INTELLIGENCE IN GEOTECHNICAL INVESTIGATIONS AT DAMS IN MINING

Marieli Biondo[1]
Vale S.A.

Rafael Piton
Vale S.A.

Viviane Costa
Vale S.A.

ABSTRACT

Geotechnical data is often not stored in an adequate way that can allow it to be used in different projects, generating repetition of boreholes due to the lack of traceability and reliability of the information. After the tailings dam accidents in Brazil, the request for geotechnical investigations was greatly increased, reaching approximately 100 kilometers per year of mixed boreholes and tests such as SPT, CPTu and special tests such as Dilatometric and Pressiometric. In the last three years, more than 200 kilometers of drilling and more than 10 thousand tests on geotechnical mining structures have been carried out and stored, with greater emphasis on dams that have undergone As Is and Decharacterization projects. To ensure the necessary speed in the planning and execution of all geotechnical campaigns, an automated process of data organization was implemented with the generation of unique identifying numbers for each borehole, which are later deposited in the official geoscientific databases. All this data is also spatialized in maps, in the same automated process, updated daily with the status of drilling execution, generating management data that is also stored in database with the same identification number of each hole. With the organization of the entire volume of data generated and establishing governance processes to certify the quality of the information, it was possible to promote integration between databases and reuse the holes already executed in future projects. More than 6 kilometers of geotechnical drilling were reduced over six months, which allowed the company's critical projects to be accelerated for the coming years. A new stage of process automation is being implemented with the aim of integrating other existing databases with environmental and geological information supported by artificial intelligence that will consume all existing data, combine it and launch it directly into models so that engineering analyses can be carried out during the execution of campaigns, which will also increase the assertiveness of the characteristic values used in the projects.

Keywords: Geotechnical Data, Geotechnical Investigation, Artificial Intelligence, Decharacterization of dams, Governance

[1] marieli.biondo@vale.com, Brazil

1. INTRODUCTION

Mining dams play a fundamental role in waste management and environmental protection, being essential for large-scale mining operations. However, Brazil has faced serious security crises related to these structures, with the tragic dam failures in Mariana in 2015 and Brumadinho in 2019, which resulted in irreparable human losses and significant environmental damage. These events alerted society and authorities to the urgent need to review and strengthen safety regulations governing the construction and maintenance of dams.

In response to these tragedies, Brazil implemented a series of regulatory reforms, notably Law 12.334, enacted in 2010, which established a national policy for dam safety, followed by Law 14.066 in 2020. The latter introduced stricter requirements, such as the prohibition of upstream raising and the need for the decharacterization of dams that do not meet the new safety standards. These legislations aim to ensure the structural integrity of dams and the protection of surrounding communities and the environment.

Currently, Brazil has over 22,000 registered dams, with approximately 5,474 of them falling under the National Dam Safety Policy (PNSB). The growing demand for geotechnical investigations, including drilling and in situ tests, reflects the need for continuous monitoring and evaluation of the stability of these structures. Vale S.A., one of the country's leading mining companies, has adopted innovative practices to optimize its drilling campaigns, utilizing modern technologies and rigorous strategic planning for data collection and analysis.

This paper aims to explore current practices in geotechnical investigations of mining dams, focusing on the methodologies employed, the organization of drilling campaigns, and the governance of the resulting data. The challenges faced in executing these investigations, the limitations of traditional techniques such as SPT (Standard Penetration Test), and the incorporation of advanced methods like CPTu (Cone Penetration Test) and laboratory tests will be discussed. Additionally, the importance of integrating historical data and implementing quality assurance and quality control (QAQC) systems to ensure the reliability of the obtained information will be addressed.

Through the analysis of actions taken by Vale S.A., this work seeks to highlight the importance of effective planning and the use of innovative technologies to optimize the processes of data processes for collecting, storing, querying, analyzing, and interpreting geotechnical data. The ultimate goal is not only to meet regulatory requirements but also to promote the safety of dams, protect human life, and minimize the environmental impacts associated with mining.

2. GEOTECHNICAL INVESTIGATIONS IN MINING DAMS

2.1. Current scenario of mining dams in Brazil

The years 2015 and 2019 were marked by tragic events in Brazil related to the collapse of mining dams, which prompted the amendment and creation of more stringent dam safety regulations and laws. The first dam safety law, Law 12.334 (BRAZIL, 2010a), was enacted in September 2010 with the aim of establishing a national policy for dams based on risk criteria such as height, reservoir size, and structural conditions. Until that year, existing laws in the country primarily had an environmental regulation character, such as licensing for dam construction in Rio Grande do Sul under Law 2.434 (BRAZIL, 1954), or criteria for building fish ladders in hydropower plants under Laws 12.488 (BRAZIL, 1997a) and 9.798 (BRAZIL, 1997b).The first Brazilian State to develop its own dam safety law was Rio de Janeiro in 2009, Law 5.549 (BRAZIL, 2009), followed by Pará with Law 7.408 (BRAZIL, 2010b). After the 2019 accident involving the B1 dam, located in the municipality of Brumadinho in Minas Gerais, Law 12.334 was amended by Law 14.066 (BRAZIL, 2020), which prohibited the construction of tailings dams using the upstream raising method and mandated the decharacterization of these structures or any other unidentified methods.

Along with the criteria established by federal law, regulatory agencies such as ANM (National Mining Agency) and state bodies like FEAM (State Foundation for the Environment) revised their regulations and decrees, instituting the requirement for more frequent submissions of stability declarations (DCE) to prove the safety of the dams.

According to the latest Dam Safety Report (ANA, 2021), Brazil has 22,654 dams registered in the national system (SNISB), of which 5,474 fall under the National Dam Safety Policy (PNSB) established by Laws 12.334 (BRAZIL, 2010a) and 14.066 (BRAZIL, 2020). Additionally, 4,313 do not meet the criteria, totalling 9,787 dams (43%). The remaining structures (57%) lack sufficient classification information, totalling 12,867 dams. Currently, mining has 935 dams registered in the safety systems, with 472 that fall under the legislation (ANM, 2024). The emergency levels of mining dams are divided into three groups: Level 1 (identified anomaly), Level 2 (anomaly without established control), and Level 3 (immediate collapse). According to the Mining Dam Safety Report (ANM, 2024), there are still two structures at Level 3 in Brazil, both located in the state of Minas Gerais and undergoing decharacterization due to being built using the upstream raising method. Considering the changes in federal and state legislation from each agency, as well as the need for decharacterization of upstream-raised dams, mining companies have scaled up geotechnical projects since 2019, significantly increasing the demand for geotechnical investigations. In 2023, Vale S.A. executed 97 km of geotechnical drilling with a fleet of 105 probes. For 2024, 114 km is planned, with the same amount expected to be maintained in the coming years. A

significant portion of the footage is executed using mixed probes, conducting SPT (Standard Penetration Test) tests associated with the extraction of samples at depth for laboratory tests and in-situ infiltration tests to determine permeability coefficients. The SPT (Standard Penetration Test) is widely used in Brazil to determine resistance to penetration in soils. During the test, the number of blows required to penetrate a certain length of soil is counted, and this value is correlated with various physical and mechanical parameters of the material. These correlations are valuable as they allow for the assessment of soil properties at depth, facilitating the interpretation of laboratory results from collected samples. This methodology is essential for safety and effectiveness in geotechnical projects, especially in the contexts of mining and civil construction.

However, the SPT has limitations related to the depth of execution, where the correlations become less accurate. In addition to depth, soft and loose soil conditions hinder the use of these tests due to the difficulty in reading the blows and the penetration of the hammer.

Considering the critical nature of the studies required for dam projects in mining, other tests have been incorporated into the portfolio of geotechnical investigations, such as an increase in CPTu/SCPTu (Cone Penetration Test with piezometer; Seismic Cone Penetration Test), dilatometers, and piezometers. These complementary methods allow for a more accurate assessment of soil conditions, providing essential data for the safety and effectiveness of dam projects. In addition to in situ tests, laboratory tests have also been significantly increased, with disturbed and undisturbed surface samples for special testing. Another advancement was the inclusion of depth samplers for collecting materials from embankments and foundations, such as Shelby, Denison, Dames and More, and Direct Push. All the methods mentioned earlier involve more than 15 geotechnical companies, design offices, consultants, laboratories, and technical monitoring of construction. To ensure that all deadlines are met, a specific organization will be incorporated, which will be mentioned in Item 2.2.

2.2. Planning of Geotechnical Drillings

Knowing that the annual forecast for conducting geotechnical drillings at Vale S.A. is around 100 km (significantly higher than what is practiced in other areas of structural and geotechnical projects), demand planning is one of the most important phases for the company. Actually, there are more than 15 geotechnical drilling companies located in the states of Minas Gerais, Pará, and Maranhão serving Vale S.A., with a total of 112 equipment to support over 13 internal clients, including: Dam Decharacterization, Geotechnical Projects, Operational Geotechnics, Capital Projects, and Projects in FEL 1 to 3 for Iron Ore. For the dam structures and their reservoirs alone, 32 km of geotechnical drillings is scheduled, with just over 27 km completed so far using a fleet of 43 probes. Differently from geological drillings, the objective of geotechnical drillings is to determine parameters in the initial phases of projects, which

concentrates the conclusion dates of all campaigns for internal clients. To maintain the agreed-upon timelines, the probes and companies have been regionalized within mining complexes to increase the speed of mobilization and demobilization between demands. To organize and implement the planning, all internal clients map their demands in the previous year and subsequently submit requests through an internal application, attaching the necessary engineering documents for the execution of the drillings, such as Technical Specifications. Figure 1 presents the internal demand application for geotechnics, along with the analysis gates and approval workflow.

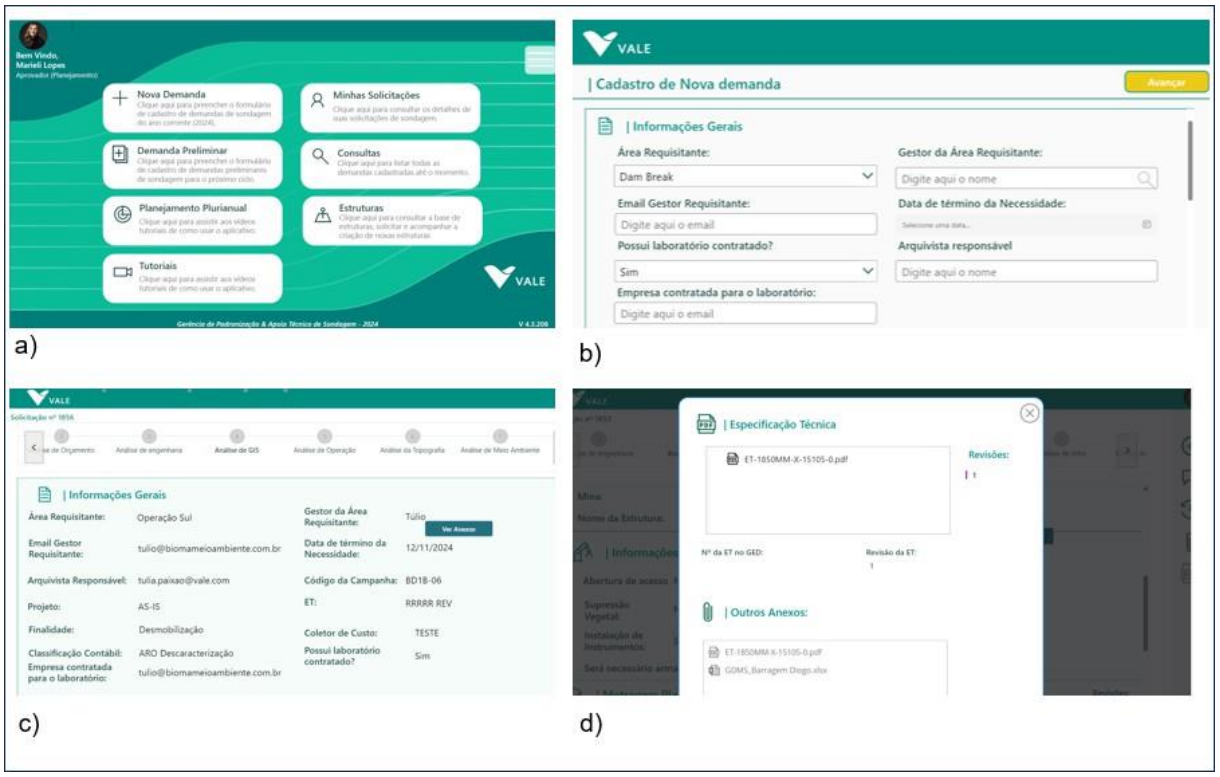


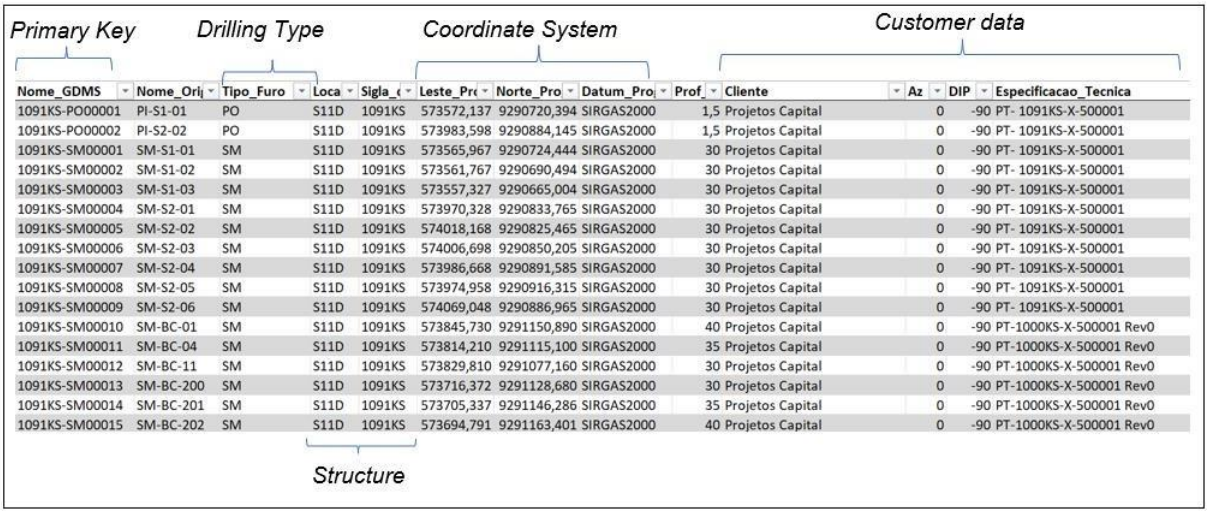
Fig. 1: Geotechnical drillings demand application: a) Opening page; b) Demand registration; c) Approval workflow; d) Attached documents.

Upon the submission of the demand by the internal client, the geotechnical drilling team evaluates the registration divided into ten gates: Preliminary Analysis, Budgeting, Engineering, Maps and GIS, Drilling Operations, Topography, Environment, Infrastructure, Strategic Planning, and Database. In the Engineering gate, the minimum requirements of the document for the execution of the drilling are evaluated, such as: type of drilling and in situ tests, type of collection of surface and depth samples, type of hole sealing, and duration of water level stabilization in the holes, as well as instrumentation when requested. Another important analysis is the historical data from the databases, which will be discussed in Item 2.3. After passing through all the other analysis gates, the last one is the database team's, which programs the activity by generating the primary key for each hole in the campaign. The primary key is a unique and standardized nomenclature automatically generated in the GDMS IG

(Fusion) system, which will be detailed below. It uses an algorithm that searches the entire company database for the last hole of the specified geotechnical structure and type of drilling and allocates the next number. The standard nomenclature of the primary key is represented by the code of the geotechnical structure, type of drilling, and a 5-digit sequential number:

<geotechnical structure code><hyphen><drilling type code><5-digit sequential number>

If it is a twin hole, the nomenclature follows the ID of the original hole with an added letter. It is at the Database gate that the entire traceability flow of the hole is initiated, as the creation of the primary key gives the hole a name that will never be repeated and that has its own attributes. The primary key of the holes is composed of the geotechnical structure code generated in the GEOTEC database, an internal system of Vale S.A. that allows for the registration of visual inspections, monitoring of geotechnical structures, recording of non-conformities, and tracking of related action plans. The registration of the geotechnical structure is determined by the creation of a code that also does not repeat among mining complexes. This ensures a unique ID for the drilling hole, considering various attributes and related databases, as presented in Figure 2.



Nome_GDMS	Nome_Ori	Tipo_Furo	Loca	Sigla_	Leste_Pr	Norte_Pro	Datum_Pro	Prof_	Cliente	Az	DIP	Especificacao_Tecnica
1091KS-PO00001	PI-S1-01	PO	S11D	1091KS	573572,137	9290720,394	SIRGAS2000	1,5	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-PO00002	PI-S2-02	PO	S11D	1091KS	573983,598	9290884,145	SIRGAS2000	1,5	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00001	SM-S1-01	SM	S11D	1091KS	573565,967	9290724,444	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00002	SM-S1-02	SM	S11D	1091KS	573561,767	9290690,494	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00003	SM-S1-03	SM	S11D	1091KS	573557,327	9290665,004	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00004	SM-S2-01	SM	S11D	1091KS	573970,328	9290833,765	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00005	SM-S2-02	SM	S11D	1091KS	574018,168	9290825,465	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00006	SM-S2-03	SM	S11D	1091KS	574006,698	9290850,205	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00007	SM-S2-04	SM	S11D	1091KS	573986,668	9290891,585	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00008	SM-S2-05	SM	S11D	1091KS	573974,958	9290916,315	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00009	SM-S2-06	SM	S11D	1091KS	574069,048	9290886,965	SIRGAS2000	30	Projetos Capital	0	-90	PT- 1091KS-X-500001
1091KS-SM00010	SM-BC-01	SM	S11D	1091KS	573845,730	9291150,890	SIRGAS2000	40	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0
1091KS-SM00011	SM-BC-04	SM	S11D	1091KS	573814,210	9291115,100	SIRGAS2000	35	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0
1091KS-SM00012	SM-BC-11	SM	S11D	1091KS	573829,810	9291077,160	SIRGAS2000	30	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0
1091KS-SM00013	SM-BC-200	SM	S11D	1091KS	573716,372	9291128,680	SIRGAS2000	30	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0
1091KS-SM00014	SM-BC-201	SM	S11D	1091KS	573705,337	9291146,286	SIRGAS2000	35	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0
1091KS-SM00015	SM-BC-202	SM	S11D	1091KS	573694,791	9291163,401	SIRGAS2000	40	Projetos Capital	0	-90	PT-1000KS-X-500001 Rev0

Fig. 2: Hole Scheduling and Primary Key Creation.

The official database used by Vale S.A. is the GDMS Fusion (Geological Database Management System) from Datamine®, which is a geological data management system designed to integrate all geological, geochemical, geotechnical, geophysical, and other data (DATAMINE, 2024). The GDMS Fusion is a geological solution used to compose the database for geological models and mineral resources. This solution has also been implemented for geotechnics as a means of standardization, organization, structuring, and centralization of data. Vale S.A. has three GDMS database solutions: “GDMS Ferrosos” for drill holes and

samples from trenches and channel samples intended for geology; “GDMS Investigações Geotécnicas” used solely for drill holes in geotechnical projects and the geomechanical description of drill holes executed in open pits; and “GDMS Sondagem” for storing management and equipment production data. Every GDMS databases maintain a governance flow, with different regulatory areas, but each one programming the drill holes through primary keys and information generated throughout the process for capturing the final technical data in the same structured database. Figure 3 illustrates one of the ways to edit and query geotechnical drilling data in the GDMS I.G. Once the Database gate is approved in the geotechnical demand application, the probs are mobilized to the designated sites, and the campaigns are initiated. The drilling companies, monitored by Vale S.A. Operations, send the technical data templates to the Database team for import into the GDM
S on a biweekly basis, with the same process occurring for data issued by laboratories.

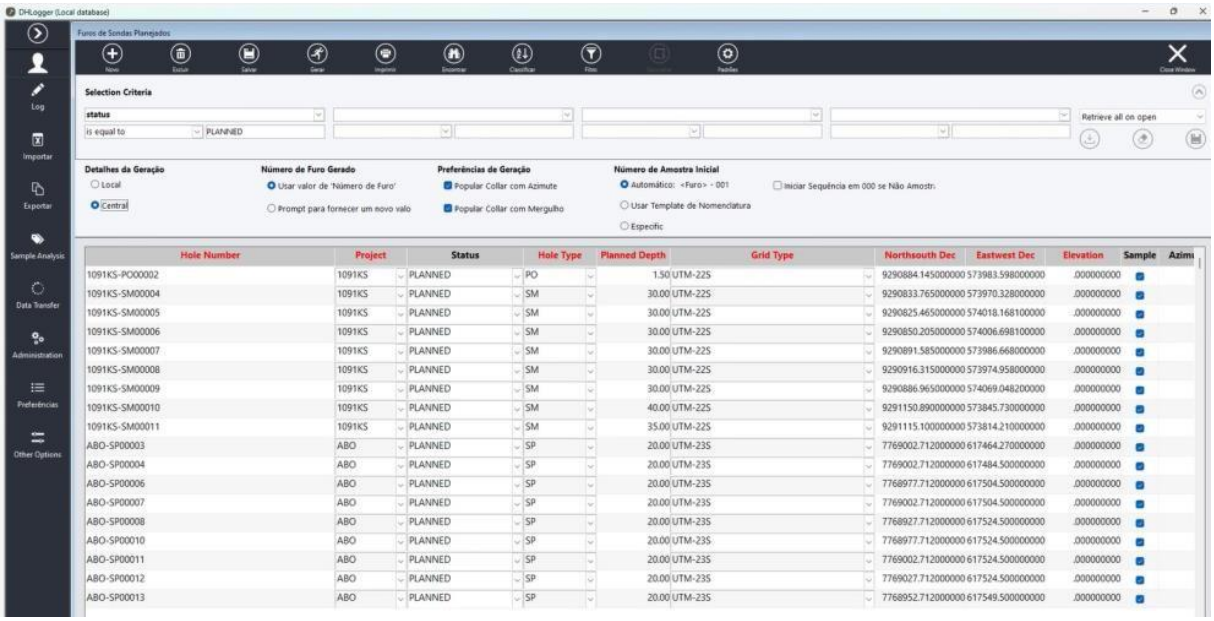


Fig. 3: Drill Hole Scheduling in GDMS I.G.

In addition to the import of all data into the GDMS Sondagem and GDMS I.G. databases, final campaign reports are provided to internal clients, containing the geological-geotechnical assessment of the area along with the drilling and testing logs for the continuation of engineering projects. It is important to highlight that all programmed and executed drill holes are spatially represented in the Mineral GIS, an ArcGIS ESRI solution, for the preparation of maps and operational monitoring (Figure 4):

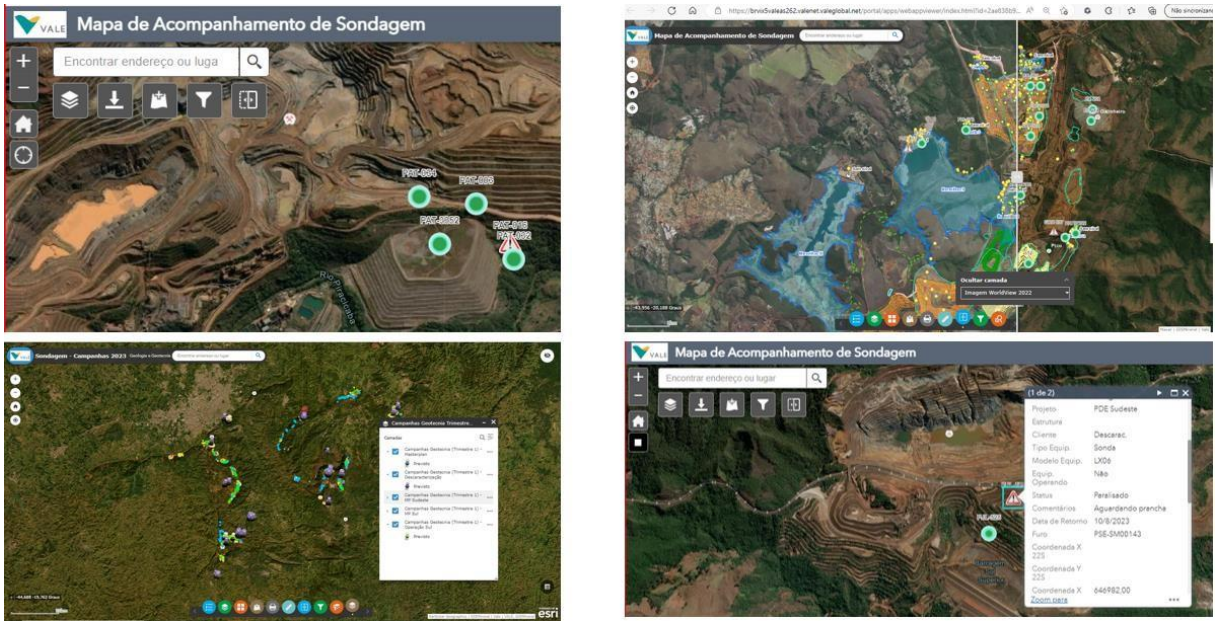


Fig. 4: Spatial Representation of Programmed and Executed Drill Holes in the Mineral GIS.

2.3. Assessment of Historical Geotechnical Data

The Engineering gate has included the assessment of historical geotechnical data from Vale S.A.'s official databases, as well as data presented in reports or other formats, with the aim of optimizing campaigns. So far, over 6 km of geotechnical drilling has been reduced through the analysis of typology, methodology, and coordinates, establishing the reuse of information. In parallel with the optimization analyses, Vale S.A. is implementing the QAQC (Quality Assurance and Quality Control) project for geotechnical data, following criteria like those already established in Geology. With the implementation of QAQC directly in the database, the business rules will automatically reject all drill holes that do not meet the quality standards required by Vale S.A., which may be linked to the data collection at the source, namely the operational drilling activities. Figure 5 shows one of the maps generated during the optimization phase for the Sul Superior Dam located at the Gongo Soco Mine in the state of Minas Gerais. The dam was at Level 3 from 2019 until mid-2024, reducing to Level 2 of emergency.

Fig. 5: Historical Data for the Sul Superior Dam

The optimization process evaluated over 30 geotechnical structures, resulting in a 33% reduction in the footage requested through the geotechnical demand application. The next step is to transcribe all unstructured historical data that is not in Vale S.A.'s official databases using Generative AI, as much historical information is still found in handwritten logs and text files. This will ensure that all drill holes have a primary key, guaranteeing data traceability and reliability. Estaire and Schweckendiek (2024) presented at the 7th International Conference on Geotechnical and Geophysical Site Characterization the evolution of the Eurocode regarding the use of historical data and the accuracy of geotechnical projects. According to the authors, European standards are being revised with a planned publication in 2027/2028, aiming to include statistical analysis of historical data to improve the parameters of geotechnical designs. In cases where historical data is not available, the Eurocode advises using characteristic parameters as represented in Figure 6. Estaire and Schweckendiek (2024) emphasize that geotechnical projects involve many uncertainties characteristics of geological formation processes and the spatialization of information. This directly affects the safety factors derived from parameters obtained through geotechnical investigations, highlighting the need for planning more accurate drilling operations with clear objectives to provide the necessary project data.

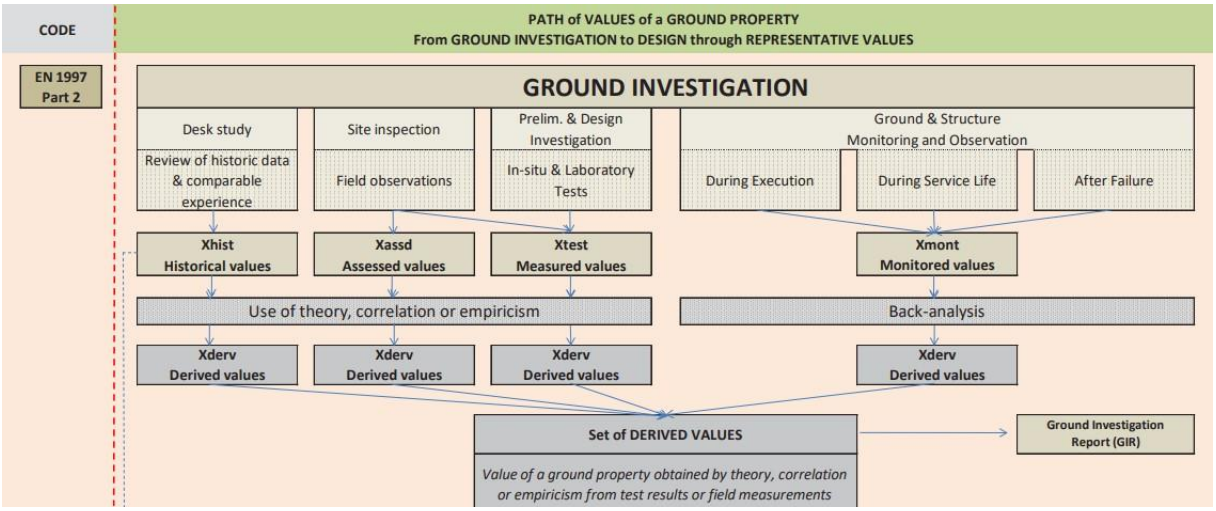


Fig. 6: Representation of the European normative flow for geotechnical investigation data. (ESTAIRE; SCHWECKENDIEK, 2024).

The optimization of geotechnical drilling campaigns also reinforces the need to store current and historical data in a single, centralized information database, with a known search and traceability flow for stakeholders to access the information whenever necessary. Another important factor is to establish quality controls for the data from the execution phase of the drilling to the importation into the databases, so that future projects are not penalized by a combination of information that does not align with expectations.

3. MANAGEMENT AND CONTROL OF GEOTECHNICAL DATA

3.1. Collection of geotechnical data in the field

The management of geotechnical data is not just a step in the phase of importing to databases and specific systems. It is a phase that begins during the execution of investigations, and for this purpose, a digital tool for capturing information, the RDS (Daily Drilling Report) Geotechnics, was implemented at the geotechnical drilling sites. The project was launched in 2023, with information being collected through an application on the mobile phones of the operational drilling inspectors and stored in a database (GDMS Drilling). All information is collected at the end of the work shift and subsequently evaluated through reports issued by the drilling companies as stipulated in the contract, establishing the reliability of the data.

Before the implementation of the RDS Geotechnics project, the lead time for capturing, storing, and validating all drilling execution data lasted 7 days, during which all field reports from the drilling companies were collected and transcribed into Excel templates by a multidisciplinary team. After the implementation of RDS Geotechnics, this lead time was reduced to 2 days, considering the acquisition of data and verification of the information imported into the GDMS Drilling.

Figure 7 shows the before (a) and after (b) of the implementation of the RDS Geotechnics project. In 2023, a mobile tool from Datamine ® called Quicklogger was implemented, which runs on iOS and Android, adapted to collect drilling execution information and directly connected to the GDMS Drilling database.

Fig. 7: Implementation of the RDS Geotechnics project: a) Before implementation; b) After implementation

In Figure 7a, it is possible to see the entire process of capturing information from 165 boreholes until the information is made available for management. The drilling companies, along with the inspectors, issued daily field reports, which were shared through work chat groups, where the data governance team collected relevant data for generating indicators and populated spreadsheets for issuing management reports.

After the implementation of the RDS Geotechnics (Figure 7b), the same 165 boreholes and inspection teams, instead of duplicating field information and filling out handwritten reports, began to fill in predefined activities in Quicklogger, with data in .json file format delivered to the governance team and imported in bulk into the GDMS Drilling system, leaving only the analysis of inconsistencies and specific corrections. What used to take 7 days to be presented in management reports can now be done in a maximum of 2 days. As a process improvement for the project, the method of acquiring field data changed to a more user-friendly Microsoft ® platform, with an approval flow that eliminates paper in the field and integrates with other databases, automatically connecting to all management reports and improving user usability. Figure 8 presents the new architecture of RDS Geotechnics with a data acquisition solution in Power Apps linked to the internal demands application shown in Figure 1 and integrated with GDMS Geotechnical Investigations, GDMS Drilling, and other auxiliary systems such as SGC (Contract Management System).

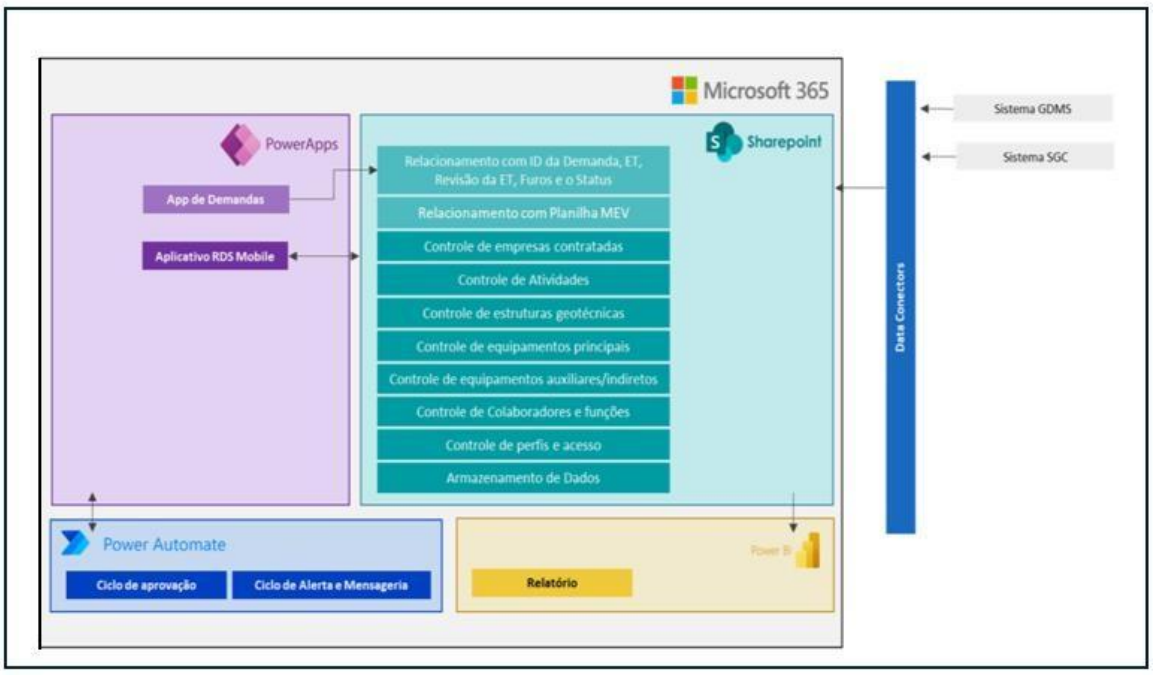


Fig. 8: Implementation of the new RDS Geotechnics solution

The main advantage of the new RDS Geotechnics solution is the online connection of field activities to the registration of requests from internal clients, considering the type of drilling to be performed, specific samples, and especially the restrictions that must be met, such as the use of water or fluids in waste structures. Thus, upon approval of the request made by the internal client, the activities are released in the RDS Geotechnics application, updated with the fleet of drilling rigs mobilized in the Contract Management System (SGC), and connected to technical databases such as GEOTEC and GDMS Geotechnical.

3.2. Visualization and interpretation of geotechnical data for projects

As previously mentioned in earlier sections, geotechnical investigations generate important data for internal management regarding equipment and companies, as well as the parameters that are computed in structural projects. Therefore, the acquisition and organization of information begins with capturing the requests, which are subsequently connected to the tools for acquiring field activities and the official databases of Vale.

Throughout the data capture process, engineering analyses are conducted to promote the optimization of campaigns and the spatialization of historical and planned data to increase the accuracy of projects. Once the campaigns are validated through optimizations, the final drilling data undergoes review by the governance team, which populates the information in the GDMS Geotechnical Investigations and GIS maps.

Figure 9 presents a summary of the flow starting from the registration of requests by internal clients and concluding with the management reports obtained through data imported into the GDMS Drilling and GDMS Geotechnical Investigations databases.

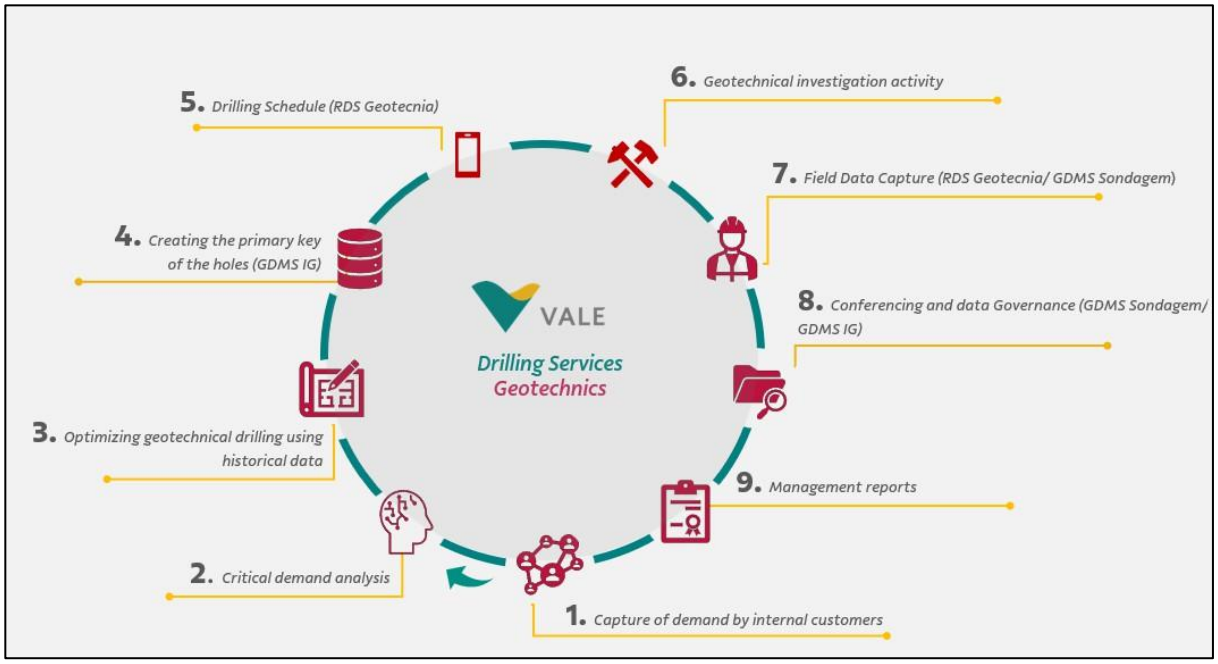


Fig. 9: Data management flow and geotechnical analyses in drilling campaigns

To support access for clients and other engineers to the technical data stored in the GDMS Drilling and GDMS Geotechnical Investigations databases, an output report was developed in Power BI that consumes both databases with geospatial and profile visualization, as shown in Figure 10:

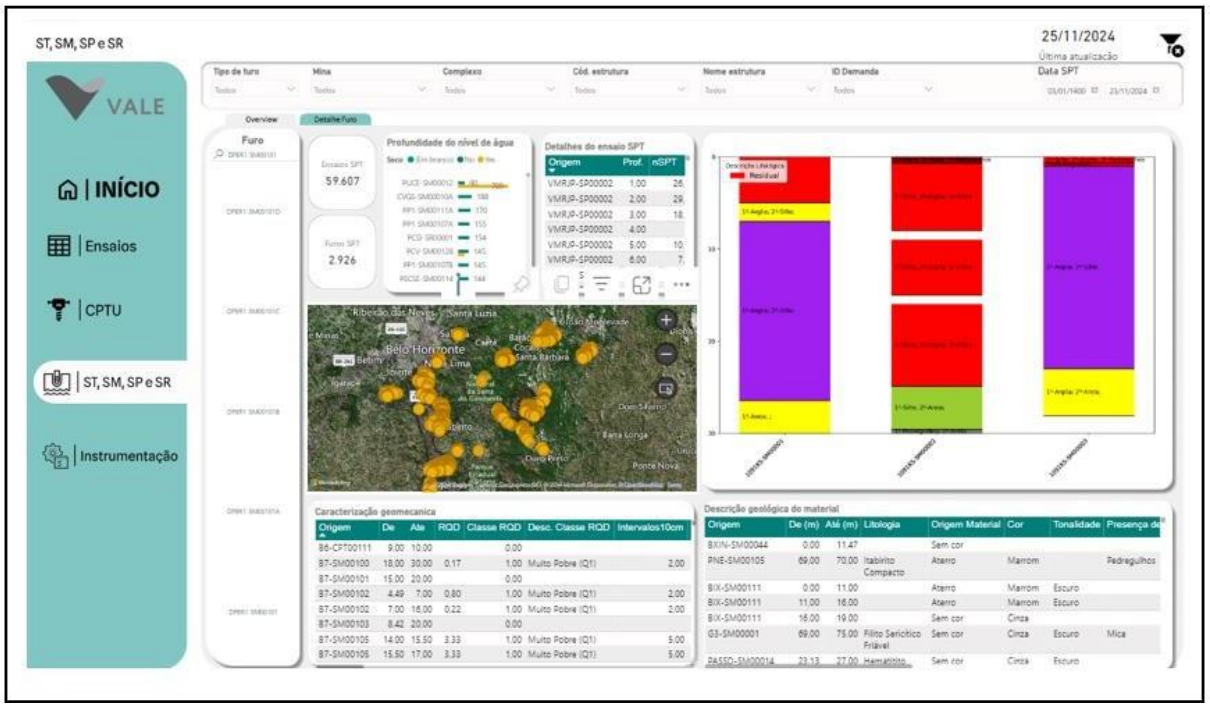


Fig. 10: Presentation of geotechnical data stored in GDMS Sondagem and GDMS Investigações Geotécnicas

The report in Figure 10 presents all the information stored in Vale's geotechnical databases since 2020, the year in which the GDMS Geotechnical Investigations was implemented in the

company following the accident at the B1 Dam in Minas Gerais. Table 1 summarizes the tests conducted at Vale and stored in the GDMS Geotechnical Investigations, associated with their import period, highlighting that historical data is in the process of being transcribed to the database and is not yet included in this summary.

Table 1: In situ tests stored in GDMS Investigações Geotécnicas.

Type of test	***Quantity (unit)***	***Year of Import***
Soil infiltration test	1.476 (*un*)	2020-2024
Water loss test in rock	242 (*un*)	2022-2024
Vane test	38 (*un*)	2022-2024
Dilatometric	72 (*un*)	2024
Dissipation	633 (*un*)	2022-2024
Seismic	497 (*un*)	2022-2024
CPTu	1.205 (*hole*)	2021-2024
SPT	2.926 (*hole*)	2020-2024

Through programming that utilizes libraries in Python, along with the conversion of geographic coordinates for the spatialization of the boreholes stored in the database, the report includes graphs (logs) generated from the results, as exemplified in Figure 11 for the borehole BGC-CPT00111B at the Campo Grande dam in Minas Gerais. This is a borehole with a CPTu test, where the graphs of tip resistance, lateral friction, and pore pressure are constructed directly from the validated data imported into the GDMS Geotechnical Investigations database.

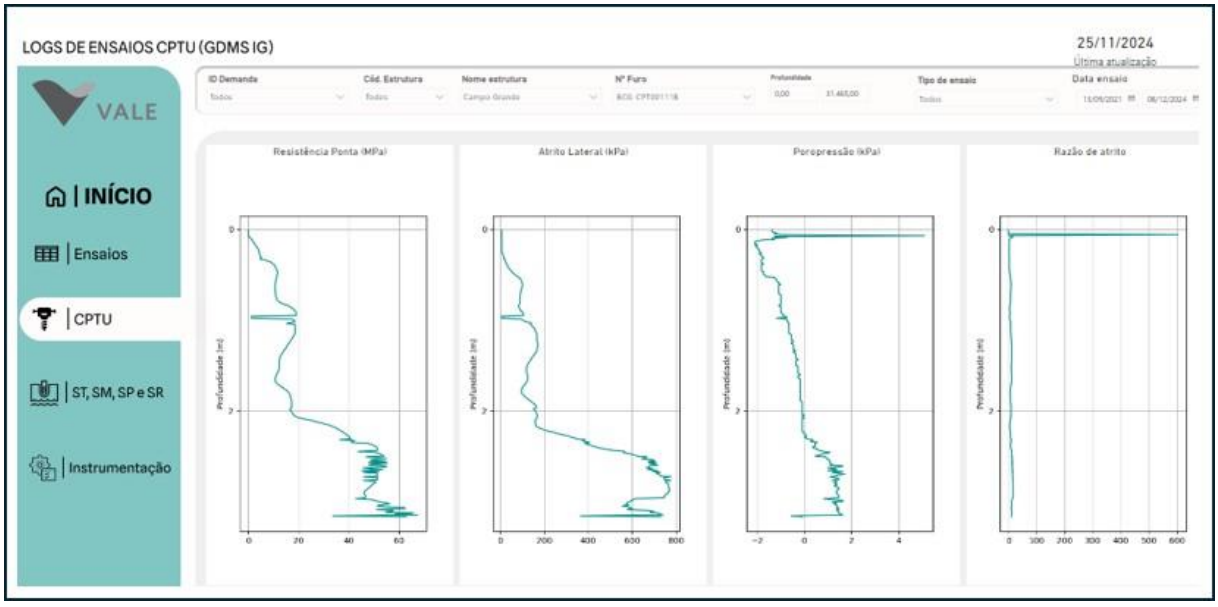


Fig. 11: CPTu test log generated from GDMS Investigações Geotécnicas data

Similarly to the CPTu tests, profiles were created for SPT tests and for mixed (SM) and rotary (SR) drilling. Figure 12 presents an example of a borehole with mixed drilling from the Campo Grande dam, indicating the geological-geotechnical description in accordance with Brazilian regulations and Vale S.A.'s internal standards.

Fig. 12: Mixed drilling log generated from GDMS Investigações Geotécnicas data

The report presented has the advantage of facilitating the consumption of information from the databases by various users without requiring them to have specific access to the platforms or to download tables for later analysis. The report allows for immediate interpretation and

decision-making, with daily updates or whenever new data is imported into the GDMS Drilling and GDMS Geotechnical Investigations databases.

4. CONCLUSIONS

In recent years, geotechnical investigations at mining dams in Brazil have undergone profound changes, especially following the tragic collapses of 2015 and 2019. These events not only shocked the nation but also spurred the creation of stricter laws, such as Laws 12.334 and 14.066, which demand a serious commitment from mining companies to ensure the safety and sustainability of their dams.

Vale S.A. has stood out in this context, heavily investing in geotechnical projects and expanding its drilling operations. The diversification of methods used, and the adoption of modern technologies have been crucial in ensuring that the data collected is both reliable and useful. For example, with the implementation of GDMS Fusion, data management has become much more efficient, allowing relevant information to be standardized and centralized in a single, traceable source, which facilitates and speeds up data analysis.

Another significant advancement was the digitization and transmission of drilling log data, which completely transformed how data is captured and validated. Previously, this process had a lead time of seven days, but with the new tool, that delivery time has been reduced to just two days. This means that information reaches decision-makers more quickly, facilitating the management of drilling operations.

Additionally, the optimization of drilling campaigns, which includes the analysis of historical data and the application of rigorous quality criteria, demonstrates a clear commitment to continuous improvement. Centralizing data in a single source and utilizing dynamic reports in Power BI make access to information much easier, enabling engineers and managers to make real-time decisions.

In summary, the future of geotechnical investigations at mining dams in Brazil looks promising, but it depends on the continued effort to enhance data management practices and adopt technological innovations. This way, we can not only mitigate risks and pr otect lives but also ensure that the mining sector follows a sustainable and responsible path.

5. REFERENCES

[1] AGÊNCIA NACIONAL DE ÁGUAS E SANEAMENTO BÁSICO (ANA). Brazil Dam Safety Report. Brasília: ANA, 2021. Available at: rsb-2021.pdf. Accessed on: November 19, 2024. (ANA, 2021).

[2] AGÊNCIA NACIONAL DE MINERAÇÃO (ANM). Mining Dam Dashboard. Brasília: ANM, 2024. Available at: Mining Dam Dashboard. Accessed on: November 19, 2024. (ANM, 2024).

[3] BRAZIL. Law 12.334, of September 20, 2010. Establishes the National Dam Safety Policy and provides other measures. Official Gazette of the Union, Brasília, DF, September 21, 2010. Available at: http://www.planalto.gov.br/ccivil_03/_Ato2007-2010/2010/Lei/L12334.htm. Accessed on: November 20, 2024. (BRAZIL, 2010a).

[4] BRAZIL. Law 2.434, of June 28, 1954. Provides for dam safety. Official Gazette of the Union, Brasília, DF, June 29, 1954. Available at: http://www.planalto.gov.br/ccivil_03/leis/L2434.htm. Accessed on: November 20, 2024.

[5] BRAZIL. Law 12.488, of August 11, 1997. Amends Law No. 6,938, of August 31, 1981, and provides other measures. Official Gazette of the Union, Brasília, DF, August 12, 1997. Available at: http://www.planalto.gov.br/ccivil_03/leis/L12488.htm. Accessed on: November 20, 2024. (BRAZIL, 1997a)

[6] BRAZIL. Law 9.798, of May 26, 1999. Provides for the National Water Resources Policy and provides other measures. Official Gazette of the Union, Brasília, DF, May 27, 1999. Available at: http://www.planalto.gov.br/ccivil_03/leis/L9798.htm. Accessed on: November 20, 2024. (BRAZIL, 1997b)

[7] BRAZIL. Law 5.549, of November 9, 2009. Provides for dam safety and provides other measures. Official Gazette of the Union, Brasília, DF, November 10, 2009. Available at: http://www.planalto.gov.br/ccivil_03/leis/L5549.htm. Accessed on: November 20, 2024.

[8] BRAZIL. Law 7.408, of August 18, 2010. Amends Law No. 12.334, of September 20, 2010, and provides other measures. Official Gazette of the Union, Brasília, DF, August 19, 2010. Available at: http://www.planalto.gov.br/ccivil_03/leis/L7408.htm. Accessed on: November 20, 2024. (BRAZIL, 2010b)

[9] BRAZIL. Law 14.066, of September 30, 2020. Amends Law No. 12.334, of September 20, 2010, and Law No. 9.433, of January 8, 1997, and provides other measures. Official Gazette of the Union, Brasília, DF, October 1, 2020. Available at: http://www.planalto.gov.br/ccivil_03/_Ato2020-2022/2020/Lei/L14066.htm. Accessed on: November 20, 2024.

[10] DATAMINE. Fusion Brochure. 2024. Available at: https://cdn-msaudata.pressidium.com/wp-content/uploads/2023/05/Fusion_Brochure_230510_PT.pdf. Accessed on: November 21, 2024. (DATAMINE, 2024)

[11] Estaire, J. and Schweckendiek, T. (2024). Treatment of uncertainties in site characterization in second-generation Eurocode 7, Plenary Lectures, 7th International Conference on Geotechnical and Geophysical Site Characterization (ISC'7).

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EVALUATION OF THE IMPACT OF CHANGING THE TAILINGS DISPOSAL METHOD IN THE RESERVOIR OF A ROCK SOIL DAM

Mateus Cruz [1]
JF Brasil

Ciro Franco[2]
JF Brasil

Katherine Flores[3]
University of Brasilia

ABSTRACT

The recent collapses of tailings dams in Brazil – Fundão (Mariana, 2015) and Córrego do Feijão (Brumadinho, 2019) – have led to changes in legislation and underscored the need for companies to implement measures that enhance the operational safety of dams. These measures should include strategies that increase the geotechnical and hydraulic safety of the structures while also reducing socio-environmental risks. Among these strategies is the alteration of the method of tailings disposal in the dam, which promotes the formation of beaches and moves the reservoir away from the embankment, reducing hydraulic loads and, consequently, interstitial pressures, hereby lowering the likelihood of piping failures.

Thus, this study aims to evaluate the influence of changing the tailings disposal arrangement in a downstream dam, enabling the formation of a beach and shifting the reservoir's water pond upstream (pond shift) in a mixed-section tailings dam (earth and rockfill) located in Brazil.

For this purpose, transient seepage analyses were carried out, with permeability and resistance parameters based on field and laboratory tests, to evaluate the time required for effective reduction of the hydraulic load in the embankment. Additionally, stability analyses were performed, with and without the presence of a beach, using the Slide2 software to estimate whether the adopted strategy impacts of the internal water flow within the dam and the reservoir. The time required for the effective reduction of the hydraulic load was also compared with the dam's lifecycle until 2030 (4 years after the change in the disposal method).

The results indicated that the formation of a beach does not influence the safety factor of the structure. This can be explained by several factors, such as the method of raising the structure, the constituent materials of the mass (notably the rockfill), the materials present in the foundation, and the nature of the saturated tailings. Although the formation of a tailings beach did not result in a substantial increase in the safety factor, it provides other important benefits. This demonstrates the importance of considering the specific characteristics and lifecycle of each structure before implementing operational changes, ensuring that such interventions are effective.

Keywords: Tailings dam safety, pond shift, hydraulic load reduction, beach formation

[1]Mateus.cruz@jfbr.com.br
[2]ciro.franco@jfbr.com.br
[3]katherine.mirella.flores@gmail.com

1. INTRODUCTION

The dam failures that occurred in Brazil, such as in Mariana (2015) and Brumadinho (2019), led to changes in legislation and emphasized the importance of companies adopting measures to improve the operational safety of dams through practices that enhance the geotechnical and hydraulic stability of these structures.

Among the strategies used by operators for dams already in operation, the formation of tailings beaches and the pond shift (relocation of the pond upstream) stand out. These measures aim to reduce hydraulic pressures on the dam's mass and mitigate the risks of failure. However, implementing beaches in already operational dams presents challenges, including changes in tailings disposal, alterations to water intake systems, and strategic assessments of the time required to achieve the expected benefits and the dam's lifespan.

This study aims to evaluate the implementation of a tailings beach, with the upstream displacement of the pond (pond shift). For this purpose, a downstream-raised dam with a mixed section (composed of soil and rockfill) in operation is considered. Using Rocscience's Slide2 software, transient and stability analyses are performed to compare scenarios with and without beach formation. This comparison allows for assessing whether the beach formation impacts aspects such as the hydraulic gradient, the phreatic level, and the dam's safety factor.

2. TAILLING DAMS AND DISPOSAL METHODS

The generation of waste is an inherent consequence of any industrial process. In mining, the increased demand for mineral inputs in recent years has significantly driven the expansion of the sector's activities [1]. This scenario has led to a growing generation of waste, which is distinct from that produced in other sectors, particularly due to the presence of solid waste from extraction – known as sterile – and from beneficiation – the tailings.

The disposal of tailings in dams is the most common method used in Brazil. This method is widely applied due to its ability to contain large volumes of tailings, which are generally disposed of as slurry (a mixture of solids and water) with a high-water content (90% to 75% water) [1].

In general, for new structures, the most common dam construction method is hydraulic fill [2]. In this methodology, from an initial dam, typically constructed with compacted soil, the tailings are transported as slurry through pipelines to the dam. During the process, the tailings are separated by hydrocyclones into fine (overflow) and coarse (underflow) fractions [3]. When the disposal is made at the crest, the fine fraction, with a high water content, is directed to the reservoir, while the coarse fraction is used to form the tailings beach [4].

The presence of water in tailings is associated with more than 75% of the recorded failures in tailings dams [5]. Therefore, understanding the water table and pore pressures is crucial to ensuring the safety of a tailings dam [6]. Over the years, several authors have proposed strategies to reduce interstitial pressures and enhance the geotechnical safety of dams throughout their operational lifespan. Among these strategies, the following stand out: beach formation, segregation of deposited tailings in the reservoir, foundation permeability, drainage systems, construction speed, and downstream slope of the structure [6][7][8][9].

This article will evaluate how the formation of a beach in the reservoir influences the geotechnical safety of the structure. To achieve this, it is proposed to change the tailings disposal method in an existing dam, replacing single-point disposal with tailings placement along the dam's crest through spigots. This is because, when tailings are disposed of from upstream to downstream, no beach formation occurs, and the reservoir lake remains in direct contact with the embankment mass, as observed in the dam analyzed in this study. Thus, the implementation of a tailings beach in an existing dam requires not only operational changes but also technical, physical stability, and hydraulic safety analyses, which are typically conducted in such projects.

3. STUDY ASSUMPTIONS

For the development of this study, the following criteria and assumptions were considered: The stability and percolation analyses were conducted using the cross-section presented in item 3.1 of this study. The resistance and hydraulic conductivity parameters of the materials were adopted based on the literature, in addition to field and laboratory tests. Regarding stability analyses, these were performed using limit equilibrium theory through deterministic analyses, considering the rupture wedge calculation methods for circular surfaces: Spencer, Simplified Bishop, and GLE/Morgenstern & Price [10].

Percolation analyses were conducted considering a transient flow regime to assess the influence of time on the lowering of the groundwater table within the reservoir. To evaluate the impact of beach formation on the groundwater level of the structure, two scenarios were considered: one with a beach, with a 0.5% slope, and another without. The water level in both scenarios was set at elevation 172.30 m, which corresponds to the elevation of the spillway crest and represents the maximum water level that can be reached within the reservoir, based on the assumption that the structure cannot overflow.

3.1. Dam Geometry

The geotechnical geological sections to be considered in this study are shown in Fig. 1. and Fig. 2.

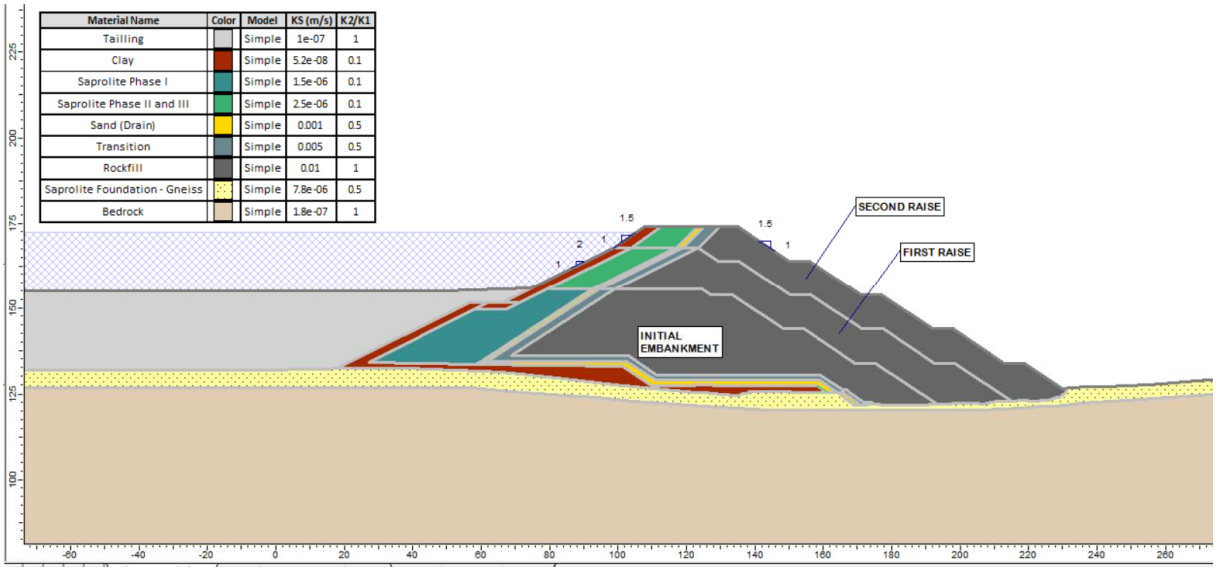


Fig. 1 – Dam Geometry without Beach

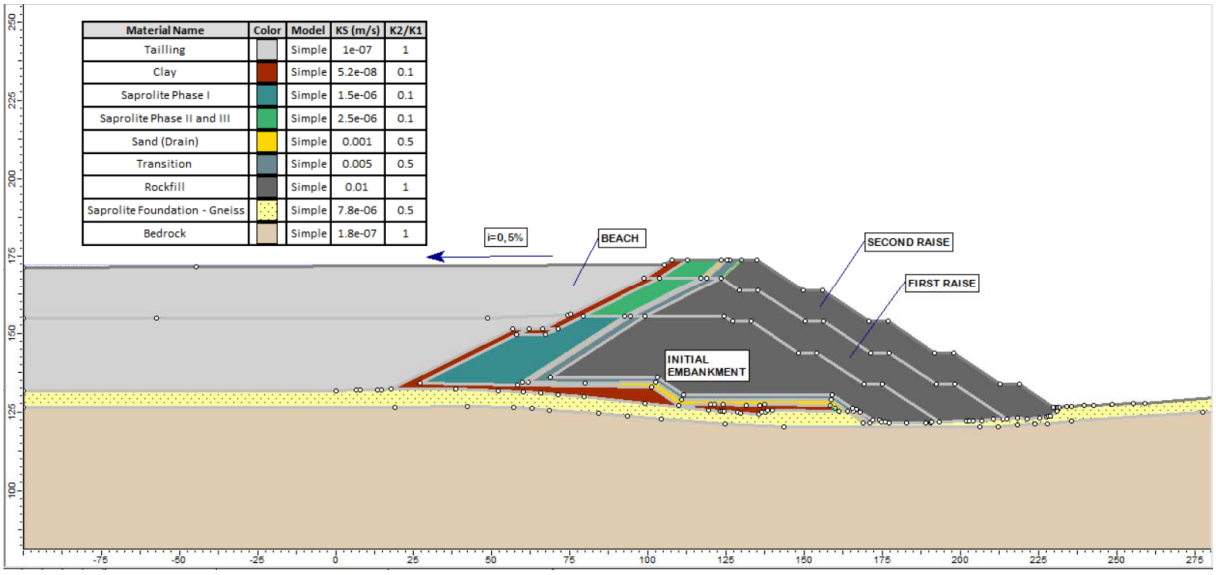


Fig. 2 – Dam Geometry with Beach, considering slope of 0,5%

As shown in Fig. 1 and in Fig. 2, the dam is constructed downstream and features two raised sections, which contribute to enhanced stability and performance. The dam has a total height of 52.00 meters and a crest width of 27.00 meters. It includes five downstream berms, each 6 meters wide, with a 1.5H:1V slope. The upstream face of the dam initially has a 2H:1V slope, which transitions to a 1.5H:1V slope starting at an elevation of 168.00 meters. The structure incorporates a clay layer on the upstream face, ensuring impermeability, while beneath this

layer, there is a compacted soil layer. Additional layers are included to form the internal drainage system, with rock fill positioned on the upstream side to further reinforce the structure.

3.2. Resistance and permeability parameters

The resistance parameters adopted for the materials in the stability analyses are presented in Table 1. These parameters were defined based on the available technical literature [5], as well as field and laboratory tests.

Table 1 – Materials

Material	Unit Weight (kN/m^3)	Cohesion (kPa)	Phi (^{o})	K (m/s)
Tailling	18	Su min -22+4.4H		1,0 x 10^{-7}
Clay	18.5	18	19	
Existing Saprolite	20	6	30	1,5 x 10^{-6}
Saprolite Phase II	18.5	16	24	2,5 x 10^{-6}
Sand (Drain)	20	0	35	1,0 x 10^{-3}
Transition	21	0	37	5,0 x 10^{-3}
Rockfill	24.5	-		1,0 x 10^{-2}
Saprolite Foundation - gneiss	18	13	30.5	1,8 x 10^{-7}
Bedrock	27	-	-	1,8 x 10^{-7}

The strength of the rockfill was defined based on the envelopes suggested by [11] for minimum strength rockfill and the specific weight according to the available technical literature [12]. The shear strength was defined using the Hoek-Brown failure criterion. UCS=100 MPa, GSI=65, mi=28, D=0.The shear strength was defined using the Hoek-Brown failure criterion. UCS=50 MPa, GSI=65, mi=27, D=0.

The hydraulic conductivity coefficients for these materials were estimated based on recommended values [6][13][14]

4. RESULTS

About the change in the concept of the layout plan, providing for beach formation from the crest and away from the water (pond shift), this study makes the following considerations based on the data analyzed

4.1. Evaluation of the Efficiency of Hydraulic Load Reduction in the Embankment

To evaluate the efficiency of the tailings beach in reducing the hydraulic loads on the embankment and its influence on the factor of safety, percolation analyses were conducted on the section presented (Fig. 1) considering the two scenarios specified in Item 3 (with and without tailings beach).

Fig. 3 through Fig. 6 present the safety factors derived for the dam under various temporal scenarios, accounting for a transient flow regime. In addition to depicting the safety factors, these figures also highlight a notable decrease in the groundwater table within the reservoir over time.

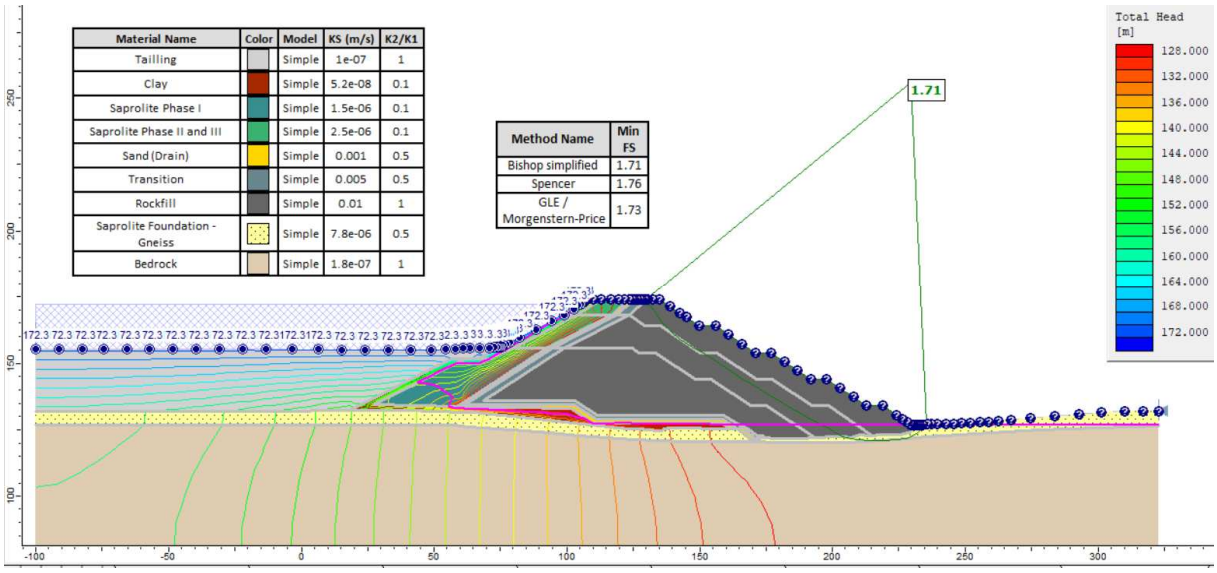


Fig. 3 – Stability and Percolation Analysis, Scenario Without Beach Formation

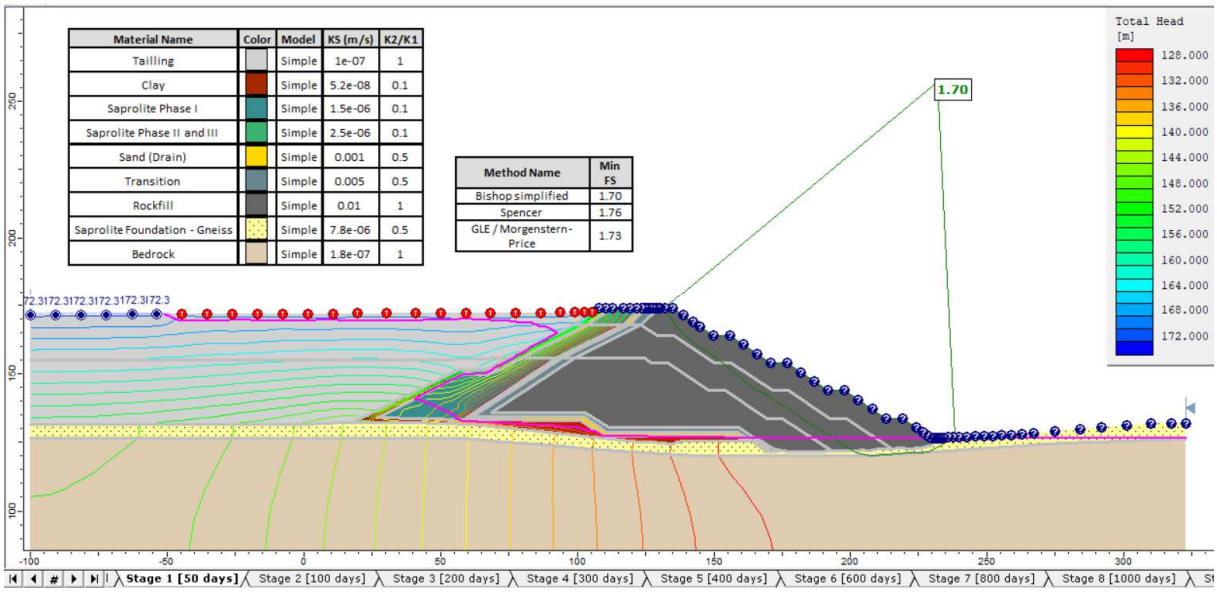


Fig. 4 – Stability and Percolation Analysis, Scenario 50 Days After Completion of Beach Formation and Cessation of Tailings Discharge

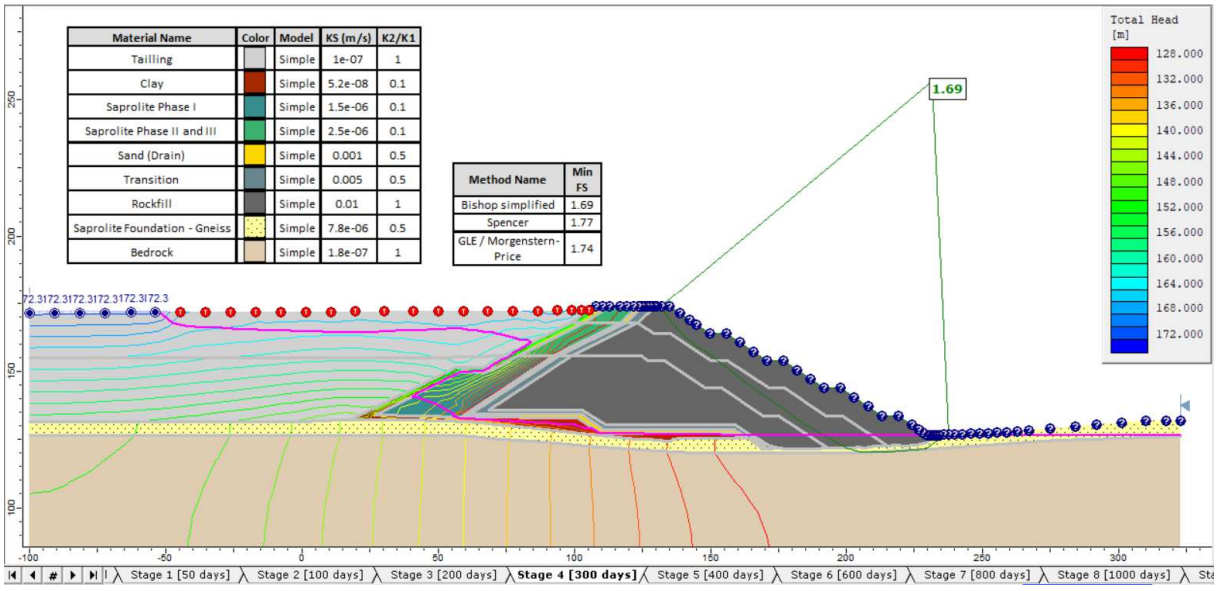


Fig. 5 – Stability and Percolation Analysis, Scenario 300 Days After Completion of Beach Formation and Cessation of Tailings Discharge

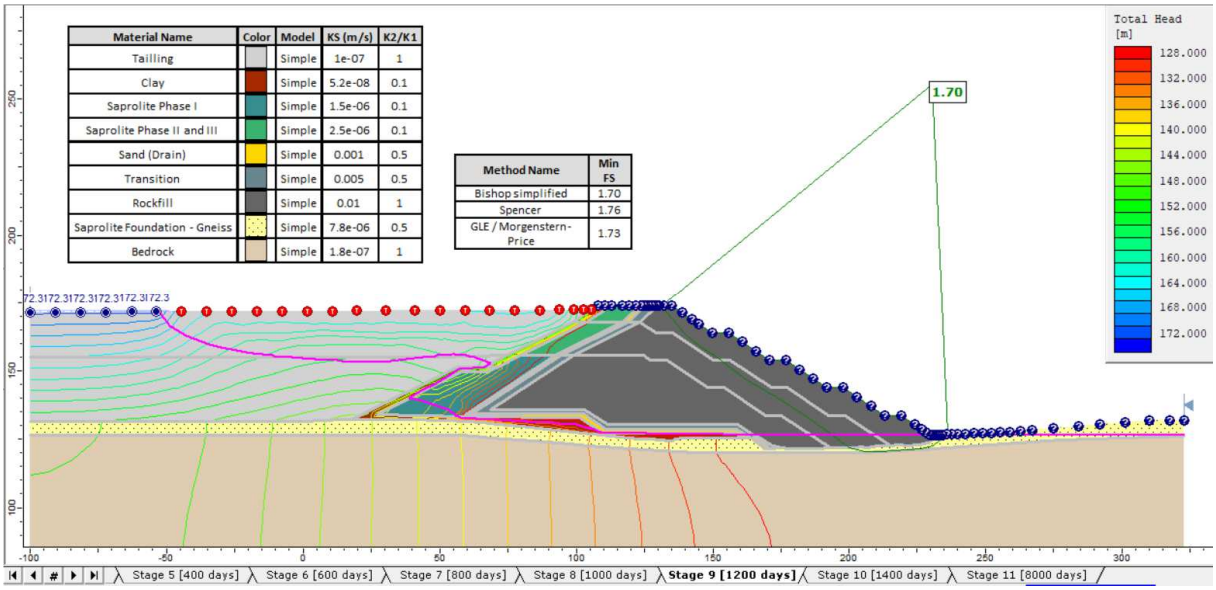


Fig. 6 – Stability and Percolation Analysis, Scenario 1200 Days After Completion of Beach Formation and Cessation of Tailings Discharge.

Regarding the results presented in the above figures, several points deserve attention:

The transient flow analysis showed that a long period is required for a significant reduction of the water table inside the tailings (around 1200 days). Considering the operational lifespan of the dam, 4 years, this period is sufficient to achieve a notable reduction in the water table within the reservoir.

Another important point is that, from an operational perspective, the tailings discharge through spigot systems is considered a hydraulic disposal method, which uses water as a fluid. Therefore, not only is the formation of the beach not immediate, but the continued recharge of the system may delay the saturation of the lower layers for several years. Thus, the formation of the beach should be assessed alongside other factors, such as the operational life of the structure and the cost-benefit analysis of the operation.

It is also worth mentioning that the potential recharge of water into the beach, either by precipitation or through the spigot system, was not considered in the percolation analysis. As a result, one might expect an even slower reduction in the water table than what was numerically simulated.

Regarding safety factors, as shown in Table 2.

Table 2 – Safety Factors of the Dam

Condition	Water Table	Safety Factor
Steady-State Flow Regime – Without Tailings Beach	172.30 m (Spillway Crest)	1.71
Transient Flow Regime – With Tailings Beach	172.30 m (Spillway Crest)	1.70

The results of the stability analyses, presented in Table 2, indicate that the downstream raised dam configuration provides an adequate factor of safety. This is due to the presence of efficient drainage elements and the reduction in the concentration of hydrostatic pressures in the massif, factors that significantly contribute to the increased stability of the structure. Furthermore, the low-permeability layer upstream acts as an additional barrier, reinforcing the safety of the dam.

Although the formation of a tailings beach did not result in a substantial increase in the safety factor for this specific configuration, it provides other important benefits. One of these is the reduction of interstitial pressures in the reservoir and the structure, an effect that, although not directly reflected in the stability analysis, can be observed by the change in the groundwater table. This decrease in the groundwater table, in turn, helps reduce the risk of piping failures, a mechanism commonly observed in dams with this configuration.

This reduction is caused by the low permeability of the sludges, which act as an auxiliary barrier, causing a significant loss of head and, consequently, lowering the groundwater table before the water approaches the dam. In this way, a more extensive tailings beach enhances the head loss, resulting in significant reductions in the water level [9].

Another observed benefit is the influence on the hydraulic gradient with the presence of the beach. Fig. 7 and Fig. 8 present the scenarios without and with the beach, respectively.

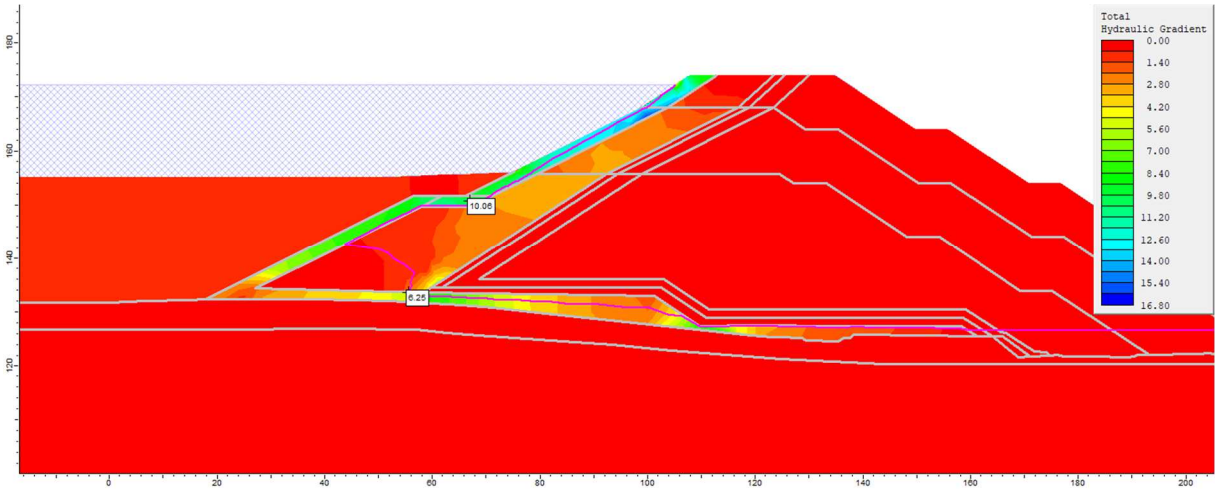


Fig. 7 – Gradient Hydraulic Analysis, Scenario Without Beach Formation

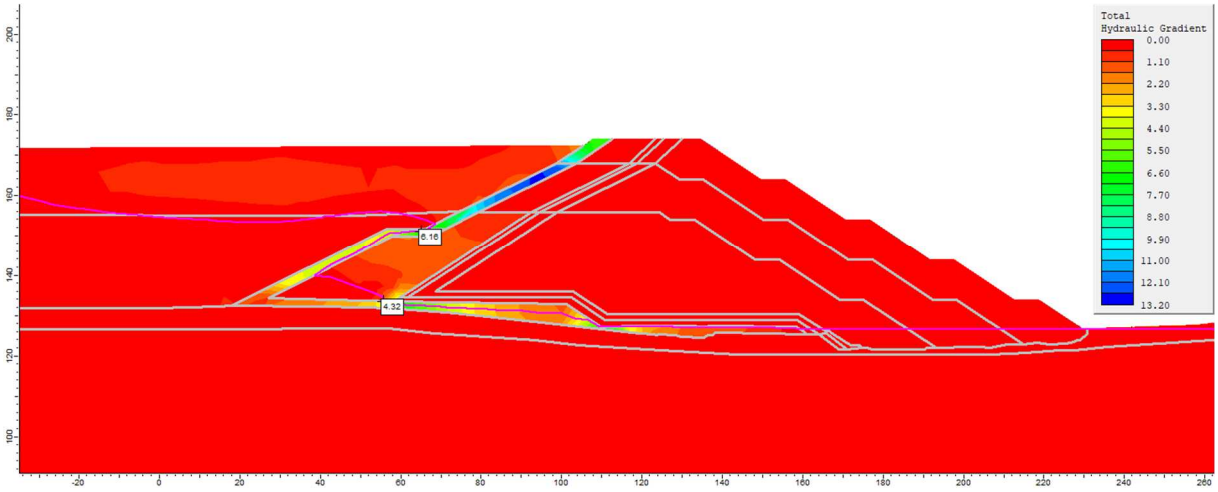


Fig. 8 – Gradient Hydraulic Analysis, Scenario of Beach Formation and Cessation of Tailings Discharge.

In Figure 7, a high hydraulic gradient is observed, particularly in the region where water comes into contact with the structure. In Figure 8, however, a reduction in the gradient can be seen due to the presence of the beach. This can be explained by the fact that the beach decreases direct contact between the water and the embankment, increasing the distance the water must travel—both within the reservoir and through the embankment—before reaching the structure's drainage system. As a result of this increased flow path, the hydraulic gradient decreases.

Additionally, in the contact region between the soil and the dam's drainage system, a reduction in the gradient is also recorded, contributing to a lower risk of piping, as evidenced by studies on the hydraulic behavior of tailings dams.[8]

4.2. Evaluation of Necessary Interventions for Changing the Tailings Disposal Location

The relocation of the tailings discharge point will require the construction of a spigot system, including pipelines and a pumping system. Additionally, a new spillway system will need to be built. These interventions involve high level of implementation complexity and significant costs for the company.

The placement of the spigots must be carefully studied, ensuring they are not positioned in locations that would negatively impact on the operation of the spillway. Therefore, the operation of these spigot points could result in spillway obstruction and changes in the physical parameters of the discharged water.

5. CONCLUSIONS

This study presents technical and operational considerations regarding the adoption of a change in the tailings disposal method in the reservoir of a downstream-raised dam, as a strategy to enhance the operational safety of the structure. The proposed method will require the relocation of the spillway system and the implementation of a spigot system along the entire crest of the dam, which entails high implementation costs. The implementation of the spigot system facilitates the formation of a beach but also presents significant challenges during the execution and operation phases.

From a geotechnical perspective, although percolation analyses under transient flow conditions indicate that a significant reduction in the water table would require a long period (at least 1200 days), the formation of the beach does influence the water table in the reservoir. This occurs because the beach contributes to the reduction of the water table within the structure and shifts the lake away from the dam, thereby increasing the internal head loss of the dam. As a result, the probability of piping failure is reduced.

As a result of the analyses conducted, it was also observed that the formation of a beach in downstream-raised mixed dams has little influence on the safety factor (FS), as the rockfill, by its more permeable nature, does not favor the development of pore pressures. Therefore, although the benefits of beach formation do not directly reflect on the safety factor, they manifest in other aspects, such as the behavior of internal water flow and the reduction of the hydraulic gradient in the reservoir and the embankment. These factors contribute to the stability of the structure by minimizing the risks of internal erosion and, consequently, structural instability.

These results highlight the importance of evaluating various factors when proposing a change in the tailings disposal method, such as the structure's configuration, the compatibility with the dam's lifecycle, implementation costs, and operational risks. Additionally, the study reveals that, through transient analyses, it is possible to preliminarily assess how beach formation contributes to increasing the geotechnical safety of the dam. If this strategy proves unfeasible, it would be possible to consider a better allocation of financial and material resources, which could also bring benefits to the population affected by the dam's operation.

REFERENCES

[1] IBRAM (2016). Gestão e Manejo de Rejeitos da Mineração /Instituto Brasileiro de Mineração.

[2] Luz, A. B.; Sampaio, J. A.; Almeida, S. L. M. (1998).Tratamento de minérios. 4 ed. Ed. Adão Benvindo da Luz, Mario Valente Possa, Salvador Luiz de Almeida. Rio de Janeiro, CETEM/MCT, 1998. 867 p;

[3] D'Agostino, L. F. (2008). Praias de barragens de rejeitos de mineração: características e análise da sedimentação, PhD Thesis, USP.

[4] Ribeiro, L. F. M. (2000). Simulação Física do Processo de Formação dos Aterros Hidráulicos Aplicado a Barragens de Rejeito, PhD Thesis, UnB.

[5] Heidarian, P. (2012). Effect of Initial Water Content and Stress History on Water-Retention Behaviour of Mine Tailings. MASc Dissertation. Carleton University

[6] Vick, S.G. (1983). Planning, Design and Analysis of Tailings Dams. John Wiley & Sons, Inc., 369 p.

[7] Klohn, E. J. (1980). The development of current tailings dam design and construction methods.Design and Construction of Tailings Dams. Colorado School of Mines Press

[8] Brisson, P. (2006). Unsaturated flow in tailings. MASc Dissertation. University of Ottawa

[9] Sarsby, R. W. (2013). Chapter 15: Tailings dams. In R. W. Sarsby, Environmental Geotechnics (pp. 365-391). ICE Publishing

[10] Rocscience (2010). Slide v6.0 - 2-D Limit Equilibrium Slope Stability Analysis. Rocscience, Inc. (www.rocscience.com), Toronto, Ontario

[11] Cruz, P. T.; Materón,B.; Freitas, M. (2020). Barragens de enrocamento com face de concreto. 2. ed. São Paulo: Oficina de Textos.

[12] Leps, T. M. (1974). Review of shearing strength of rockfill. Journal of Soil Mechanics & Foundations Div.

[13] Fell, R.; Macgregor, P.; Stapledon, D.; Bell, G. (2005). Geotechnical Engineering of Dams. Balkema, Leiden ISBN 04 1536 440 x.

[14] USBR (2014). Hydraulic conductivity estimation test impact on long-term acceptance rate and soil absorption system design. Water, 6(9), 2808-2820.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

HISTORICAL CASES OF LIQUEFACTION IN DAMS: LESSONS LEARNED

Renata Ferreira[1] **Thiago Faria**[2] **Luana Pereira**[3]

ABSTRACT

Dam failures have the potential to cause significant social, environmental, and economic impacts, making it essential for all stakeholders involved with these structures to understand their behavior. Liquefaction, a phenomenon that occurs when a material loses its shear strength under monotonic, cyclic, or dynamic loading at constant volume, leads to significant deformations, causing the material to behave like a fluid. This paper presents a detailed review of historical cases of dam failures caused by liquefaction, highlighting the conditions that led to this phenomenon in various structures and incidents worldwide. It also examines the consequences for communities, infrastructure, and the environment. Based on this analysis, the paper discusses the key lessons learned and how these events have influenced the evolution of engineering practices and changes in dam safety regulations. From these reflections, recommendations are proposed to enhance dam safety in the face of liquefaction risk, promoting a preventive and rigorous approach to risk management in dams.

Keywords: liquefaction, dams, safety, lessons

[1] engenharia.renataferreira@gmail.com, Brazil

1. INTRODUCTION

Soil liquefaction represents one of the main challenges for structures built on saturated or nearly saturated sandy soils. It can be divided into two types: cyclic liquefaction, caused by repetitive loading that can reduce the soil's effective stress to zero, resulting in a loss of stiffness; and static liquefaction, arising from soil weakening due to deformations, leading to a significant reduction in shear strength.

According to Robertson (2017), static liquefaction is particularly concerning for large structures, such as mining tailings dams and earth dams. As these structures become larger and more complex, it becomes increasingly essential to predict soil behavior under conditions of high overburden stresses. Over the decades, various procedures have been developed to estimate the liquefied shear strength in sandy soils, with emphasis on methods based on case history analyses, widely used by authors such as Seed (1987), Seed and Harder (1990), Ishihara (1993), Olson and Stark (2002), among others.

The tragic dam failures caused by liquefaction, such as those at the Fundão Dam in Mariana and Dam I in Brumadinho, have brought to light the severity of this phenomenon in Brazil. These events not only highlighted the limitations of current analysis and monitoring methods but also drove advances in procedures for assessing liquefaction susceptibility and in legislation regulating dam safety.

In this context, this study aims to provide a comprehensive analysis of the concepts, causes, and mechanisms associated with static liquefaction, addressing the impacts of this phenomenon on structures in Brazil from social, environmental, and economic perspectives. Through a detailed review of the literature, the key lessons learned from historical events were identified, and the influence of these episodes on the evolution of engineering practices and the reformulation of safety regulations was analyzed. Finally, the study proposes recommendations to improve liquefaction risk mitigation strategies and strengthen dam safety.

2. THE PHENOMENON OF LIQUEFACTION

Soil liquefaction is a phenomenon in which the soil loses its strength and rigidity due to an increase in pore pressure. This process typically occurs over a short period and can lead to the failure of the soil structure. Liquefaction can be triggered by two main mechanisms: As illustrated in Figure 1, liquefaction can occur through two main mechanisms:

(i) Monotonic loading, characterized by static loads, increased pore pressure, or reduced confinement, a condition known as static liquefaction; and

(ii) Cyclic loading, caused by dynamic or repetitive loads, usually associated with earthquakes or intense vibrations from other sources, referred to as cyclic liquefaction.

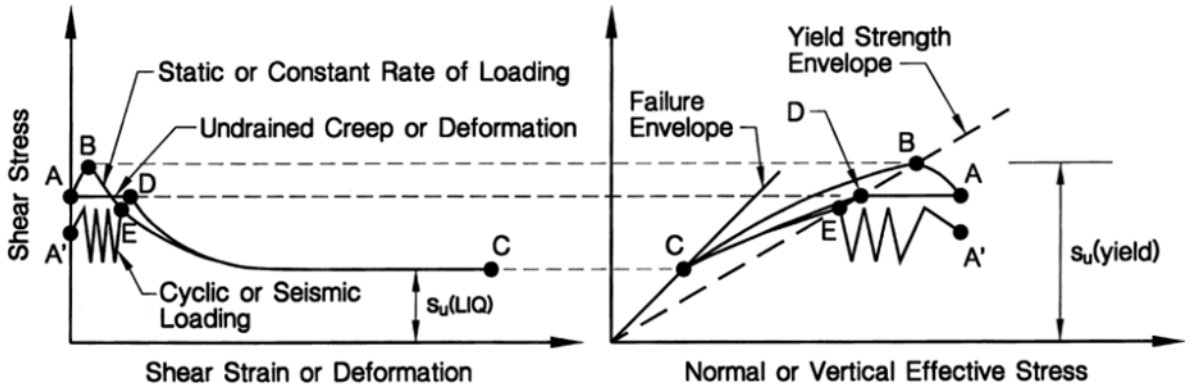


Figure 1- Schematic behavior of saturated, contractive sandy, Olson and Stark (2003).

According to Terzaghi et al. (1996), there are important geotechnical characteristics that can be identified in materials with liquefaction potential. These include: non-cohesive, loose, and saturated soils. These soils are prone to liquefaction when they exhibit high void ratios, so when subjected to shear movements, they exhibit contractive behavior. If the permeability is sufficiently low, drainage does not occur, leading to increments in pore pressure and a loss of strength.

In this context, the first step to assess a soil's susceptibility to liquefaction under static shear stresses is to determine whether it displays contractive or dilative behavior. Soils classified as contractive are more susceptible to liquefaction, and appropriate verification methodologies must be adopted for their analysis.

Tabela 1 presents some relevant bibliographical references for evaluating liquefaction susceptibility.

Tabela 1- References for evaluating liquefaction susceptibility

Source: Adapted from Paes et al., 2020.

Test Type	Parameter Assessed	Authors
S-CPTu	State Parameter	Jeffereis and Been (2016)
S-CPTu	Shear Wave Velocity (Vs1)	Keyen et al. (2013)
S-CPTu	SBT Chart (historical cases)	Robertson (2010)
SPT	Envelope (historical cases)	Olson and Stark
Characterization	Grain Size Distribution	Tsuchida (1970), Ishihara et al. (1980)
Characterization	Grain Size Distribution	Andrews and Martin (2000)
Characterization	Natural Moisture Content and Atterberg Limits	Seed et al. (2003)
Characterization	Natural Moisture Content and Atterberg Limits	Perlea et al. (1999)
Characterization	Natural Moisture Content and Atterberg Limits	Bray et al. (2004)
Triaxial	Brittleness Index	Bishop (1971) apud Souza (2018)

2.1. Liquefaction cases around the world

In 2001, the International Commission on Large Dams (ICOLD) published Bulletin 121, titled “Tailings Dams: Risk of Dangerous Occurrences, Lessons Learnt from Practical Experiences”. This document presents 221 records of tailings dam failures around the world.

According to the accident history reported by ICOLD, the main causes of dam failures include foundation issues, inadequate spillway capacity, slope instability, lack of erosion control, and deficiencies in monitoring and inspection during the post-operation period (Figure 2). All these causes are directly related to the lack of operational control during dam management and are generally associated with the inadequate presence of water in the systems (Machado et al., Azeez, 2018).

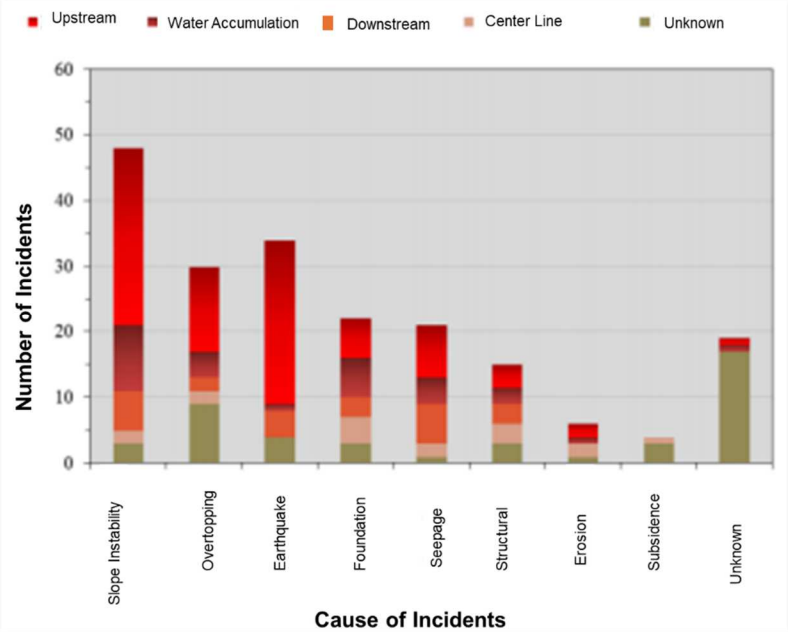


Figure 2- Causes of Tailings Dam Failures.

Source: Adapted from Machado et al. Azeez, 2018.

There is no precise information on the total number of structures that have failed due to liquefaction worldwide. However, two cases of tailings dam failures caused by liquefaction in Brazil have gained widespread media and academic attention. These events stood out due to the circumstances in which they occurred, the significant damage they caused, and the discussions they triggered regarding dam safety. Below, these cases will be analyzed in greater detail.

2.2. Dam I – Feijão – Brumadinho, Minas Gerais – Brazil

The Feijão dam, located in Brumadinho, Minas Gerais (Figure 3), belonged to the mining company Vale and stored iron ore tailings. On January 25, 2019, the structure suffered a sudden collapse, releasing a massive flow of tailings that resulted in hundreds of deaths, as well as causing severe environmental and socioeconomic damage. This case brought to light critical issues regarding the control and monitoring of tailings dams and highlighted static liquefaction as one of the main factors contributing to the structure's instability.

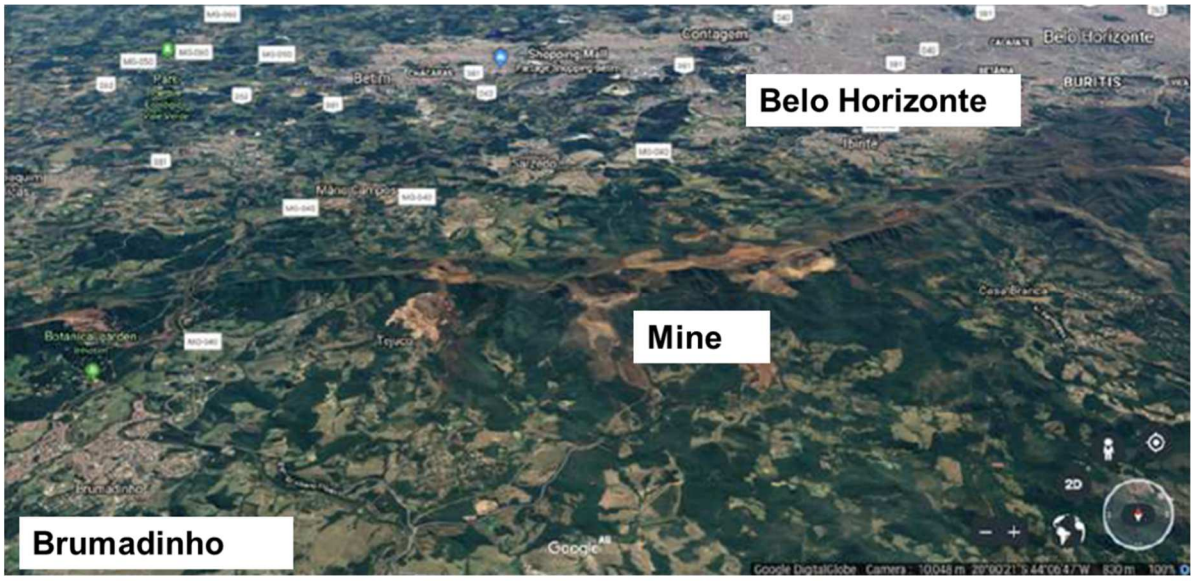


Figure 3- Location of the Córrego do Feijão Mine, with a view to the north in relation to Belo Horizonte and Brumadinho (source: Robertson et al., 2019)

Post-disaster technical studies identified static liquefaction as a determining factor for the collapse of the dam. The structure, composed of inadequately compacted layers of tailings, exhibited high susceptibility to strength loss, particularly under the saturated conditions that existed. The accumulation of water in the pores of the saturated soil generated internal pressures that drastically reduced the material's strength, leading to a progressive collapse.

According to Robertson et al. (2019), several factors contributed to the generation of instability conditions at Dam I, including:

- A design that resulted in a steep upstream-constructed slope;
- Water control within the tailings basin that sometimes allowed the water from the settling pond to approach the dam crest, resulting in the deposition of weak tailings near the crest;
- A design setback that pushed the upper parts of the slope onto weaker fine tailings;
- The lack of significant internal drainage, which led to a persistently high water level in the dam, particularly near the dam toe;
- A high iron content, resulting in heavy tailings with cementation between particles. This cementation created rigid tailings that exhibited potentially very brittle behavior when subjected to a trigger that caused an undrained response;
- High and intense regional rainfall during the rainy season, which could cause significant suction loss, producing a slight strength loss in unsaturated materials above the water table.

According to Robertson et al. (2019), the resulting collapse and mudslide were caused by the static liquefaction of the tailings within the dam. The factors described above produced a dam

composed mainly of loose, saturated, heavy, and brittle-behaving tailings, with high shear stresses on the downstream slope, resulting in a marginally stable dam (i.e., close to failure under undrained conditions). Laboratory tests showed that the amount of specific deformation required to trigger strength loss could be very small, especially in the weaker tailings. These were the key components that enabled static liquefaction.

Robertson et al. (2019) concluded that the sudden loss of strength and the resulting collapse of the marginally stable dam were due to a critical combination of continuous internal specific deformations caused by creep and a reduction in strength due to suction loss in the unsaturated zone caused by intense rainfall at the end of 2018. This followed several years of increasing precipitation after tailings deposition ceased in July 2016. Pre-failure specific deformations calculated from this combination of triggers aligned well with the small specific deformations detected in post-collapse analysis of satellite images from the year preceding the failure. The internal specific deformations and the strength reduction in the unsaturated zone reached a critical level, resulting in the observed collapse on January 25, 2019.

Figure 4 and Figure 5 show the structure before and after the failure, respectively.

Figure 4- Dam I - Feijão before the failure. Source: Globo G1, Brumadinho before and after.

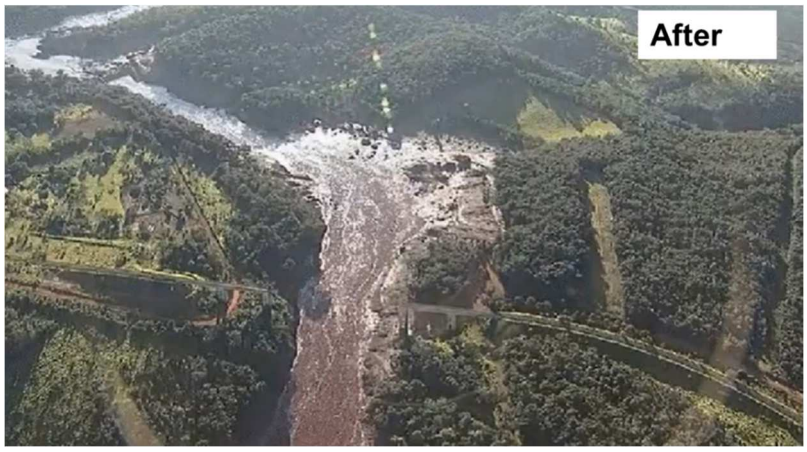


Figure 5- Dam I - Feijão after the failure. Source: Globo G1, Brumadinho before and after.

The Brumadinho disaster led to a strong push to review regulations and safety practices for tailings dams, especially those built using the upstream construction method, which was subsequently banned in several regions of Brazil. Among the main lessons learned are:

The upstream construction method poses high inherent risks, especially for structures located in areas of high saturation. In response to the Brumadinho case, alternatives such as downstream construction and dams with structures made of denser soils have been preferred.

The use of advanced technologies, such as sensors for real-time pore pressure measurement and drones for visual inspection, has become a recommended practice for identifying structural changes before a potential collapse. The implementation of piezometers and accelerometers has become an essential practice for detecting signs of imminent liquefaction.

More frequent and rigorous inspections and independent audits are essential to ensure that safety and maintenance protocols are followed. Constant verification of geotechnical and structural records allows for the identification of vulnerability signs and the implementation of preventive measures.

The collapse of the Feijão dam represents a turning point in the approach to dam safety. This case prompted regulatory and methodological advances in dam engineering, emphasizing the importance of detailed geotechnical analyses, continuous monitoring, and safer construction and maintenance practices. The incident highlighted the need for a deeper understanding of static liquefaction, particularly in tailings dams, and reinforced the importance of active monitoring and the use of technological tools to mitigate these risks

2.3. Fundão Dam – Mariana, Minas Gerais – Brazil

The Fundão Dam (Figure 6) is located in the municipality of Mariana, in Minas Gerais. Built using the upstream construction method, the structure was primarily designed to store tailings from mining operations. The dam's activities began in 2009, and in its final phase, the crest reached an elevation of 900 meters, resulting in a total height of 110 meters. Figure 7 shows the A-A' section of the structure.

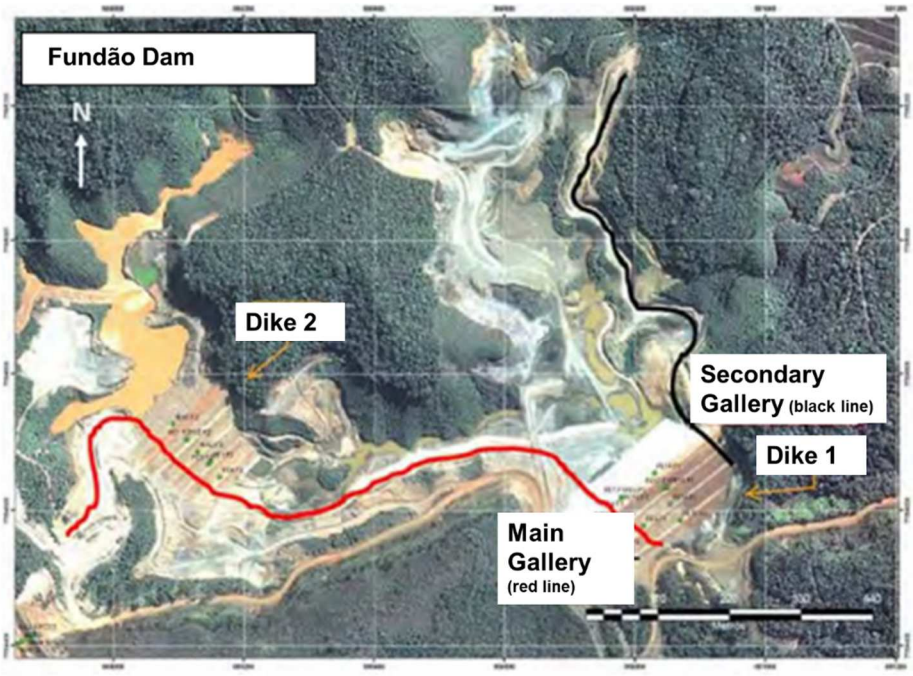


Figure 6- Dam I - Feijão after the failure. Source: Globo G1, Brumadinho before and after.

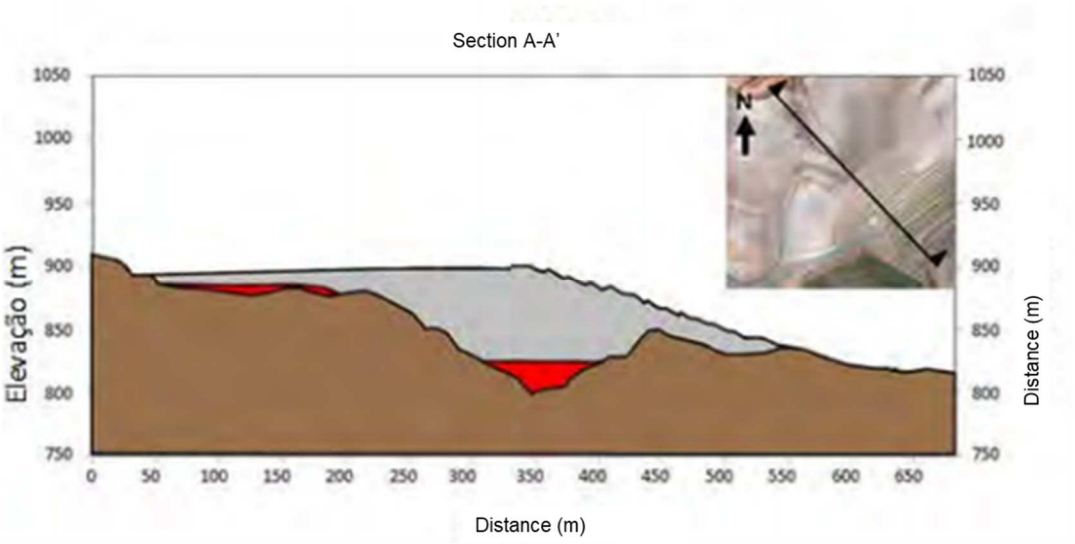


Figure 7- Original design of the Fundão Dam. Source: Report on the Immediate Causes of the Fundão Dam Failure.

According to the Report on the Immediate Causes of the Fundão Dam Failure, the Fundão Dam failed on November 5, 2015. Approximately 32 million m³ of tailings were released, representing 61% of the stored material — a significantly high proportion compared to other tailings dam failures. Figure 3.3 shows the dam before the failure (a) and the structure after the failure (b).

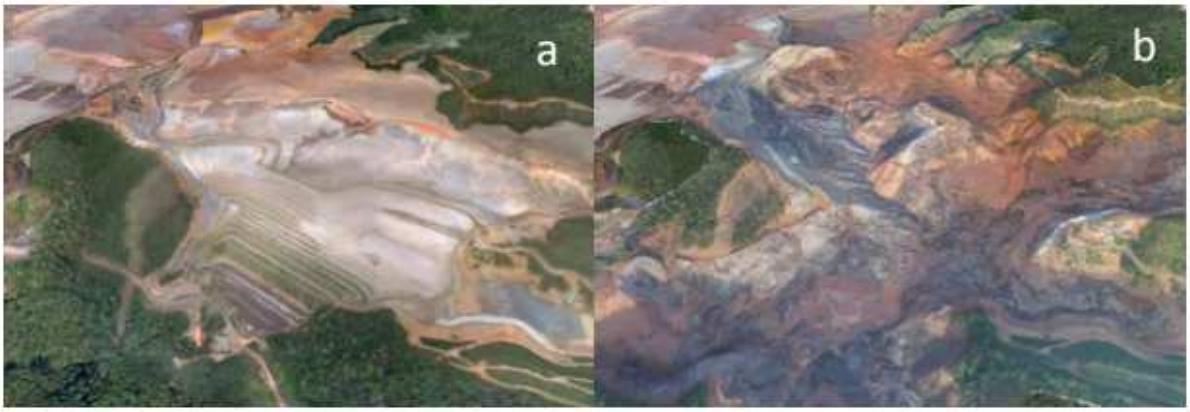


Figure 8- Fundão Dam (a) before, (b) after the failure. Source: Report on the Immediate Causes of the Fundão Dam Failure.

The collapse of the Fundão Dam occurred due to a fluid landslide triggered by liquefaction, which began at the left abutment of the structure. This failure resulted from a complex chain of events and conditions that culminated in the instability of the dam. The report highlighted several prior events and conditions that contributed to the occurrence of static liquefaction: (1) saturation of the sand, (2) loose, uncompacted sand, and (3) a trigger mechanism. Originally, the dam design included an unsaturated sand zone, ensuring robustness against liquefaction. However, design modifications resulted in the saturation of the sand, while the hydraulic deposition of tailings contributed to the formation of loose sand conditions.

The fluid landslide began at the left abutment, where the dam had been set back to accommodate repairs and raising operations. This setback caused significant loading over mud-rich deposits, which, by accumulating in an area originally intended for sand deposition, formed a barrier to downward drainage and created a zone of structural weakness.

The trigger of the landslide was identified as a lateral extrusion mechanism, where the vertical raising of the dam caused compression and lateral expulsion of the mud-rich zone, reducing the confinement of the overlying sand layers and leading to the collapse. Laboratory and computational modeling confirmed that the collapse occurred when the dam reached the height recorded on the date of failure.

Although seismic tremors were recorded shortly before the failure, their contribution was considered secondary, acting only as an accelerator of an already advanced collapse process due to the continuous loading imposed by construction activities.

This event highlighted significant failures in the design, execution, and operational control of the dam, underscoring the need for rigorous engineering standards and monitoring to prevent similar disasters in the future.

3. ASSESSMENT OF THE CONSEQUENCES

3.1 Consequences environmental, human, and socioeconomic impacts

The following are the main consequences of the collapse of the Fundão Dam and Dam 1, focusing on the environmental, human, and socioeconomic impacts.

Fundão Dam:

The collapse of the Fundão Dam, considered the largest environmental disaster in Brazil's history and one of the largest in the world, caused profound economic, social, and environmental damage. In addition to claiming the lives of 19 people, the event directly impacted 41 cities in the states of Minas Gerais and Espírito Santo, according to the Federal Prosecutor's Office. The disaster also affected three indigenous reserves, inhabited by the Krenak, Tupiniquim, and Guarani peoples.

From an environmental perspective, approximately 240.88 hectares of Atlantic Forest were severely degraded, and more than 50 million cubic meters of mining tailings were dumped onto land and into water bodies, contaminating rivers and soils. Local biodiversity was also drastically affected, with the collection of approximately 29,300 fish carcasses along the Carmo and Doce rivers, totaling 14 tons of dead fish.

Dam I:

The collapse of Dam I at the Córrego do Feijão Mine in Brumadinho (MG), which occurred on January 25, 2019, was one of the largest socio-environmental tragedies in Brazilian history. Operated by the mining company Vale S.A., the dam failed, releasing 11.7 million cubic meters of mining tailings, causing large-scale destruction. This article analyzes the main human, environmental, and socioeconomic impacts of this catastrophic event.

The human dimension of the tragedy was the most devastating. A total of 272 people lost their lives, including mine workers and local residents, and some victims remain missing, prolonging the suffering of families. In addition to the fatalities, hundreds of people were displaced, losing their homes and sources of livelihood.

The environmental damage was extensive and far-reaching. The tailings mud buried large areas of the Atlantic Forest, destroying entire ecosystems and eliminating species of fauna and flora. The Paraopeba River, one of the main watercourses in the region, was severely contaminated, making it unfit for human consumption, fishing, and agriculture.

Studies indicate that heavy metal contamination in soils and rivers could have long-term impacts, hindering the natural regeneration of the environment and the economic recovery of affected communities. Additionally, the loss of biodiversity in the region represents an irreparable loss for local environmental balance and the preservation of endemic species.

The tragedy also had significant economic repercussions. Local farmers and fishermen lost their sources of income, while the water supply for several cities, including Belo Horizonte, was affected. The need to seek alternative water sources increased costs for local governments and consumers.

3.2 Legislative consequences

The disasters at Mariana and Brumadinho served as stark reminders of the devastating consequences of inadequate safety practices in tailings dam management. In response, Brazil implemented a series of crucial regulatory changes aimed at preventing future catastrophes and ensuring stricter oversight of dam safety. One of the most significant revisions was the implementation of Resolution 95 by the National Mining Agency (ANM), which brought about sweeping reforms in the way tailings dams are constructed and managed.

Among the most pivotal changes introduced by Resolution 95 is the ban on the upstream construction method for new dams, a technique that has been directly linked to the catastrophic failures of both the Fundão and Brumadinho dams. This method, known for its high risk in areas with unstable geological conditions, is now prohibited, ensuring that future projects will adopt safer, more stable engineering practices. Moreover, the resolution mandates the decharacterization of existing upstream dams, requiring them to undergo significant modifications to enhance their stability or be dismantled altogether.

In addition to these measures, Resolution 95 emphasizes the importance of rigorous external audits. It requires independent auditors to conduct comprehensive technical evaluations, which must include a thorough analysis of the dam's susceptibility to liquefaction—a critical factor in preventing potential dam failures. This provision aims to provide an added layer of accountability and ensure that dams are assessed from multiple, unbiased perspectives.

As part of the decharacterization process, a critical deadline was established for the completion of upstream dam modifications: February 25, 2022, as stipulated in § 2, Article 2-A of Law No. 12.334 of 2010. This deadline marks a significant step in the ongoing efforts to eliminate the most dangerous dam construction methods in Brazil. However, the resolution allows for potential extensions if a technical justification is submitted and approved by the ANM and the appropriate licensing authority within the National Environmental System (SISNAMA). This flexibility ensures that the process can adapt to complex situations while maintaining a commitment to safety.

Alongside these national regulatory reforms, GISTM (Global Industry Standard on Tailings Management) was developed as a critical global framework to guide responsible tailings management. The GISTM, developed by the International Council on Mining and Metals (ICMM), the United Nations Environment Programme (UNEP), and the Principles for

Responsible Investment (PRI), sets out a set of rigorous, transparent, and accountable standards for tailings management. It emphasizes the importance of safety, environmental protection, and community engagement in managing tailings facilities. Under the GISTM, companies are required to prioritize the safety and well-being of local communities, conduct regular reviews of their operations, and ensure the implementation of best practices in design, operation, and closure of tailings facilities.

These regulatory changes, in alignment with GISTM, are a direct response to the lessons learned from the Mariana and Brumadinho disasters. By focusing on stricter technical criteria, enhancing inspection protocols, and placing a stronger emphasis on disaster prevention, these measures seek to bolster dam safety and reduce the risk of similar incidents in the future. The goal is not only to protect human life and the environment but also to foster a culture of responsibility within the mining industry.

Ultimately, the measures outlined in Resolution 95, in conjunction with the global guidelines provided by GISTM, represent a significant step forward in improving the safety of tailings dams in Brazil. However, they must be part of a broader, sustained effort that includes continued investment in technology, research, and community engagement. Only by embracing a comprehensive, proactive approach to dam safety, supported by both national regulations and international standards like GISTM, can Brazil ensure a safer future for its communities, ecosystems, and industries.

4. PREVENTIVE MEASURES

The following are suggestions for preventive measures to avoid the occurrence of similar disasters:

• **Adoption of Safer Construction Techniques**: The upstream construction method presents inherent risks, particularly in areas with high soil saturation. Following the Brumadinho disaster, safer alternatives, such as downstream construction and the use of dams built with denser soil structures, have gained preference. These methods provide better pressure distribution, enhance stability, and reduce the potential for catastrophic failure. Furthermore, employing sustainable construction practices, such as the use of eco-friendly materials, can mitigate environmental impact and reduce long-term risks associated with climate change.

• **Continuous Monitoring and Advanced Technologies**: Leveraging advanced technologies is critical for proactive risk management. The use of real-time sensors to measure pore pressure, along with drones for visual inspections, has become an essential practice for identifying structural changes before they lead to a collapse. Additionally, the implementation of piezometers and accelerometers has proven vital in detecting early signs of liquefaction and

other potential failures, enabling timely interventions to prevent disaster. The integration of Artificial Intelligence (AI) and machine learning in predictive analytics also holds significant promise for early detection and forecasting structural issues, offering an additional layer of protection against failures.

• **Regular Inspections and Independent Audits**: Frequent and rigorous inspections, coupled with independent external audits, are essential to ensure adherence to safety protocols and maintenance standards. Continuous verification of geotechnical and structural records is crucial for identifying vulnerabilities and enabling prompt preventive actions. Independent audits, in particular, bring an objective perspective, reinforcing the credibility and reliability of safety measures. Alongside inspections, maintaining detailed environmental impact assessments will ensure that the broader ecological consequences of dam operations are continuously evaluated, contributing to the long-term sustainability of these structures.

• **Crisis Management and Disaster Response Planning**: Effective crisis management frameworks must be in place to prepare for potential failures. This includes establishing clear and coordinated disaster response strategies, with predefined roles for emergency responders, local authorities, and affected communities. A critical aspect of these plans is ensuring that they are regularly tested and updated to reflect evolving risks, technological advancements, and lessons learned from past disasters.

• **Educational Initiatives and Capacity Building**: Promoting education and training on dam safety, environmental protection, and risk management at all levels – from local communities to engineers and policymakers – is fundamental in fostering a culture of safety. Regular workshops, simulation exercises, and cross-sector knowledge-sharing platforms can ensure that all stakeholders, including industry professionals and the general public, are better prepared to identify, manage, and mitigate potential risks.

The collapse of these structures has had devastating consequences, highlighting the urgent need for reform in the approach to dam safety. In response to such disasters, significant advancements have been made in both the regulatory and technical aspects of dam engineering. These include a stronger emphasis on detailed geotechnical analyses, the integration of continuous monitoring systems, and the adoption of safer construction and maintenance practices. Moreover, there is a growing recognition of the importance of designing dams with greater environmental resilience to withstand extreme weather events caused by climate change.

These incidents have also emphasized the critical need for a deeper understanding of static liquefaction mechanisms, especially in the context of tailings dams. Furthermore, they have underscored the importance of active surveillance and the integration of advanced technologies to identify and mitigate potential risks proactively. Collaborative efforts between

scientists, engineers, and regulatory bodies are essential to deepening our understanding of these complex processes and mitigating their occurrence.

In summary, these tragic events have spurred the development of more robust safety standards and technologies. Moving forward, it is essential to continue refining these practices, investing in innovation, and ensuring their widespread adoption to prevent future catastrophes and safeguard both human life and the environment. Emphasizing sustainability, community engagement, and rigorous safety protocols will be crucial in achieving a future where such disasters are minimized, and their impacts mitigated.

5. CONCLUSIONS

Static liquefaction, a phenomenon that contributed to catastrophic failures like those of the Fundão and Córrego do Feijão dams, has revealed not only significant technical shortcomings but also the devastating environmental, social, and economic consequences resulting from inadequate dam management. The collapses of these two dams stand as some of the largest environmental disasters in Brazil's history, and among the most severe globally. These events not only caused significant loss of life but also led to the displacement of entire communities and the destruction of indigenous territories.

For the Fundão Dam, the environmental impact was immense. The disaster led to the degradation of approximately 240.88 hectares of Atlantic Forest, the disposal of over 50 million cubic meters of tailings onto land and into water bodies, and the contamination of vital rivers and soils. The local biodiversity suffered a severe blow, with the death of 14 tons of fish in the Carmo and Doce rivers, underscoring the scale of the ecological damage.

At the Dam I – Feijão, nearly 13 million cubic meters of iron ore tailings flowed toward the Córrego do Feijão community in Brumadinho, resulting in the destruction of homes, vegetation, and critical urban infrastructure. This disaster further emphasized the urgency of adopting more robust safety protocols and engineering practices to prevent such tragedies.

In response to these catastrophic failures, significant regulatory reforms were introduced, emphasizing the importance of detailed geotechnical analysis, continuous monitoring, and the incorporation of advanced technologies in dam safety. For instance, the enactment of ANM Resolution 95 marked a major step forward, banning the use of the upstream construction method for new dams and requiring the decharacterization of existing ones. This resolution also strengthened audit and monitoring requirements, underscoring the need for greater oversight in dam management.

However, to truly prevent such tragedies from recurring, it is critical to go beyond regulatory measures. It is essential to deepen our understanding of the technical factors that lead to static

liquefaction, invest in further scientific research, and foster a culture of prevention that prioritizes community safety and environmental protection.

The environmental, human, and socioeconomic repercussions of these disasters highlight that dam safety is a multidimensional responsibility, requiring the integration of technical expertise, regulatory measures, and ethical considerations. Only through collaborative efforts among engineers, regulators, researchers, and society can we prevent future disasters and ensure a safer, more sustainable future for all.

REFERENCES

[1] Terzaghi, K., Peck, R. B., & Mesri, G. (1996). *Soil Mechanics in Engineering Practice* (3rd ed.). John Wiley & Sons.

[2] Olson, S. M., & Stark, T. D. (2003). *Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments*.

[3] Robertson, P. (2017). Evaluation of Flow Liquefaction: Influence of High Stresses. Available at:

[4] Machado, L. V. T., & Azeez, D. G. (2018). *Incidents and Accidents in Dams.* 7th Symposium on Environmental Management and Biodiversity.

[5] Morgenstern, N. R., Vick, S. G., Viotti, C. B., and Watts, B. D. (2016). Report on the Immediate Causes of the Failure of the Fundão Tailings Dam. Expert Panel for the Analysis of the Fundão Dam Failure.

[6] Brumadinho before and after: see images of the collapse of the Vale dam. Available at: https://g1.globo.com/mg/minas-gerais/noticia/2019/01/25/antes-e-depois-veja-imagens-do-rompimento-de-barragem-da-vale-em-brumadinho-mg.ghtml.

[7] Robertson, P., Melo, L., Williams, D., and Wilson, G. (2019). Report of the Panel of Experts on the Technical Causes of the Collapse of Dam I at the Córrego do Feijão.

[8] Paes, I. T., Carneiro, J. J. V., Oliveira, T. M. Q., and Alves, H. O. (2020). Flowchart for Preliminary Analyses of Liquefaction Susceptibility. XX Brazilian Congress of Soil Mechanics and Geotechnical Engineering, IX Brazilian Symposium on Rock Mechanics, IX Brazilian Symposium of Young Geotechnical Engineers, VI South American Conference of Young Geotechnical Engineers.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

RESCUE SYSTEMS: THE CHALLENGE OF PERFORMING GEOTECHNICAL SURVEYS IN TAILINGS DAMS AT AN EMERGENCY LEVEL

Fernando Santos[1]
Vale S.A.

Marieli Biondo[2]
Vale S.A.

Eduardo Rodrigues Kelly E Sousa[3]
Vale S.A.

ABSTRACT

In Brazil, talings dams are grouped into emergency levels according to their situation and anomalies. Level 1 indicates that the dam has some anomaly that indicates the developer's attention, but that allows the dam's operation to be maintained. Level 2 indicates that the anomaly is not controlled and requires that the operation must be stopped until full understanding is stablished. And finally, Level 3 indicates that the dam structure is in a condition of collapse or imminent rupture and according to current legislation, all the community in the downstream area is removed and simulations of the PAEBM (Emergency Action Plan) are carried out frequently. After the tailings dams failures in 2015 and 2019, raising by the upstream method was prohibited and structures already built with this type of raising or unidentified method need to be decharacterized. In Minas Gerais there is a group of tailings dams raised by the upstream method that are in the design phase for decharacterization. In order for the project to be safe and not affect downstream communities, a containment structure was built at the end of mud patch to serve as a backup in the event of a rupture. Two dams were grouped into Level 2 and one into Level 3, requiring the use of unmanned equipment for earth movements, helicopters for reading instruments and rescue systems for carrying out geotechnical surveys. Three different rescue systems were installed to carry out mixed soundings, tests such as SPT, CPTu and collection of laboratory samples in which operators work trapped in the cables and in any emergency can trigger the rescue control. It was necessary to carry out differentiated training with the survey teams that routinely work in open areas and without the need for special operations like these. In addition to training, specific procedures for geotechnical investigations were also developed in order to ensure the health and safety of the team, as well as the maintenance of the quality of the information generated.

Keywords: Emergency Level; Decharacterization; Emergency Action Plan; Geotechnical Surveys.

[1]fernando.santos9@vale.com, Brazil
[2]marieli.lopes@vale.com, Brazil
[3]eduardo.kelly@vale.com, Brazil

1. INTRODUCTION

Dams have been fundamental structures in mining operations since the beginning of the activity, whether in water storage or in the packaging of tailings. In the second case, they play an even more central role, since they are used for the final disposal of tailings from the mining operation and their construction generally uses materials with no added value for the activity – waste. As a result, they play a role in environmental and social protection and, in some cases, with the potential for future use of materials through the development of new processing technologies.

However, recent occurrences involving dams in the mineral sector – Mariana, 2015 and Brumadinho, 2019, both in the state of Minas Gerais, Brazil – demonstrated not only the need to evolve construction methods, but to increase knowledge of these structures, especially those built using the upstream raising method. It is important to highlight that the accidents increased the perception levels of companies, professionals, authorities and governmental and non-governmental bodies regarding previously minimized risks, such as the impact of removing mineral phases from tailings with increased recovery during processing, the environmental potential involving these structures, the role of water in the behavior of the structure, the rheology of materials, among others.

In addition to the technical issues involved in the process, Brazil reviewed all its laws and decrees involving dams. It is worth highlighting Law 12.334, of 2010, which established the general safety guidelines for dams and Law 14.066 of 2020, known for the increase in restrictions involving such structures, such as the prohibition of the upstream raising method and the need to de-characterize dams constructed by this method.

Brazil currently has more than 5.000 dams covered by the National Dam Safety Policy, in a universe of more than 22.000 structures. With the legal changes, the demand for surveys and research using direct methods in these structures has increased significantly, with Vale alone having an increase of 269% in demand for surveys and in situ tests in the last four years, with an increase in productivity of 76%. However, to enable the entry of equipment and people into these structures, especially those at Level 3, methodologies had to be created and major engineering works implemented for feasibility.

At this core, this article aims to demonstrate the implemented structures that allowed the development of activities in these structures, their operational principles, their limitations and general considerations about them.

2. GEOGRAPHIC LOCATION OF THE FEATURED STRUCTURES AND CHARACTERISTICS COMMON TO ALL

The tailings dams that received the rescue structures are located on the western flank of the Iron Quadrangle, an important Brazilian mineral province located in the center-south region of the state of Minas Gerais, more specifically in the Fábrica mine, located in the municipalities of Ouro Preto and Congonhas. This strip of mines located on the geological megastructures of Serra da Moeda and the western portion of Serra do Curral is internally called by Vale of Corredor Sul, covering the municipalities of Brumadinho, Belo Horizonte, Nova Lima, Itabirito, Ouro Preto, Sarzedo, Jeceaba, Belo Vale, Rio Acima and Congonhas.

The Forquilhas I, II and III dams, as well as the Grupo dam received lifelines. The Forquilhas I and II dams received the Ropeway, while the Forquilhas III dam received the Lift Line Spider (LLS). The choice of system per structure was made based on the complexity of the activities necessary to reduce emergency levels, the geometry/positioning of the work to be carried out in the structure, as well as the spatial/geometric configuration of the dam. Figure 1 shows the location of these structures depending on the dams in question.

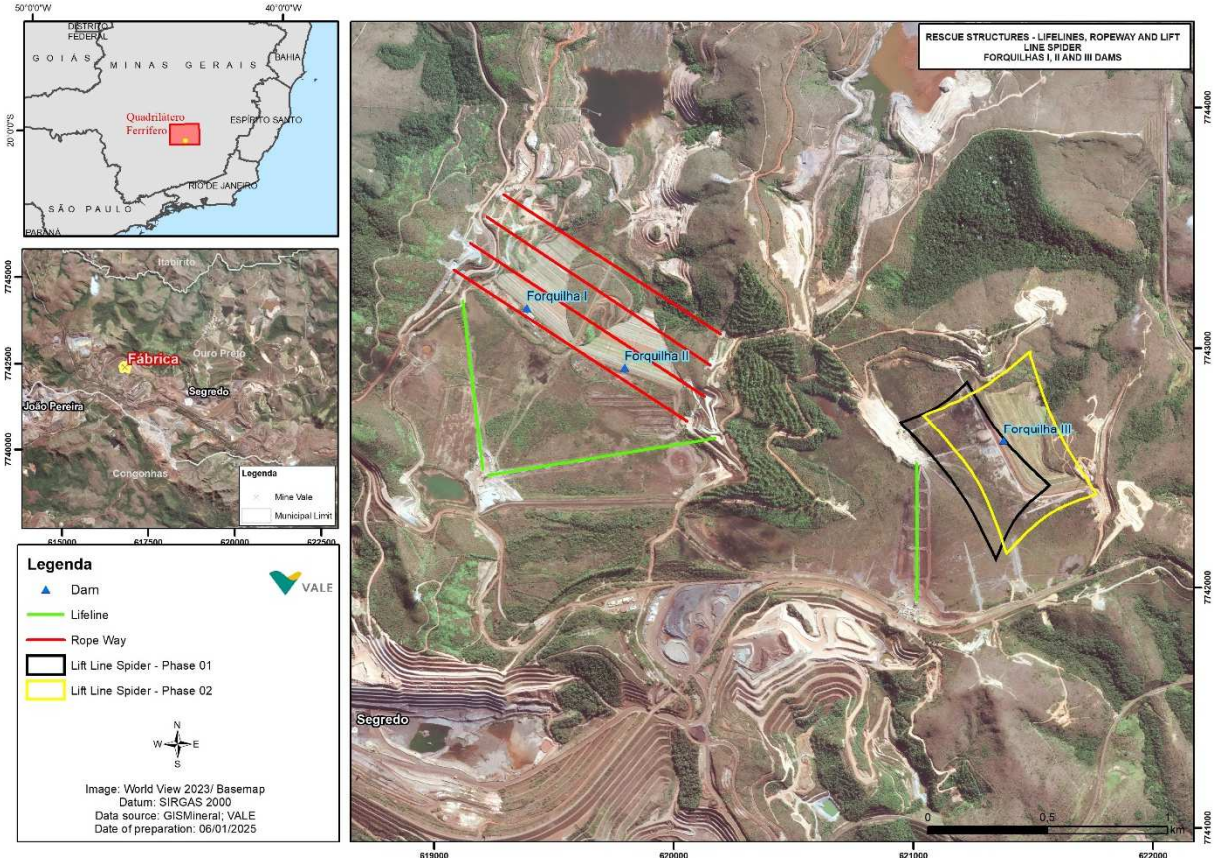


Figure 1 - Location of the dams that received the rescue structures, with emphasis on their positioning in relation to the Iron Quadrangle.

It's important to mention that Vale's planning to construct another LLS in Sul Superior dam, located in Barão de Cocais, even in Minas Gerais.

The points common to the three rescue structures include the way of entering the dam in question – in all three, entry is done by conventional walking, not requiring any type of prior lifting, as well as the need for external supervision during the operation – a individual is positioned external to the dam during all operational activity, and all devices are based on anchorages in steel cables attached to towers external to the structures. Furthermore, the three works were implemented in dams located in the Corredor Sul (South Corridor).

3. BRAZILIAN OVERVIEW OF DAMS

3.1. Regulatory System

2015 and 2019 were landmark years for Brazilian mining, especially for Vale, when it comes to tailings containment dams – in 2015 there was an accident with the Fundão dam, in Mariana, with Samarco, a subsidiary of the Vale/BHP consortium. and in 2019 the accident with the B1 dam in Córrego do Feijão, Brumadinho, both in the state of Minas Gerais. These occurrences motivated profound changes in national legislation, which had already been occurring since 2010 (September of the aforementioned year), when Law 12.334 was enacted (Brazil, 2010a), which then established a national policy for dams based on risk criteria - capacity to reservoir greater than 3Mm³, height of the massif greater than or equal to 15m, measured from the meeting of the foot of the downstream slope with the ground level to the crest of dam crowning, hazardous waste contained in the reservoir, medium or high Associated Potential Damage (DPA), in economic, social, environmental terms or loss of human life and high Risk Category (CRI), at the discretion of the competent supervisory body. Brazilian laws, until now, had an environmental bias. As an example, we can mention Laws 12.488 (BRASIL, 1997a) and 9.798 (BRASIL, 1997b), which aimed to establish the mandatory and constructive characteristics of fish ladders in hydroelectric plants. In 2009, Rio de Janeiro became the first Brazilian state to develop its own dam safety law through Law 5.549 (BRASIL, 2009). Pará follows suit with Law 7.408 (BRASIL, 2010b). With the occurrence of the B1 dam in Brumadinho in 2019, Law 12.334 was amended by Law 14.066 (BRASIL, 2020), prohibiting the use of the construction method and upstream elevations and determining the decharacterization of structures built by this method or any other methods not identified and duly registered in projects or relevant technical documentation.

In addition to the requirements established by federal law, regulatory agencies such as the recently established ANM (National Mining Agency) - arising from the extinct National

Department of Mineral Production, DNPM - and state bodies such as FEAM (State Environmental Foundation - Minas Gerais) reviewed its norms and regulations, implementing the requirement for more frequent submission of declarations of stability conditions (DCE), to be prepared by an independent professional, to certify the safety of dams.

According to the latest Dam Safety Report released by the National Water Agency in 2023, Brazil has 25.943 dams registered in the National Dam Safety Information System (SNISB), of which 5.916 fall under the National Dam Safety Policy (PNSB) established by Laws 12.334 (BRASIL, 2010a) and 14.066. (BRAZIL, 2020). Additionally, 6.162 do not meet the minimum legal criteria, totalling 12.078 dams (47%). The remaining 53% of dams lack sufficient classification information, totalling 13.865 dams. Currently, the mineral sector has 931 dams registered in the systems mentioned above, 471 of which are covered by legislation (ANM, 2024). The emergency levels of mining dams are divided into three groups: Level 1 (identified anomaly), Level 2 (anomaly without established control) and Level 3 (imminent collapse). According to the Mining Dam Safety Report (ANM, 2024), there are still three Level 3 structures in Brazil, all located in the state of Minas Gerais and in the decharacterization phase as they were built using the upstream raising method. The need to de-characterize structures built using the upstream heightening method has caused a significant increase in demand for geotechnical projects and, consequently, for investigations and tests of this nature. In 2024, Vale S.A. carried out almost 114 km of geotechnical drilling with a fleet of 107 probes. 115 km are planned for 2025, with an equivalent amount to be maintained in the coming years – judging by the variation between planning and execution, there is the possibility of executing up to 132 km this year. A significant part of the investigations is carried out with mixed probes, carrying out SPT (Standard Penetration Test) tests associated with rotary drilling with collection of cores for laboratory tests and in-situ infiltration tests to determine permeability coefficients. SPT, although widespread in Brazil given its low cost, quick execution, obtaining information on resistance to soil penetration, great possibility of correlation with other geotechnical parameters and support in the interpretation of laboratory results, presents limitations related to the depth of execution, where the Correlations become less precise due to factors such as the buckling of the rods and the consequent increase in lateral friction on them, causing dispersion of the energy of the blows.

Due to the complexity of the studies required for mining dam projects and even in projects to de-characterize existing structures, other tests were incorporated into the portfolio of geotechnical investigations or had their use expanded, such as the CPTu/SCPTu (Cone Penetration Test with piezometer; Seismic Cone Penetration Test), dilatometers, among others. The additions allow for a more accurate assessment of soil conditions, providing fundamental data for the safety and effectiveness of projects involving dams. Another

advancement was the inclusion of depth samplers for collecting materials from landfills and foundations, such as Shelby, Denison, Dames and More, and Direct Push. All the methods mentioned above involve more than 15 geotechnical companies, project offices, consultants, laboratories, a professional portfolio of more than 2.500 people and technical construction monitoring.

To support such a robust structure and guarantee the continuity of investigations in any dams, including those at higher Emergency Levels, engineering solutions were developed for the entry of conventional equipment and even specific equipment for such activities, remotely operated. Such engineering solutions will be described in the 3.2 item below.

3.2. The Vale's Challenge – How to Investigate Emergency-Level Structures

3.2.1 Lifelines

The lifelines, installed in the Grupo and Forquilhas I, II and III dams, are anchoring structures that are based on the principle of self-rescue. These rescue devices were built based on just two towers, connected by steel cables. Moored to these cables, professionals carrying out geotechnical investigations or in situ tests carry small engines, with a load capacity of up to 100kg each, which are positioned at waist height of each employee and are activated manually and directly by the employee in the event of a hypothetical rupture - Figure *2*.

Figure 2 - An operator with all lifeline equipment, starting a demonstration of lifting process.

These engines, equipped with manual accelerators like those found on motorcycles, when activated, then retract the guide cable, which is in turn attached to a trolley assembly, and this one connected to a lifeline. This recoil occurs at an average speed of 1 m/s. With the height of the towers that support lifelines varying between 5 and 26 meters, the complete lifting of the employee takes place within a period of 26 seconds at maximum. Figure 3 illustrates this instrument scheme (A) and a view of one of the systems (B).

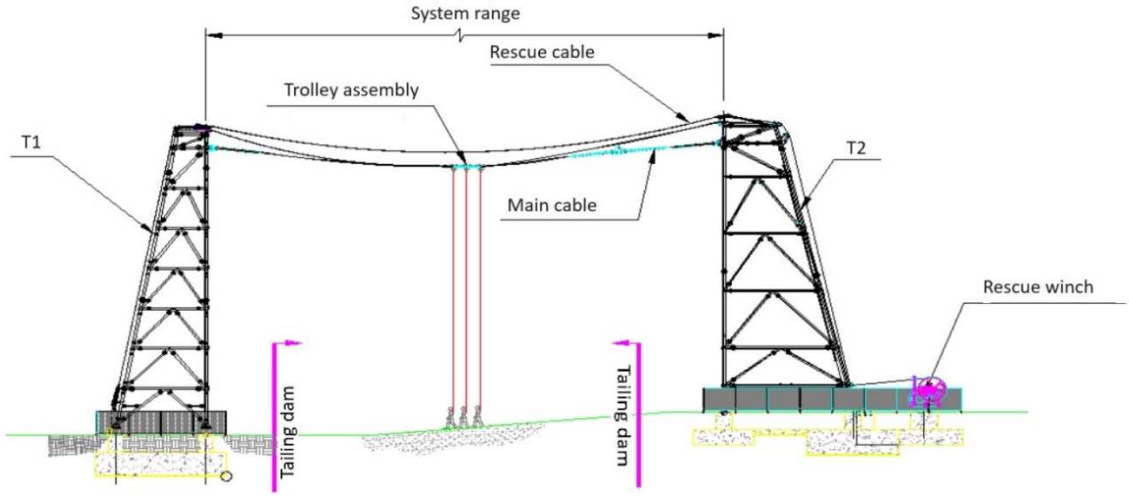


Figure 3 – (A) Lifeline scheme, installed in Forquilhas I, II, III and Grupo dams and (B) a view of the Forquilhas II lifeline.

It’s important to register that this system have a backup structure, in the event of a system malfunction, based on a rescue winch, external to the dam, that could be observed in Figure 3.

The main advantages of this safety device are based on the low installation cost when compared to others, the operational simplicity involved, considering that there are no external controls and that the functions of the lifting equipment are very simple, the low maintenance cost and frequency of these (taking into account the simplicity of the circuit) and the short time required for training the teams involved in the activity.

The negative points of these structures include the geographical rigidity imposed by the geometry generated by simplicity - just two towers interconnected by cables -, the range of operators, limited to the remainder of the cables after connection to the lifelines, the dependence on individual manual activation of each device and the relatively low lifting speed in case of a possible rupture – depending on the location of the operational team in the structure, the engines need to have their lifting speeds changed so that a possible wave of debris does not affect the physical integrity of the operators.

3.2.2 Ropeway

The Ropeway, a structure installed in the Forquilhas I and II dams, is made up of a series of lifelines, forming a reticulum that itself covers a pre-determined area, solving one of the lifeline problems – the geographic rigidity imposed by the geometry. of the solution. Figure 4 illustrates the location of this rescue structure in (A) and an illustrative scheme in (B).

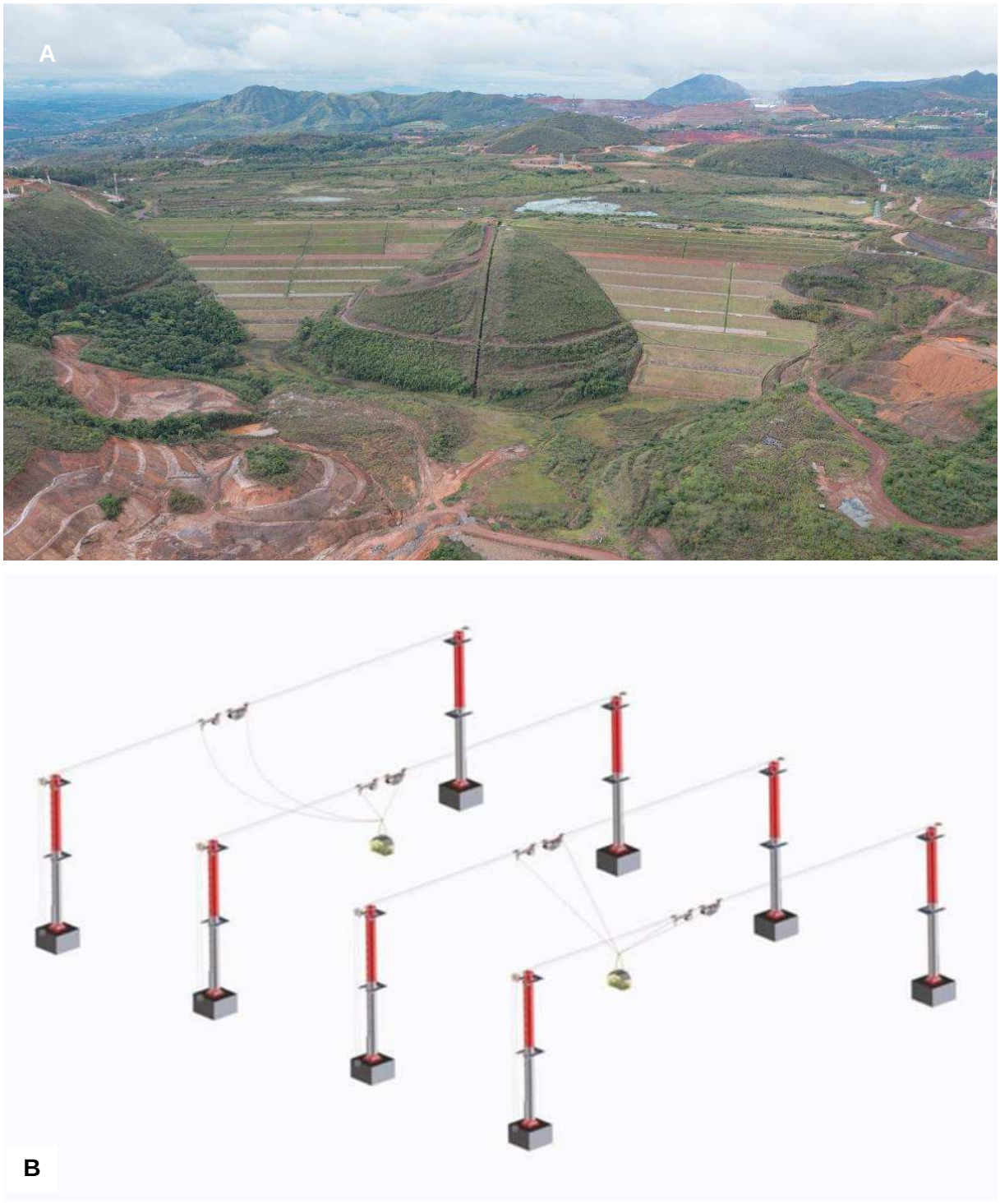


Figure 4 – (a) Ropeway installation over Forquilhas I and II dams and (b) an illustrative scheme of the rescue system.

Employees are also anchored using steel cables connected to a triangular structure called “sky bull”, which allows up to seven people to be anchored simultaneously. This structure moves along the cables attached to the support towers. This movement is carried out by own operators, that use remote controls very similar to those used in lifelines. The device is capable of lift an employee at a speed of 1m/s. With the height of the towers varying between 20 and 40 meters, the complete lifting of the employee takes place within a period of 40 seconds in

the worst case, duying the distance between the operator and the nearest tower. Figure 5 shows the skybull structure and operators preparing to use the rescue system.

Figure 5 - Sky bull structure used on Ropeway system.

The main advantages of Ropeway consist of the simplicity of the equipment, like lifelines, which implies a reduced cost when compared to the LLS solution, the greater areal coverage given the increased possibilities of lines to be covered by operators, solving a problem of lifelines and the fact that operators trained in lifelines can work on the highlighted solution.

The main disadvantages relate to the need for maintenance and inspections greater than the lifelines, given the different complexity caused by the presence of the sky bull and a greater number of towers and cables connected to each other, which, in themselves, demand more time and criteria for inspection.

3.2.3 Lift Line Spider

The Lift Line Spider (LLS), name given to the structure installed in the Forquilhas III dam, in turn, is the most complex of the three works. It was installed in this dam considering that this is the only dam that qualifies as Emergency Level 3 according to current Brazilian legislation, that is, it is the one with the greatest operational restrictions among the entire set of structures present in the Southern Corridor. Figure *6* shows the installation scheme and covered area in (A) and a illustrative configuration of the system in (B).

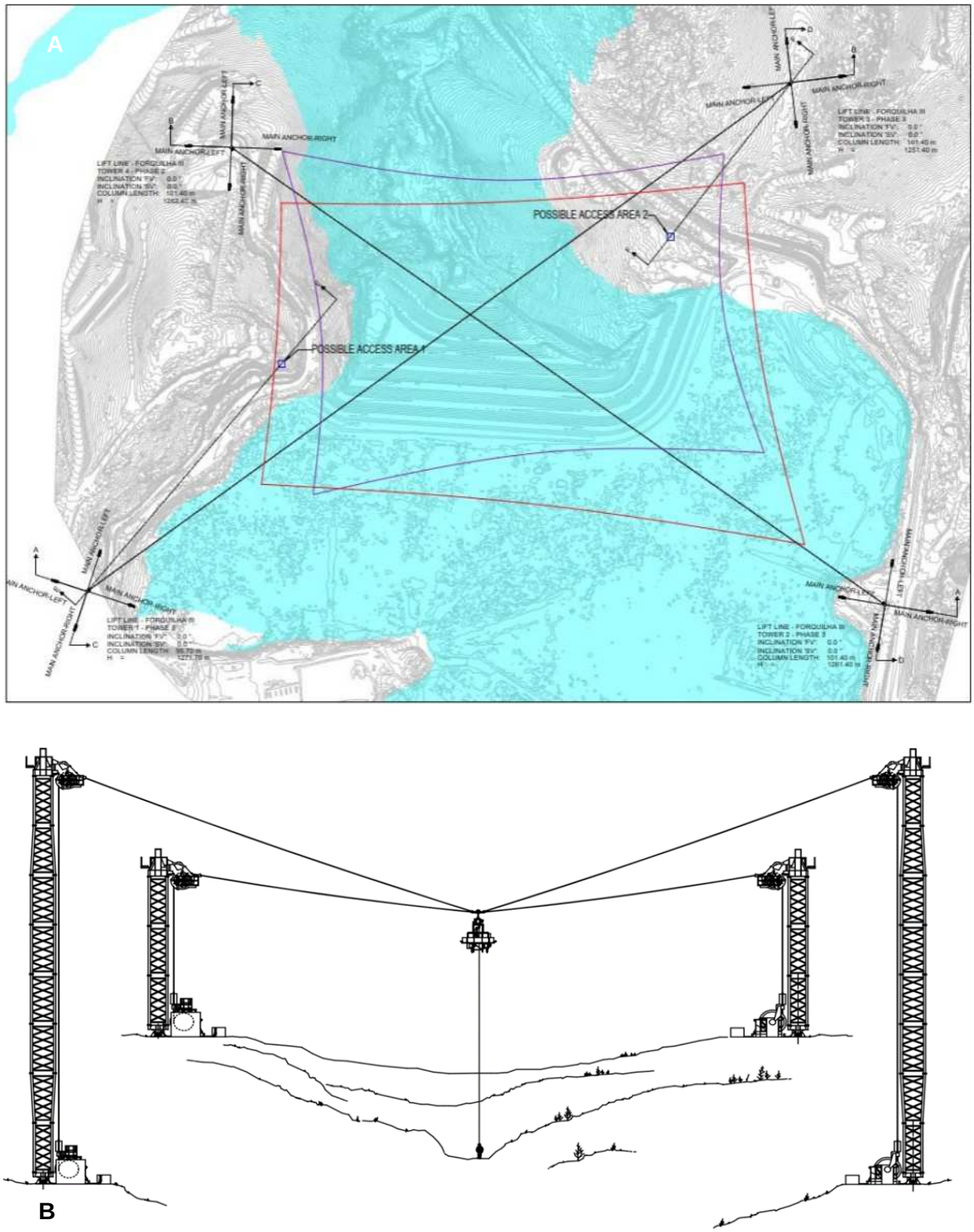


Figure 6 – (A) Installation area covered by the LLS and (B) a scheme showing the configuration of the equipment.

There was a project developed by a manufacturer of trams for access to the European Alps for ski resorts. It consists of four towers interconnected by steel cables arranged in parallel alignments, connected to the movement basket. These cables support a basket on which the equipment operator is positioned. This operator has access to a panel to carry out this

operation and outside the structure there is another control panel, continuously monitored by an employee during routine operations inside the dam. Figure 7 illustrates the basket in (A) and the Forquilhas III dam with the equipment in (B).

Figure 7 - Lift Line Spider implemented in the Forquilhas III dam. (a) The basket used to operate the equipment and (b) the entire system, focusing in towers and their positions over the dam.

Communication between the basket and the external control panel, as well as between the engines positioned in each tower, takes place via radio waves. This fact limits operations that could eventually interfere with the operation of the LLS, such as drone flights, movement of

remotely operated equipment, movement of autonomous machinery, among others. It is important to highlight that Vale carried out an interferometric study prior to the operation of the equipment.

Employees who use this type of system are moored by steel cables to the basket, having to move in line with its operation. It is important to highlight that to maintain adequate tension in the operators' support cable, they carry a backpack with additional weight (25kg) between their bodies and the cable, thus avoiding intertwining between cables from different operators - Figure 8. As the other rescue systems, this device is capable of lift an employee at a speed of 1m/s. In this case, the tower height is not important to the process. In this case, the basket height in relation to the topography is more important.

Figure 8 – An operator carrying all the LLS equipment.

The advantages of LLS include its geographical scope, given that it covers a specific area delimited by the combined range of the cables - as occurs with vertical cameras present in football stadiums and supported by analogous cable systems -, the availability even greater scope with the change of just two towers, covering the entire area of the dam studied and the greater number of controls and processes, which allows some combinations in terms of times and movements previously restricted by the other structures already described.

The main disadvantages relate to the complexity of the equipment, with internal and external operations, independent control panels, a greater quantity of components (cables, towers,

basket, radio signal replicators, among others), which imply higher costs, greater demand for time to carry out more frequent and specialized inspections. Furthermore, as there is additional weight attached to the structure, this can become an aggravating factor in the event of involuntary lifting by operators. These, in turn, need more training and have greater operational knowledge of the equipment they operate, considering that rescue in the event of breakdowns and simple problems can become a complex activity, often requiring the assistance of helicopters. – there is a record of rescue of a CPTu probe using an aircraft of this type due to mechanical problems.

3.2.4 Bonus – Deep Drive and Au-Rig

Deep Drive is the name given to a remotely operated – semi-autonomous – S-CPTu testing probe developed by the company Fugro on demand from Vale for access to emergency Level 3 structures. It consists of a probe equipped with cameras in all directions and a radio communication system for operation carried out from a cabin positioned external to the structure under study. Furthermore, the system allows piezometer-type instruments to be crimped using semi-flexible rods, wound on a specific coil for this purpose. It is important to mention that in Deep Drive the data logger responsible for recording data from the S-CPTu test is integrated into the equipment itself. Figure *9* below illustrates said equipment.

Figure 9 - Deep Drive - semi-automatic S-CPTu probe been transported to start a drillhole. Emphasis on the semi flexible coil used to install piezometers.

The Au-Rig, developed by Conetec, is equipment equivalent to the Deep Drive, that is, a semi-autonomous probe for carrying out S-CPTu type tests, operated remotely and equipped with cameras in all directions. The distinctions between the two focus on the fact that the Au-Rig uses conventional tubes for crimping piezometer-type instruments and that the probe has the S-CPTu test data logger external to the equipment itself. Figure *10* represents the highlighted equipment.

Figure 10 - Au-Rig developed by Conetec.

4. CONCLUSIONS

Occurrences with dams in the Brazilian mineral sector between 2015 and 2019 led to profound legal changes in the sector and consequently in the country. Such changes forced entrepreneurs to study their structures even more and to guarantee, through these studies and with greater frequency, the stability of the structures implemented or managed by them. In this context, large corporations had to seek engineering solutions and even promote solutions to maintain geotechnical investigation work on their structures, especially those that had their classifications classified as Emergency Level 3. In the meantime, Vale, owner of two structures

classified as Level 3, had to seek solutions to refine its projects and continue with the decharacterization of these dams. The implementation of lifelines, Rope Way and Lift Line Spider demonstrate how engineering solutions can and should be implemented to mitigate possible impacts on operational teams in the event of hypothetical ruptures. The promotion of probes such as Deep Drive and the use of technologies such as Au-Rig show the corporation's commitment to best practices and demonstrate the learning obtained after incidents.

REFERENCES

[1] AGÊNCIA NACIONAL DE ÁGUAS E SANEAMENTO BÁSICO (ANA). Brazil Dam Safety Report. Brasília: ANA, 2023. Available at: https://www.snisb.gov.br/portal-snisb/api/file/download/714/4/rsb_2023_2024_06_27_11_01_28.pdf. Accessed on: January, 03, 2025. (ANA, 2024).

[2] AGÊNCIA NACIONAL DE MINERAÇÃO (ANM). Mining Dam Dashboard. Brasília: ANM, 2024. Available at: Mining Dam Dashboard. Accessed on: November 19, 2024. (ANM, 2024).

[3] BRAZIL. Law 12.334, of September 20, 2010. Establishes the National Dam Safety Policy and provides other measures. Official Gazette of the Union, Brasília, DF, September 21, 2010. Available at: http://www.planalto.gov.br/ccivil_03/_Ato2007-2010/2010/Lei/L12334.htm. Accessed on: December 23, 2024. (BRAZIL, 2010a).

[5] BRAZIL. Law 12.488, of August 11, 1997. Amends Law No. 6,938, of August 31, 1981, and provides other measures. Official Gazette of the Union, Brasília, DF, August 12, 1997. Available at: http://www.planalto.gov.br/ccivil_03/leis/L12488.htm. Accessed on: November 20, 2024. (BRAZIL, 1997a)

[6] BRAZIL. Law 9.798, of May 26, 1999. Provides for the National Water Resources Policy and provides other measures. Official Gazette of the Union, Brasília, DF, May 27, 1999. Available at: http://www.planalto.gov.br/ccivil_03/leis/L9798.htm. Accessed on: November 20, 2024. (BRAZIL, 1997b)

[7] BRAZIL. Law 5.549, of November 9, 2009. Provides for dam safety and provides other measures. Official Gazette of the Union, Brasília, DF, November 10, 2009. Available at: http://www.planalto.gov.br/ccivil_03/leis/L5549.htm. Accessed on: November 20, 2024.

[8] BRAZIL. Law 7.408, of August 18, 2010. Amends Law No. 12.334, of September 20, 2010, and provides other measures. Official Gazette of the Union, Brasília, DF, August 19, 2010.

Available at: http://www.planalto.gov.br/ccivil_03/leis/L7408.htm. Accessed on: November 20, 2024. (BRAZIL, 2010b)

[9] BRAZIL. Law 14.066, of September 30, 2020. Amends Law No. 12.334, of September 20, 2010, and Law No. 9.433, of January 8, 1997, and provides other measures. Official Gazette of the Union, Brasília, DF, October 1, 2020. Available at: http://www.planalto.gov.br/ccivil_03/_Ato2020-2022/2020/Lei/L14066.htm. Accessed on: November 20, 2024.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

A MATHEMATICAL WAY TO CLAIM OUR EXPERIENCE: FOLLOWING THE BAYESIAN APPROACH

Jeniffer C.B. Viegas[1]
TPF Consultores de Engenharia e Arquitetura

António X.P. Gallardo
Independent consulting engineer

ABSTRACT

Every day, we encounter the seemingly simple task of defining the parameters of a linear regression that can express soil resistance or pile strength. Regardless of the process the relationship describes, the challenge persists: How do we set these parameters? What seems straightforward can quickly become daunting when each of us rely on our own not fully educated intuition and sometimes biased perceptions, seldomly on real comparable experience. It's impossible to expect everyone to subscribe to the same beliefs; each individual has their own ideals and experiences. However, what we can strive for is translating our knowledge into mathematical reasoning to enhance our parameter estimation, making it understandable and reproducible for all.

The aim of this study is to integrate our valuable experience, or "prior knowledge," into a robust mathematical model by employing a Bayesian framework, also known as the observational method or inverse probability. This approach will enable us to establish a more structured and scientifically rigorous foundation for our insights and expertise. The Bayesian approach is particularly well-suited for estimating soil parameters because often data is sparse and engineering judgment is as crucial as the data itself. Through this approach, probability is interpreted as degrees of belief, and the data is considered fixed while parameters are viewed as random entities.

The ultimate objective of this work is to underscore the significance of old data that often languishes forgotten in office drawers, yet holds the potential to enrich our parameter estimations through prior knowledge. We must acknowledge that our beliefs and experiences are our own. If we wish to share them with others, we must substantiate our beliefs with tangible evidence to validate them.

Keywords: Bayesian Methods, Soil Characterization, Parameter Estimation, Tailings Dams.

[1]jeniffer.viegas@tpf.pt, Portugal

1. INTRODUCTION

This paper explores the application of Bayesian analysis in geotechnical studies, addressing the limitations of traditional frequentist statistics when dealing with sparse datasets common in geotechnical engineering. It emphasizes the role of human judgment and experience in parameter estimation, a challenge often overlooked by frequentist approaches. The Bayesian framework allows for the integration of prior knowledge and experience, offering a more flexible and realistic methodology.

The study employs Bayesian inference for soil property estimation using a dataset of 64 undisturbed samples subjected to triaxial tests. This extensive dataset, consisting of 265 data points (64 samples), offers the advantage of providing comprehensive statistical insights, including deterministic measures such as the mean. The thorough understanding of our data, facilitated by the dataset's size, empowers us to explore methodologies and validate our approach effectively.

The methodology includes the use of Markov chain Monte Carlo (MCMC) for posterior distribution sampling, considering prior knowledge and new data. The model parameters, including slope, intercept, and standard deviation, are viewed as probability distributions.

Prior predictive checks and convergence analyses ensure the model's reliability. The study presents a comprehensive evaluation of model metrics and discusses the impact of sample size on predictive accuracy. The methodology is applied to simulate out-of-sample predictions for different scenarios, emphasizing the significance of prior knowledge in Bayesian analysis.

The investigation concludes by proposing a practical approach for incorporating new samples into the Bayesian framework, addressing the challenge of weighting new data relative to documented knowledge. A case study illustrates the methodology's application, showcasing its potential to avoid unrealistic or overly pessimistic soil parameter estimations.

The presented Bayesian approach provides a robust and flexible methodology for geotechnical studies, encouraging a shift from rigid frequentist methods to a more intuitive and informative framework. The findings contribute to the advancement of Bayesian techniques in geotechnical engineering, offering a valuable tool for soil property estimation and decision-making processes.

2. BAYESIAN APPROACH

Before delving into the specifics of the approach utilized in this study, let's first step back and discuss the significance of Bayesian analysis in the context of geotechnical studies. Traditional frequentist statistics, which we are typically taught at university, focus on concepts like confidence intervals and p-values. However, these statistics are most appropriate for large datasets, such as those found in clinical trials or political polls, where thousands of data points are available. Is this the case for geotechnical data? It's doubtful. Typically, geotechnical data tends to be sparse, whether it's from CPT, triaxial tests, SPT, or any other source. We are fortunate if we have a few spatially distributed data points.

Nonetheless, this isn't the only challenge. In geotechnical engineering, human judgment and experience play a significant role in parameter estimation. The frequentist approach offers limited room for incorporating this valuable knowledge, whereas the observational or Bayesian approach provides a more accommodating framework for its integration. As geotechnical engineers, we know what we possess: data collected from in-situ or laboratory tests, and we know what we seek – to infer the probabilities of specific soil parameters in the natural environment based on this point-wise collected data. This is precisely where inverse probability comes into play. The main difference between frequentist and Bayesian approaches lies in where uncertainty is accounted for. In the frequentist approach, uncertainty is considered within the data itself. It calculates the probability of obtaining the data, assuming the hypothesis, parameters, or models are true. On the other hand, in the Bayesian approach, uncertainty is attributed to the parameters and the model itself. Here, we calculate the probabilities of our hypothesis, parameters, or model given the data we observed. [1].

Bayesian reasoning is a natural extension of our intuition. Often, we begin with an initial hypothesis, and as we collect data that either supports or refutes our ideas, we adjust (or update) our model, we don't require an abundance of data to think in a Bayesian manner.

In Bayesian inference, the core concept revolves around redistributing credibility across possibilities as the problem evolves. The reallocated distribution of credibility is termed the posterior distribution because it represents what we believe after taking new observations into account. The distribution of credibility before observing any data is referred to as the prior distribution, reflecting our beliefs about the data before sampling it. The prior distribution can be weak (non-informative) if we lack information about the data distribution, or we can use more informative priors when we possess background experience or relevant literature to support our beliefs. The third component in Bayesian inference is the likelihood. The likelihood connects the observed data with the unknown parameters and quantifies the probability of observing the data we have, given a specific value for the parameter we are trying to estimate. While the likelihood is driven by the observed data, the prior distribution encapsulates our "prior

beliefs" about the parameter. Summarizing the explanation provided in the previous paragraph, we arrive at Bayes' rule:

$$p(\theta|D) = p(D|\theta)\, p(\theta) \,/\, p(D) \tag{1}$$

where,

$p(\theta|D)$ is the posterior distribution.

$p(D|\theta)$ is the likelihood function.

$p(\theta)$ is the prior distribution.

$p(D) = \sum_{\theta^*} p(D|\theta^*)p(\theta^*)$ is the evidence.

θ is the parameter.

D is the data.

Equation 1 says that the weight of the evidence is controlled by the likelihood function, and this function can be calculated even if we have sparse data, which is the base for learning by experience.

3. INFERENCE OF SOIL PROPRETIES

3.1. Methodology

The present study makes use of a dataset composed of 64 undisturbed samples collected to perform triaxial tests. The material under study is a fill material classified as lean silt (ML), derived from the weathering horizons of Dolomitic Phyllites from the same rock formation. Generally, the soils were placed in fills with a compaction degree mostly exceeding 97% and with water content deviations of ±2% from optimum (Standard Proctor). The samples subjected to CIUC triaxial tests were collected, either in block or using a Shelby sampler, between 2020 and 2022, from the downstream berm of two neighbouring large dams, both with homogeneous profiles, one under construction and the other with fills ranging in age from 7 to 16 years. The material is commonly known as Silty Fill. Each sample was tested at least for three different stresses, resulting in 265 points. The data was processed following the ASTM D 4767 – 95 [2], where the stress path parameters are given by:

$$\begin{cases} s' = \frac{\sigma'_1 + \sigma'_3}{2} \\ t' = \frac{\sigma'_1 - \sigma'_3}{2} \end{cases} \tag{2}$$

With the data in the s'-t space we can define a linear regression,

$$Y = A + BX \leftrightarrow t = t_0 + \tan\theta \cdot s' \quad (3)$$

From which we can obtained the soil resistance parameters,

$$\varphi' = asin(tan\theta) \quad (4)$$

$$c' = t'_0 \cdot sec\varphi' \quad (5)$$

When employing a frequentist approach, we seek to minimize the residual sum of squares, that is, we minimize the distance between the observed data and the predicted data. This method, known as Ordinary Least Squares (OLS), yields a single estimate for the model parameters: the intercept and the slope.

In contrast, the Bayesian approach involves formulating linear regression using probability distributions rather than singular point estimates, resulting in a distribution for the model parameters. The feasibility of Bayesian inference has been significantly enhanced by advancements in computing and algorithms, allowing the evaluation of the posterior distribution for complex models without the need to compute integrals (particularly when dealing with continuous data).

The methodology employed in this study entails the random sampling of numerous representative combinations of parameter values from the posterior distribution. This approach is commonly known as the Markov chain Monte Carlo (MCMC) method [3]. When performing linear regression fit in a Bayesian way, we approximate our data to a normal distribution where the mean is conditionally modelled as a combination of other variables:

$$\mu = Ax + B \quad (6)$$

$$Y \sim N(\mu, \sigma) \quad (7)$$

To carry out this Bayesian regression, we need to specify the priors for the model parameters, which, in this case, include the slope and intercept. Additionally, we need to define the prior for the standard deviation of Y (σ) and the likelihood function. Determining these components can be the most challenging aspect of the work, as it requires a good understanding of the data. However, it's precisely why we opt for a Bayesian framework in the first place.

We began our analysis by exploring our data to gain insights into its distribution. Figure 1 presents the raw data alongside the s'-t plot and the Kernel Density Estimation (KDE) for t and s'.

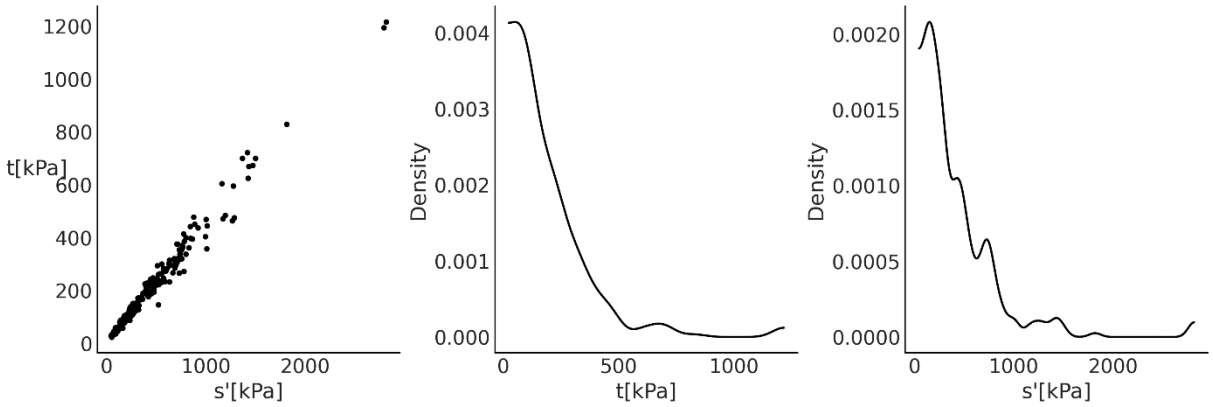


Fig. 1 – Observed data. A scatter plot representing the observed data in the s'-t space, accompanied by Kernel Density Estimates (KDE) for both t and s'.

As we need to establish the priors for both the slope and intercept, we also examine their distributions by calculating these parameters using a frequentist approach for each sample:

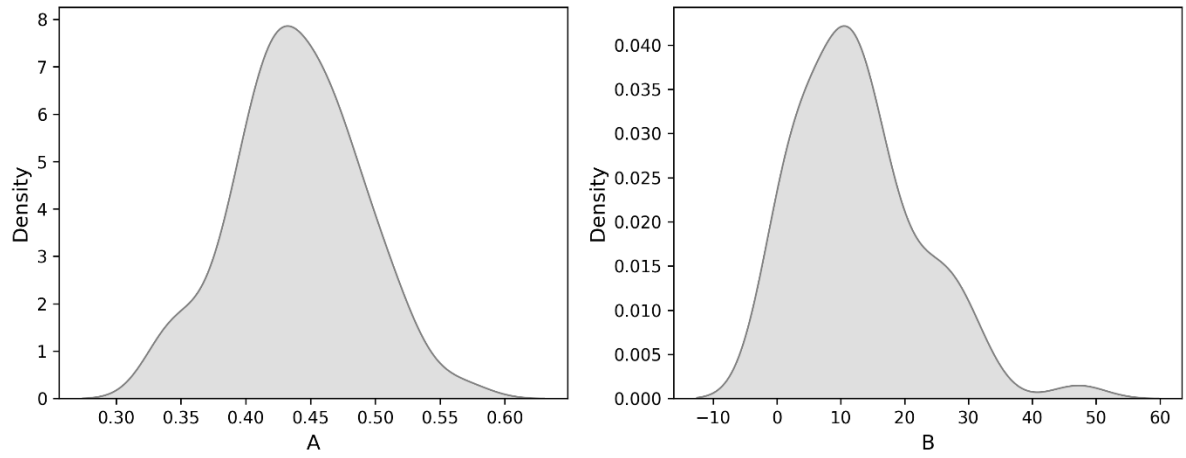


Fig. 2 – Regression Parameters. Kernel Density Estimates (KDE) for the slope and intercept parameters from the linear regression equation, t = As' + B.

In this particular scenario, our prior knowledge is notably strong due to the substantial volume of samples. As noted in previous unpublished studies from the authors, with approximately 48 samples, the mean becomes practically known, resulting in reduced uncertainty where the characteristic value [4; 5] aligns with the mean value. This aspect fortifies the robustness of our study, where we know from the beginning what to expect.

We did take a bit of a shortcut when exploring the data, and this might influence our "prior knowledge". However, for the purposes of this discussion, let's assume that we have prior

knowledge from literature reviews or experience indicating that these data typically adhere to a log-normal or truncated-normal distribution.

Once we have a good understanding of our data's distribution, we proceed by partitioning the dataset into two distinct sets:

- The first set represents our prior knowledge, comprising samples we already possess, potentially obtained from previous projects or other campaigns.
- The second set consists of new data, specifically collected for the current project, from which we aim to derive properties.

This exercise closely resembles our daily activities in projects that involve substantial volumes of data collected from various contractors, often overlooked when working with new data. In this context, we have chosen to retain 6 samples as new data (26 points) and 58 as old data (239 points). Subsequently, we developed a Bayesian framework where we assume the actual *Y* variable (derived from triaxial tests denoted as '*t*') is randomly distributed around the predicted value, denoted as $\hat{y}$, with a standard deviation represented as σ, setting the likelihood to be a Truncated Normal distribution, since the stress applied (*t*) cannot be negative. We can observe the complete model in Figure 3. The choice to utilize the half Student's t-distribution for the parameter B, as opposed to a log-normal distribution, is driven by the prolonged tail exhibited in the latter distribution and the occurrence of outliers in the intercept parameter. The elongated tail in a log-normal distribution tends to bias the mean distribution towards extreme values, making the student's t-distribution a more suitable and robust alternative, especially in the presence of outliers.

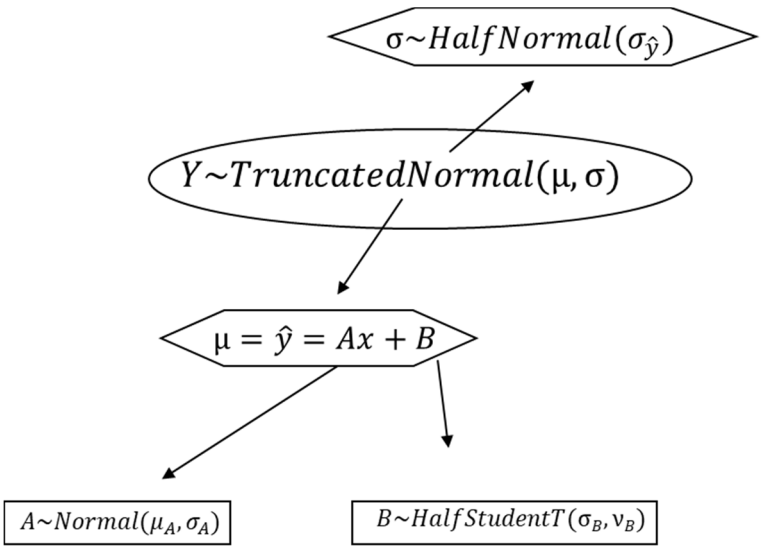


Fig. 3 – Model Workflow. Explanation of Model Parameters: The intercept parameter follows an approximate half t-student distribution, the slope adheres to a normal distribution, the

standard deviation aligns with a half normal distribution, and the likelihood is represented by a truncated normal distribution.

We implemented our approach by running four chains, each comprising 3500 draws. In addition, we conducted 2000 draws for each chain to fine-tune method parameters like step size and scaling, which were subsequently discarded. Upon careful examination of our prior beliefs and conducting prior predictive sampling, Figure 4 provides clear evidence of the alignment of our prior distribution with the intended shape.

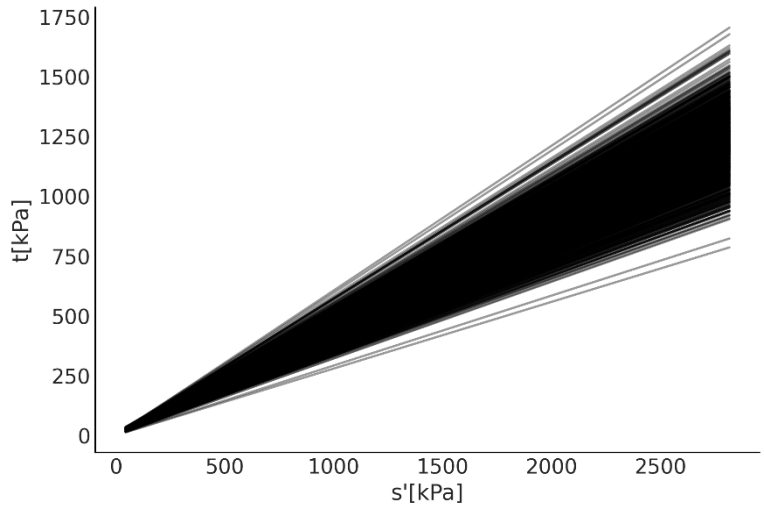


Fig. 4 – Prior predictive checks. The figure presents 400 realizations sampled from the prior distribution, which follows the expected relationship between s' and t.

By examining the cumulative distribution function (CDF) displayed in Figure 5 a) for the prior and 5 b) for the posterior distributions respectively, we validate the consistency of our model with the observed data. However, in the case of the prior distribution, there is a slight tendency to underestimate the mean as illustrated in Figure 5 a). Figure 6 shows the observed and mean predicted values.

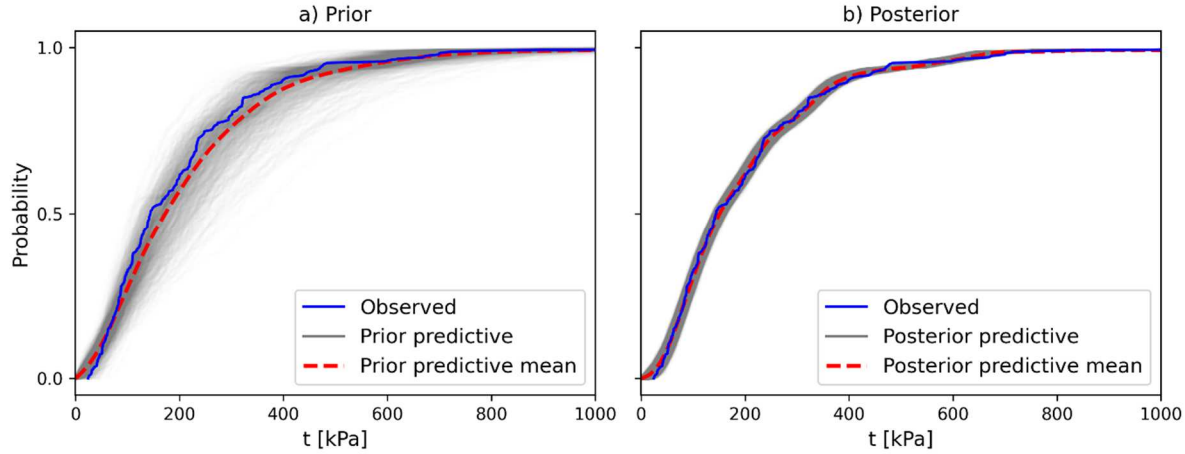


Fig. 5 – Cumulative Distribution. A comparison of the cumulative distribution between the prior and posterior distributions in relation to the observed data.

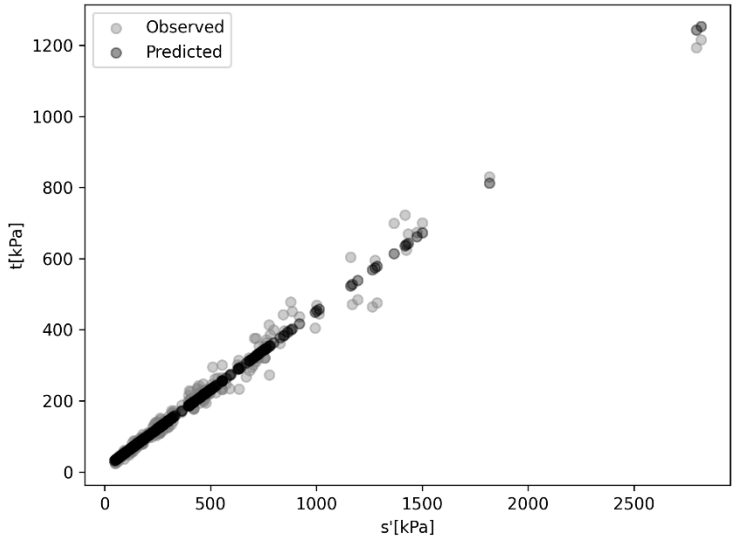


Fig. 6 – Predicted x Observed data. Scatter plot of the observed and the mean predicted data.

All models will yield results, and it is our responsibility to ensure that these results are both meaningful and consistent with the data. After assessing the agreement between the observer data and the prior and posterior distributions, we proceed to check the model's convergence. It is extremely important to ensure that the chains are mixing well and that the effective sample size (ESS) is large enough to obtain a stable estimate of uncertainty [6].

Convergence was visually analysed by plotting the histogram of the ranked posterior draws, as depicted in Figure 7. From the figure, it is evident that the draws are uniformly distributed in each chain, indicating a robust mixing of the chains.

The effective sample size (ESS), which accounts for autocorrelation within the chains, is substantial approaching the total number of draws. This characteristic is consistent both in the bulk and the tail of the distribution of the model parameters, as illustrated in Figure 8. This provides evidence that the sample size is ample for obtaining a stable estimate of uncertainty.

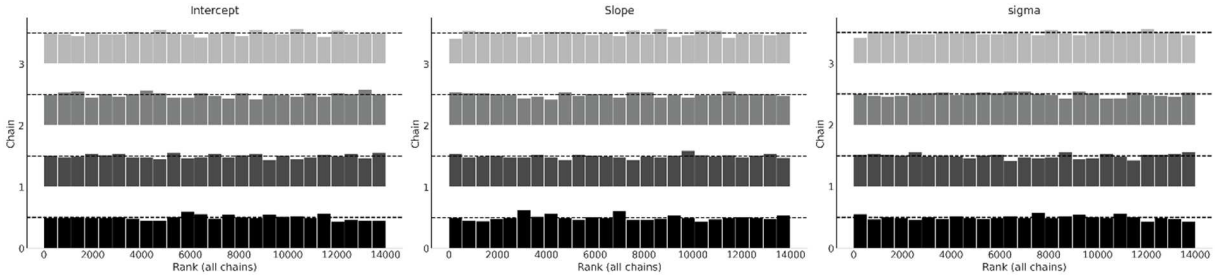


Fig. 7 – Rank Plot. Histograms depicting the ranked posterior draws, presented individually for each chain. Uniform distributions across all chains indicate successful mixing.

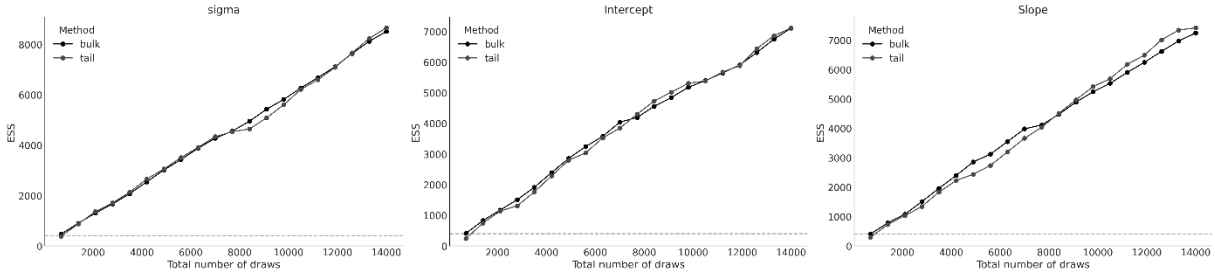


Fig. 8 – Effective sample size (ESS) Evolution. This chart illustrates the progression of the Effective Sample Size (ESS) as the number of draws increases. The upward trend demonstrates a positive correlation between the number of draws and the ESS, indicating enhanced sampling efficiency.

Figure 9 depicts the sampled parameter values (intercept, slope, sigma) across the steps in the chain, commonly referred to as a trace plot. It is evident that all four chains exhibit good mixing, yielding consistent results. The model summary, detailed in Table 1, reveals noteworthy statistics. Notably, the Gelman-Rubin statistic ($\hat{R}$) attains a value of 1 for all parameters, underscoring the convergence and reliability of the model. Additionally, the table provides the mean and standard deviation for each parameter. A succinct summary of the metrics is conveniently located below the table for quick reference. Readers are encouraged to explore Vehtari et. al,2021 [6] for a comprehensive description of these metrics.

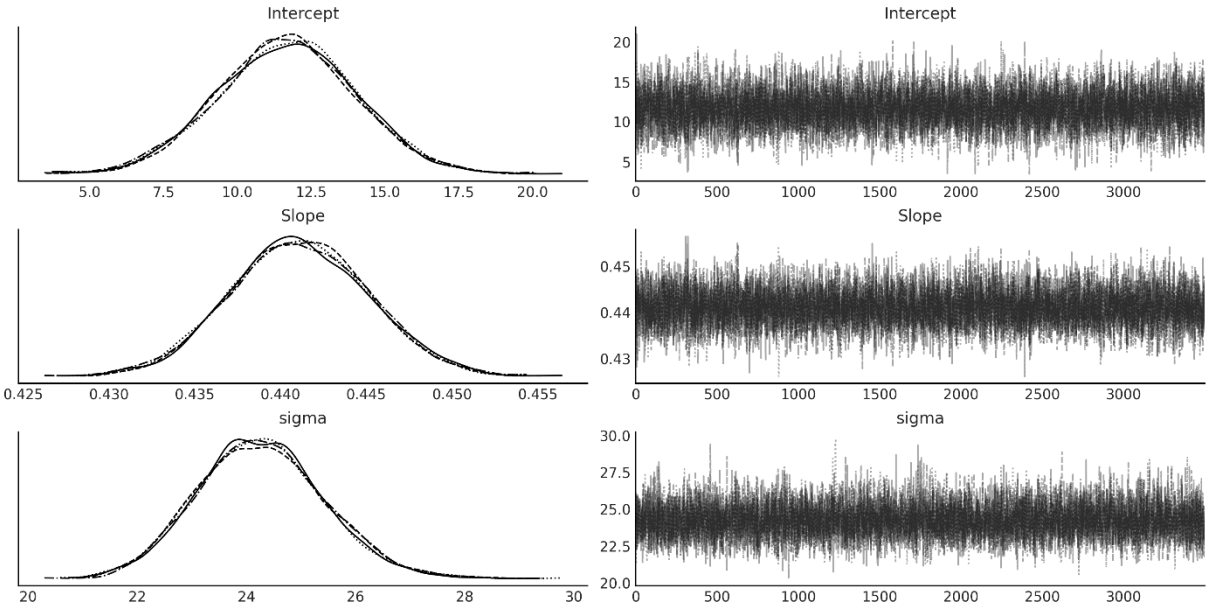


Fig. 9 – Trace Plot. The left side displays Kernel Density Estimates (KDE) for each parameter with values on the x-axis. On the right, individual sampled values at each step during the sampling process are depicted. Notably, all four chains converge towards the same values, indicating a well-mixed and convergent behaviour.

Table 1 – Model Metrics. Metrics applied to evaluate the model.

	μ	σ	$hdi_{3\%}$	$hdi_{97\%}$	$mcse_{\mu}$	$mcse_{\sigma}$	ESS_{bulk}	ESS_{tail}	$\hat{R}$
Slope	0.441	0.004	0.433	0.448	0.000	0.000	6401.0	7957.0	1.0
Intercept	11.712	2.343	7.204	16.007	0.029	0.021	6312.0	7000.0	1.0
sigma	24.226	1.135	22.077	26.307	0.012	0.009	8431.0	8357.0	1.0

Where:

μ	Mean: The average value of the parameter across the samples.
σ	Standard Deviation: A measure of the amount of variation or dispersion in the parameter values.
$hdi_{3\%}$ $hdi_{97\%}$	High Density Interval: The range of values containing the specified percentage of the highest posterior density, representing a credible interval.
$mcse_{\mu}$	Monte Carlo Standard Erro Mean: An estimate of the standard error of the mean for the parameter.
$mcse_{\sigma}$	Monte Carlo Standard Erro Standard Deviation
ESS_{bulk}	The effective sample size for the bulk of the distribution, reflecting the quality of the samples.
ESS_{tail}	The effective sample size for the tail of the distribution, indicating the quality of samples in the tails.
$\hat{R}$	Gelman-Rubin statistic: A convergence diagnostic indicating whether multiple chains have converged to the same distribution. A value close to 1 suggests convergence.

Figure 10 illustrates the distribution of each parameter in our model. The mean values from the Bayesian approach are denoted in black at the top, while those obtained from a frequentist approach for the same dataset are displayed in orange. The 95% highest density interval (HDI) is also displayed in bold, at the bottom. The HDI represents the narrowest interval containing the specified percentage of the posterior distribution, in this case 95%. Unlike confidence intervals, HDIs provide the most credible range of values for a parameter and directly reflect uncertainty about the parameters value.

Given the large dataset, the frequentist estimation closely approximates the Bayesian one, with the frequentist approach being approximately 6% higher than the Bayesian for the intercept parameter and 0.18% lower for the slope parameter.

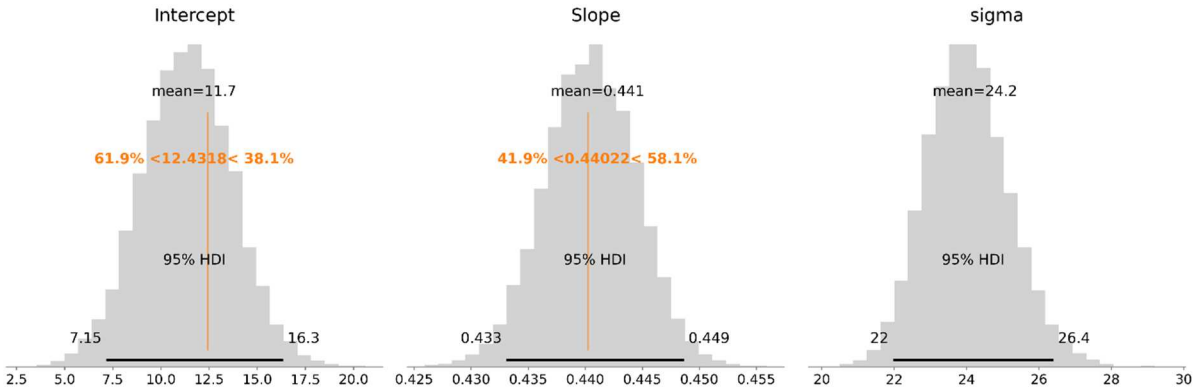


Fig. 10 – Posterior distribution densities. This figure depicts the probability density functions of the posterior distributions for each parameter. Additionally, it highlights the mean value of the Bayesian approach, in black at the top, the High Density Interval (HDI) in bold at the bottom, and in orange the estimated parameter when applying a frequentist approach.

Figure 11 presents the regression results for both approaches, with the 95% confidence interval (CI) included in the frequentist approach, depicted in a). Confidence intervals (CIs) offer a range of values above and below the sample mean, providing reliable estimates for unknown population parameters. The width of a CI, determined by the sample size, its dispersion and chosen confidence level, reflects the estimation precision. CI are constructed using statistical techniques that often rely on assumptions such as normality of the sampling distribution, that sometimes is not verified.

Despite their common use, CIs are frequently misunderstood. They represent the confidence level associated with parameter estimates but are specific to the sample from which they are derived. The confidence level holds significance only in repeated sampling scenarios. For instance, when constructing a 95% CI for the mean of a parameter, the true parameter value may or may not fall within that interval, making it impossible to ascertain the situation for any individual CI. A 95% confidence level implies that if sampling were repeated numerous times, 95% of those sample sets would yield CIs containing the true parameter value. However, since the true value is an unknown fixed parameter in the frequentist framework, identifying sample sets containing the true value remains elusive, and uncertainty regarding the true parameter value persists. It's essential to remember that in real-life scenarios, we typically have only one dataset, and there is no inherent probability associated with the inclusion of the true mean in the interval; the interval either includes it or does not [1].

In Figure 11 b) the regression from the Bayesian approach is presented, as well as the draws from the posterior distribution (light gray lines). Despite looking similar, this gray area has a

different meaning from the one shown in a), since each line represents a different estimate of the model parameters, sampled from the posterior distribution.

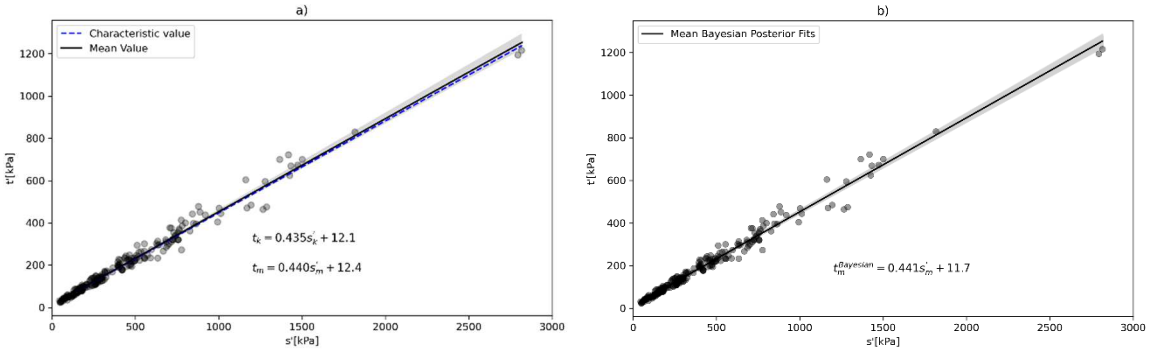


Fig. 11 – Frequentist x Bayesian Inference. In a) the regression parameters estimated with a frequentist approach. In b) the same data fitted by the Bayesian approach.

3.2. Effect of the sample size

In this section, we simulate our daily activities by drawing samples from our historical data 'bag.' This historical data, acquired from the owner or past projects, serves as our prior knowledge and has been employed in earlier sections to calibrate the model. Now, we simulate three scenarios, for out-of-sample predictions:

A) We have collected 6 new samples.

B) We have collected 4 new samples.

C) We have collected only 2 new samples.

Soil properties aren't inherently random; they possess specific characteristics. The problem is that we may lack knowledge about these characteristics, leading us to make inferences. In this example we apply our previous model to infer the soil parameters of new samples.

Figure 12 presents a comparative analysis among the observed data (depicted as black circles), Bayesian posterior fits (gray circles), the mean Bayesian posterior fit (blue line), and the frequentist results represented by the mean value (continuous black line) and the characteristic value (dashed black line), across three distinct scenarios: a) with six samples, b) with four samples, and c) with two samples. Simultaneously, Figure 13 provides a Kernel Density Estimate (KDE) comparison for the examples given in Figure 12. Larger sample sizes correlate with predicted data distributions more closely aligning with the observed data, as illustrated in Figure 13a).

In the Bayesian approach, the new data (6, 4, or 2 samples) were utilized to make out-of-sample predictions using the previously developed model. This results in a distribution for each

prediction (y) given an independent variable (x), as shown in Figure 14. What is plotted in Figure 12 is the mean value of the distribution for each observation.

We know from Figure 11 that the values of the slope and intercept for this dataset should fall between [0.440 – 0.441] and [12.4 -11.7], respectively, in the format of [frequentist – Bayesian] result. A closer examination of Figure 12 reveals that Bayesian inference is less affected by outliers. With two samples (Figure 12 c)), the Bayesian inference deviates from the mean value shown in Figure 11 b) by 4.2% for the intercept and 0.2% for the slope parameter. In contrast, the frequentist deviation for the same parameters, when compared with Figure 11 a), are 107.2% and 28.6%, respectively.

The significant deviation observed in the frequentist approach underscores the considerable penalty incurred in parameter estimation when working with limited samples and disregarding prior knowledge. This highlights the advantages of employing a Bayesian approach.

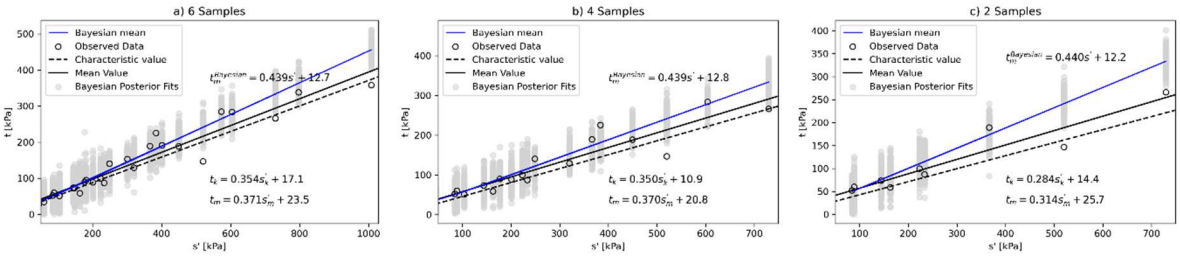


Fig. 12 – Sample size effect. Simulation of out-of-sample predictions for a) 6 new samples, b) 4 new samples and c) 2 new samples, respectively.

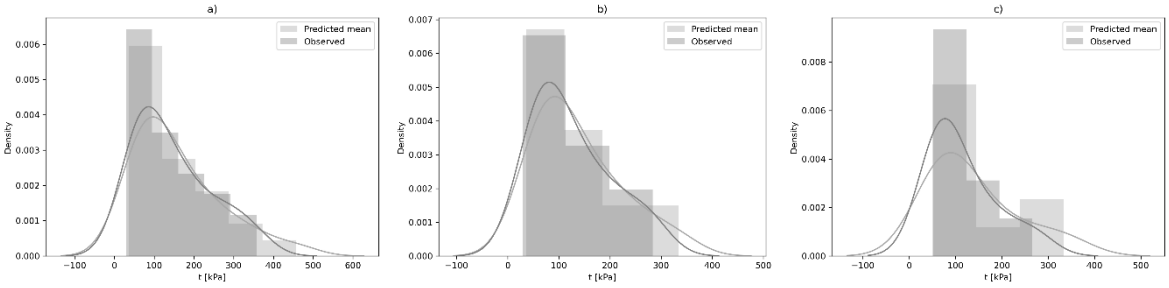


Fig. 13 – Sample size effect. Kernel Density Estimates (KDE) of the observed and predicted data for the three simulated situations: a) 6 new samples, b) 4 new samples and c) 2 new samples, respectively.

Table 2 outlines the metrics employed to evaluate the impact of sample size on out-of-sample predictions using the Bayesian model. The coefficient of determination, R^2, exceeds 0.8 with 4 or 6 samples available. However, with only 2 samples, this value decreases significantly by approximately 20%.

Table 2 – Sample size effect - Metrics. Metrics used to evaluate the sample size effect and the approximation of the predicted data to the observed data.

	N° Samples		
	6	4	2
R^2	0,87	0,81	0,64
MAE	21,61	21,90	28,14
RMSE	33,86	32,51	41,31

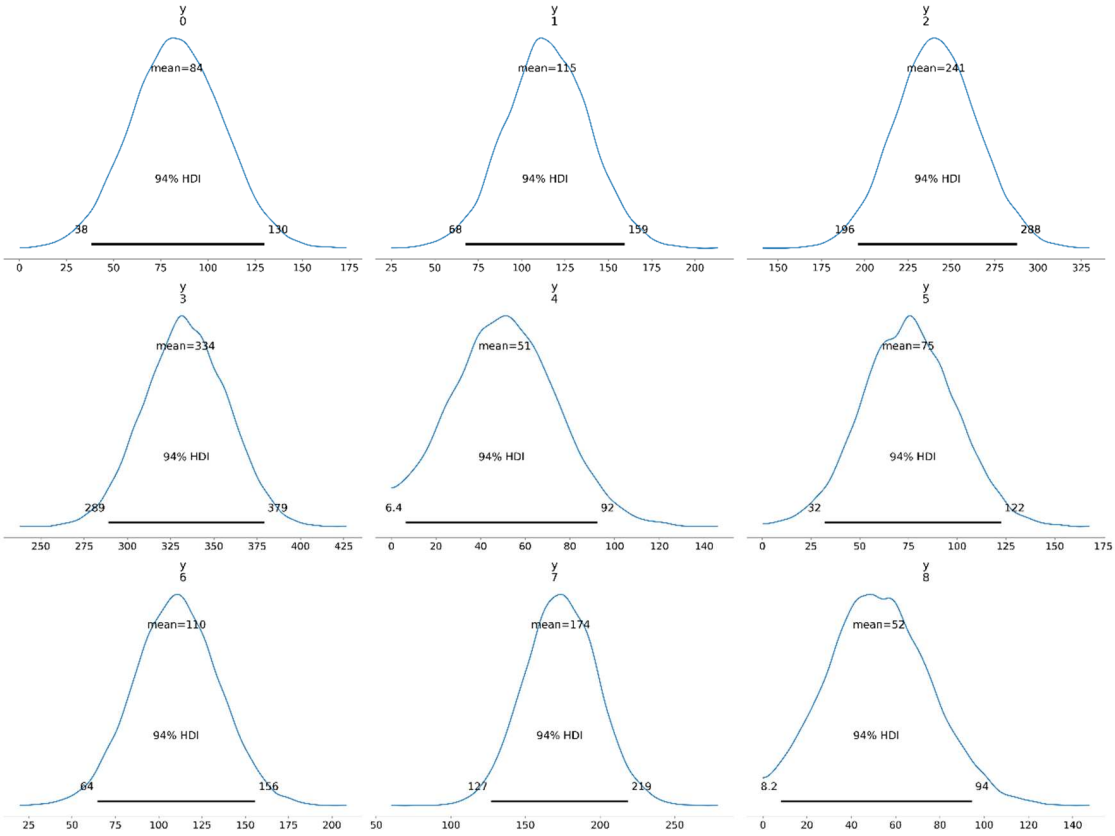


Fig. 14 – Out-of-sample prediction. Prediction for the 2 samples example.

3.3. Weighting approach

Given that the entire study, involving the application of MCMC and Bayesian inference, is rendered useless unless easily reproducible, we proceed with the rationale of regarding the work developed here as 'documented prior knowledge' in this tropical residual soil of dolomitic phyllite. We then devise a method for weighting the new samples with the current study. This approach, although simple in theory, could be practical and effective in practice, as it would help avoid unrealistic or overly pessimistic soil parameters.

Suppose we take case b) from the previous example, where we have only four samples. If we were to disregard all other information and solely rely on these two samples, we would, in a frequentist approach get the mean values of 20.8 and 0.370 for the intercept and slope parameters, respectively.

If, instead, we consider the standard error of the observed data as a ratio to weigh the importance to assign to the new data relative to the documented data, we can express this weight as follows:

$$\mathrm{w} = \frac{\mathrm{S}^2_{\mathrm{t,prior}}}{\mathrm{S}^2_{\mathrm{t,sample}}} \cdot \frac{\sqrt{\mathrm{N_{sample}}}}{\sqrt{\mathrm{N_{prior}}}} \tag{8}$$

Where, $S^2_{t,prior}$ and $S^2_{t,sample}$ are the standard deviation of the prior knowledge and at the new data, respectively.
If the standard error of the prior data, is lower than the standard error of the collected data than we could set:

$$\begin{cases} w_{prior} = 1 - w \\ w_{sample} = w \end{cases} \tag{9}$$

Instead, if the standard error of the collected data is lower than the standard error of the prior data, we would give a lower weight to the prior knowledge, this could be accomplished by setting that, in this case the w_{prior} = 10% for example.

The final regression parameters would be:

$$\begin{cases} A_w = w_{prior} \cdot A_{model} + w_{sample} \cdot A_{sample} \\ B_w = w_{prior} \cdot B_{model} + w_{sample} \cdot B_{sample} \end{cases} \tag{10}$$

Testing the weighting approach involved generating 15 random subsets, each comprising 4 samples (16 points) as illustrated in Figure 15. The aim was to observe how the weighting factor influences the estimated parameters. In Figure 15 a), focusing on the slope parameter, it's evident that the weighting factor endeavours to align the estimated parameters closer to the mean value. For instance, in subset 5, after the application of the weighting approach, the weighted slope parameter (A_w, light gray) is notably nearer to the mean (red line) compared to the non-weighted slope parameter (A, dark gray). Similar behaviour is observed with the intercept parameter. Notably, subset 5 demonstrates the most pronounced effect of weighting. In this instance, the regression of those samples yielded a negative intercept of cohesion. However, in the absence of weighting, it remained constrained to null. The weighted approach, conversely, yields a higher value, attempting to align with the mean intercept value.

Returning to our example, the estimated standard deviation for prior data is 174,8 kPa. In contrast, the standard deviation for the collected data, based on four samples, is 76.7 kPa, with $N_{sample} = 18$ and $N_{prior} = 239$, yielding a weighting factor (w) of 0.63. We have that the mean parameters from the model are: $A_{model} = 0.441$ and $B_{model} = 11.7$ as displayed in Figure

10, while for the frequentist approach we have $A_{sample} = 0.370$ and $B_{sample} = 20.8$, as depicted in Figure 12 b).

Applying equation 3.9, we obtain weighted parameters of $A_w = 0.385$ and $B_w = 17{,}39$. This results in an increase of 4,2% in the slope parameter and a decrease of 16,4% in the intercept parameter, when compared to the mean values obtained with the frequentist approach for these 4 samples. If instead with compare with the parameters obtained for all data (Figure 11) we have a difference of 12.4% and 40.2% when compared with Figure 11 a) and 6.2% and 48.6% when compared with Figure 11 b), for the slope and intercept parameters, respectively.

Plotting the mean value and the characteristic values obtained for the 4 samples against the parameter's distribution obtained with the Bayesian approach, we observe that the mean parameters lie outside the HDI interval. However, for the characteristic values, only the slope parameter falls outside the HDI.

When estimating parameters, it's crucial to consider the representativeness of the data we possess. Should we disregard all past samples and rely solely on the new ones? Wouldn't it be more prudent to incorporate this information? The selection of parameter values ultimately relies on the engineer's judgment, but this study serves as a reminder that there are alternative approaches. In this instance, considering prior knowledge and deriving weighted parameters from the new samples analyzed using the frequentist approach alongside the mean values obtained from the Bayesian approach could offer a viable method for parameter estimation with reduced penalization.

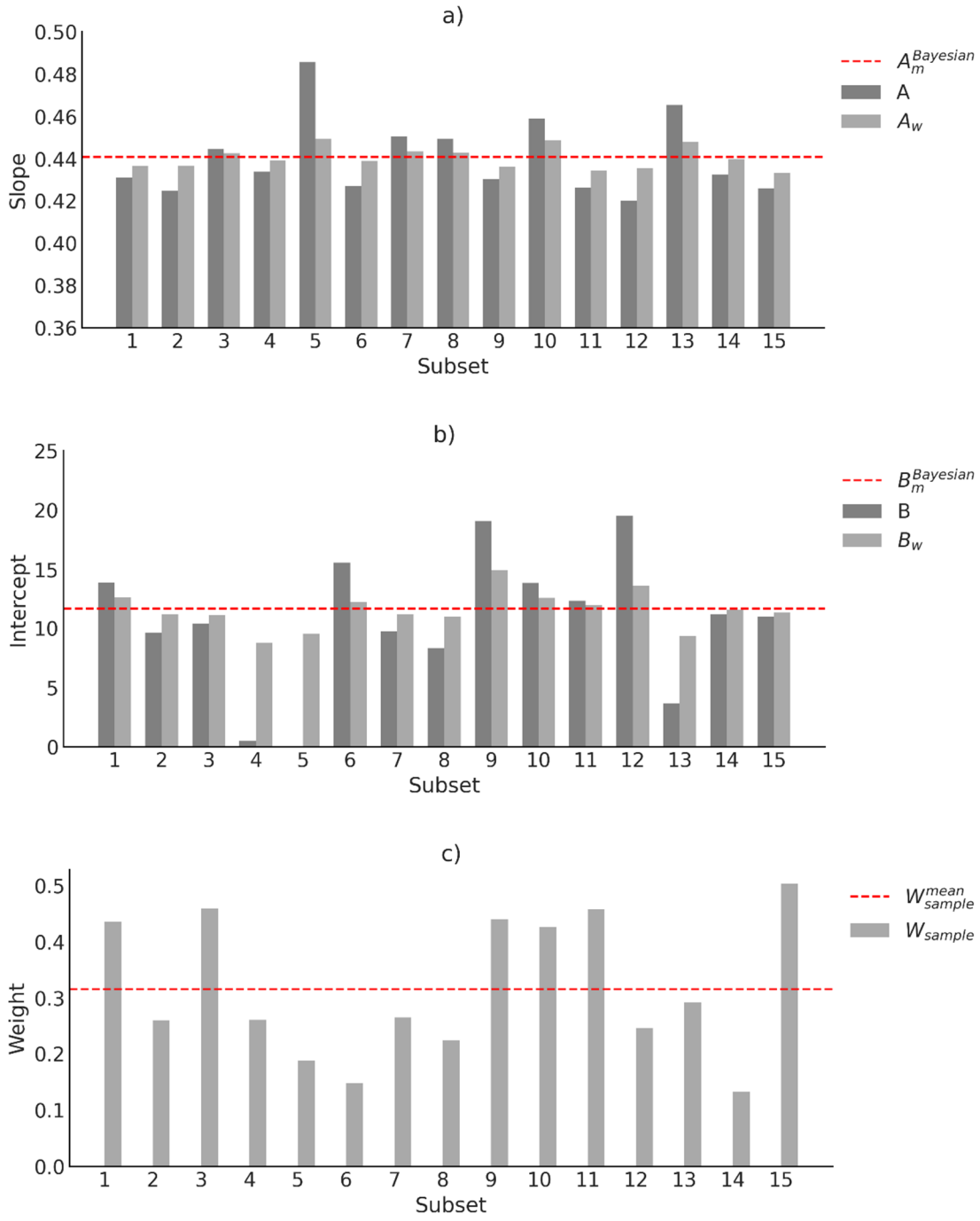


Fig. 15 – Weighting Factor. Simulation of 15 randoms subsets with 4 samples each and the effect of the weighting factor.

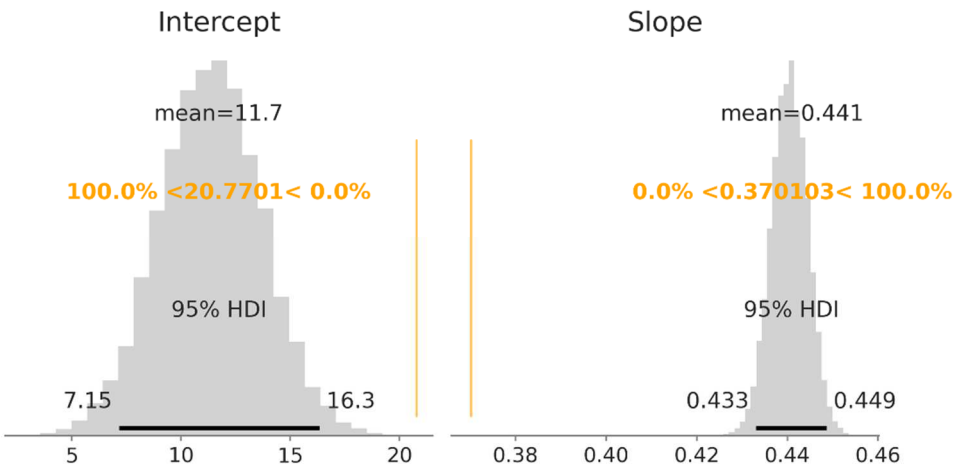


Fig. 16 – Mean Value. Comparison of the mean value obtained through the frequentist approach for four sample (orange line) against the Bayesian inference parameters.

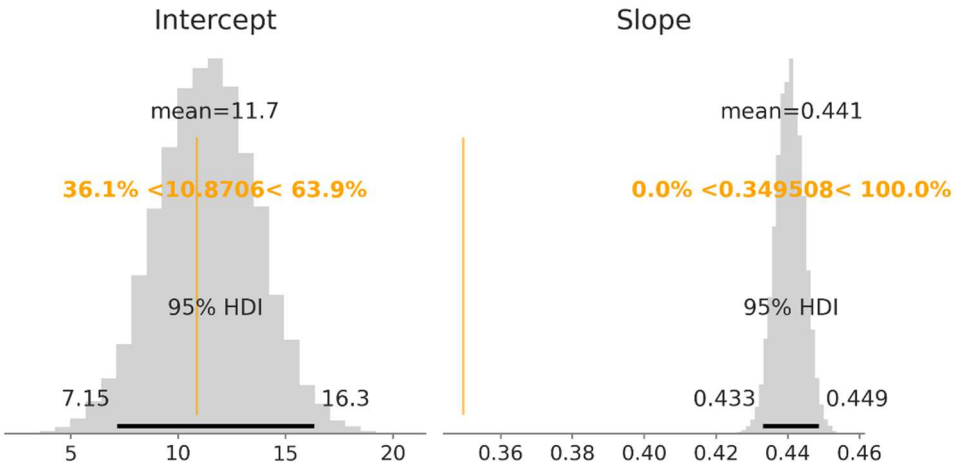


Fig. 17 – Characteristic Value. Comparison of the characteristic value obtained through the frequentist approach for four sample (orange line) against the Bayesian inference parameters.

4. CONCLUSIONS

While acknowledging that our efforts in this note represent a particular interpretation of the data, we resonate with Nietzsche's assertion: "There are no facts, only interpretations." Our endeavor is to synthesize all available information for application in new projects, thereby substantiating our expertise in a mathematical context. Our experience is valid to us, but questionable to everyone else, so we must provide evidence for others to rely on. With this study, we aim to draw attention to the importance of our historical data, emphasizing that they matter and can be very useful. Keeping data organized enables the application of Bayesian approaches, which can provide valuable insights into our parameters and translate our experience. Perhaps in the near future, we will confidently assert that we truly understand our materials, much like the London clay or the Toyoura sand.

This study underscores the pivotal role of Bayesian analysis in geotechnical studies, especially when dealing with sparse datasets and incorporating human judgment and experience in parameter estimation. Traditional frequentist statistics, while valuable in certain contexts, may fall short in adequately addressing the complexities inherent in geotechnical data analysis.

The Bayesian approach offers a flexible framework that accommodates the integration of prior knowledge and the quantification of uncertainty in parameter estimation. By formulating regression models using probability distributions, Bayesian inference provides richer insights into parameter variability and facilitates robust decision-making. With our extensive dataset, we have the opportunity to confidently understand the true parameters, which have guided our interpretation of the model results. In addition to introducing readers to the Bayesian approach, we have provided a practical example demonstrating the advantages of leveraging historical samples through prior knowledge.

REFERENCES

[1] Contreras, L. F. (2020). Bayesian methods to treat geotechnical uncertainty in risk-based design of open pit slopes. PhD Thesis. University of Queensland.

[2] Standard, A. S. T. M. (2002). *Standard test method for consolidated undrained triaxial compression test for cohesive soils*. ASTM International.

[3] Kruschke, J. (2014). *Doing Bayesian data analysis: A tutorial with R*, JAGS, and Stan.

[4] Pohl, C. (2011). *Determination of characteristic soil values by statistical methods*. ISGR Geotechnical Safety and Risk, 427-434.

[5] Pacheco, M. P., & Lima, L. S. A. (1996). *Um critério estatístico para interpretação de ensaios geotécnicos*. Solos e Rochas, 19(3), 177-188.

[6] Vehtari, A., Gelman, A., Simpson, D., Carpenter, B., & Bürkner, P. C. (2021). *Rank-normalization, folding, and localization: An improved $\hat{R}$ for assessing convergence of MCMC (with discussion)*. Bayesian analysis, 16(2), 667-718.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

TECHNOLOGICAL CHALLENGES IN TAILINGS DAMS IN THE BRAZILIAN AMAZON

P. Véliz[1]
PEK Teknep Eng. S.A.

P. Kong
PEK Teknep Eng. S.A.

ABSTRACT

This paper explores the technological, environmental, and regulatory challenges associated with tailings dam management in the Brazilian Amazon. It discusses the complexity of tailings disposal in a region with extreme rainfall, unstable lateritic soils, and logistical constraints.

While technological innovations such as filtered tailings stacking have been proposed as sustainable alternatives, their feasibility remains highly dependent on site-specific factors, and the Brazilian experience with dry tailings is still limited. The study presents a structured comparison of construction techniques, highlighting the historical reliance on conventional tailings disposal and the significant engineering challenges in transitioning to modern solutions.

Key operational and economic challenges are discussed within the constraints of publicly available data, recognizing the industry's reluctance to disclose CAPEX and OPEX figures for Amazonian operations. The research is based on publicly available ANM (Agência Nacional de Mineração) data, complemented by engineering insights on the practical difficulties of tailings management in the Amazon.

Keywords: Tailings dams, Amazon, sustainability, technological innovation, stability analysis.

[1]Ppinov2@pekteknep.com.br, Brazil

1. INTRODUCTION

Tailings dams are critical components of mining infrastructure, designed to store the by-products of mineral processing. Their construction and operation are heavily influenced by environmental, geological, and regulatory conditions. In the Amazon, tailings storage facilities face unique challenges, including extreme rainfall, unstable lateritic soils, and sensitive ecosystems requiring specialized engineering solutions.

This study evaluates the technological, regulatory, and environmental challenges associated with tailings dam construction and operation in the Amazon. The discussion contrasts these challenges with those in non-Amazonian regions and explores emerging technological innovations aimed at improving dam safety and sustainability. Special emphasis is placed on alternative disposal methods such as filtered tailings stacking, which has not been widely implemented in Brazil despite its potential benefits.

In addition, the paper highlights existing initiatives in the Amazon that incorporate dry stacking and filtered tailings technology, such as Hydro Paragominas, Mineração Rio do Norte (MRN), Bemisa, and Horizonte Minerals. These cases serve as important references for understanding the feasibility and effectiveness of alternative tailings management methods in tropical environments. Furthermore, analysis of data from Table 5, which lists 188 tailings dams, and Table 6, covering 304 tailings dams, indicates that dry stacking initiatives are largely concentrated in Minas Gerais and select operations in the Amazon. These findings suggest that dry stacking is not a uniform method but instead consists of varied approaches adapted to specific geological and climatic conditions.

Additionally, the information used to compile Tables 1 through 6 is derived from the Sistema Integrado de Gestão de Barragens de Mineração (SIGBM), a public and official database managed by the Agência Nacional de Mineração (ANM), Brazil's regulatory body for mining activities. The SIGBM provides comprehensive and up-to-date information on all registered tailings dams in the country, ensuring transparency and regulatory compliance. By utilizing this authoritative database, this study presents an accurate and nationally recognized reference for analysing tailings management practices in a country of continental dimensions like Brazil.

This dataset served as the foundation for all numerical analyses presented in this paper, ensuring that the findings are based on the most reliable and extensive database available in the country. By structuring the discussion on real-world data, this study provides an unparalleled overview of the current state of tailings dams in Brazil, serving as a primary reference for further research and policy development in the sector.

2. THE AMAZON'S SCALE AND GLOBAL COMPARISON

2.1. Territorial Scale

The Amazon rainforest spans approximately 5.5 million square kilometres, representing about 60% of Brazil's territory. As the largest tropical rainforest, it surpasses other major biomes, such as the Congo Basin and Southeast Asia, in size and complexity. The Amazon is home to 20% of the world's freshwater and over 10% of known biodiversity, necessitating environmentally responsible engineering practices [1],[2].

2.2. Climate Characteristics

The Amazon's tropical rainforest climate (Af under the Köppen-Geiger classification) includes high temperatures and abundant rainfall exceeding 2,000 mm annually. These conditions significantly influence soil behaviour and logistical planning for large-scale infrastructure projects such as tailings dams [3], [4].

3. CONSTRUCTION ENVIRONMENTS: AMAZONIAN VS. NON-AMAZONIAN

3.1. Characteristics of Non-Amazonian Environments

Non-Amazonian environments benefit from predictable climates and accessible infrastructure, allowing for standardized engineering practices. Conversely, the Amazon presents:

- Intense Rainfall: Fluctuations in water tables increase erosion risks.
- Biodiversity Sensitivity: Projects must mitigate deforestation and wildlife disruption.
- Logistical Barriers: Remote locations complicate material and equipment transport.

3.2. Soil and Material Constraints

Amazonian soils, predominantly lateritic, exhibit high porosity and low bearing capacity. Engineering adaptations often involve compacted laterite or alternative stabilizers, such as lime, to enhance stability. These solutions demand innovative approaches to overcome the scarcity of high-quality aggregates.

3.3. Geotechnical Properties of Amazonian Soils

To provide readers with a solid comparative basis, the following geotechnical parameters of Amazonian soils are presented:

Physical Properties of Amazonian Soils

Bulk Density and Total Porosity:

- Yellow Latosol (very clayey texture): Average bulk density of 1.2 g/cm³ and total porosity of approximately 55%.
- Red-Yellow Podzolic Soil: Average bulk density of 1.3 g/cm³ and total porosity around 50%.
- Terra Roxa (purple earth) Structured Soil: Average bulk density of 1.25 g/cm³ and total porosity of about 53%.

These values indicate the soil's water retention capacity and natural compaction, directly influencing its suitability for different engineering and agricultural applications [5].

Standard Penetration Test (NSPT) Values

Soils of Belém, Pará:

- Zone A (urban area): NSPT values ranging from 10 to 30 blows in the first 10 meters of depth, indicating moderate to high resistance soils.
- Zone B (peripheral area): NSPT values between 4 and 10 blows at the same depth, suggesting lower resistance and higher compressibility soils.

These data are crucial for foundation design and soil-bearing capacity assessments in construction projects [6].

Transformations in Plinthitic Soils

Moisture and Drying Cycle Studies:

- After 12 months of wetting and drying cycles, plinthitic soils showed increased penetration resistance and reduced plasticity, making them more suitable for road construction.
- These alterations highlight the importance of considering environmental conditions and soil treatment methods when planning infrastructure projects in the Amazon [7].

MCT Classification and Resilient Modulus

The MCT Method (Miniature, Compacted, Tropical) is a soil classification system specifically developed for tropical soils, considering their unique physical and mechanical properties. Unlike traditional classification methods such as the Unified Soil Classification System (USCS) or AASHTO, which were primarily designed for temperate climate soils, the MCT method better reflects the real behaviour of tropical soils in engineering applications.

Key Aspects of the MCT Method:

1. Purpose: The MCT method classifies tropical and lateritic soils, which often do not behave according to classical soil mechanics principles due to their high iron and

aluminium oxide content and intense weathering. It provides a more accurate assessment for pavement design, compaction, and foundation applications.

2. Testing Procedure: Miniature compaction: Uses a smaller soil sample (170 cm^3) than standard Proctor compaction tests.
3. Penetration resistance: A miniature cone penetrometer determines the soil's bearing capacity and shear strength.
4. Consistency tests: Plasticity, grain size, and compaction properties are assessed to determine soil behaviour under load.

Classification Groups in MCT:

- Lateritic Soils (LA, LB, LC, LD): Highly weathered, rich in iron and aluminium oxides. Typically, stable with good compaction and low plasticity. Used for road bases and compacted fill materials.
- Non-Lateritic Soils (NA, NB, NC, ND): Less weathered, higher clay content, and plasticity. More compressible and less stable compared to lateritic soils. Often require stabilization for construction use.

Engineering Applications:

- Pavements: The MCT method is widely used in road design and construction because it predicts soil behaviour under traffic loads more accurately than traditional methods.
- Earthworks: Helps determine optimal compaction conditions and soil suitability for embankments and foundation layers.
- Geotechnical projects: Essential in bridge and dam foundations, slope stability analysis, and landfill projects.

Advantages of the MCT Method:

- Better accuracy for tropical soils: Avoids misclassification that can occur with traditional methods.
- Cost-effective: Reduces unnecessary soil stabilization, optimizing material selection.
- Field applicability: Allows for on-site quick assessments of soil quality.

Application to Amazonian Soils:

Fine-textured soils collected from airport areas in the Amazon region were classified using the MCT methodology and showed resilient modulus values compatible with pavement layer requirements after dynamic triaxial testing.

The use of these methodologies allows for a precise evaluation of soil mechanical behaviour under repeated loads, essential for proper pavement design [8], [9], [10], [11], [12].

3.4. Tailings Dam Construction Techniques

The construction of tailings dams in the Amazon presents unique challenges due to high rainfall, weak lateritic soils, and logistical constraints. Choosing an appropriate dam construction method is critical to ensuring stability, environmental safety, and long-term sustainability.

The Agência Nacional de Mineração (ANM) issued Resolution No. 13/2019, which prohibits the use of the upstream method for tailings dams in Brazil due to its high failure risk. This regulation bans the construction of new upstream tailings dams and mandates the decommissioning or removal of all existing upstream dams up to August 15, 2021. Presently some tailings dams still in the decommissioning stage. This prohibition stems from high-profile dam failures in Brazil, emphasizing the need for safer, alternative construction techniques

Alternative Tailings Dam Construction Methods

Since upstream dams are no longer permitted in Brazil, two primary methods remain viable for the Amazon region:

Downstream Method (Higher cost but superior stability)

- The dam is progressively raised outward, using compacted fill material.
- Advantages in the Amazon:
- Greater resistance to high rainfall due to improved structural integrity.
- Provides better foundation support in weak lateritic soils.

Key Considerations:

- Requires larger amounts of compacted material.
- Higher construction cost and increased material transportation challenges.

Centerline Method (A balance between cost and safety)

The dam is raised partly over the tailings and partly over compacted material, balancing cost and stability.

Suitability for Amazonian Conditions:

- Requires careful tailings deposition management to maintain stability.
- Moderate reliance on drainage infrastructure to prevent water accumulation.

Soil Adaptations for Amazonian Tailings Dams

Due to the prevalence of lateritic and soft clayey soils, tailings dams in the Amazon require specific geotechnical adaptations:

Soil Improvement Techniques:

- Lime or cement stabilization: Enhances soil strength and reduces permeability.
- Geosynthetics (geotextiles, geogrids): Reinforce weak foundation layers and improve drainage efficiency.
- Rockfill or sand drains: Facilitate water drainage and increase dam integrity.

Enhanced Drainage Solutions:

- Toe drains and underdrains: Prevent excessive pore water pressure buildup.
- Seepage control barriers: Cutoff walls, clay cores, or synthetic liners reduce seepage and erosion.
- Surface water management: Diversion channels and spillways direct runoff away from the dam structure.

Sustainable Tailings Management Alternatives

Given the environmental sensitivity of the Amazon, alternative methods to reduce reliance on tailings dams are gaining importance:

Filtered Tailings (Dry Stacking):

- Eliminates the need for large impoundments, reducing failure risks.
- Requires significant investment in dewatering infrastructure.
- More sustainable but logistically challenging in remote areas.

Thickened and Paste Tailings:

- Increases tailings density to improve stability.
- Reduces water demand and enhances reclamation potential.

Hybrid Systems (Combination of Wet and Dry Methods):

- Tailings are initially thickened and later deposited in a controlled manner to minimize dam growth.

Best Practices for Tailings Dam Safety in the Amazon

- Real-Time Monitoring and Early Warning Systems:
- Deploying satellite-based and remote sensing technologies for early detection of structural weaknesses.
- Use of automated piezometers and inclinometers to track subsurface movements.

Progressive Closure and Rehabilitation Plans:

- Integrating vegetation cover for erosion control.
- Restoring hydrological balance with wetland reconstruction.

By implementing these techniques, tailings dam projects in the Amazon can achieve greater safety, regulatory compliance, and long-term environmental sustainability, mitigating risks associated with extreme weather and challenging soil conditions [13], [14], [15].

4. REGULATORY FRAMEWORK

4.1. Brazilian Legislation Governing Mining and Its Application to the Amazon

Brazil's mining sector is regulated by a comprehensive legal framework that addresses environmental protection, safety, and sustainable development. Key regulations include the Mining Code (Decree-Law No. 227/1967) and the National Environmental Policy (Law No. 6,938/1981), which outline guidelines for the exploration and exploitation of mineral resources, as well as the protection of ecological systems. Additionally, the National Water Resources Policy (Law No. 9,433/1997) governs the use and management of water resources, a critical component in tailings dam construction and operation [16].

In the Amazon, stricter regulations often apply due to the region's ecological sensitivity. Projects must adhere to specific licensing processes involving multiple agencies, including the Brazilian Institute of Environment and Renewable Natural Resources (IBAMA) and state environmental bodies. Environmental impact assessments (EIAs) are mandatory and must address deforestation, biodiversity loss, and the potential for contamination of water sources. Furthermore, compliance with indigenous rights legislation is critical, requiring consultation with affected communities under Convention 169 of the International Labour Organization (ILO.

The enforcement of these laws is challenging due to the vastness of the Amazon, logistical constraints, and illegal mining activities. Nonetheless, initiatives like the Action Plan for Prevention and Control of Deforestation in the Legal Amazon (PPCDAm) aim to enhance governance and reduce environmental impacts associated with mining operations.

4.2. National Policy on Tailings Dams (PNSB) and the Role of ANM

The National Policy on Tailings Dams (PNSB) in Brazil was established through Law No. 12,334, enacted on September 20, 2010. This law set forth a comprehensive framework for the management and safety of tailings dams across the country. Its primary objectives include the prevention of accidents, minimization of environmental impacts, and protection of human lives and property. The PNSB also defines responsibilities for dam owners, regulatory agencies, and government bodies.

Key components of the PNSB include:

Classification of Dams: Based on potential damage associated with failure, considering environmental, social, and economic impacts.

Safety Reports: Mining companies must submit periodic dam safety reports (Relatórios de Segurança de Barragens, RSB) to the National Mining Agency (ANM).

Emergency Plans: Development of Emergency Action Plans (EAPs) for high-risk dams, outlining response measures and affected area notifications.

Monitoring and Inspections: Mandated frequency of inspections and monitoring based on the dam's classification and risk level.

The PNSB emphasizes a precautionary approach, requiring proactive measures to ensure dam integrity, particularly in sensitive areas like the Amazon, where environmental and social stakes are high.

The ANM mandates that mining companies conduct comprehensive risk assessments, maintain detailed monitoring records, and adhere to emergency action plans (EAPs). Tailings dams are classified based on their potential damage level, ranging from low-risk structures to high-risk installations that require continuous oversight and advanced monitoring systems. These classifications directly impact the frequency and stringency of inspections.

In the Amazon, the PNSB is critical due to the region's unique environmental and social sensitivities. The ANM emphasizes the incorporation of robust safety measures tailored to high rainfall, challenging logistics, and the protection of biodiversity and local communities. The integration of real-time monitoring technologies, such as satellite surveillance and automated stability sensors, is increasingly encouraged by the agency to enhance compliance in remote areas [17].

4.3. Mandatory Reporting Under the PNSB: DCE and DCO Requirements

As part of Brazil’s National Policy on Tailings Dams (PNSB), mining companies must comply with specific reporting obligations to ensure the continuous safety and monitoring of their tailings dams. These reports are critical for assessing structural stability and addressing any safety risks proactively.

Declaration of Dam Condition (Declaração de Condição de Estabilidade - DCE): The DCE must be submitted twice a year, specifically in March and September, to the National Mining Agency (ANM). This document certifies the stability condition of each dam based on inspections, monitoring data, and technical evaluations conducted by qualified engineers. The DCE must indicate whether the dam is stable or unstable, accompanied by detailed justifications and proposed corrective measures if instability is detected.

Annual Operational Declaration (Declaração de Condições Operacionais - DCO): The DCO is an annual report that must be submitted by June 30th each year. This document provides an operational overview of the dam, including safety measures, maintenance activities, and compliance with the Dam Safety Plan (PSB). It serves as a broader assessment of the dam's operational and structural conditions.

Penalties for Non-Compliance: Failure to submit the DCE or DCO within the stipulated deadlines can result in immediate suspension of operations and fines. Additionally, if a company fails to communicate safety issues promptly, the dam may be interdicted, and the company's highest-ranking executive (CEO or equivalent) can be held personally liable. This accountability includes civil and criminal responsibility, with severe penalties for negligence or omission in cases of dam failure or environmental damage.

Reinforcement in the Amazon: In the Amazon, where environmental risks are heightened, these measures are even more stringent. The ANM actively monitors compliance, and any omission in reporting is swiftly addressed to prevent catastrophic events in this ecologically sensitive region [18], [19], [20], [21], [22], [23], [24], [25].

5. CURRENT TAILING DAMS SCENARIO IN BRAZIL

5.1. Design Considerations for Tailings Dams

Of the 897 registered tailings dams, a significant portion lies within the Amazon, highlighting the region's critical role in Brazil's mining sector. Alternative disposal methods, such as dry stacking, are gaining traction for their lower environmental impact.

The history of catastrophic failures, notably the Mariana (2015) and Brumadinho (2019) disasters, has placed the safety of these structures under intense scrutiny, prompting significant regulatory reforms and heightened public awareness.

In response to these events, Brazil introduced stricter regulations under the National Policy on Tailings Dams (PNSB), overseen by the National Mining Agency (ANM). The framework mandates rigorous inspection regimes, enhanced monitoring, and transparent reporting processes to minimize risks and improve accountability. Companies are now required to adopt advanced technologies such as real-time monitoring systems and perform detailed geotechnical assessments to ensure dam stability.

Despite these advancements, challenges persist. A significant number of dams are classified as high risk or high potential damage, requiring continuous oversight. Additionally, the decommissioning of upstream dams—mandated by regulatory reforms—presents logistical and economic hurdles for operators, especially in remote regions like the Amazon.

The current scenario also highlights the growing trend toward alternative tailings disposal methods, such as dry stacking and filtered tailings, which reduce the reliance on conventional dam structures. These technologies, while safer, demand substantial investment and adaptation of existing infrastructure, posing challenges for smaller operators.

Moreover, the social and environmental dimensions of tailings dams have gained prominence. Communities living near mining operations demand greater transparency and safety assurances, while environmental advocates emphasize the need for sustainable practices to protect Brazil's biodiversity and water resources.

Overall, while significant strides have been made in improving the safety and management of tailings dams in Brazil, ongoing challenges in compliance, technological adoption, and socio-environmental integration underscore the need for continued vigilance and innovation.

5.2. Tailing Dam Statistics

A detailed survey and analysis of all tailings dams registered or in operation in Brazil was conducted to provide the foundation for this paper. The survey relied on data from the "National Classification of Mining Dams" published by the National Mining Agency (ANM) in December 2024 [26].

This classification includes the following key data points for each dam:

- Dam ID
- Dam Name
- Exact Geographic Position (Coordinates N, E)
- Municipality
- State
- Primary Mineral
- Current Height (m)
- Current Tailings Volume (m^3)
- Construction Method
- Risk Category
- Associated Potential Damage
- Operational Management
- Included in the PNSB?
- Emergency Level
- DCE Status: Stability Declaration for RISR submitted by the operator.
- DCE Status: Stability Declaration for PSB submitted by the operator.
- DCO Status: Operational Conditions Declaration submitted by the operator.

- Operational Status
- Embargo Status

Brazil has a total of 897 registered tailings dams, representing a wide range of types and capacities, with dam heights varying from just a few meters to nearly 100 meters, which is the tallest recorded.

For this study, 150 dams with heights under 5 meters were excluded, as they do not significantly contribute to the study's objectives. The remaining 747 dams were categorized into two primary groups based on their geographical location:

Group 1 - Non-Amazonian Dams

Group 2 - Amazonian Dams

Each group was further classified by their construction method, following the categories defined by the ANM, which align with classical engineering concepts for this type of structure:

- Type 0 – Single Stage: Constructed in a single continuous phase.
- Type 1 – Upstream Raising: Incrementally built over time, using tailings as construction material. Note: This method is not allowed in Brazil since 2019.
- Type 2 – Downstream Raising: Built downstream of the previous dam face, offering greater stability.
- Type 5 – Centerline Raising: A hybrid method balancing material usage and stability.

Additionally, each construction method was categorized based on the mineral substance of the tailings to identify trends related to the type of mineral processed. Trends were also analyzed in terms of the storage volume of tailings, providing insights into the preferred construction methods for larger or smaller volumes.

Finally, the construction methods were compared against their relative risk profiles, as defined by ANM’s risk assessment methodology. This includes evaluations of structural stability, environmental impact, and potential damage to surrounding communities in case of failure.

This comprehensive classification and analysis provide valuable insights into the operational and safety dynamics of tailings dams in Brazil, particularly in environmentally sensitive areas like the Amazon.

These findings are critical for advancing sustainable and secure practices in the mining sector.

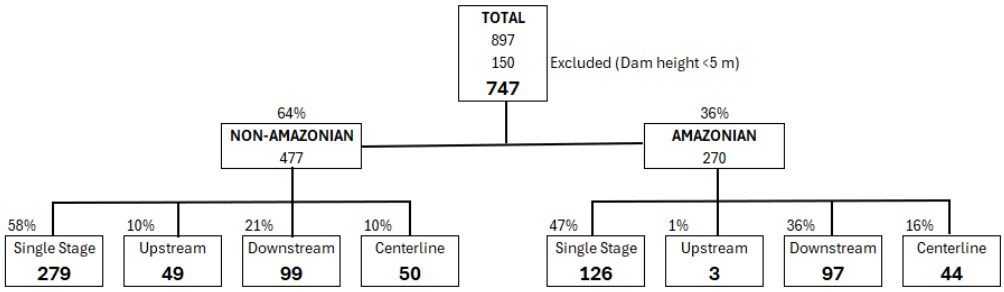


Fig. 1 – Registered Tailing Dams in Brazil as Dec.2024.

The following tables provide a comprehensive comparison of the four primary methods of tailing dam construction—Single Stage, Upstream, Downstream, and Centerline—evaluating their performance and challenges in non-Amazonian and Amazonian environments. These comparisons are structured around critical factors such as average dam height, design stages (FEL1 to FEL3), environmental licensing complexity, material availability, capital costs, dam closure plans, and post-operation monitoring requirements.

The tables aim to highlight the environmental, logistical, and technical differences between the methods when applied in distinct geographic and ecological contexts. Particular attention is given to the Amazon, where unique challenges—such as biodiversity preservation, hydrological complexity, and impacts on indigenous communities—demand tailored engineering and operational approaches.

5.3. Construction Methods applied for Construction of the existing Tailing dams

Single Stage: This method involves constructing the dam in a single phase, making it suitable for smaller dams and projects with predictable material and water management needs. While it offers moderate stability and relatively straightforward design, it is less adaptable to dynamic environments like the Amazon, where complex hydrology and biodiversity increase risks during closure and monitoring.

Upstream: The upstream method incrementally builds the dam upward using tailings as construction material. This economical method reduces material costs but has lower stability, making it particularly challenging in high-rainfall regions such as the Amazon, where erosion and seepage risks are heightened.

Downstream: This method involves constructing the dam outward and downward, requiring significant external material. It offers superior stability, making it suitable for areas with seismic risks or unstable terrain. However, in Amazonian contexts, the high logistical costs and environmental impacts of sourcing materials complicate its implementation.

Centerline: A hybrid approach, the centerline method raises the dam vertically along its initial centerline, balancing material usage and stability. While it is moderately adaptable to various

terrains, it requires detailed geotechnical studies and monitoring, especially in regions like the Amazon, where soil and hydrological conditions are less predictable.

Table 1 – Volume of Tailings per Method.

Region	Construction Method	Total Volume (Mm^3)
Amazon Legal	0 - Single Stage	254.67
Amazon Legal	1 - Upstream	24.45
Amazon Legal	2 - Downstream	602.32
Amazon Legal	5 - Centerline	153.04
Total		1034.48
Non-Amazon	0 - Single Stage	276.71
Non-Amazon	1 - Upstream	670.08
Non-Amazon	2 - Downstream	871.82
Non-Amazon	5 - Centerline	1840.99
Total		3659.60

5.4. Critical Environmental and Operational Elements

Hydrology and Rainfall: Amazonian environments demand advanced drainage systems and erosion controls due to intense and prolonged rainfall. Methods like Upstream and Single Stage are particularly vulnerable to these conditions.

Biodiversity and Ecosystem Sensitivity: Dam construction in the Amazon requires comprehensive studies and mitigation plans to minimize deforestation, protect wildlife, and ensure sustainable land restoration post-operation.

Indigenous Communities: Projects in sensitive areas must include consultations with local and indigenous populations, ensuring minimal disruption to their lands and livelihoods.

Material Availability and Logistics: The remoteness of the Amazon increases dependency on locally sourced materials like compacted laterite, which may not meet engineering standards, or on expensive transportation of external materials.

Post-Operation Monitoring: Closure plans in the Amazon require continuous monitoring, including hydrological and ecological restoration, to mitigate long-term environmental risks.

Table 2 – Average Height of Dam per Method.

Region	Construction Method	Average Height (m)
Amazon Legal	0 - Single Stage	11.65
Amazon Legal	1 - Upstream	14.46
Amazon Legal	2 - Downstream	17.39
Amazon Legal	5 - Centerline	16.64
Non-Amazon	0 - Single Stage	15.34
Non-Amazon	1 - Upstream	52.26
Non-Amazon	2 - Downstream	30.55
Non-Amazon	5 - Centerline	33.16

Table 3 – Average Risk of Dam per Method – Legal Amazon.

Region	Construction Method	Risk Category	Count	% of Total
Amazon Legal	0 - Single Stage	High	7	2.38
Amazon Legal	0 - Single Stage	Low	89	30.27
Amazon Legal	0 - Single Stage	Medium	46	15.65
Amazon Legal	1 - Upstream	High	3	1.02
Amazon Legal	1 - Upstream	Low	3	1.02
Amazon Legal	1 - Upstream	Medium	1	0.34
Amazon Legal	2 - Downstream	High	7	2.38
Amazon Legal	2 - Downstream	Low	52	17.69
Amazon Legal	2 - Downstream	Medium	40	13.61
Amazon Legal	5 - Centerline	High	9	3.06
Amazon Legal	5 - Centerline	Low	8	2.72
Amazon Legal	5 - Centerline	Medium	29	9.86
Amazon Legal	Total		294	100.0

Table 4 – Average Risk of Dam per Method – Non-Amazon.

Region	Construction Method	Risk Category	Count	% of Total
Non-Amazon	0 - Single Stage	High	11	2.45
Non-Amazon	0 - Single Stage	Low	149	33.18
Non-Amazon	0 - Single Stage	Medium	98	21.83
Non-Amazon	1 - Upstream	High	13	2.9
Non-Amazon	1 - Upstream	Low	22	4.9
Non-Amazon	1 - Upstream	Medium	10	2.23
Non-Amazon	2 - Downstream	High	4	0.89
Non-Amazon	2 - Downstream	Low	85	18.93
Non-Amazon	2 - Downstream	Medium	9	2.0
Non-Amazon	5 - Centerline	High	5	1.11
Non-Amazon	5 - Centerline	Low	35	7.8
Non-Amazon	5 - Centerline	Medium	8	1.78
Non-Amazon	Total		449	100.0

Table 5 – Dams per Method and Mineral Substance – Legal Amazon.

Construction Method	Mineral	Count	% of Total
0 - Single Stage	Bauxite	7	3.72
0 - Single Stage	Copper	12	6.38
0 - Single Stage	Gold	33	17.55
0 - Single Stage	Iron	15	7.98
0 - Single Stage	Sand	3	1.60
0 - Single Stage	Tin	11	5.85
0 - Single Stage	Zinc	1	0.53
2 - Downstream	Bauxite	1	0.53
2 - Downstream	Copper	3	1.60
2 - Downstream	Gold	42	22.34
2 - Downstream	Iron	1	0.53
2 - Downstream	Sand	23	12.23
2 - Downstream	Tin	1	0.53
5 - Centerline	Bauxite	1	0.53
5 - Centerline	Gold	29	15.43
5 - Centerline	Sand	4	2.13
Total		188	100.0

Table 6 – Dams per Method and Mineral Substance – Non-Amazon.

Construction Method	Mineral	Count	% of Total
0 - Single Stage	Bauxite	3	0.99
0 - Single Stage	Copper	1	0.33
0 - Single Stage	Gold	11	3.62
0 - Single Stage	Iron	139	45.72
0 - Single Stage	Sand	15	4.93
0 - Single Stage	Tin	1	0.33
0 - Single Stage	Zinc	3	0.99
1 - Upstream	Gold	1	0.33
1 - Upstream	Iron	28	9.21
1 - Upstream	Sand	1	0.33
2 - Downstream	Copper	14	4.61
2 - Downstream	Gold	9	2.96
2 - Downstream	Iron	41	13.49
2 - Downstream	Sand	2	0.66
2 - Downstream	Zinc	1	0.33
5 - Centerline	Bauxite	1	0.33
5 - Centerline	Copper	14	4.61
5 - Centerline	Gold	8	2.63
5 - Centerline	Iron	10	3.29
Total		304	100.0

6. INNOVATIONS IN TAILINGS MANAGEMENT

6.1. Filtered Tailings Stacking in the Amazon

Despite the challenges associated with high rainfall and soil conditions, some mining operations in the Amazon have begun implementing alternative tailings management strategies. Notably:

- Hydro Paragominas (Bauxite Mining, Pará): Uses a pioneering approach known as "Tailing Dry Backfill", where tailings are dried for 60 days before being reintegrated into mined-out areas. The system reduces the need for conventional tailings dams and integrates land rehabilitation practices. Also utilizes a Reservoir System (RP1) with multiple reservoirs for water treatment and controlled tailings deposition [27].
- Mineração Rio do Norte (MRN) (Bauxite Mining, Pará): Implemented large-scale dry stacking for tailings management as part of its sustainability strategy. Invested USD160 million in new mining plateaus to maintain production while reducing environmental impact [28].

- Bemisa (Gold Mining, Pará): Developed a dry stacking solution in its Água Azul project to eliminate the need for tailings dams. Uses a hydrometallurgical processing system that includes filtration and compacted tailings stacking [29].
- Horizonte Minerals (Nickel Mining, Pará): Implements filtered tailings stacking as part of its Araguaia project, ensuring that waste material is inert and classified as Class II. Investigates potential reuse of filtered tailings in ceramic production and asphalt mixtures [30].

These examples illustrate the feasibility of dry stacking in the Amazon and provide a basis for evaluating the broader application of this technology in other mining operations across the region.

6.2. Advanced Monitoring

To ensure safe and effective tailings management, cutting-edge monitoring technologies are being integrated into tailings dam systems. These include:

- Satellite-based Monitoring: Remote sensing techniques allow for large-scale, real-time monitoring of ground displacement, subsidence, and potential dam instability, particularly useful for remote Amazonian sites.
- IoT and Sensor Networks: High-precision geotechnical and hydrogeological sensors provide continuous data on pore pressure, slope stability, and structural integrity. These devices can be deployed across dams and tailings storage facilities to enable predictive failure analysis.
- Drones and Aerial Surveys: UAV-based inspections enhance dam monitoring by capturing high-resolution imagery, generating 3D models, and performing thermal scanning for seepage detection.
- Artificial Intelligence (AI) and Machine Learning: AI models are increasingly used for predictive maintenance, processing sensor data to detect early signs of failure and providing automated risk assessments.
- Finite Element and Discrete Element Modelling (FEM/DEM): These numerical simulation methods allow engineers to model tailings behaviour under different environmental and operational conditions, improving dam design and stability analysis.
- Digital Twin Systems for Tailings Dams: A digital twin is a virtual model that integrates real-time monitoring data, AI, and physics-based simulations to provide a comprehensive real-time view of tailings dam performance, allowing predictive analysis and enhanced decision-making.

- AI-driven Operational Simulation Models: Machine learning models trained on historical data can simulate different operational scenarios for tailings facilities, optimizing dam safety and performance under varying condition

6.3. Sustainable Practices

Sustainable approaches in tailings management focus on reducing environmental impact while enhancing safety. The most notable advancements include:

- Dry Stacking and Filtered Tailings: By removing excess water before disposal, this technique improves stability and reduces reliance on conventional dam structures. New dewatering technologies such as high-pressure filter presses and vacuum belt filters are enhancing the efficiency of this method.
- Real-time Density and Flow Monitoring: Advanced instrumentation enables continuous flow characterization and density control in pipelines transporting tailings, reducing the risk of liquefaction.
- Mass Movement Sensors and Ground-based Radar Systems: These tools provide early warnings for potential failures, improving disaster response in case of dam instability.
- Integrated Water Management Systems: With a strong focus on recycling process water, modern tailings facilities incorporate closed-loop water circuits to reduce freshwater consumption.
- AI-enhanced Stability Assessments: Using AI and big data analytics, tailings facilities can assess long-term stability risks and anticipate potential failure scenarios with greater accuracy.

These technologies and practices offer significant improvements in safety, environmental compliance, and operational efficiency for tailings management in the Amazon. However, successful implementation requires site-specific adaptation, particularly in regions characterized by high rainfall and soil instability, necessitating advanced drainage solutions and reinforced containment systems.

6.4. Technological Advancements

Filtered Tailings Technologies:

- High-pressure filter presses: More effective for fine-grained tailings but require high initial investment.
- Vacuum belt filters: Faster but not suitable for all tailing types.
- Centrifuge dewatering: Provides higher dryness levels but is energy intensive.

6.5. Engineering Transition from Conventional to Dry Tailings

The cultural shift from traditional tailings disposal (valley discharge, reservoirs) to engineered dry stacking requires significant adaptation in mining operations.

Companies face:

- Increased regulatory scrutiny—Brazilian regulators push for dry solutions but acknowledge implementation challenges.
- Workforce training gaps—mine operators must learn new filtration and stacking techniques.
- Higher operational costs—filtration plants require more power, maintenance, and logistics planning.

7. DESIGN CONSIDERATIONS FOR TAILINGS DAMS

7.1. Design Considerations for Tailings Dams

Brazil is home to one of the largest mining industries in the world, and with it, a significant number of tailings dams. The main purpose of this paper is to describe and interpret the profile of the tailing dam population in Brazil grouped in the two main environments that govern all constructions and installation of this nature.

The careful design and observation of all regulations and engineering principles and techniques are mandatory to any kind of dam, and tailings are extremely variable in their nature, its rheology, geotechnical characteristics and impact to the environment in a very wide range of matters and elements.

The design considerations included in the tables below are the result of careful study and evaluation of the main elements involved in this kind of engineering construction.

Table 7 – Design Factors for Single Stage Construction Method.

Factor	Non-Amazonian	Amazonian
Average Dam Height (Brazil)	20-30 meters	15-25 meters
Design Time Required (FEL Stages)	Short design time (FEL1 to FEL3: 1-2 years)	Extended design time (FEL1 to FEL3: 3-5 years)
Environmental Licensing Complexity	Moderate; standardized procedures	Complex; additional biodiversity studies
Construction Material Availability	High availability of quality materials	Limited availability; reliance on imports or local laterite
Capital Cost (Estimation)	Moderate capital costs	Higher capital costs due to logistics
Dam Closure and Monitoring Requirements	Simpler closure; requires periodic monitoring	Complex closure; intensive monitoring required with detailed ecological restoration plans
Post-Operation Monitoring Complexity	Moderate; routine inspections and reporting	High; includes biodiversity recovery, water quality, and community involvement
Impact on Indigenous Communities	Minimal impact, fewer displaced communities	High; potential relocation of communities
Impact on Flora and Fauna	Low; standardized mitigations in place	Significant; requires detailed impact assessments
Impact on Water Quality	Moderate, depends on tailings composition	High; high potential for contamination
Risks and Difficulties	Moderate; simpler logistics, stable terrain	High; challenging logistics and soil stability
Time Schedule Differences	Shorter time to completion	Longer completion time due to challenges

Table 8 – Design Factors for Upstream Construction Method.

Factor	Non-Amazonian	Amazonian
Average Dam Height (Brazil)	15-20 meters	10-15 meters
Design Time Required (FEL Stages)	Short design time (FEL1 to FEL3: 1-2 years)	Extended design time (FEL1 to FEL3: 3-5 years)
Environmental Licensing Complexity	Moderate; standardized procedures	High; requires hydrology and ecosystem studies
Construction Material Availability	High; uses tailings as core material	Limited; local tailings used but requires engineering adaptations
Capital Cost (Estimation)	Low; cost-effective method	Moderate; logistics and monitoring add costs
Dam Closure and Monitoring Requirements	Moderate; requires careful monitoring	Complex; prone to erosion, requiring continuous interventions
Post-Operation Monitoring Complexity	Moderate; requires stability inspections and routine checks	High; involves biodiversity recovery and advanced hydrological monitoring
Impact on Indigenous Communities	Minimal impact, localized effects	High; requires extensive consultation and mitigation plans
Impact on Flora and Fauna	Low; mitigations sufficient in non-sensitive ecosystems	Significant; detailed studies and restoration plans necessary
Impact on Water Quality	Moderate; manageable with water treatment systems	High; rainfall and erosion risks require advanced systems
Risks and Difficulties	Moderate; stability concerns in seismic zones	High; soil erosion and stability challenges
Time Schedule Differences	Shorter time to completion	Longer due to design adaptations and approvals

Table 9 – Design Factors for Upstream Construction Method.

Factor	Non-Amazonian	Amazonian
Average Dam Height (Brazil)	25-35 meters	20-30 meters
Design Time Required (FEL Stages)	Moderate design time (FEL1 to FEL3: 2-3 years)	Extended design time (FEL1 to FEL3: 4-6 years)
Environmental Licensing Complexity	Moderate; standardized geotechnical studies	High; requires advanced ecosystem and soil studies
Construction Material Availability	High; requires quality external materials	Limited; external materials must be transported
Capital Cost (Estimation)	High; material-intensive method	Very high; costly logistics and material sourcing
Dam Closure and Monitoring Requirements	Simpler; stable long-term monitoring	Complex; requires robust monitoring systems
Post-Operation Monitoring Complexity	Moderate; regular inspections suffice	High; includes biodiversity recovery and water management
Impact on Indigenous Communities	Minimal impact, localized effects	High; potential for displacement or conflicts
Impact on Flora and Fauna	Moderate; stable designs minimize risks	Significant; extensive mitigations required
Impact on Water Quality	Low; seepage minimized by design	Moderate; seepage minimized but still monitored
Risks and Difficulties	Moderate; terrain must support heavy materials	High; logistics and biodiversity considerations
Time Schedule Differences	Standard timeline for high stability	Longer due to material and logistical challenges

Table 10 – Design Factors for Centerline Construction Method.

Factor	Non-Amazonian	Amazonian
Average Dam Height (Brazil)	20-30 meters	15-25 meters
Design Time Required (FEL Stages)	Moderate design time (FEL1 to FEL3: 2-3 years)	Extended design time (FEL1 to FEL3: 4-5 years)
Environmental Licensing Complexity	Moderate; involves hybrid assessments	High; complex studies due to hybrid requirements
Construction Material Availability	Moderate; combines tailings and external materials	Limited; balanced sourcing requires logistical planning
Capital Cost (Estimation)	Moderate; balanced costs	High; costlier in remote regions
Dam Closure and Monitoring Requirements	Simpler; requires periodic inspections	Complex; long-term inspection and monitoring needed
Post-Operation Monitoring Complexity	Moderate; inspection-focused monitoring	High; includes biodiversity recovery and hydrology management
Impact on Indigenous Communities	Minimal impact, localized effects	High; requires extensive consultations
Impact on Flora and Fauna	Moderate; stable design minimizes impact	Significant; biodiversity restoration required
Impact on Water Quality	Moderate; seepage controlled effectively	High; seepage risks increase with local conditions
Risks and Difficulties	Moderate; stability depends on proper balance	High; maintaining balance under dynamic conditions
Time Schedule Differences	Standard timeline for adaptable designs	Longer due to approvals and design refinements

8. CONCLUSIONS

6.1. Key Findings

- The Brazilian experience with filtered tailings is still evolving. Not all solutions work for all types of tailings.
- Transitioning from wet to dry disposal requires significant engineering adaptation.
- Cost barriers remain a major limitation, and companies often avoid disclosing true CAPEX/OPEX figures.
- Regulations have driven innovation, but implementation in the Amazon remains complex.

6.2. Recommended Areas for Future Research

- Field studies comparing dry tailings vs. conventional methods in Amazonian conditions.
- Development of tailored filtration techniques for high-moisture tropical tailings.
- Analysis of financial feasibility of dry stacking under Amazonian logistics constraints.
- Case studies of successful (or failed) implementations to guide future projects.

REFERENCES

[1] Malhi, Y., Gardner, T. A., Goldsmith, G. R., Silman, M. R., & Zelazowski, P. (2014). "Tropicalforests in the Anthropocene." Annual Review of Environment and Resources, 39(1), 125-159.

[2] Brando, P. M., et al. (2019). "Fire, deforestation, and climate change in the Amazon." Current Biology, 29(17), R1109-R1115.

[3] Rubel, F., & Kottek, M. (2010). "World Map of the Köppen-Geiger climate classification updated." Meteorologische Zeitschrift, 19(2), 135-141.

[4] Cui, D., Liang, S., Wang, D., & Liu, Z. (2021). "A 1 km global dataset of historical (1979–2013) and future (2020–2100) Köppen–Geiger climate classification and bioclimatic variables." Earth System Science Data, 13(11), 5087-5108.

[5] Baena, A. R. C.; Dutra, S. (1982). “Propriedades físicas dos principais solos da Amazônia brasileira em condições naturais”. Belém, EMBRAPA-CPATU, 1982.

[6] Alves Vieira, M.; Nascimento Alves, C.; Santos Ferreira, Vyviane; Leite Barbosa, Cleisianne (2021). “Modeling of geological-geotechnical properties in soils of Belém do Pará”. Research, Society and Development, v. 10, n. 2, e40810212664, 2021

[7] Wanderley Neto, A.; (1995). MSc “Estudo da transformação de propriedades geotécnicas de solos plintíticos da Amazônia para fins de pavimentação”. Instituto Tecnológico de Aeronáutica. BDTD - Biblioteca Digital Brasileira de Teses e Dissertações. Ministério da Ciência, Tecnologia e Inovações, Brasil.

[8] Vertamatti, E.; (1988). MSc “Contribuição ao conhecimento geotécnico de solos da Amazônia com base na investigação de aeroportos e metodologias MCT e resiliente”. Instituto Tecnológico de Aeronáutica. BDTD - Biblioteca Digital Brasileira de Teses e Dissertações. Ministério da Ciência, Tecnologia e Inovações, Brasil.

[9] Marson, M.; (2004). MSc “Análise crítica da classificação MCT para solos tropicais”. Instituto Tecnológico de Aeronáutica. BDTD - Biblioteca Digital Brasileira de Teses e Dissertações. Ministério da Ciência, Tecnologia e Inovações, Brasil.

[10] Nogami, J. S., & Villibor, D. F. (1995). Pavimentação de baixo custo com solos lateríticos: metodologia MCT. Editora Villibor.

[11] Departamento Nacional de Infraestrutura de Transportes (DNIT), Brasil. (2023). Classificação de solos tropicais de granulação grossa – Classificação. NORMA DNIT 444/2023 – CLA.

[12] Pereira, Lázaro Fernandes. (2020). Et al. “Estudo da utilização de solos lateríticos e a metodologia MCT: Análise em trecho a ser duplicado na BR-116. Revista Científica Multidisciplinar Núcleo do Conhecimento”. Ano 05, Ed. 06, Vol. 02, pp. 72-105. Junho de 2020.

[13] Junior, B., C. Rodrigues, N. Singh and L. Souza. (2024). “How to Design a Tailing’s Dam Full Removal? Lessons Learned from a Case Study in Brazil,” in Proceedings of the Tailings and Mine Waste Conference, 10-13 November 2024. Colorado, USA.

[14] Débora Schaper, D; Lessa, R. Freitas, A.; Weeks, Björn. (2020).” De-characterization and closure of TSF: concepts of the Brazilian legislation and international criteria”. Planning For Closure 2020. 3rd International Congress on Planning for Closure of Mining Operations.

[15] Pinto, C. S., & Fonseca, A. S. (2018). "Geotechnical challenges of construction in the Amazon region." Geotechnical Engineering Journal, 42(3), 231-250.

[16] Ministério de Minas e Energia. (2017). "Código de Mineração Brasileiro e Regulações Associadas." https://www.gov.br/anm/pt-br/acesso-a-informacao/institucional/base-juridica-da-estrutura-organizacional-e-das-competencias-11/legislacao-base

[17] SIGBM - Sistema Integrado de Gestão de Barragens de Mineração. Cadastro Nacional de Barragens de Mineração https://app.anm.gov.br/sigbm/publico

[18] Almeida, F. (2020). "The Evolution of Mining Legislation in Brazil: Impacts on Environmental Management." Journal of Environmental Law in Latin America, p.203-221.

[19] IBAMA (2021). "Guidelines for Environmental Licensing of Mining Projects in the Amazon."

[20] Silva, R. & Barreto, P. (2019). "Deforestation and Mining in the Amazon: Legal and Illegal Challenges." Environmental Science Policy Review, 27, 98-110.

[21] Convention 169 of the International Labor Organization (ILO). (1989). "Rights of Indigenous Peoples: Mining and Consultation Frameworks."

[22] Santos, M. & Costa, J. (2022). "Indigenous Consultation Processes in Brazilian Mining Projects." Human Rights Journal of the Americas, 18(2), 157-176.

[23] Brasil. (2010). Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens. Diário Oficial da União.

[24] ANM. (2021). Resolução ANM nº 13, de 2020: Diretrizes para a implementação da Política Nacional de Segurança de Barragens. Agência Nacional de Mineração.

[25] Silva, L. P., & Ramos, M. P. (2020). "The National Policy on Tailings Dams: Challenges in Implementation." Brazilian Journal of Environmental Management, 15(3), 445-459.

[26] https://app.anm.gov.br/SIGBM/Publico/ClassificacaoNacionalDaBarragem

[27] https://www.hydro.com/br/global/sobre-a-hydro/a-hydro-no-mundo/americas/brasil/paragominas/mineracao-paragominas/barragens/

[28] MRN – Mineração Rio do Norte. (2022). “Projeto Novas Minas – PNM”. https://mrn.com.br/images/Cartilha%20PNM-MRN%20(1).pdf

[29] Brasil Mineral Magazine Edition 441. (2024).”Bemisa profile – Água Azul gold mine commissioning in Pará state, Brasil”

[30] Horizonte Minerals. (2021). “Annual Report. “Vermelho’s dry-stack residue storage facility option”. https://www.annualreports.com

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

TAILINGS DAMS ENGINEERING: AMAZON VS NON-AMAZON CHALLENGES

P. Véliz[1]
PEK Teknep Eng. S.A.

P. Kong
PEK Teknep Eng. S.A.

ABSTRACT

This paper comprehensively analyses the structural design, investigation techniques, and construction methodologies required for tailings dams in Amazonian versus non-Amazonian environments. While most tailings dams worldwide are built in non-Amazonian regions, where the structural calculation methods and construction practices are well-established, Amazon's unique environmental and logistical challenges demand specialized approaches. In non-Amazonian regions, tailings dams are typically designed based on well-documented theories of soil mechanics, stability analysis, and construction techniques. However, these conventional methodologies often prove inadequate in the Amazon, where high rainfall, complex hydrology, and diverse soil compositions necessitate modified or entirely new engineering solutions. This paper explores these differences in detail, examining how structural calculations must be adapted to account for the Amazon's unique conditions, such as its highly variable water table and the impact of intense and prolonged rainfall on dam stability.

Additionally, the paper investigates the distinct investigation techniques required for the Amazon, where traditional geotechnical surveys may not fully capture the variability of soil types or the potential for rapid environmental changes. We also discuss the construction methodologies that must be tailored to the Amazon's remote and ecologically sensitive nature, including the logistics of material transport and the need for construction techniques that minimize environmental impact. A significant contribution of this work is the structured analysis of these engineering practices, providing insights into how tailings dams must be designed, built, and monitored differently depending on the specific mineral or commodity being extracted. This analysis is critical for advancing the safety and sustainability of tailings dams in the Amazon, where conventional engineering approaches may not be sufficient.

Keywords: Tailings dams, Amazon, geotechnical engineering, advanced instrumentation, technological innovation.

[1]Ppinov2@pekteknep.com.br, Brazil

1. INTRODUCTION

Tailings dams are critical infrastructure in mining operations, requiring robust engineering solutions to manage waste by-products safely. The construction and operation of these dams depend heavily on the environmental and geological context. While non-Amazonian regions allow for the application of standardized methods, the Amazon's unique challenges demand innovative and adaptive technologies.

This paper focuses on the advanced tools and methods available for geological and geotechnical studies, instrumentation, and monitoring. Contrasting Amazonian and non-Amazonian environments highlights the practical application of these technologies and their role in ensuring dam safety and environmental sustainability.

2. TASKS, ACTIONS, AND TOOLS: A STEP-BY-STEP FRAMEWORK

2.1. Initial Project Phase (FEL 1)

During FEL 1, the primary goal is to gather essential data to define potential sites for tailings dam construction. In the Amazon, the challenges of high biodiversity, complex hydrology, and limited infrastructure require additional efforts compared to non-Amazonian regions.

Key Tasks and Actions:

- High-resolution terrain analysis: Using LiDAR and GIS software to create 3D models that penetrate dense vegetation and map elevation changes.
- Environmental baseline studies: Aerial and drone-based surveys to identify key ecological features, such as protected areas, water streams, and indigenous territories.
- Water flow and quality assessment: Preliminary hydrological studies to understand the water balance in potential dam sites.

Considerations for Construction Methods:

- Single-stage dams benefit from faster FEL 1 completion but may struggle with extensive environmental licensing in the Amazon.
- Upstream dams require more detailed soil studies early on, as tailings form the dam’s core [8], [9], [10].

2.2 Feasibility Studies (FEL 2)

FEL 2 focuses on refining site selection and evaluating technical and economic feasibility. For Amazonian projects, more robust hydrological and geotechnical assessments are required.

Key Tasks and Actions:

- Detailed soil characterization: Conduct Cone Penetration Testing (CPT) and borehole drilling to determine soil stability and load-bearing capacity.
- Hydrological modelling: Simulate rainfall-runoff scenarios using dynamic models to address the region's high precipitation rates.
- Contamination risk analysis: Analyse legacy mining impacts to address contamination in water and soil.

Considerations for Construction Methods:

- Upstream dams may face greater challenges in regions with high erosion risks.
- Downstream dams require early sourcing of high-quality external materials, adding logistical complexities in remote Amazon areas [8], [9], [10].

2.3 Detailed Engineering (FEL 3)

During FEL 3, the goal is to finalize the dam's design and ensure its safety under operational and extreme environmental conditions.

Key Tasks and Actions:

- Stability simulations: Using Finite Element Analysis (FEA) tools to model stress distribution and seepage behaviour in various scenarios.
- Instrumentation planning: Defining real-time monitoring systems, such as IoT-connected piezometers and inclinometers.
- Finalize construction methodology: Selecting the optimal method—single-stage, upstream, downstream, or centerline—based on site-specific data.

Considerations for Construction Methods:

- Downstream methods offer greater long-term stability, making them ideal for high rainfall areas.
- Centerline designs balance cost and stability, though they require precise hydrological management [8], [9], [10].

2.2 Construction Phase

The construction phase involves implementing the design under field conditions, ensuring quality control and environmental compliance.

Key Tasks and Actions:

- Site preparation: Clearing vegetation, levelling terrain, and installing initial drainage systems.
- Material placement: Ensuring proper compaction and layering of construction materials, particularly for downstream dams.
- Monitoring systems installation: Deploying fiber optic sensors, remote data acquisition systems, and automated inspection tools.

Considerations for Construction Methods:

- Downstream dams require extensive material transport, which can be logistically challenging in remote Amazonian regions.
- Upstream dams need careful monitoring during construction to avoid instability from tailings material [8], [9], [10].

3. ADVANCED TECHNOLOGIES IN TAILINGS DAM ENGINEERING

3.1. Instrumentation and Monitoring

Automated Piezometers: Measure pore water pressure to monitor dam stability.

Inclinometers: Detect lateral movement, critical for early failure warning.

Fiber Optic Sensors: Provide high-precision strain and temperature data, helping identify seepage and deformation.

Satellite-Based Monitoring: Synthetic Aperture Radar (SAR) ensures continuous observation of structural changes, particularly in remote Amazonian locations.

Example Application:

An Amazonian dam project used real-time data from IoT-connected piezometers and inclinometers to detect anomalies during construction. Data integration in a centralized dashboard allowed engineers to adjust the design promptly.

4. HYPOTHETICAL SCENARIOS AND NUMERICAL EXAMPLES

4.1. Scenario 1: Aging Tailings Dam in the Amazon

Description: An existing tailings dam constructed 20 years ago is experiencing seepage and minor deformation during the rainy season.

Proposed Actions:

Instrumentation Upgrade:

Install fiber optic sensors and automated piezometers to monitor seepage and pore water pressure.

Deploy inclinometer arrays to measure deformation trends.

Geotechnical Investigation:

Conduct CPT and borehole sampling to evaluate the dam's current soil properties.

Perform stability analysis using FEA software under varying rainfall scenarios.

Mitigation Measures:

Improve drainage systems with additional relief wells and filter layers.

Reinforce the dam crest with geosynthetics to reduce deformation.

Numerical Example:

- Seepage Rate Before Intervention: 50 liters/hour.
- Expected Reduction After Intervention: <10 liters/hour.
- Estimated Time for Upgrades: 6 months.
- Equipment Required: Fiber optic systems, CPT rigs, drainage installation tools.

4.2. Scenario 2: New Tailings Dam in a Remote Amazonian Location

Description: A new tailings dam is being planned for an area with high rainfall and sensitive biodiversity.

Proposed Actions:

1. **Site Selection**:
    - Use LiDAR and high-resolution GIS tools to identify a stable and environmentally low-impact location.
    - Perform hydrological studies to ensure water flow pathways are unobstructed.

2. **Design and Simulation**:
    - Model dam performance under 100-year rainfall scenarios using dynamic simulation software.
    - Select downstream construction for enhanced stability in wet conditions.
3. **Monitoring Setup**:
    - Implement IoT-enabled monitoring systems with satellite data integration for remote observation.

Numerical Example:

Design Parameters:

- Dam Height: 15 meters.
- Storage Capacity: 1.5 million cubic meters.
- Rainfall Intensity Assumed: 300 mm/day.
- Estimated Construction Time: 2 years.
- Expected Cost: USD15 million.
- Key Equipment: LiDAR, GIS systems, heavy machinery, satellite monitoring systems.

5. CONCLUSIONS

The engineering of tailings dams in Amazonian and non-Amazonian environments requires distinct approaches, driven by environmental and logistical differences. By leveraging advanced technologies and adapting them to the unique conditions of each context, the mining industry can enhance safety, sustainability, and efficiency.

Key Recommendations:

Prioritize high-resolution mapping and environmental assessments during site selection.

Use real-time monitoring systems to integrate data from multiple sources, ensuring proactive decision-making.

Transition to sustainable practices, such as dry stacking, where feasible.

Foster collaboration between stakeholders to address environmental and social priorities.

REFERENCES

[1] Pinto, C. S., & Fonseca, A. S. (2018). "Geotechnical challenges of construction in the Amazon region." Geotechnical Engineering Journal, 42(3), 231-250.

[2] Brando, P. M., et al. (2019). "Fire, deforestation, and climate change in the Amazon." Current Biology, 29(17), R1109-R1115.

[3] Rubel, F., & Kottek, M. (2010). "World Map of the Köppen-Geiger climate classification updated." Meteorologische Zeitschrift, 19(2), 135-141.

[4] Brasil. (2010). Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens. Diário Oficial da União.

[5] Santos, M. & Costa, J. (2022). "Indigenous Consultation Processes in Brazilian Mining Projects." Human Rights Journal of the Americas, 18(2), 157-176.

[6] Silva, L. P., & Ramos, M. P. (2020). "The National Policy on Tailings Dams: Challenges in Implementation." Brazilian Journal of Environmental Management, 15(3), 445-459.

[7] IBAMA (2021). "Guidelines for Environmental Licensing of Mining Projects in the Amazon." Brazilian Environmental Institute. "Rights of Indigenous Peoples: Mining and Consultation Frameworks."

[8] Safety Guidelines and Good Practices for Tailings Management Facilities (ECE 1326665) United Nations Economic Commission for Europe, 2014.

[9] Safety of Dams: Policy and Procedures (ER 1110-2-1156) United States Army Corps of Engineers, 2011.

[10] Tailings Management: Good Practice Guide International Council on Mining and Metals, 2021

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

CONSTRUCTION OF THE SOUTHERN EXPANSION OF THE CERRO DO LOBO WASTE FACILITY

João Manso[1] LNEC
Andrea Brito LNEC
Madalena Barroso LNEC
João Marcelino LNEC
Rui Galrão SOMINCOR
Mafalda Oliveira SOMINCOR

ABSTRACT

Neves-Corvo is a copper-zinc mine producing copper, zinc and lead concentrates. The mine is owned and operated by Lundin Mining's Portuguese subsidiary SOMINCOR. Lundin Mining acquired Neves-Corvo through the merger with EuroZinc Mining in 2006. It is located in the Alentejo district of southern Portugal. The ore is processed on site and the tailings are deposited at the Cerro do Lobo Waste Facility. During the operational phase, the mining waste will be deposited vertically, in stages, from the current five levels to thirteen levels (with an average slope of 5 %). In order to accommodate this process, it has been necessary to expand this infrastructure, including the southern expansion of the tailings dam. The dam will contain waste up to level 8 (at an elevation of 272 m), with a maximum height of 17 m, and walls with slopes of 1(V):2.5(H) upstream, allowing the use of geosynthetics, and 1(V):2(H) downstream. Apart from of the left abutment, the dam's foundation was constructed on the natural terrain. The dam's lining system consists of a transition layer, composed of a sandy material, a geosynthetic clay liner and a high-density polyethylene (HDPE) geomembrane.

This paper provides a detailed description of the methodology employed in the dam's construction, with particular emphasis on the use of soil-rockfill mixtures as the primary building material. It also outlines the specific techniques employed to ensure the quality and durability of the construction.

Keywords: Tailings dam, monitoring system, lining system, geosynthetics, soil-rockfill mixtures.

[1]jmanso@lnec.pt

1. INTRODUCTION

The Neves-Corvo mine is an underground zinc-copper-lead-silver mine situated approximately 200 km southeast of Lisbon, in the western part of the Iberian Pyrite Belt (Fig. 1). The ore is processed on site, and the resulting tailings are deposited at the Cerro do Lobo Tailings Storage Facility (TSF). Recently, apart from the vertical expansion of the mining waste deposit from the current five levels to thirteen levels (with an average slope of 5%), a horizontal expansion of the existing tailings facility was also executed, comprising approximately ten percent of the existing area. This expansion led to the construction of the southern embankment.

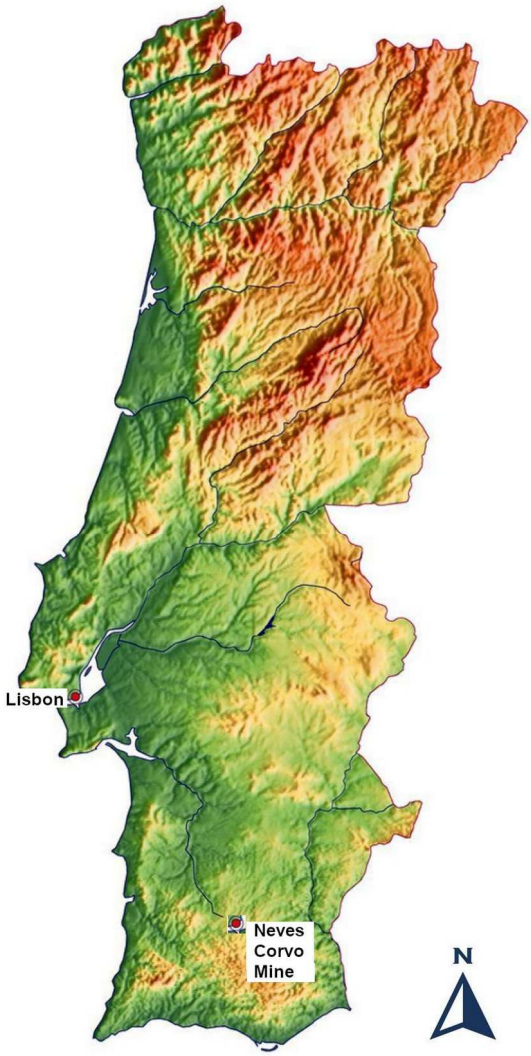


Fig. 1 – Location of Neves-Corvo mine

The southern embankment, whose location is shown in Fig. 2, is intended to contain the tailings up to level 8, at 272 m. It has a maximum height of 17 m (Fig. 3), with upstream and downstream slopes of 1(V):2.5(H) and 1(V):2(H), respectively. The body of the embankment

is composed of materials excavated on site and a liner system was installed on the upstream slope. This system comprises the following elements, from bottom to top:

- a transition layer composed of sandy material, with a thickness of 0.30 m;
- a geosynthetic clay liner (GCL) consisting of a bentonite layer (4.5 kg/m²) sandwiched between two polypropylene geotextiles, one woven (100 g/m²) and one nonwoven (200 g/m²);
- a high-density polyethylene (HDPE) geomembrane, 2.5 mm thick, with smooth surfaces on both sides.

Fig. 2 – Location of the new southern embankment within the Cerro do Lobo TSF (adapted from [1])

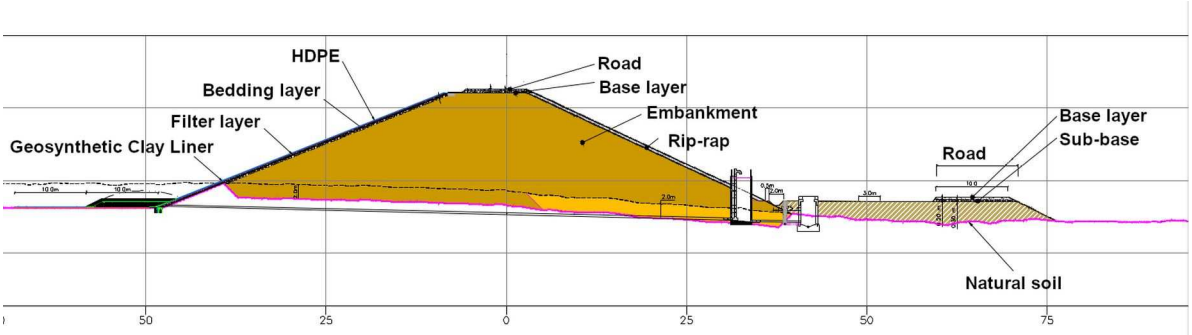


Fig. 3 – Cross-sectional profile of the new southern embankment (adapted from [1])

Fig. 3 shows an orthomosaic of the general view of the construction site at the end of the geosynthetics installation.

Fig. 4 – Orthomosaic of the ICRL, after the installation of the geosynthetics (courtesy of the company TSF)

2. MATERIAL CHARACTERISATION

2.1. Rockfill

The southern embankment is a zoned tailling dam composed by a combination of a rockfill material and a soil-rockfill mixture. This rockfill was obtained from the dismantling of a quarry using explosives. In its particle size distribution, the maximum size was 600 mm, the percentage of material passing the ¾’ sieve was 20 to 30 %, and it had an average percentage of fines of around 3 % (Fig. 5).

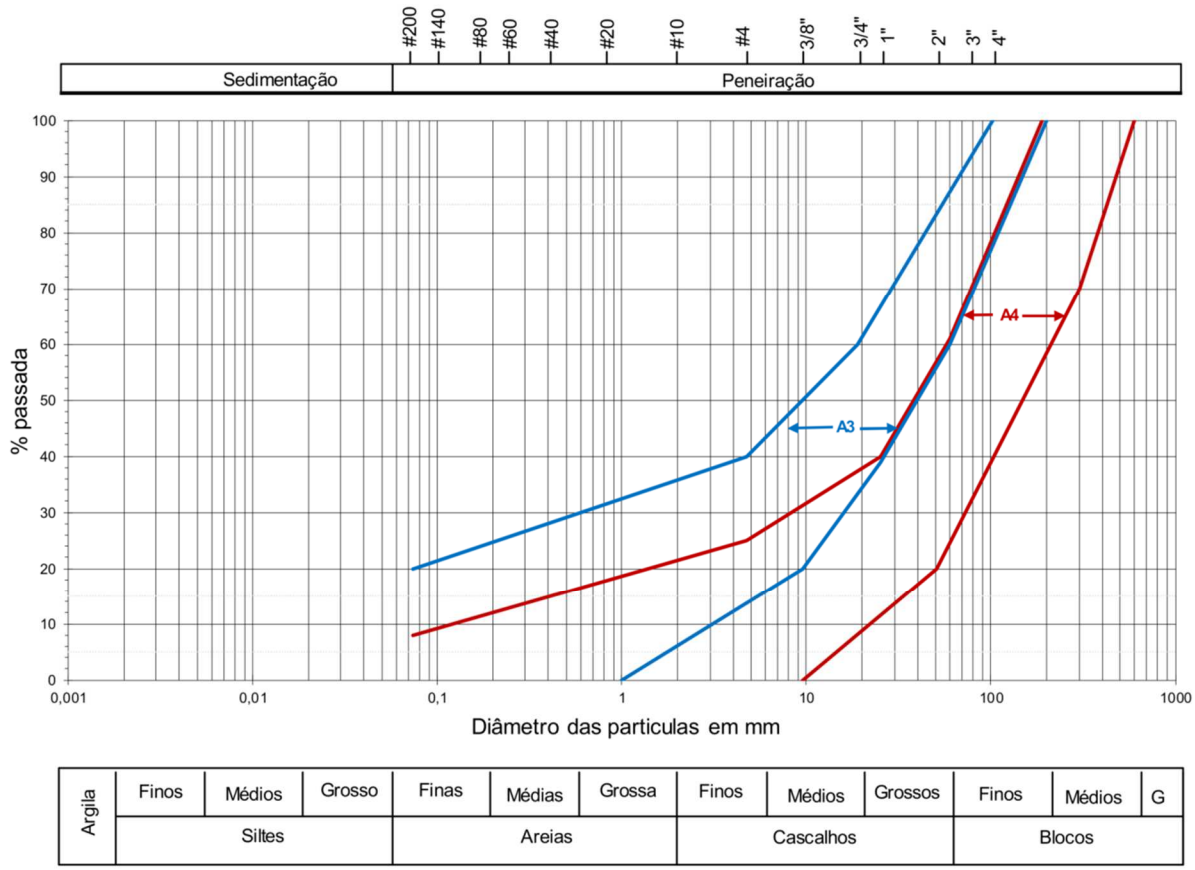


Fig. 5 – Particle size distributions of the rockfill and the soil-rockfill mixtures

Uniaxial compression tests were performed on specimens selected from the deposit, and an average unconfined compressive strength value of 29 MPa was obtained. The porosity values ranged from 9 to 10 % (average of 9.5 % and coefficient of variation of 5.2 %), and the average dry weight of the particles was 23.9 kN/m³ (coefficient of variation of 0.7 %).

The material was a graywacke rockfill, with different degrees of alteration ranging from W_2 to $W_{4\text{-}3}$, and significantly fractured. Considering an empirical classification based on index properties [2], the material generally shows low levels of resistance and is expected to be considerably influenced by water. Hence the importance of considering suitable construction specifications and compacting with abundant wetting, to prevent post-construction deformations.

In order to define the conditions under which the rockfill was applied, during the trial embankments the settlement of the lifts was analysed on eight surface marks arranged along the area of the field compaction control. In addition to this control, five macro tests were performed: one after six passes of the roller, two after eight passes and two after ten passes.

For rockfill, one layer thickness (0.8 metres) and four pass numbers (4, 6, 8 and 10 passes) were tested. During the settlement control, an increase in settlement was observed from

eight to ten passes, so it was decided to make two additional passes, totalling twelve passes. Table **Error! Reference source not found.** presents the results obtained in the macro tests.

Table 1 – Results obtained in the macro tests performed in the field compaction control

N.º of passes	γ_d(kN/m³)	e
6	20.10	0,20
8	20.84	0,14
10	21.11	0,14
12	21.48	0,12

A small particle size evolution was observed when comparing the curves of the material from the deposit with the particle size curves of the samples taken in the macro test, particularly with regard to the amount of fines, which could be attributed to the compaction of the lifts.

In general, an increase in dry density of the lift and a reduction in the void ratio were observed as the number of passes increases.

Despite the low level of resistance exhibited by the rockfill material and the fact that, following the predetermined number of passes of the roller, fines increased up to 10 %, this material was considered suitable for use in the embankments of the southern embankment, provided that suitable construction specifications were taken into account.

2.2. Soil-rockfill mixtures

The other material used in the construction of the southern embankment was a processed material obtained from mechanical excavations in the deposition area. Its particle size distribution corresponded to a soil-rockfill mixture with a percentage of fines of up to 20 per cent (Fig. 5).

Field compaction control during the trial embankments was performed in order to define its application conditions, in particular the thickness of the lifts, the number of passes of the roller, and the applied water content. Two lifts thicknesses (0.4 and 0.6 metres) and two numbers of passes (6 and 8 passes) were analysed. Macro tests were performed to control compaction, whose results are presented in Table **Error! Reference source not found.**.

Table 2 – Results obtained in the control of the field compaction tests for the soil-rockfill mixtures

Lift thickness (m)	No. of passes	γ_d(kN/m³)	w (%)	e
0.4	6	22.71	6.6	0.038
0.4	8	23.91	6.6	0.043
0.6	6	21.05	6.6	0.128
0.6	8	21.95	6.3	0.085

To analyse the results, three modified Proctor tests were performed on the fine fraction, in a large mould, of the soil-rockfill mixture. Table **Error! Reference source not found.** presents the obtained results. Since the results are very similar, their average was considered in the field compaction control.

Table 3 – Results of the macro tests on the field compaction control of soil-rockfill mixtures

	w (%)	γ_d(kN/m³)
	7.7	21.11
	7.3	21.40
	7.4	20.91
Average	7.5	21.14

In order to obtain the degree of compaction and the deviation of the water content of the soil-rockfill mixture, the results obtained in the reference test (modified Proctor in a large mould) were corrected to take into account the presence of coarse materials in the embankment. The following correction formulae were used [3]:

$$\omega_{opt}^{T} = \frac{P_F \omega_F + P_C \omega_C}{100}$$

$$\gamma_{d,máx}^{T} = \frac{100}{\frac{P_C}{\gamma_d^C} + \frac{P_F}{\eta \gamma_{d,máx}^{F}}} \quad (1)$$

where ω_{opt} represents the optimum water content of the whole material, P_F represents the percentage by weight of the fine fraction (material passing the ¾' sieve), ω_F represents the water content of the fine fraction, P_C represents the percentage by weight of the coarse fraction (material retained on the 3/4' sieve), ω_C represents the water

content of the coarse fraction, $\gamma_{d,máx}^{T}$ represents the maximum dry density of the whole material, γ_{d}^{C} represents the dry density of the coarse fraction, $\gamma_{d,máx}^{F}$ represents the maximum dry density of the fine fraction e η represents a correction coefficient given by:

$$\eta = -5 \times 10 - 5P_C^2 + 0{,}0013P_C + 0{,}9958 \quad (2)$$

The deviation in the water content of the mixture and the degree of compaction were analysed based in the macro tests. The deviation in water content varied between -1.13 % and -0.41 %, and the degree of compaction between 97.1 % and 118.91 %. All the samples were over-compacted, with the exception of two samples that had a degree of compaction of 97.3 and 98.5 %, respectively. Based on these results, it was decided that the soil-rockfill mixtures should be compacted with a 20 tonne smooth vibrating roller, in lifts 0.50 cm thick and with 8 roller passes. The water content of the material should be between optimum and optimum +2 % and the degree of compaction to be achieved in the embankment should be greater than 98 % of the modified Proctor test, after correction.

In order to control the compaction of the material, a set of three reference Proctor tests should be performed every week to serve as a reference for the following week. It was decided to perform a sand bottle test, 10 cm from the surface, and, at the same time, a macro test every 10,000 m³ of embankment. In the macro tests, it was stipulated that the dry density should be greater than 21.09 kN/m³.

The first *in situ* results showed that the deviation in water content varied between -0.65 % and 0.79 % and the degree of compaction between 98.9 % and 104.4 %. Therefore the proposed procedure proved to be effective in compacting the embankments with this material.

3. EXECUTION OF THE EMBANKMENTS

The southern embankment was built on the natural terrain, except for the left abutment, where it was partially built in an area where tailings had already been deposited. This meant that these materials had to be completely removed. In the rest of the implantation area, loose materials were removed, to an average thickness of 2 metres. In areas where greater irregularities existed, depressions were filled with mortar or simple concrete and rockfill.

The excavations conducted in the Southern Expansion Area yielded two distinct types of materials: a material from the mechanical excavations (characterised by soil-rockfill mixtures properties) and a material from the explosive excavations (characterised by

rockfill properties). In order to optimise the use of the existing materials, the cross-sectional profile of the tailings dam was designed to incorporate them, as shown in Fig. 5, which also shows the detail of the sub-surface drain connection. This change was made during the construction in order to adjust the design to the actual materials.

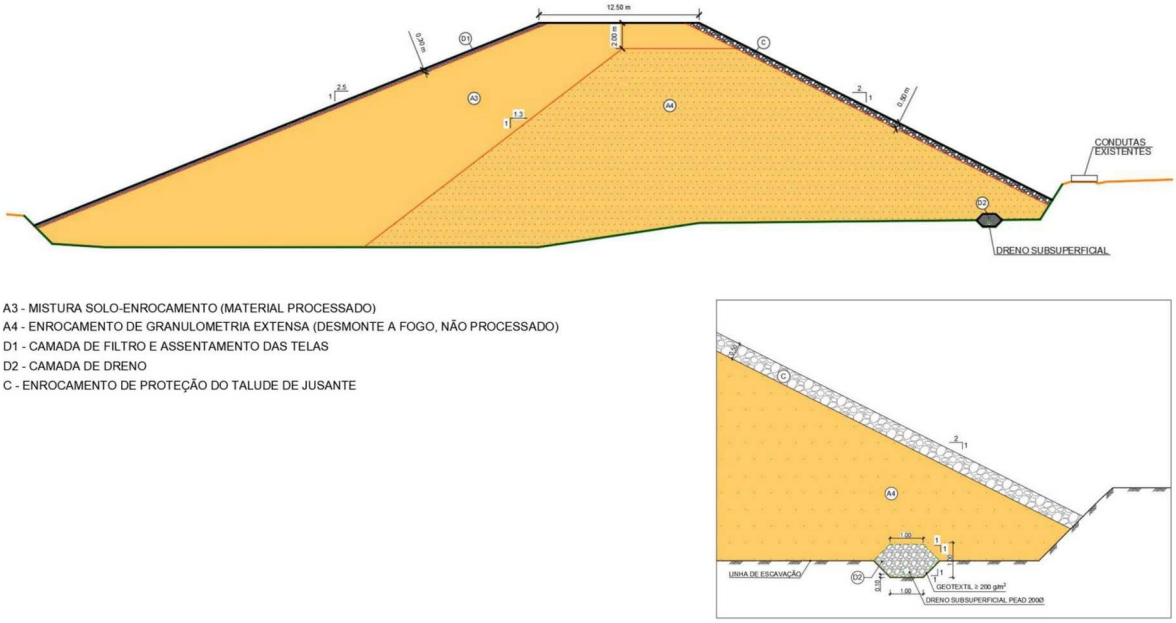


Fig. 6 – Typical cross-sectional profile (adapted from COBA, 2023)

The materials were compacted in parallel at a production rate of 3,000 to 4,000 m³ per day. The process was monitored via sand bottle tests and macro tests at every 10,000 m³. The sand bottle tests were discontinued due to the presence of coarse grains in the soil-rockfill mixtures, which resulted in difficulties being met during the testing process. Subsequently, the control frequency was increased (every 5,000 m³) and replaced by macro tests.

Fig. 7 shows the various stages of embankment construction. From the exploration of the rockfill material and soil-rockfill mixtures inside the Southern Expansion Area (Fig. 7a), to the construction of the embankment with these materials (Fig. 7b), the execution of the transition layer upstream of the embankment to support the bentonite geosynthetic (Fig. 7d and e) and the application of the waterproofing system (Fig. 7f). The waterproofing system for the southern embankment is described in [4].

a)

b)

c)

d)

e)

f)

Fig. 7 – Evolution of the construction of the southern embankment

4. CONCLUSIONS

This paper provides a detailed description of the main activities employed in the construction of the Southern Expansion of the Cerro do Lobo Waste Facility, particularly the southern embankment. Two types of material were used in its construction. A rockfill obtained from the dismantling of a quarry using explosives, and a processed material obtained from mechanical excavations (soil-rockfill mixtures). Decisions were taken to optimise the use of these materials, with benefits both in technical terms (withdrawal of the crushing procedure and adaptation of the embankment cross-sectional profiles), and in economic terms (utilization of the available materials on site). A near-perfect equilibrium of the fill/excavation was achieved at the end of the construction. Particular emphasis was given to their characterisation, including the definition of compaction control procedures and analysis of the results of the field tests carried out. Regarding the soil-rockfill mixtures, in order to obtain the degree of compaction and the deviation of the water content of the lifts, the results obtained in the reference test (modified Proctor in a large mould) were corrected to take into account the presence of coarse materials in the mixture.

REFERENCES

[1] Golder (2021). Projeto de licenciamento da expansão da IRCL para o PEZ (DGEG-LNEC). Volume I - Tomo 1.2. Análise de Consolidação dos rejeitados e de estabilidade. Relatório 1899306-132-RI.2-v2-00.

[2] Pinto, V. and Prates, M. (1997). Aterros em vias rodoviárias. Projecto, construção e comportamento. Sociedade Portuguesa de Geotecnia, ed., 6º Congresso Nacional de Geotecnia. Lisboa.

[3] JAE/LNEC (1994). Controlo de Construção e Caracterização Físico-Mecânica de Aterros de Mistura de Solo-Enrocamento. Proc. 056/1/11505. Fevereiro de 1994.

[4] Barroso, M., Manso, J., Brito, A., Marcelino, J., Galrão, R., and Olviveira, M. (2025). Installation of geosynthetics at the Cerro do Lobo Tailings Storage Facility. EuroGeo 8 – 8th European Regional Conference on Geosynthetics – Lille, France.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

SOUTHERN EXPANSION OF THE CERRO DO LOBO WASTE FACILITY. MECHANICAL CHARACTERIZATION OF SOIL--ROCKFILL MIXTURES

Andrea Brito[1]	João Manso	João Marcelino	Rui Galrão
LNEC	LNEC	LNEC	SOMINCOR

Mafalda Oliveira
SOMINCOR

ABSTRACT

The Southern Expansion of the Cerro do Lobo Waste Facility, inserted in the Neves-Corvo Copper and Zinc Mining Concession, is located in Baixo Alentejo, district of Beja, covering the municipalities of Castro Verde and Almodôvar, Portugal. During the operational phase, the mining waste will be deposited vertically, in stages. At the same time, the deposition area will expand approximately 10%. This enlargement includes the southern expansion of the tailings dam. This dam will contain waste up to level 8 (at elevation 272 m), has a maximum height of 17 m, and its walls have slopes of 1(V):2.5(H) upstream, to allow the application of geosynthetics, and 1(V):2(H) downstream. The body of the dam consists of materials resulting from excavation inside the reservoir (rockfill and soil-rockfill mixtures) and a transition layer (0.30 m thick) to support the materials that constitute the waterproofing system.

Soil-rockfill mixtures (SRM) represent an environmentally friendly material that capitalises on the reuse of the material excavated. Recently, it has been giving particular attention to SRM by the fact that many embankments, constructed with such materials, have presented a poor performance and required, in many cases, the implementation of corrective measures. To improve the knowledge about the mechanical behaviour of soil-rockfill mixtures applied in tailings dams and also to prevent the use of a less capable material behaviour, LNEC, in cooperation with the Dam Owner, carried out an experimental study. This paper presents the results of the performed experimental program, which involved undrained triaxial tests among others.

Keywords: Soil–rockfill mixtures, Hydro-mechanical behaviour, Mechanical tests, Permeability tests.

[1]andreabrito@lnec.pt, Portugal

1. INTRODUCTION

The Cerro do Lobo Tailings Storage Facility (TSF) is an active facility that is part of the Neves-Corvo Mine operation. The operation is owned and operated by Lundin Mining's Portuguese subsidiary SOMINCOR. The operation has been in continuous production since 1988. Located in the Alentejo district of southern Portugal, the operation is situated approximately 15 km southeast of the town of Castro Verde and approximately 200 km southeast of Lisbon.

It is an underground mine, of massive sulfides polymetallic volcanogenic mines, it is the largest in Portugal and also a class mine worldwide due to its abnormally high levels high in Copper, Zinc and Tin.

The processing facility at Neves-Corvo comprises two plants. The copper plant processes copper ores and has a capacity of approximately 2.8 million tonnes per annum (Mtpa). The zinc plant, which can process zinc or copper ores, has recently undergone a significant expansion to a design nameplate capacity of 2.5 Mtpa. Tailings generated from ore processing are either sent to underground workings and used as paste backfill or are deposited in the Cerro do Lobo TSF located about 3 km southeast of the processing plants.

The tailings facility currently includes the main dam, the South Embankment, and five perimeter/secondary dams (MD, PCP, ME1, ME2A-C, and Monte Branco), resulting in a total embankment length of approximately 5.6 km.

Fig. 1 presents the final configuration (after closure) of Cerro do Lobo Tailings Storage Facility Layout.

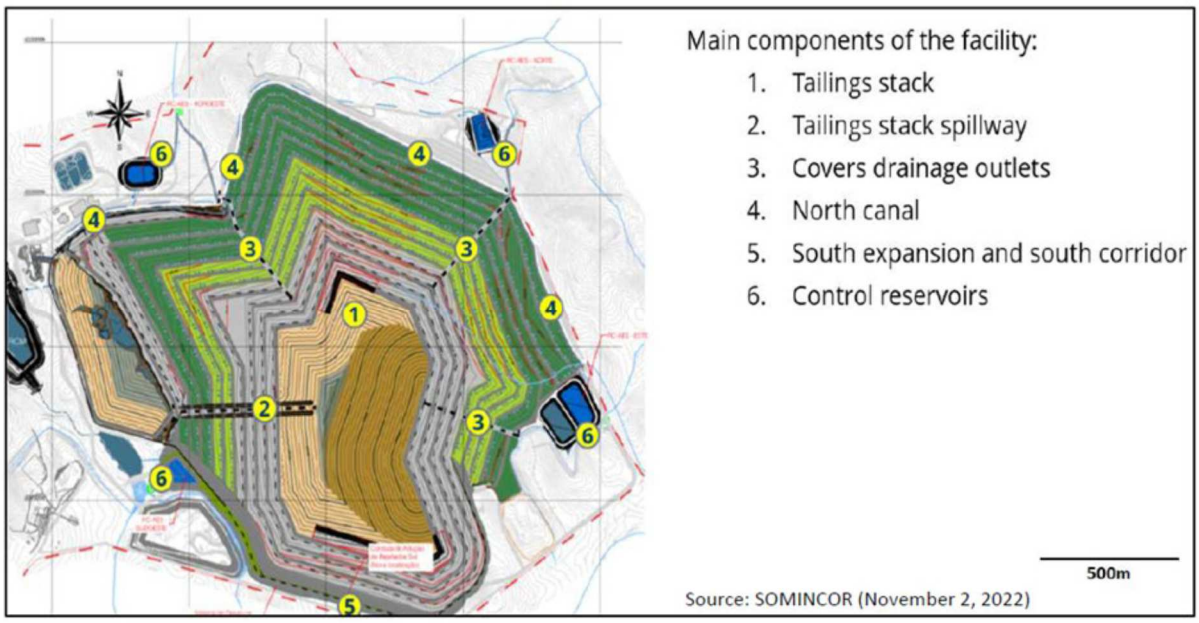


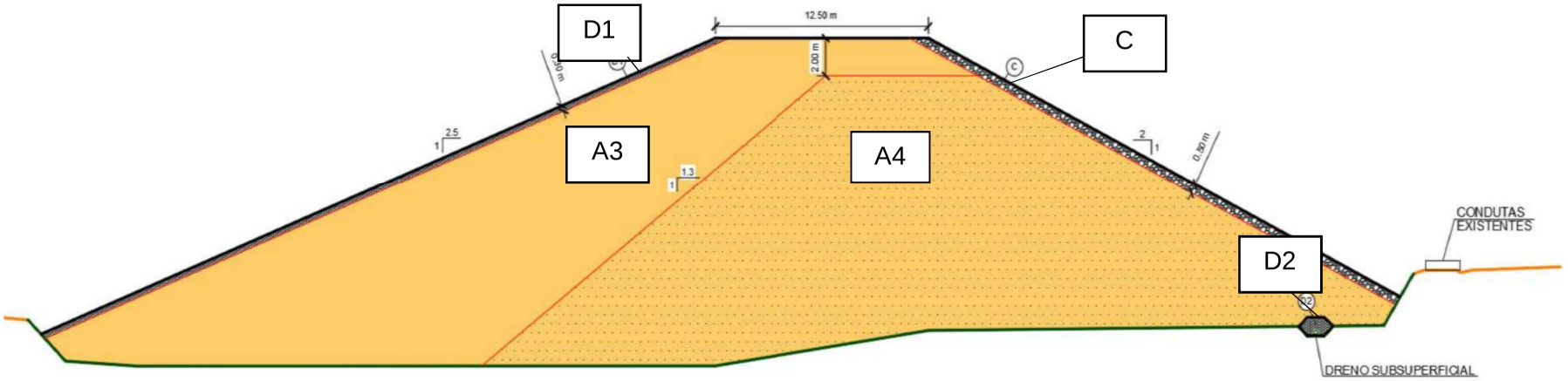
Fig. 1 - Cerro do Lobo Tailings Storage Facility Layout. Final Configuration [1]

In the exploration phase, the disposal of mining waste will be carried out vertically, in a phased manner, from the current 5 levels to 13 levels (with an average slope of 5%). At the same time, the area of Deposition will increase, in plan, by around 10%. Furthermore, it is planned to carry out: i) the Southern Embankment, consisting mainly of rockfill and soil-rockfill mixtures, ii) a new water diversion channel iii) drainage systems, iv) water control reservoirs of surface runoff, v) water management systems from buildings roofs and vi) a new thickened tailings transportation system during the operation phase. In the closing phase, it is planned to carry out the final coverings and the surface water drainage system.

Of the activities mentioned, the embankment works stand out, which include the construction of the Southern Embankment (South expansion in Fig. 1) and a set of perimeter dikes at each of the deposition levels, with an average height of about 2 m, from which the thickened tailings will be deposited. These, after reaching their final level, will first be covered with rubble, to provide conditions for trafficability for the execution of the perimeter dike of the next level, which is expected at around 40 m distance from the previous one, and then with a clean covering, formed by anti-capillarity layers, and finally clean soil.

The Southern Embankment which will contain the tailings up to level 8 (at elevation 272 m), has a maximum height of 17 m, its walls have inclinations of 1(V):2.5(H) upstream, to allow the installation of geomembranes, and 1(V):2(H) downstream (Fig. 2). The body of the dike is made with materials resulting from excavation inside the reservoir and with a transition layer (0.30 m thick) to serve as support for the materials that make up the waterproofing system. The dike was built over the natural terrain, except near the left junction, between Pk 100 and Pk 360, in which it was built, partially, in an area where waste had already been deposited, which resulted in the complete sanitation of materials in the foundation area. Foundation treatment consisted of removing loose materials, in an average thickness of 2 m, when in natural terrain, or the removal of waste, in the remainder of the section, and, in areas of foundation with greater irregularities, filling the depressions with cement mortar, or simple concrete and riprap.

One of the materials used in the construction of the Southern Embankment was a soil-rockfill mixture (SRM – material A3 in Fig. 2). This material represents an environmentally friendly material that capitalises on the reuse of the material excavated. Recently, it has been giving particular attention to SRM due to the fact that many embankments, constructed with such materials, have presented a poor performance and required, in many cases, the implementation of expensive corrective measures. To improve the knowledge about the mechanical behaviour of soil-rockfill mixtures applied in tailings dams and also to prevent the use of a less capable material behaviour, LNEC, in cooperation with the Dam Owner, carried out an experimental study. In the next points the main results obtained are presented.



A3 - Soil - rockfill mixture

A4 – Rockfill material

D1 - Filter layer

D2 – Drain layer

C – Rockfill protecting the downstream face

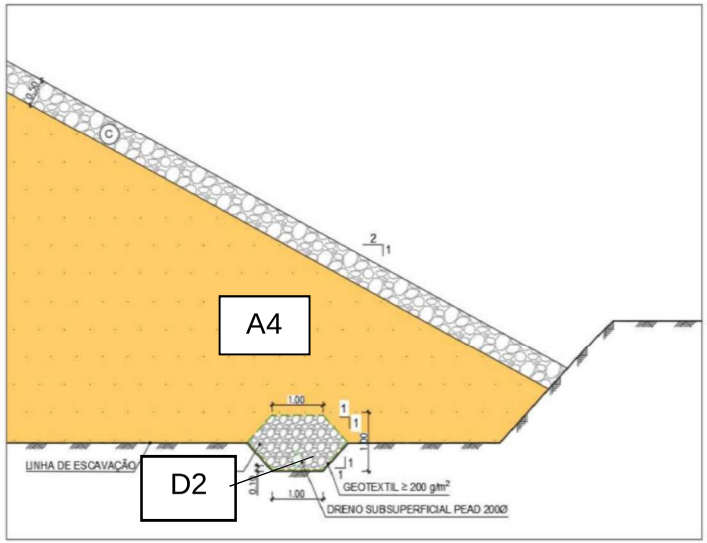


Fig. 2 – Cross section of Southern Embankment [2]

2. HYDROMECHANICAL CHARACTERIZATION OF SOIL-ROCKFILL MIXTURES

2.1. Introductory note

With the soil-rockfill mixture material collected in the borrow areas, identification, mechanical and hydraulic characterization tests were carried out, namely undrained triaxial tests and permeability tests in a triaxial chamber.

The following points present the results obtained in these tests.

2.2. Index Properties and Identification tests

The construction of the embankment for the IRCL Southern Expansion started in April 2022. Initially, the highest-quality A1 material, sourced from the Cerro da Mina reservoir (Deposit 4.2), was utilized after undergoing processing. In May 2022, material processing from mechanical excavations in the Southern Expansion Area began, and these materials were also employed in the construction of the embankments. From the execution of macro tests, the results presented in Table 1 were obtained, for the materials from Cerro da Mina (material A1) and Deposit 4.2.

Table 1 – Summary of results in macro tests carried out at embankments [2]

Sample	w_P (%)	w_L (%)	γ_d (kN/m^3)	n (%)	LA (%)
LZ099-22	10	17	24.04	9.58	39
LZ108-22	17	24	24.35	8.84	40
LZ098-22	18	25	23.57	10.56	42
LZ100-22	17	23	23.54	10.2	42
LZ109-22	15	23	23.55	14.42	42

Taking in consideration the materials available on site, two new materials were considered Type A3 – materials coming from excavations using mechanical means, processed, and Type A4 – materials coming from excavations using explosives, unprocessed, whose results are presented in Table 2 and Table 3, respectively.

2.3. Permeability tests

After the saturation phase during the triaxial compression tests, permeability tests were carried out. The values of the permeability coefficient determined in the saturation phase for each of the stresses levels applied in the consolidation phase ($\sigma_{consolidation}$) are presented in Table 4.

As can be seen, the material is very impervious with an average permeability of 5.5×10^{-10} m/s. Thus is a permeability very different from the usual permeability of rockfill materials, and it is related to the soil material in the mixture which was a very low permeability. But it may be a normal permeability value for soil-rockfill mixtures.

Table 2 – Summary of results in macro tests carried out with A3 material [2]

Sample	w_P (%)	w_L (%)	γ_d (kN/m^3)	n (%)
LZ177-23	18	25	23.49	11.79
LZ178-23	-	-	23.59	11.66
LZ195-23	17	23	23.57	9.84
LZ197-23	17	24	23.89	9.78
LZ179-23	19	26	23.72	11.73
LZ187-23	-	-	23.85	10.57
LZ199-23	17	23	23.63	11.35

Table 3 – Summary of results in macro tests carried out with A4 material [2]

Sample	w_P (%)	w_L (%)	γ_d (kN/m^3)	n (%)	LA (%)	PLT resistance (MPa)
LZ142-22	-	-	24.12	9.65	42	0.92
LZ143-22	20	26	23.67	9.98	45	0.80
LZ144-22	18	25	23.81	9.09	45	0.77
LZ145-22	-	-	23.97	9.96	-	-
LZ150-22	18	25	23.99	8.9	-	-

Table 4 – Permeability results for the soil-rockfill mixture

Sample	$\sigma_{consolidation}$ (kPa)	k (m/s)
5927	125	2.4×10^{-10}
	250	6.0×10^{-10}
	500	8.3×10^{-10}
Average		5.5×10^{-10}

2.4. Gradation curves

Before and after each sample triaxial shear, the particle size curves of the material were determined. In Fig. 3 it is possible to see the gradation curve before triaxial shear and the curves for each consolidation stress after shear, as well as the upper and lower limits of the

granulometric range interval of the material used in the construction of the Southern Embankment.

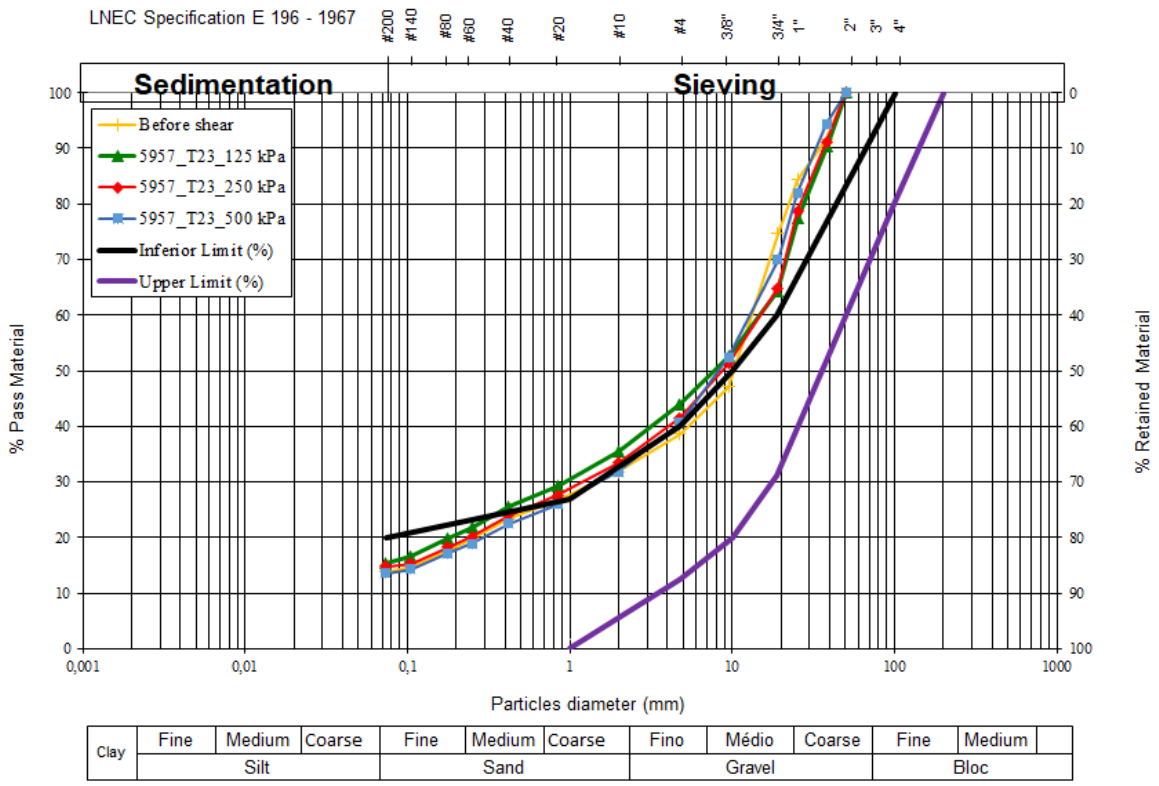


Fig. 3 – Gradation curves before and after triaxial shear

As can be seen, there is the occurrence of an evolution of the particle size for all samples tested with the compaction, with the effective mean stress and with the shear stress. According to [3] this process is more pronounced for mixtures with more coarse material, which presented the most evolutional behaviour. This is probably due to the fact that this material presents higher particles contact forces.

2.5. Undrained triaxial tests

The tests performed are undrained consolidated tests (CU), so the compression of the specimens was carried out without drainage and with measurement of the pore pressure developed in the shear phase. Due to the maximum particle size of the material, the tests were carried out in larger triaxial chambers called T23. The diameter of the samples is 22.8 cm, and the height is about 47 cm.

The results obtained from the undrained triaxial tests on sample 5957 are presented in Fig. 4. The evolution curves of the deviatoric stress (q), excess pore pressure (u) and the ratio (q/p') between the deviatoric stress and the mean effective stress (p') as a function of the variation in axial deformation (ε_a) are presented, as well as the respective stress path. After the

saturation phase, during the triaxial compression tests, permeability tests were carried out. The average permeability coefficient value determined at the end of the saturation phase is presented, together with the resistance values expressed in terms of p' and q, as well as the slope of the line (q, p´) defined by rupture and the shear resistance angle (ϕ_c') in Table 5 .

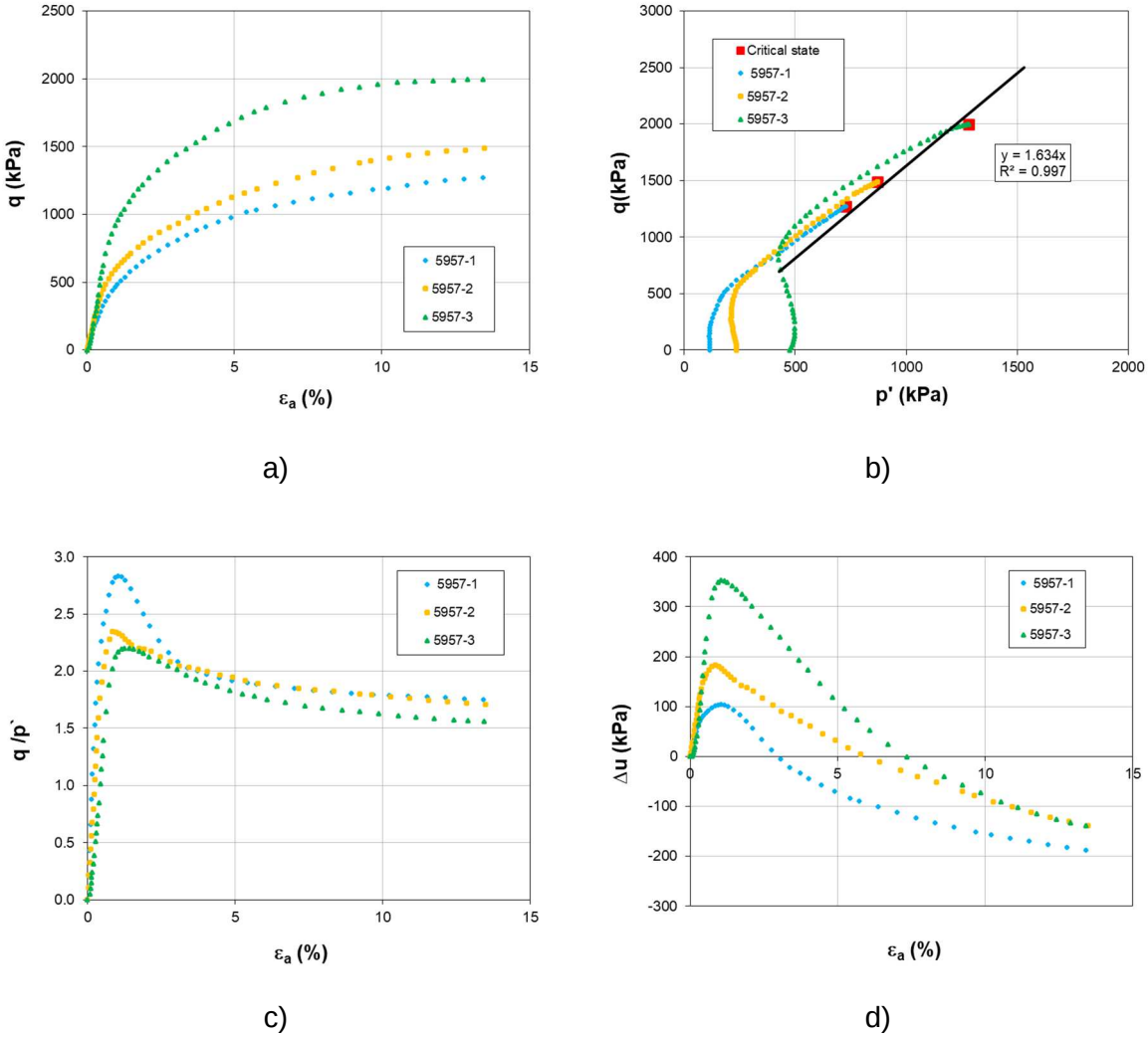


Fig. 4 - Results of the triaxial shear test for sample 5957: a) q versus ε_a; b) stress path; c) q/p' versus ε_a and d) Δu versus ε_a

Table 5 – Summary of results of the undrained triaxial test of 5957 sample

Sample	$\sigma_{consolidation}$ (kPa)	k (m/s)	p_c' (kPa)	q_c (kPa)	M	ϕ_c' (º)
5957	125	5.53×10^{-10}	726.22	1272.53	1.634	40.0
	250		870.39	1487.72		
	500		1280.14	1998.00		

All samples present the same type of behaviour: the effective stress paths show an inversion in direction and the samples exhibits a gradual transition in rigidity (reduction) when going from elastic to elastoplastic behaviour.

This material presented a shear resistance angle, in the critical state, of 40º which is a high value, showing that this material is mechanically quite resistant.

Several studies had concluded that the rockfill failure envelope is curve ([4], [5], [6], [7], among others). So, in order to verify if the SRM of the Southern Embankment also present this type of envelope the (p', q) values at the end of the tests are presented in Fig. 5. It was assumed that all the samples reached the critical state in order to be able to plot these values in this figure.

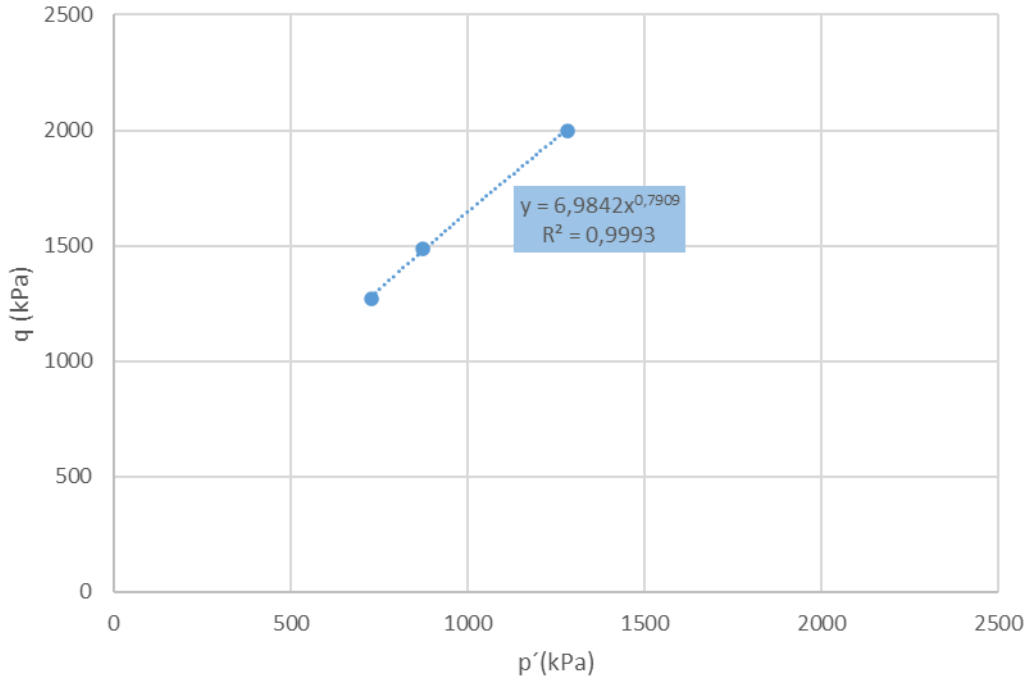


Fig. 5 - Failure values for all the samples of the SRM tested

One of the expressions used for rockfill in order to approximate the curved failure envelope results from a series of tests performed at different confining pressures has the form [6]:

$$\tau_f = A(\sigma')^b \quad (1)$$

where τ_f is the shear stress at failure and σ' is the corresponding normal effective stress.

In the same figure the expression that showed a better adjustment to the results for all the SRM is displayed considering that in (p',q) space the best fitting expression for this type of material is also a power. According to [3] the curved behaviour becomes more important with the increase of the coarse fraction value present in the mixture.

As can be seen in Fig. 5 the expression that better fits the results for the SRM used in the construction of Southern Embankment is:

$$q=6{,}984(p')^{0{,}791} \quad (2)$$

The good fitting of the results in (p',q) space, for the mixture tested, shows that this type of material also presents the same typical behaviour of rockfill, this is, the failure envelope is curved.

3. CONCLUSIONS

The Cerro do Lobo Tailings Storage Facility is an active facility that is part of the Neves-Corvo Mine operation. The operation has been in continuous production since 1988. Located in the Alentejo district of southern Portugal, the operation is situated approximately 15 km southeast of the town of Castro Verde and approximately 200 km southeast of Lisbon.

During the operational phase, the mining waste will be deposited vertically, in stages. At the same time, the deposition area will expand approximately 10%. This increase includes the construction of Southern Embankment. The body of this dike consists of materials resulting from excavation inside the reservoir (rockfill and soil-rockfill mixtures).

One of the materials used in the construction of the Southern Embankment was a soil-rockfill mixture. This material represents an environmentally friendly material that capitalises on the reuse of the material excavated. To improve the knowledge about the mechanical behaviour of soil-rockfill mixtures applied in tailings dams and also to prevent the use of a less capable material behaviour, LNEC, in cooperation with the Dam Owner, carried out an experimental study. In this paper the main results obtained were presented.

The studies showed that there is the occurrence of an evolution of the particle size for all samples tested with the compaction, with the effective mean stress and with the shear stress. Also, all samples present the same type of behaviour in the undrained triaxial tests: the effective stress paths show an inversion in direction and the samples exhibits a gradual transition in rigidity when going from elastic to elastoplastic behaviour. This material presented a shear resistance angle, in the critical state, of 40º which is a high value, showing that this material is mechanically quite resistant. The good fitting of the results in (p',q) space, for the mixture tested, shows that this type of material also presents the same typical behaviour of rockfill, this is, the failure envelope is curved.

Thus, the studies carried out at LNEC made it possible to verify that the soil-rockfill mixture (material A3) used in the work has good characteristics to be used as a dam construction material which enabled savings as the material did not all need to be processed as it was initially.

REFERENCES

[1] Lundin Mining. (2024). GISTM Public Disclosure Report. Neves-Corvo Mine, Cerro do Lobo Tailings Storage Facility. October 2024.

[2] COBA. (2023). Alteração da constituição do aterro do dique de confinamento e conformação dos aterros construídos até à data. Nota técnica 40598-PE-ES-MT-NT-0009-R(A). COBA.

[3] Brito, A. (2015). Use of soil-rockfill mixtures in embankment dams, PhD Thesis, IST.

[4] Charles, J. A. and Watts, K.S. (1980). The Influence of Confining Pressure on the Shear of Compacted Rockfill. Geotechnique, Vol. 30, No. 4, pp. 353-367.

[5] Veiga Pinto, A. (1983). “Previsão do Comportamento Estrutural de Barragens de Enrocamento”. Tese para obtenção do grau de especialista, Lisboa, LNEC.

[6] Charles, J. A. (1990-a). Laboratory Compression Tests and the Deformation of Rockfill Structures. Advances in Rockfill Structures, NATO ASI Series, Serie E: Applied Sciences- Vol.200, Chapter 4, pp. 53-72.

[7] Charles, J. A. (1990-b). Laboratory Compression Tests and the Deformation of Rockfill Structures. Advances in Rockfill Structures, NATO ASI Series, Serie E: Applied Sciences- Vol.200, Chapter 5, pp. 73-96.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

APPLICATIONS OF INSAR TECHNOLOGY IN DAM MONITORING: A SYSTEMATIC LITERATURE REVIEW

Jailson S. Alves[1]
Federal University of Pernambuco (UFPE)

Roberto Q. Coutinho[2]
UFPE

Antonio M. Ruiz-Armenteros[3]
University of Jaén

Hanna B. de A. F. Moreira[4]
UFPE

Wilson R. Aragão Júnior[5]
UFPE

ABSTRACT

Dams play a crucial role in water resource management, essential for water supply, irrigation, flood control and hydroelectric power generation, and driving socio-economic development in various regions. To ensure their structural and operational stability, continuous monitoring of various parameters such as displacements, pore pressure, infiltration, water levels, and temperature is essential. This monitoring allows early detection of signs of instability, enabling preventive actions to be taken and the necessary maintenance to be carried out to avoid catastrophic failures. Although traditional technologies such as extensometers and inclinometers are still widely used, more advanced methods have been gaining prominence in recent years, offering a more detailed and accurate analysis. One of these technologies is Interferometric Synthetic Aperture Radar (InSAR), a technique that allows large-scale monitoring of surface displacements using radar images obtained by satellites, aircraft, or ground platforms. This study aims to evaluate the capabilities of InSAR in dam monitoring, using a bibliometric analysis of scientific publications between 2018 and 2022, extracted from the Scopus database. In addition, exemplary cases were selected from the literature to illustrate the practical applications of InSAR, such as the detection of displacements and the assessment of critical conditions for the structural integrity of dams. The results indicate that InSAR has proved to be an indispensable tool for increasing the accuracy and efficiency of monitoring, contributing significantly to the safety, maintenance, and longevity of these infrastructures. The study also identifies emerging trends, key institutions, and relevant authors in the field, reinforcing the importance of InSAR as an essential technology for risk management in dams.

Keywords: Monitoring, InSAR, Dam Safety.

[1]jailson.alves@ufpe.br, Brazil
[2]roberto.coutinho@ufpe.br, Brazil
[3]amruiz@ujaen.es, Spain
[4]hanna.bfalcao@ufpe.br, Brazil
[5]wilson.ramos@ufpe.br, Brazil

1. INTRODUCTION

The safety of dams throughout their useful life is a crucial aspect, requiring continuous vigilance and rigorous monitoring of deformations to guarantee structural stability and prevent failures that could compromise their integrity [1]. Monitoring the health of the dam is essential to reduce risks, enabling preventive and corrective interventions. This approach minimizes accidents and guarantees structural safety and the safety of adjacent areas [2].

Deformations in dams occur due to horizontal stresses, mainly generated by hydrostatic pressure, and vertical stresses, associated with the structure's weight. These displacements result from factors such as the stored water load, construction characteristics, and geological conditions of the site [3]. In this sense, remote sensing, especially SAR (Synthetic Aperture Radar) technology, offers advanced tools for monitoring, allowing precise and continuous detection of structural deformations and changes in the surroundings, even in areas that are difficult to access [4].

Interferometric Synthetic Aperture Radar (InSAR) is an effective technique for mapping ground movements with high resolution and millimeter precision [5]. This tool allows continuous monitoring, even in areas that are difficult to access, helping with the early detection of anomalies and the adoption of preventive measures to ensure structural safety.

This study analyzes the capabilities of InSAR technology in dam monitoring, based on a bibliometric survey of scientific publications (2018-2022) on the Scopus database. The analysis mapped relevant trends, institutions, and authors, highlighting the potential of InSAR for accurate and efficient structural monitoring, as well as discussing advances in interferometric techniques. The results show the importance of this technology for improving dam safety and maintenance.

In addition, this work explored articles that discuss various applications of tailings, earth, and concrete dams. These analyses highlighted InSAR's ability to provide detailed data for monitoring, highlighting its relevance as a management and safety tool in dam engineering projects.

2. METHODOLOGY

The methodology adopted in this study follows a systematic approach, composed of different phases and stages, aiming to comprehensively investigate InSAR technology for dam monitoring applications. The methodological process was structured into three distinct phases, namely: (1) Database search, (2) Meta-analysis and (3) Systematic data analysis, as illustrated in the flowchart presented in Fig. 1 [6].

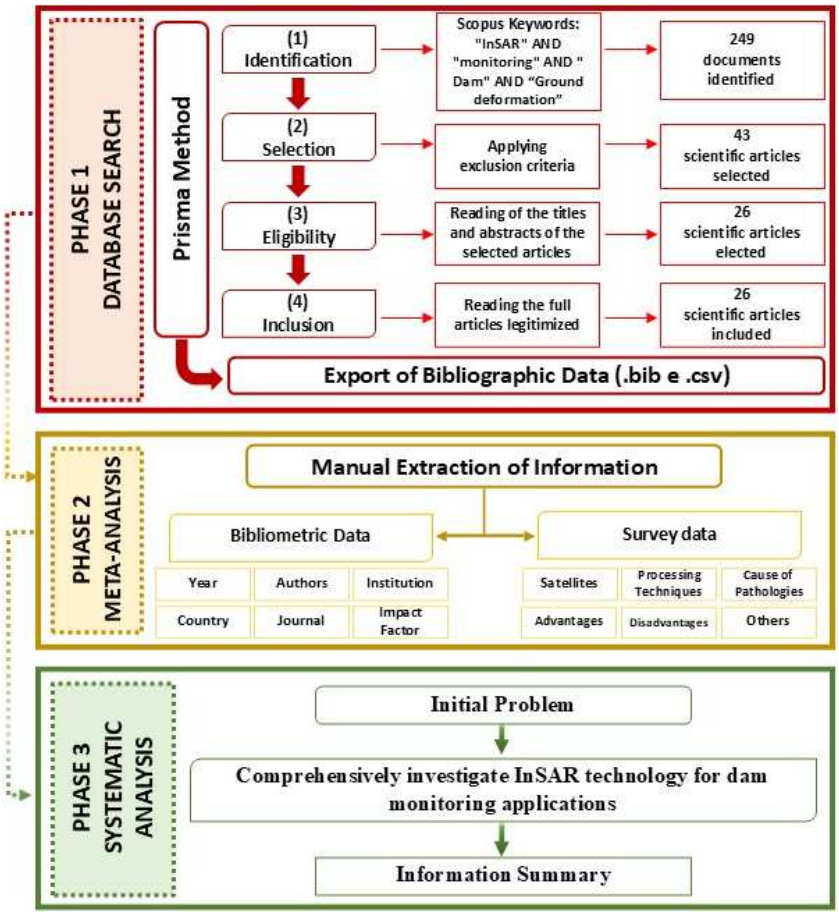


Fig. 1 – Systematic research flowchart [6]

2.1. Phase 1: Search for articles in the database

In the first phase, articles were searched in the database using the guidelines of the PRISMA Method (Preferred Reporting Items for Systematic Reviews and Meta-Analyses), which includes the stages of Identification, Selection, Eligibility and Inclusion [7]:

- Identification – a search was conducted in February 2023 in the Scopus database, through the Periodicals Portal of the Coordination for the Improvement of Higher Education Personnel (CAPES, in Brazilian Portuguese acronym), using the keywords in English "InSAR" and "monitoring" and " Dam" and “Ground deformation”, which should have been present in the title, abstract or keywords of each scientific article. At this stage, 249 documents were found.
- Selection – the papers that were found from the search were submitted to the Selection stage, where exclusion criteria were applied, including type of document (article), language (English), type of source (newspaper), publication period (from 2018 to 2022) and publication stage (final). This resulted in a reduction to 43 scientific articles.

- Eligibility – the titles and abstracts of the articles were independently evaluated by two people, to select those that were aligned with the central theme analyzed. At this stage, 26 articles were selected for the next phase.
- Inclusion – the previously selected articles were read in full and evaluated by two independent people to determine their inclusion in the systematic review. This resulted in a final sample of 26 articles.

Finally, the article information was exported from Scopus in bib and .csv formats, allowing the analysis by the bibliometrics software R Studio (Bibliometrix package).

2.2. Phase 2: Meta-analysis

Initially, the meta-analysis was carried out based on a table manually created in Microsoft Office Excel software, which contained data from the articles. This data was classified into two categories: bibliometric information from publications and data related to research. The bibliometric information of the publications included the year of publication, the authors, the authors' institutions and countries of origin, the journals and the impact metrics of these journals. Data related to the research relates to the following: the satellites and characteristics of the images used; information on processing – time series period, number of images, software, processing technique; advantages; disadvantages and limitations. Subsequently, the data was synthesized and organized according to the main topics and results identified in the included studies, allowing a comprehensive and structured analysis of the information collected.

2.3. Phase 3: Systematic analysis

In the last stage, the systematic analysis was conducted using software such as Microsoft Office Excel and R Studio (Bibliometrix package), using descriptive statistics to create graphic elements that illustrated the organized and exported information. It is essential to highlight that these tools were used exclusively to process the collected data, while the interpretation and analysis of the articles were carried out by the researchers. The results obtained from the meta-analysis were subsequently summarized and interpreted considering the objectives of the review, providing a concise and analytical exposition of the results and conclusions of the analyzed research. After bibliometric analysis, representative articles were selected that explore the capabilities of InSAR as an advanced tool in dam monitoring. These studies highlight the effectiveness of the technique in detecting displacements and identifying areas of fragility in dam structures, providing essential information for the early detection of anomalies and the continuous monitoring of surface deformations.

3. BIBLIOMETRIC RESULTS

After applying all the eligibility steps and filters, the results of the searches on the Scopus platform, related to the topic of dam monitoring using InSAR technology, are shown in Fig. 2. Over the past five years, the number of publications has reached a total of 26 articles, highlighting the ongoing relevance of the topic. While the years 2018 and 2019 recorded only two articles each, the years 2020 and 2021 added up to eight publications. It should be noted that the number of articles in 2022 may not reflect the total for the year, given that the consultation was carried out in September 2022.

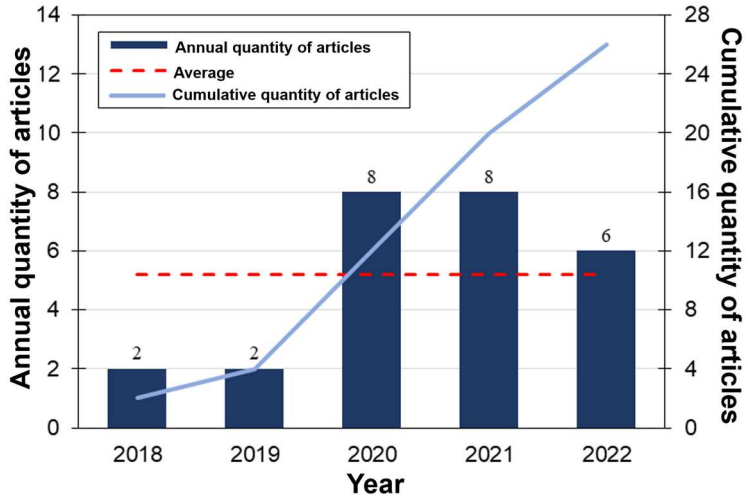


Fig. 2 - Evolution of the number of publications in the Scopus database (2018 to 2022)

The analysis of the results of indexing with registration of institutions around the world, shown in Table 1, identifies the universities and their respective countries. A total of 60 institutions were reported, but only those with at least two articles published on the subject were selected. These institutions account for 22.06% of the total volume of publications. Wuhan University in China stands out, with three articles published, corroborating the Chinese predominance in the results, which total seven publications. The United Kingdom also had six publications, while Germany contributed two.

Table 1 – Main institutions with publications on the application of InSAR in dams

Institutions	Country	Quantity of articles
Wuhan University	China	3
Central South University	China	2
Chang'an University	China	2
Durham University	United Kingdom	2
Friedrich Schiller University Jena	Germany	2
Newcastle University	United Kingdom	2
University of Nottingham	United Kingdom	2

Another relevant piece of information related to the publications is the number of citations received by the articles. Of the 26 articles selected, the total number of citations was 271. Fig. 3a highlights the 15 most cited articles, with emphasis on the work [8], which recorded 36 citations, representing 13% of the total indexed on the Scopus platform. In Fig. 3b, the data is organized according to the time of publication, showing that most of the most cited articles also stand out in this classification.

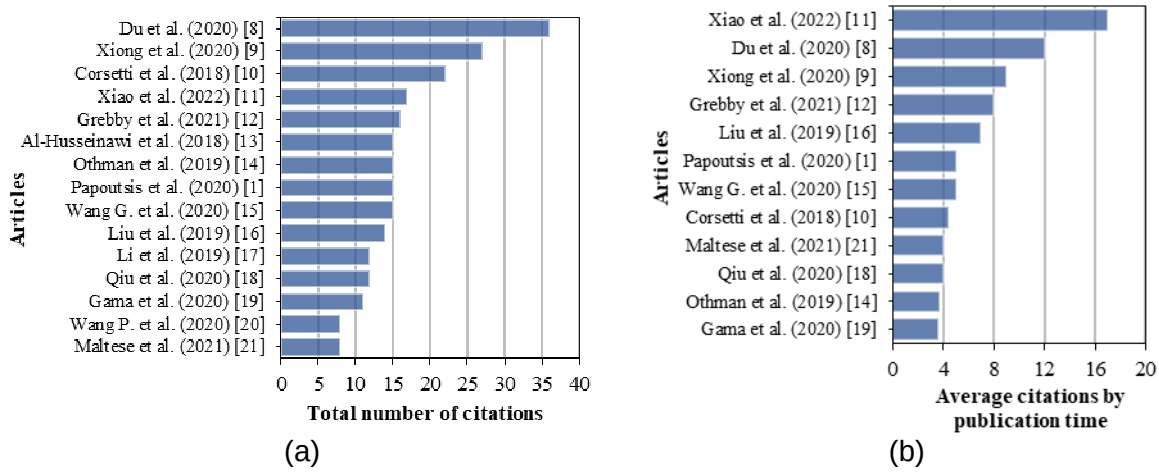


Fig. 3 – Analysis of citations of articles on the application of InSAR in dams: (a) 10 highest number of citations, 15 articles; (b) 10 highest average citations by publication time, 12 articles

The journals in which the articles were published are listed in Table 2, considering those with at least two selected publications. In total, 10 journals were identified, with the four journals highlighted accounting for 61.5% of the articles published. The journal Remote Sensing leads the way with 10 articles selected, corresponding to 38.46% of the total, showing its relevance and technical qualification in the scientific environment. The other journals presented two articles each, maintaining a satisfactory performance according to the performance and quality indicators.

Table 2 – Key Journals with Impact Metrics

	Journal	Distribution of articles		Impact metrics	
		Quantity	Contribution	CiteScore 2021	Journal Citation Reports (JCR) 2021
1	Remote Sensing	10	38.46%	7.4	5.349
2	Bulletin of Engineering Geology and the Environment	2	7.69%	5.8	-
3	Energy Geoscience	2	7.69%	-	-
4	Sensors (Switzerland)	2	7.69%	6.4	3.847
	Total	16	61.54%	-	-

4. LITERATURE CASES

Among the articles analyzed in the bibliometric survey, representative studies were selected that highlight the capabilities of InSAR as an advanced tool in dam monitoring. These papers highlight the effectiveness of the technique in analyzing displacements and identifying zones of fragility, providing crucial information for the early detection of anomalies and the continuous monitoring of surface deformations.

The cases presented below illustrate how InSAR has contributed significantly to the control and mitigation of geotechnical risks, enabling more effective management of hazards associated with structural instability. In addition to its application in technical analysis, InSAR has proved indispensable in formulating preventive strategies and reducing vulnerabilities, consolidating itself as an essential resource for dam safety.

4.1. Study of the application of InSAR in tailings dams

The monitoring of tailings dams, especially in Brazil, has gained prominence following tragedies such as the collapse of the Brumadinho dam in Minas Gerais on January 25, 2019 [19]. Traditional geotechnical instrumentation, such as piezometers, inclinometers and accelerometers, has been fundamental in this process. However, the use of InSAR stands out as an important tool for long-term monitoring, enabling the detection of displacements and subsidence, especially in risk areas such as slopes and dam bodies.

In the study [19], the application of InSAR was crucial to analyzing displacements in the Brumadinho dam before its collapse. The authors used Differential InSAR (DInSAR) data with the Small Baseline Subset (SBAS) and Persistent Scatterer Interferometry (PSI) techniques, using 26 images from the Sentinel-1B satellite between March 2018 and January 2019. The analysis revealed a gradient of displacement in the tailings, with the highest deformation values found in the lower areas of the dam, which helped to identify zones of fragility and the evolution of displacements over time.

The SBAS technique detected maximum accumulated displacements of up to -39 mm at the top of the dam and up to -79 mm in the tailings, with a deformation rate of -92 mm/year. The PSI analysis indicated displacements of up to -70 mm in the tailings, with a rate of -78 mm/year. The comparison between the results of both techniques showed that InSAR was essential for monitoring the evolution of displacements, with the PSI technique identifying greater variations in displacement compared to SBAS, especially in the last 40 days before the collapse.

In addition, the combination of displacement data with rainfall information revealed a clear influence of rain on the acceleration of movements, confirmed that climatic factors can aggravate deformation in dams. This study highlights the importance of InSAR as an effective

tool for monitoring tailings dams, providing valuable information for the early detection of deformations and disaster prevention.

4.2. Study of the application of InSAR to earth dams

The study carried out by [10] demonstrated the importance of the InSAR technique in the long-term monitoring of displacements in an earth dam in the Murge region of southern Italy. Using the DInSAR technique, the authors analyzed displacement data between 1992 and 2007, associated with information from in situ equipment such as strain gauges and bubble level gauges. The 88-meter-high dam with a low-permeability core was monitored over time to understand the evolution and spatial distribution of deformations in the body of the structure.

The InSAR technique proved essential in detecting maximum vertical displacements of up to 240 mm, mainly at the crest of the dam, while measurements at the bottom of the structure were insignificant [10]. The maximum speed of displacement was 15.5 mm/year at the crest, which indicates a concentration of settlements in this area, suggesting that the dam was giving way.

By comparing the results obtained by InSAR with data from strain gauges and bubble level gauges, the authors found a good correlation between the interferometric measurements and the in-situ instruments, which validated the accuracy of the monitoring carried out by the satellite. This comparison highlights the effectiveness of InSAR as a complementary and essential tool for long-term monitoring, especially in periods when in situ measurement data is limited.

The research [10] concludes that the SBAS-DInSAR technique not only reliably reproduced the deformation behavior of the dam, but also provided valuable data for retro analysis and the calibration of numerical models. The use of InSAR, with its ability to provide long and accurate time series, has proved to be an indispensable tool for the continuous monitoring of dams, assisting in risk management and decision-making for structural safety.

4.3. Study of the application of InSAR to concrete dams

The study by [22] looked at the integration of displacement measurements obtained by PSI and the Global Navigation Satellite System (GNSS) for monitoring a concrete dam on the Sabor river in Portugal. The dam, which is 123 meters high and has a crest length of 505 meters, has been in operation since 2016. The research used 59 SAR images from the Sentinel-1A satellite, in interferometric-wide (IW) mode, between June 2016 and May 2018. The June 2017 image was selected as a reference to minimize the perpendicular and temporal baselines, allowing for a precise analysis of displacements.

Three GNSS antennas were permanently installed on the crest of the dam, allowing relative displacements to be measured about a reference antenna installed 500 meters away in a stable location. The study analyzed 174 InSAR PS points, 17 of which were located on the dam crest. The combination of InSAR and GNSS data made it possible to adjust the monitoring network, reducing errors typical of the PSI technique, such as the unwanted movement of reference points and the effects of atmospheric debris.

Validation of the displacement data was carried out by comparing the adjusted data with the GNSS reference measurements, resulting in standard deviations ranging from 1.6 to 2.5 mm. The sensitivity analysis revealed that the detection of vertical displacements was consistent along the crest of the dam, while the sensitivity to radial displacements varied, with the highest sensitivity observed on the left bank of the dam. On the right bank, sensitivity was low, which limited the detection of anomalies in that zone. The results showed that the integration of InSAR and GNSS measurements allowed for more accurate monitoring of the dam, mitigating errors and obtaining information on displacements along the Line-of-Sight (LOS).

5. CONCLUSIONS

Dam monitoring using InSAR technology has established itself as an essential tool for detecting ground deformations and movements with high precision. The technology offers an efficient form of continuous monitoring, allowing early detection of structural anomalies, even in difficult-to-access locations. This study contributes to the understanding of InSAR's capabilities by highlighting its technical advances and the growing importance of this tool for dam safety. The use of InSAR provides a detailed analysis of deformations, which is essential for guaranteeing the stability of dams throughout their lifetime, enabling effective preventive and corrective interventions.

The bibliometric research carried out in this work was fundamental to mapping the main scientific trends on the use of InSAR in dam monitoring. Extracting information from articles published between 2018 and 2022, the study identified the main authors, institutions, and techniques related to the topic. This research also highlighted the challenges and gaps in the area, helping to guide future research and the development of new solutions for monitoring critical infrastructures.

The selected literature studies demonstrate the application of InSAR in the monitoring of tailings, earth and concrete dams, illustrating how the technology is used to identify vulnerable areas, detect structural deformations, and monitor the behavior of dams over time. In addition, these studies highlight the importance of InSAR in formulating preventive strategies, reducing risks and vulnerabilities associated with dam failure. InSAR's ability to provide accurate and

continuous data has proved indispensable in guaranteeing the safety and integrity of dams, contributing to more efficient and safer management of hydraulic infrastructures.

REFERENCES

[1] Papoutsis, I. (2020). InSAR Greece with Parallelized Persistent Scatterer Interferometry: A National Ground Motion Service for Big Copernicus Sentinel-1 Data. Remote Sensing, 12(19).

[2] Leite, H. E. S., da Costa, L. M., Guimarães, L. J. N., de Melo Ferreira, S. R., & dos Santos Júnior, O. F. (2024). Aspectos sobre Instrumentação e Monitoramento de Barragens. XXI Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, p. 8.

[3] Scaioni, M., Marsella, M., Crosetto, M., Tornatore, V., & Wang, J. (2018). Geodetic and remote-sensing sensors for dam deformation monitoring. Sensors (Basel, Switzerland), 18(11), 3682. doi:1 0.3390/s18113682.

[4] Alves, J. S. (2023). Análise Da Interferometria SAR no Monitoramento Geotécnico: Potencialidades e Limitações. Universidade Federal de Pernambuco (UFPE), Recife-PE.

[5] Aswathi, J., Binoj Kumar, R. B., Oommen, T., Bouali, E. H., & Sajinkumar, K. S. (2022). InSAR as a tool for monitoring hydropower projects: A review. Energy Geoscience, 3(2), 160–171. doi: 10.1016/j.engeos.2021.12.007

[6] Aragão Júnior, W. R., Oliveira Júnior, A. I. de. (2021). Internet das coisas na gestão de resíduos sólidos: revisão sistemática com análise bibliométrica da literatura. J. Env. Anal. Progr., 6(3), pp. 194-209. doi: 10.24221/jeap.6.3.2021.4245.194-209

[7] Moher D., Liberati, A., Tetzlaff, J., Altman, D. G. (2010). Preferred reporting items for systematic reviews and meta-analyses: The PRISMA statement. International Journal of Surgery, 8(5), pp. 336-341. doi: 10.1016/j.ijsu.2010.02.007

[8] Du, Z., Ge, L., Ng, A. H.-M., Zhu, Q., Horgan, F. G., & Zhang, Q. (2020). Risk assessment for tailings dams in Brumadinho of Brazil using InSAR time series approach. The Science of the Total Environment, 717(137125), 137125. doi: 10.1016/j.scitotenv.2020.137125

[9] Xiong, Z., Feng, G., Feng, Z., Miao, L., Wang, Y., Yang, D., & Luo, S. (2020). Pre- and post-failure spatial-temporal deformation pattern of the Baige landslide retrieved from multiple radar and optical satellite images. Engineering Geology, 279(105880), 105880. doi: 10.1016/j.enggeo.2020.105880.

[10] Corsetti, M., Fossati, F., Manunta, M., & Marsella, M. (2018). Advanced SBAS-DInSAR technique for controlling large civil infrastructures: An application to the Genzano di Lucania dam. Sensors (Basel, Switzerland), 18(7). doi: 10.3390/s18072371

[11] Xiao, R., Jiang, M., Li, Z., & He, X. (2022). New insights into the 2020 Sardoba dam failure in Uzbekistan from Earth observation. International Journal of Applied Earth Observation and Geoinformation: ITC Journal, 107(102705), 102705. doi: 10.1016/j.jag.2022.102705

[12] Grebby, S., Sowter, A., Gee, D., Athab, A., Barreda-Bautista, B. D. la, Girindran, R., & Marsh, S. (2021). Remote monitoring of ground motion hazards in high mountain terrain using InSAR: A case study of the Lake Sarez area, Tajikistan. Applied Sciences (Basel, Switzerland), 11(18), 8738. doi: 10.3390/app11188738

[13] Al-Husseinawi, Y., Li, Z., Clarke, P., & Edwards, S. (2018). Evaluation of the stability of the Darbandikhan dam after the 12 November 2017 MW 7.3 Sarpol-e Zahab (Iran–Iraq border) earthquake. Remote Sensing, 10(9), 1426. doi:10.3390/rs10091426

[14] Othman, A. A., Al- Maamar, A. F., Al-Manmi, D. A. M., Liesenberg, V., Hasan, S. E., Al-Saady, Y. I., … Khwedim, K. (2019). Application of DInSAR-PSI technology for deformation monitoring of the Mosul Dam, Iraq. Remote Sensing, 11(22), 2632. doi: 10.3390/rs11222632

[15] Wang, G., Li, P., Li, Z., Ding, D., Qiao, L., Xu, J., Li, G., & Wang, H. (2020). Coastal Dam Inundation Assessment for the Yellow River Delta: Measurements, Analysis and Scenario. Remote Sensing, 12(21), 3658. https://doi.org/10.3390/rs12213658

[16] Liu, N., Dai, W., Santerre, R., Hu, J., Shi, Q., & Yang, C. (2019). High spatio-temporal resolution deformation time series with the fusion of InSAR and GNSS data using spatio-temporal random effect model. IEEE transactions on geoscience and Remote Sensing: a publication of the IEEE Geoscience and Remote Sensing Society, 57(1), 364–380. doi: 10.1109/tgrs.2018.2854736

[17] Li, T., Motagh, M., Wang, M., Zhang, W., Gong, C., Xiong, X., He, J., Chen, L., & Liu, J. (2019). Earth and Rock-Filled Dam Monitoring by High-Resolution X-Band Interferometry: Gongming Dam Case Study. Remote Sensing, 11(3), 246. doi: 10.3390/rs11030246

[18] Qiu, Z., Jiao, M., Jiang, T., & Zhou, L. (2020). Dam structure deformation monitoring by GB-InSAR approach. IEEE Access: practical innovations, open solutions, 8, 123287–123296. doi: 10.1109/access.2020.3005343

[19] F. Gama, F., Mura, J. C., R. Paradella, W., & G. de Oliveira, C. (2020). Deformations before the Brumadinho dam collapse revealed by Sentinel-1 InSAR data using SBAS and PSI techniques. Remote Sensing, 12(21), 3664. doi: 10.3390/rs12213664

[20] Wang, P., Xing, C., & Pan, X. (2020). Reservoir dam surface deformation monitoring by differential GB-InSAR based on image subsets. Sensors (Basel, Switzerland), 20(2), 396. doi: 10.3390/s20020396

[21] Maltese, Antonino, Pipitone, C., Dardanelli, G., Capodici, F., & Muller, J.-P. (2021). Toward a comprehensive dam monitoring: On-site and remote-retrieved forcing factors and resulting displacements (GNSS and PS–InSAR). Remote Sensing, 13(8), 1543. doi: 10.3390/rs13081543

[22] Roque, D., Lima, J. N., Perissin, D., Falcão, A. P., Lemos, J. V., & Fonseca, A. M. (2021). Integrated InSAR and GNSS monitoring subsystem for an arch dam and reservoir banks. Journal of Surveying Engineering, 147(3). doi: 10.1061/(asce)su.1943-5428.0000361

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

AUTOMATED GEOTECHNICAL MONITORING PROJECT OF A BRAZILIAN MINING TAILINGS DAM

Silva. Andre[1]
Measurand

Braga. Rachel
"Mining Company"

Meireles. Henrile
"Mining Company"

Dias. Carlos
"Mining Company"

ABSTRACT

This study presents the planning, implementation, and operation of the automated geotechnical monitoring system used at a Brazilian Mining tailings dam in Minas Gerais, Brazil. The main focus is to demonstrate how the use of integrated and automated technologies directly contributes to geotechnical risk management, promoting the safety and stability of these structures. The system provides real-time data, enabling effective risk management.

The Mining geotechnical monitoring system is comprised of a set of advanced technologies, including robotic total stations, video monitoring, vibrating wire piezometers, water level indicators, and automated ShapeArray inclinometers, all connected through a remote and automated data acquisition system, complemented by rain gauges and InSAR. These technologies were selected and implemented with built-in redundancy to ensure reliability and accuracy in data collection. This study details the adopted methodology, addressing the monitored parameters, the frequency of measurements, and the decision-making process based on the information obtained. The combination of these technologies, along with geological inspections and regular surveys, enables the early identification of anomalies or unexpected behaviour in the dams, allowing for preventive or corrective measures to mitigate risks and ensure structural integrity.

As a result, the company has successfully carried out efficient risk management, preventing potential disasters and ensuring safe operational continuity. The geotechnical monitoring project stands out as an example of best practices in the mining industry, reflecting the importance of continuous and comprehensive monitoring to safeguard the dams and protect the surrounding communities.

Keywords: tailing dam, monitoring, instrumentation, monitoring plan, TARP

[1]andre.silva@measurand.com, Brazil

1. INTRODUCTION

Ensuring dam safety is a fundamental aspect of civil engineering, directly impacting human lives, environmental preservation, and economic stability. In light of past disasters such as Mariana (2015) and Brumadinho (2019), regulatory frameworks in Brazil have been strengthened, demanding more advanced monitoring techniques. This study presents the development and implementation of an automated geotechnical monitoring system designed to enhance the safety and risk management of tailings dams. This program should cover various aspects, including regularly assessing the structure's condition, identifying potential anomalies, and implementing preventive and corrective measures to mitigate risks.

According to the Dam Safety Report 2023 (RSB 2023), there are 25,923 registered dams in Brazil. As for the primary use (Figure 1), the dams are intended for: irrigation (9615 - 37%); animal watering (5433 - 21%); flow regularization (2846 -11%); human water supply (2017 - 8%); aquaculture (1813 - 7%), hydroelectric generation (1303 - 5%); disposal of mining tailings2 (931 - 4%); industrial use (596 - 2%); recreation (631 - 2%); landscaping (526 - 2%); others (232 - 1%)

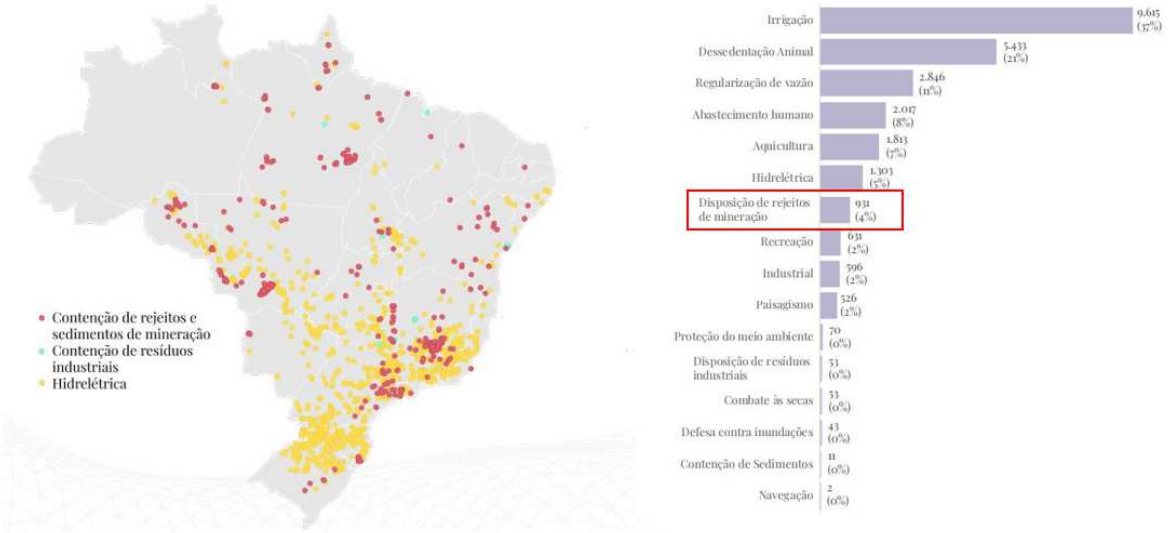


Fig. 1. - Geographical distribution of dams with primary use for the disposal of Mining Tailings, disposal of Industrial and Hydroelectric Waste; Main use of registered dams Source: RSB 2023

After the disasters of the Mariana (2015) and Brumadinho (2019) dams, there was a significant change in Brazilian legislation aimed at dam safety and substantial changes in the posture of the mining industry and other sectors responsible for these structures.

Among the main changes mentioned by Mello (2021) and Paniago (2021), the following stand out:

1. Strengthening the National Dam Safety Policy (PNSB):
   Law No. 12.334/2010, which establishes the PNSB, underwent improvements and stricter regulations after the accidents to strengthen monitoring, control, and responsibility for dam safety.
2. Restrictions on Specific Construction Methods:
   After the Brumadinho disaster, the National Mining Agency (ANM) issued resolutions banning the construction and raising of dams using the upstream method, which was deemed less safe. This has driven mining companies to invest in and adopt new tailings disposal technologies.
3. Increased Enforcement and Harsher Penalties:
   There was an increase in inspection rigour by the ANM, ANA (National Water Agency) and ANEEL (National Electric Energy Agency), with an increase in inspections, the requirement for independent technical audits, periodic safety reports and the application of heavier penalties in case of non-compliance.
4. Transparency and Participation of Society:
   The new rules reinforce the need for greater transparency, with the disclosure of reports and technical information to the public and the participation of local communities and environmental agencies in the licensing processes and monitoring of dam safety.
5. International Standards and Best Practices:
   The industry, pressured by investors, civil society and regulatory bodies, has started to adopt international safety standards, more robust risk management practices, more effective emergency plans and training for rapid response teams. In addition, companies have invested in remote monitoring technology, drones, sensors, and sound warning systems.

In this context, Brazilian mining companies, initiated a broad review of its internal safety protocols, which resulted in implementing more sophisticated monitoring plans in its projects. These plans involve the use of state-of-the-art technologies, such as remote sensors, satellites, automated warning systems and real-time data analysis, as well as the integration of digital platforms that allow continuous monitoring of the structural conditions of the dams. By aligning these innovations with new regulatory guidelines and international best practices, companies seek to anticipate potential issues, improve risk identification, and raise the level of transparency and security of their operations.

Instrumentation is one of the pillars of an effective monitoring system. Instruments such as piezometers, water level indicators, V-notch weir, surface landmarks, and inclinometers, among others, can be installed to evaluate the dam's behaviour continuously. Engineers can obtain information about structure deformation, internal pressures, percolation conditions, and other safety-relevant parameters by collecting and analyzing data from these instruments.

Data collection can be carried out automatically, with readings at a specific frequency, and manually, with periodicity defined according to the criticality of each instrument. This data is analysed by the dam management team, allowing for the early detection of anomalies and timely corrective action.

In addition to in-situ and remote satellite monitoring, periodic visual inspections are a key component of the maintenance program. Visual inspections allow the identification of anomalies that may not be detected by the instruments, such as cracks, erosions, water upwellings, excessive vegetation and other signs of deterioration. Inspections must be carried out by specialized and experienced technicians, who record their observations in detailed reports, including photographs and the precise location of anomalies. The periodicity of inspections is defined according to the dam's risk classification and the site's specific conditions.

The data collected during monitoring and inspections must be carefully documented and reported. Documentation must be organized and archived in such a way as to ensure its traceability and future accessibility. Monitoring reports must be prepared periodically, presenting the results of the analyses, conclusions about the dam's behaviour and recommendations for future actions. Effective communication between stakeholders, including the dam owner, regulatory authorities, engineering teams, and the local community, ensures the structure's safety.

In summary, this article aims to demonstrate the importance of a comprehensive monitoring and maintenance program in ensuring the safety of a dam. Advanced instrumentation, systematic visual inspections, rigorous documentation, and effective communication are fundamental pillars of this program, and they must be fully aligned with the standards and guidelines currently in force in Brazilian legislation.

2. DAM CHARACTERISTICS

The tailings dam analyzed in this study is situated in Minas Gerais, Brazil, and ranks among the largest in the country close to a urban area. Originally designed for tailings disposal from iron ore processing, it now primarily functions as a sediment containment structure. Due to its proximity to urban areas, its monitoring is critical to ensuring both structural integrity and public safety.

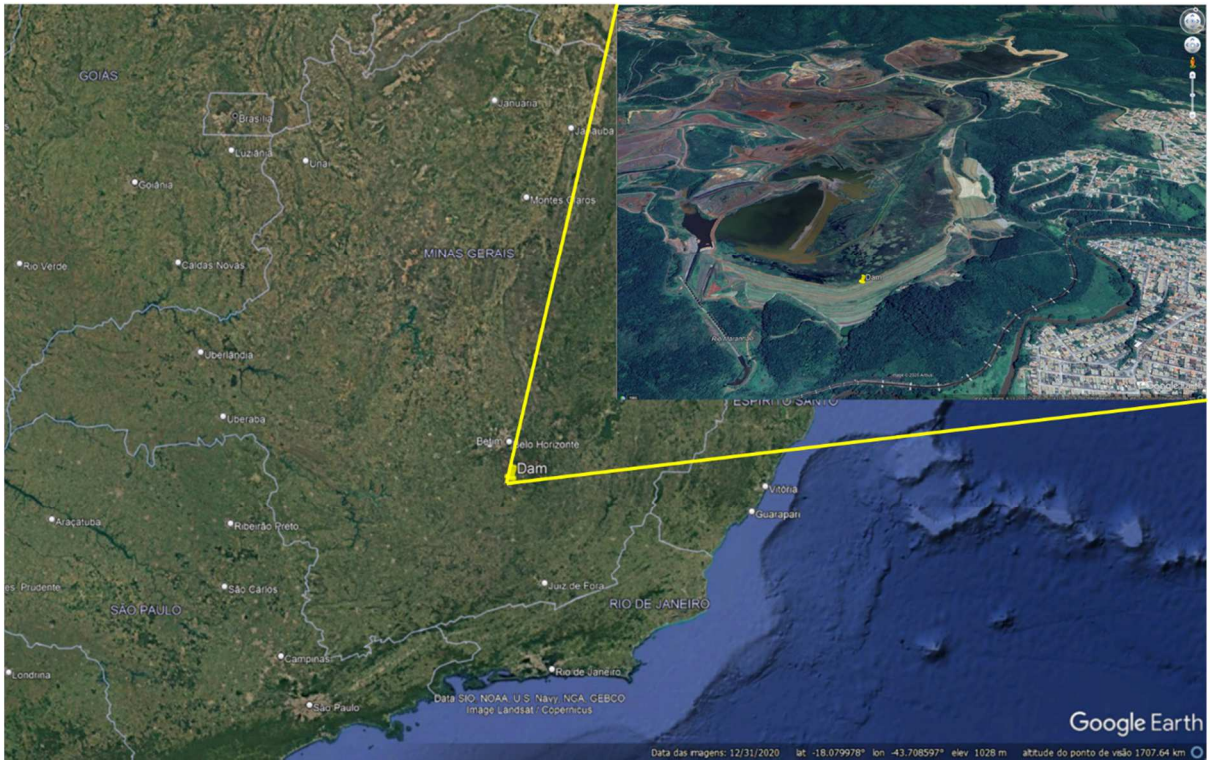


Fig. 2 – TSF location and aerial view

The dam is a homogeneous earth structure built to store water, sediment containment and, previously, for the disposal of tailings from iron ore processing. The construction of the dam was in two steps. The main dam body consists of a sequence of downstream embankment stages, while the saddle dam is constructed using centerline embankments. Currently, it is used for water recirculation and sediment containment.

With a crest of approximately 1500 meters in length, the dam has a height ranging between 79.7 and 90 meters, depending on the standard applied. The width of the crest varies between 3.95 and 6.20 meters. Its reservoir has a current volume of 65.37 million m^3, with a total capacity of 70 million m^3, covering an area of 1,902,000 m^2. This data, along with other details about the dam, are presented in Table 1

Table 1 - Dam characteristics and details

GENERAL DATA	
Supervisory body	ANM e FEAM
Purpose	Water storage and disposal of tailings from the iron ore processing process of the Dam Mine. Tailings have not been disposed of since December 2019. Currently, its reservoir retains sediments from upstream easement areas and acts to decant the water and eventually discharge it in emergencies.
Risk Rating (CRI) (ANM)*	Low
Associated Potential Damage (DPA) (ANM)*	High
Type of Dam	Homogeneous earth dam
CHARACTERISTICS OF THE DAM	
Current Crest Length (m)	~ 1500,00
Height (m) according to State Decree nº 48.140/2021	90
Height – Law 14066/2020	79,7
Construction method	Downstream and centerline
Year of start of implementation	2007
Year of Operation	2009
Year of Operation End	2019. Since 31/12/2019, the tailings have been shut down
Current Operation Status of the Dam	Active dam
Crest width (m)	between ~ 3,95 e 6,20
Slope of the Exposed Upstream Slope	1V:1,7H E 1V:2,4H
Slope of the Exposed Downstream Slope	Entre 1V:1,8H e 1V:2H

The classification of mining dams under Brazil's regulations is based on two criteria: Risk Category (CRI), which assesses the likelihood of failures, and Associated Potential Damage (DPA), which measures the possible consequences of an incident. The CRI takes into account technical factors, conservation status, and compliance with the Dam Safety Plan, while the DPA considers human losses, social, economic, and environmental impacts. Both criteria are classified as high, medium, or low**,** enabling a more accurate assessment of risks and the necessary prevention measures. That explains the possibility of having a dam with a low Risk Category but a high Associated Potential Damage

This scenario occurs when a dam is well-designed, properly maintained, and compliant with all safety regulations (low CRI), but its failure would still result in severe consequences due to its location, size, or the nature of the stored materials (high DPA). For example:

- A well-maintained tailings dam located near a densely populated area or critical infrastructure may have a low CRI (because the risk of failure is low) but a high DPA (because the consequences of failure would be catastrophic).
- A dam retaining a large volume of hazardous tailings might have low structural risk but still pose severe environmental and human risks if breached.

This distinction is crucial in dam safety management in Brazil, as even a well-maintained dam can require strict monitoring and emergency planning if its failure would lead to high social, economic, or environmental impacts.

3. GEOTECHNICAL MONITORING PLAN

A fundamental principle of geotechnical monitoring, as defined by Dunnicliff (1988), is that each instrument must be selected based on a specific question it aims to answer. This ensures that data collected is both relevant and actionable, optimizing resources and improving risk management. In this context, the present study employs a combination of high-precision instruments to monitor critical parameters, as detailed in Table 2. Incorporating this systematic approach, the monitoring plan now includes well-defined objectives, careful selection of devices, collection of relevant data, and objective analysis. This methodology optimizes resource use, reduces waste and facilitates continuous adjustments throughout the project, and consolidates a robust technical base. By promoting confidence in the results obtained, the enterprise's credibility is reinforced and, consequently, the operation's security, as Peck (1973) emphasized.

Table 1 – Stages and sub-stages of the dam monitoring plan Source: Dunnicliff (1988) and Silveira (2006) adapted by the author

Main stage	Sub-Stage	Description
Understanding Structure Behavior	Purpose of Monitoring	Identify anomalies and implement corrective measures.
	Proper Instrumentation	Choose instruments based on the identified risks.
	Strategic Instrument Location	Define strategic locations of the instruments.
	Supervision and quality control in the installation	Establish standardized procedures for installation, calibration and periodic auditing; Document with checklists, photo reports, and systematic checks
Data Collection and Analysis	Data Collection Methods	Define data collection procedures (automated or manual)
	Data Storage and Management Systems	Organize and store data in SHMS, GeoInspector, SAA Suite.
	Data Analysis and Interpretation	Analyze data, identify trends, and classify anomalies.
Corrective Action Planning	Preventive and Corrective Maintenance	Perform regular and corrective maintenance to ensure integrity.
	Emergency Action Plan	Integrar com plano de ação de emergência (PAEBM).
Communication and Documentation	Periodic Reports	Prepare periodic monitoring reports.
	Effective Communication	Establish efficient communication channels.

The monitoring plan details outlined in the previous table are further explained in the following points.

3.1. Understand structure behaviour

The Dam's monitoring plan aims to understand the structure's behaviour in line with standards and regulatory requirements, in addition to previously established measurement parameters (deformations, pore pressures, flow, etc.). In this way, a precise focus is maintained on the relevance and quality of the data collected.

The selection of instruments is based on strict technical criteria (precision, robustness, measurement range), redundancies at critical points, and up-to-date technologies are considered. Thus, greater reliability in the results is ensured, and the risk of failures is reduced.

The location of the instruments is based on structural analysis and modelling, such as the Finite Element Method (FEM), allowing positioning sensors in areas of greater relevance. This care optimizes the quantity and type of equipment, obtaining representative data, reducing costs and increasing monitoring effectiveness.

Finally, supervision and quality control in the facility include standardized assembly, calibration and auditing procedures, and systematic documentation (checklists, photographic reports, periodic checks). These practices ensure traceability, integrity, and coherence of measurements, reinforcing the structure's safety.

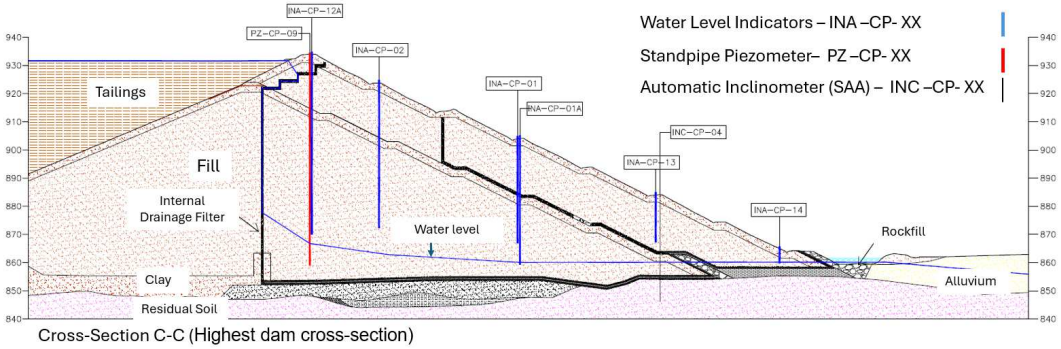
In addition to this significant and redundant set of instruments, these are complemented with rain gauges and monthly analysis of surface displacements using satellite monitoring through InSAR.

3.2. Type and Number of instruments

A comprehensive monitoring system has been implemented to ensure the structure's safety. The dam has 174 monitoring instruments, including 27 piezometers, 82 water level indicators, 52 surface landmarks, 5 V-notch weir, 1 limnimetric ruler, 7 ShapeArrays and 3 robotic stations. Data collection is carried out automatically, with readings every hour, and also manually, with periodicity defined according to the criticality of each instrument. This data is analyzed by the dam management team, allowing for the early detection of anomalies and timely corrective action.

Table 2 – Instrument types and quantities, acquisition methods, and measurement frequency

Instrument Type	Quantity	Location	Monitored Parameter	Manual Frequency	Automated Frequency
Piezometer (PZ)	27	Dam body/Foundation	Piezometric Load	Biweekly	Hourly
Water Level Indicator (INA)	82	Internal to the structure	Water level		
V-Notch Weir (MV)	5	Bottom drain and overflow	Water flow		
Superficial Landmark (MS)	52	Structure surface	Horizontal/vertical displacement	Monthly	
Automatic Total Station	3	Structure surface	Horizontal/vertical displacement		
Automatic Inclinometer ShapeArray (INC)	7	Internal to the structure	Horizontal Displacement	-	



Cross-Section C-C (Highest dam cross-section)

Fig. 3 – C-C: Highest dam cross-section showing instrument position (water level indicator-Blue line; standpipe piezometer -Red Line; Automatic Inclinometer/ShapeArrays – Black Line),water level position, downstream embankments, fill material, soil layers at the dam foundation, filters, and base drain

Fig. 4 – V-Notch Weir and piezometers with anti-vandalism protection. Automated ShapeArray is integrated with a wireless datalogger with external protection.

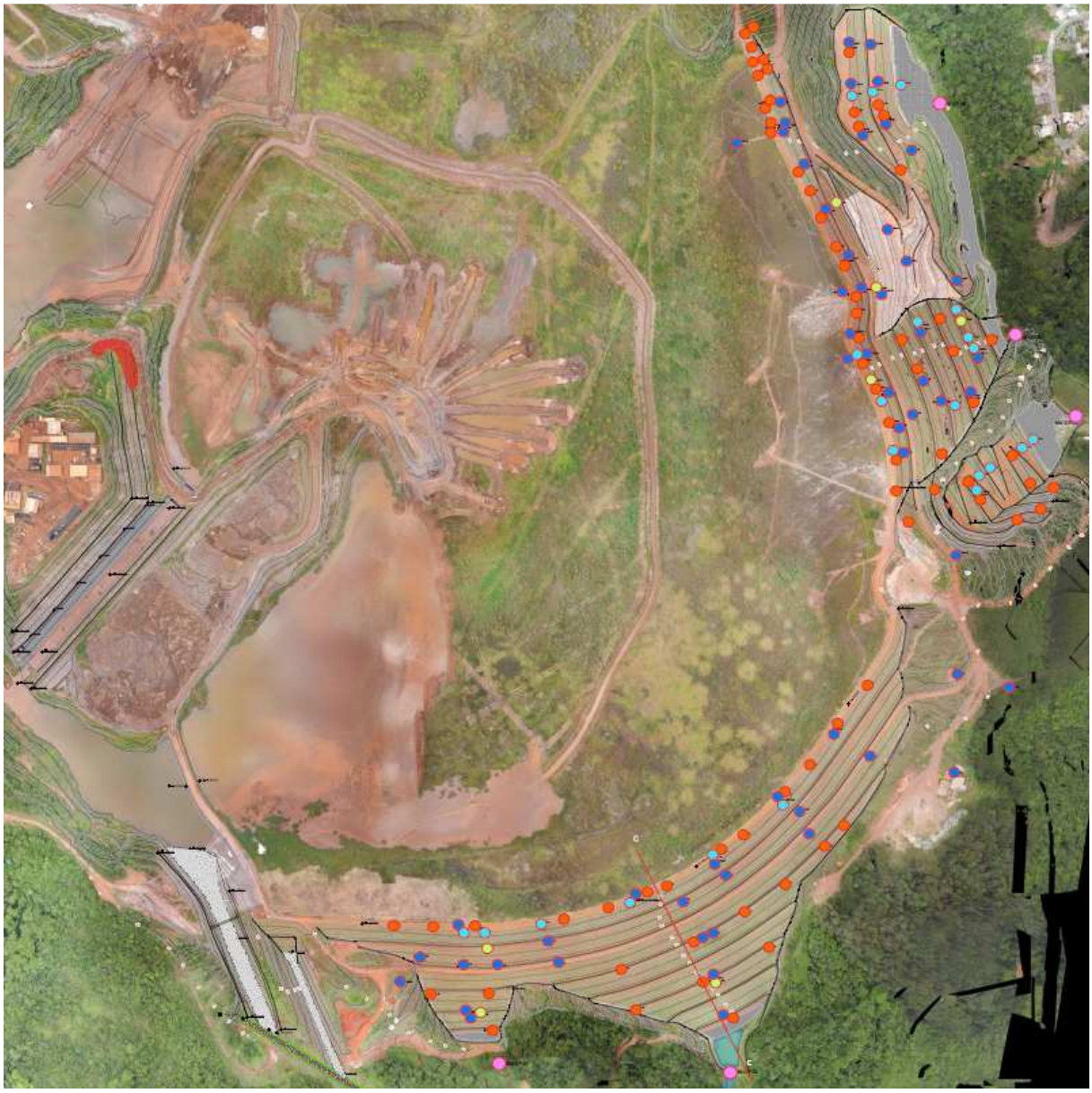


Fig. 5 – Instruments position. (Orange – Superficial Marks; dark blue – Water Level indicators; Yellow – ShapeArray inclinometers; Pink – V-Notch Weir; Red Line representing the cross-section CC the highest dam section)

3.3. Data Collection, Storage and Analysis:

The monitoring plan establishes well-defined data collection procedures, ensuring the information's consistency and reliability. These procedures include both automated and manual acquisition methods, with reading frequencies appropriate to the type of instrument used. For example, in the case of water levels, piezometers, V-notch weir, and superficial landmarks, the measurement routines and the practice of systematic comparison with previous readings are detailed, which allows the identification and correction of any discrepancies

The management of the data thus obtained is also crucial. To ensure accessibility, traceability and the possibility of historical analyses, measurements are stored on consolidated platforms such as the SHMS (Fig. 7), the GeoInspector (Fig. 9) and SAASuite (Fig. 6). These resources provide an integrated environment to consolidate the readings, facilitating the consultation and statistical treatment of the information.

Slope Health Monitoring System (SHMS):

SHMS is a data integration and analysis platform used in geotechnics, hydrogeology, and environmental engineering. It enables real-time dam monitoring by aggregating data from multiple instruments and applying artificial intelligence for anomaly detection and risk assessment.

Geo Inspector:

Geo Inspector is a platform for managing data from manual instrumentation and site inspections. It standardizes data collection and analysis, facilitating the interpretation of geotechnical monitoring results.

Both platforms were developed by the same Brazilian software company and are designed to complement each other, covering both manual and automated monitoring methods. They provide tools for data integration, analysis, and predictive assessment.

SAASuite:

SAASuite is inclinometer software that automates data conversion and analysis. It integrates with SHMS and includes additional features for advanced inclinometer data evaluation.

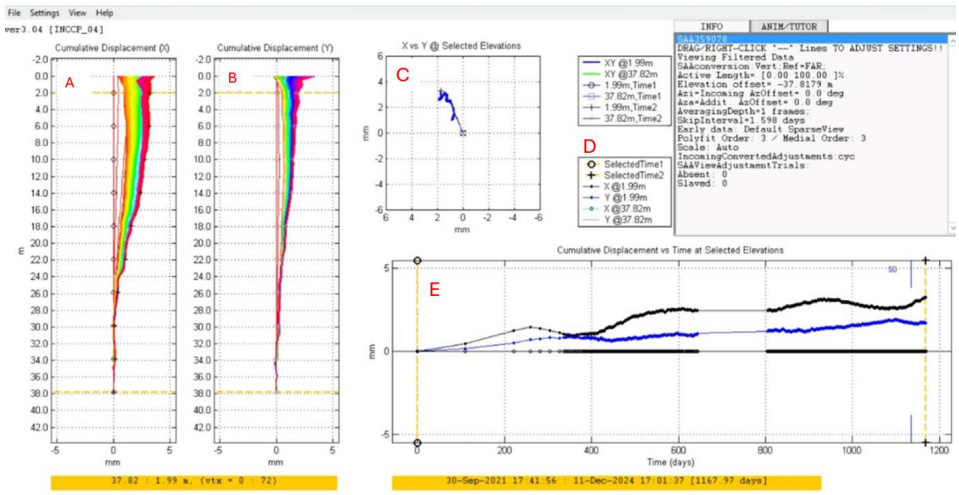


Fig. 6 – A/B – Cumulative displacements from the ShapeArray INC CP 04 inclinometer (Cross section C-C) recorded between 11/30/2021 and 12/11/2024. The X-axis (A-axis) represents the expected displacement direction (upstream-downstream), while the Y-axis (B-axis) is perpendicular to it. C – Displacement vector. D – Selected elevation. E – Displacement time plot at a depth of 2 m.

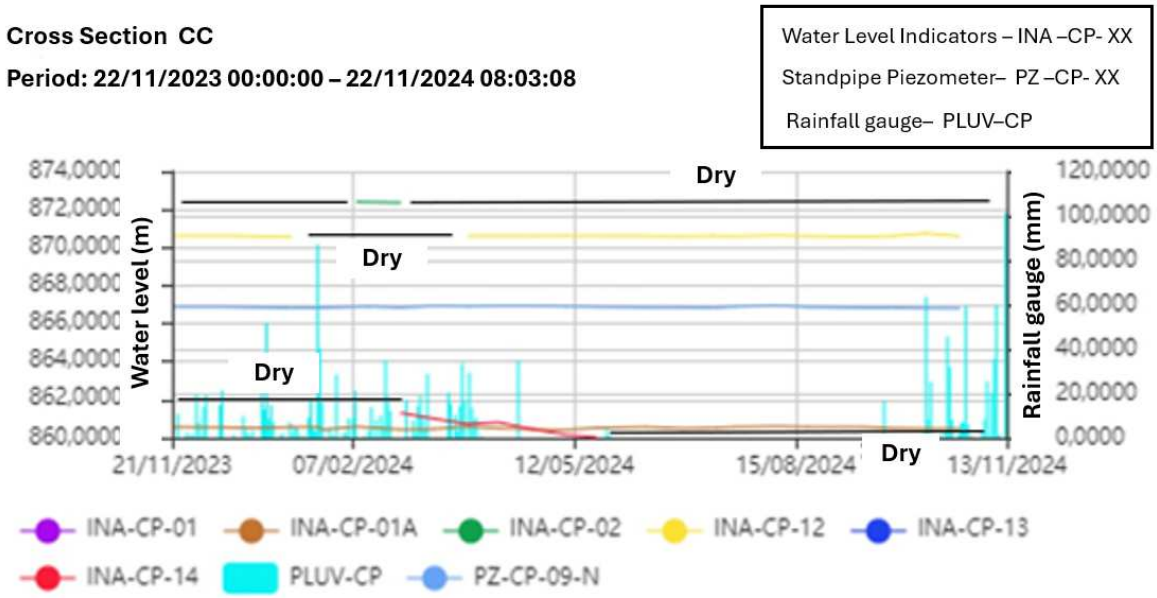


Fig. 7 – Water level variations and rainfall data for Cross Section CC over the monitoring period (22/11/2023 – 22/11/2024), showing dry or stable conditions at multiple indicator points.

Fig.8 ***–Map of landmark points (left) with real-time displacements monitored and Robotic Total Station (right) for automatic collection and transmission.***

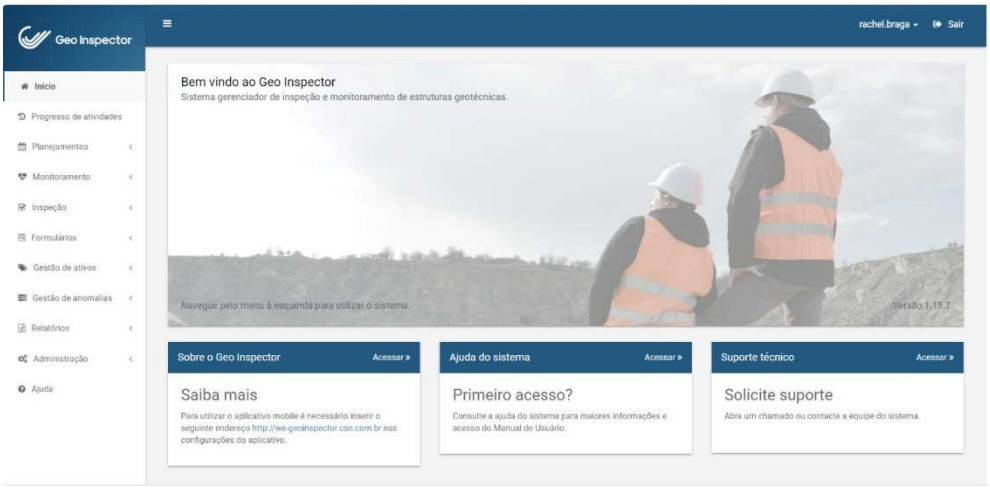


Fig.9 – Geoinspector platform allows the integration of monitoring data (manual data included), inspections, document management, reporting and other functionalities

Finally, the analysis and data interpretation are essential steps in the plan. In addition to comparing previous measurements, the process includes identifying trends, evaluating the significance of the observed variations, and classifying the severity of any anomalies, something Peck (1969) already recommended through the observational method. The Technical Assessment Team plays a central role in these activities, relying on tools such as the Classification Matrix, which provides objective criteria for framing results and deciding preventive or corrective actions

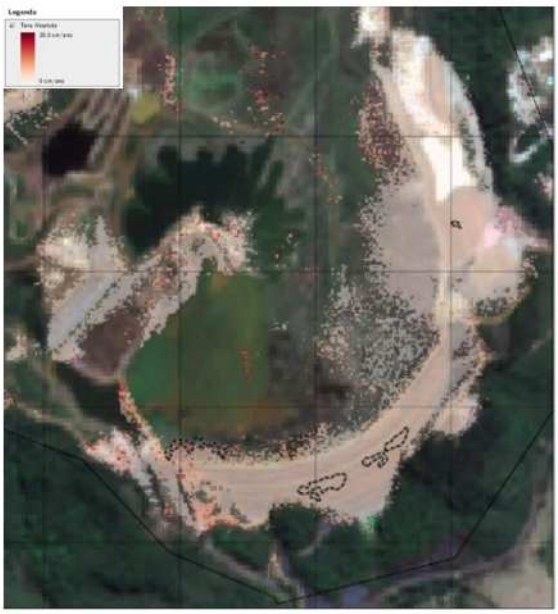


Fig. 10 – Remote monitoring using InSAR technology with monthly data acquisition (Scale 0 - 20cm/year) – Source: Mining company/3vGeomatics

At the time of writing this article, InSAR had been implemented relatively recently in the structure; therefore, there has not yet been enough time to fully observe its potential when used in conjunction with the conventional methodologies also implemented. This part of the article outlines the advantages of InSAR, conventional instruments, and how the two can complement each other.

Unique Advantages of InSAR:

- InSAR is highly effective for large-scale deformation satellite monitoring, as it can provide spatially extensive data over vast areas with high precision.
- It is a cost-effective method for monitoring remote or inaccessible regions, reducing the need for extensive fieldwork.
- The technology allows for retrospective analysis, as archived satellite data can be used to detect past deformation trends.

Strengths of Conventional Instrumentation:

- Conventional instrumentation, such as inclinometers, piezometers, and strain gauges, provides detailed, localized, and real-time data about structural and ground conditions.
- Instruments can monitor specific parameters that InSAR cannot directly measure, such as groundwater levels or internal stresses within a structure.
- It is often better suited for high-frequency monitoring or environments where real-time alerts are critical.

Complementary Roles in a Monitoring Program:

- InSAR can serve as an initial screening tool to identify areas of concern, which can then be further investigated using conventional instrumentation.
- Conventional instruments can validate and calibrate InSAR data, ensuring its accuracy and reliability.
- The integration of both methods provides a comprehensive dataset, combining spatial data from InSAR with the detailed, parameter-specific data from conventional tools.

3.4. Risk assessment analysis (“Risk chart”)

For the purpose of controlling instrumentation parameters and establishing reference alert values, it is common practice for the technical manager or structural consultant to conduct a risk analysis of the structure. This analysis considers various scenarios to derive control values for the instruments. At the time this article was being written, these values and studies could not be disclosed. The control chart or risk chart is merely an auxiliary tool for detecting

deviations with a certain classification of importance and priority in the execution of safety protocols (communication, actions, decision-making, recording). It does not directly correspond to the structural safety condition but indicates an unexpected deviation that needs to be assessed, as per the protocol outlined in the Operations Manual (TARPS).

Definition of control levels:

Control levels can be defined through:

- Computational modeling;
- Empirical reference values;
- Observational methods (analysis and statistical processing of historical data);
- Kinematic methods;
- A combination of methods.

Important common points for each method:

- Having a reliable database of instrumentation and the structure;
- Understanding the structure's condition and what is expected in terms of behavior;
- Knowing the failure modes.

This provides a comprehensive overview of how control values are determined and emphasizes the critical factors involved in the process.

3.5. Corrective action planning

The monitoring plan for the Dam includes a set of preventive and corrective maintenance actions to ensure the integrity of the structure and its components. In addition to providing regular interventions, the plan establishes specific measures to correct anomalies as soon as they are detected. According to the Operation Manual (2024), maintenance activities include pruning and conservation of vegetation cover, mowing of slopes, calibration of instruments, cleaning of drainage channels and removal of obstructions, and the immediate repair of cracks and erosion. Such tasks can be classified into predictive actions—such as pest control, instrument maintenance, pruning, preservation of vegetation cover, and calibrations—and corrective actions, applied when necessary, for example, in repairing cracks, cleaning of gutters, and removing debris from the spillway.

At the same time, the monitoring plan is integrated with an Emergency Action Plan (PAEBM), which must be activated in critical situations. This integration ensures a quick and adequate response to any structural anomalies, contributing to mitigating risks and preserving the dam's safety. Thus, the set of predictive, corrective, and emergency measures reinforces the

monitoring system's effectiveness, ensuring the proper functioning of the infrastructure and the protection of the surroundings.

3.6. Communication, documentation and report

The monitoring plan establishes the preparation of periodic reports, which consolidate information on the state of the structure, present trends identified from the data collected and describe the maintenance interventions carried out. These documents include semi-annual inspections and monthly performance reports, ensuring systematic and transparent monitoring of the dam's conditions.

In addition, effective communication between the technical team, the competent authorities, and the downstream population is a key pillar of safety management. The establishment of clear and accessible dialogue channels ensures that relevant information is readily disseminated, contributing to mutual trust and understanding, as well as to decision-making based on continuous data analysis

4. CHALLENGES

After the presentation of the various aspects of Tailings Dam monitoring plan, the daily practice highlighted a series of challenges. Such difficulties occur both in the scope of data management—collected in large volume from multiple platforms—and in the availability of technical and human resources adequate to the growth of demands. Among the main points are:

- One of the key challenges in geotechnical monitoring is handling large volumes of data efficiently. The integration of multiple monitoring platforms—each with different data formats, update frequencies, and storage requirements—demands robust solutions for secure storage and real-time analysis. For example, discrepancies between manual and automated readings require validation procedures to ensure data reliability, which can be resource-intensive.
- There is a need for multidisciplinary integration of information, overcoming barriers imposed by the use of different platforms, and the potential to evolve towards a unified system.
- A greater focus on increasing the number of well-trained professionals can enhance the efficiency of inspections, supervision of activities, and validation and interpretation of data and information.
- Continuous training is essential to keep up with the technologies currently used and those that may be incorporated in the future.

- Establishing uniform standards and protocols for data analysis can simplify efforts to ensure consistency and comparability of results.
- The dependence on a solid risk management culture, essential to guide safe practices and decisions.
- The costs and economic pressures resulting from the high Brazilian tax burden make equipment and technological solutions about 2.5 times more expensive than in North American or European countries. (example of nontechnical factors influencing monitoring projects according to Peck (1973))

These challenges highlight the need for a strategic, comprehensive, and flexible approach capable of optimizing processes, improving the quality of information, and reinforcing the capacity to respond to the growing demands of the operation.

5. CONCLUSIONS

This study presented the planning, implementation, and some of the results of the automated geotechnical monitoring system applied to a Brazilian tailings dam. Integrating automated in-situ and remote satellite instrumentation, regular inspections, and the strengthening of regulations were the pillars for developing a robust, redundant, and efficient monitoring plan.

Key highlights include:

- Integration of Instrumentation and Regular Inspections: The synergy between modern technologies such as piezometers, ShapeArray inclinometers, robotic total stations (RTS), InSAR and inspections carried out by specialized teams ensured the early identification of anomalies and the adoption of corrective measures on time.
- Influence of Legislation and Supervision: The evolution of Brazilian standards, such as the National Dam Safety Policy (PNSB) and ANM resolutions, were decisive in boosting the use of practices aligned with international standards. These changes led to the accelerated adoption of the best existing monitoring technologies and required greater transparency, technical rigour and regular audits, fostering the modernization of monitoring plans.
- Practical Results and Alignment with Best Practices: The implemented system made it possible to mitigate risks and prevent structural failures, ensuring the stability of the dam and the protection of local communities and the environment. The model aligns with international best practices, demonstrating how regulation, technology, and complementary inspections can strengthen dam management.
- Challenges and Learnings: During the implementation, challenges such as the management of large volumes of data, the need for continuous technical training, the standardization of analysis protocols, and the high costs of equipment in Brazil were

faced. These challenges highlight the importance of integrated strategies that optimize processes, improve the quality of information, and strengthen the ability to respond to growing demands.

This study demonstrates that the integration of advanced monitoring technologies, stringent regulatory oversight, and systematic field inspections plays a crucial role in ensuring the stability of tailings dams. By leveraging real-time data from automated instrumentation and remote sensing methods such as InSAR, mining companies can significantly enhance their risk management strategies. Future studies should focus on refining data integration techniques and exploring the potential of machine learning for predictive maintenance in geotechnical monitoring.

REFERENCES

[1] Agência Nacional de Águas e Saneamento Básico (Brasil). Relatório de Segurança de Barragens 2023 (RSB 2023) / Agência Nacional de Águas e Saneamento Básico. -- Brasília: ANA, 2024

[2] Mello, Flavio Miguez. Sandroni, S. Guidicini, G. (2021) Lições aprendidas com acidentes e incidentes em barragem e obras anexas no Brasil, Comitê Brasileiro de Barragens, Rio de Janeiro

[3] Paniago, Luiz (2021), Segurança de Barragens – Legislação federal brasileira em segurança de barragens comentada, Editora Insight, Brasília, Brasil

[4] Dunnicliff, J. (1988). Geotechnical Instrumentation for Monitoring Field Performance. John Wiley & Sons, Inc.

[5] Silveira, J.F.A. (2006) Instrumentação e segurança de barragens de terra e enrocamento. 1 ed., Oficina de Textos, São Paulo, SP, 416 p. ISBN 978-85-8623-861-1.

[6] Peck, R. B. (1969). Advantages and limitations of the observational method in applied soil mechanics. Geotechnique 19(2)

[7] Peck, R. B. (1973). Influence of nontechnical factors on the quality of embankment dams. Embankment Dam Engineering – CasaGrande Volume, John Wiley & Sons, New York, p 201-208

[8] “Mining Company". Dam Safety Plan Operation Manual, Minas Gerais, Brazil, 2024. Internal document.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

SAFETY EVALUATION OF AN UPSTREAM RAISED RED MUD TAILINGS DAM

Lucas Bottaro[1]
TPF Consultores

Isabel Lopes
TPF Consultores

Gonçalo Tavares
TPF Consultores

Marta Garcia
TPF Consultores

ABSTRACT

As a consequence of the last tailings dams failures, the safety evaluation of upstream raised tailings dams is a strong concern all over the world. During the last years, several documents, procedures and guides have been produced in order to have a better management of tailings dams.

The TSF's current state stability is one of the aspects that has to be assessed in order to evaluate safety and check if any preventive or mitigation measures are needed. In order to make the stability analysis, the model of the TSF has to be prepared, and it includes both the ground and the dam's structure models. Due to the age of many of the structures and/or to poor management practices of the companies, that resulted from the low awareness to the risks at the time of the dam construction, the historic archives of the original ground topography and/or details about the TSF design and construction are, often, few or inexistent. Building the original ground model is, in many situations, a very complex step but is fundamental to know, among others, the dam's height, tailings thickness, material characteristics and stored volume.

In this paper it will be presented the analysis of a red mud tailings dam, upstream raised, with 70m maximum height, going from building the dam model and tailings parametrization to the safety analysis. The results show that the bad quality geological and geotechnical data significantly increases the uncertainty in the model, resulting in marginal safety. In this situation a good operation of the structure, with tight monitoring and maintenance, along with some structure reinforcement actions have to proceed to guarantee the safety of the structure. It is essential to raise awareness among tailings dam operators and geotechnical companies about the importance of rigorous and careful practices. Reliable data collection and close collaboration are key to reducing uncertainty and ensuring long-term safety.

Keywords: tailings storage facility, red mud, safety assessment, upstream construction.

[1]lucas.bottaro@tpf.pt, Portugal

1. INTRODUCTION

As a consequence of recent tailings storage facilities (TSF) failures (e.g., Mariana 2015, Brumadinho 2019), the safety evaluation of upstream raised tailings dams is a strong concern of the mining industry all over the world. During the last years, several documents, procedures and guidelines have been produced, aiming to create better management practices for tailings dams and try to minimize the risks associated with this activity.

However, several existing TSF remote to before these guidelines have been created and implemented in the companies, and some of these facilities are still active. As such, many of them were designed and have been operating based on principles and standards that does not meet the current state of best practices, both for construction and management. These facilities must be, therefore, examined (or re-examined) under these new requirements.

The global stability of the TSF is one of the most important aspects that has to be assessed by the perspective of the current best practices, and it is essential to evaluate the overall safety condition and check if any preventive or mitigative measures should be implemented. To perform such analyses, a model of the TSF must be developed and it should reproduce, as precisely as possible, the natural ground and dam's characteristics.

Therefore, in order to perform an accurate and cautious analysis, it is necessary to characterize the foundation, define the geometry of the dam's elements, assess the engineering properties of the materials, and evaluate the current state of the tailings and their likely response to the existing and possible new loads. In this case study, the existing information regarding these aspects was little to none, and often based on documents that proved to be unreliable. In this less-than-ideal context, the geometry of the TSF and foundation had to be fully recreated, and engineering parameters and current state of the tailings had to be assessed based on field investigations and a few laboratory tests.

2. GENERAL DESCRIPTION OF THE TSF

The TSF object of this case study is part of a mining compound of bauxite ore and stores the tailings produced in the industrial process, known as red mud.

Bauxite is a weathered rock, typically with a soft structure (hardness between 1-3), containing two forms of hydrated aluminium oxide, in a quantity usually between 30-65%. These oxides, represented by the minerals Gibbsite, Boehmite and/or Diaspore, have a specific gravity (Gs) between 2.6-3.5. This ore has additional Fe_2O_3 as the main impurity. Although the minerals that compose bauxite ore have high G_s, the rock has a dry bulk density between 1.5 and 2.5 t/m^3 due to its high porosity. During the alumina extraction process, aluminium reacts with

the NaOH to form soluble sodium aluminate, leaving behind the Fe_2O_3 and other impurities. The process results in a highly alkaline pH=11-13 waste material whose mineral components can include hematite, goethite, gibbsite, rutile, calcite, sodalite, and complex silicates. Notably, there is usually absence of quartz or clay minerals in this waste. The quantities of each mineral in the red mud are very variable and depend on the source bauxite and the original mineralogical composition of the ore is one of the most important factors influencing the red mud tailings geotechnical properties [1].

According with the feasibility study, the TSF under study was built in an originally trough shaped valley surrounded by steep slopes on the northern and southern sides. This valley (E-W direction), with gradient of about 2° from E to W, before construction presented a continuous flow of water.

This TSF's starter dam was constructed in 2012 and, since then, the facility has been raised 4 times, all by the upstream method, in 2015, 2016, 2018 and 2020/2021. Currently, the TSF is about 70m in the highest section and occupies an area of about 500m x 250m. It is no longer receiving tailings and is currently under closure process.

3. RECONSTITUTION OF THE GROUND MODEL AND GEOMETRY OF THE RAISES

Building the original ground model and geometry of the raises is, in many situations, a very complex step, but is fundamental to know, among others, the dam's height, tailings thickness, material characteristics and stored volume. In this case study, due to the lack of reliable information about the site, conceptualising the TSF structure model posed as a huge challenge, both for the bedrock ground surface as for drawing the dam itself.

A good documentation and reporting system is essential for tailings management [2] [3]. When it is missing, the companies working in TSF safety assessment face a huge challenge. Without it there is no good data to work with, sometimes no data at all, and big difficulties arise as the TSF history cannot be established.

Previous information regarding the facility design and operation is very limited and consists of:

- The feasibility report;
- a few raising stability analysis reports, made previously to the raises construction;
- two site investigation campaigns, one from 2014, with two boreholes and a crosshole test, made on the crest of the starter dam, and another from 2019, with two boreholes, with very limited depth and crossing only tailings;
- and the current topography.

The difficulties in building the ground and TSF models are detailed in [4].

Fig. 1 presents the 3D topographic surface of the facility [4] which enabled the construction of the ground model and the generation of the 3D geological and geotechnical model of the TSF. This work was made with the help of an extensive site investigation campaign composed by 33 boreholes, 6 CPTu tests and 5 downhole tests, and building the TSF history was aided by the analysis of historical satellite images. Additionally, the possibility to speak to the site engineer that followed construction, provided valuable insight into the construction procedures and the TSF raising history. His memories and insights added significant information to the TSF model building.

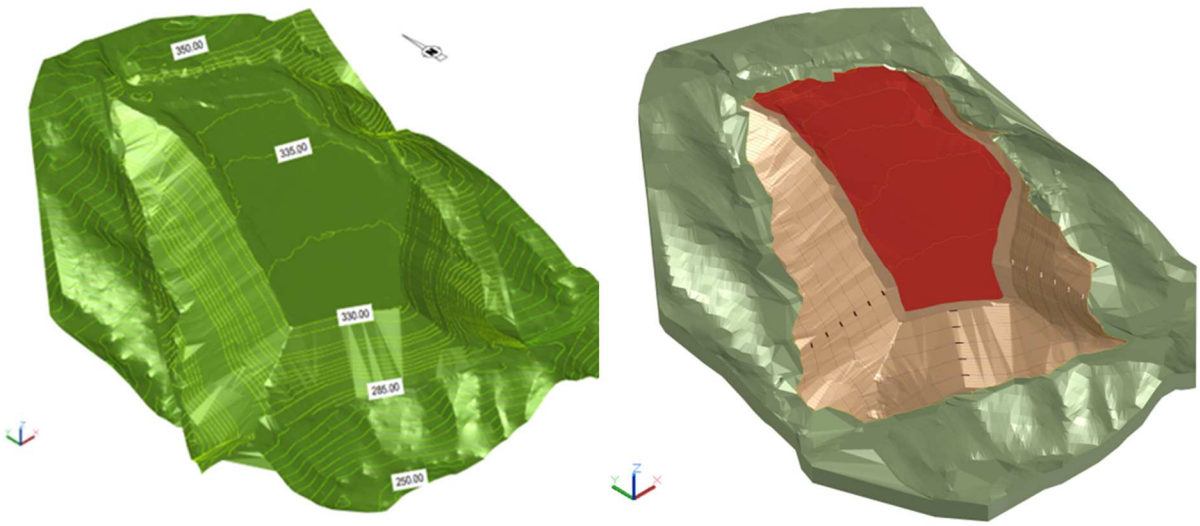


Fig. 1 – Current 3D base topographic surface [4] and the TSF final 3D model.

Building a geological model starts by identifying geological layers and materials crossed by the existing boreholes. The materials identified in the boreholes were:

- Red Mud;
- Embankment materials;
- Bedrock.

The final 3D geological and geotechnical model produced allows to have compatible sections in all directions of the TSF. Fig. 2 presents the tallest sections in the N-S and E-W directions.

4. GEOTECHNICAL PROPERTIES OF THE MATERIALS

Prior to this study, there were not enough geotechnical data to allow a well sustained parametrization, so the parameter values adopted in previous studies were far from the normal values for Red Mud [1], and not conservative.

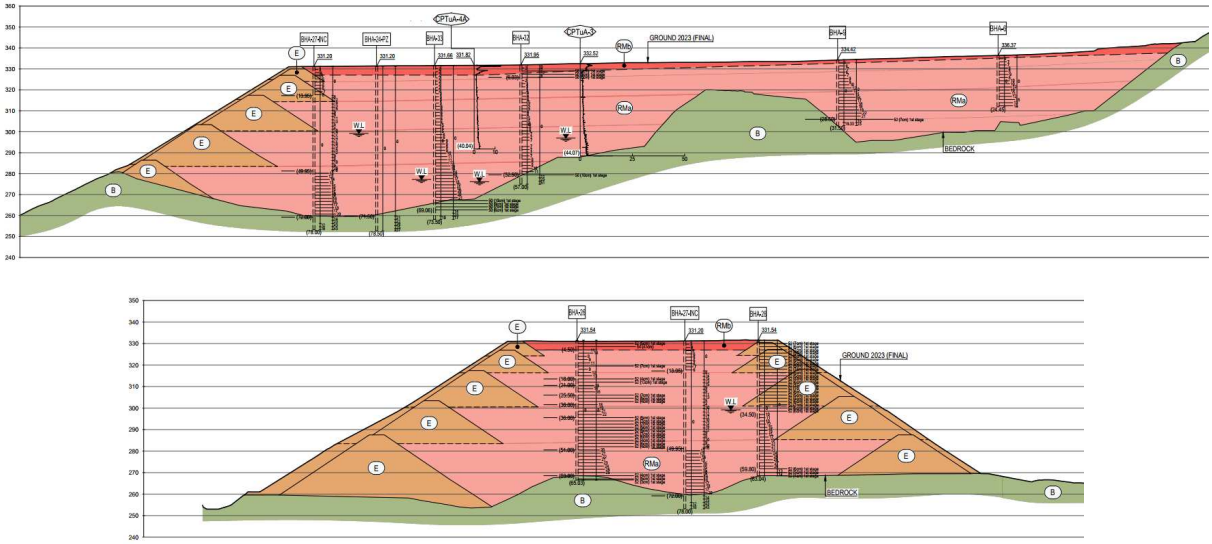


Fig. 2 – Tallest sections in N-S direction (above) and E-W direction (below).

So, all the obtained geological and geotechnical data were processed in detail to identify and parametrize the geotechnical units. These correspond to the same geological units identified in the site. The geotechnical parametrization was a complex process made through the in situ tests previously mentioned and by laboratory tests. The main aspects that lead to the geotechnical parametrization will be presented in a simplified way, focusing only on the main aspects, as detailing parametrization is not the main objective of this paper.

Besides the aforementioned in situ tests, laboratory tests were also conducted, mainly to obtain the material's identification, the physical indexes and the chemical and mineralogical analysis.

The parameters that better represent the in situ state of a soil are the bulk and dry densities, respectively γ_h and γ_d, and the void ratio, *e*. Understanding the values for the densities depends also on knowing the natural water content (w) and the specific gravity (G_s) of the materials tested. And, in cases as the red mud, where the specific gravity is high, the void ratio is the only parameter that really allows to understand the soil state.

The variability of values in the bulk density is a result of the amount of water in the soil and can be analysed trough the degree of saturation (S). The degree of saturation of the soils represents the relation of the amount of water to the void ratio. These two parameters are the ones to have into account when analysing the state of a natural soil (or tailing) in situ.

It should be noted that the values of e, γ_d and S were estimated based on the other measured physical parameters (γ_h, w and G_s) through the relations between these indexes (Fig. 3).

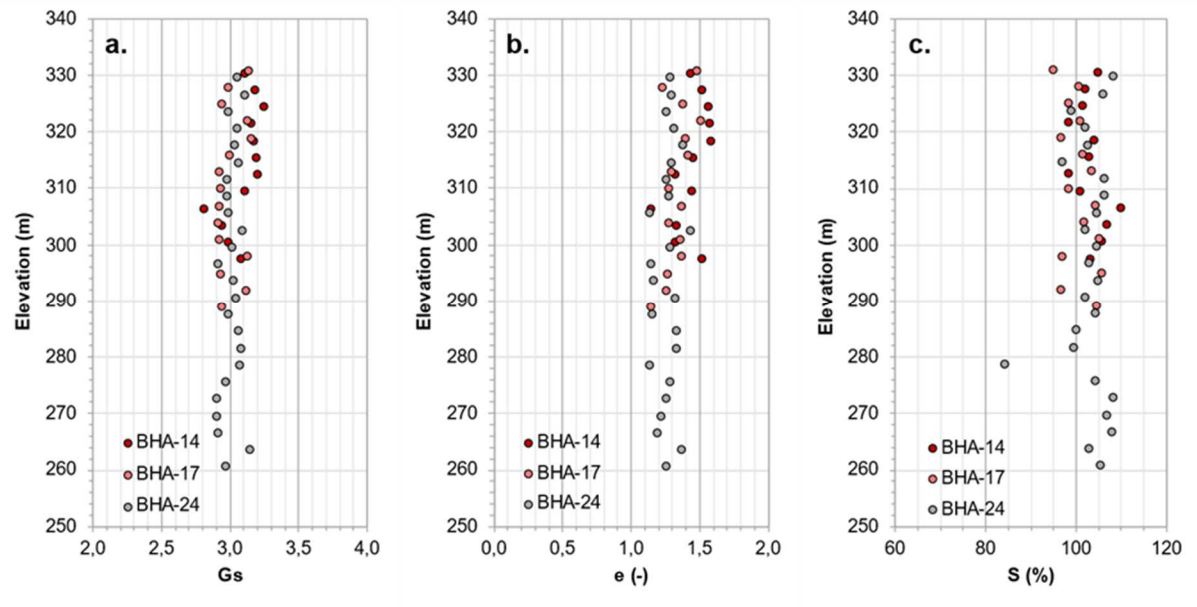


Fig. 3 – Examples physical indexes: a. specific gravity; b. void ratio; c. saturation degree

The specific gravity values obtained are in accordance with what is expected for typical red mud values and show an average value of 3, reflecting the large amount of iron left in the tailings. The red mud existing in this TSF site presents void ratio between 1.1-1.6 and the estimated degree of saturation values varies mainly from 84% to over 100% throughout the thickness of the deposit.

Even though there is a large amount of geotechnical data, not all data contributed with the same degree of relevance to the geotechnical behaviour characterization of each unit. For instance, the geotechnical behaviour of the red mud was only possible to analyse in the in situ resistance tests, mainly CPTu tests (Fig. 4). The CPTu data were processed using CPeT-IT v.3.9.2.21 software (GeoLogismiki). Due to the big amount of data and results, CPTu detailed processing and analysis will not be presented here. Nonetheless, data was processed following the Robertson recommendations (see for instance [5]) to obtain the main geotechnical parameters.

Additionally, the approach proposed by Robertson [5] was used to establish the differences in red mud behaviour and to verify its susceptibility to flow liquefaction (Fig. 5 and Fig. 6). The susceptibility to flow liquefaction was also verified using the Bishop´s brittleness index (Fig. 7), as recommended in [6].

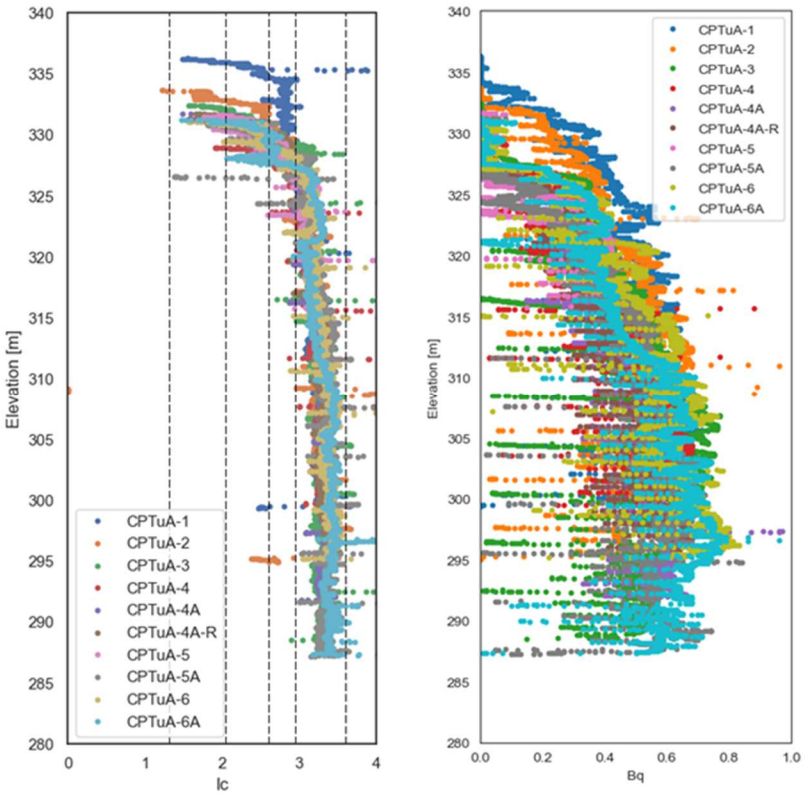


Fig. 4 – Examples of CPTu results: Behaviour index and Bq

Additionally, the approach proposed by Robertson [5] was used to establish the differences in red mud behaviour and to verify its susceptibility to flow liquefaction (Fig. 5 and Fig. 6). The susceptibility to flow liquefaction was also verified using the Bishop´s brittleness index (Fig. 7), as recommended in [6].

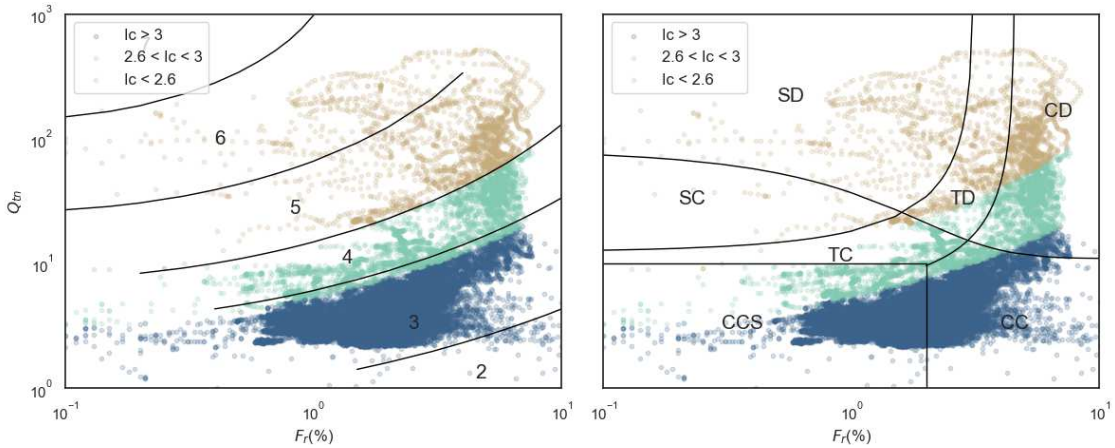


Fig. 5 – Robertson SBTn and updated SBTn charts

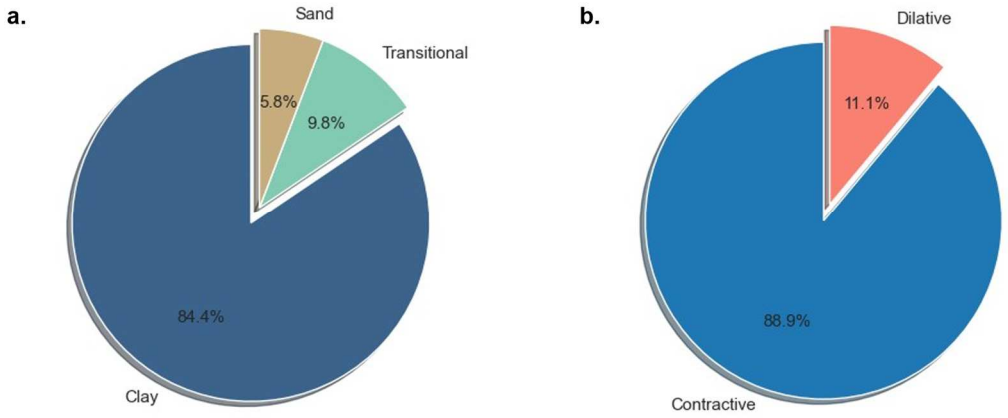


Fig. 6 – Percentage of points with: a. sand, transition and clay behaviour; b. dilative and contractive behaviour according the Roberston updated SBTn classification

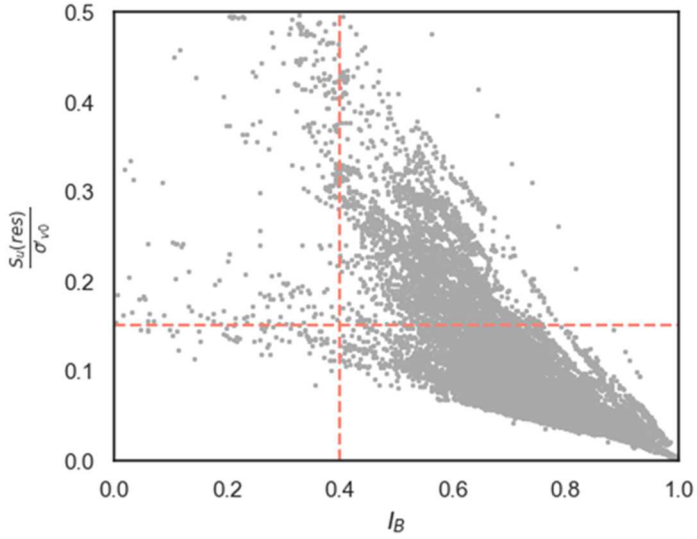


Fig. 7 – Undrained shear strength ratio at critical state versus brittleness Index (I_B) chart (87% of points are inside the critical area)

From the combined analysis of in situ and laboratory test, presented above, was verified that the red mud tailings inside the dam are mainly of clay like behaviour, normally consolidated, saturated, with contractive and brittle behaviour and so, prone to flow liquefaction.

To perform the parameterization of Red Mud geotechnical unit, the data was processed using different statistical procedures to obtain the characteristic value that best represents the variability of the deposit. According with Eurocode 7 [7] the characteristic value of a geotechnical parameter should be selected as a careful estimate of the value that affects the occurrence of a limit state. In a simplified way, two different approaches were used:

- In the case of the parameter presenting a normal distribution (for example, bulk density, specific gravity and drained resistance), the recommended procedures of Eurocode 7

were followed to determine the characteristic value of the geotechnical parameters (Fig. 8);

- In determining the characteristic value of peak and residual undrained shear strengths other recommendations were also followed. According to [8], the characteristic value of strength tends to be log-normal and, in this case, the arithmetic mean and standard deviation are not representative of this distribution. The authors recommend the adoption of the 20th percentile of the distribution, which is the parameter closest to the most frequent value in the distribution, which ensures that 80% of the material's resistance values should be higher than this value.

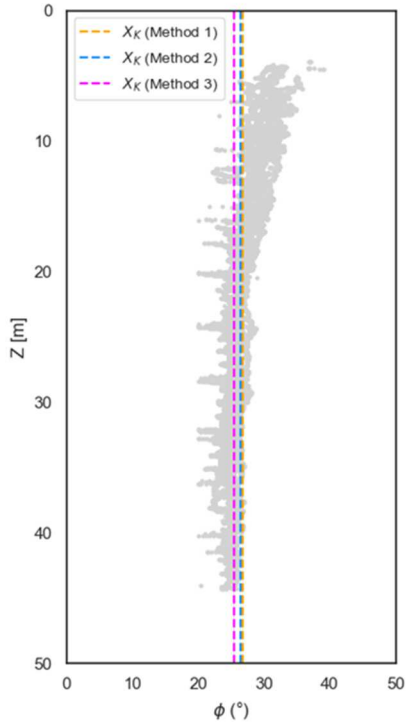


Fig. 8 – Example Red Mud parametrization process presenting the differences of the determined characteristic values (obtained by 3 methods)

The embankment is composed of a non-compacted soil-rock mixture, besides SPT tests no other information exits, and the SPT results are mainly refusals. These values are not representative of the matrix of the material but represent the existence of rock in the gravel dimension (or higher). Nonetheless, the adopted parametrization was sustained on the embankment state observations, on the lithology, on values obtained from literature and considering the team’s knowledge of the parameters of structures with similar grain size distributions.

Table 1 summarises the obtained geotechnical parameters that were used for the stability analyses.

Table 1 – Summary of geotechnical parameters for stability analyses

Geotechnical Unit	γ (kN/m^3)	Su/σ'$_{v0}$	c' (kPa)	φ'(º)
RM undrained – peak	19	0.26		
RM undrained – residual	19	0.04		
RM drained	19		0	25
Embankments	21.5		0	37
Bedrock		Impenetrable		

5. POREPRESSURE IN THE STACK

Dissipation tests executed in 6 CPTu performed in the red mud deposited in the TSF showed the existence of positive porepressure in most of the red mud. Of the 6 CPTu, 2 of them were conducted in an excavated portion in the north of the facility (CPTu-A1 and CPTu-A2) and 4 in the stack (CPTu-A3 to CPTu-A6). As presented in Fig. 9, the tests indicated though that the measured porepressure are not consistent with an elevated steady phreatic surface as they do not converge to a hydrostatic line. Since the TSF has been inactive for a few years, is located mostly above natural ground level and do not hold any water upstream, it is unlikely that this behaviour could be related to downwards seepage or severely inclined flow lines. In fact, the most likely scenario is that the measurements refer to a positive excess porepressure yet to dissipate due to the deposition of the red mud in a very loose state, and that the consolidation process is still in progress within the red mud layers.

Regardless the nature of mechanism behind the measured porepressure, they must be used in the calculations as close as possible to the in-situ measurements. Excluding the upper desiccated layer, the difference between the measured porepressure and a hypothetical hydrostatic line placed on a reference level determined by water level in the foundation tends to show a fairly constant relation with total vertical stress, both above and below that level. This behaviour was modelled in software for slope stability calculations with the $\overline{B}$ parameter in association with a phreatic surface, being $\overline{B} = {}^{\Delta u}/_{\Delta\sigma_1}$. To achieve better adjustment, the stack was zoned and two different values of $\overline{B}$ were adopted: $\overline{B} =$ 0.05 for CPTu A-1 to A-3 and $\overline{B} =$ 0.10 for CPTu A-4 to A-6, resulting in 2/3 of the errors ($u_{model} - u_{measured}$) in the positive side, with a mean of 6kPa. Although not originally intended for modelling porepressure above the phreatic level, the use of $\overline{B}$ resulted in porepressure values in the model fairly similar to the measurements, as presented in Fig. 10.

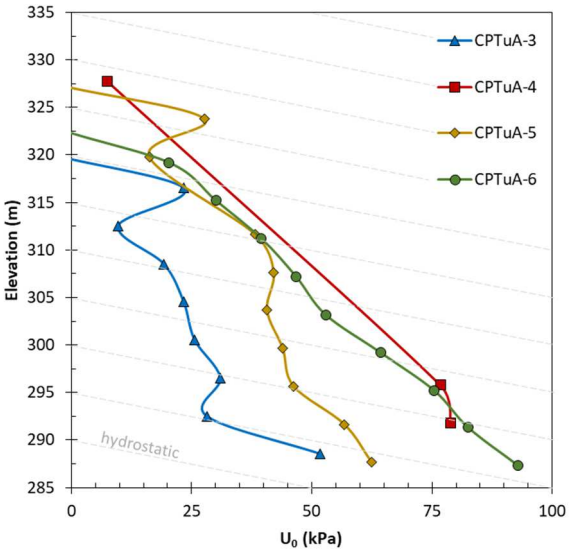


Fig. 9 – Porepressure measured at the end of dissipation tests

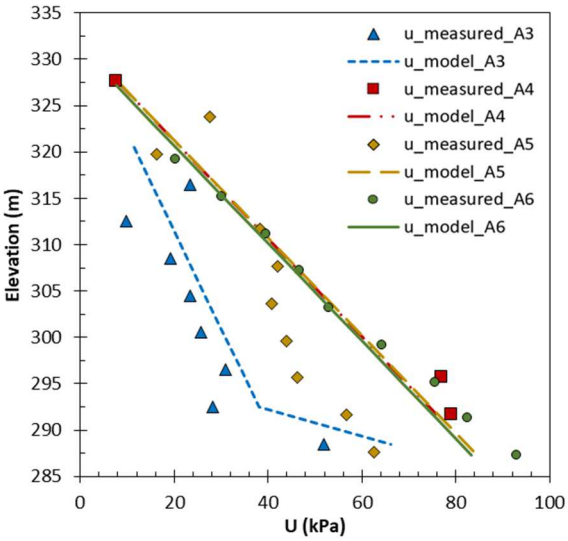


Fig. 10 – Porepressure measured and modelled

There are no oedometer tests in the material or any other reliable test that could provide a fair estimation of the expect time to dissipate this excess porepressure. The consolidation parameters estimated from the CPTu tests are in a very broad range and, therefore, a consolidation assessment based on them bears so much uncertainty that is not relevant. For that reason, it is not possible to define how the stability of the facility is expected to evolve as a function of time. Nevertheless, the factor of safety was calculated for the current state and for the future condition, when excess porepressure is dissipated.

As the facility does not have a reservoir in its currently configuration, seepage analysis considering variations in upstream boundary conditions due to extreme flood events are not essential. Since it cannot hold water permanently, changes in the upstream boundary would be transient and not long enough to create a new steady condition in the low permeability tailings. Nevertheless, a 3m rise in the phreatic level in the foundation were considered to account for potential seasonal fluctuations of the water level in the foundation.

6. GLOBAL STABILITY ASSESSMENT

The slope stability analyses of the facility were undertaken for 5 different scenarios, according to the impositions of the local code and recommendation of international organizations. The scenarios and minimum factors of safety are presented in Table 2.

Table 2 – Calculation scenarios and recommended factor of safety

	Calculation scenario	Recom. FoS
Sc1	Undrained response of the RM, static condition, current porepressure	≥ 1.3
Sc2	Undrained response of the RM, static condition, end of consolidation	≥ 1.3
Sc3	Drained response of the RM, static condition	≥ 1.5
Sc4	Undrained response of the RM, pseudo-static condition, no liquefaction	≥ 1.0
Sc5	Undrained response of the RM, post-seismic condition, liquefaction	≥ 1.0

The factors of safety against slope instability were calculated by the method of slices, formulation of Morgenstern and Price (1965) with half-sine functions for the interslice shear forces, using the software GeoStudio v.2023 for the calculations. Circular and non-circular failure surfaces were searched, with the results of the last only considered valid if judged physically credible.

Pseudo-static analysis are a requirement of the local regulatory code. Although more advanced methods based on dynamic analysis are currently considered a better practice than the pseudo-static analysis, the lack of data to perform them would require the estimation of the most significant parameters, and the study would lose its reliability. The seismic coefficient for the pseudo-static analysis was determined according to the local code for seismic design, resulting in a horizontal coefficient k_h = 0.24g and a vertical coefficient k_v = 0.12g acting upwards. The code does not state the return period associated with the reference actions, and requires the use of an importance factor of 3 for dams. Considering reasonable to assume that the reference actions relate to a return period of 475 years, an importance factor of 3 would result in actions with return period in the range of the 10000 years. For the pseudo-static analysis, 20% reduction of shear resistance was applied to all materials.

The assessment of the potential for seismic liquefaction was conducted based on the NCEER methodology [9]. Regardless of the limitations the application of this methodology may have when assessing liquefaction potential of a tailings stack and not soils, it is a requirement of the local regulatory code. It is central to mention that the code does not present return periods for the magnitude of the earthquake, but only a rudimentary map of large historical seismic events recorded in the country. An extensive literature review in that mater did not yield more reliable information. Therefore, the input magnitude for the analysis was defined as the largest earthquake recorded in the zone, acknowledging the clear weakness this lack of information adds to the analysis. The factor of safety against liquefaction is already below the unity for the reference acceleration of the code (assumed as 475 years), as Fig. 11 presents, suggesting an unacceptable probability of occurrence.

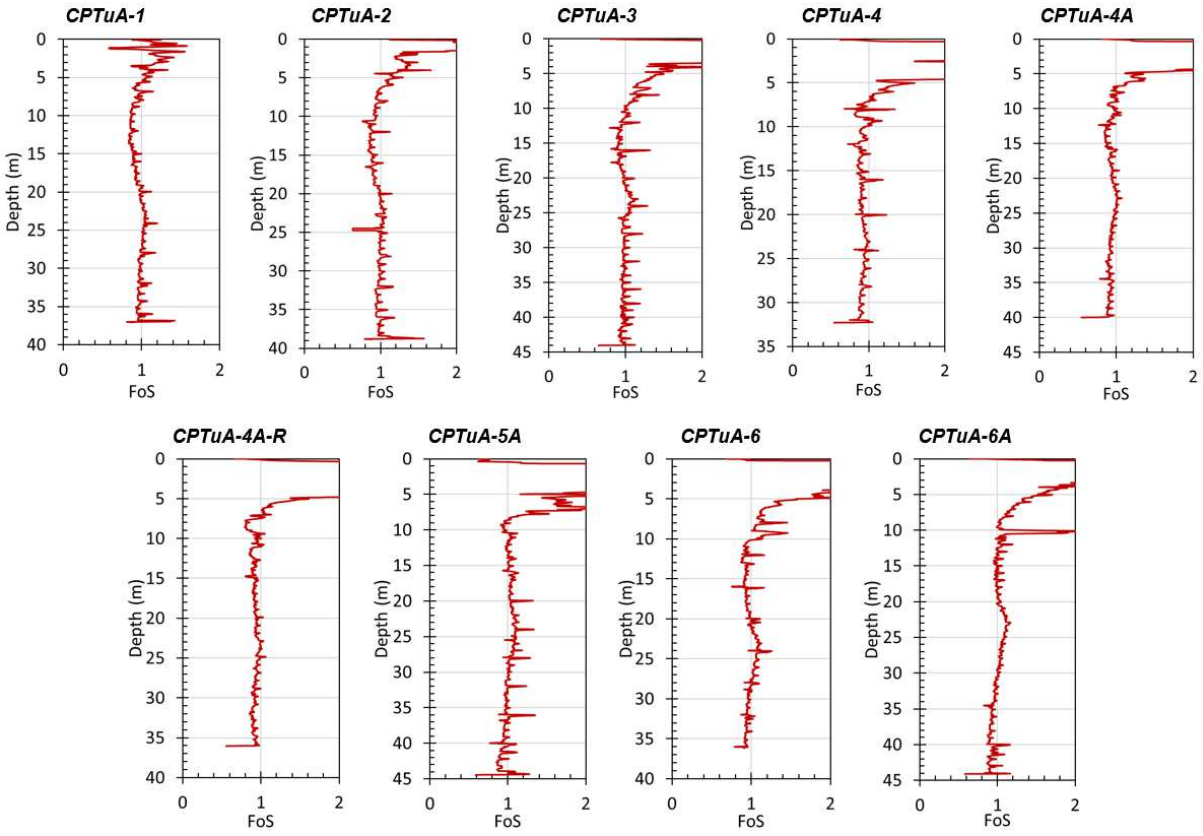


Fig. 11 – Calculated factor of safety against liquefaction

For each scenario, 3 analyses were performed, one in the N-S section and two (both directions) in the E-W sections (Fig. 2). In every case, due to the assumed regularity of the geometry of the raises, the shape of the critical slope surface and the calculated FoS were essentially identical. Therefore, the critical slip surfaces for each scenario are showed in Fig. 12 to Fig. 16 only for the N-S section. In every case, the optimisation of the critical slip surface resulted in a marginally smaller FoS, but it did not represent a physically credible failure mechanism. For that reason, only the results of the circular surfaces were considered.

As presented in Table 3, the calculated FoS is significantly smaller than the minimum required in every scenario assessed, with the facility presenting a marginal safety condition under static load and an failure situation under the assessed seismic conditions.

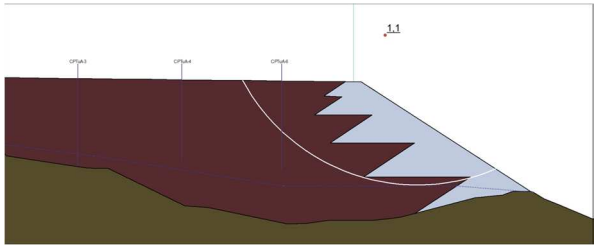


Fig. 12 – Critical slip surface for Sc1

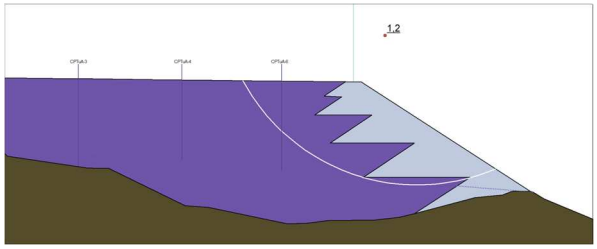


Fig. 13 – Critical slip surface for Sc2

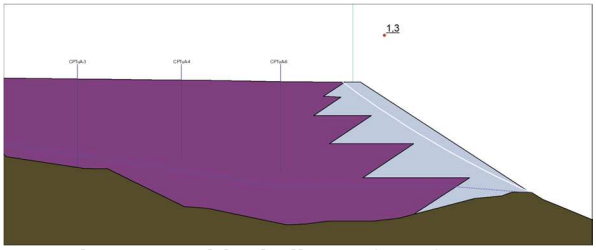


Fig. 14 – Critical slip surface for Sc3

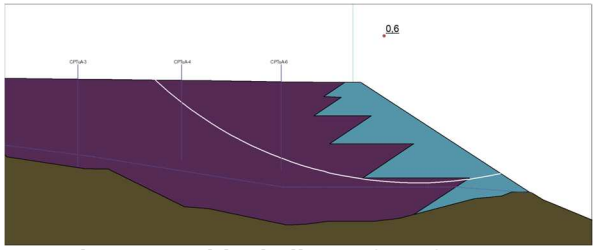


Fig. 15 – Critical slip surface for Sc4

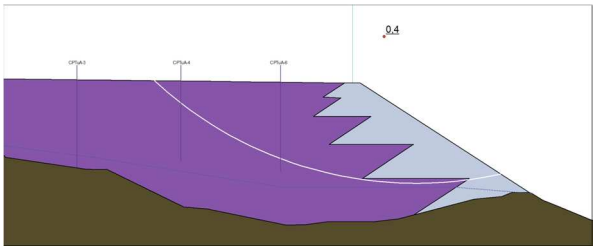


Fig. 16 – Critical slip surface for Sc5

Table 3 – Calculated factors of safety for each scenario

Calculation Scenario	Calculated FoS
Sc1	1.1
Sc2	1.2
Sc3	1.3
Sc4	0.6
Sc5	0.4

The exit point of the failure surface is the same (right above the initial embankment) for all cases in which the critical failure surface is governed by the red mud. That indicates that the calculated critical slip surface could occur in any section in which the first raise is founded on red mud. Different alignments also result in equivalent FoS, conditioned as well by the initial embankment, since it is expected that this embankment holds similar geometry in all directions.

Due to the geometry of the basin, for the phreatic surface at the bottom of the stack to have considerable influence on the critical FoS, a substantial increase on the level would be necessary. The analysis presented in Fig. 12 to Fig. 16 already considered an increase of 3m in the original estimated phreatic surface to account for the possibility of seasonal fluctuations. Yet, critical slip surfaces tend to remain above that level due to exit restriction imposed by the initial embankment. Other assessed sections present even lower phreatic surfaces.

The critical slip surface in all analysis is governed by the shear strength of the red mud, except for the scenario that assess the drained response of the tailings. For that scenario, the critical slip surface is shallow, almost linear and remains restricted to the embankment, being,

therefore, governed by the embankment's shear resistance. Although this case does not represent the most critical concern regarding the overall condition of the facility, further investigation of engineering properties of the embankment is strongly recommended, since there is still a large amount of uncertainty in the definition of the strength parameters of the embankment, as discussed previously.

7. CONCLUSIONS

This case study illustrates well how complex and challenging it may be for the industry to comply with the currently requirements that guide TSF design, management and closure. In a recent past, tailings management was frequently done as part of the mine operations and not an independent activity and, as such, did not meet the best engineering practices, as it is currently required. The TSF in case, just as many facilities all over the world, was designed and has been managed based on much looser standards, and bringing them to meet the new requirements and guidelines poses as a complex challenge. The difficulties appeared in different forms: lack of basic engineering reports, as ground characterization and topography; no construction or deposition records; poor characterization of tailings and construction materials; no safety analysis or any kind potential failure modes assessment; insufficient maintenance and monitoring.

From a more objective perspective, the primary and most important conclusion of the analysis is that the TSF in case does not present minimum safety conditions and require immediate actions to comply with basic safety requirements. The calculated deterministic FoS for the current condition is dangerously close to the unity, and the structural reliability of the facility may be even worse than the deterministic result may suggest at first glance. As discussed, the TSF does not have design or as-built records, construction drawings or reports, or any sort of previous relevant documentation that could increase the confidence on the engineering involved in the construction and operation of the facility. Geometry of the embankments, geotechnical properties of materials, zoning, porepressure conditions, and foundation had to be almost entirely recreated, extrapolated, interpolated, estimated, or simply assumed, often based on very little (good quality) information. This type of as-is analysis bears, therefore, a huge amount of uncertainty and the confidence it holds cannot be compared to a fully engineered project with properly accompanied construction and operation with good monitoring and maintenance. Therefore, the marginal deterministic FoS obtained associated with this extreme level of uncertainty indicates an unacceptable elevated probability of failure, much higher than a well-engineered facility would have for the same deterministic FoS.

REFERENCES

[1] Lopes, I.; Garcia, M., Tavares, G. (2024). Challenges in the geotechnical characterisation of bauxite tailings (Red Mud). Proceedings of the XVIII ECSMGE 2024. Lisboa.

[2] ICMM. 2021. Tailings Management. Good practice guide. International Council on Mining & Metals. Mining With Principles. 128 p.

[3] MAC. 2021. Developing an Operation, Maintenance, and Surveillance Manual for Tailings and Water Management Facilities. Version 2.1. The Mining Association of Canada. 75 p

[4] Lopes, I.; Garcia, M., Tavares, G. (2024). Difficulties in building a ground model when lacking historic data archives and its impact in TSF safety assessment. 7th International Conference on Geotechnical and Geophysical Site Characterization. Barcelona, 18 - 21 June 2024.

[5] Robertson, P.K., Cabal, K.L. (2022). Guide to Cone Penetration Testing for Geotechnical Engineering, 7th Edition, Gregg Drilling & Testing, Inc.

[6] ICOLD. 2023. Tailings dam safety bulletin 194.

[7] EN 1997-1 (2004). Eurocode 7 - Geotechnical design - Part 1: General rules, European Committee for Standardization, Brussels, 168 p.

[8] Jefferies, M., Been, K. (2016). Soil liquefaction: a critical state approach. CRC press.

[9] Youd, T. et. al (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, J. Geotechnical and Geoenvironmental Eng., ASCE 127 (10): 817–33.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

L-BAND SAR DERIVED SOIL MOISTURE MAPS FOR THE MONITORING OF EARTHEN DAMS IN SCANDINAVIA

Mark Hinton[1]
ASTERRA

Jonathan Lynch
ASTERRA

Yonatan Rabinovitch
ASTERRA

ABSTRACT.

Manmade earthwork assets are increasingly becoming susceptible to changes in the earth's climate and in particular to the more frequent and intensive rainfall events which creates extensive damage, or in the worse case causing ultimate failure of the assets.

From collapsing dams to failing embankments, disaster can strike when engineers fail to account for the presence of water in soil. L-Band SAR soil moisture analysis addresses this risk by detecting underground soil moisture as deep as 3 meters below ground using algorithms combined with AI on satellite Synthetic Aperture Radar (SAR) data.

This proprietary methodology was originally used in the search for water on Mars and has since been adapted to monitor soil moisture around critical infrastructure on Earth from orbit. The process uses data from commercial satellites equipped with L-Band SAR, which can penetrate through clouds, vegetation and soil.

The satellite imagery is then processed and analysed before delivering hard intelligence to planners, engineers, and policymakers. Initially, an installation is divided into a grid, and areas in the monitored installation known to the operator as areas of concern are marked as priority. "Alerts" or "alarms" indicate where soil moisture levels pre-defined by the operator have been exceeded. These alerts and alarms allow decision-makers to monitor problematic areas for changes in soil moisture, and make data-informed choices about the repair, maintenance, and long-term planning of above and below-ground infrastructure.

The presentation will present the case of a specific dam in Scandinavia, the challenges it faces surrounding subsurface soil moisture, image acquisition and analysis in heavy winter conditions, and results of the field investigation which demonstrate success in identifying seepage locations.

Keywords: SAR, Soil Moisture, Satellites, Earthworks, AI

[1] mark.hinton@asterra.io, UK

Introduction

According to the International Commission for Large Dams (ICOLD, 2020), there are over 62,000 registered dam structures globally. Of this number, 65% are earth dams. This data accounts for a small percentage of dams worldwide. There are many smaller unregistered structures which are not included in the ICOLD register, the number of which could be in the hundreds of thousands.

There are over 15,000 earth dams in Canada according to the Canadian Dams Association (CDA, 2024), many of which are between 50 and 100 years old, and many constructed prior to the current understanding of modern soil mechanics principles. 1,157 of these are classified as large under the ICOLD definition. 571 of these large dams are of earth construction and include the W.A.C. Bennet dam, standing 183m tall, being one of the worlds largest earthfill structures.

Given the topography of Canada flooding is a major problem due to the abundance of water and snow melt, plus the additional consequences of climate change. Within Canada there are also a significant number of earth levees designed for flood control and water management. These levees are also aging structures with growing populations, agriculture and infrastructure located in close proximity.

The number of global Tailings Storage Facilities (TSF) which hold back mine wastes rather than water, is more of an unknown with little accurate data available, which could range anywhere between 3,500 to more than 30,000, (worldminetailingsfailures.org, 2020). What is clear is that these TSF require monitoring, both regularly and properly.

Events such as the collapse at the Jagersfontein TSF in South Africa in September 2022, the Edenville dam breach in Michigan USA in 2020, and the devastating TSF failure at Brumadinho in Brazil in 2019, have brought the failures of large earth dam structures dramatically into the public attention. Around the globe, many engineers, regulators, and investors are now paying more attention to the activities of mine and dam/reservoir operators, ensuring that operators are held accountable, that regulations are followed, and that regular monitoring and maintenance is performed as it should be.

The majority of dams worldwide (excluding TSF) are embankment dams, of which 64% are earthfill dams (Adamo et al, 2020), the remaining being either rockfill dams, concrete dams or those of other types. A study conducted in 1998 by Foster et al on large embankment dams (based on ICOLD World Register of incidents up to 1986) concluded that out of a

total of 136 large dam failures, 121 of these were due to hydraulic issues (Table 1). Of these, 59 were attributed to piping failures either through the embankment, through the foundation or a combination of the two. The output from this study suggested that piping caused failures are nearly half of all the failure cases within embankment dams.

This mode of failure should also be seriously considered for all TSF's of which there are reported to be approximately 3,500 worldwide (mining.com, 2019).

Table 1. Summary of Embankment Failures, after Adamo et al, 2020

Mode of Failure	Number of Cases		Percent failures (where known)	
	All failures	Failures in operation	All failures	Failures in operation
Overtopping	46	40	35.9	34.2
Spillway/gate	16	15	12.5	12.8
Subtotal	62	55	48.4	47.0
Piping through embankment	39	38	30.5	32.5
Piping through foundation	19	18	14.8	15.4
Piping from embankment into foundation	2	2	1.6	1.7
Subtotal	59	57	46.1	48.7
Slides	7	5	5.5	4.3
Earthquakes/Liquefaction	2	2	1.6	1.7
Unknown	8	7		

Total number of failures	136	124

Many dams were constructed prior to any knowledge and understanding of climate change and the expected increase in frequency and volume of inclement rainfall events causing higher water levels and increases in potential flood waters which could affect existing dam and levee infrastructure initially not constructed for such short dramatic increases in water level.

Climate Change

The onset of climate change and its impact on the earth weather systems, sea levels and air temperature increases are widely publicized and recognized amongst most governments, industry, and the academic communities. Globally the effects of this are far reaching and for existing geotechnical infrastructure these effects need to be addressed and fed into existing asset management plans.

In more northern climates such as Canada or Scandinavia, the more rapid release of stored water through snow melt or the thawing of permafrost is something that is now occurring. In addition, these increases in rainfall and snow/permafrost melt are likely to bring with them added debris and sediment. Sedimentation over time can lead to differences in dam capacity. Although this will take a number of years to accumulate it can become significant over time (ICOLD, 2016).

Northern climates also face the problems associated with ice flow and the potential blockage of dams and spillways through ice build up leading to overtopping of the dam elsewhere. The spillways themselves if operational can also be subject to rapid water flow. More recently the effects of under design or poor maintenance were seen on the Oroville dam spillway (Figure 1) in the United States (2017) and at Toddbrook Reservoir in the United Kingdom (2019), both situations which led to the evacuation of people downstream of the dams.

Figure 1. Oroville Spillway, northern California US. Damage to spillway following significant rainfall during February 2017.

Increases in water levels in dams as a result of increased runoff and rainfall could lead to additional hydrostatic loadings onto the earth structures and therefore the potential for internal seepage could increase. The causes of these flood events such as inclement rainfall or snow melt are also influenced by other factors including vegetation and soil moisture within the catchment areas (Mostbauer et al., 2017).

Conversely the impacts of climate change include periods of prolonged drought which potentially could lead to the formation of desiccation cracking on the upstream side of dams and this in turn may lead to further impacts of seepage when dams levels return to normal (Li et al., 2018).

Seepage

One of the most frequent forms of dam failure is caused by internal erosion or seepage. Zones of weakness formed during or post construction, may be exploited by water under pressure within the dam structure, eventually find it's way to the dam surface. Depending upon the pressures and the soil types present there may be erosion of fine soil particles which can eventually be washed out leading to piping effects, causing further erosion and an increase in seepage volume and velocity. This can lead to catastrophic failure if undetected and not prevented.

The detection of seepage is therefore of paramount importance to dam operators. However, the presence of seepage is not always easily seen apart from the obvious signs

of water at the surface or changes in vegetation on the dam surface or at the toe. Seepages which have not yet manifested at the surface may take years to surface if at all, however knowledge of their positions is still vital to undertake the appropriate investigations, to make appropriate engineering assessments and to understand the dynamics of each dam, which are all different.

Typical methods of monitoring for seepage include instrumentation within the dam walls such as piezometers, fibre optic cables or temperature gauges. However, these instruments provide information which is limited to the areas they are installed. Whilst providing invaluable information the overall status of most dams remains unknown. Without blanket coverage of a dam (which would not be cost effective), the fact remains that instrumentation although useful does have its limitations.

ASTERRA EarthWorks

ASTERRA use L-Band Synthetic Aperture Radar (SAR) to identify and quantify the specific dielectric signature of soil moisture. This specific wavelength allows enables the differentiation between different types of water such as groundwater and saline water (Gadani et al, 2012). L-Band SAR sensors are set at a wavelength capable of seeing through clouds and rain, day and night, it can penetrate vegetative cover, dry snow and ice, and urban surfaces such as asphalt to see into the soil layer capturing the subsurface soil moisture levels from dielectric properties which differ depending upon water type. In this case, the SAR is mounted on a satellite in polar orbit.

The use of proven algorithms allow L-Band SAR data to be converted to soil moisture content which can be displayed as contours for engineers and asset owners to view potential areas of concern and verify existing problem areas.

Using L-Band SAR it is possible to create soil moisture maps (Figure 2), with specific information highlighting shallow subsurface soil moisture content across the entire face of any earth dam. Contoured soil moisture heat maps enable asset owners and engineers to visualise and locate their inspections for known seepage areas and more importantly target those unknown areas where the moisture has not yet manifested itself on the surface.

Figure 2. Dam with soil moisture mapping overlay. Image on the right showing smart cell application with Alert (yellow) and Alarm (red) areas highlighted.

Users of this technology can view the information within a web based viewer which may also be loaded onto a mobile device for use in the field. The same data can be downloaded for use with any GIS system enabling users to perform further geospatial analysis or loading as a layer into an existing database.

Repeatable SAR images can be taken for further analysis allowing for time series monitoring. Typically this would be on a seasonal basis however this can be altered depending upon individual requirements. A targeted investigation or supplemented monitoring and instrumentation approach may be developed based on L-Band SAR results.

Scandinavia Example

Background

Dam safety is paramount especially in regions like Sweden where freeze-thaw cycles and long-term wear can heighten the occurrence of seepage risk. Given that traditional monitoring methods have limitations, ensuring the safety and stability of dams in these regions requires innovative solutions and advanced monitoring to address these unique environmental challenges.

Challenge

Uniper, a global energy leader, faced this challenge with the Lövön Dam in Sweden. While they had existing instrumentation and sensors, they sought to improve assessment

accuracy and were keen to explore the monitoring of the dam using L-Band SAR, to determine both existing and potential seepage locations.

Solution

Uniper enlisted ASTERRA EarthWorks to evaluate the dam, aiming to identify areas with elevated concentrations of subsurface soil moisture potentially linked to seepage.

EarthWorks provides a valuable data layer to gain an overview of an entire dam surface and identify at-risk areas. The EarthWorks smart cell system established a digital grid to monitor individual areas for soil moisture levels exceeding pre-set alert or alarm thresholds, enabling early warnings for Uniper to address potential issues proactively.

Despite harsh winter conditions, EarthWorks L-Band SAR analysis effectively penetrated snow cover, capturing soil moisture data within the dam soil structure beneath snow cover. This analysis identified several areas with potentially elevated seepage, prompting the alert system to notify operators of at-risk zones.

Figure 3. Example smart cell dashboard analysis; Source: ASTERRA

This proactive approach directed field crews to specific sites around the dam for seepage inspection and verification. Despite the covering of sno, field inspections revealed the unmistakable signs of seepage, evidenced by localized snowmelt due to temperature differentials from water exfiltration. This alignment of EarthWorks' smart cell alarms with areas of higher temperatures identified by existing dam instrumentation, enhanced Uniper's confidence in the accuracy of the L-band SAR data.

Figure 4. Photo from ground truthing inspections showing areas of snow melt associated with seepage. Source: ASTERRA

Figure 5. Smart Cells in yellow (alert) and red (alarm) highlighting areas of concern, whilst those in blue are client known areas of seepage. Source: ASTERRA

Conclusion

The use of L-Band SAR for monitoring and safety has been explored for both earth dams and tailings storage facilities (TSF). Internal erosion of earth dams and TSF is one of the major causes of failure, the consequences of which are far reaching including loss of life, environmental disaster, loss of reputation and significant financial penalties. Signs of

internal erosion do not often manifest themselves at surface, hence visual inspection and non-targeted investigations or instrumentation, are unlikely to detect the tell-tale signs that seepage or erosion is present. L-Band SAR can detect moisture below the ground surface and determine the presence of high soil moisture concentrations which may be indicative of areas of internal seepage, in advance of this manifesting itself at the surface.

This crucially provides the asset owner with time and location. The time required for targeted inspection, investigation, and the design of intervention measures to prevent potential catastrophic failure of their assets.

Of course, the use of L-Band SAR alone will not prevent potential failure. Failure modes often manifest themselves in many ways with additional factors contributing toward the bigger picture. Preventative measures, rather than reactive measures, are fundamental to prolonging the life of dams and TSF's, to enable these structures to deal with the onset of climate change.

L-Band SAR can have an important part to play in the determination of failure prediction and forecasting. With a greater understanding of asset behaviour using L-Band SAR, combined with the already existing investigation and monitoring techniques, dam owners, mining companies and asset managers will be better able to direct resources and funding to the correct areas to prevent the onset of failure exacerbated by climate change effects

The Lövön Dam case study provides a demonstration of EarthWorks effectiveness in enhancing dam safety. It not only confirmed existing concerns but also unveiled new areas requiring attention, whilst demonstrating the capabilities of L-Band penetration through snow cover. Actionable insights empower dam operators to make informed decisions for proactive dam management and ensure safety.

REFERENCES

Adamo, N., Al-Ansari, N., Sissakian, V., Laue, J. and Knutsson, S. (2020). Dam Safety: Problems Related to Seepage. *Journal of Earth Sciences and Geotechnical Engineering*, 10, pp.191–239.

CDA (Canadian Dam Association). Internet accessed April 24, 2024. https://cda.ca/dams-in-canada/dams-in-canada

Foster, M., Fell, R. and Spannagle, M. (2000). The statistics of embankment dam failures and accidents. Canadian Geotechnical Journal, 37(5), pp.1000–1024. doi:10.1139/t00-030.

Gadani, D., Rana, V., Bhatnagar, S.P., Prajapati, A. and Vyas, A.D. (2012). Effect of salinity on the dielectric properties of water. *Indian Journal of Pure and Applied Physics*. 50. pp. 405-410.

ICOLD World Register of Dams. April 2020. Available at: International Commission on Large Dams (icold-cigb.org). Accessed April 2024.

ICOLD, "Global Climate Change, Dams, Reservoirs and Related Water Resources 2016," Bulletin 169.

Li, Ziyang, Wei Ye, Miroslav Marence, and Jeremy D. Bricker. 2019. "Unsteady Seepage Behavior of an Earthfill Dam During Drought-Flood Cycles" *Geosciences* 9, no. 1: 17. https://doi.org/10.3390/geosciences9010017

Mostbauer, K., Kaitna, R., Prenner, D., and Hrachowitz, M. 2017. The temporally varying roles of rainfall, snowmelt and soil moisture for debris flow initiation in a snow-dominated system, Hydrol. Earth Syst. Sci., 22, 3493–3513, https://doi.org/10.5194/hess-22- 3493-2018, 2018

About a tenth of the world's tailing dams have had stability issues. (2019). *Mining.Com* 31 Oct 2019. Available at: https://www.mining.com/about-a-tenth-of-the-worlds-tailing-dams-have-had-stability-issues/. Accessed 16 March 2024.

Estimate of World Tailings portfolio 2020. Available at https://worldminetailingsfailures.org/estimate-of-world-tailings-portfolio-2020/. Accessed April 2024

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

A GIS-GRID NONPOINT POLLUTION AND SELF-PURIFICATION MODEL FOR HYDROPOWER RESERVOIRS

José R Ribas[1] UFRJ
Maria E Arce Universidade da Coruña
Juliana C R Severo FUNDEB
Andrés S Garcia Universidade de Vigo

ABSTRACT

Some possibilities for multiple use of hydroelectric plant reservoirs depend directly on good water quality, hampered by low seasonal water availability, high concentrations of pollutants in effluents and population density around the reservoir, aggravated by changes in the Brazilian forestry code which, among them, reduced the minimum distance from the permanent environmental protection strip from 100 to 30 meters. Self-purification deals with the water body's ability to reestablish its original characteristics, through the action of chemical, physical and biological elements that help to restore dissolved oxygen. This research proposes a GIS-grid nonpoint pollution model integrated with a hydrodynamic modelling procedure for self-purification of aquatic bodies. The study area begins in the reservoir section seven km away from the central axis upstream of the Corumbá IV plant dam, in the central region of Brazil. The input data dataset comprised the interpretation of planimetric and altimetric images obtained from the Landsat 8 satellite, complemented by images captured using a drone during fieldwork. The GIS file is made up of a grid where each cell contains location information, type of land occupation, number of households, occupation limit, household growth rate, hectares of agricultural plantation, heads of cattle, direction of slope of the terrain, soil permeability, number of point loads and sewage production from point loads. The model accounts for organic matter from households and livestock and nutrients from fertilizers, concentrating on the main parameters (N, P and BOD) in specific collection points in water bodies, to Then use Qual-UFMG to simulate self-purification along the reservoir and tributary rivers until reaching the plant's dam. The reservoir pollution scenarios are a function of the predicted degree of anthropization, being called baseline, accelerated occupation and slow occupation. The study horizon coincides with the remaining concession period for the plant, which is 15 years.

Keywords: Non-point pollution, surface water, anthropization, QUAL-UFMG, Corumbá IV hydropower plant.

[1]ribas@poli.ufrj.br, Brazil

1. INTRODUCTION

Given the importance of preserving artificial reservoirs both within the scope of national energy policy and in relation to the country's socioeconomic and environmental interests, Brazil has ensured the creation of several legal instruments over the years to ensure adequate control of human activities associated with the maintenance of this environment.

The analysis of aquatic metabolism allows inferences to be made, from an ecological point of view, regarding the behavior of lakes and reservoirs in relation to contributions from the hydrographic basin (inflow and supply of organic matter and nutrients), human action on the aquatic and terrestrial ecosystem (change in land use and occupation) and climatic conditions.

The trophic state of an environment and the availability of nutrients, mainly nitrogen, phosphorus and carbon, play a fundamental role in maintaining the balance of its ecosystem, since nitrogen and phosphate compounds favor the increase in the biomass of autotrophic organisms, while the greater availability of carbon from organic matter stimulates the respiration of heterotrophic bacteria. Thus, a eutrophic environment can present both an autotrophic metabolism, due to the presence of nutrients, and a heterotrophic metabolism, due to the presence of stored organic matter [1].

Nutrients within a reservoir can have different origins. Often, the supply of nutrients is associated with the transport through surface runoff over the watershed during rain events and depending on the type and use of the soil in the watershed, the supply of nutrients will be greater or lesser. The main nutrients responsible for the balance of aquatic biota are nitrogen and phosphorus, as they are more abundant in nature and, currently, the availability of these elements has been increasing due to the use of fertilizers in agriculture and urban and industrial sewage without adequate treatment. The generation of surface runoff in fertilized areas and in urban areas (incorporating chemicals containing phosphorus), in addition to contributions associated with sewage, can be responsible for the increase in nitrogen and phosphorus levels in the aquatic environment [2]. In these situations, the concentration of nitrogen and phosphorus will depend on the qualitative characteristics of the releases, whether they are point or diffuse. Point sources refer to the release of urban and industrial effluents. In domestic effluents, nitrogen and phosphorus are associated with organic matter and the use of cleaning products. Among the diffuse release sources, agricultural activities deserve special attention, given the intensive use of phosphate fertilizers used as a means of increasing crop productivity.

Measuring diffuse pollution has been the objective of research on water quality. A measure of pollution called the Nonpoint Pollution Index (PNPI) was proposed based on three indicators:

the land cover indicator refers to an ordinal scale with a range of densely populated areas and/or intensive agriculture at one end and areas with natural vegetation and/or a small number of residents at the other end, which refers to the potential generation of non-point pollution due to the land uses of the parcel. Densely built areas and intensively cultivated fields were given the highest coefficients whereas natural and unaltered zones were placed at the opposite end of the scale; the distance indicator measured by the distance from the basin point to the water body; and the runoff indicator calculated by the average of the run-off coefficient from the basin point to the water body [3]. This method was used from 1997 to 2017 in a study on pollution generation in a river in Bolivia [4]. With a modification in the indicators, which included annual local precipitation behavior, the method was adopted to evaluate the nitrate concentration in a river in Spain [5]. It was also combined with the future land-use simulation (FLUS) model to evaluate the spatial-temporal variations of non-point risk caused by land-use change in a rapidly urbanized basin in China during 1980–2030 [6].

An index of potential environmental degradation obtained through the evaluation of some environmental indicators using arithmetic means was used in the analysis of a watershed in Brazil [7]. A GIS multicriteria method based on the flow path to water body, the efficiency of runoff generation and the climatic driving force, was adopted to evaluate the diffuse nitrogen loss potential in a basin in China [8]. The Export Coefficient Model (ECM) Net Anthropogenic Nitrogen Inputs (NANI) methods were used to estimate the spatial and temporal N loads in the Taihu Basin since 1980. The study confirmed the high correlation between population density with N inputs and exports [9]. The indicators adopted to analyze N and P loss in the Three Gorges Reservoir Area in China were crop planting, livestock production, domestic inputs and water-level-fluctuating zone [10]. An indicator called land use impact coefficient was proposed to assess the impact of land use on the export coefficient of the non-point source pollution [11]. Remote sensing data (Landsat data), known points (such as road intersections, etc.) collected from GIS topographic maps and training polygons were inputs to a supervised classification technique based on Gaussian maximum likelihood algorithm to assign the land use category nearby a lake in India [12].

2. MATERIALS AND METHODS

In this study, we analyzed the area of influence of the reservoir near the dam of the Corumbá IV hydroelectric plant, located in the central region of Brazil. The area includes the collection points located downstream of the Descoberto River water treatment plant and the water collection point of the Alagado River, two important tributary rivers. As for the reservoir, the collection point located approximately seven km from the axis perpendicular to the dam defines the boundary of the study. This delimitation is based on the assumption that the body of water

it circumscribes is the one that represents a potential effect on the dam and the plant's equipment, since the dissolution of the chemical parameters will not have sufficient distance to purify the water, allowing the concentration of these pollutants to reach the collection point immediately upstream and near the dam, and the consequent contact with the civil and electromechanical elements of the hydroelectric plant.

The adoption of a grid system aims to divide the study area into smaller parts and based on this division, facilitate the reading and processing of essential information of non-point pollution to diagnose and predict occupation in each microregion, to subsequently apply the water quality model. Each grid becomes an element of information relative to a geographic area. Its definition facilitates the computational process of referencing the various variables of interest, since each grid now has a vector of attributes and values assigned to the variables that can be treated together, in a matrix format, for example. The Corumbá IV HPP reservoir has a study area whose polygon is shown in Figure 1. A total of 204 grids are inserted into this area, each representing relevant information about the area. Having the dam as a reference, the Reservoir collection point is located 6.7 km away from de dam, the Descoberto River collection point is 38.7 km away and the Alagado River collection point is 12.7 km away, which are denoted by RES-1, DE-1 and ALA-1 respectively.

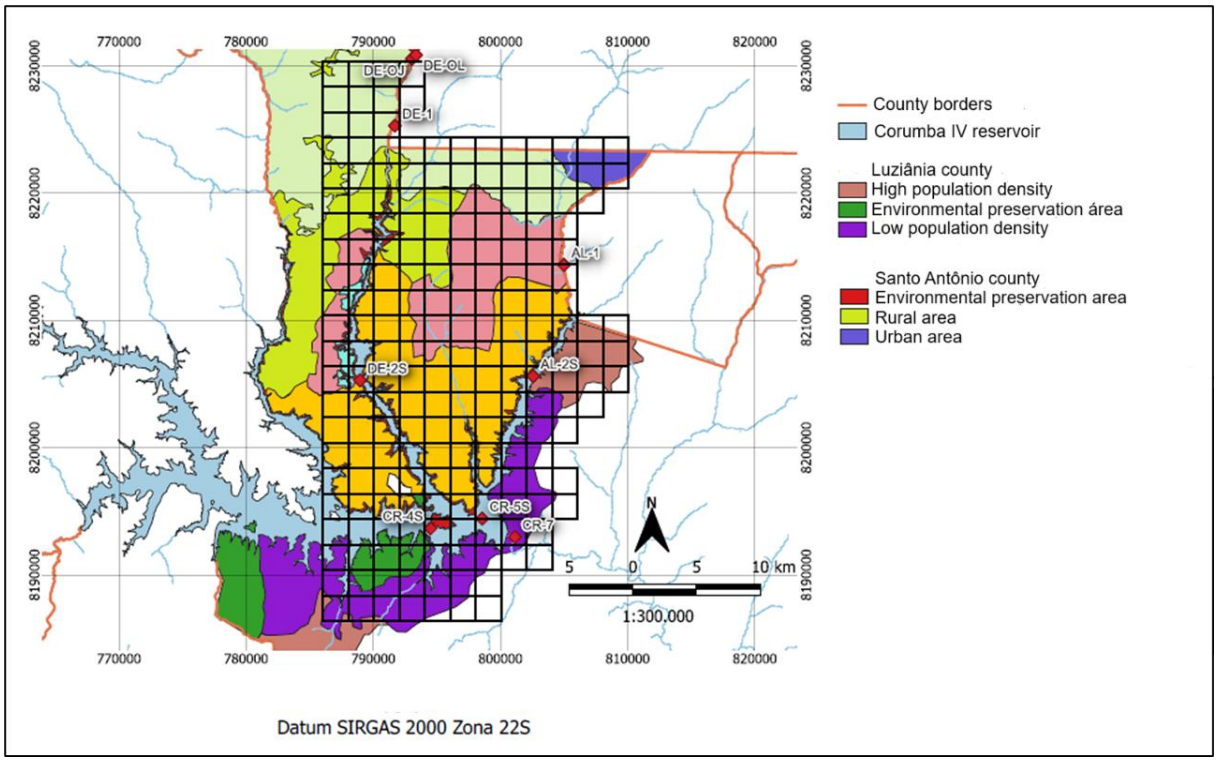


Fig. 1 – Study area

The information contained in the grids are: (a) The position of the grid, which will be located by latitude and longitude in UTM in the region 23K relative to its central point; (b) Distance in kilometers from the center of the grid to the bank of the river, provided that this location on the

bank is at a lower elevation than the grid; (c) Identification of the eight monitoring points closest to downstream of the location on the bank defined in the second; (d) Type of urban or rural zoning of which the grid is located; (e) Number of households in the location; (f) Type of household, being “U” for urban, “R” for rural and “L” for leisure home, which may be located in a condominium or isolated; (g) Limit number of households or asymptote, determined by the restrictions imposed by the predominant zoning; (h) Average number of inhabitants per household; (i) Expected growth for the households in the grid, being: “0” stagnant; “1” very slow; “2” slow; “3” moderate; “4” moderate strong; “5” strong; (j) Number of significant point loads in the grid (industries, hospitals, inns); (k) Estimated value of domestic sewage produced by point loads; (l) Estimated number of heads of cattle; (m) Expected growth for heads of cattle, horses and mules in the grid, being: “-1” decreasing; “0” stagnant; “1” very slow; “2” slow; “3” moderate; “4” moderate strong; “5” strong; (n) Agricultural area planted in hectares; (u) Expected growth of the planted agricultural area, being: “-1” decreasing; “0” stagnant; “1” very slow; “2” slow; “3” moderate; “4” moderate strong; “5” strong.

3. RESULTS AND DISCUSSION

3.1. Non-point pollution points

Groundwater sources are often contaminated by septic tanks when the safe distance between a water source and a septic tank is not properly maintained. Microbial contamination and waterborne diseases are caused by inadequate sanitation systems in many developing countries. The widespread use of septic tanks in rural and suburban areas makes them a major source of groundwater contamination. Septic tank effluents contain pathogenic bacteria, viruses, protozoa and helminths. Septic tank pathogens can leach through the soil (both unsaturated and saturated) and eventually reach groundwater.

The main contaminants in a septic tank are the microorganisms present in the well. The distance between a water body and a septic tank and the local geological and hydrogeological conditions are key factors in the spread of bacteria that contaminate the water body. When organisms are leached into soil, among other factors, the hydraulic conductivity of the soil (i.e. the volume of water that moves in a unit time under a unit hydraulic gradient across a unit area) determines how organisms move into the saturated groundwater zone. Soil hydraulic conductivity again depends on particle size, but also on textural factors such as the horizontal stratification formed by the annual deposition of silt. The term soil in this case refers to the upper 3 m of the land surface and the term sediment refers to the deposited material. The other key factors influencing the transport of bacteria in aquifer systems are the processes of horizontal physical mass transport, hydrodynamic dispersion and microbial decay. The transport of microbial pathogens in groundwater is limited by extinction and attenuation

(including filtration and adsorption). The processes of death and attenuation of bacteria occur in all groundwater aquifers. In a sand and gravel aquifer, coliforms were isolated 30 m from the source within 35 hours of initial contamination. Some pathogenic bacteria have been shown to persist in the soil for up to 42 days.

Faecal bacteria are, therefore, often found at greater distances and depths than expected, and in-depth field investigation is required to determine the conditions and environment for bacterial propagation in the subsoil. In an experiment conducted in sites where the soil composition was clay and sand, inspection wells with depths between 18 and 21 meters and located between four and eight meters from the pit showed contamination levels in all seasons of the year. Wells with the same depth and located outside this perimeter were assessed as free of any contamination [13].

Since the previous version of the forest code was established 100 meters from the maximum level of the reservoir to the permanent preservation area around the reservoir, the septic tanks of isolated homes and condominiums established there are required by law to be far from the perimeter deemed safe against the risk of contamination. The urban area of Santo Antônio Descoberto is approximately 40 km from the entrance to the Corumbá IV reservoir via the Descoberto River. Therefore, given the characteristics of urban households and the existence of a sewage treatment plant near the municipality, it is not the urban domestic sewage loads that impact the reservoir.

Therefore, contamination by domestic sewage may only be due to untreated discharge into watercourses in the rural area, where 2,093 households were in the year 2023. Of these, only those located less than eight meters from the edge of the watercourse or that have piping with direct untreated discharge are potential polluters of the reservoir [14].

A study carried out in Mozambique revealed that the soil characteristic identified in the study area was like the soil type predominant in the area of influence of the hydroelectric plant. [15]. In this country, the safe distance between a latrine and an excavated well is proposed as being 50 meters [16]. We decided to adopt this reference in our study, with the aim of estimating the decay curve of contamination of the Descoberto and Alagado Rivers by raw domestic sewage discharged into seepage pits.

Eutrophication, an excessive concentration of nutrients that results in high growth of aquatic biota and restricts the use of water in various activities, is mainly limited by phosphorus in freshwater. Nitrogen is also associated with the acceleration of eutrophication, but due to the ability of some aquatic plants to fix atmospheric nitrogen, phosphorus has received greater attention in the study.

Biochemical oxygen demand (BOD) is the amount of oxygen required to oxidize organic matter and is used to estimate the influence of organic compounds on the respiration of bacteria and algae, based on increased decomposition. The increase in BOD occurs through the respiration and decomposition of algal biomass, nitrification and eutrophication of water bodies caused by excess organic matter. The latter leads to blooms of phytoplankton and/or aquatic macrophytes, which increase the concentration of organic matter and BOD.

3.2. BOD, N and P contamination parameters

Figure 2 shows the mapping of the grid system superimposed on the contour lines of the terrain near the dam and the banks of the reservoir. From this figure, it is possible to observe the arrows in red showing the areas where there is a significant slope towards the water bodies and, therefore, are susceptible to leaching fertilizers from agricultural crops near the reservoir, reaching the water bodies and increasing the concentration of dissolved parameters in the water.

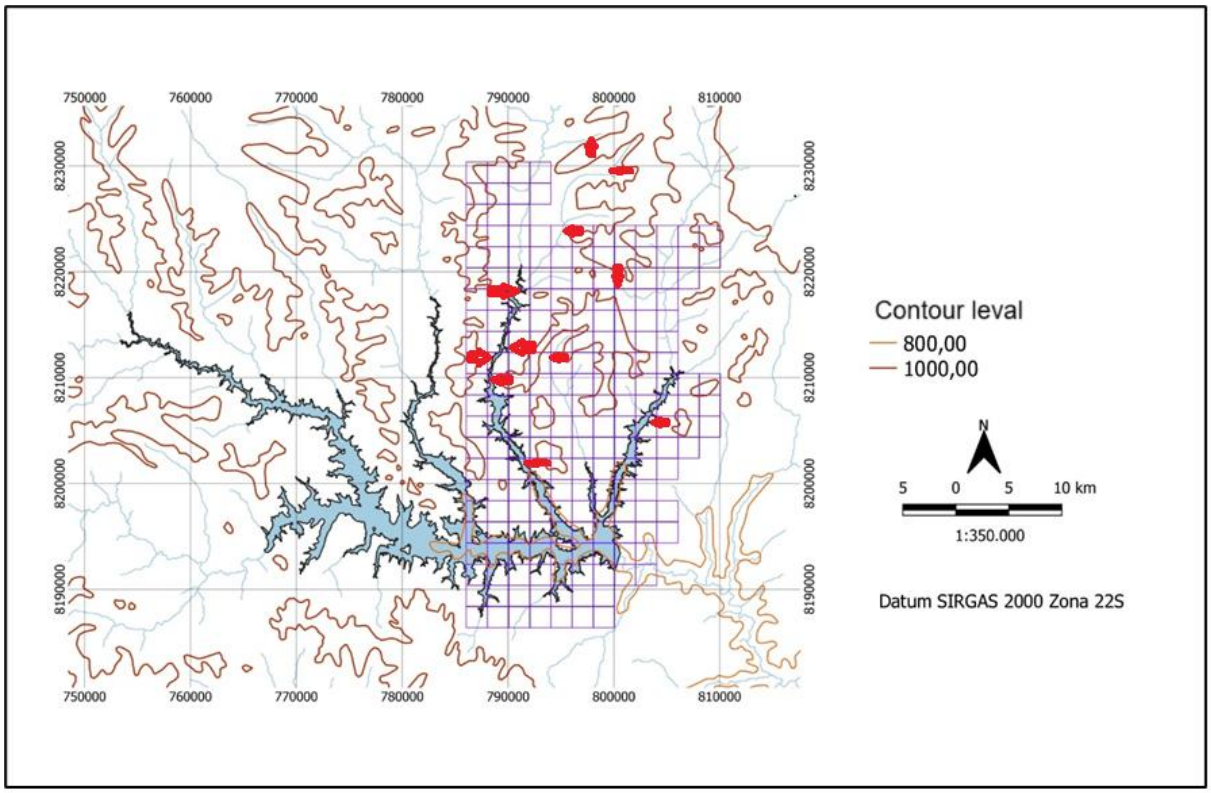


Fig. 2 – Agricultural nutrient leaching points

The parameters measured in the raw wastewater were calculated in terms of specific load, i.e., the amount discharged per inhabitant equivalent and per day. This calculation requires you to know the daily wastewater volume. The amount of domestic sewage discharged daily into the rivers of the basin can be estimated assuming a per capita BOD load of 54 g/inhabitant.day

[19]. The daily wastewater volume per capita expressed in cubic meter was easily estimated by dividing this value of 54 g by the concentration of BOD in the raw water (mg/L).

Fertilizer use in areas where soybeans are planted was estimated at 303 kg/hectare, with 20% N and 5% P. Contamination resulting from leaching was estimated at 4% of the amount of fertilizer used in the squares adjacent to the riverbanks.

The estimated values for the water quality of the reservoir and the Descoberto and Alagado Rivers can be seen in Table 1. The discharge of sewage effluents as a result of the densification of rural households near the rivers and the reservoir, as well as the contamination by N and P resulting from fertilizer leaching, for the 15-year study horizon, are shown in Table 2.

Table 1 – Estimated water quality parameters at the collection points

Collection point	DBO (mg/L)	N (mg/L)	P (mg/L)
RES-1	3.12	0.99	0.09
DES-1	11.8	25.13	2.10
ALA-1	3.90	0.90	0.32
RES-2	1.74	1.28	0.01

Table 2 – Parameters resulting from estimated contamination by effluents

Collection point	DBO (mg/L)	N (mg/L)	P (mg/L)
Reservoir	1.98	3.96	0.01
Descoberto	2.88	1.22	0.11
Alagado	2.46	0.73	0.21

According to data from UHE Corumbá, in the period from 2006 to 2018 the average inflow into the UHE was 130.67 m³/s, which corresponds to a specific flow of 18.63 L/s/km² [18]. Preliminarily, the velocity and depth coefficients were estimated to fill in the physical data of the river section. To carry out this step, it was necessary to collect data from the historical series to obtain the trend line equation and consequently its coefficients. The regression lines are shown in Equations (1) and (2):

$$\text{Average water speed} = \text{Average inflow}^{0.696},\ R^2=0.986 \quad (1)$$

$$\text{Average depth} = \text{Average inflow}^{0.268},\ R^2=0.932 \quad (2)$$

3.3. Self-purification model coefficients

The K_1 coefficient is the decomposition rate of biodegradable organic matter, which depends mainly on the temperature and composition of the effluent. Temperature directly influences the solubility of gases in water and acts directly as a controller of the metabolic activity of aquatic organisms. The increase in temperature promotes the reduction of oxygen solubility in the aqueous medium and the enhancement of oxidative processes (e.g. decomposition of organic matter and respiration of organisms). Equation (3) expresses this empirical relationship between temperature and the deoxygenation coefficient:

$$K_{1T} = K_{1:20}.\theta^{(T-20)} \quad (3)$$

Where: $K_{1T} = K_1$ at any temperature (*T*); $K_{1:20} = K_1$ at temperature *T*=20ºC; *T* is the temperature in the water body (ºC) and *θ* is the temperature coefficient.

Equation 3 is also applicable for K_d e K_s. The *θ* coefficient values 1.047 e 0.1024, respectively [20]. The deoxygenation coefficient K_1 was estimated based on a range of values [2].

The K_d coefficient is the decomposition of organic matter by the biomass suspended in the liquid mass and by the biomass in the bottom sludge. This BOD decomposition coefficient includes the sedimentation of organic matter and the removal of BOD by the bottom sludge.

$$K_d = 180.Q^{0.49} \quad (4)$$

For depth (*H*) between 0.3 and 10 meters and flow rate Q between 0.15 and 250 m³/s.

The K_s coefficient represents the quotient between the sedimentation speed of the BOD and the depth of the river [2].

$$K_s = vs/H \quad (5)$$

Where: *vs* is the sedimentation rate of particulate matter; *H* is the river depth.

The values of K_1, K_d and K_s adopted by this research in the Qual-UFMG model were 0.4, 0.425 and 0.125, respectively.

K_2 is the reaeration coefficient and depends on the diffusion of oxygen in the water and the intensity of the flow turbulence, its value increasing the more intense the turbulence. The capacity of a system to reaerate is a function of the advection and diffusion processes that occur within it and the internal sources of oxygen generation and removal. K_2 is related to the flow of the watercourse, which in turn depends on the depth and speed [21].

$$K_2 = 5 * V^{0.97} * H^{-1.67} \quad (6)$$

The study using Qual-UFMG model on the self-purification capacity of the Corumbá IV reservoir observed that, although domestic effluents and bleach from agricultural fertilizers are discharged in the rivers, as they flow towards the plant's dam, no further stretches were observed where BOD values exceeded the limits established by law. It happened because the flow in the Descoberto River, which causes greater dilution of the effluents discharged into it.

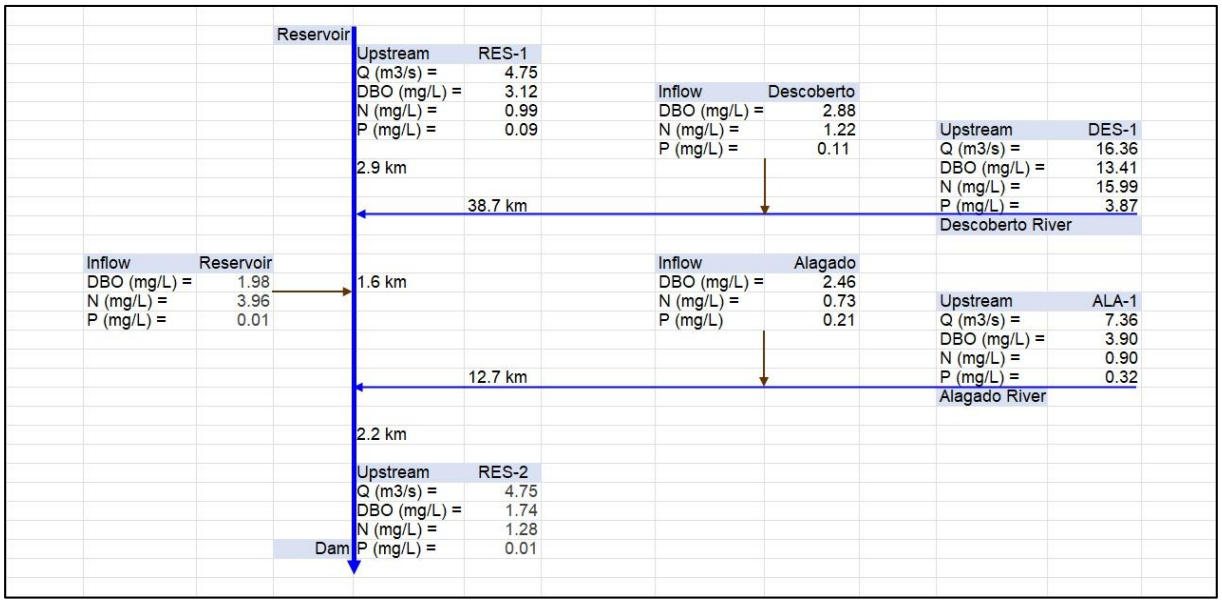


Fig. 3 – Parameters estimated by the Qual-UFMG model

Figure 3 shows that, although the volumes of BOD, N and P upstream of the dam reached the aggregate values of 27.75 mg/L, 23.79 mg/L and 4.61 mg/L, respectively, the processes of sedimentation, dissolution and dispersion of nutrients along the course of the Descoberto and Alagado rivers and the rest of the reservoir course were sufficient to purify the water, whose values that reached the dam were 1.74 mg/L, 1.28 mg/L and 0.01 mg/L, for BOD, N and P, respectively.

4. CONCLUSIONS

Considering the estimated parameters for BOD, N and P, the self-purification study of the watercourses showed values well below the limits established by Brazilian environmental regulations immediately before the hydroelectric plant dam, despite the high values for the stretches immediately after the sewage treatment plant on the Descoberto River and at the collection point on the Alagado River. Dilution is an important factor to be considered in the allocation of resources and the eventual need to establish the treatment modalities to be implemented in the reservoir. Thus, the plant management, being aware of the

estimated water quality, will be able to adopt appropriate measures for the proper protection of the plant's mechanical equipment and civil structures.

ACKNOWLEDGMENTS

This research is part of the project ANEEL PD-02262-0020/2022, which is financed by Corumbá Concessões S/A and ANEEL; and executed by Fundeb and Farol Consulting.

REFERENCES

[1] Dodds, W. K., 2006. Eutrophication and trophic state in rivers and streams. Limnology and Oceanography. 51.1, 671-680.

[2] Von Sperling, M., 2014. Introdução à qualidade das águas e ao tratamento de esgotos. 4a.ed. Belo Horizonte: Universidade Federal de Minas Gerais.

[3] Munafò, M., Cecchi, G., Baiocco, F., Mancini, L., 2005. River pollution from non-point sources: a new simplified method of assessment. Journal of Environmental Management 77(2), 93-98. https://doi.org/10.1016/j.jenvman.2005.02.016

[4] Gossweiler B, Wesström I, Messing I, Villazón M, Joel A., 2021. Impact of Land Use Change on Non-Point Source Pollution in a Semi-Arid Catchment under Rapid Urbanization in Bolivia. Water. 13(4), 410. https://doi.org/10.3390/w13040410

[6] Hualin Li, H., Zhang, J., Zhang, S., 2022. A framework to assess spatio-temporal variations of potential non-point source pollution risk for future land-use planning. Ecological Indicators 137, 108751. https://doi.org/10.1016/j.ecolind.2022.108751

[5] Contreras, E., Aguilar, C., Polo, M.J., 2023. Accounting for the annual variability when assessing non-point source pollution potential in Mediterranean regulated watersheds. Science of the Total Environment 902, 167261. https://doi.org/10.1016/j.scitotenv.2023.167261

[7] Arantes, L.T., Arantes, B.H.T., Sacramento, B.H. et al., 2023. Application of spatial environmental indicators in the assessment of degradation potential of water resources in water basins. Environmental Monitoring Assessment 195, 931. https://doi.org/10.1007/s10661-023-11499-w

[8] Zhang, H., Huang, G.H., 2011.Assessment of non-point source pollution using a spatial multicriteria analysis approach. Ecological Modeling 222(2), 313-321. https://doi.org/10.1016/j.ecolmodel.2009.12.011

[9] Hongyuan, W., Limei, Z., Hongbin, L., et al., 2018. Effects of anthropogenic activities on long-term changes of nitrogen budget in a plain river network region: A case study in the

Taihu Basin. Science of the Total Environment 645, 1212-1220. https://doi.org/10.1016/j.scitotenv.2018.06.354

[10] Zhong, S., Chen, F., Xie, D., et al., 2020. A three-dimensional and multi-source integrated technology system for controlling rural non-point source pollution in the Three Gorges Reservoir Area, China. Journal of Cleaner Production 272, 122579. https://doi.org/10.1016/j.jclepro.2020.122579

[11] Qian, Y., Sun, L., Chen, D., et al. 2021. The response of the migration of non-point source pollution to land use change in a typical small watershed in a semi-urbanized area. Science of the Total Environment 785, 147387. https://doi.org/10.1016/j.scitotenv.2021.147387

[12] Ramachandra, T.V., Sincy, V., Asulabha, K.S. et al. 2018. Optimal Treatment of Domestic Wastewater through Constructed Wetlands. Journal of Sustainability 9, 81-102. https://doi.org/11.258359/KRE-180

[13] Islam, J. S. 2016. Safe distances between groundwater-based water wells and pit latrines at different hydrogeological conditions in the Ganges Atrai floodplains of Bangladesh. Journal of Health, Population and Nutrition, 1-10. https://doi.org/10.1186/s41043-016-0063-z

[14] IBGE. 2024. Cidades: Santo Antônio do Descoberto. Available at: https://cidades.ibge.gov.br/brasil/go/santo-antonio-do-descoberto/panorama

[15] Maria, M.R., Yost, R. 2006. A Survey of Soil Fertility Status of Four Agroecological Zones of Mozambique. Soil Science 171, 902-914. https://doi.org/10.1097/01.ss.0000228058.38581.de

[16] Parker, A., Carlier, I. 2009. National regulations on the safe distance between latrines and waterpoints. Dew Point Enquiry No. A0304. Available at: https://assets.publishing.service.gov.uk/media/57a08b3c40f0b652dd000ba6/DEWPoint_A0304_Nov2009_National_regulations_safe_distance_latrines.pdf

[17] Pham, T., Yli-Halla, M., Marttila, H. Et al. 2023. Leaching of nitrogen, phosphorus and other solutes from a controlled drainage cultivated peatland in Ruukki, Finland. Science of The Total Environment 904, 166769. https://doi.org/10.1016/j.scitotenv.2023.166769

[18] Corumba IV. 1999. Estudo de Impacto Ambiental (EIA): Aproveitamento Múltiplo Corumbá IV. Relatório Interno.

[19] Economopoulos, A.P. 1993. Rapid inventory techniques in Environmental Pollution. In: Assessment of sources of air, water and land pollution. WHO Offset Publication, World Health Organization, Geneva.

[20] EPA, 1987. The enhanced stream water quality models QUAL2E and QUAL2E-UNCAS. Documentation and user model. Athens: EPA, 189 p.

[21] Churchill, M.A., Elmore, H.L., Buckingham, R.A., 1964. Advances in Water Pollution Research. Proceedings of the International Conference Held in London, September 1962, 89-136. https://doi.org/10.1016/B978-1-4832-8391-3.50015-4

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DECHARACTERIZATION PROGRAMS FOR DAMS IN BRAZIL: SOLUTIONS AND CHALLENGES

Felipe S. Santos IPT
Paula S. Nishijima[1] IPT
Gisleine C. Campos IPT
Wilson S. Iyomasa IPT

ABSTRACT

The bulletin “Dam Safety in the State of Sao Paulo – 2024” reports that, in the state of Sao Paulo, 424 dams are registered under the National Dam Safety Policy (PNSB), as per the criteria established by Federal Law No. 12,334/2010. Of these, 125 dams are used for energy generation, with 115 classified as having high Associated Potential Damage. These dams require efforts to coordinate and implement monitoring activities, communicate dam safety measures and, in certain cases, evaluate their decharacterization. Dam decharacterization involves a series of actions aimed at safely ceasing the operational activities of a dam while minimizing environmental, social, and economic impacts at the regional level. This process may encompass three primary scenarios: maintenance, partial removal and complete removal of a dam. This article explores these scenarios within the context of current Brazilian legislation and regulatory frameworks, aiming to contribute to the mitigation of accident and disaster risks associated with dams and the restoration of impacted ecosystems. In response to recent catastrophic dam failures in Brazil, mining companies have increasingly invested in dam decharacterization programs, particularly targeting upstream-raised structures. The article places particular emphasis on the geological-geotechnical aspects of various engineering solutions applicable to dam decharacterization, especially for small-scale dams constructed for water storage. Despite the growing focus on this issue, successful cases of dam decharacterization in Brazil remain limited, with no examples related to hydropower facilities in the state of Sao Paulo. To address this gap, the article presents real-world examples from other parts of Brazil, illustrating various engineering approaches to the decharacterization of hydropower dams. It highlights the benefits of such initiatives for local communities and ecosystems, potential adverse impacts, regulatory advancements and institutional challenges. It is anticipated that, within the next few decades, the state of Sao Paulo will need to align with this emerging reality.

Keywords: Hydropower dams, Brazil, Decharacterization, Engineering solutions, Geological-geotechnical aspects.

[1] ptanabe@ipt.br, Brazil, Technological Research Institute (IPT)

1. INTRODUCTION

Dams are civil engineering structures designed to serve diverse purposes, including water storage for supply, hydroelectric power generation, flood control, agricultural irrigation, navigation, storage of mining tailings or industrial waste, and recreational activities, among others. These functions are intended to address societal needs at a particular time or under specific conditions, which may evolve over time.

Thus, whether at the end of the structure's useful life or due to circumstances that alter the interests or needs of the population, dams may lose their significance years after their construction. In these cases, specific actions are required to cease their operation, rather than merely abandoning their facilities, which would result in environmental impacts [1]. Over time, the structural integrity of dams gradually deteriorates, requiring owners to fulfill their obligation to carry out repairs, reinforcements, or, in some cases, complete removal [2].

In Brazil, the guidelines, resolutions, standards, and legislation regarding dams have undergone revisions in terms of their design, construction, monitoring, operation, maintenance, and, more recently, in their decharacterization processes [3, 4].

Dam decharacterization encompasses a series of actions intended to safely cease the dam's operation while minimizing environmental, societal, and economic impacts in the region. A decharacterized dam is a legally designated structure that no longer serves its original purpose and is repurposed for an alternative use, as outlined in a specific project. The term has gained significant traction in Brazil since the implementation of Law 14.066 in 2020, thereby attaining national recognition. Nevertheless, a uniform definition or consensus regarding its meaning remains absent, necessitating further discourse and exploration.

The dam decharacterization design must be evaluated and approved by the regulatory authority, typically encompassing three distinct scenarios: maintenance, partial removal and complete removal of the dam. These scenarios are examined in this paper within the framework of current legislation and regulatory standards in Brazil, aiming to contribute to the reduction of risks related to dam accidents and disasters, as well as restoration of river ecosystems. Emphasis is placed on the geological and geotechnical aspects that dictate the various engineering solutions applicable to dams, particularly small-scale dams used for multi-purpose water storage.

It is worth noting that, in addition to the three decharacterization methods discussed (maintenance, partial removal and complete removal), there is the option of replacing the dam with a new one [5]. However, this topic is neither addressed nor detailed in this work.

2. BRAZILIAN CONTEXT

In Brazil, the majority of dams are used for water storage, with reservoirs dedicated to electricity generation accounting for 89% of the total volume assigned to dams for water accumulation [6].

According to the National Dam Safety Information System (Sistema Nacional de Informações sobre Segurança de Barragens – SNISB), Brazil had 27,386 registered dams as of November 2024, with 1,145 located in the state of Sao Paulo. Of those in Sao Paulo, 91 are classified with high-risk categories (RC) and 258 exhibit high Associated Potential Damage (APD). Of the dams with high APD, 92 are used for electricity generation (Fig. 1).

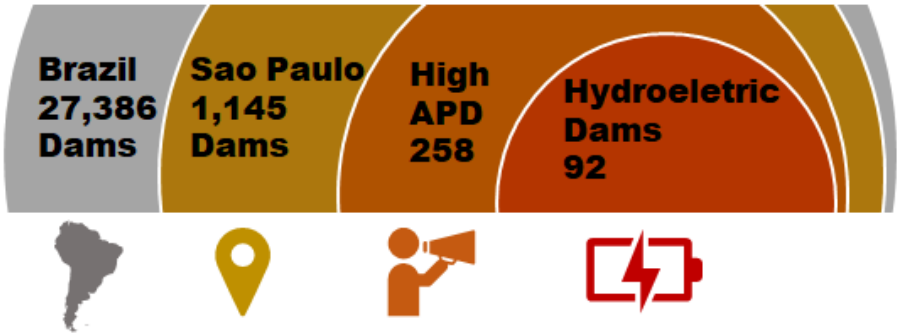


Fig. 1 – The dam scenario in Brazil and the state of Sao Paulo, according to the registry in the National Dam Safety Information System (SNISB), accessed in November 2024.

Regarding the decharacterization of dams in the state of Sao Paulo, it is known that in the SNISB [7], of the 1,145 registered dams, 28 are deactivated, with 4 of these dams having been used for water storage for electricity generation and having a height of less than 10 m. However, there are no published cases of decharacterized dams in the state of Sao Paulo that were used for hydropower generation. Thus, it is evident that the process of dam decharacterization related to the hydropower sector remains in its early stages.

In Brazil, ANEEL (National Agency of Electrical Energy) is the entity responsible for overseeing dam safety when the predominant use of hydraulic potential is for hydropower generation. Dams supervised by ANEEL are classified into Classes A, B and C, based on the RC (high, medium and low) and the APD (high, medium and low), in accordance with the provisions of Regulatory Resolution REN 1,064/2023 [8].

Based on the self-filled Dam Safety Forms (DSF) submitted by the owners [9], which differ from the information in the SNISB, it is inferred that there are 1,253 hydropower dams in the state of Sao Paulo, including HPPs[1] (Hydroelectric Power Plants), SHPs[2] (Small Hydroelectric

[1] HPP – Installed capacity exceeding 30 MW, or capacity between 5 MW and 30 MW with a reservoir area greater than 13 km².
[2] SHP – Installed capacity between 5 MW and 30 MW with a reservoir smaller than 13 km².

Plants) and HGPs[3] (Hydropower Generation Plants). Of these, 930 are classified with high APD, and 4 are classified as Class A (Fig. 2). This highlights the need to understand the potential benefits and challenges in the decharacterization process of dams, which may become a near-future reality for many existing hydropower structures. It is notable that the majority of Class C dams in Sao Paulo (245) are classified as small-scale (< 15 m), while those classified as Class A range from 15 m to 60 m in height, with two between 15 m and 30 m, and two between 30 m and 60 m.

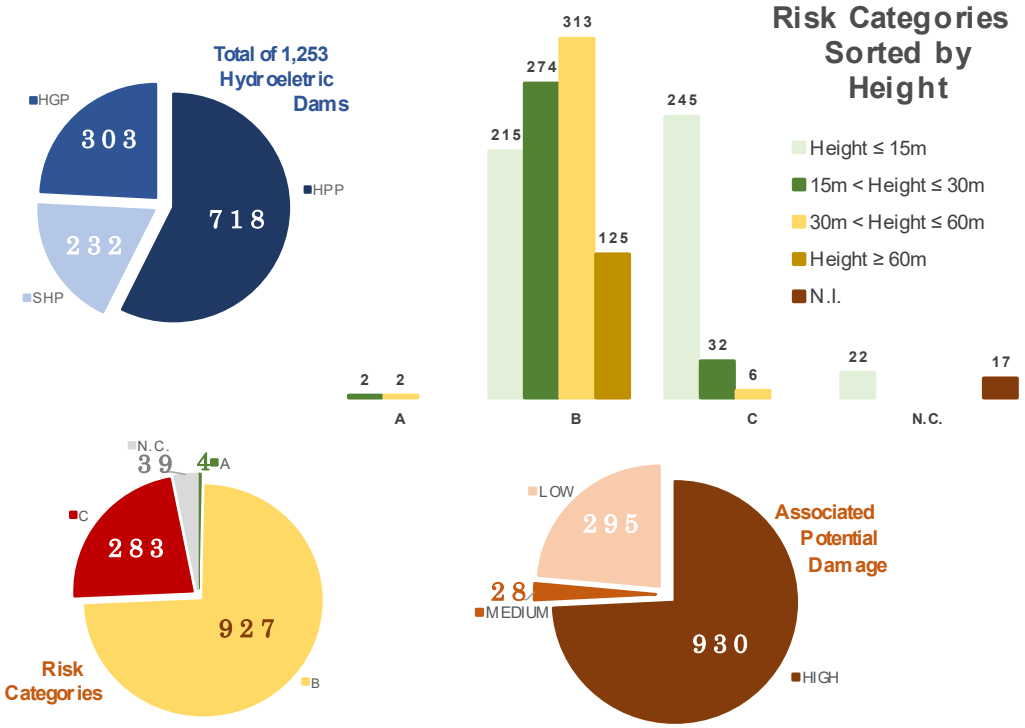


Fig. 2 – Dam scenario in the state of Sao Paulo based on the Dam Safety Forms.

2.1. Laws and regulations

Environmental concepts and regulations have evolved significantly over time. Many dams were originally licensed under less stringent environmental frameworks [5]. Today, regulatory agencies encounter challenges in relicensing these projects, as contemporary requirements were not included in the original licenses, leading to increased costs for the owners.

In Brazil, the topic of dam decharacterization was scarcely discussed or legislated until 2010, when Law No. 12,334 [10] was enacted. This law established the National Dam Safety Policy (Política Nacional de Segurança de Barragens – PNSB) and created the National Dam Safety Information System (SNISB).

In 2015, an unprecedented disaster involving a tailings dam constructed using the upstream method [11, 12] prompted the reorganization of the National Mining Agency (ANM) and the

[3] HGP – Installed capacity up to 5 MW.

proposal of a new regulatory framework. This framework was implemented between 2017 and 2019, with guidelines that consider the dam's size, the risks to downstream populations and to the environment. It introduced a more formal system of inspections, report production and registration, as well as the requirement for emergency response plans ([13]).

Additionally, the Brazilian Association of Technical Standards (ABNT), through NBR 13.028/2024 [14], established minimum requirements for the development and presentation of dam projects aimed at ensuring safety, operability, cost-effectiveness and deactivation of these structures, while minimizing environmental impacts [15]. However, it did not establish standards or guidelines for the decharacterization process of these structures [16].

In 2020, Law 12,334 was revised by Law 14,066 [17], which was enacted shortly after another catastrophic accident involving a tailings dam constructed using the upstream method [12, 13, 18, 19, 20]. To some extent, this disaster expedited the implementation of measures that had been under discussion in previous years.

In summary, Brazilian legislation stipulates that the first step in considering a dam decharacterized is the closure of the structure (decommissioning). This is followed by the second step, the establishment of hydrological and hydrogeological controls [21]. In the third step, stability must be ensured for the remaining structures. Finally, a monitoring phase is required to track the stability of the decharacterized structure [3, 22]. Fig. 3 provides a timeline of major federal legislation enacted in Brazil since 1997, along with ANEEL resolutions.

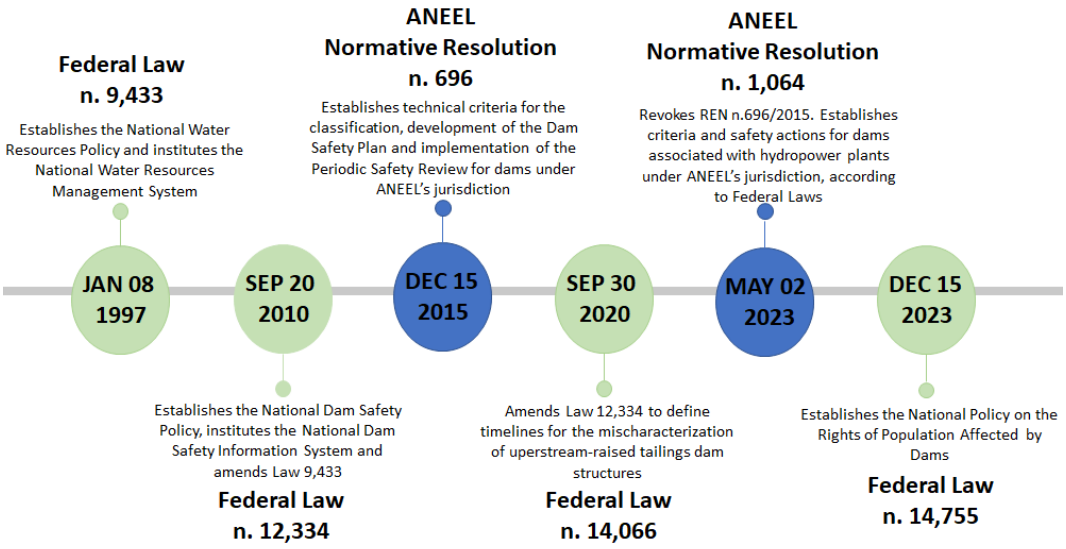


Fig. 3 – Timeline of legislation and resolutions on dams in Brazil related to hydropower projects.

In Brazil, the decharacterization of tailings dams built by the upstream method is a mandatory requirement [23]. As a result, in recent years, some dams have suffered decharacterization throughout the country [4, 21, 24, 25]. However, this study focuses on decharacterized dams

originally designed for water storage, either for hydropower generation or for public water supply. Cases fitting this classification remain relatively rare.

3. DECOMMISSIONING AND DECHARACTERIZATION

Decommissioning represents the initial stage of the decharacterization process, starting with the confirmation that the dam is no longer required within the operational context of the enterprise and can, therefore, be decharacterized. However, decommissioning alone does not render the structures immune to ruptures or failures [16]. Consequently, solutions encompassing scenarios of maintenance, partial removal, and total removal must be carefully considered to mitigate potential risks [26].

The [27] organizes the dam decharacterization strategy according to the following steps: i) Initial framing, as described below; ii) Diagnosis of the current situation regarding the presence of transverse infrastructures in watercourses; iii) Proposal of a methodology for the national inventory of transverse barriers (dams and weirs); iv) Definition of criteria for prioritizing areas for action regarding the removal of obsolete transverse infrastructures; v) Establishment of the decision-making flow related to the identification of obsolescence and the selection of the removal option; and vi) Definition of the general content of removal projects, including environmental assessment and monitoring of their effects

Three distinct situations can be considered when analyzing dams to be decommissioned and, consequently, decharacterized: obsolete infrastructures; infrastructures in operation with a valid license; and operational infrastructures without a usage title [27].

Regarding the first case, total removal is almost always the preferred option, except in situations where it is deemed impossible due to environmental, social, legal, economic, or safety reasons. In the second case, the removal option should be considered when the infrastructures present serious safety issues that are technically, environmentally, or economically unresolvable. When some dams are neglected, without any monitoring or maintenance, they become eligible for decommissioning due to the associated risk [16, 28].

In relation to operational infrastructures without a valid usage title (either because it never existed or because its validity period has expired), a multicriteria analysis should be conducted, considering the value of the uses and the infrastructure's effects on the environment. This multicriteria analysis should balance the values associated with the infrastructure and its uses with the environmental impacts it generates [29, 30, 31].

On the other hand, according to [30], in Canada, the dam removal process involves different stages, namely: i) Identifying and acting according to a pre-defined process; ii) Conducting a preliminary assessment of the structure and the river; iii) Identifying and addressing issues

related to stakeholders; iv) Assembling a team of specialists; v) Preparing detailed management plans for the process, sediment management, stream and fish restoration; vi) Removing the structure and restoring flow and its habitat; and vii) Monitoring the effects of the removal on river hydrology, ecology, and nearby infrastructure.

The two propositions above are complementary: while the [27] emphasizes the importance of conducting a general assessment of the entire dam system to better guide decision-making regarding decharacterization, the Canadian model [30] focuses on the analysis of a given dam, assuming that the global assessment has already been carried out. For the Brazilian context, it is understood that the combination of these two methodologies represents the ideal strategic model, proposing the sequence of activities illustrated in Fig. 4.

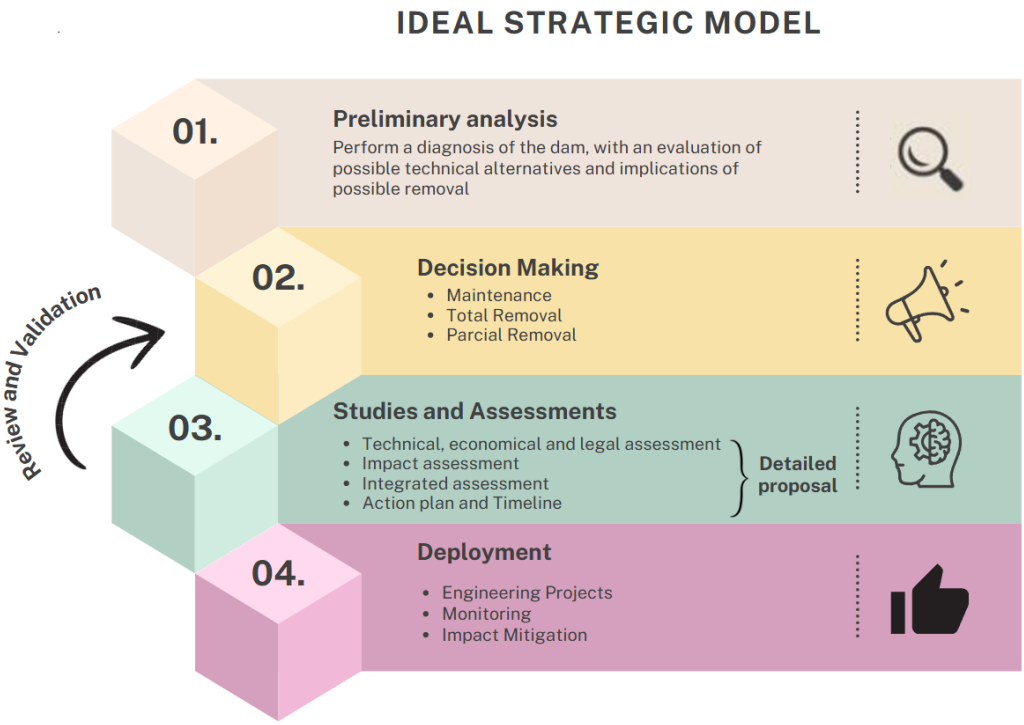


Fig. 4 – Ideal strategic model proposed.

Recognizing the complexities involved in dam removal planning, [30] proposed a decision-support matrix with multidisciplinary criteria, assigning weights to each aspect analyzed. The use of this tool can assist in identifying the benefits and negative environmental and social impacts resulting from the decision to completely remove a dam, with specific considerations for the location and data availability constraints, which are common in older structures.

3.1. Maintenance

The average lifespan of most dams is approximately 60 to 120 years, primarily due to the deterioration of structural integrity and the sedimentation of reservoirs [30, 32, 33]. In the long term, a significant number of dams will become obsolete, creating a substantial liability for society [1].

Dam maintenance refers to the scenario in which decharacterization is carried out without the physical removal of the dam, with or without modifications to its structures. An example is the adaptation of a dam originally used for water supply to recreational purposes, a situation that involves a series of structural changes, including safety improvements and additional resources [26]. The main disadvantage is the need for long-term monitoring [15].

In cases where the decision is made to maintain decommissioned dams, it is essential to have original projects in the form of manuals, detailing the main aspects of the structure and outlining the necessary maintenance activities [5]. The objective is for an entity (public or private) responsible for maintaining the structure to have a complete history of the dam, including technical specifications and incident records, thus being aware of its conservation status and, consequently, its Risk Category.

3.2. Partial removal

This scenario involves partial demolition with or without the replacement of dam structures. This may include reducing the height of the dam or the spillway to lower storage capacity. It may also involve lowering both the dam and the spillway. To achieve this, excavation work must be conducted according to the desired final height, which is determined by future demand, ecological impacts, community values, and improvements in dam safety. A range of structural changes and additional resources may also be required [26].

The choice for partial dam removal is generally applied in cases where logistical factors (such as sediment retention) or socio-political factors (such as the historical significance of the dam) require the preservation of part of the structure. For example, the portion of a dam that blocks fish passage may be removed, while other parts are maintained for historical preservation and sediment retention [33].

In this solution, for instance, one could consider creating an opening in the massif and removing the material contained in the reservoir located in the main channel bed of the ravine, while re-sloping the remaining material of the reservoir [15, 16, 34].

3.3. Complete removal

The final scenario is the complete removal of the dam structure, aiming to restore the site's original conditions prior to its construction. The full removal of the dam can be carried out in two ways [33], differentiated by the speed of structure dismantling and reservoir drawdown: instantaneous (hours or days) or sequential (months or years).

In addition to extensive excavation works and a range of additional structural features, this method requires a comprehensive sediment management strategy, particularly for sediments

that may contain chemical contaminants [34], as well as site revegetation and a decommissioning plan [26, 28].

In some parts of the world, small dams have been removed for various reasons, including structural aging, failure to meet the needs for which they were originally built, high maintenance costs, inability to renew operational licenses with regulatory authorities, unsatisfactory technical conditions and disaster risks, channel improvement, restoration of fish migratory routes, expansion of arable land, among other reasons. The primary motivations are often linked to risk reduction and the loss of their original purpose [28, 35].

According to [33], there is a higher frequency of small dam removals (less than 10 m in height) worldwide because they are generally older than large dams, tend to be structurally deficient, and their original purposes are often surpassed by current demands. In contrast, large dams are typically newer, more likely to provide significant social and economic benefits, and their removal is more arduous and faces greater socio-economic challenges.

4. MAINLY CONSTRUTIVE ASPECTS ON DECHARACTERIZATION

In addition to the issues commonly evaluated during the design phase of a dam—such as foundations resistance [25], construction materials, turbine efficiency, environmental impacts, among others—considerations related to its potential decommissioning must also be addressed. These include the size and purpose of the dam, associated costs and risks, its location, and the impacts that removal will have on the surrounding community [36]. The planning of dam decommissioning and decharacterization should begin concurrently with the initial design of the structure [37]. However, this detailed planning faces limitations, as large dam projects are designed to last for decades. Consequently, the dam's maintenance condition and the region's political, social, economic, and environmental circumstances may differ significantly from those prevailing when the project was conceived [5].

It should be considered that very old structures were designed in different eras, when the climatic scenario differed from the current one, with distinct recurrence times and different intensities and frequencies of severe events. Many old dams are not suited to current climatic conditions, which can lead to structural failures, potentially causing major disasters [38]. In Brazil, throughout 2023, 25 dam accidents[4] were recorded, with 92% of them being associated with intense rainfall events [6].

[4] Accident – According to Law 14,066, refers to the compromise of the structural integrity with uncontrolled release of the reservoir's contents, caused by the partial or total collapse of the dam or an associated structure.

The following highlights geological-geotechnical aspects, as well as engineering solutions, that should be part of the studies for decommissioning and eventual decharacterization of dams.

4.1. Geological-geotechnical aspects

To understand the geological-geotechnical characteristics of the site where a dam is constructed, direct and indirect investigations must be carried out [39, 40]. The complexity of a decharacterization project is as high as that of the original dam construction design. Therefore, it is recommended to conduct CPTu, SPT, vane tests, geophysical surveys, as well as samples of materials from the body of the dam and abutments, and from the reservoir materials, when applicable [41, 42].

For the stability studies of the dam mass and natural slopes to be reliable, correct parametrization of soils and rocks is required. Topographic and bathymetric surveys are also necessary to characterize the dam and the reservoir as accurately as possible. Complete characterization of the filling materials in the reservoir, when present, is essential, as their disposal and removal are key criteria for determining the decharacterization method [34].

The rate of reservoir drawdown in a water storage dam can cause ruptures in the upstream slope under undrained conditions; therefore, detailed analyses must be conducted to determine this rate [43]. The best studies are those that consider the coupling between flow and stress-deformation (hydromechanical model). Probable causes of failures in inactive (decommissioned) dams are associated with the rise in the water level of the reservoir ([16]; [41]), approaching the crest of the dam and thus increasing the phreatic level in the mass and downstream slope, leading to landslides and slippage. Studies on soil erodibility should also be conducted [41], as larger areas of soil will be exposed.

Seismic characteristics of the area are important, as they impact the failure mechanisms of a dam and determine the design specifications and safety factors required, as well as the instruments to be installed [22, 42, 44]. Hydrogeological and hydraulic studies are essential, as several aspects of the potential negative consequences are linked to the behavior of the reservoir. Given the ongoing climate crisis, the impermeability characteristics of watersheds, due to urban densification, have also changed. As a result, flow peaks, recurrence times, and response times are now different compared to the past.

Measurements of neutral pressure, displacements, deformations, flow rates, among others [42], constitute extremely important input data for the operation and definition of the procedures to be followed. It is important to remember that monitoring should also consider the interaction between the dam and its surrounding environment, as specified by the PNSB. An effective monitoring plan, with the proper placement and selection of instruments, allows the

identification of anomalies that, if left unaddressed, could escalate to alarming levels and result in failure of the dam mass [22, 44].

4.2. Engineering solutions

When it comes to dam removal, certain solutions come to mind, particularly those involving explosives, which require a carefully planned sequence and step-by-step monitoring. These solutions must adhere to well-established criteria to prevent damage to the structure and the remaining mass.

The dismantling can also be carried out using mechanical methods with heavy machinery, including barge-mounted equipments, such as hydraulic hammers, excavators, and tractors. All of these machines also affect the stability of the dam, so the current condition of the infrastructure must be thoroughly assessed before any intervention. The installation of cofferdams typically precedes any intervention; however, other solutions, such as the construction of channels and tunnels, can be employed for the temporary diversion of the river.

The solutions for maintaining the dam, as well as for partial removal, require studies and activities as complex as those involved in the execution of the dam itself [41]. Reinforcement of the embankment, slope stabilization, reinforcement with riprap, sediment removal from the reservoir area [32, 33], sophisticated drainage devices, reuse of materials from the dam itself, breaching of the dam, and modification of crest elevations and spillways [16] are technical aspects that must be thoroughly studied before determining the decharacterization solution for the dam.

The solution of constructing downstream containment dams is also frequently used when it is concluded that the risks and costs of adopting other measures are too high. In this option, the construction of new dams must adhere to all technical requirements established by regulatory bodies and current legislation. The adoption of green infrastructure has been gaining prominence [4, 15, 45], as it seeks to integrate the remaining infrastructure into the environment with minimal impact, aiming at the restoration of the previously existing ecosystem.

Mechanical sediment removal from the reservoir represents the option with the least impact on the downstream ecosystem; however, it is also the most expensive approach. Conversely, the spontaneous erosion of reservoir sediments by a restored river system generates negative environmental impacts downstream of the removed structure but represents the least expensive option in the short term [35]. All approaches must be carefully analyzed, with their outcomes closely monitored both during and after the decharacterization process.

5. MAINLY CONSTRUTIVE ASPECTS ON DECHARACTERIZATION

There are no concrete cases of dam decharacterization for hydropower purposes published for the state of Sao Paulo. Therefore, in this chapter, real case examples are presented in which dam decharacterization was proposed in other locations in Brazil, where the dams were used for water storage for electricity generation and public water supply.

5.1. Case 1 - Maintenance

As an example of a case where the dam structure will potentially be maintained after decharacterization, given that its structure was considered intact, the Pandeiros PCH (Small Hydroelectric Power Plant) dam (Fig. 5), located in Minas Gerais state, Brazil, built in 1957, is cited [32]. The dam had a capacity of generating 4.2 MW but became inoperative in 2008, resulting in considerable sedimentation of the reservoir.

Fig. 5 – Overview of the SHP Pandeiros [46].

According to the authors, various alternatives are being considered to enable its decommissioning; however, the presence of over 200 active gullies upstream of the dam was identified, contributing to the sedimentation of the reservoir and the Pantanal Mineiro, which is essential for regional biodiversity.

Following a global trend toward the reuse of dam structures, the study proposed periodic dredging of the sediments retained in the reservoir, converting it into a trap to reduce sedimentation in the Pantanal caused by upstream sediments, as well as creating storage volume for water needed for agriculture and human consumption. Supporting this decision, the authors conducted studies to characterize the water and sediments, as well as measurements of the river's sedimentation process during the period from 2016 to 2019, correlated with the region's rainfall indices. The studies indicated that a large portion of the accumulated sediments had already stabilized and was covered with vegetation.

In this case, the recommendation was to maintain the dam with periodic dredging of the reservoir, rather than removing it or opening its spillways for the natural removal of sediments by the Pandeiros River. In addition to these recommendations, the need for monitoring the dam and conducting technical inspections through site visits, as required by the legislation and resolutions in force in Brazil, was emphasized.

According to Chagas et al. [32], further studies should be conducted to assess the impacts of sediment suspension after dredging on water quality, as well as the selection of the disposal site for the waste, which could potentially be reused, for example, in construction.

5.2. Case 2 – Complete removal

An example of a dam decharacterization project through the proposal for complete removal of the dam is presented by [41], referring to the Ponte de Terra 2 earth dam, located in Gama (DF), Brazil. Initially used for water regulation and public supply, the dam was constructed in 1967 and operated normally until the year 2000. Between 2000 and 2013, it operated sporadically, and has been out of operation since then.

The dam consists of an earth fill mass, with a trapezoidal cross-section and an irregular crest (varying top elevations). Its length is approximately 57 m, and the maximum height of the embankment is 3.5 m. It features a concrete spillway of the thick sill type, located on its right side, and forms a reservoir with an area of 6,400 m² and a volume of about 11,764 m³, with an average depth of 1.84 m. When operational, the reservoir required periodic cleaning for sediment removal, as shown in Fig. 6, taken in 1988 during works to repair a partial rupture in the upstream slope.

Due to the lack of maintenance, the dam's spillway was ruptured and displaced. With the uncontrolled discharge, a concentration of flow began to occur in the stream section, accelerating the erosive process downstream. In response to this situation, [41] proposed a series of activities focused on the total removal of the dam, which has not yet been carried out.

Fig. 6 – Accumulated sediments and rupture at the Ponte de Terra 2 dam [41].

The proposed activities include: a) Feasibility study: evaluate the available technical alternatives, including the option of maintaining the dam as it is; b) Project design and cost estimation: consider the possibility of demolition and removal of the concrete spillway; c) Communication and approval: engage with environmental agencies and urban planning authorities, highlighting the need for funding for the works; d) access adjustment: ensure access to the dam for the passage of machinery, addressing safety and logistical requirements for decharacterization activities; e) Diversion of inflow and reservoir drawdown: necessary for any interventions to be carried out; f) Sediment management: address the significant sediment accumulation identified at the upstream slope and the impact of potential sediment transport downstream; g) Demolition and physical removal of the dam: assess the volume of waste generated and determine its final disposal; and h) Environmental and Landscape Restoration: consider the impacts on vegetation and local fish communities.

The author emphasized that "*…oftentimes, the process of decommissioning a dam is costlier than continuously performing repairs and improvements on the dam…*". Nevertheless, maintaining an inoperative unit requires investments from the owner to ensure its safety.

6. DISCUSSION

The potential benefits and drawbacks concerning the deactivation and decharacterization of dams are discussed below. However, it is important to emphasize that the conclusions are still in the early stages due to the limited number of available studies.

6.1. Their benefits

Among the positive impacts of dam decommissioning and decharacterization are the restoration of ecological, fishing, geomorphological, and even landscape processes in the watershed [33, 38, 41, 45].

According to the Douro River Basin Confederation [47], the environmental benefits of eliminating unnecessary hydraulic structures are: recovery of the natural system, contributing to increased biodiversity; restoration of floodplains and adjacent wetlands, including the coastal environment with sediment replenishment; improvement of water quality through reduced hydraulic retention time, especially when siltation occurs due to sediments and when high nutrient concentrations lead to eutrophication phenomena; redistribution of sediments and re-establishment of solid transport, enhancing river dynamics and habitat renewal; improvement in nutrient distribution and the river's self-purification capacity; and recovery of longitudinal connectivity, allowing the movement of migratory fish and other organisms.

Another benefit concerns the maintenance or enhancement of the safety factors of the dam. Through maintenance techniques, partial or complete removal, the structural risk of the dam can be mitigated or eliminated.

6.2. Potential negative impacts

Many of the negative impacts are correlated with the positive impacts, differing primarily in the existence of process control, the proper understanding of its mechanisms and consequences on the ecosystem, as well as the correct characterization of materials and ecosystems prior to the decharacterization process.

Although there may be potential ecological benefits following the decharacterization process, these should be monitored and studied. There are cases where, after the partial or total removal of a dam, there was a reduction in important native species and the introduction of exotic species, which disrupted the existing ecosystem balance [35, 48].

Another example of negative impacts is the runoff of accumulated sediments in the reservoir [1, 29], which may be contaminated by toxic substances, affecting both aquatic life and the dependent biota [30], as well as compromising groundwater quality [5, 37] and increasing water turbidity [33].

The relative amount of sediment transported from reservoirs can vary from 10% to 80% after the first year of dam removal. However, this factor is strictly dependent on the nature of the sediment, as well as the hydrological conditions and the river's erosion potential [1, 34], aspects that must be studied [32]. The sediments accumulated in reservoirs contain significant

levels of carbon, which may be released into the atmosphere during and after the dam decharacterization process, emitting substantial amounts of greenhouse gases [49, 50].

The lowering of the water table in areas adjacent to the reservoir can lead to issues such as land subsidence, mass movements, and the drying up of water supply wells [28], as well as a decrease in property value near reservoirs used for recreational purposes. Another negative impact highlighted in the technical literature is the reduction in tax revenue resulting from the cessation of dam-related activities [35]. [4] also report the intensification of traffic for the removal of materials from the reservoir as a negative aspect, as the thousands of truck trips disturb established communities and increase local pollution due to fuel combustion.

7. CONCLUSIONS

Given Brazil's vast territorial expanse, the registration and management of dams present inherent challenges, particularly due to the jurisdictional overlap among municipal, state, and federal entities. Despite notable progress in recent years, a significant information gap persists regarding the status of many dams. It is estimated that over 50% of the dams registered in the National Dam Safety Information System (SNISB) cannot be assessed for compliance with the National Dam Safety Policy (PNSB) due to incomplete data on parameters such as dam height, reservoir volume, Associated Potential Damage (APD), and Risk Category.

The substantial socio-environmental liabilities associated with decommissioned dam sites, combined with the limited resources available to stakeholders, underscore the urgent need for regulatory authorities to establish systematic methodologies and guidelines for prioritizing dam decommissioning efforts. The study by [35], along with the flowchart proposed in this work, provides a valuable foundation for addressing this complex challenge.

Accessing the complete registry of dams throughout all phases of their lifecycle remains a significant challenge. Older structures often lack detailed design information and records of modifications or issues encountered during their operational lifespan. Currently, remote technologies, such as image-based records, offer valuable tools to assist in monitoring and evaluating the condition of dams over time.

Monitoring technologies and simulation models also serve as essential tools for dam management and decision-making in decharacterization processes. For instance, post-removal monitoring programs enable the evaluation of the effectiveness of the adopted measures from various perspectives, including geomorphological (sediment removal rates from the reservoir, changes in channel morphology), environmental (physicochemical indices), and biological (recovery of flora and fauna).

Finally, the importance of involving multiple stakeholders in the dam decharacterization process is highlighted, including: i) public administration (federal and regional), policymakers, specialized scientists, and the private sector; ii) environmental organizations; and iii) local communities. While the involvement of numerous entities may lengthen the process and increase the potential for conflicts, it ensures a more democratic approach, addressing the concerns of a society heavily affected by the abandonment and collapse of poorly maintained dams.

REFERENCES

[1] Doyle, M. W.; Harbor J. M.; Stanley, E. H. (2003). Toward policies and decision-making for dam removal. Environmental Management, 31.4 (2003): 453-465.

[2] Sentinelles Petitcoadiac Riverkeeper. (2002) Reclaiming free flow — An information guide on removing small abandoned dams. Part I of the Humphreys Mill Dam Decommissioning Project, Moncton, New Brunswick.

[3] Lanza, V. C. B.; Nunes, E. G.; Scussiato, T.; Lanza, F. C. B.; do Vale, B. B. (2022). Aspectos legais relacionados a descaracterização de barragens alteadas a montante. In: 17° Congresso Brasileiro de Geologia de Engenharia e Ambiental, Belo Horizonte. Proceedings of 17° Congresso Brasileiro de Geologia de Engenharia e Ambiental. ABGE, 2022, Belo Horizonte: pp. 1-10.

[4] Massignan, R. S.; Sánchez, L. E. (2022). O que significa descaracterizar barragens de rejeitos de mineração? Uma revisão sistemática da literatura. Engenharia Sanitaria e Ambiental, 27.2 (2022): 225-234.

[5] Zwahlen, R. (2022) Assessing the environmental impacts of hydropower projects. environmental earth sciences, Gewerbestrasse, Springer International Publishing 631p.

[6] Agência Nacional de Águas e Saneamento Básico - ANA. (2024). Relatório de Segurança de Barragens 2023. Brasília: ANA, 123p.

[7] Agência Nacional de Águas e Saneamento Básico - ANA. (2024). Sistema Nacional de Informações sobre Segurança de Barragens. Base de dados. Available at: https://www.snisb.gov.br/portal-snisb/inicio. Accessed in: nov. 01, 2024.

[8] Agência Nacional de Energia Elétrica (ANEEL). (2023). Resolução Normativa ANEEL nº 1064, de 2 de maio de 2023.

[9] Agência Nacional de Energia Elétrica (ANEEL). (2024). Formulário de Segurança de Barragens. Available at: https://www.gov.br/aneel/pt-br/centrais-de-conteudos/manuais-modelos-e-instrucoes/geracao/fiscalizacao/fsb. Accessed in: oct. 10, 2024.

[10] BRASIL. (2010). Law No. 12,334, of September 20, 2010. Brasília, 20 set. 2010.

[11] Morgenstern, N.R.; Vick, S. G.; Viotti, C. B.; Watts, B. D. (2016). Fundão Tailings Dam review panel: report on the immediate causes of the failure of the Fundão dam. 1850p.

[12] Koppe, J.C. (2020). Lessons learned from the two major tailings dam accidents in Brazil. Mine Water Environ, 40 (2020): 166–173.

[13] Cambridge, M; Shaw, D. (2019). Preliminary reflections on the failure of the Brumadinho tailings dam in January 2019. Dams and Reservoirs, 29.3 (2019): 113–123.

[14] Associação Brasileira de Normas Técnicas. (2024). NBR 13,028:2024: Mining - elaboration and presentation of of design of tailings, sediment and water dams - Requirements. 4 ed. Rio de Janeiro: ABNT, 24 p.

[15] Fraga, B. A. (2019). Análises de alternativas de fechamento de uma barragem, Final Course Work (Undergratuate), Mining Engineering Department, Federal University of Ouro Preto, Ouro Preto, 40p.

[16] Fernandes, R. B.; Silva, L. de S. M. da; Sieira, A. C. F. (2020). Estudo de Caso: Descomissionamento e descaracterização de barragens de água para fins de acumulação. In: XX Brazilian Congress of Soil Mechanics and Geotechnical Engineering, 2020, Campinas. Proceedings of XX COBRAMSEG, Campinas: ABMS: 8p.

[17] BRASIL. (2024). Law No. 14,066, of September 30, 2024. Brasília, 30 set. 2020.

[18] Robertson, P. K.; Melo, L. de; Williams, D. J.; Wilson, G. W. (2019). Relatório do Painel de Especialista sobre as causas técnicas do rompimento da barragem I do Córrego do Feijão. 88 p.

[19] Rotta, L. H. S.; Alcântara, E.; Park, E.; Negri, R. G.; Lin Y. N.; Bernardo, N.; Mendes, T. S. G.; Souza Filho, C. R. (2020). The 2019 Brumadinho tailings dam collapse: possible cause and impacts of the worst human and environmental disaster in Brazil. International Journal of Applied Earth Observation and Geoinformation, 90 (2020): 102119.

[20] Rose, R. L.; Mugi, S. R.; Saleh, J. H. (2023). Accident investigation and lessons not learned: accimap analysis of successive tailings dam collapses in Brazil. Reliability Engineering & System Safety, 236 (2023): 109308.

[21] Romani, A.; Pires, I. H. (2024). Estudo de tensão deformação em uma barragem de descaracterização. In: XXI Brazilian Congress of Soil Mechanics and Geotechnical Engineering, 2024, Balneário Camboriú. Proceedings of XXI COBRAMSEG, Balneário Camboriú: ABMS, 8p.

[22] Cadete, A.; Silva, L.; Cavalcanti, E.; Vilar, J. (2023). Projeto de instrumentação complementar na descaracterização de barragens de rejeito: estudo de caso. In: XXXIV Seminário Nacional de Grandes Barragens, 2023, Foz do Iguaçu. Comitê Brasileiro de Barragens, 19p.

[23] Agencia Nacional de Mineração - ANM. (2023). Report Trimestral: descaracterização de barragens a montante. Brasília: ANM, 22p.

[24] Neri, A. C.; Sánchez, L. E.; Alves, C. F. C.; Dupin, P. C.; Silva-Sánchez, S.; Baeta, A. M.; Modesto, A.; Christofolletti, C.; Cunha, C. B.; Tanikawa, D. H.; Piló, H.; Yoshida, H. T.; Bessa Jr., J. E.; Bezerra, J. S. B.; Sánchez, J.; Dias, L. G.; Souza, L. T. M.; Santos, M. R. G.; Borghieri, M. G.; Ferracini, M.; Massignan, R. S.; Monteiro, V. L. (2021). Avaliação Ambiental Integrada das Obras de Descaracterização de Barragens Alteadas pelo Método de Montante no Estado de Minas Gerais. São Paulo: Fundação para o Desenvolvimento Tecnológico da Engenharia, 2021, 192p.

[25] Carvalhais, R. M.; Gomes, R. P.; Ribeiro, S. M.; Pereira, F. M. S; Torres, S. M. (2024). Estudo de caso para descaracterização de barragem. In: XXI Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, 2024, Balneário Camboriú. Proceedings of XXI COBRAMSEG, Balneário Camboriú: ABMS, 2024, 7p.

[26] Department of Environment, Land, Water and Planning of State of Victoria – DELWP. (2016). Decommissioning dams: A guide for dam owners. DELWP, 2016, 38p.

[27] CONSELHO NACIONAL DE ÁGUA – CNA. (2017). Grupo de Trabalho para a identificação, estudo e planeamento da remoção de infraestruturas hidráulicas obsoletas. Lisboa, 31 de março de 2017.

[28] Pinto, A. C. C. (2010). Contribuições para o estudo de descomissionamento de barragens. Master's Thesis, Departamento de Engenharia Hidráulica e Sanitária, Universidade de São Paulo, São Paulo, 216p.

[29] Stanley, E. H.; Doyle, M. W. (2003) Trading off: the ecological effects of dam removal. Frontiers In Ecology And The Environment, 1.1 (2003): 15-22,

[30] Marks, J. C.; Parnell, R.; Carter, C.; Dinger, E. C.; Haden, G. A. (2006). Interactions between geomorphology and ecosystem processes in travertine streams: implications for decommissioning a dam on fossil creek, Arizona. Geomorphology, 77.3-4 (2006): 299-307.

[31] Jumani, S.; Andrews, L.; Grantham, T. E.; Mckay, S. K.; Duda, J.; Howard, J. (2023). A decision-support framework for dam removal planning and its application in northern California. Environmental Challenges, 12 (2023): 100731.

[32] Chagas, C.J; Moreira, R.M.; Augustin, C.H.R.R; Bandeira, J.V.; Carvalho Filho, C.A. (2022). Reuse of a dam as sediment trap and water reserve. Brazilian Journal Of Water Resources, 27 (2022): 11p.

[33] Duda, J. J.; Bellmore, J. R. (2022). Dam removal and river restoration. Encyclopedia Of Inland Waters (2022): 576-585.

[34] Warrick, J. A.; Bountry, J. A.; East, A. E.; Magirl, C. S.; R., Timothy J.; Gelfenbaum, G.; Ritchie, A. C.; Pess, G. R.; Leung, V.; Duda, J. J. (2015). Large-scale dam removal on the Elwha River, Washington, USA: source-to-sink sediment budget and synthesis. Geomorphology, 246 (2015): 729-750.

[35] Habel, M., Mechkin, K., Podgorska, K.; Saunes, M.; Babiński, Z.; Chalov, S.; Absalon, D.; Podgórski, Z.; Obolewski, K. (2020). Dam and reservoir removal projects: a mix of social-ecological trends and cost-cutting attitudes. Scientific Reports, 10 (2020):192210.

[36] Fessell, B.; Dituri, F.; Platz, C.; Zielinski, D.P. (2023). A river reborn: the fulfillment of the project partnership agreement for the removal of sabin dam. Journal Of Great Lakes Research, 49 (2023): 72-77.

[37] Jesus, C. K. C.; Sánchez, L. H. (2013). The long post-closure period of a kaolin mine. Revista Escola de Minas, Ouro Preto, 66.3 (2013): 363-368.

[38] Agoramoorthy, G. (2015). The future of India's obsolete dams: time to review their safety and structural integrity. Futures, 67 (2015): 22-25.

[39] Duz, B.G.; Moreira, C. A.; Casagrande, M. F. S.; Helene L. P. I. (2023). Mapping the integrity of rock mass with GPR: case study in decommissioning mining. SN Appl. Sci., 5 (2023): 231.

[40] Oliveira, L. A.; Braga, M. A.; Prosdocimi, G.; Cunha, A. S.; Santana, L.; Gama, F. (2023). Improving tailings dam risk management by 3D characterization from resistivity tomography technique: case study in São Paulo: Brazil. Journal Of Applied Geophysics, 210 (2023): 104924.

[41] Tavares, J. P. (2023). Análise de Descaracterização de Barragens de Rejeitos pelo Método de Olson. Master's Thesis. Departamento de Engenharia Civil e Ambiental, Universidade de Brasília, Brasília, DF, 108p.

[42] Xiang, Y.; Fu, Z.; Liu, C.; Zhang, K.; Wang, Y.; Shen, G. (2024). On-site Inspection and Dam Safety Evaluation. Singapore: Springer Singapore, 375p.

[43] Azevedo, F.; Azevedo, R.; França, R. (2022). Estudo Conceitual de Rebaixamento do Reservatório de uma Barragem de Mineração para Fins de Descaracterização. In: XX Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, 2022, Campinas. Proceedings of XX COBRAMSEG, Campinas: ABMS 2022, 8p.

[44] Leite, H. E. S. C.; Costa, L. M. da; Guimarães, L. J. N.; Ferreira, S. R. M.; Júnior, O. F. S. (2024). Aspectos sobre Instrumentação e Monitoramento de Barragens. In: XXI Congresso Brasileiro de Mecânica dos Solos e Eng. Geotécnica, 21., 2024, Balneário Camboriú. Proceedings of XXI COBRAMSEG, Balneário Camboriú: ABMS, 2024, 8p.

[45] Akinaga, P.; Namba, M.; Cunha, I.; Vinueza, G.; Fontes, G. L.; Negro Júnior, A. (2010) Paisagem recriada: projeto de descaracterização das barragens da mina da cachoeira. Revista Labverde, [S.L.], n. 1, p. 117, 11 set. 2010. Universidade de São Paulo, Agencia USP de Gestão da Informação Acadêmica (AGUIA). http://dx.doi.org/10.11606/issn.2179-2275.v0i1p117-133.

[46] Chagas, C. (2023). Cemig vai testar comporta da PCH Pandeiros: ação será realizada ainda em janeiro na pequena central hidrelétrica, cuja operação está suspensa desde 2008. Available at: https://brasilenergia.com.br/energia/cemig-vai-testar-comporta-da-pch-pandeiros/. Accessed in: nov. 16, 2024.

[47] CHD - Confederación Hidrográfica del Duero (2010). Diagnóstico de la conectividad longitudinal de la cuenca del duero. Clave 452-A 611.13.04/2008.

[48] Sanches, P. V.; Nakatani, K.; Bialetzki, A.; Baumgartner, G.; Gomes, L. C.; Luiz, E. A. (2006). Flow regulation by dams affecting ichthyoplankton: the case of the Porto Primavera dam, Paraná river, Brazil. River Research and Applications, 22.5 (2006): 555-565.

[49] Pacca, S. (2007). Impacts from decommissioning of hydroelectric dams: a life cycle perspective. Climatic Change, 84.3-4 (2007): 281-294.

[50] Liang, W.; Liu, X.; Lu, X.; Yu, R.; Qi, Z.; Xue, H. (2024). Impact of dam decommissioning on greenhouse gas emissions from a reservoir: an example from the inner Mongolia grassland region, China. Journal of Hydrology, 631 (2024): 130750.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

SUSTAINABLE RESERVOIR OPERATION: BALANCING POWER GENERATION AND DOWNSTREAM ENVIRONMENTAL MAINTENANCE.THE CASE OF THE CAHORA BASSA DAM.

Rosaque Guale[1]
University of Porto

Elsa Carvalho
University of Porto

Elsa Alves
LNEC

ABSTRACT

The Cahora Bassa dam is one of the largest hydropower plants in the Southern Africa region, under the Zambezi River Basin, in Mozambique territory. The independent and standalone operation of the Cahora Bassa reservoir determines the water fluctuation in the lower Zambezi basin.

Any unregulated reservoir operation may cause adverse impacts in terms of energy production performance and downstream ecosystem sustainability. A setup of the reservoir operation rules curves can contribute to maintaining regular flows in the dry and wet seasons.

The variability of the hydrological regime in the Zambezi basin is cyclical, interspersed between long wet hydrological years and short dry periods. Historical averages between inflows and outflows are nearly 2,000 m^3/s, generating an annual water renewal in the Cahora Bassa reservoir. However, any discharge with flows exceeding 5,000 m^3/s can significantly contribute to floods in the lower Zambezi Valley, associated with contributions from river flows downstream in the rainy season.

This paper addresses the Cahora Bassa reservoir's historical operation since dam construction and its impact on downstream water demand for the environment and other relevant stakeholders.

Keywords: Reservoir operation, sustainability, lower Zambezi, Cahora Bassa Dam

[1]rguale@outlook.com, Portugal

1. INTRODUCTION

The large dams are hydraulic engineering infrastructures that play relevant role on water storage for multiples proposes, for example, hydropower generation, flood mitigation, irrigation and water supply demand [1]. However, the main challenge on operation of large dam is the reservoir water management to satisfy all demand in view of sustainable and safety dam operation.

The operation rules can play influence on a process of the inundation of the floodplain and wetland downstream and contribute on balancing the river ecosystems, which is sensitively on hydrological regime alterations.

A hydropower reservoir operation is a continuous decision-making process of determining the water level of a reservoir and the volume of the water released from it. Chang et al. [2] considered that the process in seeking the trade-offs between socio-economic benefits of reservoirs and hydropower production can cause a dilemma for water managers when defining reservoir operation policy. In fact, the real-time reservoir operation and real-time release decision is a challenge on complying with the institutional framework.

Globally, reservoirs are often part of larger river basin systems determined by hydrological flows, water demands, and operation rules [3]. Upstream reservoirs influence the inflow to downstream reservoirs. Subsequently, the hydraulic operations of such systems are required to maintain reservoir performance from flood control, power generation and downstream water release [4]; [5].

Certainly, reservoirs are operated based on their defined rule curves under the assumption that such operations are planned to maximize energy production. The rule curve represents, for a specific time, the minimum and the maximum water level required in the reservoir to meet the needs for which the reservoir is designed, and it is expressed as water surface elevation or storage capacity versus time of the year [6].

The function of the reservoir is to store enough water to satisfy the different needs upstream and downstream of the dam. The water demand downstream is one of the crucial concerns when considering the amount of water required for environmental maintenance. Environmental flow maintenance emerged to mitigate the undesirable hydrological impacts of dams and water diversions to strengthen ecological water management [7].

The environmental flows are defined as the quantity, frequency, time and quality of water and sediment flows necessary to ensure the conservation and maintenance of aquatic and riparian ecosystems and other values associated with them, namely socioeconomic, livelihoods and human well-being that depends on these ecosystems [8]; [9]; [10].

However, the motivating issue of maintaining environmental flows based on the exploitation of hydraulic infrastructures and the dependence on inter and intra-annual hydrological regime variability in each river basin must be considered and evaluated, applying methodological principles that are sustainable on balance economic, social and environmental benefits.

According to Kurigi et al. [11], the proportional dynamics that maximize energy production can impose relatively fewer changes in the hydrological regime, meaning that, for example, flow parameters, such as frequency, time, duration and rates, can be maintained close to the natural regime without necessarily reducing energy production [11].

Therefore, the protection of riverine ecosystems downstream from dams requires reservoir operations to satisfy environmental flow requirements [12]. In practice, this often involves regulation of downstream river flows within the constraints of water resource demands for human supplies and irrigation activities.

It is recognised that the importance of natural flow regimes to sustain the ecological integrity of river systems has led to increased societal pressure on hydropower projects to change their reservoir operations to improve downstream ecosystems water demand. However, a complete reinstatement of natural flow regimes is often unrealistic when balancing water needs for ecosystems, energy production, and other human uses [13].

Certainly, is necessary a clear multidisciplinary approach, that will allow to define the operation rules and the quantity and quality of water that can be provided downstream for satisfaction, not only of the hydropower water demand, but also the maintenance of the ecosystem and various stakeholders.

Environmental maintenance often requires river basin managers to shift from a strategy of maximizing hydropower production to one of multi-purpose management, incorporating a range of upstream and downstream water users and concerns in addition to hydropower.

The main goal of this paper is to analyse the relationship between power generation and reservoir operation of Cahora-Bassa, considering different periods and how those periods contribute to the maintenance of the environment downstream of the dam. Indeed, the level of water fluctuation at the lower Zambezi has been strictly related to reservoir exploitation over the years.

The paper presents the Cahora Bassa hydropower plant characterization, including the reservoir operation rules. It also approaches the methodology applied to analyse the historical data of the annual power generation and the hydrological data in terms of inflows and outflows at the Cahora Bassa and their impact on environmental dynamic at the lower Zambezi.

2. CAHORA BASSA HYDROPOWER PLANT

2.1. Dam and Hydraulic characterization

Cahora Bassa Dam is one of the largest hydropower plants in the Southern Africa region, under the Zambezi River Basin, in Mozambique territory. The hydropower plant was constructed in 1974, and the reservoir filling was completed in 1977. The dam is a double curvature (Fig. 1), thin arch, concrete wall, with a height of 171 m above the foundation and a crest length of 303 m. It is founded on a complex of pre-Cambrian crystalline rock characterized by granitic gneiss [19].

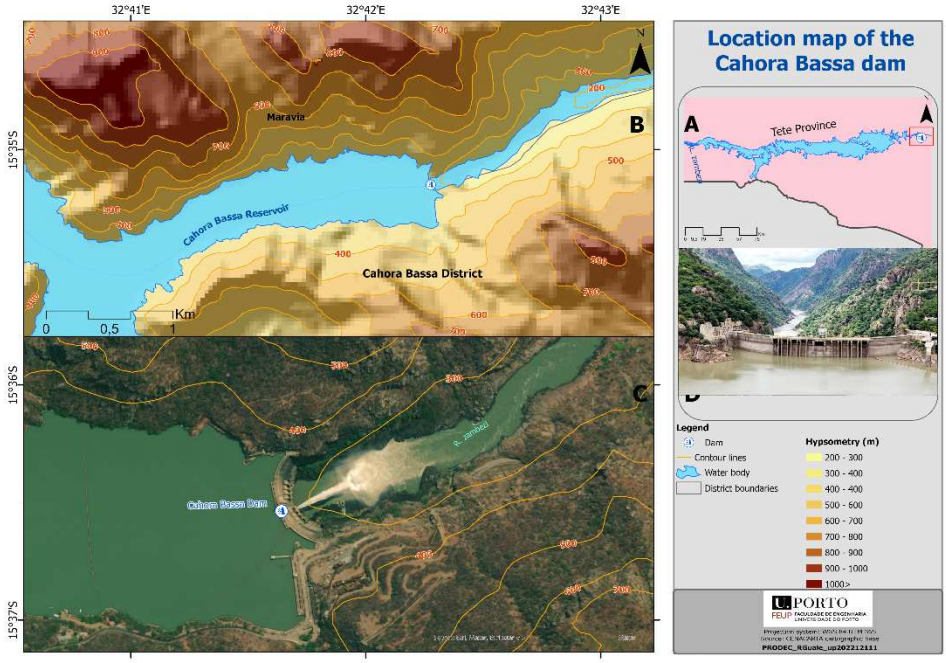


Fig. 1 - General overview of the Cahora Bassa Dam scheme.

As shown in Fig. 1, Cahora Bassa Dam was built in the narrowest part of a deep gorge of granite rock, approximately 18 km long. It is a geologically young valley with hills on the banks reaching around 700-800 m heights.

The installed capacity is 2,075 MW, divided by five ground power units of 415 MW / 480 MVA each, the called the Cahora Bassa South Bank (CBS) powerhouse. During the project development, a second powerhouse was planned on the left bank, the Cahora Bassa North Bank (CBN), also underground, intended to be built in the 2nd stage, with installed capacity estimated of 1245 MW.

Additionally, the complex of underground works on the right bank includes hydraulic circuits, namely the water intakes (threshold at elevation 275 m.a.s.l.), the five penstocks, the powerhouse (lowest area at elevation 172.8 m.a.s.l.), the caves of the two balancing chimneys

and the two escape galleries (restitution). Upstream of the south powerhouse (Cahora Bassa South Bank), the transformer room and the cable shaft are also located.

The mid-level spillways consist of eight 16mx10m rectangular stainless-steel openings with sills controlled by radial gates downstream. The dam is also equipped with a gated surface spillway that can be used to skim debris from the reservoir [14];[19];[20].

Also, there is a discharge surface spillway (volet), at the crest of the dam's central console, with a single span of 12 m and threshold at 320.8 m.a.s.l., equipped with a gate that is activated by a vertical hydraulic device [14], which is intended to allow the passage of floating material, namely vegetation and weed plants, that sometimes during rain are retained in the intake area and close of the dam hall.

2.2. Cahora Bassa reservoir characterization

The Cahora Bassa reservoir is an artificial lake located in the central province of Tete, Mozambique, about 250 km from the Kariba Dam to the mouth of the reservoir at Zumbo, in the middle course of the Zambezi River, between latitude 15°39' and 15° 35' S and longitude 30° 30' and 32° 42'E (Fig. 2). This reservoir was formed by constructing a dam on the Zambezi at the Cahora Bassa Falls in the late 1960s to produce electricity.

The Cahora Bassa reservoir has high elevations at the western end and low elevations near the dam. In Zumbo, the average elevation is around 325 m and in downstream of the dam, the average elevation is 168 m. The distance from Zumbo to the dam wall is 255 km following the main river channel course.

At the full flood control level (at elevation 326 m), the reservoir has a maximum depth of 131.68 m near the dam and an average depth along the reservoir of 23.87m. The total volume of water stored at this elevation is 64,65 km^3 and covers a flooded area of about 2,708 km^2. From 189 m downwards, the volume and the flooded area are equal to zero.

Considering that 100% of the volume available for electricity production corresponds to 326 m water level (FSL), the minimum water level for energy production (295 m) is equivalent to 23% of the total volume of the reservoir. The maximum water storage capacity of the reservoir is 73,18 km^3, and the dam has flooded up to an elevation of 329 m. The useful volumes of the reservoir point to 49,94 km^3 and 58,45 km^3 at levels 326m and 329 m, respectively. Between these elevations there is a water storage capacity of around 8.5 km^3, which represents the emergency storage buffer zone [22].

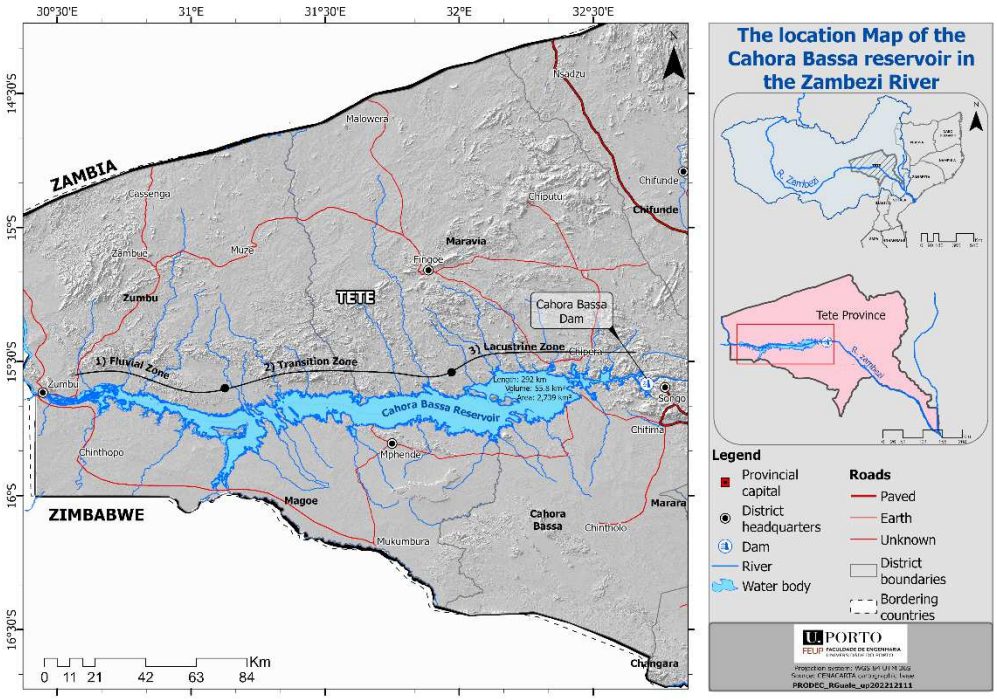


Fig. 2 - The location Map of the Cahora Bassa reservoir in the Zambezi River

The Cahora Bassa reservoir has an elongated shape, and the irregular distribution of its water masses influences the typical distinction between three zones: (1) a fluvial zone where the Zumbo Basin is located, characterized by being narrow, with higher flow velocity compared to the downstream stretch, providing greater sediment transport capacity. It is in this stretch that the main tributaries of the reservoir are located (the Zambezi and Luangwa rivers); (2) a transition zone, covering the Messenguezi, Carinde, and Mucanha basins, characterized by being a “U” shaped valley, broader and deeper than the river zone, with almost undetectable current speed. This stretch is characterized by instability and greatly influenced by fluctuations in the reservoir's water level; and (3) a lacustrine zone, that covers the Magoé, Chicoa and Garganta basins. This zone is the longest, widest, and deepest, with a longer water retention time and lower dissolved substances and nutrient concentration (Fig. 2).

The south bank is extensively flat, where the Mágoe National Park is located, covering an area of 3,558.52 km^2, including a buffer zone in the Cahora Bassa reservoir. In contrast, the north bank is predominantly mountainous.

2.3. The rules of operational hydraulic exploration

Hydraulic-operational safety is the dam's capacity to satisfy the hydraulic-operational behaviour requirements of safety and dam exploration [14]. The maximum water discharge capacity of Cahora Bassa, including from spillways, is 14,000 m^3/s [14]. The turbine flow per generator set can reach 452 m^3/s for a gross head of around 103.5 m.

The Cahora Bassa reservoir management is carried out in compliance with the power generation exploitation plan and the dam safety rules. Pursuing this objective implies that the water resource is managed based on scientific principles and probabilistic risk assessment, considering the river's historical hydrological regime, the new climate change factors, and the always fallible medium and long-term meteorological forecasts.

The rule curve of the Cahora Bassa reservoir, illustrated in Fig. 3, represents the upper safety limit of the operation and defines the maximum risk conditions that must not be exceeded, to prevent an additional risk to the safety of the dam or overtopping and, consequently, an unprecedented catastrophe at the dam and in the lower Zambezi valley [19];[20].

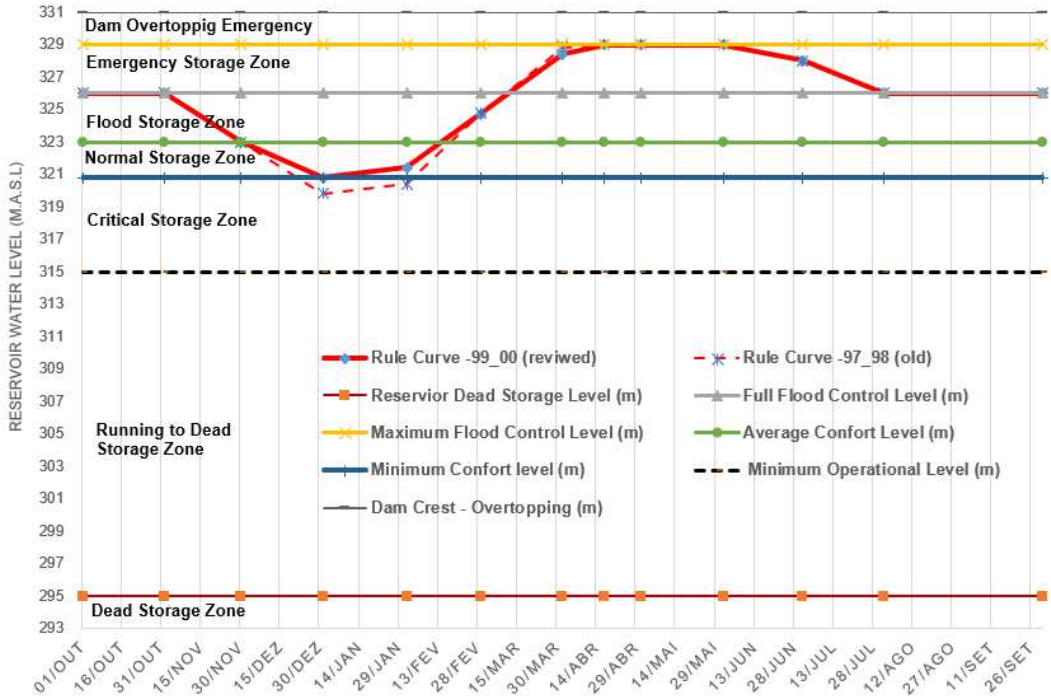


Fig. 3 - Cahora Bassa operational hydraulic rule curves [Source: adapted from Cahora Bassa Database].

It is important to highlight that revisions were made of the rule curves (1998_99_old and 1999_00_new) following the National Directorate of Water Authority (DNA) guidelines and also the recommendations of the *Seminar on the Sustainable use of the Cahora Bassa Dam and the Zambezi Valley,* held in Songo, in September 1997, in which was stated that “with the establishment of a flow and sediment regime very close to natural seasonal and inter-annual fluctuations, there will be rapid and significant ecological benefits with concomitant social impacts and general improvement of conditions in where the populations of the Lower Zambezi Valley live” [21].

The reviewed hydraulic-operational rule curve consider that the maximum flood control level is 329 m, and at the beginning of January, the reservoir must be at a level of 320.8 m to absorb

excessive volumes of water that flow in the following months, especially in February, March and April, when inflows are normally high.

The application of these management criteria made it possible to act, both in the field of minimizing the impacts of droughts and, above all, in mitigating the effects of floods. The minimum exploration level is defined at 295 m, for which useful storage is assumed to be zero (0%). However, the reservoir at this level may have a non-useful volume of around 14.5 billion m^3, corresponding to the dead storage. The full flood level (326 m) is the full storage level, corresponding to 100% of the permissible level.

In extreme flood events, the reservoir's full supply level (329 m) can be reached for flood control downstream.

According to the Dam and Reservoir Exploration Regulations, the exploitation of the Cahora Bassa reservoir must achieve the following objectives [19]; [20]:

i. *To ensure the hydraulic-operational safety of the dam and associated works, considering that the daily rise rate does not exceed 13 cm/day.*
ii. *To guarantee the storage necessary for hydroelectric production to satisfy contractual commitments with customers following exploration plans and the availability of generator sets.*
iii. *To ensure adequate satisfaction with the hydrological, ecological, and environmental regimes in the reservoir and the downstream valley.*
iv. *To ensure the safety of people and goods, navigability, and minimization of the effects of extreme events (droughts and floods), considering the needs and mitigation of flood management downstream by regulating the outflows.*

Thus, the power generation plan and meteorological forecasts of inflows constitute the determining basis for whether discharges are carried out in the rainy season and at the beginning or by the end of the wet period.

3. ANALISYS OF CAHORA BASSA DAM AND RESERVOIR OPERATION

The methodology used to analyse the Cahora Bassa dam and reservoir operation was based on the reservoir water balance concept used to calculate the hydrological parameters (reservoir water level; inflows and outflows) from 1976/77 to 2022/23. This period includes more than 45 years, with a record of the drought and flood events that affected the Cahora Bassa reservoir operation and management.

The hydrological data are based on the calculation of the monthly average of observed and calculated daily data from the Cahora Bassa database. For a clear understanding of the trend

line of the hydrological regime that governed the operation of the Cahora Bassa Dam and reservoir, four notable periods were defined and characterized according to the relevant events records during the operation and exploration of the Cahora Bassa reservoir over the years, namely:

- P1 (from1976/77 to1983/84), characterized by the commission of generators for power generation and early interruption due to the destruction of transmission lines.
- P2 (from 1983/84 and 1997/78), corresponding to a total shutdown of the energy transmission lines, which coincides with the long period of drought in Southern Africa.
- P3 (from 1997/98 to 2007/08), implementation of recommendations from the *Seminar on the Sustainable Use of the Cahora Bassa Dam and the Zambezi River Valley* (held in Songo, September 1997); completion of refurbishment of transmission lines and restarting of the full exploration of the installed power generation capacity.
- P4 (from 2007/08 to 2022/23), characterized by a combination of the long-wet seasons; cycle of the drought events and full exploitation of the installed hydropower capacity at Cahora Bassa Dam.

3.1. Historical of Cahora Bassa power generation

The historical exploration of power generation in Cahora Bassa certainly influenced the oscillation of the water level in the dam's reservoir and, evidently, the variations in the inflows and outflows into the dam (Fig. 4). The dam was built with a unique proposal for generating energy, mainly for export to neighbouring countries, such as South Africa and Zimbabwe. However, power generation also benefits Mozambique, where demand has increased over the years due to the need for upcoming domestic connections and mainly to satisfy the industrial market by developing new mineral resource projects in the country.

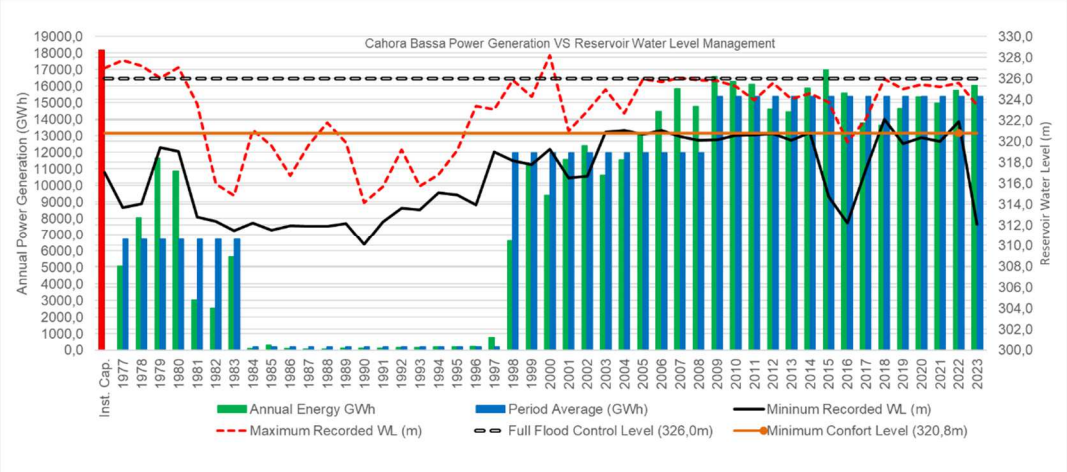


Fig. 4 - The historical annual power generation and reservoir water fluctuations in Cahora Bassa dam (1977-2023) [adapted from Cahora Bassa Database].

Considering the characterization of the four periods from 1976 to 2023, which illustrate the operation of the reservoir and the water level fluctuation, power generation is described as follows.

• P1 (from February 1977 until mid-1983): the average of power generation was 6692 GWh/year, representing 36.8% of the total installed capacity. In that period, the reservoir water level reached a maximum of 327 m, standing in flood and emergency operation zones in consecutive hydrological years.

• P2 (around end of 1983 and 1997): the average power generation was 201 GWh/year, representing only 1.1% of the installed capacity as result of a total shutdown of the energy transport system for South Africa. This coincides with the long period of drought in Southern Africa from the 1980s to the mid-1990s, with water level recorded below the operational comfort zone. In the same way, the prolonged drought generally affected the entire Zambezi Basin. Nevertheless, the civil war in Mozambican territory ended in 1992, destroying the main energy transmission systems.

• P3 (from years 1998 to 2007): the average power generation was 11972 GWh/year, representing 65.9% of the total installed capacity. The recommendation and output of operation of the reservoir from the *Seminar on the Sustainable Use of the Cahora Bassa Dam and the Zambezi River Valley* contributed to recovering the reservoir storage with record of long-wet seasons and with implantation of the reviewed operation rules curves.

• P4 (from years 2008 to 2023): the records of power generation were achieved in 2015 (16.978 GWh/year) and in 2023 (16.058 GWh/year). The average of power generation in this period was 15.381 GWh/year, representing 84.6% of the total installed capacity.

3.2. Inflow variability in the Cahora Bassa reservoir

The variability relationship of the flow regime observed in Cahora Bassa is based on historical data of annual and monthly averages recorded in each defined period, corresponding to the main phases of operation and exploitation of the dam/reservoir to produce hydroelectric energy.

The hydrograph in Fig. 5 shows that, for the period P1, the historical average of inflows was influenced by the occurrence of exceptional floods in the hydrological year 1976/78, in which the monthly average between February and April of 1978 was above 4,500 m^3/s, and it contributed to the highlights a high-water availability.

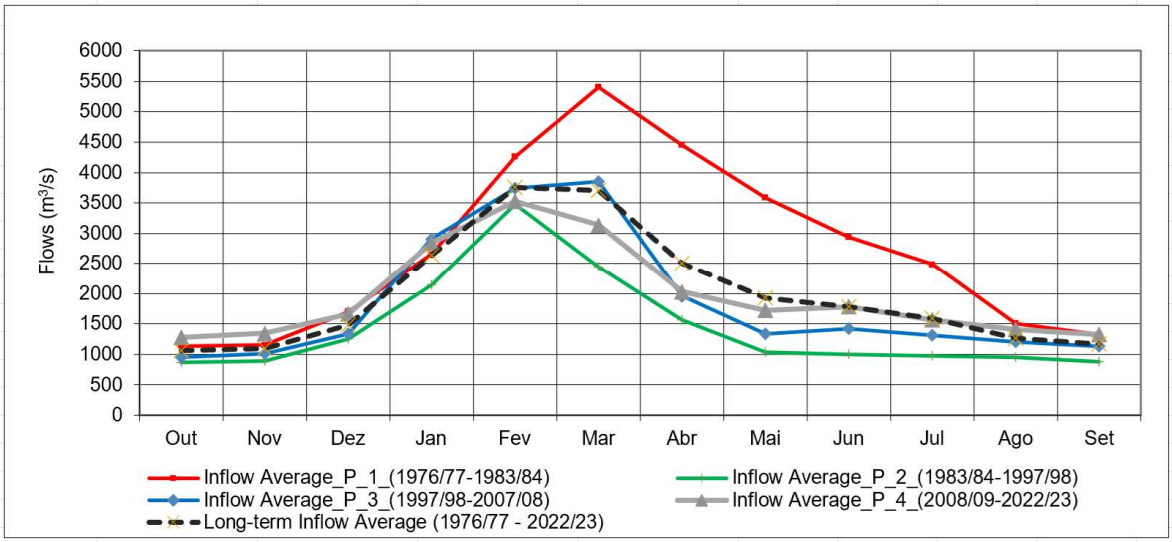


Fig. 5 - Monthly inflow average at the Cahora Bassa in four periods, representing the exploitation of the reservoir, between 1976/77 and 2022/23.

Period P2 presents extremely low inflows, with monthly average values below 2,000 m^3/s, except for the inflows recorded at the peak of the rainy season in February, whose values were slightly close to 3,000 m^3/s. In the remaining periods, P3 and P4, inflows were very close to the long-term inflows 3753 m^3/s, between February and March.

Notably, the historical inflows to the Cahora Bassa reservoir tend to decrease and currently stand at an annual average of approximately 2,000 m^3/s.

3.3. Outflow variability in the Cahora Bassa reservoir

According to the defined reservoir operations rules, the outflow at Cahora Bassa dam resulted from discharges of the turbine and spillways. The Fig. 6 shows the monthly outflows average of Cahora Bassa during the four periods. The variability of the monthly outflow average observed in the P1 may result in inadequate compliance with regular flows in the lower Zambezi and be associated with inefficient operational dam safety rule curve guidance. Also, the irregular power generation, which may have contributed to high fluctuations of water level during that period in the downstream of Zambezi River valley.

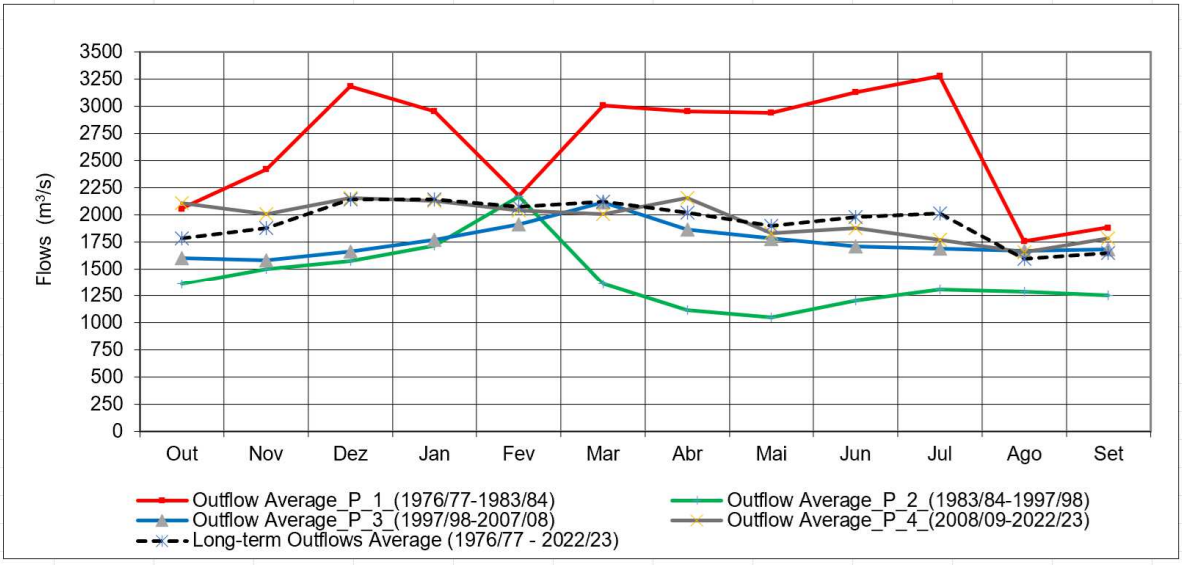


Fig. 6 - Monthly average flow of Cahora Bassa during the four periods, representing the exploitation of the reservoir, between 1976/77 and 2022/23.

As previously mentioned, the lowest outflows were observed downstream of the Cahora Bassa dam in P2 because of the generation stoppage and the extreme drought event recorded in that period, between the decade 80 and 90 in Southern Africa. During this period, the monthly outflow average was below 1,750 m^3/s in the rainy season, except in February, with an average of around 2500 m^3/s. In the dry season, between April and September, the outflow average was below 1250 m^3/s, significantly affecting the downstream of Cahora Bassa and, consequently, the dynamics of the ecosystems along the Zambezi Delta.

These flow rate values gave rise to various questions and discussions, which motivated the Seminar on the *Sustainable Use of the Cahora Bassa Dam and the Zambezi Valley* in Songo, in September 1997 [21], whose main objectives were to analyse the operation of the Cahora dam and to produce specific recommendations to ensure the sustainable regulation of flows in the Lower Zambezi.

In the P2 period, the outflows from Cahora Bassa dam were dependent on the hydraulic-operational management of the discharge to the downstream, which had to be managed uninterruptedly during dry and wet seasons until the beginning of P3.

However, P3 begins not only with concern about irregularities in outflows downstream of the dam but also with the resumption of energy transmission to leading customers, which guarantees consistency of discharge maintenance based on turbine flows and because of scheduled discharges to mitigate floods downstream.

The periods described as P3 and P4 showed consistency in the discharge of the flows downstream of the Zambezi River valley as straightforward implementation of the recommendation from the referred Songo Seminar, with global average values close to 2,000

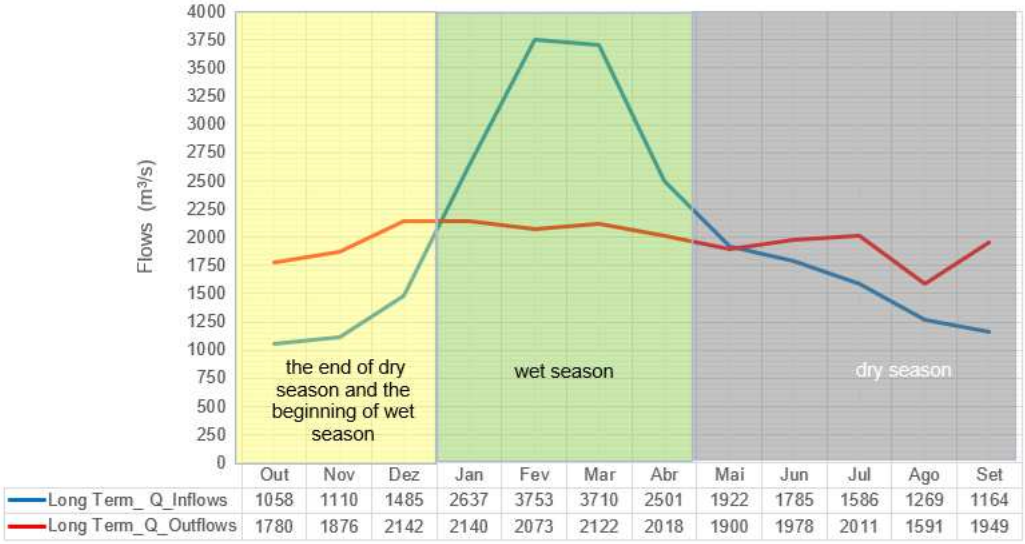
m^3/s. It indicates strict compliance with the revised rule curve for operating the dam and reservoir, which proposed adequate regulation of the hydrological regime downstream of the Cahora Bassa Dam.

Therefore, it can be assumed that a review of the rule curve and strict application of the dam and reservoir exploitation rules may have satisfactorily contributed to the regularization of the river regime in the lower Zambezi Valley over the last 25 years (1998 to 2023).

Therefore, the long-term average of outflows from Cahora Bassa is 1,940 m^3/s, representing the current average of regular discharge to the lower Zambezi River.

3.4. Impact of reservoir operation in the Lower Zambezi Basin

The Fig. 7 shows the long-term Cahora Bassa reservoir inflows, based on outflows from the Middle Zambezi catchment (Kariba and Kafue dams) upstream, including runoff from Luangwa and other contributions from the river and other ungauged catchments.



	Out	Nov	Dez	Jan	Fev	Mar	Abr	Mai	Jun	Jul	Ago	Set
Long Term_ Q_Inflows	1058	1110	1485	2637	3753	3710	2501	1922	1785	1586	1269	1164
Long Term_Q_Outflows	1780	1876	2142	2140	2073	2122	2018	1900	1978	2011	1591	1949

Fig. 7 - The long-terms inflows and outflows in Cahora Bassa dam based on annually of the hydrological years 1977/78 to 2022/23 [elaborated using HCB-Database].

Any decision regarding reservoir operations will positively or negatively affect the lower Zambezi and delta areas. However, the role of the Cahora Bassa Dam is to store flood waters and release water through hydropower generation, and, at the same time, it must play a relevant role in mitigating floods downstream.

However, 21 km^3 of storage is created annually for flood control, allowing the Cahora Bassa Reservoir to control moderate to high floods, where peak flows are attenuated by the Cahora Bassa Dam, preventing flooding in the downstream valley. These challenges require establishing an efficient inflow forecasting and decision support system for reservoir and dam operations, with less negative impact in the lower Zambezi.

In Southern Africa, hydrological year start on October, but the wet season is more observed in January to the end of April, with monthly average flow above of 3,000 m^3/s. Certainly, the dry season is longer than the wet season, from May and extended to the end of October, with monthly average of inflows below 2,000 m^3/s.

The Cahora Bassa dam's long-term outflows are slightly around 2,000 m^3/s, meaning that the energy exploration plan governs them and leads them to maintain an annual average of flows to the lower Zambezi valley.

Considering the operating rules of Cahora Bassa, additional discharges are observed in the rainy season, with flows increasing above the minimum. Meanwhile, all discharge decisions are normally notified by the water authority, in this case, the Regional Administration of Water - Centro (ARA-Centro, IP), to establish communication with downstream stakeholders, as is the case with small farmers who carry out agricultural activities on the banks of the river. On the river and small islands, communities relocated along the riverbank, where fishing and navigation commonly occur.

However, floods and cyclones are unpredictable and worse when linked to climate change events. In this case, it is requested that the management of the Cahora Bassa reservoir contributes to mitigating unusual flood peak flows that occurred in rivers downstream, such as the cases of two major cyclones, Idai (in 2019) and Ana (in 2022), which seriously affected the lower Zambezi in terms of flooding.

3.5. Downstream environmental maintenance

The record of floods that have occurred downstream of Cahora Bassa has often been associated with the discharges caused by the operation and management of the Cahora Bassa reservoir, but rather result, in part, from the high persistence of precipitation in the tributaries downstream in the case of rivers Luia-Capoche, Mavudzi, Revubue, Luenha-Mazoe, Chire and other small rivers.

The estimated water level fluctuation downstream, in the city of Tete, in result of the operation of the Cahora Bassa dam, when reach 142 m.a.s.l can cause flooding with discharges in the order of 5,000 m^3/s, without considering the contributions from tributaries downstream of the dam.

Table 1 shows the magnitude of flood events observed at the Cahora Bassa dam and the city of Tete E320. The higher upstream flows of the Zambezi and Luangwa rivers are attenuated at the Cahora Bassa Dam. However, even if flows at Cahora Bassa are reduced to less than 5000 m^3/s, it is not always possible to reduce the impact of flooding downstream, due to contributions from unpredictable flows downstream of the dam during the rainy season.

Table 1 - Major floods observed at the lower Zambezi basin between 1976/77 and 2022/23 with pick flows around 5,000 m³/s (Source: adapted from database: HCB, 2023; ARA-Centro, 2023).

Extreme Pick Flood at Cahora Bassa Dam	Cahora Bassa Inflow + Luangwa River (m³/s)	Cahora Bassa Maximum Outflow (m³/s)	Zambezi Flow at Tete City E320 (m³/s)
March 1978	18, 990	14, 753	18, 842
March 1989	12,914	4,882	5,639
February 2001	13,797	9,032	14, 849
February 2007	10, 029	8,083	9, 475
January 2008	10,364	6, 630	8,081
January 2011	8,223	6,527	8,698

After the construction of Cahora Bassa Dam, a major flood occurred in 1978. At Tete city, the maximum discharge was 18,842 m³/s after routing through Cahora Bassa Reservoir. That was really test of hydraulic project in term of dam safety. Other major flood was observed in Tete City in 2001 with maximum flows around 14,849 m³/s, with significate in impact on inundation in the lower Zambezi.

The Lower Zambezi (Fig. 8) extends from the Lupata Gorge in Mozambique to the coast, where the river exits at Chinde. It also includes those parts of the Chire River, in Malawi, below the Kapichira Falls near Blantyre. This section of the Zambezi is typified by a broad floodplain, often with many parallel channels and shifting sandbanks. At the same time, the coastal portion includes extensive grasslands and freshwater swamps, dunes and mangroves [16].

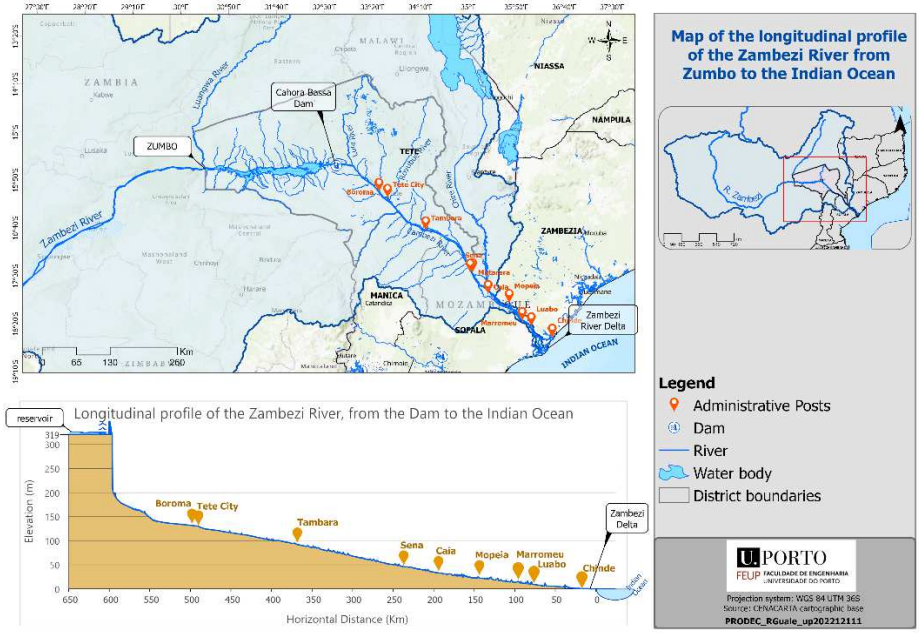


Fig. 8 - Map of the lower Zambezi with longitudinal profile

In these cases, severe flood recorded with significant impacts affected mainly the districts Mutarara, Sena, Caia, Mopeia, Luabo, Marromeu and Chinde, as showed in Fig. 8, the vulnerable flood sites.

4. CONCLUSIONS

Studying the sustainability of hydropower projects requires understanding the variability of the hydrological regime. It is necessary to identify hydrological models that can generate the minimum flow needed for different scenarios, considering that the minimum historical averages of 1,400 m^3/s were calculated since the operation of the Cahora Bassa Dam.

The variability of the hydrological regime in the Zambezi Basin is cyclical, interspersed between long wet hydrological years and short dry periods, with historical averages between inflows and outflows in the order of 2,000 m^3/s, which generates an annual renewal of water in the Cahora Bassa reservoir.

In the methodological periodization for statistical analysis of hydrological data between 1976/77 and 2022/23, four periods were defined: Period P1 (1976/77 to 1983/84) with irregular flows, P2 (1983/84 to 1996/97) with lowest recorded flows, P3 (1997/98 to 2007/08) with highest flows and P4 (2007/08 to 2022/23) with moderate flows. But, considering the output from the Seminar on the *Sustainable Use of the Cahora Bassa Dam and the Zambezi Valley*, held in Songo in September 1997, and globally analysing the operation of the Cahora Bassa reservoir during the study period from 1976 to 2023, it is concluded that the average outflows of around 1,960 m^3/s were observed after the revision of the rule curves in 1999, which contributes to the intra-annual and intra-annual regularization of the hydrological regime along the valley downstream of Cahora Bassa Dam.

In a regular discharge regime, which can generate daily Cahora Bassa flows exceeding 5,000 m^3/s, it can significantly contribute to the occurrence of floods in the lower Zambezi valley (for the districts Mutarara, Sena, Caia, Mopeia, Luabo, Marromeu and Chinde), when these are associated with contributions from river flows downstream in the rainy season.

The recommendation from the Songo Seminar (1997) contributed to the setup of the Cahora Bassa reservoir's operating rules curve and, evidently, improvements in the regularization of the hydrological regime downstream of the Cahora Bassa Dam.

Therefore, the complexity of managing river courses through the existence of hydraulic infrastructures must be guided by the principles that balance the exploitation of reservoirs to guarantee environmental maintenance, not only with a review of dam operations rules and exploitation plan but relevant commitment among dam operators and water authorities in the

Zambezi Basin, that will contribute to institutional coordination with the upstream dams, which can improve the communication system and exchange of hydrological information, for the coordinated management of reservoirs in the Zambezi Basin.

REFERENCES

1. Hager WH, Schleiss AJ, Boes RM, Pfister M. Hydraulic engineering of dams [Internet]. CRC Press; 2020 [cited 2024 May 30]. Available from: https://www.taylorfrancis.com/books/mono/10.1201/9780203771433/hydraulic-engineering-dams-willi-hager-robert-boes-michael-pfister-anton-schleiss

2. Chang J, Meng X, Wang Z, Wang X, Huang Q. Optimized cascade reservoir operation considering ice flood control and power generation. Journal of Hydrology. 2014 Nov 27;519:1042–51.

3. Vonk E, Xu Y, Booij MJ, Zhang X, M. Augustijn DC. Adapting multireservoir operation to shifting patterns of water supply and demand: A case study for the xinanjiang-fuchunjiang reservoir cascade. Water resources management. 2014;28:625–43.

4. Rani D, Moreira MM. Simulation–optimization modeling: a survey and potential application in reservoir systems operation. Water resources management. 2010;24:1107–38.

5. Singh A. Simulation–optimization modeling for conjunctive water use management. Agricultural Water Management. 2014 Jul 31;141:23–9.

6. Jiang Z, Ji C, Sun P, Wang L, Zhang Y. Total output operation chart optimization of cascade reservoirs and its application. Energy Conversion and Management. 2014 Dec 1;88:296–306.

7. Lebel L, Lebel P, Chitmanat C, Sriyasak P. Benefit sharing from hydropower watersheds: Rationales, practices, and potential. Water Resources and Rural Development. 2014 Oct 1;4:12–28.

8. Alves H, Henriques A. O caudal ecológico como medida de minimização. Métodos para a sua determinação. Actas do 6º SILUSB/1º SILUSBA, Simpósio de Hidráulica e Recursos Hídricos dos Países de Língua Oficial Portuguesa Lisboa. 1994;11:177–90.

9. Dyson M, Bergkamp G, Scanlon J. Flow: the essentials of environmental flows. IUCN, Gland, Switzerland and Cambridge, UK. 2003;20–87.

10. Brown C, Watson P. Decision support systems for environmental flows: Lessons from southern Africa. International Journal of River Basin Management. 2007;5(3):169–78.

11. Kuriqi A, Pinheiro AN, Sordo-Ward A, Garrote L. Influence of hydrologically based environmental flow methods on flow alteration and energy production in a run-of-river hydropower plant. Journal of Cleaner Production. 2019 Sep;232:1028–42.

12. Yin XA, Yang ZF, Petts GE. OPTIMIZING ENVIRONMENTAL FLOWS BELOW DAMS. River Research and Applications. 2012 Jul 1;28(6):703–16.

13. McManamay RA, Brewer SK, Jager HI, Troia MJ. Organizing Environmental Flow Frameworks to Meet Hydropower Mitigation Needs. Environmental Management. 2016 Sep 1;58(3):365–85.

14. Carvalho E, Matsinhe B, Mendes P, Oliveira S, Alegre A, Ciudad-Real M, Skolnik D, El-Idrissi M. SSHM Systems for Dams: 12 Years' Experience in Cahora Bassa Dam. Equipment, Software, and Main Results. In: Limongelli MP, Giordano PF, Quqa S, Gentile C, Cigada A, editors. Experimental Vibration Analysis for Civil Engineering Structures. Cham: Springer Nature Switzerland; 2023. p. 273–82.

15. Nones M, Ronco P, Di Silvio G. Modelling the impact of large impoundments on the Lower Zambezi River. International journal of river basin management. 2013;11(2):221–36.

16. Chirashi ME. Zambezi River-sea route for the SADC Region: A study on how to extend navigation on the Zambezi River. Zimbabwe Journal of Science and Technology. 2014;9(1):1–20.

17. Beilfuss RD, Davies BR. Prescribed Flooding and Wetland Rehabilitation in the Zambezi Delta, Mozambique. In: Streever W, editor. An International Perspective on Wetland Rehabilitation [Internet]. Dordrecht: Springer Netherlands; 1999. p. 143–58. Available from: https://doi.org/10.1007/978-94-011-4683-8_15

18. LNEC 2009. Cahora Bassa Hydroelectric scheme: Dam behaviour analysis, interpretation and precision. HCB and LNEC report 341/2009/NO, Laboratório Nacional de Engenharia Civil, proc.0402/00/17055, Lisboa. unpublished.

19. Silva, H.1997. Notas explicativa da análise da curva-guia e da simulação de gestão diária da albufeira de Cahora Bassa. HCB/DEB 97.07. 09, Songo, Mozambique. unpublished.

20. Silva, H.2004. Normas de exploração da barragem e albufeira de Cahora Bassa. Nota Tecnica. Songo, Mozambique. unpublished.

21. ARPAC.1998. Workshop sobre Uso Sustentável da Barragem de Cahora Bassa e o Vale do Zambeze – Relatório Final. ARPAC- Arquivo de Património Cultural, workshop – Songo, 29 de set-2 de out-1997. Ministério da Cultura, Juventude e Desportos, Mozambique -unpublished.

22. Subtech Group, S.G., "Survey Report – Bathymetry of the Cahora Bassa Reservoir, Airborne LIDAR and ROV surveys, final report, 2017" – unpublished.

23. https://www.hydropower-dams.com/articles/environmental-assessment-using-satellite-based-data-analytics-at-cahora-bassa-reservoir-mozambique/.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

RETHINKING ECONOMIC RISK FOR AMAZONIAN TAILINGS DAMS

P. Véliz[1]
PEK Teknep Eng. S.A.

P. Kong
PEK Teknep Eng. S.A.

ABSTRACT

This paper critically examines the gaps in traditional economic risk assessment methodologies for tailings dam projects in the Amazon. While conventional approaches focus on estimating costs related to dam failures, environmental impacts, and other feasibility factors, they often overlook the full spectrum of risks unique to this ecologically sensitive region. This study identifies all applicable risks for Amazonian projects, categorizes types of risk analysis, and proposes holistic techniques to evaluate these risks. By incorporating environmental degradation, biodiversity loss, indigenous community displacement, and climate change impacts, this paper advances a comprehensive framework for risk assessment and monetization tailored to the Amazon's unique context. The findings aim to inform engineers, regulators, and stakeholders of actionable strategies for sustainable tailings dam management.

Keywords: Tailings dams, Amazon, risk analysis, environmental impact, economic risk assessment, long-term rehabilitation.

[1]Ppinov2@pekteknep.com.br, Brazil

1. INTRODUCTION

The Amazon rainforest, often referred to as the "lungs of the Earth," is an ecological and cultural treasure of global importance. Despite its critical role in climate regulation and biodiversity, the region faces increasing threats from industrial activities, including mining. Tailings dams, integral to mining operations, present significant risks if not managed responsibly.

Traditional risk assessments for tailings dams focus on technical and financial factors but often fail to capture the complexities of operating in the Amazon. This paper highlights the unique risks associated with such projects and proposes comprehensive methods to analyze and mitigate them, fostering more sustainable outcomes.

2. APPLICABLE RISKS FOR AMAZONIAN PROJECTS

2.1. Environmental Risks:

- Deforestation and habitat destruction.
- Biodiversity loss, including impacts on endangered species.
- Water contamination and hydrological disruptions.
- Long-term soil degradation.

2.2. Social and Community Risks:

- Displacement of indigenous communities.
- Loss of cultural heritage.
- Public health risks due to pollution.
- Conflicts over land use and resources.

2.3. Operational Risks:

- Equipment failures and accidents.
- Logistical challenges in remote areas.
- Risks from natural disasters (flooding, landslides).

2.4. Regulatory and Legal Risks:

- Non-compliance with environmental and indigenous protection laws.
- Penalties for violations and regulatory delays.

2.5. Economic Risks:

- Cost overruns during construction or operation.
- Fluctuating commodity prices affecting project viability.
- Long-term costs of rehabilitation and decommissioning.

2.6. Reputational Risks:

- Public backlash from environmental and social harm.
- Reduced investor confidence due to poor ESG performance.

2.7. Climate Change Risks:

- Increased frequency of extreme weather events.
- Vulnerability of infrastructure to climate variability.

3. RISK ANALYSIS TYPES AND RECOMMENDED TECHNIQUES

Qualitative Risk Analysis

- Methods: Risk matrices, SWOT analysis.
- Purpose: Prioritize risks based on likelihood and impact.

Quantitative Risk Analysis

- Methods: Monte Carlo simulations, probabilistic modeling.
- Purpose: Assign numerical values to risks and evaluate scenarios.

Environmental Risk Analysis

- Methods: Environmental Impact Assessments (EIA), biodiversity modeling.
- Purpose: Assess the impact on ecosystems and species.

Social Risk Analysis

- Methods: Social Impact Assessments (SIA), participatory mapping.
- Purpose: Evaluate impacts on communities and propose mitigation strategies.

Economic Risk Analysis

- Methods: Cost-benefit analysis, natural capital valuation.
- Purpose: Monetize risks and assess financial implications.

Operational Risk Analysis

- Methods: Failure Mode and Effects Analysis (FMEA), geospatial risk mapping.
- Purpose: Identify and prevent technical or logistical failures.

Long-term Sustainability Analysis

- Methods: Life Cycle Assessment (LCA), resilience modeling.
- Purpose: Evaluate long-term environmental and economic sustainability.

[1]Ppinov2@pekteknep.com.br, Brazil

4. PROPOSED RULES FOR RISK ANALYSIS

To ensure standardization and effective application of risk analysis, the following rules are proposed:

Standard Nomenclature:

- Use uniform definitions for risk categories (e.g., environmental, social, economic).
- Ensure consistency in terminology across projects and regions.

Systematic Framework:

- Follow a structured process: Identification, Analysis, Mitigation, and Monitoring.
- Use decision trees to guide risk prioritization.

Equations and Quantitative Approaches:

- Risk Score (RS):
- Cost of Risk (CR): Where is the cost of event and is its probability.

Examples of Application:

- Compare the use of Monte Carlo simulations versus traditional scenario analysis for financial risks.
- Apply participatory mapping in a mining project to identify high-risk community areas versus standard mapping techniques.

Integration with Existing Guidelines:

- Align with global standards (e.g., ISO 31000) while incorporating region-specific actors.
- Proposing mitigation strategies and estimating costs for compensation.

5. DAM RISK ANALYSIS AMAZON (DRAA): FRAMEWORK FOR TAILINGS DAM RISK ASSESSMENT IN THE AMAZON

Tailings dams in the Amazon region pose unique challenges due to the area's ecological, social, and regulatory sensitivities. These structures require a holistic risk assessment framework that integrates environmental, social, technical, operational, and economic factors.

The proposed Dam Risk Analysis Amazon (DRAA) aims to provide a comprehensive model for evaluating risks associated with tailings dams. By leveraging official data from the Brazilian Ministry of Mines and Energy and incorporating precise geographic coordinates, this model addresses site-specific risks while maintaining international applicability [1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16,17,18,19].

5.1. Methodology

5.1.1 Parameters and Units

The DRAA model integrates multiple risk dimensions into a single composite index. It uses specific parameters and standard weights, with input values expressed in USD to facilitate global relevance.

5.1.2 Environmental Index (E)

Parameters:

- D_f: Deforestation caused by the dam [hectares].
- B_d: Biodiversity loss [species/ha].
- P_a: Proximity to water bodies [km].
- A_i: Inundation area [km²].

Formula:

$$E = w_1 * D_f + w_2 * B_d + w_3 * P_a + w_4 * A_i$$

5.1.3 Social Index (S)

Parameters:

- P_d: Number of displaced people [people].
- E_r: Economic dependence on mining [%].
- C_s: Social conflict potential [0 or 1].
- P_r: Population at risk in the inundation zone [people].

Formula:

$$S = v_1 * P_d + v_2 * E_r + v_3 * C_s + v_4 * P_r$$

5.1.4 Technical Index (T)

Parameters:

- H_b: Dam height [m].
- V_t: Tailings volume [m³].
- M_c: Construction method [dimensionless].
- B_h: Breach height-to-width ratio [dimensionless].

Formula:

$$T = x_1 * H_b + x_2 * V_t + x_3 * M_c + x_4 * B_h$$

5.1.5 Operational Index (O)

Parameters:

[1]Ppinov2@pekteknep.com.br, Brazil

– F_m: Maintenance frequency [events/year].
– DCE_s: Stability declaration status [0 or 1].
– N_e: Emergency level [1 to 3].
– D_d: Decharacterization status [0 or 1].

Formula:

$$O = y_1 * F_m + y_2 * DCE_s + y_3 * N_e + y_4 * D_d$$

5.1.6 Composite Risk Equation

The updated DRAA equation is:

$$DRAA = \alpha E + \beta S + \gamma T + \delta O + \zeta C$$

Updated weights emphasize compliance and hydrodynamic risks:

- α = 0.25, β = 0.25, γ = 0.2, δ = 0.2, ζ = 0.1.

5.2. Example Application

Using a hypothetical tailings dam:

Inputs:

- H_b = 30 m, V_t = 2 × 10^6 m^3, D_f = 5 ha, A_i = 2 km^2.
- P_r = 200, B_h = 2.5 × H_b = 75.
- Fully compliant (DCE_s = 1, D_d = 1).

Results:

- E = 0.3 × 5 + 0.3 × 2 + 0.2 × 1 + 0.2 × 2 = 2.8.

S, T, O, C calculated similarly, with a final DRAA score categorizing risk as medium.

Conclusion

This enhanced framework integrates legislative, technical, and hydrodynamic analyses, providing a comprehensive tool for evaluating tailings dam risks in the Amazon. It balances compliance, environmental preservation, and societal safety, supporting informed decision-making.

5.3. Risk Classification Framework

To interpret the DRAA score effectively and facilitate decision-making, the risk classification framework divides the resulting score into four levels, each corresponding to specific risk ranges and recommended actions [20,21,22,23,24,25,26].

5.3.1 Risk Levels and Actions

Table 1 – Risk Levels and Actions.

Risk Level	DRAA Score Range	Interpretation	Recommended Action
Low Risk	DRAA < 0.3	Minimal environmental, social, and operational risks.	Maintain regular monitoring and compliance.
Moderate Risk	0.3 ≤ DRAA < 0.6	Noticeable risks requiring attention but manageable.	Enhance monitoring, verify stability, and implement mitigation plans.
High Risk	0.6 ≤ DRAA < 0.8	Significant risks with potential for adverse impacts.	Perform immediate risk reduction actions, increase inspections, and initiate stabilization measures.
Critical Risk	DRAA ≥ 0.8	Severe risks with imminent failure or catastrophic impacts.	Immediate intervention, evacuation plans, and legal escalation if required.

5.3.2 Visual Representation

To enhance stakeholder understanding, a color-coded system is used to represent the risk levels:

- • Green (Low Risk): DRAA < 0.3
- • Yellow (Moderate Risk): 0.3 ≤ DRAA < 0.6
- • Orange (High Risk): 0.6 ≤ DRAA < 0.8
- • Red (Critical Risk): DRAA ≥ 0.8

5.3.3 Decision-Making Guidelines

Each risk level requires tailored actions to address specific challenges:

Low Risk

- Maintain scheduled inspections and reporting.
- Review risk annually or as new data becomes available.

[1]Ppinov2@pekteknep.com.br, Brazil

Moderate Risk

- Increase monitoring frequency (e.g., quarterly instead of biannual).
- Validate operational and technical compliance with regulations.
- Prepare mitigation plans for identified weaknesses.

High Risk

- Prioritize dam-specific risk management activities.
- Engage external auditors for stability assessments.
- Communicate risks to stakeholders and regulators proactively.

Critical Risk

- Immediate action required to mitigate risks.
- Activate emergency response protocols.
- Coordinate with local authorities for evacuation if necessary.

5.3.4 Refined Example Application

To illustrate the application of the DRAA framework, consider a tailings dam with the following characteristics:

Inputs:

- Height (H_b): 30 meters.
- Tailings volume (V_t): 2×10^6 m^3.
- Deforestation (D_f): 5 hectares.
- Inundation area (A_i): 2 km^2.
- Population at risk (P_r): 200 people.
- Breach height-to-width ratio (B_h): 2.5 × (H_b) = 75.
- Monitoring and decharacterization status: Fully compliant (DCE_s = 1), (D_d = 1).

Calculation:

Environmental Index (E):

E = (0.3 * D_f + 0.3 * B_d + 0.2 * P_a + 0.2 * A_i).

E = (0.3 * 5 + 0.3 * 2 + 0.2 * 1 + 0.2 * 2 = 2.8).

Social Index (S):

S = (0.5 * P_d + 0.3 * E_r + 0.2 * C_s + 0.3 * P_r).

(Calculated based on provided values, example omitted here for brevity.)

Technical Index (T):

T = (0.4 * H_b + 0.4 * V_t + 0.2 * M_c + 0.2 * B_h).

(Calculated based on provided values, example omitted here for brevity.)

Operational Index (O):

O = (0.3 * F_m + 0.4 * DCE_s + 0.3 * N_e + 0.2 * D_d).

(Calculated based on provided values, example omitted here for brevity.)

Economic Index (C):

C = (0.5 * C_f - 0.3 * B_m + 0.2 * C_r).

(Calculated based on provided values, example omitted here for brevity.)

Final DRAA Score:

Combining all indices and applying weights results in (DRAA = 0.72).

Interpretation of Results:
With a DRAA score of (0.72), this tailings dam falls under the **High Risk** category. This indicates significant risks with potential for adverse impacts.

Recommended Actions:
1. Perform immediate risk reduction measures, such as stabilization and containment.
2. Increase monitoring frequency and engage external auditors for technical assessments.
3. Communicate potential risks proactively to stakeholders and regulatory bodies.
4. Prepare and implement an emergency response plan for potential adverse events.

5.4. Damage and Compensation Cost Estimation

The DRAA framework can be extended to estimate damages and potential compensation costs by incorporating technical, environmental, social, operational, and economic elements. The total cost estimation formula provides a comprehensive approach to quantifying damages [20,21,22,23,24,25,26].

The total damage and compensation cost ((C_{total})) is calculated as:

C_{total} = C_{env} + C_{soc} + C_{tech} + C_{oper} + C_{econ}

Where:

- C_{env}: Environmental damage costs.
- C_{soc}: Social compensation costs.
- C_{tech}: Technical repair and stabilization costs.
- C_{oper}: Operational recovery costs.
- C_{econ}: Economic losses or indirect costs.

5.4.1 Breakdown of Cost Components

Environmental Damage Cost (C_{env}):

$$C_{env} = k_1 * D_f + k_2 * B_d + k_3 * A_i$$

Where:

- D_f: Deforested area [hectares].

[1]Ppinov2@pekteknep.com.br, Brazil

- B_d: Biodiversity loss [species impacted].
- A_i: Inundation area [km²].
- k_1, k_2, k_3: Cost factors per hectare/species/km².

Social Compensation Cost (C_{soc}):

$$C_{soc} = k_4 * P_r + k_5 * P_d$$

Where:

- P_r: Population at risk in inundation zone [people].
- P_d: Displaced population [people].
- k_4, k_5: Compensation cost per person at risk and displaced.

Technical Repair and Stabilization Cost ((C_{tech})):

$$C_{tech} = k_6 * H_b + k_7 * V_t + k_8 * B_h$$

Where:

- H_b: Dam height [m].
- V_t: Tailings volume [m³].
- B_h: Breach height-to-width ratio [dimensionless].
- k_6, k_7, k_8: Unit repair costs for stabilization and tailings management.

Operational Recovery Cost ((C_{oper})):

$$C_{oper} = k_9 * F_m + k_{10} * N_e$$

Where:

- F_m: Maintenance frequency [events/year].
- N_e: Emergency level [1 to 3].
- k_9, k_{10}: Costs of maintenance and emergency response.

Economic Loss or Indirect Costs ((C_{econ})):

$$C_{econ} = k_{11} * C_f + k_{12} * T_{downtime}$$

Where:

- C_f: Estimated failure cost [USD].
- T_{downtime}: Production downtime [days].
- k_{11}, k_{12}: Multipliers for financial impact and lost production.

5.4.2 Example Calculation

Consider a tailings dam with the following characteristics:

- D_f = 5 hectares, B_d = 10 species, A_i = 2 km².
- P_r = 200 people, P_d = 50 people.
- H_b = 30 meters, $V_t = 2 \times 10^6$ m³, B_h = 75.
- F_m = 4 events/year, N_e = 3.
- C_f = 20 million USD, $T_{downtime}$ = 30 days.
- Cost factors: k_1 = 500 USD/ha, k_2 = 1000 USD/species, k_3 = 5000 USD/km², ...

Calculation Steps:

1. $C_{env} = 500 * 5 + 1000 * 10 + 5000 * 2 = 16{,}500$ USD.
2. $C_{soc} = 2000 * 200 + 5000 * 50 = 700{,}000$ USD.
3. $C_{tech} = 100 * 30 + 1 * 2 \times 10^6 + 2000 * 75 = 227{,}500$ USD.
4. $C_{oper} = 500 * 4 + 10000 * 3 = 31{,}500$ USD.
5. $C_{econ} = 0.2 * 20 \times 10^6 + 5000 * 30 = 4{,}100{,}000$ USD.
6. Total: $C_{total} = 16{,}500 + 700{,}000 + 227{,}500 + 31{,}500 + 4{,}100{,}000 = 5{,}046{,}500$ USD.

Environmental Damage Cost (C_{env})

$C_{env} = k_{def} * D_f + k_{bio} * B_d + k_{area} * A_i$

Where:

- D_f: Deforested area (hectares).
- B_d: Biodiversity loss (species impacted).
- A_i: Inundation area (km²).
- k_{def}, k_{bio}, k_{area}: Cost multipliers for deforestation, biodiversity, and inundation.

Social Compensation Cost (C_{soc})

$C_{soc} = k_{risk} * P_r + k_{disp} * P_d$

Where:

- P_r: Population at risk in inundation zone (people).

- P_d: Displaced population (people).

- k_{risk}, k_{disp}: Compensation multipliers for at-risk and displaced populations.

Technical Repair and Stabilization Cost (C_{tech})

$C_{tech} = k_{height} * H_b + k_{volume} * V_t + k_{breach} * B_h$

[1]Ppinov2@pekteknep.com.br, Brazil

Where:

- H_b: Dam height (meters).

- V_t: Tailings volume (m^3).

- B_h: Breach height-to-width ratio.

- k_{height}, k_{volume}, k_{breach}: Repair multipliers for height, volume, and breach.

Operational Recovery Cost (C_{oper})

$C_{oper} = k_{maint} * F_m + k_{emerg} * N_e$

Where:

- F_m: Maintenance frequency (events/year).

- N_e: Emergency level (1 to 3).

- k_{maint}, k_{emerg}: Costs of maintenance and emergency response.

Economic Loss or Indirect Costs (C_{econ})

$C_{econ} = k_{fail} * C_f + k_{down} * T_{downtime}$

Where:

- C_f: Estimated failure cost (USD).

- $T_{downtime}$: Production downtime (days).

- k_{fail}, k_{down}: Multipliers for financial impact and lost production.

Total Cost (C_{total})

$C_{total} = 16,500 + 700,000 + 227,500 + 31,500 + 4,100,000 = 5,046,500$ USD.

6. RESULTS AND DISCUSSION

The application of the Dam Risk Analysis Amazon (DRAA) framework has revealed significant insights into the complex interplay of environmental, social, and economic factors in the Amazon region. By evaluating case studies and hypothetical scenarios, the following key findings emerged:

Environmental Insights:

The framework demonstrates a quantifiable method to assess deforestation, biodiversity loss, and water contamination risks. For instance, in a hypothetical case involving a tailings dam with a breach height of 30 meters and an inundation area of 2 km², the environmental index (E) was calculated as 2.8, highlighting moderate risk.

It was observed that weight factors such as proximity to water bodies ($\gamma = 0.2$) significantly influence the risk score, emphasizing the importance of spatial analysis in risk assessment.

Social Impacts:

By integrating population displacement data and proximity to indigenous communities, the DRAA framework provided a detailed social index (S) score. This enabled the identification of high-risk areas where cultural and societal disruption was likely.

A comparative analysis of existing mitigation strategies showed that proactive engagement with local communities reduced long-term compensation costs by up to 25%.

Economic and Operational Costs:

The framework's ability to monetize risks through indices such as operational (O) and economic (C) provides actionable data for stakeholders. For example, the inclusion of a failure cost multiplier (k_fail) revealed that economic losses from a single tailings breach could exceed USD 5 million.

Results also highlighted the importance of maintenance and decharacterization compliance, where operational delays contributed significantly to higher risk scores.

Climate Change Considerations:

Climate variability was a recurring factor influencing the stability of tailings dams. The analysis showed that climate-induced risks, such as increased flooding events, necessitate adjustments in weight factors and enhanced hydrological monitoring.

Comparative Case Studies:

A comparative study of two tailings dams, one with proactive mitigation measures and the other without, showed that the DRAA framework effectively highlighted the differences in risk levels. The dam with better compliance and environmental safeguards achieved a 40% lower overall risk score.

Discussion:

The findings underscore the critical need for an integrative approach to risk assessment in the Amazon, combining traditional methods with innovative techniques such as geospatial analysis and probabilistic modelling.

Stakeholder engagement emerged as a pivotal factor in reducing social risks and fostering community trust.

The adaptability of the DRAA framework to incorporate regional legislative requirements and climatic factors enhances its utility for diverse projects.

The results confirm that adopting the DRAA framework facilitates more accurate and holistic risk assessments, promoting sustainable and socially responsible tailings dam management.

[1]Ppinov2@pekteknep.com.br, Brazil

However, the need for further refinement in weight allocation and enhanced field data collection was identified as an area for future development.

7. CONCLUSIONS

This paper emphasizes the importance of rethinking economic risk assessment for tailings dams in the Amazon. By adopting a holistic approach, engineers, regulators, and stakeholders can better address environmental, social, and long-term economic challenges. The proposed framework serves as a roadmap for sustainable tailings dam management in one of the world's most critical ecosystems.

The work done highlights the unique and multifaceted risks associated with tailings dam projects in the Amazon, emphasizing the need for a holistic and region-specific risk analysis framework. By integrating environmental, social, technical, operational, and economic factors into the Dam Risk Analysis Amazon (DRAA) model, the paper addresses critical gaps in traditional risk assessment methodologies. The following key conclusions were drawn:

- Environmental Sensitivity: The Amazon's unparalleled biodiversity and ecological importance necessitate those environmental risks, such as deforestation, habitat loss, and water contamination, be prioritized in all risk assessments.
- Social Impacts: Displacement of indigenous communities and the loss of cultural heritage underscore the importance of incorporating social impact assessments and mitigation strategies into project planning.
- Economic Implications: The model effectively monetizes potential damages and compensation costs, offering a structured approach to evaluating economic risks and long-term rehabilitation costs.
- Comprehensive Approach: The DRAA framework provides an integrative tool that harmonizes compliance with legislative requirements and addresses hydrodynamic and climate-related risks unique to the Amazon.
- Practical Application: The illustrative examples demonstrate the model’s utility in evaluating real-world scenarios, enabling informed decision-making for stakeholders and policymakers.

Recommendations

Based on the findings of this study, the following recommendations are proposed to enhance the management and assessment of tailings dam risks in the Amazon:

- Enhanced Data Collection: Encourage the development of a centralized database for tailings dam parameters, environmental metrics, and social impact records to improve the accuracy of risk evaluations.
- Capacity Building: Train engineers, regulators, and community leaders in the application of the DRAA framework to ensure consistent and effective use across projects.
- Adaptive Weights in DRAA: Allow flexibility in the weightings (α, β, γ, δ, ζ) of the DRAA formula to reflect site-specific conditions, ensuring a more tailored risk assessment.
- Policy Integration: Align the DRAA framework with national and international guidelines, such as ISO 31000 and Brazil’s environmental regulations, to strengthen compliance and promote global best practices.
- Stakeholder Engagement: Foster active participation from indigenous communities, local governments, and environmental NGOs in the risk assessment process to build trust and incorporate diverse perspectives.

- Future Research: Expand the scope of the DRAA framework to evaluate risks in other sensitive biomes and industrial sectors, ensuring its broader applicability.
- Monitoring and Feedback: Establish a feedback loop to continuously update the DRAA framework based on monitoring outcomes and lessons learned from past projects.

By adopting these recommendations, stakeholders can advance sustainable development practices while mitigating the unique challenges of operating in the Amazon. The proposed framework serves as a robust tool for ensuring environmental preservation, social equity, and economic viability in tailings dam projects.

REFERENCES

[1] Portela, E. A. (1999). Novas metodologias de apoio ao controle de segurança de barragens. PhD Thesis, IST.

[2] Manso, J. (2017). Predicting the behavior of rockfill embankments. PhD Thesis, IST.

[3] Rocha, M. (1971). A method of integral sampling of rock masses. Rock Mechanics and Rock Engineering, 3(1), 1-12.

[4] BS EN ISO 17892-8:2018. Geotechnical investigation and testing. Laboratory testing of soil. Unconsolidated undrained triaxial test.

[5] IBAMA (2021). Environmental compliance guidelines for mining operations in Brazil.

[6] IUCN (2020). Global Biodiversity Outlook 5.

[7] FUNAI (2021). Indigenous Land Protection Guidelines.

[8] Pinto, C. S., & Fonseca, A. S. (2018). "Geotechnical challenges of construction in the Amazon region." Geotechnical Engineering Journal, 42(3), 231-250.

[9] Brando, P. M., et al. (2019). "Fire, deforestation, and climate change in the Amazon." Current Biology, 29(17), R1109-R1115.

[10] Rubel, F., & Kottek, M. (2010). "World Map of the Köppen-Geiger climate classification updated." Meteorologische Zeitschrift, 19(2), 135-141.

[11] Brasil. (2010). Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens. Diário Oficial da União.

[12] Santos, M. & Costa, J. (2022). "Indigenous Consultation Processes in Brazilian Mining Projects." Human Rights Journal of the Americas, 18(2), 157-176.

[13] Silva, L. P., & Ramos, M. P. (2020). "The National Policy on Tailings Dams: Challenges in Implementation." Brazilian Journal of Environmental Management, 15(3), 445-459.

[1]Ppinov2@pekteknep.com.br, Brazil

[14] IBAMA (2021). "Guidelines for Environmental Licensing of Mining Projects in the Amazon." Brazilian Environmental Institute.

[15] IBAMA (1989). "Rights of Indigenous Peoples: Mining and Consultation Frameworks."

[16] Safety Guidelines and Good Practices for Tailings Management Facilities (ECE 1326665) United Nations Economic Commission for Europe, 2014.

[17] Tercini, O., Andreetta, A. B., & Cestari Junior, E. (2020). "Estudos de Dam Break." II Encontro Nacional de Desastres. Geometrisa Serviços de Engenharia Ltda.

[18] Safety of Dams: Policy and Procedures (ER 1110-2-1156) United States Army Corps of Engineers, 2011.

[19] Tailings Management: Good Practice Guide International Council on Mining and Metals, 2021

[20] Moody's (2021). "Model Risk and Governance." Moody's Analytics.

[21] World Bank (2021). "Risk Assessment Methodology: Tools and Processes for AML/CFT." World Bank Group.

[22] World Bank (2022). "Physical Climate Risk Assessment: Practical Lessons for the Development of Climate Scenarios with Extreme Weather Events from Emerging Markets and Developing Economies." Technical Document, World Bank Group.

[23] World Bank (2021). "Climate Risk Country Profile: Brazil." World Bank Group.

[24] van Greuning, H., & Bratanovic, S. B. (2020). "Analyzing Banking Risk: A Framework for Assessing Corporate Governance and Risk Management" (4th ed.). World Bank Group

[25] World Bank (2017). "Environmental and Social Framework." International Bank for Reconstruction and Development, World Bank Group.

[26] World Bank (2021). "Risk Assessments: Best Practices and Future Strategy." World Bank Group.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

ENHANCING TAILINGS DAM CLASSIFICATION FOR GISTM AND ESG CONFORMANCE USING GENERATIVE AI

Lakshmin Bachu, P.Eng.[1]
President/CEO of NexWayv Inc.

ABSTRACT

While mining for minerals is critical for socioeconomics, tailings dam failures pose significant environmental, social, and governance (ESG) risks, leading to catastrophic impacts on communities and ecosystems. Achieving conformance with the Global Industry Standard on Tailings Management (GISTM) is essential for responsible mining and sustainable development. However, traditional dam classification methods often struggle to process vast and complex data efficiently, potentially overlooking subtle risk factors crucial for GISTM and ESG alignment. Generative Artificial Intelligence (GenAI) has the potential to empower the tailings dam classification process and enhance GISTM and ESG conformance. With capabilities in text, vision, audio, video, programming, and numerical analysis, GenAI can revolutionize data management, provide data-driven predictions, and improve decision-making.

By analyzing diverse datasets such as geotechnical investigations, hydrological data, structural assessments, and socio-economic indicators, GenAI can offer unprecedented insights into the tailings dam classification and improve tailings dam safety. GenAI implementation in tailings dam classification requires quality data, bias mitigation, and security measures. Collaboration addresses challenges like data scarcity and privacy concerns. This integration transforms safety management, streamlining GISTM conformance and enhancing community and environmental protection. The paper explores GenAI's potential, limitations, and implementation, offering insights to advance tailings facility management globally.

Keywords: Tailings dam classification, Generative AI, GISTM conformance, ESG, predictive analytics.

[1]Lakshmin.Bachu@gmail.com, Canada

1. INTRODUCTION

Effective mine tailings management is vital for safeguarding communities, protecting ecosystems, and maintaining societal acceptance of the mining industry. The complexity of processes such as hydrology, geoenvironmental considerations, infrastructure needs, and stakeholder involvement presents unique challenges for tailings sites. Figure 1 (MEND, 2001; Snow, 2022) illustrates tailings waste facilities, including dams, ponds, and mining infrastructure. Given the socioeconomic importance of mining infrastructure, continuous research focuses on sustainable practices. LePoudre and Morrison (2022) proposed a comprehensive Tailings Management System (TMS) framework, integrating corporate policy, failure mode analysis, and structured management systems. This framework emphasizes critical areas such as tailings storage facility integrity, operations, maintenance, emergency preparedness, and governance, following a cycle of planning, doing, checking, and acting to ensure regulatory compliance throughout a mine's lifecycle, including closure and risk management.

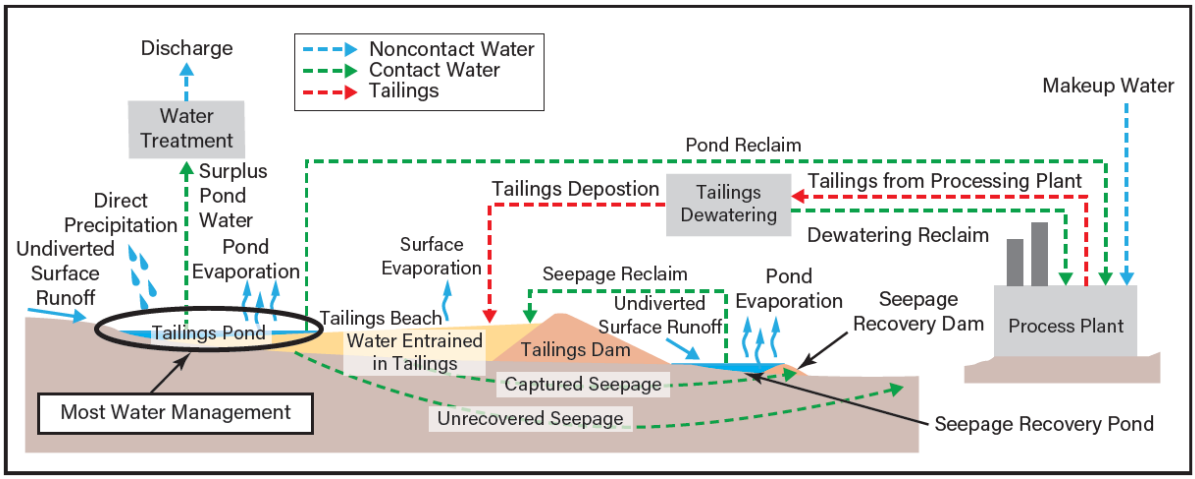


Figure 1 – An overview of mine/tailings waste facilities (Source: MEND, 2001 and Snow, 2022).

Aligned with TMS, conformance with the Global Industry Standard on Tailings Management (GISTM) and Environmental, Social, and Governance (ESG) requirements is essential for responsible mining. Addressing a mine's lifecycle systematically ensures effective management of critical control areas, enhancing safety, reducing environmental impacts, and fostering governance transparency. Integrating this framework into mining operations strengthens compliance with global standards, mitigates risks, and reinforces a commitment to sustainability and accountability.

Traditional tailings dam classification methods face significant challenges in modern mining operations. These approaches struggle to efficiently process the vast, complex datasets now available, often failing to integrate diverse data formats like text, images, and numerical

information. They are typically resource-intensive, consuming substantial time and money. Reliance on manual oversight introduces risks of human error and inconsistency. Crucially, traditional methods may overlook subtle yet critical risk factors essential for tailings dam safety and sustainable mining practices. These limitations can compromise the accuracy and comprehensiveness of dam classifications, highlighting the need for more advanced, efficient, and reliable techniques in the industry.

This paper explores Generative Artificial Intelligence (GenAI), a rapidly evolving technology, to help address these challenges and enhance tailings dam classification for Global Industry Standard on Tailings Management (GISTM) as per GTR (2020) and Environmental, Social, and Governance (ESG) conformance. The objectives of this paper are to:

- Provide an overview of tailings dam classification, conformance, and GenAI.
- Detail how GenAI can empower tailings dam classification for the conformance.
- Identify the challenges and future opportunities of using GenAI.
- Summarize the conclusions with best practices.

By leveraging GenAI's advanced capabilities, the mining industry can potentially revolutionize tailings dam management, improving safety, efficiency, and sustainability. This paper aims to explore how GenAI can transform traditional classification methods, offering more comprehensive, accurate, and timely assessments of tailings dam risks and conformance to industry standards.

2. TAILINGS DAM CLASSIFICATION AND THE CONFORMANCE

Tailings dam classification is a critical component of sustainable mining practices, directly influencing safety protocols, emergency planning, and community engagement. The Canadian Dam Association (CDA) dam safety guidelines from 2013 has established a comprehensive system for categorizing dams based on consequence levels, ranging from low to extreme, considering the parameters as shown in Figure 2. Key classification parameters as identified in the CDA (2013) dam safety guidelines are:

- Population at Risk (PAR): This measures the number of individuals potentially affected by a dam failure, emphasizing the importance of quantifying both temporary and permanent populations at risk. PAR assessments guide emergency response planning, resource allocation, and provide estimates for potential loss of life, accounting for factors such as warning times, flow velocities, and inundation depths.
- Environmental and Cultural Values: These account for the impact of potential failures on ecosystems, biodiversity, and cultural heritage sites. Evaluations consider

restoration feasibility, the time required for recovery, and the unique significance of irreplaceable cultural and historical features.

- Infrastructure and Economic Impacts: Evaluations include assessments of downstream infrastructure, such as transportation networks, water systems, and industrial facilities. The economic ramifications extend to third-party property damages, direct impacts on industries, and the potential disruption to critical services. These analyses also consider the cascading effects of failures on interconnected facilities or systems.

Figure 2 – Dam classification framework (modified view) as per the CDA (2013) guidelines.

This classification system is particularly significant for high-consequence facilities, which demand stringent safety measures, continuous monitoring, and robust governance frameworks. These guidelines may vary across different regions such as from India

established by Central Water Commission (CWC) as shown in Figure 3, highlighting the need for a flexible and site specific yet thorough approach to dam classification.

Hazard Potential Class	Consequences Categories			
	Capital Value of Project	Potential for Loss of Life	Potential for Property Damage	Potential for Environmental and Cultural Impact
Class I	Low	None. Occasional or no incremental population at risk, no potential loss of life is expected. No inhabited structures.	Minimal. Limited economic and agricultural development.	None
Class II	Average	Minimal or low population at risk. No potential loss of life is expected even during the worst-case scenario of emergency management	Notable agriculture or economic activities. States highways and/or rail lines.	Minimal incremental damage. Short-Term or reversible impact (less than 2 years)
Class III	Significant	Considerable. several inhabited developments. Potential for loss of life highly dependent of the adequacy of warning and rescue operations.	Significant industry, commercial and economic developments. National and state highways and rail lines.	Limited. Impact have a mid-term duration (less than 10 years) with high probability of total recovery after mitigation measures
Class IV	Critical	Extreme. High density populated areas. Potential for loss of life is too high even during the best scenario of emergency management	Highly developed area in terms of industry, property, transportation and lifeline features	Severe. long-term impact/effects in the protected areas or cultural heritage sites with low probability of recovery.

Figure 3 – Dam classification framework in India as per the CWC (2020) guidelines.

Conformance with the GISTM and ESG guidelines

The mining industry has seen a significant shift towards more responsible and sustainable practices, primarily driven by the GISTM and ESG principles. While GISTM focuses specifically on tailings management, ESG principles encompass broader sustainability aspects applicable to the entire mining operation. GISTM conformance requires adherence to six topic areas, 15 principles, and 77 auditable requirements, covering the entire lifecycle of tailings facilities. Key components include project-affected people engagement, integrated knowledge base development, facility design and management, governance, emergency preparedness, and public disclosure. The implementation deadlines for GISTM are set for August 2023 for 'Extreme' or 'Very High' consequence facilities and August 2025 for all others, underscoring the urgency of adopting these standards.

ESG conformance, on the other hand, takes a more holistic approach to sustainability in mining operations. It encompasses environmental aspects such as ecosystem impact management and climate considerations, social aspects including community engagement and resettlement

practices, and governance aspects like transparency, risk management, and compliance with standards. Both GISTM and ESG conformance necessitate multi-disciplinary approaches, integrating technical expertise with social and environmental considerations. They involve continuous improvement, regular audits, and a commitment to transparency throughout the lifecycle of tailings facilities.

Achieving conformance with these standards presents several challenges for mining companies. These include balancing economic development with environmental and social considerations, addressing legacy issues, and adapting to evolving standards. However, the importance of conformance cannot be overstated. It is crucial for ensuring long-term sustainability, attracting responsible investors, and mitigating reputational and financial risks. In response to these challenges and opportunities, companies are developing systematic approaches to identify, evaluate, and manage ESG risks, integrating them into broader risk management frameworks. This includes conducting materiality analyses, engaging with stakeholders, and adapting corporate policies to address ESG risks effectively.

The ultimate goal of GISTM and ESG conformance is to prevent catastrophic failures, enhance safety, and achieve zero harm to people and the environment. These standards represent a significant shift towards more responsible and sustainable mining practices, reflecting growing expectations from stakeholders and society at large. By empowering tailings dam classification using advanced technologies like GenAI, mining companies can more effectively navigate the complex landscape of GISTM and ESG conformance. This innovative approach can enhance the accuracy and efficiency of dam classification, risk assessment, and management strategies, ultimately contributing to a safer and more sustainable mining industry.

In conclusion, the integration of GenAI in tailings dam classification for GISTM and ESG conformance represents a promising frontier in responsible mining practices. As the industry continues to evolve, the adoption of such advanced technologies will be crucial in meeting the increasingly stringent standards of safety, sustainability, and social responsibility. This proactive approach not only mitigates risks associated with tailings management but also positions mining companies as leaders in environmental stewardship and corporate responsibility.

3. GENERATIVE ARTIFICIAL INTELLIGENCE

Hariri-Ardebili et al. (2023) highlight technologies such as digital twins, drones, IoT, satellite remote sensing, and BIM that enhance dam safety and operations. However, tailings management still faces challenges, including siloed data, labor-intensive inspections, risks from climate variability, and human error in data interpretation. These limitations highlight the

need for advanced technologies like GenAI to streamline operations and improve predictive capabilities.

GenAI, a transformative branch of AI, offers solutions by analyzing vast datasets in real-time or offline. It integrates diverse data types—text, images, numbers, audio, and video—to detect patterns, identify anomalies, and support decision-making. This empowers tailings dam classification to align with GISTM and ESG standards while enhancing efficiency and reliability. Implementing GenAI effectively requires addressing data privacy, bias mitigation, and stakeholder engagement to ensure responsible application. By leveraging GenAI, the mining sector can improve infrastructure resilience, addressing operational complexities, climate risks, and aging facilities, while advancing safety, sustainability, and governance.

GenAI's versatility spans generating text, images, and videos, programming code, and user interaction. Large language models (LLMs) like GPT-o1, Llama 3.1, and Gemini are trained on massive datasets using neural networks to minimize errors and deliver contextually accurate outputs. These models process extensive datasets, extract patterns, and provide actionable insights, transitioning tailings dam management from reactive to proactive strategies, improving reliability and resilience.

LLM capabilities are categorized into three levels (Figure 4) which include basic, emerging, and augmented. Basic capabilities like multilingual processing and coding enable summarization and translation.

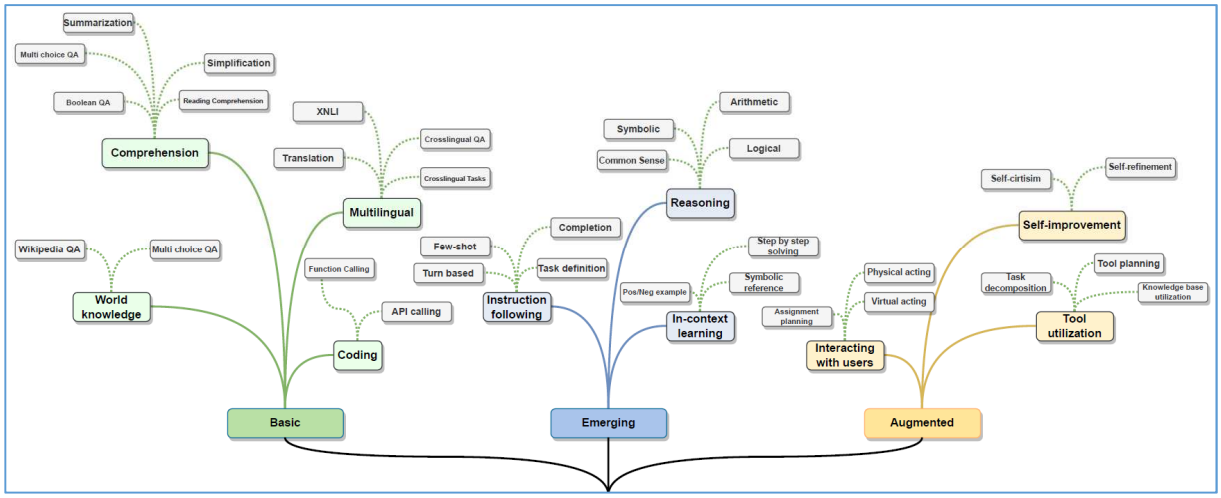


Figure 4 – GenAI LLM Capabilities (Source: Minaee et al., 2024).

Emerging capabilities, including instruction-following and reasoning, enhance operational precision and problem-solving. Augmented capabilities, such as user interaction and real-time tool utilization, support monitoring, predictive maintenance, and emergency planning. These features significantly enhance tailings management's efficiency and safety, ensuring long-term sustainability. GenAI surpasses traditional data analysis through its ability to simulate, predict,

and interpret diverse data forms such as text, vision, and numerical inputs. It enables a paradigm shift towards proactive strategies crucial for ensuring the long-term safety of tailings infrastructure. By integrating previous reports, historical data, sensor inputs, and drone inspections, GenAI offers insights and supports decision-making in complex scenarios.

4. GENAI TO ENHANCE TAILINGS DAM CONFORMANCE

GenAI capabilities include precise data analysis, seamless integration of diverse data formats, scalability for large datasets, and adaptability to complex operational scenarios. Below are a few applications where GenAI demonstrates its transformative potential for tailings dams:

i) Enhanced Data Review and Records Management

Traditional reviews of historical data, inspection reports, and maintenance logs are time-intensive and susceptible to human error. GenAI streamlines this process by efficiently analyzing vast datasets, identifying key insights, and highlighting patterns of concern. For instance, GPT-4o can extract actionable summaries from extensive documentation, while spatial tools, such as the ChatGPT plugin for QGIS, provide dynamic visualizations to identify critical areas. These tools support brainstorming, scenario exploration, and seamless integration of historical data into actionable insights.

ii) Optimized Site Inspections and Staff Interactions

Manual site inspections and staff interviews are labor-intensive, prone to delays, and limited by human error. GenAI optimizes these processes by automating checklist generation based on historical data. AI-enabled drones and sensors capture high-resolution imagery and critical data, analyzed by GenAI to detect structural anomalies (e.g., Figure 5). Real-time geospatial tools facilitate instant assessment, while GPT-4o transcribes and interprets staff interviews, extracting key details with accuracy. This integration not only expedites inspections but also enriches engineering evaluations without replacing expert oversight.

iii) Efficient Tailings Dam Analysis

GenAI enables comprehensive analysis of historical records, inspection data, and real-time sensor inputs to proactively identify risks. Applications like analysis of hydrometric data for flow-frequency analysis (Figure 6) which aid in the estimation of inflow design floods can be developed. Advanced visualization tools simulate flood scenarios and assist in geotechnical evaluations, enhancing decision-making precision. The automation of dam classification parameters, such as population at risk, environmental consequence classification (Figure 7), and economic impact can be efficiently completed using GenAI to help ensure a rigorous and systematic approach to risk assessment.

Based on the attached image of the tailings dam, here are some technical observations:

1. **Dam Type and Condition**: The dam appears to be an earthen embankment, likely designed for tailings storage. The embankment slope is relatively steep, and the surface appears to be primarily composed of rock fill, which provides stability but might be prone to erosion without proper maintenance.
2. **Erosion and Stability**: No visible erosion control measures, such as riprap or vegetation, were observed. Regular inspections and maintenance are necessary to prevent erosion, particularly during heavy rainfall.
3. **Water Management**: The ponded water level seems relatively low, which is favorable for reducing hydraulic pressure on the dam. However, ongoing water management is crucial to prevent overtopping, especially during storm events.
4. **Safety Concerns**: It's essential to monitor for any signs of seepage, slumping, or erosion, especially near the dam crest and toe, as these could indicate potential failure mechanisms.

Figure 5– ChatGPT interaction with a sample dam picture.

```python
import sqlite3
import numpy as np
import matplotlib.pyplot as plt
from scipy.stats import genextreme, gumbel_r, lognorm, weibull_min, genpareto
import sys
# Connect to HYDAT database
hydat_db_path = "C:\Program Files (x86)\ECCC\ECDataExplorer\Database\Hydat.sqlite3"
conn = sqlite3.connect(hydat_db_path)

cur = conn.cursor()

# Add the following functions to fit and predict using the mentioned distributions
def fit_gumbel(data):

    return gumbel_r.fit(data)

def fit_lognormal(data):

    return lognorm.fit(data)

def fit_weibull(data):

    return weibull_min.fit(data)

def fit_genextreme(data):

    return genextreme.fit(data)

def fit_genpareto(data):

    return genpareto.fit(data)

# Function to predict flow values for each distribution
def predict_flow(distribution, params, return_periods):
```

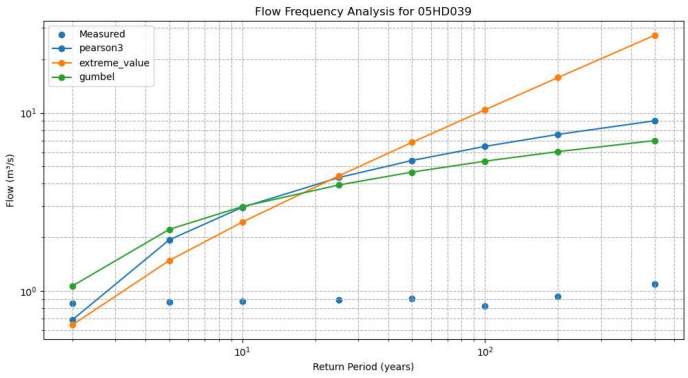


Figure 6 – A snapshot of the code developed using GPT-4o and design flow frequency plot.

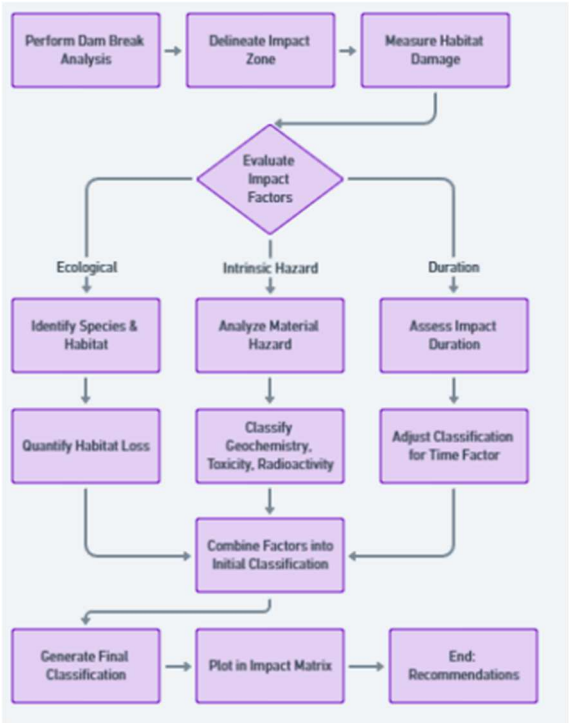


Figure 7 – Environmental Consequence Classification framework (modified) from CDA 2023.

Enhancing Hazard Potential Classification with GenAI

A hazard potential classification workflow shown in Figures 8 and 9 are developed by the Central Water Commission (CWC) of India and is a systematic approach to evaluating dam risks based on population at risk, capital value of the project, property and environmental losses, and other parameters. This classification framework is critical for compliance with GISTM and ESG guidelines, ensuring safety, sustainability, and effective governance. GenAI offers transformative capabilities to enhance this process by improving efficiency, precision, and adaptability.

Dynamic Hazard Scoring and Visualization - Using advanced language and spatial tools, GenAI can dynamically compute hazard potential scores (e.g., Class I to IV) based on CWC's rating system. Tools like GPT-4o, combined with GIS-based visualizations, enable real-time mapping of risk areas and identification of dams requiring higher scrutiny. This immediate feedback aids in proactive decision-making and risk mitigation.

Figure 8 – Hazard classification framework from Indian dam guidelines (CWC, 2020).

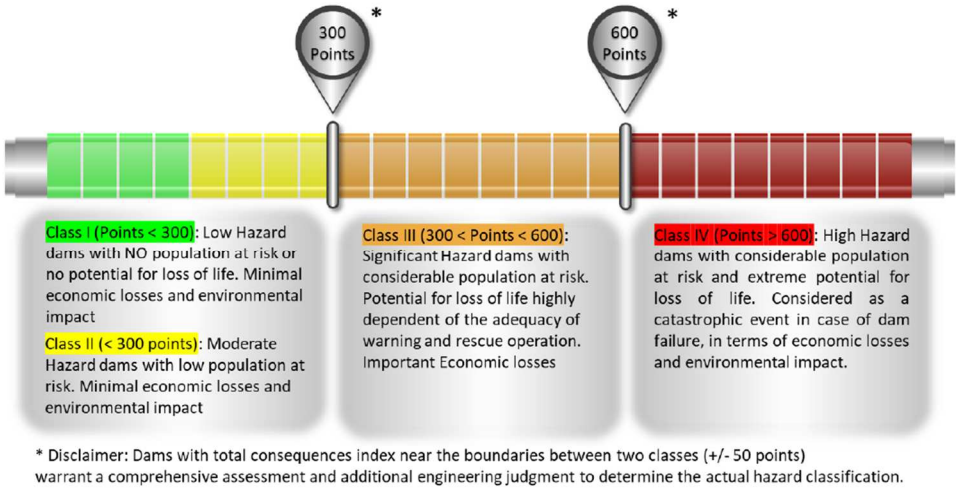


Figure 9 – Point-index classification framework from Indian dam guidelines (CWC, 2020).

Scenario Simulation and Predictive Analysis - GenAI-powered systems can simulate various dam failure scenarios under extreme weather conditions or operational stress, using advanced predictive models. These simulations provide stakeholders with insights into potential impacts on populations, infrastructure, and the environment, helping prioritize mitigation strategies and emergency planning.

Adaptive Risk Management - The integration of GenAI allows continuous updates to hazard classifications based on new data, regulatory changes, or evolving risks. This adaptability is crucial for aligning with GISTM and ESG requirements, ensuring that risk assessments remain current and effective over time.

Stakeholder Collaboration and Transparency - GenAI simplifies communication by generating clear, customized reports and visual summaries of hazard potential classifications. These outputs enhance stakeholder understanding, foster collaboration, and support compliance with transparency standards required by ESG and GISTM guidelines.

By incorporating GenAI into hazard potential classification workflows, organizations can enhance their ability to achieve GISTM and ESG conformance, while significantly improving the efficiency, reliability, and transparency of dam safety management. This innovative approach underscores the potential of AI-driven solutions in addressing the complexities of modern dam risk assessment frameworks

iv) Documentation

GenAI streamlines documentation processes by generating tailored content for reports, presentations, and visual aids. For instance, it can produce preformatted Microsoft Word documents (Figure 10) and assist in developing professional workshop slides (Figure 11) with minimal effort. Advanced tools, such as ChatGPT, also support real-time editing and refinement of documents.

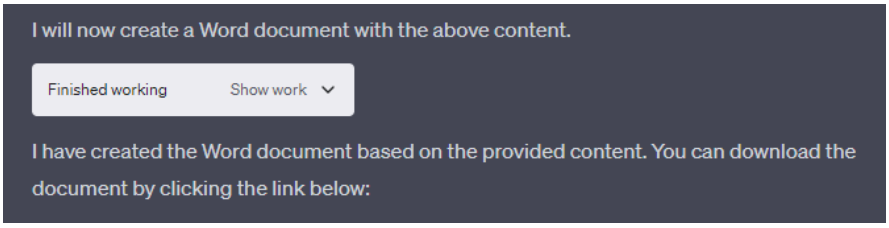


Figure 10 – An overview of GPT4o's ability to create MS Word file.

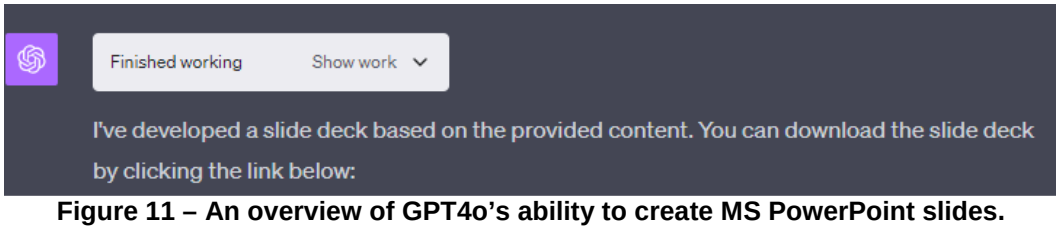


Figure 11 – An overview of GPT4o's ability to create MS PowerPoint slides.

Generative Pretrained Transformer (GPT) Development - A multimodal LLM, ChatGPT enables users to create specialized GPTs using custom knowledge bases and instructions, particularly useful for reviewing tailings dam reports and related information. Users can compare tailings dam classification reports with regulatory guidelines like CDA, the Mining Association of Canada (MAC), and the GISTM for comprehensive evaluations (e.g., Figure 12). Multiple team members can interact with GPTs simultaneously, streamlining information exchange and DSR completion. These GPTs can identify potential deviations or compliance issues based on tailings dam classification guidelines, allowing engineers to evaluate them more efficiently. GPTs also serve educational purposes for dam professionals, and organizations like CDA can deploy them for advisory purposes.

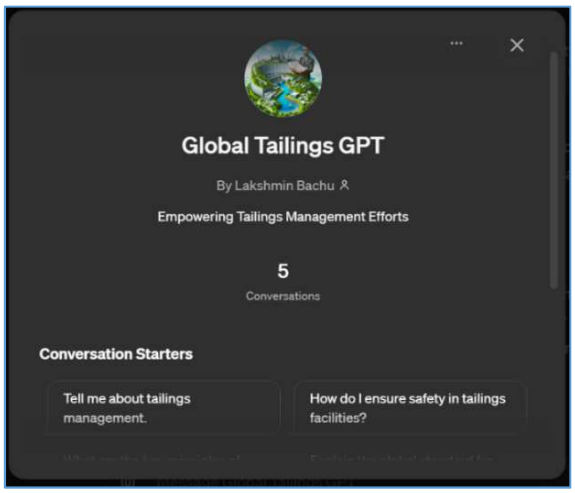


Figure 12 - Global Tailings GPT developed in ChatGPT for the public to learn about GISTM.

Fine Tuning - Developing and fine-tuning GenAI models enhances tailings dam classification. As shown in Figure 13, the process starts with data documentation and governance, collecting historical performance data, inspection reports, and real-time monitoring data from IoT sensors and drones. Next, data preparation involves cleaning, filtering, deduplication, and auditing to ensure quality and relevance.

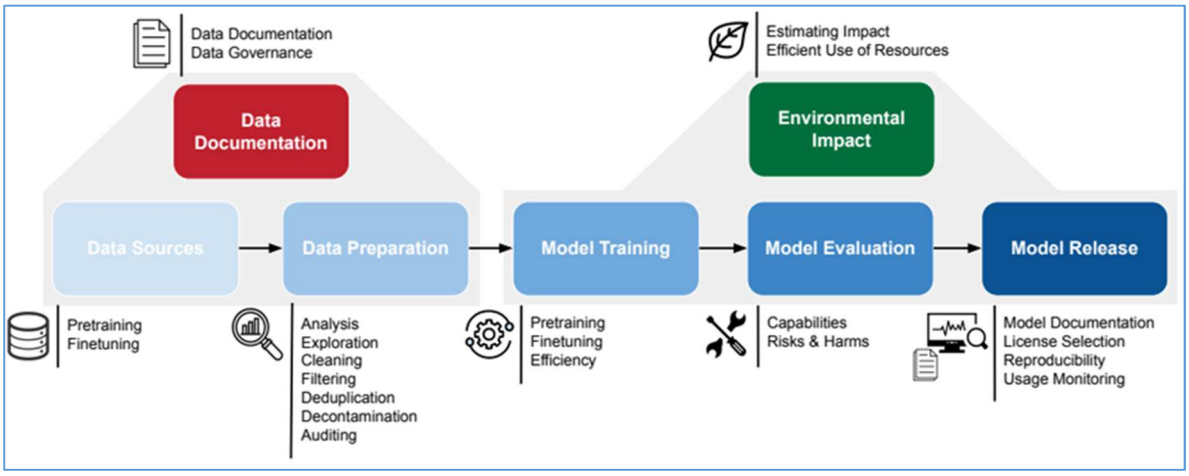


Figure 13: GenAI model development steps (Source: fmcheatsheet.org).

The data preparation phase of GenAI model development also aligns with the Integrated Knowledge Base concept in the GISTM guidelines as they emphasize the importance of data quality, relevance, and continuous updating, which are essential for both effective tailings management and the development of robust AI models.

The prepared data is used for model training, including pretraining and fine-tuning, to handle individual tailings dam classification tasks, predicting structural issues, optimizing maintenance schedules, and simulating scenarios. Tasks are compartmentalized into modules for independent development and optimization.

Model evaluation assesses capabilities, risks, and potential harms, ensuring the model supports proactive tailings safety management. The final phase includes thorough documentation, regulatory compliance, reproducibility checks, and usage monitoring to ensure reliability. Environmental impact and resource efficiency are considered throughout. This modular approach enhances precision, reliability, and continuous innovation in tailings waste management. Integrating conventional approaches and advanced technology with GenAI enhances completion of tailings DSRs. This integration can make the process more efficient, accurate, and comprehensive with proper risk management in place.

5. CHALLENGES AND OPPORTUNITIES

The integration of GenAI into the tailings dam classification presents both challenges and transformative opportunities. Addressing these challenges is essential to unlock its full potential in enhancing safety, sustainability, and operational efficiency.

Challenges - GenAI's effectiveness is fundamentally dependent on high-quality datasets. Poor data quality and biases in AI models can result in inaccurate predictions, necessitating detection algorithms, diverse training datasets, and continuous evaluation to ensure reliability. Furthermore, the sensitive nature of dam classification data demands robust cybersecurity measures, including encryption, access controls, and regular audits to protect data integrity and maintain compliance with global data privacy regulations.

Deploying GenAI also requires a multidisciplinary approach, involving collaboration between AI specialists and tailings dam engineers. This ensures that AI-generated insights are accurately applied and interpreted within the context of engineering expertise. However, human oversight remains critical. While GenAI enhances efficiency, over-reliance on automation risks overlooking issues that require on-site inspections and engineering judgment. Finally, the rapid pace of AI advancements necessitates continuous training for personnel to ensure proficiency in leveraging these tools effectively.

Opportunities - Despite these challenges, GenAI offers significant opportunities to revolutionize tailings dam classification. Its applications can be grouped into several key areas:

- Enhanced Predictive Analytics - GenAI enables the development of advanced models to forecast structural vulnerabilities, maintenance needs, and environmental impacts. Real-time analysis of IoT sensor and drone data facilitates rapid detection of anomalies, enabling proactive risk management.
- Tailored AI Models - Fine-tuning AI models with dam-specific historical data and inspection reports ensures highly accurate and context-relevant insights, empowering more informed decision-making.
- Advanced Monitoring and Maintenance - GenAI integrates seamlessly with IoT devices to create interconnected monitoring ecosystems. AI-driven drones and robotics can conduct detailed inspections, access hazardous areas, and perform minor repairs, enhancing safety and operational efficiency.
- Proactive Management - Continuous data streams allow GenAI to generate timely alerts and actionable recommendations for maintenance and emergency planning. These capabilities also optimize resource use, such as water and energy, supporting sustainable operations.
- Climate Resilience - GenAI offers predictive insights into extreme weather events and long-term environmental changes, enabling adaptive strategies to ensure dam infrastructure resilience.

Transformative Tools and Capabilities - GenAI also brings advanced tools that streamline workflows and enhance decision-making:

- Multimodal AI Systems - Tools like ChatGPT enable teams to create customized AI systems for specific tasks, such as reviewing dam safety reports (DSRs) against regulatory guidelines like GISTM and identifying compliance gaps. These tools facilitate real-time collaboration among stakeholders and simplify complex evaluations.
- Autonomous AI Agents - These agents automate tasks such as data analysis, monitoring, and documentation while continuously predicting potential failures. AI-enhanced drones and IoT devices provide high-resolution data and real-time environmental and structural monitoring, reducing the need for manual intervention.
- Specialized Small Language Models (SLMs) - Designed for tailings management, SLMs deliver fast, accurate insights with lower computational requirements, making them ideal for remote or resource-constrained settings.

- Retrieval-Augmented Generation (RAG) - RAG models combine data retrieval and AI generation to improve decision-making. They integrate diverse datasets, simulate scenarios, and automate compliance documentation, driving both safety and operational efficiency.
- Physical AI – It is revolutionizing the dam industry by integrating AI with physical systems. This technology enhances dam operations through improved real-time monitoring, forecasting, and adaptive control. In dam classification and safety management, AI techniques analyse vast amounts of data to make informed decisions, potentially increasing overall efficiency in the face of climate change and growing energy demands.

Overall, to leverage GenAI in tailings dam management, organizations need an integrated approach. This includes developing task-specific AI models, ensuring data integrity aligned with GISTM standards, and implementing robust cybersecurity measures. Stakeholder engagement is crucial for aligning AI initiatives with regulatory and community expectations. Ongoing personnel training is necessary to effectively utilize GenAI innovations. Addressing these aspects can transform tailings dam classification, enhancing safety, sustainability, and operational standards in the mining industry.

6. CONCLUSIONS

This paper underscores the transformative potential of GenAI in enhancing tailings dam classification and management. By leveraging GenAI's capabilities in text, vision, audio, video, programming, and numerical analysis, mining dam operations can enhance safety, efficiency, and sustainability. Advanced methodologies, including GPT-4o, Small Language Models (SLMs), Retrieval-Augmented Generation (RAG), Multi-AgentRAG systems, and Physical AI developed with a lifecycle perspective, offer the capability to improve every aspect of tailings dam classification. Importantly, GenAI does not replace human expertise but rather amplifies its efficacy, enabling more informed and precise decision-making.

The successful implementation of GenAI demands rigorous attention to data integrity, bias mitigation, robust cybersecurity, and a multidisciplinary approach. Collaboration between AI specialists, tailings engineers, and regulatory bodies is essential to ensure the responsible and impactful deployment of these technologies. The integration of GenAI with physics-based models introduces a synergistic framework, empowering professionals to address complex challenges such as extreme climate variability and aging infrastructure while upholding accountability.

The potential of GenAI to revolutionize tailings dam classification is unparalleled. It enables the industry to proactively manage risks, optimize resource use, and enhance compliance with global standards. This paper delivers actionable insights and a forward-looking roadmap for integrating GenAI into tailings waste management, positioning the mining industry as a leader in sustainability and innovation on a global scale.

REFERENCES

[1] CDA (Canadian Dam Association). 2013. *2007 Dam Safety Guidelines, 2013 Edition.*

[2] Central Water Commission. (2020). *Guidelines for Classifying the Hazard Potential of Dams.* Dam Safety Rehabilitation Directorate, Central Dam Safety Organization, Government of India.

[3] ECCC, Environment and Climate Change Canada. 2018. *National Water Data Archive: HYDAT.*

[4] FMBCHEATSHEET. 2024. “Foundation Model Training Overview.” fmbcheatsheet. Retrieved from https://fmcheatsheet.org; accessed July 2024.

[5] Hariri-Ardebili, M. Amin & Mahdavi, Golsa & Nuss, Larry & Lall, Upmanu. 2023. “The role of artificial intelligence and digital technologies in dam engineering: Narrative review and outlook.” *Engineering Applications of Artificial Intelligence, 126.* 10.1016/j.engappai.2023.106813.

[6] LePoudre, C & Morrison, KF. 2022. “Tailings Management Systems,” in KF Morrison (ed.), *Tailings Management Handbook: A Life-Cycle Approach.* Society for Mining, Metallurgy & Exploration, Englewood.

[7] McKinsey and Company. 2024. *The promise and the reality of Gen AI agents.* Accessed July 2024.

[8] MEND 5.4.2d. (2001). *MEND Manual, Volume 4 – Prevention and Control,* Ed. G.A. Tremblay and C.H. Hogan, February.

[9] Minaee, S., Mikolov, T., Nikzad, N., Chenaghlu, M., Socher, R., Amatriain, X., & Gao, J. (2024). *Large Language Models: A Survey.* arXiv preprint arXiv:2401.00001.

[10] OpenAI. 2024. *Announcing GPT-4o in the API!* Retrieved from https://www.openai.com; accessed May 2024.

[11] Snow, R. 2022. “Overview of Tailings Management,” in KF Morrison (ed.), *Tailings Management Handbook: A Life-Cycle Approach.* Society for Mining, Metallurgy & Exploration, Englewood.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

THE DEVELOPMENT OF RESERVOIR LEGISLATION IN THE UK – PAST, PRESENT & FUTURE – A CRITICAL REVIEW AND RECOMMENDATIONS

Dr Andy K Hughes[1]
Reservoirs & Dams Ltd

ABSTRACT

In the UK we tend to react to incidents and failures leading to legislative change rather than think proactively. The biggest loss of life due to a failure was in 1864 prompting the first legislation – the Waterworks Clauses Act. Failures at Skelmorlie in Scotland and Dolgarrog in Wales in 1925 resulting in the loss of life of some 21 people, led to the Reservoir (Safety Provisions) Act 1930. Prompted by a review of the legislation in 1966, the Reservoirs Act 1975 was born, bringing in the role of the Supervising Engineer, a trained engineer who visits each year, and the formation of a Regulator to ensure the provisions of the Act are applied.

Changes in the number of Regulators and devolution has led to different legislation in England and Wales, in Scotland and in Northern Ireland.

After floods in 2007 and a worldwide interest in applying risk-based approaches, it was decided that the volume which causes a reservoir to be subject to the legislation be reduced from 25,000 to 10,000 cubic metres. Wales has applied this change, but no other country has yet to apply the criteria.

In 2019 an incident at Toddbrook Reservoir led to an investigation and report by Government. This has led to changes, mainly administrative and the generation of significant amounts of paperwork which, in the opinion of the author, does not in general make a dam safer. The number of Supervising Engineers (SE) has reduced from 500 to 120 in the last 35 years. The number of All Reservoirs Panel Engineers (ARPE) has reduced from 100 to 28. The author's view is that we are not encouraging engineers to become reservoir engineers, not training them properly and not actively supporting them.

Featuring on a national TV programme in 2017, the author stated that he believes that if the probability of a dam failure involving loss of life is rising to an unacceptable level that a failure involving loss of life could occur in his lifetime (next 20 years) – the author still believes this to be a distinct possibility.

Keywords: Legislation, Regulation, Training, Incidents, Failures

[1]andy.hughes@damsandreservoirs.co.uk, UK

1. INTRODUCTION

This paper will explain how reservoir legislation has developed in the UK and how it continues to be developed.

It will show that in general we have reacted when an incident or failure occurs at a dam and that we change our regulations to suit, rather than to think ahead and take a proactive approach. These changes are sometimes made without consultation and gaining the advice of practising engineers.

This paper will present the author's views that the changes and the way in which the profession is developing, which is partly due to generational changes but also a number of other reasons, is not necessarily the most appropriate. A few years ago, the author stated on British National TV that he I believed the result of inappropriate changes could lead to a failure of a dam involving loss of life.

2. THE HISTORICAL DEVELOPMENT OF LEGISLATION IN THE UK

When one looks at how legislation was developed, there was no legislation relating to dams and reservoirs until 1863 with clauses in the Waterworks Clauses Act[1]. This was a reaction to the failure of a dam called Bilberry in Yorkshire when some 80 people were killed.

The Waterworks Clauses Act 1863 stated that if someone was worried about the condition of a dam they could complain to two Justices of Peace, who would then investigate the issue and then arrange for the cause of the complaint to be removed. There are no records of this legislation be used.

Around the same time there was a case where the owner of a coal mine, which existed below a reservoir, sued the owner of the dam after reservoir water seeped out of the reservoir into the mine. The case of Rylands and Fletcher forms the basis of the Law of Tort and applies to cases of a civil wrong that results in loss or harm to another.

In 1864 the Dale Dyke Dam, situated 8 miles to the west of Sheffield, failed. This was a new dam, which was 20 metres high and therefore 'large' in the UK, which released large volumes of water into the centre of Sheffield where more than 245 people were killed. The worst loss of life incident in the UK to date.

The author researched this failure some 150 years after the event and found that the Coroner's inquest was, in his opinion, a bit of a cover up. One good recommendation from the jury, was "all dams and reservoirs should, by an Act of Parliament, be subject to frequent, sufficient and regular inspections".

To review this idea, in true British fashion, the UK set up a ‘Select Committee’ to look at what could be done to improve reservoir safety. Nothing was done until 1925 when there were two dam failures – one in Scotland and one in Wales, which resulted in the loss of 21 lives and increased public awareness and opinion on reservoir safety.

The result, some 5 years later, was the Reservoirs (Safety Provisions) Act 1930[2].

One of the failures in 1925 involved a reservoir retaining 5 million gallons of water and it was therefore decided that this should be the threshold to which the Act applied – no other reason unfortunately. Thus, the Act was applied to some reservoirs which posed no risk to persons and property downstream but equally there were some reservoirs below the 5 million gallon threshold which should they fail would cause loss of life.

The Reservoirs (Safety Provisions) Act 1930 was brought into force on the 1st January 1931 and it applied to reservoirs which store more than 5 million gallons above the level of the natural ground.

The Act introduced the “Panel Engineer” system whereby an individual, independent of the owner, and assessed as competent by the Secretary of State, undertook an inspection and made recommendations in the ‘interests of safety’. The maximum period between statutory inspections was 10 years. The Act also required the owner to keep records of water levels etc.

A “Panel” is just a list and at the moment there are four lists covering all reservoirs; the All Reservoirs Panel, giving the names of All Reservoirs Panel Engineers (ARPE’s) who can inspect, supervise, design and oversee the construction of all types of reservoir in terms of construction type and function; the Non-Impounding Reservoir Panel (NIR), as above but only applying to non-impounding reservoirs; the Service Reservoir Panel (SR), as above but applying to service reservoirs, and the Supervising Engineer Panel (SE), those engineers who carry out the annual examination and reporting thereafter via a statement.

However, there was no provision for enforcement of the Act and indeed no requirement for an owner even to admit they owned a reservoir! Thus, we had a situation where the water companies registered their reservoirs, had them inspected and did any work necessary but there were many owners, particularly in the private sector, who didn’t even register their reservoirs and certainly many did not do the works recommended to be done, even if registered!

The UK then applied the Reservoirs (Safety Provisions) Act, and there were no more failures involving loss of life, but there were a significant number of incidents and a number of failures but none involving loss of life. The failures at Malpasset in France and Vaijont in Italy, together

with a meeting of the International Commission on Large Dams (ICOLD) in Scotland at about the same time, caused the UK to look at whether the legislation was still appropriate.

A committee was set up and that reported that there were 12 areas of major improvement which could be implemented - the main ones being the creation of an Enforcement Authority and the creation of the role of the Supervising Engineer. The main changes proposed meant that there would be a "policeman", the Enforcement Authority, to enforce the Act, who would also create a register of reservoirs - because we didn't actually know how many reservoirs we had or where they were! The creation of the role of the Supervising Engineer (SE) was to put a person in place who would visit the reservoir at least once a year and keep the owners advised of its behaviour and make sure the owner was maintaining the reservoir, reading instruments etc and if necessary, bring forward the independent inspection by a Panel Engineer called the Inspecting Engineer.

Once again, in true British fashion, nothing happened immediately, and it took nearly 10 years to get the Reservoirs Act 1975[3] through Parliament and onto the statute book. It then became an Act on the 8th May 1975.

This was a major, positive step forward without being too complex involving statutory inspections at no more than 10 yearly intervals by an Inspecting Engineer, a frequent and regular examination by a Supervising Engineer, whose role was to keep the owner advised of any behaviour of the dam which might affect safety, and the enforcement role of the Act was given to more than 160 local authority councils and metropolitan borough councils in England and Wales and more than 36 in Scotland. These organisations were required to ensure the provisions of the Act were followed by the owner.

The volume which was used to define whether a reservoir was subject to the legislation or not was 25,000 cubic metres – a slight rounding up from the 1930 volume of 5 million gallons (22,700 cubic metres) – and so just before the Act became active a significant number of owners tried to reduce their retained volume to less than 25,000 cubic metres – say to 24,500 cubic metres which really didn't change the consequence of failure and the effect on people downstream but nevertheless the reservoir would no longer be subject to the legislation.

The British Legal System requires a Bill to be written, it then has to pass a vote in the House of Commons, followed by a vote in the House of Lords, and then must achieve Royal Assent. It then becomes an Act, however, the Act doesn't start being applied/working until there is a Commencement Order. So, the Reservoirs Act 1975 became an Act in 1975, but it was not commenced until 1987! This delay was caused, in the author's opinion, by political pressures being applied by influential groups. There was even a television programme on national TV in 1981 trying to get Government to commence the Act. The Act was finally commenced in 1987.

Over the next decade it became evident that with so many Enforcement Authorities the standards of enforcement were very different. The Water Act of 2003[4] changed the enforcement role such that the role in England and Wales was carried out by one Agency, the Environment Agency (EA) and in Scotland by the Scottish Environmental Protection Agency (SEPA).

In addition, the creation of onsite plans as part of the Flood Plan in the case of emergency was something suggested. The Flood Plan is made up of three elements, the onsite plan – how an owner would react in an emergency; the inundation map identifying those areas affected should the dam fail both in a sunny day scenario and a wet day scenario; and an offsite plan written by the emergency services detailing how they would respond in an emergency.

An event which caused some significant flooding across the UK occurred in 2007. No dam subject to the legislation failed but a number of small reservoirs (<25,000 cubic metres) associated with the industrial archaeology of the northeast and northwest which essentially were abandoned caused a few problems. The Government commissioned a report carried out by Sir Michael Pitt. This resulted in the 'Pitt Report'[5] which had a chapter on reservoirs. Sir Michael Pitt accompanied the author to some reservoir sites and his conclusion was that:

- Inundation maps should be produced for every reservoir illustrating where the water goes should the dam fail.
- That the likelihood of a dam failure is very low.
- That the Institution of Civil Engineers (ICE) should take action to encourage more people to enter the profession in order to ensure an adequate succession of 'Panel Engineers'.
- That we should move to a risk based approach, and that
- There should be a new legislative framework for reservoir safety.

This then lead on to the Flood and Water Management Act of 2010[6] which amended a number of elements of the Reservoirs Act 1975 including:

- A change to the retained volume to 10,000 cubic metres.
- The application of a risk designation in which decides whether the Act applies and to what designation viz 'high risk' v 'not high risk' – those dams which should they fail would kill someone against those who wouldn't kill anyone.
- A Construction Engineer is required for reductions in capacity as well as increases in a reservoir's capacity.
- The application of dates by which measures recommended should be completed.
- The submission of inspection reports within 6 months of a visit (9 months in Scotland).
- Increased powers for the Supervising Engineer and the ability to order "directions".

- The preparation of flood plans.
- Assessment of reports and statements by the Regulator.
- The ability for the Regulator to make changes to their role.

So again, a reactionary approach.

The Flood and Water Management Act 2010 brought in many of his recommendations although not all have been applied even to this date.

The cubic metre threshold volume to which the Act applied was reduced to 10,000 cubic metres. It was suggested that rather than a portfolio of reservoirs based on volume retained alone, a risk-based approach should be followed. This requires the calculation of not only the consequence of failure but the probability of failure, which is of course more difficult.

With the devolution of Government in the UK, and assemblies and parliaments in Edinburgh, Cardiff and Belfast, things also changed. Suddenly we had the Reservoirs Act 1975 as modified by the Flood and Water Management Act 2010, directed by Defra (The Department for Environment, Food and Rural Affairs) and regulated by the EA (Environment Agency) in England and NRW (Natural Resources Wales) in Wales. So, one piece of legislation for England and Wales but what is bizarre and difficult to comprehend is that Wales adopted the 10,000 cubic metre threshold but England has not!

Equally the Regulator operates in a very different way in England to the one in Wales which makes application of the legislation difficult.

In Scotland they developed their own legislation called the Reservoirs (Scotland) Act 2011[7] which is broadly similar to that in England and Wales but has some subtle, and in my opinion, better differences. Northern Ireland is also developing its own Act, yet to be regulated and commenced called the Reservoirs Act, Northern Ireland, 2015.

So, it can be seen that differences in approaches and even regulation have started to creep in. Four countries, three pieces of legislation applied in different ways. Surely we could have picked the best ideas from each country and had one Act? Is this sensible in such a small group of countries served by the same group of consultants providing design, inspection and supervision on dams and reservoirs which are essentially the same?

In 2017 the author appeared on a national TV programme called ‘Countryfile’; a programme about farming, rural practices and environmental issues and aired across the UK. On that programme the author voiced concern about the state of the stock of reservoirs and its associated support by consultants, engineers and contractors and the author stated he believed that the probability of a failure of a dam involving loss of life was becoming more and more likely and was likely to occur in his lifetime. That comment attracted little or no comment!

In August 2019 an incident occurred at Toddbrook Reservoir above Whalley Bridge in Derbyshire. An auxiliary spillway, some 50 years old, constructed from reinforced concrete and built over the crest of the dam, broke up during a storm estimated to have a return period of some 1,000 years. Through the excellent work of a contractor, who installed a bank of pumps very quickly to pump the water level down, in the author's opinion, the likelihood of the dam failing was significantly reduced.

Two reports were produced; one for Government colloquially known as the 'Balmforth Report' written by Mr David Balmforth and another by the author for the owner of the dam, The Canal and River Trust.

Fig. 1 – Toddbrook Reservoir

They had very different conclusions and suggested very different failure modes. The author's report concluded that a design fault meant that the spillway started failing from the first day of its use back in 1970 and erosion was taking place under the slabs every time water reached the underside of the slab. After some 3,500 events the voids underneath were so large, that when the spillway came into operation in August 2019, the slabs collapsed. In the author's opinion, there had been a significant number of warnings associated with dirty water flows at the base, and cracking etc which were not picked up by five All Reservoirs Panel Engineers (ARPE's) who inspected the dam over 50 years and a large number of Supervising Engineers visiting every year over that period. Even when the last Inspecting Engineer who recognised a problem could exist and asked the owner to investigate, the owner's inhouse Supervising Engineer failed to recognise the importance of the issue. In the author's opinion, a failure on the part of all involved.

The Inspecting Engineer gave the owner 18 months to investigate whether the spillway had a cutoff under the slabs. Unfortunately, that period was too long, and failure occurred before the investigation took place. However, in the author's opinion, had the Supervising Engineer gone

to the owner and said we need to give some urgency to this investigation then the Toddbrook incident would not have happened.

The Balmforth Report, called the Independent Reservoir Safety Report (March 2021)[8] concluded the causes of spillway damage at Toddbrook were very different to those identified by the author. The author believes the report did not highlight the correct mode of failure or identified the real issues of the site. A second report by Balmforth was to look at current legislation and its implementation. It was supposed to explore developing a risk-based approach, engender a continuous improvement to safety and secure a proportionate regulatory approach.

The report covered the effectiveness of current legislation, it reviewed the competence of reservoir engineers, the safety management of UK infrastructure, a review of international practices of reservoir safety management, a chapter on compliance and safety, a review of the impact of climate change and ended with a summary of findings and recommendations.

The recommendations started with the statement that over 2.4 million people in England are at risk from 2,095 reservoirs.

The recommendations made by Balmforth were as follows: - there is no prioritisation and the author will comment at the end of the paper.

RECOMMENDATION 1. High risk reservoirs should be divided into three classes depending on the hazard created by the reservoir:

a) **The degree of effort and detail required in reservoir safety management should depend on the Class**. Reservoirs in the lowest of the classes of hazard (3) should be subjected to the same regime of inspection and supervision by qualified reservoir engineers that exists at present, c. every 10 years. Higher hazard reservoirs should be progressively subject to a more detailed approach. For reservoirs in the highest hazard class (1), the maximum period between inspections should be reduced to 5 years and a detailed Design Review undertaken every 20 years.
b) **The threshold between classes should be determined by the Environment Agency, in consultation with Defra (Department for Food, Environment and Rural Affairs), the ICE (the Institution of Civil Engineers) and the BDS (the British Dam Society)**. As hazard changes over time, the thresholds that determine the different classes should be kept under review.
c) **Government should review the current threshold for designating high risk reservoirs**. They should determine if a threshold based on risk (hazard) would be more appropriate in the light of other recommendations in this report that recommend a risk based approach for managing reservoir safety.

RECOMMENDATION 2:

a) **The Environment Agency should make owners fully aware of their duties and responsibilities**. This should include the need to exercise due diligence and to respond to the directions of reservoir engineers in a comprehensive and timely manner.
b) **The Environment Agency should work with the owner to help develop their capability and that of their operational staff and to foster continuous improvement to their reservoir(s)**. The expected capability should depend on the reservoir hazard (class).
c) **The Environment Agency should have the powers to charge owners for their regulatory services and to do this in a way that incentivises responsible behaviour**. Charges should reflect the amount of effort that is needed to regulate the reservoir(s).
d) **The Environment Agency should establish a procedure for managing and adjudicating any disputes with an owner and resolving them**. This should replace the current system that allows owners to appeal inspection reports.

The author comments that there are few or no appeals known.

RECOMMENDATION 3. Owners should adopt a systematic and well documented approach to reservoir safety management, and this should be approved annually:

a) **The reservoir owner (undertaker) should prepare and implement a Reservoir Safety Management Plan (RSMP). The extent of the plan should reflect the classification of the reservoir**. The plan should detail all the necessary surveillance, monitoring, operation, maintenance and periodic inspections required at the reservoir, and include a log of all activities to demonstrate that the plan is being effectively delivered. The RSMP should also include, all available details of construction and alteration of the reservoir, copies of reports of periodic inspections by Supervising and Inspecting Engineers, and certificates to demonstrate that the requirements of Supervising and Inspecting Engineers, have been complied with.
b) **The RSMP should be kept as part of the Prescribed Form of Record and the requirements of that Record should be amended to accommodate this and avoid duplication**.
c) **The Supervising Engineer should review and approve the RSMP annually and certify that the owner's actions have been carried out in accordance with the Plan**. Approved and certified RSMPs should be submitted annually by the owner to the Regulator.

d) **The Owner should ensure that all personnel with responsibility for delivering the RSMP are appropriately competent to do so**. For reservoirs in I classes 1 and 2 this should include an appropriate means of certifying their competence.
e) **The Environment Agency should produce guidance for owners for the production and delivery of RSMPs, including exemplars for the different classes of reservoir**. Implementation of these recommendations should recognise that a number of owners already meet many of the requirements of RSMPs and this should not, therefore, impose an undue burden on them.

RECOMMENDATION 4. Supervising Engineers should be fully engaged in assuring the day to day surveillance, operation and maintenance of the reservoirs that they supervise:

a) **As part of their routine visits to a reservoir, Supervising Engineers should fully engage with surveillance and operational staff, review the records demonstrating tasks completed and verify that the RSMP is being delivered as planned. These are in addition to their existing duties**. Where instrumentation records are being taken, the Supervising Engineer's reports should include a copy of the results, ideally in graphical form, together with a commentary explaining how they demonstrate that the reservoir's behaviour is remaining within safe limits. They should discuss any recommendations with the reservoir owner and where necessary give directions for improvement.
b) **Each year the Supervising Engineer should formally certify compliance with the RSMP as part of the annual statement and approve the RSMP for the following year**. The Environment Agency should be able to review the annual statements of Supervising Engineers and RSMPs and to challenge them.

RECOMMENDATION 5. The Periodic Inspection of Reservoirs by Inspecting Engineers should be systematic, detailed and impartial, and their findings and requirements communicated in a clear and evidence-based manner:

a) **As part of their inspection of a reservoir, Inspecting Engineers should identify all the potential failure modes of the dam and other reservoir structures**. They should determine the significance and credibility of each of these and then evaluate each to understand the overall likelihood of failure. On the basis of this they should require any measures in the interest of safety (MIOS) or amendments to the RSMP.
b) **Where an Inspecting Engineer considers that further investigations are needed, an interim report should be issued together with any associated MIOS or precautionary measures pending the completion of the investigation and the issuing of a final report and requirements for further measures, where required**.

c) **For class 1 and 2 reservoirs, Inspecting Engineers should undertake or update, as necessary, a risk assessment for the reservoir.** Where MIOS are required as a result of a risk assessment, these should be specified so as to reduce risk to ALARP, and evidence should be provided to demonstrate that.
d) **Appropriate and clearly defined timescales should be attached to each MIOS depending on the urgency of implementation**. In assessing the condition of the reservoir, the Inspecting Engineer should not rely on visual observations alone but should make use of the data and information collected during routine surveillance activities, previous reports from Inspecting and Supervising Engineers and records of the reservoir's design and construction. In any report they should express their findings, recommendations and requirements in a clear and evidence-based manner.
e) **Where precautionary measures may compromise the beneficial use of a reservoir, their implementation should be determined on the basis of managing the risk to be ALARP**.

RECOMMENDATION 6:

a) **Any MIOS that is urgent should be clearly and unambiguously indicated as such in the Inspecting Engineers Report and an appropriate completion date specified.**
b) **For urgent MIOS the Owner should ensure that a Qualified Civil Engineer (Construction Engineer) is appointed to oversee their implementation as soon as practicable but no later than 14 days from the issue of the IE's report.**
c) **Urgent MIOS should be completed as soon as practicable and a certificate issued by the QCE not later than the specified completion date.**
d) **The certificate issued on completion of MIOS should provide details of the measures certified.**
e) **Required amendments to the RSMP should include dates by which each amendment should be implemented. The RSMP should be updated with the amendments within 14 days of issue of the IE's report and their implementation reviewed by the SE immediately as soon as practicable after the specified date of implementation and thereafter during their regular visits.**

RECOMMENDATION 7: Defra and the Environment Agency, working with their counterparts in the other administrations of the UK, owners and employers should commission the ICE to undertake a thorough review of the supply and development of Supervising and Inspecting Engineers to ensure future supply. This should include the technical and professional requirements, the value of the sector to the employers' business, and the commercial models and practices for procurement. In particular the attractiveness of

this branch of the profession should be compared with other sectors where recruitment is not so much of a challenge.

RECOMMENDATION 8. The ICE Reservoirs Committee should review the attributes of reservoir engineers to ensure that they appropriately match the role that they play and foster smoother career progression without dropping standards:

a) **The Reservoirs Committee should review the designation of Reservoir Panels to ensure that they are appropriate for future needs. In particular they should consider if a single set of attributes is appropriate for the inspection of reservoirs with different levels of hazard**. For example, it may be appropriate for engineers qualified to inspect lower hazard reservoirs of certain types of construction to have less experience and attributes compared with those qualified to inspect higher hazard reservoirs or reservoirs of more complex construction. Likewise, the inspection of extremely high hazard reservoirs may require greater experience/attributes. There may also be an argument for not requiring construction experience for all Inspecting Engineers, though this should be retained for engineers qualified to oversee and certify construction or alteration of a reservoir and other changes to reservoir structures, or measures in the interests of safety requiring construction works.
b) **The ICE should review the process for supporting prospective candidates for Supervising and Inspecting Engineers through comprehensive training, mentoring and guidance**. The potential to introduce probationary periods for newly qualified Inspecting and Supervising Engineers should also be considered.

RECOMMENDATION 9. The Environment Agency, working with Defra and their respective counterparts in Wales, Scotland and Northern Ireland, should work with the ICE and others to improve the capacity and capability of reservoir engineers through better knowledge sharing, better incident reporting and improved guidance:

a) **The Environment Agency should work with the ICE, BDS, CIRIA, ICOLD and others to ensure that all reservoir engineers have better access to national and international learning as it develops**. This should be achieved through promoting conferences, seminars and meetings which can be attended by all reservoir engineers in a convenient and affordable way. More attention should be placed on the continued learning and development of reservoir engineers and their personal contribution to knowledge before their initial application and at the time they apply for reappointment.
b) **The Environment Agency, in consultation with Defra, should review the incident reporting framework and report content, and work with owners to draw out more comprehensive lessons in a timely manner**. This will ensure that incidents are

brought to the attention of reservoir engineers without delay and that lessons learnt from any subsequent investigations are shared in a timely and comprehensive manner.

c) **The Environment Agency should review the definitions in the reporting framework to allow for the reporting of near misses and for anonymous reporting**. In doing this they should review how this is done in other infrastructure sectors.

d) **The Environment Agency should work with the ICE to comprehensively update guidance on reservoir supervision, reservoir inspections and risk assessments**. Appropriate pro-formas should be produced as part of the guidance with the aim of assuring that reports are consistent and complete. However, pro-formas should avoid being over prescriptive so as not to encourage a 'tick box' approach. Ultimately the safety of reservoirs is best assured by the expert judgement of the professionals involved.

e) **In the event of new legislation or regulations, Defra should commission the !CE to update its guidance to the Reservoirs Acts**.

RECOMMENDATION 10. Class 1 and 2 high risk reservoirs should be managed and operated on the basis of risk, to ensure their ongoing safety:

(a) **Reservoir owners should manage the safety of these reservoir(s) by ensuring the risks that they pose are managed to be as low as is reasonably practicable (ALARP)**. The assessment of risk should include a quantification of the probability of failure of the dam and other significant reservoir structures, based on an appropriate assessment of potential failure mechanisms, the consequences arising from an uncontrolled release of water on the area downstream of the reservoir, and the effectiveness of the RSMP. It should also take the owners competence into account.

(b) **The risk assessment should be based on recognised good practice**. The Environment Agency should give guidance to owners on the appropriate approach to risk assessment, which should include an assessment of uncertainty. However, it should recognise that some owners already have robust risk assessment methods in place. Owners should not be unduly constrained in the methods that they use.

(c) **MIOS implemented as a result of the risk assessment should be such as to reduce the risk to be both tolerable and ALARP**.

(d) **Where the probability and/or consequence of failure of a reservoir cannot be practicably reduced to a level where the risk becomes tolerable the reservoir should be decommissioned**.

RECOMMENDATION 11. The Environment Agency should promote and champion good practice. It should provide guidance to owners and engineers, and have the duties and powers commensurate with regulators in other infrastructure sectors:

a) **The Environment Agency and Defra should work with the ICE and BDS, to produce a Code of Practice in Reservoir Safety Management**. This should set out clearly the roles and responsibilities of all the parties to reservoir safety management, in particular owners, surveillance, monitoring, operational and maintenance personnel, reservoir engineers, the Regulator and the ICE Reservoirs Committee. The Code should not only reflect legislation but include wider good practice. In due course Government should consider if such a Code should become statutory.

b) **The Environment Agency should have a duty of assuring the necessary systems and procedures are in place and applied by all to ensure all parties engaged in reservoir safety management fulfil their duties and responsibilities**. The Environment Agency should promote reservoir safety (with others as necessary) through promoting the Code of Practice and by fostering good practice through timely reports, guidance and training, to ensure that the measures recommended in this Review are properly implemented in a consistent manner.

c) **The Environment Agency should have a duty to review a sample of Supervising Engineers' reports and annual statements, inspection reports, risk assessments and RSMPs each year**. This duty will ensure an on-going national assessment of the quality of inspections and reporting which should be used to update guidance and training where necessary. The Environment Agency should also have the duty and powers to review specific reports and/or RSMPs beyond the annual sample where it finds this appropriate.

d) **The Environment Agency should have powers to question or challenge Supervising Engineer's annual statements and reports, inspection reports, risk assessments and RSMPs**. Where there are concerns over the appropriateness of a statement, report, risk assessment or RSMP or its delivery or requirements, which cannot be resolved with the owner and their engineer(s), the Regulator should have the powers to appoint their own reservoir engineer(s) to undertake an inspection and/or risk assessment and require MIOS and/or amendments to the RSMP and enforce their implementation.

e) **The Environment Agency should have the duty to spot check the surveillances, operation, maintenance, and the condition of a sample of regulated reservoirs each year. They should have the powers to do this at any reasonable time and without warning and to issue improvement, stop and other notices to owners as a result of such checks or for any other reason**. These powers should be in addition

to any current powers. The Environment Agency should also have powers to spot check the surveillance, monitoring, operation maintenance and condition of reservoirs beyond the annual sample.

RECOMMENDATION 12. The Reservoir Regulatory and Enforcement: Function within the Environment Agency should be properly resourced, and its independence fully protected:

a) **Routine costs for regulation and enforcement should be fully recovered from reservoir owners in a fair and equitable manner (for example according to reservoir hazard and the amount of regulatory effort required)**. This includes additional costs associated with specific enforcement measures, intervening and taking over the duties of a reservoir owner, and all costs associated with emergency action in the interests of safety. Government should meet any further reasonable costs that cannot be recovered, for example costs relating to emergency intervention where a reservoir owner becomes insolvent.
b) **The Environment Agency should have the powers to make charges to Owners and to issue fines and other penalties as a result of a lack of compliance**. Such powers should allow the Environment Agency to use changes and fines as a means of fostering good practice amongst owners.
c) **The independence of the reservoir regulatory section of the Environment Agency should be strengthened**. The Environment Agency should ensure that it has reporting and resourcing procedures in place that protect its independence, and periodic scrutiny of its independence using external assessors should be established.

RECOMMENDATION 13:

a) **The current research project commissioned by the Environment Agency into PMP and PMF should allow for the non-stationarity of climate. It should give guidance on estimating the frequency of present day and future extreme flood events suitable for use in reservoir risk assessment**. This should be based on data from multiple scenarios of computer-generated weather using the best available tools and incorporating the latest rainfall climatologies.
b) **The Environment Agency should review recent and ongoing research on the impact of climate change on constructed embankments and determine if further research is needed to better understand the implications for earth embankment dams and other reservoir structures**. If necessary, further research should be commissioned.

RECOMMENDATION 14:

a) **The Environment Agency should produce an annual report of its activities and an objective assessment of the overall state of the nation' reservoir stock. The report should be public facing and explain the role that the Environment Agency plays in reservoir safety and its objectives**. The Environment Agency should establish key performance indicators so that it can demonstrate the continued improvement in reservoir safety year on year. The public should be able to readily understand how reservoir risk is managed and be assured of its effectiveness and independence.

b) **Owners should publish key information about each of their regulated reservoir(s), to an agreed format, such that the public can be assured that safety is being appropriately managed**. The public should be able to access appropriate information at individual reservoirs without compromising security needs. The Environment Agency should produce a pro-forma to assist with this.

RECOMMENDATION 15. The recommendations made in this Review should be supported by the development of appropriate legislation and regulations:

a) **Government should consider how they might best implement the recommendations made in this Review and determine what changes are needed to legislation and regulations in order to gain the full benefits to reservoir safety in the long term**.

b) **In considering future reservoir safety legislation and associated regulations, Government should take into account the more recent wider developments in safety management and technology, and ensure it reflects the broad principles of safety management as embedded in Health and Safety legislation and practice**. It may, for example, be appropriate for the Secretary of State to determine suitable standards and thresholds from time to time through regulations and/or statutory guidance rather than embedding these in primary legislation. This would allow maximum flexibility for change as knowledge and practice on reservoir safety develops.

c) **When amending current legislation and drafting new legislation the enforcement policy should recognise that it is not always appropriate to take formal enforcement action in every case**. This will avoid parties who make every effort to comply but marginally miss compliance on occasion being recorded as being non-compliant.

d) **Government should consider whether these changes are substantial enough to propose a new Reservoirs Act, rather than continuous amendments to current legislation, to ensure our approach to reservoir safety remains fit for purpose in**

the future. In the intervening time Government, in consultation with the reservoir profession, should consider how many of the recommendations in this review might be sensibly delivered in the short term through secondary legislation and improved guidance.

In the author's opinion much of this is already being done. Other elements create a lot of paperwork which again, in the author's opinion, does not make a dam safe.

One of the benefits of the current process is the simplicity of the system. Many of the recommendations involve a significant amount of paperwork and complexity which in the author's opinion some engineers will not be able to cope with and certainly will result in extra costs which many of the owners of dams will not be able to afford.

Many of the recommendations give more power to the Regulators and takes away the ability of experienced engineers to exercise judgement and pragmatism. Up until now the Regulator is an administrative role – many of the recommendations are already in place.

The report does not differentiate between those elements which already exist and what is new. It paints a picture of a system which is broken – which is certainly not true.

It also provides more powers to the Regulator to change from an administrative role to one who can challenge the engineering capability of all involved including the Supervising Engineer who will attract additional liabilities that they do not want and who may not be equipped to take on!

Some of the recommendations use terms which are incorrect demonstrating a lack of understanding. Equally many of the recommendations recommend a move towards more standardisation and takes away the ability for an engineer to exercise pragmatism and make judgements – recommendations which are in conflict with others. Each dam is unique – that is what makes our job interesting, and standardisation is difficult.

The impartiality of the Inspecting Engineer has always been a pillar of the UK's reservoir legislation – the erosion of that impartiality and the removal of the engineers' ability to exercise pragmatism and judgement is, in the opinion of the author, an unwelcome change.

3. ENGINEERS

The Inspecting Engineer is engaged by the owner who is responsible for the fees charged. The report that is produced and any recommendations is submitted to the owner who then has to carry out any recommendations. The current role of the Regulator is to ensure the recommendations are carried out.

To become a Panel Engineer, appointed by the Secretary of State (SofS), one has to apply through the President of the Institution of Civil Engineers (ICE) who has been delegated the powers to decide whether a candidate seeking appointment to a Panel has the necessary 'attributes'. The President of the ICE has formed a committee, known as the Reservoirs Committee, who receives the applications from members and if first time appointees, arranges a face to face interview where the depths and quality of their experience is explored. Interviews are conducted by a group of 3 from the 12 people who sit on the committee. The views of the interview group are relayed back to the full committee of 12 persons and then a recommendation goes to the Secretary of State to appoint a candidate or not.

Some 40 years ago when the author was considering application to the All Reservoirs Panel to become an All Reservoirs Panel Engineer (ARPE), there was very little information on what experience you needed to become a Panel Engineer, no written rules and no attributes. There were written notes which suggested you had to be about 45 years of age, a member of a consulting engineer's staff, and to have built a new dam. At 32, the author worked for a large water authority looking after some 300 dams on a day to day basis, and he looked after a stock of ageing (average age about 120 years) mainly embankment dams. The author applied and was appointed to the Non-Impounding Reservoir Panel (NIR) even though he was looking after mainly impounding reservoirs and was carrying out the flood study and flood routing calculations for the company's reservoirs. The author was clearly told that if he wanted to achieve AR status he would have to get involved with designing and building a new dam. This would entail leaving his employer of the time, North West Water, uprooting his family and moving to one of the major consultancies in the UK and becoming a consulting engineer. Luckily, since the community of dam engineers and firms doing dam engineering in the UK is small and were working in the UK and oversees, it was a very short period of time before a consultant offered the author the senior RE spot on a project improving the water supply to the UK military base of Episkopi in Cyprus, which included the construction of a 40 metre high rockfill dam with upstream HDPE membrane.

The author took up the post in 1989, and by 1991 he was appointed to the All Reservoirs Panel as well as becoming a Fellow of the Institution Civil Engineers (FICE). So, to achieve his desires he had to uproot my family and move to Cyprus for 3 years, but in fact his employer NWW (North West Water) did not want him to leave and so he was seconded away from the company. Later the author left NWWA and went straight in as a partner of Rofe Kennard and Lapworth (RKL). Subsequently he worked for ARUP, KBR and Atkins before venturing out on his own with his own company Dams and Reservoirs Ltd. At the time he was appointed to the All Reservoirs Panel and was No. 100, and the youngest appointed to the Panel at 37. In 1987

when the Panels were set up there were 487 Supervising Engineers (SE's) on the SE Panel, covering nearly 2000 dams.

If we look at the picture now, we have about 120 Supervising Engineers and 27 All Reservoirs Panel Engineers – the situation is, in the author's opinion, now critical with insufficient engineers with insufficient experience, and he reinforces that as already predicted, a failure involving loss of life in the UK may well occur within the next 25 years.

So why the reduction in numbers? In the author's opinion a number of factors contribute to the situation we face today, including:

- British firms being taken over by foreign firms who dislike the risk/reward profile applicable to dam inspections and supervision;
- Generational changes from the 'baby boomer' era through to today;
- Increases in responsibilities and liabilities;
- Lack of financial resources available to some owners;
- Increasing costs of construction and repair;
- 'Frameworks' where large companies sign up to one or two contractors and associated consultants for up to 5 years;
- and lately, the aggressive nature of some Regulators;
- More paperwork.
- The erosion of the ability of the engineer to exercise judgment and pragmatism

In years gone by the author would say Panel Engineers had the best job in the world – lately the engineers live in fear of being challenged by the Regulator and this is causing some engineers not even willing to apply to be a Panel Engineer, and many on the Panel choosing not to reapply.

The UK, in the opinion of the author, is certainly not encouraging engineers to become Panel Engineers and whereas the standards set years ago to become a Panel Engineer, appears to have reduced so much over the years, caused by a Government and Regulator panicking, that, in the opinion of the author, a significant number of engineers on the Panel do not have the experience to be on the Panel. The author has witnessed decisions in meetings and recommendations in reports which are at times ambiguous, even dangerous, sometimes with no technical basis, and sometimes even incredulous. Unfortunately, the future is not good because we are not training people properly, and other members of the profession are retiring before passing on their knowledge.

The main outcome from the Balmforth Report seems to be more paperwork, more liability placed on the Supervising Engineer, the one person often least able to accept that liability, and an aggressive Regulator in England – but not in Wales or Scotland. Engineers, in general,

don't like paperwork, they want to do their job out on the dams. The author feels engineers are being challenged on what they write when the Regulator is supposed to be an administrative role – the thought that they can be challenged at any time when they have carried out their role for many years quite successfully generates a fear and therefore engineers now question if they want to carry on doing the job – particularly when situations/incidents occur. However, there is inconsistency when at Toddbrook, and a few other cases known to the author, those engineers have not been challenged.

In the author's opinion, the Reservoir Legislative System in the UK is in a difficult and potentially dangerous situation. Historically and up to recent times we have reacted to situations, never have we thought about how we can improve things and work in a proactive way. We have an ageing population of dams (average age of about 130 years) and a dwindling number of experienced Panel Engineers and a young group who, in my opinion lack experience and knowledge, and the recent actions of our Regulator means the pragmatism and judgement that engineers used to be able to apply is moving to a 'tick box' mentality. Paperwork is now all important and the way engineers are judged and maybe even punished. Fear pervades the profession – who will be challenged next and forced to resign for no good reason?

It appears that the younger profession and the Regulator believe that paperwork will save a dam failing more than allowing engineers to use experience, judgement and pragmatism. The other side of that is that if that paperwork is not 'adequate' as defined by other engineers, then an engineer can face penalty.

If you speak out you are likely to receive aggressive intervention from the Regulator and face 8-hour long interviews, and temporary suspension from the Panel which in some cases have lasted more than 12 months with no income and the individual forced into retirement despite being highly respected and with more than 100 letters of support being sent to the Regulator and Government.

Another 'rule' which has no foundation in law or statute that the Regulator in England, is now known as the rule of 51! A number of engineers were acting for small private owners and farmers as Supervising Engineer and had often built up a portfolio of 100 or more, sometimes 140 reservoirs which they had to visit over a year. These dams are simple structures, particularly the farm/irrigation dams with no tunnels, no valve shafts, often no overflows, with water pumped over the crest via temporary pipes from the local stream or boreholes when water was available and then pumped back to the fields. The dams are often only 3 or 4 metres high and situated in unpopulated areas used to irrigate crops. If grouped geographically in an area it was quite easy to visit 6 or 7 reservoirs on a summer's day, and then edit previous reports and statements. So, visits to over 140 reservoirs over a year when one is well organised

is very easy. This of course would be very different if the dams were water supply reservoirs, with heights of 15-20 metres, close to populations, with large spillways, valve houses and tunnels, multiple valves which have to be exercised – often a day's visit or more is required to give justice to these sorts of structures.

Some engineers, through threats of not being reappointed to the Panel or by subject to sanctions, have been told to reduce their portfolio to 51 or less. That number has no basis in law, it is not specified whether these need to be the simple farmer type dams or the complex water authority dams, it takes no account how familiar the engineer is with the dam often over decades of visits to the site, and yet no one can tell the profession why this so called rule is being applied – no one will answer the call for clarity, they all hide behind the title of their organisation. Quite bizarre but another reason why engineers who need to know the reason for things happening are very uncomfortable with the situation in the UK at the moment. Recently the Regulator has issued a statement that they didn't say that there was a restriction!

In conclusion, the author states very sadly and clearly that he believes unless things change soon, there could well be a failure of a dam in the UK within the next 25 years which will involve loss of life. Failure in 1925 brought about new legislation – let's hope failures in 2025 do not cause us to react again.

The comments and opinions expressed in this paper are those of the author and in no way is attributed to any other engineer or author.

REFERENCES

[1] Waterworks Clauses Act (1863), HMSO

[2] Reservoirs (Safety Provisions) Act 1930, HMSO.

[3] Reservoirs Act 1975, HMSO.

[4] Water Act 2003

[5] Pitt Report

[6] Flood and Water Management Act 2010

[7] Reservoirs (Scotland) Act 2011

[8] Independent Reservoir Safety Report, March 2021

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EVALUATION OF DAM SAFETY PRACTICES IN MOZAMBIQUE THROUGH RISK ASSESSMENT

Luis Mabunda
Direcção Nacional de Gestão de Recursos Hídricos

Ignacio Escuder-Bueno
iPresas Risk Analysis

Adrián Morales-Torres
iPresas Risk Analysis

Davide Mancini
SGI Studio Galli Ingegneria S.r.l.

Sergio Romero
I iPresas Risk Analysis

Marco Silva[1]
iPresas Risk Analysis

ABSTRACT

Mozambique has a substantial number of medium and large dams serving purposes such as electricity generation, irrigation, and water supply, many of which have been in operation for over 50 years. While these dams contribute significantly to the country's development, their age and operational risks highlight the growing importance of dam safety.

As part of the project *"Preparation of the National Water Resources Development Program (NWRDP) and Assessment of Dam Safety Practices in Mozambique"* led by the National Directorate of Water Supply and Sanitation (DNAAS) and funded by the World Bank, a semi-quantitative risk assessment was conducted to evaluate current dam safety practices. Key activities included a review of the regulatory framework, development of a Risk-Based Screening Tool, and a Potential Failure Mode Assessment (PFMA) for six pilot dams (Macarretane, Chicamba, Muda, Nampula, Mugica, and Nacala).

The results validated the screening tool as an effective preliminary risk assessment method and led to the identification of 86 measures aimed at reducing risks, enhancing monitoring efforts, and addressing data uncertainties. These recommendations encompass both structural and governance measures, aiming to strengthen dam safety management across Mozambique's portfolio.

Keywords: Dams, PFMA, Screening, Risk, Assessment

[1]mpereira@ipresas.com, Spain

1. INTRODUCTION

The safe management and operation of dams are critical for fostering socioeconomic development, particularly in nations such as Mozambique, where these infrastructures play a pivotal role in electricity generation, irrigation, and water supply. However, the aging of numerous dams—many exceeding 50 years since their construction—intensifies the risks associated with structural degradation. These challenges are further compounded by the limited availability of updated technical information, insufficient monitoring mechanisms, and a lack of implementation of advanced risk assessment tools. Such shortcomings underscore the vulnerability of the existing regulatory framework governing dam safety in the country.

In response to these pressing issues, the project entitled “Preparation of the National Water Resources Development Program (NWRDP) and Assessment of Dam Safety Practices in Mozambique,” executed by the National Directorate of Water Supply and Sanitation (DNAAS) with financial support from the World Bank, sought to evaluate the current state of dam safety practices and propose targeted interventions. Central to this initiative was the development and application of a risk-based screening tool for Mozambique’s dam portfolio tested with six selected dams (Macarretane, Chicamba, Muda, Nampula, Mugica, and Nacala), alongside the performance of Potential Failure Mode Analyses (PFMA) for these infrastructures.

This methodology helps evaluate the dams current state and reduce the uncertainties regarding their behaviour, thanks to the exchange and sharing of relevant information on each of the evaluated dams as well as the joint review of the available information and the inspection of all the dams. Complementing these technical efforts, a comprehensive review of Mozambique’s regulatory framework was undertaken to identify critical gaps and opportunities for governance improvements.

The methodology included the facilitation of three regional workshops with Mozambique’s Regional Water Administrations (ARA`s) from the South, Center, and North regions. These workshops enabled a structured evaluation of potential failure modes for the selected dams, culminating in a set of recommended measures. These recommendations encompassed structural interventions, enhancements to monitoring systems, and governance reforms aimed at mitigating risks, advancing technical understanding, and fortifying dam safety management across Mozambique’s infrastructure portfolio.

The process, outlined in Fig. 1, begins with a review of the current dam safety regulatory framework, followed by the development and application of a risk-based screening tool and semi-quantitative risk assessment for six pilot cases. These form the basis for the roadmap, which will then be extended to other dams managed by the ARAs.

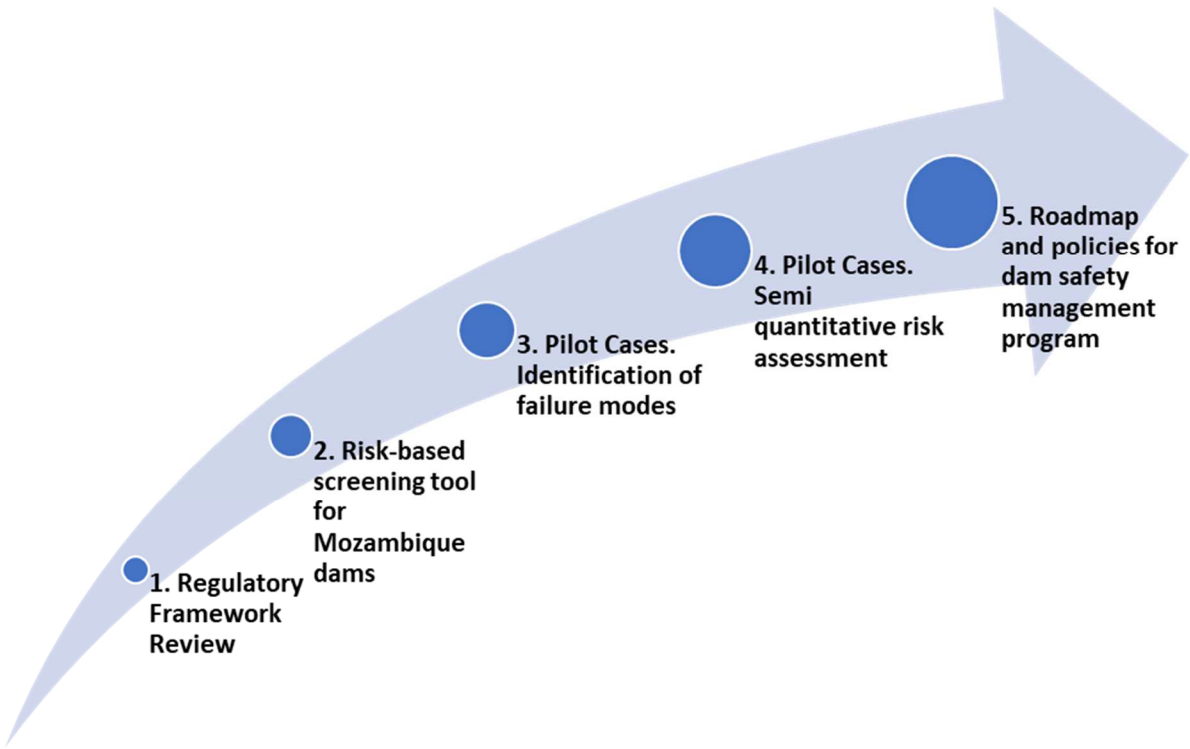


Fig. 1 – Process for assessment of dam safety practices in Mozambique.

2. REGULATORY FRAMEWORK REVIEW

The starting point for this project was the analysis of Mozambique's regulatory framework to identify key gaps and needs. This review focused on the legal, institutional, and regulatory aspects of dam safety, including the roles and responsibilities of relevant entities, and diagnosed shortcomings in normative, organizational, and technical components.

This section highlights the identified gaps and needs, addressing the legal framework, institutional arrangements, and technical considerations. Dam safety must be prioritized from both a management and regulatory standpoint, adapting to the context, objectives, and constraints. The summary diagram in Fig. 2 depicts the essential elements for effective dam safety management and legislation.

The review of Mozambique's regulatory framework reveals significant gaps in dam safety governance. Key issues include the lack of a specific regulatory body or dam safety units, absence of a national dam inventory, and no formalized guidelines for dam safety or risk management. Coordination challenges exist among water, land use, and local authorities, compounded by weak institutional capacity and unsustainable governance structures. Technical shortcomings include insufficient training, lack of emergency action plans, and the absence of risk-informed management practices. Addressing these gaps is crucial for harmonizing legal frameworks, enhancing cross-sectoral coordination, and improving dam safety standards nationwide.

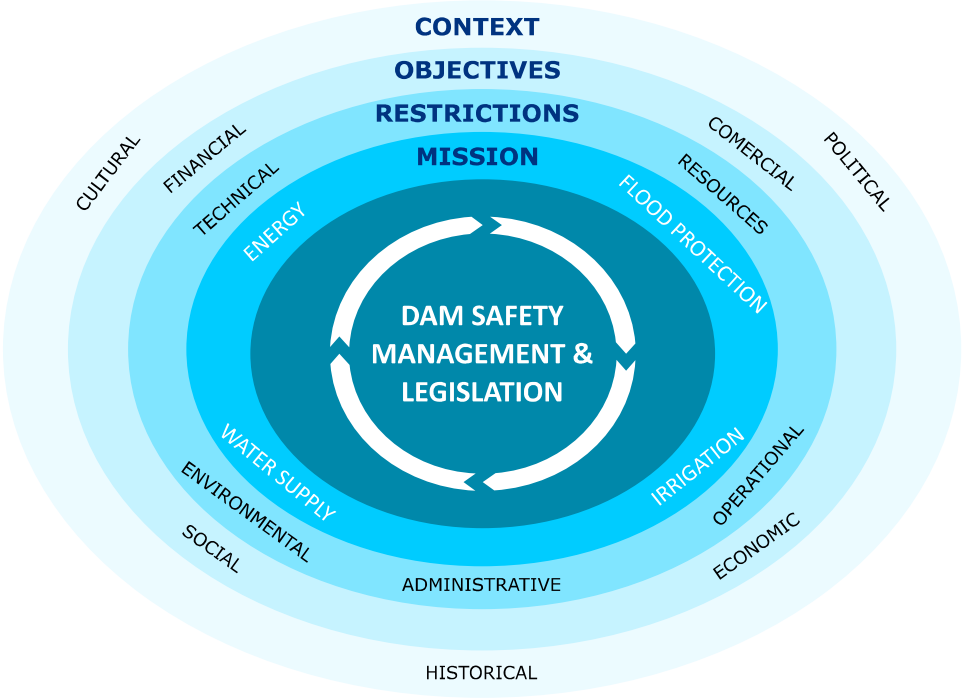


Fig. 2 – Dam safety management and legislation. Own source.

3. RISK-BASED SCREENING TOOL

The Risk-Based Screening Tool is designed to prioritize dam safety by assessing key safety factors, termed Risk Sub-Indexes, and compiling them into a Global Risk Index. This approach enables rapid evaluations, highlights vulnerabilities, identifies data gaps, and guides action priorities. Though effective for preliminary assessments, detailed analyses are required for precise risk diagnosis and mitigation, making the tool a foundation for broader risk management.

Tailored by the iPresas team for Mozambique, the tool integrates global risk assessment expertise with local needs. It evaluates hydrological and seismic hazards, design vulnerabilities, and potential consequences while incorporating an Uncertainty Index to address data quality gaps. By combining indices for Failure Probability and Consequences, the tool provides a comprehensive yet adaptable framework for dam safety management. The general structure of the screening tool developed is presented in Fig. 3, outlining the safety factors considered for assessment.

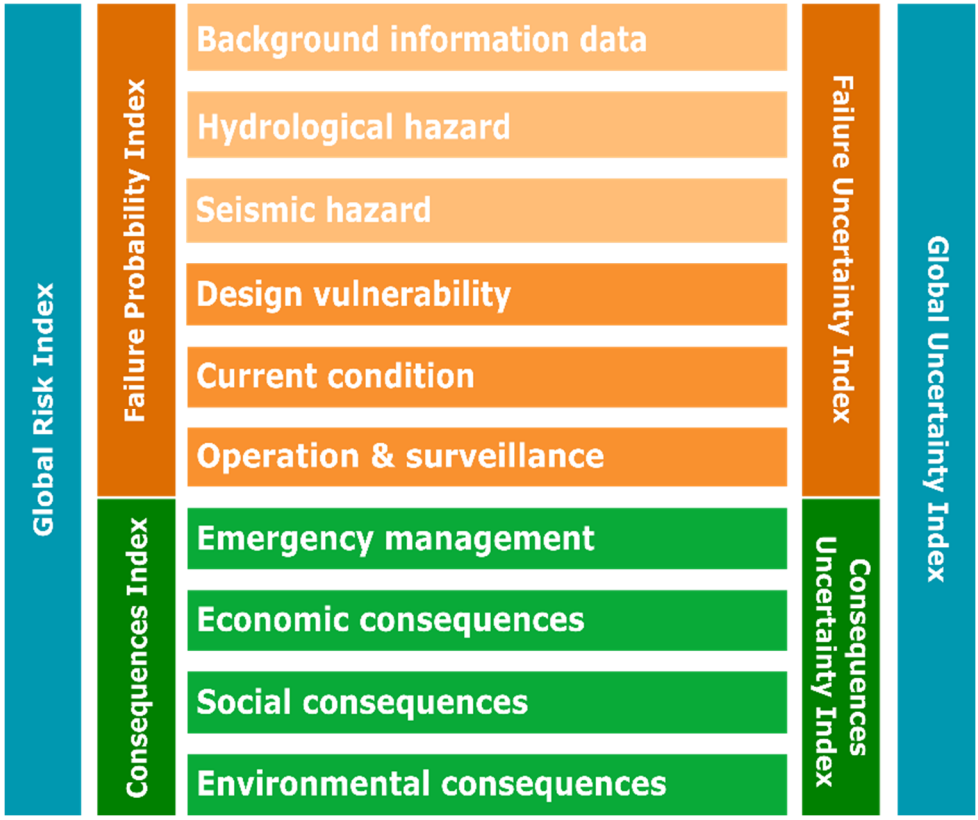


Fig. 3 – Structure of the applied screening tool.

The Global Risk Index (GRI) and Uncertainty Index (UI) are calculated as follows:

$$\text{GRI} = \text{Failure Probability Index (FPI)} \times \text{Consequence Index (CI)} \quad (1)$$

$$\text{UI} = \text{Uncertainty in Failure Probability} + \text{Uncertainty in Consequences} \quad (2)$$

The Failure Probability Index (FPI) measures the likelihood of dam failure by evaluating factors such as hydrological and seismic hazards, design vulnerabilities, and operational conditions. The Consequence Index (CI) assesses the potential impacts of failure, including emergency preparedness and economic, social, and environmental consequences. Together, these indices provide a holistic assessment of risk and uncertainty, with weights for each sub-index determined through stakeholder consultations, ensuring alignment with Mozambique's priorities.

The tool adapts established frameworks, such as those from the US Bureau of Reclamation (USBR) [1] and Brazilian dam safety standards [2], to address Mozambique’s specific needs. Key modifications include an Uncertainty Index to assess data quality, a sub-index for dam condition with safety triggers, and adjustments to account for upstream dams as a risk factor. For mixed dam types, the highest risk between concrete and embankment components is applied. The Consequence Index was also expanded to include emergency preparedness.

The tool identifies high-risk dams, prioritizes failure mode analyses, and recommends enhanced monitoring and data collection. It supports regular inspections, personnel training,

and emphasizes the critical need for comprehensive Emergency Action Plans to improve preparedness and resilience. The adaptations ensure the tool's relevance to Mozambique, providing a solid foundation for the development of a risk-informed dam safety management as outlined in Fig. 4

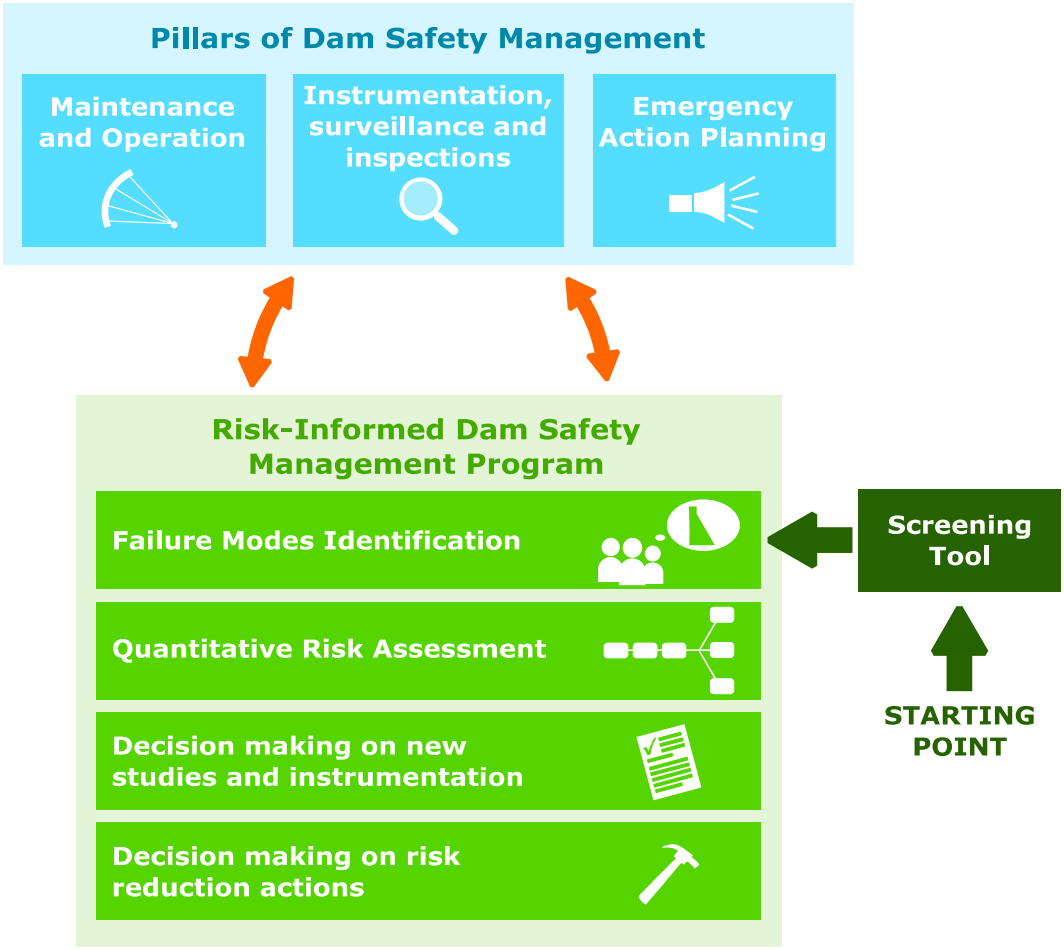


Fig. 4 – Development of a dam safety programme based on a screening tool [4].

The scores for each sub-index, including the Failure Probability Index (FPI), Consequence Index (CI), Global Risk Index (GRI), and Uncertainty Index (UI), were determined based on input from **50 participants**.

The results of the application of the Risk-Based Screening Tool are presented in both Fig. 5 and Fig. 6. Fig. 5 displays the Failure Probability Index against the Consequences Index, the product of which is the Global Risk Index, allowing an assessment of the infrastructures in these terms. Three areas are distinguished in the graph according to whether the risk is low, medium or high, the higher the failure probability and the consequences, the higher the risk. From the six dams evaluated, Muda dam has the highest risk, while Nacala has the lowest. Mugica and Chicamba have the highest consequence indices (60-70), while Nacala, Nampula, and Macarretane range between 40-50.

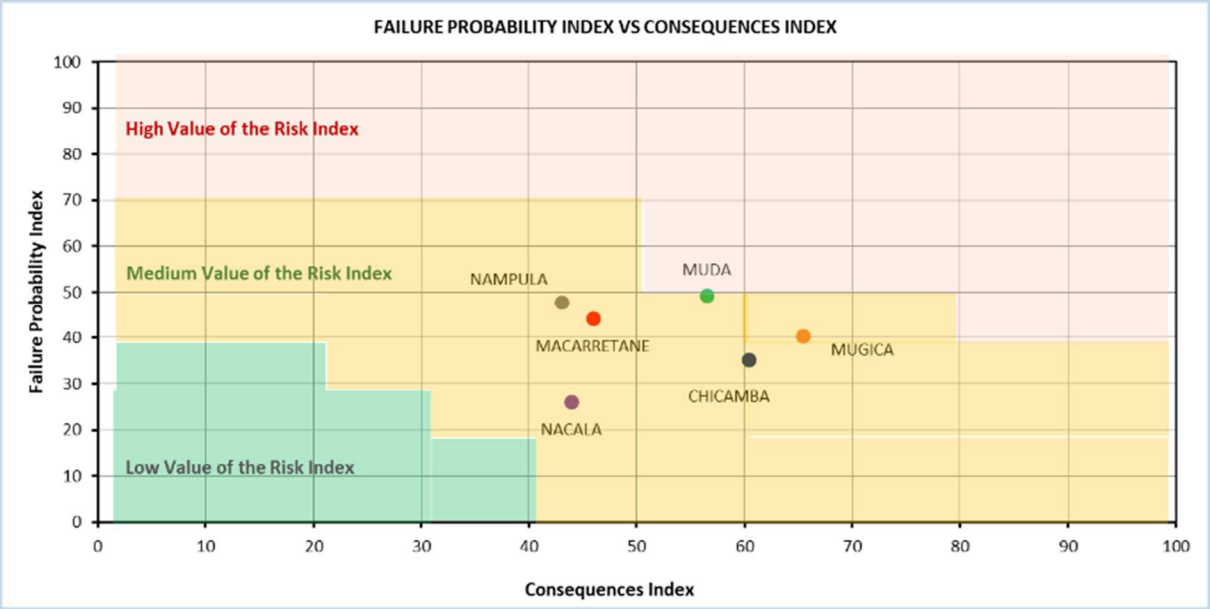


Fig. 5 – Failure Probability Index versus Consequence Index.

Fig. 6 shows the Global Risk Index against the Global Uncertainty Index, which allows to have a quick overview of the uncertainty of the risk results obtained. The higher the uncertainty, the stronger the recommendations to carry out more studies or intensify the instrumentation of the dam. Three areas are distinguished in the graph (green, yellow and red), each one indicating respectively whether the knowledge of the dam is considered sufficient, or whether, it is advisable or highly advisable to carry out more studies and intensify the instrumentation of the dam in order to improve the knowledge of the dam. Nampula, Chicamba, Mugica, and Muda have high uncertainty indices, strongly recommending further studies and improved instrumentation. Macarretane and Nacala are slightly lower but still require similar actions due to their risk and uncertainty levels.

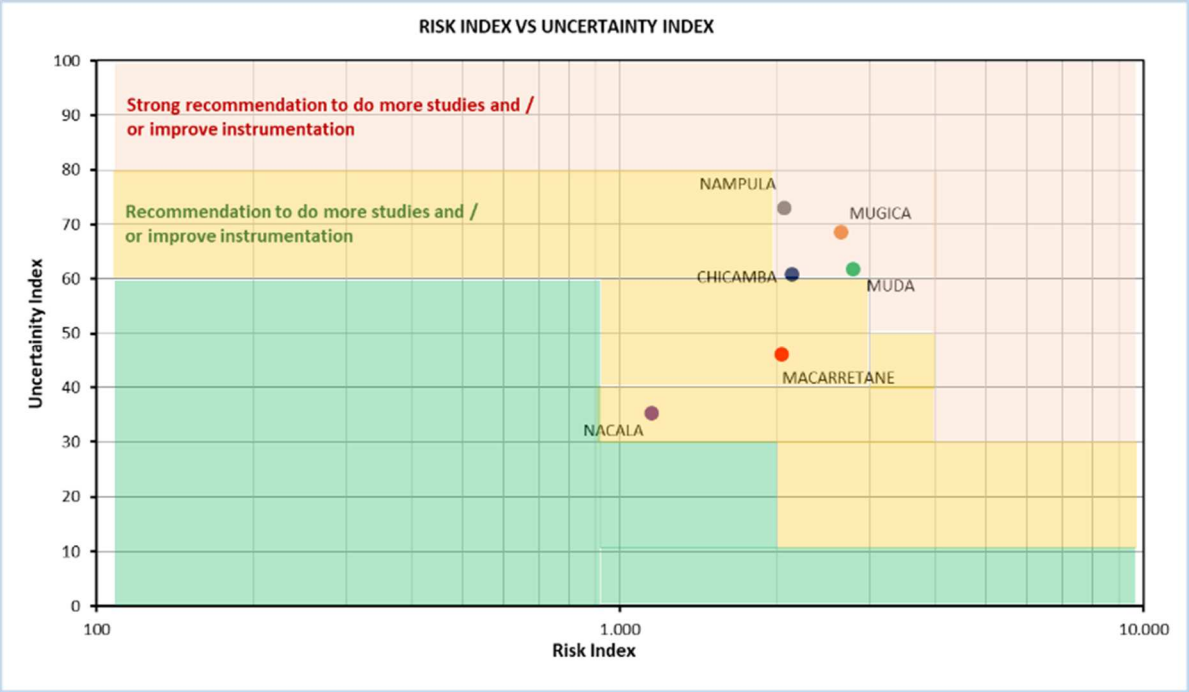


Fig. 6 – Global Risk Index versus Global Uncertainty Index.

4. POTENCIAL FAILURE MODE ANALISYS

The comprehensive risk assessment of the six pilot dams has been conducted using a semiquantitative risk analysis through participatory workshops, based on the specific risk assessment methodology for dams described in the Technical Guideline published by SPANCOLD [4], USACE [5] and the methodology described in [6]. These sessions brought together national and international experts, dam design and operation personnel, and representatives from organizations responsible for dam safety in Mozambique. Leveraging over 20 years of experience and specialized tools from iPresas, the methodology provided a structured approach to identify vulnerabilities and propose safety measures for hydraulic infrastructures.

The assessment process, illustrated in Fig. 7, combined knowledge exchange during workshops with a detailed review of available data, offering critical insights into the current state of dam safety. This collaborative approach was essential for a comprehensive evaluation and for guiding future risk management actions.

Fig. 7 – Methodology for the semiquantitative risk analysis.

The seminars were organized into three missions, each including visits to specific dams. The first mission, conducted with ARA-Sul during the week of April 15, involved a visit to the Macarretane dam. The second mission, with ARA-Centro, took place during the week of April 22 and included visits to the Chicamba and Muda dams. Finally, the third mission, conducted with ARA-Norte during the week of June 3, covered visits to the Nacala, Mugica, and Nampula dams. This analysis with approximately **64 participants**, underscored the importance of stakeholder involvement in the risk assessment process, ensuring a collaborative and comprehensive approach to dam safety management. Fig. 8 presents an example of some of the activities carried out during the in-person workshops.

Fig. 8 - Example of some activities of an in-person workshop.

4.1 PFMA Methodology

Step 1: Review of available information

A comprehensive review of the information provided by the ARAs was conducted for each dam, allowing the data to be enriched with insights from participants. This process included discussions on aspects where the available information was unclear or incomplete.

Step 2: Site Visit

After reviewing the information, a dam inspection was conducted to assess its current condition and identify potential issues. Special attention was given to critical points identified during the information review.

Step 3: Failure mode identification

The analysis of potential failure modes is a crucial step in risk assessment, helping dam owners understand safety and improve monitoring systems to detect key failure mechanisms. A failure mode (FM) refers to a sequence of events, starting with an initiating event, leading to failure and negative consequences as Fig. 9 presents.

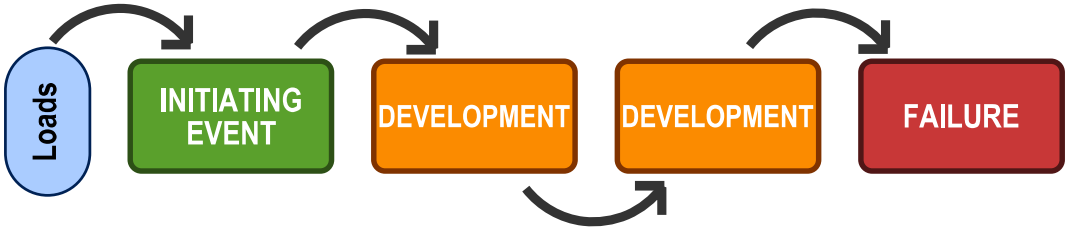


Fig. 9 – Generic structure of failure mode.

The process is divided into individual and group phases to ensure a comprehensive analysis of potential failure modes. In the individual phase, participants propose potential failure modes using PFM booklets (Fig. 10), which review all relevant factors. Each failure mode must be clearly described, including all necessary events for its development.

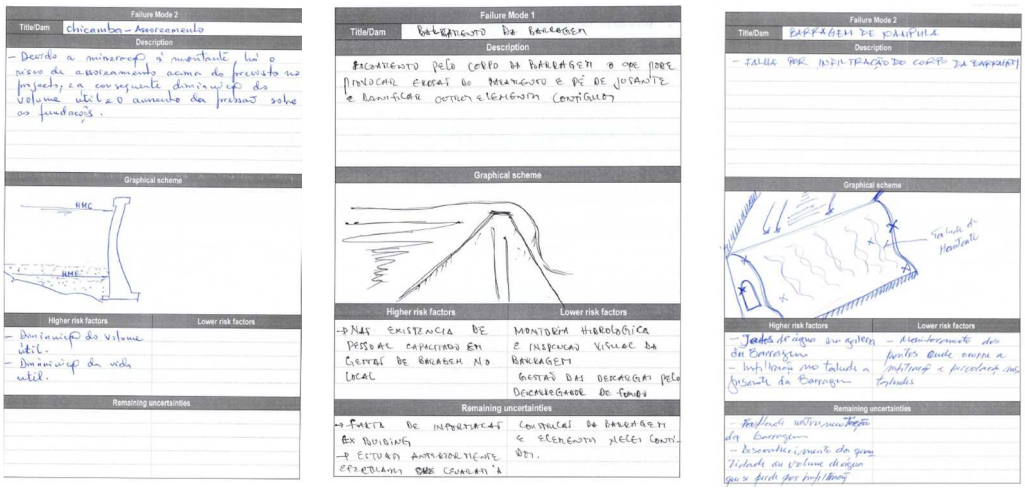


Fig. 10 - Example of failure mode identification booklets completed during the workshops.

Subsequently, the group work phase is conducted, during which participants present the identified failure modes with the objective of consolidating them and reaching a consensus within the group. This phase is coordinated by a facilitator, who ensures adequate time is allocated to analyse each potential failure mode in detail. An example of the outputs from this PFM identification is shown below in Fig. 11, where the sequence of development of the failure mode is mapped out.

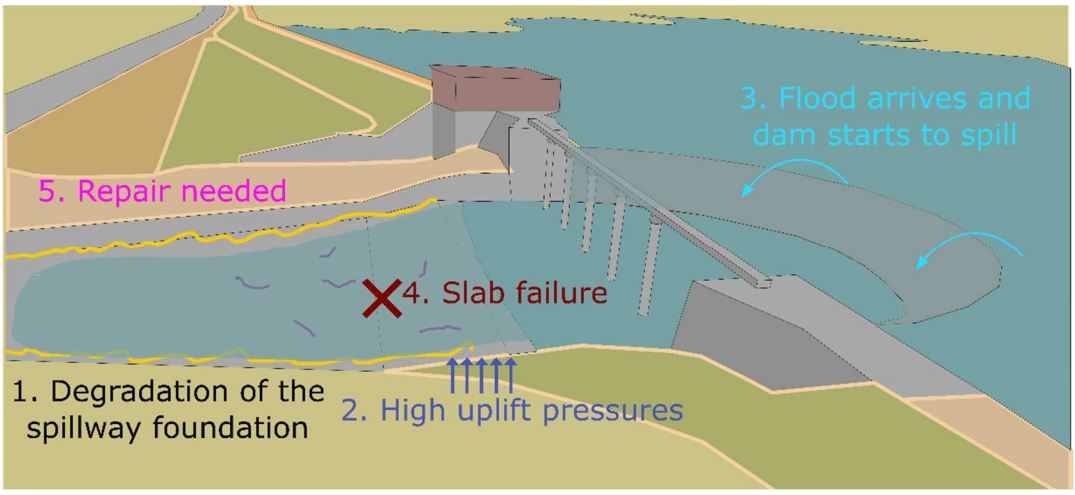


Fig. 11 – Example of failure scheme.

Step 4: Failure mode Classification

The semi-quantitative classification serves as the foundation for proposing measures and new studies.

The semi-quantitative classification assigns categories to failure probability and consequences for each identified failure mode. Six categories for failure probability (P1 to P6) and consequences (C1 to C6) were defined, ranging from least to most severe.

An example of how the classification is conducted can be observed in Fig. 12, where five hydrological, one structural, one seismic and two of other type failure modes identified were classified.

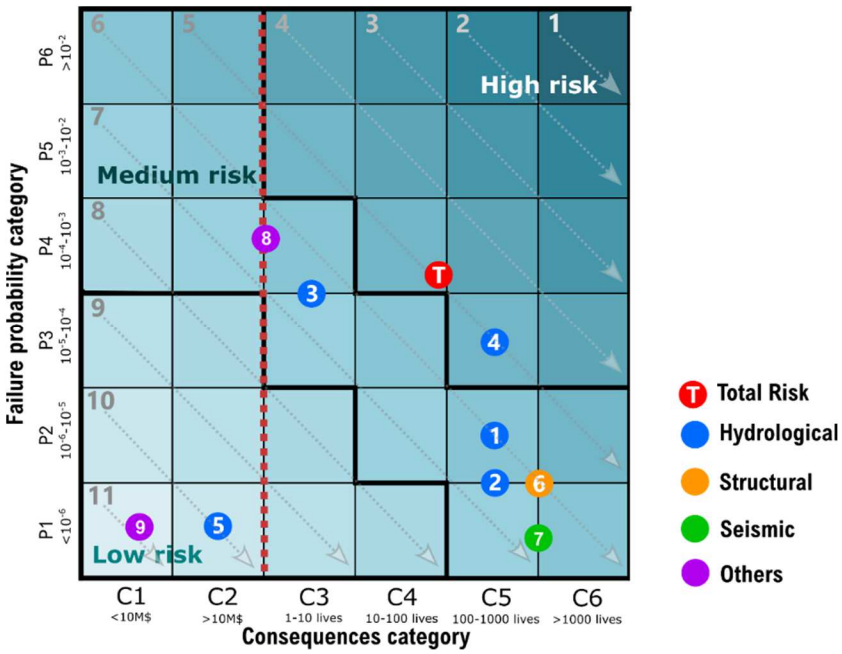


Fig. 12 – PFM classification matrix.

Step 5: Recommendations

The final stage of the semi-quantitative analysis involves proposing measures to mitigate risks for each identified failure mode. These include actions to improve monitoring, recommend new studies to address uncertainties, and enhance understanding of the dam system and also being the basis for the roadmap.

4.2 PFMA Results

A comparative risk classification of six dams is shown in Fig. 13. Four dams (Chicamba, Muda, Mugica, and Nampula) are in the high-risk zone, while Macarretane and Nacala are in the medium-risk zone.

Muda has the highest risk, due to a past overtopping event and high potential loss of life. Nacala, with lower failure probability and past rehabilitations, has the lowest risk, despite significant potential consequences. Macarretane has high failure probability but low consequences, while Mugica poses the highest social risk, highlighting the need for improved monitoring and maintenance.

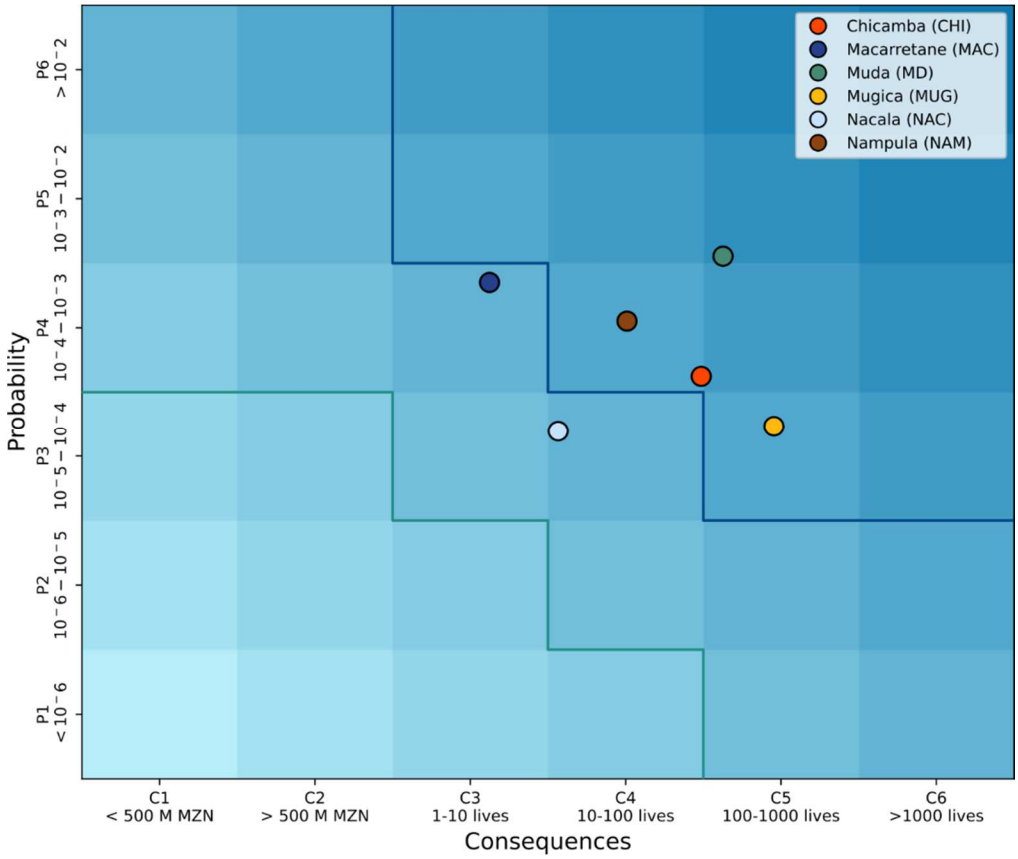


Fig. 13– Total Risk of each dam. SQRA classification.

For the six dams, a total of 44 failure modes were identified during the potential failure mode analysis workshops.

The analysis shows strong consistency between the two risk classification methods, confirming the reliability of the assessment. Macarretane, Mugica, Chicamba, and Nampula dams share similar risk levels, with Chicamba and Mugica having higher consequences due to nearby populations, while Macarretane and Nampula show higher failure probabilities. Nacala dam is classified as the lowest risk. Muda, Chicamba, Nampula, and Mugica are high-risk dams requiring urgent action to ensure safety and protect downstream communities. Implementing the proposed measures is essential to address vulnerabilities and ensure the long-term safety of the dams.

5. DEVELOPED ROADMAP

This roadmap aims to integrate risk-informed decision-making within the ARAs. Fig. 14 illustrates the proposed roadmap, describing how it is divided into short, medium, and long-term actions. Additionally, it simultaneously highlights the governance measures that apply uniformly across all dams.

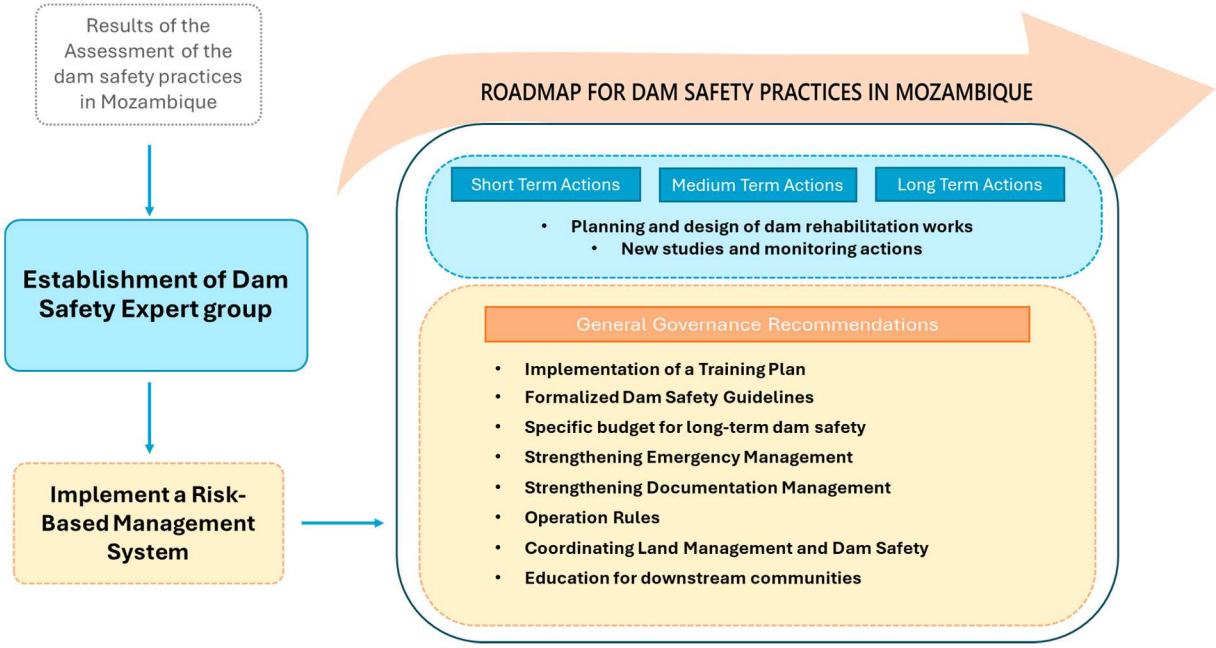


Fig. 14 – Proposed roadmap to develop a risk-inform dam safety program in ARA´s.

The goal of this Roadmap is to prioritize and plan the previously defined measures for each dam, serving as a guide for future actions. This roadmap is based on the review of the regulatory framework and incorporates the 86 proposed measures, which will guide the subsequent steps for improving dam safety and ensuring the long-term reliability of Mozambique's dams. These measures include both structural improvements, such as upgrades to civil works for outlet systems, and non-structural actions, such as conducting new hydrological studies.

Fig. 15 serves as the starting point for the roadmap to improve dam safety in Mozambique, prioritizing the previously identified measures for each studied dam. The prioritization is based on levels ranging from 1 to 11, with measures associated with high-risk failure modes considered the most urgent and scheduled for the short term. Medium-risk failure modes will be addressed in the medium term, while measures for low-risk failure modes will be planned for the long term.

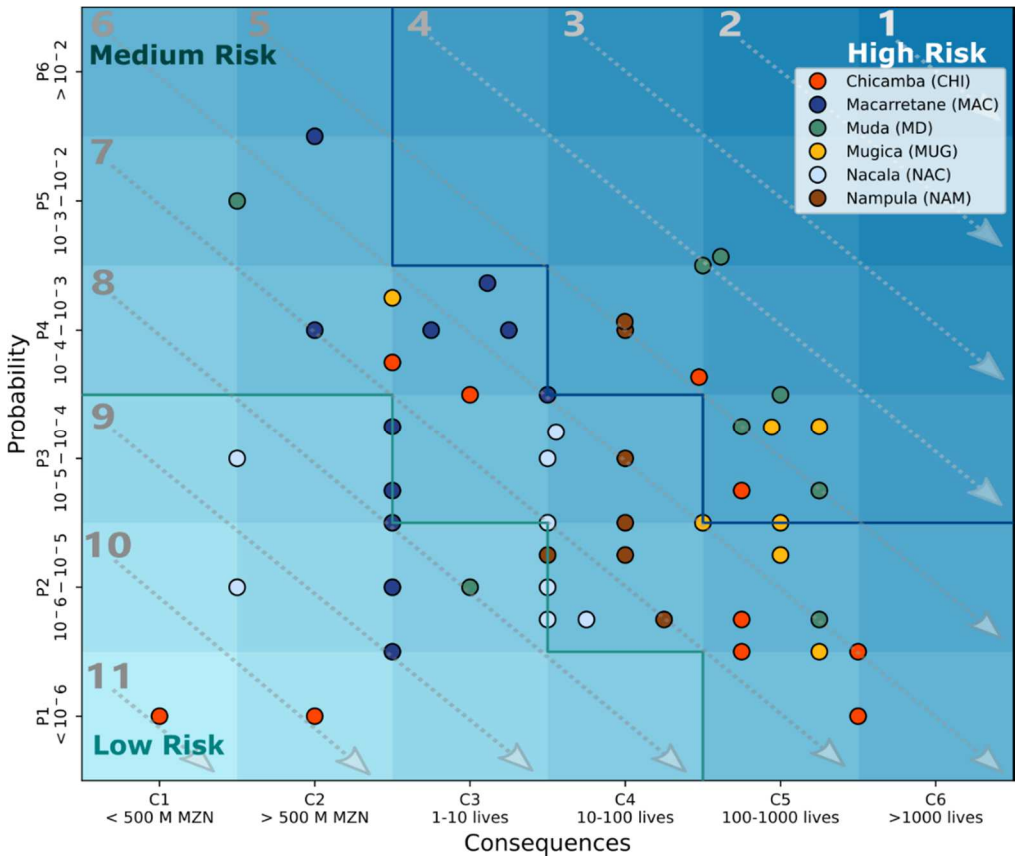


Fig. 15 – Semiquantitative classification of the failure modes of all the dams and Priority levels distribution.

6. CONCLUSIONS

This study provides a structured approach to dam safety management in Mozambique, emphasizing the application of a risk-based screening tool and semi-quantitative risk assessment (SQRA) for the identification and mitigation of risks associated with dams. The research highlights the importance of a systematic methodology for evaluating dam safety, integrating both technical and operational perspectives, and fostering collaboration among national and international stakeholders. Through the analysis of six pilot dams, key failure modes and vulnerabilities were identified, which served as the foundation for proposing targeted mitigation measures.

The comparison of the screening tool results with those obtained through the SQRA demonstrated a high degree of consistency, validating the effectiveness of the proposed risk assessment framework. The findings support the need for prioritizing high-risk dams and implementing appropriate measures to address structural and operational deficiencies, thus ensuring the safety of downstream communities and the integrity of critical infrastructure.

The proposed roadmap, developed based on the risk assessments, offers a clear path forward for enhancing the regulatory framework for dam safety in Mozambique. It emphasizes the necessity of continuous risk monitoring, the implementation of preventive measures, and the allocation of resources to manage vulnerabilities effectively. This project contributes to advancing dam safety practices in Mozambique, offering a reliable and comprehensive tool for future risk management efforts and guiding policy and decision-making in the context of dam safety.

REFERENCES

[1] USBR, United States Bureau of Reclamation, 2011. Dam Safety Public Protection Guidelines – A Risk Framework to Support Dam Safety Decision-Making, United States.

[2] J. M. Alonso, R. Ribeiro, I. Escuder, I. Guerra, M. Sánchez, and A. Morales. (2018). “Plan of Strategic Actions for the Rehabilitation of 164 dams of Brazil based on the Brazilian Risk-based Methodology,” in 26th International Commission on Large Dams World Congress.

[3] CWC, Central Water Commission. Ministry of Water Resources India, 2019. Guidelines for Assessing and Managing Risks Associated with Dams, India.

[4] SPANCOLD, Spanish Committee of Large Dams, 2012. Risk Analysis applied to the dam safety. Technical Guide Nº 8, vol. 1, Spain.

[5] USACE, United States Army Corps of Engineers, 2014. Safety of dams – Policy and procedures. Engineering and design, United States.

[6] I. Escuder-Bueno, A. Morales-Torres, F. D. García-Brook & H. C. Yarritu-Sánchez. (2024). Beyond conventional practices: Enhanced methodology for dam potential failure modes screening. 92nd ICOLD Annual Meeting – New Delhi, India.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

PROPOSAL TO OBTAIN A CORRELATION OF CONCRETE PROPERTIES FROM STANDARDIZES RADIOLOGICAL TESTING

João B.M.Carmo[1]
VALE S.A.

Luiz C. Meira-Belo
CDTN

Fernando S. Lameiras
CDTN

José M. Filho
UFPR

ABSTRACT

This work aims to present the methodology implemented to obtain different exposure conditions for conventional and roller-compacted concrete molded in the laboratory, as well as in cores extracted from a containment structure downstream of VALE in Minas Gerais. This research was developed at the Nuclear Technology Development Center – CDTN in Belo Horizonte and presented in a thesis recently in 2023. In this work, a test methodology was adopted that subjected concrete samples to a standardized exposure to ionizing radiation with the nature of the source of cobalt 60, and which could be adopted as a standard for future tests with comparative parameters. The physical property density was evaluated in the thesis, but other properties of concrete could be studied, such as compressive strength, tensile strength and modulus of elasticity. The concretes analysed were composed of gneissic rock aggregates and industrial hematite residue and in all concretes, cement was used with the addition of ground blast furnace slag, classified by Brazilian standards as type III.

Keywords: Dams, Concrete, Radiologic testing, Tailings

[1]jboscomc70@hotmail.com, Brazil

1. INTRODUCTION

In mid-2019, the mining company VALE S.A. began construction of containment structures in Roller Compacted Concrete (RCC) to protect the areas downstream of the dams, which, in a critical state of stability, could cause accidents due to potential ruptures with enormous repercussions. This initiative sought fast construction alternatives that could generate the best benefit in terms of safety for those affected, with the best executive quality available at the time. The definition of RCC construction allowed the development and research of a concrete that, in addition to providing an interesting construction speed, generated an increase in the density of this concrete with the use of aggregates originating from industrial waste from the iron ore industry in the region of the works. This increase in density was essential to ensure a reduction in the volume of the structure of around 20%, consequently reducing the execution time of the works, which at that time was crucial to achieving the objective of those works. However, during the execution of the tests, the possibility of using this heavier-than-normal concrete as shielding of structures against the effects of ionizing radiation was envisioned. This was the motivation for developing the thesis at the *Centro de Desenvolvimento da Tecnologia Nuclear* CDTN in Belo Horizonte, Brazil. These tests carried out at CDTN were published by [1] and the methodology used to determine the attenuation coefficient in concrete specimens will be highlighted.

2. DEVELOPMENT

2.1. Definitions

The principle of obtaining the attenuation coefficient for ionizing radiation was considered the starting point for obtaining the property that could express the capacity to shield photons emitted under a standardized condition and that was also reproducible through a standard procedure. The linear attenuation coefficient (μ) is a quantity used to determine the probability of a radiation photon interacting with the molecular structure of a given material per unit path length. This coefficient is essential for calculating and experimentally determining radiation shielding and depends on the physical state of the shielding material. In the case of concrete, there can be a large variation in material density due to variations in composition, humidity, and moisture (Bashter, 1997). According to Kaplan (1989, cited in Bashter, 1997), the linear attenuation coefficient can be expressed as a mass attenuation coefficient, which is the linear attenuation coefficient per unit mass of the material. In the case of concrete, many physic-chemical properties could affect the mean linear attenuation coefficient. These include the concentration and type of cement, the curing time, the mineral composition and grain size distribution of the aggregates, as well as the moisture content, residual humidity, and methods

of homogenization and compaction of the material. The presence of cracks, inclusions (liquid or gaseous), and vacancies could affect the shielding properties of the material [1].

2.2. Methodology

The methodology proposed here is based on the principle that by maintaining a standardized distance between a given test specimen and a controlled radioactive source, a procedure can be established to be adopted and, in a certain way, support a normative proposal so that other tests can be carried out in any other laboratory and the results obtained can be correlated with the properties of the concrete. Thus, as it was easy to carry out the testing campaign carried [1], it is intended to offer this methodology as a standard to support new research in the future.

2.3. Test standardization

The attenuation coefficient of photons absorbed by a material exposed to a radioactive source can usually be determined by the following expression, according to the Beer-Lambert law, given by:

The following example presents an equation in a single line:

$$I = I_0 \, e\text{^}(-\mu x) \tag{1}$$

where μ is the linear attenuation coefficient, x is the thickness of the absorbing material layer, and I and I_0 are the intensity of the radiation field at a predefined distance from the radiation source with and without the absorbing material, respectively. The coefficient μ depends on the effective atomic number of the medium and the energy of the radiation beam. In the case studied, concrete samples made in the laboratory and obtained from cores extracted from the structure were used.

The radiation field was provided by a multipurpose 60Co panoramic irradiator, model IR 214, manufactured by MDS Nordion, installed at the Gamma Irradiation Laboratory (LIG/CDTN/CNEN). For practical purposes, the 60Co source presents a pair of gamma photopeaks with energies of 1,173.2 keV and 1,332.5 keV, with a half-life of 5.27 years. Based on this principle of subjecting the concrete to this exposure to the defined source, the distance of 50 cm from the wall of the irradiator chimney to the closest face of the test specimen was determined as constant. The samples were predetermined with faces of 15 cm and/or core diameter also of 15 cm. The radiation recording was made using radiochromic film of the same dimension as the cross-section of the test specimens. GafchromicTM EBT3 LOT 01252202 films, validated until 2024-01-25, were used to obtain detailed images of the absorption profiles to determine the absorbed dose. The irradiation dose value was standardized at 30 Gy.

2.4. Future research

This methodology aims to standardize tests to correlate some concrete properties with the radiological attenuation coefficient obtained in the test in question. The study carried out by CARMO 2023 presented the correlation of the attenuation coefficient with the density of different concretes. It is understood that the same standard procedure could be used to obtain other concrete properties, including compressive strength, tensile strength, modulus of elasticity, permeability, and thus be able to establish traceable parameters. For this research to be applied in construction sites, where the need to know such properties is urgent, with the removal of cores from structures or when the use of portable equipment is possible, it may be possible to estimate values for these desired properties in a clear and expeditious manner.

2.5. Results obtained

The results obtained in the research developed at CDTN with the made concrete and cores extracted from the RCC structure are presented below, correlated with the densities obtained in the field.

Considering the presence of several elements in the raw image that do not contribute to the relevant information on the distribution of radiation doses recorded by the radiochromic film, only the central region was used to determine the average dose recorded, as well as the number of intensities counts per pixel per dose interval. This description is depicted in Figure 1, where the left plot represents the central region of the slice, and the right plot illustrates the distribution of pixel counts by dose range.

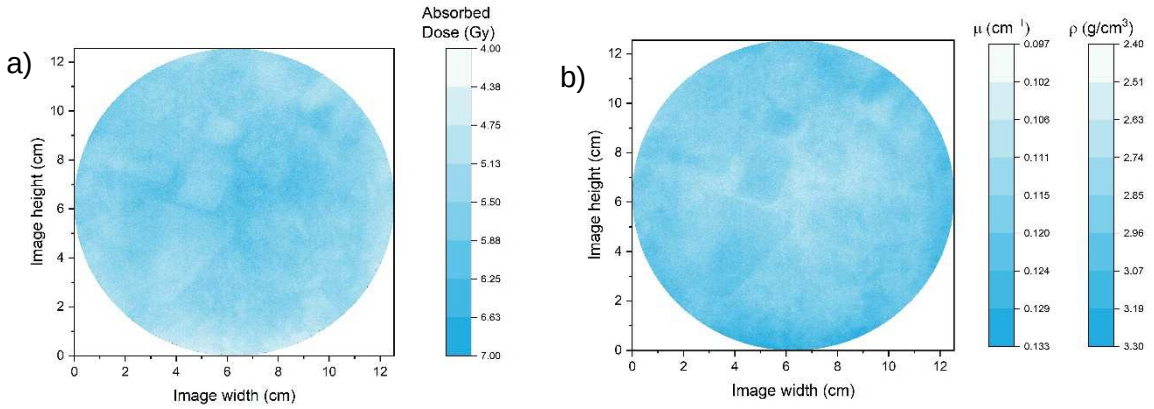


Figure 1. Region of interest of the radiographic record of 15 cm of RCC dam core converted to a) absorbed dose values and b) linear attenuation coefficient and density.

Table 1 presents the analysis of the acquired data from the central portion of the digitized image. The data was organized to determine the contribution of each range to the average absorbed dose recorded by the radiochromic film. For a real conventional absorbed dose value of 30.0 Gy, the evaluated average dose was 5.34 Gy, indicating an average attenuation of

82% of the radiation field intensity by the RCC sample. This corresponds to an average attenuation of 5.5% per cm^3 of RCC concrete.

In fact, the summation of the i-th density terms corresponds to the average density.

Table 1. Consolidated information obtained from the region of interest of Figure 4.

Absorbed dose range center (Gy)	Pixels by range	Fraction of pixels per range	Absorbed dose by range (Gy) (± 2%)	μ (cm^{-1}) (± 0.002)	ρ (g/cm^3) (± 0.05)
2.65	4	0.00000	0.00	0.162	3.99
3.15	44	0.00004	0.00	0.150	3.70
3.65	334	0.00027	0.00	0.140	3.46
4.15	2,819	0.00228	0.01	0.132	3.25
4.65	93,958	0.07590	0.35	0.124	3.06
5.15	610,438	0.49311	2.54	0.117	2.90
5.65	490,800	0.39647	2.24	0.111	2.74
6.15	38,361	0.03099	0.19	0.106	2.60
6.65	1,051	0.00085	0.01	0.100	2.48
7.15	106	0.00009	0.00	0.096	2.36
7.65	9	0.00001	0.00	0.091	2.25
8.15	4	0.00000	0.00	0.087	2.14
8.65	1	0.00000	0.00	0.083	2.04
9.15	1	0.00000	0.00	0.079	1.95
Total	1,237,930	1	5.34		

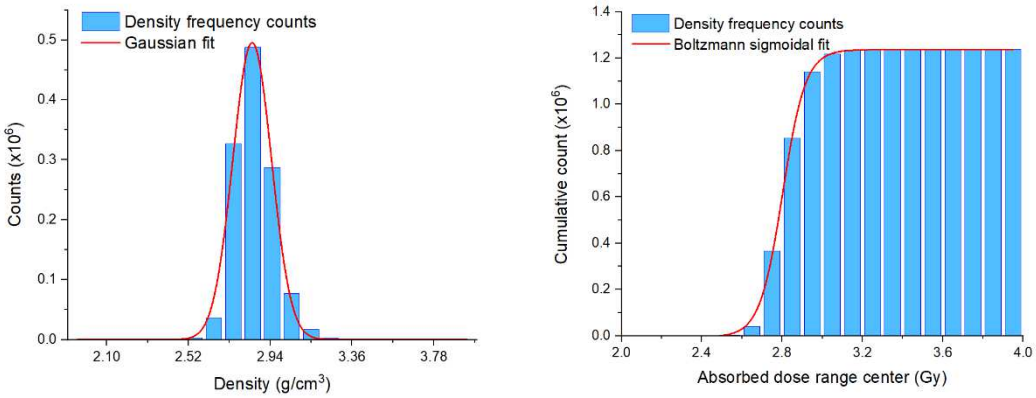


Figure 2. Graphical representation of the data in Table 1. a) Frequency counts of density values distribution (blue) and the respective Gaussian adjustment (red). b) Cumulative frequency counts of density values distribution (blue) and the respective Boltzmann sigmoidal adjustment.

3. CONCLUSIONS

Based on the results obtained in the tests carried out in the CDT Center in Belo Horizonte, the aim is to make available the standard methodology used in the tests carried out to complete the thesis presented by CARMO 2023, and thus be able to determine other properties of hardened concrete in addition to the specific mass. Through correlations with the standardized exposure of photons from a reliable and safe source of ionizing radiation. These properties can be studied, as well as laboratory and field tests capable of predicting property values without destroying test specimens or the structure under analysis.

4. ACKNOWLEDGMENTS

The authors are grateful to Centro de Desenvolvimento de Tecnologia Nuclear (CDTN), Conselho Nacional de Desenvolvimento Científico e Tecnológico (CNPq), Coordenação de Aperfeiçoamento de Pessoal de Nível Superior (CAPES), Fundação de Amparo à Pesquisa do Estado de Minas Gerais (FAPEMIG) and Comissão Nacional de Energia Nuclear (CNEN) for supporting these studies. Thanks are also extended to VALE S.A. for contributing to the development of concrete technology in the construction of downstream containment structures in RCC in Minas Gerais Brazil and all the partnerships for technology development and Brazilian engineering.

REFERENCES

[1] CARMO, J. B. M. (2023). Evaluation of Roller Compacted Concrete – RCC Composed with Hematite Waste for Massive Structures – Comissão Nacional de Energia Nuclear – CNEN –Centro de Desenvolvimento da Tecnologia Nuclear – CDTN – Belo Horizonte – Brazil - PhD Thesis, 2023.

[2] PEREIRA, F., CARMO, J. B. M., ANDRADE, W.P., Use of Roller Compacted Concrete – RCC for Containment of Tailings Dams. COBRAMSEG – Camburiu – Brazil - 24.

[3] ANDRADE, W.P., FURNAS - E.Concretos: Massa Estrutural, Projetado e Compactado com Rolo. 1st ed. São Paulo – Editora Pini, 1997.

[4] LADEIRA, L.C.D., PINTO, F.C., Relatório de Análise de Segurança do Laboratório de Irradiação Gama. Capitulo 1 – Centro de Desenvolvimento da Tecnologia Nuclear – CDTN. Belo Horizonte, 2002 (RASIR.LIG.CDTN Rev1) – Restricted Access.

[5] CARMO, J. B. M., LAMEIRAS, F.S., FILHO, J.M., ANDRADE, W.P., RCC Roller Compacted Concrete with Hematite Aggregate REM, Int. Eng. J., Ouro Preto, 76(3), 247-254, jul.sep 2023.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

EMERGENCY ACTION PROTOCOL IMPLEMENTED BY INEA AT THE AÇUDE DA CONCÓRDIA DAM

Spitz Dias, Fernanda[1]
State Environmental Institute – Inea/RJ

Pereira Batista Amaral, Daniele[2]
State Environmental Institute – Inea/RJ

Alves da Silva, Milena[3]
State Environmental Institute – Inea/RJ

Bielschowsky, Cauê[4]
State Environmental Institute – Inea/RJ

Henrique Zampieri Pereira, Thiago[5]
State Environmental Institute – Inea/RJ

ABSTRACT

This work presents the emergency actions taken by Rio de Janeiro State Environmental Institute (in Portuguese, Inea), regulation body, towards Açude da Concórdia dam, located in Valença city, State of Rio de Janeiro, after internal erosion (piping) identification in the dam mass. Considering these issues are recurring causes of failure in this type of structures and, also considering lack of definition about dam owner and based on prevention principle, Inea, after problem identification and awareness, summoned the Civil Protect and Defense Agencies (State and Municipal) and Valença City Hall, aiming to coordinate and take actions to ensure safety of the dam and downstream population. The Açude da Concórdia dam is located in a conservation unit, the Açude da Concórdia Municipal Natural Park. In Park's Contingency Plan scope, the institute prepared and included an emergency action protocol, laying at defining downstream vulnerability points and regions where it is considered there is not enough time for competent authorities to intervene due to a possible dam failure. This protocol also included a preliminary delimitation of a simplified flooding map, aiming to guide authorities decision-making and Civil Protection and Defense Agency's task in population evacuation in case of an imminent dam accident. It was also considered that, if an eventual accident has occurred and it has been declared an emergency situation, downstream population should be evacuated to safe shelter locations and highways and local roads should be interdicted, aiming to preserve human lives. After identifying new internal erosion points, Inea performed emergency measures to minimize dam failure risk by emptying the reservoir, aiming to avoid damage and impacts on population downstream. Finally, it was recommended to hire a specialized and qualified company or professional(s) to verify and certify real and current dam safety conditions and indicate the necessary and appropriate interventions to recover its structure to guarantee and mitigate risks to the population residing downstream.

Keywords: Emergency, Dam Safety, Prevention.

[1] fernandasd.inea@gmail.com, Brazil
[2] danielepba.inea@gmail.com, Brazil
[3] milena.inea@gmail.com, Brazil
[4] caue.inea@gmail.com, Brazil
[5] thiagopereira.inea@gmail.com, Brazil

1. INTRODUCTION

Dams perform a crucial role in socioeconomic development of a society, especially in Brazil, a country of continental dimensions, by promoting water and energy security. Currently, the main discussions surrounding dams are related to their management aspects: design, construction, and maintenance, focusing on safety, since each dam presents an inherent risk given the great destructive potential due to the water volume that could be released in case of a collapse [1].

Dam failures often occur during flood periods, and the main causes of failure in earth dams are overtopping and internal erosion. Overtopping is the water overflow from reservoir over its crest, leading to the dam's embankment rupture. The primary factors associated with overtopping are insufficient spillway capacity, spillway obstruction, or unforeseen high inflow upstream. Regressive erosion, also known as "piping", consists of soil particles disaggregation within the dam, caused by action of water seepage through embankment or its foundation. Key factors contributing to piping include poorly compacted layers, intake pipe system issues, internal drainage system deficiencies, presence of permeable layers, or foundation fractures [2].

The 1988 Brazilian Federal Constitution [3] enshrined in its Art. 225 the right of all citizens to enjoy an ecologically balanced environment, essential for ensuring population quality of life, recognizing it as a fundamental right of individuals and society. It also established environmental protection as public authorities’ duty [4].

In “Environmental Law” there are principles that shape its regulations and applications, being observance and compliance essential for disaster prevention. The prevention principle stands out as one of the most crucial, especially regarding environmental protection, when aiming to anticipate measures to mitigate environmental risks. This requires an understanding of potential damages that certain actions may cause to an environment, so the necessary measures can be taken to avoid them.

In this context, this article presents the actions taken out by the Rio de Janeiro State Environmental Institute (in Portuguese, Inea), the environmental agency responsible for water resource management and dam safety surveillance for industrial waste dams, which acted as a licensing body, and water accumulation dams located in rivers under state jurisdiction in Rio de Janeiro (RJ, Brazil), excluding dams which main purpose is energy generation, in partnership with civil protection and defense agencies, after internal erosion identification in

Açude da Concórdia dam located in Valença (RJ, Brazil), aiming to minimize the risks of structural collapse and to protect downstream population.

2. DAM LOCATION AND TECHNICAL CHARACTERISTICS

The Açude da Concórdia dam is located in the Açude da Concórdia Municipal Natural Park (in Portuguese, PANAMAC) in the city of Valença, in Rio de Janeiro State, Brazil, as shown in Fig. 1.

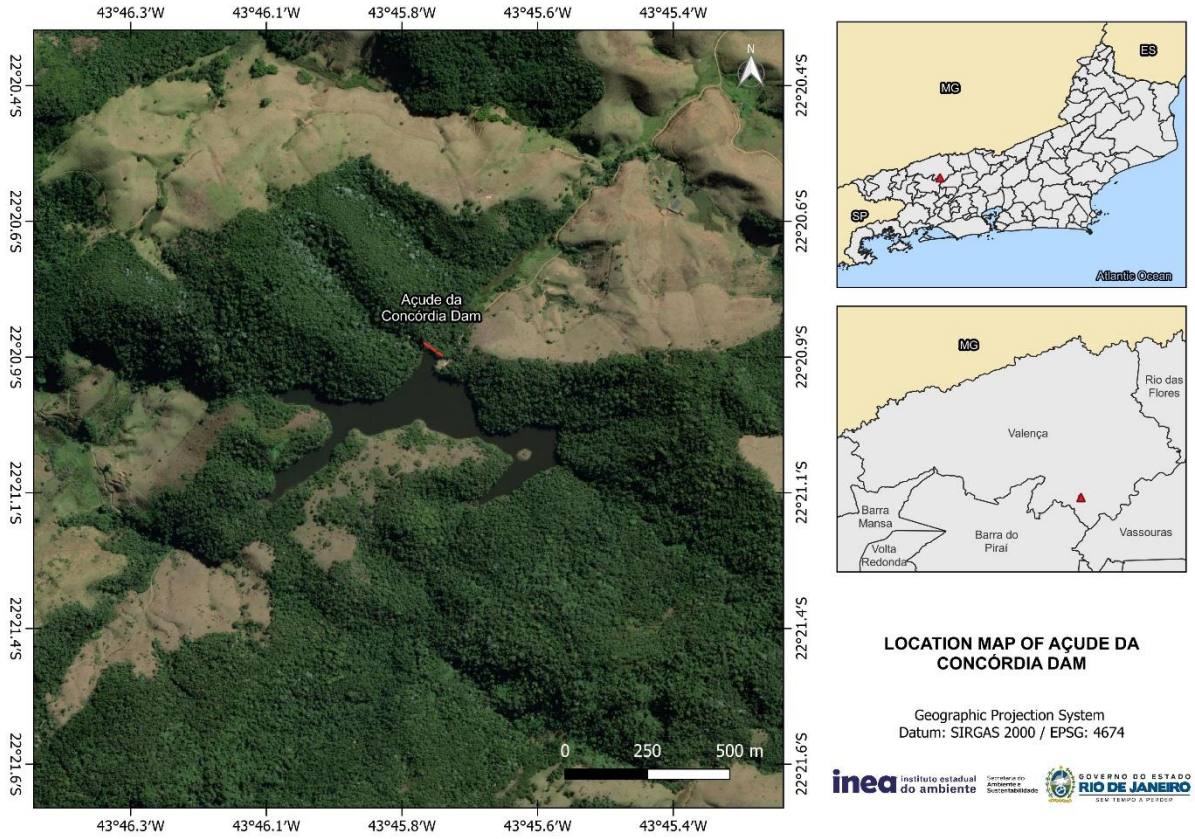


Fig. 1 – Location map of Açude da Concórdia dam

PANAMAC is an integral protection conservation unit established by Municipal Decree no. 65 on September 12, 2001 [5], located approximately 130 km from state capital, and covering an area of 22.99 hectares. Its vegetation is classified as Atlantic Forest, specifically semideciduous seasonal forest. Since January 2018, it has been managed by Inea, considering the assignment and management agreement between Inea and the Valença Municipal Government (in Portuguese, PMV).

The park is a city tourist attraction, open to the public from Thursday to Sunday, with an average of 350 visitors per month. It features various hiking trails and waterfalls, an entertainment area to barbecue, and fitness equipment.

The Açude da Concórdia dam is an earth dam constructed by the Santa Mônica Baron in the mid-1890s. Its reservoir has always served a landscape purpose and was also used to supply water to Valença city until 1980s [6].

The dam does not have design or technical documentation (as-built), making it hard to determine its exact dimensions. The earth embankment height was estimated between 10 and 12 meters, based on visual analyses and reservoir depth data.

Basic measurements taken during an on-site inspection made it possible to determine the dam crest (Fig. 2) is approximately 6 meters wide and 50 meters in length. Along the crest, there is a concrete walkway about 1 meter wide and 20 to 25 cm thick, connecting the park's headquarters to the wooden deck located by reservoir and the entertainment area. The walkway also contains several drains for electrical conduits along its length.

Fig. 2 – Dam crest

The reservoir depth (Fig. 3) ranges from 6 to 14 meters and has 4.1 km perimeter approximately, according to informational signs posted in PANAMAC. Satellite images were used to determine the area and estimate the reservoir's volume at 1.5 hm^3, which presents dense vegetation preserved in the surrounding area. Also, no other dams were observed upstream.

Fig. 3 – Reservoir view from right bank

The Açude da Concórdia Dam watershed covers an estimated 2.61 km^2 and remains unoccupied, with dense and preserved vegetation (Fig. 4). Currently, its tributaries lack instrumental monitoring, with water levels being visually inspected by park employees on a regular basis.

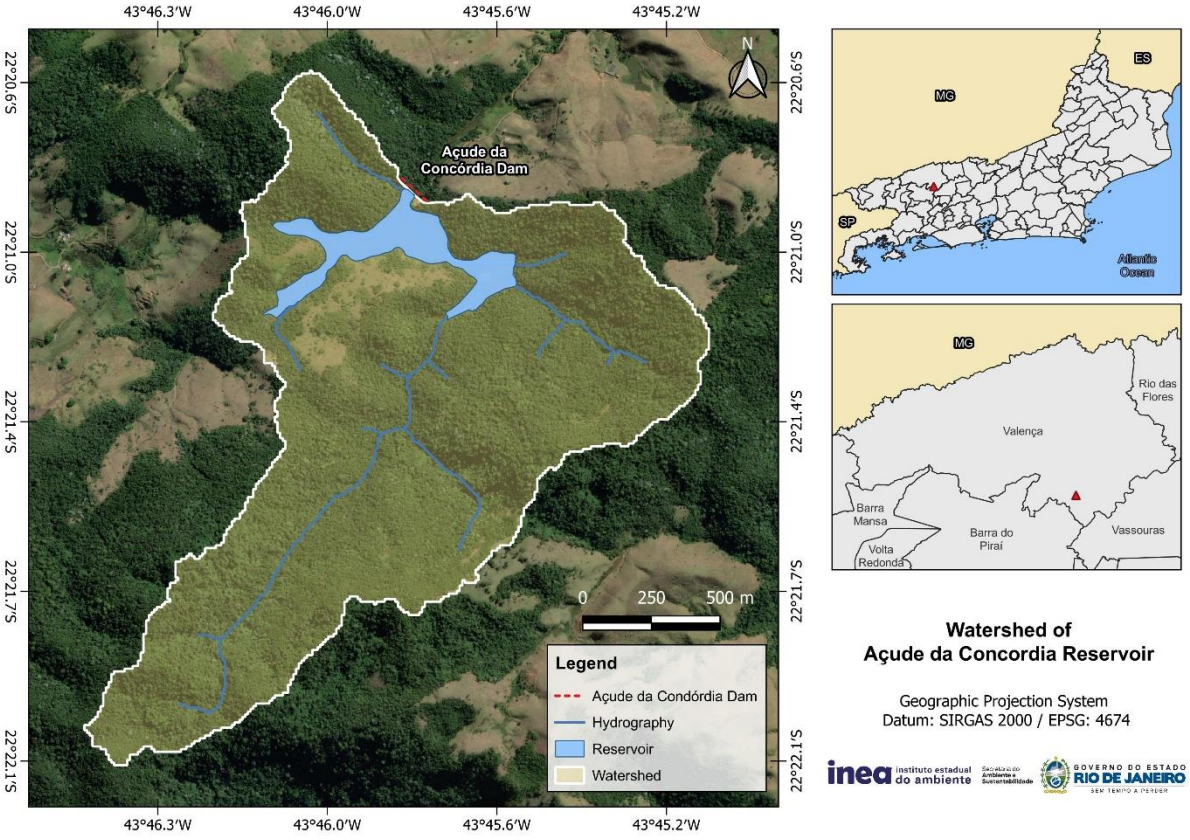


Fig. 4 – Açude da Concordia Dam Watershed

During on-site inspections conducted in 2023, significant siltation was observed throughout the entire reservoir length, therefore, its actual volume is probably lower than initially estimated. Considering potential accident scenarios, the reservoir's siltation could help mitigate the impacts as the lower water volume reduces the potential consequences. Regarding instrumentation, there is a limnimetric ruler adjacent to the wooden deck.

The dam has a spillway located about 1.3 km from its axis, a functional intake structure with a valve, where the tilting gate allows flow regulation and an artificial waterfall in dam downstream (Fig. 5), and a deactivated water intake structure on right abutment (Fig. 6).

Fig. 5 – Waterfall downstream of the dam

Fig. 6 – Deactivated water intake structure

On Jequitibá Trail, which provides access to dam downstream slope, presenting dense and large arboreal elements on the massif, ruins of old Water Treatment Plant (WTP) can be found, as shown in Fig. 7.

Fig. 7 – Decommissioned old Water Treatment Plant (WTP), highlighting large arboreal elements on dam downstream slope (background)

3. CONTEXTUALIZATION

In February 2023, PANAMAC management team identified a seepage on dam downstream slope and has contacted Inea department responsible for dam safety surveillance to assess structure safety conditions. Inea agents conducted an on-site inspection in early March 2023, and an internal erosion was identified, as shown in Fig. 8.

Fig. 8 – Seepage identified on dam downstream slope (March, 2023)

In May 2023, agents revisited the dam and found that the initially observed condition had worsened, verifying an increase in the number of erosion points, as illustrated in Fig. 9.

Fig. 9 – Observed increase in seepage on dam downstream slope (May, 2023)

Although projects increasingly incorporate automated processes and new technologies, failures in dam embankments internal drainage systems are a frequent incidents and accidents cause in structures of these kind, with internal erosion processes accounting for nearly half of all registered dam failures [7].

According to International Commission on Large Dams (ICOLD), internal erosion onset is governed by particle equilibrium mechanism, while its continuation is governed by grain transport mechanism [8].

Once the internal erosion process begins and not being halted, through the action of a filter or other measures for flow control and protection, erosion may progress, potentially leading to embankment collapse and/or its foundation. This process develops in four fundamental stages:

i. Internal erosion onset;
ii. Internal erosion continuation;
iii. Internal erosion progression; and
iv. Breaches formation and embankment/foundation collapse [9].

The phenomenon known as piping is a specific type of internal erosion that begins with regressive erosion and results in a channel or "pipe" formation, through which significant flows continuously percolate between dam embankment upstream and downstream or its foundation. This can lead to cracks formation and even massif failure [10].

Therefore, when internal erosions are detected, actions must be taken to control and repair them. For this reason, Inea implemented emergency measures to minimize failure risk of Açude da Concórdia dam.

4. MEASURES TAKEN BY INEA

In addition to several on-site inspections carried out on dam after embankment erosion points identification, Inea dam safety surveillance department recommended, among other actions:

i. safety inspection to assess dam integrity and associated structures by a competent and duly qualified professional; and
ii. suspending park visitation, considering prevention principle.

Additionally, State and Municipal Civil Protection and Defense Agencies were notified about Açude da Concórdia dam situation and were called upon to support emergency measures to prevent harm and impacts on downstream population.

The emergency measure implemented to minimize dam failure risk was reservoir drawdown. Thus, in mid-May 2023, considering prevention principle, Inea's technical team began the emptying process, initially by creating an auxiliary channel near dam's spillway (Fig. 10) and pumping water using motorized pumps.

Fig. 10 – Lateral opening near spillway, created to assist water release from the reservoir

After three days of pumping, agents evaluated that emptying could be carried out using a set of four pipelines installed over the dam crest, functioning as a siphon (Fig. 11), measure adopted since then.

Fig. 11 – Reservoir drawdown: pipes over dam crest

The interventions and maneuvers executed by Inea to empty the reservoir were supported by the State Water and Sewer Company (in Portuguese, CEDAE), a public state company (Rio de Janeiro, Brazil), which provided equipment and materials, such as pipes used at emptying process.

During an on-site inspection conducted two weeks after reservoir drawdown began, a significant water level reduction was observed based on visual analysis. This reduction was observed on limnimetric ruler, which in previous inspections has been submerged due to the high reservoir water level, as shown in Fig. 12.

Fig. 12 – Reservoir water level lowering register (limnimetric ruler on right bank)

It is worth noting that after heavy rainfall in dam region at end of March 2024, the reservoir drawdown process was taken again (by reducing extra volume), and no new erosion points were identified. Since then, continuous monitoring of climate conditions and rainfall has been implemented to detect potential changes in reservoir conditions, enabling drawdown restart process if necessary.

Following problem identification in May 2023, Inea also implemented non-structural measures to address the issue. In this context, an interinstitutional Emergency Response Board was established involving Inea, CEDAE, PMV, and Civil Protection and Defense Agencies towards to coordinating efforts, monitoring and updating information about Açude da Concórdia dam's situation and risk-reduction measures.

Inea presented and incorporated into PANAMAC's Contingency Plan (in Portuguese, PLANCON) an emergency action protocol.

PLANCON, initially developed by park management team, included general information about the conservation unit, such as contacts information for emergencies, available infrastructure, equipment, vehicles, and access routes. It also included relevant biotic and abiotic conditions, support points and vulnerable areas, and provided details about locations for food, medical

assistance, and institutional partners contacts such as the Fire Department, Civil Protection and Defense Agencies, Police and others, focusing on biodiversity issues, park activities, and its touristic role.

However, there was no specific protocol for handling dam-related emergencies. So, considering Açude da Concórdia dam particularities, an emergency protocol was developed to guide Inea, Civil Protection and Defense Agencies, and other agents in responding to specific circumstances and/or anomalies that could lead to dam incidents or accidents, considering current condition identified during previous inspections.

Thus, based on 5th paragraph of Art. 13 of the National Water Resources Council (in Portuguese, CNRH) Resolution no. 230/2022 [11], an Emergency Action Protocol was developed and attached to PANAMAC's PLANCON, containing:

i. dam general information;
ii. the location of communities, infrastructure, and environmental protection and conservation units potentially affected; and
iii. entities list and its contacts to be notified in case of an incident or accident.

This document also pointed regions and vulnerability points downstream of the dam where intervention by competent authorities would not be able to respond in time in case of an accident. These areas were designated as Self-Rescue Zone (in Portuguese, ZAS) and the Secondary Rescue Zone (in Portuguese, ZSS).

Dam failure simulations results and downstream flood wave propagation studies, aimed at identifying populations and areas potentially affected in case of dam accidents (e.g., structural collapse), are essential, as they guide the decision-making and actions by competent authorities during structural failure situations.

To conduct these studies, it is necessary to gather field data with satisfactory scope, quality, and resolution. Key data include topobathymetric, cadastral, topographic, and hydrometric surveys, geotechnical studies on dam stability, and information on riverbanks and crossings along water stretches.

Insufficient project data hindered the creation of a detailed flood map. However, a survey of available data was conducted to draw a simplified preliminary flood map (Fig. 13), identifying approximately 80 properties that could potentially be affected in case of Açude da Concórdia dam failure. This map was developed to guide decision-making and Civil Protection and Defense Agencies actions for population evacuation in case of imminent dam accident.

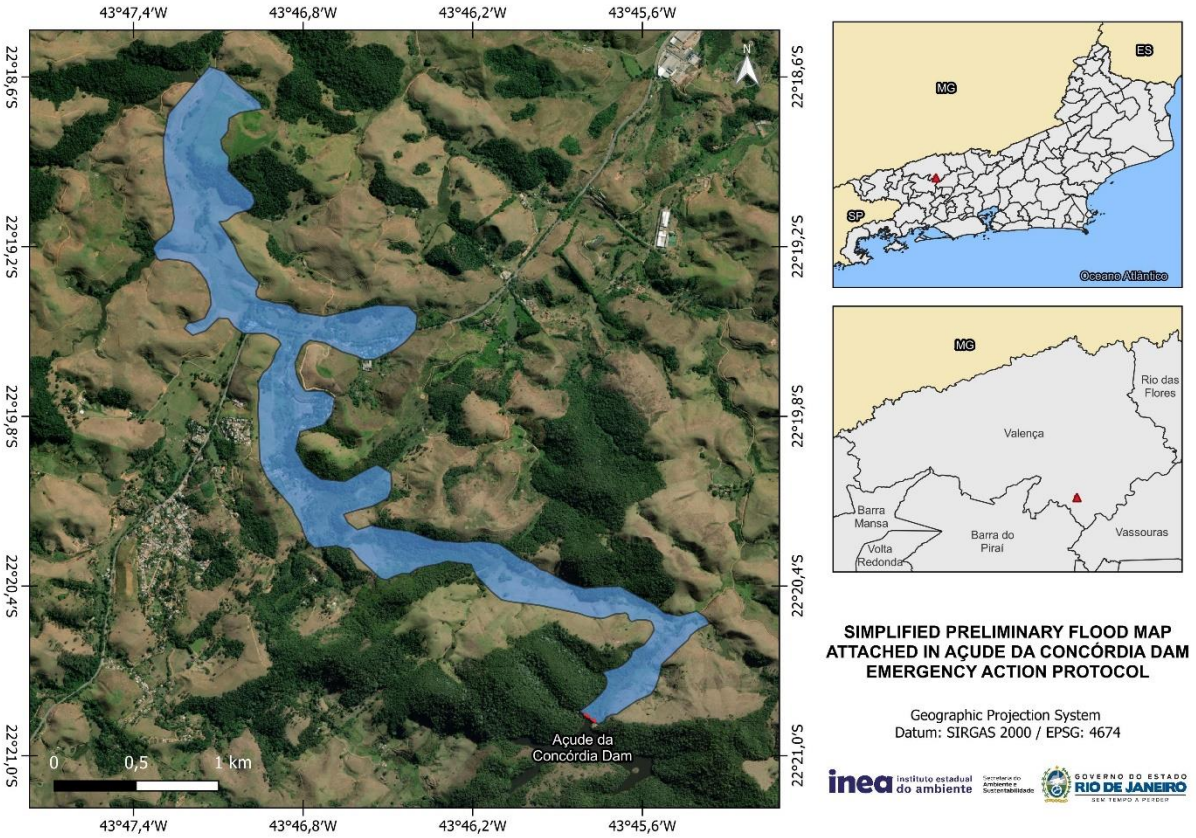


Fig. 13 – Simplified preliminary flood map attached in Açude da Concórdia Dam Emergency Action Protocol

In collaboration with the Portuguese National Laboratory of Civil Engineering (in Portuguese, LNEC), the Brazilian National Water and Basic Sanitation Agency (in Portuguese, ANA) developed a simplified methodology for generating flood maps resulting from dam failure, using solely data related to height, volume, and location, and leveraging the Shuttle Radar Topography Mission (SRTM) as a Digital Elevation Model (DEM). Notwithstanding the inherent simplifications, this methodology was deemed suitable for dam classification purposes with respect to Associated Potential Hazard (in Portuguese, DPA) [12].

Given its successful application, the methodology was subsequently adopted by ANA and Inea for DPA classification of their dams. It is important to note that the simplified preliminary flood map development relied on the available cartographic data for Rio de Janeiro State, provided by Brazilian Institute of Geography and Statistics (in Portuguese, IBGE), at a 1:25.000 scale. Additionally, some parameters were estimated due to the lack of project dam data and bathymetric surveys.

The methodology employs an empirical formula to estimate the maximum downstream flood extent. Topographic data derived from DEM are used to determine river profiles and key hydraulic characteristics [13].

Flood wave propagation is estimated using empirical equations based on dam volume and distance. Cross-sectional profiles are extracted from the DEM along the river path to calculate expected water heights. These elevations are then interpolated to produce the flood extent map.

Finally, the flood zone is overlaid on satellite imagery to evaluate potential impacts on infrastructure and settlements, aiding hazard classification and emergency planning. This method offers a rapid and practical approach for preliminary flood risk assessment.

Synthetically, the simplified methodology contains the following steps:

1. Empirical calculation of downstream flooded area;
2. Calculate maximum flow rate at dam section;
3. Verify downstream limit zone;
4. Calculate attenuated rupture flow rate;
5. Obtain altimetry of valley sections;
6. Calculate maximum flood wave level;
7. Create maximum flood surface map;
8. Obtain flooded area;
9. Consider correction factors (if needed);
10. Delimit affected area.

If necessary, the delimitation is refined after overlaying the flood zone on satellite imagery, to address any inconsistencies that may be identified.

Based on prevention principle, vulnerable regions, corresponding to outlined areas within simplified preliminary flood map and nearby areas, were pointed. These nearby areas were also included due to model simplifications, because they could potentially be affected too. The protocol also included contingency actions with procedures for incidents and/or accidents, population evacuation to safe locations and shelters, and road and local highway closures to safeguard human lives.

Inea also recommended Civil Protection and Defense Agencies conduct on-site inspections in areas previously classified as potentially affected vulnerable regions, considering available data limitations and region's dense vegetation, which could hide facilities, infrastructure, residences, roads, and communities not identified during preliminary assessment.

The protocol also outlined response process (Fig. 14), including:

- Location(s) for setting up emergency operation center;
- Notification and alert procedures;
- Emergency response actions for each dam potential incident.

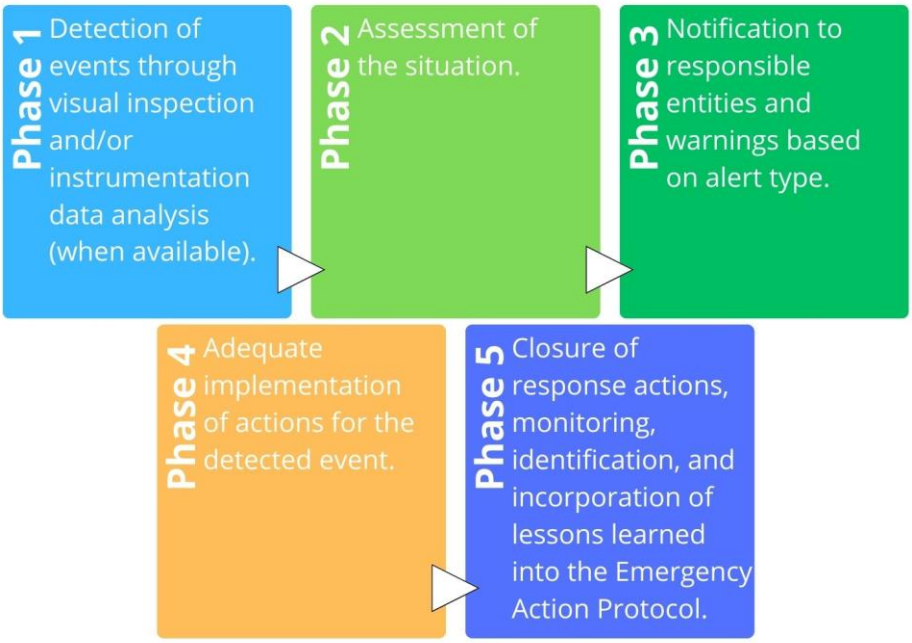


Fig. 14 – Emergency Response Process attached in PANAMAC's PLANCON

Notification and alert procedures aim to identify names and contacts of responsible entities, define communication channels, and establish the communication and alert system for population residing in dam downstream vulnerable areas in case of imminent or occurring dam accidents.

In addition to the emergency actions, Inea also initiated a bidding process to hire a specialized company to develop studies, a safety inspection, and an assessment of Açude da Concórdia dam conditions and situation. These activities should be performed by qualified professionals and should also include recommendations for appropriate interventions to address existing issues, along with related costs and schedules. However, the bidding process was declared unsuccessful due to a lack of interested companies. Until the conclusion of this paper, a new bidding process was in progress.

5. CONCLUSIONS AND RECOMMENDATIONS

Incidents and accidents involving dams in Brazil over the past decades highlight the need to revise procedures and criteria employed in dam safety, aiming to ensure more effective control mechanisms for improving dam security.

In this context, the National Dam Safety Policy (in Portuguese, PNSB) [14], published in 2010 with significant revisions in 2020, and the State Dam Safety Policy (in Portuguese, PESB) [15], published in 2016 and applicable in Rio de Janeiro State, stand out as important frameworks.

Thus, dam safety has assumed a substantial role, emphasizing dam owner (or user) responsibility and their commitment to ensure dam operation and maintenance are carried out by experienced and qualified professionals.

Towards erosion issues (piping) identified at Açude da Concórdia dam in 2023, it can be concluded that Inea acted preventively, considering that these issues are recurring causes of structural failure in dams of this kind.

Below is a summary of the actions taken by Inea in response to piping issue at Açude da Concórdia dam:

- Periodic on-site inspections to assess safety conditions;
- Interinstitutional Emergency Response Board establishment;
- Reservoir drawdown;
- Emergency action protocol development;
- Simplified preliminary flood map;
- Identification of vulnerable regions and points in case of dam failure; and
- Bidding process for perfoming studies, safety inspections, and an evaluation of dam's situation (in progress).

The establishment of an emergency response board, implementation of emergency structural measures (especially the reservoir drawdown), dam safety conditions monitoring, and development of an emergency action protocol significantly contributed to reducing disaster risks at Açude da Concórdia dam.

Until the conclusion of this paper, the situation was under control, with the dam’s water level lowered, and water percolation at identified erosion points had ceased.

Given that no precise evaluation of dam's stability and integrity has been conducted and that no interventions have been made to recover its structure, Inea recommended continuing reservoir emptying actions. This is essential since significant rainfall could lead to a rapid increase in reservoir’s water level, imposing a risk of collapse.

Therefore, Inea continues monitoring dam's conditions and situation and reservoir water levels to detect any progression in the observed scenario due to summon relevant institutions with the necessary urgency in case of an incident and/or accident at the dam.

Finally, it is essential to emphasize the importance of preventive actions, such as regular safety inspections and dam and associated structures proper maintenance, including spillways, floodgates, and electromechanical equipment. The prevention principle implementation is particularly significant in situations like the one presented in this paper. Through state's environmental agency (Inea) proactive efforts, in collaboration with other agencies, it was possible to prevent a potential tragedy, preserve lives, and mitigate potential economic and environmental impacts.

6. REFERENCES

[1] Sampaio, M. V. N. (2014). Segurança de Barragens de Terra: Um Relato da Experiência do Piauí, Dissertação de Mestrado. Universidade Federal do Ceará. Fortaleza, CE.

[2] Miranda, A. N. (2010). Curso de Treinamento sobre Segurança de Barragens. XIV Fecon – Feira e Congresso Internacional de Engenharia e Arquitetura. Teresina, PI.

[3] Brasil. (1988). Constituição da República Federativa do Brasil. Brasília, DF.

[4] Silva, G. F. & Sant'Anna, F. M. (2021). Degradação Ambiental e Desastres Socioambientais: O Princípio da Prevenção Como Meio de Proteção da Saúde e do Meio Ambiente – Uma Análise do Caso de Brumadinho. Open Journal Systems, Vol. 3, 2.

[5] Valença. (2001). Decreto Municipal nº 65, de 12 de setembro de 2001. Cria o Parque Natural Municipal do Açude da Concórdia (PANAMAC). Valença, RJ.

[6] Portal Valença RJ. (2018). Parque Natural Municipal do Açude da Concórdia. Portal Valença RJ (Online). Accessed 22 Nov. 2024. *https://www.portalvalencarj.com.br/acude_concordia/*.

[7] Filho, M. de Á. P. (2013). Análise da Erosão Interna de Solos em Barragens com Base na Distribuição de Vazios, Dissertação de Mestrado. Escola de Engenharia, Universidade Federal de Minas Gerais. Belo Horizonte, MG. 124 p.

[8] ICOLD. (2017). Internal Erosion of Existing Dams, Levees and Dikes, and their Foundations. Vol. 1 & 2, Bulletin 164.

[9] Carneiro, H. F. (2021). Erosões Internas no Aterro da Barragem Granjeiro: Estudo de Caso, Dissertação de Mestrado. Departamento de Engenharia Ambiental, Universidade Federal da Bahia. Salvador, BA. 114 p.

[10] Fell, R., Foster, M., Davidson, R., Cyganiewicz, J., Sills, G., & Vroman, N. (2008). A unified method for estimating probabilities of failure of embankment dams by internal erosion and piping. UNICIV Report R, 446, 10-8.

[11] Brasil. (2022). Resolução CNRH n° 230, de 22 de março de 2022. Estabelece diretrizes para fiscalização da segurança de barragens de acumulação de água para usos múltiplos. Conselho Nacional de Recursos Hídricos. Brasília, DF.

[12] Petry, A., Bonfim, M., Laus, F., & Anderaos, A. (2018). Classification of dams by its hazard potential: The experience of the Brazilian National Water Agency. Third International Dam World Conference. Foz do Iguaçu, Brazil. 10 p.

[13] Rolo, R. M., Marodin, F. A., Carrard, G. P., Cardone, L. B., & Silveira, C. J. S. (2022). A python implementation for the simplified dam-break flood modeling. Brazilian Journal of Water Resources – RBRH, v. 27, e8. https://doi.org/10.1590/2318-0331.272220220013.

[14] Brasil. (2010). Política Nacional de Segurança de Barragens (PNSB). Lei Federal nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens destinadas à acumulação de água para quaisquer usos, à disposição final ou temporária de rejeitos e à acumulação de resíduos industriais. Brasília, DF.

[15] Rio de Janeiro. (2016). Política Estadual de Segurança de Barragens (PESB). Lei Estadual nº 7.192, de 06 de janeiro de 2016. Dispõe sobre a Política Estadual de Segurança de Barragens (PESB) e regula o Sistema Estadual de Informações sobre Segurança de Barragens (SEISB) no âmbito do Estado do Rio de Janeiro. Rio de Janeiro, RJ.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

THE CHALLENGES OF THE BRAZILIAN ELECTRIC ENERGY REGULATORY AGENCY ON MITIGATE EFFECTS FROM 2024 RIO GRANDE DO SUL FLOODS

Leite, Sérgio R.[1]
Brazilian Electricity Regulatory Agency

Silva, Ludimila L.[2]
Brazilian Electricity Regulatory Agency

Gomes, Luiz R.[3]
Brazilian Electricity Regulatory Agency

ABSTRACT

Evaluating the occurrence of extreme events in large civil structures, such as dams, is of great importance in a more complete risk analysis study, both for managers and decision-makers.

The prevention of new tragedies, however, depends on the lessons we learn from previous accidents, regardless of their size. Accidents act as revelations of organizational dysfunction, questioning our capacity for analysis and diagnosis. Generally speaking, the solution to mitigating these risks requires, in addition to rigorous periodic inspections, the adoption of more efficient structural and non-structural studies and techniques.

Against this backdrop, the National Electric Energy Agency (ANEEL), which is responsible for overseeing the construction and operation of hydroelectric power plants, has been restructuring its activities with a view to improving the knowledge and techniques linked to the dam monitoring process, based on risk assessment and management approaches.

Between the end of April and the beginning of May 2024, the Brazilian state of Rio Grande do Sul was hit by flooding due to precipitation, which led the local government to classify the situation as "the biggest climate catastrophe in the state's history". The excessive rainfall affected more than 60% of the state's territory.

In this context, out of a total of 126 potentially affected hydroelectric plants, only two had their dams partially breached, and their effects were restricted to material damage. No loss of human life was recorded.

In a risk-oriented perspective, in which actions, performance and consequences are treated in an integrated manner, the correct classification of the levels of commitment of a structure or system makes it possible to detect dangerous events in good time.

The aim of this article is to present an analysis of the climatic events recorded in the state of Rio Grande do Sul, their impacts on the procedures adopted by ANEEL to mitigate their effects, as well as the challenges faced by the Agency in monitoring the safety of hydroelectric dams in this region.

Keywords: Dam Safety, Risk Management, Regulation, Enforcement, Climate Changes.

[1] sergioleite@aneel.gov.br, Brazil
[2] ludimila@aneel.gov.br, Brazil
[3] lrgomes@aneel.gov.br, Brazil

1. INTRODUCTION

Climate change impacts on water resources are increasingly affecting the vulnerability of global hydropower generation. Climate risk assessments for South American countries such as Brazil, the country most dependent on this type of energy on the continent [1], are becoming increasingly severe, suggesting that critical infrastructure and energy and water security strategies are not adequate to respond to climate change [2].

According to Belt [3], higher temperatures and changing weather patterns are altering evaporation, river flow, rainfall patterns, frequency of extreme weather and glacial melting rates. Although the response of flood magnitudes to precipitation extremes is not entirely clear due to the still large uncertainties associated with climate models [4], studies indicate an increasing trend of vulnerability to flooding in urban systems and reservoirs.

In Brazil, according to data from the Dam Safety Report [5], 2023 edition, prepared by the National Water and Sanitation Agency (ANA), there are at least 26,000 dams, from small to large. Most of them were built based on pre-climate change hydrological assumptions, i.e. without considering the risks arising from the intensification of extreme events.

The effects of this potential fragility were observed in May 2024, when the Brazilian state of Rio Grande do Sul faced climatic events that impacted on the safety of its dams, causing partial collapse and flooding of its powerhouses. This context triggered a series of actions by the Brazilian Electricity Regulatory Agency (ANEEL) and other government entities involved, which highlighted the importance of preventive inspection in standardizing emergency procedures and updating safety standards.

The aim of this exploratory article, from the perspective of the dam safety policy in force in Brazil, is to discuss the events recorded in the state of Rio Grande do Sul, their impact on the procedures adopted by ANEEL to mitigate their effects, and to present the initiatives adopted to deal with this critical scenario.

2. THE NATIONAL DAM SAFETY POLICY AND ANEEL'S RESPONSIBILITY

On September 20, 2010, Federal Law No. 12.334 [6] was passed in Brazil, establishing the National Dam Safety Policy (NDSP) for the accumulation of water for any use, the final or temporary disposal of tailings and the accumulation of industrial waste. The NDSP establishes that the public authorities are responsible for overseeing the safety of dams, according to their purpose, by the bodies that grant or authorize the exploitation of the hydraulic potential. The regulation of Law 12.334/2010 with the specificities of the electricity sector is currently governed by ANEEL Normative Resolution no 1.064 of May 2, 2023 - REN 1064/2023 [7].

These regulations define that the safety conditions of structures are checked by means of inspections and by following up with the recommendations proposed by the technicians responsible for submitting the safety reports.

The law subdivides safety inspections into regular and special. The regular one is carried out by the dam's own safety team, and the resulting report must be made available to the inspection body and civil society. The special one must be drawn up as directed by the inspection body, by a multidisciplinary team of specialists, depending on the risk category and potential damage associated with the dam. The reports resulting from the safety inspections must indicate the actions to be taken by the developer to maintain the safety of the dam.

The regulation also provides for Periodic Safety Reviews (PSRs), the aim of which is to check the overall safety status of the dam, taking into account the current state of the art for the design criteria, updated hydrological data and changes in conditions upstream and downstream of the dam.

In addition, the Resolution defined criteria for classifying dams and delimited which of these structures are covered by the Agency's regulatory provisions. Under the terms of REN 1.064/2023, the classification of dams is initially based on self-assessment information on the integrity of dams submitted annually by entrepreneurs, based on the set of risk factors established by the Resolution of the National Council of Water Resources - CNRH No. 144, of July 10, 2012 [8].

Based on the principles of the Scoring Method, the regulation establishes a matrix structure for classifying dams by Risk Category (RC), by Associated Potential Damage (PDA) and by their volume, summarized in Fig. 1.

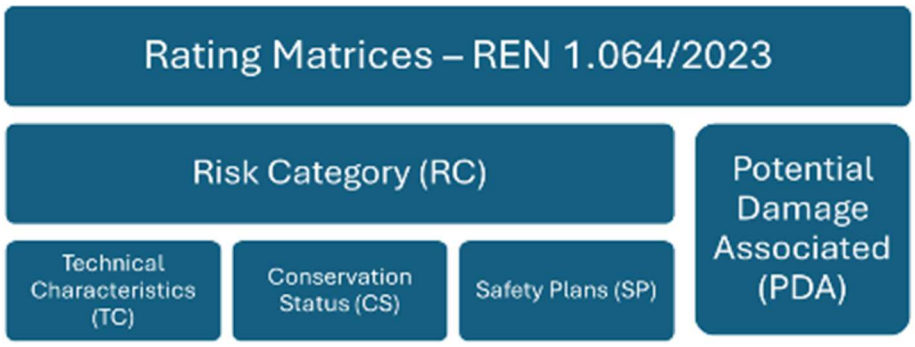


Fig. 1 - Matrix Structure for Evaluating Dams. ANEEL, 2023

The RC classification takes into account structural and functional aspects of the dam. Its purpose is to represent, in accordance with existing rupture statistics, the massif's vulnerability to an accident, in terms of its: (i) intrinsic Technical Characteristics (TC); (ii) current State of Conservation (SC) of its structures; and (iii) Safety Plans (SP) or maintenance documentation and procedures.

The classification in terms of PDA relates to the consequence to which all dams are leveled when considering the downstream effects of the flood induced by the dam failure in different scenarios of loss of human life, environmental impact and socio-economic impact.

The combinations of the results obtained for these give rise to the final classification of the dam, as established in the specific Classification Matrix proposed by REN 1.064/2023 - ANEEL (Fig. 2).

RISK CATEGORY	POTENTIAL DAMAGE		
	High	Medium	Low
High	A	B	B
Medium	B	C	C
Low	B	C	C

Fig. 2 - Dams final classification matrix. ANEEL, 2023

Where:

A. Dams that present a high potential risk and damage category and whose anomalies require short-term intervention to maintain safety conditions, not always signifying an immediate risk of rupture;
B. Dams that present medium or low risk category and average potential damage and whose anomalies must be controlled, monitored and interventions can be implemented over the time to maintain safety conditions; and
C. Dams that present medium or low risk and potential damage and which do not present anomalies and existing ones do not compromise the safety of the dam.

In order to monitor the results of the obligations of the generation agents covered by the law, ANEEL has developed a specific monitoring structure to collect information on the characteristics that define the classification of dams in terms of Risk Category and Associated Potential Damage.

The processing of this information determines ancillary obligations that must be observed by generation agents, notably those associated with the preparation of the Dam Safety Plan (DSP), the Regular Safety Inspection (RSI) and the Emergency Action Plan (EAP), as well as the respective deadlines for their completion.

In the context of the NDSP, ANEEL must ensure that the companies responsible for hydroelectric dams maintain an up-to-date safety plan and implement preventive actions to avoid structural failures. In addition, ANEEL must ensure that they draw up and apply the Emergency Action Plan (EAP) in the event of any imminent risk to the integrity of the dams or to the populations located in the areas of their direct influence.

In events such as those recorded in May 2024 in Rio Grande do Sul, ANEEL must act not only as an inspector and regulator, but also as an entity that guides and coordinates the response in conjunction with other authorities, such as Civil Defense, the National Water and Sanitation Agency (ANA) and state and municipal governments.

3. HYDROLOGY AND DAM INFRASTRUCTURE IN RIO GRANDE DO SUL

3.1. Regional Hydrology

According to Fritzsons *et al* [9], the climate is mainly influenced by relief, altitude, latitude and distance from the sea. The state of Rio Grande do Sul (RS) is located in the southern region of Brazil between longitudes -49º and -58º and latitudes -27º and -34º, at 10m above sea level. In terms of relief, it is made up of plateaus, depressions, mountain ranges and the coastal plain.

The state capital and Brazil's fifth-largest metropolitan city, Porto Alegre, is situated on the Guaíba, which is formed by the confluence of nine river basins and leads to Lagoa dos Patos, the continent's largest freshwater Lagoon, as show in Fig. 3.

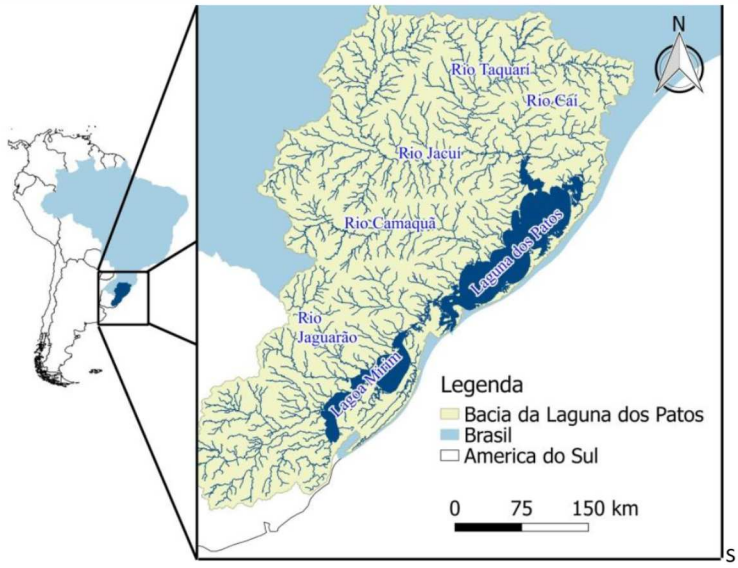


Fig. 3 - Map showing some of the state's major river systems emptying into Lagoa dos Patos, at the northwestern tip of which Porto Alegre is located. VRKIC, Dora *et al*., 2024

As far as vegetation is concerned, it is characterized by the presence of two important Brazilian biomes: the Atlantic Forest and the Pampa. The Atlantic Forest is located in the north-northeast of the country, occupying 37% of its area. However, as explained by Rodrigues *et al* [10], its distribution is inhomogeneous and highly fragmented. It is estimated that the remaining area corresponds to only 7.5% of the original vegetation cover. The remaining 63% of the state's area is occupied by the Pampa, which stretches across the entire southern half and the

western border, with the presence of grasses, creeping plants and some trees, shrubs and bushes.

In the context of climate variability, rainfall is one of the most important variables, as its behavior influences environmental, economic and social dynamics [10]. The hydrology of the state of Rio Grande do Sul is characterized by a rainfall regime that varies throughout the year. Alternating periods of drought and heavy rainfall create constant challenges for water management. Precipitation is well distributed throughout the year, due to the fact that the region is located in a band of mid-latitudes that receives precipitation systems of tropical and extratropical origin [10, 11]

However, average monthly rainfall is not spatially homogeneous and varies seasonally. According to Reboita *et al*. [12], the northern half of RS generally has higher precipitation figures, except in the coldest half of the year, when the presence of southern polar air masses makes the distribution of precipitation more homogeneous, causing the south of the state to have its highest volumes in this period.

In addition, there are changes in the precipitation regime when phenomena such as La Niña and El Niño occur.

3.2. Dam Infrastructure in Rio Grande do Sul

The state's water infrastructure was designed for various purposes, such as flood control, water supply, irrigation and electricity generation, and is therefore made up of a series of dams, locks, dikes and pumping systems. This extensive and diverse hydrographic network is essential for managing the state's water resources [13,14, 15].

According to data from the Dam Safety Report [5], of the 26,157 dams registered in Brazil's 27 federal units, 10,412 are located in Rio Grande do Sul, making it the Brazilian state with the highest number of dams in the country [9]. Of these, 98% are identified as dams for water storage purposes and are under the direct supervision of the state government.

Considering the dams whose main purpose is to generate hydroelectric power, the state has 143 structures in operation, corresponding to 125 plants (Fig. 4), which are monitored by ANEEL's Dam Safety Monitoring System (FSBWeb) [16]. Of this group, 34 dams are classified as Type B, characterized by LOW risk and HIGH potential damage. 70 dams are classified as not covered by the Safety Law (12.334/2010).

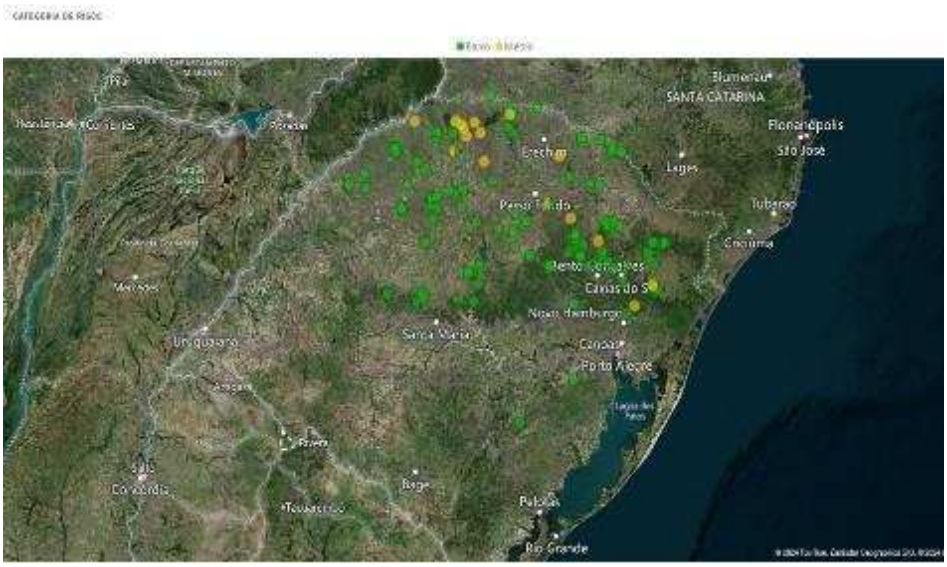


Fig. 4 - Dams monitored by ANEEL in the state of Rio Grande do Sul, identified by Risk Classification (Type B - Yellow, Type - C - Green). ANEEL, 2023

A large proportion of these projects, around 87%, are small hydroelectric plants with little or no regularization capacity [15]. Even the large capacity plants are mostly run-of-river plants. In terms of the electricity generation matrix, the hydroelectric plants installed in Rio Grande do Sul account for 58.6% of the state's installed capacity [13]. This figure is proportional to the national installed capacity (53.15%) [15].

In terms of Safety Level, based on the results obtained from Regular Safety Inspections (RSI), requested by ANEEL and collected via the monitoring system, dams have their safety level measured on a scale that takes into account the combined effect of the anomalies observed in their structure or operation.

Safety levels identified as 'Normal' or 'Warning' indicate that the anomalies identified do not compromise the safety of the structure (they can be understood as neglect and poor maintenance), or compromise its integrity in the short term, and should be controlled and monitored over time. On the other hand, 'Alert' or 'Emergency' classifications indicate short-term compromise, or an imminent risk of rupture or collapse, and measures should be taken to eliminate the problem.

Before the extreme weather event, the state had three plants in a state of WARNING for reasons unrelated to the events observed: PCH Guarita, PCH Passo do Inferno and CGH Herval [16]. The latter two were not covered by Dam Safety Law.

4. BACKGROUND TO THE EVENTS OF MAY 2024

The climatic event in Rio Grande do Sul has its origins in the warming of the waters of the Pacific Ocean, a phenomenon studied and recorded in numerous publications, known as El

Niño, with the combination of special meteorological conditions installed over the state from April 29, 2024.

An atmospheric block caused by a high-pressure system in central-southern Brazil prevented the movement of typical weather systems (extratropical cyclone, cold front, trough) that cause precipitation [17]. Temperatures where the anticyclone acted were five to ten degrees Celsius above what is historically recorded by the National Institute of Meteorology (INMET). Because of this, the areas of instability were confined to the state of Rio Grande do Sul, characterizing a stationary front (Fig. 5).

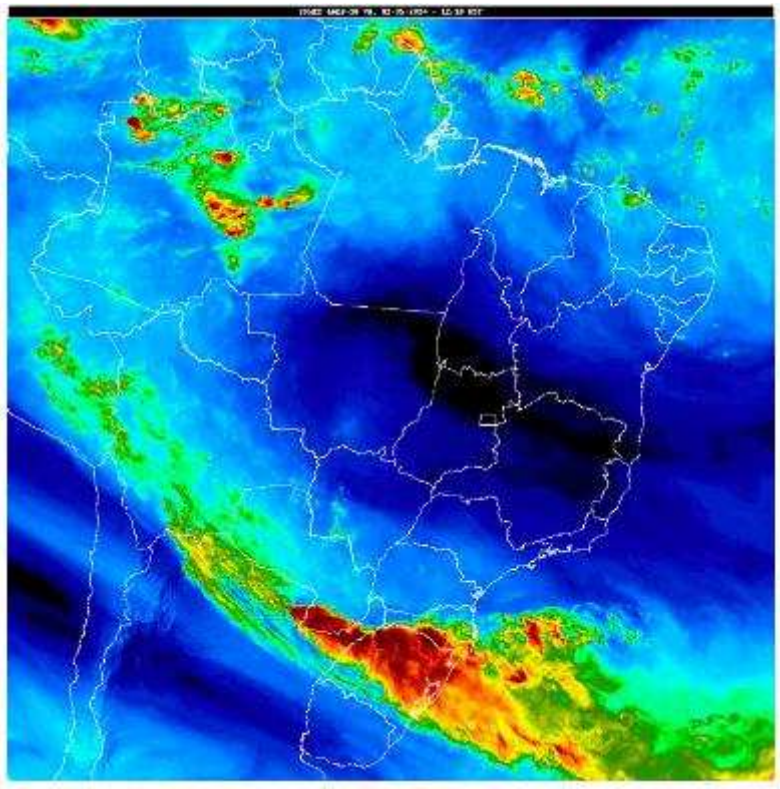


Fig. 5 - Climatic conditions in Rio Grande do Sul on 02/05/2024. INMET, 2024

Before the first warning of heavy rainfall, issued on April 29 by Inmet, areas in the Rio Pardo Valley, in the central region of the state, had already been hit by heavy rain and hail since April 27. On April 28, the Civil Defense of Rio Grande do Sul issued the updated Meteorological Warning 254 [18], warning of the risk of disruption due to isolated thunderstorms and occasional heavy rains, causing rapid rises in water levels and overflows in streams, small creeks and riverside regions.

On Monday the 29th, the situation spread throughout the state. On the 30th, at around 2pm, Meteorological Warning 254 was updated, warning of a "high risk of disruption due to isolated storms and heavy, persistent rainfall". In the days that followed, the warnings continued, always with forecasts for adverse weather.

On the night of May 2, the Federal Government recognized a state of calamity in the region. On May 3, the State Civil Defense issued a flood alert for the Gravataí and Sinos rivers and a severe situation for the Ibirapuitã and Jacuí rivers, starting in the municipality of Dona Francisca (Fig. 6). It also warned of the partial collapse of the 14 de Julho dam.

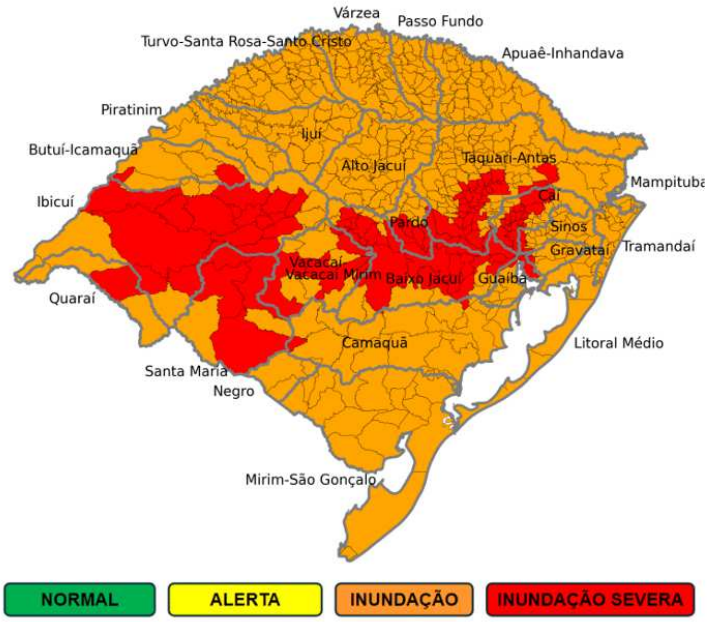


Fig. 6 - Alert issued by Civil Defense on May 3, 2024. INMET, 2024

During this period, several cities received between 500 and 700 mm of rain, equivalent to a third of the historical average rainfall for an entire year, and in many others the rainfall was between 300 and 400 mm (Fig. 7).

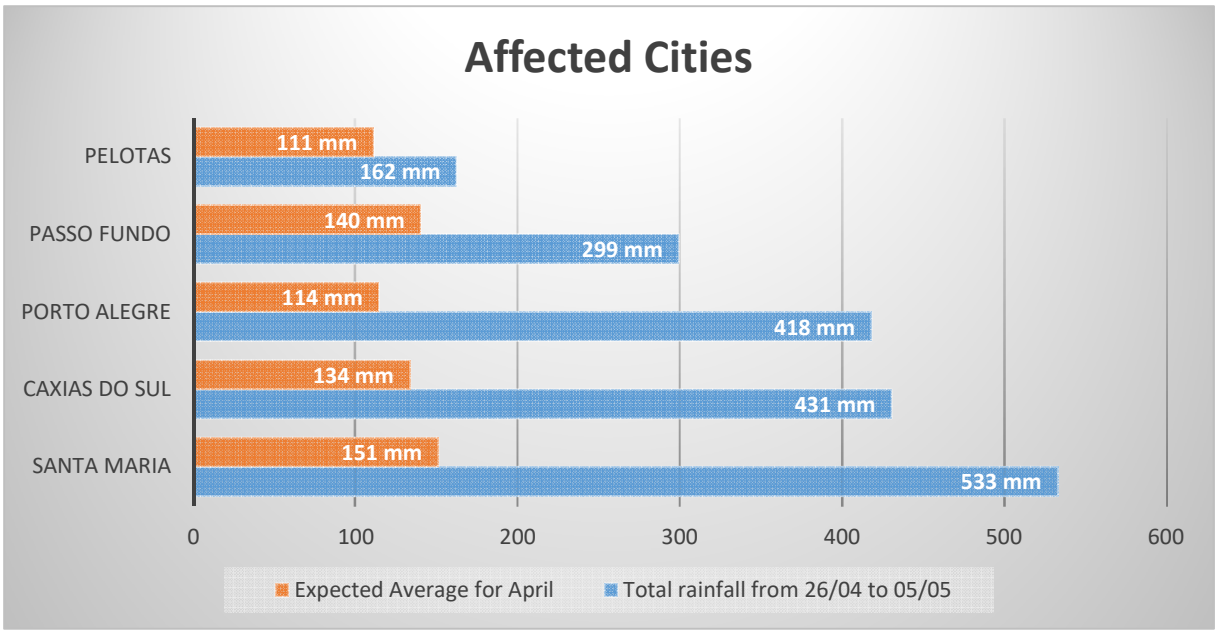


Fig. 7 - Precipitation levels recorded in some cities in Rio Grande do Sul. INMET, 2024

Lake Guaíba, which bathes the capital Porto Alegre, then surpassed the historic mark of 1941, reaching a level of 4.77 meters, causing flooding in several districts of the city, including the Historic Center. With the warning of a partial collapse of the dam at the 14 de Julho Hydroelectric Power Plant in the Rio das Antas basin, residents of seven municipalities in the metropolitan region were warned to leave risk areas. It was reported that the collapse created a wave of approximately 2 m, aggravating the flooding in already inundated communities nearby [19].

After a few days without rain, the advance of a high-altitude warm front over the colder air that was acting in the state, which became semi-stationary, accompanied by a low-level jet stream and a low-pressure center, led to the formation of a line of instability, and from May 10 onwards, a new period of intense precipitation began, concentrated in the northern half of the state, affecting regions where the headwaters of several rivers that had overflowed but were in the process of subsiding are located, such as the Jacuí, Taquari, Antas, Caí, Sinos, Paranhana and Gravataí.

The new wave of instability also caused two tornadoes to form, both on the 11th, one in the municipality of Gentil, with an estimated intensity of between F0 and F1 on the Fujita Scale, which hit the countryside and caused no damage or victims [20] and the other in the urban area of the municipality of Cambará do Sul, in the Vila Santana district, with estimated winds of 140 km/h, damaging 57 houses, a school and a health center.

A third period of rain between May 23 and 24 was produced by the formation of a low-pressure area to the east of Rio Grande do Sul, accompanied by the arrival of a strong mass of cold air of polar origin and winds from the southern quadrant with storm force, estimated at 70 to 90 km/h. Although several inland rivers rose again, this episode affected the Porto Alegre region more intensely.

The figures portray a scenario of destruction caused by the rains and floods in 463 municipalities in the state. Together the cities recorded 169 deaths, 41 missing, more than 500,000 displaced and a total of 2.33 million people affected.

According to Civil Defense, at the beginning of June, the figures were as follows:

- 76,955 people in shelters;
- 1,078 schools affected, totaling 378,000 students;
- 240,000 consumer units without electricity and 136,188 customers without water supply;
- Six municipalities without telephone and internet services;
- 37 federal highways and 49 state highways were completely blocked.

4.1. Considerations on Recorded Weather Events

According to a study produced by the World WeatherAttribution [21] Rio Grande do Sul is no stranger to floods due to its largely low-lying physiographic location myriad of river systems, and land-use changes driving vegetation loss. Illustratively, between 2013-2023, it saw 953 emergency decrees due to rain or associated hazards such as landslides [22].

Fig. 8 - Most of Rio Grande do Sul, Brazil's southernmost state, has faced damaging floods. Image:Copernicus Program, May 2024

The study characterized the weather and climate conditions in force between April 26 and May 5, 2024. The probability and intensity of consecutive heavy rains over periods of ten and four days were assessed, including the relationship with the El Niño phenomenon.

According to the authors, the heavy rainfall event was "extremely rare" and is expected to occur once every 100 to 250 years in the current climate. However, without global warming, it would have been even rarer. The article indicates that the probability of these events has increased by a factor of 2, with an increase in intensity of between 6% and 9%.

In addition, El Niño played an important role in the event. According to the observations made, for all the data sets analyzed, compared to a neutral El Niño phase, there was a threefold increase in the probability and a 7% increase in the intensity of these events.

However, these were not the only floods that Rio Grande do Sul has experienced. In Porto Alegre, as Valente [23] points out, the record of floods is almost as old as the city itself. The first known flood in the capital of Rio Grande do Sul dates back to 1795 - just 23 years after the city was founded. Due in part to its location, Porto Alegre has faced major floods in 1873, 1928, 1936, 1941, 1967, and 2023.

Between April and May 1941, Rio Grande do Sul was hit by torrential rains that lasted more than four weeks. The rainfall was probably influenced by El Niño, although the phenomenon had not yet been identified.

As happened in 2024, the rains raised the water level of the river basins that flow into the Guaíba. The rainfall exceeded 1,000 millimeters in some regions of the state and reached 791 millimeters in Porto Alegre. Similarly, the strong wind coming from Lagoa dos Patos reduced the speed of the runoff, damming up the water in the Guaíba.

The difference between the 1941 and 2024 events lies in the fact that this time they were more intense (22 days of rain on average in 1941 and now 6, less than a week, in 2024). When comparing the events of 1941 and 2024, the researchers [22, 23] indicate that in the first episode it took 22 days for the water level in Lake Guaíba to reach 4.76 m above normal levels. This year, however, the Guaíba exceeded 5 m in just five days.

Another condition to consider is that although both events occurred during an El Niño period, so did other cases listed by the authors over the last 124 years. However, the 1941 flood occurred at the height of an El Niño that lasted from 1939 to 1942; while the 2024 flood occurred during the end of the 2023 and 2024 episode.

Fig. 9 shows the years in which floods were recorded in the state. It can be seen that in every decade there were at least three episodes. Even in the 1950s, considered the driest decade of the 20th century, there were still episodes of flooding.

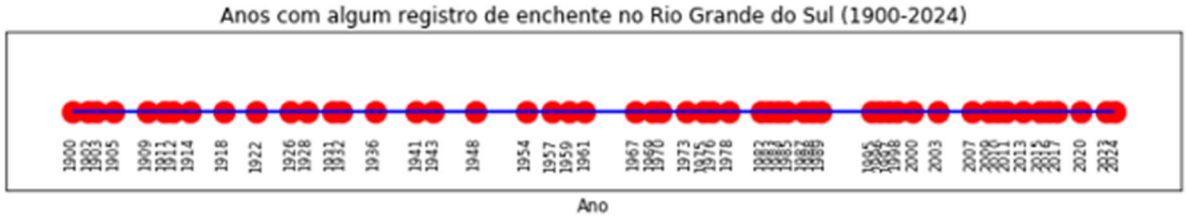


Fig. 9 - Rainfall history in Rio Grande do Sul – Years with Some Record of Flooding in Rio Grande do Sul (1900-2024) [23]

This brief analysis of vulnerability and exposure indicates that RS is likely to face more frequent and serious impacts related to flooding in the future. It should be noted that the variability of river flows does not always follow the behavior of rainfall, demonstrating that factors other than climate, such as those associated with water use, infrastructure operation, land use and occupation, among others, have a major impact on the events observed.

In this context, there is a need to invest in hydrological studies, considering more robust analyses, including verification of the hypotheses of stationarity of series, risk analyses and scenarios of probable maximum floods.

5. RECORDED SAFETY ACCIDENTS AND INCIDENTS AND THEIR IMPLICATIONS

The NDSP transfers to the supervisory body the power to determine the preparation of Emergency Action Plans (EAP) according to the risk category and potential damage associated with the dam and must always require this for dams classified as having HIGH potential damage.

In this context, in addition to a monitoring policy based on risk classification, the Agency analyzes the Safety Level of the projects under its supervision through the reports it receives. The safety levels guide those involved in defining the degree of danger in emergency situations, helping with decision-making and indicating the steps to be followed once a situation has been identified that could jeopardize the safety of the dam.

Fig. 10 shows the historical evolution of the main plants impacted and monitored during the month of May in Rio Grande do Sul, followed by a brief description of the events observed at the impacted power plants:

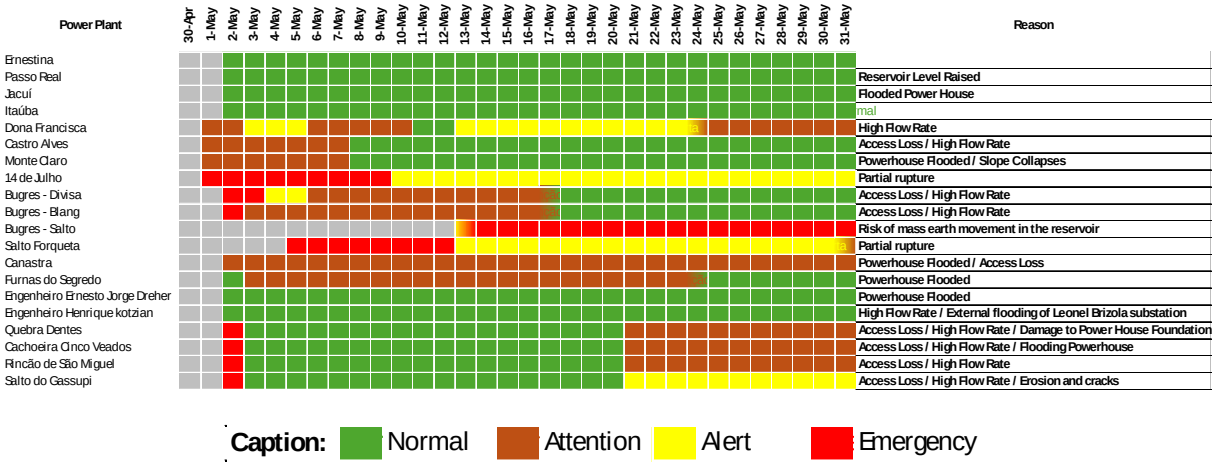


Fig. 10 - Safety Level Observed at Hydroelectric Power Plants During the Flood Period - May 2024. ANEEL, 2024

5.1. Hydroelectric Power Plant 14 de Julho

The 14 de Julho Hydroelectric Power Plant, part of the Rio das Antas Complex, located in the municipality of Cotiporã, was affected by heavy rainfall between 30/04/2024 and 03/05/2024, with accumulated values of around 520mm.

Fig. 11 – 14 de Julho Dam. Source: https://ceran.com.br/usinas/usina-14-de-julho

The high volume of rainfall caused various impacts, such as: loss of access, collapse of slopes and hillsides, damage to road structures and erosion. Power and communication supplies were also interrupted.

By design, the 14 de Julho dam was dimensioned to withstand a flood that represents El. 111.50m in the upstream reservoir, and the downstream level at El. 110.52m. Thus, in the limit load situation, the expected upstream/downstream gradient is less than 1.0m.

On 02/05/2024 at 13:23h, with the reservoir level at 111.61m, there was a partial breach of the structure of the 14 de Julho Hydroelectric Power Plant from the right shoulder, the controlled spillway gates were partially open and unable to continue the opening process.

Due to the loss of measurement of the level downstream of the dam, it was estimated from a photo taken on 01/05/24, with the downstream level approximately at the trunnion beam, which represents El. 98.00m. In this condition, there is an upstream/downstream gradient of approximately 13.6m. This gradient is a consequence of the impossibility of opening the gates fully and represents a load above the limit for which the structure was designed. Fig. 12 shows the damage caused to the dam structure after collapse.

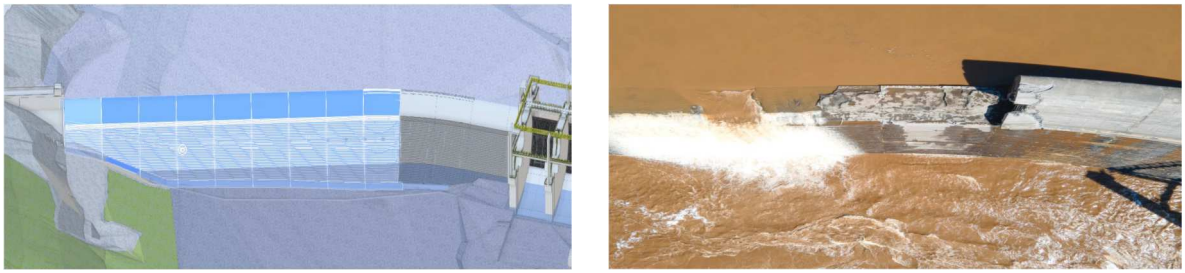


Fig. 12 - 14 de Julho dam after partial collapse. CERAN, 2024

The EAPs have been operational since 30/04/2024. When the EAP was activated during the event, there were no difficulties in liaising with the regional and municipal Civil Defenses and other parties involved.

On spillway region, the post-event inspections results identified some adhesion loss between the roller-compacted concrete (RCC) and conventional concrete (SCC) layers, which could indicate that no bonding mortar was used between the RCC layers. Between the construction joints of the face layers of SCC, it was also not possible to identify the treatments between layers.

In view of the observations regarding concrete technology, it was recommended that an investigation plan be carried out, using boreholes, water loss tests, borescopy, ultrasound and laboratory tests, in order to support the preparation of the executive project and any treatments.

Nevertheless, post-event inspections showed that the remaining structures are stable for the plant's operating flows. Although there is no structural risk, the plant has been maintained at a Safety Level of 'Alert' until the dam restoration is completed, in accordance with REN 1.064/2023.

5.2. Salto Forqueta Small Hydroelectric Power Plant

The dam located on the Forqueta River, in the municipalities of Putinga and São José do Herval, partially broke due to structural damage on its right bank caused by landslides. In addition to the emptying of its reservoir, there was flooding in its Powerhouse.

During the flood events, the flow rate measured in the region reached 2,817.07 m³/s, 56% above the maximum capacity of the plant's spillway, designed to handle a flow of 1,802.10 m³/s.

Fig. 13 - Salto Forqueta Dam after partial collapse. ANEEL, 2024

5.3. CGH Nova Palma

Built in 1926 and deactivated since 2010, the dam has been integrated into nature, forming a green tunnel to access it. On June 4, 2024, ANEEL was informed of its collapse, probably on April 30, 2024, due to climatic events.

Fig. 14 - Nova Palma dam after partial collapse. ANEEL, 2024

As it has a low volume of water, no impacts were recorded in its area of influence. As the dam was not in use, it was decided not to rebuild it, preserving the river in its original bed.

5.4. Bugres Hydroelectric Power Plant – Salto Dam

The Salto dam, which is part of the Bugres hydroelectric power station, although it did not break, did register a risk of slope landslides around its reservoir due to the high volumes of rainfall, with a potential risk of wave formation capable of damaging buildings in its area of influence, as shown in Fig. 15. Cracks compatible with large mass movements were detected in the vicinity of the water intake, near the left shoulder, with consequent infiltration of water into the terrain.

Fig. 15 - Alert issued by local Civil Defense on May 14, 2024 for the Salto Dam. Source: Civil Defense of the Municipality of São Francisco, 2024.

5.5. Dona Francisca Hydroelectric Power Plant

The Dona Francisca dam, located in the central region of the state, activated its Emergency Action Plan after recording historic levels of water flow. Historical inflows of around 10,600 m³/s were recorded, close to the maximum capacity of the spillway, equivalent to a decadal flood level, causing the dam to go into 'Attention' mode. According to the plant's safety procedures, when the flow rate exceeds 5,000 m^3/s, a Special Safety Inspection (SSI) is carried out, as determined in the project's Dam Safety Plan (DSP). A total of 3 (three) Special Inspections were carried out: in all of them, the dam's safety and stability situation was considered 'Normal'.

6. ANEEL'S ROLE IN THE MAY 2024 EVENTS

ANEEL's actions during adverse weather events were characterized by a rapid response to declared emergencies. On April 29, 2024, as soon as special weather conditions were confirmed over the state of Rio Grande do Sul, the Agency set up a situation room within the scope of its Superintendence for the Supervision of Electricity Services in order to monitor and take the necessary steps to ensure that the energy supply was guaranteed or resumed in Rio Grande do Sul. From that moment on, the area began to provide the ministry and other interested parties with information about the rains and their effects on the structures it monitors.

On May 2, 2024, with the recognition of a state of public calamity in the state of Rio Grande do Sul, the Crisis Committee was set up with the aim of identifying and coordinating the necessary measures and managing ANEEL's actions to deal with the impacts of ongoing climatic events. Inspection teams were sent to monitor the situation on site.

The committee's meetings were held daily in person and by videoconference. The use of the videoconferencing system made it possible to include new players, such as the National Center for Monitoring and Warning of Natural Disasters (Cemaden) and the National Electricity System Operator (ONS).

Depending on the need, other stakeholders were involved in the discussions. This was the case with agents who had experienced dam failure or events that had drastically altered the safety level of their dams. The dynamic of meetings via videoconference made it possible to hold meetings as often as necessary and to involve representatives with decision-making power.

At the same time, the Agency intensified its inspection activities at various dams in the state, guiding sector agents to activate Emergency Action Plans (EAP) at dams in critical situations. Together with Civil Defence, Climate Monitoring Centres and state authorities, ANEEL worked

collaboratively to organize preventive evacuation efforts in risk areas, ensuring that people living near dams were moved to safe places.

Regulatory measures were adopted to ensure that generators complied with the necessary emergency measures, as well as to mitigate the effects of the state of calamity on market agents affected by the calamity in their market operations.

The calamity not only impacted the agents in physical and operational terms, but also generated significant social and environmental impacts. Due to the need to control the level of the reservoirs, equalize the cascade and maintain the safety of the dams, the flow of excess water over the spillways had the side effect of flooding rural areas, resulting in substantial agricultural losses, compromising local ecosystems and evacuating riverside communities and those in flood areas.

ANEEL monitored these operations with the concessionaires and the System Operator to ensure that the dams were operated in such a way as to minimize damage. Additional concerns were raised by the affected populations regarding water quality in the impacted regions. In this context, when necessary, ANEEL advised the environmental agencies on mitigating these impacts.

7. CHALLENGES FACED BY ANEEL IN SUPERVISION AND MONITORING

According to Puga [24], crises are important endurance tests for governance systems in any area, testing the adaptive capacity of their components, testing their limits and demonstrating their bottlenecks. In the case of water resources, explains the author, "extreme events can reveal more information than periods of stability, as they activate different actors and other mechanisms".

The Situation Room helped to reduce the asymmetry of information between the participants, guaranteeing them participation and a voice under more homogeneous conditions of knowledge. The functioning of the Situation Room, therefore, proved to be a governance structure capable of involving government (national and sub-national) and civil society actors with a common goal, despite different perspectives and specificities

In practical terms, one of the biggest problems faced was the lack of technological resources for continuous and effective monitoring of the dams under ANEEL's responsibility. The absence of a real-time monitoring system capable of reporting anomalies or imminent risks has contributed to the perceived vulnerability of these structures during periods of heavy rainfall or other extreme weather events.

Miscommunication between the concessionaires responsible for the dams and the regulatory authorities exacerbated these effects. There was a delay in responding to the first evidence of structural failure, resulting in greater exposure of communities to the risks of dam failure.

Even so, it can be understood that the previous monitoring and inspection work carried out by the Agency to expand and deepen its audits and regulations, gradually demanding greater levels of efficiency, effectiveness, transparency and accountability, contributed to mitigating the effects observed.

Table 1 below shows the historical series of hydroelectric plants installed in Rio Grande do Sul and monitored by ANEEL, classified according to their Safety Level, in accordance with the criteria established in REN 1064/2023.

Table 1 - Safety Level of Dams in Rio Grande do Sul, ANEEL, 2024

Year	Normal	Attention	Alert	Emergency	Total
2019	56	13	1	-	70
2020	67	7	-	-	74
2021	112	5	-	-	117
2022	121	2	-	-	123
2023	120	4	-	-	124

This indicates a positive trend in terms of risk mitigation. Table 2 shows the historical series from the perspective of drawing up Safety Plans, Emergency Action Plans and carrying out Safety Inspections, taking into account projects which, given their characteristics, are required by law to implement safety measures.

Table 2 - Compliance with the Normative Obligations of the NDSP, ANEEL, 2024

Year	DSP	EAP	RSI	SSI	PSR
2019	54	21	56	5	0
2020	55	23	59	4	13
2021	56	33	58	5	13
2022	66	44	63	3	38
2023	67	49	62	16	38

The evolution of which shows a strengthening of the safety culture and the search for regulatory compliance. The increase in the number of Special Safety Inspections carried out

in 2023 reflects the number of plants impacted by similar severe weather events in September and November.

Contributing to this progress are communication campaigns about the obligation to comply with rules and standards, the disclosure of monitoring results with performance indicators and the establishment of a deadline for proof of self-regularization of non-compliant situations.

8. MITIGATION POLICIES AND MEASURES ADOPTED AFTER THE EVENTS

Reversing the impacts caused by the storms in Rio Grande do Sul depends on several factors. This is because dam safety is a multidisciplinary process, made up of structural and non-structural activities. After overcoming the critical moments in Rio Grande do Sul, ANEEL initiated and mapped out a series of measures focused on improving inspection and dam safety protocols. These include:

8.1. Review of Dam Safety Plans (DSP)

The occurrences recorded indicate that Rio Grande do Sul is about to face more frequent and severe impacts related to flooding in the future. Especially if there is no investment in mitigation and adaptation responses. Dam Safety Plans and Emergency Action Plans should serve as a complementary tool to support decision-making for each alert level defined in risk management.

In this sense, the concessionaires operating dams in Rio Grande do Sul were urged to revise their Safety Plans, considering the implementation of new monitoring and continuous inspection technologies, improved communication processes and data review. Although flood forecasts and warnings were available almost a week earlier, media reports reinforce the idea that many of those affected did not receive the warnings or did not understand the seriousness of the situation.

On the other hand, sophisticated simulation methods are of little use if the data is precarious, since no methodology generates information, but only better exploits existing data [25]. In a recent study, Hora *et al*.[26] analyzed decamillennial flood estimates for stretches of the Taquari-Antas river basin in Rio Grande do Sul. The results achieved differed when compared to those defined by the Generation Agents. The article concluded by recommending a review of the design flows of the Rio das Antas Energy Complex spillways, with special emphasis on the Castro Alves Hydroelectric Power Plant spillway.

As the authors make clear, the assertion that a decadal or higher value has been measured in a given watercourse should be viewed with caution. On the contrary, this could be an indication of an underestimation of the design flow, resulting from a probability distribution that

does not represent the river, or from a short series that is insufficient to safely extrapolate the calculation to this magnitude.

8.2. Carrying out Special Safety Inspections (SSIs)

According to REN 1.064/2023, the Special Safety Inspection (SSI) aims to maintain or restore the dam's safety level to the normal category and should be carried out by a multidisciplinary team of specialists, as a substitute for the Regular Safety Inspection, whenever there is a critical change in the dam's safety level, or it shows signs of significant degradation, or is subject to extreme stress, such as seismic tremor, overtopping, flooding or hydraulic operation of overflows under exceptional conditions.

Considering this determination and understanding that adaptation works will be necessary, both in terms of enlarging the overflows and raising the dams, in addition to diagnosing the general state of safety of their dams, the owners were advised during the process to review the hydrological information in their respective river basin and the design criteria for their dam structures, in order to adapt them to the occurrence of more intense rains and floods.

Also, according to REN 1.064/2023, the extent and detail of these studies is proportional to the complexity of the dam and its area of influence, and must be sufficient to guarantee adequate safety conditions for structures and people. Thus, it is likely that a differentiated approach will be observed for small dams compared to large ones: the former were designed with more limited safety criteria and are impacted differently.

8.3. Strengthening Regulation

Strengthening legislation and creating new regulatory instruments are important steps to ensure that dams operate within the expected and required standards. It is therefore essential to constantly review and update safety regulations, ensuring that they are effective and reflect the best technical and regulatory practices. Regulation must be dynamic, adapting to changes in the sector and new technologies. As an opportunity, lessons from international experiences, including disasters, can be adapted to the Brazilian context. More intelligent and effective operating solutions will also be needed, such as improved hydro-meteorological monitoring and forecasting and a review of reservoir operating rules.

8.4. Investment in technology

The geotechnical, environmental and hydraulic risks to which a dam is exposed mean that the storage structures, reservoirs, foundations and dam shoulders must be monitored and controlled, just like the water stored or flowing. In this reality, the use and development of intelligent monitoring tools such as sensors, embedded systems and drones has proved to be

an effective strategy. These resources enable real-time analysis of structural conditions, facilitating early identification of problems. Applications can go beyond monitoring. Artificial Intelligence resources can be incorporated into methods for assessing and validating the Potential Damage of projects. The regulator should encourage the adoption of these technologies by dam operators in order to ensure more effective supervision. Partnerships with technology companies and research institutions should be encouraged.

8.5. Continuous Training and Interinstitutional Cooperation

Inter-institutional cooperation between ANEEL and other regulatory bodies can result in a more integrated and effective approach to inspection and regulation. The same can be said for the need to integrate databases from federal and state institutions. The exchange of experiences and resources through regular training and workshops improves actions to prevent, repair and respond to security events.

8.6. Community participation

The communities affected by emergency plans and risk management must be involved in order to ensure that they are aware of the procedures and prepared for possible emergency situations. According to Villagrán de León [27], warning systems must be planned, implemented and operated with the aim of empowering the people who need them most. Empowerment refers to adequate information about the risks they face and ways to minimize losses in the event of a catastrophic event. People-centered (bottom-to-top) warning systems, in contrast to technically oriented (top-to-bottom) systems, take as their starting point the communities that could be affected and plan the response system according to the characteristics of those communities. In technically oriented systems, the people at risk are incorporated at the end. For the author, the failures of communication in disasters are in contrast to the current trend of risk warning and communication systems.

8.7. Risk communication to society (Fake News)

The occurrence of negative events can make public perceptions more important than reality itself. In the last decade, major dam accidents in Brazil due to safety failures have projected a negative image of these structures, contributing to them not being perceived by the majority of public opinion, as a rule, as controlled and safe assets. These effects have been amplified by the massive dissemination of information. When society is unable to distinguish between true and false information, its trust in institutions and their ability to provide accurate and reliable information is shaken. The risks in a tragedy are great and disinformation amplifies them. Fake news diverts attention to non-existent problems. As one of the regulators of the dam safety process, it is up to ANEEL to expand its activities in promoting practices to combat erroneous

assumptions by making data and reports on dam safety available to the public in an objective and accessible way.

8.8. Attention to Small Dams and their Dissemination

Small dams on watercourses, although they have important and diverse purposes, are not taken into account by the regulatory guidelines established in Brazil, where the regulatory system has been stimulated by discussions analyzing the advantages and disadvantages of large dams only for large enterprises. It must be acknowledged that small dam failures also have disastrous consequences. In China, in 1975, the collapse of the Shimantan and Banquia dams resulted in the cumulative collapse of more than 60 small dams downstream, resulting in the deaths of 230,000 people. According to Viseu [28], the ruptures of large dams receive more attention than those of smaller dams. Ruptures of small dams, built on private property, occur more frequently. This is because, as dams are relatively expensive to build, their owners tend to make them less expensive, generally sacrificing their safety. In terms of historical records, the probability of a small dam breaking is estimated at 2% [29], a figure considerably higher than that associated with large dams, estimated at 0.5% [30]. Weaknesses encountered, whether in licensing or safety aspects, are the main factors driving the practice of establishing small irregular dams on watercourses, disregarding technical and environmental criteria.

9. CONCLUSIONS

The catastrophic floods that hit Rio Grande do Sul between April and May 2024 highlighted the region's vulnerability to extreme weather phenomena. The record rainfall affected around 90% of the state and 2.3 million people, with 640,000 of them losing their homes. Studies published by researchers from various countries project that events of this category in the region will become more frequent and intense in the future.

The worsening is mainly due to the consequences of climate change and the intensification of phenomena such as El Niño, combined with a lack of investment in a protection system. For the authors, the episode exposes the urgent need to improve flood infrastructure.

Dam safety is a critical issue for Brazil, especially given the challenges posed by its dependence on hydroelectric power. The results of the management of dams carried out in the state by ANEEL and other government agencies demonstrate the evolution of the stability conditions of the structures. Out of a universe of 126 potentially affected hydroelectric plants, only two had their dams partially breached, with the effects being restricted to material damage.

However, any dynamic has its resilience limits. Continuous and strategic investments, especially in emerging technologies, are needed to ensure sustainable growth in the long term.

Means of preventing and minimizing engineering risks to prevent extreme events have not yet been fully exhausted.

Future dams can be designed to better balance climate change risks, social and environmental impacts and economic considerations by better evaluating all aspects of the project before construction. New hydroelectricity technology can help to compensate for vulnerabilities to climate change and can make electricity generation more efficient.

In-depth exploration with the identification of new criteria and risk factors to improve the Dam Safety Policy is left as a proposal, in order to guarantee managers a wider range of options. The application of a structured risk analysis process, even at the initial maturity level, promotes efficiency gains to the process while reducing information asymmetry levels.

Finally, it should be emphasized that the reflections contained in this article are part of a subject of significant importance, not only because of the need to understand regulatory guidelines for the treatment of dam safety by the constituted authorities, but also as proposals to support decisions in the technical field by all the actors involved.

REFERENCES

[1] HYDROPOWER (2024), I. E. A. World Hydropower Outlook. Opportunities to Advances Net Zero. (2024).

[2] ABDENUR, A. E.; KUELE, G.; AMORIM, A. (2019). Climate and Security in Latin America and the Caribbean. Igarapé Institute, (2019).

[3] Belt, J. (2015) How Will Latin America Deal with Its Hydropower Problem, Greenbiz 20 May. Available at: <https://www.greenbiz.com/article/how-will-latin-america-deal-its-hydropower-problem>.

[4] BRUNNER, M. I. (2021). et al. An Extremeness Threshold Determines the Regional Response of Floods to changes in rainfall extremes. Communications Earth & Environment, v. 2, n. 1, p. 173 (2021).

[5] BRAZIL. (2023): National Water Agency - Dam Safety Report. Technical Report, National Water Agency - ANA, Brasília (2023). Available at: <https://www.snisb.gov.br/portal-snisb/documentos-e-capacitacoes/rsb/. 2023>

[6] BRAZIL. (2010). Law No. 12.334, of September 20, 2010. establishes the National Dam Safety Policy. Official Gazette of the Federative Republic of Brazil, Section 1, September 21, 2010, p.1, 2010. Available at: <http://www.planalto.gov.br/ccivil/Leis/L9427cons.htm>.

[7] BRAZIL. (2023). National Electric Energy Agency – ANEEL. Normative Resolution No. 1,064, of May 2, 2023. Available at: < https://www2.aneel.gov.br/cedoc/ren20231064.pdf>

[8] BRAZIL. (2012). Resolution No. 144 of July 10, 2012 of National Water Resources Council (CNRH). Establishes guidelines for implementing the National Dam Safety Policy, applying its instruments and operating the National Dam Safety Information System, in compliance with art. 20 of Law No. 12.334, of September 20, 2010. Available at: <https://sistemas.anm.gov.br/publicacao/mostra_imagem.asp?IDBancoArquivoArquivo=7234>

[9] FRITZSONS, E.; WREVE, M. S.; MANTOVANI, L. E. (2015) Altitude and Temperature: A Study of the Thermal Gradient in Rio Grande do Sul. Revista Brasileira de Climatologia, v. 16 (2015).

[10] RODRIGUES, A. A. et al. Rainfall Trend and Variability in Rio Grande Do Sul, Brazil.

[11] SATYAMURTY, P.; NOBRE, C. A.; SILVA DIAS, P. L. (1998) South America. In: Meteorology of the Southern Hemisphere. American Meteorological Society, Boston, MA, p. 119-139, (1998). Available at: <https://link.springer.com/chapter/10.1007/978-1-935704-10-2_5>

[12] REBOITA, M. S.; GAN, M. A.; ROCHA, R. P.; AMBRIZZI, T. (2010) Precipitation Regimes in South America: a Literature Review. Revista brasileira de meteorologia, v. 25, p. 185-204. (2010). Available at: <https://www.scielo.br/j/rbmet/a/yhrG5QPXhBCkZXcGKgyzWvf/?lang=pt&format=pdf>

[13] RIO GRANDE DO SUL. (2023) Socio-economic Atlas of Rio Grande do Sul. Department of Planning, Governance and Management - SPGG. Porto Alegre: (2023).

[14] RIO GRANDE DO SUL. (2023). State Dam Safety Report. Secretariat for the Environment and Infrastructure - SEMA. Porto Alegre: (2023).

[15] BRAZIL. (2023). National Electric Energy Agency – ANEEL. Dam Safety Inspection. 2024. Available at: https://dadosabertos.aneel.gov.br/dataset/fsb-fiscalizacao-de-seguranca-de-barragens. Accessed on: Aug. 19, 2024.

[16] BRAZIL. (2024). National Electric Energy Agency – ANEEL. Dam Safety Panel. Available at: <https://app.powerbi.com/view?r=eyJrIjoiMzQ4MmMyZDItNWJhOC00NWYwLTgyMDAtNjk1Zjc0ZWI1NTNhIiwidCI6IjQwZDZmOWI4LWVjYTctNDZhMi05MmQ0LWVhNGU5YzAxNzBlMSIsImMiOjR9>

[17] SIAS E. (2024). MetSul, ed. Heat Wave in Brazil Will Worsen with Historic Records and Up to 40ºC. Consulted on May 7, 2024.

[18] RIO GRANDE DO SUL. (2024). Hydrometeorological Warning 254-Update" (PDF). SEMA – RS.

[19] IWPDC (2024). International Water Power & Dam Construction. Available at: <https://www.waterpowermagazine.com/news/dam-of-14-de-julho-power-plant-in-brazil-partially-collapses-fatalities-reported-11737530>

[20] "Tornado in Rio Grande do Sul Amid Rain Disaster". MetSul Meteorologia. May 11, 2024.

[21] VRKIC, D. (2024) Climate Change, El Niño and Infrastructure Failures Behind Massive Floods in Southern Brazil (2024).

[22] ROMANY, I. (2024). The Brazilian Report. (2024). Available at: <https://brazilian.report/liveblog/brazil-floods/2024/05/09/south-brazil-devastating-floods/>

[23] VALENTE, P. T. (2018) Extreme Precipitation Events in Rio Grande do Sul in the 20th Century Based on Reanalysis Data and Historical Records. (2018).

[24] PUGA B. P. (2018) Governança dos Recursos Hídricos e Eventos Climáticos Extremos: a Crise Hídrica de São Paulo (2018).

[25] CEEE/IPH. Companhia Estadual de Energia Elétrica / Instituto de Pesquisas Hidráulicas. Regionalization of flows in Rio Grande do Sul - Volume I: Development of Regionalization. Tucci, C. E. M. (Coord.). Porto Alegre. (1991)

[26] DA HORA, A. F. et al. (2023). Evaluation of Decamillennial Flows in Hydroelectric Plants and in the Middle and Lower Stretches of the Rio das Antas/RS. DELOS: Desarrollo Local Sostenible, v. 16, n. 49, p. 3647-3662 (2023).

[27] VILLAGRÁN DE LEÓN, J.C (2012). Early Warning Principles and Systems. In: WISNER, B; GAILLARD, J.C; KELMAN, T. Handbook of Hazards and Disaster Risk Reduction, (2012).

[28] VISEU, T. (2006). Segurança dos Vales a Jusante de Barragens: Metodologia de Apoio à Gestão do Risco. Doctoral Thesis, Instituto Superior Técnico, Lisboa (2006).

[29] LEMPÉRIÈRE, F. (2017). Dams and Floods. Engineering, v. 3, n. 1, p. 144–149 (2017).

[30] ICOLD (1995): Dam Failures: A Statistical Analysis. ICOLD Bulletin, n. 99. International Commission on Large Dams ICOLD, Paris (1995).

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

THE ROLE OF THE BRAZILIAN ELECTRIC ENERGY AGENCY IN DAM SAFETY - A REVIEW OF RECENT YEARS AND PROSPECTS FOR THE FUTURE

Leite, Sérgio R.[1]
Brazilian Electricity Regulatory Agency

Silva, Ludimila L.[2]
Brazilian Electricity Regulatory Agency

Mendes, Marcelo P.[3]
Brazilian Electricity Regulatory Agency

ABSTRACT

Increased discussion of the risks imposed on society as a result of the construction of reservoirs has demanded greater preparation on the part of dam owners and the authorities.

For the former, it is not enough just to strive to meet standardized criteria, such as minimum resistance values and safety factors. For the authorities, in addition to promoting stricter regulatory environments, there is a need to format mechanisms and performance indicators that promote better management of their information base.

In the Brazilian context, dams are aging, which favors the emergence of deficient behavior on the part of these structures, thus reinforcing the need for care with regard to their safety.

Such care requires not only structural activities, such as maintenance and reinforcement works, but also non-structural actions that reduce the risk through preventive operative measures or that make it possible to detect dangerous events in good times, such as monitoring.

Against this backdrop, the Brazilian Electricity Regulatory Agency (ANEEL), which is responsible for overseeing the construction and operation of hydroelectric power plants, has restructured its activities in recent years, taking as its guideline, the improvement of knowledge and techniques linked to the process of monitoring dams from approaches based on risk assessment and management.

The aim of this article is to present a sensitivity analysis which seeks to assess the impact of the procedures adopted by ANEEL in recent years, as well as the challenges faced by the Agency in monitoring the safety of hydroelectric dams.

Conjecturally, risk mapping based on the classification indices established by Brazilian legislation has seen reductions of over 5% in the last decade.

From this perspective, it was found that the responsive model enabled a broad analysis of the performance and integrity of the structures evaluated, serving as a basis for identifying specific vulnerabilities, and the results presented for the scenarios evaluated were considered to be in line with the preferences of the regulators and the decision-maker.

Keywords: Dam Safety, Risk Management, Regulation, Enforcement.

[1]sergioleite@aneel.gov.br, Brazil
[2]ludimila@aneel.gov.br, Brazil
[3]marcelomendes@aneel.gov.br, Brazil

1. INTRODUCTION

Dams are structures designed to retain liquids or mixtures of liquids and solids in a controllable and safe manner [1]. They are an important vector of socio-economic and territorial development, settling populations and consolidating nations. Their existence cannot be ignored or ignored, and their history is closely linked to that of humanity.

Despite their benefits, like any infrastructure project, they are subject to failures and operational problems that can compromise their performance. Jansen [2] states that the risk of dam failure is one of the unavoidable burdens that humanity must bear.

Although small, the risk of a dam collapse is a potential reality and has been a public concern. According to the Dam Safety Report [3], 2023 edition, drawn up by the National Water and Sanitation Agency (ANA), Brazil records an average of twenty dam accidents every year.

The risks associated with dam failures are, in most cases, major. They involve threats to human life, material, environmental and social damage. In addition, the spatial arrangement of a dam favors the spread of disaster on a large scale, since they are commonly located in river valleys, streams and tributaries occupied downstream by urban centers and other forms of population densification.

In this context, dam safety policies and programs have become increasingly important, demanding greater preparation from dam owners and the authorities. For owners, it is not enough just to strive to meet standardized criteria, such as minimum resistance values and safety factors. For policymakers, in addition to stricter regulatory environments, there is a need to develop mechanisms and performance indicators that promote better management of their information base [4].

Dam safety is a crucial issue in a country like Brazil, where around 60% of the energy generated comes from hydroelectric plants. Dams play a vital role in the energy matrix, but they also pose significant risks, especially if they are not properly maintained and managed. ANEEL, as the regulatory body for the electricity sector, is responsible for overseeing the safety of these structures, ensuring that they comply with rules and regulations designed to protect the population and the environment.

This text sets out to explore the various aspects related to ANEEL and the challenges of dam safety, analyzing current legislation, safety standards, disasters and lessons learned. We will also discuss mitigation strategies and recommendations for improving the safety of these essential structures for the energy sector.

2. ANEEL'S HISTORY

2.1. Creation and Mission

At the end of the 1990s, there were experiments in transforming the bureaucratic public administration model into a managerial one in Brazil. In some sectors, the state no longer acted as the sole and direct provider of public services, but as a regulatory agent for the provision of these services [5].

In a context of privatization of state-owned companies, predominant in the provision of public services, the Brazilian Electricity Regulatory Agency (ANEEL) emerged on the Brazilian institutional scene in 1996 as part of a movement to restructure its electricity sector.

The regulatory agency for the electricity sector was created to regulate and supervise the production, transmission, distribution and commercialization of electricity, in accordance with the policies and guidelines of the federal government, promoting competition, low tariffs and universal access to energy.

In this context, with regard to the regulatory inspection activity, it was established by regulation that the agency's objectives should be achieved primarily through actions aimed at providing guidance and preventing conduct that violates the law and contracts. ANEEL was also given the authority to apply regulatory and contractual penalties to the agents it regulates.

With the expansion of hydroelectric power, dam safety has become a fundamental part of the agency's responsibilities.

2.2. Evolution of Regulation

Since its creation, ANEEL has gone through several phases of regulatory evolution, adapting to the scenarios and technological advances arising in the sector. In 2015, the Agency started the Strategic Inspection Project, using as its main theoretical reference regulatory inspection techniques presented by the OECD [6], which are part of a set of documents focused on disseminating principles of good practice for regulatory policies.

The principles presented by the OECD are: 1. evidence-based supervision; 2. selectivity; 3. focus on risks and proportionality; 4. responsive regulation; 5. long-term vision; 6. coordination and consolidation; 7. transparent governance; 8. integration of information; 9. clear and fair process; 10. incentives for compliance; 11. professionalism.

Some of these principles (7 and 9) were already practiced in ANEEL's traditional inspection model and were reaffirmed, while the incorporation of others (1, 2, 3, 4, 10) were the focus of the reform reported here and some (5, 6, 8, 11) remain current challenges.

In the context of Dam Safety, among the models for differentiating regulatory risk, the concept of responsive inspection was adopted, which suggests modulating inspection actions according to the response of sector agents to regulatory commands and their capacity and commitment in the pursuit of regulatory compliance.

This means that agents who systematically commit violations are considered to be at greater risk and are inspected and punished more frequently. Agents with a history of good regulatory compliance practices are seen as lower risk and can be inspected less frequently and given chances to improve the quality of the services provided. [7].

This idea is often represented schematically by means of a pyramid of regulatory compliance expressed visually in Fig. 1 below:

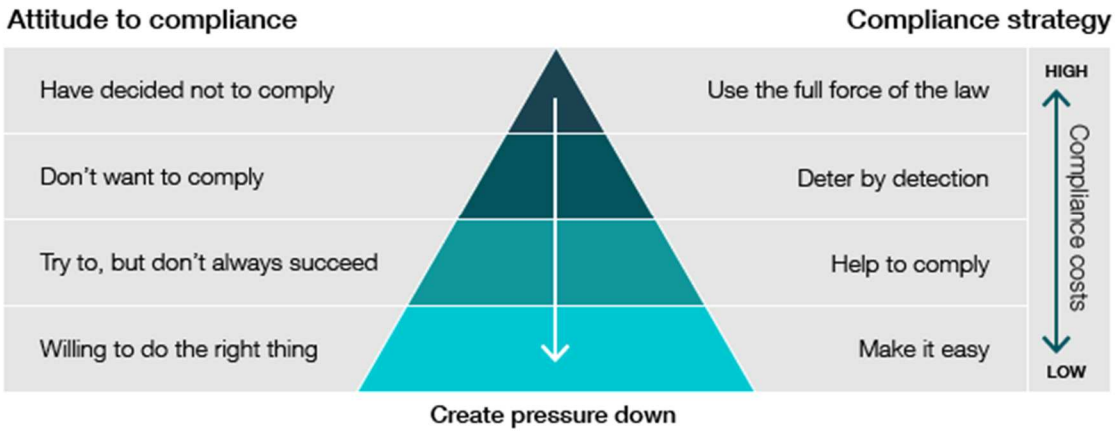


Fig. 1 - Compliance model. Adapted from Australian Taxation Office (ATO), 2024

To incorporate this concept, a 3-level inspection model was formulated in 2015 [8], as shown in Fig. 2.

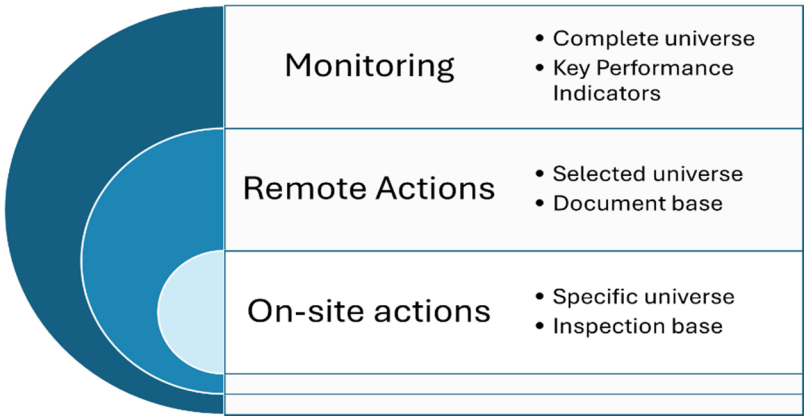


Fig. 2 - 3-Level Inspection Model.

The regulation of dam safety has been stepped up over time, especially after disasters that revealed shortcomings in the supervision and maintenance of these structures.

3. LEGISLATION AND SAFETY STANDARDS FOR DAMS

3.1. National Dam Safety Policy

As recorded by Costa [9], the starting point for a historical overview of dams in Brazil can be considered to be the year 1877, when a major drought hit the northeast of the country, affecting the resident population and livestock farming. At the time, this caused great concern to the Emperor of Brazil, who set up a commission to solve the problem of drought in the region.

At this time, the first officially known dam was built: the Cedros Dam, in the state of Ceará, on the Jaguaribe system. Since then, countless dams have been built all over the country for various purposes, such as irrigation, electricity generation and supplying large cities.

According to the author, dams first came into being with the aim of supplying a certain region, but were later also used to produce electricity and to contain floods, among other needs. In view of the various issues related to the construction of dams, there was a concern and a need to create rules to guarantee orderly exploitation and relations between the various entities involved.

However, regulation in Brazil did not take place in full, i.e. it was not legislated to cover all the issues inherent to dams. During the imperial period, the only legislation implemented was a charter of 1819, which extended to Brazil some paragraphs of a Portuguese charter of 1804 that regulated the construction and use of water in canals at the request of sugar mill owners [10].

Except for these small initiatives, no study or legal text has addressed the issue of the use of water or any other related resources [11].

In Brazil, the regulatory framework for dam safety only came about in 2010 with the enactment of Law No. 12.334 [12], which established the National Dam Safety Policy - NDSP and created the National Dam Safety Information System - SNISB, as a result of society's recognition of the high level of organizational problems responsible for the general state of abandonment of hundreds of Brazilian dams, with latent vulnerabilities in the design, construction and operation of existing structures [13].

The NDSP is a regulatory framework that defines responsibilities for various bodies, including ANEEL. Among the instruments foreseen by law, the following should be highlighted: the classification of dams by risk category and potential damage associated, as well as the need for a Dam Safety Plan, and its minimum content, if the project fits among the categories of medium or high damage in terms of economic, social, environmental or loss of human life.

The Standards Based Approach is the rule in dam engineering [14]. In it risks are controlled by compliance with established rules for design situations, structural resistance, safety

coefficients, besides the adoption of defensive design measures [15]. The risk management concept incorporation in the law text reflects the maturing of the theme in the electricity sector, indicating that security is not an absolute condition, but rather a tolerable situation, with residual risk levels always present [16].

Such directive forces the dam owner to deal with actions, performance and consequences in an integrated manner by properly considering the enterprise's nature and all external factors that affect it. This position change also includes a paradigm shift. The adoption of a bottom-up approach where local actors should participate in decision-making process around the community in which they belong, in defining the strategy to be used in accident prevention and response and in the selection of priorities forces the interaction between communities, companies and authorities to achieve better global results, stimulating the development of a collaborative governance culture.

3.2. ANEEL Resolutions

The regulation of Law No. 12.334/2010 with the specificities of the electricity sector is currently governed by ANEEL Normative Resolution No. 1.064 of May 2, 2023 - REN 1064/2023 [17]. As provided in article 7 of Federal Law No. 12.334 , the dams will be classified by the inspection agents by Risk Category - RC, by Potential Damage Associated - PDA and by their volum, based on general criteria established by Brazilian National Water Resources Council (CNRH). These criteria, synthesized in 4 matrices, are part of Annex II of Resolution CNRH No. 143/2012, of July 10, 2012 [18]. The matrix structure established for dams classification by RC and PDA is summarized in Fig. 3.

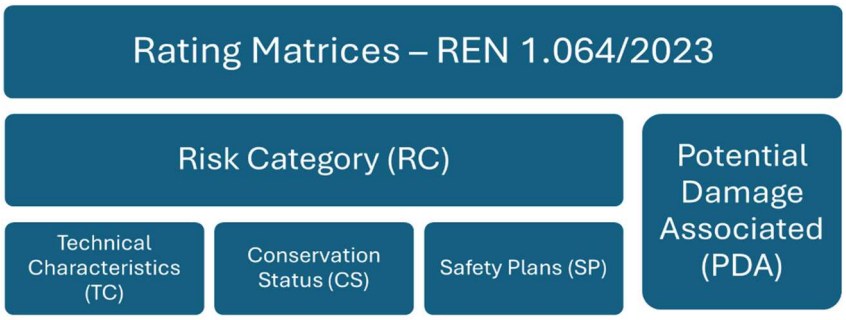


Fig. 3 - Matrix Structure for Evaluating Dams. ANEEL, 2023

As for dam Risk Category, the classification criteria defined by law are intended to represent, according to existing statistical studies of rupture, the vulnerability index into an accident in the massif, in terms of their: (i) intrinsic Technical Characteristics (TC); (ii) structures current Conservation Status (CS); and (iii) Safety Plans (SP) or maintenance procedures.

On the other hand, the classification in terms of Potential Damage Associated is related to the consequence in which all dams are leveled when considering downstream effects, highlighting

the damages that can be caused by the flood induced by the dam break in different scenarios of human life losses, environmental and socioeconomic impact.

It can be seen that the characteristics that lead a dam to be classified under the NDSP are, in some way, related to the greater potential for causing damage downstream, whether economic, environmental or social, demanding proportionally greater demands on safety management, with a view to preventing incidents, accidents and disasters.

The scores obtained by descriptors are summed by criterion, in order to classify the dam as RC and PDA, in high, medium or low. The results combinations obtained give the dam final classification, as established in Specific Risk Classification Matrix, as follows.

RISK CATEGORY	POTENTIAL DAMAGE		
	High	Medium	Low
High	A	B	B
Medium	B	C	C
Low	B	C	C

Fig. 4 - Dams final classification matrix. ANEEL, 2023

Where:

A. Dams that present a high potential risk and damage category and whose anomalies require short-term intervention to maintain safety conditions, not always signifying an immediate risk of rupture;
B. Dams that present medium or low risk category and average potential damage and whose anomalies must be controlled, monitored and interventions can be implemented over the time to maintain safety conditions; and
C. Dams that present medium or low risk and potential damage and which do not present anomalies and existing ones do not compromise the safety of the dam.

Based on the results of the classification, the resolution defines criteria for drawing up Safety Plans, Regular Inspections (RI), Special Safety Inspections (SSI) and Periodic Safety Reviews (PSR) for dams, as well as establishing guidelines for drawing up Emergency Action Plans for dams considered to be high or medium risk. The standard is an important tool for implementing good safety practices.

Since the principles of a law are to be clear (easy to understand), precise (not prolix or sparse), and ostentatious (easy to notice or perceive), it can be seen that Law No. 12.334/2010, was based on principles of the Scoring Method to achieve its goals.

It is, therefore, an Index-Based Qualitative Risk Analysis process that enables an expeditious risk-based classification and that employs specific dam engineering knowledge. In terms of

dam safety this approach is not considered inappropriate, although quantitative analyses are more consistent as they employ numerical values of probabilities and consequences.

The classification is also important for the prioritization of actions by regulatory agencies. According to the OECD [6], regulators in many countries are increasingly pressured to do "more with less." While increasing demands for better protection of the environment, the health and safety of citizens, economic crises compel governments to reduce government spending, including regulatory oversight.

4. ANEEL'S ROLE IN REGULATION AND SUPERVISION

ANEEL has a supervisory role with regard to dam safety. This involves analyzing construction projects and inspecting the operating conditions of dams in operation. The agency is also responsible for authorizing the operation of new dams, taking into account structural and environmental safety.

In order to ensure that operators comply with safety standards, ANEEL's inspection processes include periodic inspections, analysis of monitoring reports and assessment of the operational conditions of dams.

For the power generation segment, the strategies used to apply the principle of promoting regulatory compliance are modulated, taking into account the very diverse universe of company characteristics: size, experience, time on the market, process technology, among others.

This condition acquires significant contours in the scope of Hydropower Plants in Brazil. It must be recognized, however, that greater security rigors impose costs on companies, affecting their economic and financial balance. In addition, some owners do not easily absorb the philosophy of the security program to be adopted, making it difficult to implement, requiring a convincing job by the regulator.

From this perspective, for example, according to current regulations, Dam Safety Plan must be available and accessible before the first filling of the reservoir begins as a condition for the plant's release beginning commercial operations of its generating units.

Thus, each proposed measure must have its cost-benefit carefully analyzed, otherwise a good safety program will never be implemented.

5. CHALLENGES FACED BY ANEEL IN IMPLEMENTING THE NATIONAL DAM SAFETY POLICY

The evaluation of public policies is a theoretical and practical field based on different protocols or methodological roadmaps that adapt to the context in which they are carried out and to the very nature of the objectives involved [19].

With regard to Dam Safety, the main problem lies in the difficulties or shortcomings faced in fully exercising preventive risk management in these structures throughout their life cycle, from design to decommissioning.

The methodology used in this article to evaluate the National Dam Safety Policy was based on the *Ex-post* Analysis of Law No. 12.334 [20], of 2010, carried out by the Ministry of Integration and Regional Development - MIDR, in 2021, based on the guidelines of the *Ex-post* Evaluation Guide, by the Civilian House of the Presidency and Applied Economic Research Institute (IPEA) [21]. This process was combined with the stages of document collection, data collection and, subsequently, conversations and semi-structured interviews with focal points (regulators and experts) working in the dam safety and inspection process, totaling 30 interviewees.

The approach adopted was more formative than summative, in the sense of evaluating the functioning of the policy, given the NDSP nature, which with only fourteen years of defined national legal framework can be said to still be in a consolidation phase.

This analysis made it possible to identify the extent to which the NDSP had achieved its objectives (to varying degrees and depths), but above all, what challenges remain. The main points identified are presented and discussed below.

5.1. Regulations

As discussed, enforcement aims to guarantee compliance with legislation and ensure that entrepreneurs behave appropriately. The general perception is that, with the exception of specific points, there is no lack of rules, nor is the current regulatory framework of poor quality. In this context, the problem is restricted to legal fragmentation or disarticulation and the fragility of its implementation.

Given its cross-cutting nature, there are many players involved in implementing the NDSP, which in addition to the entrepreneur and the public authorities acting at various federal levels, includes civil protection and defense and environmental agencies. The lack of a nationally unified regulatory system (norms and rules), with points of legal overlap and fragmentation, and the content of the norms act as factors that hinder its implementation.

An example of this would be the application of legal norms that are indistinguishable from the type or size of the development or the low technical-regulatory detail for the implementation of EAPs, Contingency Plans (PLANCON) or the development of flood maps.

In turn, the aspect of regulatory fragmentation or disarticulation stems, according to the interviewees, from the absence of institutional instruments such as a high-level national authority for regulatory development, or from the very constitutional condition of autonomy for states and municipalities. This autonomy obviously makes the problem more complex, requiring different strategies for articulation and persuasion.

In this context, as recorded in the Dam Safety Report [3], drawn up by the National Water and Sanitation Agency, it is understood that the circumstances of the evaluation of the implementation of the NDSP in the national scenario bring results with still discrete developments.

To compensate for this, a measure adopted by ANEEL to promote greater incentive for regulatory compliance has been the drafting or revision of internal procedures, guides for inspectors on how to select objects based on risk assessment and how to proceed with inspections. These tools are important because the lack of consistency in the interpretation of requirements between inspectors and the lack of predictability as to the objects and methods of inspection do not contribute to the overall level of compliance and create burdens for companies, especially small and medium-sized ones.

5.2. Communication

The risks associated with dam collapses are, in most cases, of major proportions. In the last decade, Brazil has experienced several dam disasters, the most serious of which was the collapse of the tailings dam in the Córrego do Feijão region, in the Brazilian municipality of Brumadinho, 65 km from Belo Horizonte, in Minas Gerais, on January 25, 2019.

The national repercussions of accidents of this size have contributed to dams no longer being seen by the majority of public opinion as social and economic assets linked to the development of a region or country.

Part of this perception can also be attributed to the nature of the NDSP itself: guaranteeing safety presupposes the possibility of a situation of insecurity, whether minor or major. This perception produces a bias that forms a negative image of the subject (dams) in the population.

Aware of this structural difficulty, the agency understands that functional or institutional factors that corroborate the prevailing alarmist view are: (1) the absence of communication strategies that contain effective initiatives to reach the general public in terms of image and perception; (2) technical language and concepts are not decoded (risk and risk categories); (3) the National

Dam Safety Information System (SNISB) is not suitable for broad communication and does not reach opinion formers and (4) Emergency Action Plans (EAPs) and Civil Protection and Defense Contingency Plans (PLANCONs) for dams are not didactic and have low communicability.

5.3. Ageing Structures

Regardless of initial forecasts, dams can present age-related deficiencies, requiring corrections to maintain adequate levels of safety. In the Brazilian context, the hydroelectric generating matrix is old, with 52% of its large hydroelectric plants built by the end of the 1980s [22].

This reality favors the emergence of deficient behavior on the part of these structures, thus reinforcing the need for care with regard to their safety. Continuous safety inspections and rehabilitation activities are therefore essential to mitigate this risk.

It should also be noted that construction techniques and safety standards that were considered adequate when these structures were built may no longer meet current requirements.

The recurrence of accidents and incidents in dams, even in the case of structures that fall under its legal competence, led ANEEL to question the reasons why the control measures adopted were not having the expected effects. In a study carried out in 2019 [4, 23], the agency questioned its regulated entities in order to validate factors defined as risk in its regulations and in the NDSP.

Apart from the causes of possible maintenance failures, outdated documentation and monitoring systems have been identified as one of the main risk factors in old dams. The availability of design and construction information is essential for understanding the behavior of structures and for enabling safety analyses and assessments. Poorly written work instructions and operating procedures, inaccurate drawings or designs were pointed out as one of the main points of uncertainty in the safety of the dams evaluated.

Taking these factors into account, ANEEL's regulations and inspections have started to demand more explicitly, especially for developers of old dams, whose designs do not exist or are unknown, cadastral surveys of their structures in order to obtain: geometry, information on the foundations and construction materials.

However, as Melo and Fusaro [24] explains, the level of detail of this survey depends on the complexity of the structures, but it must allow for an analysis of the overall stability of the dam and the determination of the capacity of the overflows.

ANEEL has no regulation governing the de-characterization of dams, even in cases where it is economically advantageous, considering issues of social and environmental relevance.

5.4. Climate Change

Evaluating the occurrence of extreme events in large civil structures, such as dams, is of great importance in a more complete risk analysis study, both for managers and decision-makers.

The prevention of new tragedies, however, depends on the lessons learned from previous accidents, regardless of their size. Accidents act as revelations of organizational dysfunction, questioning our capacity for analysis and diagnosis. In general, the solution to mitigating these risks requires, in addition to rigorous periodic inspection, the adoption of more efficient structural and non-structural studies and techniques.

For example, as recorded in the Dam Safety Report [3], it was found that in 2023 in Brazil, of the 50 accidents and incidents reported, the majority were caused or aggravated by heavy rainfall. Heavy rainfall was reported in 92% of the accidents (23 cases), and in 8% (2 cases) there was no record of extreme events.

Among the main failure modes, the greatest occurrence was attributed to overtopping, slips and "unknown factors". In the majority of cases, the structures damaged were the downstream slopes or walls, followed by damage to the embankment in the case of accidents and, in the case of incidents, damage to the spillway. In 34 (68%) cases, the damage was to earth or rock-fill dams, and only 2 (4%) to concrete structures.

As for the main use: 33 cases (66% of accidents and incidents) were water storage dams, except for hydroelectric generation, and in 20% of cases the use was not informed.

In a risk-oriented perspective, in which actions, performance and consequences are treated in an integrated manner, the correct classification of the levels of commitment of a structure or system makes it possible to detect dangerous events in good time.

In this context, ANEEL has been restructuring its activities with a view to improving the knowledge and techniques linked to the dam monitoring process, based on risk assessment and management approaches.

The strategies chosen to prevent or promote compliance include: (1) the publication of reports indicating the performance category of each company for the market and society; (2) the issuing of warnings to companies with inadequate performance and guidance on correction; and (3) the establishment of improvement plans for companies with insufficient performance and intermediate risks to the quality of service provision.

Between the end of April and the beginning of May 2024, the Brazilian state of Rio Grande do Sul was hit by flooding due to precipitation, which led the local government to classify the situation as "the biggest climate catastrophe in the state's history". The excessive rainfall affected more than 60% of the state's territory.

Out of a total of 126 potentially affected hydroelectric plants, only two had their dams partially breached, and the effects were restricted to material damage. No loss of human life was recorded.

The result observed, although insufficient to definitively validate the inspection strategy adopted to incorporate the principles pointed out, indicates a good level of response to the policies and guidelines implemented. ANEEL, however, has yet to consider the effects of the climate transition in its regulations and to ensure that the Emergency Action Plans for dams and other associated structures are adapted to this new reality.

5.5. Supervision and Limited Resources

In this respect, the circumstances for evaluating the implementation of the NDSP refer to CNRH Motions No. 76 and 79, of November 2022 [25], addressed to the federal and state levels of government, highlighting the need to establish legal norms that provide financial resources for structuring civil protection and defense agencies, as well as the creation of a specific budget line for dam safety actions, especially for maintenance, operation and compliance with Law No. 12,334.

This position reveals the lack of a sufficiently robust financing model in the NDSP to enable the full implementation of all the instruments, rules and obligations set out in the policy.

As highlighted in the *ex-post* analysis of Law No. 12.334 [20], this issue has a dual nature. In the public sphere, the only source for the NDSP comes from the budgets of the entities involved, which is considered insufficient to achieve the objectives. On the private side, there is low capitalization of small entrepreneurs. Also according to the report, with the exception of some energy and mining cases, the document concludes that the artifact, whether public or private, dam or reservoir, is not treated as an economically profitable or viable asset.

As for the teams from the supervisory bodies, it is important to consider the scale of the brazilian energy matrix, which is still predominantly hydroelectric. According to data from the Brazilian Energy Research Office (EPE) [26], Brazil's hydroelectric potential is estimated at 172 GW, of which more than 60% has already been tapped. Approximately 70% of the untapped potential is located in the Amazon and Tocantins-Araguaia river basins, against the agency's capacity to carry out periodic inspections. It should be noted that, in addition to the lack of a minimum number of dedicated technicians, professionals with specialized training and the development of technologies are needed to meet the requirements set out in the NDSP.

As a mitigating response, ANEEL strongly incorporated the concept of evidence-based inspection into its approach, in which inspections are based on evidence and measurement. To this end, it was necessary to intensify and systematize monitoring activities in the sector in

order to collect and evaluate data. Inspection needs and opportunities were then identified based on indicators of risk to compliance and the quality of regulatory results.

The choice of topics, companies and facilities to be inspected became the result of monitoring; the topics and objects of inspection themselves were revised, created or extinguished; criteria and procedures for planning and carrying out inspections were modified. To maximize the availability of evidence, instruments and procedures were improved for intensive and electronic data collection, data storage in structured databases, as well as processing using analytical intelligence tools to determine performance indicators in terms of compliance and regulatory results.

5.6. Risk Management and Emergency Plans

One of the most important definitions of dam safety contained in the NDSP is that of the responsibilities of the owner / operator and the supervisory bodies. Before this, it was unclear who was responsible, with a clear lack of coordination between public authorities and environmental agencies dealing with accidents in isolation [27].

According to the NDSP, the primary responsibility lies with the owner, who is legally liable for the safety of the dam, for damage resulting from its failure, leakage or malfunction, and is responsible for taking measures to ensure its safety and repair damage, regardless of fault.

With regard to the responsibilities of the owner, twenty-two competencies have been defined by the NDSP, of which the following stand out: arranging for the preparation and updating of the Dam Safety Plan (DSP) and presenting Emergency Action Plans (EAP).

Considering that the preparation of DSP and EAP plans demands challenges for the entrepreneur, both financial and technical, the availability of models or standards can contribute substantially to the regularization of dams, as well as small entrepreneurs. In addition, the lack of a uniform format can lead to inconsistencies and difficulties in the assessment of projects by regulatory bodies.

In scenario studies, for example, the worst or most serious scenario must be considered, within the line of possible causality and technical-mathematical bases. Flood simulations, on the other hand, are subject to a high degree of uncertainty and imprecision, identified as factors that make the results sensitive, such as the cause of the breach, the width and formation time of the breach, among others, calculated in various ways [28].

In turn, simplified hydrograph models such as simplified triangular and parabolic models can be used to estimate reservoir emptying times, and there are also various approaches to determining maximum flows. As noted by Sampaio [29], such variation in analysis and technical assessment must be subject to strict control by the public authorities. Thus, it is to

be expected that the inspection body will have professionals capable of examining the studies carried out by the developer and pointing out corrections when necessary.

Brazilian legislation is deficient in this respect, as it does not create mechanisms that allow for an in-depth examination of the EAP drawn up by the entrepreneur. This gap reduces the EAP to a duty to deliver a mere document to some public authorities and to provide additional information. In addition, in terms of implementation, it introduces an untimely factor of uncertainty, dependent on a serious environmental compliance and safety policy on the part of the entrepreneur or their fear that failure to comply with the standard, in the event of an accident or disaster, will be costly to them [29].

5.7. National Civil Protection and Defense Plan

Although Article 179 of Brazil's first constitution already mentioned guaranteeing public assistance [30], it was only after the 1988 Federal Constitution that a national civil defense system was created. Over a period of 24 years, this system has undergone successive changes [31]. Therefore, it can be said that it is a structure that is still in search of a better institutional design, not yet fully consolidated. The existence of an integrated Civil Defense system acting across the board would make a major contribution to achieving public policies on: (1) territorial planning, by controlling the use and occupation of land downstream of dams; (2) climate change and (3) water resource management with a view to protecting the population.

6. RESULTS OBTAINED

Medeiros [32] defines that in order to develop an efficient dam safety regulatory structure, it is necessary to have the direct participation of all the entities involved in the issue, as shown in Fig. 5.

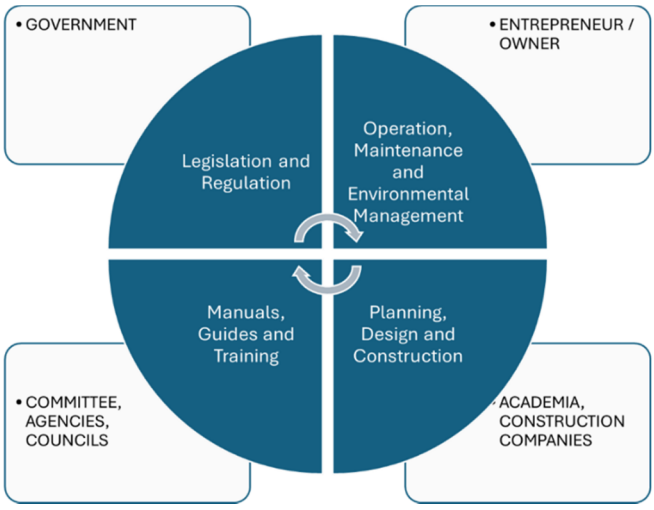


Fig. 5 - Dam safety regulatory structure [32].

Thus, in order to develop a dam safety culture, the existence of legislation alone is not enough. Among other factors, it is necessary for the supervisory bodies to be independent, to have the necessary resources, a trained technical staff and a number compatible with the number of dams that need to be inspected within their remit [4].

Despite these challenges, the regulatory body has made progress by incorporating new theoretical concepts and good practices validated by other institutions, to the extent that the objectives and targets are now related to promoting compliance and regulatory results for the agency's inspection teams and senior management.

A significant result in the category of improving the level of regulatory compliance concerns the significant increase in the rate of compliance with the obligation to send information to ANEEL on the classification and classification of dams. This information, which is self-declaratory in nature, is submitted periodically by electronic form and reports compliance or non-compliance with the obligations established by the NDSP by companies that have received authorization to operate hydroelectric dams [33].

This increase in scope can be seen in Fig. 6, which shows the annual historical series of dams assessed and classified by ANEEL in terms of Risk Category and Associated Potential Damage, according to the methodology detailed in section 3.2. Evaluated dams which, due to their reduced impact, do not fit the criteria for application of the NDSP, receive the classification NC (Not Covered).

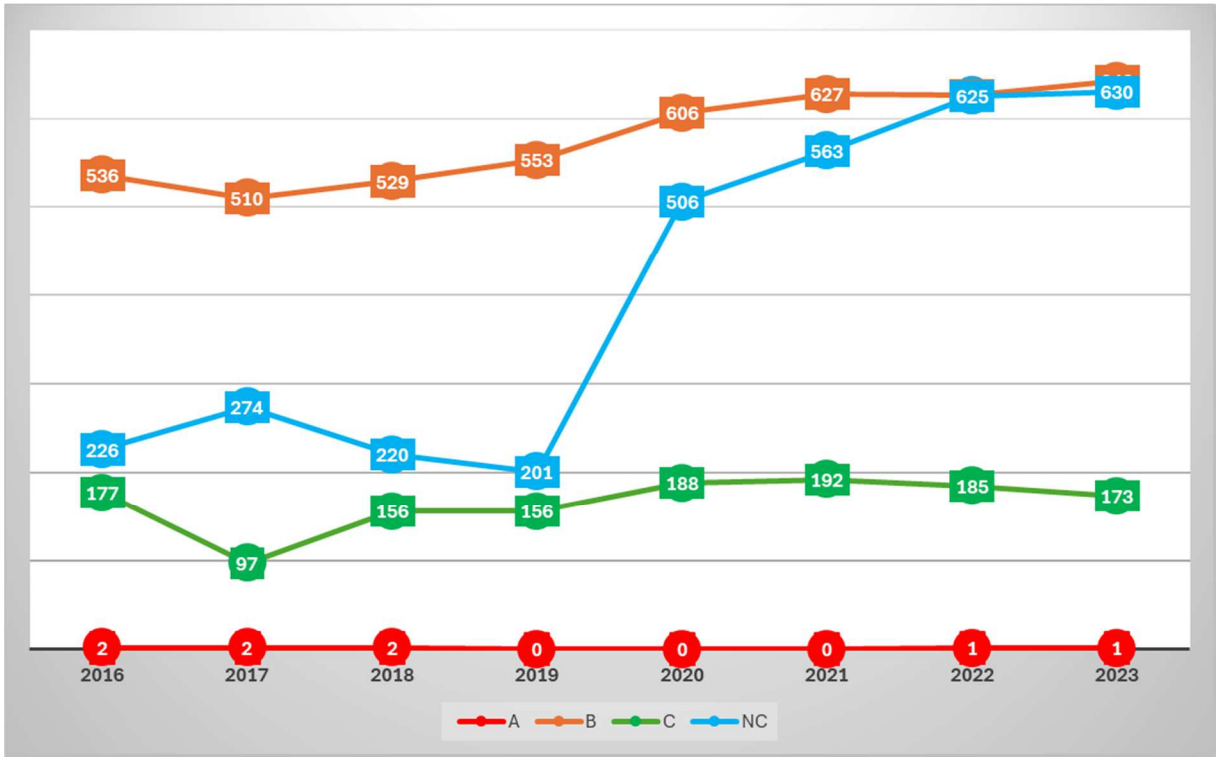


Fig. 6 - Risk Classification for Brazilian Hydroelectric Dams. ANEEL, 2023 [33]

It can be seen that most of the dams that are part of the Brazilian energy matrix and monitored by ANEEL have a Type B Risk Classification, i.e. they are characterized as low risk and medium potential for damage. In addition, based on the results of the Regular Safety Inspections, collected via a form, dams have their safety level measured on a scale that takes into account the combined effect of the anomalies observed in their structure or operation (Fig. 7).

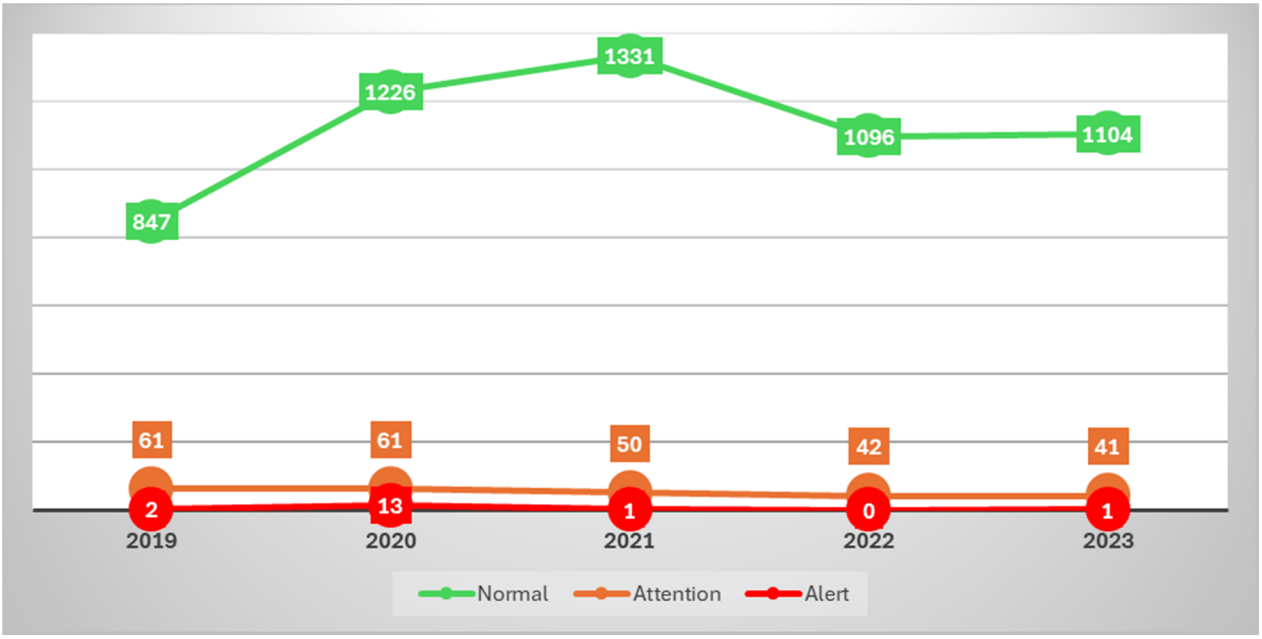


Fig. 7 - Safety Level for Brazilian Hydroelectric Dams. ANEEL, 2023 [33]

The classification of the structure's levels of damage, combined with the possibility of periodic prevention and maintenance, contributes significantly to reducing the occurrence of dam ruptures, promoting greater reliability and, consequently, greater safety for the populations distributed throughout its area of influence.

Safety levels identified as 'Normal' or 'Warning' indicate that the anomalies identified do not compromise the safety of the structure (they can be understood as neglect and poor maintenance), or compromise its integrity in the short term, and should be controlled and monitored over time. On the other hand, 'Alert' or 'Emergency' classifications indicate short-term compromise, or an imminent risk of rupture or collapse, and measures must be taken to eliminate the problem. In these cases, by virtue of regulation, the regulatory body must be notified immediately of the risk condition identified. In the case of Brazil, the indices have remained stable over the last few years.

In addition to the significant increase in the number of structures mapped over the last few years, it is important to highlight the evolution of the results obtained from the analysis of the risk factors that make up the dam classification matrix. Figs. 8 and 9 below show the annual global average of the risk classification scores attributed to the set of 560 dams common to all

the assessment cycles carried out. The risk scale is graded from 1 to 10, with grade 1 corresponding to situations of minimal risk.

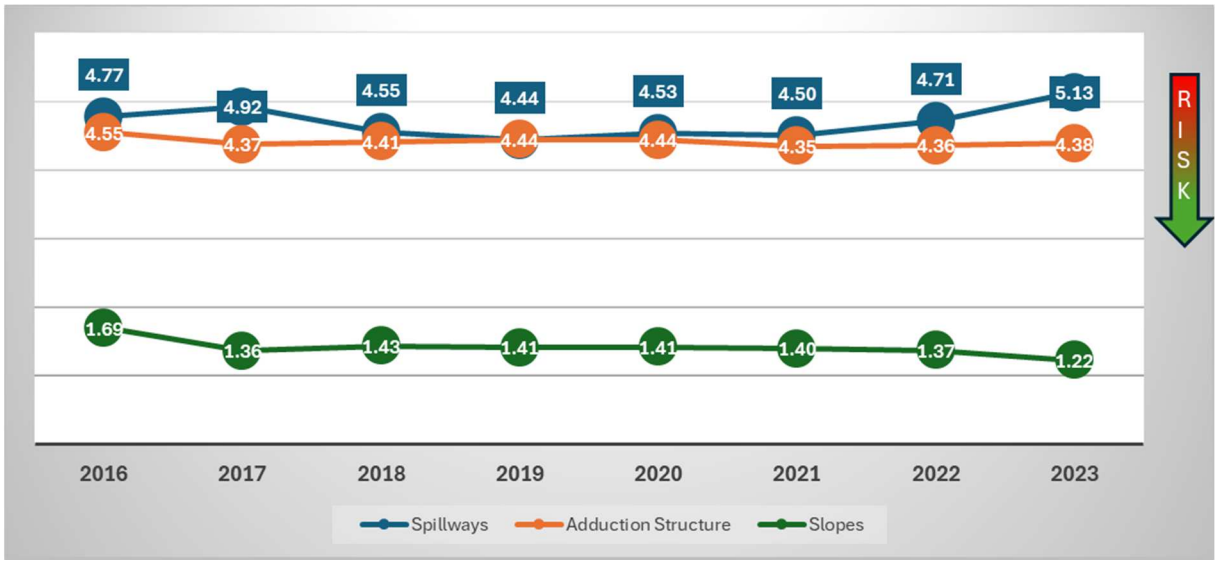


Fig. 8 - Evolution of Risk Factors Related to Conservation Status, ANEEL 2023 [33]

The risk factors associated with the classification matrix by Conservation Status (CS) in Fig. 8 cover activities that maintain the integrity of a dam and are the explicit responsibility of the owner, as established in Law No. 12.334/2010.

As highlighted by Melo and Fussaro [24] and ratified in section 5.4, historically dam accidents are more frequently caused by issues linked to: (1) hydraulic failure due to abnormal levels, including overtopping; (2) mass movement caused by extraordinary loads and (3) internal deterioration and erosion, identified by the occurrence of piping,

The descriptors listed in Fig. 8 aim to capture this context. Overtopping scenarios, for example, have a strong correlation with the integrity of overflow structures. For the indices calculated, in addition to the low gradation, there is stability or a downward trend in the medium term, indicating a good level of control.

In the years 2022 and 2023, there is an increase in overall risk levels for the overflow structures parameter after successive falls. This condition is attributed to the regulatory obligation to carry out Periodic Safety Reviews (PSR), the aim of which is to diagnose the general safety status of the dam in greater detail, taking into account, for example, design criteria and the updating of hydrological information in the respective river basin.

The increase is due to the application of strict controls to protect operational technologies, reflecting the improvement in the inspection process.

As discussed, the Dam Safety Plan is an instrument of the NDSP. Evaluated dams that fit the criteria for applying the policy are obliged to implement it. The existence and operation of a

Safety Plan is assessed in the Rating Matrix. Thus, insufficient or unsatisfactory compliance with its guidelines negatively affects the dam's risk classification. The indexes evolution that assesses these risks can be seen on Fig. 9.

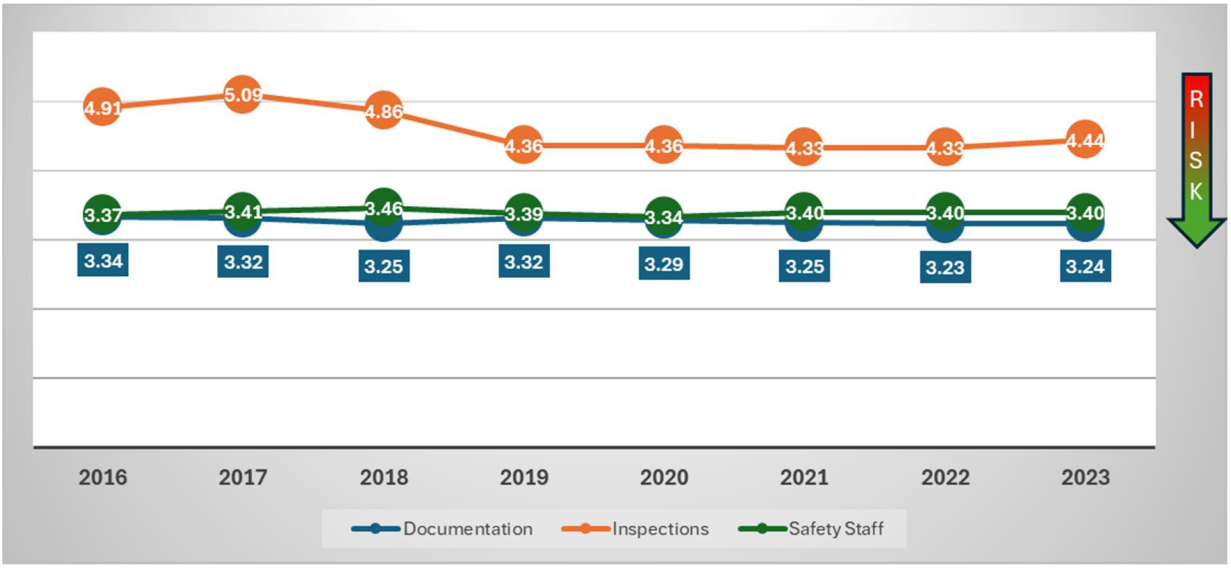


Fig. 9 - Evolution of Risk Factors Related to Security Plans, ANEEL 2023 [33]

The risk factors associated with the dam's Safety Plan (SP) classification matrix identify and qualify the tools available in the developer's organizational structure, both from the point of view of the qualifications of the team in charge and the routine and characteristics of inspection operations and analysis of their results.

As explained in section 5.3, inconsistent documentation is seen as a relevant risk factor because it adds uncertainties that could have been avoided. The EAP, for example, is a document that must be adapted to the life stage of the work, the operating circumstances and the safety conditions.

The indices calculated show a low degree of risk, suggesting an increase in the level of maturity of the entities inspected in terms of risk perception and the search for compliance.

7. RECOMMENDATIONS FOR THE FUTURE

The risk imposed by a dam is rarely caused by a single factor. It arises from a combination of adverse conditions, including contributions from the site, the design, the construction and maintenance methods, rainfall conditions and seismic activity in the area [4].

Dam safety is therefore a multidisciplinary process, made up of structural and non-structural activities. This condition has the potential to stress the model practiced if it is not properly exercised and updated.

In order to harmonize these aspects and guarantee the safety of dams in Brazil, some recommendations are essential:

7.1. Strengthening Regulation

Strengthening legislation and creating new regulatory instruments are important steps to ensure that dams operate within the expected and required standards. It is therefore essential to constantly review and update safety regulations, ensuring that they are effective and reflect the best technical and regulatory practices. Regulation must be dynamic, adapting to changes in the sector and new technologies. As an opportunity, lessons from international experiences, including disasters, can be adapted to the Brazilian context.

7.2. Investment in technology

The geotechnical, environmental and hydraulic risks to which a dam is exposed mean that the storage structures, reservoirs, foundations and dam shoulders must be monitored and controlled, just like the water stored or flowing. In this reality, the use and development of intelligent monitoring tools such as sensors, embedded systems and drones has proved to be an effective strategy. These resources enable real-time analysis of structural conditions, facilitating early identification of problems. Applications can go beyond monitoring. Artificial Intelligence resources can be incorporated into methods for assessing and validating the Potential Damage of projects. The regulator should urge dam operators to adopt these technologies in order to ensure more effective supervision. Partnerships with technology companies and research institutions should be encouraged.

7.3. Continuous Training and Interinstitutional Cooperation

Inter-institutional cooperation between ANEEL and other regulatory bodies can result in a more integrated and effective approach to inspection and regulation. The same can be said for the need to integrate databases from federal and state institutions. The exchange of experiences and resources through regular training and workshops improves actions to prevent, repair and respond to security events.

7.4. Community participation

The communities affected by emergency plans and risk management must be involved in order to ensure that they are aware of the procedures and prepared for possible emergency situations. According to Villagrán de León [34], warning systems must be planned, implemented and operated with the aim of empowering the people who need them most. Empowerment refers to adequate information about the risks they face and ways to minimize losses in the event of a catastrophic event. People-centered (bottom-to-top) warning systems,

in contrast to technically oriented (top-to-bottom) systems, take as their starting point the communities that could be affected and plan the response system according to the characteristics of those communities. In technically-oriented systems, the people at risk are incorporated at the end. For the author, the failures of communication in disasters are in contrast to the current trend of risk warning and communication systems.

7.5. Risk communication to society (Fake News)

In the last decade, major dam accidents in Brazil due to safety failures have projected a negative image of these structures, contributing to them not being perceived by the majority of public opinion, as a rule, as controlled and safe assets. These effects have been amplified by the massive dissemination of information. When society is unable to distinguish between true and false information, its trust in institutions and their ability to provide accurate and reliable information is shaken. The risks in a tragedy are great and disinformation amplifies them. Fake news diverts attention to non-existent problems. As one of the regulators of the dam safety process, it is up to ANEEL to expand its activities in promoting practices to combat erroneous assumptions by making data and reports on dam safety available to the public in an objective and accessible way.

7.6. Attention to Small Dams and their Dissemination

Small dams on watercourses, although they have important and diverse purposes, are not taken into account by the regulatory guidelines established in Brazil, where the regulatory system has been stimulated by discussions analyzing the advantages and disadvantages of large dams only for large enterprises. It must be acknowledged that small dam failures also have disastrous consequences. In China in 1975, the collapse of the Shimantan and Banquia dams resulted in the cumulative collapse of more than 60 small dams downstream, resulting in the deaths of 230,000 people. According to Viseu [35], the ruptures of large dams receive more attention than those of smaller dams. Ruptures of small dams, built on private property, occur more frequently. This is because, as dams are relatively expensive to build, their owners tend to make them less expensive, generally sacrificing their safety. In terms of historical records, the probability of a small dam breaking is estimated at 2% [36], a figure considerably higher than that associated with large dams, estimated at 0.5% [16]. Weaknesses encountered, whether in licensing or safety aspects, are the main factors driving the practice of establishing small irregular dams on watercourses, disregarding technical and environmental criteria.

8. CONCLUSIONS

Based on the knowledge and practice acquired, ANEEL has gathered the necessary skills to systematically measure and evaluate its compliance results and regulatory performance. This ability has been progressively institutionalized in its internal procedures aimed at improving governance and regulatory results in the dam safety process, as well as integrated into other derived initiatives.

A subsidiary challenge to be met in order to continue this reform includes information management and the training of civil servants. The new enforcement model is data-intensive and the agency's ability to acquire, store, process and analyze it properly will greatly influence the efficiency and effectiveness of its processes.

The figures presented were achieved after successive annual inspection cycles based on the concepts of promoting regulatory compliance, focusing on risk and selectivity. The improvement seen contributes to defining strategies and carrying out assertive planning, operation and maintenance activities for the dam safety system with a view to reducing the number of failures and, consequently, increasing the level of reliability of these structures.

This circumstance is important for better management of the operation and maintenance of equipment by its owners, for increasing the ability to anticipate safety risks and for qualifying information for enforcement action. This dynamic can have its impact and visibility increased through the formation of collaborative inter-institutional networks.

However, any dynamic has its resilience limits. Continuous and strategic investment, especially in emerging technologies, is needed to ensure sustainable long-term growth.

In-depth exploration with the identification of new criteria and risk factors to improve the Dam Safety Policy is left as a proposal, in order to guarantee managers a wider range of options. The application of a structured risk analysis process, even at the initial maturity level, promotes efficiency gains to the process while reducing information asymmetry levels.

Finally, it should be emphasized that the reflections contained in this article are part of a subject of significant importance, not only because of the need to understand regulatory guidelines for the treatment of dam safety by the constituted authorities, but also as proposals to support decisions in the technical field by all the actors involved.

REFERENCES

[1] PERINI, D. S (2009). Estudo dos processos envolvidos na análise de riscos de barragens de terra. Master Thesis. Faculdade de Tecnologia, Universidade de Brasília – UnB (2009).

[2] JANSEN, R. B. et. al. (1983). Water resources technical publication. In Dams and Public Safety. Saffron Walden: Books Express Publishing (1983).

[3] BRASIL, Agência Nacional de Águas (2023). Relatório de Segurança de Barragem. Relatório Técnico, Agência Nacional de Águas – ANA (2023). Available at: <https://www.snisb.gov.br/portal-snisb/documentos-e-capacitacoes/rsb/>

[4] LEITE, S. R. (2019) Modelo para Avaliação de Riscos em Segurança de Barragens com Associação de Métodos de Análise de Decisão Multicritério e Conjuntos Fuzzy. Master Thesis. Departamento de Ciência da Computação, Universidade de Brasília – UnB (2019).

[5] FERNANDES, C. A. G (2018). A Reforma do Modelo de Fiscalização do Setor Elétrico Brasileiro. Escola Nacional de Administração Pública (ENAP), Brasília (2018).

[6] OECD (2014). Regulatory Enforcement and Inspections. OECD Publishing, Paris (2014).

[7] AYRES, I. and BRAITHWAITE J. (1995). Responsive Regulation: Transcending the Deregulation Debate. New York: Oxford University Press (1995).

[8] HIRATA, I.; LEITE S. R. et. Al (2015). Fiscalização em 3 Níveis – Aplicando o Conceito de Diferenciação de Risco Regulatório na Fiscalização de Empreendimentos de Geração de Energia. v. 20. In CONGRESSO BRASILEIRO DE REGULAÇÃO, 9, EXPOABAR, Brasília (2015).

[9] da COSTA, D. C., FAIS, L. M. C. F. and FREIRIA R. C. (2020). Segurança de Barragens: Panorama Histórico da Legislação Brasileira. Athenas: Revista de Direito, Política e Filosofia (2020).

[10] PORTUGAL (1804). Alvará de 27 de novembro de 1804. Dá providências a bem da Agricultura e das herdades da Província do Alemtejo, em Portugal. Available at: <http://www1.ci.uc.pt/ihti/proj/filipinas/l4pa1020.htm>.

[11] POMPEU, C. T. (1972) Regime Jurídico da Concessão de Uso das Águas Públicas. In: Revista de Direito Público, n.º 21, p. 160-173 (1972).

[12] BRASIL, Lei nº 12.334, de 20 de setembro de 2010. Estabelece a Política Nacional de Segurança de Barragens. Diário Oficial da República Federativa do Brasil, Seção 1, 21 de setembro de 2010, p. 1 (2010). Available at: <http://www.planalto.gov.br/ccivil/Leis/L9427cons. htm>.

[13] MEDEIROS, C. H. (2017): Curso Guia de Diretrizes para Elaboração de Projeto de Barragens e Revisão Periódica - Módulo 1: Etapas dos Estudos e Projetos, Universiade Federal do Ceará, UFC. Rio de Janeiro (2017). Available at:: <http: //www.snisb.gov.br/portal/snisb/downloads/>.

[14] McGRATH, S. (2000). To Study International Practice and Use of Risk Assessment in Dam Management. The Winston Churchill Memorial Trust of Australia (2000).

[15] GOMES, A.S, ALMEIDA, A.B., CASTREO, A. T., PAIXÃO, J., CALDEIRA, L, PIMENTA, L and T VISEU. 1º Relatório de Progresso – Grupo de Trabalho de Análise de Riscos em Barragens. Comissão Nacional Portuguesa das Grandes Barragens – CNPGB, Lisboa, Portugal (2005).

[16] ICOLD (1995): Dam Failures: A Statistical Analysis. ICOLD Bulletin, n. 99. International Commission on Large Dams ICOLD, Paris (1995).

[17] BRASIL (2023). Agência Nacional de Energia Elétrica (ANEEL). Resolução Normativa nº 1.064, de 2 de maio de 2023. Available at: < https://www2.aneel.gov.br/cedoc/ren20231064.pdf>

[18] BRASIL, Conselho Nacional de Recursos Hídricos (2012). Resolução Nº 143/2012. Diário Oficial da República Federativa do Brasil, Seção 1, 10 de julho de 2012 (2012). Available at: <https://www.snisb.gov.br/Entenda_Mais/legislacao-aplicada/resolucao-cnrh-143-2012.pdf>:

[19] CAPELLA, A.C.N. Perspectivas Teóricas sobre o Processo de Formulação de Políticas Públicas. In: Hochman,G., Arretche, M., Marques, E. (Orgs.) Políticas Públicas no Brasil. Rio de Janeiro: Editora Fiocruz (2007).

[20] BRASIL (2021). Ministério da Integração e do Desenvolvimento Regional. Avaliação Ex-post da Política Nacional de Segurança de Barragens. Available at: <https://www.gov.br/mdr/pt-br/assuntos/seguranca-hidrica/cnrh/camaras-tecnicas/ctsb/snisb/3-relatorio-final.pdf>

[21] BRASIL (2018). Presidência da República. Casa Civil. Avaliação de Políticas Públicas: Guia Prático de Análise Ex post. Instituto de Pesquisa Econômica Aplicada (IPEA). – Ipea (2018). Vol. 2. Available at: <http://www.ipea.gov.br/portal/index.php?option=com_content&view=article&id=34504>.

[22] BRASIL (2024). Agência Nacional de Energia Elétrica (ANEEL). Sistema de Informações de Geração da ANEEL. Available at: <https://dadosabertos.aneel.gov.br/dataset/siga-sistema-de-informacoes-de-geracao-da-aneel>.

[23] LEITE, S. R., GOMES, L. R. CAETANO, R. E., CHAIM, R. M., NERY, M. M. (2023). Validação dos Fatores de Risco Associados à Segurança de Barragens sob a Ótica dos

Profissionais Envolvidos –XXXIV Seminário Nacional de Grandes Barragens Tema 127 Foz do Iguaçu, Paraná (2023).

[24] MELO, A. V.; FUSARO T. C. (2015). Avaliação de Métodos de Análise de Risco Aplicados a Barragens. In Seminário Nacional de Grandes Barragens, Foz do Iguaçu, Paraná (2015).

[25] BRASIL (2022). Ministério da Integração e do Desenvolvimento Regional (MIDR). Available at: <https://www.gov.br/mdr/pt-br/assuntos/seguranca-hidrica/cnrh/deliberacoes-cnrh-1/mocoes?b_start:int=0>.

[26] BRASIL (2024). Empresa de Pesquisa Energética (EPE). Available at: <https://www.epe.gov.br/pt/areas-de-atuacao/energia-eletrica/expansao-da-geracao/fontes>

[27] BRASIL (2011). Agência Nacional de Águas e Saneamento Básico (ANA). Relatório de Segurança de Barragens 2011 (2011).

[28] MORRIS, M.W., GALLAND, J.C. (2000). Dambreak Modelling – Guidelines and Best Practice,Conclusions from the CADAM Concerted Action Project, Available:at: <http://zip.net/bts79Y>

[29] SAMPAIO, J. A. L. (2016). As Deficiências do Plano de Ação Emergencial das Barragens no Brasil. Revista Brasileira de Direito, v. 12, n. 2, p. 7-17. (2016).

[30] UFSC. Universidade Federal de Santa Catarina – CEPED. Atlas Brasileiro de Desastres Naturais:- 1991 a 2010. Florianópolis (2012).

[31] GANEM, R. S. (2012) Gestão de Desastres no Brasil, Biblioteca Digital da Câmara dos Deputados, Brasília (2012).

[32] MEDEIROS, C. H. (2020). Segurança de Barragens: O que fazer para Convergir Teoria e Prática. In: XXVI Seminário Nacional de Grandes Barragens, 2005 CBDB (2005).

[33] BRASIL (2024). Agência Nacional de Energia Elétrica (ANEEL). Fiscalização de Segurança de Barragens. (2024). Available:at: <https://dadosabertos.aneel.gov.br/dataset/fsb-fiscalizacao-de-seguranca-de-barragens>

[34] VILLAGRÁN DE LEÓN, J.C (2012). Early Warning Principles and Systems. In: WISNER, B; GAILLARD, J.C; KELMAN, T. Handbook of Hazards and Disaster Risk Reduction, (2012).

[35] VISEU, T. (2006). Segurança dos Vales a Jusante de Barragens: Metodologia de Apoio à Gestão do Risco. Doctoral Thesis, Instituto Superior Técnico, Lisboa (2006).

[36] LEMPÉRIÈRE, F. (2017). Dams and Floods. Engineering, v. 3, n. 1, p. 144–149 (2017).

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DEVELOPMENTS OF INTEREST FOR DAM INDUSTRY REGARDING SEISMIC ACTION IN THE EUROCODE 8 OF NEXT GENERATION

Pierre B. Labbé[1]
Ecole Spéciale des Travaux Publics

ABSTRACT

Although dams are not covered by the Eurocode 8, the purpose of this paper is to focus on those changes that might have an impact on earthquake engineering practices of dam industry, namely:

- In place of Peak Ground Acceleration (PGA), the standard response spectrum is anchored by its plateau value, S_α, and its value at 1 s spectral period, S_β.
- An informative annex is introduced, with two European hazard maps, one for S_α, and one for S_β, both defined for 475 years return period. Performance factors are introduced for extrapolation to other return periods.
- The site categorization is revised by introducing two new screening parameters in addition to the shear wave velocity: the bed rock depth and/or the observed site natural frequency.
- New site amplification factors are introduced, which account for the non-linear response of sediments under strong input motions.
- A model of spatial variability of ground motion is introduced, which can used for description of input motion variability along dam foundations.
- In view of time-history analyses, and for the selection of seismic input motions (accelerograms), guidance is provided in the form of a table of conventional magnitudes and strong motion duration.
- Criteria for the acceptance of a set of accelerograms are significantly relaxed, compared to the current version. The philosophy behind is that a series of actual seismic records is more legitimate than a non-realistic standard spectrum to represent the seismic input motion.

Keywords: Eurocode 8, Response spectrum, Site categorisation, Time response analysis.

.

[1]pierre.labbe@estp.fr, France

1. INTRODUCTION

1.1. Outlines of Eurocodes development process

Eurocodes are developed under the guidance and co-ordination of Centre Européen de Normalisation, Technical Committee 250 "Structural Eurocodes". The Eurocode 8 “Design of structures for earthquake resistance”, is under the responsibility of Subcommittee 8 (SC8). Part 1-1 “General rules and seismic action” [1], designated here as Eurocode 8-1-1, was formally approved in June 2024, concurrently with Part 5 “Geotechnical aspects, foundations, retaining and underground structures”. The next generation of Eurocodes should be enforced in 2027.

1.2. Eurocode text *versus* national choices

In the general Eurocode development framework, questions that fall under physics laws should be introduced in the text without the possibility for CEN National Members to modify them, while, on the opposite, questions that fall under safety considerations are under national responsibilities. Consequently, parameters that concern safety are National Determined Parameters (NDPs). The values of them applicable in a country are national choices. However, for each of them, a recommended value is proposed by the Eurocode, decided by consensus among the Subcommittee delegates.

1.3. Generic Eurocode 8 concepts that interact with seismic action

For the purpose of safety, Eurocodes classify structures into Consequence Classes (CC1, CC2, CC3-a, CC3-b)), depending on the consequences of failure or malfunction of the structure in terms of loss of human life or personal injury, as well as in terms of economic, social or environmental consequences. Structures of the highest consequence class, CC4 (e.g. nuclear facilities, large dams), are covered by dedicated rules and regulations, not by the Eurocodes. However, it is recognized that Eurocodes are reference documents for CC4 structures, principally because CC4 provisions are basically more severe than CC3 ones.

The Eurocodes approach for the design of structures is based on the concept of Limit States (LS). We distinguish between ultimate limit states, the non-exceedance of which controls safety of structures, and serviceability limit states. Regarding seismic protection, four limit states are considered. Two of them are ultimate limit states, namely Near Collapse (NC) and Significant Damage (SD), the two other ones are serviceability limit states, namely Damage Limitation (DL) and Fully Operational (OP).

1.4. Eurocode philosophy of the seismic action

As stated in Eurocode 8-1-1, and only measurable in probabilistic terms, the fundamental performance requirement is that, in the event of an earthquake, human lives are protected, damages to structures are limited, and facilities important for civil protection remain operational. To meet these performance objectives, Eurocode 8-1-1 requires that structures are designed in such a way that specified limit states are not exceeded under prescribed seismic actions. Such actions are to be regarded as conventional and cannot be related to any specific earthquake event.

For the design of new structures, it is required that, at least, the non-exceedance of the SD limit state is verified. The Eurocode 8 is conceived in such a way that, for a large majority of new structures, the SD non-exceedance requirement implies avoiding NC exceedance under a seismic action meaningfully more severe than that of design, as well as avoiding DL exceedance under a seismic action less severe than that of design.

As compared to the current Eurocode 8, the next generation anticipates the expected development of time-response analyses and, concurrently with the development of seismic strong motion database, pushes in favour of using natural input motion. Such an evolution is of interest for the dam industry, which is familiar with time response analyses, and is commented further.

1.5. A major evolution in standard response spectra anchoring system

A major evolution, which plays a role all along the document, is the new standard response spectra anchoring system. New spectra are still presented in the form of pseudo-absolute acceleration response spectra. However, instead of being anchored with the Peak Ground Acceleration (PGA), the next generation standard response spectra are anchored with two parameters (Fig. 1): S_α, which is the spectrum value on a plateau that covers a certain spectral period range, and S_β, which is the spectrum value at 1 s spectral period. In Eurocode 8-1-1,

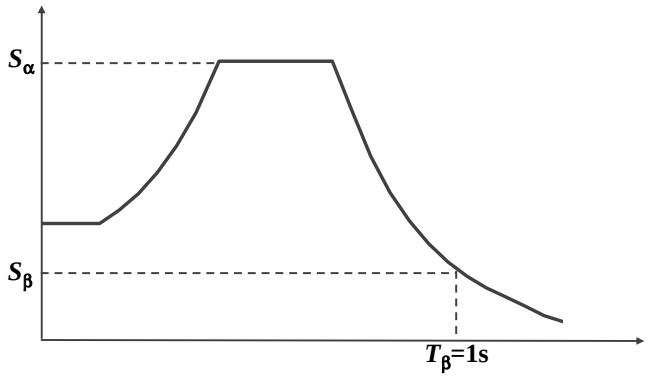


Fig. 1: Anchoring system of standard spectra in next Eurocode 8

subscripts α and β may be supplemented by additional subscripts for specification of S_α and S_β values under consideration. A more detailed presentation of the development of seismic action in Eurocode 8-1-1 is given by Labbé & Paolucci [2].

2. SAFETY-RELATED FACETS OF THE SEISMIC ACTION

2.1. Seismic action classes

A seismic action index is introduced at the very beginning of Eurocode 8-1-1, as given by Equation 1. In this formula, the last three terms, introduced further in this paper, are constitutive of the seismic action, while the first one, δ, is a coefficient that depends on the consequence class of the considered structure. Recommended values of this new NDP are reported in Table 1. However, countries may apply their respective safety policies by selecting their own δ values. According to the seismic action index value, the situation under consideration is qualified by its seismic action class, as presented in Table 2.

$$S_\delta = \delta F_\alpha F_T S_{\alpha,475} \quad (1)$$

Table 1. recommended δ values in different parts of Eurocode 8

Consequence class	CC1	CC2	CC3-a	CC3-b
Buildings and Bridges	0,6	1	1,25	1,6
Silos, tanks, pipelines, towers, masts, chimneys	0,6	1	1,4	1,8
Geotechnics and foundations	0,6	1	1,5	

Table 2. Seismic action index values and associated seismic action classes

Range of seismic action index	Seismic action class
$S_\delta < 1{,}30$ m/s²	Very low
1,30 m/s² $\leq S_\delta <$ 3,25 m/s²	Low
3,25 m/s² $\leq S_\delta <$ 6,50 m/s²	Moderate
$S_\delta \geq 6{,}50$ m/s²	High

2.2. Return periods and performance factors

It is requested that seismic actions be specified by selecting appropriate return periods, $T_{LS,CC}$, depending on the specified limit state, LS, that should not be exceeded under the seismic action for a structure of consequence class CC. By definition, the reference return period, T_{ref}, is equal to $T_{SD,2}$, the return period associated to SD limit state for CC2 structures. In most countries, $T_{SD,2}$=475 years. $T_{LS,CC}$ are NDPs, the recommended values of which are specified in the relevant parts of Eurocode 8. As a matter of example, values given in Part 2 (Bridges)

are reported in Table 3. For instance, a 2500-year return period is recommended for verification of a CC3-a bridge at the near collapse limit state.

In this context, $S_{\alpha,RP}$ and $S_{\beta,RP}$, in particular $S_{\alpha,ref}$ and $S_{\beta,ref}$, are values of S_α and S_β, (on category A site) for the RP return period. $S_{\alpha,RP}$ and $S_{\beta,RP}$ values may be obtained from spectral acceleration maps for a set of selected return periods. Alternatively, they may also be derived from performance factors, $\gamma_{LS,RP}$, by applying formulas given in Equation 2. Recommended values of $\gamma_{LS,RP}$ for bridges are presented in Table 3. For the above-mentioned example, a 1.8 performance factor is recommended.

$$S_{\alpha,RP} = \gamma_{LS,CC}\, S_{\alpha,ref} \;\; ; \;\; S_{\beta,RP} = \gamma_{LS,CC}\, S_{\beta,ref} \tag{2}$$

Table 3. Recommended return period (years) and performance factors for bridges

Limit State	CC1	CC2	CC3-a	CC3-b	CC1	CC2	CC3-a	CC3-b
NC	800	1600	2500	5000	1,2	1,5	1,8	2,2
SD	250	475	800	1600	0,8	1,0	1,25	1,5
DL	50	60	100	200	0,5	0,5	0,6	0,7

Acronyms: NC: Near Collapse, SD: Significant Damage, DL: Damage Limited, CC: Consequence Class

3. SITE CATEGORISATION

3.1. Site stability and site-specific hazard analyses

Provisions relating to site conditions introduce general considerations on stable sites and refer to Part 5 (Geotechnics) for assessment of unstable sites. Criteria controlling the necessity of site-specific studies are provided, considering the consequence class of the structure, the maximum earthquake magnitude possibly generated by the fault, and the distance to the fault. As opposed to the current Eurocode 8, site-specific hazard studies are permitted for every site, and guidance given in the Annex C.

3.2. Definition of site categories

Eliminating current ambiguity in site categorisation was an objective of the Eurocode 8 revision. Additionally, although the shear wave velocity, v_s, is recognised as a very good predictor for site categorisation, it should be also recognised that at least one additional parameter is necessary to better cover the variety of site profiles. Two proxies were selected, namely H_{800} and $v_{s,H}$. H_{800} is the depth of the conventional bedrock formation, identified by a shear wave velocity of at least 800 m/s. In case H_{800} is larger than 30 m, $v_{s,H}$ is taken as equal to $v_{s,30}$, as defined in the current Eurocode 8. Otherwise, it is the propagation time-averaged shear wave velocity down to H_{800}.

On these bases, site profiles are categorised as presented in Table 4. Categories A to F are used further to control site amplification factors. Additional site categories may be introduced in case they are deemed necessary to better distinguish the seismic response features (e.g. hard rocks, very deep basins, very shallow and soft sites, irregularly layered soil profiles) at a regional or urban scale.

Table 4. Standard site categorisation

	Ground class	Stiff	Medium	Soft
Depth class	v_{sH} in m/s	800-400	400-250	250-150
very shallow	$H_{800} \leq 5$ m	A	A	E
Shallow	5 m < $H_{800} \leq 30$ m	B	E	E
Intermediate	30 < $H_{800} \leq 100$ m	B	C	D
Deep	H800 > 100 m	B	F	F

3.3. Alternative identification of site category

In case $v_{s,H}$ and/or H_{800} are not documented, alternative ways of site categorisation are provided in Annex B. They can be separated into quantitative and qualitative ways. In the quantitative way, in case H_{800} is not documented, a major development is that the site category can alternatively be derived from f_0, the natural frequency of the soil profile, as reported in Table 5. In addition, H_{800} can also be derived from f_0 if necessary. Categorisation of sites based on f_0 is based on researches by Pitilakis et al. [3]. Consistency of categorisation based on H_{800} *versus* f_0 is discussed by Paolucci et al. [4].

Table 5. Site categorisation based on $v_{s,H}$ and f_0

Combination of f_0 (Hz) and $v_{s,H}$ (m/s)	Site category
$f_0 \geq 10$ and $v_{s,H} \geq 250$	A
$f_0 < 10$ and $400 \leq v_{s,H} < 800$	B
$v_{s,H}/250 \leq f_0 < v_{s,H}/120$ and $250 \leq v_{s,H} < 400$	C
$v_{s,H}/250 \leq f_0 < v_{s,H}/120$ and $150 \leq v_{s,H} < 250$	D
$v_{s,H}/120 \leq f_0 < 10$ and $150 \leq v_{s,H} < 400$, or $f_0 \geq 10$ and $150 \leq v_{s,H} < 250$	E
$f_0 < v_{s,H}/250$ and $150 \leq v_{s,H} < 400$	F

In the qualitative way, a simplified identification of site category can be obtained by the combined use of two tables. For instance, site category E is described as follows in Table B.1: “Shallow deposits consisting of prevailing alluvium layers, underlain by rock-like materials as described for site category A. Representative values of the geotechnical parameters are in the range defined in Table B.2 for medium stiffness or soft ground.” And concurrently, the mentioned Table B.2 provides correspondence between ground classes (stiff, medium stiffness, soft) and geotechnical parameters resulting from classical tests: SPT, CPT, Field or laboratory vane test, Pressuremeter test.

4. SEISMIC HAZARD MAPS

4.1. Mapping rules and seismicity levels

Eurocode 8-1-1 requires that territories be mapped depending on the local seismic hazard. Concurrently, the mapped representation of seismic hazard values applicable to design remains a national responsibility. It is required that hazard maps provide the reference spectral acceleration values of $S_{\alpha,ref}$ and $S_{\beta,ref}$, on a sufficiently dense grid throughout the concerned territory. Four seismicity levels are introduced as presented in Table 6, based on $S_{\alpha,475}$.

Table 6. Seismicity levels and f_h values

$S_{a,475}$ (m/s^2)	Seismicity level	f_h
$S_{a,475} < 1,0$	Very low	0,2
$1,0 \leq S_{a,475} < 2,5$	Low	0,2
$2,5 \leq S_{a,475} < 5,0$	Moderate	0,3
$5,0 \leq S_{a,,475}$	High	0,4

Seismicity levels play two roles. First, possibility is given that seismic hazard maps take the form of seismic zones across which the same reference spectral accelerations apply. In such a case, these zones should be delimited, and their seismicity levels qualified, according to Table 6. The second role is relating to $S_{\beta,ref}$ values. Indeed, an open national choice is either to map $S_{\beta,ref}$ from seismic hazard studies, in parallel to $S_{\alpha,ref}$, or to derive it from $S_{\alpha,ref}$ according to Equation 3, where f_h takes values 0,2, 0,3 and 0,4 in areas of respectively very low and low, moderate and high seismicity.

$$S_{\beta,ref} = f_h \, S_{\alpha,ref} \tag{3}$$

4.2. Informative European hazard maps

Two small scale maps of $S_{\alpha,475}$ and $S_{\beta,475}$ are provided in the Eurocode 8-1-1 Annex A. These maps are derived from deliverable ESHM20 of the SERA research project (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe), which received funding from the EU Horizon 2020 research and innovation programme to provide updated information on the seismic hazard in Europe. Outputs of the ESHM20 model, published by Danciu et al. [5], are available on http://hazard.efehr.org. Those $S_{\alpha,475}$ and $S_{\beta,475}$ maps are reported in Fig. 2.

$S_{\alpha,475}$ and $S_{\beta,475}$ maps presented in the Eurocode 8 Annex A are derived from ESHM20 median uniform hazard spectra (UHS) obtained across Europe for the 475-year return period. $S_{\alpha,475}$ is calculated as the average spectral value on the range of periods [0,5 T_{peak} , 1,5 T_{peak}], where T_{peak} is the spectral period that corresponds to the UHS peak. $S_{\beta,475}$ is directly taken as the UHS value for a spectral period of 1 s.

In order to obtain a consensus on maps, it is indicated in a note of Annex A that “CEN does not pass judgment on other results” of ESHM20. This is in particular the case for hazard values at other return periods, and even at 475-year return period for parameters other than $S_{\alpha,475}$ and $S_{\beta,475}$, which are neither endorsed nor rejected by SC8.

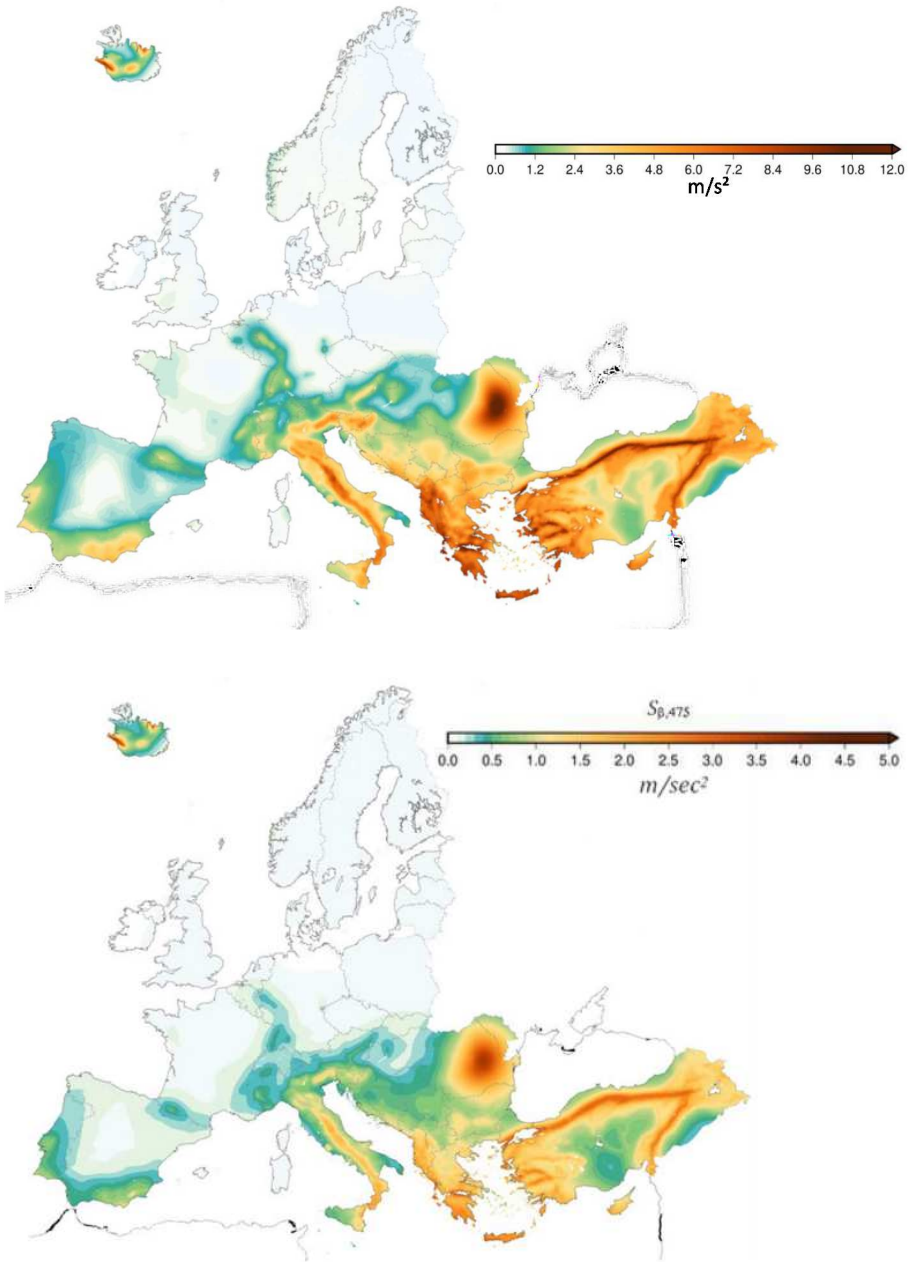


Fig. 2: Representation of $S_{\alpha,475}$ (top) and $S_{\beta,475}$ (bottom) European map for rock sites, based on ESHM20

5. SPECTRAL DESCRIPTION OF THE SEISMIC ACTION

5.1. Standardized spectral shapes

Standardized spectral shapes, designated as “elastic response spectra”, take the form presented in Fig. 4. First, S_α and S_β,are defined according to Equation 4, where $S_{\alpha,RP}$ and $S_{\beta,RP}$ are introduced in the above section 2.2, while F_a, F_b and F_T are presented below. In addition, four additional NDPs are introduced to finalize the spectral shape, default values of which are presented in Table 7.

$$S_\alpha = F_\alpha F_T S_{\alpha,RP}\,;\;\; S_\beta = F_\beta F_T S_{\beta,RP} \tag{4}$$

Table 7. Recommended values of T_A, F_A, χ, T_D

T_A (s)	F_A	χ	T_D (s)	
0,02	2,5	4	2	if $S_{\beta,RP}$≤1m/s²
			1+$S_{\beta,RP}$	if $S_{\beta,RP}$>1m/s²

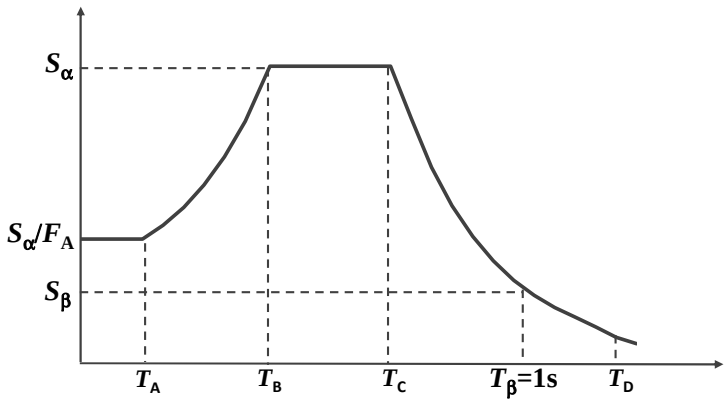


Fig. 4: Parametric description of standard spectra in next Eurocode 8

The spectral shape is then completed as follows: According to the postulated equation of the corresponding branch of the spectrum, the T_c corner frequency is automatically derived by $T_C = (S_\beta/S_\alpha)T_B$, with $T_B = 1s$. The T_B corner frequency is calculated as $T_B = T_C/\chi$, being however constrained to stay in the window [0,05 s – 0,1 s]. The spectrum is finalized, first by introducing the short cut-off period T_A associated with the PGA value defined by S_α/F_A and considering a linear variation between T_A and T_B, and, second by introducing the corner frequency T_D at the beginning of the first constant displacement branch of the spectrum.

5.2. Site amplification factors

F_α and F_β are the short period and intermediate period site amplification factors, values of which, depending on the site category, are presented in Tables 8. The column in the middle

provides continuous values of F_α or F_β, which may be used in case H_{800} and $v_{s,H}$ are documented, or in case the quantitative way of Annex B is adopted. The right column provides discrete default values, to be used if the qualitative way of Annex B is adopted. The role of r_α is to consider that, on a given site, the stronger the input motion, the smaller the site amplification factor. The scientific background for F_α and F_β is presented by Paolucci et al. [4]. and a contribution to the knowledge of input motion strength effect is given by Pitilakis et al. [6].

Table 8.
Site amplification factors F_α for the standard site categories

Site category	H_{800} and $v_{s,H}$ available	Default value
A	1,0	1,0
B	$\left(\frac{v_{s,H}}{800}\right)^{-0,4\, r_\alpha}$	$1{,}3\,(1-0{,}1\,S_{\alpha,RP}/g)$
C		$1{,}6\,(1-0{,}2\,S_{\alpha,RP}/g)$
D		$1{,}8\,(1-0{,}3\,S_{\alpha,RP}/g)$
E	$\left(\frac{v_{s,H}}{800}\right)^{-0,4\, r_\alpha \frac{H}{30}\left(4-\frac{H}{10}\right)}$	$2{,}2\,(1-0{,}5\,S_{\alpha,RP}/g)$
F	$0{,}9\,\left(\frac{v_{s,H}}{800}\right)^{-0,4\, r_\alpha}$	$1{,}7\,(1-0{,}3\,S_{\alpha,RP}/g)$
$r_\alpha = 1-\frac{S_{\alpha,RP}/g}{v_{s,H}/150}$; H in m, $v_{s,H}$ in m/s, $g = 9{,}81$ m/s^2		

Site amplification factor F_β for the standard site categories

Site category	H_{800} and $v_{s,H}$ available	Default value
A	1,0	1,0
B	$\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_\beta}$	$1{,}6\,(1-0{,}2\,S_{\beta,RP}/g)$
C		$2{,}3\,(1-0{,}3\,S_{\beta,RP}/g)$
D		$3{,}2\,(1-S_{\beta,RP}/g)$
E	$\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_\beta \frac{H}{30}}$	$3{,}2\,(1-S_{\beta,RP}/g)$
F	$1{,}25\,\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_\beta}$	$4{,}0\,(1-S_{\beta,RP}/g)$
$r_\beta = 1-\frac{S_{\beta,RP}/g}{v_{s,H}/150}$; H in m, $v_{s,H}$ in m/s, $g = 9{,}81$ m/s^2		

The effects of site amplification factors on standardized response spectra are illustrated in Fig. 5 with respect to situations of high and low seismicity areas. It is clear that the effect of site amplification factors is more significant in low seismicity areas than it is in high seismicity areas. In particular the elastic response spectrum plateau value for $v_{s,H}$ =150 m/s is lower than it is for $v_{s,H}$ =300 m/s in the representative situation of high seismicity, while it is not the case in low seismicity situation.

In alternative to using the site amplification factors, site-specific ground response studies may be carried out, for whatever ground type and consequence class of the structure, according to guidance given in Annex C, provided that the resulting site-specific elastic response spectrum is not lower than 75% of the standard spectrum. These site-specific ground response studies are mandatory: a) in case of high seismic action class, with $v_{s,H}$ < 200 m/s; and b) when site conditions cannot be associated with the standard site categories referred to in Table 5. In case they are mandatory, site-specific ground response studies should provide elastic response spectra not lower than the standard one for the same ground conditions.

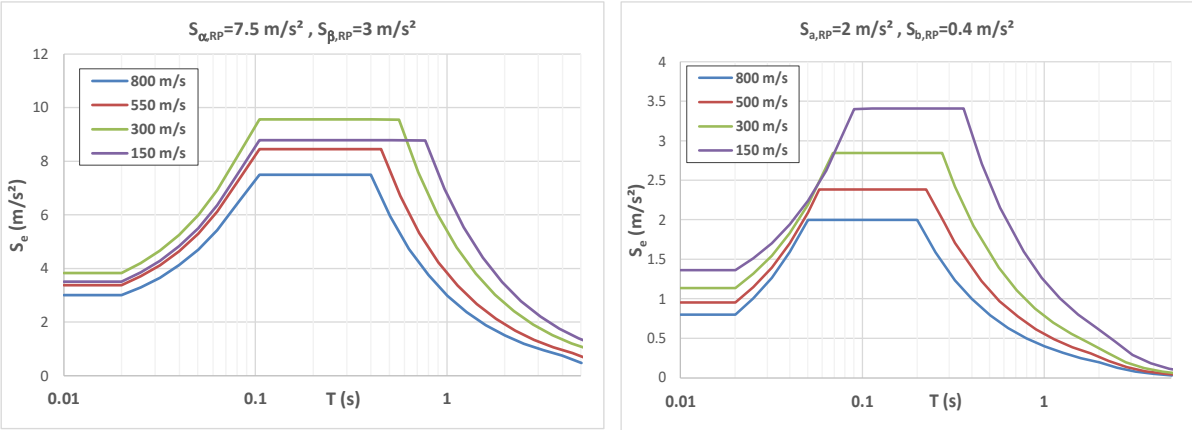


Fig. 5: Effects of site amplification factors on elastic response spectra in a high seismicity area (left) and low seismicity area (right) for different $v_{s,H}$ values

5.3. Additional considerations on standard response spectra

Topographic amplification factor. It is the F_T factor that appears in Equation (4). It takes values 1, 1,2 or 1,4 depending on the topographic configuration of the site. There is no change as compared to the similar factor given in Part 5 of the current Eurocode 8.

Damping correction factor. The above description of the standard spectrum is for a 5% damping spectrum. A formula is provided in Eurocode 8-1-1 for other damping values. It is the current formula for $T \geq T_B$, while it is calibrated to be equal to 1 for $T \leq T_A$, with linear transition.

Vertical elastic response spectrum. This spectrum is obtained through formulas like those of the horizontal spectrum, except that S_α and S_β are replaced by $S_{\alpha v}$ and $S_{\beta v}$, given in Equation 5, and the corner spectral period, T_{Bv}, is fixed to 0,05 s. A consequence of this fixed T_{Bv} value is that the vertical-to-horizontal spectral ratio may be larger than 1 in a range of short to intermediate spectral periods.

$$S_{\alpha v} = f_{v\alpha}\, S_\alpha \,;\; S_{\beta v} = f_{v\beta}\, S_\beta \,, \text{with}$$
$$f_{v\alpha} = \begin{cases} 0{,}6 & \text{if } S_\alpha < 2{,}5\ \mathrm{m/s^2} \\ 0{,}04\, S_\alpha + 0{,}5\ (\text{with } S_\alpha \text{ in m/s}^2) & \text{if } 2{,}5\ \mathrm{m/s^2} \le S_\alpha \le 7{,}5\ \mathrm{m/s^2} \\ 0{,}8 & \text{if } S_\alpha > 7{,}5\ \mathrm{m/s^2} \end{cases} \quad (5)$$
$$f_{v\beta} = 0{,}6$$

Elastic displacement response spectrum. Two additional corner periods are introduced, $T_E = \max[T_D, 6\ \mathrm{s}]$ and $T_F = 10\ \mathrm{s}$ to determine the elastic displacement response spectrum, the formula of which is not reported here. A long period site amplification factor, F_L, given by Equation 6 controls the constant displacement spectrum branch for $T > T_F$.

$$F_L = \left(\frac{v_{s,H}}{800}\right)^{-0{,}4} \quad (6)$$

Ground motion peak values. In addition to the PGA, formulas are provided for peak ground velocity (PGV) and peak ground displacement (PGD), based on analyses of seismic records carried out by Paolucci and Smerzini [7], as reported in Equations (7) and (8), where PGV is in m/s, PGD in m, S_α, S_β and $S_{\beta,RP}$ in m/s², T_B and T_D, in s. As compared to the current standard, and consistently with the absence of observed correlation between them, the PGD is not anymore calculated as proportional to the PGA.

$$PGV_e = 0{,}06\left(S_\alpha\, S_\beta\right)^{0{,}55} \quad (7)$$

$$PGD_e = S_{De}(T_F) = 0{,}025\, \eta T_\beta T_D F_L\, F_T\, S_{\beta,RP} \quad (8)$$

5.4. Site specific response spectra

In most countries, determining site-specific spectra is mandatory for CC4 structures such as large dams. According to the next Eurocode 8, determining such site-specific spectra is also mandatory for conventional structures under certain conditions. In addition, site-specific response spectra may be used for whatever the ground type and the consequence class of the considered structure. Accordingly, guidance is given in the Eurocode 8 (Annex C) on how to establish such spectra. The shape of the site-specific spectrum may differ from the standard shape; for instance, it can be a uniform hazard spectrum derived from a probabilistic hazard analysis. In most cases, the calculated site-specific horizontal response spectrum should not be lower than 75% of the standard horizontal elastic response spectrum for the same $v_{s,H}$ and site category and for the corresponding return period, at least in a range of spectral periods from 0,2 T_1 to 1,5 T_1, where T_1 is the fundamental period of the structure.

6. MAGNITUDE, TIME SERIES AND SPATIAL VARIABILITY

6.1. Conventional magnitude and strong motion duration

Although standard spectra are not representative of a specific event, conventional magnitude, M_w, and strong motion duration, D_R, are additional parameters that may be useful or even necessary, for instance in view of liquefaction risk assessment in Part 5 or for the selection of input motions in the form of time series. Values of those two parameters are provided as functions of $S_{\beta,RP}$, which is the reference spectral acceleration more closely related to Magnitude. Furthermore, duration is made dependent on the site category, as reported in Table 9. Magnitude values are valid for epicentral distance less than about 20 km, which is the most frequent situation inside Europe.

Guidance on magnitude and strong motion duration to be associated to a given response spectrum are very useful for engineers who intend to run time response analyses. Indeed, such guidance legitimates selection of strong motions out of databases.

6.2. Input motions in the form of time series

The seismic action may be represented by time series in terms of accelerogram and associated time dependant velocity and displacement. In such cases, recorded accelerograms, available from qualified strong motion databases, should be the preferred representation of the seismic action, before simulated accelerograms (obtained by physically constrained numerical simulations of the source to site wave propagation) or artificial accelerograms (obtained as samples of stochastic processes). A minimum of seven input motions is recommended and the average peak response should be considered for estimating the seismic action effects.

Table 9. Conventional magnitudes and strong motion durations

Range of $S_{\beta,RP}$ (m/s^2)	M_w	D_R (s)		
		Site cat. A	Site cat. B & C	Other
$S_{\beta,RP} \leq 0{,}08$	4,5	0,5	0,6	0,75
$0{,}08 < S_{\beta,RP} \leq 0{,}2$	5,0	1,0	1,2	1,5
$0{,}2 < S_{\beta,RP} \leq 0{,}5$	5,5	2,0	2,4	3,0
$0{,}5 < S_{\beta,RP} \leq 1{,}2$	6,0	4,0	4,8	6,0
$1{,}2 < S_{\beta,RP} \leq 2{,}5$	6,5	8,0	9,6	12
$2{,}5 < S_{\beta,RP} \leq 4{,}0$	7,0	16	19	24
$S_{\beta,RP} > 4{,}0$	7,5	32	38	48

Criteria are given in Eurocode 8-1-1 Annex D for the selection of accelerograms in databases. As far as possible, it should consider the regional tectonic environment, magnitude, source-to-site distance, and local conditions.

Selected accelerograms should approach the target spectrum, which is the standard elastic spectrum presented above or the local UHS if available, in a range of spectral periods from 0,2 T_1 to 1,5 T_1, where T_1 is the fundamental period of the structure in the direction in which the accelerogram applies. To improve compatibility with the target spectrum, recorded accelerograms may be scaled by a constant factor, limited in the 0,5-2 range either for geotechnical applications, or in case of highly non-linear response. In alternative, spectral scaling is also allowed, in which scaling is selectively applied in the frequency domain, until the response spectrum of the modified record approaches the target.

A set of selected (and scaled) accelerograms may be regarded as satisfactory if, in the above specified range, the ratio of the 5%-damped average response spectrum of the set to the target spectrum falls within the band between 0,75 and 1,3 and has an average value larger than 0,95. In addition, the 5%-damped response spectrum of each accelerogram should not fall below 50% of the target spectrum in the same range.

These criteria are significantly laxer than the current ones. This is a question of principle that artificial standard response spectra do not have a higher legitimacy than actual records have for representing the seismic action. In particular, although it is convenient for spectral analyses, the piecewise shape of standard spectra exhibits unrealistic slope changes that should not be taken as constraints in the selection or calibration of accelerograms.

Such laxer criteria are intended, in case time response analyses are necessary, to enhance using actual strong motion instead of artificial ones. This is of major importance in situations when non-linear analyses need to be carried out, because artificial strong motions trend to considerably increase the amount of energy in input signals.

6.3. Spatial variability of the seismic input motion

Regarding structures for which the assumption of the same excitation at all supports cannot reasonably be made, it is recommended that spatial models of the seismic action be used. If the seismic action is represented by a set of response spectra, for every couple of supports with indices k and l, a correlation coefficient, ρ_{kl}, between action effects is introduced given by Equation (9), where L_{kl} is the distance between supports k and l and $L_{g,k}$ and $L_{g,l}$ are characteristic lengths given in Table 10 as functions of the site category of the considered supports k and l, respectively. For instance, in case of a bridge, of which piles 1 and 2 are implemented on sites of type A and C, with a 300 m distance between piles, the correlation

coefficient, ρ_{12}, is obtained through Formula (9) with $L_{g,1} = 400\ m, L_{g,2} = 250\ m, L_{12} = 300\ m$. Formula (9) is derived from analyses carried out by Papadopoulos and Sextos [8].

$$\rho_{kl} = \exp\left(-\frac{2\,L_{kl}}{a_{kl}(L_{g,k}+L_{g,l})}\right) \text{ with } a_{kl} = \exp\left(-\frac{|L_{g,k}-L_{g,l}|}{500}\right) \quad (9)$$

In case the seismic action is represented in the form of time series, the normalised cross-correlation coefficient between two input motions applied in the same direction at supports *k* and *l* should not exceed the largest of ρ_{kl} given by Equation (9), and 0,2.

In the Eurocode 8 framework, spatial variability is principally considered for large bridges and surface pipelines. However, the above correlation values could be used as references for selecting spatial variable input motions applicable to large dams and calculating their effects.

Table 10. Characteristic lengths

Site category	A	B	C	D	E	F
L_g (m)	400	300	250	200	300	200

7. CONCLUSIONS

Definition and representation of the seismic action in the Eurocode 8 of next generation present some significant changes in comparison with the current version. Likely, the most visible change for engineers is the new anchoring system of standardized response spectra, which is not anymore based on the PGA but on two spectral values: S_α, the plateau spectrum value, and S_β, the spectrum value at 1 s spectral period. An option enables deriving S_β from S_α, to have only one anchoring parameter, as requested by some CEN National Members.

Another significant change is site categorisation, now given by a two-entry (H_{800} and $v_{s,H}$) table, resolving ambiguities that appeared with the current classification. Site categorisation may also be derived from the site natural fundamental frequency, f_0, which is easy to acquire on site. Based on this new categorisation, site amplification factors are updated according to the most recent scientific progress on the subject, including the effect of input motion intensity.

In the spirit of site categorisation based on f_0, the new draft aims at encouraging users acquiring better quantified knowledge of the considered sites, in terms of seismic hazard and of geological and geotechnical data. In this spirit, the text is drafted in such a way that the user can take benefit of any effort in favour of a well-documented site.

In accordance with the general Eurocode policy, every effort was made to limit introduction of national determined parameters (NDPs) to those parameters that concern national safety choices. Also, in the Eurocode spirit of enhancing harmonisation of engineering practices

across Europe, two informative maps of seismic hazard are introduced in the new draft, representing S_α and S_β values on type A sites for a 475-year return period. However, in the Eurocode terminology, “informative” means that S_α and S_β values operative for design are defined in the NDP list and should be provided by every country.

In the light of dam industry practices, important developments in the Eurocode 8 of next generation are linked to the use of time-response analysis. First, guidance is given for the selection of accelerograms out of databases, so that they are compatible with the site standard spectrum in terms of the magnitude that originates the seismic input motion and of duration of the strong motion. Second, acceptance criteria for the compatibility of a set of accelerograms with the site standard spectrum are significantly laxer than in the current version. The philosophy of the standard being now that, under some limited constraints of compatibility with a target spectrum, a set of natural input motions is more legitimate to represent the appropriate seismic input motion than a non-realistic response spectrum would be.

This standard philosophy is also adapted to the fact that the new standard should last for about 15 years, and that it is anticipated that in the coming years, time response analyses will play a growing role in earthquake engineering, concurrently with the development of more realistic representation of seismic responses of structures in non-linear regimes, particularly when dealing with highly non-linear structures such as embankment or earthfill dams.

REFERENCES

[1] CEN/TC250 (2024). Eurocode 8 Earthquake resistance design of structures, Part 1-1, General rules and seismic action. FprEN1998-1-1 :2024.

[2] Labbé P., Paolucci R. Developments relating to seismic action in the Eurocode 8 of next generation in Progresses in European Earthquake Engineering and Seismology, 3rd European Conf. on Earthquake Eng. and Seismology – Bucharest, 2022, Springer

[3] Pitilakis K, Riga E, Anastasiadis A (2013) New code site classification, amplification factors and normalized response spectra based on a worldwide ground-motion database. Bull Earthq Eng 11(4):925–966

[4] Paolucci, R., Aimar, M., Ciancimino, A., Dotti M., Foti S., Lanzano G., Mattevi P., Pacor F., Vanini M. (2021). Checking the site categorization criteria and amplification factors of the 2021 draft of Eurocode 8 Part 1–1. Bull Earthquake Eng 19, 4199–4234.

[5] Danciu L., Nandan S., Reyes C., Basili R., Weatherill G., Beauval C., Rovida A., Vilanova S., Sesetyan K., Bard P-Y., Cotton F., Wiemer S., Giardini D. (2021) - The 2020 update of the European Seismic Hazard Model: Model Overview. EFEHR Technical Report 001, v1.0.0, https://doi.org/10.12686/a15

[6] Pitilakis K., Riga E., Anastasiadis A., Fotopoulou S., Karafagka S., 2019, “Towards the revision of EC8: Proposal for an alternative site classification scheme and associated intensity dependent spectral amplification factors”, Soil Dynamics and Earthquake Engineering, vol. 126, DOI: 10.1016/j.soildyn.2018.03.030

[7] Paolucci R., C. Smerzini (2018). Empirical evaluation of peak ground velocity and displacement as a function of elastic spectral ordinates for design. Earthquake Engineering and Structural Dynamics, 47: 245-255.

[8] Papadopoulos S.P., Sextos A.G. (2020). Simplified design of bridges for multiple-support earthquake excitation. Soil Dynamics and Earthquake Engineering 131 106013

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

OPPORTUNITIES AND CHALLENGES IN APPLYING BIM METHODOLOGIES WITHIN THE PORTUGUESE DAM SAFETY FRAMEWORK

Carlos Serra[1]
National Laboratory for Civil Engineering (LNEC)

João Conde Silva
National Laboratory for Civil Engineering (LNEC)

Jorge Sampaio Carvalho
Portuguese Environment Agency (APA)

ABSTRACT

In the current context of rapid implementation of the Building Information Modelling (BIM) methodology, there is a need to discuss BIM within the dam safety environment with an underlining attention to dam design, construction and operation aspects. Under these three stages, dam safety activities are key to ensure the longevity and adequate operation of the dam structures and appurtenant works.

BIM methodology allows for the definition of a systematic approach to the development of dam safety control activities, within the existing regulations and practices. The advantages of this approach include better interaction and exchange of information between stakeholders, the definition of universal nomenclatures for the various elements and activities, standardization of the information needed to the development of each activity, development of three-dimensional models for the support of the design, assessing construction activities and operation of infrastructures, development of automation tools for processing monitoring and decision support data, among others.

The opportunities and challenges of implementing BIM methodologies within the dam safety framework are discussed in this paper, bearing in mind the necessary contributions from the different stakeholders and the role of the Portuguese authority in dam safety regulation.

Keywords: Building Information modelling (BIM), Dam Safety, Portuguese dam safety authority

[1] cserra@lnec.pt, Portugal

1. INTRODUCTION

Hydraulic structures constitute complex systems that include the main structure, appurtenant hydraulic structures, the foundation mass and the reservoir. Each of these elements involves a potential risk to downstream valleys, that may result in significant loss of lives and economic and environmental damages. Dam safety regulations and guidelines define the legal requirements concerning the design, construction, first filling, operation, maintenance and removal of these structures and national supervision authorities are key players to ensure safety through the hydraulic structure's lifespan.

Structural safety is generally divided into three main activities: i) monitoring, i.e., operating monitoring systems and carrying out regular visual inspections and characterization tests; ii) analysis and interpretation of monitoring results through statistical models and numerical modelling; iii) safety assessment and decision making. Monitoring the loads and response of the structures is one of the main activities related with safety control, not only in the short-term regarding ongoing maintenance works, but also in assessing structural behaviour under extreme events, as well as assessment of deterioration scenarios.

The Building Information Modelling (BIM) methodology allows defining a systematic approach for the development of dam safety control activities, improving the interaction and exchange of information between stakeholders, by considering the definition of universal nomenclatures for the different elements and activities, standardization of information for each activity, development of three-dimensional models to support the design, construction and operation phases and development of automation tools for processing monitoring and decision support data, among others.

According to a BCG research, in 2026 a "full-scale digitalization in non-residential construction will lead to annual global cost savings of 13% to 21% ($0.7 billion to $1.2 billion) engineering and construction phases and 10% to 17% ($0.3 billion to $0.5 billion) in the operations phase" [1].

Argyroudis *et al.* [2] explored the possibilities and challenges related to using digital technologies to increase the resilience of infrastructures to climate action. They refer to the Internet of Things (IoT), Digital Twins, virtual and augmented reality, point clouds, Artificial Intelligence and BIM methodology as emerging technologies to support reliable decisions and reduce uncertainties related to safety assessment. The bibliographical work review by Panah and Kioumarsi [3] gives a global view of the growing importance of BIM applications in monitoring and maintenance of structures. Jeong *et al.* [4] present a framework for managing information associated with bridge monitoring with the integration of instruments installed on

site into a three-dimensional model. In turn, Sacks *et al*. [5] demonstrate the potential of an integrated bridge monitoring management system that includes the systematic use of point clouds, from a scan to BIM perspective. This work presents the added value relying on an Industry Foundation Classes (IFC) extension to bridges. Regarding dam engineering, Zhang *et al.* [6] present an integrated BIM-based technology system for digital design, construction and operation which accounts for key features of BIM methodology and several practical examples.

There are local examples of specific studies applied to concrete dams [7–9] and embankment dams [10]. Díaz *et al.* [9] present some applications of the BIM methodology applied to hydraulic structures, including the development of the ODIS – Online Dam Information System platform for integrating monitoring data into a concrete dam three-dimensional model. Heleno's study [10] focused particularly on embankment dams and allowed the development of a methodology for organizing and exploring monitoring data and integrating it into a three-dimensional model in BIM.

This paper aims to present some insights regarding the benefits and issues arising with the application of the BIM methodology in dam safety activities and points out the expected challenges during its implementation.

2. DIGITAL TRANSFORMATION OF DAM SAFETY ACTIVITIES

Building Information Modelling (BIM) is changing the Architecture, Engineering, Construction and Operation (AECO) industry by providing new work methodologies supported with powerful technological advancements. BIM can be defined as methodology for sharing information and communication between all stakeholders and during all phases of a construction's life cycle, based on a digital representation of the physical and functional characteristics of an asset. It is intended to be a shared knowledge resource for information to be used as the basis for decision-making [11]. The word “building” in BIM stands for any type of built environment, new or existing, “information” means the combination of geometrical and alphanumerical data associated with an object and “model” refers to the digital representation of the information (geometrical and data).

In large projects, such as dam design and construction, the adoption of BIM is known to be beneficial due to its size and scale, budget, long project duration, complexity, risk and environmental and regulatory issues [12], regarding not only hydropower infrastructures but also all types of water management works. Each country, usually coordinated with the International Commission on Large Dams (ICOLD), has its own standard and guidelines to regulate the challenges associated with design, building and operating these complex structures.

To summarise, BIM enables firstly and foremost a structured and standard methodology for the process of design, construction and operation of building assets, defining technical aspects and roles, interactions between stakeholders, deliverables, information storage and share. Thus, BIM implementation is an opportunity to review, optimize and define work processes, adapted to the existing standards and guidelines, ensuring the use of digital technologies to aid in simple and complex tasks or activities and taking advantage of the digital transformation.

Despite all the challenges of design and construction, the operation phase is the period in which the advantages of BIM implementation are more relevant, when evaluating the complete lifecycle of the infrastructure, especially if a BIM model is used since the design phase. The existence of a digital twin, with judicious data-enrichment as the real structure is being built, is the optimal solution for day-to-day operation and maintenance of new developments, working as information repository linked to geometrical data and enhancing analysis and ultimately decision-making.

Dam safety activities are present since the beginning of the process and are essential to ensure the life, environmental and asset safety.

In Portugal, dam safety regulation and coordination are under the responsibility of a number of Public Administration entities (Portuguese Environment Agency, APA, National Laboratory for Civil Engineering, LNEC, and National Emergency and Civil Protection Authority, ANEPC, and the Dam Safety Commission, CSB) and the dam owner. These entities interact with one another, from the design to dam removal phase, in order to maintain an adequate safety level of these structures. The challenge associated with safety is considerable, given the complexity and amount of information generated in this type of activity. Furthermore, decades can go by since the beginning of the project to the completion of construction.

The Portuguese Dam Safety Regulation (RSB) [13, 14] frames the activities related to the safety of large dams in Portugal and defines the various entities and stakeholders and their responsibilities in this regard. RSB defines large dams as hydraulic infrastructures with a height of more than 15 m (from the lowest point of the foundation to dam crest) or dams with a height equal to or greater than 10 m that create a reservoir with a volume greater than 1,000,000 m^3.

The RSB structure defines the responsibility of each entity and stakeholder and the relationship between them. In recent years, a number of studies and work carried out linking dam safety procedures and BIM methodologies [15–18], highlighted the interest in establishing a strategy for BIM implementation based on RSB premisses, goals and procedures. BIM methodologies should be perceived as set of tools to enable efficiency and improve collaboration, within the existing regulations and practices. It should be highlighted that the existing regulation does not restrict the use of emerging tools to support the development of dam safety activities.

It is known that the definition of a strategy for the Portuguese dam safety framework requires assessment by of various entities and stakeholders, namely APA, LNEC, ANEPC, CSB and dam owners. The different investment capacities and specialized technical support available by existing entities and dam safety stakeholders in Portugal constitute crucial challenges to implement BIM methodologies, particularly in dam developments currently under operation. Nevertheless, development of current BIM methodologies and related projects may set the bases for a future expansion of this procedures on a wider scale.

This paper introduces a number of elements that may be useful to discuss the implementation of the BIM methodologies and identifies various existing components in dam safety, in particular those included in the information management system and the BIM model. The structure of the processes during the application of the BIM methodology must follow, with the necessary adaptations, the standards EN ISO 19650-1:2018 [19], EN ISO 19650-2:2018 [20] and CEN/TR technical report 17439 [21].

Establishing a parallel with the current activities of the AEC industry, the nature of the dam safety activity may be considered close to asset maintenance, which corresponds to an advanced level of the information production process [19].

One of the main challenges of implementing the BIM methodology in the context of dam safety relates to adapting the use of a management information system integrated into the BIM model, particularly during the operation phase, when incorporating inspection results. Under the BIM methodology, the production and exchange of information in the various phases of the project and the transposition of the basic concepts defined in the standards in a transversal way are likely to benefit from further discussion in the technical environment.

Regarding technological aspects, a number of options recently adopted in several projects can be included in BIM workflows and power the digital improvement of dam safety activities, namely:

- The use of reality capture techniques such as laser scanning or photogrammetry to obtain useful point clouds at a given time or reliable georeferenced orthophotos to aid the inspection activities. Drones are being used to obtain these products in distant or remote areas. Similarly, Remotely Operated Vehicles (ROV) are being deployed for inspecting pipelines, tunnels, and other confined spaces, reducing the risk to humans;
- The development of BIM models enriched with relevant data from design and construction, combining the different technical areas (geotechnical, civil, hydraulic, mechanical, electrical), to be the basis for an efficient asset management using digital twins [22];

- The use of automated data systems (ADS), modern monitoring devices and its integration with digital twins in real, based on the internet of things (IoT) concepts, enabling early detection of potential issues [23, 24];
- Augmented reality technology can assist maintenance workers by overlaying digital information onto the physical world, enhancing accuracy and efficiency and reducing the likelihood of errors [25, 26].

3. THE ROLE OF THE PORTUGUESE DAM SAFETY AUTHORITY WITH A VIEW TO IMPLEMENT A BIM METHODOLOGY

In Portugal, the National Dam Safety Authority duties are carried out by the Portuguese Environment Agency (APA) which acts as the regulator within the applicable legislation framework related with the safety of the dams, having the specific task to verify compliance with dam safety regulations.

The main Dam Safety Regulations are published by Decree-Law nr. 21/2018 in 28 of march, which, together with the Dam Safety Guidelines (Documentos Técnicos de Apoio) published by APA in April 2018 [13, 14], define the essential framework regarding the safety of the Portuguese dams. These are crucial documents to minimize the potential damages to people and property in the downstream valley, establishing rules for the design, construction and operation of dams, inspection, monitoring and maintenance programs, emergency plans and the appropriate technical qualifications of the agents involved in these tasks.

Besides APA, acting as the dam safety authority, other organizations are broadly involved in safety, namely a number of public administration entities, such as the National Laboratory for Civil Engineering (LNEC), which supports dam safety activities as a consultant to the Authority, the National Authority for Emergency and Civil Protection (ANEPC), which coordinates national civil protection activities, and the Dam Safety Commission (CSB), which independently analyses the status on safety of the Portuguese dams, with the support of the APA, LNEC and relevant stakeholders.

The dam owners are responsible for the dams before the Authority (APA), and have the assignment to maintain, operate, monitor, repair the structures and related equipment, report incidents and accidents, and bear all inherent costs involved in dam safety, having the obligation to comply with all dam regulation matters.

It is crucial to take into consideration that the dam owners are very diverse, and we can find several different entities in the state administration, hydropower companies, agriculture associations and multiple purpose developments. This broad diversity of dam owners brings

additional challenges to the role of the Portuguese Dam Safety Authority, due to the different financial and technical capabilities of the same, in the context of safety.

Another important role of the Portuguese dam safety authority is the classification and archive of administrative and technical documents, both in digital and physical support. Whilst part of this work is focused on obtaining digital documentation files collecting data of older dams, whose management remains the responsibility of the state, a vast collection of documentation remains to be digitized, associated with a set of 270 large dams and a register of 700 smaller dams.

In the later years, a growing concern has arisen regarding the large volume of digital information dealt with by the dam safety Authority, which is currently increasing exponentially, challenging the way in which it is managed and organized. This is particularly the case with the documentation related to the implementation of emergency action plans, though the new dam developments under assessment or currently being built, are also cause of concern in this regard.

In this context, development of specific information management tools in a systematic approach is likely to optimize the access to technical information in dam safety activities. Therefore, the implementation of a new methodology to interact between the authority and the dam safety agents, namely dam owners, by allowing to automatically upload information, is an important step further. BIM may be seen as an enabling tool to improve this information management system, though it is important to acknowledge that a substantial investment in software capabilities and human resources is needed to enable the administration to cope with the increasing technological demands.

On a different level, APA is also carrying out a relevant task within the National Portuguese Committee on Large Dams (CNPGB), which represents Portugal as member of ICOLD, thus participating in the activities of the existing technical committees.

Under this task, availability of essential technical information on dams to the general public is to be considered, work that is currently being carried out within the scope of the CNPGB, this requiring permanent updating and validation of data, whilst harmonizing the national register of large dams in coordination with the ICOLD Committee O criteria.

Following the development of the BIM methodology related to dam safety in Portugal, an interest has been identified in considering the recent work that has been undertaken under Committee O, focusing on documentation and the world register of dams.

The criteria standardized by ICOLD, following the work of Committee O in the technical characterization of large dams, emerges as promising support to promote the standardization

of nomenclature, context and procedures when defining the different components of the BIM methodology.

4. OPPORTUNITIES AND CHALLENGES OF BIM IMPLEMENTATION IN DAM SAFETY ACTIVITIES

4.1. Initial remarks

As previously described, BIM is a powerful methodology to revolutionize the AECO industry by setting uniform standards across the “building” lifecycle to ensure efficiency, automation and an overall collaboration between stakeholders. Although it is currently established for new buildings, important steps are being taken towards its implementation on large infrastructure projects, such as bridges, railway, airports, and specifically to the operation phase (facility management) [3, 27, 28]. The use of BIM for dams is still limited to some dam owners and contractors and an approach for dam safety activities (which can be fitted in the operation phase) are even more scarce.

This section discusses more in detail the opportunities and challenges related to the BIM implementation aiming for a holistic management across the project lifecycle [12], for an increase on efficiency and coordination between stakeholders and data integration for various sources [29]. The focus will be on dam safety activities.

4.2. Opportunities

As BIM allows for the integration of different technical fields in a federated BIM model, one of the primary benefits is medium to long-term cost reduction, as it facilitates the identification of design clashes and optimizes resource use, ultimately leading to lower overall project costs throughout the construction lifecycle. Another main feature is parametric modelling based on object-oriented strategy, in which BIM objects are defined using specific geometric parameters and have relations with other BIM objects. This is an advantage for making consistent and quick changes in the model, avoiding errors/mismatches and producing 2D drawings, instantaneously and automatically. The automatic quantity take-off, a task prone to errors, is also seen as one of the main benefits of a well-structured BIM model. Interoperability between modelling software and structural analysis software can introduce a major gain in the iterative design process. BIM can provide valuable “as-built” documentation for existing buildings, which is crucial for maintenance, warranty, and service information management. This documentation aids in quality control, assessment, monitoring, energy and space management, emergency management, and retrofit planning [11].

Closely related to early clash identification during design and construction, the potential for seamless information exchanges and collaboration between each technical team is a key element of BIM, powered by BIM Collaboration Format (BCF) which allows for “different BIM applications to communicate model-based issues and other kinds of topics with each other by leveraging IFC models that have been previously shared among project collaborators” [30]. This type of communication is model-based and can be used during operation phase to register and report, for example, maintenance issues, anomalies or, even, safety issues related to a given dam element [31, 32].

It is known that BIM implementation can be costly and challenging at an initial stage, due to cultural change, slow learning curve, software licenses, training and difficulty of acquiring skills for stakeholders, as well as a greater need for team involvement, interoperability, copyright and accountability [15, 33], but by leveraging its advantages it can be an introduce new workflows and procedure that benefit dam owners, designers, contractors, government institutions and the general public [34].

The work of Monteiro and Lima [18] describes, in general, the challenges of implementing BIM methodologies in the construction industry, typifying the implementation profiles (project teams, construction companies, building operations managers and construction owners and promoters) and presents some operational principles in order to optimize the process. The operational principles described are generic and valid in a high-level framework but can be transposed to the context of dam safety, namely, the systematization of modelling processes and work processes in general, the interaction of BIM models with external databases and the use of collaborative platforms for sharing and formal submission of models. Within the Portuguese context, Serra [32] lays out a roadmap for BIM regarding structural safety activities, by identifying the existing types of information obtained during design, construction and operation, setting the relation between the BIM model and an information management system and proposing several steps towards a full BIM implementation.

The recent review paper of Wangchuk *et al.* [12] lists several practical examples of BIM uses for hydropower projects, separated by: i) general uses - development of BIM model for asset management towards a digital twin, collaborative design and project optimization, development of a design and analysis-integrated platform, 3D modelling for structural and hydraulic analysis, as well as integration of GIS data for planning; ii) monitoring uses - development of BIM model for project monitoring during construction, BIM-based ontology technology for dam safety monitoring [35], workflow for dynamic BIM-augmented UAV safety inspection [36], and real-time data monitoring using BIM model [37].

The Web Dam Information System (ODIS) developed by the company OFITECO and ViCon is an example of a BIM tool that incorporates monitoring data and a three-dimensional BIM

model [38, 39]. The system allows the visualization of data through time series, the definition of statistical models and warnings and alerts for the management of emergency plans through a three-dimensional model of the dam where information can be accessed and visualized, and incorporates the information collected in visual inspections of the database. In turn, Heleno [16], using several technological platforms, proposed a workflow for integrating monitoring data into a BIM model of an embankment dam, allowing a better perception of the structure's behaviour. Serra *et al.* [31] proposed some openBIM concepts regarding concrete dam inspection, including a new IFC schema extensions for better interoperability between dam owner and national safety authority.

These examples demonstrate the possibilities and advantages of an integrated system, however, they only constitute the technological part of the BIM methodology applied to structural safety. In order to leverage the investment associated with BIM implementation, it is highly recommended to follow standards and guidelines to increase interoperability, ease information exchange and avoid misconceptions and errors.

4.3. Challenges

The ability of different BIM applications to exchange and utilize data seamlessly is often restricted, particularly in hydraulic engineering, where varying standards and formats complicate data sharing [40] which is still a missing piece for full-scale BIM implementation. Technically, there are also investments to be taken into account to properly store and maintain the vast and diverse data and information obtained during construction (geometric information, building schedules, material information, testing results, documentation, …) and from monitoring devices and inspections over time.

Another needed investment is in training programs, private and public, to leverage these advanced technologies and introduce them into the day-to-day activities. Private businesses, such as dam owners, designers and contractors should invest in digitalisation and BIM methodology training to begin to gradually shift to more efficient workflows. National authorities may also have a role in the dissemination of knowledge and in the promotion of specific training courses for the technical community.

Even though specific dam-related BIM standards and guidelines are still to be published or even drafted, dam engineering has a good backbone regarding technical standards and regulations, provided by national institutions and by ICOLD, which is followed by many countries worldwide. Alongside these technological developments, those involved in this type of activity, including the different dam safety stakeholders and public institutions, may contribute to the standardisation of concepts, terms and processes, at a national level, in order to improve communication and information exchange. Similarly to BIM standard EN ISO 19650

[19, 21], these specific standards often define many requirements necessary to follow BIM implementation, such as roles, responsibilities and relationships between stakeholders, dam engineering concepts and taxonomy, activities, information exchange and deliverables regarding dam safety over time. Although there is a long road for uniform standardisation of dam engineering activities, there is also a structured background to work with.

Besides the use of standards and guidelines for practitioners and contractors, a number of public and government institutions are requiring the use of openBIM concept, formats and workflows to ensure interoperability between different stakeholders and software, neutral standards, reliability for data exchanges and good collaboration workflows [41].

Furthermore, the lack of real-time data integration limits the effectiveness of BIM in managing these structures during their operational and maintenance phases, hindering timely decision-making.

Concerns related to data security and privacy issues are seen as an important matter for the Portuguese dam safety authority when it comes to digital applications. The protection of sensitive data and personal information is crucial not only for businesses but also, for national protection and safety. Transfer of physical to digital information should include redundancy in the archiving process, and remote access to documents in connection with the ability to defend these actions against cyber-attacks from external sources. In this context, it is opportune to recall the situation of severe loss of information occurred with the Scottish Dam Safety Authority [42].

It is also critical in this process related with the implementation of BIM methodologies in dam safety activities, from the Portuguese dam safety authority point of view, that this update is consider the diversity of dam owners in terms of technical and financial support, as well as allowing enough time and resources for the process to evolve, in accordance with each activity sector capabilities. To address these limitations, enhancing data interoperability through standardized protocols could facilitate better data exchange among BIM applications. Additionally, improving modelling techniques to accommodate the complexities of hydraulic structures and ensuring that BIM tools are up to date to reflect current regulatory requirements would be beneficial. Finally, developing better integration strategies with legacy systems can streamline workflows and improve overall project efficiency. By tackling these challenges, the effectiveness of BIM applications in hydraulic engineering can be significantly improved to exploit all their inherent benefits.

4.4. Discussion on BIM implementation

According to EN 17412-1:2020 [43], the basis for the level of information required is the definition of the BIM objective or use (reason for which the information is defined), the moment

of production and delivery of the information, of who and for whom the information is and about what object the information is processed. The information is defined in terms of Geometric Information, Alphanumeric Information and Documentation (Figure 8 of EN 17412-1:2020 [43]). By specifying a necessary level of information for a given object, in each phase of the process, a geometric and informational detail of the model is defined that is appropriate to the needs of each BIM use. The needs for geometric detail and alphanumeric information detail may not coincide for many of the objects related to safety and monitoring, for example, recording an occurrence during a visual inspection using the Patch concept with simplified geometry, georeferenced location in the BIM model and the various properties that relate to it [44] or the representation of monitoring devices in a simplified form but containing information relating to the date of installation, maintenance forecasts, suppliers, calibration constants, operational status, monitoring results, among others.

The Dam Safety Regulation (RSB) [14] determines that the National Laboratory for Civil Engineering(LNEC) must create and maintain a digital file updated with data and monitoring results from large Portuguese dams. To this end, LNEC developed an information system, gestBarragens [45, 46] which includes a structured database, as well as different operational modules that allow the management of data and monitoring results, visual inspections and documentation. technique, visualization of graphical results and support for the interpretation of behaviour [46]. It is worth noting that during the implementation of the gestBarragens information management system, a structure was already developed that defines the relationships between certain objects (for example, the relationships between structural elements and sub-elements: dam is composed of blocks and joints of contraction; the relationship between sub-elements and instruments: groups of extensometers embedded in the blocks and joint movement meters in the contraction joints).

The criteria standardized by ICOLD, following the work of Committee O in the technical characterization of large dams, and the structured database used in gestBarragens information management system, may be the basis for defining the BIM model, including the necessary information requirements for parametric BIM objects concerning structural elements, equipment and monitoring devices.

With the objective of developing or using a BIM model that integrates the information necessary to carry out the various activities of a given project, the high degree of complexity and existing information implies the organization and systematization of each element in a universal way. Thus, one of the first challenges in the BIM implementation of dam safety activities is the structuring of the main concepts, definitions, elements and activities that make up the dam-foundation-reservoir system.

Data structuring must integrate an existing Construction Information Classification System (CICS), following pre-defined assumptions, to have greater compatibility between specialties and a common basis for the identification of objects and for the interconnection between the BIM model and information management system.

This task can be greatly facilitated by the structure already proposed by the International Commission on Large Dams (ICOLD) through the provision and codification of several terms related to dam engineering [47].

In conjunction with integration into a classification system, it is also necessary to extend these elements to the Industry Foundation Classes (IFC) format, following the standard [48], similar to what has been done for other types of infrastructure (ports, roads railways, roads) and for concrete dams [49].

Following the above, it is essential to develop BIM models that include not only geometric information, through parametric BIM objects built according to pre-defined modelling criteria, but also the information necessary for their practical use. The integration of information collected in visual inspections and monitoring and its visualization in an effective and intuitive way is also a point to be considered [10].

Ideally, the BIM model should start in the design phase, be adapted during the construction phase and made available as an as-Built georeferenced BIM model for the different safety actors, integrating the different specialties, including the monitoring system.

The pre-existence of a classification system adapted to the reality of dams, rules for modelling and creating objects and instructions for use and export to an open format are requirements for the functioning of an integrated BIM system. The Portuguese BIM Standardization Technical Commission (CT197) has been contributing with recommendations for the standardization of BIM modelling that could be adopted in the development of BIM objects in the context of dams [50].

5. FINAL REMARKS

This paper presents some advantages, opportunities and challenges of the use of BIM methodologies for the main dam safety activities, considering the Portuguese framework.

Resource optimization, interoperability and collaboration are critical advantages of BIM, leading to efficient use of materials, human and time resources. The discussed advantages underscore the transformative impact of BIM on the construction of hydraulic structures and dams. Despite that, BIM applications to dam safety activities face several limitations, primarily due to data interoperability issues, modelling complexity, regulatory compliance challenges, and integration with legacy systems.

Addressing these challenges requires a collaborative approach involving stakeholders from various sectors, continuous investment in research and development, and the adaptation to regulatory frameworks. Overcoming these barriers will pave the way for more widespread and effective use of digital technologies in infrastructure safety control.

REFERENCES

[1] BCG (2020). The Transformative Power of Building Information Modeling. In: BCG Global. https://www.bcg.com/publications/2016/engineered-products-infrastructure-digital-transformative-power-building-information-modeling. Accessed 17 Dec 2024.

[2] Argyroudis, S. A.; Mitoulis, S. A.; Chatzi, E., *et al* (2021). Digital technologies can enhance global climate resilience of critical infrastructure. Climate Risk Management 100387–100387. https://doi.org/10.1016/J.CRM.2021.100387.

[3] Panah, R. S.; Kioumarsi, M. (2021). Application of building information modelling (BIM) in the health monitoring and maintenance process: A systematic review. Sensors (Switzerland) 21:1–26. https://doi.org/10.3390/s21030837.

[4] Jeong, S.; Hou, R.; Lynch, J.P.; *et al* (2017). An information modeling framework for bridge monitoring. Advances in Engineering Software 114:11–31. https://doi.org/10.1016/J.ADVENGSOFT.2017.05.009.

[5] Sacks, R; Kedar, A.; Borrmann, A.; *et al* (2018). SeeBridge as next generation bridge inspection: Overview, Information Delivery Manual and Model View Definition. Automation in Construction 90:134–145. https://doi.org/10.1016/j.autcon.2018.02.033.

[6] Zhang, Z.; Zhang, S.; Zhao, Z.; *et al* (2023). HydroBIM—Digital design, intelligent construction, and smart operation. Journal of Intelligent Construction. https://doi.org/10.26599/JIC.2023.9180014.

[7] Marques, A.; Azenha, M.; Ferreira, A. (2016). BIM no projeto de estruturas de obras hidroelétricas. Universidade do Minho, Guimarães, pp 163–172.

[8. Fernandes, L. (2015). An integrated model for simulation of construction phasing of arch concrete dams. Universidade do Minho.

[9] Díaz, S.; Galán, A.; Arrieta, A.; *et al* (2019). Building Information Modeling (BIM) en Ingeniería Hidráulica. Toledo, pp 1–5.

[10] Heleno, A.; Azenha, M. (2022). Aplicação da metodologia BIM na monitorização da segurança de barragens de aterro. In: Figueiredo B, Azenha M, Lino JC (eds) 4o Congresso Português de Building Information Modelling vol. 1 - ptBIM, 1st ed. UMinho Editora, pp 447–457.

[11] Volk, R.; Stengel, J.; Schultmann, F. (2014). Building Information Modeling (BIM) for existing buildings - Literature review and future needs. Automation in Construction 38:109–127. https://doi.org/10.1016/J.AUTCON.2013.10.023.

[12] Wangchuk, J.; Banihashemi, S; Abbasianjahromi, H; Antwi-Afari MF (2024). Building Information Modelling in Hydropower Infrastructures: Design, Engineering and Management Perspectives. Infrastructures 9:98. https://doi.org/10.3390/infrastructures9070098.

[13] APA (2025). Legislação e guias | Agência Portuguesa do Ambiente. https://apambiente.pt/prevencao-e-gestao-de-riscos/legislacao-e-guias. Accessed 6 Jan 2025.

[14] APA (2018). Regulamento de Segurança de Barragens. Decreto-Lei nº 21/2018. Diário da República, Lisboa.

[15] Marques, A. (2015). Implementação de metodologias BIM na Direção de Engenharia de Barragens da EDP: Casos de estudo de projeto de estruturas em obras Hidroelétricas. Universidade do Minho.

[16] Heleno, A. (2021). BIM for information management in structural safety control of embankment dams BIM for information management in structural safety control of embankment dams.

[17] Cruz, P.J.S.; Azenha, M. (2018). The challenges of adopting BIM for setting and infrastructure management of University of Minho. E3S Web of Conferences 48:4–7. https://doi.org/10.1051/e3sconf/20184802002.

[18] Monteiro, A.; Lima, J. (2018). Implementação de processos BIM: desafios na aplicação da metodologia em diferentes enquadramentos da indústria da construção. In: Costa AA, Azenha M (eds). Instituto Superior Técnico, Universidade de Lisboa, Lisboa, pp 89–98.

[19] CEN (2018). EN ISO 19650-1:2018 - Organization and digitization of information about buildings and civil engineering works, including building information modelling (BIM) — Information management using building information modelling — Part 1: Concepts and principles. 34–34.

[20] CEN (2018). EN ISO 19650-2:2018 - Organization and digitization of information about buildings and civil engineering works, including building information modelling (BIM) — Information management using building information modelling — Part 2: Delivery phase.

[21] CEN (2005). CEN/TR 17439:2020 - Guidance on how to implement EN ISO 19650-1 and -2 in Europe.

[22] Singh, M.; Fuenmayor, E.; Hinchy, E.; *et al* (2021). Digital Twin: Origin to Future. ASI 4:36. https://doi.org/10.3390/asi4020036.

[23] Liu, R.; Luo, Z. (2023). Application of digital intelligent construction in the field of water conservancy and hydropower engineering. J Phys: Conf Ser 2565:012025. https://doi.org/10.1088/1742-6596/2565/1/012025.

[24] Mavrova-Guirguinova, M. (2023). The Digital Era in Hydraulic Engineering Comes with Applications of Artificial Intelligence. Copernicus Meetings.

[25] Trindade N.; Ferreira A.; Oliveira S. (2019) DamAR: Augmented Reality in Dam Safety Control. International Journal on Hydropower and Dams 5:56–62.

[26] Trindade, N.; Ferreira, A.; Pereira, J.; Oliveira, S. (2023). Extended reality in AEC. Automation in Construction 154:105018. https://doi.org/10.1016/j.autcon.2023.105018.

[27] Cruz, P.J.S.; Azenha, M.; Mateus, R.; *et al* (2019). BIM as a tool for setting and infrastructure management. 1–7.

[28] Mohamed, A.G; Khaled, A.; Abotaleb I.S. (2023). A Bridge Information Modeling (BrIM) Framework for Inspection and Maintenance Intervention in Reinforced Concrete Bridges. Buildings 13:2798. https://doi.org/10.3390/buildings13112798.

[29] Biancardo, S.A.; Intignano, M.; Paola, F.D.; *et al* (2024). VIASTRATA®: The New Frontiers of BIM for the Digitalisation and Management of Infrastructures. In: Recent Topics in Highway Engineering - Up-to-Date Overview of Practical Knowledge. IntechOpen.

[30] buildingSMART International (2024). BIM Collaboration Format (BCF). In: buildingSMART Technical. https://technical.buildingsmart.org/standards/bcf/. Accessed 20 Dec 2024.

[31] Serra, C.; Silva, J.C.; Granja, J.; Azenha, M. (2024). BIM no controlo de segurança de barragens de betão: uma ferramenta de apoio às inspeções visuais (em processo de revisão).

[32] Serra, C. (2022). As potencialidades e desafios da implementação BIM adaptada ao controlo de segurança estrutural de barragens de betão. In: Figueiredo B., Azenha M., Lino J.C. (eds) 4o Congresso Português de Building Information Modelling vol. 1 - ptBIM, 1st ed. UMinho Editora, pp 371–382.

[33] Lino, J.C.; Azenha, M.; Lourenço, P. (2012). Integração da metodologia BIM na engenharia de estruturas. Encontro Nacional Betão Estrutural -BE2012. 1–10.

[34] Pinnacle (2023). The Cost-Benefit Analysis of BIM Services for Construction Companies. In: Pinnacle Infotech. https://pinnacleinfotech.com/cost-benefit-analysis-bim-services-construction-companies/. Accessed 19 Dec 2024.

[35] Zhou, Y.; Bao, T.; Shu, X.; *et al* (2023). BIM and ontology-based knowledge management for dam safety monitoring. Automation in Construction 145:104649. https://doi.org/10.1016/j.autcon.2022.104649.

[36] Liu, D.; Chen, J.; Hu, D.; Zhang, Z. (2019). Dynamic BIM-augmented UAV safety inspection for water diversion project. Computers in Industry 108:163–177. https://doi.org/10.1016/j.compind.2019.03.004.

[37] Ji, S.; Hu, T. (2019). Construction Simulation and Real-time Monitoring Research of Concrete Dam Based on BIM. IOP Conference Series: Earth and Environmental Science 340. https://doi.org/10.1088/1755-1315/304/5/052056.

[38] Ortuño, M.G.M.; Hoppe S. (2015). Dam monitoring Spain. The International Journal on Hydropower and Dams.

[39] Hoppe, S; Ortuño, M. G. M.; Fleitz, J. (2017). Implementation of a web based platform for portfolio dam monitoring. In: Proceedings of the 4th International Conference. Graz University of Technology, Tehran, Iran.

[40] Daniel, E.I.; Oshodi, O.S.; Nwankwo, N.; *et al* (2024). Barriers to the Application of Digital Technologies in Construction Health and Safety: A Systematic Review. Buildings 14:2386. https://doi.org/10.3390/buildings14082386.

[41] buildingSMART International (2020). openBIM - buildingSMART International. https://www.buildingsmart.org/about/openbim/. Accessed 20 Dec 2024.

[42] SEPA Cyber-attack | Beta | SEPA | Scottish Environment Protection Agency (2025). https://beta.sepa.scot/about-sepa/cyber-attack/. Accessed 2 Jan 2025.

[43] CEN (2020). BS EN 17412-1:2020 - Building Information Modelling - Level of Information Need - Part 1: Concepts and principles.

[44] Sousa, G.; Azenha, M.; Matos, J.; Brito, V. (2018). Implementação BIM no contexto de inspeção e gestão da manutenção de obras de arte em betão armado: proposta de metodologia e aplicação piloto. 2o Congresso Português BIM 519–528.

[45] Silva, A.; Galhardas, H.; Barateiro, J.; Matos, H. (2006). Data Analysis Features of the GEST BARRAGENS System. In: Second International Conference of Innovative Views of .NET Technologies. Florianópolis, Brazil.

[46] Castro, A.T.D.; Mata, J.; Barateiro, J.; Miranda, P. (2012). Information Management Systems for Dam Safety Control . The Portuguese Experience. 54° Congresso Brasileiro do Concreto 1–13.

[47] ICOLD (2025). ICOLD Dictionary. https://www.icold-cigb.org/GB/dictionary/dictionary.asp. Accessed 6 Jan 2025.

[48] ISO (2018). ISO 16739-1:2018 - Industry Foundation Classes (IFC) for data sharing in the construction and facility management industries — Part 1: Data schema. 1474–1474.

[49] Zhang, S.; Jiang, P. (2021). Implementation of BIM + WebGIS Based on Extended IFC and Batched 3D Tiles Data: An Application in RCC Gravity Dam for Republication of Design Change Model. KSCE Journal of Civil Engineering 1–20. https://doi.org/10.1007/s12205-021-0115-9.

[50] CT-197 (2021). Regras de modelação de objetos BIM. SECCLasS - Sustainability Enhanced Construction Classification System.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DONA FRANCISCA HPP CRISIS MANAGEMENT: EXTREME FLOOD EVENT RESPONSE

Giorgia Moecke[1]
Fractal Engenharia

Pedro Guilherme de Lara[2]
Fractal Engenharia

Maria Eduarda Fonseca[3]
Fractal Engenharia

ABSTRACT

During May 2024, the Southern region of Brazil was struck by heavy rains that triggered historic natural floods in the state of Rio Grande do Sul, leading to the loss of human lives and incalculable financial losses. In the area of the Dona Francisca Dam, three major flood peaks were recorded, with the first two approaching the dam's design flood (ten-thousand-year flow). In this scenario, the Dona Francisca Hydroelectric Plant (UHE Dona Francisca) activated its Emergency Action Plan (EAP), implementing communication with civil defense to safeguard the downstream population affected by the natural floods. During the plan's activation, a crisis committee was formed, involving key stakeholders such as dam safety officials, cascade agents, and the hydrology and meteorology team. The focused work of the crisis committee, along with the swift and effective action of regional and municipal civil defense, ensured the safety of the population located in the downstream valley, with no loss of life recorded in the area downstream of the dam, despite the extreme hydrological conditions faced. This article presents the challenges encountered during the EAP activation, the conditions of the event experienced, and highlights the importance of an effective EAP, the relationship with civil defense, and the training of the riverside community in building a resilient population capable of overcoming adversities like those experienced in 2024.

Keywords: Dam Safety, Flood, Crisis Committee, Emergency Action Plan, EAP.

[1] giorgia@fractaleng.com.br, Brazil
[2] pedro@fractaleng.com.br, Brazil
[3] mariaeduarda@fractaleng.com.br, Brazil

1. INTRODUCTION

Between April and May 2024, an extreme weather event struck the state of Rio Grande do Sul, located in southern Brazil. This event triggered severe flooding in several municipalities, leading to extensive material losses and, most tragically, the loss of 183 human lives.

One of the areas impacted by the flooding was the Dona Francisca Hydroelectric Power Plant (HPP), owned and operated by Dona Francisca Energética S.A. (DFESA). The plant is situated on the Jacuí River in the municipality of Nova Palma, Rio Grande do Sul. Given the severity of the situation, it became necessary to activate the plant's Emergency Action Plan (EAP) to safeguard the surrounding communities and infrastructure.

To manage the crisis, a crisis committee was established. This committee was tasked with assessing the evolving situation, discussing critical issues, and coordinating the actions required to address emerging challenges. The committee included representatives from DFESA, civil defense, and other key stakeholders responsible for ensuring a prompt and effective response.

The crisis committee faced numerous challenges as it worked to address the adversities caused by the extreme weather event. Despite the complexities and pressures of the situation, the activation of the Dona Francisca HPP’s EAP was ultimately regarded as a success. The population within the Self-Rescue Zone (SRZ) — the area immediately downstream of the dam where evacuation is self-directed — was protected, and no fatalities were recorded within the region.

The following sections outline the key aspects of the event, the challenges encountered during the activation of the EAP, and the issues identified in the process. Additionally, lessons learned from the experience are highlighted, offering valuable insights to improve future responses to extreme hydrological events.

2. DONA FRANCISCA DAM

The Dona Francisca Dam is a water retention structure built for hydroelectric power generation. The hydroelectric power plant (HPP) has a total installed capacity of 125 MW.

The dam is a gravity-type structure constructed using roller-compacted concrete (RCC), with sections of conventional concrete. It has a maximum height of 51 m, with the crest located at an elevation of 102.0 m. Under normal operating conditions, the reservoir has a storage capacity of 335 hm^3 and covers a flooded area of 19 km^2.

The spillway system consists of an uncontrolled (free crest) spillway, with a crest elevation of 94.50 m and a length of 335 m. The structure, designed with a Creager-type profile, has a

discharge capacity of up to 10,600 m³/s, corresponding to the 10,000-year flood. This event results in a reservoir surcharge of 5.8 m, raising the reservoir level to 100.30 m.

Fig. 1 – General view of the project (September 2020).

3. DOWNSTREAM VALLEY

The downstream valley of the Dona Francisca Dam is characterized by a vast floodplain, where large rice plantations extend along the riverbanks and small urban settlements, or villages, are scattered throughout. These settlements include the municipalities of Agudo, Dona Francisca, Nova Palma, Cachoeira do Sul, Candelária, Faxinal do Soturno, Formigueiro, Paraíso do Sul, Restinga Seca, São João Polêsine, and São Sepé, all located in the state of Rio Grande do Sul.

According to a cadastral survey conducted for the dam's Emergency Action Plan (EAP), approximately 9,000 residents would be affected if flooding occurred within the mapped extent of a 10,000-year return period event. According to the EAP, the entire region is served by the regional civil defense of Santa Maria, which also provides support to other municipalities on the eastern side of the state.

4. EXTREME WEATHER EVENT – MAY 2024

The extreme weather event that occurred in Rio Grande do Sul between April and May 2024 was driven by a unique combination of meteorological factors. These included the formation of an atmospheric block in the Pacific Ocean, a cold air mass to the south, and a robust mass of hot, dry air over central Brazil, which hindered the southward progression of the cold front. Together, these factors confined atmospheric instabilities over Rio Grande do Sul, concentrating weather systems that triggered thunderstorms and heavy rainfall across the state for several days.

A total of 478 municipalities in the state of Rio Grande do Sul were affected by floods, landslides, and slope failures. Approximately 2.4 million people experienced the impacts of the

intense rainfall across the state. Among those affected, over 442,000 residents were displaced from their homes, with around 18,000 seeking refuge in shelters.

The disaster also caused widespread disruptions to essential services. More than 640.000 residences experienced interruptions in water supply, while over 440,000 customers faced power outages. Landslides, flooding, road surface damage, dam breaches, and fallen trees resulted in blockages along dozens of points on state highways, further complicating rescue and recovery efforts.

This event was classified as the largest insurance loss caused by a single event in Brazilian history. By May 27th, claims submitted by policyholders had exceeded 1.6 billion Brazilian reais.

The impact of rainfall at the Dona Francisca Dam site and in the downstream valley can be observed by comparing Figure 2 with Figures 3 and 4.

Fig. 2 – Pictures of the Dam before the event, during dry season (May 2023).

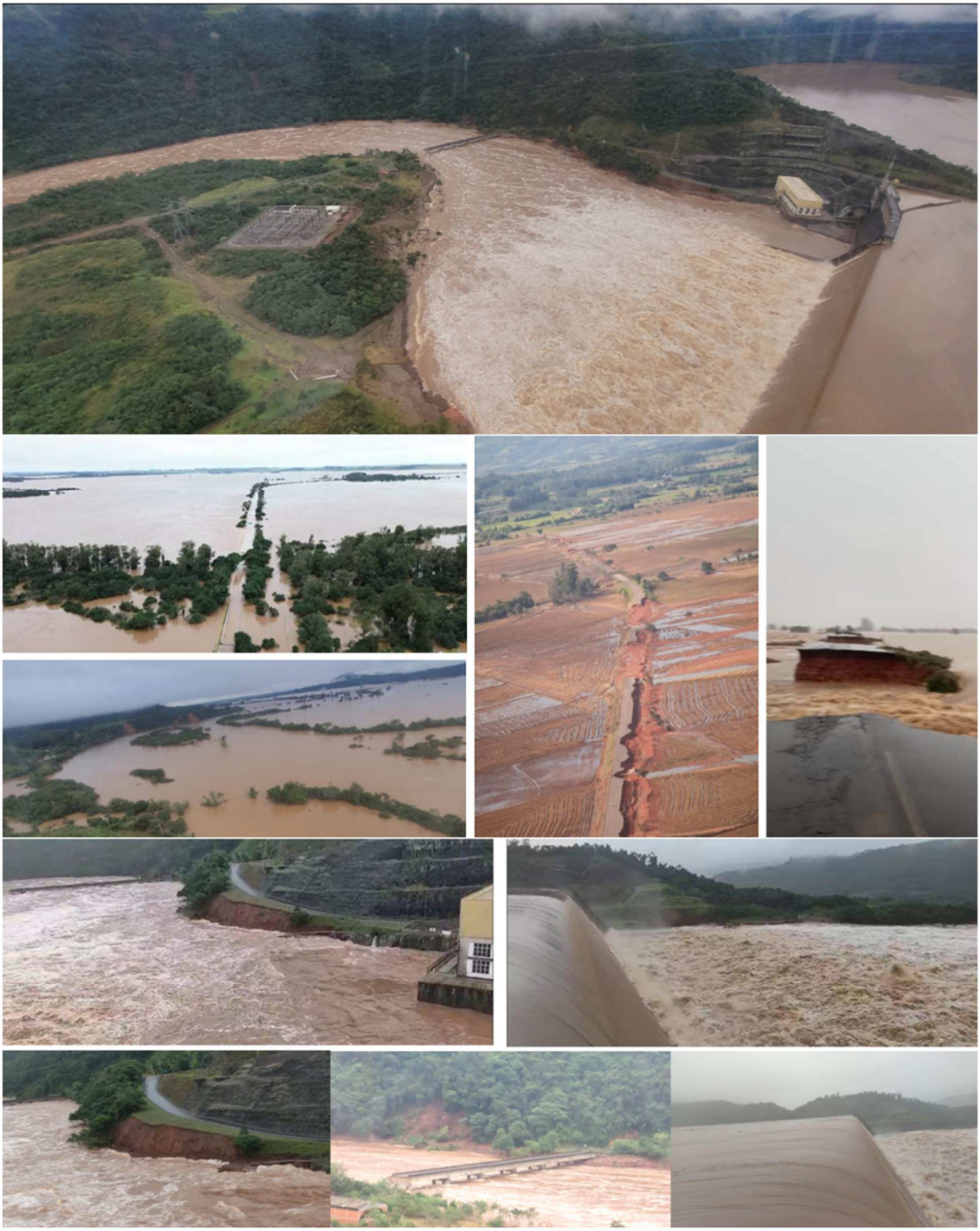


Fig. 3 – Pictures of the impact of the floods occurred in May 2024.

Fig. 4 – Pictures of the destruction caused by the floods in May 2024.

In the Jacuí River basin, upstream of the Dona Francisca Hydropower Plant (HPP), the majority rainfall occurred between April 29 and May 2, with 511 mm of rain recorded at the Arroio Carijinho station. Over a broader timeframe, from April 22 to May 5, the station registered a total of 738 mm rainfall. Nearly 70% of the total rainfall during the event fell within just four days in localized areas of the basin.

As a result, at the Dona Francisca Dam, the event caused an extraordinary rise in the reservoir's water level. On April 30, 2024, at the flood's peak, the reservoir level reached 100.40 meters, approaching the design elevation of 100.50 meters. This rise corresponds to a flood with a 10,000-year recurrence interval.

The water level rose by 4 meters in just 8 hours, shifting from a 2-year recurrence flood to a 10,000-year flood event. No meteorological model had predicted such an extreme impact on the basin.

Fig. 3 depicts the water levels recorded at the Dona Francisca HPP during this exceptional event.

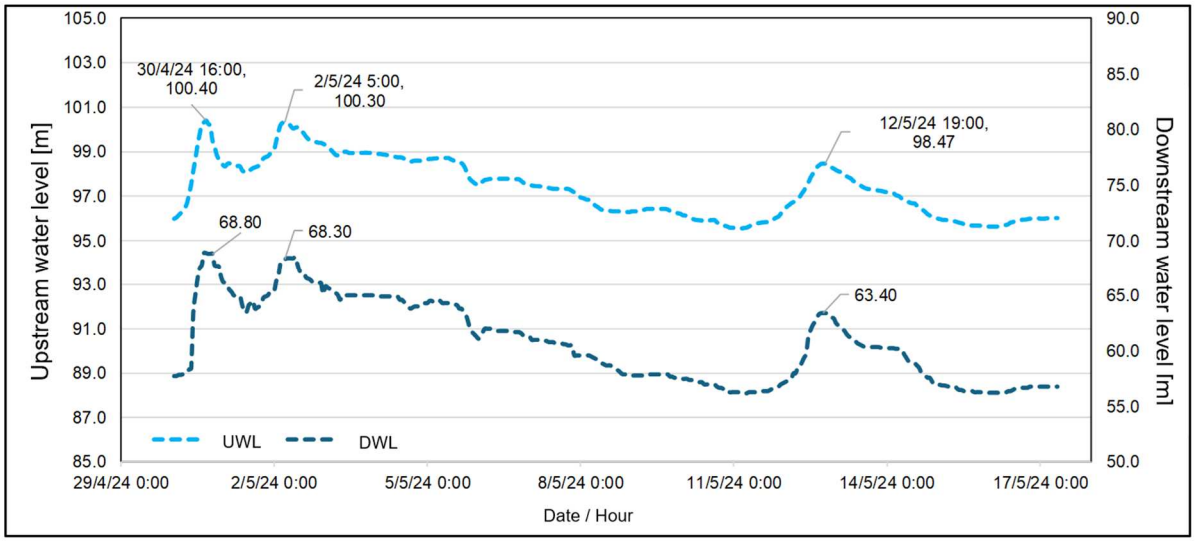


Fig. 5 – Upstream and downstream levels of The Dona Francisca Dam during the May 2024 event.

5. EVENT TIMELINE - TRIGGERING THE EAP

Due to the severity of the hydrological event and the rapid rise in reservoir water levels, the Emergency Action Plan (EAP) for the structure had to be activated to implement measures ensuring the safety of the dam, the workers on-site, and the downstream population.

On April 30, 2024, a crisis committee was established to coordinate critical actions during the activation of the EAP at the Dona Francisca Dam. The EAP required the establishment of an emergency room for crisis management whenever the dam reached response levels of Attention, Alert, or Emergency. The designated location for this command center, as outlined in the EAP, was in the city of Agudo, located downstream of the dam. However, due to the extreme hydrological conditions, an in-person meeting was not possible. Instead, a virtual emergency room was created, enabling the crisis committee to convene via an online platform. The meeting brought together key stakeholders, including dam safety personnel, cascade operation agents, and hydrology and meteorology experts.

The committee's immediate priorities included:

- Ensuring the structural integrity of the dam.
- Protecting workers operating at the Dona Francisca HPP.

- Safeguarding downstream communities from potential flood impacts.

The crisis committee was activated at the onset of the event, as water levels began to rise. During the first days of the event, continuous updates were provided on the structure's status, operations, and weather conditions, with meetings lasting throughout the day and extending into the early hours of the morning, discussing possible actions to ensure the safety of both the structure and the people. As the severity of the event decreased, meetings were held three to four times per day. However, the entire team remained on standby in case any situation escalated.

Since the beginning of the event, DFESA issued hydrological bulletins to civil defense authorities to keep them informed of anticipated conditions. This proactive measure provided civil defense with additional time to prepare and implement appropriate protective actions.

Despite the effective activation of the EAP, the extreme magnitude of the event exposed several challenges, both in operational response and in the documentation of emergency procedures.

On April 30, 2024, the Dona Francisca HPP lost all communication with external agents, and land access to the plant was cut off. As a result, operator shifts could not be rotated, leaving the on-site team isolated and on high alert due to rising flood levels.

- The reservoir and downstream water levels were continuously monitored, but instrumentation data on structural conditions remained unavailable, as the structure does not have automated instrument monitoring, and no personnel were on-site to perform the readings.
- A critical fiber optic cable to the substation was severed, disrupting communication between the plant and the substation.
- Diesel generators were activated to maintain auxiliary systems, but limited fuel supply posed a significant risk, as refueling was impossible by air or land due to inaccessibility.
- Generating Unit 1 (UG1) experienced a malfunction, leading to the closure of its floodgate.

As the situation escalated, the crisis committee raised the dam's response level to Attention, the first of three possible response levels (Attention, Alert and Emergency), due to the loss of communication and access, the inability to obtain instrumentation readings, and the lack of concrete information regarding the dam's conditions.

On May 1, 2024, the crisis committee, in collaboration with civil defense authorities, the Brazilian Army, and the Brazilian Air Force, secured a difficult land access route. Through this route a small quantity of diesel was delivered to the plant. Simultaneously, an army helicopter

managed to approach the site, despite adverse weather conditions, enabling a new team to take over operations, inspect instruments, and monitor the structure. The previously isolated employees were evacuated during the same flight.

On May 2, 2024, the dam experienced another flood peak, with flows approaching a 10,000-year return period magnitude. This event required the immediate refueling of diesel generators at the substation. Operators had to navigate the drainage gallery and partially flooded downstream sections to perform essential maneuvers, ensuring the continued operation of critical systems.

Instrumentation readings taken on May 2 revealed increased pressure levels within the dam, with measurements indicating rises proportional to the upstream and downstream water level elevations observed during the event. Three instruments reached Attention levels but remained well below the Alert thresholds established in the design.

This condition, combined with the forecast of additional rainfall in the basin and the continued lack of access to the site, led the crisis committee to elevate the dam’s response level to Alert.

In the days that followed, daily instrumentation campaigns were initiated to monitor evolving conditions and structural responses to the ongoing hydrological event.

Data from these campaigns, along with operational insights, were reviewed in crisis committee meetings to assess the structure’s status and determine appropriate response levels. Rainfall forecasts were also analyzed daily to inform the planning of subsequent measures. This systematic approach ensured comprehensive oversight and facilitated timely decisions during the critical period.

On May 5, the dam's response level was downgraded to Attention, and on May 8, to Normal as rainfall diminished and reservoir water levels began to recede.

On May 12, another flood peak occurred, though it was less severe than the previous two. At this time, Generating Unit 1 (UG1) remained inoperative, and communication and access to the substation were still unavailable.

On the same day, preparations began for connection adjustments to perform the reverse energization maneuver at the plant. Due to the operating conditions at the Dona Francisca HPP, combined with rainfall and flow forecasts, the dam's response level was once again elevated to Alert.

In the days that followed, rainfall in the region decreased again, leading to a further reduction in reservoir levels. On May 20, communication between the plant and the substation was restored, and the cleaning of UG1's gate was completed. On the same day, UG1 was successfully synchronized and brought back online. By May 24, the dam's response level was

reduced again to Attention, and by June 1, regular shift changes and diesel supply to the plant were re-established. On July 5, land access to the plant was finally restored.

With the normalization of dam operations, the crisis room was officially closed on August 8, 2024.

Throughout the event, the crisis committee held daily meetings and maintained close coordination with the National Electric System Operator (ONS) and the National Electric Energy Agency (ANEEL), both of which provided essential support.

The Emergency Action Plan (EAP) was activated at the Alert level three times, triggering special safety inspections under Law No. 14.066/2020 and ANEEL Resolution No. 1,064/2023. These inspections confirmed that, despite significant damage to surrounding areas, the dam's civil structures remained intact and safe.

The successful management of this extreme weather event was largely attributed to:

- The existence of a robust EAP and the preparedness of the team responsible for its activation.
- The relentless efforts of the crisis committee and the swift response of regional and municipal civil defense teams.
- The structural resilience of the dam, which withstood flood magnitudes comparable to the first and second peaks.

It is important to note that at no time during the critical event did the Dona Francisca Dam present a structural risk that could lead to failure or more severe consequences — only a natural response to an event that subjected the structure to its design load, as evidenced by the three special inspections conducted in May.

From the onset of the event, the downstream population was notified and evacuated by civil defense authorities due to the occurrence of natural large-magnitude floods. The alert notifications issued by DFESA were always based on flow forecasts and reservoir water level projections, rather than any anomalies or structural issues with the dam.

It is worth mentioning that the local civil defense had access to flood maps of recurring natural floods, provided by DFESA, through a mobile application that also served to issue flood notifications. Additionally, public alerts had been issued on previous occasions, albeit for floods of lesser magnitude. As a result, both civil defense authorities and the local population were already familiar with the necessary procedures to follow during flood events.

Despite the challenges, no lives were lost in the region as a result of the flooding.

This outcome highlights the critical importance of proactive planning, continuous training, and collaboration among agencies during extreme hydrological events.

5.1. LESSONS LEARNED

The Emergency Action Plan (EAP) is a critical document designed to guide decision-making and establish a clear sequence of actions during a crisis. Its development requires comprehensive analysis, alignment between the project developer and civil defense authorities, and meticulous attention to detail. Despite its importance, the EAP is rarely activated, as emergency situations at most hydropower facilities are uncommon.

However, during the May 2024 crisis, the EAP for the Dona Francisca Dam had to be activated, providing a real-world test of its effectiveness and uncovering valuable lessons.

One of the most important lessons learned was the need for prior coordination between the dam owner and civil defense authorities. This alignment ensures that:

- Each crisis situation is properly assessed.
- The appropriate response actions are identified.
- Responsibilities for each task are clearly defined.

During the crisis, challenges highlighted areas where the EAP could be improved. As a result, a revised version of the plan was developed, focusing on clarity, conciseness, and simplicity to enable faster and more confident decision-making under emergency conditions.

Beyond refining EAP documentation, the crisis emphasized the importance of clearly defining roles and responsibilities in decision-making.

Regular and direct communication with municipal civil defense teams proved critical. By sharing forecasts for upcoming hours and providing timely updates on the evolving situation, civil defense authorities were able to act proactively. This open line of communication facilitated the protection and, when necessary, the safe evacuation of the affected population, with no fatalities reported as a result of the incident.

A major enhancement to the EAP was the inclusion of protocols for hydrological risk scenarios, which had previously focused primarily on structural risks. This update introduced:

- A procedure for issuing flood level bulletins to civil defense authorities.
- Improved forecasting methods to enhance emergency preparedness.

These updates ensure that civil defense teams can anticipate and respond to potential flood scenarios more effectively, improving overall emergency management.

Crisis committee meetings also highlighted the need for continuous dialogue between hydropower cascade operators to improve coordination during extreme flood events.

By evaluating potential flood scenarios collectively, stakeholders can develop a unified response strategy that better protects downstream populations and infrastructure.

The May 2024 event reinforced the need for a well-structured and regularly updated EAP. Additionally, it underscored the importance of ongoing training for crisis response teams to ensure preparedness and clear communication and coordination among all involved parties to facilitate an effective emergency response.

This event served as a critical reminder that preparedness, coordination, and adaptability are key to successfully managing crisis situations. By applying these lessons, future emergency responses can be even more effective, minimizing risks to infrastructure, workers, and downstream communities.

6. CONCLUSIONS

The extreme weather event that struck southern Brazil in May 2024 was a catastrophic incident, marked by an intensity and scale that exceeded prior forecasts. Tragically, the event resulted in significant loss of life across the affected areas.

In response to the crisis, the Dona Francisca HPP, located along the Jacuí River within the flood-impacted region, was required to activate its Emergency Action Plan (EAP). This activation was carried out with the support of the crisis committee, which played a crucial role in assisting the dam owner and civil defense teams. However, the process presented a range of challenges for all parties involved.

The event served as a learning experience, revealing critical insights into crisis management and emergency response. Several opportunities for improvement were identified, prompting revisions to the EAP document and adjustments to the planned sequence of actions. The goal of these enhancements was to create a more effective, streamlined, and user-friendly plan, ensuring that future activations could be executed more efficiently.

The use of the EAP during this extreme event underscored the value of having a well-structured, clearly defined emergency plan. It also highlighted the need for continuous training of the agents responsible for activating and executing the plan. Proper training enables faster and more decisive response actions during high-pressure crisis situations.

Despite some challenges and limitations identified in the EAP document used at the time, its activation was deemed a success. Thanks to effective coordination, clear communication, and timely action, the affected population was safeguarded and rescued from danger. No loss of life was recorded as a direct result of the extreme hydrological conditions downstream of the dam. This outcome demonstrated the vital role of preparation, alignment, and collaboration in mitigating the impacts of extreme weather events.

REFERENCES

[1] ABRAGE – ASSOCIAÇÃO BRASILEIRA DAS EMPRESAS GERADORAS DE ENERGIA ELÉTRICA. Orientações para elaboração do PAE das barragens de usinas hidrelétricas – v3.0. Belo Horizonte, Outubro, 2017.

[2] AGÊNCIA NACIONAL DE ENERGIA ELÉTRICA (ANEEL). Resolução nº 1.064, de 02 de maio de 2023.

[3] Lei Federal n° 14.066 de 30 de Setembro de 2020. Brasília, 2020.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

GISTM REQUIREMENTS CONFORMANCE AND THE INFLUENCE OF THE KEY ROLES FOR THE TAILINGS GOVERNANCE FRAMEWORK: A STATISTIC APPROACH

Borges, Marina [1]
Federal University of Ouro Preto (Brasil), Brazil

Menezes, Danielle [2]
Federal University of Ouro Preto (Brasil), Brazil

Delgado, Bruno [3]
Federal University of Ouro Preto (Brasil), Brazil

Mota, Hernani [4]
Federal University of Ouro Preto (Brasil), Brazil

ABSTRACT

The Global Industry Standard on Tailings Management (GISTM), launched in 2020, was developed by means of a partnership between the International Council on Mining and Metals (ICMM), the United Nations Environment Program (UNEP) and the Principles for Responsible Investment (PRI). That standard was created in response to tailings dam disasters such as the failure of Brumadinho tailings dam in Brazil in 2019. It sets forth 77 requirements with guidelines on tailings dam management focusing on safety, transparency, governance and responsibility.

Governance in the context of tailings dams risk management refers to the set of policies, regulations and practices guiding decision making and the supervision of those facilities. This paper presents an assessment and a correlation, using multivariate statistics techniques to analyze compliance with GISTM requirements regarding key roles in the Tailings Governance Framework. The check was performed according to the responsibilities assigned by the GISTM to the following positions: Responsible Tailings Facility Engineer (RTFE), Engineer of Record (EOR), Independent Technical Review Board (ITRB or ISTR) and Accountable Executive (AE).

The methodology adopted in this study counted on the Conformance Protocol (ICMM, 2020) to set forth a GAP analysis and check conformity with the GISTM. A databank was created based on 35 Tailings Storage Facilities (TSF) belonging to 7 mining companies in 5 countries. A semiquantitative analysis was performed based on that set of data to review the tailings management system and the influence of the key governance roles on improving process efficiency.

As a result, the research presents a comparative assessment of the main roles in the Tailings Governance Framework. This study offers insights on the GISTM implementation and shows the strong value of the organizational structure to meet the Standard conformity to reduce TSF failure risks and, consequently, to avoid catastrophic impacts both on the communities and the environment.

Keywords: Tailings storage facility, Tailings governance, Responsibility and Accountability.

[1]marina.bp@aluno.ufop.edu.br, Brazil

1. INTRODUCTION

In the past years, the mining company has been facing a significant increase on concerns related to the safety of tailings dams after accidents such as the dam failures in Mariana (2015) and Brumadinho (2019) in Brazil. Such events resulted in profound environmental and social impacts as well as increased pressure on companies and regulatory authorities for the implementation of best governance and risk management practices for tailings dams. In response to those failures, the Global Industry Standard for Tailings Management (GISTM) was published. It set forth international guidelines for the safe and transparent management of Tailings Storage Facilities (TSF) with a focus on the reduction of risks and the protection of affected communities.

The GISTM consists of 77 requirements organized into six main topics, one of which is Management and Governance. Within the context of tailings management, a governance system refers to a set of policies, standards and practices guiding decision making and tailings dams supervision [3].

In that sense, the GISTM introduces several key governance roles to strengthen risk technical and corporate management [4]. Those functions are also conceived to ensure appropriate supervision, independent technical review and accountability, thus minimizing the risk of failures with catastrophic impacts.

The most relevant key governance roles are described below:

•**Responsible Tailings Facility Engineer (RTFE):** an engineer appointed by the Operator as the person responsible for TSF. The RTFE should be permanently available during the construction, operation and closure phases of a TSF. The RTFE has a clearly defined responsibility delegated by the tailings storage facilities management and has appropriate qualifications and experience compatible with a TSF. The RTFE is responsible for all the activities related to a TSFand for its budget, as well as for the associated risk management. The RTFE can delegate specific tasks and responsibilities regarding tailings management to qualified employees, except accountability concerning the role [3].

•**Engineer of Record (EOR):** a qualified engineering company responsible for confirming that the tailings storage structure was designed, built and deactivated with due attention to facility integrity and that it meets and follows the applicable legislation, statutes, guidelines, codes and standards. The Engineer of Record can delegate responsibilities except accountability and responsibility over its function. In some highly regulated jurisdictions, especially in Japan, the role of the EoR is played by the regulatory authorities in charge [3].

•Independent Technical Review Board (ITRB): a board that is technically responsible for reviewing the design, construction, operation, closure and management of tailings storage facilities. Independent reviewers are third parties who are not and have never been directly involved in the design or the operation of the tailings disposal facility at stake. The experience of ITRB members must cover the entire range of issues that are relevant to the facility, its context and the complexity of those issues. In some highly regulated jurisdictions, especially in Japan, the role of the ITRB is played by the regulatory authorities in charge [3].

•Accountable Executive (AE): One or more executives directly subordinated to the executive director in regard to matters related to this standard and having a communication channel with the Board of Directors, and who are responsible for the tailings storage facilities safety and for minimizing the social and environmental consequences of an eventual failure of the TSF. The Accountable Executive can delegate responsibilities except accountability and responsibility over its function [3].

•Board of Directors (BoD): the Operator's highest governance body, whose members are usually elected by the Operator's shareholders. The Board of Directors is the Operator's body having final decision authority to define policies, objectives and the overall directions, and to supervise the company executives, among other tasks. On the purposes of the Standard, the term encompasses any individual or body having control over the Operator, including the company's holder or holders. When the State acts as an Operator, the term Board of Directors refers to the government manager having the highest responsibility for the Operator's final decisions [3].

The main purpose of establishing those functions is to guarantee that all phases of a tailings facility lifespan — design, construction, operation, closure and post closure – are conducted according to the highest level of safety and risk minimization. Moreover, they aim to foster tailings management transparency and to promote engagement and respect for communities located near operations.

This paper aims to present an assessment of the correlation between those key roles and the requirements set forth by the GISTM, evaluating how the implementation of those functions contributes for risk mitigation and for improving governance practices in the sector.

2. METHODOLOGY

This study aims to assess the impact of governance functions on compliance with GISTM requirements. To evidence that relationship, a detailed analysis of how those functions interact within the GISTM structure and how they contribute to mitigate the risks associated with the tailings dams was performed.

The Tailings Governance Framework [6] provided a base to structure GISTM requirements and measure the efficiency of governance and tailings management processes. The main purpose is to develop a methodology to comparatively analyze those storage facilities and their perspectives by using GISTM requirements as the assessment framework.

The research was conducted in three phases. Phase 1 involved problem analysis and development of the analysis framework. Phase 2 focused on data analysis while Phase 3 focused on study interpretation.

Phase 1 aimed at setting forth a correlation between the GISTM [3] and the main functions defined by the Tailings Governance Framework [6].

GISTM Conformance Protocols [5] were used to identify key parameters and group relevant variables to assess the TSF. That process involved analyzing, organizing and filtering conformance criteria to make sure assessments are consistent and eliminate redundancies.

Since the GISTM's main purpose is to mitigate risks regarding TSFs [1], this study incorporates the concept of risk to check effectiveness of the main tailings governance tasks proposed by the GISTM to control risk factors influencing both the probability and the consequence.

Phase 2 used Menezes et al. (2024) [7] GAP analysis system, which was designed to quantify and evaluate GISTM requirements. Questions in the GAP analysis derived from GISTM protocols [5] and aligned to the main tailings governance principles based on risk and applied to 35 TSFs belonging to seven mining companies in five countries.

The GAP analysis limited the answers to "Yes", "No" or "Non applicable (NA)" to ensure clarity and to avoid subjective interpretation. That allowed for a semiquantitative check. Answers "Yes" and "NA" were equally pondered to avoid penalizing non-applicable questions, while "No" indicated non-compliance or ongoing efforts to comply with the requirements. The option "partially compliant" was excluded to eliminate ambiguity, making sure that incomplete actions were ranked as "No" for a consistent and objective assessment, as described by Menezes et al. [7].

The gap analysis form included 271 objective questions that were later answered by representatives appointed by the mining companies to check compliance with the requirement. For this study, the questions were ranked according to their influence on the consequence and the probability.

The influence class is related to the topic the question aims to assess. For example, a question verifying technical problems that may improve the facility safety in its lifespan cycle such as geotechnical, operational and governance controls is directly related to the probability of tailings facility failure and, therefore, is ranked as a Probability.

In turn, questions assessing controls related to the potentially affected area, community engagement, environmental impact and emergency preparedness and response plans to reduce the impact of an eventual TSF failure are ranked into Consequence.

Moreover, the questions were grouped according to the main governance roles of the RTFE, EoR, ITRB, AE, BoD and IA, as pointed out by GISTM Conformance Protocols [5] and detailed in the Tailings Governance Framework [6]. That intends to provide a structure appropriately reflecting TSFs governance system and risk management, together with its responsibilities and accountabilities.

In phase 3, TSFs scores recorded in Menezes et al. [7] set of data were standardized to a base 10 to ease result interpretation. A comparative analysis was conducted between GISTM Principles and the influence of key roles to assess tailings management systems effectiveness. That analysis focused on the interactions between RTFE, EoR, ITRB, AE, BoD and IA roles, examining how those roles collectively contributed to meet GISTM requirements and reduce TSF risks.

3. RESULTS AND DISCUSSIONS

3.1 Problem Assessment and Structure Development

On the purposes of this research, a case study was conducted using a gap analysis form. That form was originally developed by Menezes et al. [7] and then expanded as part of his research. The focus of this study was to check TSF compliance with GISTM requirements for Tailings Storage Facilities (TFs).

Figure 1 shows the percentage representativeness of the questions according to the influence class by key role. The number of questions representing governance controls that may influence the consequence is 13%, which is very low compared to the Probability (87%).

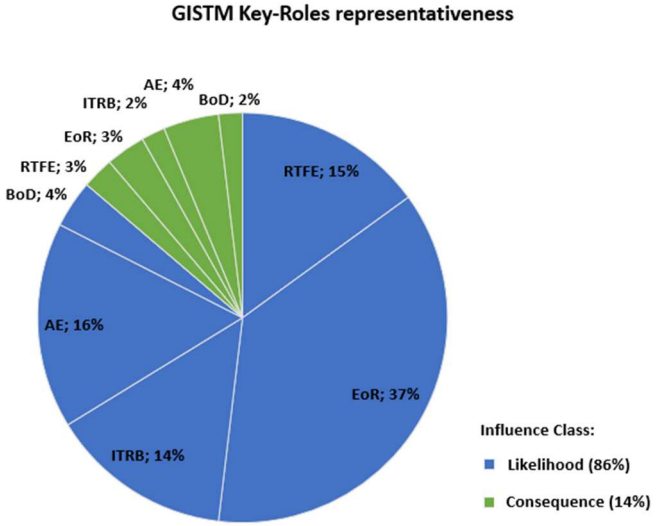


Fig 1 – GISTM Key-holes representativeness by influence class.

Considering that almost 50% of the TSFs registered in the set of data are of the Upstream-Dam type, those built on tailings placed, and the risk is directly connected to credible failure modes, that kind of structure should be addressed with emphasis in terms of controls for the Consequence influence class.

Fig. 2 shows the TSF percentage representation according to the construction method.

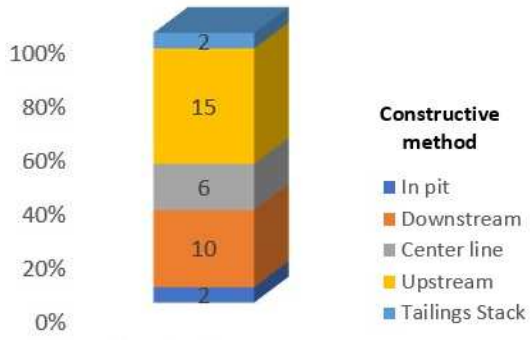


Fig 2 – Upstream Dam at Dataset.

3.2 Interpretation of the case study

The level of conformity of TSFs registered by Menezes et al. (2024) [7] was standardized for base 10 to ease result interpretation.

The discoveries provide insights on the importance of a governance structure that is well defined to ensure tailings dams safety. ***Fig.*** *3 shows the outliers for the tailings facilities under assessment. This exercise aims to improve understanding of the representation and the influence of key roles for GISTM Principles to check conformity.*

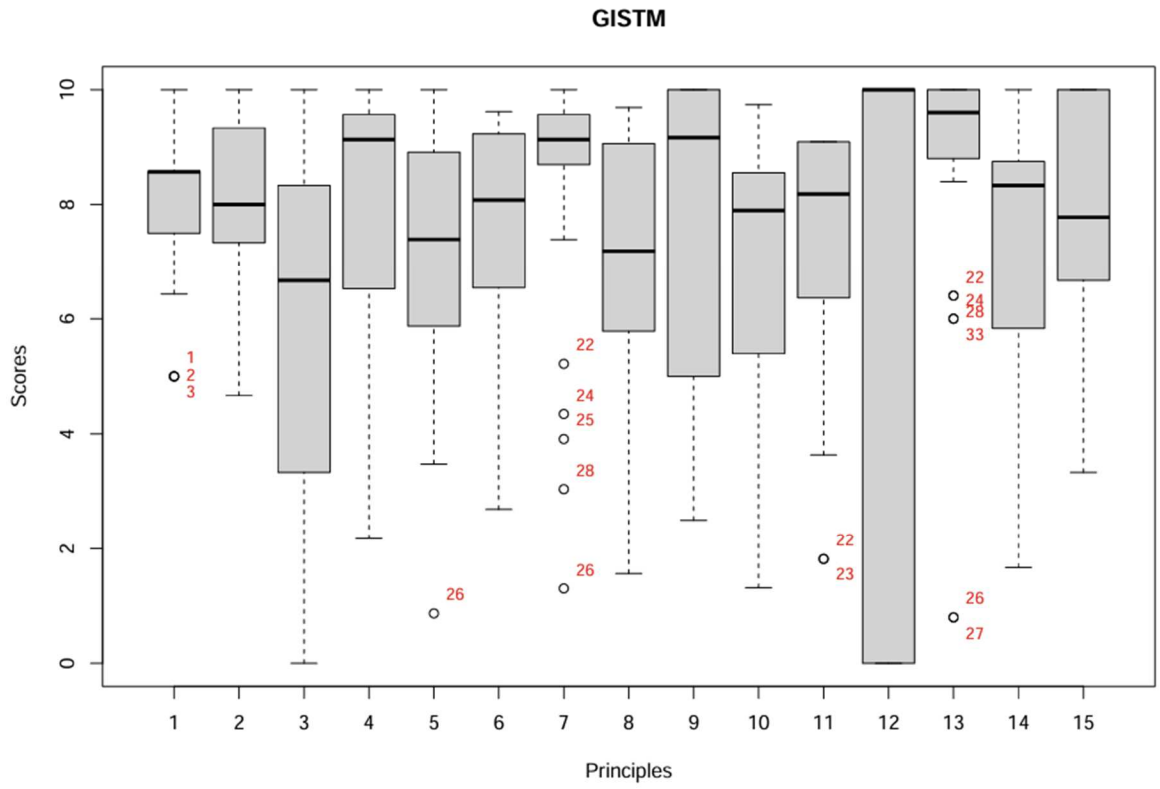


Figure 3 – GISTM Principles conformance boxplot (Menezes et al., 2024).

The comparative analysis between GISTM Principles and the influence of key roles was performed to check tailings management systems effectiveness. This analysis concentrated on the interactions between RTFE, EoR, ITRB, AE and BoD roles. The results can be seen on ***Fig.* 4**, which presents outliers in the analysis.

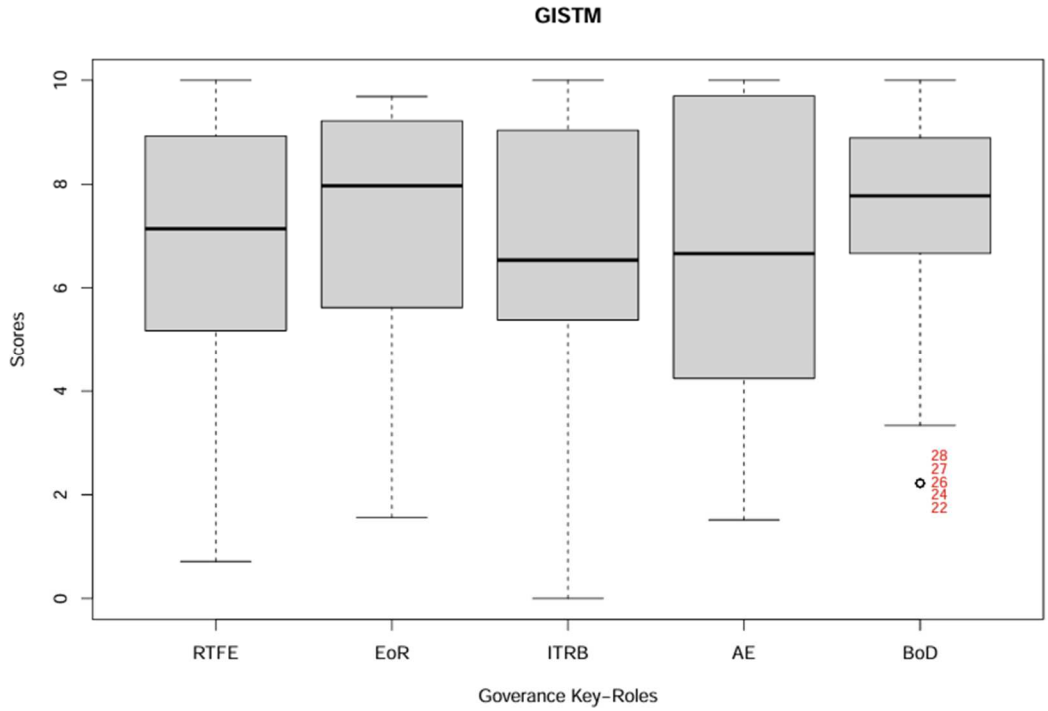


Fig 4 – GISTM Key-Roles conformance boxplot.

Considering that the risk is directly connected to the credible failure mode and the upstream method is not only related to most of the events but is also very important in case of a flow failure, since 40% of those accidents evolved to liquefied tailings release and 88% of flow failures under the construction method described above were upstream dams [2].

Figure 5 shows the average score of the construction method separated by Influence Class: Probability.

(a) and Influence Class: Consequence (b).

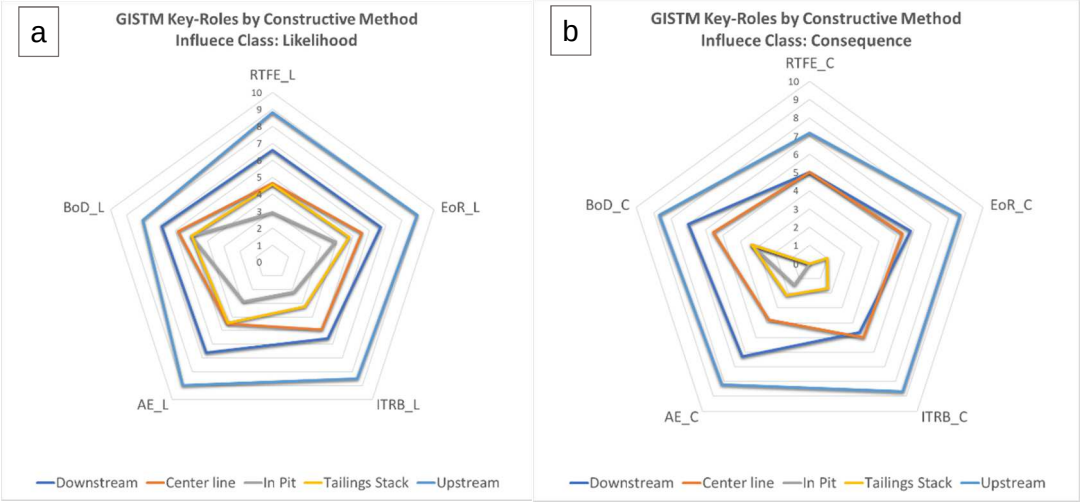


Fig 5 – GISTM Key-Roles by Constructive Method: (a) Influence Class: Likelihood; (b) Influence Class: Consequence.

4. CONCLUSIONS

The results of the statistical analysis and the comparative assessment showed relevant correlation between effective governance for key roles and the level of compliance with GISTM requirements.

In fact, if governance procedures are inserted into the company cultures, GISTM results in a control for risk mitigation and, consequently, the Tailings Management System will show good performance.

Governance maturity can be seen concerning the upstream construction method.

There is an opportunity to improve governance performance in controls related to the consequences of an eventual TSF failure, especially in case of flow failure.

In general, key tailings governance roles have greater influence on controls related to influence class Probability. That shows strong commitment, intrinsic to GISTM requirements, to improving best tailings management practices throughout the industry,

addressing technical, operational and governance aspects related to TSFs safety, in line with the ICMM [3].

Implementation of well-defined governance roles within the context of the TSF is essential to improve tailings facility management and, consequently, to prevent accidents and safeguard lives. They perform vital functions to guarantee tailings management operations integrity by reducing risks and ensuring compliance with environmental and safety standards.

Interaction and collaboration among governance functions contribute to the creation of a robust tailings management system considering both facility safety and environmental impacts. Moreover, the effective implementation of those functions strengthens transparency and reliability in the sector by regulatory agencies and local communities.

To ensure responsible and safe tailings facility management with the support of a solid governance system may be crucial to avoid tragedies resulting from tailings facilities failure and to guarantee sustainable and socially responsible operation continuity in what concerns the mining industry.

5. REFERENCES

[1]GTR, 2020. Global Industry Standard on Tailings Management, GTR. ed. London.

[2]Halabi, A.T.M., Siacara, A.T., Sakano, V.K., Pileggi, R.G., Futai, M.M., 2022. CASE HISTORY-PEER-REVIEWED Tailings Dam Failures: A Historical Analysis of the Risk.

[[3]ICMM, 2020. Global Industry Standard on Tailings Management, ICMM. ed. ICMM, London.

[4]ICMM, 2021a. Tailings Management: Good practice guide, ICMM. ed. ICMM, London.

[5]ICMM, 2021b. Conformance Protocols: Global Industry Standard on Tailings Management, ICMM. ed. ICMM, London.

[6]ICMM, 2016. Position Statement Tailings Governance Framework.

[7]Menezes, D., Santos, T., Lima, H., 2024. Analysis of GISTM requirements as failure risk controls for tailings storage facilities. Tailing Conference.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

GENERAL CRITERIA FOR IMPLEMENTATION OF DAM SAFETY MANAGEMENT SYSTEM

Luiz Sergio Sieber Padilla[1]
Eletrobras Chesf

ABSTRACT

Considering the technological advancement of information systems, within a connected world, in which instant and secure information is sought, the engineering that takes care of dam safety needs to modernize with the implementation of structured database systems to better manage these structures. The preparation of this process depends on some precautions that must be taken before implementing an SGSB (Dam Safe Manage System), for a better transition and ensuring better benefit from the solution adopted. The definition of both the preparation and the functional technical requirements need to be well defined and are covered in this article. This work aims to describe and evaluate the necessary measures before, during and after the implementation of a dam safety management system, as well as guide the general criteria necessary for a system to be developed or contracted, using the data science.

Keywords: database, dam security, criteria, implementation, system

[1] lsieber@eletrobras.com, Brazil

1. INTRODUCTION

The conception, design, construction and operation of dams is a multidisciplinary process, due to the technical complexity that involves this type of project, in all stages from planning, construction to operation and maintenance. Monitoring, data management, plays an important role in all these phases, ensuring that information is collected, transformed into engineering parameters, analyzed and used in a timely, efficient and safe manner, as well as ensuring that appropriate preventive and corrective measures are taken to minimize risks and protect communities as well as the environment, maintaining history and complying with DSP (Dam Safety Plan), which is the entrepreneur's obligation to implement.

In view of the growing need for control and mitigation of the risk associated with Brazilian dams and the difficulty observed in obtaining, processing and analyzing data on the design and execution of dams built in past decades, the description of the development of a dam monitoring and safety system is presented. Regular inspection of dams is extremely important to monitor the useful life and check for potential problems in the structures and propose improvements and maintenance, whether predictive, corrective or preventive maintenance. Dam safety management is important because it can directly avoid and/or reduce the consequences of damage from an eventual incident and/or accident caused. The safety of the neighboring community and water security for public supply are paramount. Considering water security in times of flood and drought, in multi-use dams, especially those for public supply, it is essential to monitor the reservoir levels in correspondence to the stored volume, the values measured by the instrumentation, as well as the chemical characteristics of the water, monitoring irregular occupation in the surroundings and erosion and sedimentation of the reservoir (LOBO, A.V.L, GRAEFF, 2018).

According to LOBO, PENA, BELLINI, *et al.* (2023), the results to be achieved provide access to information in a quick and interactive way, as well as preliminary analyses that direct the subsequent evaluations carried out by the specialists, enabling agile decision-making, ensuring the maintenance and safety of the project. The optimization of the analysis of structures through computational tools, together with the integration of monitoring modules, is a fundamental part of the constant development of companies. These efforts aim to allow the improvement of the tools used by the entrepreneur and/or company and facilitate the obtaining of accurate and timely information. These advanced approaches allow preliminary analyses to be conducted, providing valuable guidance for the more detailed analyses performed by the experts. These initiatives aim to allow for more efficient auscultation, providing valuable information to ensure the safety and integrity of structures. Through these continuous

advances, Sanepar maintains its position at the forefront of technology, constantly seeking to improve its practices and processes.

Before the emergence and popularization of computers, the monitoring and data management of dams relied mainly on paper records, which presented several challenges and risks, including the loss of information. Some reasons why the loss of paper information:

- Brittleness and deterioration: Paper records are subject to deterioration over time due to factors such as exposure to sunlight, moisture, insects, and rodents. This can result in the gradual loss of important information as documents wear out or deteriorate.
- Limited physical space: Storing large amounts of paper documents requires a lot of physical space. Businesses and organizations often faced the challenge of finding adequate space to store a large amount of paper and records of data and documents, which could lead to the loss of documents due to poor organization or accidental disposal.
- Difficulty of retrieval: Locating specific information in paper records can be a time-consuming and error-prone task. Without an integrated, effective indexing and organization system, documents can get lost in disorganized files or be mistakenly archived, making it difficult to retrieve them when needed.
- Vulnerability to disasters: Paper documents are highly vulnerable to natural disasters such as fires, floods, and earthquakes. The loss of documents in these situations could result in the irreparable loss of information that is essential to an organization's operations.
- Limitations on sharing and collaboration: With paper records, sharing information between different departments or geographic locations was challenging and often involved physical copies, which could be lost or damaged in the process.
- Risk of human error: The manual manipulation and recording of paper data, or important technical documents, increased the risk of human errors, such as: incorrect data entry, accidental loss of documents or failures in the transcription of information.

Even with all this inconvenience of managing technical documentation, paper data, that is, not digital, most dams in Brazil had to be managed in this way in the past. It turns out that this way of managing dam safety engineering has been digitized and computerized, bringing many gains to management, monitoring, inspection, operation, maintenance, information sharing and safety of the projects.

Modernity is guided by data science, where there is a modern and powerful approach to managing information effectively. It focuses on extracting valuable *insights* and making data-

driven decisions by utilizing statistical methods, machine learning algorithms, and data visualization techniques.

The implementation of a dam safety management system is challenging in terms of the preparation of data and other important technical information for importation, as well as the criteria for its contracting or development. Data needs to be organized and tabulated so that a structured database system can recognize it. In addition, this data needs to be verified, validated, and structured so that there are no import errors, so that the data is converted into information.

The benefits of a dam management system go far beyond storing data. Some of the main advantages in structured data are:

- Efficient organization: Structured data is organized in a defined format with predefined fields, which makes it easy to organize and store the data efficiently.
- Ease of query: Because the data is organized into predefined fields, queries and searches can be done quickly and efficiently, allowing for quick retrieval of specific information.
- Data consistency: The defined structure of the data ensures consistency in the formats and types of data used, which reduces errors and inconsistencies in the data.
- Ease of analysis: Structured, parameter-transformed, and integrated data is easier to analyze and process than unstructured data, making data analysis more accessible and efficient.
- Simplified integration: Structured data is easier to integrate with existing systems and tools, facilitating interoperability between different systems and applications, as well as allowing for web (cloud) storage.
- Scalability: Structured data is more scalable, which means it can better handle large volumes of data and grow as needed without compromising performance.
- Security: Data structuring makes it easy to implement security measures, such as access control and encryption, to protect data from unauthorized access.
- Standardization: Structured data often follows well-defined standards and conventions, which facilitate communication and data sharing between different systems and organizations.

Dam safety is a complex process that involves a series of steps from its conception, design, construction, operation, monitoring, inspections and maintenance of the dam and its associated structures, which must be accompanied by a multidisciplinary team and dam

specialists, throughout the useful life of the project and its associated structures such as spillways, water intake, bottom unloader and others.

2. PREPARATION FOR DEPLOYMENT

The Control and Monitoring System (CMS) and the Database Management System (DBMS) are software, developed by the dam safety team, responsible for managing, monitoring, storing, organizing and efficiently retrieving data in a database. The system offers an interface between users and the database, allowing users to perform operations such as inserting, updating, deleting, and querying data easily and securely.

Database Management Systems (DBMS) and/or Control and Monitoring Systems (CMS) often also provide features to ensure data integrity, consistency, and security, such as transaction mechanisms, concurrency control, access control, and backup/recovery.

Before implementing a Database Management System, it is essential to have the data collected, whether it is data from instrumentation readings, inspection records, or data from the Emergency Action Plan (EAP) – escape routes, sirens, individuals, guardians – or other data considered important should preferably be stored in electronic spreadsheets in order to facilitate the migration process to a structured database transformed into engineering variables. Even old data, which is stored on paper, must be migrated to electronic spreadsheets.

Importing the data is part of this process of making the data as consistent as possible. Searching for typos, calculation errors, checking if the data types are compatible with the system that will receive them, checking the column names, data type, transformation formulas are some of the necessary tasks.

DBMS work with several tables that relate to each other to make the data storage as structured as possible. This is intended to compartmentalize each type of data into dimension tables and fact tables. To better exemplify, a table for registering instruments would behave like a dimensional table. On the other hand, the table of the registration of the instrumentation readings would behave as a fact table, since it plays a dynamic role in the data structure by using the data from the registration table of the instruments.

Table 1 - Example of a cadastral table of auscultation instruments

ID	Identifier	Structure	Status	Instrument Type	Instrument subtype	It's automated
28987	PZ-01	Earth Dam	Active	Piezometer	Standpipe Piezometer	No
29666	MV-32	Earth Dam	Active	Seepage measurement	Triangular Seepage measurement	No
28075	EH-37	Powerhouse	Active	Extensometer	Rod extensometer	No
28076	PZ-28	Earth Dam	Active	Piezometer	Standpipe Piezometer	No
21712	MT-23	Powerhouse	Active	Join meter	Triaxial join meter	No
32123	EH-01	Powerhouse	Active	Extensometer	Rod extensometer	No
23455	PZ-10	Earth Dam	Active	Piezometer	Standpipe Piezometer	No

To improve data import, it is important to take some measures. Among them we can list dealing with missing, duplicate or inconsistent data; format the data records for import from spreadsheet to the database; check for calculation errors in the transformations of readings; format spreadsheet and inspection record data; organize technical data of the structures; verify instrumentation registration data.

2.1. Handle missing, duplicate, or inconsistent data

Errors are very common when you have a database organized in an electronic spreadsheet, when they have passed through several operators over the years of the monitored and/or inspected project, it is almost impossible not to have human error, so this work, of organizing the data (history of the dam) and information about the project, must be very thorough. Errors in records with extra zeros, when the comma is placed in the wrong place or even errors in very discrepant values, duplicate, illogical, empty or incomplete records should be sought. In general, instrumentation has a certain logic of variation. Sometimes they vary according to the level of the reservoir, other times they are dependent on temperature, depending on the section and position there may also be a correlation between the instruments, whose evaluation, interpretation and/or correction, selection of this data is made by the dam safety specialist or civil engineer.

One way to also evaluate the history of instrumentation records is to make a visual analysis of the graph, seeking to find marked variations and whether these variations are expected behaviors or not that may indicate errors in their histories.

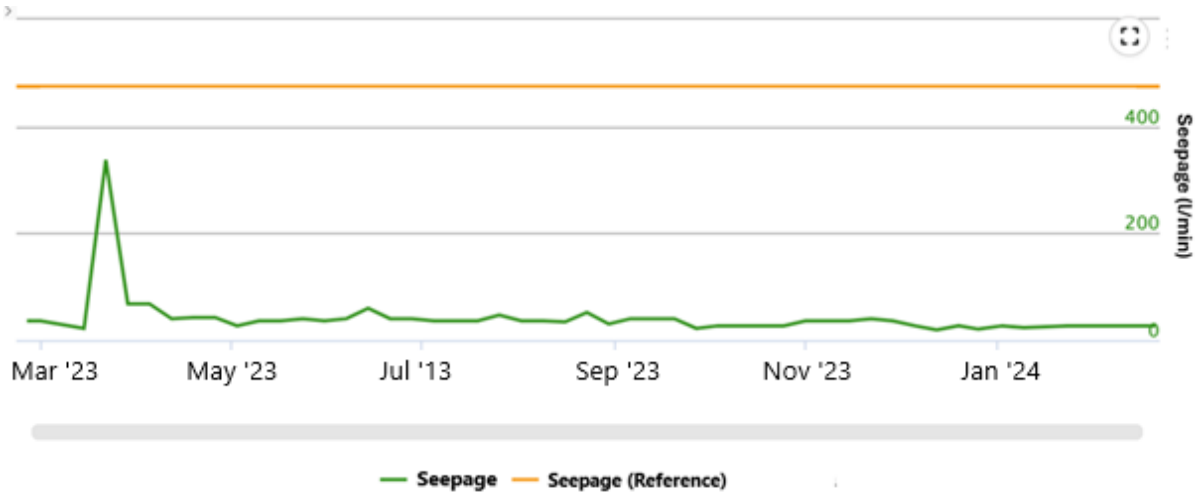


Fig. 1 - Chart with point variation

2.2. Formatting the data records for spreadsheet import into the database

The way to record the data is preferable to be line by line. For example, in the case of instrument readings, it greatly facilitates the future migration process, if the reading records were being made for each line a record. To exemplify, table 1 exemplifies the way to import this organized data into a DBMS. In Excel software, for example, there are specific commands that perform this task automatically. However, it is good to remember that, in a very large database, for example with more than 100 instruments and/or more than 10 years of records, this task becomes difficult and can take a long time to organize and feed the SGSB database.

Table 2 - Way of storing data that facilitates future import into a DBMS

Date	Instrument	Reading (input) (m)	Piezometry (output) (m)
05/03/2023	PZ-01	4,50	372,00
05/03/2023	PZ-02	4,80	372,30
05/03/2023	PZ-03	3,00	374,00

Table 2 shows the step of transforming data into engineering variables, which is basically making transformations of these data from table 2, into the data contained in table 1. In the

market there are software applications for sale but depending on the data and information that is desired, it does not make this task a simple transformation, without an analysis by an expert of what is wanted, which data relates to another to bring the information and not just the data.

Table 3 - Way of storing data that makes it difficult to import into a DBMS in the future

Data	PZ-02	PZ-02	PZ-03
05/03/2023	4,50	4,80	3,00

2.3. Check for Calculation Errors in Readout Transformations

It is not uncommon when we analyse a historical series of an instrument, to find incorrectly typed formulas in electronic spreadsheets. Eventually, the units of measurement are wrong, other times there are errors in the constants in the recording of reading data and transformation of data into information from the instruments.

In general, these formula errors happen when there are changes in instruments and measurement in which the information is not passed on to the office, where the measurements are processed.

2.4. Format spreadsheet and inspection log data

This data also needs to be tabulated in an electronic spreadsheet with specific columns, which will vary according to the DBMS that will import them.

It is important to check consistency data, such as whether the anomaly registration has been completed. The date formats, if the description of the date does not exceed the characters allowed in the DBMS.

It is important to note that there is new information that the DBMS is able to store and that the old spreadsheets do not have. As an example, we can cite the geographic coordinates of the anomaly detected.

2.5. Organize technical cadastral data of the structures

In general, these data are fewer numbers than those already mentioned in the previous topics, there is not so much concern in organizing them with many criteria. Basically, it is important to have them on paper and most likely they will be manually typed into the systems.

2.6. Verify instrumentation registration data

This is very important data, because in addition to information such as name, installation date, responsible company, there is the data of the constants that will be used in the transformation formulas of the readings entered for the desired outputs. Therefore, a constant can bring many errors in cases of direct import only of the data from the readings.

3. CRITERIA REQUIRED FOR THE IMPLEMENTATION OF A DAM SAFETY MANAGEMENT SYSTEM

The development and/or implementation of an existing DBMS and/or BCMS system specific to Dam Safety requires a series of technical engineering knowledge. The criteria presented in this work will be useful for contracting an existing system in the market or for its development.

It is verified that companies that develop this type of solution are usually opened to adaptations requested by customers, which customizes and values the information for each customer project.

The planning of hiring/developing an IT (Information Technology) solution, in conceptual terms, is a project, as it has a beginning, middle and end.

The quality of the hiring/development planning depends on the team assigned to conduct it. Therefore, the team must be chosen carefully (TCU, 2012). The team that will define the hiring/development must be formed by professionals in civil engineering of dams, specialists in design, construction, operation and monitoring and inspection, and specialists and/or information technology team.

Regarding the functional requirements of the system, dam safety specialists are the best prepared for this definition. Those responsible for the IT (Information Technology) area will be mainly concerned with the type of solution to be adopted, hosting (local or cloud), performance, interface, programming language, among others.

The inherent risks of hiring/development should also be analysed. Among the main risks:

- Deficiency in the solution planning process, failing to foresee requirements that may later be difficult to execute through contractual amendments.
- Lack of interaction between those who developed the hiring requirements and the team that will verify their fulfillment during implementation.
- Interruption of the contracted solution due to problems in the supplier company, which must provide an alternative plan for data export, for example.

3.1. Main functional requirements

3.1.1. Registers

- Allow the registration and editing of dam data, with at least the following fields per dam: dam name, technical construction characteristics of the dam (height, type of construction, etc.), location (full address, geographic coordinates), manager, technical responsible and photos.
- Allow the registration and editing of the data of the structures that make up the dams, with at least the following fields per structure: name of the structure, dam to which it is linked, technical constructive characteristics of the structure (height, type of construction), location (complete address, geographic coordinates) and photos.
- Allow the registration of types and subtypes of auscultation instruments, where it will be possible to enter data on constants, engineering variables and formulas.
- Allow the registration and editing of auscultation instruments, with at least the following fields per instrument: instrument code (current and old), QR-CODE of the instrument or similar, instrument type, reading unit of measurement, photos, installation data (quotas, geographic coordinates, among others), registration of constants and integrations between the same types of instruments and/or others.
- Allow the registration and editing of the data of the dam maintenance management bodies, with at least the following fields per body: name of the body, manager of the body, contact data, technical team that makes up the body.
- Allow the registration and editing of different types of access to the system, such as administrator, normal user, among others. Access types need to be configured by the administrator.
- User authentication must also be done through integration with AD (*Active Directory*) or another *keycloak* authentication system used.

3.1.2. Collecting readings

- The instruments must be able to be visualized on a map of the earth, with their respective geographic coordinates, for instruments installed in outdoor environments. For internal instruments, the system must allow the inclusion of a full vector drawing of the structure where the instruments will be installed, so that, when the user passes the mouse over this structure, it will stand out and can be enlarged to see the instruments positioned inside.
- The system should allow the monitoring of the execution of the readings, so that on a given date it is possible to measure the number of readings planned and performed in a given period, this should be shown in a calendar.

- The system must allow the creation of personalized reading profiles so that the user can activate them according to the need. For example: normal profile means the normal periodicity of readings. If the user creates a profile for full, the intention is probably to increase the frequency of readings.
- The System should allow the creation of groups of readings that will have recurrences and instruments to be configured by the user. The User may also assign a profile to this group of readings (normal, full, etc.).
- It should allow the creation of cross-sections for various types of dams, where the water level of the upstream and downstream level reservoirs can be inserted, as well as the instruments should be graphically positioned to show what they are measuring.
- Allow the creation of custom line charts to represent the historical series of the instruments. These graphs should be able to combine multiple instruments, as well as reservoir operational data and hydrometeorological data.
- Allow the configuration of personalized dynamic dashboards for each user, allowing to insert graphs (2D Bars, 3D Bars, line graphs, pie charts, combination charts, tree maps, kpis, among others), drawings, images, texts using the data tables registered in the software as a data source.
- Allow the search of inputs and outputs of readings, making it possible to manually edit readings, in addition to allowing edit, delete, disable these readings.
- Allow the import of existing readings in spreadsheet tables (with and without transformation of the reading into a unit of measure for analysis).

3.1.3. **Monitoring (instrumentation)**

- Provide applications for mobile devices that allow the insertion of readings manually, with the creation of validation rules for these readings to minimize human typing error.
- Offline operation of the application with transmission of readings when the device is connected to the available wireless network.
- Allow traceability, through geographic coordinates of the readings.
- Allow you to enter a new constant, variable and instrument control limits, after maintenance, directly on the mobile device.
- Allow the user who performed the readings to record data, such as name, time, functional key, signature, among others.
- Allow the reading of QR codes of the instruments, so that the mobile reading application can identify you, without the need to enter your code, in order to facilitate reading.
- Allow you to select the way in which a certain instrument will be read, if there is more than one way.

3.1.4. **Inspections**

- Allow registration and editing inspection types.
- Allow individual structures to be selected to carry out the inspection, as well as make it possible to combine inspections on different structures as a single inspection.
- Allow the registration of a personalized inspection *checklist* for each structure and make it possible to link it to the structures.
- Provide an application for mobile devices where you can record existing anomalies and follow up on them. The anomaly register must contain photos, description, date, geographic coordinates, checklist item, danger level, magnitude.
- Anomalies must be able to be monitored, with records of future until their possible fulfillment.
- It must be able to create action plans with recommendations linked to anomalies detected or even not linked, imported from an external inspection.
- After the inspection is completed, the system must issue an inspection checklist report with anomaly data, photo registration, among others.
- The system must have a GUT matrix (Priority Matrix – Severity, Urgency and Trend) to classify anomalies/recommendations.
- Allow the import of external recommendations, arising from periodic reviews or inspections by external organizations.
- Allow the insertion of a digital model of the instrumented sections, plan and section with a link to historical data and the possibility of temporal and/or integrated analysis.

3.1.5. **Safety Plan, Emergency Plan and Periodic Safety Review**

- Allow the inclusion of volumes of the Dam Safety Plan (DSP) according to the definition of the inspection agency and its updates.
- Allow the control of changes to the dam safety plans (DSP), when periodic or extraordinary reviews occur in case of change at any time.
- Allow the organization and standardization of the information of the DSP (Dam Safety Plan), as well as the automatic generation of the DSP and the PRDS (Periodic Review of Dam Safety).
- Allow the operationalization of the EAP – Emergency Action Plan.
- Allow you to view in the system the communication flowcharts with the contacts and the actions expected for each of the agents (internal and external), with a view to:
    - the level of emergence of the structure.

  - the emergency records with the description of the control/response actions expected at each emergency level for the different failure modes of the structure (overtopping, liquefaction, internal erosion, destabilization).
  - the information from the Hypothetical Rupture Study (Dam Break), such as the location of the rupture breach, flood spots of the simulated scenarios and their respective arrival times, depth, velocity, duration and hydrodynamic risk.
  - the delimitation of the Self-Rescue Zone (ZAS) and the Secondary Safety Zone (ZSS).
  - photo and description of the main affected locations.
- Allow the inclusion of other general data about the location such as hydrography, accesses and municipal and state boundaries. Public data on conservation units, traditional communities, archaeological sites, among others, must also be able to be entered into the system.
- Allow the creation of Alert and Evacuation Plans and CDS – Civil Defence System.
- The Evacuation plan must allow, through alerts:
  - enter the registration data of the affected people/properties (name, age, contact, animals, reduced mobility, photo of the place, among others).
  - visualize the escape routes with indication of the direction of flow and the maximum travel time.
  - visualize the meeting points, with information on the number of people expected.
  - view the signage (location, model and quantitative).
  - enter information about the alert system, including the area covered by the sirens, in addition to obtaining the activation history and sending alert messages to the population.
- The Emergency Action Plan (PAE) must allow:
  - enter information about the places where people and animals are sheltered, including the necessary resources (hygiene items, food, water, others).
  - view the access road interdiction points.
  - view alternative routes; view the location of the Emergency Operation Center (EOC)/Situation Room.
  - list the logistical resources and first aid supplies available.
- Provide its own application for mobile devices that allows the registration in the system of the locations and populations around the dam, with at least the following information: georeferenced location, photo, construction standard, condition of the site, purpose, family income, name, telephone number, age, reduced mobility, profession, level of education, among others.

- Provide its own application or integration with existing applications for mobile devices that serves as a means for those interested in the dam, such as the general population, Civil Defence and Aneel, ANA and inspected state agencies, to:
    - update your registration information,
    - receive "messages" from the entrepreneur,
    - receive instant notifications on the cell phone in emergencies,
    - share your location in real time,
    - receive evacuation route to the nearest meeting point,
    - monitor your commute in real time,
    - view basic operational data of the dam (flows, quotas, etc.).
- The notifications described in this requirement must provide at least: instant notifications in the application, messages via *WhatsApp*, SMS, voice call and *email*.

3.1.6. **Stability Analysis**

- Allow the insertion of data with the characteristics of the structure for geological modelling, linked to the profiles created in the monitoring module.
- Allow drawing form visualization of structure section.
- Allow stability analysis using the limit equilibrium method, in which the calculation of the safety factor, probabilistic analyses, alarms and predictive analysis are obtained.
- Allow stability analysis using the Finite Element method, in which the probable failure surface, stress map and strain map are obtained.
- Allow automatic updating of stability calculations based on new scenarios.
- Allow the display of the results of these analyses through graphs and reports, including in a dynamic dashboard.
- Enable stability analysis in an integrated manner with the monitoring system and emergency procedures.
- Stability analyses should consider the data from the installed instruments, on a periodic basis.
- This analysis should also enable integration with the EAP (Emergency Action Plan), allowing the sending of (configurable) notifications of emergencies that are detected by this analysis.

3.1.7. **Important criteria that should be part of the dam safety and monitoring system**

- Allow traceability in case of changes to system records.
- Allow the export of data to electronic spreadsheets.
- Allow to generate PDF reports in all areas of the system.
- All modules should be interrelated to avoid duplicate data.

- All modules must be able to send notifications within the system and through text messages, emails, SMS. These alerts should predict at least instruments that are outside the control level, anomalies with execution delay, and delays in instrumentation reads.
- The system must provide for periodic backups to ensure data security.
- There must be a single control of users to customize access in a unified way to each part of the system.

4. CONCLUSIONS

In view of what has been presented in this work, it is essential to plan all documentation before implementing a dam safety management system, observing the proper transition of physical documents and documents that are stored on computers without a structured logical structure of a database to maximize the future use of the migrated data. Having data migrated with errors can hinder future analyses that will have difficulties identifying where these problems occurred from. In addition, the correct definition of requirements, from non-functional requirements (those that affect the usability and general security of the software), to functional requirements (those that impact technical use, are fundamental for a good use of the tool for the main objective, which is to ensure that structures are at an adequate level of security.

In addition, a good tool, properly specified, can streamline the daily processes of monitoring, inspection, reporting, and problem detection in enough time for eventual corrections.

REFERENCES

[1] LOBO, A.V.L, GRAEFF, C. Desempenho de uma seção de barragem de terra antes e após a execução de um dreno no piso DE PÉ DE TALUDE, Revista Brasileira de Engenharia de Barragens do CBDB, Comitê Brasileiro de Barragens, 11 de janeiro de 2018.

[2] LOBO, PENA, BELINI et al. 2023, SISTEMA INTEGRADO DE MONITORAMENTO DE BARRAGENS: CASO SANEPAR

[3] Brasil. Tribunal de Contas da União. Guia de boas práticas em contratação de soluções de tecnologia da informação: riscos e controles para o planejamento da contratação / Tribunal de Contas da União. – Versão 1.0. – Brasília: TCU, 2012. 527 p.

[4] BRASIL. Lei nº 14.066, de 30 de setembro de 2020. Altera a Lei nº 12.334, de 20 de setembro de 2010, que estabelece a Política Nacional de Segurança de Barragens (PNSB), a Lei nº 7.797, de 10 de julho de 1989, que cria o Fundo Nacional do Meio Ambiente (FNMA), a Lei nº 9.433, de

8 de janeiro de 1997, que institui a Política Nacional de Recursos Hídricos, e o Decreto-Lei nº 227, de 28 de fevereiro de 1967 (Código de Mineração).

[5] COMITÊ BRASILEIRO DE BARRAGENS – CBDB. Guia Básico de Segurança de Barragens. São Paulo, SP. 2001.

[6] MENESCAL, R.A. (2009). Gestão da Segurança de Barragens no Brasil: Proposta de um Sistema Integrado, Descentralizado, Transparente e Participativo. Tese (Doutorado) - Departamento de Engenharia Hidráulica e Ambiental, Universidade Federal do Ceará, Fortaleza, 769 p.

[7] TAVARES, L. F.; SILVA, M. A.; SOUZA, J. C. Implantação de um sistema de gestão de banco de dados para monitoramento de barragens. In: CONGRESSO BRASILEIRO DE ENGENHARIA CIVIL, 25., 2022, São Paulo. Anais… São Paulo: ABENC, 2022. p. 123-130.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

ELETROBRAS' EXPERIENCE IN IMPLEMENTING EMERGENCY ACTION PLANS FOR DAMS

Roberta F. S. Fernandes[1]	Guilherme R. Peplau[2]	Lizandra P. de Azevedo[3]	Leonardo de O. G. Deotti[4]	Vinícius F. Vianna[5]
Eletrobras	Eletrobras	Eletrobras	Eletrobras	Eletrobras

ABSTRACT

The Emergency Action Plan (EAP) for dams is a crucial component of Brazil's National Dam Safety Policy, established by Law 12,334/2010 and regulated by Brazilian National Electric Energy Agency (ANEEL) in 2015. This law mandates dam operators in Brazil to implement specific plans to respond to potential emergencies. At Eletrobras, the development of EAPs began with different approaches, considering the unique characteristics of projects implemented across the country.

In 2022, Eletrobras transitioned into a private corporation, prompting the integration of its subsidiaries across multiple operational activities, including the strategic implementation of Emergency Action Plans (EAPs) in downstream communities. This process involved developing communication plans, training sessions, and evacuation drills with local communities. Eletrobras subsidiaries - CGT Eletrosul, Chesf, Eletronorte, and Furnas - were responsible for implementing EAPs, addressing the challenging task of nationwide compliance with legal requirements while ensuring operational safety.

This article outlines Eletrobras' experience in implementing Emergency Action Plans (EAPs) at its power plants, focusing on the challenges encountered in aligning schedules and meeting regulatory requirements across approximately 40 hydroelectric plants distributed throughout Brazil. The sharing of this technical knowledge aims to benefit other hydroelectric operators by providing valuable insights into best practices for dam safety management.

Keywords: Emergency Action Plan (EAP), Implementing EAP, Eletrobras

[1]roberta.fernandes@eletrobras.com, Brasil
[2]gpeplau@eletrobras.com, Brasil
[3]lizandra.azevedo@eletrobras.com, Brasil
[4]lguerra@eletrobras.com, Brasil
[5]vvianna@eletrobras.com, Brasil

1. INTRODUCTION

In Brazil, Law No. 12.334/2010 [1], amended by Law Nº. 14.066/2020 [2], established the National Dam Safety Policy (PNSB in initials in portuguese). Among its guidelines is the mandatory Dam Safety Plan (PSB in initials in portuguese), which includes the Emergency Action Plan (EAP). The EAP details the procedures for managing risks and responding to emergency situations, as well as identifying the agents who must be notified.

The PNSB requires the preparation of an EAP for dams that accumulate or dispose of mining tailings and for all dams classified as having medium or high associated potential damage or high risk, as assessed by the enforcement authority.

In the EAP, possible emergency scenarios are analyzed, including flood maps, to identify the worst associated potential damage - considering human losses, environmental and socio-economic impacts. Preventive, corrective and emergency response procedures are established. In addition, the Self-Rescue Zone (ZAS in initials in portuguese) is delimited, where the authorities' reaction time is insufficient, and the Secondary Safety Zone (ZSS in initials in portuguese), defined as the remainder of the impact area according to the flood maps.

All of Eletrobras' projects included in the PNSB have EAP drawn up and submitted to the municipal authorities in the potentially affected areas on time. Since their creation, the EAP have been revised periodically, with updates sent to the competent authorities whenever there are significant changes in the data.

Implementing the EAP involves communicating with external agents, installing signaling infrastructure and warning systems, as well as internal and external training and drills. However, this implementation is challenging due to the absence of a culture of prevention in Brazil, including among public representatives and civil defense agencies.

In the context of the privatization and reorganization of Eletrobras, the integration of safety guidelines required significant efforts to adapt planning and meet new deadlines and regulatory requirements.

Dam safety and the effectiveness of EAPs are fundamental to protecting communities and the environment. In countries like the United States and Sweden, strict protocols ensure rapid responses to incidents, including continuous monitoring, evacuation drills and clear communication with the population. In China, the central government takes responsibility for emergency management, investing in advanced monitoring technologies.

In Brazil, legislation places responsibility for dam safety and the implementation of EAPs on the operators. This regulatory framework requires each operator to develop its own strategies

for managing emergencies, highlighting the importance of an integrated and systematic approach to meeting safety demands.

2. ELETROBRAS AND THE EMERGENCY ACTION PLANS

Eletrobras was created in 1962 with the aim of coordinating national energy development and promoting the expansion of the country's electricity infrastructure. At the time of its foundation, there were already Companhia Hidroelétrica do São Francisco - Chesf and Furnas Centrais Elétricas, public generation companies that operated in the northeast and southeast regions of Brazil, respectively. Later, CGT Eletrosul was created, with operations in the three southern states of Brazil, and Eletronorte with generation and transmission activities in the north and center-west regions (Figure 1).

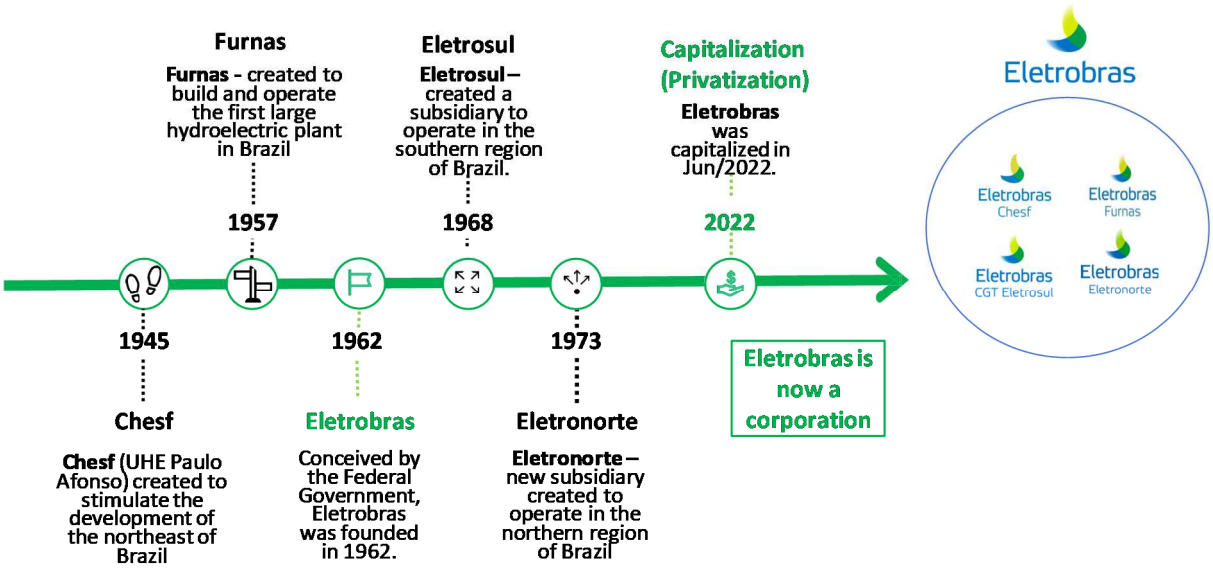


Fig.1 - Timeline of the creation and evolution of Eletrobras

Eletrobras currently owns a portfolio of 38 projects operated and maintained by the group's companies, as shown in Figure 2 below.

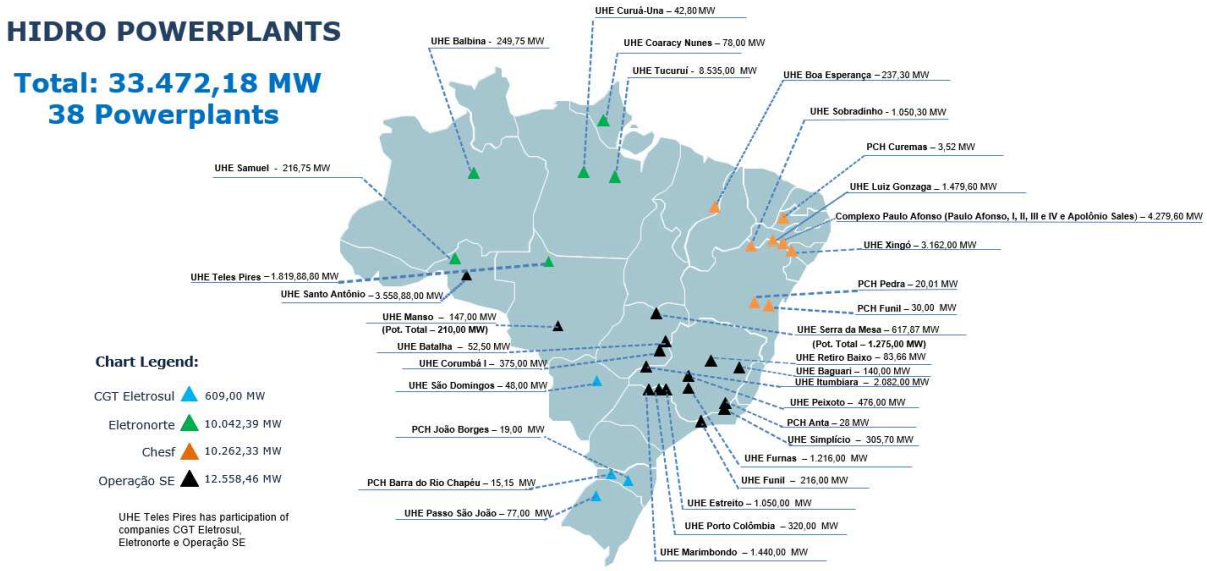


Fig. 2 - Eletrobras' portfolio of hydroelectric plants (Dec/24)

Despite belonging to the same group, the companies have always acted autonomously in technical and management terms, but with regard to the EAPs drawn up, all the companies considered the same basic stages to be met as milestones, with reference to the support material suggested by the Brazilian Association of Energy Generating Companies - ABRAGE (in initials in Portuguese) [3]. The following activities were defined as milestones for implementing an EAP.

Table 1 – ACTIVITIES OF IMPLEMENTATION OF AN EAP (ABRAGE, 2024)

ITEM	ACTIVITY
1	Work plan for implementing the EAP
2	Internal evacuation drill for the Emergency Action Plan - EAP
3	Implementation of the dam safety communication and alert system
4	Signposting of escape routes and meeting points
5	Dissemination and communication actions for communities
6	Registration and up-to-date mapping of the existing population in the ZAS
7	External evacuation drill for the Emergency Action Plan - EAP

With the advent of the group's privatization, the new management of Eletrobras opted to integrate dam safety processes into its new organizational structure, creating in

November/2023, among other actions for this purpose, the Executive Management of Dam and Reservoir Safety - OOXS.

One of the major challenges presented by Eletrobras' Vice-Presidency of Operations and Safety to the newly created OOXS was to monitor and provide integrated guidelines for the implementation of the PAEs of the subsidiaries. It is worth adding that this process was already underway in each company, each with its own established schedule and at different stages of execution.

3. THE IMPLANTATION PROCESS

3.1. Regulatory timelines

In August 2020, in response to that year's inspection, the companies Eletrobras Chesf, Eletrobras Eletronorte and Eletrobras Furnas sent ANEEL letters containing Action Plans for the implementation of the EAP, with the activities planned for each hydroelectric generation asset related to communication and warning infrastructure, implementation of escape routes and meeting points with simulated exercises for each of the plants listed.

After the EAP schedule was submitted to ANEEL, Law nº. 14.066 of September 30, 2020, which amended Law nº. 12.334/2010, came into force, creating new obligations to be met in the plan, such as a cadastral survey and updated mapping of the existing population in the Self-Rescue Zone (ZAS in initials in portuguese), identification of social vulnerabilities and coordination between the entrepreneur and municipal and state civil protection and defense agencies.

Thus, after the change in legislation establishing the National Dam Safety Policy (PNSB in initials in portuguese), the Eletrobras companies rescheduled their schedules in line with the new responsibilities imposed by the law. Precisely for this reason, REN 1.064/2023 [4], which came into force on June 1, 2023, provided for an additional deadline for updating the existing EAPs, within 12 months of the publication of this standard.

Figure 3 shows the timeline of the evolution of the planning of EAP implementation activities, in accordance with the new requirements arising from Law nº 14.066/2020 and REN nº 1.064/2023.

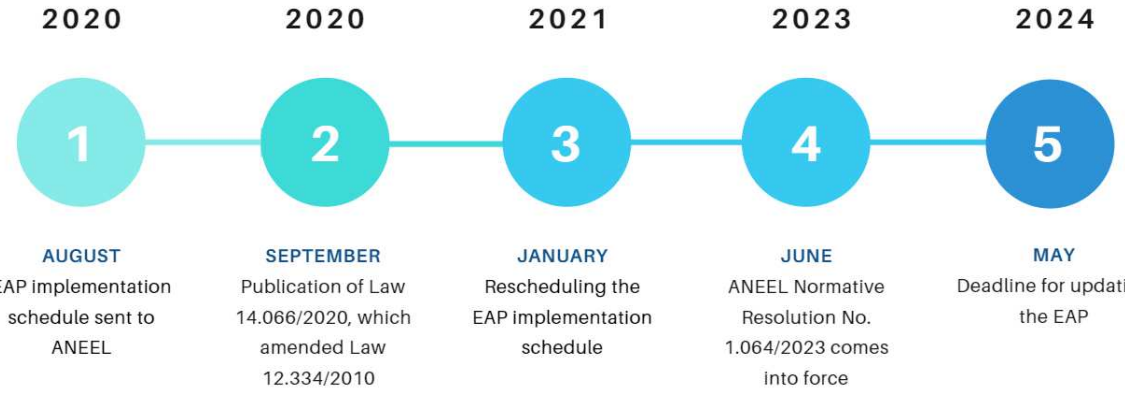


Fig. 3 - Timeline of Dam Safety regulatory developments towards the implementation of the EAP

In November 2023, Eletrobras Eletronorte's projects completed all the external simulations. This work resulted in valuable lessons learned, which could be replicated in planning the activities of the other subsidiaries.

In January 2024, the National Electric Energy Agency - ANEEL issued a circular letter, stating its understanding that the actions to implement the EAP should be completed and that they would be subject to inspection by the agency. In view of the above, additional efforts have been made to make it possible to bring forward the timetables initially planned:

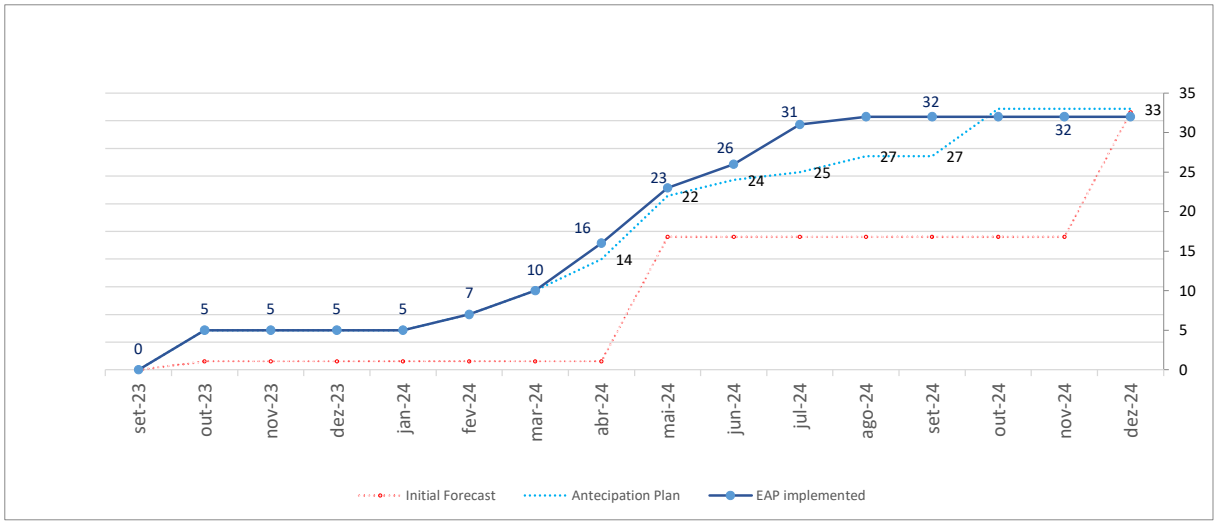
- Actions in the land and environmental areas to speed up obtaining the agreements and consents needed to set up the external infrastructure.

- Acquisition of mobile sirens to mitigate the risk of delays in setting up the definitive communication system and make it possible to anticipate external drills.

- Hiring a company to provide support in the planning and implementation stage of external drills.

The results of the anticipation plan were positive, achieving success in 2024 in carrying out external simulations in the ZAS municipalities of all its projects. Chart 1 shows the temporal evolution of the EAP implementation in Eletrobras hydroelectric plants, considering the initial forecast, the anticipation plan and the effective implementation of EAPs

Chart 1 - Anticipation Plan - Temporal Evolution of EAP Implementation



The EAP implementation process involved several stages, including interaction with external agents, implementation of communication infrastructure, communication campaigns, and training and simulations. The following items present each phase of the EAP implementation in Eletrobras hydroelectric plants.

3.2. Eletrobras' Strategies for Implementing EAPs

Each company has adopted specific strategies to meet the demands of implementing the EAP for its hydroelectric assets. In this section, we will concisely present how Eletrobras, in general, has dedicated itself to fulfilling the activities envisaged in this context.

3.2.1. Work Plan for Implementing the EAP

Eletrobras drew up a work plan which began in 2018, shortly after the EAPs were officially handed over to the relevant authorities. Since then, several meetings and seminars have been held (Figure 4) with the aim of integrating civil defense and municipalities. This process culminated in continuous collaboration in the drafting and revision of the plans, which lasted until 2024.

Fig. 4 - Seminar to present the EAP to civil defense authorities - November/2018.

This period was marked by intense integration (Figure 5), which contributed significantly to improving the document in subsequent revisions. It was also essential for raising awareness of the risks associated with dams. Clarifying these risks, especially with regard to structural failures, was particularly important since, although dams are monitored and safe, the introduction of the EAP generated a great deal of mistrust about the condition of these structures in general.

Fig. 5 - Extraordinary meeting of the "PAE/PLANCON" integration committee.

Also during this phase, the companies organized themselves into working groups and dam safety committees, with the aim of coordinating the implementation of the EAP in all their enterprises. These groups drew up the work plans and monitored the execution of all the activities related to the EAP, which generated a positive synergy through the exchange of experiences (Figure 6) between the representatives of each enterprise or group of plants.

Fig. 6 - Presentation of the EAP to the HPP operations team in 2019.

3.2.2. PAE tabletop drill and internal evacuation drill

Before the internal and/or external evacuation drills, exercises called “tabletop drills” were also carried out (Figure 7), with the aim of training the team involved in emergency management. These exercises were based on a hypothetical situation, conducted using a game. As hydrological conditions or possible anomalies were simulated, the dam was gradually brought into a fictitious emergency situation. This allowed decision-makers to face adverse conditions and be challenged to “trigger” their facility's Emergency Action Plan.

Fig. 7 – Tabletop exercises

Internal facility evacuation drills (Figure 8), with the hypothetical scenario of a dam structure collapsing, began to be carried out in 2020. Until the external drills were carried out, the companies continued to conduct their internal area abandonment drills, with the aim of promoting the internalization of the EAP guidelines among employees.

Fig. 8 - Simulated area abandonment exercises carried out at Eletrobras hydroelectric plants.

The aim of this exercise is to test the effectiveness of the internal procedures for an emergency evacuation, as well as to assess the range of the sirens installed, the time and conditions of

the participants' displacement, possible impediments or obstacles that could hinder rapid displacement, among other parameters.

3.2.3. Implementation of the Dam Safety Communication and Alert System

The implementation of the EAPs included the installation of warning sirens and a communication system aimed at dam safety. Critical infrastructure was planned with the implementation of audible alarm systems and signaling of evacuation routes.

In order to adequately respond to civil defenses in different Self-Rescue Zone (ZAS in initials in portuguese) contexts, Eletrobras implemented a mixed system, consisting of fixed siren towers and mobile sirens (Figure 9), the latter coupled to a vehicle exclusive to the project.

Fig. 9 - Sound alarm systems (fixed and mobile sirens).

In this way, redundancy was created in the warning system, guaranteeing full coverage of the ZAS, in accordance with the guidelines established by the dam safety best practice guides. The fixed siren towers were positioned at strategic points, focusing on areas of high population density and regions of greater hydrodynamic risk. The mobile sirens were accompanied by circulation rotagrams, which were defined after field visits.

3.2.4. Implementation of Escape Route and Meeting Point Signs

Signposting escape routes and meeting points was a priority, with the installation of indicative signs (Figure 10). These signs were strategically positioned to ensure good visibility and easy understanding, especially in emergency situations.

Fig.10 - Escape Route and Meeting Point signs used

All the signs installed followed a strict standard, in accordance with the rules and guidelines established by the local civil defense agencies, which monitored and approved the project and the installation of the signs.

3.2.5. Community Outreach and Communication

Communication campaigns aimed at the population (Figure 11) were developed to raise awareness among both internal and external audiences about dam safety and the holding of simulated evacuation exercises in the ZAS.

Fig.11 - Campaign to hand out materials to publicize the simulations

Another important action was the involvement of schools (Figure 12) in the communication and dissemination plan for the EAP, addressing prevention and safety aspects within the context of the Emergency Action Plan.

Fig. 12 - Actions in public schools.

3.2.6. Registration and Updated Mapping of the Existing Population in the ZAS

In 2023 and 2024, people and homes in the Self-Rescue Zones (ZAS) were registered. During this process, all residents were informed about the defined meeting points, which contributed to the safety and organization of evacuation in emergency situations.

At this stage, the sociocultural, economic and biophysical attributes of the population were cataloged, such as the elderly, children, people with disabilities, people without schooling and/or access to means of communication, among others, with the aim of identifying social vulnerabilities (Figure 13). A previously prepared digital form was used to collect the information.

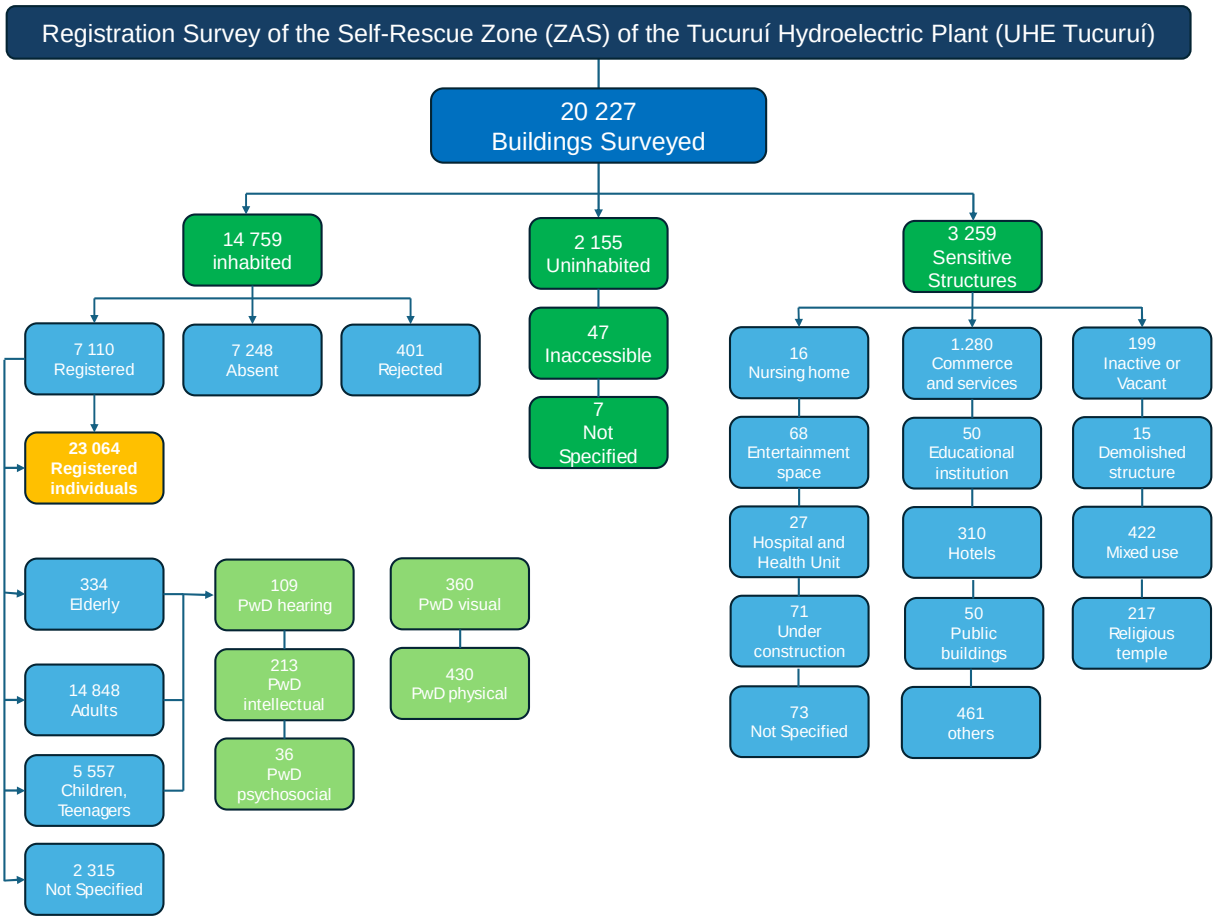


Fig. 13 - Cadastral survey of the ZAS of an Eletrobras hydroelectric plant.

3.2.7. External EAP Evacuation Simulation

As a final stage in the effective implementation of the EAP, external simulated exercises were carried out with the population in the ZAS (Figure 14), involving local communities in practical exercises to abandon the area and occupy the meeting points defined in collaboration with the civil defenses. This initiative was part of an ongoing effort to integrate the population into the security and emergency preparedness process.

Fig. 14 - Representation of the ZAS and main control points for the external simulation

These actions reflect Eletrobras' ongoing commitment to ensuring the safety of hydroelectric plant operations and the protection of neighboring communities. The implementation of EAPs is a dynamic process that requires collaboration between various stakeholders and the adoption of integrated safety practices.

Fig.15 - External simulated ZAS evacuation exercise

The plan is for these exercises to be carried out periodically, at least every three years, with the aim of guaranteeing the continuity and improvement of training, as well as strengthening community involvement, promoting and consolidating a culture of preventive safety.

Fig.16 - Meeting points during external simulated exercise closing events

4. CONCLUSIONS

The implementation of the Emergency Action Plans (EAP) at Eletrobras' hydroelectric plants represents a significant milestone in the company's commitment to operational safety and the protection of neighboring communities. Throughout a dynamic and challenging process, Eletrobras adopted an integrated approach, involving not only its subsidiaries, but also civil defense, municipalities and the local population. The implementation stages, which involved everything from planning and carrying out internal and external drills to installing communication and signaling systems, were essential to ensuring the effectiveness of the EAP and raising awareness of the risks and evacuation procedures in dam emergencies.

Collaboration between the various stakeholders, along with the use of safety practices based on the best national guidelines, was crucial to the success of the process. The actions carried out, such as registering and mapping the populations in the Self-Evacuation Zones (ZAS in initials in portuguese), installing sirens and meeting points and promoting awareness campaigns, ensured that both internal and external audiences were prepared to act in a coordinated and effective manner in case of need.

In addition, the anticipation of the implementation schedules, in line with the adaptation to new legal requirements, demonstrated Eletrobras' ability to adjust to regulatory changes and challenges in an agile manner. The success of the activities, evidenced by the external simulations carried out with the populations of the ZAS, reflects the company's ongoing commitment to the safety of its facilities and to the process of developing a culture of prevention.

Finally, Eletrobras is constantly updating its EAPs, ensuring the continuity and improvement of training and integration with communities. This continuous effort demonstrates the importance of dam safety, which not only seeks to guarantee the integrity of operations, but also to promote the trust and tranquility of the communities involved.

The implementation of the EAPs, therefore, not only meets a regulatory requirement, but also establishes a model of social responsibility, consolidating Eletrobras as a benchmark in the energy sector, committed to safety, sustainability and the well-being of the communities around it.

REFERENCES

[1] BRASIL. Lei Nº 12.334 de 20 de setembro de 2010. Presidência da República - Casa Civil - Subchefia para Assuntos Jurídicos. Brasília, 2010.

[2] BRASIL. Lei Federal nº 14.066 de 30 de setembro de 2020. Diário Oficial da União. Brasília-DF, 2020.

[3] ABRAGE - Associação Brasileira das Empresas Geradoras de Energia Elétrica. (2024) Boas Práticas para implantação do Plano de Ação de Emergência (PAE) em barragens de usinas hidrelétricas V1.0. Belo Horizonte, Brasil.

[4] ANEEL – Agência Nacional de Energia Elétrica (2023). Normativa ANEEL n.º 1.064 de 02 de maio de 2023. Brasília-DF.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DAM SAFETY KPI AT ELETROBRAS: DEVELOPMENT AND APPLICATION OF TACTICAL KEY PERFORMANCE INDICATORS

Roberta F. S. Fernandes[1]	Leonardo de O. G. Deotti[2]	Vinícius F. Vianna[3]	Guilherme R. Peplau[4]	Lizandra P. de Azevedo[5]
Eletrobras	Eletrobras	Eletrobras	Eletrobras	Eletrobras

ABSTRACT

This article presents the tactical key performance indicators utilized by Eletrobras for dam safety management in its hydroelectric dams. These indicators were initially developed by a working group composed of representatives from Eletrobras Holding and its subsidiaries since 2019, and they have undergone iterative refinements to enhance their applicability and precision. These tactical indicators are specifically developed to evaluate the effectiveness and quality of processes related to dam safety, serving as a strategic tool for management oversight.

Eletrobras currently works with four key indicators: (1) IEPA, which quantifies the effectiveness of the execution of the annual plan concerning regular inspections and instrumentation readings; (2) IDEA, which assesses the structural performance of dams based on instrumentation data and detected anomalies; (3) IPIR, which monitors the implementation of corrective actions for identified anomalies and compliance with recommendations outlined in technical reports; and (4) an indicator that evaluates the scores of manageable risk items as defined by the risk matrix adopted by ANEEL, the sector's regulatory agency.

The article details the current methodologies and formulas applied by Eletrobras in the consolidation of these indicators, presenting empirical data from their application and correlating results with field conditions. The implementation of these tactical indicators has proven to be an effective tool for enhancing strategic dam safety management, facilitating the identification of potential issues and driving continuous improvement of internal safety processes. This approach positions Eletrobras as a potential benchmark for other hydroelectric operators, promoting a more robust safety culture aligned with engineering best practices.

Keywords: Dam Safety, KPI, Eletrobras, Key Performance Indicators.

[1] roberta.fernandes@eletrobras.com, Brasil
[2] lguerra@eletrobras.com, Brasil
[3] vvianna@eletrobras.com, Brasil
[4] gpeplau@eletrobras.com, Brasil
[5] lizandra.azevedo@eletrobras.com, Brasil

1. INTRODUCTION

Dam safety is essential in the infrastructure sector, especially in the context of hydroelectric plants, due to their strategic importance for energy generation and the potential impact of incidents [1]. The Brazilian National Dam Safety Policy (PNSB), established by Law 12.334/2010 [2], is a crucial regulatory milestone in Brazil, setting guidelines for monitoring, evaluating, and mitigating risks associated with these structures.

Despite possessing a comprehensive body of knowledge, in 2019, Eletrobras acknowledged the need to improve the strategic management of its dams, considering the vast number of dams operated by the company and their distribution across the entire national territory. As a result, it was formed a task force responsible for developing strategic indicators designed to facilitate ongoing monitoring of compliance with legal obligations and good engineering practices.

The article details the current methodologies and formulas applied by Eletrobras in the consolidation of the strategic dam safety KPIs, presenting empirical data from their application and correlating results with field conditions.

The implementation of these tactical indicators has proven to be an effective tool for enhancing strategic dam safety management, facilitating the identification of potential issues and driving continuous improvement of internal safety processes. This approach positions Eletrobras as a potential benchmark for other hydroelectric operators, promoting a more robust safety culture aligned with engineering best practices.

1.1. **Eletrobras' hydroelectric park**

Eletrobras was created in 1962 with the aim of coordinating national energy development and promoting the expansion of the country's electricity infrastructure. At the time of its foundation, there were already Companhia Hidroelétrica do São Francisco - Chesf and Furnas Centrais Elétricas, public generation companies that operated in the northeast and southeast regions of Brazil, respectively. Later, CGT Eletrosul was created, with operations in the three southern states of Brazil, and Eletronorte with generation and transmission activities in the north and center-west regions (Figure 1).

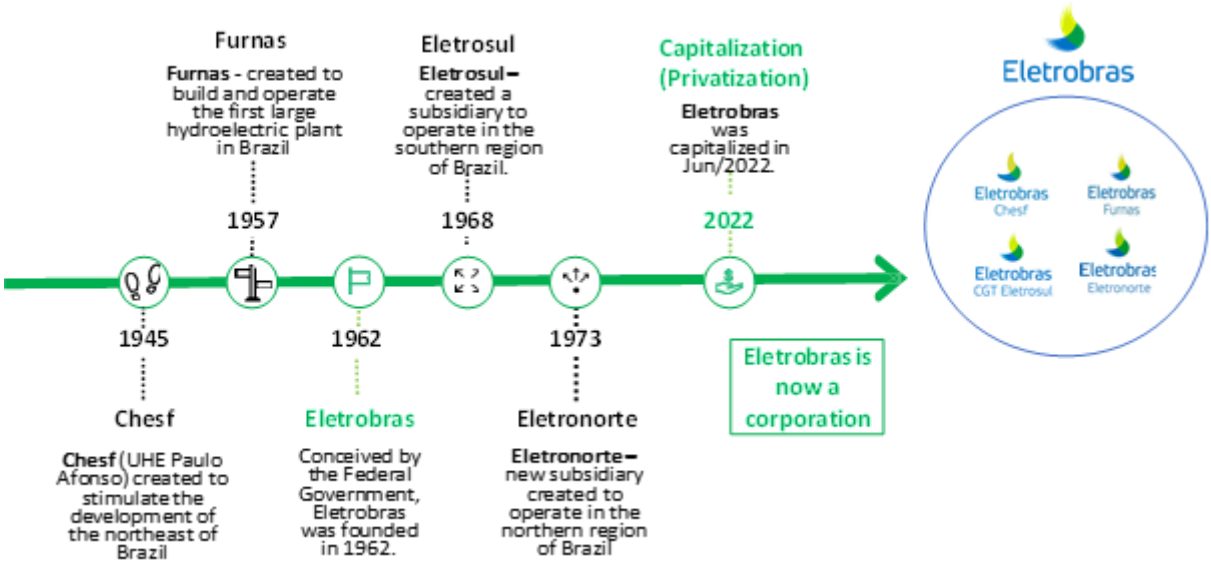


Fig.1 - Timeline of the creation and evolution of Eletrobras

Eletrobras currently owns a portfolio of 38 hydroelectric powerplants operated and maintained by the group's companies [3], as shown in Figure 2 below. The company is the largest generator of hydroelectric power in Brazil, with more than 33 GW of power installed in all the country's geographical regions (Figure 1).

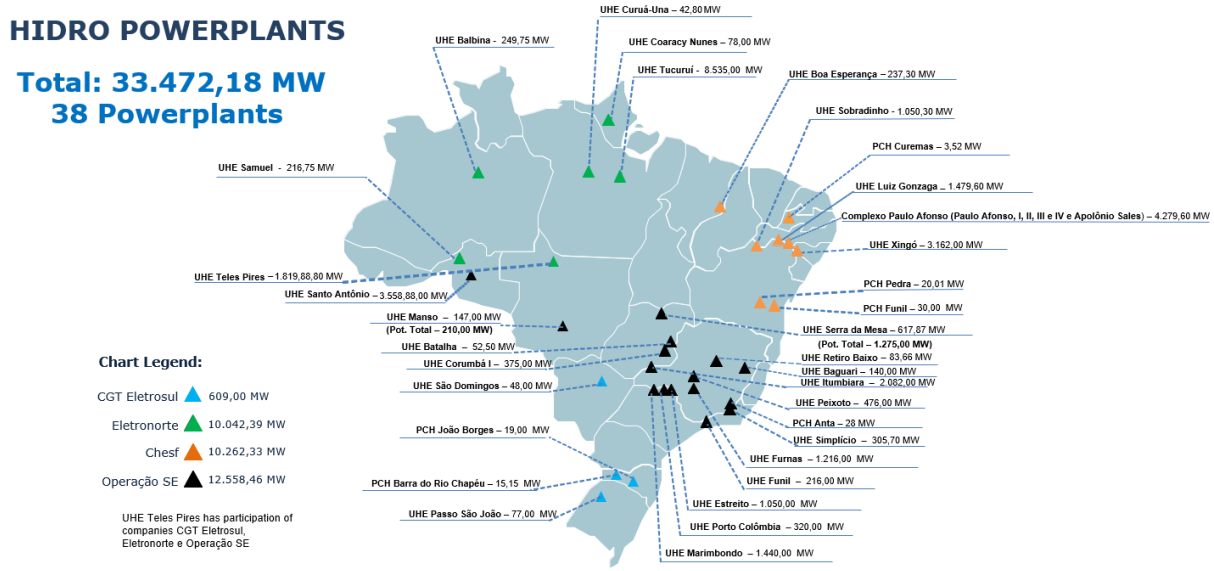


Fig. 2 - Eletrobras' portfolio of hydroelectric plants (Dec/24)

.

1.2. Brazilian National Dam Safety Policy (PNSB)

The National Dam Safety Policy (PNSB), established by Law No. 12.334/2010, aims to ensure the safety of dams in Brazil, focusing on the protection of human life, public and private property, and the environment. The law sets out a series of guidelines and obligations that dam operators must follow to mitigate risks and prevent accidents.

Law No. 12.334/2010 outlines various responsibilities for dam operators, which are entities or companies responsible for operating and maintaining dams in Brazil. The main obligations imposed by the law include:

a) Dam Safety Plan (PSB): Operators must prepare, implement, and keep updated a Dam Safety Plan (PSB) for each dam under their responsibility. The PSB must include risk assessments, actions to mitigate these risks, emergency plans, and protocols for managing risk situations.

b) Risk Classification of Dams: Dams must be classified according to the potential risks they pose in the event of accidents, considering factors such as environmental and social impacts in case of failure. These classifications must be reviewed periodically.

c)Monitoring and Inspection: Operators must carry out continuous monitoring of the conditions of the dams, using appropriate instruments and methods to identify anomalies or signs of deterioration. Periodic inspections are also required, both in relation to the physical structure of the dam and the monitoring and control systems

d) Reports and Communication: The law requires operators to prepare technical reports on the condition of the dams and their safety assessments to ensure transparency and compliance with regulations. Operators must immediately notify competent authorities and the public, if necessary, in the event of imminent risk of an accident.

e) Emergency Actions and Contingency Plans: Operators are required to develop emergency plans and contingency plans, which must be updated periodically and tested through drills, to prepare teams for emergency situations.

f) Dam Safety Periodic Review (RPS): is an additional mechanism in the monitoring and inspection process chain, characterized by redundancy and technical auditing. The RPS is carried out by agents independent of the project owner and aims to review the implemented processes, hydrological studies, stability analyses, and the control levels of the instrumentation, in periodic cycles of 5, 7 or 10 years.

g) Regulatory Agencies: Government regulatory agencies have the authority to oversee and audit the process, conducting regular monitoring campaigns to ensure that entrepreneurs comply with the requirements of the dam safety policy. In the specific case of the Brazilian

electric sector, regulation is carried out by the National Electric Energy Agency (ANEEL), which establishes criteria and safety actions for dams associated with hydroelectric power plants through Normative Resolution No. 1064/2023 [4].

h) Dam Safety Level (NSB): the monitoring system uses the regulatory macro indicator related to the Dam Safety Level (NSB), which provides the updated and consolidated diagnosis of the structural safety condition. The classifications used in the NSB are listed below:

i) Normal: there are no anomalies or contingencies, or those that exist do not compromise the dam's safety, but should be controlled, monitored, or repaired over time;

ii) Attention: anomalies or contingencies do not compromise the dam’s safety in the short term, but require increased monitoring, control, or repair in the medium or long term;

iii) Alert: anomalies or contingencies represent a risk to the dam's safety, requiring immediate actions to maintain safety conditions;

iv) Emergency: anomalies or contingencies represent an imminent risk of failure, requiring urgent actions to prevent and mitigate human and material damage.

2. DEVELOPMENT AND IMPLEMENTATION OF DAM SAFETY KEY PERFORMANCE INDICATORS AT ELETROBRAS

The year 2022 was a milestone for Eletrobras as it transitioned into a corporation, integrating its subsidiaries. This integration aimed to standardize dam safety plans and procedures while implementing agile monitoring systems capable of identifying critical points within a vast and complex dataset. Eletrobras' projects are distributed across Brazil, a country of continental dimensions, marked by diverse cultures and varying social conditions.

These projects vary in age, size, technical characteristics, and are influenced by regional management specificities. Consequently, the monitoring indicators proposed must be designed to deliver results that consider the heterogeneous nature of the company's portfolio.

The dam safety operational process is fully operated within the regional dam safety teams. The results of the operational process are continuously monitored through indicators by the Eletrobras executive department of dam safety (an entity established to standardize and control the quality of the process), which consolidates and analyses the results, providing periodic reports to the company's senior management and making recommendations to operational areas, thereby forming a comprehensive governance process.

Based on the work carried out by the Indicators Working Group (GT Indicadores), four outcome indicators were defined to be used in monitoring the process, detailed in the following subsections:

2.1. IEPA (Acronym in Portuguese) – Monitoring Plan Execution Index:

IEPA measures the fulfilment of planned monitoring activities, including the execution of Dam Safety Inspections (Regular and Routine Inspections) and the implementation of instrumentation reading campaigns. This index produces a value reflecting the efficiency in executing these plans.

Deficiencies in this indicator may point to challenges such as difficulties in contracting services, insufficient staffing for routine tasks, extended maintenance of instrumentation, improper reading intervals, issues with the data management system for instrumentation, or low adherence to the process in favour of other activities.

A low IEPA score is associated with reduced data availability for dam safety diagnostics, potentially affecting the quality of the assessments provided.

The IEPA indicator reflects the effectiveness of monitoring teams in executing regular planning (routine or preventive activities) related to monitoring and maintenance.

$$IEPA = (I1 + I2\,)/2 \tag{1}$$

Where:

$$\mathrm{I}_1 = \left(\frac{\text{Number of Measurements performed}}{\text{Number of Measurements planned}}\right);\ \mathrm{I}_2 = \left(\frac{\text{Number of Inspections performed}}{\text{Number of Inspections planned}}\right);$$

2.2. IDEA (Acronym in Portuguese) – Performance Structures Index

This indicator is based on the two fundamental and complementary monitoring principles (qualitative and quantitative assessments), which provide support for diagnosing issues detected during dam safety campaigns: a) the number of anomalies identified in the structures during inspections, with weights assigned according to the identified Risk Level, and b) the number of instruments readings out of the reference limit.

Deficiencies in this indicator may indicate poor performance in the conservation of the project, requiring increased maintenance directed at the identified anomalies, or the need for further studies on the instrumentation, re-evaluating reference indexes, calibrating equipment, or confirming behaviours outside the expected conditions, implementing more detailed monitoring on the evolution of measurements and assessing potential impacts.

Low results from this indicator may reflect chronic conservation problems, the existence of relevant anomalies to be addressed, or the need for reassessment/deepening of the study on the behaviour of the instrumentation.

$$\mathrm{IDEA} = G_i \,.\, \mathrm{A}_{\mathrm{quant}} + (1 - G_i) \,.\, \mathrm{A}_{\mathrm{quali}} \quad (2)$$

Where:

Gi = level of instrumentation of the dam

Gi = 0.5; for dams with good instrumentation levels. The number of instruments is enough to understand all relevant phenomenal occurring in the dam.

Gi = 0.4; for dams with medium instrumentation levels. The number of instruments is enough to understand the most relevant phenomenal occurring in the dam.

Gi = 0.3; for dams with poor instrumentation levels. The number of instruments is not enough to understand the most relevant phenomenal occurring in the dam.

$$Aquali = 1 - \sum_{n=(0,1,2\ ou\ 3)} NA_n * F_n \quad (3)$$

Where:

n = Hazard level n = 0 (null), hazard level n = 1 (Low), hazard level n = 2 (Medium), hazard level n = 3 (High), as recorded on the inspection sheets.

NA_n= Number of anomalies detected with hazard level n;

F_n = Multiplication factor assigned to each hazard level. (F_0 = 0.0001; F_1 = 0.001; F_2 = 0.01; F_3 = 0.1).

The values assigned by the multiplication factors for each of the hazard levels were obtained empirically, considering 2 years of data, including the evaluation of the anomalies identified and their impact on the general condition of dam.

Each dam operator must define these values considering its own characteristics, such as the age of the dams, construction methods, risk tolerance, among others.

$$Aquanti = \frac{IR\ r}{IR\ t} \quad (4)$$

Where:

IR r: Number of instrumentation records within the instruments reference levels.

IR t: Total number of instrumentation records.

2.3. IPIR (Acronym in Portuguese) – Plan of Recommendations Index

The IPIR Indicator presents an index that reflects the timely completion of recommendations derived from different routines in the dam safety process (Regular Safety Inspections, Periodic Safety Reviews, Regulatory Agencies Inspections, etc). The indicator aims to measure the degree of compliance with the dam safety recommendations within the schedule associated with the action plan for the identified issues.

Deficiencies in this indicator highlight difficulties in meeting the proposed schedules in action plans. These difficulties may arise from excessively tight timelines, challenges in contracting necessary actions, limitations in technical and budgetary resources, difficulties in designing appropriate technical solutions, low prioritization of recommended actions, or inadequate staffing for managing these actions.

Results from this indicator may reflect increased complexity in finding solutions for recommended actions, leading to higher costs for resolution or worsening of the reported condition. Low indices for this indicator also increase regulatory risk, as meeting deadlines for actions is required by regulatory agencies.

$$IPIR = \frac{\sum_{i=(1,\ 2\ ou\ 3)} NR_i * PP_i}{\sum_{i=(1,\ 2\ ou\ 3)} NT_i * PP_i}$$

where:

i = Priority level: 1 (Low Priority), 2 (Medium Priority), or 3 (High Priority), as defined by the priority in the recommendations' action plan.

NR_i = Number of priority *i* recommendations completed or scheduled within the deadline;

NT_i = Total number of priority *i* recommendations;

PP_i = Multiplication factor assigned to each priority level i (PP_1 = 1; PP_2 = 2; PP_3 = 3).

2.4. IECSB (Acronym in Portuguese) – Dam Conservation/Safety State Index

The IECSB indicator presents an index derived from the Dam Risk Classification Matrix formalized by ANEEL in Normative Resolution 1.064/2023, with classification criteria to be used. The index extracts from the matrix the items manageable by the operator, related to the Conservation State of the Project "EC", and the Dam Safety Plan "PS" (thus excluding items inherent to the project's design characteristics).

The Conservation State (EC) matrix updates the condition of variables related to the operational condition of spillways and electromechanical devices, the conditions of percolation control of structures, evaluation of deformations, settlements and deterioration on slopes. The tables 2 and 3 illustrated below present the criteria and scores for the Dam's Conservation Status.

Table. 1 (a) - Regulatory Risk Matrix – Conservation Status (CS)

Reliability of Extravasor Structures	Reliability of Adduction Structures	Percolation
Civil and mechanical structures in full operation / unobstructed approach or restitution channels or spillway (free sill type) (0)	Civil structures and mechanical devices in proper maintenance and operation conditions (0)	Percolation fully controlled by the drainage system (0)
Civil and mechanical structures prepared for operation, but without emergency power supply sources / channels or spillway (free sill type) with erosion or obstructions, but without risks to the slope structure. (4)	Compromised civil structures or mechanical devices with identified problems, with reduced flow capacity and with corrective measures in implementation (4)	Moisture or upwelling in downstream areas, walls, slopes or jambs stabilized and/or monitored (3)
Compromised civil structures or mechanical devices with problems, with reduced flow capacity and with corrective measures in implementation / channels or spillway (free sill type) with erosion or obstructed, with risk of compromising the structure. (7)	Compromised civil structures or mechanical devices with identified problems, with reduced flow capacity and without corrective measures (6)	Moisture or upwelling in downstream areas, walls, slopes or jambs without treatment or in the diagnostic phase (5)
Compromised civil structures or mechanical devices with identified problems, with reduced flow capacity and without corrective measures / obstructed channels or spillway (free sill type) or with damaged structures (10)	-	Upwelling in downstream areas, slopes or jambs with material carrying or with increasing flow (8)

Table. 1 (b) - Regulatory Risk Matrix – Conservation Status (CS) - Continuation

Deformations and Settlements	Slope Deterioration	Dam Lock
Non-existent (0)	Non-existent (0)	Non-existent (0)
Existence of cracks and subsidence of small extent and no impact (1)	Failures in the protection of slopes and walls, presence of shrubs of small extension and zero impact. (1)	Well-maintained and functioning civil and mechanical structures (1)
Existence of cracks and subsidence of considerable impact generating the need for additional studies or monitoring (5)	Surface erosion, exposed hardware, widespread vegetation growth, generating the need for monitoring or corrective action (5)	Compromised civil structures or mechanical devices with identified problems and corrective measures being implemented (2)
Existence of cracks, slumps or significant slips, with the potential to compromise safety (8)	Accentuated depressions on slopes, landslides, deep erosion grooves, with the potential to compromise safety (7)	Compromised civil structures or mechanical devices with identified problems and no corrective action (4)

The Safety Plan (PS) matrix considers scores related to the existence of technical documentation and monitoring procedures and manuals, existence of reservoir operational rules and evaluation of the company's organizational structure. The table 3 illustrated below presents the criteria and scores of the Dam Safety Plan (PS) matrix.

Table. 2 - Regulatory Risk Matrix - Dam Safety Plan (DSP)

Existence of project documentation	**Organizational structure and qualification**	**Safety inspection and monitoring roadmap procedures**	**Operating rule of dam discharge devices**	**Safety inspection reports with analysis and interpretation**
Has Executive design and "as built" (0)	It has an organizational structure with a technician responsible for the damsafety (0)	Owns and enforces inspection and monitoring procedures (0)	It has or Free Sill Type Spillway (0)	Regularly issues reports (0)
Has Executive design or "as built" (2)	It has a technician responsible for the safety of the dam (4)	Owns and applies only inspection procedures (3)	It does not have (6)	Issues reports without periodicity (3)
Has basic design (4)	It does not have an organizational structure and or professional responsible for the damsafety (8)	It has inspection and monitoring procedures, but does not apply them (5)	-	Does not issue reports (5)
Only a conceptual design of a dam (6)	-	Does not have and does not apply procedures for monitoring and inspections (6)	-	-
No project documentation (8)	-	-	-	-

Low indices in this indicator indicate opportunities for improvement related to the Dam Safety process that could improve the project's score in the risk classification matrix.

Results from this indicator represent the condition of the structures and the effectiveness of the Safety Plan, with potential improvements that may be required by the regulatory agency.

$$IECSB = 1 - \left(\frac{\sum CS\ Score \quad + \sum DSP\ Score}{\sum Maximum\ possible\ Score}\right)$$

Where:

$\sum CS\ Score$ = Summation of the score obtained in the Conservation Status Matrix;

$\sum DSP\ Score$ = Summation of the score obtained in the Dam Safety Plan Matrix;

$\sum Maximum\ possible\ Score$ = Summation of the Maximum possible scores for each item of the corresponding classifications (CS+DSP)

2.5. Range of values and classifications

For the four indicators previously outlined, a scale was developed that extends from 0, representing the least favourable outcome, to 1, indicating the most favourable outcome.

Furthermore, through observational analysis, four categories of results were established, each accompanied by their corresponding ranges and characteristics, as delineated in the table below.

Table 3 - Indicator classification ranges.

Indicator Index	Classifier	Evaluation
1,00 – 0,90	Excellent	Optimal line, from which the deviations of the indexes are considered residual.
0,89 – 0,75	Good	Defined as the target indicated for monitoring. Result assessed as effective, where the necessary process and result improvement opportunities can be implemented within operational routines.
0,74 – 0,60	Insufficient	Classifier with result indicating the need for attention to the process to implement additional efforts to improve results. Demand Specific Action Plan
Below 0,60	Poor	Classifier indicates the need for short-term action to address the activities/anomalies associated with the result. Demand Specific Action Plan

RESULTS OF DAM SAFETY KPI APPLICATION

For each evaluation cycle, it is entirely possible to effectively consolidate the indicator results by thoughtfully considering individual assets, various asset groups, or even encompassing the entire portfolio of assets. Groups can be formed based on a variety of factors, including the geographical locations of the assets, the companies responsible for their operations, or any other relevant and meaningful grouping criteria that may apply. The resulting data can be aggregated using simple averages or weighted by various factors that are deemed more significant or influential, such as the number of instruments available, the total power capacity, the dimensions of the dams involved, or any other pertinent metrics that could enhance the analysis. This comprehensive approach allows for a more nuanced understanding of performance across different segments of the asset portfolio.

This enhanced flexibility in the process of group selection truly enables a more thorough and in-depth analysis that is specifically tailored to the unique context at hand. This approach significantly contributes to an evaluation that more accurately reflects the intricate realities of the company, allowing for a more nuanced understanding of its operations and needs.

This section presents actual and detailed data derived from a total of 12 unique assets, the names of which have been deliberately anonymized for confidentiality reasons and subsequently grouped into four distinct categories based on the operating companies responsible for their management. Fig.3 displayed below illustrates the 12 selected assets, labelled from PP1 to PP12, organized into four clear and distinct groups, which are identified

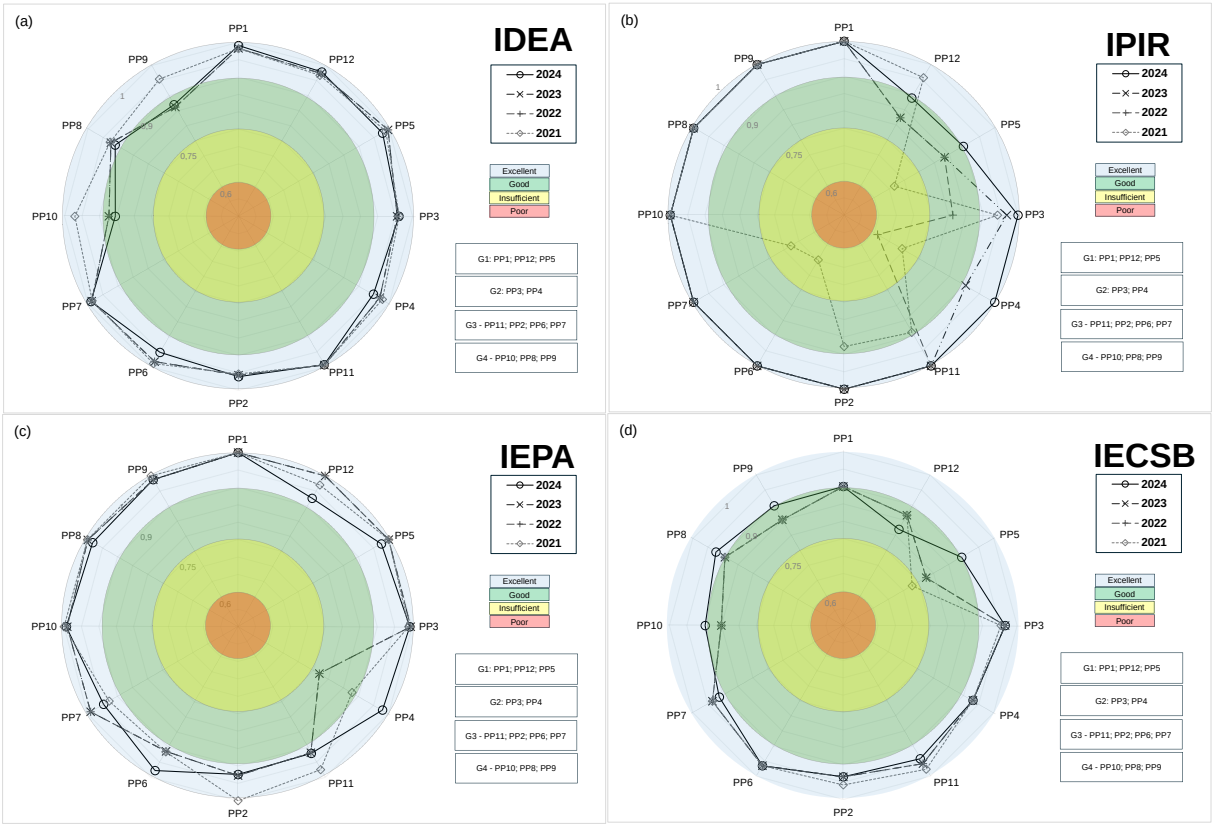
as G1 to G4 for easier reference: G1 comprises PP1, PP12, and PP5; G2 includes PP3 and PP4; G3 consists of PP11, PP2, PP6, and PP7; while G4 encompasses PP10, PP8, and PP9.

Fig. 3 – Separation of Assets by Companies or Groups

2.6. Individual Asset Analysis

This section provides a detailed analysis of each asset based on the performance indicators. Each asset is evaluated independently to identify strengths, weaknesses, and areas that require improvement. The individual analysis allows for a deep dive into the specific characteristics of each asset, considering factors such as operational efficiency, maintenance needs, and compliance with performance targets. This approach ensures a targeted strategy for each asset, facilitating more precise decision-making and resource allocation.

Fig 4 – Individual KPI Results: 2021 to 2024



In Fig. 4 (a), a significant drop in the IDEA Indicator classification from "Excellent" to "Good" is notably identified for the assets PP9 and PP10, which both are categorized under Group G4, with this status being maintained until the year 2024. The underlying reason for this downgrade was thoroughly assessed and traced back to recent changes in the reference values that have been established for certain critical instruments during the Dam Safety Periodic Review that was completed at the conclusion of the year 2021. It has been determined that these instruments are currently operating outside of the defined and acceptable limits that were previously set. This outcome has consequently led to the necessity for conducting an in-depth study, which is currently being developed, to verify and assess the adequacy of the new reference limits in relation to the current conditions of the assets. For the remaining assets under consideration, a temporal stability of results is consistently observed, which remains firmly within the "Excellent" classification.

In Fig. 4 (b), it is distinctly noted that in the years 2021 and 2022, several assets were classified as "Insufficient" based on the IPIR indicator. Specifically, for these assets, which include PP5, PP4, PP6, and PP7, a thorough internal assessment revealed a significant shortage of personnel actively involved in critical activities, as well as identifying opportunities to enhance local process management. This process management prioritization specifically focused on ensuring strict adherence to the established schedules associated with each recommendation put forth. As a direct result of the implementation of a comprehensive action plan, there was a marked improvement in the overall performance of the indicator. By the year 2024, it was determined that all projects had been reclassified as either "Excellent" or were found to be on the borderline between the classification of "Good" and "Excellent," illustrating a noteworthy advancement in asset management and operational efficiency.

In Fig. 4 (c), asset PP4 recorded "Good" IEPA indicator results during the years 2022 and 2023, although these results were very close to reaching the "Insufficient" threshold. A notable deficiency in the number of personnel assigned to handle routine activities for this specific asset was identified and addressed. After strengthening local teams in 2024, a significant improvement in this indicator's results was observed, showcasing the impact of these changes. For the other assets under consideration, fluctuations in performance levels remained consistently within the "Excellent" classification, which is a positive sign of overall stability.

Finally, Fig. 4 (d) illustrates the evolution of the IECSB indicator results spanning from 2021 to 2024. For asset PP5, the initial result in 2021 was categorized as "Insufficient". Following an internal evaluation, a comprehensive action plan was meticulously developed to improve the redundancy systems of the gate operation systems. Upon completing this action plan in 2024, the asset successfully achieved the commendable "Excellent" classification. Overall, a general

improvement in the classifications of the various assets is observed, reflecting a concerted effort toward enhancing operational efficiency and effectiveness.

2.7. Group results over time

The assessment that is based on a variety of groups of projects can also be conducted in a thorough and comprehensive manner. The careful grouping of these projects can be organized by taking into account regional specificities, the different company divisions, or any other types of grouping where it is especially relevant to conduct a collective evaluation, and an in-depth analysis of the safety processes associated with dams. In the specific case that was studied, the results of the performance indicators for the groups established were meticulously derived through the simple average of the individual results of the numerous assets that belong to each respective group.

Fig. 5 provides a presentation of the diverse results obtained for groups G1 through G4 concerning each of the four indicators. This analysis encompasses the entire period from 2021 to 2024 in a thorough and detailed manner.

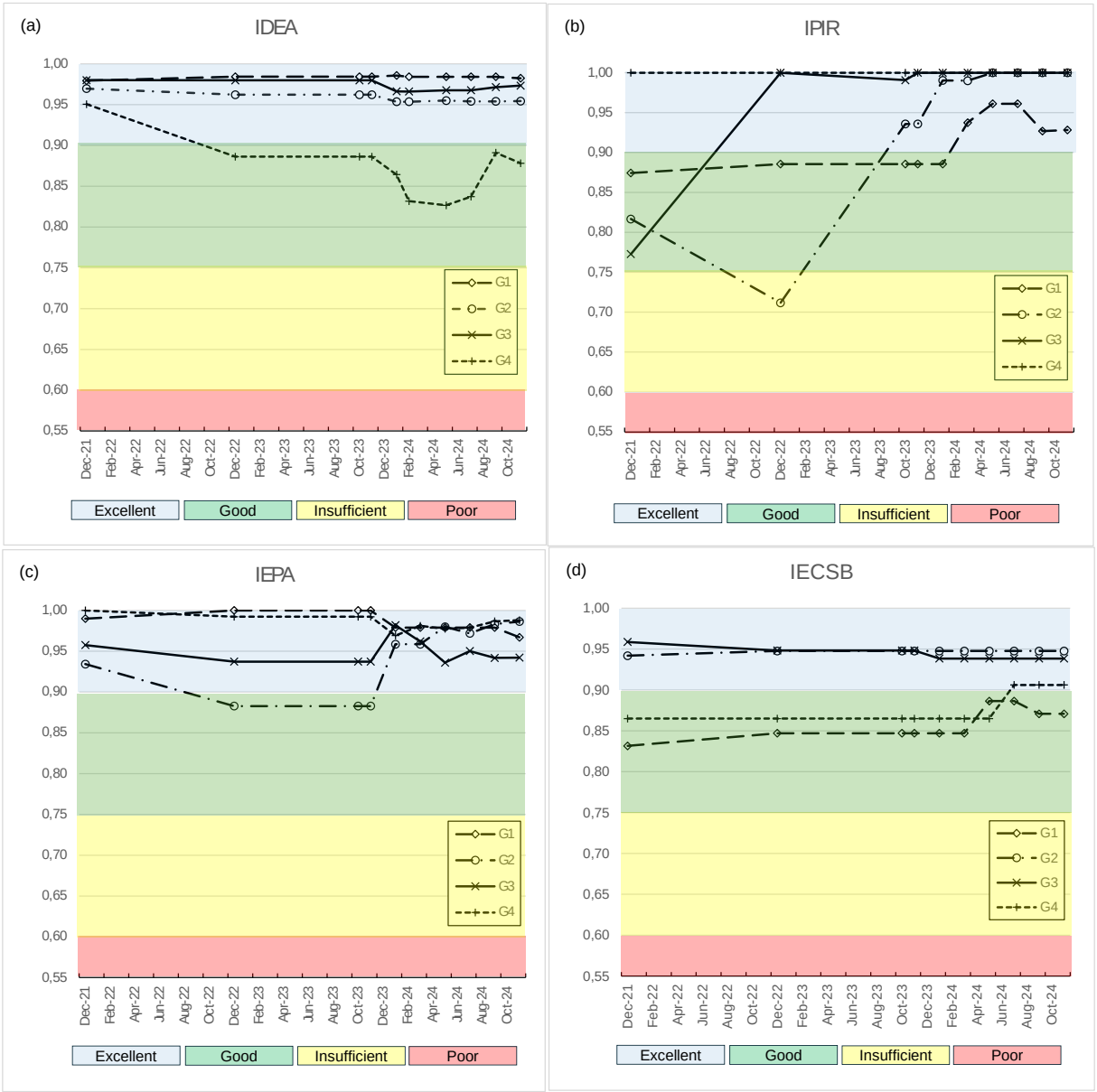
The evaluation cycles of the indicators throughout the years 2021 and 2023 were conducted on an annual assessment frequency. As the teams internalized the measurement processes more effectively by 2023, a significant shift took place, resulting in the consolidation of the results occurring every two months instead of once a year. This transformation allowed for a more regular and detailed analysis of the indicators.

This type of analysis is incredibly valuable as it plays a crucial role in identifying systemic issues that are related to a specific group of assets. For example, as illustrated in Fig. 5(a), from the years 2021 to 2024, a noticeable reduction in the IDEA indicator values can be clearly observed in Group G4, which comprises the assets PP9 and PP10. This decline in the indicator values was directly associated with the recent changes made in the reference limits of certain instruments during the periodic dam safety review process, as it was reported in the Section 3.1. Another compelling example can be seen in Fig. 5(b), where a significant improvement in the results of the IPIR indicator was observed from 2021 through to 2024. This trend demonstrates that the implementation of well-structured action plans had a positive and meaningful influence on the timely fulfillment of the various recommendation response plans put in place.

However, it is important to emphasize that very specific negative outcomes stemming from a particular asset, which do not necessarily signify any systemic issues within a company or an entire group, can often be masked by the overall set of positive results achieved by the group. Consequently, group analysis or evaluation does not serve as a substitute for the individual and detailed assessment of each asset. A pertinent example to illustrate this point is Group

G2, which underwent a reduction in its performance classification, dropping from the "Excellent" category to the "Good" category during the years 2022 and 2023. Nonetheless, as elaborated in Section 3.1, it is crucial to note that asset PP4 experienced a far more substantial decline in performance relative to the overall performance of the group, coming precariously close to falling into the "Insufficient" classification. This situation exemplifies the importance of conducting meticulous individual evaluations alongside broader group analyses to fully understand performance nuances.

Fig 5 – KPI Group results over time: 2021 to 2024



3. CONCLUSIONS

The analysis of dam safety indicators presented in this study provides a comprehensive view of the performance of assets over a four-year period (2021-2024), allowing for the identification of evolution patterns and critical areas that required specific interventions. The use of indicators such as IDEA, IPIR, IEPA, and IECSB proved essential for objectively assessing structural, operational, and safety management aspects, providing key insights for decision-making.

The approach based on groups of ventures, such as those analysed (G1 to G4), stood out as an efficient tool for understanding collective dynamics and identifying regional or sectoral trends in dam safety processes. This methodology demonstrated the usefulness of the simple average of individual indicators to create an overview that facilitates the prioritization of corrective and preventive actions at a macro level. Furthermore, the temporal analysis highlighted that sustained improvements in safety depend on continuous strategic planning, coupled with the systematic evaluation of assets.

The results pointed to significant progress in assets initially classified as “Insufficient.” The implementation of specific action plans, such as strengthening local teams and revising reference limits for critical instruments, proved to be an effective approach in reversing unfavourable classifications and ensuring greater operational reliability. Examples of this include assets PP5 and PP4, which, after focused interventions, evolved to performance levels classified as “Excellent” or close to this classification.

Despite the observed advances, the study also revealed the importance of maintaining continuous monitoring and robust management of human and technological resources, strengthening these aspects throughout the analysed period was decisive for the overall improvement in asset classifications.

Finally, this study reinforces the importance of integrating individual and collective assessments in dam safety management. The use of well-structured indicators, combined with temporal and group approaches, proved to be a robust strategy for improving the safety and reliability of dam infrastructures. The need for continued evaluation efforts and adaptation to new realities is reiterated, ensuring that reference limits and analysis methods remain aligned with the operational and environmental conditions of the assessed assets.

REFERENCES

[1] ENAP – National School of Public Administration (2021). Dam Safety Management – International Experiences (in Portuguese).

[2] BRAZIL. Law No. 12.334, of September 2010. Presidency of the Republic - Civil House - Subsecretariat for Legal Affairs. Available at: https://www.planalto.gov.br/ccivil_03/_ato2007-2010/2010/lei/l12334.htm. Accessed on: Jan. 14, 2025.

[3] ELETROBRAS. Annual Report 2023. Available at: https://eletrobras.com/en/Documents/Annual-Report-2023.pdf. Accessed on: Jan. 14, 2025.

[4] ANEEL – National Electric Energy Agency (2023). Normative Resolution No. 1.064 of May 2, 2023. Brasília-DF.

[5] CBDB – Brazilian Committee on Dams. Basic Dam Safety Guide (in Portuguese). São Paulo, SP, 2001.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

PLANNING THE SEMI-ARID REGION: LAND USE CAPABILITY ALLIED TO UNDERGROUND DAMS AS A MEASURE FOR MITIGATING DETERIORATION

Danilo Duarte Costa e Silva [1]
Universidade Federal de Goiás

ABSTRACT

Over the past years, although the building of dams is still the chosen solution for tackling the water - deficit of the Brazilian semi-arid region, this reality has been changing with alternative water harvesting techniques being disseminated in the region. Among such alternative techniques, research works have been recommending the introduction of an alternative water infrastructure known as the underground dam, which is developed through the obstruction of ground water using impervious materials. Despite the advantages obtained with the introduction of underground dams, issues may arise if their building sites are not properly assessed, posing a threat to the environment. The present study analyses a methodology for planning the introduction of underground dams in the Brazilian semi-arid region through data obtained from Land Use Capability maps. After calculations, it has been observed that the area classified as agricultural land is the most adequate for the introduction of underground dams.

Keywords: Management, Land Capability Map, Underground dam, Semi-arid region, Watershed.

.

[1] daniloduarte@ufg.br, Brazil

1. INTRODUCTION

Currently, although most of the infrastructure directed to tackle the water-deficit in the semi-arid region of the Brazilian northeast are water dams [1], such reality has changed and in recent years the introduction of rainwater harvesting techniques has been increased. Plate water tanks, mesh water tanks, ferrocement water tanks, dug wells and underground dams ([2], [3]) are some examples of alternative rainwater harvesting technologies that are currently in use within the region. Among these technologies, the underground dam can be highlighted due to its easiness of construction, low cost (in relation to other techniques) and the maintenance of the captured water amidst high evaporation rates environments ([4], [5]). Gnadlinger [2] classifies the underground dam as a technology directly linked to the increase in agricultural production. Brito et al. [5] highlights the underground dam as an alternative capable of enabling agriculture in the semi-arid region of Brazil, increasing productivity. Cirilo
[6] states that this technique, from a socio-economic stand, is perfectly suited to the semi-arid conditions of the Brazilian northeast, and may become one of the best alternatives to improve the earnings and living conditions of low-income families in the area.

Amid this context, even though underground water dams are being used as a support to agricultural activity, it can be noticed that only few researches have been carried out aiming at defining the best sites to introduce this technology. Considering the overall context of introduction of this technology at already deteriorated environments, that may lead to an intensification of the issues already present in the area. That is, introducing an underground dam (for agricultural use) where soil fitness is not proper for agriculture may increase local deterioration, culminating in erosion problems, which in turn contribute to the desertification of the semi-arid region [7]. Rocha & Kurtz [8] point out that the inadequate use of land (different from its suitability) causes conflicts that are among the main agents responsible for erosion, siltation of dams, rivers and water reservoirs, floods and drought effects.

Therefore, if, on the one hand, building an underground dam can sometimes mitigate a problem (tackling water scarcity and enhancing agriculture), on the other, it can contribute to the deterioration of the semi-arid region (by giving support to agricultural activities in unsuitable land). Based on the aforementioned, there is need for coherent planning before introducing such technologies, so as not to contribute to the deterioration of fragile environments.

As an alternative for soil conservation in deteriorated areas (such as those undergoing desertification), Bertoni & Lombardi Netto [7] bring up the land use capability map , which is devised in a way that indicates land suitability and, thus, mitigates land misuse from a conservationist point of view. Araújo [1] highlights the social need to discipline land use, since its misuse implies in reducing water availability, which can have severe consequences,

especially in semi-arid regions. Rocha [9] presents a methodology for elaboration of land use capability maps (an integral part of the integrated watershed management approach). This methodology has already found some applications within the semi-arid region of Brazil ([10], [11]), but has been little used as a mechanism for planning the location of underground dams . This methodology has the particularity of drawing the land use capability map from the roughness coefficient, regarded by Pereira Filho ([13], apud [14]) as being equivalent in 99.9% of the cases to the land use capability .Hence, using the roughness coefficient for the elaboration of a land use capability map is a faster and economical method that does not require the collection of field samples.

Considering the above, in spite of the dissemination of studies based on the use of the survey of land use capability as subsidy to environmental planning ([15], [16], [17]), it is clear that their use as a management index for the introduction of water harvesting techniques for the agricultural sector is still precarious.

The present article aims to analyse the effectiveness of elaborating the land use capability map aiming at the subsequent introduction of an underground dam for agricultural purposes in the Brazilian semi-arid region.

2. METHODOLOGY

The delimited subwatershed for the present study is part of the Piranhas-Açu watershed and is located in the rural area of the municipality of Currais Novos (RN), in the Brazilian semi-arid region, with an area of approximately 16,566 ha, as shown in Figure 1 below:

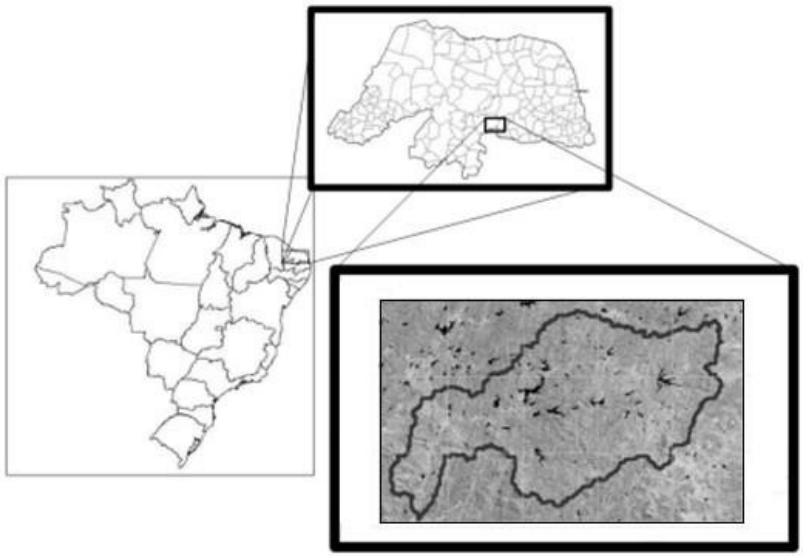


Fig. 1 - Location of the subwatershed in the rural area of Currais Novos – RN - Brazil

The methodology used in this study is that proposed by Rocha [9] and by Rocha & Kurtz [8], with its application being focused on predominantly rural watersheds, subwatersheds and micro-watersheds. The development of the land use capability map is based on physical-conservationist diagnosis methodology, developed from the roughness coefficient. The roughness coefficient is evaluated for each compartment of the subwatershed and in order to do so, it is necessary to split the subwatershed in smaller compartments named micro-watersheds, with this procedure taking in consideration the hydrographic network and the local terrain.

2.1 Elaboration of the land use capability map based on the physical conservationist diagnosis

The land use capability map, according to the physical-conservationist diagnosis [9] is developed from the calculation of the roughness coefficient, given by the following equation:

$$\mathrm{RN} = \mathrm{D\ X\ H} \quad (1)$$

Where:

RN = roughness coefficient (dimensionless).

D = drainage density of the evaluated compartment, in km/ha.

H = average slope of the evaluated compartment, in %.

The average slope is found based on information from the corresponding information plan. The use of planialtimetric charts is one of the sources of information that can be used to find the average slope. The calculation of the average slope is given by the expression:

$$H = (\Sigma LCN\ x\ \Delta h\ /\ A)\ x\ 100 \quad (2)$$

Where:

H = average slope, in %.

ΣLCN = sum of lengths of all contour lines in the mapped Micro-watershed, in hm or m.

A = area of the Micro-watershed, in ha (or m2).

Δh = the equidistance between contour lines, in hm (or m).

Regarding the drainage density “D” (in a similar way to what occurs with the average slope) it is based on the existence of an information plan (in this case, planialtimetric charts) for calculation. This is given by the following expression:

$$D = \Sigma (R, C, T) / A \quad (3)$$

Where:

ΣR,C,T = sum of lengths of ravines, channels and tributaries of each soil unit, in km.

A = area of compartment, in ha.

D = drainage density, in km/ha.

Once the average slope and drainage density values are found, multiplication is performed (according to Equation 1) to find the roughness coefficient. The roughness coefficients are found for each micro-watershed (sub-watershed divisions), therefore having different values corresponding to the micro-watershed number. Once the specific values for each micro-watershed are found, the amplitude is calculated according to the following equation:

$$A = Highest\ RN - Lowest\ RN \quad (4)$$

Where:

RN = Roughness coefficient.

After obtaining the amplitude value, for obtaining the interval values all that has to be done is divide the amplitude by four. Thus, the intervals are given by the following equation:

$$I = A/4 \quad (5)$$

Where:

A = Amplitude.

After the intervals are found, the lowest interval value is classified as being suitable for agriculture (A), followed by pasture (B), pasture/afforestation (C) and, finally, class (D) as suitable for afforestation. With the conclusion of this procedure, the initial roughness coefficient values are observed and classified according to the intervals proposed by the application of Equation 5. The classification of each micro-watershed according to the intervals is the final step for composing the land use capability map.

2.2 Final Stage of Underground Dam Site Selection

As the final step in selecting the underground dam location, key parameters were applied to determine the most suitable site within Brazil's semi-arid micro-basin [14]:

1. Climate & Streams
2. Topography & Geology
3. Hydrogeology
4. Rainfall & Evaporation

3. RESULTS AND DISCUSSION

3.1. Division of the Subwatershed

Before anything else, in order to execute the division of the area into micro-watersheds it was necessary to delimit the larger area (Figure 2, below) i.e. the own subwatershed. To this end, georeferencing was initially carried out based on known coordinates. Subsequently, the image was exported to the ArcGis software where the subwatershed was delimited with the extension rhec_goohmf (according to Figure 3 below).

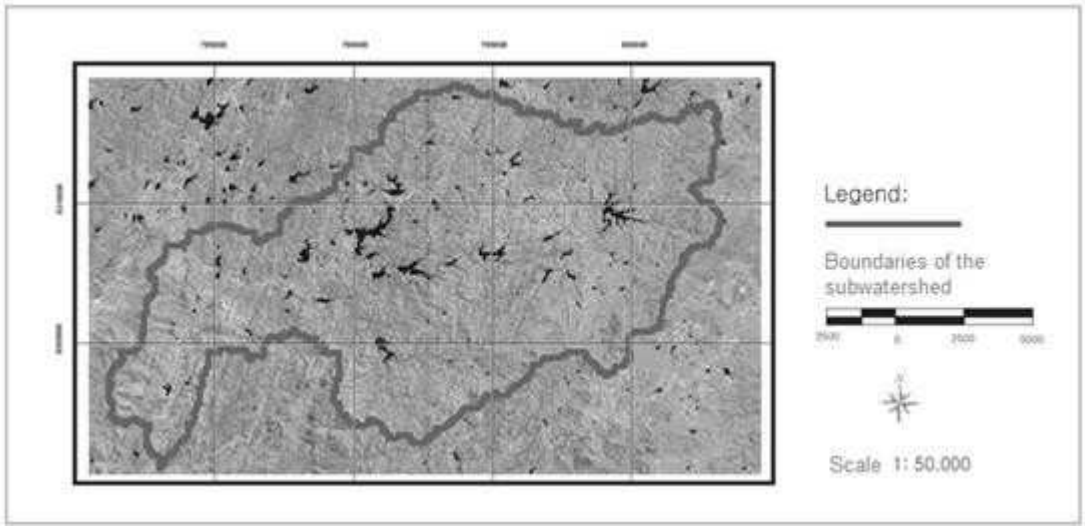


Fig. 2 - Delimitation of the Subwatershed

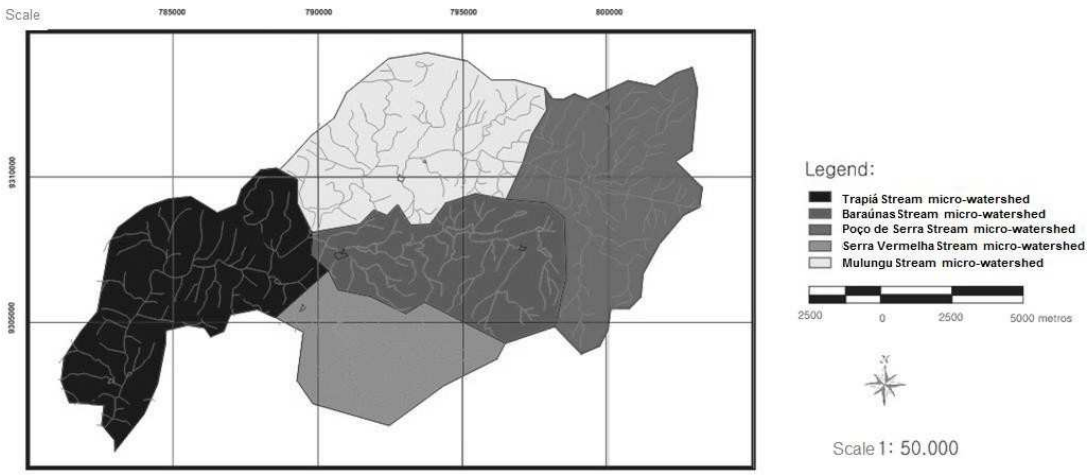


Fig. 3 - Map displaying the subdivisions of the subwatershed in rural micro-watersheds in Currais Novos (RN)

3.2 Elaboration of the Land Use Capability map

After the division in micro-watersheds, roughness coefficients were calculated for each subdivision. Since the roughness coefficient is given by the multiplication of the drainage density by the average slope, these two quantities were calculated prior to the roughness coefficient.

3.2.1 Evaluation of the average slope

In order to obtain the first required component for calculating the roughness coefficient (average slope), a digitized topographic chart[1] on a scale of 1: 100,000 was used, and using AUTOCAD software it was possible to select the contour lines and divide them by the micro-watersheds. After dividing the contour lines of each compartment, the results were summed. Once the sum of the contour lines of each compartment was evaluated, the result was multiplied by 50 m (the equidistance between contour lines as described in Equation 02 - "Δh") and divided by the respective areas of each compartment. Table 1 below presents the results of the average slope for each compartment:

Table 1 - Average slope of the micro-watershed by compartments

Micro-watershed	Average slope Result (%)
1	14.3072
2	12.3827
3	15.8773
4	13.8911
5	14.3116

[1] [18]

3.2.2 Evaluation of the drainage density

The drainage density of each compartment was obtained from the digitalized topographic chart on a scale of 1: 100,000, where initially the drainage network by compartment was measured (using AUTOCAD software) and later divided by the area. Table 2 displays the results by compartment:

Table 2. Drainage density of the micro-watershed by compartment

Micro-watershed	Drainage density
1	0.0198
2	0.0195
3	0.0194
4	0.0203
5	0.0183

3.2.3 Evaluation of the Roughness Coefficient

Once the values of average slope and drainage density were calculated, the next step was to multiply these two variables in order to obtain the roughness coefficient. Table 3 shows the values of roughness coefficient for each micro-watershed.

Table 3 - Evaluation of the roughness coefficient

Roughness coefficient (RN) = (Dm x Dd)				
Micro-watershed - 1 RN = 0.2717	Micro-watershed – 2 RN = 0.23522	Micro-watershed – 3 RN = 0.30153	Micro-watershed – 4 RN = 0.2762	Micro-watershed – 5 RN = 0.25758
RN x 10^2 = 27.17	RN x 10^2= 23.522	RN x 10^2= 30.153	RN x 10^2= 27.62	RN x 10^2= 25.758

3.2.4 Land Use Capability Map

After calculating the roughness coefficient values, the greater interval was calculated in accordance with Equation 4 (by subtracting the lowest roughness coefficient from the highest roughness coefficient) and divided by 4 to establish the specific intervals, so as to compose the land use capability of each micro-watershed.

Tables 4, 5 and 6 show the calculation sequence executed for enabling the elaboration of the land use capability map.

Table 4 - Analysis of land use capability

Analysis of land use capability	
Highest roughness coefficient (RN1)	30.15
Lowest roughness coefficient (RN2)	23.52
RN1-RN2	6.63
(RN1-RN2)/4	1.66
First Interval A	25.18
Second Interval B	26.84
Third Interval C	28.49
Fourth Interval D	30.15

Table 5 - Intervals of land use capability

Intervals	
23.5-25.1	A (Agriculture)
25.2-26.8	B (Pasture)
26.9-28.4	C (Pasture/Afforestation)
28.5-30.1	D (Afforestation)

Table 6 - Classification of Micro-watersheds

Classification of land use capability	
Micro-watershed 1	B (Pasture)
Micro-watershed 2	A (Agriculture)
Micro-watershed 3	D (Afforestation)
Micro-watershed 4	C (Pasture/Afforestation)
Micro-watershed 5	B (Pasture)

3.3 Assessment of the Baraúnas Stream for Underground Dam Implementation

After selecting the agriculturally suitable area (Baraúnas Stream) key parameters were applied to determine the best location for an underground dam in Brazil's semi-arid sub-basin [14].

1. Climate & Streams: Irregular rainfall occurs in a few months, with temporary streams enabling periodic recharge and intermittent rivers supporting water storage.

2. Topography & Geology: Stream narrowing enhances retention, while latosols, neosols, and crystalline rock formations require site-specific selection to prevent infiltration.

3. Hydrogeology: Low-capacity aquifers make underground dams essential, with natural rock barriers reducing water loss.

4. Rainfall & Evaporation: Precipitation varies from 400-800 mm annually, with high evaporation requiring shaded areas. Seasonal rainfall ensures recharge during wet months.

The construction of **underground dams** was carried out at coordinates **S 6º 15'27.7'' and W 36º 20'13''**.

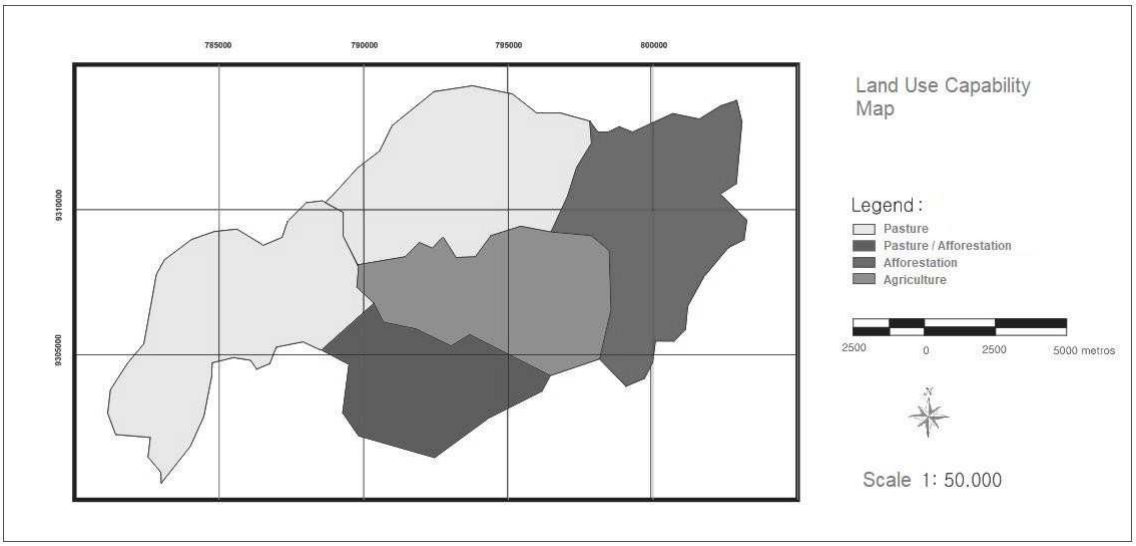


Fig. 4 - Land use capability map

Figure 4 above shows the land use map (result of applying the methodology). In this map, the region classified as agriculture was chosen for the introduction of the underground dam.

3.4 Management of water resources through the application of an agricultural technique

Based on the suitability found (for agriculture) on the land use capability map and other parameters [3.3 above], an area was selected within the Baraúnas Stream micro-watershed for building an underground dam. The construction was carried out at coordinates S 6º 15'27 7" and W 36º 20'13, in the rural area of Currais Novos, Rio Grande do Norte, Brazil, within the semi-arid region of the Northeast. The underground dam was built with a **depth of 5 meters and a cross-sectional cut of 30 meters,** as shown in Figure 5 (below).

Fig. 5 - Management of water resources through application of an agricultural technique

4. CONCLUSIONS

The use of the land use capability map (derived from the methodology for integrated management of watersheds) as an instrument responsible for defining the location where to build an underground dam proved to be effective, first due to the fact that since the methodology is based on micro-watersheds (composed of rivers, streams and brooks) it is suitable for the most likely places where an underground dam is normally introduced (characterized by barring the flow of water from rivers, streams and brooks through an underground obstacle). Secondly, the very environmental consequence of planning the introduction of an underground dam by consulting a land use capability map, adapting a technique that will contribute to the agricultural activity in regions that are suitable for cultivation. It can be noticed that given the physical and human particularities of the Brazilian semi-arid region, the re-introduction of underground dams guided by the elaboration of the land use capability map can constitute a powerful alternative to contribute to the mitigation of water-scarcity

REFERENCES

[1] ARAÚJO, José Carlos. As barragens de contenção de sedimentos para conservação de solo e água no semi-árido. In: KÜSTER, Angela; MARTÍ, Jaime Ferré; MELCHERS, Ingo (Org.). **Tecnologias Apropriadas para Terras Secas:** Manejo sustentável de recursos naturais em regiões semi-áridas no Nordeste do Brasil. Fortaleza: Fundação Konrad Adenauer e Gesellschaft Für Technische Zusammenarbeit, 2006, 212 p.

[2] GNADLINGER, Johann. Tecnologias de captação e manejo de água de chuva em regiões semi-áridas. In: KÜSTER, Angela; MARTÍ, Jaime Ferré; MELCHERS, Ingo (Org.).**Tecnologias Apropriadas para Terras Secas:** Manejo sustentável de recursos naturais em regiões semi-áridas no Nordeste do Brasil. Fortaleza: Fundação Konrad Adenauer e Gesellschaft Für Technische Zusammenarbeit, 2006, 212 p.

[3] CAMPELLO NETTO, M. S. C., COSTA, M. R. & CABRAL, J.J.S.P. Manejo integrado de água no semi-árido brasileiro.In: CIRILO, J. A. et al. (Org.) O uso sustentável dos recursos hídricos em regiões semi-áridas. Recife: ABRH – Editora Universitária UFPE, 2007. p.508.

[4] ONDER, H.; YILMAZ, M. Underground Dams. European Water, Ankara, n. , 2005, p.35-45.

[5] BRITO, L. T.et al. TECHNOLOGICAL ALTERNATIVE TO INCREASE WATER AVAILABILITY IN SEMI-ARID REGION OF BRAZIL. Campina Grande, PB. Revista Brasileira de Engenharia Agrícola e Ambiental, v.3, n.1, 1999, p.111-115.

[6] CIRILO J. A. ET AL.. Soluções para o Suprimento de Água de Comunidades Rurais Difusas no Semiárido Brasileiro: Avaliação das Barragens Subterrâneas. Brazilian Journal of Water Resources, 2003, (8) 4: p. 5-24.

[7] BERTONI, J. E LOMBARDI NETO, F. Conservação do solo. São Paulo: Ícone, – 4th edition, 1999.

[8] ROCHA, J. S. M. e KURTZ, S. M. J. M. Manual de Manejo Integrado de Bacias Hidrográficas, Santa Maria: Edições UFSM CCR/UFSM, 2007.

[9] ROCHA, J. S. M. da. Manual de projetos ambientais, Santa Maria: Imprensa Universitária, 1997.

[10] BARACUHY, J.G.V. Integrated management of hydrographic basins in the northeastern semi-arid region: a case study. Campina Grande: UFPB, 2001, 221p. Doctoral Dissertation

[11] MENDONÇA, Izaque Francisco Candeia de. Adequação do uso agricola e estimativa da degradação ambiental das terras da microbacia hidrografica do riacho Una, Sape-PB.

2005. 158p. Tese (doutorado) - Universidade Estadual de Campinas, Faculdade de Engenharia Agricola, Campinas, SP.

[12] UNITED NATIONS CONVENTION TO COMBAT DESERTIFICATION – UNCCD – 3rd edition. Brasília: Ministry of the Environment – MMA, s/d

[13] PEREIRA FILHO, W. Capacidade de uso da terra em função do coeficiente de rugosidade. 1986. 48p. Monografia (Especialização em Interpretação de Imagens Orbitais e Suborbitais) – Universidade Federal de Santa Maria, Santa Maria, 1986.

[14] BARACUHY, J.G.V. Técnicas Agrícolas para contenção de solo e água. Impressos Adilson, Campina Grande, 2003.

[15] SOUSA, F.A. & LIMA, C.V. Capacidade de uso das terras como suporte ao planejamento ambiental na bacia hidrográfica do ribeirão Santo Antônio – Iporá-Go. Boletim Goiano de Geografia Goiânia - Goiás - Brazil v. 27 n. 3 p. 91-101 jul. / dez. 2007

[16] TORRES, J. L. R., BARRETO, A.C & PAULA, J.C. Capacidade de uso das terras como subsídio para o planejamento da microbacia do córrego lanhoso, em Uberaba (MG). Caminhos de Geografia, Uberlândia v. 8, n. 24 DEZ/2007 p. 22 – 32.

[17] MENDONCA, Izaque F. C.; LOMBARDI NETO, Francisco; VIEGAS, Ricardo A. Classification of land use capacity of micro basin of Una stream in Sapé, PB, Brazil. Rev. bras. eng. agríc. ambient., Campina Grande, v. 10, n. 4, Dec. 2006 Available at <http://www.scielo.br/scielo.php?script=sci_arttext&pid=S1415-43662006000400016&lng=en&nrm=iso>. access on Mar 11, 2012. http://dx.doi.org/10.1590/S1415-43662006000400016.

[18] SRHRN. Cerro-Corá topographic chart, page : SB. 24 - Z- B - III, scale 1:100.000, from the Pró-agua Semi-árido programme of the Water Resources Secretariat of the state of Rio Grande do Norte, Brazil, 2003 [CD].

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

21ST CENTURY CLIMATE CHALLENGES AND DEBRIS DISASTERS: ASSESSING LEGACY DAMS AMID RECORD CLIMATE EVENTS

Captain Jared McCully[1]
Worthington Products

Paul Meeks[2]
Worthington Products

Paul Rollins[3]
Worthington Products

ABSTRACT

In the aftermath of the debris blockage at the Rapidan Dam in the summer of 2024, the conversation resurfaces, requiring dam operators globally to assess whether historical precipitation and debris models can be reliable for future events. This paper analyzes debris-related incidents at legacy dams built using outdated climate models and predictions from the 20th century.

Dams worldwide, including the Rapidan Dam, are now confronted with pressing threats of record precipitation events and the accumulation of debris clogging spillways, reducing storage capacity, and increasing pressure on dam structures, elevating the risk of overtopping and structural failure. These legacy dams, constructed before the 1970s, relied on historical hydrological data, proven inaccurate in current precipitation patterns. This discrepancy poses significant risks to these aging infrastructures' structural integrity and operational functionality. The paper identifies specific dams at risk, emphasizing the heightened vulnerability of those constructed without consideration of present-day climate anomalies. Increased precipitation and more frequent storms lead to higher volumes of debris being transported into dam reservoirs. This debris can obstruct critical inspection and maintenance activities, compromising dam integrity.

To mitigate the risks, the paper recommends a comprehensive reassessment of legacy dams, incorporating updated climate predictions and debris management strategies. Proactive measures, including installing advanced monitoring systems and implementing public safety and debris barriers, are essential to enhance the resilience of these critical infrastructures. The analysis underscores the urgent need for investment in dam safety to address the emerging challenges posed by climate change and debris accumulation. The paper highlights the importance of adaptive management strategies by examining the impact of contemporary climate predictions on legacy dams, explicitly focusing on the role of debris accumulation. These strategies are critical for ensuring dams' long-term safety and functionality, particularly in the face of evolving climate patterns and the increasing frequency of extreme weather events. The findings advocate for an integrated approach to dam safety that includes updated hydrological data, advanced engineering practices, and robust maintenance protocols.

Keywords: dam safety, debris management, climate resilience, legacy dams, spillway discharge

[1] jared@tuffboom.com USA

1. INTRODUCTION – A WORLD CHANGING WITH DAMS AGING

Dam safety represents an essential global water resource management pillar, bridging critical domains of structural integrity, operational efficiency, and environmental stewardship. The reliance on dams for various functions, ranging from irrigation, flood mitigation, and municipal water supply to hydropower, underscores their critical nature to our society. Sole reliance on hydropower is tempered by the growing vulnerabilities inherent in aging infrastructure and the cascading impacts of climate change. With a collected perspective of three-plus decades of increasing even frequency, focusing on the increasing incidence of debris accumulation and its implications for dam safety and climate resilience. Dams serve as multi-functional infrastructure, regulating water systems to accommodate the needs of industry, agriculture, and urban populations. Structural failures or operational inefficiencies can result in catastrophic consequences, including loss of life, displacement of communities, environmental degradation, and long- term economic setbacks. Most dams worldwide were constructed during the post-World War II industrial expansion, characterized by large-scale infrastructure projects designed to meet burgeoning urban and agricultural demands. Many of these structures, built with a projected lifespan of 50 to 100 years, are approaching or exceeding their design limits. This is particularly evident in the United States, where over 85% of dams will be over 50 years old by 2030 [1]. As is the human body, aging dams are increasingly vulnerable to wear and tear, specifically material degradation, sediment accumulation, and structural stress. Aging infrastructure compounds risks during extreme weather events. Dams constructed under outdated design standards cannot often accommodate increased hydrological variability caused by climate change. The inability of legacy infrastructure to adapt to modern conditions highlights a critical need for proactive rehabilitation and retrofitting programs.

Climate change and extreme weather events have emerged as defining challenges for dam safety in the 21st century, amplifying existing vulnerabilities and introducing new threats. Changes in precipitation patterns, the intensification of extreme weather events, and the increased variability of hydrological cycles have profoundly altered the standard operating procedures for dam owners. Probable Maximum Flood (PMF) rainfall events have become more frequent and intense, contributing to higher runoff rates and more significant sediment and debris transport. Prolonged droughts, on the other hand, reduce reservoir levels, complicating the balance between water storage and downstream flow requirements [2]. The secondary effects of climate change further exacerbate risks. Wildfires, which have increased in frequency and severity in regions such as California, destabilize upstream catchments, leading to heightened erosion and debris flows during subsequent rainfall. Similarly, landslides triggered by saturated soil conditions can deposit significant debris into reservoirs. A complex balance of monitoring, communication, and multiagency and international communication ills the community of dam safety activists [3]. There

is never just one symptom, but factors collectively demand an adaptive approach to dam design and management, incorporating climate resilience as a core principle.

1.1 Extreme events with extreme consequences

Debris accumulation represents one of the most pressing yet often overlooked threats to dam safety. Floating debris, ranging from tree trunks and branches to vegetation and anthropogenic materials, poses significant operational and structural challenges. Accumulated debris obstructs spillways, reducing their discharge capacity and leading to elevated reservoir water surface elevations (WSE). When combined with extreme inflow conditions, this phenomenon dramatically increases the risk of overtopping, which remains one of the leading causes of dam failures worldwide. Recent events, such as the 2024 near-miss at Rapidan Dam caused by extreme precipitation and debris blockage and the challenges faced by dams during Hurricane Helene, underscore the critical need for effective debris management and monitoring strategies. While still developing a fuller understanding, the incidents reveal how inadequate debris mitigation can amplify structural vulnerabilities, necessitating advanced approaches in design, monitoring, and operational responses to safeguard dam infrastructure under modern climatic extremes. Today's engineers and operators must respond and troubleshoot our unfortunate miscalculations of tomorrow's climate Despite the challenges of debris in dams worldwide, the collected studies and challenges faced in the subfield of debris management unfortunately dispersed to regional agencies and authorities. The multiple ICOLD bulletins have sought to address the concern of debris mitigation and spillway effects but often are confined to regional representation that may struggle to encompass or attract the global audience. A limitation of this study and prescriptive analysis is the dearth of peer incorporation from significant debris and climate-challenged regions of Asia. For the backdrop of the analysis, empirical studies, including the USBR Spillway Debris Physical Model Study provide critical insights into the mechanics of debris accumulation and its impacts [4].

1.2 The intersection of aging infrastructure dilemma and climate challenges

Most dams globally were constructed during the mid-20th century to meet the demands of urbanization, agriculture, and hydropower generation. For instance, the average damage in the United States is 57 years, with nearly 17% classified as a high hazard [1]. These aging structures are particularly vulnerable to wear and tear, exacerbated by material degradation, changing hydraulic forces, and the limitations of design standards that did not anticipate contemporary climatic extremes. In any risk analysis, the predictive capabilities of infrastructural capacity are only designed to withstand the projections that could only be considered at the time with available data.

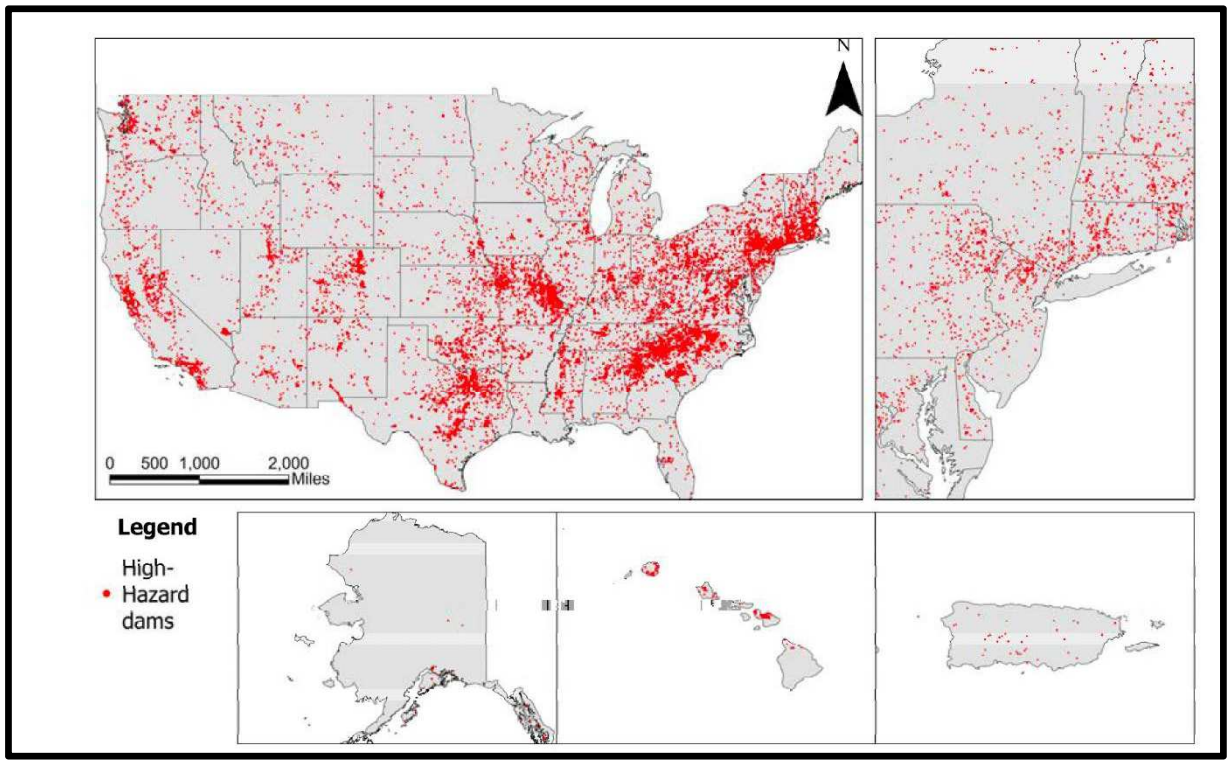


Fig .1 ***Modernizing Probable Maximum Precipitation Estimation (2024) [3]***

Case studies have highlighted catastrophic failures resulting from infrastructure vulnerabilities. For instance, the Palagnedra Dam blockage, inundated by 25,000 cubic meters of debris, underscores the severe impact of aging infrastructure when combined with unanticipated debris accumulation. These incidents underscore the urgent need to integrate advanced monitoring systems and revisit strategies to enhance the lifespan and performance of legacy dams [5].

Climate change has amplified the frequency and intensity of extreme weather events, with cascading effects on water resource infrastructure. The USBR Spillway Debris Physical Model Study underscores that heightened precipitation variability accelerates the transport of woody debris, sediment, and anthropogenic materials into reservoirs. Several primary considerations exist from the analysis of the United States Bureau of Reclamation, USBR, conducting laboratory tests on this modeling topic. Key drivers identified include:

I. **Severe Rainfall Events**: Rising global temperatures intensify atmospheric moisture retention, resulting in extreme downpours. Increased runoff mobilizes debris from upstream catchments, creating challenges for spillway operations.

II. **Post-Wildfire Erosion**: Burned landscapes, weakened by wildfires, contribute substantial debris volumes during subsequent rainfall events. Notable examples include dams in California affected by the 2020 wildfire season. In previous years, Wildfires across Tennessee and North Carolina provided the perfect recipe for debris issues during significant storm events in 2024.

III. **Slope Instability and Landslides**: Climate-induced saturation destabilizes slopes, leading to landslides that deposit large debris loads into reservoirs [4].

Presented in various cases, the report will emphasize the need for predictive tools and resilient designs to address the compound impacts of climate and debris flows. Floating debris presents a multifaceted challenge to dam safety, affecting spillway efficiency, structural stability, and downstream ecosystems. The Swiss Committee on Dams Report provides comprehensive metrics for assessing spillway vulnerability to debris blockages:

a) **Debris Blocking Probability**: This parameter is influenced by trunk size, buoyancy, and spillway geometry. For instance, logs exceeding 80% of spillway width are deemed high-risk for obstructions.

b) **Reservoir Backwater Effects**: Obstructions can elevate WSE by up to 30%, heightening the risk of overtopping.

c) **Hydraulic Loads**: Accumulated debris amplifies stresses on piers and gates, potentially leading to structural failures. Multiple bridges and access roads were severely damaged or washed away during the 2024 river surges across North Carolina.

Fig .2 ***Debris Blockage USBR Laboratory, Demonstrating reduced spill gate capacity with simulated debris blockage of 80% [5]***

2. EVENTS HAPPENING IN REAL TIME

The first of two recent storm-related events in the United States rejuvenated the discussion of the country's infrastructure readiness and ability to adapt to evolving climate challenges. The first of 2024 swept the Midwest by surprise, with one event catching significant national attention: the Rapidan Dam. In a matter of hours, the real-time coverage of the Rapidan Dam's partial failure may not have been the most devastating flood event in the US, but it certainly raised alarms on aging infrastructure in the face of increasingly taxing extreme weather [6]. Months after the notorious images of Rapidan, the Southern United States was severely struck by Hurricane Helene. Traditionally, coastal regions are the primary coverage of damage and disaster recovery, but the worst devastation came across hundreds of miles inland at dams and reservoirs

throughout North Carolina and Tennessee. With over 56 dams overtopping and an estimated 75 requiring removal following the historical storm and rainfall, Hurricane Helene, and the extreme rainfall events that followed [7]. While only a select case of the many extreme hydrological events that impacted dams in 2024, the relevance and widely broadcasted case can not only serve as excellent resources for lessons learned in future dam engineering but also raise the call to action to prepare operators to mitigate these pending threats before it is too late. In the case studies from the United States, dam safety and emergency planning are often more developed and codified than in some parts of the world but still experience lapses and challenges of readiness when confronted with extreme events. The prescriptive approach motivates emergency planners, hydro leaders, and dam operators to collaborate on active and future dam safety programs.

2.1 Rapidan dam - a near miss

In June of 2024, a combination of heavy rainfall and debris accumulation caused the partial failure of the Rapidan Dam in Minnesota. The Blue Earth River, part of the Mississippi River watershed, rose rapidly, with a peak of 34,800 cubic feet per second during the week of June 22, 2024. This marked the second-largest flood in the dam's history, built in 1910. The former hydroelectric power generator dam is a concrete gravity design consistent with this region and the era. Near misses were not a new dilemma for this location, as a once-considered 500-year flood in 1965 brought heavy ice flow downstream that destroyed six tainter gates. Eventually refitted and reinforced in the 1980s, the new gates and intakes were designed to prevent a repeat of the 1965 conditions but still caused noticeable issues with concerns of vortexing from the intake positioning and foundational inadequacies. The retrofit of 1983 took into account historical climate data from the 20th century and did not consider the potential for deteriorating climate conditions that would eventually be faced 40 years later. The Rapidan Dam continued to serve as a hydroelectric provider, with improvements to spillway capacity and trash racks to mitigate clogging. Eventually, the business case for continuously improving the structure was determined in 2019, and the site did not warrant further power generation, and the dam's condition soon deteriorated [8].

Known as a potential hazard, the US National Inventory of Dams rated the dam in poor condition and announced significant concern for failure potential. During the June 24 events, rainfall had subsided, but the downstream flow and release of debris toward the Rapidan Dam spillway channels started to carry floating and submerged debris. Immediate reports indicate a debris accumulation at Rapidan developing on the evening of June 23. With difficulties in site access and rapid response capabilities, the large fields of primarily woody debris soon became a problem that could not be managed. By the time debris had passed the upstream bridge support piles, the flow for traditional debris removal had become too severe for extraction craft to deploy. With gates open, increasing water flow, and delayed reaction to counteract debris, the woody debris piled against spill gates reducing releasing capacity. With this presence of extreme hydrological forces,

the fear of an overtopping event was imminent. The designed spillway capacity notes that the dams can manage 41,9000 cfs, but the potential for debris blockage to reduce up to 50% of discharge capacity forced water to find alternative paths [8].

Known as a potential hazard, the US National Inventory of Dams rated the dam in poor condition and announced significant concern for failure potential. During the June 24 events, rainfall had subsided, but the downstream flow and release of debris toward the Rapidan Dam spillway channels started to carry floating and submerged debris. Immediate reports indicate a debris accumulation at Rapidan developing on the evening of June 23. With difficulties in site access and rapid response capabilities, the large fields of primarily woody debris soon became a problem that could not be managed. By the time debris had passed the upstream bridge support piles, the flow for traditional debris removal had become too severe for extraction craft

Fig. 3 Rapidan Dam Woody Debris Blockage and Water Redirection, NBC News June 2024

to deploy. With gates open, increasing water flow, and delayed reaction to counteract debris, the woody debris piled against spill gates reducing releasing capacity. With this presence of extreme hydrological forces, the fear of an overtopping event was imminent. The designed spillway capacity notes that the dams can manage 41,9000 cfs, but the potential for debris blockage to reduce up to 50% of discharge capacity forced water to find alternative paths [8].

The west abutment faced severe erosion, with water seepage tearing away at the softened soil. The former generating house forced more water into this new path, creating a new emergency spillway. The dam and the foundational bedrock held to avoid a complete failure, but this new erosion of the western abutment created an uncontrolled release. With the events playing out on national television, emergency response was ready to assure that downstream structures or residents were more prepared than the initial efforts of 24-28 hours before. The inability to control this release and remove immediate debris blockage at Rapidan Dam forced operators and authorities to monitor the situation until cleanup and repair efforts could be initiated [9].

Since the event, the county commissioners have voted to remove the dam and monitor and or repair the damaged bridge upstream from the site. The bridge pilings were exposed to the flood's high flows and shifting sediments, creating safety concerns to avoid a repeated failure of critical infrastructure. Following the partial failure, the US dam safety community has commenced investigations to determine the factual narrative of the partial failure and how to prevent a future incident. Multiple dam operators and former site managers noted the delayed response in removing surface debris flowing downstream. Floating debris on the river was not a new site, especially following such a heavy rain. As familiar with seasonal rains, dead vegetation and trees from along the watershed are susceptible to enter the waterway and slowly creep towards dams. The rainfall event of the days leading to the June 24 partial failure should have been a significant alert to mobilize and prepare for emergency debris management and spill gate clearing [10]. Although much of the public attention on the dam’s partial failure shows consistent imagery of developing debris at the bridge buoys and eventually compiling at the dam wall, there can be additional factors of reduced spill gate capacity that could have contributed to the event.

The county authority has traditionally been charged with managing the dam since the termination of power generation in 2019. It has contracted services to monitor and remove debris when concerns arise. A combination of unavailable removal equipment, untimely decision points, and the rapidness of extreme rainfall all played a factor in the inability to remove the debris that contributed to the forced divergence of the river flow. At an unmonitored site like Rapidan, the lack of early capture or containment systems allowed the threats of mass debris fields to develop on the dam wall quickly. The public safety buoys upstream at the bridge piles captured small amounts of debris but were not intended to capture debris. Rapidan Dam demonstrates an event where conditions, decisions, and uncontrolled variables mix perfectly to send shockwaves across the dam safety community. In similar sites across Minnesota or the US Midwest, several legacy dams that are nearly a century old installed debris booms and trash racks to either stop or quickly remove debris prior to massive blockages. With the known historical incidents of the 1965 ice clogging and previous debris removal requirements, Rapidan Dam did not become a management mistake that caused further damage or possible loss of life. Understanding dam spillway capacity and considering increased debris flow are vital for mitigating risks to infrastructure and communities. Debris carried by extreme hydrodynamic conditions can obstruct spillways, reducing discharge capacity and increasing flooding risk. At dam sites worldwide, the case for debris management and its intersection with dam and public safety.

2.2 Hurricane Helene

Across the United States, the conversation of correlating extreme weather events to challenges in dam safety can occasionally become partisan, but the lives lost and property damages incurred

across the American South following the severe precipitation of Hurricane Helene in September 2024 warrant an honest conversation. Hurricane Helene brought record-breaking rainfall and exposed significant climate resiliency challenges for regional dams. Several dams faced critical conditions during this extreme weather event, highlighting the need for improved infrastructure and preparedness.

In North Carolina, two dams faced potential failure amid the flooding caused by Hurricane Helene. The Walters Dam in Haywood County and the Lake Lure Dam in Rutherford County experienced critical conditions as excessive rainfall caused water to crest behind the dams and flow around their wall.

The most extreme rainfall occurred in the mountain regions of western North Carolina and eastern Tennessee:

- Western North Carolina recorded upwards of 30 inches over 3 days
- Busick, North Carolina: 30.78 inches
- Mount Mitchell State Park: 24.20 inches
- Asheville airport: 13.98 inches in three days

Several locations across Florida, Georgia, and the Carolinas recorded more than a foot of rain from Helene. Estimated rainfall totals across the southern Appalachians had an Annual Recurrence Interval greater than 1,000 years over a wide area [11].

Fig. 4 Aftermath of Hurricane Helene. Major Releases at Waterville Dam, Oxford Dam, and Lake Susan. Major debris fields flow downstream

In Tennessee, the Nolichucky Dam near Greeneville came close to failing, necessitating the evacuation of nearby communities. This near miss, along with the incidents in North Carolina, illustrates how many dams are unprepared for the challenges posed by climate change and increasingly severe weather patterns. Debris accumulation played a significant role in exacerbating dam challenges during Hurricane Helene. In Douglas Lake, Tennessee, the Tennessee Valley Authority (TVA) had to install a one-mile-long floating barrier to collect flood debris washed into the lake by the hurricane. The massive amount of debris, including floating and submerged materials, posed risks to the dam's operations and the safety of the surrounding areas.

An additional threat that has developed from the multiple regional and federal authorities that manage the utilities and hydropower divisions of the TVA and adjacent areas is what will be entering the water next. Although recovery efforts and reactive debris barriers can momentarily capture and corral debris threats at a reservoir, extreme weather events can deteriorate the situation quickly. After Helene, there is a concern about loose debris along the banks of reservoirs and tributaries upstream that will be suspectable in entering the waterway and causing potential hazards to dam operations. In the past, dam owners could better manage the regular presence of debris at dams with available extraction resources, trash rakes, debris booms, or manual removal. Looming with the enhanced possibility of future debris complications with miles of riverbanks littered with loose debris, the rebuilding and recovery efforts are suspectable to floating hazards that can damage weakened structures or incapacitate spillways [13]. The rapid response lesson of debris management and enhancing programs in the wake of disaster is a very fluid development process as the known risk mitigation tactics of the past are forced to be altered.

The impacts of Hurricane Helene on dams and water infrastructure extended beyond just the immediate threat of failure. The storm's flooding severely damaged water infrastructure in North Carolina, Florida, Georgia, Tennessee, and Virginia, including sewage systems, wastewater treatment plants, and pipes that deliver drinking water to residents. These widespread impacts demonstrate the interconnected nature of water management systems and the cascading effects that extreme weather events can have on critical infrastructure.

As climate change intensifies extreme weather events, the resilience of dams and related infrastructure will become increasingly crucial. The experiences during Hurricane Helene serve as a wake-up call for policymakers, engineers, and communities to reassess and reinforce these vital structures to ensure they can withstand the challenges of a changing climate [7].

3.COLLABORATIVE FRAMEWORKS FOR CLIMATE RESILIENT DAMS

A holistic approach to dam safety extends beyond engineering solutions, incorporating governance, community engagement, and policy integration. Collaborative efforts between stakeholders, engineers, environmental scientists, policymakers, and local communities are essential to implementing sustainable strategies that address human and ecological needs. Regions with significant logging operations and or familiar with vast fields of woody debris can assist in reporting and mitigating upstream threats [15]. Hydrological cycles have become increasingly unpredictable due to climate change, altering water inflows and reservoir dynamics. This unpredictability complicates dam operations, particularly during extreme flood or drought events. In such scenarios, debris accumulation acts as a force multiplier, magnifying the operational challenges faced by dam operators [16].

3.1 Debris technologies and mitigation strategies

Despite the critical importance of dam safety, many countries lack robust regulatory frameworks for monitoring, maintaining, and upgrading dams. The absence of dedicated funding streams for dam modernization has hindered progress in implementing proactive safety measures globally.

Technology-driven advancements are reshaping how debris management aligns with climate adaptation strategies. Predictive analytics powered by artificial intelligence (AI) can analyze rainfall and hydrological patterns to forecast the likelihood and volume of debris inflows. By integrating this data with dam operations, managers can schedule preemptive water releases or activate barriers before debris accumulates to dangerous levels. Furthermore, drones equipped with high-resolution imaging and lidar can assess debris load in reservoirs and identify hotspots for sedimentation. Such tools improve monitoring efficiency and provide actionable insights, reducing reliance on manual labor and minimizing response times during emergencies. Providing access to these resources to regions struggling with regular dam safety stimulus can effectively provide global access and collaboration to dam safety and climate resilience [2].

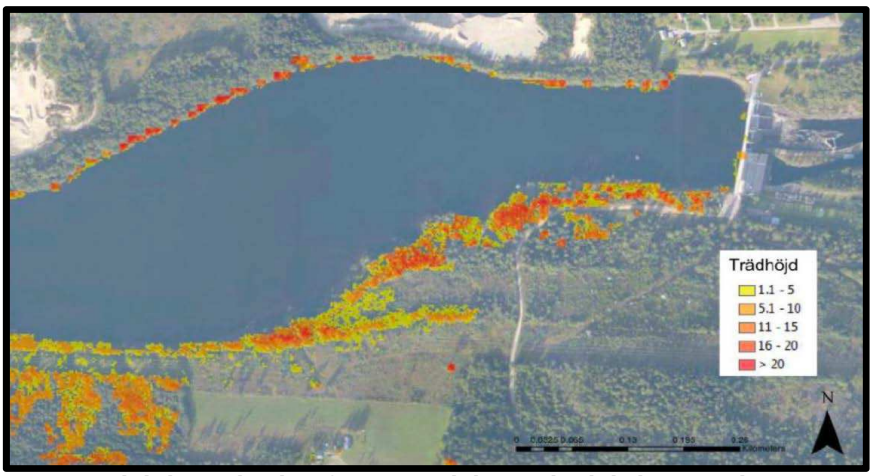


Fig. 6 Geospatial data plotting for potential woody debris entry into reservoir [16]

Community-driven initiatives can complement technical measures, especially in regions prone to climate-induced hydrological variability. Local communities upstream of dams can actively reduce debris at its source through sustainable land use practices such as reforestation, erosion control, and waste management programs. For example, planting native vegetation along riverbanks stabilizes soil and reduces the amount of loose sediment washed into reservoirs during heavy rains. Collaborations between dam operators, environmental organizations, and government agencies can also establish integrated debris management systems that align with regional water management policies and climate adaptation plans. These collective efforts create a more resilient framework for protecting dam infrastructure and the communities that depend on it.

3.2 A deteriorating situation

Regions worldwide face mounting challenges as these events compromise spillway capacity, elevate reservoir levels, and exacerbate sediment and debris inflow. Certain areas are particularly vulnerable to these trends. In the United States, the Northeast has experienced a 60% increase in extreme precipitation events since 1958, with projections showing continued growth. Similarly, the Midwest has seen a 45% rise, threatening aging dams in these regions that were not built to handle such intense inflows. In South Asia, extreme precipitation events

Fig. 7 Major debris field of organics at Xayaburi Dam, Laos Mekong River Source WPI 2022

could become up to 15 times more frequent under a 3°C warming scenario, which poses a significant risk to dams and reservoirs in densely populated areas. These failures could lead to widespread flooding and catastrophic socio-economic impacts, highlighting the urgent need for proactive flood and debris management systems [2]. Extreme precipitation events could become

annual, dramatically increasing the risk of dam overtopping and failures. This is particularly alarming for regions lacking dam modernization and flood control resources, further emphasizing the need for robust dam safety measures [15].

The rise in extreme precipitation has created a new reality for dam safety worldwide. Addressing these risks requires integrating predictive modeling into dam management systems. By combining real-time data with advanced forecasting tools, dam operators can better anticipate and mitigate the impacts of sudden inflows and debris accumulation. Furthermore, investment in spillway enhancements, debris barriers, and sediment management is critical to adapting aging dams to changing climate realities. These measures are essential for ensuring the resilience of dams and the safety of the communities and ecosystems that depend on them.

3.3 Debris management designs and planning your worst day

The installation and design of debris management systems have become more prevalent since the 1990s, typically motivated by the increased efficiency in power generation capacity with unclogged intakes. Although the financial incentive for controlled debris and water hyacinth diversion is enough to motivate a safety debris boom line, the risks and concerns of clogged emergency spillways and spill gates should motivate further action.

After reviewing 30 years of successful and unsuccessful debris booms across unique and challenging environments, dam owners often struggle with designing and deploying effective measures. When designing a debris barrier solution to combat increasing climate challenges and increased debris problems, operators and engineers must consider several factors to be prepared for future unknowns.

1) Debris load and type
2) Upstream entry possibilities
3) Hydrodynamic forces and surface velocity
4) Varying operating conditions (gates open, PMF)
5) Debris removal and consolidation
6) Monitoring capabilities
7) Wind Speed and wave action
8) Bathymetric variables
9) Controlled emergency management
10) Anchoring strength and long-term resilience

Not limited to this list of preliminary design elements for a debris boom protecting the head pond of a reservoir, these key elements must be considered in conceptual design and analysis [15]. Unfortunately, the belief that off-the-shelf solutions are the instant band- aid to triage debris situations worldwide has caused exacerbated debris issues in some cases. In the event at

Rapidan Dam, a proper debris boom upstream of the spill gates could have provided operators an early warning of compiling debris and a safer mechanism to remove debris rather than risk personnel and equipment.

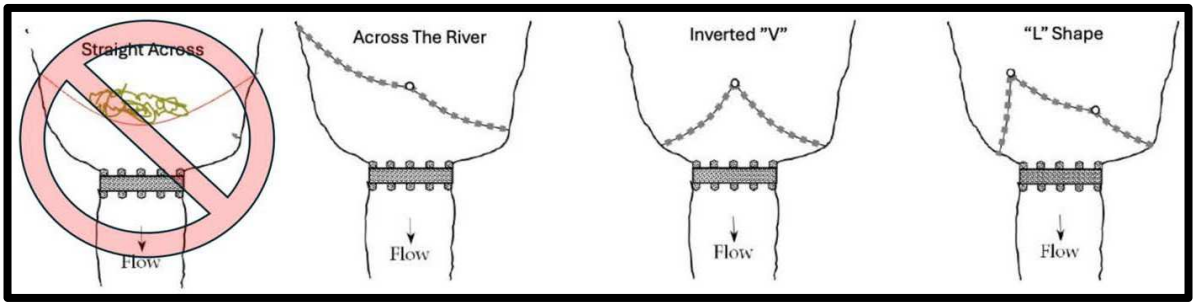


Fig. 7 Boom Line Configuration [15].

Solutions such as debris barriers, trash rakes, or screens could help alleviate the situation. In the provided graphic, shoreline diversion concepts avoid the possible oversaturation of debris that will flow beneath a barrier and create a threatening sag. Preparing debris management and dam safety for the threat of known unknowns in a risk analysis will permit a broader understanding of being prepared for a dam owner's worst day [17] [18] [19].

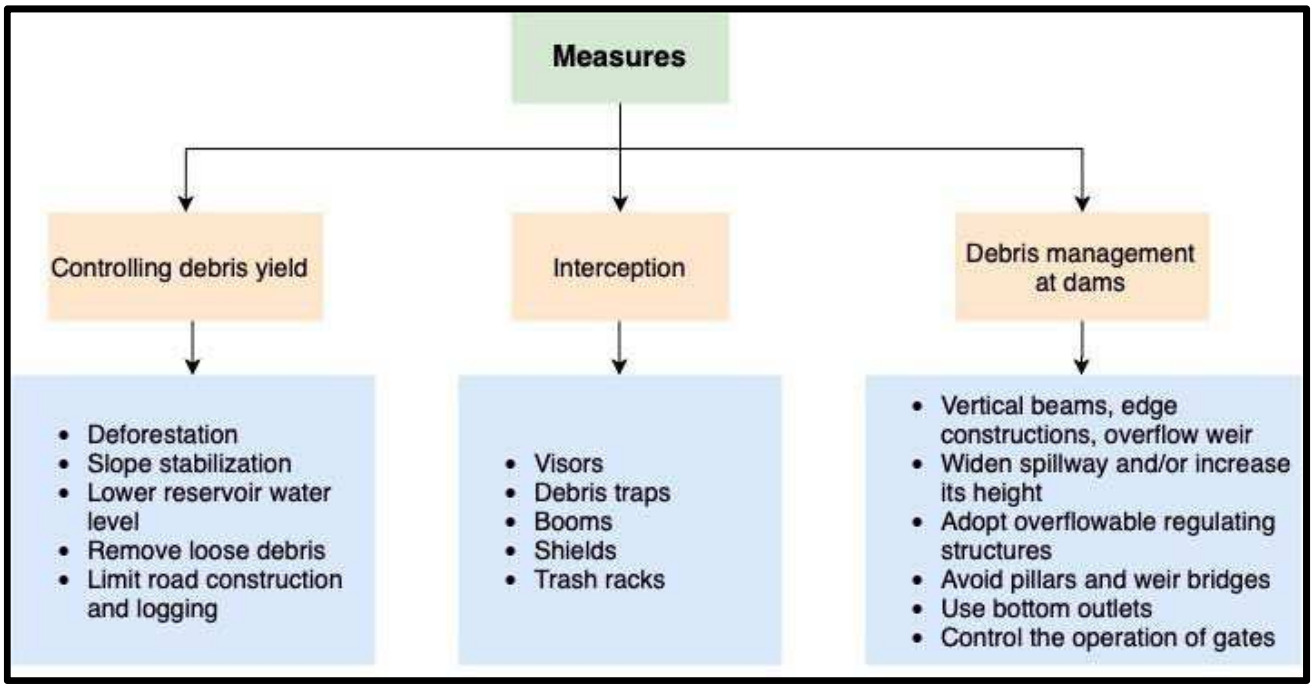


Fig. 8 Debris control mitigation matrix [5].

4. CONCLUSIONS

Dam safety remains a critical global challenge, requiring a concerted effort to modernize infrastructure, adapt to climatic realities, and mitigate debris-related risks. By leveraging cutting-edge technology, fostering collaboration, and addressing funding gaps, dam resilience can be significantly enhanced for future generations.

Debris removal technologies upstream can help intercept and clear debris, maintaining spillway efficiency even during extreme weather events. Furthermore, predictive analytics and hydrological modeling give dam operators critical foresight into potential debris loads and flow patterns, enabling informed decisions on reservoir management and water releases. However, the technological advancements alone are not enough. Collaborative frameworks that bring together engineers, policymakers, environmental scientists, and local communities are critical to the success of climate-resilient dam safety strategies. Engaging communities in upstream erosion control, reforestation, and waste management efforts can mitigate debris at its source, reducing the strain on dam infrastructure. Moreover, public awareness campaigns and educational initiatives are instrumental in fostering a culture of preparedness and resilience. This is particularly relevant for regions prone to climate-induced hydrological variability, where local knowledge and participation can amplify the effectiveness of dam safety measures.

The intersection of climate change, extreme weather events, and debris management presents both a challenge and an opportunity for dam safety. By investing in adaptive technologies, fostering multi-stakeholder collaboration, and addressing funding gaps, we can ensure that dams remain reliable and resilient, and capable of serving both human and ecological needs. As the climate evolves, a comprehensive and integrated approach to dam safety will be critical to safeguarding the lives, livelihoods, and environments that depend on these vi

REFERENCES

[1] Baecher, G. B. (2020, January 12). *FLOATING DEBRIS MANAGEMENT* [DAM SAFETY
INTEREST GROUP (DSIG)]. CEATI International Inc.

[2] Smith, J., Cooley, D., & England, J., Jr (2024). *Modernizing Probable Maximum Precipitation Estimation*. National Academies of Sciences Engineering Medicine. https://doi.org/10.17226/27460

[3] Wark, R. J. *BLOCKAGE OF RESERVOIR OUTLET STRUCTURES BY FLOATING DEBRIS*
[INTERNATIONAL COMMISSION ON LARGE DAMS 2017].

[4] Walker, K., & Shinbein, M. (2020). Spillway Debris Physical Model Study Ogee Crest Spillway. *Hydraulic Laboratory Report HL-2020-09*, (Dam Safety Office). https://doi.org/Department of the Interior

[5] Boes, R. (2017, November 15). *FLOATING DEBRIS AT RESERVOIR DAM SPILLWAYS*
[Report of the Swiss Committee on Dams on the state of floating debris issues at dam spillways]. Swiss Committee on Dams.

[6] Dance, S. (2024, June 29). This dam survived another flood, but exposed what ' s at stake as storms intensify. *The Washington Post*.

[7] A. P. (2024, June 29). This dam survived another flood, but exposed what ' s at stake as storms intensify. *Poltico*. www.politico.com/news/2024/09/29/hurricane-helene-widespread- destruction-south

[8] (Barr, Ayres,Kleinschidt, USACE)

[9] Stocekmann, E. (2024). St. Paul District responds to summer flooding. *Cross Currents USACE St Paul District*, *50*(3).

[10] Reyes, R. (2024, June 25). Crazy images show major dam breach in Minnesota — With
buildings and roads destroyed in flooding. *The New York Post*. https://nypost.com/2024/06/25/us-news/minnesotas-rapidan-dam-breach-destroys- buildings-and-roads

[11] Greene,M. (2024, November 4). *TVA Releases Helene Data*. TVA. https://www.tva.com/newsroom/press-releases/tva-releases-helene-data

[12] Thiem, H. (2024, November 7). *Hurricane Helene's extreme rainfall and catastrophic inland flooding*. NOAA. https://www.climate.gov/news-features/event-tracker/hurricane-helenes- extreme-rainfall-and-catastrophic-inland-flooding

[13] Press Release (2024, September 29). *Hurricane Helene Impacts to Bluestone Dam*.

US Army Corps of Engineers. https://www.lrd.usace.army.mil/Media/Media-Releases/Article/3921660/hurricane-helene-impacts-to-bluestone-dam/

[14] Schleiss, A. (2022, June 2). *BLOCKAGE OF RESERVOIR OUTLET STRUCTURES BY FLOATING DEBRIS* [Committee C - Hydraulics for Dams Workshop]. ICOLD.

[15] Schleiss, A. (1994, March 1). *Dam Safety Investigations Guidelines for Design of Booms* [Hydro Electric Engineering Division]. BC Hydro.

[16] Hassan, A. (2020, June 1). *Handling river floating debris for dam safety – the state of the practice* [Architecture and the Built Environment Master Thesis in Concrete Structures]. KTH.

[17] Lewin, J., Ballard, G. and Bowles, D.S. (2003). Spillway gate reliability in the context of overall dam failure risk. 2003 USSD Annual Lecture, Charleston, South Carolina. April.

[18] Boes, R., Lutz, N., Lais, A., and Lucas, J. (2013). Hydraulic modelling of floating debris conveyance for a spillway upgrade at a large rockfill dam. VAW (2013). Trängslet Dam Stepped Spillway - Physical Model Investigation. VAW-Report 4305 (unpublished), Laboratory of Hydraulics, Hydrology and Glaciology (VAW), ETH Zurich.

[19] FHWA. (2005). Debris Control Structures Evaluation and Countermeasures (Third Edition). Federal Highway Administration, Washington, DC.

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

IMPACT OF CLIMATIC DRIFT ON ARCH DAMS BEHAVIOUR

Louis SUCHIER[1] EDF
Emmanuel ROBBE[2] EDF
Alexandre SIMON[3] EDF
Grégory COUBARD[4] EDF

ABSTRACT

Climate change is having two main effects on temperatures: an upward climatic drift over the long term, and more marked exceptional phenomena such as heatwaves. Objectives of this publication are first to evaluate the impact of climatic drift in France on the behaviour of arch dams, based on monitoring data collected on approximately fifty arch dams, and second compare it to heatwaves, in order to conclude whether or not it is worth modelling them in studies of arch dam behaviour.

Measurements of the displacement of arch dams, using pendulums or planimetry, provide relevant data for assessing the thermal effects on their behaviour. These monitoring data, processed using HST (Hydrostatic Seasonal Temporal) and HSTT (Hydrostatic Seasonal Temporal Thermal) models developed by EDF, make it possible to assess the effects of heating and compare them with the usual thermal loads. These effects are fairly small and give rise to median equivalent deformations at the crest of around 0.6 µm/m/year between 2010 and 2019, or around 15 µm/m over 40 years, with variability depending on the structure. Median upstream displacements of the crest length are estimated at around 1 to 2 mm over the same period.

Because of the irreversible nature of climatic drift cumulated with reversible seasonal effects, particularly in summer, the question arises as to how it should be incorporated into numerical modelling of the structure's behaviour. Indeed, behaviour of arch dams is only marginally affected by climatic drift. While thin arch dams are much more sensitive to exceptional climatic episodes, rather than the slight increase due to the gradual rise in temperature, thick arch dams are much less sensitive to these exceptional events and to climatic drift. It therefore seems more appropriate for thin arch dams to study exceptional climatic episodes in numerical modelling to assess the behaviour of the dam with regard to its potential failure modes.

Regarding the orders of magnitude and the envelope load cases already studied, this climatic drift is considered negligible and not taken into account when it is the only source of irreversibility. For structures affected by a concrete swelling pathology, it appears to be of the second order and often cannot be dissociated from the swelling phenomenon.

Other effects of climate change, namely ones related to sudden increase of water level, due to extreme floods, and to the effects of long periods of empty reservoir, due to droughts, are not studied in this paper.

Keywords: Climate change, Climatic drift, Heatwave, Arch dams, Numerical studies.

[1] louis.suchier@edf.fr, France
[2] emmanuel.robbe@edf.fr, France
[3] alexandre-gilles.simon@edf.fr, France
[4] gregory.coubard@edf.fr, France

1. INTRODUCTION

In France, a global warming trend has been observed since 1980. There have been several phases: a marked warming up to around 1995 (+ 1°C), then stabilization until 2010, before a further, more significant increase to the present day (+ 1°C). Fig. 1 below shows this increase as a rolling average over 11 years.

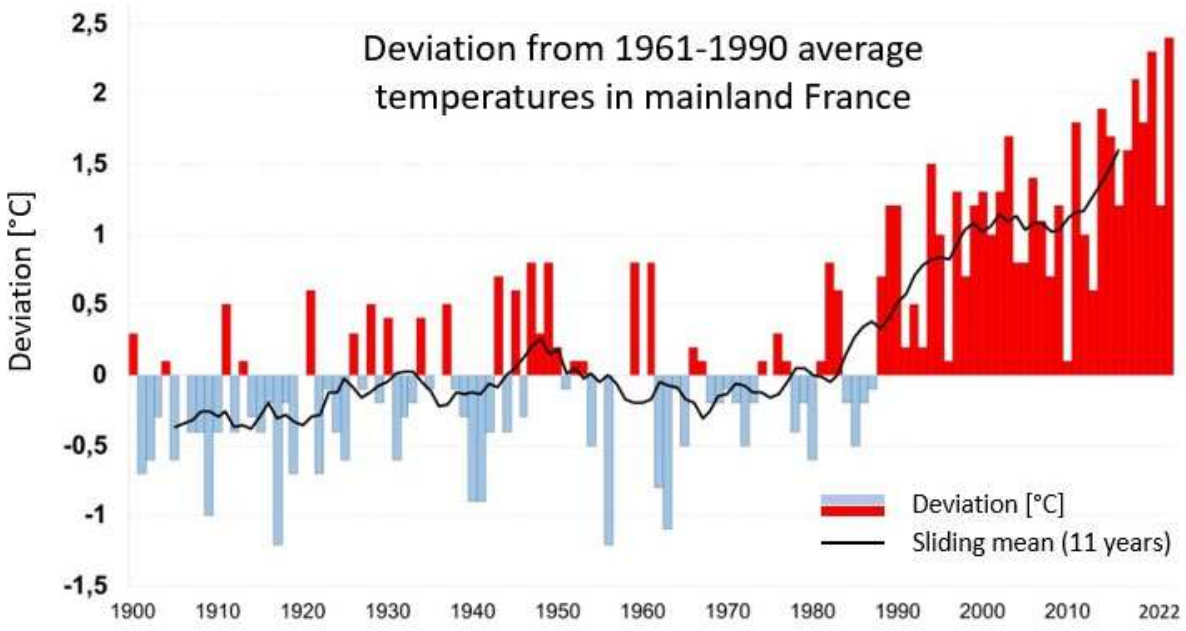


Fig. 1 – Deviations from 1961-1990 mean temperature normals for mainland France, 1900-2022, and 11-year rolling average (black).

Météo France wrote on this subject in 2020 [1] :

> « *Average annual temperatures in mainland France have been rising since 1900. The rate of warming has varied, with a particularly marked increase since the 1980s. Over the period 1959 - 2009, the observed trend is approximately +0.3°C per decade. The three years with the highest average temperatures were observed in the 21st century, in 2018, 2014 and 2011 respectively.* »

Since then, 2022 and 2023 have become the 2 hottest years worldwide [2]. By 2024, climate drift in mainland France since the 1960s can be estimated at around 2°C.

Climate change is having two main effects on temperatures: an upward climatic drift over the long term, and more marked exceptional phenomena such as heatwaves.

While these two effects have no impact on gravity or embankment dams, it does cause arch dams, and in particular their crests, to move upstream. The long-term monitoring and modelling methods used at EDF Hydro enable us to measure and quantify the effect of climatic drift and heatwaves on existing structures. While the effect of exceptional events has been described

in recent publication [3], the aim of this publication is to assess the impact of climatic drift on arch dams compared to heatwaves, in order to conclude whether or not it is worth modelling them in studies of arch dam behaviour.

In the first part, this paper uses monitoring data to characterise the irreversible response of arch dams to climatic drift and highlight the orders of magnitude of its consequences. In the second part, two studies on arch dams of different typology are carried out using finite element models, to evaluate the impact of climatic drift on arch dams compared to heatwaves.

2. CHARACTERIZING THE IRREVERSIBLE RESPONSE OF ARCH DAMS TO CLIMATIC DRIFT AND QUANTIFYING ITS CONSEQUENCES

2.1. Monitoring data and the HSTT model

The Hydrostatic Seasonal Temporal Thermal model (HSTT) was developed by EDF [4] to improve the well-known HST model following the 2003 heatwave to consider average seasonal thermal effect as well as real mean daily air temperature. The statistical model, calibrated using multiple linear regression on chosen monitored measurement, allows to highlight:

- Irreversible effect related to time *t*:

$$g_1(t) = b_1 \cdot t \tag{1}$$

- Reversible hydrostatic effect related to the reservoir level *h*, normal water level *NWL* and reservoir level when empty *WLempty*:

$$g_2(Z) = b_2 \cdot Z + b_3 \cdot Z^2 + b_4 \cdot Z^3 + b_5 \cdot Z^4 \tag{2}$$

with:

$$Z = \frac{NWL - h}{NWL - WL_{empty}} \tag{3}$$

- Thermal effect related to the temperature distribution in the structure, which can be divided into two terms:
  - A sinusoidal "seasonal" term representing at best the mean annual evolution of temperatures related to a seasonal variable S going from 0° the 1st of January and 360° the 31st of December,

$$g_{3a}(S) = a_6 \cdot \cos(S) + a_7 \cdot \sin(S) + a_8 \cdot \cos(2S) + a_9 \cdot \sin(2S) \tag{4}$$

  - A component accounting for temperature deviation from the seasonal average.

$$g_{3b}(t) = K \cdot \Delta\theta_R(\mathrm{t}) \tag{5}$$

with $\Delta\theta_R(t)$ the delayed temperature deviation from the mean seasonal temperature computed from the following formula involving the characteristic time T_0 of the thermal diffusion inside the dam.

$$\Delta\theta_R(t+dt) = \Delta\theta(t+dt)\,(1 - e^{-\frac{dt}{T0}}) + \Delta\theta_R(t)\,e^{-\frac{dt}{T0}} \quad (6)$$

The climatic drift is included in the $g_{3b}(t)$ term. The chronicles used to calculate delayed temperature deviations, which depend on the T_0 parameter (Eq. 6), are taken from 8 reference temperature stations covering the whole of mainland France. The impact of climatic drift on arch dams is estimated on the basis of their own climatic drift and the parameter *K* (Eq. 5).

2.2. Methodology for estimating the impact of climatic drift

In order to estimate the effect of climatic drift, the chosen displacement monitoring systems are systematically located near crest of the arch dam, close to the crown if possible. This assumption is based on concrete elements common to all arch dams:

- This is the phenomenon with maximum displacement under thermal load.
- The displacement between the toe and crest is representative of the arch dam's overall behaviour and “integrates” the structure's behaviour over its entire height.
- Today, the majority of EDF arch dams are monitored at the crest, around half by pendulums, the other half by topographic measurements. Furthermore, these monitoring systems regularly have a history stretching back several decades.

Where reliable or sufficient data are not available for certain crest monitoring systems, they are not included in this summary.

Note that in the graphs below, the arch dams have been annotated in relation to their thickness, according to a ratio between their thickness at the base (E_b) and their height (H). If E_b /H < 0.15, the arch dam is considered “thin”, “medium” between 0.15 and 0.25, and “thick” above that.

2.3. Irreversible response of arch dams to climatic drift

Results are presented for the period 2010-2019, with a climatic drift of around 1°C, or for the period 1980-2019, with a climatic drift of around 2°C in mainland France. The choice of these two periods was dictated by the fact that the rate of climatic drift was homogeneous over each.

On the following graphs :

- The number at the top of each column of Fig. 2 and Fig. 4 corresponds to the crest radius of each arch dam.
- The numbering of the arch dams in Fig. 2 has been retained for Fig. 3 to Fig. 5.

Fig. 2 shows the peak arch crest elongation due to climatic drift, in µm/m/year, over the period 2010-2019, as a function of arch dam thickness.

The median peak arch crest elongation is of the order of 0.6 µm/m/year, or around 6 µm/m over 10 years. However, this median value masks significant disparities between structures, since the rate of elongation ranges from 0 to 1.6 µm/m/year.

Logically, the thinnest arch dams (in dark blue for the thinnest and light blue for those of intermediate thickness) are the most affected by climatic drift. Their average arch crest elongation is higher than that of thicker dams (0.75 vs. 0.4 µm/m/year).

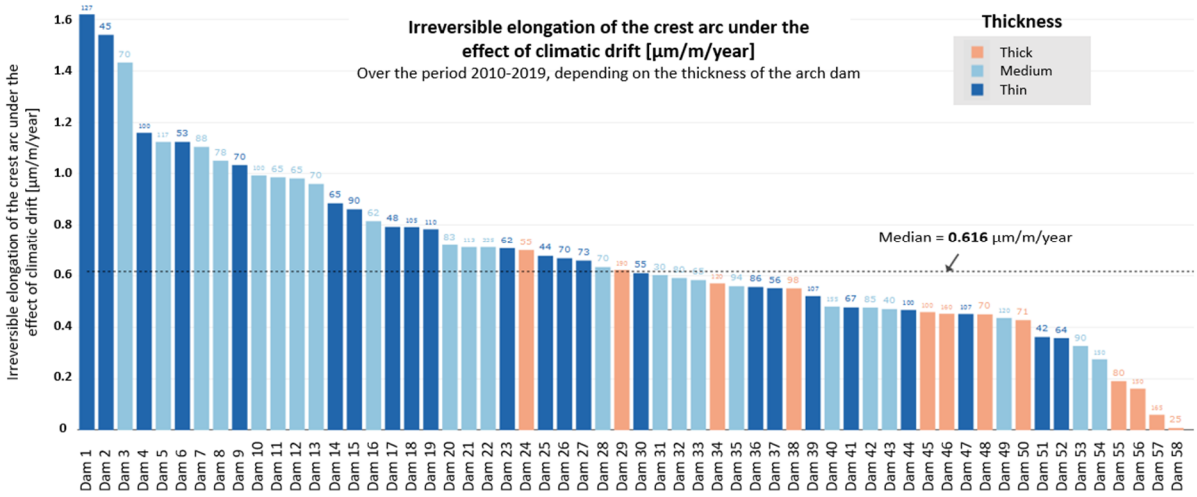


Fig. 2 – Irreversible elongation of the arch crest under the effect of climatic drift (µm/m/year), over the period 2010-2019, depending on the thickness of the arch dam.

Fig. 3 shows upstream displacement undergone by the crest monitoring system studied for each of the structures under the effect of climatic drift, in mm/year, over the period 1980-2019.

This graph shows that the irreversible upstream displacement caused by climatic drift over the period 1980-2019 on EDF's arch dams ranges from 0 to 5 mm. For two-thirds of the dams, climatic drift has caused an upstream shift of the order of 1 to 2 mm over 40 years.

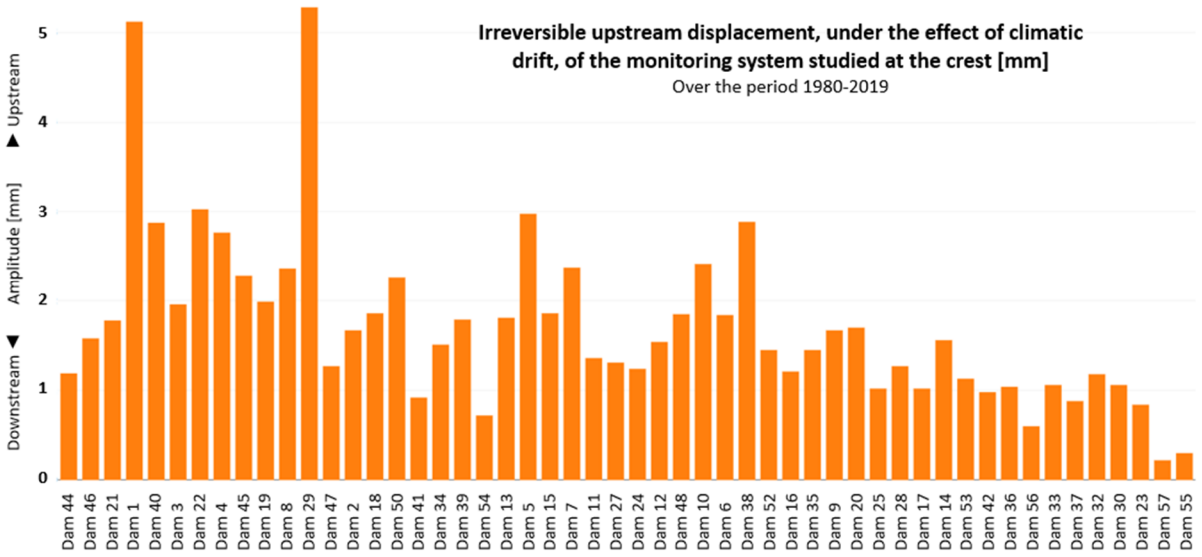


Fig. 3 – Irreversible upstream displacement, under the effect of climatic drift, of the monitoring system studied at the crest for each of the structures (mm/year), over the period 1980-2019, depending on the thickness of the arch dam.

Taking all uncertainties into account, the average upstream velocity of the crest of the structures studied can be estimated at between 0.03 and 0.05 mm/year over the period 1980-2019. However, this median velocity is 0.075 mm/year for the decade 2010-2019[1]. It is therefore very likely that this will now be closer to 0.1 mm/year since the early 2020s and the observed acceleration of climatic drift. [5]

Finally, Fig. 4 gives an idea of the rate of upstream displacement of the arch crest, with the result scaled according to dam height: displacement in mm/year is based on a 100-meter-high dam. This approach may be questionable as it results from a simple ratio using the height of the dam and does not take into account the effect of crest radius or length.

Thus, over the period 2010-2019, the median upstream displacement is 0.145 mm/100m/year, a value that once again erases major disparities since results range from 0 to 0.4 mm/100m/year. Nonetheless, two-thirds of the structures have displacement rates of between 0.1 and 0.2 mm/100m/year.

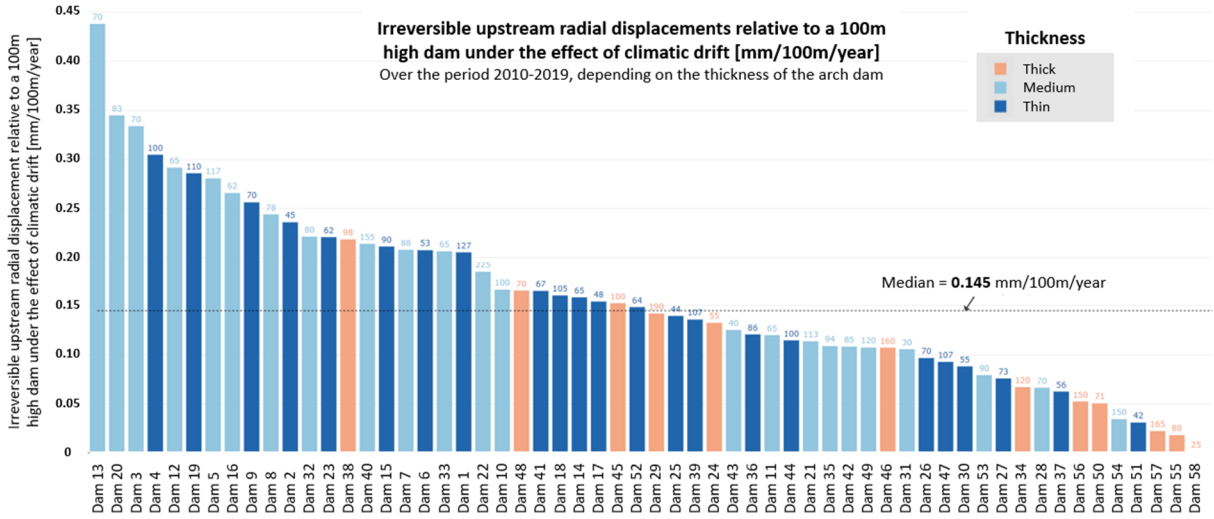


Fig. 4 – Irreversible upstream radial displacement relative to a 100 m high dam under the effect of climatic drift (mm/100m/year), over the period 2010-2019, depending on the thickness of the arch dam.

2.4. Share of climatic drift in global irreversible amplitude

Fig. 5 below compares, for each structure, the irreversible displacements attributable to climatic drift over the period 1980-2019 (in orange) with the total irreversible displacements suffered over the same period (in blue).

This anonymised graph should nevertheless be analysed with caution, as the irreversible displacements highlighted :

[1] This is linked to the lesser warming over the 1995-2010 period.

- Do not date from the construction ;
- Are linked to different sources of irreversibility, with different kinetics, without distinction.

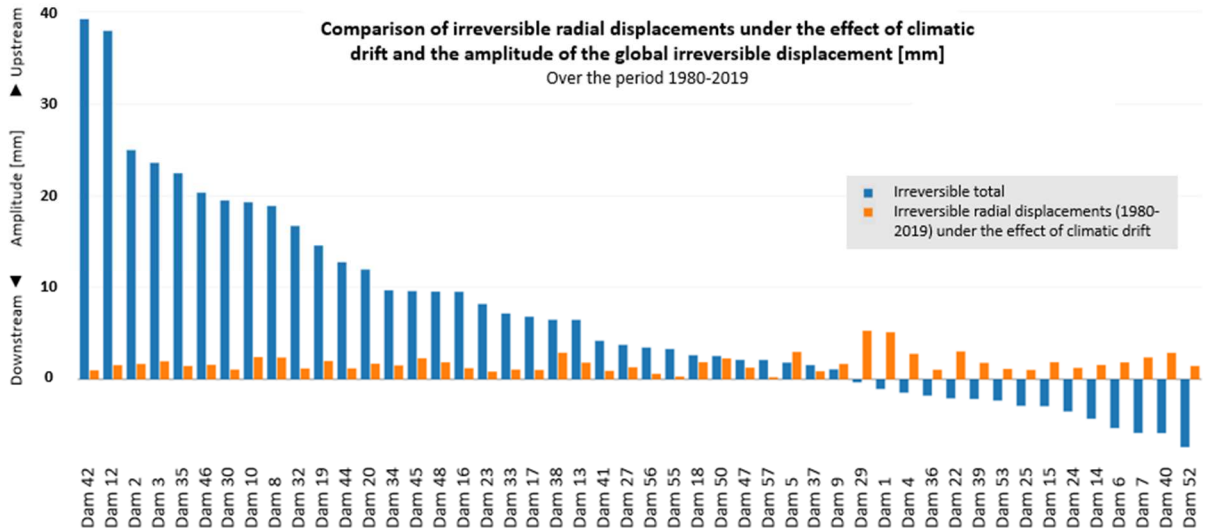


Fig. 5 – Comparison of irreversible radial displacements under the effect of climatic drift and the amplitude of the global irreversible displacement (mm/year), over the period 1980-2019.

The graph shows that the thermal effects of climatic drift are additional to the overall irreversible displacement of a structure due to any pathology, and can be minimal (structures furthest to the left) or of the same order of magnitude (structures in the middle). Conversely, climatic drift can also have a “beneficial” effect on certain structures (those furthest to the right), by “counteracting” their natural irreversible displacement, initially downstream, by moving them further upstream.

2.5. Orders of magnitude

Arguments in the general case highlight the negligible effect of this climatic drift on our structures. In addition, the way in which climatic drift is modelled in the HST and HSTT models allows us to conclude whether or not it is necessary to take climate drift into account:

- A climatic drift of around 2°C over 40 years (1980-2020) with a coefficient of expansion $\alpha = 1.0.10^{-5}$ °C^{-1} generates equivalent deformations of the order of 0.5 µm/m/year, or 20 µm/m, whereas some structures suffering from swelling pathology see equivalent deformations of several hundred or even thousands of µm/m.
- The same 2°C climatic drift over the last 40 years has caused irreversible upstream displacements of between 0 and 5 mm, usually around 1 or 2 mm, which is still fairly negligible. In fact:
    - If we use the HST model, the global irreversible (the “T” in HST) considers all the irreversible sources (possible pathology and/or climatic drift).

So, when an HST model is used to calibrate the irreversible, it takes climatic drift into account.

- When using the HSTT model, the irreversible is split into two parts: possible pathology in the “temporal T” and the effect of the climatic drift is modelled in the “thermal T” (delayed temperature deviations).

  If an HSTT model is used for calibration, the effect of the climatic drift is associated with the term “thermal T”, and thus is not exhibited by the irreversible effect. Regarding the order of magnitude:

  - Either the irreversible effects are modelled in the case of a proven pathology (e.g., swelling), and in this case the order of magnitude of climatic drift is negligible when compared to the effect of pathology;
  - Either the irreversible effects are not modelled (no known pathology) because they are very low. In this case, as the effect of climatic drift is low in every cases (less than 5 mm in 40 years, mostly 1-2 mm i.e. close to model uncertainties), the irreversible upstream shift attributable to climatic drift is not considered.

Effects of climatic drift are therefore covered by our methodology.

3. FINITE ELEMENT ANALYSES OF TWO ARCH DAMS

In this part, two studies on arch dams of different typology are carried out using finite element models, to assess the impact of climatic drift on arch dams compared to heatwaves.

In fact, the effect of exceptional events as heatwaves (by simulating 10-year summers) has been described in publication [3], and the previous chapter highlighted the small influence of climate drift on monitoring data on the crest displacement of arch dams.

These two studies will make it possible to assess the impact of climatic drift on arch dams behaviour compared to heatwaves, in order to conclude whether or not it is worth modelling them.

3.1. Presentation of the two dams and the finite-element models

Dam 1 and Dam 2 are modelled using a finite element mesh. They have a minimum of 2 elements in their thickness. The behaviour law used for the quadratic volume elements is linear elastic. For Dam 1 only, joint elements of zero thickness are added to the dam-foundation contact and allow the opening at the dam-foundation interface from the upstream toe, particularly under the combined effect of the hydrostatic load and winter thermal load: this effect is not analysed in this publication.

Parameters of both models have been calibrated by comparison with monitoring data (not presented here).

Their main geometric characteristics are as follows in Table 1:

Table 1 – Main geometric characteristics of the two dams.

Characteristic	Dam 1 (thin)	Dam 2 (thick)
Type	Single Curvature	Double Curvature
Date of construction	1956	1960
Height above natural ground	34.5 m	87.0 m
Height above foundations	37.0 m	90.0 m
Thickness at crest	1.7 m	4.3 m
Maximum thickness at toe	4.9 m	30.4 m
Radius of curvature at crest	48.0 m	71 m
Crest length	130.3 m	120.0 m

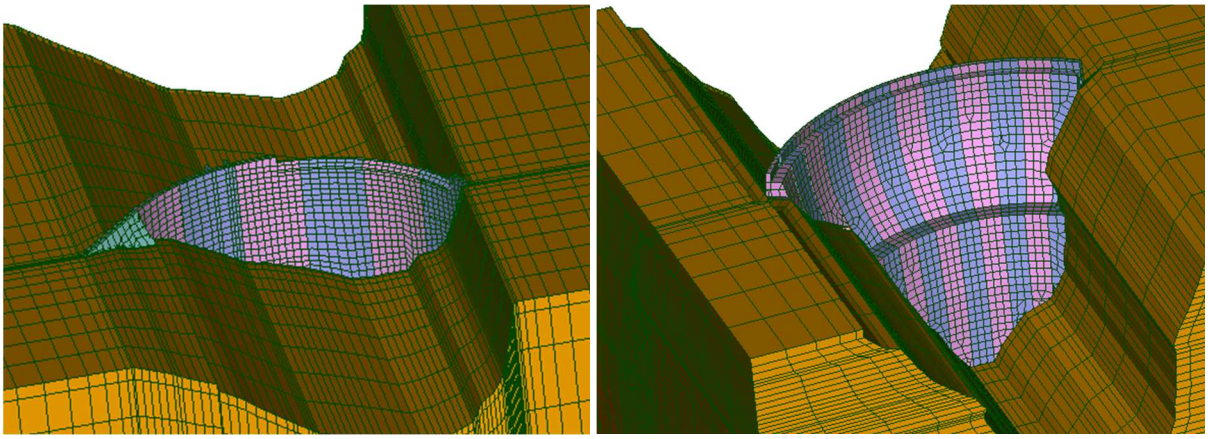


Fig. 6 – Finite-element mesh of Dam 1 (left) and Dam 2 (right).

3.2. Modelling temperature rises

In the numerical model, climatic drift and heatwaves are modelled in two different ways:

- Climatic drift of the order of 2°C is modelled by increasing the temperatures for the entire dam in steady state. This assumption could be challenged since we did not collect data on the water temperature rises in the reservoir and more particularly at the upstream toe of the dam.
- The 10-year thermal scenario is applied by increasing the air temperature boundary conditions, so it is essentially the downstream face that is warmed up. The upstream face, part of which is under water, does not see any variation in temperature at the toe where the temperature is that of the water, and this variation is reduced over the entire

part of the face where the temperature is in transition between the water temperature and the air temperature.

In this case, several annual temperature cycles are modelled in order to reach a steady state in the dam, before increasing the temperatures of the last summer using the methodology described in the publication [3].

Fig. 7 below highlights the difference in modelling between climatic drift and heatwaves. The temperature increments modelled for heatwaves are explained in paragraph 3.4.

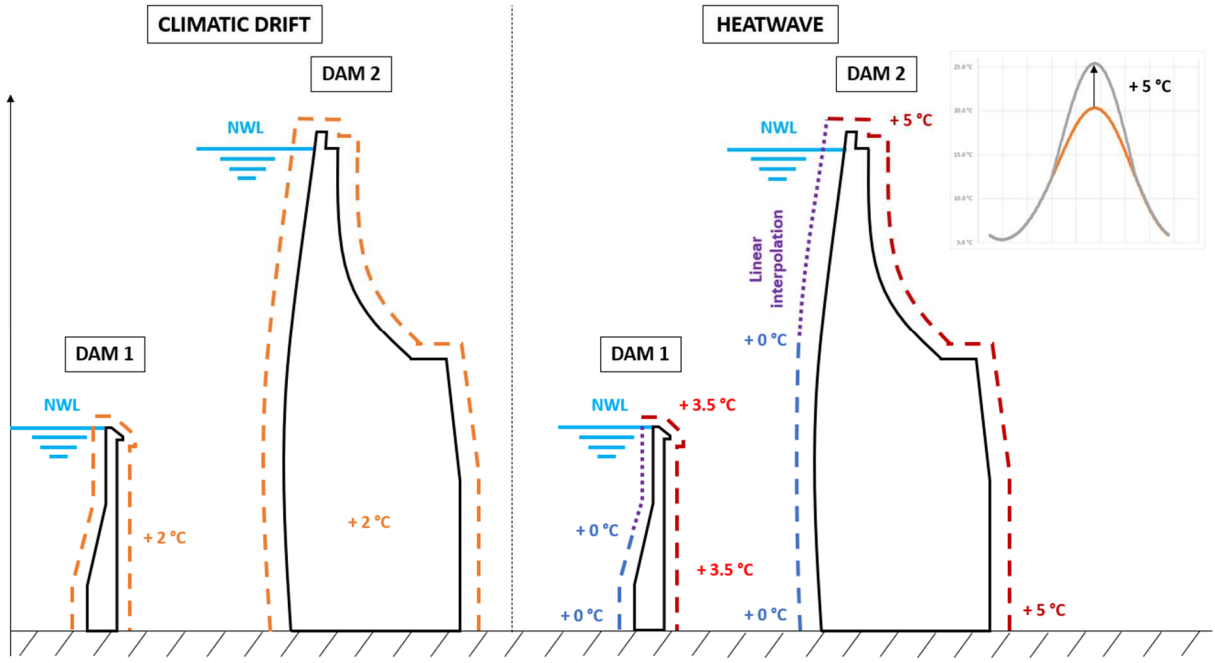


Fig. 7 – Modelling climatic drift and heatwaves for Dam 1 and Dam 2.

Paragraphs 3.3 and 3.4 describe the effect of a 2-degree rise in temperature and a heatwave on Dam 1 and Dam 2. These results are analysed in section 3.5.

3.3. Impact of climatic drift on the dams behaviour

Without climatic drift, the main effects of summer thermal loading on these structures are as follows:

- For Dam 1 and Dam 2 : An increase in compressive stresses in the upper part of the structure, close to the abutment, in the horizontal direction. These stresses are induced by a bending mechanism of the crest arch in this zone (moment about the vertical axis), clearly visible on the deformations of the structure.
- For Dam 1 : A change in the distribution of tensile stresses in the downstream face, with the appearance of a tensile zone at the downstream toe of one of the bank blocks, resulting in a slight opening of the dam-foundation contact.
- For Dam 2 : An increase in compressive stresses on the banks in low downstream face.

Logically, these effects are amplified by the climatic drift load. The increased displacement at the crest of the central cantilever is:

- For Dam 1 : 1.3 mm, for an annual seasonal amplitude of around 10.5 mm;
- For Dam 2 : 1.2 mm, for an annual seasonal amplitude of around 14.2 mm.

Figures below compare the most dimensioning stresses between situations NWL + Summer and NWL + Summer + 2°C, for Dam 1 in Fig. 8 and Fig. 9, and for Dam 2 in Fig. 10 and Fig. 11.

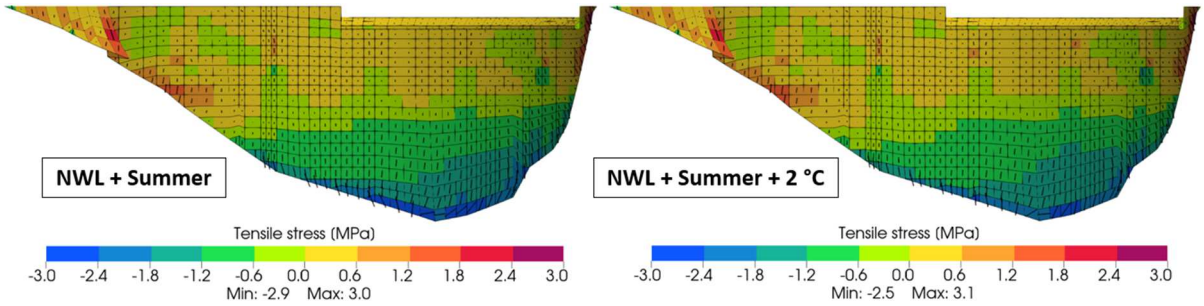


Fig. 8 – Dam 1: Tensile stresses downstream at NWL + Summer and NWL + Summer + 2°C.

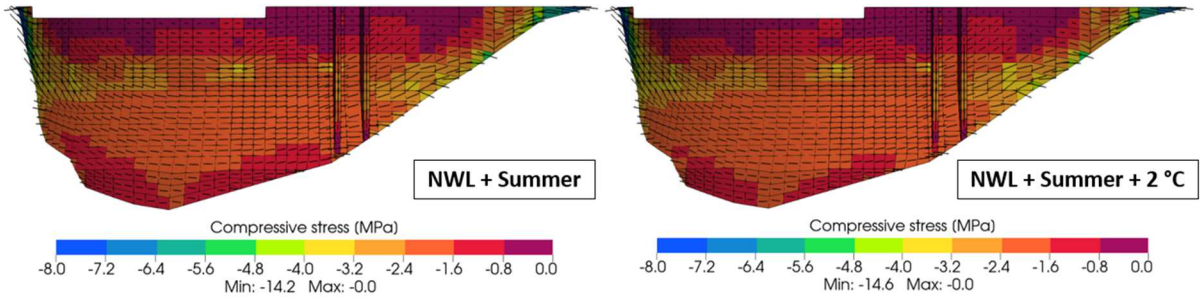


Fig. 9 – Dam 1: Compressive stresses upstream at NWL + Summer and NWL + Summer + 2°C.

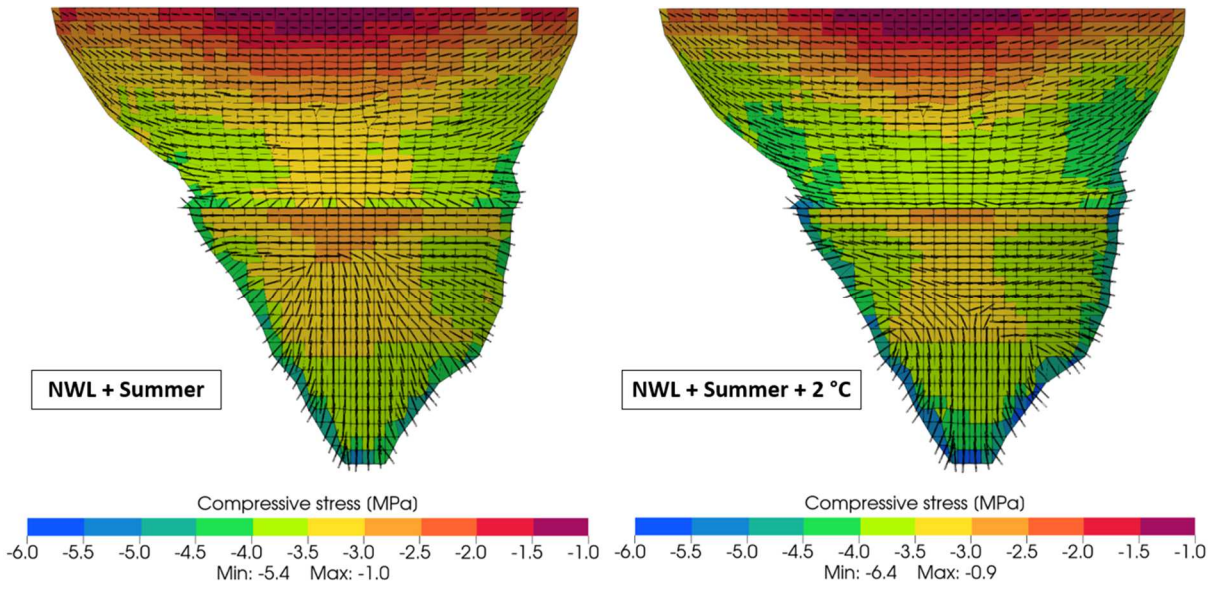


Fig. 10 – Dam 2: Compressive stresses downstream at NWL + Summer and NWL + Sum. + 2°C.

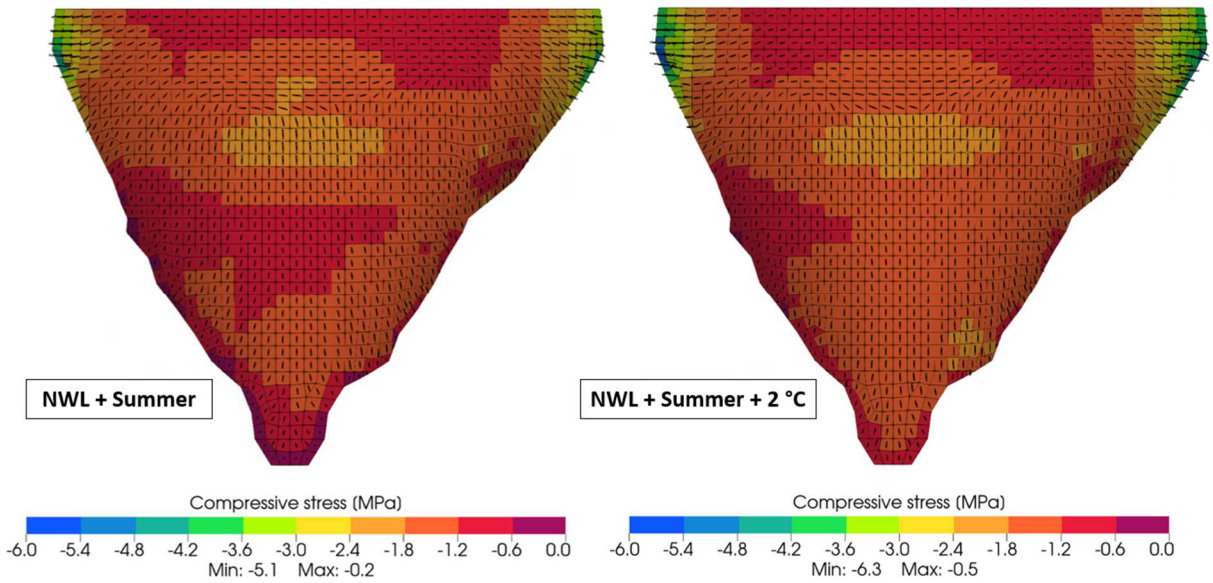


Fig. 11 – Dam 2: Compressive stresses upstream at NWL + Summer and NWL + Summer + 2°C.

3.4. Impact of a heatwave

The simulated heatwave corresponds to a 10-year Summer (one chance in ten of occurring each year) and has been particularized at the same dams as in the previous section, according to a methodology implemented in recent years and presented in the publication [3].

This heatwave corresponds to a prolonged rise in temperature. This increase is modelled under transient conditions. The maximum temperature rise (specific to the dam) is around 3.5 °C for Dam 1 and around 5 °C for Dam 2 and is applied in midsummer, when the dam is already at its maximum upstream.

The additional key displacement caused by this increase in summer thermal conditions is:

- For Dam 1 : 2.7 mm, for an annual seasonal amplitude of around 10.5 mm;
- For Dam 2 : 2.8 mm, for an annual seasonal amplitude of around 14.2 mm.

Figures below compare the most dimensioning stresses between situations NWL + Summer and NWL + 10-year Summer, for Dam 1 in Fig. 12 and Fig. 13, and for Dam 2 in Fig. 14 and Fig. 15.

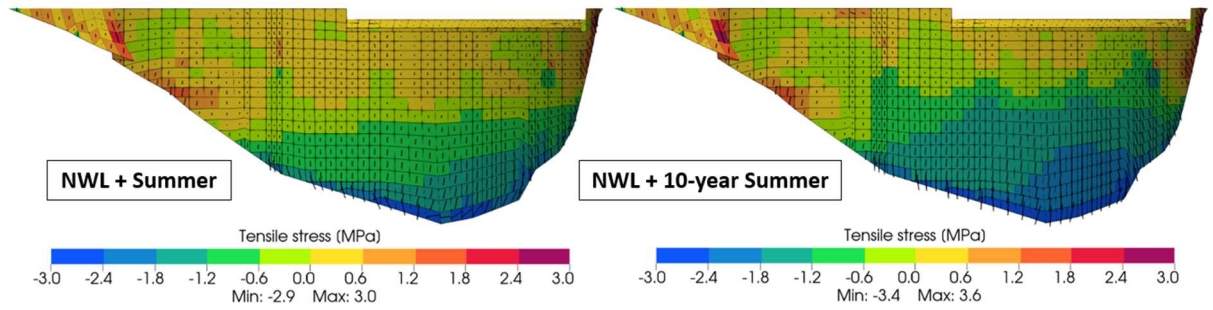


Fig. 12 – Dam 1: Tensile stresses downstream at NWL + Summer and NWL + 10-year Summer.

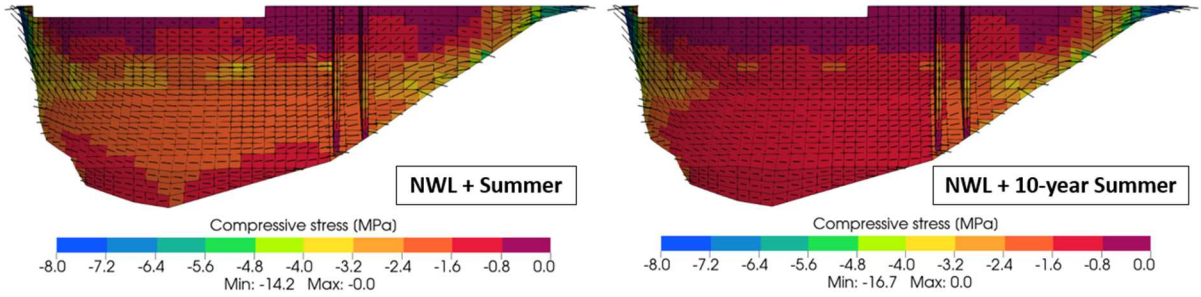


Fig. 13 – Dam 1: Compressive stresses upstream at NWL + Summer and NWL + 10-year Sum.

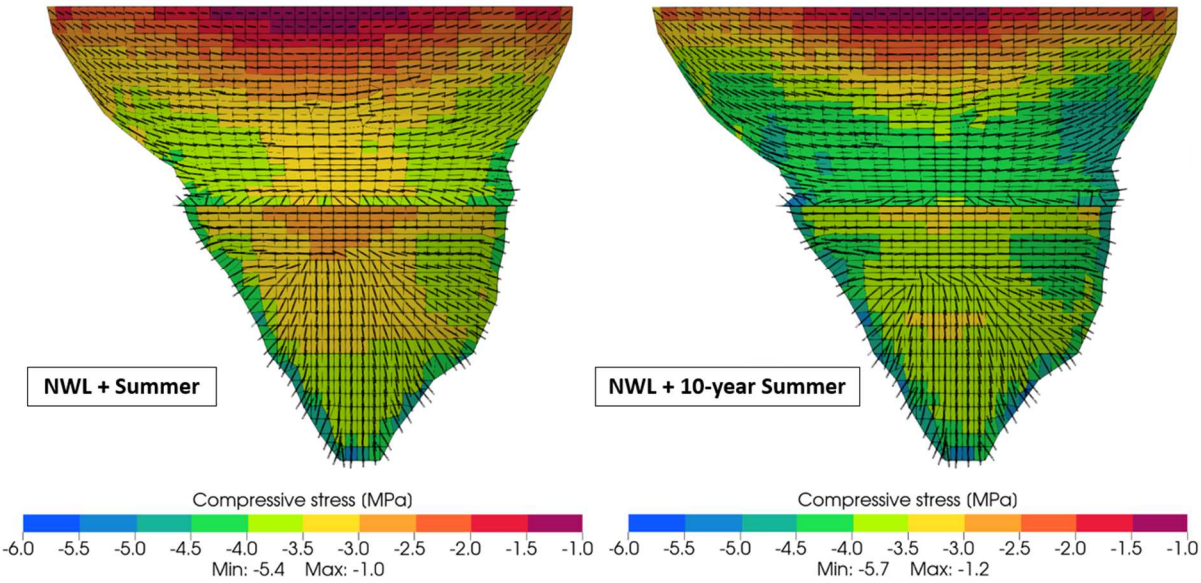


Fig. 14 – Dam 2: Compressive stresses downstream at NWL + Summer and NWL + 10-year Summer.

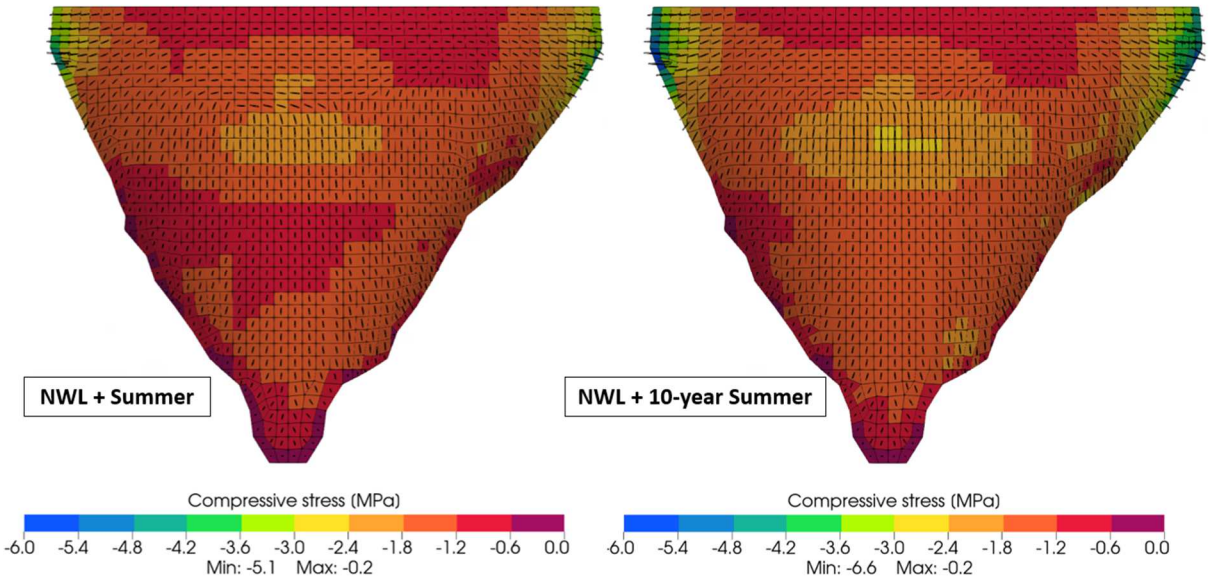


Fig. 15 – Dam 2: Compressive stresses upstream at NWL + Summer and NWL + 10-year Sum.

3.5. Comparison and discussion

Table 2 and Table 3 summarizes the maximum compressive and tensile stresses obtained with the different scenarios. Variations from the NWL + Summer scenario are shown in brackets for the last two scenarios (NWL + Summer + 2°C and NWL + 10-year Summer).

For Dam 1 (thin), we can see a very small difference between the NWL + Summer and NWL + Summer + 2°C scenarios, whereas the difference is greater and significant under the influence of the heatwave.

For Dam 2 (thick), the stress increments generated by either of the scenarios, climatic drift or heatwave, are fairly small and do not influence the behaviour of the arch dam.

Table 2 – Dam 1 – Maximum compressive (negative) and tensile (positive) stresses, in MPa, obtained with the different scenarios (in brackets the variations compared with NWL + Summer).

	Upstream face		Downstream face	
Scénario	Compressive stress	Tensile stress	Compressive stress	Tensile stress
NWL	- 8.5	2.1	- 6.3	2.2
NWL + Summer	- 14.2	2.1	- 8.3	3.0
NWL + Summer + 2°C	- 14.6 (- 0.4)	2.1 (+ 0.0)	- 8.5 (- 0.2)	3.1 (+ 0.1)
NWL + 10-year Summer	- 16.7 (- 2.5)	2.3 (+ 0.2)	- 9.0 (- 0.7)	3.6 (+ 0.6)

Table 3 – Dam 2 – Maximum compressive (negative) and tensile (positive) stresses, in MPa, obtained with the different scenarios (in brackets the variations compared with NWL + Summer).

	Upstream face		Downstream face	
Scénario	Compressive stress	Tensile stress	Compressive stress	Tensile stress
NWL	-2.9	0.5	-4.1	-0.3
NWL + Summer	-5.1	0.4	-5.4	0.1
NWL + Summer + 2°C	-6.3 (- 1.2)	-0.1 (- 0.5)	-6.4 (- 1.0)	0.3 (+ 0.2)
NWL + 10-year Summer	-6.6 (- 1.5)	0.5 (+ 0.1)	-5.7 (- 0.3)	0.4 (+ 0.3)

In these examples :

- For Dam 1 (thin), the 10-year heatwave scenario generates a 4 to 5 times greater increase in stresses than the 2°C climatic drift.
- For Dam 2 (thick), stress increments between a heatwave and climatic drift are of the same order of magnitude, and the influence of climatic drift is sometimes more marked than that of a heatwave.

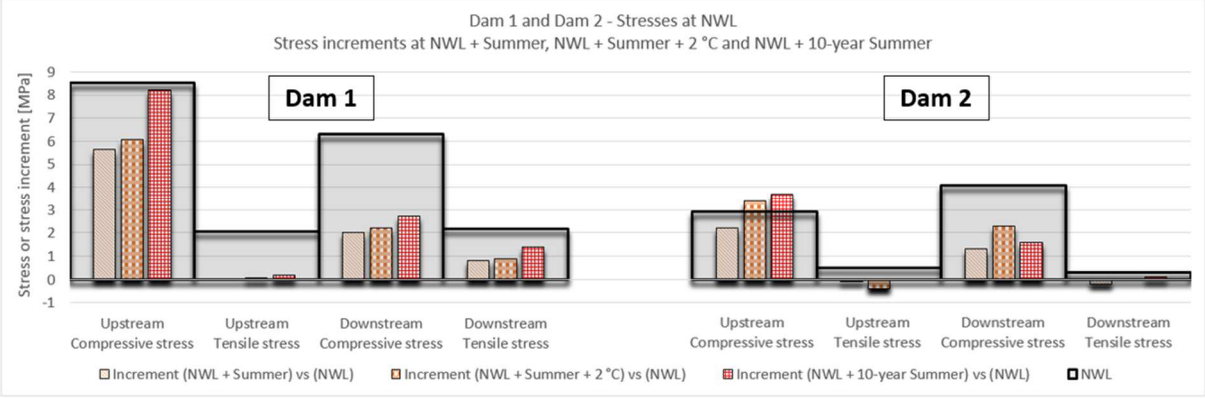


Fig. 16 – Maximum stresses at NWL (black) and stress increments at NWL + Summer, NWL + Summer + 2°C and NWL + 10-year Summer for Dam 1 and Dam 2.

In summer, as the dam moves upstream, it tends to increase the forces on its abutments. EDF analyses this effect by using the shear strength parameters (angle of friction ϕ and cohesion mobilised) required to prevent sliding. For ease of comparison, the angle of friction is arbitrarily set at 45°, so that only mobilised cohesion is compared. This lesser effect of climatic drift compared to the heatwave is also reflected in the cohesions mobilized at the dam-foundation interface under the bank blocks for Dam 1, as illustrated in Fig. 17.

In fact, we can see that climatic drift of 2°C results in an increase of 50 kPa under block 6 compared with the NWL + Summer scenario, whereas the 10-year Summer generates an increase of around 200 kPa in mobilized cohesion.

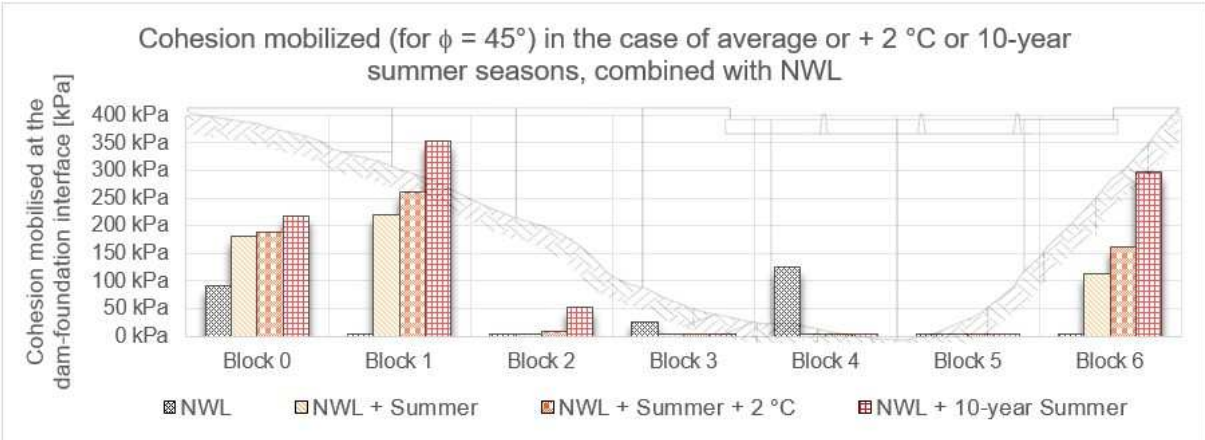


Fig. 17 – Dam 1 – Mobilized cohesions under the arch dam blocks at NWL, NWL + Summer, NWL + Summer + 2°C and NWL + 10-year Summer.

For Dam 2, cohesion increments mobilised by the heatwave and climatic drift are of the order of a few tens of kPa at most, and of the same order of magnitude, that's why they are not shown here.

It seems that the way in which climatic drift and heatwaves are modelled has a strong influence on displacements, stress and cohesion increments mobilised. The difference in modelling shown in Fig. 7 explains why the displacement caused by the heatwave is not proportional to that caused by climatic drift. In fact, the cantilever effect caused by higher temperatures on the downstream face during a heatwave tends to increase the upstream displacement of the dam.

For Dam 1 (thin), an increase in temperature on one of the faces of a thin arch dam in summer seems to have a much greater influence on the behaviour of the structure than a slow, widespread climatic drift of 2°C over forty years.

For Dam 2 (thick), it is difficult to draw conclusions about the prevalence of climatic drift or heatwaves in stress increments. It seems that both phenomena lead to stress increments of the same order, and that these are relatively negligible compared to the stresses already developed in the dam, which does not change the conclusions about the behaviour of the dam.

Finally, these examples – and others, not shown in this article – shows that:

- For a thin arch dam as Dam 1, an exceptional climatic event (in this case, a heatwave) is much more important for assessing the behaviour of a dam with regard to its potential failure modes under increased summer thermal conditions.
- For a thick arch dam as Dam 2, the influence of climatic drift and exceptional climatic events on the stresses and cohesions mobilised at the dam-foundation contact is much less marked, in particular because of the inertia of this type of arch dam.

In the end, applying a thermal gradient to a thin structure that can bend has more effect in terms of stress than applying a uniform compressive stress. If the structure is thicker, it has more bending inertia, so it is less sensitive to the thermal gradient than the thin structure.

4. CONCLUSIONS

Temperatures are estimated to have risen by around 2 degrees since 1980 in temperate regions, with significant local disparities. Measurements of the displacement of structures, using pendulums or planimetry, provide relevant data for assessing the thermal effects on the behaviour of arch dams. These monitoring data, processed using the HST and HSTT numerical models developed by EDF, make it possible to assess the effects of heating and compare them with the usual thermal stresses. These effects are fairly small and give rise to median equivalent deformations at the crest of around 0.6 µm/m/year between 2010 and 2019,

or around 15 µm/m over 40 years, with variability depending on the structure. Median upstream displacements of the crest arc are estimated at around 1 to 2 mm over the same period.

Because of the irreversible nature of climatic drift and its addition to reversible seasonal effects, particularly in summer, the question arises as to how it should be incorporated into numerical modelling of the structure's behaviour. Indeed, behaviour of arch dams is only marginally affected by climatic drift. While thin arch dams are much more sensitive to exceptional climatic episodes, rather than the slight increase due to the gradual rise in temperature, thick arch dams are much less sensitive to these exceptional events and to climatic drift.

Regarding the orders of magnitude and the envelope load cases already studied, this climatic drift is considered negligible and not taken into account when it is the only source of irreversibility. For structures affected by a concrete swelling pathology, it appears to be of the second order and often cannot be dissociated from the swelling phenomenon.

In conclusion, in finite-element models of thin arch dams, it seems more sensible to take into account exceptional climatic events, such as heatwaves, rather than the slight increase due to gradual temperature rise. For thick arch dams, it is not necessary to take into account exceptional climatic events or climatic drift. An exception could be made for structures for which the subject has already been raised historically, or those for which a particularly extreme sensitivity to thermal conditions has been proven.

Other effects of climate change, namely ones related to sudden increase of water level, due to extreme floods, and to the effects of long periods of empty reservoir, due to droughts, are not studied in this paper.

REFERENCES

[1] Météo France (February 2020 [Online]). Climate change in France. https://meteofrance.com/changement-climatique/observer/le-changement-climatique-en-france

[2] Copernicus (January 2024 [Online]). Copernicus: 2023 is the hottest year on record, with global temperatures close to the 1.5°C limit. https://climate.copernicus.eu/copernicus-2023-hottest-year-record

[3] Suchier, L., Robbe, E. (2023). Characterization of arch dam’s behaviour during increased thermal load such as heat or cold wave, 12th ICOLD European Club Symposium, 5-8 September 2023, Interlaken.

[4] Penot, I., Fabre, J.-P., Daumas B., 2009. Analyse et modélisation du comportement des ouvrages de génie-civil par la prise en compte des températures de l’air : méthode

HST Thermique. Q91-R60, Commission internationale des grands barrages, Brasilia, 2009.

[5] ONU (August 2021 [Online]). Global warming is accelerating at an unprecedented rate and it is definitely due to human activity (IPCC).

https://news.un.org/fr/story/2021/08/1101402

Fifth International Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

INCREASING RESERVOIR STORAGE CAPACITY BY USING THE FUSEGATE® SYSTEM IN THE BRAZILIAN STATE OF BAHIA

Cesare De Simone[1]
Hydroplus

Franck Del Rey[2]
Hydroplus

ABSTRACT

Over the last few years, the Water Resources Department of the Bahia State in Brazil has been working on developing a strategic plan to increase water supply for agriculture and human consumption. This plan aims to mitigate the impacts of droughts and to respond to the demographic increase in certain areas of the country by implementing solutions with low carbon emission and minimal environmental impact.

In this regard, many actions have been taken such as construction of new dams, optimization of water infrastructure in large cities, and organizing educational campaigns on water consumption.

The strategic plan also includes increasing the reservoir storage capacity of three dams through the installation of the innovative Fusegate® System. Previously, feasibility studies were performed to evaluate the benefits of additional storage and to verify that the dams could withstand the additional hydrostatic loads without compromising their integrity. The Fusegate® solution has proven to be cost-effective, and at the same time maximize efficiency and sustainability.

The reservoirs that benefited from this optimization involve Ponto Novo, França and Rio Tapera dams. For each of them, an additional water storage capacity of +24%, +29%, and +40% was achieved, respectively.

This article presents the different phases of each project covering the innovative approaches used, the cost savings involved, and the benefits gained by the Bahia State Government and the local populations.

Keywords: Fusegate®, storage, sustainability, innovative.

[1] cesare.desimone@hydroplus.com, Brazil

[2] franck.delrey@hydroplus.com, France

1. INTRODUCTION

The Bahia State Water Resources Department has been developing a strategic plan to enhance their water supply infrastructure in recent years. Approximately two-thirds of Bahia falls within Brazil's semi-arid "Drought Polygon," and the demand for water in this region is rapidly increasing.

The strategic plan includes constructing new dams, expanding existing reservoirs, establishing water lines, improving infrastructure in densely populated areas, and promoting water conservation through educational campaigns.

As part of this initiative, storage capacity has been increased in three reservoirs using the Fusegate® system. This corresponded to an increase by approximately 24% in Ponto Novo, 29% in França, and 40% in Rio Tapera, boosting water availability across the region.

2. DESCRIPTION OF THE DAMS

2.1. Ponto Novo Dam

Built in 1998, Ponto Novo is an earthfill dam with a central RCC spillway. Initially designed to supply water to the municipalities of Ponto Novo and Filadelfia, and to irrigate 1,000 hectares of land, its coverage area has since expanded to support Senhor do Bonfim's water supply system and transfer water to the Pedras Altas reservoir feeding the SISAL Pipeline System.

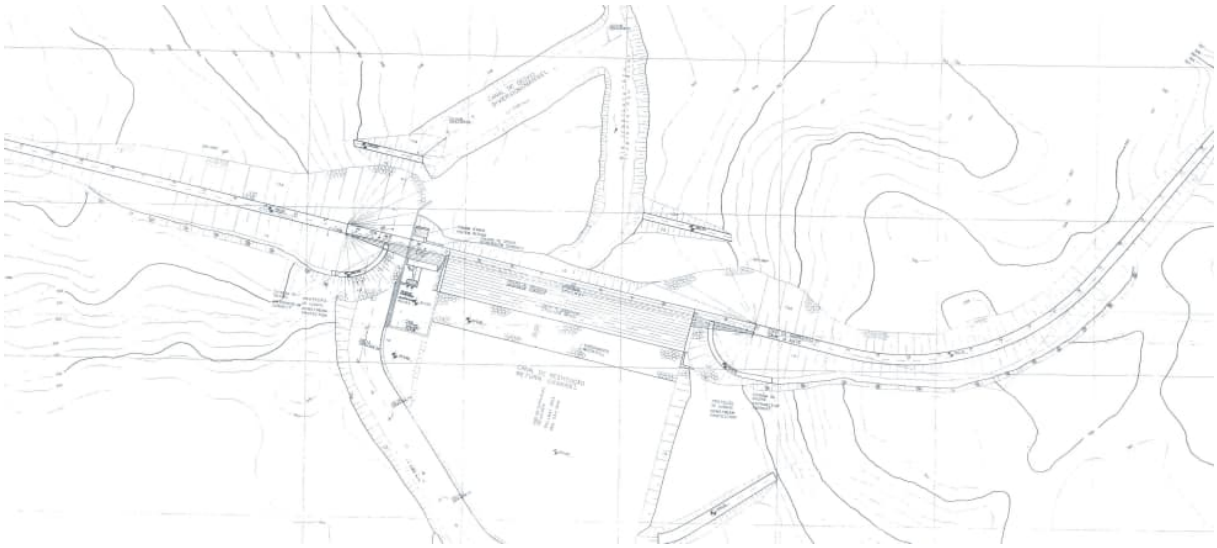


Fig. 1 – Ponto Novo Dam

By the early 2000s, prolonged droughts reduced the reservoir's storage to 21% by October 2012. In response, the Companhia de Engenharia Hídrica e de Saneamento da Bahia (CERB),

which owns the dam, opted to increase storage capacity without raising the dam height or requiring active operation.

Main characteristics of the dam are described in Table 1

Table 1 – Ponto Novo Dam characteristics

Dam	
Type	Earthfill dam with central core
Height	25.50m
Full Supply Level (FSL)	El. 360.30m
Maximum Water Level	El. 363.94m
Dam Crest elevation	El. 365.50m
Volume at FSL	38.56 x $10^6 m^3$
SPILLWAY	
Crest elevation	El. 360.30m
Length	160.00m
Maximum outflow	2 385 m^3/s

The selected solution to increase storage capacity was to raise the Full Supply Level (FSL) of the dam by 1.20 meters by installing 58 Fusegate® units along the spillway, adding 9.4 million cubic meters (+24%) to the reservoir's capacity.

The main characteristics of the adopted solution are listed below:

Table 2 – Fusegates® characteristics

Type of Fusegates®	Labyrinth
Number of units	58
Fusegate® height	1.20m
Fusegate® width	2.75m
Length of the spillway equipped with Fusegates®	160.00m
Fusegate® crest elevation	El. 361.30m
Volume at FSL	47.96 x $10^6 m^3$

2.2. França Dam

França Dam is a rockfill dam, which was built in 1996 and impounds Jacuipe River. Located near the municipalities of Piritiba and Miguel Calmon (BA), it is used for supplying water for drinking and irrigation. The reservoir has a capacity of 24.2 x $10^6 m^3$ at the FSL (512.00m).

CERB selected Fusegates as the spillway control system to increase the capacity of this reservoir without raising the dam crest. The solution adopted consists of raising the FSL from elevation El. 512.00m to El. 513.50m using 1.50m high Fusegates. This 1.50m increase in FSL provided an additional storage capacity of 7.0 x $10^6 m^3$ (+29% increase).

Fig. 2 – França Dam

Main characteristics of the dam are described in Table 3

Table 3 – França Dam characteristics

Dam	
Type	Rockfill dam
Height	25.50m
Full Supply Level	El. 512.00m
Maximum Water Level	El. 517.14m
Crest elevation	El. 522.00m
Volume at FSL	24.19 x $10^6 m^3$
SPILLWAY	
Crest elevation	El. 512.00m
Length	55.00m
Maximum outflow	1 300 m^3/s

As part of the upgrade, nine precast concrete Fusegates measuring 6.11m long and 1.50m high were installed on the 55.00m long spillway. The main characteristics of the adopted solution are listed below:

Table 4 – Fusegates® characteristics

Type of Fusegates®	Straight crested
Number of units	9
Fusegate® height	1.50m
Fusegate® width	6.11m
Length of the spillway equipped with Fusegates®	55.00m
Fusegate® crest elevation	El. 513.50m
Volume at FSL	31.19 x $10^6 m^3$

2.3. Rio Tapera Dam

Rio Tapera Dam is an earth fill dam with a side spillway, which was built to also impound Jacuipe River. Its purpose is to supply water to the integrated supply network of Itaparica and Vera Cruz.

The reservoir has a stored volume of 4.54 x $10^6 m^3$ at elevation El.15.60m. EMBASA selected Fusegates as the spillway control system to increase the reservoir's storage capacity to meet the increasing demand due to the implementation of the Salvador-Itaparica Island bridge project. The solution adopted consists of raising the FSL from El.15.20m to El.17.04m using 1.84m high Fusegates. The 1.84m rise in FSL provides an additional storage capacity of 1.8 x $10^6 m^3$ (+40% increase).

Fig. 3 – Rio Tapera Dam

Main characteristics of the dam are described in Table 5

Table 5 – Rio Tapera Dam characteristics

Dam	
Type	Earthfill dam
Height	18.00m
Full Supply Level	El. 15.20m
Maximum Water Level	El. 18.50m
Crest elevation	El. 19.00m
Volume at FSL	4.54 x $10^6 m^3$
SPILLWAY	
Crest elevation	El. 15.20m
Length	8.00m
Maximum outflow	40 m^3/s

As part of this upgrade, two straight crested Fusegates, 4.00 m long and 1.84 m high each, were installed on the 8.00 m long spillway. The main characteristics of the adopted solution are listed below:

Table 6 – Fusegates® characteristics

Type of Fusegates®	Straight crested
Number of units	2
Fusegate® height	1.84m
Fusegate® width	4.00m
Length of the spillway equipped with Fusegates®	8.00m
Fusegate® crest elevation	El. 17.04m
Volume at FSL	6.34 x $10^6 m^3$

3. OPERATION OF THE FUSEGATE SYSTEM

The Fusegate System operates as a straight-crested or a labyrinth spillway divided into individual segments or "Fusegates", each forming a component of the spillway. These modules are installed on the concrete spillway sill and remain in place solely by gravity.

Concrete or steel toe abutment blocks anchored to the spillway sill prevent Fusegates from sliding downstream. When the water level reaches a predetermined elevation, the designated Fusegate(s) overturn and washed away downstream, thereby increasing the spillway's discharge capacity.

The modules are rotationally fused and the tipping is triggered by water entering the Fusegate's pressure chamber through a supply chimney called an inlet well installed at a certain height. Once water enters this pressure chamber, the water pressure within it increases rapidly, causing the Fusegate to rotate around the toe abutment blocks and tip downstream.

By modifying the elevation of each inlet well, each Fusegate can be designed to activate at a different elevation so that an excessive increase in flow downstream does not occur.

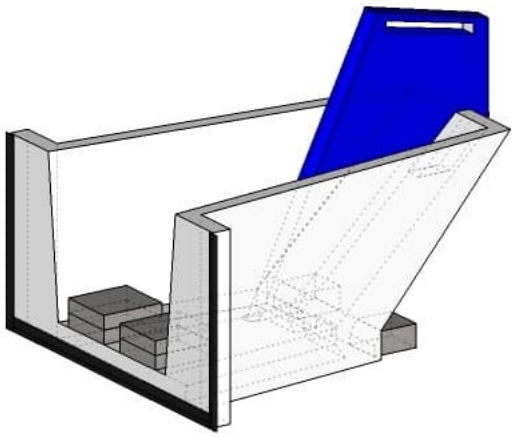


Fig. 4 – Labyrinth Fusegate

The discharge per unit length over a labyrinth Fusegates is calculated by the following formula:

$$q(m^3/s) = A \cdot h + B \quad (1)$$

where:

h = hydraulic head over the Fusegate crest

A, B = hydraulic coefficients of a particular Labyrinth Fusegate

The discharge per unit length over a straight crested Fusegates is calculated by the following formula:

$$q(m^3/s) = C_{FG} \cdot \sqrt{2g} \cdot h^{\frac{3}{2}} \quad (2)$$

where:

h = hydraulic head over the Fusegate crest

C_{FG} = discharge coefficient of the straight crested Fusegate

Once Fusegates have tipped, the discharge per unit length over the spillway is defined by:

$$q(m^3/s) = C_d \cdot \sqrt{2g} \cdot h^{\frac{3}{2}} \quad (3)$$

where:

h = hydraulic head over the spillway sill

C_d = discharge coefficient of the spillway sill

4. DESIGN OF THE FUSEGATE SYSTEM

The design phase of the Fusegate system generally involves the three steps listed below.

Hydraulic and dam stability check

In this initial stage, which is necessary prior to any modification of the spillway, the stability of the dam should be verified under the new loading conditions to which the dam will be exposed due to the installation of the Fusegates.

Thus, the current stability of the dam should be analysed based on the new normal loading conditions (FSL) as well as the new extreme loading conditions (MWL).

Design of Fusegates

The design of Fusegates includes the structural design of the modules, which can be made in pre-cast reinforced concrete, cast-in-place reinforced concrete (in case they are too large to be lifted) or steel. It also includes the design of all components of the system (inlet wells, toe abutment blocks, watertightness sealing system, deflectors, etc.). In addition to the structural design, the stability of each Fusegate module and its hydraulic behaviour is checked against different design floods.

Model tests

In order to validate the Fusegates' design, hydraulic models can be performed to verify (i) the correct activation of the Fusegate modules after feeding of the inlet well, and (ii) the water profiles in the vicinity of the inlet wells.

Fig. 5 – Ponto Novo Fusegate model test – Hydroplus lab.- 2017

5. FUSEGATE SYSTEM INSTALLATION

5.1. Ponto Novo Dam

The works include the installation of 58 Fusegate modules in pre-cast concrete with 1,20 m height during a season with the lowest probability of flooding of site (from September to December). Due to the challenges of the work, especially with regard to the installation of the Fusegate modules, a specific methodology was developed for handling them on the spillway.

Construction of pre-cast Fusegates

The Fusegates were constructed at a pre-cast plant in the vicinity of the dam to minimize the transportation distance. The weight of each module is 5 tons. To facilitate the removal of the formwork, lifting hooks were included in the structure. Prefabrication also allowed for better control of the construction works.

Fig. 6 – Ponto Novo Fusegate formwork removal

Spillway adaptation

The crest of the spillway sill had to be adapted to retrofit the prefabricated Fusegates. Therefore, the adaptation of the spillway and the construction of the new broad crested weir were carried out in parallel with the prefabrication of the Fusegates.

Fig. 7 – Spillway adaptation. Ponto Novo dam

Fusegates installation

At the final stage, the Fusegates we installed on the new spillway sill. To carry out this task, a crane was used to lower the modules one after another on the spillway sill.

Fig. 8 – Fusegate installation. Ponto Novo dam

Due to the length of the spillway (160 meters), the Fusegates were moved using a wheel system that allowed each module to be placed exactly in the position required by the project.

Fig. 9 – Wheeling system. Ponto Novo dam

Shortly after the completion of the works, which coincided with the arrival of the wet season, the reservoir level reached the FSL, that is, the crest of the Fusegate, thus immediately demonstrating the benefits of raising the spillway.

Fig. 10 – Outflow at Ponto Novo spillway equipped with Fusegates

Behaviour of the spillway equipped with Fusegates is presented in the rating curve hereafter. Tipping of the 1st sequence of Fusegates occurs for an outflow greater than 1250m³/s.

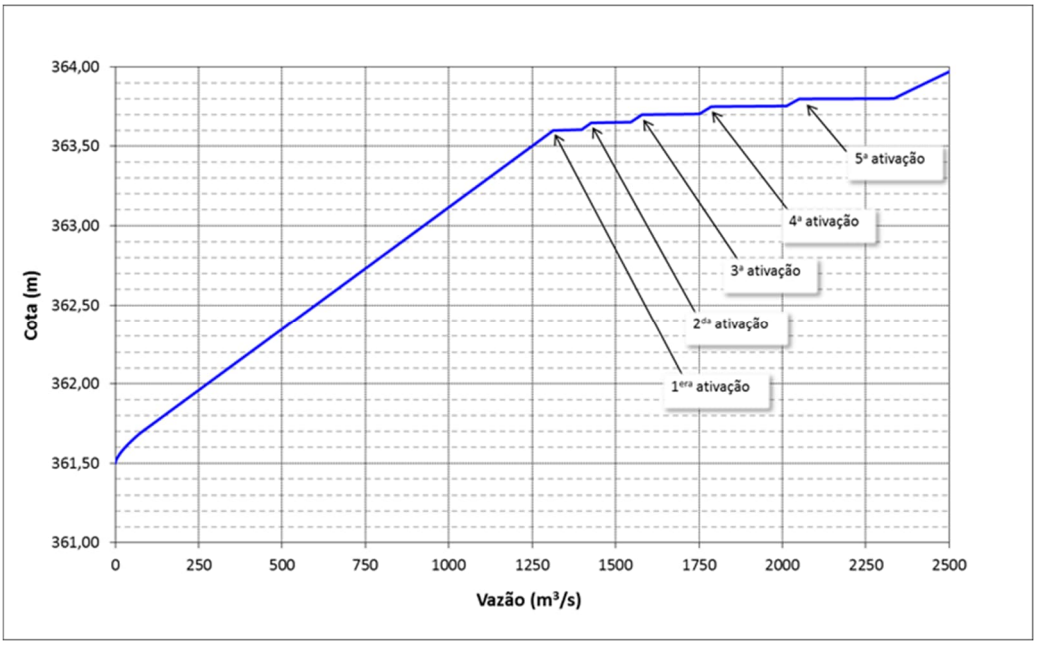


Fig. 11 – Ponto Novo spillway rating curve

5.2. França Dam

In the case of França dam, there were nine pre-cast concrete Fusegate modules having a height of 1,50m. The installation was carried out during the dry season when upstream of the spillway was still accessible.

Fig. 12 – Upstream view of França dam spillway

Fabrication of the Fusegates

Also for this project, it was possible to find a precast company in the vicinity of the dam. The weight of each element is 11 tons.

Fig. 13 – Pre-cast concrete Fusegates. França dam

Spillway sill adaptation

The crest of the spillway sill needed to be adapted to be able to retrofit the prefabricated Fusegates. Therefore, the adaptation of the spillway and the construction of the new broad crested weir were carried out in parallel with the prefabrication of the Fusegates.

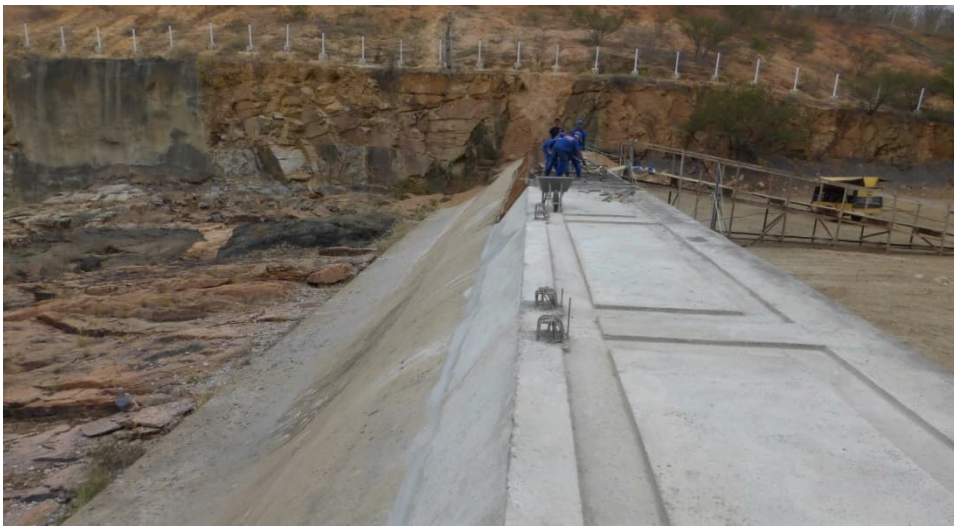


Fig. 14 – Execution of the spillway sill. França dam

A mobile crane set-up at the upstream of the spillway sill was used for the installation of the pre-cast Fusegates, which took just one week.

Fig. 15 – Installation of the Fusegates using a mobile crane. França dam

The Fusegates were successfully tested a few months after their installation, when a flood of approximately 100m³/s has occurred. There was no tripping of any of the Fusegate modules as the reservoir level did not reach the level of the inlet wells.

Fig. 16 – Flood at França dam

Performances of the spillway equipped with Fusegates are described in the rating curve hereafter. It is also presented a comparison between previous and actual spillways.

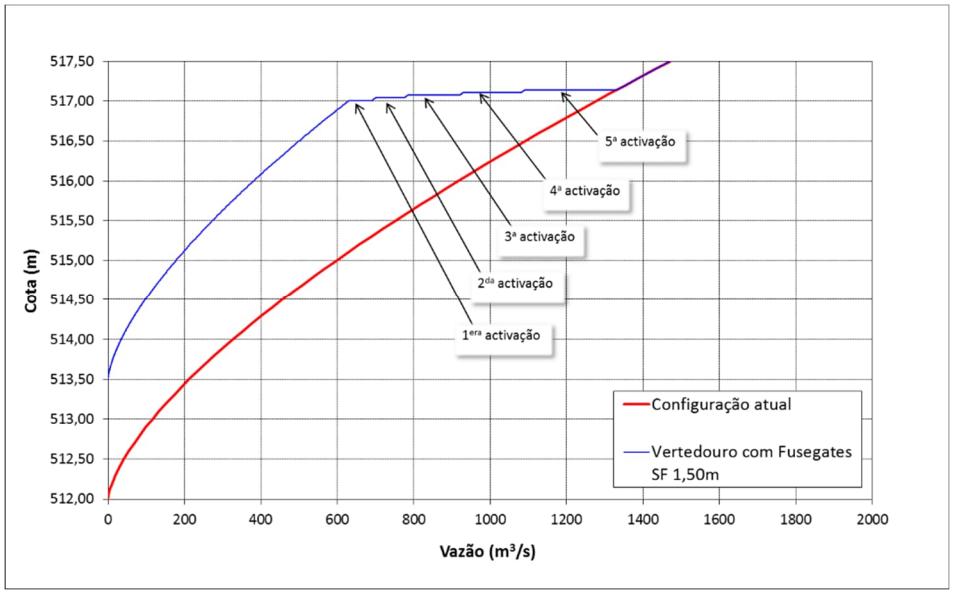


Fig. 17 – França spillway rating curve

5.3. Rio Tapera Dam

Two pre-cast concrete Fusegates with 1,84 m height were needed for the Rio Tapera spillway which has a length of only 8.00 m.

Fabrication of the Fusegates

For this project, the inlet wells are not located at the crest of the Fusegates, and are part of a remote intake structure with a conduit at the upstream of the spillway sill. This allows for better hydraulic efficiency of the system as well as greater safety against floating debris of a certain size that can be flushed during floods.

Fig. 18 – Fusegates and the inlet wells. Rio Tapera dam

Spillway adaptation

As the spillway sill was not wide enough for the retrofit of the Fusegates, a 1.50 ft wide extension of the weir was need at the upstream section. The following photo shows the detail of the toe abutment blocks as well as the recess on the platform that will accommodate the beams of the Fusegates.

Fig. 19 – Spillway sill. Rio Tapera dam

Fusegates installation

At Ponto Novo dam, a mobile crane was also set-up at the upstream of the spillway allowing for a quick and efficient installation of the two Fusegate units.

Fig. 20 – Fusegates installation. Rio Tapera dam

At Rio Tapera dam, floods are frequent, but of moderate intensity. Due to the frequency of such floods, the reservoir regularly reaches the FSL before summer, a period of high water demand, when the local population multiplies by a factor of three.

Fig. 21 – Outflow at Rio Tapera spillway equipped with Fusegates

Performances of the spillway equipped with Fusegates are described in the rating curve hereafter. It is also presented a comparison between previous and actual spillways.

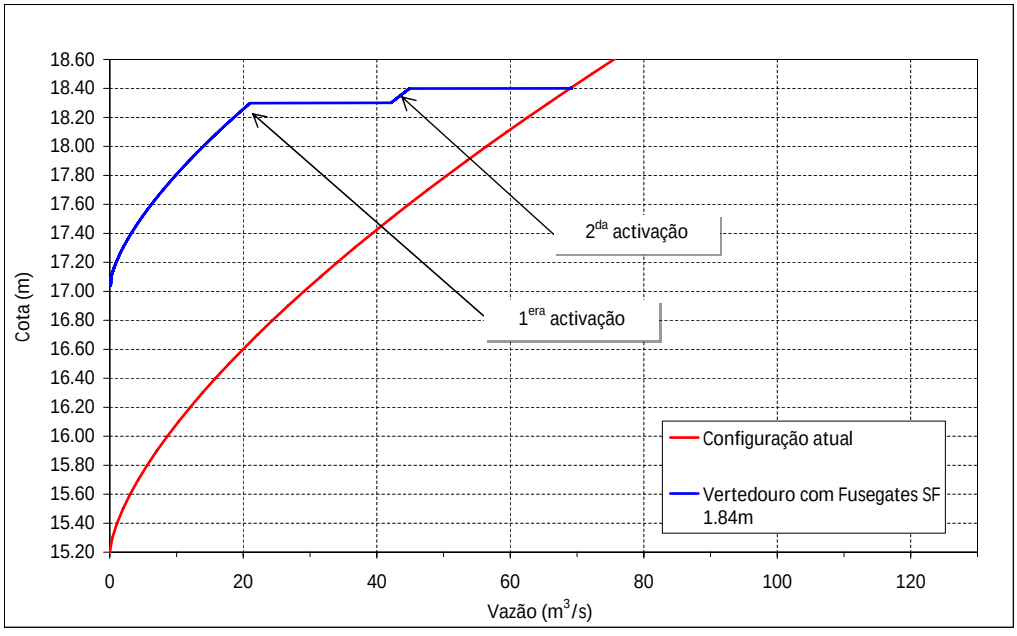


Fig. 22 – Rio Tapera spillway rating curve

6. CONCLUSIONS

The Fusegate® System offers an economical and effective means of increasing reservoir capacity without the need for any electrical or mechanical systems nor human intervention. By adding nearly 18.2 million m³ of water storage across three dams, the gains from the overall project corresponds to the equivalent capacity of a new medium-sized reservoir in the State of Bahia, contributing significantly to water security in drought-prone regions.

REFERENCES

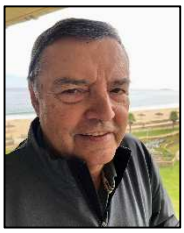
[1] Hydroplus Ltda, 2013. HYD7601R001AFCB, Aumento da capacidade do reservatório da Barragem do França com o Sistema Fusegate®, Relatório hidráulico e de estabilidade.

[2] Hydroplus Ltda, 2016. HYD7602R002ASBB, Prestação de serviço de instalação com fornecimento do equipamento Fusegate, a ser instalado na Barragem Rio Tapera. Relatório hidráulico e de estabilidade do Sistema Fusegate®.

[3] Hydroplus Ltda, 2018. HYD7603R002ASBB, Elaboração de projeto, fabricação, fornecimento e instalação de dispositivos do Sistema Fusegate visando ao aumento de capacidade de armazenamento de água do reservatório da barragem de Ponto Novo. Relatório hidráulico e de estabilidade do Sistema Fusegate®.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

CLIMATE RESILIENCE IN RESERVOIRS: RECOMMENDATIONS FROM CHILE



Carlos Garrido[1]
SMI Ingenieros Ltda.

Luis San Martín
SMI Ingenieros Ltda.

ABSTRACT

Chile faces unique challenges in water resource management due to its varied geography and the increasingly intense effects of climate change. This phenomenon has led to significant reductions in annual mean flows, especially in the northern and central regions of the country, with projections of decreases between 10% and 40% by 2050. Additionally, an increase of 15% to 30% in extreme event flows is anticipated during the same period.

The impact of climate change is evident in reservoirs such as Valle Hermoso (1,670 meters above sea level) and Cogotí (690 meters above sea level) located in semi-arid climates that have not been able to reach their maximum storage capacity in recent years. These situations demonstrate how extreme climatic conditions are surpassing the original design capacities of these reservoirs.

On the other hand, the Chironta reservoir (1,760 meters above sea level), located in a desert climate and affected by the altiplanic winter, and the Ancoa reservoir (750 meters above sea level), located 2,450 km to the south in a Mediterranean climate zone, have faced an increase in flood flows linked to the increase in maximum precipitation and the rise of the 0-degree isotherm. In Chironta, this phenomenon negatively impacted the diversion works during its construction, while in Ancoa, already in operation, it caused significant damage downstream of the energy dissipator.

These cases show how climate change affects both reservoirs in arid northern regions and temperate southern climates in Chile. It is crucial to integrate these effects into the design, construction, and operation of future reservoirs.

This paper synthesizes Chilean experience in adapting reservoirs to climate change, covering design and operation. Key recommendations include the implementation of advanced hydrological methodologies, snow line estimation, the incorporation of drainage and regulation works, multipurpose use of reservoirs, and dynamic management measures.

These recommendations are essential to ensure the sustainability and resilience of reservoirs in Chile and other regions facing similar challenges, protecting water resources for future generations.

Keywords: Climate change, resilience, reservoirs, hydrology, sustainability.

[1]cgarrido@smi-chile.cl, Chile

1. INTRODUCTION

Climate change represents one of the greatest global challenges of the 21st century, directly impacting natural, social, and economic systems. In particular, water resources have been severely affected by rising temperatures, changes in precipitation patterns, increased frequency of droughts, and extreme weather events such as floods (CEDEX, 2017; Chilean Ministry of the Environment, 2020). This phenomenon has significant implications for water security and the management of hydraulic infrastructure, especially in highly vulnerable countries such as Chile (Bautista Yáñez, 2020). Figure 1 illustrates the impacts of climate change on Chile's macro-zones, highlighting critical sectors such as mining, agriculture, and energy, along with temperature and precipitation projections up to 2100, which reflect the country's differentiated vulnerability.

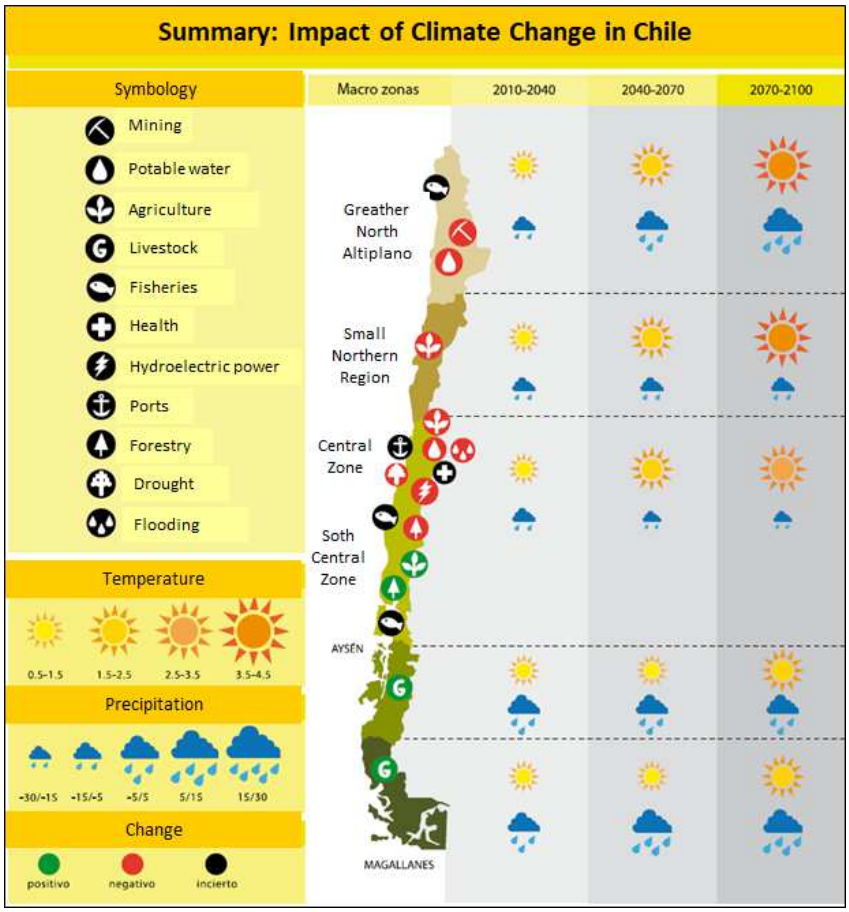


Fig. 1 – Impact of climate change in Chile

Chile faces a unique context where its geographic and climatic diversity exacerbates the effects of climate change. The northern region of the country, characterized by its aridity, is already experiencing a critical decline in its water reserves, while the central and southern zones, historically reliant on reservoirs for irrigation, hydroelectric generation, and human supply, are facing increased hydrological variability and extreme weather events (Chilean

Ministry of the Environment, 2020). In this scenario, reservoirs become key infrastructures for ensuring the country's water security but are also particularly vulnerable to the challenges posed by climate change (CEDEX, 2017; Bautista Yáñez, 2020).

The main issue lies in the limited adaptive capacity of many dams and reservoirs to new climatic regimes. Recent studies highlight challenges such as the reduction of average water flows, increased sedimentation that decreases storage capacity, and infrastructure overloads caused by more frequent and intense floods (Bautista Yáñez, 2020). Furthermore, the traditional management of these structures must evolve toward more dynamic approaches, integrating resilience and sustainability criteria into their design, operation, and planning (AEMET, 2008; Chilean Ministry of the Environment, 2020). Figure 2 highlights the key functions of reservoirs: potable water supply, irrigation, energy generation, flood control, recreation, and ecological support.

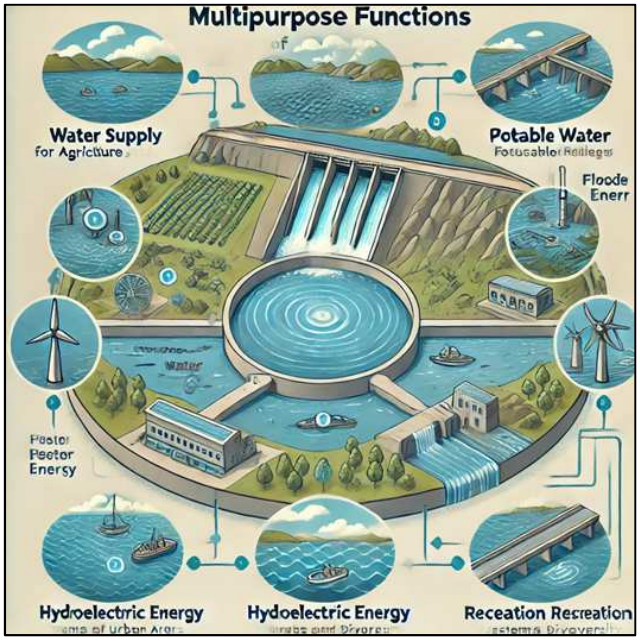


Fig. 2 – Multipurpose functions of reservoirs

This study aims to analyze Chile's experience in adapting reservoirs to climate change, focusing on four specific cases (Chironta, Valle Hermoso, Cogotí, and Ancoa) that illustrate the technical and operational challenges in this context. Through these case studies, the goal is to highlight lessons learned and propose practical recommendations to enhance the resilience of these essential infrastructures. This analysis is not only relevant locally but also provides valuable insights applicable to other regions worldwide facing similar challenges, contributing to global knowledge in this field (CEDEX, 2017; Bautista Yáñez, 2020).

2. CLIMATE CHANGE AND RESERVOIR MANAGEMENT IN CHILE

2.1. Climate Projections in Chile: Decrease Flows and Extreme Events

Climate projections in Chile reveal a significant trend toward decreasing river flows and an intensification of extreme events such as droughts and floods. Regarding precipitation, studies indicate a reduction of up to 30% in rainfall by the end of the 21st century in the central-southern regions under high-emission scenarios, directly affecting water availability for human consumption, agriculture, and hydroelectric generation (Ministry of the Environment, 2023; Lagos-Zúñiga et al., 2024). Simultaneously, an increase in the frequency of extreme precipitation events is expected in certain areas of the Altiplano and southern Austral zones, with significant impacts on infrastructure and local ecosystems.

Regarding extreme events, recent analyses identify complex patterns. For instance, while central Chile shows a decrease in intense rainfall events (Rx1day), the northern and southern regions exhibit significant increases in such events during summer and winter, respectively (Lagos-Zúñiga et al., 2024). This is illustrated in the following figure.

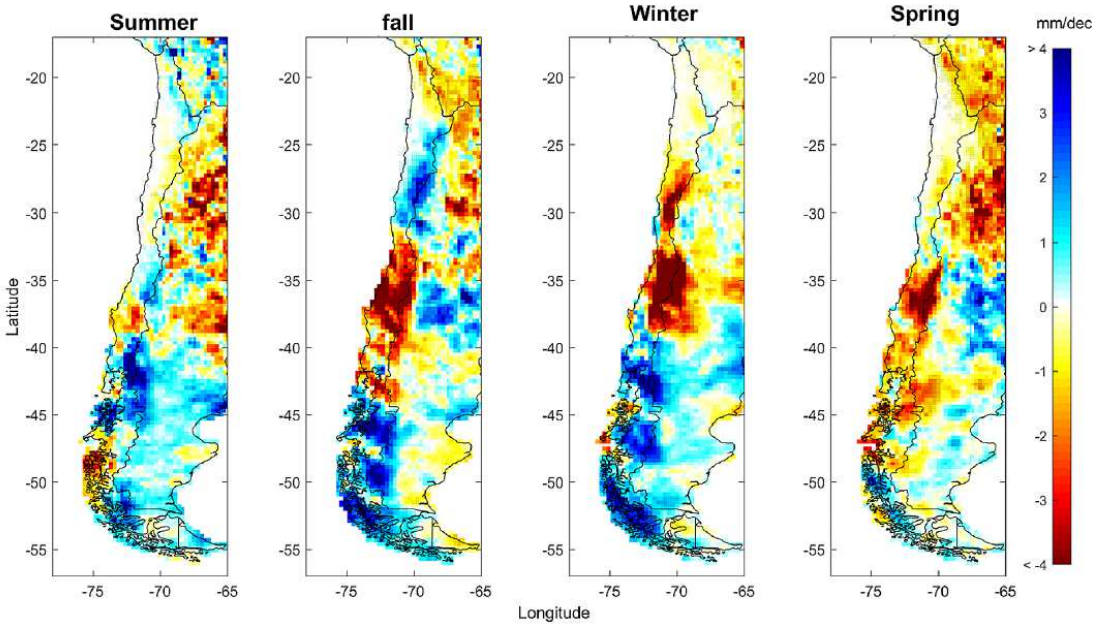


Fig. 3 – Future projections of extreme events such as Rx1day (maximum daily precipitation)

These trends are associated with the poleward expansion of the Hadley cell and an intensification of water vapor transport in southern regions during extreme precipitation events, exacerbated by atmospheric rivers (Aceituno et al., 2021). Additionally, climate projections suggest that rising temperatures contribute to an increase in the altitude of the 0°C isotherm, heightening the risk of floods caused by accelerated snowmelt and extreme rainfall events in mountainous basins (Mardones and Garreaud, 2020).

These phenomena pose critical challenges for the management of reservoirs and other water infrastructures, which must adapt to increased hydrological variability by enhancing their capacity to handle prolonged droughts and sudden floods. The integration of historical trend data and climate projections will be key to designing resilient strategies to address climate uncertainty.

2.2. Climatic Diversity and Its Implications for Reservoir Design

The following table summarizes the main climatic characteristics and the projected impacts of climate change in each zone, highlighting the importance of adaptive strategies in the design of water infrastructure (Mardones & Garreaud, 2020; Ministry of the Environment, 2023).

Table 1 – Climatic Subdivision of Chile and Climate Change Impacts on Water Resources and Extreme Events

Zone	Latitude	Climate	Climate change impacts
Greather North	17°S - 25°S	Arid climate, low rainfall, predominance of deserts	Increased water scarcity, sporadic torrential rains causing landslides
Small North	26°S - 32°S	Semi-arid climate, transition to higher rainfall	Reduced precipitation, increased frequency and intensity of droughts
Central Zone	33°S - 35°S	Mediterranean climate, high agricultural activity	Decreased river flows, flood risks, and lower water availability for irrigation and hydropower
South-Central Zone	36°S - 38°S	Mediterranean climate in Ñuble, temperate rainy in Maule and Biobío, abundant watersheds	Prolonged droughts, river floods impacting reservoirs like Ancoa
Southern Zone (Patagonia)	39°S - 56°S	Cold and rainy climate, presence of glaciers and fjords	Intense storms, accelerated melting affecting hydropower generation and glacier ecosystems

Chile, with its vast latitudinal range and geographical diversity, exhibits remarkable climatic variability that directly impacts the design and management of reservoirs. This climatic diversity spans from the arid climate of the Norte Grande, characterized by low precipitation and high evaporation rates, to the cold and rainy climate of Patagonia, home to abundant glaciers and fjords (Ministry of the Environment, 2023). Between these extremes, there is the semi-arid climate of the Norte Chico, transitioning toward higher precipitation levels, the Mediterranean climate of the Central Zone, ideal for agricultural activities, and the temperate rainy climate of

the South-Central Zone, dominated by watersheds and forests (Aceituno et al., 2021; Lagos-Zúñiga et al., 2024).

Therefore, reservoirs must be designed to account for both efficient storage capacity and resilience to extreme events.

2.3. Observed Impact on Specific Reservoirs

Chile faces climatic diversity that differentially impacts the operation and sustainability of its reservoirs, particularly in the context of climate change. Below, the observed impacts on the Chironta, Valle Hermoso, Cogotí, and Ancoa reservoirs are detailed, considering variations in water flows, extreme events, sedimentation, and geological risks. The location of these projects is presented in the following figure.

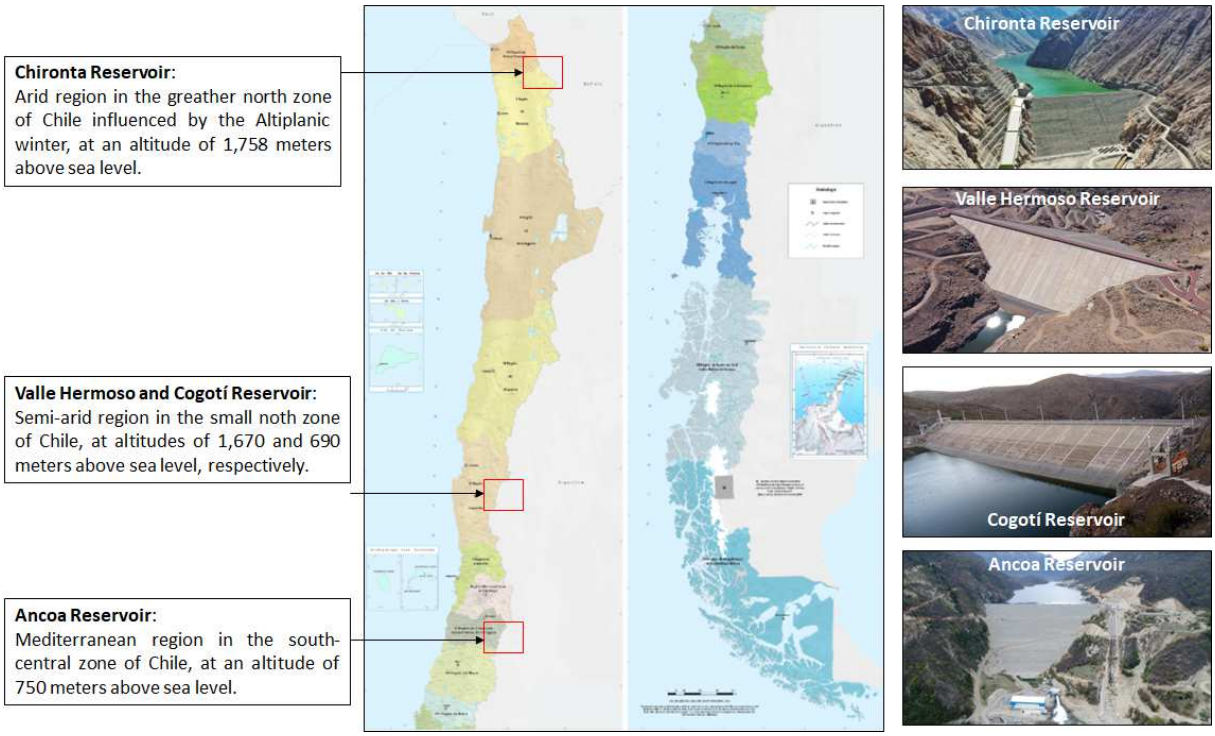


Fig. 4 – Location map of Chironta, Valle Hermoso, Cogotí, and Ancoa Reservoirs

The Chironta reservoir, located in an arid region of the Norte Grande but influenced by the Altiplanic winter, relies on water inputs from seasonal storms in the mountains. This climatic pattern can cause sudden floods and extreme flows, complicating water regulation. A significant example occurred on January 28, 2019, when a storm destroyed the diversion cofferdam during the reservoir's construction, highlighting the vulnerability of these infrastructures to intense events. Additionally, rains associated with the Altiplanic winter increase sedimentation, affecting operational capacity and raising the risk of landslides on surrounding slopes.

Located in the semi-arid region of Norte Chico, the Valle Hermoso and Cogotí reservoirs face significant challenges related to water availability and infrastructure operation. Due to the prolonged drought affecting this area, water levels in these reservoirs have been severely impacted. For instance, in August 2023, reservoirs in the region averaged only 12% of their total capacity. In the case of the Cogotí reservoir, it was reported in March 2024 that the reservoir had reached 0% capacity, with the remaining water insufficient to exit through the valves, compromising its ability to regulate water supply and meet agricultural demands.

On the other hand, the Ancoa reservoir, located 2,450 km to the south in the Ñuble Region of the south-central zone, operates in a Mediterranean climate influenced by significant winter rains. During July and August 2023, intense rainfall and a rise in the 0°C isotherm caused damage to riverbank protection works downstream of the ski jump energy dissipation pool.

3. RESERVOIR CASE STUDIES

3.1. Reservoir in a Desert climate: Chironta and the effects of Altiplanic Winter

The original design of this project dates back to 2004 and did not account for the effects of climate change. Construction took place between 2017 and 2022. The reservoir features a CFGD (Concrete Face Gravel Dam) with a height of 76 meters from the foundation seal and a crest length of 274 meters, situated at an elevation of 1,759.8 meters above sea level. Its total storage capacity is 17 hm^3, of which 9.5 hm^3 is usable volume and 7.5 hm^3 is designated as dead storage for sedimentation over its lifespan.

The original design includes a 473-meter-long diversion tunnel capable of handling a flow of 381 m^3/s (Ppmax over 24 hours of 49.2 mm), and a flood evacuation system with a capacity of up to 1,275 m^3/s, designed for an event with a 1,000-year return period.

In terms of functionality, the original design of the Chironta Reservoir was solely intended to improve agricultural irrigation in the Lluta River Valley, with a sufficient dead storage volume to accommodate incoming sediment for a 50-year lifespan.

Climate projections for the coming years (2030–2060) indicate a temperature increase in the reservoir’s basin of approximately 2 to 3°C, while annual precipitation is expected to remain stable on average but with increased interannual variability, intensifying extreme events. This can be observed in the following figure.

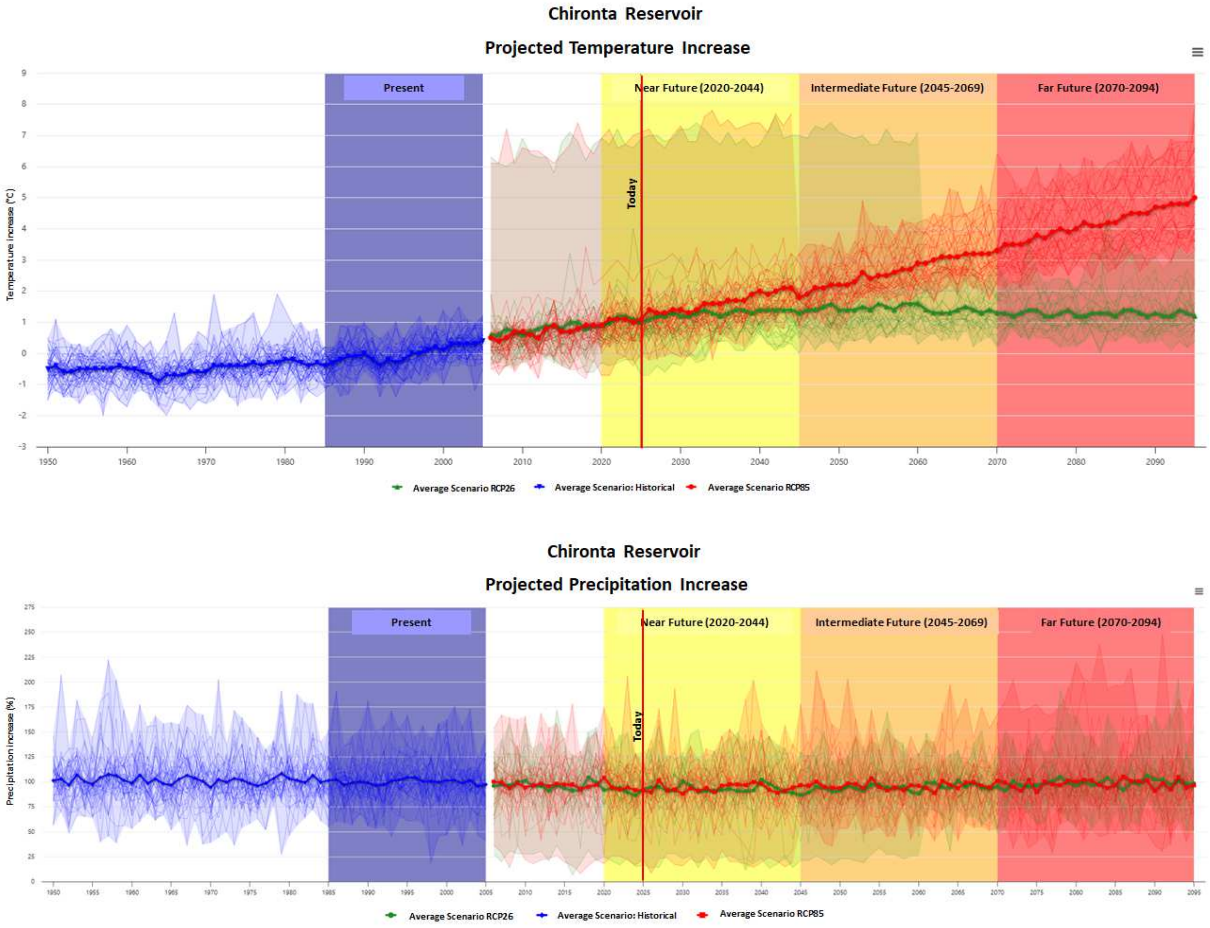


Fig. 5 – Projected Temperature and precipitation with climatic change

The Chironta Reservoir, located in the Lluta River Valley, faces significant challenges stemming from the Altiplanic winter, a seasonal phenomenon characterized by intense rainfall in the high Andes. These water inputs create significant impacts on inflows, the frequency of extreme events, and sediment accumulation, affecting both its operation and sustainability.

The Altiplanic winter causes substantial increases in inflows during January and February, as shown by seasonal runoff projections. These increases, which in some climate change scenarios and models exceed 100% compared to historical baselines, reflect a significant rise over recorded values. This pattern, typical of arid climates, amplifies intermonthly variability and complicates reservoir regulation. Looking ahead, in climate scenarios such as RCP85, a greater concentration of inflows over short periods is expected, increasing the challenges of water storage and surplus management.

Projections of maximum daily precipitation indicate an increase in the frequency and intensity of severe storms for both near-term (2020–2044) and mid-term (2045–2069) scenarios. These storms, associated with the Altiplanic winter, generate sudden and highly concentrated flow peaks that could exceed the reservoir's evacuation capacity. A significant example occurred on January 28, 2019, when a maximum 24-hour precipitation event reached 67 mm, along with

a high position of the snowline. This corresponded to a 200-year return period, far exceeding the 20-year return period considered in the design of the diversion structure, causing damage to the cofferdam during construction and highlighting the infrastructure's vulnerability to such events.

In this case, being located in an arid zone, the rise in the snowline affects the contributing area for runoff but does not impact the runoff curve number, as both the upper and lower areas lack vegetation coverage. Consequently, the increase in flows is directly proportional to precipitation and the contributing area.

As a result of the projected increase in the frequency and intensity of floods, and to protect agricultural areas and housing downstream of the reservoir (which were established within the river's natural course due to the regulation provided by the reservoir), the Public Ministry decided not to fill the reservoir to its full capacity. Instead, it is maintained 2 meters below the crest spillway threshold as a safety cushion for flood regulation, sacrificing 2 hm^3 of usable capacity (20% of total capacity).

Additionally, the Altiplanic winter brings a significant influx of sediments from the watersheds feeding the reservoir. Intense rains erode unstable soils, transporting large amounts of sediment into the system. This phenomenon progressively reduces the reservoir's usable capacity by filling the dead storage designated for sedimentation. Given the increasing frequency of extreme events, this process is expected to accelerate, further complicating the reservoir's long-term sustainability.

In response, and considering the large dead storage of this project—equivalent to 44% of the reservoir's total volume—it was decided to complement the original design during construction with the implementation of two bureau-type bottom gates. These gates are intended to facilitate flushing operations and sediment removal. Their efficiency for such tasks is currently under evaluation by the Public Ministry.

Regarding the bureau-type gates, they are currently being alternatively used to rapidly lower the reservoir levels during flood events, facilitating the creation of the regulation volume required for flood management.

An operational drawback that has arisen with the implementation of these structures is their use for delivering irrigation flows. These flows are released with a high sediment content and imprecise regulation, as the gates lack the accuracy of the Howell-Bunger valves originally designed for this purpose. This alternative use has also led to wear and damage on the bureau gates due to their continuous use, as they were initially intended for occasional operation.

3.2. Reservoirs in Semi-Arid climates: Valle Hermoso and Cogotí

The Valle Hermoso and Cogotí Reservoirs, located in the Coquimbo Region, are two essential water infrastructures for the Limarí River basin, a region highly dependent on the regulation of its water resources due to its semi-arid climate. Although designed using different technologies and constructed during different periods, these reservoirs play complementary roles in supplying water for agricultural irrigation, human consumption, and recreational activities.

The Valle Hermoso Reservoir, whose original design did not account for the effects of climate change, dates back to 2011 and was constructed between 2015 and 2019 in the Pama River basin. It features a CFRD (Concrete Face Rockfill Dam) with a height of 116 meters and a crest length of 388 meters, providing a total storage capacity of 20 hm^3. In addition to benefiting 1,500 hectares of irrigated farmland, this reservoir supplies drinking water to 17 rural communities.

The Cogotí Reservoir, built between 1934 and 1939, also without considering the effects of climate change, has a rock-fill dam with a concrete curtain measuring 87.7 meters in height and 160 meters in length, with a maximum storage capacity of 150 million m^3. In 2003, an inflatable curtain system ("Rubber Dam") was added to increase its operational capacity to 158 million m^3. However, this system has not performed as expected, as in recent years, the reservoir has failed to reach full capacity, often remaining at 0 volume for entire years.

A critical challenge faced by both reservoirs is their inability to fill in recent years. For instance, the Valle Hermoso Reservoir has only reached 20% of its originally projected capacity, and in March 2024, the Cogotí Reservoir reported 0% capacity following the filling season. This is primarily due to the drastic reduction in water resources in the region, driven by decreased precipitation associated with climate change and a prolonged period of extreme drought affecting the entire Coquimbo Region. The inability to fill limits their capacity to meet their objectives of water regulation, irrigation supply, and human consumption, significantly reducing their effectiveness in a context of increasing water demand.

Additionally, the outlook for the coming years is not promising. Projections indicate a sustained rise in temperatures within the watersheds of these reservoirs, while annual precipitation is expected to remain consistent with the historical drought period, offering little relief to the current challenges. This can be observed in the following figure.

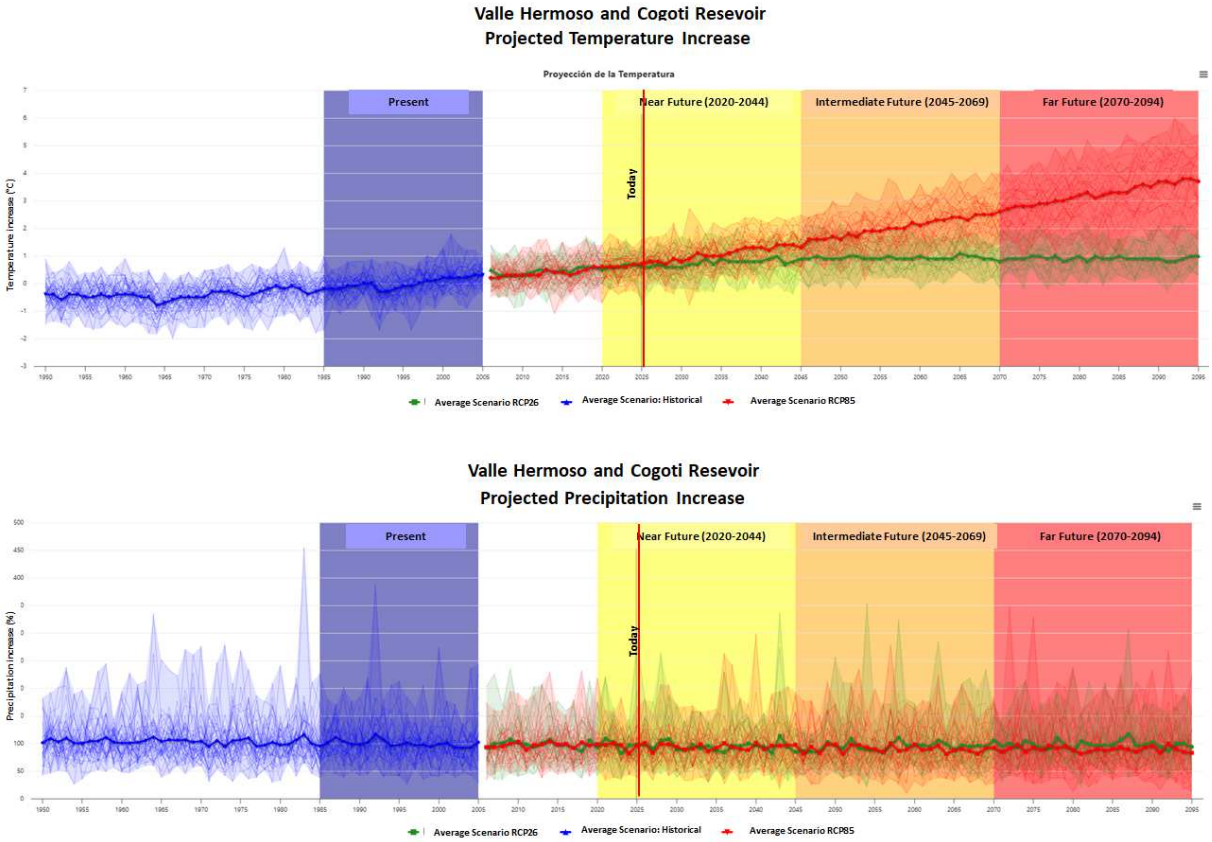


Fig. 6 – Projected Temperature and precipitation on Valle Hermoso and Cogotí reservoir with climatic change

Seasonal projections indicate that most precipitation and water inflows are concentrated during the winter months (May–July), leaving prolonged dry periods for the rest of the year. This means that reservoir operation depends on optimal management during these short recharge periods. Climate change exacerbates the situation by reducing snow accumulation in the mountains and advancing the timing of snowmelt. This not only decreases base flows during dry months but also creates more abrupt flow peaks during extreme precipitation events.

Despite having robust flood evacuation systems (such as the free-fall spillway at the Cogotí Reservoir, with a capacity of 5,000 m^3/s, and the lateral spillway at the Valle Hermoso Reservoir, designed to handle 346 m^3/s), the primary concern today is the reduction of inflows. This decline highlights the need to implement adaptive measures, including integrated watershed management, water conservation programs, and infrastructure modernization, to optimize the use of available resources.

In the case of the Valle Hermoso Reservoir, the original design envisioned an inter-annual regulation project, but the expected flow behavior has not materialized. Flows have remained constant year after year, with no observable increases influenced by the El Niño phenomenon along the Chilean coast, which had been evident in hydrological statistics prior to 2012. The same effect has been observed in the Cogotí Reservoir. Based on available flow statistics from

2003, the Public Ministry decided to incorporate rubber dam systems into the spillway to increase the reservoir's storage capacity. However, the reservoir has not been able to fill to its capacity under current conditions.

3.3. Reservoir in a Mediterranean climate: Ancoa and Its Operational Challenges

The Ancoa Reservoir, whose design study did not account for the effects of climate change, dates back to 2005 and was constructed between 2008 and 2012. It is located in the foothills of Linares Province, Maule Region, and serves to regulate water resources from the Ancoa River to meet irrigation demands for 36,000 hectares, benefiting over 3,000 downstream farmers. It also facilitates hydroelectric generation using the passing flows.

The Ancoa Reservoir features a Concrete Face Gravel Dam (CFGD) with a height of 140 meters, a storage capacity of 80 hm^3, and an inundated area of 230 hectares. The flood spillway is a free-flow lateral spillway with a chute ending in a ski jump, with original design flows (2004) corresponding to the 1,000-year flood (884 m^3/s) and verified for the 10,000-year flood (1,191 m^3/s). These floods were estimated without considering the effects of climate change, assuming a snowline position at 1,800 meters above sea level.

During June and August 2023, storms raised the snowline to 2,150 meters above sea level, resulting in a 13% increase in flood flows compared to the original design assumptions. This caused significant overflow and damage to the protection works in the energy dissipation pool, along with considerable morphological changes downstream of the Ancoa River channel.

Additionally, projections for the coming years indicate a sustained temperature rise in the reservoir's basin, while annual precipitation is expected to decrease on average. However, extreme precipitation events are projected to increase by approximately 20%, further exacerbating operational challenges.

The Ancoa Reservoir, located in the Ñuble Region, faces significant challenges associated with climate change, particularly regarding water resources and the management of extreme events. Climate projections indicate a sustained rise in temperatures, which will increase evaporation and reduce water contributions from snowmelt, affecting water availability during dry months. Additionally, although precipitation levels are expected to remain relatively stable, greater seasonal variability is projected, with flows concentrated in winter, complicating annual water regulation. This is illustrated in the following figure.

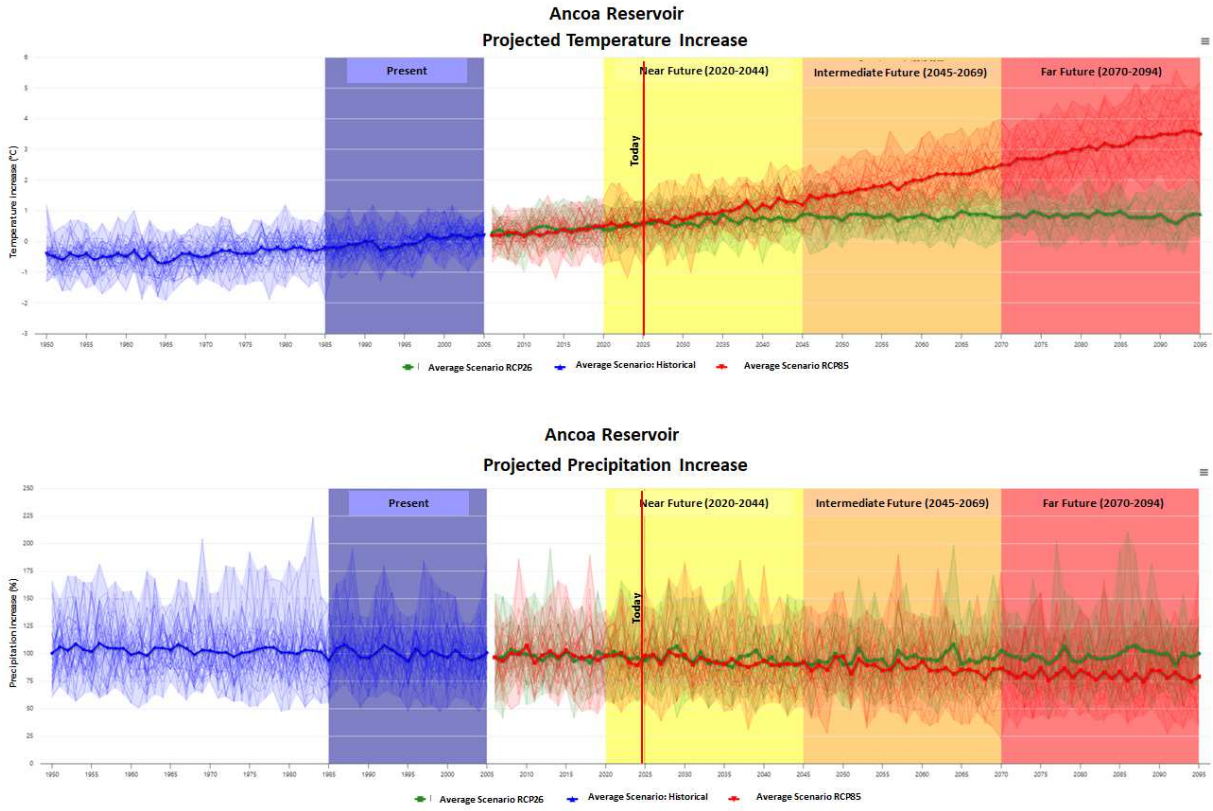


Fig. 7 – Projected Temperature and precipitation with climatic change on Ancoa Reservoir

Extreme events, such as torrential rains and abrupt snowmelt, are expected to increase the frequency of intense floods, overloading regulation infrastructure and causing damage similar to those observed downstream of the reservoir during the rains of July and August 2023. Analysis of the effects of climate change concluded that, in the case of the Ancoa Reservoir, the originally designed 10,000-year flood (1,400 m^3/s) will become the future 1,000-year flood. Additionally, the rise in the 0°C isotherm will alter the hourly distribution of precipitation, resulting in more intense rainfall events.

These events will also intensify sedimentation. With the rise in the 0°C isotherm, it is estimated that by 2060 the entire basin could transition from a pluvio-nival regime to a pluvial regime, increasing not only flood flows but also the sediment loads expected for the reservoir.

In this context, the reservoir currently relies only on irrigation delivery valves for level regulation, while the flood spillway is a free-flow type, providing no flow regulation beyond the typical volume adjustment capacity of a reservoir.

In the context of adaptation, the Ministry took steps to modify the reservoir to address climate change impacts. The first measure involved estimating future extreme flows to adjust the height of the flood spillway and implement adaptations to the ski jump and energy dissipation pool. Additionally, the riverbank protection works were reinforced, shifting their design criteria from a 100-year return period to a 1,000-year return period.

The operational manual for the reservoir was also updated to include additional uses beyond irrigation, such as flood regulation, human water supply, and hydroelectric generation. Consequently, an analysis of the reservoir's regulation capacity during floods was conducted based on its installed Howell-Bunger delivery valves, which allow discharge rates of 6 to 35 m^3/s. The analysis determined that the reservoir could regulate a volume of 9 hm^3 over three days, representing 8% of its total capacity and corresponding to a 5-meter reduction in reservoir level.

Furthermore, the instrumentation system across the basin is being upgraded to enable real-time measurement of snowline position and precipitation. This will support the projection of flood events in the study area, allowing decisions to be made well in advance and reducing uncertainties associated with these projections.

4. RECOMMENDATIONS FOR CLIMATE RESILIENCE

4.1. Methogology for Adapting Existing Reservoir structures

For the development of adaptation works on existing reservoirs under these new climatic conditions, it is necessary to review and modify the structures to meet these challenges. The following work methodology is suggested for this purpose.

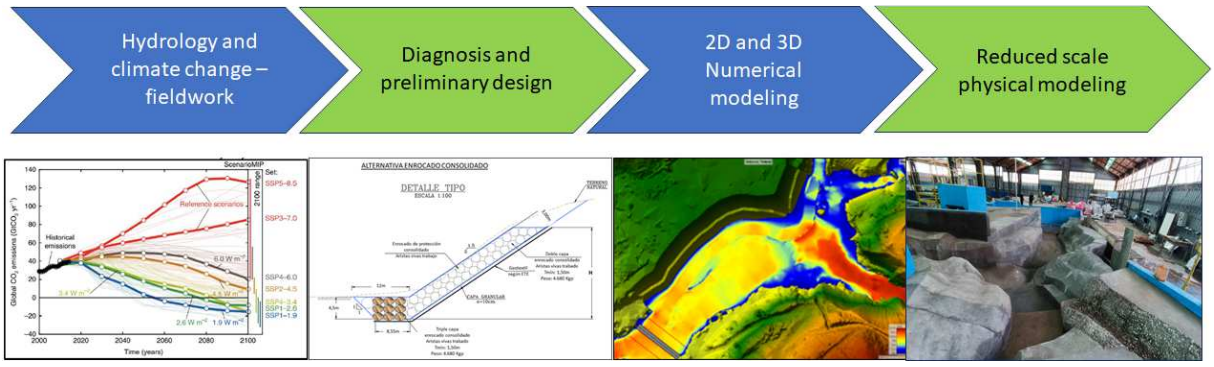


Fig. 8 – Work Methodology for the Adaptation of Existing Reservoir Structures

The figure illustrates a comprehensive methodology for designing reservoirs adapted to climate change, dividing the process into four key stages. First, a hydrological and climate change analysis is conducted, including fieldwork and projections of future scenarios (RCP/SSP) to assess risks such as prolonged droughts, extreme floods, and accelerated sedimentation. Next, a preliminary design is developed, proposing structural alternatives to address these threats, such as sediment control systems and high-capacity spillways. In the following stage, 2D and 3D numerical modeling is applied to simulate flows, validate reservoir performance, and refine designs to manage extreme events. Finally, reduced-scale physical

models are constructed, allowing realistic evaluations of structural functionality and the optimization of critical features such as energy dissipators and evacuation systems. This methodology ensures more resilient and secure reservoirs, prepared to face the challenges posed by climate change.

4.2. Inclusion of Climate Change Effects in Hydrology

The design of future reservoir structures must incorporate the implementation of climatic change scenarios and models to estimate a range of modifications in inflows at the project site, including extreme event flows and sedimentation effects in the contributing basin. These studies generate a spectrum of possibilities, from which the final design conditions should be defined based on the specific characteristics of each project. Since climate conditions lead to varied responses, this study identified the following challenges for different climates:

- In desert climates, the primary challenges include increased intensity of floods and sedimentation issues.
- In semi-arid basins, the main concern is the reduction of inflows to reservoirs, requiring particular attention to medium-flow scenarios.
- In Mediterranean basins, challenges involve both a decrease in inflows and an increase in flood flows.

These insights highlight the need for tailored hydrological considerations in the design of reservoir structures to address the distinct impacts of climate change in each climatic zone.

4.3. Recommendations for Reservoirs in Arid Climates: Chironta Reservoir

Main Challenges:

High variability in inflows due to the Altiplanic winter, with abrupt increases in seasonal rainfall.

High sediment loads transported by intense rains, reducing the reservoir's usable capacity.

Vulnerability to extreme events.

Recommendations:

Sediment Management: Implement upstream control works, such as sediment retention basins, reforestation, and terrace stabilization, to reduce sediment transport.

Hydrological Monitoring: Develop early warning systems to predict severe storms and adjust reservoir operations in real time.

Robust Design: Adapt infrastructure to handle extreme events with higher return periods by incorporating more flexible flood evacuation systems, including valves or flushing gates, and increasing the delivery capacity to regulate reservoir volumes during floods.

Technological Modernization: Incorporate advanced hydrological models that account for climatic variability and future scenarios. Use real-time hydrometeorological monitoring systems to adjust operations proactively.

4.4. Recommendations for Reservoirs in Semi-Arid Climates: Valle Hermoso and Cogotí Reservoirs

Main Challenges:

Sustained decrease in inflows due to reduced precipitation and prolonged droughts.

Inability to reach maximum storage levels over the past five years.

High dependency on recharge periods during winter and low contributions during dry months.

Recommendations:

Technological Modernization: Incorporate advanced hydrological models that account for climatic variability and future scenarios. Use real-time hydrometeorological monitoring systems to proactively adjust operations.

Integrated Basin Management: Strengthen coordination within the basin, promoting efficient water use across sectors.

Adaptive Management: Introduce dynamic operation models that prioritize storage during periods of high availability and regulate supplies for critical demands.

4.5. Recommendations for Reservoirs in Mediterranean Climates: Ancoa Reservoir

Main Challenges:

Increased frequency of extreme events, such as torrential rains and abrupt snowmelt, causing significant damage to downstream infrastructure.

Reduction in base flows due to the rise of the 0°C isotherm and increased evaporation.

Flood flows exceeding the original design capacity due to climate change.

Recommendations:

Infrastructure Reinforcement: Increase the capacity of flood evacuation and energy dissipation systems and strengthen riverbank protections downstream to mitigate future damages.

Predictive Models: Incorporate advanced tools to project the impacts of climate change, such as snowline variations and extreme flow scenarios.

Seasonal Management: Optimize reservoir operations to store water during wet months and prioritize its use during dry months.

Channel Stabilization: Implement fluvial control structures in the most vulnerable downstream areas to minimize erosion and sediment transport.

Dynamic Operations: Develop adaptive operational manuals, integrating flood regulation, irrigation, water supply, and hydropower generation.

5. CONCLUSIONS

5.1. Regional Challenges

Arid Climate (Chironta Reservoir): Affected by the Altiplanic winter, with intense short-duration rainfall that causes sedimentation and sudden floods.

Semi-Arid Climate (Valle Hermoso and Cogotí Reservoirs): Prolonged droughts that hinder filling and efficient operation, with significant reductions in storage levels.

Mediterranean Climate (Ancoa Reservoir): Increased temperatures, evaporation, and extreme rainfall events that overload flood control systems, along with a reduction in average inflows.

5.2. Integrating Climate Change into Reservoir Design studies

Reservoir designs must integrate climate change modeling to evaluate the potential impacts of future scenarios on hydrology and infrastructure performance. High-resolution climate models, such as RCP (Representative Concentration Pathways) and SSP (Shared Socioeconomic Pathways), should be utilized to simulate diverse conditions including changes in precipitation, temperature, snowline, and extreme hydrological events.

The focus should be on generating projections for multiple scenarios:

- RCP4.5 and RCP8.5 Scenarios: Assess moderate and high-emission pathways to identify long-term risks such as prolonged droughts and intense floods.
- Extreme Event Simulations: Model rare but high-impact occurrences, like 100- or 1,000-year floods, and their effects on sediment transport and storage capacity.
- Seasonal and Annual Variability: Explore changes in hydrological cycles, focusing on seasonal recharge during wet months and prolonged dry periods.

Design solutions must incorporate this data to create robust and adaptive infrastructure. Examples include flexible spillway configurations to manage increased flood volumes and sediment flushing systems to mitigate storage losses. Ensemble modeling techniques and sensitivity analyses should be applied to refine designs and optimize decision-making. This ensures that reservoirs are resilient to both predicted and unexpected future climatic shifts.

5.3. Proposed Adaptation Mesures

Thus, the recommendations for adapting reservoir structures to climate change across the various climatic conditions analyzed are summarized as follows.

Table 2 – Climatic Subdivision of Chile and Climate Change Impacts on Water Resources and Extreme Events

Climate	Chilean Zone	Representative Reservoir	Challenges	Adaptation Measures
Arid climate, low rainfall, predominance of deserts	Greather North	Chironta	- High variability in inflow. - Significant sedimentation due to intense rainfall. - Damage from floods.	Physical Works: More robust evacuation systems, bottom gates for sediment flushing. Baseline Studies: Advanced hydrology, specific climate projections. Design: Increase dead storage for sedimentation. Operation: Real-time monitoring.
Semi-arid climate, transition to higher rainfall	Small North	Valle Hermoso and Cogotí	- Sustained decrease in inflows. - Inability to fill to full capacity. - Prolonged droughts.	Physical Works: Anti-evaporation works. Floating pumping systems. Multi-Level Water Delivery Works Baseline Studies: Integrated watershed management. Design: Dynamic storage systems. Operation: Intersectoral coordination of water use.
Mediterranean climate in Ñuble, temperate rainy in Maule and Biobío, abundant watersheds	South-Central Zone	Ancoa	- Extreme floods exceeding design capacity. - Reduction in base flows due to evaporation. - Reduction of runoff due to reduction of precipitation and temperature increase	Physical Works: Reinforcement of riverbank protections and energy dissipation systems. Baseline Studies: Predictive models for floods and snow line. Design: Expand flood evacuation capacity. Operation: Adaptive multi-purpose operation manuals.

6. REFERENCES

[1] AEMET. (2008). Escenarios climáticos regionalizados para España. Agencia Estatal de Meteorología.

[2] Bautista Yáñez, Á. (2020). ¿Están nuestras presas adaptadas al cambio climático? Ministerio para la Transición Ecológica y el Reto Demográfico.

[3] CEDEX. (2017). Evaluación del impacto del cambio climático en los recursos hídricos y sequías en España. Centro de Estudios y Experimentación de Obras Públicas.

[4] Gobierno de Chile. (2016). Guía de presentación y aprobación de proyectos de modificación de cauces. Dirección General de Aguas.

[5] Aceituno, P., Garreaud, R., & López, L. (2021). Impact of atmospheric rivers on extreme precipitation events in southern Chile. Journal of Hydrometeorology, 22(5), 1231-1247. https://doi.org/10.1175/JHM-D-21-0001.1

[6] Aldunce, P., & Vicuña, S. (2019). Cambio climático y recursos hídricos en Chile: Escenarios y estrategias de adaptación. Santiago: Universidad de Chile.

[7] Comité Científico COP25. (2019). Informe sobre impactos del cambio climático en Chile. Santiago, Chile: Ministerio de Ciencia, Tecnología, Conocimiento e Innovación.

[8] Lagos-Zúñiga, A., Vargas, X., & Olivares, G. (2024). Extreme precipitation trends across Chile in a changing climate. Climate Dynamics, 32(1), 45-62. https://doi.org/10.1175/CD-2024-0034.1

[9] Mardones, M., & Garreaud, R. (2020). Projected changes in 0 °C isotherm altitude and their implications for hydrology in central Andes basins. Hydrology and Earth System Sciences, 24(3), 2045-2060. https://doi.org/10.5194/hess-24-2045-2020

[10] Ministerio del Medio Ambiente de Chile. (2023). Plan Nacional de Adaptación al Cambio Climático. Santiago, Chile: Gobierno de Chile.

[11] Ministerio de Medio Ambiente, Gobierno de Chile. Guía metodológica para la consideración del cambio climático en el SEIA.

[12] Juan P. Boisier. (2023). CR2MET: A high-resolution precipitation and temperature dataset for the period 1960-2021 in continental Chile. (v2.5) [Data set]. Zenodo. DOI: 10.5281/zenodo.7529682

[13] Sharifi, E., Saghafian, B., & Steinacker, R. J. J. O. G. R. A. (2019). Downscaling satellite precipitation estimates with multiple linear regression, artificial neural networks, and spline interpolation techniques. Journal of Geophysical Research: Atmospheres, 124(2), 789-805.

[14] Aboitiz C., Chadwick C., & Barochivich J. (2023) Métodos de escalamiento espacial de los modelos de cambio climático: Estudio comparativo sobre diferentes metodologías aplicadas en la zona centro y sur de Chile. XXVI Congreso de Ingeniería Hidráulica.

[15] Cannon, A. J., S. R. Sobie, and T. Q. Murdock. (2015). Bias correction of GCM precipitation by quantile mapping: How well do methods preserve changes in quantiles and extremes? J. Climate, 28(17), 6938-6959, https://doi.org/ 10.1175/JCLI-D-14-00754.1.

[16] SMI, I. (2004). Diseño construcción, embalse Ancoa, región del maule.

[17] SMI, I. (2024). Diseño sistema de disipación de energía, embalse Ancoa, región del maule.

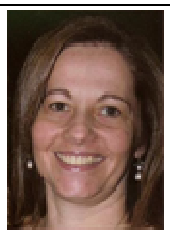
[18] SMI, I. (2010). Diseño embalse Chironta, región de Arica y Parinacota.

[19] MN, I. (2011). Diseño embalse Valle Hermoso, región de Coquimbo.

Fifth International Dam World Conference

dam world 2025

URBAN WATER DAMS DECOMMISSIONING USING NATURE-BASED SOLUTIONS: THE CASE OF MONJOLINHO DAM, IN BRAZIL



Povinelli S C S[1]
UFSCar

Vasconcelos A F[2]
UFSCar

Vannucchi S H L M[3]
UFSCar

Rother D C[4]
UFSCar

Gonçalves L M[5]
UFSCar

ABSTRACT

Decommissioning of urban water dams represents an important challenge that can be characterized as a complex environmental problem. The case addressed in this paper appears as an opportunity for the development of public policies for dam safety, since there is no legislation in Brazil for the dismantling and restoration of these dams. Many structures, which in the recent past were built to store water and control floods, have become obsolete or present significant risks to the environment and nearby communities. The Monjolinho Dam is in the Tietê Hydrographic Basin in the city of São Carlos, state of São Paulo, Brazil, in its section that passes through the Federal University of São Carlos (UFSCar). The University has responsibilities provided for in the Dam Safety Legislation, by which the dam is classified as having high risk and high associated potential damage, which is the main problem related to the dam. Negative impacts may also be associated with the Monjolinho damming, such as habitat fragmentation, alteration of aquatic ecosystems, loss of species and sedimentation. The need for constant maintenance and the costs associated with operating the structures to comply with legislation become high. Since the University is unable to bear the maintenance costs of dams of this type within the Campuses, decommissioning emerges as a viable alternative to mitigate risks. In this scenario, the use of Nature-based Solutions (NbS) emerges as an innovative and sustainable approach. Transforming the dam area into wetlands or rehabilitating riverbeds can improve water quality, but also allows aquatic ecosystems and their surroundings to recover and maintain themselves over time. Revegetation of areas adjacent to the old dam could stabilize the soil, prevent erosion and improve the filtration of pollutants. The implementation of rain gardens, retention basins and permeable pavements helps to manage runoff and infiltration, reducing pressure on drainage systems and contributing to flood mitigation. Finally, it is important that decommissioning combined with the use of Nature-based Solutions is accompanied by continuous monitoring to maximize the benefit of communities and the effectiveness of the process.

Keywords: dam decommissioning, nature-based solutions, waterbody renaturalization, watershed management

[1] silviapovinelli@ufscar.br, Brasil
[2] anai.vasconcelos@ufscar.br, Brasil
[3] sergiomattos@ufscar.br, Brasil
[4] deborarother@ufscar.br, Brasil
[5] lucianamg@ufscar.br, Brasil

1. INTRODUCTION

The dilemma of decommissioning dams arises with the advent of ageing dams and environmental problems [1] [2] [3]. Over time, other factors have been incorporated into the initiative to decommission a dam. In the USA, removal arises as a response to threatened and endangered fish species due to dams, the fragmentation of river systems and the intended return to a more natural regime to restore local ecological conditions [1]. In a survey of 417 dams removed in the USA, 153 of which had justified reasons, Pohl [4] shows that the main reasons were due to safety, interest in environmental issues and restoration. Lejon et al. [3] point out four main reasons for considering dam removal in Sweden: safety, law and politics, economics and ecology. According to Silva [5], in Brazil the main reasons for considering decommissioning dams have been safety problems, siltation due to age and failure to fulfill the purposes of the project. Currently, there are more than 90,000 dams in the USA, some of which have exceeded their average life expectancy, have become structurally unsafe and require expensive repairs to bring them into compliance with state or federal safety regulations. All repairs are the responsibility of the dam owner. The cost to repair and rehabilitate the structure may exceed the financial capabilities of the owner [6], causing owners to consider removal. In Australia, many dams are intended for supply. In Victoria they are generally considered for decommissioning when alternative supply arrangements are identified, when there are ongoing water quality problems or dam safety problems, when the costs of upgrading are disproportionate to the benefits of maintaining a dam or a combination of these reasons [7].

The decision to decommission a dam depends on many factors, such as geographic location, local politics, social aspects, etc. According to Habel [8], dam removal can be different for each country or region, since each one differs greatly in terms of politics, economics, social and cultural awareness, and human relationships with the environment. While one state or country may have removals for environmental reasons, others may do so for economic or safety reasons [4]. It is a fact that in several countries that already have guidelines for decommissioning dams, there is a clear indication that the area covered by the decommissioning must be restored. In Canada, in places such as British Columbia, it is expected that the channel will be restored to its original alignment and slope, and that fish issues will be discussed according to the needs of each area [9]. In countries like Australia, particularly in states like Victoria, the recovery of the area would involve rehabilitating the land to reproduce the natural environment, for example, through the planting of native vegetation [7]. In Brazil, there are still no guidelines for decommissioning and the aim is to support the implementation of the best guide for the country. In the case in question, the suggestion is to use Nature-based Solutions (NbS) to recover the areas previously occupied by the dam surrounding complex.

NbS are innovative actions inspired, sustained or copied from nature, used to address contemporary societal challenges, such as climate change, urbanization and biodiversity loss [10] [11] [12]. Their objectives include promoting positive outcomes in multiple dimensions [13]. NbS are closely related to green infrastructure and ecosystem services, but they focus on sustainability and resilience, making them an even broader tool for achieving sustainable development goals [14]. The impact of NbS is evident in several areas. In urban environments, they contribute to improving public health by mitigating heat island effects, providing recreational spaces and promoting mental well-being [15]. In addition, they boost economic activities such as ecosystem restoration, ecotourism and the development of green technologies, offering alternatives for sustainable growth [16] [17]. These solutions also play a critical role in the resilience of landscapes and cities, helping them to face environmental challenges without compromising economic and social goals [18] [19].

2. STUDY AREA LOCATION

The case presented in this article is in the city of São Carlos, central region of the state of São Paulo, southeastern Brazil, geographic position UTM: 23K: 201514E / 7564351S (Figure 1). The municipality has a population of 254,822 inhabitants, with a population density of 224.14 inhab./km². It has a GDP per capita of R$ 55,044.88 and the Municipal HDI is 0.805. The urbanized area is 79.87%, with afforestation of public roads of 89.2% and urbanization of public roads of 34.2%. The main biomes are Cerrado and Atlantic Forest, its climate is subtropical with a dry season: rainy summer, dry winter and average precipitation of 1,558.3 mm/year. All these data were taken from [20]. São Carlos stands out for being considered the capital of technology and for having three university centers (University of São Paulo - USP, Federal University of São Carlos - UFSCar and Central Paulista University Center - Unicep), two of which are public, which places it in an important role in the generation and dissemination of knowledge, in addition to the training of professionals who work throughout the country.

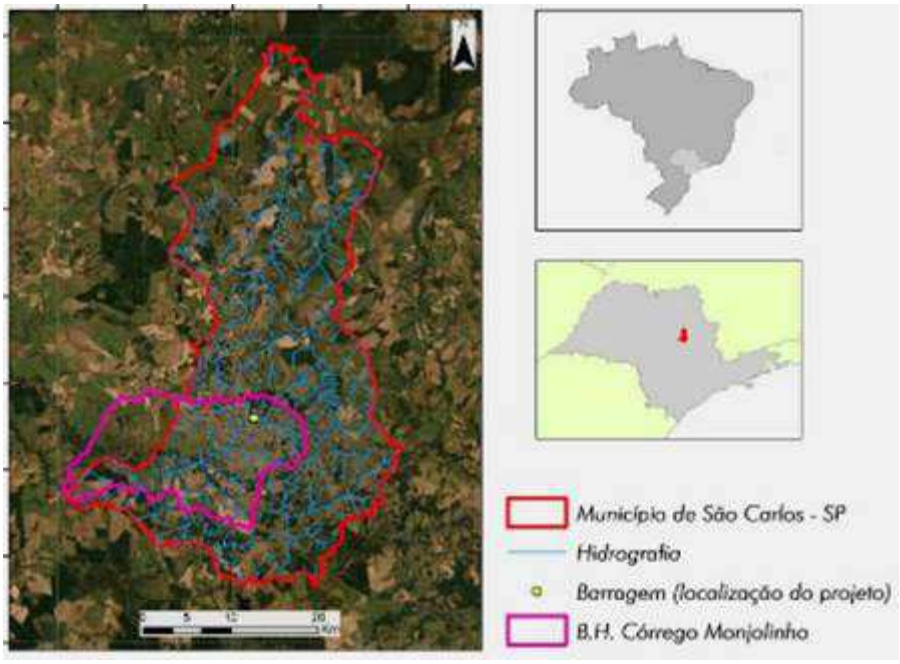


Fig. 1 – Location of the city of São Carlos

The Monjolinho Dam is located in the Monjolinho Stream, in the Tietê-Jacaré Hydrographic Basin, in the city of São Carlos, in its section that passes through the Federal University of São Carlos (UFSCar), Latitude 21º 59´09”S and Longitude 47º 52´55”W, with the contribution area shown in Figure 2.

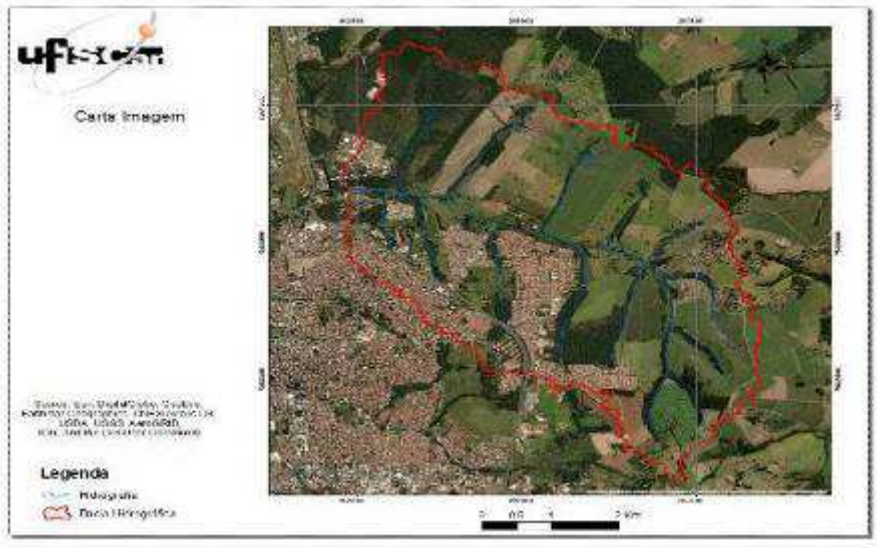


Fig. 2 – Monjolinho contribution basin to the UFSCar dam

Monjolinho dam dates to the late 1960s/early 1970s. Its construction was probably between 1970 and 1974. Its original function was as a water source for public supply, for which it is no longer used. So, the UFSCar lake formed by the dam is not a natural component of the landscape. The type of authorization currently in force is the granting of the right to use water resources for recreational/landscaping purposes for 30 years, granted by Ordinance 2717 of

08/28/2015 by the Department of Water and Electric Energy of the Secretariat of Sanitation and Water Resources of the State of São Paulo. In 2017, an inspection by the Civil Defense and the Department of Water and Electric Energy of the State of São Paulo (DAEE) was carried out, which found damage to the dam structure and recommended declaring a state of emergency, implementing mitigation measures (consisting of banning vehicle traffic and maintaining the spillways open), determining a safety plan, costs and a study to classify potential damage. UFSCar is legally responsible for the dam, but it does not have governance over the territorial occupation policy of the basin, which directly interferes with the dynamics of land use upstream of the project area, affecting hydrological characteristics of the basin, such as concentration time, volume and flow of the upstream. The lack of governance of the upstream areas increases the fragility of structural interventions that retain large quantities of water, such as dams, and, therefore, makes the idea of preserving this dam even more problematic. The project area is located upstream of major highways, industries and the urban area of the municipality of São Carlos, which increases the damage associated with the flood wave caused by an eventual rupture. What to do with the Monjolinho dam and the lake it forms can be considered a typical complex environmental problem.

3. DECISION MAKING PROCESS ON THE FUTURE OF THE DAM

As previously stated, the end of a dam's useful life can occur due to several factors, such as silting of the reservoir, dam safety, deterioration of water quality, environmental, ecological, social and economic problems (profits are lower than operating costs), aging of the structure, lack of authorization, etc. An American study [4] regarding the reasons for removing dams in the country shows that 39% of dams were removed for environmental reasons, 34% for dam safety reasons, 18% due to economic reasons, 5% due to failures and 4% due to other causes. The dam presented in this work has some of these problems, as shown below.

3.1. Lake silting

Due to the urban upstream occupation, the lake has been suffering from silting over the years, having lost much of its water storage capacity and increasing the occurrence of overtopping. The evolution of silting in the lake of the UFSCar dam can be seen in Figure 3. There are regions that already have consolidated vegetation and, hence, can no longer be desilted, due to current environmental legislation. It is important to mention that UFSCar has no control over this silting, nor over the imperviousness increase of the soil that increases surface runoff, since in the upstream area the municipality is responsible for ordering the occupation.

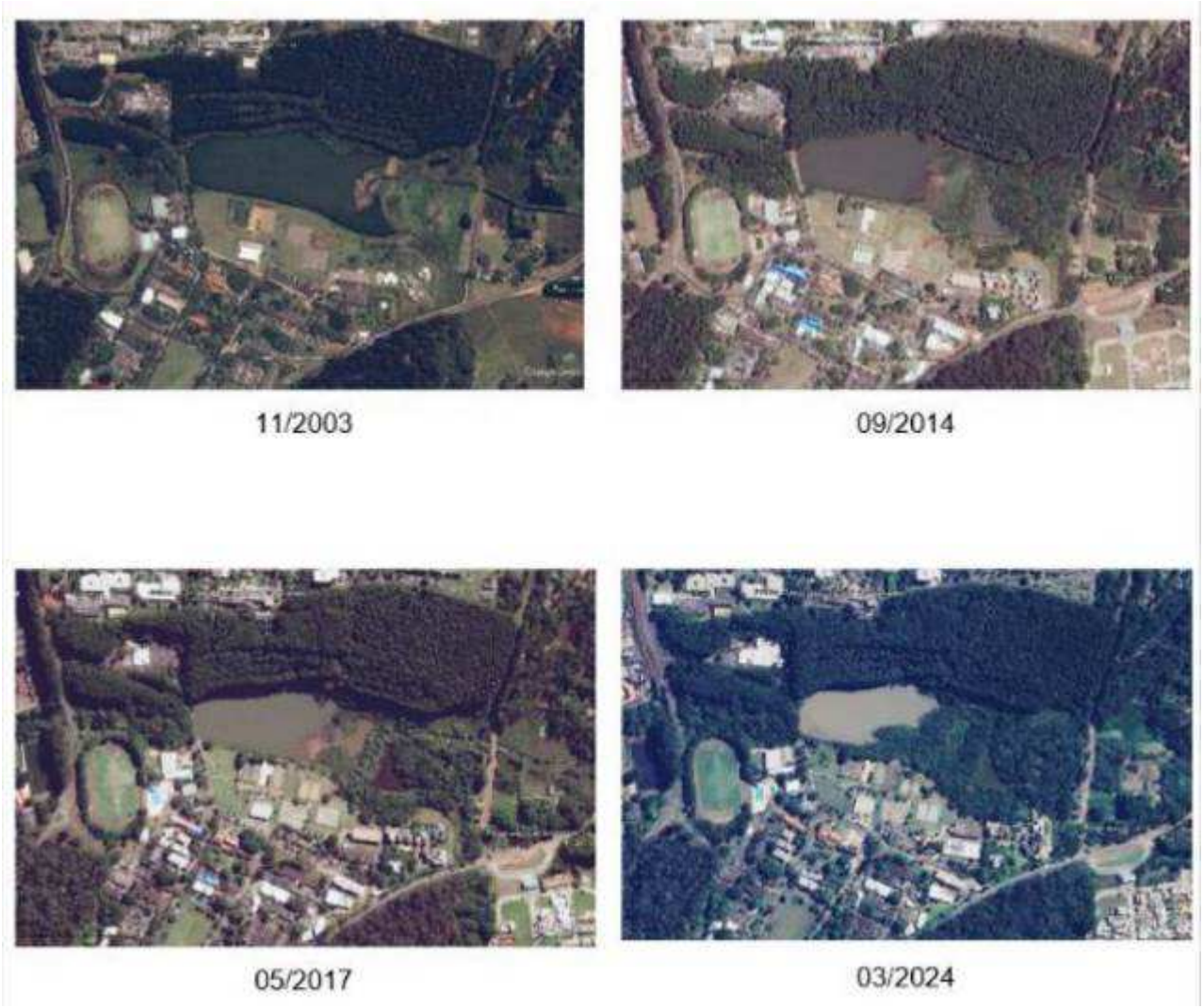


Fig. 3 - Silting of the UFSCar lake. Images from 2003, 2014, 2017 and 2024

3.2. Climate change and floods

In recent years, there have been some extreme events that have damaged the lake region. On January 13, 2013, there was 116 mm of rainfall in 24 hours and the water rose above the crest of the Monjolinho dam. On December 28, 2022, the dam was overtopped, creating a backlog downstream by the Av. do Bosque massif, the main internal road on campus, which was not designed for this purpose, due to an extreme rainfall event. The occurrence of this event caused such a large flood that the water passed over the dam, spreading throughout the University and flooding the Physical Education Department, which had its facilities and equipment rendered useless. The dam is not sufficient to contain extreme rainfall. The UFSCar Dam has a concession for Leisure/Landscaping purposes and, therefore, does not have devices to control floods or regulate free-flowing flows. As previously stated, there is no dam project. Therefore, UFSCar cannot carry out flood control in this reservoir.

3.3. Deterioration of water quality

The water in the UFSCar lake constantly presents quality problems. This is due to a sewage interceptor that runs along the stretch of the Monjolinho River that is within UFSCar and is in very poor condition, which causes leaks that reach the reservoir and promotes the deterioration of water quality and the death of aquatic fauna. One of these episodes can be seen in Figure 4.

Fig. 4 – Deteriorated quality of the UFSCar lake and presence of dead fish - Photo of the lake taken on August 28, 2021, by Paola Camargo Sartori

3.4. Security and economic problems

The current conditions of the dam are precarious, as can be seen in Figures 5 and 6. The costs of building and maintaining a new dam are prohibitive for the university, especially due to the obligations imposed by Federal Law No. 14,066/2020 [22].

View of the dissipation basin

Spillway view

Right slope

Left slope

Fig. 5 – Deterioration of the dam condition

Extravasive structures

Deformities in the asphalt

Fig. 6 – Deterioration of the dam condition

3.4.1. Dam safety legislation

On September 20, 2010, Law No. 12,334 [21] came into force, known as the Dam Safety Law, which establishes the National Dam Safety Policy for dams intended for the accumulation of water for any use, for the final or temporary disposal of waste and for the accumulation of industrial waste, and creates the National Dam Safety Information System. On October 1, 2020, Law No. 14,066 [22] amends Law No. 12,334 of September 20, 2010, which establishes the National Dam Safety Policy.

The general criteria for classifying dams by risk category, associated potential damage and volume are established by Resolution No. 143 [23], of July 10, 2012, of the National Water Resources Council. This Resolution presents important definitions such as: a) potential damage associated with the dam: damage that may occur due to rupture, leakage, infiltration into the soil or malfunction of a dam, regardless of its probability of occurrence, to be graded according to the loss of human lives and social, economic and environmental impacts; b) risk category: classification of the dam according to the aspects that may influence the possibility of an accident or disaster occurring. The supervisory body may adopt technically justified complementary criteria. The application of the Law to a dam depends on the height of its massif, which must be greater than 15 m (the Monjolinho dam is 4.28 m), the total capacity of the reservoir, which must be greater than 3,000,000 m² (Monjolinho is 58,914 m²), the presence of hazardous waste (Monjolinho is a water accumulation reservoir), or the presence of a medium or high risk or potential associated damage category. Since the first three categories are not met, the dam will only be subject to the law if it has a medium or high risk or potential associated damage. The calculations for the risk category and potential associated damage for the UFSCar dam are presented below.

A study was conducted on November 24, 2017, at the Monjolinho dam, which found a score of 67 for the risk category. The limit for the high-risk category is 60, above which the dam can be classified as “high risk category”. The calculation for potential associated damage was 20 points. Above 16 points, the dam can be classified as having “high potential associated damage”.

National Water Resources Council of Brazil (CNRH) Resolution No. 241, of September 10, 2024 [24] was created to establish new general criteria for classifying dams by potential associated damage, by volume and by risk category. Once again, high potential associated damage was found for the UFSCar dam.

It should be noted that the potential associated damage is given by the conditions downstream of the dam and, therefore, does not change if a new dam is built in the same location.

Therefore, the potential associated damage in this location will remain high for any dam, and the Developer must respond to the Dam Safety Law.

Therefore, it is concluded that the University has responsibilities towards Dam Safety Legislation, which is the main problem related to the dam: safety. On the other hand, the University is not able to bear the costs of maintaining a dam within the Campus, since in this location, any dam, new or old, will be subject to the Dam Safety Law.

Due to all the problems reported, the UFSCar University Council opted to decommission the dam, as it was not feasible to maintain it in adequate conditions. The decommissioning of the dam was approved on March 25, 2022, at a meeting of the University Council - Normative Act CONSUNI No. 187.

However, the UFSCar lake is an emblematic and emotional setting for the entire community, so this process requires special care to create an environment that meets the needs of the community that will no longer be met by the lake.

3.5. Governance

Federal Law 14,066/2020 [22] defines that the safety of the dam is a legal responsibility of the developer, including damages resulting from its rupture, leakage or malfunction, and regardless of the existence of fault for the repair of such damages. In the case of the present study, the developer is UFSCar. This law also defines that the supervisory body is the public authority, responsible for the actions of inspection of the safety of the dam under its jurisdiction. In this case, the supervisory body is the Department of Water and Electric Energy of the State of São Paulo (DAEE). In countries such as Canada, in British Columbia, the Dam Safety Regulation (DSR) identifies any of the following as a "restricted activity" in the area: (a) removing all or a significant portion of a dam; (b) decommissioning a dam; (c) deactivating a dam, or interrupting the normal operation of a dam, for a period greater than one year. Before undertaking a restricted activity, a dam owner must provide written notice to a Dam Safety Officer (DSO) of their proposed work. This is the first step in a process that may include several phases of planning, consultation, and reporting before receiving approval to begin work [9].

As already mentioned, the dam in question is located within the campus of UFSCar, in São Carlos - SP. As it is a federal university, the Union is responsible for the area. However, as it encompasses a state-owned water body that has its sources upstream and flows downstream of the campus boundaries, it has direct interaction with the governance of public and private spaces located in the surrounding areas, within the boundaries of the municipality of São Carlos. In this context, the project addresses a complex problem, which involves several actors

with different interests and potential interferences, as well as different levels of government. To accommodate this diversity, the Monjolinho Dam Management Group was created, composed of internal members (professors, students and technicians) from UFSCar, which coordinates the development of the project and mediates communication with external members. A cooperation between UFSCar and the City of São Carlos was also established, initially through the Municipal Secretariat for the Environment, Science, Technology and Innovation, which was split into two secretariats in early 2023: the Municipal Secretariat for Science, Technology and Innovation and the Municipal Secretariat for the Environment and Sustainable Development.

4. SCOPE AND DEVELOPMENT OF THE PRELIMINARY PROJECT

4.1. Solution

To meet the demands presented above, the project aims to decommission and replace the dam currently located on the UFSCar campus in São Carlos with a multifunctional river park, which will include the renaturalization of the waterbody and its surroundings using Nature-based Solutions (NbS). The park will integrate the floodplain areas, restore the natural morphology of the Monjolinho River, restore the riparian forest, and recover and expand the flooded area. It will also be part of a System of Open Spaces planned for the city of São Carlos and serve as a free, healthy leisure option surrounded by nature for the local population. There are some areas consolidated as urban parks in the city that will possibly be annexed to the central region of the project, thus enabling the creation of biodiversity corridors. It is worth noting that the decommissioning process of the UFSCar dam could serve as a model to be followed in Brazil in similar cases, given that there are thousands of inactive and at-risk water dams in the country. Furthermore, the renaturalization of the Monjolinho River could provide the opportunity for the development of teaching, research and extension activities related to the implementation and monitoring of this project. In this way, the proposed project contributes to 11 Sustainable Development Goals (SDGs) (Figure 7).

Fig. 7 - SDGs related to project implementation

For the central core of the park, the proposal is to implement a project that includes several treatment basins along the watercourse (such as for solids retention, quality improvement, etc.), using NbS to reduce the speed of the water and retain it. It is also intended to install NbS around the area currently occupied by the lake, to compose the multifunctional landscape of the park. Examples of the structures initially proposed are the bioswales along the main road that runs parallel to the river, and rain gardens in roundabout areas. In the area bordering the watercourse, a multifunctional park will be implemented, with places for activities for the community integrated into the region and focused on maintaining local biodiversity. The intention is to connect this area with existing municipal parks and green areas of the university itself. The best afforestation techniques will be studied, considering the biodiversity and geographic location of the project. Regarding the problem with the sewage interceptor that runs alongside the river and has frequent leaks, it will be rebuilt by the responsible concessionaire. To enable any maintenance that may be necessary in the interceptor and, at the same time, integrate it into the park, the intention is to create a permeable pavement over it to be used as a cycle path, pedestrian crossing and as access for people with reduced mobility.

The multifunctional park project has already been selected among 10 of almost 100 projects by WRI to carry out training in NbS. In addition to training, the University has carried out several studies necessary for the decommissioning of the dam and implementation of a multifunctional park using NbS. According to [3], to facilitate dam removal, the reasons and effects of dam removal must be carefully explained, and the public and stakeholders must be kept informed.

4.2. Project stages

To properly carry out the project, several specific studies are required. Some of the main studies to be carried out for the implementation of the project are described below: Topographic survey of the intervention region in the basin and bathymetry of the lake; Geological survey of the site; Survey of all existing documentation on the project region; Hydraulic-hydrological calculations and calculations of hydraulic structures in the Monjolinho River, including flood levels within the UFSCar campus; Study of the integration of the renaturalization project with those of urban parks of the City Hall of São Carlos; Design of the crossing of the Monjolinho River that will replace the dam to be decommissioned; Dimensioning of the passage section under 'Avenida do Bosque' (the connecting avenue between the north and south areas of the campus); Topographic survey, design and execution of reconstruction of the interceptor that runs along the Monjolinho River in the area of the UFSCar campus and which is currently damaged, compromising the quality of the river's water; Studies of the fauna and flora existing in the project area and surroundings; Definition of the flooded area, as well as the technique that will be applied for its recovery; Survey of other areas within the campus that have already proven viable for connection to the multifunctional park; Study of areas within UFSCar that can be used for the implementation of nature-based solutions for rainwater retention such as micro drainage; Selection of nature-based solutions to be implemented throughout the project region; Architectural and engineering design of the multifunctional linear park. Subsequently, with these studies and the basic and executive projects in hand, the Monjolinho multifunctional park itself will be implemented. Its works are divided into a few main stages, namely: Decommissioning of the dam; Adaptation of the crossings downstream of the project area; Implementation of the NbS; Implementation of the multifunctional park.

Finally, it is important to consider the monitoring and maintenance of the implemented structures, which, in addition to ensuring their proper functioning, enables learning from this project, enabling teaching and research at the university and training professionals for similar projects, already with the proposal of improvements based on this experience.

4.3. Project benefits and impacts

The assessment of ecosystem services and the benefits and impacts that will be generated in the environmental, social, and economic spheres has been the focus of academic work, such as [25], in addition to the results of work developed in the modules of the WRI Brasil NbS Accelerator. In general, significant positive impacts have been observed in the environmental, social, and economic spheres. Two important aspects that should be highlighted are linked to the increased resilience of the landscape in the face of climate change and the increase in

biodiversity that should result from the renaturalization process. A clear example of the impacts of the current system that will be mitigated with the implementation of the project is presented in the benefits and impacts table (Table 1). The project has the potential to generate several positive impacts that mitigate negative impacts associated with human actions. Examples of these positive impacts include reducing the overflow of water from a watercourse, minimizing erosion and sediment transport, and restoring the ecosystem. The project could also remove up to 8,932.26 tons of CO_2 equivalent from the atmosphere, improve the quality of life of the surrounding population and reduce inequality in access to leisure areas by creating a free multifunctional park open to the public. In addition to these, the new cultural, scientific and educational role of the space is noteworthy.

Table 1 - Benefits and impacts expected from project implementation

Benefits		Impacts
Environmental	Carbon Reduction/ Sequestration	Greater opportunities for CO_2 offsetting, with the increase in green areas and implementation of NbS
		Removal of up to 8,932.26 tons of CO_2 equivalent
	Change in Water Flow and Quality	Greater resilience to flooding with areas that can become flooded during the rainy season and dry during the dry season
		Improvement in water quality providing improvements in aquatic life and surroundings
		Reduction in sewage overflows, impacting, for example, the reduction in fish mortality
	Biodiversity	Improved ecosystem health with restoration of natural conditions
		Increased habitat provision, including increasing habitat diversity such as lotic and lentic aquatic environments
		Habitat recovery for native species, since the lake was an anthropogenic altered environment and altered the entire balance of the local ecosystem
		Connection of ecosystems with the integration of the park into the municipality's green areas system
		Seed dispersal, with improved habitats and connection of green areas
		Alignment with national and international targets, as this is the UN Decade of Ecosystem Restoration and the recently concluded Decade of Biodiversity
	Siltation and soil degradation	Erosion control, due to the increase in the riparian zone and APPs, which help to preserve the soil, thus reducing erosion and the consequent silting of the region
	Climate	Greater resilience to climate change, which has caused a gradual increase in precipitation and temperatures
Social	Air Quality	Improvement in quality of life, with better regulation of air quality and humidity, due to the implementation of the park
		Improved health, with improved air quality and reduced associated respiratory diseases
	Change in Water Flow and Quality	Lower risk of disruption to civic life, given that 16,000 people pass through UFSCar daily and the area is located upstream from the urban area of the municipality
	Temperature	Greater comfort/reduced stress due to better temperature regulation by the park area (green areas and watercourses)
	Health and Wellbeing	Improving physical and mental health, with the provision of a space for sports and leisure activities in contact with nature
		Greater attractiveness of the location, due to the creation of a more pleasant environment to transit and stay in, with attractions related to local nature and NbS
		Improved social cohesion, with the creation of a new living space in the city and on the UFSCar campus
		Improvement of local cultural and traditional aspects, with the promotion of leisure in a natural environment and access to systemic eco-diversity and biodiversity, in addition to contact with NbS, which would allow society to become more involved in this global trend. Arenas will also be created in the park area to hold cultural events.
		Implementation of a community garden, which will bring engagement and well-being to the university community, in addition to food security for the vulnerable part of this community
	Biodiversity	Greater attractiveness of the location with relaxation and leisure areas and opportunities for activities related to biodiversity, such as bird watching
		Strengthening educational networks, training, research and community outreach, related to the use of UFSCar areas, fostering science, innovation and professional training for similar projects
Economics	Air Quality	Reduction in hospitalizations/illnesses related to poor air quality
	Temperature	Reduction in hospitalizations/illnesses related to temperature extremes, especially high temperatures
	Change in Water Flow and Quality	Lower risk of damage to asset(s). Due to flooding in the area, the Physical Education Department was lost
		Lower risk of interruption to local services and businesses, which are affected by flood waters
	Local Economy	Greater attractiveness of the location, promoting tourism and associated economic benefits
		Increased income for local businesses, with the potential to implement new businesses related to the park's operations, such as restaurants, snack bars and leisure equipment rentals
		Financial costs avoided due to the reduction of legal obligations related to the reconstruction and maintenance of the dam

4.4. Replicability

The creation of a fluvial park on the Monjolinho River could serve as a reference for the decommissioning of dams in Brazil using NbS and for integration with Open Space Systems. Furthermore, as this is a pioneering case of decommissioning of water accumulation dams using NbS in the country, it will serve as an example for similar projects, with great potential for replicability, since there are several dams in these conditions currently.

5. CONCLUSIONS

Due to the risk conditions and high maintenance costs of the dam, UFSCar decided to decommission it. Since then, the university's working group responsible for solving this problem has been developing studies and projects for the renaturalization of the lake area and the construction of a crossing at the site of the dam. To this end, planialtimetric surveys, hydrological impact studies, a conceptual urban project for the multifunctional river park, among other technical and administrative documentation necessary for the development of the proposal have already been carried out. Since this is a very large and complex project, community involvement at all stages is essential to maximize its acceptance and benefits. Since this is an innovative project for decommissioning the dam and renaturalizing the area using NbS, especially because it occurs within a university campus, its capacity for generating knowledge and replicability is enhanced. The publication of this article makes it possible to disseminate at an international level the most innovative measures that have been taken in Brazil to decommission water dams in line with the most modern and sustainable solutions for renaturalizing the area, with the use of NbS.

6. REFERENCES

[1] Grant, G. (2001). Dam Removal: Panacea or Pandora for Rivers. Hydrological Processes 15, 1531 – 1532.

[2] Doyle, M. W.; Harbour, J. M.; Stanley, E. H. (2003). Toward Policies and Decision-Making for Dam Removal. Environmental Management, v. 31, n. 4, 453 – 461.

[3] Lejon, A. G. C.; Renöfält, B. M.; Nilsson, C. (2009). Conflicts Associated with Dam Removal in Sweden. Ecology and Society **14**(2): 4.

[4] Pohl, M.M. (2002). Bringing down our dams: trends in American dam removal rationales. Journal Of the American Water Resources Association, v. 38, n. 6.

[5] Silva, M. B. F. A. (2012). Índice de remoção de barragens cearenses (IREB) sob um enfoque multicritério. Tese de doutorado em Engenharia Civil, pela Universidade Federal do Ceará.

[6] ASDSO Resources (2022). Dam Removal. Dam Ownership - Fact Sheet, American Rivers.

[7] Australia, Victoria (2022). Decommissioning dams. A guide for dam owners. Department of Environment, Land, Water & Planning.

[8] Habel, M.; Mechkin, K.; Podgorska, K.; Saunes, M.; Babiński, Z.; Chalov, S.; Absalon, D.; Podgórski, Z. & Krystian Obolewski, K. (2020). Dam and reservoir removal projects: a mix

of social-ecological trends and cost-cutting attitudes. Scientific Reports | (2020) 10:19210.

[9] Canadá, British Columbia (2019). Dam Decommissioning Guidelines. Dam Safety Program. Ministry of Forests, Lands, Natural Resource Operations and Rural Development.

[10] Castelo, S., Amado, M., & Ferreira, F. (2023). Challenges and Opportunities in the Use of Nature-Based Solutions for Urban Adaptation. Sustainability. https://doi.org/10.3390/su15097243.

[11] Seddon, N., Daniels, E., Davis, R., Chausson, A., Harris, R., Hou-Jones, X., Huq, S., Kapos, V., Mace, G., Rizvi, A., Reid, H., Roe, D., Turner, B., & Wicander, S. (2020). Global recognition of the importance of nature-based solutions to the impacts of climate change. Global Sustainability, 3. https://doi.org/10.1017/sus.2020.8.

[12] Seddon, N., Smith, A., Smith, P., Key, I., Chausson, A., Girardin, C., House, J., Srivastava, S., & Turner, B. (2021). Getting the message right on nature-based solutions to climate change. Global Change Biology, 27. https://doi.org/10.1111/gcb.15513.

[13] Cohen-Shacham, E., Andrade, A., Dalton, J., Dudley, N., Jones, M., Kumar, C., Maginnis, S., Maynard, S., Nelson, C., Renaud, F., Welling, R., & Walters, G. (2019). Core principles for successfully implementing and upscaling Nature-based Solutions. Environmental Science & Policy. https://doi.org/10.1016/J.ENVSCI.2019.04.014.

[14] Martín, E., Giordano, R., Pagano, A., Van Der Keur, P., & Costa, M. (2020). Using a system thinking approach to assess the contribution of nature-based solutions to sustainable development goals. The Science of the total environment, 738, 139693. https://doi.org/10.1016/j.scitotenv.2020.139693.

[15] Kolokotsa, D., Lilli, A., Lilli, M., & Nikolaidis, N. (2020). On the impact of nature-based solutions on citizens' health & well-being. Energy and Buildings, 229, 110527. https://doi.org/10.1016/J.ENBUILD.2020.110527.

[16] Bush, J., & Doyon, A. (2019). Building urban resilience with nature-based solutions: How can urban planning contribute? Cities. https://doi.org/10.1016/j.cities.2019.102483.

[17] Kooijman, E., Mcquaid, S., Rhodes, M., Collier, M., & Pilla, F. (2021). Innovating with Nature: From Nature-Based Solutions to Nature-Based Enterprises. Sustainability. https://doi.org/10.3390/SU13031263.

[18] Dumitru, A., Frantzeskaki, N., & Collier, M. (2020). Identifying principles for the design of robust impact evaluation frameworks for nature-based solutions in cities. Environmental Science & Policy. https://doi.org/10.1016/j.envsci.2020.05.024.

[19] Lafortezza, R., Chen, J., Van Den Bosch, C., & Randrup, T. (2018). Nature-based solutions for resilient landscapes and cities. Environmental research, 165, 431-441. https://doi.org/10.1016/j.envres.2017.11.038.

[20] IBGE, (2024) https://cidades.ibge.gov.br/brasil/sp/sao-carlos/historico.

[21] Brasil. (2010). *Lei Federal n° 12.334, de 20 de setembro de 2010*. Estabelece a Política Nacional de Segurança de Barragens destinadas à acumulação de água para quaisquer usos, à disposição final ou temporária de rejeitos e à acumulação de resíduos industriais, cria o Sistema Nacional de Informações sobre Segurança de Barragens e altera a redação do art. 35 da Lei no 9.433, de 8 de janeiro de 1997, e do art. 4o da Lei no 9.984, de 17 de julho de 2000.

[22] Brasil. (2020). *Lei Federal nº 14.066, de 30 de setembro de 2020*. Altera a Lei nº 12.334, de 20 de setembro de 2010, que estabelece a Política Nacional de Segurança de Barragens (PNSB), a Lei nº 7.797, de 10 de julho de 1989, que cria o Fundo Nacional do Meio Ambiente (FNMA), a Lei nº 9.433, de 8 de janeiro de 1997, que institui a Política Nacional de Recursos Hídricos, e o Decreto-Lei nº 227, de 28 de fevereiro de 1967 (Código de Mineração).

[23] Brasil. (2012) - Conselho Nacional de Recursos Hídricos (CNRH). (2012). *Resolução 143, de 10 de julho de 2012*. Estabelece critérios gerais de classificação de barragens por categoria de risco, dano potencial associado e pelo seu volume, em atendimento ao art. 7° da Lei n° 12.334, de 20 de setembro de 2010.

[24] Brasil. (2024) - Conselho Nacional de Recursos Hídricos (CNRH). (2024). *Resolução 241, de 10 de setembro de 2024.* Estabelece critérios gerais de classificação de barragens por dano potencial associado, por volume e por categoria de risco, em atendimento ao art. 7° da Lei n. 12.334, de 20 de setembro de 2010.

[25] Lustosa, G. E. D. P. (2021). Avaliação de vantagens, desvantagens e desafios da renaturalização do lago da Universidade Federal de São Carlos (São Carlos - SP). Trabalho de conclusão de curso (bacharelado em Ciências Biológicas), Universidade Federal de São Carlos (UFSCar).

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

STATE-OF-THE-ART REVIEW ON ADVANCEMENTS FOR DAM MONITORING TECHNOLOGIES

Silva, Andre[1]
Measurand

Levesque, Christiane
Measurand

ABSTRACT

Advancements in earth dam monitoring technologies have significantly enhanced the ability to maintain safety, reliability, and operational efficiency in these critical infrastructures. This review explores these innovations, focusing on their advantages, limitations, and the challenges they present, while also highlighting their future potential for improving dam management.

Remote sensing technologies, such as Interferometric Synthetic Aperture Radar (InSAR) and Light Detection and Ranging (LiDAR), have revolutionized the way surface deformation is monitored. These technologies provide large-scale, high-resolution tracking of subsidence and deformation in dam structures and surrounding areas. Fiber optic sensing (FOS), including Distributed Temperature Sensing (DTS) and Distributed Strain Sensing (DSS), offers highly precise, real-time internal structural condition monitoring through embedded fiber optics.

The Internet of Things (IoT) has further improved real-time data acquisition with wireless sensor networks (WSN), piezometers, inclinometers, strain gauges, extensometers, among others. These systems continuously collect data on key variables such as water pressure, tilt and strain. ShapeArray Sensors (SAA) deliver detailed measurements of dam displacements, while Global Navigation Satellite System (GNSS) technologies provide highly accurate, real-time geodetic measurements to track surface movement and displacement (ATS).

Artificial intelligence (AI) and machine learning play an increasingly critical role in processing large datasets generated by these sensors. These tools enable the prediction of potential failures, early detection of anomalies, and optimization of maintenance strategies. Complementing these efforts, vibration and seismic monitoring systems, including accelerometers and geophones, aid in detecting seismic events and internal instability within dam structures.

Innovative inspection tools such as drones and unmanned aerial vehicles (UAVs) offer efficient visual assessments of hard-to-reach areas, while Automated Total Stations (ATS) provide precise, automated surface deformation measurements. Ground Penetrating Radar (GPR) detects internal structural defects, and hydrological monitoring systems—including automated weather stations and reservoir level sensors—deliver vital environmental data.

Despite these technological advancements, human expertise remains essential for interpreting data, ensuring system functionality, and making critical operational decisions. The integration of advanced technologies with human oversight creates a comprehensive monitoring system, offering new potential for improving dam safety and risk mitigation.

This paper addresses the challenges in implementing these technologies and highlights their potential to further revolutionize dam monitoring in the future.

Keywords: geotechnical monitoring, IOT, AI, human role, remote sensing

[1]andre.silva@measurand.com, Brazil

1. INTRODUCTION

Dams are an important component in infrastructure (e.g. hydroelectric dams and water reservoirs) as well as in mining (e.g. tailings storage). As past failures of these structures have shown, dam monitoring is a crucial practice to ensure that risks are properly mitigated, keeping people and the environment safe [63].

This paper focuses on monitoring of earth dams. It will cover the history of monitoring as well as current technologies used in monitoring. Finally, it contains a discussion of the future of dam monitoring including the use of artificial intelligence tools and challenges which require further research [2].

2. HISTORY OF EARTH DAM MONITORING: A COMPREHENSIVE OVERVIEW

The history of earth dam monitoring has evolved significantly over the past century, shaped by advancements in technology, a growing understanding of geotechnical processes, and an increasing focus on safety [3]. From rudimentary visual inspections to the integration of artificial intelligence and remote sensing technologies, the field has transformed into a sophisticated domain that underpins the safety and reliability of critical infrastructure. This comprehensive overview traces the evolution of earth dam monitoring, highlighting key milestones and advancements [4].

The history of earth dam monitoring reflects a trajectory of increasing sophistication and technological integration. From traditional empirical methods to cutting-edge artificial intelligence and remote sensing, the field has evolved to meet the demands of modern infrastructure management [5]. These advancements not only enhance safety but also provide valuable insights into long-term dam behavior, ensuring the integrity and sustainability of these critical structures. As technology continues to advance, the future of earth dam monitoring holds even greater promise for innovation and resilience [6].

2.1. Early Practices: Empirical and Rudimentary Monitoring

In the 19th century and earlier, earth dam monitoring relied heavily on visual inspections and empirical observations [7]. Engineers searched for external signs of distress, such as seepage, deformation, or cracking, without the tools to understand subsurface processes. The limited instrumentation of the time, such as simple piezometers, provided only basic insights [8]. Monitoring was largely reactive, and dam failures often occurred without warning, underscoring the need for better predictive capabilities [9].

2.2. Early 20th Century: Foundations of Systematic Monitoring

As dams grew larger and more complex, systematic monitoring practices began to emerge [10]. Simple mechanical devices like settlement gauges and inclinometers were introduced to measure surface deformations. The catastrophic failure of the St. Francis Dam in 1928 highlighted the critical need for comprehensive monitoring systems, leading to increased research into geotechnical behavior [11]. By the mid-20th century, engineers started integrating instruments during dam construction, focusing on parameters such as pore water pressure and stress distribution to better understand dam stability [12].

2.3. Post-War Era: Advancements in Instrumentation

The period between the 1940s and 1970s marked a turning point with the advent of advanced instruments like vibrating wire piezometers, extensometers, and electrical strain gauges [13]. These tools allowed for more accurate and quantitative data collection, supplementing traditional methods. Organizations like the International Commission on Large Dams (ICOLD) promoted standardized practices and facilitated global collaboration [14].

2.4. Modern Instrumentation Era: Digital Transformation

From the 1970s to the early 2000s, digital technology revolutionized earth dam monitoring [15]. Automated data loggers, fiber optic sensors, and geophysical techniques like seismic and resistivity measurements became widely adopted. These innovations enhanced the resolution and scope of data collection, enabling continuous monitoring and early detection of potential issues [16]. This era also marked the beginning of integrating digital analysis tools, which paved the way for intelligent monitoring systems. This era also saw the establishment of early warning systems, driven by lessons learned from failures such as the Banqiao Dam disaster in 1975 [17].

2.5. Intelligent Monitoring and Remote Sensing

The 1990s marked the introduction of artificial intelligence and machine learning into monitoring practices [18]. Researchers explored neural networks and genetic algorithms to predict seepage behavior, significantly improving the analysis of monitoring data. Bayesian networks further enhanced intelligent decision-making, allowing for more robust interpretations of seepage and deformation trends [19].

Simultaneously, remote sensing technologies such as ground-based radar interferometry and three-dimensional laser scanning transformed the field [20]. These methods provided precise measurements of displacement and structural integrity, especially for older dams lacking embedded sensors. Ground-based radar, for instance, has proven valuable for assessing seismic risks, while laser scanning has facilitated detailed geometric monitoring [21].

2.6. Contemporary Period: Wireless and Satellite Technologies

The 21st century has seen a surge in wireless sensor networks and satellite-based monitoring systems [22]. Wireless sensors have enabled real-time detection of seepage paths, preventing potential collapses [23]. Advanced algorithms, like the Parallel Small Baseline Subset (P-SBAS), have improved the accuracy of deformation monitoring. Satellite technologies, including GNSS and interferometric synthetic aperture radar (InSAR), have revolutionized large-scale monitoring, providing continuous data over extended periods and in inaccessible regions [24].

3. CURRENT TRENDS AND FUTURE DIRECTIONS

The integration of IoT, AI, and real-time data processing has defined the current era of earth dam monitoring. These technologies provide unprecedented levels of precision and predictive capabilities, allowing for proactive management of dam safety. Moreover, advancements in cloud computing, AI and big data analytics promise to further enhance the efficiency and reliability of monitoring frameworks.

Given the significant number of available technologies, the authors chose to organize the most relevant information into a table. This format highlights essential aspects such as what these technologies are, what they do, where they are applied, and their development expectations for the next 5 to 10 years

Table 1 – Monitoring technologies summary table

Technology	How It Works	Current Applications	Limitations	Future (5–10 Years)	Key References
Remote Sensing (InSAR, LiDAR)	InSAR (Interferometric Synthetic Aperture Radar): Uses radar signals (often from satellites) to measure ground displacement. Phase differences between radar images acquired at different times enable detection of sub-centimeter changes in the Earth's surface. LiDAR (Light Detection and Ranging): Emits laser pulses and measures return time to create high-resolution 3D models.	- Large-scale deformation monitoring (dam surfaces, slopes, urban subsidence, infrastructure monitoring).- High-resolution terrain mapping.- Rapid detection of topographic changes in remote/inaccessible areas.- Infrastructure health assessment (subsidence or uplift).	- InSAR: Atmospheric distortions, signal decorrelation in vegetated/water areas, requires stable reflectors.- Resolution: Spatial resolution may be insufficient for small-scale features.-Cost and Accessibility: High-resolution data from commercial satellites can be expensive. 3D Deformation Challenges: Difficulty in accurately measuring north-south displacements due to satellite orbit geometries.- LiDAR: Potentially high cost for repeated surveys, dependent on weather/atmospheric conditions, requires line-of-sight.	- InSAR: Near-real-time monitoring from constellations of small satellites; improved coherence for vegetated areas; larger historical and spatial coverage.- Multi-Sensor Integration: Combining InSAR with LiDAR, GNSS, and ground-based sensors for comprehensive monitoring. -Broader Accessibility: Open-data initiatives (e.g., Copernicus program) will make high-quality InSAR data more accessible to researchers and industries. Big Data Processing: Cloud infrastructures and high-performance computing enable large-scale Persistent Scaterere Interferometry (PSI) processing, even at national scales.- LiDAR: Cheaper, higher-frequency systems (e.g., drone-mounted); near-real-time 3D mapping with machine learning for quick hazard detection.- Integrated monitoring with GNSS and ground sensors for multiparametric insights.	25,26,27
Fiber Optics	Detects strain, temperature, or acoustic signals in fiber optic cables via changes in the light's phase, polarization, or intensity. Common methods include Distributed fiber-optic sensors (e.g., Brillouin or Rayleigh-based)	- Real-time monitoring of cracks and strain in critical structures (dams, tunnels, pipelines).- Early detection of leaks or geotechnical instabilities.- High spatial resolution over long distances with a single cable. Integration with existing FOS network. Resistant to lightning strikes and eletromagnetic interferences. -High frequency data collection	- High cost and complexity of installation (especially for retrofits).- Fibers can be fragile during construction.-Installation expertise required- Readouts still expensive compared with convencional systems. Data interpretation expertise and large data storage capacity	- Multi-parameter sensing (strain, temperature, vibration) in a single cable generally applied.- Enhanced ruggedness of fibers for harsh conditions.- Integration with AI for automated anomaly detection across critical infrastructure networks. Synergy with other Sensing Technologies. Retrofit and Smart Grid Integration.-Wider Adoption of Standards and Regulations	28,29,30,31
IoT Devices (Dataloggers & Gateways)	Low-power sensors (strain gauges, temp sensors, accelerometers) connect to dataloggers; gateways transmit data (Wi-Fi, Lo(ng) Ra(nge), cellular) to a central or cloud-based platform.	- Continuous, remote data collection from distributed networks (geotechnical, structural).- Near-real-time alerts and dashboards for decision-making.- Reduced need for manual readings and site visits.	- Connectivity issues in remote areas (lack of stable power/cell coverage).- Battery life constraints in large-scale deployments.- Data security and standardization challenges (cyberattacks).-Handling and processing large volumes of data collected at high frequencies can be complex and resource-intensive.	- Increased edge computing for on-site data processing and anomaly detection; Ultra-low-power communication protocols ((Narrow Band) NB-IoT, LoRaWAN) and satellital; Energy-efficient designs; enhanced onboard storage capacities; Self-calibrating networks and standardized IoT frameworks for large-scale structural health monitoring. AI powered loggers for Early warning System ((EWS) System improvements	32,33,34
Inclinometer (Manual, IPI, ShapeArrays)	- Manual: A casing is installed in a borehole; a probe measures tilt at depth intervals.- In-Place Inclinometers (IPI)/ ShapeArrays: Permanently installed sensor arrays (different spacing resolution options) that continuously measure tilt or shape changes. The installation could be vertical or horizontal	- Monitors lateral displacements in soil/rock for slope stability or retaining walls or vertical displacements of landfills, embankments, earth dams.- Early warning of dangerous movements in embankments, foundations, dams.- Captures incremental movement over time. -Reusable in different projects	- Requires borehole installation (costly, intrusive) or open trenchs for horizontal installations- Measurements can be affected by casing deformation/installation and human errors.- Limited coverage (one sensor string per location).- Large installations could be challenging and resource-intensive. Need a local reference	- Higher-resolution arrays with wireless data transmission to reduce manual readouts.- Integration with real-time alert systems and advanced analytics.- Lighter and longer installations. Robotized data colletion with manual probes. FOS alternatives for vertical and horizontal applications. -User-Friendly Interfaces and Enhanced Visualization. - Hybrid/Multi-Parameter Sensing (Strain, Temperature, Pressure, 3D displacement)	35,36,37,38
Piezometers, Water Level Indicators, Standpipe	- Piezometers (vibrating wire, pneumatic, standpipe) measure pore water pressure.- Water Level Indicators/ Standpipes measure water levels in wells/boreholes (manually or automatically).	- Tracks groundwater or reservoir level fluctuations (seepage, hydrostatic pressure).- Inputs for slope stability and seepage models.- Critical for dam safety monitoring and dewatering programs.	- Point-based measurement (one sensor per location).- Sensor clogging or damage in harsh environments.- Manual instruments require frequent site visits for data collection. Measurements can be affected by installation and human errors.	- Wireless IoT piezometers for real-time water pressure data.- Integrated multi-parameter probes (e.g., temperature, conductivity).- Self-diagnosing sensors that detect anomalies (drift, clogging). - Harmonized Guidelines for Data Quality. - Plug-and-Play Monitoring Packages.-Robust and Miniaturized Designs	39,40,41,42, 43

Technology	How It Works	Current Applications	Limitations	Future (5–10 Years)	
Automatic Total Stations	Automated robotic theodolites measure angles/distances to reflectors on structures/slopes. Provide millimeter-level accuracy in 3D displacement measurement.	- Continuous/periodic measurement of 3D displacements in real time (e.g., dam deformation).- Quick detection of movement trends or anomalies.- Widely used in construction, tunneling, and geotechnical monitoring.	- Requires uninterrupted line-of-sight to reflectors.- Sensitive to atmospheric conditions (temp, humidity).- Prone to obstruction or reflector displacement (vandalism, vegetation).	- Enhanced automation/tracking of multiple reflectors.- Integration with GNSS for robust positioning solutions.- Advanced onboard data processing to reduce measurement noise/drift.	44,45,46
GNSS	Global Navigation Satellite Systems (GPS, GLONASS, Galileo) triangulate satellite signals for positioning. High-precision modes (RTK, PPP) can achieve cm- to mm-level accuracy.	- Monitoring structural/ground movements (subsidence) at discrete points in real time.- Commonly used for dam and bridge displacement, landslide tracking.- Geodetic surveys and baseline references for other instruments.	- Requires open sky visibility (signal loss under obstructions).- Multipath errors near large structures.- Precision depends on satellite geometry and correction services. - Power and connectivity requirements for remote places	- More robust constellations giving higher accuracy with fewer satellites.- Advanced multipath mitigation and data fusion (INS, LiDAR).- Miniaturized, low-power GNSS integrated into IoT sensor networks. Decreasing costs of GNSS hardware and software may make the technology more accessible for smaller dam projects.	47,48,49,50, 51
Seismic & Vibration Monitoring	Uses triaxial accelerometers or geophones to measure ground/structural accelerations. Detects seismic waves or vibration signatures from natural/anthropogenic sources. Track peak ground acceleration (PGA) amplitudes according to predefined threshold criteria. In addition, ambient vibration techniques—such as Frequency Domain Decomposition (FDD) and the H/V spectral ratio (Nakamura method)—are commonly employed for spectral evaluations.	- Real-time earthquake and structural response monitoring (e.g., dams, buildings).- Identifies resonance frequencies, potential fatigue in structures.- Event-triggered logging for abrupt incidents (blasts). Integrate with expert geophysical studies	- Highly sensitive to noise (traffic, construction).- Requires calibration for various frequency ranges.- Complex data interpretation needing specialized expertise.- Factors that could influence the interpretation (Background noise, Soil-structure interaction, Canyon effect, Water level, Temperature, Climatic effects, Instrument aging, Sensor installation, Instrument positioning, Earthquake characteristics).- Measurements may be compromised by installation errors, such as improper sensor placement.	- Dense “smart seismic arrays” for detailed structural health mapping.- AI to distinguish harmless vibrations from critical events.- Ultra-low-power wireless sensors for large-scale deployment. - Combining Geodetic and Remote Sensing Methods - Advanced Signal Processing and Machine Learning (Automated Anomaly Detection, Event Classification); General integration with other geophysical methods	52,53,54,55
Inspection Tools (Drones & UAVs)	UAVs with cameras, LiDAR, or thermal sensors collect structural or terrain data. Automated/semiautomated flights access hard-to-reach areas.	- High-resolution imagery/point clouds for visual inspection (cracks, corrosion).- Thermographic surveys to detect moisture or delamination.- Rapid post-disaster assessment.	- Limited flight time/battery capacity.- Regulatory restrictions in certain zones.- Skilled operators needed; data processing can be time-intensive.	- Fully autonomous UAV fleets with real-time collision avoidance and analytics.- Advanced sensors (hyperspectral, GPR).- AI-driven automatic defect recognition and condition scoring. Cost reductions in drone hardware and software, making the technology more widely available.	56,57,58
Geophysical Techniques	Non-invasive methods (Electrical Resistivity Tomography, Ground Penetrating Radar, Seismic Refraction) measure subsurface properties. Variations in conductivity, wave velocities, or radar reflections reveal anomalies.	- Locates seepage paths, voids, saturation zones in embankments.- Evaluates geological conditions for foundations.- Monitors subsurface changes (internal erosion in dams).	- Resolution/depth penetration vary by method.- Interpretation can be ambiguous without ground-truth data.- Field logistics can be time-consuming and require skilled operators.	- Permanent real-time geophysical arrays for continuous subsurface imaging.- Machine learning-based inversion for faster, automated data processing/interpretation.- Integrated with hydro-mechanical models for improved early warning.	59,6
Hydrological Monitoring Systems (Automated Weather Stations, Reservoir Level Sensors)	- Automated Weather Stations: Measure rainfall, temperature, humidity, wind, etc., via connected sensors.- Reservoir Level Sensors (ultrasonic, pressure transducers): Continuously measure water levels in reservoirs, rivers, canals.	- Real-time weather and reservoir level tracking for flood forecasting and water resource management.- Inputs for hydrological modeling and early warning systems.- Trend analysis of climate impact on water resources.	- Sensor calibration/maintenance in harsh conditions (lightning, flood, sediment).- Communication outages in remote sites (satellite/cellular).- Potential data gaps from sensor drift/fouling.	- Enhanced sensor fusion (weather, flow, structural data) for predictive dam safety.- Advanced AI for real-time flood forecasting and dynamic reservoir operations.- Low-cost, solar-powered weather stations with mesh networking in remote areas.	61,62
Data Processing and Analysis	-data processing and analysis is often based on automated visualizations which are evaluated by engineers reviewing actual behaviour and comparing it with expected behaviour as determined using constitutive models and analyses -Artificial intelligence has been used for many years to evaluate material properties, and assess stability, but limited in terms of predicting future stability	Analysis is used for assessing slope stability, seepage impact, mitigation actions, and overall risk management	Large data sets are not easily parsed by people, and if changes are slow, people may not identify pattern changes	- increased use of AI to predict stability through Machine Learning -Development of multi-agent systems to make AI systems more fault tolerant and easier to scale	63,64,65

From Table 1, it is possible to identify points of convergence among various technologies, with the following highlights:

Integration of Multi-Parameter Sensing:

- Combining multiple sensing technologies (e.g., strain, temperature, displacement, and vibration) in a single system for comprehensive monitoring across various applications.

Artificial Intelligence and Machine Learning:

- AI is expected to play a major role in anomaly detection, predictive maintenance, and the fusion of data from diverse sources.
- Automated data processing and analytics will make large datasets more actionable and insights more accessible.

Real-Time Monitoring and Automation:

- Increased adoption of real-time systems using IoT devices, wireless technologies, and automated data collection tools.
- Enhanced edge computing for real-time data processing, reducing reliance on centralized systems.

Cost Reductions and Broader Accessibility:

- Advances in GNSS, InSAR, and LiDAR technologies are driving down costs, making high-resolution data more accessible for smaller-scale projects.

Improved Durability and Adaptability:

- Development of rugged and miniaturized sensor designs for use in harsh environments.
- Enhanced wireless and energy-efficient solutions, such as solar-powered sensors and low-power communication protocols (e.g., LoRaWAN, NB-IoT).

Hybrid Systems and Multi-Sensor Integration:

- Combining ground-based systems with satellite remote sensing (InSAR, GNSS) for comprehensive monitoring.
- Integration of fibre optics, geophysical methods, and UAVs for a more complete understanding of subsurface and surface conditions.

User-Friendly Interfaces and Data Visualization:

- Simplified user interfaces with enhanced visualization capabilities, making advanced analytics more accessible to engineers and decision-makers.

Standardization and Guidelines:

- Development of harmonized standards and guidelines for geotechnical monitoring to ensure data quality and interoperability across systems.

Environmental and Safety Enhancements:

- Adoption of predictive models for disaster prevention (e.g., landslide and flood forecasting).
- Increased focus on sustainability, with low-cost, solar-powered, and autonomous systems for remote and inaccessible areas.

These advancements collectively aim to make geotechnical monitoring more efficient, reliable, and scalable, addressing current challenges such as data management, cost barriers, and real-time decision-making. Given the exponential development and growing impact of artificial intelligence, the authors decided to dedicate a specific chapter to the topic.

4. ARTIFICIAL INTELLIGENCE IN GEOTECHNICAL MONITORING: ENHANCING DATA MANAGEMENT, ANALYTICS, AND MONITORING SYSTEMS

4.1. Data management and analytics

Increases in the automation of geotechnical monitoring is inherently increasing the volume of data which must be managed. The volume of data generated, and the velocity at which this data is produced has created an environment with much more complex data management needs. However, the greater volume of data also allows engineers to use analytics or AI to gain insights which would not have been possible previously. These insights include identification of trends, creation of visualizations to tell a story with the data, and heretofore unknown correlations.

An important issue with large data sets was highlighted in several references [63] [66] [67] [68]. This is the need to carefully inspect the data and ensure it is as free of errors as possible. Ideally, engineers can work with data wranglers or scientists to ensure that the data is as accurate as possible. Data management in this case grows beyond the traditional “Collect, Store, Use, Archive, and Destroy” used in business. It must now instead include “Generation, Collection, Processing, Storage, Management, Analysis, Visualization, and Interpretation” [69].

Engineering judgement is required for creating instructions for the rates at which the data is generated (e.g. number of readings per day), the collection rate (e.g. once hourly), and processing (e.g. what data should be removed, for what reasons, and how can this be determined?). In many cases, these tasks are then undertaken by others such as technicians or data managers. The storage and management of data also requires engineers to think about the use of the data. Any data must be collected for a reason, since collecting data with no known goal will typically result in high data maintenance cost, as well as potentially lead to confusion. Engineers must also carefully plan and execute analyses, visualization and interpretation to ensure the results are reasonable.

Four main types of data analytics have been used in business. These are descriptive analytics (looks to answer what happened), diagnostics analytics (looks to find root cause and answer why something happened), predictive analytics (try to predict what might happen in the future), and prescriptive analytics (try to determine what should be done next) [70]. Analytics used in geotechnical engineering have been concentrated on determining and updating geotechnical parameters such as soil or rock properties [63], as well as predicting processes such as seepage [66] or slope stability [63] [71]. Although descriptive and predictive analytics techniques have been most studied, the opportunities which diagnostic and prescriptive analytics can provide in dam maintenance and risk management are interesting, and it is likely that these analytical techniques will be explored and used more frequently in dam management within the next decade.

4.2. Artificial Intelligence (AI) tools

Research in the use of AI in geotechnical engineering has been taking place for decades [64], starting with the use of expert systems, followed by fuzzy logic systems, and artificial neural networks (ANN). Since the first papers discussing ANN were published, it has become the most used and researched AI methodology. Ebid's State of the Art review on the use of AI in geotechnical engineering showed 48% of papers published between 1984 and 2019 using this technique [64].

Although ANN and Machine Learning (ML) are often discussed separately, it is important to note that ANN are a subset of algorithms used in ML. Table 2 shows the most used types of ANN in geotechnical engineering as well as a description of how these neural networks can be used [63].

Table 2. Types of ANN used in geotechnical engineering and their proposed uses [63]

Type of ANN	Proposed Uses
Feed Forward Neural Network	Used for complex pattern identification and learning
Back Propagation Neural Network	Used to gain understanding of complex relationships
Bayesian Regression Neural Networks	Uses Bayesian methods to measure uncertainty, helpful in complex models
Differentiated Evolution Neural Network	Used for complex optimization tasks
Generalized Regression Neural Network	Used for regression assignments
Radial Basis Function Neural Network	Used for pattern recognition and regression tasks

Other ML algorithms are also becoming more popular in geotechnical engineering in the last 5 years [63]. There are 5 broad categories of machine learning: supervised learning, unsupervised learning, semi-supervised learning, self-supervised learning, and reinforcement learning [72]. To date, most geotechnical engineering researchers used supervised learning, unsupervised learning, and reinforcement learning techniques [63]. Common algorithm types for these three types of learning are presented in the following discussion.

In supervised learning, training is done using a known or labelled set of data before setting the algorithm onto unlabelled data [63] [72]. This type of machine learning is particularly suited for use in risk assessments, classification and predictive analytics. Various algorithms used in supervised learning are provided in Table 3. In geotechnical engineering, supervised learning has been used in slope stability analyses [71], design of deep and shallow foundations, soil or rock classification, seepage analyses [63] [64] [66].

Table 3. Algorithms used in supervised machine learning [63]

Algorithm type	Use
ANN	Identify complex patterns, understand complex relationships, optimization, and regression tasks
Classification algorithms (e.g. Logistic regression, k-nearest neighbours, support vector machines)	Predict categorical output by labelling pieces of input data
Naïve Bayes classifiers (generative learning algorithm)	Used for classification tasks when large datasets are encountered
Random Forest Algorithms	Predict a value or category using results from multiple decision trees
Regression algorithms (e.g. linear regression)	Predict output values by identifying linear relationships between continuous values (e.g. temperature)

Unsupervised learning techniques are trained on unlabelled data [63] [72]. The goal of training on unlabelled data is to learn the inherent patterns or structures of the data set. In general,

unsupervised machine learning algorithms are most used in cluster analyses, in which the data are categorized according to similar values [72]. However, Yaghoubi et al [63] found that geotechnical applications of unsupervised learning techniques could be broken into 4 categories shown in Table 4. Typically, unsupervised learning methods are used for site characterization [63].

Table 4. Types of algorithms used in unsupervised machine learning [63]

Algorithm type	Use
Anomaly detection (e.g. isolation forests, one-class support vector machine)	Identify data points that deviate from the normal or expected behaviour
Clustering (e.g. K-means clustering, hierarchical clustering, probabilistic clustering)	Group similar data points based on specific features. Grouping similar values together
Density estimation (e.g. Gaussian Mixture Model)	Estimate the probability density function for a dataset
Dimensionality reduction (e.g. Principal Component Analysis (PCA)	Reduce the number of features while maintaining important information

Reinforcement learning techniques involves using human feedback to train the algorithm or agent to learn best strategies [72]. Table 5 contains a list of reinforcement methods [63]. Reinforcement techniques are useful for optimization tasks such as resource management or for training a model in which the agent will need to predict dam stability based on new data [63] [66].

Table 5. Reinforcement machine learning methods and their uses

Training method	Use
Value based method	Focus on value functions to guide action selection
Policy based method	Learns policies to choose actions that lead to expected rewards
Actor-critic method	Combines value based and policy-based optimizations
Model-based methos	Learning a model of the environment to plan and make decisions

4.3. Challenges and the future of AI in dam monitoring

In general, interest in the use of ML has been increasing. This increase is leading to a skills gap in effectively applying and evaluating results from ML leading to a preference of ANN techniques which have been far more used to date [63] [64]. Although the use of ML methods are believed to lead to more robust results and improved reliability [68], the emergence of a new tool also leads to questions about what training is most important for geotechnical engineers [63] [64] [67]. With the current global population trends showing a reduction in young

people on most continents [72], it is important to ensure that new engineers are trained in properly evaluating the results from the ML models. Data driven models must be evaluated to ensure that they coincide with the physical world [63] [66] [67] [68]. The constitutive models used to implement AI methods, and the choice of the parameters to be evaluated are key to the success of the use of AI in engineering. Sound engineering judgement is still very important, to ensure that the AI does not become a black box outputting data of unknown quality. Teaching good critical thinking skills and improving young engineers grasp on theory, or the whys, of geotechnical engineer is extremely important. Since training on bad models or data can lead to large issues, the skills required to critically evaluate the AI Results should be taught as early as reasonably possible at University.

Although AI is becoming more popular, many existing sites have large monitoring processes which do not yet include the use of AI. Integration of AI into existing workflows is complicated and will require careful planning to ensure the best use of data collected from existing monitoring systems, as well as new monitoring systems [63] [66] [68].

Use of multi-agent systems is also taking place in some geotechnical projects. rather than putting everything into a single centralized agent or system [73]. In traditional systems, the system is controlled by a single agent. This agent is built up and trained for specific models, and if new parameters are to be added, the system will require a complete overhaul. In multi-agent systems each agent is specialized, and communicates with others sharing information, which is used by each agent. By decentralizing the system, it allows the data to be analysed in parallel and makes the system more fault tolerant [73]. The use of these systems is relatively new to the field of geotechnical engineering, but it is likely to expand considering the need for system stability when safety is on the line.

5. FINAL REMARKS: THE ROLE OF HUMAN EXPERTISE IN MONITORING

Recent developments in geotechnical monitoring point toward an increasingly integrated, intelligent, and accessible future. Multi-parameter sensing systems are converging, enabling simultaneous measurement of strain, temperature, displacement, and vibration for more holistic data collection. Artificial intelligence and machine learning are emerging as key drivers, supporting automated anomaly detection, predictive maintenance, and efficient data fusion across large datasets. Real-time capabilities, strengthened by IoT devices and edge computing, reduce latency and enhance system autonomy. Meanwhile, cost reductions in GNSS, InSAR, and LiDAR have broadened accessibility, facilitating the use of high-resolution data in smaller-scale projects. Sensor designs are becoming more durable and adaptable, with rugged, miniaturized, and energy-efficient solutions—such as solar-powered devices—expanding the operational range into remote and harsh environments. Increasingly, hybrid

frameworks blend ground-based systems with satellite remote sensing for a more comprehensive view of subsurface and surface conditions. These advances are supported by improvements in user-friendly interfaces and data visualization, coupled with standardization efforts that ensure data quality and interoperability. Collectively, these trends not only enhance the safety and reliability of geotechnical infrastructure but also foster sustainability and resilience in the face of environmental challenges.

In 1972 Ralph Peck opined that “An instrument too often overlooked in our technical world Is the human eye connected to the brain of an intelligent human being.” [74]. This was in response to an increase in the use of instrumentation, often with a lack of clear goals around what should be monitored. The warning that counting on technology too much opens an engineer to significant dangers is still valid. Engineers must know how to ensure the data or insights from monitoring equipment is in line with reality. Human beings with appropriate training and expertise are still needed in this new data driven age, especially when dam safety is considered.

Although moving towards more data and automation will make it easier to automate many aspects for dam monitoring, engineers will still be an integral part of ensuring dam safety. These systems should be considered a set of tools which can make the engineering more reliable. Not a replacement for proper engineering judgement. Engineers will still need to use their skills to deal with:

- Ethical considerations such as safety, transparency, and environmental impacts
- Design and implementation of automated monitoring systems
- Ensure data quality processes are in place
- Choose constitutive models and parameters
- Work with experts to effectively use ML
- Evaluate results and insights generated by AI
- Manage risks using insights gained from monitoring and AI.

6. REFERENCES

1. Dunnicliff, J. (1993). "Geotechnical Instrumentation for Monitoring Field Performance." Wiley.
2. ICOLD (2021). "Dam Safety Guidelines and Best Practices." International Commission on Large Dams.
3. Fell, R., MacGregor, P., & Stapledon, D. (2005). "Geotechnical Engineering of Dams." Taylor & Francis.
4. USACE (2005). "Engineering and Design - Instrumentation of Embankment Dams." U.S. Army Corps of Engineers.
5. Terzaghi, K., Peck, R.B., & Mesri, G. (1996). "Soil Mechanics in Engineering Practice." Wiley.
6. Charles, J.A. (2005). "Lessons from Historical Failures of Dams." Geotechnical Engineering.
7. Casagrande, A. (1965). "Role of the ‘Critical Void Ratio’ in Soil Mechanics." Journal of the Soil Mechanics and Foundations Division.
8. USGS (2015). "Advances in Seepage Monitoring Technologies." U.S. Geological Survey.
9. Peck, R. B. (1969). "Advantages and Limitations of Observational Methods in Applied Soil Mechanics." Geotechnique.
10. ICOLD (2013). "Technical Bulletins on Embankment Dams." International Commission on Large Dams.
11. Seed, H.B., & Idriss, I.M. (1982). "Ground Motions and Soil Liquefaction During Earthquakes." Earthquake Engineering Research Institute.
12. Cording, E.J. (1974). "Instrumentation for Safety Monitoring of Earth Dams." ASCE Journal of Geotechnical Engineering.
13. Wilson, C. (2001). "Advances in Structural Health Monitoring of Dams." Journal of Infrastructure Systems.
14. Fell, R., & Fry, J.J. (2007). "State of Practice in Dam Safety Risk Assessment." Geotechnical Safety Journal.
15. Bazant, Z.P., & Planas, J. (1997). "Fracture and Size Effect in Concrete and Other Quasibrittle Materials." CRC Press.
16. Soga, K., & Mohamad, H. (2014). "Instrumentation and Monitoring for Geotechnical and Structural Performance." Journal of Civil Engineering Research.

17. ICOLD (2017). "Guidelines for Dam Monitoring." International Commission on Large Dams.
18. Johanson, R. (2019). "Applications of AI in Geotechnical Monitoring." Geotechnical Innovations Journal.
19. NASA (2020). "Satellite-Based Monitoring for Infrastructure Stability." NASA Earth Science Division.
20. Zhang, Y., & Chen, L. (2021). "Remote Sensing Techniques in Dam Safety." Remote Sensing Journal.
21. Xu, W., & Li, P. (2018). "Wireless Sensor Networks in Structural Health Monitoring." Structural Engineering Journal.
22. Li, J., & Wang, X. (2019). "Geospatial Techniques for Dam Monitoring." Geospatial Science Journal.
23. Smith, D. (2020). "Machine Learning Applications in Geotechnical Engineering." AI in Civil Engineering.
24. ESA (2021). "Advancements in InSAR for Large-Scale Monitoring." European Space Agency.
25. El Kamali, M., Abuelgasim, A., Papoutsis, I., Loupasakis, C., & Kontoes, C. (2020). A reasoned bibliography on SAR interferometry applications and outlook on big interferometric data processing. Remote Sensing Applications: Society and Environment, 20.
26. Ferretti, A., Prati, C., & Rocca, F. (2001). IEEE Transactions on Geoscience and Remote Sensing, 39(1), 8–20.
27. Shan, J., & Toth, C. (Eds.). (2018). Topographic Laser Ranging and Scanning: Principles and Processing (2nd ed.). CRC Press.
28. Barrias, A., Casas, J. R., & Villalba, S. (2016). Sensors, 16(5), 748.
29. Zou, L., Wu, W., & Chen, L. (2019). Photonic Sensors, 9(3), 193–208.
30. Glisic, B., & Inaudi, D. (2007). Fiber Optic Methods for Structural Health Monitoring. John Wiley & Sons.
31. Inaudi, D., & Blin, R. (2016). Monitoring dams with distributed fiber optic sensing. Hydro 2016, Montreux, Switzerland.
32. Wubbena, G., Bagge, A., & Schmitz, M. (2014). In Proceedings of the ION GNSS+ Conference.
33. Gomes, D. G., Rodrigues, J. J., et al. (2018). Journal of Network and Computer Applications, 143, 198–222.

34. Chandroth, J., Saxena, N., Roy, A., & Rastogi, E. (2021). A new design and analysis of power saving for IoT gateway. IETE Technical Review, 38(1), 1–8. https://doi.org/10.1080/02564602.2021.1880349
35. Mikkelsen, P. E. (2002). Geotechnical News, 20(1), 46–51.
36. Lucking, G., et al. (2021). Journal of Geotechnical and Geoenvironmental Engineering, 147(10), 04021099.
37. Júnior, W., Evangelista da Silva, M., Coutinho, R., & Dantas, B. (2024). Inclinometer: An overview of the evolution from manual to automatic.
38. Allasia, P., Godone, D., Giordan, D., Guenzi, D., & Lollino, G. (2020). Advances on measuring deep-seated ground deformations using robotized inclinometer systems. Sensors, 20(13), 3769. https://doi.org/10.3390/s20133769
39. U.S. Army Corps of Engineers (USACE). (2014). Engineer Manual EM 1110-2-4300: Instrumentation for Fills. Washington, DC.
40. International Commission on Large Dams (ICOLD). (2018). Bulletin 165: Dam Surveillance and Monitoring – Best Practices. Paris.
41. Wayne, J., & Singh, R. (2020). Performance analysis of vibrating wire piezometers in earth dam monitoring. Journal of Dam Safety, 12(2), 114–128.
42. Chen, B., Li, F., & Zhang, W. (2019). Advanced wireless piezometer networks for real-time dam safety monitoring. Sensors, 19(12), 2641.
43. Kaynia, A. M. (2021). Advanced monitoring and condition assessment of geotechnical structures. In G. Lollino et al. (Eds.), Engineering Geology for Society and Territory – Volume 6. Springer.
44. Stempfhuber, W. (2016). Journal of Applied Geodesy, 10(1), 45–62.
45. Fumagalli, A., et al. (2019). ISPRS Annals of Photogrammetry, Remote Sensing, and Spatial Information Sciences, IV-2/W5, 293–299.
46. Lienhart, W. (2019). State-of-the-art of geotechnical monitoring with geodetic techniques. Geotechnical Engineering, 50.
47. Blewitt, G. (2015). In Treatise on Geophysics (2nd ed.). Elsevier, 307–338.
48. Kashani, I., et al. (2015). Canadian Journal of Forest Research, 45(7), 821–828.
49. Trimble. (n.d.). R9s GNSS Receiver Manual.
50. Leica. (n.d.). GS18 User Manual.
51. Topcon. (n.d.). HiPer HR GNSS Receiver Manual.
52. Darbre, G. R. (1995). Strong-motion instrumentation of dams. Earthquake Engineering and Structural Dynamics, 24, 1101–1111.

53. Darbre, G. R., & Proulx, J. (2006). Knowledge gained from the dam strong-motion network in Switzerland. In Proceedings of the First China National Hydraulic Earthquake Engineering Congress, Hehai University, Nanjing, October 28–30, 2006.
54. Mihailov, V., & Dojcinovski, D. (2004). Strong-motion instrumentation of dams in Macedonia: Some experience and results. 13th World Conference on Earthquake Engineering, Vancouver, Canada, August 1–6, 2004.
55. Syscom. (n.d.). Manual Guidelines for Tailings Storage Facilities. https://www.syscom.ch/tailings-storage-facilities-tsf
56. Rakha, T., & Gorodetsky, A. (2018). Automation in Construction, 93, 252–264.
57. Siebert, S., & Teizer, J. (2014). Automation in Construction, 41, 1–14.
58. Leite, D. T., Marcelino, J., & Gonçalves, S. . Visual inspection of embankment dams. 4th DamWorld Conference, Lisbon, Portugal.
59. Panthulu, T. V., Krishna, S., & Shirke, J. M. (2001). Engineering Geology, 59(3–4), 281–295.
60. Loke, M. H., & Barker, R. D. (1996). Geophysical Prospecting, 44(1), 131–152.
61. Buytaert, W., et al. (2012). Climate and Development, 4(1), 66–79.
62. Verma, R., et al. (2021). Water, 13(10), 1400.
63. Yaghoubi, E., Yaghoubi, A., Khamees, A., & Vakili, A. H. (2024). A systematic review and meta-analysis of artificial neural network, machine learning, deep learning, and ensemble learning approaches in geotechnical engineering. Neural Computing and Applications, 36, 12655–12699.
64. Ebid, A. M. (2020). 35 years of (AI) in geotechnical engineering: State-of-the-art review. Geotechnical and Geological Engineering, 39, 637–690.
65. Firoozi, A. A. F., & Firoozi, A. A. (2023). Intelligent multi-agent systems for advanced geotechnical monitoring. IntechOpen.
66. P. A. Garsole, S. Bokil, V. Kumar, A. Pandey e N. S. Topare, “A review of artificial intelligence methods for predicting gravity dam seepage, challenge and way-out,” *AQUA: Water Infrastructure, Ecosystems and Society,* vol. 72, nº 7, pp. 1228-1248, 2023.
67. Yu Wang e Hua-Ming Tian, “Digital geotechnics: from data-driven site characterisation towards digital transformation and intelligence in geotechnical engineering,” *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards,* vol. 18, nº 1, pp. 8-32, 2024.

68. H. Liu, H. Su, L. Sun e D. Dias-da-Costa, “State-of-the-art-review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artificial Intelligence Review,* vol. 57, nº 196, 2024.

69. T. Tobierski, “8 Steps in the Data Life Cycle,” 2 February 2021. [Online]. Available: https://online.hbs.edu/blog/post/data-life-cycle.

70. C. Cote, “4 Types of Data Analytics to Improve Decision Making,” 19 October 2021. [Online]. Available: https://online.hbs.edu/blog/post/types-of-data-analysis.

71. K. Chand, V. Kumar, P. Raj, N. Sharma, A. K. Mankar e R. Koner, “Artificial Intelligence Tool for Prediction of Mine Tailings Dam Slope Stability,” *Journal of Mining and Environment (JME),* vol. 16, nº 1, pp. 127-142, 2025.

72. C. R. China, “Five machine learning types to know,” IBM, 20 December 2023. [Online]. Available: https://www.ibm.com/think/topics/machine-learning-types. [Acesso em 15 December 2024].

73. PopulationPyramid.net, “Population Projections,” 01 12 2024. [Online]. Available: https://www.populationpyramid.net/population-projections/africa+asia+europe+latin-america-and-the-caribbean+northern-america+oceania/. [Acess em 01 12 2024].

74. R. Peck, “Observation and Instrumentation: Some Elementary Considerations,” *Highway Focus,* vol. 4, nº 2, pp. 2-5, 1972.

Fifth International
Dam World Conference
PORTUGAL • LISBON • LNEC • APRIL 13-17, 2025
dam world 2025

DIVERSIFYING ELECTRICITY SOURCES IN WATER-STRESSED REGIONS: A MULTI-CRITERIA APPROACH FOR THE SÃO FRANCISCO RIVER BASIN

Clécio Barbosa Souza Júnior[1]
Eletrobras

ABSTRACT

The article investigates the challenges faced by water-stressed regions, where climate change exacerbates issues such as water scarcity, environmental degradation, and conflicts over water use. This situation highlights the urgent need to transition to more sustainable and resilient electricity sources. The study emphasizes the importance of using Multi-Criteria Decision Analysis (MCDA) to reform the energy matrix in semi-arid regions, addressing economic, technical, environmental, and social dimensions.

The São Francisco River Basin in northeastern Brazil, which relies heavily on hydroelectric power, serves as a case study. The region has faced drought, leading to a reduction in hydropower generation and escalating conflicts between the agricultural and electricity sectors. This reliance on hydropower has exposed vulnerabilities within the regional electricity matrix, which climate change has further intensified, underscoring the need for a more diversified and resilient energy approach.

To address these challenges, the study employed the TOPSIS (Technique for Order of Preference by Similarity to Ideal Solution) method to evaluate four electricity matrix scenarios, including hydropower, wind, solar, and thermoelectric power. The analysis revealed that scenarios involving a reduction in hydropower generation, particularly those emphasizing the expansion of wind energy, were the most viable. In contrast, scenarios that increased the share of gas-fired thermoelectric power were considered less favorable.

The findings underscore the critical need to diversify energy sources to mitigate the impacts of climate change in water-stressed regions. By adopting a multi-faceted approach to energy planning, the study highlights the importance of transitioning towards more sustainable and resilient energy solutions to address both current and future challenges.

Keywords: TOPSIS, MCDA, Climate Change, Electricity Matrix.

[1]clecioj@eletrobras.com, Brazil

1. INTRODUCTION

The energy sector plays a pivotal role in the economic and social development of nations, serving as the backbone for industrial growth, technological advancement, and improved living standards [1]. However, achieving a balance between economic profitability, environmental sustainability, and social equity remains a significant challenge [2]. Traditional approaches to energy planning, often reliant on single-criterion models, have struggled to address the multidimensional complexities of modern energy systems [3].

In response to these challenges, governments worldwide are increasingly prioritizing energy efficiency and adopting integrated tools to address pressing issues such as water governance and the expansion of renewable energy sources [4, 5, 6]. This shift has placed Multi-Criteria Decision Analysis (MCDA) methods at the forefront of energy planning and policy development [7].

MCDA methods offer a structured approach to evaluating competing priorities and scenarios. For example, Amer and Daim [8] demonstrated their utility in Pakistan, identifying wind power and biomass as the most favorable renewable energy technologies. In Saudi Arabia [9] and Turkey [10], MCDA applications have underscored the strategic potential of solar and wind energy. Similarly, in India and Sri Lanka, these models have been used to pinpoint barriers to renewable energy adoption, offering actionable insights for overcoming challenges [11, 12]. Across diverse global contexts—including Italy, Greece, Kazakhstan, and Taiwan—MCDA has proven instrumental in crafting energy policies that reflect local priorities and constraints [13, 14, 15, 16].

In Brazil, the integration of hydropower with renewable energy sources like wind has also been recognized as a promising pathway. Koch et al. [17] highlighted the complementary nature of these systems, which can enhance energy security and resilience.

This is particularly relevant in the Brazilian Northeast (NE), where the São Francisco River Basin plays a central role in the region's energy matrix (Fig. 01). Hydropower accounts for 41% of the NE's installed capacity, primarily relying on large reservoirs such as Sobradinho, Itaparica, Complexo Paulo Afonso, and Xingó [18]. As of 2017, the region's electricity mix included hydropower (41.4%), wind power (27.7%), thermoelectric plants fueled by fossil fuels (24.2%), and smaller contributions from biomass, solar, and small hydropower plants [18].

However, persistent meteorological and hydrological droughts since 2013 have severely impacted water flows into reservoirs, leading to decreased hydropower generation. This has

exacerbated conflicts over water allocation, highlighting the urgent need for integrated management strategies that balance energy production, environmental preservation, and social demands [19, 20].

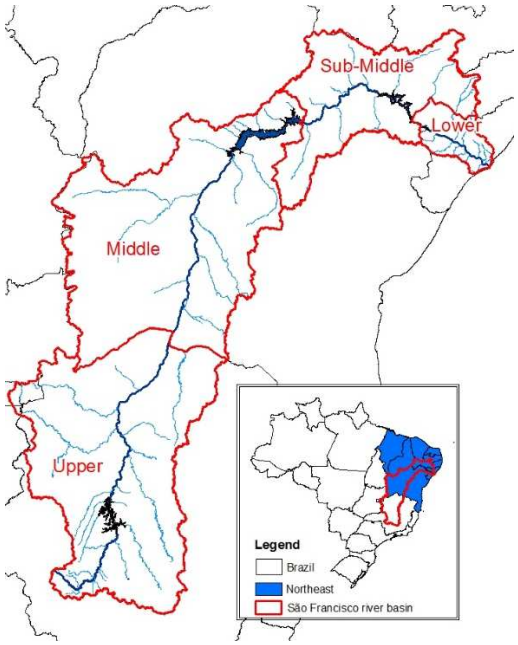


Fig. 1 - Northeast Region and São Franscisco River basin

This prolonged hydrological drought has necessitated compensatory measures, such as increased reliance on thermoelectric generation, while also driving significant growth in wind energy production [18]. According to Brazil's Decennial Energy Expansion Plan 2024, the Northeast region holds the greatest potential for solar and wind energy, with a projected capacity of approximately 114 TWh/year—more than half of Brazil's total potential [21]. Solar power, in particular, shows great promise in the São Francisco River Valley, where solar irradiation levels range between 6 and 6.5 kWh/m²/day, surpassing the national average of 5.4 kWh/m²/day [22].

However, challenges persist, including the high implementation costs of solar technologies, despite a noticeable decline in prices over recent years [23]. Figure 2 illustrates the hydroelectric generation scenario in the Northeast between 2002 and 2022, highlighting a marked reduction in electricity production from 2012 to 2018 (Fig. 2). Notably, the PDE 2026 (Ten-Year Energy Expansion Plan) identifies this period as the most severe water shortage in the 84-year history of recorded measurements.

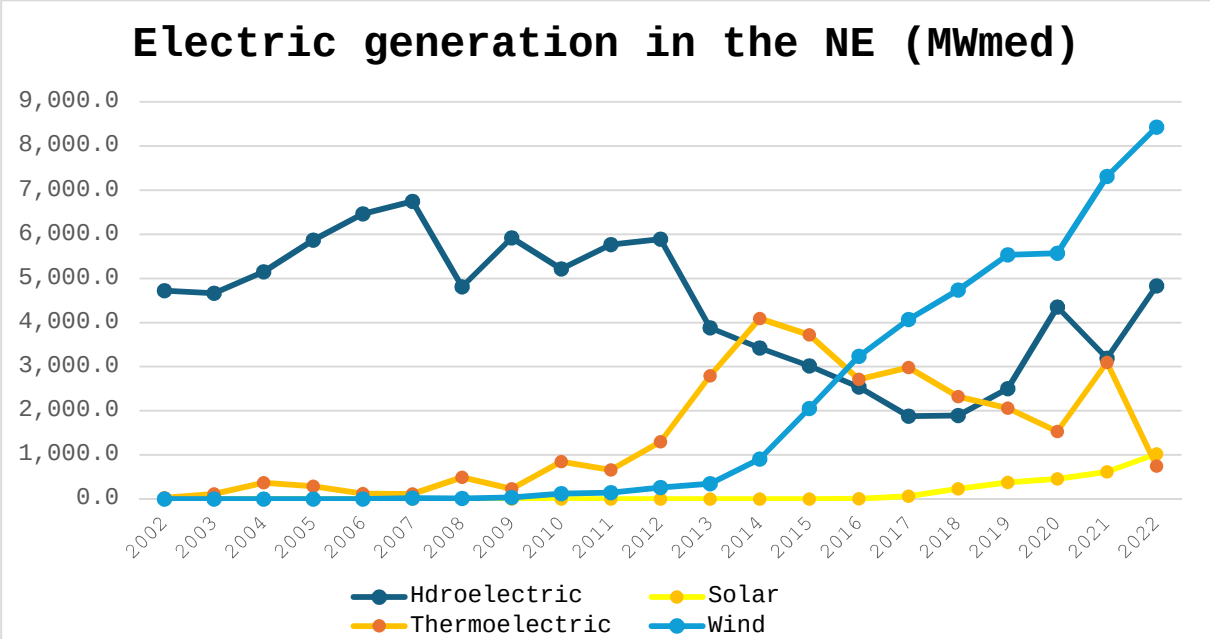


Fig. 2 Average annual electric power generation in the NE between 2002 and 2022 [23]

The unsustainable reliance on hydropower in a region experiencing severe droughts has intensified water use conflicts, particularly in the Sub-Middle and Lower São Francisco regions [24]. These challenges are further exacerbated by conflicting priorities among reservoir managers and regulatory agencies, which complicate water governance [26]. To address this issue, researchers advocate reducing dependence on hydropower by investing in alternative energy sources and establishing clear and consistent rules for reservoir management [24, 26, 27, 28].

A sustainable electricity matrix for the Northeast should prioritize self-sufficiency, energy efficiency, and the expansion of renewable energy, while minimizing the negative impacts of fossil fuels and addressing climate change [12]. Achieving this requires an integrated approach that balances the complex interplay of social, environmental, and economic factors [29, 30]. Multi-criteria decision tools, such as the Technique for Order of Preference by Similarity to Ideal Solution (TOPSIS), have proven effective in supporting such decisions [29]. TOPSIS is particularly advantageous for evaluating energy sources, as it efficiently manages trade-offs among multiple conflicting criteria, providing clear and reliable rankings [29, 31].

Given this context, the present study explores potential configurations for the Northeast's electricity matrix using the TOPSIS model. By simulating scenarios and defining criteria to assess water-energy sustainability, it aims to determine whether the region's electricity system can meet demand under persistent drought conditions. The ultimate goal is to contribute to sustainable development by ensuring a balance among economic, social, and environmental objectives in the São Francisco River Basin.

2. METHODOLOGY

Considering the intrinsic link between water management and hydropower generation, the methodology proposed in this paper aims to support electricity planning through tools that promote effective hydro-energetic governance. Multi-Criteria Decision Analysis (MCDA) is one of the most widely used approaches in the energy sector and in decision-making processes for water resource management [32].

The procedural steps for modelling the MCDA are illustrated in Fig. 3. The process is primarily divided into two main phases: 1) the structuring phase, which involves identifying criteria and defining alternatives, and 2) the evaluation phase, which involves applying the MCDA method and assessing the results.

This paper focuses on the structuring phase, encompassing problem definition, scenario development, and the selection of relevant criteria and sub-criteria.

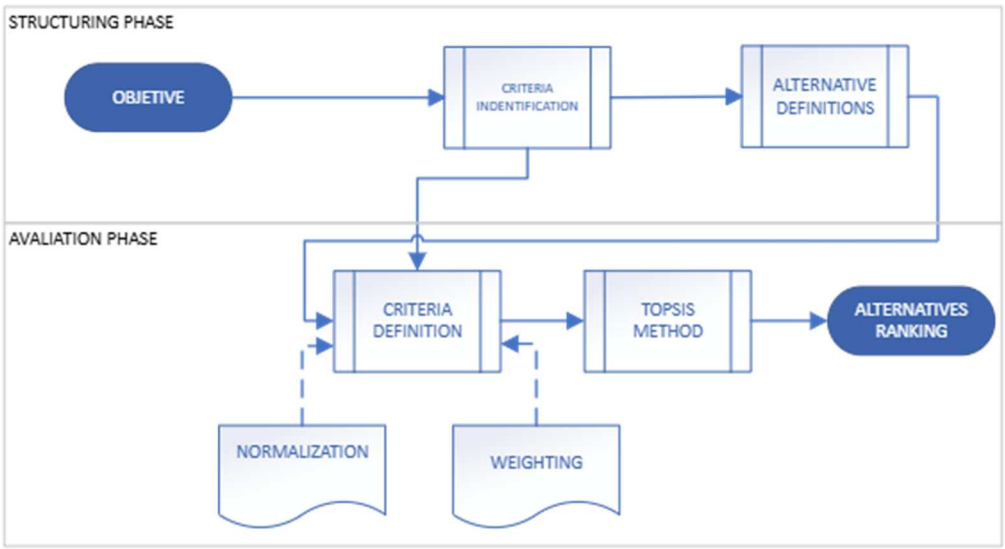


Fig. 3- Flowchart explaining the methodology used to apply the MCDA model

2.1. Objective structuring

The literature offers numerous methods for addressing multi-objective problems through multi-criteria decision-making approaches. Among these, four problem-structuring methods stand out: Strategic Options Development and Analysis (SODA), Soft Systems Methodology (SSM), Strategic Choice Approach (SCA), and Value-Focused Thinking (VFT) [33]. This study adopts SSM due to its effectiveness in clarifying the problem context, identifying stakeholders and their relationships, and structuring the fundamental objective for MCDA [27, 34] (Fig. 4).

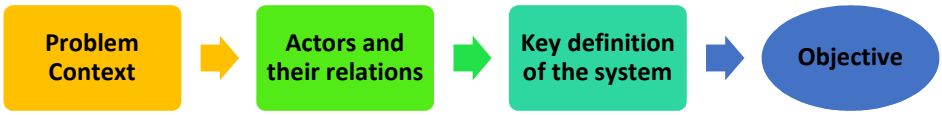


Fig. 4 Phases in the structuring of the problem using SSM, leading to the identification of the objective

SSM employs tools like the rich picture—a diagrammatic representation that captures the complexity of the situation from multiple perspectives [33, 35, 36] to help identify the problem context and the relationships among actors. To further define the system, the CATWOE framework is used, addressing key elements:

- Customers: Beneficiaries or affected parties of the system transformation.
- Actors: Those responsible for implementing activities.
- Transformation: The change the activities aim to achieve.
- Worldview: The rationale behind pursuing transformation.
- Owner: The individual or entity with authority over the system.
- Environment: External constraints, such as cultural or political factors.

This structured approach ensures a comprehensive understanding of the system, facilitating effective problem resolution.

2.2. Criteria identification

The methodology used for identifying criteria and sub-criteria, as well as the participatory decision-making process, was adapted from Cloquell-Bellester et al. [37]. The procedure for defining the criteria consisted of three stages: a) selection and preliminary assessment of criteria, b) interviews with professionals and scientists, and c) aggregation of results (Fig. 5).

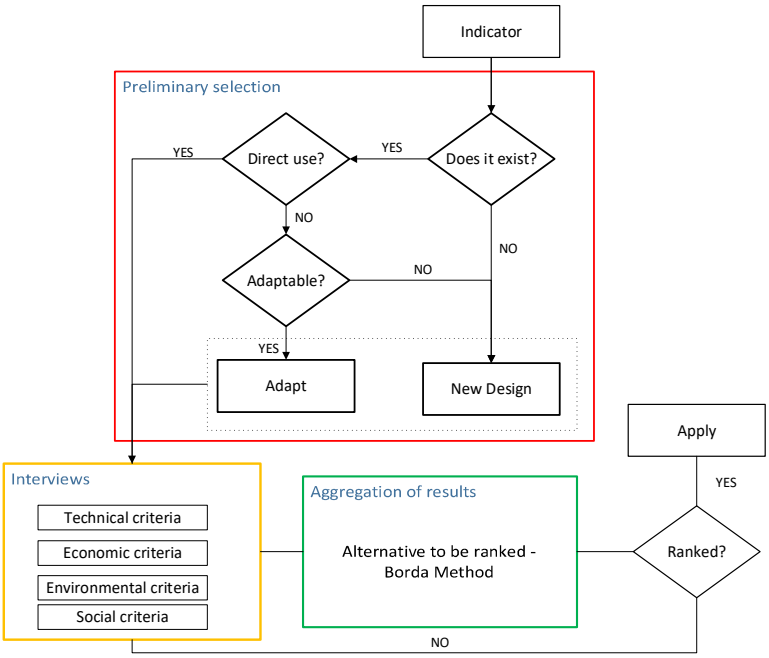


Fig. 5 Process of criteria and sub-criteria selection for MCDA. Adapted from Cloquell-Bellester et al. [37]

The following questions guided the selection of stakeholders/actors for subsequent administration of the interviews, as suggested by Almeida et al. [33]:

Stakeholders/actors and their needs:

I. Who are the decision-makers?
II. What are the roles of the stakeholders/actors regarding the problem context?
III. Which stakeholders are affected?
IV. Did the stakeholders/actors collaborate in previous situations?

Knowledge level:

I. How familiar with the problem context are the stakeholders/actors?
II. Which aspects of the problem context are members aware of?
III. How will knowledge sources be obtained during the process?

Possibility of use of support tools:

I. How much time and effort can group members invest in the process?
II. Which methodological tools are best aligned with the needs and possibilities (workshops, video conferences, online search forms, etc.)?

Presence of uncertainties:

I. How much is known about the different decision alternatives and their consequences?
II. Can the uncertainties substantially be reduced?

Applying the listed questions, around 35 institutions were selected amongst private and public companies in the electrical sector, institutions related to water and environment management, as well as universities. The aim was to select the stakeholders/actors from each institution who had direct relation and familiarity with each of the four main groups of criteria: technical, economic, environmental and social. Around 100 stakeholders/actors were pre-selected for interviewing, and around 60 participated in the interviews. The inquiries were made through personal interviews by video conference, telephone and online forms. At the start of each interview a short introduction was made about the research and the facilitator, and some questions were asked to verify the level of knowledge of the interviewee with the problem and decisions at stake.

Given the need for group decisions, it was essential to apply methods for aggregating the results from individual interviewees. Since the interviewees formed a heterogeneous group with differing views and objectives, making consensus difficult, Forman and Peniwat [38] recommended using the Aggregation of Individual Priorities (AIP) method. The interviewees were divided into four sub-groups to separately analyze preferences for each technical, economic, environmental, and social criterion, with each decision-maker grouped according to their area of expertise. Using survey forms, the relative importance of each criterion was assessed for the development of an MCDA model to support strategic decisions in the electricity sector. This approach made each sub-group more homogeneous, and as a result, the aggregation of judgments was carried out in such a way that the sub-group acted as a “new” individual (AIJ method).

The ranking methodology used was the Borda Method [33, 39]. This method involves ordering each sub-criterion of every decision-maker by preference, assigning points such that the most preferred sub-criterion receives the most points, and the least preferred receives fewer. Almeida et al. [33] suggest a parameterization where the worst alternative (least preferred) is assigned a, the second-worst gets a + b, the third-worst a + 2b, and so on. Thus, the ranking of results follows a decreasing order, from the criterion with the most points to the one with the fewest. The Borda method was chosen for its simplicity and robustness. The ranking involved summing the points for each sub-criterion across all decision-makers in the group, producing a combined judgment from all interviewees to determine the most appropriate results for each sub-criterion within the multi-criteria model.

2.3. Alternative Definition

Climate change is impacting water balance and demand, necessitating new management strategies and risk mitigation, especially in regions with uneven precipitation distribution, which

negatively affect various water uses, including hydropower generation. The IPCC provides climate change projections and carbon emission pathways (RCPs), which are based on economic activity, energy use, population growth, and socio-economic factors up to 2100. The study selects two RCPs for scenario simulation: RCP2.6, which assumes immediate greenhouse gas reductions through political intervention, and RCP8.5, which assumes continued population growth and slow technological development, resulting in high emissions. The study uses MIROC for RCP2.6 and HadGEM for RCP8.5 to simulate climate impacts on the São Francisco River Basin, considering both drought and wet future projections. Two sets of climate models were applied: MIROC RCP2.6 and HadGEM RCP8.5, with downscaled and corrected data from the CMIP5 project.

Souza Júnior et al. (2019) [40] used the SWIM (Soil and Water Integrated Model) to simulate hydroelectric generation in various reservoir management scenarios, taking climate change into account. For this study, the focus was on prioritizing reservoir use for hydroelectric power generation in the São Francisco basin through 2050, with the results used to inform a TOPSIS model for water-constrained electricity expansion.

The analysis included 07 (seven) alternatives (Table 1), with A1 as the reference alternative. Alternatives A2, A4, and A6 incorporated climate change, simulated using the MIROC RCP2.6 model, while A3, A5, and A7 used the HadGEM RCP8.5 model. No new hydroelectric plants were included in the alternatives, and generation changes were only based on climate factors.

The electricity matrix considered hydroelectric (excluding small plants), wind, solar, and thermoelectric sources, with gas-fired thermoelectric generation accounting for the majority in the region.

Reference Alternative

This scenario is based on the expansion of energy sources considering the average annual generation in 2022 for the Northeast (NE) region. It assumes proportional increases in wind, solar, and thermoelectric electricity generation according to their share in the energy matrix of that year. The hydroelectric generation for 2022 is based on the HadGEM RCP8.5 model results.

Alternative Wind Expansion

This scenario focuses on growing energy supply primarily from wind power. It combines the National Energy Plan (PNE) 2050 scenario, which targets a 100% renewable energy matrix, with a limited expansion of solar and thermoelectric power according to PNE 2030.

Alternative Solar Expansion

This alternative prioritizes solar power expansion. It uses PNE 2050 projections for a 100% renewable energy matrix, with limited wind expansion and thermoelectric growth as defined by PNE 2030.

Alternative Thermoelectric Expansion

This scenario assumes the growth of solar and wind power as defined in the wind and solar expansion alternatives, respectively. To meet demand, it compensates by increasing thermoelectric generation.

Table 1 - Electricity supply alternatives up to 2050 in the Northeast

Alternatives / Electricity Sources		Hydroelectric (mW)	Wind (mW)	Solar (mW)	Thermoelectric (mW)
Referenc	A1	5100	77400	5730	6120
Wind	A2	2950	74000	5600	11800
	A3	5100	71850	5600	11800
Solar	A4	2950	17000	62600	11800
	A5	5100	17000	60450	11800
Thermoelectric	A6	2950	17000	5600	68800
	A7	5100	17000	5600	66650

Sources: data based on [40,41,42].

2.4. Criteria Definition (normalization and weighting)

Ten (10) criteria were selected through stakeholder/actor interviews, and the results were calculated using Borda's method (item 2.2). In accordance with each electricity source used in the alternatives framework, the scoring for each criterion was defined, as shown in Table 2.

Table 2 - Criteria and scoring by electrical sources before the standardization procedure.

Criteria		Electrical Sources (scoring)			
		Wind power	**Solar power**	**Thermal power**	**Hydro power**
C1	Energy efficiency (%)	35 (a)	9,5 (a)	39 (a)	80 (a)
C2	Generation capacity (%)	25 (b)	11 (b)	38 (b)	52 (b)
C3	Technological maturity (from 0 to 7)	3,7 (c)	5,6(c)	6,0 (c)	4,5 (c)
C4	Investment cost (U$/kW)	4500 (d)	4000 (d)	5100 (d)	10200 (d)
C5	Operation and maintenance cost (U$/kW h)	90 (d)	50 (d)	150 (d)	50 (d)
C6	Land use and occupation (Km²/1000 MW)	100 (e)	35 (e)	2,5 (e)	750 (e)
C7	CO2 emissions (Kg CO2e/MWh)	12 (a)	48 (a)	820 (a)	2 (a)
C8	Public acceptance (%)	83 (f)	88 (f)	26 (f)	65 (g)
C9	Job creation (employees/100 MW)	1140 (h)	2500 (h)	1770 (h)	1110 (h)
C10	Fatalities (deaths/GW years)	0 (a)	0 (a)	157 (a)	3 (a)

Sources: (a) [41]; (b) [42]; (c) [43]; (d) [44]; (e) [45]; (f) [48]; (g) [49]; (h) [50].

Almeida et al. (2021) [51] recommend consulting experts with specific knowledge of the study area to evaluate the 10 criteria in a multicriteria decision model. A workshop was conducted at the São Francisco Hydroelectric Company (CHESF). Almeida (2021) [51] explains that expert consensus can be reached using two approaches: 1) mathematical models, or 2) modifying opinions until agreement is reached. In this study, the consensus was considered a "calculated opinion," based on a mathematical framework rather than complete agreement. Sixteen experts evaluated the importance of each criterion using a cardinality scale. The linear opinion pool method (Stone, 1961) was applied to combine individual expert probabilities, weighted by

their relative quality. In this case, all experts were given equal importance, meaning the combined probability was calculated as an arithmetic mean. The evaluation scale ranged from 0 to 60 points, with a verbal scale to aid in value judgment. After applying the method and performing normalization, the results were presented in Table 3.

Table 3 - Criteria weights used in the TOPSIS model

Criterion		Weight (Pn)	Criterion		Weight (Pn)
Technical	C1	0.092	**Environmental**	C6	0.077
	C2	0.089		C7	0.070
	C3	0.082			
Economic	C4	0.091	**Social**	C8	0.081
	C5	0.086		C9	0.076
				C10	0.074

2.5. TOPSIS Method

The basis of the TOPSIS multi-criteria method is the minimization of the distance from the Positive Ideal Solution Point (PIS) and the maximization of the Negative Ideal Solution Point (NIS) for each alternative evaluated [52]. Then, based on the Euclidean distances between these two points, the relative proximity (RC) is calculated.

The topics below describe the recommended procedures for implementing the method:

Decision matrix

The decision matrix (equation (1) comprises the alternatives to be evaluated (Am), the selected criteria (Cn), and the scores for each criterion before the normalization procedure (Xmn).

$$D = \begin{array}{c} \\ A1 \\ A2 \\ \vdots \\ Am \end{array} \begin{array}{c} \begin{array}{cccc} C1 & C2 & \dots & Cn \end{array} \\ \begin{bmatrix} X11 & X12 & \dots & X1n \\ X21 & X22 & \dots & X2n \\ \vdots & \vdots & \vdots & \vdots \\ Xm1 & Xm2 & \cdots & Xmn \end{bmatrix} \end{array} \quad (1)$$

Scale transformation procedure (normalization)

Since any quantitative criterion score must have a single scale for comparison purposes, all values must be normalized. In other words, the results of criteria with different units must be

normalized under the same measurement scale. Several methods for normalization can be used, depending on the type of problem. Normal, linear and fuzzy methods are the most commonly adopted. In this study, the method used for normalization is the division by the maximum value approach according to equation (2) [33, 52]:

$$Nmn = \frac{Xmn}{\sqrt{Max\ Xmn}} \tag{2}$$

Where: Nmn = normalized element and Xmn = scoring value for each criterion.

As a result, the normalized decision matrix (Dn) is obtained according to equation (3) below.

$$Dn = \begin{array}{c} \\ A1 \\ A2 \\ \vdots \\ Am \end{array} \begin{array}{c} \begin{array}{cccc} C1 & C2 & \dots & Cn \end{array} \\ \begin{bmatrix} N11 & N12 & \dots & N1n \\ N21 & N22 & \dots & N2n \\ \vdots & \vdots & \vdots & \vdots \\ Nm1 & Nm2 & \cdots & Nmn \end{bmatrix} \end{array} \tag{3}$$

Weighted Decision Matrix

Weighted decision matrix (Dp) is the result of multiplying the vector of p-values by the normalized matrix in equation (3).

$$P = [P1, \quad P2, \quad \cdots \quad Pn] \tag{4}$$

$$Dp = \begin{array}{c} \\ A1 \\ A2 \\ \vdots \\ Am \end{array} \begin{array}{c} \begin{array}{cccc} C1 & C2 & \dots & Cn \end{array} \\ \begin{bmatrix} N11 & N12 & \dots & N1n \\ N21 & N22 & \dots & N2n \\ \vdots & \vdots & \vdots & \vdots \\ Nm1 & Nm2 & \cdots & Nmn \end{bmatrix} \end{array} x\ P = \begin{array}{c} \\ A1 \\ A2 \\ \vdots \\ Am \end{array} \begin{array}{c} \begin{array}{cccc} C1 & C2 & \dots & Cn \end{array} \\ \begin{bmatrix} P11 & P12 & \dots & P1n \\ P21 & P22 & \dots & P2n \\ \vdots & \vdots & \vdots & \vdots \\ Pm1 & Pm2 & \cdots & Pmn \end{bmatrix} \end{array} \tag{5}$$

Where: Pmn = Weighted matrix element

Setting the positive (PIS) and negative (NIS) ideal solution point

PIS (Positive Ideal Solution Point) and NIS (Negative Ideal Solution Point) are calculated using the Euclidean method as shown in equations (6) and (7) [33].

$$D^{+} = \sqrt{\sum_{j=1}^{n} (Vn^{+} - Pmn)^{2}} \quad (6)$$

$$D^{-} = \sqrt{\sum_{j=1}^{n} (Vn^{-} - Pmn)^{2}} \quad (7)$$

Where D^{+}= distance to PIS, D^{-}= distance to NIS, Vn^{+} = ideal alternative; Vn^{-} = anti-ideal alternative.

Determining relative proximity (RC)

Equation (8) represents relative proximity (RC). The closer the alternative is to the PIS and the further away it is from the NIS, the higher it will be. The best choice is the one with the highest RC, which is the coefficient of ranking preference order [31].

$$RC = \frac{D^{-}}{D^{+} + D^{-}} \quad (8)$$

3. RESULTS

To analyze different energy matrix configurations to meet electricity demand as hydroelectricity sources come under stress due to climate change, 7 alternatives were divided into groups in order to compare options with common characteristics (Table 5). Three clusters were separated by electricity source: cluster 1 wind expansion (alternatives A2, A3), cluster 2 solar expansion (alternatives A4, A5) and cluster 3 thermoelectric expansion (alternatives A6, A7). These groups were analyzed from the perspective of moderate influence and more drastic influence of climate.

Table 5 shows the framework with the RC (relative proximity) of each alternative. The best option is the one with the highest RC. The nearer the RC is to 1, the more viable this alternative is in terms of the criteria and weights defined.

Considering the reference energy source expansion scenario, which is based on average annual generation in 2022 [22], the Reference Scenario (A1) would be one reasonably good option, with an RC of 0.6805. It is worth noting that this alternative considered a milder climate change, as the hydroelectric source in 2022 had an annual average close to that calculated by

the HadGEM RCP8.5 model, so the result simulated by it was used. It also considered a higher percentage of wind expansion, which proved to be a very favorable source in the multicriteria model adopted. A smaller increase in thermoelectric generation was also taken into account among all the alternatives analyzed.

Table 4 - Framework of alternatives analyzed in the TOPSIS model and their rankings

Group	Alternative	Description		RC	Overall Ranking
A1		Reference		0.6805	3º
01	A2	Set A	Wind Expansion	0.6637	5°
	A3	Set B		0.6640	4°
02	A4	Set A	Solar Expansion	0.6965	2º
	A5	Set B		0.7064	1º
03	A6	Set A	Thermal Expansion	0.3208	7°
	A7	Set B		0.3290	6°

The alternatives in Group 2, which emphasize a greater expansion of solar energy, ranked first and second in the evaluation. Alternative A5, which assumes a lower impact of climate change on hydropower generation, achieved the best performance. The model identifies an ideal scenario, based on the selected criteria, that favors increased investment in solar energy expansion combined with minimal climate change influence, enabling a significant contribution from hydropower sources.

Following the reference option (A1), which also considers hydropower sources with minimized climate effects, Group 1—focused on greater integration of wind energy—ranks 5th and 4th for alternatives A2 and A3, respectively. Similarly, the alternative with the highest share of hydropower emerges as the most advantageous within this group.

An important consideration for understanding why the wind option is less preferable than the solar option lies in the significant weighting assigned by experts to technical criteria, particularly C4 (Investment cost), C5 (Operation and maintenance cost), and C8 (Public acceptance). In these criteria, wind power significantly underperforms compared to solar energy, which negatively impacts its overall ranking.

Group 3, which emphasizes greater thermoelectric expansion, occupies the lowest positions in the evaluation. The worst-case scenario among all analyzed alternatives is A8, characterized by increased thermoelectric expansion combined with no climate change effect, i.e., greater use of hydropower. This group is identified as the least favorable option, largely due to its poor performance in criteria associated with high CO2 emissions (C9), low public acceptance (C10), and the risk of fatal accidents linked to the operation and maintenance of power plants (Table 2).

4. CONCLUSIONS

The alternatives that simulate less reduction in hydroelectric power represent more efficient energy matrices, according to the TOPSIS multicriteria model developed. However, this model does not consider water use conflicts in the São Francisco basin. According to Souza Junior et al. (2017) [55], hydroelectric power generation is one of the activities that most frequently generates conflicts among reservoir water users. Therefore, a model that could consider the effects of reservoir management prioritized for hydropower generation and its impact on water use conflicts in the basin would be more comprehensive and aligned with the practices of integrated water resources management. One way to account for these effects would be to include a criterion that considers, for example, the importance of maintaining the minimum flow for the transportation of goods such as soy, corn, and cotton (navigation), as well as the minimum flow required to support economic activities such as agriculture, industry, aquaculture, and domestic water supply, among others, in contrast to release rules designed to optimize hydropower generation.

According to the PDE 2022, the assumptions include greater investment in wind and solar power and no hydroelectric expansion in the São Francisco region. Given this, the scenario appears to align well with the results of this study. However, the plan does not directly address the issue of climate change, merely recommending that weather changes be considered as a parameter in future supply expansion studies. Another aspect is that the environmental, social, and technical nuances are not clearly incorporated into the scenario studies of the energy plans. Instead, the analysis of the various scenarios tends to rely on a mathematical model for investment decisions, which indicates that the optimal expansion involves minimizing investment and operating costs.

Innovations in the proposed multicriteria model include incorporating the effects of climate change on hydroelectric generation. This approach contributes to better alignment between basin management plans and energy plans, as it brings the conflicts between basin water

users into the electricity expansion project discussion. Additionally, it enables a multicriteria study to develop alternatives for electricity development.

Several criteria selected for use in the model had scores derived from international publications, where values vary greatly across different bibliographic sources. For example, item C10, which addresses public acceptance of electricity sources, is a criterion strongly influenced by cultural and regional factors. However, since the data used were sourced from research by Sharpton et al. (2020) [48], there is a lack of local studies to inform this criterion. Furthermore, some criteria, such as C9 (CO_2 emissions), present significant variability between authors. For instance, Dipto et al. (2020) [43] estimate that 2 kg of CO_2 are emitted per MWh of hydroelectric power produced, whereas Amer and Daim (2011) [53] report 40 kg per MWh. These discrepancies introduce uncertainty into the model and, depending on the values used, could alter the ranking of alternatives. To address this issue, efforts were made to use case studies from similar research lines to obtain more accurate and representative data for the local context. Moreover, a broader survey that better distributed experts among economic, ecological, social, and technical criteria groups could have yielded more effective results, as participation from environmental specialists was limited.

For future improvements, it would be beneficial to compare this multicriteria approach with other established methodologies, such as AHP and ELECTRE, to assess potential differences in the ranking of alternatives. Another avenue for enhancement would be to develop a linguistic TOPSIS model (Fuzzy TOPSIS). Koutsandreas and Keppo (2023) [4] utilized a modified fuzzy TOPSIS method for energy planning decision support, employing fuzzy numbers and linguistic variables to prioritize criteria and rank alternatives. And finally, it is recommended to conduct a sensitivity analysis of the model. This sensitivity analysis will assess the model's robustness regarding both input data and parameters. The objective of this analysis is to evaluate the impact on the model output caused by variations in its input [33].

The findings of this study underscore the significant potential and strategic importance of transitioning to a more sustainable energy matrix in Northeast Brazil. As global energy dynamics shift towards cleaner and renewable sources, Northeast Brazil is uniquely positioned to leverage its abundant natural resources, such as solar and wind energy, to meet both regional and national energy demands.

Implementing renewable energy solutions aligns with environmental sustainability and offers socio-economic benefits, such as economic growth and job creation. Reducing reliance on fossil fuels also enhances energy security and mitigates climate change impacts.

An MCDA tool is essential for evaluating and prioritizing energy alternatives, considering costs, environmental benefits, and social impacts. It enables a structured analysis of multiple options, facilitating decision-making and ensuring transparency.

In conclusion, transitioning to a renewable energy matrix in Northeast Brazil is crucial for a resilient, sustainable future. Collaboration among policymakers, stakeholders, and the community is key to overcoming challenges and harnessing renewable energy's full potential. This study provides valuable insights to guide future energy policies.

REFERENCES

[1] Shen, Y.; Chou, C. J.; Lin, G.T. (2011). The portfolio of renewable energy sources for achieving the three policy goals. Energy. 36, 2589–2598.

[2] Neves, L.P.; Dias, L.C.; Antunes, C.H.; Martins, A.G (2009). Structuring an MCDA model using SSM: A case study in energy efficiency. European Journal of Operation Research. 199, 834-845.

[3] Cristóbal, J. R. S. (2010). Multi-criteria decision-making in the selection of a renewable energy project in Spain: The Vikor method. Renewable energy. 36, 498-502.

[4] Abulfotuh, F. (2008). Energy efficiency and renewable technologies: the way to sustainable energy future. Desalination. 209, 275–82.

[5] Bilgen, S; Keles, S.; Kaygusuz, A.; Sari, A.; Kaygusuz, K. (2008). Global warming and renewable energy sources for sustainable development: a case study in Turkey. Renewable and Sustainable Energy Reviews. 12, 372–96.

[6] Riahi, K.; Krey, V.; Rao, S.; Chirkov, V.; Fischer, G.; Kolp, P.; Kindermann, G.; Nakicenovic, N.; Rafai, P. (2007). RCP-8.5: exploring the consequence of high emission trajectories, Climatic Change. doi: 10.1007/s10584-011-0149-y.

[7] Blanco, G.; Amarilla, G.; Martinez, C.; Llamosas, V.; Oxilia, V. (2017). Energy transitions and emerging economies: A multi-criteria analysis of policy options for hydropower surplus utilization in Paraguay. Energy Policy. 108, 312-321.

[8] Amer, M.; Daim, T. U. (201). Selection of renewable energy technologies for a developing country: A case of Pakistan. Energy for Sustainable Development. 15, 420–4432.

[9] Garni, H.; Kassem, A.; Awasthi, A.; Komljenovic, D.; Al-Haddad, K. (2016). A multicriteria decision making approach for evaluating renewable power generation sources in Saudi Arabia. Sustainable Energy Technologies and Assessments. 16, 137–150 (2016).

[10] Kahraman, C.; Kaya, I.; Cebi, S. (2009). A comparative analysis for multiattribute selection among renewable energy alternatives using fuzzy axiomatic design and fuzzy analytic hierarchy process. Energy. *34*, 1603–1616.

[11] Luthra, S.; Kumar, S.; Garg, D.; Haleem, A. (2015). Barriers to renewable/sustainable energy technologies adoption: Indian perspective. Renewable and Sustainable Energy Reviews. 41, 762–776.

[12] Wijayatunga, P., D. C.; Siriwardena, K.; Fernando, W. J. L. S.; Shrestha, R. M.; Attalage, R. A. (2006). Strategies to overcome barriers for cleaner generation technologies in small developing power systems: Sri Lanka case study. Energy Conversion and Management. 47, 1179–1191.

[13] Cannemi, M.; García-Melón, M.; Aragonés-Beltrán, P. (2014). Modeling decision making as a support tool for policy making on renewable energy development. Energy Policy. 67, 127–137.

[14] Kambezidis, H., D.; Kasselouri, B.; Konidari, P. (2011). Evaluating policy options for increasing the RES-E penetration in Greece. Energy Policy. 39, 5388–5398.

[15] Karatayev, M., Hall, S.; Kalyuzhnova, Y.; Clarke, M. L. (2016). Renewable energy technology uptake in Kazakhstan: Policy drivers and barriers in a transitional economy. Renewable and Sustainable Energy Reviews. 66, 120–136.

[16] Strantzali, E.; Aravossis, K. (2016). Decision making in renewable energy investments: A review. Renewable and Sustainable Energy. 55, 885-898.

[17] Koch, H.; Silva, A.; Azevedo, R.; Souza, W.; Köppel, J.; Souza Júnior C.B.; Hattermann, F. (2018). Integrated hydro- and wind power generation: a game changer towards environmental flow in the Sub-middle and Lower São Francisco River Basin? . Online first .doi: 10.1007/s10113-018-1301-2.

[18] Østergaard, P.A. (2009): Reviewing optimization criteria for energy systems analyses of renewable energy integration. Energy. 34, 1236–1245.

[19] Agência Nacional de Águas. Accessed 15 January at http://www2.ana.gov.br/.

[20] Agência Nacional de Águas. Resolução n.º 74272017.Accessed 06 July 2017 at http://arquivos.ana.gov.br/resolucoes/2017/224-2017.pdf

[21] MME/EPE. Plano Decenal de Expansão de Energia 2024. Accessed 23 May 2016 at http://www.epe.gov.br/PDEE/.

[22] Brasil, 2023. National Energy Operator - ONS. Operation History. Accessed April 03, 2023 at https://www.ons.org.br/.

[23] Brasil, 2022. Energetic Research Company – EPE. Ten-Year Energy Expansion Plan – PDE 2031. Accessed november 18, 2023 at http://www.epe.gov.br/pt/.

[24] Resnier, M.; Wang, C., D, P.; Chen, J. (2007). The promotion of sustainable development in China through the optimization of a tax/subsidy plan among HFC and power generation CDM projects. Energy Policy. 35, 4529–44.

[25] Silva, F. (2016). Oficinas Usos Múltiplos das Águas do Rio São Francisco. Accessed 17 December 2016 at http://cbhsaofrancisco.org.br/wp-content/uploads/.

[26] Silveira, S.; Filho, F.; Martins, E.; Oliveira, J.; Costa, A.; Nóbrega, M.; Souza, S., Silva, S. (2016). Mudanças Climáticas na bacia do São Francisco: uma análise para precipitação e temperatura. Revista Brasileira de Recursos Hídricos. 21, 416-428.

[27] CBHSF. (2016): Plano de Recursos Hídricos da Bacia Hidrográfica do São Francisco: resumo executivo do plano de recursos hídricos. Accessed 01 December 2016 at http://cbhsaofrancisco.org.br/planoderecursoshidricos/relatorios/.

[28] Garjulli, R. (2016). Oficinas Usos Múltiplos das Águas do Rio São Francisco. Accessed 17 December 2016 at http://cbhsaofrancisco.org.br/?wpfb_dl=993.

[29] Cavallaro, F.: Multi-criteria decision aid to assess concentrated solar thermal technologies. Renewable Energy. 34, 1678-1685 (2009).

[30] Afgan, N., Carvalho, M.: Multi-criteria assessment of new and renewable energy power plants. Energy. 27, 739-755 (2002).

[31] Rahim, R.; Putera, A.; Siahaan, U.; Wijaya, R.; Hantono, H.; et. al. (2021). Technique for Order of Preference by Similarity to Ideal Solution TOPSIS) Method for Decision Support System in Top Management. International Journal Engineering & Technology. v. 7 (3,4): 290-293.

[32] Strantzali, E.; Aravossis, K. (2016). Decision making in renewable energy investments: A review. Renewable an Sustainable Energy. 55, 885-898.

[33] Almeida, A.T.; Morais, D.C.; Costa, A.P.C.S.; Alencar, L.H.; Daher S.F.D. (2012) Decisão em Grupo e Negociação. São Paulo: Atlas.

.

[34] MME/EPE.: Nota Técnica DEA 13/15: demanda de energia 2050. http://www.epe.gov.br/PDEE/Forms/EPEEstudo.aspx. Accessed 23 May 2017.

[35] Coelho, D.; Antunes, C.H.; Martins A.G. (2009) Using SSM for Structuring an MCDA Model for Sustainable Urban Energy Planning. 5th International Vilnius Conference, 102-107.

[36] Medeiros, Y.; Freitas, I.; Stifelman, G.; Freire, O´Keeffe, J. (2013). Social participation in the environmental flow assessment: the São Francisco River case study. Revista Eletronica de Gestão e Tecnologia Ambientais. 1, 122-130 (2013).

[37] Cloquell-Ballester, V. A.; Monterde-Diaz, R.; Santamarina-Siurana, M. C. (2006). Indicators validation for the improvement of environmental and social impact quantitative assessment. Environmental Impact Assessment. 26, 79-105.

[38] Ram, C.; Montibeller, G.; Morton, A. (2011). Extending the use of scenario planning and MCDA for the evaluation of strategic options. Journal of the Operation Research Society. 62 (5), 817-829.

[39] ONS. (2016). Inventário das restrições operativas hidráulicas dos aproveitamentos hidrelétricos. Operador Nacional do Sistema Elétrico.. Acessed 25 September 2016 at http://ons.org.br/.

[40] Souza Junior, C.B.; Koch, H.; Siegmund-Schultze, M.; et al.; 2019. An exploratory scenario analysis of strategic pathways towards a sustainable electricity system of the drought-stricken São Francisco River Basin. Energy Syst. v. 12: 563-602. https://doi.org/10.1007/s12667-019-00343-1.

[41] Brasil, 2020. Energetic Research Company - EPE. National Energy Plan – PNE 2050. Accessed July 28, 2023 at http://www.epe.gov.br/pt/.

[42] Brasil, 2014. Energetic Research Company - EPE. National Energy Plan – PNE 2030 Accessed August 23, 2022 at http://www.epe.gov.br/pt/.

[43] Dipto, A.S.; Bari, A.; Nabil, S.T., 2020. Sustainability Analysis of Different Types of Power Plants Using Multi-Criteria Decision Analysis Methods. Journal of Engineering Advancements. v. 01 (03): 94-100. https://doi.org/10.38032/jea.2020.03.004.

[44] Bolson, N.; Pietro, P.; Patzek, T., 2022. Capacity factors for electrical power generation from renewable and nonrenewable sources. Proceedings of the National Academy of Sciences of United States of America., v.119 (12): 2205429119. https://doi.org/10.1073/pnas.220542911.

[45] Brasil,2017. National Electric Energy Agency - ANEEL. Technological Prospecting in the Electric Power Sector: National Technological Evolution in the Electricity Generation and Energy Storage Segment, 2017. Brasília, DF: Centro de Gestão e Estudos Estratégicos. v.3.

[46] Brasil, 2022. Energetic Research Company – EPE. Ten-Year Energy Expansion Plan – PDE 2031. (Accessed November 18, 2023) at http://www.epe.gov.br/pt/.

[47] Gómez-Romero, J.; Martínez-Rangel, M; Hernández-Aguilar, J; et. al., 2023. Relationship between the main economic, environmental and social impacts of hydroelectric dams. In: Capatan, A. Estudos sobre o funcionamento, desenvolvimento e organização das sociedades on the Brazilian Journals, Paraná, pp. 90-113.

[48] Sharpton, T.; Lawrence T.; Hall, M.,2020. Drivers and barriers to public acceptance of future energy sources and grid expansion in the United States. Renewable and Sustainable Energy Reviews. v.126: 109826. https://doi.org/10.1016/j.rser.2020.109826.

[49] European Commission. Energy Technologies: Knowledge, Perception, Measures. Special Eurobarometer 262 / Wave 6.3 – TNS Opinion & Social. Accessed Janeiro 15, 2021 at https://europa.eu/eurobarometer/screen/home.

[50] Hanna, R.; Heptonstall, P. and Gross, R.(2024). Job creation in a low carbon transition to renewables and energy efficiency: a review of international evidence. Sustain Sci. v. 19: 125–150. https://doi.org/10.1007/s11625-023-01440-y

[51] Almeida, A.T.; Morais, D.C.; Costa, A.P.C.S; Alencar, L.H.; Daher S.F.D. (2021). Decisão em Grupo e Negociação. Interciência, Rio de Janeiro, 292 p.

[52] Golfam, P.; Ashofet, P.; Rajee, T. et al. (2019). Prioritization of Water Allocation for Adaptation to Climate Change Using Multi-Criteria Decision Making (MCDM). Springer Nature. pp 3401-3416. https://doi.org/10.1007/s11269-019-02307-7.

[53] Amer, M.; Daim, T. U. (2011). Selection of renewable energy technologies for a developing country: A case of Pakistan. Energy for Sustainable Development. 15, 420–4432.

[54] Koutsandreas, D.; Keppo, I., 2023. A stochastic fuzzy multicriteria methodology for energy planning decision support: Case study of the electrification of the Greek road transport sector. Energy Strategy Reviews. v.50: 101 - 233.

[55] Souza Júnior, C.B.; Siegmund-Schultze, M.; Köppel, J. et al. (2017). Sinais de um problema crônico: a governança hídrica carece promover os comitês de bacias, coordenar planos e gerir informações. Rev. Ambient. Água [online]. v.12 (6):1054-1067. ISSN 1980-993X. http://dx.doi.org/10.4136/ambi-agua.2044.

SUPPORTERS

apa
agência portuguesa do ambiente

U.PORTO
FEUP FACULDADE DE ENGENHARIA
UNIVERSIDADE DO PORTO

NOVA
FACULDADE DE
CIÊNCIAS E TECNOLOGIA

1 2
9 0
FACULDADE DE
CIÊNCIAS E TECNOLOGIA
UNIVERSIDADE D
COIMBRA

edp

SOMINCOR
Sociedade Mineira de Neves-Corvo, S.A.
a subsidiary of lundin mining

CIGB
ICOLD
CBDB
Comitê Brasileiro de Barragens

LECM
澳門土木工程實驗室
LABORATÓRIO DE ENGENHARIA CIVIL DE MACAU
Civil Engineering Laboratory of Macau

CDA ACB
Canadian Dam Association
Association canadienne des barrages

Sociedade
Portuguesa
de Geotecnia

ORDEM
DOS
ENGENHEIROS

https://dw2025.lnec.pt/home
damworld@lnec.pt

AV DO BRASIL 101 • 1700-066 LISBOA • PORTUGAL
tel. (+351) 21 844 30 00
lnec@lnec.pt **www.lnec.pt**