Caniçada dam complementary spillway. Design, hydraulic model and ongoing works

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ABSTRACT: In order to meet the updated requirements of the Portuguese Dam Safety Legislation, Caniçada dam (double-curvature arch dam, concluded in 1955 and located in the Cávado River) hydraulic-operational safety assessment has been developed by EDP, including the review of the design flood and the evaluation of safety devices capacity. Thereafter some corrective measures were implemented, including the construction of a new complementary spillway. The paper summarizes the main features of the new spillway design, developed by AQUALOGUS. The selected solution is a gate-controlled ogee crest, followed by a 200 m long tunnel scoped in the slope of the left bank, designed for a discharge of 2062 m^3/s. The hydraulic performance of the designed structure was analysed by a study developed on a hydraulic physical model, built at LNEC on a scale of 1/62. Results and conclusions that allowed improving the design structure shapes are described, with particular emphasis on the use of the reduced physical model. Some aspects of interesting and nowadays challenging ongoing works of this new spillway are also presented.

Keywords: Dam, Spillway, Physical Model, Safety, Hydraulic Design.

1. INTRODUCTION

Caniçada dam is located in the Cávado River, which flows through north-western Portugal, being part of the hydro electrical system of Cávado-Rabagão-Homem owned and operated by EDP. It was built in 1955 and is a double-curvature arch dam with a maximum height of 76 m and at that maximum height a length of 196 m. For the normal water level (NWL), set at elevation (152.50), the flooded area is 522 ha and the maximum storage capacity is 153 hm^3.

The Caniçada scheme is formed by the dam and respective safety devices. The original safety devices are a spillway and a bottom outlet. The spillway, located in the central part of the dam body, Figure 1, consists of four rectangular orifices equipped with Stoney gates. The maximum discharge capacity is about 1700 m^3/s for the maximum water level (MWL_{OP}) defined in the original project (153.00).

The energy dissipation of the discharged free jets is made by impact on the river bed and on the water pool created by a weir built 100 m downstream from the dam. This weir is a concrete arch structure with a maximum height of 24 m above the river bed, crossed by two equal rectangular outlets 4.5 m wide and 3.0 m high, equipped with manual flat gates.

In order to meet the updated requirements of the Portuguese Dam Safety Legislation, RSB (2007), the hydraulic-operational safety assessment of Caniçada dam has been developed by EDP since 2006. The first phase of this assessment included a review of flood studies (in order to validate the previous design flood or to establish a new one), a suitability analysis of discharge devices and the outline of corrective structural measures, as detailed in Oliveira et al. (2012). The second phase consisted in the design and implementation of corrective measures, which included the construction of a new complementary spillway, as described in Oliveira and Dias da Silva (2012). This paper summarizes the main conclusions of the aforementioned dam safety assessment and presents the main features of the new spillway design, developed by AQUALOGUS (2012).

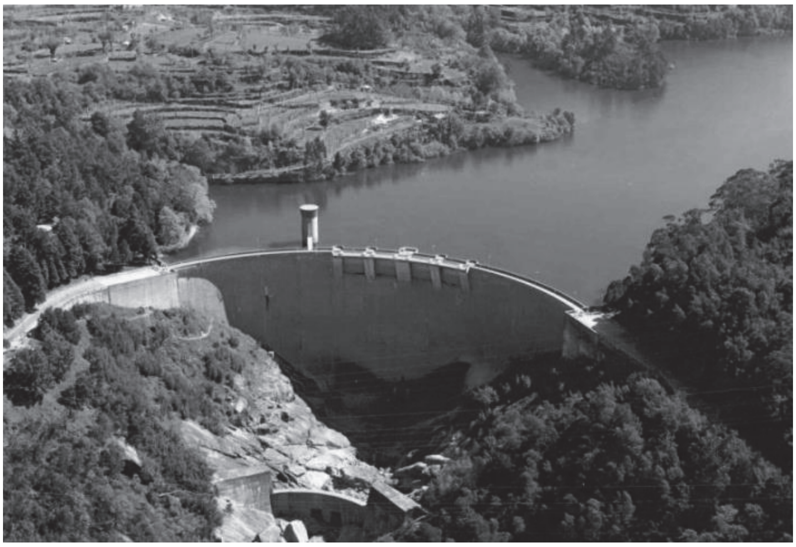


Figure 1. Caniçada dam.

The adopted solution for the new spillway is composed by a tunnel in the left bank. The design flow capacity is 2062 m^3/s for the new MWL, (152.83).

The hydraulic performance of the designed structure was analysed by a study on a physical model, carried out by Laboratório Nacional de Engenharia Civil, LNEC. This model intended the evaluation of the flow conditions under the operation of the new spillway or both spillways simultaneously and allowed testing alternative shapes to improve the hydraulic performance of the spillway (Couto et al. 2014). The studies in the physical model are mentioned in the paper.

Some aspects of the ongoing works are also referred.

2. SAFETY ASSESSMENT STUDIES

2.1 *Hydrologic study*

The review of the design flood was justified by the need to update the original flood studies, in some cases based on methods and criteria now considered obsolete or improper for local conditions, and/or based on short annual maximum flow series, often unreliably recorded.

Caniçada dam flood study was performed using the application of empirical formulas (Iskowski, Knichling, Forti, Possenti, Fuller, Creage, Giandotti and Gibrat) to establish the value of the peak flow and the Giandotti method for the definition of the flood hydrograph. The maximum discharge flow was 2400 m^3/s and the return period (T) is not clearly defined in the design.

According to the current Portuguese Dam Safety Legislation, RSB (2007), the return period of the design flood is defined taking into account the dam characteristics (type and height) and its potential hazard. Thus, the design flood for Caniçada dam (double arch type, 76 m maximum height) was defined with a 1000 years return period, and the check flood with a 5000 years return period.

The flood studies were undertaken based on new and updated hydro-meteorological data (obtained not only from over 35 years of daily maximum rainfall records in several stations within the catchment and surrounding areas, some of which with continuous recording gauges, but also from the available exploitation records of the dams) and the application of a rainfall-runoff model (HEC-HMS). This model was calibrated with some recorded hydro-meteorological events (October 1987, December 1988 and March 2001 floods). Obviously, these studies take into account the existence and exploitation of all the reservoirs in the Cávado-Rabagão scheme.

Figure 2 includes the calculated and observed hydrographs for the Cávado River at Caniçada dam section for the March 2001 flood. Figure 3 includes the calculated flood hydrographs for the Cávado River at Caniçada dam section associated with a return period of 1000 years (resulting from rainfall durations (d) between 6 and 30 hours and temporal distribution according to the Huff 2nd quartile) along with the initial design hydrograph.

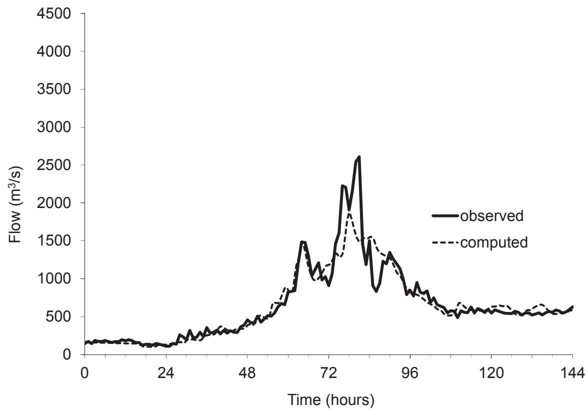


Figure 2. Computed and observed hydrographs at Cávado River in the dam section for the March 2001 flood.

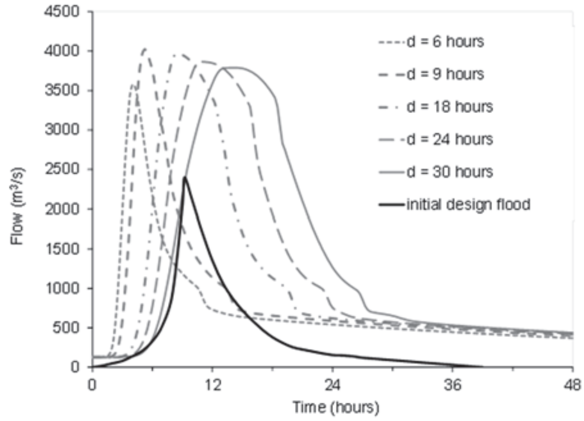


Figure 3. Computed flood hydrographs at Cávado River in the dam section.

Comparing the initial design hydrograph with that obtained in the reviewed study, it becomes evident that volumes and peak discharge flows are considerably higher than the initially adopted ones.

2.2 *Analysis of discharge devices*

These studies have shown that for a 1000 years return period flood, the existing spillway is not able to guarantee that the reservoir level does not exceed the actual MWL. In fact, the flood routing simulations indicated that the water level would be about 3.0 m above the dam upstream parapet wall, and even for the 100 years flood, the spillway would not ensure sufficient capacity and the water level would reach about 1,2 m above the upstream parapet wall.

Bearing in mind such conclusions, several corrective solutions were envisaged and studied in a preliminary analysis. These solutions were fitted in two groups, which are related with two hypothesis of reservoir exploitation: i) maintenance of Caniçada NWL and increase of actual discharge capacity by means of a new spillway; ii) conditioning the normal reservoir operation at Caniçada itself and at the existing reservoirs upstream during the rainy months (from October until the end of April), creating a storage volume for flood regulation.

The comparative analysis of the technical, economical and environmental aspects associated with each solution led to the decision of construction of a new spillway to work simultaneously with the existing one.

3 THE COMPLEMENTARY SPILLWAY DESIGN

3.1 *Justification and description of the adopted solution*

Beyond the dam safety legislation, in the choice and design of the spillway, the following essential aspects were taken into account: the location of the existing discharge devices and the existing and new hydraulic circuits; the topographical and geological characteristics of the downstream valley; the need to minimize any interference with the dam's body (thin arch) without the excessive lengthening of the new spillway structure; and the maintenance of the traffic between the two river banks during construction period. Due to the high discharge capacity required (≈2100 m3/s), the low difference between the maximum and normal reservoir levels (≈0.5 m) and the space constrictions, a solution with a gated spillway was selected.

The adopted solution for the spillway was designed near the left dam abutment and is formed by a gate-controlled ogee crest, followed by a 200 m long tunnel, ending in a ski jump structure. It has discharge capacity of 2062 m^3/s under the new reservoir maximum water level (MWL = 152.83).

The excavation of a horizontal approach platform at elevation (131.50), immediately upstream from the control structure, was planned. The ogee crest, at the elevation (138.50), is divided into two equal 8.75 m wide bays, fitted with radial gates. In each span the crest has a WES profile, with 1:1.5 (H:V) slope at the upstream face and 12.5 m of hydraulic design head. The flow separation at the crest is performed using a pier with hydrodynamic shape in plan.

The control structure is followed by a lined concrete tunnel with an approximate length of 200 m and variable cross-section, extending from the elevation (134.30) to the elevation (98.35). The longitudinal profile of the tunnel has two straight stretches, having slopes of 77% and 10%, concordant by a 50 m radius circular curve, Figure 4a. In the horizontal plan, the tunnel has a straight alignment, Figure 4b. A septum wall, in the continuity of the pier that separates the two spans of the control structure, divides the tunnel section along the entire tunnel length. In the initial stretch, along approximately 60 m, the cross-section is convergent between approximately 2 x 105 m^2 to a constant section of about 2 x 56 m^2. For the design discharge, the maximum water level inside the tunnel does not exceed 75% of the cross-section height, as design restriction criteria.

a)

b)

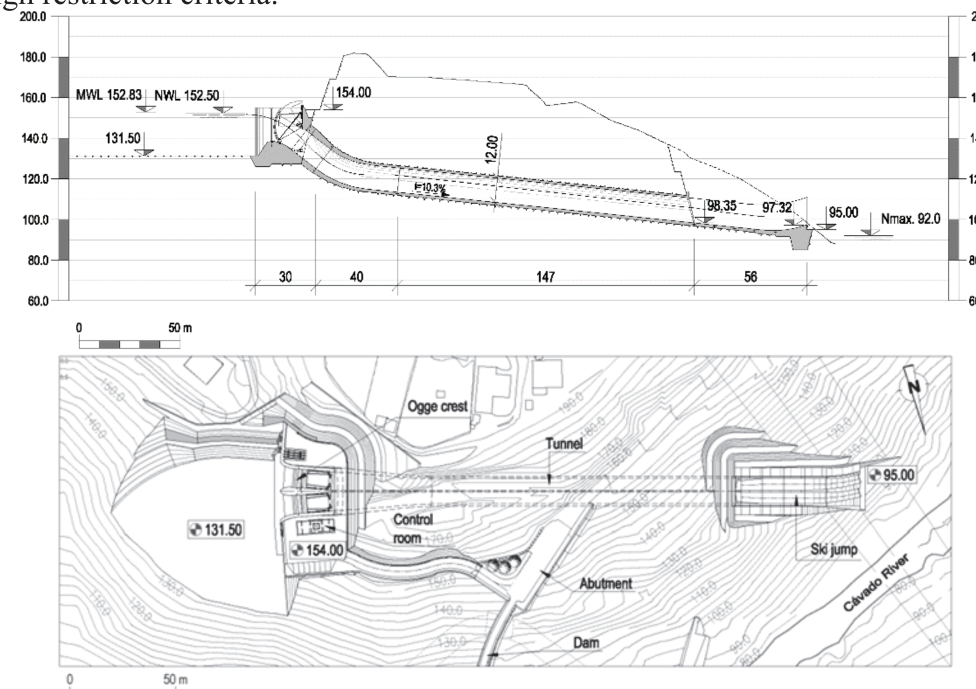


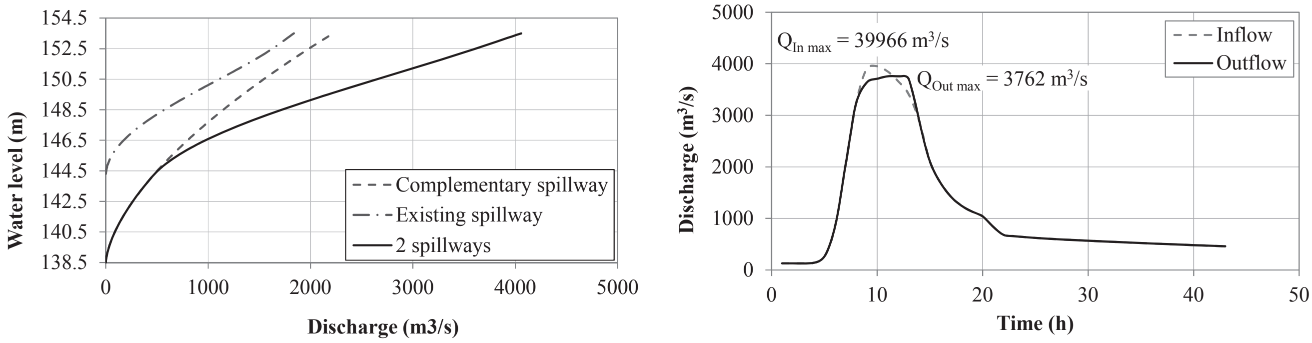
Figure 4. Complementary spillway: a) longitudinal profile view; b) horizontal plan view.

The downstream part of the spillway, 60 m long, is an open channel, ending by a ski jump structure, also divided by the septum. According to the design calculations, the impact zone of the discharged jets will be located about 45 m downstream of the terminal section of the spillway.

3.2 *Hydraulic design*

The complementary spillway was designed for a flood discharge with a 1000 years return period (T) and the condition of not exceeding the reservoir maximum water level (MWL_{OP} = 153.00, original project), considering its simultaneous operation with the existing spillway. It was checked for a 5000 years return period flood and for a 100 years flood in the case of one gate being out of service.

Flood routing in the reservoir was simulated using the reservoir storage curve, the spillways rating curves and the flood hydrographs determined in the flood revision (Figure 5). The complementary spillway design discharge was 2062 m^3/s (T = 1000 years) and the checking discharge was 2197 m^3/s (T = 5000 years). For the 1000 years return period flood, the total flow discharged through both spillways (existing and complementary) would be 3762 m^3/s for the new maximum water level (MWL = 152.83).



a) b)

Figure 5. a) Rating curves. b) Design flood inflow and outflow hydrographs (T = 1000 years).

The flow characteristics in the tunnel were analysed using a 1D model developed to compute gradually varied flow profiles in closed sections, Figure 6. The friction head losses were computed using Manning's coefficients n = 0.0133 and 0.0125. According to the Karman-Prandtl equation, in Quintela (1981), for the tunnel hydraulic diameter (between 8 and 12 m), these values are equivalent to an absolute roughness of 0.2 to 0.5 mm (rough concrete). The air entrainment effect on the water surface was not considered due to the short length of the tunnel.

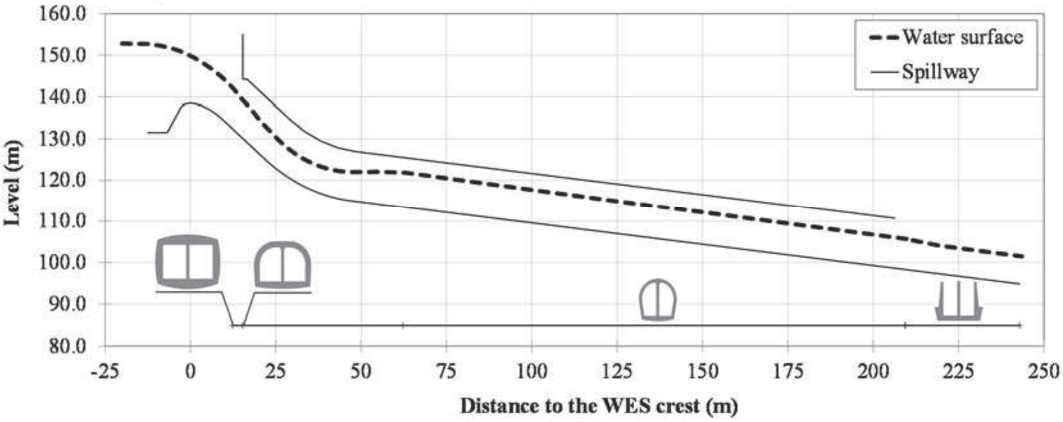


Figure 6. Water surface profile for design discharge (Q = 2062 m^3/s, n=0.0133).

The maximum flow depth in the tunnel was about 71% of the cross-section height for the design discharge (2062 m^3/s) and 75% of the cross-section height for the check discharge (2197 m^3/s). Therefore the tunnel discharge capacity was considered adequate.

It was verified that the maximum flow depths are located in the end of the cross-section convergent. Reduce the rise of the flow depth in this section would require a longer convergent and the expansion of the open-cut area. This would be a constructive and economic disadvantage.

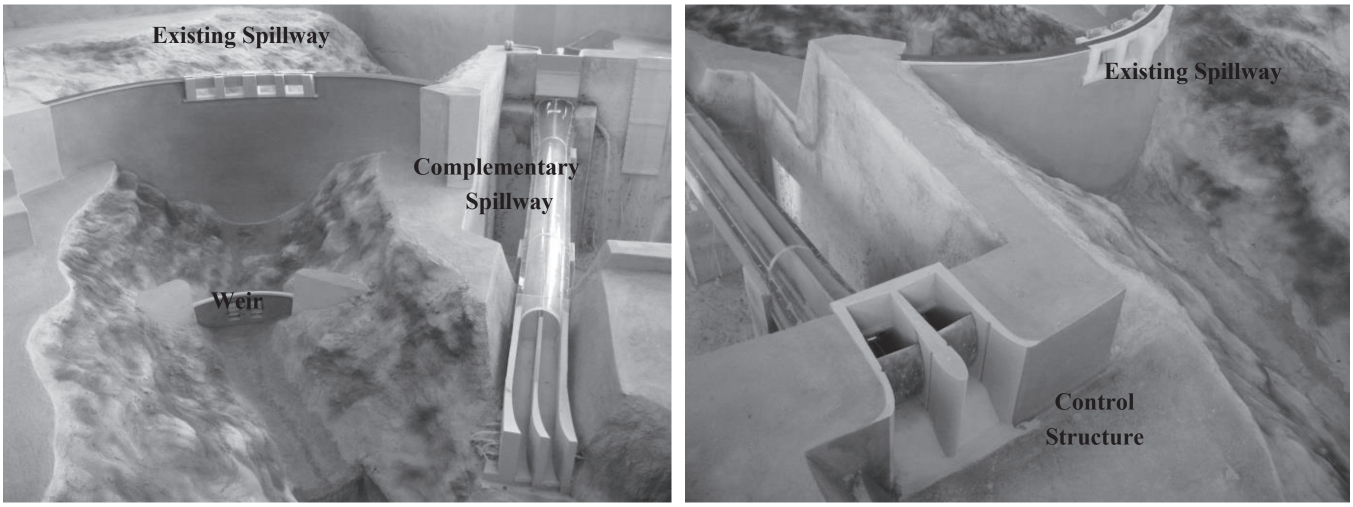
Due to the rock quality of the river bed downstream of the dam (Figure 1), a ski jump designed to direct the jet to the river bed was selected. To direct the jets to the riverbed, the spillway describes a curve in plan view, in its final 26 m length, which ends with an angle of 15° with the tunnel axis. In this final stretch, a convergent was considered to reduce the final sections width to 4.00 m and inclined lips. The ski-jump final geometry was adapted within the study on physical model, as described in the following section.

4 PHYSICAL MODEL HYDRAULIC STUDY

4.1 *Model description*

A non-distorted physical model with a 1/62 scale factor was built at LNEC with 2.3 m height, 6.7 m width and a length of 13.3 m. Froude similarity was used in this study. The complementary spillway, the dam and the original spillway were reproduced and significant reaches of the reservoir and of the downstream valley, including the river bed and the respective banks, to ensure an accurate reproduction of the flow conditions in the river.

Figure 7a presents the downstream view of the model, with the dam, the existing spillway in the middle, the complementary spillway in the right part and the protection weir downstream the dam. Figure 7b includes an upstream view of the model, where the control structure of the complementary spillway, the upstream face of the dam and the existing spillway are represented.



a) b)

Figure 7. Hydraulic model: a) Downstream view, b) Upstream view.

Both spillway structures were moulded in cement and sand mortar. The border areas in contact with the flow were coated with cement paste, which simulates the roughness of the concrete surfaces of the prototype. The top of the tunnel was built with transparent acrylic material ("Perspex") to enable the visualization of the flow and measuring water levels achieved therein. Alternative shapes were moulded using gypsum and wood pieces. After the trials, the proposed shapes were reconstructed with cement mortar in order to withstand more prolonged use in tests without deterioration. In the downstream reach of the model, where scour of the river bed can occur, mobile material consisted of natural gravel was used, which simulates approximately the expected probable fracturing of existing rocks.

The water intake is integrated in the experimental pavilion network, where a 500 mm diameter pipe has regulation valves and electromagnetic flowmeters. The water flows to the model through a tranquilization system. The water levels upstream and downstream from the dam were controlled by two hydrometers, one upstream, inside the reservoir and another downstream, close to the boundary of the model. Water levels were measured in the reservoir model at a section located 100 m upstream from the spillway crest. For the tested flow rates, the water levels downstream of the dam were reproduced in the model using a plane gate located in the downstream area of the model. The reproduced flow levels were based on the natural rating curves.

4.2 *Objectives and tested conditions*

The main objectives of this study on a physical model were the analysis of the complementary spillway design shapes, the hydraulic performance and definition of alternative shapes, if necessary, in order to have more favourable technical solutions.

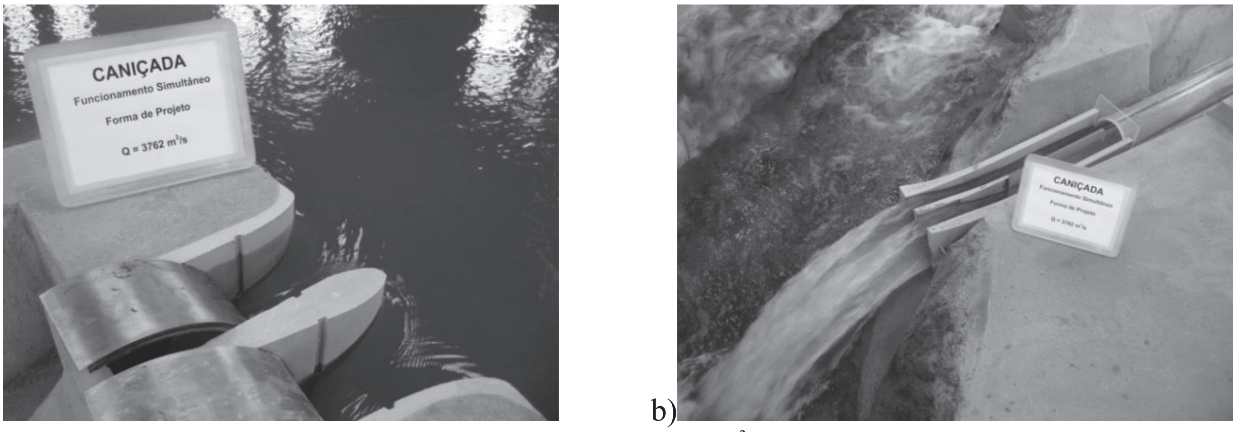
Therefore, the use of the physical model included: i) Analysis of the general flow conditions in the approach zone, on the control structure, inside the tunnel, over the ski jump structure and in the outlet zone; ii) Determination of the spillway stage-discharge curves for different openings of the gates; and iii) Analysis of the simultaneous operation of the two spillways, in particular regarding the approach conditions and possible influence of the new spillway release with the existing spillway releases damped by the downstream weir.

To achieve these objectives, tests with different discharges were performed, namely design and check flood discharge in the complementary spillway (2062 and 2197 m^3/s) or simultaneous operation of both spillways (3762 and 4012 m^3/s). Tests were also run with 1055 m^3/s over the complementary spillway through one of the gates, simulating the case of one gate being out of service.

4.3 *Preliminary design structures performance*

The first tests on the hydraulic model, with the preliminary design shapes, allowed the conclusion that the flow conditions were generally acceptable. Nevertheless, some efforts were done to improve these conditions as much as possible.

The results of the test with the design shapes, near the control structure, running the design flow for both spillways operation, are represented in Figure 8a. Flow contractions were observed near the wing walls and the central pier. Additionally, turbulence uplift occurred inside the tunnel caused by the contractions upstream. Figure 8b presents a view of the release over the ski jump structure with the same test conditions. This structure's shape created two very compact jets that would cause excessive scour on the river bed at the impact zone and, to some considerable extend, impact on the right bank.



a) b)

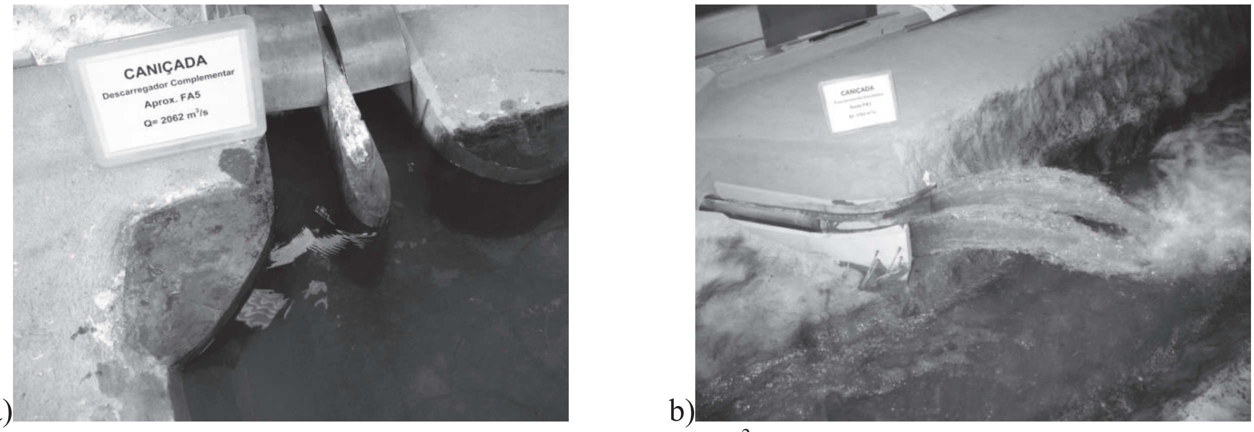
Figure 8. Design shapes and discharge, simultaneous operation (3762 m^3/s): a) control structure, b) ski jump structure

The jets impact, with the design discharge, obtained from formulation of Martins (1977), is 77 m. The impact extension measured in the test showed in Figure 8b) was 82 m.

The maximum river bed scour measured in the tests was 12 m, after letting the design discharge flow during the equivalent time of eight hours pick flood. Calculations of this scour depth using the criteria of Martins (1977) pointed out the result of 14 m.

4.4 *Modifications to the spillway design shapes*

The mitigation of the flow contractions initially observed near the control structure was accomplished by testing six modifications. Figure 9a represents some of the alternative shapes tested, namely the extension and reshaping of the wing walls and the upstream extension of the central pier. The reduction of the contractions is clear in the figure and the consequent reduction of the turbulence uplift in the tunnel was achieved



a) b)

Figure 9. Alternative shapes and design discharge (2062 m^3/s): a) control structure, b) ski jump structure

Both bank erosion and riverbed scour could be minimized through series of testing eight alternative shapes for the ski jump structure that orientated the two jets to a more convenient impact zone. In Figure 9b, one example of these tests is presented, where improvement in the jets dispersion and impact zone can be visualized.

4.5 *Proposed structures performance*

The main modifications in the control structure and approaching area, within the final proposed shape, included: a reduction in the excavation slope in the left bank; redefinition of the hydraulic shape of the left and right wing walls; extension of the central pier; maintenance of the cofferdam abutment in the left bank slope.

Consideration of part of the cofferdam permanently integrated in bank and the reduction in the excavation slope was decided upon economic and constructive constraints. The tests showed that this solution will not affect the flow conditions. The study also considered the final shape of the left bank in the approach area, which resulted in a reduction of excavation volumes, with all its advantages in terms of amount of works and costs.

The proposed overall shapes allow fewer contractions near the control structure. Due to the reduction of the contractions, the flow uplift inside the tunnel was extinguished. In Figure 10a the results of the test with the design discharge are included, where a significant improvement in terms of visualized contraction from Figure 8a can be highlighted.

Regarding the flow inside the tunnel, tests revealed that the free surface never reaches the design criteria of 75% of free height of the tunnel. This is a highly recommended safety concern, bearing in mind the spillway solution through a tunnel.

The proposed changes for the ski jump structure include mainly: a variation in extension of the two channels; extension of the left discharge channel; inclusion of a wedge in the right wall of the right channel; reduction of the extension of the right channel; inclusion of a wedge in the right wall of the left channel; consequent slight uplift of the lateral walls.

The jet obtained for the design discharge is illustrated in Figure 10b. This causes shallower holes and the effect of its dispersion is evident. The jets impact, with the design discharge, extends from 44 to 62 m for the right jet and from 43 to 74 m for the left one. The maximum river bed scour measured in the tests, after letting the design discharge flow during the equivalent time of eight hours pick flood, was reduced from 12 m, obtained with the design shapes, to 8 m, achieved with the changes in the ski jump structures. This was partly a result of having two different zones of impact after the variation in extension of the two channels ending section. Measurements also pointed out that the scour extension can have a maximum length of 75 m.

a) b)

Figure 10. Proposed shapes and design discharge, simultaneous operation (3762 m^3/s): a) control structure, b) ski jump structure.

The stage-discharge curves obtained theoretically in the design step were confirmed in the model tests. Additionally, it was concluded that no significant changes in the stage-discharge curves could be measured when running the several alternative shapes in the control structure. In Figure 11, the stage-discharge curves for the simultaneous operation of the two spillways and for the isolated operation of the complementary spillway are presented. The values obtained in the model are very similar to the calculated values. The most significant difference is in the high head of the simultaneously operation curve. This difference may be due to the fact that the flow around the gates in the existing spillway show some disturbances in the tests.

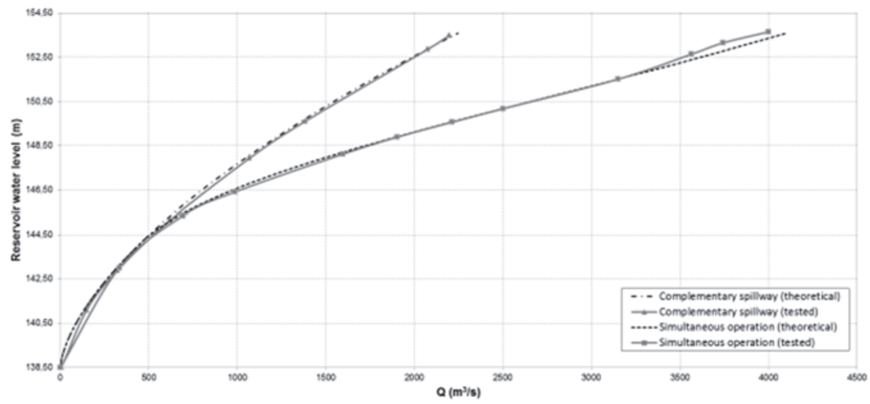


Figure 11. Stage-discharge curves for the isolate operation of the complementary spillway and for the simultaneous operation.

Several tests with combinations of different gate openings in the new spillway were performed. It was concluded that the successive symmetric opening of both gates is more favourable. According to the tests, if one of the gates is eventually not functioning, no critical situation was detected and the operation of the working gate can proceed until the necessary opening.

Additionally, flow depths were measured inside the tunnel in ten cross-sections for five different gate openings. The results were used to calibrate a numerical model, namely the commercial computational fluid dynamics “FLOW-3D”, used to simulate the flow characteristics along the new spillway. Measurements are detailed in Muralha et al. 2014.

5 CONSTRUCTION PHASE

The minimization of restrictions to normal reservoir operation was a fundamental factor to take into account during the construction of Caniçada complementary spillway.

In this case, due to the conditioning imposed by the important touristic activity along the reservoir, the minimum operation levels during dry season are usually kept above elevation (144.00). Under these circumstances, the cofferdam planned to protect the worksite around the spillway intake was designed to be constructed and demolished without the need to empty the reservoir below that level.

The adopted solution comprises a concrete gravity wall, with a maximum height of 7 m and the bottom at elevation (146.50), with ground foundation treated mostly by a double curtain of jet grouting piles, 1 m diameter and maximum depth of about 30 m.

The construction of the new complementary spillway started in January 2014 and will be finished in September 2016.

Figure 12 illustrates the main aspects of such works (February 2015), including the construction of the cofferdam, excavations for the tunnel and an overall view of the working area around the dam and existing spillway.

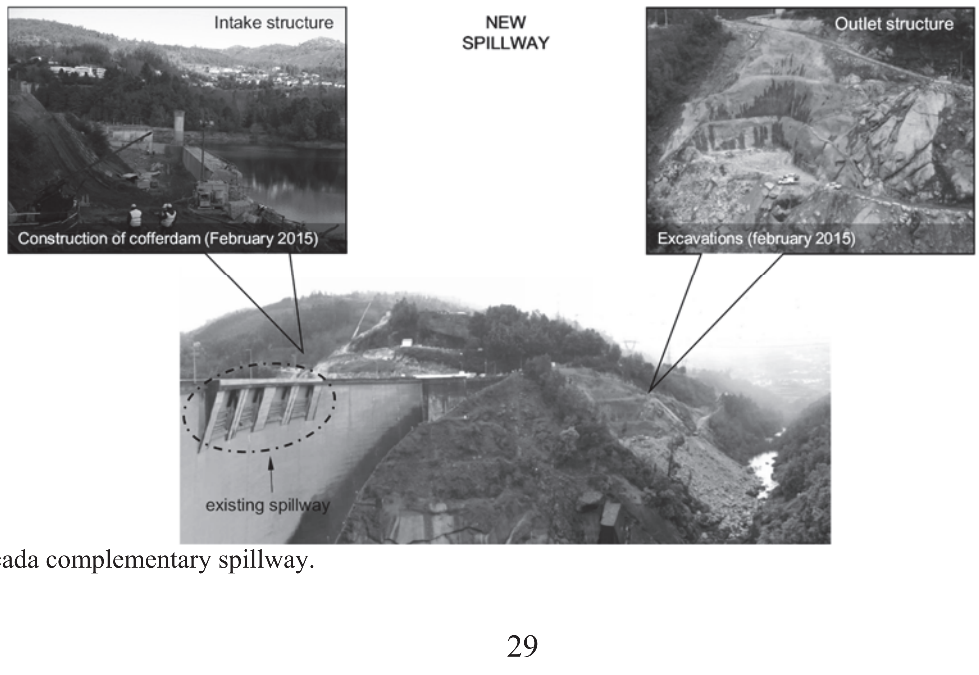


Figure 12. Caniçada complementary spillway.

6 CONCLUSIONS

This paper presents the design, hydraulic tests and ongoing works of the complementary spillway of Caniçada dam.

After the safety analysis carried out by EDP, it was concluded that an additional discharge structure would be necessary, namely the complementary spillway for Caniçada dam. This new structure, designed by AQUALOGUS, is a gated spillway, controlled by an ogee crest, followed by a tunnel, designed for free surface flow, and a ski jump which directs the jet into the river bed.

The hydraulic performance of this new spillway was studied in a physical model built at LNEC with a 1/62 scale factor (Figure 7).

Based on the model testing facilities, some modifications of the preliminary design were considered, namely in the control structure wing walls shape, on the left bank shape and volume of digging upstream the intake structure, incorporation of the cofferdam abutment and central pier extension to upstream. This model allowed particularly testing several alternative shapes or the detailed definition of the ski jump structure, namely a variation in extension of the two channels, minimizing the impact on the downstream banks and the scour on the riverbed. The study included several scenarios with various gate openings of the complementary spillway.

The construction of the new complementary spillway started in January 2014 and will be finished in September 2016.

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