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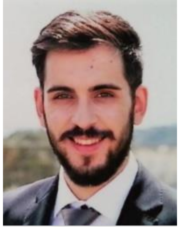
NON-LINEAR SEISMIC ANALYSIS OF ARCH DAMS CONSIDERING JOINT MOVEMENTS AND A CONCRETE DAMAGE MODEL



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ABSTRACT

This paper presents a study on the non-linear seismic response of Cabril arch dam (Portugal), for a load combination including the dam's self-weight, the hydrostatic pressure (full reservoir) and the seismic loading (10 s generated accelerograms) applied at the dam base. The main results are presented considering two accelerograms, with peak ground accelerations of 0.2g and 0.6g. The investigation is focused on the effects of the non-linear behaviour of joints on the structural response of an arch dam under stronger earthquakes and on the resulting concrete damage distributions.

The numerical simulations are performed using *DamDySSA4.0*, a 3D finite element program developed in LNEC for dynamic analysis of concrete dams, including a recently developed module for non-linear seismic analysis. The dynamic behaviour of the dam-reservoir-foundation system is simulated using a coupled model in displacements and pressures, considering the dam-water interaction and non-proportional damping, and using a sub-structuring technique to simulate the foundation. The non-linear seismic response is computed using a time-stepping algorithm, based on the Newmark method, and the stress-transfer method. The non-linear behaviour of concrete is considered using an isotropic damage model with softening and two independent damage variables (d^+ for tension and d^- for compression), and the opening/closing/sliding movements of joints are simulated using joint finite elements and the classic Mohr-Coulomb law, for vertical contraction joints, dam-foundation interface and existing cracks. The aim is to summarise the implemented formulations and show the potential of *DamDySSA4.0* for predicting the non-linear seismic behaviour of concrete dams and to support seismic safety verifications.

Keywords: Non-linear seismic analysis, Arch dam, Concrete damage model, Non-linear joints, Dam-reservoir-foundation dynamic interaction

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1. INTRODUCTION

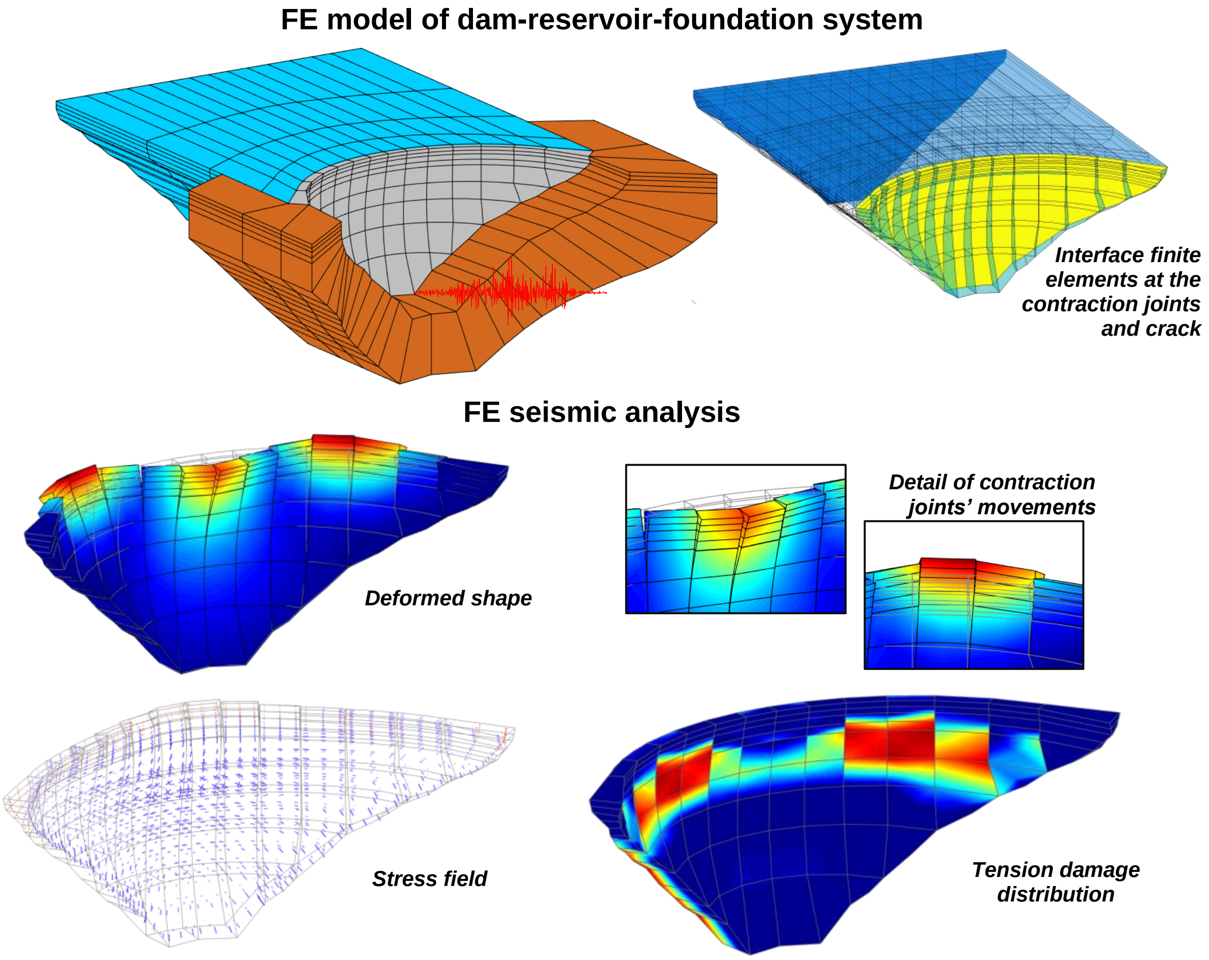
Large concrete dams play an increasingly important role in the management of water resources due to effects of climate change, by contributing decisively to water supply, energy production, irrigation and flood control. As civil engineering structures with high potential risk [1], according to the International Commission on Large Dams (ICOLD) it is fundamental to ensure the safety of dams for hazards from the natural and man-made environment [2]. Therefore, the seismic safety of large concrete dams is a fundamental research area on dam engineering, particularly for dams located in seismic areas, aiming to evaluate their design and performance under strong earthquakes. In this scope, it is essential to invest (i) in sophisticated finite element (FE) programs and models for numerical analysis and (ii) in systems for monitoring of the dynamic behaviour of dams under ambient/operational vibrations and during seismic events [3,4].

The numerical modelling of the dynamic behaviour of concrete dams is a research field that has been widely studied, leading to the development of various formulations and models to simulate the dynamic response of dam-reservoir-foundation systems based on the Finite Element Method (FEM) [5], considering linear and non-linear behaviour (Fig. 1). Nevertheless, this a complex field where important challenges are still presented.

For simulating the reservoir and dynamic dam-water interaction, there are the classic added water mass models, formulated in displacements, using Westergaard’s solution to compute the reservoir mass based on the water pressures [6] and considering proportional Rayleigh damping (this approach requires the use of an added mass reduction factor for dynamic analysis [7]). Alternatively, there are the coupled models, formulated in displacements and pressures [8] or velocity potentials [9], which consider dynamic dam-reservoir motion coupling and the propagation of pressure waves in water (with radiation damping). As for the foundation and seismic input modelling, the simplest models are the ones based on the massless approach [10], considering only the foundation flexibility and uniform seismic input at the foundation boundaries. Then, the models based on the substructure method [11] simulate the foundation as an elastic, massless substructure, considering stiffness and damping components at the dam-rock interface and uniform seismic inputs at the dam base. In alternative, there are energy dissipating models, taking into account the dam-foundation interaction and radiation damping in the rock mass [12], e.g. using viscous-spring boundaries [13], while the seismic input is modelled by imposing the free-field ground motions directly to the dam-rock interface [14] or by compression and shear waves vertically propagation from the foundation base [15].

The seismic response analysis of arch dams can be carried out considering linear-elastic behaviour, using linear constitutive models for concrete and assuming closed joints. This is a valid approach for earthquakes that induce vibrations with lower amplitudes, more commonly measured on site using continuous or trigger-event monitoring systems. Nevertheless, strong earthquakes might cause the opening of contraction joints [16], resulting in a significant release of stresses in the arch direction and an increase in the cantilever stresses, and the damage cracking in concrete under tension [17], thus requiring more complex non-linear models. In what concerns concrete behaviour, constitutive damage models with softening can be used, considering damage under tension and compression [18,19], while the opening/closing/sliding movements of joints can be simulated using joint elements [20] and appropriate models.

Aiming to contribute for knowledge in this field, a study on the non-linear seismic response of Cabril arch dam is presented in this paper, considering two seismic acelerogramas with peak ground accelerations of 0.2g and 0.6g, in order to investigate the effects of vertical contraction joints in the structural response of the dam under stronger earthquakes and on the resulting concrete damage distributions in concrete. The numerical simulations are carried out using *DamDySSA4.0*, a 3D finite element program developed in LNEC for dynamic analysis of concrete dams.


FE model of dam-reservoir-foundation system

FE seismic analysis

Fig. 1 – FE models of dam-reservoir-foundation systems and results of FE seismic analysis.

2. CASE STUDY: CABRIL DAM

Cabril dam, located on the Zêzere river, has been in operation since 1954 and is the highest dam in Portugal (Fig. 2). It is a 132 m high double curvature arch dam, founded on granite mass rock foundation of good quality. The crest is at el. 297 m and its arch is 290 m long. The maximum width is of 20 m at the base, in the central cantilever, and the minimum is of 4.5 m at el. 290 m, 7 m below the crest (which has a greater width).

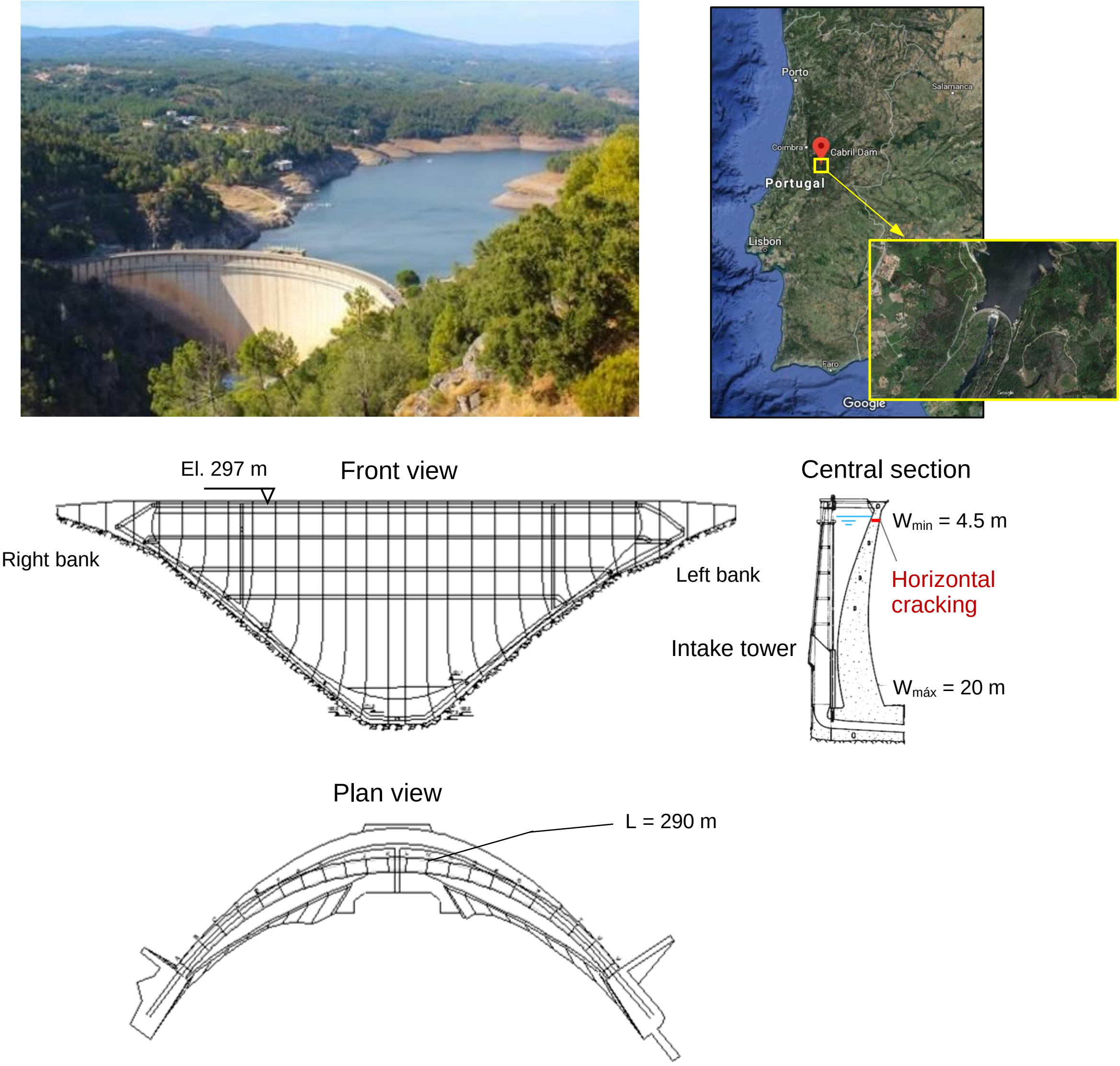


Fig. 2 – Cabril arch dam (132 m high). Upstream view, central cantilever and top view.

Regarding the structural health, horizontal cracking was detected at the upper part of the dam, around el. 280-290 m, during the first filling of the reservoir, and concrete internal swelling was detected in the late 90's. Nevertheless, recent studies based on monitoring data and FE results have shown that the dam's behaviour is normal [4]. The water level usually ranges from about el. 265 m to el. 295 m. A reinforced concrete intake tower, with the same height of the dam, was built near the upstream face, and it is connected to the central cantilever at the crest level via a concrete walkway, with a joint in the contact surface.

Cabril dam has been under continuous dynamic monitoring since 2008, using a pioneer Seismic and Structural Health Monitoring system in Portugal [3]. Specific software has been developed in LNEC for analysing continuous monitoring data and for FE numerical analysis, which have allowed to conduct several studies on the dynamic behaviour of Cabril dam under ambient/operational vibrations and during seismic events, as well as to calibrate, validate and improve the developed numerical models.

3. DamDySSA4.0. FORMULATION FOR NON-LINEAR SEISMIC ANALYSIS

The numerical simulations were carried out using *DamDySSA4.0*, a 3D FE program developed in LNEC for dynamic analysis of concrete dams, including modal analysis [21], linear seismic analysis [22] and a recently developed module for non-linear seismic analysis. The developed formulation is summarised as follows.

3.1. Coupled problem and FE formulation

The dynamic behaviour of the dam-reservoir-foundation system (Fig. 3) is described by a coupled problem [5], based on the governing equations for the solid and fluid domains. Specific boundary conditions are established to prescribe null displacements at the dam base and consider the hydrostatic pressures at the upstream face of the dam, as well as to simulate the dynamic dam-reservoir interaction, the propagation of pressure waves in water and the free surface condition [8].

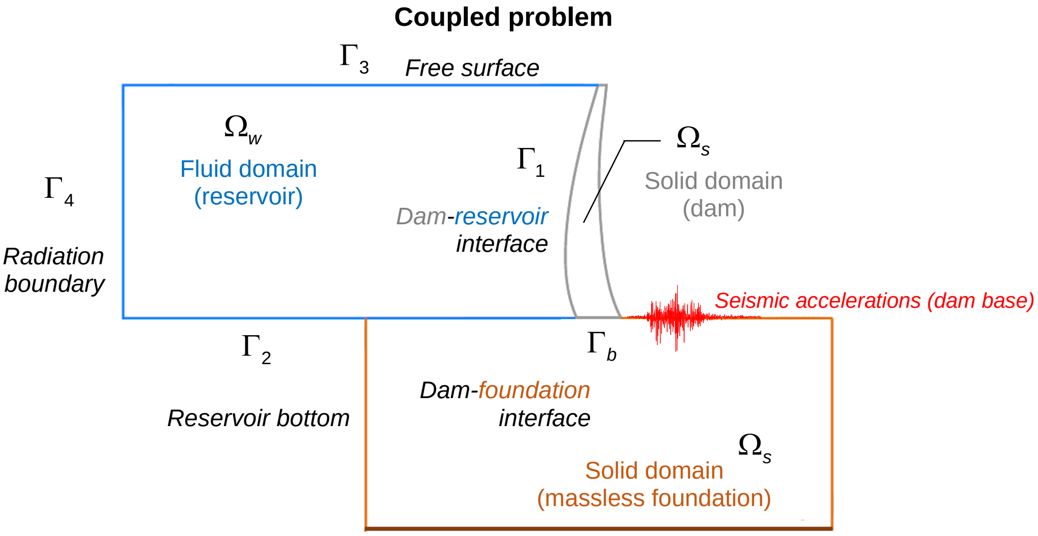


Fig. 3 – DamDySSA. Coupled problem and main interfaces.

To solve the dynamic coupled problem, the dam-reservoir-foundation system is discretized considering displacement FE for the dam and foundation and pressure FE for the reservoir, using 3D isoparametric cubic elements with 20 nodes, and compatible joint elements with 16 nodes for vertical contraction joints, the dam-base surface and existing cracks (Fig. 4).

Therefore, using a FE formulation in displacements and pressures, considering the boundary conditions, the dynamic equation for the discrete coupled problem is obtained,

$$\begin{bmatrix} \underline{m} & \underline{0} \\ \rho_w \underline{Q}^T & \underline{S} \end{bmatrix} \begin{bmatrix} \ddot{\underset{\sim}{u}} \\ \ddot{\underset{\sim}{p}} \end{bmatrix} + \begin{bmatrix} \underline{c} & \underline{0} \\ \underline{0} & \underline{R} \end{bmatrix} \begin{bmatrix} \dot{\underset{\sim}{u}} \\ \dot{\underset{\sim}{p}} \end{bmatrix} + \begin{bmatrix} \underline{k} & -\underline{Q} \\ \underline{0} & \underline{H} \end{bmatrix} \begin{bmatrix} \underset{\sim}{u} \\ \underset{\sim}{p} \end{bmatrix} = \begin{bmatrix} \underset{\sim}{F}_s \\ \underset{\sim}{F}_w \end{bmatrix} \tag{1}$$

where $\underline{m}$, $\underline{c}$ and $\underline{k}$ are the mass, damping and stiffness matrices for the solid domain, $\underline{S}$, $\underline{R}$ and $\underline{H}$ indicate the corresponding terms for the reservoir domain, and $\underline{Q}$ is the dam-reservoir coupling matrix. Rayleigh damping is calculated element by element for the solid domain, while radiation damping due to propagation of pressures waves in water is simulated for the reservoir. The stiffness and damping components of the existing joints elements are incorporated into the corresponding global matrices. A sub-structuring technique [11] is used to simulate the foundation block as an elastic, massless substructure. With this method, the dynamic analysis is performed only for the dam-reservoir system.

For seismic analysis, the nodal force vectors are $\underset{\sim}{F}_s = \underset{\sim}{F}_s(t) = -\underline{m}\ \underline{s}\ \underset{\sim}{a}_S(t)$ for the dam and $\underset{\sim}{F}_w = \underset{\sim}{F}_w(t) = -\rho_w\ \underline{Q}^T\ \underline{s}\ \underset{\sim}{a}_S(t)$ for the reservoir, where $\underset{\sim}{a}_S(t)$ is the vector with the seismic acceleration time histories, applied uniformly at the dam base, and $\underline{s}$ is a distribution matrix to obtain the nodal forces in the upstream-downstream, cross-valley and vertical directions.

The above equation (1) can be written in a simpler format as

$$\underline{M}\,\ddot{\underset{\sim}{q}} + \underline{C}\,\dot{\underset{\sim}{q}} + \underline{K}\,\underset{\sim}{q} = \underset{\sim}{F} \tag{2}$$

where $\underline{M}$, $\underline{C}$ and $\underline{K}$ are the global mass, damping and stiffness matrices, and

$$\underset{\sim}{q} = \underset{\sim}{q}(t) = \begin{bmatrix} \underset{\sim}{u} \\ \underset{\sim}{p} \end{bmatrix} \tag{3}$$

is the global unknowns vector, which includes the problem's unknowns: the displacement vector $\underset{\sim}{u} = \underset{\sim}{u}(t)$ and the hydrodynamic pressures vector $\underset{\sim}{p} = \underset{\sim}{p}(t)$.

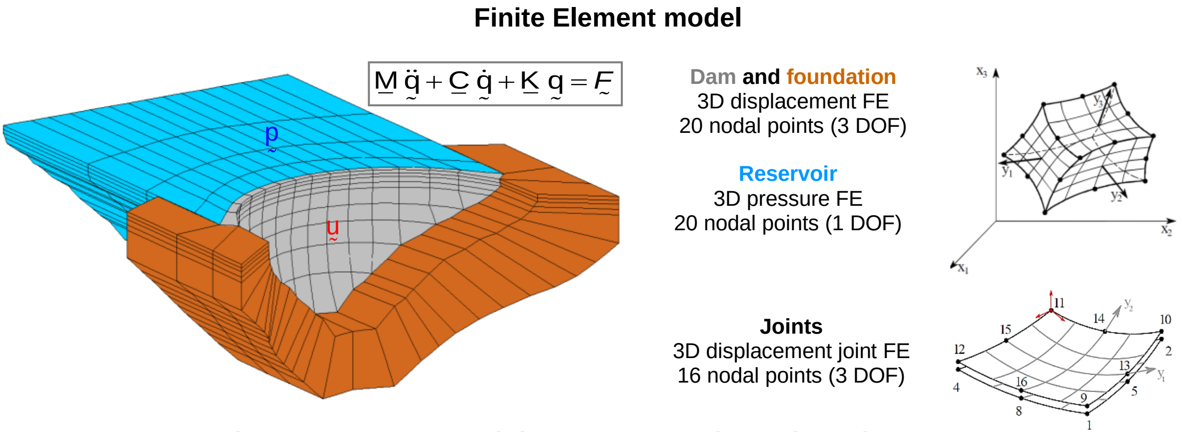


Fig. 4 – *DamDySSA*. Finite element model and used elements.

3.2. Seismic analysis: time-stepping procedure

The dynamic response of the coupled system under seismic loading is calculated using a new time-stepping procedure for numerical integration [21], based on the Newmark method [22]. The aim is to solve the coupled dynamic equation at time t+Δt,

$$\underline{M}\,\ddot{\underset{\sim}{q}}_{t+\Delta t} + \underline{C}\,\dot{\underset{\sim}{q}}_{t+\Delta t} + \underline{K}\,\underset{\sim}{q}_{t+\Delta t} = \underset{\sim}{F}_{t+\Delta t} \tag{4}$$

The relations between the coupled unknown and the respective time derivatives at t+Δt are obtained from Taylor series expansions, assuming a linear variation for the second time derivatives within each time step Δt and using Newmark's parameters β and γ, resulting in the following expressions

$$\underset{\sim}{q}_{t+\Delta t} = \underset{\sim}{q}_t + \Delta t \cdot \dot{\underset{\sim}{q}}_t + \Delta t^2 \cdot (\frac{1}{2} - \beta) \cdot \ddot{\underset{\sim}{q}}_t + \Delta t^2 \cdot \beta \cdot \ddot{\underset{\sim}{q}}_{t+\Delta t} \tag{5}$$

$$\dot{\underset{\sim}{q}}_{t+\Delta t} = \dot{\underset{\sim}{q}}_t + \Delta t \cdot (1-\gamma) \cdot \ddot{\underset{\sim}{q}}_t + \Delta t \cdot \gamma \cdot \ddot{\underset{\sim}{q}}_{t+\Delta t}$$

Therefore, this dynamic problem is written terms of the coupled unknown at t+Δt, using the equivalent matrix $\underline{A}^*$ with the internal forces,

$$\underline{A}^* \cdot \underset{\sim}{q}_{t+\Delta t} = \underset{\sim}{F}^*_{t+\Delta t} \tag{6}$$

which is solved at each time step to compute the coupled system's response in time domain.

3.3. Non-linear seismic analysis

For seismic analysis under strong earthquakes, the structural response might be influenced by the non-linear behaviour of concrete in the dam body (tension and/or compression damage) and by the opening/closing/sliding movements of vertical contractions joints or existing cracks.

The non-linear dynamic problem is solved by combining the previous time-stepping procedure with an iterative stress-transfer method, using the dam's damping and stiffness matrices corresponding to the linear elastic state. The goal is to simulate the non-linear response, considering the redistribution of unbalanced stresses arising due to concrete deterioration and/or movements of joints, by solving the coupled dynamic equation at time t+Δt

$$\underline{M}\,\ddot{\underset{\sim}{q}}_{t+\Delta t} + \underline{C}\,\dot{\underset{\sim}{q}}_{t+\Delta t} + \underline{K}\,\underset{\sim}{q}_{t+\Delta t} = \underset{\sim}{F}_{t+\Delta t} + \underset{\sim}{\Psi}_{t+\Delta t} \tag{7}$$

or, in the equivalent form

$$\underline{A}^* \cdot \underset{\sim}{q}_{t+\Delta t} = \underset{\sim}{F}^*_{t+\Delta t} + \underset{\sim}{\Psi}_{t+\Delta t} \tag{8}$$

where $\underset{\sim}{\Psi}_{t+\Delta t}$ represents the vector of nodal forces equivalent to the unbalanced stresses, which is recalculated in every iteration.

In each time-step t+Δt, the convergence of the non-linear calculation is verified at the end of every iteration, by analysing the unbalanced forces or the corresponding displacements, which must be smaller than a certain tolerance value. If the structure has enough resistant capacity and can redistribute the unbalanced stresses, the process converges; otherwise, global equilibrium is not achieved, and the process diverges (structural collapse may occur).

3.4. Concrete and joints non-linear behaviour

3.4.1. Damage model for concrete

The non-linear behaviour of concrete up to failure is simulated using a constitutive damage model with softening and two independent damage variables [18], namely d^+ for tension damage and d^- for compression damage. This is a computationally efficient model, which considers specific damage criteria, to limit the linear elastic behaviour, and damage evolution laws for d^+ and d^-, in order to account for the progression and the irreversible nature of concrete deterioration. Damage occurs when the material's strength is exceeded (hence unbalanced stresses arise). Also, the damage law allows to account for the concrete resistance under compressive stresses inside a certain region after tension damage occurs.

The following constitutive law is used to calculate the real stress tensor $\underset{\sim}{\sigma}$ from the corresponding effective stresses $\underset{\sim}{\tilde{\sigma}}$, at each Gauss integration point of all elements of the dam.

$$\underset{\sim}{\sigma} = (1-d^+)\underset{\sim}{\tilde{\sigma}}^+ + (1-d^-)\underset{\sim}{\tilde{\sigma}}^- \tag{9}$$

The computed stress-strain diagrams for uniaxial tension and compression are shown in Fig. 5, where E_0 is Young's modulus for the undamaged state, f_0^+. and f_0^- indicate the maximum admissible tension and compression for linear elastic behaviour and f_c represents the peak compressive stress.

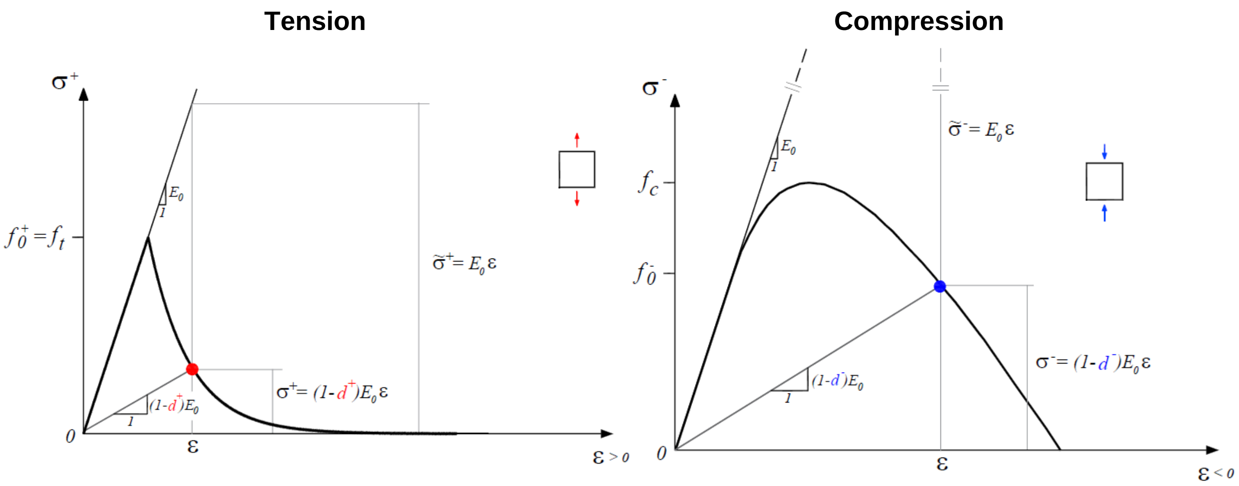

Tension **Compression**

Fig. 5 – DamDySSA. Damage model for concrete: uniaxial stress-strain diagrams.

3.4.2 Mohr-Coulomb model for joints

The non-linear behaviour of joints, including vertical contraction joints, dam-rock surface or existing cracks, is simulated using joint elements and a model based on the Mohr-Coulomb failure criterion, assuming the joints are closed under compression and tend to open under tension. Therefore, this model considers the material's elastic properties, namely the normal K_N and shear K_T stiffnesses, and the strength properties, which are the cohesion *c* and the friction angle ϕ.

With this formulation, the normal σ_N and shear stresses τ are calculated for each surface Gauss integration point on both faces of the joint elements and compared with the uniaxial tensile strength f_t and shear strength τ_R, in order to verify if the joint's strength is exceeded, which leads to local joint failure, and thus simulate the opening, closing and sliding movements. The uniaxial tensile strength f_t and the shear strength τ_R, are defined as

$$\tau_R = c + \sigma_N \times \tan(\phi) \tag{10}$$

$$f_t = c \times \frac{2\cos(\phi)}{1+\sin(\phi)}$$

and can be related to the cohesion and friction angle as shown in the Mohr's cycle (Fig. 4).

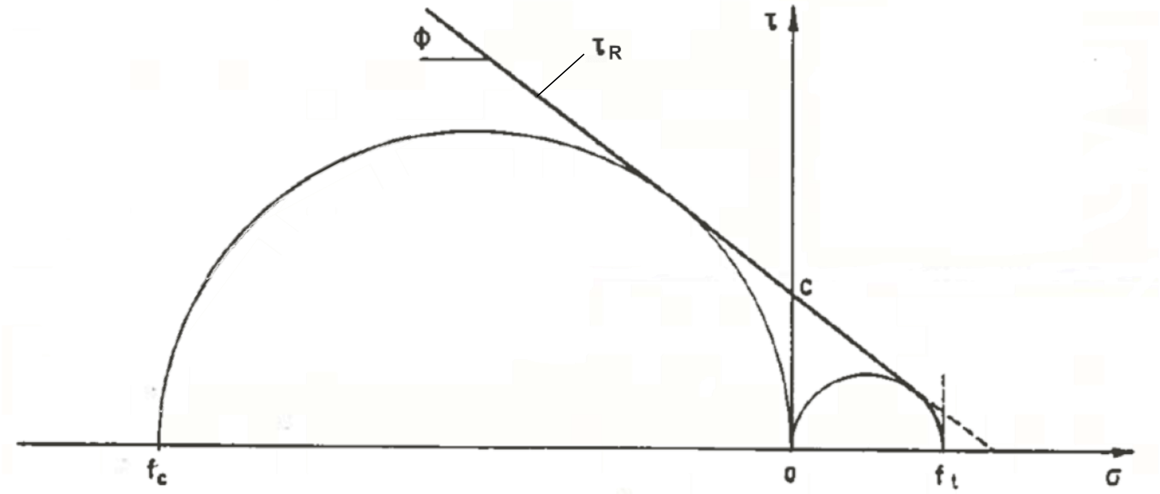


Fig. 6 – DamDySSA. Non-linear joint behaviour. Mohr's circle: strength properties.

4. NON-LINEAR SEISMIC ANALYSIS OF CABRIL DAM

The seismic response of Cabril dam is analysed for a load combination including the dam's self-weight (SW), the hydrostatic pressure (HP) for full reservoir (el. 297 m) and the seismic loading (SeismicL). The seismic accelerogram, a generated 10 s earthquake (Fig. 7) is applied at the dam base in the upstream-downstream direction and uniformly distributed along the dam-rock interface. The calculations are carried out using two scaled accelerograms, with peak ground accelerations of 0.2g (1,96 m/s^2) and 0.6g (5,89 m/s^2). In order to investigate the influence of the non-linear behaviour of joints in the structural response of an arch dam and on the concrete damage distributions, the results of non-linear seismic analysis are compared with the response calculated using a simple model without joints and concrete damage.

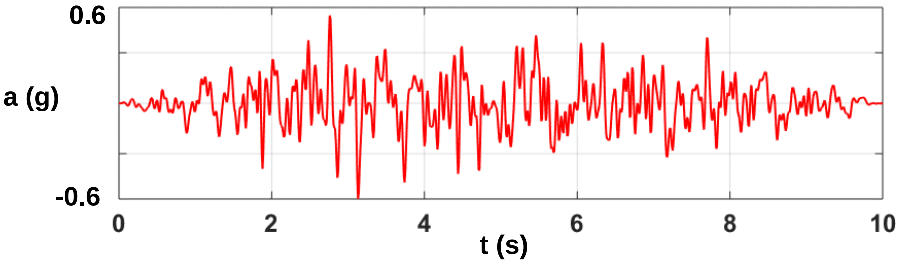
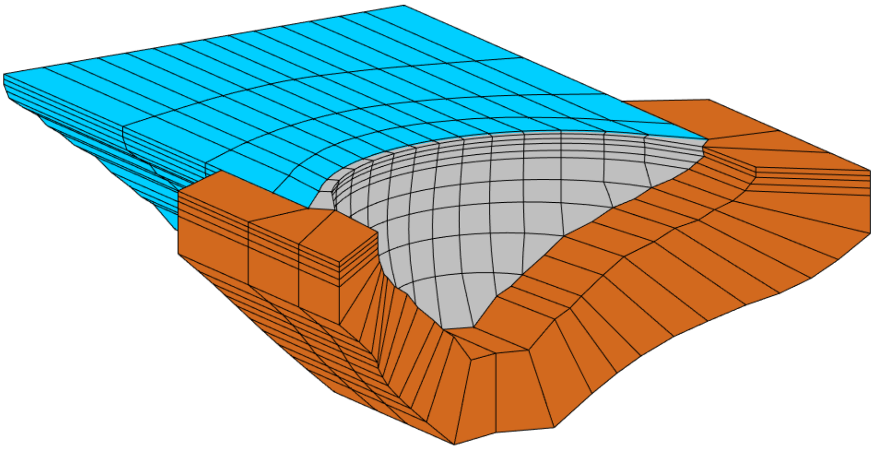


Fig. 7 – Seismic accelerogram (10 s) used for non-linear seismic analysis of Cabril dam.

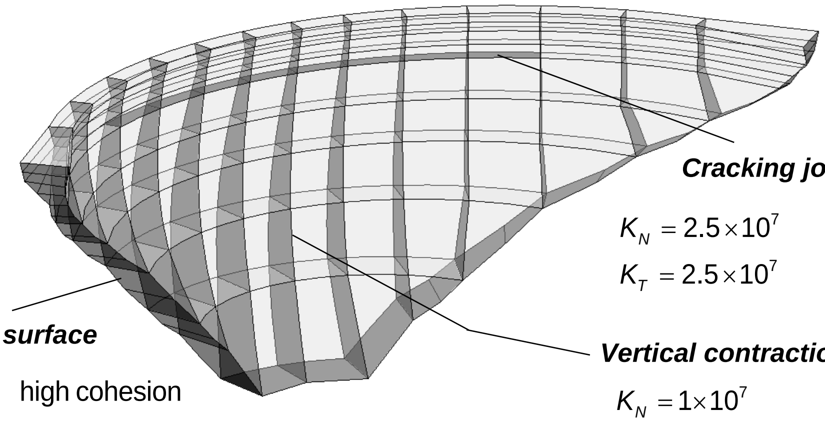
The seismic calculations are carried out with *DamDySSA4.0*, using a 3D FE model of the dam-reservoir-foundation system of Cabril dam (Fig. 8). This model has been previously tested, calibrated and validated based on the comparison of computed natural frequencies and mode shapes with identified modal parameters over time [3,4].

The mesh has 4822 nodal points and 626 elements, including 106 in the dam, 96 in the foundation and 424 in the reservoir. All vertical contraction joints, the joint along the dam-rock interface and the existing cracking (around el. 280 m) are incorporated into the model, using a total of 137 joint elements. The dam concrete and the foundation rock are assumed as isotropic materials, considering Young's modulus E = 25 GPa and Poisson's ratio $v = 0.2$, while the water in the reservoir is an inviscid and compressible fluid, with a velocity c_w = 1440 m/s for pressure waves' propagation. A Rayleigh law with $\alpha = 2$ and $\beta = 0.006$ is used for all dam elements, resulting in a damping ratio of about 10% around the frequency band from 2 to 3 Hz (first vibration modes); in the foundation, a damping matrix proportional to the stiffness matrix is computed, to get around 20% in the same frequencies [4]. The non-linear behaviour of concrete is simulated using a constitutive law with tensile strength f_t = 3 MPa and compressive strength f_c = - 30 MPa. As for the joints, null cohesion is considered for the vertical contraction joints and the cracking joint, to simulate the opening under tensions, while high cohesion is assumed for the joint at the dam-rock interface. The high friction angle values are used to account for the existing shear keys in the contraction joints of Cabril dam.

Mesh of the Cabril dam-reservoir-foundation system



Reservoir	**Dam**	**Foundation**
c_w = 1440 m/s γ = 10 kN/m^3	E = 25 GPa ν = 0.2	E = 25 GPa ν = 0.2
Full reservoir (el. 297 m)	γ = 24 kN/m^3 $\xi \approx$ 10%	(massless) $\xi \approx$ 20%



Cracking joint

$K_N = 2.5 \times 10^7$ $c = 0$

$K_T = 2.5 \times 10^7$ $\phi = 30^o$

Dam-rock surface

$K_N = 2.5 \times 10^8$ high cohesion

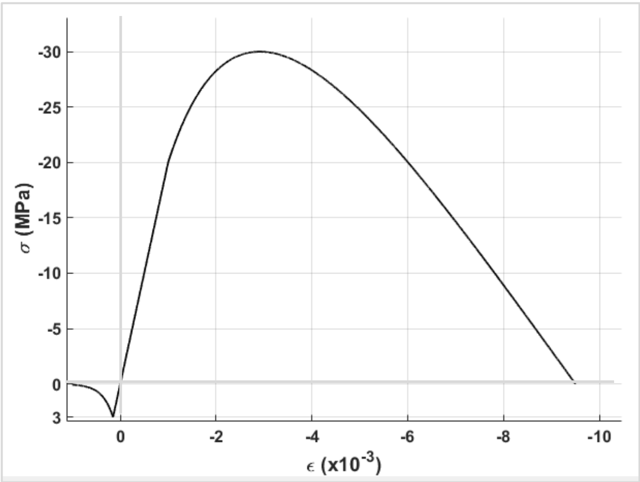
$K_T = 2.5 \times 10^8$ and friction angle

Vertical contraction joints

$K_N = 1 \times 10^7$ $c = 0$

$K_T = 0.5 \times 10^7$ $\phi = 30^o$

Non-linear constitutive law



f_t = 3 MPa

f_c = –30 MPa

Fig. 8 – Cabril dam. 3D model of the dam-reservoir-foundation system and material properties.

4.1. Calculation for *SW+HP297+SeismicL (0.2g)*

First, Cabril dam’s seismic response is analysed for the load combination *SW+HP297+SeismicL*, under the seismic accelerogram scaled for a peak ground acceleration of 0.2g. The results include the deformed shapes and stress fields, for the time instants of the maximum upstream and downstream seismic displacements, as well as the tension damage distributions.

The seismic response calculated with a linear model is presented in Fig. 9. The maximum displacement towards upstream, in relation to the deformed shape for static loads, occurs at the top of the central section and is ~ 15 mm in the downstream direction. The maximum downstream displacement (~ 81 mm) is also computed at the top of the central cantilever. Regarding the stress field, the dam is globally under compressions, with high stresses (- 6.2 MPa) in the arch direction at the top. The higher tensions arise at the upper part of the dam (2.5 MPa) in the vertical direction, around el. 280-285 m, and along the dam base (2.8 MPa), normal to the insertion.

The seismic response considering non-linear behaviour is presented in Figs. 10 and 11. As expected, the dam’s behaviour is influenced by the opening/closing/sliding movements of the vertical contraction joints. In this case, the maximum displacements in the upstream direction (~18 mm) are calculated in the lateral cantilevers, and not in the central section. The maximum downstream displacements (~ 84 mm) occur in the central section, at the crest. In what concerns the stress fields, the maximum compressions (- 8.9 MPa) are calculated at the top of the dam in the arch direction. The joints’ opening releases arch stresses at the top and increases the cantilever stresses near the base, around the central part of the dam, resulting in some areas with low values of tension damage.

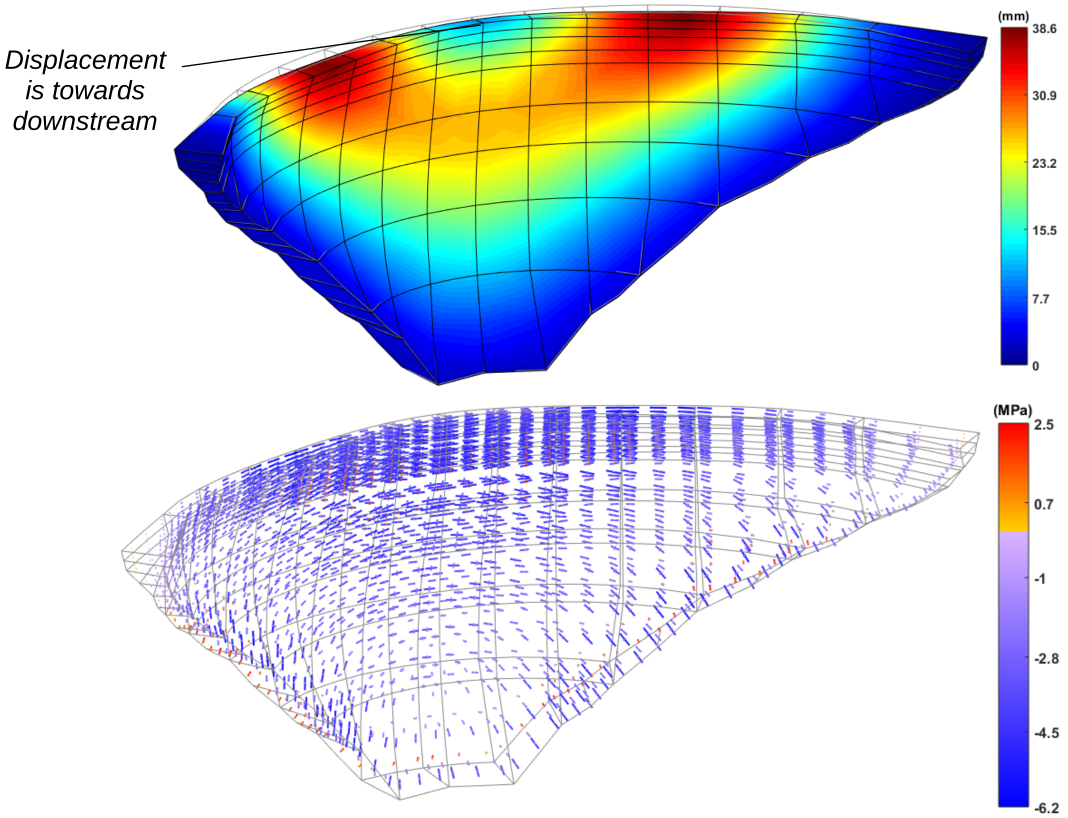
4.2. Calculation for SW + HP297 + SeismicL (0.6g)

Cabril dam’s seismic response is now analysed for the combination *SW+HP297+SeismicL*, considering the 0.6g seismic accelerogram. The same graphical representations are shown.

The seismic response considering linear behaviour is presented in Fig.12. The maximum upstream (~55 mm) and downstream (~156 mm) displacements occur at the top of the central section. Regarding the stress field, the higher compressions are calculated at the top of dam, in the arch direction, with a maximum value of - 18.8 MPa. Relatively to the first calculation with the 0.2g accelerogram, there is an increase in the areas subject to tensions, namely at the top of the dam, at the crest (in both sides of the central section), and in the lateral cantilevers (near the base). The higher tensions are calculated at the crest of the central section (7.9 MPa), in the arch direction, as well as at the crest of the lateral cantilevers (6.4 MPa), also in the arch direction, and near the base, normal to the insertion (6.5 MPa).

Linear seismic response: *SW+ HP297+SeismicL (0.2g)*

Time instant of the maximum ***upstream*** displacement


Displacement is towards downstream

Time instant of the maximum ***downstream*** displacement

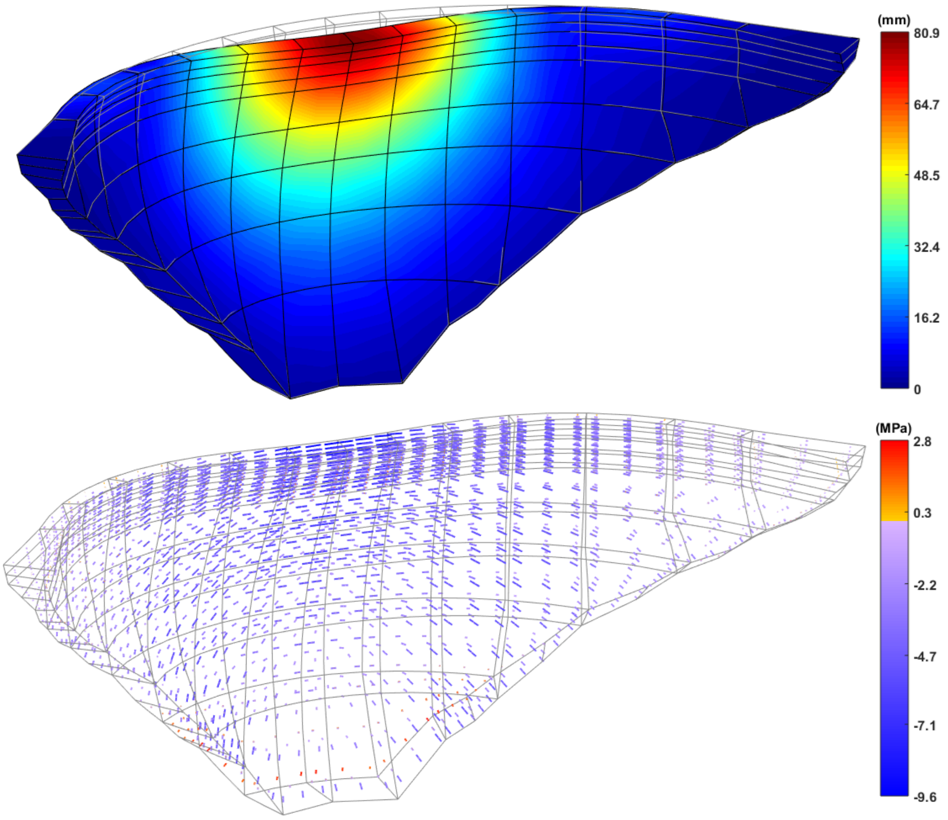
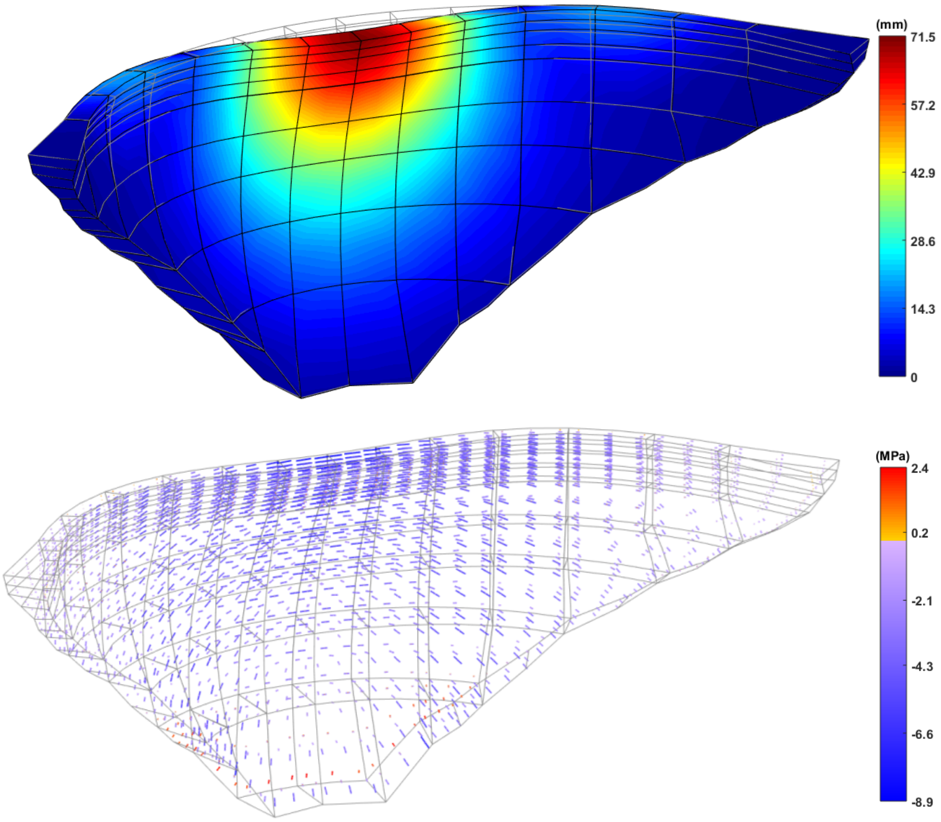


Fig. 9 – Cabril dam. Linear seismic response. Deformed shapes and stress fields.

Non-linear seismic response: *SW+HP297+SeismicL (0.2g)*

Time instant of the maximum *upstream* displacement



Time instant of the maximum *downstream* displacement

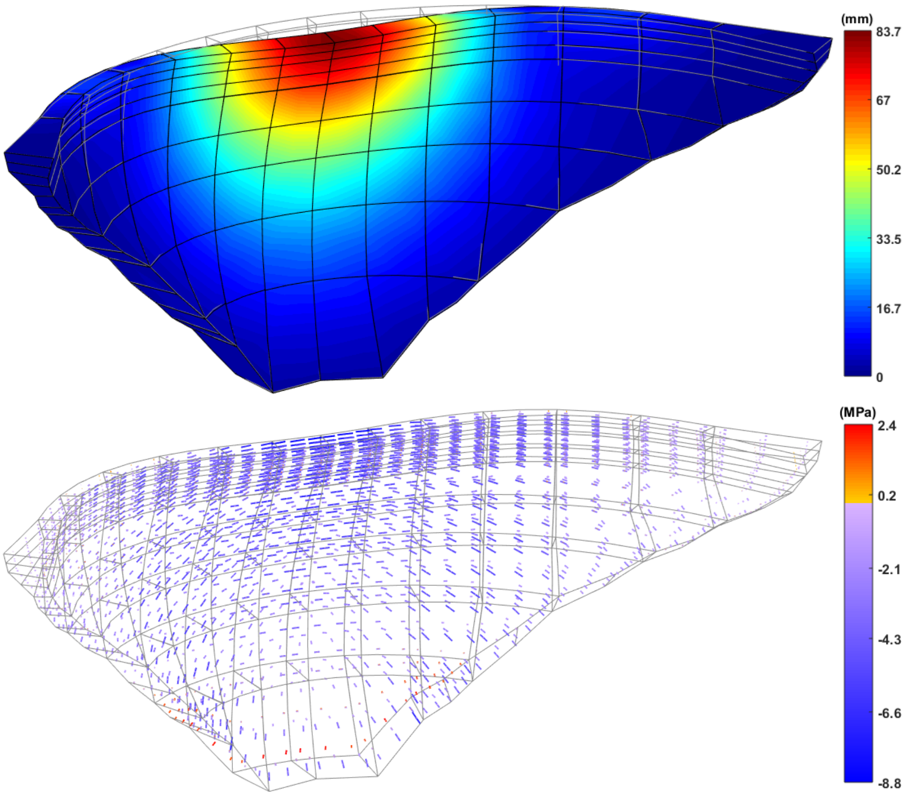


Fig. 10 – Cabril dam. Non-linear seismic response. Deformed shapes and stress fields.

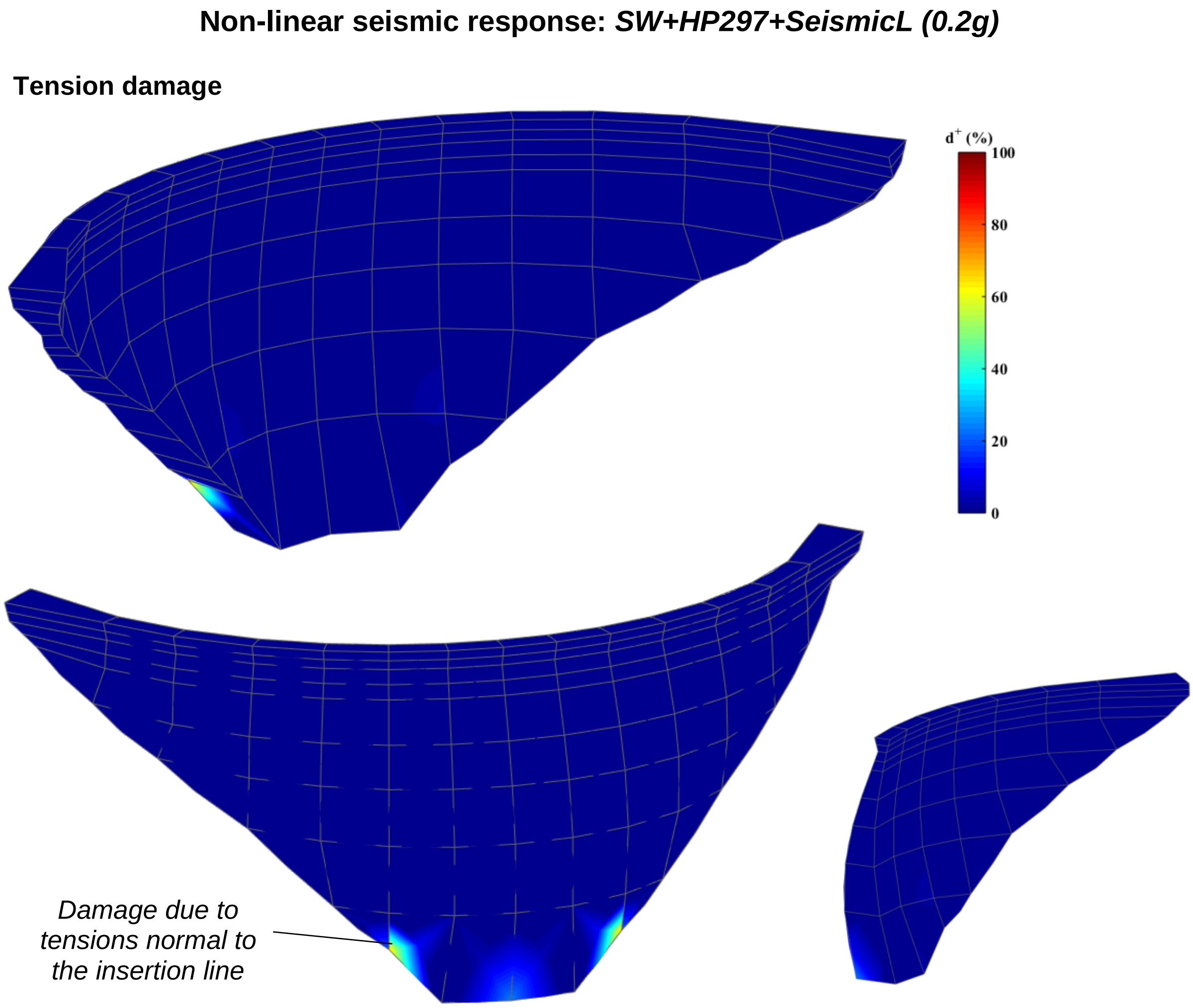
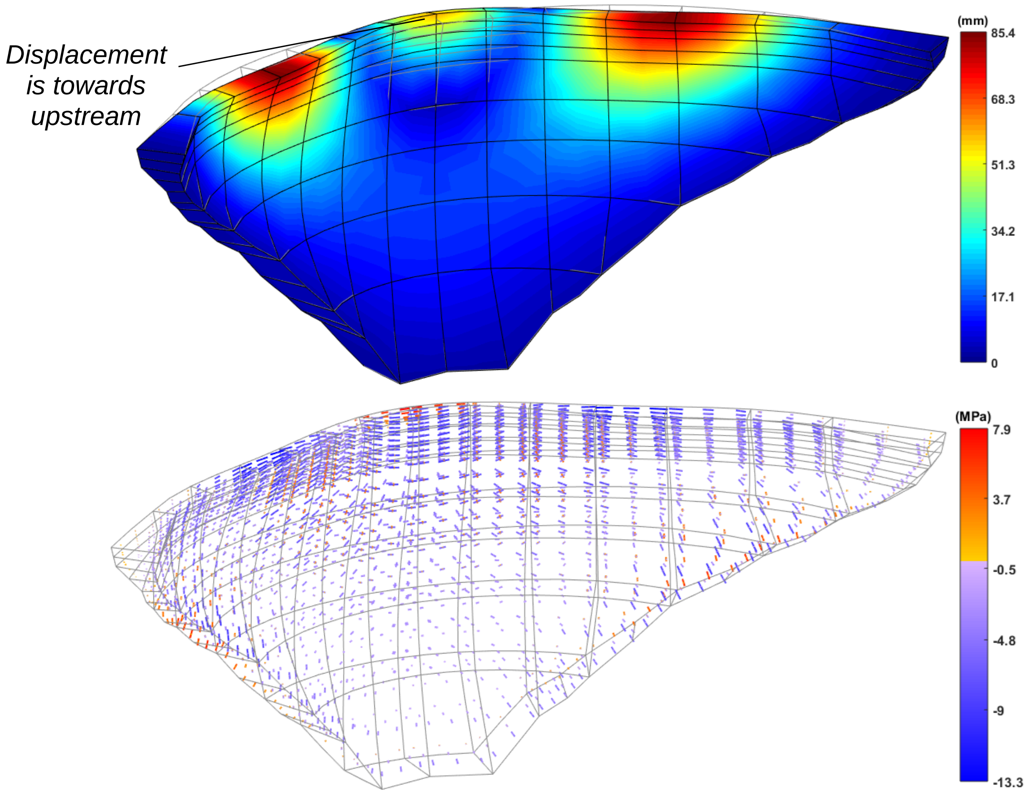


Fig. 11 – Cabril dam. Non-linear seismic response. Tension damage.

The non-linear seismic response, using the model with joints movements and considering non-linear behaviour in concrete, is presented in Figs. 13 and 14. For a stronger earthquake, Cabril dam's seismic behaviour is clearly affected by the opening of the vertical contraction joints. The maximum upstream displacements (~ 95 mm) are calculated at the lateral cantilevers, where the greater joint openings occur. As for the stress fields, the maximum compressions arise at the top of the dam (- 14.4 MPa), in the arch direction, while the maximum tensions (3 MPa) are computed at the crest of the lateral cantilevers, in the arch direction, and along their height in the cantilever direction. In this case, the opening of the vertical contraction joints led to a significant release of the arch stresses at the top of the dam, resulting in an increase of the cantilever stresses near the base, along the insertion, and along the upper part of the lateral cantilevers. In this case, high concrete tension damage values are obtained near the base, along the insertion, at the upstream face, and at the upper part of the lateral cantilevers (due to the suffered deformation), at the downstream face. In some points, d^+ values of 100% are calculated, where concrete failure occurs. Nevertheless, regarding structural safety, it is possible to conclude that the dam presents enough resistant capacity to withstand an earthquake with a peak ground acceleration of 0.6g.

Linear seismic response: *SW+HP297+SeismicL (0.6g)*

Time instant of the maximum ***upstream*** displacement



Displacement is towards upstream

Time instant of the maximum ***downstream*** displacement

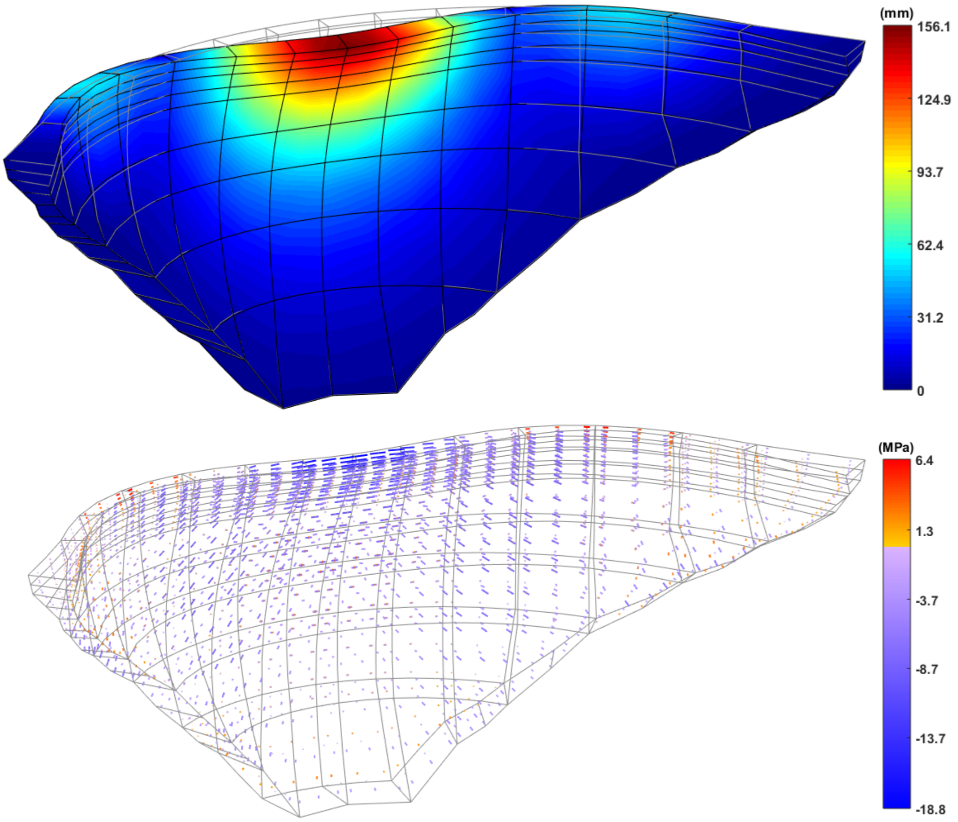
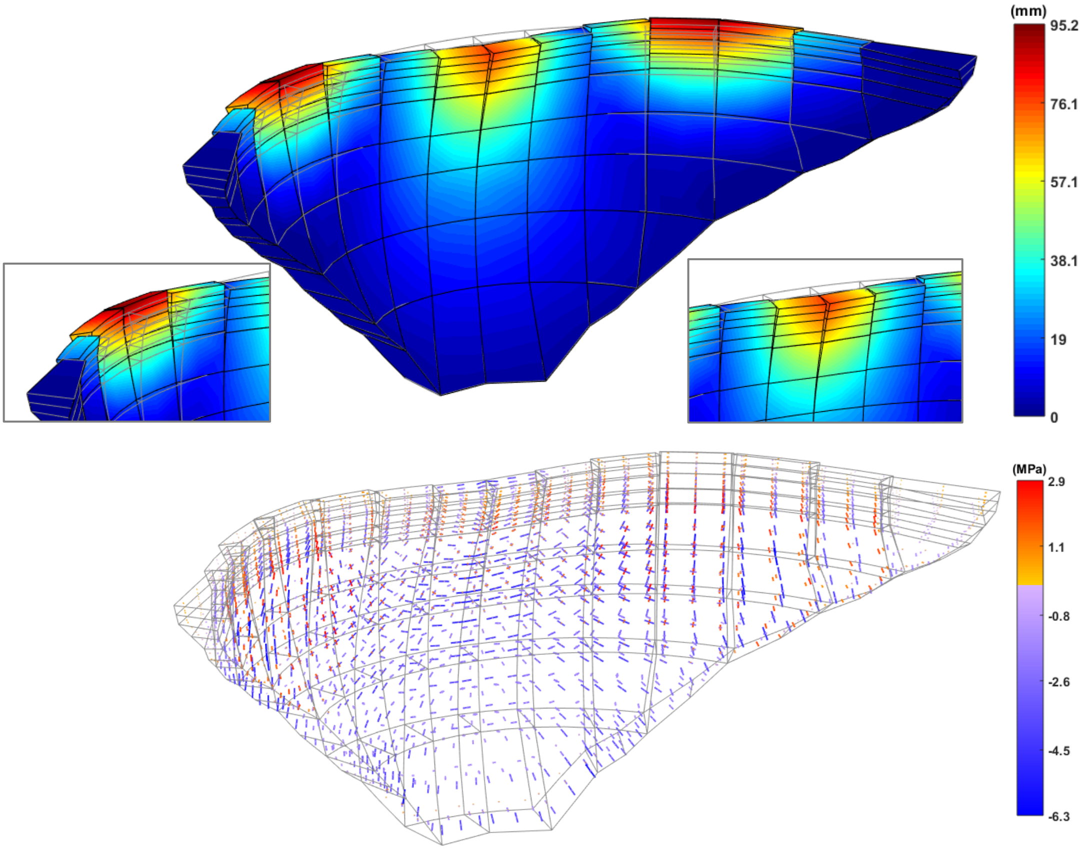


Fig. 12 – Cabril dam. Linear seismic response. Deformed shapes and stress fields.

Non-linear seismic response: *SW+HP297+SeismicL (0.6g)*

Time instant of the maximum ***upstream*** displacement



Time instant of the maximum ***downstream*** displacement

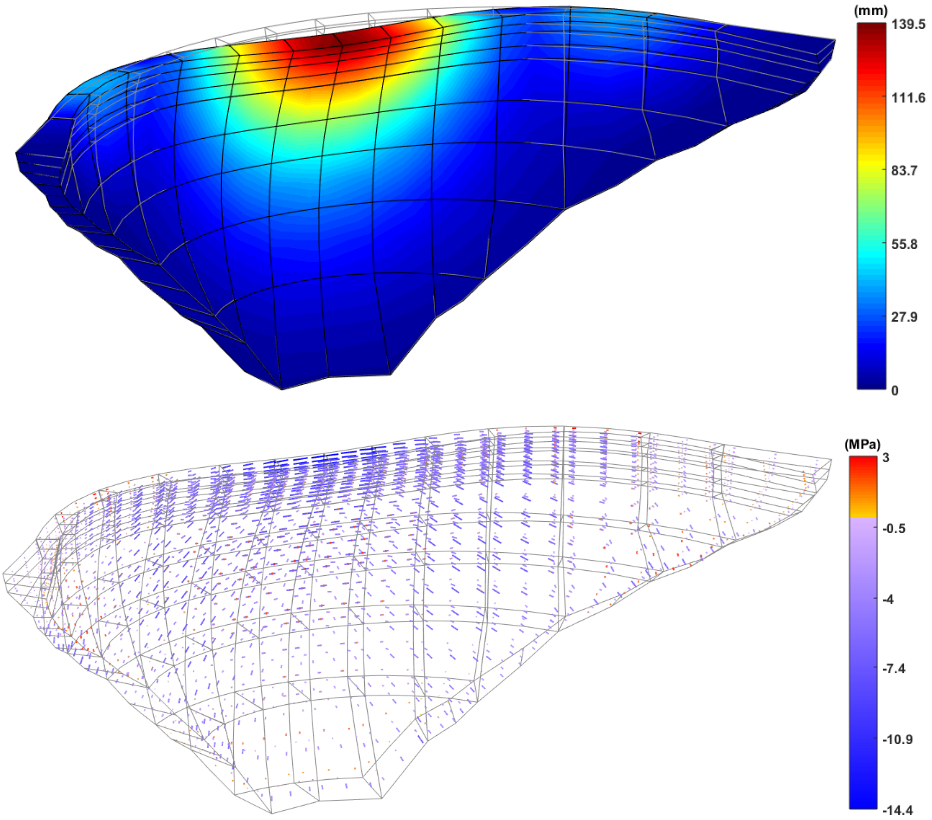
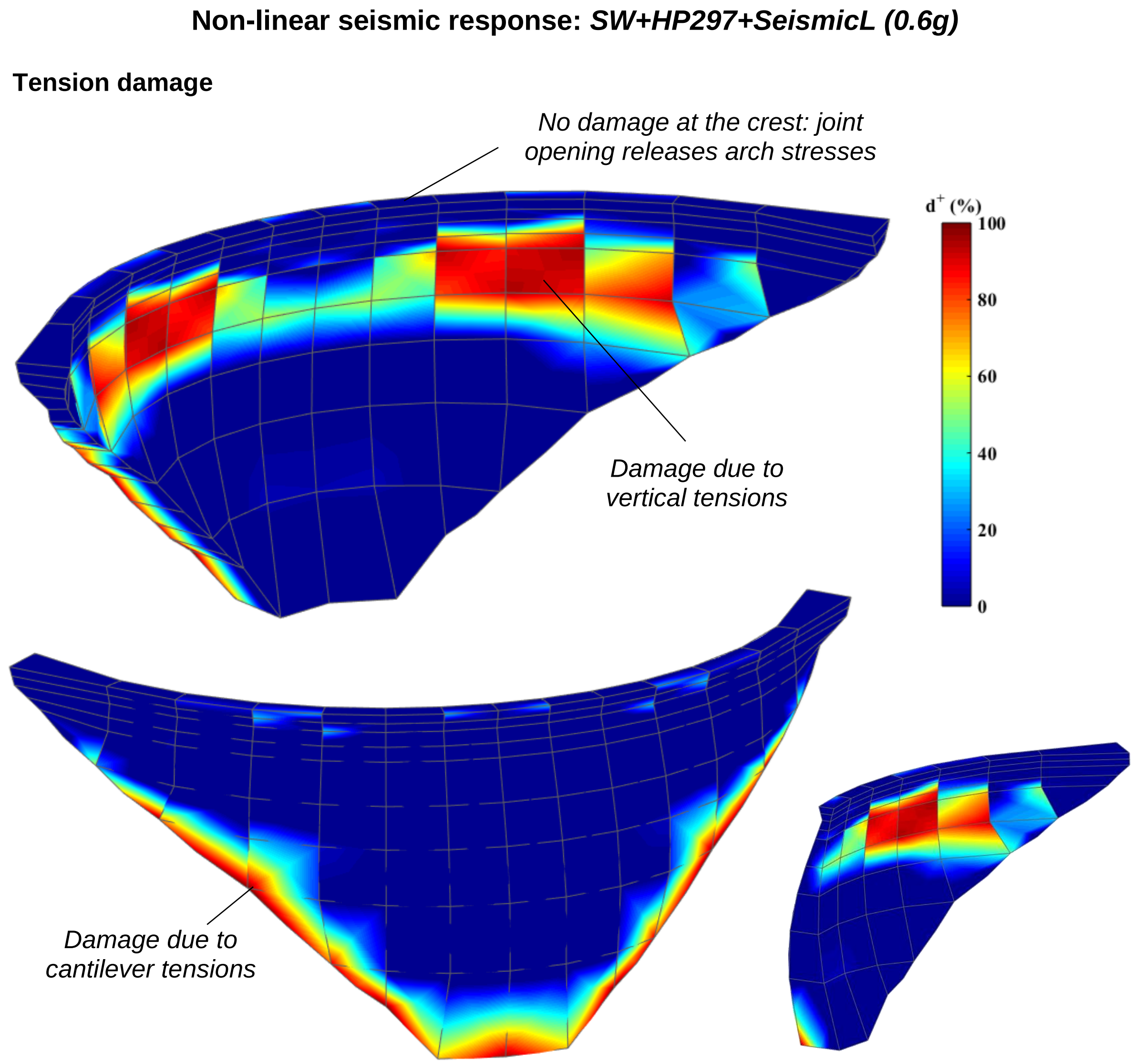


Fig. 13 – Cabril dam. Non-linear seismic response. Deformed shapes and stress fields.


Non-linear seismic response: *SW+HP297+SeismicL (0.6g)*

Tension damage

No damage at the crest: joint opening releases arch stresses

Damage due to vertical tensions

Damage due to cantilever tensions

Fig. 14 – Cabril dam. Non-linear seismic response. Tension damage.

5. CONCLUSIONS

This paper presented a study on the non-linear seismic response of Cabril arch dam for a load combination considering the self-weight, the hydrostatic pressure (full reservoir) and the seismic loading (10 s generated seismic accelerogram). The main results of this study were shown for two scaled accelerograms with peak ground accelerations of 0.2g and 0.6g.

The numerical calculations were performed using *DamDySSA4.0*, a 3D finite element program developed in LNEC for dynamic analysis of concrete dams. The formulation of the recently developed module for non-linear seismic analysis was summarised in this paper.

For the case of Cabril dam, the seismic analysis under a strong earthquake has shown how taking into consideration the movements of the vertical contraction joints and the non-linear behaviour of concrete influenced the dam’s structural response. Namely, the opening of the contraction joints led to a significant release of the arch stresses at the crest, and consequently to an increase of the cantilever stresses in the lateral cantilevers, near the base and at the upper part. In terms of structural safety, it was possible to conclude that Cabril dam presents enough resistant capacity to withstand a strong earthquake with a 0.6 peak acceleration

Finally, this work allowed to demonstrate the potential of *DamDySSA4.0*'s recently developed module for predicting the non-linear seismic behaviour of concrete dams and to support seismic safety verifications.

Given that this was one of the first studies conducted with this module, a coarser mesh has been used. In the future, the aim is to develop and analyse more refined meshes for Cabril dam and for other large arch dams, as well as to further investigate aspects related to the opening of contraction joints and the stresses developed along their height.

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