Simulation of concrete crack development in seismic assessment of existing gravity dams

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ABSTRACT
Most of dams around the world were built before the introduction of seismic regulations, or in those regions that were classified as seismic at a later time. Only few new dams are now being built, so older ones, which are inexorably ageing, are required to fulfil a longer life expectancy, as they are a critical component of our energy production infrastructure. Consequently, a better understanding of the seismic risk of these structures is required.
As no concrete gravity dams are known to have failed catastrophically during an earthquake, modelling aspects assume great importance to predict their collapse behaviour.
In this paper a plane strain model of an existing gravity dam has been analysed in order to simulate the concrete crack development within a continuum damage framework. The study has been performed by using a nonlinear constitutive equation that takes into account the bounded tensile, compressive and shear strength of the material. The collapse behaviour of the dam has been compared with the results obtained by means of models based on the plasticity theory, fracture mechanics or contact elements.

1 INTRODUCTION

Dams probably were among the earliest major structures to be created by humans; the reservoirs retained by dams were key elements in water supply. Nowadays they are a critical component of our energy production infrastructure and, due to economic and environmental reasons, just few new dams are now being built. Most of the existing dams were built before the introduction of seismic regulations or in those regions that were classified as seismic at a later time, so a better understanding of their seismic behaviour is required.

Seismic safety of existing dams is an issue that has been receiving increasing attention in many parts of the world during recent years. It is partly due to the continue increase of population at risk in locations downstream of major dams and also to the awareness that the seismic design concepts in use at the time most existing dams were built were inadequate. The hazard posed by large dams has been demonstrated by the failures occurred in many parts of the world. However, no failure of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failures in the foundation rock. This means that seismic failure modes for concrete dams are not well understood (Anderson et al. 1998). Several dams have been subjected to seismic shaking, and it is possible to learn from their damage.

A significant instance of earthquake damage to concrete dams occurred in the 1960s for Koyna Dam in India. The damage was severe enough, but not so much to determine the uncontrolled water release. From the beginning, many authors (Chopra and Chakrabarti 1972) and (Chakrabarti and Chopra 1972) have come to the numerical simulation of the damage of this dam which has become the most known case study in the world.

Nowadays, high performance computing capabilities allow experimenting numerical simulations under different hypotheses of material constitutive behaviour and various loading conditions.

In this paper the interesting comparison among different models of the Koyna Dam reported in (Roth et al. 2015) is extended by introducing further approaches. More specifically, the crack behaviour of the vertical section of the Koyna Dam has been investigated under statically increasing load using different models. The one is constituted by linear elastic blocks separated by frictional interfaces; the other is based on a nonlinear material with elastic-plastic-damage formulation. Furthermore, the two new models have been subjected to an acceleration time history. Results obtained have been analysed and discussed.

2 COMPUTATIONAL MODELS FOR THE SEISMIC ASSESSMENT OF EXISTING CONCRETE GRAVITY DAMS

Numerical simulation of concrete dams subjected to earthquake loading is a very complex task that includes not only the structure and its material properties, but also the effect of soil-structure and reservoir-structure interaction. For this reason, simple but realistic material laws, which take into consideration seismic loading conditions, are required for the structural analysis.

2.1 Nonlinear models for the seismic assessment of gravity dams

The assumption of linear behavior may not be appropriate in the analysis of seismic response of concrete gravity dams. There are several approaches to model the complicated stress-strain behavior of concrete (Akköse and Şimşek 2010), some of which are based on plasticity models and some others on fracture mechanics.

Elasto-plastic models (Lee and Fenves 1998) can overcome overstressing problems encountered in the linear analysis of a concrete dam and may predict more realistic stress distribution in the dam body during earthquake ground motion. They are considered very useful to determine plastic regions in concrete dams.

Unfortunately they do not reproduce the real behaviour of the concrete that is usually employed in dam construction. In fact, during an earthquake, several parts of the dam may suffer tensile loading with subsequent crack formation; the safety of these structures is thus controlled by the tensile behaviour of the material (Brühwiler and Wittmann 1990). The upper cracks usually initiate from the upstream or downstream face of the dam and propagate horizontally or at an angle toward the opposite face. The consequence of cracking, if extended through the dam section, may lead to sliding or rotational instability of the separated blocks (Ghanaat 2004) (Zhu and Pekau 2007). The rocking stability of a gravity dam with penetrated cracks was first studied by (Saini and Krishna 1974), for the highest monolith of the Koyna Dam.

Traditionally, a no-tension stress criterion has been used in the design of concrete dams (NRC 1990). However, microcracking is always present in concrete, and the acceptance of moderate tensile cracking that does not impair the function of a dam may be a realistic point of view (NRC 1990).

The greatest impediment to effective nonlinear analysis at present is the lack of knowledge about the real nonlinear properties of the mass concrete which is typically used in dams, the so called "dam concrete". Due to differences in the grain size and in the nature of the aggregates, an extrapolation from common concrete to dam concrete cannot be made directly (Brühwiler and Wittmann 1990). In this regard, important efforts have been undertaken from the past in order to study, both theoretically and experimentally the behaviour of concrete under high loading rates (Topçu and Uğurlu 2007), (Wu et al. 2016).

In recent years, the nonlinear dynamic response of gravity dams under earthquake actions including cracking of concrete has attracted more attention from engineers (Hariri-Ardebili et al. 2016).

As a rough method to account for cracking and its consequences on the stability of the dam is that to introduce predefined cracks in a Finite Element (FE) model, following (CFBR 2012).

In the present work a similar model having no-tension frictional interfaces between stacked linear elastic elements were proposed. The definition of potential failure surfaces is suggested by construction joints that may be considered as weak planes.

2.2 Models for Concrete Cracking

Stress and crack response of concrete dams may be analysed by means of many nonlinear models, commonly applied in most of engineering analysis (Pal 1976), (Ghrib and Tinawi 1995), (Pekau et al. 1991), (Ghaemian and Ghobarah 1999), (Guanglun et al. 2000).

Cracking process may be represented by numerous approaches that can be classified into two macro-categories: the geometrical approach, that considers the crack a geometrical entity and, if needed, allows updating discretization model with crack growth; and the non-geometrical approach, which only updates the constitutive relationship during the propagation of cracks, the mesh remaining unchanged (Ingraffea 2004).

The first one, which concerns the discrete cracks, contains two main groups, the linear elastic fracture mechanics (LEFM) and the nonlinear fracture mechanics (NLFM). Regarding the latter, there are two basic procedures of modelling cracks commonly used in numerical analysis; they are the fictitious crack model (FCM) presented by (Hillerborg et al. 1976) and the crack band model (CBM) proposed by (Bazant and Cedolin 1979) and (Bazant and Oh 1983), both of which take the effects of strain softening into account. The FCM overcomes the limitation of LEFM and a nonlinear constitutive relation can be introduced in fracture analysis according to the strain-softening mechanism.

In this regard, in (Pan et al. 2014) a general investigation is presented, in order to evaluate whether the nonlinear responses of concrete dams obtained from different fracture modelling approaches are comparable in terms of crack propagation and failure modes.

The second macro-category regards the continuum models and includes smeared cracks and damage mechanics. In this category, two groups may be identified, the constitutive methods and the kinematic ones. The continuum damage model (CDM), belonging to the constitutive methods, offers the possibility to model areas where damage causes a multitude of micro-cracks that are not necessarily localized. In particular, in the CDM approach introduced by (Rashid 1968), the coalescence of one or more cracks in a volume will result in a deterioration of the stiffness and strength of this volume. The Extended FE Method (XFEM) approach belongs to the kinematic methods and describes the crack geometry independently of the background mesh by enriching the standard displacement-based FE approximation with some pre-knowledge of the physics of crack.

The crack is represented, however, either in the material constitutive model or in the kinematic model, as an intense localization of strain.

In this context, (Roth et al. 2015) proposed a crack model that combines the damage mechanics approach and the XFEM in order to predict the propagation of the crack path within the dam section. The CDM (in this case, it comes of a rotating anisotropic damage model) offers the possibility to model areas where damage causes a multitude of micro-cracks that are not necessarily localized. It can efficiently predict and continuously adjust crack directions during their evolution. The cohesive XFEM, instead, allows a discontinuous displacement field to be well represented across a localized crack. The use of the CDM allows any initial misprediction of the crack direction to be corrected as a crack grows.

In this paper, a model constituted by nonlinear material with elastic-plastic-damage formulation is introduced. It is compared with the one proposed by (Roth et al. 2015) and with the others reported in the same work. The new constitutive model is presented in the following.

2.3 A new constitutive equation for the material

Recently, a new material model has been developed by generalising the constitutive equation of the masonry-like material (Lucchesi et al. 2008).

The masonry-like material (Di Pasquale 1982) is a simplified model that considers masonry as an isotropic non-linear elastic material. It belongs to a class of nonlinear elastic materials that are incapable of withstanding tensile stresses and have linear elastic behaviour when subjected to compressive stresses. For this reason, the stress tensor must be negative semidefinite. The strain tensor is decomposed into the sum of an elastic part, from which the stress is linearly dependent, and the fracture part, that is positive semidefinite and belongs to the normal cone of the stress range in correspondence with the current stress. The material is moreover allowed to extend freely in directions of zero stress.

The constitutive law is fully specified by the tensor $\mathbb{C}$ of the elastic moduli, assumed to be symmetric and positive definite, and by the stress range (the set of all admissible stress tensors) that is a closed and convex subset of the space of the symmetric tensors. Under these hypotheses, an application of the Minimum norm Theorem (Lucchesi et al. 2008) assures the existence and the uniqueness of the stress, when a strain tensor is given.

Later on, this constitutive equation has been generalized, in order to account for a limit to the tensile and compressive stresses (Lucchesi et al. 2008) and, more recently, also for a limit to the tangential component of the stress, proportionally to the normal stress component (Lucchesi et al. 2017). In addition to tensile (fracture) and compressive strains, the anelastic shear strain tensor has been introduced.

In this case, the stress range has been modified, remaining still a closed and convex set, so that the material maintains all the properties of a “normal elastic material” (Del Piero 1989). The obtained material is hyperelastic with a constrain on the stress, but it can also be framed within the so-called 'deformation theory of plasticity' (Kachanov 1971).

The constitutive equation has been explicitly deduced in the isotropic 2D and 3D cases and has been implemented in the FE code MADY (Lucchesi et al., in preparation). The program code now allows for two- and three-dimensional, static and transient dynamic analyses.

Recently, an isotropic damage criterion is introduced. More specifically, when the norm of the fracture strain attains a defined value, the limit of the tensile strength and the cohesion of the material are diminished, according to a negative exponential law that depends on the difference between the fixed fracture strain value and the current one.

This constitutive law seems suitable to describe the damaging of the dam concrete. In this paper, the material model presented above is used for the first time to assess the seismic behaviour of an existing concrete gravity dam.

3 COMPARISON AMONG MODELS

3.1 *The benchmark model*

Koyna Dam, a 103-m-high concrete gravity dam in India, is a benchmark problem which has been widely examined by several investigators for evaluation of their proposed material models.

In the case of Koyna Dam the earthquake forces, based on a seismic coefficient of 0.05 uniform over the height, were expected to cause no tensile stresses; however, the earthquake of 1967 caused significant cracking in the dam. The higher monolith of the non-overflow section suffered the worst damage during the earthquake, endangering their stability during future earthquake shocks. It is believed that this exaggerated damage resulted from an elevator tower that extended 50 ft above the top of the block and therefore was subjected to greatly increased inertial forces.

A static analysis of the Koyna Dam on cases of reservoir overflow was performed using fracture mechanics and plasticity-based models. The results have not been validated experimentally, however numerous authors have published numerical results using the same geometry and material parameters (Gioia et al. 1992), (Bhattacharjee and Léger 1994), (Ghrib and Tinawi 1995) and (Cai 2007).

In (Roth et al. 2015) a comparison among different models of the Koyna Dam is reported. For this case, the presence of unkeyed contraction joints enables the use of 2D plane stress models of individual monoliths to predict the earthquake response of the dam under moderate and intense ground motions. FE model is made of a four-nodes quad plane stress mesh (fig. 1). An initial notch facing the change of slope on the downstream face with a depth that corresponds to 10% of the length of the section is included in the model by disconnecting the nodes of the elements. Material properties are reported in table 1. The value f_t’ = 1.0 MPa is fairly typical for actual tensile tests on concretes customarily used in gravity dams. The model is subjected to gravity, hydrostatic pressure of a full reservoir level and overflow pressure. The water pressure inside the cracks is neglected.

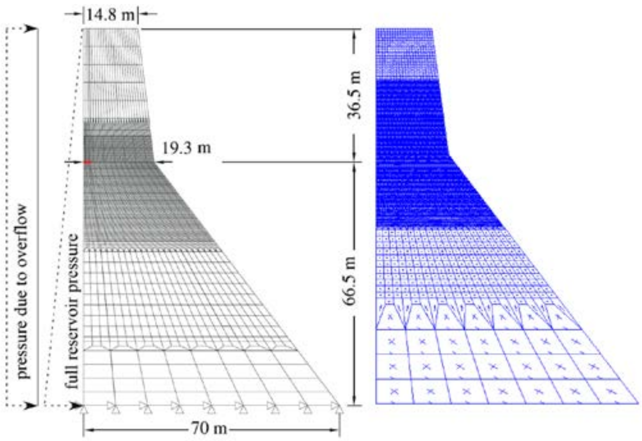

Figure 1. The Koyna Dam section: model of (Roth et al. 2015) (left) and MADY model (right).

Table 1. Material properties for the overflow analysis

E [MPa]	f_c' [MPa]	f_t' [MPa]	ν	ρ
25000	10.00	1.0	0.2	2450

3.2 *Two new models for the Koyna Dam*

Two FE plane strain models of the Koyna dam, having very different characteristics, have been developed in this work. Model A, created with MADY code, schematizes the dam via a continuum model having an elastic-plastic-damage material; model B, created with Ansys r.17.2 research, represents the dam divided in 21 stacked linear elastic elements separated by no-tension frictional interfaces. The interfaces simulate planes of weakness that may exist at lift joints and can be a major influence on crack location.

The FE model A (Figure 1) is composed by 658 plane four-nodes elements and 706 joints. The base nodes are fixed along the two directions.

The FE model B (Figure 3) is formed instead by 3531 eight-nodes plane strain elements (element type “plane183”) having quadratic shape functions, and 12210 nodes. The base nodes are restrained along the two directions. The interfaces are modelled with 2733 contact no-tension elements (element type “CONTA172”) with frictional behaviour following the Mohr Coulomb law (Augmented Lagrange Formulation).

In both models, the foundation is also assumed to be rigid, which means the dam–foundation interaction is neglected.

Firstly, the two models have been subjected to quasi- static load conditions, caused by an hypothetical overflow. Results coming from both models have been compared with those obtained in the literature for the overflow analysis on the Koyna Dam.

For the sake of comparison, in the overflow analysis material properties of model A have the same values adopted for the models reported in the literature (Table 1); in addition, the friction coefficient μ is set equal to 1 and the cohesion c to 1MPa. Cohesion values, or zero normal load intact shear strengths, are typically about 10% of static uniaxial compressive strengths based on direct shear tests of concrete core samples, and coefficients of friction are typically near 1 (NRC 1990). Material properties of Model B have the same values of mass and elastic coefficients reported in table 1 for the linear elastic material, whereas friction coefficient μ of the no-tension interfaces is set equal 1.

During the overflow simulations, the model initially undergoes its self-weight and the hydrostatic load is applied on the whole height of the dam. Afterwards, it is subjected to a step by step constant pressure increase of 1 cm due to overflow, starting from the full reservoir water level, up to the water height that brings the dam to the collapse limit state. The water uplift is neglected.

Afterwards, both models have been subjected to a transient analysis. In this case, different values of material properties have been adopted. More specifically, in model A different cases, having different values of tensile strength f_t' and cohesion c have been considered.

- Case 1: masonry–like material (no-tension elastic material).
- Case 2: elastic-plastic damage formulation (f_t' = 1MPa, c = 1MPa and μ=1).
- Case 3: elastic-plastic damage formulation (f_t' = 0.05 MPa, c = 0.5 MPa and μ=1).

In model B, two different values of friction angle, 45° and 71°, have been adopted, corresponding to μ=1 e μ=3, respectively, in addition to the material properties previously defined.

The Rayleigh damping for all cases is set by putting the mass proportional coefficient α equal to 3/s and the stiffness proportional coefficient equal to 4.2E-4 s.

Finally, Westergaard added mass model is used to simulate the hydrodynamic effect (Westergaard 1933) induced by the reservoir in addition to the hydrostatic component. So, both models have been equipped by added masses, suitably distributed along the height of the dam.

The dynamic excitation includes only a component in the horizontal plane of earthquake records (Figure 2). It belongs to one of the Italian strongest events occurred in the last 30 years, the earthquake of Central Italy of October 30th 2016 (06:40:17 UTC, 6.5 MW). The duration of transient load is 30 s and the sampling is 5/1000 s. The time step of the analysis ranges from 0.0001s to 0.005s. The analysis is carried out by assigning to the model, in addition to the self-weight and the hydrostatic load, the seismic shaking in form of volume loads.

Also in this case, no uplift pressure is considered for both models.

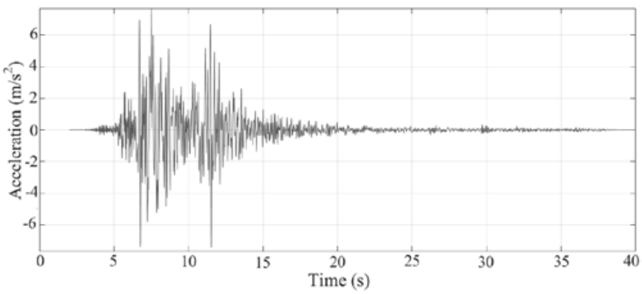

Figure 2. The acceleration time history for the transient analysis.

3.3 *Results of the overflow analysis*

The results of the overflow analysis conducted on model B (Ansys) are reported in Figure 3 in terms of minimum principal stresses. The deformed shape clearly shows an opening of the contact elements that is concentrated at the base of the neck.

Results obtained from model A (MADY) are reported in Figure 4 in terms of anelastic shear strain. One may observe that the initiation of the cracking is the same in both models A and B.

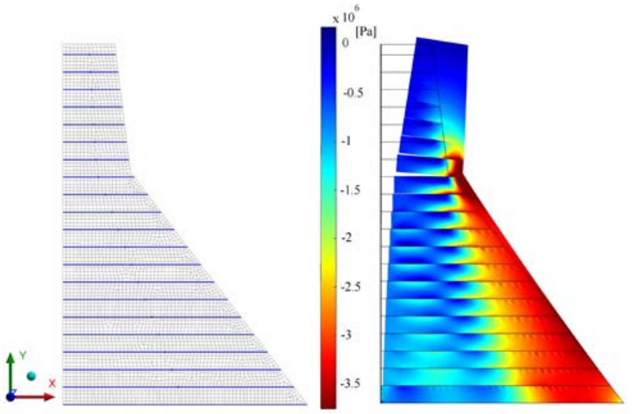

Figure 3. The Ansys 17.2 mesh for Model B (left). Results of the overflow analysis of Koyna Dam in terms of minimum principal stresses (right).

The crack path resulting from model A is compared with those given in the literature resulting within a small bandwidth (Figure 4). Unlike the other models of the literature, the anelastic shear strain exhibits in this case a bifurcation, one branch is quite horizontal and the second one follows the crack path of the other models. Anelastic shear strain may be detected also downstream, at the base of the neck, and upstream, at the base of the dam. In all cases the reservoir overflow increases compressive stresses on the downstream face of the dam and drives the crack downward. This shows that overturning of the top part of the dam is the principal consequence of cracking in this region.

Anyway, it should be stressed that all the models taken from the literature were characterized by an initial notch facing the change of slope of the downstream face, whereas model A doesn't need the definition of a weak area from which the fracture starts. Nevertheless, in this case the fracture is triggered at the same point, probably due to the particular geometry of the section.

The response of the structure is represented by the overflow height versus the horizontal crest displacement. In Figure 5 the response of models A and B are shown and are put in comparison with those of the models cited above and reported in the work of (Roth et al. 2015). In particular, the combined model proposed in (Roth et al. 2015) is called "Léger", the model based on the plasticity theory "Gioia" (Gioia et al. 1992), those based on smeared cracks "Bhattacharjee" and "Cai" (Bhattacharjee and Léger 1994) and (Cai 2007), and the model based on damage mechanics (Ghrib and Tinawi 1995) is called "Ghrib".

The response of model A (MADY) is in agreement with the plasticity model of "Gioia". In this latter case (Gioia et al. 1992), a perfectly plastic model with an associated flow rule and a yield surface as proposed for plain concrete by (Ottosen 1977) is adopted. That is, no-tension assumption is implemented as a special case of plasticity, in which the tensile yield limit tends to zero (the no-tension design ought to be considered as the limit of the plastic designs as the tensile yield limit tends to zero).

The response of Model B, instead, is in agreement with smeared cracks models and damage mechanics.

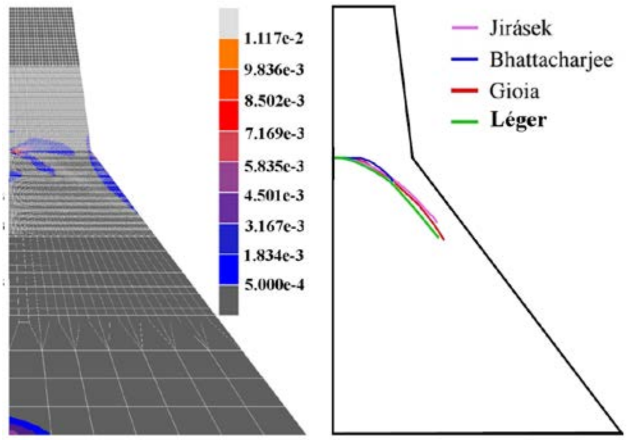

Figure 4. Results of the overflow analysis on Model A (Mady) in terms of anelastic shear strain (left), compared to the cracking paths obtained in the literature (right).

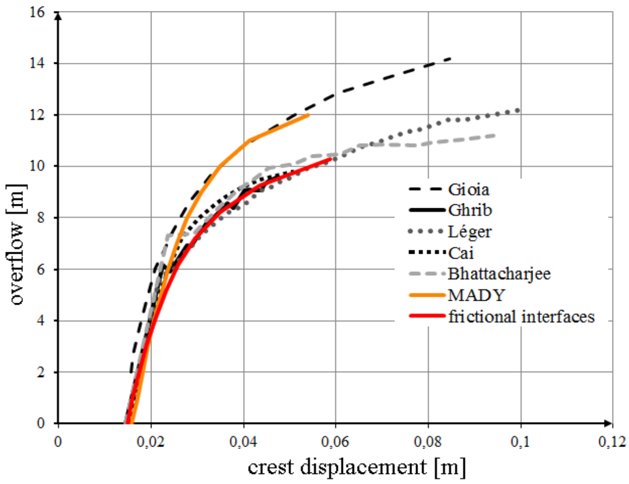

Figure 5. Structure response under overflow in term of overflow vs. crest displacement.

Both new models exhibit a lower capacity in respect to "Gioia", "Léger" and "Bhattacharjee". The reason can be furthermore investigated by both using different values of the parameters that are not in common with the other models and suitably modifying convergence settings.

3.4 *Results of the transient ground shaking analysis*

Transient analysis was applied for a first attempt to evaluate the ability of both models to describe the dam behaviour during earthquake. For model A, different values of tensile strength and cohesion, in relation to the three cases described above, have been considered. For model B, two different friction angle values have been considered. The analysis has been carried out under the hypothesis of large displacements and the results are obtained in terms of crest displacements and base shear time histories.

In the Figures 6 and 7 the response of model B in terms of base shear and crest displacement, respectively, is reported.

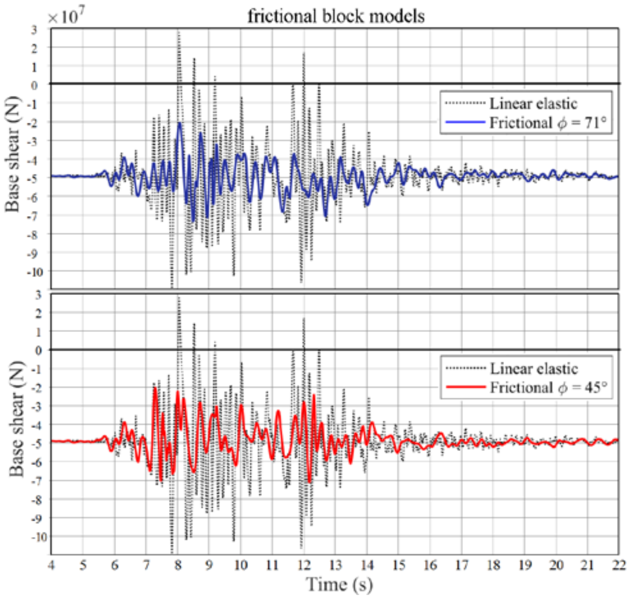

Figure 6. Base shear resultant force - model B (friction angles of 45° and 71°).

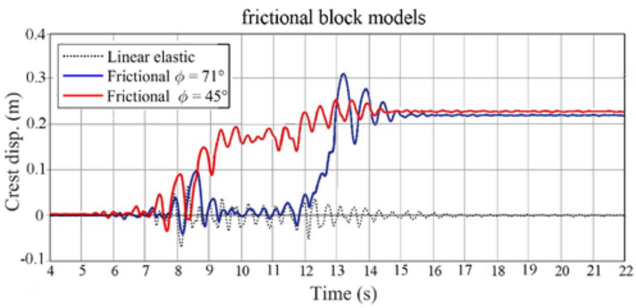

Figure 7. Crest displacement – Model B (friction angles of 45° and 71°).

As for the three MADY models, in the Figures 8 and 9, the history of dam crest displacement and that of the base shear resultant force for the combined static and earthquake loads are shown. In any case, the curve for the linear elastic model is always reported as benchmark.

As for the base shear values, the response of model B is lower than in the linear elastic case. The response of model A is greater than that of model B, but of the same order of magnitude. More specifically, in the case 2 of model A, including damage and the higher values of tensile and shear strength, peaks sometimes exceed the values of the linear elastic model. The response in terms of crest displacement of the three models A is comparable.

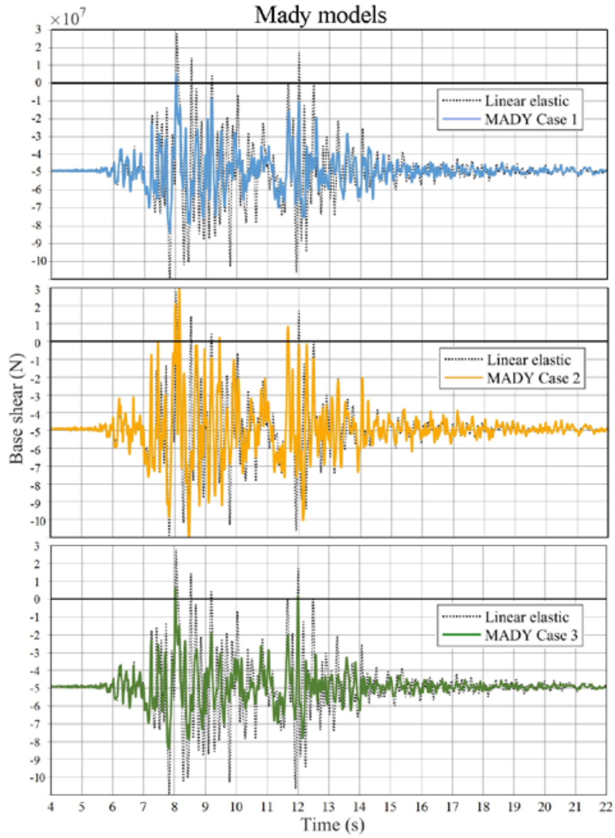

Figure 8. Base shear force - model A.

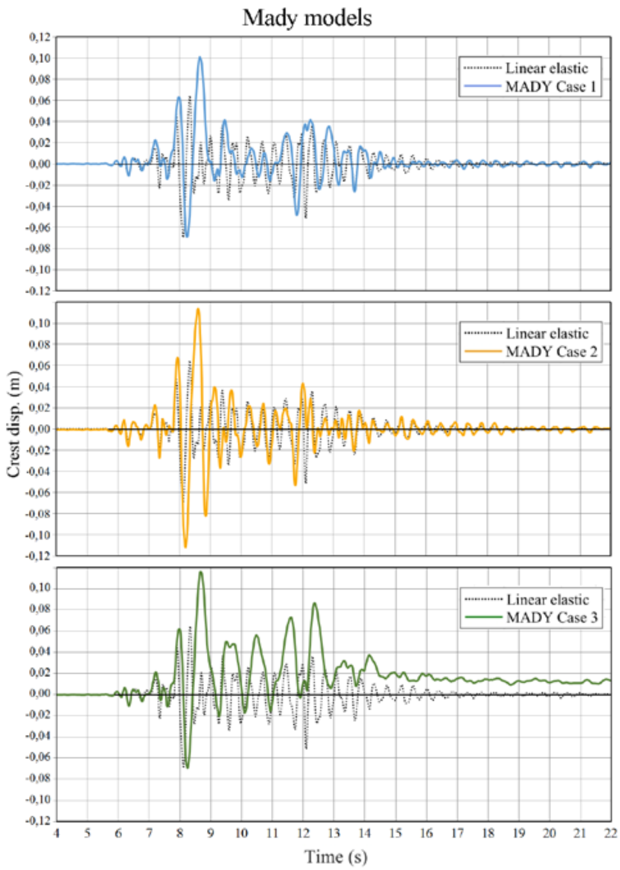

Figure 9. Crest displacement – Model A.

The response of case 1 is the most similar to the elastic linear one. The response of case 2 exhibits the greatest amplification due to the nonlinearity. Finally, the response of case 3, having the lowest tensile and shear strength, shows a residual displacement due to the occurrence of sliding after tension damaging.

The comparison between models A and B in term of base shear is hard to be carried out. During early shaking, the displacement values in models A are similar to those in models B. Successively, models B exhibit a residual displacement that attains values of an order of magnitude higher in respect to the case 3 of model A. Moreover, the different values of friction angle in models B do not influence the total amount of residual displacement, but the way it occurs.

In Figures 10 and 11 the images of the deformed shapes of model B at different times instants, respectively, for friction angle of 45° and 71° are reported. The minimum principal stresses map is also shown.

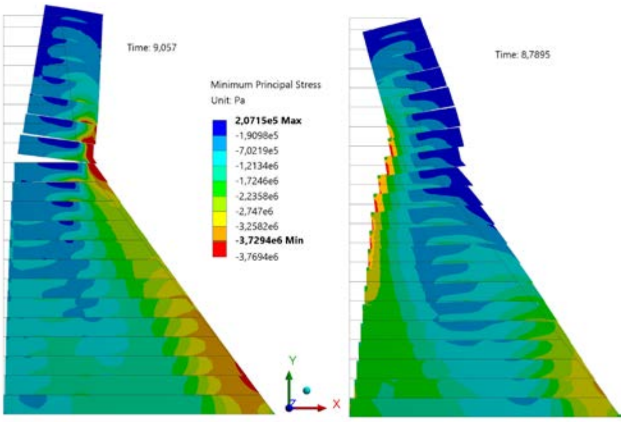

Figure 10. Response of model B (ϕ=45°) in term of minimum principal stress, at 9.057s (left) and at 8.789s (right) – deformed shape.

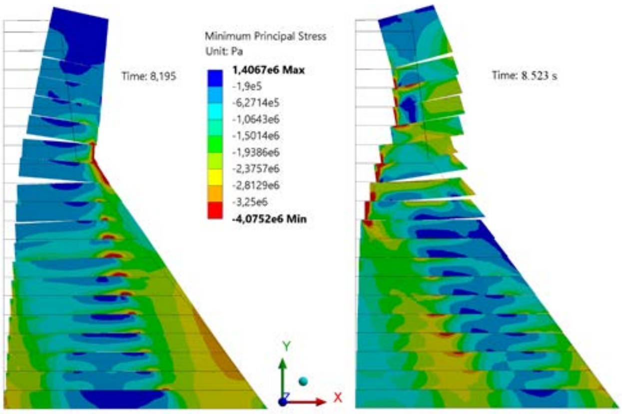

Figure 11. Response of model B (ϕ=71°) in term of minimum principal stress, at 8.195s (left) and at 8.523s (right)– deformed shape.

One can observe that cracks form near the change in downstream slope at both faces. Moreover, for lower values of friction angle, cracking occurs only upstream at the base of the neck, while, for higher values of friction angle, it also occurs downstream. The tendency to slide at the base of the neck is particularly evident in the downstream zone in both cases, particularly in the case of lower friction angle.

As for model A, in Figure 12 the map of anelastic shear stress for the case 3 (low tensile strength) accumulated during shaking is shown. In Figure 13 the map of tensile fracture strain for the same case is reported. As in models B, the tendency to slide, especially at the downstream face near the change of slope, is evident. Cracking occurs at the base of the neck on both upstream and downstream faces, at different levels, as is the case of model B.

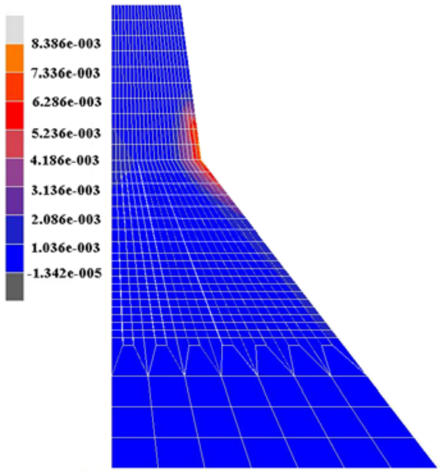

Figure 12. Anelastic shear strain – Model A – case 3.

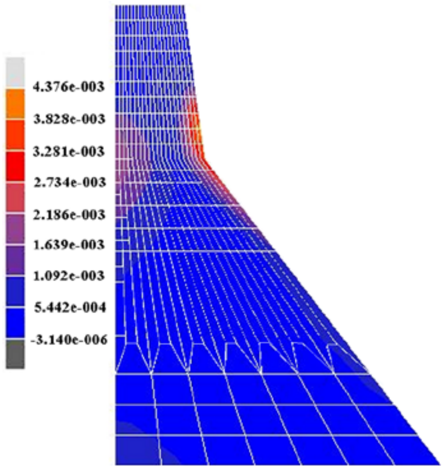

Figure 13. Tensile fracture strain – Model A – case 3.

In Figures 14 and 15 the maps of the damage level in terms of ratio between residual tensile strength and initial tensile strength are reported. The unitary value indicates that material is unchanged and 0 value means that material is totally damaged. In Figure 14 damage of model A - case 3 after the end of the shaking is shown. The section at the base of the neck is damaged on its entire length and this fact is in agreement with the presence of residual displacement shown in Figure 9. Another crack is located upstream at the base of the dam.

The behaviour of model A - case 2 is different (Figure 15). Due to the higher values of the tensile strength and cohesion of this model in respect to the case 3, a particular type of cracking occurs. Cracks firstly form at the downstream face, near the change in slope, and proceeds downward to the upstream face assuming an “S” shape. The section at the base of the neck is not entirely damaged, so the crest does not exhibit permanent displacements after the end of the shaking (Figure 9). The formation of the crack on the upstream face is possible only in the case of lower tensile strength.

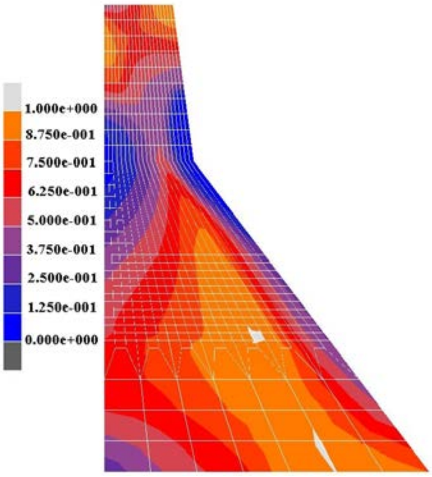

Figure 14. Damage level after shaking – model A, case 3.

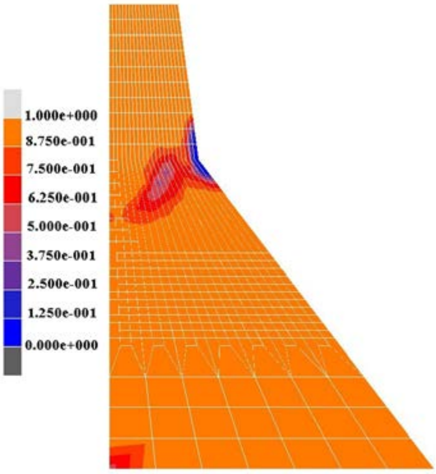

Figure 15. Damage level after shaking – model A, case 2.

The resulting crack path is worth to be compared with other examples in the literature. As a significant example, in (Omidi et al. 2013) a study of the Koyna Dam is carried out by using a plastic-damage model, whose constitutive relations are fully described by Lee and Fenves (Lee and Fenves 1998). The crack paths obtained from the simulation under shaking are surprisingly similar to that shown by Figure 15.

Crack paths are also well comparable with those obtained in (Pan et al. 2011), where a general evaluation of different fracture procedures in terms of nonlinear response and cracking development is performed.

4 CONCLUSIONS

This paper proposes two new models for the dam crack behaviour investigation. The first one is based on a continuum elastic-plastic damage formulation, while the second one is made up of stacked linear elastic blocks with no-tension frictional interfaces. The analyses were performed under both quasi - static conditions, caused by an hypothetical overflow, and earthquake excitation.

The numerical results of overflow analysis, compared with other results in the literature, show the aptitude of the first model to reproduce the behaviour of other plasticity models, and the ability of the second model to catch the load-displacement behaviour of the fracture mechanics models.

The transient analysis demonstrates that the two models are both applicable to predict the shear load and the displacement histories of the Koyna Dam under earthquake conditions. The first one, providing only a rough crack initiation localization, captures the order of magnitude of the resultant forces history. Due to the constrained crack paths, it essentially fails in the prediction of crack propagation. The second one, more promising, catches more suitably the damage extent and the response of the structure in terms of displacements. From the practical point of view, it provides some advantages. On the one hand, the availability of the explicit form of the constitutive law, together with the tangent matrix, improve the convergence of the FE analysis. On the other hand, the direct control of the different types of fracture strains allows observing the different failure mechanisms (sliding-rocking) and then no failure section has to be fixed a priori.

Following this first attempt, a more appropriate calibration of the two models is scheduled, in order to provide more precise indications.

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