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# Earthquake analysis of an old cyclopean concrete dam and its seismic retrofit with post-tensioning anchors.

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## Summary

The dynamic analysis and especially the seismic design of dams are an important area of research in the past years. This research has, however, focused on concrete or embankment dams. There are a significant number of dams in the world, which have been built of rough masonry, mainly in the early 20th century when the use of concrete was not yet widespread. The dynamic behavior of these dams was not thoroughly investigated, and the seismic calculations in seismicity areas were carried out often under simplified assumptions. After about a century, many of these dams are still in use and must fulfill the requirements of the current anti-seismic standards. Using the finite element method, these structures can now also be investigated in order to obtain important information about the seismic behavior of such structures during an earthquake.

The article deals with the seismic analysis of a gravity dam with a height of about 50 m, which was built of rough masonry in the early 20th Century. First, the properties of the material are analyzed computationally on the basis of a 2D finite element calculation at the meso-scale level in order to formulate the nonlinear constitutive model of the heterogeneous material for the whole model in a-as realistic as possible-way. Subsequently, the material constitutive law is applied as a continuous material for the seismic analysis of the dam. In the loading of the dam all other loads or influences such as hydrostatic pressure, pore pressures in the dam and the foundation rock and temperature, are taken into consideration. The analysis is performed for two seismic excitation cases (a) non-seismic upgrading, and (b) with a seismic upgrading with unbonded tendons. The nonlinear calculations are performed for these two cases, in order to quantify the contribution of the latter on the seismic response of the dam.

## Introduction

The seismic analysis of concrete dams has to take into account many phenomena such as the soil-structure-interaction, the fluid-structure-interaction, the sliding of the dam over the rock foundation, the rocking response of the dam, the seismic wave propagation in the infinite media, the radiation absorption of the seismic waves, the discontinuities in the rock foundation, etc. The difficult problem of the seismic behavior of concrete dams can become more complicating, if there is much uncertainty about the constitutive law of the material. The strength of the material can be influenced by many factors, such as creeping, alkali-aggregate reaction, the aging of the concrete and many other reasons. The complexity of accounting of the material characteristics can increase, if there is not any financially effective solution to estimate the material characteristics. The common practice of taking samples of the dam material or trying to manufacture samples with such material characteristics similar to the dam ones is often either inapplicable or too expensive or refrains much from the reality.

## Layout of paper

This document is divided in two parts; the first part presents a procedure to obtain the material characteristics of the dam and the second part analyses the dam for the seismic excitation, taking into account the real state of loads at the time that the earthquake occurs. Additionally, it gives the results obtained from a type of seismic retrofitting, the one of the post-tensioned anchors.

**Part A – The constitutive law of the material**

The analyzed dam was constructed at the beginning of the 20th century, has a height of about 50 m, a length of 400 m and a total volume of material of about 300.000 $m^3$. The material is often mentioned in the dam literature either as rubble masonry or as cyclopean concrete. It consists of huge rubble stones, with a size of 50-60 cm, bonded with truss-lime mortar. The stones cover about 60% of the volume of the compounded material. Such a construction is difficult to be characterized as masonry or concrete one. The samples taken from the dam in the form of drilling carrots with a diameter of up to 23 cm cannot represent adequately the material. Hence the laboratory tests for the definition of the strength characteristics were seen with aloofness and only the values for the mortar could be taken into account (also with aloofness because the small amount of the material may not represent the real condition of the mortar).

The idea was to model the material of the dam and analyze it computationally. Such an assumption seems very reasonable for two reasons: firstly, there is almost impossible to take representative samples of the dam or to manufacture similar ones in a financially efficient way and secondly, the computer abilities nowadays allow us to attempt such a computation. Despite the big size of the aggregates, they are small related to the big dimensions of the dam, allowing us to consider that the Hill's theorem is satisfied. The Representative Volume Element (RVE) has a rectangle shape with dimensions 2,5 m × 2,5 m, which means that about four to six illusions fit at the length dimension and that about fourteen RVEs fit at the dam basis of 36 m (Figure 1).

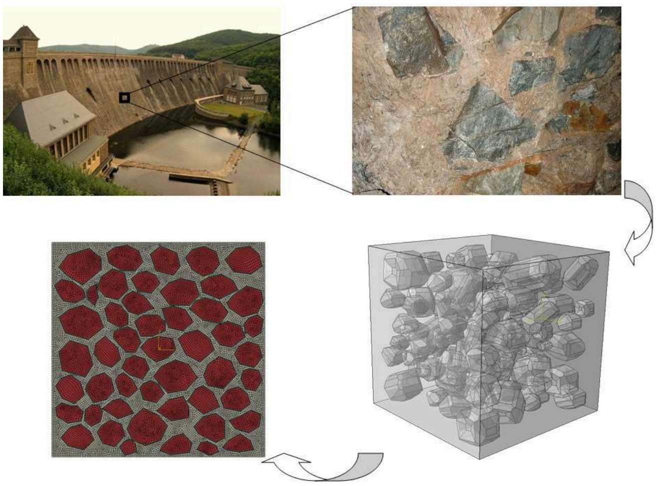

Figure 1: The Representative Volume Element

**Finite elements and mechanical characteristics**

The RVE was modeled in two dimensions. The stones (aggregates) have only elastic characteristics. To the mortar was assigned the material model of concrete damaged plasticity [2], which is available in Abaqus. The ITZ was modeled also as continuum medium for computational sufficiency despite its negligible size. For the Interfacial Transition Zone the 60% of the mechanical strengths of the mortar were assigned. The mechanical characteristics are presented at the Table 1. The RVE was numerically investigated in order to find the mechanical characteristics (Elasticity modulus, Fracture energy, etc.) of the equivalent smeared material. Reduced integration plain strain elements and the explicit computation code were used.

TABLE 1: MECHANICAL CHARACTERISTICS OF RVE

| | **Stones** | **Mortar** | **ITZ** |
|---|---|---|---|
| E (MPa) | 40.000 | 8.000 | 4.800 |
| v | 0,25 | 0,20 | 0,20 |
| $f_c$ (MPa) | - | 17,6 | 10,6 |
| $f_t$ (MPa) | - | 5,0 | 0,3 |

The determination of the size dependent fracture energy of the compound material was carried out with numerical spitting test following the work of J. Šejnoha et al [3]. The numerical experiments are shown at the Figure 2.

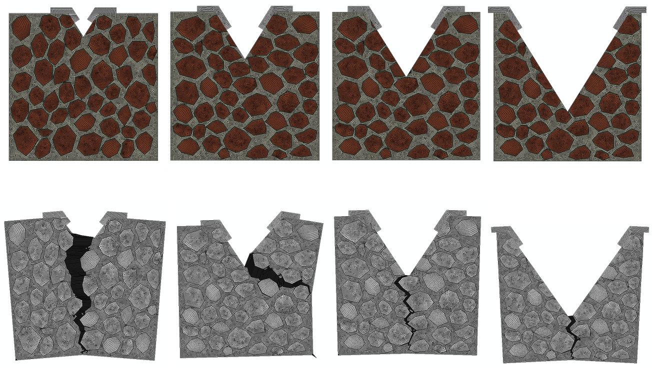

Figure 2: The numerical wedge splitting test

The size dependent fracture energy is given from:

$$G_f(a) = G_F \cdot \frac{W-a}{2 \cdot a_l}\text{, for} \quad a \geq W - a_l \tag{1}$$

$$G_f(a) = G_F \cdot \left[1 - \frac{a_l}{2 \cdot (W-a)}\right]\text{, for} \quad a \leq W - a_l \tag{2}$$

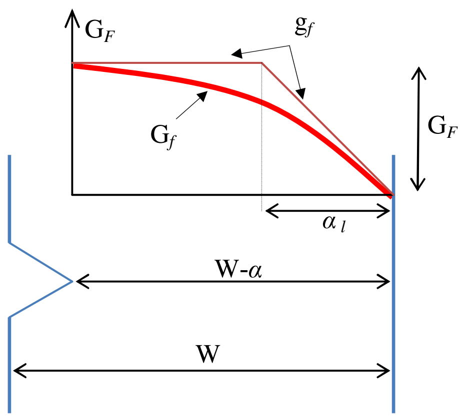


Figure 3: Graphical representation of fracture energies

The fracture energy was estimated to be 26 N/m. The characteristics of the homogenized material are presented at the Table 2.

TABLE 2: MECHANICAL CHARACTERISTICS OF THE HOMOGENIZED CONCRETE

| | **Cyclopean Concrete** |
|---|---|
| E (MPa) | 15.600 |
| v | 0,20 |
| $f_c$ (MPa) | 23,4 |
| $f_t$ (MPa) | 1,0 |
| $G_F$ (N/m) | 26 |

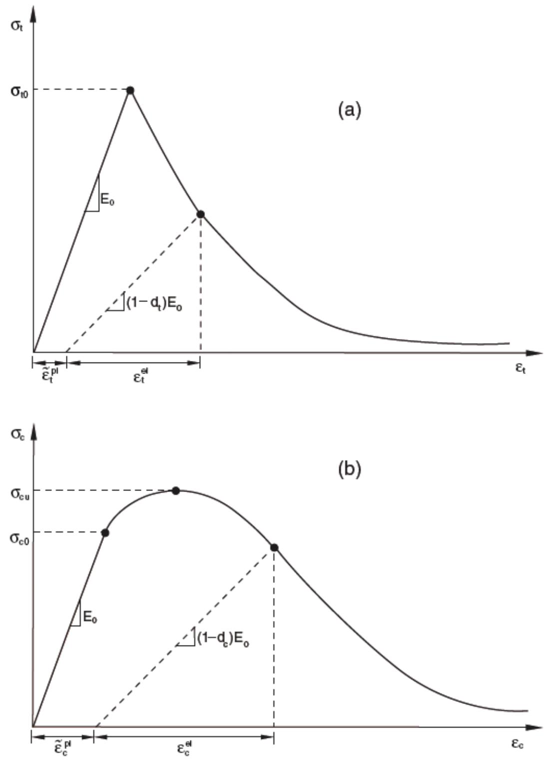


Figure 4: The concrete damaged plasticity model.

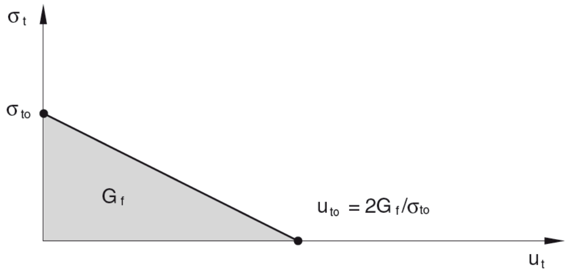


Figure 5: The tension stiffening as linear relation with the fracture energy.

**Part B – The pre-seismic analysis**

The whole analysis was performed as a 2-D plane strain model with the commercial finite element program Abaqus [1]. As mentioned before the curvature of the dam has no significant contribution to the transferring of the loads to the surrounded rock, so it is also applicable to perform only a two dimensional plane strain analysis. The pre-seismic analysis includes several steps in which a real stress state condition of the dam and the rock foundation will be established before the earthquake occurs. These steps consist of: a) a geostatic stress distribution to account for the stress condition in the rock foundation under its self weight for the years before the dam construction, b) the self weight of the dam after the dam construction, c) the hydrostatic forces on the reservoir bottom and on the dam after the fill of the reservoir, d) the pore pressure that developed in the dam and rock material all these years, e) the post tensioning of the dam with anchors (this step is omitted for the initial analysis without the post tensioning retrofit of the dam), f) the thermal load due to seasonal air and water temperature change and finally g) the earthquake event.

The geostatic step includes the determination of the gravity stresses of the rock from a previous analysis, which are inserted as initial stress conditions for the rock foundation. At the geostatic step a self equilibrium is established in order not to have deformations due to the self weight of the rock foundation.

After the geostatic stresses are established, the dam is inserted in the model and its self weight is accounted for the new stress state of the rock foundation.

The next step concerns the application of the hydrostatic loads on the foundation and the dam due to the reservoir fill. This step is followed by the pore pressure distribution in the dam and the rock. Both materials, the concrete and the rock, have voids, through which the water can flow. The presence of water in the voids causes effective stresses, which are higher comparing to a dry condition assumption. For the pore pressure analysis a steady state condition was assumed.

As next analysis step is considered the application of the post tensioning anchors. This step concerns only the retrofit analysis and is omitted for the initial analysis without seismic upgrading. However, this step shall be performed before the thermal loading, so as the thermal changes are taken into account for the stress state of the anchors. The post tensioned anchors were placed in order to upgrade the dam's carrying behaviour for static loads. With this analysis their contribution as a seismic upgrading is examined. The necessary compression force was calculated by a former analysis to be 2000 kN/m, which corresponded to a tensioned force of 4500 kN per anchor (considering that the anchors are placed in spaces of 2,5 m). In this analysis were a plane strain condition is considered, the dam and the foundation rock have a depth of 1 m. Therefore a tension force of 2000 kN rather than 4500 kN is applied to the anchor. Such an assumption fails to determine the real stresses at the anchorage level of

the anchors in the rock foundation, but succeeds in determining the real stress distribution in the dam. The anchors are unbonded and the tension force was applied with the technique of temperature change in the element. With the expansion of the steel material known, the temperature change is calculated in order to achieve the same strain for the anchor which corresponds to 2000 kN tension force ($E_{steel}$=210 GPa).
The thermal loading takes into account the seasonal temperature changes in the air and the water. A period of 10 years of cycling thermal loading is considered in order to obtain a realistic stress distribution in the dam. The thermal loading ends at the two extreme temperature conditions (winter and summer).
As a realistic stress distribution is established for the dam for the characteristic quasi permanent loads, the seismic event occurs.

**The seismic analysis**

The dam-foundation-reservoir system is subjected to an earthquake excitation. Regardless the real seismic risk of Germany and the German regulations, which determine a return period of 2500 years for this size of dams, a hypothetical ground acceleration of 0,36 g was assumed. This size of ground acceleration is of greater interest as it applies in many European seismic regulations (although for a return period of 475 years). First the response spectrum, which corresponds to this ground accelerations was determined according to DIN EN 1998-1 (EC-8) and then five matched time histories were generated considering the limitations of DIN EN 1998-1(Figure 4) [4]. Then the time histories were deconvoluted [6] by the common used program SHAKE91 (Figure 5) [5].
The seismic input was applied in form of equivalent shear stresses at the lower foundation elements. As the rock foundation is considered to behave linearly during the seismic excitation and because of the elastic characteristics of the rock, a bedrock level is difficult to assign. Moreover the finite element model has to consider the wave absorption at the model boundaries and not reflect the seismic waves back to the model.

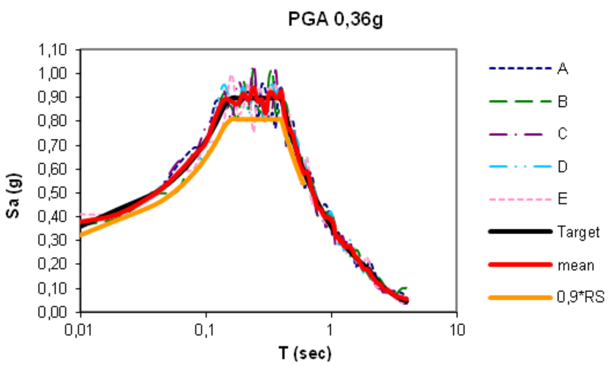


Figure 6: Time histories according to EC-8.

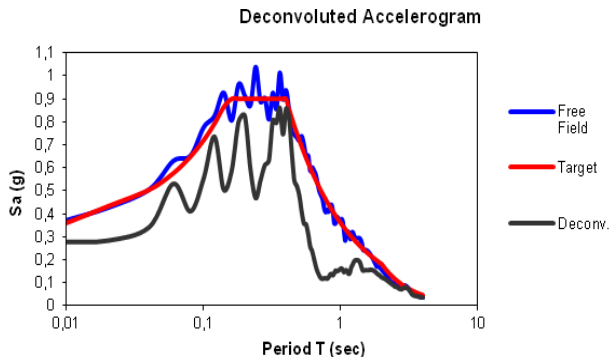


Figure 7: Deconvoluted time history.

The FE program Abaqus offers infinite elements, which behave as absorbing boundaries during a dynamic analysis. The damping at the boundaries is proportional to the density of the medium and the wave velocities [1]:

$$d = \rho \cdot c \tag{3}$$

Where $d$ represents the damping factor, $\rho$ the density of the medium and $c$ the wave velocity.
However, this wave absorption has to be taken into account for the seismic input [7]. The accelerations are converted to time history velocities and then are multiplied with the shear wave velocity and the density of the medium in order to obtain stresses. The factor 2 accounts for the damping at the boundary.

$$c_s(t) = 2 \cdot \rho \cdot C_s \cdot v_s(t) \tag{4}$$

$c_s(t)$ means the shear traction as a time history, $\rho$ the density of the medium and $C_s$ the shear velocity of wave propagation in the medium, $v_s(t)$ the earthquake velocity time history.

**The finite element model**

The dam is about 36 m wide at its base and has about 50 m height. Therefore the foundation is extended $3 \times h_{dam}$ in both sides and depth (Figure 6). The size of the foundation elements was chosen to be smaller than the 1/8 of the wave length to the direction of propagation [9]. As the frequency cut-off of the excitation depends on the element size, we can determine the element size according to frequencies, which do not affect much the dam response. As mentioned before the wave absorption at the boundaries is succeeded with infinite elements. The reservoir was modelled with acoustic elements. The acoustic elements are active only for longitudinal deformations and have zero shear stiffness. The added mass approach does not account for the compressibility of the reservoir water and the use of fluid or solid elements presents oft computational issues. The infinite end of the reservoir was modelled in this analysis with acoustic infinite elements. The effectiveness of the acoustic elements to model the reservoir was shown by other researchers [8]. The interface between the dam and the reservoir can be modelled either with tie constraints or with acoustic-solid interface elements. Both techniques transform the acceleration of the solid medium into pressures in the fluid medium. The dam and the foundation were modelled with two types of elements; plane strain elements with pore pressure as additional degrees of freedom and plane strain elements with temperature as additional degrees of freedom. As the finite element program does not offer elements with both temperature and pore pressure as degrees of freedom for a two dimensional analysis, the overlay technique was used. With the overlay technique, the different types of elements share the same nodes. At the one type of the element a very small young modulus was assigned and the stiffness of the element is given by the other type of the element. Care must be taken for not counting twice gravity loads because both elements material density need to be assigned. If no density is assigned for the pore pressure elements, then we obtain only the excess pore pressure and we don't have the results for the effective stresses. Just before the dynamic analysis the one element type must be taken away from the model, otherwise the double density will affect the wave propagation and thus the seismic excitation. The anchor was modelled with truss elements with temperature degrees of freedom. The joint between the anchor and the dam/foundation was achieved by constraining the translation degrees of freedom at both ends of the anchor. For this analysis was assumed that the dam is tied with the rock foundation and that no separation can occur. The finite element types used are showed in Figure 7.

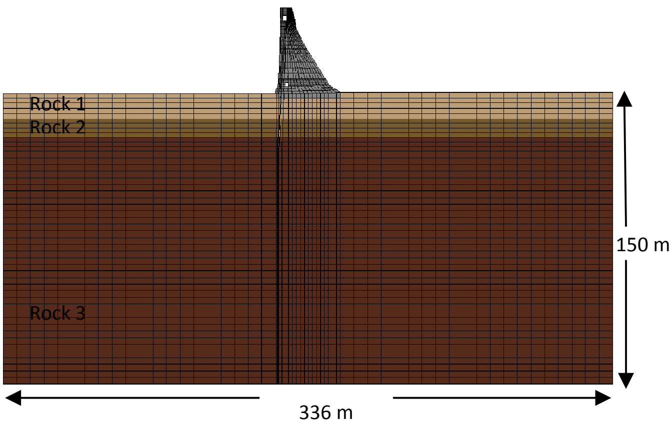


Figure 8: The dimensions of the FE model.

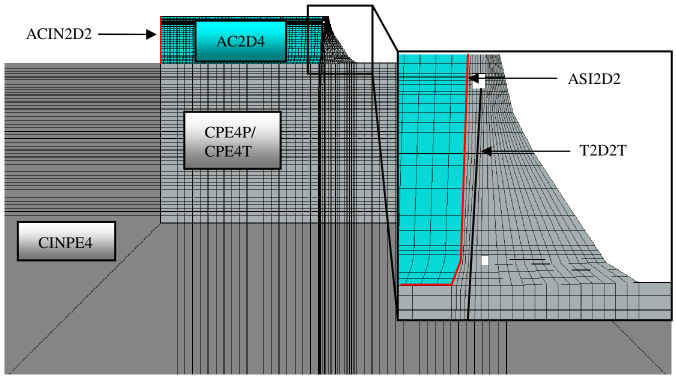


Figure 9: The finite element types.

TABLE 3: MECHANICAL CHARACTERISTICS OF THE FINITE ELEMENT MODEL

| Model Part | Permeability $k_f$ (m/s) | Young Modulus E (Gpa) | Poisson Nr. v | Density (kg/m$^3$) |
|---|---|---|---|---|
| **Rock 1** | $1{\cdot}10^{-5}$ | 5 | 0,25 | 2773 |
| **Rock 2** | $1{\cdot}10^{-6}$ | 5 | 0,25 | 2773 |
| **Rock 3** | $1{\cdot}10^{-7}$ | 5 | 0,25 | 2773 |
| **Cyclopean Concrete** | $1{\cdot}10^{-6}$ | 15,6 | 0,20 | 2200 |
| **Blanket** | $5{\cdot}10^{-7}$ | - | - | - |
| **Anchors** | - | 210 | 0,29 | - |
| **Water** | | K=2,2 | | 1000 |

| Model Part | Conductivity (J/m·K·s) | Specific Heat (kJ/kg·K) | Expansion (1/K) |
|---|---|---|---|
| **Rock 1** | 3,10 | 1000 | $6{\cdot}10^{-6}$ |
| **Rock 2** | 3,10 | 1000 | $6{\cdot}10^{-6}$ |
| **Rock 3** | 3,10 | 1000 | $6{\cdot}10^{-6}$ |
| **Concrete 1** | 3,10 | 1000 | $6{\cdot}10^{-6}$ |
| **Concrete 2** | 3,10 | 1000 | $6{\cdot}10^{-6}$ |
| **Blanket** | - | - | - |
| **Anchors** | 43 | 500 | $12{\cdot}10^{-6}$ |

The analysis showed that the post tensioned anchors contribute to the seismic upgrading of the dam as the damage observed from the two analyses is limited only at the dam heel (Figure 14). The analysis without the seismic upgrading, i.e. without the post tensioned anchors, shows severe damage of the dam not only at the heel but also at the middle of the dam height (Figure 13). However, the mechanical characteristics of the dam with the small compression and tension strength as well as the small Young's modulus play an important role for the damage development although the ground acceleration is moderate.

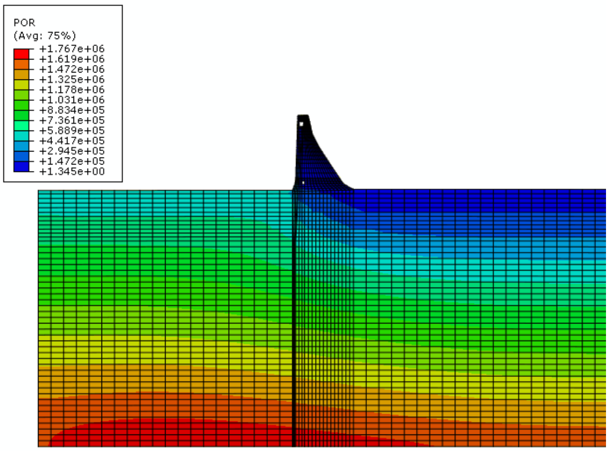


Figure 10: The pore pressures in the foundation rock.

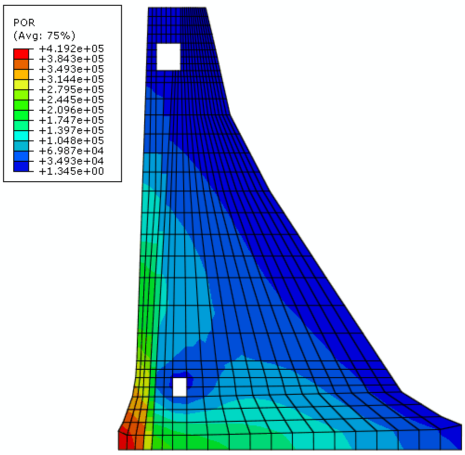


Figure 11: The pore pressures in the dam.

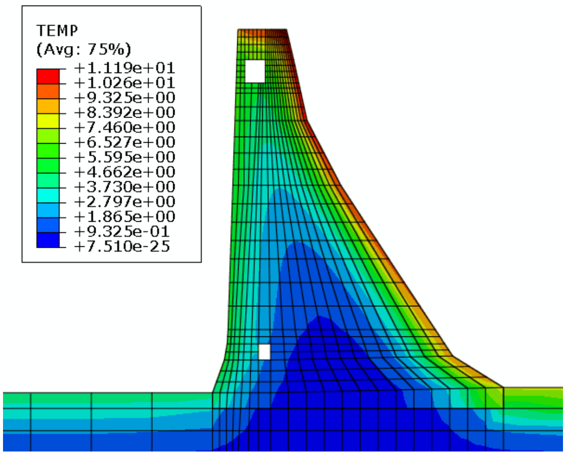


Figure 12: The temperature distribution in the dam.

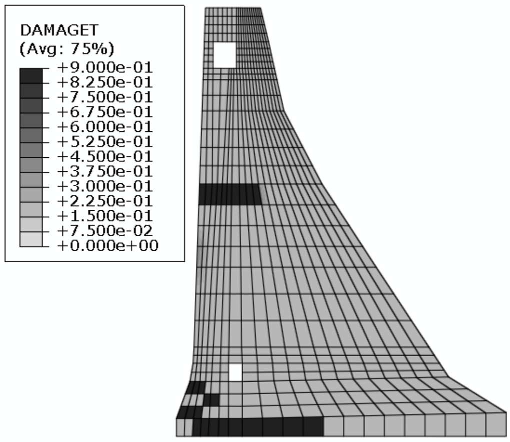


Figure 13: The tension damage for the non upgraded phase of the dam.

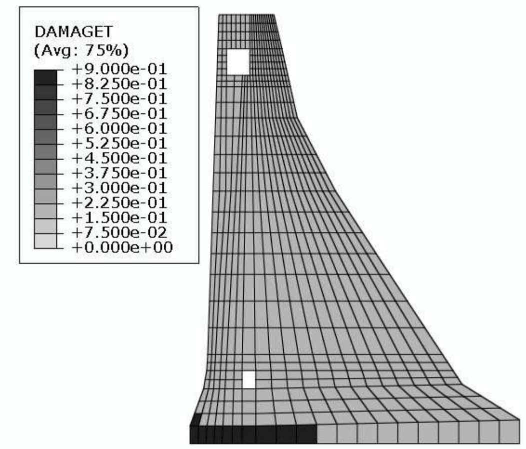


Figure 14: The tension damage for the upgraded phase of the dam with anchors.

## Conclusion

The seismic upgrading of an old cyclopean concrete dam with post tensioned anchors was shown to be effective up to a grade. The damage is limited to a smaller region of the dam. The advantage of the post tensioning as a seismic upgrading method is that the anchors add negligible weight to the dam, so no additional inertial forces are developed. The compression stresses due to the anchors eliminate a big part of the tension stresses caused by the overturning moment.

## Acknowledgements

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