Vienna Congress on Recent Advances in
Earthquake Engineering and Structural Dynamics 2013 (VEESD 2013)
C. Adam, R. Heuer, W. Lenhardt & C. Schranz (eds)
28-30 August 2013, Vienna, Austria
Paper No. 58

The influence of post-tensioned anchors to the seismic behaviour of an old masonry gravity dam

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Abstract. The dynamic analysis and specially 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 built of rough masonry, mainly in the early 20th century when the use of concrete was not yet widespread. The dynamic behaviour of these dams was not thoroughly investigated, and the seismic calculations in seismicity areas were carried out often under simplistic assumptions. After about a century, many of these dams are still in use and must fulfil the requirements of 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 behaviour of such structures during an earthquake.

The article deals with the seismic analysis of a gravity dam with a height of about 50m, which was built of rough masonry in the early 20th Century. First, the properties of the material are analysed 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, uplift pressures and thermal loading, are taken into consideration. The model is taking into account the base joint and the pore pressures in the crack after the fracture has occurred. The analysis is performed for two seismic excitation cases; with and without the post tensioned anchors. 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.

Keywords: Masonry and concrete gravity dams; Soil-Fluid-Structure Interaction; Nonlinear seismic analysis; elasto-plastic constitutive law

1 INTRODUCTION

1.1 Material properties of old gravity dams

Many dams in Germany, Italy, France and Spain were constructed at the beginning of the 20th century. All these dams have some common characteristics: they are slightly curved but they undertake the loads much more as gravity rather than arched dams and they are constructed with big or huge rubble stones bonded with a weak truss-lime mortar. It is a very difficult task to estimate the mechanical characteristics of this material. Many studies handle this material as rubble masonry, other as mass concrete and other are using constitutive laws from rock mechanics. There are also various methods to estimate the mechanical characteristics of such a dam body; deformation and vibration monitoring, bore core taking etc. These methods have their advantages and disadvantages, i.e. the monitoring can estimate the modulus of elasticity of the dam but sometimes it is not an easy task to differ the elasticity of the dam and the rock's one, the ultrasonic scans can find weak areas or indicate the deterioration in the dam body, the bore cores are the most straightforward method to find the mechanical characteristics but is applicable and makes sense only when the material consists of small aggregates compared with the core's diameter. However, it is widely expected that such dams have very low strength characteristics and elastic modulus.

1.2 Constitutive law of the material

The analysed dam was built in the 1914, has a height of about 50 m and a length at the crest of about 400 m. It consists of huge greywacke rubble stones, with a size of about 50-60 cm, bonded with truss-lime mortar. As the common bore cores fail to abstract a representative sample of the dam body, the idea was to estimate numerically the mechanical characteristics of the material. The mechanical characteristics of the mortar and the stones, such as the modulus of elasticity, compression and tension strength, can be obtained for each material separately from laboratory tests. Then a cube with dimensions 2,5 m × 2,5 m was modelled with Abaqus, where the stones were also modelled as discrete parts. The dimensions of the cube were chosen in order to fulfil the Hill's theorem, according to which a representative volume element (RVE) must be only a small part of the whole domain investigated. Here, about six stones fit in the RVE's width and about fourteen RVEs fit in the dam's basis of 36 m.

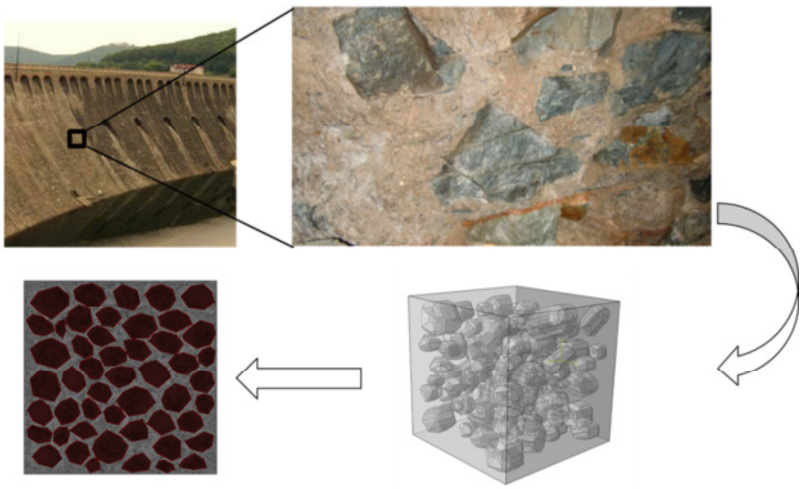

Figure 1. The Representative Volume Element (RVE)

Table 1. Mechanical characteristics of the Representative Volume Element

	E	v	f_c	f_t
	GPa	–	MPa	MPa
Stones	40.0	0.25	–	–
Mortar	8.0	0.20	17.6	1.5
ITZ	4.8	0.20	10.6	0.9

The finite element model consists of rectangular and triangle plain strain elements with reduced integration. The stones have only elastic characteristics whereas the concrete damage plasticity model was assigned to the mortar and the interfacial transition zone. The interfacial transition zone (ITZ) was modelled with continuum elements for computational efficiency. The RVE was subjected to a numerical splitting test in order to obtain the size depended fracture energy of the equivalent smeared material following the work of J. Šejnoha et al.

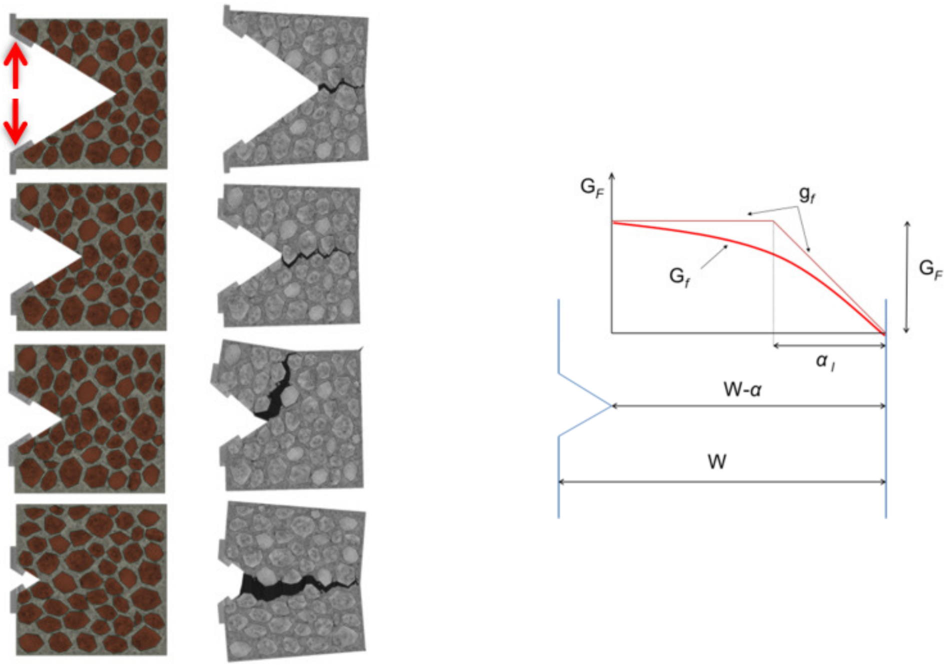

Figure 2. The numerical splitting test (left) and the illustration of size depended fracture energy (right).

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

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

The fracture energy was calculated to be 26 N/m. The characteristics of the homogenized material are presented at the Table 2. The smeared material was also modelled with the concrete damage plasticity model.

Table 2. Mechanical characteristics of the homogenized material

	E	v	f_c	f_t	G_F
	GPa	–	MPa	MPa	N/m
Rubble masonry	15.6	0.20	23.4	1.0	26

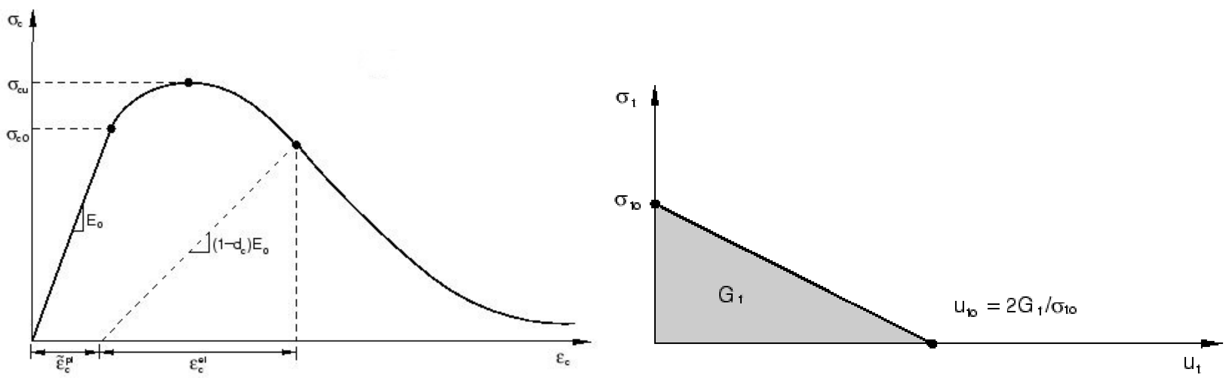

Figure 3. The concrete damage plasticity model. Compression softening (left) tension softening (right).

2 THE SEISMIC ANALYSIS

2.1 Analysis' steps

The analysis was performed with a 2D plain strain model with the commercial finite element program Abaqus. 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. At the beginning the internal stresses of the rock due to its self-weight were calculated and applied as initial conditions to the model. These stresses are in equilibrium with the self-weight of the rock mass and they don't cause strains or deformations at the foundation. After establishing the stress state of the rock before the dam construction, the self-weight of the dam is applied to the foundation by activating the dam part and by applying gravity loading. Subsequently, the hydrostatic pressures of the full reservoir are applied on the dam and the rock foundation. Then the pore pressure distribution is calculated for the rock foundation and the dam. Both materials are considered permeable despite the very small void ratio. The presence of water in the voids causes effective stresses, which play significant role for the start of cracking. 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 temperature conditions corresponding to winter and summer.

As a realistic stress distribution is established for the characteristic quasi permanent loads, there is no damage or cracks observed in the dam.

2.2 The seismic analysis

According to the German legislations (DIN 19700) the dams of the category 1 (in the category 1 are listed the dams with a height bigger than 15 m or with a reservoir volume greater than 1.000.000 m^3) have to be designed for an earthquake with an annual probability of exceedance of 4×10^{-4}. Taking into consideration the service life of the dams in Germany (100 years) this leads to a return period of 2500 years. An estimate of the peak ground acceleration using the standard seismic map of Germany, which is designed for a return period of 475 years, and the importance factor, is not always realistic. This amplification of the importance factor does not consider the real tectonic situation of the region and both conservative and non-conservative peak ground accelerations can be calculated. In order to overcome this limitation, seismic hazard maps with the desired return period shall be used. Despite the small seismicity of Germany, these maps are available by the German Research Centre for Geosciences (GFZ-Potsdam). In this study, for research reasons and in order the accelerations to be comparable with other regions of Europe with higher seismicity, the highest peak ground acceleration is used (about 0,2 g near the city of Tübingen) for the considered return period of 2500 years (Figure

4). According to the German annex of EN 1998-1 the vertical component of the seismic action is 50% of the horizontal component. However, a value of 70% was assumed in order to conform to the values suggested by countries with higher seismicity. Moreover, Germany has adopted the Response Spectrum Type II, which is valid for surface Magnitude Ms < 5,5. The adopted ground acceleration of 0,2 g is about at the limit of this magnitude when applying the most attenuation relationships proposed for Europe, so it is acceptable. The response spectrum, which corresponds to this ground acceleration, was determined according to the German annex of EN 1998-1 and then three matched time histories were generated considering the limitations of EN 1998-1. Then the time histories were deconvoluted by the common used program SHAKE91.

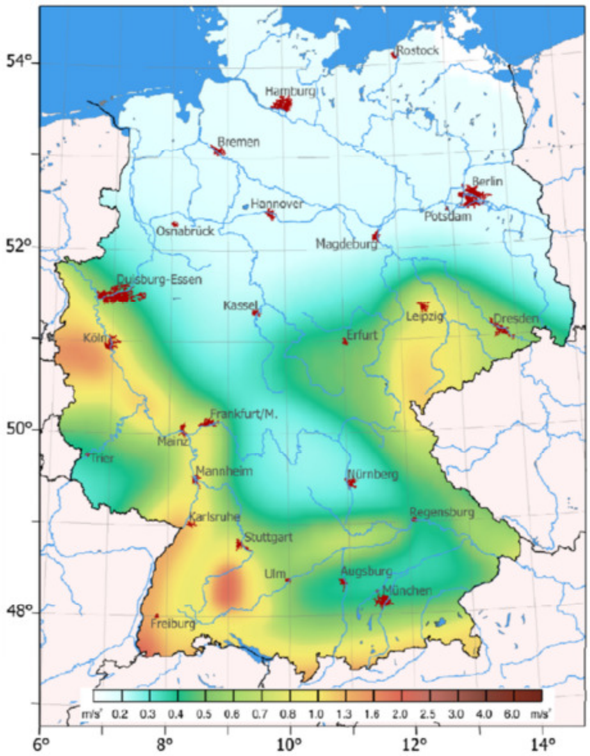

Figure 4. PGA for rock foundation for a return period of 2500 years (GFZ Potsdam).

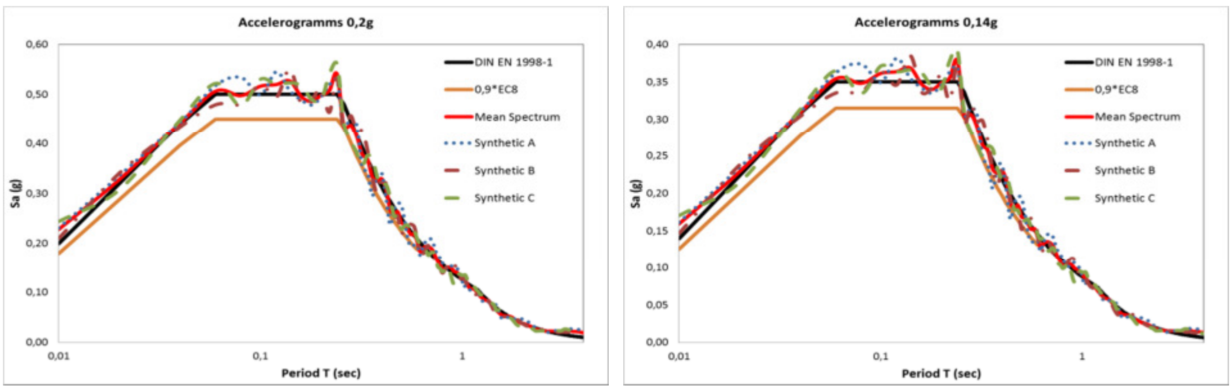

Figure 5. Response Spectra of the synthetics time accelerogramms according to DIN EN 1998-1.

The Figure 5 shows the three synthetic time histories, which were generated following the restrictions of DIN EN 1998-1. According to these restrictions the mean response spectrums of the generated time histories must not have a PGA smaller than the one of the elastic design spectrum, at least three time histories have to be generated, and none of the spectral values of the mean response spectrum must be smaller than the elastic design spectrum at the range between 0,2 × T_1 and 2 × T_1, where T_1 the natural first period of the structure. Two of the generated time histories are shown at Figure 6. For the generation of the time histories were used three different envelope shapes. However, because of the Type II of the elastic spectrum used, where the plateaus of the horizontal and vertical spectra are at the same periods, there is no clear difference at the frequency content of the generated time histories (i.e. no shift but only reduction factor is adopted for the spectrum of Type II).

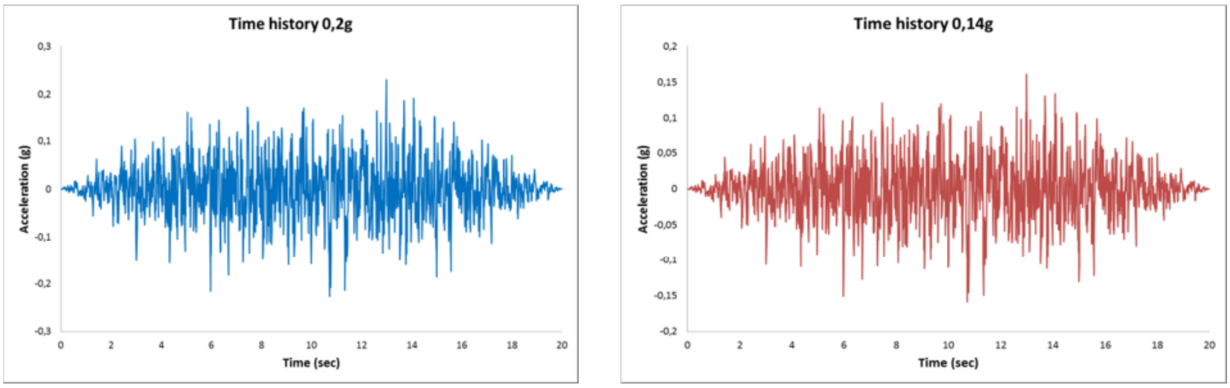

Figure 6. Synthetic record excitation at outcrop rock: horizontal component (left) and vertical (right).

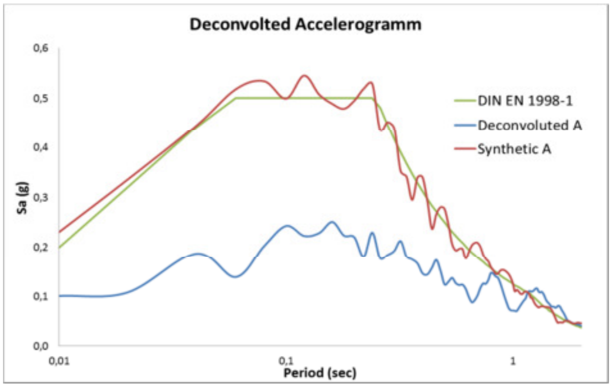

Figure 7. The deconvoluted accelerogramm applied at the base of the finite element model.

The seismic input was applied in form of equivalent forces 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.

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:

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

where d represents the damping factor, ρ the density of the medium and C_s the wave velocity.

However, this wave absorption has to be taken into account for the seismic input. The accelerations are converted to time history velocities and then are multiplied with the shear wave velocity, the density of the medium and the contributing area around the node in order to obtain forces. The factor 2 accounts for the damping at the boundary.

$$F_S(t) = 2 \cdot \rho \cdot C_S \cdot v_S(t) \cdot A \tag{4}$$

$F_s(t)$ is the time history of the equivalent forces at the base nodes, ρ 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 and A the contributing area around the node.

2.3 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 8). The size of the foundation elements was chosen to be smaller than the 1/8 of the wave length to the direction of propagation. 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. 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. If no density is assigned for the pore pressure elements, then we obtain the excess pore pressures.

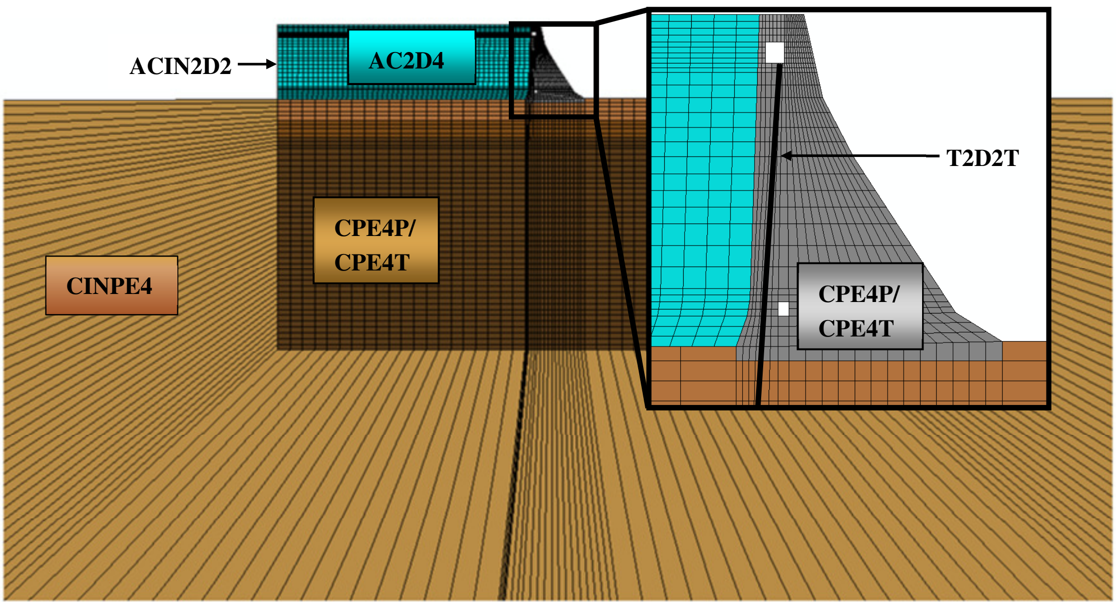

Figure 8. The finite element model with the element types of Abaqus.

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 shown in Figure 8.

Table 3. Mechanical characteristics of the different parts of the finite element model

Model Part	Young modulus	Poisson Nr.	Density	Permeability k_f	Conductivity	Specific heat	Expansion
	GPa	-	kg/m^3	m/s	J/m·K·s	kJ/kg·K	1/K
Rock 1	5.0	0.25	2773	$1\cdot10^{-5}$	3.10	1000	$6\cdot10^{-6}$
Rock 2	5.0	0.25	2773	$1\cdot10^{-6}$	3.10	1000	$6\cdot10^{-6}$
Rock 3	5.0	0.25	2773	$1\cdot10^{-7}$	3.10	1000	$6\cdot10^{-6}$
Masonry	15.6	0.20	2200	$1\cdot10^{-6}$	3.10	1000	$6\cdot10^{-6}$
Blanket	-	-	-	$5\cdot10^{-7}$	-	-	-
Anchors	210	0.29	7850	-	43	500	$12\cdot10^{-6}$
Water	K=2.25	-	1000	-	-	-	-

3 RESULTS

The analysis results are shown at the following pictures. First the pore pressures established after 100 years of service are presented (steady state condition), then the temperatures for the summer and winter period. For the quasi permanent loads, no damage is observed. The post tensioned anchors affect positively the stress condition of the dam as they increase the compression stresses at points where possible tension stresses could occur. For the earthquake analysis without the anchors we observe the crack along the basis of the dam up to a length of 13,83 m (visualised as the tensioned damaged elements). It can be seen from the seismic analysis with post tensioned anchors that the anchors not only upgrade the static stability of the dam but also moderate the cracking on the dam-rock interface during an earthquake event.

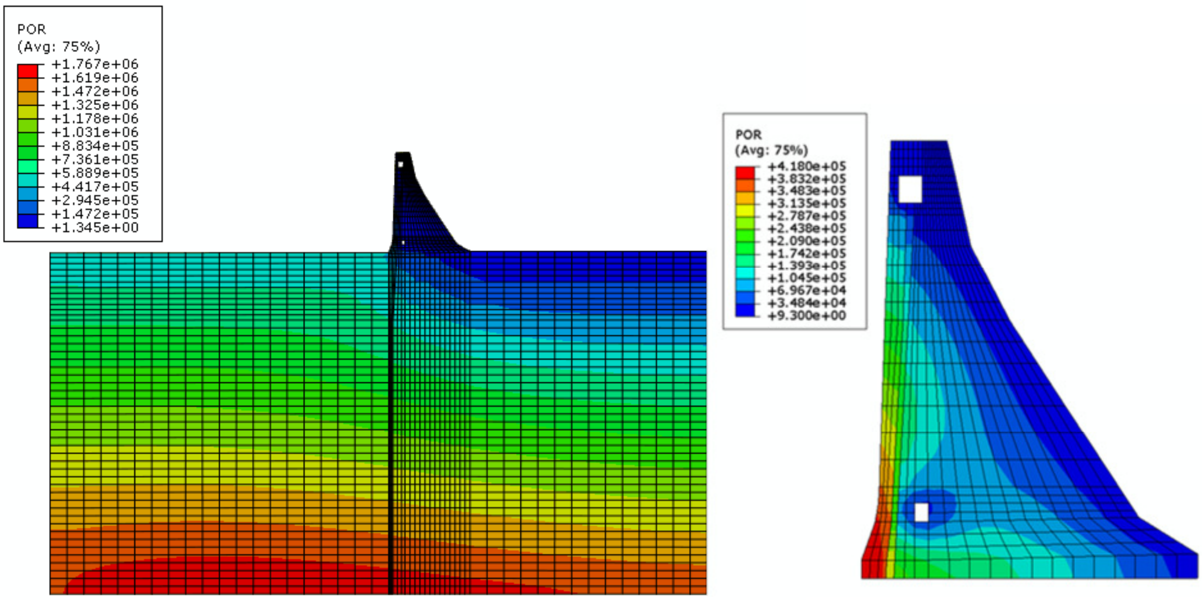

Figure 9. The pore pressures in the foundation rock (left) and in the dam (right).

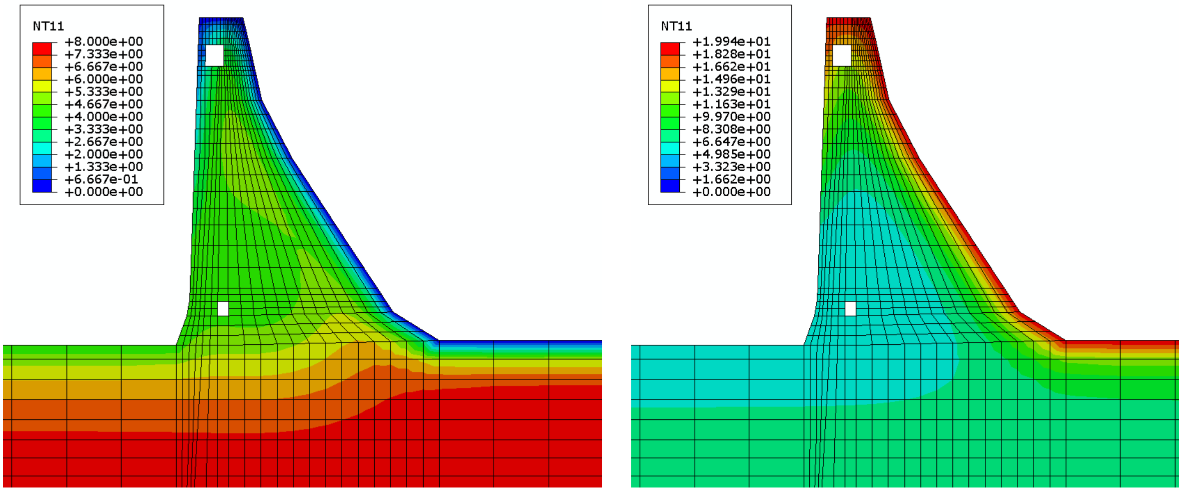

Figure 10. The temperature distribution in winter (left) and in summer (right).

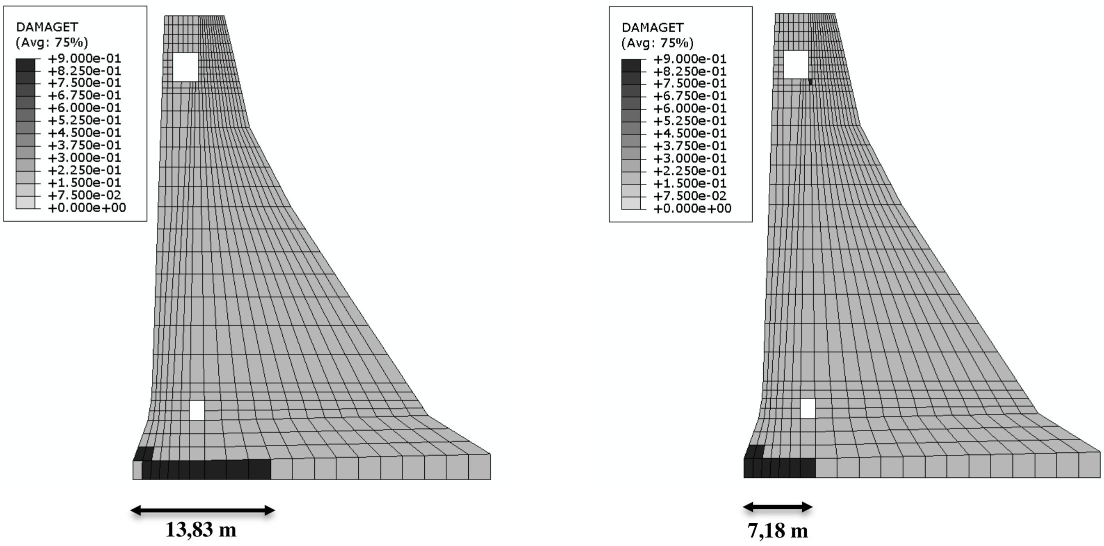

Figure 11. The tension damage of the dam for without upgrading (left) and upgrading with post tensioned anchors (right).

4 CONCLUSIONS

The positive effect of post-tensioned anchors, placed for static upgrading, at an old cyclopean concrete dam was shown also for an earthquake loading. The damage is limited to a smaller region of the dam. The advantage of the post tensioning 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. The position of the damage is indicating that the dam body may not be damaged at all as the tension crack happens at the heel and dam base indicating that the crack will happen at the dam-rock interface. The requirements of the German Standards (DIN 19700) were fulfilled for this conservative seismic loading.

ACKNOWLEDGEMENTS

This paper consists a part of the research project „Earthquake Analysis and Design of Hydraulic Structures“, which is funded by the Federal Waterways and Research Institute of Germany in cooperation with the Institute of Concrete Structures of the Karlsruhe Institute for Technology. The contribution of both participated Institutes and persons involved is highly acknowledged.

REFERENCES

ABAQUS (2011),User’s manual, Version 6.11. Dassault Systèmes Simulia Corporation, Providence RI, USA.

DIN 19700-10: Stauanlagen. Teil 10: Gemeinsame Festlegungen. (2004). Beuth, Berlin.

DIN 19700-11: Stauanlagen. Teil 10: Talsperren. (2004). Beuth, Berlin.

Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. German version EN 1998-1:2004 + AC:2009

Grünthal, G., Bosse, Ch. and Stromeyer, D. (2009) Die neue Generation der probabilistischen seismischen Gefährdungseinschätzung der Bundesrepublik Deutschland Version 2007 mit Anwendung für die Erdbeben-Lastfälle der DIN 19700:2004-07 „Stauanlagen”. Scientific Technical Report STR09/07.

Idriss, I. and Sun., J. (1992). User's Manual for SHAKE91, “A computer program for conducting equivalent linear seismic response analysis of horizontally layered soil deposits.” University of California, Davis: Davis, CA.

Kuhlmeyer, R. and Lysmer, J. (1973). Finite element method accuracy for wave propagation problems. *Journal Soil Mechanics and Foundations Div*., ASCE, 99(SM5), pp 421-427.

Lee, J., and Fenves G.L. (1998), Plastic-damage model for cyclic loading of concrete structure. *Journal of Engineering Mechanics*, **124** **(8)**: 892–900.

Mejia, L., Dawson, E. (2006). Earthquake deconvolution for FLAC. In: Hart & Varona(eds.), *4th International FLAC Symposium on Numerical Modeling in Geomechanics – 2006*, Paper: 04-10

Muto, M., von Gersdorff, N., Duron,Z. and Knarr, M. (2012). Effective modelling of dam-reservoir interaction effects using acoustic finite elements, in Proceedings of Innovative Dam and Levee Design and Construction for Sustainable Water Management, *32nd Annual USSD Conference*, New Orleans, Louisiana, April 23-27, 2012, pp 1161-1168.

Reimer, R. (1973). Deconvolution of seismic response for linear systems. Report No EERC 73-10, University of California, Berkeley.

Sarkar,R., Paul, D.K. and Stempniewski,L. (2007). Influence Of Reservoir And Foundation On The Nonlinear Dynamic Response of Concrete Gravity Dams. *ISET Journal of Earthquake Technology*. **44(2)**, pp. 377–389.

Šejnoha, J., Šejnoha, M., Zeman, J., Sýkora, J. and J. Vorel (2008). Mesoscopic study on historic masonry. *Structural Engineering and Mechanics* **30(1)**: 99-117.

Yong,L., Zhenhuan, S., Zhenguo, T. (2010). Analysis of dynamic response of concrete using a mesoscale model incorporating 3D effects. *International Journal of Protective Structures* **1(2)**:197-217