Dynamic analysis study and its consequences for the design of the concrete face slab of CFRD

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***Abstract*— The study deals with a approx. 140 m high CFRD founded on sound rock in a highly seismic area.**

The dam embankment consists of sandy gravels from the alluvial borrow areas and rockfill obtained from the mandatory excavations, adding up a total volume of approximately 6 million m³. The upstream slope of the dam is 1.5:1.0 (H:V) whereas the downstream slope is on average 1.6:1.0 (H:V) - variable between 10m wide berms (access road). The crest is 8.40 m wide and 570 m long.

This paper presents the study of the dynamic analysis on a three-dimensional finite difference model of the dam and its effects on the concrete face slab. The high seismic hazard of the site, which considers a PGA of 0.507g for the Maximum Credible Earthquake and 0.344g for the Design Earthquake, leads to the potential risk of high permanent displacements corresponding to the maximum level of acceptance (1% of the height of the dam). In full reservoir condition and due to the high rigidity of the concrete face, tensile stresses build up in the slab inducing the upper third of the face of the dam to crack when adopting the concrete strength limits based on the current practice in structural analysis of CFRDs. Consequently, high leakage may occur impacting on the downstream slope stability.

In order to estimate the cracking and to control the leakage potential, a parametric study is performed for the dimensioning of the slab by varying the amount of reinforcement, including horizontal joints and a vertical drain to improve the permeability of the dam materials. The study indicates that the steel reinforcement required to withstand the seismic effects is much higher than that adopted in the usual design practice. The structural behavior can be improved with non-usual mitigation measures for the concrete slab (i.e. including horizontal joints, additional reinforcement), however, there is lack of knowledge in the state of the art about its behavior because there are no precedent analyses of embankments with similar design conditions. Furthermore, there is no evidence of large CFRDs that have had experienced such high ground motion at full impounding level.

Keywords-CFRDs, Seismicity, Dynamic Analysis, Design

I. INTRODUCTION

Rockfill dams with concrete face are suitable for many different site conditions. They are being widely used around the world, tending to increase their height as new projects are developed. Likewise, CFRDs are often built also in highly seismic regions.

Still today their design is mainly based on empirical rules and on the experience of the designer. While most of them perform well under static conditions, recent earthquakes of higher magnitude have caused damages to the concrete face of several dams.

The progresses over the last 10-15 years in computation technology and software are progressively enhancing the development of the design of earth and rockfill dams. Sophisticated numerical modelling suggests that the empirical design of the concrete face is not sufficiently reliable for all loading conditions.

While the design approaches under static conditions can easily be verified at real scale, the numerical modelling of the dams under seismic conditions comes along with bigger uncertainty. For large CFRDs built in highly seismic regions there is not much evidence about their performance under seismic loading conditions similar to the design cases of this study. The design approaches can therefore not be calibrated directly on the observed behavior.

Several large CFRDs have experienced strong ground motion and their behavior and performance have been analyzed thoroughly by several authors. All these cases can, be considered as a good reference but are not representative of their behavior since the ground motion experienced did not correspond to the MCE (Maximum Credible Earthquake) of the site, and of even more importance, the reservoirs were only partially impounded.

In the present paper the design performance of a large CFRD in a highly seismic region is presented. The results indicate that the design of the concrete face based on three-dimensional numerical modelling differs significantly from the design based on empirical approaches. Those differences are analyzed and discussed.

II. Description of the Dam and the Foundation

A 140 m high CFRD in a highly seismic region is object of the present study. The dam body with a total volume of approximately 6 million cubic meters consists mainly of sandy gravels from the alluvial borrow areas and rockfill obtained from the excavations required for construction.

The upstream slope of the dam is 1.5:1.0 (H:V) whereas the downstream slope is on average 1.6:1.0 (H:V) - variable between 10m wide berms (access road). The crest is 8.40 m wide and 570 m long.

The foundation of the dam consists of igneous rocks of volcanic origin interbedded with sedimentary rocks. Rock outcrops and site investigations show a good quality of the base rock with only few discontinuities. A relatively high stiffness (elastic modulus E = 35 GPa) has thus been assigned to the base rock.

In contrast to the stiff base rock the stiffnesses of the dam materials are by factor 100-150 lower. A deformation modulus depending on the confinement stress was considered for the dam materials and discussed hereafter.

III. Numerical 3D Calculation Model

For the analyses of the behavior of the dam in static and dynamic conditions, a tree-dimensional calculation model was used. It is made of approximately 177’000 volumetric elements.

The finite-difference code FLAC 3D in its current version 6.0 was used. FLAC is indicated especially for geotechnical and geomechanical problems in static as well as in dynamic conditions.

In order to avoid border effects on the calculation, a sufficiently large extension of the modelled area is required. In the upstream-downstream direction the model has an extension of 1’400 m, while in the left-right direction 1’000 m. In the river flow direction there is a distance of 550 m between the dam and model boundaries, while the lateral distance is 220 m. Underneath the dam base 270 m of bedrock is modelled.

A view on the numerical 3D model is provided in Fig. 1 while in Fig. 2 a vertical section of the dam with the zoning is presented.

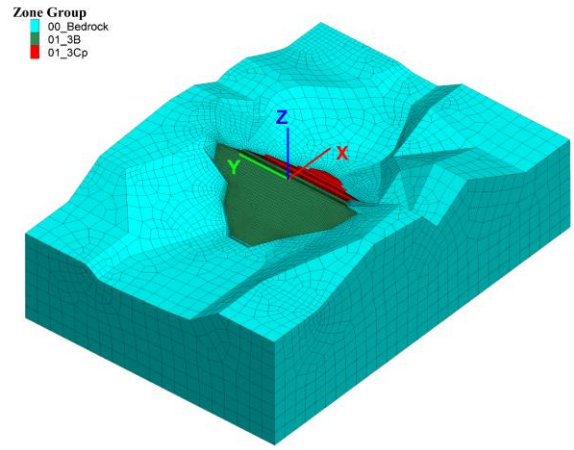

Figure 1. View on three-dimensional calculation model (downstream on the right, reservoir on the left)

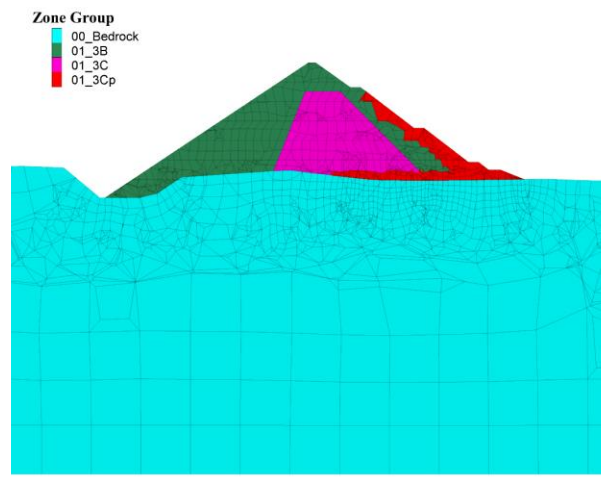

Figure 2. Cross-section of the three-dimensional calculation model (downstream on the right, reservoir on the left)

An elastic-perfectly plastic constitutive model with a Mohr-Coulomb failure criterion was adopted.

Stiffness as well as strength parameters of the dam materials were defined to depend on the confinement stress.

For the elastic modulus the following expression (1) was used.

$$E = E_{ref} \cdot (\sigma_3 / p_a)^n \tag{1}$$

- E = Young’s modulus for a confinement pressure σ_3;
- E_{ref} = reference value of Young’s modulus for the atmospheric pressure p_a
- p_a = atmospheric pressure (100 kPa)
- σ_3 = confinement pressure
- n = empirical coefficient

For the friction angle expression (2) was used and limited to a maximum value of 60° to avoid unrealistic assumptions.

$$\phi' = \phi_0' - \Delta\phi' \cdot \log(\sigma_n' / p_a) < 60° \quad (2)$$

- ϕ_0' = friction angle for a confinement pressure σ_n'
- $\Delta\phi'$ = inclination of curve in the plane (ϕ_0', $\log(\sigma_n' / p_a)$)
- σ_n' = confinement pressure
- p_a = atmospheric pressure (100 kPa)

For the dynamic analyses the constitutive model was completed with a hysteretic damping model. This model was calibrated on the decay law for the shear modulus in function of the shear strain, which characterizes the dam materials when subject to cyclic loading.

The dynamic shear modulus for small strains was calculated with expression (3).

$$G_0 = 70 \cdot K_{2,max} \cdot (\sigma_m)^{0.5} \quad (3)$$

- G_0 = dynamic shear modulus for small strain
- $K_{2,max}$ = material constant
- σ_m = average principal stress

The reduction of the shear modulus in function of the shear strain and the hysteretic damping function are presented in Figure 3. .

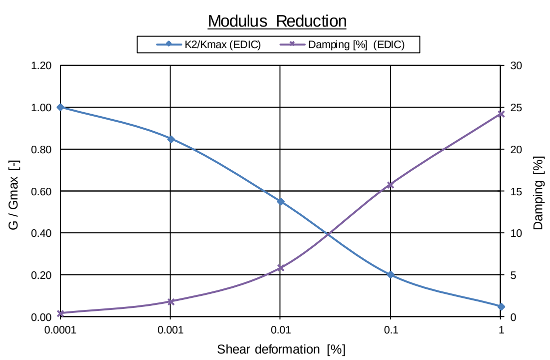

Figure 3. Shear modulus in function of the shear strain and the hysteretic damping.

Since the hysteretic damping is not suitable for small strains, a Rayleigh damping of 0.2 % was adopted for small strain conditions.

To keep calculation time down to reasonable values, the concrete face was modelled explicitly with shell elements only for the calculations for static and quasi-static conditions.

Under dynamic conditions, calculation time would be excessive if the concrete face is included in the model, given the high non-linear behavior of the reinforced concrete. To reach a satisfying precision in the calculation process of cracking and consequent stiffness reduction within the face, small elements would be required. For dynamic conditions the design of the face was done with a separate model, based on the calculated deformation of the dam. More details are given later in this paper.

IV. Stability Analysis in Static and Quasi-Static Conditions

Since the properties of the dam materials (resistance and stiffness) were defined as stress-dependent, the stress-history of the dam was reproduced in the calculation model to obtain a proper basis especially for the dynamic analyses.

The construction process is therefore of importance and the self-weight of the dam, by definition a static load, becomes a quasi-static loading condition. The material constituting the dam body was placed in layers of varying thickness depending on the material type. Each layer was adequately compacted before the next one was placed.

A total of 7 phases were simulated in the numerical model and the material properties were recalculated after each phase. The construction sequence in layers is shown in Figure 4. .

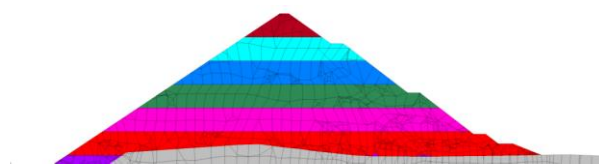

Figure 4. Construction phases of the dam as conisdered in the numerical model (reservoir on the left side)

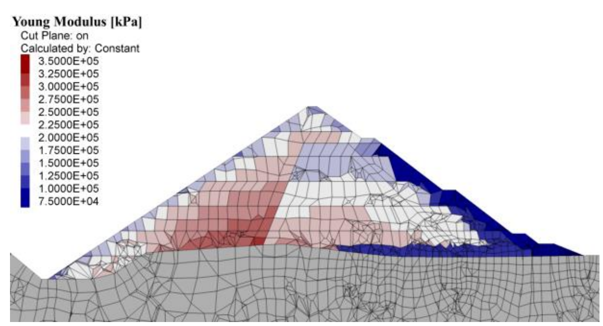

Figure 5. Elastic moduli of dam body materials at the end of construction (reservoir on the left side)

The elastic moduli of the dam body materials at final stage after construction of the dam are shown in Figure 5. while the friction angle is presented in Figure 6. When interpreting the results, one has to keep in mind the dam zoning shown in Fig. 2.

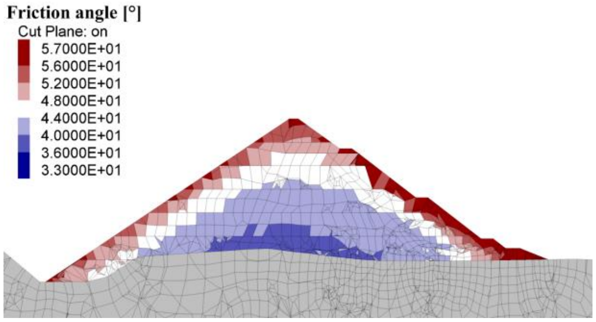

Figure 6. Friction angle of dam body materials at the end of construction (reservoir on the left side)

In Fig. 7 the vertical displacement following the stress-induced compaction of the dam body materials at the end of construction is presented. Settlements concentrate in the central part of the dam.

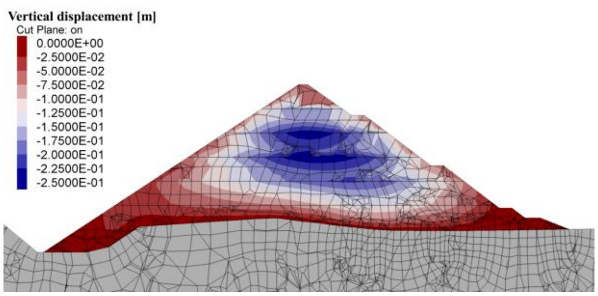

Figure 7. Vertical displacement at the end of construction (reservoir on the left side)

Since the material properties were defined as stress-dependent, also the first filling of the reservoir was simulated in steps. For the present case, 10 steps were chosen and after each step the material properties were recalculated.

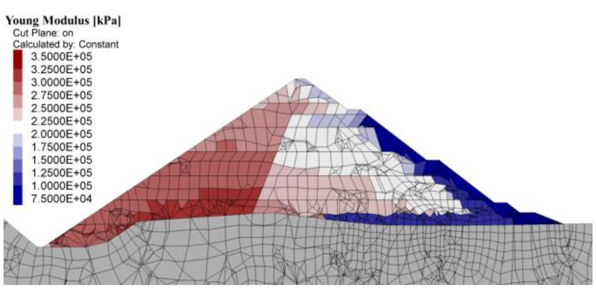

Figure 8. Elastic moduli of dam body materials after reservoir filling (reservoir on the left side)

The elastic moduli of the dam materials at the end of the reservoir filling are shown in Fig. 8. The increase of the moduli upstream (left of the dam) due to the water load on the concrete face can be clearly noted.

Fig. 9 shows the total displacements of the dam after reservoir filling. As one can note, settlements concentrate principally beneath the concrete face in its central part. However, as the displacement vectors in Fig. 9 indicate, the upper part of the dam body moves also downstream. This movement has to be followed by the concrete face which is nearly glued to the dam body by the water pressure.

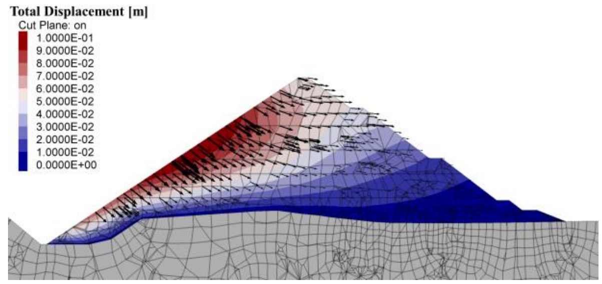

Figure 9. Total displacement of dam body after reservoir filling (reservoir on the left side)

V. STABILITY ANALYSIS IN DYNAMIC CONDITIONS

For the behavior analysis of the dam in dynamic conditions, the constitutive model was completed with a hysteretic damping model. A detailed description is provided in chapter III of the present paper.

The stability analyses were carried out by means of a time-history calculation considering real strong motion seismograms in south America scaled to a MCE (Maximum Credible Earthquake, PGA = 0.51 g, duration 110 seconds, Arias intensity 12 m/s) and to an OBE (Operational Basis Earthquake, PGA = 0.344 g, duration 110 seconds, Arias intensity 5.5 m/s). The input seismogram for the horizontal component for the OBE is shown in Fig. 10.

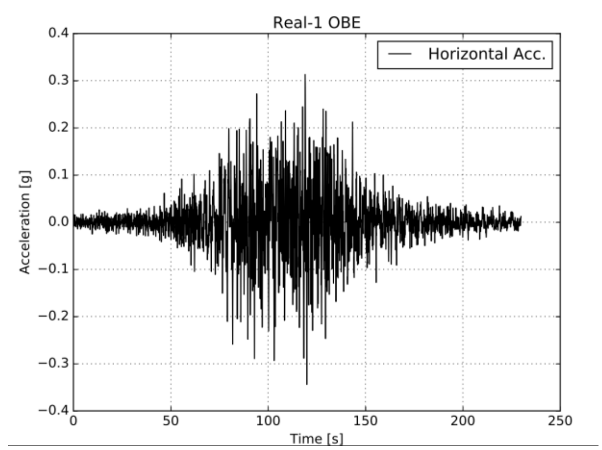

Figure 10. Horizontal input seismogram for the OBE

The dynamic response of the dam highly depends on the water load on the concrete face. In case of a full reservoir, the spectral amplification of the PGA over the dam height is more important than for a low reservoir level. This is shown in the following Fig. 11 (MCE, full reservoir) and Fig. 12 (MCE, empty reservoir), respectively. The figures show the spectral amplification of the PGA for different heights over the dam body.

As one can note, spectral amplification at crest level reaches 3.6 times PGA in case of a full reservoir, while it reaches only 3.4 times PGA in case of an empty reservoir. Spectral amplification of PGA over the dam height is generally higher over all periods of oscillation.

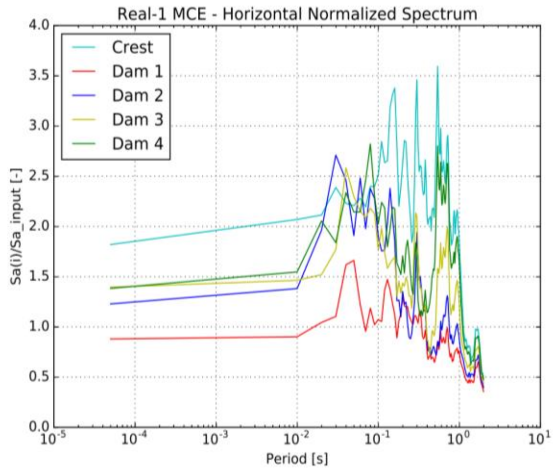

Figure 11. Spectral amplification of PGA for an MCE (horizontal component) – Full reservoir

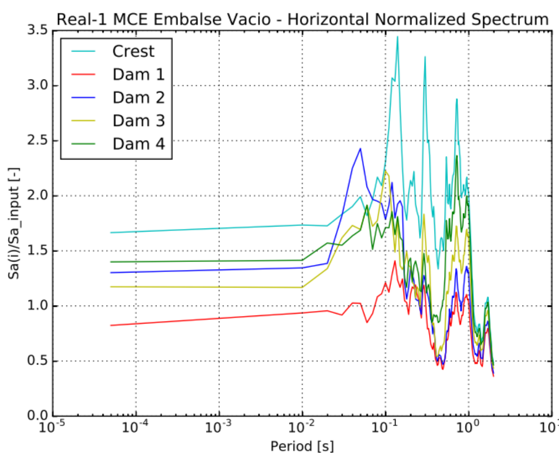

Figure 12. Spectral amplification of PGA for an MCE (horizontal component) – Empty reservoir

As already stated in the introduction, the fill level of the reservoir is an important factor for the dynamic response of a CFRD and is crucial for the design of the concrete face, as discussed later in this paper.

An important aspect for a correct interpretation of the behavior of the concrete face is the permanent deformation of the dam caused by strong motion. The results obtained for the permanent deformation of the dam are shown in Fig. 13 (MCE, full reservoir) and in Fig. 14 (MCE, empty reservoir).

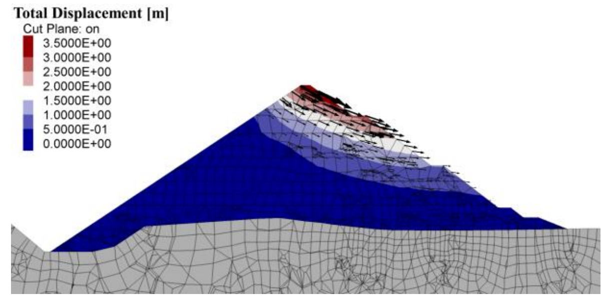

Figure 13. Permanent deformation of the dam for an MCE and full reservoir (reservoir on the left side)

In case of an empty reservoir the deformation is mainly vertical, i.e. settlement of the crest, because instabilities occur towards downstream and upstream cyclically during an earthquake. At full reservoir level only instabilities towards downstream can occur due to the acting water pressure, leading to a downstream movement.

The settlement of the crest reaches approx. 1% of the height of the dam in case of an MCE and full reservoir, which is considered acceptable. In case of an empty reservoir the settlement of the crest reaches nearly 1.5% of the dam height, being irrelevant for the safety of the dam since the reservoir is empty.

The permanent horizontal displacement, however, exceeds the vertical one in case of a full reservoir. The consequences of such permanent deformation on the concrete face are discussed later in this paper.

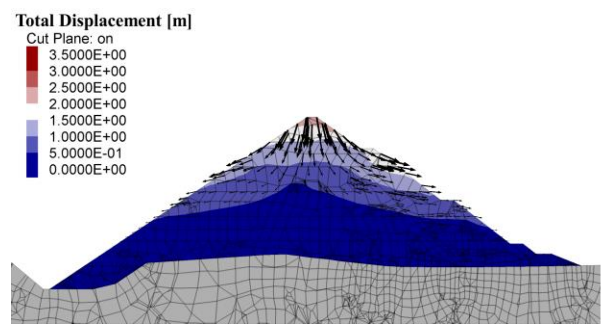

Figure 14. Permanent deformation of the dam for an MCE and empty reservoir (reservoir on the left side)

VI. BEHAVIOR OF THE CONCRETE FACE

The concrete face of a CFRD is the only sealing element between dam and reservoir. Damaging of the face, i.e. mainly cracking, cause infiltrations into the dam body and has therefore to be avoided as far as possible.

The concrete face is bonded to the dam body in a frictional manner. The amount of friction activated in the interface between face and embankment depends not only on the material properties but also on the self-weight of the face and, in a more important way, on the water load on the face. This is where the reservoir level becomes important again.

Concrete is a relatively stiff material with a low deformation capacity as long as it remains elastic. Under static conditions, an elastic behavior of the face is assumed as the deformation of the dam body after construction of the face is often of little importance. Ongoing settlements cause mainly compressive stresses in the concrete.

In seismic conditions, however, important permanent displacements are to be expected. Now we come to the point where the reservoir level plays an important role.

In case of an empty or partially empty reservoir, mainly a vertical settlement of the upper part of the dam is expected. This downward movement leads to an axial compression of the face and thus to increased compressive stresses. The thickness and the compressive strength of the concrete become the principal influencing factors on whether it comes to damaging of the face or not. Damaging would most likely be a compressive failure of the concrete, as it has been observed in various CFRDs after a strong motion event.

If the reservoir is full or at least in the upper third of the dam's capacity, permanent deformations in downstream direction are to be expected in the upper part of the dam, as shown in the previous chapter. Since the horizontal component of the deformation is bigger than the vertical one, the resultant vector of the deformation is oriented upwards with respect to the normal direction towards the upstream slope of the dam. As a consequence, this leads to a deformation in the axial direction of the concrete face (Fig. 15).

This condition is considered relevant basically for all CFRDs worldwide for which permanent deformations are expected during strong motion and in case of a full or nearly full reservoir.

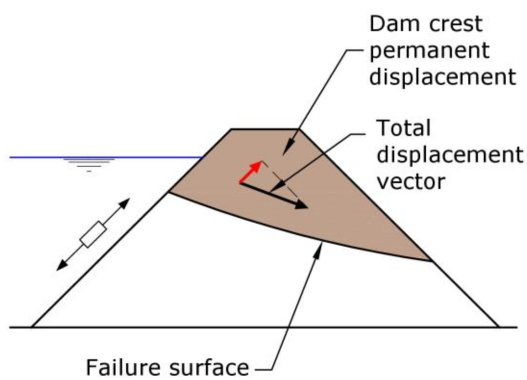

Figure 15. Permanent deformation of the upper part of the dam during strong motion in case of a full or nearly full reservoir.

This axial deformation in the concrete face corresponds to an elongation and causes therefore tensile stresses in the face. Considering the magnitude of permanent deformation in the analyzed case, the deformations overcome the concrete elastic limit resulting in cracking on the face. To control the crack pattern and to avoid excessive leakage through the face, steel reinforcement is used.

The stresses building up in the face cannot directly be deduced from the deformation of the dam as the tensile strain in the face depends on the friction between face and dam body. Where the frictional resistance is exceeded, sliding of the face occurs and tensile strain decreases. Where the face, however, remains "glued" to the dam by friction, the deformation of the dam is directly translated into tensile strain.

The tensile stress σ_t at a given point is defined as shown in Fig. 16, where L is the length of the element from the crown, s the thickness of the face, α the inclination of the face and τ the mobilized friction along the element.

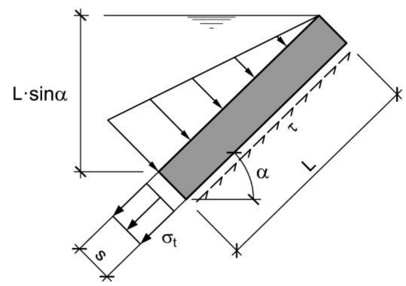

Figure 16. Scheme of the interaction between concrete face and dam body.

A further factor determining the behavior of the face and the cracking pattern is its axial stiffness. When subject to compression, the stiffness depends on the concrete only and is constant. When subject to tension the axial stiffness depends on the interaction between the concrete and reinforcement or in other words, it depends on the crack pattern (crack spacing, crack opening). The behavior of the concrete face becomes therefore highly non-linear when in tension.

Fig. 17 illustrates the non-linear evolution of axial stiffness of reinforced concrete with increasing strain and stress.

The first branch (A) corresponds to the elastic stiffness of the uncracked concrete. Once exceeding the tensile strength of the concrete, it comes to a progressive cracking and activation of the reinforcement (branch B). After formation of all cracks it comes simply to a progressive opening of them and the axial stiffness corresponds to that of the completely cracked cross-section (branch C). When reaching the yield strength of the steel, the axial stiffness significantly decreases and a small increase of tensile stresses up to the tensile strength of the reinforcement steel comes together with significant elongation (branch D).

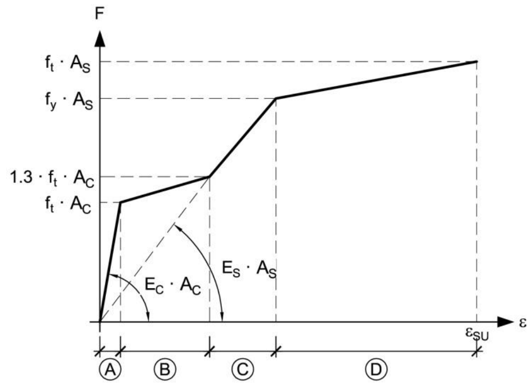

Figure 17. Axial stiffness of reinforced concrete

VII. DESIGN OF THE CONCRETE FACE

As described earlier, the concrete face was explicitly modelled with shell elements only in the calculations for static and dynamic conditions. The concrete face is designed so that for static conditions no cracking is expected and the face remains in elastic conditions.

In dynamic conditions, the assumption of elastic behavior would lead to excessive tensile stresses in the face, as explained in the previous chapter.

Considering the earlier presented permanent displacements of the dam body, damaging of the face is expected. The behavior of the reinforced concrete becomes highly non-linear as its axial stiffness in tension depends on the crack pattern. Simulating this highly non-linear behavior with adequate accuracy in an already highly non-linear three-dimensional time-history, calculation with a model of this size would require an excessive amount of calculation effort and cannot be handled for "every-day" use.

The design of the face was therefore disconnected from the numerical model and an iterative procedure was developed to calculate adequately the strains and stresses along the face, based on the displacement profile of the upstream face of the dam. This is admissible as the dam displacement is not influenced by the presence of the concrete face.

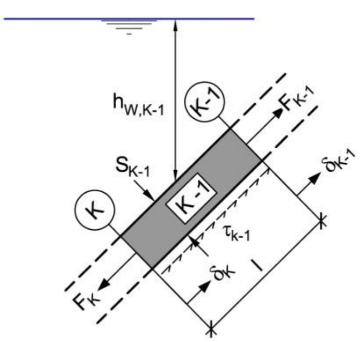

Figure 18. Equilibrium of forces and displacement at any point along the concrete face

The equilibrium of forces and deformations at any point along the concrete face is defined as shown in Fig. 18, where F is the force in axial direction, δ the axial deformation, τ the friction between face and dam body and s the water load on the element.

Given the displacement profile of the upstream dam face, the stress and strain profile along the concrete face can be calculated by integration. The design of the face becomes an iterative process as the reinforcement content plays an important role. Based on the crack pattern (distribution, opening) and the permeability of the dam materials, also the leakage through the concrete face can be calculated.

In Fig. 19 the axial displacement profile along the face is shown for the analyzed case. The profiles were calculated with the numerical three-dimensional model. As one can deduce by the entity of the displacements, the assumption of an elastic behavior for the face would lead to unrealistic and excessive tensile stresses. For the analyzed case the calculated tensile stresses exceeded 40 MPa for a MCE.

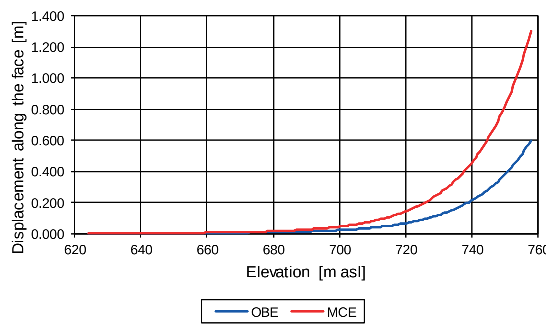

Figure 19. Displacement profiles along the concrete face

As a result, the profiles of stresses, strains, the cracking pattern (location of cracks and crack opening) and the leakages in function of the reinforcement ratio are obtained.

For the analyzed dam the displacements are important already for an OBE and a reinforcement ratio much higher than usual empirical ratios are required to avoid the formation of single cracks with openings of several tenth of centimeters. Even with a reinforcement ratio at the limit of feasibility of 4% the calculated crack opening resulted to be in the order of centimeters.

As the only possibility to limit the crack width, and hence the leakage through the face, and to avoid excessive reinforcement ratios resulted the inclusion/installation of two horizontal joints in the upper third of the concrete face. These joints are provided with a complex sealing system and allow a controlled opening without affecting the performance of the concrete face as principal water retaining element.

As a consequence, the reinforcement quantity could be reduced considerably compared to the solution without horizontal joints. Nevertheless, in the upper third of the face a reinforcement ratio exceeding usual empirical ratios of 0.3-0.5% is still required to limit cracking and leakages to acceptable values.

Fig. 20 illustrates the concrete face and the vertical and horizontal joints. Vertical joints can be of three different types; compression (blue), tension (green) or hybrid (violet), while the horizontal joints are tension joints only (orange).

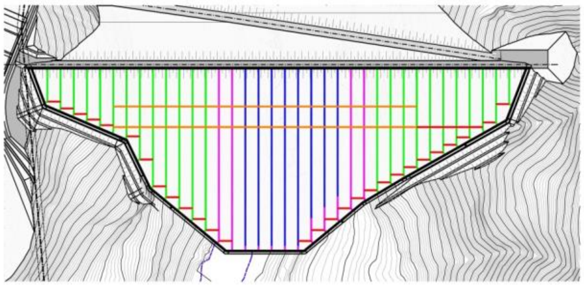

Figure 20. Concrete face with vertical and horizontal joints

In order to identify the most convenient design for the concrete face considering also the possible variability of equivalent permeability of the dam materials, a parametric study was carried out, varying reinforcement ratio as well as permeability. The results for an OBE are shown in Fig. 21.

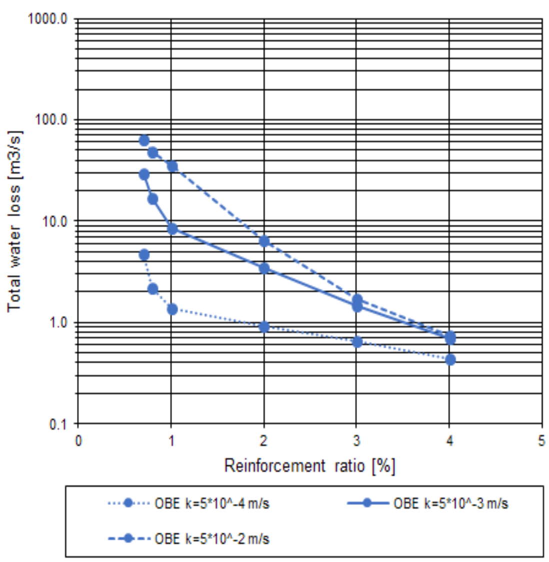

Figure 21. Leakages through concrete face as a function of reinforcement ratio and equivalent permeability of the dam material

In the following figures the results of the design of the concrete face of the analyzed case are illustrated. Design assumption was a ground motion corresponding to an OBE (PGA = 0.344 g), a reinforcement ratio of max. 1%, two horizontal dilatation joints in the upper third of the dam and an equivalent permeability of $5*10^{-4}$ m/s.

Fig. 22 shows the displacement profile of the upstream face of the dam and of the concrete face. As one can observe, the concrete face does not follow the dam deformation in the upper part. With the progressively reducing water load on the face towards the crest, also the frictional forces that can be mobilized are progressively reduced. At a certain point, mainly depending on the reinforcement ratio, the face begins to slide on the dam. In this sliding zone the deformation of the dam is not directly transformed into strain in the face and hence also the tensile stresses do not rise up.

The presence of the two horizontal joints is clearly visible and the joint opening can be read for proper design of the joint and the sealing system.

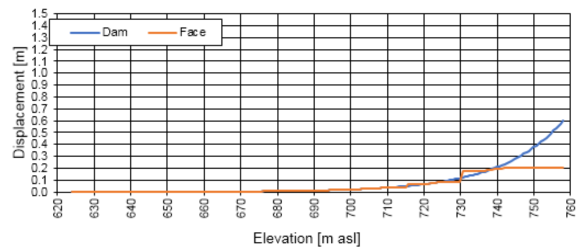

Figure 22. Displacement profile of concrete face and upstream face of dam in axial direction of the face

The axial stiffness of the concrete face expressed as equivalent elastic modulus is illustrated in Fig. 23. Where the concrete is cracked the stiffness is reduced.

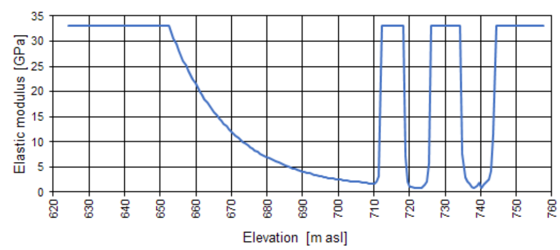

Figure 23. Profile of axial stiffness along the concrete face expressed as equivalent elastic modulus

Fig. 24 shows the equivalent tensile stresses in the concrete face referred to the concrete cross-section. The presence of the horizontal joints is visible also here as the tensile stresses are nil at the corresponding elevations.

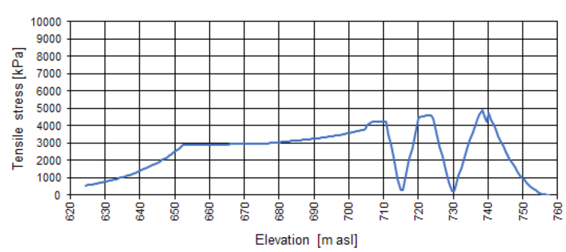

Figure 24. Profile of equivalent concrete tensile stresses along the concrete face

Fig. 25 and Fig. 26 finally show the cracking pattern of the concrete face with the relative crack opening, as well as the resulting water leakages through the cracks.

For the calculation of the leakages the shape of the dam was considered, i.e. to each elevation a different dam width was assigned.

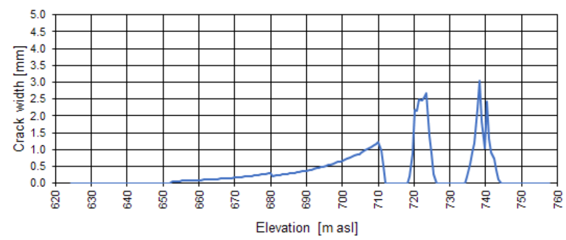

Figure 25. Profile of crack distribution and crack opening along the concrete face

The total leakages that are expected after a ground motion corresponding to an OBE are in the range of 2-3 m³/s and therefore acceptable. The structural integrity and the serviceability of the dam is not affected negatively after an OBE.

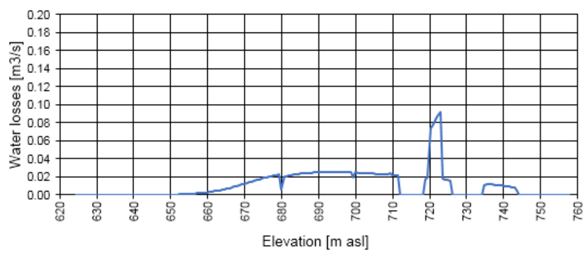

Figure 26. Profile of leakages along the concrete face

VIII. CONCLUSIONS

The numerical dynamic analysis of a large CFRD in a highly seismic region presented in this paper highlights several important aspects regarding dam safety. As shown, the reservoir level at the moment of ground motion is decisive for the expected behavior of the concrete face.

Recent experiences of large CFRDs subject to strong ground motion showed usually a quite good performance also when the structural damages to the concrete face sometimes were substantial. All cases known to the authors have in common, that the reservoir was partially empty during ground motion. There is no evidence of large CFRDs subject to strong ground motion at full impounding level.

The analyses carried out show that, at full reservoir level, the permanent horizontal displacements of the upper part of the dam are important. Since the vertical displacement is smaller, the vector of the residual force induces tensile forces in the concrete face due to the frictional forces between the dam and concrete under full water load.

As a consequence of those tensile forces the concrete face cracks. To limit cracking and hence leakage through the face to acceptable limits, the often used empirical approaches for the design of the concrete face turned out to be not suitable.

In the analyzed case a reinforcement ratio of locally at least 1% together with two horizontal dilatation joints was required to limit cracking and water leakages to acceptable limits.

Up to date there is no common state-of-the-art approach for the design of the concrete face of large CFRD in highly seismic regions and many aspects are still to be analyzed in detail.

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