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COMPARISON BETWEEN 2D AND 3D ANALYSES OF SEISMIC STABILITY OF DETACHED BLOCKS IN AN ARCH DAM

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ABSTRACT

The seismic safety of the 147 m high Gigerwald arch dam in Switzerland was assessed for a Safety Evaluation Earthquake (SEE) with a return period of 10,000 years. The results of the linear dynamic analysis showed that the seismic shaking would cause high tensile stresses in the arch direction in the central crest region in spite of the compressive stresses due to the static loads. As the tensile strength of the vertical contraction joints is quite low, they would most likely open during the earthquake shaking. This could be verified by a 3D nonlinear analysis of a dam model containing these joints.

When the vertical joints are open, the upper portion of a dam monolith behaves as a vertical cantilever, leading to high vertical tensile stresses. Since the tensile strength of horizontal lift joints is usually considerably lower than that of monolithic mass concrete, a horizontal crack may form, causing the upper portion of a dam monolith to become fully detached from the rest of the dam. The dynamic stability analysis of such possibly detached blocks using a simplified 2D approach as well as a more rigorous 3D approach showed that they could undergo some sliding and rocking motions. However, they would remain stable during and after the earthquake. Therefore, it is concluded that the earthquake loading could cause some limited damages, but an uncontrolled release of reservoir water is unlikely. Therefore, the dam satisfies the safety requirements for the SEE.

The comparison of the results of the 2D and 3D dynamic stability analyses shows that the simplified 2D approach is quite conservative, as the frictional resistance at the vertical contraction joint on each side of the detached block is neglected in this approach.

INTRODUCTION

In a linear elastic dynamic analysis of an arch dam subjected to earthquake loading, usually high horizontal tensile stresses are obtained in the arch direction in the central crest area even in a region with moderate seismicity. These tensile stresses would usually exceed the dynamic tensile strength of the vertical contraction joints, which are substantially weaker in tension than the monolithic mass concrete. Thus, the contraction joints would open and cracks would develop at the horizontal lift joints, possibly leading to formation of detached concrete blocks in the crest area, which is subjected to high earthquake accelerations. Such detached blocks could thus undergo sliding and rocking motions during a strong earthquake.

The dynamic stability of a possibly detached block in an arch dam is a highly nonlinear three-dimensional (3D) problem. In view of the difficulty to numerically solve this 3D problem, a simplified procedure using 2D models is usually employed in engineering practice, for instance, as proposed by Malla and Wieland (2006). Such a 2D analysis is conservative, as the frictional resistance on the sides of the detached block is neglected. Due to the recent advances in the software and hardware

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capabilities, it has nowadays become feasible to carry out also a more rigorous 3D dynamic stability analysis of detached blocks.

In this paper, the dynamic stability of detached blocks in the Gigerwald arch dam in Switzerland is investigated using both 2D and 3D analyses and the results are compared.

MAIN FEATURES OF GIGERWALD ARCH DAM

The main features of the Gigerwald dam (see Fig. 1) are as follows:

- Dam type: double-curvature arch dam
- Construction period: 1974-1976
- Number of monoliths (blocks): 24
- Concrete volume: 446,000 m^3
- Maximum height: 147 m
- Crest length: 430 m
- Dam thickness: 7.0 m (crest) to 22.0 m (base)
- Lombardi slenderness coefficient: 14.5

SEISMIC HAZARD

According to the Swiss guidelines, the Gigerwald arch dam is a class I dam, whose safety has to be verified for a Safety Evaluation Earthquake (SEE) with a return period of 10,000 years. Under the SEE ground motion, significant structural damage to the dam is acceptable, as long as there is no uncontrolled release of water from the reservoir. At the location of the Gigerwald dam, the SEE has an intensity of 8.0 on the MSK scale (see Fig. 1). The corresponding horizontal and vertical peak ground accelerations (PGA's) are 0.19 g and 0.13 g, respectively.

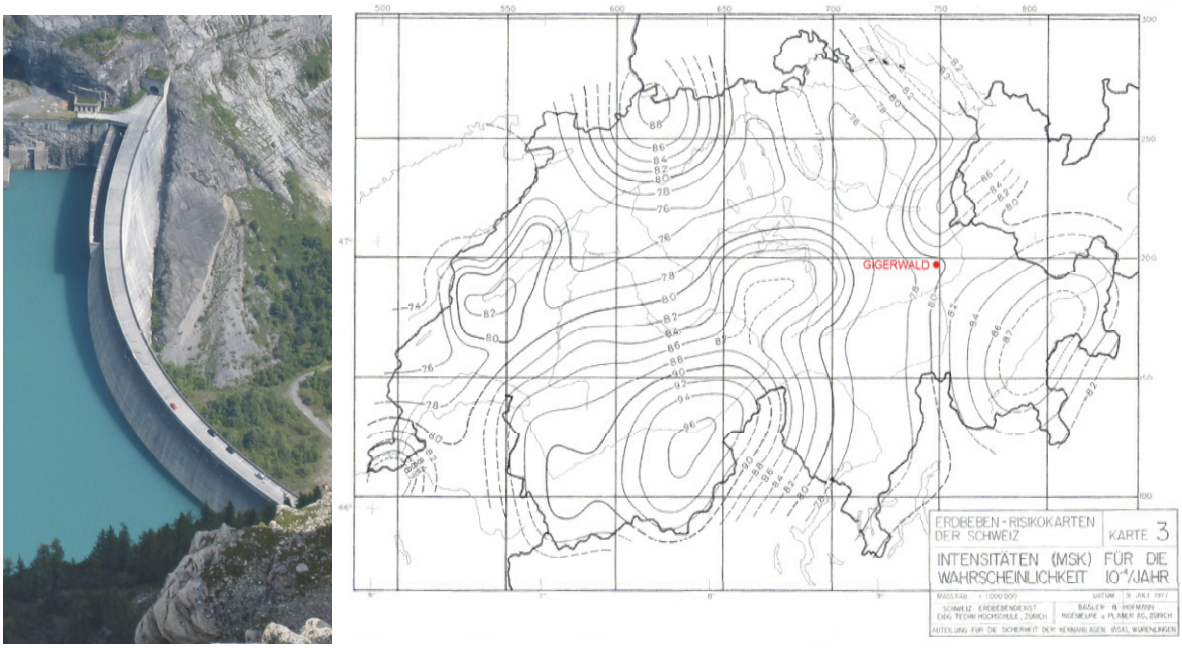

Figure 1. Gigerwald arch dam and seismic hazard map of Switzerland for a return period of 10,000 years

For the dynamic analysis, spectrum-compatible horizontal and vertical ground motions were artificially generated using the software SIMQKE (Gasparini and Vanmarcke, 1976). In total, 3 sets of ground motions designated as earthquakes 1, 2 and 3 were produced, each having a total duration of 35 s. An example of the artificially generated ground motion is illustrated in Fig. 2.

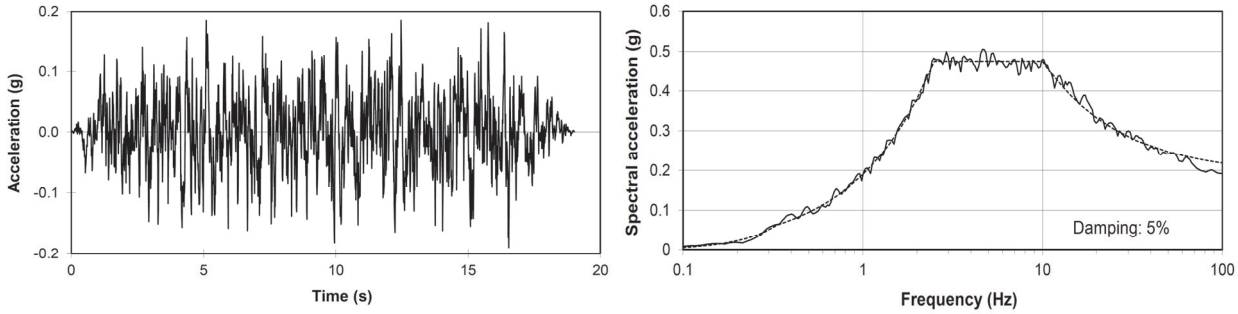

Figure 2. Time history and response spectrum of horizontal component of SEE ground motion (earthquake 1, along-stream component)

LINEAR DYNAMIC ANALYSIS

First, a linear dynamic analysis was performed using a 3D finite element (FE) model of the arch dam and the foundation rock, as shown in Fig. 3. In the dynamic analysis, the foundation rock was assumed to be massless and the hydrodynamic effect of the reservoir was simulated by added masses acting normal to the upstream face of the dam. The dam body is assumed to be monolithic in the linear analysis, but it is also possible to simulate all the vertical contraction joints as frictional contact surfaces in order to later perform the nonlinear analysis of the 3D FE model.

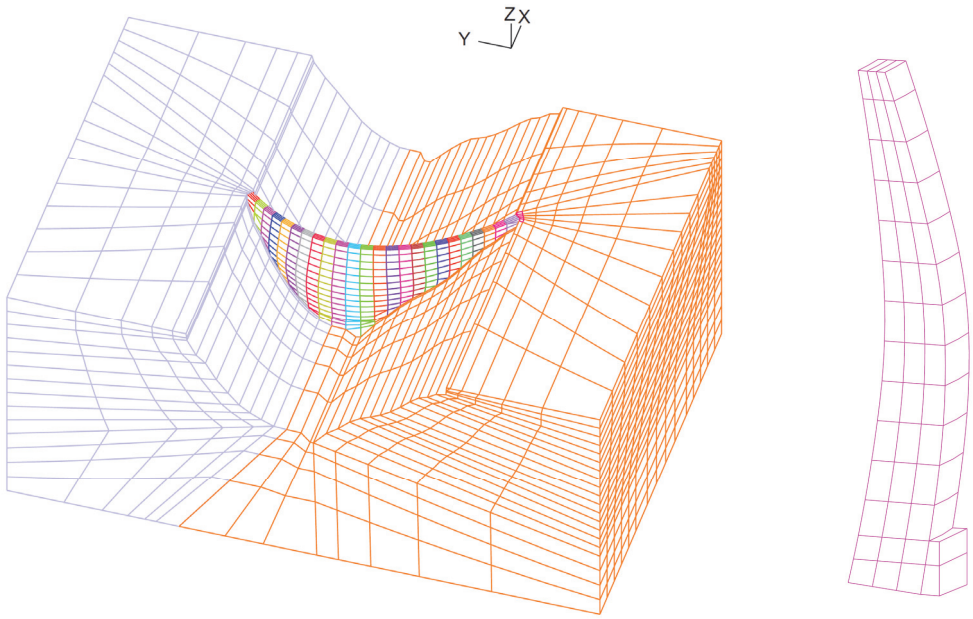

Figure 3. 3D FE model of dam-foundation system and central monolith (block 11)

The elastic properties of concrete and rock were calibrated on the basis of measured displacements during seasonal variations of the reservoir level and the temperature distribution in the dam body. As recommended in the Swiss guidelines, the dynamic E-modulus was assumed to be 25% higher than the static E-modulus.

The dynamic material properties of the mass concrete were taken as follows: E_d = 57 GPa, ν = 0.20 and ρ = 2550 kg/m³, where E_d, ν and ρ stand for the dynamic E-modulus, the Poisson's ratio and the mass density, respectively.

The foundation rock consists of two distinct stiff limestone formations, whose properties were taken as follows:

- Quintner limestone (left abutment rock shown blue in Fig. 3): E_d = 52 GPa, ν = 0.25
- Schilt limestone (middle/right abutment rock shown orange in Fig. 3): E_d = 37 GPa, ν = 0.25

The first 10 eigenfrequencies of the arch dam and the corresponding mass participation factors are listed in Table 1. The fundamental mode is antisymmetric and has an eigenfrequency of 2.14 Hz under the full reservoir condition (see Fig. 4). However, the 2nd, 3rd, 5th and 10th modes are more important from the viewpoint of the earthquake response of the arch dam. These symmetric modes excite in total about three-quarters of the total mass of the dam-reservoir system in the along-stream direction.

Table 1. First 10 eigenfrequencies and mass participation factors

Mode	Eigenfrequency (Hz)	Mass participation factor		
		Along-stream (X) direction	Across-stream (Y) direction	Vertical (Z) direction
1	2.14	0.09%	**17.10%**	0.01%
2	2.45	**32.80%**	0.00%	0.02%
3	3.31	**19.73%**	0.06%	0.03%
4	4.32	0.03%	3.12%	0.00%
5	4.52	**15.26%**	0.00%	3.40%
6	5.38	0.48%	**8.14%**	0.01%
7	5.62	1.00%	3.53%	0.03%
8	6.68	0.22%	0.64%	0.36%
9	7.01	0.06%	0.55%	0.03%
10	7.53	**8.31%**	0.00%	**5.04%**

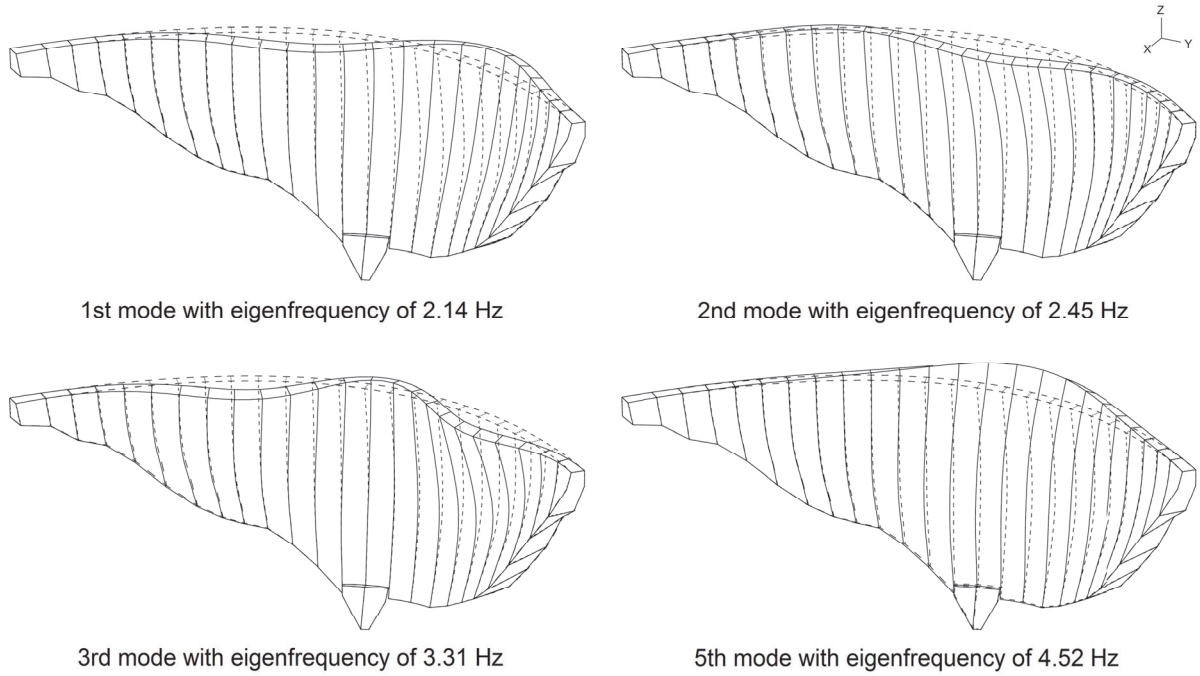

Figure 4. Important vibration modes of dam under full reservoir condition

For the linear dynamic analysis, a Rayleigh damping model with the following parameters was employed: $\alpha = 1.77\ s^{-1}$ and $\beta = 0.00150$ s. This model results in a damping of around 7% in the relevant frequency range from 2 to 15 Hz. At the most important eigenfrequencies lying between 2.5 and 10 Hz, the damping ratio varies from about 5% to 6% (see Fig. 14). All the dynamic calculations were performed using the general-purpose FE software ADINA (ADINA R & D, 2008).

The most important results of the linear time history analysis of the dam subjected to 3 different earthquakes are listed in Table 2, which also shows the results obtained from a simplified analysis using the response spectrum method. The central crest region experiences amplified horizontal accelerations as high as 2.0 g, which is about 10 times larger than the horizontal PGA of 0.19 g of the 10,000-year SEE.

Table 2. Maximum responses obtained from linear dynamic analysis of dam subjected to SEE ground motion (without static loads) under full reservoir condition

Dynamic response (envelope)	Time history analysis (linear)			Response spectrum analysis
	Earthquake 1	Earthquake 2	Earthquake 3	
Relative crest displacement (mm)				
• Along-stream direction	48.0	45.6	44.5	42.0
• Across-stream (left-right) direction	15.3	16.0	18.0	15.1
• Vertical direction	6.6	7.4	6.9	6.0
Absolute crest acceleration (g)				
• Along-stream direction	1.83	2.04	1.98	1.65
• Across-stream (left-right) direction	0.84	0.86	0.80	0.70
• Vertical direction	0.75	0.78	0.71	0.70
Principal tensile stress (MPa)	8.7	9.8	8.7	9.3
Principal compressive stress (MPa)	-10.3	-9.5	-9.6	-9.3

The earthquake shaking produces dynamic tensile stresses of nearly 10 MPa in the central crest region of the dam, as shown in Fig. 5 depicting the principal stress vectors at time t = 14.70 s, when the highest tensile stress occurs in the main dam body during simulated earthquake 2.

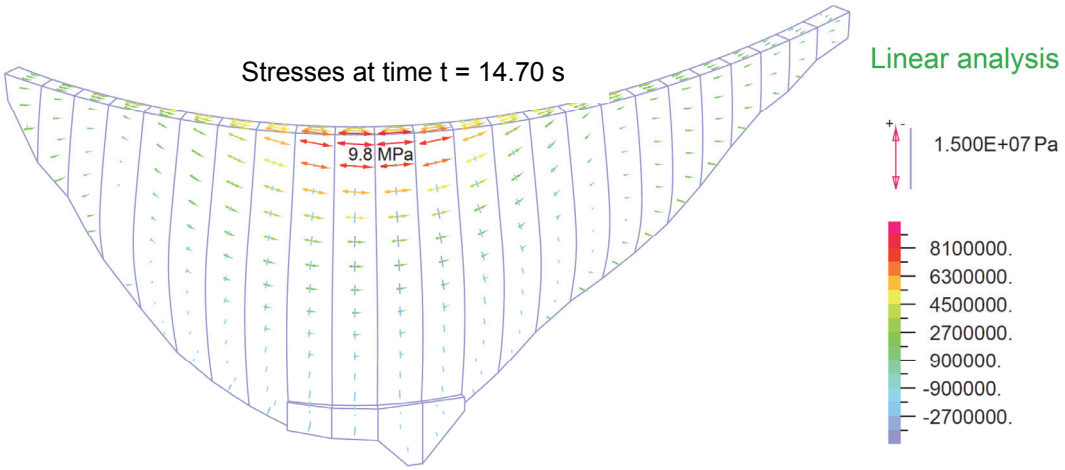

Figure 5. Principal stress vectors on upstream face due to earthquake 2 (without static loads) at time t = 14.70 s obtained from linear dynamic analysis under full reservoir condition

Even after the combination with the compressive stresses due to the static loads (self-weight and hydrostatic water load), high horizontal tensile stresses of up to about 6 MPa still remain in the arch direction (see Table 3 and Figs. 6 and 7). In reality, such tensile stresses cannot develop due to the presence of the vertical contraction joints with a low tensile strength. Thus, the earthquake shaking is likely to lead to the opening of these joints, as discussed in the next section.

High local stresses are computed at the upstream and downstream edges of the dam-rock interface in a linear elastic analysis due to the stress singularities at these reentrant corners, but they do not pose a problem for the global safety of the arch dam.

Table 3. Maximum stresses in dam obtained from linear dynamic analysis under full reservoir condition (excluding reentrant corners with stress singularities at dam-rock interface)

	Self-weight + Water load + Earthquake 1	Self-weight + Water load + Earthquake 2	Self-weight + Water load + Earthquake 3
Largest tensile stress (MPa)	5.6	6.4	5.3
Largest compressive stress (MPa)	-15.7	-14.3	-14.1

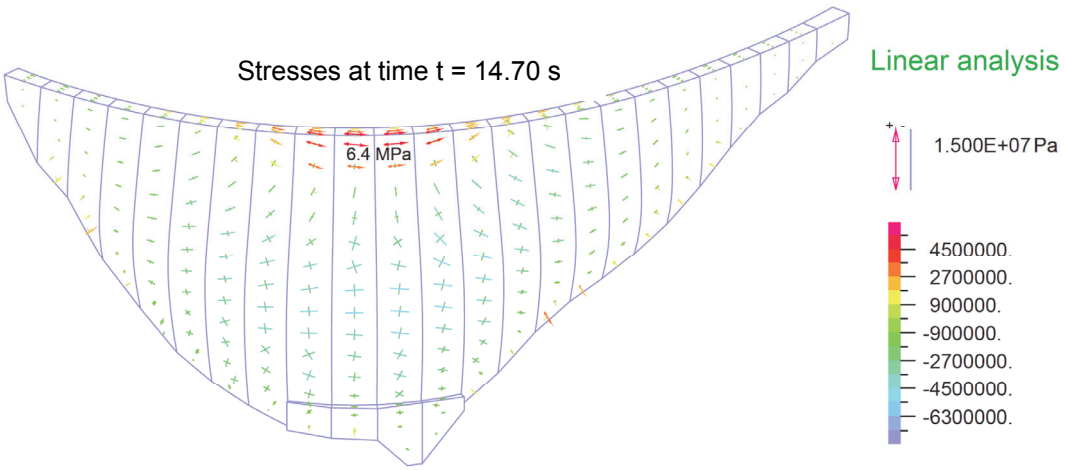

Figure 6. Principal stress vectors on upstream face due to combination of static loads (self-weight and water load) and earthquake 2 at time t = 14.70 s obtained from linear dynamic analysis under full reservoir condition

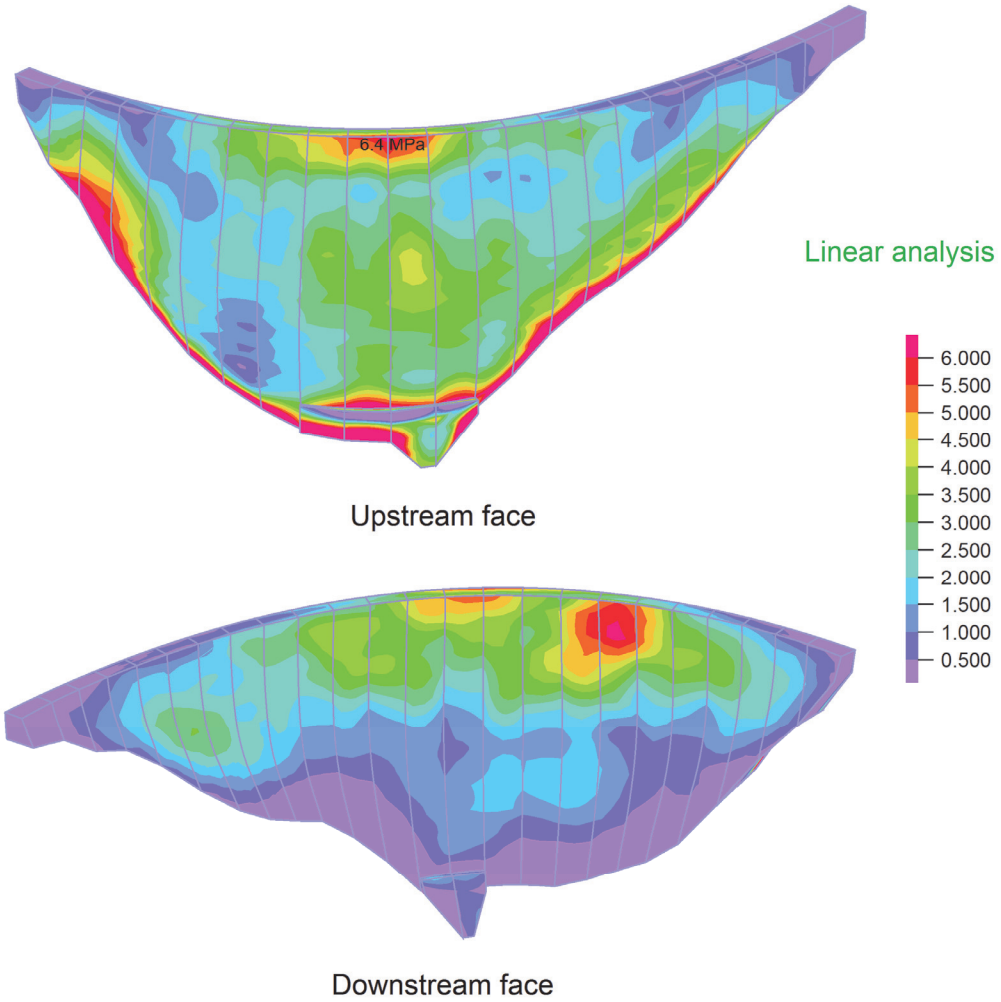

Figure 7. Maximum (envelope) tensile principal stresses obtained from linear dynamic analysis of dam subjected to combination of static loads (self-weight and water load) and earthquake 2 under full reservoir condition

NONLINEAR DYNAMIC ANALYSIS OF DAM WITH CONTRACTION JOINTS

The nonlinear behaviour of the dam subjected to the SEE ground motion was analysed using a 3D FE model in which all 23 vertical contraction joints were simulated as frictional contact surfaces and the dam concrete was assumed to be linear elastic (uncracked). The Rayleigh damping coefficients for this nonlinear analysis were reduced to $\alpha = 1.27\ s^{-1}$ und $\beta = 0.00107$ s, so that the damping ratio in the relevant frequency range of 2 to 15 Hz is around 5%, compared to around 7% in the case of the linear analysis.

The vertical contraction joints open during the seismic shaking as illustrated in Fig. 8, which depicts the deformed shape of the dam at time t = 14.71 s, when the largest joint openings were computed in the dam subjected to earthquake 2 (see also Fig. 9). This corresponds roughly to the time when the largest tensile stresses were computed in the corresponding linear analysis (see Fig. 6).

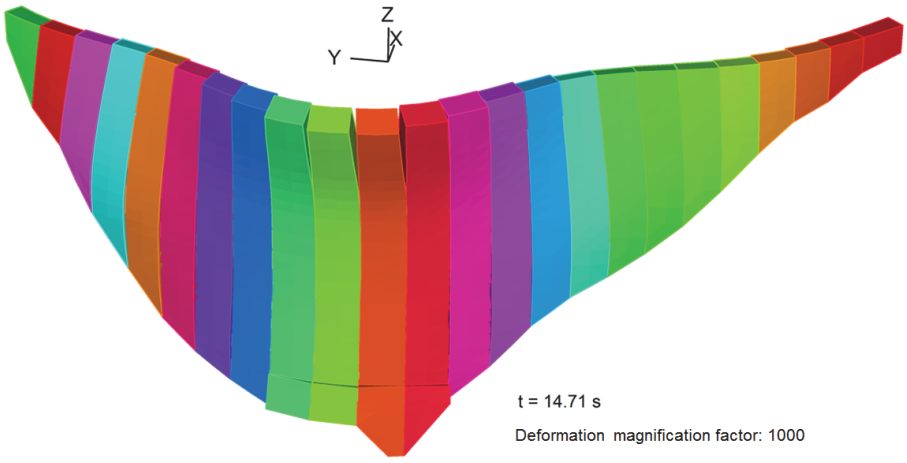

Figure 8. Deformed shape at time t = 14.71 s obtained from nonlinear dynamic analysis of dam with vertical contraction joints subjected to earthquake 2 under full reservoir condition

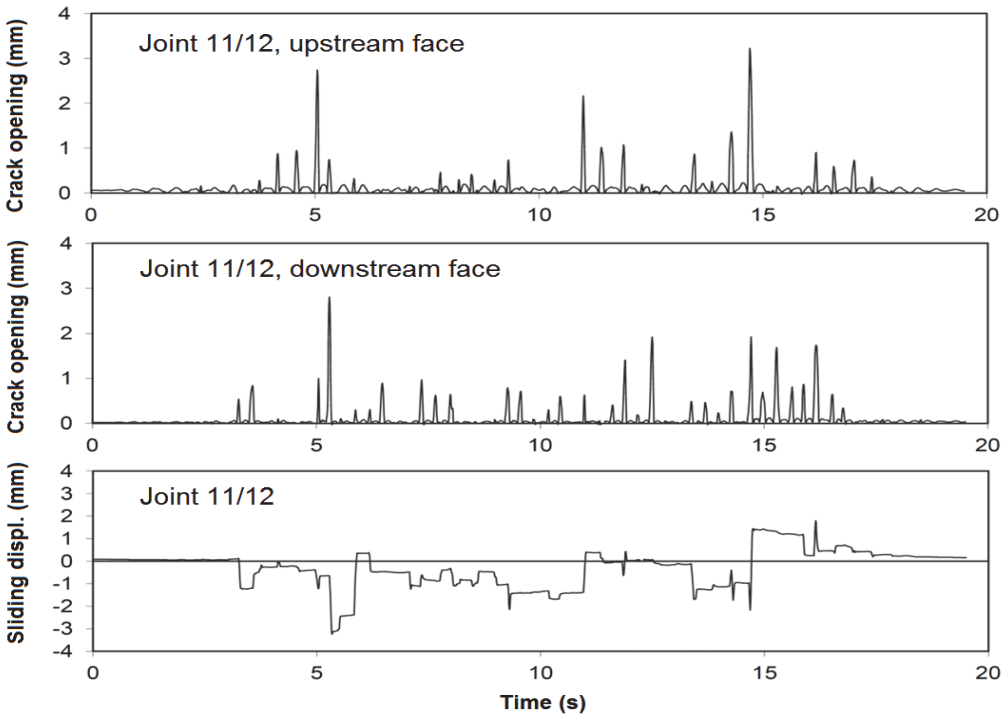

Figure 9. Time histories of opening and sliding displacements of vertical contraction joint 11/12 at crest level due to earthquake 2 under full reservoir condition

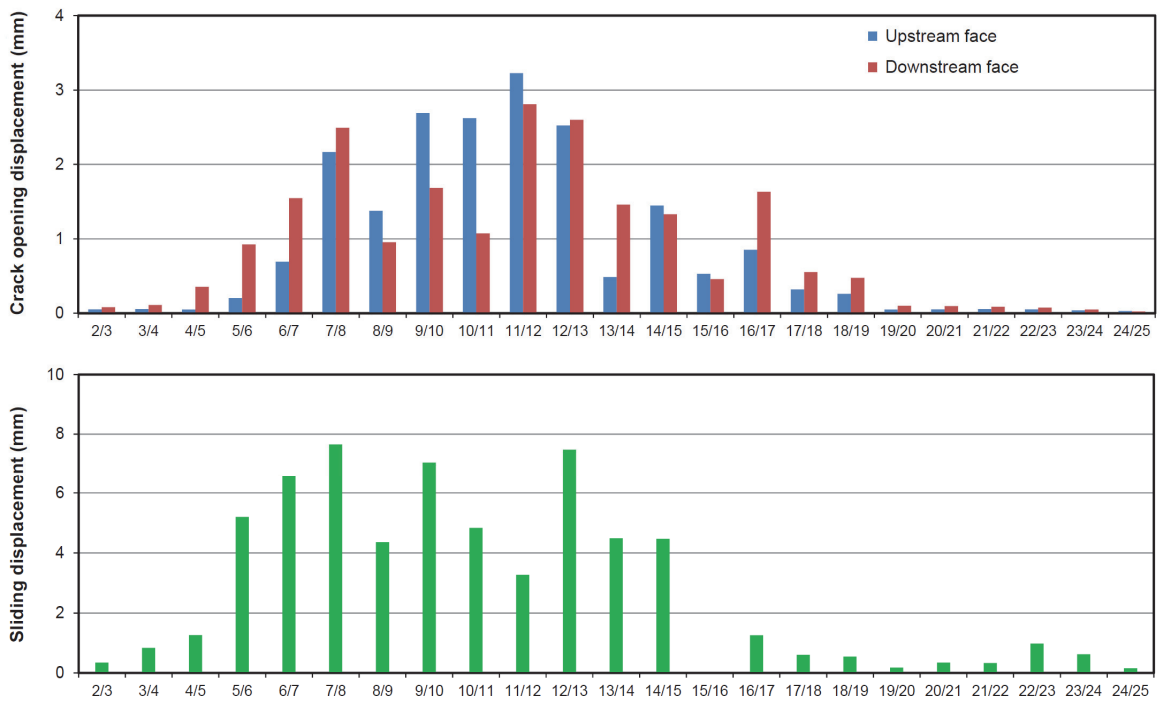

Figure 10. Maximum opening and sliding displacements of vertical contraction joints at crest level due to earthquake 2 under full reservoir condition

The maximum opening and sliding displacements of the vertical contraction joints at the crest level due to earthquake 2 are shown in Fig. 10. The maximum joint displacements due to the three simulated earthquakes are listed in Table 4. The largest relative sliding displacement between two adjacent monoliths is about 10 mm and the contraction joints open by up to about 4 mm.

Table 4. Maximum opening and sliding displacements of vertical contraction joints obtained from nonlinear dynamic analysis of dam under full reservoir condition

	Earthquake 1	Earthquake 2	Earthquake 3
Maximum joint opening on upstream face of dam	3.5 mm (Joint 12/13)	3.2 mm (Joint 11/12)	2.3 mm (Joint 9/10)
Maximum joint opening on downstream face of dam	2.5 mm (Joint 7/8)	2.8 mm (Joint 11/12)	2.2 mm (Joint 7/8)
Maximum sliding displacement between two adjacent monoliths	7.5 mm (Joint 7/8)	7.7 mm (Joint 7/8)	9.5 mm (Joint 7/8)

In the nonlinear analysis with the vertical contraction joints, horizontal tensile stresses in the arch direction are no longer present in the central crest region (see Fig. 11). However, during the brief openings of the contraction joints, the upper portion of a dam block acts temporarily as a cantilever, due to which relatively high transitory vertical tensile stresses exceeding 6 MPa appear on the downstream face of the dam, as shown in Fig. 12. Hence, horizontal cracks are likely to form, especially at the lift joints, possibly causing the uppermost portion of a central monolith to get detached from the rest of the dam body.

Table 5 shows that the largest compressive stress in the dam obtained from the nonlinear analysis approaches nearly -19 MPa (excluding the corner singularity at the dam-rock interface), which is not a problem for the dam concrete. In comparison, the maximum compressive stress in the case of the linear analysis is about -16 MPa (see Table 3).

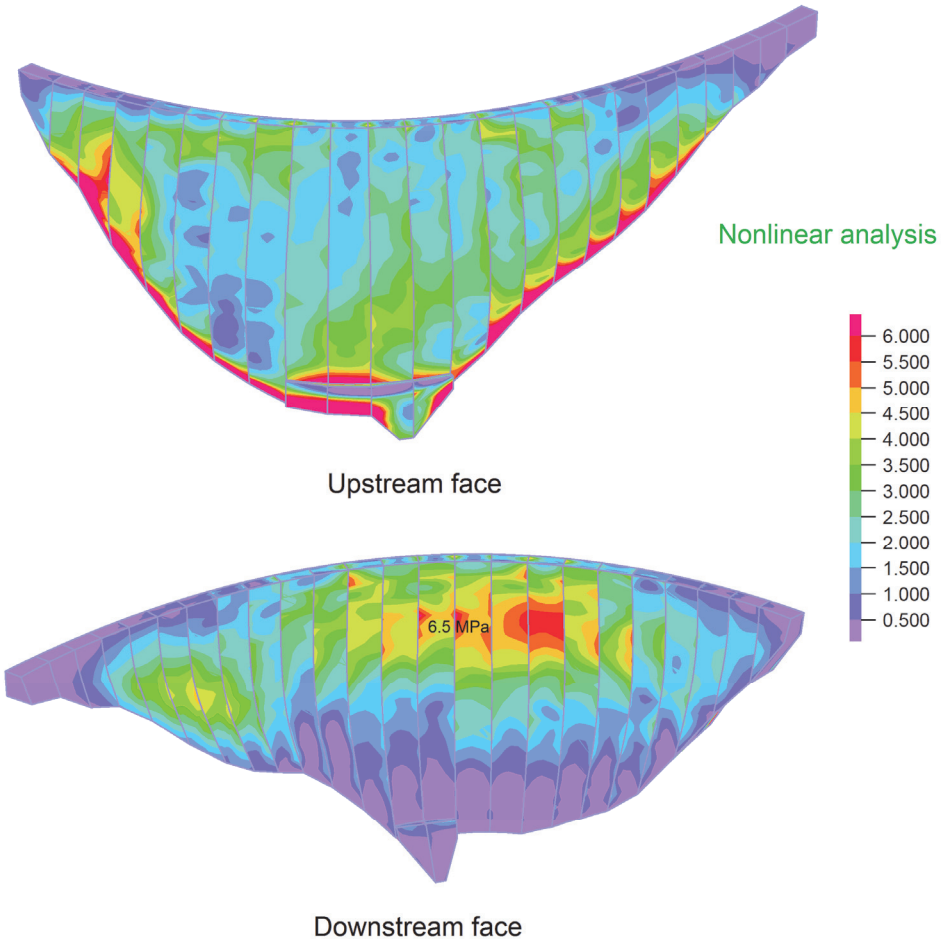

Figure 11. Maximum (envelope) tensile principal stresses due to combination of static loads (self-weight and water load) and earthquake 2 under full reservoir condition obtained from nonlinear dynamic analysis of dam with vertical contraction joints

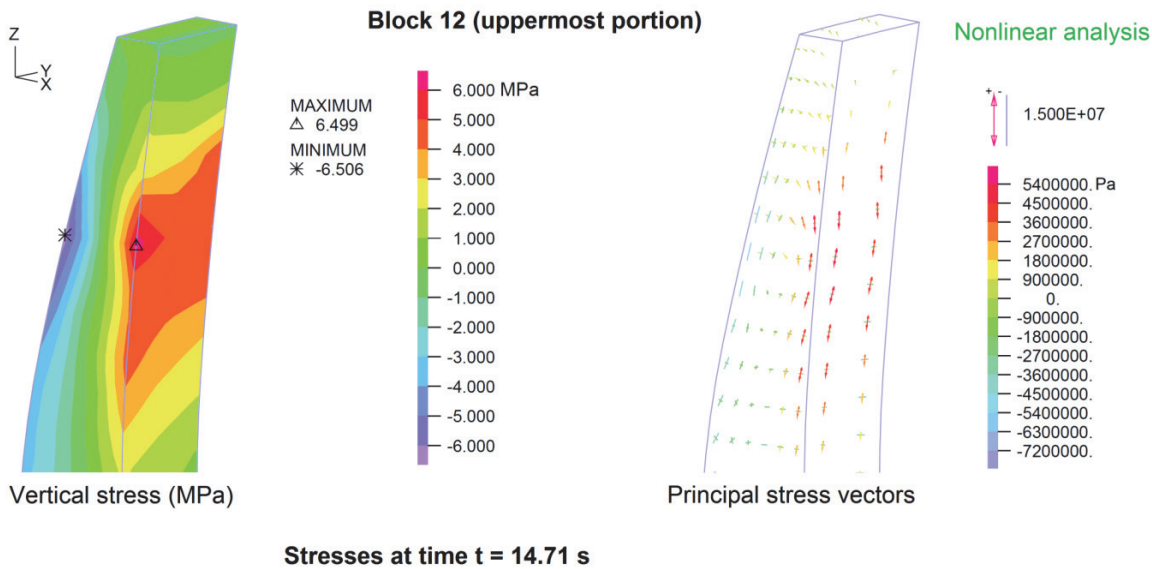

Figure 12. Stresses in uppermost portion of monolith (block) 12 at time t = 14.71 s due to combination of static loads (self-weight and water load) and earthquake 2 under full reservoir condition obtained from nonlinear dynamic analysis of dam with vertical contraction joints

Table 5. Maximum stresses in dam with vertical contraction joints obtained from nonlinear dynamic analysis under full reservoir condition (excluding reentrant corners with stress singularities at dam-rock interface)

	Self-weight + Water load + Earthquake 1	Self-weight + Water load + Earthquake 2	Self-weight + Water load + Earthquake 3
Largest tensile stress (MPa)	5.8	6.5	5.3
Largest compressive stress (MPa)	-18.6	-17.9	-18.2

In spite of the joint displacements, the dynamic displacements and accelerations of the dam computed in the nonlinear analysis do not deviate significantly from those in the corresponding linear analysis, as can be seen from the comparison of the crest displacements obtained from the linear and nonlinear analyses in Fig. 13.

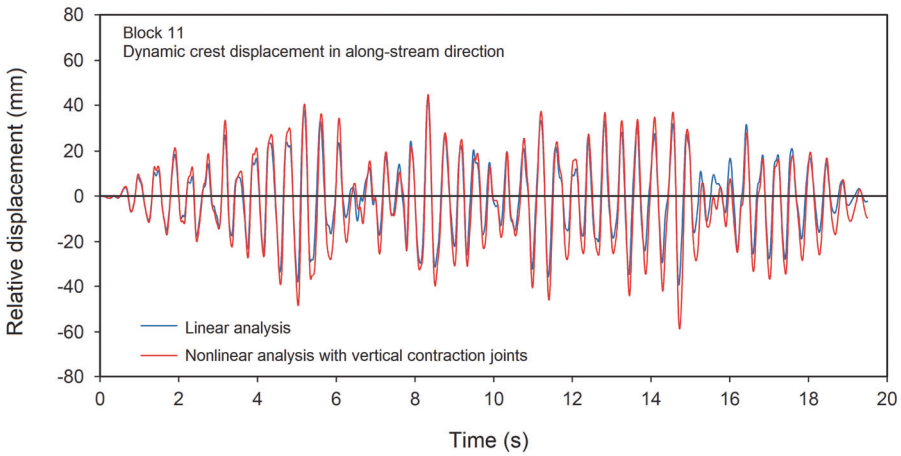

Figure 13. Comparison of time histories of crest displacement in central monolith (block 11) obtained from linear and nonlinear dynamic analyses (positive displacement: towards downstream)

2D DYNAMIC STABILITY ANALYSIS

The seismic stability of possibly detached blocks in the central upper portion of the dam crest was first checked by a simplified two-dimensional (2D) analysis procedure as proposed by Malla and Wieland (2006). From this analysis, sliding and rocking motions of the detached block were computed at the assumed horizontal crack, which was modelled using contact surfaces. In view of the arch curvature, the detached block can move only towards the reservoir. This is taken into account in the 2D model using gap elements, which prevent the movement of the detached block beyond the downstream face of the dam (see Fig. 14). The vertical contraction joints on the sides of the detached block are assumed to remain open throughout the analysis in this simplified 2D approach. In reality, the joints would open only sporadically during the earthquake shaking.

In a dynamic analysis involving rigid body motions, it is considered prudent to use only the stiffness-proportional part of the Rayleigh damping model, as the mass-proportional part corresponds to external viscous dampers connected to the nodes of the model (Hall, 2006). Accordingly, three different Rayleigh damping models were considered in the stability analysis, as shown in Fig. 14.

The 2D dynamic stability analysis of an 8 m high detached portion shows that it could slide by up to about 50 cm (see Fig. 15) towards the reservoir during the SEE ground shaking and the rocking motion would result in crack opening displacements of up to about 7 cm. The tendency of the detached block to slide towards the upstream side can be explained by its asymmetric geometry, which causes the rocking response towards the downstream to be stronger than that towards the upstream. Since a dynamic impulse is shared by a combination of rocking and sliding motions, the smaller rocking response towards the upstream is more likely to be accompanied by a larger sliding response in this direction. This explains why the sliding movement in the upstream direction is of a cumulative nature.

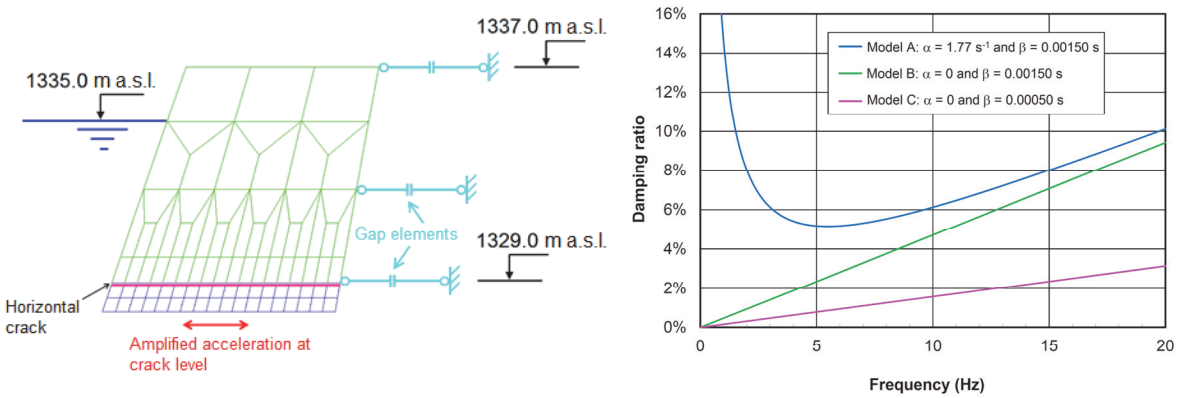

Figure 14. 2D FE model of 8 m high detached upper portion of central monolith (gap elements prevent any movement beyond the downstream face) and various Rayleigh damping models

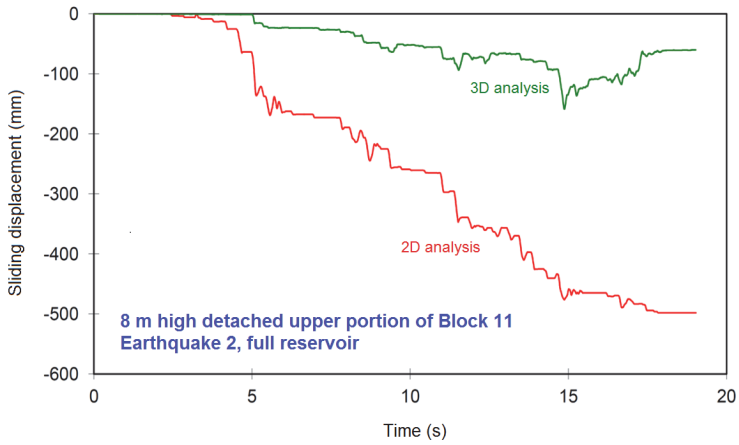

Figure 15. Time histories of sliding displacement of 8 m high detached upper portion of central monolith of dam subjected to earthquake 2 under full reservoir condition obtained from 2D and 3D dynamic stability analyses using Rayleigh damping model C for the detached portion

The dynamic stability analysis was performed also for a 17 m high detached block. The rocking and sliding displacements in this case were found to be significantly smaller than those of the 8 m high detached block.

3D DYNAMIC STABILITY ANALYSIS

For a more realistic dynamic stability assessment, a nonlinear analysis was also performed using a 3D model containing the vertical contraction joints and an assumed horizontal crack along a lift joint. This analysis showed that an 8 m high detached block in the central crest region would slide towards the reservoir by up to 16 cm, which is only about one-third of the result obtained in the corresponding 2D analysis (see Figs. 15 and 16). The substantially smaller sliding displacement in the more rigorous 3D analysis can be mainly attributed to the additional frictional resistance at the vertical contraction joint on each side, an effect that could not be taken into account in the simplified 2D approach. The maximum crack opening also decreased from 74 mm in the 2D analysis to 28 mm in the 3D analysis.

The nonlinear dynamic analysis of the 3D model of the uncracked dam (i.e. without any horizontal crack) with the vertical contraction joints showed that the highest vertical tensile stresses due to the cantilever action during the opening of the vertical joints would occur approximately 27 m below the dam crest (see Figs. 11 and 12). A further nonlinear dynamic analysis was performed by modelling an assumed horizontal crack at this location in the 3D model. The results of this analysis showed that the horizontal crack would open by maximum 7 mm only. The detached portion above this crack tends to get wedged between the adjacent monoliths due to the frictional resistance at the vertical contraction joints. As a result, the detached block cannot return to its original position and a crack opening of 6 mm would remain at the end of the earthquake, as illustrated in Fig. 17.

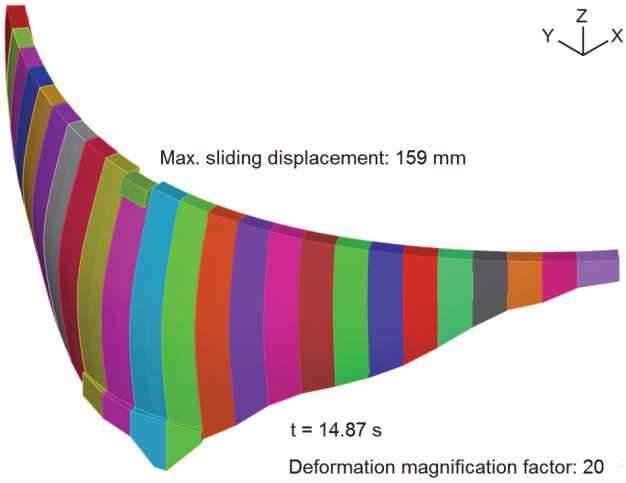

Figure 16. Deformed configuration showing sliding displacement of 8 m high detached block in central crest region at time t = 14.87 s during earthquake 2 obtained from 3D dynamic analysis under full reservoir condition

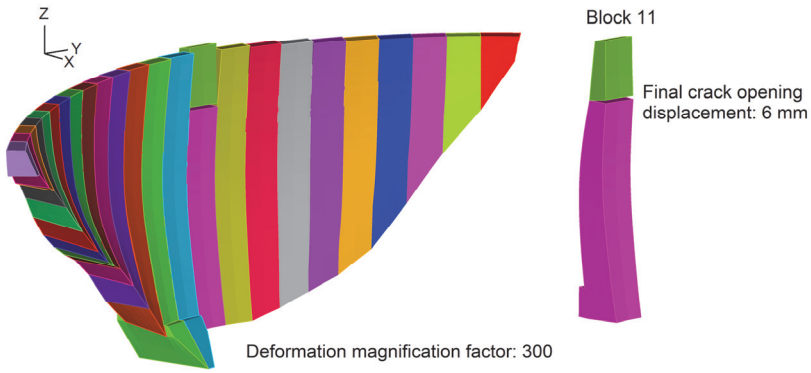

Figure 17. Final deformed configuration of dam with 27 m high detached upper portion in central monolith obtained from 3D dynamic analysis under full reservoir condition showing crack opening of 6 mm remaining at end of earthquake 2

CONCLUSIONS

Based on the results of the dynamic stability analysis, it is concluded that a possibly detached block in the central crest area could be subjected to some sliding and rocking motions during the SEE ground shaking, but it would remain stable during and after the earthquake and an uncontrolled release of reservoir water would not occur. Therefore, the safety requirements for the SEE are satisfied. The comparison of the 2D and 3D dynamic stability analyses shows that the simplified 2D approach is quite conservative, as the additional frictional resistance at the vertical contraction joint on each side of the detached block is neglected in this approach.

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