Seismic Strengthening of an Arch-Gravity Dam

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SUMMARY:
The 25 m high Illsee dam is being strengthened to meet the requirements of the Swiss guidelines for earthquake safety assessment of dams. The stability of the dam was checked at an existing horizontal crack in the curved portion on the left side and at the concrete-rock interface in the straight portion on the right side. The factors of safety against sliding and overturning are adequate under the usual static loading, but are less than 1.0 under the earthquake loading. Hence, anchors carrying a total force of 1900 kN/m length of dam are being installed to strengthen the curved portion. In the straight portion, an anchor force of 410 kN/m length will be provided to meet the stability requirement for the ice loading only, as the 10,000-year SEE excitation with a horizontal PGA of 0.44 g would cause only a relatively minor sliding displacement even without any anchors.

Keywords: concrete dam, strengthening, anchors, rehabilitation

1. INTRODUCTION

The Illsee dam is located in the canton of Valais in south-west Switzerland in an area with relatively high seismicity. The earthquake safety of the dam was analyzed according to the Swiss guidelines for earthquake safety assessment of dams (BFE/BWG, 2003). Based on the results of this analysis, the dam is currently being strengthened. Various other measures are being implemented at the same time to further improve the safety of the dam.

2. MAIN FEATURES OF DAM

The Illsee dam was originally built in 1926-27 and was later heightened by 7 m in 1941-43. It consists of a curved portion (arch-gravity dam) on the left side and a straight gravity dam on the right side, as shown in Figs. 1 and 2.

The main features of the dam are as follows:
- Maximum dam height: 25 m
- Crest level: 2360.40 m a.s.l.
- Crest length: 270 m
- Thickness at crest level: 3.00 m
- Maximum thickness: 18.75 m
- Concrete volume: 24,000 m^3
- Maximum reservoir level: 2360.00 m a.s.l.
- Reservoir volume: 6.5 million m^3

In the 1960's, the first signs of chemical expansion of the dam concrete were observed (Bossoney and Balissat, 2005; Otto, 2007). In the meantime, this has caused the central part of the curved portion to move irreversibly towards the reservoir by about 50 mm, while the junction between the curved and straight portions has moved by about 35 mm towards the downstream. Moreover, a horizontal crack can be seen on the downstream face of the curved portion at an elevation of 2353 m a.s.l., which corresponds to the crest level of the older concrete dam of 1926-27 prior to the heightening.

Figure 1. Illsee dam

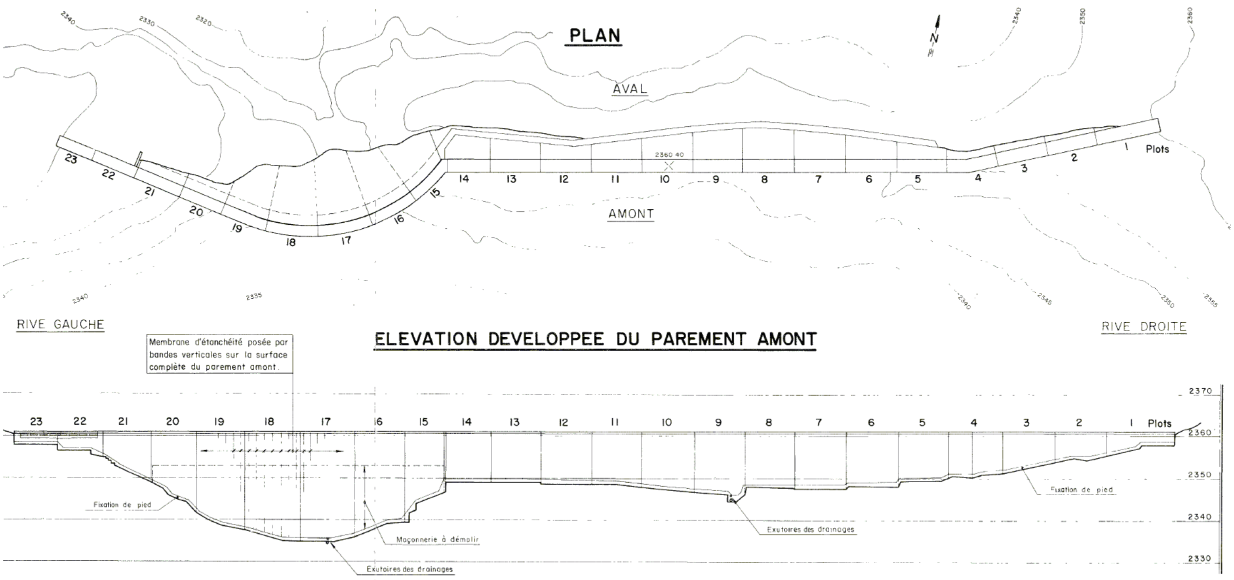


Figure 2. Layout plan and developed elevation showing upstream face of Illsee dam

3. SEISMIC HAZARD

According to the Swiss guidelines, the Illsee dam had to be checked for a Safety Evaluation Earthquake (SEE) with a return period of 10,000 years, as it is a class I dam (BFE/BWG, 2002, 2003). At the dam location (see Fig. 3), the SEE has an intensity of 9.4 in the MSK scale (Basler & Hofmann and SED, 1977). The corresponding horizontal and vertical peak ground accelerations (PGA's) are 0.44 g and 0.29 g, respectively. 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 used, each having a total duration of 35 s. Figure 4 shows an example of the artificially generated ground motion.

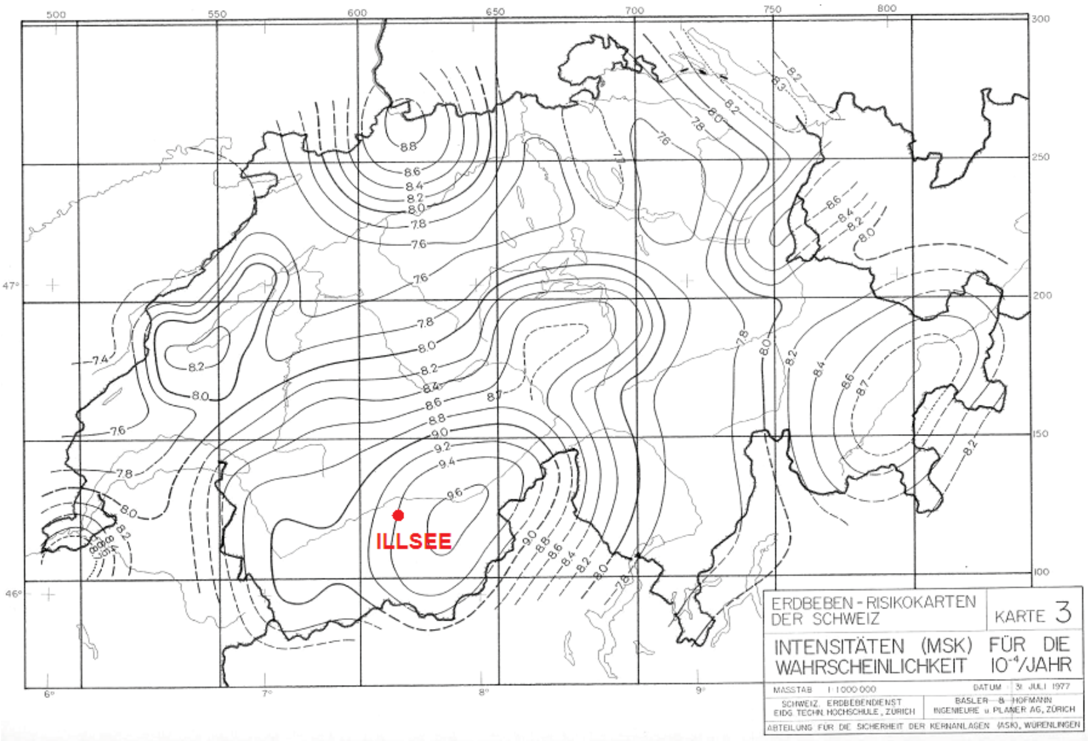


Figure 3. Seismic hazard map showing isolines of MSK intensity with a return period of 10,000 years and location of Illsee dam

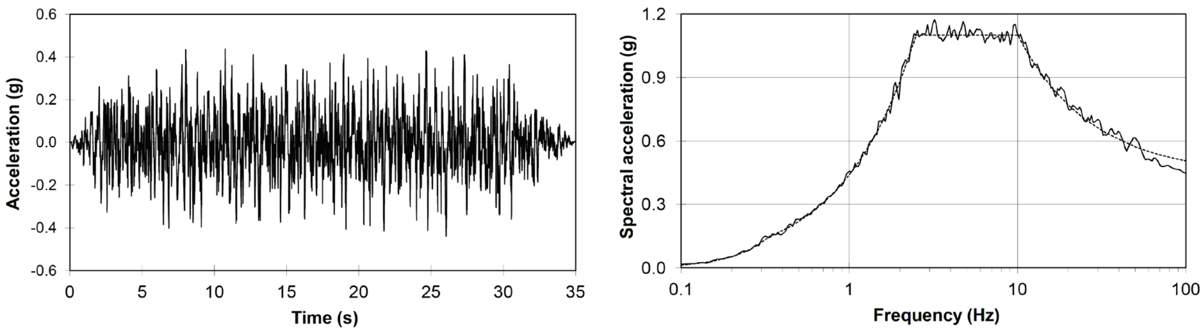


Figure 4. Time history and response spectrum of horizontal component of SEE ground motion (earthquake 1)

Under the 10,000-year SEE ground motion, significant structural damage to the dam is acceptable, as long as there is no uncontrolled release of water from the reservoir. It is common practice to also consider the Operating Basis Earthquake (OBE) with a significantly shorter return period than the SEE. The dam shall remain operable after the OBE, which may cause only easily repairable minor damage (ICOLD, 1989). As the consequences of exceeding the OBE is just economical, there are no fixed criteria for the OBE. The International Commission on Large Dams (ICOLD) recommends a return period of about 145 years, but sometimes longer return periods are considered (ICOLD, 1989; Reilly, 2004; Wieland, 2005). Although not specifically required by the Swiss guidelines, it was decided to consider also the OBE with a maximum horizontal PGA of 0.20 g, which has a return period of 475 years at the location of the dam.

4. LOADS AND STABILITY REQUIREMENTS

The following loads were considered in the stability analysis:

(i) **Gravity load (G)**: The unit weight of mass concrete was taken as 23 kN/m^3.

(ii) **Water pressure (W)**: The stability was checked for the full reservoir level of 2360 m a.s.l. The specific weight of water was taken as 10 kN/m^3.

(iii) **Uplift (U)**: The uplift pressure in the assumed sliding surface was assumed to reduce linearly from 100% to 0% over the dam thickness.

(iv) **Ice load (I)**: An ice load of 275 kN/m was considered, corresponding to a pressure of 250 kPa exerted by an ice layer with a thickness of 1.1 m, which is the maximum ice thickness at an altitude of 2500 m a.s.l. in Switzerland.

(v) **Seismic load (S)**: Both the SEE and the OBE ground motions were considered.

The load combinations and the minimum required factors of safety are listed in Table 1. In the case of the seismic load, it is permissible for the factor of safety to be less than the minimum required value or even fall below 1.0, provided that the ensuing damage (sliding, crack opening) would not lead to an uncontrolled release of reservoir water.

Table 1. Load combinations and minimum required factors of safety

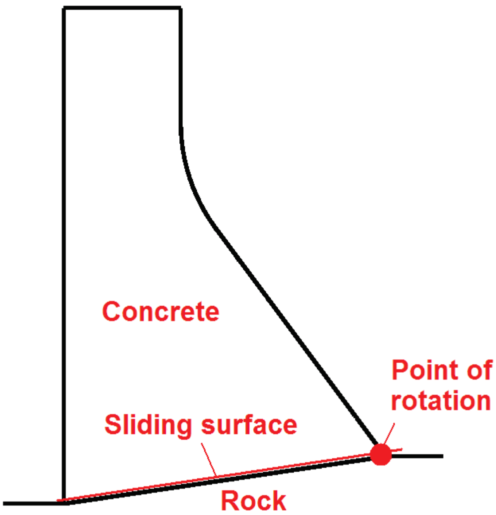
No.	Load combination	Type	Minimum factor of safety (FOS)
1	Normal operation: G + W + U	Usual	1.5
2	Ice load combination: G + W + U + I	Unusual	1.3
3	Seismic load combination: G + W + U ± S	Extreme	1.1*

* If FOS < 1.1, structural damage shall not be severe, so that there is no uncontrolled release of reservoir water.

5. ANALYZED CROSS-SECTIONS

The following two typical cross-sections of the dam were investigated:

(i) In the straight portion of the dam, the stability of block 5 was analyzed at the concrete-rock interface at the base. In this portion, this is the most critical block having a nearly horizontal base, as shown in Fig. 5. All the remaining blocks have bases sloping upwards at a steeper angle towards the downstream side and thus possess a significantly greater resistance to sliding.



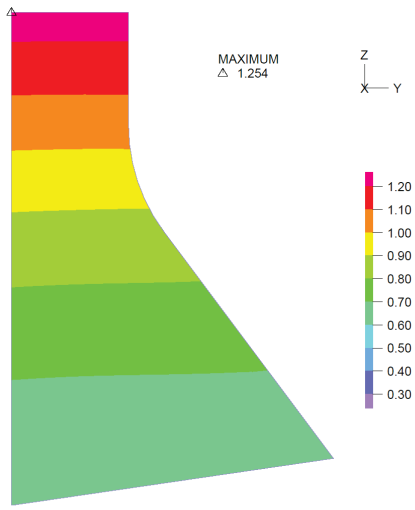


Figure 5. Sliding surface at dam-rock interface considered in stability analysis of block 5 (straight portion) and peak absolute horizontal acceleration in g under SEE ground motion (earthquake 2)

(ii) In the curved portion, the highest cross-section representative of blocks 17 and 18 was analyzed. The stability checks were performed for the known horizontal crack at elevation 2353 m a.s.l., as indicated in Fig. 6. The arch action was not taken into account, as the upper part of the arch-gravity dam does not have a proper rock abutment on the right side, where it is supported only by the adjacent end block (block 14) of the straight portion. Moreover, the planned cutting of slots in the dam to relieve stresses due to the chemical expansion of the concrete will further weaken the arch action in the upper part of the curved portion. A sliding instability is highly unlikely at the base of the curved portion, as the dam-rock interface is sloping steeply upwards towards the downstream and also in view of the geometrical constraint posed by the arch action in the lower part in this portion.

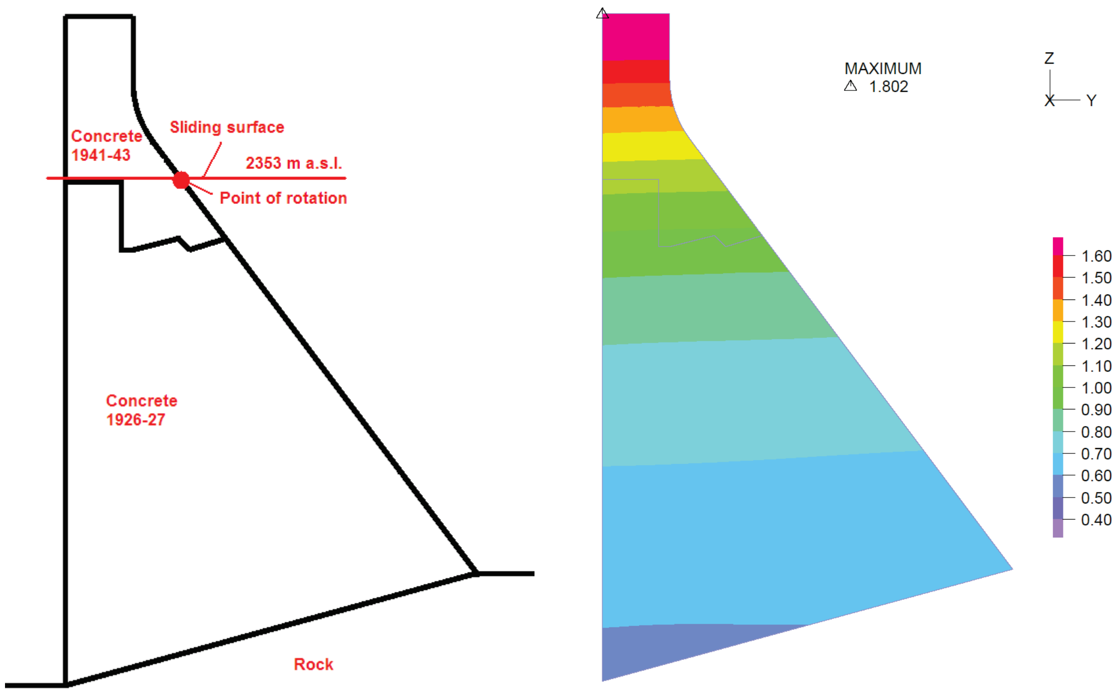


Figure 6. Sliding surface at level of observed horizontal crack at elevation 2353 m a.s.l. considered in stability analysis of block 17/18 (curved portion) and peak absolute horizontal acceleration in g under SEE ground motion (earthquake 3)

6. LINEAR DYNAMIC ANALYSIS

The dynamic earthquake forces were computed using two-dimensional finite element models, in which a massless foundation was assumed and the hydrodynamic effect of the reservoir was simulated by added masses acting normal to the upstream face of the dam.

The dynamic material properties of concrete were estimated based on the known compressive strength as follows:

- elastic modulus E = 36 GPa (1926-27 concrete) and 41 GPa (1941-43 concrete)
- Poisson's ratio ν = 0.20
- mass density ρ = 2300 kg/m³

and those of the foundation rock were estimated based on the geological information as follows:

- elastic modulus E = 10 GPa
- Poisson's ratio ν = 0.25

The fundamental modes of the analyzed cross-sections of the straight and curved portions of the dam have natural frequencies of 11.9 and 5.8 Hz, respectively, in the full reservoir condition.

For the linear-elastic dynamic analysis, the damping ratio was assumed as 10% for the SEE excitation and 7% for the OBE excitation. The effective damping based on Fenves und Chopra (1986), taking into account also the radiation damping in the foundation rock, would have been theoretically as high as 20%. However, it was decided to limit the damping ratio to 10%.

Typical distributions of peak absolute horizontal accelerations in the analyzed cross-sections of the straight and curved portions are shown in Figs. 5 and 6, respectively. The dynamic normal and tangential forces acting on the sliding surface and the dynamic overturning moment about the point of rotation were determined from the computed stresses by numerical integration.

7. STABILITY ANALYSIS OF EXISTING DAM

The friction angle was taken as 45° and the cohesion was ignored in the stability analysis. The factors of safety for the various load combinations are listed in Table 2. In the case of the seismic load combination, the time histories of the factors of safety against sliding and overturning (see Fig. 7) were computed for 3 artificial earthquakes and the minimum values were determined.

Table 2. Safety factors in existing dam (S_S: sliding safety factor; S_O: overturning safety factor)

Cross-section	Normal operation: G + W + U		Ice load combination: G + W + U + I		Seismic load combination: G + W + U ± S			
					10,000-year SEE (PGA: 0.44 g)		475-year OBE (PGA: 0.20 g)	
	S_S	S_O	S_S	S_O	S_S (min.)	S_O (min.)	S_S (min.)	S_O (min.)
Block 5 (straight portion)	1.67	1.50	1.17	0.95	0.30	0.50	0.59	0.74
Block 17/18 (curved portion)	1.67	1.68	0.79	0.68	0.01	0.28	0.24	0.44

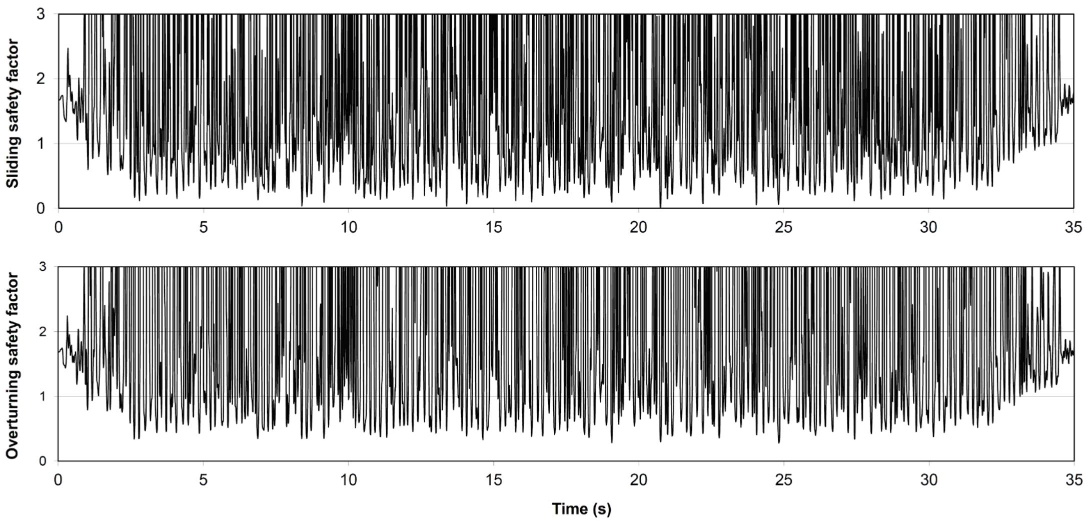


Figure 7. Time histories of factors of safety against sliding and overturning of upper part of curved portion above horizontal crack at elevation 2353 m a.s.l. during SEE ground motion (earthquake 1)

Both the investigated sections have the minimum required factors of safety of 1.50 against sliding and overturning for the normal static load combination. For the ice load combination, however, the factors of safety are less than the required minimum value of 1.30. In fact, a pneumatic system is installed to create air bubbles in order to prevent the formation of an ice layer next to the dam, but this was not taken into account in the stability analysis.

Under the earthquake load combination, the factors of safety against sliding and overturning are well below 1.0 in both the curved and straight portions. This implies that sliding and rocking motions would occur during the earthquake shaking.

8. ANCHORING FORCES REQUIRED TO SATISFY STABILITY REQUIREMENTS

It is possible to satisfy the stability requirements by strengthening the dam with anchors. The anchoring forces required to raise the factors of safety to the minimum required values of 1.30 and 1.10 under, respectively, the ice and earthquake load combinations are listed in Table 3. The time histories of the safety factors in block 17/18 subjected to the SEE ground motion after installation of anchors carrying a force of 1900 kN/m length are illustrated in Fig. 8.

Table 3. Anchoring forces required to satisfy stability requirements for ice and seismic load combinations

Cross-section	Required anchoring force (kN/m)		
	Ice load combination: G + W + U + I	Seismic load combination: G + W + U ± S	
		10,000-year SEE (PGA: 0.44 g)	475-year OBE (PGA: 0.20 g)
Block 5 (straight portion)	410	1520	650
Block 17/18 (curved portion)	490	1900	940

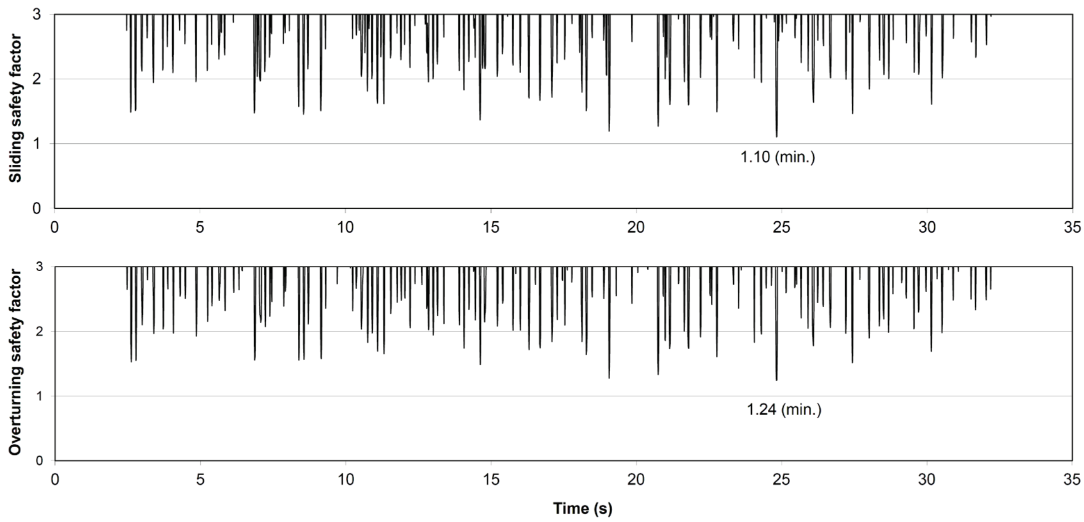
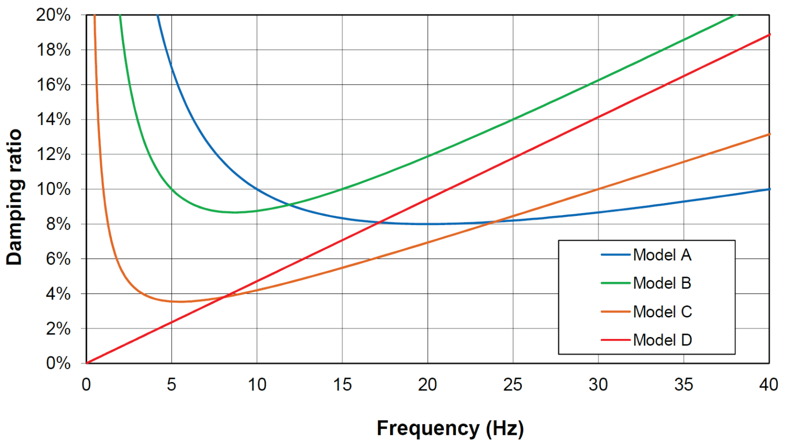


Figure 8. Time histories of factors of safety against sliding and overturning of upper part of block 17/18 (curved portion) during SEE ground motion after installation of anchors carrying a force of 1900 kN/m length of dam (earthquake 1)

9. NONLINEAR DYNAMIC ANALYSIS

Nonlinear analysis was carried out to compute the dynamic sliding and rocking displacements in the existing dam (i.e. without anchors) during the earthquake shaking. The assumed crack in each investigated cross-section was modeled as frictional contact surfaces, which cannot transmit tensile stresses and whose shear strength is governed by the Coulomb friction law. First, a static analysis was performed to compute the static stresses due to the self-weight, the water load on the upstream face and the triangular uplift pressure (i.e. varying from 100% to 0% over the dam thickness) in the crack surface. The sliding-rocking response of the dam subjected to the earthquake shaking was then computed in a dynamic restart analysis, in which the static loads were kept constant.

The energy dissipation in the nonlinear analysis is simulated by means of the Rayleigh damping, in addition to the frictional sliding in the assumed crack surface. Strictly speaking, the Rayleigh damping model is applicable only to linear models, in which it results in frequency-dependent damping, as illustrated in Fig. 9. The Rayleigh damping is widely adopted also in nonlinear analysis for practical reasons, although its applicability is arguable. In particular, it is recommended to use only the stiffness-proportional term in a dynamic system subjected to rigid body motions, which tend to be excessively damped by the mass-proportional term (Arros, 2003; Hall, 2006). In view of the uncertainties associated with the Rayleigh damping, the sliding-rocking analysis of the dam was carried out for various damping models shown in Fig. 9. Models A and B would cause about 10% damping in the dominant modes in a linear (i.e. uncracked) analysis of blocks 5 (straight portion) and 17/18 (curved portion), respectively. In model C, the damping is reduced in the frequency range of interest. Finally, model D includes only the stiffness-proportional term. In the finite element software ADINA (ADINA R & D, 2008) used for the nonlinear dynamic analysis, the Rayleigh damping matrix does not contain any terms linking nodes on two sides of the sliding surface.



Model	α (s^{-1})	β (s)
A	10.05	0.000637
B	4.71	0.001592
C	1.22	0.001027
D	0.00	0.001500

Figure 9. Various Rayleigh damping models considered in sliding-rocking analysis and frequency-dependence in case of linear response (α and β: coefficients of, respectively, mass-proportional and stiffness-proportional terms of damping matrix)

The displaced configuration of block 17/18 after being subjected to the SEE is shown in Fig. 10 along with the time histories of the sliding displacement and the crack opening displacement at the upstream face. As the sliding displacement increases, the restoring moment due to the self-weight decreases, which causes the amplitude of the rocking motion of the detached upper part to become ever larger.

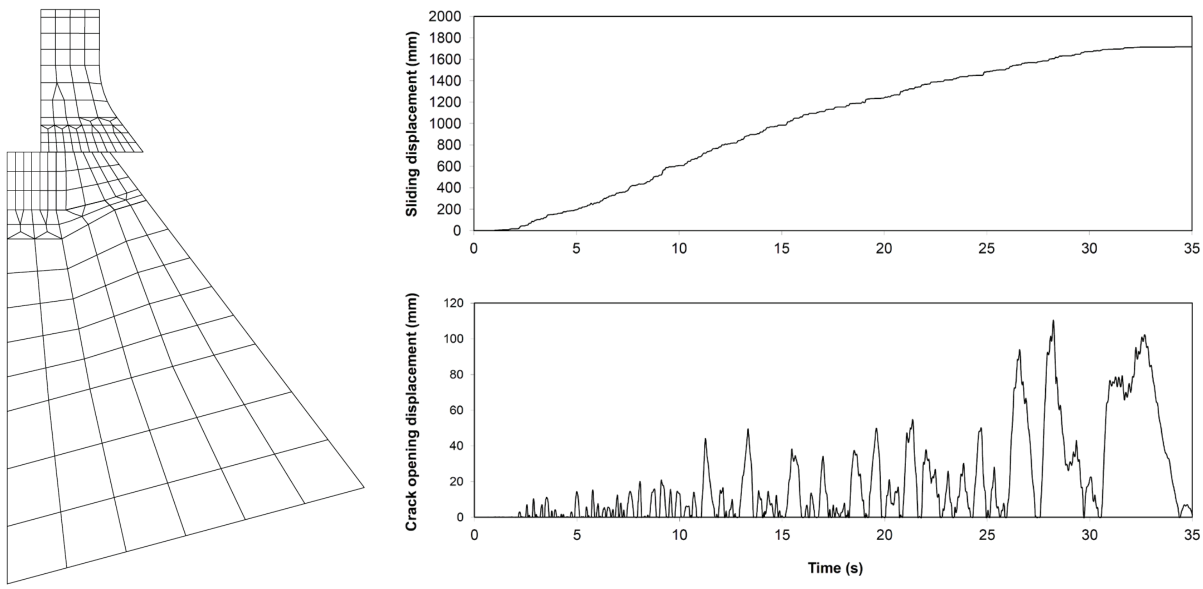


Figure 10. Displaced configuration of block 17/18 after SEE shaking and time histories of sliding displacement and crack opening displacement at upstream face (earthquake 1, Rayleigh damping model C)

The maximum values of the displacements obtained from the nonlinear dynamic analysis for 3 artificial earthquakes are listed in Table 4.

Table 4. Maximum **s**liding and crack opening displacements from nonlinear analysis of dam subjected to SEE shaking (each cross-section was analyzed for 3 artificial earthquakes and the maximum results are shown below)

Cross-section	Rayleigh damping model	Max. sliding displacement (mm)	Max. crack opening at upstream face (mm)	Max. crack opening at downstream face (mm)
Block 5 (straight portion) with sliding surface at base (dam-rock interface)	A	282	2.1	2.5
	C	566	6.5	7.3
	D	596	4.0	6.3
Block 17/18 (curved portion) with sliding surface at level 2353.0 m a.s.l.	B	1070	34	22
	C	1830	130	44
	D	1950	291	61

The upper part of the curved portion of the dam above the horizontal crack at elevation 2353 m a.s.l. would slide towards the downstream side by 1 m to as much as 2 m and crack openings of up to about 300 mm would occur under the SEE excitation. If the friction angle would be smaller than 45°, the sliding displacement would be even larger and the stability of the dam could be endangered.

In the case of block 5 with the most unfavorable cross-section in the straight portion, the nonlinear analysis without any anchors showed that the SEE shaking would cause a sliding displacement of up to 60 cm and minor crack openings of 2 to 7 mm. The remaining blocks in the straight portion would undergo substantially smaller displacements owing to the greater sliding resistance.

10. STRENGTHENING MEASURES

The curved portion of the dam will be strengthened by installing anchors (see Fig. 11) to achieve a factor of safety of at least 1.10 during the SEE ground motion. About 50% of the required anchoring force of 1900 kN/m length (i.e. 22,800 kN in a 12 m long block) will be in the form of post-tensioned anchors, which will ensure that there is practically no damage under the OBE loading. The rest of the anchoring force will be provided by passive anchors to prevent any substantial damage also in the event of the SEE.

An anchor force of 410 kN/m length will be provided in the straight portion of the dam by installing post-tensioned anchors to meet the stability requirement for the ice load combination only. The SEE ground motion could cause relatively small sliding displacements in this portion without endangering the stability of the dam.

11. CONCLUDING REMARKS

The static and dynamic stability analysis of the Illsee dam showed that the stability requirements were not satisfied for the ice and earthquake loading conditions. Therefore, the dam is currently being strengthened by installation of anchors.

The following rehabilitation measures are also being implemented to further enhance the safety of the 85-year old dam (see Fig. 11):

(i) Slots are being cut in the dam to relieve the stresses due to the chemical expansion of the dam.
(ii) New drainage boreholes are being drilled to reduce uplift and thus improve the dam stability.
(iii) The downstream face is being cleaned and resurfaced with shotcrete to repair the frost damage.
(iv) The existing spillway is being replaced by a new spillway to increase the discharge capacity and satisfy the freeboard requirements.

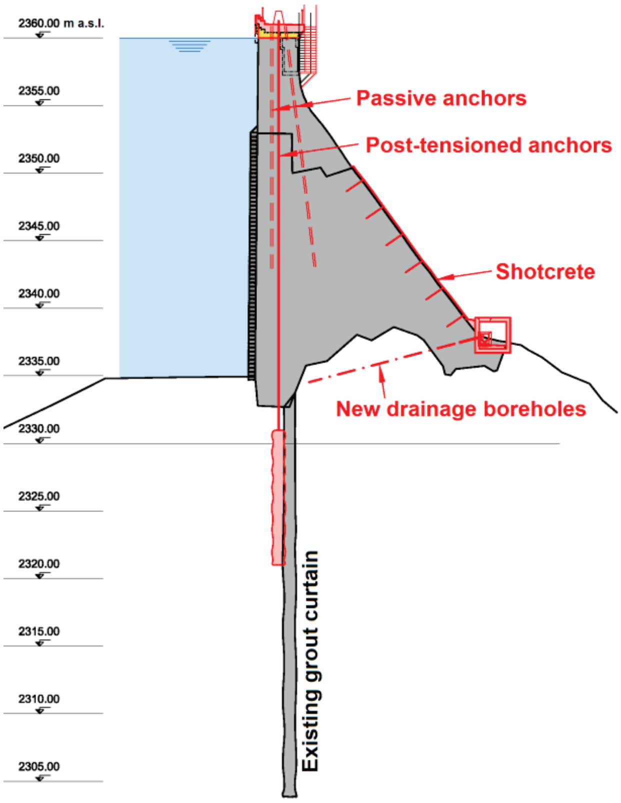


Figure 11. Strengthening with anchors and other rehabilitation measures

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