Seismic safety assessment of a weir dam and appurtenant structures - 3D static and dynamic analyses

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SUMMARY:
The seismic safety assessment of Verbois Dam, a weir dam on the Rhone River, is presented. An integral three-dimensional finite element model of the dam, including its appurtenant structures, steel gates and foundation is prepared. Calibration and validation of the numerical model are carried out through comparisons between measured and calculated dam temperatures and displacements. Static analyses of the dam are first conducted to evaluate the initial stresses and the displacements due to usual load cases, i.e. self-weight, hydrostatic pressure, silt pressure and temperature gradient. Then, dynamic analyses are run by direct time-step integration for three sets of accelerograms. Calculated compressive and tensile stresses in the dam and the appurtenant structures are compared to the dynamic strength of concrete. Then, dam's sliding and overturning stability is evaluated considering the maximal dynamic response. Finally, the seismic safety of the hydro-mechanical structures is evaluated regarding steel strength, gates jamming and jacks buckling.

Keywords: weir dam, hydraulic gates, seismic safety, stress analyses, stability analyses

INTRODUCTION

The seismic safety assessment of Verbois Dam and its appurtenant structures has been carried out in the framework of the respective program of the Swiss Federal Office of Energy (SFOE), the highest supervisory authority for all dams in Switzerland. The study has been has been performed in accordance to the Ordinance on the Safety of Dams and its directives for application issued by the Dams Department of the Swiss Federal Office of Energy [OFEG, 2002, 2003].

The study consists of the following main steps: (1) collection of data regarding the geometry and the configuration of the dam and its gates; (2) definition of the dam Class; (3) definition of the design seismic solicitations; (4) collection and analysis of data regarding the static and dynamic material properties of the dam and the gates, the geological and geotechnical conditions of the dam foundation, and the air and reservoir water temperature variation; (5) definition of the design load cases and load combination; (6) preparation of a detailed numerical model of the dam-gates-reservoir-foundation system; (7) calibration of the numerical model by comparing the calculated structural response to the corresponding data available from the monitoring of the structure; (8) eigenvalue analysis of the structures; (9) seismic analysis by direct time-step integration; (10) verification of the local stability of the dam and the gates; (11) verification of the global stability of the structures and the safe operation of the gates; and (12) assessment of the seismic safety of the structure. This paper presents the studies that have been carried out in order to assess the seismic safety of Verbois Dam.

2. DESCRIPTION OF THE STRUCTURE

Verbois Dam is located on the Rhone River, at about ten kilometers downstream of Geneva. It is a 36 m high, 410 m long concrete gravity dam with gated spillway and bottom outlet and an adjacent runoff-the-river power plant (Fig. 2.1). The volume of the impounded reservoir is 13.8 million cubic meters; the head between the normal upstream and downstream levels is approximately 20 m. The dam is composed of five structures separated by vertical joints, which are as follows from the left to the right bank:

(1) Left Bank Dam: a 25 m high, 130 m long concrete gravity dam shouldered by an earth fill shell on the downstream slope.
(2) Gated Spillway and Bottom Outlet Dam. The spillway has four 14 m wide bays that are separated by 5.5 m wide piers. Each bay is equipped with a 4.2 m high radial gate at the bottom and a 4.0 m high flap gate at the top. The total length of the spillway dam is 81 m; its height is 35 m. It is divided by a vertical joint in the middle of the central pier.
(3) Powerhouse. This building houses four units of a vertical-axis Kaplan turbine and an alternator. The structure is 70 m long and its maximum height is 36 m. A vertical joint divides the Powerhouse into two blocks.
(4) Service Building. The upstream wall of the Service Building constitutes a 26 m high, 55 m long gravity dam.
(5) Right Bank Dam. It is a concrete gravity dam which, similarly to the Left Bank Dam, is supported by an earthfill shell on the downstream slope. In plan, the longitudinal axis of the Right Bank Dam turns at 45 degrees towards upstream. Its length is 80 m and its maximum height is 24 m. Four vertical joints divide the dam into five blocks.

Figure 2.1. Verbois Dam and Hydropower Plant

The rock foundation consists of molasse sandstones. Glacial and alluvium deposits, as well as marls layers have been excavated in order to found the dam on a relatively hard rock, as are the sandstones.

3. METHODOLOGY OF THE STUDY

The evaluation of the seismic safety of Verbois Dam has been carried out in accordance to the Swiss regulations following the prescriptions as detailed in [OFEG 2002, 2003]. The seismic input, the type of numerical modelling and the method of numerical solution are determined according to the corresponding requirements of OFEG [2003] for a dam Class defined from OFEG [2002]. With a "reservoir height" of approximately 20 m and a reservoir capacity of 13.8 million m3, Verbois Dam is of Class I, i.e. the highest Class (see Fig. 3.1). Since the dam foundation is composed of hard rock, the foundation Class is defined as Class A.

The Swiss Norms [OFEG, 2003] require that the seismic verification of Class I dams should be carried out for an earthquake of a 10 000-year return period. This requirement has been followed for Verbois Dam. The peak ground horizontal acceleration a_h has been determined based on the MSK intensity of the dam site from the seismic map given in [OFEG, 2003] as $a_h=0.27g$. The peak ground vertical acceleration a_v has been adopted as equal to two thirds of a_h, i.e. $a_v=\frac{2}{3}\cdot a_h = 0.18g$. An elastic response spectrum with 5% damping corresponding to the class of the site foundation has been used in the current study.

The seismic behaviour of the structure has been investigated by means of the direct time-step integration method. Three series of three stochastically independent acceleration time histories have been generated so as to contain the defined peak ground accelerations, as well as to be compatible with the retained response spectrum. The dam-foundation system is subjected to the defined acceleration time histories applied on the numerical model boundaries in the cross-stream, the stream, and the vertical directions. The rock foundation is assumed to be massless.

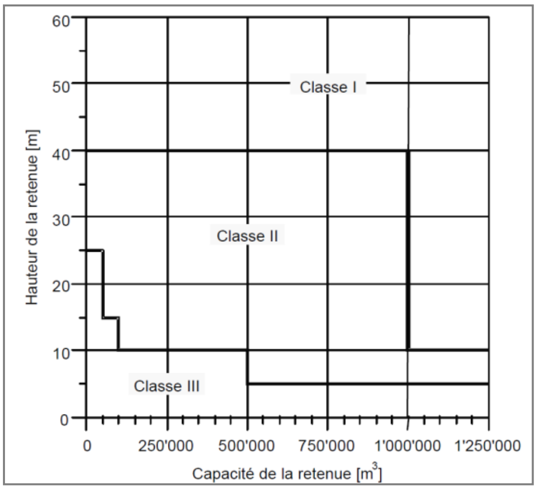

Figure 3.1. Classification of the dams in Switzerland according to [OFEG, 2002]

The geometry of all the concrete structures, as well as of the steel gates, has been defined on the basis of the Execution Drawings [SIG, a and b]. The location of the monitoring devices has been determined from the Instrumentation Layout Drawings [Gicot, 2005]. The dam and its foundation, as well as the gates are modelled in a single three-dimensional finite element model. The dam-reservoir interaction has been considered by means of the added mass method according to Westergaard approach [Westergaard, 1931; Kuo, 1938]. The numerical modelling, the computations and the post-processing of the results have been carried out by means of DIANA Finite Element Program, version 9.4.2 [DIANA, 2010].

Linear-elastic and isotropic material models are associated with the dam concrete, the foundation rock and the gates steel. The material parameters of the concrete and the rock have been evaluated on the basis of recent investigations on Verbois Dam, as well as on the preserved test data from the construction period of the dam. The characteristics of the steel used for the gates have been obtained from literature data. The dynamic Young's moduli of the concrete and the rock have been obtained from the static ones by increasing the latter by 25%. The dynamic modulus of the steel has been assumed equal to its static modulus.

The vertical contraction joints between the adjacent monolith parts of the concrete structures can cause non-linear effects in the structural behaviour. In case of relative displacements normal to the contact surfaces, the joints can open or close. In case of tangential displacements and closed joints, friction occurs between the adjacent concrete blocks. However, since nonlinear dynamic calculations would demand significant computational effort, we have associated linear-elastic properties with the joint elements used to model the connection between the adjacent blocks. The rigidity of the joints elements has been selected so as to allow for relative tangential displacements and to prevent opening at the joints. Finally, the calculated tensile stresses in the cross-stream direction have been evaluated in order to account for the fact that the joints can open.

The dynamic strengths of the concrete and the mass rock have been defined based on the available data on the static strengths and in using the recommendations of SFOE. Thereupon, the dynamic compressive strengths have been obtained by increasing by 50% the corresponding static compressive strengths:

$$f_{cd} = 1.5 f_{cs} \tag{3.1}$$

The dynamic tensile strengths have been obtained from the static compressive strengths as follows:

$$f_{td} = 0.1 f_{cd} \leq 4\ MPa \tag{3.2}$$

The shear strength of the concrete-rock interface, expressed by the friction angle, the dilation angle and the cohesion, is estimated on the basis of data published in the technical literature.

In order to verify the hypothesis made in the numerical modelling of the dam, the finite element model is validated for the static loading condition by comparing calculated displacements against the corresponding ones measured by the pendulums installed in one of the piers of the dam. Along the same line, eigenvalue analyses have been performed for each part of the dam, as well as for each of the steel gate types. These analyses help to define the eigen shapes and frequencies which activate the most of the effective mass and hence engender the most important dynamic responses. In order to evaluate the significance of the added mass of water, the eigenvalue analyses have considered the cases of both empty and full reservoir.

The structural safety is evaluated by checking the local and the global stability of the dam.

The local stability is checked by comparing the calculated stresses against the strength of the respective material (concrete, rock, steel). The local stability is guaranteed if the evaluated maximum compressive and tensile stresses do not exceed the corresponding strengths.

The global stability of the dam concerns the safety against sliding and the safety against overturning of each of the five parts of the dam. The global stability is checked based on the stresses calculated by the FEM analyses. For each structure, the stability against sliding is defined for a surface at the contact between the structure and the foundation. The stability is guaranteed if the peak shear resisting force is greater than the driving shear force, according to the following equation:

$$C + \sum V \tan(\varphi + i) \geq \sum H \tag{3.3}$$

where C, φ and i are respectively the cohesion, the friction angle and the dilatation angle of the concrete-rock interface, and $\sum V$ and $\sum H$ are respectively the sums of the forces normal and tangential to the potential sliding surface.

The stability against overturning is guaranteed if the maximum tensile stresses at the dam-foundation interface are lower than its tensile strength. If this is not the case, it would be necessary to check whether the sum of the stabilising moments is greater than the sum of the overturning moments.

4. FINITE ELEMENT MODEL

The concrete structures and the foundation have been modelled by 8-node and 6-node solid elements. Each node of the solid elements has three translational degrees of freedom. The joints between the five parts of the dam have been modelled by structural interface elements. Curvilinear shell and beam elements of both translational and rotational nodal degrees of freedom have been used to model the steel gates. Finally, the added masses representing the dynamic interaction between the water and the structures have been defined by means of point mass elements. In total, the model contains 81 011 nodes and 67 536 elements of which 55 959 solid elements for the concrete structures and the foundation, 4 960 shell elements and 1 872 beam elements for the steel gates, and 4 745 point mass elements for the Westergaard added mass. The Finite Element model of Verbois dam and its appurtenant structures is shown on Fig. 4.1.

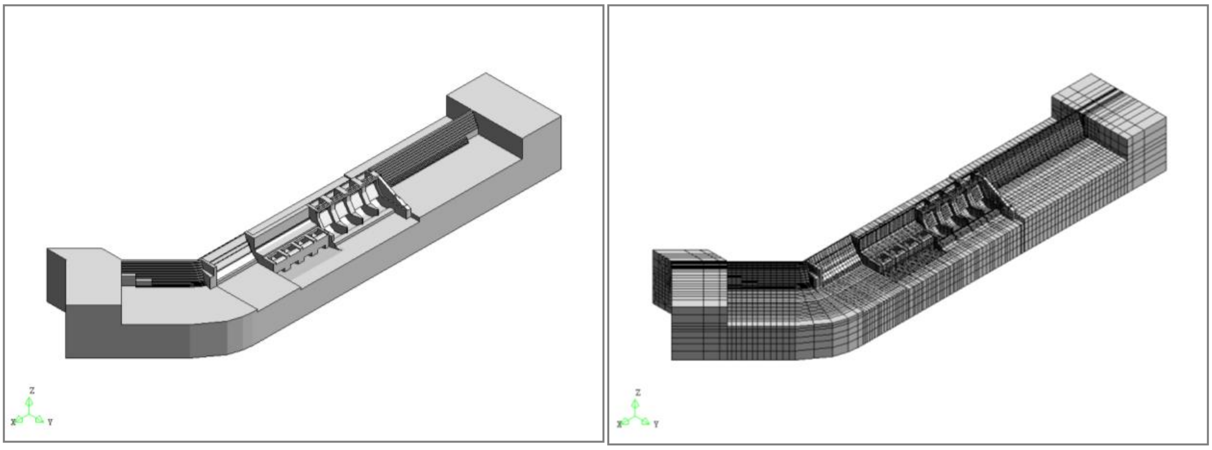

Figure 4.1. Geometry and integral finite element model of Verbois Dam including the steel gates

The local stability has been investigated for the load combinations given in Table 4.1. To define the global stability of the structure, the uplift acting on the dam at its interface with the rock foundation has additionally been taken into account, whilst for this check the temperature gradients in the dam body have not been considered.

Table 4.1. Investigated static and dynamic load combinations

Load combinations		Static			Dynamic								
Loads		SU0	SU1	SU2	DE0			DE1			DE2		
Self-weight		1	1	1	1	1	1	1	1	1	1	1	1
Earth pressure		1	1	1	1	1	1	1	1	1	1	1	1
Hydrostatic pressure		1	1	1	1	1	1	1	1	1	1	1	1
Silt pressure		1	1	1	1	1	1	1	1	1	1	1	1
Temperature gradients	Winter		1					1	1	1			
	Summer			1							1	1	1
Earthquake	Series 1				1			1			1		
	Series 2					1			1			1	
	Series 3						1			1			1

The earthquake excitation has been specified by three series of stochastically independent and response spectre compatible acceleration time histories in cross-stream, stream and vertical direction. The components of one of the series are presented in Fig. 4.2.

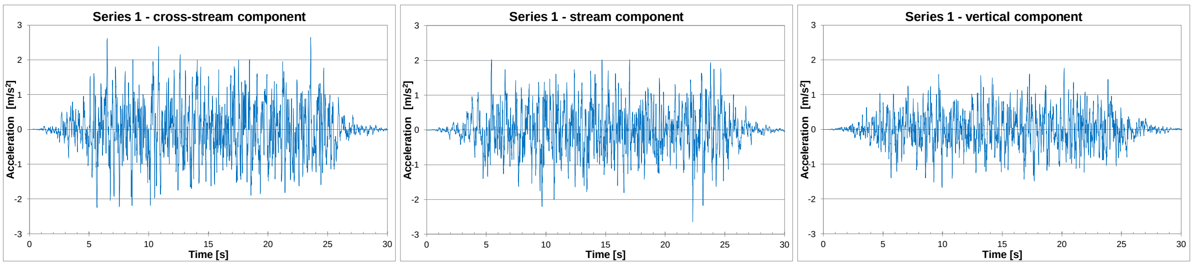

Figure 4.2. Generated input acceleration time histories (Series 1)

The material parameters of concrete, rock and steel have been estimated as follows (Table 4.2):

Table 4.2. Material parameters of concrete and rock

Material parameters		Units	Concrete		Rock		Steel
			Static	Dynamic	Static	Dynamic	Static and Dynamic
Modulus of elasticity	E	GPa	29	36	3.5	4.4	210
Poisson's ratio	ν	-	0.2	0.2	0.32	0.32	0.3
Density	ρ	t/m^3	2.4	2.4	0	0	8
Thermal expansion	α	K^{-1}	10^{-5}	-	10^{-5}	-	-
Thermal conductivity	λ	W/(m*K)	2.22	-	2.22	-	-
Volumetric heat capacity	c	$J/(m^3*K)$	$2.2*10^6$	-	$2.2*10^6$	-	-
Compressive strength	f_c	MPa	25	37.5	20	30	350
Tensile strength	f_t	MPa	2.5	3.75	2	3	350

5. MODEL CALIBRATION

The numerical model has been calibrated for (1) the increase of the hydrostatic pressure in case of reservoir filling that follows a reservoir draw-down and (2) the variation of the static loads.

Calibration (1) has been used to determine the apparent elastic modulus of the foundation rock mass. The geological and geotechnical conditions of Verbois dam site has been profoundly investigated by Joukowski [1938], Lugeon [1938, 1939] and Zschokke [1938] during the construction period, as well as by recent studies [SIA, 2003]. All investigations confirm that the dam is founded on molasse sandstones whose elastic modulus is more than five times lower than the one of concrete. To evaluate the apparent elastic modulus of the rock mass, we have determined the crest displacements of Pier III of the Gated Spillway and Bottom Outlet Dam due to rising of the reservoir water level from the bottom to Normal Water Level. Such conditions correspond to the reservoir filling following a draw-down, which was actually performed approximately ten years ago. By comparing the calculated to the measured displacements, we have evaluated the rock mass elastic modulus to be approximately equal to 3500 MPa.

The purpose of Calibration (2) is to adjust the thermal and thermo-mechanical properties of concrete so that the temperatures and the displacements obtained by the numerical model are approximately equal to the corresponding ones measured by means of the dam instrumentation devices. These calculations deal mainly with the changes of the ambient temperatures since the hydrostatic pressure during the normal operation of the facility remains almost constant with the reservoir at the Normal Water Level. To this end, we have utilised the data obtained from the instrumentation system for a period of five years, viz. January 2003 to December 2007. The strain and stress states of the dam have been calculated for load condition combining self-weight, constant u/s and d/s water levels corresponding to the normal operation conditions, and varying temperature gradients in the dam body.

Calibration (2) has been performed in two stages, the first of which consist in defining the transient temperature fields in the concrete structures and the foundation by means of a transient heat flow analysis. To define the boundary conditions, we used the air and water temperatures recorded by the dam instrumentation system for the investigated period of five years. The air temperatures measured have been increased by 1°C, 2°C and 6°C for, respectively, the winter, spring/autumn, and summer seasons. The initial temperature of the dam has been specified equal to the multi-annual mean air temperature of the site. The concrete temperatures are measured in four locations of Pier III: the upper two are on the upstream and the downstream sides 2 m below the crest (T1 and T3), and the other two are located some 6 m below the top ones (T2 and T4). The calculated vs. measured temperature comparison is shown in Fig. 5.1.

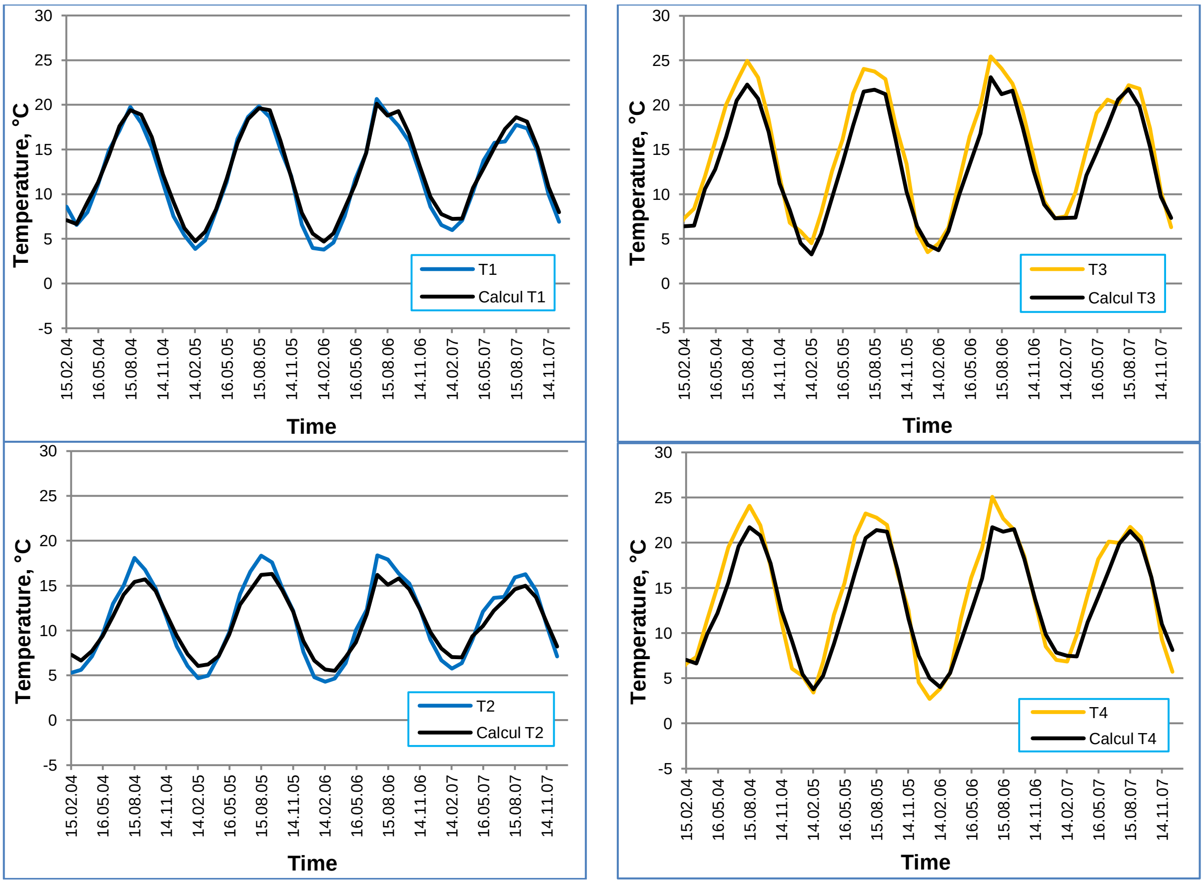

Figure 5.1. Calibration of calculated to measured temperatures

The second stage of Calibration (2) consists in a transient structural analysis aimed at defining the displacements due to the considered combination of static loads. The calculated displacements are compared to the ones measured by means of the inverse pendulum installed in Pier III in Fig. 5.2.

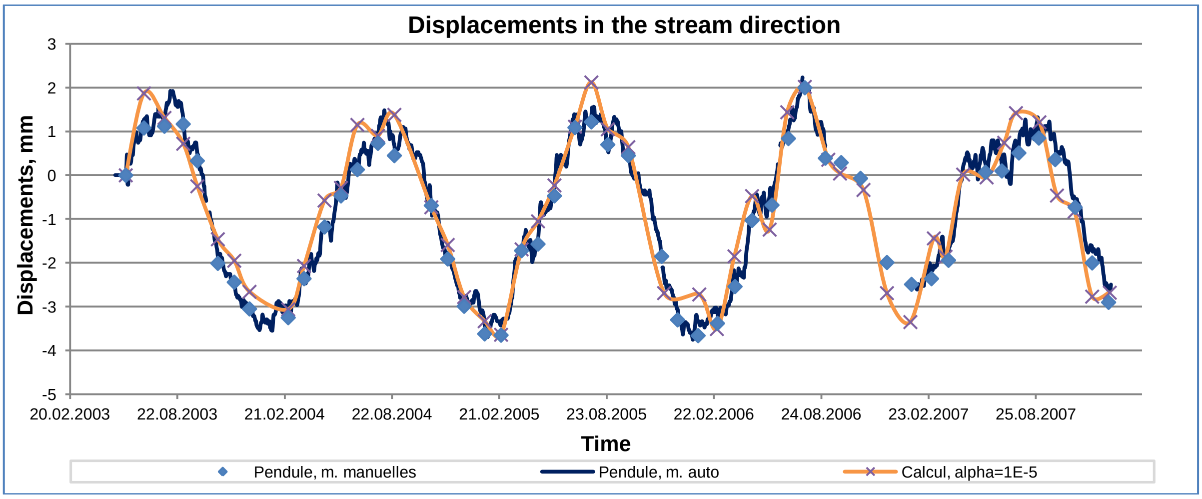

Figure 5.2. Comparison of calculated to measured displacements

The calibration is an iterative procedure and it has been repeated until a good comparison between the measured and the calculated quantities (temperatures and displacements) has been achieved.

6. STRESS AND STRAIN STATIC AND DYNAMIC ANALYSES

6.1 Verification of the local stability

The results from the static analyses show stress values that are quite low and meet the standard requirements toward the factors of safety related to the tensile and compressive strength of the concrete material. The load combination SU1 (winter) yields tensile stresses in the cross-stream direction, which in fact are eliminated by the opening of the vertical contraction joints of the structure. On the other hand, one can observe an increase of the compressive stresses in the same direction for load combination SU2 (summer), which is completely realistic. The characteristic static stresses are summarised in Table 6.1 (note that positive stress means tension).

Table 6.1. Static load combinations. Characteristic stresses of the structures

Structure	Principal stresses, MPa					
	SU0		SU1		SU2	
	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$
Left Bank Dam	0.27	-1.7	1.4	-1.7	0.28	-3.8
Gated Dam	1.0	-4.0	2.0	-4.0	1.0	-7.0
Powerhouse	0.75	-2.0	1.5	-2.0	0.7	-4.0
Service Building	0.1	-1.2	1.4	0	0.6	0
Flap gates	60	-60				
Radial gates	50	-150				

The characteristic dynamic stresses are given in Table 6.2. For the concrete structures, the envelopes of the dynamic stresses are almost identical for the three series of acceleration input.

Table 6.2. Dynamic load combinations. Characteristic stresses of the structures

Structure	Principal stresses, MPa					
	DE0		DE1		DE2	
	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$	$\sigma_{1,\,max}$	$\sigma_{3,\,min}$
Left Bank Dam	1.5	-3.5	2.0	-3.5	1.8	-4.0
Gated Dam	3.0	-6.0	3.5	-6.0	2.0	-7.0
Powerhouse	2.0	-6.0	2.8	-6.0	3.5	-8.0
Service Building	1.5	-3.5	2.0	-3.5	2.0	-4.5
Flap gates	350	-300				
Radial gates	200	-250				

The maximum tensile stresses are calculated in the Gated Spillway and Bottom Outlet Dam for the DE1 (winter) load combination (Fig. 6.1). They reach 3.5 MPa and approach the dynamic tensile strength of concrete. On the other hand, the maximum compressive stresses are obtained in the Powerhouse for the DE2 (summer) load combination. They approach -8 MPa and are well below the dynamic compressive strength of the material.

The maximum stresses in the flap gates reach the adopted strength of steel of 350 MPa. They occur instantaneously in the vertical beam “Chevron” in the middle of the gates. On the other hand, the maximum stresses in the radial gates of the bottom outlet reach 250 MPa. This stress occurs in the left most and the right most gates’ arms. Envelope stresses in the gates are shown in Fig. 6.2.

The most important displacement are in the stream direction (6 mm in the upstream wall of the Service Building and 22 mm at the top of the flap gates), followed by those in the vertical direction. The highest displacements in the cross-stream direction are obtained in the Gated Spillway and Bottom Outlet Dam and are in the order of 3 mm.

The stress and strain state analyses show that the local stability criteria are met for both the concrete and the steel structures of the dam.

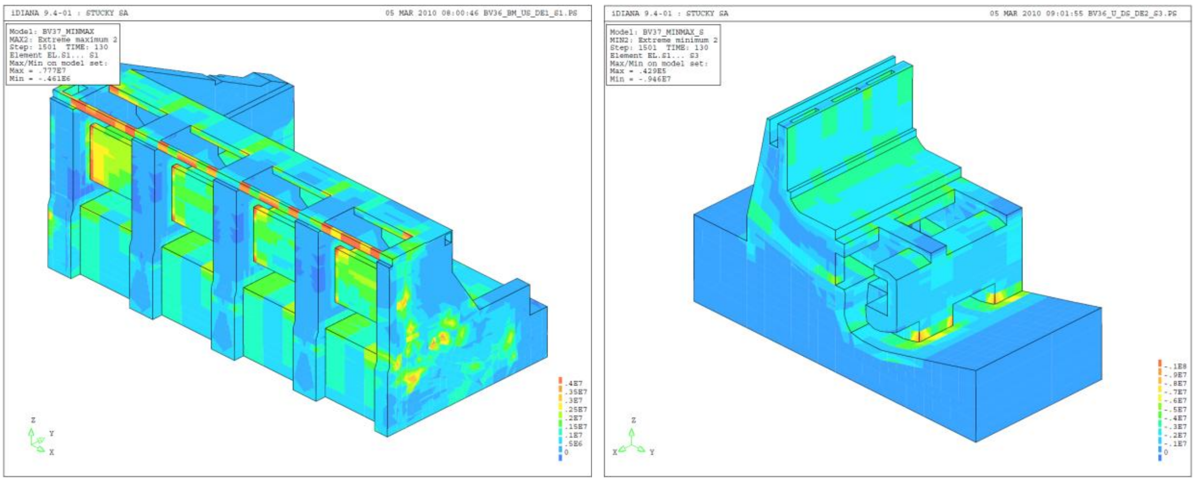

Figure 6.1. Envelopes of the maximum tensile stresses (Pa) for DE1 in the Gated Dam (left) and of the maximum compressive stresses (Pa) for DE2 in the Powerhouse

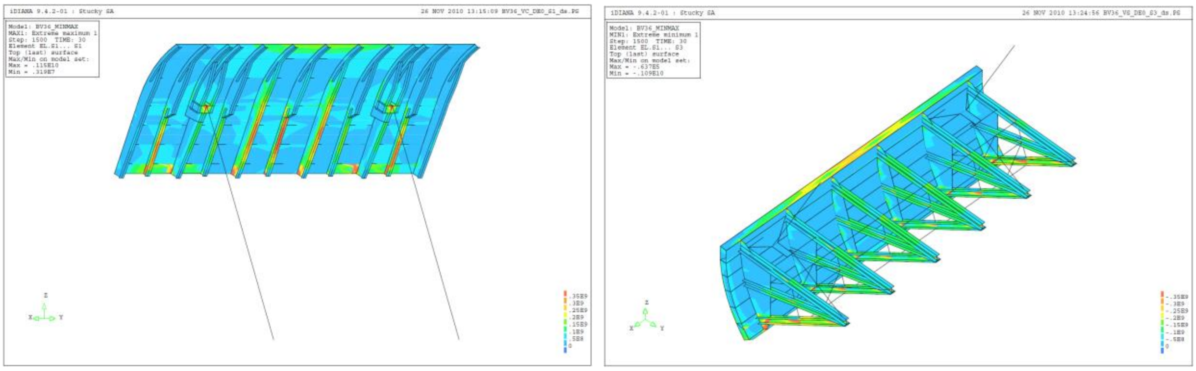

Figure 6.2. Envelopes of the maximum stresses (Pa) for DE0 in the flap gates (left) and the radial gates (right)

6.2 Verification of the gates jacks safety against buckling

The flap gates transfer the compressive loads to the concrete structure of the dam. The increase of the normal forces in the jacks can lead to buckling if the critical value of the normal force is reached. The latter is defined by the following formula (note that the negative value means compressive force):

$$N_{cr} = -\frac{\pi^2 EI}{l_0^2} \tag{6.1}$$

where l_0 is the length of the jack, E is the Young's modulus of steel, and I is the moment of inertia of the jack. For the flap gates jacks, N_{cr} = -6652 kN. The maximum effort obtained from the dynamic analysis for the investigated three series of acceleration time histories yield maximum normal force of -2525 kN, -2265 kN and -1946 kN. The corresponding factors of safety are 3.1, 2.9 and 3.4, and the corresponding compressive stresses are -40 MPa, -43 MPa and -37 MPa, respectively. Therefore, the safety against buckling of the jacks is guaranteed.

6.3 Verification of the gates jacks safety against jamming

During earthquake, the oscillations in the lateral direction of the piers of the Gated Spillway and Bottom Outlet Dam and of the gates can cause a decrease of the available clearance between the former and the latter. It is therefore necessary to verify that the serviceability of the gates is not endangered at any time during the strong ground motion due to a reduction of the clearance.

For the load combination DE0, the maximum displacements of the dam in the stream direction are of the order of 3 mm, whilst those of the flap gates and the radial gates are of the order of 2.2 mm and 1.2 mm, respectively. Hence, the maximum reduction of the clearance between the gates and the piers is 5.2 mm for the flap gates and 4.2 mm for the radial gates. Since the available clearance is 20 mm, it is concluded that at all times during earthquake the serviceability of the gates is guaranteed.

7. VERIFICATION OF GLOBAL SAFETY

It was proved necessary to verify only the safety against sliding since the tensile stresses at the dam-foundation interface are lower than its estimated tensile strength.

According to the recommendations in [Schleiss, 2004], we have adopted an angle of friction of the interface $\phi' = 56°$ and zero cohesion. The assumption for the angle of friction corresponds to a rock foundation of medium quality.

The normal and the driving forces at the potential sliding surfaces assumed for the parts of the dam have been obtained for each time step of the dynamic calculations. The minimum factors of safety computed are 2.53, 1.93, 3.68 and 1.07 for, respectively, the Left Bank Dam, the Gated Spillway and Bottom Outlet Dam, the Powerhouse and the upstream wall of the Service Building. All of them are greater than 1.0. Therefore, it has been concluded that the global safety of the dam is guaranteed.

8. CONCLUSIONS

The seismic safety assessment of Verbois dam has been performed by means of a three-dimensional finite element model of the dam, its appurtenant structures and gates. Performed static and dynamic analyses have shown that both local and global stability of the dam are guaranteed as well as the operation of hydro-mechanical equipments. Therefore it is concluded that Verbois dam fulfils all seismic safety requirements according to the Swiss regulations.

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