Reassessment of Dam Foundation Stability – the Case of Three Swiss Dams

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Abstract

The paper addresses the case of three Swiss dams, namely Spitallamm arch-gravity dam (in the scope of a project of rehabilitation and heightening), Seeuferegg gravity dam (heightening), and les Toules arch dam (rehabilitation). In 2009, the first two above cases are in the design phase, with construction scheduled during the next decade, whilst the last dam is currently under rehabilitation (2008-2011).

The three projects are reviewed with respect to the foundation stability conditions. The approach is similar in all cases: first the existing dam is back-analyzed, with emphasis on the foundation conditions, allowing the cross-checking of the design results with the available monitoring data (benchmarking). Then on this basis, the heightening and/or rehabilitation project is prepared and the most optimized solution in terms of safety but also of construction schedule and technology is recommended.

Introduction

In view of the rehabilitation or heightening of some existing dam, the initial project has to be retrieved from the archive (if ever available) and re-evaluated. Due to some specificities of the existing structure, in particular its behavior as monitored during its lifespan, but also with due consideration to the objectives of the rehabilitation or heightening and the interaction with the existing dam, the studies provide interesting and sometimes surprising outcomes.

This is especially true considering the progress made in the past decades in the field of rock mechanics and foundation stability analyses, namely:

- A better understanding of the rock mass behavior and the corresponding dam foundation aspects, which over the past decades have been more and more considered amongst the most critical issues of any dam project. This has been especially revealed following some terrific dam failures such as Malpasset arch dam occurred in 1959, involving foundation failure.
- The development of more sophisticated rock mass failure criteria, which mainly occurred in the 80s and 90s. The use of statistics relying on large data bases coming from many different sources throughout the world (job sites, mining excavations) greatly helped gain confidence into the approaches.
- The advent of computer science and the development of numerical modeling allowing the achievement of a large number of loading cases and scenarios during the design phase, to assess the behavior and safety of the dam foundation.

Based on the above three pillar aspects, the rock mechanics science applied to the dam foundation stability analyses has become a critical dam design issue that cannot be overlooked by any means and has to be addressed with due care and thorough consideration.

Below, the case of les Toules arch dam, Seeuferegg gravity dam and Spitallamm arch-gravity dam are reviewed, focusing on the foundation aspects of the design. Interestingly all three are rehabilitation or heightening projects, making the issue even more sensitive and tricky.

Rehabilitation of les Toules Arch Dam

Les Toules arch dam is located in Switzerland close to the southern border with Italy, in Canton of Valais. The dam was built in 1960-63 and enjoys a particular design, e.g. a pre-pack joint, very slender shape without abutment thickening in a large valley, high vertical curvature and no shear keys, etc. The dam is 86 m high and the crest across the valley 460 m long. The concrete volume is 235’000 m^3. The owner of the dam is Forces Motrices du Grand-St-Bernard (FGB).

The dam foundation lies on gneiss and mica schist rocks, forming alternate subvertical strips almost parallel to the valley. The gneiss appears to have better mechanical properties and to be more rigid than the mica schist; thus the dam rests on an alternate series of rock formations which have variable properties and stiffness, especially on the left bank (Figure 1).

Since 1980, monitoring surveys and pendulums show some irreversible displacements towards downstream with an increasing tendency. Preliminary analyses showed that the safety of the dam was marginal and thus the reservoir level was lowered as a safety measure; meanwhile more sophisticated studies were carried out and a strengthening concept was developed. More detailed analyses allowed understanding the behavior and weak points of the dam. Dynamic analyses revealed the necessity of cantilevers strengthening as well as shear keys in the middle part of the dam. On this basis, some strengthening alternatives were studied and compared. The final solution comprises:

- A unique downstream strengthening in a form of abutment thickening (concrete volume around 60'000 m^3), transferring the load from over-loaded cantilevers to the thickened arches;
- The creation of shear keys in the vertical joints;
- Local foundation treatment, in particular on the left bank at dam bloc No. 6, which is founded into the weakest rock formation of the site (mica schist);
- And some other minor rehabilitation or improvement works.

Figure 1 shows the dam elevation from downstream, where the strengthening is highlighted in grey. Figure 2 shows the typical cross-section of the dam with the strengthening (left), whilst the particular case of bloc No. 6, which is further explained below, is shown on the right hand side.

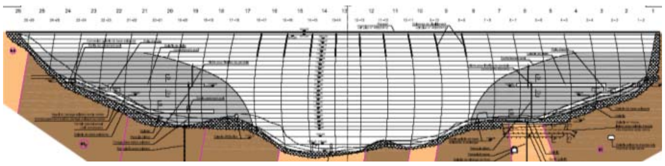

Figure 1: Rehabilitation of Les Toules arch dam, Downstream elevation, Strengthening project shown in grey

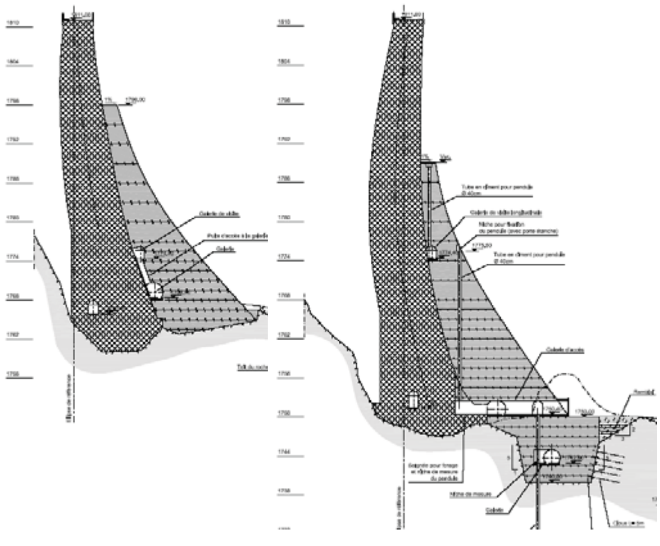

Figure 2: Rehabilitation of Les Toules arch dam, Typical cross-section (left), Cross-section at bloc No. 6 (right) with local deepening of the strengthening

In a first step, the consideration of the geological conditions and geometrical constraints of the project made it clear that the most critical sliding mode of the dam foundation would be through the rock mass, and not along pre-existing joint sets (plane sliding or wedge), which all are observed to be subvertical or steeply dipping into the foundation.

On this basis, the dam foundation stability conditions have been assessed, and found barely acceptable if not below the standards published by the Federal Office of Energy (FOE), in charge of dam safety in Switzerland [3, 4]. This was especially true for the extreme loading case (earthquake) on the left bank by bloc No. 6, where the dam is founded into mica schist rock featuring the weakest rock mechanics parameters of the whole dam site.

Then the stability conditions were checked with the consideration of the rehabilitation solution. Due to the particularly poor conditions of the rock foundation of bloc No. 6, the strengthening had locally to be deepened by some 10 m to found the dam further down into the rock mass (Figure 2, right) and reach regular safety conditions.

Of particular interest is the execution of the local deepening of the strengthening foundation. The excavation was carried out by blasting, complying with very stringent requirements in terms of allowable wave velocity (12 mm/s). Such requirement was imposed by the upstream proximity of the arch dam. It is also worth noting that for safety reasons during the rehabilitation works, the reservoir water level is lowered by 30 m. Figure 3 shows a picture of the local deepening of the foundation (left), while the critical cross-section of bloc No. 6 after excavation is presented on the right. Although such unusual dam section with a hole right downstream was temporary (for some weeks only, until the strengthening is concreted and the hole filled up again), a particular attention was given in the design of this temporary loading case, and during the job site supervision. The realization of this delicate job took place in 2008 and occurred without any particular problem.

Figure 3: Rehabilitation of Les Toules arch dam, Bloc 6, Excavation of the local deepening of the strengthening

After concreting of both left bank and right bank strengthening structures against the downstream face of the existing dam, the rock mass below the dam and new foundation will be extensively grouted and eventually a drainage system will be provided. The completion of this challenging dam rehabilitation job site is scheduled for Spring 2011.

Heightening of Seeuferegg Gravity Dam

The KWO Plus project foresees the heightening by 23 m of the Grimsel reservoir level, from 1908.74 masl to 1931.74 masl. This implies the heightening of both Seeuferegg gravity dam and Spitallamm arch-gravity dam (Figure 4). Both dams are located in Canton of Bern and the owner of both dams is Kraftwerke Oberhasli A.G. (KWO).

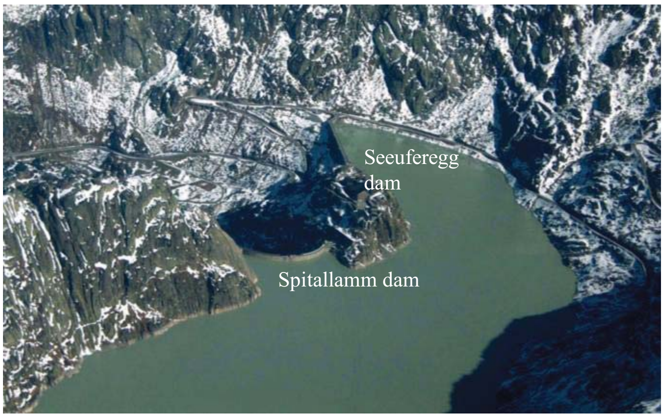

Figure 4: Picture with Seeuferegg gravity dam and Spitallamm arch-gravity dam

The existing Seeuferegg gravity dam is 42 m high and was built in 1932. To increase the reservoir volume for winter production, the dam is projected to be turned into a hollow gravity dam, the heightening being built against the upstream face of the existing dam inside the reservoir (Figure 5). Due to the reservoir level heightening, the water pressure acting on the dam face will be notably higher than at the existing dam; consequently the uplift pressure developing in the dam foundation will be higher as well. Therefore the dam foundation stability conditions must be carefully assessed and checked. One of the design advantages of turning a gravity dam into a hollow-gravity dam is the reduction of uplift pressure acting in the foundation thanks to the drainage provided by the hollow cavities.

The dam is founded in a very hard, jointed rock mass (mainly granodiorite). In such hard rock the stability issue is usually governed by the joint sets, the stress field playing a very minor role. The stability analysis is performed by using joint-based design methods and considering three different but complementary approaches that will allow cross-checking:

- A 2D analysis (failure through the rock mass);
- A 2D analysis considering the joints observed on site that are in a particular unfavourable position (dip and dip direction toward downstream – failure on a single joint);
- A 3D analysis considering the geometrically possible wedges with the joints observed on site by the geologist.

Salient outcomes of the above analyses are reviewed and discussed below.

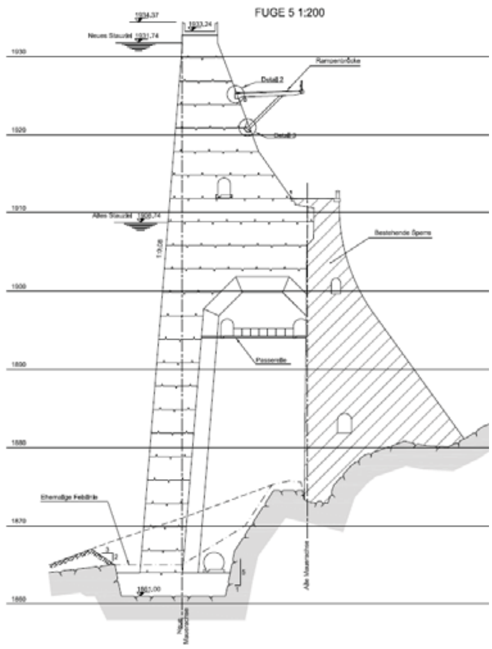

Figure 5: Heightening of Seeuferegg gravity dam, Typical cross-section of the heightened hollow-gravity dam (existing dam on the right, drawn with hatch)

Failure through the rock mass

Sliding through the rock mass is a pure bi-dimensional case. Such sliding mechanism is likely to occur in hard, jointed rock mass such as encountered at Seeuferegg dam site.

All critical dam cross-sections are checked with the most unfavourable sliding lines, given with the prevailing topographical conditions.

The analysis is performed by considering a foundation drainage effectiveness of E=50%, meaning that the drainage obtained from the heel gallery and with the hollow cavity is only 50% efficient.

The first relevant result is that heightening the dam reduces the factor of safety by 25-50% as per the section considered.

Furthermore, the analysis reveals that the different loading cases meet the requirements published by the FOE.

A sensitivity analysis performed on the drainage effectiveness (applying E=0%, thus no drainage effect considered) shows that the reduction rate of the factor of safety is above all dependent on the water height acting on the upstream dam wall. The maximal effect obtained with E=0% reduced the safety factor by 14%, which remains above the requirements.

Dam foundation sliding on two existing planes

The review of all joint sets observed by the geologist allows identifying a critical combination on the left bank abutment. Figure 6 shows that a tensile crack propagating into the rock mass below the upstream dam face might combine with joint set No. 1 (daylighting on the downstream slope and clearly localized on site) and joint set No. Q, to form a potential hazardous wedge.

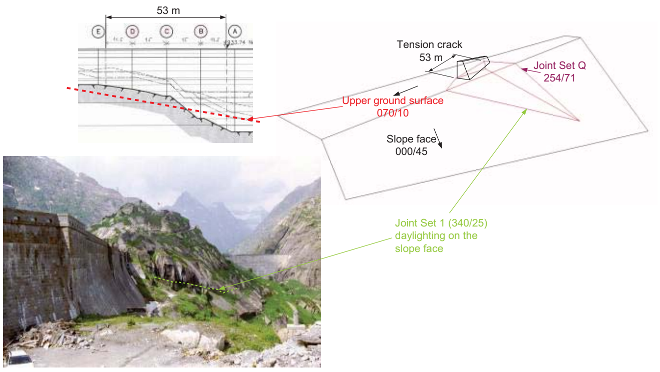

Figure 6: Heightening of Seeuferegg gravity dam, Foundation stability analysis of a critical wedge formed by two existing joint sets

Interestingly the wedge defined and presented above is not touched by the existing gravity dam, since it is located on the top of the left bank, above the current dam crest elevation.
First, the wedge stability conditions are checked for the present conditions, and the analysis shows that the extreme loading case (earthquake) is not complied with.
Then, the wedge stability conditions are checked with the heightened dam. The results obtained from the analysis are clear-cut: in no cases the safety requirements are reached, even when the uplift pressure acting on both joint planes is strongly reduced (drainage efficiency). Therefore the implementation of remedial measures to increase the safety factor is necessary.

Remedial measures to improve dam foundation conditions
Several options are studied, such as:
- Excavation of galleries in the foundations to intercept the joint planes and filling up the galleries with concrete (creation of shear key galleries);
- Implementation of prestressed anchors in the downstream slope to increase the stress acting perpendicular to the critical joint and thus add resistance against sliding;
- Deepening the dam foundation level below the elevation of the critical joints.

A technical and economical analysis of the alternatives shows that the third alternative is the most radical in the sense that it gets rid of the joint presence problem. Both other alternatives endeavour to improve the joint conditions by increasing either the cohesion along the joint (shear key galleries), or the force acting perpendicular to the joint (prestressed anchors). With respect to that, deepening the dam foundation elevation is most probably the most reliable alternative. It is also the most flexible solution to implement, since it can be adapted on the spot to the effective rock conditions faced during excavation.
On this basis, the dam heightening design was adapted in the area of the left bank, going deeper into the rock mass to found the dam below the elevation of the critical joints, and the wedge stability issue was resolved.

Rehabilitation and Heightening of Spitallamm Arch-Gravity Dam

As presented above (Figure 4), the Seeuferegg gravity dam and Spitallamm arch-gravity dam form together the Grimsel reservoir that will be heightened by 23 m. Therefore, the project of rehabilitation and heightening of these two dams is been carried out in parallel.
The Spitallamm dam is a 114 m high, single curvature arch-gravity dam, built in 1932. The crest length is 258 m and the concrete volume 340’000 m^3. According to the actual project, the dam is heightened by 23 m, in the form of a double curvature structure applied on the upstream face of the existing dam, after removing the existing upstream face which is not in good conditions (Figure 7).

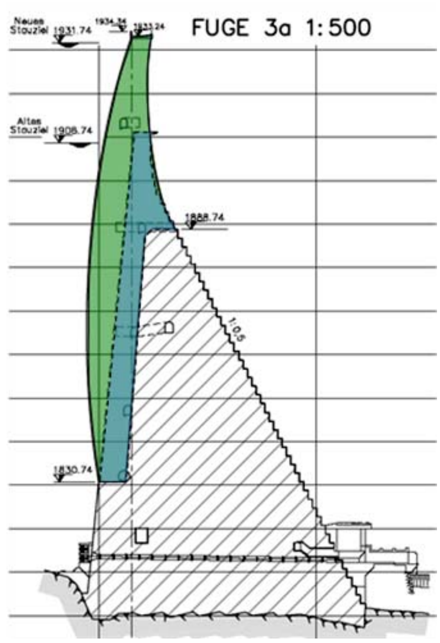

Figure 7: Typical cross-section of Spitallamm arch-gravity dam (in blue: existing dam concrete to be removed; in green: heightening project)

Due to the relatively important increase of the reservoir level, one of the major tasks of the rehabilitation and heightening project is the foundation stability analysis. The general approach of the abutment stability analysis presented above is also employed in the case of Spitallamm dam.
In-situ investigation campaigns revealed that the Spitallamm dam is founded in a very hard, jointed rock mass (Grimsel granodiorite), and that the right bank is partially founded in a mylonite. This important change of the rock mass on the right bank did not impact on the dam behavior until now. According to the geological surveys of the site, no major joint sets are identified on the left bank. Therefore, the stability analysis on the left bank can be discarded and consequently only a 2D analysis is carried out.

Several joint sets were identified on site with some discrepancies between different sources. Combining the available sources, a stereographical plot (low hemisphere projection) of the joint sets and orientation (Nord-East) of Spitallamm dam is prepared (Figure 8). In the same plot, potentially unstable wedges are indicated by a full circle. Potentially unstable wedges are situated in the intersection of joints "K", "S" and "C" with a horizontal joint "4". Empty circles show wedges based on joints with lower confidence; they are observed by one source only or by very old geological examinations.

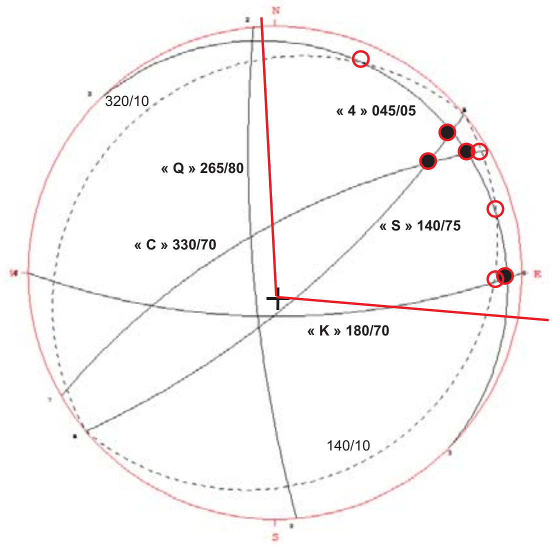

Figure 8: Stereographical projection (lower hemisphere) of the joint sets for Spitallamm dam

Based on the available geomechanical data, the following values of friction angle meet the general consent:
- Basic friction angle: $\phi_b = 32°$
- Angle reflecting joint roughness: $i = 8°$
- Representative value of friction angle: $\phi = \phi_b + i = 40°$.

For the stability analysis, the residual value of cohesion c' = 0 kPa is used.

Methodology and design criteria

The most complete and reliable 3D foundation stability analysis has been proposed by Londe [1]. The method provides very comprehensive and logical results in the case of well-defined main discontinuities in the rock mass. In the case of non-present (or not observed) discontinuity, the analysis has to be carried out with other methods. Lombardi [2] reports the possibility to underestimate the factor of safety in the case of two blocks that superpose; in this case the method takes into account two times resistant forces. Because of a very conservative hypothesis, such 2D analysis could be useful as a first estimate of a safety factor. Thus the method is employed to assess the stability with the usual loading case. On the other hand, the 3D analysis (Londe) considers three loading cases described below.

The 3D foundation stability study is coupled with static and dynamical analyses of Spitallamm dam using the finite element method. The overall procedure can be explained as follows: a 3D finite element analysis of the dam body and gravity abutments is used as a means of obtaining interface forces between the dam and its foundation for the load cases presented below. The analysis employs a representative domain, i.e. with an appropriate extent and boundary conditions and appropriate material properties. The zones or parts of zones, which interact with specific wedges, are specified in the wedge stability analysis, thus making the wedge stability calculations independent of the calculation of the dam stresses and interface forces. Therefore, the wedge stability analysis is performed independently of the stress and interface force analysis of the dam/foundation system. Displacement of abutments is allowed to interact with the dam body and thus additional forces imposed from the dam due to such displacement are feed-back from the 3D finite element analysis of the dam body into the wedge stability analysis.

A review of existing international codes in the matter of factor of safety shows that two philosophies tend to clash:

1) the US approach, which requires a high factor of safety, taking into account cohesion values;
2) the European approach, which indicates a lower factor of safety but without considering any cohesion.

The Swiss Federal Office for Energy FOE issued directives concerning the safety of dams [3, 4] and the present study is carried out in accordance with the standards. The directive recommends three loading cases that have to be considered in the stability analysis and corresponding factors of safety (Table 1).

TABLE 1: REQUIRED FACTORS OF SAFETY FOR THE THREE LOADING CASES, WITHOUT CONSIDERATION OF COHESION

Loading cases		
Usual loading case	Unusual loading case	Extreme loading case
1.50	1.30	1.10

Two-dimensional analysis

Based on a 2D analysis it can be concluded that the calculated factor of safety in the case of usual load case is not sufficient (1.26) on the right bank. However, this result must be considered very carefully and put into perspective, in order to evaluate the real possibility to have a failure. The analysis is carried out by implementation of Lombardi's approach, using some conservative hypotheses. According to the proposition of the author [2], it is not realistic to assume that the rock wedge of unit thickness is sliding along one discontinuity; it would rather mobilize the shear strength within the rock mass. Thus, it is proposed to introduce a factor of correction for the friction angle:

$$\mathrm{tg}\phi_c = M \cdot \mathrm{tg}\phi \qquad (1)$$

where M depends on angles ω and β that form the unstable block. In this study, no correction of the friction angle is introduced.

The second conservative hypothesis concerns the hydrostatic pressure at the dam heel. The above shown results assume 100% of hydrostatic pressure acting on the dam heel (0% effectiveness of the drain). According to references [5, 6], the drain effectiveness varies from 25% up to 67% with a mean value of about 50%. Taking into account 50% of drain effectiveness, the safety factor raises to 1.94 (instead of 1.26). In our opinion, this value can be considered as more realistic than the previous one.

Three-dimensional analysis

The following general points were applied to the wedge stability analysis:

i) The geometry of the 3D wedges are defined by 2 or 3 sliding planes and one tension plane (P4 in Figure 9);
ii) The sliding resistance for a given plane is specified by various strength options which may be different for individual planes;
iii) Application of static forces on each plane from uplift or other loads such as passive earth or rock pressure;
iv) Application of body forces due to the wedge self-weight and the buoyancy effects from saturated wedge rock, plus instantaneous loads given by the three-component instantaneous seismic acceleration;
v) Application of static and seismic forces imposed from the dam body onto the dam/wedge-rock interface which belongs to the specific wedge;
vi) Calculation of time dependent safety factors and cumulative deformation if sliding occurs.

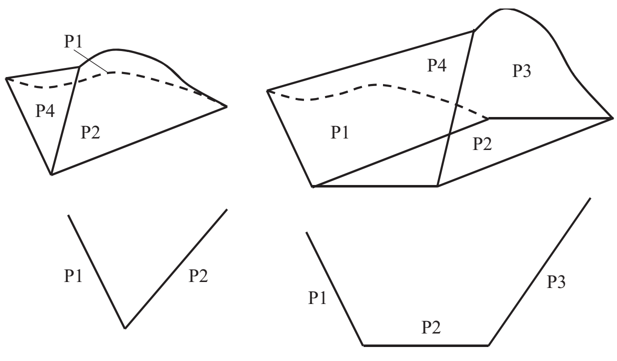

Figure 9: Schematic representation of 3- and 4-plane 3D wedges

Possible wedge geometries are illustrated in Figure 9 for both 3-plane and 4-plane wedges. Wedge sliding can take place on any of the individual slide planes: P1, P2 or P3 or along the intersection between two adjacent planes P1/P2 or P2/P3. After careful consideration of possible wedge sliding, only a three planes solution is used here. The static and dynamic values of the wedge safety factors are shown in Table 2.

TABLE 2: STATIC SAFETY FACTOR OF WEDGE AND MINIMUM DYNAMIC SAFETY FACTORS FOR THE THREE RECORDS USED IN THE DYNAMIC ANALYSIS

	Static loading	**Dynamic loading – DE1**		
		MCE1	MCE2	MCE3
FS	3.90	1.78	2.06	2.17

The 3D stability assessment of potentially unstable wedges on the right bank shows clearly that the critical wedge meets the required safety factors for static as well as for dynamic loading conditions. The dynamic stability analysis has been carried out for the three seismic records used for the dynamic finite element analysis of the dam.

Conclusion

A methodology employed for the assessment of the foundation stability of three existing Swiss dams: Spitallamm arch-gravity dam, Seeuferegg gravity dam, and les Toules arch dam, has been presented in the frame of heightening or rehabilitation dam projects. The approach is similar in all cases: first, the existing dam is analyzed with emphasis on the foundation stability; among others this allows cross checking the results with available monitoring data. Then, the heightening and/or rehabilitation project is developed and the most optimized foundation solution in terms of safety but also of construction schedule and technology is recommended.

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