ARCH DAM ANALYSIS WITH BASE JOINT OPENING

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1. ABSTRACT

The Schlegeis Arch Dam was designed as double-curvature arch-gravity dam in a wide spanned valley. The design and safety analyses were carried out with linear numerical models. As it is shown, the linear analysis provides an appropriate representation of the overall bearing behavior of the structure. Since the first impounding the dam is operated under planned conditions.
However, during the first filling of the reservoir unexpected high seepage into the bottom gallery was monitored. This was explained by local deformations at the dam's upstream heel. The construction of an elastic cut off wall solved the seepage problem.
To find an appropriate model for the interpretation of measured data and to derive an answer for the structural safety and integrity – considering this local phenomena - a detailed finite element model was developed. The linear analysis results show tensile stresses at the upstream heel of the dam. Paying attention to the geological site condition the model was updated with a perimetral base joint at the abutment, to allow the separation of the dam from the rock. To reduce the computational effort afterwards a coarse model was discretized.

This contribution deals with the numerical model assumptions and discussion of results gained with a continuous base joint over the entire dam abutment. The results show an opening of the base joint. The arch dam bearing behavior enables a safe redistribution of stresses. The results of these investigations are intended for further discussions in the community of Dam Engineers.

Keywords: Arch Dam Analysis, Base Joint Opening,
Stress Analysis, Dam Safety Assessment

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2. DESCRIPTION OF SCHLEGEIS DAM

The Schlegeis arch dam is the main structure of the Zemm power plant in Austria. The dam was concreted between 1969 and 1971. The first filling was commenced in 1970 and full storage level was reached 1973.
The main data of the dam are as follows:

- Height 131m
- Crest length 725m
- Crest width 9m
- Maximum dam thickness 34m
- Total Concrete Volume 960000m^3
- Live Storage 127 Mill.m^3

The foundation of the dam consists of fairly uniform gneiss. It's schistosity plane strikes approximately parallel to the right bank abutment and has a very steep dip towards downstream. The intercalation of soft biotite schists in the schistosity plane of the gneiss has a thickness of up to several decimeters.
The grout curtain was built vertical at a distance of about 4m off the upstream dam toe. Due to the grout curtain a permeability of 1 Lugeon was achieved. About 3m off the grout curtain an inspection gallery which is open towards the rock was located directly on the surface of the foundation rock. Drainages were drilled from this gallery and from downstream to relieve the pressures in the lower part of the dam foundation.

Behavior of the Dam

The dam is installed with different kind of instrumentation system as these are:

- Plumb lines down to 80 m depth into the foundation
- Extensometers and Piezometers
- Uplift pressure cells embedded on the dam foundation.

The readings taken during the first filling confirm the behavior of the dam predicted by computations. However for the dam foundation a different behavior was envisaged.

During the first filling of the reservoir, a maximum rate of seepage of about 250 l/sec were encountered. The extensometer readings suggested a clear relationship between the width of rock joints beneath the upstream toe of the dam and the seepage flow.

Measurements of uplift pressure showed values of about 100% of the reservoir head upstream the inspection gallery, whereas downstream of the gallery the uplift pressure was reduced to 10% of the reservoir head in maximum.

Measures taken to improve the situation

Based on extensometer readings it could be elaborated, that high strains occur in the uppermost part of the foundation rock (of about 5m in depth) in a close vicinity to the upstream dam base. These led to the opening of cracks into the grout curtain and resulted in high seepage inflow into the gallery. Due to the presence of the gallery the uplift pressure could be reduced significantly.
To prevent this water seeping into the gallery an elastic cut off wall was constructed with 6 m into the rock and made by boreholes of 128mm in diameter. The upper part of the cut off wall is integrated into a reinforced concrete vault sealing the inspection gallery towards downstream.
After completion of the grouting works a drainage curtain inclining towards downstream was drilled.
After completion of the works the amount of seepage were reduced to 25 l/sec without increasing the uplift pressure which impairs the stability of the dams.

3. SYSTEM DESCRIPTION

Finite Element Mesh

The Finite-Elemente discretization for the dam and a sufficient portion of the foundation is made on the basis of quadratic, isoparametric 20 and 15 node volume elements. For the fine mesh 6 elements are foreseen in radial direction at the base of the highest blocks and three at the crest. The dam has 2553 elements and the foundation 8170 (see fig. 1). The coarse mesh has three elements in radial direction over the whole dam with 246 elements for the dam and 896 elements for the foundation. In both cases contact elements are introduced between the dam and the foundation.

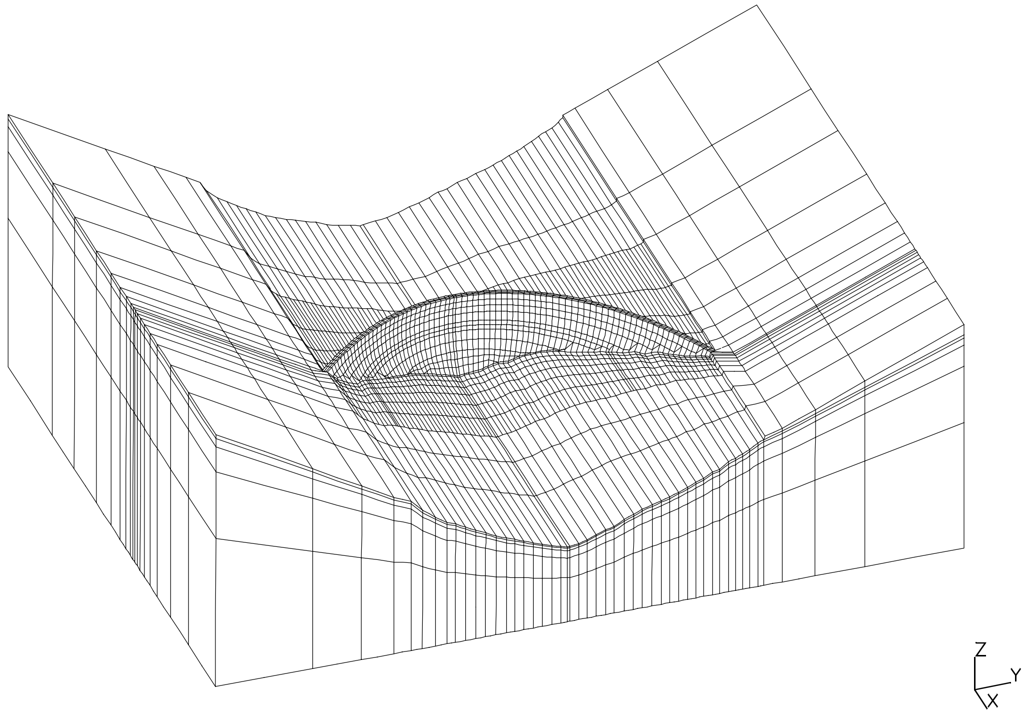

Figure 1: Downstream View of the Fine Finite Element Mesh

Material Behavior

The material behavior for the concrete is anisotropic for the loading dead weight to simulate the construction of the dam. For the subsequent loading cases an elastic material behavior is assumed. The rock foundation is modeled with an orthotropic material law according to the direction of the schistosity.
The material behavior are given in the following table:

	Rock	Concrete
Young's modulus E [GPa] (for rock E_{II})	30	25
Young's modulus E [GPa] (for rock E^)	10	
Poisson ratio n	0.17	0.17
Density [kg/m^3]		2400
a_T		8.10^{-6}

The joint behavior of the interface between dam and rock foundation is modeled with isotropic Coulomb friction. The friction angle is $j=45^o$ and no tensile stresses are allowed for being transmitted across the surface.

Applied Loading

For the investigation of the behavior of this arch dam two different analyses were carried out - first a linear analysis with a closed base joint and second a non-linear one with the possibility of opening of the base joint.

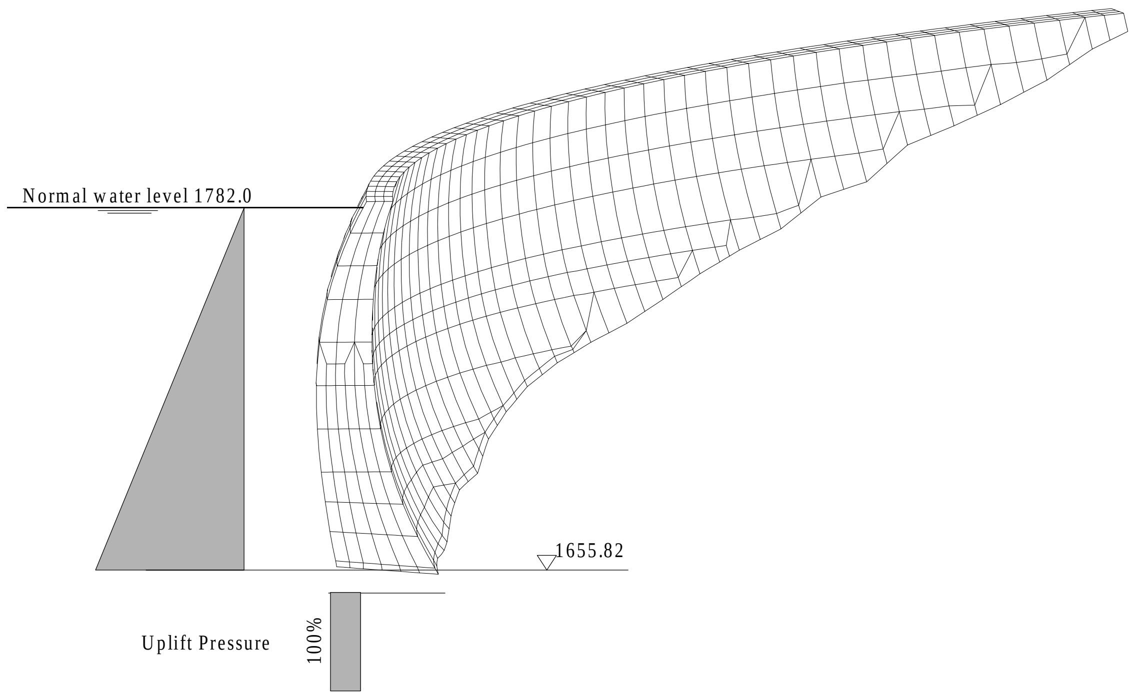

Figure 2: Dam Model, Water and Uplift Loading

The assumed loading conditions for each case is as following:

- dead weight loading - with independent columns
- water loading
- temperature loading
- and uplift pressure loading.

The water loading acts at the upstream face of the dam. The uplift pressure is included in about one third of the radial direction with 100% of the related water level height. The water loading and uplift pressure assumptions are shown in fig. 4.
For the loading case temperature the distribution of a measured summer-temperature field relative to the joint closing temperature is used.

4. NUMERICAL PROCEDURE

Contact Algorithm

The contact procedure required accounts for opening, closing and for frictional behavior in the defined interface. Numerical investigations are used to verify the numerical performance during static and dynamic analysis of joint elements (ICOLD benchmark, 1994 and 1996). Within the IV^{th} benchmark the evaluated results by nonlinear block joint behavior during earthquake excitation with different finite element codes were compared against each other.

The contact condition together with the defined numerical parameters for contact formulation are displayed in fig. 3. Isotropic Coulomb model with a friction angle of $j = 45$ is used. The allowable shear stress transmitted within the contact surface is calculated with $t_{max} < m*p$ for sticking state; the surfaces are glued to each other. If the current shear stress t equals t_{max}, the sticking state changes to sliding. This sliding is controlled in the program by $g_{elastic}$ value, which is an allowed "sliding deformation" prior t_{slide} is reached.

The contact algorithm itself is realized by a so called softened contact condition. This needs the definition of a contact pressure p_0 at the status closed. At a relative deformation between the two contact surfaces of c_0 zero pressure is transmitted in this contact zone. The pressure clearance relationship itself is an exponential function, resulting in a penetration of two bodies if the contact pressure is above p_0.

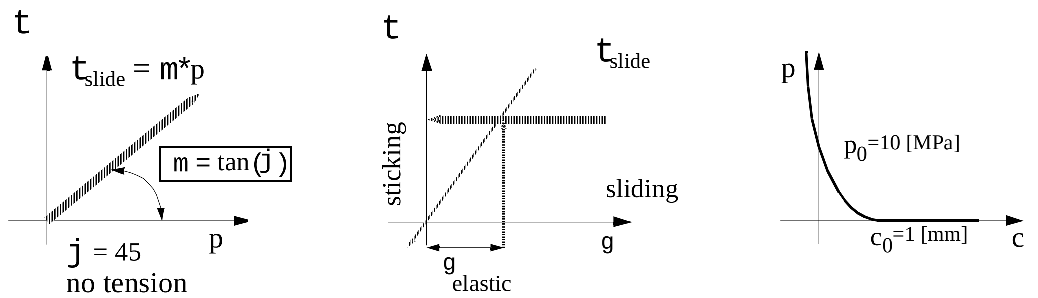

Figure 3: Coulomb friction with softened contact condition

5. LOADING SEQUENCE

The dead weight loading case is modeled taking the construction procedure into account. The concrete blocks are constructed as columns separated by vertical block joints. After completion of construction the block joints are grouted at a specific temperature. After the block joint grouting the arch dam supporting behavior is enabled for further loading cases. In this analysis the dead weight is applied for the entire dam structure with a material model with only significant vertical stiffness but reduced tangential and radial stiffness.
In the context of the used finite element code the change of material parameter – as this is necessary from dead weight application to water loading – would lead to a stress redistribution in subsequent loading cases. Therefore the stress state due to dead weight is written on file and reread into the model as initial stress state by applying dead weight loading. The equilibrium iteration carried out gives none deformations, but a changed structural behavior for the subsequent loading cases as these are water, temperature and uplift pressure loading.

6. ALLOCATED COMPUTER RESOURCES - PROGRAM USED

The computations are carried out on a Silicon Graphics Origin 200 Computer with 2 CPU’s. The computer runs Irix 6.5 operating system and has 1024 MB memory installed. The finite elemente code used is Abaqus 5.8.

	Fine Model	Coarse Model
Scratch file	3.7 GB	255 MB
Result file	100 MB	100 MB
CPU Time all loading cases	42 h	43 min

7. PRESENTATION OF RESULTS

For reference purpose the results of a linear model are presented in terms of minimum principle stress and middle principle stress. The minimum principle stress represent for dead weight the vertical stress and for further loading hoop stress in the dam body. The middle principle stress represents the vertical stress component in the dam body. In the vicinity of the abutment the direction of the principle stresses have no specific orientation. The nonlinear model - with the proposed base joint - is presented for fine and coarse discretization.

Linear Model – Closed Base Joint

The presented dead weight loading is the first reference step for further calculations. This loading shows minimum principal stresses at the upstream heel of the dam at about 6MPa. At the downstream face of the dam the maximum principle stresses are between 0 to 0.5MPa for most of the surface and for a small portion in the vicinity of the abutment this tensile stresses reach values up to 0.8MPa (fig. 4 and fig. 5).

For the full loading case (Dead Weight, Water, Temperature and Uplift) under the assumption of a closed base joint the minimum principle stresses at upstream face of the dam body are hoop stresses at about 4 to 5MPa. The maximum tensile stresses at the heel of the dam are higher than 2MPa.

At the downstream face of the dam the minimum principle stresses in the dam body are hoop stresses and are between 4 to 5MPa. At the abutment the compressive stresses are higher than 6MPa. The maximum principle stresses are small and are less than 0.5MPa (fig. 6, fig. 7 and fig. 8).

The maximum radial deflection under full loading condition is 48mm. The deformation at the highest concrete block at the interface level concrete to rock and in radial direction is 6mm (see fig. 9).

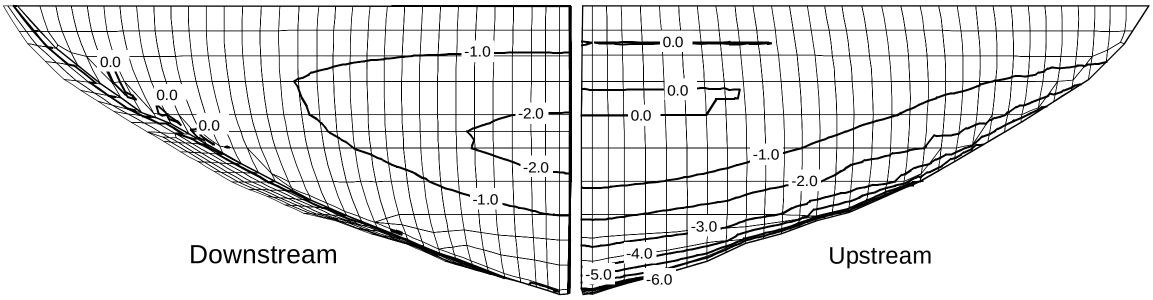

Figure 4: Dead Weight - Minimum Principle Stress

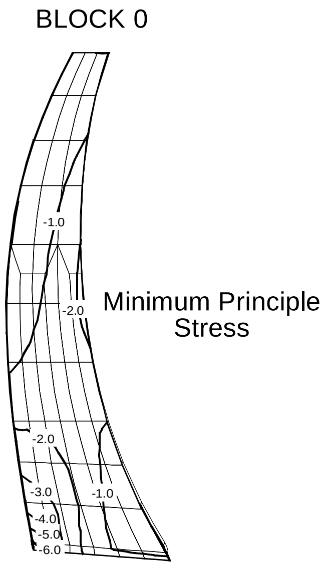

Figure 5: Dead Weight - Minimum Principle Stress - Cantilever

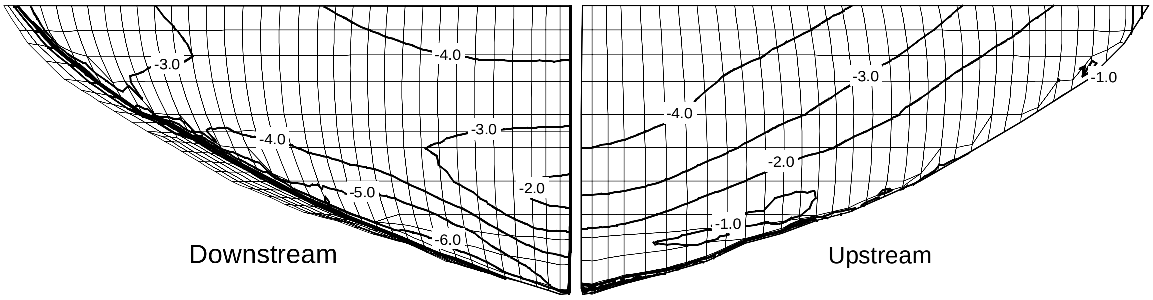

Figure 6: Full Loading - Closed - Min. Prin. Stress

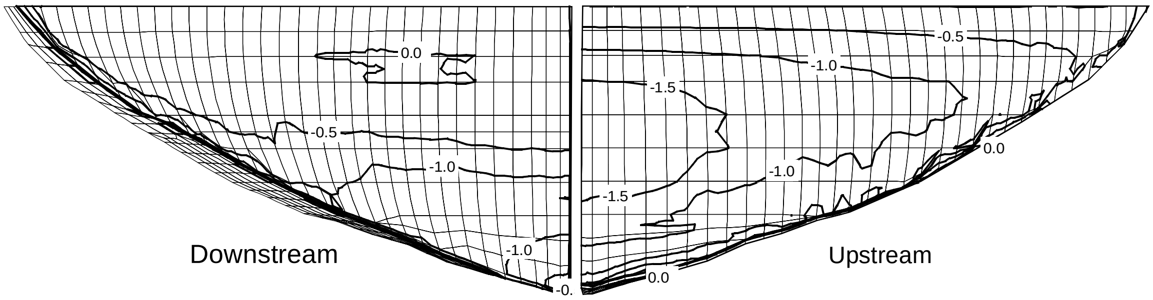

Figure 7: Full Loading - Closed - Middle Prin. Stress

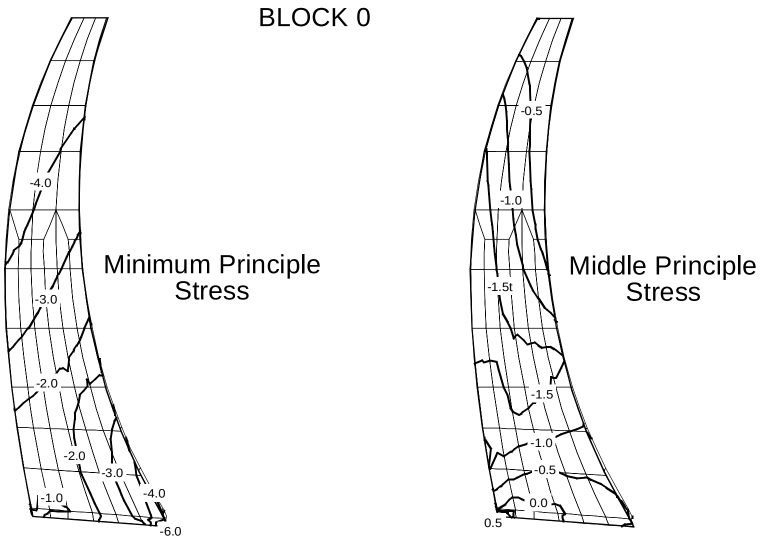

Figure 8: Full Loading - Closed - Cantilever

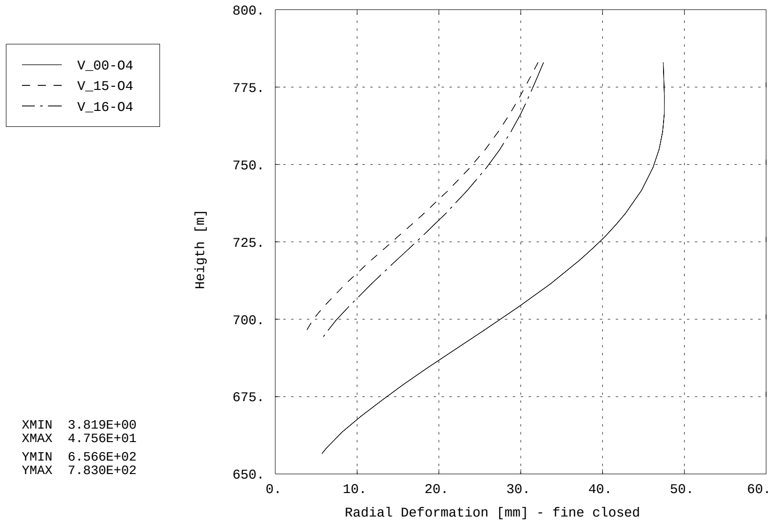

Figure 9: Full Loading - Closed - Radial Disp. Block 0, 15, 16

Nonlinear Model with base joint

The stress distribution for dead weight loading is equivalent to the results calculated with the help of the closed model. Under full loading conditions the base joint opens to a certain extent and the gained results are discussed.

In total two geometrically different discretized models are investigated. These are the so called fine model and the coarse model.

Results of the ”Fine Model”

Compared to the linear model, in general an increase in compressive stress occurs for hoop stresses at the upstream face of the dam. This can also be seen for compression stress at the downstream abutment (fig. 10, fig. 11 and fig. 12). Due to the presence of uplift pressure the upstream heel of the dam is still under compression, though the dam itself has separated from the rock.

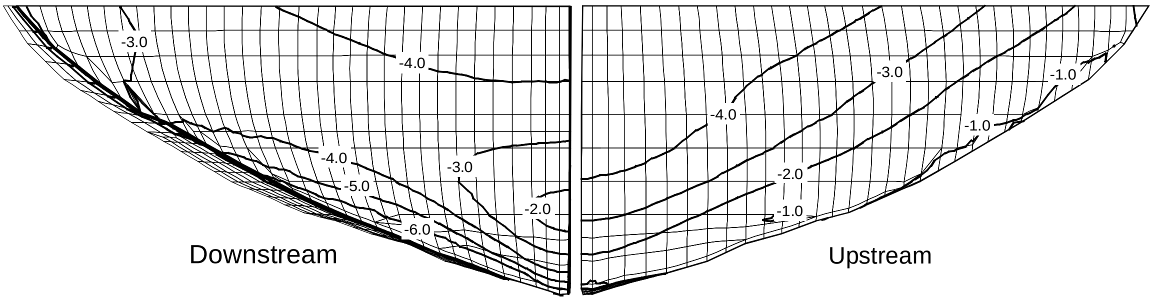

Figure 10: Full Loading - Open - Min. Principle Stress

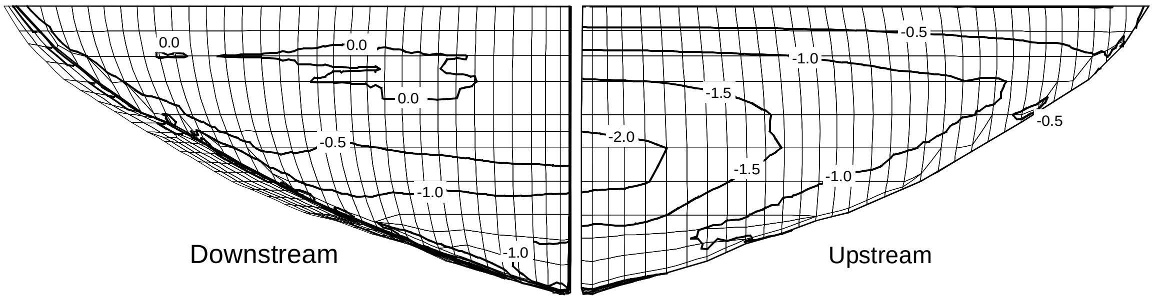

Figure 11: Full Loading - Open - Middle Principle Stress

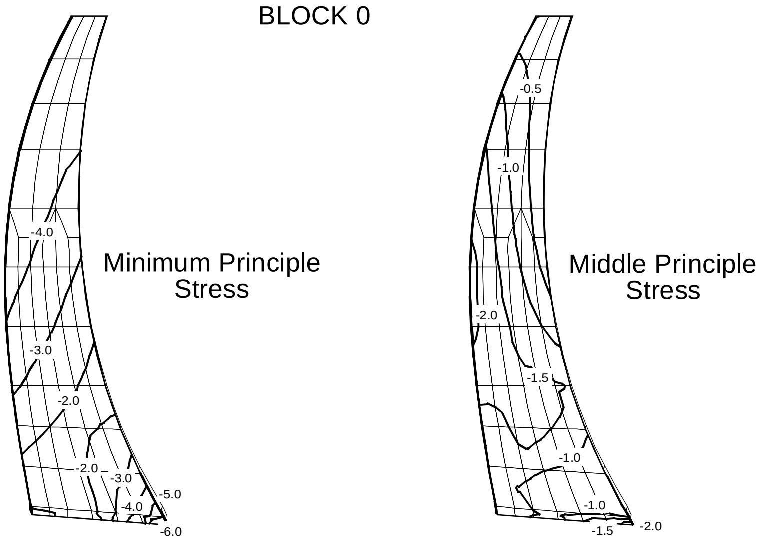

Figure 12: Full Loading - Open - Cantilever

The opening of the base joint along the entire dam abutment is shown in fig. 13. To a relative large extent the dam's base joint is open. (The displayed values are corrected against c_0, which is a numerical value of 1 mm). Within the base joint under compression, the resultant forces have to be transmitted.

The calculated radial displacements are at about 52mm for the block 0, and reduce to 34mm for block 16 and 39mm for block 15 (fig. 14).

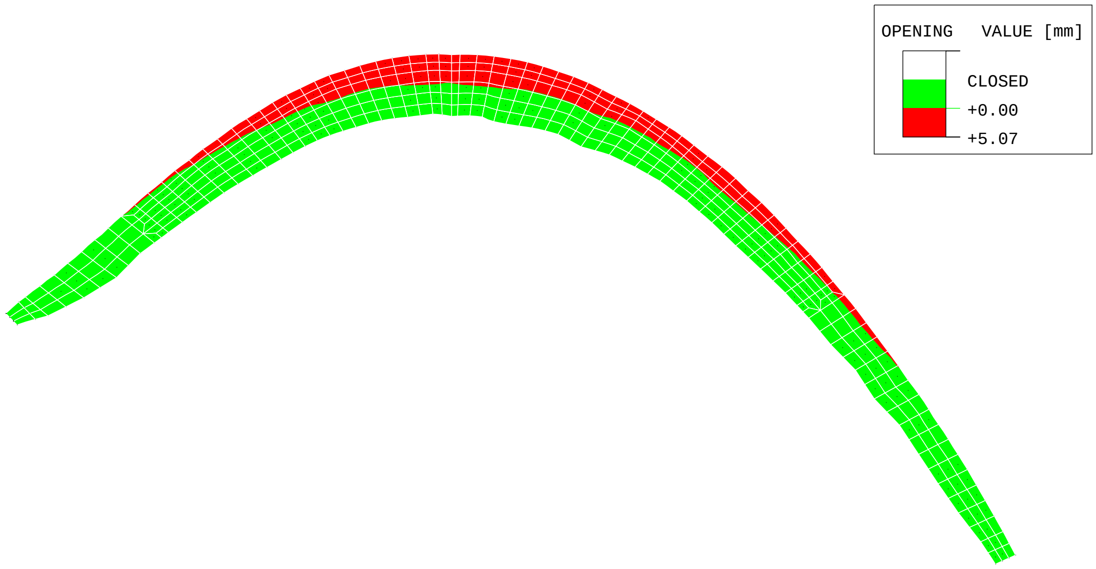

Figure 13: Full Loading - Opening Base Joint

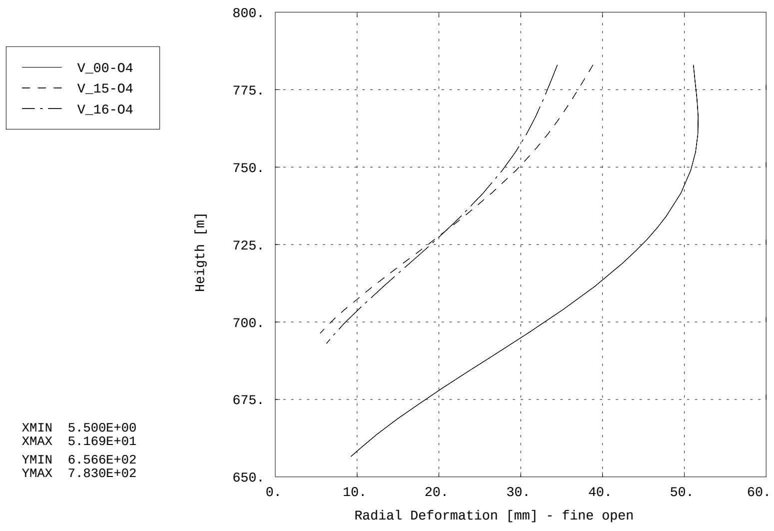

Figure 14: Full Load - Open - Radial Disp. for Block 0, 15, 16

The stress distribution along a radial line is shown on fig. 17. For the highest concrete blocks the diagram shows a base joint opening of about 42%. Nearly within the entire remaining cross section the contact failure condition is reached and sliding occurs.
Within blocks 15/16 the joint opens to an amount of 30% and 22% respectively. As it can be seen, sliding is only within the first 10m of the joint significant.

Results of the Coarse Model

The coarse model gives in general the same stress distribution along the entire dam as it can be calculated with the fine model. Due to the different discretization with the same element family, the stresses as well as the deformations gradients are less for the coarse model.
From the subsequent figures (15,16,18) it can be seen, that the stress distribution is in line with Fig. 10, 11 and 12 of the fine model.
The differences between the fine and the coarse model in terms of normal stress and base joint opening are shown on fig. 19.

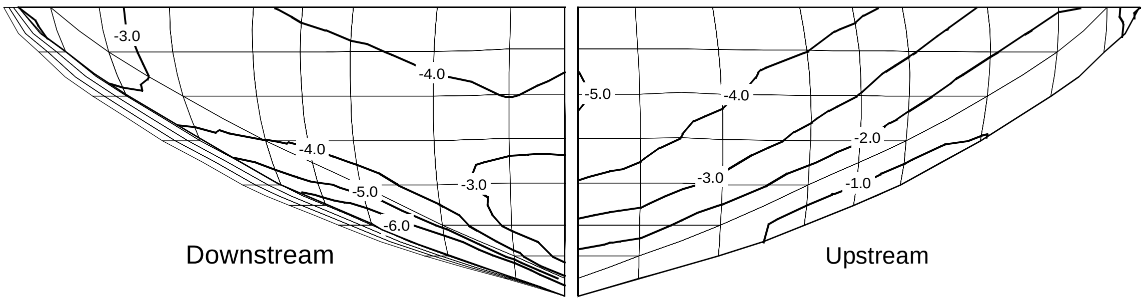

Figure 15: Full Loading - Minimum Principle Stress - Surface

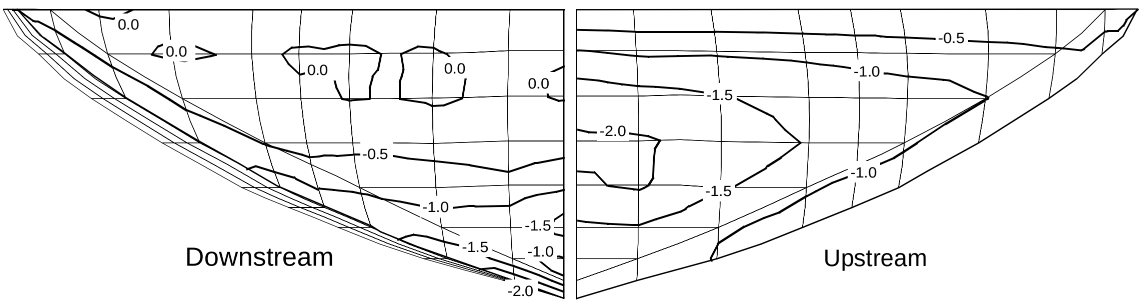

Figure 16: Full Loading - Middle Principle Stress - Surface

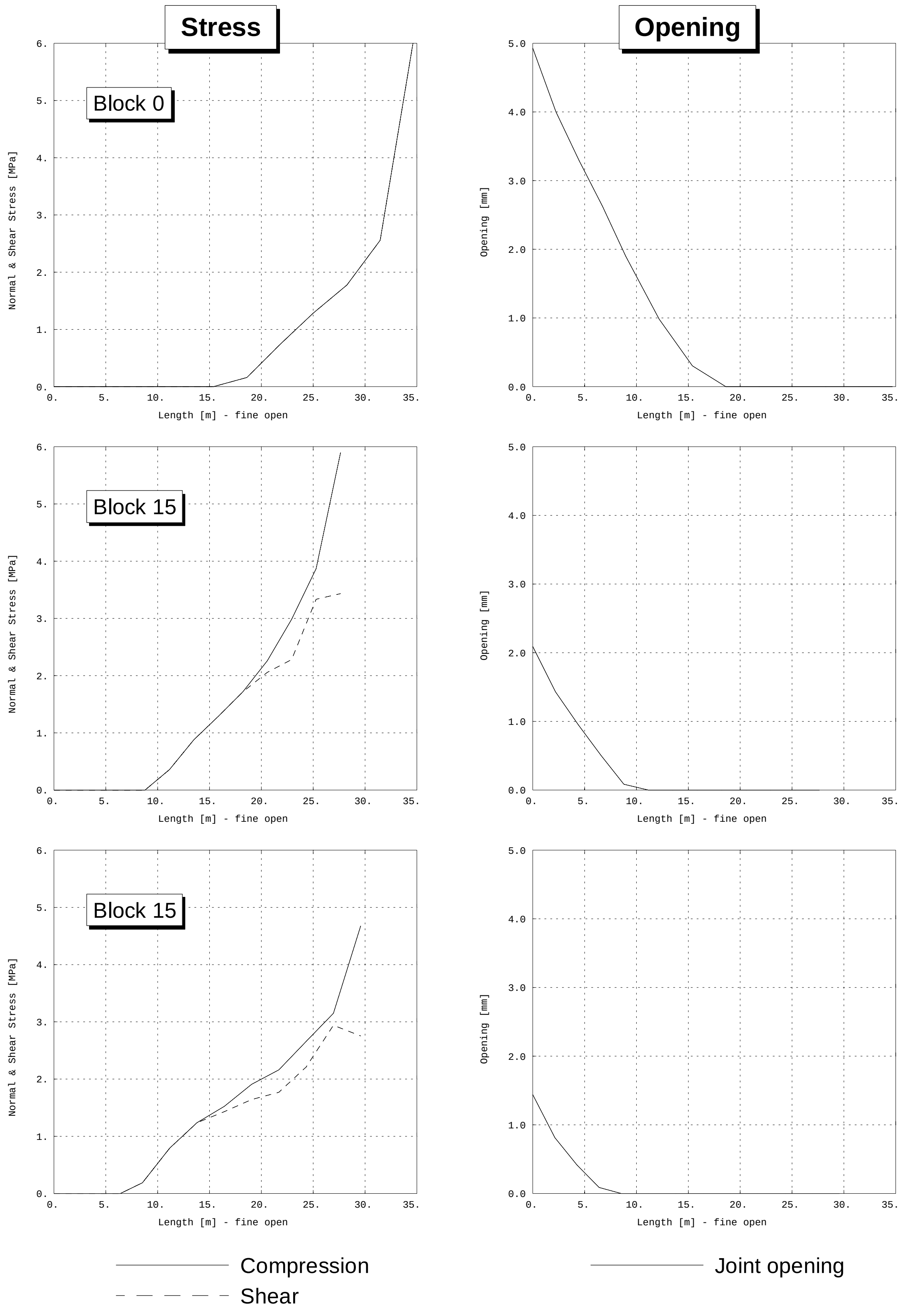

Figure 17: Full Loading - Compressive Stress - Joint Opening

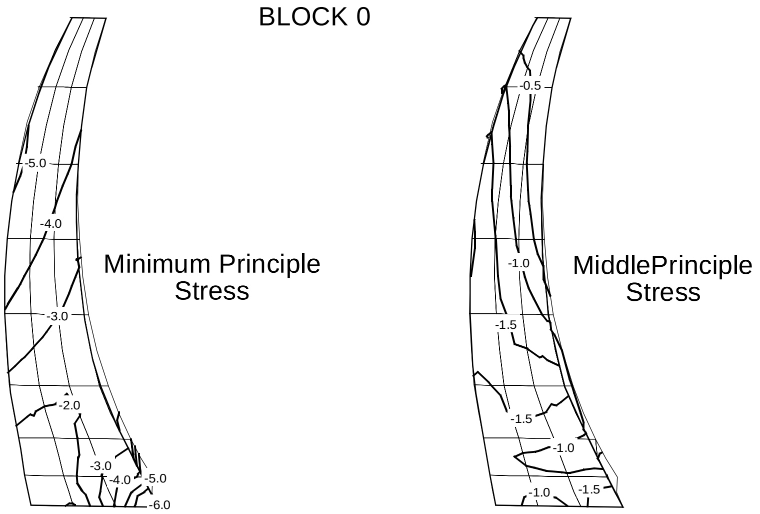

Figure 18: Full Loading - Cantilever

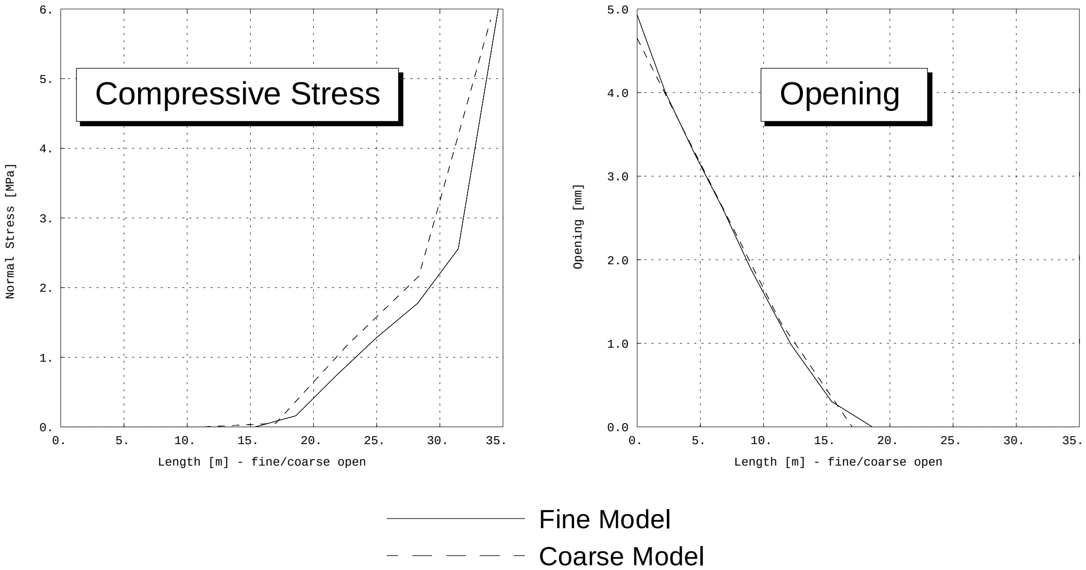

Figure 19: Comparison Fine - Coarse Model

8. SLIDING STABILITY

Displayed on the level of stresses in the interface the friction criteria is processed for block 0, 15 and 16 for full loading condition. The applied Coulomb failure criteria – residual friction angle of j = 45 only - is reached nearly all over the entire cross section for block 0 (see fig. 20).

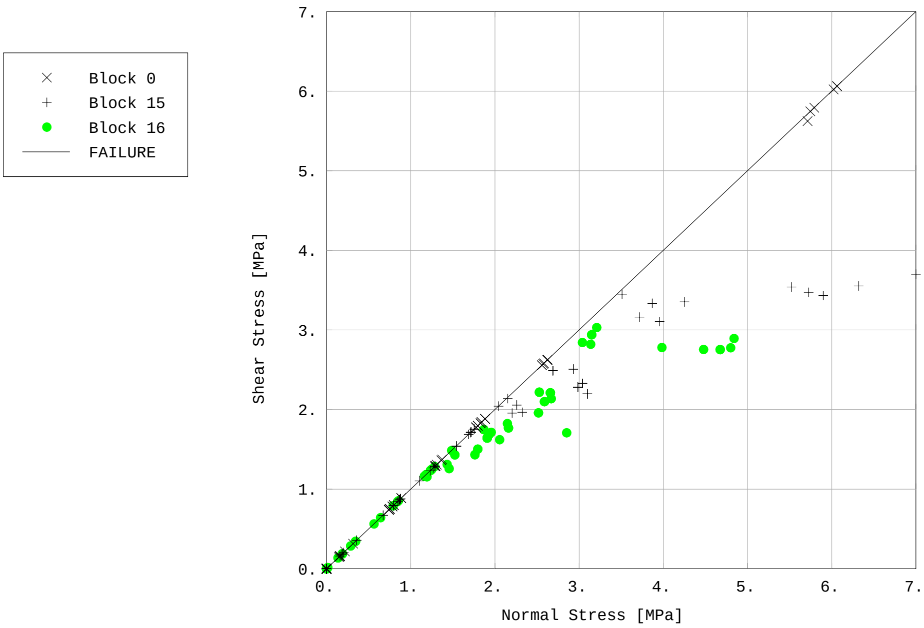

Figure 20: Coulomb Failure Criteria for Block 0, 15, 16

However, for the sliding safety assessment of the structure the shear strength, found by material tests, is t_{max}= 4.0 + 0,75 s_N.

The sliding deformations are of about 5.2 mm.

The distribution of the vertical reaction force along the projection of the crown axis at the abutment is shown on fig. 21. The reaction force per m in MN for dead weight, dead weight and water loading and under full loading conditions is displayed. It can be seen, that the water loading and water loading together with the applied uplift pressure results in a significant vertical unloading of the dam.

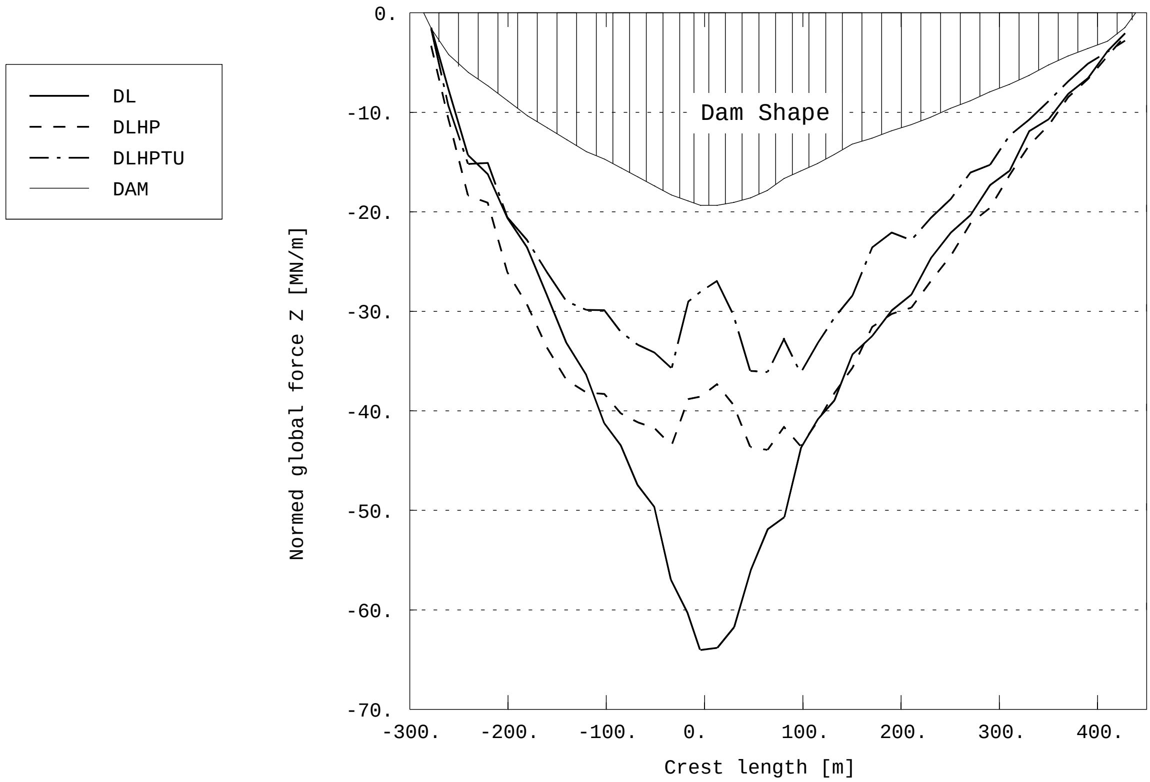

Figure 21: Normed vertical global force along the Crown Axis

9. CONCLUSIONS

Under a given model assumption the numerical analysis for Schlegeis Arch Dam is carried out. Respectively dead weight, water, temperature and uplift pressure loading are investigated.
For these loading cases the first analysis run was a standard finite element analysis with a linear model. The evaluated results show tensile stresses at the upstream heel of the dam.

Additionally to the linear analysis a model is employed, which gives the dam the possibility to separate from the foundation rock. For the entire dam in general the results calculated show compressive stresses slightly higher than for the linear model. Under full loading condition the upstream heel of the dam separates for an amount of 5mm from it’s rock foundation. Due to the uplift pressure acting in

the base joint no tensile stress is apparently in the dam abutment.
Due to the nonlinear analysis a more favorable stress state in the dam structure can be evaluated. The hoop stresses in the dam increase slightly and the tensile stresses at the abutment decrease. Additionally to this investigation the sliding stability considerations at the dam abutment needs a reevaluation.

Based on the nonlinear analysis it is shown that the compressive stress level increases slightly in the dam body. Compared to the linear model a safe redistribution of stress occurs, due to the base joint opening.

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