

Arch dam excavation in a steep gorge

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Introduction

An Austrian-Turkey Consortium is constructing the hydro power plant Ermenek in Turkey. The plant consists of an arch dam, a large grout curtain, a pressure tunnel, a power plant and appurtenant structures.

Starting with a description of the general layout of the scheme the geological situation is explained. The considerations and investigations for the design and construction of the dams' abutment are detailed. The design steps and analyses for the arch dam structure are described. The dam transmits high abutment forces to the foundation rock for which the results of the stability calculations are concluded.

To support the rock surface during excavation by blasting prestressed anchors are designed. As an alternative a local support block at the right dam abutment near the crest level is investigated. The appropriate technical and economically favorable solution with prestressed anchors is under construction and provides the basis for the actual situation site report.

1. General layout of scheme

The scheme is located on the Ermenek river upstream of the existing Gezende – hydro power plant.

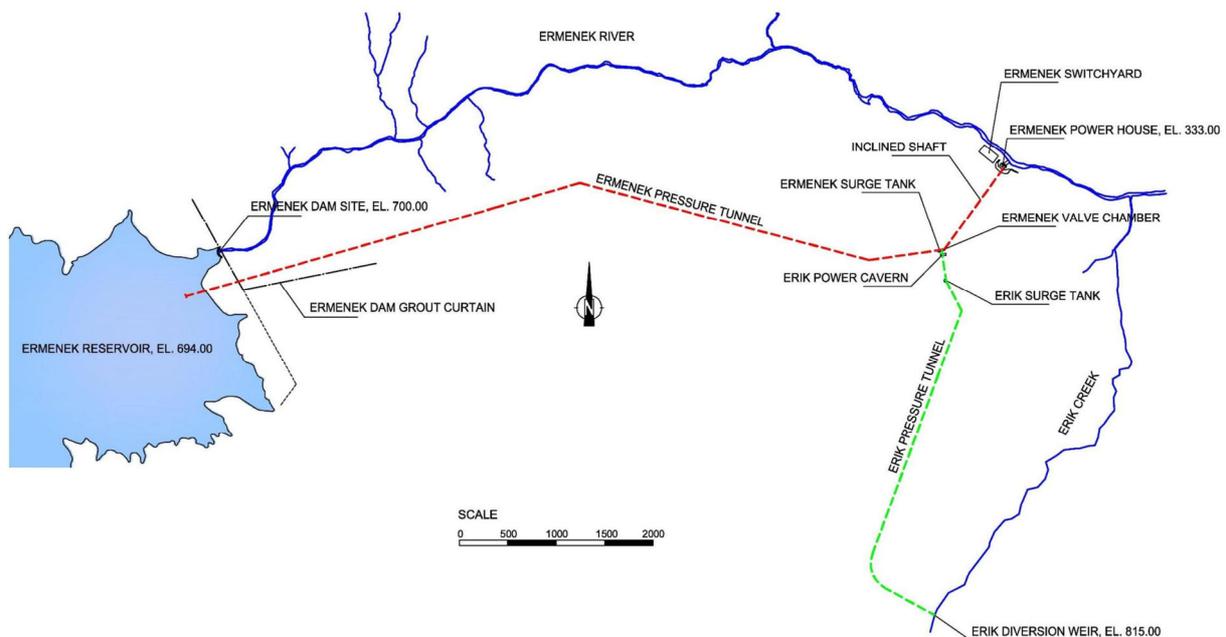


Fig. 1. General scheme layout

The concrete arch dam will be built up a reservoir of 4600 Mio m³ at maximum operation level of 694 masl. The entire length of the reservoir will be 27,3 km. The mean discharge of the river at the dam site is 42,3 m³/s. The mean annual flow is calculated with 1300 Mio m³/year.

The pressure tunnel with an inner diameter of 5,6m and a entire length of about 8km allows for the maximum operation discharge of 106m³/s from the reservoir to the power house.

The electricity generation will be provided with 2 Francis units with a total capacity of 300MW. With the net rated energy head of 327m the annual energy production is expected to be about 1000GWh.

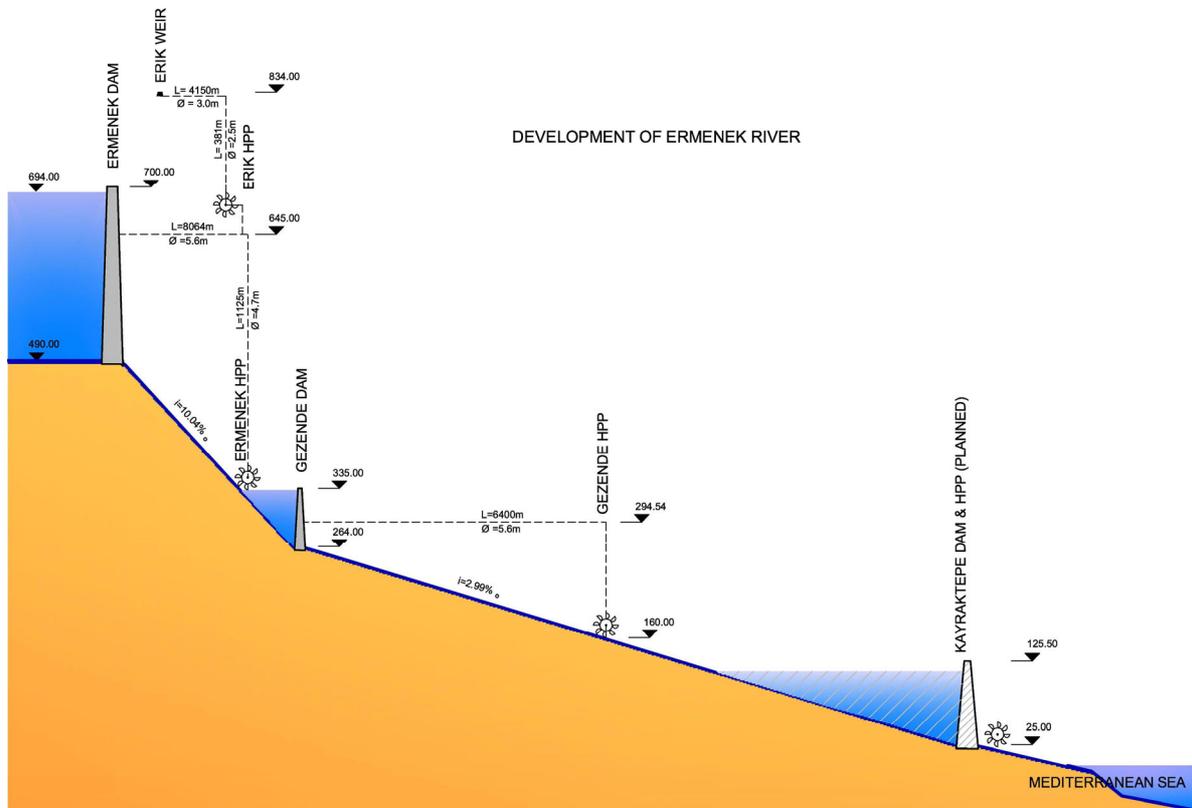


Fig. 2. Longitudinal section along river

1.1 Geology at dam site

During different project stages - feasibility, final and tender design - the knowledge about foundation conditions increase. However, during execution of work design measures need to be adopted or changed if necessary. This process is described thereafter.

The Ermenek valley is a deep cut through several geological units. The deeper unit of pretertiary age consists of sedimentary and magmatic rocks which were overthrust and folded during Upper Eocene. After a subsequent erosion period marls and limestone were deposited on top of the basement unit and were not affected by further folding but by uplifting.

The dam abutment is situated in the Nadire limestone. The limestone is either massive or very thick bedded, usually light beige in colour. The matrix is fine grained, hard to very hard and homogenous. The rock mass is affected by karstification. Large cavities can be seen in the gorge faces and reach dimensions up to several meters and frequently follow main joints. In the galleries the degree of karstification is rather low. It seems, that karstification mainly affected the area of the present gorge.

The following rock mass types of Nadire limestone (NA) have been defined at dam site. The massive type NA 1 is the typical prevailing type of limestone, with usually closed joints. The rock mass is characterised by abundant non persistent discontinuities and by moderately to widely spaced persistent joints. The bedding planes dip with 40° - 65° in NW direction. The type NA 2 is encountered close to the surface of terrain and is characterised by open joints.

The rock mass type NA 3 is called "fractured" and is characterised by a closer joint spacing and abundant filled joints. Filled karst cavities have been observed frequently, the filling material is dry and hard, partly recemented.

The rock mass types have blocky structure. The blocky rock structure is characterised by the dominant role of discontinuities, defined by orientation and strength characteristics.

Six main discontinuity sets have been measured. The data refer to the geological conditions of the dam site area and therefore the orientations are valid for both sides of the river. Three discontinuity sets (set 2, 3, 5) are dominant. Mean dip directions and dip angles for main discontinuity sets are shown in Table 1.

The joints can be generally defined as tightly closed thus ensuring a rock wall contact. Sometime the joints are widened by karstification and the openings are filled up by clay material. Generally these joint openings have local appearance. On the joints no strength tests have been performed.

Discontinuity set	Dip direction [°]	Dip angle [°]
System 2	319	90
System 3	55	70
System 5	346	60

Tab. 1. Main Discontinuity sets at dam site

2. Arch dam layout

The arch dam is ideally located in a narrow gorge, with a natural width at the base of 20m and at the crest of 110m only, at a dam height of 210m. The geometry of the dam is set up by means of ellipses at horizontal sections.

The thickness of the structure at the center line is at the dams' base 25m and at the crest 7m. The entire concrete volume is 272000m³ with a rock excavation for the dam abutment of 363000m³.

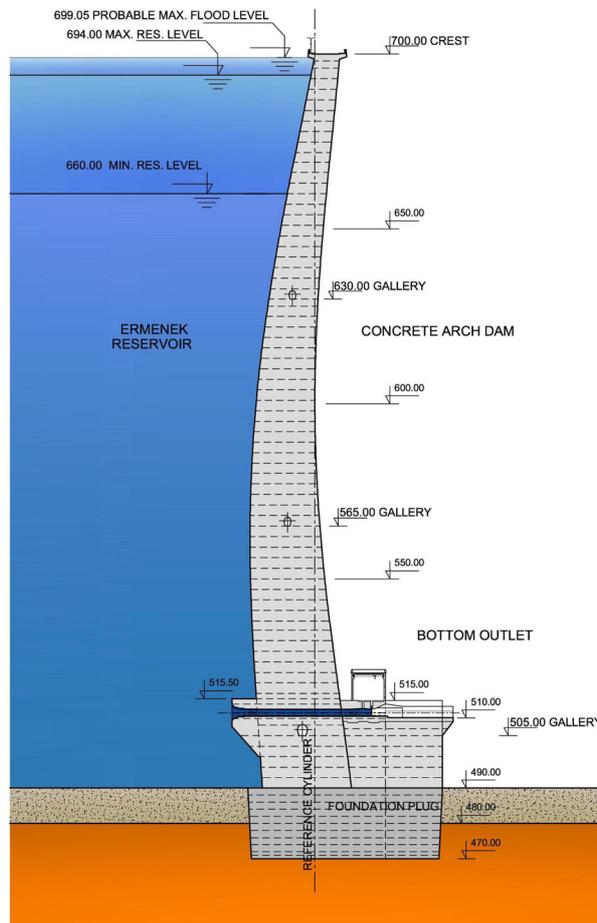


Fig. 3. Dam vertical cross section

The dam is equipped with a bottom outlet, consisting of two cylindrical steel pipes with a diameter of 2m. Each has an emergency and one operating valve. The amount of discharge under full hydraulic head totals 280m³/s.

For flood release two concrete lined spillway tunnels with a diameter of 6m are designed. At full service level the maximum discharge amounts to 986m³/s. The energy dissipation will be achieved by collision of the water jets in valley axis.

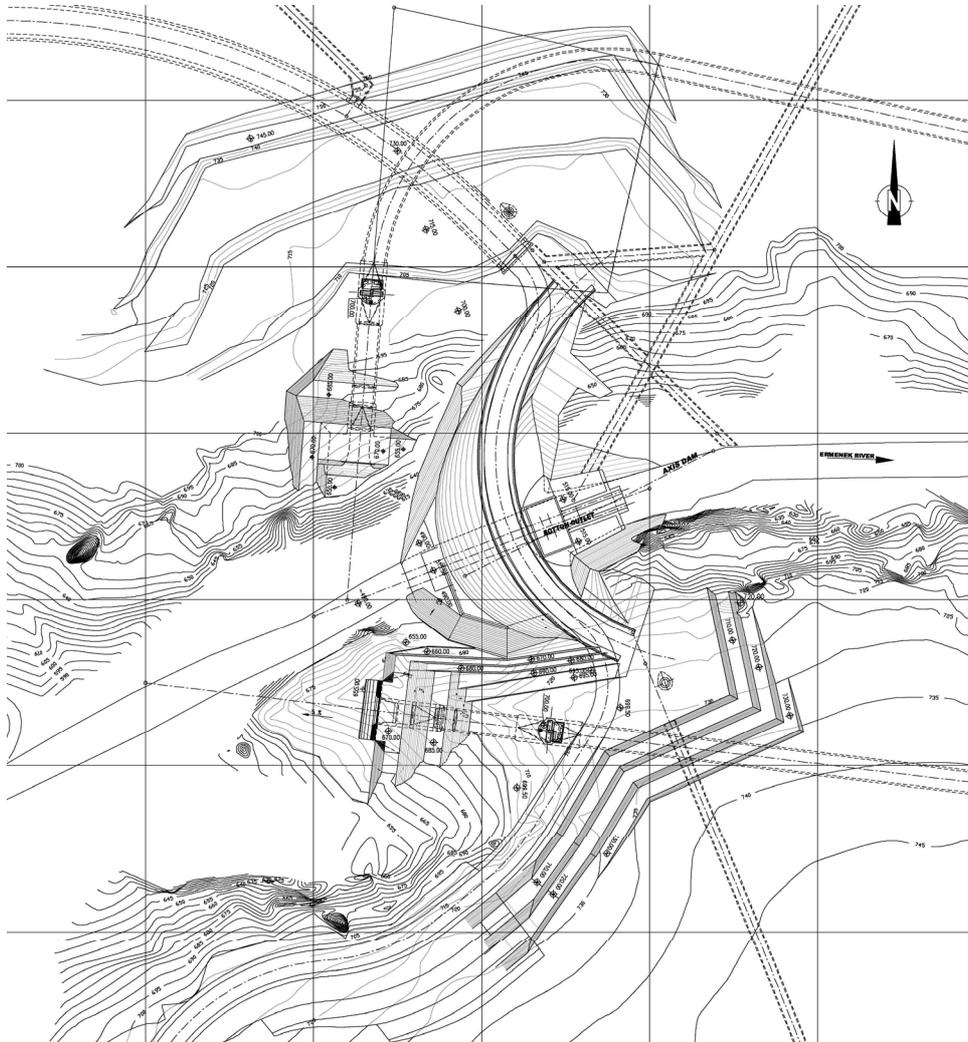


Fig. 4. Layout of dam and spillway

The dam will be constructed in concrete blocks, each at about 20m in thickness measured at the centerline. The concrete will be poured in layers of 3m height. After a certain height reached, the joints between the concrete blocks will be grouted. After joint closing by grouting the structures will act as an arch rather than individual cantilevers.

The dam is equipped with three horizontal galleries and one gallery at the vicinity of the dam abutment to connect with grout- and drainage curtain at the abutment. The horizontal galleries are at the same level of the grouting galleries. In total the grout curtain will be 2150m in horizontal length, 1450m at the dams' right and 550m in the left abutment. The maximum depth of the grout curtain is about 240m below the lowest gallery and over the entire abutment has 680.000m².

3. Arch dam analysis

Based on the dams design concept the analyses are carried out. The main investigated loading steps are:

- Dead weight, block joint grouting
- Water loading with different hydraulic heads
- Summer and winter temperature distribution
- Abutment uplift water pressure
- Earthquake.

The size of the extreme hoop stresses are in the range of -4MPa compression at dams' upstream surface and at +1MPa tension stress at the downstream surface during winter loading condition. In the loading case summer the stresses are not that large as calculated for the winter-load case.

The minimum vertical stresses are in the range of -3 to -6MPa depending on the actual loading condition. Maximum vertical stresses are in the range of +1MPa tension during load case winter at the upper part of downstream face.

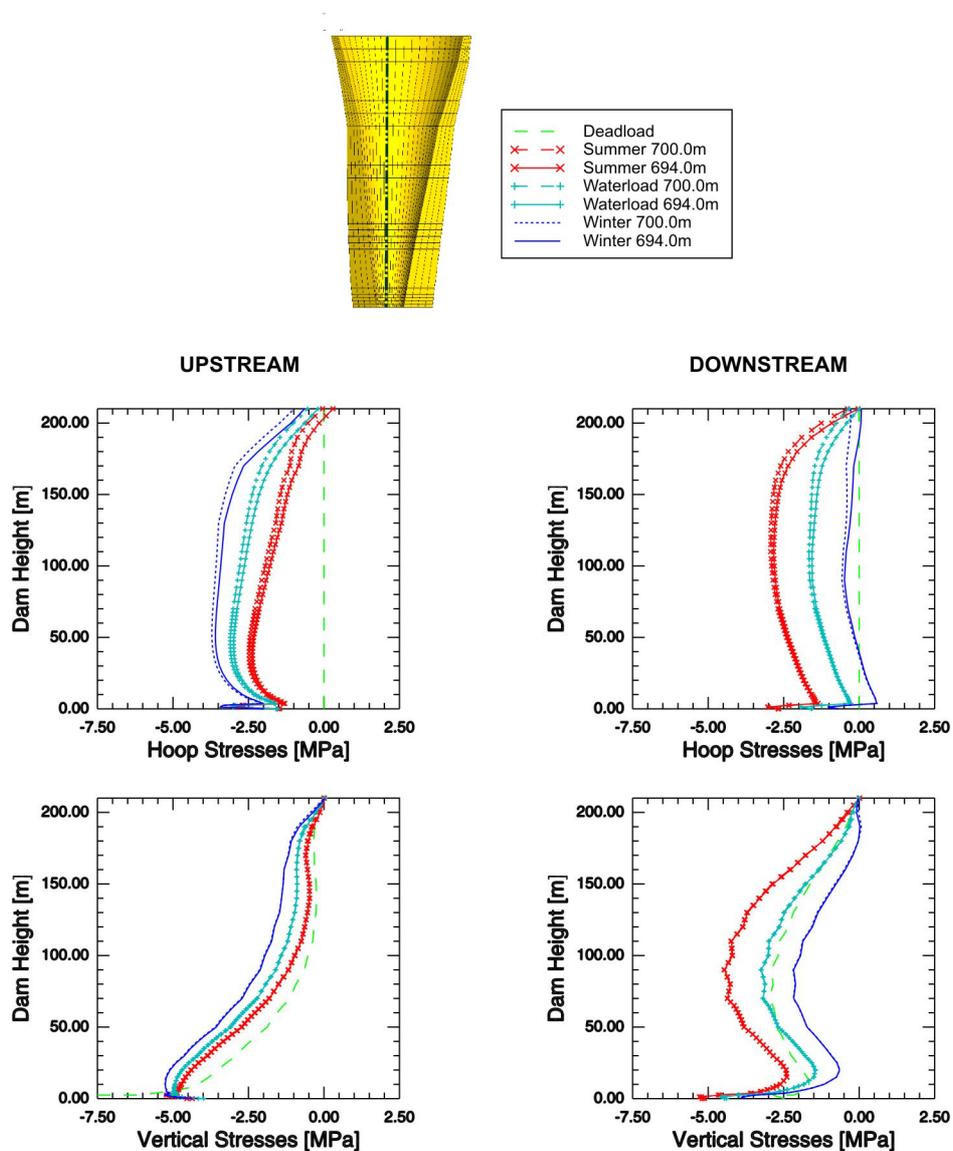


Fig. 5. Dam stress distribution – Normal loading condition (US / DS)

The minimum sliding safeties are calculated in the middle blocks during load case summer down to 1.8, and during load case winter down to 1.4. Both values are given for unusual loading cases. For usual loading cases these safety values are even higher (1.8 during summer versus 1.5 during winter).

4. Arch dam foundation

For maximum loading cases, the dams' abutment forces are calculated at different elevations. The figure 6 provides the abutment force distribution. These abutment forces together with the joint orientation are used to investigate the stability of the abutment.

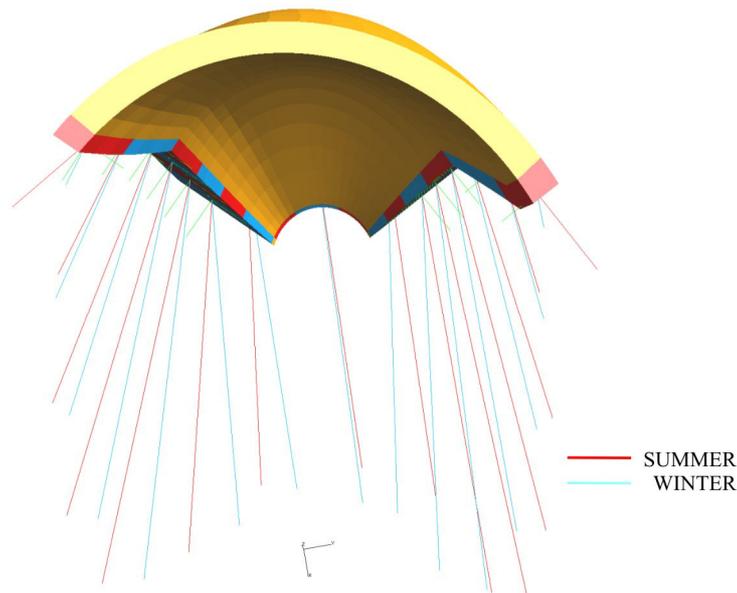


Fig. 6. Abutment force distribution

5. Rock mechanical parameters

5.1 In Situ rock tests – plate loading tests

Plate loading tests were carried out in grouting galleries at the right and on the left abutment. According to site investigations, the intact and competent rock portion will be dominant at the dam abutment. If karstified zones appear in the dam abutment, these zones have locally to be removed and replaced by concrete. Therefore the dominant rock portion can be characterized with the measured elastic loading and unloading deformation value.

From measurements taken in the left gallery GL2 the results give a mean value from 12,5 and 20,7 of $E_d=16,6\text{GPa}$. For gallery GL4 a value of 14,1GPa could be achieved. On the right abutment in gallery GR3 the mean of the values of 10,8 and 16,4 results in a rock mass deformation value of $E_d=13,6\text{GPa}$. In gallery GR4 the mean value gives 17,4GPa.

From plate loading tests carried out a mean deformation value of rock mass of $E_d=15,3\text{GPa}$ could be gained. As this value is determined on the rock surface of galleries, which were excavated by blasting, this value is a lower bound value of rock mass deformation behavior.

From correlations according classification by Bieniawski and Hoek the deformation modulus of rock mass can be awaited to be in between 20GPa to 27GPa. As a lower bound value 12GPa could be correlated for highly fractured rock mass portions.

For the elastic stress analysis a deformation modulus of 17GPa will be used. A distribution of different rock mass deformation modulus will depend on geological mapping and measurements. This value will be confirmed

by comparisons with deformation measurements of prestressed anchors and measurements taken from extensometer readings.

5.2 To investigate the joint roughness

The joint behavior of rock joints is dependent on the waviness of the surface and the initial bonding. In case of contact of these joints, material bridges need to be sheared through to allow for a relative movement of rock blocks. To account for this behavior within an analysis the published theory from Hoek [1995] is used to calculate the ratio of shear strength and normal stress in rock joints.

$$\tau = \sigma_n \tan \left(\varphi_b + JRC \cdot \log_{10} \frac{JCS}{\sigma_n} \right)$$

JRC joint roughness coefficient
 JCS joint wall compressive strength
 φ_b basic friction angle of the surface (equal to $\varphi_{\text{residual}}$)
 σ_n is the normal stress acting on the surface
 τ is the shear stress acting on the surface

As an example, the in situ measured values for joint R-J5g and R-J5e respectively are shown, as these values are the most critical ones for the abutment stability.

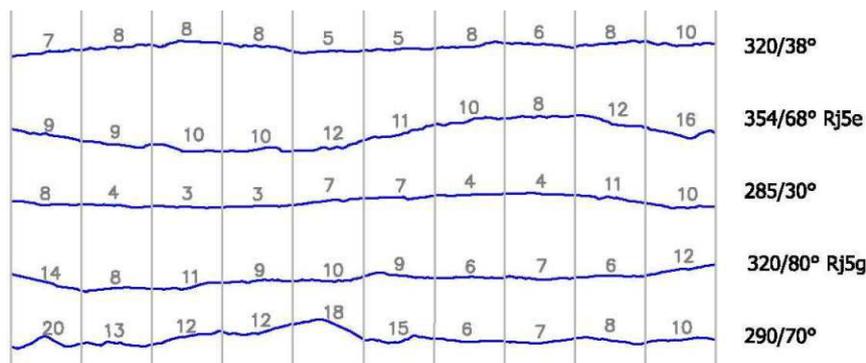


Fig. 7. Assigned JRC value

6. Rock excavation – Abutment stability

For the excavation of the dam abutment and the stability of the abutment during operation of the several analyses are carried out:

- Stability investigation during excavation
- Overall abutment stability

With the distribution of abutment forces calculated at the dam rock interface the stability analysis is carried out. Based on the geological – geotechnical model of the rock abutment significant rock blocks are identified.

Based on the rock joint distribution and the shape of the surface to be excavated a volume model is developed. This 3D model forms the basis for stability analysis.

In general for this analysis the rock mass is assumed to behave rigid; the gravity forces and forces due to additional loadings are considered and according to the sliding mode achieved, the forces are considered along sliding planes for stability analysis.

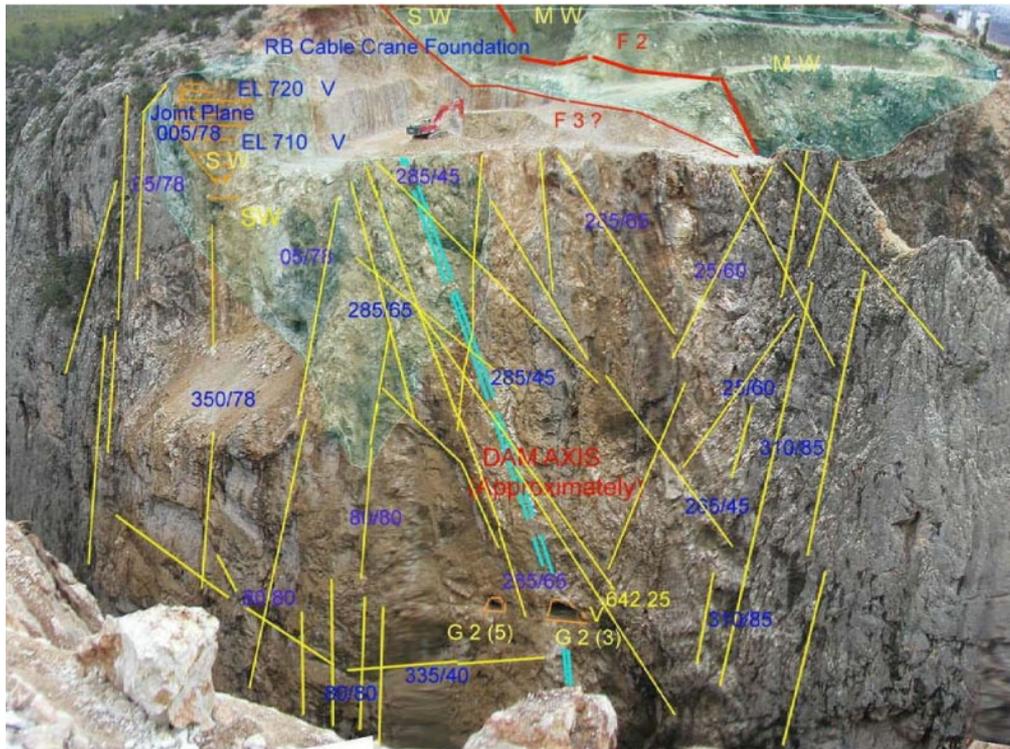


Fig. 8. Rock joint distribution at right abutment



Fig. 9. Status of dam excavation – May 2006

From this analysis it can be concluded, that all possible wedges are stable based on the derived material properties. Forces transmitted via arch dam to the abutment provide no reduction of the sliding factor of safety. For the process of excavation it is necessary to provide additional support to the rock by means of prestressed anchors. These will help to prevent preexisting joints – at the moment entirely closed – to open during blasting. Therefore the initial bonding of the surfaces at joints can be maintained.

6.1 Investigation of local concrete thrust block support

As it is mentioned, some part of the abutment appears locally to be karstified. Especially in the vicinity of the upper right abutment, apart from sound foundation rock, a karst cavern exists. The contractor brought up to investigate this locally weak zone and wanted to prevent the usage of prestressed anchors. The idea was to

remove additional part of the abutment rock, shift the location of the rock foundation, provide a new dam structure and prevent the usage of prestressed anchors.

Therefore investigations were carried out to compare.

Stress distribution changes in the dam due to different abutment flexibility abutment stability with a concrete thrust block.

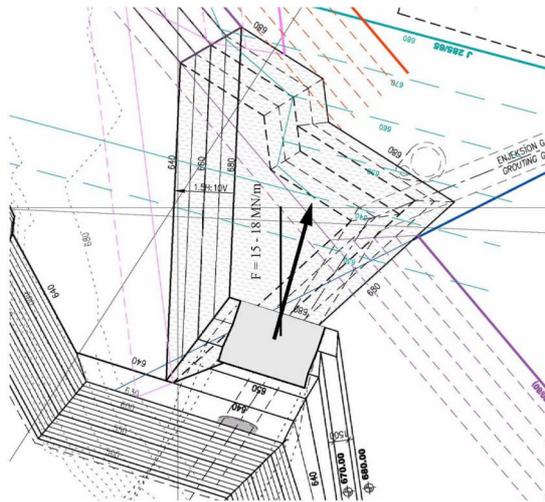


Fig. 10. Forces transmitted from arch dam abutment via concrete thrust block

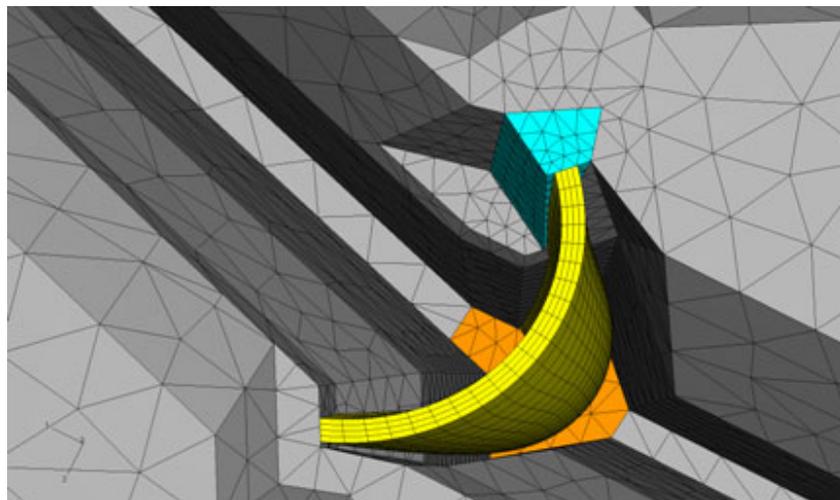


Fig. 11. Pulvino in the model

The calculations, as anticipated show that the influence of the flexibility at the abutment does not provide a significant change in the arch dam stress distribution. With the help of a concrete thrust block the support of the dam could be achieved. Instead, the change of the dam shape would lead to an unsymmetrical bearing and a significant change of the structural stiffness at the elevation 640masl, where the concrete thrust block is designed to start.

The explained investigations show, that the initially designed excavation with pre-stressed anchor support is more favorable than the anticipated solution with a concrete thrust block. Together with the additional support by anchors and implemented force monitoring cells an in situ measurement program is designed to monitor the excavation progress. The equipment consists of inclinometers, extensometers and geodetic reference points. The interpretation of the monitored data will help for further decisions and if required additional measures.

7. Status of work

The present status of the excavation work in July 2006 is shown on Fig.12 for the left bank (about 40m are excavated) and in Fig.13 for the right bank, where about 20m are excavated, out of totally 210m. The support by means of pre-stressed anchors is following the excavation progress step by step. The blasting work is guided by an Austrian blasting expert.

At the present stage the excavation procedure is satisfactorily. However, the speed has to be accelerated after some starting problems within the next time period. The dam body excavation is scheduled to be finalized in March 2007.



Fig. 12. Left bank dambody excavation – July 2006



Fig. 13. Right bank dambody excavation – July 2006

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