# Open air excavations at Deriner dam

*Olivier Muller provides details on the excavation works carried out as part of the Deriner arch dam project on Turkey's Coruh River*

Figure 1 – Views before excavation works (May 1999) and after completion of excavation (November 2005)

DERINER DAM is a double-curvature arch dam located on the Çoruh River in Northeastern Turkey. With a height of 249m and a volume of concrete of 3.5Mm$^3$, it will be the highest arch dam in Turkey once complete in 2011. The installed capacity of the powerhouse will be 670MW (with four Francis turbines) with annual generation of 2118GWh. State Hydraulics Works (DSI) of Turkey owns the project, with construction work being carried out by a consortium led by Turkish contractor ERG.

In 1997, Stucky Ltd, a Swiss consultant, was appointed to bring technical assistance to the contractor. Firstly the company conducted, under the direction of DSI, a full review of the final design project from 1991, and then performed a complete detailed design of the dam and its appurtenant structures. A number of measures were introduced to improve safety and reduce construction costs and time.

Excavation began in October 2000 on the Left Bank. In August 2005, all open air excavations for the dam foundation were completed. The total volume of excavated material amounts to 8.7Mm$^3$. Figure 1 shows the project site before and after excavation.

On 11 December 2005, the mass concrete works for the dam began. These works are currently ongoing with the project expected to be complete by the end of 2011. This article provides details of the open air excavation works for the dam body and appurtenant structures. The design aspects of the project, together with a general layout plan, were detailed in an article published in the July 08 issue of IWP&DC [1], and will therefore not be covered in this paper.

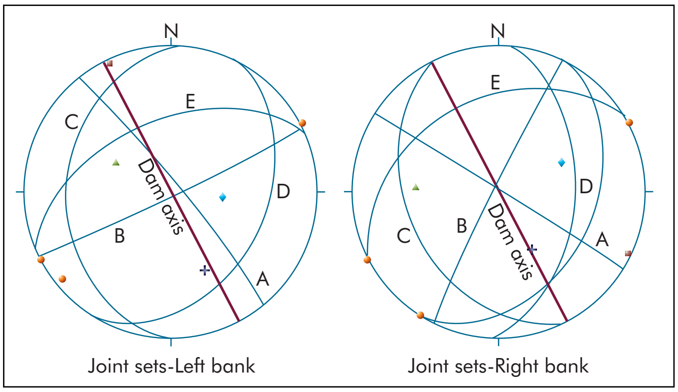



**Figure 2 – Stereographic projection of the five joint sets for the left and right banks, with dam axis**

## Geological and geotechnical parameters

The geology at the Deriner site is mainly composed of granodiorite intruded by diabase dykes, which are generally very suitable for the foundation of an arch dam. Nevertheless, the upper layer of the rock is decompressed and heavily jointed, which means that it has to be excavated so as to ensure sound rock for the dam foundations.

Since both main rock types, Diabase and Quartzdiorite, are quite homogeneously distributed over the site and show more or less similar properties of intact rock, there is no need to specifically take into account the lithology in defining the design values of intact rock parameters.

Unconfined Compressive Strength results on dry rock samples show mean values of $\sigma_{ci}$ = 120 MPa for the Left Bank and $\sigma_{ci}$ = 80 MPa for the Right Bank. The modulus of elasticity of the intact rock is worth $E_{Si}$ = 65 GPa.

Despite the relatively favourable strength and deformation parameters of the intact rock, the behaviour of the rock at the Deriner site is mostly controlled by the properties of the discontinuities since their spacing can be considered as close with respect to the scale of the structural elements to be built.

Five main joint sets have been identified at the site: Joint sets A and B are dominant and represent conjugate sets of fractures; thus, they intersect at right angles. Their spacing ranges from 1 to 5m. Joints C and D are subordinate, but can still be observed at most locations and their spacing ranges from 10 to 15m. Fractures from joint set E are very rare and the spacing is over 20m.

Figure 2 shows the stereographic projection of these five joint sets for each bank. The dam axis corresponds to the valley axis. The strength of the rock mass can only be considered as isotropic when it is heavily jointed, i.e. when it exhibits several joint sets and low joint spacing, or when it is intact, i.e. when no joint crosses the mass. With five joint sets and joint spacing orders of magnitude smaller than the structures it will support, the rock mass at the Deriner dam site can be considered, at a large scale, as homogeneous and isotropic. Figure 3 illustrates the numerous joints existing at Deriner site.

Direct shear tests on joints were performed and three properties have been measured: the peak shear strength; the residual shear strength; and the reverse residual shear strength. Based on the results, the average peak friction angle on joints is 44°, the residual friction angle on joints 37° and the peak cohesion about 0.5 MPa. The Rock Mass Rating (RMR) ranges from 45 to 60, which classifies the rock mass as 'fair rock'.

From empirical studies as well as in-situ laboratory tests, rock mass parameters have been widely discussed throughout the project. Finally for the rock mass, the overall modulus of deformation recommended at the time of the final design is $E_{sm}$ = 9 GPa and the Mohr-Coulomb failure parameters are $\phi_{pm}$ = 35° to 40° and $c_m$ = 0.15 to 0.20 MPa.

## Design of the excavated slopes

The excavation work has been carried out respecting the following geometrical principles:

1. Berm heights: 15m
2. Slopes of berms: 1/5 (Horizontal / Vertical)
3. Berm widths: 5m

Excavations, which start from el. 578 at the Right Bank and from el. 620 at the Left Bank, extend down to el. 144. It means that the natural slopes are excavated on over 430m height for the Right Bank and nearly 480m for the uppermost point at the Left Bank. Twenty nine berms at the Right Bank and 33 berms at the Left Bank have been excavated.

As a general rule, two types of stability analyses for the surface excavations have been carried out throughout the project site:

*a) Global stability*

Global stability of the excavated slopes is verified, and proper support designed, over multiple berms height assuming two-dimensional and prismatic behaviour. The general method used to study the global stability is based on the limit equilibrium of unit thickness rock mass slices.

Failure through the rock mass along the most unfavourable hypothetical plane is considered for each cross-section. For this analysis, cohesion of the rock mass is always taken into account.

In some situations, the relative orientation of one or several joint sets with respect to the natural/excavated slope is unfavourable. In these cases, failure along joints is also analysed, taking into account the joint shear strength parameters.

Global toppling failure mode is also considered. The results show that, according to the very low cohesion required to ensure stability, this mode of failure is ruled out for most of the excavated slopes.

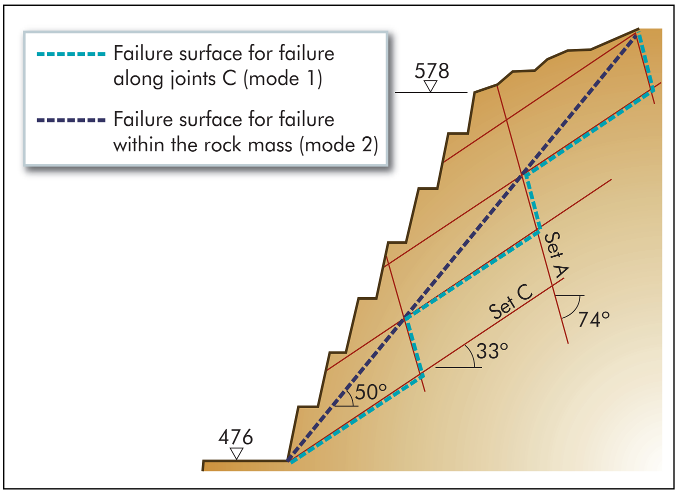
Geometrical as well as shear strength parameters have been adapted throughout the project according to observations made on site during the first excavations. An assessment of global stability has been carried out on numerous cross-sections of the Right and Left Banks, taking into account different rock mass parameters, shear strengths parameters on joint and exact geometry. This global stability analysis generally results in the installation of active pre-stressed anchors.

The global stability analysis is a large-scale analysis (several tens of meters high), which implies that the sliding plane is deep inside the rock mass (more than 10m), making it impossible to use passive anchors to increase the sliding resistance. Only pre-stressed anchors are considered if required by the global stability analysis.

Figure 4 represents an example of a global stability analysis considering failure within the rock mass and failure along joints C.

*b) Local stability*

Local stability is verified and proper support designed over a single berm height assuming a three-dimensional (wedge) behaviour. The




**Below: Figure 3 – Picture of the rock mass at the right bank with the overflow spillway inlet in the centre at elevation 360m asl**

**Right: Figure 4 – Right bank – failures assumptions for global stability analysis during excavation;**

general method used to study the local stability is based on the limit equilibrium of three-dimensional rock wedges bound by two joints. Local stability generally induces a light supporting system on almost every berm, with passive anchors (rockbolts), weld mesh and shotcrete.

The local stability analysis is a small-scale analysis (usually 10m corresponding to one berm height), taking into account the joint sets. No cohesion on the joints is considered in the computation, as this figure is very uncertain and scattered around its average value, and greatly influences the computation results. This analysis results in both active and passive anchors, with priority for passive anchors (rockbolts, easier to place and less expensive than pre-stressed anchors). Active anchors are designed in extreme cases, where the stability of the rock surface cannot be ensured with rockbolts only.

Generally, four different load cases are considered for the global stability analysis:

1. *Structural Loads* – in addition to the weight of the potential rock wedge, these loads are the loads imposed by any structure which affects the stability of the slope (for example, the overflow spillway inlet structure weight when checking the stability of the slope located at the downhill side).
2. *Rapid drawdown of the reservoir* – it is assumed that some water pressure will build up on the failure plane if the reservoir is drawn down to the minimum operating level relatively rapidly. This is for the following reasons: grouting is undertaken in the slopes for passive grouted anchors and active prestressed anchors. The fault planes are expected to have a lower permeability than the normal rock mass. There will possibly be some reduced efficiency of the installed drainage measures. This particular load case is difficult to quantify due to the number of uncertainties, such as permeability of the failure plane, softening effect of water on the fault properties, flow conditions within the fault, and water recharge from the mountain side, which could have a significant impact on the final design value. A value of 30% of the change in water pressure during drawdown is applied to the fault planes as agreed with the Owner.
3. *Earthquake* – the operating basis earthquake (OBE) horizontal acceleration of 0.2 g is assumed.
4. *Reservoir level* – for cross-sections below the dam crest (397m asl), the reservoir operating levels is considered with the following cases: normal operating level (392m) and minimum operating level (34 m).

**Above: Figure 5 – Excavation at the Right Bank at chainage 310m between elevations 420 and 427m asl**

**Right: Figure 6 – Installation of pre-stressed anchors in December 2004 on the Right Bank at el. 345 and 348.5m asl; Far Right: Figure 7 – Current view of the left bank (20 February 2009)**

The stability design takes into account a maximal factor of safety of 1.5 in general when only structural load case applies, and a minimum factor of safety of 1.0 in general when considering that all loads occur at the same time.

## Excavation and support

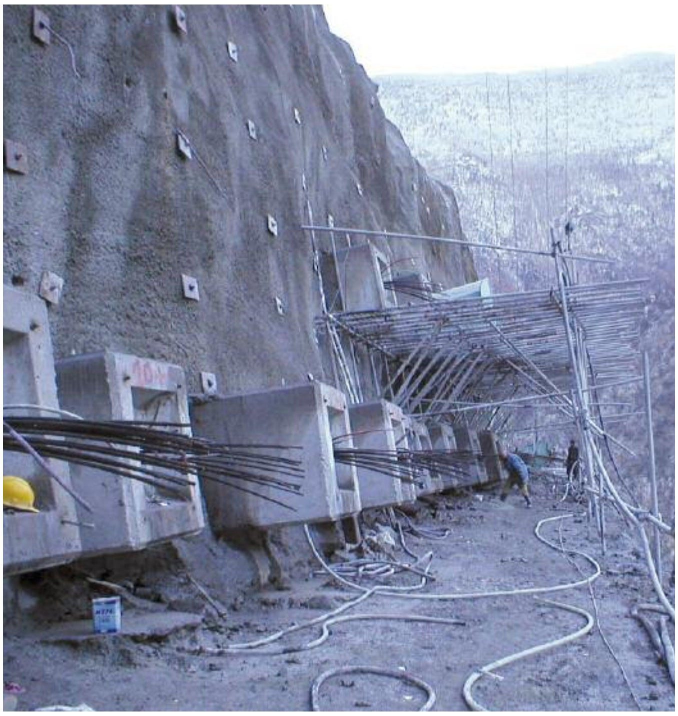
The first phase of excavation is the removal of the decompressed rock mass, which is not suitable for use as aggregate for concrete production. The thickness of this layer is observed to be from 5 to 10m, parallel to the natural rock surface. The extremely weathered layer close to the surface makes it impossible to apply a regular blasting pattern. Therefore dozers or excavators (generally back-hoes) are used for excavation of the overburden.

The second phase of excavation consists of the large mass excavation between the first phase and the foundation. The most important point in this excavation is to provide a smooth geometry and not to disturb the design excavation line in the interfaces with the berms and avoid damage to the foundations.

Drilling and blasting using the pre-splitting method has been undertaken at the Deriner site for the majority of the excavations since the start of the works. However, the first sections of the dam foundation were excavated using a different method – a controlled blasting method which, as applied on site, has been observed to yield less favourable results. Pre-splitting on the dam foundation below el. 297m on the Left Bank and el. 342m on the Right Bank showed improvements in the excavation method. Figure 5 shows the rock surface of half a berm just after blasting.

Where found necessary, pre-stressed anchors are installed. The nominal load $T_n$ of the prestressed anchor is set to 1860 kN and therefore induced a Lock-off Load $T_0$ of 2050 kN and an ultimate Load $T_u$ of 3350 kN, which induce a material factor of safety of 1.80, according to the normative reference EN 1537. Suitability tests performed at the very beginning of the project indicate that a bond length of 8m is adequate for this kind of rock. Figure 6 shows the installation of prestressed anchors on the Right Bank.

The tension in all anchors is checked at one, 30 and 100 days, by means of the tensioning jack. If the increase / decrease in the load of

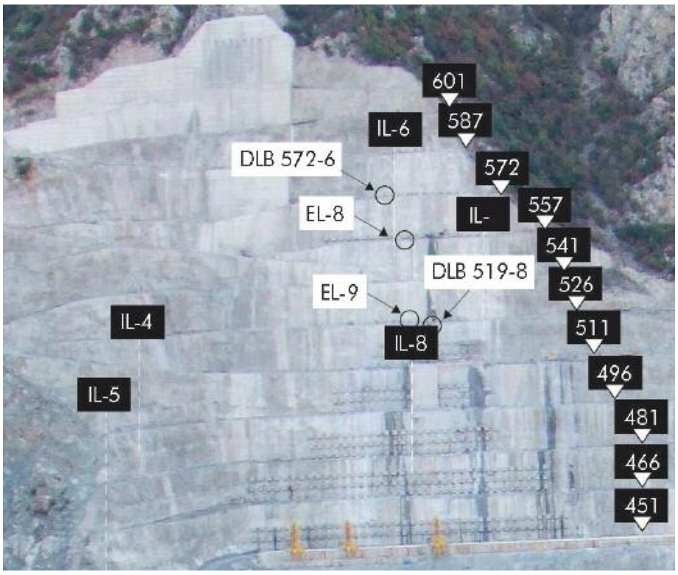
a pre-stressed anchor amounts to more than 5%, additional lift-off tests are undertaken and an updated stability analysis is performed, taking into account all other geological and geotechnical parameters, as well as the continuous monitoring results.

Figure 7 illustrates the current view on the Left Bank. The necessary support is heavier in the Right Bank than in the Left Bank. This is partly due to the slightly lower rock quality in the Right Bank, but mainly as a result of the less favourable joint orientation with respect to the slope orientation in that bank – for example the C-joint, which dips nearly parallel to the slope.

From the very first design reports, that have been prepared by Stucky during the construction stage, based mainly on theoretical considerations, to the current status of open air excavations, approximately 10% more pre-stressed anchors have been installed. The difference is due to a re-assessment of the stability for some particular areas where the geological conditions have been found less favourable.

The design methodology for the slopes stability has been conducted in a safe and economical way. An over-conservative design would have resulted in more anchors installed, from which some would have been ineffective. It has to be pointed out that no major incident or large-scale instabilities have occurred within the whole dam excavation area. Some small rock slides occurred, but without any personal or material damage. For example, on 2 September 2002, a rock portion of some 300–400m$^3$ failed and broke out of the excavated berm at 357–372m asl on the Right Bank excavation, upstream of the dam at approximate chainage 140m, as illustrated in Figure 8.

This critical area is directly above the Power Intake Structure and is also within 20m of the Right Bank Overflow Spillway at that elevation. The rock slide cracked the foundations of the cable crane loading quay at the southern end. Based on the location and the geometry of the slide, it appears that sliding occurred on the C-joints, which had already been identified as the less




**Left: Fig 8 – Rockslide from 2 September 2002 with 300-400m³ of rock.**
**Right: Fig 9 – Monitoring of the left bank between elevations 451 and 620m asl**

favourable joints. This has been the most notable incident that has occurred during excavation, and is generally expected on projects of this scale.

## Surface excavation monitoring

Monitoring of the excavated slope is achieved via numerous different devices, which have been continuously improved and checked throughout the project.

On the Right Bank, 26 inclinometers, six extensometers and 22 Load Cells are installed, whereas these figures amount to 24 Inclinometers, seven extensometers and nine Load Cells for the Left Bank. The regular lift-off tests carried out for each installed pre-stressed anchor complete the monitoring devices installed.

In April 2004, a reliable geodetic monitoring system was installed over the whole site. The average semi-axis of the ellipses of error is of a maximum 2.1mm, which shows the very high accuracy of the geodetic measurements.

With all the above devices and measurements, the monitoring system of the surface excavations is deemed to be accurate and complete. It gives a close follow-up of the surface excavations and allows potential problematic areas where rock movements could occur to be accurately identified

## Monitoring displacements

Excavation of the slopes began in October 2000 (Left Bank – el. 620m asl) and June 2001 (Right Bank – el. 578m asl). From April 2004 and the installation of the geodetic networks, important displacements have been recorded at the top of the excavations.

Of particular note is the area at the top of the Left Bank, above el. 511, which shows important geodetic displacements as well as load increases in the load cell installed at el. 519m asl. Additional lift-off tests have been carried out and show that the four rows of anchors from 511 to 526m asl increase their load, which means that this area is still not totally stabilised, therefore monitoring of the slope in this area is still ongoing.

Figure 9 shows a picture of this area with the installed monitoring devices (Inclinometers IL, Extensometers EL and Load Cells DLB).

These displacements, as well as load increases in the pre-stressed anchors, are currently under investigation. Continuous and close follow-up of the monitoring results, observations and reappraisal of stability analysis are being performed. Additional pre-stressed anchors are also currently being installed in this area.

## Conclusions

The large open air excavations for the Deriner arch dam have been completed and required the installation of many pre-stressed anchors based on both local and global stability studies. To date, the accurate and complete monitoring system and geodetic measurements allows a close follow-up of the behaviour of the excavated slopes. This also allows the identification of particular areas which are still not stabilized, resulting in the installation of additional pre-stressed anchors. Since no major instabilities occurred during the excavation, it is believed that the design approach adopted in this project was adequate. IWP&DC

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Olivier Müller obtained his civil engineer MsC at the Ecole Polytechnique Fèdèrale de Lausanne (EPFL), Switzerland, in 2003.

He joined Stucky Ltd the same year and worked at the Head Office for several projects, mainly about rock mechanics. Particularly from 2003 to 2005, he was deeply involved in the design and follow-up of the Sebeillon-Tridel Railway Tunnel, which is a 3.5km low depth urban railway tunnel in Switzerland.

From December 2005 to June 2006, always on behalf of Stucky, he was resident engineer for the Enguri rehabilitation project in Georgia, which is a 271.5m high arch dam.

In 2007, he spent one year as resident excavation engineer for the construction of the Deriner arch dam in Turkey and brought technical assistance to the Turkish Contractor.

In 2008, he returned to Head Office in Switzerland and is currently working for different dams or underground works projects.

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## Reference

**[1] Wieland, M., Aemmer, M. and Ruoss, R (2008) Designs Aspects of Deriner Dam. Int. Water Power & Dam Construction, Volume 60 Number 7, pp19-23 (Available online at http://www.waterpowermagazine.com/story.asp?storyCode=2050150)**