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PLANNING OF PRESSURIZED HEADRACE TUNNEL IN ALBANIA

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Abstract

A major hydropower project is under planning in the eastern parts of Albania. The 10.7km long pressurized headrace tunnel is designed based on the Norwegian "unlined waterway" principle, only including the required rock support.

1 Background and Project Description

A major hydropower project is under planning in the eastern parts of Albania, consisting of three hydropower plants along the Devoll river with an installed capacity of approximately 272MW, which will generate 800GWh once the plants are finished and operating, corresponding to an increase of electricity production in Albania by 20%.

The upper hydropower plant, HPP Moglicë, is designed according to Norwegian design principles with an unlined pressure tunnel, and utilizes a head of 300m along an approx. 22km long stretch of Devoll River, as schematically shown in Figure 1.

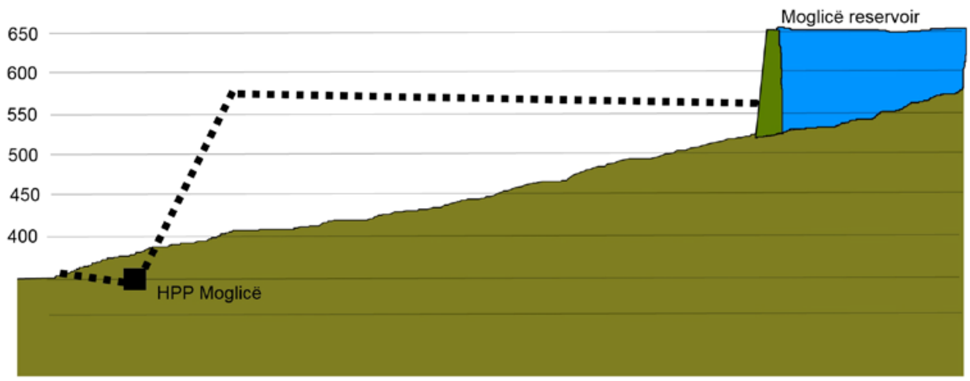

Figure 1: Schematic overview of HPP3

The intake is situated in the Moglicë reservoir created by the approximately 150m high Moglicë Dam, outlined in Figure 2.

A headrace tunnel of length 10.7km conveys the water to the powerhouse located underground on the north bank of Devoll River. The tailrace tunnel is approx. 900m long leading to the Devoll River. HPP Moglicë powerhouse is equipped with two Francis units with a total combined capacity of 171MW and an average annual energy production of 445GWh.

Figure 2: Outline of Moglicë dam

The project owner is a joint venture between Norwegian Statkraft, and Austrian EVN, established in Albania under company name Devoll Hydropower Project (DHP). Main consultants for the concept study, feasibility study and tender design have been Norconsult AS, supported by Multiconsult AS on all underground design works.

2 Ground conditions and investigations

2.1 General geology

In any hydropower project the ground conditions are a great project uncertainty, and great efforts should be made to provide a good understanding of the geological conditions affecting the project. The geological units encountered in the project area vary considerably both in their origin and in their mechanical properties. There are two main lithological units, the ophiolitic rocks, mainly variants of peridotite, shown in green color in Figure 3, and various sedimentary rocks, shown in pink and yellow colour in Figure 3. The border between these two units is of tectonic nature and of very poor quality.

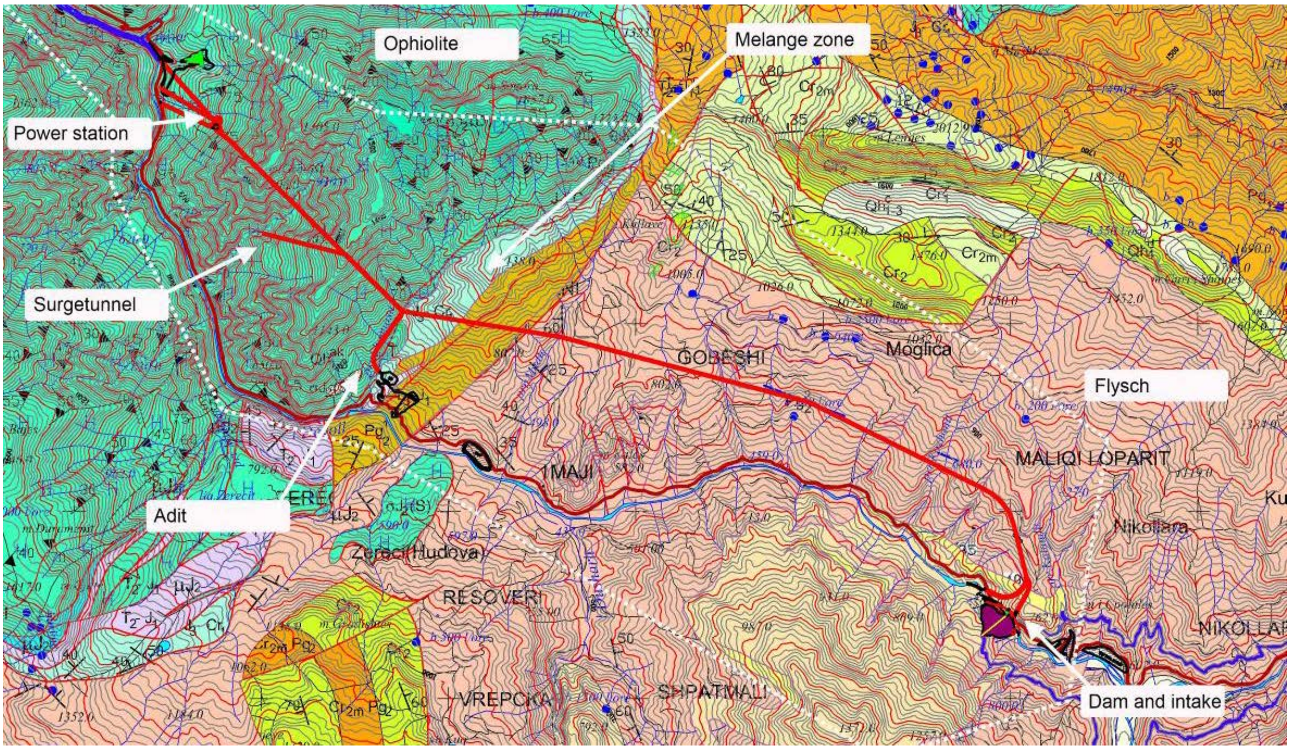

Figure 3: Excerpt of the geological map of Albania with the tunnel system shown in red

2.2 Rock mass conditions

2.2.1 Introduction

On the mechanically sound end of the rock mass scale are the ophiolitic rocks found in the downstream part in this project. The main concern regarding rock mass quality within the ophiolite has been localized serpentinization of the peridotite and fault zones.

The flysch typically consist of alternating layers of claystone, siltstone, sandstone and rare conglomerates, with the stronger sandstones typically creating a rigid “skeleton” within the weaker silt- and claystones. The mechanical characteristic of the rock mass is thus governed not only by the characteristics of each individual layer, but also by the proportion of the different rock types.

While the ophiolite found in the project area is fairly homogenous and sound, this is not the case with the Flysch which can be extremely heterogeneous, and locally of very poor rock mass quality. A geological cross section showing the various rock types from core drilling is shown in Figure 4.

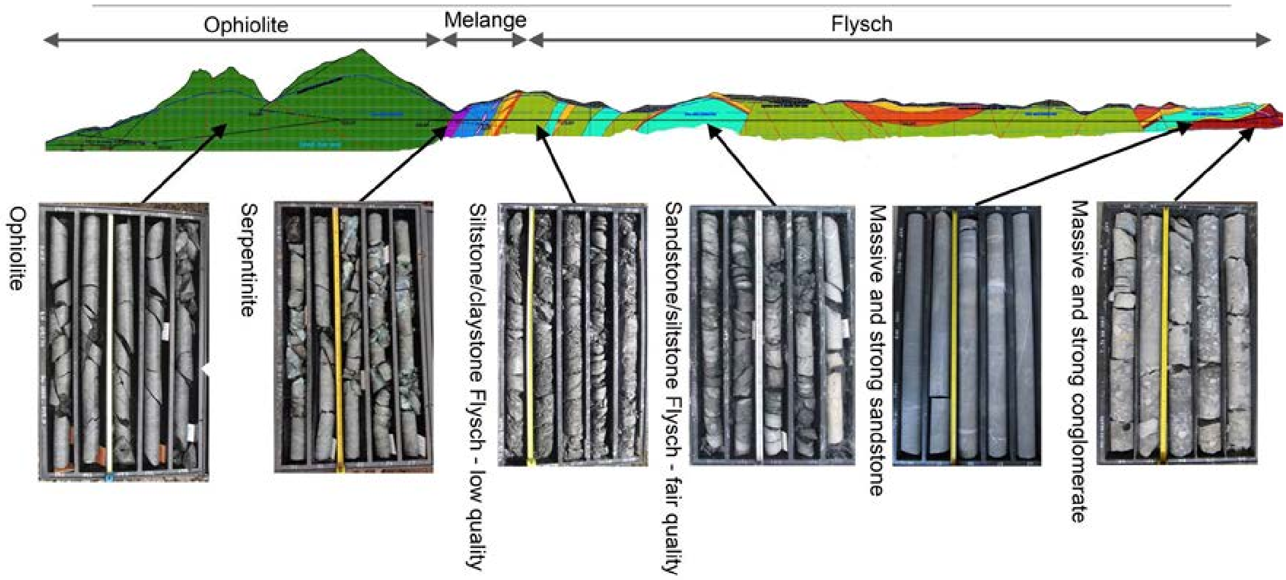

Figure 4: Overview of geology along the tunnlel alignment

3 Ground investigations

Ground investigations for the underground part of HPP Moglicë consisted mainly of rotary core drillings and refraction seismic measurements. From the core drilling representative samples of rock along the headrace tunnel alignment were taken for geological study and laboratory testing.

Field testing of rock mass permeability, groundwater level and long term monitoring of ground water level were performed in all investigation boreholes. Additionally, stress measurements were performed in selected boreholes near the inclined pressure tunnel.

A verification of the interpreted rock mass quality, ground water and in-situ stress conditions will be done by a detailed ground investigation program performed concurrent with excavation of tunnels and caverns belonging to Moglicë power station area, including the pressure tunnel.

3.1 Rotary core drilling

A total of six core drilling locations were performed for the headrace tunnel, giving a total of 860 m core material. Due to the difficult accessible drilling locations, the rotary core drilling was performed by belted drilling rigs, as shown in Figure 5.

Besides providing essential information about the sub-surface rock mass and hydrogeological conditions, the core drilling was aimed at providing representative core samples from all rock types that could be encountered during the tunnelling works. All holes along the headrace tunnel alignment were equipped with stand-pipe piezometers enabling surveying of ground water levels.

Figure 5: Core drilling in the Flysch

3.2 Laboratory sampling and testing

Although any experienced engineering geologist could obtain extensive knowledge about rock strength and petrography aided only by a hammer and a magnifying glass, exact knowledge of the mechanical properties of intact rock require laboratory testing of representative samples of intact rock. Besides the standard index tests, such as density, porosity and thin sections, the following tests were considered necessary:

- Uniaxial compressive strength, UCS
- E-modulus, E
- Point load, Is_{50}
- Brazilian tensile strength, BTS
- Sound Velocity, v_p
- Petrographic analysis/thin section
- Drilling rate index/Cutter life index (DRI/CLI)
- Cerchar scratch test
- Slake durability

3.3 Rock stress measurements

To evaluate the rock stress levels and orientation several hydraulic fracturing (HF) tests were performed. The HF testing had as objective to present indications on the minimum rock stress (σ_3) in the area as close as possible to critical design components of the project. The purpose of the investigation is to provide information on the state of stress in the rock-mass at depth to:

- Confirm that σ_3 is higher than the maximum planned head with a safety factor, avoiding any unwanted hydraulic fracturing and loss of water
- Enable an optimum orientation of the long axis of the power- and transformer caverns to the σ_1

4 Design Basis and Experience

4.1 General

Already at an early stage during the planning it was evident that both topographical and geological conditions were suitable for an unlined "Norwegian" design of the inclined part of the headrace tunnel. Unlined in this context means a water tunnel without steel lining or hydraulic concrete lining, with rock support only consisting of rock bolts alone or in combination with sprayed concrete applied only on parts of the tunnel surface, thicker reinforced sprayed concrete or shorter concrete sections where required.

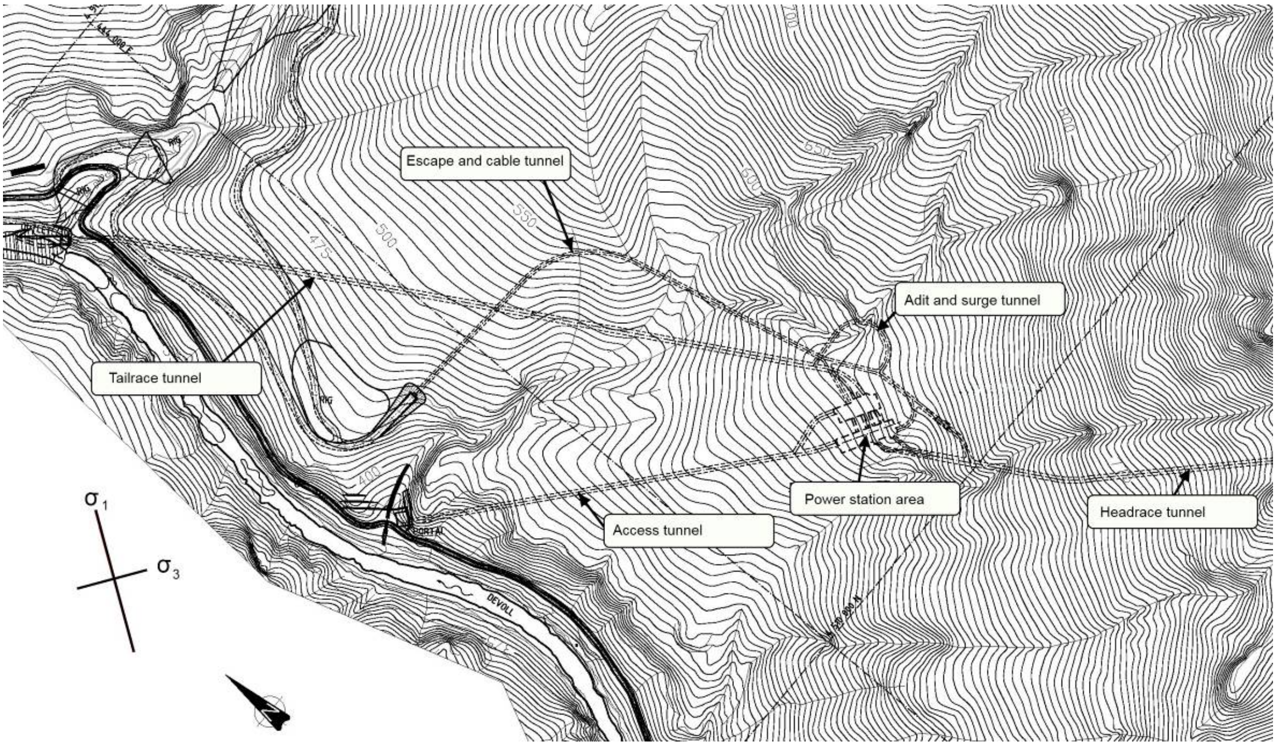

Figure 6: Power station layout and stress field directions

4.2 Design features

The requirements to an unlined pressure tunnel are quite straight forward – to remain stable for the life of the project under the various loading conditions without undue water loss, nor severe maintenance requirements. For a successful design of the unlined pressure tunnel, the following desirable geological characteristics should be present:

1. Sufficient confinement; the entire tunnel must be set deeply enough within the rock mass to ensure that adequate in-situ compressive stress is available to prevent hydraulic jacking.
2. Suitable rock mass; the rock material must be long-term durable and preferably have good and fair tunnelling qualities without soluble or weak fillings.
3. Sufficient long-term tunnel stability; i.e. no slide, cave-in must take place during operation of the power plant.
4. Other important conditions, such as:
   - Low rockmass permeability, and
   - Sufficiently high groundwater level.

Once the initial stress field was estimated from a crude overburden assessment, analytical solutions were used to evaluate the critical tunnel water pressure where hydraulic jacking may take place. The location of the tunnel was verified by checking the in-situ stress conditions from hydraulic fracturing tests in deep drill holes. Later, during the construction phase, further hydraulic fracturing tests performed in drill holes in the vicinity of the penstock area shall verify that sufficient stress conditions are met at the final tunnel location.

The choice of the factor of safety is influenced by the complexity and degree of knowledge of the geology, the accuracy with which the in-situ stresses and the maximum tunnel water pressure are known. As geology and the water rock stresses is known to a satisfactory degree, a value of F = 1.2 to 1.3 is used for the maximum dynamic pressure, and F = 1.3 to 1.5 for the maximum static pressure.

4.3 Measures to reduce risks of undesired failures in the unlined pressure tunnel

Special attention *during the design* of the unlined pressure tunnel was made to the:

- location of the tunnel with respect to the topography; ensure that the headrace tunnel at no point has less than 100 m overburden, and in flysch not more than 250m overburden
- investigation of geological conditions and understanding of the geology, as presented in Chapter 3, and
- magnitude of the in-situ least principle stress.

The key measures to reduce the risk of failure are:

- The use of a design where it is possible to decide the final location after stresses can be measured in the tunnel during construction
- Geological mapping and decision on final support means by qualified personnel after each blast
- In-situ testing and monitoring *during construction*
- Observations of water seepage into the tunnel together with remedial grouting
- The installation of rock support will be followed-up to ensure appropriate support quality.

5 Tender Design

5.1 Layout

The tunnel layout presented below is the result of adapted design to findings from the ground investigations, together with the project specific minimum requirements.

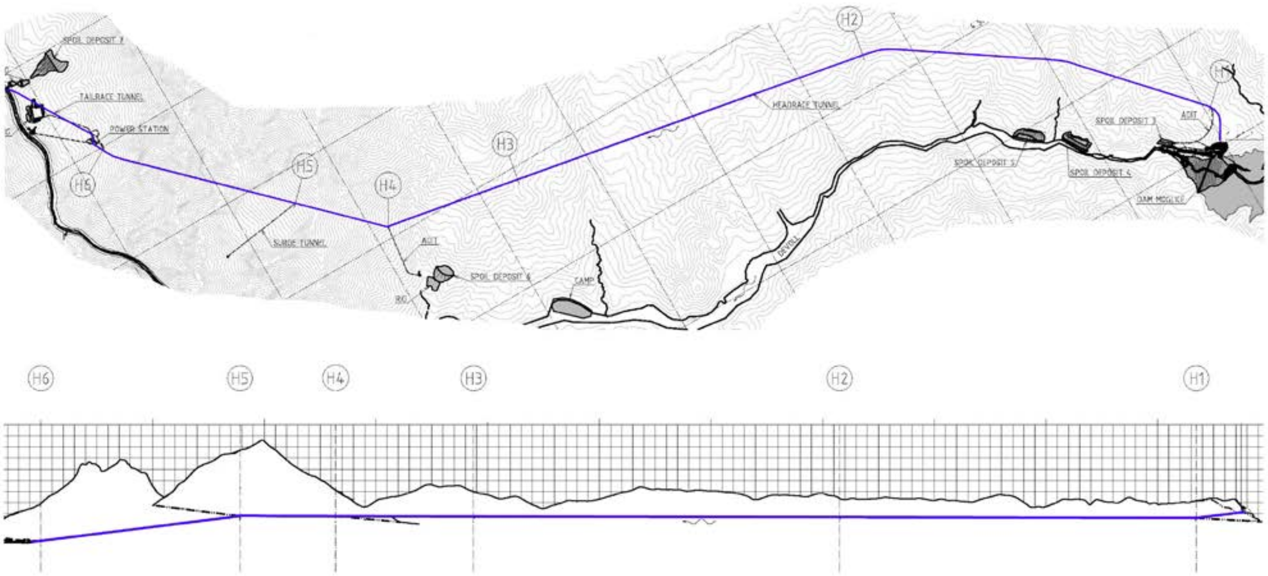

Figure 7: Layout of hydropower scheme - waterway highlighted blue

Table 1: Selected Tunnel Design Concept for each tunnel section

Headrace tunnel section	Internal water pressure (m)	Terrain overburden (m)	General rock formation	Tunnel Design Concept/ Excavation method
Intake to H1	0-90	0-150	Flysch	Unlined/sprayed concrete (D&B)
H1-H3	90	100-250	Flysch	Segmental lining (TBM). Drained and without gaskets
H3-H4	80	100-300	Melange	Concrete lining (D&B), drained
H4-H6	80-340	350-800	Ophiolite	Unlined/sprayed concrete (D&B), drained
Penstock tunnels	340	350	Ophiolite	Steel lined (D&B)

5.2 Engineering Geological follow up during construction

It should be recognized that the Tender Design quantities do not represent the real and final distribution of rock support and grouting quantities to be used during construction, neither the required measures for investigation, instrumentation or tests.

During construction the observational method is used for establishing the final rock support and grouting design. In general the method is described as observation of the behaviour of the newly excavated face, decide on the necessary support (including water treatment) and keep the tunnel under surveillance for a period of time to verify the functioning of the installed support. If the support is found insufficient, additional support has to be placed until stability is assured. This cause the real support to be distributed quite independent of the support classes listed in Engineering Geological Report.

The geology at each of the tunnel face will be mapped by qualified and experienced Engineering Geologists, and the rock mass quality classified immediately following the blasting of a new round. The appropriate rock support resources will then be selected based on the findings.

The permanent support will be designed incorporating the initial rock support and released for execution at suitable intervals.

6 Tunnelling Contract format

The aim of rock support is both to provide safe working conditions during construction and sufficient long-term stability of the underground opening. Safe working conditions, which are the responsibility of Contractor, are to be taken care of by the initial rock support. For long-term stability, the extent of additional rock support will be decided by Employer.

The contract format carefully selected for this project, is the FIDIC red book, and based on Employers design and mainly unit prices for the tunnelling works.

Basically, the Contractor has responsibility for his unit prices and unit capacities, whereas the Employer has the risk and responsibility for the total quantities defined and installed during construction. All rock support and grouting works elements have a separate payment item in the BoQ`s.

Basic principle is that each cost element (item) shall be measured and paid according to installed and approved quantities, not according to "rock mass quality" or "rock support class" as may be specified in some projects.

7 Recommendations on filling and dewatering of the unlined pressure tunnel

The initial water filling of the pressure tunnel should be carefully controlled to limit differences in pressures between the groundwater and the tunnel water. During construction, the tunnel has been open for several years, and drainage of the rock massive has taken place. Slowly filling the tunnel allows pressure equalization to occur, and thereby limits deformation of the rock and supportive structures. The rate should depend on the rock mass conditions and the types and extent of rock support installed. An infilling rate of 5 to 20m head/hour has been found adequate where good rock mass conditions.

Dewatering of pressure tunnels should be done even more carefully, preferably at a rate between < 5 to 10m head/hour, utilizing slower rates for high-head plants. Ground water changes should be noted as the dewatering takes place. A detailed inspection of the tunnel should be done immediately after the dewatering is complete. Records of inflow, local failures of the rock or rock support, cracking or other distress should be recorded.

8 Control of the tunnel during power production

An unlined water tunnel cannot be considered completed until the tunnel has been dewatered and its performance verified, included installation of any clean-up works and necessary additional support works. For this, it is generally recommended that such tunnel should be dewatered within approximately one year of operation, so that such works could be undertaken during the contractor's mandatory defects remediation responsibility.

Head loss should be constantly recorded during power production and even head losses in the range of a few centimetres should be carefully analysed, as this may indicate local rock falls of several m^3.

9 Conclusion

An "unlined" pressure tunnel design concept is adapted to the local geological and topographical conditions at the HPP Moglicë. The tender design is based on extensive geological mapping and ground investigations whereas the final tunnel and rock support design shall be defined based on observational methods during construction phase. Construction is planned to start in 2015 and the client DHP has decided to continue to final design according to the principles described in this article due to the great economical savings compared to a more "standard" lining design.

10 Acknowledgement

We would like to thank Devoll Hydropower Sh. A. for allowing us to present this interesting project.

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