The Tauernmoos pressure tunnel: a multidisciplinary approach to verify and ensure design rock mass conditions during construction

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ABSTRACT: A 1.6 km long headrace tunnel has been constructed for the Tauernmoos hydro power plant. For the dimensioning of the tunnel lining, a minimum elastic modulus requirement of 15 GPa for the surrounding rock mass was formulated. The objective for the geological documentation was therefore the identification of zones with an elastic modulus below the required minimum. This was achieved by detailed mapping of the embrasure as well as by correlating the UCS, determined by Schmidt hammer, with the elastic modulus. For parts of the headrace tunnel, additional dynamic elastic moduli were established using refraction seismic surveying. That way, critical areas in terms of deformational behavior along the headrace tunnel could be determined. A consolidation grouting campaign with primary and secondary injection rings was carried out to improve the elastic moduli. The success of this measures was confirmed with a post-grouting seismic survey.

Keywords: headrace tunnel, elastic modulus, Schmidt hammer, refraction seismic, grouting.

1 THE TAUERNMOOS PUMPED STORAGE HYDRO POWER PLANT

1.1 Project outline

Currently, the Austrian Federal Railways (ÖBB) are constructing the Tauernmoos pumped-storage hydro power plant in the Stubach valley (Salzburg, Austria). The difference in altitude between the existing upper (Weißsee) and lower reservoir (Tauernmoossee) equals 230 m. A 1.6 km long headrace tunnel connects the upper reservoir to the power cavern (Figure 1).

1.2 Design of the headrace tunnel

The bottom 216 m of the pressure tunnel will be equipped with an embedded steel penstock with 4 m inner diameter. The remaining part of headrace tunnel with an inner diameter of 5.2 m will be supplied with a PVC-membrane and an in-situ concrete lining without reinforcement. This section is designed as prestressed concrete lining. The design is based on the method defined by Seeber (1999), considering that the internal water pressure, causing extension and tensile stresses in the concrete lining, will be taken partly by the surrounding rock mass and partly by the concrete lining. In the concrete lining, tensile forces could cause cracks and seepage of water out of the tunnel. To prevent such a seepage that could have a destabilizing effect on the rock mass, high-pressure annual gap grouting is carried out. The bearing capacity of the surrounding rock mass is defined by the rock mass overburden and rock mass stiffness characteristics. If the minimum primary stress along the headrace tunnel is lower than the internal water pressure, hydrofracturing of the surrounding rock mass by water seeping out of the tunnel cannot be excluded. As an additional safety against water seeping out of the penstock, a 3 mm PVC membrane is installed between the concrete lining and the surrounding rock mass. The membrane must bridge possible tensile cracks in the rock mass and the system requires a deformation modulus for the latter of at least 15 GPa.

1.3 Geological outline

The project area is located within the Tauern window and is part of the Venediger nappe system (Frisch 1977), containing Variscan metagranites called the "Zentralgneis". Virtually all excavation work for the headrace tunnel was done in this granitic gneisses (Figure 1). Schists and mafic gneisses that occur occasionally in the area did not play a significant role and are therefore not expounded on.

Three rock mass types have been defined for the Zentralgneis (Forstinger & Stadlmann 2019). Ranges of their elastic moduli and selected representative values are listed in Table 1.

Elastic moduli below the required 15 GPa could partially be expected in rock masses GA 1b and GA 2. As it is not possible readily to estimate an elastic modulus for a rock mass, some reliable, yet simple method for determining it during the standard geological documentation, had to be found.

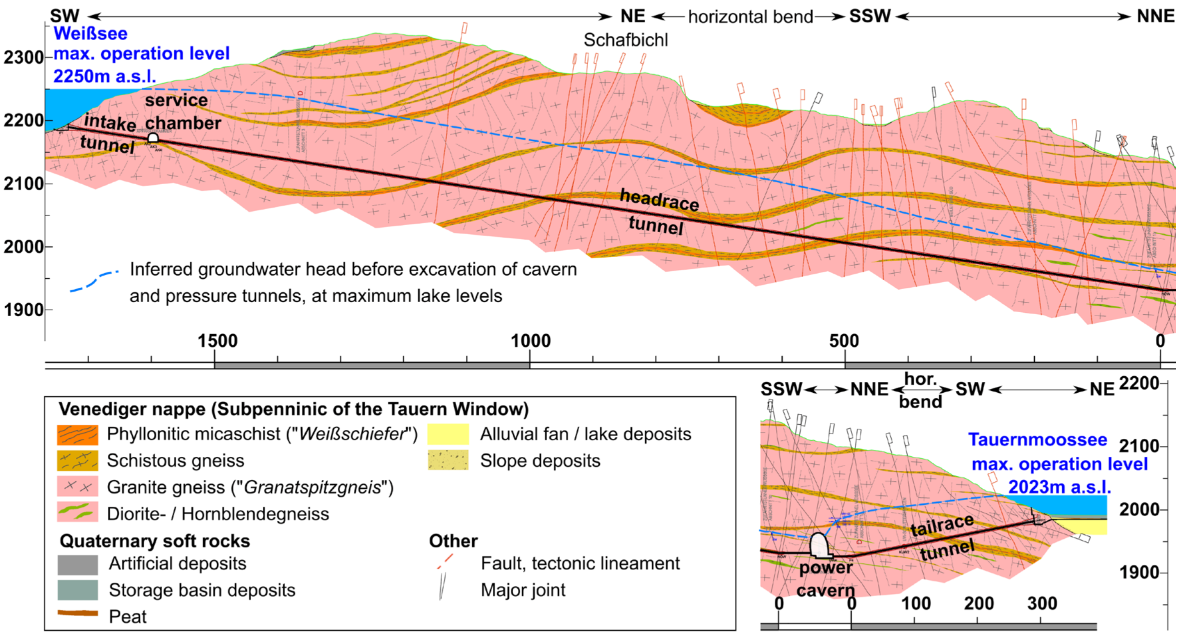

Figure 1. Geological cross-section along the headrace and tailrace tunnels.

Table 1. Rock mass types for the Zentralgneis and their Young´s moduli E in GPa.

Rock mass type		$E_{representative}$	E_{min}	E_{max}
GA 1a	Gneiss, compact	25	15	37
GA 1b	Gneiss, blocky	19	10	32
GA 2	Gneiss, strongly foliated	7	2	22

2 METHODOLOGICAL APPROACH

A three-pronged approach consisting of an extended geological documentation, Schmidt hammer measurements and refraction seismic surveying was chosen to determine the elastic moduli.

2.1 *Extended geological documentation*

Rock types, joints and faults that could negatively affect the Young's modulus were recorded at the heading face, which, for the purpose of interpretation, necessitates interpolation between two successive documentations. To minimize uncertainties concerning the extent of a structure, a detailed mapping of the tunnel's embrasure was carried out. The results were depicted in a vertical cross section along as well as in a horizontal projection 2 m above the tunnel axis. GSI – values (Geological Strength Index, Marinos & Hoek 2000) were estimated for structurally homogenous parts.

2.2 *Correlation of uniaxial compressive strength and elastic modulus*

During earlier stages of this project, the UCS as well as E were determined on various Zentralgneis samples from the Tauernmoos area. When plotted against each other (Figure 2), they show a linear correlation that is given as

$$E = 126.61 \times UCS + 9230 \qquad (1)$$

With a determination coefficient of $R^2 = 0.80$, the correlation can be considered as significant, allowing for the calculation of an elastic modulus based on UCS.

The latter were acquired by in situ Schmidt hammer measurements, utilizing an Proceq Schmidt – type hammer which allows for a direct conversion of rebound values in UCS up to 70 MPa. The conversion of rebound values yielding a higher UCS was based on literature data (Merkel & Breit 2018).

With those UCS values, elastic moduli could be calculated according to formula (1), representing the rock, but not the rock mass. To establish values for the latter, some sort of reduction had to be applied accounting for the influence of joints. This was done based on the method suggested by Hoek & Diederichs (2006) using the GSI – value.

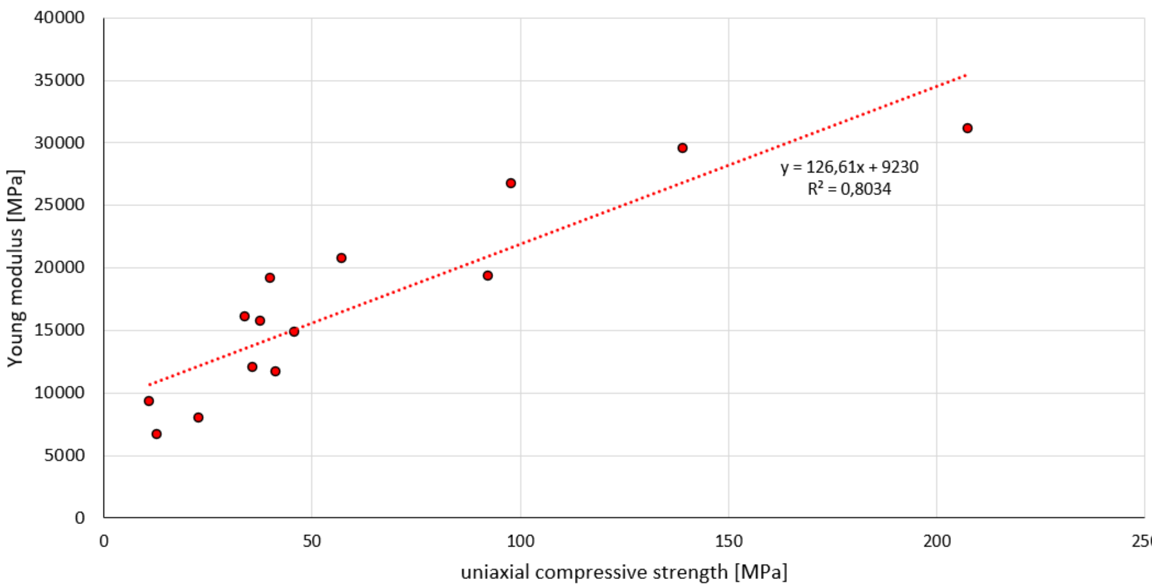

Figure 2. Relation between UCS and E for Zentralgneis – samples (data from Forstinger & Stadlmann 2015).

2.3 *Refraction seismic surveying*

To evaluate and quantify the Young’s modulus of the rock mass before and after consolidation measures, seismic surveys had been planned within selected parts of the headrace tunnel. Aim of the seismic investigations was primarily to determine the dynamic elastic modulus before and secondarily to show the effect of the consolidation measures within a certain part of the headrace tunnel after grouting.

The seismic measurements were done on a profile along the left side of the open tunnel wall from the power cavern upwards. Primary seismic investigations were executed from chainage 0+020 m to 0+590 m, the follow-up investigation after grouting was done from 0+195 up to 0+410 m. Single geophones had been drilled into the tunnel wall at a height of about 2 m above ground and at a spacing of 1 m. Hammer blows against the open tunnel wall utilizing a recoilless hammer had been done at a spacing of 0.5 m. We used a moving, fixed spread layout using 144 active stations while rolling along half of the geophone spread when moving forward.

The seismic data sets have been processed for refracted waves as a first guess and then evaluated using a surface-tomographic approach. Outcome of this approach is the two-dimensional distribution of the seismic pressure v_p and shear v_s velocities along the tunnel wall. Using the basic laws of elasticity in isotropic media one may calculate dynamic elastic moduli from the seismic velocities and the density of rock (Barton, 2007). The latter was given as an average of 2670 kg/m^3. The differences in strain amplitude by static and dynamic tests, the presence of pores, cracks or joints leads to a mismatch of static and dynamic moduli, except where rock quality is very high and strains are very small. Then the static and dynamic moduli are likely to be very close (Barton, 2007). Based on numerous studies on the ratio of dynamic and static elastic moduli derived from laboratory tests (e.g. Davarpanah et al. 2020), it was decided to reduce the dynamic moduli by 50 %.

3 CONSOLIDATION GROUTING

3.1 *Determination of grouting necessity*

In rock masses with an elastic modulus < 15 GPa consolidation by grouting was necessary. As a first step, the results from the geological documentation and the Schmidt hammer measurements were compared with the results from the refraction seismic surveying.

It was clearly possible to correlate certain fracture corridors with seismically determined areas displaying elastic moduli < 15 GPa. The fractures in question were displaying some aperture (up to 5 mm, mostly below 1 mm) with weathered surfaces and occasionally soft fillings of silty composition. Their spacing was mostly in the range from 6 to 60 cm.

Furthermore, the Young´s moduli derived from Schmidt hammer measurements were slightly higher than the reduced seismically derived values. Nevertheless, both depicted the same basic trends and independently determined coinciding areas of low elastic moduli.

As a second step, the previously described methods were applied to define areas to be consolidated by grouting. The only exception from this were the first 200 m of the prestressed concrete lining that were grouted indiscriminately due to the high internal pressures expected there. Overall, the areas marked for consolidation grouting amounted to a length of 720 m.

3.2 *Grouting concept*

The grouting concept envisioned primary injection rings of 8 boreholes with a length of 3.5 m each and a 2.5 m interval along the tunnel axis. If necessary, secondary injection rings with the same basic parameters were to be placed halfway between the primary rings. As the main joint orientation was characterized by steep dip angles > 80° and a strike perpendicular to the tunnel axis, the injection rings were tilted 15° in axis direction and rotated sideways by 15°.

For the slurry, Portland cement with a Blaine value of 4400 was used. The w/c – ratio was specified as 0.8, with an additional 1 % of grouting aid. Marsh Cone flow times of 35 – 40 s and a maximum bleeding of 5 % had to be observed. The holding pressure was defined with 15 bar. A borehole was considered to have been successfully grouted if the intake did exceed 4 l (=borehole volume), a pressure of 15 bar was reached and could be held above at least 10 bar for 2 minutes.

3.3 Grouting implementation

The grouting campaign comprised 297 primary and 97 secondary injection rings. The whole grouting process was documented automatically with a permanent recording. Furthermore, handwritten notes were taken, containing key parameters and additional information such as interconnections with other boreholes, equipment malfunctions and so on. Based on this information, the geologist responsible deduced the grouting success and decided about the necessity of further measures.

4 ASSESSMENT OF GROUTING MEASURES

4.1 Evaluation of the grouting success

Generally, all injection rings could be finished successfully according to the criteria above. While numerous boreholes in the primary injection rings showed a significant intake of grout (up to 540 l), no interconnection between neighboring boreholes or injection rings could be observed. This led to the likely conclusion that the cement slurry did not spread enough to sufficiently consolidate the rock mass in between two injection rings. Accordingly, secondary injection rings were situated between primary rings displaying the characteristics mentioned above.

The assumption of an insufficient consolidation between the primary rings is supported by the fact that most of the secondary rings showed similar intakes as their neighboring primary ones (Figure 3) and similar average grout intakes (169 l primary rings, 155 l secondary rings).

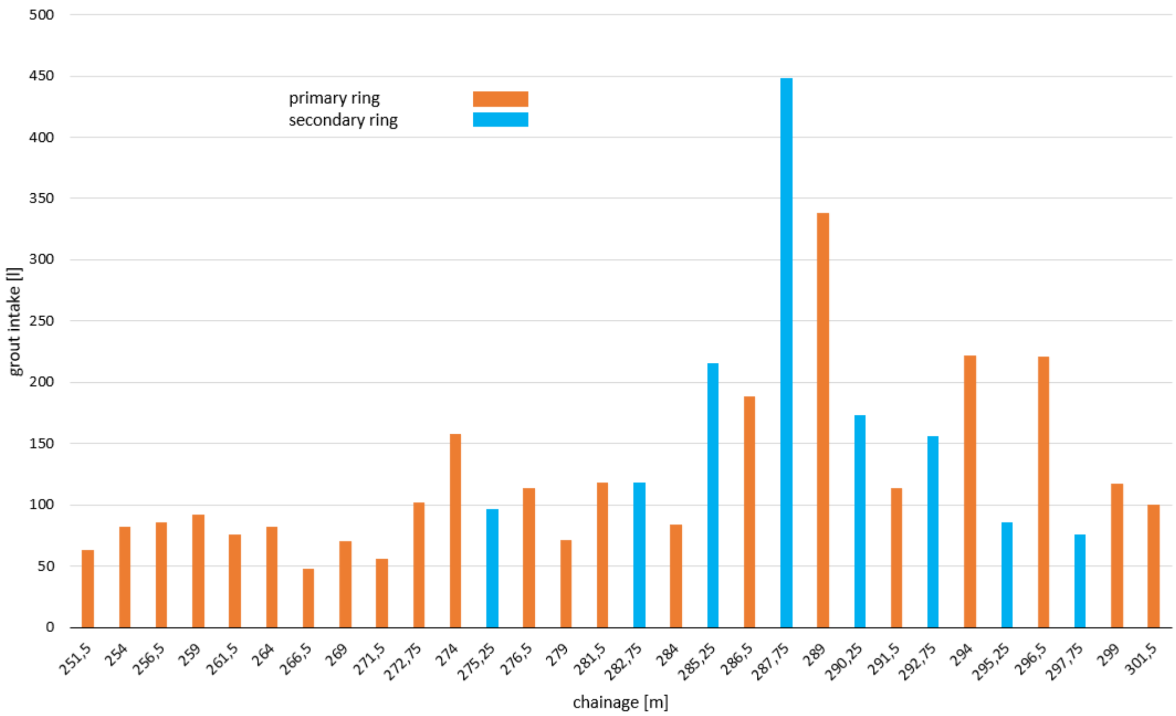

Figure 3. Exemplary intake of slurry per grouting ring between chainage 250 and 300 of the headrace tunnel.

4.2 *Pre- and post-grouting seismic survey*

Figure 4 shows the dynamic elastic moduli before (light) and after (dark) grouting measures along a 200 m long section of the headrace tunnel. One clearly can see the improvement in the local minima, which had been confirmed by geological mapping as zones of weakness before.

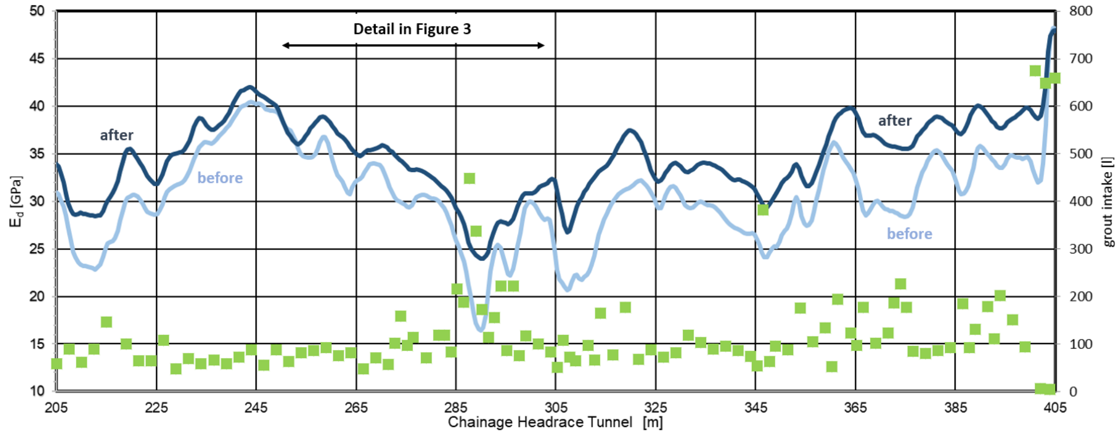

Figure 4. E_{dyn} along the tunnel wall before and after grouting underlain with total grout-intake.

In summary the seismic investigations showed an overall improvement and an increase of about 20 up to 30 % for the Young´s modulus of the rock mass.

5 DISCUSSION AND CONCLUSION

The grouting measures applied resulted in an overall improvement of the elastic modulus, with seismically determined zones of low moduli displaying the highest grout intakes. In these areas, grouting raised the elastic moduli by 20 to 30 %. Even in areas with comparatively low intake, a measurable improvement could be achieved. Overall, the elastic modulus could mostly be raised to the minimum level of what was deemed necessary. The methods described above can be considered a reliable means to determine whether the design rock mass criteria are met in situ.

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