15th ISRM Congress 2023 & 72nd Geomechanics Colloquium. Schubert & Kluckner (eds.) © ÖGG
ISRM 15th International Congress on Rock Mechanics and Rock Engineering – Challenges in Rock Mechanics and Rock Engineering
October 9-14, 2023, Salzburg, Austria
ISBN 978-3-9503898-3-8

Assessment on the in-situ rock stress condition along an unlined pressure tunnel/shaft of a Norwegian Hydropower Project using numerical modeling

Bikash Chaudhary
Norwegian University of Science and Technology, Trondheim, Norway

Krishna Kanta Panthi
Norwegian University of Science and Technology, Trondheim, Norway

Nghia Quoc Trinh
SINTEF, Trondheim, Norway

ABSTRACT: Confinement is a pre-requisite in the "Norwegian state-of-art design principles" for unlined pressure tunnels/shafts where the minimum principal stress must exceed internal water pressure. Reliable estimation of in-situ stress state is crucial in implementing unlined pressure tunnels/shafts so that hydraulic jacking/fracturing in the rock mass is avoided. The Norwegian thumb rule may not offer a complete solution in all unlined pressure tunnels/shafts since these are based on the 2D geometry of terrain and thus do not fully represent the in-situ stress environment in 3D. This article evaluates the minimum principal stress state of the unlined headrace system of Bjørnstokk powerplant using 3D numerical modeling. The 3D model is developed with the integration of geological, topographic, and geotectonic features. The model is calibrated using measured in-situ stress data of fixed locations along the waterway system. Discussions are made on how in-situ rock stress differs upon changes in topography and geological setting.

Keywords: Unlined Pressure Tunnels, Minimum Principal Stress, In-situ Stress, Shear Planes, Numerical Modeling.

1 INTRODUCTION

Norwegian Hydropower Projects (HPs) are famously known for cost efficiency, environmental sustainability, unique design, and construction concept of "unlined pressure tunnels/shafts". More than 95% of the total waterway systems of HPs are unlined in Norway (Panthi, 2014). These tunnels/shafts are constructed in such a way that the rock mass itself shall resist the internal water pressure acting during powerplant operation (Broch, 1982). The "Norwegian confinement criteria" proposed by Selmer-Olsen (1969), Bergh-Christensen and Dannevig (1971) and Broch (1982) are the rule of thumbs used for the design of unlined pressure tunnels/shafts. The state-of-art rule of thumb criteria considers both the overburden and lateral cover of the rock mass to secure the risk against hydraulic failure. It is assumed that the minimum principal stress is governed by the vertical overburden and lateral cover. Since criteria are based on 2D geometry, these may not fully represent the true picture of in-situ stress state (Basnet & Panthi,2018). Hence, there are cases of unlined pressure tunnels/shafts failures which were originally designed using the rule of thumb. The conditions such as irregular topography, stress perturbations due to faulting at local level and tectonic movement create a complex in-situ stress state. If the minimum in-situ principal stress is not large enough to balance the water pressure, hydraulic jacking and/or fracturing may occur (Basnet & Panthi, 2019). Such failure will lead to an opening of existing joints or even create new fractures causing water leakage out from the tunnels/shafts (Figure 1). In the rock mass, a 3D in-situ stress state exist which can be represented by the magnitude and orientation of three different principal stresses, i.e., maximum (S_1), intermediate (S_2) and minimum (S_3) principal stresses (Hudson & Harrison, 2000). The flowing water in the unlined pressure tunnel induces pressure (P_w) to the surrounding rock mass equivalent to static head (H). Reliable quantification of in-situ minimum principal stress is therefore crucial.

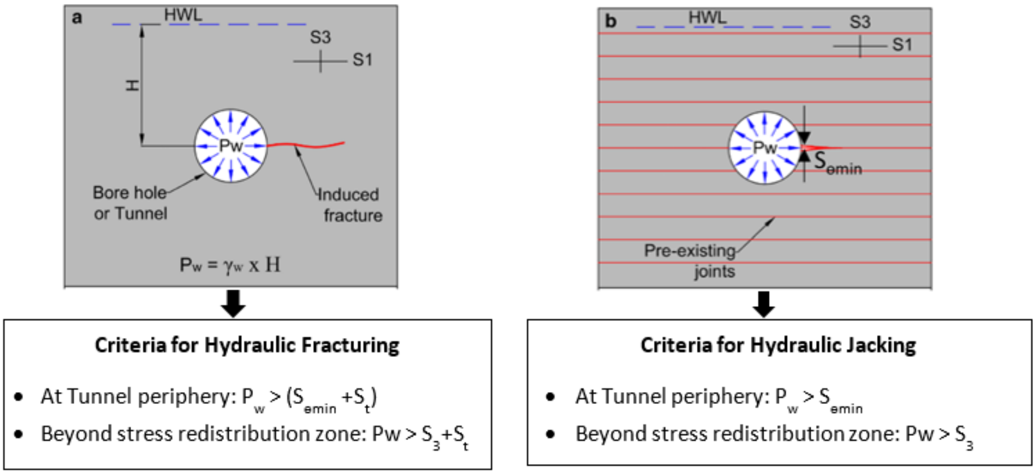

Figure 1. Representation of typical hydraulic fracturing phenomenon in a relatively intact rock mass; b) typical hydraulic jacking phenomenon in a rock mass with pre-existing joint (Basnet & Panthi, 2019).

Therefore, it is important that a hydraulic failure is avoided, and this can be achieved if the unlined pressure tunnels are designed in such a way that the minimum principal stress (S_3) is greater than water pressure (P_w). The main aim of this article is to evaluate the in-situ stress state along the unlined pressure tunnel and shaft alignment of Bjørnstokk powerplant where complex geology and topography exist. The spatial distribution of minimum principal stresses along the unlined pressure tunnel is investigated and quantified using 3D numerical model. It is demonstrated that if relevant constitutive laws and reliable rock mass and interface/weakness planes properties are established, it is possible to assess the in-situ stress state along the whole waterway system.

2 CASE: BJØRNSTOKK POWERPLANT

2.1 Project Layout and Geology

The Bjørnstokk powerplant is located at Tosbotn, Norway. The waterway system of the plant consists of a 250m long inclined pressure shaft followed by a 600 m long unlined pressure tunnel ending at a 10 m long concrete plug and a 330 m long penstock tunnel leading to the powerhouse (Figure 2a). The powerplant has a gross head of 264 m creating a pressure of 2.64 MPa. Initially, the concrete plug in the waterway was positioned using the Norwegian rule of thumb as the design basis. It was assumed that the overburden and lateral cover would provide sufficient confinement along the unlined pressure tunnel and therefore no need was felt to measure the minimum principal stress. However, during the first and second water filling, a hydraulic jacking occurred leading to water leakage. This forced to empty the waterway for inspection and several cracks were found in the tunnel wall upstream of the concrete plug (Solli,2018). Consequently, the initially placed concrete plug was relocated 300 m upstream following the results of in-situ stress measurements carried out.

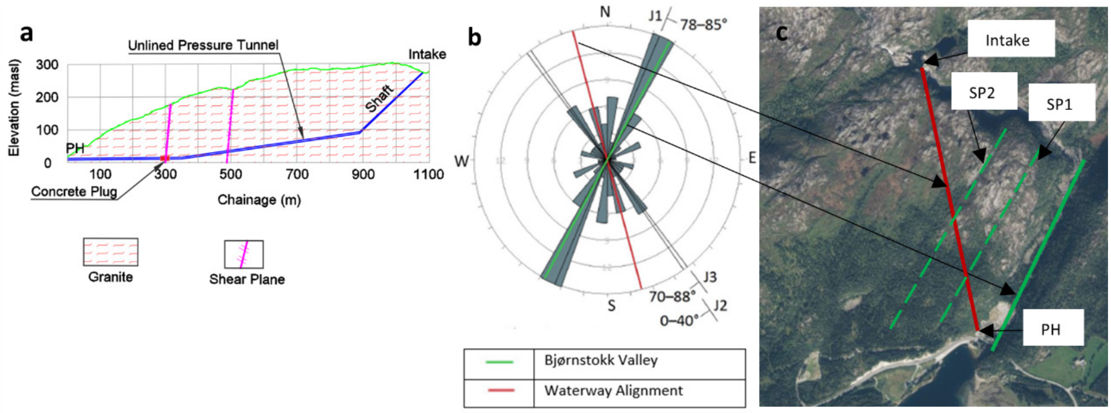

Figure 2. a) Longitudinal profile with geology of waterway system of Bjørnstokk powerplant (modified after Solli, 2018); b) Joint rosette of the main joint sets; c) project plan, shear planes and waterway alignment.

The rock mass in the area falls within the rock mass class of medium to good quality. The rock type along the waterway is granodiorite/granite. The most prominent lineaments found along the waterway represent shear planes (SP1 & SP2) that are oriented NE-SW (Figure 2b & 2c). The laboratory test results of the intact rock sample brought from the shear plane area indicated similar mechanical and mineralogical properties as that of the surrounding rock (Solli, 2018).

2.2 Rock Stress Measurement

After the hydraulic failure that occurred in April 2016, the waterway was emptied, and rock stress measurements were carried out by SINTEF using hydraulic fracturing (HF) method to find out a secured place for the new concrete plug from where the unlined pressure tunnel could start (Solli, 2018). The results of the measurement are shown in Table 1.

Table 1. In-situ rock stress measurement data using HF conducted by SINTEF (Solli, 2018).

Chainage	498	504	510	550	556	562	600	606
Minimum principal stress S_3 (MPa)	2.00	2.10	2.50	2.40	2.70	2.70	2.80	3.20
Internal water pressure Pw (MPa)	2.34	2.33	2.32	2.26	2.25	2.25	2.19	2.18
Safety Factor (S_3/P_w)	0.86	0.90	1.08	1.06	1.20	1.20	1.28	1.47

Table 1 clearly indicates that the minimum principal stresses (S_3) were lower than the induced water pressure (Pw) along the tunnel stretch between the concrete plug and chainage 504m. It is noted here that several new cracks were identified during the inspection of this tunnel segment.

3 NUMERICAL MODELING AND DISCUSSIONS

A FLAC3D numerical model is used to simulate the in-situ stress state at Bjørnstokk project. The main aim is to evaluate the influence of 3D topography as well as the two shear planes shown in Figure 2. The constitutive equations derived for an isotropic linear elastic model are used to exhibit the behavior of the modeled rock mass. Table 2 shows the laboratory test results of intact rock strength (σ_{ci}), elasticity modulus (E_{ci}), unit weight (γ), and Poisson's ratio (ν). As suggested by Panthi (2006), the values of rock mass strength (σ_{cm}) and rock mass deformation modulus (E_{rm}) are calculated using Equation 1 & 2 respectively.

$$\sigma_{cm} = \frac{{\sigma_{ci}}^{1.6}}{60} \quad \& \quad E_{rm} = E_{ci} * \left(\frac{\sigma_{cm}}{\sigma_{ci}}\right) \qquad (1) \; \& \; (2)$$

Table 2. Properties of intact rock and rock mass and interface parameters for numerical modelling.

Parameters	σ_{ci} (MPa)	E_{ci} (GPa)	ν	γ (kN/m3)	σ_{cm} (MPa)	E_{rm} (GPa)	k_n (GPa/m)	k_s (GPa/m)	Friction Angle (^{0})
Values	130	61	0.3	26.7	40	19	188	72.5	27

The shear planes (SP1 & SP2) are modeled as planes of weakness and considered as an interface between the same rock formations. The normal stiffness (k_n), shear stiffness (k_s), and frictional angle are the input parameters for simulating interface planes in the FLAC3D. The stiffness can be estimated if values of Young's modulus (E_0) and the thickness (t) of the shear plane are known (Basnet & Panthi, 2019; Li et al., 2009). Young's modulus of the material at the shear plane is considered equal to the deformation modulus of the surrounding rock mass (E_{rm}). The joint stiffness are estimated using Equation 3 & 4.

$$k_n = \frac{E_0}{t} \quad \& \quad k_s = \frac{E_0}{2(1+\nu)t} \qquad (3)\ \&\ (4)$$

3.1 Simulation Process

The initial process is the simulation of the actual topography which is made using the digital elevation model data of Tosbotn area. A block representing the topography of Bjørnstokk project area of size approximately 2160 m x 1600 m with an additional depth of 2000 m below the sea level is made (Figure 3). The bottom of the model is fixed at -2000 masl so that the boundary conditions at the base do not affect the stress assessment along the pressure tunnel alignment. The shear planes that cross the pressure tunnel are also incorporated in the model as interfaces (SP1 & SP2).

After the geometry is defined, 3D tetrahedral volume grids of different sizes are created where the grid size are finer nearby the surface topography, location of the unlined pressure tunnel, and the shear planes. After the discretization of the block, rock mass parameters are assigned along with the boundary conditions. The boundary faces of north, south, east, west, and bottom are prevented from normal displacement by fixing corresponding velocities to zero values. The surface topography is set free to generate the gravity loading in the area.

Once the 3D Model is ready with above-mentioned steps, it is run to initialize the gravity-induced vertical and horizontal stresses. The model is then converged to the equilibrium, and the total horizontal stresses in both x and y direction (tectonic stresses) are iterated to match the magnitude of the measured minimum principal stress. After calibrating the model, the results of the minimum principal stress generated in the model are ready for assessment.

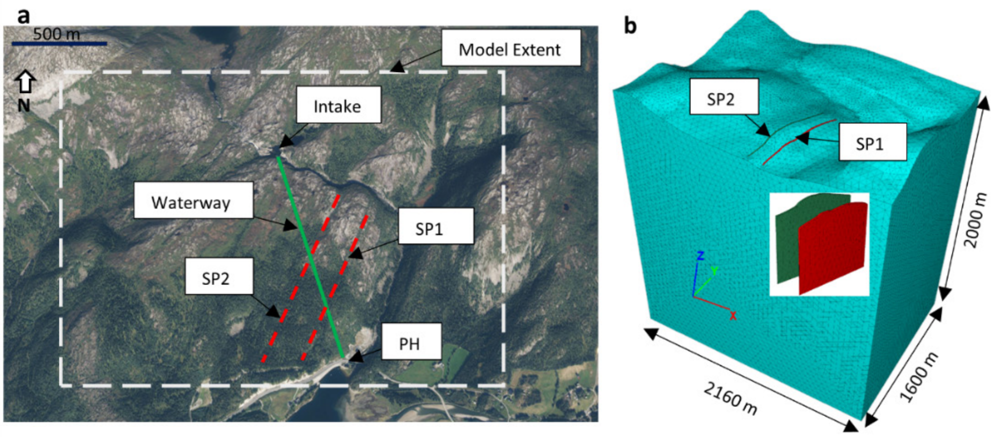

Figure 3. a) Model Extent over Bjørnstokk area; b) 3D geometry with topography, grids, and shear planes.

3.2 Analysis results and discussions

The model is run with two scenarios, i.e., with and without shear planes to evaluate the magnitude of minimum principal stress (S_3). The model results indicated that the magnitude of minimum principal stresses are significantly influenced by the presence of shear planes SP1 & SP2 (Figure 4).

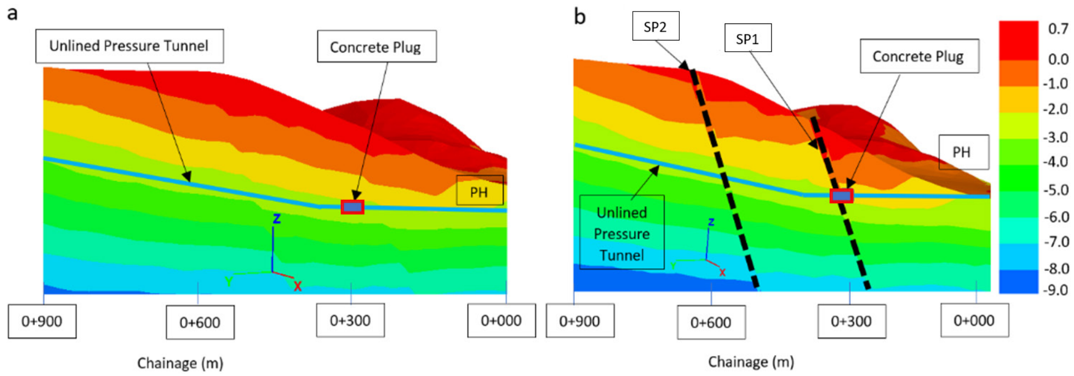

Figure 4. Minimum principal stress (MPa) simulated along the tunnel: a) without and b) with shear planes.

The minor principal stress (S_3) at the concrete plug area (chainage: 290-310) ranges between 1 to 2 MPa which indicates considerable influence by the shear planes in the stress magnitude (Figure 4b) as compared to the magnitude of stresses without shear planes (Figure 4a). It is noted here that the magnitude of minimum principal stresses (S_3) are much lower values than the magnitude of induced water pressure (Pw) which is about 2.6 MPa (Figure 5). This means that the stretch of unlined pressure tunnel near the concrete plug area clearly had the possibility of hydraulic fracturing in the intact rock mass and followed by hydraulic jacking along the shear planes leading to water leakage.

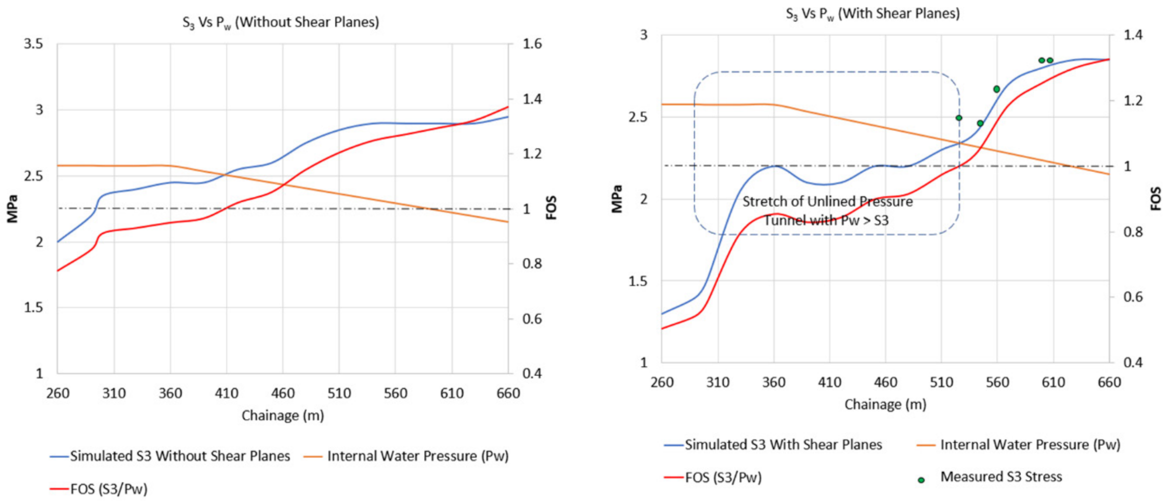

Figure 5. Minimum Principal Stress vs Water Pressure along tunnel alignment with and without shear planes.

Figure 5 further indicates that the factor of safety (FOS) represented by the ratio of minimum principal stress and the static water pressure (S_3/P_w) along the unlined pressure tunnel from chainage 290 to chainage 530 is below 1 indicating the possibility for hydraulic failure. This finding well matches with the inspection made along the unlined pressure tunnel in autumn 2017 where many new cracks were observed along the same chainages of pressure tunnel. This substantiates that the numerical modelling provides possibility to simulate in-situ stress state of an area where the unlined pressure tunnel could be located.

4 CONCLUSION

The hydraulic failure incident in the unlined pressure tunnel of Bjørnstokk project shows that it is equally important to assess the in-situ stress state considering topographic, geological, and geotectonic conditions. The stress measurement data from the project as well as the numerical model results indicate that the minimum principal stress is lower than the induced water pressure in the downstream stretch of the pressure tunnel which was not considered during the placement of the concrete plug. In addition, the numerical modeling results demonstrates that the conditions such as irregular topography and stress perturbations due to the presence of local shear planes can develop complex in-situ stress states in a rock mass. Such complex in-situ stress state can only be estimated at greater reliability if all possible geological and geotectonic conditions that may influence the state of stress are identified and incorporated into the 3D numerical model.

ACKNOWLEDGEMENT

The manuscript is written based on the data published in the MSc thesis written by MSc student Snorre Andersen Solli with the supervision of Professor Krishna Kanta Panthi who is the second author of this manuscript and was also involved in the inspection of this tunnel after the incident of hydraulic failure. The authors acknowledge Helgeland Kraft for arranging the visit to the site for the inspection that took place in September 2017.

REFERENCES

Basnet C. B. and Panthi K. K. 2018. Analysis of unlined pressure shafts and tunnels of selected Norwegian hydropower projects. Journal of Rock Mechanics and Geotechnical Engineering. Vol. 10, pp 486-512.

Basnet C. B. and Panthi K. K. 2019. Evaluation on the Minimum Principal Stress State and Potential Hydraulic Jacking from the Shotcrete-Lined Pressure Tunnel: A Case from Nepal. Rock Mechanics and Rock Engineering, vol. 52, pp. 2377–2399.

Bergh-Christensen J., Dannevig N. T. 1971. Engineering geological evaluations of the unlined pressure shaft at the Mauranger hydropower plant. Technical report. Oslo, Norway: Geoteam A/S

Broch E. 1982. The development of unlined pressure shafts and tunnels in Norway. ISRM International Symposium. International Society for Rock Mechanics

Hudson J. A., Harrison J. P. 2000. Engineering rock mechanics: an introduction to the principles. Elsevier

Li G., Mizuta Y., Ishida T., Li H., Nakama S., Sato T. 2009. Stress field determination from local stress measurements by numerical modelling. Int J Rock Mech Min Sci 46(1):138–147

Panthi K. K. 2006. Analysis of engineering geological uncertainties related to tunneling in Himalayan rock mass conditions. Doctoral thesis at NTNU 2006:41, Norwegian University of Science and Technology (NTNU). ISBN 82-471-7825-7.

Panthi K. K. 2014. Norwegian design principle for high pressure tunnels and shafts: it’s applicability in the Himalaya. Hydro Nepal: J Water Energy Environ 14:36–40

Selmer-Olsen R. 1969. Experience with unlined pressure shafts in Norway, Proc. Int. Symposium On Large Permanent Underground Openings, Oslo

Solli A. S. 2018. Evaluation on Hydraulic Splitting at Bjørnstok Hydropower Project. MSc. Thesis under Department of Geoscience and Petroleum, NTNU