Proceedings of the NROCK 2023
The IV Nordic Symposium on
Rock Mechanics and Rock Engineering
ISBN 978-9935-9436-2-0
DOI 10.33112/nrock2023.2

Engineering Geology in Hydropower Engineering

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ABSTRACT

Norway has more than 100 years of experience in the design and construction of hydropower plants consisting waterway systems that included unlined pressure tunnels and shafts. The waterway systems are in general very long and consist of unlined pressurized headrace tunnels, unlined high-pressure shafts, underground powerhouse caverns, access, and tailrace tunnels. The maximum static head that the unlined pressure tunnel has reached is 1047 meter, which is equivalent to almost 10.5 MPa. This is a world record, and it is obvious that the rock mass in the periphery of unlined pressure tunnels and shafts experience high hydrostatic pressure exerted by the flowing water discharge. Experienced gained from the construction and operation of these unlined pressure tunnels and shafts were the key to develop design criteria and stability assessment principles by giving focus on engineering geology, rock mass quality and geo-tectonic environment. As a result, these criteria and principals have got worldwide acceptance. However, the success of these criteria depends on the engineering geological and geo-tectonic environment prevailing in the are of concern and the operational regime adopted in the hydropower plants. This key-not lecture reviews some of the first attempts of the use of unlined pressure tunnels and shafts concept, highlights major failure cases, discusses the gradual development of design criteria for the unlined pressure tunnels and shafts and highlights recent operational trends that have direct influence on the stability of unlined pressure tunnels and shafts of hydropower plants.

KEYWORDS

Hydropower; Hard rock mass; Water pressure; Design criteria; Operation

1. INTRODUCTION

A typical layout of the hydropower schemes in Norway before 1920s consisted horizontal headrace tunnel, steel penstock along the surface topography and powerhouse on the bottom of the valley. Early 1920s attempts were made to build underground pressure shafts (both steel-lined and unlined) and underground powerhouse. The first such hydropower scheme with underground powerhouse was built in the year 1916. However, emphasis was given to keep all waterway system and powerhouses inside the mountain mainly after the completion of World War II. Today, according to Panthi and Broch (2022), Norway has over 200 underground powerhouse caverns and over 4300 km hydropower tunnels. Experiences gained through the design, construction and operation of hydropower schemes has made it possible to apply unlined high-pressure tunnels and shafts concept due to the favorable engineering geological and geo-tectonic conditions that persist in the Scandinavia. It is estimated that over 95% of the waterway length of Norwegian hydropower schemes are left unlined (Panthi, 2014; Panthi and Broch, 2022).

The success history of the development of hydropower schemes with unlined pressure tunnels and shafts in Norway used to be almost 99 percent with very little stability problems until the de-regulation of power market in early 1990s. However, after the de-regulation of power market the waterway systems are facing new operational challenges (Neupane et al., 2020). This key-note lecture highlights about the developed design criteria that consider geology, rock mass and in-situ stress. In addition, challenges associated to topography, geo-tectonic conditions, and changes in operation after de-regulation of power market are highlighted.

2. BRIEF HISTORY OF DEVELOPMENT

In Norway, the use of unlined pressure tunnel and shaft in hydropower projects started early 1920 (Vogt, 1922). Four projects were implemented around this time. Three out of these four projects had problems during initial water filling and these problems were solved by extending the penstock pipe and by carrying out extensive grouting. All four projects were designed with low pressure headrace tunnel, unlined inclined pressure shafts, and horizontal penstock tunnel as waterway system connecting the powerhouse located at surface (Figure 1a). Although three out of four hydropower schemes with unlined pressure shafts were operating perfectly after some initial problems were fixed, it took almost 40 years to beat the world record of static water head of 152 m with unlined high-pressure shaft of Svelgen hydropower project (Figure 2). The Tafjord K3 hydropower project with a static head of 286 m was the one to beat this record, which was successfully put into operation in 1958 (Broch, 1982). After the construction of this project the hydropower industry in Norway had a new confidence in the application of unlined pressure tunnels and shafts concept. The general layout design used for the design of hydropower schemes after Tafjord K3 is shown in Figure 1b. This type of design uses very limited length of steel lining near the powerhouse (mostly not exceeding 75m) in order to avoid the leakage from unlined pressure shaft to the underground powerhouse cavern. In areas where topography restricted the use of unlined high pressure shaft all the way from near powerhouse cavern to downstream end of headrace tunnel, an layout arrangement consisting steel lined lower pressure shaft and part of the horizontal pressure tunnel downstream of unlined upper pressure shaft and unlined headrace tunnel (Figure 1c) become common hydropower design solutions after around 1960.

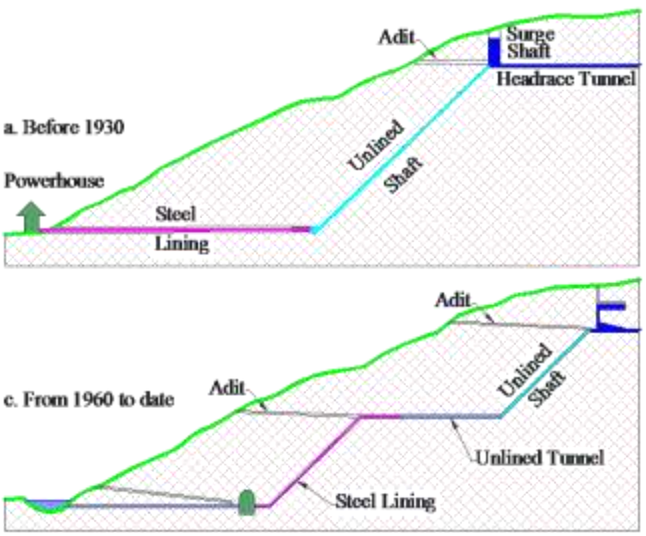

Figure 1. Layout design history of hydropower projects in Norway (Panthi and Basnet, 2016)

Until the beginning of 1970s, all the hydropower schemes consisted of the vented surge chamber to dampen the water hammer and oscillation waves (upsurge waves) produced due to sudden stoppage of turbines or operational changes in the turbine units. However, at Driva hydropower project which came in operation in 1973 had a very steep topography which restricted to build access road to intermediate adit and top of vented surge shaft. As a result, an Air Cushion Surge Chamber with a solution as indicated in Figure 1d was implemented (Selmer-Olsen, 1974; Panthi and Broch, 2022). Today, Norway has 10 hydropower schemes where Air Cushion Surge Cambers are used to control the water hammer and oscillation waves generated in the headrace system due to sudden changes in the operation mode of the plant.

The benefit of this solution is that a hydropower scheme can avoid an inclined or vertical shaft. Instead, a long unlined high-pressure headrace tunnel may connect the intake directly with underground powerhouse through a very short steel penstock shaft near the powerhouse. At present, Norway has many unlined pressure tunnels and shafts of varying static heads with maximum static water head of 1047m at Nye Tyin hydropower project, which came in operation in 2004 (Figure 2).

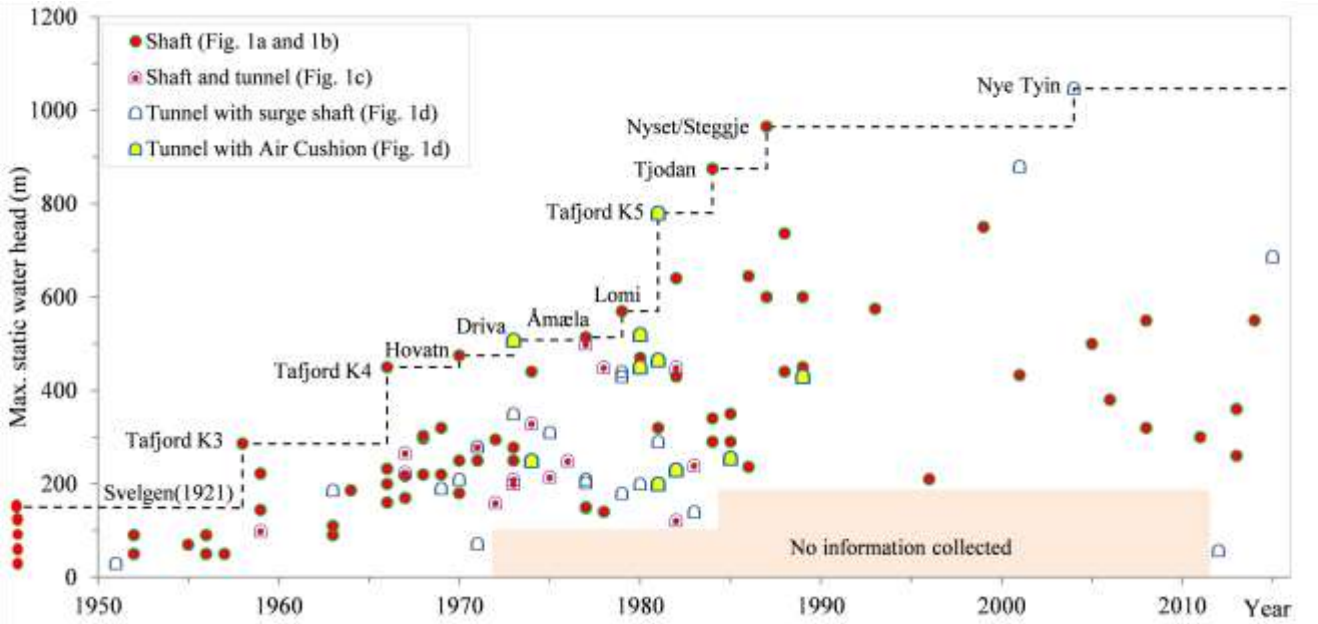

Figure 2. The static head variation of Norwegian unlined pressure tunnels and shafts over time. The figure is an updated version from Broch (2013) (Panthi and Basnet, 2016)

Most of the unlined pressure tunnels and shafts have been and are being successfully operated with no long-term instability problems excluding few exceptions, which were the basis for the development of design principles and criteria. Even though, all unlined high-pressure tunnels and shafts follow the developed design principles and criteria, there are some cases of failures even in modern time where further investigations were needed with substantial mitigation measures applied after the first water filling. Some of the major failure cases of tunnels and shafts with rock type and construction completion year are presented in Table 1.

Table 1: Failure of unlined pressure shafts and tunnels in Norway (Broch, 1982; Selmer-Olsen, 1985; Solli, 2018)

Project	Year	Waterhead (m)	Rock types	Cross-section Area (m^2)	Failure condition
Herlandsfoss	1919	136	Mica-schist	8.0 (Tunnel)	Hydraulic fracturing
Skar	1920	129	Gneiss-granite	Tunnel	Completely failed
Svelgen	1921	152	Sandstone	4.5 (Shaft)	Minor leakage
Byrte	1968	303	Granite Gneiss	6.0 (Shaft)	Hydraulic jacking
Åskåra	1970	210	Sandstone	9.0 (Tunnel)	Hydraulic jacking
Bjerka	1971	72	Gneiss	10.0 (Tunnel)	Leakage
Holsbru	2012	63	Dark Gneiss	18.0 (Tunnel)	Leakage
Bjørnstokk	2017	264	Granodiorite/granite	Tunnel and shaft	Hydraulic fracturing

As shown in Table 1 the first failure case was at Herlandfoss where the unlined pressure tunnel was partly failed, and considerable leakage occurred, and steel lining was further extended as a final solution. At Skar the waterway system was mostly failed due to very low rock cover. At Svelgen, the leakage was observed during the first filling of the pressure shaft. At Bryte the unlined pressure shaft failed due to hydraulic jacking through unfavorably oriented fracture system and faults. Similar was the case at the unlined pressure tunnel of Åskåra hydropower project. At Bjerka, leakage occurred through the pressure tunnel. A recent case of leakage through the pressure tunnel is at Holsbru hydropower project, which came in operation in 2012. The recent case of hydraulic fracturing was at Bjørnstokk, which came in operation in 2017. Most of these cases experienced failure at first water filling and the remedial measures were taken to bring these projects to operation.

3. BASIC DESIGN PRINCIPALS

The design of underground structures for the hydropower projects should be made in such a way that the design provides cost effective, long-term stable and sustainable solution. This can be achieved by considering rock mass as the part of a structural element that counteracts any load or pressure exerted by either unloaded rock mass or hydrostatic water head acting during operation (Edvardsson and Broch, 2002). In addition, combination of tunnel rock support consisting of rock bolts and sprayed concrete applied during construction to achieve safe working environment should be considered as part of the permanent support. It is, however, emphasized here that the sprayed concrete (shotcrete) is a permeable material and hence does not restrict water to penetrate to the rock mass (Panthi and Basnet, 2017). Thus, any design should make sure that there is no possibility of hydraulic fracturing/jacking that may cause water leakage out from the waterway system and constructed powerhouse and transformer caverns are long-term stable. Any design considerations should be based on the results from comprehensive engineering geological investigations. The aim of the design should be to avoid stability and long-term functionality of the underground structure in consideration (Panthi and Broch, 2022).

3.1. Placement of unlined pressure tunnels and shafts

The success history of the implementation and operation of unlined pressure tunnels and shafts in Norway is very good example of the capacity of rock mass that is capable of self- supporting. As shown in Figure 2, the unlined pressure tunnels and shafts built in Norway have varying static heads with maximum water head of 1047 m at Nye Tyin hydropower project. Over 99 percent of unlined pressure tunnels and shafts have been successfully operated with no noticeable long-term instability problems until the de-regulation of power market that took place in early 1990s. The Norwegian experience of development of unlined pressure tunnels and shafts gave good basis for location design of waterway system of hydropower plants and are famously recognized by the world as Norwegian Confinement Criteria (NCC). Equation 1 and Equation 2 are the two criteria that are related to vertical and lateral rock covers (Figure 3-left). The understanding is that both vertical and lateral rock covers should confine the pressure given by the static water head against hydraulic fracturing at any location of the pressure tunnel and shaft (Panthi and Broch, 2022).

$$h > \frac{P_w}{\gamma_r \times \cos\alpha} \qquad (1)$$

$$L > \frac{P_w}{\gamma_r \times \cos\beta} \qquad (2)$$

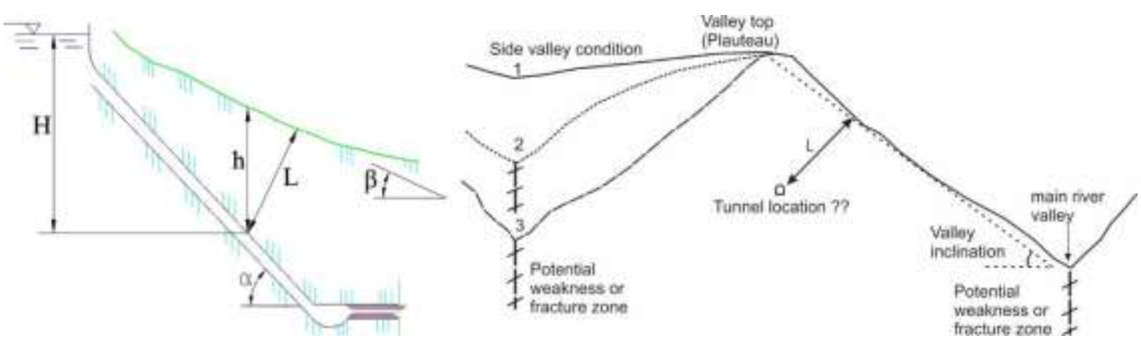

Figure 3. Idealized topography with geometrical parameters used in Norwegian confinement criteria (left) and different topographic conditions that may prevail in a hydropower scheme (right).

In Equation 1 and 2 and Figure 3, *h* is the vertical rock cover, H is the hydrostatic head acting in the tunnel or shaft, γ_w is the specific weight of water, γ_r is the specific weight of the rock, and α is the inclination of shaft / tunnel with respect to horizontal plane, L is the shortest distance from valley side slope topography to the tunnel location and β is the angle of valley side slope with respect to horizontal plane.

In general, the start of highest static pressure point and remaining upward (towards intake) alignment of an unlined headrace system assessed using Equation 1 and Equation 2 provide good result against hydraulic fracturing (jacking) for a topography representing almost no existence of secondary valley (side valley condition 1 in Figure 3-right). However, if the topography consists of more than one deep valleys like shown in Figure 3-right, the location assessment made using these two equations may not provide needed safety margin against hydraulic failure. Therefore, it is important to assess the magnitude of minimum principal stress (σ_3) along the pressurized headrace system which should always be more than the hydrostatic water head (Equation 3).

$$\sigma_3 > P_w \qquad (3)$$

The confinement criteria developed in Norway are mainly for tunnels and shafts that are mostly unlined excluding areas with weakness zones lined with in-situ concrete. Similarly, these criteria are equally relevant for tunnels lined with sprayed concrete (shotcrete) since sprayed concrete is a permeable support and almost equal water pressure will act on the rock mass as that on the sprayed concrete.

In addition, in relatively unjointed and massive rock mass as in Norway, it is important that the pressure tunnels and shafts should be placed in such a way that minimum instability challenges associated to induced stresses are met. The use of proper assessments methods is therefore essential for a meaningful instability assessment of rock burst / rock spalling condition in tunnels. Figure 4 should be used as preliminary basis to locate pressure tunnels and shafts so that rock spalling or rock burst (strain burst) along the alignment are minimized.

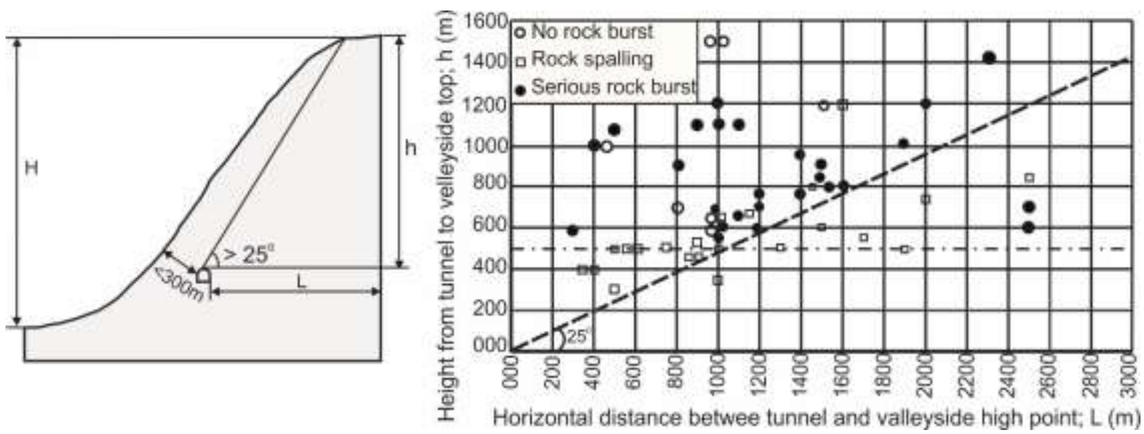

Figure 4. Location of tunnels and shafts with respect to topographic conditions (left), and a plot of rock burst / spalling in relation to height (h) from tunnel to top of valley-side and horizontal distance from tunnel to the top of valley side (L) (Panthi, 2018).

As indicated in Figure 4, most of the tunnels that had vertical height (h) between tunnel and plateau less than 500 meters and angle between tunnel location and plateau less than 25 degrees did not experienced rock burst / rock spalling. The tunnels that had exceeded this threshold were met stability challenges associated to rock burst / rock spalling. However, exceptions are made for the vertical shafts, the white circles located above the separation line in Figure 4-right.

3.2. Leakage assessment from unlined pressure tunnels and shafts

In addition to the placement design of the unlined headrace system, an assessment on the potential water leakage from the headrace system should be carried out. In general, the permeability of rock mass is governed by discontinuities and their engineering geological characteristics. Hence, among the most important aspect of unlined or shotcrete lined headrace system is to control water leakage while the system is in operation at full hydrostatic pressure so that the water leakage is within an acceptable limit boundary which should be less than 1.5 liters per minute per meter tunnel (Panthi, 2006). In an unlined or shotcrete lined pressure tunnel, water gives pressure (Pw) to the rock mass equivalent to the hydrostatic water head (H) as indicated in Figure 5.

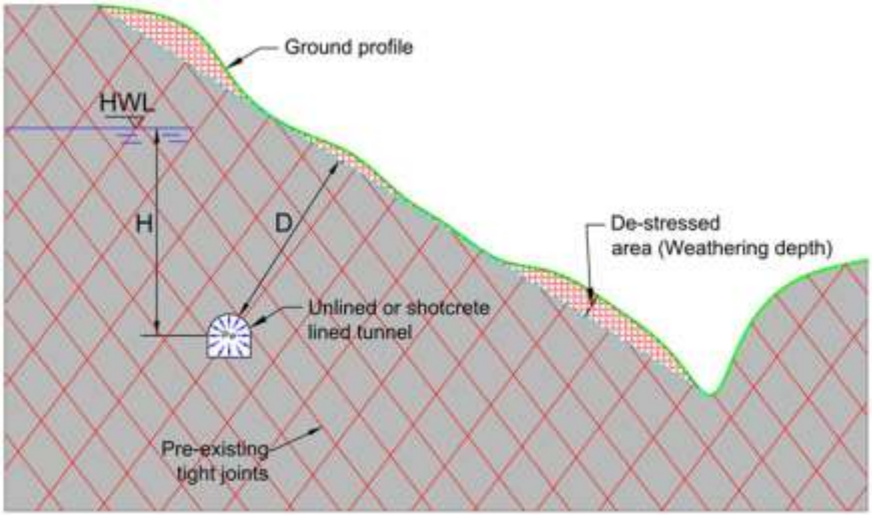

Figure 5. Typical topographic condition surrounding an unlined pressure tunnel / shaft (Panthi and Basnet, 2021).

As shown in Figure 5, due to presence of joints and discontinuities, the rock mass behaves differently when it is exposed to water pressure. The leakage potential through an unlined pressurized headrace system is therefore governed by degree of jointing in the rock mass and condition within different joint sets such as joint aperture, joint infilling conditions, spacing of the must unfavorable joint set and joint persistence. In addition, hydrostatic water head and shortest distance from the waterway to the topographic slope surface are very crucial to be assessed. Equation 4 proposed by Panthi (2006) may be used to estimate specific leakage (q_t) from an unlined or shotcrete lined pressurized headrace system. In Equation 4, H is the hydrostatic water head (Figure 4), J_n is joint set number, J_r is joint roughness number and J_a joint alteration number as described by Barton et al (1974) in the Q-system of rock mass classification. The joint permeability factor (f_a) given in Equation 4 can be estimated using Equation 5 as recommended by Panthi and Basnet (2021).

$$q_t = f_a \times H \times \frac{J_n \times J_r}{J_a} \qquad (4)$$

$$f_a = \mathcal{L} \times \frac{J_p}{D \times J_s} \qquad (5)$$

In Equations 4 and 5, f_a is a joint permeability factor with unit l/min/m^2 which may vary between 0.001 and 0.25 and is related to joint spacing (J_s) and joint persistence (J_p) measured in meters, shortest distance from tunnel to surface topography of the valley side slope (D) and $\mathcal{L}$ which is equivalent to 1 lugeon (1 l/min/m).

3.3. Shape and size of unlined pressure tunnels and shafts

The extent of frictional head-loss of a headrace tunnel or shaft depends on the shape and size. TBM excavated tunnels and shafts are circular in shape and have smooth wall surface and are hydraulically ideal in shape. However, it is not always feasible to use TBM as an excavation method for these tunnels and shafts since success of TBM application is largely dependent on the length of the tunnel and shaft to be excavated (Panthi, 2015). In general, drill and blast method of excavation is preferred construction method due to its flexibility in making quick engineering decisions if unforeseen geological conditions arise. However, the tunnels and shafts excavated using drill and blast method have undulated surface of varying smoothness due to overbreak caused by blasting and presence of fractured rock mass. In addition, shape and size of an unlined pressure tunnel / shaft will be determined mostly by construction requirements and easiness. The most practical tunnel shapes excavated using drill and blast method are inverted D or horseshoe shaped. The profile of excavated tunnels and shafts may either be unlined / shotcrete lined or concrete / steel lined depending on the rock mass and in-situ stress condition. The excavated surface of the tunnel walls will have undulation which depends on the quality of rock mass and proficiency of the contractor involved in the construction.

The optimum hydraulic shape of a water tunnel occurs when wall height of a tunnel is between 1 to 1.3 times the radius of the tunnel curvature above spring level. Rougher the surface, pronounced will be the flow resistance due to large undulations. Following Lysne et al (2003) and Basnet and Panthi (2018), frictional headloss (Equation 6 and 7) can be calculated using coefficient of resistance called hydraulic roughness represented by either friction factor (f) or manning coefficient (M_R) and calculated by Equation 8 and Equation 9 which largely dependent on both surface roughness (ε_R), the Reynolds number (R) and the hydraulic radius (R_h) of a tunnel / shaft which is a function of area (A) and perimeter (P) (Equation 10). In Norway, it is normal to keep water velocity in an unlined or shotcrete lined pressure tunnels between 1 to 2 m/sec.

$$H_f = \frac{f\,L\,v^2}{2g(4R_h)} \qquad (6)$$

$$H_f = \frac{L\,v^2}{M^2 {R_h}^{4/3}} \qquad (7)$$

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon_R}{14.8\,R_h} + \frac{2.51}{R\sqrt{f}}\right) \qquad (8)$$

$$M_R = \frac{24}{{\varepsilon_R}^{1/5}} \qquad (9)$$

$$R_h = \frac{A}{P} \qquad (10)$$

In addition, singular losses that usually are formed entrance loss, trash rack loss, gate loss, bend loss, transition loss, niches loss, rock trap loss, exit loss etc. should be considered. For detail on calculation methods one can read Basnet and Panthi (2018). These singular losses cannot be avoided but could be minimized. It is therefore important to optimize the size of a tunnel taking consideration on the shape and size governing the frictional headloss and overall construction cost.

3.4. Shape and size of underground caverns

The size of an underground cavern is optimized based on the desired functional need. The shape of the cavern is designed as such that it achieves evenly distributed stresses along the whole periphery (roof and walls). The evenly distributed stress condition can be achieved by giving the cavern a simple shape as possible with an arched roof and with limited protruding corners. If a cavern roof is designed with a protruding corner which many do so to accommodate the space for crane beam, there is a chance that the cracks are developed in the corners between the transition of wall and arched roof. Such design may reduce stability considerably and the failure may extend further down to the cavern walls. It is emphasized here that the in-situ stress measurements should be carried out so that the magnitude and direction of the stresses are know. A comprehensive stability assessment should be carried out to ascertain that there is no serious stability problem that may cause serious damage to both walls and roof of the cavern.

4. OPERATION OF UNLINED PRESSURE TUNNELS AND SHAFTS IN HARD ROCKS

Norway has almost half of the reservoir capacity in Europe and thus has a great potential for providing the much-needed flexibility for the European power market in the future. After de-regulation of the power market in early 1990s, power price volatility has increased considerably which is intensifying in past 20 years. As a result, the operation of hydropower plants in Norway is becoming very dynamic. Operating the existing and new power plants with dynamic operational regime confronts with various technical challenges and operational risks. The Norwegian Research Centre for Hydropower Technology (HydroCen) is conducting research in several areas to assess such technical challenges and provide sustainable solutions to meet the future flexibility requirements in Norwegian hydropower system. The scope of research ranges from long-term stability of underground structures (especially the unlined pressure tunnels and shafts), electrical and mechanical systems, environmental impacts, and market conditions (Neupane et al, 2021). The assessment of the production data of some Norwegian hydropower plants (Figure 6) indicated that the dynamic operational regime looks dramatic in most of the hydropower plants which has direct influence on the long-term stability of unlined pressure tunnels and shafts.

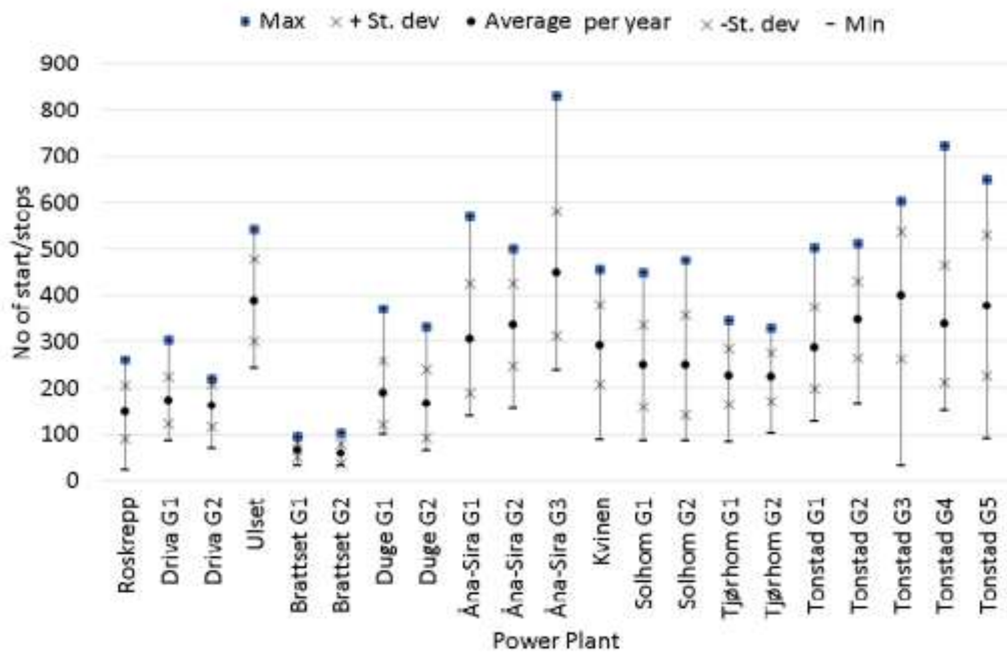

Figure 6. Statistical values of start/stops of some hydropower plants in Norway (Neupane et al, 2021).

As seen in Figure 6 the starts/stops (operational change) sequences of some hydropower plants show a clear distinction between hydropower plants with or without operational restrictions. Both average values and standard deviation are much smaller for hydropower plants with operational restrictions. The lowest number of starts/stops among all power plants is 65 per year per unit for Brattset. All other powerplants experience an average of 200 to 400 starts/stops sequences in annual average. Since over 95 percent of pressure tunnels and shafts of Norwegian hydropower plants are unlined, water is in direct contact with the rock mass and the pressure transients resulting from operational changes has direct impact on the discontinuities in the rock mass, which in long-term are causing block falls because of cyclic fatigue due to frequent pressure pulsations (Figure 7).

Figure 7. Examples of block falls and collapses witnessed in pressure tunnels and shafts of Norwegian HPP.

The analysis carried out by Neupane et al (2020) for two-year long real-time monitored data from the unlined headrace tunnel of Roskrepp hydropower plant in Southern Norway indicates that there occurs time lag between the water pressure in the tunnel and pore-water pressure in the rock mass deep into the tunnel wall (Figure 8).

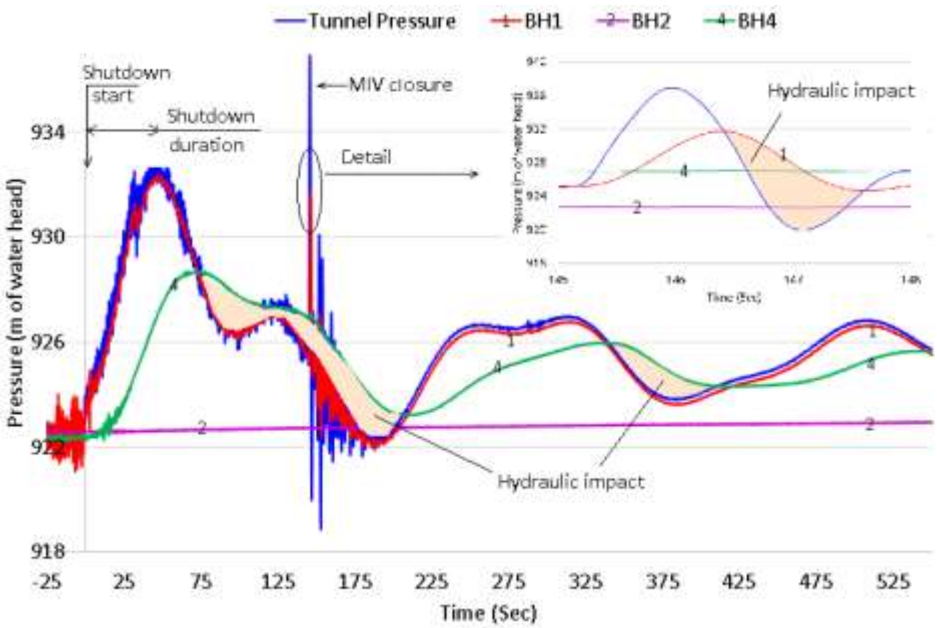

Figure 8. Tunnel pressure transient with pore pressure responses from boreholes (Neupane et al, 2021)

Figure 8 shows a pressure transient in the headrace tunnel and the rock mass pore pressure during a typical shutdown event at Roskrepp hydropower plant. The rock mass pore pressure measured in three boreholes, along with the hydraulic impact during the transient are shown in the figure. Both water hammer and mass oscillation are recorded by the pressure sensor because the measurement is done at a location between the turbine and the surge shaft. As seen in Figure 8, the boreholes which intersect the conductive joints in the rock mass i.e., BH1 and BH4 strongly respond to pressure transients whereas other boreholes are non-responsive indicating that there is no direct hydraulic contact. As one can see, BH1 registers a stronger response to pressure transients but there is very little time-lag during mass oscillation, resulting in very little to zero hydraulic impact during mass oscillation and significant hydraulic impact during water hammer. On the other hand, BH4 shows a clear time-lag during both mass oscillation and water hammer. But the amplitude of pore pressure in BH4 in response to the water hammer is smaller as compared to BH1. This difference in the response is due to different resistance to the flow through joints in the rock mass, which is a function of void geometry of joints and the length of flow path i.e., joint length between tunnel wall and its intersection points with individual boreholes. The distance between tunnel wall and boreholes (length for flow path) at BH1 and BH4 are 2.3 m and 8 m, respectively.

From a theoretical point of view, it can be said that the hydraulic impact on the joints in the rock mass depends on the magnitude of change of discharge during shutdown and the duration of shutdown event. These two parameters govern the nature of transient pressure pulses which travel into the joint wall surface in the rock mass causing additional forces. Another important parameter is the static pressure before transient which governs the resistance to flow through joints during transients. The joint hydraulic aperture is influenced by the effective stress across joints. During the operation of a power plant, the effective stress across the joints can vary depending on reservoir levels, which may change the initial hydraulic aperture before transients. Such changes of transient and changes in pore water pressure in the rock joints will cause a fatigue over the long period of the operation of hydropower plants causing a new crack in the rock mass leading to block failure as shown in Figure 7.

5. OPERATION OF PRESURE TUNNELS IN SWELLING ROCKS

It is important to note here that the rock mass in the periphery of hydropower water tunnels is unloaded and drained during tunnel construction and then the tunnel is exposed to cyclic wetting and drying processes during the operational lifetime of the project. If the pressure tunnels are aligned through weak and weathered rocks of sedimentary and volcanic origin such as flysch, volcanic sediments, andesites; and the pressure tunnel is shotcrete lined; there is a risk of collapse due to swelling of rocks (Figure 9). This is because, the interaction between rock mass and flowing waters through pressure tunnels may cause swelling of weak and weathered rocks of sedimentary and volcanic origin.

Figure 9. Collapsed tunnels passing through weak and weathered rocks of volcanic and sedimentary origin (left and center) and a cored highly weathered and weak flysch rock (right).

It is emphasized that the surrounding rock mass in pressure tunnels supported with shotcrete lining comes in direct contact with water which may lead to time-dependent deformation caused by both swelling and squeezing causing instability in tunnels as indicated in Figure 9. Hence, proper stability assessment and support measures are applied in pressure tunnels passing through swelling rocks. To do so, the swelling potential of intact rocks are first assessed by conducting mineralogical test to identify swelling clay minerals such as montmorillonite, anhydrite, zeolite etc. The next step will then be to carry out swelling pressure tests in intact rock samples in repeated cycles of drying and wetting (Selen et al, 2021). An example of such tests is shown in Figure 10.

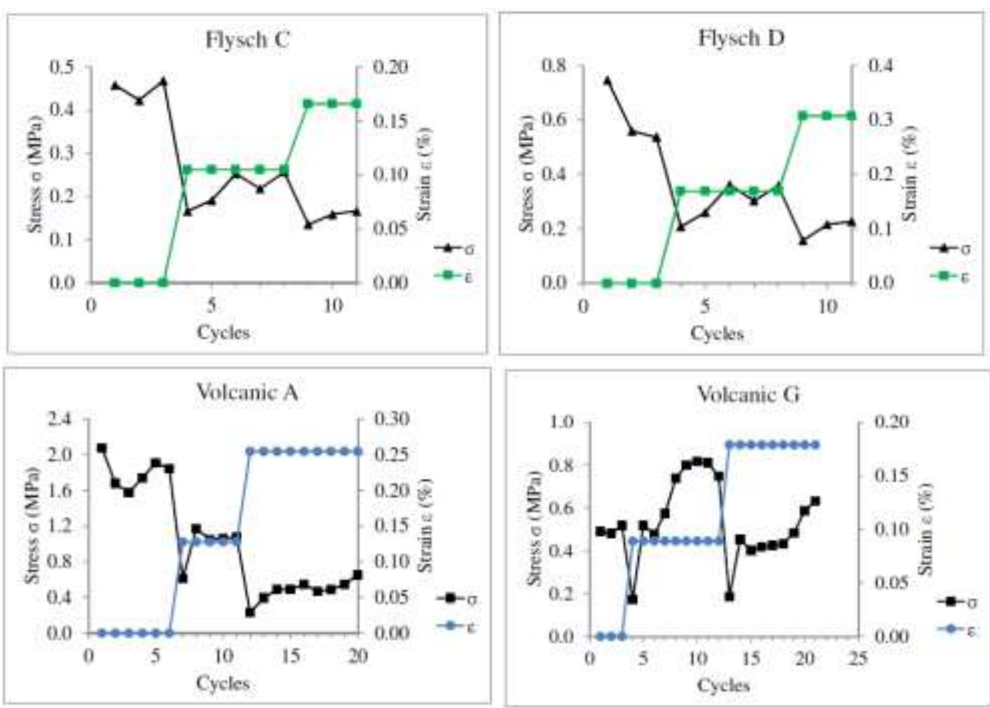

Figure 10. Cyclic swelling pressure development of intact rocks under controlled deformation in oedometer

It is noted here that extensive moisture fluctuations are special features of pressure tunnels of hydropower plants compared to tunnels built for infrastructure projects. The recent rend of the development of wind and solar power which is dependent on the wind intensity and day light conditions, respectively, the hydropower plants functions as energy balancing agents and are seldom operated to their base load. This changed scenario causes fluctuation in the operation regime of the power plants. To determine the effect of moisture fluctuations on the swelling behavior of weathered rocks surrounding pressure tunnels, repeated wetting and drying cycles of swelling tests should be performed on intact rock samples.

6. CONCLUSIONS

The experience gained from the construction and operation of Norwegian unlined pressure tunnels and shafts helped to develop design criterion and stability assessment principles focusing on engineering geology and rock mass quality. These design criteria and principals have got worldwide acceptance. As have been highlighted in this manuscript, the success of these criteria and principals depends on the engineering geological, geo-tectonic and topographic environment prevailing at selected locations where the hydropower plants to be built. As have also been demonstrated, recent operational trends of hydropower plants with more frequent start stop sequences have caused more dynamic load due to pore water fluctuation in the rock mass which is resulting to both long-term fatigue in hard rock mass and plastic deformation in weak and weathered rock mass of sedimentary and volcanic origin due to swelling of rock mass.

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