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Underground Hydropower Plants

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00607. Underground hydropower plants

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Synopsis:

Most of the hydropower plants developed in a mountainous topography consist different underground structures such as headrace tunnels, pressure tunnels and shafts, powerhouse caverns, tailrace, and access tunnels. Design of underground structures in a cost effective and optimum way is a challenge and demands step wise investigations on the engineering geological condition of the area where these structures will be located. Orientating, shaping, and sizing plays an important role for the overall stability of each structures regarding both hydraulic requirements and long-term stability viewpoint. This manuscript briefly discusses on development history of underground hydropower plants in Norway, discusses steps of engineering geological investigations, highlights potential stability problems an underground opening may experience and elaborates on the design aspects practiced.

1 Introduction

Historically, the electricity generation technology was developed by the beginning of 1880s which helped to accelerate industrial growth in the world (Hveding, 1992). The modern era of use of artificial intelligence would not have been possible without the development of electrical energy. With development of electricity generation technology, possible sources that could be used to generate electricity were extensively explored in the world. Hydropower became among the most popular renewable sources that is being exploited to generate electricity along the river valleys. The development of hydropower started becoming popular by the beginning of 1900 and took momentum especially after the first world war.

Generation of hydropower requires head difference between two points so that the prevailing potential energy is converted to the electrical energy. Meaning, a typical hydropower plant consists of a dam in a river from where the water is diverted and discharged (flown) along the headrace system utilized to bring water to the powerhouse located somewhere in the downstream and then the water is discharged to the tail reservoir or river using tailrace system. Most of the hydropower projects built in the mountainous area that have installed capacity over 5 MW use underground space one or another way. This is due to the fact that the hydropower plants that utilize underground space help protect the surrounding environment and are more sustainable solutions because the development of hydropower plants using underground space minimizes excavation at the surface topography which helps to reduces potential geohazards to avoid impacts on the environment.

Well planned investigations and evaluation of stability conditions are key issues of the planning, design and construction activities of the hydropower projects consisting underground caverns, pressure shafts (both inclined and vertical), medium to high pressure headrace tunnels, tailrace, and access tunnels (Figure 1). The cost and construction time optimization play important role for the sustainable and economically favorable hydropower projects. Use of underground space mostly helps to achieve these goals. Hence, an optimum design involves the utilization of innovative thinking and positive attitude that accepts certain level of unforeseen geological risks.

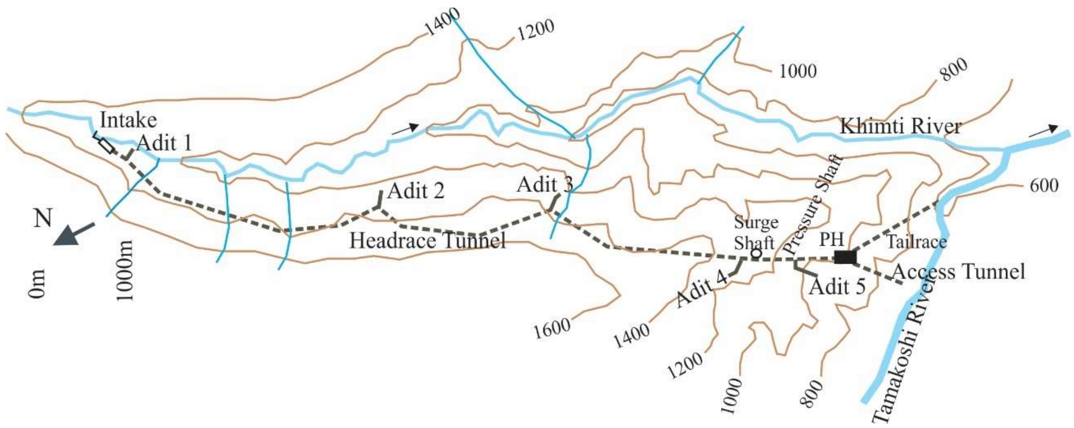

Figure 1. Topographic plan showing different underground elements of Khimti I hydropower project. The project that has 8 km long headrace tunnel, was designed using Norwegian design concept construction completed in 2001.

An example of the successful use of underground space for hydropower development is Norway where more than 200 underground powerhouse caverns and over 4300 km tunnels as waterways were successfully built. During the peak period of hydropower development in Norway (between 1960 and 1990) more than 100 km of hydropower tunnels were excavated every year (Broch, 2016a).

1.1 Underground structures in hydropower projects

As shown in Figure 1, the connection between the dam and tail reservoir/river in general involves extensive use of underground space such as construction of headrace tunnel, surge shaft, high pressure shafts (inclined or vertical), powerhouse cavern, tailrace tunnel, access tunnels at different location and diversion tunnel at the dam/reservoir area. The placement of these underground structures demands favorable conditions regarding geology, the geo-tectonic and overall rock mass conditions and are key factors in the design (Panthi, 2014).

In the Figure 2-left, a typical layout example of an underground powerhouse area of a hydropower plant consisting different underground elements are shown. The figure is schematic which shows a simplified plan and cross section of a medium hydropower plant with single turbine unit. It is emphasized here that most of the Norwegian hydropower plants (medium and large) have similar layouts with a water head varying from 200 to 600 m. The figure is to some extent self-explanatory. A critical point for the location of the powerhouse will normally be where the unlined pressure tunnel / shaft ends, and the steel lining starts. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures.

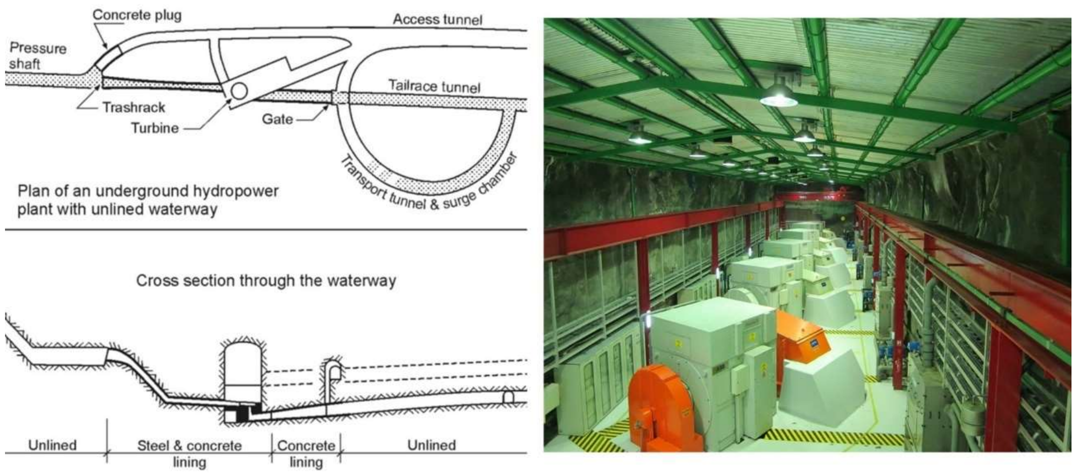

Figure 2. Typical arrangement of different underground elements at powerhouse area (left) and underground powerhouse cavern of 60 MW Khimti I hydropower project in Nepal (right).

The Figure 2-right shows a construction completed underground powerhouse for 60 MW Khimti I hydropower project for better visualization. The power plant has five Pelton turbine units having installed capacity of 12 MW for each unit.

1.2 Stages of development on underground solutions

The use of almost horizontal headrace tunnel and exposed steel penstock along the surface topography all the way down to surface powerhouse located at the bottom of the valley was the start of utilization of underground space in hydropower project (Figure 3-top). This type of design solution is still in use around the world. Especially this is the case for small hydropower projects with an installed capacity less than 25 MW. However, in Norway, during and shortly after the World War I there was a shortage of steel which lead to the shortage of steel making uncertain delivery and high prices. With the lack of steel for a penstock, the obvious alternative was to try to bring the water as close to the powerhouse as possible through a pressure tunnel or a shaft (Figure 3) resulting the introduction of unlined pressure tunnels and shafts already as early as in 1916 (Broch, 1984a). After World War II, however, emphasis was given to move most of the hydropower elements including powerhouses inside the mountain citing security threat. Most of the powerhouses, pressure tunnels and shafts have been successfully designed and operated. It is emphasized here that more than 96 percent of a total annual production of 136 TWh of electric energy in Norway is generated from hydropower. The success history of the operation of unlined pressure shafts and tunnels in Norway is almost 99 percent with very little stability problems along the waterway system excluding some exceptions where problems were registered during the initial phase of the development of unlined concepts (Panthi and Basnet, 2016).

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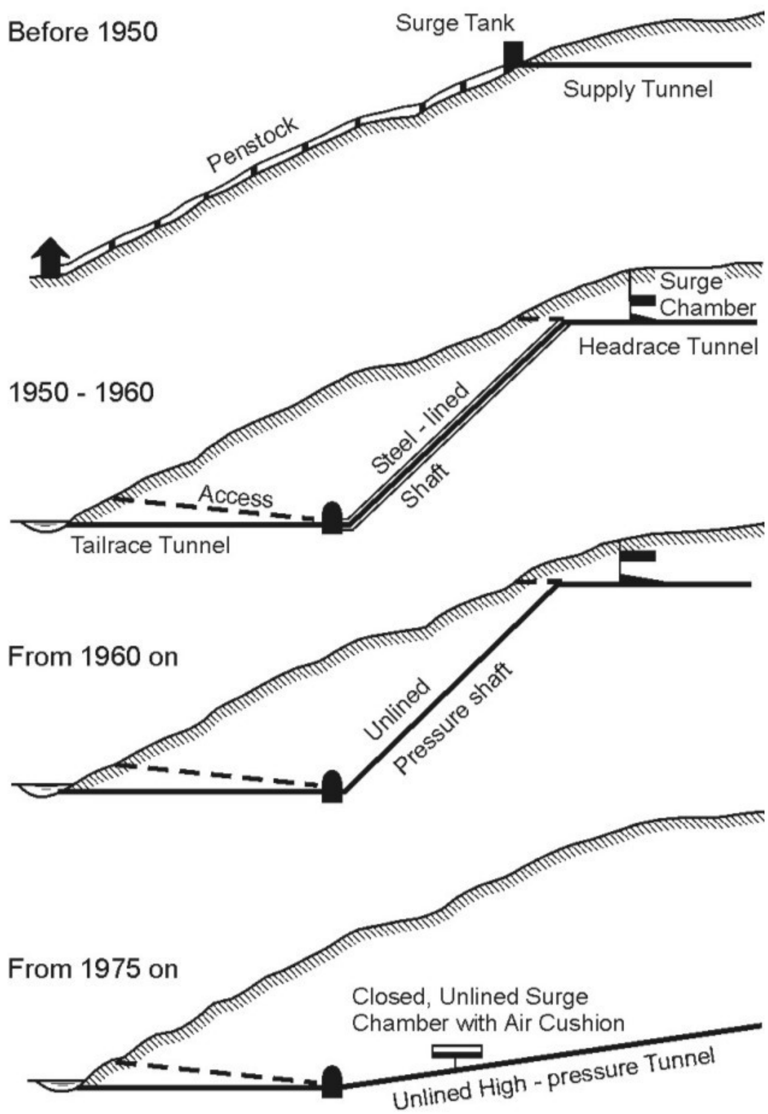

Figure 3. The development of general layout of hydropower plants in Norway (Broch, 2016a)

Through the design, construction, and operation of all these tunnels and underground powerhouses in Norway, valuable experience was gained. The experience gained through design and excavation of underground space for hydropower scheme made it possible to develop design methods and advanced tunneling technology in connection with both excavation and support philosophy. Innovative way of thinking and implementation in the design and construction continued since the beginning. As Figure 3 indicates, one of the Norwegian innovation is the use of unlined high-pressure tunnels and shafts as waterway system and another is the development of so called air cushion surge chambers that helped give alternative solution against conventional vented surge chamber in certain topographically difficult hydropower schemes (Broch, 1984a and Panthi, 2014).

This experience has been of great importance for the general development of tunneling technology, and not least for the use of the underground. Many underground powerhouses excavated in rock mass of varying quality are to a large extent the forerunners for the varied use of rock caverns which are all around the world today (Edvardsson & Broch, 2002). An example of an underground powerhouse from the early 1950s is shown in Figure 4-left. In this case a concrete building has been constructed inside a rock cavern. The powerhouse has in fact false windows to give people a feeling of being above ground rather than underground. Later people became more confident in working and staying underground, and powerhouses were constructed with exposed rock walls, often illuminated to show the beauty of the rock mass such as demonstrated by the powerhouse commissioned around 1970 as shown in Figure 4-right (Broch, 2016a).

Figure 4: The Aura underground hydropower station, commissioned in 1952 (left) and Tafjord K-5 underground powerhouse station commissioned around 1970s (right)

Some special techniques and design concepts have over the years been developed by the Norwegian hydropower industry. Most of the Norwegian hydropower tunnels have only 2 – 4 percent concrete lining. Only in a few cases has it been necessary to increase from this threshold. The low percentage of lining is due not only to favorable tunneling conditions but also a support philosophy which accepts some falling rocks during the operation period of a waterway system. A reasonable number of rock fragments spread out along the headrace or tailrace tunnel floor will not disturb the operation of the hydropower station if a rock trap is located at the downstream end of the headrace tunnel (Broch, 2016b). Serious collapses or local blockages of the tunnel must, of course, be prevented using a proper and adequate tunnel rock support consisting either sprayed concrete or in-situ concrete lining.

The experience gained in the design, construction and operation of waterway system and underground powerhouse helped to exploit innovative and cost-effective solutions (Panthi and Basnet, 2016). Figure 5 below shows a gradual innovation in the design of hydropower schemes in Norway consisting different underground solutions. The design solutions shown in Figure 5 are equally practiced around the world. Some examples of use of unlined pressure tunnels are Chivor and Gauvio projects in Columbia (Broch, 1984b), Lower Kihansi hydropower Project in Tanzania (Marwa, 2004), Las Lajas and other projects in Chile (Palmstrom and Broch, 2017), Venda Nova II and Venda Nova III (Lamas et. al, 2014) and in Nepal (Panthi and Basnet, 2017).

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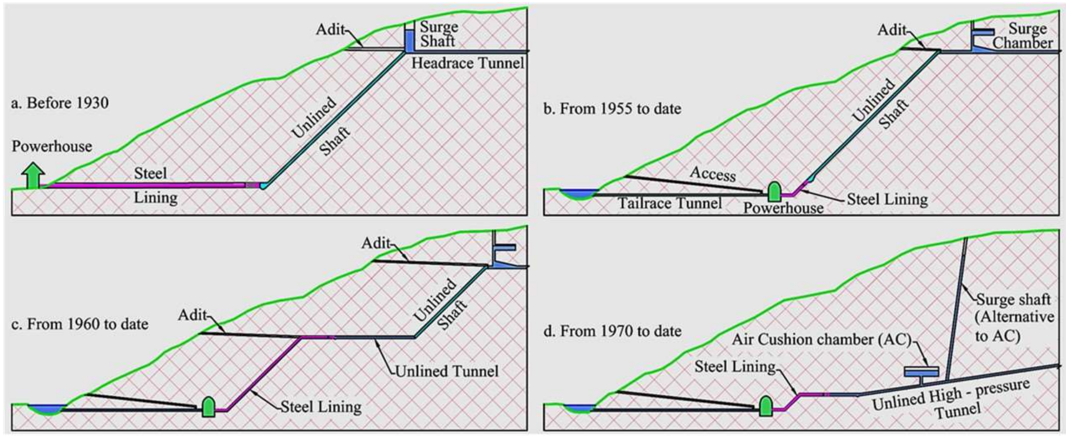

Figure 5: Development steps of underground solutions in Norway since 1920 (Panthi and Basnet, 2016)

In areas where topography restricted on the use of unlined high pressure shaft all the way from near underground powerhouse, a layout arrangement consisting steel lined pressure shafts and unlined/shotcrete lined headrace tunnel have been successfully practiced. In general, a hydropower scheme consists over 70 percent of its civil structures underground consisting elements. A proper planning and design are hence crucial.

2 Engineering geological investigations

It is a well-known fact that the rock mass is a heterogeneous medium usually made up of an interlocking matrix of discrete blocks. The blocks are generally separated by the sets of discontinuities such as bedding planes, foliation planes and other systematic or random joints and faults oriented at different directions. The discontinuities in the rock mass are subjected to lateral movement and shearing caused by tectonic or other mechanical course of actions occurred during their geological lifetime (Panthi, 2006). Such movements always cause alteration and weathering to the rock mass to varying degrees and the contact surfaces between the blocks may vary from very clean, fresh, and rough to clay filled, smooth and slickenside (Hoek, 1998). Thus, the mechanical characteristics of rock mass are not uniform and vary greatly.

The rock mass quality predictions and stability analysis for the underground structures are based normally on very limited information established by surface and subsurface site explorations and laboratory testing. As a result, the degree of uncertainty and risk associated to quality of rock mass remain higher at planning phase. Hence, both rock quality knowledge and level of uncertainty are time and project stage dependent (Figure 6). The rock mass condition along the alignment of any hydropower project consisting different element of underground structures is decisive with respect to the development cost and time required to complete the construction work. To assess the economic viability, the rock mass quality along the waterway system should be examined and estimated quantitatively during the pre-construction phases. To do so an in-depth engineering geological investigation should be conducted at this early stage of the hydropower project. Because the rock mass is a complex material with many variable parameters.

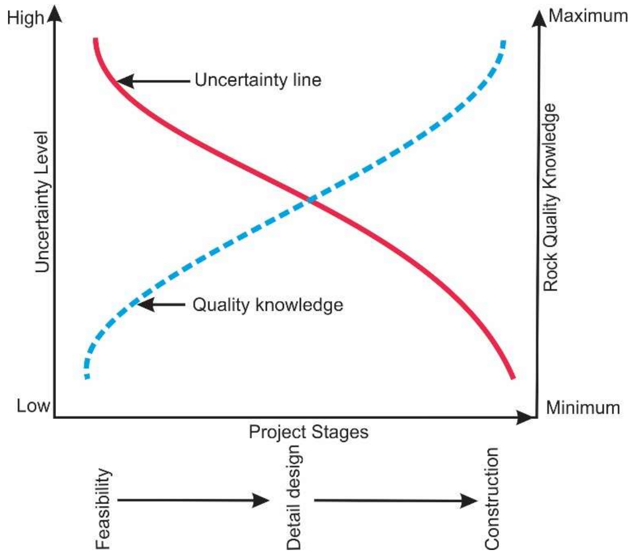

Figure 6: Schematic illustration of uncertainty level and rock quality knowledge at different project stages (Panthi, 2006)

However, it is generally expected that there will be some degree of deviations between predicted and actual rock mass conditions. This deviation should be within the acceptable limit so that excessive cost overruns and required construction time are controlled. The only way to control quality deviation is to carry out a well-planned and organized pre-construction phase engineering geological investigations. If the procedures have satisfactorily high quality, the final investigation results with desired limit of deviations are possible to obtain despite geological uncertainties and risk (Panthi, 2006). Though, the experience suggests that many tunnel projects suffer with varied and quite different ground conditions from what was anticipated during pre-construction phase planning and design. The rock mass quality variation results additional cost (Figure 7) and considerable delay in the project completions.

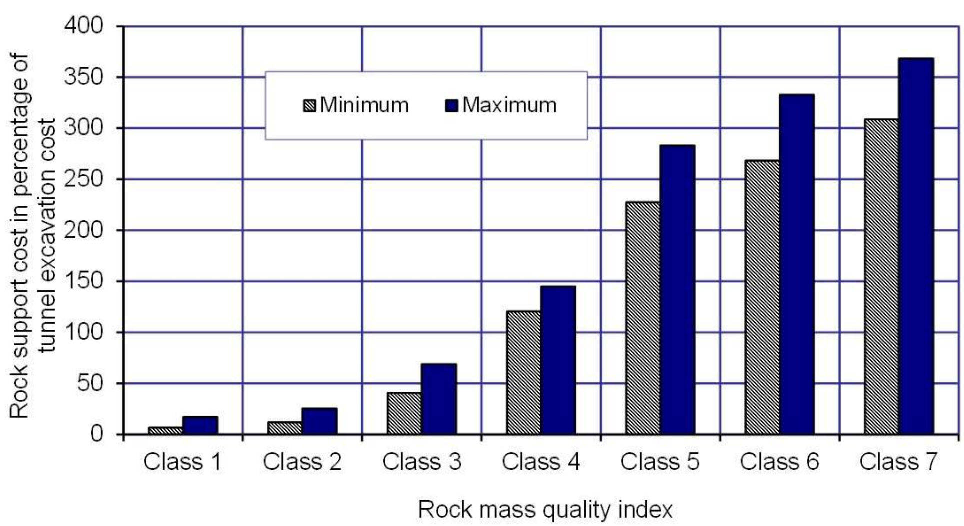

Figure 7: Approximate rock support cost in relation to tunnel excavation costs for different rock mass classes (Panthi and Nilsen, 2007)

It is not always an easy task to predict and estimate the rock mass conditions along the tunnel alignment accurately enough in advance so that variations can be kept within the acceptable limit. The quality variation from as has been in the contract documents may lead to the contractual dispute between the client and the contractor such as reported by Hencher (2019) at Glendoe hydropower project which may cause additional financial burden to the project.

The engineering geological investigations should always aim to acquire knowledge about the rock mass quality condition of the project in question so that an optimum, technically as well as economical viable planning and design of the hydropower project is achieved. Therefore, a well-thought and stage-wise (Figure 8) engineering geological investigation program consisting different investigation approaches should be made and followed.

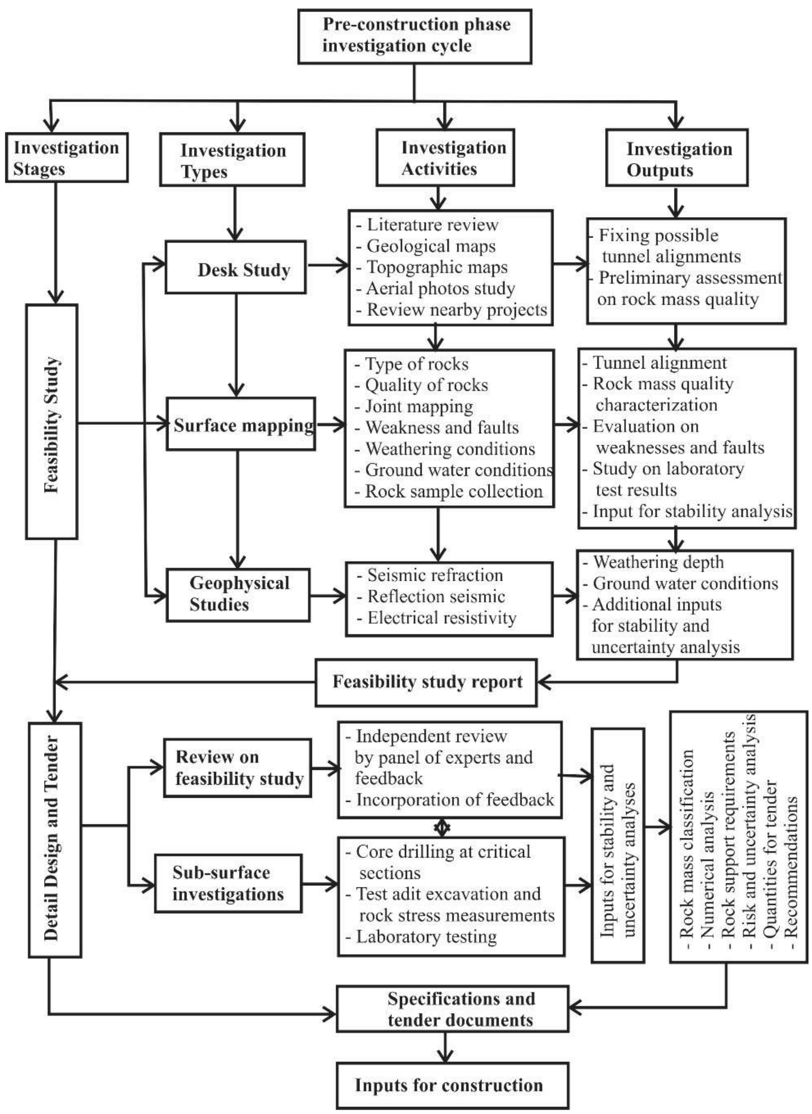

Figure 8: Recommended pre-construction phase engineering geological investigations for hydropower projects consisting underground elements (Panthi, 2006)

The program made for engineering geological investigation should be able to provide aimed investigation results (Figure 8) so that there is a very good basis to acquire background material to carry out calculation and analysis of various kinds which helps to make a proper decision on the investment viability (Panthi, 2021). The investigation must be oriented to the selection of methods addressing local geology and accessibility conditions, a well-defined purpose of investigation, stepwise investigation and continuity regarding execution and the extent of investigations according to geological complexity of the project area where hydropower project is situated.

The workflow of any hydropower project from planning to completion follows ground investigations, planning and detail design, preparation for tender documents and tendering, and finally development of the project.

3 Factors influencing instability

A rock mass is a heterogeneous medium which is characterized by two main features i.e., 1) rock mass quality and 2) the mechanical processes acting on the rock mass (Panthi, 2006). These two features are interlinked to each other and are not independent. The first one is related to rock mass strength, deformability properties, strength anisotropy, presence of discontinuities and weathering effect over the geological history. The second one on the other hand is linked to in-situ rock stress and groundwater conditions. The stability of tunnels and underground caverns is therefore a function of these two features. The stability is also influenced by project specific characteristics such as location, orientation, size, and shape of the underground structure. Hence, stability problems in an underground opening can be defined as inability to sustain failure after the excavation.

3.1 Rock mass quality

The geology, geo-tectonic, topographic environment as well as weather influence the overall rock mass quality condition in the vicinity of concern. The rock mass quality is mainly defined by rock mass strength and deformability properties. The stability assessment of an underground opening or a rock slope or a foundation is not possible without reliable estimates of the strength and deformation characteristics of rock mass (Hoek, 2007). Hence, the strength and deformability properties of both intact rock and rock mass are crucial basis to define failure mode and to assess stability of an underground cavern or tunnel.

The intact rock strength is established by carrying out laboratory test of the relatively small rock specimen following ISRM (1979) recommendation where a 50 mm diameter cored intact rock specimen with length 2.5 to 3 times the diameter is used. In general, the intact rock specimen contains very few or no discontinuities and is homogeneous, which in general results much stronger strength than the rock mass itself. The laboratory tested rock specimen therefore does not represent the strength and deformability properties of the rock mass. The strength and deformation modulus of the rock mass are significantly lower than the strength and young’s modulus of an intact rock specimen tested in the laboratory.

The rock mass strength and the rock mass deformation modulus are difficult to estimate directly in the field or by laboratory testing. Different scholars such as Bieniawaski (1989), Hoek et al (2002) and Barton (2002) have suggested empirical equations for the estimation of both rock mass strength and rock mass deformation modulus. All these empirical equations are linked to rock mass classification systems consisting Rock Mass Rating (RMR), Geological Strength Index (GSI) and Q-system, respectively. Since the rock mass classification systems are subjective tools (Palmstrom and Broch, 2006) estimated values differ from person to person based on their expertise, the use of rock mass classification in the estimation of rock mass strength and rock mass deformation modulus may not provide a real picture of the ground condition. Therefore, it is best that the estimation of rock mass strength and rock mass deformation modulus are directly linked with the laboratory tested results.

It is the known fact that the rock mass strength is defined as ability to withstand stress and deformation. Overall strength and deformation modulus of the rock mass are influenced by features such as intact rock strength, fractures or discontinuities in the rock mass, foliation or schistocity planes persisting in the rock mass, orientation of these features relatively to the direction in which the strength and deformation modulus are assessed and on the overall weathering condition of the rock mass. Strength anisotropy is common in many rocks mainly because of preferred orientations of mineral grains and directional stress history. Distinct anisotropy is very common for sedimentary and metamorphic rocks (Figure 9-left). Moreover, weathering influences significantly on intact rock strength and young's (elasticity) modulus of rocks (Figure 9-right). The weathering process in the rock mass starts from the fractures or discontinuities and migrates to the rock minerals through discoloration and staining, change in texture and fabric, disintegration, and decomposition.

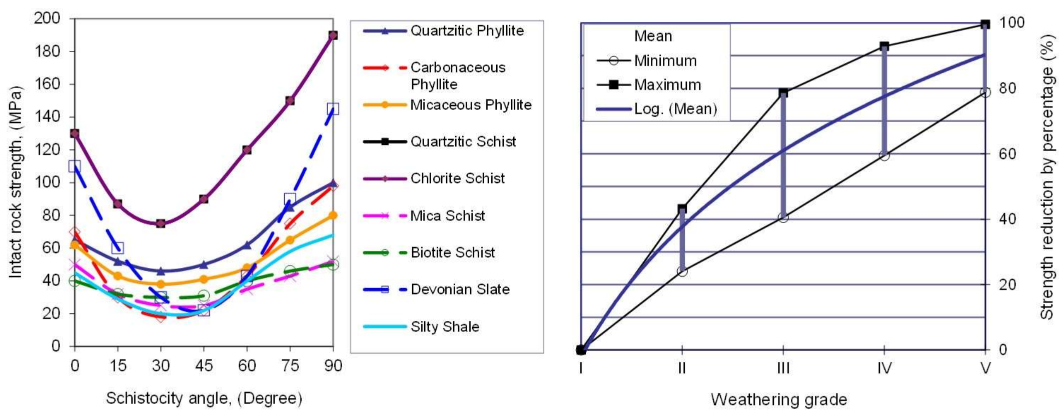

Figure 9. Variation of intact rock strength at varying schistocity angle (left) and reduction of intact rock strength with degree of weathering (Panthi, 2006). ISRM (1978) weathering grade: I = Fresh rock; II = Slightly weathered; III = Moderately weathered, IV = Highly weathered and V = Completely weathered.

Considering all of the influencing factors Panthi (2006 and 2018) suggested relations (Equation 1, 2 and 3) to estimate rock mass strength (σ_{cm}) and rock mass deformation modulus (E_{cm}) by directly linking to the laboratory tested intact rock strength (σ_{ci}) and young's modulus (E_{ci}) of the rock specimen following ISRM (1979) standard.

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.5}}{60} \qquad (1)$$ (Panthi, 2006) for foliated and schistose rock mass

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.6}}{60} \qquad (2)$$ (Panthi, 2018) for brittle and homogeneous rock mass

$$E_{cm} = E \times \left(\frac{\sigma_{cm}}{\sigma_{ci}}\right) \qquad (3)$$ (Panthi, 2006) for both brittle and foliated rock mass

3.2 In-situ rock stress

Unlike other materials used in engineering design, rock mass is preloaded by in-situ stresses. When an excavation is made in the rock mass, the in-situ stresses are redistributed and tangential stresses are induced in the vicinity of an underground opening (Hoek and Brown, 1980). The knowledge on the in-situ stress condition is essential for a meaningful assessment of the instability likely to be caused by induced stresses in tunnels and underground openings (Hudson and Harrison, 1997). The magnitude of principle stresses may be established either by in-situ stress measurement or by back calculation using numerical modeling or combination of both. In principle, if the rock mass strength is less than induced stresses, overstressing should occur in the periphery of an underground opening leading to stress induced instability. In relatively unjointed and massive strata, if the rock mass strength is less than the induced stresses, the instability may mainly be associated with rock spalling or rock burst (strain burst). Conversely, if the rock mass is weak, schistose, sheared, and thinly foliated / bedded, squeezing is the most likely scenario (Panthi, 2012a). The use of proper assessments methods is essential for a meaningful instability assessment in an underground opening.

3.2.1 Assessment on rock burst

Bothe empirical and semi-analytical methods are commonly used for the assessment of rock burst / rock spalling in an underground opening. Three most prominent and widely accepted rock burst assessment methods are Norwegian Rule of Thumb proposed by Selmer-Olsen (1965), Uniaxial Compressive Strength and Tensile Strength Approach described by Diederichs (2007) and Maximum Tangential Stress and Crack Initiation Strength Approach proposed by Martin and Christiansson (2009), which are be briefly presented below.

Most of the hydropower, road and railways tunnels built in Norway run through steep valley-side slopes where stress an-isotropy is a very common phenomenon. Hence, tunnels experiencing rock burst / rock spalling are quite common instability issues that are being faced while tunneling through hard and brittle rocks mass. In this respect, the knowledge associated to brittle failure in tunnels is not new in Norway (Panthi, 2018). Already in 1965, Professor Rolf Selmer-Olsen of Norwegian Institute of Technology (NTH) studied over 60 tunnels passing parallel with valley-side slope where rock burst and rock spalling were experienced during tunnel excavation (Selmer-Olsen, 1965). Most of these tunnels were passing through a topography where vertical rock cover directly above the tunnel were relatively small in comparison to the vertical height between tunnel and top of the valley-side slope, the plateau. In addition, most of these tunnels had relatively short distance (mostly not exceeding 300m) from the surface (Figure 10).

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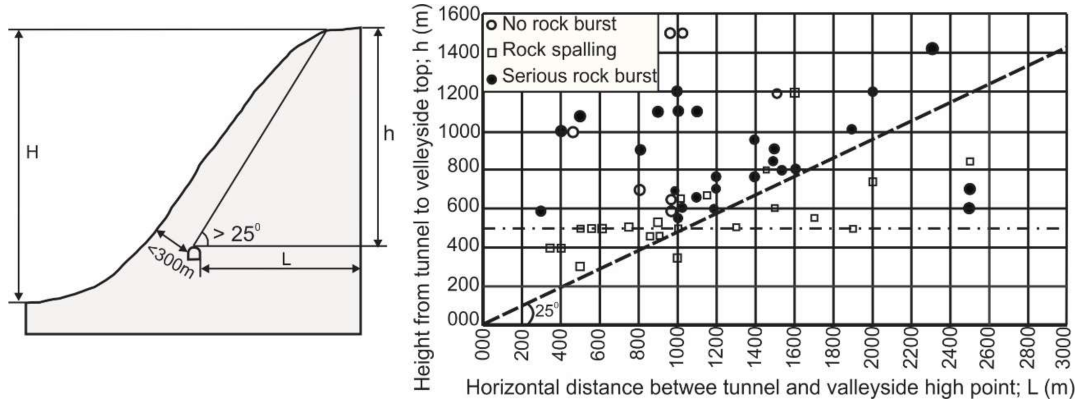

Figure 10. Horizontal distance from tunnel to valley-side top (left), and rock burst/spalling in relation to height from tunnel roof to top of valley-side, the plateau (right). Drawn based on Selmer-Olsen (1965).

The Figure 10 shows tunnels with no rock burst activity, medium rock burst (spalling) condition and high rock burst activity in relation with vertical height between the tunnel and top of valley-side slope (the plateau) and horizontal distance between tunnel and the top of valley-side slope. Selmer-Olsen (1965) concluded that 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 mostly did not experienced rock burst / rock spalling activities. However, tunnels that had exceeded this threshold mostly had stability problems associated with rock burst / rock spalling. However, exceptions are made for the vertical shafts, the white circles located above the separation line in Figure 10.

It is noted here that the Figure 10 shows results of rock spalling / rock burst in tunnels aligned parallel with the valley side slope where tunnels are located within 500 m distance from the slope topography. This rule of thumb can be used at early phase of planning and design to make ascertain that the tunnels and underground openings are placed at the best location possible. This early phase placement design may work as first check on whether there is a potential rock spalling / rock burst activity in tunnel or underground opening under consideration or not.

Similarly, Diederichs (2007) proposed a qualitative approach to the assessment where the author assumed that there is an influence of tensile strength on the rock burst / rock spalling in tunnels and underground openings. The author linked rock burst with uniaxial compressive strength (UCS) and tensile strength (T) of the intact rock (laboratory tested results). It is assumed that in the hard, strong, and brittle rocks under compression, the crack initiation (CI) occurs due to internal heterogeneities and strain an-isotropy and the crack initiation (CI) is strongly influenced by internal tensile strength (T) (Figure 11). It is emphasized here that the extension fracture may develop in the rock mass before the actual rock burst event by forming parallel and thin slabs in the tunnel periphery. As per Figure 9, higher the uniaxial compressive strength (UCS) of the rock material and higher the ratio between UCS and tensile strength (T), more violent and extensive will be the rock burst damage potential in the tunnel wall.

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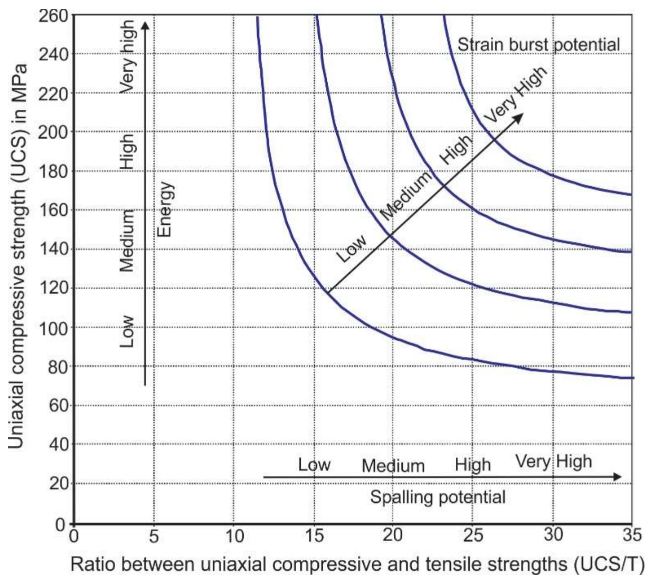

Figure 11. Diederichs (2007) qualitative approach for the assessment of rock spalling/rock burst based on compressive and tensile strengths of the rocks (Panthi, 2018).

Nevertheless, the major weakness of Diederichs (2007) approach is that it considers no in-situ stress condition that prevails in the rock mass. Hence, the assessment using this approach should be taken as indicative and in addition to this method other methods that include in-situ stress condition in the rock-mass in the assessment.

The two approaches discussed above provide qualitative assessment of the rock burst and therefore do not provide clear picture on the severity of the rock burst depth-impact (S_d) into the rock mass behind the tunnel wall as shown in Figure 12. The knowledge on the rock burst depth-impact (S_d) is crucial in making decision on the application of rock support (Panthi, 2012).

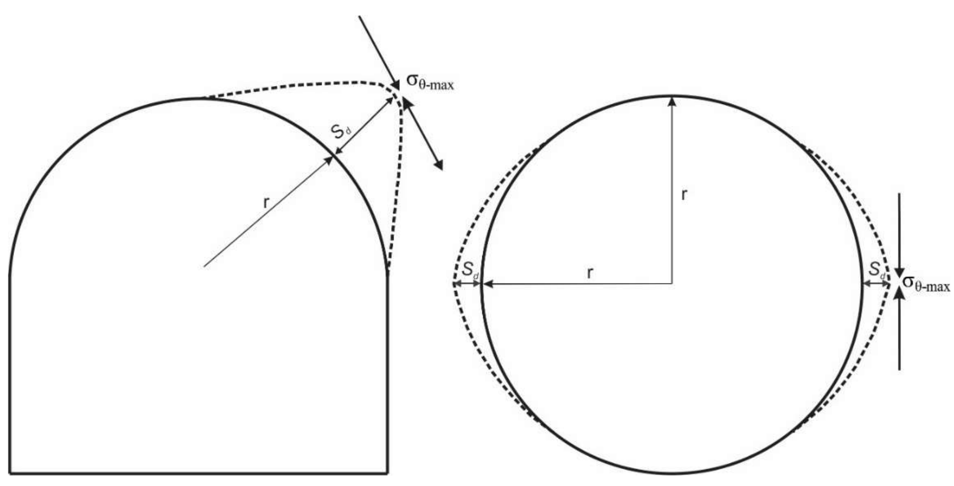

Figure 12. Both drill and blast and TBM excavated tunnels showing potential damage in the tunnel wall (depth-impact, S_d) due to induced tangential compressional stress.

This is particularly important while deciding the length and type of rock anchors and other support means such as mesh, rock straps and steel fiber reinforced shotcrete so that needed safety and long-term stability of an underground opening is well taken care off.

Martin and Christiansson (2009) proposed Equation 4 to assess the extent of rock burst / rock spalling depth-impact in the tunnel wall expressed by S_d which is linked to tunnel radius (r), maximum tangential compressional stress ($\sigma_{\theta\text{-max}}$) and rock mass spalling strength (σ_{sm}) which is equivalent to rock mass strength (σ_{cm}) that can be calculated using Equation 1 and Equation 2 (section 3.1) depending upon rock mass character (i.e. massive or schistose/foliated).

$$S_d = r \times \left[0.5 \times \frac{\sigma_{\theta-max}}{\sigma_{cm}} - 0.52\right] \qquad (4)$$

The rock burst / spalling depth-impact assessment using Equation 4 requires knowledge on both in-situ stress condition and rock mass strength of the area where planned tunnel will be located. In addition, method to calculate maximum tangential compressional stress ($\sigma_{\theta\text{-max}}$), which can be done using Kirsch's equation (Equation 5) defined by maximum principal stress (σ_1) and minimum principal stresses (σ_3), is needed.

$$\sigma_{\theta-\max} = 3\sigma_1 - \sigma_3 \qquad (5)$$

It is noted here that the rock mass strength (rock-mass spalling strength) for rocks with high degree of schistocity is mostly below 0.3 times the intact rock strength. Hence, Equation 1 is appropriate to be used for the rock mass influenced by schistosity. On the other hand, Equation 2 should be used to calculate rock mass strength (σ_{cm}) for the rock mass which is massive, homogeneous, brittle and has relatively high intact rock strength (σ_{ci}) (exceeding 150 MPa) since rock mass spalling strength lies between 0.3 to 0.45 of the intact rock strength (σ_{ci}).

3.2.2 Assessment on rock squeezing

When an underground opening is subjected to induced tangential stress, weak and schistose rock mass behave differently from the isotropic and stronger rock mass. Unloading caused by excavation, a visco-plastic zone of micro-fractured rock mass is formed deep into the tunnel wall if the rock mass is schistose (extent foliation/bedding) leading to a time dependent inward movement of the rock material. As a result, the applied support experiences gradual build-up of pressure. This time-dependent inward movement (plastic deformation) of the rock material towards the tunnel center (Figure 13) when subjected to tangential stress is defined as tunnel squeezing (Panthi, 2012b). Dealing with this type of stability problem is a serious issue experienced by the designer and tunnel construction crews during the excavation of tunnels and underground caverns for hydropower project. Coping tunnel squeezing demands good understanding on the behavior of weak and schistose rock mass so that a proper and well-planned strategy is made regarding stabilizing measures. This means, reliable prediction on the extent of squeezing is the key issue here. In the following two of such prediction approaches, i.e. Hoek and Marinos (2000) approach and Panthi and Shrestha (2018) approach, are presented.

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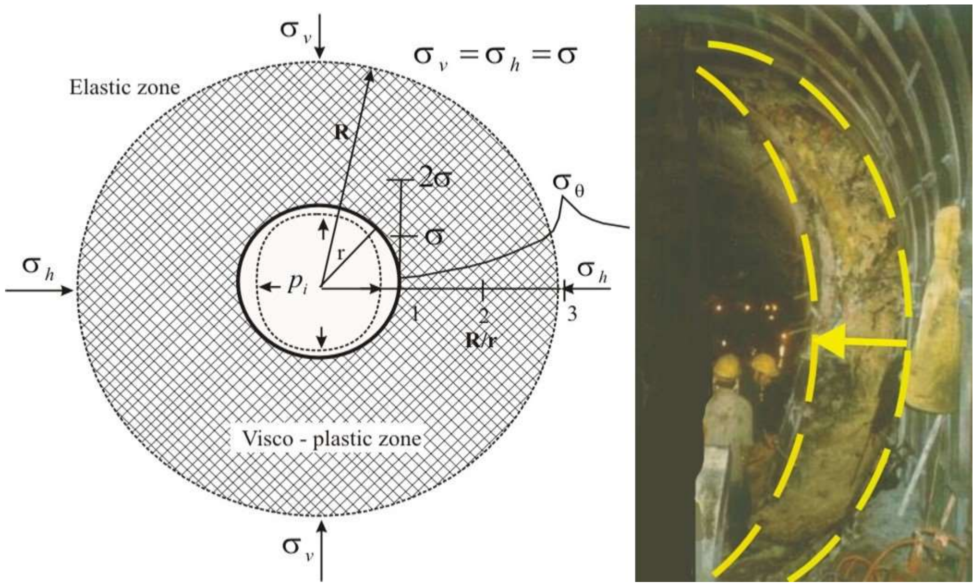

Figure 13. An illustration showing formation of visco-plastic zone and plastic deformation (squeezing) in a tunnel wall (Panthi, 2006).

An analytical approach of predicting tunnel squeezing was proposed by Hoek and Marinos (2000). The authors argue that the rock mass strength and the overburden stress (pressure) are the two key parameters for estimating plastic deformation (tunnel squeezing) of an underground opening and suggested a relationship that gives total tunnel strain (the ratio of deformation versus tunnel diameter). The proposed relationships are the function of the ratio between rock mass strength and in-situ overburden stress expressed by Equation 6 and Equation 7. The method was developed using CCM (Convergence Confinement Method) concept and Monte Carlo simulation approach and considers isostatic stress field and circular tunnel shape, which is not always the case in reality.

$$\varepsilon_t = 0.2 \times \left(\frac{\sigma_{cm}}{\sigma_v}\right)^{-2} \qquad (6)$$

$$\varepsilon_t = \left(0.2 - 0.25 \times \frac{p_i}{\sigma_v}\right) \times \left[\frac{\sigma_{cm}}{\sigma_v}\right]^{\left(2.4 \times \frac{p_i}{\sigma_v} - 2\right)} \qquad (7)$$

In Equation 6 and Equation 7, ε_t is tunnel strain in percentage, σ_v is overburden stress in MPa, σ_{cm} is rock mass strength in MPa which can be estimated using Equation 1 and p_i is support pressure experienced by applied support in MPa.

The authors assume that very weak rock mass is incapable of sustaining significant differential stress and that failure occurs before in-situ horizontal and vertical stresses have been equalized. This is the main reason for their consideration of overburden pressure instead of the tangential stress, which is always greater in magnitude than the overburden stress (Figure 13). However, it is noted that the in-situ principal stresses are in most occasion an-isotropic and tunnels are not always circular in shape excluding tunnels excavated using TBM.

Considering the constraint on tunnel shape and stress an-isotropy in the estimation of tunnel deformation, Panthi and Shrestha (2018) carried out comprehensive assessment on the long-term monitored and recorded deformation data of three headrace tunnels excavated using drill and blast method. All three tunnels pass through highly schistose, thinly foliated and weak rock formations. The authors found out that the total deformation in a tunnel periphery varies greatly and is an-isotropic in magnitude and distribution confirming significant influence of stress an-isotropy in the total tunnel wall deformation. Their comprehensive assessment resulted Equation 8 and Equation 9 which can be used to estimate both instantaneous and final tunnel wall deformation (tunnel strain). The authors argue that the rock mass shear modulus (G) is more appropriate parameter to be linked for squeezing analysis of highly schistose, thinly foliated/laminated and weak rock mass. In addition, the proposed Equations include the gravity (overburden) stress (σ_v), stress ratio (k) and support pressures (p_i).

$$\varepsilon_{IC} = 3065\left(\frac{\sigma_v(1+k)/2}{2G(1+p_i)}\right)^{2.13} \qquad (8)$$

$$\varepsilon_{FC} = 4509\left(\frac{\sigma_v(1+k)/2}{2G(1+p_i)}\right)^{2.09} \qquad (9)$$

Shrestha (2014) developed a simplified chart which incorporates ratio of rock mass shear modulus (G) and overburden stress (σ_v), and stress ratio (k) for different support pressures (Figure 12), which can be used to estimate required support pressure (p_i) for Equations 6 to 9.

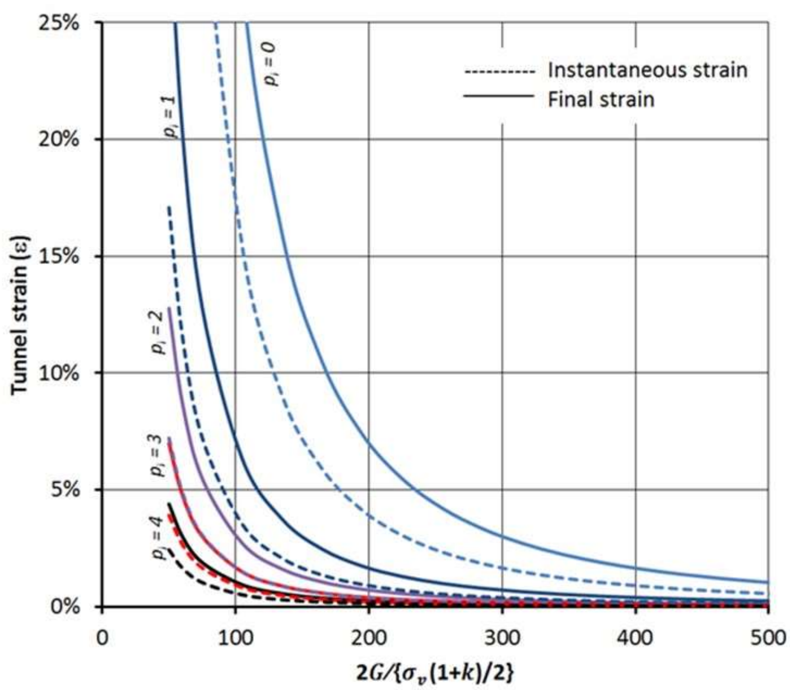

Figure 14. Tunnel strain vs. rock mass shear modulus and in-situ stress condition for different support pressure magnitude (Shrestha, 2014)

Carranza-Torres and Fairhurst (2000) suggests that the rock mass shear modulus (G) can be estimated using rock mass deformation modulus (E_{rm}) and Poison's ratio (ϑ) expressed by Equation 10.

$$G = \frac{E_{rm}}{2(1+\vartheta)} \qquad (10)$$

The rock mass deformation modulus (E_{rm}) and rock mass strength (σ_{cm}) are related to Young's modulus (E) and strength (σ_{ci}) of intact rock and can be estimated using Equation 1 and Equation 3. It is noted that in addition to in-situ stress condition and support pressure, the most relevant rock mass parameter regarding tunnel wall deformation estimation is the rock mass shear modulus (G).

3.2.3 Groundwater

The rock mass is composed of both intact rock and discontinuities. Excluding some high-porosity rocks such as young sandstones and certain volcanic rocks, most of the intact rocks have in general very low porosity. The visual observations in many unlined tunnels indicate that most of the water leakage occurs in the part of the tunnel which is either closest to the surface or is at a water bearing fractures, fault and weakness zones (Nilsen and Thidemann, 1993). Water inflow into the tunnel and leakage out of the tunnel are challenging issues for hydropower tunnels. At the tunnel face, water inflow during excavation may reduce work safety considerably and drilling and detonation may become very difficult. Stability of the tunnel may be reduced considerably due to reduction in the rock mass strength. Excessive inflow through weakness / fracture zones may cause severe stability problems and in extreme case tunnel may collapse (Panthi, 2006). Similarly, there is equal risk of water leakage out from the pressurized unlined/shotcrete lined waterway tunnels and shafts of hydropower projects during operation causing significant loss of valuable water (Figure 15).

Figure 15. Water inflow in a headrace tunnel striking a weakness zone (left) and leaked water from a headrace tunnel during test water filling (right).

Figure 15 demonstrates the importance of understanding the permeability, inflow and leakage potential from a pressurized headrace and tailrace tunnel system of the hydropower projects.

In general, the permeability of rock mass is governed by discontinuities and their engineering geological characteristics. In an unlined or shotcrete lined pressure tunnel, water gives pressure (P_w) to the rock mass equivalent to the hydrostatic water head (H) as indicated in Figure 16. The rock mass behaves differently when it is exposed to the water pressure. Therefore, the interaction between water pressure and rock mass is an important issue for unlined or shotcrete lined pressure tunnels (Panthi and Basnet, 2021).

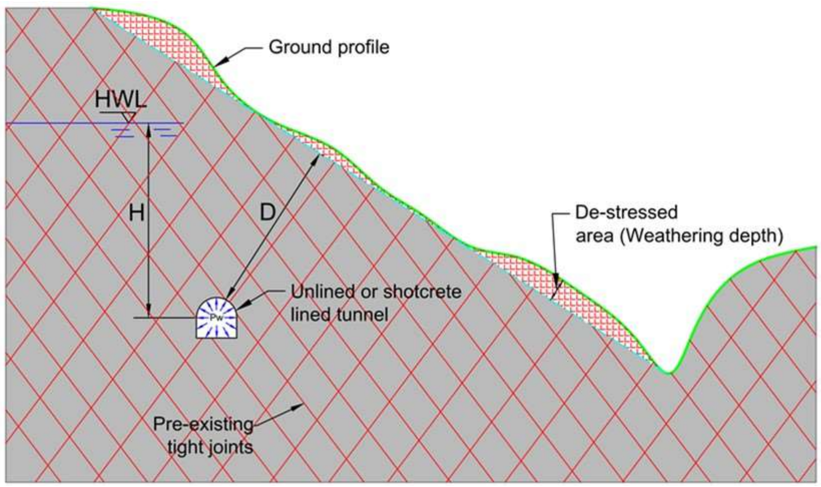

Figure 16. An idealized topographic arrangement explaining various condition surrounding a water tunnel (Panthi and Basnet, 2021)

Fluid flow through joints is in general non-laminar and unevenly distributed in the joint surface due to roughness and infilling condition of the joint walls. However, it is difficult to mathematically express governing laws for such turbulent flow through the joints. For simplicity, the fluid flow through joints is assumed to be laminar and represented by the Darcy's equation (Equation 11) described by the flow velocity (v) and hydraulic conductivity (K) with measuring units of m/s and the hydraulic gradient (i).

$$v = K \times i \qquad (11)$$

If the joint surface is assumed to be planner, the flow may be idealized by means of the parallel plate model (Louis, 1969). The joint hydraulic conductivity and flow rate (q) per unit width can thus be expressed by Equation 12 and Equation 13. In these Equations, a is the joint hydraulic aperture, k_j is a permeability factor (Equation 14), γ is unit weight of the water, ΔP is the pressure drop when water flows between two adjacent flow domains, *l* is the length assigned to the contact between the domains and µ is dynamic viscosity of water which according to Kestin et al (1978) is equal to 1.306 x 10-3 Pa-s at 10^0C (Panthi and Basnet, 2021).

$$K = \frac{a^2 \times \gamma}{12} \qquad (12)$$

$$q = -k_j \times a^3 \times \frac{\Delta P}{l} \qquad (13)$$

$$k_j = \frac{1}{12\mu} \quad (14)$$

According to Panthi (2006), among the most important aspect of unlined or shotcrete lined water tunnels is to control water leakage while tunnel is in operation at full hydrostatic pressure and to limit this leakage to an acceptable limit boundary. The leakage limit for a unlined or shotcrete lined water tunnel maybe defined maximum up to 1.5 liters per minute per meter tunnel. The rock mass permeability is mainly governed by degree of jointing and condition within different joint sets represented by joint aperture, infilling conditions, spacing of the must unfavorable joint set, joint persistence, hydrostatic water pressure in the rock mass and shortest distance from the water tunnel to the topographic slope surface (Figure 16).

The Equation 13 and 14 are not that practical to be used in the quantification of possible water leakage from the pressured headrace system during operation of a hydropower projects since it is very difficult to estimate rock mass permeability coefficient of the whole tunnel length. Yet, all tunnels are geologically well mapped and rock mass classification methods are used to assess overall rock mass quality. Panthi (2006) exploited comprehensive data records of Q-value (Barton et al, 1974) parameters and systematic water leakage test carried out ahead of the tunnel face while excavating the 8 km long headrace tunnel of Khimti I hydropower project in Nepal (Figure 1). The author came out with a suggestion of a semi-empirical equation (Equation 15) that can be used to estimate specific leakage (q_t) from an unlined or shotcrete lined pressure tunnel or shaft.

$$q_t = f_a \times H \times \frac{J_n \times J_r}{J_a} \quad (15)$$

$$f_a = \mathcal{L} \times \frac{J_p}{D \times J_s} \quad (16)$$

In Equations 15 and 16, f_a is a joint permeability factor with unit l/min/m2 which varies from 0.001 to 0.25 and can be estimated using Equation 16 (Panthi and Basnet, 2021). This factor is related to the physical condition of the joint sets, particularly, joint spacing (J_s) and joint persistence (J_p) with a maximum value not exceeding 25 m, 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). H is the hydrostatic water head (Figure 16), J_n is joint set number, J_r is joint roughness number and J_a joint alteration number of the Q-system of rock mass classification.

4 Basic design aspects

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,

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emphasized here that the sprayed concrete (shotcrete) is a permeable material and is not a structural element that either acts against excessive load or water pressure (Panthi and Basnet, 2017). Hence, 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 (Figure 17).

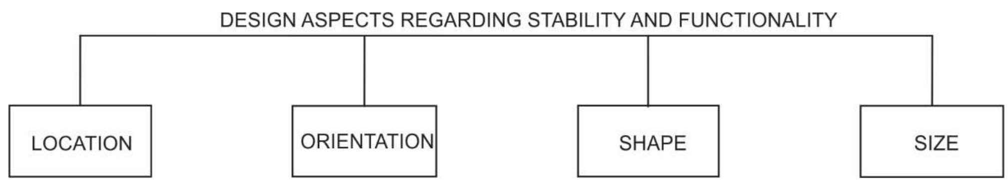

Figure 17. Elements controlling stability and long-term functionality of an underground opening

4.1 Location design

The most important aspects of engineering geological investigation are to find the best location for the underground structure of a hydropower scheme since an investigation defines the quality of rock mass that is prevailing in the vicinity of concern. According to Hudson (1993), there are mainly two modes of failure that may occur in an underground excavation. These are block failure when pre-existing blocks in the roof and side walls of an underground opening become free to move after the excavation has been made. The second one is stress failure when induced stresses around the excavation exceed the rock mass strength. Hence, most of the instabilities in underground excavation are depth dependent. Near the surface, the in-situ stresses are in general an-isotropic and discontinuities mainly control the stability. On the opposite, deep into the rock mass, the in-situ stress magnitudes are increased, and frequency of discontinuity occurrence are reduced due to enhanced confinement, and hence, in-situ stability is controlled by induced stresses. It means the stability challenges vary greatly depending upon the way a hydropower scheme is designed with respect to location of different underground structures such as headrace and tailrace tunnels, shafts, underground caverns, and access tunnels.

4.1.1 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. 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 (Figure 18). According to Edvardsson and Broch (2002), almost 99 percent of unlined pressure tunnels and shafts are successfully operated with no noticeable long-term instability problems. 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).

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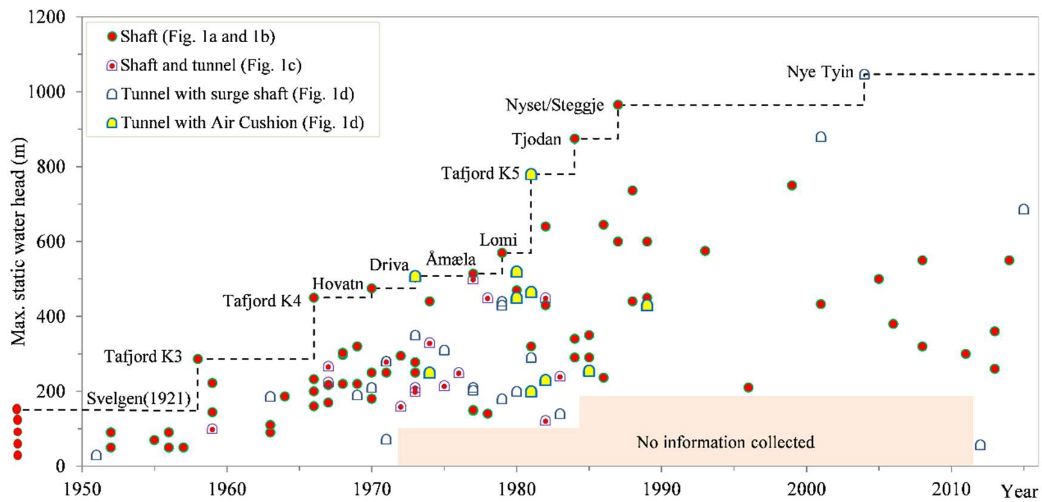

Figure 18. Development history and maximum static head of Norwegian unlined pressure tunnels and shafts. The figure is an updated version from Broch (2013) (Panthi and Basnet, 2016).

The developed criteria (Equation 17 and Equation 18) are based on the principle that both vertical and lateral rock covers (Figure 19-left) should provide basis to confine the pressure given by the static water head against hydraulic fracturing at any location of the pressure tunnel and shaft (Selmer-Olsen, 1969 and Broch, 1984a).

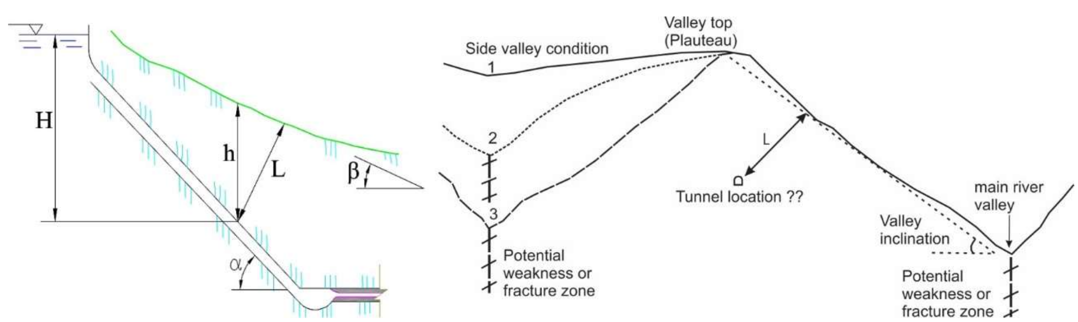

Figure 19. Idealized topography with geometrical parameters used in Norwegian confinement criteria (left) and different topographic conditions that may prevail in a hydropower scheme (right).

$$h > \frac{P_w}{\gamma_r \times \cos\alpha} \tag{17}$$

$$L > \frac{P_w}{\gamma_r \times \cos\beta} \tag{18}$$

In Equation 17 and 18, h is the vertical rock cover above tunnel alignment, H is the hydrostatic head acting in the tunnel, 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 (Figure 19).

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In general, the location assessment made by using Equation 17 and Equation 18 provides good result against hydraulic fracturing for the topography representing almost no existence of secondary valley (side valley condition 1 in Figure 19-right). However, if the topography consists more than one valley, the location assessment made using these equations may not provide needed safety margin. Therefore, it is important to assess the magnitude of minimum principal stress (σ_3) along the pressurized waterway which should always be more than the hydrostatic water head (Equation 19).

$$\sigma_3 > P_w \qquad (19)$$

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, a through leakage and potential hydraulic jacking assessment should be carried out using the methods described in Section 3.

4.1.2 Underground caverns

The greatest challenge associated to the design of underground caverns is the site location. Failing to select a proper location will generate greatest risk on both short-term and long-term stability which may result economic disaster to the hydropower project. It is important to keep in mind that the decision on where the underground powerhouse cavern should be placed is often made at an early stage of planning when there is limited rock quality knowledge (Figure 6). It is best to choose more than one possible location at this early stage of planning. A successful planning should result powerhouse cavern to be in best possible location regarding the quality of rock mass and in-situ stress conditions. Therefore, Broch (1984b) recommends that highly experienced experts in rock engineering field should be consulted and engaged. It is important that certain unfavorable rock types, such as soapstone, serpentinite, thinly bedded and foliated weak rocks with swelling potential, highly schistose and sheared rock mass, and areas with high degree of fracturing where de-stressing may have occurred should be avoided for the location of cavern (Edvardsson and Broch, 2002).

In addition, the cavern should be placed in such a way that not only the powerhouse cavern is in good quality rock mass where medium (neither high nor low) level of in-situ stresses with relatively less stress an-isotropic condition exists. If there exist a boundary fault or weakness zone between relatively schistose or fractured rock formations at the outer part of topography (along access and tailrace tunnels) and a competent rock formation where the cavern is to be located (Figure 20), a fairly sufficient distance (D > height of the cavern) between the fault and the underground cavern should be kept to avoid high level of stress an-isotropy and to achieve needed confinement in the rock mass. A favorable site has to be found within a limited area so that it is possible to avoid unnecessarily long access tunnel and the hydraulic condition along unlined or shotcrete lined high pressure headrace system guarantees no serious hydraulic fracturing or leakage potential.

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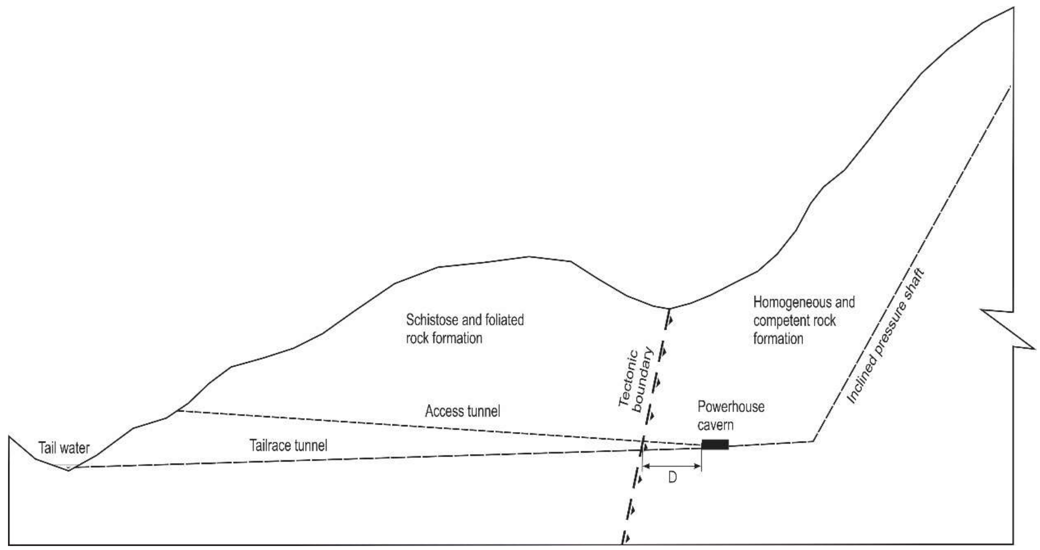

Figure 20. An idealized longitudinal profile indicating arrangements for different underground elements.

It is also emphasized here that all weakness/fault zones are identified, mapped, and projected all the way down to the alignment. Maximum attempt should be made to avoid such zones of weakness / faults from intersecting the cavern alignment. The stress induced stability assessment should be made using approaches discussed in Section 3 and by using both 2D and 3D numerical modeling. The appropriate rock support measures should be recommended.

4.2 Fixing orientation

The major and systematic joint and discontinuity systems are the decision-making factors in the choice of location of any underground structure. Therefore, it is vital that detailed discontinuity information is mapped, congregated, and systematized. During planning phase mapping, the major discontinuity systems such as bedding / foliation planes, cross joints, major fault/weakness zones present in the locality are identified and their orientation are measured (Nilsen and Thidemann, 1993). The basic rule that should be adopted is to orient the length axis of an underground opening along the bisection line of the maximum intersection angle between the two predominant joint systems (Figure 21) and parallelism with other minor joint systems should be avoided as possible.

During mapping and evaluation, not only the orientation and system of joints are important but also the character of discontinuity surfaces which control the frictional properties. For powerhouse cavern having relatively high walls, it is important to achieve an angle of at least 25 degrees to steeply dipping planner joint or joint system filled with clay material. In addition, the length axis alignment of an underground opening should avoid parallelism with the orientation of major bedding/foliation and other cross joints. This is especially the case for large scale caverns such as powerhouse and transformer caverns.

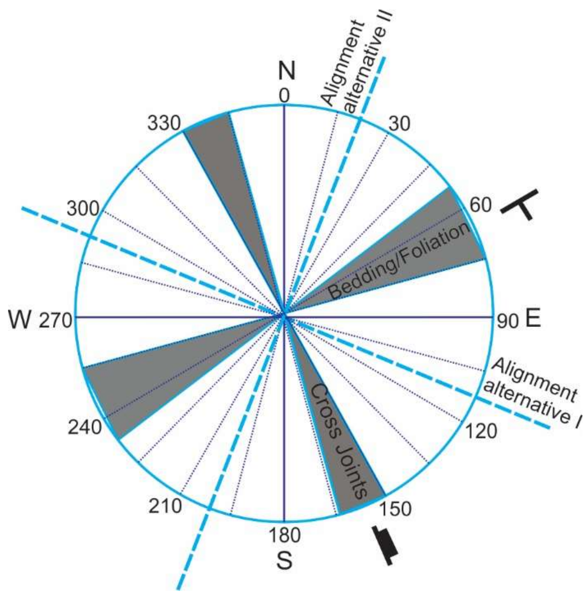

Figure 21. Discontinuity interpretation in a joint rosette with possible length axis aliment alternatives.

The directions of the major principal stresses are equally important while orientating the powerhouse and transformer caverns because stress induced stability problems are concentrated in areas of the contour where principal stress orientation are tangential (parallel) to the contour surface. Edvardsson and Broch (2002) recommend that the most stable orientation of the length axis of an underground opening is when it makes an angle between 15-25 degrees with respect to the horizontal projection of a major principal stress.

4.3 Shape and size

A shape and size of an underground opening constructed for hydropower projects depends on its's function. Most of the tunnels (access, headrace, and tailrace) of a hydropower plant may have an inverted D shape if the tunnel is excavated using drill and blast method. Similarly, the tunnels will have circular in shape if these are excavated using TBM method. On the other hand, most of the vertical shafts are circular in shape and inclined shaft may either be circular or inverted D shaped depending upon the excavation method used. The caverns are mostly of inverted D shaped with deep walls and are excavated using drill and blast method. The size of tunnels, shafts and caverns are mainly dependent on the purpose and requirement for the hydropower project in question.

4.3.1 Shape and size of waterway tunnels

The hydraulic efficiency or extent of frictional head-loss of a tunnel or shaft depends on the shape and size. TBM excavated tunnels are circular in shape and have smooth wall surface in comparison to the tunnels excavated with drill and blast method and therefore are hydraulically ideal in shape. However, it is not always feasible to use TBM as an excavation method for these tunnels since success of TBM application is largely dependent on the geological condition and length of the tunnel to be excavated (Panthi, 2015). In general, drill and blast method of excavation is the dominating construction method for these tunnels due to flexibility in making actions if unforeseen geological conditions arise. However, it is underlined here that tunnel walls excavated using drill and blast method have undulated surface of varying smoothness and shape of tunnel will be determined mostly by construction necessities and easiness (Lysne et al., 2003). The most practical tunnel shapes excavated using drill and blast method are inverted D or horseshoe shaped (Figure 22).

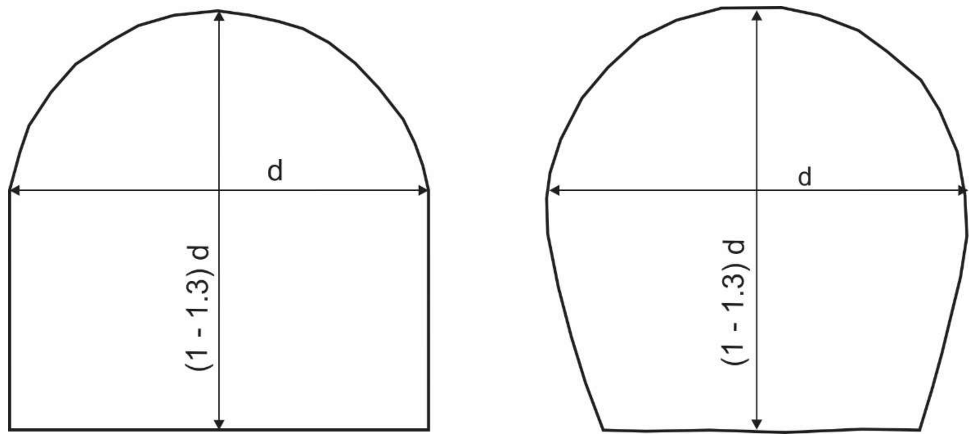

Figure 22. Recommended inverted D-shape (left) horseshoes shape (right) tunnel (Panthi, 2015).

The inverted D-shaped tunnel is recommended for tunnels passing through good quality rock mass where the mode of failure is brittle. However, if the rock mass is highly schistose, weak, and deformable in nature with possibility of tunnel squeezing, slightly curved shaped (horseshoe shaped) tunnel helps strengthen the overall stability (Panthi, 2015).
The excavated waterway tunnel profile may either be unlined / shotcrete lined or concrete / steel lined depending on the rock mass and in-situ stress conditions. The shotcrete lined tunnels end up similar with the excavated surface shape, which is undulated (Figure 23). The extent of undulation depends on the quality of rock mass and proficiency of the contractor involved in the construction.

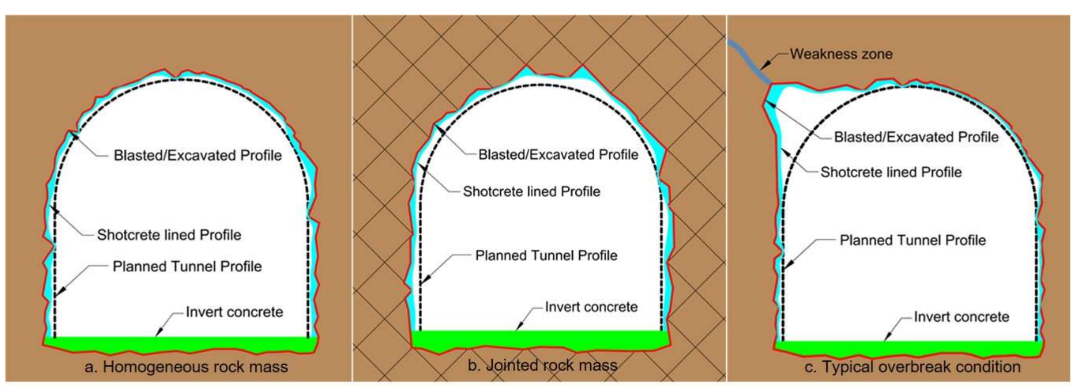

Figure 23. Shape of a shotcrete lined water tunnel influenced by geological condition (Basnet and Panthi, 2018).

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In principle, the optimum hydraulic shape of a water tunnel occurs when wall height (length between tunnel invert to the spring level where curvature starts) of a tunnel is between 1 to 1.3 times the radius of the tunnel curvature above spring level.

Figure 23a is seldom achieved in the jointed rock mass, so Figures 23b and 23c are most common contour profiles in the blasted tunnels. Overbreak in Figures 23a and 23b may be defined as normal overbreak whereas localized enlarged area in Figure 23c may be expressed as excessive overbreak. Such localized enlarged area may also be formed due to stress induced rock spalling and rock burst in hard rocks and due to squeezing in deformable rocks (Basnet and Panthi, 2018). The wall surface of a water tunnel and shaft is either unlined or shotcrete / concrete / steel lined. Rougher the surface, pronounced will be the flow resistance due to large undulations (Figure 21). Following Lysne et al (2003) and Basnet and Panthi (2018), frictional headloss (Equation 20 and 21) 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 22 and Equation 23 which largely dependent on both surface roughness (ε_R), the Reynolds number (R) and the hydraulic radius (R_h) of a tunnel which is a function of area (A) and perimeter (P) (Equation 24). In Norway, it is normal to keep water velocity in an unlined or shotcrete lined water tunnels between 1 to 2 m/sec.

$$\mathrm{H_f} = \frac{\mathrm{f\,L\,v^2}}{\mathrm{2g(4R_h)}} \qquad (20)$$

$$\mathrm{H_f} = \frac{\mathrm{L\,v^2}}{\mathrm{M^2 R_h}^{4/3}} \qquad (21)$$

$$\frac{1}{\sqrt{\mathrm{f}}} = -2\log\left(\frac{\varepsilon_R}{14.8\,\mathrm{R_h}} + \frac{2.51}{\mathrm{R}\sqrt{\mathrm{f}}}\right) \qquad (22)$$

$$\mathrm{M_R} = \frac{24}{\mathrm{\varepsilon_R}^{1/5}} \qquad (23)$$

$$R_h = \frac{A}{P} \qquad (24)$$

In addition, different types of singular losses that usually are formed across the waterway tunnels such as entrance loss, trashrack loss, gate loss, bend loss, transition loss, nitches 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 however 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.

4.3.2 Shape and size of powerhouse cavern

The stability of an underground cavern is either dependent on the shear strength of discontinuities which is a function of mobilized normal stress or on the in-situ stress condition prevailing at the area where the cavern will be located. In addition to the location and orientation, shape and size of a cavern are very important aspects regarding over stability. Therefore, it is important that the size of a cavern is optimized based on the desired functional need and the shape is made in such a way that it achieves evenly distributed stresses along the whole periphery (roof and walls). The evenly distributed stress condition can according to Edvardsson and Broch (2002) be achieved by giving the cavern a simple shape as possible with an arched roof and with limited (best to avoid) protruding corners (Figure 24).

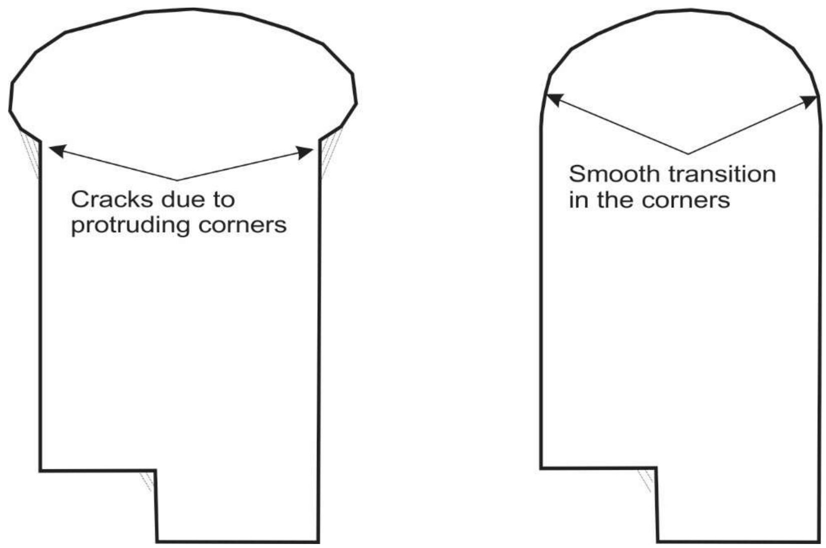

Figure 24. Shape of a powerhouse cavern with protruding (left) and smooth corners (right).

As seen in Figure 24 left, if a cavern roof is designed with a protruding corners which many do 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 will in general reduce the stability considerably and the failure may extend further down to the cavern walls. Therefore, the cavern roof should be designed with smooth transition as in the right figure. It is further emphasized here that the in-situ stress measurements should be carried out so that the magnitude and direction of the stresses are determined. 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.

5 Conclusions

Most of the hydropower projects built in a mountainous area with an installed capacity exceeding 5 MW in general consists different underground elements such as headrace tunnel, pressure shaft, powerhouse cavern, tailrace, and access tunnels. A comprehensive and step-wise engineering geological investigations are necessary to evaluate the overall geological condition in the area, to assess system of joints and their engineering geological characteristics, to judge overall quality of the rock mass and to find out in-situ stress condition. The geological investigation results and prevailing design principles as discussed here should be the basis in selecting the best possible locations and orientation of different underground elements in the most favorable way possible. It is important that the shape and size of an underground opening are also very important elements to secure a stable, optimum and sustainable development of a hydropower project.

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