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Methods applied for the stability assessment in rock engineering

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ABSTRACT

The rock mass is exposed to stress changes after the excavation is made for the construction of infrastructures such as roads, railways, hydropower and irrigation projects. The expansion of the roads requires slope cuts which cause the natural discontinuities to daylight leading to the rock slope failure of different types and magnitudes. The construction of tunnels and underground caverns for different infrastructure projects will pass through rock mass with varying rock cover leading to different stability challenges. The changes in in-situ stress condition therefore will have direct impact on the stability of infrastructure projects. This key-note lecture will highlight the methods used to make a stability assessment of rock slopes and underground excavations. The focus will be given on the most important aspect of design and stability assessment covering rock slope failures, block falls in tunnels and issues associated to in-situ stress conditions on elastic brittle and elastic plastic rock material. Tunnels and underground caverns located in shallow overburden are subjected to de-stresses conditions causing block falls. On the other hand, tunnels and caverns located deep into the rock mass (high rock cover) are subject to instabilities caused by induced rock stresses. If the rock mass is relatively unjointed and massive, the instability is associated to brittle failure called rock spalling/ rock bursting. On the other hand, if the rock mass is weak and deformable, the instability is associated with plastic deformation called squeezing. Therefore, stability assessment in rock engineering is a challenging task and needs deep knowledge on the behavior of rock mass and is of the challenging issue for engineering geologists and rock engineers.

Keywords: Rock mass, rock stress, rock slope, tunnels, caverns, stability assessment methods

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INTRODUCTION

Instabilities in tunnels and rock slopes are geological incidences associated to the rock mass quality. The rock mass is characterized as inhomogeneous, and its quality is affected by the existence of discontinuities, weathering, strength anisotropy, presences of in-situ stress and groundwater. The rock slope failures are influenced either by natural physical processes or man-made interventions along the natural rock slopes. The stability analysis of a potentially unstable rock slope is one of the key engineering design issues in civil engineering and open pit mining. The design of underground structures in a cost-effective way with minimum instabilities is also very important issue in civil and mining engineering. Both these areas demand step wise investigations on the engineering geological condition. Hence, the importance of economic impact caused by either over design or under design of rock slope and underground opening should not be ignored. Every effort should be made to come to an optimum design solution (Panthi, 2006). The aim of this manuscript is to briefly present the methods that are applied and can be applied in future for the stability assessment of both rock slope and underground openings.

ROCK SLOPE STABILITY ASSESSMENT

All rock slopes consist numbers of significant structural features and variety of geological parameters. Presence of such variable geometrical and geological conditions in a rock slope influence on the safe and optimum slope design and is a challenging task. Among the very important parameter that influences in rock slope design is the shear strength of the discontinuities, which is governed by different geological and non-geological parameters (Panthi, 2021). Before carrying out the stability assessment of a rock slope (natural or cut slope) it is important to first evaluate whether the rock slope in question is analyzed for short-term or long-term stability perspective considering extent of risk to be taken and economic resources to be used. This is because a short-term stability signifies on the relatively low social and economic damages to society if the slope fails and is hence associated to a slope for short-term purpose or a slope that has insignificant damage impacts. On the other hand, a long-term stability signifies to extensive social and economic damages with significant impacts if the rock slope fails.

Most of the rock slope failures occur along the existing shear planes, weakness zones, and discontinuities. Therefore, the orientation of these geological structures in relation to the orientation of the rock slope itself will define potential failure mode. Following Hoek (2009) four main modes of failures may be met along the rock slope depending upon the orientation of discontinuities and rock slope face as indicated in Figure 1.

Basic principles of stability analysis

Now a day variety of sophisticated tools for analyzing rock slope stability exist and the analysis itself can be done with a high degree of accuracy. However, great deal of uncertainty cannot be overruled as such tools/models require comprehensive geological data input during analysis. If the accuracy of geological data acquired are in question, the results obtained from such analysis will be misleading. Therefore, very strict procedure of slope stability analysis should be followed. According to Nilsen and Palmstrøm (2000), there are three steps involved in rock slope stability assessment, which are.

- Definition of potential problem
- Quantification of input parameters and,
- Calculation or evaluation of stability

Definition of potential problem

Decision on what information to collect for defining the potential problem is a very crucial task in rock slope stability analysis. Accurate engineering geological and geotechnical data collection is thus a key issue at this stage. The data collection can be done by extensive field mapping of the discontinuities, by analyzing lithological, structural, hydrological, and tectonic information and by studying photographs and core logs of the rock slope under question. Different rock mass classification methods can also be used to measure different geotechnical parameters of the discontinuity surfaces. It is noted here that the great majority of rockslides occur along the major geological discontinuities such as lithological boundaries, bedding planes, faults, and weakness zones. Therefore, geometrical, and geological properties of rock slope and discontinuities in it will make it possible to identify the type of the failure mode as given in Figure 1. The first step to be used for such assessment is the use of stereographic projection technique (Fig. 2).

It is emphasized that in some cases, the geometry of the potentially critical rock slope is complex especially in highly weathered rock mass and defining the potential failure mode may become difficult. Still, in great majority, the definition of potential failure mode is relatively easy task.

Quantification of input parameters

There are two most prominent factors governing the rock slope stability, i.e., the geometry of the rock slope and shear strength (frictional properties) of the potential failure surface (Fig. 3). These two factors are controlled by different other parameters like groundwater, seismic acceleration, irregularities in the discontinuity surface etc.

The inputs require to establish these two factors for analyzing rock slope stability includes a very comprehensive data base consisting geometrical and structural geological information, hydrological models for groundwater pressure estimation, strength and deformation properties of the rock mass discontinuities and estimation of the external forces like earthquake and blasting vibrations. The geometrical and structural geological information required for slope stability analysis can be defined by field mapping and stereographic analysis. The challenging task however is to establish shear strength properties of the discontinuity surfaces consisting of friction, cohesion, normal stress, and water pressure.

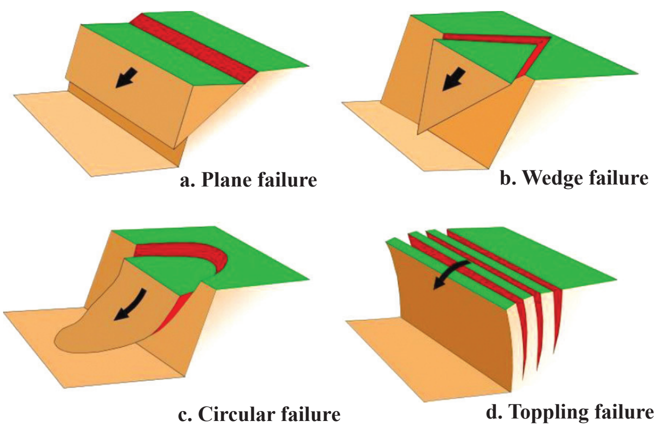

Fig. 1: Simplified illustrations of most common slope failure modes (Hoek, 2009).

Evaluation of stability

When geometry and the potential failure mode have been defined and quantification of input parameters have been made, the overall evaluation of the stability assessment is a straightforward task. For reliable calculation of stability, it is crucial to have reliable input parameters. For large scale slopes in complex geology, numerical analysis may supplement the quality of assessment. For an excavated cut-slopes, limit equilibrium analysis where the calculation of factor of safety (FS) is involved is normally the preferred method defined by Equation 1 which should be used first.

$$FS = \frac{Stabilizing\ forces}{Destabilizing\ forces} > 1.3\ to\ 2.0 \qquad (1)$$

In general, for all civil engineering structures the factor of safety must be above 1.3 and may reach to up to two for very important structures.

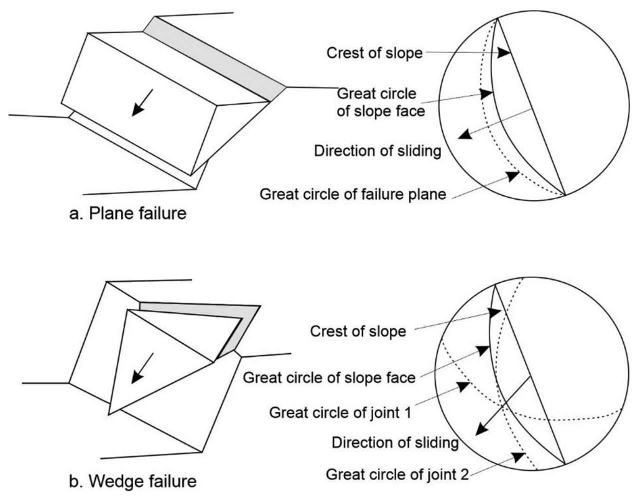

Fig. 2: Typical plane and wedge failures with stereo plot of slope face and discontinuities.

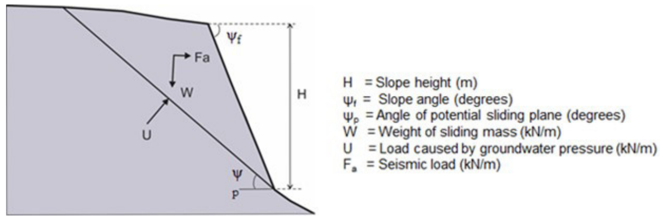

Fig. 3: Typical geometry for potential plane failure (Panthi, 2022).

TUNNEL STABILITY ASSESSMENT METHODS

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. To assess the economic viability, the rock mass quality along the tunnel alignment should be examined and estimated quantitatively during planning and design phases. To do so, an in-depth engineering geological investigation should be conducted (Panthi and Broch, 2022). A rock mass, which is a heterogeneous medium, is characterized by two main features i.e., (1) rock mass quality, (2) the mechanical processes acting on the rock mass (Panthi, 2006). The first one is related to the rock mass strength, deformability properties, strength anisotropy, presence of discontinuities and weathering effect. 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. Various methods that could be used in the assessment of stability condition in tunnels and underground caverns are briefly presented here under.

Empirical approach

Rock mass classification systems, which are empirical approaches of stability assessment, are the means used extensively in the Himalayan region to assess the quality of rock mass and to estimate/ decide rock support requirement at both planning, design and construction phases. According to Panthi and Nilsen (2007) there are mainly two areas where rock mass classification systems have been widely used in the Himalaya, i.e., for evaluating rock mass conditions and for estimating required tunnel rock support during pre-construction and construction phases. Both RMR (Bieniawski, 1973) and Q (Barton et al., 1974) methods of rock mass classifications are widely used in Nepal (the details of these classification systems are available widely and will not be discussed here further).

The benefit of the use of RMR method of classification is that the overall quality rating can be used to assess the stand-up time (Fig. 4) of an underground structure under excavation. The classification allows time for a rock engineer or engineering geologist to make decision on the equipment and rock support measure to be used for the stabilization of underground opening.

The benefit of use of Q method of classification on the other hand is that it is directly hooked with the recommended rock support decision chart (Fig. 5) which a rock engineer or engineering geologist can use to estimate the rock support measure during planning and design phases and to decides preliminary rock support requirement at spot after classifying the rock mass at the face of an underground opening.

The rock support categories in Figure 5 are described as; (1) unsupported, (2) spot bolting, (3) Systematic bolting and 5-6 cm thick unreinforced shotcrete, (4) systematic bolting and 6-9 cm thick fiber-reinforced shotcrete, (5) systematic bolting and 9-12 cm thick fiber-reinforced shotcrete, (6) systematic bolting and 12-15 cm thick fiber-reinforced shotcrete, (7) systematic bolting, >15 cm thick fiber-reinforced shotcrete + reinforced ribs of shotcrete, (8) Concrete lining.

The applicability of the rock mass classification methods as tools to assess stability conditions are always questionable and needs verification by other methods such as analytical calculations and numerical modeling. The experience suggests that if carefully used the classification system may provide good results in tunnels located in medium rock cover (100 to 300 m) and having rock mass quality class with RMR value exceeding 45 and Q value exceeding 1, respectively.

Tunnels in shallow overburden

The tunnels at shallow overburden have low gravitational stresses in the rock mass. The low vertical stress causes higher level of stress anisotropy. In addition, near surface rock mass is prone to high degree of weathering. Therefore, tunnels and underground caverns will be exposed to low level of interlocking effect between the rock blocks which will result in the reduction of arching effect leading to potential block falls from the roof and side walls. Therefore, the stability assessment should be associated to this type of failure as indicated in Figure 6.

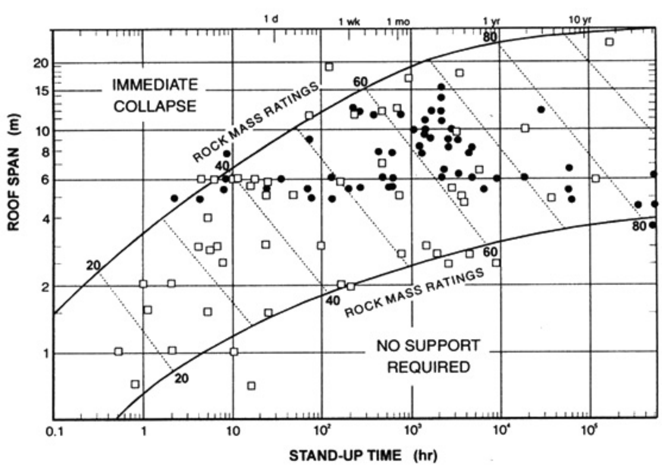

Fig. 4: Stand-up time vs. roof span of an underground opening and RMR value (Bieniawaski, 1989).

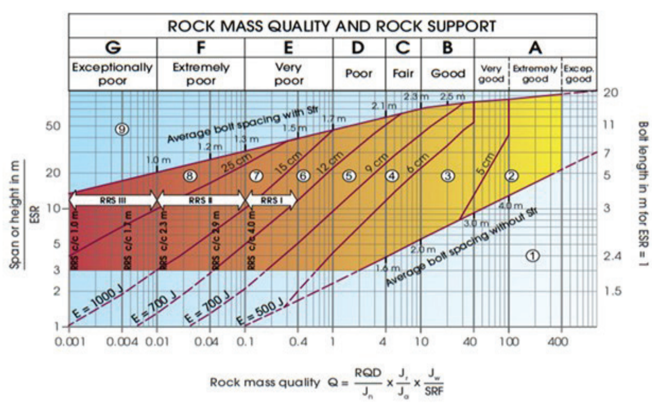

Fig. 5: Updated rock support chart for tunnels and caverns (NGI, 2015).

The assessment approach to this kind of instability is similar as stability assessment of a wedge failure used in the rock slope stability assessment. Equation 2 provides an insight to the approach of calculation of factor of safety (FS).

$$FS = \frac{Capacity\ of\ the\ anchors}{Weight\ of\ the\ block} > 1.3 \qquad (2)$$

It is emphasized that the confinement produced by the horizontal stress may be ignored while calculating the factor of safety and while selecting the capacity of the applied anchors.

Tunnels in competent rocks with high overburden

Upon excavation of an underground opening in the rock mass, in-situ stresses are redistributed. As a result, tangential stresses are induced in the vicinity of an underground opening (Panthi, 2012). If the rock mass strength is less than induced stresses, overstressing will occur in the periphery of an underground opening leading to stress induced instability. In relatively unjointed and massive rock mass, the tunnel instability will thus be associated with rock spalling or rock burst (strain burst). The use of proper assessments methods is essential for a meaningful instability assessment of rock burst / rock spalling condition in tunnels (Panthi and Broch, 2022). In the following two such methods are presented.

Norwegian rule of thumb

The tunnels built in Norway for hydropower, road and railways run through steep valley-side slopes where stress an-isotropy is a very common phenomenon. The tunnels experiencing rock burst / rock spalling are quite common instability issues while tunneling through hard and brittle rocks mass (Panthi, 2018). Selmer-Olsen (1965) studied over 60 tunnels passing parallel with valley-side slope where rock burst and rock spalling were experienced during tunnel excavation. Most of these tunnels were passing through a topography where vertical rock cover directly above the tunnel was relatively small in comparison to the vertical height between the tunnel and the top of the valley-side slope (the plateau). In addition, most of these tunnels had relatively short distance (mostly not exceeding 300 m) from the valley surface as indicated in Figure 7.

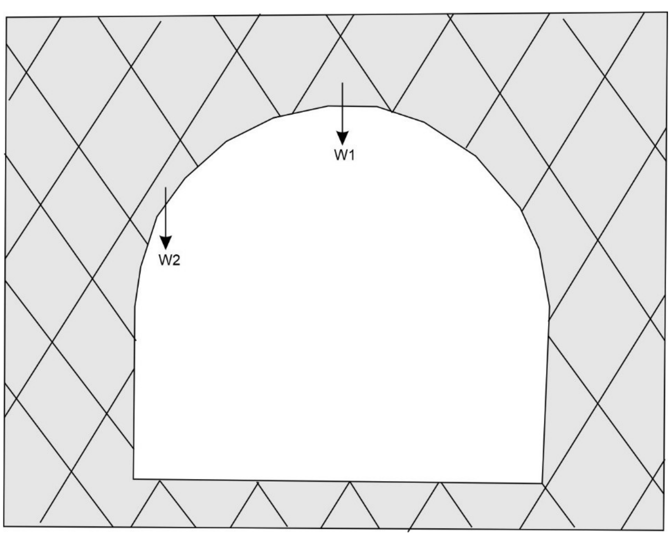

Fig. 6: Potential scenario of block falls in tunnels located at shallow overburden.

As can be seen in Figure 7 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 activities. 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 7 (right). The Norwegian rule of thumb can be used as first approach during early phase of planning and design of tunnels so that tunnels are placed at the best possible locations possible.

Semi-analytical method

Norwegian rule of thumb presented above provides qualitative assessment of rock burst and hence do not provide a clear picture on the severity of the rock burst depth-impact (S_d) into the rock mass behind the tunnel wall (Panthi, 2018) as shown in Figure 8. The knowledge on the rock burst depth-impact (S_d) is crucial in making decision on the application of rock support (Panthi, 2012).

Martin and Christiansson (2009) proposed a semi-analytical approach (Eq. 3) to assess the extent of rock burst/ rock spalling depth-impact (S_d) in a tunnel prone to rock burst/ spalling.

$$S_d \approx r \times \left[0.5 \times \frac{\sigma_{\theta-max}}{\sigma_{sm}} - 0.52\right] \qquad (3)$$

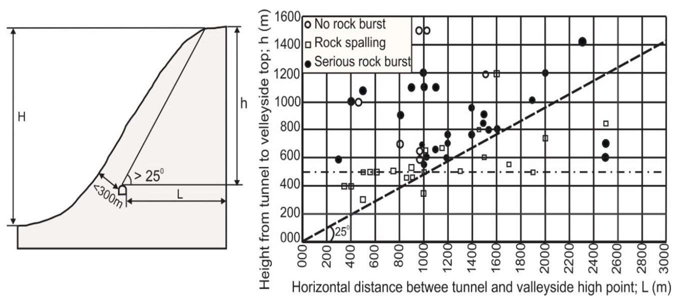

Fig. 7: Location of tunnels with respect to topographic conditions (left), and plot of rock burst/spalling in relation to height (h) from tunnel roof to top of valley-side and horizontal distance from tunnel to the top of valley side (L) (Panthi, 2018).

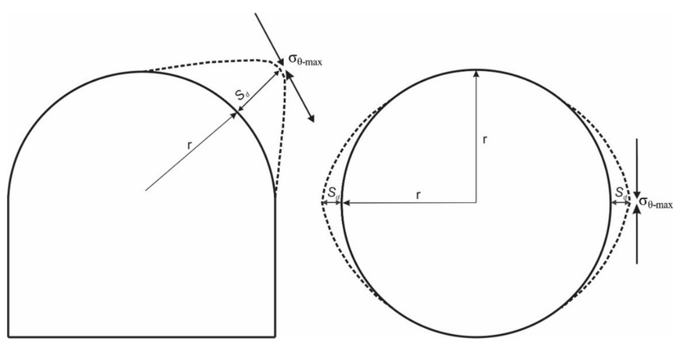

Fig. 8: Both drill and blast and TBM excavated tunnels showing potential damage zone in the tunnel wall (depth-impact, S_d) due to induced tangential compressional stress (Panthi, 2018).

As seen in Equation 3, the rock burst/ spalling depth-impact (S_d) 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 either using Equation 4 for schistose and foliated rock mass (Panthi, 2006) or Equation 5 for brittle and massive rock mass (Panthi, 2018) depending upon rock mass character.

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.5}}{60} \tag{4}$$

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.6}}{60} \tag{5}$$

The rock burst/ spalling depth-impact assessment using Equation 3 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 ($\sigma_{\theta\text{-max}}$) is needed which can be done using Kirsch's equation (Equation 6) defined by maximum principal stress (σ_1) and minimum principal stresses (σ_3).

$$\sigma_{\theta-max} = 3\sigma_1 - \sigma_3 \tag{6}$$

Panthi (2018) highlights that the rock mass strength (σ_{cm}) for rocks with high degree of schistocity is in general below 0.3 times the intact rock strength. Hence, Equation 4 is appropriate to be used for the rock mass influenced by high degree of schistosity. On the other hand, Equation 5 should be used to calculate rock mass strength (σ_{cm}) for massive, homogeneous, brittle rock mass with relatively high intact rock strength (σ_{ci}), i.e., exceeding 150 MPa.

Tunnels in weak and deformable rock mass

When a tunnel is subjected to induced stresses, weak and schistose rock mass behave differently from the isotropic and stronger rock mass. If the rock mass is highly schistose, upon unloading a visco-plastic zone is formed deep into the tunnel wall leading to a time dependent inward movement of the rock mass material. This phenomenon is described as tunnel squeezing. A reliable prediction on the extent of squeezing is therefore a key issue while tunneling through weak and highly schistose, sheared, and thinly foliated/ bedded rock mass (Panthi, 2006). In the following two of such prediction approaches, i.e., Hoek and Marinos (2000) approach and Panthi and Shrestha (2018) approach, are presented.

Semi-analytical method

A semi-analytical method for predicting tunnel strain (deformation) was proposed by Hoek and Marinos (2000). The method focuses on the rock mass strength (σ_{cm}) and overburden stress (σ_v) as two key parameters controlling plastic deformation (squeezing) in tunnels. The authors proposed two relationships (Eq. 7, 8) where tunnel strain (ε_t) in percentage is a function of rock mass strength (σ_{cm}), overburden stress (σ_v) and support pressure (p_i).

$$\varepsilon_t = 0.2 \times \left(\frac{\sigma_{cm}}{\sigma_v}\right)^{-2} \tag{7}$$

$$\varepsilon_t = \left(0.2 - 0.25 \times \frac{p_i}{\sigma_v}\right) \times \left[\frac{\sigma_{cm}}{\sigma_v}\right]^{\left(2.4 \times \frac{p_i}{\sigma_v} - 2\right)} \tag{8}$$

The rock mass strength (σ_{cm}) can be calculated using Equation 4 which is related to relatively weak, highly schistose, and deformable rock mass.

Analytical method

The semi-analytical method presented above considers that the stress conditions are isostatic, tunnels are circular, and the vertical stress mainly governs the plastic deformation in tunnels and caverns. However, this is not always true since in-situ principal stresses are an-isotropic, and tunnels are not circular in shape excluding those excavated using TBM. Considering the constraint on tunnel shape and stress an-isotropy in the estimation of tunnel deformation, Panthi and Shrestha (2018) proposed equations 9 and 10 to estimate both instantaneous and final plastic deformation in tunnels (tunnel strain in percentage). The authors recommend that the rock mass shear modulus (G) is more appropriate parameter to be linked with plastic deformation (squeezing) analysis for highly schistose, thinly foliated/laminated, and weak rock mass.

$$\varepsilon_{IC} = 3065 \left(\frac{\sigma_v(1+k)/2}{2G(1+p_i)}\right)^{2.13} \tag{9}$$

$$\varepsilon_{FC} = 4509 \left(\frac{\sigma_v(1+k)/2}{2G(1+p_i)}\right)^{2.09} \tag{10}$$

As seen, the Equation 9 and Equation 10 include the overburden stress (σ_v), horizontal to vertical stress ratio (k) and support pressures (p_i). According to Panthi and Shrestha (2018), the rock mass shear modulus (G) can be estimated using rock mass deformation modulus (E_{rm}) and Poison's ratio (ϑ) expressed by Equation 11 and rock mass deformation modulus (E_{rm}) can be calculated using Equation 12.

$$G = \frac{E_{rm}}{2(1+\vartheta)} \tag{11}$$

$$E_{cm} = E \times \left(\frac{\sigma_{cm}}{\sigma_{ci}}\right) \tag{12}$$

It is highlighted here that a use of proper assessments methods is key for a meaningful stability assessment in an underground opening associated to block fall and stress induced instabilities. The methods presented above will give a good guide to the engineering geologists, designer and researcher working in rock engineering field.

CONCLUSIONS

Proper stability assessment demands good knowledge in geology and rock engineering. Use of only rock mass classification methods is not enough for the assessment of instabilities that may occur in the cut slope and underground openings. Empirical methods may be useful during preliminary planning. However, at detailed design and construction phases it is essential to use analytical methods. The results achieved through analytical methods should be verified by numerical modeling. In addition to the methods highlighted, it is necessary to assess potential groundwater condition, shear strength properties, inflow, and leakage in tunnels and from the tunnels, respectively. The final rock support design must be based on the results of the total stability assessment.

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