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An Overview of Design and Construction practices of Himalayan Hydropower tunnels

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An Overview of Design and Construction practices of Himalayan Hydropower tunnels

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Abstract: Due to steep terrain and fast flowing rivers in the Himalayan region of Nepal, medium to mega size hydropower projects are constructing day-by-day. Tunnel is one of the best and short routes for water conveyance system for power production. Hundreds of kilometers of tunnels have been constructed and new tunnels are planned in this region. The availability of high head for hydropower generation, the tunnel cross sections are relatively small, up to 6 m diameter in size, and there is high rock cover above the tunnel alignment. This paper focuses on the design and construction practices of hydropower tunnel passes through weak rock mass with high rock cover of the Himalayan region of Nepal. Most of the hydropower tunnels undergo excessive deformation and support failure during the tunnel construction, which delayed the project as well as increases the cost of project. The paper first discusses the available construction practices for tunnel and underground structures. In the second part, the current practice employed during construction in the Himalayas is discussed along with the shortcomings of the methods and how it is addressed in the region.

Keywords: Hydropower tunnel construction, design methods, deformation, support failure, field investigation

1. Introduction

Construction of underground space is time and money consuming task. In country like Nepal, which is dominated by high hills and steep terrain, underground support provides economical solution for water conveyance and transportation [1],[2]. But since the geology of Himalayas is fragile and with high overburden, tunnels and underground structure undergo excessive deformation and support failure during the construction [3]. Many hydropower tunnels constructed in the region has faced the problem of heavy squeezing, roof collapse, water inflow and face failure. The Chameliya HRT is one of the prime examples of heavy squeezing. The tunnel of 5.2m diameter and around 4km length was dominated by slate, talcosic phyllite, limestone and dolomite schist. As the tunnel passed through a major fault region, the section of the tunnel faced a squeezing of upto 1m (Figure 1a). Squeezing was also reported in HRT of Kaligandaki A HEP, Khimti HEP, Middle Bhotekoshi HEP. Similarly, Modi HRT faced the problem of squeezing along with heavy groundwater inflow (Figure 1c); Sanjen HRT suffered face collapse (Figure 1b) during construction [4].

Thus, the design of underground support has been a crucial part and many methods have been developed over the long period of time. In ancient times, support system like masonry wall of stone and brick were used and in many cases the timber was used as the structural member. But the use of these members became obsolete as it would take a lot of time for installation of the timber sections and in

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extreme conditions, the support itself would take up one-third and sometimes more of the excavated section. The major problem with the traditional support was that the support yielded, creating loosening pressures in the rock mass which would later be the cause of failure of the structures. These problems were later addressed after the introduction of structural member like steel ribs, shotcrete and concrete linings, etc. [5].

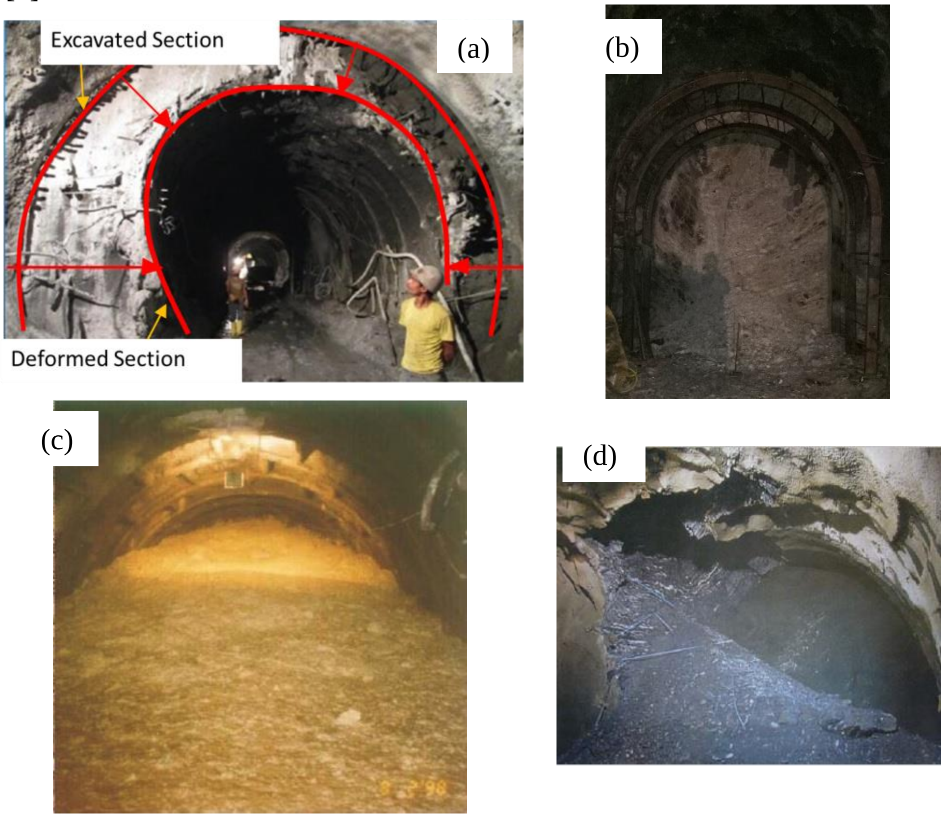

Figure 1. Tunnelling Problems in the Himalayas (a) Squeezing in Chameliya HRT, (b) Face collapse in Sanjen HRT, (c) Water inflow in Modi HRT and (d) Roof Collapse in Kaligandaki A HRT [2],[4],[6].

2. Design methods for tunnel supports for Himalayan tunnels

During the design of the tunnel supports, in-situ stress plays a vital role. Knowledge of the stress is vital for safe and economic design of the support system. Apart from field study, numerous alternatives are available for the estimating the incoming stress around the excavation with a certain degree of accuracy. Empirical, Analytical and Numerical techniques are established for preliminary estimate of the rock stress and thus the support requirement. In the Himalayas, empirical and analytical method are the mostly used methods. The design of the support system is heavily relied on the empirical rock mass classification technique. There is limited practice of using the numerical modelling technique and ay changes required to the support based on the geological condition is done based on the experience of the site engineer.

2.1. Empirical Method

Empirical methods are based on the experience gathered by researchers in various parts of the world. Several methods have been developed over the time for accessing the ground conditions [7], for estimating support pressure [10] and estimating the tunnel strain [16],[17] and improvements have also been made with relation to the experience of newer ground conditions. These empirical methods provide an estimate to the ground conditions based on which support are designed. However, they only provide and initial estimate and final designed must be improved according to the ground condition faced during the construction of the project.

2.1.1. Squeezing Assessment. Squeezing assessment is done to determine squeezing from non-squeezing ground. Squeezing in one of the major problems during underground works in the Himalayas, so it is essential to identify the squeezing ground. Various empirical relations have been provided to identify the squeezing ground among which the relations provided by Singh et al. *(Equation* 1) and Goel et al. *(Equation* 2) are the most popular [7],[8]. Recently, Jimenez and Recio *(Equation* 3) have also proposed a new relation to identify squeezing ground [9]. The relations provided by the researchers *(Equation 1, 2, 3)* all give a value of critical overburden. If the rock cover above a tunnel section is greater than the critical value, the section is categorized as squeezing else it is non-squeezing. A log-log graph is plotted to determine the squeezing sections.

$$H = 350 * Q^{0.33} \tag{1}$$

$$H = (275 * N^{0.33}) * B^{-0.1} \tag{2}$$

$$H = 424.4 * Q^{0.32} \tag{3}$$

2.1.2. Rock Mass Classification. Following the ground assessment, the rock mass classification is done to determine the support requirements. Three distinct methods are available for classifying the rock mass: The Rock Mass Rating (RMR) [18], the rock Quality Index (Q-system) [12] and the Geological Strength Index (GSI) [27]. The RMR and Q-system is used to classify the rock mass and it also provides an estimate to the support system for each rock class. The GSI, however, would place greater emphasis on basic geological observations of rock mass characteristics; reflect the material, its structure, and its geological history; and would be developed specifically for the estimation of rock mass properties rather than for tunnel reinforcement and support.

The preferred method of rock classification is the Q-system in the Himalayas due to its simplicity and as it is an Observational Method, assessment of the variations in ground conditions is done during construction and required modifications can be done to adapt to the actual ground condition. Although, Q-system is used for rock mass classification, the support systems provided differ from the standard of the system i.e. supports which are not recommended in the original classification is used for strengthening the weak rocks of the regions. A comparison has been made for the supports used in the headrace tunnel (HRT) Sanjen Hydropower Project (Table 1). The HRT has a dimension of 3.5x3.75m with a length of 3629m and the geology is dominated by four different types of rocks: Graphitic Schist, Dolomite, Quartzite and Psammatic Schist. The rock cover above the tunnel varies from 34.36m to 355.33m. It is to be noted that the support described in Table 1 is obtained from the support drawings of the HRT for different classes of rocks and the actual support provided in the tunnel might differ from the ones mentioned due to change of the support as required by the geology. Also, the rock classes are based on the HRT drawings and corresponding support obtained from Q-chart [33] for the defined rock class are mentioned.

Table 1. Comparison of tunnel support recommendations in actual site and standard Q-system of classification [33],[34].

Q-value	Support Category	Support Details provided in site	Support Details according to the Q-system
>4	I	Spot Bolting	Spot Bolting
1-4	II	S(fr) of 50mm with pattern bolting at 1.8m c/c	S(fr) of 50-60mm + Bolting at 1.7 to 2.1m c/c
0.5-1	III	S(fr) of 50mm with pattern bolting at 1.6m c/c	S(fr) of 60-90mm + Bolting at 1.5 to 1.7m c/c

0.1-0.5	IV	S(fr) of 100mm and 50mm at crown and wall respectively and pattern bolting at 1.4m c/c	S(fr) of 90-120mm + Bolting at 1.3 to 1.5m c/c
0.01-0.1	V	S(fr) of 150mm and 100mm at crown and walls respectively, pattern bolting at 1.2m c/c and concrete lining of 300mm thickness	S(fr) of 90-150mm + Bolting at 1 to 1.3m c/c + RRS (I or II)
<0.01	VI	S(fr) of 150mm, pattern bolting at 1m c/c, steel sets at 1m c/c with concrete lining of 300mm thickness	S(fr) of 150mm + Bolting at 1m c/c + RRS (II or III)

Note:

i) The rock bolts provided in the site are of 20mm diameter and 2 m length while the length of rock bolts in Q-system are given in the actual chart or can be obtained from the relation provided in Barton et al. (1997).

ii) S(fr) refers to fibre reinforced shotcrete

2.2. Semi-empirical method

Hoek and Marinos provided semi empirical relation for estimating tunnel strain using curve fitting technique [16]. This method assumes a circular tunnel geometry in hydrostatic stress field and the analysis is based on the simple closed-form solution. The support is considered to act uniformly over the entire boundary of the underground opening. This method gives a good first estimate of potential tunneling problems due to squeezing conditions in weak rock at significant depth below surface. Hoek and Marinos gave a chart based on Equation (4) and (5) which can be used to classify the tunnel sections in five different categories (Figure 2) based on the strain (tunnel deformation/tunnel radius) value.

$$Strain\ (\varepsilon) = \left(0.002 - 0.025 * \left(\frac{P_i}{P}\right)\right) * \left(\frac{\sigma_{cm}}{P}\right)^{\left[2.4*\frac{Pi}{P}-2\right]} \quad (4)$$

$$\sigma_{cm} = (0.0034 * m_i{}^{0.8}) * \sigma_{ci}(1.029 + 0.025 * e^{-0.1m_i}) * GSI \quad (5)$$

Where, Strain is the obtained with respect to diameter of the underground opening; P_i is the internal support pressure; P is the in-situ stress; σ_{cm} is the Uniaxial Compressive Strength of the Rock mass; m_i is the rock mass constant defined by friction characteristics of rock mass; σ_{ci} is the Uniaxial Compressive Strength of Intact Rock mass and GSI is Geological Strength Index.

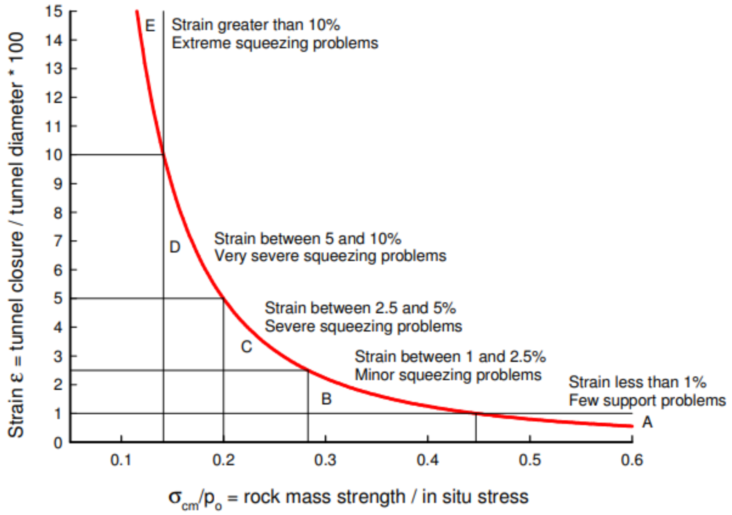

Figure 2. Semi-empirical method for classifying rocks based on strain values [16].

2.3. Analytical Method
The analytical approach addresses the nature of interplay between the rock mass that may vary and the installed support, and the effect of variation in assumed rock properties on the support load. The analytical method uses three distinct curves to study the interplay between the ground and support system. The Convergence-Confinement Method (CCM) is the most widely used method for analytical analysis of underground excavation [35]. The CCM method comprises of three curves: Ground Reaction Curve (GRC), Longitudinal Displacement Profile (LDP) and Support Characteristic Curve (SCC).

This method assumes a circular tunnel of radius R subjected to a uniform isotropic field stress σ_o. Carranza-Torres and Fairhust [35] have explained the effect of tunnel face for the stability of section as the support does not carry the full load of the earth pressure where it is installed. Some part of the load is carried by the tunnel face and as the face moves farther from the section under consideration, the load on the support increases. When a tunnel face is about twice the tunnel diameter far from the section considered, only then the maximum deformation occurs in the section. Similarly, the deformation is zero not on the face of the tunnel but about twice diameter of tunnel ahead of the face. This is represented by a curve known as the Longitudinal Deformation Profile (LDP). GRC shows the nature of the ground as the tunnel is excavated. Initially the ground is in elastic state and as the tunnel is excavated, its nature changes to plastic. The SCC on the other hand defines the capacity of the support installed in the section. The interaction of the GRC and the SCC gives the amount of pressure that the support must bear and the deformation that occurs during the installation and after the support has taken the full load imposed over it.

CCM is originally developed for a circular tunnel subjected to hydrostatic stress, the method only gives a preliminary idea about the excavation behavior and must not be fully relied upon for the final design of the tunnel. Vlachopoulos and Diederichs have provided an improved method for the calculation of the LDP which relates the displacement to the normalized plastic radius of the tunnel (plastic radius/tunnel radius) [36]. This improved LDP which was also initially proposed for circular excavation, can be well used for non-circular case provided the aspect ratio of the tunnel is small [37].

2.4. Numerical Modelling
Numerical modelling utilizes computing power and, using various modelling techniques, can be a precise way of solving very complex problems, however, its use is very limited during the design of underground structures in the Himalayas. Only recently have the modelling techniques been put in practice. Due to the dependency on empirical, analytic methods and experiences, heavy support is provided in the tunnels of Himalayas. This has a negative effect of the cost and time consumption of the project.

The precision of numerical modelling lies in the accuracy of the input parameters used to produce the model. So, it is very important to obtain precise data on the properties of the rock mass during preliminary investigation. With the help of modelling, we can obtain a safe and optimum support for every support class. The rock failure mechanism also plays an important role during modelling of underground spaces. Hence, it is important to identify whether the rock fails by Elastic Plastic or by Strain softening method. The failure mechanism has an effect on the residual values of the rock. Khakda has provided a method of selecting this residual GSI value during the modelling based on case studies from the Himalayan region (Table 2) [4]. These values have been used by Karki et al. for modelling of another case study and thus the values can be used with confidence during modelling of the Himalayan tunnels and underground spaces [3].

Numerical modelling must be implemented from the investigation stage of the project to obtain an accurate image of the ground behavior and the support requirement. The results must also be checked against the results of empirical method to confirm the accuracy of the generated models.

Table 2. Selection of residual GSI values [4].

GSI	Rock mass quality	Remarks
20<GSI<30	Extremely poor rock mass, Highly Jointed and weathered rocks	No reduction in peak GSI
30<GSI<50	Very poor to Poor, Moderately Jointed and weathered rocks	Reduce between 60 and 70% of peak GSI
50<GSI<65	Fair to good rock	Reduce between 40 and 50% of peak GSI

3. Tunnel Excavation Methods

Tunnel excavation is a challenging task as there are many uncertainties associated with the ground. For safe and economic construction of underground spaces, all the factors like ground stress, tectonic stress, groundwater, etc. must be well considered. Excavation in the ground has been practices from an early time and with time, the mechanics and approaches for construction has been improving. Currently, there are two distinct tunneling approaches: The New Austrian Tunnelling Method (NATM) and the Norwegian Method of Tunnelling (NMT). These methods have been developed from the past construction experiences addressing the challenges and problems associated with tunneling.

3.1. New Austrian Tunnelling Method (NATM)

The New Austrian Tunnelling Method is an improvement in the Austrian Tunnelling Method (old) where the excavation was performed in multiple drifts and which used heavy support of timber, masonry walls and dry rocks. The method was pioneered by L. von Rabcewicz, L. Müller and F. Pacher from 1957 to 1964. According to Muller, "NATM is rather a tunneling concept than a method, with a set of principles, which the tunneller tries to follow" [38]. NATM is an empirical tunnel construction approach which involves the whole sequence of tunneling aspects from investigation during design, engineering and contacting to construction and monitoring. The method was developed for weak grounds which utilizes the surrounding rock mass strength in supporting the excavation [39]. To achieve this, initial support comprising of rock bolts and sprayed concrete (shotcrete) are provided in the excavation.

The introduction of rock bolts and shotcrete is the main characteristic of the NATM. The shotcrete and bolts are provided as initial support to the excavation which are later closed with final lining of concrete (steel sets are also provided in very weak sections). In case of very weak grounds, closing of invert is also done while providing the initial shotcrete. NATM suggests carrying out the excavation in full face where the openings can be stabilized with initial support and in case of very weak section with very little stand-up time, sequential excavation should be done with heading-benching or multiple drifts. Since NATM is a 'Build as you Go' approach, instrumentation and monitoring is an important aspect of the method [39]. NATM classifies the rock qualitatively and the support are provided based on the description of the rock, so it is important to monitor the changes occurring in the opening so further excavation and support works can be adjusted according to the observations.

3.2. Norwegian Tunnelling Method (NMT)

NATM was primarily developed for tunneling in weak grounds where a smooth profile can be obtained during the excavation. This posed a problem while tunneling in hard, foliated rocks which was subjected to overbreak during tunneling. Norwegian Method of Tunnelling (NMT) was developed by Barton et al. (1992) in Norway [13] to address the problem of overbreak during the tunneling. The combination of systematic bolting and wet process steel fibre reinforced shotcrete constitute the main feature of the tunneling process [5],[14]. NMT is suitable for hard, foliated rocks due to the use of more flexible support i.e. steel fibre reinforced shotcrete (SFRS) [S(fr)]. Bolting is the dominant form of rock support since it mobilized the strength of the surrounding rock mass in the best way possible and S(fr) is used to supplement the rock bolts in potentially unstable rock mass with clay filled joints and discontinuities [5]. The advantage of using S(fr) also lies in zero tensile bending stress even in an irregular shotcrete lining when a good bond between the rock and shotcrete forms. Also, shotcrete can be applied in several layers without worrying about the bonding between subsequent layers. Another advantage of S(fr) is that fiber corrosion and loss of reinforcing effect is not seen when they crack and it can be applied of same thickness without regards to the irregular surface which makes it more flexible and economic than the wire mesh shotcrete [41].

Another aspect of NMT is the use of Q-system for rock classification and rock support. The Q-value (Equation 6) gives a quantitative description of the rock mass which aids in determining the rock support [12]. For determining the rock support, safety requirement and dimension of the opening are also required along with the Q-value. The safety requirement is given by the Excavation Support Ratio (ESR) value. Barton et al. (1992) have provided a chart for determining the rock class and the support requirement. It should also be noted that the Q-system does not recommend the use of concrete lining or steel ribs, instead it suggests using Reinforced Ribs of Shotcrete (RRS) for very weak sections [33].

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} \tag{6}$$

Where RQD = Rock Quality Designation, J_n = Joint set number, J_r = Joint roughness number, J_n = Joint alteration number, J_w = Joint water reduction factor and SRF = Stress Reduction Factor

The difference between the two methods have been highlighted in **Table 3**. It is to be noted that the difference have been summarized from various authors, inclusive of [12], [13], [14], [38], [39], [41].

Table 3. Major differences between the two methods of tunnelling.

New Austrian Tunnelling Method (NATM)	Norwegian Method of Tunnelling (NMT)
• Suitable for weak rocks where a smooth profile can be obtained after excavation	• Suitable for hard, highly foliated rocks subjected to overbreak
• Gives qualitative description of rocks	• Gives quantitative description of rocks
• Both initial and final support are required	• Initial support forms a part of permanent (final) support and for good rocks, initial support can suffice
• Use of wire mesh shotcrete	• Use of steel fibre reinforced shotcrete
• Pre investigation not as important as in NMT	• Pre-investigation plays an important role and must be done carefully
• Concrete lining is used for final support along with steel sets for very weak sections	• No provision of concrete lining or steel sets, instead RRS is used for very weak sections
• Monitoring during construction must be done to obtain a clear picture of the rock and to adjust further design and construction	• Monitoring is not as important as in NATM

4. Construction practice in the Himalayas

The excavation of tunnels and underground sections in the Himalayas follows the NMT method where the rock classification is done by Q-system and the design of supports are based on the same. Comparatively, NMT is also better suited for the region than NAMT because of presence of hard rocks which are highly foliated. Although the support is designed based on the Q-system, as the rock mass gets weaker, engineers make use of steel sets and concrete lining contrary to the standard Q guidelines (Table 1) (Figure 4). This is due to fact that, even though the geology comprises of hard rocks, the rock mass is weak due to the high rock cover. Despite being hard, the strength of the rocks fails to withstand the high overburden pressure, thus heavy supports must be provided in the region. Also, there is a practice of providing concrete lining as a final support for every kind of hydropower tunnel.

The tunnel excavation in the Himalayas is done by drill and blast method which disturbs the rock mass surrounding the excavation. As there is limited practice of controlled blasting, a large portion of the surrounding rock mass gets disturbed thus demanding a heavier support.

The practice of defining support in the Himalayan region involves determining preliminary supports from pre-investigation of the geology of the proposed tunnel alignment and its surrounding area. This is followed by detailed geological investigation to obtain the final design of the tunnel supports. Later during the construction of the project, face mapping is done to obtain an accurate data on the geology and the changes in the support are done as required. Khadka has summarized the process in the form of flowchart provided in Figure 3 [4].

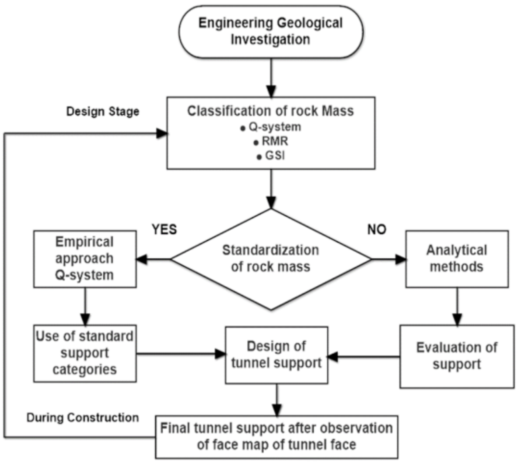

Figure 3. Design Process for estimation of tunnel support [4].

Figure 4. Supports provided in the HRT of Super Madi HEP (4.2m diameter tunnel).

During the final design of the tunnel supports, supports are designed for each class of rocks. Firstly, the results of pre-investigation are used to identify the range of expected rock conditions along the tunnel alignment and based on its results, the ground is classified into a number of classes. Then a support is designed for each class such that it can withstand the extreme condition in the specified class. This process of designing tunnel supports is more or less the same for all the tunnels excavated with drill and blast methods [41]. Singh & Goel recommends using the relation provided by Bhasin et al. (Equation 7) to estimate the average Q-value (Q_{av}) which is then used with the Q-chart to define the support system for the rock class (this method is especially useful for designing supports in the neighboring of the shear zone) [5],[42].

$$\log Q_m = \frac{b.\log Q_{wz} + \log Q_{sr}}{b+1} \tag{7}$$

Where, Q_{av} is the mean value of rock mass quality for finding the support pressure, Q_{wz} is the Q value of the weak zone/shear zone, Q_{sr} is the Q value of the surrounding rock and b is the breadth of the weak zone in meter.

The entire sequence of tunneling in the Himalayas involves sequentially the steps of fixing alignment, drilling, charging, blasting, mucking/hauling, rock bolting and shotcreting and finally providing final support in the form of concrete lining and steel ribs for very weak sections.

From the past experiences from the tunneling in the Himalayas, it is clear that the empirically established Rock Mass Classification technique is insufficient for the region [1],[3]. Only relaying on these methods can cause huge loss for the project. It is thus important to realize the importance of geotechnical studies and proper analysis must be done before the execution of project. Geological considerations, hydrological conditions, in-situ stress, tunnel dimension, support elements and construction methodology are the prerequisites for successful tunnel design and these parameters must be carefully measured during the investigation stage so as to obtain more precise estimate of the support system. Since experiences have a lot of positive impacts on facing uncertainties, it is important for engineers and contractors to realize the value of recording the problems and its associated remedies undertaken.

5. Conclusions

The major geological risks associated with tunneling in the Himalayas include weak rock conditions, rock bursting, heavy water ingress and geothermic heat. These are aggravated by heavy rock cover over the underground structure and frequent seismic activities in the region. During design of the underground structures, such factors must be considered for safe and economic design. The Rock Mass Classification is inadequate for the Himalayan geology, thus numerical modelling must be done to obtain a clearer picture of the geology and support system designed according to it. Proper monitoring and instrumentation of the project must be done during construction. Preparation of database of recorded properties before, during and after construction will aid in the future projects as well saving time and money for preliminary data collection.

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