Assessment and Numerical Analysis of Hydropower Tunnel in Lesser Himalayan Region of Nepal- A Case Study

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Abstract: This paper is focused on the assessment and numerical analysis of hydropower tunnel of the Lesser Himalayan Region of Nepal. This region lies between two major faults namely Main Boundary Thrust (MBT) and the Main Central Thrust (MCT) with weak rock mass like phyllite, schist, gneiss, phyllitic schist, etc. Thus, to overcome the stability problems during underground construction, proper rock support system must be installed. Rock mass classification systems are commonly used for estimating the rock support system in this region, but this approach is inadequate to address the underground stability problems. In this study, numerical analysis is done to define the requirement of support and the result compared to actual support provided in selected case study. Analytical approach is used along with two-dimensional Finite Element Analysis using the software, RS² provided by RocScience for the study. Finally, required modification of the provided support has been suggested to overcome the problem faced in the selected tunnel.

Keywords: Squeezing, Rock mass classification, Finite Element Analysis, rock support

1. Introduction
Due to steep terrain and fast flowing rivers in the Lesser Himalayan region of Nepal, medium to mega size hydropower projects are constructing day-by-day. The availability of high head for hydropower generation the tunnel cross sections are relatively small, up to 6 m diameter in size. Excessive tunnel deformation and support failure were encountered in this region during the construction as it passes through very weak rock masses with high overburden pressure. The selection of tunnel supports is carried out by empirical methods, basically rock mass classification approaches. Among them, Rock Mass Rating (RMR) by Bieniawski (1984) and Q-system rock classification proposed by Barton et al. (1974) are mostly used in the Himalayan region to prescribe tunnel support. Drill and blast is a common method for construction of the tunnel in this region. During blasting, the surrounding rock mass gets disturbed which is not considered in rock mass classification approach. This paper focuses on the analysis and design of tunnel support by using empirical, analytical and numerical methods. In this study, Middle Bhotekoshi Hydropower tunnel was selected for the case study, which is an under construction project located in the Lesser Himalayan region of Nepal.

2. Design Overview of Hydropower Tunnel in Lesser Himalayan Region of Nepal
In this section, the general design approaches which are commonly used to design or prescribe the support system in the lesser Himalayan region of Nepal is discussed focusing on the Middle Bhotekoshi hydropower tunnel.
2.1 The Project
Middle Bhotekoshi Hydroelectric Project (MBKHEP) is a run-of-river project located in Sindhupalchowk District of Bagmati Zone of Nepal with a capacity of 102 mega-watts. The gross head of the project is 235 m with a net head of 222 m. Headrace tunnel is one of the major structure of the project with a total length of 7124 m from inlet to surge tank. The tunnel is of inverted D shape cross-section with a dimension of 5.7 m x 5.7 m. Geologically, it is located in the Lesser Himalayan Region in the Northern part of Nepal (Figure 1). The main rock types along the alignment of headrace tunnel are quartzite, dolomite, talcose phylite and phylitic schist. Up-to chainage 50 m good quality of quartzite is present. From chainage 50 to 1120 m dolomite is present and quartzite from 1120 m to 2120 m. Poor rock mass phyllitic schist is present from chainage 2420 m to 7198 m, end of headrace tunnel. [5].

![Figure 1. Geological map showing the location of selected case study [4].](image)

2.2 Rock Mass Classification-empirical approach
Rock mass classification approach is the first stage of design and analysis of tunnel support in the Himalayan region of Nepal. It gives the idea of tunnel support and cost estimation of the project. For the estimation of tunnel support, geological investigation is the first step which is followed by classification of the rock mass. The rock mass classification can be done by Rock Mass Rating (RMR) approach, Rock Mass Quality Index (Q-System) and the Geological Strength Index (GSI). The RMR and the Q-system also provides an estimate of the rock support to be installed based on the rock quality while the GSI only classifies the rock mass without giving an estimate of rock support. Standardization of rock mass is then performed and the results compared to that obtained from the classification. Empirical analysis and analytical analysis is done based on the results of standardization and rock classification. Finally, the tunnel support is designed followed by face mapping of tunnel face to obtain the field parameters. These parameters are again used to classify the rock mass and design of support in a similar manner. For the selected case study, the classification approach adopted was RMR classification.

2.2.1 Rock Mass Rating
Bieniawski (1974) developed the Rock Mass Rating (RMR) on the basis of his experiences in shallow tunnels in sedimentary rocks [10]. Since the introduction of RMR, this system has undergone several evolutions in 1975, 1976, 1979 and 1984. The RMR classification commonly used is of 1979 where the ISRM (1978) rock mass description was adopted. Any reference to RMR in this paper will imply to RMR1979. RMR method of classification uses six parameters for classifying the rock mass, Uniaxial Compressive Strength (UCS), Rock Quality Designation (RQD), Joint or discontinuity spacing, Joint condition, Groundwater condition and Joint orientation. Certain rating is provided for each parameter
based on the condition of the rock. Details of the rating can be found in [10]. The sum of all the ratings gives the final value of RMR according to which the rock mass can be classified from very good to very poor. Along with the classification, guideline for estimating the rock support has also been provided. For the tunnel sections of the case study, the rock masses have been classified from R-I to R-V (for very good to very poor quality rock mass respectively) based on the rock quality and the support requirement [5]. The classification has been provided in Table 1.

3. Stability Assessment

For estimation of deformation, the relation provided by Panthi and Shrestha [6] incorporates the stress anisotropy of the tunnels. Since, anisotropy has been the reality in most of the tunnel cases, the same relation has been used for this study. Panthi and Shrestha [7], on their study of 24 tunnels sections representing four different rock types of three hydropower tunnels located in the Lesser Himalayan region of Nepal have developed the following relation for instantaneous and final closure of the tunnel section as given in Equation (1) and Equation (2) respectively.

$$\varepsilon_{IC} = \frac{3065 \times \left( \sigma_v \times (1 + k)/2 \right)^{2.13}}{2G \times (1 + P_i)}$$  \hspace{1cm} (1)

$$\varepsilon_{FC} = \frac{4509 \times \left( \sigma_v \times (1 + k)/2 \right)^{2.09}}{2G \times (1 + P_i)}$$  \hspace{1cm} (2)

Where $\varepsilon_{IC}$ and $\varepsilon_{FC}$ represent instantaneous and final closure in mm, $P_i$ is the support pressure in MPa, $G$ is the shear modulus and $\sigma_v$ is the vertical pressure both in MPa and k is the in-situ stress ratio.

| Tunnel Chainage | H (m) | RMR | Rock mass | Support class | $\sigma_v$ (MPa) | $\sigma_h$ (MPa) | G (MPa) | $P_i$ (MPa) | Closure (%) |
|-----------------|------|-----|-----------|---------------|----------------|----------------|---------|-----------|-------------|
| 2886.6          | 523.56 | 51  | Fair      | R-III         | 14.14          | 4.57           | 698.99  | 1.45      | 0.19        |
| 2912.2          | 505.93 | 52  | Fair      | R-III         | 13.66          | 4.52           | 749.78  | 1.45      | 0.15        |
| 3171.7          | 357.23 | 66  | Good      | R-II          | 9.65           | 4.07           | 1911.56 | 0.71      | 0.02        |
| 3190            | 355.13 | 30  | Poor      | R-IV          | 9.59           | 4.07           | 158.03  | 2.59      | 1.00        |
| 3248.2          | 347.1  | 36  | Poor      | R-IV          | 9.37           | 4.04           | 240.78  | 2.59      | 0.39        |
| 3264.4          | 344.98 | 34  | Poor      | R-IV          | 9.31           | 4.03           | 209.07  | 2.59      | 0.53        |
| 3283.2          | 339.85 | 62  | Good      | R-II          | 9.18           | 4.02           | 1480.09 | 0.71      | 0.04        |
| 3297.5          | 338.14 | 63  | Good      | R-II          | 9.13           | 4.01           | 1579.56 | 0.71      | 0.03        |
| 3756            | 201.61 | 58  | Fair      | R-III         | 5.44           | 3.60           | 1134.00 | 1.45      | 0.01        |
| 3793.4          | 205.56 | 39  | Poor      | R-IV          | 5.55           | 3.62           | 297.94  | 2.59      | 0.11        |
| 4002.6          | 174.22 | 44  | Fair      | R-IV          | 4.70           | 3.52           | 425.51  | 1.45      | 0.09        |
| 4011.8          | 170.92 | 45  | Fair      | R-III         | 4.61           | 3.51           | 456.94  | 1.45      | 0.08        |
| 4017.5          | 168.38 | 47  | Fair      | R-III         | 4.55           | 3.51           | 526.81  | 1.45      | 0.06        |
| 4028.5          | 163.35 | 50  | Fair      | R-III         | 4.41           | 3.49           | 651.47  | 1.45      | 0.03        |
| 6772            | 161.45 | 19  | Very poor | R-V           | 4.36           | 3.48           | 75.24   | 2.49      | 1.59        |
| 6777.3          | 158.49 | 17  | Very poor | R-V           | 4.28           | 3.48           | 66.12   | 2.49      | 2.04        |
| 6856.6          | 115.19 | 39  | Poor      | R-IV          | 3.11           | 3.35           | 297.94  | 2.59      | 0.05        |
| 6867.5          | 112.55 | 22  | Very poor | R-V           | 3.04           | 3.34           | 91.66   | 2.59      | 0.63        |
| 7082            | 58.86  | 41  | Fair      | R-III         | 1.59           | 3.18           | 343.56  | 1.45      | 0.05        |
| 7106.5          | 52.49  | 49  | Fair      | R-III         | 1.42           | 3.16           | 607.05  | 1.45      | 0.01        |
| 7124.8          | 53.19  | 44  | Fair      | R-III         | 1.44           | 3.16           | 425.51  | 1.45      | 0.03        |

Note: $H$=Overburden, RMR=Rock Mass Rating, $\sigma_v$= Vertical stress, $\sigma_h$= Horizontal stress, $\varepsilon_{IC}$= instantaneous closure, $\varepsilon_{FC}$= final closure, $G$= Shear Modulus, $P_i$= Support pressure

The deformation values for tunnel sections have been plotted in Figure 2. The obtained deformations were then classified as very mild (closure 1-2%), mild (closure 2-3%), mild to moderate (closure 3-4%), moderate (closure 4-5%), high (closure 5-7%) and very high squeezing (closure >7%) as defined by Singh and Goel [10]. The details of the analysis are given in Table 1. All the provided sections had Phyllitic Schist rock and among the sections two sections suffered mild squeezing and
two of them suffered high squeezing. All the other sections had closure less than 2% i.e. very mild squeezing.

![Figure 2](image)

**Figure 2.** Comparison of Convergence for different support conditions.

From the available sections, four sections (2886.6, 3190, 3283.2 and 6867.5), each representing different rock class were selected for the analysis. Figure 3 shows the selected sections and the support system required for the rock class as defined the rock mass rating guidelines.

![Figure 3](image)

**Figure 3.** Designed rock support system of tunnel at different chainage as per RMR.

### 4. Rock-Support interaction - analytical assessment

The rock support interaction curves, suggested by Carranza-Torres and Fairhurst [12] is one of the simple and useful method to design tunnel support. It can be drawn by assuming the tunnel is circular with radius R through a rock mass that is to be subject initially to a uniform field far field stress $\sigma_0$ as shown in Figure 4 (b). For the simplicity, it is assumed that all the deformation occurs in a plane perpendicular to the axis of the tunnel. Radial displacement $u_r$ and internal pressure $p_i$ i.e. the reaction of support on the walls of the tunnel are uniform at the section. Figure 4 (c) shows that the circular annular support of thickness $t_c$ and external radius R is installed at the section A-A'$. The pressure $P_s$ represent uniform load transmitted by the rock - mass of the support.
Figure 4. a) Cylindrical tunnel of radius R driven in the rock mass. b) cross section of rock mass at section A-A'. c) Cross-section of the circular support installed at section A-A' [12].

If the support is installed immediately in the vicinity of the face, it does not carry the full load which it is supposed to as some part of the load is carried by the face itself, this is known as the face effect. This means that when the excavation proceeds further, the face effect decreases and the load imposed to the support increases gradually. Based on these, the CCM can be represented by three curves: Longitudinal Displacement Profile (LDP), Ground Reaction Curve (GRC) and the Support Characteristics Curve (SCC). Although this approach assumes the case of an isotropic circular tunnel which is seldom in practice, this method is used as a first step in the design of underground structures. Analytical solutions assume isostatic vertical stress conditions which overestimates the support pressure and only relying on this approach will lead to increasing cost of the project. So this method should only be used as a tool for study and not be relied upon for final design. The LDP (Figure 5), GRC and SCC plot (Figure 6) was generated for the selected tunnel sections. From Figure 5 it is clear that the deformation of tunnel occurs even ahead of tunnel face where excavation has not yet occurred. From the plot, the deformation stops completely at a distance of around twice the diameter ahead of tunnel face and the maximum deformation occurs at twice the diameter behind the tunnel face.

The support pressure, which the provided support must bear is shown in Figure 7 and the deformation at that particular sections is shown in Figure 8. It can be seen that the support pressure for chainage 2886.6m is the highest when the deformation is very low. This is due to the fact that analytical approach determines the support pressure on the basis of the rock mass cover. Since the rock cover is highest in the section, the obtained support pressure is also very high. The support pressure for designed support and support according to the RMR guidelines has been compared in the graph of Figure 7. It is clear from the figure that the support provided in the project is robust than required. In Figure 8, the deformation obtained from empirical calculation and the deformation obtained from analytical analysis is compared. The results are almost similar, the difference in the results is due to the assumptions in analytical analysis.
5. Numerical Analysis

5.1 Estimation of rock mass parameters

Rock mass parameters such as Hoek-Brown constants, Modular Ratio (MR), Uniaxial Compressive Strength (UCS), etc. are very much essential while conducting numerical analysis. Due to this, the significance of the rock mass parameters has increased during the past decades and there have been numerous studies on the subject [9]. Generalized Hoek-Brown strength criterion is used for this study, which obtained from slight modification to the original Hoek-Brown criteria. This criterion is an empirical failure criterion which establishes the strength of rock in terms of major and minor principal stresses and is expressed as:

\[
\sigma_1 = \sigma_3 + \sigma_{ci} \left( m_b \sigma_3 + s \right)^a
\]

(3)

Where \(\sigma_1\) and \(\sigma_3\) are the axial (major) and confining (minor) effective principal stresses respectively, \(\sigma_{ci}\) is the Uniaxial Compressive Strength (UCS) of the intact rock material, \(m_b\) is a reduced value (for the rock mass) of the material constant \(m_i\) (for the intact rock), \(s\) and \(a\) are constants which depend upon the characteristics of the rock mass. Hoek et al. [3] have provided separate empirical relations to estimate these rock constants. The GSI here refers to the Geological Strength Index and is calculated by decreasing 5 points from the RMR value [2].

\[
GSI = RMR - 5
\]

(4)

Rock mass strength is estimated by Equation (5) as suggested by the Hoek et al. [3].

\[
\sigma_{cm} = \sigma_{ci} \frac{(m_b + 4s - a(m_b - 8s))(\frac{m_b + 4s}{4} + s)^{a-1}}{2(1 + a)(2 + a)}
\]

(5)

Where, \(\sigma_{cm}\) is the unconfined compressive strength of rock mass in MPa, \(\sigma_{ci}\) is the uniaxial compressive strength of intact rock in MPa.

Previous studies show that the density of weak rock is taken as 2.7 gm/cm\(^3\) and the Poisson’s ratio as 0.1 for the analysis of the hydropower tunnels in the Himalayan region [4],[6].

For vertical stress Equation (6) is used while for horizontal stress Equation (7) is used where tech refers to the tectonic stress which is taken as 3MPa as stated in [6].

\[
\sigma_v = \gamma * H
\]

(6)

\[
\sigma_h = \frac{1}{1 - \nu} \sigma_v + \sigma_{tech}
\]

(7)

During the field visit of case study, it is found that the UCS of Phyllite was obtained as 34.5 MPa, which is closed to the value from laboratory testing of Phyllite from Kaligandaki hydropower project as stated by Panthi [6]. The rock mass modulus was calculated by the following equation [3].

\[
E_{rm} (GPa) = \left(1 - \frac{D}{2}\right) \sqrt{\frac{100 \sigma_{ci}^{651-10}}{100}}
\]

(8)

Table 2. Estimation of rock mass properties for Numerical Analysis

| Chainage (m) | H (m) | GSI | \(\gamma\) | \(\nu\) | \(m_i\) | \(\sigma_{ci}\) | \(E_{rm}\) | \(\sigma_v\) | \(k\) | \(\sigma_h\) |
5.2 Numerical Modelling

For numerical analysis, the finite element software, RS2 developed by RocScience has been used in this study. RS2 is a two dimensional hybrid finite-element/boundary element program for underground excavations in rock [8]. It provides most convenient way to model the underground structures where non linearity can occur close to excavation boundary, and elastic behavior is observed far from the boundary. The nearest section is thus modelled using a finite element method which is then linked at its outer boundary to a boundary element method [9]. In this study, the rock mass has been modelled for Generalized Hoek Brown failure criterion by developing a plain strain model that relaxes the internal pressure of the excavation from a value equal to in-situ stress to zero. Anisotropic stress condition is assumed and the rock mass modelled using strain softening and elastic completely plastic post-failure for good and poor quality rock mass respectively. Khadka [4] has suggested the different rock model to analyze the rock mass in terms of peak and residual strength for the rock mass from the Himalayan region of Nepal. For extremely weak rocks with GSI values less than 30, the elastic-plastic model is used where no residual parameters are required. For weak to good quality rocks, strain softening model is used where the residual parameter is obtained by reducing the peak GSI between 60 and 70% for very poor to poor rock (30<GSI<50) while it is reduced between 40 and 50% for fair and good rocks (50<GSI<65).

A D-shaped model of the tunnel has been prepared with the excavation boundary as six times the diameter of the tunnel. The disturbance factor of 0.5 has been considered to account for the disturbances from the drill and blast method of tunneling in this region [4]. Selected four tunnel sections have been modelled and studied with and without support respectively. The closure is determined by knowing the maximum displacement during unsupported condition and the radius of plastic zone. This closure value is then used to determine the relaxation stage of the tunnel which is the stage where maximum deformation has occurred and further deformation occur at a very slow rate. The analysis is then carried out for the three stages: the initial stage, the relaxation stage and the final stage. Support is installed on the relaxation stage and the deformation is noted during supported condition. The support is provided according to the RMR guidelines.

### Table 3. Input parameters for Numerical Modelling.

| Chainage (m) | 2886.6 | 3190 | 3283.2 | 6867.5 |
|-------------|-------|------|--------|--------|
| Rock Class  | R-II  | R-III| R-I    | R-IV   |
| GSI mb      | Peak  | Residual | Peak  | Residual | Peak  | Residual | Peak  | Residual |
| 46          | 16.1  |          | 25    |          | 57    | 19.95    | 17    |          |
| 0.687       | 0.165 |          | 0.253 |          | 1.1613| 0.1989   | 0.172 |          |
| 0.007       | 1.38E-05 | 4.54E-05 | 0.0032| 2.32E-05 | 1.56E-05 |          |
| 0.5075      | 0.556 | 0.531    | 0.5035| 0.5438   | 0.553 |          |

### Table 4. Material properties of concrete and steel rib.

| Parameters                  | Unit | Concrete | Steel Rib |
|-----------------------------|------|----------|-----------|
| Young’s modulus             | GPa  | 30       | 200       |
| Poisson’s ratio             | 0.2  | 0.2      | 0.25      |
| Cross sectional area        | mm²  | 2170     |           |
| Moment of inertia           | mm⁴  | 8.39×10⁶ |           |
| Section inertia             | mm   | 0.150    |           |
5.3 Results of Numerical Analysis

The selected four tunnel sections were analyzed by numerical method for support system according to RMR guidelines. The provided support failed at bottom corners of the excavation. This is due to the accumulation of excessive stress at the corners of the excavation. The support was modified such that it could withstand all the induced stress. Concrete lining was provided in addition to the original support as defined by RMR guidelines. For poor and very poor rock mass, the section of the steel sets was also increased in order to limit the thickness of the concrete lining. Figure 9 shows the extent of failure in the surrounding rock mass before and after support installation. It can be seen that the extent of failure at poor and very poor rock mass is larger than the other. The installation of support has reduced the extent of the failure but the yield zone still lies outside the region of installed rock bolt. This shows that the support provided by the rock mass classification approach is not adequate to control the failure around the tunnel as compare to the modified support Figure 9. The support capacity plot for the modified support of two sections, fair and poor rock mass respectively, has been provided in Figure 10. It can be seen that all the points lie inside the factor of safety curve for both the section showing the validity of the support to withstand the stress. The capacity plot for other two sections also gave the same results.

| Details (m) | Before Support | After Support (RMR Guidelines) | Modified Support |
|-------------|----------------|--------------------------------|-----------------|
| Chainage: 2886.6m | Rock Type: Fair | Rock Class: R-III | Type = end anchored E=200GPA; Φ =20mm; Tensile capacity =0.1MN |
| Chainage: 3100m | Rock Type: Poor | Rock Class: R-IV | Type = end anchored E=200GPA; Φ =20mm; Tensile capacity =0.1MN |
| Chainage: 3283.2m | Rock Type: Good | Rock Class: R-II | Type = end anchored E=200GPA; Φ =20mm; Tensile capacity =0.1MN |
| Chainage: 6867.5m | Rock Type: Very Poor | Rock Class: R-V | Type = end anchored E=200GPA; Φ =20mm; Tensile capacity =0.1MN |
The deformation around the tunnel was recorded for every section. The results are shown in the graph of Figure 11. It can be seen that the deformation obtained from numerical analysis for the modified support has reduced compared to that obtained from support defined by the RMR guidelines. The modified support was also used to calculate the strain from the relation given by Panthi and Shrestha [7]. It can be seen that the strain obtained from empirical analysis for the modified support is less than that obtained from the numerical analysis Figure 11. The reason may be due to the assumption used while calculating the parameters for modelling. The parameters of modelling were also obtained from empirical relations which can cause slight deviations in the results.

Figure 9. Extent of failure of tunnel with and without support.

Figure 10. Support Capacity Plot for (a) fair rock (b) poor rock.

Figure 11. Comparison of deformation for selected tunnel sections.

6. Conclusion
In this study, analytical assessment is carried out in conjunction with numerical analysis to study the support requirement of the headrace tunnel of Middle Bhotekoshi Hydroelectric Project, located in the
Lesser Himalayan Region of Nepal. Convergence Confinement Method is used for analytical study and two dimensional finite element analysis carried out using the software RS2 for numerical analysis. Four tunnel sections, each representing different quality rock mass were used for the study among which two sections have poor quality rock mass which faced severe squeezing while the other two sections had very less squeezing due to the presence of good quality rocks despite having large rock cover.

For numerical analysis the rock mass parameters were obtained using empirical relations. The support system was designed according to RMR guidelines. This support was unable to control the deformations in the tunnel and modified support was applied by introducing concrete lining to the original support. The deformations obtained from each type of support system was calculated and it was found that the deformation due to modified support was less than the RMR guidelines’ support. The modified support was also used to calculate the strain from the empirical approach defined by Panthi and Shrestha [7]. It was found that the results were close to each other with small difference in the values which may be caused due to the assumptions used while calculating the numerical parameters. The closeness in the results showed that the modified support can safely control the deformation in the tunnel. Also the support capacity plot of the modified support, as shown in Figure 10, shows that it is able to withstand the imposed stress in the boundary of the excavation. Therefore, rock mass classification approach only is not adequate to design and estimation of tunnel support. Numerical analysis is very helpful to estimate the tunnel support in such geological region where rock masses are very poor with high rock cover.

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