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Empirical and Numerical Study for Support Estimation for Stability of Underground Opening: Case Study of Two Tunnels in Lesser Himalaya Region of Nepal

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Abstract

Tunnel stability evaluation methods are divided into three categories: empirical methods, observational methods and analytical or numerical modeling. In this study the support estimated from empirical methods (Q-system) has been analysed with Numerical Modeling. The headrace tunnel of the two projects i.e. Solu Khola (Dudh Koshi) Hydroelectric Project and Ghar Khola Hydro Power Project both lies in the lesser Himalaya Region of Nepal has been used as a case study. The support estimated from the empirical method was sufficient for achieving the stability in the circular shaped tunnel but not inverted D and horse shoe shape tunnel. For the weak rock mass and at high stress field condition is was difficult to achieve the stability in the inverted D shape. So the horse shoe shape is preferred and modification of the support from the Q-system for the stability has been suggested on the basis of Numerical Modeling.

Keywords

Q-Value, Numerical Modeling, RRS, Rock Support

1. Introduction

1.1 Background

The construction of the tunnel is being common in Nepal for the conveyance of water to generate electricity. Also planning and construction of some transportation tunnels are being carried out for the mobility of the people and traffic. In the future tunnel will play an important role in the sustainable development of hydropower and the transportation sector. There are numerous difficulties encountered during the construction of such projects, despite being attractive schemes for the development of the hydropower and transportation sector. The presence of the steep slopes in Nepal Himalaya, the presence of jointed rock mass, and sheared and weak zones formed due to the movement of the tectonic plates make the planning of the underground structures more challenging.

A little misunderstanding during the early design stages might have expensive and time-consuming effects during the construction and lifetime of the underground openings. Misunderstanding of ground conditions account for fifty three percentage of all tunnel failures worldwide. [1]. To assure safety and stability, the geotechnical and geological ground conditions must be identified correctly.

Tunnel stability evaluation methods are divided into three categories: empirical methods that based on experience, Observational method which uses the tunnel wall deformation and analytical methods which analyses the stress distribution and deformation [2]. In this study Empirical Method using Q-system and the analytical method i.e. numerical modeling for estimation of the rock support has been compared. Empirical methods use the various rock mass classification systems for classifying the rock mass to understand the typical behavior and relate the experience gained in rock conditions at one site to another for the assessment of the stability of the underground opening. Where as the stability assessment by the numerical modeling is for the particular rock mass at the site considering in-situ stress conditions and its mechanical properties rather than comparing it to the similar nature of rock mass. Comprehensive information on the rock mass parameters, in-situ stress conditions, and mechanical properties of rock masses are unavailable during the feasibility and basic design stages of a project so empirical methods are used rather than numerical modeling.

The headrace tunnel of Solu Khola (Dudh Koshi) HEP and Ghar Khola HPP both in Lesser Himalaya are used as the Case Study in this research.

1.2 Study Area

The Himalayan range is the result of the ongoing collision of the Indian and Eurasian plates, which started around 55 million years ago [3]. Five geological units in Nepal's Himalaya correspond to morphotectonic zones. Each zone is distinguished by its morphological, geological, and tectonic characteristics, and each one is divided by various geological tectonic structures that are Terai, Siwalik, Lesser Himalaya, Higher Himalaya and Tibetan tethys zone [4].

1.2.1 Solu Khola (Dudh Koshi) Hydroelectric Project (SKDHEP)

The project is located in the Solukhumbu district of Sagarmatha Zone in Nepal's Eastern Development Region. This project is located in the Lesser Himalaya of the Okhaldhunga Window. Based on the field mapping done in the project area during the investigation it was found that the project area consist of the Augen gneiss and green phyllite with quartzite bands. The tunnel section of interest used for analysis passes through gray-colored medium-coarse grained, foliated, moderately to highly weathered and disintegrated Augen gneiss with phillitic bands. The rock consists of quartz, feldspar, muscovite, and biotite as principle minerals. The joint surface is smooth, planar with a tight to open aperture with a zone of clay or disintegrated or crushed rock, medium to a low over consolidated or softening fillings. The joints have medium to high persistency. The overall RQD of the exposure is poor. Ground water condition is dripping from the left wall.

1.2.2 Ghar Khola Hydroelectric Project (GKHPP)

The project lies in Myagdi District of Gandaki zone in the Western Development Region of Nepal. This project is located in the Lesser Himalaya. The project area consists of green phyllite and white quartzite of Lesser Himalaya. The tunnel section of interest used for analysis passes through the rock mass consisting of thinly foliated, crenulated, slightly weathered, dark gray to black phyllite with quartz veins. The rock mass is highly deformed and crushed. Joint one is low to medium spacing, medium persistence of greater than one meter, slightly weathered, smooth to rough planar. Several joints are crushed which varies from low to medium persistence and slightly weathered in condition. The overall RQD of the exposure is poor.

2. Objectives

The main of objective of this research is to check the reliability of the rock support estimated for the stability of underground opening with the numerical modeling. Specific objectives are:

1. Identification of the rock mass parameters at the tunnel from the two case study projects at lesser Himalaya of Nepal.
2. Determining the mechanical properties of the rock mass from back analysis using the measured deformation at a wall of the tunnel.
3. Determining the rock support from the empirical method and analyzing it with the Numerical modeling.
4. Checking the reliability of the rock support from the empirical method for the particular rock mass condition with Numerical Modeling.

3. Methodology

The study uses the following approach to meet its Objectives:

3.1 Determination of Rock Mass Properties

The tunnel excavation was done by the drill and blast method. The Q-value was calculated during excavation in tunnel. The rock mass classification has been done based on the tunnel mapping done during the excavation. Following equation has been used to compute the Q-value based on six parameters:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \tag{1}$$

Where, RQD is Degree of Jointing (Rock Quality Designation), J_n is Joint set number, J_r is Joint roughness number, J_a is Joint alteration number, J_w is Joint water reduction factor and SRF is Stress Reduction Factor

The deformation modulus of the intact rock is very high than the jointed rock mass. Due to the unavailability of the in-situ test for determining deformation modulus in the field, the various empirical relation provided by Bieniawski [5], Hoek and Brown [6], Barton [7] and Hoek and Diederichs [8] are used.

Bedding plane, joints, folds, faults, shear zones and dykes are the major structural features of the rock mass [9]. The strength of rock mass often is dependent upon the discontinuities and the foliation or the bedding planes and the orientation of these structural features. An intact rock specimen is stronger than the rock mass and homogeneous with fewer discontinuities. So the rock mass strength is completely different from that of the intact rocks. Empirical relations developed by Bieniawski [10], Hoek [11] and Barton [7] for estimating the unconfined compressive strength of the rock mass (σ_{cm}).

3.2 Failure Criterion

Failure criteria given by Hoek – Brown is used for modelling of rock mass as suggested by Figure 1. The method is based on the estimation of rock block interlocking and shear conditions in the joints. This method was developed to assess the strength of a jointed rock mass when the rock blocks are small in comparison to the excavation [11]. The criterion has been presented in form of the equations below.

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \cdot \frac{\sigma_a}{\sigma_{ci}} + s\right)^a \tag{2}$$

Where, σ_1 and σ_3 are the maximum and minimum principal stress at failure, m_b is the Hoek-Brown constant for the rock mass, s and a are constants related to the rock mas characteristics and σ_{ci} is the uniaxial compressive strength of intact rock sample.

$$m_b = m_i + e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{3}$$

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \tag{4}$$

$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \tag{5}$$

Where, m_i is the Hoek-Brown constant for the intact rock samples, D is the disturbing factor considering the disturbance from blasting and GSI is the Geological Strength Index.

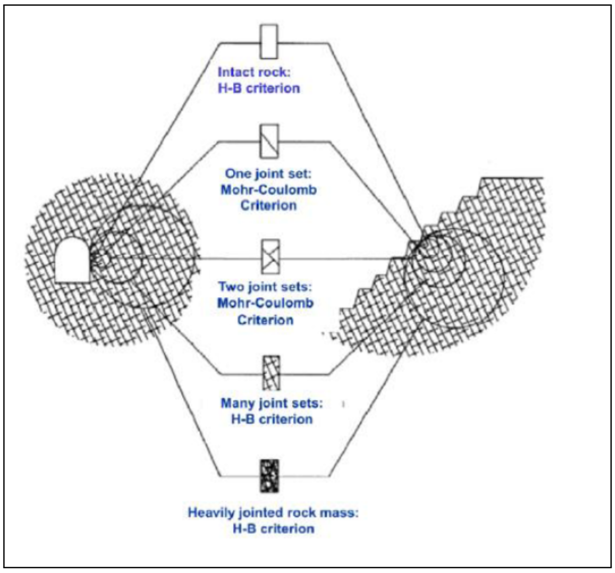

Figure 1: Selection of failure criterion according to the rock mass condition [11]

3.3 Calibration of the Model

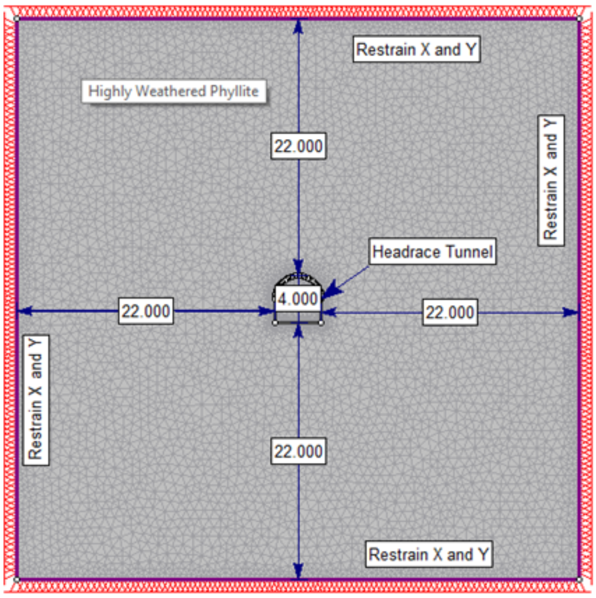

Figure 2: Model Setup of Solu Khola (Dudh Koshi) HEP tunnel in Phase2

The main objective of the calibration of the model is to analyze the model to identify the amount of deformation around the tunnel. Another objective is to use the measured deformation to calculate the rock mass parameters by back analysis [12]. The Phase2 program has been selected for the analysis. Firstly the model is created in Phase2 and the input parameters for the intact rock and rock mass are set based on the geotechnical investigation and tunnel mapping done during excavation.

3.3.1 Elastic Analysis

The material type is considered elastic, which means the rock mass acts elastically. The main goal of this analysis is to determine the strength factor around the tunnel periphery to identify whether the rock mass behaves in an elastic or plastic manner. The strength factor and total displacement around the tunnel have been examined and compared for both the supported and unsupported cases.

3.3.2 Plastic Analysis

If the material yields in the elastic analysis the plastic analysis is done on the same model to identify the amount of plastic deformation in the tunnel. The deformation achieved from the Numerical Modeling has been compared with the measured value. If there is any discrepancy between calculated and measured deformation, the rock mass parameters are redefined. The deformation was measured after the installation of the support. So the results should be compared to those obtained after applying for support in the Numerical Modeling.

3.4 Semi Analytical Method for Deformation

3.4.1 Hoek and Marinos Method

Hoek and Marinos method is one of the most common semi-analytical methods used for analyzing plastic deformation. In this method the relation between percentage strain (ε) and the competence factor (σ_{cm}/p_o) has been suggested [13]. The simulation is carried out under unsupported conditions, which means there is no internal pressure (p_i), for which the strain value can be calculated. Further internal pressure (p_i) was introduced to simulate the effect of support [13].

3.5 Analytical Method for Deformation

3.5.1 Convergence Confinement Method (CCM)

The convergence confinement method is an analytical method for the calculation of the deformation in a tunnel and the construction of ground support interaction. CCM has three components that are Longitudinal Displacement Profile (LDP), Ground Reaction Curve (GRC), and Support Characteristics Curve (SCC). Relations proposed by Carranza-Torres and Fairhurst [14] have been used to construct GRC, LDP and SCC.

3.6 Support estimation using Empirical Methods

3.6.1 Support Estimation using Q-chart

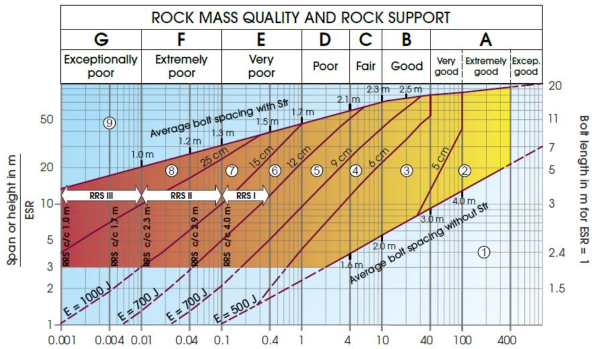

Figure 3: Permanent support recommendation based on Q-value and span/ESR (NGI, 2015)

The tunnel mapping has been done during the excavation of the tunnel based on the Q-system which was developed by the Norwegian Geotechnical Institute (NGI). Q-value calculated during the tunnel mapping is used to estimate the support using the empirical method. The three factors i.e. rock mass quality (Q-value), span or height of the opening, and safety requirement (ESR) are decisive for the estimation of support. The support chart shows at what distance the rock bolts are spaced and how thick the sprayed concrete is to be applied.

3.7 Support design using Numerical Modeling

The estimated support from the chart has been analysed using Numerical Modeling using the Phase2 program. A plane model of the tunnel is built to determine the deformation far from the face of the tunnel and the radius of the plastic zone. The amount of deformation in the wall of the tunnel before the support installation is determined. Then the support is added and whether the tunnel is stable, the deformation meets the specified requirements and the tunnel lining meets certain factors of safety requirements been identified.

4. Results and Discussion

4.1 Calibration of the Model

4.1.1 Elastic Analysis

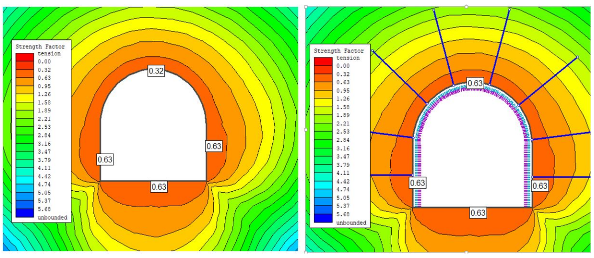

Figure 4: Strength factor before (left) and after (right) application of support at chainage 2+068 of SKDHEP

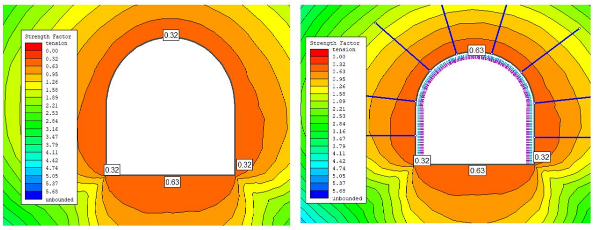

Figure 5: Strength factor before (left) and after (right) application of support at chainage 2+077 of SKDHEP

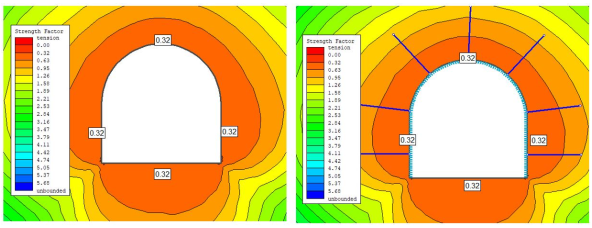

Figure 6: Strength factor before (left) and after (right) application of support at chainage 0+403 of GKHPP

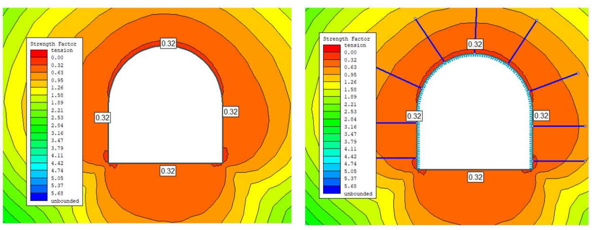

Figure 7: Strength factor before (left) and after (right) application of support at chainage 0+417 of GKHPP

From the elastic analysis for with and without support, the strength factor is obtained as less than one which indicates that the respective rock mass does not behave elastically Hence, plastic analysis is required. Also, the displacement of the tunnel wall closure is much less compared to the measured value.

4.1.2 Plastic Analysis

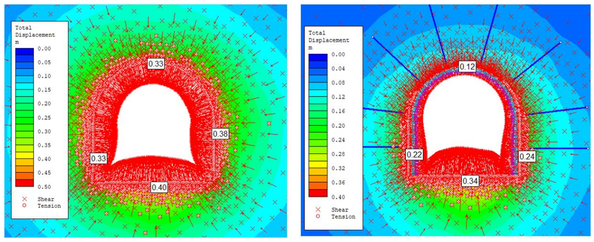

Figure 8: Deformation obtained from the plastic analysis without (left) and with (right) application of support at chainage 2+068 of SKDHEP

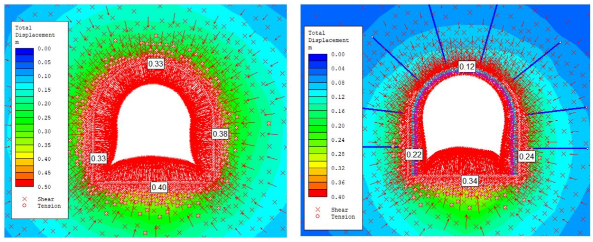

Figure 9: Deformation obtained from the plastic analysis without (left) and with (right) application of support at chainage 2+077 of SKDHEP

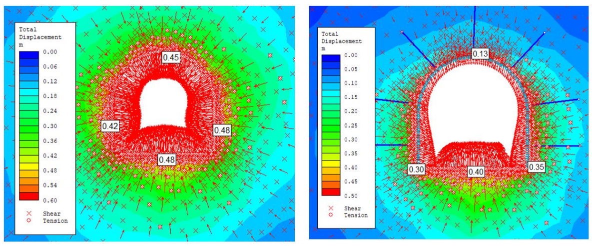

Figure 10: Deformation obtained from the plastic analysis without (left) and with (right) application of support at chainage 0+403 of GKHPP

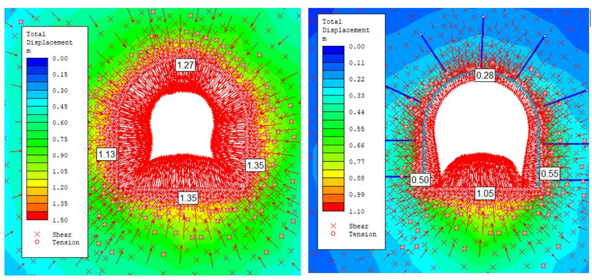

Figure 11: Deformation obtained from the plastic analysis without (left) and with (right) application of support at chainage 0+417 of GKHPP

The displacements obtained from the analysis were different from the measured value. Therefore the rock mass parameter is changed and the reanalysis is done unless the displacements are more or less equal to the measured value. Finally, the better rock mass parameters are obtained. As in the plastic analysis, the liner elements and the bolt elements failed as in the field. Hence, the model with better input parameters accurately replicates the actual site situation.

4.2 Hoek and Marinos Approach

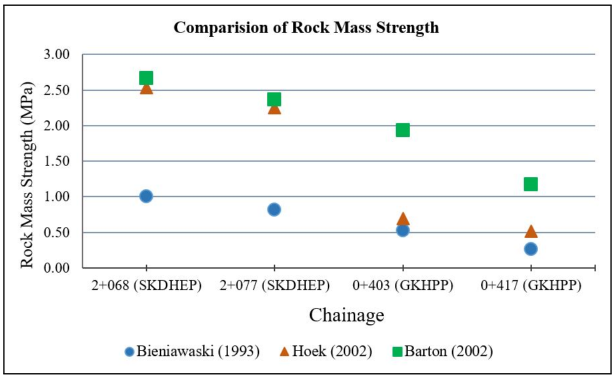

Figure 12: Comparison of Rock mass strength form different empirical relations

For the calculation of the deformation of the underground opening using the Hoek and Marinos approach the rock mass strength is one of the input parameters.The comparison of rock mass strength from different researchers has been shown Figure 12. The deformation calculated from the Hoek and Marinos approach using the rock mass strength calculated by using the empirical relation is shown in Figure 13.

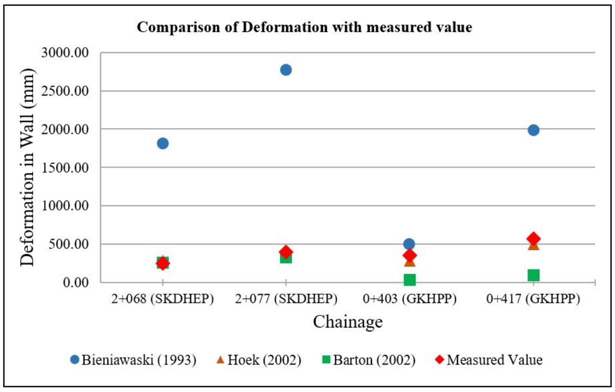

Figure 13: Comparison of Deformation from Hoek and Marinos approach with measured value

4.3 Convergence Confinement Method

The main assumptions of CCM are that the shape of the tunnel is considered to be circular and the stress field is hydrostatic. The analysis has been done considering the equivalent hydrostatic field stress and circular tunnel with a radius equal to the equivalent radius of the section.

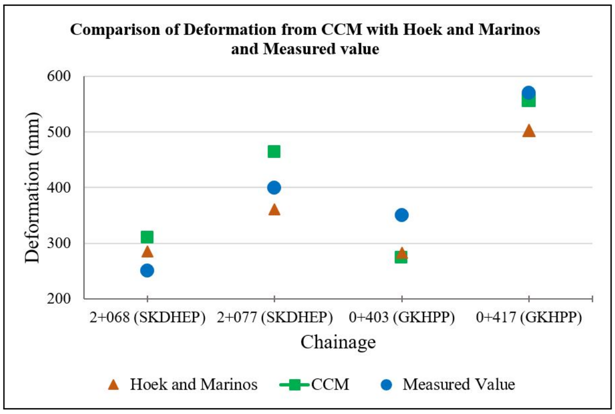

Figure 14: Comparison of deformation from CCM with Hoek and Marinos approach and measured value

4.4 Support Design using Empirical Method

4.4.1 Q-system

Table 1: Tunnel rock support from Q-system for SKDHEP Tunnel

Support	2+068	2+077
Rock Bolt Length (m)	2.4	2.4
Rock Bolt Spacing (m)	1.2 c/c	1.2 c/c
Initial Shotcrete (cm)	12	12
RRS Spacing (m)	3.2	2.7
Reinforcement	6 Nos.	6 Nos.
Reinforcement Dia (mm)	16-20	16-20
Layers	Single	Single
Total Rib Thickness (cm)	35	35

Table 2: Tunnel rock support from Q-system for GKHPP Tunnel

Support	0+403	0+417
Rock Bolt Length (m)	2.0	2.0
Rock Bolt Spacing (m)	1.1 c/c	1.0 c/c
Initial Shotcrete (cm)	12	15
RRS Spacing (m)	2.5	1.9
Reinforcement	6 Nos.	10 Nos.
Reinforcement Dia (mm)	16-20	16-20
Layers	Single	Double
Total Rib Thickness (cm)	35	40

4.5 Numerical Modeling for Support Check

The supports obtained from the empirical method mentioned in the chapter above have been analyzed whether the empirical approaches provide enough rock support for the stability of the tunnel.

Table 3: Summary of the yielded liner elements and bolts for inverted D-shape tunnel sections at SKDHEP

Chainage	Liner Failed	Bolts Failed
2+068	87 %	50 %
2+077	98 %	63 %

Table 4: Summary of the yielded liner elements and bolts for inverted D-shape tunnel sections at GKHPP

Chainage	Liner Failed	Bolts Failed
2+068	53 %	14 %
2+077	48 %	0 %

The support installed as per the Q-chart seems to fail in the side walls and the invert of the inverted D-shape of the tunnel.

Table 5: Summary of the yielded liner elements and bolts for Horse shoe shaped tunnel sections at SKDHEP

Chainage	Liner Failed	Bolts Failed
2+068	55 %	33 %
2+077	59 %	44 %

Table 6: Summary of the yielded liner elements and bolts for Horse shoe shaped tunnel sections at GKHPP

Chainage	Liner Failed	Bolts Failed
2+068	24 %	0 %
2+077	20 %	0 %

The support installed as per the Q-chart also fail Horse Shoe shape of the tunnel. As compared to the inverted D-shaped tunnel the fewer liner elements fail and less number of rock bolts yield in the horseshoe-shaped tunnel. But the support installed as per the Q-chart does not fail in the case of the circular-shaped shape of the tunnel. The liner elements, as well as the rock bolts, are capable to withstand the load in a circular shape. Hence, it can be interpreted that the support estimated from the Q-chart could be sufficient for the stability in the case of the circular tunnel.

Reanalysis of the tunnel with shape inverted D-shape and horseshoe shape has been done for achieving the stability of the underground opening. In the case of the GKHPP and SKDHEP tunnel section inverted D shape is not preferred because the support system becomes too uneconomical to achieve stability.

For the horse shoe shape the support from the Q-system has been modified by analyzing it through the Numerical Modeling which is presented in Table below.

Table 7: Support from Q-system and Modified support for the horse shoe shape tunnel of 4m dia at chainage 2+068 of SKDHEP

Support	Q-system	Modified
Initial Shotcrete (cm)	12	15
RRS Spacing (m)	3.2	2.0
Reinforcement	6 Nos.	10 Nos.
Reinforcement Dia. (mm)	16-20	20
Layers	Single	Double
Total Rib Thickness (cm)	35	40

Table 8: Support from Q-system and Modified support for the horse shoe shape tunnel of 4m dia at chainage 2+077 of SKDHEP

Support	Q-system	Modified
Initial Shotcrete (cm)	12	15
RRS Spacing (m)	2.7	1.5
Reinforcement	6 Nos.	10 Nos.
Reinforcement Dia. (mm)	16-20	20
Layers	Single	Double
Total Rib Thickness (cm)	35	40

Table 9: Support from Q-system and Modified support for the horse shoe shape tunnel of 3.3m dia at chainage 0+403 of GKHPP

Support	Q-system	Modified
Initial Shotcrete (cm)	12	12
RRS Spacing (m)	2.5	2.0
Reinforcement	6 Nos.	10 Nos.
Reinforcement Dia. (mm)	16-20	20
Layers	Single	Double
Total Rib Thickness (cm)	35	35

Table 10: Support from Q-system and Modified support for the horse shoe shape tunnel of 3.3m dia at chainage 0+417 of GKHPP

Support	Q-system	Modified
Initial Shotcrete (cm)	15	15
RRS Spacing (m)	1.9	1.5
Reinforcement	10 Nos.	10 Nos.
Reinforcement Dia. (mm)	16-20	20
Layers	Double	Double
Total Rib Thickness (cm)	40	40

5. Conclusion and Recommendation

The support obtained from Q-system applied to the circular tunnel achieved the requirement for stability, but not for the inverted D and horseshoe-shaped tunnel. The horseshoe shape tunnel was found out to be more stable in comparison to the inverted D-shape. Reanalysis has been done of the inverted D and horseshoe-shaped tunnels by increasing the thickness and reinforcement to the liner. For the weak rock mass and at the high field stress the inverted D-shaped tunnel is not preferred because it was very hard to obtain stability even though heavy support has been applied. Support obtained from Q-chart should be modified except for the circular shape tunnel in particular case.

The numerical modeling of the rock mass requires the exact measurement of the mechanical properties of the rock mass and in-situ stress. Modeling of the rock mass without exact input parameters could mislead the interpretations. So it is recommended to conduct the laboratory test of the rock mass at the exact location to get reliable geotechnical properties as well as geological condition of the field.

Acknowledgments

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References

[1] Lance. G., J.M. Anderson, and D.R. Lamot. Third party safety issues in international urban tunnelling. *Proc World Tunnel Congress, Prague.*, 2007.

[2] A Palmström and B. Nilsen. Engineering geology and rock engineering handbook. *Norwegian Soil and Rock Engineering Association*, 2000.

[3] Subodh Dhakal. Geological divisions and associated hazards in nepal. 2014.

[4] Subodh Dhakal. *Fundamentals of Geology. A Reference book for students of Geology, Civil Engineering, Environmental Science and Forestry*. Kalyani Subedi Publishers., 2012.

[5] Bieniawski and T. Zdzisław. *Engineering rock mass classifications: a complete manual for engineers and geologists in mining, civil, and petroleum engineering*. John Wiley & Sons, 1989.

[6] E. Hoek and E.T. Brown. Practical estimates of rock mass strength. *International Journal of Rock Mechanics and Mining Sciences*, 1997.

[7] N. Barton. Some new q-value correlations to assist in site characterisation and tunnel design. *International Journal of Rock Mechanics and Mining Sciences*, 2002.

[8] E. Hoek and M.S. Diederichs. Empirical estimation of rock mass modulus. *International Journal of Rock Mechanics and Mining Sciences*, 2006.

[9] B.H.G. Brady and E.T. Brown. *Rock Mechanics: For underground mining*. Springer Netherlands., 2007.

[10] Z.T. Bieniawski. *Classification of Rock Masses for Engineering: The RMR System and Future Trends*. 1993.

[11] E. Hoek, C. Carranza-Torres, and B. Corkum. Hoek-brown failure criterion-2002 edition. 2002.

[12] Chhatra. Basnet, Pawan Shrestha, and Krishna Panthi. Analysis of squeezing phenomenon in the headrace tunnel of chameliya project, nepal. *Hydro Nepal Journal of Water Energy and Environment*, 2013.

[13] E. Hoek and P Marinos. Predicting tunnel squeezing problems in weak heterogeneous rock masses. *Tunnels and tunnelling international*, 2000.

[14] C. Carranza-Torres and C. Fairhurst. Application of the convergence-confinement method of tunnel design to rock masses that satisfy the hoek-brown failure criterion. *Tunnelling and Underground Space Technology*, 2000.