Tunnel support design in fault zone in hydropower project in the Nepal Himalaya: a case study

Bimal Chhushyabaga¹, Sujan Karki¹ and Shyam Sundar Khadka¹*

¹Department of Civil Engineering, Kathmandu University, Nepal
*Corresponding author (sskhadka@ku.edu.np)

Abstract. Tunnelling through faulted rock mass is associated with long lasting displacements, stability problems, overburden squeezing, high deformations and dynamic stress conditions. With the dominance of faults in the Nepal Himalaya, determination of behaviour of the rock mass around tunnels and physical parameters for tunnel support is very challenging and demanding. Tunnel support and rock mass stability in tunnel are always dominated by fault. In this paper a hydropower tunnel in the vicinity of a fault has been used as a case study for the design and application of tunnel support. Rock mass is characterized, behaviour of the unsupported rock mass is evaluated, and support is designed. Geological Strength Index (GSI) is used to characterize the rock mass and a methodology has been used to design the support using numerical analysis. The system behaviour of the rock mass and the tunnel support are analysed for the various combinations of geomechanical ground conditions. The displacement, stress, yielded elements and plastic region are studied for the influence of the fault and its stress conditions of the designed tunnel supports.

Keywords: Fault, Geological Strength Index (GSI), Tunnel Support, Numerical Analysis

1. Introduction

A fault is an unfavorable and undesirable geological structure which is a planner or gently curved fracture in the rock of the Earth’s crust, where compressional or tensional forces cause relative displacements on the opposite sides of the fracture. Tunnelling in the fault zones is associated with frequently changing rock mass, groundwater conditions, and long-lasting displacements in comparison to other sections of the tunnel. The main instability situations associated to the faulted rock are collapses, squeezing, flow ground [1]. Deformation from squeezing, swelling of faulted rocks, excessive over breaks, instability of the face are principal geotechnical difficulties that are likely to be encountered when driving the tunnel through the faulted rock mass [2].

Location and mechanical properties of the fault control the failure pattern of the surrounding rock and support. The stresses in the rock mass around the tunnel and sprayed concrete due to properties such as rock mass quality, fault thickness, location [3]. There is the variation in the ground stresses in the presence of the tectonic structures such as a fault and initial ground stress and it is one of the important influencing factors on the behavior of tunnel [4]. High tectonic stresses and ground stresses, lateral stress coefficient...
cause diverse deformation and the second stress field in the rock mass around the tunnel [5] There is rock slides along the fault plane, the downfall of blocks, and wedges cut by faults or minor joints inward the cavern [6].

2. Methodology of Tunnel Support Design in Fault Region

A geomechanical design procedure (Figure 1) is followed for the design of the tunnel support in the faulted rock mass for the headrace tunnel of Kali Gandaki ‘A’ Hydropower Project. The rock mass is characterized and the behaviour of the unsupported rock mass are evaluated.

**Figure 1.** Geomechanical design procedure of tunnel support in faulted zone

**Figure 2.** Criteria for the support design, Goricki et al. [7]
Figure 3. Maximum support capacities for ribs, 3-bar lattice girder, bolts, and concrete in circular tunnel after Hoek[8]

Tunnel support are designed after the excavation analysis of the tunnel and the system behaviour of the rock mass and tunnel support are analysed. The rock mass parameters around the tunnel and in the actual face are determined during the construction. The change of the asymmetric displacement vectors in the tunnel cross section indicates that a fault is crossing the axis of the tunnel in an acute angle. Actual ground conditions of the weak sheared and faulted rock mass are obtained by determination of parameters such as intact rock strength and rock mass strength, in situ site stress, deformation, orientation of joint to the tunnel axis, ground water conditions and overburden. The model for the design of the tunnel is improved according to the obtained actual ground conditions. Geological Strength Index (GSI) [9] is used for the determination of the important rock mass parameters from the properties of the intact rock and discontinuities of the rock mass in sheared and weak rock masses GSI system cannot be used when the intact rock properties and discontinuities in the sheared weak rock mass cannot be described separately. In such case the properties of

| Support type | F(mm) | D(mm) | W(Kg/m) | C  | P(MPa) |
|--------------|-------|-------|---------|----|--------|
| Wide flange rib | 305   | 305   | 97      | 1  | $P_l = 19.9D^{0.23}$ |
|               | 203   | 203   | 67      | 2  | $P_l = 13.2D^{-0.3}$ |
|               | 150   | 150   | 32      | 3  | $P_l = 7.6D^{-1.4}$ |

Note: $F = $ Flange width, $D = $ Section depth, $W = $ Weight per meter, $C = $ Curve number, $P = $ Maximum pressure, $T = $ Thickness – mm, $A = $ Age, $U = $ Uniaxial Compressive Strength
the weak rock mass are evaluated using the Block-in-Matrix (BIM rock) or testing of the rock mass in suitable scale. For the design of the tunnel support in faulted rock mass, rock mass strength, deformation characteristics, primary stress condition are most important quantitative parameters and design can be done by two methods as suggested by by Goricki et al, and Hoek.

In the method by Goricki et al, the application of the support categories is defined in the form of a matrix diagram (Figure 2) in which the intact rock strength($\sigma_{ci}$), rock mass strength($\sigma_{rm}$) and GSI are used as parameters to describe the properties of the rock mass. The quadrant 1(Figure 2) and quadrant 2 (Figure 2) are used for the modification of the $\sigma_{rm}$ due to the discontinuities and ground water conditions. The orientation of the joint to the tunnel axis is very important in determining $\sigma_{rm}$. The parallel orientation is unfavourable to that of perpendicular orientation. Similarly, higher ground water decreases $\sigma_{rm}$. The stress factor obtained from overburden and modified $\sigma_{rm}$ in quadrant 3 (Figure 2) and measured displacement are used to obtain support category in the quadrant 4 (Figure 2). The measurement of the joint orientation to the tunnel and the ground water table are very essential for obtaining the support category. In method proposed by Hoek, the measured support pressure is used to obtain the tunnel support (Note: $F$ = Flange width , $D$ = Section depth, $W$ = Weight per meter, $C$ = Curve number, $P$ = Maximum pressure, $T$ = Thickness - mm, $A$ = Age, $U$ = Uniaxial Compressive Strength).

3. Case study: Kali Gandaki ‘A’ Hydroelectric Project

Headrace tunnel of Kali Gandaki ‘A’ Hydroelectric Project has been selected. It is located in the Lesser Himalaya, The length of 5950 m long horseshoe shaped headrace tunnel (Figure 4). The headrace tunnel passes through highly deformed Siliceous and Graphitic Phyllite that varies in mineral composition and degree of metamorphism (Figure 4). The rock mass in the area has been subjected to shearing, folding and faulting due to active tectonic movement. The orientations and dips of the joints sets are highly scattered due to extreme folding and shearing, giving no distinct joint system except for foliation joints. The tunnel is close to Main Boundary Thrusts (MBT) and different shear zones. The tunnel section at the chainage 2+000 m with 620 m is selected for the design of the tunnel support as there is a thrust at the boundary of Siliceous Phyllite and Graphitic Phyllite [10].

![Figure 4. Longitudinal profile of headrace tunnel of Kali Gandaki A Hydroelectric Project [10]](image)

4. Estimation of Geotechnical Properties

The Hoek Brown failure criterion [11] is linked closely to the GSI for the heavily sheared rock masses which have homogeneity, behaves isotopically and hence it is used in estimating the strength parameters of rock mass used in modelling. The input parameters in modelling of tunnel in Siliceous Phyllite are presented in Table 1 and Table 2. The vertical in-situ site stress acting in the rock mass in which the tunnel is excavated is given by the product of depth(H) below the surface and the unit weight of the rock equation (1).
\[ \sigma_1 = \Upsilon \cdot H \]  

(1)

Where, \( \Upsilon \) = Unit weight of rock mass and \( H \) = Overburden of rock mass

The horizontal in-situ site stress in the rock mass depends upon the tectonic history of the area, stiffness and local topography. In such an area there is in situ stress variation between 0.5:1 and 2:1[12]. The ratio of in-situ vertical site stress to horizontal site stress in the context of the faulted rock mass in the Himalaya can be calculated by using equation (2) [13]. The calculated in-situ vertical and horizontal stress are presented in the Table 2.

\[ \frac{\sigma_1}{\sigma_3} = \left( \frac{U^2 + 1}{U^2 + 1 + U} \right)^{3/2} \]  

(2)

Where, \( U \) = Poisson’s ratio

Intact rock modulus\( (E_i) \) and rock mass modulus\( (E_{rm}) \) have been calculated using equation (3) and equation (4) as suggested by Hoek and Diederichs [14] and Cai et al [15].

\[ E_i = (1 - \frac{D}{2}) \sqrt[100]{\frac{\sigma_{ci}}{10}} \times 10^{-40} \]  

(3)

\[ E_{rm} = E_i (0.02 + \frac{1-D/2}{I+e^{(60+15D-GSI)/11}}) \]  

(4)

Equation (5) suggested by Priest and Brown [16] have been used to calculate the strength parameters for the jointed rock mass i.e, \( m_j \) and \( s_j \), which are used in Hoek Brown failure Criterion.

\[ m_j = m_i e^{(RMR-100) \frac{15.4}{6.3}} \]  

\[ s_j = e^{(RMR-100) \frac{6.3}{6.3}} \]  

(5)

Similarly, Bieniawski [17] have been used to estimate Rock Mass Rating (RMR). Ramamurthy [18] have suggested equation to obtained to calculate GSI equation (13)

\[ GSI = 9 \ln(Q) + 44 \]  

(6)

4.1. Joint Factor Method

The joint factor equation (7) method relate the strength ratio \( r_{cm}/r_c \) to a joint factor that is related to joint frequency, joint orientation and joint strength [18]. The calculated joint factor parameters are shown in Table 1.

\[ J_f = \frac{J_0}{n_r} \]  

(7)

Where, ‘\( J_0 \)’ is the joint frequency (number of joints per meter), which is simply obtained by dividing the number of joints by the specimen length in meters. ‘\( n \)’ is an inclination parameter depending on the orientation of the joint, ‘\( \beta \)’ is the angle between the loading direction and the joint plane and ‘\( r \)’ is the joint strength parameter depending on the joint condition, Figure 5 (a) and Figure 5 (b).

Figure 5. (a) Loading on a rock mass element ‘A’ in a circular tunnel. (b) Enlarged element ‘A’ [19]
To incorporate the joints in determination of the rock mass compressive strength, equation (8) suggested by Jade and Sitharam [20] have been used.

$$\frac{\sigma_m}{\sigma_{ci}} = 0.039 + 0.893e^{(r - Jf) / 160.99}$$ (8)

**Table 1.** Joint factor parameters and Hoek Brown strength parameters [16], [21], [22]

| Jn/meter | N | r | Jf | Q | GSI | σci (MPa) | D |
|---------|---|---|----|---|-----|-----------|---|
| 6.3 | 0.09 | 0.18 | 389 | 0.016 | 22 | 50 | 0 |

**Table 2.** Rock mass parameters of Siliceous Phyllite in the faulted region [10], [21], [22]

| Overburden(m) | Rock Density(gm/cm³), | Specific Weight(MN/m³), | Intact Rock Strength(σci ,MPa), | Rock mass strength(σrm,MPa) | Poisson’s ratio(U), | Vertical Stress(MPa) | Horizontal Stress(MPa) | Measured Support Pressure(MPa) | Percentage of measured deformation per radius of the tunnel | Intact rock mass modulus(MPa) | Rock mass modulus(MPa) |
|---------------|------------------------|--------------------------|-----------------------------|-----------------------------|------------------|---------------------|------------------------|-----------------------------|--------------------------------|--------------------------|-----------------------|
| 620           | 2.77                   | 0.027                    | 50                          | 4.62                        | 0.26             | 16.74               | 10.01                  | 1.27                        | 4.4% (i.e, 19.14 cm)                 | 770.5                    | 27500                 |

5. Numerical Modelling
RS2 is a 2D finite element program for soil and rock applications. RS2 is used for the tunnel lining and excavation design. Complex, multi-stage tunnel models can be easily created and quickly analyzed in weak or jointed rock. Progressive failure, support interaction is used for the design of the tunnel lining. It offers a wide variety of support elements for support design including bolts, liners, beams and piles. Different types of loading can be modelled, restraints and boundary conditions can be easily applied and also meshing is automatic with tetrahedral elements [23]. A number of options are available to view and display the results in 2D. The goal of this design procedure is to design the tunnel support suggested by Goricki et al or by Hoek with a factor of safety greater than 1.4 in RS2. Since, we have measured support pressure and deformation form our case study, design of the tunnel support is done by the method suggested by Hoek with following modelling procedure. The horse shoe tunnel of radius 4.35 m is developed in RS2 with the overburden of 620 m with surrounding rock mass of Siliceous Phyllite. The obtained major in plane principal stress are 16.74 MPa vertical Stress and 10.01 MPa horizontal Stress. The outer boundary is taken as 3D, where D is the diameter of the tunnel, equal to 8.7 m. The strength parameters of Siliceous Phyllite used in the modelling is represented in Table 2. The Rock mass strength (σrm) obtained from equation (5) is 5.94 MPa which is greater than 4.62 MPa i.e., to design in the critical state, smaller value of σrm have been used, i.e Hoek Brown Failure Criterion. As a tunnel is excavated, there is a certain amount of deformation before the support is installed, usually 35-45% of the final tunnel wall deformation. The total amount of tunnel wall deformation prior to support installation is determined using the observed field values. The final displacement obtained in the tunnel is 4.4 % of the radius of tunnel. i.e 19.14 cm (Table 2). So, deformation of the tunnel prior to the support installation is allowed to 35 percent (Hoek et al [24]) of the final
displacement i.e, 6.70 cm. The maximum total displacement obtained at the invert of the tunnel i.e 7.1 cm (Figure 6) in the numerical modelling is very close to that of the allowed final displacement of 6.7 cm before the support installment.

6. Results
The maximum support pressure using statistical distribution in the tunnel is 1.27 MPa (Table 2). The tunnels in the seismic area are affected near the portals and in vicinity of faults and thrusts. The effects of the seismicity are observed to be up to a distance along tunnel within ratio of B on the both sides of faults and thrusts, where B is the span or size of the opening. The design support pressure is taken as 1.25 times of ultimate support pressure [25]. So, the designed support is taken as 1.59 MPa. The design is done using the equivalent circular diameter of 8.7 m. The support with Design Support 1(Figure 7(a)) will give a support pressure approximately of 1.8 MPa and Design Support 2 (Figure 7(b)) will give the support pressure excess of 1.6 MPa. Details of Design Supports are in Table 3.

| Support          | Details                                                                 |
|------------------|-------------------------------------------------------------------------|
| Design Support 1 | Three bar lattice girder bar size 26 mm 34 mm, spaced at 1 m embedded in a 150 mm thick shotcrete layer (25 MPa) |
| Design Support 2 | 1 m ×1 m pattern of 34 mm diameter of rock bolts together with a 200 mm thick shotcrete (25 MPa) and ISMB 310 |

6.1 Analysis of Design support 1
The model without the support show that total number of the yielded elements and total displacement are 1062 and 7.1 cm respectively (Figure 8 (a)), which decreased to 853 and 3.2 cm respectively when the support of the Design Support 1 was applied (Figure 8 (b)). The support capacity curve (Figure 9) for the Design Support 1 show that some points are outside the support capacity curve i.e., the concrete failed in the points at the base of the walls of the tunnel. Hence, the Design Support 1 was not enough as designed tunnel support.

6.2 Analysis of Design support 2
The number of the yielded elements decreased to 694 and 0.73 cm, when Design Support 2 was applied (Figure 8 (d)). The support capacity curve (Figure 10) show that the all the points are inside the support capacity curve. Hence, the Design Support 2 have enough load bearing capacity and can be used in the tunnel construction as the required support.
Figure 7. Tunnel Design Support

Figure 8. Application of Design Supports
The primary rock support applied during construction mainly consisted of steel ribs of ISMB 125 spaced at an interval ranging from 0.6 to 1.5 m, steel fiber reinforced shotcrete having thickness ranging from 15 to 60 cm and 4-m-long 25-mm diameter fully grouted rock bolts [26]. The provided support is giving the support pressure equivalent to Design 2. Hence, the methodology applied in this paper to design the tunnel support provides the tunnel support as per requirement of the field.

Figure 9. Support capacity curve for Design Support 1

Figure 10. Support Capacity curve for Design Support 2

7. Conclusion
The main instability situations associated to the faulted rock are collapses, squeezing, flow ground. The distribution of the stress and deformation of the faulted rock mass can be studied using 2D Numerical modelling in RS2 and design tunnel support can be done for the required support pressure with allowable deformations.
The support design methodology is based on the numerical modelling of the monitoring based data and characterization of the rock mass using GSI during excavation of tunnel. It provides more realistic scenario in the design of the tunnel support as the parameters of the faulted rock mass and in situ site stress are calculated. The uncertainty in obtaining the realistic geological data and mechanics in the context of the jointed and faulted rock mass is reduced. Hence, methodology proposed in this paper can be used to design the tunnel support in weak, sheared and faulted rock mass in the Himalaya.

8. Acknowledgement
This study has been carried out under the project “Technical Investigation of Tunnel Support Technology of hydropower Projects Located in the Himalayan Region of Nepal” (ENEP-RNEP-II-18-02). The project has been funded by Energize Nepal and Department of Civil Engineering, Kathmandu University. The authors would like to express the deepest gratitude to Energize Nepal and acknowledge all the project partners: Seoul National University (SNU), Norwegian University of Science and Technology (NTNU) and Hydro Tunnelling and Research Pvt. Ltd. (HTR)

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