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Stability Evaluation and Review of Applied Support for Headrace Tunnel of Middle Mewa Hydropower Project, Taplejung, Nepal

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Abstract

In the Nepal Himalayas, predicting rock mass quality precisely and assessing stress induced issues have been difficult tasks, where stess induced problem such as tunnel squeezing is a common phenomenon in weak rock and in weakness zones. Incompetent rock with high or moderate rock stress due to moderate to high overburden when tangential stress exceeds the strength of the rock mass may have squeezing problems in tunnels. Rock spalling or rock burst problems occur when tangential stress exceeds the strength of the rock mass in competent and brittle rock with high-stress levels due to high overburden. Prior knowledge of possible stress-induced problems in the tunnel help address problem in advance or prepare an action plan accordingly. Often, rock mass properties and state of in-situ stresses are not known fully until tunnel excavations are made. With the response of rock mass upon excavation in the form of tunnel convergence and failures, the rock mass properties and in-situ stresses are back analyzed to ascertain realistic input parameters. The headrace tunnel of the Middle Mewa Hydropower Project also has both competent and incompetent rock at varying overburden. As the tunnel is being excavated, rock mass response to excavation is known and rock supports estimated can be reviewed. This paper presents a stability assessment of the headrace tunnel using various Empirical, Semi-Analytical, Analytical and Numerical Methods. Finally, the support system is reviewed.

Keywords

Tunnel squeezing, Rock spalling, Rock mass properties

1. Introduction

Due to the varied geology and challenging terrain of Nepal, study on tunnel stability requires specific attention. The hilly topography results in significant overburden pressure in the subsurface structure, which squeezes weak rock masses and produces other stability issues like rock bursting and spalling in competent rock masses. Tunnel squeezing is a typical occurrence in Nepal's Himalayas [1] in weak rock and the weakness zone (eg:mudstone, slate, phyllite, schist, and schistose gneiss) which are not capable of withstanding high stress. The rock mass in the Himalayas is highly weathered, faulted, fractured, and sheared as a result of the significant tectonic activity in the area. It is highly possible that tunnels pass through several weaknesses, fissures, and fracture areas. Most of these weak zones are often highly conductive and could be sources of groundwater aquifer from where leakage might occurs from finished unlined or shotcrete-lined tunnels [2]. Predicting rock mass quality and analyzing stress-induced problems have been challenging tasks in this region. In absence of proper geological investigation, known rock mass properties, and state of stress, a fairly conservative support system may be adopted to avoid stress induced problems. The headrace tunnel of Middle Mewa Hydropower Project is being excavated, and tunnel behavior is observed where rock mass parameters are now known, a review of tunnel stability and rock support is necessary.

2. Brief on the Project

Located at a distance of 700 Km northeast of Kathmandu, Middle Mewa Hydropower Project is located in the Taplejung District of the Eastern Nepal. The Project shall utilize the design discharge of 18.76 m3/s and the gross of 475 m through a 5430 m long headrace tunnel. To facilitate the excavation of the headrace tunnel two construction adits have been provisioned.

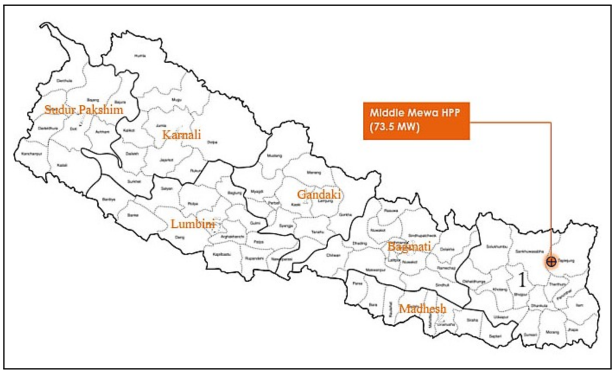

Figure 1: Location of Middle Mewa HPP

2.1 Geology of the area

The project is situated in the Higher Himalayan zone near the Main Central Thrust (MCT) zone having low-grade metamorphic rocks of the Taplejung Group. Higher and Lesser Himalayan crystalline rocks are found in that area which consists of thick banded and augen gneiss. The headrace tunnel alignment lies mainly in the gneiss, schistose gneiss, and schist units. In the headrace tunnel exit portal area, the rock mass is generally composed of the coarse-grained schistose gneiss with biotite dipping towards the North.

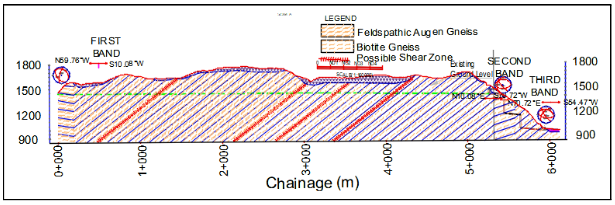

Figure 2: Geological profile of MMHPP [3]

2.2 Mapped rock mass quality

The rock mass classification by the Q-system [4] along the headrace tunnel (HRT) was carried out along its excavated 1950 m length of tunnel alignment during tunnel excavation. The rock mass having Q-Value greater than $4(Q > 4)$ is considered Class I rock mass, Q-value between 4 and 1 $(4 > Q > 1)$ is considered as Class II rock mass, Q-value between 1 and 0.4 $(1 > Q > 0.4)$ is considered as class III rock mass, Q-value between 0.4 and 0.1 $(0.4 > Q > 0.1)$ is considered as Class IV rock mass, Q-value between 0.1 and 0.05 $(0.1 > Q > 0.05)$ is considered as Class V-A rock mass, and Q-value between 0.05 and $0.01(0.05 > Q > 0.011)$ consider as Class V-B rock mass. The highest percentage with 47.21% of overall Rock mass classification consists of Rock class II, the second highest with 25.89% consists of Rock Class III and 21.83% of Rock Class IV was observed, 5.076% of Rock Class V was observed.

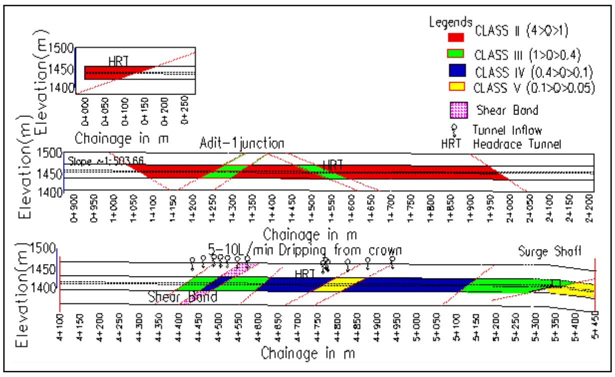

Figure 3: Rock mass quality along the Headrace Tunnel of MMHPP

Figure 3 shows the various types of rock mass classes present along the headrace tunnel which mainly vary from the Rock Class II to Rock Class V. Near the inlet portal and up to chainage 0+165 m, Rock Class II has been found. The upstream and downstream sections from the junction of Adit-1 and the headrace tunnel have Rock Class II except for the junction location. The junction at Adit- 1 and the headrace tunnel consists of Rock Class III Similarly the rock mass class at the Chainages around 1+500 m to 1+550 m consists of Rock class III. The upstream section and downstream section from Adit -2 junction have Rock Classes III, IV, and V. Minor inflow of groundwater was observed around these Chainages sections. Around Chainage 4+4500 m, a shear band was observed in which an inflow of around 5-10 Litre/min into the tunnel from the crown was observed. The rock mass of Rock Class III was observed around the Chainage of 4+450 m to 4+800 m and at the Chainage of 5+175 m to 5+350 m. Rock Class IV was observed at the Chainage of 4+800 m to 4+750 m, and 4+850 m to 5+100 m. The weak rock mass of Rock Class V was observed at the change of 4+750 m to 4+850 m in which minor inflow into the tunnel was also observed. The sections for stability evaluation was chosen based on the overburden depth, shear band observations, the amount of water inflow inside the tunnel, Rock Class quality, and the junction area of Adit-1, and Adit-2 with the headrace tunnel.

Rock mass classification using the Q-system was adopted to evaluate the rock mass quality. The rock mass parameters were taken from the face mapping data at respective critical chainage of the headrace tunnel alignment. The rock mass classification based on the Q-system shown in Table 1.

Table 1: Rock mass classification along section

HRT Chainage m	Rock Type	Q Value	Rock class
0+135	slightly weathered Gneiss	3.75	II (Fair)
1+275	light to grey color Gneiss	0.89	III (Poor)
4+500	Biotite gneiss with shear band at face and crown	0.17	IV (Very Poor)
4+560	Biotite gneiss with schist	0.66	III (Poor)
4+700	Biotite Rich Gneiss	0.33	IV (Very Poor)
4+760	Gneiss	0.08	V (Extr. Poor)
5+050	Schistose Gneiss	0.39	IV (Very Poor)
5+265	Gneiss	3.75	II (Fair)
5+314	Gneiss	1.00	III (Poor)

3. Stability Assessment

Various Empirical, Semi-Analytical, Analytical and Numerical Methods were used to evaluation of the stability and estimation of tunnel deformation at critical tunnel sections. Empirical method using Singh et al (1992) and Goel et al (1995) approach were used to check if the selected tunnel sections would have squeezing problem or not, Semi-Analytical method using Hoek and Marinos (2000) approach was used to estimate the magnitude of deformation also. The Hoek and Brown (1980) method was used to assess if rock bursting or rock spalling would occur in overstressed but competent rocks. The Semi-Analytical method using Shrestha and Panthi (2015) approach was used for the deformation estimation with various support pressures in stress anisotropy conditions. The convergent confinement Method (Carranza-Torres and Fairhurst, 2000) with the Hoek and Brown failure criterion (Hoek et al., 2002) and 2D finite element numerical analysis using RS2 were used to evaluate tunnel stability.

3.1 Singh et al (1992) approach

The empirical method based on the Q-method of classification system for rock masses was used to determine whether or not the rock mass would squeeze.

Table 2: Empirical method for Squeezing Prediction using Singh et al (1992) method

Description	Ch 4+500m	Ch 4+760m	Ch 5+050m
overburden (H)	214.6	195.4	268.21
Q Value	0.17	0.08	0.39
H’	191.97	152.37	255.47
squeezing prediction	squeezing	squeezing	squeezing

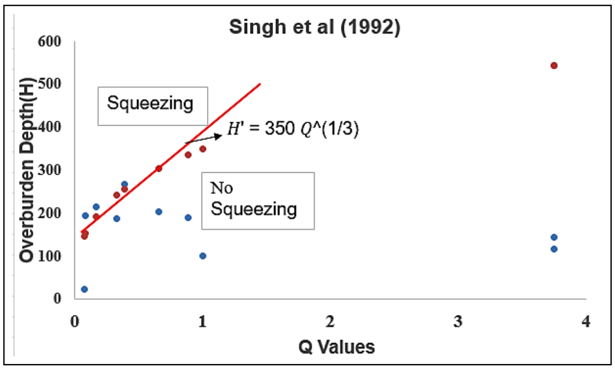

Figure 4: Singh et al (1992) curve for squeezing prediction

Figure 4, shows that squeezing is likely at the three chainages 4+500 m, 4+760 m, and 5+050 m in which the overburden depth lies above the equation line proposed by Singh et al (1992) out of all critical sections that were selected for the study interest.

3.2 Goel et al (1995) approach

This empirical method based on the Q-method of classification system for rock masses was used to determine whether or not the rock mass would squeeze in the same line as Singh et al except that they used rock mass number (N).

Table 3: Empirical method for Squeezing Prediction by Goel et al (1995) method

Description	Ch 4+500m	Ch 4+760m	Ch 5+050m
Overburden (H)	214.6	195.4	268.21
Q value without SRF	0.41	0.41	1.94
H”	175.31	175.31	292.43
Squeezing prediction	Yes	Yes	No

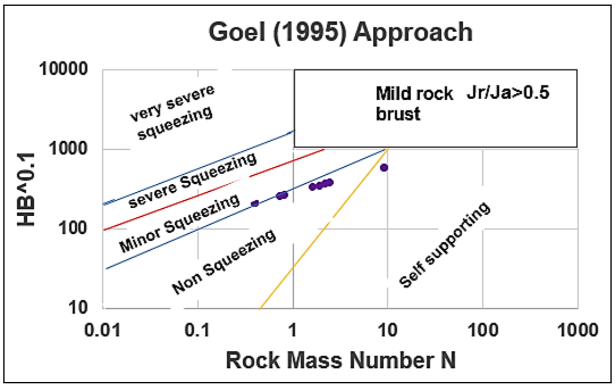

Figure 5: Goel et al (1995) curve for squeezing prediction

Figure 5 shows that the squeezing occurs only at the two Chainages 4+500 m and 4+760 m due to the two points lying inside the minor squeezing zone whereas other points lie in non-squeezing zones where no squeezing was observed. Table 3 also predicted the squeezing phenomenon occurs along these two particular Chainages 4+500 m and 4+760 m.

3.3 Hoek and Marinos (2000) approach

Hoek and Marinos (2000) method is a semi-analytical method based on a general closed-form solution for a circular tunnel with a hydrostatic stress field, where the support is assumed to act evenly around the tunnel's perimeter. It is used to predict squeezing potential and identify its magnitude.

Table 4 shows that squeezing occurs at the Chainage of 4+500 m and has a high magnitude of deformation 46.08 mm. Similarly, the magnitude of deformation at Chainage of 4+760 m and 5+050 m is 32.05 mm and 39.90 mm respectively in 4.41 m wide tunnel. Figure 6 shows the relationship between strain percentage at support pressure 0 MPa, 0.5 MPa, and 1 MPa with the ratio of rock mass strength to the vertical stress which indicates that an increase in the support pressure decreases the strain percentage gradually. The strain at 1 MPa internal support pressure has a lesser value than 0.5 MPa and 0 MPa. In above Table 4, σ_{cm} is estimated as 2.56 MPa, 2.8 MPa, and 3.44 MPa at Chainages 4+500 m, 4+760 m, and 5+050 m respectively. σ_v is estimated as 0.027 times the overburden.

Table 4: Squeezing Prediction by Hoek and Marinos (2000) method

Parameters	Ch. 4+500 m	Ch. 4+760 m	Ch. 5+050 m
$\sigma'cm/\sigma$ v	0.44	0.78	0.66
Strain (ε%) when Pi =0	1.04	0.73	0.90
Squeezing Prediction	Yes	No	No
Strain (ε%) when Pi=0.5 MPa	0.79	0.55	0.73
Strain (ε%) when Pi=1 MPa	0.59	0.42	0.59
Deformation (δi) (mm) for Pi = 0 MPa	46.08	32.05	39.90

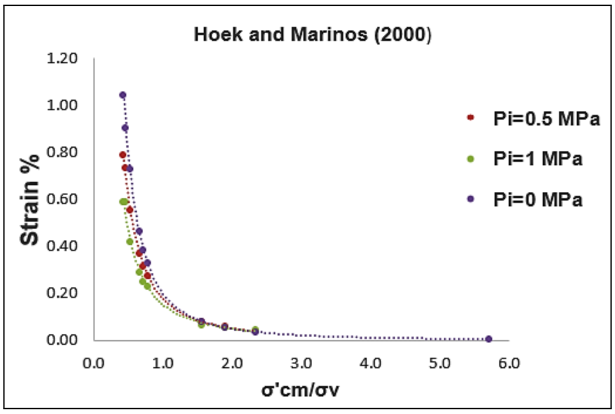

Figure 6: Strain percentage vs Ratio of rock mass strength to vertical stress

3.4 Shrestha and Panthi (2015) method

Shrestha and Panthi [5] studied the long-term squeezing phenomenon of three different hydropower tunnels in the Himalayas of Nepal and found a relationship between time-independent and time-dependent strain using a convergence equation as proposed by Sulem et al. (1987). A relationship between tunnel strain (both instantaneous and final tunnel strain), vertical gravitational stress σ_v, horizontal to vertical stress ratio (k), support pressure (p_i), and shear modulus of rock mass was attempted (G). The shear mass modulus can be estimated as

$$G = E/2(1+\nu) \tag{1}$$

where ν is Poisson's ratio which is estimated as 0.29 (as Lab Report MMHPP), E is rock mass modulus estimated using Hoek and Diederic method (2006). Sress anisotropy factor k is estimated as ratio of horizontal stress (σ_H) to vertical stress (σ_v). Horizontal stress (σ_H) is estimated assuming 4MPa of in plane tectonic stess (σ_{tec}) and the horizontal component of the vertical stess (σ_h) [6].

Table 5: Estimation of Deformation, Shrestha & Panthi (2015)

Chainage	Ch 4+500m	Ch 4+760m	Ch 5+050m
Initail closure when (Pi=0MPa)(%)	0.13	0.08	0.08
Final Closure when (Pi=0MPa)(%)	0.23	0.15	0.14
Initail closure when (Pi=1MPa) (%)	0.03	0.02	0.02
Final Closure when (Pi=1MPa) (%)	0.05	0.04	0.03
2G/σv(1+k)/2	112.80	139.02	142.9

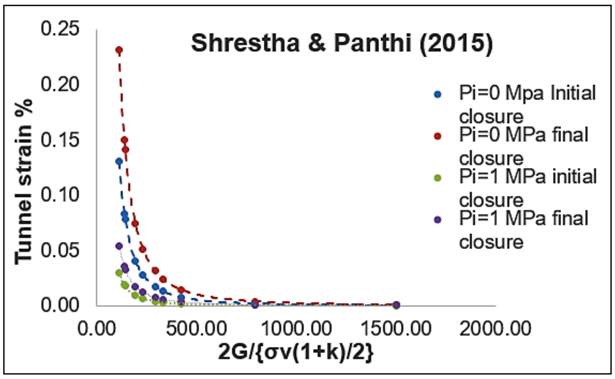

Figure 7: Tunnel strain percentage vs the ratio of shear modulus (G) and in-situ vertical stress (σ v)

Figure 7 shows the strain percentage decrease with the increase in support pressure. The unsupported and 1MPa support pressure is applied then its initial and final closure is computed along all the critical sections chosen for the study. The maximum initial and final closure of 0.13% and 0.23% respectively occur at Chainage 4+500 m among the selected Chainages at unsupported condition.

3.5 Convergence Confinement Method (Analytical Approach)

The relationship between the Ground Reaction Curve (GRC), Longitudinal Deformation Profile (LDP), and Support Characteristics Curve (SCC) is determined using the Convergence Confinement Method (CCM). For the optimization of support, these methods play a vital role [7].

Ground Reaction Curve (GRC) describes the relationship between the decreasing internal pressure (P_i) and the increasing radial displacement of the wall u_r.

The Longitudinal Deformation Profile (LDP) is a graphical representation of radial displacement occurring along the axis of an unsupported cylindrical excavation, for sections behind and ahead of the face.

Support Characteristic Curve (SCC) is the plot between increasing pressure P_s on the support and increasing radial displacement u_r of the support.

$$P_s = K_s u_r \tag{2}$$

K_s denoted the elastic stiffness of the support.

Support characteristics curve has been constructed by using shotcrete or concrete, rock bolts, and steel ribs. The various parameters of shotcrete or concrete linings are unconfined compressive strength of 30 MPa, thickness 75-120 mm(as per rock class), Poisson's ratio 0.2, and Young's modulus of elasticity 30 GPa. The various parameters of fully grouted rock bolts are length of 2.3 m, diameter of 25 mm, bolt modulus pf 200 GPa, and Peak Tensile strength of 0.16 MN with a spacing of 1.3 m c/c distance. The maximum pressure provided by the shotcrete lining is given as [7].

The various input data were used for plotting CCM such as the average radius of the tunnel is 2.21 m, unit weight of rock γ= 0.0273 MN/m3, radial loading of 5.86 MPa, 5.33 MPa, and 7.32 MPa, the GSI of 25, 28, and 35 at Chainages at 4+500 m, 4+760 m, and 5.050 m respectively. The uniaxial compressive strength σ ci = 28.74 MPa, Hoek and Brown parameter mi =28, Poissons ratio ν= 0.29, dilation angle ψ = 0°, the face effect is taken as 1.5 m.

Table 6 shows that the deformation increases going further behind the tunnel face and reaches its maximum value when the face effect becomes zero when no internal support pressure is installed. At

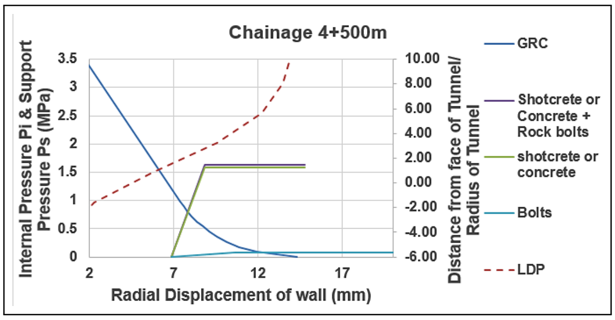

Figure 8: Schematic representation of GRC,LDP and SCC at Chainage 4+500 m

Chainage 5+050 m the value of plastic radius or maximum deformation becomes lesser in comparison to 9.41 mm whereas the value of maximum deformation becomes more at Chainage 4+500 m as 14.32 mm. The internal support pressure value P_s^{max} has a value greater than the critical pressure P_i^{cr} at all Chainages 4+500 m, 4+760 m, and 5+050 m. So the support provided is adequate.

Table 6: Data output from plotting GRC,LDP & SCC curves at various Chainage

Parameters	Ch 4+500m	Ch 4+760m	Ch 5+050m
P_i^{cr} MPa	0.99	0.79	1.12
u_{rmax} mm	14.32	10.2	9.41
u_r at face mm	4.41	3.14	2.9
u_r at 1.5m mm	6.88	4.9	4.52
P_s^{max} MPa	1.63	2.04	1.63
Remarks	No Failure	No Failure	No Failure

3.6 Results of Numerical Analysis

To identify the principal stresses two-dimensional topographical valley model was generated for a needed cross-section of the headrace tunnel in the RS2 FEM model. The bottom boundary of the model was restrained in Y directions and the left-right sides of the model were constrained on the X axis. The model's top was left open in both directions. The four corners of the model were restrained in both X & Y directions. The field stress was set as a gravity type with the actual ground surface.

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Table 7: Input parameters for RS2 in valley model

Description	Ch 4+500m	Ch 4+760m	Ch 5+050m
Tectonic Stress(σ tec)	4	4	4
Trend of Tectonic Stress (θ t)	N5°W	N5°W	N5°W
Angle between σh and Length axis of HRT (θ)	74.8	74.77	74.77
Locked in horizontal stress(In Plane)	0.28	0.28	0.28
Locked in horizontal stress(Out of Plane)	3.72	3.72	3.72
Total stress Ratio(horiz/vert in plane)	0.46	0.46	0.45
Total stress Ratio(horiz/vert out of plane)	1.04	1.11	0.92

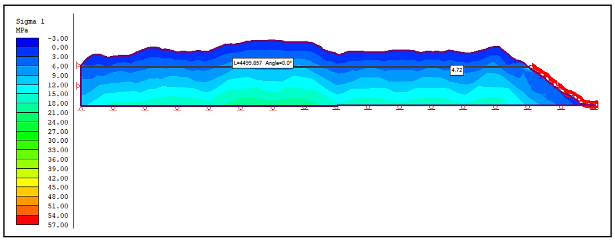

Figure 9: Valley model construction for headrace tunnel alignment at chainage 4+500 m

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Table 8: Output Parameters from Valley Model

Parameters	Ch. 4+500m	Ch. 4+760m	Ch. 5+050m
σ 1(MPa)	4.72	4.48	6.3
σ 3(MPa)	2.63	2.65	2.54
σ z(MPa)	8.6	8.65	9.21
θ°(CCW)	80	99	90

3.6.1 Model setup

For the analysis of the critical tunnel section, the 2D box model of the tunnel width with the width of five times its excavation was constructed. The in-situ stress from the valley model σ_1, σ_3, and σ_z with angle θ is used in this 2D model. The boundary was restrained in both directions. In the 2D model, the material properties should be defined by choosing the initial loading element as field stress and body force. The unit weight and Poisson's ratio were input along with the identification of strength parameters by using the Generalized Hoek Brown method.

Table 9: The Rock mass parameter value set for analysis of various chainage

Parameters	Ch 4+500m	Ch 4+760m	Ch 5+050m
Overburden (m)	214.6	195.4	268.21
Density (MN/m3)	0.027	0.027	0.027
Poisson's Ratio	0.29	0.29	0.29
Ei (MPa)	17244	17244	17244
σ ci (MPa)	28.74	28.74	28.74
mi	22	22	22
GSI	25	28	35
σ 1 (MPa)	4.72	4.48	6.3
σ 3 (MPa)	2.63	2.65	2.54
σ Z (MPa)	8.6	8.65	9.21
θ°CCW	80°	99°	90°

Chainage 4+500 m

The chainage was selected based on the presence of a shear band at the face and the crown portion containing biotite gneiss which is a weakness zone. The overburden at this chainage is 214.6 m and the model was carried out in this section.

Elastic Analysis

Figure 10 shows that the strength factor before installation of support is less than one in the overall periphery of the tunnel, which means that further analysis of the failure of material by plastic analysis is needed. So further plastic analysis is being carried out.

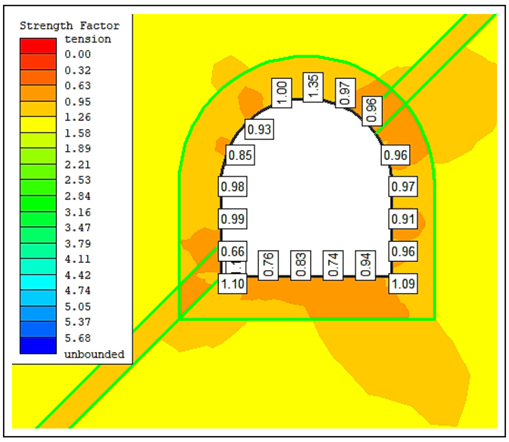

Figure 10: Strength factor before installation of support at chainage 4+500 m

Plastic Analysis

The total displacement which is also called the maximum closure (u_{max}) of the tunnel is 17.93 mm. This is about 0.44% of the tunnel span. The extent of the plastic zone (R_{pl}) is 4.26 m. The unsupported section (X) will be a maximum of 1.5 m distance from the tunnel face. The ratio of the distance from the tunnel face to tunnel radius (X/R_t) is 0.68 and the plastic zone to tunnel radius (R_{pl}/R_t) is 1.93. By using Vlachopoulus and Diederichs method, the above values are plotted to give a ratio of closure to maximum closure equal to 0.55. Therefore, the closure equals 9.86 mm which means 55% of total deformation will already take place before support is installed. An internal pressure factor of 0.2 yields the tunnel wall displacement computed above for the point of support installation.

The total displacement after support installation is shown in below Figure 11.

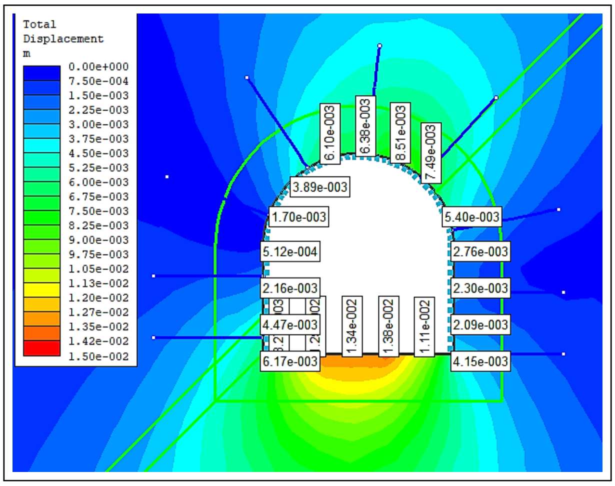

Figure 11: Total displacement after support installation at chainage 4+500 m

The support installed as adopted by the project, the support capacity seems inadequate, there is no yielding of rock bolts however 8 yielded liner element was found so the model was run many times increasing the thickness of concrete along with the use of reinforcement I-beam(W): W150x37.1 in each model. The support capacity plot which is presented as Thrust vs Shear Force and Thrust vs Moment for the support system as suggested in Figure 12 is generated for the support considered.

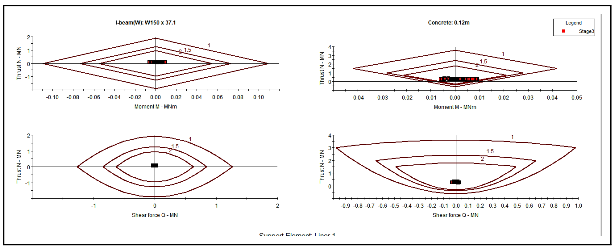

Figure 12: Support capacity curve after revised support at chainage 4+500m

For shotcrete or concrete element in the support capacity curve, all the points come inside all three envelopes as FOS given were 1, 1.5, and 2. So the support for the similar geological condition with the similar Q-value, steel sets, and shotcrete are preferred. Similarly, after analysis of the model in RS2 software at the chainage of 4+760 m, the support capacity seems adequate but in 5+050 m the support installed as adopted by the project needs to be revised with additional reinforcement I-beam(W): W150x37.1.

4. Conclusion

In the Himalayan region of Nepal and the tectonically active zone, underground structures is have threat to its stability such as squeezing, groundwater problems, and roof collapses, etc. It is highly vulnerable to work in such an environment. So for safety issues and economic benefit of the project, stability assessment has great importance. Following are the major conclusion drawn from this paper.

1. In headrace tunnel with excavation length of 1960 m, there is variation of rock mass quality estimated. As excavation revealed, a maximum of 47.21% are fair to good class rock, 25.81% are poor class rock, 21.83% are very poor class rock, and 5.76% are extremely poor class rock with rock type of gneiss, schistose gneiss, and biotite gneiss.

2. Minor squeezing occurs at Chainages 4+500 m (shear band) with 46.08 mm deformation along with the potential squeezing that occurs at Chainages 4+760 m and 5+050 m.

3. Analytical method shows that the internal support pressure p_{smax} has a higher value than the critical pressure P_i^{cr} at all chainages. So the support installed as adopted by the project is adequate but the Numerical method shows inadequate support at chainages 4+500 m and 5+050 m and needs revised support.

Acknowledgments

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