Numerical Modeling for Engineering Analysis and Designing of Rock Support for Headrace Tunnel

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Abstract - *This paper is focused on the empirical and numerical design approaches which are very important for the design of support system in Underground excavation. This research consider rock mass classification and finite element analysis for the rock support estimation and analysis. The tunneling quality index (Q-system) and rock mass rating (RMR) were used as empirical methods based on real time geological and site geotechnical data for rock mass classification. The physical and strength properties of the rock samples collected from the project office. These supports are compared to the project support and found similar. All the support systems were calculated analytically for critical support pressure and numerically 2D elastoplastic finite-element method (FEM). The analysis was used for the study of rock mass behavior, in situ and redistribution stresses, plastic zone thickness around the tunnel, and performance of the design supports. Based on the result, for the estimation of tunnel support finite element analysis is most important tools to optimize the support instead of designing support from rock mass classification approach.*

Key Words: **Rock Support, Rock Mass Classification, RMR, Q, Intact Rock, Finite Element Analysis, Support Capacity**

1. INTRODUCTION

In the Himalayan region of Nepal, very steep slope and high velocity of water in the river tends to increase the possibilities of Hydro-Electric projects. In the hydropower development, some project use pipe but most of the project use tunnel to carry water in low gradient so that a significant head can gain at some distance. While tunnel construction, excessive tunnel deformation and support failure occurs where very high overburden pressure appears and encountered very weak rock masses during the construction. In such case support designing becomes very challenging. Selection and design of support systems are only two of many inter-related factors in the overall design of serviceable and economical tunnel. The type of support, the method of excavation, and the character of ground are inseparable considerations for support design.

The reliable rock mass properties are much efficient for the design and construction of rock tunnel structure with safe excavation and satisfactory performance (Hoek and Brown 1980a).

Therefore, support required for a tunnel in rock is a complex function of the properties and condition of the rock, the geometry and orientation of the tunnel, and the construction procedures. The tunneling has the stability is the main problem, sufficient support may require for the safety of tunnel alignment.

Geological condition of throughout tunnel alignment must be done carefully for the investigation of rock type, discontinuities and other parameters. These field study directly affect the rock support and hence economize the project and completion on time. Intact rock strength varies widely within same rock because of rock weathering, that is depends on different mineralogical contents, therefore direct measurement is recommended (Panthee et al, 2016a).

Analysis and design of tunnel support by using empirical, analytical and numerical methods are common. In this study, Super Madi-Hydro-Electric Project headrace tunnel section was selected for the case study, which is an under construction project located in the lower part of Higher Himalayan Region of Nepal.

The specific objectives of this research work are the analysis of support design for different ground condition using empirical and numerical method by Finite Element Method and compare these result.

2. DESCRIPTION OF PROJECT AREA

Super Madi Hydroelectric Project is located in Kaski Districts, Provience number 3 of Nepal. The project lies in the Namarjun and Parche Village Development Committees of Kaski District. The headworks is located at the foothill of the Sikles Village and the powerhouse is located just opposite of Sodha village. In general, this project has the installed capacity of 44 MW; design discharge of 18 m^3/s. Net head of 295m and net saleable annual energy is 243.125GWh. This project is simple run-off hydropower project (Himal Hydro 2009). The figure 1 shows the geological map of country with location of selected case study. This project is lies on the lower part of Higher Himalayan Region.

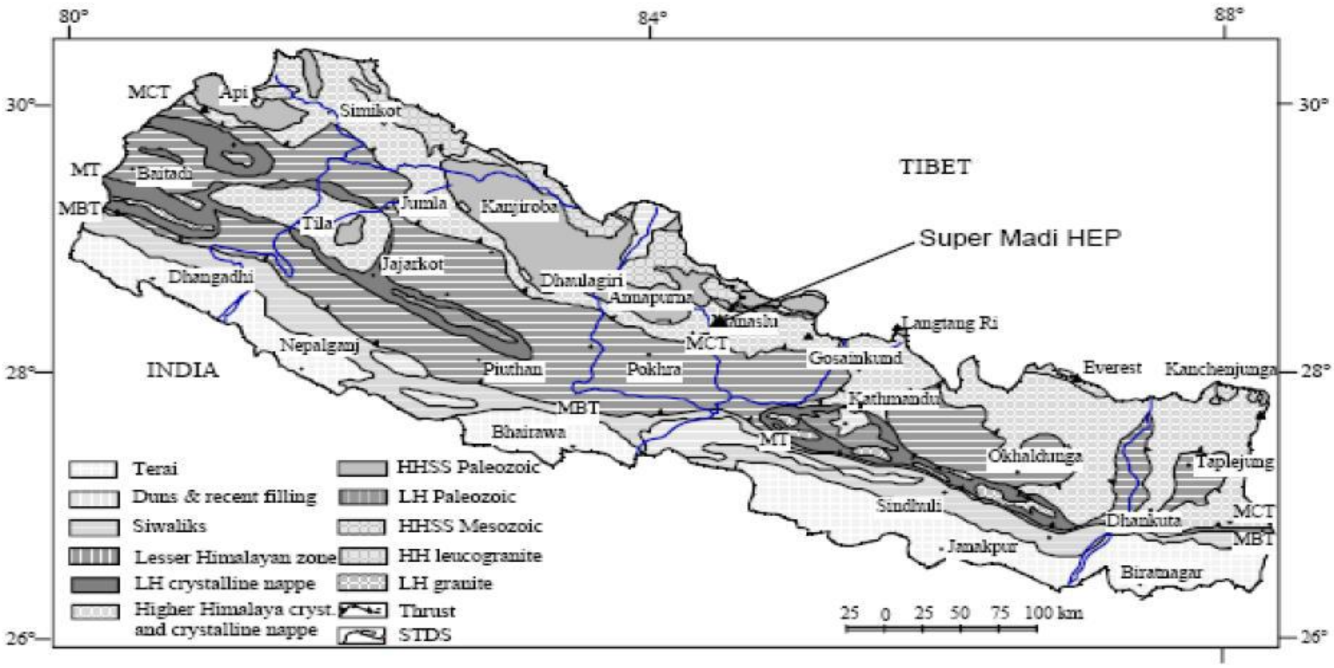

Fig - 1 Geological map showing the location of selected case study (HH, 2009)

3. GEOLOGY ALONG TUNNEL ALIGNMENT

Geologically, project area lies in the Higher Himalayan succession. Higher Himalayana is sandwiched between the Southern Tibetan Detachment System (STDS) in north and the Main Central Thrust (MCT) in south. The MCT is the major regional thrust in Himalayan which lies in about 2 km (aerial distance) south from the proposal powerhouse area.

This zone comprises mainly high-grade metamorphic rocks such as Kyanite-silliminitae bearing gneiss, schist and quartzite. Geologically the project location belongs to Higher Himalayan Crystalline Zone consisting of percambrian gneiss. The main lithology of the project area is banded gneiss micaceous gneiss, schist and garnetiferous schist. The banded gneisses are fresh to moderately weathered, whereas the mica gneiss is moderately to highly weathered. The overall rock mass condition of the project area is fair to good which is thickly to massively foliated, slightly fractured to highly fractured with intercalation of quartzite and schist.

The geological condition of the inlet tunnel area, underground settling basin, flushing tunnel and adit-1 are fair to good and favorable. The rock mass of the area is slightly deformed and foliated banded gneiss with few partings of schist. The less deformed massive rocky mountain with sufficient vertical and horizontal rock cover above and valley side is a perfect location for the underground tunnel. The geological condition of the area is favorable even for bigger size. The overburden is 306.42m at first section (i.e. chainage 1+000m) and tends to decrease along the alignment and exposed to the Kalbandi Khola around chainage 1+600m.

The head race tunnel is aligned along the left hill side of the Madi River. The total length of headrace tunnel is approximately 5.28km excluding settling basin. The tunnel will be exposed at both banks of Kalbandi Kholsi which will be used as audits during construction and shall be connected by penstock pipe at the final stage with river crossing structure. The rock condition along the tunnel are interpreted by extrapolation of rock mass condition from the tributaries and foot trails in the vicinity of tunnel alignment. The maximum overburden is approximately 450m at section between Kalbandi and Ghatte Khola. Folding and shearing are quite significant along the surface slopes.

Chainage (1+000) To (1+300)

The tunnel alignment makes less angle with the strike of major discontinuity with the excavation driving against dip. The rock overburden within this stretch is between 200m to 300m. This tunnel section consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is dry to damp. The overburden varies from 306.42 m to 219.05m along the tunnel alignment in this section. Figure 2 and 3 indicate the geology along the tunnel in plan and overburden in profile.

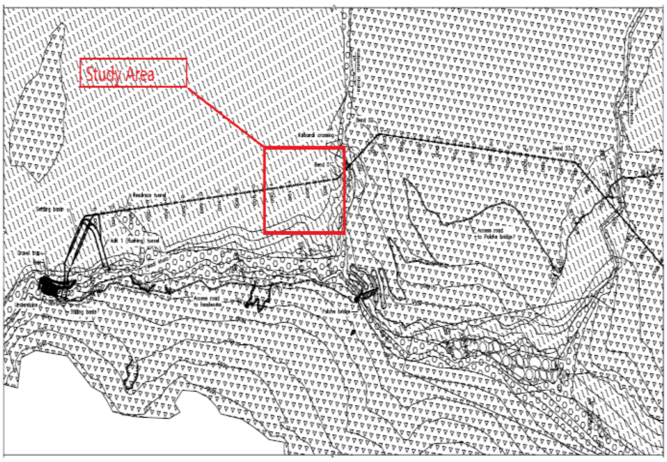

Fig - 2 Geology along Headrace Tunnel-Plan- (Himal Hydro-2009)

Chainage (1+300) To (1+400)

The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss. Three major joint sets along with other joints shall be encountered. The rock overburden within this stretch is between 15m to 225m. The Excavated tunnel section consists gray colored, medium grained, foliated, moderately weathered, Medium strong to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 10-60 cm thick. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3 mm) aperture with clay coating and filling in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp. This section is further exposed to the Kalbandi Khola at around 200m ahead. (Himal Hydro 2009). The overburden is less compared to previous section with minimum 167.63m at the last chainage. This section carries less deformed rock mass with less frequency of joint set.

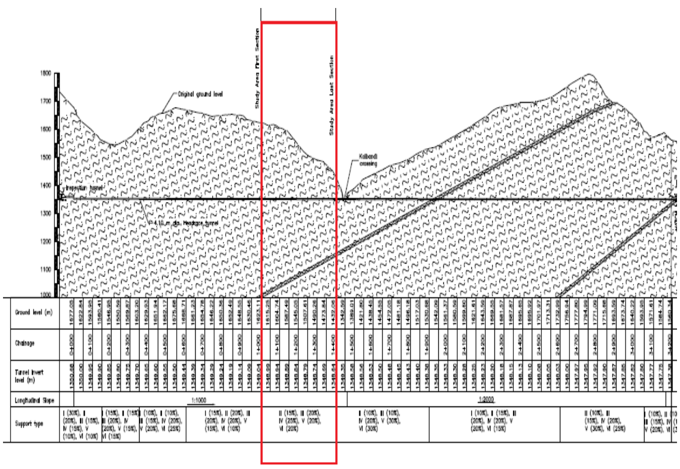

Fig - 3 Geology along Headrace Tunnel-Profile (Himal Hydro 2009)

4. TUNNEL GEOMETRY

The tunnel excavation size without pay line is 4.2 m*4.2 m (B*H) which is clearly shown in figure 4. The elevation of the tunnel section of this study varies from 1444.68 m to 1634.69m above mean sea level. The tunnel is exposed at the bank of Kalbandi Kholsi which will also be used as adits during construction and shall be connected by penstock pipe at the final stage with river crossing structure. Overburden in the selected section for this study varies from 116.66m at Kalbandi Kholsi to 306.42 m at 500m upstream from the Kholsi.

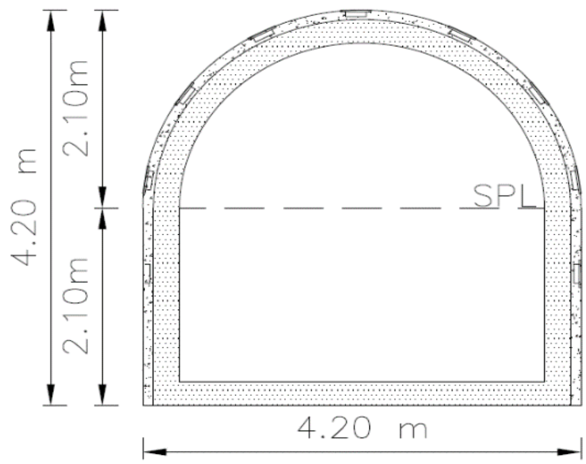

Fig - 4 Tunnel Geometry

5. ROCK MASS CLASSIFICATION

Various rock mass classification system has been developed based on civil and mining engineering case studies by different researchers, among all rock mass rating (RMR) of Bieniawski (1976) and tunneling quality index (Q-system) by Barton in 1974 were used in this research for assessment and classification of rock mass.

All the required information was collected from the site and calculated Q value using Eq. 1. Q value is found from lowest 0.038 at chainage 1+200m to 1.25 at chainage 1+300m. The Q value for the chainage 1+000m to 1+100m is found between 0.2 to 0.4 that was classified as Rock Class E, i.e. very poor rock mass. And this is followed by extremely poor rock of class F because of Q value is 0.038 to 0.075 up to chainage 1+250m. From chainage 1+300m to 1+350, the rock class D, i.e. poor rock mass. From this classification we can divide the study section at three basic category of Poor Rock, Very Poor Rock and Extremely Poor Rock.

Similarly, the RMR value was found from 19 to 58 along the section. This can categorize the study section among three categories, first one is from chainage 1+000m to 1+150m as IV rock mass class i.e. poor rock class followed by rock class of V i.e. Very Poor Rock up to chainage 1+250m and after that rock class of III i.e. Fair Rock mass.

Table - 1 Q and RMR Value along the Tunnel Chainage

CHAINAGE								
1+000	1+050	1+100	1+150	1+200	1+250	1+300	1+350	1+400
Q -VALUE								
0.344	0.583	0.229	0.075	0.038	0.070	1.250	1.083	0.271
ROCK CLASS								
VERY POOR	*Very Poor*	*Very Poor*	*Extremely Poor*	*Extremely Poor*	*Extremely Poor*	*Poor*	*Poor*	*Very Poor*
RMR								
35	35	35	19	19	19	58	58	35
ROCK CLASS								
POOR	*Poor*	*Poor*	*Very poor*	*Very poor*	*Very poor*	*Fair*	*Fair*	*Poor*

6. ROCK MASS PROPERTIES

The in situ stresses are determined by using the overburden determined from the profile of the study section. The developed empirical models were used for determination of vertical and horizontal stresses. In this study, the vertical stress was determined by:

$$\sigma_v = \gamma H \quad \text{Eq. 2}$$

$$\sigma_h = \left(\frac{\nu}{1-\nu}\right)\sigma_v \quad \text{Eq. 3}$$

The rock joint parameter like spacing, separation, persistence, play the significant role on tunnel deformation. The study of joint parameter must be taken seriously for the accurate deformation analysis. (Panthee 2016b). The rock mass along the tunnel axis was classified into three geotechnical units (G-1, G-2, and G-3).

Bieniawski (1978) defines Emass as below:

$$E_{mass} = 2RMR-100 \quad \text{Eq.}$$

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Serafim and Pereira (1983) proposed the following formula

$$E_{mass} = 10^{\frac{RMR-10}{40}} \quad \text{Eq. 5}$$

Grimstad and Barton (1993) suggested the following equation for calculating E_{mass} for Q>1 and generally for hard rock:

$$E_{mass} = 25 \log Q \text{ (GPa)} \quad \text{Eq. 6}$$

Hoek and Brown (1998) found a correlation between Emass and GSI:

$$E_{mass} = \sqrt{\frac{\sigma ci}{100}}\, 10^{\frac{GSI-10}{40}} \text{ (GPa)} \quad \text{Eq. 7}$$

Read et al. (1999) proposed the below equation for calculating E_{mass} based on RMR value of rock mass:

$$E_{mass} = 0.1\left(\frac{RMR}{10}\right)^3 \text{ (GPa)} \quad \text{Eq. 8}$$

The deformation modulus of a rock mass (E_m) is an important parameter in rock mechanics and engineering and its determination is a difficult task. Therefore, several equations were used in this research as suggested S. Panthee 2016.

Hoek and Brown (1980b):

$$\sigma_{cm} = \sigma ci.\, exp^{\left[\frac{RMR-100}{18}\right]} \quad \text{Eq. 9}$$

Yudhbir et. al. (1983):

$$\sigma_{cm} = \sigma ci.\, exp^{\left[\frac{7.65(RMR-100)}{100}\right]} \quad \text{Eq. 10}$$

Ramamurthy (1986):

$$\sigma_{cm} = \sigma ci.\, exp^{\left[\frac{RMR-100}{18.75}\right]} \quad \text{Eq. 11}$$

Kalamaris and Bieniawski (1995):

$$\sigma_{cm} = \sigma ci.\, exp^{\left[\frac{RMR-100}{24}\right]} \quad \text{Eq. 12}$$

Shoery (1997):

$$\sigma_{cm} = \sigma ci.\, exp^{\left[\frac{RMR-100}{20}\right]} \quad \text{Eq. 13}$$

Barton (2002):

$$\sigma_{cm} = 5\gamma \left[Q \left(\frac{\sigma ci}{100}\right)\right]^{1/3} \quad \text{Eq. 14}$$

Hoek et. al. (1980):

$$\sigma_{cm} = \sigma ci.\, s^a \quad \text{Eq. 15}$$

These group are form under the Q-values Classification. The support systems for each geotechnical unit were designed. First group contain the rock mass of poor quality and rock mass of very poor and extremely poor are categorized in second and third group respectively.

Rock mass properties such as Hoek–Brown constants, deformation modulus of rock masses (table 2) and uniaxial compressive strength of rock mass (table 3) were calculated using empirical relations. The deformation modulus and uniaxial compressive strength of of rock mass was calculated by taking average of the determined values from various empirical relations mentioned from equation 4 to equation 15.

Table - 2 Deformation Modulus of Rock Mass

Group	*Em*					*Average Em*
	Bieniawski (1978)	Serafim and Pereira (1983)	Grimstad and Barton (1993)	Read et al.	Hoek & Brown (1998)	
G_01	-	4.217	-	4.2875	3.113	3.872
	-	4.217	-	4.2875	4.094	4.200
	-	4.217	-	4.2875	2.523	3.676
G_02	-	1.679	-	0.6859	1.415	1.260
	-	1.679	-	0.6859	0.988	1.117
	-	1.679	-	0.6859	1.368	1.244
G_03	16	15.849	2.422750325	19.5112	6.076	11.972
	16	15.849	0.869052656	19.5112	5.642	11.574

Table - 3 USC for the Rock Mass

Group	σ_{cm}						
	Hoek & Brown (1980b)	Yuhbir et al (1983)	Ramamurthy (1986)	Kalamaris & Bieniawski (1995)	Sheorey (1997)	Barton (2002)	Average
G_01	6.755	1.731	7.805	16.662	9.694	14.051	9.450
	6.755	1.731	7.805	16.662	9.694	23.844	11.082
	6.755	1.731	7.805	16.662	9.694	9.367	8.669
G_02	2.777	0.509	3.325	8.555	4.356	3.066	3.765
	2.777	0.509	3.325	8.555	4.356	1.533	3.509
	2.777	0.509	3.325	8.555	4.356	2.874	3.733
G_03	24.243	10.059	26.615	43.443	30.614	51.094	31.011
	24.243	10.059	26.615	43.443	30.614	44.281	29.876

7. ROCK SUPPORT ESTIMATION

Based on the Q values and RMR Values, underground rock support was estimated at each section separately (table 4). From the Q chart, the support was estimated for Q value and ratio of tunnel span to ESR. The support for the roof and wall are look similar that recommend to use a single type support for each section. From the Bieniawski, 1989 support table required supports are estimated. Among these two method the Q chart gives the most adequate and economic support for the tunnel which are further taken for the numerical analysis with some modification in reference of RMR system.

Using the Q chart based on Q value at each section the rock support is estimated, rock bolt of 2 meter length with spacing 1 m to 2m center to center is estimated at various section shown in table 4. The concrete shotcrete is also estimated for the crown and wall. The Q- value is very low at chainage 1+150m to 1+250m, so the steel ribs are estimated.

Table - 4 Rock Support Estimation & Parameter for Numerical Modelling

Chainage	*Unit Weight (g/cm3)*	*Poisson's Ratio (v)*	*GSI*	*Hoek & Brown Constant*			*Q- System Support*	*RMR Support*
				mb	s	a		
1+000	2.60	0.19	34	0.56	4.81562E-05	0.517	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 50 mm, RRS I	Systematic 20mm diameter bolts 4 - 5 m long, spaced 1–1.5 m in crown and wall with wire mesh, shotcrete with a thickness range of between 100mm and 150m in crown and 100mm in the sides of tunnel, and Light to medium ribs spaced 1.5 m steel set where required
1+050	2.60	0.19	39	0.75	9.90537E-05	0.512	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 40mm	
1+100	2.60	0.19	31	0.45	2.77038E-05	0.521	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 50 mm, RRS I	
1+150	2.60	0.19	21	0.25	6.0408E-06	0.542	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 100 mm +RRS II	Systematic 20mm diameter bolts 5-6 m long, spaced 1–1.5 m in crown and walls with wire mesh; bolt invert, shotcrete with a thickness range of between 150mm and 200m in crown and 150mm in the sides of tunnel and 50mm on face, and Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required; close invert
1+200	2.60	0.19	14	0.17	2.1929E-06	0.565	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 150 mm +RRS II	
1+250	2.60	0.19	20	0.25	5.5318E-06	0.543	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 100 mm +RRS II	
1+300	2.60	0.19	46	1.13	0.00028	0.508	Systematic Bolt of φ-20mm, length 2m at 2m c/c, unreinforced shotcrete of 50mm	Systematic 4m long systematic bolts of 20mm diameter and fully grouted, spacing range between bolts of 1.5–2m in crown and walls with wire mesh in crown, shotcrete with a thickness range of between 50mm and 100m in crown and 30mm in the sides of tunnel, and no steel set required
1+350	2.60	0.19	45	0.48	3.35313E-05	0.520	Systematic Bolt of φ-20mm, length 2m at 2m c/c, unreinforced shotcrete of 50mm	
1+400	2.60	0.19	32	0.49	3.47942E-05	0.522	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 50 mm, RRS I	Systematic 20mm diameter bolts 4 - 5 m long, spaced 1–1.5 m in crown and wall with wire mesh, shotcrete with a thickness range of between 100mm and 150m in crown and

	100mm in the sides of tunnel, and Light to medium ribs spaced

8. NUMERICAL MODELLING

For numerical analysis, the finite element software, Phase[2] developed by RocScience has been used in this study. 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. 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. Finite Element Method reduces the degrees of freedom from infinite to finite with the help of discretization or meshing (nodes and elements). One of the purposes of meshing is to actually make the problem solvable using Finite Element. An inverted D-shaped model of the tunnel has been prepared with the excavation boundary as five times the diameter of the tunnel. The disturbance factor of 0.8 has been considered to account for the disturbances from the drill and blast method of tunneling in this region. Selected all the nine 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 support install stage. Support is installed on the third stage and the deformation is noted during supported condition. The support is provided according to the Q values with some modification and estimated using analytical method. The Phase model is prepared including the joint properties at the sections. Material Properties and other input parameter are taken from the site condition data and some are calculated using empirical relations.

Rock joints can be subject to different types of boundary conditions in the field ranging from constant normal stress to constant normal displacement.

Joint shear strength depends on the nature of those boundary conditions. Fully grouted rock bolts are often used for systematic rock bolting. Shotcrete with concrete grade M30 is applied as liner.

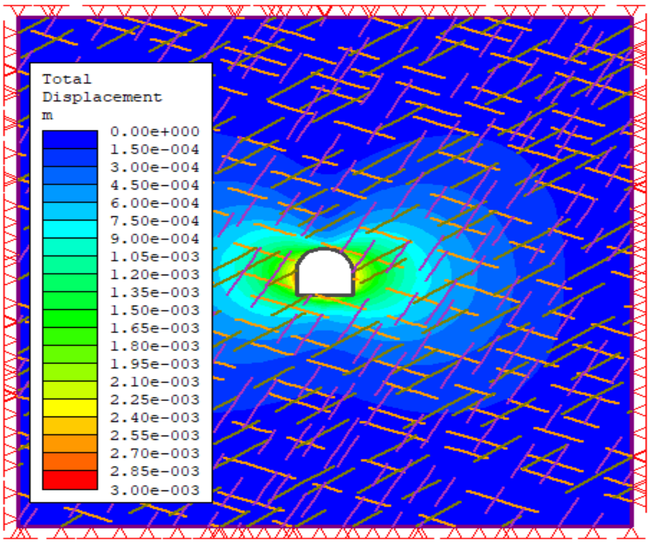

Fig - 5 Maximum Displacement at Chainage 1+000m

The input parameter of support liner used in this research are listed in table-5. Steel Ribs ISMB 150*6.6 Kg/m³ and Concrete of grade M_{30} is used.

Table - 5 Material Properties of Concrete & Steel Ribs

Parameter	*Unit*	*Steel Ribs*	*Concrete*
Young's Modulus	GPa	200	30
Poisson's Ratio		0.25	0.25
Sectional Depth	m	0.152	-
Cross-Sectional Area	m^2	0.000832	-
Moment of Inertia	m^4	3.01E-06	-

The strength factor at all the section is found greater than one there for this model needed plastic analysis for the stability.

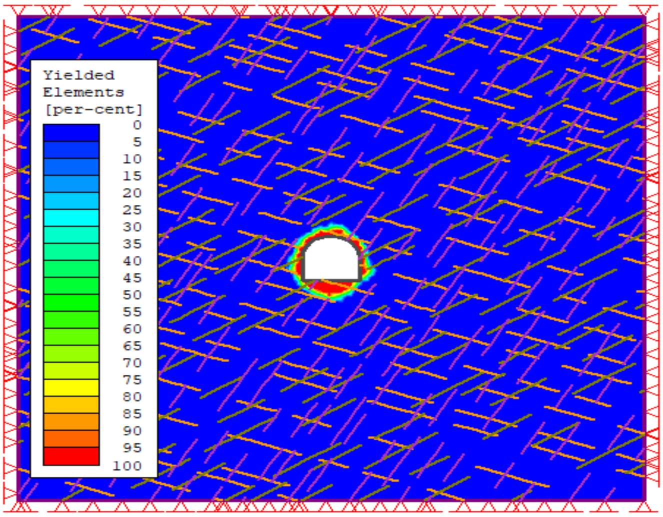

Fig - 6 Phase Model Showing Plastic Zone at Chainage 1+000m

The above two figure 5 & 6 shows the maximum displacement and yielded portion of tunnel opening at chainage 1+000 from which we can determine the radius of plastic zone.

9. SUPPORT CAPACITY ANALYSIS

The selected tunnel sections were analyzed by numerical method for support system according to Rock Mass Classification. The provided support failed at bottom corners of the excavation at some section. 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 is provided generally and steel sets was also increased in order to limit the thickness of the concrete lining.

The support system from the Rock Mass Classification was also compared to the support adopted by the project and these support can be reduced using analytical method by comparing support pressure and critical support pressure. All the three model were tested in phase model for the stability analysis.

The total displacement of the tunnel is calculated at each section. Then the extend of the plastic zone (Rp) is about is measured from yielded portion of the interpret model. For the ratio of distance from tunnel face to tunnel radius (X/Rt) is 1.19 and plastic zone to tunnel radius (Rp /Rt). By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal. Therefore, the closure was found. That represent the total deformation will already take place before support is installed. The support was applied after the tunnel relaxation stage. The supports are applied and analyzed after this stage. The support s are applied and check for the safety using support capacity plot. At every stage the support capacity plot was analyzed for different rock support. The minimum support for which it shows safe capacity plot is taken at the last. All these result were also compared to the rock support applied by the project and found over support, it may be for better safety factor.

At chainage 1+100m the estimated support is failed for the shear force and moment that can be redesigned for the better stability. Similar process of redesign was also applied to 1+200m and 1+350m. Using the numerical model support system were optimized at each section.

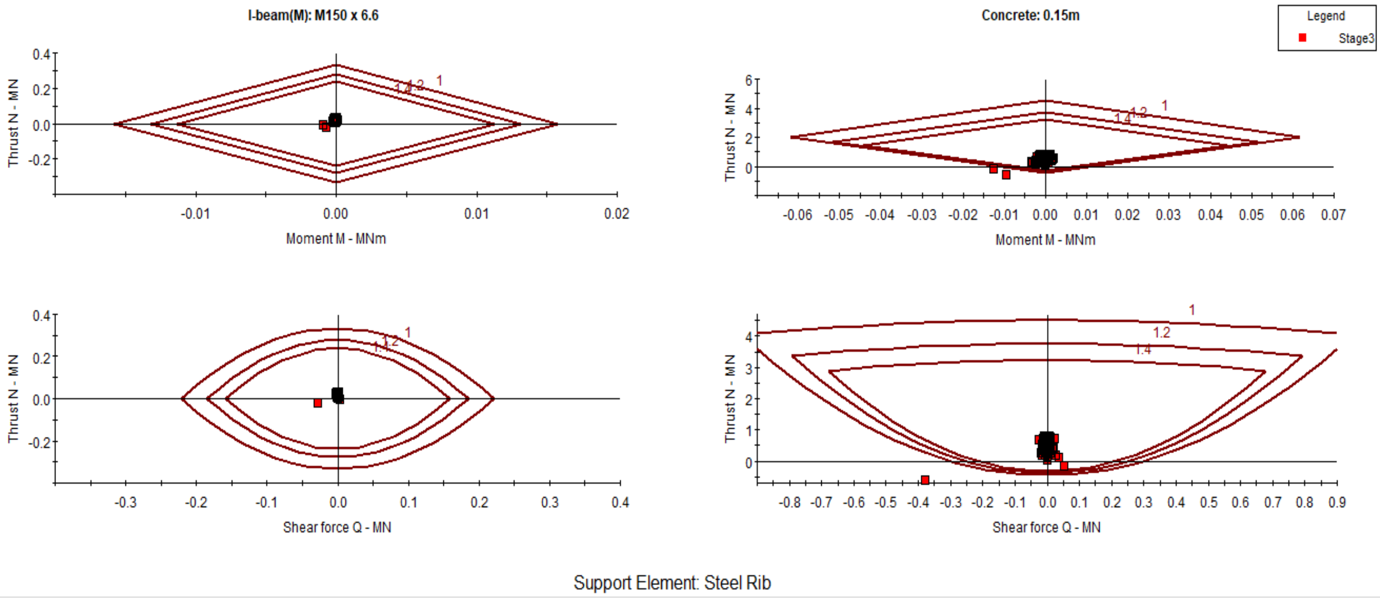

Fig - 7 Support Capacity Plot at Chainage 1+100m

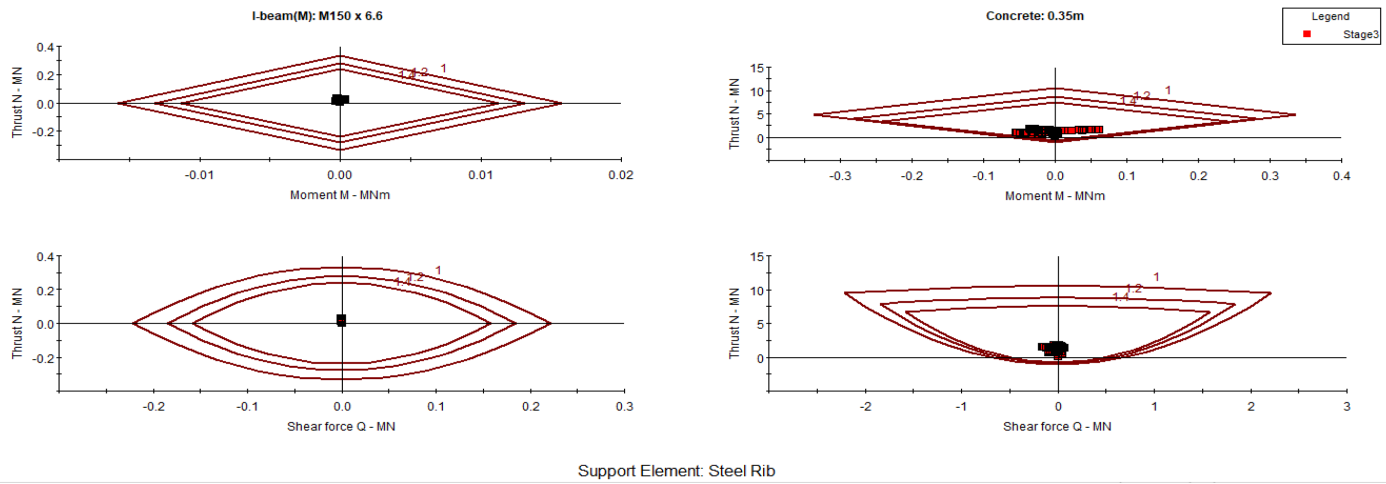

Fig - 8 Support Capacity Plot at Chainage 1+100m after redesign

Among all the selected section 1+200m section is classified as extremely poor section. The Discontinuities at that section is very close and the Q-value is 0.038. For the stability analysis of rock support at this section the concrete liner is used of 200mm and steel ribs applied very close.

All the section were tested in phase model for the various support system estimated from Rock Mass Classification, from critical support pressure criteria and support applied by the project. Rock support applied at chainage 1+100m was failed at the initial stage than it would be again analyzed.

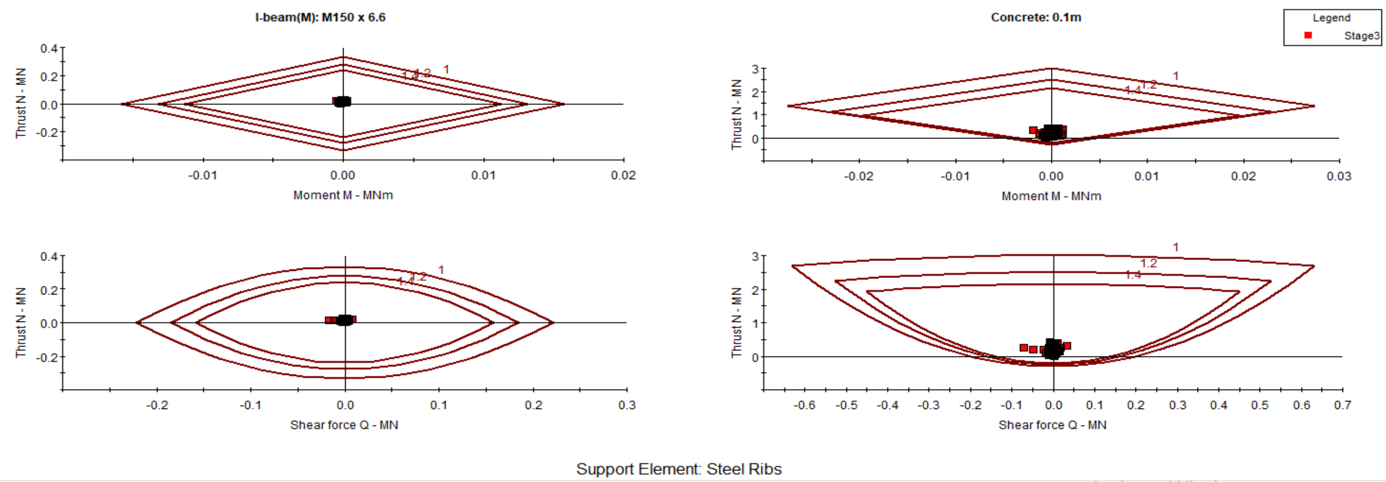

Fig - 9 Support Capacity Plot at Poor Rock 1+300m

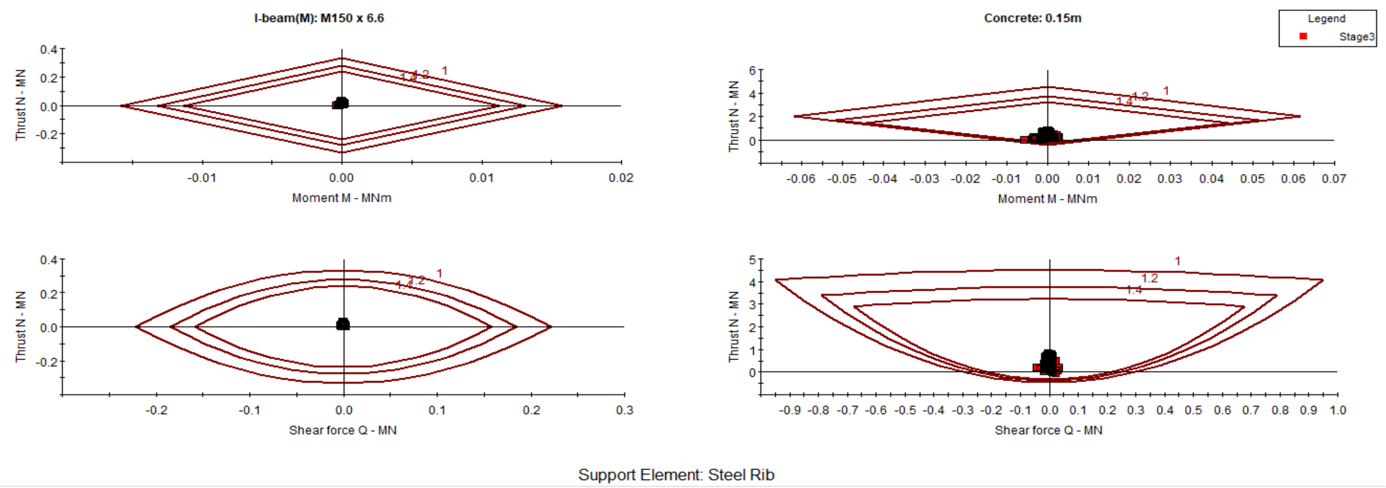

Fig - 10 Support Capacity Plot at Very Poor Rock 1+050m

The rock masses are classified into three main category that is poor rock, very poor rock and extremely poor rock mass along the study section. The chainage from 1+000m to 1+150m is very poor rock mass, chainage from 1+150m to 1+250m is classified at extremely poor rock mass and at chainage 1+300m and 1+350m is classified as poor rock mass. The support system is studied among the classified three rock mass quality for general support recommendation. In this research supports are optimized for the safety and economic purpose. At each section we have did separate finite element analysis for the optimum support design and these minimum support were tested and the support capacity plot is shown in figure 9, 10 and 11 respectively for poor, very poor and extremely poor rock class.

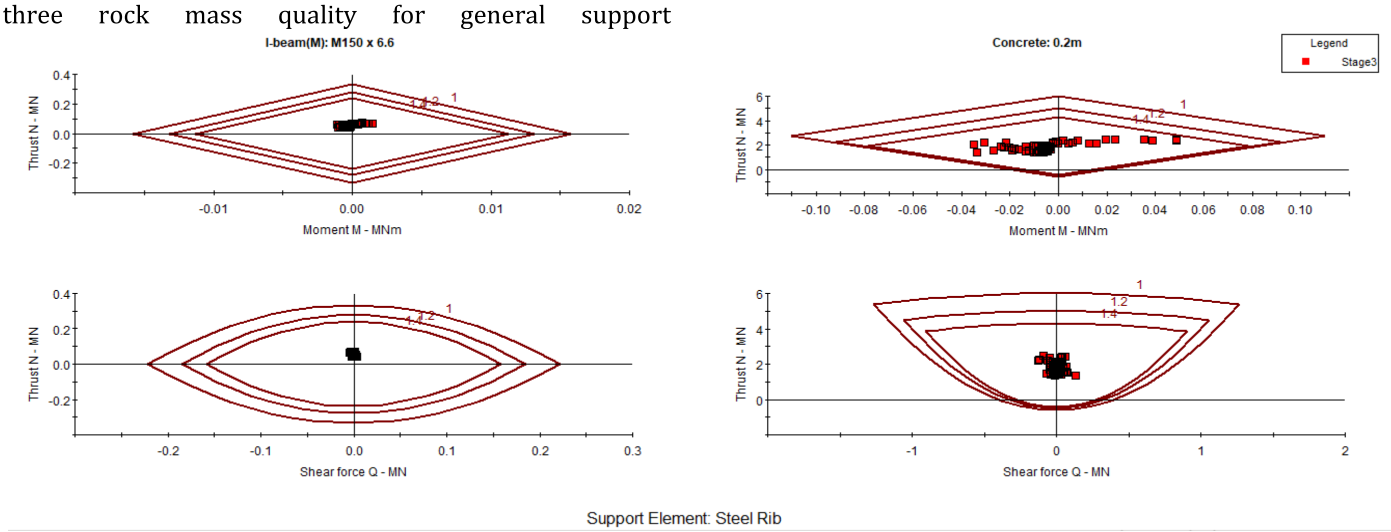

Fig - 11 Support Capacity Plot at Extremely Poor Rock 1+200m

10. DISCUSSION & CONCLUSION

In this study, rock mass classification is carried out in conjunction with numerical analysis to study the support requirement of the headrace tunnel of Super-Madi Hydroelectric Project, located in the Lesser Himalayan Region of Nepal. These simulated models were developed based on the following assumptions: Supports were installed instantly after excavation, Elastoplastic behavioral model using generalized Hoek–Brown criterion is used to simulate the models and Tunnel model is 2D considering plane strain problem.

For numerical analysis, three-stage models were adopted to conform the in situ ground stresses. In First stage of simulated model, ground stress distributions were examined. In the next stage, induced stress distributions, yield points, and the induced displacement were analyzed. In the final stage, behavior of the recommended support systems was investigated.

The deformation was calculated using the empirical relations given by E. Hoek et. al. The critical pressure, radius of plastic zone and total inward displacement were calculated and compare with the result obtained from finite element method. 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 is presented on figure 12. The highest displacement was found at chainage 1+200m that it has lowest Q value of 0.038 and the very close and much fractured joint present on that section. The critical support pressure was calculated using the relation:

$$P_{cr} = \frac{2Po - \sigma cm}{1+k} \qquad \textit{Eq. 16}$$

If the internal support pressure is greater than the critical support pressure, no failure occurs and the behavior of the rock mass surrounding the tunnel is elastic. The inward radial elastic displacement of the tunnel wall is given by:

$$u_{ie} = \frac{ro\,(1+\nu)}{E}\,(Po - Pi) \qquad \textit{Eq. 17}$$

Where E is the young's modulus or deformation modulus and ν is the Poisson's ratio.

When the internal support pressure is less than the critical support pressure failure occurs and the radius of the plastic zone around the tunnel is given by:

$$r_p = r_o\left[\frac{2(Po(k-1)+\sigma cm)}{(1+k)((k-1)Pi+\sigma cm)}\right]^{\frac{1}{k-1}} \qquad \textit{Eq. 18}$$

The total inward radial displacement of the walls of the tunnel is given by:

$$u_{ip} = \frac{ro\,(1+\nu)}{E}\left[2(1-\nu)(Po - Pcr)\left(\frac{rp}{ro}\right)^2 - (1-2\nu)(Po - Pi)\right]$$

Eq. 19

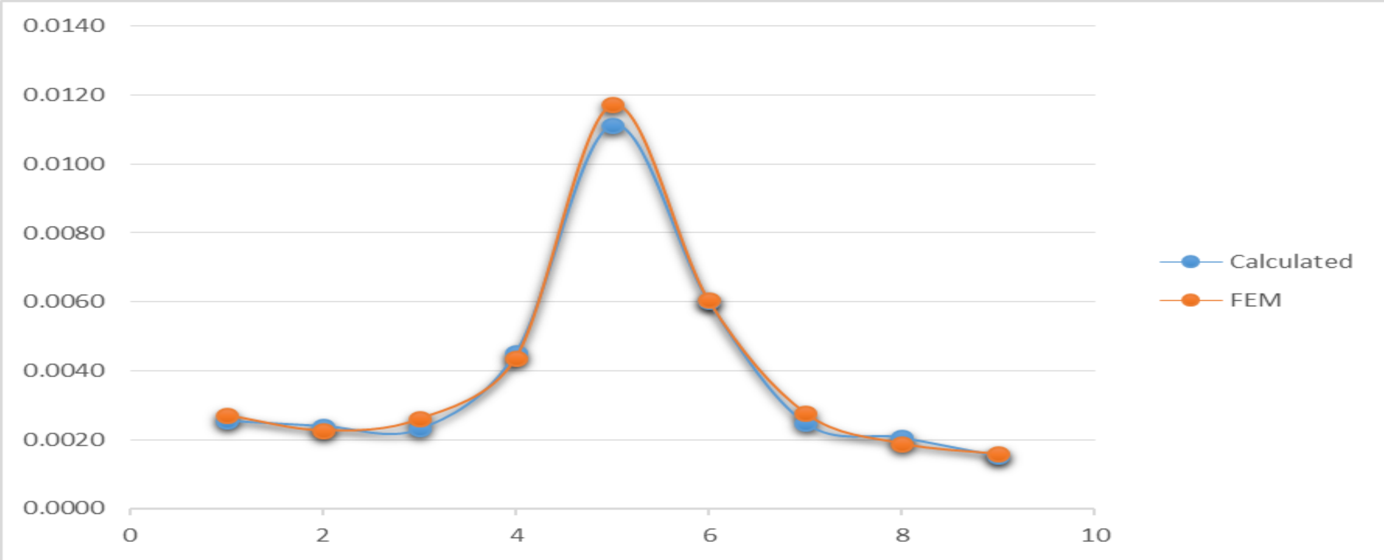

Chart - 1 Total Displacement from Calculated and FEM.

The stability analysis of models developed for each geotechnical unit in Phase2, was carried out after installment of support estimation from rock mass classification (Q & RMR system), Analytical Method (using critical pressure criteria) and support used by the project. From all the result and discussion of that, this paper conclude as following:

- Some input parameter for the numerical analysis were calculated form empirical relations.
- The estimated support are found over estimated at some section using Q-chart. The supports are than reduced and redesigned. The modified support were applied to the model and support capacity plot was observed for the stability of tunnel. It showed that the modified support can safely control the deformation in the tunnel.
- The support capacity plot are very useful for the analysis of rock support estimation. That can be used for the optimizing supports.
- 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.
- Effect of ground water is not considered as it may create problem during excavation. Therefore, it is suggested to make drain holes to pass out the possible water.

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