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Fault and Shear zone Induced Georisks in Tunnel: A case study of Rahughat Mangale Hydroelectric Project, Myagdi, Nepal

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Abstract

This paper focuses the case study of georisks in the hydropower tunnels of Nepal. The headrace tunnel of Rahughat Mangale Hydroelectric Project has several shear zones, encountered overbreak and rock squeezing problem in different chainages, selected for the study of georisks. Serious georisks in tunnel construction is generated by faults and shear zones. The presence of fault in the rock mass increases the existing in-situ stress beyond its critical level and strength. Due to which squeezing and overbreak are frequently encountered in sheared, schistosed, deformed and jointed rock-masses. This study discussed the characterized rock-mass, behavior of the unsupported rock mass, and analyzed support. A numerical analysis is carried out using the geological data, shear zone and rock mass of the tunnel along with comparision between the analysis result of tunnel with and without fault and shear zone.

Keywords

Fault, Shear zone, Georisks, Rock squeezing, Overbreaks

1. Introduction

Geologically, Nepal is divided into five tectonic zone from south to north respectively; the Gangetic plane (Terai), the Siwaliks zone, the Lesser Himalayan zone, the Higher Himalayan zone and the Tibetan-Tethys zone separated by Main Frontal Thrust(MFT), Main Boundary Thrust (MBT), Main Central Thrust (MCT), South Tibetan Detachment System(STDS). MBT and MFT are active faults, which creates major difficulties during tunnel construction whereas MCT is not active and will not create major problems. There are also some minor faults and shear zones whose thickness varies from few meter to tens of meter. Some examples are the Barigad fault, Talamarang Fault, Baseri fault, and Dhabang Fault,etc.[1]

In Nepal Himalaya, generally two types of shear or fault zone can be identified, Ductile shear or fault zone and Brittle shear or fault zone. Ductile fault or shear zone generally occurs at the deeper parts of earth crust with no significant change in mechanical properties of rock mass with minimum effect in construction of tunnel whereas brittle fault or shear zone occurs close to earth surface with sheared rock matrix of strong to very weak blocks which can be problematic during tunnel construction. Faults and shear zone are undesirable geological structure found in rock of earth's crust which can be planner or gently curved. In such zones, compressional or tensional forces causes relative displacement on the opposite sides of fracture. During Tunneling in fault or shear zones, we should consider frequently changing rockmass,groundwater conditions and long term deformation in comparision to section without fault or shear zone. Squeezing, collapse, flow ground are main instability conditions in faulted rock. Tunneling through faulted or sheared rock mass causes geotechnical difficulties such as deformation from squeezing, swelling of faulted rocks, excessive over breaks, instability of the face.[2]

2. Brief on the Project

Rahughat Mangale Hydroelectric Project (RMHEP) is located in Myagdi District, Gandaki Province of Nepal. The project components of RMHEP are located in Rahuganga Rural Municipality. The Project shall utilize the design discharge of 11.6 m3/s and the gross head of 365 m through a 5180 m long headrace tunnel. To facilitate the excavation of the headrace tunnel two construction adits have been provisioned.[3]

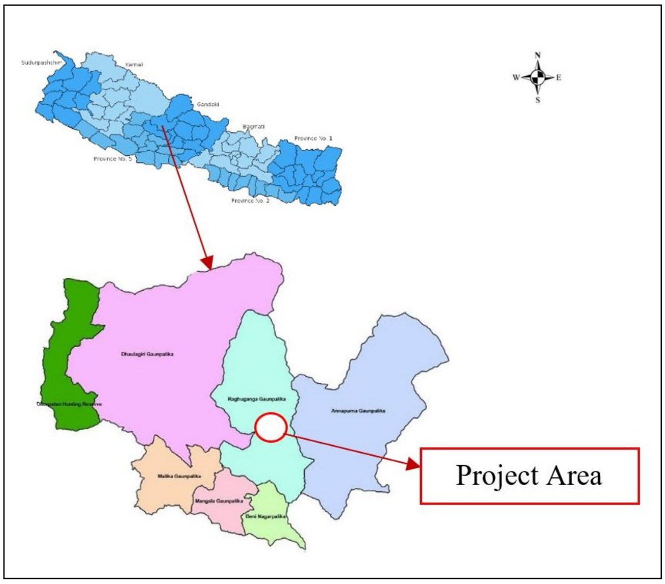

Figure 1: Location of Rahughat Mangale HEP

2.1 Geology of the area

Project lies in the Lesser Himalaya which occupied by low grade metamorphic rocks and structurally located on the south of the Main Central thrust (MCT). The main rocks types found in the project area are phyllite, slate and quartzite belonging to Kunchha Formation, Benighat Slate and Fagfog quartzite.Majority of tunnel alignment will pass through weak to medium strong and thin to medium foliated phyllite with alternating quartzite with shear or weak zones.

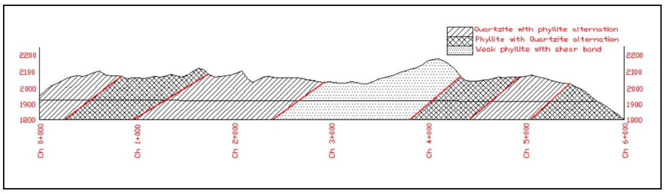

Figure 2: Geological profile of RMHEP [3]

2.2 Faults and shear zones identification

During the tunneling, the majority of fault and shear zones were found parallel to the foliation plane and minor steep shear zone across the foliation plane. Faults encountered during the tunneling was clayey brittle fault majority developed in Phyllite. In clayey brittle fault, Clay content is high and impermeable natures holding water which causes the stability problem during the construction of the tunnel. Similarly in blocky brittle fault and shear zone, clast contain is more than that of clay which enables ground water flow in tunnel and affects its stability. The field mapping data of the tunnel excavation and test report of rock mass along different section have been studied. Only shear zone, RMR and presence of water are mapped during field mapping. From the observed data, two types of shear zones are identified, i.e, Clayey Brittle shear zone and Blocky Brittle shear zone as shown in Table 1.

Table 1: Shear zone classification

HRT Chainage m	Rock Type	Shear or Fault zone
0+061	Quartzite	Blocky brittle shear zone
1+650	phyllite	clayey brittle shear zone
3+750	phyllite	clayey brittle shear zone

Knowing the information about the type of shear zone inside the tunnel, the georisks that can occur due to specified shear zone and other shear band can be predicted and analyzed.

2.3 Rock mass Classification

The rock mass classification by the Q-system [4] along the headrace tunnel (HRT) was carried out along its excavated 5180 m length of tunnel alignment during tunnel excavation. The rock mass having Q-Value greater than 1($Q > 1$) is considered Class I rock mass, Q-value between 1 and 0.4 ($1 > Q > 0.4$) is considered as Class II rock mass, Q-value between 0.4 and 0.1 ($0.4 > Q > 0.1$) is considered as class III rock mass, Q-value between 0.1 and 0.04 ($0.1 > Q > 0.04$) is considered as Class IV rock mass, Q-value between 0.04 and 0.01 ($0.04 > Q > 0.01$) is considered as Class V rock mass, and Q-value less than 0.01 ($Q < 0.01$) consider as Class VI rock mass. The highest percentage with 32.21% of overall Rock mass classification consists of Rock class IV, the second highest with 25.89% consists of Rock Class III and 17.83% of Rock Class V was observed, 5.076% of Rock Class VI was observed and 18.99% of Rock Class II was observed.

Along the tunnel we have selected eight section with different type of rock mass and rock support class. Also two section,0+123 and 3+750 with presence of shear band and shear zone also taken for the analysis.The rock mass classification based on Q-system used in RMHEP shown in Table 2.

Table 2: Rock mass classification along section

HRT Chainage m	Rock Type	Q Value	Rock class
0+012	phyllite	0.006	VI
0+036	phyllite	0.0125	V
0+061	Quartzite	0.02	V
0+123	phyllite with shear band	0.006	VI
0+201	Quartzite	0.1	IV
0+225	Quartzite	0.1	IV
0+276	Quartzite	0.04	IV
3+750	phyllite with shear zone	0.004	VI

3. Georisks Prediction

Overbreak and squeezing are the major georisk problems faced during tunnelling in Nepal. The overbreak and squeezing are determined using different formulas. Various empirical, semi-analytical, and numerical methods are used to evaluate the stability of tunnel and estimate the squeezing at critical tunnel sections. Empirical method using Singh et al (1992) and Goel et al (1995) approach were used to check the squeezing problem. Semi-analytical method using Hoek and Marinos (2000) approach was used to estimate the magnitude of deformation. The Semi-analytical method using Shrestha and Panthi (2015) approach was used for the deformation estimation with various support pressures in stress anisotropy condition. The 2D finite element numerical analysis using Phase 2 were used to evaluate displacement with and without shear zone.

3.1 Overbreak

Overbreak is major problem during excavation of tunnel in a blocky brittle and to lesser extent, clayey brittle shear or fault zone.Based on data collected from the tunnel, the tunnel was predicted for overbreak at different chainage using Barton overbreak formula.[5] i.e Overbreak will occur at section if Jn/Jr value is greater than 6 and Q value is less than 0.1 as represented in Table 3.

Table 3: Prediction of overbreak based on Barton overbreak formula.

Description	Ch 0+061m	Ch 0+225m	Ch 3+750m
Q Value	0.02	0.1	0.004
Jn/Jr	15	10	15
Overbreak expected?	Yes	Yes	Yes

3.2 Squeezing

Rock squeezing is a common problem in Nepal Himalaya while tunneling through low strength rock, fault and shear zone containing mainly non swelling clay. Various empirical, semi-analytical, and numerical methods were used for the prediction of squeezing.

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

Description	Ch 0+123m	Ch 0+225m	Ch 3+750m
overburden (H)	132	177	338
Q Value	0.006	0.1	0.004
H'	63.6	162.46	23.82
squeezing prediction	squeezing	squeezing	squeezing

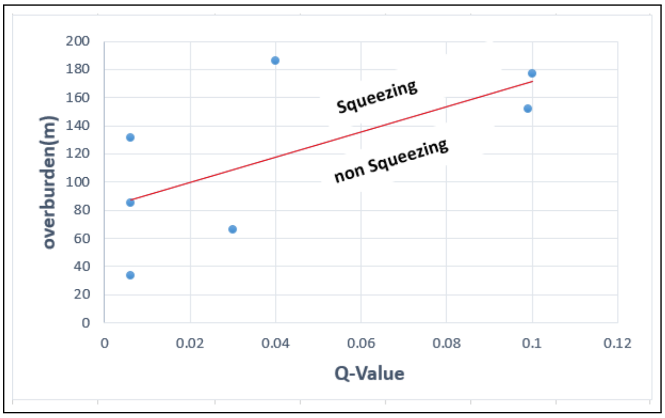

Figure 3: Singh et al (1992) curve for squeezing prediction

Figure 3, shows that squeezing are likely at the three chainage 0+123 m, 0+225 m, and 3+750 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.

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).

Figure 4 shows that the squeezing occurs only at the two Chainages 0+123 m and 3+750 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 5: Empirical method for Squeezing Prediction by Goel et al (1995) method

Description	Ch 0+123m	Ch 0+225m	Ch 3+750m
Overburden (H)	132	177	338
Q value without SRF	0.06	0.5	0.04
H”	94.13	189.5	84.97
Squeezing prediction	Yes	No	Yes

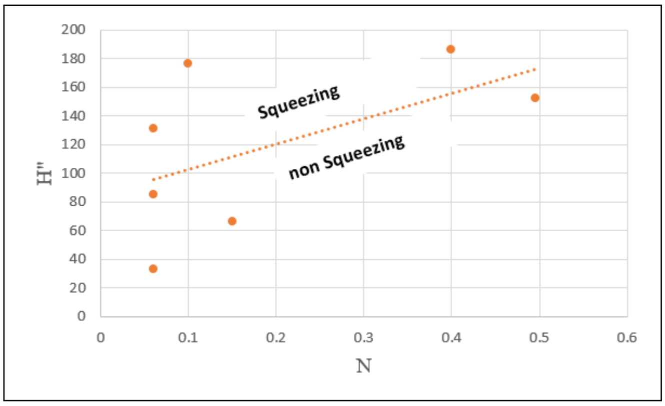

Figure 4: Goel et al (1995) curve for squeezing prediction

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 6 shows that squeezing occurs at the Chainage of 3+750 m and has a high magnitude of deformation 89.70 mm. Similarly, the magnitude of deformation at Chainage of 0+123 m and 0+225 m is 23.57 mm and 9.7 mm respectively in 4.41 m wide tunnel. Figure 5 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.

Table 6: Squeezing Prediction by Hoek and Marinos (2000) method

Parameters	Ch. 0+123 m	Ch. 0+225 m	Ch. 3+750 m
σ’cm/σ v	0.54	0.85	0.32
Strain (ε%) when Pi =0	0.67	0.28	2.56
Squeezing Prediction	No	No	Server
Strain (ε%) when Pi=0.5 MPa	0.35	0.18	1.68
Strain (ε%) when Pi=1 MPa	0.30	0.18	1.12
Deformation (δ i) (mm) for Pi = 0 MPa	23.57	9.7	89.70

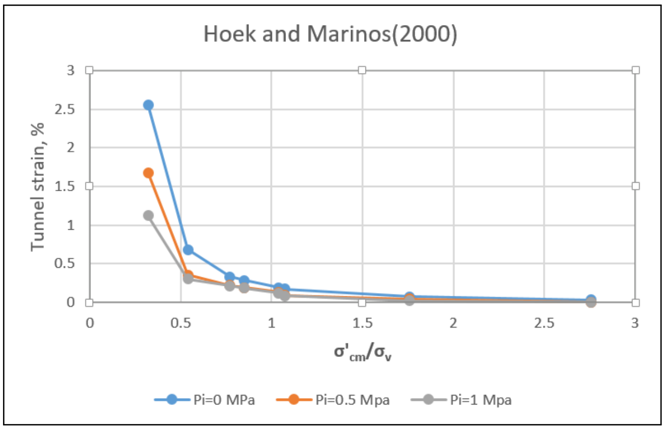

Figure 5: Strain percentage vs Ratio of rock mass strength to vertical stress

Shrestha and Panthi (2015) method

Shrestha and Panthi [6] 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).

Table 7: Estimation of Deformation, Shrestha & Panthi (2015)

Chainage	Ch 0+123m	Ch 0+225m	Ch 3+750m
Initail closure when (Pi=0MPa)(%)	0.28	0.2	0.82
Final Closure when (Pi=0MPa)(%)	0.53	0.39	1.54
Initail closure when (Pi=1MPa) (%)	0.17	0.1	0.44
Final Closure when (Pi=1MPa) (%)	0.24	0.17	0.70
$2G/\sigma v(1+k)/2$	151.0	158.30	103.30

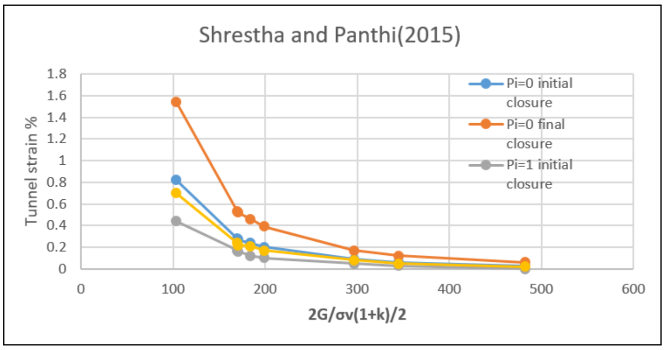

Figure 6: Tunnel strain percentage vs the ratio of shear modulus (G) and in-situ vertical stress (σv)

Figure 6 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 0.82% and 1.54% respectively occur at Chainage 3+750 m among the selected Chainages at unsupported condition due to presence of shear zone.

4. 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 Phase 2 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 8: Input parameters for Phase 2 in valley model

Description	Ch 0+123m	Ch 0+225m	Ch 3+750m
Tectonic Stress(σ tec)	4	4	4
Trend of Tectonic Stress (θt)	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)	0.47	0.47	0.42
Total stress Ratio(horiz/vert out of plane)	1.13	1.04	0.89

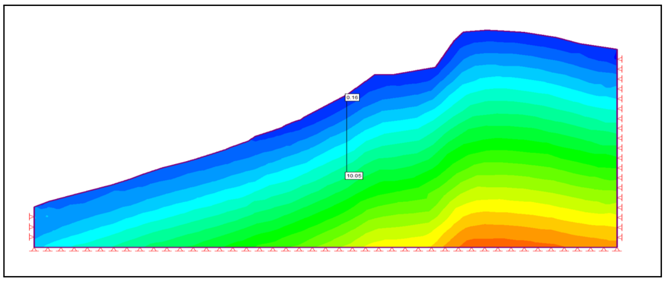

Figure 7: Valley model construction for headrace tunnel alignment at chainage 3+750 m

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Table 9: Output Parameters from Valley Model

Parameters	Ch. 0+123m	Ch. 0+225m	Ch. 3+750m
σ 1(MPa)	4.73	4.78	10.09
σ 3(MPa)	2.4	2.89	4.21
σ z(MPa)	7.46	7.81	12.52
θ°(CCW)	88	88	90

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

Parameters	Ch 0+123m	Ch 0+225m	Ch 3+750m
Overburden (m)	131.36	176.61	338
Density (MN/m3)	0.027	0.027	0.027
Poisson's Ratio	0.2	0.25	0.2
Ei (MPa)	17000	30000	17000
σ ci (MPa)	45	100	45
mi	7	20	7
GSI	25	28	35
σ 1 (MPa)	4.73	4.78	10.09
σ 3 (MPa)	2.4	2.89	4.21
σ Z (MPa)	7.46	7.81	12.52
θ° CCW	88°	88°	90°

Chainage 3+750 m

The rock mass of this chainage is highly fractured weak phyllite with presence of clayey shear zone of about 1.2 m. The overburden at this chainage is 338 m and the model was carried out in this section.

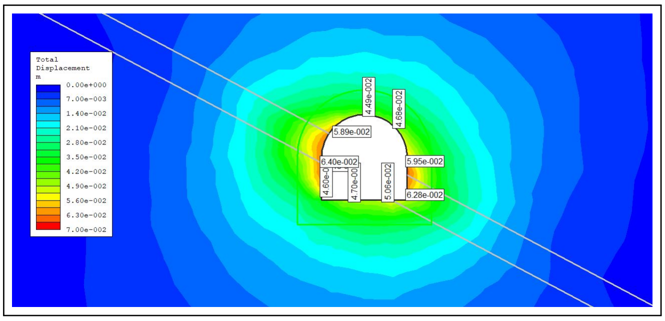

Figure 8: Total displacement before installation of support at chainage 3+750 m with shear zone.

Here the plastic analysis is carried out for the determination of displacement. At this chainage there is presence of shear zone. So for the analysis of displacement is done for both condition i.e with shear zone and without shear zone which gives clear idea about the effect of presence of shear zone in tunnel. From the model with shear zone,total displacement of 62 mm is generated and mainly on the portion of shear zone as shown in Figure 8. similarly total displacement of about 46.96 mm is obtained from model without considering shear zone as shown in Figure 9.

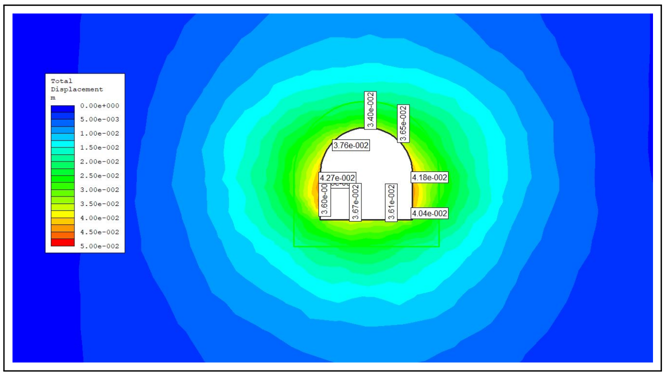

Figure 9: Total displacement before installation of support at chainage 3+750 m without shear zone.

Chainage 0+123 m

The rock mass of this chainage is highly fractured weak phyllite with quartzitic alternation with presence of shear band of about 0.25 m. The overburden at this chainage is 131.36 m and the model was carried out in this section.

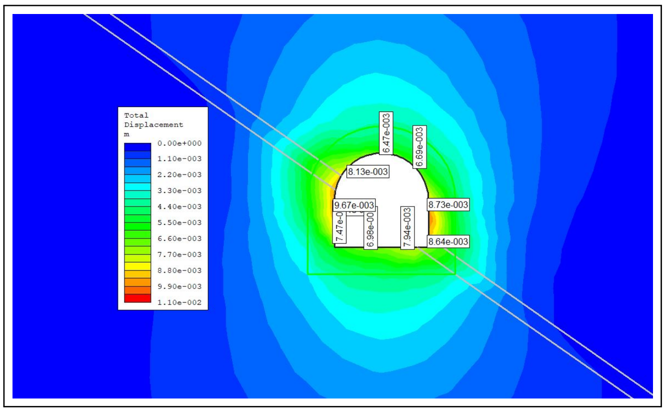

Figure 10: Total displacement before installation of support at chainage 0+123 m with shear band.

Here also the analysis is done both with presence of shear band.From the model with shear band,total displacement of 10.91 mm is generated mainly on the portion of shear band as shown in Figure 10. similarly total displacement of about 6.7 mm is obtained from model without considering shear band as shown in Figure 11.

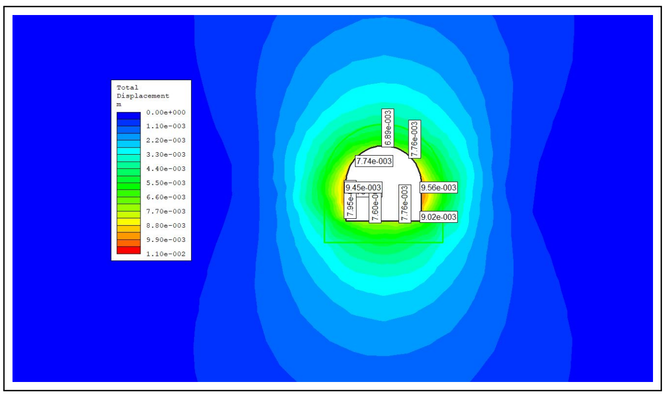

Figure 11: Total displacement before installation of support at chainage 0+123 m without shear band.

Support Analysis

The support installed as adopted by the project, the support capacity seems inadequate, there is no yielding of rock bolts however 4 yielded liner element was found so the model was run many times increasing the thickness of concrete along with the use of reinforcement ISMB 75X150 mm 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.

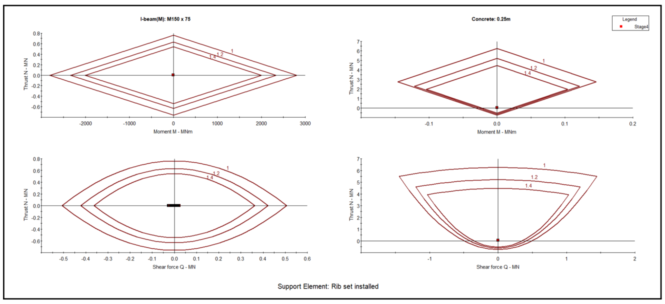

Figure 12: Support capacity curve after revised support at chainage 3+750m

For shotcrete or concrete element in the support capacity curve, all the points come inside all three envelopes as FOS given were 1, 1.2, and 1.4. 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 Phase 2 software at the chainage of 0+123 m and 0+225, the support capacity seems adequate.

5. Conclusion

In Rahughat Mangale Hydroelectric Project (RMHEP), brittle faults and shear zone are found. Those zones are more geotechnically problematic. Following are the major conclusion.

1. Predection of overbreak using Barton overbreak formula only gives an idea of overbreak, it doesn't totally relay with the project. The prediction of squeezing using emprical, semi emprical method gives an idea of squeezing and deformation in the tunnel.

2. Analyzing the tunnel same section with and wihout shear zone at chainage 3+750 and 0+123, numerical result shows that displacement is higher in tunnel section with presence of shear zone.

3. From numerical analysis of support installed among the selected chainage, it seems adequate support adopted at the chainage 0+123 and 0+225 as well as additional thickness of concrete were provided at the chainage 3+750.

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