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Rock Squeezing Analysis in Young Mountains: A Case Study of Headrace Tunnel of Supermadi Hydroelectric Project in Nepal

Dev Raj Joshi[1] and Suman Panthee[2]

[1] Department of Civil Engineering, Pulchowk Campus, Institute of Engineering, Nepal

[2] Central Department of Geology, Tribhuwan University, Nepal

E-mail: *drj.devrajjoshi@gmail.com*

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Abstract

The tunnel stability was assessed by analysing stress in the opening. Squeezing is the major problem faced while excavation a tunnel alignment in Himalayan region. This research was done in Super Madi hydroelectric project of 44 MW, in which main focus on the rock mass section of headrace tunnel of Inverted D shaped. The prediction of squeezing is done using various empirical, semi empirical, semi analytical and numerical modeling for the three different rock class. Finite element analysis was done using Rock science Phase2 software and output was verified by site investigation. The main lithologies of the area along the tunnel axis are banded gneiss. Based on the site condition, this study recommend the accurate method of predicting rock squeezing in the lesser Himalayan region with similar site conditions. It is found that the higher deformation occurs in high tunnel depth with low Q value.

Keywords: Rock squeezing, slope stability, Rock Support, headrace tunnel

1. Introduction:

The failure mechanism in tunneling is generally of two mechanism [1] . These are structurally controlled and stress induced. Both the mechanisms are dependent mainly on the rock mass quality and in-situ stress condition. During excavation in jointed rock masses at relatively shallow depth, the most common type of failure are the deformation and wedge falling from the roof or sliding out of the side walls of the opening. Unless steps are taken to support, the stability of the roof and side walls of the opening may deteriorate rapidly [2]. The stress level acting around the underground openings is another factor that may cause tunnel stability problems. It is evident that a tunnel fails when the stress exceeds the strength of rock mass around the opening [3].

For the stability analysis of underground structures is importance to success the project. This study aims to determine the underground squeezing and their stability willanalyzed and calculation of stress analysis to find out the rock squeezing problem in headrace tunnel at greater depth and compare the result of various method to recommend accurate method on similar geological condition. The deformation in the underground structure is directly affect by the tectonic activities in that area. Being highly schistose and weaker in their mechanical characteristics, the rocks such as slate, phyllite, phyllitic schist, schists, mica gneiss and rock mass of the tectonic fault zones of the Himalaya lack sufficient bonding (confinement), and hence have considerably reduced self-supporting capability

[4]. Therefore, the main issue of this study regarding prediction of tunnel squeezing using various empirical, semi-analytical and numerical modelling. Methods used in this study is also used to predict tunnel squeezing along the headrace tunnel segment of the Middle Marsyangdi, which was under construction at that time [5].

2. Geology of the Project Area:

Super Madi Hydroelectric Project is located in Kaski Districts, Gandaki Province 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 243.125 GWh. This project is simple run-off hydropower project [6]. The geological map of country with location of selected case study is shown in Figure 1. This project is lies on the lower part of Higher Himalayan Region. This study was carried out for 400 m length of headrace tunnel from 1+000 m chainage to 1+400 m chainage which include three geological variation on rock type.

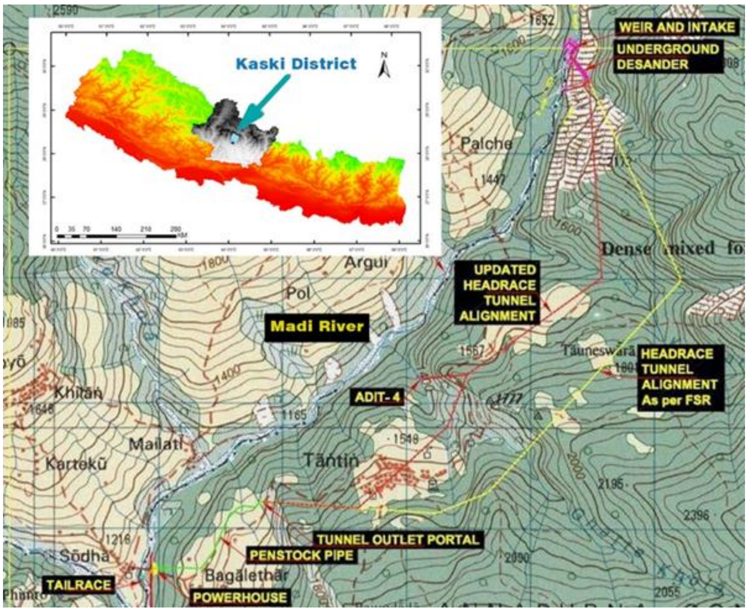

Figure 1: General Layout of Project Area

Geologically, project area lies in the Higher Himalayan succession. Higher Himalayan 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 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 200 m to 300 m. 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 219.05 m along the tunnel alignment in this section. 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 15 m to 225 m. 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.

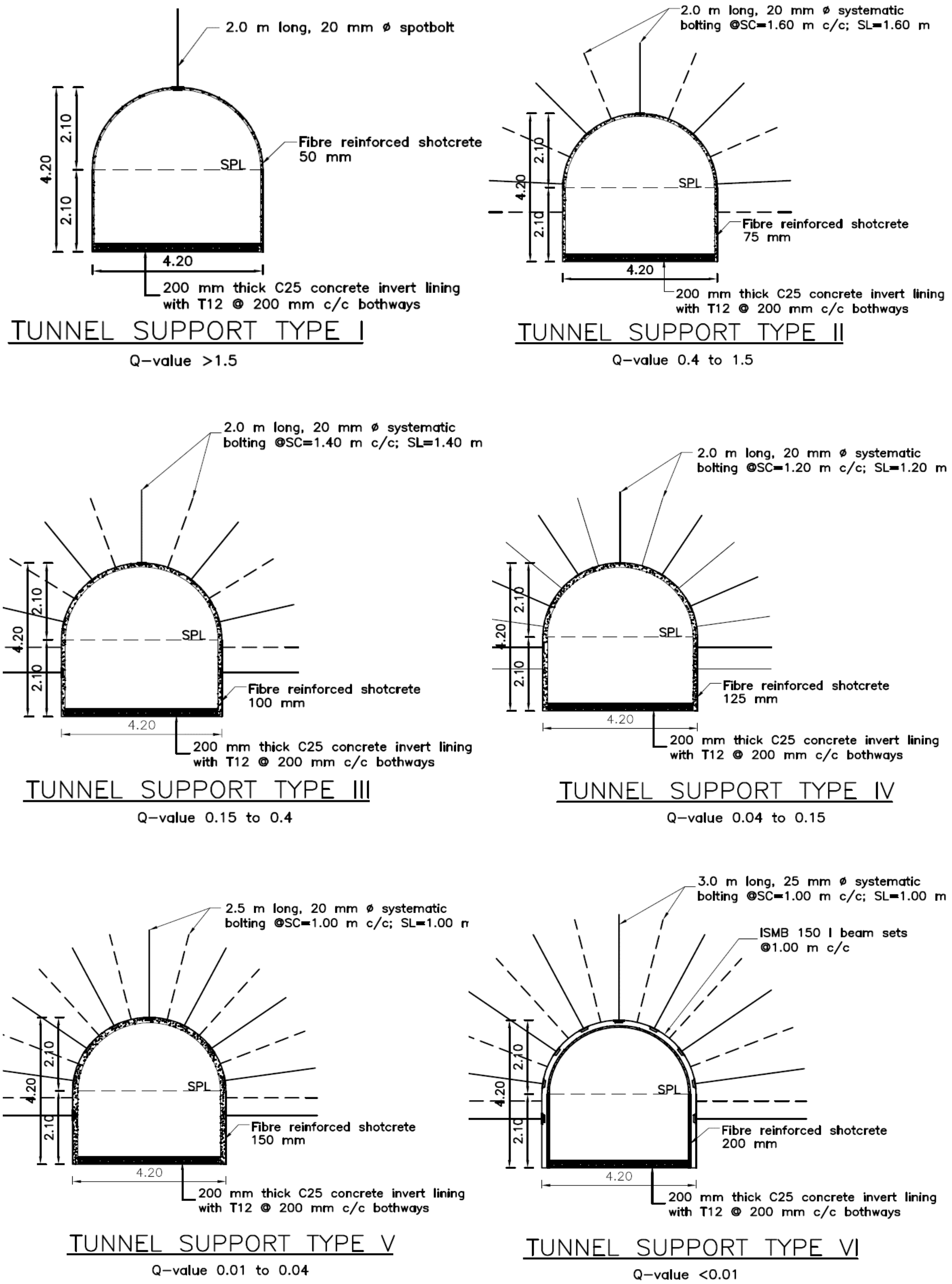

Figure 2: Different Tunnel Support Types

3. Literature Review:

Steiner (1996) [7] studied about the case histories in tunneling in squeezing rocks and concluded that squeezing ground conditions are influenced by the factors such as rock type, strength of fragmentation if rock mass, orientation of the rock structure, stress state (overburden), water pressure, construction procedures and support system, not all of which contribute to the sapme degree.

N. Vlachopoulos (2009) [8] recoomend the method for developing displacement profile of Deep tunnels and tunnel strain [9], effect of stress [10], plastic deformation [11] behaviour of the various rock mass present in Nepal himalayas was studies and recommend accurate method, input parameter by P. Shrestha and K.K. Panthi.

Basnet et. al. (2013) [12] assesses the squeezing phenomenon along headrace of Chameliya Project in which tunnel stretch through evaluation of rock mass properties and support pressure. He approaches three different methods (two analytical and one 2D finite element numerical modeling program) for the analysis. His finding is that it is possible to predict extent of squeezing in tunnel if more than one method is used to verify rock mass mechanical properties.

3.1 Instability Along the Headrace Tunnel:

Along the tunnel alignment there were minor stability problems. Tunnel alignment passes through different rock classes, different weakness zones. The major stability problem was found in soft soil section of this tunnel.

The actual geology in this area was found to be quite different from the initial level study. Due to lake of clear geological investigation along the alignment, the risk of deformation exists. Rock mass along the selected 400 m chainage is massively foliated and slightly weathered banded gneiss. This study considered the three major joint sets along the chainage. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3 mm) apertures 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 200 m ahead. Himal Hydro Study Report [6]. The overburden is less compared to previous section with minimum 167.63 m at the last chainage. This section carries less deformed rock mass with less frequency of joint set.

3.2 Tunnel Excavation and Support System:

The method of excavation used in this headrace tunnel is conventional drill and blast method. In the most fractured rock site other alternative method applied with controlled blasting and manual excavations. Different types of rock support were applied simultaneously with the excavation Figure 3. Various support types were categorized based on the Q-value along the tunnel alignments and is shown in Figure 2. The support of 20 mm diameter spotbolt and 50 mm fibre reinforce shortcrete which is the type I support applied for the rock mass of Q-value greater than 1.5. Similarly type II and type III support was applied for Q-value from 0.4 to 1.5 and 0.15 to 0.4 respectively. Most of the section from study area was applied type III and Type IV support types. Then weakest zone among the study area was at 1+200 m chainage with the Q value of 0.038 and design with the support type V i.e. 2.5 m long bolt and 150 mm thickness fibre shortcrete.

Figure 3: Excavation and Tunnel Support

3.3 Squeezing Analysis:

For the squeezing prediction at each section Singh et al. (1992) [13], Goel (1994) [14], Jethwa et. al (1984) [15], and Hoek and Marinos (2000) [16] methods was use for the geological conditions. Squeezing was further analyzed using Numerical modelling from Phase2 in the basis of finite element method. Rockscience software Phase2 was used for the finite element analysis. The result was verified at the site and accurate method was recommended for the similar geological condition.

Singh et al. (1992) [13] approach has given a demarcation line to differentiate squeezing condition from non-squeezing condition. This approach was developed by collecting data on rock mass quality Q (Barton et al., 1974 [17]) and overburden depth H.

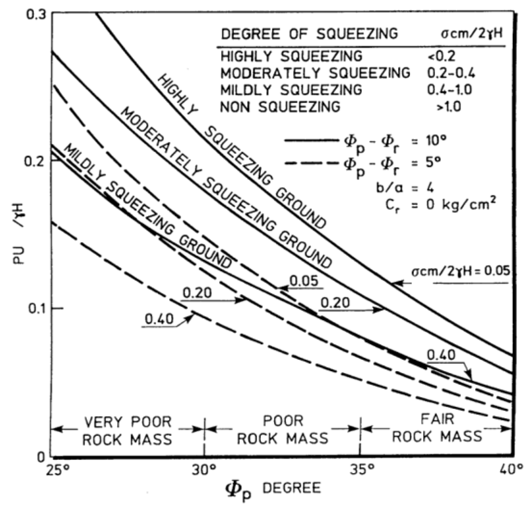

Figure 4: Degree of Squeezing Analysis (Jethwa et al.)

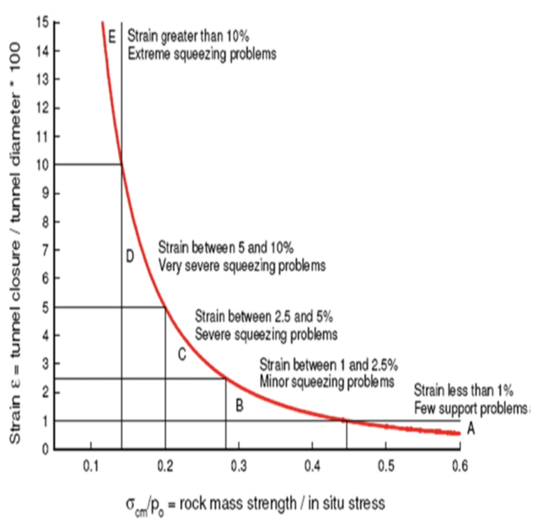

Figure 5: Squeezing Analysis (HM method)

Goel (1994) [14] developed an empirical approach based on the rock mass number *N*, defined as Q with SRF = 1. *N* was used to avoid the problems and uncertainties in obtaining the correct rating of parameter SRF in Q method. Considering the overburden depth *H*, the tunnel span or diameter *B*, and the rock mass number *N* from our tunnel sections, we have plotted the available data on log-log diagram between *N* and HB0.1.

All the section is studied for the squeezing problems and found five section fall for squeezing and rest lies for non-squeezing condition.

Jethwa et. al. (1984) [15] approach calculate the degree of squeezing as in the Fig. 4 which is described using coefficient Nc which is equal to the ratio of rock mass uniaxial compressive strength (UCS) to insitu stress. Based on the degree of squeezing, type of behavior of tunnel can be estimated from Figure 5.

A semi-analytical approach given Hoek and Marinos (2000) [16] have been used for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel.

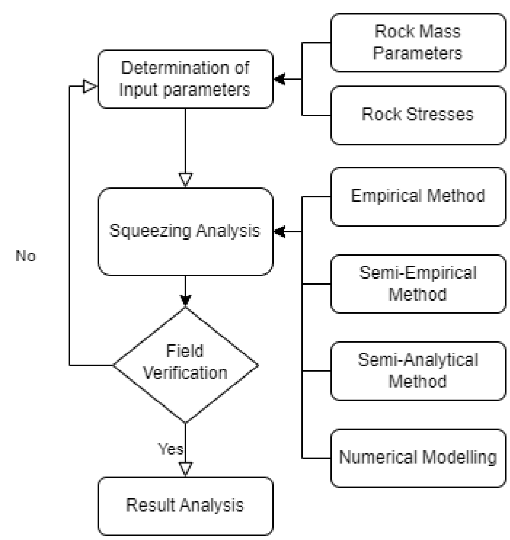

Figure 6: Methodology Flowchart

Hoek and Marinos showed that a plot of tunnel strain (ξ) against the ratio cm/po could be used effectively to assess tunneling problems under squeezing condition. Hoek and Brown's criteria for estimating the strength and deformation characteristics of rock masses assume that rock mass behaves isotopically.

However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will have been disrupted and the rock may behave as an isotropic mass. Thus, this criterion can be adapted to weak heterogeneous rock masses too.

The research was started from the secondary date of geological investigation was collected from the project office. Firstly, the input parameters were calculated from the various empirical methods (Table 1) and was compared to the project data. Secondly squeezing was predicted using the empirical, semi-empirical, semi-analytical and finite element modelling and was verified with the site condition. The summary of the methodology was shown in Figure 6.

Table 1: Input Parameters

Chainage	Rock Type	Tunnel Depth (m)	Q value	mi	GSI
1+000	IV	306.42	0.344	28	30
1+050		292.61	0.583	28	30
1+100		285.26	0.229	28	30
1+150	V	275.37	0.075	20	14
1+200		274.05	0.038	20	14
1+250		253.04	0.070	20	14
1+300	III	219.05	1.250	28	53
1+350		186.85	1.083	28	53
1+400	IV	167.63	0.271	28	30

4. Results and Discussions:

Singh et al. (1992) [13] has given a demarcation line to differentiate squeezing condition from non-squeezing condition. This approach was developed by collecting data on rock mass quality Q (Barton et al., 1974 [17]) and overburden depth H based on 41 tunnel section data. Out of 41 data, 17 data were taken from case histories in Barton et al. (1974) and 24 tunnel section data were obtained from tunnels in Himalayan region. The ground condition prediction based on rock mass quality Q is shown in Figure **7**. From the graph we can conclude that most of the area (about 67%) falls under the squeezing zone and the remaining (33%) falls under the non- squeezing zone. The section from 1+000 m to 1+250 m where high overburden pressure lies falls under squeezing condition and rest of the section with lesser overburden pressure is safe for squeezing problem.

The calculation of the equation of line for prediction of squeezing is tabulated in Table 2, this can clearly determine the squeezing problem occurs up to 1+250 m chainage and rest of the section is stable against deformation. Goel (1994) [14] developed an empirical approach based on the rock mass number *N*, defined as Q with SRF = 1. *N* was used to avoid the problems and uncertainties in obtaining the correct rating of parameter SRF in Q method.

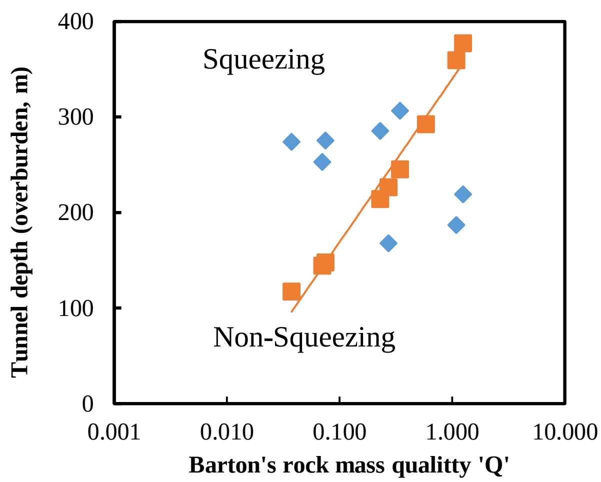

Figure 7: Squeezing Prediction using Singh et al. (1992) Approach

Considering the overburden depth *H*, the tunnel span or diameter *B*, and the rock mass number *N* from our tunnel sections, we have calculated (Table 3) and plotted the available data on log-log diagram between *N* and $HB^{0.1}$. All the section is studied for the squeezing problems and found five section fall for squeezing and rest lies for non-squeezing condition.

Table 2: Calculation for Singh et al. (1992) Approach

Chainage	Overburden (m)	Q value	Equation of Line (H)	Remarks
1+000	306.42	0.344	245.18	Squeezing
1+050	292.61	0.583	292.44	Squeezing
1+100	285.26	0.229	214.18	Squeezing
1+150	275.37	0.075	147.60	Squeezing
1+200	274.05	0.038	117.15	Squeezing
1+250	253.04	0.070	144.46	Squeezing
1+300	219.05	1.250	377.03	Non-Squeezing
1+350	186.85	1.083	359.46	Non-Squeezing
1+400	167.63	0.271	226.45	Non-Squeezing

Table 3: Calculation for Goel (1994) Approach

Chainage	Overburden (m)	Equation of Line 'H'	$HB^{0.1}$	Remarks
1+000	306.42	284.86	328.82	Squeezing
1+050	292.61	339.17	391.51	Non-Squeezing
1+100	285.26	249.18	287.64	Squeezing
1+150	275.37	216.66	250.09	Squeezing
1+200	274.05	172.36	198.96	Squeezing
1+250	253.04	212.09	244.82	Squeezing
1+300	219.05	346.98	400.53	Non-Squeezing
1+350	186.85	330.98	382.05	Non-Squeezing
1+400	167.63	263.31	303.94	Non-Squeezing

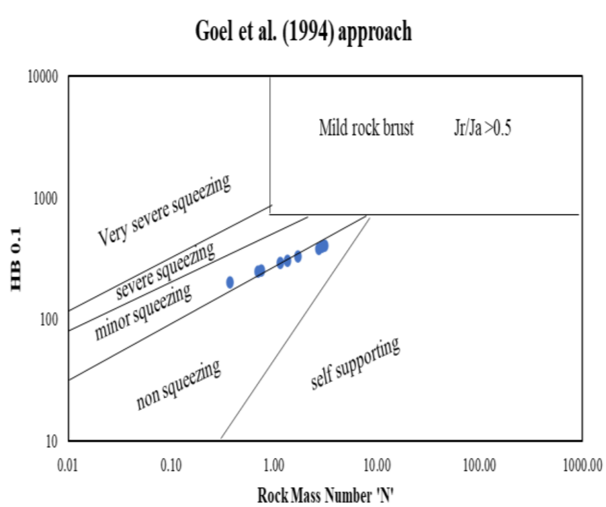

Figure 8: Squeezing Prediction using Goel (1994) Approach

The Figure 8 represent the ground condition at the study section for different value of '*N*', the section 1+050 m which is rock type IV is fall under the non-squeezing due to the less joint set rather than other section of same rock type. Squeezing problem is not severe along the study zone but this method identifies additional rock support to prevent squeezing problem. The degree of squeezing in Jethwa et al. (1984) **[15]** approach is described using coefficient *Nc* which is equal to the ratio of rock mass uniaxial compressive strength (UCS) to in-situ stress. Based on this value, type of behavior of tunnel can be estimated. Jethwa et al. (1984) define the degree of squeezing as shown in Table 4. Based on the coefficient the initial section up to 1+100m was found to be greater than 0.2 and less than 0.4 that identify the moderate squeezing but for the rock type V this method identifies highly squeezing with *Nc* less than 0.2. These sections need special rock support for the tunnel stability and the section with rock type III is from 1+300 m to 1+350 m has no squeezing problem. A semi-analytical approach given Hoek and Marinos (2000) have been used for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel.

The Hoek and Marinos (2000) [16] approach for predicting tunnel squeezing given by Eqs. 1 and 2. Similarly Eq. 3 suggested by Panthi (2006) [18] for estimating rock mass strength are used as a basis for the analysis below.

$$\varepsilon_t = (0.2 - 0.25 * \frac{pi}{\sigma v}) * (\frac{\sigma cm}{\sigma v})^{(2.4*\frac{pi}{\sigma v}-2)} \quad (1)$$

$$\varepsilon_t = 0.2 * (\frac{\sigma cm}{\sigma v})^{-2} \quad (2)$$

$$\sigma_{cm} = \frac{\sigma ci^{1.5}}{60} \quad (3)$$

Where; ε_t is tunnel strain in percentage, σv is overburden stress in MPa, σcm is rock mass strength in MPa and pi is rock support pressure in MPa.

The

Table **5** describe the strain in percentage at every section that lies in the < 1 %. That indicate the minor support problem not the squeezing problem. The support from the Q-chart will be safe for the installation. It generally happens in weak rock such as shale, phylite, and slates or at weakness zone. In case of SMHEP, rock mass seems (from surface) strong and brittle, there is less chance of squeezing

In case semi analytical method of squeezing analysis, Hoek and Marinos (2000) [16] approach was used with correlation with ground condition. This method shown quite acceptable result and is more recommended by different scientists in rock engineering. In this method, tunnel strain ($\mathcal{E}$) was calculated by using different parameters such as Tunnel overburden depth, vertical stress, intact strength, material constant, GSI, rock mass strength and support pressure. Support pressure *p* was calculated by using the supports used in the tunnel such as steel ribs, shotcrete and rock bolts.

Modeling of rock mass is a very difficult job due to the presence of discontinuities, anisotropic, heterogeneous, and nonelastic nature of rock mass, using empirical and numerical methods. Complex nature and different formation make the rock masses a difficult material for empirical and numerical modeling. During initial stages of excavation projects, the detailed data are not available about strength properties, deformation modulus, in situ stresses, and hydrological of rock masses. To handle the no availability of the detailed project data, the empirical methods like rock mass classification systems are considered to be used for solving engineering problems. empirical methods used to define input parameters in designing of any underground structures, recommendation of support systems, and determination of input parameters for numerical modeling.

The total displacement of the tunnel is 2.71 mm. This is about 0.06% of the tunnel span. The extend of the plastic zone (*R*p) is about 3.684 m. The ratio of distance from tunnel face to tunnel radius (*X*/*R*t) is 1.19. And plastic zone to tunnel radius (*R*p /*R*t) is 1.754. By using Vlachopoulos and Diederichs [8] method, the above values are plotted gives ratio of closure to maximum closure equal to 0.74. Therefore, the closure equals 2.005 mm. This is about 74% of the total closure 2.71 mm. 74% of total deformation will already take place before support is installed. Internal pressure factor of 0.04 yields the tunnel wall displacement computed above for the point of support installation.

Table 4: Calculation for Jethwa et. al (1984) Approach

Chainage	Q	Nc	Remarks
1+000	0.344	0.28	Moderately Squeezing
1+050	0.583	0.34	Moderately Squeezing
1+100	0.229	0.27	Moderately Squeezing
1+150	0.075	0.06	Highly Squeezing
1+200	0.038	0.05	Highly Squeezing
1+250	0.070	0.06	Highly Squeezing
1+300	1.250	1.27	Non-Squeezing
1+350	1.083	1.43	Non-Squeezing
1+400	0.271	0.48	Mildly Squeezing

Table 5: Calculation for Hoek and Marinos (2000)

Chainage	Tunnel Depth (m)	ξ (%)	Category	Remarks
1+000	306.42	0.139%	< 1%	Few Support Problems
1+050	292.61	0.134%	< 1%	Few Support Problems
1+100	285.26	0.127%	< 1%	Few Support Problems
1+150	275.37	0.210%	< 1%	Few Support Problems
1+200	274.05	0.210%	< 1%	Few Support Problems
1+250	253.04	0.177%	< 1%	Few Support Problems
1+300	219.05	0.050%	< 1%	Few Support Problems
1+350	186.85	0.043%	< 1%	Few Support Problems
1+400	167.63	0.081%	< 1%	Few Support Problems

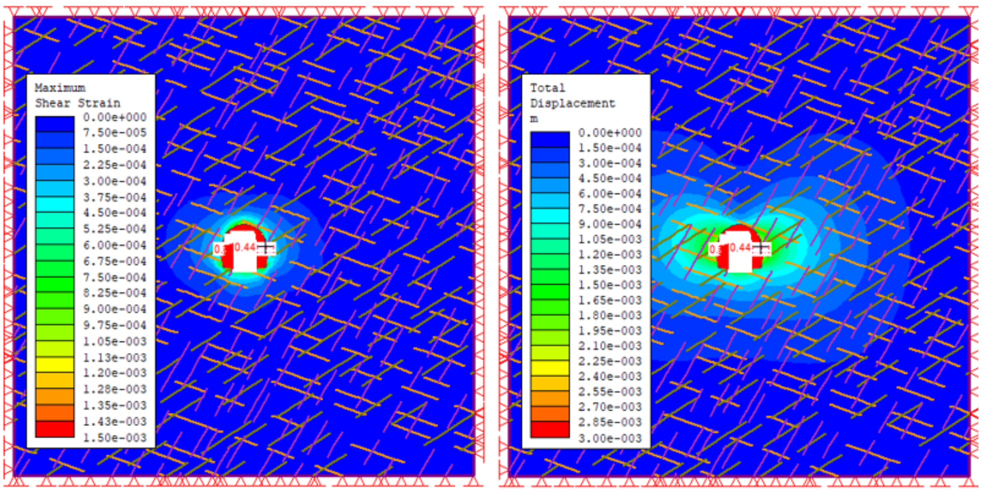

Figure 9: Maximum Strain and Total Displacement at 1+100m (Rock Type IV)

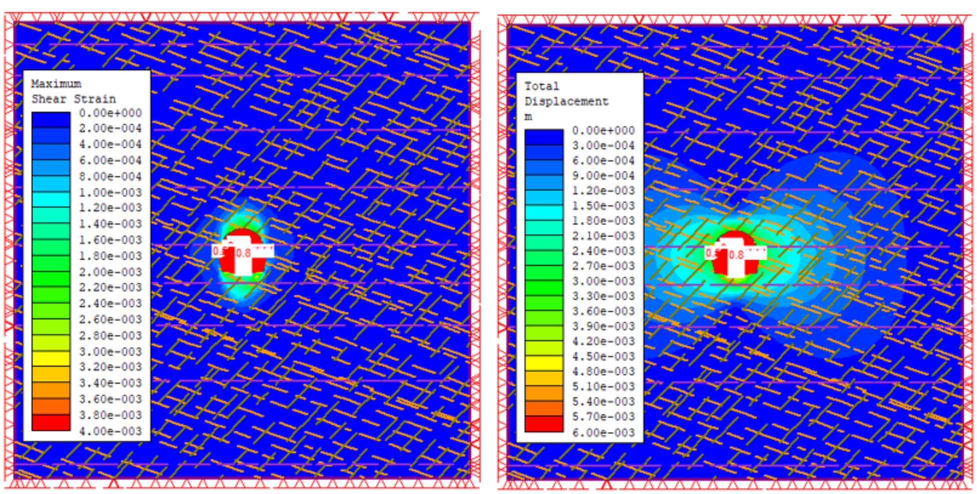

Figure 10: Maximum Strain and Total Displacement at 1+200m (Rock Type V)

This is found to be minor support problem not the squeezing problem. In Plastic analysis, uniform distributed load is added to the tunnel in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face. The Maximum strain and total displacement at representing three sections of three different rock types are shown in Figure 9, Figure 10 and Figure 11.

In the section 1+200 m the total displacement of the tunnel is 12.71 mm. This is about 0.303% of the tunnel span. The extend of the plastic zone (*R*p) is about 9.125 m. The ration of distance from tunnel face to tunnel radius (*X*/*R*t) is 1.19. And plastic zone to tunnel radius (*R*p/*R*t) is 4.345. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.44. and for the section 1+350 m the total displacement of the tunnel is 1.88 mm. This is about 0.045% of the tunnel span. The extend of the plastic zone (*R*p) is about 3.577 m. The ration of distance from tunnel face to tunnel radius (*X*/*R*t) is 1.19 and plastic zone to tunnel radius (*R*p /*R*t) is 1.703.

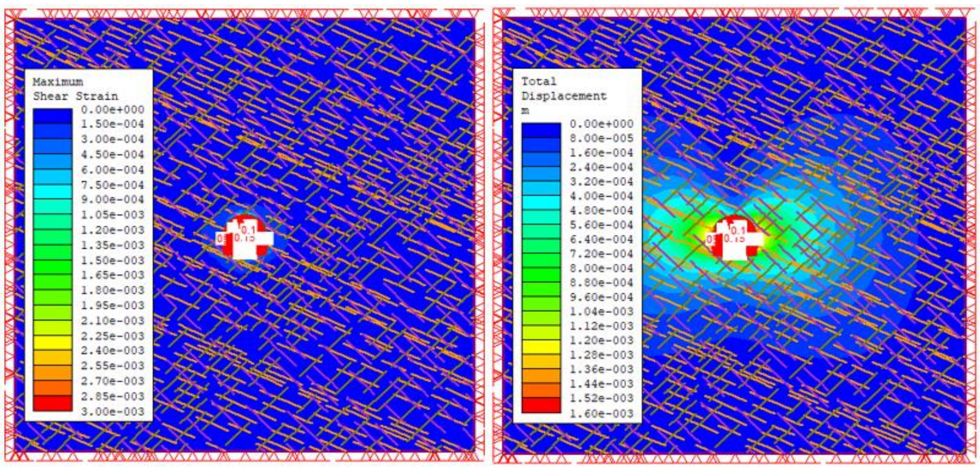

Figure 11: Maximum Strain and Total Displacement at 1+350m (Rock Type III)

By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.82. all the section has minor deformation which is due to minor support problem and the squeezing problem. This method gives the result similar to the HM methods with no squeezing problem along the tunnel section.

The accuracy in predicting tunnel squeezing depends on the reliability of the estimated input variables and equations that are used for such analysis. The selection of the representative probability distribution functions (pdf), the input variables related to equations used and reliability of the equations in use for such predictions are key factors. Five main methods have been used to analyze squeezing: empirical Singh et al. (1992) [13] Approach, and Goel et. al. (1994) [14] Approach, semi empirical Jethwa et. al (1984) [15] approach, semi-analytical Hoek and Marinos (2000) [16] approach, and 2D finite element numerical modeling program Phase2. Rock mass parameter and rock stresses are the input parameter for each method. The achieved analysis results indicated that accuracy of analysis largely depends upon the correct estimation of input parameters. The squeezing analysis have been done for all the ten tunnel sections along the squeezed part of the headrace tunnel. However, only three selected tunnel sections were presented in this paper using Phase2 numerical modeling. These three sections were selected such a way that it represents all the three types of rock mass i.e., rock class III, IV and V.

From the field verification, (Figure 12) it is found that there is no any squeezing problem along the section. The same result was found by Hoek and Marinos (2000) [16] and Numerical Modelling. The tunnel deformation is calculated using improved rock mass parameters as input to different approaches such as Hoek and Marinos, and checked by using Phase 2 taken in Figure 13, Figure 15 and

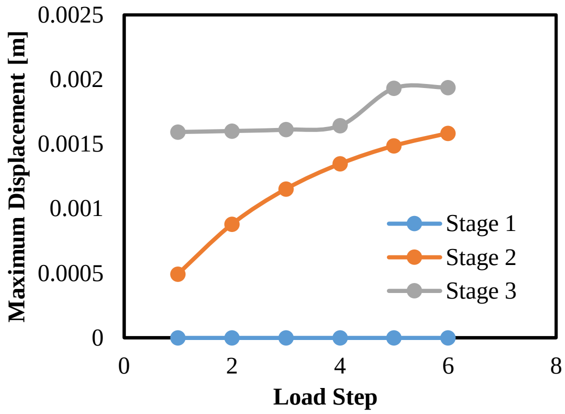

Figure 16.

Figure 12: Site Picture of the Tunnel Alignment

The analysis also indicates that if carefully used it is possible to exploit all these three methodologies to evaluate the squeezing phenomenon.

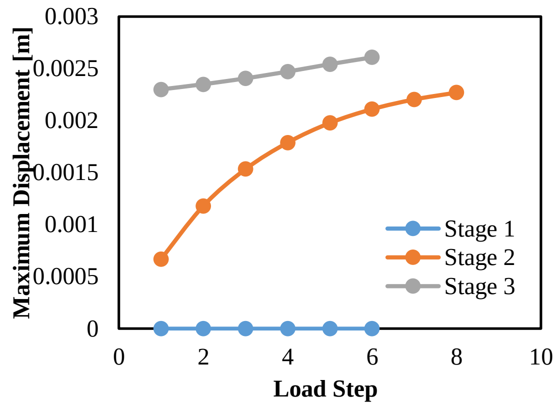

Figure 13: Maximum Displacement vs. Load Curve at 1+000m (Rock Type IV)

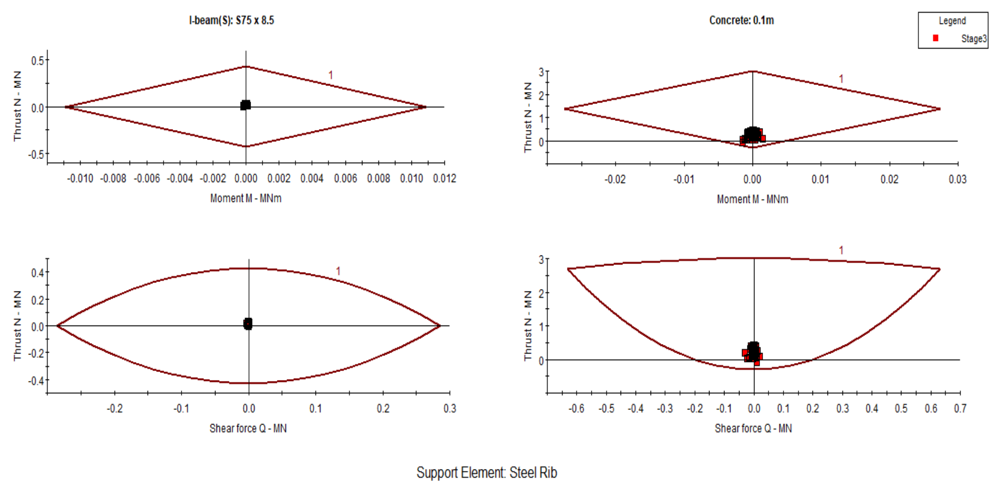

Figure 14: Support Capacity Curve at 1+000m (Rock Type IV)

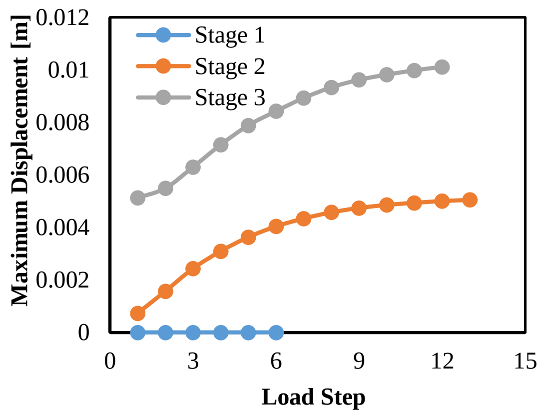

Figure 15: Maximum Displacement vs. Load Curve at 1+200m (Rock Type V)

Maximum displacement vs load curve shows the displacement for three different stage i.e. Initial stage, relaxation stage and support installation stage which is falls in the safe zone which is shown in support capacity curve. The estimated support capacity curve which is safe for the installation are shown in Figure 14, Figure 17 and Figure 18.

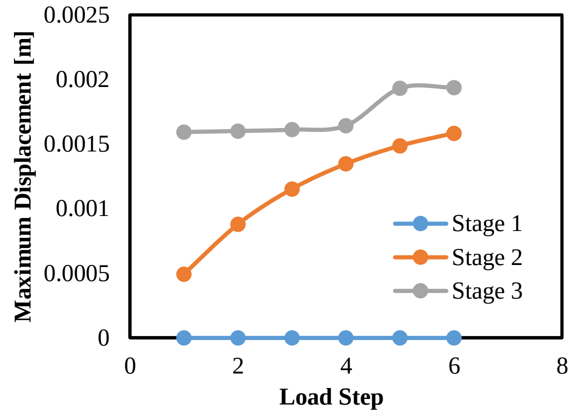

Figure 16: Maximum Displacement vs. Load Curve at 1+350m (Rock Type III)

The portion of the study is safe to install steel ribs. These supports are estimated using empirical methods and analyzed using numerical modeling and checked for the stability against deformation. This research found no any squeezing problem along the 400 m section of headrace tunnel of Super madi HEP that recommend no any special treatment for the addition of support along the wall and crown.

This study concludes that the semi-analytical Hoek and Marinos (2000) [16] method give accurate results for the prediction of squeezing. It should be emphasized that the input variables be carefully selected and that more than one methodology be used in predicting the severity of squeezing.

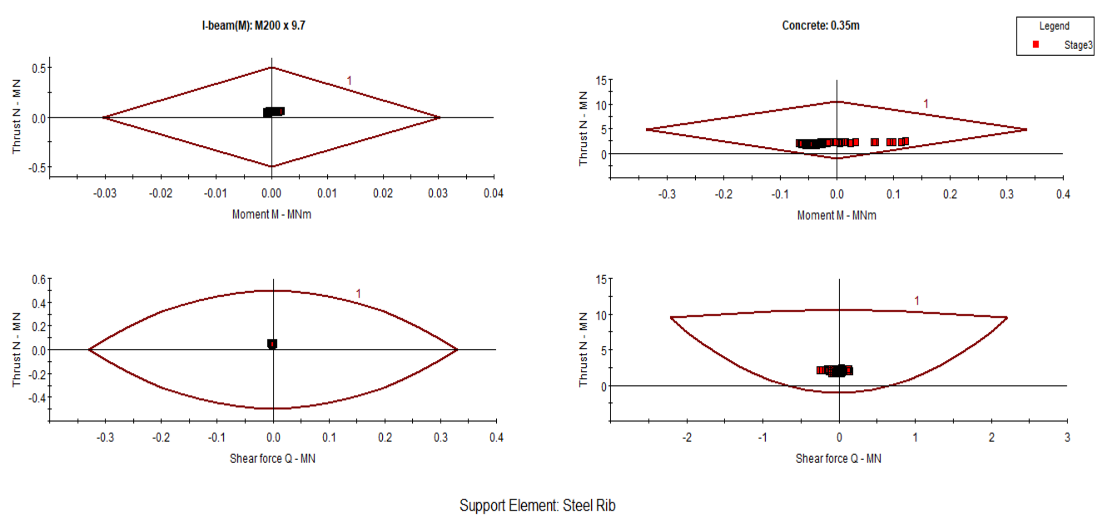

Figure 17: Support Capacity Curve at 1+200m (Rock Type V)

For the similar geological condition HM method and Numerical modelling is recommended for the prediction of squeezing problems in underground construction. 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. The headrace tunnel section of supermadi HEP along rock section does not have squeezing problem and it would be safe with the designed support using Q-chart.

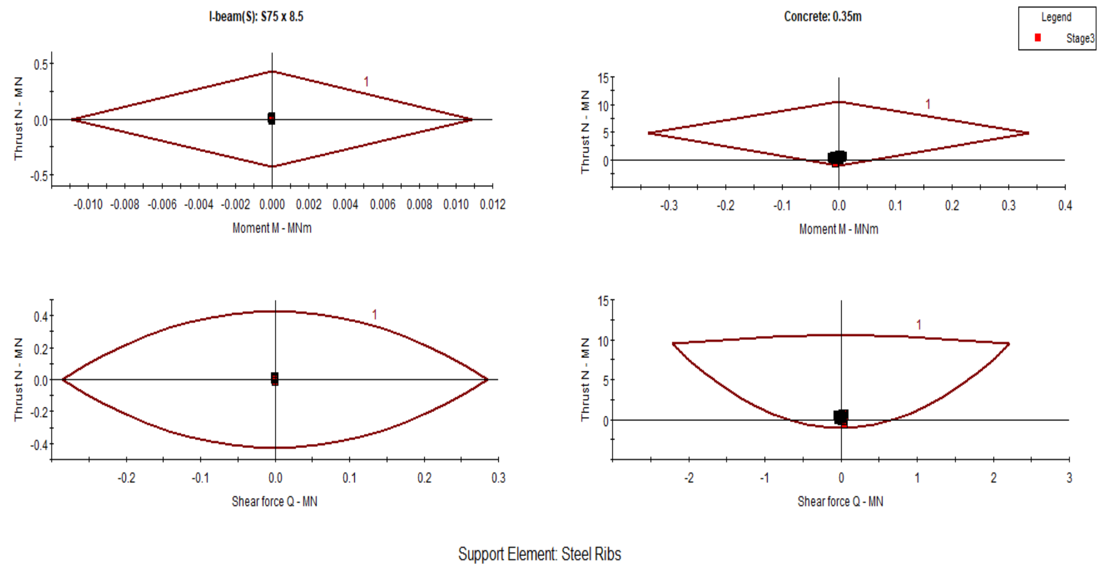

Figure 18: Support Capacity Curve at 1+350m (Rock Type III)

5. Conclusion:

Four important methods have been used to analyze squeezing in the Headrace Tunnel of Super Madi Hydroelectric Project. The inputs to squeezing analysis in each method were rock mass parameters and rock stresses. The achieved analysis results indicated that accuracy of analysis largely depends upon the correct estimation of input parameters. The squeezing analysis have been done for ten tunnel sections along the headrace part of the tunnel using empirical, semi-empirical, Semi-Analytical and Finite element approaches. However, only three selected tunnel sections were analyzed using Phase2 numerical modeling that represent all the three types of rock.

HM method is to be found accurate squeezing prediction method for such geological condition which is verified by the finite element method i.e., Phase2 analysis and field verification. Based on the analysis and comparison HM and Numerical modelling is recommended for the squeezing analysis for underground structures.

Therefore, Empirical approach only is not adequate to Analyze and estimation of tunnel Squeezing. Numerical analysis and HM Method was very helpful to estimate the tunnel squeezing in such geological region where rock masses are very poor with high rock cover.

The effect of water has not been considered in the analysis. The result can be improved by considering the water effect in the analysis. The seismic effect in the tunnel is also not considered in this research. The result can be improved by applying the seismic effect.

Limitations:

Field measurement for the deformation of tunnel face was not taken in the project. The model was compared to the calculated deformation from empirical relations for the validation. The hydrostatic pressure and earthquake pressure was not considered in the research that will be considering for future extension.

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