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Possible Squeezing Effect and Design of Support System at the Headrace Tunnel of Dudhkoshi Storage Hydroelectric Project

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Abstract

Rock mass strength, techniques used in excavation and stress and deformation characteristics of rock mass is prime challenges which should be taken in account to design underground excavation and construction. Thus, proper calculation of possible worse problematic and disastrous condition during excavation and give safer and cost-effective engineering solutions bears prime necessity before excavation of tunnel. Due to the weak composition and younger rock formation of Nepal, squeezing phenomenon is one of the common problems in excavation in Himalayan region of Nepal. Squeezing is such phenomenon where weak rock mass moves radially inward leading to decrease in size of excavated portion. So, different methods of squeezing analysis have been used for the estimation of probable squeezing phenomenon. For the simulation of deformation and support behavior, numerical approach has been carried out for the case study of Dudhkoshi Storage Hydroelectric Project (635 MW). RMR and Q-Value were found to be poor to fair for Phyllite, Schist, Quartzite, Limestone with overburden varying from 120 m to 1100 m for headrace tunnel of the same project. Accordingly, support system has been designed for the potential squeezing section.

Keywords

Tunnel, Squeezing, Excavation, Deformation, Rock mass, Convergence, Hydropower projects, Himalayas.

1. Introduction

Tunneling for water conveyance system is most preferable technique to Hydropower Projects in Himalaya region to gain higher elevation difference in shortest span. Most of the Himalayan terrain of Nepal formed over the rocky geology. Thus, almost all tunneling works has to be done through rock. Rocks are naturally occurring solid mass and composed of several kind of minerals and geological structures which makes rocks complex in nature. Study of type, index properties, engineering properties and attitude of the rock bears prime necessity before excavation and construction of any type underground structures in rock. Several tunneling projects in Nepal had met many fatal problems such as squeezing, swelling, spalling and rock bursting. Such challenges in tunneling are the result of rock mass's strength, techniques used for excavation and stress and deformation behavior of rock mass. Thus, enumeration of such problems and preparation of remedies for such problems before construction, during construction and after construction is mandatory.

Tunnel is shortest and economic option compared to other methods water conveyance system. In accordance with the study of Water and Energy Commission Secretariat (WECS) shows more than 850 km of tunneling needs to be done to develop priorly planned hydropower potentials in Nepal. Squeezing phenomenon is common problem in Nepal on excavating tunnel through weak rock and high overburden [1]. Squeezing is an inward displacement induced into tunnel excavation due to stress gradient created around the tunnel after excavation. Whereas stress gradient is induced due to losing existing either confinement or one of the stress components in rock making plastic rock free to converge inward to the excavation. Deformation due to squeezing is associated with the high horizontal compressive stresses in the rock. Study of stresses around underground excavation is very fruitful in order to alleviate devastating failure due to the deformation of tunnel caused by squeezing phenomenon could be

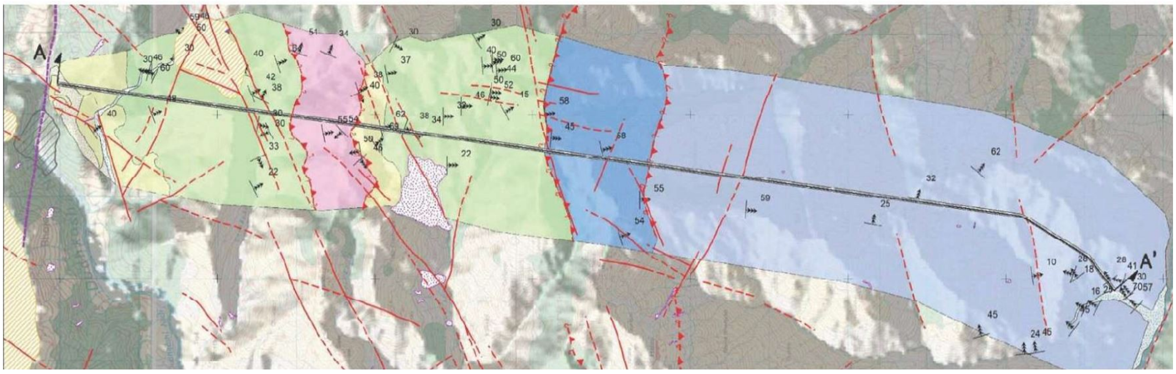

Figure 1: Tunnel Alignment Plan

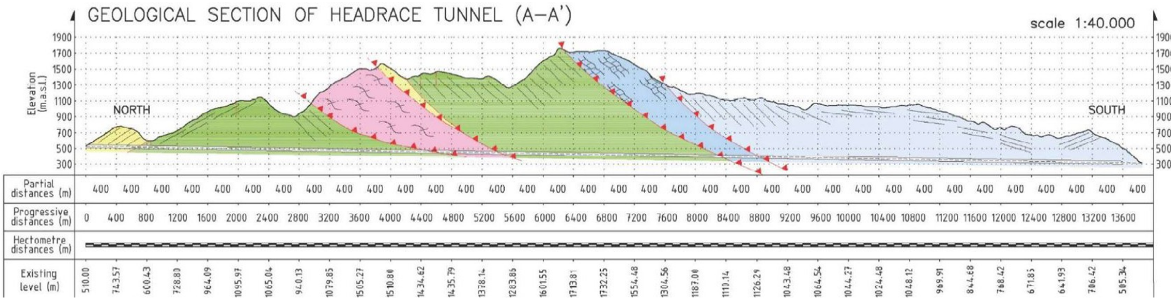

Figure 2: Geological Section of Headrace Tunnel

fruitful. Also, proper planning for the precautions and solutions by enumerating the possible amount of deformation due to squeezing, method of excavation and required support system during design phase can tackle the problems like fetal accidents, cost overrun and time unconformity on construction of tunnel although New Austrian tunneling method (NATM) is most common tunneling system. Several tunnel projects in Nepal had encountered severe squeezing and tunnel stability complications which made delayed project and increased the cost. The main objective of this study is to assess the possibility of squeezing phenomenon in tunnel before construction and calculate necessary support system to crosscut the problems.

1.1 Study Area

Dudhkoshi Storage Hydroelectric Project's headrace tunnel has been taken as study area of the study. Dudhkoshi Storage Hydroelectric Project is a storage type hydroelectric project having installed capacity of 635 MW and located at the boundaries of Okhaldhunga and Khotang Districts of Nepal. The dam of the project is located at gorge almost one kilometer downstream of the confluence of Dudhkoshi River and Thotne River and horseshoe shaped tunnel with 8.3m diameter having length of is 13.23 KM is provided to navigate water from dam to the power house. Headrace tunnel of the project lies in the lesser Himalaya zone. The majority of the rocks in the project area are characterized by a low metamorphic grade, such as quartzite, phyllite, mica schist, limestone, gneiss (both schistose and granite).

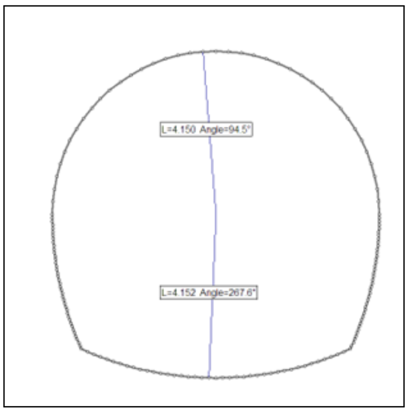

Figure 3: Section of Tunnel

1.2 Input Parameters

The required data to analyze squeezing followed by designing support systems like Q-value, RMR, Intact Rockmass Strength (UCS), Young's Modulus etc. have been accumulated by assessing the Updated Feasibility Study and Detailed Design of Dudhkoshi

Storage Hydroelectric Project.

Table 1: Input Parameters

Chainage	Overburden(m)	RMR	Q
0+000	180	51.30	2.250
0+915	180	34.11	0.333
1+129	350	41.41	0.750
2+353	500	6.94	0.016
2+853	750	43.33	0.928
3+916	750	12.23	0.029
4+077	750	37.18	0.469
4+774	750	29.79	0.206
5+524	950	42.27	0.825
6+274	1100	1.15	0.009
6+917	1000	43.91	0.990
7+587	850	26.23	0.139
7+909	700	40.26	0.660
9+919	600	24.22	0.111
10+133	600	50.15	1.980
10+908	500	12.81	0.031
11+208	230	70.87	19.800
12+683	120	13.67	0.034
12+783	180	68.28	14.850
13+227	250	65.69	11.138

2. Literature Review

As tunnel in hydropower projects is an underground structure which is dug to navigate water from water source to the electromechanical units for power generation. Tunnel is constructed by digging through the existing soil/earth/rock, providing necessary supports and keeping closed except for entrance, audits and exit. Weak as well as over-stressed rock mass may encounter squeezing phenomenon during and after excavation. Squeezing phenomenon is time dependent, slow and hazardous challenge often faced on tunneling due to the rock mass around the excavation drops its existing strength because of the influence of in situ stresses. Loss in inherent strength may cause mobilization of high support pressure and tunnel closures. Stress condition, strength and deformability of the rock mass, rock types, orientation of the geological structures and construction method and support system are causes of squeezing in tunnel excavation.

Many researches related to the assessment of possibility of squeezing, quantification of squeezing and designing of necessary support system prior to the construction of the tunnel are available. Where, methodologies adopted and recommended by the authors has been categorized as Empirical, Semi-empirical, Analytical and Numerical Modeling categories in accordance with their approaches. Author has adopted following methods to analyze the squeezing phenomenon of the study area.

2.1 Empirical Approach

2.1.1 Singh et. al. (1992):

According to the Singh's approach, possibility of squeezing phenomenon is related with the limiting overburden. If overburden is greater than limiting overburden $350Q^{1/3}$ then squeezing may occur which is as shown in Table-2.1 below [2].

$$H = 350Q^{(1/3)} * m \tag{1}$$

Table 2: Summary of squeezing criteria of Singh et. al.(1992)Approach

Approach	Squeezing Condition	Non Sqeezing Condition
Singh's Approach	$H>350Q^{1/3}$	$H<350Q^{1/3}$

2.1.2 Grimstad and Barton (1993):

According to the Grimstad and Barton's approach also called as Q-system, possibility and type of squeezing phenomenon is related with the ratio of $\sigma_{\theta\ \max}$ and σ_{cm}. which are as shown in Table-2.2 below [3].

$$Q = \frac{RQD}{J_{\mathrm{n}}} * \frac{J_{\mathrm{r}}}{J_{\mathrm{a}}} * \frac{J_{\mathrm{w}}}{SRF} \tag{2}$$

$$\sigma_{\theta\max} = 3\sigma_1 - \sigma_3 \tag{3}$$

$$\sigma_{\mathrm{cm}} = 0.7\gamma Q^{1/3} \tag{4}$$

Table 3: Summary of squeezing criteria of Grimstad and Barton (1993) Approach

$\sigma_{\theta\ \max}/\sigma_{\mathrm{cm}}$	Squeezing Type
<1	No Squeezing
1 to 5	Mild Squeezing
>5	Heavy Squeezing

2.2 Semi Empirical Approach

2.2.1 Jethwa et. al. (1984)

Possibility of squeezing in this approach is accounted as per the value of N_c which is proportion of rock mass uniaxial compressive strength (UCS) over in-situ stress. In accordance with the value of the Nc possible squeezing phenomenon can be categorized as shown in Table-2.3 below [4].

$$N_c = \frac{\sigma_{cm}}{P_O} = \frac{\sigma_{cm}}{\gamma * H} \tag{5}$$

Table 4: Squeezing behavior according to Jethwa et al. (1984)

Nc	Type of behavior
<0.4	Highly Squeezing
0.4-0.8	Moderately squeezing
0.8-2.0	Mildly squeezing
>2.0	Non squeezing

2.2.2 Hoek and Marinos (2000)

Degree of squeezing can be calculated based on the Hoek and Marinos (2000) method. In this method of predicting and estimating the squeezing, plot of tunnel convergence against the ratio of rock mass strength to in situ stress in case of unsupported tunnel is analyzed.

Hoek and Marinos (2000) suggested that the classifications of squeezing severity based on the strain percentage. There are five suggested the classes of squeezing classes based on the strain percentage. There are five classes of squeezing problems from few support problems to extreme squeezing problems i.e. from A to E. The ranges of these classes and their description are shown in Figure [5].

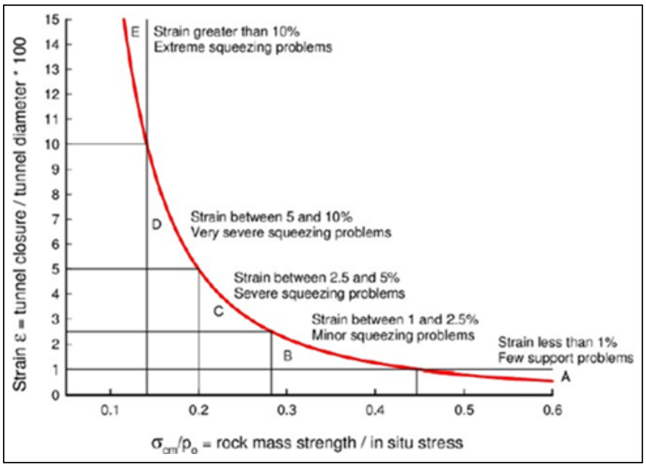

Figure 4: Squeezing prediction and quantifying curve after Hoek and Marinos (2000)

2.2.3 Analytical Method (Convergence Confinement Method)

Convergence Confinement Method (CCM) had been developed by Carranza Torres and Fairhurst (2000) and this method is capable to estimate tunnel designing parameters like strain, closure, support pressure etc. CCM has three components, Longitudinal Displacement Profile (LDP), Ground Reaction Curve (GRC) and Support Characteristics Curve (SCC) [6].

- Ground Reaction Curve (GRC)
  GRC is the graphical representation of relation between decreasing internal pressure (pi)and increasing radial displacement of tunnel wall (ur). The relationship depends upon mechanical properties of rock mass and can be obtained from the elasto-plastic solution of rock deformation around an excavation.
- Longitudinal Displacement Profile (LDP)
  LDP is the graphical representation of relation between the radial displacement that occurs along the axis of unsupported cylindrical excavation i.e. for the sections located ahead of and behind tunnel face. The diagram indicates that at some distance behind tunnel face the effect of face is negligibly small, so that beyond this distance the tunnel has converged by final value. At some distance ahead of face, the tunnel excavation has no effect on the rock mass and the radial displacement is zero.
- Support Characteristics Curve (SCC)
  SCC is the plot between increasing pressure (Ps) on the support and increasing radial displacement (ur) of the support:

1. Available support for Concrete or Shotcrete Linings. The stiffness constant Kc is as follows:

$$K_c = \frac{E_c[r_i^2 - (r_i^2 - t_c^2)}{(1+V_c)(1-2vc)r_i^2 + (r_i - t_c)^2} \tag{6}$$

The maximum support pressure developed by concrete or shotcrete lining van be calculated from the following relationship which is based on the theory of hollow cylinders.

$$P_{max} = \sigma_{cconc}(1 - (\frac{(ri-tc)^2}{ri^2}) \tag{7}$$

2. Available support for ungrouted bolts and cables: The maximum pressure provided by the support system, assuming that the bolts are equally space in the circumferential direction, is given by;

$$P_s^{Max} = \frac{T_{bf}}{S_c S_i} \tag{8}$$

And the stiffness is given by,

$$\frac{1}{K_s} = \frac{S_c S_t}{ri}\left(\frac{4L}{\Pi {d_b}^2 E_s} + Q\right) \tag{9}$$

Where,
d_b is the bolt or cable diameter [m]
l is the free length of bolt or cable [m]
T_{bf} is the ultimate load obtained from a pull out test [MN]
Q is a deformation load constant for the anchor and head [m/MN]
Es is Young's modulus of bolt or cable [MPa]
Sc is the circumferential bolt spacing [m]
Sl is the longitudinal bolt spacing [m]

3. Available support for steel set support
The maximum support pressure of the set is (Hoek's Corner)

$$P_s^{max} = \frac{A_s \sigma_{ys}}{SI.R} \tag{10}$$

And the stiffness is;

$$K_s = \frac{E_s A_s}{SI.R^2} \tag{11}$$

Where,
$sigma_{ys}$ is the yield strength of the steel [MPa]
Es is the young's modulus of the steel [MPa]
As is the cross sectional area of the section[m]
Sl is the set spacing along the tunnel
R is the radius of the tunnel [m]

4. Combined effect of support system n this case, the stiffness of the combined system is determined as the sum of the stiffness of the individual components.

$$K = K_1 + K_2 \tag{12}$$

Where,
K_1 = stiffness of the first system and K_2 = stiffness of the individual components.

2.2.4 Numerical Modeling

Phase[2] is adopted for the estimation of stress, deformation and stability of tunnel. The detail assessment using computer software is carried out only for those section which was identified as critical section. The properties of rock mass for numerical modelling are adopted as far as practicable and closer to real values. The properties of rock mass were estimated using Geological Strength Index (GSI) and blast factor D from correlations. The blast damage factor was first introduced in the year 2002 version of Hoek Brown criterion and it is used to estimate Hoek's constant. GSI is calculated from empirical formula as a function of rock mass rating (RMR) value. Input Parameters for Phase[2].

$$m_b = m_i exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{13}$$

$$S = exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{14}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - E^{-20/3}\right) \tag{15}$$

3. Results and Discussion

3.1 Squeezing analysis with empirical and semi-empirical methods

As per the Empirical and Semi-empirical method, 7 segments of the tunnel seem to face squeezing problem during construction. The result of Empirical and Semi-Empirical methods of squeezing analysis are as mentioned below:

Table 5: Result of Squeezing analysis with empirical methods

Tunnel Segment	Singh et. al. (1992)	Barton and Grimstad (1993)
0 to 915	No Squeezing	No Squeezing
915 to 1129	No Squeezing	Mild Squeezing
1129 to 2353	Squeezing	Mild Squeezing
2353 to 2853	Squeezing	Mild Squeezing
2853 to 3916	Squeezing	Heavy Squeezing
3916 to 4077	Squeezing	Heavy Squeezing
4077 to 4774	Squeezing	Heavy Squeezing
4774 to 5524	Squeezing	Heavy Squeezing
5524 to 6274	Squeezing	Heavy Squeezing
6274 to 6917	Squeezing	Heavy Squeezing
6917 to 7587	Squeezing	Heavy Squeezing
7587 to 7909	Squeezing	Heavy Squeezing
7909 to 9919	Squeezing	Heavy Squeezing
9919 to 10133	Squeezing	Mild Squeezing
10133 to 10908	Squeezing	Mild Squeezing
10908 to 11208	Squeezing	Mild Squeezing
11208 to 12683	No Squeezing	No Squeezing
12683 to 12783	Squeezing	No Squeezing
12783 to 13227	No Squeezing	No Squeezing
13227 to 13419	No Squeezing	No Squeezing

Table 6: Result of Squeezing analysis with semiempirical methods

Tunnel Segment	Jethwa et. al. (1984)	Hoek and Marions (2000)
0 to 915	No Squeezing	Few Support Problems
915 to 1129	No Squeezing	Few Support Problems
1129 to 2353	Mild Squeezing	Few Support Problems
2353 to 2853	Moderate Squeezing	Few Support Problems
2853 to 3916	Highly Squeezing	Few Support Problems
3916 to 4077	Highly Squeezing	Extreme Squeezing Problems
4077 to 4774	Highly Squeezing	Few Support Problems
4774 to 5524	Highly Squeezing	Minor Squeezing Problems
5524 to 6274	Highly Squeezing	Minor Squeezing Problems
6274 to 6917	Highly Squeezing	Extreme Squeezing Problems
6917 to 7587	Highly Squeezing	Minor Squeezing Problems
7587 to 7909	Highly Squeezing	Severe Squeezing Problems
7909 to 9919	Highly Squeezing	Minor Squeezing Problems
9919 to 10133	Moderate Squeezing	Few Support Problems
10133 to 10908	Mild Squeezing	Few Support Problems
10908 to 11208	No Squeezing	Few Support Problems
11208 to 12683	No Squeezing	Few Support Problems
12683 to 12783	No Squeezing	Few Support Problems
12783 to 13227	No Squeezing	Few Support Problems
13227 to 13419	No Squeezing	Few Support Problems

3.2 3.2 Squeezing analysis with analytical method (Convergence and Confinement Method)

After the determination of the squeezing phenomenon at tunnel segments. The tunnel section from the CAD drawings has been determined. Then the analytical method for deformation calculation and support system design has been done using Confinement Convergence Method. Where maximum deformation of tunnel is 0.42 m at chainage 6+684 and then the 25 mm thick M40 grade fiber shotcrete with steel rib W-310 and 25 mm dia. rock bolt of length 4.5m has been provided. The deformation after providing support is limited to 0.157 m. Thus, calculated GRC, LDP, SCC and support interaction curve are as shown in figures 5 to 8 below:

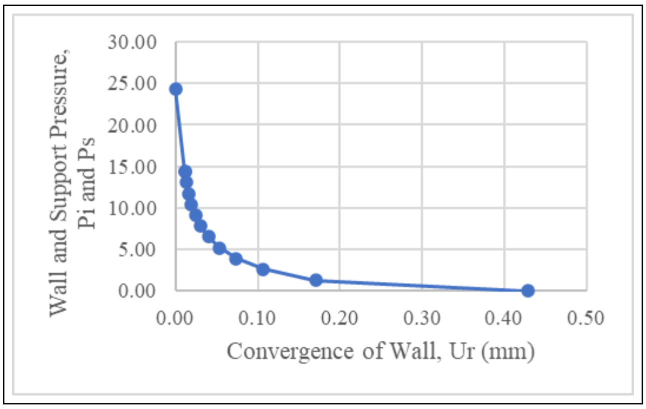

Figure 5: Ground Reaction Curve (GRC)

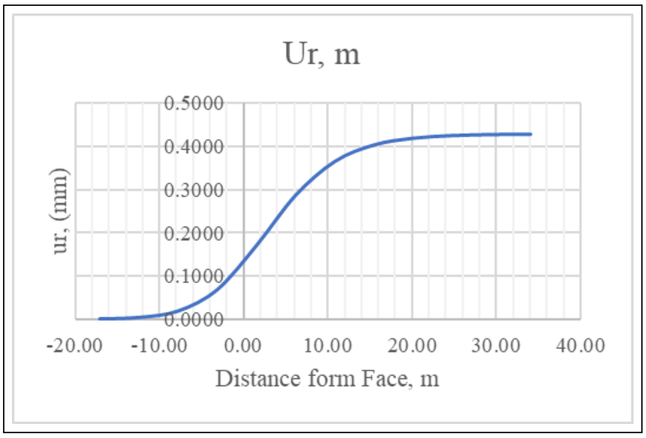

Figure 6: Longitudinal Displacement Profile (LDP)

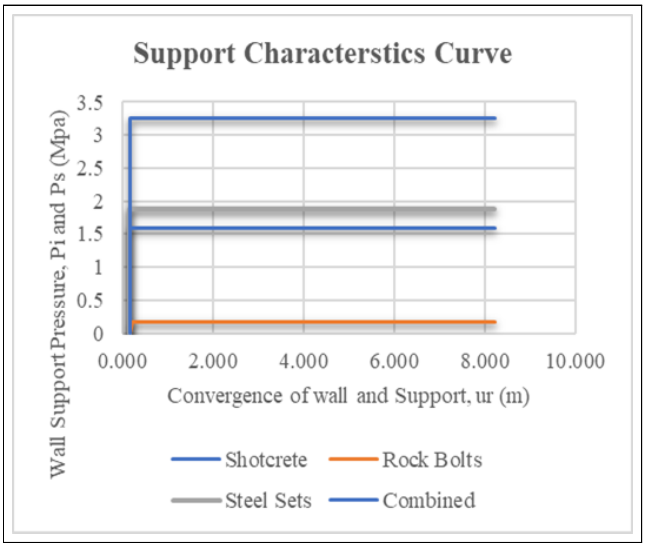

Figure 7: Support Characteristics Curve (SCC)

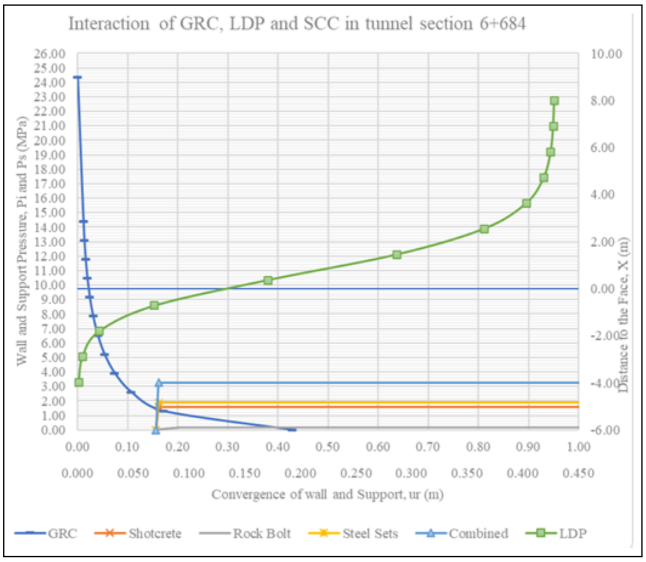

Figure 8: GRC, LDP and Support Characteristic interaction curve

In Figure 5-8, if the support is applied at the face of tunnel there will be 0.132 m displacement at tunnel wall. At the face of the tunnel, the maximum pressure that the support can experience is 9.14 MPa whereas the maximum support capacity for combined support (shotcrete+ rock bolt+ steel ribs) is only 4.65 MPa. So, the support will fail before it experiences 9.14 MPa pressure. To overcome the failure of support, either support capacity should be increased to the value more than support pressure when support is applied at tunnel face or the support can be applied at some distance behind tunnel face. Both of these solutions have some difficulties such as for the first case increase of support capacity can be achieved with concrete lining but application of concrete lining at the face of tunnel is very challenging work. And for the second case, tunnel size will be reduced to some extent more than acceptable limit but that could lead to total collapse if support is delayed and then support application will also be very challenging task. In the Figure 8, if the support is applied 1m behind the face. The tunnel wall deformation at this distance will be 0.157m which gives 36.6 percent strain and support pressure will be 1.40 Mpa. The rock bolt and steel sets will be failed before they reach their capacity. Shotcrete will sustain the support pressure with F.O.S is equal to 1.63 (2.276/1.4) and combined support will be working with F.O.S 3.31(4.65/1.4).

3.3 Calculation Verification with Numerical Modeling

Then based on the field database and literatures the following input parameter for phase[2] has been considered and used to verify the result from Semi-empirical and analytical method.

Table 7: Input Parameters for Phase2

CH	sigmaci, Mpa	RMR	GSI	m_i	d
3+916	60	25	20	7	0.1
5+100	60	40	35	7	0.1
6+008	60	55	50	7	0.2
6+684	55	25	20	7	0.1
6+917	50	55	50	7	0.2
7+887	45	40	35	9	0.1
8+589	40	55	50	12	0.2

After modeling and analyzing the individual tunnel section with distinct properties of rock and insitu stress condition, the verification of squeezing phenomenon may occur on tunnel excavation has been verified. During the verification by numerical method the result from analytical method came closer with the numerical model's output rather than empirical and semi empirical methods. Where maximum of tunnel deformation without support is 0.41 m at chainage 6+684 and then the 25 mm thick fiber shotcrete with steel rib W-310 and 25 mm dia. rock bolt of length 4.5m has been provided. The deformation after providing support is limited to 0.23 m. Some representative results are determined as shown in figures below:

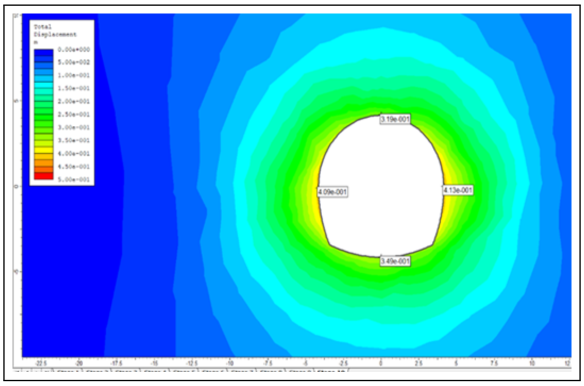

Figure 9: Deformation before installation of support and Plastic zone Radius

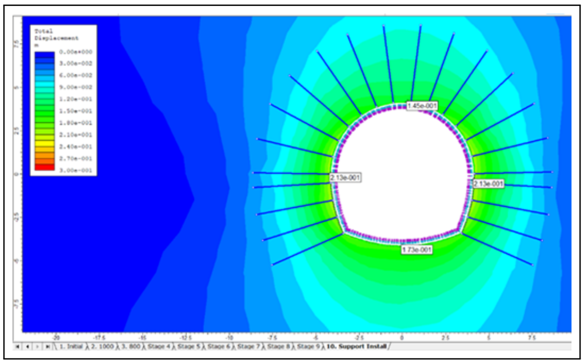

Figure 10: Deformation after Installation of Support

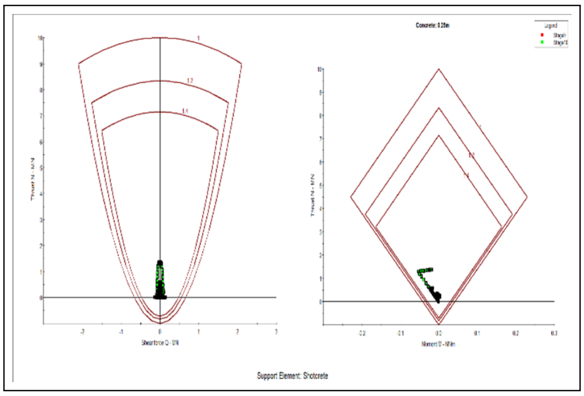

Figure 11: Support capacity Curve for Fiber Shotcrete

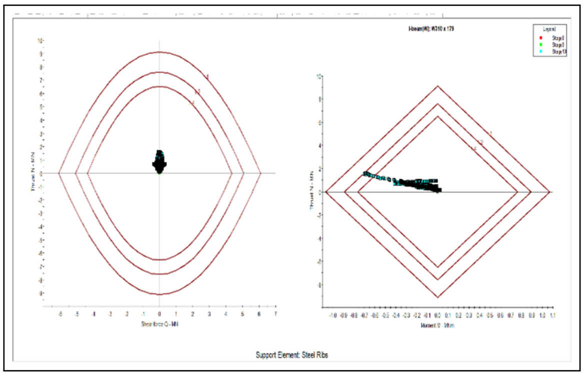

Figure 12: Support capacity Curve for Steel Rib

4. Conclusion and Recommendation

4.1 Conclusion

Possible squeezing phenomenon has been predicted as well as quantified based on the empirical, semi-empirical method and analytical method and verified with the numerical method. Where results from semi empirical method (Hoek and Marinos 2000) and Analytical Method (Convergence Confinement Method) are closer to the values calculated from numerical method. Also, the analysis for design of support system against deformation induced by squeezing is done using empirical (RMR and Q-System) and semi-empirical (Convergence Confinement Method). Then verified with numerical modeling. For designing of support system, results from Convergence Confinement Method coincides the results from numerical model.

4.2 Recommendations

Recommendations after the study are:

- Tectonic stress has been calculated based on the literatures thus detail seismic study is advised.
- The effect of water needs to be analyzed in detail to calculate the change in the rock mass strength due to rise in pore water pressure.
- Maximum value of insitu stress due to overburden only is estimated 29 MPa thus fore poling and benching is advantageous during construction.

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