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Evaluation on the Squeezing and Design of Support System in Headrace Tunnel of Middle Modi Hydroelectric Project

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Abstract

Tunneling through a weak rock possess a great challenge. These challenges depends upon type of rock, excavation method, stress and deformation behavior of the rock etc. Squeezing is one of the major problem which is likely to occur during excavation of tunnel through weak rock. A reliable prediction of the extent of squeezing is essential so that a strategy can be established regarding stabilizing measures and for optimizing the support well in advance.

In this paper, Middle Modi Hydroelectric Project located in the Kaski District has been taken as the case study. In this project, huge squeezing problem occurred at about chainage 1+140m. At this section deformation has been recorded well over 65cm. Hence, this paper basically deals with squeezing analysis using different approaches. Rock types along the headrace tunnel alignment are sheared phyllite and fractured quartzite. Mostly, intercalation of phyllite and quartzite has been found in the squeezed section. Rockmass quality found in the squeezed section is extremely poor to exceptionally poor. Four main methods have been used to evaluate the squeezing phenomenon viz.;empirical methods such as Singh (1992) and Goel (2000),semi-empirical such as Hoek and Marinos (2000), analytical method such as (Convergence Confinement Method, 2000) and numerical program Phase^2.The main factors that control the squeezing phenomenon are the rock mass parameters and rock stresses. The uniaxial unconfined compressive strength of intact rock has been back calculated from measured deformations using phase2 program and found to be in the range of 10 to 15Mpa in the squeezed section. Deformation was calculated using CCM and Compared with phase^2 result. Due to excessive deformation temporary supports were provided at several locations, steel ribs are buckled and shotcrete lining is also cracked. All these have to be removed before application of final lining. Finally two different approaches have been studied using phase^2 program to address the existing problems in squeezed section of headrace tunnel i.e. design of support system considering either circular shape with final lining (shotcrete and steel rib) or reshaping existing D-shaped tunnel into horseshoe shaped and providing final concrete lining.

Keywords: Tunnel; Squeezing; deformation; rock mass; Hydropower projects

1 INTRODUCTION

Himalaya is young mountain with complex geology and the tunneling activity in various projects in Himalayas are suffered by diverse geological problems such as difficult terrain conditions, thrust zones, shear zones, folded rock sequence, in-situ stresses, rock cover etc. All these challenges may result in increased cost and extended completion period. High stresses in weak rock mass are among the major causes for plastic deformation in tunnels. Excessive deformation in the periphery of a tunnel eventually causes it to collapse. The plastic rock which was confined before excavation loses its confinement or one of its stress components and thus is free to move into the excavation. This movement creates high horizontal compressive stresses in the rock.

Weak and deformable rocks such as phyllite, schist, schistose gneiss and rock mass in weakness and fault zones are incapable of sustaining high tangential stress thereby resulting in squeezing of the tunnel section. Therefore, Engineering principles and applications are pre-requisite to ensure safe and economic solutions of the problems. D-Shaped tunnel is generally preferred as it provides flat floor for equipment and provides a pleasant working platform. This shape is suitable for shallow tunnels in good quality soil but stress concentration is high at the corners where the sidewalls meet the floor or invert in D-shaped tunnel. In case of poor rock like phyllite, shape of tunnel should be modified to Horseshoe or circular. Good knowledge of stresses around underground excavation helps to mitigate or reduce the squeezing.

The best way to deal with severe squeezing is to build a strategy well in advance (during planning and design) regarding stabilizing measures for minimizing stability problems and optimizing the support. Hence, reliable prediction on the extent of squeezing is essential. A probabilistic approach of uncertainty analysis that focuses on the effect of variation in each input parameter is the most reliable way of predicting the extent of squeezing.

Project Background

Middle Modi Hydroelectric Project is a run-off-river hydropower project located in Kaski District of Gandaki Province in Western Development Region, Nepal. The approximately 2840 m long headrace tunnel is proposed to be located along the right bank of Middle Modi. Mainly three types of rock mass are present along the alignment of the headrace tunnel, they are Phyllite, Schist and quartzite. Rock class throughout the tunnel belongs to category poor to exceptionally poor which is very challenging during construction and after construction.

Objective

- Review existing theory on the stability issues in tunneling with particular focus on tunnel squeezing
- Calculation of rock mass properties using empirical methods and literature review.
- Document on the rock support principle used while tunneling in the middle Modi and document on measured deformation along the tunnel alignment.

- Assessment of squeezing using empirical, semi empirical, analytical approaches.
- Attempt to produce a support characteristic curve based on applied support, measured final deformation and reviewed theory.
- To carry out numerical modelling for stability assessment of tunnel
- Compare and discuss the analysis results from empirical, semi-analytical, analytical and numerical approaches.

2 LITERATURE REVIEW

Criteria for squeezing ground condition

According to Mohr's theory, squeezing occurs if maximum tangential stress at the face of excavation is greater than UCS of rock mass. Singh (1992) determined squeezing phenomena on the basis of Barton's Q-value of rock mass and height of overburden. Similarly, Goel (1994) approach expressed squeezing phenomena based on rock mass number, width of tunnel and height of overburden. Here, Rock mass number is the Q-value where Strength Reduction Factor (SRF) is equal to one. These two approach are empirical method and their criteria are well described in section 3. Likewise, semi-analytical approaches that are used for the analysis of tunnel squeezing phenomenon are Jethwa et al. (1984), Hoek and Marinos (2000), etc. For analytical approach, Convergence Confinement Method (CCM) is used which gives detail estimate of stress and deformation. It is also used for the appropriate location for support design and to find support pressure required to resist that deformation. This method only consider circular shape of the tunnel and hydrostatic stress condition only. But in reality D shaped and Horseshoe shape is mostly used and stress condition is not perfectly hydrostatic. So, Numerical modelling is needed for the further assessment of squeezing. For this purpose, computer software Phase^2 has been used.

3 METHODS OF SQUEEZING ANALYSIS AND SUPPORT DESIGN

Data Collection

The relevant data on Middle Modi Hydroelectric Project was provided by Senior Geologist "Druba Mishra". Data were collected from two sources. General data such as unit weight, modulus of elasticity, Poisson's ratio, uniaxial compressive strength of intact rock and other properties of rock mass were obtained from literature and expert advice. Whereas specific data like tunnel dimension, plan, and ground profile of tunnel alignment, rock type, and rock mass classification were obtained from Geological Report of Middle Modi Hydroelectric Project. Input parameter like UCS of intact rock was back calculated for measured deformation using Phase^2 software.

Table 3-1: Input Data of Middle Modi Hydroelectric Project (48.5MW)

From	To	Overburden	Q-value	RMR
0	140	50	1	45
140	1160	315	30	85
1160	1400	275	0.005	20
1400	1600	250	0.03	20
1600	1760	200	0.4	35
1760	2220	210	3	50
2220	2300	150	0.3	30
2300	2700	125	2	50
2700	2840	80	0.4	35

There have been many researches and papers for the prediction of methods of squeezing analysis and support design and a number of approaches have been proposed by many authors. New and highly effective technologies have been developed in the last few years. Similarly, many case histories have been documented with prediction of the deformation and support requirement for squeezing rocks. The main approaches can be categorized in the five main categories.

i. Empirical methods
ii. Semi-Empirical methods
iii. Analytical methods
iv. Numerical modelling methods

Empirical methods

Singh et al. (1992) approach

This approach was developed by collecting data on rock mass quality Q 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 equation of the line is

$H = 350\ Q^{1/3}$

For Squeezing condition, $H > 350\ Q^{1/3}$ (m)

For Non- Squeezing condition, $H < 350\ Q^{1/3}$ (m)

Goel (1994) approach

Goel (1994) developed an empirical approach based on the rock mass number N, defined as Q with SRF = 1. Considering the overburden depth H, the tunnel span or diameter B, and the rock mass number N from 99 tunnel sections.

The equation of this line is

$H = (275N^{0.33})\,B^{-1}$

For squeezing conditions,

$H \gg (275N^{0.33})\,B^{-1}[m]$

For Non- Squeezing conditions

$H \ll (275N^{0.33})\,B^{-1}[m]$

Semi-Empirical Method

Hoek and Marinos Approach (2000)

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

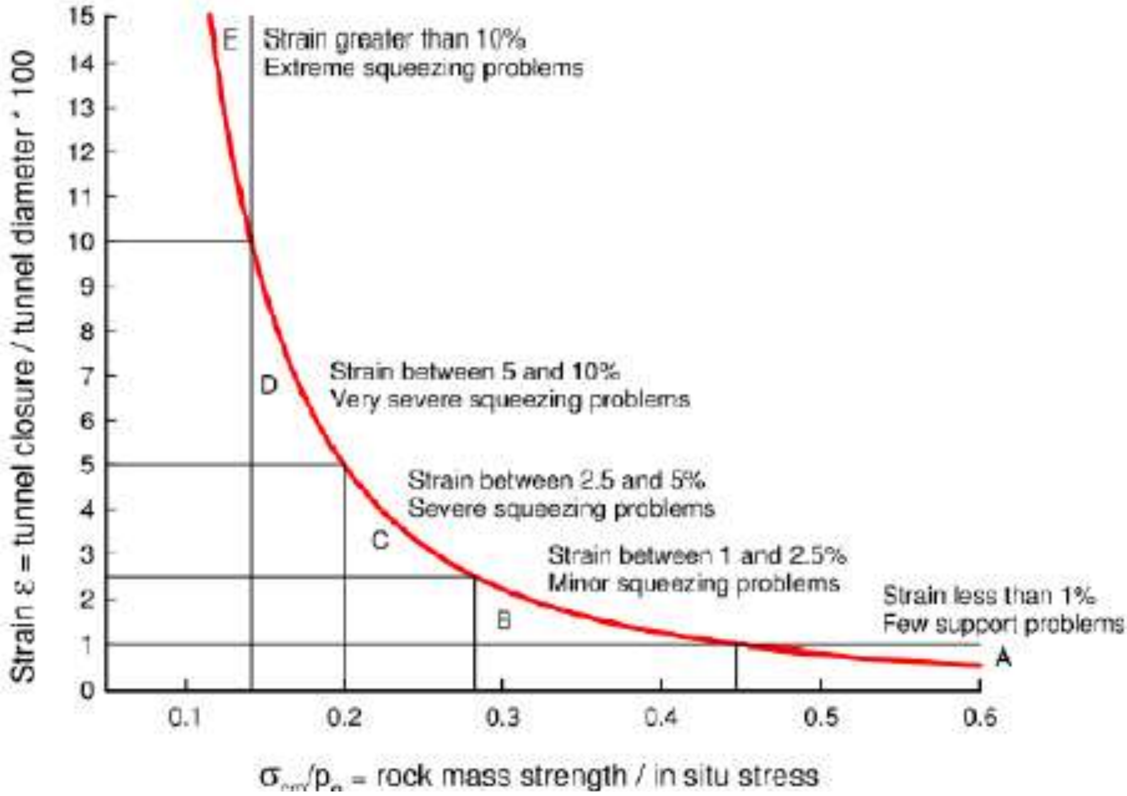

Figure 3-1 Appropriate relationship between strain and the degree of difficulty associated with tunneling through squeezing rock in case of unsupported tunnel(Hoek and Marinos, 2000)

Jethwa et al. (1984) approach

The degree of squeezing is defined by Jethwa et al (1984) on the basis of the following,

$$N_c = \frac{\sigma_{cm}}{p_o} = \frac{\sigma_{cm}}{\gamma H}$$

Where:

σ_{cm} = rock mass uniaxial compressive strength;
p_o = in situ stress
γ = rock mass unit weight
H = tunnel depth below surface

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

Analytical methods

Convergence Confinement Method (CCM) using Hoek-Brown Criteria

Carranza-Torres and Fairhurst (2000) concluded that CCM has three basic components viz. the Longitudinal Displacement Profile (LDP), the Ground Reaction Curve (GRC) and the Support Characteristics Curve (SCC).

1. Longitudinal Displacement Profile (LDP),
2. Ground Reaction Curve (GRC) and
3. Support Characteristics Curve (SCC).

Longitudinal Displacement Profile (LDP)

It is the graphical representation of radial displacement that occurs along the axis of unsupported cylindrical excavation i.e. for the sections located ahead of and behind tunnel face.

Ground Reaction Curve (GRC)

It is the relationship between decreasing internal pressure pi and increasing radial displacement of tunnel wall u_r.

Support Characteristics Curve (SCC) is defined as the relationship between increasing pressure pi on the support and increasing radial displacement u_r of the support.

4 DESIGN OF SUPPORT SYSTEM

Empirical methods

Q-value and the six appurtenant parameter values give a description of the rock mass. Based on documented case histories a relation between the Q-value and the permanent support is deducted, and can be used as a guide for the design of support in new underground projects. To express safety requirements, a factor called ESR (Excavation

Support Ratio) is used. A low ESR value indicates the need for a high level of safety while higher ESR values indicate that a lower level of safety will be acceptable (Grimstad and Barton, 1993).

Closed form solution methods

Available support for Concrete or Shotcrete Lining

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

$$p_{max} = \sigma_{c.conc}\left[1 - \frac{(r_i - t_c)^2}{r_i^2}\right]$$

The elastic stiffness constant is given by

$$K_s = \frac{E_c\{r_i^2 - (r_i - t_c)^2\}}{(1+v_c)\{1-2v_c)\, r_i^2 + (r_i - t_c)^2\}}$$

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;

$$P_s^{max} = \frac{T_{bf}}{S_c\, S_l}$$

And the stiffness is,

$$\frac{1}{\ } = \frac{S_c\, S_t}{\ }\left[\frac{4l}{\ } + Q\right]$$

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 pullout test [MN]
Q is a deformation load constant for the anchor and head [m/MN]
E_s is Young's modulus of bolt or cable [MPa]
s_c is the circumferential bolt spacing [m]
s_l is the longitudinal bolt spacing [m]

Available support for steel set support

The maximum support pressure of the set is (Hoek's Corner)

$$P_s^{max} = \frac{As\,\sigma_{ys}}{S_l\, R}$$

And the stiffness is;

$$K_s = \frac{E_s A_s}{S_l . R^2}$$

Where

σ_{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]
S_l is the set spacing along the tunnel axis[m]
R is the radius of the tunnel [m]

Combined effect of support system

In this case, if two supports having the elastic stiffness K_{s1} and K_{s2} and maximum pressure $p_{s1}{}^{max}$ and $p_{s2}{}^{max}$ respectively are installed in the same location, their combined stiffness can be computed as

$$K_S{}^{eq} = K_{s2} + K_{s2}$$

The maximum possible elastic deformations for the two support are $u_{r1}{}^{max}$ and $u_{r2}{}^{max}$ respectively. The support with the lowest maximum value, $u_r{}^{max}$ determines the maximum support pressure available for the supports acting together which can be calculated as

$$ps = K_s u_r$$

Numerical analysis

Phase2 has been used for the modeling purpose for the calculation of deformation, stress and stability of tunnel. Generalized Hoek and Brown approach has been adopted only for those section which was identified as critical section that is at chainage 1+140. UCS of intact rock (σ_{ci}) was first taken as provided in the geological report of Middle Modi and deformation is calculated and compared with field measurement. Again, (σ_{ci}) is back calculated for causing measured deformation in phase^2 modelling and the revised one is used for further modelling. The properties of rock mass was estimated using Geological Strength Index (GSI) and blast factor D. Blast factor is taken as 0 in case of excavation and less than 0.8 in case of blasting.

$$m_b = m_i \, exp\left(\frac{GSI - 100}{28 - 14D}\right)$$
$$s = exp\left(\frac{GSI - 100}{9 - 3D}\right)$$
$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

GSI is calculated from empirical formula as a function of rock mass rating (RMR) value. The relation between GSI and RMR is given by relationship

GSI = RMR – 5 (Hoek et al, 1995).

The input parameters and the resulting output in the form of rock mass parameters have been tabulated below. Maximum rock cover of 270m has been used for this computation which is present at Chainage 1+140.

Input parameters	Input Value	Remarks
Backcalculated intack rock strength	12.5	
Geological strength index	15	Calculated using formula
Rock mass constant, mi	7	
Disturbance factor (D)	0	
Unit weight of rock mass (kg/cm3)	0.026	
Depth to tunnel, m	270	
Intact Modulus(Ei)	7000	As per collected data
mb	0.336	
s	0.00007	
a	0.5611	
Vertical rock sress	7.15	
Insitu stress ratio	0.5	Calculated using empirical relation

5 RESULT AND DISCUSSION

From Figure 5-1 and Figure 5-2 it can be seen that factor of safety at chainage 1+140 is lowest which is less than 1. Hence, it can be concluded that this chaiange is susceptible to squeezing.

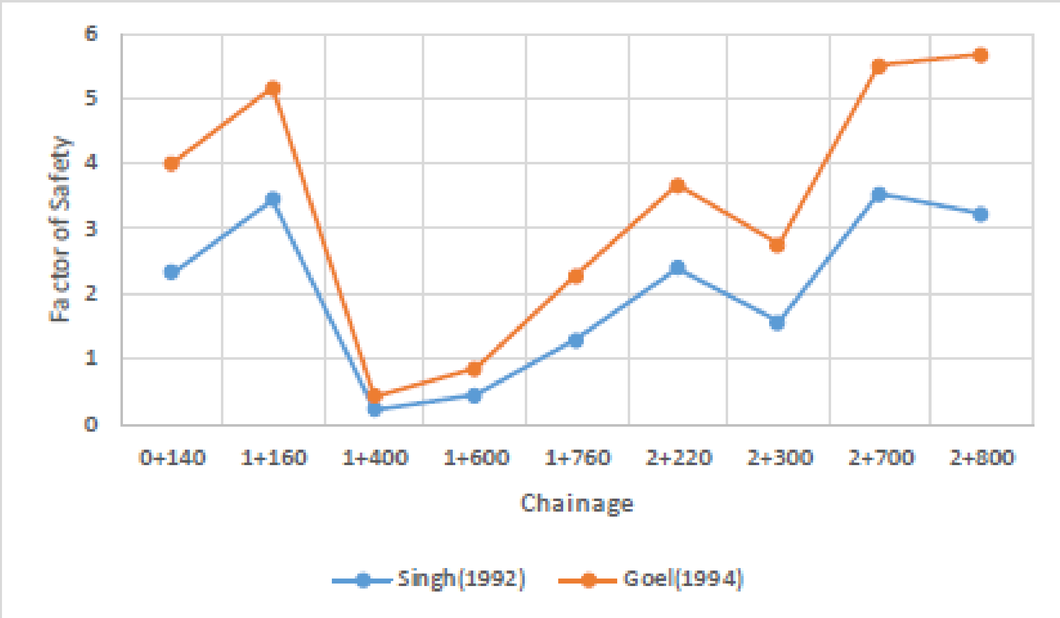

Figure 5-1 Results showing Factor of safety from Singh's approach and Goel's approach.

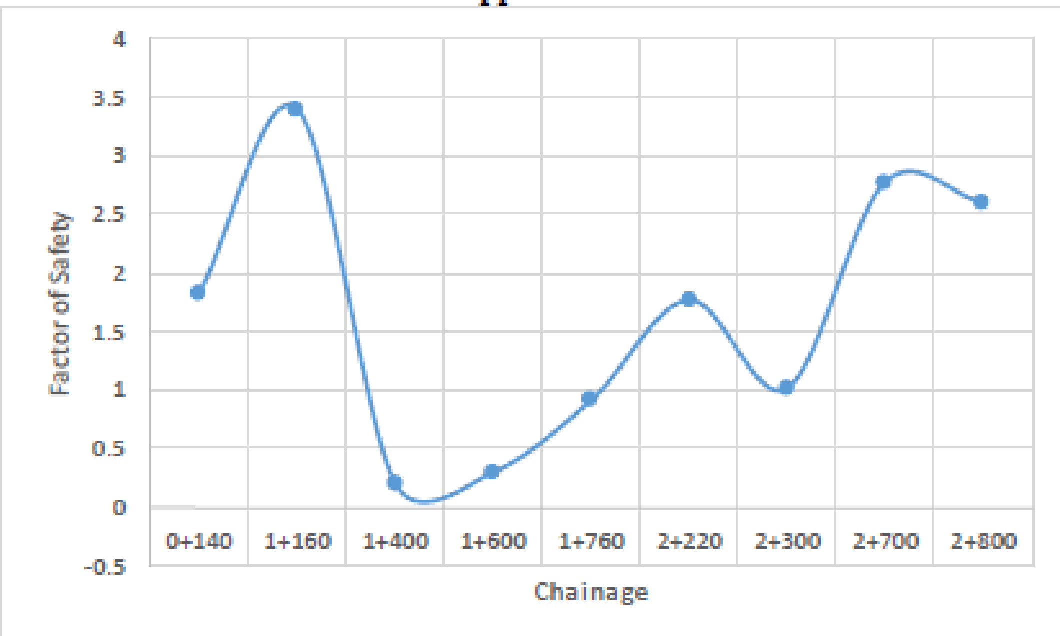

Figure 5-2 Graph showing Chainage against Factor of safety from Jethwa et.al (1984)

Deformation and stress was calculated at this section using analytical method CCM .Firstly,elastic deformation was calculated which is 9mm. After that, Plastic deformation is estimated. The maximum plastic deformation was found to be in the range of 810 mm. Radius of plastic zone was found to be 14m. Support interaction curve was plotted as shown in **Figure 5-3.**

If support is applied at face of tunnel there will be 0.101m displacement at tunnel wall.The maximum pressure that he support can experience at the face of tunnel is 1.12Mpa whereas the maximum support capacity for combine support is only 0.78Mpa. So the support will fail before it experience 1.12Mpa pressure.
To overcome the failure of support.,either support 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. But both are very challenging option. Deformation if support is applied behind 2m from face is found to be 360mm(16% strain) and support pressure at this place is 0.25Mpa. The rock bolts and steel sets will be failed before they reach their capacity.Shocrete will sustain the support pressure with F.O.S equal to 2.4 (0.6/0.25) and combined working with F.O.S equal to 3.12(0.78/0.25).

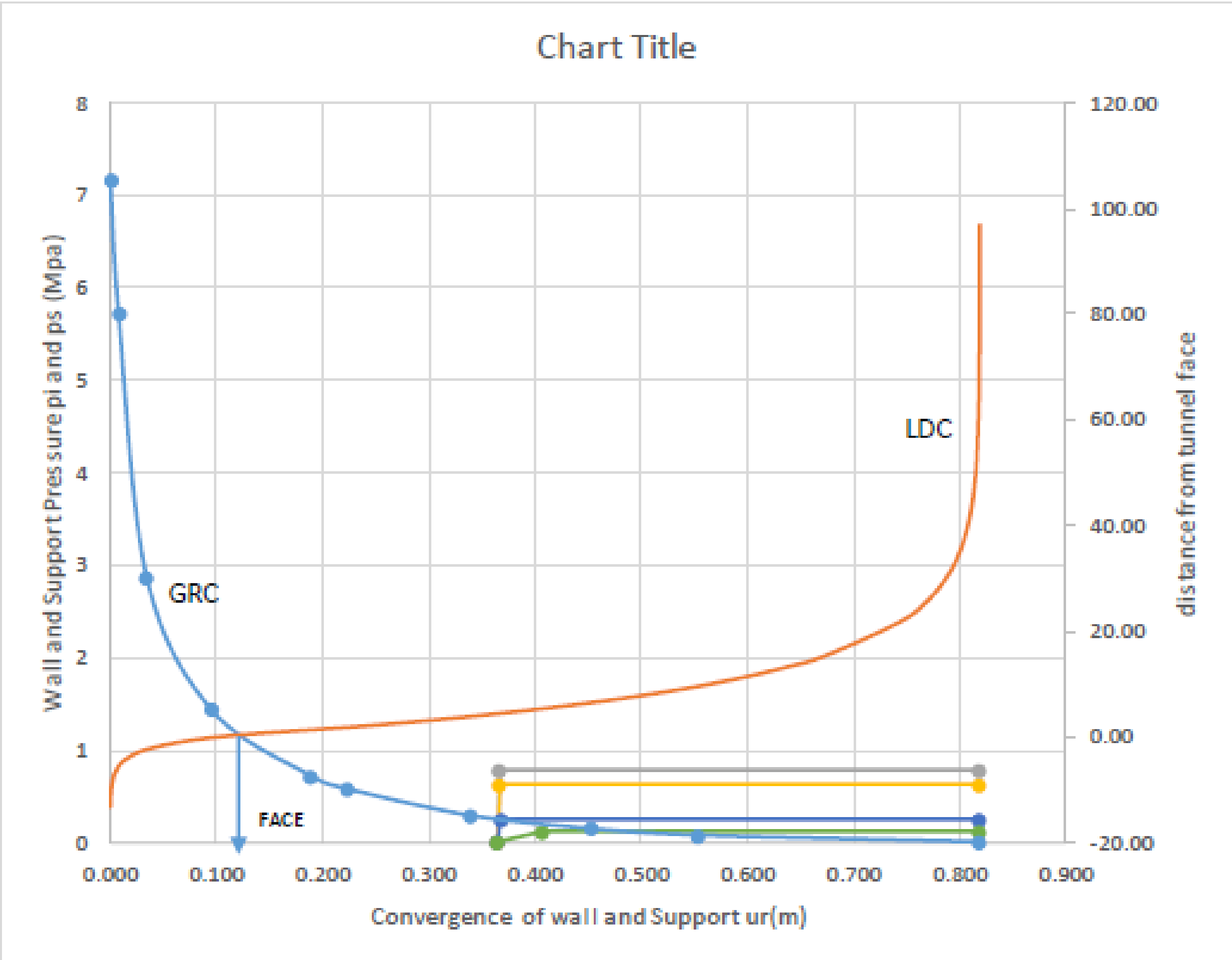

Figure 5-3 Interaction of GRC,LDP and SCC in tunnel section 1+140m

Deformation calculated from CCM not comparable with measured value and radius of plastic zone is also so high. CCM considers circular shape of tunnel with hydrostatic stress condition only so for comparison of field deformation and design of different support system phase2 analysis is most required.Phase2 is used to calculate stress and deformation and it is applicable for any type of shape. First of all, using measured deformation actual UCS of rock mass has been calculated. It gives UCS of intack rock in the range of 10 to 15 MPA. Both elastic and plastic analysis was carried out using Mohr coulomb method for elastic deformation and Generalized Hoek and Brown for plastic deformation.

The model for Chainage 0+140 has been created in Phase² program. For the loading, field stress type is chosen as a constant. Model has been generated for both elastic and plastic analysis and also for analysis with and without support application .The typical D-shaped Phase² model for tunnel section 0+140 for strength factor is shown in **Figure 5-4** .

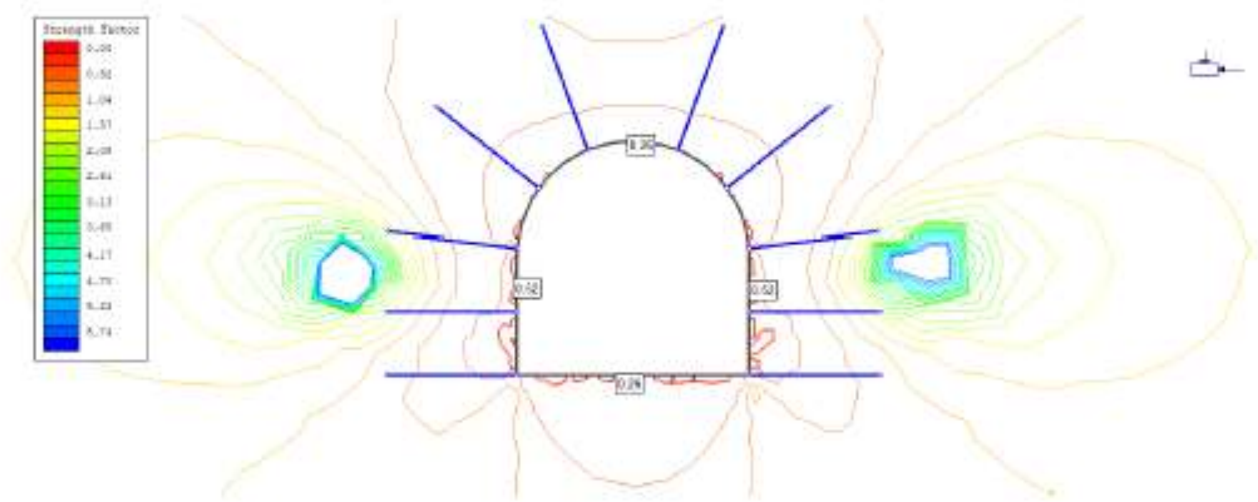

Figure 5-4 Strength factor before and after support application for section 1+140m (Elastic Deformation)

Here, strength factor is less than one means all the support will get failed, and for more additional information plastic analysis would be necessary.

Table 5-1 Stress and deformation values from phase2 program at Chainage 1+140m

Mode of deformation	Sigma1(Mpa)	Deformation (m)	Strength factor	Measured deformation in the field(m)
Elastic deformation before using support	1.5	0.07	0.52	
Elastic deformation after using support	1.5	0.07	0.52	
Plastic deformation before using support	0.3	1.12	1.3	
Plastic deformation after using support	1.8	0.75	1.04	0.65

The displacement with support obtained from Phase2 analysis is almost equal to measured tunnel wall closure. Shotcrete used as per the information was 100mm along with steel rib(300mm) ISMB300 and rockbolt of length 2.5m at a spacing of 1.1m.It can be seen that due to tensile failure extent of plastic zone could not be marked well. Here, in the phase2 modelling support has been provided just after excavation. But, all the support installed got failed. So, D-shaped tunnel could either be modified to Horseshoe shape and final concrete lining is provided or it could be made circular and final lining (shotcrete+steel rib) are recommended.

Concrete lining in horse shoe shape

After reshaping of tunnel in horseshoe shape, the concrete lining has been applied as 0.4m.Phase[2] program has been used to analyze the stability and to determine deformation around tunnel. Support with concrete lining 0.4m was proposed by the project. The concrete lining having young's modulus 35000mpa, Poisson's ratio 0.2, compressive strength 35MPa and tensile 3 MPa has been used. The bolts and shotcrete that were applied at the time of excavation has been neglected because most of them are already failed and most of them will be removed during re-excavation. The analysis shows deformation of around 4%.

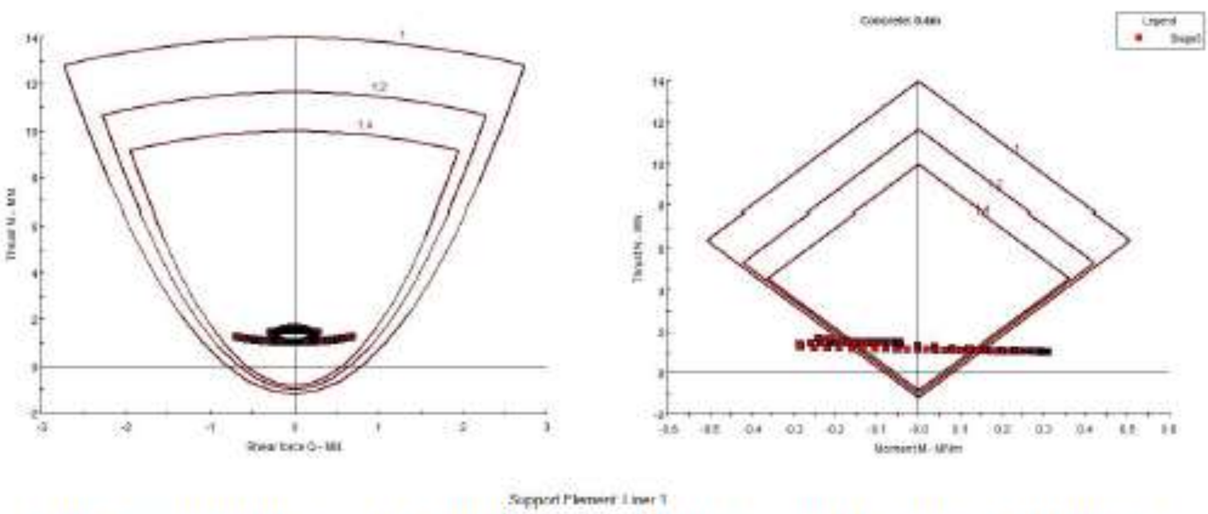

Figure 5-5 Support capacity plot of concrete lining in tunnel sections at chainage 1+140m for horseshoe shape tunnel

The lining element have factor of safety less than 1.4. Hence, the support capacity with this factor of safety is inadequate that means there will be high chance that support will fail in near future with time dependent long term deformation.

Steel ribs and shotcrete in circular shape

Another solution to the squeezing section will be apply the final lining after reshaping of tunnel into circular shape with diameter 4.4m. Again phase2 program has been used for the stability analysis and deformation calculation. The final lining will steel rib and shotcrete with thickness 0.6m.The steel ribs will be W 250*22. with spacing 0.5m and yield strength 400 MPa. The shotcrete lining will consist of 0.6m thick,25000 MPa young's modulus , 0.25 Poisson's ratio,30 MPA compressive strength and 2 MPa tensile strength .The effect of rockbolts and shotcrete linings that were applied at the time of excavation has been neglected. Analysis shows deformation is within 2.5 to 3 % .The higher deformation is due to exceptionally poor rockmass condition.

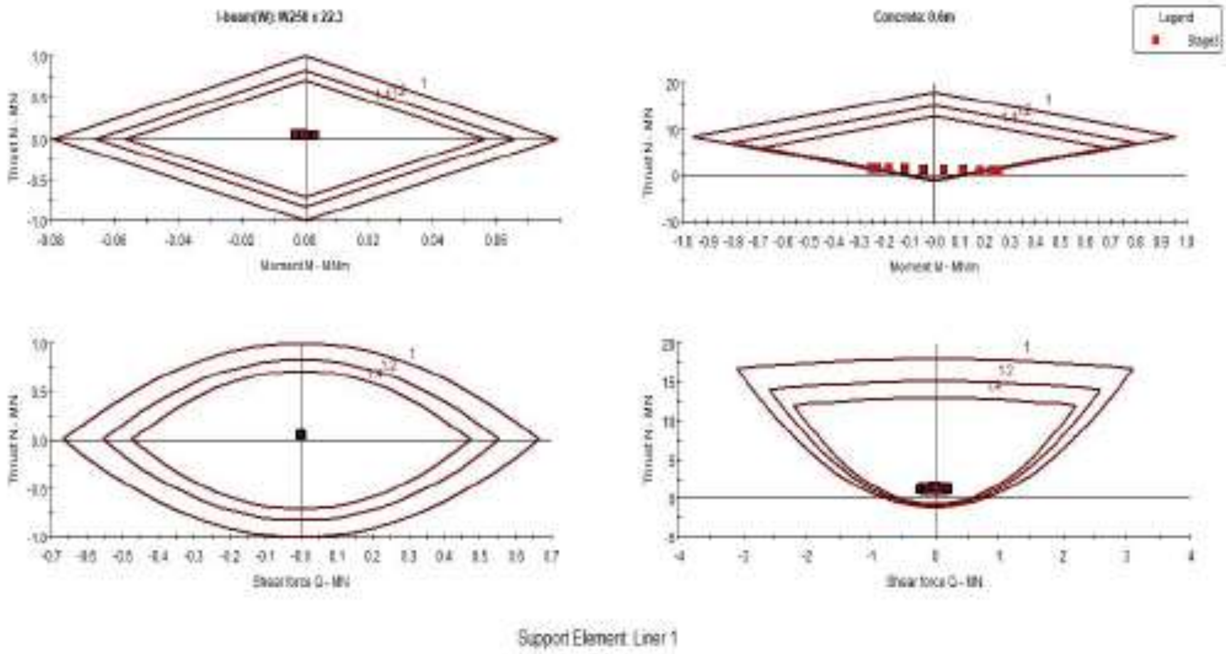

Figure 5-6 Support capacity plot of concrete lining in tunnel sections at chainage 1+140m for circular shape tunnel

6 CONCLUSION

Empirical method and semi-empirical method of predicting squeezing rock condition gives similar result. The squeezing potential was observed at chainage 1+140m of headrace tunnel of AKHPP. However assessment from empirical and semi-empirical methods only give reasonable result for the preliminary study. For detail design, more accurate approach should be adopted such as CCM and Phase2. The result from CCM suggests that to control failure of the support and displacement capacity of support system should be increased when it is applied at the face or rockmass properties should be improved before excavation by application of forepoling, pre injection of grouting etc. Phase2 concludes that squeezing is controlled by modifying shape to horseshoe or circular and using final lining. Final deformation after the application of support is 50mm only.

7 RECOMMENDATIONS:

- Here, only one critical section has been considered for the stability assessment of tunnel. It would be better to design support system throughout the tunnel section.
- Analytical method such as CCM are not adaptable for all shapes of tunnel and in uneven stress distribution. Further, numerical analysis has to be performed which is suitable in all conditions.
- Tectonic stress and ground water table has not been considered in the squeezing analysis due to lack of field data. Thus, for better result, ground water table can be valuable parameter to assess squeezing ground.

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