Analysis Of Tunnel Squeezing Phenomenon In Hydropower Project.

Season Maharjan

Abstract: The challenges in underground excavation depends upon rock mass strength, method of excavation and stress and deformation behavior of rock. It is important to obtain reliable estimates of these problem as early as possible in order to carry out construction with least difficulties. Squeezing is one of the common problems in excavation in Himalayan region where weak rock mass moves radially inward leading to decrease in size of excavated portion. Different methods has been used for the estimation of potential squeezing phenomenon. As rock consists of complex formations and exhibit a wide range of behaviors, the analytical methods provided some approximate solutions only. For the simulation of deformation and support behavior, numerical approach has been carried out for the case study of Ankhu Khola Hydropower Project (42.9MW). RMR and Q-Value were found to be poor to fair for Phyllite schist with overburden varying from 49 m to 613m for headrace tunnel of the same project. Consequently, support system has been designed for the potential squeezing sction.

Index Terms: Tunnel, Squeezing, deformation, rock mass, case study, Hydropower projects, Himalayas.

1 Introduction

During the construction of linear structures, the shortest alignment for most of the case is tunnel alignment. Topography of Nepal is not favorable for easy construction. Tunnel being shortest and economic option compared to other option most of the hydropower projects adopt tunnel as water conveyance system. A study carried out by Water and Energy Commission Secretariat (WECS) indicates that more than 850 km of tunneling needs to be done to develop already planned hydropower potentials in Nepal. However, tunnel poses the possibility of various kind of challenges such as swelling, squeezing, spalling, rock burst etc. [1]. In tunneling through weak rock, the common challenge we face is tunnel squeezing in Nepal. It is a type of displacement into an excavation due to stress gradient created around the tunnel by excavation. 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. Good knowledge of stresses around underground excavation helps to mitigate or reduce the squeezing. Majority of tunnel in the design phase decision (for example selecting tunnel alignment and predicting the rock mass quality and rock support requirement) has direct influence on the overall cost and time requirement of any tunneling project. The past tunneling experience indicated that majority of the tunnel projects developed have had suffered severe stability problems that made delay in completion and cost overruns [2]. The main objective of this research work is to determine possibility of squeezing phenomenon in tunnel prior to construction and calculate necessary support system for the given condition.

2 Literature Review

A tunnel is an underground passageway, dug through the surrounding soil/earth/rock and enclosed except for entrance and exit, commonly at each end. Weak over-stressed rock mass would experience squeezing ground condition [3].

- *Season Maharjan is currently pursuing master's degree program in geo-technical engineering in Tribhuvan University, Nepal*
  E-mail: seasonmaharjan@gmail.com

It is very slow and hazardous process because the rock mass around the opening loses its inherent strength under the influence of in situ stresses. This may result in mobilization of high support pressure and tunnel closures. Rock squeezing is a time dependent process that typically occurs in weak over-stressed rock masses and that could have a significant and negative influence on the budget and time needed for successful completion of a tunneling project. Factors influencing squeezing of the tunnel are stress condition, strength and deformability of the rock mass, rock types, orientation of the geological structures and construction method and support system. Some of the squeezing problems are listen in TABLE 1.

2.1 Criteria for squeezing ground condition

Several research has been performed to determine squeezing potential of rock mass prior to construction. According to

TABLE 1: *Stability Problem Related To Squeezing In Nepal In Hydropower Projects*

Project	Geological rock types
Chameliya HEP	slate, phyllite, schists, quartzite, limestone, dolomite, etc.
Modi HEP	highly fractured quartzite and highly sheared and highly deformed phyllite green schist
Middle Marsyangadi HEP	quartzite, phyllite and meta-sandstone
Kali Gandaki HEP	Headrace tunnel mostly passes through highly deformed phyllite
Khimti HEP	banded gneiss and augen mica gneiss

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 [4]. Similarly Goel's (1994) approach expressed squeezing phenomena based on rock mass number, width of tunnel and height of overburden [5]. 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 shown in

Table 3. Likewise, semi-analytical approaches that are used for the analysis of tunnel squeezing phenomenon are Jethwa et al. (1984), Kovári (1998), Aydan et al. (1993), Hoek and Marinos (2000), etc. In this article, only Jethwa et al. approach (1984) is used which is a function of rock mass uniaxial compressive strength (UCS) and in-situ stress. Criteria for this approach is shown in

Table ***4***. For analytical approach, convergence confinement method (CCM) is used which gives detail estimate of stress and deformation. CCM is also used for support design. The simplicity of the method has led to its widespread use since the end of the seventies. The important hypotheses on which it is based are as follows:

1. Circular and deep tunnels (boundary conditions of the problem to infinity)
2. Lithostatic stresses of a hydrostatic type and constant in the surrounding medium of the tunnel (the variation of the stresses with depth due to the weight of the rock is neglected)
3. Continuous, homogeneous and isotropic rock mass
4. Bi-dimensional problem and plane stress field

For verifying the estimation and calculation, numerical modeling has been used. This is an approximate method and precision of this method depends on the permissible error adopted. For this purpose computer software $PHASE^2$ has been used.

2.2 Study area

The proposed study area lies Himalaya region in Ri VDC of Dhading district located in Baghmati zone, Central development region. Ankhu Khola Hydropower Project (AKHPP) is RoR type project with generating capacity 42.9MW. It lies within longitude 28°04'00"N to 28°07'00"N and latitude 84°58'35"E to 85°01'04"E. All related structures are located along the right bank of the Ankhu Khola on the alluvial and colluvial deposits as well on bedrocks. The project is in feasibility stage. Three types of rock types are found which are quartzite, phyllite and phyllitic schist. Headrace tunnel of AKHPP lies between phyllite and phyllitic schist which are weak rock and are prone to squeezing. This is one of the reason to choose this site for analysis.

3 Material and Methods

3.1 Data collection

The necessary data required to perform analysis are rock mass properties and tunnel dimension. Rock mass properties include Q-value, RMR, UCS of rock, Elasticity etc. and tunnel properties include alignment, shape, size, overburden height, length etc. These data were extracted from "Main feasibility report" of AKHPP which was obtained from Hydro Solutions Pvt. Ltd. [6].

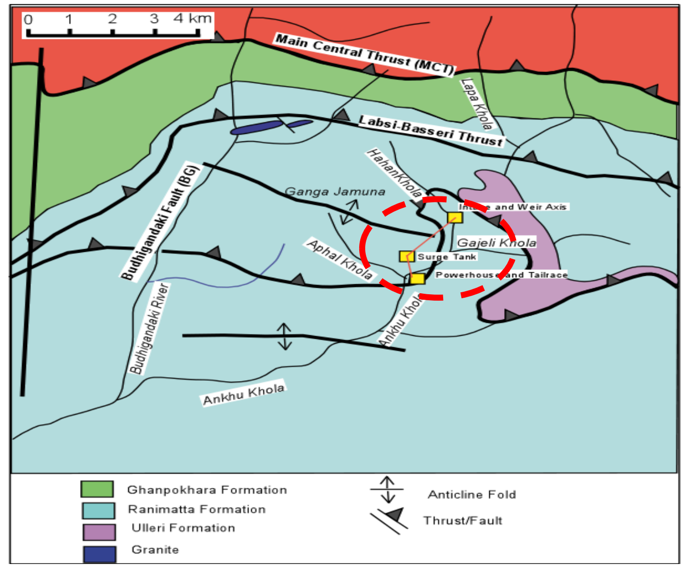

Figure 1: *Regional Geological Map of the Ankhu Khola Area (after DMG, 1987)*

Table 2: *Input Data Of Ankhu Khola Hydropower Project (42.9MW)*

Chainage m	Overburden m	RMR	Q
0+500	162.34	54	6.88
1+000	121.47	54	6.13
1+500	214.74	54	9.72
2+000	345.03	48	6.88
2+500	613.37	48	6.88
3+000	384.85	54	5.44
3+500	212.46	54	6.13
4+000	308.45	42	8.66
4+500	290.53	31	4.82
5+000	164.98	31	6.13
5+500	14.39	31	6.13

3.2 Empirical approach for squeezing

Singh's approach (1992):

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

And Goel's approach (1994):

$$H > (275\ N^{0.33})\ B^{-0.1}$$

Criteria for empirical approaches are given in Table 3.

Table 3: *Summary of criteria of Empirical approach*

Approach	Squeezing condition	Non squeezing condition
Singh's approach	$H > 350\ Q^{1/3}$	$H < 350\ Q^{1/3}$
Goel's approach	$H > (275\ N^{0.33})B^{-0.1}$	$H < (275\ N^{0.33})B^{-0.1}$

3.3 Semi-empirical approach

The degree of squeezing in this 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 [7]. The degree of squeezing is defined by Jethwa et al. (1984) on the basis of following relation:

$$Nc = \frac{\sigma cm}{Po} = \frac{\sigma cm}{\gamma H}$$

Where, σcm is rock mass uniaxial compressive strength, Po is in-situ stress, γ is unit weight of rock mass and h is tunnel depth below surface.

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

3.4 Convergence confinement method (CCM)

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). The detail of these components is explained further in this chapter [8].

Longitudinal Displacement Profile (LDP)
LDP 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. 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.

Ground Reaction Curve (GRC)
GRC is the relationship 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 (Carranza-Torres and Fairhurst, 2000).

Support Characteristics Curve (SCC)
Support characteristic Curve is the plot between increasing pressure Ps on the support and increasing radial displacement u_r of the support.

3.5 Calculation of available support

1. Available support for Concrete or Shotcrete Linings

The stiffness constant Kc is as follows:

$$\mathrm{Kc} = \frac{\mathrm{Ec}\{\mathrm{r_i^2} - (\mathrm{ri}-\mathrm{tc})^2\}}{(1+\mathrm{vc})\{1-2\mathrm{vc})\,\mathrm{r_i^2} + (\mathrm{ri}-\mathrm{tc})^2\}}$$

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.

$$\mathrm{pmax} = \sigma_{\mathrm{c.conc}}\left[1 - \frac{(\mathrm{ri}-\mathrm{tc})^2}{\mathrm{r_i^2}}\right]$$

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;

$$\mathrm{P_s^{max}} = \frac{\mathrm{Tbf}}{\mathrm{Sc\ Sl}}$$

And the stiffness is given by;

$$\frac{1}{\mathrm{Ks}} = \frac{\mathrm{Sc\ St}}{\mathrm{ri}}\left[\frac{4\mathrm{l}}{\pi \mathrm{d_b^2 Es}} + \mathrm{Q}\right]$$

Where,
db is the bolt or cable diameter [m]
l is the free length of bolt or cable [m]
Tbf 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)

$$\mathrm{P_s^{max}} = \frac{\mathrm{As}\ \sigma_{\mathrm{ys}}}{\mathrm{Sl\ R}}$$

And the stiffness is;

$$\mathrm{Ks} = \frac{\mathrm{EsAs}}{\mathrm{Sl.R^2}}$$

Where
σys 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 axis[m]
R is the radius of the tunnel [m]

4. Combined effect of support system

In this case, the stiffness of the combined system is determined as the sum of the stiffness of the individual components.

$$\mathrm{K} = \mathrm{K1} + \mathrm{K2}$$

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

3.6 PHASE^2 Modelling

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 which are at chainage 2+500m. The properties of rock mass for numerical modelling is adopted as far as practicable and more close to real values. The properties of rock mass was estimated using Geological Strength Index (GSI) and blast factor D from correlations [9]. 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.

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

$$\mathrm{s} = \exp\left(\frac{\mathrm{GSI}-100}{9-3\mathrm{D}}\right)$$

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

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 – 4.

TABLE 5: *INPUT VALUE FOR PHASE2 FOR NUMERICAL MODELLING.*

Rock mass properties	**Values**
Rock type	Phylllitic schist
Poisson's ratio	0.15
σci (MPa)	75
mi	10
GSI	48
Rock surface condition	Slightly rough
D	0.8
Ei (MPa)	14926.11
mb	1.093
s	0.00037
a	0.5066
Unit wt. (kN/m3)	27
Vertical stress, γH (MPa)	16.37

4 RESULTS AND DISCUSSION

From Figure 2, lowest factor of safety at chainage 2+500m which is less than one. Hence, it can be concluded that this section is susceptible to squeezing. Similarly, semi-empirical method also indicates that factor of safety is least for the section at chainage 2+500m (**Error! Reference source not found.**). Therefore, support should be designed for that section using analytical method (CCM). This method was used to determine stress and deformation at chainage 2+500m with 2.3m radius of tunnel. First elastic deformation was calculated which is 19mm. After that, plastic deformation is estimated. The maximum plastic deformation was found to be 92.45 mm. Deformation of tunnel is plotted in graph against internal pressure which is shown in **Error! Reference source not found.**. For the design of rock bolt, radius of plastic zone is determined using Phase 2 model. The plastic radius or radius of broken zone is 5.3m. Therefore, rock bolt of 25mm diameter and at least 3m long is required at 0.5 m center-to-center spacing. The bolt shall be install when tunnel is deformed by 30mm. But permissible deformation was not obtained with rock bolt only.

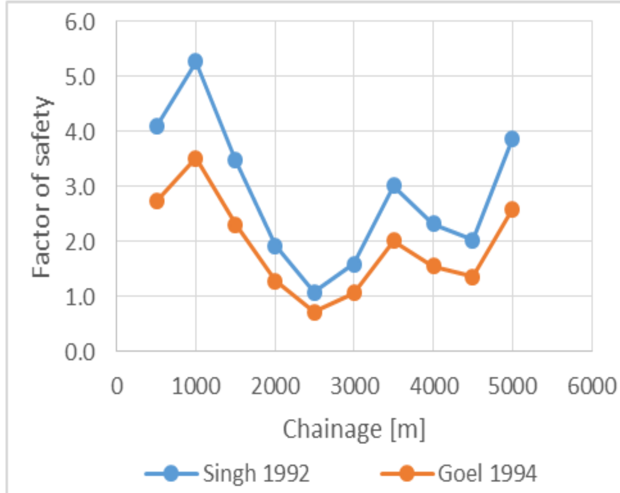

Figure 2 *Results showing Factor of safety from Singh's approach and Goel's approach.*

Shotcrete was used with rock bolt for deformation control. The required calculated shotcrete was 500mm but it would be too thick. So, 250mm thick shotcrete was used with grade of M40. In addition, steel rib was used having depth of 150mm i.e. ISMB150 should be used. Using composite support, final deformation was achieved as 35mm which is within permissible limit. The factor of safety of 4.35 was achieved which is shown in Figure 5.

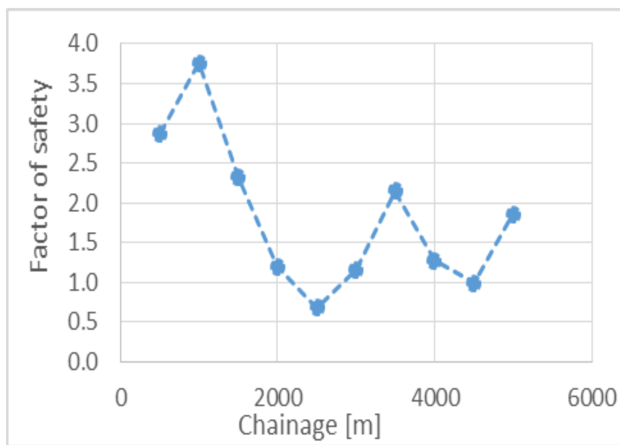

Figure 3: *Graph showing chainage against Factor of safety from Jethwa et al. approach*

TABLE 6: *SUMMARY OF CALCULATION OF HOEK'S CONSTANT USING GEOLOGICAL STRENGTH INDEX*

Chainage	2+500
Overburden	613.37
RMR	48
Q	6.88
GSI	43
m_b	1.093
s	0.00037
a	0.5066

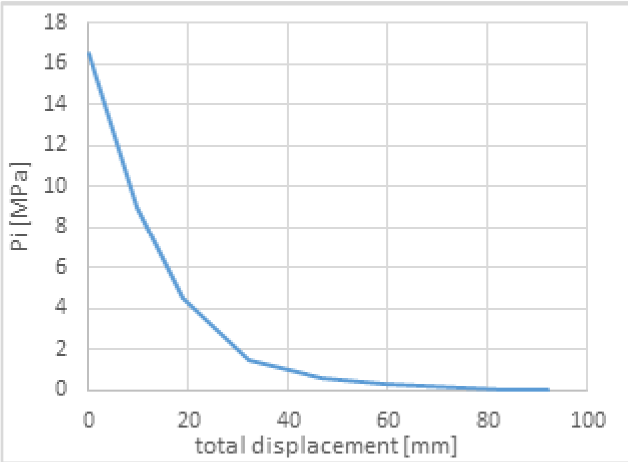

Figure 4 *Ground responsive curve for tunnel section at 2+500m*

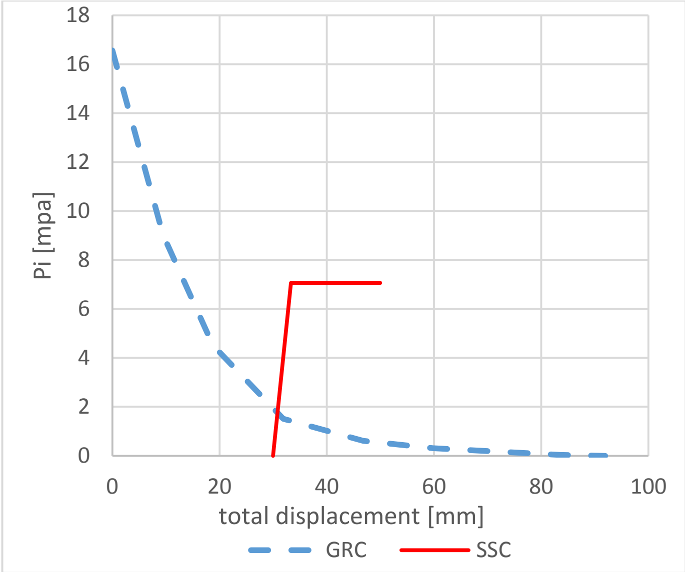

Figure 5: *Ground reaction curve (GRC) and Support characteristic curve (SCC) plotted in same graph to determine stability of tunnel section against squeezing.*

5 CONCLUSION
Empirical method and semi-empirical method of predicting squeezing rock condition gives similar result. The squeezing potential was observed at chainage 2+500m 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. The result from CCM suggests that to control displacement, composite support system should be used. The support system consist of 25mm diameter 3m long rock bolt at 0.5m c/c spacing along with 0.15m thick shotcrete of M40 and ISMB150 steel rib with factor of safety 4. The final displacement with and without support is 32.5mm and 115.63mm.

RECOMMENDATION
Some of the recommendation for further study are as follows:

- Sometimes tectonic stress is larger than in-situ stress. Therefore, it is important to study about magnitude of tectonic stress especially in Himalaya Range which is seismically active region.
- Variation in ground water condition has not been considered in this study. Thus, for better result, ground water table can be valuable parameter to assess squeezing ground.
- Here, support system is provided for squeezed section only. Thus, for economic design of tunnel, different category of support should be designed throughout the tunnel length.

ACKNOWLEDGEMENT
The author gives sincere thanks to Institute of Engineering, Tribhuwan University for the encouragement. Also thanks to the Hydro Solutions Pvt. Ltd. for their help in collecting relevant reports and information needed in writing this paper.

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