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Analysis of plastic deformation around an underground opening: A case study of Tanahun Hydropower Project

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Abstract

Various methods have been devised over the years to determine the amount of deformation and extent of the plastic zone around the underground opening and can be categorized into empirical, semi-empirical, and analytical methods. In addition to this, 2D finite element numerical models are also utilized to determine the plastic deformation and the extent of the plastic zone surrounding the underground opening. This study focuses on the analysis of plastic deformation using the aforementioned approaches and discusses their applicability in the context of Himalayan geology. The results of the analysis of plastic deformation around the headrace tunnel of the Tanahun Hydropower Project show that along with the use of semi-empirical and analytical approaches, numerical analysis should be used for the assessment of plastic deformation, especially in case of weak rock mass in high-stress conditions where the stability of rock ahead of the tunnel face is essential. The selection of input parameters plays a vital role in the outcome of the analysis. This study also reveals that it is possible to predict the extent of plastic deformation in the Himalayan rock mass by using different approaches through the correct identification of rock mass parameters.

Keywords

Deformation, Finite Element Analysis, Headrace tunnel, Plastic behaviour, Support Pressure

1. Introduction

Since more than 70% of the country lies in the hilly and mountainous region, there is a massive scope of underground construction in the hydropower, storage, and transportation sectors in Nepal. However, constructing underground structures in this region is a difficult task because the rock mass is relatively weak and highly deformed, schistose, weathered, and altered as a result of active tectonic movement and dynamic monsoons. The ongoing collision of the continents has resulted in several thrusts and faults in the Himalayas [1]. Therefore, it is paramount to develop a great deal of knowledge and expertise in the field of rock engineering to deal with these challenges. The stresses that previously existed in the rock mass are disturbed after the underground excavation and are redistributed in the local vicinity of the opening. This causes the concentration of stress in the rock mass surrounding the excavation [2]. In the case of weak rock mass, high-stress concentration around the underground opening causes plastic deformation if the redistributed tangential stress exceeds the strength of rock mass. The maximum stress value is located at some distance from the tunnel contour due to the formation of a plastic zone around the excavation. In the case of drill and blast tunnel, the damage caused by blasting also contributes to the formation of the plastic zone [3]. Adequate support systems must be designed considering the plastic deformation and extent of the plastic zone around the underground opening.

Various methods have been devised over the years to determine the amount of deformation and extent of the plastic zone around the underground opening and can be categorized into empirical, semi-empirical, and analytical methods [4]. In addition to this, numerical models are also utilized to determine the plastic deformation and the extent of the plastic zone surrounding the underground opening.

The main objectives of this study are:

i) To determine the magnitude of plastic deformation and extent of plastic zone in supported and unsupported conditions of the headrace tunnel using semi-analytical and analytical approaches.

ii) To use 2D Finite Element Modeling to assess the plastic deformation and provide necessary supports for its stabilization.

iii) To discuss the applicability of these various methods in the context of Himalayan geology.

2. Study Area

2.1 Tanahun Hydropower Project

Tanahun hydropower project is a 140-megawatt storage type hydroelectric project located in the upper part of the Seti River, (a tributary of the Trishuli River) flowing in the central part of Nepal about 150km west of Kathmandu. The Seti River originates at the Annapurna (7,555 m height above sea level) of the Himalaya Mountains and joins the Madi River 2 km downstream from the Dam site after flowing about from north to south. The length of the Seti River from the origin to the Dam site is about 120 km, and a catchment area at the Dam site is 1,502 km2.

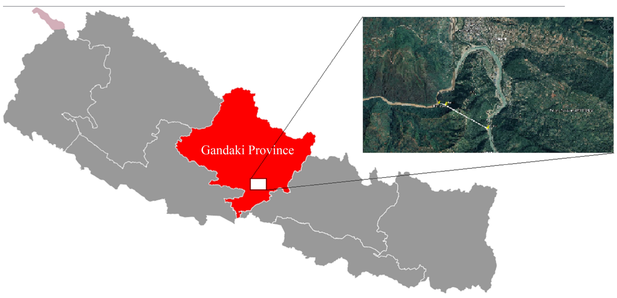

Figure 1: Location of study area

2.2 Geology of the study area

Among the five tectonic provinces of the Himalayan geology, the Tanahun Hydropower Project area is located in the Lesser Himalayan zone. The project area comprises of Norpul, Dhading Dolomite and Benighat Slate formations [5] as shown in Figure 2. Rock types along the tunnel alignment mainly consists of slate, phyllite, and dolomite as shown in Figure 4. Penstock and underground powerhouse site are founded in slate and dolomitic or siliceous slate. The slate layers strike almost parallel to the tunnel route and dip 45 to 50 degrees to the south [5]. Most of slate layers are moderately hard except for phyllitic sections due to faults etc., and dolomitic or siliceous portions are very hard. They are generally thin bedded and less than 10 cm in thickness. An approximately 80 m thick hard dolomite block is exposed at the outlet site. However, this dolomite becomes thin to the upstream and to the mountain side. These strata trend E-W to WNW-ESE, namely parallel to the Seti River, and dip southward as shown in Figure 3

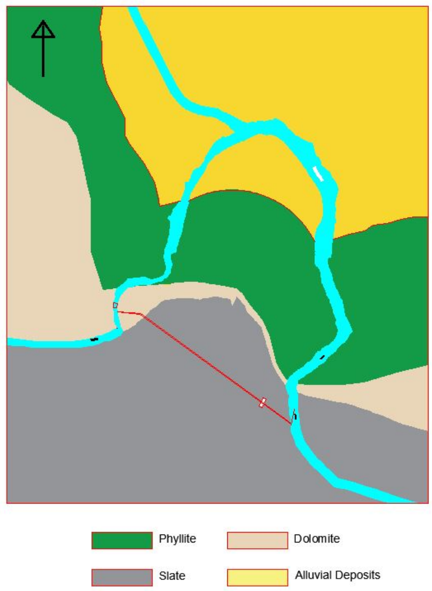

Figure 2: Geological map of the study area (modified after JICA [5]

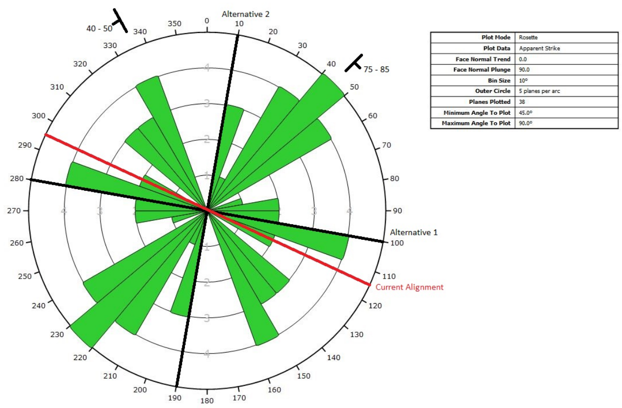

Figure 3: Rosette plot of joint mapping along the headrace tunnel

2.3 Headrace tunnel of Tanahun Hydropower Project

The waterway from the intake up to the underground Powerhouse consists of a 1430 m long horseshoe shaped headrace tunnel of 7.4 m diameter, a 28m high Headrace Surge Tank of 61m diameter, and a 160 m long Penstock ranging from 5.7 m to 4 m in diameter.

Water will be discharged to the Outlet via the Draft Tunnel of 100 m in length and a 248 m long Tailrace Tunnel of 7.4 m in diameter. The powerhouse is underground type with dimensions 89m(length) X 45m(Height) X 22m(Width).

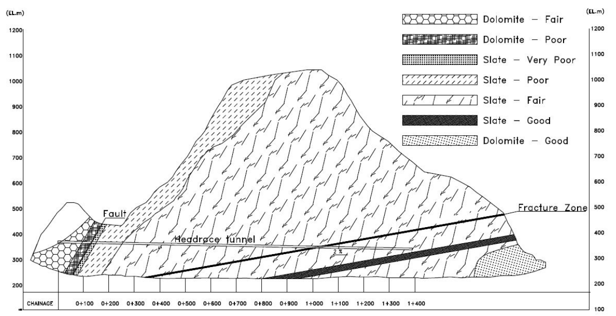

Figure 4: Geological profile of the head race tunnel (modified after NEA [6])

The planned intake is located in the right bank slope about 250 m upstream of the dam axis, and the headrace tunnel is excavated from the intake site for 1.5 km in WNW-ESE direction. The headrace tunnel crosses the ridge of EL.600m 1,000m, and the rock cover of the headrace tunnel is 200 m 690 m and that of the underground powerhouse is about 300 m.

3. Analysis of Plastic Deformation

Various Empirical approaches are commonly used for quAlitative analysis of plastic deformation surrounding an underground opening. Hoek and Marinos [7] and Panthi and Shrestha [8] are two useful semi-empirical techniques. Analytical methods such as the Convergence Confinement Method (CCM) [9] and numerical methods using finite element software such as Rocscience are also highly effective for quantifying squeezing around tunnel contours. Plastic deformation is analysed using the semi-empirical, analytical and numerical approach in this study. The inputs for these methods are rock mass parameters and rock stresses.

3.1 Selection of sections for analysis

The head race tunnel of Tanahun hydropower project lies mostly in thinly bedded slate of fair (RMR=41-60) and poor (RMR=21-40) rock mass quality. Based on the rock mass quality, sections at chainage 1+060 and 1+080 are selected for study for representing the two-rock mass class. In addition to this, the section with maximum overburden i.e., 691m at chainage 1+035, the section with minimum overburden i.e., 65m at chainage 0+235 and the section at which a transition zone between slate and dolomite of very poor rock mass (RMR = 10-20) is likely to intersect the tunnel at chainage 0+145 are also considered for this study.

3.2 Determination of rock mass parameters

The headrace tunnel of Tanahun Hydropower Project mostly passes through slate of poor and fair quality based on RMR Classification of rock mass. The rock mass quality in terms of RMR, GSI and Q values are summarized on Table 1.

Table 1: Summary of determination of rock mass quality using various empirical approaches

Chainage	RMR	GSI	Remarks
1+080	50	45	Fair
1+060	35	30	Poor

The rock properties are determined from reports of various laboratory testing of rock samples like the Uniaxial Compression Test (UCS) and in situ testing of rock mass using Block Shear Test and Plate Bearing Test provided by the Project and the results obtained are summarized in Table 2.

Table 2: Intact rock properties from laboratory testing

Rock mass quality	Fair	Poor
Density, γ (gm/cc)	2.77	2.7
UCS of intact rock, σ_{ci} (MPa)	62	25
Modulus of Elasticity, Ei (GPa)	30	22
Poisson's Ratio, μ	0.3	0.35
Friction Angle, ϕ	39	32
Cohesion, c	1.7	0.8

From hydro-fracturing and Diametrical core deformation analysis (DCDA) carried out by NEA [6], the maximum and minimum in situ stresses have been determined at the powerhouse location of Tanahun Hydropower Project. Based on this data various in plane and out of plane stresses with respect to the tunnel alignment have been determined as shown in Table 3 .

Table 3: Summary of stress calculations

Vertical Stress (σ_v)	9.80
Density (γ)	2.72
Poisson's Ratio (μ)	0.30
Trend of Tunnel (θ)	126.30
Minimum horizontal stress (σ_h)	2.20
Maximum horizontal stress (σ_H)	10.40
Angle between minimum horizontal stress and out of plane stress (α)	57.70
In plane horizontal stress ($\sigma_{h,in}$)	4.54
Out of plane horizontal stress ($\sigma_{h,out}$)	8.06
Horizontal stress due to gravity only ($\sigma_{h,gravity}$)	4.20
In plane horizontal /Vertical stress ($\sigma_{h,in}/\sigma_v$)	0.46
Out of plane horizontal/Vertical stress ($\sigma_{h,out}/\sigma_v$)	0.82
Locked in stress (In plane)	0.34
Locked in stress (out of plane)	3.86

3.3 Semi-empirical Approach

The semi-empirical procedures presented below provide indicators for forecasting squeezing as well as some techniques for predicting the expected deformation surrounding the tunnel opening.

3.3.1 Hoek and Marinos Approach (2000)

The ratio of rock mass uniaxial compressive strength σ_{cm} to in situ stress p_0 was generally used as an indication of probable tunnel squeezing difficulties. Hoek and Marinos [7] demonstrated, in particular, that a plot of tunnel strain ε_t versus the ratio σ_{cm}/p_0 may be utilized successfully to identify tunnelling issues under squeezing conditions. Plastic deformation or strain can be calculated using [7] as shown in equation (1).

$$\varepsilon = \frac{\delta_i}{d_0} = \left(0.002 - 0.0025\frac{p_i}{p_0}\right)\left(\frac{\sigma_{cm}}{p_0}\right)^{2.4\left(\frac{p_i}{p_0}\right)-2} \quad (1)$$

And the size of the plastic zone can be determined using equation (2).

$$\frac{d_p}{d_0} = \left(1.25 - 0.625\left(\frac{p_i}{p_0}\right)\right)\left(\frac{\sigma_{cm}}{p_0}\right)^{\frac{p_i}{p_0}-0.57} \quad (2)$$

The strain has been calculated for the selected sections in unsupported and supported conditions and the result has been plotted as shown in Figure 5.

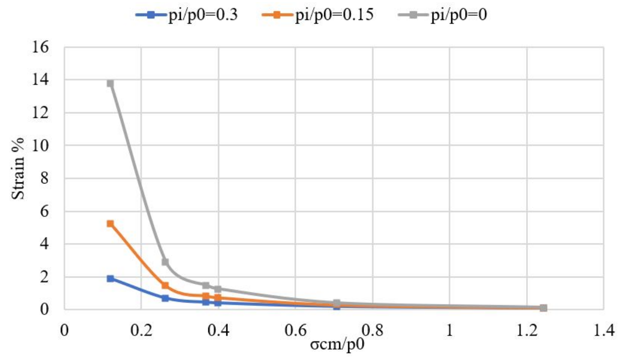

Figure 5: Estimation of tunnel strain using [7]

The diameter of plastic zone was calculated using equation (2) to be 16.3m for fair quality rock mass and 30m for poor quality rocks under simillar loading conditions in case of headrace tunnel of THP according to Hoek and Marinos [7].

3.3.2 Panthi and Shrestha Approach (2015)

Panthi and Shrestha [8] investigated the long-term squeezing phenomena of three separate hydropower tunnels in Nepal's Himalaya and discovered a link between time independent and time dependent strain using convergence equations. According to Panthi and Shrestha [8], total plastic deformation in tunnels passing through schistose and weak rock mass comprises of both time-independent and time-dependent deformations. Panthi and Shrestha [8] established a relationship between tunnel strain (instantaneous and total tunnel strain), vertical gravitational stress (σ_v), horizontal to vertical stress ratio (k), support pressure (p_i), and rock mass shear modulus (G). The instantaneous closure (ε_{inst}) and final closure (ε_{final}) values are directly related to the in-situ stress conditions and indirectly proportional to the rock mass shear modulus and support pressure values and are given by equation (3) and (4).

$$\varepsilon_{\text{inst}} = 3065 * \left(\frac{\sigma_v * \left(1+\frac{k}{2}\right)}{2G\left(1+p_i\right)}\right)^{2.13} \quad (3)$$

$$\varepsilon_{\text{final}} = 4509 * \left(\frac{\sigma_v * \left(1+\frac{k}{2}\right)}{2G\left(1+p_i\right)}\right)^{2.09} \quad (4)$$

The instantaneous and final tunnel closure is calculated for the selected sections in supported and unsupported conditions as shown in Figure 6.

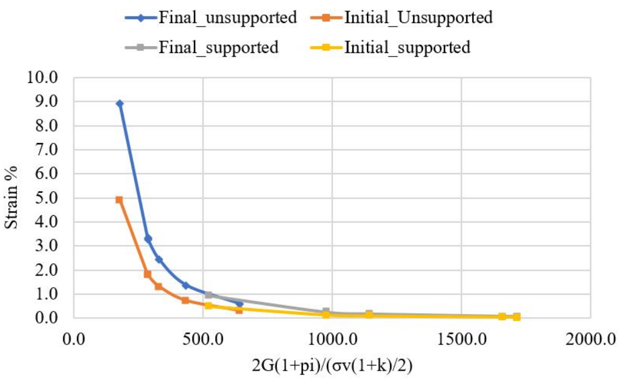

Figure 6: Estimation of tunnel strain using [8]

3.4 Analytical Approach

Estimation of the support required to stabilize a tunnel excavation, especially in the vicinity of the face, is essentially a four-dimensional problem. Time dependent weakening of the rock compounds the three-dimensional redistribution of forces around the excavation, and the nature of the rock is uncertain until it is exposed in the face [9].

3.4.1 Convergence Confinement Method (CCM)

CCM is an analysis tool that provides rock engineers a simplified appreciation of the nature of the interplay between the (variable) rock-mass and the installed support, and the effect of variation in assumed rock properties on the support loads [9].The three basic components of CCM are,

Longitudinal Deformation Profile (LDP) LDP is a graphical depiction of radial displacement that happens along an unsupported circular excavation axis ahead and behind the tunnel face.

Ground Reaction Curve (GRC) GRC shows the relation between decreasing internal pressure (pi) and increasing radial deformation of wall (ur).

Support Characteristic Curve (SCC) The SCC is defined as the interaction between rising pressure ps on the support and increasing radial displacement ur of the support.

3.4.2 Results of CCM

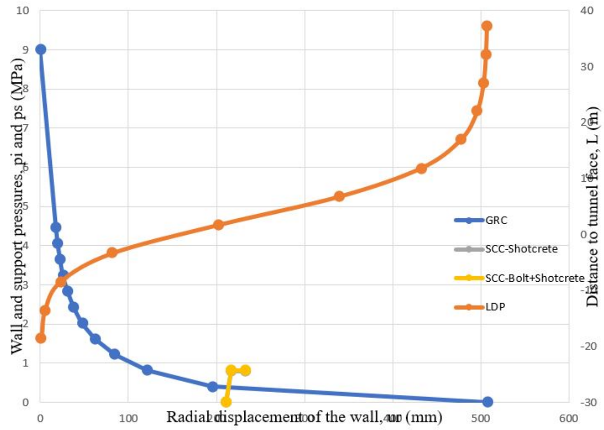

Figure 7: Rock support interaction analysis for poor rock mass at Ch. 1+060

From Figue 7 and 8 the maximum possible deformation for poor quality rock mass as determined from the rock support interacton analysis was about 500 mm which was controlled at 200 mm and for fair quality rock mass was 27 mm and controlled at 11 mm by the application of bolts and shotcrete at 2 m behind the face of the tunnel.

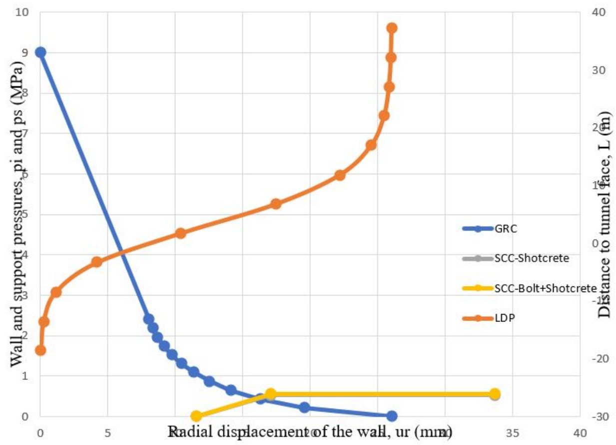

Figure 8: Rock support interaction analysis for fair rock mass at Ch. 1+080

3.5 Numerical Modeling

3.5.1 Model setup

Cross sectional model of the tunnel is created in RS2. The external boundaries of the models are placed with sufficient distance from the tunnel, and restrained in both X and Y direction. Continuum modelling is carried out where the rock material is regarded as one homogeneous mass as shown in Figure 9.

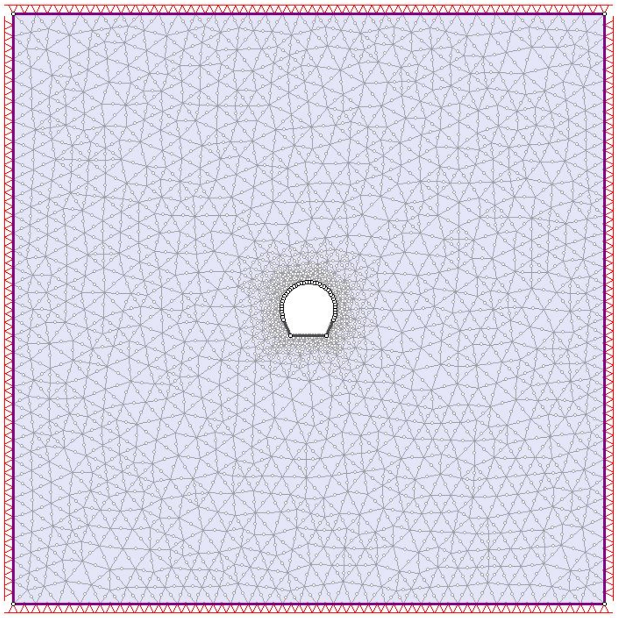

Figure 9: Continuum model geometry in Phase2

3.5.2 Input parameters

The loading has been determined by resolving the measured principal stress in the direction and perpendicular to the direction of tunnel alignment. The material properties are obtained from laboratory testing report provided by Tanahun Hydropower Project. The Hoek-Brown parameters for the Hoek-Brown failure criteria [10] are determined using D = 0.8 for Drill & Blast tunnel and m_i = 9 for slate. The support properties are used in agreement with the actual supports used in the project.

Table 4: Material properties and stress state

Loading	
Major principal stress	9.8
Minor principal stress	4.54
Intermediate principal stress	8.6
Angle from horizontal to σ_1	90
Material properties	
Poisson's ratio	0.3
Young's modulus, MPa	20000
mi	9
UCS, MPa	62
s	0.002
mb	0.34
a	0.508
Unit weight, gm/cc	2.7
Material Type	Plastic
Failure criterion	H-B

The input parameters used for numerical modelling in Phase2 are summarized in Table 4.

3.5.3 Model Validation

The deformation from the model after the installation of supports (Stage 7) was compared to the monitoring data because the monitoring devices were placed at the section after the installation of supports. The results are shown in Figure 10. This validated model is then utilized for further numerical analysis.

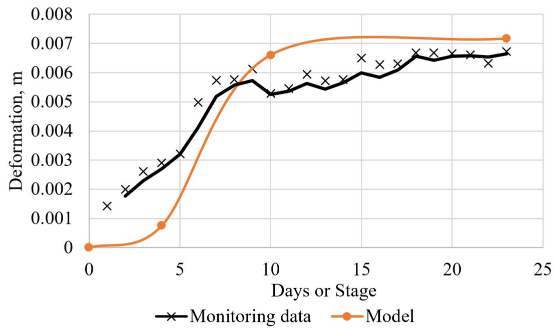

Figure 10: Comparison between model and in-situ deformation

3.5.4 Results of numerical modeling

Analysis of the model in elastic condition reveals the strength factor to be less than 1 around the excavation contour as shown in Figure 11. This means that further analysis should be done in plastic condition.

The results of plastic analysis in unsupported condition shows that the maximum deformation is about 220 mm as shown in Figure 13. Plastic zone is formed upto 7 m from tunnel contour all around the tunnel as shown in Figure 12.

In order to stablize the large plastic zone formed around the excavation, adequate support system needs to be designed. The supports obtained empirically from Q-chart were found to be inadequate to adress the plastic deformation in high overburden condition. Hence, 30 cm thick reinforced concrete lining (6 mm diameter rebars with 20cm spacing) as final lining and 20 cm shotcrete with steel ribs (ISMB 150*18) in 1m spacing as initial lining has been considered. The length of the bolts are increased to 6m to address the large plastic zone surrounding the excavation. The results of analysis using improved support is shown in Figure 15. The final maximum deformation has been reduced to about 90 mm and the radius of plastic zone is decreased to 9m.

RCC capacity envelopes in figure above show that the 30 cm thick concrete lining is found to be adequate as verified in Figure 16. The maximum strain in bolts is also less than 1% as shown. Hence this improved support system is recommended for the headrace tunnel in very poor-quality rock mass at high overburden.

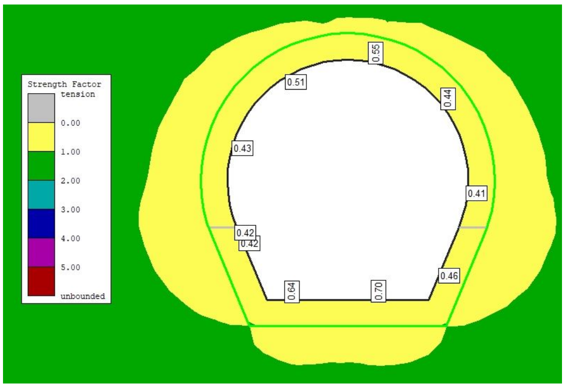

Figure 11: Strength factor in elastic analysis

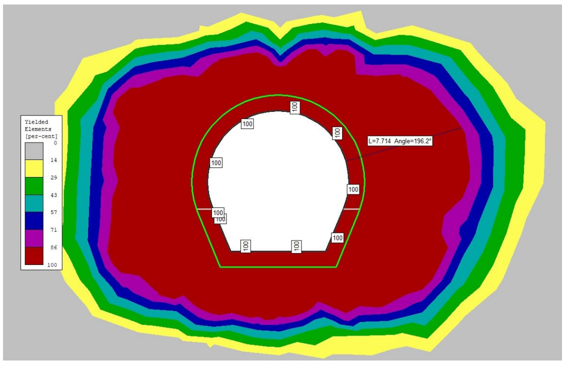

Figure 12: Extent of plastic zone surrounding the tunnel contour in unsupported condition

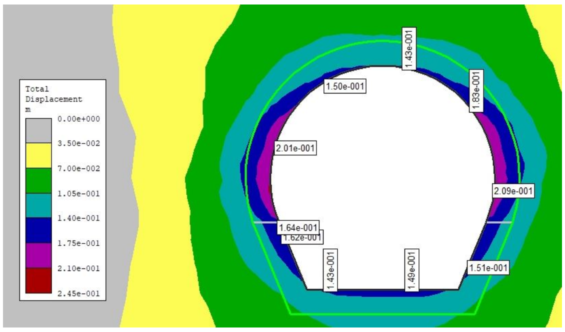

Figure 13: Deformation around the tunnel contour in plastic analysis under unsupported condition

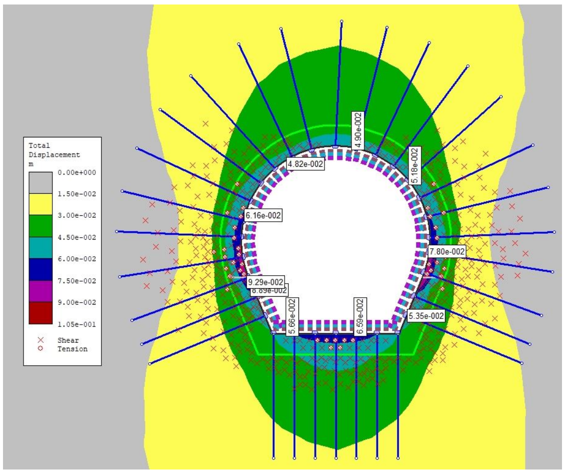

Figure 14: Deformation around the tunnel contour in plastic analysis under supported condition

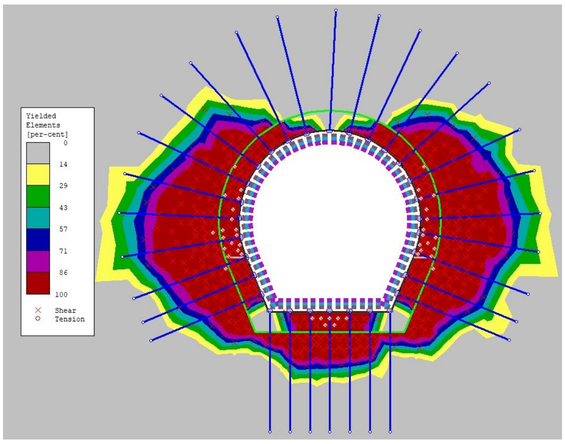

Figure 15: Extent of plastic zone surrounding the tunnel contour in unsupported condition

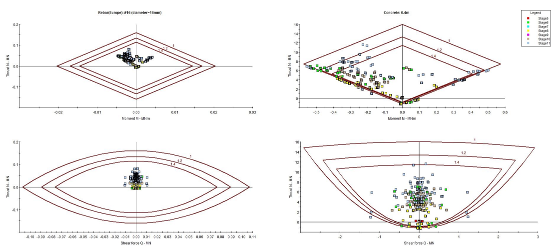

Figure 16: Support capacity plot for rebar (left) and concrete (right)

4. Discussion on the results

Semi empirical method by Hoek and Marinos [7] is developed for a circular tunnel with limited section size. In addition to that the method depends on how well the quality of rock mass estimation has been carried out. Panthi and Shrestha [8] is applicable to all types of tunnels irrespective of shape. In addition to that, the method has been devised based on various cases in hydropower tunnels in Himalayan region. So, it seems to represent the Himalayan rock mass more accurately.

In case of the results of CCM, the method has many drawbacks such as it is devised for circular tunnels under hydrostatic stress. The headrace tunnel of THP is a horse shoe shaped tunnel and the stress field surrounding the tunnel are anisotropic. In addition to this, the time-dependency of plastic deformation is completely neglected in CCM. However, its importance in analysis of plastic deformation cannot be undermined as it reinforces the understanding of mechanisms behind the plastic deformation.

Estimation of input parameters present quite a challenge during the analysis of plastic deformation using Phase2. The results of the analysis are applicable as long as the input parameters correctly represent the actual ground conditions.

5. Conclusions

i) The maximum tunnel strain according to Hoek and Marinos [7] was 14 % and according to Panthi and Shrestha [8] was 9 % in unsupported conditions. When the ratio of internal support pressure and the hydrostatic stress acting on the rock mass was taken as 0.15, the maximum tunnel strain was reduced to 5.2 % in case of Hoek and Marinos [7] and to 0.6 % in case of Panthi and Shrestha [8]. The radius of plastic zone was calculated to be about 16m for fair quality rock mass and 19m for poor quality rock mass.

ii) The results of rock support interaction analysis using CCM shows that maximum tunnel strain in unsupported condition in poor rock mass was 6.75 % and that in fair rock mass was 0.3 %. Analysis results of SCC show that in both the fair and poor section the applied support pressure is greater than demand; hence, confirming that the support system is enough to withstand plastic deformation.

iii) The analysis of the section at Ch. 1+060 in plastic condition shows the deformation was 220mm in unsupported condition which was limited to 90mm after the application of adequate supports. The extent of plastic zone was also decreased from 13m to 9m. The results from numerical simulations are in agreement with the deformation monitoring data. This proves that these FEM programs can be used in designing tunnels in weak rock masses in the Himalayan region as long as the input parameters are precisely and carefully determined.

Acknowledgments

The authors would like to express their gratitude to Nepal Electricity Authority (NEA) and Tanahun Hydropower Limited (THL) for providing required data and access needed to complete this study.

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