ISSN No:-2456-2165

Tunneling in Weak Rock Mass - an Evaluation on Stability Condition of Headrace Tunnel of Setikhola Hydropower Project, 22MW

Bikram Bhusal[1], Dr. Chhatra Bahadur Basnet[2], Ghan Bahadur Shrestha[3], Samrat Poudel[4]
[1]MSc student, Rock and Tunnel Engineering, Department of Civil Engineering, Pashchimanchal Campus, Institute of Engineering, Tribhuvan University, Nepal;
[2]Visiting Associate Professor, Pashchimanchal Campus, Institute of Engineering, Tribhuvan University, Nepal;
[3] Senior Geologist, Clean Energy Consultant Pvt. Ltd., Kathmandu, Nepal;
[4] Assistant Professor, Pashchimanchal Campus, Institute of Engineering, Tribhuvan University, Nepal;

Abstract:- The upstream section of the headrace tunnel (HRT) of the Setikhola Hydropower Project passes through calcareous and silicious phyllite with scant foliation which is weak and deformable rock mass. Bands of quartzite and metasandstone are intercalated locally within phyllite. These weak rocks may undergo plastic deformation. In such rock mass, there is a high chance that the tunnel might experience squeezing. Approximately 92 percent of the tunnel alignment has an overburden greater than 100 meters, with a maximum overburden above the tunnel of more than 500 meters. The portion from chainage 1+100 to 1+600 m along the tunnel alignment due to substantial overburden and intercalation of silicious phyllite and meta-sandstone, is critical in terms of tunnel squeezing. The tunnel at this section is analyzed for squeezing phenomenon using empirical techniques such as Singh et al. (1992), Goel et al. (1995), semi-analytical techniques such as Hoek and Marinos (2000), and Shrestha and Panthi (2015), analytical techniques such as the Convergence Confinement Method (Carranza Torres and Fairhurst, 2000) as well as numerical programs such as Phase^2 and RS^3. According to the empirical and semi-analytical squeezing prediction criteria, there is high chance of significant squeezing particularly in the selected headrace segment at chainage 1+580 m. The numerical analysis was carried out at the section where maximum deformation was anticipated from the prediction criteria. Numerical model also shows considerable deformation at this section to cause heavy squeezing. The support system estimated primarily using empirical methods are applied in the numerical modeling. The support system is inadequate because the model shows considerable deformation even after the application of the support and also failure occurs at the support. To overcome this effect either a heavy support system needs to be applied, reducing the pull length and adopting top heading and benching tunneling method or strengthen the rock mass by pre-injection grouting.

Keywords:- *Weak And Deformable Rock Mass, Stability Analysis, Empirical Relations, Semi-Analytical Technique, Analytical Technique, Numerical Modeling, Support System.*

I. INTRODUCTION

The use of underground spaces and tunnels has been increasing year by year. In the context of Nepal, we are very familiar with hydropower tunnels. However, it is a very challenging task, especially in the case of a country like Nepal with weak, sheared, fractured and deformed rock mass. Tunneling work is directly related to the geology of the tunneling site and also depends upon other non-geological parameters such as construction technology, available manpower, etc. So, to effectively carry out the construction of the underground structures, a detailed study of the geology of the area is very important. Nepal is slowly entering the tunneling era, and the planning and construction of road tunnels and hydro tunnels are also increasing. The complex geology and ongoing tectonic activity of the Himalayan region, however, have increased geological uncertainty and greatly impacted the stability of tunnels and underground caverns (Panthi, 2006). Tunnel squeezing is a common occurrence in the Himalayan rock mass with a high degree of schistocity, fracturing and deformability (Basnet, 2013).

A rock is a heterogeneous substance made up of solid aggregates of one or more minerals that occur naturally in smaller and larger blocks or chunks. The physical characteristics of these minerals vary greatly from one another. Additionally, the size, shape, orientation, and binding forces of the minerals have a significant impact on the physical and mechanical properties of rocks (Chaudhary, 2022). These two characteristics are extremely interdependent and play a crucial role in the stability of the tunnel, as seen in figure 1.

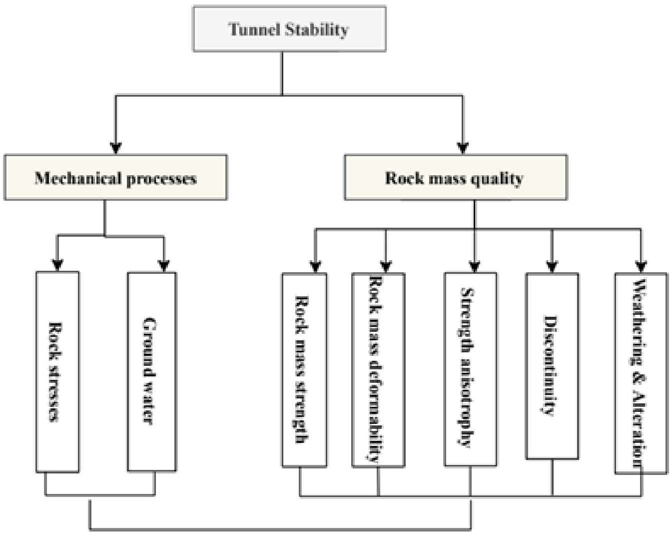

Fig1: Factors influencing tunnel stability (Shrestha, 2020)

A rock mass's natural distribution of structural elements is referred to as its structure. Folds, faults, joints, shear zones, and dykes are a few examples of structural characteristics. The existence of structural characteristics inside the rock mass controls its qualities (Brady & Brown, 2007). The weakest part of a rock mass is its joints, which are the most common structural element within it. Joint systems are formed when two parallel joint sets intersect. The most unfavorable joint has rough, uneven, and planner-type joint surface conditions, with soft infilling material over 5mm and a persistence range of 3-10m. The characteristics of a rock mass are influenced by various factors such as the composition of accompanying minerals, the quality of intact rock, the presence of discontinuities and joints, groundwater conditions, and the in-situ stress. In rock engineering, the strength and elastic characteristics of the rock are crucial and it can be determined in Laboratory or field test.

II. PROJECT LOCATION AND GEOLOGY

The Nepal Himalaya can be divided into five east-west trending major tectonic zones based on faults, thrusts, rock type, and age. These are the Tibetan Tethys Zone, Terai, Siwalik, Lesser Himalaya, and Higher Himalaya. The project site is located in Lesser Himalaya region. Geographical coordinates of project site are Latitude: 28°05'00"N to 28°08'05"N and Longitude: 84°04'15"E to 84°05'28". This tunnel has been proposed with a length 2940 m and an intake and powerhouse site easily accessible from Prithivi Highway (Setikhola Hydropower Project, 2018). The upstream section of the headrace tunnel (HRT) of the Setikhola Hydropower Project passes through calcareous and silicious phyllite with scant foliation which is weak and deformable rock mass shown in figure 3. Bands of quartzite and metasandstone are intercalated with the local phyllite. The aim of this paper is to perform stability analysis and rock support design using Empirical, Semi- Analytical, Analytical and Numerical Modelling and to predict the suitable tunneling methods in case of weak/squeezing rock mass and difficult ground conditions.

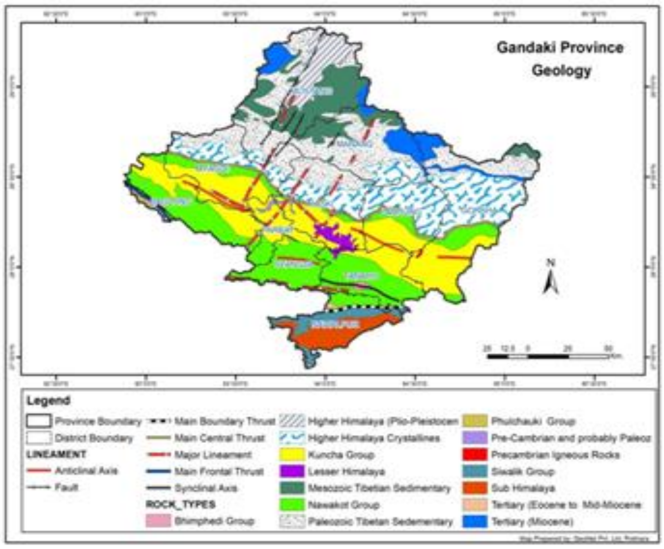

Fig2: Geological map of Gandaki

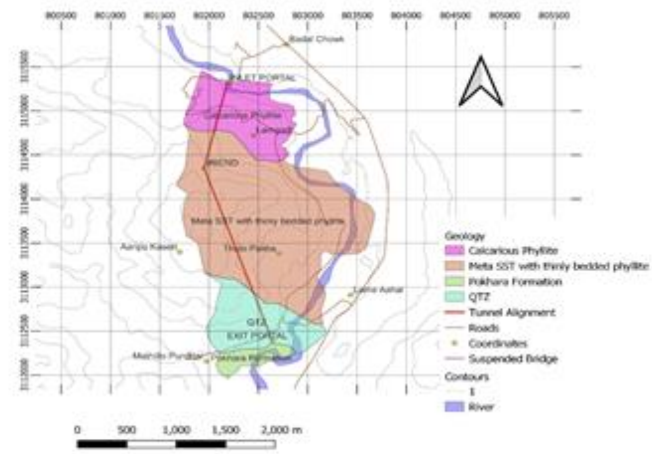

Fig 3: Geological Map of Project site

III. METHODOLOGY

Empirical, semi-empirical, analytical, and numerical methods have all been used in the analysis and design of SHEP's headrace tunnel. Initially by adopting empirical and semi-empirical methods, critical length (some stretch from throughout of the tunnel length) has been predicted. The silicious phyllite rock mass has shown the most deformation, hence this section has been chosen and produced as Phase^2 file utilizing RS^2 section creation tools for further study and tunnel design. The tunnel's design and analysis were done using the convergence confinement method, and a plot of the GRC, LDP, and SCC curve at the section has been made. Further the section exported from RS^3 is further analyzed in Phase^2 program and found that the similar safety factor as CCM generates. The topography surface of the stretch has been extracted as .stl file using civil 3D. The topography surface has been imported in RS^3 program from Rocscience package and analysis of the stretch has been performed. And hence, finally results and conclusions have been withdrawn. This overall procedures/methodology can be summarized in the following flow chart. The figure 4 shows the methodology steps followed during study.

ISSN No:-2456-2165

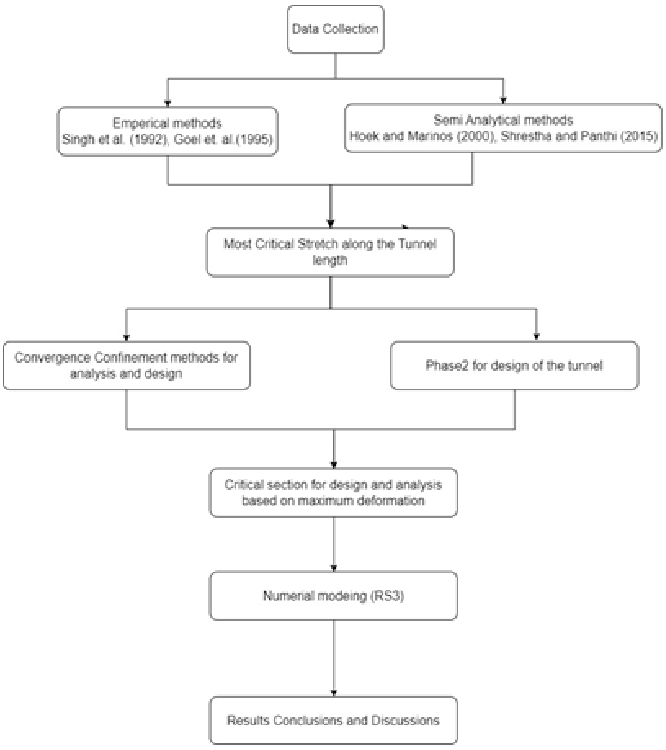

Fig 4: Flow Chart of Thesis Work

To achieve the above-mentioned objective, headrace tunnel of Setikhola Hydropower Project, Kaski has been selected for the estimation of the input parameters during numerical modeling. The methodology steps followed during the study is summarized below.

A. Literature Review:

- Background theories on the rock mass properties.
- Literature on Stresses and instability in tunnel.
- Literature on instability evaluation methodologies.

B. Data Collection

The collected data consists of rock mass properties from the lab test report, and face mapping of the headrace tunnel. These data have been collected from Setikhola Hydropower Company Pvt. Ltd. Kathmandu. Some primary data has been collected during the field visit, and based on a variety of published material, the remaining information has been hypothesized.

C. Input Parameters

Parametric analysis has been performed to estimate the input parameters for numerical modeling. With reference to the different available literature, density of rock, rock mass strength, and deformation modulus of the rock mass have been estimated.

D. Numerical modeling

Phase^2 and RS^3, a well-known and useful rocscience program that can be used for the investigation of subterranean or surface excavation in rock mass or soil, have performed numerical analysis of the models for stability. Basically, model, compute and interpret are the three fundamental program modules available in the Phase^2 and RS^3 program. The detail of the numerical modeling process is available in the later chapter.

E. Interpretation and Report Preparation

Lastly, the information generated from the interpret module has been analyzed based on the total strain generated at the excavated surface. Based on these results final report will be prepared.

IV. RESULT AND DISCUSSION

The upstream part of the headrace tunnel of Setikhola Hydropower Project passes through weak and deformable rock mass. Plastic deformation may take place in these fragile rocks. From figure 5, the largest overburden above the tunnel is about 500 meters high, and more than 92 percent of the tunnel alignment has overburden that is larger than 100 meters high. Quartzite, phyllite, and meta sandstone with intercalation features are the main types of rocks in this project, as shown in the figure. The downstream portion of the headrace tunnel is situated in quartzite that is of fair to good quality and is highly to moderately weathered, fractured, and foliated. The remaining upstream segment generally passes through thinly foliated calcareous and siliceous phyllite. The phyllite of the area is intercalated with bands of quartzite and meta-sandstone. Due to substantial overburden and the intercalation of two rock types, namely silicious phyllite and meta-sandstone, the section of the tunnel alignment from chainage 1+100 to 1+600 is critical in terms of tunnel squeezing. Thus this section has been chosen for assessment in this work.

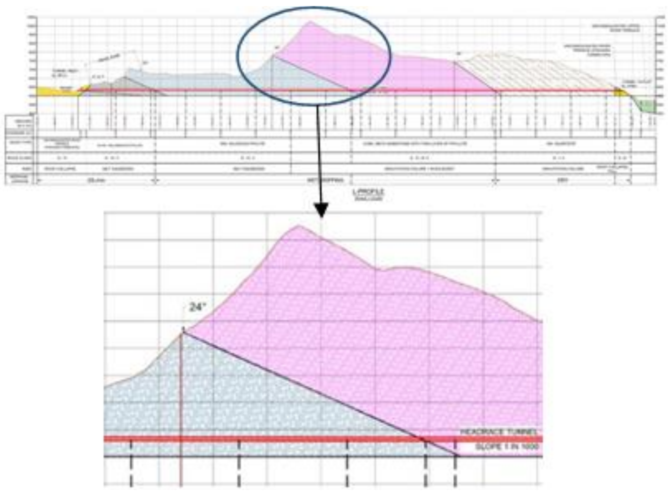

Fig 5: Portion Selected for Analysis

❖ *Squeezing Analysis*

In this paper work, squeezing analysis has been done in two stages. Squeezing problems have been predicted utilizing empirical techniques in the first stage by using Singh et al. (1992) and Goel et al. (1995) approaches. Using the methods of Hoek and Marinos (2000), Shrestha and Panthi (2015), Carranza-Torres and Fairhurst (2000), and numerical modeling, a second stage analysis with greater detail was conducted in the areas where there was possibility of substantial squeezing problem.

ISSN No:-2456-2165

A. Squeezing Prediction Criteria

The squeezing phenomenon in the SHEP headrace tunnel has been predicted using above mentioned technique.

Table 1: Deformation from different method

Rock Type		Silicious Phyllite	Silicious Phyllite	Meta sandstone	Meta sandstone
Overburden depth, m		100	330	170	330
Q		0.37	0.37	1.11	1.11
Span of the Tunnel		6	6	6	6
SRF		5	5	10	10
Singh et al (1992)	Limiting value of H, m	42.92	42.92	130.37	130.37
	Squeezing condition	YES	YES	YES	YES
Goel et al., 1995	Limiting value of H, m	275.98	275.98	500.58	500.58
	Squeezing condition	NO	YES	NO	NO
Hoek and Marinos (2000)	Strain % without support, ε	1.14%	12.41%	0.34%	1.26%
	Squeezing condition	Minor squeezing problem	Extreme squeezing problem	Few Support	Minor squeezing problem
Shrestha and Panthi (2015) approach	Short term deformation (m)	0.638	2.26	0.378	0.794
	Long term deformation (m)	1.13	3.94	0.681	1.41

The analysis shows the mixed result for the different sections at different chainage. Nonetheless, the outcomes for silicious phyllite at chainage 1+580 m with overburden 330m are essentially same. According to Singh et al. (1992) and Goel et al. (1995), there will be squeezing in this section. Hoek and Marinos (2000) predict that there will be a severe squeezing issue. The short-term deformation and long-term deformation, according to Shrestha and Panthi (2015) method, are 2.26 and 3.94 m, respectively. Therefore, the section at chainage 1+580 m portion is critical from squeezing point of view. This region was therefore selected further for analytical as well as numerical examination.

B. Convergence Confinement Method

The convergence confinement method (CCM) is an analytical solution in which rock mass and support interaction can be understood using three basic components. This technique provides guidance on where to put supports and how much support pressure to apply to keep deformation within the limit. The Ground Reaction Curve (GRC), Longitudinal Deformation Profiles (LDP), and Support Characteristics Curve (SCC) are the three main components of the CCM. An analysis has been done to predict the tunnel deformation under in the case of zero internal support pressure using the estimated input values. Since circular tunnels with hydrostatic far field stress conditions are the only ones for which CCM is relevant, the size of an inverted D tunnel has been utilized as a stand-in for a circular tunnel. For the calculation, the excel sheet formula proposed by (Carranza-Torres & Fairhurst, 2000) has been incorporated and the following parameters were used.

Table 2: Properties of support used in CCM calculation

Beam		Reinforced Shotcrete	
Young's Modulus (MPa)	30000	Young's Modulus (MPa)	200000
Poisson's Ratio	0.2	Section Depth (m)	0.162
Area (m2)	0.5	Thickness (m)	0.2
		Poisson's Ratio (concrete)	0.15

Shotcrete or Concrete		Bolt	
σcc [Mpa]	25	d_b [mm]	25
Ec [GPA]	20	L [m]	3
νc	0.15	T_{bf} [MN]	0.1
tc [mm]	20	Q [m/MN]	0.341
		E_B [GPA]	200
		n_{bolt}	10
		s_t [m]	0.5

ISSN No:-2456-2165

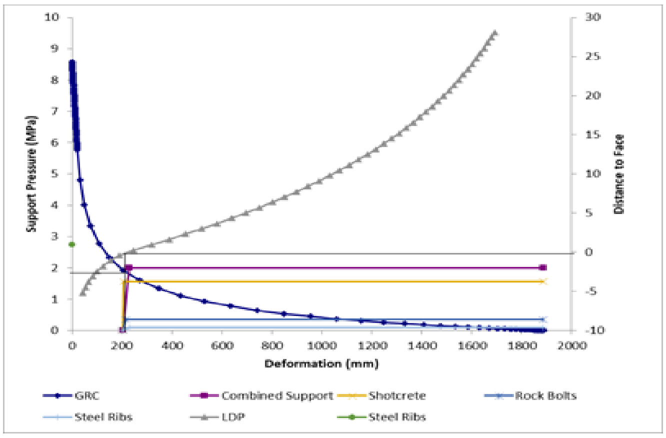

Fig 6: Interaction of GRC, LDP and SCC in tunnel section at 1+580 m

From figure 6, when the support is applied at face of tunnel there will be 203.58 mm displacement at tunnel wall. At the face of tunnel, the maximum pressure that the support can experience is 1.93 MPa whereas the maximum support capacity for combined support i.e., shotcrete, bolts and steel ribs is 1.99 MPa. Thus, the combined support withstands the pressure at face with factor of safety of 1.03. But the support capacity of all other individual support is less than the support pressure on face thus they will fail.

For successful application of the support, either the support capacity should be increased to a value greater than the support pressure, or the support is applied a distance behind the tunnel face. Both of these operations are demanding in different ways. For example, while concrete lining can increase support capacity in the first scenario, applying concrete lining to the tunnel's face is a highly difficult task. In the second scenario, the tunnel size will converge somewhat over the allowable limit, but if support is delayed, it could result in total collapse, making it a very difficult operation to apply support.

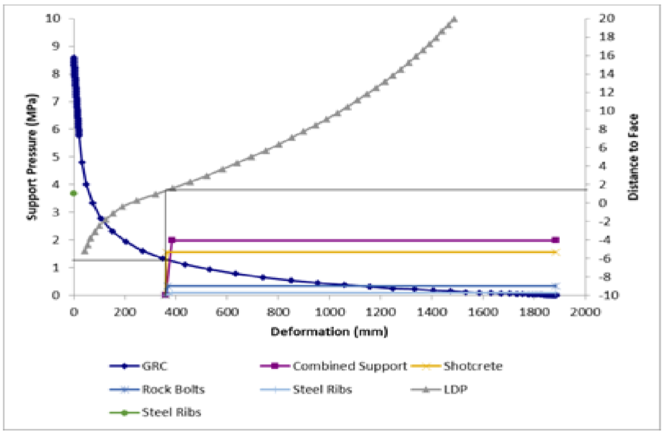

Fig 7: Interaction of GRC, LDP and SCC when support is at 1.5m from face

Figure 7 shows that the displacement increases to 380 mm and the maximum pressure that the support may encounter decreases to 1.28 Mpa when the support is applied at 1.5 meters from the face of the tunnel. Shotcrete, bolts, and steel ribs have a combined maximum support capability of 1.99 MPa. Hence, with a safety factor of 1.55, the combined support can bear pressure applied to its face. The shotcrete withstands the pressure with low factor of safety. Nevertheless, none of the other individual supports can handle the pressure that they are under, therefore they will all fall short.

This method is applicable only when far field stresses normal to the long axis of the tunnel are constant and the shape of tunnel is circular. Due to these assumptions, there is question in the validity of the methods in the cases in which the far field stresses are not equal and the tunnel section is other than circular. To overcome the limitations of CCM, numerical analysis has been done.

C. Numerical Analysis

To find the stresses and deformation in a rock mass numerical model are very useful. They provide the sophisticated interface to solve complex rock engineering problems with reference to inhomogeneity, isotropy, groundwater and topography. The tunnel closure in this thesis can be calculated using the RS^3 and $Phase^2$ tools. The importance of the tunnel closure will influence whether or not the ground is compressed. The Poisson's ratio is utilized in this application to determine the gravity field stress. The program has also incorporated material parameters from the literature, such as the density of the rock mass under the weakest condition, the unconfined compressive strength of intact rock (σ_{ci}), the Hoek-Brown constant (mi), the Geological strength index (GSI), the Young's modulus of intact rock (Ei), and Poisson's ratio (ν).

➢ *2D Numerical Modelling*

Table 3: Input rock parameters for numerical modelling

Rock Type	Field stress type	Initial element loading	Elastic type	Failure Criterion	Material type	Density, MN/m^3	Poisson's ratio	Erm Mpa	σ_{ci}, Mpa	mi	GSI
Silicious Phyllite	Constant	Field stress only	Isotropic	Generalized Hoek-Brown	Elastic and plastic	0.026	0.2	492.718	30	7	20
Metasandstone	Constant	Field stress only	Isotropic	Generalized Hoek-Brown	Elastic and plastic	0.026	0.18	651.065	40	16	30

By using RS^2 section profile available in RS^3 software, the Phase^2 model was exported. For the 2D analysis and design the section was prepared at which maximum deformation was observed (7 m ahead the face in RS^3 model) as shown in figure 8.

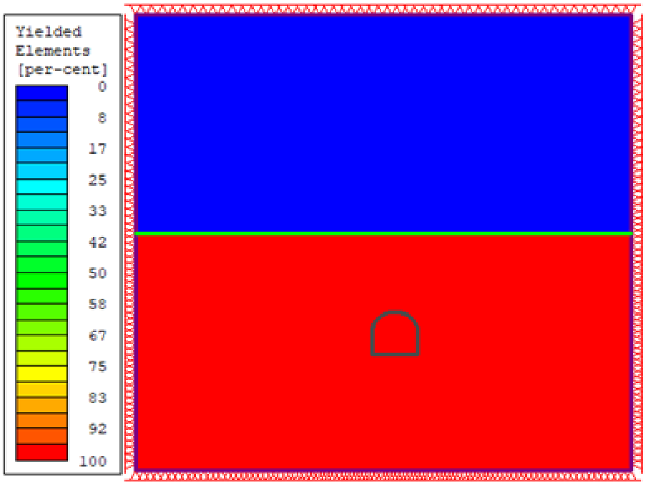

Fig 8: Yielded rock mass in insitu stage

After excavation, the plastic zone surrounding the tunnel is found by using the core replacement technique. To find the plastic zone in phase^2 program, the radius of excavated tunnel is measured upto the point in surrounding rock mass where there is extent of failure in rock mass. At the field condition depicted in figure 8, the rock mass in the tunnel segment at chainage 1+580 m has already yielded. As a result, it is very difficult to calculate the radius of the plastic zone in this tunnel segment. So, in this thesis work, Vlachopolous technique of core replacement method has not been used for the support application. At stage 2 of the model for the tunnel sections, the support is applied immediately after excavation.

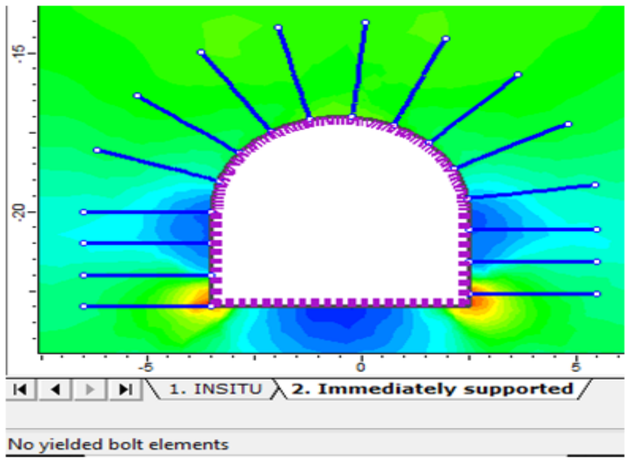

Fig 9: Yielded bolt

As shown in figure 9, the stage 2 support system is constructed using shotcrete, steel ribs with concrete liners, and bolts. It was discovered that none of the bolts failed. As shown in figure 10 the support capacity plot of the support system has been plotted with a safety factor of 1.03 (as seen in CCM). This outcome demonstrate that several lining components have safety factors less than one and are prone to failure.

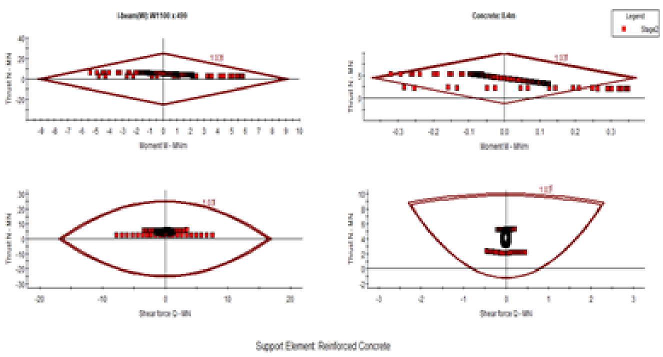

Fig 10: Support Capacity Plot

Both tunnels with and without support underwent a displacement analysis. Figure 11 shows the displacement of tunnel before application of tunnel.

ISSN No:-2456-2165

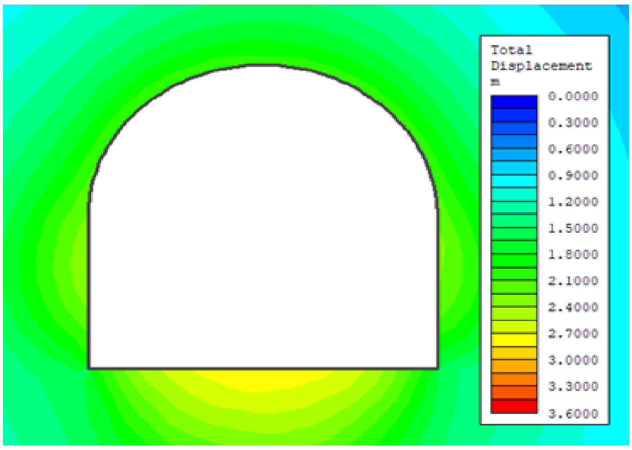

Fig 11: Displacement before application of support

The application of support causes a decrease in the overall displacement. The lack of support at the invert in figure 12 may be the reason why the displacement at the invert is not considerably lower than at the wall and crown.

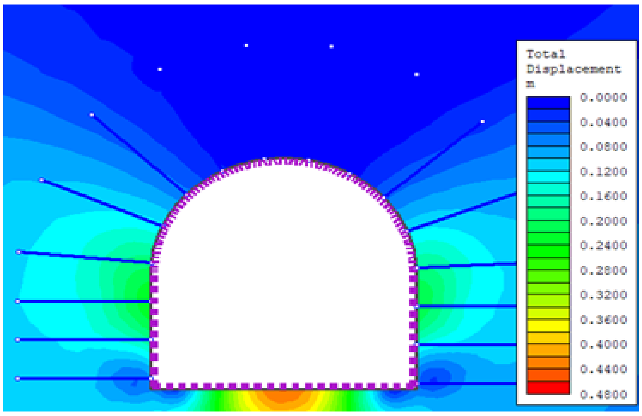

Fig 12: Displacement after application of support

All the methods discussed above are based on single section estimation and give single section result. Due to which there is high discrepancy in result from real problem. To lower the discrepancy and verify the result from the above methods, 3D numerical modelling was adopted for analysis. 3D modelling is considered to be better than other methods since it covers 3 dimensional aspects and give more realistic result.

➢ *3D Numerical Modelling*

For the 3D analysis the valley model was generated of size 45 m along X axis and 25 m along Y axis shown in figure 13. The analysis was done for chainage 1+580 only and is compared with result from all the above methods

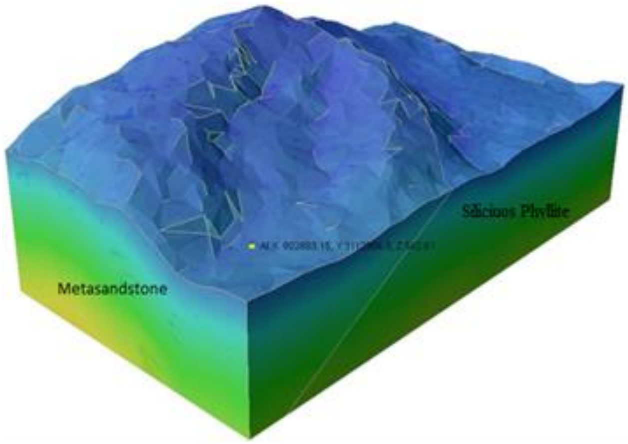

Fig 13: Valley model of selected section

A three-dimensional model is required to examine the stress and deformation pattern surrounding tunnels since underground excavation is a three-dimensional challenge. In this paper, the redistribution pattern of stresses and the strength factor were examined using elastic analysis, and the displacement was examined using plastic analysis. The model was initially created using the aforementioned input parameters. In the silicious phyllite zone, the strength factor is less than 1, whereas it is close to 1 in the metasandstone rock mass zone. The fact that the strength factor at the tunnel's excavation border less than one indicates that the material has yielded. This type of plastic analysis was done. The plastic analysis was conducted using the elastic perfectly plastic material model.

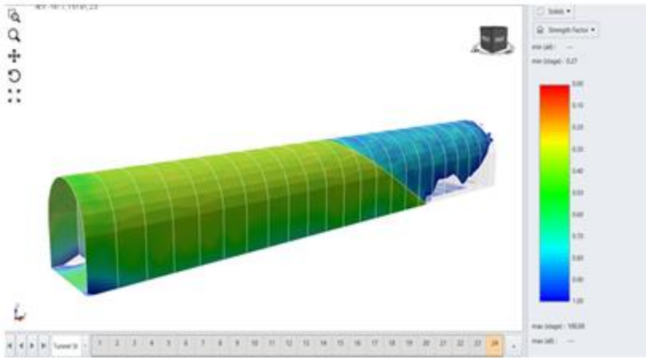

Fig 14: Strength factor during elastic analysis

RS^3, a 3-dimensional elasto-plastic finite element program, was used to study the total deformation of the tunnel at different chainages. It is depicted in Figure 14.

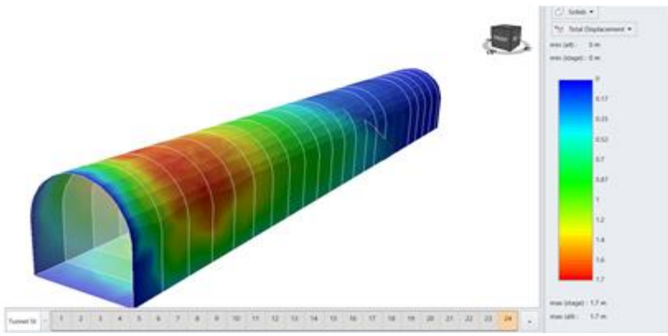

Fig 15: Deformation around tunnel

The entire displacement is centered along the roof of the HRT at stages 2, 3, 4, and 5, with a maximum value of 1.7 m, according to an elastic analysis of figure 15 utilizing generalized Hoek and Brown failure criteria.

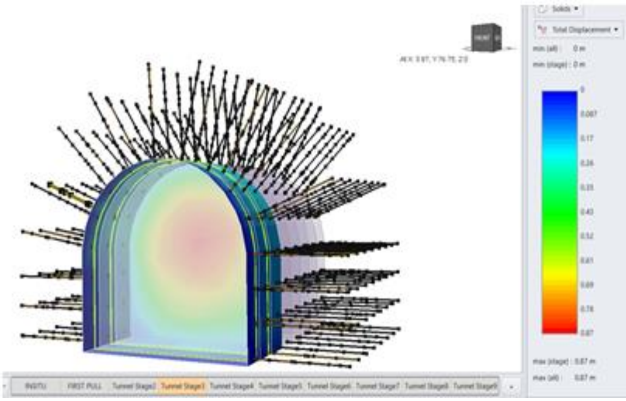

Fig 16: Displacement after application of support at 1m from face

Initially, supports were applied 1 m behind the face, maximum displacement of 0.87 m has been observed at the face of the tunnel after 3rd pull shown in figure 15. After this, the analysis has been done by applying supports at the face of the tunnel, the total displacement at the face has been reduced to 0.54 m shown in figure 16.

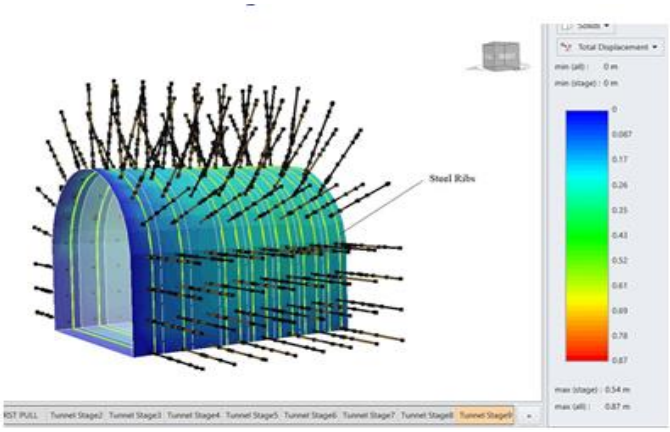

Fig 17: Displacement after application of support at the face of tunnel

D. Comparison of the Result

In this section, the result obtained from different methods are compare with each other.

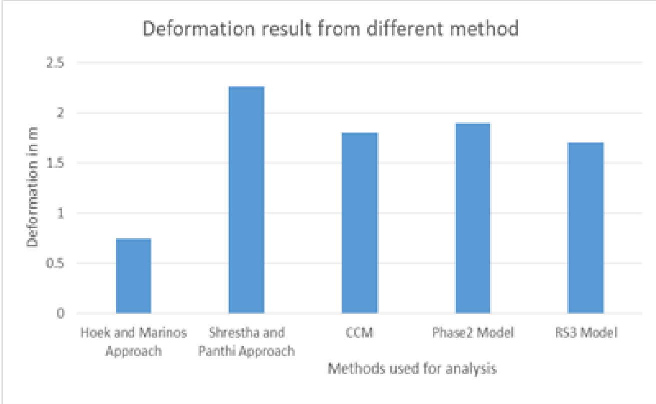

Fig 18: Result from different methods

Figure 18 shows that there is more discrepancy in result obtained from semi-analytical methods from numerical modelling. This result also illustrates that the Hoek and Marinos approach is quite conservative as compared to other methods because it gives very lower value. The result from CCM, Phase^2 and RS^3 model is almost same. This is may be due to the same rock parameters and rock supports are used for analysis under same conditions. The discrepancy in result from semi-analytical, analytical and numerical modelling may be due to the different assumption that are considered by different method.

V. CONCLUSION

The headrace tunnel of the Setikhola Hydropower Project traverses a weak rock mass. Plastic deformation is likely to takes place in these fragile rocks. The most important aspect of this study is to conduct a stability analysis of the headrace tunnel of the Setikhola Hydropower Project. The goal is to predict and assess plastic deformation using empirical techniques, semi-analytical techniques, analytical techniques, and numerical. For each of these techniques, accurate in-situ stress and rock mass quality assessments are necessary. The empirical techniques used by Singh et al. (1992) and Goel et al. (1995) predict whether or not there will be ground squeezing, but we are unable to determine the magnitude of tunnel deformation and support pressure. According to Singh et al. (1992) and Goel et al. (1995), there will be squeezing in this work. In this work Hoek and Marinos (2000) predict that there will be a severe squeezing issue at chainage 1+580. This result also illustrates that the Hoek and Marinos approach is quite conservative as compared to other methods because it gives very lower value. The short-term deformation and long-term deformation, according to Shrestha and Panthi (2015) method, are 2.26 and 3.94 m, respectively. The analytical approach, CCM, which takes into account face effect, can be used to determine the amount of support pressure needed to keep the deformation of the tunnel wall within a given range. Although the approach is designed for circular tunnels, some shape effects can be detected in the calculations. For calculations involving tunnels of different shapes, equivalent tunnel diameter has been employed. When the support is applied at face of tunnel there will be 203.58 mm and when support is at 1.5 m from face 380 mm deformation. Phase[2] and RS[3] have been used to determine the tunnel deformation and is found to be 1.9 and 1.7 m respectively. The outcome of numerical modeling can be compared with the outcomes of analytical and semi-analytical methods. This demonstrates that support design for constructing tunnels in weak rock masses is possible using these numerical model systems

RECOMMENDATION

In this paper, the stability of HRT is assessed for a particular rock mass condition. So that there are many limitations in analysis. Following recommendations made for further study.

Since these input parameters are the most crucial elements for accurate analysis, field observations and laboratory test data are particularly significant for accurate calculation of rock mass attributes. The study did not take into account the impact of ground water, which will result in a slightly higher value for the rock mass strength used to determine stability. The rock yields at insitu condition and applied support system fails during analysis. To overcome this effect either strengthening the rock mass by pre injection grouting, heavy support system needs to be applied, reduce the pull length or adopt top heading and benching tunnelling method.

REFERENCES

[1]. Basnet, C. B. (2013). Evaluation on the Squeezing Phenomenon at the Headrace Tunnel of Chameliya Hydroelectric Project , Nepal. June.
[2]. Brady, B. H. G., & Brown, E. T. (2007). Rock Mechanics for Underground Mining (Third, Vol. 15, Issue 2).
[3]. Carranza-Torres, C., & Fairhurst, C. (2000). Application of the Convergence-Confinement Method of tunnel design to rock masses that satisfy the Hoek-Brown failure criterion. Tunnelling and Underground Space Technology, 15(2), 187–213. https://doi.org/10.1016/S0886-7798(00)00046-8
[4]. Chaudhary, B. (2022). Assessment on plastic deformation at the powerhouse cavern and tailrace tunnel of andhikhola hydroelectric project. February. https://doi.org/10.13140/RG.2.2.30784.81924
[5]. Dhital, M. (2015). Geology of the Nepal Himalaya. In Springer International Publishing Switzerland. http://link.springer.com/10.1007/978-3-319-02496-7
[6]. Goodman, R. E. (1989). Introduction to Rock Mechanics.
[7]. Hoek, E., Carranza, C., & Corkum, B. (2002). Hoek-brown failure criterion – 2002 edition. Narms-Tac, September 2015, 267–273.
[8]. Hoek, E., & Diederichs, M. S. (2006). Empirical estimation of rock mass modulus. 43, 203–215. https://doi.org/10.1016/j.ijrmms.2005.06.005
[9]. Hoek, E., & Marinos, P. (2000). Predecting tunnel squeezing problem in weak heterogeneous rock masses. Tunnels and Tunnelling International, 45–51.
[10]. Hudson, J. A., & Harrison, J. P. (1997). Engineering rock mechanics an introduction to the principles. In elsevier science ltd.
[11]. Labuz, J. F., & Zang, A. (2012). Mohr-Coulomb Failure Criterion.
[12]. Palmström, A. (1995). Characterizing Rock Burst and Squeeing by the Rock Mass Index. Design and Construction of Underground Structures, c(February), 23–25.
[13]. Panthi, K. (2006). Analysis of Engineering Geological Uncertainties Related to Tunnelling in Himalayan Rock Mass Conditions. In Norwegian University of Science and Technology (Vol. 5, Issue February).
[14]. Panthi, K. K. (2012). Evaluation of rock bursting phenomena in a tunnel in the Himalayas. Bulletin of Engineering Geology and the Environment, 71(4), 761–769. https://doi.org/10.1007/s10064-012-0444-5
[15]. Panthi, K. K. (2017). Review on the Prevailing Methods for the Prediction of Potential Rock Burst / Rock Spalling in Tunnels. Technology, 2014, 1–8.
[16]. Setikhola Hydropower Project. (2018). Updated Feasibility Study Report.
[17]. Shrestha, G. lal. (2021). Rock Engineering Handbook on Design of Tunnel and other Underground Structures.
[18]. Shrestha, N. (2020). Evaluation on stability condition along the headrace tunnel of Kulekhani-III Hydroelectric Project. June.
[19]. Shrestha, P. K. (2021). *Lecture notes on Stability Analysis and Support Design*.