Proceedings of 14th IOE Graduate Conference
Peer Reviewed
Year: 2023 Month: December Volume: 14
ISSN: 2350-8914 (Online), 2350-8906 (Print)

Deformation Study and Stability Assessment of Headrace Tunnel: A Case Study of Khimti-2 Hydroelectric Project

Sangit Lamichhane [a], Saphal Lamichhane [b], Uttam Lamsal [c]

[a] *Department of Civil Engineering, Pashchimanchal Campus, IOE, Tribhuvan University, Nepal*
[b] *Adyanta Engineering Pvt. Ltd.*
[c] *Peoples Energy Limited*
[a] song.geet53@gmail.com, [b] saphallamichhane@gmail.com, [c] geologistuttam@gmail.com

Abstract

Rock engineering involves the comprehensive examination of both rock mechanics and engineering geology. Evaluating the various parameters of rock engineering plays a crucial role in effective planning, design, and construction of subterranean structures. Two primary parameters, strength and deformability, are pivotal in understanding the mechanical properties of the rock mass. These properties are typically determined using a range of commonly employed empirical methods. Additionally, rock engineering delves into engineering geological factors such as the characteristics of rock joints and in-situ stress conditions, all of which are meticulously studied to ensure the success of underground projects.This paper describes about the interaction of rockmass and support applied after the excavation is made in the rockmass. The geological conditions in the project area have been thoroughly examined through a combination of literature review and on-site surface mapping. These investigations have revealed that the project site is situated within the Augen Gneiss unit, which is part of a meta-sedimentary rock sequence equivalent to the Nuwakot group rocks from the Paleoproterozoic era. Support estimation using Q-chart and Numerical Modelling with Phase-2 software is carried out and compared with the support used in the project.

Keywords

Rock engineering, Deformability, Squeezing, Geological parameters

1. Introduction

Nepal presents a unique geological landscape, characterized by its intricate geology and significant variations in elevation within a relatively small geographical area. In this context, tunneling has emerged as a crucial field of study and practice. Despite limited site coverage, the diverse topographical and geological conditions, particularly in the lesser Himalayan zone, pose formidable challenges for the construction of underground structures.To address these challenges effectively, a comprehensive understanding of rock engineering is imperative. This knowledge is essential for devising appropriate solutions to the geological complexities encountered in Nepal. It enables engineers and researchers to tackle the intricacies of constructing underground structures in a region characterized by its diverse and challenging terrain.

Rock engineering encompasses the examination of both rock mechanics and engineering geology. Rock mechanics includes the study of mechanical behavior of rock such as strength and deformability whereas engineering geology includes the application of geological knowledge in engineering analysis, planning, design and construction (Nilsen & Palmström, 2000). The strength of rock is contingent on factors such as its mineral composition and structural orientation. Additionally, comprehensive assessments must consider geological factors like prominent weak zones, the condition of rock jointing, in-situ stress conditions, weathering status, and groundwater presence. To evaluate the engineering characteristics of the rock mass, thorough surface and subsurface investigations are imperative. Furthermore, both laboratory and field tests are essential to establish the input parameters required for the stability analysis of subterranean structures. In conclusion, a thorough understanding of rock engineering principles is essential for the effective planning, design, and construction of tunnels and other infrastructure projects involving rock formations.

This study is undertaken to understand the basic aspects that play a crucial role in the preliminary design of headrace tunnel in a hydropower project. What are the types of instabilities encountered while tunneling in Himalayan rockmass and major factors affecting the stability of tunnels in the Himalayan rock mass? How can we design a suitable and efficient support system to address these stability issues based on the rock mass characterization? Furthermore, the tunnels are being constructed after limited field investigation and the supports are assigned as per the support chart of Q-system only, which results the frequent design change. This study attempts to provide solution to these problems.

2. Rockmass Properties & Classification

The intact rock is strong and homogeneous with few discontinuities and stronger than the rock mass [1]. Intact rock strength and deformability are determined from lab test or insitu tests[2] . Uni-axial compressive strength ($\sigma_c i$) test is the most common method to test mechanical characteristics of rock where intact rock specimen cylinder is loaded till failure. Value of sci is useful in calculating the rockmass strength (σ_{cm}). The intact rock strength depends on mineralogical composition of rock, size of specimen, strength anisotropy, water effect, weathering and alteration etc.

A rockmass is a complex geometrical and mechanical assemblage resulting from a long history of tectonic forces and other natural environmental effects. The intact rock specimen is usually strong and homogeneous, with few discontinuities and therefore doesn't represent the strength of the total rockmass[3]. The displacement, strength and failure properties of a rockmass are determined by the mechanical properties of the intact rock, the geometrical properties of the discontinuities, and the mechanical properties of the discontinuities. [4]

Table 1: Empirical estimation of rockmass strength

Proposed By	Empirical Relationships
Bieniawaski (1993)	$\sigma \mathrm{cm} = \sigma \mathrm{ci}^{\times} \exp\left(\frac{RMR-100}{18.75}\right)$
Hoek et al. (2002)	$\sigma \mathrm{cm} = \sigma \mathrm{ci}^{\times} \left(\frac{(mb+4s-a(mb-8s))\left(\frac{\mathrm{mb}}{4}+s\right)^{a-1}}{2(1+a)(2+a)}\right)$
Barton (2002)	$\sigma \mathrm{cm} = 5\gamma^{\times} {Q_C}^{1/3} = 5\gamma\left[\frac{\sigma ci}{100} \times 10^{\frac{RMR-50}{15}}\right]^{1/3}$
Panthi (2006)	$\sigma \mathrm{cm} = \frac{\sigma(ci)^{1.5}}{60}$

Rock Mass Classification is the process of placing a rock mass into groups or classes on defined relationships [5], and Assigning a distinct identifier (such as a label or numerical code) based on shared properties or features, enabling the anticipation of the behavior of the rock mass. Rock mass is referred to an assemblage of rock material separated by rock discontinuities, mostly by joints, bedding planes, dyke intrusions and faults etc. Bedding planes, dyke intrusions and faults are not so common as compared to joints and are dealt individually [6]. Rock mass classification systems allow the user to follow a guideline and place the object in an appropriate class. The assessment is done using Q-Syatem of rockmass classification.

3. Rockmass Deformability

The deformability of a rock mass is a critical engineering parameter essential for designing underground structures and conducting stability analyses.

Table 2: Relationship to estimate rock mass deformation

Proposed By	rock mass deformation
Serafin and Pereira (1983)	$E_m = 10^{\frac{RMR-10}{40}}$
Hoek and Diederichs (2006)	$\mathrm{E_m} = \mathrm{E_{ci}}\left[0.02 + \frac{1-\frac{D}{2}}{1+e^{\frac{60+15D-GSI}{11}}}\right]$
Hoek and Brown (1997)	$E_m = \sqrt{\frac{\sigma ci}{100}} \times 10^{\frac{GSL-10}{40}}$
Barton (2002)	$E_m = 10 \times \left(\frac{Q \times \sigma ci}{100}\right)^{1/3}$
Panthi (2006)	$E_m = Ei \times \frac{\sigma cm}{\sigma ci} = \frac{\sigma ci^{0.6}}{60} Ei$

It provides insights into the mechanical characteristics of the rock mass. While it's possible to determine the rock mass deformation modulus through various field tests, these methods can be time-consuming To streamline the process, the elastic modulus of intact rock is initially determined in a laboratory setting. Subsequently, the rock mass deformation modulus is derived using a range of empirical equations, allowing for a more efficient assessment of deformability properties.

4. Rock stress

The geological materials are are preloaded by in-situ stresses. While excavation is done in the rockmass, the in-situ stresses are redistributed, which induces tangential stresses in the vicinity of the underground opening [7]. When the induced Tangential stress becomes larger than the rockmass strength, the rockmass becomes overstressed which renders the rockmass susceptible to stress induced stability issues during underground construction. Therefore, The magnitudes of in-situ stresses play a vital role in stability of underground openings.

4.1 In-situ stresses in rock mass

The In-situ stress in the rockmass is due to gravity, toppgraphy, residual stress and plate tectonics. The stress induced by gravity can be calculated by;

Vertical stress $\sigma_v = \gamma \cdot H$; Horizontalstress $\sigma_h = \frac{\mu}{1-\mu} \times \gamma \cdot H + \sigma_{tec}$

Where,$\sigma_v, \sigma_h, \sigma_{tec}$ are the vertical, horizontal and tectonic stresses in MPa,

γ is the unit weight in MN/m3,

H is depth in meters, and

μ is the Poisson's ratio.

4.2 Stress Redistribution around a tunnel

The in-situ stress in the rock mass are altered once an underground aperture is excavated. Stresses are transferred throughout the excavation's edge. The redistribution of stresses around a circular hole in an elastic material in isostatic stress condition can be represented using the Kirsch equation.

The tangential stresses (σ_θ) and radial stresses (σ_r) at the periphery of a circular opening in a fully iso-static stress and for a elastic rock material will be twice and zero times the iso-static stress, respectively. As the ratio of radial distance (R) to the opening radius (r) grows, stresses become normalized. The magnitude of σ_θ and σ_r are:

$$\sigma_\theta = \sigma \times \left[1 + \frac{r^2}{R^2}\right] \quad \text{and} \quad \sigma_r = \sigma \times \left[1 - \frac{r^2}{R^2}\right]$$

5. Failure Criteria

5.1 M-C failure Criterion I

The Mohr-Coulomb failure criteria is a linear criterion used to analyze the tunnel stability in isotropic, unjointed, elastic rock mass. The Mohr Coulomb failure criterion demonstrates the link between shear and normal stress at failure. Because an internal friction for material expressed with friction angle is utilized, it is sometimes referred to a as the inner friction criteria.

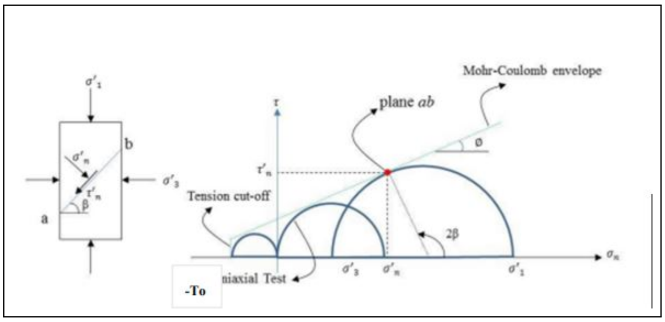

Figure 1: Mohr coulomb failure criterion with tension cutoff (Goodman,1989)

5.2 Hoek and Brown criterian

This is a non-linear criterion which is useful for schistose and jointed rock mass of homogeneous character and is based on triaxial test. The generalized Hoek Brown criterion for the estimation of rock mass strength is expressed as [8]. $\sigma'_1 = \sigma'_3 + \sigma_{ci}(mb \cdot \frac{\sigma'_3}{\sigma_{ci}} + s)^a$

Where mb, s and a are the rock mass material constants.

$mb = mi \exp(\frac{GSI-100}{28-14D})$

$s = \exp \frac{GSI-100}{9-3D}$

$a = \frac{1}{2} + \frac{1}{6} \cdot (\exp(\frac{-GSI}{15}) - \exp(\frac{-20}{3}))$

6. Study Area

The Khimti-2 Hydroelectric Project (KH2HEP) is a Run of the River (RoR) project whose installed capacity is 48.8MW. This project is located in Jiri Municipality and Tamakoshi Rural Municipality of Dolakha District (Previous Jiri, Thulopatal and Hawa VDC of Dolkha District) and Gokulganga Rural Municipality of Ramechhap District (Previous Rasnalu VDC of Ramechhap District). This Project is choosen as study area as the tunnels are being constructed after limited field investigation and the supports are assigned as per the support chart of Q-system only, which results the frequent design change.Furthermore,the supports suggested by Q chart are insufficient as per the site condition as the Steel Ribs get buckled at Chainage 2+500.63m.

6.1 Geology of the Project Area

Geologically, The Khimti-2 Hydropower project site is situated in Central Nepal, specifically within the Lesser Himalayan Midland zone. The geological characteristics of this area involve the presence of tectonic elements such as the Jiri thrust, Midland Thrust, and Vicholo Thrust. The primary rock formations within this region consist of augen gneiss, banded gneiss, schist, phyllite, and metasandstone. Notably, the dominant rock types in the project area are augen gneiss, schist, and banded gneiss. This location is closely situated near the Midland thrust fault, with a general occurrence direction of NE-SW and a northwestward dip.

6.2 Headrace Tunnel of Khimti-2 Hydropower Project

Table 3: Predicted Rockmass in headrace tunnel

Rock Class	Predicted Length((m)	Percentage
C	662	10.00%
D	993	15.01%
E	1985	30.00%
F	2315	34.99%
G	662	10.00%
Total	6617.00	100.00%

7. Evaluation of Tunnel Stability

Exploring the impact of rock engineering parameters on tunnel stability involves the utilization of Phase 2 software. This software enables the calculation of maximum radial deformation and plastic radius of the tunnel while altering various rock engineering parameters. Factors such as the uniaxial compressive strength of rock, Poisson's ratio, Young's modulus of elasticity, and geological strength index are systematically varied in the model to investigate their influence on tunnel stability.

7.1 Selection of Section along HRT for analysis

The headrace tunnel of Khimti-2 hydropower project lies mostly in intercalated Augen gneiss and schist of rock class C To G as per preconstruction phase investigation. Total 29 sections are studied based on empirical and semi empirical methods. Depending on the rockmass quality encountered during tunneling, Based on rockmass quality, overburden and problem encountered while tunneling the Chainage 2+446.65-2+464.65 and 2+491.40-2+506.6 are selected for this study purpose. The average Q-value at Chainage 2+446.65-2+464.65 is 0.036 having overburden of 160- 168m, similarly for Chainage 2+491.40-2+506.6 the average Q value is 0.05 having similar overburden.

8. Stability Assessment

The Empirical, Semi-Analytical, Analytical and Numerical Methods were utilized for the evaluation of the stability of Tunnel and estimation of Tunnel deformation at critical sections regarding Overburden and Q-values. The Squeezing was predicted using Empirical methods like [9] and [10] approach. The Semi-Analytical method such as [11] approach was used to estimate the magnitude of deformation .The Semi-Analytical method using [12] approach was used for the deformation estimation with various support pressures in stress anisotropy conditions. The squeezing phenomenon was studied using two methods: 2D finite element numerical analysis using Phase2 and the convergent confinement Method [13] with the Hoek and Brown failure criteria [8]

8.1 Empirical Methods Singh et al (1992) and Goel et al (1995) approach

The Goel et. al method of squeezing prediction shows no squeezing zone whereas 10 Chainage are predicted to face squeezing problem while predicting through Singh et al approach. Singh et al approach predicted the squeezing problem in the area of our concern.

8.2 Hoek and Marinos (2000) Approach

The Hoek and Marinos (2000) method represents a semi-analytical approach founded on a comprehensive closed-form solution tailored for circular tunnels operating within a hydrostatic stress environment. This method assumes uniform support distribution along the tunnel perimeter, aiming to forecast the potential for squeezing and estimate its scale. However, only few support problems are encountered while analysing by this approach.

8.3 Shrestha and Panthi (2015) method

This approach takes into account the anisotropic stress conditions of the material and is suitable for tunnels of various shapes, making it versatile in its applicability. At Chainages 2+450.63 and 2+500.63, The Initial Inward Deformation (δ_I) and Final Inward deformation (δ_F) are comparatively more, which Closely Depicts the Site Condition.

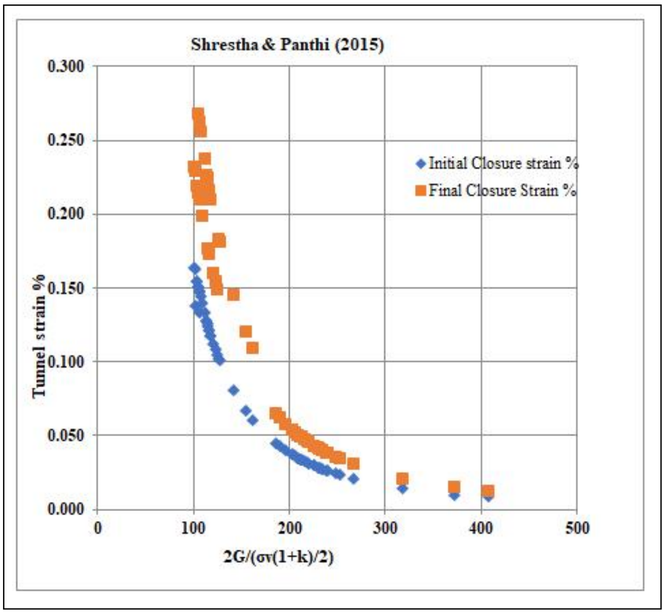

Figure 2: Tunnel strain Vs ratio of shear modulus (G) and in-situ vertical stress

8.4 Analytical Method (CCM)

Based on the overburden, rock type and Q and GSI value Three sections are selected between Chainage 2+410 to 2+510 m along the section for the analysis using Convergence-Confinement Method (CCM). Most of the input parameters are taken from site conditions, some are taken from related literature reviews and some of the input parameters are assumed for the study purpose. The Support Characteristics Curve is developed using shotcrete or concrete, rock bolts, and steel ribs. Shotcrete or concrete linings have parameters like an unconfined compressive strength of 35 MPa, thickness of 5-100 cm (depending on rock class), Poisson's ratio of 0.2, and Young's modulus of elasticity of 20 GPa. Fully grouted rock bolts are 2-3 meters long, 20 mm in diameter, have a bolt modulus of 200 GPa, and a peak tensile strength of 0.1 MN, with a spacing of 1-2 meters center to center. The Rock support interaction curve is found as Fig 3 graph which uses circular tunnel section and 2m face distance.

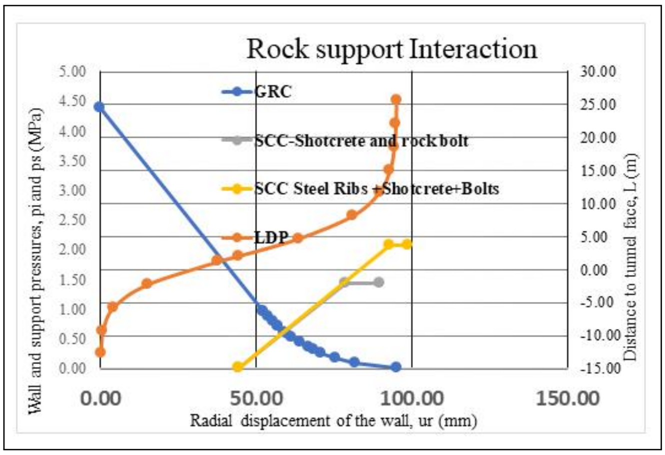

Figure 3: Result of Rock Support Interaction Analysis at Chainage 2+500.63m

8.5 Block stability analysis

Discontinuities intersect to form blocks. In specific locations, the right combination of joints creates wedges. In tunnel excavation, rock wedges can emerge due to these joints. Blocks might collapse because of gravity and other forces. Both roof and wall wedges can fail by falling or sliding. Analyzing these blocks and wedges is crucial for ensuring the stability of the excavation. This analysis involves studying data obtained outside the tunnel to assess block stability. Based on the data at 27°34'55.60" 86°12'2.46", the UNWEDGE SOFTWARE gave following result;

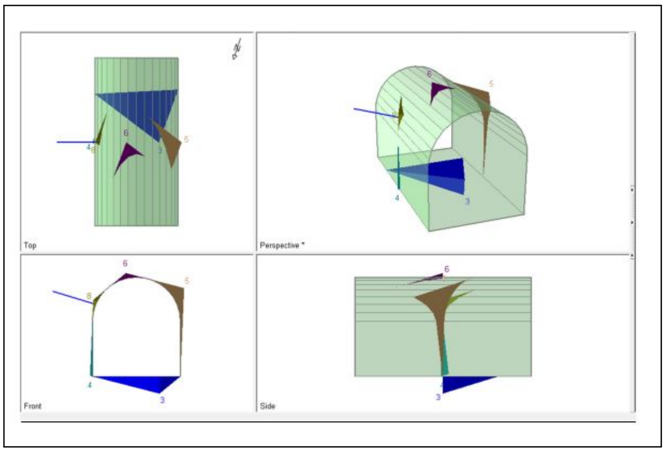

Figure 4: Perimeter wedge formation (27°34'55.60" 86°12'2.46")

The total of five wedges formed around the perimeter of the tunnel. Wedge 4 is in the lower left wall, wedge 5 is in upper right wall, wedge 3 is on the invert of tunnel which is stable, wedge 6 is in the roof. Wedge 8 is on upper left wall. For which factor of safety are larger and hence the tunnel is safe from wedgefall even if small quantity of rockbolts are installed.

8.6 Numerical modeling with Phase2

The numerical modeling is done with phase2 Software. The modal is prepared in such a way that, the supports are recorded in order to get the displacement that was actually occurred in the side to achieve the rockmass parameters and stress factor. After doing hundreds of trial bu varying some rockmass parameters and stress factors, the displacement was achieved very near to surface condition. As the empirical, semi empirical method didn't predicted the actual site condition, numerical modeling is carried out varying parameters to get the closest displacement. In the site at Chainage some symptoms of deformation greater than the allowable limit was encountered at Chainage 2+446.65-2+264.65 and 2+491.4-2+506.6m. Being similar geological condition and overburden, Chainage 2+500.63 is tried to analyze in Phase2

8.7 Model Setup

For the analysis of the critical tunnel section, a 2D box model is prepared for the tunnel width, which is five times the width of its excavation. The in-situ stress from the valley model σ_1, σ_3, and σ_z with an angle is utilized in this 2D model. The model was prepared in two steps. At first step, Core Modulus at the tunnel face is identified creating 10 stages. In Second step, The model was divided into 3 stages with initial insitu condition at stage 1, excavation at stage 2, support installation at stage 3.1m of Disturbed zone was created in the model.The boundary were confined in two ways. In the two-dimensional model, material characteristics needed specification by selecting the initial loading element as the field stress and body force as the unit weight. Additionally, Poisson's ratio was included, along with determining strength parameters using the Generalized Hoek Brown method. The 2D model is computed and interpreted until the absolute displacement at the crown part of the tunnel matches its value to the field monitoring convergence data. The various input parameters for Phase2 analysis are presented in next section.

8.8 Back Analysis and Input Parameters

The model is set up first giving the geometry of tunnel. The model was prepared in two steps. At first step, Core Modulus at the tunnel face is identified creating 10 stages. In Second step, The model was divided into 3 stages with initial insitu condition at stage 1, excavation at stage 2, support installation at stage 3. 1m of Disturbed zone was created in the model. The input parameters like stress condition , Young's Modulus of the rock etc. were varied until the deformation data becomes very close to the exact site condition. The 2D model is computed and interpreted until the absolute displacement at the crown part of the tunnel matches its closest value to the field monitoring convergence data after applying the primary supports at Chainages 2+410.63m to 2+500.63m. The various input parameters for Phase2 analysis are presented in next

Table 4: Input Parameters for rockmass and Intact Rock

S.N	Parameters	Value/Condition
1	Rock Type	Augen Gneiss
2	Initial Element Loading	Field Stress+Body Force
3	Unit weight MN/m^3	0.026
4	Poisson's Ratio (ν)	0.3
5	Young's Modulus (MPa)	13710
6	Failure Criterion	Generalized H&B
7	Material type	Plastic
8	Intact Rock Compressive Strength (Mpa)	56
9	Disturbance Factor	0.5
10	Field Stress Type	Constant
11	σ_1 (MPa)	7.04
12	σ_3 (MPa)	3.50
13	σ_z (MPa)	5.48
14	Angle	13

section. The following input parameters are used for the analysis of deformation at Chainage 2+500.63m

8.9 Support Properties

Based on site visit and design drawing of the Khimti-2 Hydropower Project, Following supports were seen in the studied section, whose effect are computed in the phase2 analysis.

Table 5: Support Types and Conditions

Sn	Support Type	Value/Condition
		@1.5m c/c
2	Wiremesh 150x150x6	@1.5m c/c
3	Bolt 20mm dia	2.5m long @1.2m c/c
4	Shotcrete (Plain+SFR)	200mm

8.10 Plastic analysis

The total displacement which is also called the maximum closure (umax) of the tunnel is 12-13 cm .at crown and 9-10cm at wall while monitored via total station in 14 consecutive days and time after the extra support RRS and Horizontal runner (Square hollow Pipe 50x50) which were placed after encountering the Buckling of steel rib. So analysis was carried to acquire such displacements, this called back analysis which was carried out to achieve some rockmass parameters and stress parameters for further analysis.

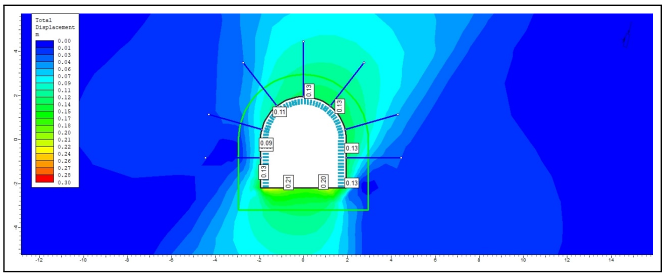

Figure 5: Result showing total displacement after support installation at chainage 2+500.63 m.

Table 6: Comparison of deformation data from site and model at Chainage 2+500.63

Location	Displacement at site (mm)	From This Model (mm)
At crown	130	110
At Left wall	90	100
At Right wall	112	120

8.11 Verification of Model

The numerical model is tried to validate using the site deformation data at Chainage 2+500.63 m. This specific section is used to model because the deformation was monitered contineously and more stability problem was encountered than other chainages.

9. Conclusion

The squeezing prediction using (Singh, et al., 1992) shows total 10 section of squeezing and using (Goel, et al., 1995) only one section is predicted to be squeezed.

Hoek & Marinos introduced a method that assesses the level of plastic deformation and the size of the plastic zone around an underground excavation. It categorizes the extent of squeezing or plastic deformation by analyzing the overall strain within the tunnel. This approach takes into account the pressure applied to support the excavation. However, it's worth noting that this method specifically applies to circular tunnels and doesn't account for the presence of stress variations in underground excavations. Only Few Support Problems are encountered while analyzing via this approach. The maximum deformations are encountered at Chainage 2+450.63 and 2+500.63 when initial pressure was nil.

Panthi & Shrestha developed a method to quantify both instantaneous and final deformations comprehensively. This approach takes into account the anisotropic stress conditions of the material and is suitable for tunnels of various shapes, making it versatile in its applicability. At Chainages 2+450.63 and 2+500.63, The Initial Inward Deformation δI and Final Inward deformation δF are comparatively more. Which Closely Depicts the Site Condition.

Regarding the results obtained from the Convergence Confinement Method (CCM) as proposed by (Carranza-Torres & Fairhurst, 2000) it's important to note several limitations. First, the CCM was originally designed for circular tunnels exposed to hydrostatic stress conditions. However, the headrace tunnel of Khimti-2 HEP has a Inverted-D shape, and the stress environment around it is characterized by anisotropy, which makes the direct application of CCM less suitable. One significant drawback of CCM is it neglects of the time-dependent aspects of plastic deformation. This omission is notable because considering time-dependency is crucial for a comprehensive understanding of the mechanisms driving plastic deformation. It is evident that for all Chainages, the maximum internal support pressure value (Psmax) exceeds the critical pressure (Pcr). This finding indicates that the support system chosen and implemented for the project is indeed sufficient and meets the requirements. However, the factor of safety for Chainage 2+500.63m is being less than FOS of similar other section. So this is assessed using numerical approach.

2D Finite element models developed in Phase2 software were validated by comparison of deformation from the model and from deformation monitoring data. The validated models were then used to determine the plastic deformation and required supports for its stabilization at other Chainages. The supports added after the problem encountered at Chainage 2+500.63m are adequate for the stability. And For other Chainages, within the project environment and insitu rockmass condition, the provided supports are adequate.

Finally,The project lies in such location where there is no possibility of wedge fall. As the headrace tunnel alignment passes through high overburden and comparatively weak rock like schist,Schistose Gneiss and Weathered gneiss, there is possibility of squeezing in several location. So special attention must be given while tunnelling i.e. Face mapping should be done accurately and Rocksupport must be given sufficiently.

References

[1] Z. Bieniawski. Determining rock mass deformability: Experience from case histories. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 1978.

[2] B. Nilsen and A. Thidemann. *Rock Engineering*. Norwegian Institute of Technology, 1993.

[3] K. Panthi. Analysis of engineering geological uncertainties related to tunnelling in himalayan rock mass conditions. 2006.

[4] J. A. Hudson and J. P. Harrison. *Engineering Rock Mechanics*. 1997.

[5] Z. Bieniawski. *Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering*. John Wiley & Sons, Inc., 1989.

[6] Z. Bieniawski. Classification of rock masses for engineering: The rmr-system and future trends. *Comprehensive Rock Engineering*, pages 553–573, 1993.

[7] E. Hoek and E. Brown. *Underground Excavation in Rock*. Institute of Mining and Metallurgy, 1980.

[8] E. Hoek, C. Carranza-Torres, and B. Corkum. Hoek-brown failure criterion. 2002.

[9] B. Singh, J. Jethwa, and A. Dube. Correlation between observed support pressure and rock mass quality. *Tunneling and Underground Space Technology*, 7, 1992.

[10] R. Goel, J. Jethwa, and A. Paithankar. Indian experiences with q and rmr systems. 1995.

[11] E. Hoek and P. Marinos. Gsi: A geological friendly tool for rock mass strength estimation. In *Proceedings of the GeoEng 2000 at the International Conference on Geotechnical and Geological Engineering*, pages 1422–1446, 2000.

[12] P. K. Shrestha and K. K. Panthi. Analysis of the plastic deformation behavior of schist and schistose mica gneiss at khimti headrace tunnel, nepal. 2014.

[13] C. Carranza-Torres and C. Fairhurst. Application of the convergence-confinement method of tunnel design to rock masses that satisfy the hoek-brown failure criterion. *Tunnelling and Underground Space Technology*, 15:187–213, 2000.