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Deformation Study of Gharkhola Hydroelectric Project Tunnel Emphasis on Squeezing, West Central Nepal

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Abstract: This research investigates the deformation and squeezing phenomena in the headrace tunnel of the Gharkhola Hydroelectric Project in Nepal, emphasizing the challenges posed by weak rock masses in the Himalayan region. The study employs a multifaceted methodology, integrating empirical, semi-analytical, and numerical approaches to evaluate the deformation modulus (Em) and predict tunnel stability. Utilizing regression analysis, the sensitivity of (Em) and predict tunnel stability. Utilizing regression analysis, the sensitivity of Em to rock mass classification Q, GSI, and RMR is scrutinized, revealing that euations proposed by Barton (2002), Hoek and Diederichs (2006), and Gokceoglu et al. (2003) exhibit the least sensitivity to rock mass variability, rendering them particularly efficacious for deformability estimation. The analysis identifies severe squeezing conditions in specific tunnel sections, with maximum wall closure reaching 0.432803 meters at an overburden height of 134 meters. Empirical and semi-analytical methods, including Singh et al. (1992) and Hoek and Marinos (2000), corroborate the prevalance of squeezing, while finite element modelling quantifies deformation, validating field observations. The study underscores the imperative of employing diverse methodologies to corroborate rock mass parameters, thereby enhancing the accuracy of squeezing predictions. The findings advocate for meticulous parameter selection and interdisciplinary validation to mitigate instability in tunneling projects within geologically complex terrains.

Keywords: ***Deformability, Squeezing, Support Pressure, numerical modeling.***

Introduction

In general, tunnel construction in hard rock can be taken as safe, and in weak rock, it is considered the most challenging based on the rock mass's mechanical behavior. In Himalayan countries like Nepal, Bhutan, and Northern parts of India there is good potential for waterpower generation tunnel construction is the most option given the economy and safety. Highly trustworthy rock mass qualities are necessary for the design and construction of rock tunnel structures, including support installation, safe excavation, and good performance (Hoek and Brown 1980a, Hoek and Diederichs 2006). In the complicated and fragile geological conditions, the occur several kinds of challenges to the underground works so, the rising risk of the excavations is managed by understanding the subsurface ground conditions, and understanding the stress conditions on the periphery of the excavation is the most necessary work.

The change in shape, size and structure resulting due to the stress is referred to as strain or deformation. Kayabasi et al. (2003) have shown that the deformation modulus is the most representative parameter of the pre-failure mechanical behavior of the rock material and a rock mass. Excavation in weak rock has greater stability problems, so the proper design of the underground openings is needed to consider the stress condition. The deformation modulus of a rock mass (Em) is one of the key parameters in rock engineering among the rock mass parameters. The design and successful completion of rock engineering projects depend heavily on the deformation modulus. In a tunnel, the anisotropic stress conditions generate problems like squeezing, rock bursting, and many stability-related problems (Selmer-Olsen and Broch, 19970). The instability in the tunnel that occurred in the weak rock is defined by the term squeezing. In this research, ground deformation is studied by details analysis of the discontinuity properties such as its pattern, block size, persistency, etc. Jaiswal et al. (2023) used a modeling technique to aid stress estimation and tunnel support evaluation for retards the problems that come in future development conducted by Mushahary et al. (2020). The direct evaluation of the tunnel squeezing is the most difficult task so using the several types of the empirical relationship given by the different researchers in the past using this empirical relation evaluation of the tunnel deformability was estimated.

A handful of the equations were based on the rock quality designation (RQD; Zhang and Einstein 2004), while the majority were based on the rock mass rating (RMR; Bieniawski 1973), the tunneling quality index (Q; Barton et al. 1974), and the geological strength index (GSI). In this study, the RMR, GSI, and Q classification systems are used to estimate the deformation modulus values of the rock mass along the tunnel of the Gharkhola Hydroelectric Project using existing empirical equations. Squeezing will happen when a plastic zone forms around the tunnel, creating severe deformation in the tunnel's perimeter if the rock mass is very weak and malleable. As a result of the fault zones and weak rocks (such as mudstone, shale, slate, phyllite, schist, and extremely schistose gneiss) that make up the mountains, tunnel squeezing is a frequent occurrence in the Himalayas (Panthi, 2006).

Along the headrace tunnel of the Gharkhola Hydroelectric Project, excessive tunnel squeezing has taken place. The project management had substantial difficulties in battling the squeeze. In its research by utilizing intact rock properties and other index variables, the commercial computer application Roclab (Rocscience Ltd., 2011) is used to evaluate rock mass parameters. In addition to mechanical characteristics like rock mass strength, deformation modulus, and tensile strength, these rock mass qualities also contain strength factors like cohesion and friction angle (Hoek and Brown 1980 b). Tunnel squeezing analysis was carried out by using the commercially available software Phase 2 by using the different rock mass input parameters. It enables accurate modeling of an underground excavation where the rock exhibits plastic behavior close to the excavation and elastic behavior farther away from the excavation (Khadka et al., 2019). In this research work, the Gharkhola Hydroelectric Project is selected for this study. The main aim of this research is to study the sensitivity of the deformability equations and to identify the tunnel's stability conditions.

Study Area

Geological study area lies in the Lesser Himalaya of Nepal Himalaya at Myagdi district is located in western Nepal, which is part of Gandaki province (Figure 1). The district lies at latitude 28o 20' 32.49" N and longitude 83o 33' 57.78" E and has a maximum altitude is 8, 167 m (Dhaulagiri Himal) and a minimum altitude is 792 m (Ratnechaur). The district is surrounded by Mustang, Manang, Kaski, Parbat, Baglung, Rukum, and Dolpa districts. The location of the study area (Figure 1) is 304 km away from Kathmandu Valley towards the west. The study project lies at Annapurna Rural Municipality wards no. 5 and 6. The project is a run-of-river type project. The geographic coordinates of the study project are latitude 28o 27' 02.53" and longitude 83o 39' 53.36". The region is a part of the Lower Nuwakot Group of the Lesser Himalaya geologically. It mostly consists of the white quartzite from the Fagfog Quartzite and the green phyllite from the Kuncha Formation, respectively. The foliation then dips moderately to steeply between 26° and 49° towards the north, with a strike of NW-SE.

GEOLOGY OF PROJECT AREA

Geologically, the area lies in the Lesser Himalaya (Lower Nuwakot Group) region. Which dominantly consists of the green phyllite and white color quartzite of the Kuncha Formation and Fagfog Quartzite respectively. Along the tunnel alignment, phyllite with amphibolite bands and Quartzite were present as the dominant rock types. Then the foliation strikes NW-SE with moderate to steep dipping about 26o-49o towards the north.

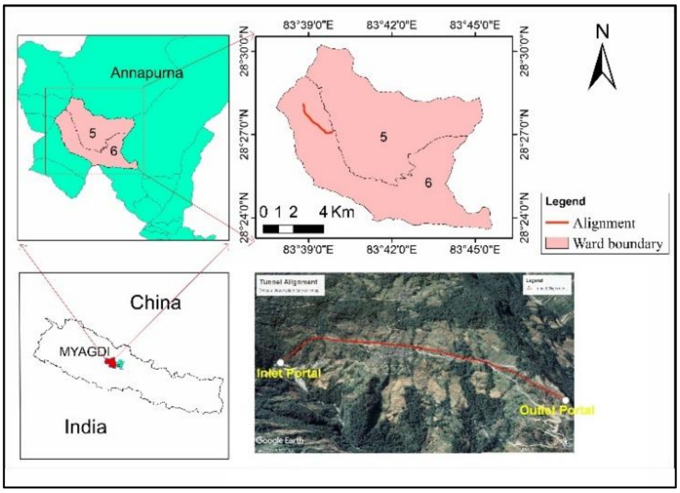

Figure 1, Location map of the study area.

Methodology

With that several methodologies were used to assess the different rock mass properties. The details of the methodology are discussed below.

Calculation of deformation modulus

There exist different types of equations which were derived from the regression analysis. In this study, Q, RMR, and GSI are used to estimate the deformation modulus value along the tunnel alignment. The deformation modulus value is calculated by the equation proposed by Barton (1995), Palmstrom, and Singh (2001), Barton (2002), and Grimstand and Barton (1993).

Barton (2002)

$$\mathrm{Em\ (Gpa)} = 10 \times \left(\mathrm{Q} \times \frac{\sigma c}{100}\right) 1/3 \quad (1)$$

Barton (1983)

$$\mathrm{Em\ (Gpa)} = 10 \log \mathrm{Q}\ (\mathrm{Minimum}) \quad (2)$$

Palmström and Singh (2001)

$$\mathrm{Em(Gpa)} = 8\mathrm{Q}^{0.4} \quad (3)$$

Grimastad and Barton (1993)

$$\mathrm{Em\ (Gpa)} = 25 \log \mathrm{Q} \quad (\mathrm{Average}) \quad (4)$$

$$\mathrm{Em\ (Gpa)} = 40 \times \log \mathrm{Q}\ (\mathrm{Maximum}) \quad (5)$$

Then, the deformation modulus using GSI along the tunnel alignment the equation proposed by Hoek and Brown (1997), Carvalho (2004), Hoek and Diederichs (2006), Beiki et al (2010) and Hoek and Didérichs using E_i.

Beiki et al (2010)

$$\mathrm{Em} = [\tan\,(1.56\ +\ (\ln\,(\mathrm{GSI}))\hat{}2]0.5 \times (\sigma_{ci})1/3 \quad (6)$$

Hoek and Diederichs (2006)

$$\mathrm{Em} = \left[\frac{10^5(1-0.5\mathrm{D})}{1+e^{\left(\frac{(75+25\mathrm{D}-\mathrm{GSI})}{11}\right)}}\right] \quad (7)$$

Hoek and Diederichs using Ei

$$Em = Ei\left[\frac{0.02+(1-0.5D)}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right] \quad (8)$$

Hoek and Brown (1997)

$$Em = \left(\frac{\sigma ci}{100}\right)^{0.5} \times 10\left(\frac{GSI-10}{40}\right) \quad (9)$$

Carvalho (2004)

$$Em = Ei(s)^{0.25}, S = Exp\left(\frac{GSI-10}{9-3D}\right) \quad (10)$$

Similarly, the deformation modulus using RMR along the tunnel alignment the equation proposed by Gokceoglu et al. (2003), Diederichs and Kaiser (1999), Mitri et al. (1994), and Serafim and Pereira (1983).

Gokceoglu et al. (2003)

$$Em = 0.07555\ RMR \quad (11)$$

Diederichs and Kaiser (1999)

$$Em = \sqrt[7]{10^{(RMR-44)/21}} \quad (12)$$

Mitri et al. (1994)

$$Em = 0.1\left(\frac{RMR}{100}\right)3 \quad (13)$$

Serafim and Pereira (1983)

$$Em = 10^{\left(\frac{RMR-10}{40}\right)} \quad (14)$$

Tunnel squeezing analysis

In its research, the different 51 sections of the tunnel were taken based on the problems that arose in the tunnel after the excavation and the rock mass conditions existing tunnel. In its research, the following approaches were used to study the tunnel squeezing phenomenon.

Empirical approaches

Semi-analytical approaches

Numerical modeling approaches

Empirical approaches

Rock mass classification approach

Rock mass classification approaches in tunnel squeezing analysis Singh et al. 1992, and Goel et al. 1995, were used in this research.

Singh et al approach (1992)

Singh et al. 1992 give a clear demarcation line to find out the squeezing and non-squeezing conditions. In its research total of 51 sections of the tunnel, data is collected to delineate whether the squeezing or not by using the Singh et al. concept and to compare the analysis ground condition and real field condition. The following relationships were used for this.

The equation of a line

$$H = 350\ Q^{1/3}\ (m) \quad (15)$$

The compressive strength of the rock mass was calculated by using a relation.

$\sigma cm = 0.7\ \gamma\ Q^{1/3}$Mpa) (H= Overburden height) (16)

Goel et al. (1995) approach

Goel et al. 1995 give an empirical approach to estimating the stability of tunnels based on rock mass number N, depth of overburden H, and tunnel diameter. In its research, the different tunnel section data were taken from the Gharkhola Hydroelectric Project to study the stability and squeezing conditions in the tunnel. The data were calculated using the empirical relationship given by Goel et al. (1995) and a graphical plot was carried out as per the Goel et al. standard.

$$H = (275N^{0.33})B^{-0.1} \quad (17)$$

In which n rock mass number, B = Tunnel diameter, and H = Overburden height.

Semi-analytical approach

The Hoek and Marinos (2000) approach were used as the semi-analytical approach to find the tunnel squeezing. This approach is generally used to evaluate the deformation initiated by squeezing and similarly the determination of support pressure by taking the different data related to tunnel support. As per Hoek and Marinos the tunnel strain value plot against the σ_{cm}/P_o used to evaluate the tunneling issues.

The Hoek and Marinos curve obey the following relationship.

$$\varepsilon = \frac{\delta_i}{d_o} = \left[0.002 - 0.0025\ \frac{P_i}{P_o}\right]\left(\frac{\sigma_{cm}}{P_o}\right) \wedge \left(2.4\ \frac{P_i}{P_o} - 2\right) \quad (18)$$

For the unsupported condition in the above equation, Pi is zero. Then the compressive strength of the rock was calculated by using the following relation.

$$\sigma_{cm} = (0.0034m_i^{0.8})\ \sigma_{ci}\{1.029 + 0.025\ e^{(-0.1m_i)}\}\ GSI$$
(19)

Where,

GSI =Geological Strength Index,

m_i = material constant,

σ_{cm} = Compressive strength of rock mass,

P_i = Internal support pressure,

P_o = In-situ stress,

ε = Tunnel strain,

σ_{ci} =Intact rock strength

In its research, the above-mentioned relationship is used to calculate the tunnel strain, and this is interpreted as per Hoek and Marino's standard to find out the types of squeezing in the tunnel section.

Numerical modelling approaches

There exists several numerical modeling software then in this study Phase2 software was used for the different analyses. Phase 2 is the two-dimensional modeling software that can be used to display the different stability conditions of the rock mass in an underground structure. Phase2 is one of the most familiar modeling software which calculates stress and can add different support structure designs for the underground structure. It uses the two-dimensional finite element method to encircle the opening and extend past the boundary element interface due to automatic generation in the pre-processor. The most common numerical techniques for designing tunnels and subterranean excavations in rock engineering were illustrated by Barala (2016).

Rocscience software is a finite element analysis application that has been used in this study by calculating the required parameters and techniques developed by Hoek and Brown (1997). The growing trend of the rock mass's failure and the support's reaction could be observed in Phase 2 over several of the excavation's subsequent phases.

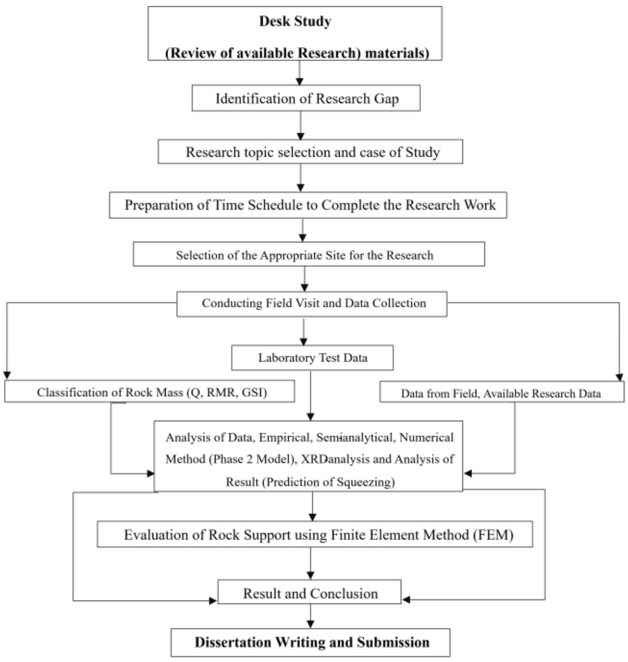

Figure 2, Methodological flowchart of the research work.

Results

Geology of headrace tunnel alignment

Geologically, the area lies in the Lesser Himalaya (Lower Nuwakot Group) region. Which dominantly consists of the green phyllite and white color quartzite of the Kuncha Formation and Fagfog Quartzite respectively. Along the tunnel alignment, phyllite with amphibolite bands and Quartzite were present as the dominant rock types. Then the foliation strikes NW-SE with moderate to steep dipping about 260-490 towards the north. The geological cross-section of the tunnel study section is shown below (Figure 3). The Main Central Thrust is the major tectonic structure that separates the Lesser Himalaya rocks from the Higher Himalaya rocks sequences. In the study area, the MCT lies about 3.6 km upstream of the project site near Dana. In the project area, a few centimeters to a few meters thick shear zones were found which are characterized by crenulation, weak to highly soft materials presence, and bright grey to green clay gouge with the presence of illite and chlorite.

Tunnel deformability study

The different available empirical relations to calculate the deformation modulus of them deformation modulus using Q, GSI, and RMR were used for the study.

Deformation modulus using Q-method

Using this Q method the deformation modulus was calculated by using the empirical relationship given by Grimstad and Barton 1993, Palmstrom and Singh 2001, and Barton 1995. Due to the low value (<1) of Q some applied relations do not give an adequate result. Then the graphical plot of the deformation modulus from different empirical relations along the selected chainage is shown below (Figure 4). According to the chainage of each type of rock, the observed Em values for the relationships indicated above were plotted on a graph. The average, minimum, and maximum Em values for various rock types along the tunnel are displayed in Figure 3 below. Values are derived from Q.

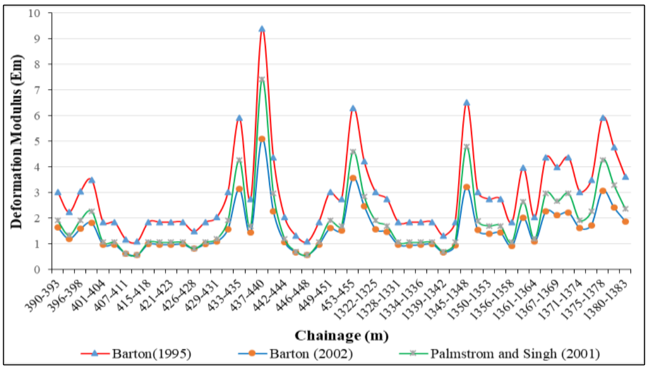

Figure 3, Comparison of deformability value using different empirical methods using Q.

From the above graphical plot (Figure 3) the different values of the deformation modulus were plotted and compared with each other. Among the three applicable empirical relations i.e. Barton 1995, Palmstrom and Singh 2001, and Barton 2002 the sensitivity of the different empirical relations was checked with rock mass type and rock class. In them, Barton 2002 seems less sensitive than Barton 1995 and Palmstrom and Singh 2001.

Deformation modulus using GSI

Similarly, the deformation modulus of rock mass using GSI was also calculated using the different empirical relationships given by rock science researchers. Based on GSI the empirical relation given by Hoek and Brown 1997, Carvalho 2004, Hoek and Diderichs 2006, Beiki et al 2010 and Hoek and Diderichs using Ei were used in its research. Then obtained value of the deformation modulus using different empirical relationships was compared by Plotting them (Figure 4). From this plot, Hoek and Diederichs using Ei seem less sensitive to change in rock class and rock mass type than the other empirical relation to calculating the deformation modulus. So, Hoek and Diederichs give a more satisfactory result compared to other methods.

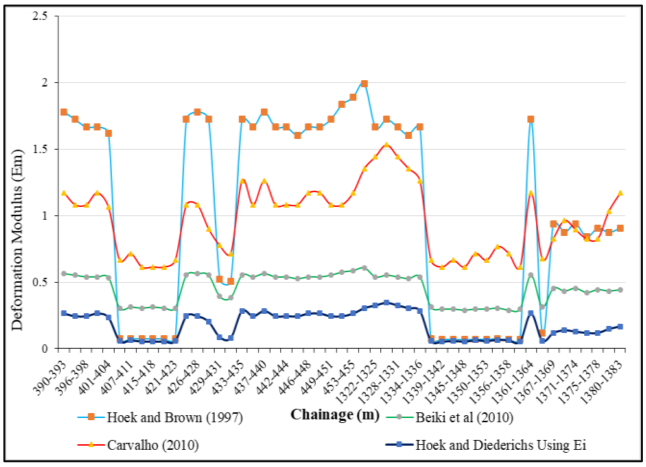

Figure 4, Comparison of deformation modulus from the empirical relationship using GSI.

Deformation modulus using RMR

Similarly, deformation modulus using RMR values were also used to calculate the deformation modulus of the rock mass. Different empirical relationships were used i.e. Serafim and Pereira 1983, Mitri et al. 1994, Diderichs and Kaiser 1999, Gokceoglu et al. 2003, and Palmstrom 2000. From this study (Figure 5), Gokceoglu et al. 2003 seems less sensitive to changes in rock mass types and class than the other equation used to estimate Em. So, it gives more satisfactory results than others.

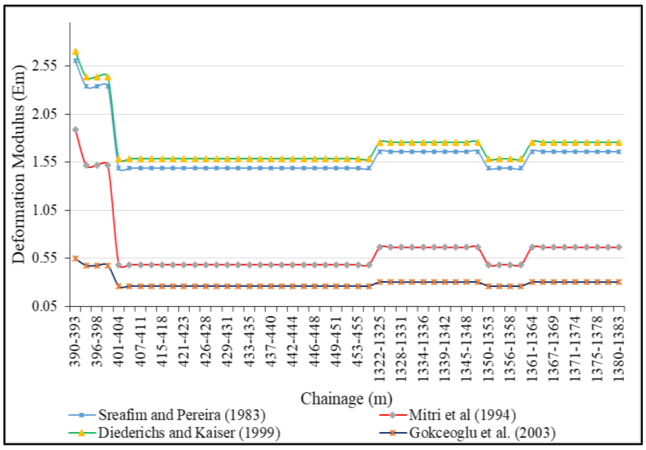

Figure 5, Comparison of deformation modulus values calculated using GSI.

Tunnel squeezing analysis

At locations where the squeezing issue was apparent, the tunnel's deformation was measured. Neither throughout the research period nor when the excavation was taking place were any tests on the parameters related to rock mass taken. Face mapping of the tunnel involved estimating the Q-value and recording the rock and support types. Other factors, such as the complete rock's unconfined compressive strength, Young's modulus, density, and Poisson's ratio, were not examined in the lab. So, all these required parameters were taken from the literature review based on the ground condition in field. In its research, the following approaches were used to study the tunnel squeezing phenomenon.

- Empirical approaches
- Semi-analytical approaches
- Numerical modeling approaches.

Empirical approaches

In its research work, rock mass classification approaches were used.

Rock mass classification approach

To predict tunnel squeeze, the rock mass classification approach was given in the past by different researchers. In this research, the following approaches were used to predict tunnel squeezing.

Singh et al approach (1992)

In the past, Singh et al. gave the concept of finding out the ground condition and whether squeezing occurred or not. In which they take the different tunnel data and develop the standard graphical plot to delineate the squeezing ground conditions. They give a clear demarcation line to differentiate whether the ground is squeezing or not. Then the Q value (Barton 1974) and Height of the overburden were collected. In its study from the graphical plot of the study section (Figure 6) based on rock mass quality Q value of different selected sections of the tunnel, it seems 10% of the area of the tunnel section lies under the non-squeezing zone and 90 % of the area lies under the squeezing zone. The chainage 0+390, 0+396, 0+398, 0+433, and 0+437 lies under the non-squeezing zone which has overburden heights are 133 m, 132 m, 132 m, 142 m, and 143 m respectively. Then the remaining chainage lies under the squeezing zone.

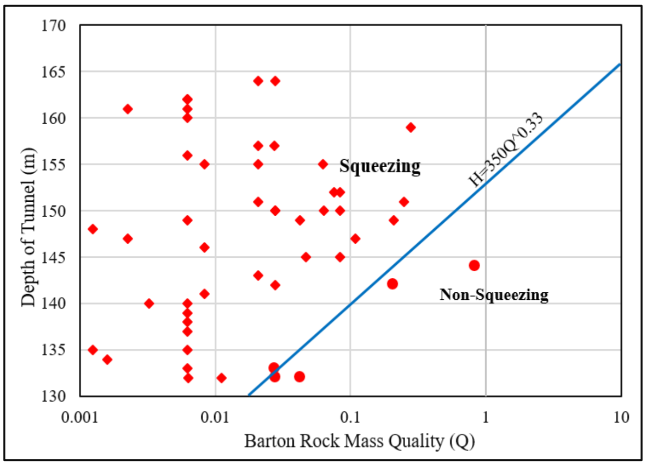

Figure 6, Prediction of the ground condition based on Q.

Goel approach (1994)

The rock mass number N which is referred to as Q with stress reduction factor (SRF) =1 was used by Goel (1994) to build an empirical method. N value was utilized to get around issues and uncertainties in the Q method's ability to rate the parameter stress reduction factor correctly. In its research, we have plotted the information that is currently available in log-log diagrams between N and HB0.1 considering the rock mass number N, depth of overburden H, and tunnel diameter B from the study tunnel. All the selected sections of the tunnel were plotted on the graph given by Goel to delineate whether the ground squeezing or not. A clear demarcation of lines separates this condition.

From the graphical plot, there seems that 41% of the area lies under the severe squeezing area, and about 59% fall under the minor squeezing zone. This result seems more closely related to the ground conditions in the real field.

Semi-analytical approach

Among these different semi-analytical methods, Hoek and Marinos 2000 were used in their research.

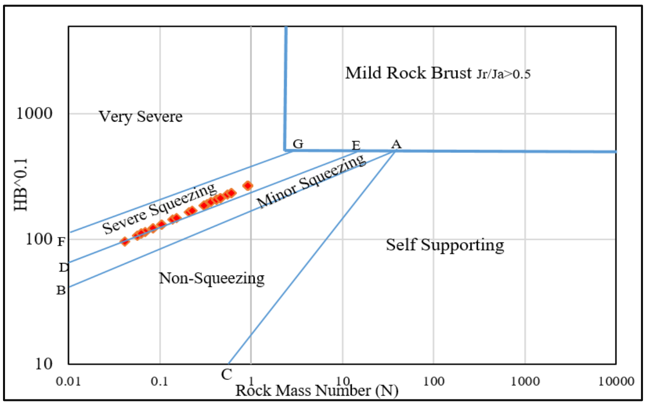

Figure 7, Prediction of the ground using rock mass number N.

Hoek and Marinos 2000 approach

Hoek and Marinos's (2000) squeezing analysis method was applied to the semi-analytical method scenario with correlation to ground conditions. Using various parameters including the intact rock strength, material constant, depth of overburden Geological Strength Index (GSI), rock mass compressive strength, intact rock strength, and support pressure the tunnel strain was computed. The support employed in the tunnel like rock bolts, steel ribs, and shotcrete was used to calculate the tunnel support pressure Pi. Then the tunnel strain Pimax was computed using the Pi value that was determined. In its research in the selected tunnel section, the support was installed so using the Hoek and Marinos concept is better for the evaluation of the tunnel squeezing. To allow the internal support pressure in the tunnel the semi-analytical approach is best. In its research majority of the tunnel sections selected fall inside the < 1% tunnel strain. This means there are few support problems in the tunnel rather than the tunnel squeezing. Some sections fall inside the 1 to 2.5 % which means minor support problems. Then very few sections of the selected section tunnel fall inside the 2.5 to 5% which means the severe types of squeezing. Then the graphical plot of the tunnel strain at zero support pressure condition is shown below (Figure 8).

In the study project at the selected tunnel section, the support was installed so, using this Hoek and Marinos concept the value of the internal support pressure is calculated to find the tunnel strain after the support installation in the tunnel. So, to calculate the internal support pressure (P_{imax}) in the tunnel by considering the different supports used in the tunnel. Then from the calculation, the value of P_{imax} is 2.5 for the whole selected tunnel section. Then, the value of the tunnel strain ranges from 0.002 to 3.40 %. Then the comparison between the tunnel strain at conditions of with support and without support is shown below (Figure 8).

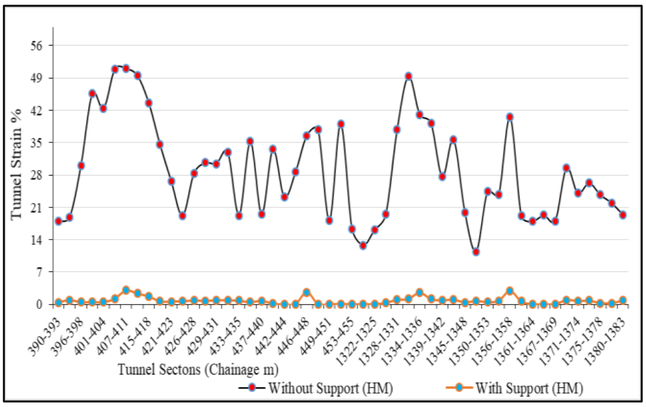

Figure 8, Comparison of the tunnel strain without the support and with support.

This shows that the support helped to control the tunnel failure at a certain point and then further after support installed there still the tunnel strain value is higher in some sections which indicates there is also some defect in the support used and in ground conditions. At chainage 0+407, 0+446, 1+334, and 1+356 there seems to squeeze problem in the field and the calculations. Then the final tunnel strain value is calculated by using the effective value of support pressure. From the Hoek and Marinos (2000) analysis of tunnel squeezing at the different sections of the tunnel different problems of squeezing were found such as few support problems, Minor squeezing problems, and severe squeezing problems (Table 1). The result obtained from this analysis seems more appropriate than the other method of tunnel squeezing prediction with ground conditions.

Numerical modelling approach

In its research work, generally Phase 2 software was used for tunnel deformation modeling. Based on the tunnel squeezing condition or based on tunnel strain value the different tunnel sections on which high squeezing occurs were selected for the modeling of tunnel deformation. The chainage 0+407 m, 0+446 m, and 1+334 m were selected for the deformation analysis on which maximum squeezing occurred.

Table 1, Tunnel strain and types of problems at selected section of Tunnel.

Chainage (m)	Tunnel Strain (ε%)	Remarks
390 to 404	0.45 – 0.95	Few support problems
404 to 407	1.27	Minor squeezing problems
407 to 411	3.04	Severe squeezing problems
411 to 415	1.74	Minor squeezing problems
415 to 446	0.02 to 0.93	Few support problems
446 to 448	2.66	Severe squeezing problems
448 to 458	0.01 to 0.07	Few support problems
1322 to 1331	0.06 to 0.94	Few support problems
1331 to 1334	1.18	Minor squeezing problems
1334 to 1336	2.56	Severe squeezing problems
1336 to 1339	1.15	Minor squeezing problems
1339 to 1356	0.49 to 0.97	Few support problems
1356 to 1358	2.51	Severe squeezing problems
1358 to 1383	0.02 to 0.96	Few support problems

Chainage 0+407: The excavated rock mass consists of thinly foliated, slightly to highly weathered, dark grey to black phyllite (pelitic) with little quartz veins. The existing rock mass is highly deformed and crushed, and the joints are low to medium spacing, with a medium persistence of >1m. Slightly weathered, smooth. Then the excavated rock mass of the left wall below the sp level is highly sheared and deformed in conditions. But the right wall seems slightly better than the left wall of the tunnel. Then the details of the rock mass parameter used in this modeling are shown below (Table 2).

The parameter was used in the numerical modelling of tunnel deformation, then the result observed is shown below (Figures 9, 10 11 and 12).

Using the finite element method several ground conditions were modeled, and the results are shown as deformation behavior under the different low to medium stress conditions. In this study, overburden mass seems to correspond to the highest and lowest stress level range.

Table 2, Rock mass input parameter for chainage 0+407.

Parameter type	Rock mass (sheared)		Rock (mass sound)	
	Peak	Residual	Peak	Residual
Types of rock	Phyllite (Pelitic)		Phyllite (Pelitic)	
UCS (Mpa)	13		20	
Unit weight (KN/m^3)	26		26	
GSI	6	6	16	16
m_i	7	7	7	7
D	0	0.5	0	0.5
m_b	0.235	0.0759	0.3362	0.12224
S	2.60e-0	3.15e-006	7.91e-00	1.19e-00
A	0.612	0.61921	0.56110	0.561101
Poisson ratio (θ)	0.2	0.2	0.2	0.2

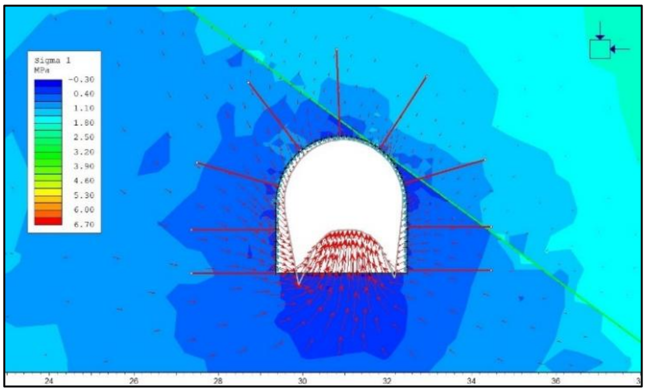

Figure 9, the principle stress (σ_1) applied Ch. 0+407 m

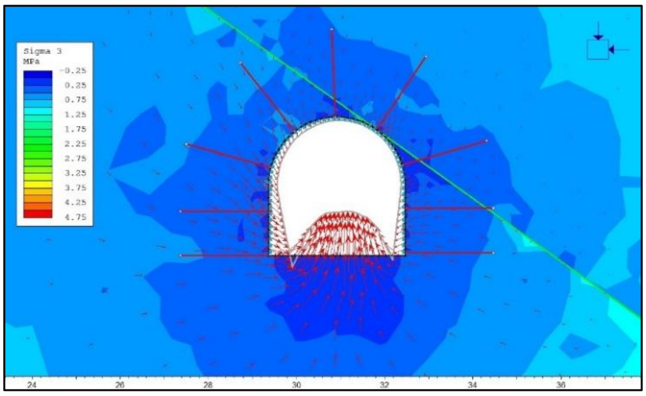

Figure 10, the principle stress (σ_3) applied at Ch. 0+407 m

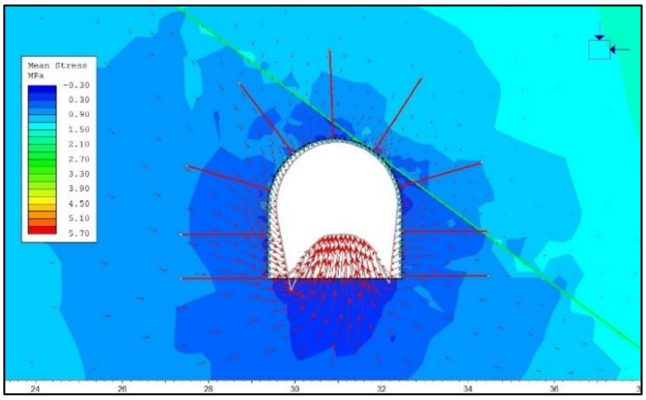

Figure 11, the mean stress applied at Ch. 0+407 m

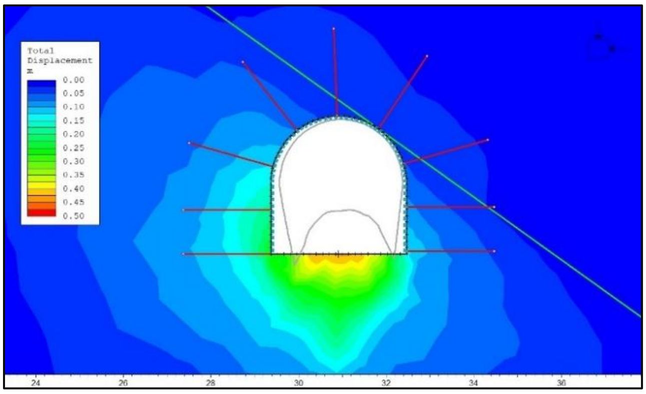

Figure 12, Tunnel deformation boundary at ch. 0+407 m

Finally, in this section, the value of the total maximum displacement obtained is 0.432803 m.

Chainage 0+446: The rock mass consists of thinly foliated, slightly to high weathering dark grey to black phyllite (pelitic) with branching quartz veins. The rock mass is highly deformed and crushed phyllite. J1 is low to medium spacing, medium persistence of >1m, slightly weathered, and smooth to the rough planar surface. Other crushed joints are low to medium persistence, and slightly weathered conditions. The excavated rock mass of the right wall below the sp level is lightly shared and deformed in conditions. Most of the properties of the rock mass are like the rock mass condition of chainage 0+407 m. The details of the rock mass parameter used in the modeling are shown in the table below (Table 3).

Table 3, Rock mass input parameter for chainage 0+446 m.

Parameter type	Rock mass (sheared)		Rock (mass sound)	
	Peak	Residual	Peak	Residual
Types of rock	Phyllite (Pelitic)		Phyllite (Pelitic)	
UCS (Mpa)	14		21	
Unit weight (kN/m3)	26		26	
GSI	7	7	18	7
mi	7	7	7	7
D	0	0.5	0	0
mb	0.0796	0.09637	0.37431	0.079637
s	3.60e-0	3.64e-006	0.00011	3.6045e-00
a	0.61151	0.611508	0.54999	0.611508
θ	0.2	0.2	0.2	0.2

The above-tabulated parameter was used in the numerical modelling of tunnel deformation then the result observed is shown in Figures 13, 14, 15, 16. Using the finite element method several ground conditions were modeled, and the results are shown as deformation behavior under the different low to medium stress conditions. In this study, overburden mass seems to correspond to the highest and lowest stress level range. The different above-mentioned rock mass parameters of them are collected from the literature review like ground condition, some of them were taken from the project site office and some were from the field measurement and test. In this section, the value of the total maximum displacement obtained is 0.200786 m.

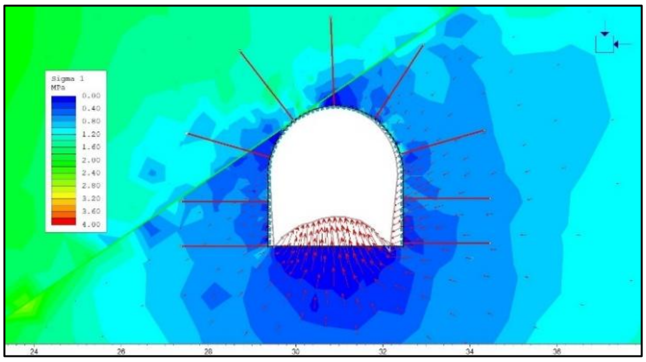

Figure 13, the principle stress (σ_1) applied Ch. 0+446 m

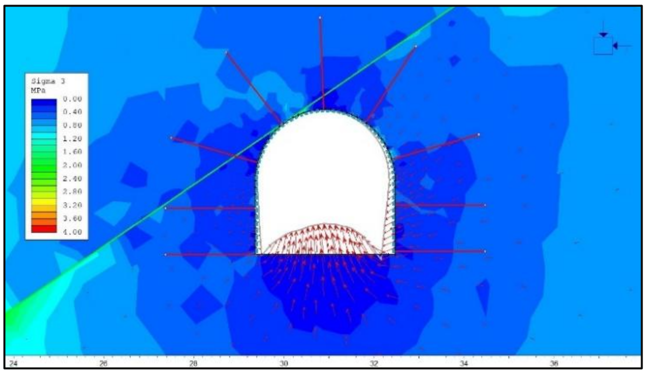

Figure 14, the principle stress (σ_3) applied Ch. 0+446 m

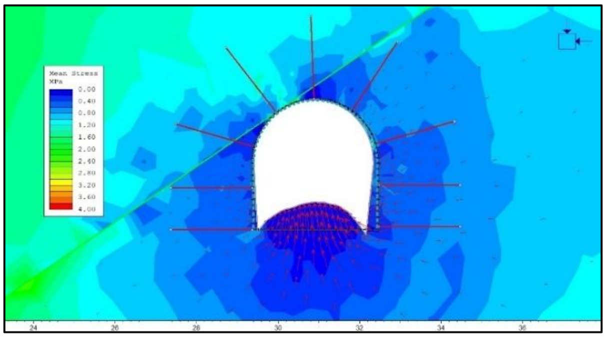

Figure 15, the mean stress applied at Ch. 0+448 m

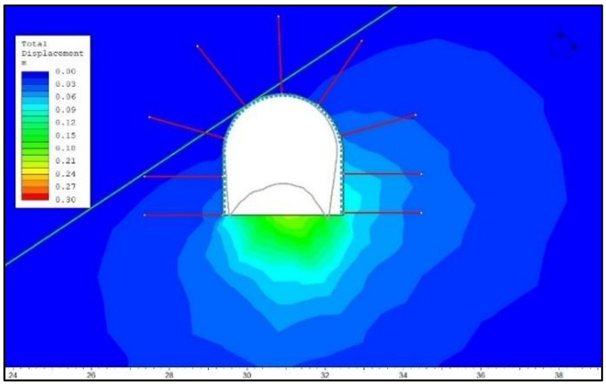

Figure 16, The deformation contour at Ch. 0+446 m

Chainage 1+334: The rock mass excavated consists of thinly foliated, mildly to heavily worn dark grey to black phyllite (pelitic) with extensively branching quartz veins, which makes up the excavated rock mass. The rock mass is severely crushed and distorted and J1 has low to medium spacing and persistence of more than 1 m, lightly worn, open joint, smooth planar surface, and J2 and J3 with low persistence (<1 m). The Rock mass is damp to little dripping in conditions. In its study, this section does not seem much of a drastic variation in the properties of the materials between the left and the right wall of the tunnel section. So, in the modeling of the deformation of the tunnel the same properties were used in both walls of the tunnel. The summary of the rock mass input parameter for the modeling is given below (Table 4).

Table 4, Rock mass input parameter for chainage 1+334 m.

Parameter type	Rock mass (sheared)	
	Peak	Residual
Types of rock	Phyllite (Pelitic)	
UCS (Mpa)	14	
Unit weight (kN/m3)	26	
GSI	11	11
mi	7	7
D	0.5	0.5
mb	0.101045	0.325358
s	7.0206e-006	2.93922e-00
a	0.579839	0.523561
Poisson ratio (θ)	0.2	0.2

The above-tabulated parameter was used in the numerical modelling of tunnel deformation then the result observed is shown below (Figures 17, 18, 19, 20). Using the finite element method several ground conditions were modeled, and the results are shown as deformation behavior under the different low to medium stress conditions. In this study, overburden mass seems to correspond to the highest and lowest stress level range. The different above-mentioned rock mass parameters of them are collected from the literature review like ground condition, some of them were taken from the project site office and some were from the field measurement and test.

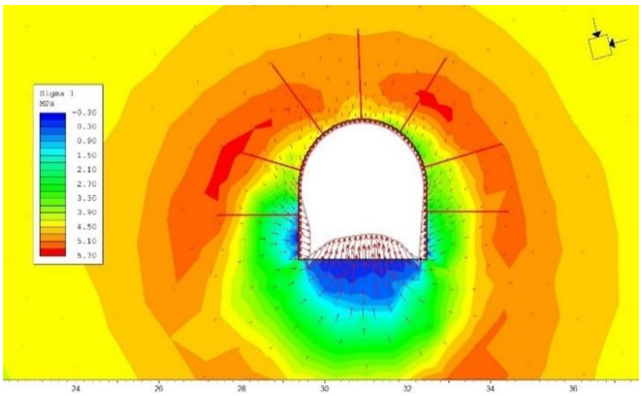

Figure 17, The principle stress (σ_1) applied at Ch.1+334 m

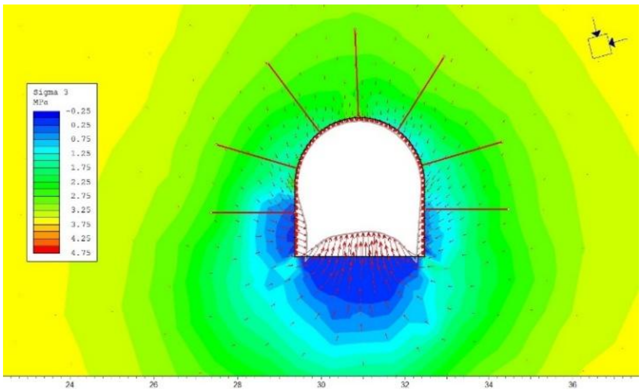

Figure 18, the principle stress (σ_3) applied at Ch. 1+334 m

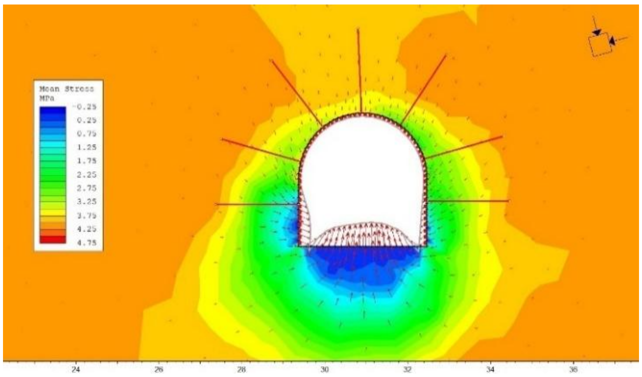

Figure 19, the mean stress applied at Ch. 1+336 m

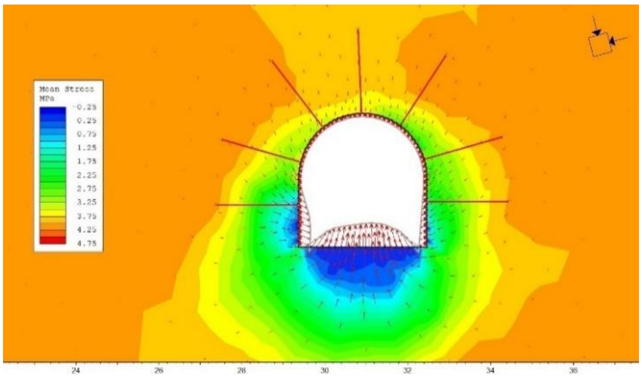

Figure 20, the deformation contour at Ch. 1+334 m

Finally, in this section, the value of the total maximum displacement obtained is 0.101413 m.

Table 5, Rock mass input parameter for chainage 1+334 m.

S.N.	Chainage (m)	Total maximum displacement (m)
1.	0+407	0.432803
2.	0+446	0.200786
3.	1+334	0.101413

Discussion

To study the deformability at different tunnel section rock mass classification was conducted from three different methods i.e. Rock Quality Index (Q), Geological Strength Index (GSI), and Rock Mass Rating (RMR) were carried out in which the Q value ranges from 0.0016 to

0.25, GSI ranges from 11 to 28, and RMR value ranges from 17 to 19. In my research using the different well-accepted equations given by Grimstad and Braton (1993), Palmstrom and Singh (2000), Barton (2002), Hoek and Brown (1997), Beiki et al (2010), Carvalho (2004), Hoek and Diderichs using Ei, Sreafim, and Pereira (1983), Diderichs and Kaiser (1999), Mitri et al (1994) and Gokceoglu et al (2003) were used to estimate the value of deformation modulus for the different rock types and rock class along the tunnel section. The different values obtained from the calculation were plotted to know the similarity or disparity obtained from the rock types and classes of rock. Due to the various values of the deformation modulus obtained from the various empirical formulas there exists a big challenge to pick up the value which is more appropriate to design the tunnel support and the various civil engineering structures. In the past, such kinds of studies were carried out by Panthee (2016), Kayabasi (2000), and Hoek and Diderichs (2005) but the significant result was not formed.

The values of deformation modulus in some empirical relations show higher sensitivity and some of them display lesser sensitivity with the class of the rock mass. In its study, the deformation modulus value range of some equations gives a lesser deformation modulus value and some of them show higher values of the deformation modulus with the rock mass and class. In its research using the Q value the deformation modulus obtained from Barton, 2002 shows less sensitivity to the Q value than Barton (1995) and Palmstrom and Singh (2002). Then the deformation modulus using GSI by using the empirical relationship Hoek and Diderichs using Ei, Beiki, et al, Carvalho, Hoek, and Brown the Deformation modulus value using the Ei shows less sensitivity than the deformation modulus from Beiki et al, Carvalho, Hoek and Brown (1997) with the GSI value. Similarly, using RMR values from the different equations Gokceoglu et al (2000), Mitri et al (1994), Sreafim and Pereira (1983), and Diderichs and Kaiser (1999) of Gokceoglu et al (2000) show less sensitivity than the other empirical equations with RMR value. From its research, I discovered that the deformation modulus value increases along with rising Q, GSI, and RMR values. When the value of the deformation modulus changes in a highly sensitive equation the accompanying change in the value of Q, GSI, and RMR will be significant. The equation given by Barton (2002) based on Q, Hoek, and Diederichs using Ei based on GSI and Gokceoglu et al. (2003) based on RMR are the best for the deformability estimation because all these equations show less sensitivity with the rock mass class.

The tunnel squeezing analysis uses different approaches, i.e. empirical approaches in which Singh et al. gave 90 % area of the tunnel under squeezing and Goel (1974) gave 41 % of the tunnel area of a tunnel under the severe squeezing and 59 % under the minor squeezing zone. In semi-analytical approaches Hoek and Morinas (2000) were used to find the tunnel squeezing. This method is found to be more acceptable than the other method. Using the numerical modeling approach tunnel squeezing was carried out in which Finite Element Method (FEM) was used. Using this method, total displacement was estimated and then the amount of the tunnel deformation using numerical modeling. The maximum total displacement was found in the chainage 0+407 i.e. 0.432803 m, then at chainage 0+446 and 1+334 the total displacement was found 0.200786 m and 0.10143 m respectively. In the selected tunnel section, most of them suffered from support problems.

Conclusion

The intricate challenges associated with tunnel deformation and squeezing in Gharkhola Hydroelectric Project, emphasizing the paramount importance of accurately estimating the deformation modulus (Em) of rock masses. By employing several empirical approaches based on the values of Q, GSI, and RMR reported by different studies where the equations proposed by Barton (2002), Hoek and Diederichs (using Ei), and Gokceoglu et al. (2003) exhibit superior reliability due to their diminished sensitivity to rock mass variability, thereby facilitating more precise predictions of tunnel behavior. The investigation reveals that severe squeezing phenomena, manifesting as tunnel wall closures up to 0.432803 meters, predominantly occur in sections with overburden heights approximating 134 meters, corroborating the necessity for robust support systems. Furthermore, the synthesis of empirical analyses, Hoek and Marinos' semi-analytical approach, and Phase2 numerical modeling underscores the indispensability of integrating diverse techniques to mitigate uncertainties and enhance predictive accuracy. This study not only advances the comprehension of rock mass deformability in geologically complex terrains but also proffers pragmatic insights for optimizing tunnel design and support strategies in analogous hydroelectric ventures. Ultimately, the research advocates for the judicious selection of input parameters and the amalgamation of multiple analytical paradigms to ameliorate the fidelity of squeezing assessments and ensure the structural integrity of subterranean excavations.

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