Proceedings of 12th IOE Graduate Conference
Peer Reviewed
ISSN: 2350-8914 (Online), 2350-8906 (Print)
Year: 2022 Month: October Volume: 12

Rock Engineering Assessment Along the Headrace Tunnel of Khimti-2 Hydroelectric Project

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Abstract
Rock engineering is the study of both rock mechanics and engineering geology. The assessment of the parameters of the rock engineering is essential for proper planning, design and construction of any underground structures. This article focuses on the study of these parameters and their effect on tunnel stability. Strength and deformability are the two main parameters of rock mass that define the mechanical behavior of rock. The strength and deformability of the rock mass is calculated using different empirical methods that are commonly in use. The engineering geological parameters such as joint characteristics, in-situ stress, and ground water condition are also studied in detail. The in plane and out of plane stresses are calculated at different chainages considering gravity as well as tectonic effect. The possibility of water leakage is studied at selected chainages and specific leakage is calculated and checked with the limiting value. Norwegian Confinement Criteria is also checked to study about the possibility of unlined/shotcrete lined tunnel.To study about the effect of rock engineering parameters on tunnel stability, parameters such as UCS, GSI, Young's modulus of elasticity, poisson's ratio are varied on Phase 2 software and corresponding value of deformation and plastic radius are compared.

Keywords
Rock engineering,rock engineering assessment,deformability

1. Introduction

Nepal is a country with complex geology i.e fault and fold structure and tunneling is an emerging field in context of Nepal. The high variation of elevation (north-south) in small area leads to the different topographical and geological condition even if our site coverage is less. Especially in the lesser Himalayan zone, construction of underground structures is a very challenging task. To find out the appropriate solutions to these problems, good knowledge about the rock engineering is required.

Rock engineering is the study 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 [1]. The strength of the rock is determined by its mineral composition and its orientation. The geological features such as major weakness zones, jointing condition, in-situ stress condition, weathering condition and ground water condition also must be studied properly. In order to assess the engineering properties of the rock mass, proper surface and sub-surface investigation must be carried out. The tests also should be carried out in lab as well as in the field for the establishment of the input parameters required for the stability analysis of any underground structures.

To sum up, proper study of rock engineering issues is required in proper planning, design and construction of the tunnel and any other construction works in/on rock.

1.1 Statement of Problem

Majority of the tunnel failure cases are related to ground condition. So, correct assessment of the mechanical behavior of the rock mass and the engineering geology of the area around tunnel alignment is very important to ensure stability as well as safety.

1.2 Objectives

The main objective of the present study is to assess the rock engineering parameters along the headrace tunnel of Khimti-2 Hydroelectric Project. The broad objective will be obtained from integration of the following objectives.

- To study about the mechanical behaviour and geological parameters of the rockmass.
- To discuss about the effect of rock engineering parameters on tunnel stability.

2. Literature review

Rock engineering assessment can be better demonstrated by the figure given below.

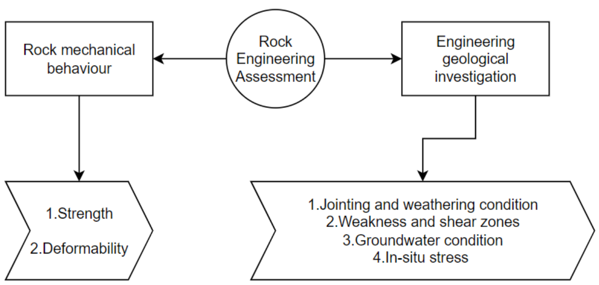

Figure 1: Parameters for rock engineering assessment

2.1 Rockmass strength

Rock mass strength is an ability to resist stress and deformation. It is difficult to measure in-situ rock mass strength. So, it has to be estimated from the lab test results of intact rock incorporating other geological observations.The rock mass strength is calculated using different empirical methods. Bieniawaskil(1993)

$$\sigma_{cm} = \sigma_{ci} \times \exp\left(\frac{RMR - 100}{18.75}\right) \tag{1}$$

Hoek et al. (2002)

$$\sigma_{cm} = \sigma_{ci} \times \frac{(m_b + 4s - a(m_b - 8s))(m_b/4^{+s})^{a-1}}{2(1+a)(2+a)} \tag{2}$$

Barton (2002)

$$\sigma_{cm} = 5\gamma \times Q_c^{1/3} = 5\gamma \left[\frac{\sigma_{ci}}{100} \times 10^{\frac{RMR-50}{15}}\right]^{1/3} \tag{3}$$

Singh et al.(1992)

$$\sigma_{cm} = 0.7 * \gamma * Q^{1/3} \tag{4}$$

Panthi (2017)

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.6}}{60} \tag{5}$$

2.2 Rockmass deformability

Rock mass deformation modulus can be determined in field by different tests but that is very time consuming. So, the Elastic modulus of intact rock is first determined in lab and the rock mass deformation modulus is determined by using different empirical equations.

Serafin and Pereira (1983)

$$E_m = 10^{\frac{(RMR-10)}{40}} \tag{6}$$

Hoek and Diederichs(2006)

$$E_m = E_{ci} \times \left[0.02 + \frac{(1 - D/2)}{(1 + e^{\frac{(60+15D-GSI)}{11}}}\right] \tag{7}$$

Hoek and Brown (1997)

$$E_m = \sqrt{\frac{\sigma_{ci}}{100} \times 10^{\frac{GSI-10}{40}}} \tag{8}$$

Barton(2002)

$$E_m = 10 \times \left[\frac{Q \times \sigma_{ci}}{100}\right]^{\frac{1}{3}} \tag{9}$$

Panthi (2006)

$$E_m = E_i * \sigma_{cm}/\sigma_{ci} = \frac{\sigma_{ci}^{0.6}}{60} \times E_i \tag{10}$$

2.3 In-situ stresses in rock mass

In-situ stress in rock mass is caused by gravity, topography, plate tectonics and residual stress.

$$\text{Vertical stress } (\sigma_v) = \gamma h \tag{11}$$

$$\text{Horizontal stress } (\sigma_H) = \frac{v}{v-1}\gamma h + \sigma_{tect} \tag{12}$$

Tectonic stress is governed by the plate tectonics. Tectonic stress in central Nepal is mostly oriented in north south direction and fairly towards west in eastern part. Tectonic stress is the main reason for stress anisotropy. Topographic stress is the stress caused by the topographic condition. The stresses that are locked into the rock material during early stages of its geological history is called residual stress.

2.4 Groundwater inflow and leakage estimation

In an unlined/shotcrete lined tunnel, it is important to understand the behavior of rock mass when exposed to water pressure [2].

A relationship established between specific tunnel leakage (qt),joint set number(Jn), joint roughness number(Jr) and joint alteration number(Ja) is expressed by

$$q_t = \frac{f_a \times H \times J_n \times J_r}{J_a} \tag{13}$$

Where, fa is a joint permeability factor with unit l/min/m2 and H is the static water head . The joint permeability factor factor varies from 0.001 to 0.25. The joint permeability factor (fa) is related to joint spacing (Js), joint persistence (Jp) and the shortest perpendicular distance (D) from the rock slope topography to valley side tunnel roof [3].

$$f_a = \frac{J_p}{D \times J_s} \tag{14}$$

2.5 Norwegian Confinement Criteria

In an unlined/shotcrete lined tunnel, water gives pressure to the rock mass surrounding the tunnel equal to the water head at the point of consideration. To balance this water pressure, there must be sufficient vertical and lateral cover as given by the Norwegian criteria for confinement. The factor of safety for the vertical and lateral cover is expressed by the given formula

$$FoS_1 = h \times \frac{\gamma_r * cos\alpha}{P_w} \tag{15}$$

$$FoS_2 = L \times \frac{\gamma_r * cos\beta}{P_w} \tag{16}$$

$$Pw = \gamma w \times H \tag{17}$$

Where, h is the vertical rock cover above tunnel, H is the hydrostatic head acting in the tunnel, γw is the specific unit weight of water, γr is the specific unit weight of the rock, and α is the inclination of shaft/tunnel with respect to horizontal plane, L is shortest distance from the ground profile to the tunnel location and β represents the angle of valley side slope with respect to horizontal plane.

3. Research Methodology

The main aim of this research work is to study about the rock engineering parameters that influence the stability of the headrace tunnel of the Khimti-2 hydroelectric project. To fulfil this objective, the following research methodology is adopted in the study as shown in the flow diagram below.

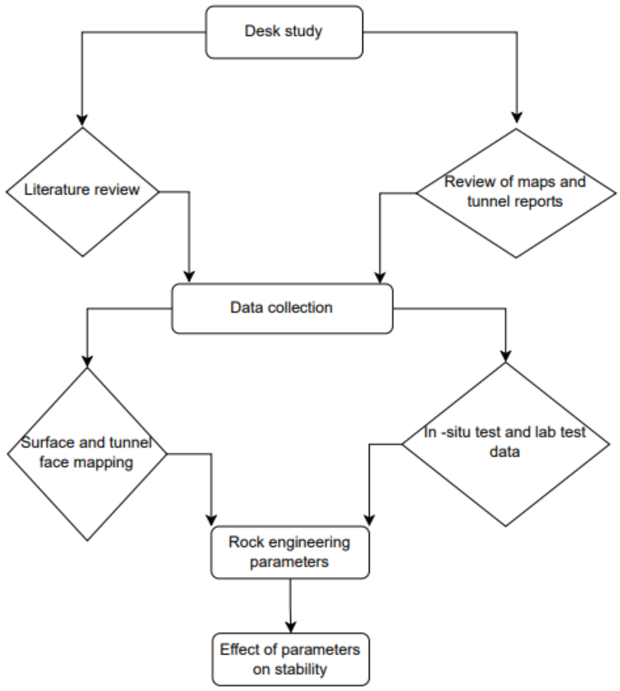

Figure 2: Flow diagram for research methodology

3.1 Desk study

Desk Study includes literature review about different rock engineering parameters and their effect on tunnel stability . It also involves the study of topographical and geological map of the project area. Different reports like feasibility report, basic design report and progress report were also taken as references.

3.2 Data collection

Different types of data required for the assessment of rock engineering parameters are collected from the site. Data of face mapping was collected from the project and surface mapping was carried out during the site visit. Lab test data of rock sample and in-situ stress data was collected from the project and the data that are not available in the project are extracted from the literature review of the nearby hydropower projects.

3.2.1 Study area for data collection

Khimti-2 Hydroelectric Project (KH2HEP) is a RoR project with an installed capacity of 48.8MW. This project is located in Jiri Municipality and Tamakoshi Rural Municipality of Dolakha District and Gokulganga Rural Municipality of Ramechhap District . The gross head of the project is 355m and net head is 342.92m.The project is located in the border of Ramechhap and Dolakha District of Nepal. The project is being developed on Khimti River (a major tributary of Tamakoshi River).

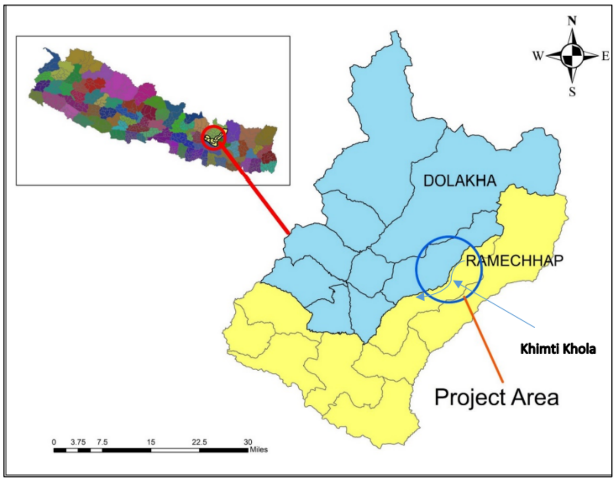

Figure 3: Location of Khimti-2 Hydroelectric Project

3.2.2 Geology of the Project Area

Geologically, the project site is located within the Lesser Himalayan Midland zone of Central Nepal. These units in the project area are accompanied tectonically by the Jiri thrust, Midland Thrust and Vicholo Thrust. Main rocks in this region are augen gneiss, banded gneiss, phyllitic schist and metasandstone. In particular, the project area is dominated by augen gneiss, schist and banded gneiss. This region is located near Midland thrust fault, with the occurrence generally NE-SW and dipping northwestwards. The rock types found in the site are schist and augen gneiss.

Bed rock is rarely exposed around the left bank of the project area as it is covered by thin to thick overburden material of colluvium deposit. Augen gneiss is well exposed at right bank of the Khimti River along the headrace tunnel alignment, whereas the surrounding hillslope of the left bank is fully covered by thick colluvium and landslide deposit. The schist is present in alternative repetition with gneiss having different interval parallel to the foliation plane of bed rock. The augen gneiss is slightly to highly weathered, massive to foliated, jointed blocky and seamy. Three plus random joint sets are predominant within the whole project area. Among which two nearly vertical joint sets are developed locally in different areas. Roughness is rough, planer to undulating, which consists silt as infilling material. Schist is highly weathered, thinly foliated, sheared and deformed. Roughness of joint set is rough irregular to undulating, whereas in some part the joint roughness is rough planer.

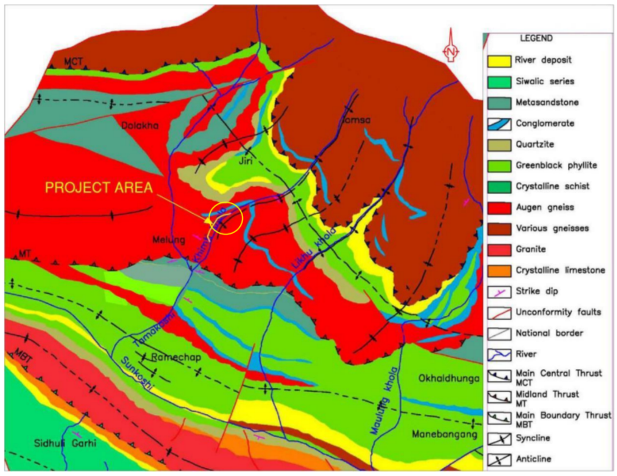

Figure 4: Regional Geological Map of Project area (Source:Basic Design Report)

3.3 Rock engineering parameters

Based on the data collected from the site, lab test data and data from literature review, different rock engineering parameters are calculated. Rockmass mechanical parameters are calculated from given strength and young's modulus of elasticity of intact rock using different empirical relations. The geological parameters are also calculated similarly. The value of stresses are calculated using overburden and the tectonic stress data obtained from literature review. The probability of leakage of unlined or shotcrete lined tunnel is also evaluated. The topography above the headrace tunnel is evaluated using Norwegian confinement criteria for the possible use of unlined or shotcrete lined tunnel.

3.4 Effect of parameters on stability

To study about the effect of rock engineering parameters on the stability of tunnel, Phase 2 software is used. Using Phase 2, the value of maximum radial deformation of tunnel and plastic radius is calculated by varying the rock engineering parameters. The values of rock engineering parameters such as uniaxial compressive strength of rock, poisson's ratio, young's modulus of elasticity and geological strength index values are varied to study their effect on tunnel stability .

4. Results and Discussions

4.1 Rockmass Mechanical Properties

Rock mass strength has been calculated using five empirical methods. Among these methods, Singh et al.(1992) gives the minimum value whereas Barton(2002) gives the maximum value. Panthi(2017) and Hoek et al.(2002) gives medium value. Deformation modulus of the rock mass also has been calculated using five methods. Among these methods Serafin and Pareira(1983) gives the maximum value and Hoek and Diederich(2006) gives the minimum value.Since Panthi(2006) gives the average value and also the formula was formulated based on the research carried out on the Himalayan geology, the value obtained from Panthi(2006) better approximates our site condition.

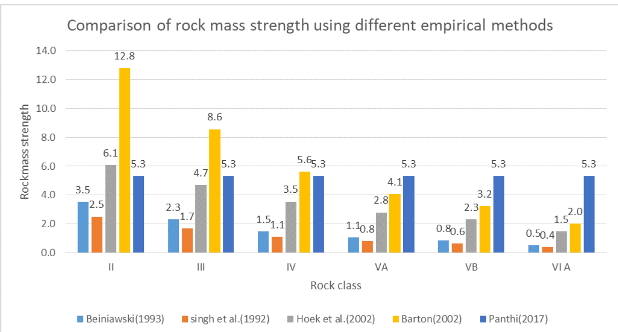

Figure 5: Comparison of rock mass strength using different empirical methods

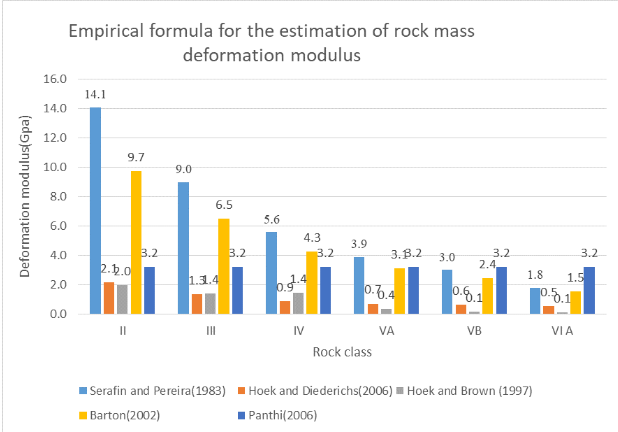

Figure 6: Comparison of deformation modulus using different empirical methods

4.2 In-situ Stress Condition

In-situ stress in rock mass is caused by gravity, topography, plate tectonics and residual stress. The topographic stress is calculated using unit weight of rock and overburden at respective sections. Since in-situ stress measurement has not been carried out yet, the tectonic stress is determined from the nearby project with similar geological environment. The resultant horizontal tectonic stress at khimti-I is estimated to be 3Mpa and the mean value direction of the tectonic stress at this region is N15W [4].As Khimti-I is the nearest project with similar geological environment, we will consider this value for our calculation.

Table 1: Calculation summary of in-situ stress

chainage	Vertical Stress (Mpa)	In plane Horizontal Stress(Mpa)	Out of plane Horizontal stress(Mpa)
000+300	5.27	1.34	2.84
000+800	5.27	3.49	0.68
001+600	3.95	3.35	0.53
002+250	0.92	2.75	0.45
002+400	3.16	3.00	0.70
003+200	6.85	3.41	1.11
004+000	4.21	3.12	0.82
004+800	3.56	2.41	1.38
005+400	8.03	2.91	1.88
005+600	5.79	2.66	1.63
006+300	1.98	0.24	3.20

4.3 Leakage estimation

A hundred meter stretch of headrace tunnel, chainage from 000+265 to 000+355, is taken for the assessment of the leakage potential. The average value of specific leakage for this section is 7.84 l/min/m. A leakage limit maximum up to 1–1.5 l/min/m tunnel is recommended, which is achievable and is very cost effective solution [3]. All the values of specific leakage in this section are greater than this value .So, Proper measures should be applied for the control of leakage in this tunnel section.

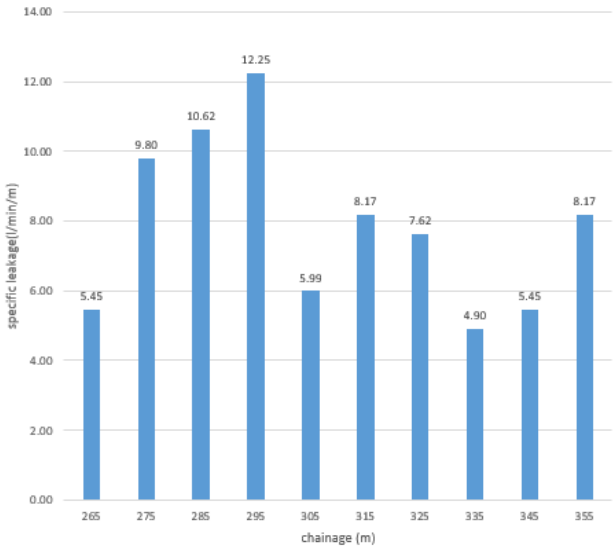

Figure 7: Leakage estimation in unlined headrace tunnel

4.4 Norwegian Confinement Criteria for unlined/shotcrete lined tunnel

Five chainages are selected for checking the factor of safeties that fulfil the Norwegian Confinement Criteria. Usually, the factor of safety for the cover is recommended as 1.3. Both factor of safeties i.e. for vertical cover (FoS1) and for lateral cover (FoS2) respectively are greater than 1.3. This concludes that the both the covers satisfy the criteria for required confinement.

Table 2: Factor of safeties for Norwegian Confinement Criteria

Ch.	000+800	002+400	003+200	004+000	005+600
H	10	10	15	20	30
Pw(Mpa)	0.10	0.10	0.15	0.20	0.30
γ_w	0.02634	0.02634	0.02634	0.02634	0.02634
h	142.25	92.7	202.45	155.85	229.05
α	0	0	0	0	0
β	6.5	27.5	9.1	11.3	21.7
L	141.35	81.25	199.15	157.15	212.4
$L\cos\beta$	140.44	72.07	196.64	154.10	197.35
FOS1	37	24	36	21	20
FOS2	37	19	35	20	17

4.5 Jointing Condition

Joint is a discontinuity plane along which there has not been any displacement. The area mainly consists of three plus random joint sets. According to the surface mapping data, the spacing of the joints is 0.2 to 2m. The persistence of the joint in the area is 3 to 20m. The aperture is 1 to 5mm to greater than 5mm with soft clay infilling. The roughness of the joint is slightly rough to rough. The joints in the exposed surface are mostly slightly weathered.

Table 3: Joint properties from surface mapping

S.N	Spacing	Persistence	Aperture	Roughness	Infilling	Weathering
1	0.6-2m	10-20m	1-5mm	S.rough	clay	S.weathered
2	0.6-2m	10-20m	1-5mm	S. rough	clay	S.weathered
3	<60mm	>20m	>5mm	slicken sided	clay	S.weathered
4	0.2-0.6m	10-20m	>5mm	rough	none	S.weathered
5	0.2-0.6m	10-20m	1-5mm	S. rough	clay	S.weathered
6	0.2-0.6m	3-10m	> 5mm	rough	clay	M.weathered
7	0.6-2m	10-20m	>5mm	rough	clay	S.weathered

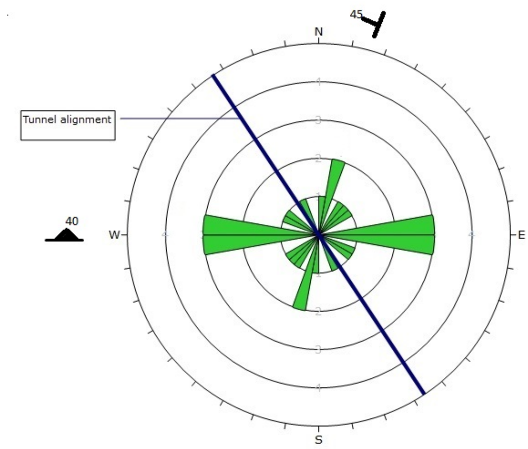

Figure 8: Tunnel alignment based on joint orientation

4.6 Factors affecting tunnel stability

To study about the effect of different rock engineering parameters on the stability of the tunnel, these parameters are varied in the Phase 2 software and the corresponding effect on total deformation and plastic radius is studied. The parameters like poisson's ratio, young's modulus of elasticity, UCS of intact rock and GSI values are varied and the graph is plotted to study the sensitivity of the parameters on deformation and plastic radius.

Table 4: Input parameters used in Phase 2

Properties	Value	Unit
UCS	36.73	Mpa
Poisson's ratio	0.1	
Unit weight	26.34	kN/m3
Intact rock constant(mi)	28	
Ei	22	Gpa
Rock type	Augen Gneiss	
Initial element loading	Field stress only	
Failure criterion	Generalized Hoek and Brown	
Material type	elastic/plastic	

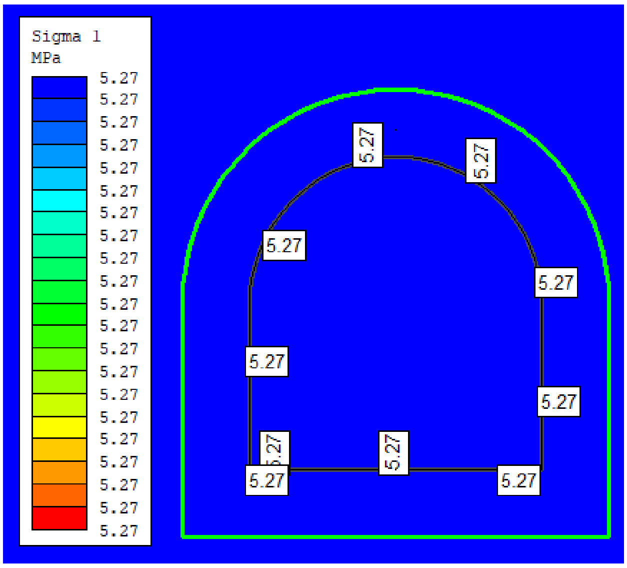

Figure 9: Tunnel section used in Phase 2 (4.3*4.6)

4.6.1 Poisson's ratio

The variation of deformation and plastic radius with respect to poisson's ration is shown in the graph below. As the poison ratio increases, the deformation in the tunnel decreases. The plastic radius seems less sensitive to the change in poisson's ratio.

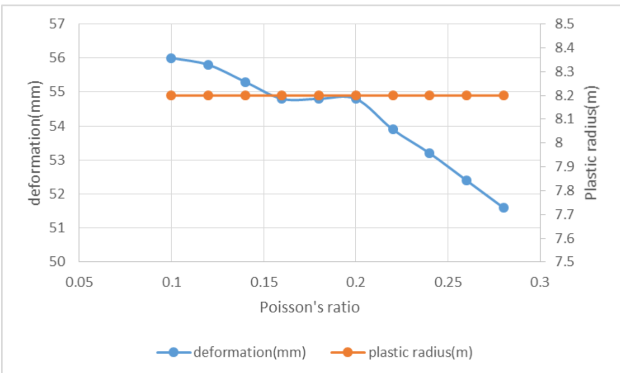

Figure 10: Variation of deformation and plastic radius with respect to poisson's ratio

4.6.2 Uniaxial Compressive Strength (UCS)

The value of uniaxial compressive strength (UCS) is varied from 30 to 75 MPa and the effect on deformation and plastic radius is studied. As we can see from the graph that both the deformation and plastic radius decrease with respective to increase in UCS. The graph shows that both the parameters are equally sensitive towards UCS.

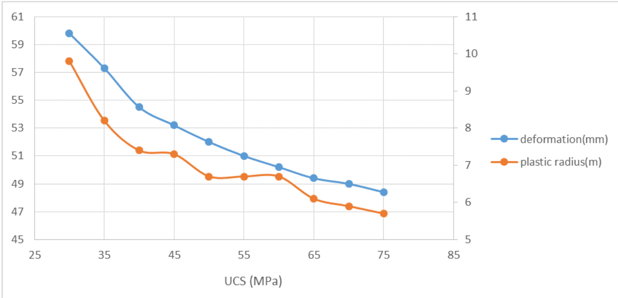

Figure 11: Variation of deformation and plastic radius with respect to UCS

4.6.3 Geological Strength Index (GSI)

The value of uniaxial compressive strength (UCS) is varied from 30 to 75 MPa and the effect on deformation and plastic radius is studied. As we can see from the graph that both the deformation and plastic radius decrease with respective to increase in UCS. The graph shows that both the parameters are equally sensitive towards UCS.

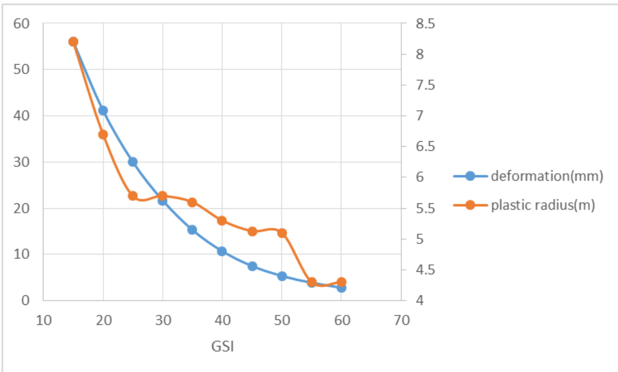

Figure 12: Variation of deformation and plastic radius with respect to GSI

4.6.4 Young's Modulus of elasticity (Ei)

The value of young's modulus of elasticity is increased with an increment of 2 from 20 to 38 GPa and the value of deformation and plastic radius is measured. The data plotted in the graph shows that the plastic radius very less sensitive to the change in modulus of elasticity whereas the deformation decreases on increasing modulus of elasticity.

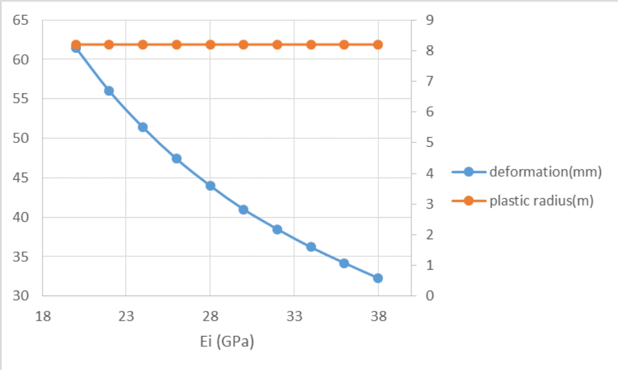

Figure 13: Variation of deformation and plastic radius with respect to Ei

5. Conclusions

In this paper, the assessment of different rock engineering parameters and their effect on tunnel stability is carried out. Based on this assessment, following conclusions can be drawn.

1. A 100m length of tunnel around the 1st bend of the headrace tunnel is selected for the estimation of leakage. As the specific leakage in this portion exceeds the leakage limit, proper leakage control measures such as grouting should be used in this stretch of tunnel.

The results from the Norwegian confinement criteria suggests that the vertical and lateral cover is enough for the confinement of the tunnel to obtain the required factor of safeties (1.3).

2. The effect of these rock engineering parameters is studied using the data extracted from Phase2 software. The result concludes that poisson's ratio has least effect on both plastic radius and deformation. Young's modulus of elasticity of rock has maximum effect on deformation and UCS has maximum effect on plastic radius.

Acknowledgments

The authors are thankful to Peoples Energy Limited for providing data required for the research work.

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