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Stability Evaluation of Tunnel: A Case Study of Headrace Tunnel of Khimti-2 Hydroelectric Project, Dolakha and Ramechhap, Nepal

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Abstract

This study focuses on the stability evaluation of the headrace tunnel of the Khimti-2 hydroelectric project located in Dolakha and Ramechhap, Nepal. The objective of this present study is to assess the impact of rock stress distribution during tunnel excavation on the surrounding rock strength and evaluate the effectiveness of different support types and excavation techniques in maintaining tunnel stability. The study specifically targets weak rock mass conditions and investigates deformation, failure zone size, convergence percentage, factor of safety, and the strength of the rock mass surrounding the tunnel. To analyze the behavior of the underground tunnel, axisymmetrical and plain strain models were developed using the Phase^2 program. Three excavation methods were simulated: full-face excavation, two-stage excavation, and three-stage excavation. In each method, rock support systems consisting of steel ribs, wire mesh, and steel fiber-reinforced shotcrete were installed immediately after excavation. Predicting the best location for support system (linear) installation from the tunnel face was challenging. The results were presented and discussed in terms of strength factor, extent of yielding zones,and rock mass convergence percentage. The excavation pattern that demonstrates the safest condition was found to be the three-stage excavation. The study findings provide valuable insights into the stability performance of the Headrace Tunnel and offer guidance for similar projects in weak rock mass conditions.

Keywords

Weak Rock Mass, Plain Strain, Axisymmetrical, Factor of safety

1. Introduction

The modern rock tunnels and underground structure, the deformation/stability of the tunnel and underground structure is controlled by a combination of reinforcement and support system. Construction of underground structures such as tunnels, caverns and shafts encounter various risk and uncertainties. The major stability problems during tunneling in Lesser Himalayan region of Nepal are: weak rock mass quality, high degree of weathering and fracturing, rock stress and ground water effect [1]. In Himalayas of Nepal, tunnel squeezing is a common phenomenon as the fault zone weak rocks (eg: mudstone, slate, phyllite, and schist highly schistose gneiss) that compose the mountains are not capable of with standing high stress.

The squeezing behaviour is associated with poor rock mass deformability and strength properties; based upon previous experience, there are a number of rock complexes where squeezing will occur, if the loading conditions needed for the onset of squeezing are present: gneiss, micaschists and calcschists (typical of contact and tectonized zones and faults), claystones, clay-shales, marly-clays, etc. [2, 3]. Terzaghi gives a behavioural description of squeezing rock as follows: "Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite of squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity " [4]. Severe tunnel squeezing cases have been encountered in many hydropower tunnels in Nepal like Kaligandaki HEP, Khimti HP, Modi HP, Middle Marsyangdi [5], Chameliya HEP and many more.

Khimti-2 Hydroelectric Project is also one among them which faced similar problem, located in Dolakha district, Central Province of Nepal. There was significant deformation and slight (minor) squeezing during construction in the Headrace tunnel at the Chainage of 2+.516.83 m to 2+475.43 m. Because of very weak, highly schistose and fractured rock types and high tectonic stress squeezing has been experienced even in the lower overburden. Khimti-2 Hydroelectric Project is facing challenges related to poor rock mass quality and high overburden leading to compressive stress exceeding the strength of the rock mass which is seen as buckling of steel rib as shown in Figure 1.

Figure 1: Buckling of Steel Rib on crown right wall of upstream of heardrace tunnel.

2. Project location and Geology

The headrace tunnel runs to the south-west direction at the beginning up to 200 m and turn towards North West direction. Again it turns towards south west direction up to 6323 m and then it turns towards south east direction till the surge tunnel located at Hawa of Dolkha. The location boundary is as shown in Table 1 The Project is located in Midland, the Lesser Himalaya, central Nepal. According to the study Ishida and Ohta (1972) Okhaldhunga Phyllite, Melung gneiss and Jiri crystalline schist are three major unite in the project area [6]. Main rocks in the study area of headrace tunnel are augen gneiss and schist. Generally foliation plane is NE-SW and dip NW. The augen gneiss is greenish white to greyish white, moderately weathered, jointed and fractured. Whereas the schist intercalated with gneiss is greenish grey, thinly foliated, highly weathered, fine to medium grained. The study area map and regional geological map of Khimti-2 Hydroelectric project is shown in Figure 2 and Figure 3.

Table 1: Location Boundary

Coordinates	From	To
Latitude	27°33' 07" N	27°35' 13" N
Longitude	86°09' 26" E	86°14' 18" E

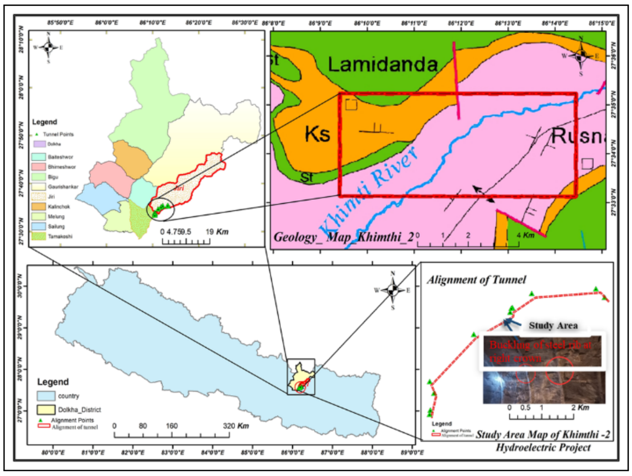

Figure 2: Study area map of Khimti -2 Hydroelecric Project.

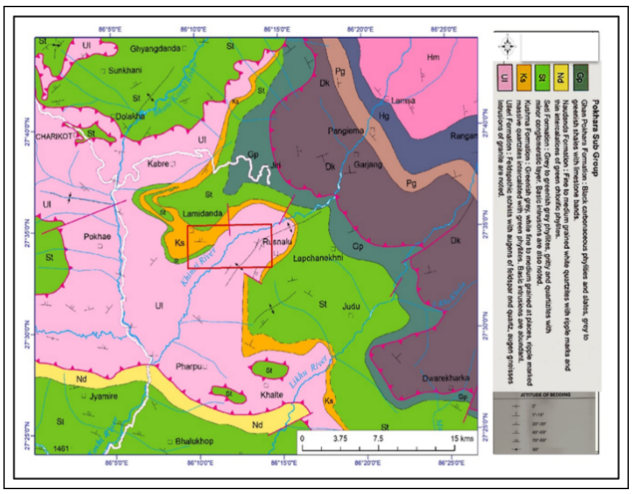

Figure 3: Regional Geological map showing Project Area.

3. Rock mass classification and support class of study area.

The geological condition of study area along headrace tunnel has augen gneiss rock type ,thinly foliated and moderately to highly weathered, fractured rock mass approximate RQD ranging from 40 % to 65 % with predicated Q-value of 1.63 (i.e Rock class D, Poor Rock mass quality) and RMR value of 43 (i.e Rock Class III, Fair rock mass quality) and the actual geological condition of study are shown in Figure 4. The rock quality in study area is poor i.e Q-value 1.63 that means rock bolts with 50-60 mm fibre reinforced sprayed shotrete (B+Sfr) is enough to solve the problems, if any. But actually during tunnel advance the Q-value is found to be 0.07 -0.027 i.e Rock class F, Extremely poor rock mass and RMR value of 28-34 (i.e Rock Class IV, Poor rock mass quality) that means it may required to provide fibre reinforced sprayed shotrete (120-150mm) and bolting + steel ribs and bolting and spiling bolts of 4m @ 20-30mm c/c. Additional support depend on tunnel closure that occour during tunnel advance.

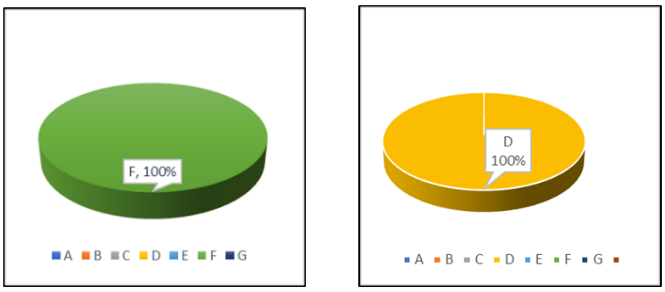

Figure 4: Predicated (left side) and actual (right side) rock mass classification of study area along headrace tunnel upstream of Adit-1.

4. Estimation of Rock mass parameters

Uniaxil compressive strength of intact rock σ_{cm}= 38 MPa and modulus of elasticity of intact rock 22 Gpa has been taken from paper Shrestha and Panthi, 2014 [5]. The value of rock mass parameter are calculated by using empirical formula as shown by Equation 1 to 5 [7, 8, 1]. The summary of rock mass parameter along study area are shown in Table 2.

$$GSI = \frac{52 * \frac{J_r}{J_a}}{1 + \frac{J_r}{J_a}} + 0.5 * RQD \tag{1}$$

$$\sigma_{cm} = \sigma_{ci}\left(exp\left(\frac{GSI - 100}{9 - 3D}\right)\right)^{a} \tag{2}$$

&

$$\sigma_{cm} = \frac{\sigma_{ci}^{1.5}}{60} \tag{3}$$

$$E_{rm} = E_{ci}\left[0.02 + \frac{1 - (D/2)}{1 + e^{(60+15D-GSI)/11}}\right] \tag{4}$$

&

$$E_{rm} = E_{ci}\left(\frac{\sigma_{cm}}{\sigma_{ci}}\right) \tag{5}$$

Table 2: Rock mass parameter along the study area of headrace tunnel

Chainage (m)	Over-burden (m)	Q-Value	GSI	ν MPa	γ MPa	σ_{ci} MPa	σ_{th} MPa	σ_{cm} MPa	σ_v MPa	σ_H MPa	k	E_i GPa	E_{rm} GPa	G GPa
2+511.83	157.18	0.031	37	0.10	0.027	38	2.46	3.90	4.24	2.46	0.58	22	2.26	2.05
2+506.83	156.05	0.06	38	0.10	0.027	38	2.46	3.90	4.21	2.46	0.58	22	2.26	2.05
2+501.83	155.02	0.07	39	0.10	0.027	38	2.46	3.90	4.19	2.46	0.59	22	2.26	2.05
2+496.83	154.12	0.07	39	0.10	0.027	38	2.46	3.90	4.16	2.46	0.59	22	2.26	2.05
2+491.83	153.53	0.07	39	0.10	0.027	38	2.46	3.90	4.15	2.46	0.59	22	2.26	2.05
2+486.83	153.53	0.02	37	0.10	0.027	38	2.46	3.90	4.15	2.46	0.59	22	2.26	2.05
2+481.83	153.24	0.02	37	0.10	0.027	38	2.46	3.90	4.14	2.46	0.59	22	2.26	2.05
2+476.83	153.23	0.02	37	0.10	0.027	38	2.46	3.90	4.14	2.46	0.59	22	2.26	2.05
2+471.83	152.48	0.034	38	0.10	0.027	38	2.46	3.90	4.12	2.46	0.6	22	2.26	2.05
2+466.83	152.15	0.034	38	0.10	0.027	38	2.46	3.90	4.11	2.46	0.6	22	2.26	2.05
2+461.83	151.64	0.034	38	0.10	0.027	38	2.46	3.90	4.09	2.46	0.6	22	2.26	2.05
2+456.83	150.77	0.034	38	0.10	0.027	38	2.46	3.90	4.07	2.46	0.6	22	2.26	2.05

5. Determination of Deformation.

There are many methods used to determine the deformation of a tunnel. Among these methods, Carranza-Torres and Fairhurst, (2000) [9], Shrestha and Panthi, (2015) [10], and numerical modeling by Phase^2 and Rocsupport were used in this research work.

5.1 Shrestha and Panthi (2015) method

Shrestha and Panthi (2015) investigated the long-term squeezing behavior observed in three distinct hydropower tunnels located in the Himalayan region of Nepal. They established a connection between time-independent and time-dependent strains using a convergence equation, as originally proposed by Sulem et al. (1987) [10]. Time-independent deformation is mostly dominating and often the most crucial part of the plastic deformation, which takes place immediately after the tunnel excavation and until the tunnel face efect is stoped [10]. Shrestha and Panthi found out correlation between time-independent (instantaneous) and time-dependent (final) rock mass deformation with rock mass deformability properties, in situ stress anisotropy and support pressure. Estimation of instantaneous and final closure of the tunnel as a tunnel strain and given by the Equation 6 & 7 [5]

This approach links the shear modulus (G), in-situ stress condition and support preesure to tunnel strain calculation. Calculated strain have been presented in Table 3. Generally these approach is applicable for all type of tunnel without considering the shape.

$$\varepsilon_{IC} = 3065\left(\frac{\frac{\sigma_v(1+k)}{2}}{2G(1+P_i)}\right)^{2.13} \tag{6}$$

$$\varepsilon_{IF} = 4509\left(\frac{\frac{\sigma_v(1+k)}{2}}{2G(1+P_i)}\right)^{2.09} \tag{7}$$

5.2 Convergence Confinement method

The convergent confinement method provides guidance on where to place supports and how much support pressure is needed to keep deformation within the limit. The study has been carried out separately for each unique rock mass situation because CCM only takes into account a single rock type per section. Figure 6 illustrates a longitudinal deformation profile (LDP) for various chainages and Table 4 shows the tunnel closure % at various chainage.

In the Figure 5 ,the support (Blocked steel + shoterete) is applied 2.5 m ,1.5 m behind the tunnel face and as well as at the face itself. The radial deformation are 6.289mm, 5.055 mm and 3.179 mm,when P_i=0. When maximum support pressure (P_s^{max}) is 1.41 Mpa the radial deformation is 7.669 mm and it increase with maximum limit of 11.809 mm.The Blocked steel + shoterete sets will be failed before they reach their equilibrium support pressure (p_s^D) 0.15 Mpa, 0.5 Mpa, and 0.80 Mpa with factor of safety (FOS) equal to 9.40, 2.82, 1.763 respectively.

If the support is not installed the radial deformation goes on increasing and moves to its maximum limit (i.e 10.328 mm) but when the support set are installed the tunnel convergence

Table 3: Deformation of tunnel advace and % of closure at different chiange

Chainage (m)	Over-burden (m)	Q-Value	Strain ε_{FC}% P_i = 0 MPa	Strain ε_{IC}% P_i = 0 MPa	Deformation (mm) P_i = 0 ,Initial	Deformation (mm) P_i = 0 ,Final
2+516.83	159.38	0.031	0.161	0.083	6.76	3.49
2+511.83	157.18	0.031	0.159	0.081	6.68	3.4
2+506.83	156.05	0.060	0.156	0.080	6.55	3.36
2+501.83	155.02	0.070	0.157	0.080	6.59	3.36
2+496.83	154.12	0.070	0.155	0.079	6.51	3.32
2+491.83	153.53	0.070	0.154	0.079	6.47	3.32
2+486.83	153.53	0.020	0.154	0.079	6.47	3.32
2+481.83	153.24	0.020	0.153	0.078	6.43	3.28
2+476.83	153.23	0.020	0.153	0.078	6.43	3.28
2+471.83	152.48	0.034	0.153	0.078	6.43	3.28
2+466.83	152.15	0.034	0.153	0.078	6.43	3.28
2+461.83	151.64	0.034	0.151	0.077	6.34	3.23
2+456.83	150.77	0.034	0.150	0.076	6.30	3.19

Table 4: Deformation of tunnel advace and % of closure at different chiange

Chainage (m)	Overburden (m)	Q-Value	Deformation Pi=0	Strain(ε) %
2+516.83	159.38	0.031	11.186	0.27
2+511.83	157.18	0.031	11.084	0.264
2+506.83	156.05	0.06	10.984	0.262
2+501.83	155.02	0.07	10.561	0.251
2+496.83	154.12	0.07	10.497	0.25
2+491.83	153.53	0.07	10.492	0.25
2+486.83	153.53	0.02	10.497	0.25
2+481.83	153.24	0.02	10.46	0.249
2+476.83	153.23	0.02	10.46	0.249
2+471.83	152.48	0.034	10.396	0.248
2+466.83	152.15	0.034	10.396	0.248
2+461.83	151.64	0.034	10.332	0.246
2+456.83	150.77	0.034	10.328	0.246

will begins to load the support. The convergence of the rock mass occurs at 1.01 mm, 0.195 mm, and 0.111 mm before the equilibrium between the rock mass and support systems is reached. Tunnel radial deformation and support pressure for other tunnel section has been estimated in simillar manner.

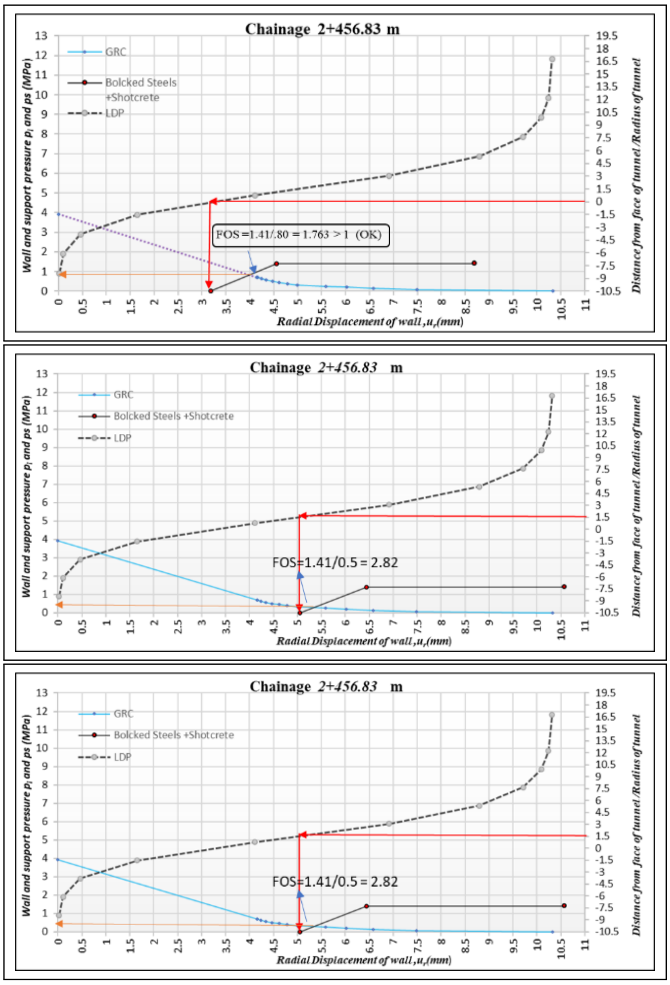

Figure 5: FOS when face effect varies from 0 m to 2.5 m

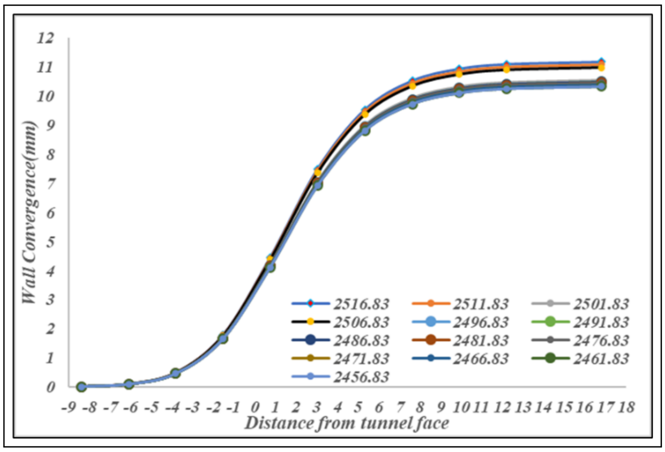

Figure 6: Original LDPs for all selected section

5.3 Numerical Modeling

Modelling in 2D is carried out as plane strain analysis and axisymmetric analysis in Phase^2 version 8 [11]. RocSupport is used to analysis tunnel face effect and to visualize support interaction with various support system and with tunnel advance. The input parameters have been finalized by calibrating the model using project report of Khimti-2 Hydroelectric Project, project drawings, geological information of the area, data from near by projects.

5.3.1 Construction of Valley model

The in-situ stress condition along tunnel alignment and normal to the tunnel alignment at the study area are determined. The models were run for various stress ration (K), to achieve the stress at study area. The value of principle stress σ_1, σ_3, σ_z and stress direction angle were identified as shown in Table 5. The in-situ stress value from valley model $sigma_1$,

Table 5: Result of a stress analysis with various stress ratios at different chainage of study area

Chainage(m)	K_H	K_h	σ_1	σ_3	σ_z	Angle
2+516.83	1.50	0.980	6.03	3.9	4.175	12^0
2+501.83	1.10	0.980	4.38	3.6	4.175	15^0
2+481.83	1.20	0.945	4.95	3.7	4.050	19^0
2+456.83	1.52	1.100	6.03	3.9	4.625	9^0

$sigma_3$, $sigma_z$ and stress direction angle are used in 2D model of study area. Input loading type is set as Field stress (gravity type) with use of actual ground surface. The two model (i.e parallel & perpendicular to alignment of tunnel) were run with various stress ratio condition. The stress ratio was selcted such that it lies with in the range of Equation 8 provided by Hoek and Brown, (1982) [12].

$$\frac{100}{H} + 0.3 \leq k \leq \frac{1500}{H} + 0.5 \tag{8}$$

5.3.2 Tunnel Excavation Sequences Modelling

Tunneling Method and supports are utilized to hold the loads and strengthen the rock in order to prevent collapse and stability problems as the tunnel advances. The Khimti-2 Hydroelectric Project study area HRT Tunnel was excavated on a weak rock mass (Q values range from 0.027 to 0.07), suitable excavation methods are required to reduce collapse and stability issues throughout the excavation operation. To evaluate the stability of the Khimti-2 Hydroelectric Project due to tunnel excavation pattern in construction site, three excavation patterns, including full face tunnel excavation, are modelled on phase[2]. Blocked steel, shoterete, and rock bolts are the support system chosen for analysis. Following excavation, the support is installed. Different failure evaluation criteria, including the strength factor for the rock mass, the size of the failure zones, and rock mass deformation, have been used to assess the stability of the Khimti-2 HRT tunnel.

In these section failure due to stregth factor is discussed. A strength factor greater than 1 indicates stable conditions, while a strength factor less than 1 indicates a stability problem due to the produced stress exceeding the rock mass strength. Based on the results of Figure 7 full face excavation at chainge 2+510.83 m, it can observed that the stability issue in the HRT tunnel is indicated by the strength factor value in the right and left crowns being less than 1. The excavation plan used on site for the weak rock mass condition was the cause for the steel ribs buckling at these sections, just as it takes place in reality at the Khimti-2 Hydroelectric Project. From above discussion it can be concluded that the excavation method followed on site should be changed and three stage excavation method should be used for the stability of tunnel in weak rock mass condition.

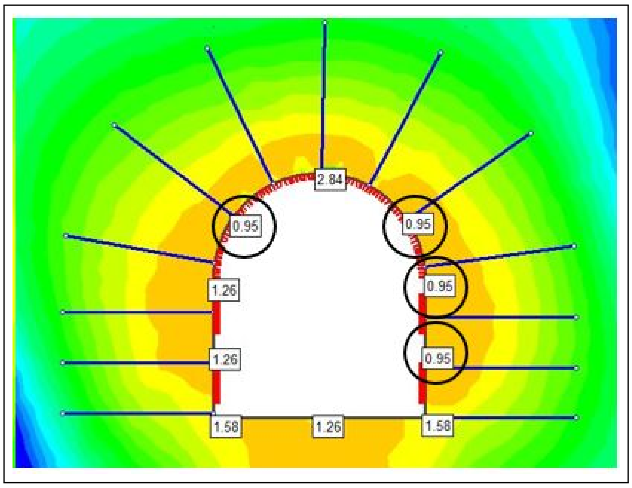

Figure 7: strength factor contour around the tunnel after support installation at chiange 2+501.83 m

5.3.3 Tunnel convergence and stability due to tunnel face effcect

The convergence of rock mass around tunnel opening is the another failure criterion. The distance between the tunnel face and the support installation is a key factor in the stability of tunnel excavation, and computational simulation shows that this distance will increase tunnel convergence when the support installation is delayed. The total tunnel convergence value around tunnel opening is less then 1 %. Therefore, the performance of tunnel is satisfactory. Long-term deformation behavior of a tunnel and support system can be used to demonstrate a tunnel's stability. The long-term qualities of the rock mass surrounding a tunnel gradually decrease over time. Possible lead stability issue. RocSupport demonstrates a long-term deformation prediction. For this analysis, strength is reduced by 30 %. In order to determine the properties for the long-term ground response curve, the properties of the rock mass will be decreased by this proportion. The input parameter for rock mass is obtained from Tables 2 and stress value from Table 5.

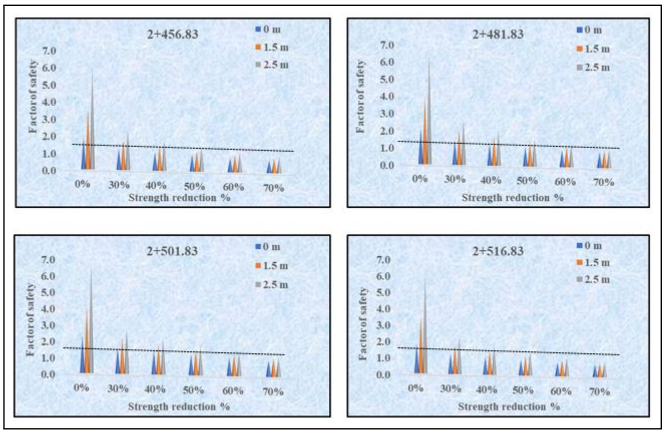

Figure 8: Relationship between factor of safety and strength reduction percentage at different support installation distance from tunnel face.

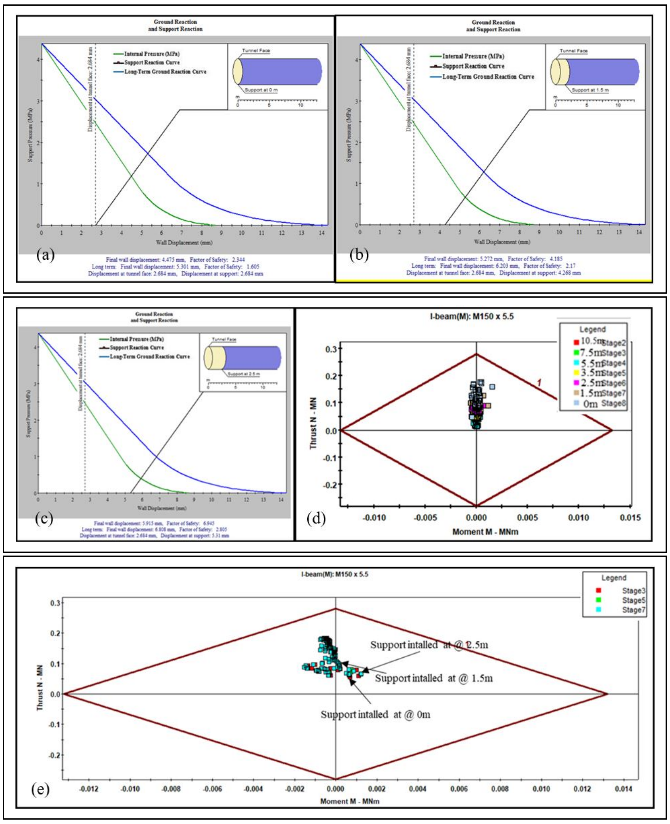

Figure 9: RocSupport analyis for ground reaction analyis and support reaction of lined tunnel installed at o m (a), at 1.5 m (b) and at 2.5 m (c).Summary of axisymmetrical analysis of tunnel linear at different distance from the face (d).Plain strain phase2 analysis of tunnel linear at different distance from the face (e).

From these analyses, it can be conclude that the Khimti-2 Hydroelectric Project can withstand a strength loss of up to 50 % while still maintaining stability, but if the percentage of strength reduction exceeds 50 %, failure will occur in the Khimti-2 headrace tunnel at study area at the chainage from 2+456.83 m to 2+516.83 m. Therefore, a value of 50 % strength reduction is taken as the starting point for stability problems as shown in Figure 8.

Early installation of a support system can overload the support syatem and cause failure in weak rock ground. A late installation will result in excessive ground displacement and ground instability. Therefore, it is essential to accurately identify the support installation location inside a staged 2D modeling. For the installation of supports at various distances from the tunnel face, a number of asymmetrical analyses and plain strain analyses are conducted. The factor of safety is then determined from the RocSupoort study. As the moment thrust capacity envelope is provided for a factor of safety of 1, it can be concluded from Figures 9 (d, e) that the support system installed at a distance of 1.5 m to 2.5 m provides the best result. Steel ribs on the right crown of the Khimti-2 Hydroelectric Project buckled at the chainage of 2+510.83 m as well as at other chainages, which may be a result of an incorrectly support installed distance from the tunnel face.

6. Conclusion

The most frequent issues include stability issues resulting from the excavation technique used on site, support system (Linear) installation distance from tunnel face, and plastic deformation (Long term and instantaneous deformation). The installation distance also has an impact on the plastic zone and tunnel wall displacement. According to this research, there should be 1.5 to 2.5 m distance between the face and the liner. The 2D modeling carried out with Phase^2 indicates the importance of providing sufficient support for the tunnel face and leaving a suitable distance between the face and the support to reduce displacements in the Khimti-2 tunnel.

Since less displacement appears after three stages of excavation, it is suggested that dividing the tunnel face into sections is an effective excavation technique. The whole face excavation cause the stability problem at the right wall and left and right crown levels at the study area of Khimti-2 Hydroelectric Project. The key findings from this research work are as follows:

✓Over the HM method and CCM Method, the Shrestha and Panthi (2015) method has some advantages. The method accounts for stress anisotrophy in the rock mass, takes into account the shape of the tunnel, and predicts both instantaneous and long-term deformations around the tunnel.

✓50% strength reduction is taken as the starting point for stability problem.

✓The accuracy of the input parameters limits numerical modeling, which makes a number of simplifications. However, the evaluation of the stability and required support installation distance of the Khimti-2 headrace tunnel shows numerical modeling to be effective.

7. Recommendations

- ✓ Field observations and laboratory tests are essential for accurate estimation of rock mass properties, as these input parameters are the most critical factors in analysis.
- ✓ To assess stability in response to the sequence of tunnel excavation, 3-dimensional numerical modeling will be required while tunneling through a weak rock mass condition.
- ✓ Instrumentation should be installed properly to measure actual deformation.

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