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Stability analysis and evaluation of rock support in headrace tunnel of Khimti-2 Hydroelectric Project

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ABSTRACT

The tunnelling activities have been increasing considerably in Nepal with the development of many hydropower projects. During the design phase, tunnel alignment selection and rock mass quality prediction along with rock support requirements have a direct impact on overall cost and time requirements of the project. The major decisions that must be made in planning, designing and constructing a tunnel is mainly influenced by the geology along the tunnel alignment. An assessment on the stability of the Headrace Tunnel (HRT) of Khimti-2 HEP has been done. Ten excavated sections of headrace tunnel have been selected for the study at the critical zones. Squeezing and spalling phenomena have been assessed by empirical and semi-analytical method. The wedge block stability analysis has been done using UNWEDGE software. Further, rock support estimation from Q-method as well as support optimization have been done from numerical method with phase2 software developed by Rocscience. From the numerical analysis, the support has been optimized based on the Q-values.

Keywords: Stability analysis, support optimization, numerical methods, Q-value

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INTRODUCTION

Nepal is a landlocked and mostly mountainous country. Within a very short width the altitude of the country varies greatly from about 100 masl at its southern border to its maximum up to 8848.86 masl in the northern border.

The increasing population trend and urbanization is becoming a major challenge in economic development of Nepal. The major economic resources of the country are water resources, agriculture, tourism and agro-tourism based industries. The maximum utilization of these resources seems a gateway to the development. This is not possible unless we develop infrastructures such as hydropower projects, irrigation schemes, road networks, drinking water system, storage facilities and so on. For the development of those infrastructures will need to utilize underground space like tunnels and underground caves (Panthi, 2004). Particularly for hydropower development and good road network development in the Himalayan region, the need of tunnel is enormous.

In this regard, the tunnelling activities have increased considerably with the development of many hydropower projects. Most tunnelling projects developed in the past have faced severe stability issues. The complex geological setting of the Himalaya poses a major challenge in solving most tunnel instabilities.

For a tunnel to design, the decision in selecting tunnel alignment and predicting the rock mass quality and rock support requirement has direct influence on the overall cost and time requirement. The major decisions that must be made in planning, designing and constructing a tunnel is mainly influenced by the geology along the tunnel alignment.

The main objective of this research work is to evaluate tunnel support and optimize in order to stabilize the headrace tunnel. To achieve the main objective the following specific objectives are set to: (a) review the headrace tunnel alignment, (b) analyze the stability of the headrace tunnel, and (c) estimate the support in HRT using numerical method and optimize.

STUDY AREA

Peoples Energy Limited has been developing Khimti-2 Hydroelectric Project (48.8 MW) located in the border of Dolakha and Ramechhap districts of Bagmati Province (Fig. 1). The project is being developed on Khimti River which is a tributary of Tamakoshi River which is a major drainage of Saptakoshi river system of Nepal. The Khimti River originates

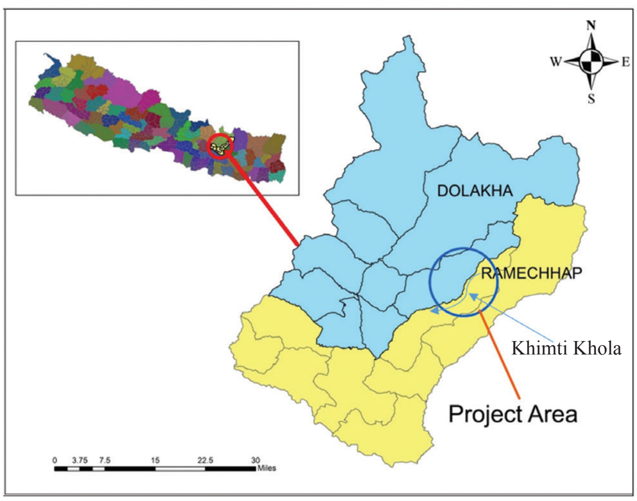

Fig. 1: Location of the Khimti-2 Hydroelectric Project (HEP).

from elevation 4500 m and converges at elevation 600 m with the Tamakoshi River. The total drainage area of the river is 492.4 km^2, and the drainage area in front of the intake is 295.34 km^2. From dam site to powerhouse the river section is about 7 km long. The ridges on both banks are at elevation 2,300 to 3,000 m. The riverbed at the headworks is at an elevation of 1627 m and that at the powerhouse tailrace is at elevation of 1278 m with a drop of about 349 m. The gross head of the project is 355 m and net head is 342.92 m.

The study section

While doing the research the HRT of the project has been excavated around 1500 m in total. Around 250 m HRT has been excavated from Adit-2 (Ch. 6+030 to 6+280), around 681 m HRT has been excavated from Adit-1 (Ch. 3+326 to 2+645) whereas around 605 m HRT has been excavated in inlet tunnel (from Ch. 0+000 to 0+605). It is very much time consuming to do analysis of the whole section of the HRT. Hence number of sections have been selected based on the lowest, medium and highest overburden depth. Total 10 sections were selected for the research purpose at the critical zones, out of which, five sections are from HRT via Adit-1 and five from HRT via Adit-2 (Fig. 2).

Geology of the project area

The project area lies within the Lesser Himalayan Zone of east-central Nepal. It falls in the Augen Gneiss unit within a meta-sedimentary rock sequence equivalent to Nawakot Group rocks of Paleoproterozoic age. Major rock types of the gneissic unit are: augen mica gneiss and phyllitic schist (Fig. 3). The schist is present in alternative repetition with gneiss at various intervals parallel to the foliation plane. The augen gneiss is slight to highly weathered, foliated, and massive to jointed. Three plus random joint sets are predominant in the rocks of the whole project area including foliation. Likewise, schist is slight to highly weathered, but because of its phyllitic nature, it is generally very weak and problem posing in tunnel. Quartz, feldspar, muscovite, biotite and tourmaline are common minerals in gneiss whereas the schist consists commonly of biotite, muscovite, chlorite, sericite, quartz and feldspar.

METHODOLOGY

Before moving to the field visit, desk study had been conducted and collected all the data like a topographical map, data record format preparation etc. The field study was carried out with the help of common geological tools like hammer, compass and GPS. The available laboratory data were also utilized. The analyses were carried out with the help of some specialized software like DIPS and Phase2 developed by Rocscience. General methodology is shown in Figure 4.

RESULTS AND DISCUSSIONS

The study focuses on the stability analysis of the HRT of Khimti-2 HEP lying at the Lesser Himalayan augen gneiss unit. Existing empirical and analytical methods for the stability of the headrace tunnel of Khimti-2 HEP are used for the estimation of support. A detailed numerical study has been carried out in finite element analysis to evaluate the tunnel rock support.

Rock mass classification

The rock mass quality along HRT based on the Q-system classification during excavation of tunnel is measured during the face mapping. The face mapping data were collected from the site office. Table 1 shows the value along the study section.

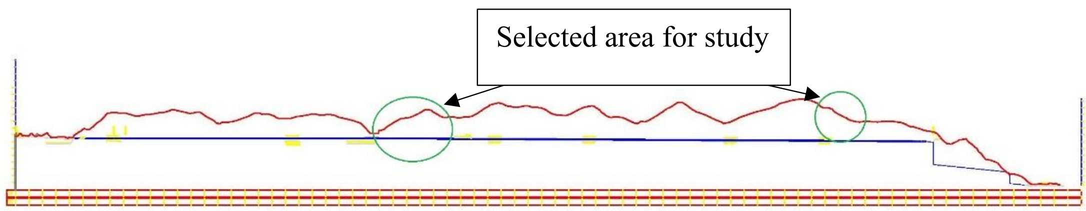

Fig. 2: L-profile of tunnel alignment with indicated sections for study.

Fig. 3: Gneiss-schist intercalation observed on road-cut section, and augen gneiss observed in HRT via Adit-2.

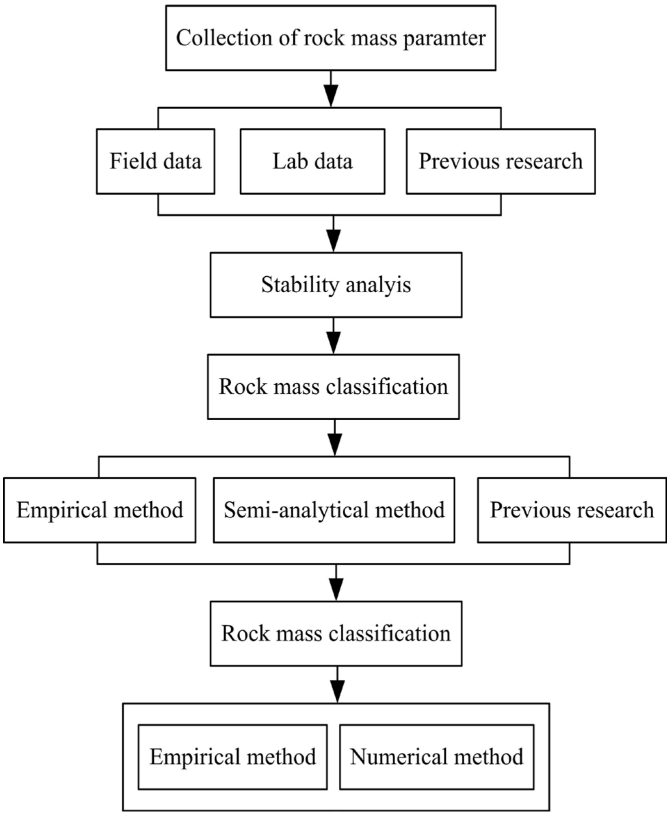

Fig. 4: Generalized flowchart of methodology.

Conversion of Q-value to RMR and GSI

The Q-value of rock obtained is converted to RMR and GSI value by using following equations 1 (Barton, 1995) and 2 (Hoek et al., 2013).

$$RMR = 15\,LogQ + 50 \qquad (1)$$

$$GSI = \frac{52 * Jr/Ja}{1 + Jr/Ja} + RQD/2 \qquad (2)$$

Tunnel stability evaluation

Block stability analysis

Various data were collected from the field study to evaluate the rock mass as well as to anticipate the tunnelling with respect to the discontinuities. The discontinuities data were plotted to make the Rosettes with the help of DIPS software. Two sites viz. upstream and downstream excavation via Adit-1 and upstream excavation via Adit-2 has been considered. The data were presented in Rosettes (Fig. 5). It indicates that the alignment of the HRT via Adit-1 is parallel or sub-parallel to the strike of major discontinuity i.e., foliation whereas that via Adit-2 is considerably oblique to the strike. However, it is worth considering that the accuracy of field measurement may be affected due to undulated surface hence the range of the attitude may be higher. Even though the rock mass stability is the result of many other factors, the rosettes indicate the more favourable condition to tunnel prevails in HRT via Adit-2 than that through Adit-1. However, tunnelling from two opposite fronts will face different challenges; one with 'driving with dip' and another, 'driving against dip'. Through Adit-2 and upstream

Table 1: Conversion of Q-value to RMR and GSI value.

Chainage	RQD	Jn	Jr	Ja	Jw	SRF	Q value	RMR value	GSI
2+684	20	9	2	4	1	2.5	0.44	44.72	27.33
2+750	30	12	1.5	4	1	7.5	0.13	36.45	29.18
2+900	40	6	1.5	10	0.66	10	0.07	32.29	26.78
2+970	35	12	1.5	2	1	2.5	0.88	49.13	39.79
3+016	20	12	1.5	4	1	2.5	0.25	40.97	24.18
6+077	65	12	1.5	4	1	2.5	0.81	48.65	46.68
6+157	40	12	1.5	6	1	2.5	0.33	42.84	30.40
6+237	30	12	1.5	10	1	7.5	0.05	30.48	21.78
6+257	60	12	3	6	1	2.5	1.00	50.00	47.33
6+277	35	12	1.5	6	1	7.5	0.10	34.82	27.90

pull through Adit-1, the tunnelling is being 'drive with dip' whereas downstream pull through Adit-1 it is 'driving against dip'. The drive with dip is more favourable than against dip.

Wedge stability analysis using UNWEDGE software

For the analysis of wedge stability, UNWEDGE software from Rocscience was used. It is a three-dimensional numerical tool designed to examine the geometry and stability of underground wedges, which are defined by the intersecting structural discontinuities of a rock mass surrounding an excavation. Safety factors are calculated for potentially unstable wedges.

Input parameters: The input parameters are given in Table 2.

Table 2: Input parameters for wedge analysis.

Chainage	Trend	Plunge	c (Mpa)	φ (°)	γ (MN/m³)	Tensile strength
2+684	55	0.3	0.1748	47.90	0.0262	15.89
2+750	55	0.3	0.3095	42.98	0.0262	15.89
2+900	15	0.3	0.4338	37.18	0.0262	15.89
2+970	250	0.3	0.6448	41.93	0.0262	15.89
3+016	250	0.3	0.4841	33.77	0.0262	15.89
6+077	341.5	0.3	0.8319	43.33	0.0262	15.89
6+157	341.5	0.3	0.5103	38.10	0.0262	15.89
6+237	341.5	0.3	0.3546	35.21	0.0262	15.89
6+257	341.5	0.3	0.6473	46.74	0.0262	15.89
6+277	161.5	0.3	0.4283	38.22	0.0262	15.89

Model Used: Mohr-Coulomb: Assumed water pressure and waviness are zero. The wedges are subjected to gravity loading only, stress field are not taken consideration. All the section was analyzed for the wedge failure but only one section was discussed.

Chainage 2+684:

Three joint sets are present in this section (Fig. 6). The factor of safety is greater than 2 in all the selected section. Hence there is less probability of wedge failure.

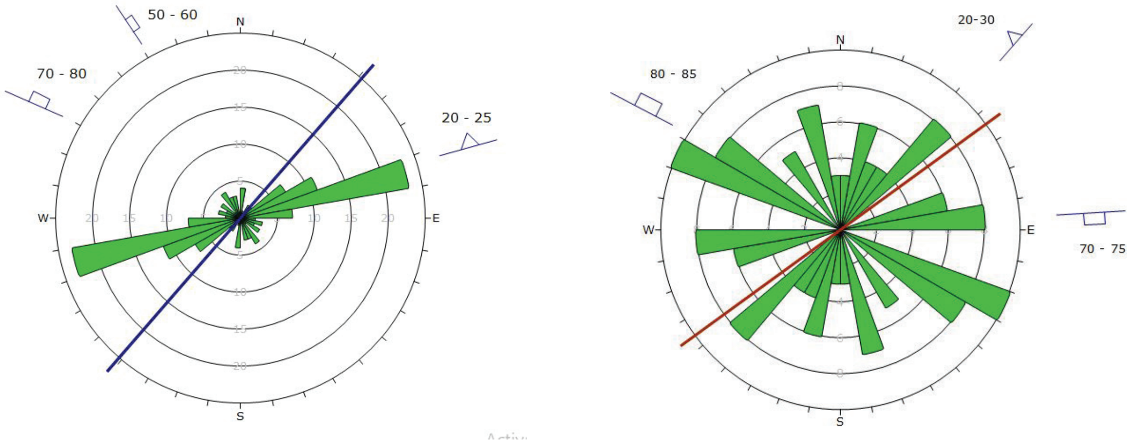

Fig. 5: Discontinuities rosettes regarding Adit-2 (left) and Adit-1 (right) area.

Squeezing analysis

Squeezing of rock is the time dependent large convergence occurring during excavation of tunnel. Squeezing prediction were analysed by empirical, semi analytical and numerical modelling. Summary of the results from empirical and semi analytical approach has been presented below;

Singh et al. (1992) approach: Singh et al. (1992) has suggested an empirical relationship between the overburden depth and rock mass quality (Q-value) both in logarithmic system. The approach is simple but correct estimation of SRF in Q-value estimation is challenging. The equation of the line is (Eq. 3).

$$H = 350\, Q^{\frac{1}{3}} \tag{3}$$

Goel et al. (1995) approach: The Goel et al. (1995) has developed an empirical approach based on the rock mass number (N) which is defined as the Q-value with SRF value of 1. It avoids the problem in obtaining the correct rating of SRF parameter. The Equation of demarketing line which separates squeezing from non-squeezing condition (Eq. 4).

$$H = 275 N^{0.33} B^{-0.1} \tag{4}$$

Hoek and Marinos (2000) approach: Hoek and Marinos (2000) approach is a semi-analytical approach of predicting tunnel squeezing. The purely analytical approach may get problem in using widely varied rock mass properties even with in a single meter.

The curve can be generated using the following equation (Eq. 5).

$$\varepsilon = \left(0.2 - 0.25\frac{p_i}{p_o}\right) * \frac{\sigma_{cm}}{p_o}^{\left(2.4\frac{p_i}{p_o} - 2\right)} \tag{5}$$

Panthi and Shrestha (2018) method: Panthi and Shrestha (2018) proposed a relationship between time independent and time dependent strain using a convergence equations (Eq. 6, 7).

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

$$\varepsilon FC = 4509 \times \left[\left(\frac{\frac{\sigma_v(1+k)}{2}}{2G(1+P_i)}\right)^{2.09}\right] \tag{7}$$

where, εIC = initial closure, εFC = final closure, σ_v = vertical stress, G = rock mass shear modulus, k = horizontal to vertical stress ration, P_i = support pressure

The summary of results are shown in Table 3.

Numerical modeling

Phase2 software by Rocscience was used for the analysis of the headrace tunnel. The Phase2 program calculates stresses and displacements around underground excavations using two-dimensional elastoplastic finite element method.

Out of 10 sections of study 6 sections has been chosen for numerical modelling based on the Q-values as project has adopted the support class based on Q-values. The Q-value ranges from 0.05 to 1 in selected 10 sections. Where four types of rock support class lie in the range. Class III for Q-range 0.4 <Q<1, Class IV for 0.1<Q<0.4, Class V-A for 0.05<0.1 and Class V-B for 0.01 <Q<0.05.

Material properties

The intact rock properties taken in design are summarized in the Table 4.

The model boundary is set as approximately 3 times the tunnel diameter on all sides to avoid boundary effects. In Khimti-2 HRT as it is a deep excavation, constant field stress has been applied. Due to the lack of stress data in the field it has been assumed the following stress data for the analysis. The tectonic stress data has been taken reference from Khimti-I hydropower project of 3 MPa with orientation N15ºW (Shrestha and Panthi, 2014).

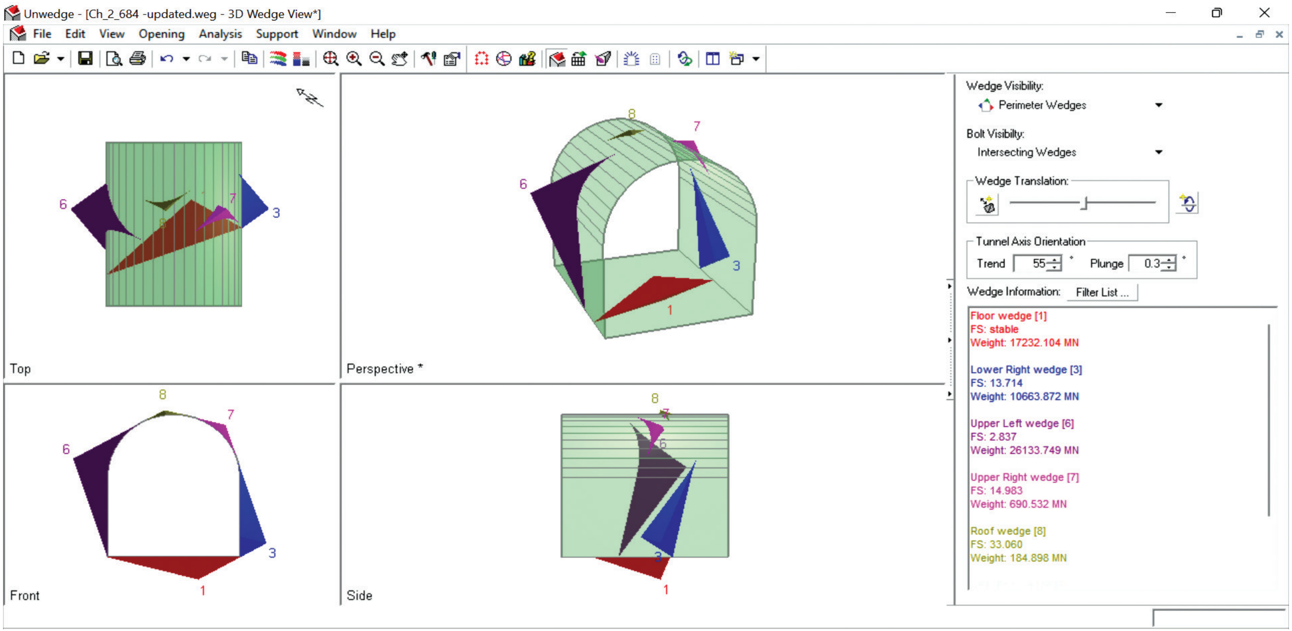

Fig. 6: 3D wedge view of wedge analysis at Chainage 2+684.

The rock was modeled as a plastic isotropic material and failure governed by the generalized Hoek-Brown criterion. The material properties for the disturbed and non-disturbed zones are applied under the generalized Hoek-Brown criterion 'GSI calculator'.

The tunnel has been excavating by drill and blast method; therefore, a disturbance zone has been considered to account for the blast damage. Disturbed zone with the disturbance factor of 0.5 all around the excavated area with the 1m thickness is considered.

Chainage 3+016

Plastic analysis has been done in each chainage. In the initial stage a uniform distributed load to the tunnel is considered. The factor is chosen so that it will gradually decrease the magnitude of the pressure. In consequence, tunnel deformation will increase as the pressure is reduced to zero. This stage simulates the reduction of support as the tunnel face advances by removing the internal pressure.

The total maximum displacement of the tunnel (U_{max}) is 21.84 mm. The radius of the plastic zone (R_P) is 4.730 m. The ratio of distance from tunnel face to the tunnel radius (X/R_T) is 1.11 and the ratio of plastic radius to tunnel radius (R_P/R_T) is 2.102.

Using Vlachopoulos and Diederichs method, the values are plotted giving the ratio of radial displacement to maximum displacement equal to 0.66. This means that 66% of the deformation will have been already taken place before the support installation. At stage 4 (internal pressure factor 0.2) yields the tunnel wall displacement.

Support is provided which was adopted by project to find the support capacity diagram which is shown in figures 7 and 8.

The support of invert concrete is safe for the installation as it comes under all the three envelops of the factor of safety. But in the shotcrete capacity plots some points are out of the all the three factors of safety envelop. Hence the support in the roof and wall should be increased to be safe. The thickness of the shotcrete is increased to 150 mm thickness from 100 mm thickness. And the invert concrete is left as it is. The result found was shown in Figure 9.

Now the shotcrete support comes within the factor of safety of 1 envelop. The support type adopted was of type IV for Q-value 0.1<Q<0.4 but still it yields. Hence the support can't be generalized based on the Q-value only, the overburden depth should also be considered. Total displacement before support and after support optimization are given in Figure 10a,b.

DISCUSSION

From stability analysis of HRT there is not much serious stability issues found in the studied section of the alignment. Numerical modelling is a useful tool in assessing stability of the underground structures. With the analysis, the headrace tunnel is found safe, no significant tunnel deformation has been observed. The deformation in all section of the study is all in acceptable range with the support system applied in the existing headrace tunnel. The six sections are numerically analysed and optimized rock support comparing with the support adopted by project has been presented in the Table 5.

At Chainage 3+016 the support type adopted was of type IV for Q-value 0.1<Q<0.4 but still it yields unlike at Chainage 2+750. At Chainage 2+750 the Q-value was 0.13 even less than this 0.25 value at the Chainage 3+016 (Table 6). The difference was in overburden depth at Chainage 2+750 it was 84.4 m and at Chainage 3+016 the depth is 205.25. Therefore, the support can't be generalized based on the Q-value only, the overburden depth should also be considered. Where overburden depth is greater than 100 m the GSI value required will be higher (Shrestha, 2021). Hence support type can be re-categorized based on Q-values and overburden depth higher than 100 m.

Table 3: Results from empirical and semi-analytical approaches.

Chainage	Overburden (H)	Q-value	Empirical methods		Semi analytical method	
			Squeezing prediction			
			Singh et al. (1992)	*Goel et al. (1995)*	*Hoek and Marions (2000)*	*Shrestha and Panthi (2018)*
2+684	38.55	0.813	No	No	No	Few stability problems
2+750	84.40	0.333	No	No	No	Few stability problems
2+900	154.95	0.050	Yes	No	No	Few stability problems
2+970	178.00	1.000	No	No	No	Few stability problems
3+016	205.25	0.097	No	Yes	No	Few stability problems
6+077	213.35	0.444	No	No	No	Few stability problems
6+157	171.75	0.125	No	No	No	Few stability problems
6+237	143.40	0.066	Yes	No	No	Few stability problems
6+257	141.90	0.875	No	No	No	Few stability problems
6+277	145.15	0.250	No	No	No	Few stability problems

Table 4: Intact rock properties used in modeling.

Intact rock properties	Value	Unit	Source
Unconfined compressive strength	36.732	Mpa	Laboratory data of project
Poisson's ratio	0.1		Panthi (2006)
Unit Weight	26.213	kN/m^3	Laboratory data of project
Intact rock constant - m_i	28		Hoek and Brown (2007)
Young's Modulus (GPa)	22	GPa	Panthi (2006)

Table 5: Revised rock support comparing with project adopted support.

Chainage	Support type	Support adopted by project	Recommended support
2+684	III (0.4<Q<1)	Rock bolts Ø20 mm, L=2.5 m @1.7x1.7 m and Fiber reinforced shotcrete 75 mm thickness plus invert concrete lining of 150 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1.7x1.7 m and Fiber reinforced shotcrete 50 mm thickness plus invert concrete lining of 150 mm thickness
2+750	IV (0.1<Q<0.4)	Rock bolts Ø20 mm, L=2.5 m @1.5x1.5 m and Fiber reinforced shotcrete 100 mm thickness plus invert concrete lining of 200 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1.5x1.5 m and Fiber reinforced shotcrete 50 mm thickness plus invert concrete lining of 150 mm thickness
2+900	V-A (0.05<Q<0.1)	Rock bolts Ø20 mm, L=2.5 m @1.3x1.3 m and Plain shotcrete 50 mm first layer + Fiber reinforced shotcrete 100 mm thickness plus Steel ribs ISMB 150 @ 1-1.5 m c/c +wire mesh Ø6 mm 150x150 mm in arch + invert concrete lining of 250 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1.3x1.3 m and Plain shotcrete 50 mm first layer + Fiber reinforced shotcrete 100 mm thickness plus Steel ribs ISMB 150 @ 1-1.5 m c/c +wire mesh Ø6 mm 150x150 mm in arch + invert concrete lining of 250 mm thickness
3+016	IV (0.1<Q<0.4)	Rock bolts Ø20 mm, L=2.5 m @1.5x1.5 m and Fiber reinforced shotcrete 100 mm thickness plus invert concrete lining of 200 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1.5x1.5 m and Fiber reinforced shotcrete 150 mm thickness plus invert concrete lining of 200 mm thickness
6+077	III (0.4<Q<1)	Rock bolts Ø20 mm, L=2.5 m @1.7x1.7 m and Fiber reinforced shotcrete 75 mm thickness plus invert concrete lining of 150 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1.7x1.7 m and Fiber reinforced shotcrete 50 mm thickness plus invert concrete lining of 150 mm thickness
6+237	V-B (0.01<Q<0.05)	Rock bolts Ø20 mm, L=2.5m @1x1 m and Plain shotcrete 50 mm first layer + Fiber reinforced shotcrete 100 mm thickness plus Steel ribs ISMB 150 @ 1-1.5m c/c +wire mesh Ø6mm 150x150 mm in arch and walls + invert concrete lining of 250 mm thickness	Rock bolts Ø20 mm, L=2.5 m @1x1 m and Plain shotcrete 50 mm first layer + Fiber reinforced shotcrete 100 mm thickness plus Steel ribs ISMB 150 @ 1-1.5 m c/c +wire mesh Ø6 mm 150x150 mm in arch and walls + invert concrete lining of 300 mm thickness

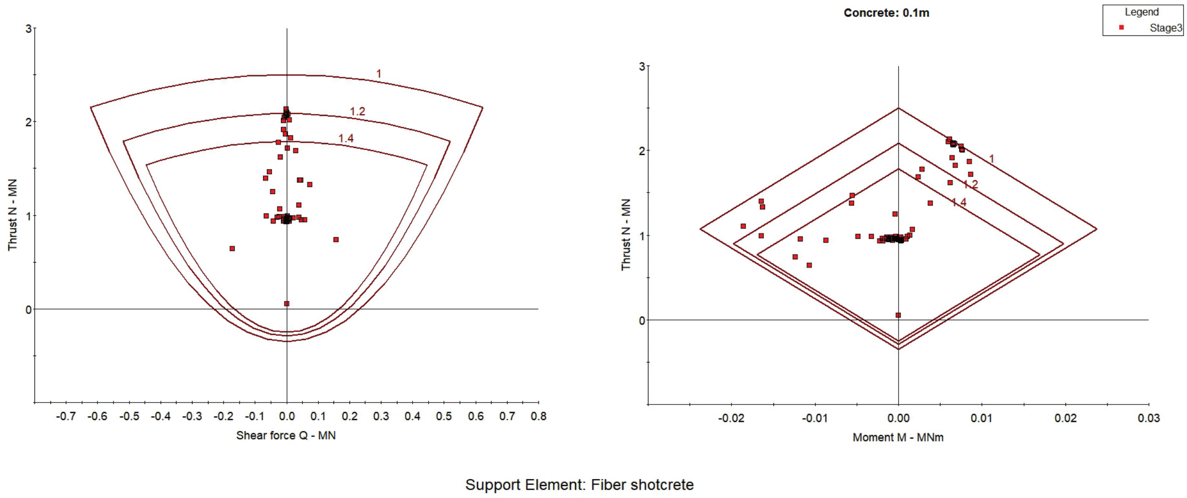

Fig. 7: Support Capacity plot of shotcrete at Chainage 3+016.

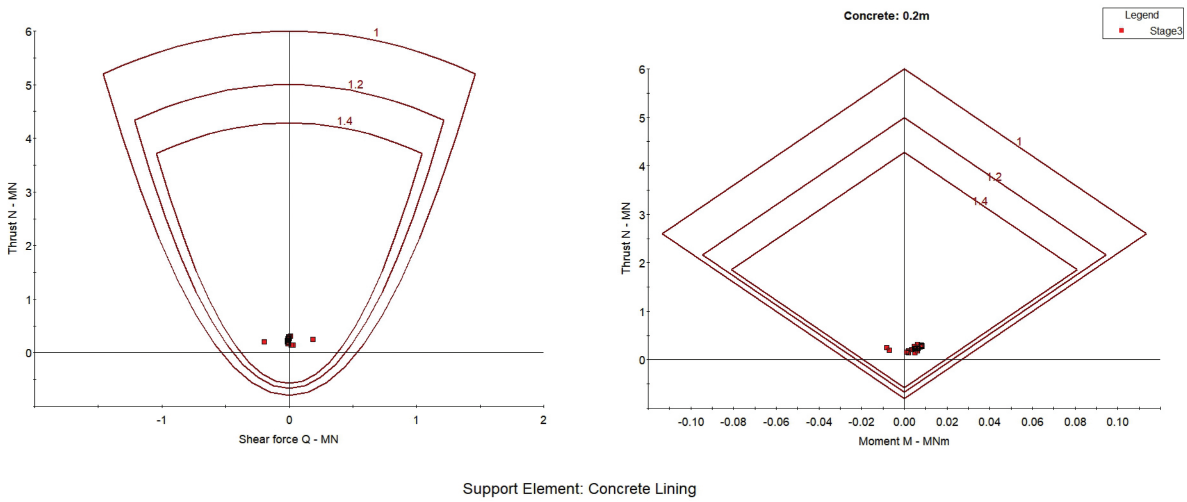

Fig. 8: Support capacity plot of invert concrete lining at Chainage 3+016.

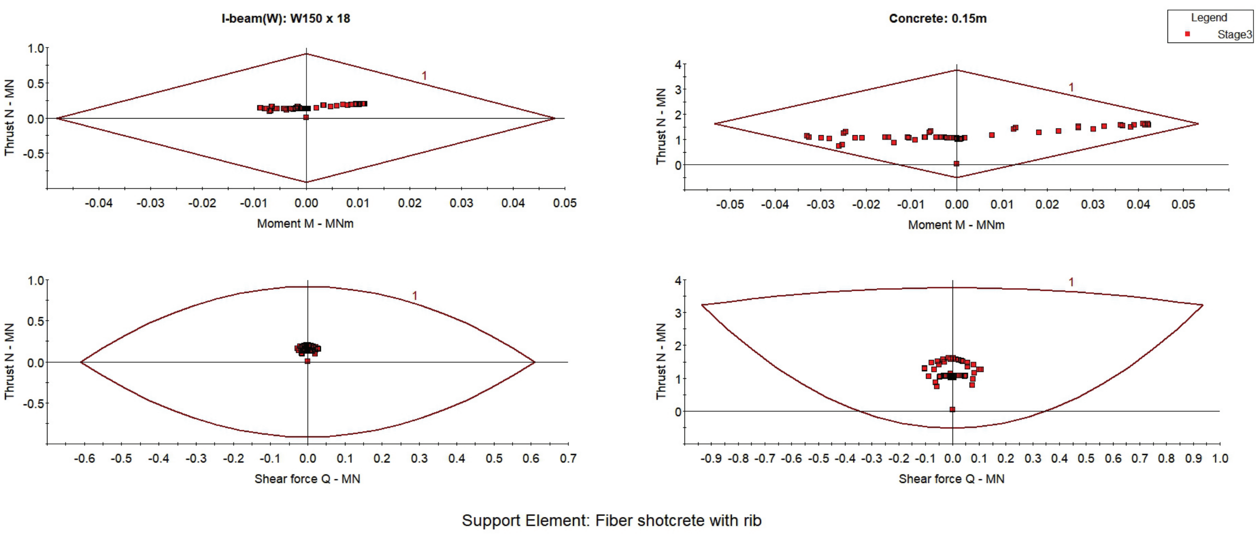

Fig. 9: Support capacity plot after increment of shotcrete at Chainage 3+016.

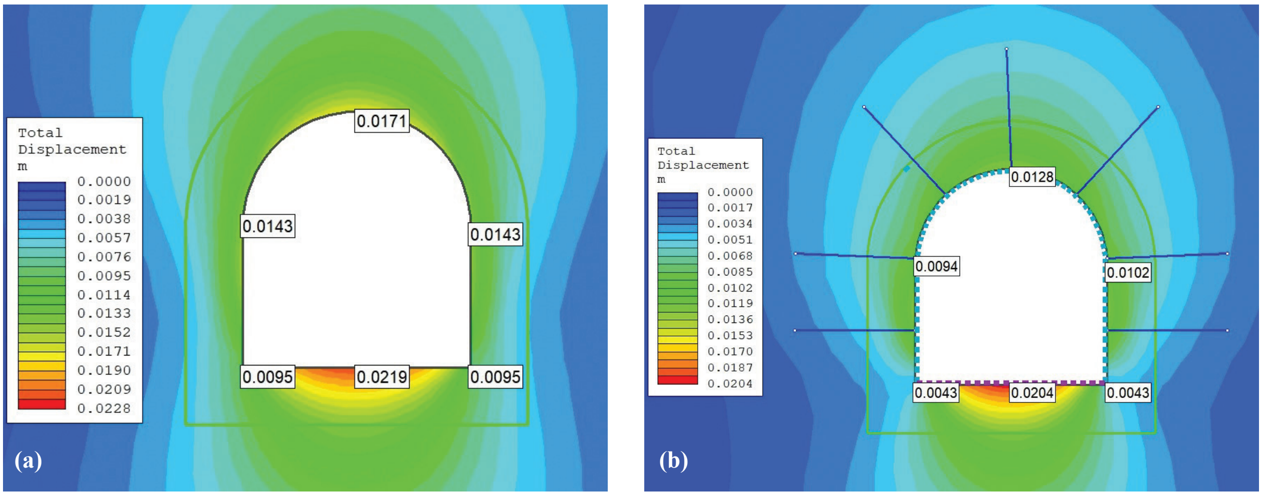

Fig. 10: Tunnel displacements (a) before support, (b) after support optimization.

Table 6: Summary of support evaluation result.

Chainage	Overburden	Q-value	Support type	Remarks
2+684	38.55	0.44	III (0.4<Q<0.1)	Optimized
2+750	84.40	0.13	IV (0.1<Q<0.4)	Optimized
3+016	205.25	0.25	IV (0.1<Q<0.4)	Yields
6+237	143.40	0.05	V-B (0.05<Q< 0.1)	Yields

CONCLUSIONS

The study of headrace tunnel of Khimti-2 hydroelectric project has been done which lies in the Lesser Himalayan augen gneiss region. Three methods have been used to analyse the stability of tunnel and rock support evaluation i.e., empirical methods, semi-analytical methods and numerical modelling method using the program Phase2. The inputs for the analysis in each method are rock mass parameters and rock stresses. Hence the quality of analysis, totally depends upon the correct estimation of those input parameters. Following are the major conclusions from the study:

- The current alignment of Khimti-2 HEP has been adopted by the project considering many factors like its length, rock cover and even the length of Adits. The alignment throughout its stretch, has made various angles with the foliation strike ranging from sub parallel to even 30-35 degree. The maximum vertical cover is 306 m at chainage 5+834 and minimum is 38 at chainage 2+668. Otherwise, it is between 100 to 300 m everywhere. It seems not to pose serious instability problem.
- Block stability analysis using UNWEDGE software shows that the alignment of HRT is good. All the wedges have factor of safety >2.
- No any serious squeezing problem was found while analysing the HRT for squeezing problem from various empirical, semi-analytical and finite element modelling.
- The project has adopted the different rock support class based on Q-values only. However, if the rock cover is higher than 100 m the stress increases and the designed support may not be adequate. Hence support type can be re-categorized in case where rock cover exceeds 100 m.

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