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Rock mass characterization and support analysis of pressurized headrace tunnel of the Upper Balephi "A" Hydroelectric Project, Nepal

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ABSTRACT

Detailed geological, engineering geological and geotechnical assessment are prerequisites for any design works in underground excavation. Due to complex geology in young Himalayan, it is mandatory. Any misjudgment in support design may results huge losses of cost and time of the project. This paper encompasses detailed analysis carried out for examining the rock mass properties for optimum support along a pressurized headrace tunnel of Upper Balephi "A" Hydroelectric Project, Nepal. Empirical, analytical and numerical methods were applied for safe tunnel design. The rock mass quality and support in these areas were estimated using the rock mass rating (RMR), geological strength index (GSI) and rock mass quality (Q) systems. The detailed rock engineering assessment indicated that there are some critical locations along the headrace tunnel alignment. Rock mass quality values derived from different methods were used for calculating modulus of deformation, Hoek-Brown constants, strength of rock mass, in situ stresses, squeezing and support pressure using available empirical equations. The support determined from the empirical methods were evaluated for the overall stability of the required excavation by using finite element method. The analysis showed that the support pressure and deformation can be predicted very well and magnitude of the displacements and extent of the plastic zones can be reduced significantly by application of the support installation. The numerical modelling reveals that the support suggested by empirical methods are appropriate. Both empirical and numerical approaches are necessary for the confirmation of reliable support design of underground structure.

Keywords: Rock mass classification systems, tunneling, numerical modelling, empirical methods, Nepal Himalaya

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INTRODUCTION

The Engineering geological behaviour of the rock mass in and around the underground excavation play significant role with respect to the stability of the underground structures. The engineering geological investigations consisting engineering geological surface mapping, sub-surface investigations and laboratory testing of the intact rock samples are more important and has vital role on designing the underground structure. Rock mass characterization of the project area are very important in support estimation and support design of such structure (Jembere and Yihdego, 2016).

The rock mass classification methods are the best tools used extensively to quantitatively describe the quality of rock mass. There are a number of classification systems that exist to classify the rock mass based on a variety of parameters developed by various researcher that has gone a long way on its history (Cai et al., 2007). Most of these classification systems provide the empirical value to different rock mass parameters and the overall rating value of the rock mass is determined by combining all the parametric values (Akram and Zeeshan, 2018). Different Support Categories (type of support) for different rock mass classes are defined by these values. Similarly, the classification systems have become a tool for evaluating the geotechnical parameters of different rock masses from various empirical equations (Ozsan et al., 2009; Ghiasi et al., 2011; Panda et al., 2014).

The commonly practiced rock mass classification systems in the underground excavation work are the Rock mass rating (RMR) by Bieniawski (1989), Tunnelling quality index (Q system) by Borton et al. (1974), Geological strength index (GSI) by Hoek (1994) and the RMi system (Palmstrom, 1995). Among them most commonly and frequently used are RMR and Q system in the entire world for support design in underground works (Stille and Palmstrom, 2007; Barton and Bieniawski, 2008; Barton and Grimstad, 2014; Pells et al., 2017; Akram et al., 2018; Rehman et al., 2018). In the contest of Nepal Himalaya, Q and RMR system is frequently used and practiced in underground excavation design and support works. Similarly, this classification system has been used for the determination of strength of rock mass in Himalayan region.

The rocks mass behaviour depends upon modulus of deformation, in situ stress condition and strength of rock mass (Akram et al., 2018). Empirical relationship is developed from various rock mass classification system to describe characteristic behaviour of rock mass. However, deformations around the tunnel, support performance and stress distributions cannot be always satisfactorily calculated by rock mass classification systems (Ozsan et al., 2009). Absence of homogeneous geology, active tectonic environment, complex topography lead for the necessity of cross check of support design by alternative method. Proposed support system by empirical methods should be analysed for verification by using numerical simulation (Ozsan et al., 2009; Panda et al., 2014; Kanik et al., 2015; Akram et al., 2018).

The present study focuses on detailed rock mass characterization and support analysis of 4.3 km long headrace tunnel of Upper Balephi "A" Hydroelectric Project (UBAHEP). The topography, discontinuities survey and rock mass at UBAHEP area are studied in detail to characterize the project rock mass condition. RMR, Q and GSI system were used to assess, analysis and classify the rock mass of tunnel alignment. The geotechnical evaluation and tunnel support are analysis in detailed using empirical equation derived from various classifications system. The most important factors for designing a tunnel or other underground structure is to provide the required stability. In addition to the empirical methods, the FEM based numerical analyses were also undertaken in order to define the stress distributions, deformations developed around the tunnel and to control the performance of empirical support design.

STUDY AREA

The study area lies on Sindhupalchowk District of Bagmati Province of Nepal (Fig. 1). The project site is located at about 90 km northeast of Kathmandu. The headwork site is located at about 8 km north of Kartike and the powerhouse site is located near Baikunthe village just upstream of the existing suspension bridge over the Balephi River at Kartike near its confluence with Lapse Khola. Most of the components of the project are located at the right bank of the river within Jugal Rural Municipality. Geographically, the project area is located in between the longitudes 85º47'40"E and latitudes 27º57'00"N (at Headwork site) and 85º45'30"E and 27º53'45"N (at Powerhouse site). Physiographically, the project area belongs to the Higher Himalayan zone. The elevation range within the river valley from powerhouse to headwork is 1044 m and 1252 m above mean sea level (amsl), respectively.

Upper Balephi "A" Hydroelectric Project

The Upper Balephi-A Hydroelectric Project is a run of the river scheme designed to divert 20.80 m^3 flow from Balephi River by approx. 45 m long concrete gravity type Diversion Weir with side Intake. The diversion weir has been designed as a simple free overflow weir without control gates. The water from the diversion weir is then diverted through the side intake on the left bank of the Balephi River. The water then passes to the settling basin and headrace pipe followed by about 4235 m long inverted D-shaped 3.90 m diameter headrace tunnel connected to the Surge Shaft having finished diameter of 8 m. To the downstream of the surge shaft, partly surface and partly underground penstock pipe transport the water to the powerhouse located on right bank of Balephi River. Finally the water will be diverted to the Balephi River again through a cut and cover type Tailrace Conduit. The Gross head of the project is 203 m. The installed capacity of projects is 36 MW and generate 212.834 GWh energy per annum. To generate this energy, water has to pass total 4.89 km waterway including headrace pipe, headrace tunnel, penstock pipe and tailrace canal. The design discharge of the project is 20.80 m^3/s at 41.6% exceedance flow.

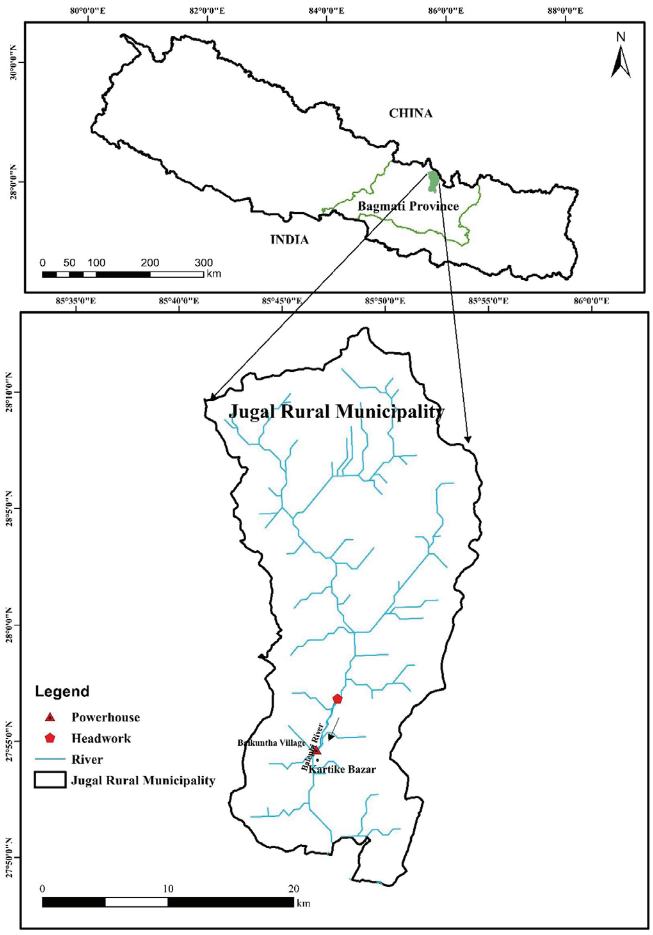

Fig. 1: Location map of study area.

GEOLOGY OF THE PROJCET AREA

Upper Balephi 'A' Hydroelectric Project lies predominantly in the Higher Himalayan physiographic province. However, the project area is very close to one of the major tectonic boundary called Main Central Thrust (MCT). The MCT is within 1-3 km distance from the powerhouse location. As a result, the valley slope between Baramchi Bazaar and Kartike Bazaar are highly disturbed and active with respect to valley slope stability along the left bank of the Balephi River. However, upstream valley from Kartike Bazaar where the project area is located is sound and stable, especially the right bank of the Balephi River, where all engineering structures are located.

The Balephi River has cut through the major discontinuity trending north-east-north to south-west-south in the higher Himalayan crystalline rocks of Palaeozoic to Precambrian age and extended through the valley cutting the rocks of the lesser Himalayan meta-sedimentary sequence and joins with Sunkoshi River at Balephi Bazaar. In this process, this river crosses the MCT zone that lies between Kartike Bazaar and Baramchi Bazaar. The geology in and around the project area is shown in Figure 2a.

The rock mass in the project area is of Palaeozoic to Precambrian age that mainly consists of garnet bearing biotite gneiss, kyanite bearing biotite gneiss, garnetiferous mica schist, mica gneiss. The geology of the project area and cross section of the headrace tunnel alignment are given in Figure 2a,b.

The area is characterized by the varied topography. The landform is controlled mainly by tectonic processes, subordinately by mass wasting and deep share failures. Rugged hills, numerous deep gorges, steep slopes and some of unstable surface failure caused by deep shear failure, and active gullies represent the erosional landform of the area.

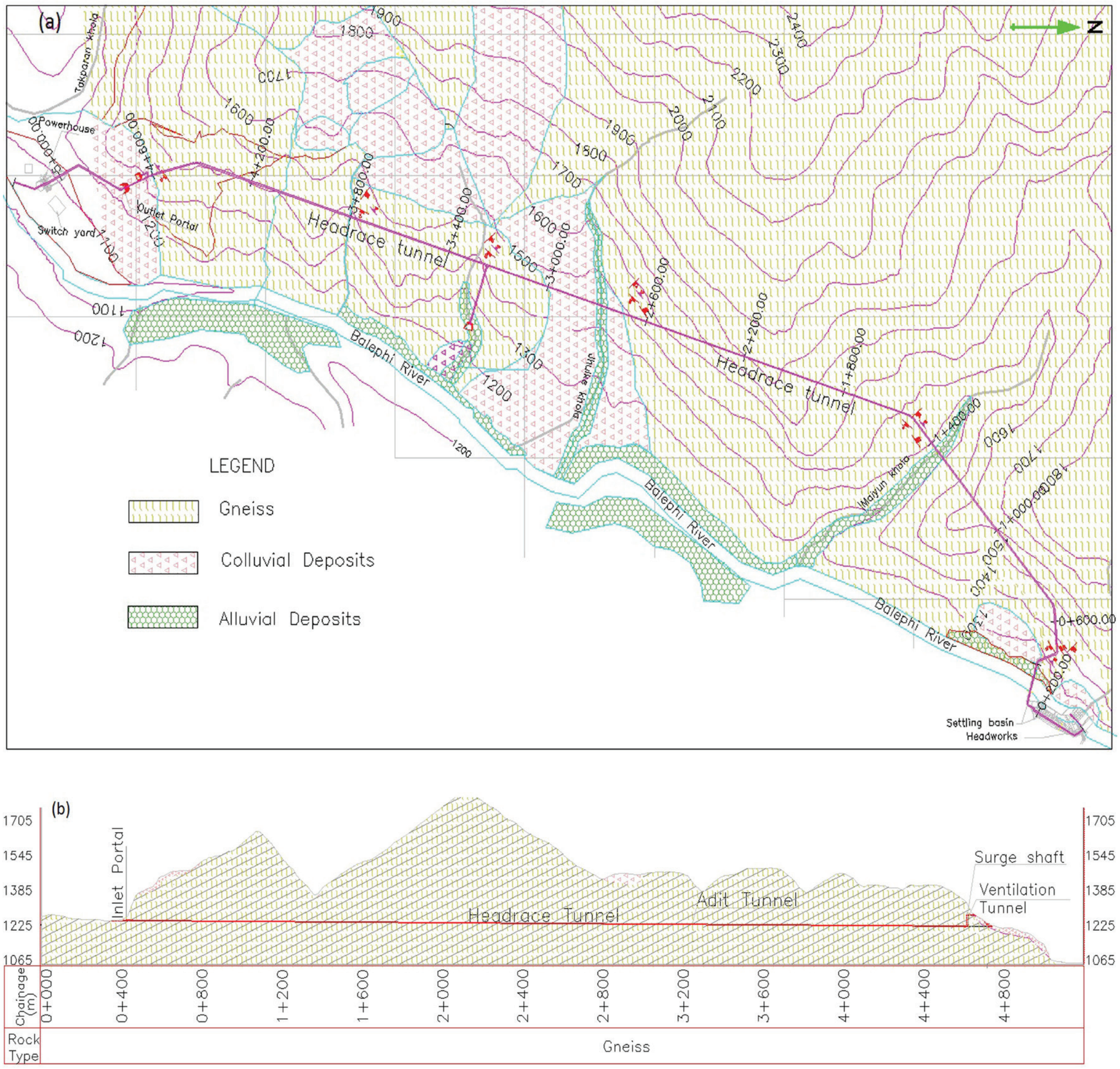

Fig. 2: (a) Geological map of the project area, (b) Longitudinal profile of the headrace tunnel.

Characteristics of headrace tunnel

The main focus on this paper is the engineering geological evaluation of headrace tunnel which is 3.9 m wide and 3.9 m high, inverted D-shaped with 13.58 m^2 cross section area. The total length of the headrace tunnel is 4235 m. The crest level of the weir is 1257 m and the lowest level in the tunnel is at invert of outlet portal, which is 1218 m. Thus, during the normal operation there will be 39 m constant head in the tunnel. The total head is of 203 m. The velocity of water in the headrace tunnel will be less than 1.53 m/s during design flow. The summary of general parameter of headrace tunnel alignment is presented in Table 1.

Table 1: General parameter of tunnel alignment.

Parameters	Properties	Parameters	Properties
Length of headrace tunnel	4295 m	Minimum overburden	180 m
Length of adit tunnel	200 m	Water head at inlet tunnel	10 m
Diameter of tunnel	3.9 m	Water head at adit tunnel	24 m
Maximum overburden	595 m	Water head at outlet tunnel	39 m

MATERIALS AND METHODS

Data collection

Tunnel routes were divided into segments and various traverses were made along the tunnel alignment to gathered necessary geological and geotechnical data. Intensity of jointing, weathering condition of different joint sets, roughness of the joint plane and shearing effect are other most important factors that will govern the rock mass quality (Panda et al., 2014). The rock cover from the surface, distance of an underground structure from the surface topographic slope and tectonic shearing govern the degree of weathering of the joint sets. Hence, physical parameters of all discontinuities such as orientation, spacing, persistence, aperture, roughness, joint sets number, infill material and ground water conditions were closely examined.

Data analysis

The collected geological and geotechnical data in the fields along with physical parameters of all discontinuities sets, geological maps, Q, RMR and GSI values were compared for characterizing and analysing the rock mass condition of the project area. The most important parameter for the rock mass characterization: Hoek-Brown Constants, Modulus of deformation, In situ Stress, Strength of rock mass, Squeezing condition, and Support pressure were selected for the evaluation. Different empirical relationships developed from various researcher were used for geotechnical parameter and tunnel support evaluation and analysis purpose. Dips v.5.1 (Rocscience, 2002a). Unwedge (Ver. 3.005) (Rocscience Inc., 2010), Phase2 (Ver 7.009) (Rocscience, 2001) software package were used for data analysis.

Rock mass strength parameters

Hoek-Brown Constants

The most important and necessary parameter for geotechnical evaluation is Hoek-Brown constants. The Generalised Hoek-Brown failure criterion (Hoek et al., 2002) for jointed rock masses is defined by Eq. (1):

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b\frac{\sigma'_3}{\sigma_{ci}} + s\right)^a \qquad (1)$$

Where σ'_1 and σ'_3 are the maximum and minimum effective stresses at failure respectively. m_b is the value of the Hoek-Brown constant, mi for the intact rock mass and is given by equations 2 to 5 (Hoek et al., 1995; Singh et al.,1997; Hoek and Brown, 1998; Hoek et al., 2002) respectively.

$$\frac{m_b}{m_i} = 0.135(\mathrm{Q}')^{1/3} \qquad (2)$$

$$\frac{m_b}{m_i} = 0.135(\mathrm{Q_N})^{1/3} \qquad (3)$$

$$\frac{m_b}{m_i} = \exp\left(\frac{RMR-100}{28}\right) \qquad (4)$$

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI-100}{28-14D}\right) \qquad (5)$$

s and a are constants which depend upon the characteristics of the rock mass and is given by the following relationships equations 6 to 10 (Hoek et al., 1995; Singh et al., 1997; Hoek and Brown, 1998; Hoek et al., 2002) respectively.

$$\mathrm{s} = 0.002\mathrm{Q}' \qquad (6)$$

$$\mathrm{s} = 0.002\mathrm{Q_N} \qquad (7)$$

$$\mathrm{s} = \exp\left(\frac{RMR-100}{9}\right) \qquad (8)$$

$$\mathrm{s} = \exp\left(\frac{GSI-100}{9-3D}\right) \qquad (9)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \qquad (10)$$

where, D is disturbance factor that's depends upon the degree of disturbance due to blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses, such as those excavated by a tunnel boring machine, to 1 for very disturbed rock masses that have been subjected to very heavy production blasting. The tunnel in this project will be excavated by drill and blast method and the disturbance factor will be intermediate between 0 to 1. Hence, D=0.5 will be used for the numerical modelling.

Modulus of deformation

The in-situ deformation modulus of a rock mass is an important parameter in any form of numerical analysis and in the interpretation of monitored deformation around underground openings. Modulus of deformation is the ratio of stress to corresponding strain during loading of the rock mass, including elastic and inelastic behaviour. Since this parameter is very difficult and expensive to determine in the field, several attempts (equations 11 to 16) have been made to develop methods for estimating its value (Table 6), based upon rock mass classifications.

Based on the analyses of a number of case histories, Bieniawski (1978), Serafim and Pereira (1983), Mitri et al. (1994), Read et al. (1999) proposed the following relationship (equations 11 to 14) for estimating the in-situ deformation modulus, E_{mass}, from RMR:

$$\mathrm{E_{mass}} = 2\mathrm{RMR} - 100 \quad (11)$$

$$\mathrm{E_{mass}} = 10^{\frac{(\mathrm{RMR}-10)}{40}} \quad (12)$$

$$\mathrm{E_{mass}} = \mathrm{E_i}\left[0.5\left\{1-\left(\cos\pi\frac{\mathrm{RMR}}{100}\right)\right\}\right] \quad (13)$$

$$\mathrm{E_{mass}} = 0.1\left(\frac{\mathrm{RMR}}{10}\right)^3 \quad (14)$$

Grimstad and Barton (1993) have found good agreement between measured displacements and predictions from numerical analyses using in situ deformation modulus values estimated from Equation 15:

$$\mathrm{E_{mass}} = 25\log \mathrm{Q} \quad (15)$$

Hoek and Brown (1998) proposed the following relationship Eq. 16 for estimating the in situ deformation modulus, E_{mass}, from GSI:

$$\mathrm{E_{mass}} = \sqrt{\frac{\sigma_{ci}}{100}}\,10^{\left(\frac{\mathrm{GSI}-10}{40}\right)} \quad (16)$$

where, σ_{ci} is the uniaxial compressive strength of the intact rock.

Strength of rock mass

During the initial stages of engineering design, approximate estimates of rock mass strength parameters are frequently necessities (Basarir, 2006). Several authors have published several empirical calculation, some of the popular useful empirical calculations were used to determine the strength of rock mass (σ_{cmass}) based on different classification systems.

Hoek and Brown (1980), Ramamurthy (1986), Kalamaris and Bieniawski (1995), Aydan et al. (1997), Sheorey (1997), Trueman (1988) and Aydan and Dalgic (1998) calculated the strength of rock mass by using RMR from equations 17 to 23 as below:

$$\sigma_{cmass} = \sigma_{ci}\sqrt{exp\ \frac{(RMR-100)}{9}} \quad (17)$$

$$\sigma_{cmass} = \sigma_{ci}exp\left[\frac{(RMR-100)}{18.75}\right] \quad (18)$$

$$\sigma_{cmass} = \sigma_{ci}exp\left[\frac{(RMR-100)}{24}\right] \quad (19)$$

$$\sigma_{cmass} = 0.0016RMR^{2.5} \quad (20)$$

$$\sigma_{cmass} = \sigma_{ci}exp\left[\frac{(RMR-100)}{20}\right] \quad (21)$$

$$\sigma_{cmass} = 0.5\exp\,(0.06RMR) \quad (22)$$

$$\sigma_{cmass} = \frac{RMR}{RMR+\beta(100-RMR)}\sigma_{ci} \quad (23)$$

Bhasin and Grimstad (1996), Barton (2000), Barton (2002) expressed rock mass strength (σ_{cmass}) using the normalization of Q-values expressed by equations 24 to 26 respectively, as below:

$$\sigma_{cmass} = \left(\frac{\sigma_{ci}}{100}\right)7\gamma Q^{1/3} \quad (24)$$

$$\sigma_{cmass} = 5\gamma\left(Q\,\frac{\sigma_c}{100}\right)^{1/2} \quad (25)$$

$$\sigma_{cmass} = 5\gamma(Q_c)^{1/3} \quad (26)$$

Squeezing evaluation

For the stability analysis in Himalayan rock mass, most important parameter that has to be examined is squeezing phenomenon. Different empirical approaches is applied for evaluating squeezing condition of the rock mass at different chainages along the headrace tunnel alignment. For this purpose, equations 27 to 29 proposed by Singh et al. (1992), Goel et al. (1995) and Jethwa et al. (1984) has been examined.

$$\mathrm{H} = 350\mathrm{Q}^{1/3} \quad (27)$$

$$\mathrm{H} = (275\mathrm{N}^{0.33})\mathrm{B}^{-0.1} \quad (28)$$

$$\mathrm{N_c} = \left(\frac{\sigma_{\mathrm{cmass}}}{\gamma\times\mathrm{H}}\right) \quad (29)$$

where, H = overburden in meter (m), N = rock mass number, B = tunnel diameter (m), σ_{cmass} = rock mass uniaxial compressive strength and γ = unit weight of overlying rock mass

Support pressure evaluation

For evaluating support pressure, relationships developed by different researcher has been examined. Equations 30 to 33 proposed by Bieniawski (1974), Barton et al. (1974) and Goel et al. (1995) respectively has been analysed.

$$P_{roof} = \left(\frac{100-RMR}{100}\right) W\gamma \tag{30}$$

$$P_v = \frac{0.2}{J_r} \times Q^{-1/3}\text{, when } J_n > 3 \tag{31}$$

$$P_v = \frac{0.2 \times {J_n}^{1/2}}{3 \times J_r} \times Q^{-1/3}\text{, when } J_n < 3 \tag{32}$$

$$P = \frac{7.5B^{0.1} \times H^{0.5} - RMR}{20RMR} \tag{33}$$

where, Proof = support pressure (kg/m^2) γ = unit weight (t/m^3), P, Pv = support pressure (MPa), W and B = tunnel width (m) and H = tunnel depth (>50 m) below the surface (m).

Stereographic projection

Joints play a major vital role in the determination of support type necessary for the safe construction of the underground structure. The orientation of different joints sets along the alignment of headrace tunnel had analysed by preforming, a joint frequency assessment using the joint mapping data that were collected during engineering geological surface mapping of the project area. The orientation of different joints sets were measured and were plotted on a stereographic projection net by using Dips v.5.1 software (Rocscience, 2002a).

Rock mass characterization

Rock mass classification

Three empirical methods namely Geomechanics Classification or Rock Mass Rating (RMR) proposed by Bieniawski (1989), the Tunnelling Quality Index (Q) proposed by Barton et al. (1974) and Geological Strength Index (GSI) proposed by Hoek and Brown (1997) schemes are used for determination of rock mass characteristics and tunnel support requirements of Upper Balephi 'A' Hydroelectric Project.

For rock mass classification, the proposed tunnel alignment is divided into several geological regions, such that each region would be geologically similar and would require one type of support, i.e., it will not be economical to change tunnel support until rock mass conditions change distinctly; that is, a new structural region can be distinguished.

Geomechanics classification (RMR)

Rock mass rating (RMR) system known as Geomechnical classification was developed by Bieniawski during 1972-1973 in South Africa to assess the stability and support requirements of tunnels (Bieniawski, 1974). These system comprises of six parameters to classify a rock mass. They are uniaxial compressive strength of rock material, rock quality designation (RQD), Spacing of discontinuities, Condition of discontinuities, Groundwater conditions and Orientation of discontinuities. These parameters are assigned numeric values based on their conditions. All the values are algebraically summed for the first five given parameters and then adjusted by the sixth parameter depending on the excavation orientation. The summation of the numeric values for all the parameters is the rating of the rock mass (Eq. 34).

$$RMR89 = R1 + R2 + R3 + R4 + R5 + R6 \tag{34}$$

where, R1 = Uniaxial compressive strength of rock material, R2 = Rock Quality Designation (RQD), R3 = Spacing of discontinuities, R4 = Condition of discontinuities, R5 = Groundwater conditions, R6 = Orientation of discontinuities

Tunnelling quality index (Q)

Barton et al. (1974) at the Norwegian Geotechnical Institute (NGI) proposed a Tunnelling Quality Index, Q-system for estimating rock support in the tunnels. The Q-value gives a description of the rock mass stability of an underground opening in jointed rock masses. Q-value is the product of the ratio of parameters (Hoek, 2007). Based on six parameters, the Q-value is calculated using the following Equation 35:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} \tag{35}$$

where, RQD = Rock Quality Designation, Jn = joint set number, Jr = joint roughness number, Ja = joint alteration number, Jw = joint water reduction factor, SRF = stress reduction factor

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter, which they called the equivalent dimension (De) of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the excavation support ratio (ESR). The value of ESR is related to the intended use of the excavation and to the degree of security, which is demanded of the support system installed to maintain the stability of the excavation. Barton et al. (1974) suggest value of 1.6 for ESR for water tunnel for hydropower projects. The equivalent dimension, De, plotted against the value of Q, is used to define a number of support categories.

Geological strength index (GSI)

Because rock mass classification requires time consuming procedures and has some limitations, Hoek and Brown (1997) suggested a more practical index, called GSI, to be used as an input parameter by their empirical failure criterion. In fact, the only one system that is directly touched to the engineering parameters such as Mohr–Coulomb, Hoek–Brown strength parameters or rock mass modulus is GSI (Cai, 2004).

Sonmez and Ulusay (1999) made an attempt to provide a more quantitative numerical basis for evaluating GSI by introducing new parameters and ratings, such as surface condition rating (SCR) and structure rating (SR). In this modification, the original skeleton of the GSI System has been preserved, and SR and SCR are based on volumetric joint count (Jv) and estimated from the input parameters of RMR scheme (e.g. roughness, weathering and infilling), respectively.

Finite element analysis method

The finite element software package Phase2 (Ver 7.009) (Rocscience, 2001) was used to determine the deformations and failure zones developed around the tunnel excavation and to verify the results of the empirical methods by performing numerical analysis. To design the support system for underground excavation three steps are performed. In the early stage, the amount of tunnel wall deformation prior to support installation is determined. This deformation is determined by using empirical relationship. In the middle stage, the internal pressure that yields the amount of tunnel wall deformation at the point of and prior to support installation is determined. At last, the support is assessed and it is checked whether i) the tunnel is stable, ii) tunnel wall deformation meets the specified requirements, and iii) the tunnel lining meets certain factor of safety requirements. For factor of safety, capacity envelopes are plotted in axial force versus moment space and axial force versus shear force space. Values of axial force, moment and shear force for the liners are then compared to the capacity envelopes.

RESULT AND DISCUSSION

Discontinuity study

The main rock types in the project area are garnetiferous schist, garnet and kyanite bearing banded gneiss and mica gneiss intercalation. However, the intercalation is different from place to place and the intensity of this intercalation effect will mainly govern on the quality of the rock mass. In areas with the domination of garnetiferous schist, the rock mass is weaker in comparison to the areas where garnet bearing banded gneiss dominates the strata.

The main dominating joint set in the project area is the foliation joint (Jf) and there are other two major and systematic joint sets (J1 and J2) that exist at surface rock outcrop. In addition to these joint sets, occasional random joins can be recorded along the rock outcrop of the surface topography. However, the effect of random joints is relatively less in the tunnels that have rock cover exceeding 75 meters. The orientation range of measured systematic joints sets of the project area are given in Table 2.

Table 2: Description and orientation of joint sets.

Description of joint sets	Designation	Orientation of joint sets	
		Dip direction	***Dip amount***
Foliation joints	Jf	N60W– N40W	25 – 50°
Joint set one	J1	N45E – N60E	30 – 50°
Joint set two	J2	S30E – S45E	50 – 70°
Joint set three (Random)	J3	S20W – S45W	20 – 40°

Depending upon joint apertures the surface of joint sets has different degree of weathering and vary from stained joints to highly weathered one. The quality of rock mass was mainly governed by the characteristics of the joint sets of the project. The foliation joints in garnetiferous schist are tight and relatively impermeable whereas the other two cross joints are either open and stained or filled with silty clay. Discontinuity values for Q and RMR ratings of joint parameters are given in Table 3.

The orientation of different joints sets along the alignment of headrace tunnel had been analysed by preforming, a joint frequency assessment using the joint mapping data that were collected during surface engineering geological mapping of the project area. The field data of different joints were plotted on a stereographic projection net by using Dips v.5.1 software (Rocscience, 2002a). The headrace alignment is divided into three sections for the analysis. The results of the joint frequency (joint rosette) and the alignment of different segment of headrace tunnel has been plotted on the joint rosette (Fig. 3, 4).

Table 3: Discontinuity values for Q and RMR ratings.

Section/Class Q	RMR	I		II		III		IV		V	
RQD		80		70		50		25		25	
	UCS		12		7		7		7		4
Jn		12		9		12		12		15	
	RQD		13		13		8		3		8
Jr		1.5		1.5		1.5		1		1	
	Spacing		10		10		8		5		6
Ja		1		2		3		4		8	
	Condition		26		20		13		11		9
Jw		1		1		1		1		0.2	
	water		15		15		10		15		2
SRF		1		2.5		2.5		7.5		10	
	Orientation		-5		-5		-5		-5		-2

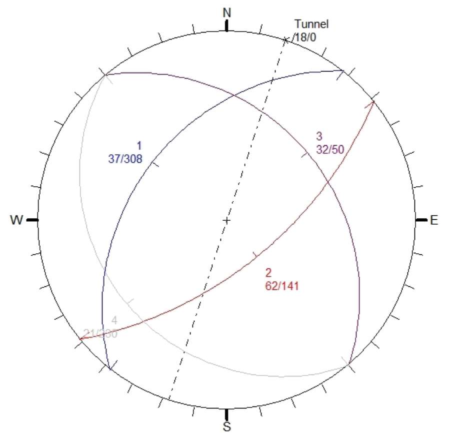

Fig. 3: Stereographic projection of average dip and dip directions of three discontinuities sets in a rock mass and trend of tunnel axis.

(a)

(b)

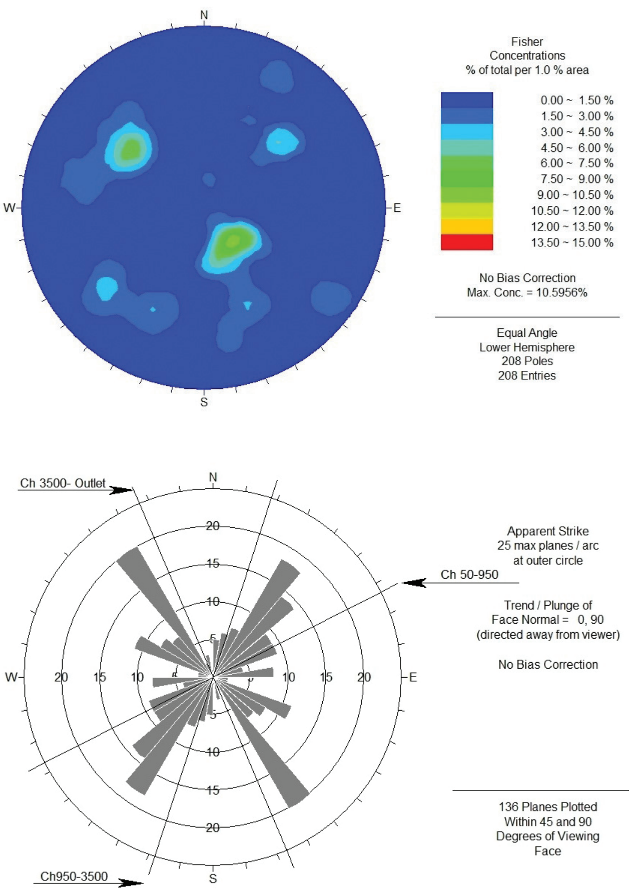

Fig. 4: (a) Fisher concentrations of joint sets (b) Rosette diagram for joints in the study area.

As shown in Figure 4 the first stretch of headrace tunnel up to chainage 950 meters has an angle exceeding 30 degrees from the mean values of the foliation joint (Jf) and other two joint sets (J_1 and J_2). Hence, this stretch of the headrace tunnel is medium favourable with respect to the orientation of the joint sets.

The second stretch of headrace tunnel from chainage 950 to 3500 meters (2550 m tunnel) passes almost parallel to foliation joint (Jf) and joint set two (J_2) and is considered to be unfavourable. Knowing this, the alignment was not changed to reduce the tunnel length. Since these joint sets are very close to similar strike and are dipping opposite of each other, there is a risk of wedge failure if the joints are closely spaced.

The third stretch of headrace tunnel from chainage 3500 meters till its outlet has an angle almost perpendicular to the foliation (Jf) and joint set two (J_2) and has an angle exceeding 30 degrees from the mean of the joint set one (J_1) and hence is favourably oriented.

Rock mass classification

For more authentic result of rock mass classification more than one classification systems should be used (Bieniawski, 1989; Geni et at., 2007). Both qualitative and quantitative assessment of the rock mass along the alignment has been carried out. RMR and Q-method of rock mass classification has been used as a basis for quality assessment of the rock mass and design of rock support. The strength of this method is that it gives quantitative assessment of the different parameters of the rock mass and also suggests the rock support requirement for the respective rock mass class. Table 4 gives a summary of chainage wise classification of rock mass obtained using the RMR, Q and GSI methods, for a number of chainages (section) along the tunnel alignment axis (Fig. 2a,b).

Geotechnical evaluation

The rock mass constant calculated using equations 2 to 10 at different chainages along the tunnel alignment are presented in Table 5. Constant m calculated using equations 2 to 5 (Hoek et al., 1995; Singh et al., 1997; Hoek and Brown 1998; Hoek et al., 2002) are 2.639-9.598, 1.543-9.598, 0.737-11.714 and 0.221-6.232 respectively for weak to hard rock. Similarly constant s calculated using equations 6 to 9 (Hoek et al., 1995; Singh et al., 1997; Hoek and Brown 1998; Hoek et al., 2002) are 0.0004-0.02, 0.00008-0.02, 0.0003-0.0398 and 0.00002-0.0094 respectively for weak to hard rock. The m and s calculated by Hoek et al. 2002 and Hoek et al. 1995 are found to be comparatively low and high as compare to other approaches.

The in situ deformation modulus (E_{mass}) calculated using Eq. 11 to 16 at different chainages along the tunnel alignment are given in Table 6. E_{mass} calculated using RMR relationship by Bieniawski (1978), Serafim and Pereira (1983), Mitri et al. (1994) and Read et al. (1999) from equations 11 to 14 are 0-42 GPa, 2.661-33.49 GPa,12.7-17.1 GPa and 1.968-35.791 GPa respectively for weak to hard rock. The calculated E_{mass} by Grimstad and Barton (1993) using Q relationship Eq. 15 is 0-25 GPa. Similarly, by Hoek and Brown (1998) using GSI Eq. 16, E_{mass} calculated 0.974-21.474 GPa respectively for weak to hard rock. The calculated by GSI system is found to be comparatively low as compare to RMR and Q syatem.

The rock mass strength (σ_{cmass}) values calculated using equations 17 to 26 at different chainages along the tunnel alignment are given in Table 7. The calculated σ_{cmass} by Hoek and Brown (1980), Ramamurthy (1986), Kalamaris and Bieniawski

Table 4: Chainage wise classification of rock mass of the project.

Section/ Class	Chainage	UCS	RQD	Q	Q'	Q_N	Q_c	RMR	GSI	Rock mass Class Q	Rock mass Class RMR
I	1+697–1+730	82	80	10	10	10	8.2	71	65	Fair rock	Good rock
II	0+000 –1+100, 2+481–-2+775, 3+830– 4+150	82	70	2.333	5.833	5.833	1.91	60	50	Poor rock	Fair rock
III	1+100 – 1+697, 1+730 – 2+465, 2+885 – 3+830	65	50	0.833	2.083	2.083	0.541	41	35	Very poor rock	Fair rock
IV	2+752–2+885	53	25	0.069	0.520	0.520	0.036	36	25	Extremely poor rock	Poor rock
V	2+465– 2+481, 4+150 – 4+296	30	25	0.004	0.208	0.041	0.001	27	20	Exceptionally Poor Rock	Poor rock

Table 5: Rock mass constants from different equations.

Section/ Class	Equation No.										
	m					s					a
	2	3	4	5	Average	6	7	8	9	Average	10
I	9.5980	9.5980	11.7141	6.2328	9.28575	0.0200	0.0200	0.0398	0.00940	0.0223	0.50197
II	8.0194	8.0194	7.90848	3.0512	6.74966	0.0116	0.01166	0.0117	0.00127	0.0090	0.50573
III	5.6895	5.6895	4.01230	1.4937	4.22127	0.0041	0.00416	0.0014	0.00017	0.0024	0.51595
IV	3.5824	3.5824	1.01701	0.2811	2.11577	0.0010	0.00104	0.0008	0.00004	0.0007	0.53126
V	2.6395	1.5436	0.73744	0.2215	1.28556	0.0004	0.00008	0.0003	0.00002	0.0002	0.54372

Table 6: Deformation modulus in GPa from different equations.

Section/ Class	Equation No.						
	11	12	13	14	15	16	Average
I	42	33.497	17.1	35.791	25.000	21.474	29.144
II	20	17.783	5.52	21.600	9.198	9.055	13.859
III	0	5.957	14.1	6.892	0.000	3.400	5.058
IV	0	4.467	13.6	4.666	0.000	1.726	4.076
V	0	2.661	12.7	1.968	0.000	0.974	3.051

(1995), Aydan et al. (1997), Sheorey (1997), Trueman (1988) and Aydan and Dalgic (1998) by using RMR from equations 17 to 23 are 0.5197-16.3725 MPa, 0.6113-17.4624 MPa, 1.4326-24.4929 MPa, 6.06079-67.9619 MPa, 0.7797-19.2347 MPa, 2.5265-35.4049 MPa and 1.7419-23.7632 MPa respectively for weak to hard rock. Similarly, the calculated σ_{cmass} by Bhasin and Grimstad (1996), Barton (2000), Barton (2002) using the normalization of Q-values expressed by equations 24 to 26 are 0.9074-33.3894 MPa, 0.4734-38.6581 MPa and 1.35-27.2231 MPa respectively for weak to hard rock. The calculated values of σ_{cmass} by Hoek and Brown (1980) is found to be low as compare to other derived from RMR system. Similarly, the calculated values of σ_{cmass} by Barton (2000) is found to be low as compare to other derived from Q-system.

Tunnel support analysis

The squeezing evaluation condition has been calculated and is presented in Table 8. The Table 9 shows the squeezing potential of rock mass along the tunnel alignment varies from 55.55-754.05, 83.64-513.12 and 0.0048-0.072 according to Singh et al. (1992), Goel et al. (1995) and Jethwa et al. (1984) respectively for weak to hard rock. The values generated from the Table 9 indicated the necessities of suitable support lining for controlling squeezing.

Some of the relationships developed by researchers for calculating support pressure has been calculated and the corresponding results are summarised in Table 9. Support pressure has been calculated for five sections of the tunnel

Table 7: Rock mass strength from different equations.

Section/ Class	Equation No.										
	17	18	19	20	21	22	23	24	25	26	Average
I	16.3725	17.4624	24.4929	67.9619	19.2347	35.4049	23.763	33.3894	38.65811	27.2231	30.3963
II	8.8861	9.71223	15.4877	44.6167	11.0974	18.2991	16.400	21.4735	19.93799	16.7498	18.2660
III	2.4512	2.79468	5.56253	17.2218	3.40208	5.85240	6.7468	11.5451	9.91583	11.0001	7.64928
IV	1.5139	1.74535	3.68262	12.4416	2.16039	4.33556	4.5428	4.10853	2.58164	4.45760	4.15701
V	0.5197	0.61131	1.43265	6.06070	0.77973	2.52654	1.7419	0.90749	0.47346	1.35000	1.64037

Table 8: Squeezing evaluation from different equations.

Section/ class	Equation No.		
	27	28	29
I	754.0521	513.1286139	0.072165954
II	464.2012	429.5073354	0.032682054
III	329.3187	305.7691685	0.004769298
IV	143.5548	193.4226533	0.003648411
V	55.55904	83.64594766	0.004898447

Table 9: Support pressure evaluation from different equations.

Section/ class	Equation No.			
	30	31	33	Average
I	0.00305	0.06189	0.02559	0.03018
II	0.00421	0.10053	0.05303	0.05259
III	0.00621	0.14171	0.20542	0.11778
IV	0.00674	0.48762	0.19518	0.22985
V	0.00769	1.25992	0.12721	0.46494

alignment which varied from 0.00305-0.00769 MPa, 0.06189-1.2721 MPa and 0.02559-0.12721 MPa according to Bieniawski (1974), Barton et al. (1974) and Goel et al. (1995) respectively for weak to hard rock. The support pressure generated by Bieniawski (1974) is found to be comparatively low as compare to Barton et al. (1974) and Goel et al. (1995).

Analysis of tunnel support design

Different rock mass classification systems provide significant input to design rock engineering parameters for tunnel support analysis (Panda et al., 2014). This paper deal with both empirical and numerical methods for support analysis. The most commonly utilized rock mass classification systems such as RMR, Q and GSI were employed to characterize the rock masses along the tunnel alignment and to conduct empirical preliminary support design.

Empirical support estimation

The rock excavation class and respective rock support class with required tunnel support system in different rock mass class are summarized in Table 10. The 3.9 m diameter of inverted D-shaped tunnel is driven through good to extremely poor-quality rock consisting monotonous sequence of garnet schist with occasional bands of banded gneiss. The worst-case scenario is also considered while support designing. Table 4 gives a summary of the values obtained using the RMR, Q and GSI methods, for a headrace tunnel. However, the empirical preliminary support systems proposed by the Q system (Barton, 2002) were applied for the headrace tunnel support design (Fig. 5). Based on Table 10 for drilled and blast tunnel excavation methods, support types proposed for fair rock to exceptionally poor rock mass quality are 50 mm thick steel fibre reinforced shotcrete with Spot bolting of 25 mm diameter 2.5 m long grouted rock bolts to 20 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.1 x 1.3 m spacing with steel set ISMB150 @ 1.0 m spacing respectively.

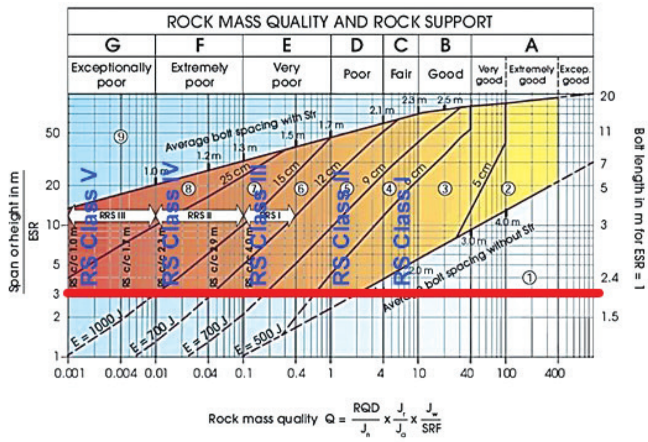

Fig. 5: Empirical support estimation as per Q-chart (after Grimstad and Barton, 1993).

Unwedge analysis

The alignment of headrace tunnel has been divided into three sections for the analysis. The trend of tunnel alignment from the inlet to the outlet with their average attitude of foliation plane which is the prominent discontinuity is given in Table 11. Similarly, the Illustration of wedges formed at headrace tunnel with trend 063°, 018° and 023° are shown in Figure 6.

For the first section (N63) three wedges are taken in consideration. The roof wedge (7) will slide on joint 3. The floor wedge (2) is completely stable and requires no further considerations. The sidewall wedges (6) have different factors of safety as sliding occurs on joint 2. Therefore, to stabilize these wedge blocks, Spot bolting of 2.5 m long and 25 mm diameter was required.

For the second section (N18) five wedges are taken in consideration. The roof wedge (7) will slide on joint 3, and the floor wedge (2) is completely stable and requires no further considerations. The three sidewall wedges (4, 5, 8) have different factors of safety as sliding occurs on different

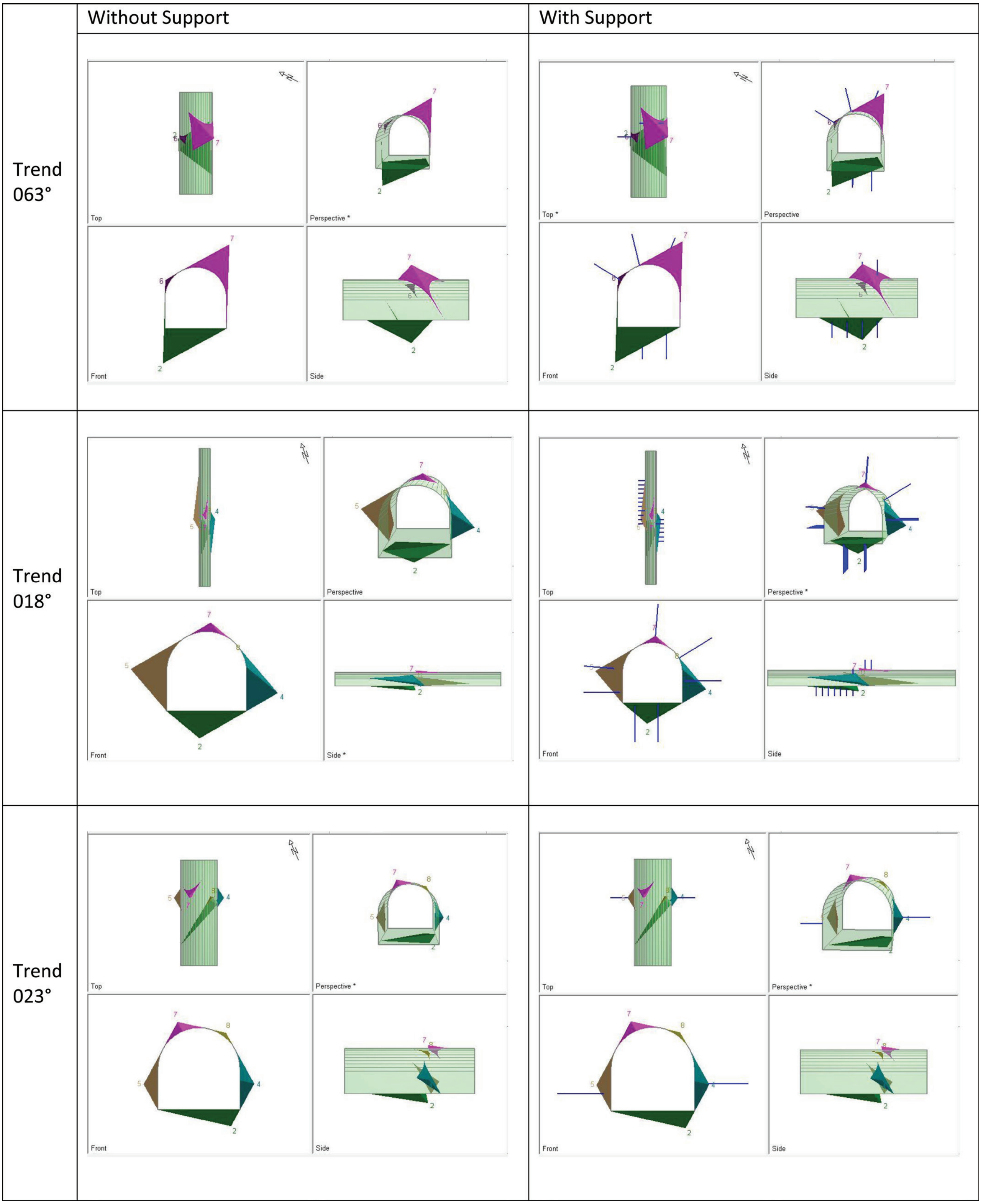

Fig. 6: Wedge formed in the roof, floor and side walls of headrace tunnel with trends of tunnel alignment 063°, 018°, and 023°.

Table 10: Empirical tunnel support categories for the rock masses along the headrace tunnel.

Rock Mass Class	Q value	Rock Support Class	Support Description
Fair Rock and better	>4	RS Class I	5 cm thick steel fibre reinforced shotcrete and Spot bolting.
Poor Rock	1-4	RS Class II	7.5 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.5 x 1.5 m spacing.
Very Poor Rock	0.1-1	RS Class III	10 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.3 x 1.5 m spacing
Very Poor Rock	0.1-1	RS Class III A (for overburden >500 m above tunnel)	15 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.3 x 1.5 m spacing.
Extremely Poor Rock	0.01-0.1	RS Class IV	15 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.1 x 1.3 m spacing.
Exceptionally Poor Rock	<0.01	RS Class V	20 cm thick steel fibre reinforced shotcrete and 25 mm diameter 2.5 m long grouted rock bolts @ 1.1 x 1.3 m spacing with steel set ISMB150 @ 1.0 m spacing

Table 11: Orientation of tunnel alignment with major joints.

Tunnel stretch (km)	Direction	J1	J2	J3
I	63	30°/330°	65°/100°	65°/020°
II	18	20°/310°	50°/110°	50°/065°
III	23	45°/335°	70°/145°	65°/055°

surfaces in their cases. Spot bolting of 2.5 m long and 25 mm diameter was required to stabilise the above wedges formed.

Similarly the last segment (N23) five wedges are formed. The roof wedge (7) will slide on joint 3, and the floor wedge (2) is completely stable and requires no further considerations. The three sidewall wedges (4, 5, 8) have different factors of safety as sliding occurs on different surfaces in their cases. Spot bolting of 2.5 m long and 25 mm diameter was required to stabilise the above wedges formed. Summary of the wedges are illustrated in Figure 7 and is given in Table 12.

Finite element analysis

The numerical analyses were performed for unsupported as well as supported cases for each rock class. The maximum total displacement values around the tunnel show the progression of displacement on the excavation boundary before the support installation. The maximum total displacement values (umax) of tunnel vary between 0.0010–0.044 m.

It can be seen from Table 13 that the maximum total displacement value for class 5 is much higher than all the other sections. The overall total displacement values for the tunnel are very small. The location of this displacement is found at both the walls and floor of the excavation. Displacement of both wall is seen in class II, III, IV, V and at the floor in class I (Table 13).

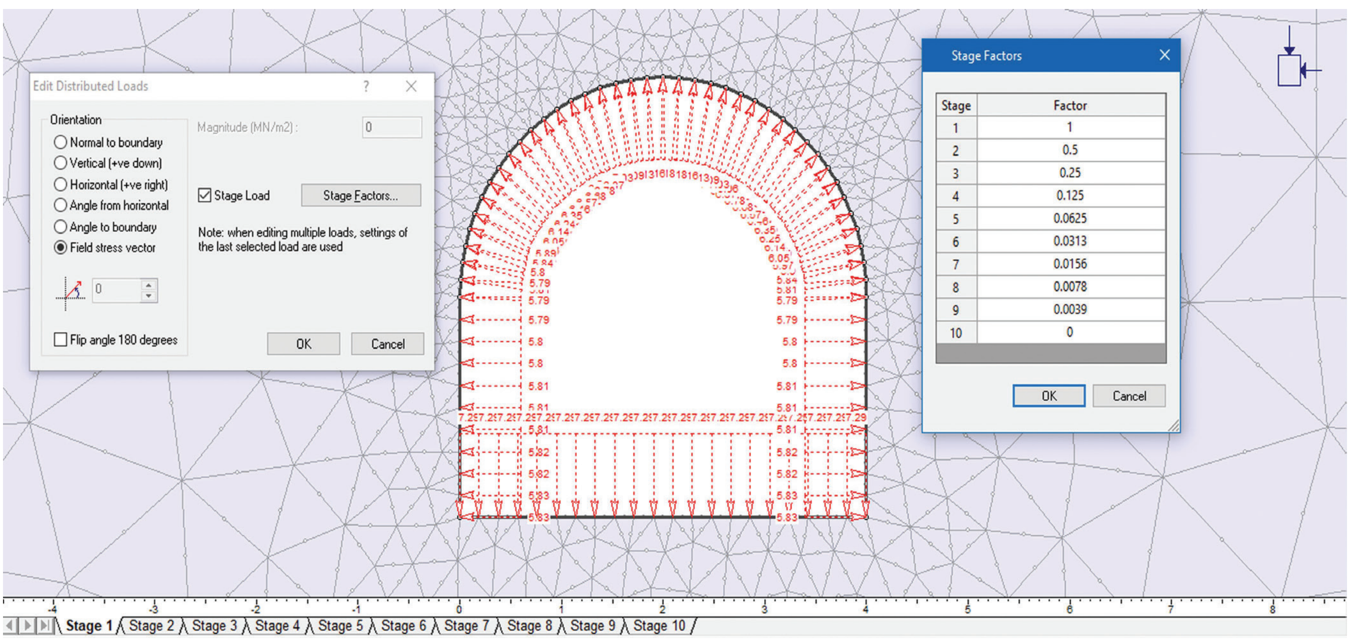

Fig. 7: Various stages of internal support pressure applied for numerical analysis.

Table 12: Summary of wedges formed at headrace tunnel with trend 63°, 018° and 023°.

SN	Wedge	Weight (tonnes)	Failure Mode	FS before Support	Support Required	FS after Support
Trend 063°	Roof wedge (7)	8.756	Sliding on joint 3	0.327	Spot bolting	3.098
	Upper left wedge (6)	0.138	Sliding on joint 2	0.327	25 mm diameter, 2.5 m long	47.944
	Floor wedge (2)	23.939	Stable	Stable		Stable
Trend 018°	Roof wedge (7)	1.287	Sliding on joint 3	5.854	Spot bolting	11.261
	Lower Right wedge (4)	24.293	Sliding on joint 1	17.159	25 mm diameter, 2.5 m long	9.654
	Lower Left wedge (5)	37.221	Sliding on joint 2 and 3	37.221		4.639
	Upper Right wedge (8)	0.0	Falling	0		0
	Floor wedge (2)	28.557	Stable	Stable		Stable
Trend 023°	Roof wedge (7)	0.282	Sliding on joint 3	0.327	Spot bolting	11.439
	Lower Left wedge (5)	0.975	Sliding on joint 2 and 3	0.544	25 mm diameter, 2.5 m long	29.342
	Lower Right wedge (4)	0.987	Sliding on joint 1	0.700		21.623
	Upper Right wedge (8)	0.035	Falling wedge	0		0
	Floor wedge (2)	2.040	Stable	Stable		Stable

Table 13: Summary of maximum displacement, extent of the plastic zone.

Section / class	Maximum displacement (umax)	Displacement location	Extent of the plastic zone (Rp)	unsupported section from face (X)	Distance from Tunnel face/ Tunnel radius (X/Rt)	Plastic zone radius/ Tunnel radius (Rp/Rt)	closure/ maximum closure	Tunnel displacement before support installation	Tunnel displacement after support installation
I	0.001096	Floor	2.383	4	2.051282	1.22205	0.94	0.00103	0.001094
II	0.002702	Floor, wall	2.931	2	1.025641	1.50307	0.74	0.001999	0.002663
III	0.022242	Floor, wall	5.024	2	1.025641	2.57641	0.58	0.012901	0.018386
IV	0.033044	Floor, wall	5.698	2	1.025641	2.92205	0.56	0.018505	0.025674
V	0.044406	Floor, wall	7.541	2	1.025641	3.86717	0.46	0.020427	0.026725

However, the extent of the plastic zones shows that there would be a stability problem in tunnel section, if they are not supported. The plastic zones developed around the unsupported tunnel boundary are illustrated in Figure 8. Therefore, it is more important to consider the extent of then plastic zone rather than the magnitude of the displacements. According to the 2D models, the extents of the plastic zone for the sections are 2.3 m, 2.9 m, 5.02 m, 5.69 m and 7.54 m respectively. Table 15 shows material properties of the rock masses for the numerical analyses for each stretches of the tunnel alignment of the project.

The empirical support design obtained from the Q-system classification was then calculated using unsupported analysis models. The support patterns such as shotcrete thickness, length of rock bolt and its spacing, and steel ribs were used as those proposed in Table 10, and their characteristics applied in numerical analyses are presented (Table 15). After support applications, the changes behaviour in the maximum total displacements and extent of the plastic zones were analysed and results were compared with the unsupported cases. After support installation, it was found that the magnitude of the displacements was slightly reduced. Similarly, the extent of the plastic zones has been reduced significantly by application of the shotcrete and rock bolts for all sections (Fig. 8).

The support capacity diagrams, which are presented as Thrust vs. Shear Force and Thrust vs. Moment, for all classes are generated and presented in the Figure 8. The result shows that, all the points fall within the factor of safety envelope on both plots. Likewise, no yielding was observed in any of the rock-bolts. The support system has achieving a required factor of safety. 2D finite element analyses proved that the empirical support design suggested by the Q-system was sufficient to eliminate the stability problems in the tunnels.

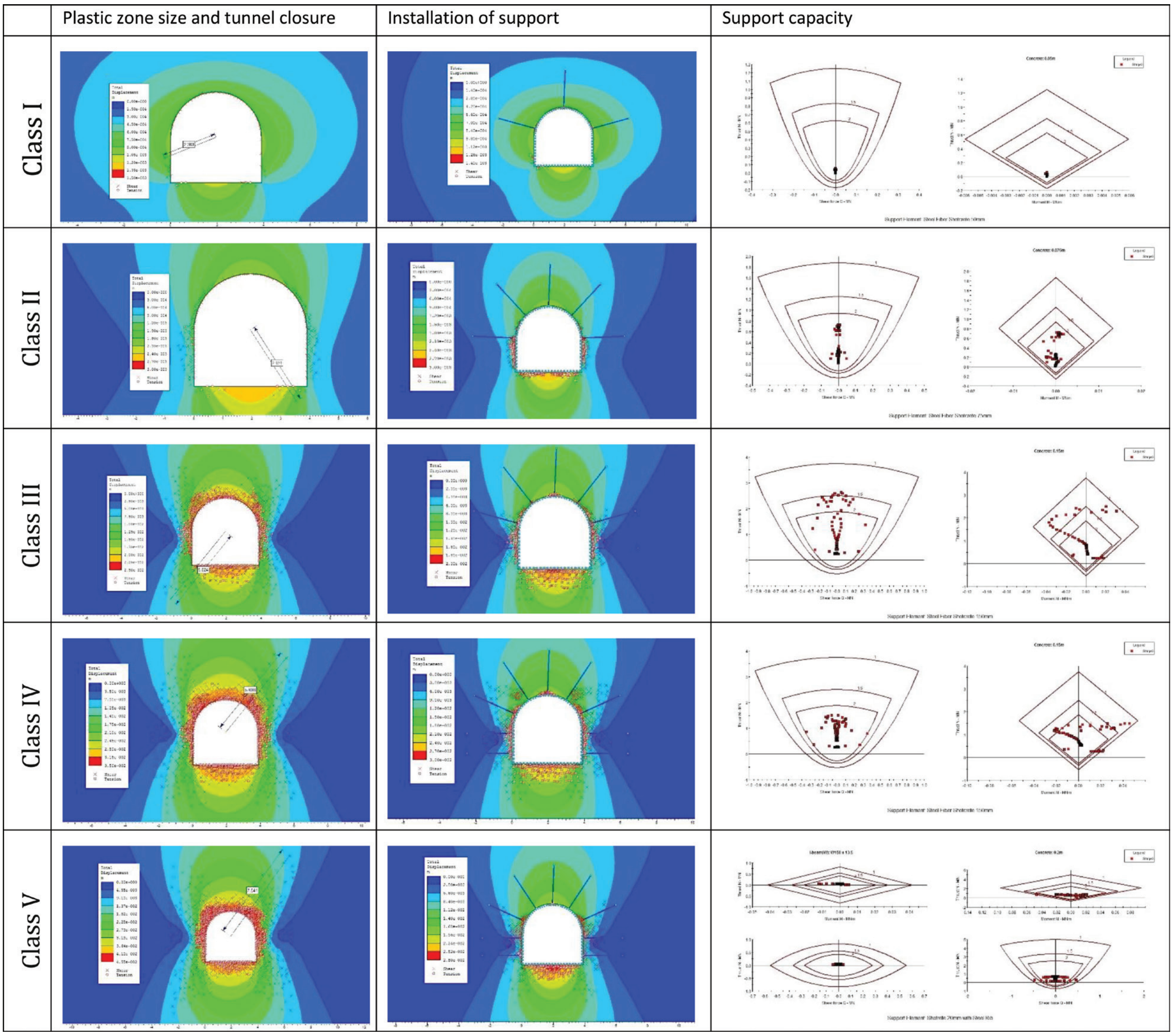

Fig. 8: Numerical analysis of the support estimation.

Table 14: Material properties of the rock masses for the numerical analyses.

Section/ Class	σci (MPa)	mi	MR	D	z (m)	c (MPa)	Φ (deg)	Em (MPa)
I	82	28	675	0.5	218	1.734	55.46	19514.69
II	82	28	675	0.5	207	1.184	50.7	8133.24
III	65	28	650	0.5	350	1.125	38.83	2414.23
IV	53	28	650	0.5	235	0.627	35.7	1220.22
V	30	28	600	0.5	124	0.289	33.6	537.51

Table 15: The characteristics of the support units used in the numerical analyses.

Steel ribs (MB150)		Shotcrete (fibre reinforced) and Concrete	
Section depth	0.15 m	Young's Modulus	25000 MPa
Area	1.91 x 10^{-3} m^2	Poisson Ratio	0.2
Moment of Inertia	7.18 x 10^{-6} m^4	Compressive Strength	25 MPa
Young's Modulus	200000 MPa	Tensile Strength	3.5 MPa
Poisson Ratio	0.25	Rock-Bolts (fully bonded, 2.5 m long)	
Compressive Strength	435 MPa	Diameter	25 mm
Tensile Strength	435 MPa	Bolt Modulus	200000 MPa
Weight	15 kg/m	Tensile Capacity	0.1 MN

CONCLUSIONS

The proposed headrace tunnel of Upper Balephi "A" Hydroelectric Project was designed and analysed using the empirical methods, analytical methods and numerical methods. The RMR, Q and GSI rock mass classification systems were used to characterize the rock mass of the headrace tunnel. The Q-system was used for necessary consequence support determination. The analysis of rock mass condition indicates that headrace tunnel passes through the few weakness zones in small section along the tunnel alignment. Minor rock spalling or plastic deformation is likely to occur in minor tunnel stretch with a rock cover exceeding 500 meters from Ch. 1+600–Ch.2+000. The rest section of the tunnel lie on the fair rock mass condition without stability problem. The conclusion obtained from the analysis is explained as: (1) the support patterns proposed for various rock types by empirical methods was cross-checked by Finite Element Analysis. It was found out that the magnitude of the displacements and extent of the plastic zones can be reduced significantly by the installation of support system, (2) it was found that the support patterns proposed by empirical method was satisfactory for the stability of underground opening. This was observed during the excavation of the tunnel, (3) it was found that both empirical and numerical method was equally essential for estimation of support for underground structures (4) the orientation of different joints sets along the alignment of headrace tunnel had been analysed by preforming, a joint frequency assessment using the joint mapping data that were collected during surface engineering geological mapping of the project area. The alignment of tunnel was designed with respect to the orientation of the joint sets for stabilization of the tunnel, (5) the unwedge analysis assist in identifying the probable wedge failure locations during the excavation of headrace tunnel. The installed support against the wedge was adequate for the stability of the tunnel, (6) therefore, it can be concluded that the combination analysis of both methods will be very useful for reliable accurate support determination.

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