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Control of rock joint parameters on deformation of tunnel opening

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ABSTRACT

Tunneling in complex rock mass conditions is a challenging task, especially in the Himalayan terrain, where a number of unpredicted conditions are reported. Rock joint parameters such as persistence, spacing and shear strength are the factors which significantly modify the working environments in the vicinity of the openings. Therefore, a detailed tunnel stability assessment is critically important based on the field data collection on the excavated tunnel's face. In this context, intact as well as rock mass strength and deformation modulus is obtained from laboratory tests for each rock type encountered in the study area. Finite element method (FEM) is used for stability analysis purpose by parametrically varying rock joint persistence, spacing and shear strength parameters, until the condition of overbreak is reached. Another case of marginally stable condition is also obtained based on the same parameters. The results show that stability of tunnels is highly influenced by these parameters and the size of overbreak is controlled by joint persistence and spacing. Garnetiferous schist and slate characterized using high persistence show the development of large plastic zones but small block size, depending upon joint spacing; whereas low persistence, low spacing and low shear strength in marble and quartzite create rock block fall condition.

1. Introduction

Construction and stabilization of underground openings in complex geological terrain are a challenging work. Opening created for any purpose provides avenues for the release of large amount of pre-existing stress and causes the material to deform elastically. Further, if the stresses are sufficiently high, rocks start to behave inelastically, causing fractures in rock mass and overall reduction in the bearing capacity (Ewy and Cook, 1990). Analysis of in-situ measurements and analytical modeling of excavations show that an area of 2*D* (*D* is the diameter) is mostly affected in terms of stress redistribution and resulting strain (Brown et al., 1983; Kontogianni et al., 2008). Singh et al. (2004) observed that anisotropy in deformational behaviors of rocks is induced if the number of joint set is not very large, and modeling such behaviors of intact rocks as well as joint properties should be incorporated. The deformation and failure of surrounding rocks are widespread and the associated deformation mechanism has been a matter of great concern to researchers (Singh et al., 2011; Kainthola, 2015; Zou and Yan, 2015).

Schubert and Schubert (1993), Schubert (1996), and Steindorfer (1998) have studied the effect of geological structure on deformational behavior of rocks surrounding tunnel using Alpine tunnels' data. The deformation behaviors of rocks surrounding tunnels in varying conditions have been also studied, and different opinions and classifications are proposed accordingly. Five geomechanical modes of classes of rock deformation and failure were proposed by Zhang et al. (1981). The classes were creep–crack, slip-pressure-induced crack, bending-crack, plastic flow–crack, and slip–bending. Furthermore, Wang et al. (1984) analyzed and summarized the proposed classification in actual underground engineering basis and discussed rock deformation mechanisms, structures, methods and characteristics of the classification. Hu and Zhao (2004) recommended three types (roof falling stones, dome transverse tensile collapse and sidewall tangential squeeze slide) of deformation and failure of caverns in low stress condition. Variation of block sizes and shapes not only changes the failure mode, but also leads to considerable changes in the stress distribution around the tunnel (Solak and Schubert, 2004). Pan and Brown (1996) carried out research on the effects of out-of-plane stress and dilation on the convergence and stability of the surrounding rocks and found these parameters to be the major parameters for understanding the failure mechanism around tunnel surroundings. The size of underground excavation and types of rocks also influence

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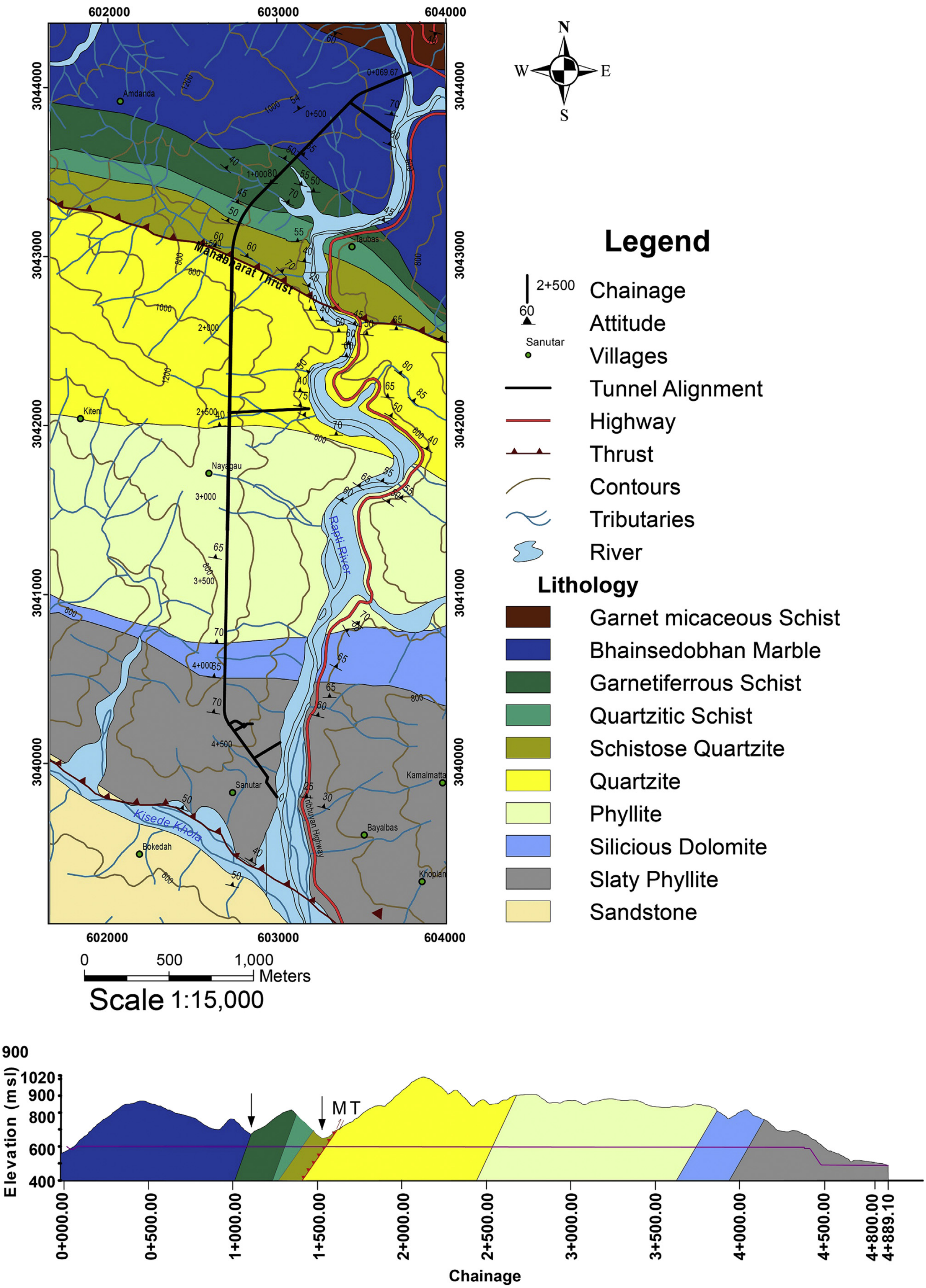

Fig. 1. Geological map of the study area showing tunnel alignment and L-section along the tunnel.

Table 1
Stratigraphy of the Lesser Himalaya Central Nepal (Modified after Stöcklin and Bhattarai, 1977; Stöcklin, 1980).

Complex	Group	Unit	Lithology	Thickness (m)	Age
Kathmandu complex	Bhimphedi Group	Markhu formation	Marble, schist	1000	Late Precambrian
		Khulekhani formation	Quartzite, schist	2000	Precambrian
		Chisapani quartzite	Quartzite	400	Precambrian
		Kalitar formation	Schist, quartzite	400	Precambrian
		Bhainsedhovan marble	Marble	800	Precambrian
		Raduwa formation	Garnet schist, quartzite	1000	Precambrian
		Mahabharat thrust (MT)			
Nawakot complex	Upper Nawakot Group	Robang formation	Phyllite, quartzite schist	200-1000	Paleozoic
		Malekhu limestone	Limestone, dolomite	800	Paleozoic
		Benighat slate	Slate	500-3000	Paleozoic

Note: The data in the dashed box indicate geological formation concerned in the present study.

Table 2
Geotechnical properties of intact rock obtained from laboratory testing.

Stratigraphy	Lithology		σ_{ci} (MPa)	σ_{ti} (MPa)	E_i (GPa)	γ (kN/m^2)	ν	c_i (MPa)	ϕ_i (°)
Bhainsedobhan marble	Marble	Mean (8)	122	11.7	26	26.81	0.18	42	37
		Range	109–138	9.2–12.5	13–38	–	–	–	–
Raduwa formation	Garnetiferous schist	Mean (6)	78	8.6	19	27.40	0.16	32	26
		Range	53–101	6.8–10.3	13–21	–	–	–	–
	Psammatic schist	Mean (8)	91	10.3	22	26.84	0.20	43	31
		Range	61–118	6.9–13.1	14–29	–	–	–	–
	Schistose quartzite	Mean (7)	109	13.8	29	27.13	0.17	38.5	38
		Range	78–124	9.8–14.6	19–34	–	–	–	–
Robang formation	Quartzite	Mean (8)	190	23.2	33	27.32	0.17	60	41
		Range	173–232	21.5–27.1	21–52	–	–	–	–
	Phyllite	Mean (5)	82	10.4	9	26.82	0.26	27	26
		Range	77–94	9.8–11.7	7.2–12.7	–	–	–	–
Malekhu limestone	Siliceous dolomite	Mean (7)	169	21.3	51	28.16	0.18	36	29
		Range	127–213	15.8–24.1	36–68	–	–	–	–
Benighat slate	Slate	Mean (7)	93	6.4	1.5	26.84	0.23	31	28
		Range	76–110	5.3–7.8	0.6–1.8	–	–	–	–

Note: The numbers in the bracket such as (8) are the total number of samples tested for a particular rock type.

Table 3
Estimated geotechnical properties of rock mass.

Stratigraphy	Lithology		σ_{cm} (MPa)	σ_{tm} (MPa)	E_m (GPa)	c_m (MPa)	ϕ_m (°)	m_i	*GSI*
Bhainsedobhan marble	Marble	Mean	20	0.3	37.56	0.9	56	9	49
		Range	8–40	0.03–2.4	109–5	0.2–5.0	48–60		16–77
Raduwa formation	Garnetiferous schist	Mean	12.8	0.09	37.2	0.7	48.5	12	38
		Range	5.3–22	0.01–0.15	0.6–85.92	0.3–1.3	40–52		13–57
	Psammatic schist	Mean	21	0.25	9.6	1.6	52	14	40
		Range	8–26	0.02–0.26	1.14–11.2	0.6–2.1	44–52		26–54
	Schistose quartzite	Mean	20	0.07	4.7	1.5	51	17	40
		Range	10.4–29	0.02–0.2	1.3–11.7	0.7–1.6	46–55		27–52
Robang Formation	Quartzite	Mean	33	0.07	3.5	1.5	53	20	34
		Range	18–53	0.02–0.21	0.8–13.2	0.9–3	45–53		16–47
	Phyllite	Mean	10.8	0.18	2	1.1	41	7	45
		Range	3.5–18.5	0.01–0.8	0.2–7.2	0.35–2.3	27–46		11–62
Malekhu limestone	Siliceous dolomite	Mean	20	0.14	5.7	1.1	47	9	35
		Range	15–34	0.1–0.5	4–20	0.9–2.1	44–52		3–49
Benighat slate	Slate	Mean	10.2	0.1	0.02	0.6	45	7	36.9
		Range	5.6–15.5	0.03–0.3	0.03–0.5	0.3–1.2	41–52		22.1–47.6

Note: σ_{cm} (MPa) is the rock mass compressive strength, σ_{tm} (MPa) is the rock mass tensile strength, E_m (GPa) is the deformation modulus, c_m (MPa) is the rock mass cohesion, ϕ_m (°) is the angle of internal friction, and m_i is the Hoek-Brown material constant for intact rock.

Table 4
Geomechanical classification on the basis of Q-system.

Lithology	Chainage (m)		Q_{max}	Q_{min}	Q_{ave}	*SD*	*CV* (%)
	From	To					
Marble	0+000	0+795.00	18.75	0.03	2.76	2.67	96.62
Garnetiferrous schist	0+795	1+029.73	2.50	0.05	1.05	0.69	65.49
Psammatic schist	1+029.73	1+339.00	3.00	0.37	1.35	0.78	57.68
Schistose quartzite	1+339.00	1+420.00	2.71	0.27	1.26	0.74	58.85
Quartzite	1+420.00	2+476.00	6.25	0.10	1.48	1.06	71.69
Phyllite	2+476.00	3+826.00	6.25	0.07	1.97	1.11	56.07
Siliceous dolomite	3+826.00	4+073.00	1.41	0.17	0.79	0.39	49.48
Slate	4+073.00	4+400.00	1.50	0.17	0.92	0.47	50.91

Table 5
Values of different parameters used in FEM simulation.

Lithology	Poisson's ratio, ν	*K*	Overburden, *h* (m)	σ_1 (MPa)	σ_3 (MPa)
Marble	0.18	0.22	256	6.90	1.50
Garnetiferrous schist	0.16	0.19	165	4.52	0.90
Psammatic schist	0.16	0.19	205	5.50	1.00
Schistose quartzite	0.16	0.19	173	4.70	0.90
Quartzite	0.17	0.20	345	9.40	1.90
Phyllite	0.26	0.35	274	7.34	2.60
Siliceous dolomite	0.15	0.18	167	4.70	0.80
Slate	0.23	0.30	142	3.81	1.10

Note: σ_1 and σ_3 are the vertical and horizontal principal stresses, respectively.

the type and size of deformation. Hatzor et al. (2010) observed the following parameters to be responsible for the stability of shallow karst caverns against collapse, which are listed in order of importance: (1) height of rock cover; (2) span of the opening; (3) intensity of jointing characterized by the number of principal joint sets, mean spacing, and mean persistence discontinuities; (4) orientation of discontinuities; (5) shear strength of discontinuities; (6) strength of intact rock; and (7) groundwater conditions.

Clearly, a number of variables are associated with the deformation of tunnel and many researches have been done to understand them (Wang et al., 2004), but only few can be found on the effect of structural control of joint (mainly joint persistence and spacing) and shear strength of the rock joint. This research focuses on the observation of problems related to these three parameters by taking a case study from a tunnel of Kulekhani III hydroelectric project, Nepal.

2. Study area

2.1. Geological setting

The study area lies partly in the Lesser Himalaya and partly in the Higher Himalaya Zone of the Nepal Himalayas. The tunnel of Kulekhani III passes through five geological formations and eight rock types (Fig. 1). The geological setting of the study area is proposed by Stöcklin and Bhattarai (1977). The present study site lies in the southernmost part of the Mahabharat synclinorium, which consists of Kathmandu complex and Nawakot complex differentiated by the varying metamorphic grade (Table 1). Bhainsedhovan marble comprises coarse-crystalline marble, thickly-to-thinly well bedded, massive with subordinate schist intercalations. The

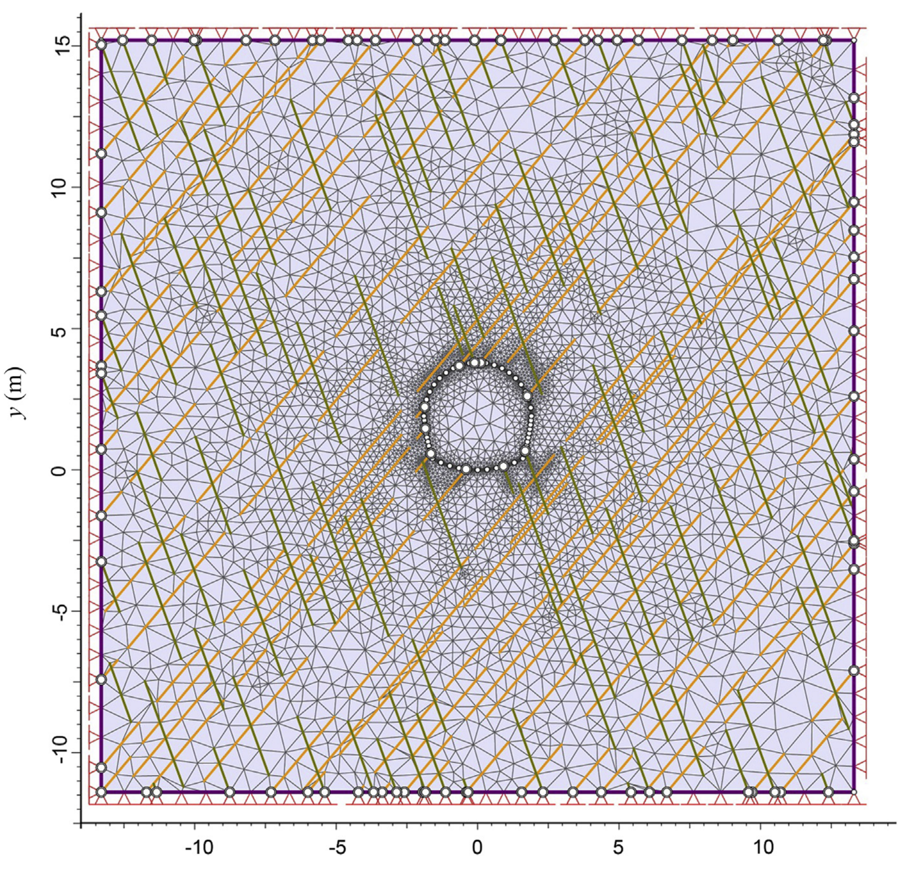

Fig. 2. Diagram showing geometry of the model and boundary condition used in the simulation (joint and other parameters were changed for different rock types).

Raduwa formation comprises coarse-crystalline, highly garnetiferous mica-schist, locally gneissic schist, some quartzite, abundant segregation quartz with green chlorite-schist at base. Benighat slate comprises dark-grey slates, whereas Malekhu limestone is composed of white to grey, siliceous, fine crystalline limestone and dolomite. Robang formation comprises blue–green chloritic phyllites, partly with inter-bedding Dunga quartzite beds.

2.2. Geotechnical details

Rock samples were collected from each of the eight rock units for the determination of seven geotechnical properties, i.e. unit weight (γ), uniaxial compressive strength (σ_{ci}), tensile strength (σ_{ti}), Young's modulus (E_i), Poisson's ratio (ν), angle of internal friction (ϕ) and cohesion (c_i), following the ISRM standards. The observed values of intact rocks are given in Table 2. The strength values show a large range in the measured data and notably mean values are close to the upper range.

The presence of quartz content increases the tensile strength of rocks considerably; however substantial reduction in tensile strength is observed in slate due to the absence of quartz and marked lamination. The intact rock elastic modulus is the highest in dolomite and the lowest in slate as expected. Using Roclab, a Rocscience package, rock mass strength was estimated based on

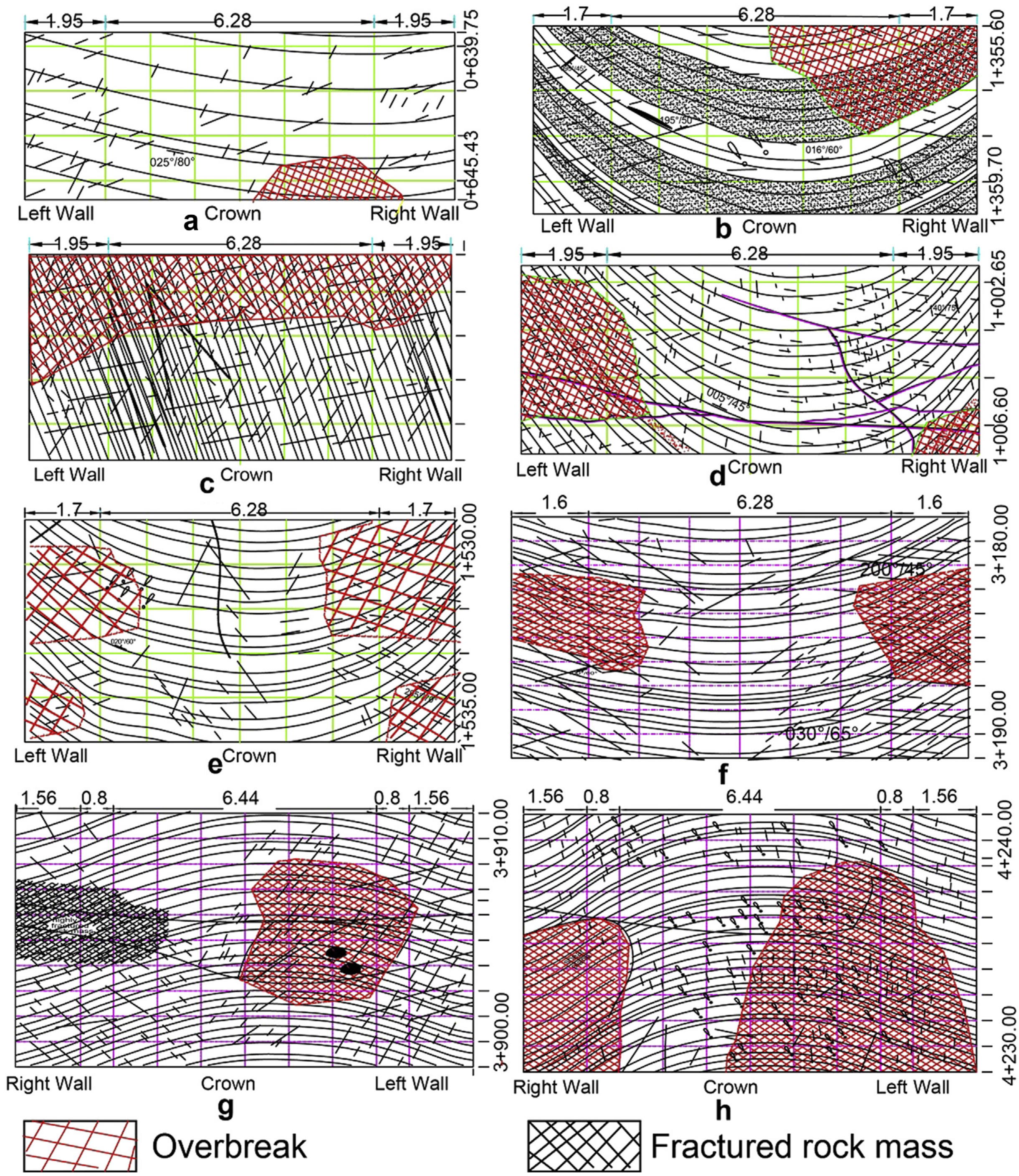

Fig. 3. Tunnel logs of the different lithologies showing geological condition and overbreaks: (a) Marble, (b) Garnetiferous schist, (c) Psammatic schist, (d) Schistose quartzite, (e) Quartzite, (f) Phyllite, (g) Siliceous dolomite, and (h) Slate, at different chainages.

geological strength index (GSI), intact rock properties and depth of overburden (Hoek and Brown, 1997, see Table 3). A remarkable reduction in rock mass strength is observed which is possibly due to the disposition of joints and overburden load. The tunneling quality index (Q-system) proposed by Barton et al. (1974) was used to classify the rock mass of study area as well as for support design. Q-system was preferred over rock mass rating in this study because of the past experiences and successful application of Q-system in the Himalayan condition. The observed values of classification and the maximum, minimum, average values, standard deviation (SD) and coefficient of variation (CV) are given in Table 4.

A number of intact rock strengths and elastic moduli were obtained by rigorous laboratory testing, followed by estimation of rock mass strength and deformation modulus using the intact rock properties. These properties can be used for similar rock conditions and environments with a very high degree of reliability.

3. Numerical modeling

Application of high-end numerical tools has become an important part in the design and construction phases of many engineering structures. These methods are proven useful in simulating the behavior of rock mass and estimate of other parameters such as distribution of stresses, zones of stress and strain localization, etc., due to imposition of any engineering load. A number of researchers have used different models depending upon the conditions to simulate rock mass behaviors in the vicinity of the openings (Jing and Hudson, 2002; Verma and Singh, 2010; Verma et al., 2011; Qiu et al., 2013).

In this study, finite element method (FEM) is used to model the influence of rock joint persistence, spacing and shear strength on the stability of tunnel and subsequent estimation of parameters that are responsible for creation of maximum zone of overbreak to resemble the field condition. FEM has been used previously by various researchers, showing versatility of the method towards successful implementation in various rock engineering problems (Eberhardt, 2001; Vermeer et al., 2003; Lee, 2009; Kainthola et al., 2012; Singh et al., 2013, 2015).

3.1. Material model

The tunnel has been modeled using shear strength reduction (SSR) technique, commonly used in various rock engineering environments. The rock mass is allowed to deform elastoplastically to converge to the final solution. Equivalent Mohr-Coulomb shear strength parameters (c and ϕ) have been obtained from linear curve fitting method, using generalized Hoek-Brown failure criterion. This criterion allows incorporation of GSI into the model, and the benefit of which is the fact that GSI includes rock mass deformation parameters in addition to disturbance factor (Sonmez and Ulusay, 1999).

3.2. Geometry, boundary condition, and meshing

The tunnel serves the purpose of headrace tunnel of a hydroelectric project. The excavation is horseshoe-shaped with a diameter of 3.8 m. The in-situ vertical and horizontal stresses are obtained from the overlying column of rock and the lateral stress ratio (K, a ratio of horizontal to vertical stress) was calculated from the values of Poisson's ratio (Table 5). Since the main emphasis of this study is to observe the deformation around the opening, the far-end boundaries have been restrained in both horizontal and vertical directions (Fig. 2). The discontinuities were then incorporated into the model based on field conditions. The meshing used in the model is graded 3-node triangle, which is further refined near the opening.

4. Results and discussion

From the tunnel logs, different types of deformations can be observed owing to different geological and geotechnical conditions (Fig. 3). The nature and occurrence of joints have a major effect on the failure of specific rock types. Irrespective of rock types, overbreak was frequently observed along the tunnel section. However,

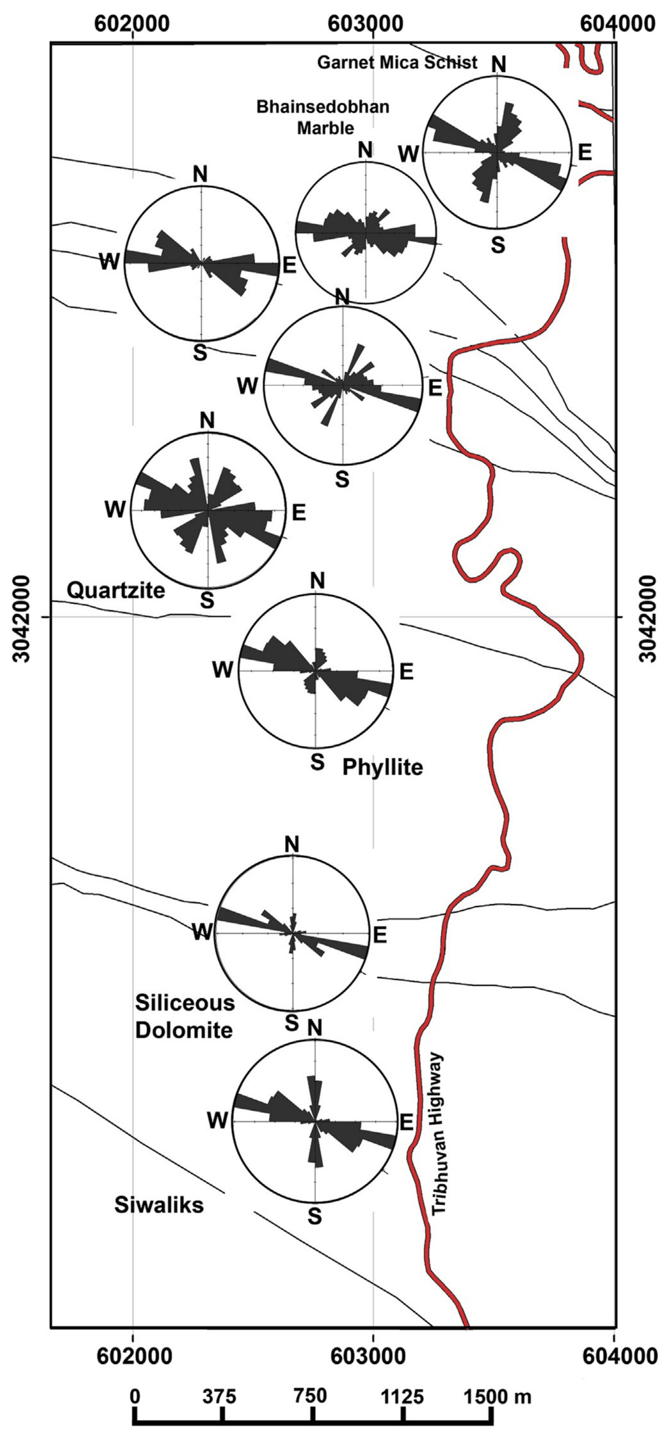

Fig. 4. Rose diagram for joint pattern analysis for all the rock types along the tunnel.

Table 6
Joint conditions for failure and marginal stable conditions for overbreak.

Rock type	Joint	Failure condition				Marginal stable condition			
		Spacing (m)	Persistence (m)	c (MPa)	ϕ (°)	Spacing (m)	Persistence (m)	c (MPa)	ϕ (°)
Marble	J1	2.0	15 (0.5)	0.05	20	2.0	15 (0.4)	0.06	22
	J2	0.3	1 (0.4)	0.08	22	0.8	1 (0.5)	0.10	24
	J3	0.5	1 (0.75)	0.08	22	0.5	1 (0.5)	0.09	24
Garnetiferrous schist	J1	0.5	10 (0.7)	0.01	18	1.0	10 (0.7)	0.01	20
	J2	0.4	1 (0.3)	0.012	20	0.8	1 (0.5)	0.02	20
	J3	0.3	1 (0.2)	0.012	20	1.0	1 (0.5)	0.02	20
Psammatic schist	J1	0.5	0.8 (0.4)	0.1	25	0.6	1 (0.5)	0.11	25
	J2	0.8	0.8 (0.3)	0.1	25	1.0	1 (0.4)	0.11	25
	J3	0.5	15 (0.8)	0.07	20	0.6	8 (0.5)	0.10	22
Schistose quartzite	J1	0.5	1 (0.3)	0.1	23	0.6	1 (0.3)	0.11	23
	J2	0.4	15 (0.8)	0.08	20	0.5	10 (0.6)	0.10	22
Quartzite	J1	0.3	0.8 (0.2)	0.12	26	0.5	0.8 (0.2)	0.15	26
	J2	0.6	15 (0.9)	0.1	22	0.8	15 (0.8)	0.12	25
	J3	0.4	0.8 (0.2)	0.12	26	0.5	0.8 (0.2)	0.15	26
Phyllite	J1	0.7	0.8 (0.4)	0.015	22	1.0	0.6 (0.4)	0.02	22
	J2	0.5	0.8 (0.3)	0.015	22	0.6	0.5 (0.3)	0.02	22
	J3	0.4	15 (0.8)	0.01	18	0.5	10 (0.8)	0.15	22
Siliceous dolomite	J1	0.6	1 (0.4)	0.015	22	1.2	0.8 (0.5)	0.02	25
	J2	0.3	1 (0.4)	0.015	22	0.6	1 (0.6)	0.02	25
	J3	0.4	15 (0.8)	0.01	20	0.8	20 (0.8)	0.15	22
Slate	J1	0.6	1 (0.5)	0.012	20	0.6	1 (0.3)	0.02	22
	J2	0.6	0.5 (0.4)	0.012	20	0.6	0.5 (0.4)	0.02	22
	J3	0.2	15 (0.9)	0.01	18	0.4	8 (0.6)	0.015	20

Note: The data in bracket are equivalent persistence used in FEM model which varies from 0 to 1 (1 being fully persistent joint).

block failure, roof collapse and side wall failure, which are all a type of overbreak, were also frequently observed. In situations where rock masses are consistently disturbed due to excavation processes, persistent joints may be exposed, enabling kinematic feasibility. There will be very little internal deformation for kinematic release in the cases where discontinuities are relatively persistent and adversely oriented. On the other hand, if the kinematic release of the blocks does not occur through the pre-existing planar discontinuities, the failure mechanism will require sufficient internal deformation, and in such cases rock mass is degraded by localized slip along the joints (Eberhardt et al., 2004). Within the same rock type, the persistence and spacing are frequently varied along with variable confining stress. Barton et al. (1974) performed detailed experimental analysis of shear strength of rock and rock joints, and suggested that at very low effective normal stresses, rock joints exhibit a wide range of shear strength owing to surface roughness and variable rock strength.

Eight different rock types are encountered along the tunnel alignment. Almost all rock types have three sets of discontinuities. Discontinuity distribution in slate, quartz-schist, phyllite, psammatic schist and dolomite are more regularly distributed in comparison to quartzite, garnetiferous schist and marble (Fig. 4). As a result, the joint parameters like spacing persistence vary randomly in hard rocks in this region. The tunnel is excavated by drill-and-blast method and supports are given based on Q-system. The Q-value along the tunnel is also varying because of variable discontinuity characteristics. The intact strength of rocks is relatively high but some parts show various degrees of alteration due to water percolation. It is interesting to note substantial reduction in rock mass strength owing to joint alteration, nature and occurrence of joints and change in confining stress condition.

Various ground conditions were modeled using FEM and the results were obtained in the form of deformation behavior under different geological conditions. Parametric study was done by varying joint persistence, spacing, and shear strength parameters under low to medium level stress condition. The maximum and minimum ranges of stress levels correspond to the overburden depth observed at each section.

In numerical modeling, joint conditions like spacing and persistence were varied as observed in ground condition till the rock failure like overbreak resembled ground observation, followed by estimation of the optimum stability conditions (no overbreak) due to these parameters. The limit of optimum stability conditions is presented as “marginal stable condition” in Table 6.

The persistent joints are the most vulnerable in terms of stability. Einstein et al. (1983) suggested that joint persistence is among the parameters which significantly affect the rock mass strength. In the present study, all the sections in different rocks have joints which are highly persistent along which variable spacing is observed in rocks. Out of the eight rocks, garnetiferous schist, phyllite and slate show the most consistent joints with very less spacing. This causes the formation of small block size as is observed during tunnel excavation. While in the other rocks, the block size is relatively large which in this study directly influences the overbreak zone except in slate and garnetiferous schist where persistent joints are arranged in such a way that kinematic sliding initiates along that plane. This is also a result of reduction in joint shear strength values. Tsesarsky and Hatzor (2003) observed that multi-joint rocks around the openings are stable when joint shear strength is higher. The present result also shows as joint cohesion and friction angle increase, the total displacement decreases which leads to the stability of the tunnel surrounding rocks. The observed displacement conditions and their corresponding tunnel logs are presented in Figs. 3–5. Rose (1982) studied the deformation characteristics in terms of degree of jointing based on RQD and classified the deformation of rocks on the basis of Terzaghi’s classification. According to his findings, the expected overbreak is

$$W = 0.5B \text{ to } 0.2(B + h_t)\ (RQD = 75\text{–}85) \tag{1}$$

$$W = (0.2\text{–}0.6)\ (B + h_t)\ (RQD = 30\text{–}75) \tag{2}$$

where B and h_t are the width/span and height of opening, respectively.

This study correlates well for the failure condition when RQD is 30–75, overbreak is $(0.2\text{–}0.6)\ (B + h_t)$, but the result contradicts his

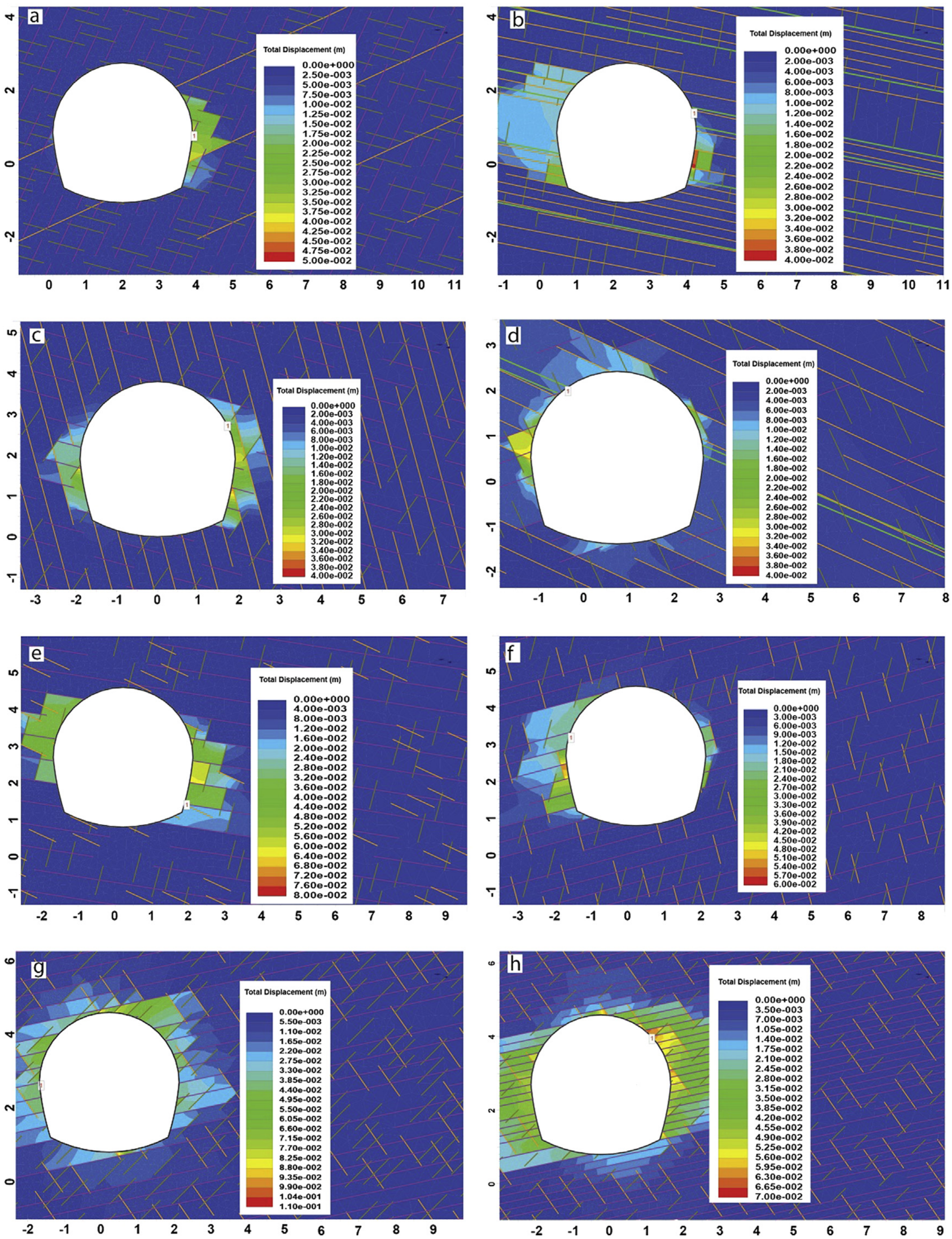

Fig. 5. Displacement contours for different rock types in overbreak condition. (a) Marble, (b) Garnetiferous schist, (c) Psammatic schist, (d) Schistose quartzite, (e) Quartzite, (f) Phyllite, (g) Siliceous dolomite, and (h) Slate.

observations on situations where marginal stable conditions (critical or limiting stability) are observed. In such conditions, deformation is observed but there were no overbreaks observed in all rock types.

From the above analyses, it is found that with the exception of garnetiferous schist and slate, the extent of overbreak is less than 1 m. No squeezing conditions were observed but block fall either from roof or from side walls is common as was also observed in the field (Figs. 3–5). The maximum probability of rock block fall condition was found at the lowest values of joint shear strength along with low joint spacing and low persistence like in quartzite and marble. Therefore, it is established that joint strength, spacing and persistency are very sensitive parameters for block failure in tunnel surrounding for all analyzed rock types. However, Tsesarsky and Hatzor (2006) found that the extent of loosening above excavation in blocky rock masses is predominantly controlled by joint spacing and only to a lesser extent by joint shear strength. Yeung and Leong (1997) studied the effects of joint attributes on tunnel stability using discontinuous deformation analysis (DDA) and found that block volume based parameter may be more appropriate than joint spacing as a measure of the effects of block size on tunnel stability, and the tunnel excavated in a blocky rock mass is likely to be more stable if joint spacing is larger with higher joint friction angle. The present study also confirms that with an increase in joint friction and spacing, the stability of the rock mass around tunnel increases.

5. Conclusions

This paper attempts to address some of the issues observed during tunneling in low stress conditions by taking a case of Kulekhani III hydroelectric project, Nepal. A detailed field study was done to collect rock joint parameters, mostly persistence and spacing, which have a major influence on stability of openings. The following observations are drawn.

(1) Stability analysis was done using FEM by parametrically varying rock joint persistence, spacing and shear strength parameters until the condition of overbreak was achieved. Another case of marginally stable condition was also obtained based on the same parameters.
(2) The purpose of FEM is to obtain the optimum condition for the formation of overbreak similar to ground conditions. It was observed that the size of overbreak is controlled by joint persistence, spacing and shear strength of rock joints.
(3) Garnetiferous schist and slate having high persistence show the development of large plastic zone but smaller block size owing to low joint spacing, whereas low persistence and low spacing in marble and quartzite create rock block fall condition and large block size along with significant reduction in the overbreak zone.
(4) Squeezing condition was not observed during FEM simulations as well as field condition; however block failure and side wall collapse were observed in almost all the rock types.
(5) The present study gains huge significance for the Himalayan region. A number of infrastructure-related activities are being undertaken in the fragile Himalayan terrain and the results obtained from this study can be used as markers for excavation in similar litho-tectonic units. The result can come in handy for design and execution of the similar works.

Conflict of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

References

Barton N, Lien R, Lunde J. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics 1974;6(4):189–236.

Brown ET, Bray JW, Ladanyi B, Hoek E. Ground response curves for rock tunnels. Journal of Geotechnical Engineering 1983;109(1):15–39.

Eberhardt E. Numerical modelling of three-dimension stress rotation ahead of an advancing tunnel face. International Journal of Rock Mechanics and Mining Sciences 2001;38(4):499–518.

Eberhardt E, Stead D, Coggan JS. Numerical analysis of initiation and progressive failure in natural rock slopes—the 1991 Randa rockslide. International Journal of Rock Mechanics and Mining Sciences 2004;41(1):69–87.

Einstein HH, Veneziano D, Baecher GB, O'Reilly KJ. The effect of discontinuity persistence on rock slope stability. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 1983;20(5):227–36.

Ewy RT, Cook NGW. Deformation and fracture around cylindrical openings in rock - I. Observations and analysis of deformations. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 1990;27(5):387–407.

Hatzor YH, Wainshtein I, Mazor DB. Stability of shallow karstic caverns in blocky rock masses. International Journal of Rock Mechanics and Mining Sciences 2010;47(8):1289–303.

Hoek E, Brown ET. Practical estimate of rock mass strength. International Journal of Rock Mechanics and Mining Sciences 1997;34(8):1165–86.

Hu XS, Zhao FS. Basic deformation and failure types of surrounding rock mass of underground excavation in the low ground stress region. Hydrogeology & Engineering Geology 2004;3:79–81 (in Chinese).

Jing L, Hudson JA. Numerical methods in rock mechanics. International Journal of Rock Mechanics and Mining Sciences 2002;39(4):409–27.

Kainthola A, Singh PK, Wasnik AB, Sazid M, Singh TN. Finite element analysis of road cut slopes using Hoek & Brown failure criterion. International Journal of Earth Science and Engineering 2012;5(5):1100–9.

Kainthola A. Tunnelling challenges and solutions - Indian scenario. Tunneling Association of India 2015;4(2):89.

Kontogianni V, Papantonopoulos C, Stiros S. Delayed failure at the Messochora tunnel, Greece. Tunnelling and Underground Space Technology 2008;23(3):232–40.

Lee JS. An application of three-dimensional analysis around a tunnel portal under construction. Tunnelling and Underground Space Technology 2009;24(6):731–8.

Pan XD, Brown ET. Influence of axial stress and dilatancy on rock tunnel stability. Journal of Geotechnical Engineering 1996;122(2):139–46.

Qiu Y, Yang X, You C, Xu Q. Numerical simulation test of tunnel's deformation under different levels of horizontal stress. In: Fourth International Conference on Transportation Engineering. Chengdu, China: American Society of Civil Engineers; 2013. p. 2071–5.

Rose D. Revising Terzaghi's tunnel rock load coefficients. In: Goodman RE, Hueze F, editors. Proceedings of the 23rd Symposium on Rock Mechanics. Berkeley, CA: Society of Mining Engineers, AIMMP; 1982. p. 951–60.

Schubert W. Dealing with squeezing conditions in Alpine tunnels. Rock Mechanics and Rock Engineering 1996;29(3):145–53.

Schubert W, Schubert P. Tunnels in squeezing rock: failure phenomena and counteractions, assessment and prevention of failure phenomena in rock engineering. Istanbul: A.A. Balkema; 1993. p. 479–84.

Singh AK, Singh KK, Vashisht NK, Boruah P. Pre-grouting in the HRT of Teesta III hydroelectric power project, North Sikkim, India. In: International Conference on Underground Space Technology (ICUST 2011). Bangalore, India: ISEG & NIRM; 2011. TT-12-1-9.

Singh M, Singh B, Choudhari JB, Goel RK. Constitutive equations for 3-D anisotropy in jointed rocks and its effect on tunnel closure. International Journal of Rock Mechanics and Mining Sciences 2004;41(Supp. 1):652–7.

Singh PK, Wasnik AB, Kainthola A, Sazid M, Singh TN. The stability of road cut cliff face along SH-121: a case study. Natural Hazards 2013;68(2):497–507.

Singh PK, Kainthola A, Prasad S, Singh TN. Protection measures on the failed cut-slope along the free expressway, Chembur, Mumbai, India. Journal of Geological Society of India 2015;86(6):687–95.

Solak T, Schubert W. Influence of block size and shape on the deformation behavior and stress development around tunnels. In: EUROCK 2004, Proceedings of the 53rd Geomechanics Colloquium. Salzburg: Austria; 2004. p. 1–6.

Sonmez H, Ulusay R. Modifications to the geological strength index (GSI) and their applicability to stability of slopes. International Journal of Rock Mechanics and Mining Sciences 1999;36:743–60.

Steindorfer A. Short term prediction of rock mass behaviour in tunneling by analysis of displacement monitoring data. PhD Thesis. Graz: Institute for Rock Mechanics and Tunneling, Graz Technical University; 1998.

Stöcklin J, Bhattarai KD. Geology of Kathmandu area and Central Mahabharat range, Nepal Himalaya. Report of Department of Mines and Geology. Kathmandu: UNDP; 1977. p. 86.

Stöcklin J. Geology of the Nepal and its regional frame. Journal of the Geological Society of London 1980;137:1–34.

Tsesarsky M, Hatzor YH. Deformation and kinematics of vertically jointed rock layers in underground openings. In: Ming Lu, editor. Development and Application of Discontinuous Modeling for Rock Engineering. Lisse: Swets & Zeitlinger; 2003. p. 93–102.

Tsesarsky M, Hatzor YH. Tunnel roof deflection in blocky rock masses as a function of joint spacing and friction – a parametric study using discontinuous deformation analysis (DDA). Tunnelling and Underground Space Technology 2006;21(1):29–45.

Verma AK, Singh TN. Assessment of tunnel instability — a numerical approach. Arabian Journal of Geosciences 2010;3(2):181–92.

Verma AK, Bajpai RK, Singh TN, Narayan PK, Dutt A. 3D instability analysis of an underground geological repository—an Indian case study. Arabian Journal of Geosciences 2011;4(7):1173–88.

Vermeer PA, Möller SC, Ruse N. On the application of numerical analysis in tunnelling. In: Proceedings of the 12th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering. Singapore: World Scientific; 2003. p. 1–6.

Wang S, Yang Z, Liu Z. Rock stability analysis of underground works. Beijing: Science Press; 1984 (in Chinese).

Wang S, Liu J, Tang C, Li LC, Zhao XD. Stability analysis of a large-span and deep tunnel. International Journal of Rock Mechanics and Mining Sciences 2004;41(Supp. 1):870–5.

Yeung MR, Leong LL. Effects of joint attributes on tunnel stability. International Journal of Rock Mechanics and Mining Sciences 1997;34(3/4):348.e1–348.e18.

Zhang Z, Wang L, Wang S. Principle of engineering geology. Beijing: Geological Publishing House; 1981.
Zou H, Yan EC. Surrounding rock deformation and failure mechanism of Wudang group schist tunnel: a case study of Tongsheng tunnel. The Electronic Journal of Geotechnical Engineering 2015;20(Bund. 11):4457–78.

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