Severity Prediction of Stress Induced Instability during Subsurface Excavation in the Himalaya: Case Studies

Ghosh Roy, Mainak[1], Singh, Satinder[2] Ganvir, Sunil, J[3] and Singh, A.P[4]

[1,2,3]NHPC Limited, Faridabad 121003, Haryana, India
[4]Director, Explore Engineering Consultants Pvt. Ltd., Noida, India
mainakgr@yahoo.co.in

Abstract. Time dependent deformation in rock under high in-situ stress conditions in the Himalaya poses a number of unique challenges in underground excavation leading to time and cost overrun. Objective of this paper is severity prediction of potential stress induced instabilities based on the back analysis of already encountered rock bursting and squeezing incidences in two deep seated tunnels located in two different geological formations in the Himalaya. Geo-mechanical properties of the encountered rock and different in-situ stress factors like σ_θ (tangential stress), σ_V (vertical stress), σ_H (horizontal Stress), σ_{cm} (compressive strength of rockmass) and SRF (stress reduction factor) have been determined from the already encountered zone for severity analysis. Based on the analysis, three prediction theories were adopted for identifying potential zones of stress induced instabilities during tunnel excavation. First prediction approach was attempted utilizing storage energy potential of the rock based on its stress-strain characteristics under uniaxial compression testing in the laboratory. The second approach involves utilization of different in-situ stresses to determine Modified Overload Factor (OFM) proposed by Deere et al. (1969). The third approach is to determine in-situ vertical stress to intact rock strength ratio and incorporate in Empirical stability classification proposed by Hoek and Brown (1980). These stress factors are then incorporated in empirical rockmass classification RMR to propose modified RMR for true assessment of rockmass classification in rock bursting and squeezing ground conditions. In squeezing ground, support pressure has been determined from Goel (1994) approach.

Finally, based on the above analysis, a site specific severity prediction model has been proposed which can be used as an impetus for future study. This attempt may assist in framing a comprehensive Geo-technical Baseline Report by predicting potential geotechnical hazards under stress-induced conditions often encountered during tunnelling in the Himalaya and adopting suitable risk mitigation measures.

Keywords: Rock Bursting, Squeezing, Linear elastic energy, Modified Overload Factor, Empirical stability classification, Goel (1994) approach

1.0 Introduction

The high mountainous topography causes high overburden pressure in the underground structures causing rock bursting in hard, massive rock, squeezing in soft, jointed rock and other stability problems. When the tectonic stresses do not get path for release through joint or any other weak plane in the rock, localized stress concentration takes place. When these locked-in stresses exceed the yield strength of the rock, elasto-brittle failure results. Depending on the magnitude of the stress release, popping/spalling/rock bursting occur leading to rupture of the rock. The second type of common stress induced deformation during underground excavation is squeezing which causes failure of soft and jointed rock due to overburden pressure. Unlike rock bursting, sudden failure does not take place during squeezing, rather it is a time dependent phenomenon where elasto-plastic deformation takes place before eventual failure. Prediction and control of both rockburst and squeezing are challenging issues across the world (Cai, 2016). Few prediction theories such as Brittleness coefficient (Peng and Wang, 1996), Linear elastic energy (Qiao and Tian, 1998), Burst energy coefficient (Goodman,1980; Li et al.,1996), Strain energy storage coefficient (Kidybiski,1981; Wang et al.,1998), Discriminant Index (Tao) for rockburst and various empirical (Singh et al, 1992, Goel, 1994, Jain et al, 2022), semi-analytical (Aydan et al, 1993, Hoek & Marinos, 2000) and analytical (Duncan-Fama,1993, Carranza-Torrest & Fairhurst,2000) models to predict squeezing are proposed over the past three decades based on the laboratory test of physico-mechanical properties of rock mass but most of them are assumptive or theoretical. Few attempts have been made to incorporate in situ or field stress parameters and modify existing rockmass classifications like RMR for judicious characterization of rockmass in overstressed conditions.

The severity prediction is often estimated by back analyzing case histories where examples of failure have been carefully documented (Sakurai 1993). Keeping this in view, the present study intended to carry out investigations of rock bursting and squeezing events experienced in two of the prestigious projects in the Himalayas based on the back analysis of geomechanical characteristics of rock and in-situ stress conditions. An attempt has been made to incorporate these in-situ stress components into Bieniawski's empirical rockmass classification, RMR and propose a site specific Modified RMR under high in-situ stress conditions.

2.0 Case study-1: HRT in Dhauladhar range, Himalaya

The study area selected is a 9km long segment of 32km long head race tunnel (HRT) located within Dhauladhar range of NW Himalaya as a part of development of hydroelectric power. The tunnel is presently under excavation in Quartzite belonging to Kullu group of rocks comprising of quartzites, granite gneiss and mica schist. Excavation of the tunnel is being carried out from two faces viz. face-3 and face-4. In the reach between face-3 and face-4 of HRT, rock cover reaches a maximum of upto ±1600m. No noticeable rock spalling/rock burst observed in granite gneiss during excavation. However as tunnel entered into brittle and massive quartzite, rock bursting incidences have been reported intermittently towards crown and spring level of HRT (Fig. 1). Thus, for evaluating the tunnel from the viewpoint of potential rock bursting, tunnelling in Quartzite has been selected and segmented based on rock strength and high superincumbent cover.

(a) (b)

Fig. 1. Rock bursting observed from (a) crown of HRT Face-4 and (b) right wall of HRT Face-3

2.1 Severity prediction of rockburst

Three prediction theories for identifying potential zones of overstressed conditions in the tunnel has been adopted in this paper to utilize both the geomechanical properties of the rock as well as in-situ stress characteristics of the surrounding tectonic environment. These theories quantify the bursting potential of rock based upon release of stored elastic energy (Klein et al., 2001), depth of overburden cover, orientation and magnitude of principal tectonic stresses (Deer et al., 1969) and strength of the intact rock (Hoek and Brown,1980). These are the most accepted and well applied theories and the main advantages of these theories are their relative simplicity in application for real-time prediction/ assessment of rock bursting during excavation of tunnel in stress induced conditions. Moreover, the above theories used strength of the intact rock and insitu stress values, both of which are the important parameters that have been used in the Modified RMR by the authors.

Severity prediction based upon the geomechanical properties from laboratory test. The behavior of the rock under overstressed conditions depends on the stress-strain characteristics of the rock (Klein et al., 2001). Rock that fails in a brittle manner will fracture when overstressed. For validating the rock bursting potential of quartzite from the study area, core samples were collected from the rock burst area encountered in the tunnel and rock mechanic test of the core samples (Table 1) was carried out in the laboratory. The core samples exhibit core disking which is a typical manifestation of stress effect (Fig. 2).

The stress-strain curve and failure pattern of Manikaran quartzite under uniaxial compression as shown in Fig. 3, depicts elasto-brittle type of failure with axial splitting and violent rupture typically exhibited by overstressed rock during rock bursting. Thus, stress-strain behaviour of quartzite from the study area in the laboratory, depicts brittle behaviour and has a tendency for bursting under overstressed condition.

Table 1. Geomechanical properties of quartzite

Sl No.	Properties	Location of Samples	
		HRT Face-3	HRT Face-4
1.	Unconfined compressive strength (σ_{ci}), MPa	100-145	72.7-86.9
2.	Brazilian Tensile strength, MPa	13	6-10
3.	Modulus of Elasticity (E_S), GPa	41	38
4.	Shear strength parameters (Cohesion, c and Angle of Internal Friction, Ø)	c=31 MPa Ø=48.5°	c=25.5 MPa Ø=42.5°
5.	Poisson's ratio	0.13-0.26	0.17-0.22

Fig. 2. Core disking due to stress effect in Quartzite

In uniaxial compression test, the elastic energy stored in rock specimen before reaching peak strength is defined as "linear elastic energy" (Qiao and Tian,1998). It is one of the judging index of the rock bursting tendency of the rock based on laboratory testing. Based on the geomechanical properties of quartzite determined from the rock mechanic test in the laboratory, rock bursting potential of the rock was analyzed using linear elastic energy approach as shown in Table 2. The linear elastic energy, W_e calculated from the enveloped area of the three stress-strain curves in Fig. 3 varies from 54.76 kJ/m^3 to 72.70 kJ/m^3.Thus according to the intensity scale proposed by Qiao and Tian (1998), the rock burst tendency of the quartzite of the study area is medium. Based on the experience of rock bursting in a deep ore mine comprising of quartzitic sandstone and tuffaceous rock, Ma et al. (2018) suggested that there may be spalling type of rockburst when the elastic strain energy is less than 200 kJ/m^3. The area from where the quartzite samples were collected in HRT Face-3 and Face-4 encountered spalling and rock bursting of medium intensity in the crown and sidewall portion of the tunnel as shown in Fig. 1. Thus the severity prediction of the rock bursting tendency based on laboratory geomechanical properties of quartzite, validate the in-situ rock bursting phenomena observed in the tunnel.

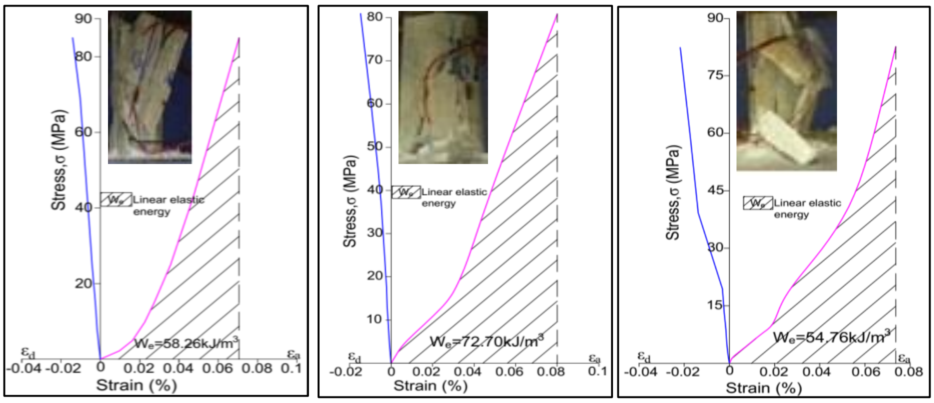

Fig. 3. Stress-strain curve of quartzite based on UCS testing in the laboratory; Inset Failure pattern

Table 2. Judging result of rockburst tendency based on Linear Elastic energy

Rock	σ_c (MPa)	E_s (GPa)	W_e (kJ/m^3)	Rock Burst Tendency
Quartzite	80-85	60-79	54.76-72.70	Medium

Severity prediction of rockburst based upon in-situ stresses. The depth of overburden cover above a tunnel section and the orientation and magnitudes of the in-situ principal stresses are mainly responsible for spalling and rock bursting. Orientation of principal stresses at a low angle to the major discontinuities of the rock encountered in a tunnel will lead to its kinematic instability (Laubscher et al, 2000) whereas orientation of principal stresses at a high angle to the tunnel direction leads to stress induced instability.

From the World Stress Map (2016) prepared based on the seismicity records, it was found that principal horizontal stress, σ_H is aligned at N065°E near the study area. Between HRT Face-3 and Face-4, the approximate azimuth of the tunnel is N15ºE-S15ºW. This means that the principal horizontal stress (σ_H) is oriented at approximately 50º to HRT direction between Face-3 and Face-4. The major discontinuity sets (045-060°/55-65°) in quartzite in HRT Face-3 & 4 are oriented almost parallel to the direction of principal horizontal stress,σ_H (N065ºE) (Fig. 4).

The above two factors signify that the section of the HRT between Face-3 and Face-4 are subjected to maximum stress concentration which may promote spalling/rock bursting during excavation.

Modified Overload Factor for Rock Bursting. Based on the depth of overburden cover and magnitude of principal stresses, Deere et al. (1969) developed a factor called the modified overload factor (OFM), which can be used to evaluate proneness of the rock to bursting under overstressed conditions:

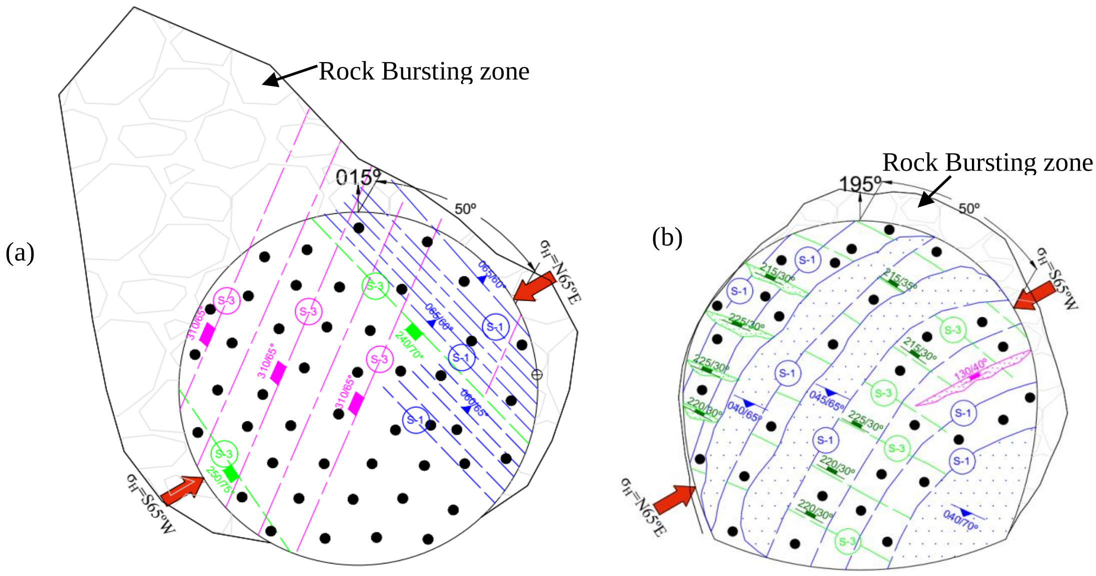

Fig. 4. Geological Face log of (a) HRT F-3 (TBM) and (b) HRT F-4(DBM) showing angular relationship between S-1 and horizontal tectonic stress σ_H for stress adjustment

$$OFM = \sigma_\theta / UCS \quad (1)$$

where σ_θ is the tangential stress, UCS is uniaxial compressive strength of the rock. Overstressed rock conditions develop around a tunnel when OFM is greater than one.

The tangential stress (σ_θ) around a circular tunnel can be estimated using following equations based on the depth of the overburden and in-situ principal horizontal and vertical stresses, σ_H and σ_v respectively (after Deere et al, 1969):

$$\sigma_\theta = (3K_0 - 1).P_z, \text{ at the crown and invert} \quad (2)$$
$$\sigma_\theta = (3 - K_0).P_z, \text{ at the springline} \quad (3)$$

where field stress, K_0, is the ratio of insitu principal horizontal to vertical stresses ($K_0 = \sigma_H/\sigma_v$) and P_z or σ_v is the vertical overburden stress at a depth Z above the tunnel section. As per Hoek & Brown (1980) relation, $P_z = \gamma.Z$ where γ is the unit weight of the overlying rock and Z is the depth of the overburden cover above the tunnel section.

Principal horizontal stress σ_H was calculated from Hoek and Brown (1980) empirical relationship for depth <2000m:

$$\sigma_H \sim 40 + 0.5\sigma_V \quad (4)$$

The magnitude of in-situ principal stresses σ_V and σ_H are calculated to determine Modified Overload Factor (OFM) for severity prediction of the rock bursting zones in HRT Face-3 and Face-4 as shown in Table 3.

Table 3 indicates that when K_0 is greater than one, the maximum tangential stresses are at the crown and invert of the tunnel developing more overstressed conditions in crown compared to springline.

Table 3. Severity prediction based on OFM in rock bursting areas of HRT Face-3 and Face-4

HRT RD (m)	Z (M)	$P_Z=\sigma_V$ $\gamma.Z$ (MPa)	σ_H (from eq.4) (MPa)	K_0 σ_H/σ_v	σ_θ (MPa) at crown & invert	Av. UCS (MPa)	OFM σ_θ/UCS	Severity Prediction
1585-3715 (Face-3)	990-1628	27-44	53-62	1.4-2	(3-5P_Z) 133-142	120	1.11-1.18	Overstressed conditions as OFM >1
3642-5184 (Face-4)	911-1367	25-37	52-58	1.6-2	(4-5P_Z) 132-138	80	1.65-1.72	

From the above analysis, it can be inferred that the tangential stresses that develop around the tunnel excavation play a significant role in initiating failure due to rock bursting. The rock around the tunnel is susceptable to failure when the tangential stresses induced by the excavation exceeds the strength of the rock mass. Since the magnitude of the tangential stress around the tunnel depends on the depth of overburden cover, a relationship between the tangential stress, σ_θ and depth of overburden, Z has been developed in the rock bursting zones in the tunnel section between HRT sFace-3 and Face-4.

$$\sigma_\theta = A * Z + B \; ; R^2 = 1 \text{ for } 1000<Z<2000 \text{ at crown and invert level} \quad (5)$$

$$\sigma_\theta = A * Z - B \; ; R^2 = 1 \text{ for } 1000<Z<2000 \text{ at spring level} \quad (6)$$

Fig. 5 and eqn. 5 and 6 indicate that the gradient of tangential stress increases with depth of overburden. Thus in a steeply inclined mountain, the principal horizontal stress increases with depth towards hill side compared to valley side where the principal horizontal stress becomes more relaxed. As determination of field stresses through in-situ rock mechanic tests is a time taking and cumbersome process, calculation of tangential stress from the empirical equation and assessment of its behaviour with the depth of overburden in the already encountered rock bursting zones of a tunnel shall further assist to identify the area of maximum stress concentration in similar geo-environment and adopt suitable remedial/preventive measure to mitigate rock bursting.

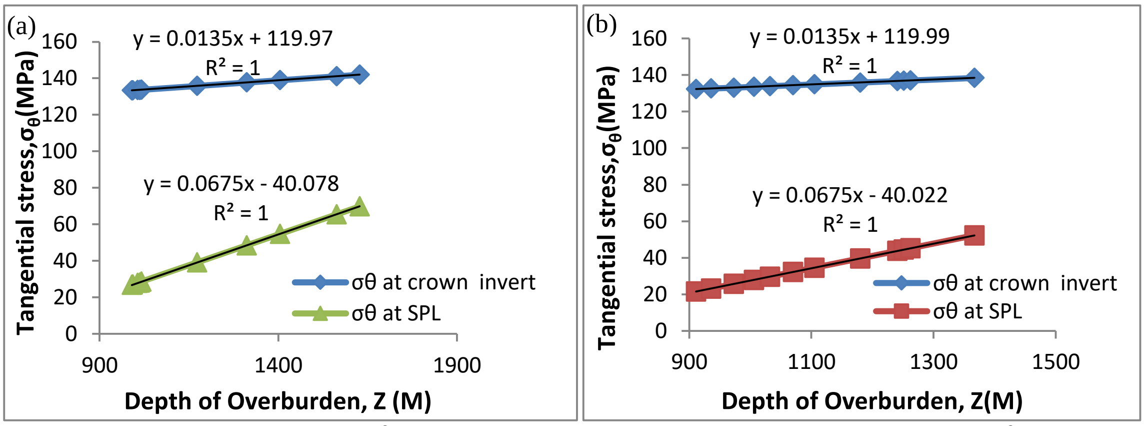

Fig. 5. Relation between the depth of overburden and tangential stress in the encountered zones of rock bursting conditions in HRT (a) Face-3 and (b) Face-4

Fig. 6 shows the tunnel section between Face-3 and Face-4 with calculated OFM values in the zones that have witnessed rock bursting conditions.

Empirical stability classification based on insitu vertical stress/intact rock strength ratio. In brittle rock masses, failure around tunnels occurs in the form of spalling or fracturing, and back-analysis involve establishing the stresses required to cause this fracturing. Ortlepp et al. (1972) compiled experience from tunnelling in brittle rocks in South African gold mines and suggested that the stability of these tunnels could be assessed using the ratio of the in-situ vertical stress, σ_V to the laboratory uniaxial compressive strength, UCS. The depth of the overburden cover and magnitude of in-situ vertical stresses are calculated for severity prediction in the rock bursting sections of HRT Face-3 and Face-4 based on Ortlepp et al. (1972) relation and is shown in Fig. 7. Hoek and Brown (1980) compiled additional South African observations from underground mining in massive brittle rocks and suggested the empirical stability classification as given in Fig. 8. Severity prediction pof rock bursting zones in HRT Face-3 & Face-4 based on Hoek's Empirical Stability classification is shown in Table 4.

Table 4. Severity prediction based on Empirical stability classification

HRT/ RD (m)	Z (M)	UCS (MPa)	σ_V (MPa)	σ_V/UCS (MPa)	Severity Prediction
Face-3 1585-3715	990-1628	120	27-44	0.22-0.36	Sidewall Spalling
Face-4 3642-5184	911-1367	80	25-37	0.31-0.46	

Calculated σ_V/UCS values along the tunnel section between Face-3 and Face-4 is plotted in Fig. 8 for severity prediction based on Hoek and Brown (1980) model.

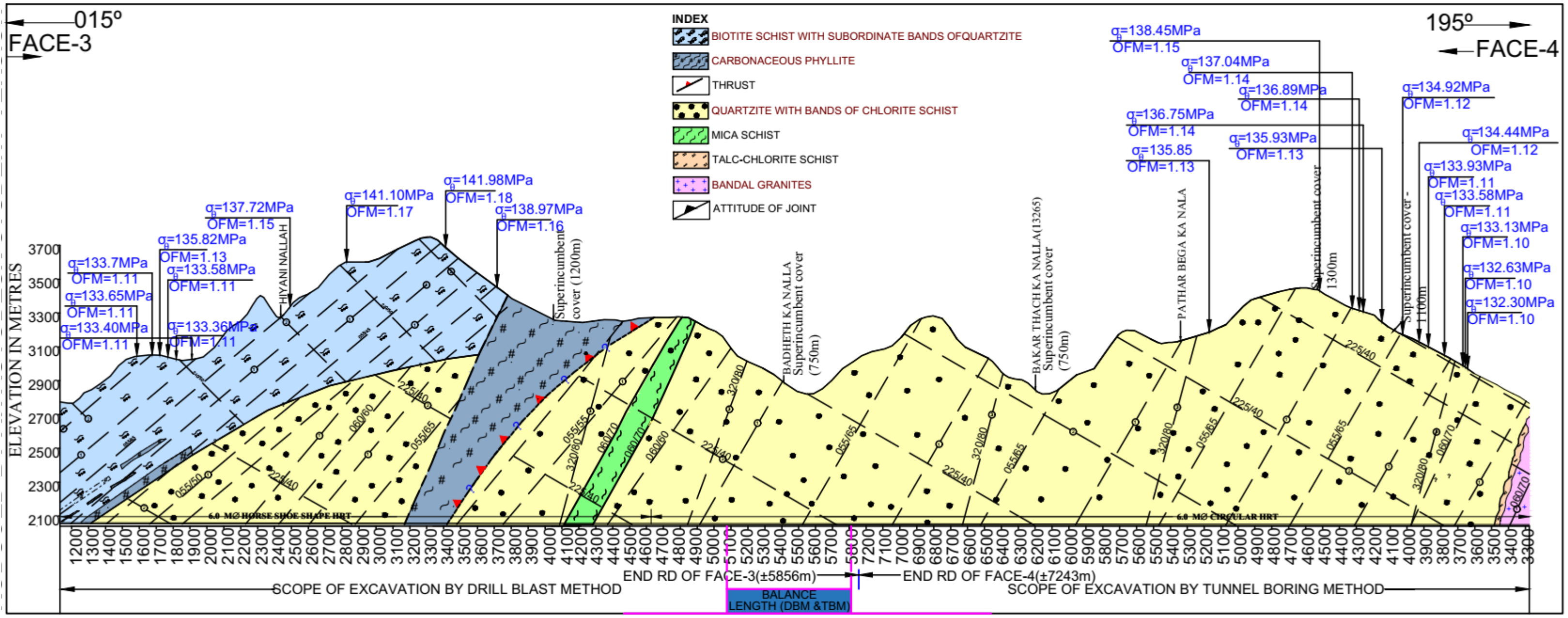

Fig. 6. Prediction analysis based on OFM between HRT Face-3 and Face-4 in the zones of rock burst

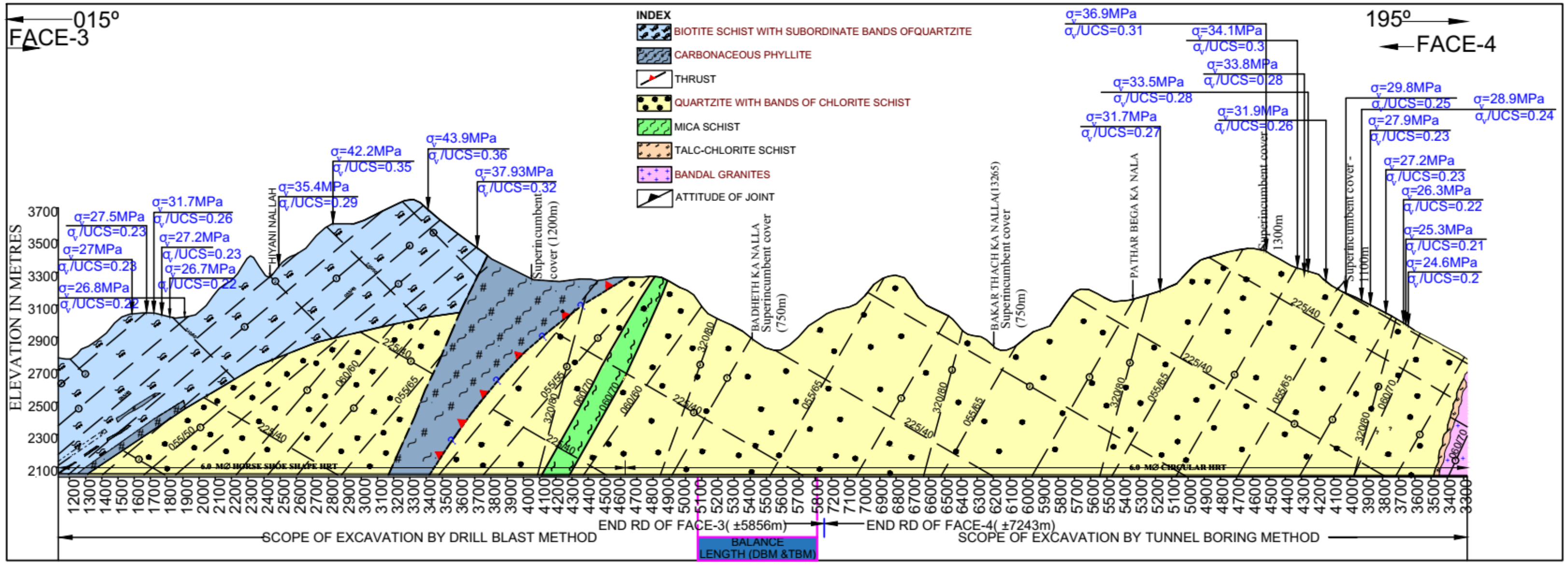

Fig. 7. Prediction analysis based on empirical stability classification between HRT Face-3 and Face-4 in the zones of rock burst

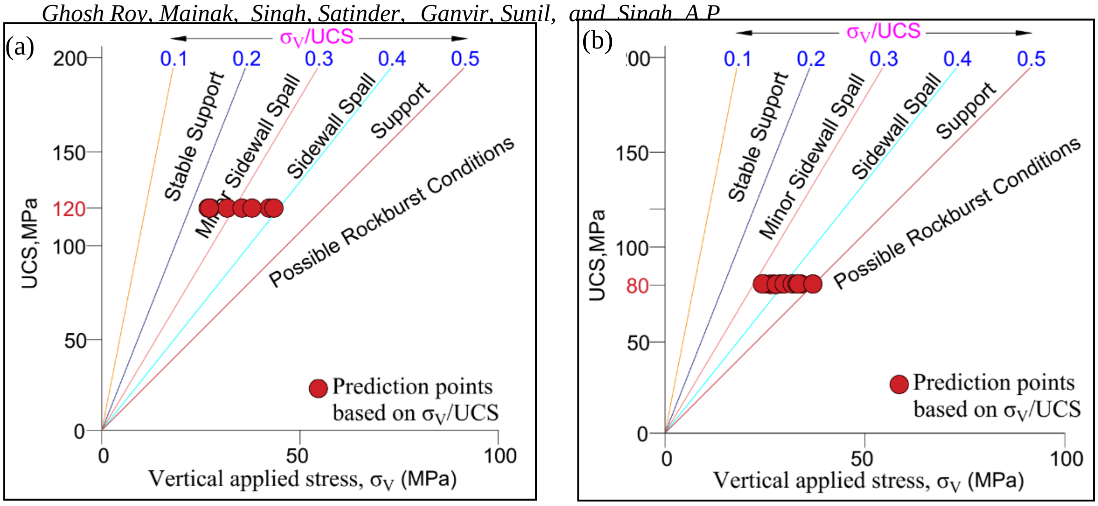

Fig. 8. Severity prediction based on empirical stability classification (Hoek and Brown, 1980) in (a) HRT Face-3 and (b) HRT Face-4

The severity prediction based on the empirical stability classification of the zones in HRT Face-3 and Face-4 under the influence of high principal vertical stress (σ_V) as shown in Fig. 8 indicates that rock bursting of different intensities shall mostly be limited to minor sidewall to sidewall spall with occassional heavy support requirement.

3.0 Case study-2: HRT in Pir Panjal range, Himalaya

The study area selected is a 23.2km long HRT passing through Pir Panjal range of Himalaya located in north-western part of India. Excavation of this HRT has been completed successfully. Out of total length of 23.2km, excavation of 14.75km was carried out through TBM and balance 8.5km was carried out through DBM. Six major lithological units encountered along HRT were Panjal volcanics, metasiltstone, Graphitic schist, Phyllitic quartzite, Quartzitic phyllite and Granodiorite. Weak metasiltstone/graphitic schist belonging to Razdhan, Hasthoji and Hafkhalan formation (Fig.9) with the potential of squeezing was the main focus of the study area.

Based on the laboratory measured geomechanical properties of Graphitic Schist (Table 5), range of in-situ stresses responsible for squeezing were calculated. Bending of ribs due to squeezing is shown in Fig. 10.

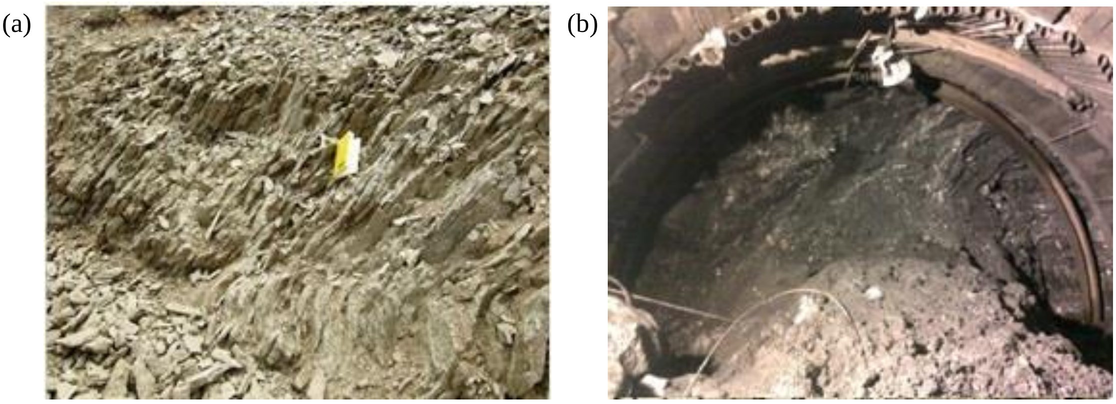

Fig. 9. (a) Surface outcrop of metasiltstone and (b) graphitic schist rock in HRT Face-2 (DBM face)

3.1 Severity prediction of squeezing

Modified Overload Factor proposed by Deere et al. (1980) was used for evaluating potential squeezing condition in Graphitic Schist rock. Superincumbant cover above the tunnel in Graphitic schist varies from 350 to 400m.

Principal horizontal stress σ_H was calculated from Stephansson (1993) empirical relationship for depth <1000m:

$$\sigma_H \sim 2.8 + 1.48\sigma_V \quad (4)$$

Table 5. Geomechanical properties of Graphitic Schist

Sl No.	Properties	Location of Samples: HRT Face-5	HRT Face-2
1.	UCS (σ_{ci}), MPa (Graphitic Schist/Phyllite)	15-50 (Av.-25)	
2.	Specific Weight (γ) (MN/m^3)	0.025	
3.	Modulus of Elasticity (E_s), GPa	15	
4.	Mode of failure	Ductile	
5.	Poisson's ratio	0.1	
6.	Joint Volume (m^3)/ RQD	25-28/<25%	
7.	RMR/Q	RMR-19-37 (Very Poor to Poor)	Q-0.016-0.025 (Extremely Poor)

Fig.10. Bending of ribs due to squeezing in HRT Face-2

The magnitude of in-situ principal stresses σ_V and σ_H are calculated to determine Modified Overload Factor (OFM) for severity prediction of the squeezing zones in HRT Face-5 and Face-2 as shown in Table 6.

Table 6. Severity prediction based on OFM in rock bursting areas of HRT Face-5 and Face-2

HRT RD (m)	Z(M)	$\sigma_V = P_Z$ (MPa)	σ_H (MPa)	$K_0 = \sigma_H/\sigma_v$	Av. UCS (MPa)	σ_θ (Av.) at crown & invert (MPa)	OFM σ_θ/UCS	Remarks
Face-5 (TBM)	334-457	9-12 (G.Sc.)	16-21(G.Sc.)	1.6-1.8	25	(2.7-2.8) P_Z	1-1.3	Overstressed condition in Graphitic schist(G.Sc.) OFM>1; Normal in metasediments (M.S.) &Volcanics (P.V.)
Face-2 (DBM)	345-628	9(G.Sc) 9.5(M.S.) 17.6(P.V.)	16(G.Sc) 17(M.S.) 29(P.V.)	1.6-1.8 1.8 1.65	25(G.Sc.) 50 (M.S.) 162(P.V.)	(2.6-2.8)P_Z 4.4P_Z 3.95P_Z	~1 (G.Sc.) 0.84 (M.S.) 0.43 (P.V.)	

From the above analysis, it can be inferred that the Graphitic Schist rockmass under high overburden cover in HRT Face-5 and Face-2 are overstressed and is conducive to squeezing. Fig. 11 and Fig. 12 show the tunnel section between Face-5 (TBM) and Face-2 (DBM) with calculated OFM values in the zones that have witnessed squeezing conditions.

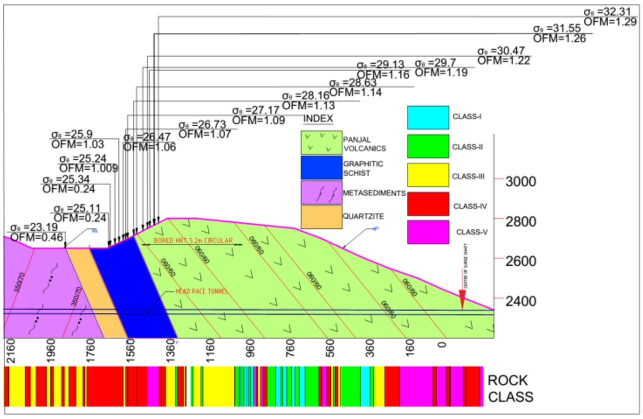

Fig. 11. Prediction analysis based on OFM within the zones of squeezing in HRT Face-5 (TBM side)

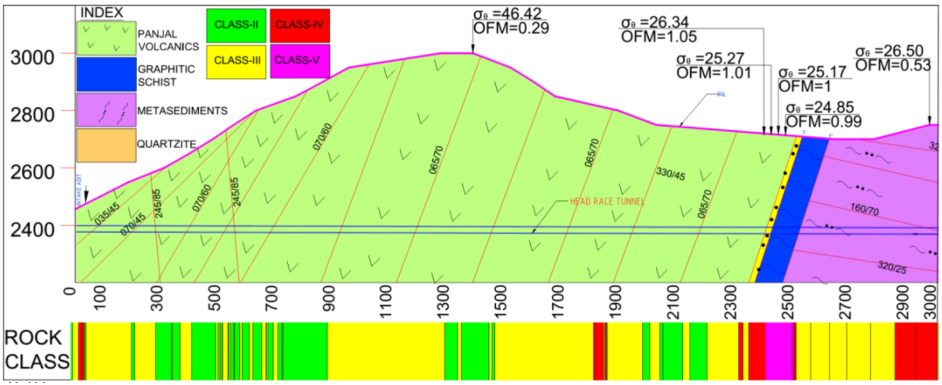

Fig. 12. Prediction analysis based on OFM within the zones of squeezing in HRT Face-2 (DBM side)

From comparison of OFM values in different rock types encountered along HRT Face-2(DBM) as shown in Fig. 13, it is found that OFM varies with strength of the rock e.g. for Graphic schist with UCS 25MPa, threshhold OFM (>1) is overcome at Z>300m; for metasediments with UCS 50MPa, threshhold OFM is overcome at Z>800m whereas in Quartzite and Panjal volcanics with UCS 106MPa and 160MPa respectively, threshold OFM is not crossed even at large depth >1400m.

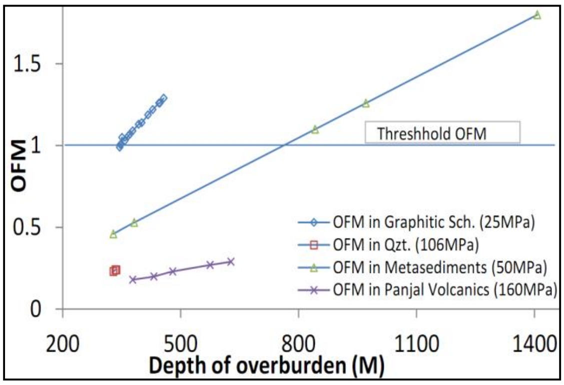

Fig. 13. Comparison of OFM value in different rock encountered along HRT in Kishanganga

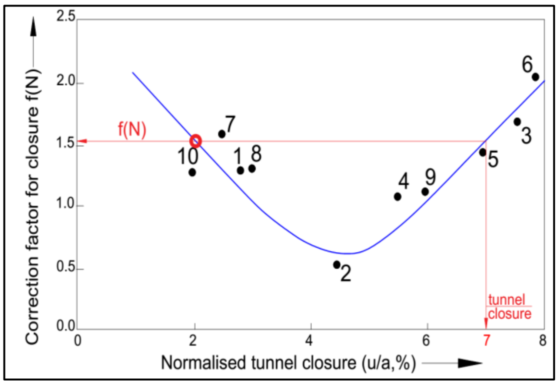

Fig.14. Estimated closure in squeezing ground condition based on support pressure curve, Goel (1994)

3.2 Estimation of Support Pressure

Goel (1994) estimated ultimate support pressure for squeezing ground condition with the following equation

$$P_{sqult}(N) = \left\{\frac{f(N)}{30}\right\} 10^{\left(\frac{H^{0.6}a^{0.1}}{50N^{0.33}}\right)} \tag{5}$$

$P_{sqult}(N)$ = Estimated ultimate support pressure in squeezing ground conditions in MPa using N
From fig.13, f(N)=1.5 taking tunnel closure as 7% (av.) and av. Radial deformation=180mm
H=average depth of overburden=400m, a=tunnel radius=260cm, N=Rock Mass Number =0.5(av.) in Graphitic schist
Thus estimated support pressure is calculated as $P_{sqult}(N)$= 1.97MPa and shown in Fig. 14.

4.0 Assessment of rock bursting and squeezing zones using empirical rockmass classifications

Due to the limitation of RMR classification of Bieniawski (1973) in predicting potential rock bursting and squeezing conditions in the tunnel due to absence of any stress parameters, stress adjustment factor has been applied to RMR to determine Modified RMR. Modified RMR classification system was initially introduced by Laubscher (1990) for caving operations which takes into account three adjustment factors-Blasting damage (A_B), Induced stress (A_S) and Fracture orientation (A_O). Each adjustment factor assigned is then multiplied with RMR value to get modified RMR. In case of tunnel, induced stress adjustment is calculated on the basis of orientation of principal horizontal stress w.r.t major joint set (S-1).

Fig.15(a) and (b) show the calculated MRMR in the rock bursting and squeezing zones of the two case studies discussed above based on the application of in-situ parameters A_B, A_S and A_O. A comparative analysis of RMR and MRMR as shown in Fig. 15 (a) and (b) indicate that MRMR accounts for a more conservative and practical rating approach while characterizing rockmass in overstressed region. During tunnelling, it is experienced that due to absence of any stress parameters, RMR often overestimates rockmass qualities in overstressed areas leading to wide disparities between class based support and hindrance based support. Modified RMR may reduce this disparity as adequate support within the stand-up time of the rock may successfully negotiate the stress induced instabilities within the rockmasses.

Following empirical relationship between RMR and Modified RMR (MRMR) as shown in Fig. 16 and eqn. 6 is proposed in stress induced conditions using statistical analysis of the two case studies:

$$RMR = (1.45 - 1.85)MRMR + (0 - 1), R^2 = 0.98 - 1 \tag{6}$$

5.0 Results and Discussions

Rock bursting and Squeezing are the most well-known stress induced deformations which often culminate into a potential hazard during underground excavation in the tectonically unstable mountain ranges of Himalaya. Implementation of in-situ stress components in the severity prediction of overstressed conditions and its incorporation in empirical rockmass classification are the basis for assessing the stability conditions of underground openings.

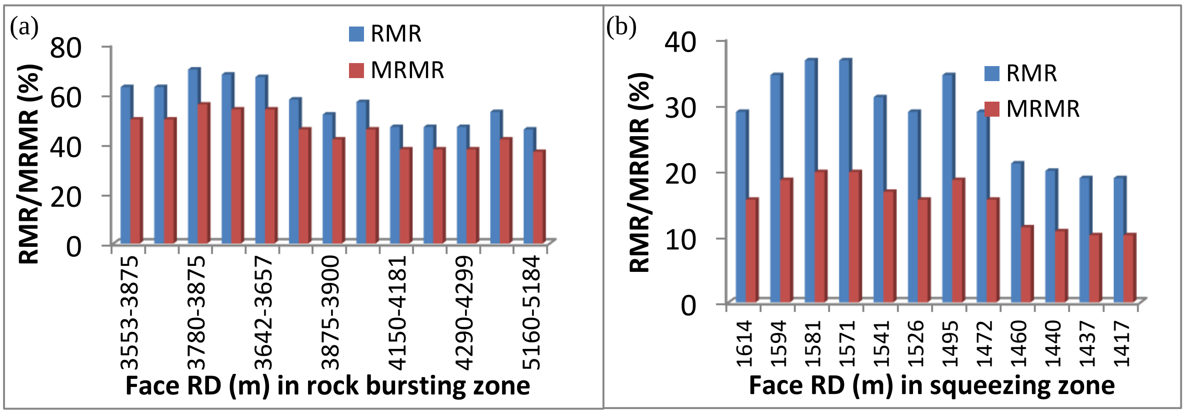

Fig. 15. Comparative analysis of RMR and MRMR in (a) rock bursting and (b) squeezing zones of HRT

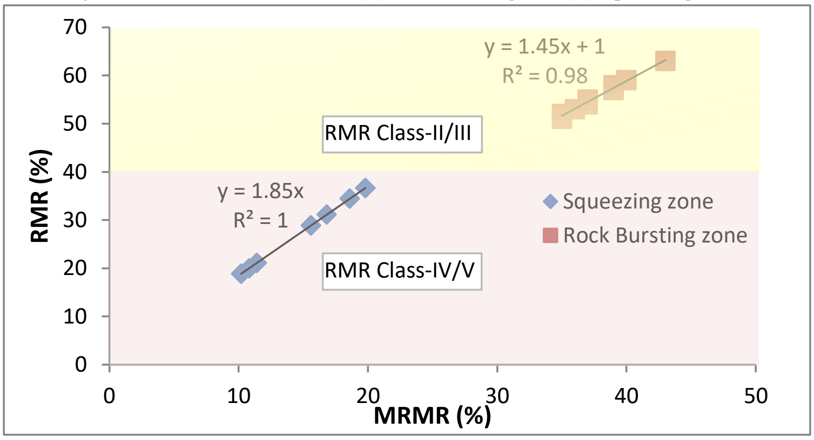

Fig. 16. Empirical relationship between RMR and MRMR in rock bursting and squeezing zones

In view of above, two of the mega hydroelectric projects situated in Dhauladhar and Pir Panjal ranges of Himalaya which have experienced few incidences of rock bursting and squeezing in the past as well as during present tunneling conditions, are chosen as the case studies.

Following results are drawn based on the severity prediction analysis carried out in the rock bursting zones of the study area as described in the preceding sections:

(i) Based on the geomechanical properties of quartzite determined from the laboratory testing, it is observed that amount of stored elastic energy released during failure under uniaxial compression ranges from 55-73 kJ/m^3 indicating proneness of the rock to medium scale bursting.

(ii) OFM analysis of the rock bursting and squeezing zones based on the insitu stresses indicates that OFM exceeds the threshold value of 1.0 in all these zones thus signifying overstressed conditions

(iii) Severity prediction based on empirical stability classification interpret that the intenstity of rock bursting in HRT Face-3 and Face-4 of the study area shall mostly be limited to minor sidewall to sidewall spall.

(iv) Based on Goel (1994) approach, support pressure in squeezing ground condition has been estimated to be 1.97MPa

(v) Looking to the limitations of RMR in stress-induced conditions, a modified RMR (MRMR) is proposed which incorporates the insitu stress conditions and adopt a more realistic characterization of rockmass. Under moderate rock bursting conditions, MRMR value may reduce upto 20% to 45% of the RMR value depending on other tunneling conditions.

(vi) Finally, a site specific severity prediction model based on the strength of the rock, in-situ stresses, depth of overburden and empirical rockmass classification has been proposed and shown in Table 7.

6.0 Conclusions

The investigated section of the tunnels in the study area experience several incidences of rock bursting and squeezing under a geo-environment which is conducive to the stress induced instabilities. Although many methods, such as case analysis, in situ stress measurements, rock mechanics tests, microseismic monitoring, numerical simulations, etc. have been applied to study stress induced deformations in different projects worldwide, however, fundamental mechanisms of rockbursts and squeezing are yet to be understood due to their complexity and hidden nature of the subsurface geological conditions.

As most of the tunneling projects in the Himalaya experience similar geo-enviroment, a need arises for utilizing the most accepted and easy to use theories for real-time prediction/ assessment of overstressed conditions during excavation of tunnel in a deep seated environment.

Based on the back analysis of the rock strength, depth of overburden and principal in-situ stress conditions of the already encountered rock bursting and squeezing zones, an attempt has been made in this paper to develop a simplified prediction model for rock bursting and squeezing under similar geological conditions. Moreover, looking to the limitation of RMR in stress-induced conditions, a modified RMR is proposed which incorporates the insitu stress conditions and adopt a more realistic characterization of rockmass. The outcome of the present geotechnical investigations may be supportive in preparing a comprehensive geotechnical baseline report incorporating the support estimation for stabilizing the burst prone area during the planning and execution stages of the project.

Successful implementation of suggested prediction models may also help in creating an early warning system and adopt suitable preventive measures to reduce the intensity of rock bursting and squeezing. The authors are hopeful that the public and private executing agencies of the nation involved in various road, hydropower and other infrastructure projects shall be benefited from the outcome.

Table 7. Site specific **s**everity prediction model

Prediction Principles	**Kinematic properties**	**Strength of intact rock (UCS)**	**Mode of failure**	**Stress Induced phenomenon**
Geomechanical properties based on laboratory test	Massive to moderately jointed	>80MPa	Elasto-brittle with violent rupture	Spalling/Rockburst
	Closely jointed	25-40MPa	Elasto-plastic with splitting along joint plane	Squeezing
	Massive	< 25MPa	Elasto-visco plastic with shear failure	Creep
Prediction Principles	Formulation	Empirical Value	Rock Burst/Squeezing Intensity	Comments
Field Stress, K_0	$K_0 = \frac{\sigma_H}{\sigma_V}$	<1 1-2 >2	No rockburst/squeezing Sidewall spalling / convergence Rockburst in crown	Based upon the principal horizontal stress (σ_H) and vertical stress (σ_v)
Modified Overload Factor, OFM	$OFM = \frac{\sigma_\theta}{UCS}$	<1 1-2 >2	No rockburst /squeezing Sidewall spalling/ convergence Rockburst/Squeezing in crown	Based upon tangential stress (σ_θ) and strength (UCS) of the rock
Virgin stress analysis, VSA	$VSA = \frac{\sigma_v}{UCS}$	<0.2 0.2-0.4 >0.4	No rockburst Sidewall spalling Rockburst in crown	Based upon vertical stress (σ_v) and the strength (UCS) of the rock
Empirical rockmass classification, ERC	$ERC = \frac{MRMR}{RMR}$	~1 0.6-1 <0.6	No rockburst/squeezing Minor rock burst/ squeezing Major rock burst/squeezing	Based upon Modified Rock Mass Rating (MRMR) and Rock Mass Rating (RMR) of the rock

7.0 Limitations of the study and scope for future work

More incidences of rock bursting and squeezing in tunnels in different rock formations should be studied to establish approximate range of tectonic stress components capable of causing stress induced deformation in the tectonically active region like Himalaya. Instability of the ground above the tunnel crown due to overstressing when field stress, K_0 is greater than one is a serious concern in terms of ground support and safety.Therefore, it is critical to accurately determine K_0 in order to assess the behaviour of both strong and weak rocks in a tunnel with confidence. In such areas, stress measurements through hydrofracturing, dilatometer or pressuremeter should be carried out to determine magnitude and orientation of principal stress components. The actual field stresses determined can be used for any further analysis. A need for modification of existing and popular empirical rockmass classification like RMR by incorporating a stress parameter arise to make it a more prudent tool for site specific characterization of rockmass in stress induced conditions. Under this context, the proposed site specific severity prediction model attempted in Table 7 can only be used as an impetus for future study.

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