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Critical strain and squeezing of rock mass in tunnels

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Abstract

The squeezing of tunnels is a common phenomenon in poor rock masses under high in situ stress conditions. The critical strain parameter is an indicator that allows the degree of squeezing potential to be quantified. It is defined as the strain level on the tunnel periphery beyond which instability and squeezing problems are likely to occur. Presently, in the literature, the value of critical strain is generally taken as 1%. It is shown in this study that the critical strain is an anisotropic property and that it depends on the properties of the intact rock and the joints in the rock mass. A correlation of critical strain with the uniaxial compressive strength, tangent modulus of intact rock and the field modulus of the jointed mass is suggested in this paper. It is also suggested that the modulus of deformation being anisotropic in nature should be obtained from field tests. In absence of field tests, use of a classification approach is recommended, and, expressions are suggested for critical strain in terms of rock mass quality Q. A rational classification based on squeezing index (SI) is proposed to identify and quantify the squeezing potential in tunnels. Applicability of the approach is demonstrated through application to 30 case histories from the field.

Keywords: Critical strain; Squeezing; Tunnel deformation; Rock mass classification; Rock mass strength; Stress–strain response

1. Introduction

Squeezing is an unique problem faced by rock engineers while excavating tunnels through rock masses of very poor quality under high rock cover. High deformability, low shear strength and the high in situ stress state are the major factors that govern the tunnel wall stability and extent of closure. Prediction of squeezing conditions is of great importance to a designer for designing a stable support system of the tunnel.

It is suggested in this study that there is a threshold value of tangential strain at the tunnel periphery above which instability and support problems are likely to occur. This threshold value of strain is termed as the critical strain. It is also suggested that the critical strain may be obtained from the properties of the intact rock and the jointed rock mass. If the observed strain exceeds this value, squeezing is likely to occur. This work is based on Aydan et al. (1993), which is applied to an experimental study conducted on specimens of jointed rock mass by Singh (1997). A relationship is suggested for computing the critical strain. For a reasonable prediction of the critical strain, the modulus of deformation of the rock mass will be required. In absence of field tests, the critical strain is linked with Barton's rock mass quality Q. The approach is demonstrated by applying it to several case studies on squeezing and non-squeezing tunnels from the literature.

2. Squeezing and critical strain

Squeezing stands for large time dependent convergence during the tunnel excavation (Barla, 2001). Though the fundamental mechanism of squeezing is yet to be fully understood, it is well known, that the excavation of tunnel redistributes the stresses in the tectonically stressed rock mass. The tangential stresses around the tunnel periphery become large and exceed the uniaxial compressive strength of the rock mass in the tangential direction at that point. The rock mass at the periphery therefore fails and the

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broken zone progresses slowly in the radial direction giving rise to time-dependent-large-tunnel convergence. Many attempts to 'quantify' squeezing potential are based on concept of comparing the rock mass strength with the induced stress in one way or the other (Singh et al., 1992; Jethwa et al., 1984; Hoek and Marinos, 2000).

A slightly modified approach is suggested by Aydan et al. (1993), which compares the strains to define squeezing potential. The approach is based on the analogy between the stress–strain response of rock in laboratory and the tangential stress–strain response around tunnels. Five distinct states of stress–strain response were expressed during loading of a specimen at low confining stress σ_3 (Aydan et al., 1993). Based on strain, the normalized strain level η_p was defined as

$$\eta_p = \frac{\varepsilon_p}{\varepsilon_e} \tag{1}$$

where ε_e is the elastic strain limit and ε_p is the strain levels at the peak of stress–strain curve.

If ε_θ^a $(= u_a/a)$ is the peak tangential strain at the periphery of the tunnel and ε_θ^e is elastic strain, the ratio $\varepsilon_\theta^a/\varepsilon_\theta^e$ may be used to define various degrees of squeezing as suggested by Aydan et al. (1993). Hoek (2001) also used tunnel strain to define squeezing potential. A comparison of squeezing conditions suggested by Hoek (2001) and Aydan et al. (1993) is presented in Table 1 (Barla, 2001). The approach is also used by Sakurai (1997) where critical strain is used to define various warning levels for severity of construction in a tunnel.

The approach of comparing the strain and not the strength is advantageous in that the deformations are easy to measure and the field engineer will appreciate it more than the approach based on strength computation. Also, the damage to the rock mass depends directly upon strain. Moreover, if permissible limit of deformations based on critical strain is already available, one can modify the support system from the observation in the field as the project progresses.

It is observed from Table 1, that the critical strain, above which the construction problems due to squeezing are likely to occur, is taken as 1%. Hoek (2001), however, has mentioned that there are some tunnels which suffered strains as high as 4% but did not exhibit stability problems. It is suggested in this paper that the critical strain should not be taken as 1%, rather it should depend on the properties of the intact rock material and jointed rock mass. The critical strain in this paper is defined as an empirical level of tangential strain at the periphery of the opening above which the jointed rock mass fails under uniaxial loading condition. It should be noted that the critical strain is an anisotropic property and will be different at different points on the periphery of the opening. Expressions have been suggested for critical strain. The strain actually occurring at the periphery of the opening may be obtained through numerical modelling or through monitoring and analysis of the field data. The ratio of observed strain to the critical strain may then be used to quantify the squeezing potential and modify support systems accordingly. Sakurai (1997) observed that construction problems occurred in non-squeezing ground conditions, where observed tangential strain exceeded far above the predicted critical strain (i.e. ratio between UCS and modulus of elasticity of that rock material). The degree of severity of construction problems will therefore increase in proportion of the ratio between the actual strain and the critical strain.

3. Experimental programme and expression for critical strain

One of the best ways to understand the mechanical response of jointed rock masses is to conduct physical model studies. The rock mass samples of a scale suitable for testing in the laboratory are difficult to retrieve from the field in a relatively undisturbed state. Thus for physical modelling of jointed rock masses, fabrication of blocky systems from available materials is preferred. An extensive experimental study was carried out by testing more than 80 specimens to study the strength and deformation behaviour of jointed rock mass under uniaxial stress state. The complete details of the study are available else where (Singh, 1997; Singh et al., 2002; Singh and Rao, 2005). Cut blocks of lime silica bricks were used in the study to simulate a weak rock. The properties of the model material and the joints are indicated in Table 2. The specimens of jointed mass were prepared by arranging elemental blocks in a particular fashion. The size of the specimen was $15 \times 15 \times 15$ cm and, on an average, more than 260 blocks were used to form a specimen. The various configurations of joints adopted in the experimental study are shown in Fig. 1. The majority of the specimens belonged to type-A (Fig. 1). These specimens consisted of three sets of joints. Joint Set-I was continuous and was inclined at variable

Table 1
Comparison of approaches (Barla, 2001)

Class number	Hoek (2001)		Aydan et al. (1993)[a]	
	Squeezing level	Tunnel strain ε_t	Squeezing level	Tunnel strain
1	Few support problems	$\varepsilon_t < 1\%$	No squeezing	$\varepsilon_\theta^a/\varepsilon_\theta^e \leqslant 1$
2	Minor squeezing problems	$1\% < \varepsilon_t < 2.5\%$	Light squeezing	$1 < \varepsilon_\theta^a/\varepsilon_\theta^e \leqslant 2.0$
3	Severe squeezing problem	$2.5\% < \varepsilon_t < 5\%$	Fair squeezing	$2.0 < \varepsilon_\theta^a/\varepsilon_\theta^e \leqslant 3.0$
4	Very severe squeezing problem	$5\% < \varepsilon_t < 10\%$	Heavy squeezing	$3.0 < \varepsilon_\theta^a/\varepsilon_\theta^e \leqslant 5.0$
5	Extreme squeezing problem	$\varepsilon_t > 10\%$	Very heavy squeezing	$\varepsilon_\theta^a/\varepsilon_\theta^e > 5.0$

[a] UCS of rock mass was taken as 1 MPa.

Table 2
Properties of the model material and joints

Property	Value
Dry density (kN/m^3)	16.86
Porosity (%)	36.94
UCS σ_{ci} (MPa)	17.13
Brazilian strength σ_{ti} (MPa)	2.49
Tangent modulus E_i (GPa)	5.34
Poisson's ratio v	0.19
Cohesion c_i (MPa)	4.67
Friction angle of intact material ϕ_i	33°
Friction angle along the joints ϕ_j	37°
Deere and Miller (1966) classification of the material	EM
Normal stiffness of joints k_n (MPa/m) (σ_n = normal stress on joints in MPa)	$11190\ (\sigma_n)^{0.627}$
Shear stiffness of joints k_s (MPa/m)	588.6 MPa/m

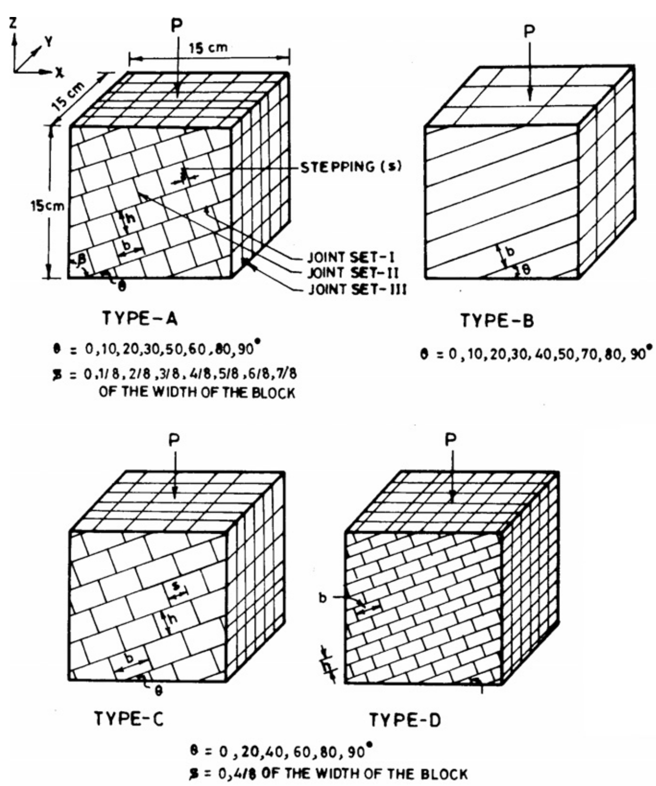

Fig. 1. Configuration of jointed block system (Singh, 1997).

angle θ with the horizontal. The joint Set-II was constructed with stepping 's' for each θ; and Set-III was always kept vertical. The stepping 's' was varied with each θ as shown in Fig. 1. Besides type-A, additional tests were performed on types B, C and D specimens by changing the geometry of the cut blocks.

The tests were performed under uniaxial loading condition by applying uniformly distributed load at the top of the specimen. Two teflon sheets smeared with silicon grease were used on the top and the bottom of the specimen to reduce end friction. A strain controlled machine was used to apply load at an uniform strain rate. During testing, the deformations were continued beyond the failure of the specimen until the load reduced to about half the peak load. Deformations of the specimen in X, Y and Z directions were measured during loading. The axial stress at any given instant was computed by applying area correction (see Singh et al., 2002).

After the test was over, the failure mode was identified based on visual observation. Four distinct failure modes were identified as: (i) splitting of intact material, (ii) shearing of intact material, (iii) rotation of blocks and (iv) sliding along the critical joints. The failure mode was found to be dependent of the joint configuration. It may be noted that almost all types of failure modes could be achieved through different configuration of joints.

Axial stress vs axial strain and axial stress vs transverse strain curves were plotted for all the specimens. The transverse strain in Y direction for most of the cases was found to be negligibly small compared to the transverse strain in X direction. The mass may, therefore, be assumed to be acting under nearly plane strain condition. The peak stress was taken as the rock mass strength σ_{cj}, and the tangent modulus E_j was obtained by measuring gradient of tangent drawn to the axial stress–strain curve at a stress level equal to half the rock mass strength σ_{cj}. It was possible to record strain levels ε_e and ε_p for all cases. Most of the stress–strain curves obtained were 'S' shaped (Fig. 2). The curves showed an initial concave upward portion due to the joint closures and the initial seating; thereafter a linear middle portion exhibiting elastic deformations, and, a convex upward portion due to plastic deformations near failure. The axial strains were corrected for initial seating effect by drawing a tangent at the middle straight line portion of the stress–strain curve. The point of intersection of this tangent with the strain axis was considered as the point of zero strain. The strain levels, i.e. ε_e and ε_p are explained in Fig. 2.

The Modulus ratio (M_{rj}), as defined by Deere and Miller (1966), for each specimen was obtained as follows:

$$M_{rj} = \frac{E_{tj}}{\sigma_{cj}} \tag{2}$$

where E_{tj} is the tangent modulus and σ_{cj} is the UCS of the mass. The modulus ratio is a measure of inverse of failure strain. A plot of results between ε_p and M_{rj} (Fig. 3)

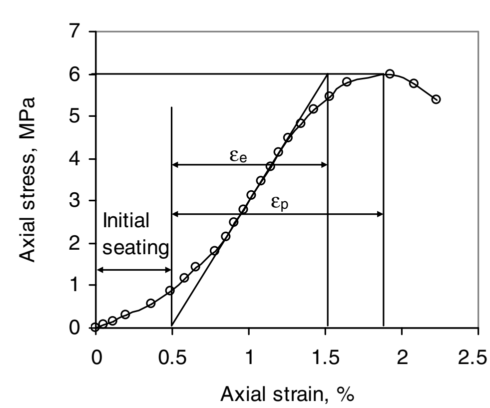

Fig. 2. Typical stress–strain curve and different strain levels.

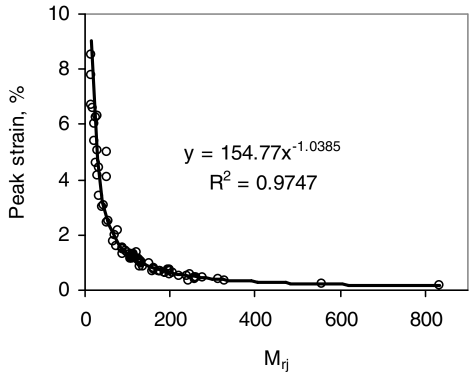

Fig. 3. Correlation of peak strain with modulus ratio.

indicates a strong correlation between them. The following correlation is found for the peak failure strain ε_p.

$$\varepsilon_p = 154.77(M_{rj})^{-1.04} \tag{3}$$

where ε_p is peak failure strain in percent.

The critical strain as defined by Sakurai (1997) represents the elastic strain in Fig. 2, and may be obtained as

$$\varepsilon_{cr} = \varepsilon_e = \frac{\sigma_{cj}}{E_{tj}} \tag{4}$$

The results from tests on tangent modulus E_j are plotted against the strength σ_{cj} as shown in Fig. 4. The intact rock position is also indicated on this plot. It is interesting to see that, the E_j vs σ_{cj} plot is scattered about a line passing through the intact rock position I. The gradient of this empirical line is found to be 1.6. A correlation between the strength and tangent modulus of intact and jointed rock mass may be obtained as given below (Singh and Rao, 2005):

$$\text{Gradient of the line} = \frac{\log E_i - \log E_{tj}}{\log \sigma_{ci} - \log \sigma_{cj}} = 1.6$$

$$\Rightarrow \frac{\sigma_{cj}}{\sigma_{ci}} = \left(\frac{E_{tj}}{E_i}\right)^{0.63} \tag{5}$$

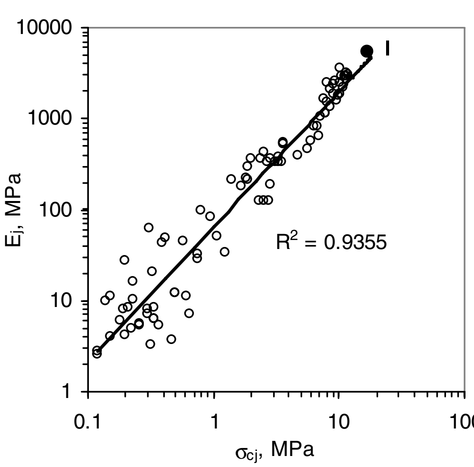

Fig. 4. Relationship between intact and rock mass properties (Redrawn from Singh and Rao, 2005).

where E_i and σ_{ci} are tangent modulus and uniaxial compressive strength (UCS) of the intact rock, respectively.

Substituting σ_{cj} from Eq. (5) into Eq. (4), the critical strain may be obtained as

$$\varepsilon_{cr} = \frac{\sigma_{ci}}{E_{tj}^{0.37} E_i^{0.63}} \times 100 \text{ (percent)} \tag{6}$$

Also from Eqs. (2)–(4) the peak strain may be obtained as:

$$\varepsilon_p = 154.77\left(\frac{E_{tj}}{\sigma_{cj}}\right)^{-1.04} \approx 154.77(\varepsilon_{cr})^{1.04} \approx 1.5\varepsilon_{cr} \tag{7}$$

Eqs. (6) and (7) may be used to predict the critical strain (i.e. the strain level for which the stress will reach the elastic limit) and the peak failure strain ε_p, (i.e. the strain at which failure will occur as light squeezing).

It may be noted that E_{tj} is generally anisotropic in nature and, in the field, it should be obtained at the periphery of the tunnel in the desired direction. The value may be obtained from uniaxial jacking test results which are routinely performed at the project sites. For the crown, the test in horizontal direction and for side walls, the test in vertical direction, will give E_{tj} for Eq. (6). Actually there is some in situ stress along a tunnel axis, which will increase both σ_{cj} and E_{tj} but this effect is not considered in this simplified analysis.

4. Use of classification approach

Many times field tests may not be feasible especially during preliminary stages of site selection. Classification approaches serve as a powerful tool in the absence of the field tests. Field modulus E_{tj} may be obtained from a Joint Factor (Ramamurthy and Arora, 1994), RMR, GSI or Q system. Barton's Q system is the most extensively used and well tested classification for tunnels and underground structures especially for squeezing ground conditions. An approximate value of critical strain may be obtained from Q as given below.

4.1. Using Singh et al. (1997) correlation

Singh et al. (1997), based on the back analysis of several tunnels, have suggested the following relation for rock mass strength σ_{cj}.

$$\sigma_{cj} = 7\gamma Q^{1/3} \text{ MPa}, \quad \text{for } Q < 10,\ J_w = 1.0, \quad \sigma_{ci} < 100 \text{ MPa} \tag{8}$$

where, γ is the density of rock mass in gm/cc, σ_{cj} and σ_{ci} are UCS in MPa of the rock mass, Q is the actual (post-construction) rock mass quality and J_w is joint water reduction factor used in Q. The critical strain may, therefore, be obtained from the intact rock properties and the Q value as follows:

$$\frac{1}{\varepsilon_{cr}} = \frac{E_{tj}}{\sigma_{cj}} = \frac{E_i}{\sigma_{cj}}\left(\frac{\sigma_{cj}}{\sigma_{ci}}\right)^{1.6} = E_i(\sigma_{cj})^{0.6}(\sigma_{ci})^{-1.6}$$
$$= E_i(7\gamma Q^{1/3})^{0.6}(\sigma_{ci})^{-1.6}$$

$$\Rightarrow \varepsilon_{cr} = 31.1\frac{\sigma_{ci}^{1.6}}{E_i\gamma^{0.6}Q^{0.2}} \text{ (percent)} \tag{9}$$

4.2. Using Barton (2002) correlation

Barton (2002) has suggested the following correlation for the long term modulus of deformation in the field:

$$E_{tj} = 10\left[\frac{Q\sigma_{ci}}{100}\right]^{1/3} \times 10^3 \text{ MPa} \tag{10}$$

Substituting the above expression in Eqs. (4) and (5), the critical strain may be obtained as,

$$\varepsilon_{cr} = 5.84\frac{\sigma_{ci}^{0.88}}{Q^{0.12}E_i^{0.63}} \text{ (percent)} \tag{11}$$

The observed or expected strain may be obtained from numerical modelling or preferably from actual monitoring in the field. The squeezing index SI may be defined as:

$$\text{SI} = \frac{\text{Observed or expected strain}}{\text{Critical strain}} = \frac{u_a/a}{\varepsilon_{cr}} \tag{12}$$

where u_a is the radial closure and a is the radius of the opening.

A classification similar to Aydan et al. (1993) may now be adopted as proposed in Table 3.

5. Field application

The methodology suggested above is applied to some case studies available from the literature. The data required for computing critical strain was collected for several tunnels (Table 4). The critical strain was computed using Eqs. (9) and (11). The observed or expected strain values, if available, are also listed in the table. The expected tangential strain was assessed by the respective authors mainly through numerical modelling. The intact rock properties for the cases referred from Jethwa et al. (1984) and Singh et al. (1992) were obtained from Mehrotra (1992). Arithmetic average values of σ_{ci} and E_i were used for analysis, where as geometric average ($Q_{av} = \sqrt{Q_1Q_2}$) was used for Q values ranging between Q_1 and Q_2.

It is observed that for the majority of the cases of non-squeezing tunnels, the critical strain is below 1%. It is also observed that for all these cases the squeezing index is less than one, indicating no squeezing. In some cases the critical strain is higher than 1% and it may be expected that the tunnel is not likely to pose problems for even large deformations as far as squeezing is concerned. Therefore, permissible deformations may be computed for the tunnels based on the critical strain.

For many of the cases the critical strain is well below the expected or observed strain. This is an indicator of squeezing or probable problems to be encountered during construction. A warning is therefore given by the value of critical strain and suitable measures could be adopted. For example the case study of Kaletepe tunnel, Turkey (Sari and Pasamehmetoglu, 2004) indicates a squeezing condition without support. As rock bolts are provided, the expected strain reduces resulting also in a reduction of the squeezing index. However, for some cases, the squeezing conditions still prevails. A further improvement is achieved by providing rock bolts with shotcrete which reduces the squeezing index to less than one, which indicates no squeezing and a likely stable condition.

A quantitative evaluation of the performance of the support is, therefore, possible through critical strain. It is, therefore, suggested that the idea of critical strain and permissible deformation will serve as an additional tool for safety management especially for those sites where critical strain is less than 1%. It is also observed from Table 4 that the squeezing potential defined through Singh et al. (1997) and Barton (2002) matches with each other and also verify the observed ground condition in almost all cases barring few exceptions. The observed good consistency validates the applicability of the Eqs. (6), (9) and (11) in estimating the critical strain.

Table 3
Proposed classification for squeezing potential in tunnels

Class number	Squeezing Level	SI
1	No squeezing (NS)	SI < 1.0
2	Light squeezing (LS)	1.0 < SI ⩽ 2.0
3	Fair squeezing (FS)	2.0 < SI ⩽ 3.0
4	Heavy squeezing (HS)	3.0 < SI ⩽ 5.0
5	Very heavy squeezing (VHS)	5.0 < SI

6. Concluding remarks

Assessment of squeezing potential of tunnels in jointed rocks is an important aspect in the tunnel design and construction. The tunnel closures are easily measured and give better indication of likely damage to the rock mass and associated construction problems than the strength of the rock mass. If the permissible deformations are known before hand, measures like the design of steel fibre reinforced shotcrete (SFRS) may be taken up in advance to solve the problem. The critical strain is defined as the tangential strain level at a point on the opening periphery beyond which the squeezing problems may be encountered during construction. The value of critical strain, presently in vogue is 1%, which is based on past experience. It is suggested in this paper that the critical strain is an anisotropic property and depends on the properties of the intact rock and the orientation of discontinuities in the mass. The conclusion is based on the outcome of an experimental programme, in which, specimens of jointed rock mass were tested under uniaxial loading condition. A correlation has been derived for critical strain in terms of the intact rock properties and modulus of deformation in the field in the appropriate direction. Expressions are also suggested for computing critical strain if field modulus is not known and Q is known. However, the Eq. (6) is recommended for calculation of critical strain taking into

Table 4
Some case studies evaluated for squeezing potential

Sl	Rock type	Reference	Project	Q	γ (gm/cc)	Observed or computed strain (%)	σ_{ci} (MPa)	E_i (GPa)	Critical strain (ε_{cr}, %)		Squeezing index (SI)		Classification		Observed ground condition
									Singh et al. (1997)	Barton (2002)	Singh et al. (1997)	Barton (2002)	Singh et al. (1997)	Barton (2002)	
1	Moderately fractured quartzites	Jethwa et al. (1982)	Maneri-Bhali hydro project, India	3.6	2.5	0.06	67–128 Av = 97.5	28.25–49.8 Av = 39.03	0.54	0.36	0.11	0.17	NS	NS	No squeezing
2	Foliated metabasics	Jethwa et al. (1982)	Maneri-Bhali hydro project, India	3.4–6.8 Av = 4.81	2.5	0.05	70.9–104 Av = 87.45	21–22.4 Av = 21.70	0.77	0.46	0.06	0.11	NS	NS	No squeezing
3	Sheared metabasics	Jethwa et al. (1982)	Maneri-Bhali hydro project, India	0.3–3.3 Av = 0.99	2.5	0.4	70.9–104 Av = 87.45	21–22.4 Av = 21.70	1.06	0.55	0.38	0.72	NS	NS	No squeezing
4	Grade-I phyllites, massive and distinctly jointed	Dube et al. (1982)	Khara hydel project, India	5	2.64	–	38–133 Av = 85.5	6.68–7.07 Av = 6.88	2.26	0.92	–	–	–	–	No squeezing
5	Crushed red shales, moderately squeezing	Jethwa et al. (1982)	Chibro-Khodri tunnel, India	0–0.1 Av = 0.05	2.73	2.8	16.8–37.0 Av = 26.9	10.80	0.56	0.44	3.36	4.29	HS	HS	Moderate squeezing
6	Crushed red shales, highly squeezing	Jethwa et al. (1982)	Chibro-Khodri tunnel, India	0–0.5 Av = 0.08	2.73	6.00	16.8–37.0 Av = 26.9	10.80	0.51	0.41	11.76	14.63	VHS	VHS	High squeezing
7	Soft plastic black clays, moderately squeezing	Jethwa et al. (1982)	Chibro-Khodri tunnel, India	0–0.03 Av = 0.02	2.64	4.5	1.86–8.27 Av = 5.065	0.26–2.83 Av = 1.55	0.32	0.38	14.06	11.84	VHS	VHS	Moderate squeezing
8	Very blocky and seamy slates	Jethwa et al. (1982)	Giri hydro tunnel, India	0.3–0.82 Av = 0.51	2.5	7.6	1–38.0 Av = 19.5	20.00	0.12	0.17	63.33	44.7	VHS	VHS	Moderate squeezing
9	Crushed phyllites	Jethwa et al. (1982)	Giri hydro tunnel, India	0.1–0.32 Av = 0.2	2.3	12.4	38–133 Av = 85.5	6.68–7.07 Av = 6.88	4.67	1.36	2.66	9.12	FS	VHS	High squeezing
10	Crushed shales	Jethwa et al. (1982)	Loktak hydro tunnel, India	0–0.04 Av = 0.02	2.7	7	16.8–37 Av = 26.9	10.80	0.66	0.48	10.61	14.58	VHS	VHS	Moderate squeezing
11	Highly fractured quartzites	Sharma (1985)	Maneri-Bhali hydro project, India	0.5	2.5	7.9	67–128 Av = 97.5	28.25–49.8 Av = 39.03	0.80	0.46	9.86	17.17	VHS	VHS	Not reported
12	Highly jointed dolomites	Singh et al. (1992)	Salal hydel tunnel, India	1.2–1.7 Av = 1.43	2.8	–	46.94	29.0	0.25	0.26	–	–	–	–	–
13	Grade-II phyllites with banded structure of argillaceous material	Singh et al. (1992)	Tehri dam project, India	0.8	2.65	0.38	38–133 Av = 85.5	6.68–7.07 Av = 6.88	3.25	1.15	0.12	0.33	NS	NS	No squeezing

14	Competent massive quartzites of very high strength	Singh et al. (1992)	Kolar gold mines, India	100–200 Av = 141	2.8	–	67–128 Av = 97.5	28.25–49.8 Av = 39.03	0.24	0.23	–	–	–	–	No squeezing
15	Argillaceous conglomerates	Singh et al. (1992)	Khara hydro project, India	0.4	2.7	0.42	105.5	46.19	0.77	0.45	0.97	1.66	NS	LS	No squeezing
16	Argillaceous conglomerates	Singh et al. (1992)	Khara hydro project, India	0.4	2.7	0.75	105.5	46.19	0.77	0.45	0.97	1.66	NS	LS	No squeezing
17	Thinly bedded shales with calcite bands	Singh et al. (1992)	Upper Krishna project, India	15	2.67	0.18	16.8–37 Av = 26.9	10.80	0.18	0.22	1.00	0.82	NS	NS	No squeezing
18	Thinly bedded shales with calcite bands	Singh et al. (1992)	Upper Krishna project, India	15	2.67	0.08	16.8–37 Av = 26.9	10.80	0.18	0.22	0.44	0.36	NS	NS	No squeezing
19	Faulted sandstone	Dalgic (2002)	Istanbul metro tunnel	0.06	2.66	1.05	55	13.00	1.43	0.71	0.74	1.47	NS	LS	–
20	Mudstone	Dalgic (2002)	Istanbul metro tunnel	0.21–2.43 Av = 0.71	2.69	1.05	31	12.00	0.37	0.34	2.82	3.12	FS	HS	High squeezing + heaving
21	Graphite schist (Section-ISK-8)	Kockar and Akgun (2003)	Iliksu tunnel, Turkey	0.9	2.71	2.70 0.92	50	26.60	0.34	0.30	7.94 2.71	9.00 3.07	VHS FS	VHS HS	[a]
22	Highly weathered tuff (BH-1)	Ozsan and Basarir (2003)	Urus dam site, Turkey	0.0018	1.8	0.26	12	8.70	0.46	0.36	0.57	0.72	NS	NS	[a]
23	Moderately weathered tuff (BH-2)	Ozsan and Basarir (2003)	Urus dam site, Turkey	0.11	1.99	0.26	19.9	11.60	0.33	0.29	0.79	0.89	NS	NS	[a]
24	Andesite (BH-3 and BH-4)	Ozsan and Basarir (2003)	Urus dam site, Turkey	0.56	2.39	0.26	23.7	41.90	0.08	0.12	3.32	2.10	HS	FS	[a]
25	Limestone (Section-II)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	1.52	2.68	1.37 1.36 0.08	72	87.20	0.17	0.18	8.06 8.00 0.47	7.61 7.56 0.44	VHS VHS NS	VHS VHS NS	[a]
26	Limestone (Section-III)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	11	2.69	0.96 0.85 0.12	90	67.00	0.21	0.21	4.57 4.05 0.57	4.57 4.05 0.57	H S H S N S	H S H S N S	[a]
27	Limestone (Section-IV)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	11.5	2.69	0.32 0.30 0.11	63	78.70	0.10	0.14	3.2 3.0 1.1	2.29 2.14 0.79	HS FS LS	FS FS LS	[a]
28	Limestone (Section-V)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	10.8	2.65	0.24 0.23 0.095	81	76.80	0.16	0.18	1.5 1.44 0.59	1.33 1.28 0.53	LS LS NS	LS LS NS	[a]
29	Limestone (Section-VI)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	12.3	2.63	0.32 0.31 0.14	75	77.00	0.14	0.16	2.23 2.21 1.00	2.00 1.94 0.88	FS FS NS	LS LS NS	[a]
30	Limestone (Section-VII)	Sari and Pasamehmet oglu (2004)	Kaletepe tunnel, Turkey	9.99	2.63	0.24 0.24 0.11	85	77.00	0.17	0.18	1.41 1.41 0.65	1.33 1.33 0.61	LS LS NS	LS LS NS	[a]

NS, non-squeezing; LS, light squeezing; FS, fair squeezing; HS, heavy squeezing; VHS, very heavy squeezing.

[a] Observed strain was not available, computed strain was used in SI.

account the anisotropy of the jointed rock mass. The observed or, expected strain in the opening will depend on the properties of the rock mass and in addition, size, shape of the opening and the in situ stress state. The expected strain may be assessed through numerical modelling or can preferably be monitored in the field. A Squeezing Index (SI) which is ratio of actual strain to the critical strain, defines the likelihood of squeezing or the occurrence of probable problems during construction. Several case histories analysed systematically prove the usefulness of the Squeezing Index (SI) in tunnels. It is also shown that the efficacy of the adequate support system to reduce squeezing may also be ascertained through the SI approach suggested here.

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