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CASE REPORT

Engineering Geological Study of NWCT Tunnel in Iran with Emphasis on Squeezing Problems

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Abstract This research is concerned with the assessment of squeezing potential along of North Water Convey Tunnel in Iran (NWCT) and its stability analysis has been carried out. In this study, the most suitable methods are utilized for the stability analysis and design of support of the tunnel. For the empirical investigation, the rock mass were classified based on RMR, RMi, GSI and Q systems. The geomechanical properties of the rock mass were determined from the laboratory and field investigations. The results obtained from the analysis show that the tunnel is highly unstable due to the presence of a fault and hence strong supports are need in these regions. Because of high overburden (up to 600 m) and the presence of faults and crushed zones, it was necessary to evaluate squeezing potential along the critical section of the tunnel (Lot2). Therefore, empirical and semi empirical approaches and a new method as critical strain have been used for evaluation of squeezing potential. Some experimental equations were used for estimation of critical strain and also Flac2D program has been used to calculate tunnel convergence. Consequently, squeezing index was estimated. This index is used for determination of squeezing degree. As a result, fair squeezing potential was found in crushed zones along the tunnel route.

Keywords Engineering geology · Rock mass classification · Squeezing · Critical strain · Flac2D

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Introduction

Squeezing is a large time dependent deformation which occurs around the tunnels. It is essentially associated with creep caused by exceeding a limitation of shear stress. Deformation may terminate during construction or continue over a long time period [7]. The effects of squeezing of rock, as the redistribution of stress and formation of a plastic zone around a deep tunnel were first identified by Wiesmann [45] during construction of the Simplon tunnel in Switzerland. Recently, the squeezing phenomenon was addressed and extensively investigated by Singh et al. [42], Aydan et al. [1], Dube [16], Barton and Grimstad [8] and Goel et al. [17, 20, 21].

Prediction of squeezing conditions is of great importance to a designer for designing a stable support system of the tunnel. A number of relatively extensive studies have been carried out on the prediction of squeezing problem [1, 2]. In this study, in addition to numerical modeling and finite element methods, prediction of squeezing has been done using experimental and critical stress methods [9, 15, 38].

A threshold value of tangential strain at the tunnel periphery above which instability and support problems are likely to occur existed. The threshold value of strain is termed as the critical strain [1]. Using Flac2D program, deformation of tunnel has been calculated. This value is employed to predict squeezing index (SI). In this paper, in addition to critical strain, empirical and semi empirical equations have been used for endorsement squeezing analysis in the critical section of the NWCT.

Study Area Characteristic

The NWCT tunnel, located in north of Iran, provides a part of water requirement to the north Iran tropical plains

(Fig. 1). The inlet and outlet portals of the tunnel are 635 and 625 m higher respectively than the free water level and the maximum overburden point of this tunnel is 800 m.

This study reflects the findings of the 40 km long tunnel area constructed by the TBM. The geometry of the tunnel shape is circular and the parameters of tunnel properties have been shown in Table 1. The geological study included the field and laboratories investigations and based on the results, the tunnel alignment of lot1 and lot2 was divided into 12 and 21 lithology types respectively. This paper presents the geological study of the lot2.

Geological Conditions

Based on the results of the samples and boreholes carried out, the transfer tunnel path passes through argillaceous sandstone and shale type of soil/rock as shown in Fig. 2. Some stability problems were predicted at some locations along the alignment and hence a more detailed exploration was carried out. These locations are as follows:

1. Tunnel entrance.
2. Distance from some parts of the tunnel alignment to ground level may be low because at these places, the rocks are weak.
3. Depth of weathered rocks is high.
4. Shear regions and comminuted, and
5. Aquifer horizons.

Based on the results of studies made in the engineering geological zone, along the total route of the tunnel, regardless of surface sediments or hypothesis, the lithology of the rock/soil has been identified to include debris such as argillaceous and shale, sandstone and shale. The investigation of the types of lithology in tunnel path of lot 1 and lot2 was carried out. According to the field and visual investigation, including geological and geotechnical investigation (borehole, core logging and laboratory testing), the rock/soil mass

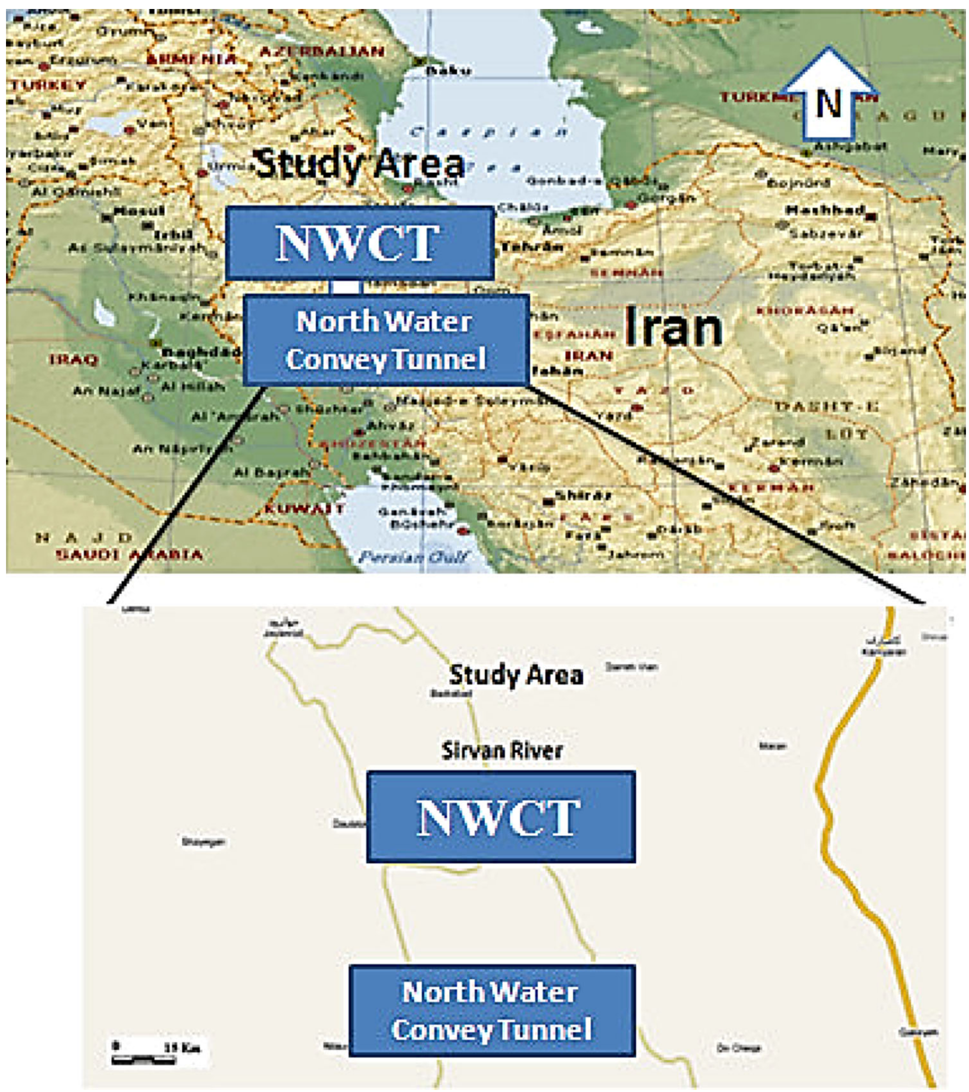


Fig. 1 Location of studied area in Iran

Table 1 General parameters of the tunnel path

Parameters	Properties
Length of tunnel path	
Lot 1	14 km
Lot 2	26 km
Inlet tunnel free water surface (m)	625
Outlet tunnel free water surface (m)	635
Radius of tunnel boring operation (m)	2.3
Maximum overburden	800
Dip	8/1,000

has been identified to consist of 12 lithography types in lot 1 and 21 lithography types in lot2. The types of lithology identified are shown in Tables 2 and 3. The boundaries of types of lithography are according to the stratigraphy and in many cases for the geomechanical features; the lithography was the main factor in separation and classification.

The category classification of massif regional characteristics in geomechanical features is illustrated in Table 4.

As is evident from Table 2, the rock mass, along the tunnel path in lot1, varies from very weak, thinly bedded, crushed and unstable to moderately strong, thick bedding and stable. Similarly, the rock mass along the tunnel path in lot2 (Table 3) varies from very weak, thin bedding, crushed and unstable too weak to moderately strong, crushed, medium bedding and unstable. The physical and mechanical properties of the rock units along tunnel alignment in lot 2 were evaluated and are presented in Table 5.

The estimated rock mass classification systems and Physical and geotechnical properties of the rock along tunnel alignment (lot 2) are presented in Ghiasi et al.'s [22], paper.

Engineering Geological Studies

Units of studied tunnel have been distinguished on the basis of some characteristics such as lithology of layers, differences of structural features and geotechnical characteristics [30]. In general, by considering the repeated units in different parts of the tunnel route, 21 engineering geological units of lot 2 are divisible. Meanwhile, in the critical section, 7 units are located. Most units of the critical sections have a pyroclastic source including many types of argillaceous, sandstone and shale.

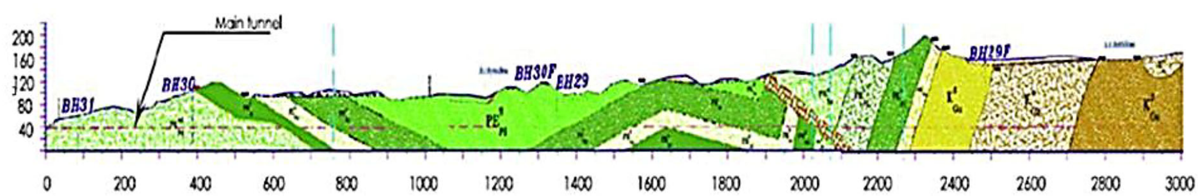
For the study of strength and deformability properties of rock masses, a number of boreholes were drilled and the needed core and block samples for laboratory studies have been selected. Based on the results, some of the geotechnical characteristics of intact rock that are essential for evaluation of squeezing problem are presented in Table 6.

Engineering Classification of Rock Masses

Some rock mass classifications such as rock mass rating (RMR), quality system (Q), geological strength index (GSI) and rock mass index (RMi) systems have been performed on the engineering geological units of NWCT tunnel. The rock mass properties were determined using these system results.

Rock mass rating (RMR) system was initially developed by Bieniawski [10] on the basis of his experiences in shallow tunnels. In this research the version 1989 of RMR [11] has been used. The GSI, is a new rock mass classification system that was developed by Hoek [23].

Palmstrom [32] proposed that RMi is used for general characterization, calculation of the constants in the Hoek–Brown failure criterion for rock masses and preliminary rock support.



Geology	SH-ML1, SH-ML2	ML-SH1	ML-SH2	ML-SH3	ML-SH2	SH-ML3	ML-SH2	ML-SH1	ML-SH4	ML-SH5	SH-LS1	SH-LS2	SH-LS3
Lithology	Argillaceous and shale										Argillaceous and shale, sandstone and shale		
Structure	Layered, jointed, folded		Layered	Layered, jointed, folded	Layered				Fractured		Jointed, folded		Layered, , folded
Average permeability (m/sec)	$15e^{-8}$		$2.5e^{-8}$						$15e^{-8}$		$1e^{-7}$		

Fig. 2 Longitudinal geological profiles of the excavated sections of NWCT (lot 2)

Table 2 Lithology of rock mass along tunnel (lot 1)

Geology	Stability state	
	Description	Type
LI-SH1	Weak to moderately strong, crushed, moderately bedding, unstable	B
LI-SH2	Moderately weak, thin bedding, crushed almost unstable	C
LI-SH3	Moderately weak, thin bedding, crushed almost unstable	C
LI-SH4	Weak to moderately strong, crushed, moderately bedding, unstable	B
LI1	Moderately strong, thick bedding, little crushing, stable	A
LI2	Moderately strong, thick bedding, little crushing, stable	A
LI3	Moderately strong, thick bedding, little crushing, stable	A
LI4	Moderately strong, thick bedding, little crushing, stable	A
LI5	Weak to moderately strong, crushed, bedding medium, unstable	B
SI	Moderately weak, thin bedding, crushed almost unstable	C
CZ	Very weak, thin bedding, crushed and unstable	D
FZ	Moderately weak, thin bedding, crushed almost unstable	C

Table 3 Property of rock mass in different units (lot 2)

Geology	Stability state	
	Description	Type
SH-ML1	Moderately weak, thin bedding, crushed almost unstable	C
SH-ML2	Very weak, thin bedding, crushed and unstable	D
SH-ML3	Very weak, thin bedding, crushed and unstable	D
MLI-SH1	Weak to moderately strong, crushed, bedding medium, unstable	B
ML-SH2	Moderately weak, thin bedding, crushed almost unstable	C
ML-SH3	Moderately weak, thin bedding, crushed almost unstable	C
ML-SH4	Moderately weak, thin bedding, crushed almost unstable	CB
ML-SH5	Weak to moderately strong, crushed, bedding medium, unstable	
SH-LS1	Very weak, thin bedding, crushed and unstable	D
SH-LS2	Moderately weak, thin bedding, crushed almost unstable	C
SH-LS3	Relatively weak, thin bedding, crushed almost unstable	C
SH-LS4	Very weak, thin bedding, crushed and unstable	D
LI2	Very weak, thin bedding, crushed and unstable	D
LI3	Weak to moderately strong, crushed, bedding medium, unstable	B
LI4	Moderately weak, thin bedding, crushed almost unstable	C
LI5	Weak to moderately strong, crushed, bedding medium, unstable	B
LI6	Weak to moderately strong, crushed, bedding medium, unstable	B
LI-MA	Moderately weak, thin bedding, crushed almost unstable	C
LI-	Moderately weak, thin bedding, crushed almost unstable	C
SH	Very weak, thin bedding, crushed and unstable	D
FZ	Very weak, thin bedding, crushed and unstable	D

Table 4 Category classification of massif regional characteristics and geomechanical features

Type	Geomechanical features
A	Very strong, massive, the average distance between discontinuities being significantly more than half a meter
B	Semi-solid to solid, medium to thick layers, the average distance between discontinuity being significantly less than half a meter
C	Semi-solid to weak, thin to medium layer, the average distance of discontinuity significantly less than 0.2 m
D	Poor, crushed

Table 5 Physical and geotechnical properties of the rock along tunnel alignment (lot 2) [22]

Geology	Uniaxial compressive strength (MPa)	Tensile strength (MPa)	Modulus of deformation (GPa)	Dry density (g/cm^3)	Porosity (%)	Weathering in surfaces
SH-ML1	25–50 (35)	1–3	4–6	2.4–2.5	10–15	Moderately weathered
SH-ML2	25–50 (35)	1–3	4–6	2.3–2.5	10–15	Highly weathered
SH-ML3	25–50 (35)	1–3	5–6.5	2.3–2.5	5–15	Moderately weathered
ML-SH1	50–100 (75)	1–3	4–6.5	2.5–2.6	2–5	Slightly weathered
ML-SH2	25–50 (35)	1–3	4–6.5	2.2–2.5	5–10	Slightly weathered
ML-SH3	25–50 (35)	2–4	5–6	2.05–2.5	5–15	Moderately weathered
ML-SH4	25–50 (35)	2–4	4–6	2.2–2.5	5–10	Moderately weathered
ML-SH5	50–100 (75)	5–10	5.5–7	2.3–2.6	3–5	Slightly weathered
SH-LS1	5–25 (20)	1–3	4–6	2.4–2.5	3–15	Slightly weathered
SH-LS2	25–50 (35)	2–4	5–6	2.3–2.6	3–10	Moderately weathered
SH-LS3	5–25 (20)	1–3	4–6	2.3–2.5	5–15	Moderately weathered
SH-LS4	25–50 (35)	1–2, 2–5	4–6,2–4	2.3-2.6	5–10	Moderately weathered
LI2	100–150 (125)	2.5–6	15–30	2.5–2.6	2–5	Slightly weathered
LI3	100–150 (125)	5–10	15–30	2.5–2.6	2–5	Slightly weathered
LI4	50–100 (75)	2.5–6	5–10	2.5–2.6	2–5	Slightly weathered
LI5	100–150 (125)	2.5–6	5–10	1.9–2.7	5–15	Slightly weathered
LI6	100–150 (125)	5–10	15–30	2.5–2.5	2–5	Slightly weathered
LI-MA	25–50 (35)	1–3	2–5	2.2–2.6	2–5	Moderately weathered
LI-SH	5–50 (35)	1–3	2–5	2.2–2.6	2–5	Moderately weathered
CZ	5–150	5–10	5–7	2.5–2.6	2–5	Highly weathered
FZ	5–150	1–3	4–6	2.3–2.5	5–15	Highly weathered

Table 6 Geotechnical characteristics of intact rock [22]

Engineering geological units	m_i constant	Dry density (g/cm^3)	Deformation modulus (GPa)	Uniaxial compressive strength (MPa)
SH-ML2	6	2.3–2.5	4–6	25–50 (35)
SH-ML3	6	2.3–2.5	5–6.5	25–50 (35)
SH-LS1	8	2.4–2.5	4–6	5–25 (20)
SH-LS4	8	2.3–2.6	4–6, 2–4	25–50 (35)
LI2	6	2.5–2.6	15–30	100–150 (125)
CZ	6	2.5–2.6	5–7	5–150
FZ	6	2.3–2.5	4–6	5–150

The Q-system was developed as a rock tunneling quality index by the Norwegian Geotechnical Institute (NGI) [12] and the last update was released in 2004. The Q-value can be calculated as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{1}$$

This classification system includes six parameters of rock quality as following:

1 Rock quality designation (RQD).
2 Number of joint sets (J).
3 Joint surface roughness (J_n).
4 Degree of joint weathering and alteration (J_r).
5 Joint water reduction factor (J_w).

Stress Reduction Factor (SRF)

A stress free form of Q was defined later by Goel et al. [17, 20, 21] as Q. In order to calculate Q, SRF is taken as one, which is given in Eq. (2):

$$Q_n = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times J_w \tag{2}$$

Table 7 The estimated rock mass classification systems [22]

Engineering geological units	RQD (%)	RMR	Q	GSI	RMi	Q_n	Q'
SH-ML2	50–75	40–45	1.5–2	25–35	0.5–1	1.75	1.75
SH-ML3	50–75	40–50	2–2.5	40–45	1–2	2.25	2.25
SH-LS1	50–75	40–45	2–3	35–40	0.5–1.5	2.50	2.50
SH-LS4	75–90	40–50	1.5–2	40–45	1–2	1.75	1.75
LI2	75–90	55–65	4–5	55–60	6–7	4.50	4.50
FZ	25–50	30–40	1–2	25–30	0.5–1	1.50	1.7
CZ	5–25	20–30	0.5–1	15–25	0.025–0.75	0.83	0.96

Table 8 The proposed empirical equations for calculation of σ_{cmass}

Researcher	Equation nos.	Equations	Notes
Ramamurthy [37]	(4)	$\sigma_{cmass} = \sigma_{ci}\ \exp\left[\frac{7.65(RMR-100)}{18.75}\right]$ (MPa)	σ_{ci} is the strength of intact rock (MPa)
Kalamaris and Bieniawski [29]	(5)	$\sigma_{cmass} = \sigma_{ci}\ \exp\left[\frac{(RMR-100)}{24}\right]$ (MPa)	
Palmstrom [32]	(6)	$\sigma_{cmass} = RMi = \sigma_c \times j_p$ (MPa)	
Aidan et al. [3, 5]	(7)	$\sigma_{cmass} = 0.0016RMR^{2.5}$ (MPa)	
Sheorey [39]	(8)	$\sigma_{cmass} = \sigma_{ci}\ \exp\left[\frac{(RMR-100)}{20}\right]$ (MPa)	
Trueman [43]	(9)	$\sigma_{cmass} = 0.5\ \exp(0.06RMR)$ (MPa)	
Aydan and Dalgic [4]	(10)	$\sigma_{cmass} = \frac{RMR}{RMR+6(100-RMR)}\sigma_{ci}$ (MPa)	
Barton [13]	(11)	$\sigma_{cmass} = \gamma\left(Q\frac{\sigma_c}{100}\right)^{\frac{1}{3}}$ (MPa)	γ is the density of rock mass (t/m^3)

Hoek et al. [24] proposed the modified Tunneling Quality Index, Q′, calculated in the same way as the standard Q rock mass classification, except that the SRF and joint water reduction factor (J_w) was set to 1.00.

$$Q' = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \tag{3}$$

These classification systems are used in order to estimate rock mass parameters along the critical section. The results are presented in Table 7.

Rock Mass Properties

Rock mass properties such as deformation modulus of the rock mass and uniaxial compressive strength and Hoek–Brown constants of the rock mass have been determined using empirical equations based on Q_n, Q, RMR, RMi and GSI systems.

It should be noted that each of experimental relations includes a part of characteristics of rock masses (based on the classification system of rock applied in that equation). Therefore, as the different relationships are averaged, this error is reduced in calculating the characteristics of rock masses [19, 34]. Of course, SD was used with the effort to minimize this value as least as possible so that a real average value is obtained. (In some cases, this aim was achieve by removing maximum and minimum).

Strength of Rock Mass

Different researchers have proposed different empirical equations to calculate the strength of rock mass(s) based on rock mass classification systems. The most widely used equations are tabulated in Table 8. The calculated s mass c mass values are given in Table 9.

Deformation Modulus of Rock Mass

In-situ determination of the deformation modulus of rock mass (E_{mass}) is costly and often very difficult. Thus, empirical methods are generally used in estimating E methods, E_{mass}. By means of the empirical can be easily obtained. The proposed equations by different researchers are presented in Table 10.

The calculated E_{mass} values have been given in Table 11. The E_{mass} was calculated using various relationships as mentioned above and it was observed that it varies from a low of 0.86 GPa to a high of 21.6 GPa.

Table 9 Strength values (σ_{cmass}) obtained from different equations

Equation nos.	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	Average	SD
SH-ML2	2.267	3.19	2.96	18.84	1.97	6.40	3.84	9.39	6.11	5.71
SH-ML3	6.288	3.54	2.96	21.73	2.24	7.44	4.20	10.65	7.38	6.42
SH-LS1	1.296	1.82	1.69	18.84	1.13	6.40	2.19	8.66	5.25	6.14
SH-LS4	6.288	3.54	2.96	21.73	2.24	7.44	4.20	9.59	7.25	6.36
LI2	10.210	23.61	10.56	44.62	16.92	18.30	25.00	30.24	22.43	11.33
FZ	0.030	0.67	0.85	11.60	0.39	4.08	0.82	4.94	2.92	3.95
CZ	0.001	0.44	0.85	5.00	0.24	2.24	0.53	3.29	1.57	1.78

Table 10 The proposed empirical equations for calculation of E_{mass}

Researcher	Equation nos.	Equations	Notes
Bieniawski [14]	(12)	$E_{mass}(\text{GPa}) = 2RMR - 100$	For RMR > 50
Sefarim and Pereira [40]	(13)	$E_{mass}(\text{GPa}) = 10^{\left(\frac{RMR-10}{40}\right)}$	For RMR < 50
Grimstad and Barton [18]	(14)	$E_{mass}(\text{GPa}) = 25\,\text{Log} \cdot Q$	For Q > 1
Aydan et al. [3, 5]	(15)	$E_{mass}(\text{MPa}) = 0.0097 \times RMR^{3.54}$	
Read et al. [35]	(16)	$E_{mass}(\text{GPa}) = 0.1\left(\frac{RMR}{10}\right)^3$	
Palmstrom [33]	(17)	$E_{mass}(\text{GPa}) = 5.6RMi^{0.375}$	For RMi > 0.1
Barton [9, 15]	(18)	$E_{mass}(\text{GPa}) = 10\left(Q\frac{\sigma_{ci}}{100}\right)^{\frac{1}{3}}$	

Table 11 Calculated E_{mass} values from empirical methods for different rock units

Equation nos.	(12)	(13)	(14)	(15)	(16)	(17)	(18)	Average	SD
SH-ML2	–	6.49	6.08	5.64	7.68	5.03	8.49	6.57	1.30
SH-ML3	–	7.50	8.80	6.90	9.11	6.52	9.23	8.01	1.19
SH-LS1	–	6.49	9.95	5.64	7.68	5.60	7.94	7.22	1.66
SH-LS4	–	7.50	6.08	6.90	9.11	6.52	8.49	7.43	1.17
LI2	20.00	17.78	16.33	19.12	21.60	11.30	17.78	17.70	3.30
FZ	–	4.22	4.40	2.84	4.29	5.03	5.31	4.35	0.86
CZ	–	2.37	0.00	0.86	1.56	3.97	4.22	2.16	1.69

Constants of Rock Mass

Hoek–Brown failure criterion for rock masses is based on m_m and S_m constants. Some suggested equations based on the empirical methods are used to calculate these constants. These equations are presented in Table 12. The calculated m_m and S_m constants are given in Table 13.

Estimation of In Situ Stress

The main origins of in situ stresses are geological conditions and geological history of the area. In general, estimating of in situ stresses requires a detailed characterization of the site geology and considerable judgment [6]. Different expressions have been proposed in the literature for the coefficient K (ratio of horizontal to vertical tress).

Rummel [36] presented an extensive literature review of stress variations with depth from deep hydraulic fracturing stress measurement conducted in various parts of the world and presented in Eq. (27) in determining K_H and K_h depth. In this research, no field or laboratory tests have been done for the determination of stresses. Thus, they were calculated as:

$$\sigma_V = \gamma H \tag{27}$$

where σ_v is the vertical stress (MPa), γ is the unit weight of rock mass (MN/m^3) and Z is the tunnel depth below surface in m for K_H and K_h the following equations are used [36]:

$$K_H = 0.98 + 250/Z;\ K_h = 0.65 + 150/Z. \tag{28}$$

The results of the equations are presented in Table 14.

Table 12 The proposed empirical equations for calculation of m and s constants of rock mass

Researcher	Equation nos.	Equations	Notes
Hoek et al. [24]	(19)	$\frac{m}{m_i} = 0.135(Q')^{1/3}$	
Hoek et al. [24]	(20)	$S = 0.002Q'$	Jp = jointing parameter for undisturbed rocks
Palmstrom [33]	(21)	$s = j_p^2$	D = disturbance factor
Palmstrom [33]	(22)	$m_{mass} = m_i \times j_p^{0.64}$	
Palmstrom [33]	(23)	$m_{mass} = m_i \times j_p^{0.875}$	
Hoek et al. [25]	(24)	$s = \exp\left(\frac{GSI-100}{9-3D}\right)$	
Hoek et al. [25]	(25)	$\frac{m}{m_i} = \left(\frac{GSI-100}{28-14D}\right)$	
Hoek et al. [25]	(26)	$\alpha = \frac{1}{2} + \frac{1}{6}\left[\exp\left(-\frac{GSI}{15}\right) - \exp\left(-\frac{20}{3}\right)\right]$	

Table 13 Calculated constants of rock mass from empirical methods for different engineering geological units

Equation nos.	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	Average	
									m	s
SH-ML2	0.9761	0.0035	0.0005	0.5128	0.2079	0.0821	15.0000	0.5223	4.17	0.03
SH-ML3	1.0614	0.0045	0.0018	0.7992	0.3812	0.1283	12.3214	0.5096	3.64	0.04
SH-LS1	1.4658	0.0050	0.0025	1.1761	0.5817	0.1073	17.8571	0.5135	5.27	0.04
SH-LS4	1.3015	0.0035	0.0018	1.0656	0.5083	0.1283	16.4286	0.5096	4.83	0.04
LI2	1.3373	0.0090	0.0027	0.9045	0.4515	0.2192	9.1071	0.5034	2.95	0.08
FZ	0.9667	0.0034	0.0056	1.1434	0.6221	0.0751	15.5357	0.5264	4.57	0.03
CZ	0.7991	0.0019	0.0016	0.7647	0.3589	0.0574	17.1429	0.5437	4.77	0.02

Table 14 Empirical results of stresses in NWCT tunnel

Engineering geological units	Unit weight of rock mass (MN/m^3)	Overburden (m)	Vertical stress (MPa)	K_H	K_h	σ_H (MPa)	σ_h (MPa)
SH-ML2	0.0240	500	12.00	1.48	0.95	17.76	11.40
SH-ML3	0.0240	400	9.60	1.61	1.03	15.41	9.84
SH-LS1	0.0245	500	12.25	1.48	0.95	18.13	11.64
SH-LS4	0.0245	450	11.03	1.54	0.98	16.93	10.84
LI2	0.0255	500	12.75	1.48	0.95	18.87	12.11
CZ	0.0255	600	15.30	1.40	0.90	21.37	13.77
FZ	0.0240	500	12.00	1.48	0.95	17.76	11.40

Identification and Quantification of Squeezing Potential

Empirical Approaches

The empirical approaches are essentially based on classification schemes. Two of these approaches are mentioned below.

Singh et al.'s [42] approach is based on 39 case histories, by collecting data on rock mass quality (Q), and overburden (H), defined an equation for predicting squeezing conditions:

$$H = 350Q^{\frac{1}{3}}[m] \tag{29}$$

where H is the overburden (m), and Q is the rock mass quality.

For squeezing condition:

$$H \gg 350Q^{\frac{1}{3}}$$

Goel et al. [17, 20, 21] have proposed a simple empirical approach which is based on the rock mass number (Q_n), as follows:

$$H = \left(27 \times Q_n^{0.33}\right)B^{-0.1} \tag{30}$$

where H is the overburden (m), B is the tunnel span or diameter (m), and Q_n is the rock mass number.

If the right side of the equation is equal or bigger than the left side, squeezing conditions will occur.

Semi-empirical Approaches

With using semi-empirical approaches, deformation around the tunnel has been calculated. In this regard, the support pressure needed for squeezing analysis will be obtained by using analytical solutions for a circular tunnel shape in a condition of hydrostatic stress field.

Jethwa et al. [28] defined Eq. (31) for determining the degree of squeezing on the basis of the rock mass uniaxial compressive strength and the tunnel depth below the surface as:

$$N_c = \frac{\sigma_{cm}}{\gamma \times H} = \frac{\sigma_{cm}}{P_o} \tag{31}$$

The classifications of squeezing potential according to Jethwa et al. [28] approach are given in Table 15.

Hoek and Marinos [26] used the ratio of the rock mass uniaxial compressive strength (σ_{cm}) to the in situ stress (P_o), as an indicator for evaluation of tunnel squeezing potential. With decreasing ratio of rock mass strength to in situ stress and consequently increasing strain, the potential of tunneling problems will be increased due to squeezing conditions. The equation proposed by Hoek and Marinos [26], can be used for assessment of tunneling problems under squeezing conditions, Eq. (32). According to this equation, for the case which tunnel strain (%) is equal or less than 1, no squeezing occurs.

$$\varepsilon_t(\%) = 0.15\left(1 - \left(\frac{P_i}{P_o}\right)\right)\frac{\sigma_{cm}^{-\left(\left(\frac{3P_i}{P_o}\right)+1\right)/\left(\left(\frac{3.8p_i}{P_o}\right)+0.54\right)}}{P_o}. \tag{32}$$

where ε_t is the tunnel strain in percentage, P_i is the support pressure (MPa), σ_{cm} is the rock mass uniaxial compressive strength (MPa) and P_o is the in situ stress (MPa).

Estimating Support Pressure

Rock mass classification systems formed the backbones of the empirical methods that were applied for estimation of support pressure and design of tunnel support. In this research, for estimation of support pressure (P), Q and RMR classification systems have been used.

Goel et al. [17, 20, 21] based on RMR has defined Eq. (33) for predicting squeezing conditions in tunnels with depth of >50 m:

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

P_i is support pressure (MPa), B is tunnel width (m) and H is tunnel depth below the surface (m).

Unal [44] has presented Eq. (34) which can be used to calculate the tunnel support pressure based on RMR:

$$P_i = (100 - RMR)\gamma \times \frac{B}{100} \tag{34}$$

where P_i is the support pressure (kg/m^2), B is the tunnel width (m), and g is the unit weight (t/m^3).

The amount of support pressure has been calculated for all engineering geological units of the tunnel route (Table 16). The results of empirical and semi-empirical equations for identification of squeezing problems in this tunnel are presented in Table 17.

Table 15 Classification of squeezing behavior according to different approaches

Equation no.	(31)
No squeezing	>2
Mildly squeezing	0.8–2
Moderately squeezing	0.4–0.8
Highly squeezing	<0.4

Table 16 Calculated support pressure using empirical equations for different engineering geological units

Equation nos.	(33)	(34)	Average of Pi (MPa)
SH-ML2	0.180	0.062	0.121
SH-ML3	0.144	0.060	0.102
SH-LS1	0.180	0.064	0.122
SH-LS4	0.156	0.061	0.108
LI2	0.113	0.046	0.079
CZ	0.256	0.075	0.165
FZ	0.341	0.081	0.211

Critical Strain

Excavation of tunnel redistributes the stresses within 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 broken zone progresses slowly in the radial direction giving rise to time dependent-large-tunnel convergence. 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 [1]. 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

Table 17 Evaluation of squeezing behavior according to empirical and semi-empirical approaches for rock units along the tunnel

Equation nos.	(29)		(30)		(31)		(32)	
	Result	Behavior	Result	Behavior	Result	Behavior	Result	Behavior
SH-ML2	421.77	Squeezing	278.80	Squeezing	0.27	Highly squeezing	0.15	No squeezing
SH-ML3	458.62	No squeezing	302.91	Squeezing	0.40	Moderately squeezing	0.15	No squeezing
SH-LS1	475.01	Squeezing	313.62	Squeezing	0.23	Highly squeezing	0.15	No squeezing
SH-LS4	421.77	Squeezing	278.80	Squeezing	0.34	Highly squeezing	0.15	No squeezing
LI2	577.81	No squeezing	380.76	Squeezing	1.25	Mildly squeezing	0.15	No squeezing
CZ	400.64	Squeezing	276.14	Squeezing	0.10	Highly squeezing	0.15	No squeezing
FZ	318.00	Squeezing	228.68	Squeezing	0.08	Highly squeezing	0.14	No squeezing

Table 18 Proposed classification for squeezing potential in tunnels with uses of squeezing index [1, 2]

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

exceeds this value, squeezing is likely to occur. This work is based on specimens of jointed rock mass by Singh et al. [41]. An approximate value of critical strain may be obtained from Q as given below.

Singh et al. [41]

$$\varepsilon_{cr} = 31.1 \frac{\sigma_{ci}^{1.6}}{E_i \gamma^{0.6} Q^{0.2}} (\%). \tag{35}$$

Barton [9, 15]

$$\varepsilon_{cr} = 5.84 \frac{\sigma_{ci}^{0.88}}{E_i^{0.63} Q^{0.12}} (\%). \tag{36}$$

ε_{cr} is the critical strain uniaxial σ_{ci} is the compressive strength of intact rock (MPa), E_i is the modules of deformation of intact rock (MPa), and γ is the density of rock mass (gr/cm^3).

And then, SI may be defined by the following equation [1, 2]:

$$SI = \frac{u_a/a}{\varepsilon_{cr}}. \tag{37}$$

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

The observed or expected strain may be obtained from numerical modeling or preferably from actual monitoring in the field. Proposed classification for squeezing potential in tunnels on the basis of SI is presented in Table 18.

In this research, for determining strain at the tunnel periphery, the computer software Flac2D, was used for calculating deformation. FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation. This program simulates the behavior of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. Software calculates elastic–plastic strain in tunnel walls by use of intact rock strength, Hoek and Brown constants, deformability modulus of rock mass and tunnel geometry (Itasca [27]).

Table 19 Results of squeezing analysis with use of the critical strain and final squeezing analysis, along the critical section of NWCT tunnel

Units	Critical strain (%)		u_a/a (%)	Squeezing index (SI)		Behavior		Final squeezing analysis
	Singh et al. [40]	Barton [9, 15]		Singh et al. [40]	Barton [9, 15]	Singh et al. [40]	Barton [9, 15]	
SH-ML2	0.97	0.58	0.16	0.16	0.27	NS	NS	NS
SH-ML3	0.80	0.52	0.52	0.65	1.00	NS	LS	LS
SH-LS1	0.37	0.34	0.64	1.75	1.87	LS	LS	LS
SH-LS4	1.20	0.67	0.15	0.13	0.22	NS	NS	NS
LI2	1.32	0.62	0.16	0.12	0.26	NS	NS	NS
CZ	0.11	0.18	0.33	3.04	1.88	HS	LS	HS
FZ	0.16	0.21	0.40	2.58	1.87	FS	LS	FS

NS no squeezing, *LS* light squeezing, *FS* fair squeezing, *HS* high squeezing

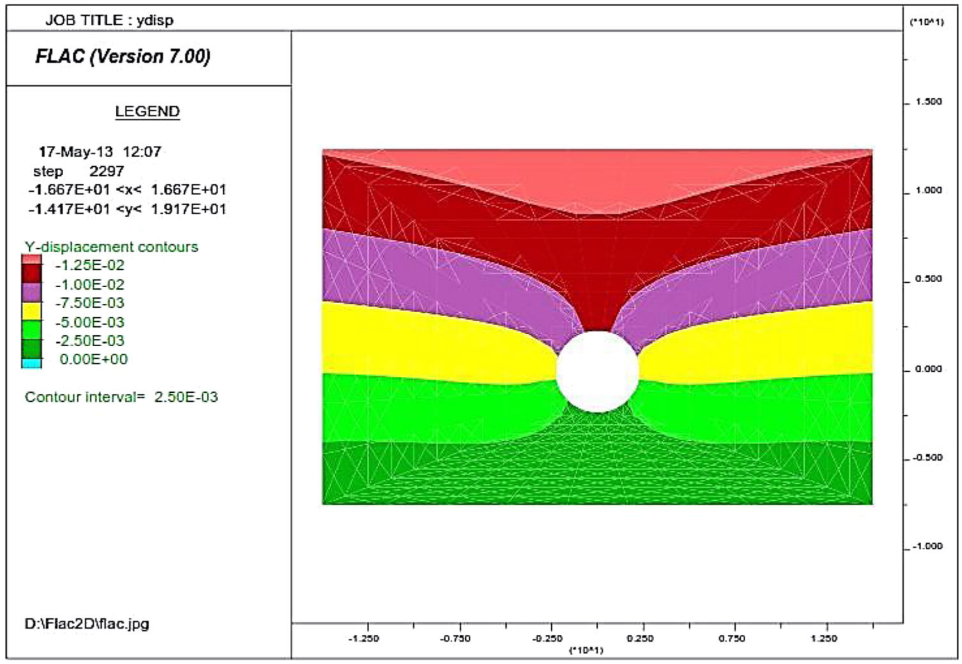


Fig. 3 Total displacements and development of yield zone for CZ zone with 600 m, overburden

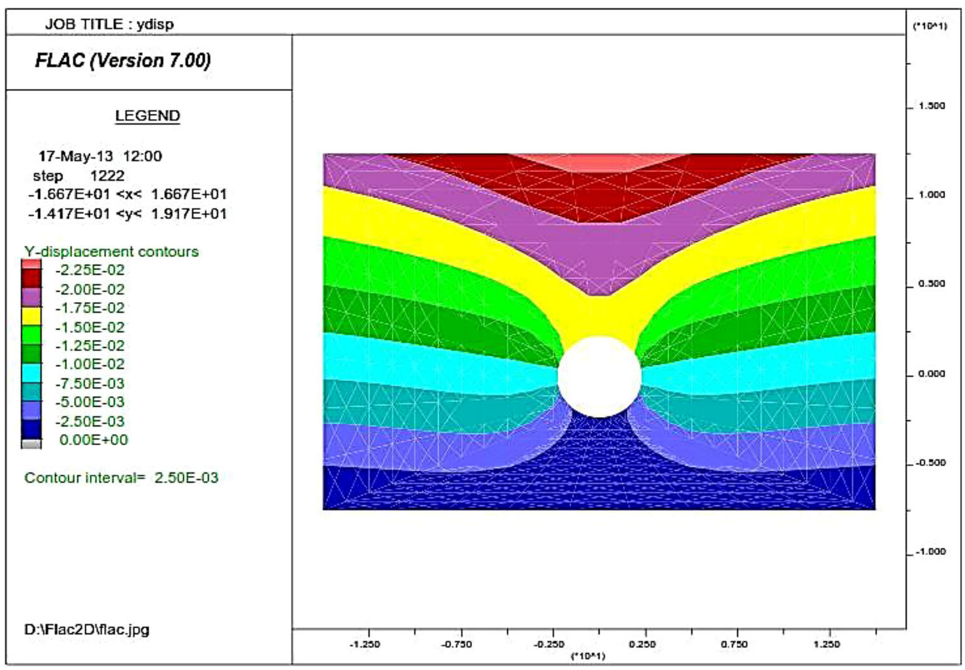


Fig. 4 Total displacements and development of yield zone for FZ zone with 500 m, overburden

For deformability analysis in elasto-plastic mode using Flac2D program, in addition to the above parameters, some extra parameters such as dilation angle and residual strength parameters of rock mass are needed. With note to this point and because of depth of tunnel and also presence of high in situ stresses along the tunnel route, dilation angle has been assumed to be zero [31].

In this analysis, the default boundary condition is fixed (i.e. zero displacement, the Fixed XY condition) for the external boundary. The outer model boundary was set at a distance 10 times as long as that of the tunnel radius.

Results of final squeezing analysis in the critical section of NWCT tunnel are presented in Table 19. Based on the results and because of weak geotechnical characteristic and high overburden, crushed zone (CZ) and fractured zone (FZ) from the 21 geological engineering units in the tunnel route show fair and heavy squeezing problems (Figs. 3, 4). This can be concluded from the distribution of yield zone around the tunnel.

In fact, deformation analysis performed by Flac2D program, eventually, leads to determination of squeezing along the tunnel (in critical strain method). It can be considered as more perfect experimental methods and really a combination of empirical methods each of which includes some parts of factors influencing squeezing. Among the empirical and semi-empirical methods, only Hoek and Marinos [26] method is highly consistent with the critical strain method due to the effect of influential factors on the deformation of rock masses around the tunnels.

Discussion

Knowledge of ground conditions is a key factor in adopting an excavation method and designing a support system for underground openings. In mediums where the rock masses are tectonically disturbed and overburden is high, the rock masses are already under stress before an underground opening is ever excavated. Therefore, prediction of squeezing problem along the tunnel route has been done by using critical strain that combine empirical equations and numerical modeling. Furthermore, other ways such as empirical and semi-empirical approaches were used to evaluate squeezing. Due to interaction between more parameters of rock masses and based on Hoek and Marinos [26] method, totally, crushed and fractured engineering zones show fair potential of squeezing.

Conclusion

A comprehensive engineering geological assessment of rock masses has been carried out at the NWCT in north of Iran. The geotechnical properties of these rocks have been carefully assessed based on laboratory and field investigations for assessing squeezing problem along the tunnel.

Therefore, by using empirical and semi-empirical equations, tunnel route have been evaluated. Results show, a fair squeezing potential in CZ and FZ zones. In order to complete the squeezing analysis, critical strain was employed and using Flac2D program, deformations of tunnel have been computed. Results of critical strain analysis were used in order to identify squeezing, supplement and good agreement with semi-empirical equations that showed a fair squeezing in CZ and FZ zones along the tunnel route. These zones, including crushed (CZ) and FZ, resulted from fault activity along the tunnel rout. For example:

Flac2D program (see Fig. 3) = 1.25 cm

$$CZ \rightarrow \frac{U_a}{a}(\%) = \frac{0.33}{100} \times 4.6\,\mathrm{m} = 1.52\,\mathrm{cm}$$

Flac2D program (see Fig. 4) = 2.25 cm

$$FZ \rightarrow \frac{U_a}{a}(\%) = \frac{0.40}{100} \times 4.6\,\mathrm{m} = 1.84\,\mathrm{cm}$$

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