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The effect of engineering geology on the rock load and squeezing potential in Lot2 of Imamzadeh-Hashem tunnel

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Abstract

The load entering from the ground surrounding on the tunnel's lining is one of the effective parameters in the designation of a tunnel. The amount of this parameter, which is called Rock load in a rocky environment, depends on several factors such as the overburden thickness, geological and geomechanical conditions and the depth of tunnel. In the present study, the amount of rock load for the Lot 2 of Imamzadeh Hashem Tunnel (L2IHT) has been evaluated using various methods and the effect of geological characteristics of the rock units on the tunnel route has been investigated. For this purpose, different methods for estimating the rock load were introduced firstly, and then the amount of this parameter for the tunnel was estimated using these methods. To evaluate the thickness of the plastic zone around the tunnel, the numerical method, finite difference code of FLAC3D software, has been used. Comparison of the results obtained from different methods and their analysis indicates that the rock load applying from H-4 and H-10 rock units is more than the other units. This happens because of the specific geological characteristics of these rock units such as high degree of crushing due to the geological factors and being in the faulted zone. Therefore, it is required for the stronger lining of the tunnel in the range of these units.

Keywords: *Rock load, Squeezing, Engineering Geology, FLAC 3D, Lot 2 of Imamzadeh-Hashem Tunnel*

1. Introduction

The rock masses existing at the overburden of a tunnel create a load on the tunnel lining, called Rock Load, which depend mainly on the depth of tunnel, the density of the rock masses and engineering geological factors. The amount of this load, having a close relationship with the squeezing phenomenon, is one of the most important parameters in the designing of the tunnel's lining and the other types of underground structures. Lack of proper estimation of rock load can lead to the improper designation of the lining and ultimately impose costs and increase the risk of project implementation and utilization. Quantifying of this parameter requires taking into account different parameters that the geological and geotechnical characteristics of the tunnel route and its materials are the most of them. In addition to the rock load, other loading components are involved in the design of the mechanized tunnel lining, including the water load (The hydrostatic pressure of the ground-water contained in the host rock masses of the tunnel) the pressure of the contact grouting behind the segment, the pressure of the trust-jacks of TBM (Tunnel Boring Machine), loads during the transportation of the segment and load of earthquake.

Preceding studies about this subject mostly include the methods and equations proposed by several researchers to evaluate the amount of rock load and the squeezing potential based on the geometrical dimensions of tunnel (such as depth and diameter of tunnel) and engineering geological characteristics of rock masses existing in tunnel route.

In other words, the literature review section of this study comprises the review of the methods that will be discussed through next section. Various methods have been proposed to estimate rock load by several researchers. These methods include empirical, analytical and numerical methods. Each of the methods is able to estimate the rock load based on the particular assumptions. In the present study, the amount of rock load applied to the final lining of the Lot 2 of Imamzadeh Hashem Tunnel (L2IHT) is determined using any of the specified methods and the impact of the geological factors on it, will be investigated.

2. Estimation of rock load

Several studies have been done to estimate the rock load applied to the tunnel lining. The proposed method of Terzaghi (1946) can be considered as one of the most initial and, at the same time, the most basic studies in this regard. In general, the methods of the estimation of rock load can be divided into three experimental, analytical and numerical categories. Each of these methods will be investigated further.

2.1. Experimental method

Different methods and equations have been proposed for estimating rock load by considering dependent variables such as tunnel geometry and rock geomechanical

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properties. Each of the methods has been developed in accordance with the specific conditions and has its own limitations. Generally, these methods can be divided into three categories:

1) Methods developed based on the Terzaghi's rock load.
2) The methods proposed based on rock mass classification.
3) Proposed quantities in technical manuals and regulations.

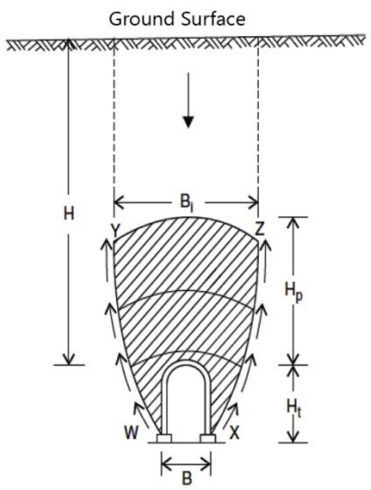


Fig 1. The concept of rock load in tunnels by Terzaghi method

2.1.1. Terzaghi's rock load theory (arc pressure)

The estimation of the load by the method of pressure arc theory is first proposed by Terzaghi in 1946. The general concepts of this method and the parameters used in it are shown in Fig 1. The Terzaghi's method is based on the following equation:

$$P_v = \gamma . H_P$$

Where, H_P and γ are the overburden thickness and unit weight, respectively. Some of the constraints of the Terzaghi's method, such as the inadequacy for the large tunnels, the large extent of the estimated range and development based on the drilling and blasting method, have led to its subsequent modification by Deer et al. 1970. According to this modification, the rock load calculated from this method decreases by 20~25% in mechanized drilling. Using this method in areas that have the potential of squeezing or tunnels with widths of 6 to 13 meters leads to conservative results (Singh and Goel 1999). By studying various tunnelling projects, some adjustments were considered to the theory of Terzaghi's load factor. Accordingly, the rock load is proportional to the geomechanical conditions of the rock mass and is independent of tunnel dimensions in normal conditions.

According to Singh and Goel (1999), in sheared and faulted zones, clayey shales, crushed areas, and in sections of the tunnel that are likely to fall in rocky wedges, rock load is proportional to the dimensions of the tunnel. These methods are summarized in Table 1.

2.2. Methods proposed based on rock mass classification

The second group of the methods has been developed based on rock mass classification. These methods mostly include equations which are presented in Table 2. Two categories of equations based on the Q parameter and the number of the roughness of the joints were developed by Barton, et al. (1974). In this regard, it can also be referred to the equations provided by Bhasin and Grimstad (1996). Unal (1983) presented an equation based on the information obtained from coal mines and RMR classification rock mass and dimensions of the underground space.

Table 1. Empirical methods proposed based on arc pressure

Class	Rock mass properties	Terzaghi (1946)	Deere et al. (1970)			Singh and Goel (1999)		
		Rock Load	RQD* (%)	Rock Load	Considerations	Class	Pv** (MPa)	Ph*** (MPa)
I	Hard and intact	Zero	95~100	Zero		I	Zero	Zero
II	Hard, stratified and schistose,	0~0.5B	90~99	0~0.5B		II	0.0~0.04	Zero
III	massive to moderately jointed,	0~0.25B	85~95	0~0.25B		III	0.04~0.07	Zero
IV	moderately blocky and seamy,	0.25B~0.35(B+Ht)	75~85	0.25B~0.35(B+Ht)	Types IV, V, and VI reduced by about 50 % from Terzaghi values because water table has little effect on rock load	IV	0.07~0.1	0~0.2Pv
V	very blocky and seamy	(0.35-1.1)(B+Ht)	30~75	(0.2-0.6)(B+Ht)		V	0.1~0.2	0~0.5Pv
VI	completely crushed but chemically intact	1.1(B+Ht)	3~30	(0.6-1.1)(B+Ht)		VI	0.2~0.3	0.3~1.0Pv
			0~3	(1.1-1.4)(B+Ht)				
VII	squeezing rock at moderate depth	(1.1-2.1)(B+Ht)	NA	(1.1-2.1)(B+Ht)		VIIa	0.3~0.4	Depends on primary Stress
						VIIb	0.4~0.6	
VIII	squeezing rock at great depth	(2.1-4.5)(B+Ht)	NA	(2.1-4.5)(B+Ht)		VIIc	0.6~1.4	
XI	swelling rock	Up to 250 ft. (80m), irrespective of the value of (B+Ht)	NA	Up to 250 ft. (80m), irrespective of the value of (B+Ht)		VIIIa	0.3~0.8	Depends on type and content of clay
						VIIIb	0.8~1.4	
						VIIIc	1.4~0.2	

* Rock Quality Designation
** Pv: vertical Pressure
*** Ph: horizontal Pressure

Table 2. Empirical methods based on rock mass classifications and technical manuals

	Methods	Equation		Considerations
methods proposed based on rock mass classification	Barton et al. (1974)	$P_v = \left(\frac{0.2}{J_r}\right) Q^{-1/3}$ $P_h = \left(\frac{0.2}{J_r}\right) Q_w^{-1/3}$	*Jr*: Joint roughness number *Qw*: Wall factor	Support capacities is independent of the opening size
	Bhasin, and Grimstad, (1996)	$P_v = \left(\frac{40B}{J_r}\right) Q^{-1/3}$	*Jr*: Joint roughness number *B*: Width of tunnel's span	Q should be lower than 4
	Unal (1983)	$P_v = \left[\frac{100-RMR}{100}\right] \gamma.B$	*γ*: Density *B*: Width of tunnel's span	No suitable for squeezing conditions
	Goel and Jethwa (1991)	$P_v = \frac{7.5B^{0.1}.H^{0.5} - RMR}{20RMR}$	*H*: Tunnel's depth *B*: Width of tunnel's span	No suitable for blasting without steel rib supports
	Singh et al. (1992)	$P_v = \frac{0.2Q_i^{-\frac{1}{3}}}{J_r} \times f \times f' \times f''$	*Jr*: Joint roughness number *f*: correction factor for overburden *f'*: correction factor for tunnel closure *f''*: correction factor for the time after excavation	Developed based on NATM* tunnels *New Austrian Tunneling Method
	Goel et al. (1995a)	$P_v = \left[\frac{f(N)}{30}\right].10^{\left[\frac{H^{0.6}.a^{0.1}}{50.N^{0.33}}\right]}$	*f(N)*: correction factor for tunnel closure *H*: Tunnel's depth *B*: Width of tunnel's span *N*: Number of rock mass	Developed based on NATM tunnels
Relationships provided by technical manuals	US Army (1997) , EM 1110-2-2901	$P_v = 3 \times H_t \times \gamma$ $P_v = 3 \times 1.5 \times H_t \times \gamma$ $P_v = 0.6 \times H_t \times \gamma$ $P_v = (1-2) \times H_t \times \gamma$	*Ht*: Tunnel's depth *γ*: Density	In the case of NATM tunnels, the values increase by the factor of 30%

This equation is not as good as Terzaghi's relationship for spaces with a diameter of more than 6 meters and squeezing grounds. Another equation has been developed based on RMR (Rock Mass Rating) and structural dimensions by Goel and Jethwa (1991). Other equations have also been proposed by Singh et al. (1992), as well as Goel et al. (1995a) for different geological conditions. The equations and parameters used in each of the methods are presented in Table 2.

2.3. Relationships provided by technical manuals and regulations

The third group of methods is the experimental relationships provided by technical manuals and regulations. For example, a method for estimating the rock load for rock tunnels proposed by the US Army (1997) in the form of regulations and its relationship is presented in Table 2.

2.4. Analytical method

This method, which is based on the closed form equations, is provided for circular tunnels in a hydrostatic state. The output of this method is to plot the displacement (or convergence) curve against the pressure on the support system of the tunnels, called the Convergence-Confinement curve, and determines the radius of the plastic zone around the tunnel. Different methods have been proposed for drawing the Convergence- Confinement curve.

In this study, the method developed by Carranza-Torres and Fairhurst (2000), has been used. In addition to the methods mentioned in this section, numerical methods are also used to determine the rock load in deep tunnels, which will be discussed in detail in future sections. In the following, the general conditions of the project will be mentioned and the geological characteristics of the tunnel route will be discussed.

3. Case study Tunnel Description

Lot 2 of Imamzadeh Hashem Tunnel (L2IHT) is located in the northeastern part of Tehran province on the border with Mazandaran province and the between of Abali and Mosha cities along the Haraz road (Figs 2 and 3).

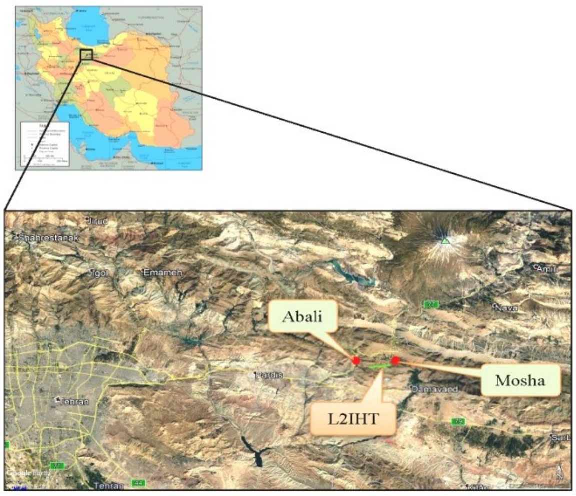


Fig 2. Geographical Position of Study Area

Fig 3. The route of L2IHT tunnel

This tunnel is a part of the road development project, which is responsible for increasing transportation capacity and reducing road accidents in the area of Imamzadeh Hashem region. The length of the tunnel is about 3200 meters and its transverse section is circular with a diameter of about 12.27 meters. The longitudinal slope of the tunnel is 2.5%. According to the initial designation, the tunnel will be excavated mechanically using a Double Shield Tunnel Boring Machine (D.S.TBM). The summary of the machine parameters are shown in Table 3. The primary support system of the tunnel is the concrete segments with 30 cm in thickness. In this construction procedure, the tunnel is excavated with a diameter about a few centimetres larger than the outer diameter of the segment. A Part of this drilling supplement is compensated by the convergence of the tunnel wall and the remainder is filled with p-gravel and grouting. The empty space behind the segment is about 15 centimetres.

Table 3. Machine parameters of TBM

Parameter	Value
Excavation Diameter (m)	12.27
Cutter-head Speed (rpm)	1.95 ~ 3.9
Maximum Trust Force (KN)	16500
Number of disc-cutter	76
Disc-cutter Diameter (mm)	432

4. Geology

The study area is tectonically located in the Alborz zone and lithologically composed of dark limestone, volcanic formations (including volcanic lavas, tuffs, and agglomerate), red sandstone and shale, light pink sandstones and pink to red conglomerates.

The presence of several faults in the tunnel route and, consequently, the tectonization of the range, as well as the influence of the volcanic activities in this region, have created crushed (fragmented) zones around the faults in the tunnel route (Hassanpour et al. 2014b). Fig 4 shows the engineering geological profile of L2IHT route. As seen in this figure, the rock masses along the studying tunnel are categorized into eight engineering geological types (or rock units). The basis for this categorization is most of the engineering characteristics of the rock mass, especially the apparent strength and the general state of discontinuity. The samples of the rock types in the field (surface outcrops) are shown in Fig 5. Table 4 present the geological, geotechnical and geomechanical characteristics of the rock types which have been achieved through geological studies.

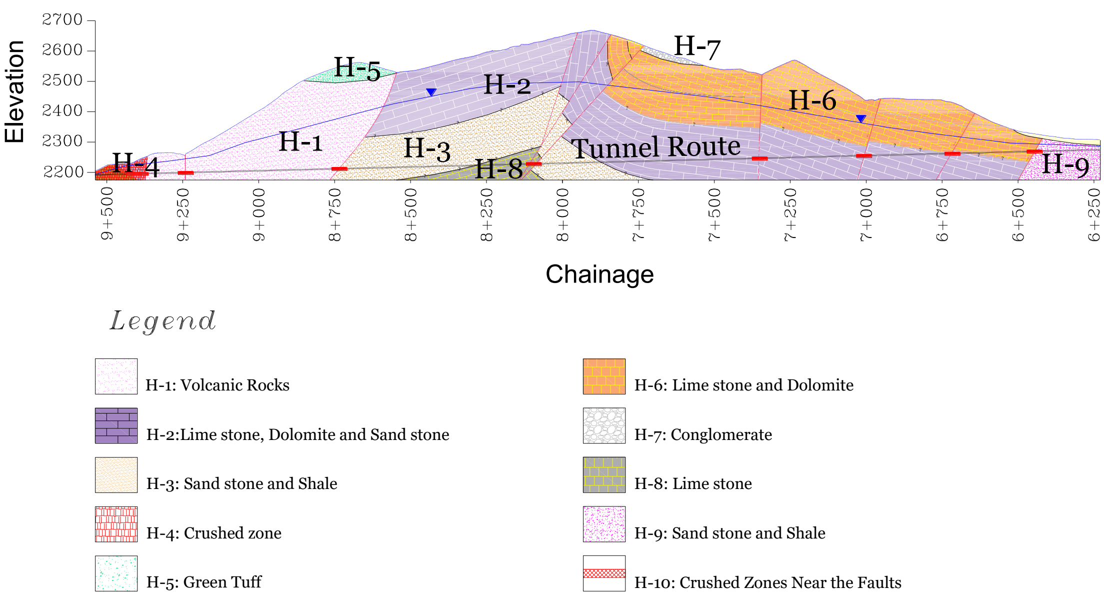
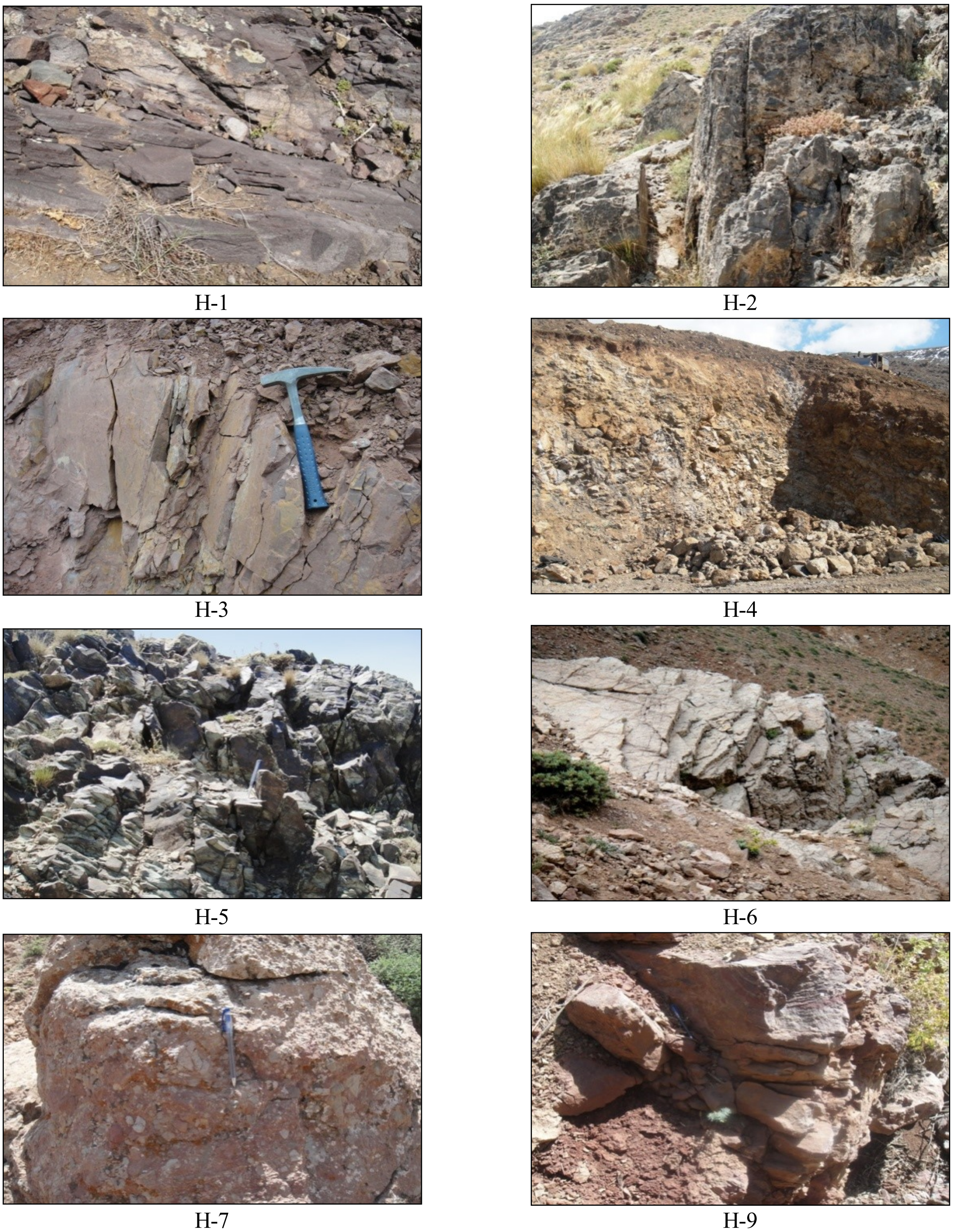


Fig 4. Engineering geological section of L2IHT with categorized rock units (Hassanpour et al. 2014a)



H-1 H-2 H-3 H-4 H-5 H-6 H-7 H-9

Fig 5. Outcrops of the categorized rock units at the studying area

Table 5 present the engineering characteristics of the discontinuities of the rock units. Study on the discontinuities and recording the characteristics were done based on the recommendations of International Society of Rock Mechanics (ISRM 1978).

4.1. Rock Mass Classification

The uniaxial strength of the rock types was evaluated based on Schmidt hammer test as well as point load test, and then rock mass was classified by various methods. In this regard, the determination of RQD, the BGD (Basic Geotechnical Description) classification, as well as the engineering classification of rock mass was done. Some empirical methods such as Q, RMR and GSI (Geological Strength Index, Fig 6) were used to classify rock masses in the studied area.

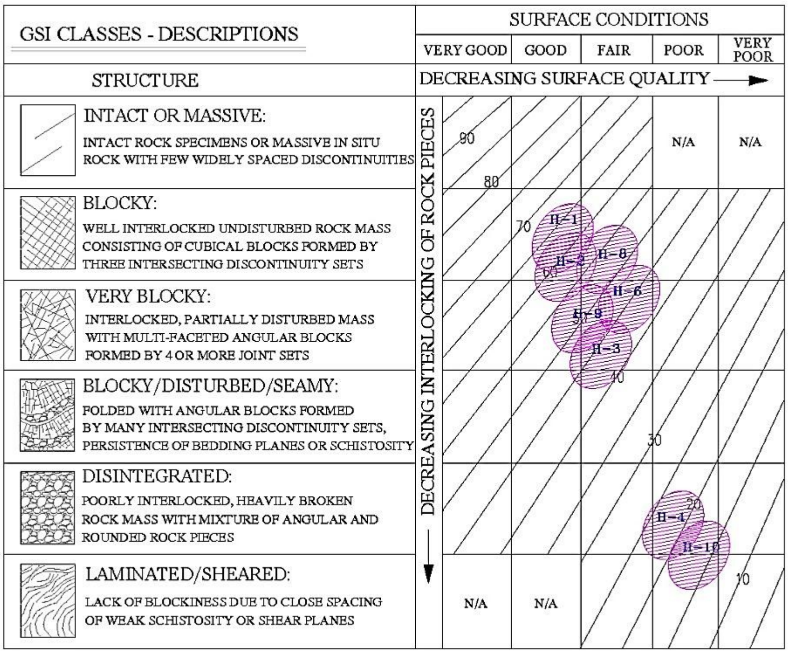


Fig 6. GSI classification of rock units

Table 4. Geological, geotechnical and geomechanical properties of rock units

Engineering Geological Units	Lithological Description	Geological Formation	BGD[1] Classification	RQD (%)[2]	RMR[3]	Q[4]	GSI[5]	UCS (MPa)[6]	Shear Strength Parameters of Rock Mass		Elasticity Modulus	Density (Kg/m3)
									c (MPa)	φ (°)	E (GPa)	
H-1	Volcanic Rocks	Eocene	Volcanic, L2, F3, S3, A3	85~95	63	9	62~67	110~130	2.5	47	20	2600
H-2	Lime stone, Dolomite and Sand stone	Roote, Doroud	Dark Lime & Sandstone, L2, F3, S3, A3	80~85	59	8	57~62	100~120	1.75	45	16	2400
H-3	Sand stone and Shale	Doroud	Sandstone, L5, F3, S4, A3	50~55	43	0.49	43~48	50~60	0.8	42	5	2500
H-4 (Cz)	Crushed zone	-	-	20~25	19	0.02	17~22	20~30	0.2	30	1	2400
H-6	Lime stone and Dolomite	Elika	Limestone and Dolomite, L4, F3, S3, A3	60~65	45	2.52	45~50	40~50	0.7	40	5	2500
H-8	Lime stone	Mobarak	Limestone, L3, F3, S3, A3	45~55	50	1.95	50~55	70~80	0.5	35	7	2400
H-9	Sand stone and Shale	Barout	Sandstone and Shale, L3, F3, S4, A3	40~50	55	2	45~50	20~30	0.3	32	6	2500
H-10 (Fz)	Faulted zone	-	-	20~25	<20	0.02	12~17	20	0.1	30	1	2300

Notes:
1 Based on Anon (1995)
2 Based on Palmstrom (1982)
3 Based on Bieniawski (1989)
4 Based on Barton et al. (1974)
5 Based on Hoek and Brown (1997)
6 Unconfined Compressive Strength

Table 5. Engineering characteristics of the discontinuities of the rocky units

Engineering Geological Units	Type of Discontinuity	Orientation: Dip Direction	Orientation: Dip	Spacing (m)	Opening (mm)	Filling	Persistence (m)	Roughness	Waviness	Weathering
H-1	Layering	-	-	-	-	-	-	-	-	-
	Set 1	90	20	0.4~0.5	2~5	-	0.5~2	Rough	Planar	Moderately
	Set 2	126	86	0.2~0.8	<10	-	1~2	Slightly rough		Slightly
	Set 3	269	77	0.1~1	0.5~1	-	3~6	Rough		Slightly
H-2	Layering	180	85	0.5~1	5~10	Calcite	High	Slightly rough	Planar	Moderately
	Set 1	75	36	0.5~0.8	5~10	-	1~3	Rough	Planar	Slightly
	Set 2	275	76	0.1~0.5	<5	Calcite	1~2	Slightly rough	Planar	Moderately
	Set 3	345	62	0.3~0.6	<5	-	1~2	Slightly rough		Slightly
H-3	Layering	195	35	0.4~0.7	2~6	Clay	High	Slightly rough	Planar	Moderately
	Set 1	324	71	0.3~0.5	5~8	Clay	0.5~1	Slightly rough	Planar	Moderately
	Set 2	76	73	0.3~0.6	2~5	-	0.4~1	Rough	Planar	Slightly
	Set 3	265	77	0.1~0.4	3~6	Clay	0.5~1	Slightly rough	Planar	Moderately
H-4	-	-		-	-	-	-	-	-	Very high
H-6	Layering	45	56	0.1~0.3	10~30	Calcite	High	Very rough	Planar	Moderately
	Set 1	252	73	0.3~0.5	10~20	Calcite	0.5~2	Rough	Planar	Moderately to high
	Set 2	334	79	0.3~0.6	10~25	-	0.4~0.8	Rough	Planar	Moderately to high
	Set 3	306	43	0.2~0.4	10~20	-	0.5~1	Rough	Planar	Moderately
H-8	Layering	170	40	0.5~2	10~20	-	High	Slightly rough		-
	Set 1	155	45	0.1~1	2~5	Clay	0.5~1	Rough		Moderately
	Set 2	330	30	0.5~0.8	<10	-	1~2	Rough		Slightly
	Set 3	55	70	0.1~0.5	1~2	-	0.5~1	Rough		Slightly
H-9	Layering	145	53	0.2~0.5	10~20	-	High	Slightly rough	Planar	Moderately
	Set 1	245	74	0.1~0.3	5~10	Calcite	1~2	Rough	Planar	Slightly
	Set 2	103	58	0.2~0.5	<5	-	0.4~1	Rough	Planar	Moderately
	Set 3	286	31	0.4~0.5	<5	-	0.5~1	Slightly rough		Slightly
H-10	-	-		-	-	-	-	-	-	-

These methods are based on the engineering parameters of intact rock and the discontinuities which should be determined exactly in field and laboratory. For this purpose, in the outcrops of each rock units, the characteristics of discontinuities and intact rock have been recorded and sampled in different stations. Then, by performing some simple tests such as Point Load and Schmidt Hammer tests, recording the engineering characteristics of discontinuities, statistical analysis, and finally engineering judgment, the engineering classification was done on the rock masses (Table 4).

5. Research method

As already mentioned, the rock load on the tunnel lining in mechanized tunnelling is one of the most important parameters that need to be determined exactly through the early stage of studies and ultimately used in the final designation of the tunnel. Different methods have been proposed to estimate rock loads that have been discussed earlier. It should be noted that the use of any of these methods requires consideration of assumptions. In this regard, the existence or absence of stress (Stress Induced Instability) should be investigated first. In the absence of these conditions, the thickness of the plastic zone around the tunnel is calculated and the proportional rock load is determined (Shamsoddin and Maarefvand 2014). Additionally, empirical relationships and formulas, such as Terzaghi's method and rock mass classification (RMR and Q) methods, are also provided to calculate the amount of rock load. Each of these methods is discussed in following sections.

6. Investigation on the Squeezing Potential

Before determining the rock load, it is necessary to identify the parts of the tunnel with squeezing potential. This is important because of the amount of rock load in these parts will be directly linked to the stress-induced instability. When the weak rock types, such as shale, slate or phyllite, are existent in studying area or in presence of the faulted and crushed zones and when the stress of the region is more than rock resistance, the rock mass behaves differently from that of a homogeneous and solid rock. In this case, under the effect of tangential stresses applying on tunnel's wall, a Visco-plastic area, called plastic zone, extends around the tunnel (Fig 7).

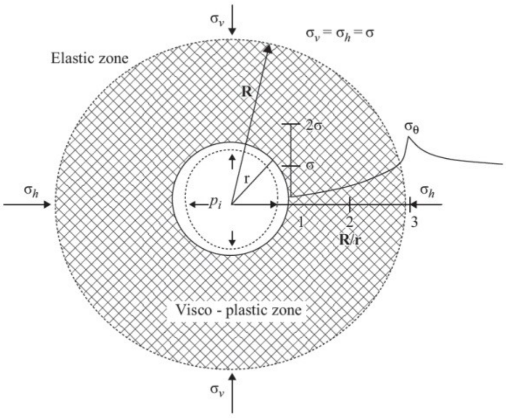


Fig 7. Squeezing behavior in a circular opening (Goodman 1989)

As a result, a time-dependent convergence (squeezing phenomenon) will occur in the tunnel wall, which, will increase pressure on the support system if there is such system in the tunnel. Squeezing, however, is actually a creep behaviour (Panthi 2006). In the literature, momentary squeezing, creep-dependent squeezing and time-dependent squeezing behaviour are expressed. According to Goodman (1989), when an underground space, such as tunnel is created, it causes turbulence in the in-situ stresses. Due to the impossibility of transferring the stress from the created space, the stress concentration is generated in the tunnel wall, as shown in Fig 7. Various methods have been proposed to estimate the squeezing potential of rock masses. In the present study, the methods of Singh et al. (1992) and also Goel et al. (1995b) as empirical methods and the methods presented by Jethwa et al. (1984) and also Hoek and Marinos (2000) as a semi-analytic method (Fig 8) were used for determining the areas with squeezing potential in the route of L2IHT. The relationships used to determine the squeezing potential are presented in Table 6 and the results of the investigation of the potential of occurrence of this phenomenon are presented in Table 7. It is noticeable that the weak rock types such as H-3, H-8, and H-10 are the units with squeezing potential.

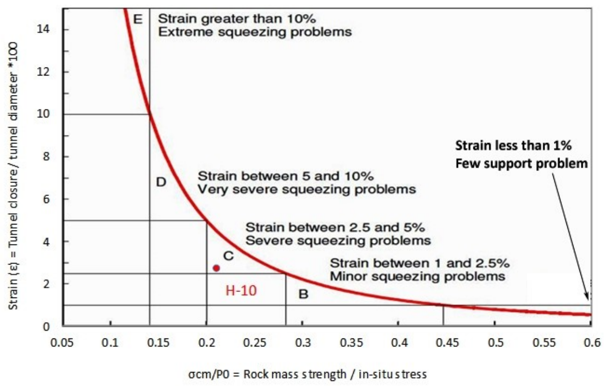


Fig 8. Evaluation of squeezing using the semi-analytical method (Hoek and Marinos 2000)

7. Evaluation of the amount of rock load in the route of L2IHT

Various methods that were discussed in the previous sections have been used to estimate the rock load. Also, by adding numerical analyses to the results of experimental and analytical methods, it is tried to obtain a fairly accurate estimate of this parameter.

Table 6. Methods used to determine the squeezing potential in tunnel route

Method	Presented by	Equation	Consideration	
Empirical	Singh et al. (1992)	$H = 350Q^{1/3}$ H: Critical Overburden	h>>H h<<H	High-Squeezing Non-Squeezing
	Goel et al. (1995b)	$H = (275N^{0.33})B^{-0.1}$ H: Critical Overburden B: tunnel span or diameter	h>>H	High-Squeezing
	Jethwa et al. (1984)	$N_c = \frac{\sigma_{cm}}{\gamma h}$	$N_c < 0.4$ $0.4 < N_c < 0.8$ $0.8 < N_c < 2$ $2 < N_c$	High-Squeezing Moderate-Squeezing Mild-Squeezing Non-Squeezing
Semi-Analitical	Hoek and Marinos (2000)	$\varepsilon_t(\%) = 0.15(1 - p_i/p_0)\frac{\sigma_{cm}}{p_0}^{-(3p_i/p_0+1)/(3.8p_i/p_0+0.54)}$		Diagram of Fig. 8

Table 7. The results of squeezing investigation in tunnel route

Units	Squeezing Probability based on the several methods			
	Singh et al. (1992)	Goel et al. (1995b)	Jethwa et al. (1984)	Hoek and Marinos (2000)
H-1	Non-Squeezing	Non-Squeezing	Non-Squeezing	Non-Squeezing
H-2	Non-Squeezing	Non-Squeezing	Non-Squeezing	Non-Squeezing
H-3	Moderate-Squeezing	Mild-Squeezing	Mild-Squeezing	Non-Squeezing
H-4	Non-Squeezing	Non-Squeezing	Non-Squeezing	Non-Squeezing
H-6	Non-Squeezing	Non-Squeezing	Non-Squeezing	Non-Squeezing
H-8	Non-Squeezing	Mild-Squeezing	Mild-Squeezing	Non-Squeezing
H-9	Non-Squeezing	Non-Squeezing	Non-Squeezing	Non-Squeezing
H-10	Moderate-Squeezing	Moderate-Squeezing	High-Squeezing	Moderate-Squeezing

Table 8. The amount of rock load obtained via experimental methods (ton/m^2)

Units	Terzaghi (1946)		Deere et al. (1970)		Singh and Goel (1999)		Unal (1983)	Goel and Jethwa (1991)	Barton et al. (1974)		Bhasin and Grimstad (1996)	Goel et al. (1995a)		Singh et al. (1992)		US Army, EM 1110-2-2901 (1997)	
	P_{max}	P_{min}	P_{max}	P_{min}	P_{max}	P_{min}			P_v	P_h		Squeezing	Elastic	P_v	P_h	P_v	P_h
H-1	8	0	6	0	4	0	12	9	7	5	13	-	8	10	4	9.6	4.8
H-2	22	16	9	6	10	4	13	12	7	5	14	-	9	11	5	9.6	4.8
H-3	70	22	28	9	20	10	18	18	17	12	31	-	16	26	11	31	16
H-4	70	38	56	31	30	20	26	13	34	34	125	-	28	60	35	19	9.6
H-6	70	22	28	9	20	10	18	8	11	8	21	-	11	12	5	9.6	4.8
H-8	70	22	28	9	20	10	14	12	11	8	20	-	12	15	7	9.6	4.8
H-9	70	22	28	9	20	10	16	2	11	8	19	-	9	9	4	9.6	4.8
H-10	287	134	230	107	140	60	27	59	52	52	191	175	43	151	88	63	32

7.1. Experimental methods

As mentioned, several experimental methods have been proposed to evaluate the rock load. In the present study, these methods have been used and the amount of this parameter has been determined in the route of L2IHT. The results of these studies are presented in Table 8 and Fig 9. It is noticeable that, with a slight difference in the results of different methods, units H-10, H-4 and H-3 will apply higher load on the tunnel than other units.

This issue can be analysed and interpreted according to the geological characteristics of these rock types (Table 4) as well as the parameters required to estimate rock load (Table 2). H-10, H-4 and H-3 units often have relatively low values of Q and RMR and therefore, the rock loads induced from these units will also be higher.

7.2. Analytical methods

As said previously, the output of the analytical method is to draw the displacement curve (or convergence) against the pressure imposed on the tunnel's support system and determination of the radius of the plastic zone around the tunnel. The estimated rock load using this method is determined for the various units in the tunnel route and the corresponding convergence-confinement curves can be plotted. The parameters required to plot the curves according to the Carranza-Torres and Fairhurst (2000) and the estimated loading weight are presented in Table 9.

Table 9. the results of analytical methods

Units	Displacement (Convergence) of Tunnel's Wall (mm)	Pressure of Support System (MPa)	Rock Load (ton/m^2)
H-1	3.2	0.41	0
H-2	4.79	0.53	0.6
H-3	24	0.59	6
H-4	13.2	0.12	3.1
H-6	5.76	0.31	2.1
H-8	11.56	0.65	2.8
H-9	2.59	0.17	0.82
H-10	648	0.75	75

The convergence-confinement curves for the unit H-3, for example, are drawn (Figs 10 and 11). As seen in these figures, the tunnel convergence in this unit is approximately 4 percent, equal to 2.4 centimetres which is less than the thickness of the empty space behind the segmental lining of the tunnel. Typically, the time distance between tunnel excavation and back-grouting behind the segment is about a few hours (in some situations, 1 to 2 days). There is also a distance between the tunnel face and the segments at which the back-grouting is done behind of them. This distance is about 15 to 20 meters. The existence of this spatial and temporal distance, in addition with the over-excavation, provides conditions that the convergence of the tunnel happens and cause to the radial component of the induced stress in the tunnel wall is zero or close to zero. Therefore, the actual load from the ground on the tunnel lining would be zero, unless the ratio of the compressive strength of the rock mass to the induced stresses is reached to the values that the tunnel convergence does not lead to the self-retaining condition. In this case, if the convergence of tunnel is not stopped, for example in case of the tunnel without suitable support, the tunnel failure will occur.

7.3. Numerical modelling

Numerical modelling is one of the useful and current methods used to evaluate the extension of the plastic zone and consequently, the amount of rock load on the segmental lining of the tunnel. In this study, the finite difference code of FLAC3D (Itasca 2006) was used to model the excavation of (L2IHT). The models were constructed based on the recommendations of Baghban Golpasand et al. (2018), Zhao et al. (2012), Hasanpour et al. (2014b) and Ramoni and Anagnostou (2010). Geometrical characteristics of the model are shown in Fig 12. As seen in this figure, the model of the studying tunnel contains 35000 zones and 38046 grid-points.

Because of symmetry conditions of the problem (geometry and loading), only a half of the tunnels was considered. Appropriate boundaries were applied along lateral sides of the model to prevent any movement in the x, y and z directions, whereas the upper surface in the z-direction is free to move. In addition, the bottom boundary in z-direction has been fixed too. It should be noted that to simulate the overburden load (induced by the weight of upper layers), a vertical pressure was used equally to the weight of upper layers.

Selection of the modelling sections for all rock unites was carried out according to the worst (weakest) condition of the units. This condition almost includes the most thickness of the tunnels overburden. This was done because of the increasing of the safety factor thorough the excavation of tunnel.

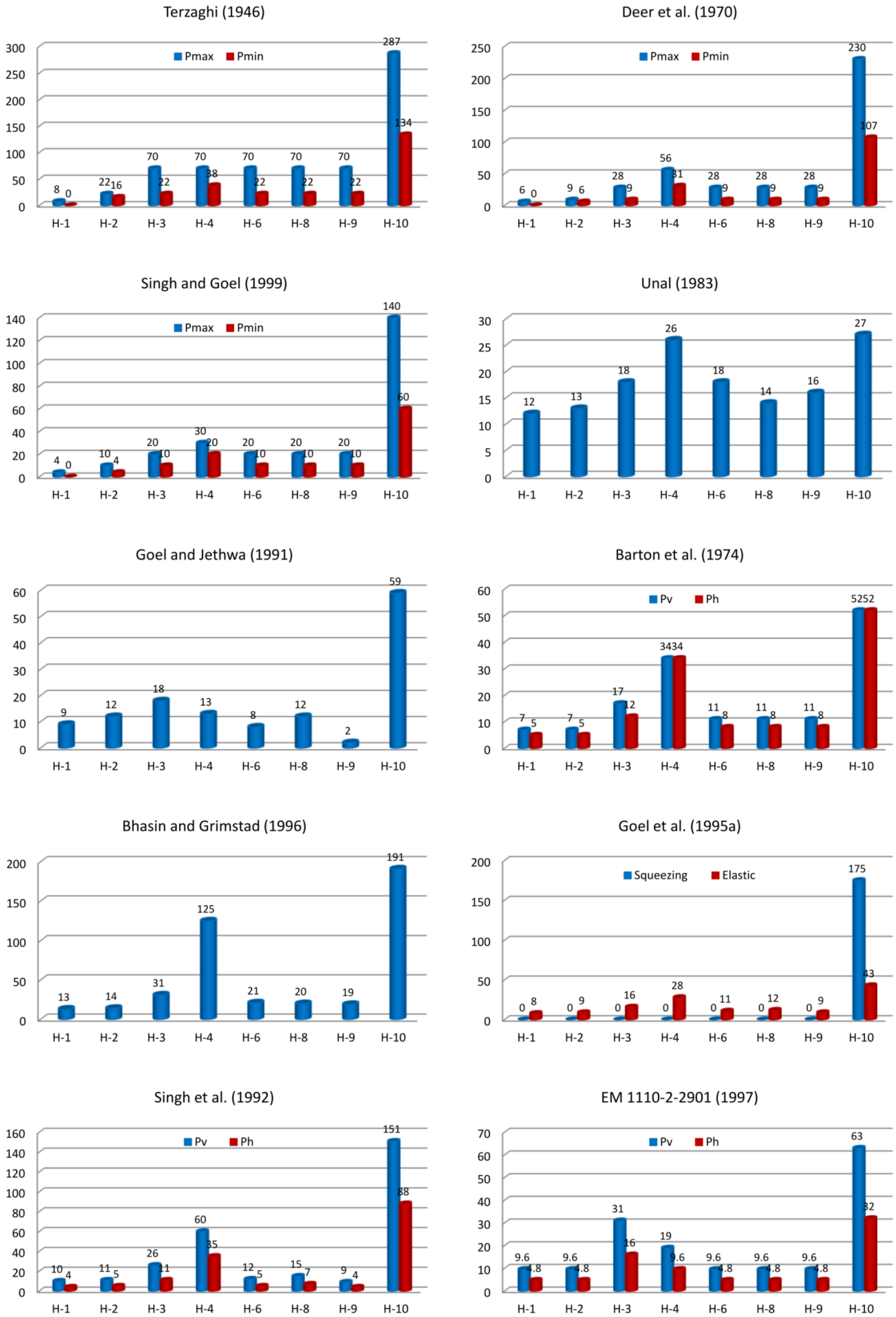


Fig 9. Bar chart of the rock load amounts obtained from experimental methods (ton/m2)

Excavation and advancing of TBM were simulated with respect to the actual processes that are taken place during the construction of the tunnel. The mechanized tunnelling is basically a sequential process. All of its stages have been considered during numerical modelling. These stages, respectively, are:
• Excavation of tunnel equal to the length of a segmental lining ring;
• Generation of EPB machine elements for the new excavation length of the tunnel;
• Application of the face pressure on the new excavation face of the tunnel;
• Allowing the model for relaxation and movement (convergence) of the soil;
• Solving the model to reach the equilibrium state;
• Repeating the above stages.
The shield was modelled by shell element with technical properties presented in Table 10.
Some of the engineering parameters of the materials in tunnel route should be specified before construction of the model. These parameters can be categorized into three groups:

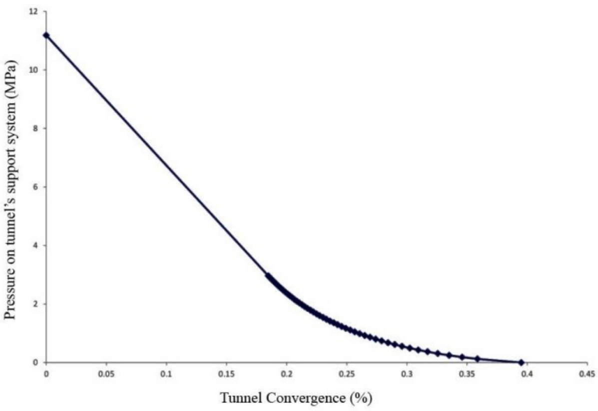


Fig 10. Convergence-confinement curves for H-3 rock type before installation of tunnel's support system

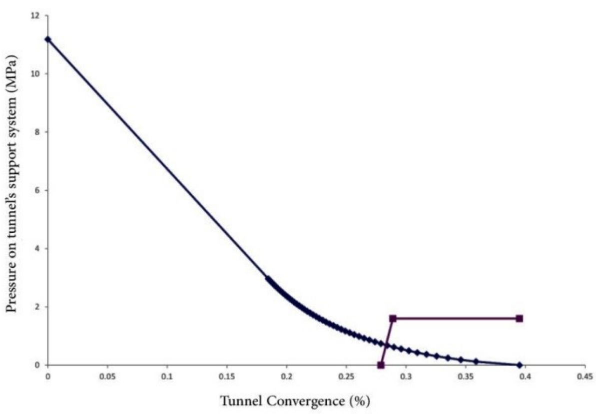


Fig 11. Convergence-confinement curves for H-3 rock type after installation of tunnel's support system at distance of 10 meters from the face of the tunnel

• Physical parameters of rock mass such as density.
• Strength and deformability parameters of rock mass such as Elasticity Modulus (E), Poisson's Ratio (υ), Cohesion (c), Internal Friction Angle (φ), Lateral Earth Pressure Coefficient (K), and etc.
• Other engineering geological characteristics of the tunnel environment such as water-table level, layering of ground and etc.
Physical and geotechnical properties of rock units used in numerical models were previously presented in Table 4. The materials of ground (rock units) were modelled by an elastic-perfectly plastic constitutive model based on Mohr-Coulomb failure criterion.

7.3.1. Numerical results

To evaluate the rock load using the numerical modelling, the extension of the plastic zone as well as the displacement (convergence) of tunnel wall must be considered carefully. The studies of Shaffiee Haghshenas et al. (2017) shows that rock load is in a relationship with the extension of the plastic zone around the tunnel. According to Shaffiee Haghshenas et al. (2017), the pressure exerting from surrounding rocks on the tunnel lining is equal to rock load if the following conditions would be prepared:
A) Convergence of tunnel be less than the clearance (gap) between tunnels interior wall and the segmental lining
B) No failure to be happens through the surrounding rocks.
The outputs of FLAC 3D modelling for H-3 unit, for example, are presented in Figures 13 and 14. Table 11 indicates the extension of the plastic zone around the tunnel and the amounts of rock load obtained from the results of FLAC 3D. As seen in this Table, the thickness of plastic zone, as well as the convergence of tunnels wall, are various for different rock units. H-10 and H-4 are the rock units having similar geological characteristics, the thickness of plastic zone and the amount of rock load for these two units are considerably higher than the others. To compare the results, they are shown by bar chart in Fig 15. The abnormal amounts of rock loads are clearly obvious for the H-4 and H-10 rock units in this figure.

8. The effect of engineering geological factors

The load imposed by the surrounding rock masses on the tunnel's lining as well as the squeezing are originally geological events. In the previous sections, after categorizing the rock units and describing the engineering geological characteristics of them, the squeezing potential and then the amount of rock load of each of these units applying on the tunnel's lining was estimated using several methods which generally proposed based on the geological and geometrical characteristics of tunnel and surrounding rock masses. In this section, the effect of the engineering geological factors on these objects is investigated.

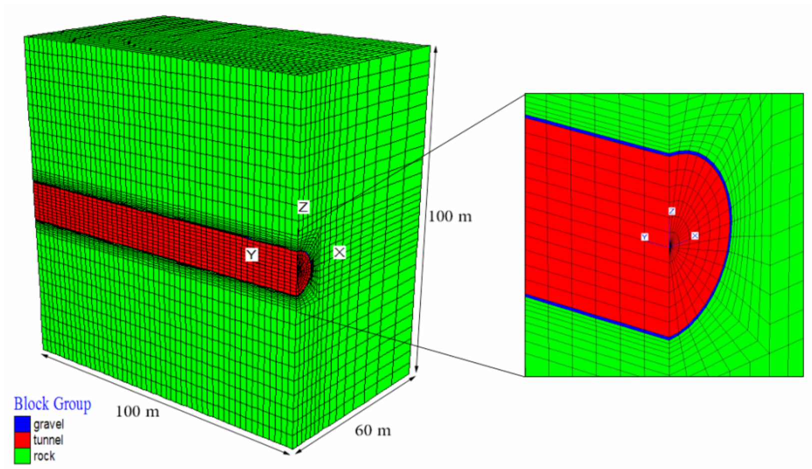


Fig 12. Geometric dimensions of the numerical model

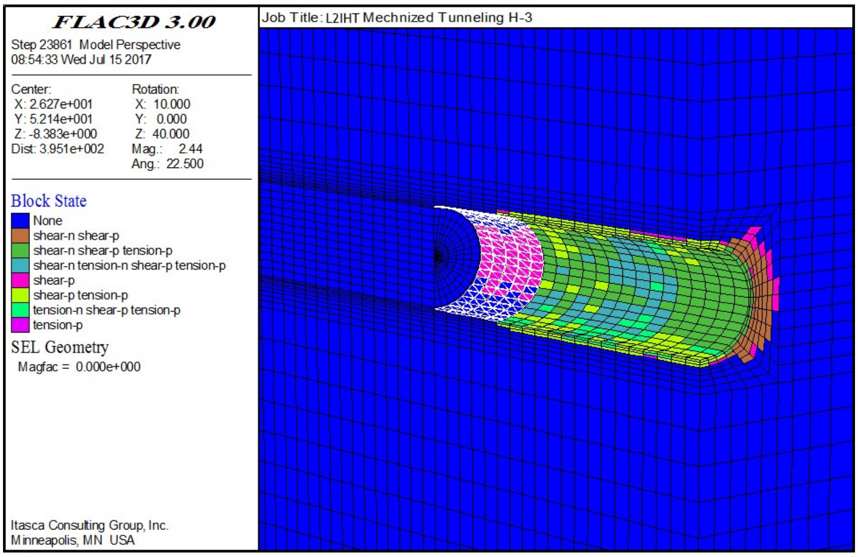


Fig 13. Extension of the plastic zone around the tunnel for H-3 rock unit

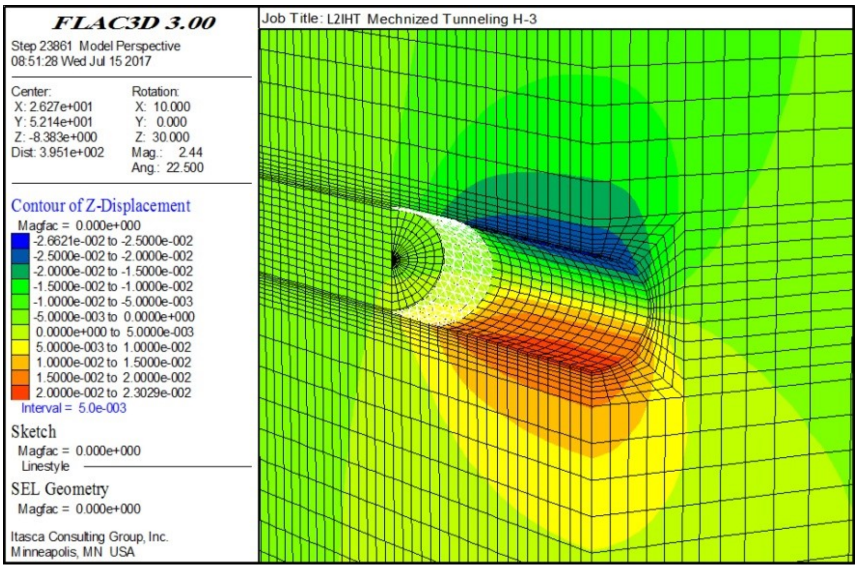


Fig 14. Z-Displacement of the tunnel for H-3 rock unit

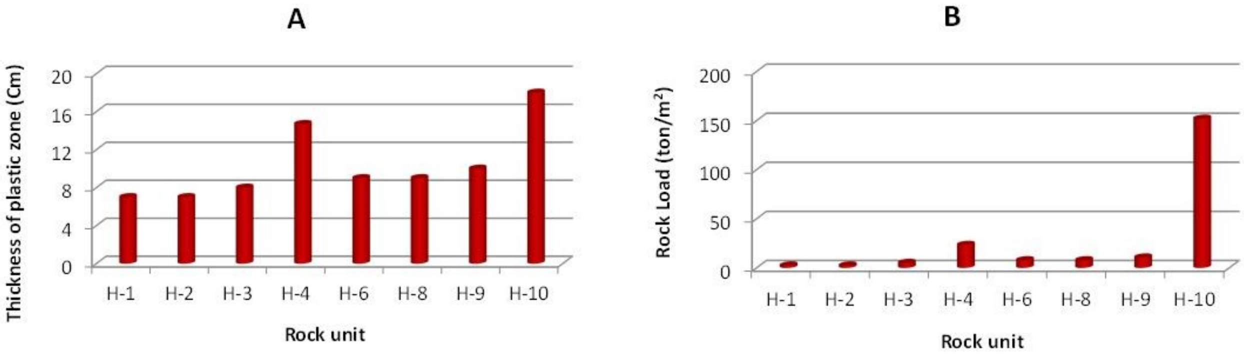


Fig 15. Bar chart of the thickness of plastic zone (A) and the amounts of rock load (B)

Table 10. Technical properties of the shield

Thickness (cm)	Elasticity modulus (GPa)	Poisson's ratio	Density (kg/m^3)
20.0	200	0.20	7800

Table 11. The results of numerical modelling to evaluate extension of plastic zone and rock load

Units	Extension of Plastic Zone (Cm)	Rock Load (ton/m^2)
H-1	7	2.4
H-2	7	2.4
H-3	8	5.1
H-4	14.7	23.4
H-6	9	7.8
H-8	9	7.8
H-9	10	10.6
H-10	18	152

8.1. Squeezing Potential

Squeezing potential thorough the route of L2IHT was studied using the methods and equations presented in the Table 6 and the results were presented in Table 7. As pointed previously, the occurrence of this phenomenon is probable in rock unites of H-3, H-10 and H-8. To investigate the cause of this issue, Table 6 and Fig 4 were studied more carefully. According to Table 6, the parameters of Q and UCS, which play important role in occurrence of squeezing, have low values then the results are rational from this aspect. The remarkable point in this connection is that these parameters in units H-4 and H-9 also have low values, however, and contrary to the previous units, there is no squeezing potential in these units. The reason of this issue is shown in engineering geological section of the tunnel (Fig 4).

According to this figure, it is clear that the tunnel depth in units H-4 and H-9 is much less than the tunnel depth in units H-3, H-10 and H-8. Since the thickness of overburden (tunnel depth) plays a major role in the occurrence of squeezing, the absence of the possibility of the occurrence of squeezing in the units H-4 and H-9 seems rational due to the low thickness of the tunnel overburden in these units. Therefore, it is once again emphasized that in order to study the squeezing potential in the tunnel, it is necessary to investigate the engineering geological characteristics of rock mass as well as the depth of the tunnel concurrently along the tunnel route.

8.2. The Amount of Rock Load

Similar to squeezing, the amount of rock load thorough the route of L2IHT was studied using several methods and the results were presented via Tables and Figures. Basically, the rock load is induced due to the several factors:

a) Weight of rock mass;

b) The changes in the state of the initial stress at tunnel range which occurs due to the excavation of the tunnel;

c) The thickness of the overburden;

d) Most importantly, the engineering geological properties of rock mass that are directly related to the resistance and deformability parameters of the rock mass.

Various methods have been proposed for estimating this parameter. Each of these methods calculates rock load using a set of rock mass engineering parameters as well as other parameters. The results of determination of rock load using these methods presented in Table 12 and comparing them in the form of a bar graph are shown in Fig 16. It is observed that the amounts of rock load obtained from the empirical methods are generally higher than the other methods. It is necessary to note that these methods are mainly developed on the basis of field observations and measurements. In some of the proposed equations of the experimental methods, the amount of rock load is considered independent of the dimensions of the tunnel.

Given that the tunnel is subject to stress-induced instabilities in some sections, it can be said that the assumption is incorrect and therefore the use of these methods should be done with caution.

Table 12. The amounts of rock load obtained from Several Methods

Units	Rock Load (ton/m^2)		
	Empirical Methods	Analytical Method	Numerical Method
H-1	6	0	2
H-2	10	1	2
H-3	22	6	5
H-4	39	3	23
H-6	17	2	8
H-8	17	3	8
H-9	16	1	11
H-10	111	75	152

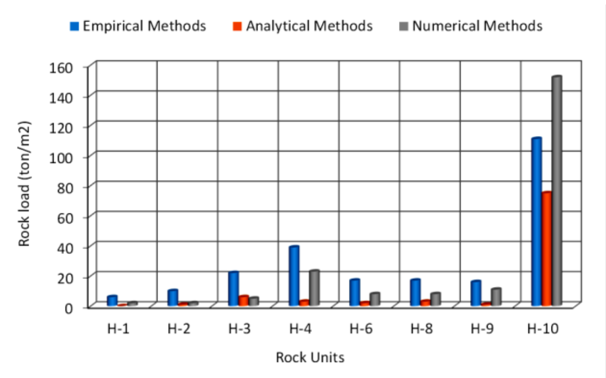


Fig 16. Comparison of the rock loads obtained from several methods

In other words, it is necessary that the results obtained from experimental method not to be supposed as definitive results and they should be controlled by the results of other methods.
The results obtained by analytical methods are generally lower than the other methods then it is necessary to consider adequate safety factors when using the results of this method.
The numerical methods release rational values. The reason for this claim is that this method uses all of the geological and geometrical characteristics of the tunnel and host ground such as engineering properties of rock masses, groundwater condition, overburden thickness and other parameters. Then, the results of numerical methods are more trustable and can be used confidently to evaluate the parameter of rock load.

9. Conclusion

In this paper, various methods for estimating the rock load on the lining of the Lot 2 of Imamzadeh Hashem Tunnel (L2IHT) as well as the potential of squeezing through the tunnel route have been investigated. Geologically, the tunnel route is composed mainly of limestone, sandstone, and volcanic rocks. Unless the sheared zones around the faults and the crushed zones located in entrance portal of the tunnel, the rest of rock units are classified in terms of moderate to good and very good categories and have relatively higher engineering geological parameters.
Investigation on the squeezing potential showed that in the units H-3 and H-8, there is a possibility of mild and moderate squeezing potential. In the unit H-10, there is a possibility of moderate to high squeezing potential. It seems that in units H-3 and H-8, the thickness of the tunnel overburden and in the case of the unit H-10, the geological characteristics of this unit, especially the structural crushing as well as poor specification of strength and deformability characteristics, are the main factor in the occurrence of this potential.
Experimental, analytical and numerical methods were used to estimate the rock load and the results were compared with together. The comparison showed that, the amounts of rock load obtained from experimental methods are generally higher than the results of analytical and numerical methods. The reasons for this were discussed in pervious section.
Estimated rock loads in good to moderate rock units such as H-1, H-2 in the approximate range of 0 to 10 tons per square meter. In the crushed and sheared zones around the faults such as units H-4 and H-10, the value of rock load on the tunnel is about 50 to 170 tons per square meter. It seems that in these zones the calculated rock load is affected by the impact of the load caused by the squeezing phenomenon. Then, the increase of rock load in these zones is directly related to the geological characteristics of these weak units. In other words, the concrete lining of the tunnel in the range of weak geological units, such as units H-4 and H-10, will be subjected to the higher loads than the other parts of the tunnel then the lining in this range requires more reinforcement.

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