

Estimation of the Specific Energy of TBM Using the Strain Energy of Rock Mass, Case Study: Amir-Kabir Water Transferring Tunnel of Iran

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Abstract The specific energy (SE) is the most important parameter to estimate the energy consumption in tunnel boring machines (TBMs). It is defined as the amount of required energy to excavate a unit volume of rock mass which used to predict the performance of TBMs. Several models are used to estimate the SE based on different parameters such as the rock mass properties, disc cutter dimensions and cutting geometry. The aim of this work is to propose new relations between the SE and the strain energy of rock mass (W) using the geological mappings of rock mass and TBM operational parameters from Amir-Kabir Water Transferring Tunnel of Iran. W is an appropriate criterion to estimate SE because it is a function of different parameters such as rock mass behavior, pre and post failure properties and peak and residual strains. In this study, to increase the correlation coefficient of relation between the mentioned parameters, the rock mass is classified in two methods, in the first method according to the geological strength index (GSI) all data is classified in three classes such as weak, fair and good and in the second method using the drop to deformation modulus ratio (η) the

classification of data is performed in three classes such as $\eta < 0.05$, $0.05 \leq \eta < 10$ and $\eta \geq 10$. The results show that there are direct relations between both parameters. It is suggested to estimate SE in all rock mass classes using the proposed relations based on GSI classification.

Keywords Specific energy · Post failure behavior · Strain energy · Geological strength index · Drop to deformation modulus ratio

1 Introduction

The specific energy of excavation (SE) is defined as the energy consumption for the drilling unit volume of rock mass. The concept of the SE has been presented first time by Teale (1965) in the petroleum industry and is an index for evaluation of the drilling processes and excavating result in rock masses. SE can be determined by the collected data from performance of a drilling machine or a TBM. Boyd (1987) calculated rate of penetration based on SE of rock mass (Ramezanzadeh et al. 2004). Correlation of this parameter to the mechanical properties of the rock mass makes it very attractive for researchers (Acaroglu et al. 2008). Some researchers such as Muirhead and Glossop (1968) achieved a good correlation between this parameter and uniaxial compressive strength (UCS). Altindag (2003) found that there is a

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meaningful relationship, between the SE and the fragility of rock mass (Altindag 2003). Bieniawski et al. (2006) found correlation between Rock Mass Excitability (RME) and SE. Acaroglu et al. (2008) proposed a method by using fuzzy logic procedure in the process of rock cutting to predict SE requirement of constant cross-section disc cutters. Atici and Ersoy (2009) estimated the effect of brittleness and destruction energy on SE. Zhang et al. (2012) introduced the mechanical analysis of the shield excavating process into the nonlinear multiple regression of the on-site data and achieved a diagnosis model of the SE. Wang et al. (2012) developed new SE equations with change in disc cutter radius.

The aim of this study is to predict the specific energy of TBM, SE, using the strain energy of rock mass, W . For this reason estimation of SE is conducted in three methods. In the first method prediction of SE is done without considering the post peak behavior of rock mass. In the next step, the classification of post peak behavior of rock mass is considered using two indexes including the Geological Strength Index, GSI, and the Drop to Deformation modulus ratio, η , and finally the relations between both parameters (SE and W) in any classes are estimated.

The proposed method to estimate the SE in detail is mentioned in the following section.

2 The Proposed Methods to Estimate SE

As mentioned above, the required energy to excavate a unit volume of rock mass is named SE, that is dependent on the operational parameters of TBM (Wang et al. 2012). So SE is a function of two components such as the rolling (torque) and the axial (thrust) forces of the cutterhead (Zhang et al. 2012).

Teale (1965) proposed the SE as the following expression (Teale 1965):

$$SE = \left(\frac{F}{A}\right) + \left(\frac{2\pi}{A}\right) \left(\frac{NT}{u}\right) \quad (1)$$

where SE is the specific energy for drilling (MJ/m^3), F is the total thrust (KN), A is the drilling face area (m^2), N is the rate of cutter head revolution (rps), T is the cutter head torque (KN m), and u is the average of penetration rate (m/s).

Equation (1) has two terms, the first representing the SE of the cutter head thrust from static loading, Whereas the second part is the SE of rotation incurred by the rotating cutter head.

SE is also used for estimation and comparison of cutting efficiency (production rates) of mechanized systems and one of the most important factors to define their optimum cutting geometries (optimum ratio of depth of cut to line spacing) for a given rock sample (Bilgin et al. 2013). SE is estimated according to Eq. 2 (Roxborough and Phillips 1975):

$$SE = \frac{FC}{Q} \quad (2)$$

where SE is specific energy (MJ/m^3), FC is cutting force acting on the tool (KN), and Q is yield or rock volume cut in unit cutting length (m^3/km).

According to Eq. 3, as seen in Fig. 1 the SE is equal to integral of complete stress–strain curve in uniaxial compression test (Atici and Ersoy 2009):

$$SE = \int \sigma \cdot d\varepsilon \quad (3)$$

where SE is the specific energy (MJ/m^3); σ is the stress (MPa); ε is the strain. In the next section concept of the W theory is explained in detail.

3 Strain Energy Theory

The area under complete stress–strain curve is called strain energy of rock mass (W). W is affected by rock mass behavior, pre and post failure properties, peak strain and residual strain. Based on Hoek–Brown's proposal, rock mass behavior is considered strain softening, because it can accommodate purely brittle behavior and elastic perfectly plastic behavior (Alejano et al. 2009, 2010; Hoek and Brown 1997). In this research, estimation of the W is carried out based on Dehkordi's proposal (Dehkordi et al. 2011). In this method, pre and post peak behavior of rock mass is assumed linear and the modified Hoek and Brown's criteria is used to estimate principle stresses (Hoek 1994, 2000; Hoek et al. 1998, 2002; Hoek and Diederichs 2006; Hoek and Karakas 2008). The drop modulus is estimated using Alejano's equations (Alejano and Alonso 2005; Alejano et al. 2009, 2010). The value of peak strain is evaluated based on Sakurai's

Fig. 1 Estimation of the destruction specific energy (SE_s) out of the stress–strain curve of a rock sample under unconfined compression test (Atici and Ersoy 2009)

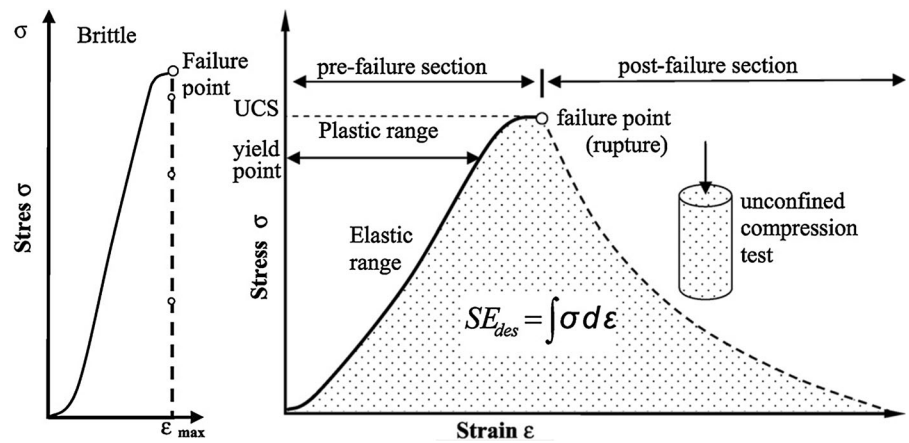
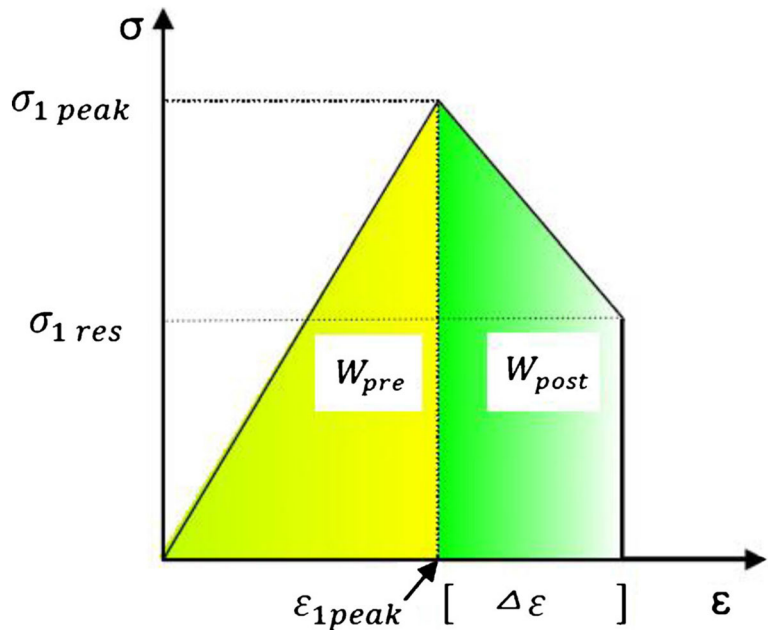


Fig. 2 The stored and residual strain energies in pre- and post-failure, respectively



suggestions (Sakurai 1993, 1997). For estimation of residual parameters of rock mass, GSI_{res} is estimated using Cai’s method (Cai et al. 2004, 2007). These concepts are shown in Fig. 2.

Dehkordi et al. (2011, 2013) proposed a method to estimate the stored strain energy of pre failure and the residual strain energy of post failure according to Eqs. 4, 5 respectively (Dehkordi et al. 2015a). The W is equal to summation of the residual and stored energies (Eq. 6).

$$W_{pre} = \frac{1}{2} \sigma_{1peak} \cdot \epsilon_{1peak} \tag{4}$$

$$W_{post} = \frac{1}{2} [\sigma_{1peak} + \sigma_{1res}] \cdot \Delta\epsilon \tag{5}$$

$$W = W_{post} + W_{pre} \tag{6}$$

where W_{pre} and W_{post} are stored and residual strain energy in pre- and post-failure, respectively and W is the total strain energy.

Table 1 Initial approach to roughly estimating GSI_{res} starting only from the GSI_{peak} (Alejano et al. 2010)

GSI_{peak}	GSI_{res}
75	35–45
70	30–40
60	28–37
50	25–33
40	23–30
30	21–27
25	20–25

The most important parameters to estimate the residual and stored energies of rock mass are peak and residual strength and strain parameters. According to the Dehkordi et al. (2011), peak and residual properties of rock masses were respectively estimated based on the Hoek–Brown failure criterion–2002 edition and Cai et al. (2007) proposals. Cai et al. (2007), proposed estimating the residual strength of rock masses by adjusting peak GSI to the residual GSI value by using of two major controlling factors in the GSI system including residual block volume V_{rb} and residual joint condition factor J_{rc} . In order to obtain Peak failure criterion can be used of GSI_{peak} in Hoek and Brown equations and the residual failure criterion is similarly obtained by changing the value of the peak geotechnical quality GSI_{peak} to that of the residual geotechnical quality GSI_{res} .

The guidelines given by Cai et al., (2007) used to estimate GSI_{res} is mentioned in Table 1. Once this GSI_{res} value has been obtained, a standard technique (for instance, RocLab freeware) can be used to obtain the residual parameters m_{res} and s_{res} (together with the non-changing σ_{ci}) that define the Hoek–Brown residual failure criterion.

As mentioned above, peak and residual strains are important factors to estimate the strain energy. Peak strain is obtained using Sakurai's suggestion. Sakurai (1983, 1997) has suggested that the stability of tunnels can be assessed by using of the strain in the rock mass surrounding the tunnel. The maximum strain is defined by the ratio of tunnel closure (δ_i) to tunnel diameter (d_o). Sakurai proposed the following equation to estimate the percentage strain:

$$\varepsilon_{PC} = A\sigma_{cm}^B \quad (7)$$

where σ_{cm} is the rock mass strength and A, B are constants.

Sakurai (1997) has proposed three warning levels of risk in tunnels according:

$$\text{Log}\varepsilon_c = -0.25\text{Log}E_{rm} - 0.85 \quad (8)$$

$$\text{Log}\varepsilon_c = -0.25\text{Log}E_{rm} - 1.22 \quad (9)$$

$$\text{Log}\varepsilon_c = -0.25\text{Log}E_{rm} - 1.59 \quad (10)$$

where ε_c is critical strain and E_{rm} is deformation modulus of rock mass.

Equations 8, 10 are shown short and long term of stability of tunnels (Sakurai 1997). In this paper, Eq. 8 was used to estimate the critical strain related to peak strength of rock mass.

The proposed method by Dehkordi et al. (2011), to estimate the residual strain, which is highly dependent on the post failure behavior of rock mass, is described in the next section.

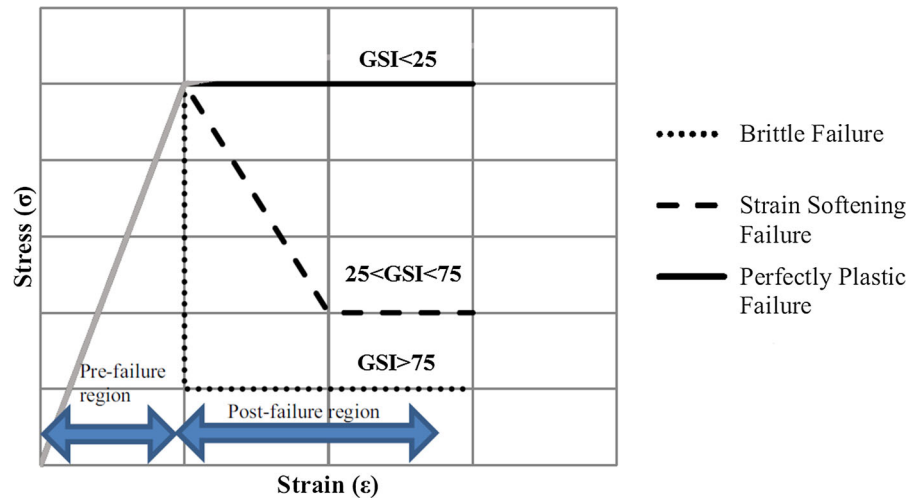
4 Post Failure Behavior of Rock Mass Modes

Hoek and Brown (1997) suggested guidelines to estimate the post failure behavior types of rock mass according to rock mass quality. These guidelines are based on rock types (Hoek and Brown 1997):

- For very good quality hard rock masses, with a high GSI value ($GSI > 75$), the rock mass behavior is elastic brittle and dilation angle is $\frac{\theta}{4}$ (where θ is the friction angle).
- For averagely jointed rock ($25 < GSI < 75$), the strain softening is assumed and dilation angle is $\frac{\theta}{8}$.
- For very weak rock ($GSI < 25$), the rock mass behaves in an elastic perfectly plastic manner and no dilation are assumed (according to Fig. 3).

The brittle and perfectly plastic behaviors are special cases of the strain softening behavior because the strain softening behavior can accommodate brittle and perfectly plastic behaviors (Alejano et al. 2009). One of the most important parameters to identify the strain softening behavior of rock mass is the drop modulus (M) which is the slope of the post failure section of stress–strain curve. The amount of the mentioned parameters is an index to determine the post failure of rock mass, if this modulus approaches to

Fig. 3 Different post-failure behavior modes for rock masses (Tutluoglu et al. 2015)



zero, perfectly plastic behavior is obtained and inversely the brittle behavior is observed if it approaches to infinity.

Based on the previous researches, it is cleared that the drop modulus of rock mass is highly related to rock mass quality (Cai et al. 2007; Hoek and Brown 1997) and the confinement stress level (Alejano and Alonso 2005; Wawersik 1970). Based on these observations, Alejano et al. (2009) proposed the next equations to estimate the drop modulus of the rock mass by using the deformation’s modulus, E_{rm} , GSI_{peak} , intact rock strength, σ_{ci} , confinement stress, σ_3 and s^{peak} .

$$M = -\omega \cdot E_{rm} \tag{11}$$

$$\omega = \left[0.0046e^{0.0768 \cdot GSI^{peak}} \right] \left(\frac{\sigma_3}{\sqrt{s^{peak}} \cdot \sigma_{ci}} \right)^{-1} \tag{12}$$

for $\frac{\sigma_3}{\sqrt{s^{peak}} \cdot \sigma_{ci}} \geq 0.1$

$$\omega = \left[0.0046e^{0.0768 \cdot GSI^{peak}} \right] \left(\frac{\sigma_3}{2\sqrt{s^{peak}} \cdot \sigma_{ci}} + 0.05 \right)^{-1} \tag{13}$$

for $\frac{\sigma_3}{\sqrt{s^{peak}} \cdot \sigma_{ci}} \leq 0.1$

The residual strain is estimated according to the following equations:

$$M = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{\sigma_{Ires} - \sigma_{Ipeak}}{\varepsilon_{Ires} - \varepsilon_{Ipeak}} \tag{14}$$

$$\Delta\varepsilon = \frac{\sigma_{Ires} - \sigma_{Ipeak}}{M} \tag{15}$$

where M is the drop modulus, ε_{Ipeak} is the strain in peak point, and $\Delta\varepsilon$ shows the post peak strain (changing the strain from peak to residual strength).

Dehkordi et al. proposed a new classification of rock mass based on the η ratio to predict the post failure behavior of rock mass. The η ratio is equal to the absolute value of the drop to deformation modulus ratio and obtained from following equation:

$$\eta = \left| \frac{M}{E_{rm}} \right| \tag{16}$$

The classification of post peak behavior of rock mass based on η ratio is as below (Dehkordi et al. 2015b):

1. if $\eta < 0.05$, the rock mass behaved in plastic manner
2. if $0.05 \leq \eta < 10$, the post peak behavior is strain softening
3. if $\eta > 10$, the post peak behavior is brittle

As mentioned above, there are two indexes to estimate the post failure properties of rock mass including the η ratio and the GSI.

In this study the classification of Amir-Kabir Water Transferring Tunnel is adapted according to both of mentioned indexes.

Fig. 4 Location of Amir-Kabir tunnel (Lot-2) (SCE 2009)

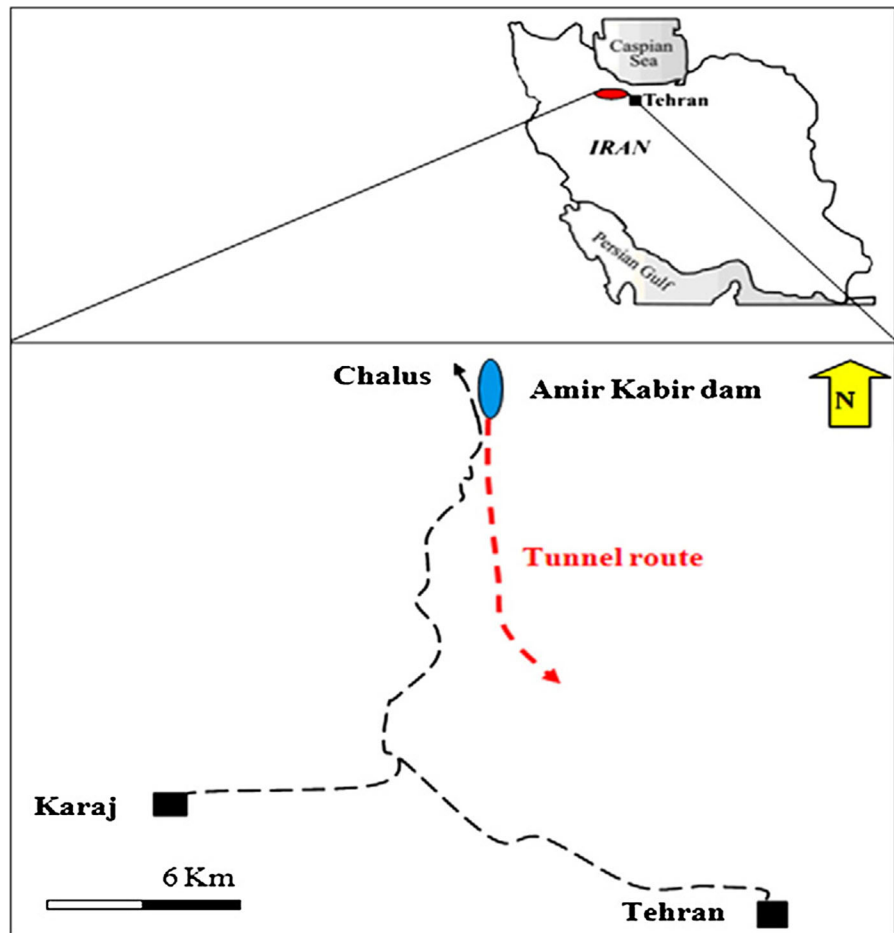


Table 2 Double shield TBM specifications

Excavation machine type	Double shield TBM
Length of machine	166 m
Length of shield	10.6 m
Weight of machine	750 ton
Weight of shield	170 ton
Weight of cutter head	45 ton
Number of disc cutters	31
Average spacing of disc cutters	75 mm

5 Projects Description and Geology

Amir-Kabir Water Transferring Tunnel of Iran is a water transferring tunnel with $16 \text{ m}^3/\text{s}$ water flow capacity from Amir-Kabir dam to Tehran. Location of tunnel is shown in Fig. 4.

Total length of tunnel is 30 km and it is divided into two sections. The Lot2 with 13.45 km in length and 4.66 m in diameter is excavated with a double shield TBM Herrenknecht model S323. The basic specifications of utilized TBM are shown in Table 2.

The tunnel alignment contains a series of asymmetric faults and folded formations. The lithology of this area consists of a sequence of Karaj formations and a variety of pyroclastic rocks, often interbedded with sedimentary rocks. The rock mass types of tunnel are gray tuff, siltstone, sandstone, monzodiorite and monzogabbro. The engineering geological profile of the tunnel is shown in Fig. 5.

In general, by considering the repeated units in different parts of the tunnel route, 20 engineering geological units were distinguished. Geological and geomechanical properties of rock mass types are mentioned in Table 3 (SCE 2009).

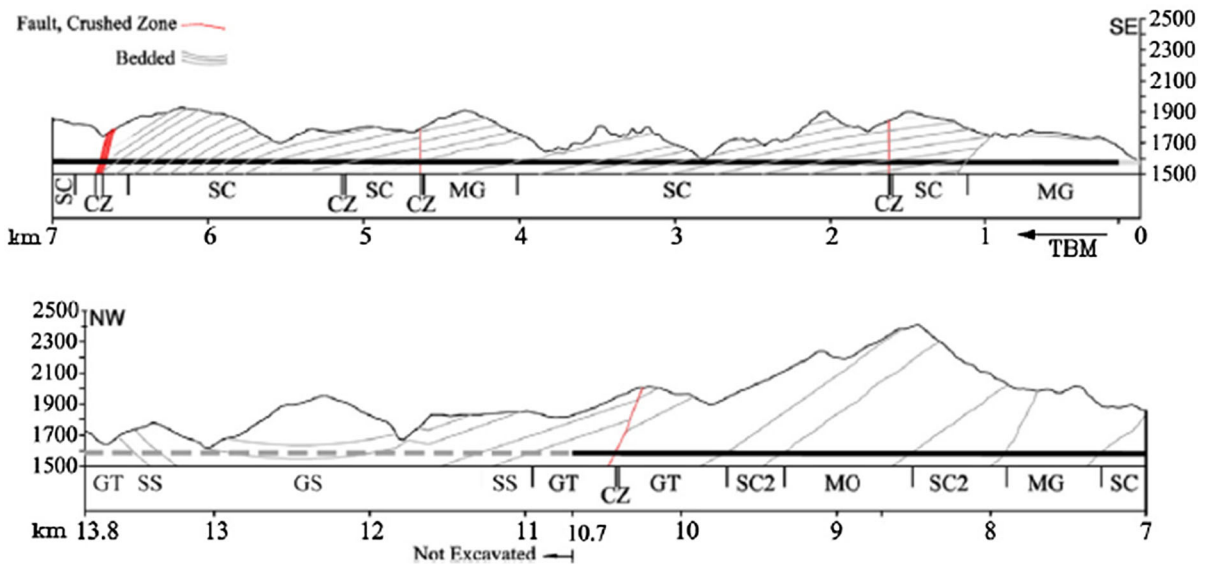


Fig. 5 Engineering geological profile of Amir-Kabir tunnel (Lot-2) (SCE 2009)

Table 3 Geomechanical properties of engineering geological units of tunnel

Eng. Geo. unit	Sign	GSIPeak	σ_{ci} (MPa)	σ_{cm} (MPa)	E_m (GPa)
Diorite	DIO	65–75	100–200	61.7	15
Gabbro	GA	60–70	100–200	57.4	12.5
Lithic crystal tuff	LCT	40–50	50–100	15.2	5
Ash and lithic tuff	AL	25–35	50–100	13.7	4
Lithic and lapili tuff	ll	60–70	50–100	32.6	7.5
Lapili tuff	LT	50–60	50–100	16.2	5
Gabbro rubble	BG	55–65	50–75	17.1	7.5
Thick lithic tuff	LC	50–60	100–150	31.2	7.5
Lithic and ash tuff	LA	55–65	50–100	20.8	7.5
Massive lipili tuff	MLT	50–60	50–100	26.3	5
Monzonite	MO	70–80	100–200	69.3	15
Gray tuff	GT	50–60	50–100	16.6	5
Lithic lapili tuff	LLT	65–75	100–150	43.5	10
Ash tuff	AT	25–35	50–100	8.8	2.5
Cream tuff	CT	70–80	100–150	44.5	10
Gray lithic tuff	GLT	45–55	50–100	22.5	5
Green and cram tuff	TU	45–55	100–150	24.7	7.5
Ash lithic tuff	ALT	40–50	50–100	9.9	4
Fractured zone	FZ	25–30	–	8.1	2.5
Crushed zone	CZ	20–25	–	6.3	1.5



project no.: S-323

advance 5789

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start		end	
at: 30.04.2000	at: 14:44:05	at: 30.04.2000	at: 15:25:26

time:	[hs:min:sec]	[hs:min:sec]	[hs:min:sec]	[hs:min:sec]
advance:	00:17:14	ringbuild:	00:02:27	stop:
			00:21:29	total:
				00:41:19

cutter head:	average	minimum	maximum		
rotation speed [rpm]	6.0	5.8	6.5	rotating direction left [%]	0
torque [kNm]	825.15	62.09	1252.13	rotating direction right [%]	100
current drive [%]	52				
torque drive [%]	825.15				

thrust cylinders mode double shield:					
	average [kN]	average [bar]	minimum [bar]	maximum [bar]	
group A	644	52	28	85	
group B	1016	83	36	105	
group C	955	78	38	99	
group D	719	58	15	101	
	start [mm]			end [mm]	
group A	181			1488	
group B	172			1489	
group C	168			1497	
total force [kN]	3335	advance speed [mm/min]	76.5	penetration [mm/rpm]	12.7

thrust cylinders mode single shield:					
	average [kN]	average [bar]	minimum [bar]	maximum [bar]	
group A	690	45	30	78	
group B	152	10	4	48	
group C	74	5	0	31	
group D	317	21	11	39	
	start [mm]			end [mm]	
group A	1723			1472	
group C	1678			1756	
group D	1708			1786	
total force [kN]	1232	advance speed [mm/min]	0.0	penetration [mm/rpm]	0.0

others:	average	minimum	maximum		average	minimum	maximum
tank gantry [°C]	31			pressure gear oil 1 [bar]	23.0	22.6	23.8
gear oil [°C]	35			pressure gear oil 2 [bar]	52.0	51.4	52.7
roll [mm]	0	0	0	strokes grease main drive	32	1	64
current machine belt [%]	46	39	56	methane 1 [% LEL]	0.00	0.00	0.00
current crossbelt [%]	39	37	43	methane 2 [% LEL]	0.00	0.00	0.00
current back-up belt [%]	34	30	36	methane 3 [% LEL]	0.00	0.00	0.00
advance pump [bar]	116	55	149	oxygen 1 [Vol. %]	20.71	20.43	20.75
coupling front shield [kN]	84	77	172	oxygen 2 [Vol. %]	20.80	20.79	20.81
coupling gantry [kN]	45	26	57	carbon dioxide 1 [%]	0	0	0
pressure foam system [bar]	0.1	0.0	1.3	carbon dioxide 2 [%]	0	0	0
quantity water [l]	4	4	4	carbon monoxide 1 [ppm]	0	0	0
quantity liquid [l]	46	46	46	carbon monoxide 2 [ppm]	0	0	0

◀ **Fig. 6** A recorded TBM operational data sheet from Amir-Kabir tunnel (SCE 2009)

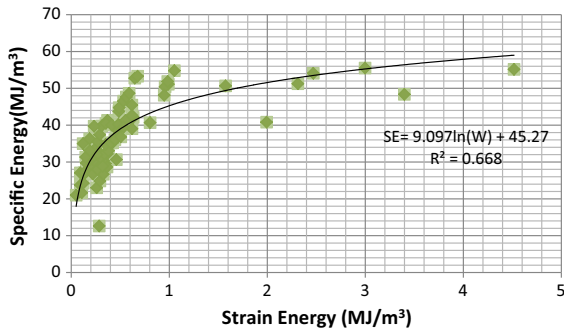


Fig. 7 Relation between SE and W for all rock mass classes

In the next section SE of TBM is estimated based on W with considering both indexes of post failure behavior of rock mass.

6 Estimation of SE Using W

The main purpose of this section is to predict SE of TBM using W. For this reason, 83 sections in different zones of Amir-Kabir Water Transferring Tunnel of Iran were considered and estimation of SE was conducted using of recorded actual information in TBM operational data sheets of mentioned tunnel. One of the operational data sheets was shown in Fig. 6.

The minimum and maximum of W varies between 0.04 and 4.51, depending on the quality of rock mass and confinement stress. The regression analysis is used to investigate the kind of relation between SE and W without considering the post failure behavior of rock mass.

As illustrated in Fig. 7, there is a direct relation between both parameters and a new logarithmic equation with $R^2 = 0.668$ (Eq. 17) is proposed to predict SE based on W.

$$SE = 9.097 \ln(W) + 45.27 \tag{17}$$

where SE is the Specific Energy of TBM (MJ/m^3), and W is the Strain Energy of rock mass (MJ/m^3).

Table 4 Rock mass classification based on GSI

GSI	Engineering geological units	Post failure behavior
$GSI < 25$	FZ–CZ	Perfectly plastic
$25 < GSI < 75$	GA–LCT–AL–LL–LT–BG–LC–LA–MLT–GT–LLT–AT–GLT–TU–ALT	Strain softening
$GSI > 75$	DIO–MO–CT	Brittle

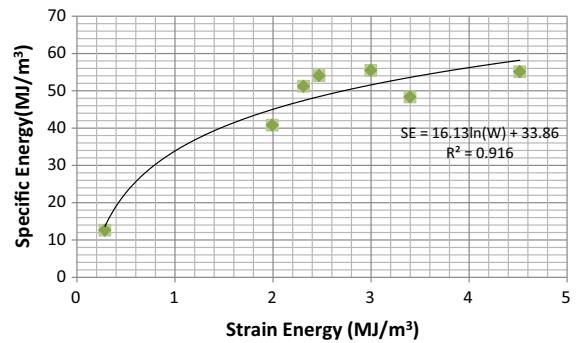


Fig. 8 Relation between SE and W in perfectly plastic rock mass

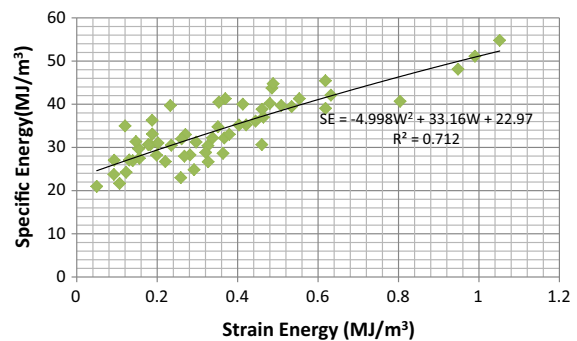


Fig. 9 Relation between SE and W in strain softening rock mass

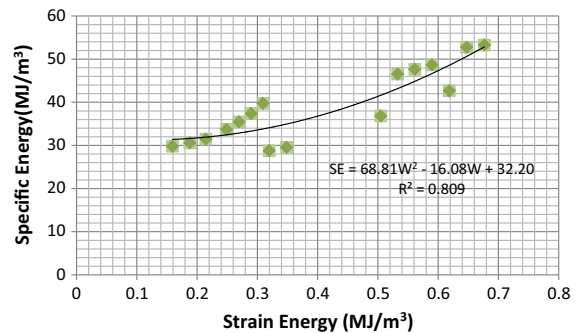


Fig. 10 Relation between SE and W in brittle behavior rock mass

Table 5 The proposed equations to estimate the specific energy using the strain energy of rock mass based on the GSI classification

Eqs	GSI	Equations	Correlation coefficient
18	$GSI < 25$	$SE = 16.13 \ln(W) + 33.86$	$R^2 = 0.916$
19	$25 < GSI < 75$	$SE = 4.998W^2 + 33.16W + 22.97$	$R^2 = 0.712$
20	$GSI > 75$	$SE = 68.81W^2 - 16.08W + 32.20$	$R^2 = 0.809$

Table 6 Rock mass classification based on η

η	Engineering geological units	Post failure behavior
$\eta < 0.05$	AT–AL–FZ–CZ	Perfectly Plastic
$0.05 < \eta < 10$	LCT–LT–BG–LC–LA–MLT–GT–LLT–GLT–TU–ALT	Strain Softening
$\eta > 10$	LL–GA–DIO–MO–CT	Brittle

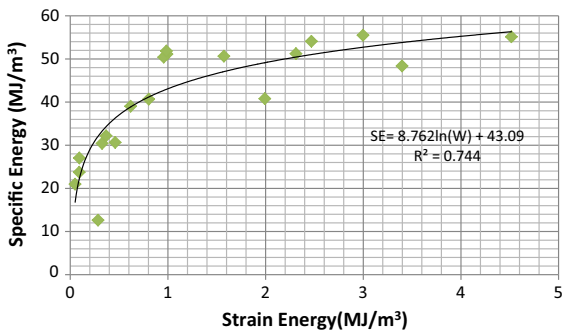


Fig. 11 The relationship between SE and W in very weak rock mass

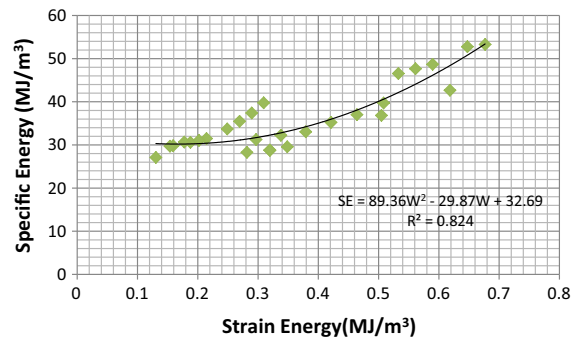


Fig. 13 The relationship between SE and W in good rock mass

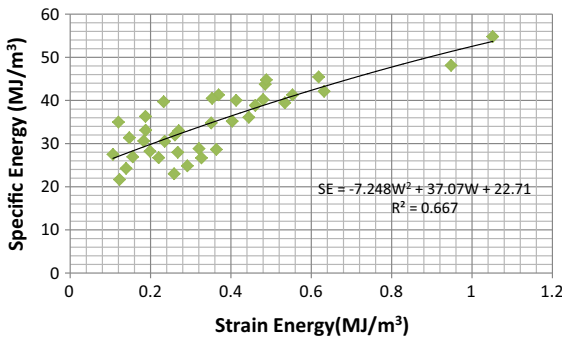


Fig. 12 The relationship between SE and W in favorable rock mass

It is cleared that the correlation coefficient between both parameters is low, so to increase it, the classification of rock mass according to the post failure behavior of rock mass is considered.

7 Estimation of SE with Considering the Post Failure Behavior of Rock Mass

As mentioned above there is two indexes to identify the post failure behavior of rock mass such as GSI and η ratio.

Table 7 The proposed equations to estimate the specific energy using the strain energy of rock mass based on the η

Eqs	η	Equations	Correlation coefficient
21	$\eta < 0.05$	$SE = 8.762 \ln(W) + 43.09$	$R^2 = 0.744$
22	$0.05 < \eta < 10$	$SE = -7.248W^2 + 37.07W + 22.71$	$R^2 = 0.667$
23	$\eta > 10$	$SE = 89.36W^2 - 29.87W + 32.69$	$R^2 = 0.824$

7.1 Estimation of SE with Considering GSI

The classification of rock mass formations with considering the Hoek's proposal is performed as shown in Table 4 and the statistical analysis to estimate SE using W is performed in any class.

As seen in Figs. 8, 9, and 10, the relation between SE and W in any class is direct, and the proposed equations (Eqs. 18–20) are shown in Table 5.

Where SE is the Specific Energy of TBM (MJ/m^3), and W is the Strain Energy of rock mass (MJ/m^3).

7.2 Estimation of SE with Considering η Ratio

As mentioned above η ratio is an index to distinguish post failure behavior of rock mass. So the classification of rock mass with considering it, is performed as shown in Table 6.

As seen in Figs. 11, 12, and 13, the relation between SE and W in any class is direct, and the proposed equations (Eqs. 21–23) are shown in Table 7.

8 Conclusion

In this study the post failure behavior of rock mass was assumed strain-softening behavior, because it can clarify both brittle and plastic behaviors. The results of model showed that increasing the quality of rock mass and decreasing the minimum principal stress can cause to increase the drop modulus because the rock mass behavior changes from elastic plastic to elastic brittle and drop modulus intended to infinite and it can be true inversely.

In this research, the prediction of the specific energy of TBM in Amir-Kabir Water Transferring Tunnel of Iran was performed using the strain energy of rock mass and it was predicted in three methods such as without considering the post peak behavior of rock mass, using the Geological Strength Index, and using the Drop to Deformation modulus ratio.

At the first method, estimation of SE using W without considering the post failure behavior of rock mass was carried out in Amir-Kabir Water Transferring Tunnel of Iran and a new logarithmic equation (Eq. 17) with $R^2 = 0.668$ was proposed to predict SE based on W . Because of high variation of SE and low correlation coefficient of the mentioned equation, the

classification of all data was performed with considering the post failure behavior of rock mass using GSI and η ratio.

At the next step, the classification of rock mass was conducted based on GSI and all data was classified in three classes such as plastic ($GSI < 25$), strain softening ($25 < GSI < 75$) and brittle ($GSI > 75$) and the best correspondence between SE and W was proposed using the statistical analysis in any class. Based on the statistical analysis, the maximum correlation between both parameters was achieved using Eqs. 18–20 to estimate the SE.

Finally using η ratio, the classification of data was performed in three classes such as $\eta < 0.05$, $0.05 \leq \eta < 10$ and $\eta \geq 10$. Also a statistical analysis was performed to estimate SE using W in any class. The results show that there were direct relations between both parameters and the best equations with highest correlation coefficient were achieved and Eqs. 21–23 were proposed to estimate SE.

The results showed a strong correlation between the mentioned parameters using the classification of post peak behavior of rock mass based on GSI, because the correlation coefficients of the mentioned equations (Eqs. 18–20) were higher than other equations. So it was proposed to estimate SE using W considering post failure behavior of rock mass based on GSI classification.

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