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A new strain-based criterion for evaluating tunnel stability

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Abstract. Strain-based criteria are known as a direct method in determining the stability of the geomechanical structures. In spite of the widely use of Sakurai critical strain criterion, it is so conservative to make use of them in rocks with initial plastic deformation on account of the considerable difference between the failure and critical strains. In this study, a new criterion has been developed on the basis of the failure strain to attain more reasonable results in determining the stability status of the tunnels excavated in the rocks mostly characterized by plastic-elastic/plastic behavior. Firstly, the stress-strain curve was obtained having conducted uniaxial compression strength tests on 91 samples of eight rock types. Then, the initial plastic deformation was omitted making use of axis translation technique and the criterion was presented allowing for the modified secant modulus and by use of the failure strain. The results depicted that the use of failure strain criterion in such rocks not only decreases the conservativeness of the critical strain criterion up to 42%, but also it determines the stability status of the tunnel more accurately.

Keywords: failure strain; critical strain; modified secant modulus; tunneling

1. Introduction

In view of the fact that the strain measurement is easier and more practical than stress and as it does not need to consider constitutive equations, strain-based criteria are mostly applied when evaluating the stability of the underground structures. In the stress-based criteria, the behavior equation between the stress and strain is assumed on the basis of the plane stress and Hooke’s law. Such relations rarely comply with the in-situ conditions and the mechanical properties of rocks (Li 1990). The first strain based criteria was proposed by St.Verrant in 1870 as the title maximum elastic strain theory in order to express the condition of brittle failure of metals (Nickolson 1985). The development of the strain-based criteria can be relevant to the progress in the fabrication of the monitoring devices in 1980s. Researches on this matter were commenced by Sakurai (1981), Stacey (1981) and continued by Li (1990), Aydan *et al.* (1993), Fujii *et al.* (1994, 1998), Li *et al.* (2000), Barla (2001), Li and Villaescusa (2005), Singh *et al.* (2007), Kwasniewski and Takahashi (2010), Yim *et al.* (2011), Zhang and Goh (2015), Wasseloo and Stacey (2016) and Cui *et al.* (2017). In accordance with the lab studies performed on several rock samples, Stacey (1981) stated that if the extension strain is more than the critical strain, the rock would start fracturing. He determined the critical strain causing dilation at stress level roughly equal to 30% of the final strength. Sakurai (1981) carried out lab studies on different rock samples and presented a model to evaluate the tunnel stability based on the critical strain. He stated that the tunnel would become unstable, if the maximum principal strain measured in the tunnel exceeds the rock critical strain. Fujii *et al.* (1994, 1998) conducted some tests such as uniaxial and triaxial compression strength and Brazilian tensile strength tests on sandstone, granodiorite, granite and andesite samples and stated that the rock would fracture when the minimum principal strain is equal to the critical tensile strain. During their lab investigations, Li *et al.* (2000) focused on the sandstone and concrete samples to recognize if the critical tensile strain criterion of Fujii and the critical strain of Sakurai affected by the ambient conditions (water content, confining pressure and strain rate variations) are correct. Barla (2001) used the critical strain for the identification and quantification of squeezing rock conditions. Li and Villaescusa (2005) had lab studies on different rocks to assess the relationship between the intact rock critical strain and rock mass and their relation to the Young's modulus and compression strength. Singh *et al.* (2007) had some tests on Lime silica bricks to assess the relationship between the critical strain, compression strength, tangential modulus of intact rock and rock mass amongst the jointed rocks. Kwasniewski and Takahashi (2010) conducted some uniaxial, triaxial and multi-axial compression strength tests on sandstone samples. They stated that the tests results do not approve the critical tensile strain model. Zhang and Goh (2015) regarded the critical strain as the tunnel strain associated with the occurrence of the tunnel collapse when the global factor of safety is 1.Wasseloo and Stacey (2016) reconsidered the correct interpretation of the results, proper conditions for using the extension strain criterion and make use of it as an index to determine the damage to the rock. In Table 1 the major strain-based criteria are listed. While some of them are considered as an academic records.

Amongst the strain-based criteria, the critical strain criterion of Sakurai is more comprehensive than others; on

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Table 1 The major strain-based criteria

Name of criterion	Proposer	Date
Maximum elastic Strain	St. Verrant	1870
Constant elastic energy of deformation	Beltrami	1885
Extension strain	Stacey	1981, 2016
Critical strain	Sakurai	1981, 1997
Critical tensile strain	Fujii *et al.*	1994, 1998

a way that the British Tunneling Society (2004) has accepted it as the criterion for measuring the tunnel stability and included it in their code. This criterion is so conservative in spite of its widely usage as also stated in the studies performed by Park *et al.* (2008), Kim and Kim (2009) & Park and Park (2014). The Sakurai criterion is conservative due to the following reasons:

• The difference between the critical strain of the intact rock and the rock mass

• The difference between the critical strain and failure strain, particularly in rocks with plastic-elastic/plastic behavior

The first reason corresponds with the scale effect; on a way that the criterion is determined on the basis of the lab tests results and is applied in the in-situ conditions. In such cases, the criterion would have a factor of safety, in view of the higher strain of the rock mass than the intact rock. According to Sakurai (1997), rock mass to intact rock critical strain ratio in fact demonstrating safety factor including in criterion. Because rock mass critical strain higher than intact rock and hence by using the criterion, which is based on intact rock results in order to rock mass stability evaluation includes this safety factor. On the other hand, critical strain of intact rock is smaller than failure strain which this ratio states another safety factor in Sakurai critical strain criterion. In accordance with Sakurai (1997), Kim and Kim (2009) & Park and Park (2014), the critical strain of the rock mass is equal to 3, 8 and 3.4 times of the critical strain of the intact rock. The second reason for the high conservativeness of Sakurai's criterion is due to that fact that it is based on the critical strain. In accordance with Sakurai (1981), Singh *et al.* (2007), Park *et al.* (2008), & Kim and Kim (2009), the failure strain of the rock mass is equal to 5, 1.5, 4 and 1.8 times of the critical strain, whilst this ratio is higher than the aforesaid values in rocks with initial plastic deformation. In order to lessen the degree of conservativeness of the critical strain criterion, this paper aims to represent a criterion based on the failure strain making use of the results of uniaxial compression strength tests carried out on 91 samples of eight rock types.

2. Critical strain criterion

Due to the inaccurateness of the back analysis method in realizing the displacements, Sakurai (1981) used Direct Strain Evaluation Technique (*DSET*) based on the critical strain so as to evaluate the stability of tunnels. In this method, the stability of the excavated space is directly determined based on the measured displacements. Critical

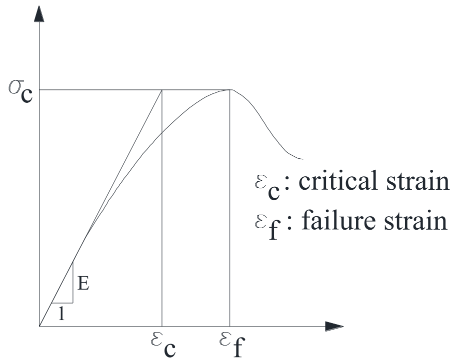


Fig. 1 Critical strain concept (Sakurai 1986)

strain, which is shown in Eq. (1) defined as the ratio of uniaxial compression strength to initial tangent modulus (Sakurai 1981). The critical and initial tangent modulus are shown in Fig. 1.

$$\varepsilon_c = \sigma_c / E_i \tag{1}$$

where ε_c , ε_f, σ_c *and* E_i *are* critical strain, failure strain, uniaxial compression strength and initial tangent modulus.

Sakurai (1982, 1986, 1997) proposed a relationship between critical strain and initial tangent modulus and uniaxial compression strength (Fig. 2). He presented three hazard warning levels so as to evaluate the tunnel stability in the construction stage on the basis of the results of several rock and soil critical strain lab tests. In this method, the tunnel stability is evaluated by determining the intersection point of the uniaxial compression strength parameters and the strain measured through monitoring of the tunnel. The lower and upper bounds are considered as stable and unstable zones and the middle part is considered as the basic level of the design. It should be noted that the critical strain is not much influenced by moisture content and temperature (Sakurai 1997). Sakurai (1986) defined the hazard warning levels (HWLs) on the basis of the relation between the critical strain-initial tangent modulus as Eqs. (2)-(4).

$$\log\varepsilon_c = -0.25\log E - 0.85 \quad \text{HWL III} \tag{2}$$

$$\log\varepsilon_c = -0.25\log E - 0.85 \quad \text{HWL II} \tag{3}$$

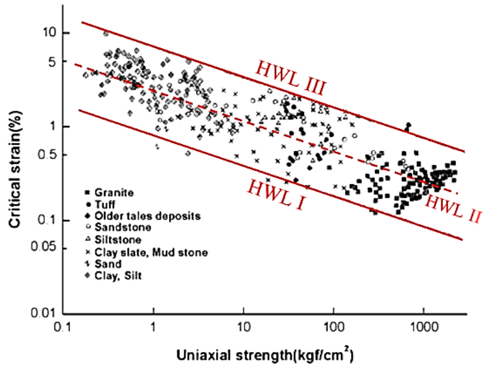
$$\log\varepsilon_c = -0.25\log E - 0.85 \quad \text{HWL I} \tag{4}$$

where E (*Kg/cm*^{2}) and ε_c indicate initial tangent modulus and critical strain.

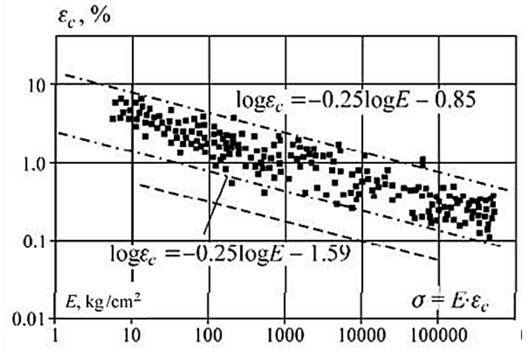
According to field observations and monitoring, Sakurai (1983) proposed that tunnel strain levels in excess of approximately 1% are associated with the onset of tunnel instability and with difficulties in providing adequate support. Field observations by Chern *et al.* (1998) and Hoek (2001), confirm Sakurai’s idea.

3. Initial and modified plastic deformation

Deere and Miller (1966) presented six typical behavior curves based on the results of the uniaxial compression



(a) Based on critical strain- uniaxial compression strength



(b) Based on critical strain-initial tangent modulus

Fig. 2 Sakurai's bounds (Sakurai 1982, 1986)

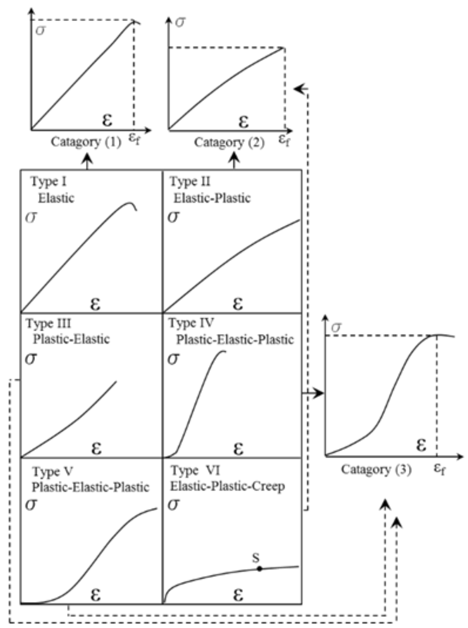


Fig. 3 Stress-strain typical curves in uniaxial compression strength test and classifications according to the similarity of the curve (Daraei and Zare 2018)

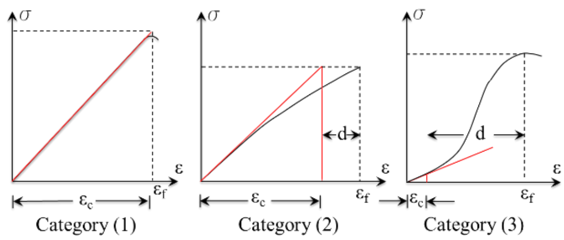


Fig. 4 Critical strain in various category (Daraei and Zare 2018)

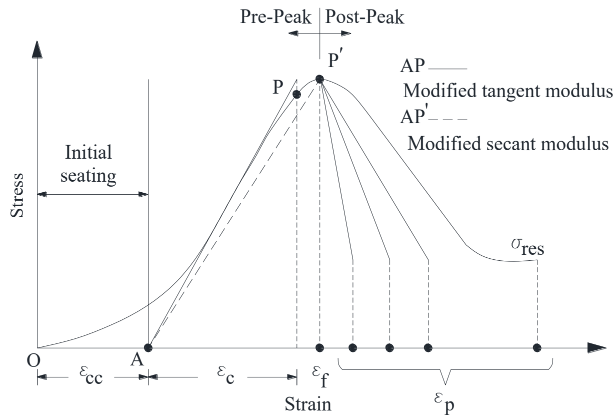


Fig. 5 Axis translation technique

strength tests carried out on 28 samples taken from 13 rock types, (Fig. 2). Taking into account the three behaviors of elastic, elastic-plastic and plastic-elastic/plastic, it is possible to define three similar categories comprising the curve of type *I* rocks, the category including the rocks with behavior curve types *II* and *VI* and the category consisting the rocks with behavior curve types *III*, *IV* and *V* based on the behavioral stages. It should be considered that the curve type *VI* to point *S* in Fig. 3 (start of creeping) is similar to the curve *II* from behavior point of view.

The rocks of category 1, which are mostly including of high strength rocks, have linear elastic behavior. Such rocks have a high brittleness index; on a way that at the failure point, the energy is suddenly released while loading. The rocks of category 2 have elastic behavior at the beginning and in the middle sections. They have gradual plastic behavior after the yield point. This type of rocks has elastic-plastic behavior and has less strength than the ones classified as category 1. The category 3 rocks have plastic-elastic/plastic behavior. There might be no plastic section after the yield point in some rocks. The presence of upward initial concavity at the beginning of the stress-strain curve is one of the outstanding features of these rocks. Such a behavior is mainly observed in weak rocks or high porosities rocks.

3.1 Modification of initial plastic deformation

Practically, critical strain is determined by drawing the initial tangent modulus on the stress - strain curve (Fig. 1). The tangent modulus, critical and failure strains (ε_c, ε_f) in three behavior curves are presented as Fig. 4 (Daraei and Zare 2018).

In general, making use of the critical strain in the behavior categories of (1) and (2) results in a reasonable factor of safety on account of the inconsiderable difference between the critical and failure strains (d). In regard to the rocks of category (3) with upward concavity at the beginning of the curve, if the initial tangent modulus is drawn on the basis of ASTM (2004) and ASCE (1996)

Standards, the considerable difference between the critical and failure strains would induce high safety factor in the Sakurai criterion. The initial concavity existing in the behavior curve category (3) occurs at the beginning of the sample loading stage and in response to the closure of the micro-cracks and the initial seating of the sample under the test apparatus. At this stage, the samples have nonlinear behavior and their presence and propagation depend on the density and geometry of the micro-cracks (Peng *et al.* 2015). According to Santi *et al.* (2000), such cracks have occurred during drilling and sampling and they do not belong to the rock natural features. Therefore, axis translation technique is used in order to modify the initial concavity of such curves as shown in Fig. 5. In this method, the origin is transferred to the intersection point of the middle straight line portion of the stress-strain curve and the strains axis (point *A*). It should be noted that the initial plastic deformation part of the graph (*OA*) is eliminated.

By modifying the behavior curve as stated above, the initial tangential modulus of the critical strain criterion would be changed to the modified tangential modulus (Line *AP*). Although such modifications lessen the conservativeness of the criterion to some extent, in most of the rocks (the rocks without sudden fracture–Class *I* of the classification of Wawersik and Fairhurst 1970), the load-bearing ability of the rock would not disappear suddenly by increasing the strain amount even after fracturing. Several studies depict that the post-peak failure behavior has strain softening stage and residual strength (Hajiabdolmajid and Kaiser 2003, Bogusz and Bukowska 2015, Jianxin *et al.* 2012, Wang *et al.* 2014, Abdullah and Amin 2008). In accordance with Cai *et al.* (2007), in most of rocks with strain softening behavior mechanism, the residual strength determines the plastic deformation existing in the general deformation. It is obvious that the rock is ruptured under such conditions relying upon the amount of brittleness index and stress path in a strain higher (ε_p) than the failure strain (ε_f). So, even by the stability criteria based on the failure strain (ε_f), in addition to the safety factor resulted from the presence of the strain after the post-peak failure, the difference between the intact rock strain–rock mass would cover the uncertainties under consideration. Therefore, the stability criterion based on the failure strain (instead of the critical strain) is more logic and has less safety factor. It also plays an important role in preventing from overdesigning of the tunnel supporting system. According to Fig. 5, having modified the initial concavity, the modulus represents failure strain and modified secant modulus (*AP'* line in Fig. 5). The modified secant modulus covers a wider range of strains and stresses amongst the deformation moduli comprising the initial, secant and tangential moduli that is of vital importance in underground structures. Besides, the modified secant modulus has less variation factor (percentage of the ratio of standard deviation to the average value) and more repeatability (the possibility of obtaining similar results during different tests under similar conditions) comparing to other moduli (Santi *et al.* 2000).

4. Test Method and stages

4.1 Preparation of samples

In order to perform uniaxial compression strength tests, a number of samples were obtained from 11 different sites

Table 2 Lithology and number of prepared samples

Location	Lithology	No. of samples	Grade	Term
1	Siltstone	16	R2-R3-R4	Weak to Strong
	Conglomerate 1	6	R4	Strong
2	Sandstone	13	R3-R4	Medium to Strong
3	Limestone 1	13	R3-R4	Medium to Strong
4	Conglomerate 2	7	R2-R3	Weak to Medium
5	Slate	2	R2	Weak
6	Phyllite	6	R2	Weak
7	Limestone 2	1	R3	Medium
8	Conglomerate 3	2	R2	Weak
9	Shale	2	R2	Weak
10	Schist	3	R2	Weak
11	Limestone 3	20	R3-R4	Medium to Strong

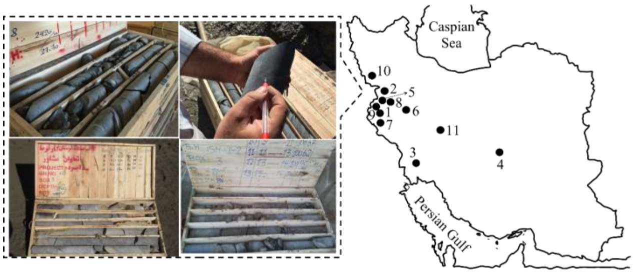


Fig. 6 Location and the cylindrical cores obtained from different projects

(a) Clip and chain-type extensometers (b) Dial gauges

Fig. 7 Installation of strain gauges on samples

Table 3 Gauges reading intervals in indirect method

Axial Load (*kN*)	Recording Deformation Intervals
Up to 30	Each 5 *kN*
30-100	Each 10 *kN*
100-200	Each 20 *kN*

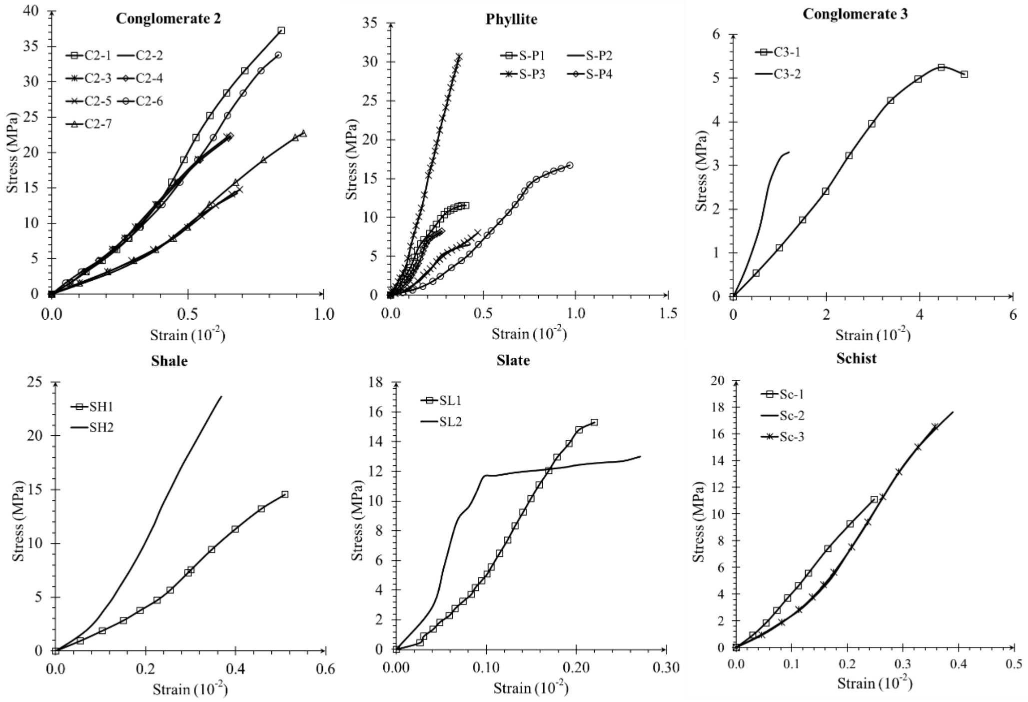


Fig. 8 The samples with weak to medium strength

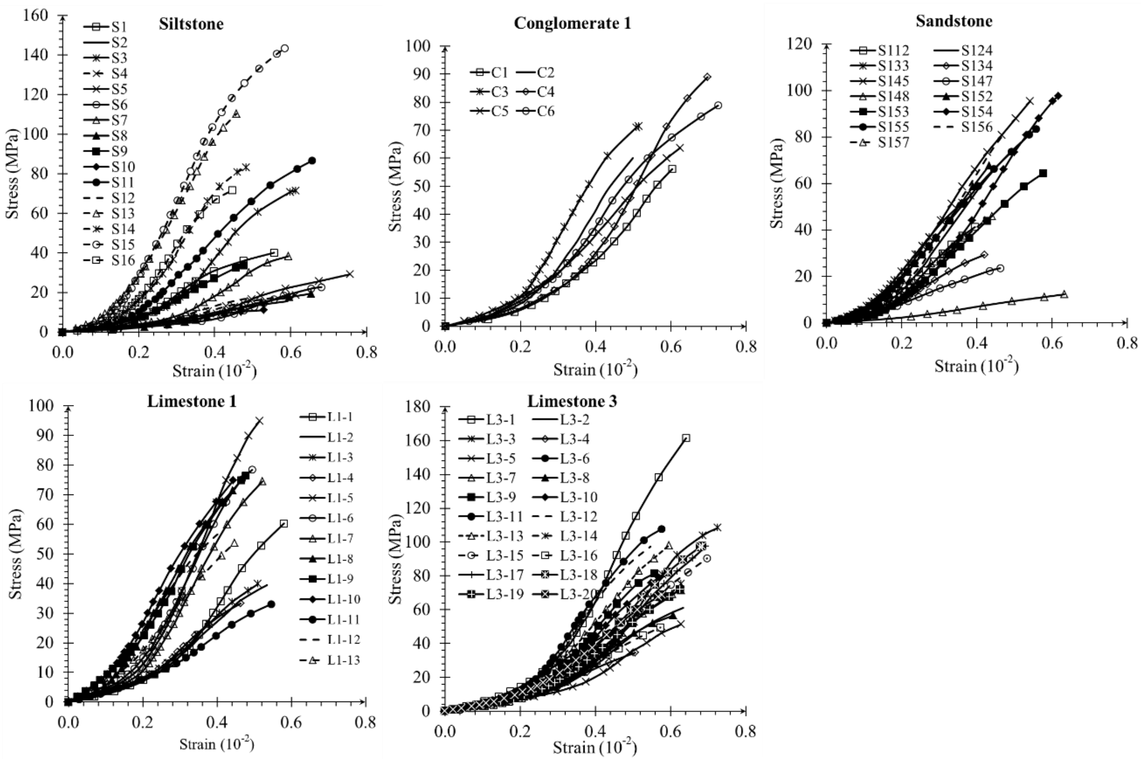


Fig. 9 The samples with strong strength

existing in Iran (Fig. 6). Samples were obtained either as *NX*-size cylindrical cores from the field, or as block quarry samples which were then core-drilled in the laboratory. In both cases, the samples were in an air-dried state. Taking into account the aforementioned conditions, 91 samples (Table 2) were provided out of eight different lithology including conglomerate, sandstone, limestone, schist, phyllite, slate, shale and siltstone.

So as to attain the stress-strain curves with initial plastic deformation, either (*i*) low strength or (*ii*) high porosity was taken into consideration while selecting the samples. In spite of the fact that the priority was first given to the selection of low-strength samples, due to the difficulties in taking samples and controlling the loading rate during the test, most of the samples were taken from the porous rocks with strengths of more than 30 MPa.

Given the above, 31 samples were taken from low-strength rocks and the rest were selected from the rocks with high porosity. Regarding easy drilling and take sound samples, 74 mm diameter bits have been use. The procedure recommended in ISRM (1979) for sample preparation was followed. According to samples quality, strain values were measured applying either direct or indirect methods. The loading rate was considered 0.5-1 MPa/s. Based on samples quality and strength, two types of strain measurement are applied i.e., direct and indirect. The strain value in samples with desirable quality and free of weak layers were measured based on direct method by using clip and chain-type extensometers as shown Fig. 7(a). In low quality specimens due to the high sensitivity of the clip and chain-type extensometers as well as the inhomogeneity of the samples, the strains were determined indirectly. In this method, deformations were initially measured by use of the dial gauge installed on the sample as shown in Fig. 7(b).

Attached dial gauges measure axial deformation in certain intervals of axial load as Table 3. Thereafter, axial strain of sample calculated from ratio between the changed and its initial length. For instance, the deformation of sample up to 50 kN axial load, were recorded in 5, 10, 15, 20, 25, 30, 40 and 50 kN, respectively (Daraei and Zare 2018).

As seen in Figs. 8 and 9, the extension of the crack closure area in the curves related to the porous rocks is more than the ones in the low-strength rocks; on a way that it can be said that the parameter of porosity has more effect on the formation of initial plastic deformation in the behavior curve than the strength. By increasing the strength and stiffness of the sample, the behavior curve tends to linear elastic behavior. In addition, any increase in the brittleness of the samples would give rise to less failure and critical strains. Hence, it can be said that the failure and critical strains value depends completely on the mechanical behavior of the samples. Having compared the behavioral categories shown in Fig. 4 and the curves resulted from tests, it was realized that category 3 behavior is mainly related to the weak rocks, category 2 behavior is mainly related to the weak to medium strength rocks (25-50 MPa) and category 1 behavior is mainly related to the moderate to high strength rocks (UCS > 50 MPa).

4.2 Determination of failure strain and modified secant modulus

In stress-strain curves, the deformation corresponding to the rock maximum uniaxial compression strength was considered as the failure strain. In addition, after modifying the initial concavity of the curves, the modified secant modulus was determined via the ratio of the maximum compression strength to the failure strain.

4.3 Determination of the ratio of failure strain to critical strain

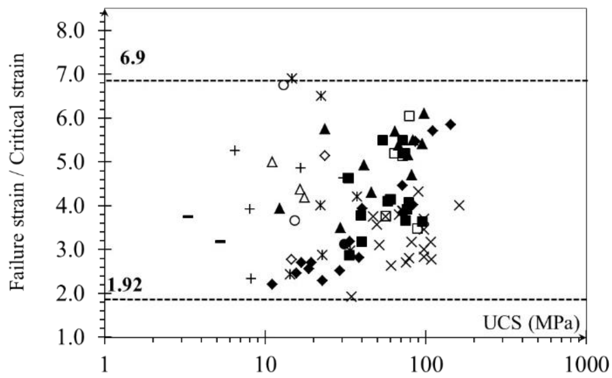
The ratio of failure to critical strains can be considered as a parameter to indicate the conservativeness existing in the criterion. In general, Sakurai (1981) presented the relation between the critical and failure strains as Eq. (5).

$$\varepsilon_f = \frac{\varepsilon_c}{1 - R_f} \tag{5}$$

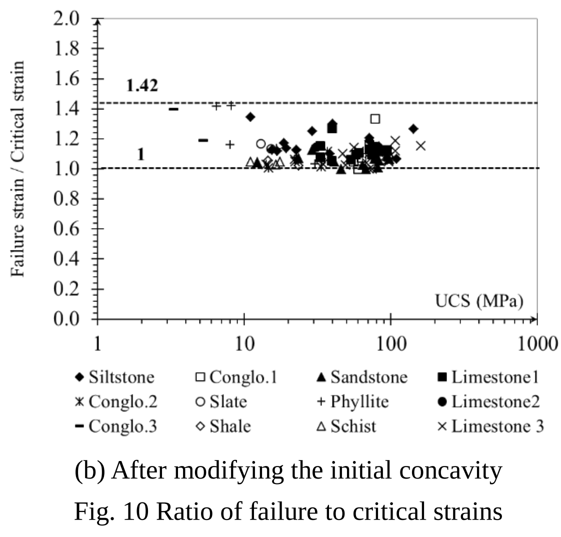
where ε_f: failure strain, ε_c : critical strain and R_f is failure strength parameter.

Based on Sakurai (1981), the R_f domain located between 0.05 and 0.8. In this case, the failure strain of the intact rock would be 1 to 5 times of the critical strain. Cai (2011) stated the wide range of R_f proposed by Sakurai, requires more discussions. He said R_f located in most of hard rocks between 0.1 and 0.3. In this case, the failure strain value in hard rocks would be 1.1 to 1.42 times of the critical strain. According to Singh *et al.* (2007) and Kim and Kim (2009), this ratio is 1.5 and 1.8. As the selected samples are classified as category 3, the ratio of failure to critical strains can be considered as two parts of before and after modifying the initial plastic deformation (Fig. 10). In case of not modifying the initial concavity, the above ratio in the tested samples would be 1.92 to 6.9 and in case of modifying the initial concavity, the above ratio would be 1 to 1.42.

Table 4 shows the results of the uniaxial compression strength, failure strain, modified secant modulus and the failure to critical strains ratio.



(a) Before modifying the initial concavity



(b) After modifying the initial concavity

Fig. 10 Ratio of failure to critical strains

Table 4 Parameters determined by the uniaxial compression strength test

Lithology	UCS (*MPa*)		Failure strain (10^{-2})		Mod E_{sec} (*GPa*)		$\varepsilon_f / \varepsilon c$ in intact rock: Before correction		$\varepsilon_f / \varepsilon c$ in intact rock: After correction	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Siltstone	11	143	0.28	0.48	2.35	34.4	2.2	5.85	1.07	1.34
Conglomerate 1	56	89	0.29	0.52	14.4	25.4	3.49	6.04	1	1.33
Sandstone	12.3	97.6	0.26	0.51	2.4	26.4	3.49	6.1	1	1.13
Limestone 1	33	94.8	0.3	0.39	8.4	29.6	2.88	5.5	1.06	1.27
Conglomerate 2	14.2	37.2	0.52	0.69	2.7	5.6	2.43	6.9	1.01	1.11
Slate	15.3	13	0.17	0.23	9	5.6	3.67	6.75	1.13	1.17
Phyllite	6.4	30	0.21	0.77	1.9	9.2	2.33	5.25	1.03	1.42
Limestone 2	31		0.79		3.9		3.13		1.14	
Conglomerate 3	3.3	5.2	0.95	4	0.12	0.34	3.18	3.75	1.19	1.4
Shale	14.5	23.6	0.29	0.39	3.7	8.1	2.78	5.14	1.02	1.05
Schist	11.1	17.6	0.22	0.29	5	6.3	4.19	5	1.03	1.05
Limestone 3	34.6	161	0.33	0.44	10.1	42.5	1.92	4.31	1	1.19

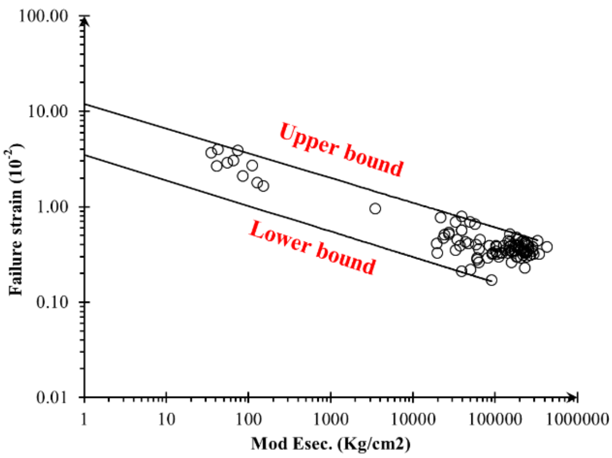


Fig. 11 Relation between failure strains and modified secant modulus

According to Fig. 10 and Table 4, the ratio of the failure to critical strains in rocks with initial plastic deformation can be discussed in two cases. If initial tangent modulus is used, the ratio of failure strain to the critical strain would be 1.92 to 6.9. This ratio indicates that making use of the initial tangent modulus in rocks with initial plastic deformation would considerably increase the criterion conservation. This ratio decreases to maximum 1.42 by modifying the initial concavity and making use of the initial tangent modulus in defining the critical strain. In this case, even with the above modifications, there are 42% of safety factor on account of the difference between the critical and failure strains in the modified Sakurai criterion. In view of the sufficiency of the safety factor required for compensating the uncertainties in relation to the rock mass (elaborated in Section 3.2), the omission of this section by use of the failure strain and modified secant modulus would decrease a portion of the excessive conservations exiting in the modified Sakurai criterion.

4.4 Proposed criterion

The relation between failure strain values and the modified secant modulus was drawn in a logarithmic graph. As per Fig. 11, the failure strain values are situated in between two parallel levels; on a way that their relation can be expressed in the framework of Eqs. (6) and (7).

$$\log \varepsilon_f = -0.25 \log Mod.E_{sec} - 0.92 \quad \text{Upper bound} \tag{6}$$

$$\log \varepsilon_f = -0.27 \log Mod.E_{sec} - 1.45 \quad \text{Lower bound} \tag{7}$$

where $Mod.E_{sec}$ (Kg/cm^2) and ε_f indicate modified secant modulus and failure strain.

The lines depict the maximum and minimum failure strains of the rocks. The rock enters the failure stage when strains are more than the upper bound. In addition, the rock is stable when the strain value is less than the lower bound. The data of some monitoring stations of tunneling projects existing in Iran, which are characterized by collapsing, stable and critical conditions, have been utilized in order to validate the failure strain criterion levels (Table 5). The accuracy of the graph was verified by plotting the measured strains of these three types of tunnels in the proposed levels according to Fig. 12 and comparing the resulted conditions to the real conditions observed in the tunnels.

In general, three following scenarios can be foreseen for the tunnel by plotting the strains resulted from monitoring:

• The measured strain is more than the upper bound; in this case, it is very likely to have collapse in the tunnel

• The measured strain is less than the upper bound; in this case, the tunnel is stable

• The measured strain is moderate; in this case, the tunnel is under critical conditions

It is notable that the condition between the stable and unstable is called critical condition. In critical conditions, it would be necessary to install or increase the supporting system to mitigate deformation. When the deformation rate does not become fix, even by taking supporting systems, the tunnel would absolutely have collapse allowing for the third creeping stage of the surrounding rocks. This would likely happen when the strain values reach to the upper bound. In this case, the tunnel strain will exponentially increase, and the stability control of the tunnel require special consideration including the use of forepoling and face reinforcement and/or multi-sectional excavation methods such as central diaphragm and side wall drifts.

4.5 Scale effect; using proposed criterion in field

Due to difference in sample volume in lab and field, their rock mechanical parameters are different. Such an effect which studied by various researchers in terms of scale effect is due to the presence of discontinuities and different environmental conditions in both status (Heuze 1980, Cuisiat and Haimson 1992, Zhang *et al.* 2011). The question may now arise as to how to extend the proposed criterion obtained from lab tests on small specimens to large-scale in-situ soils and rocks. According to Sakurai (1997), the strains obtained from lab tests may be almost the same as that for in-situ soil masses. But, in rock masses there is no

Table 5 Monitoring results in some tunnels in Iran

Project	UCS (*MPa*)	E (*GPa*)	Measured by	Failure Strain (10^{-2})	Real Condition	Location
1	39	12	Extensometer	0.989	Collapsed	
	48	17.4	Extensometer	0.76	Collapsed	
	49	17.7	Extensometer	0.585	Collapsed	
	40	19.3	Extensometer	1.36	Collapsed	
	39	32.3	Extensometer	1.36	Collapsed	
2	25	14	Extensometer	0.48	Collapsed	
3	50	8	Extensometer	0.05	Stable	
	35	12.5	Extensometer	0.06	Stable	
4	69	15.9	Extensometer	0.062	Stable	
	113	21.9	Extensometer	0.008	Stable	
5	60	7.5	Extensometer	0.044	Stable	
6	--	6.5	Extensometer	0.289	Critical	
	--	15	Extensometer	0.15	Stable	
	--	6.5	Extensometer	0.211	Critical	
7	33	9.9	Extensometer	0.01	Stable	
8	< 1	0.08	Target point	0.84	Critical	
		0.0648	Target point	0.1	Stable	
			Target point	0.26	Stable	
			Target point	0.1	Stable	
			Target point	0.2	Stable	
9	30	11.9	Extensometer	0.03	Stable	
	88	23.5	Extensometer	0.02	Stable	

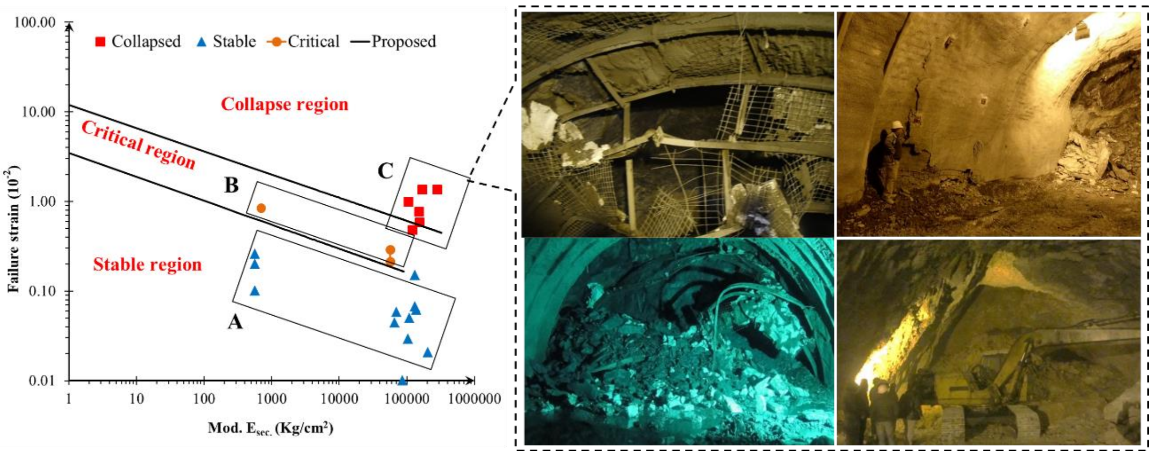


Fig. 12 Validating of the failure strain criterion levels

situation present. Sakurai (1997) by using Eq. (8) linked intact and rock mass critical strains together. He presented reduction factor (*m/n*) by lab tests. The (*m/n)* ratio depictsthe decreasing effect of the uniaxial compression strength and deformation modulus of the rock mass proportionate to the intact rock. Sakurai (1997) stated rock mass critical strain 1 to 3 times higher than intact rock. In this study in order to determining the relationship between intact and rock mass failure strains, the results of lab and unstable monitoring stations in two tunnels were applied.

So, determining four parameters as follow are necessary:

• Failure strain and modified secant modulus of intact rock

• Failure strain of rock mass

• Deformation modulus of rock mass

Failure strain and modified secant modulus of intact rock determine by using lab tests. By application of convergency monitoring graphs, strains which there were crack in shotcrete, falling rock bolts and/or steel ribs

Table 6 Ratio of rock mass to intact rock failure strains

Tunnel	$^*\varepsilon_{f\text{-}m}$ (%)	*GSI*	E_m (GPa)	Lab results: No. of Samples	$^{**}\varepsilon_{f\text{-}i}$ (%)	*Mod-E*$_{sec.}$ (GPa)	σ_{ci} (MPa)	$\varepsilon_{f\text{-}m}/\varepsilon_{f\text{-}i}$
Ilam-Mehran	0.47	15	0.44	3	0.2	5.04	11.1	2.35
	0.42	15	0.56		0.28	6.07	17.6	1.5
	0.56	15	0.54		0.26	6.35	16.5	2.15
Shibli	0.6	30	1.2	2	0.38	3.7	14.5	1.58
	0.56	30	1.2					1.47
	0.35	25	1.1		0.29	8.1	23.6	1.21
	0.69	25	1.1					2.38
	0.65	25	1.1					2.24

$^*\varepsilon_{f\text{-}m}$: rock mass failure strain; $^{**}\varepsilon_{f\text{-}i}$: intact rock failure strain

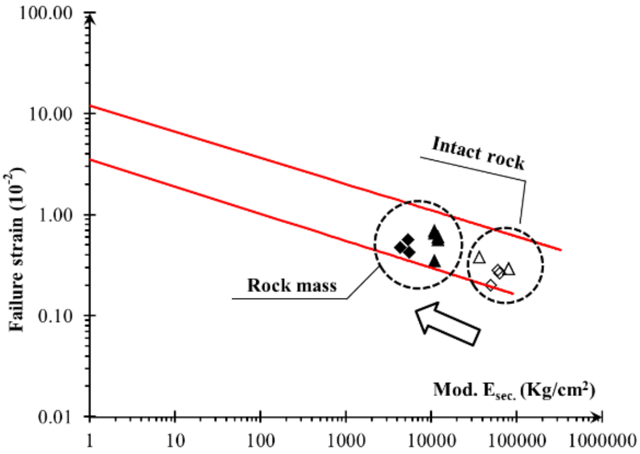


Fig. 13 Relationship between failure strains of intact rocks and in-situ rock masses

buckling occurred, considered as rock mass failure strain. Then by using Eq. (9) deformation modulus of rock mass was calculated. According to Hoek and Diederichs (2006) studies, deformation modulus of rock mass which obtained by using Eq. (9), is the secant modulus. The results are shown in Table 6. By plotting the results according to Fig. 13, it is observed that scale effect has no influence in general condition presented by proposed criterion and it could applied in field. The main point is average of rock mass to intact rock failure strain ratio which is equal to 1.9, i.e. by using intact rock failure strain, safety factor as 1.9 is include in proposed criterion.

$$\varepsilon_{0R} = \left(\frac{m}{n}\right)\varepsilon_0 \tag{8}$$

$$E_{rm} = E_i\left(0.02 + \frac{1-D/2}{1+e^{(60+15D-GSI/11)}}\right) \tag{9}$$

where *m*, *n* are reduction coefficient for uniaxial compression strength, reduction coefficient for deformation modulus ($0<m, n\leq 1$), ε_{0R}, ε_0 indicate rock mass and intact rock critical strains, E_{rm} and E_i are deformation modulus of rock mass and intact rock, *GSI* is Geological strength index and *D* indicate disturbance factor.

5. Discussions

The graphs shown in Fig. 11 depict that the increase in the modified secant modulus would cause decrease in the failure strain and allowable deformations. In order to compare Sakurai and proposed criteria, stability evaluation levels of them are shown in Fig. 14. As seen, the variation domain of the failure strain is less than the critical strain that is due to the lower modified secant modulus than the tangent modulus and less error in determining the failure strain. The less error can be ascribed to the presence of a certain point in the stress-strain curve so as to precisely determine the failure strain. Because, the failure strain can be determined with a high accuracy via the horizontal axis of the stress-strain curve by linear connection between the origin and the peak strength point, whilst there is no point or condition indicating the occurrence of critical strain in this curve. In both cases, the critical strain should be determined by drawing a tangent line on the stress - strain curve. So, the human error involved in determining the failure strain would be less than the critical strain. On the other hand, the failure strain would also be affected in view of the influence of some parameters as the water content and temperature on the rock strength. Nevertheless, in accordance with the investigations carried out by Daraei and Zare (2018) and Sakurai (1997), the effect of the water content and temperature on the critical strain is negligible. Therefore, this factor can also be involved in the increase in the accuracy and more real conditions for the proposed criterion.

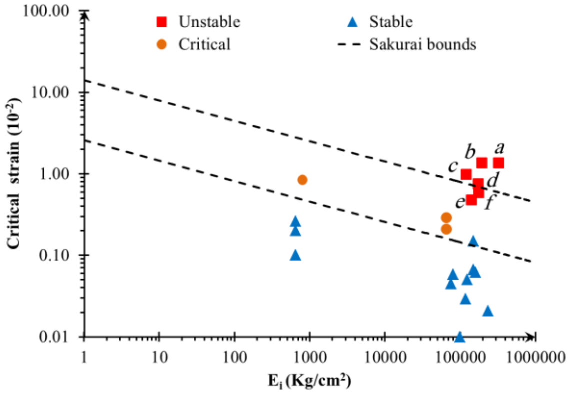
In addition to the above difference, the proposed criterion has not the same doubts as the ones in the Sakurai criterion and determines the tunnel status more accurately and explicitly. This issue was assessed in more details by comparing the practical results with the output of proposed and Sakurai strain levels (Table 7). In real status, all stations *a*, *b*, *c*, *d*, *e* and *f* (Fig. 14) have had collapses in their walls, crown or both of them.

The following results would be obtained by plotting the measured strain values of these stations on the above common graph:

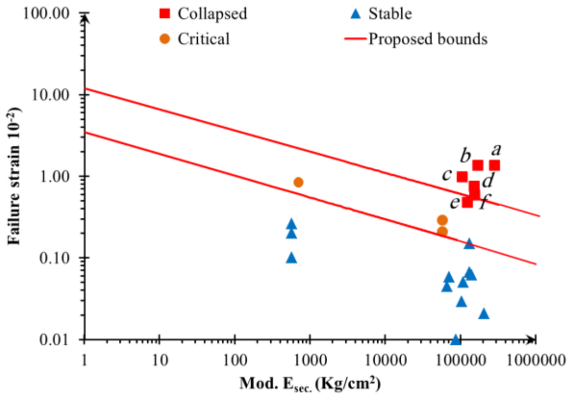
• In accordance with the Sakurai criterion, the stations *e* and *f* have critical conditions and the stations *a*, *b*, *c* and d are unstable.

• In accordance with the proposed criterion, the stations *a*, *b*, *c*, *d* and *f* have collapses and the station *e* has critical conditions. But, according to the earlier descriptions, provided that the strain measured in this criterion is moderate with an upper bound trend, the surrounding host rocks would have stage 3 of creeping. Hence, it is very likely to have collapse therein. Accordingly, Station *e* can also be considered as a station with collapsing conditions.

Taking into consideration the results of Table 7, seeing that the status determined by the proposed criterion are more compatible with the real status of the stations, as it can be said that the proposed criterion determines the tunnel conditions more precisely. In addition, the presence of doubt in the Sakurai criterion can be inferred from the keyword of "*Unstable*" and the wide range of such conditions in this criterion. According to Sakurai (1997), the unstable status of the tunnel include difficulties in



(a) Sakurai criterion



(b) Proposed criterion

Fig. 14 Stability evaluation levels

Table 7 Comparison between the results of Sakurai and proposed criteria

Station	Real status	Condition based on	
		Sakurai criterion	Proposed criterion
a	Collapsed	Unstable	Collapsed
b	Collapsed	Unstable	Collapsed
c	Collapsed	Unstable	Collapsed
d	Collapsed	Unstable	Collapsed
e	Collapsed	Critical	Collapsed
f	Collapsed	Critical	Critical

maintaining tunnel face, failure or cracking in shotcrete, buckling of steel ribs, breakage of rock bolts, fall-in of roof and swelling at invert. Sakurai said that "*Even if the measurement values are still smaller than the hazard warning level, however, engineers should always pay attention to what happens after the lapse of a certain period of time*". Taking into consideration such a wide range of instability, in case of running into a strain more than the upper bound in the Sakurai criterion, the project experts concern on what case would happen amongst the aforesaid cases would have direct effect on the increase in duration and cost of the project. Because, under such concerned conditions, particularly in underground structures and ambiguous type of instability, decision would normally be made on increasing the factor of safety by increasing the supporting system or decreasing the excavation step or both of them. But, in the proposed criterion, the upper bound has been considered as the "*collapse area*" by eliminating some conservations existing in Sakurai criterion making use of the definition of failure strain and the modified secant modulus. In this case, the closeness of the strain to the upper bound only indicates the "*collapse condition*". Such an explicit assertion not only removes ambiguities and doubts relevant to the type of a probable instability, but also plays an important role in better decisions to be made by the engineers and more suitable plans to be drawn up for implementing the tunnel secondary supporting system in time.

6. Conclusions

Initial plastic deformation in the stress - strain curve gives rise to a considerable difference in the failure strain rather than the critical strain up to 6.9 times. This ratio decreases to 1.42 by modifying the initial plastic deformation by use of axis translation technique and modified tangent modulus. Taking into consideration the compensation of the uncertainties existing in the rock mass by the difference in the strain of the rock mass-intact rock and the capability of the rock load bearing after the failure point, making use of the failure strain and the modified secant modulus in defining the proposed criterion may omit a portion of the high factor of safety existing in the Sakurai criterion up to 42%. In addition, results show that the tunnel conditions are determined more accurately and explicitly by making use of the proposed criterion. The main reasons for the increase in the accuracy can be due to the removal of ambiguities existing in the type of the probable instability, more effects of the environmental conditions on the failure strain and having a different definition on the unstable condition. In engineering practice, with using proposed failure strain criterion, the factor of safety from 1 to 1.9 is automatically included, because the failure strain of in-situ rock masses is always 1 to 1.9 times greater than that of intact rocks.

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