Assessment of Hard Rock Tunnel Stability: A Note on the Influence of Post-peak Strength Degradation

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ABSTRACT: Tunnelling depth is increasing rapidly in Taiwan. The effect of brittle failure on hard rock tunnelling is, however, rarely studied. For this, in this paper, a study is carried out on the post-peak form of the Hoek-Brown failure criterion and the strength loss experiment, whereby a relationship between strength loss parameter and confining stress is established. Subsequently, a numerical analysis model, so-called strength degradation model, is proposed and applied to predict the impact of the post-peak strength degradation on an actual tunnel. The analysis shows that the effect of the post-peak strength degradation on excavation deformation is becoming more and more pronounced with increasing depth. Severe excavation deformation may endanger tunnel stability during construction in deep overburden. The strength degradation beyond brittle failure shall play an exceptionally important role in the stability of deep tunnelling.

KEYWORDS: Brittle failure, Numerical analysis, Post-peak strength degradation, Tunnel stability

1. INTRODUCTION

Large amount of tunnels were constructed in mountainous area of Taiwan to establish the roundabout traffic system. Abundant tunnelling experiences are, therefore, accumulated. Many investigations on tunnelling behaviour and construction measures have been proposed especially for squeezing weak rock (e.g. Chern *et al*., 1998; Jeng *et al*., 2002; Wang & Huang, 2011). However, the issue of hard rock tunnelling was seldom studied in Taiwan in the past because the excavation in hard rock is normally stable at shallow depth except for wedge failure. It is often found that only few supports, even without any support, were installed for underground mining cavern and for highway tunnel in the metamorphic hard rock of Taiwan. However, the tunnelling depth is progressively increasing in Taiwan, e.g. the maximum overburden of 1188 m for Chungren Tunnel in the Suhua Highway Improvement Project is being constructed.

At depths where high in-situ stresses and the stress concentration near the excavation boundary might result brittle failure occurrence due to hard rock overstressed. The brittle failure of hard rock is different from the squeezing failure of weak rock. Figure 1 shows the typical features of brittle failure for hard rock, that is, relatively small deformation occurs prior to failure during loading, and under continued loading, the integrity of rock would be destroyed with degrading rock strength. In terms of weak rock, failure would occur beyond large deformation and the post-peak strength would only slightly degrade because the integrity and continuity of rock material are generally maintained, as shown in Figure 2. The well-known ground response curve (Figure 3) clearly portrays that tunnel displacements and required supported pressure are closely related to the constitutive model of rock mass. There is a significant difference in tunnel displacement for different constitutive model of rock mass (elastic-brittle, elastic-plastic or strain-softening) even the same support pressure and the same install timing adopted. It depends on the intensity of post-peak strength degradation during the brittle failure process. However, the simplified elastic-ideally plastic model is basically not suitable for hard rock excavation. The elastic-brittle or the strain-softening model would be more appropriate in the simulation of post-peak strength degradation of hard rock.

The impact of brittle failure on underground excavation stability is normally focused on violent rock burst. Rock burst is recognized as a dynamic instability phenomenon of the rock mass surrounding a tunnel in high geostatic stress and caused by the violent release of strain energy stored in intact hard rock. Rock burst occurs during underground excavation in the form of stripe of rock slices or rock fall or throwing of rock fragments, sometimes accompanied by crack sound. Because it occurs suddenly and intensely, rock burst usually causes injury including death to workers, damage to equipment, etc. However, rock burst is normally occurred in the condition of intact rock strength reaches to 200-300 MPa according to the case histories of Gotthard Base Tunnel in Switzerland, Laerdal Tunnel in Norway, Jinping II Diversion Tunnel and Qinling Tunnel in China. The uniaxial compressive strength of hard rock in Taiwan is mainly in the range of 100-150 MPa. Instead of violent rock burst, non-violent spalling or slabbing failure (as depicted in Figure 4) would be the significant problem for deep hard rock tunnelling in Taiwan.

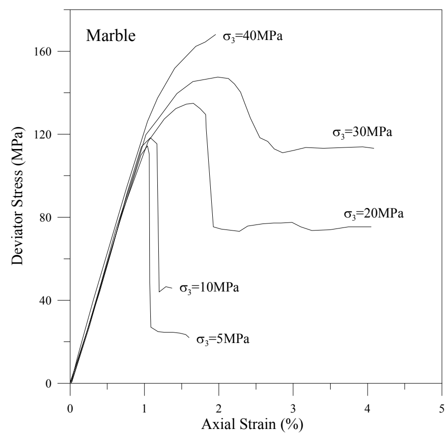

Figure 1 Triaxial compressive test results of Tailuko marble in eastern Taiwan (Lu *et al.,* 1993)

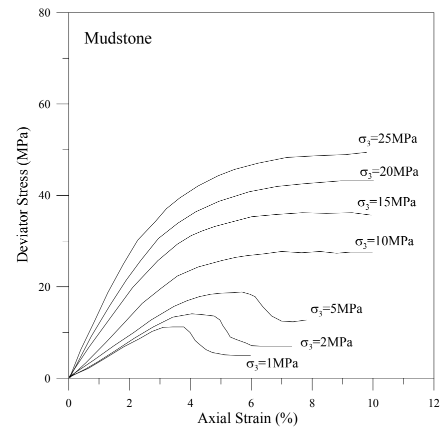

Figure 2 Triaxial compressive test results of Kanhsialiao mudstone in southern Taiwan (Tsai *et al.,* 2009)

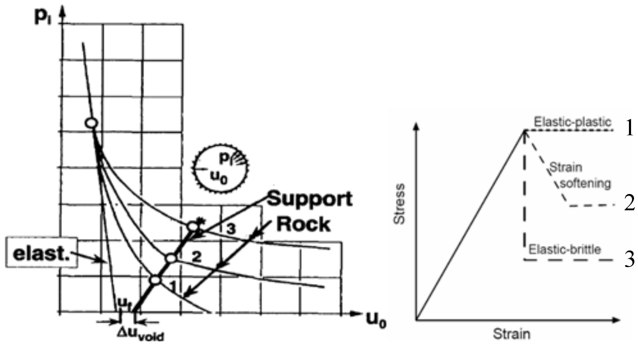

Figure 3 Correlation between ground response curve and support pressure for different stress-strain characteristics of rocks

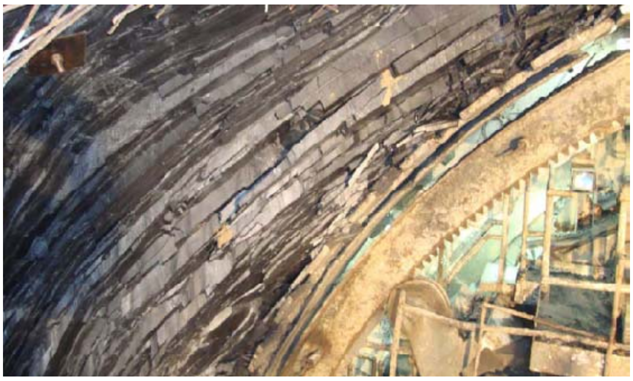

Figure 4 Spalling failure of the rock mass around the No. 3 diversion tunnel in Jinping II Hydropower Station in China (photographed by Wang in 2011)

2. BRITTLE FEATURE OF INTACT ROCK

Spalling-type failure is generated by extension crack and cracks propagation. Based on the experimental data, Kaiser and Kim (2008) demonstrated that lower confining stress would facilitate the creation, growth, and propagation of cracks and thus the rock strength would reduce obviously. The significant strength reduction often occurs as the confinement lowering than $0.1\sigma_c$, where σ_c is the uniaxial compressive strength of rock core. Since traditional shear failure criteria (e.g. Hoek-Brown failure criterion) could not well account for the strength reduction at lower confinement condition, Kaiser and Kim (2008) proposed a tri-linear brittle failure envelope, by using damage initiation threshold, spalling limit and shear failure criterion, to illustrate the scenario of strength reducing process. Diederichs *et al*. (2010) further describes the in-situ damage of massive rocks under different confining stress based on the spalling envelope, as shown in Figure 5.

Under high confinement condition, the behaviour of brittle hard rock is controlled by shear failure. The failure induced by extension crack will be gradually dominant with decreasing the confining stress. Finally the mechanical behaviour is predominate by tensile or extensional failure. The transition between shear failure and extensional failure is distinguished by the spalling limit, which can be evaluated by the stress ratio of σ_1/σ_3, where σ_1 is the maximum principal stress and σ_3 is the minimum principal stress. For highly heterogeneous rock types, the spalling limit is often less than 10 but may be more than 10 in homogeneous materials. Furthermore, the lower bound field strength (e.g. damage initiation threshold) in the envelope denotes the tensile or extensional crack initiation stress. Below the threshold, no damage occurs within the rock. Above this threshold, micro-cracks initiate at the grain scale. At low confinement near an excavation wall, rock that is stressed will incur spalling damage as new extension cracks and old cracks are allowed to propagate in an unstable fashion. The stresses above damage initiation threshold but at high confinements these initiating micro-cracks will quickly stabilize as they propagate away from the nucleation site.

To understand the mechanical characteristics of brittle hard rock in Taiwan, six sets of laboratory tests on marble, including uniaxial compressive test and triaxial compressive test, were carried out. Marble is one of major metamorphic rock types in eastern Taiwan. It can be massive or thick-bedded, ranging from fine- to coarse-grained. High quality marbles with pure white color are mined as dimension stone or ornamental stone. The tested specimens were acquired from the marble quarries in the region. The specimens were circular cylinders having a height to diameter ratio of 2.0 and a diameter of 54 mm. The confining stress from $0.05\sigma_c$ to $0.8\sigma_c$ were applied for each triaxial compressive test. The post-peak strength of marble was obtained by using the performance of MTS rock mechanics test system with an axial strain rate of 0.2 mm/min.

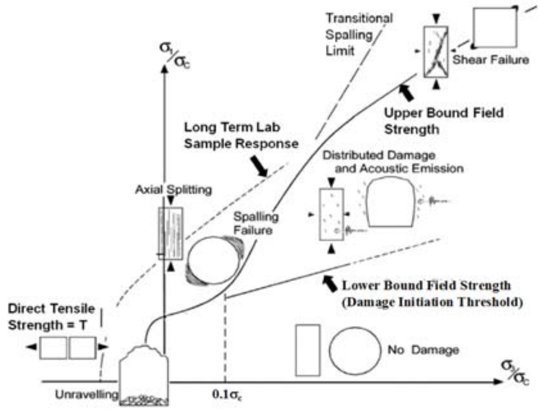

Figure 5 Conceptual model of damage, spalling and confined yield in non-porous rocks (after Diederichs *et al.,* 2010)

From the results of laboratory test (as summarized in Table 1), the uniaxial compressive strength of marble is about 61-94 MPa. The typical stress-strain curves under different confining stresses are illustrated in Figure 6. It is found that post-peak curves are strongly affected by the confining stress. At low confining stress, the strength rapidly drops down to a residual value after reaching the peak strength, showing a typical brittle feature. With increasing confining stress, the residual strength progressively increases and the stress-strain curve does not drop down immediately after reaching its peak strength. Simultaneously, the curve remains at the peak strength for a period of time previous to dropping down. Rock behaviour become more and more ductile and eventually acts like an ideally plastic medium with the increase of confining stress. The possible critical confining stress ($\sigma_{3,crit}$) for the transition from brittleness to ductility is about 0.2 times of maximum principal stress proposed by Seeber (1999). The test results show that the transition from brittleness to ductility on the marble of Taiwan can be defined as the equation of $\sigma_{3,crit}= 0.23\sigma_1$, which approximately consists with the suggestion of Seeber in 1999.

Moreover, the slope of the stress-strain curve prior to failure is gradually sharp and meanwhile the peak strength becomes greater with the increasing of confining stress, as showed in Figure 6. It means both the stiffness and the peak strength are increased with increasing confining stress. Figure 7 shows the normalization of the maximum principal stress (σ_1) and the minimum principal (σ_3) stress divided by the uniaxial compressive stress (σ_c) for each triaxial compressive test. It is discovered that the rock strength of marble has a great reduction under a lower confining stress of less than $0.2\sigma_c$, which is approximately agreed with the characteristic of aforementioned brittle failure envelope, that is, the rock strength ould intensely reduce under a low confining stress. The stress field near tunnel wall is actually in low confinement due to the radial

Table 1 Summarization of laboratory test on marble in eastern Taiwan

Test No.	Experimental results (stress unit: MPa)						
1	Confining stress (σ_3)	0	4	8	16	34	64
	Peak strength (σ_1^p)	94	101	131	149	191	281
	Residual strength (σ_1^r)	–	24	43	86	144	268
2	Confining stress (σ_3)	0	3.75	7.5	15	30	60
	Peak strength (σ_1^p)	75	95	112	143	206	299
	Residual strength (σ_1^r)	––	31	42	75	144	270
3	Confining stress (σ_3)	0	3	6	12	24	48
	Peak strength (σ_1^p)	61	68	81	98	137	204
	Residual strength (σ_1^r)	––	24	32	58	111	198
4	Confining stress (σ_3)	0	4.5	9	18	36	63
	Peak strength (σ_1^p)	88	119	135	169	209	275
	Residual strength (σ_1^r)	––	59	37	91	167	263
5	Confining stress (σ_3)	0	3.75	7.5	15	30	60
	Peak strength (σ_1^p)	74	127	108	133	183	287
	Residual strength (σ_1^r)	–	24	45	69	144	261
6	Confining stress (σ_3)	0	4.25	8.5	17	34	60
	Peak strength (σ_1^p)	88	98	111	166	215	264
	Residual strength (σ_1^r)	––	34	58	97	159	227

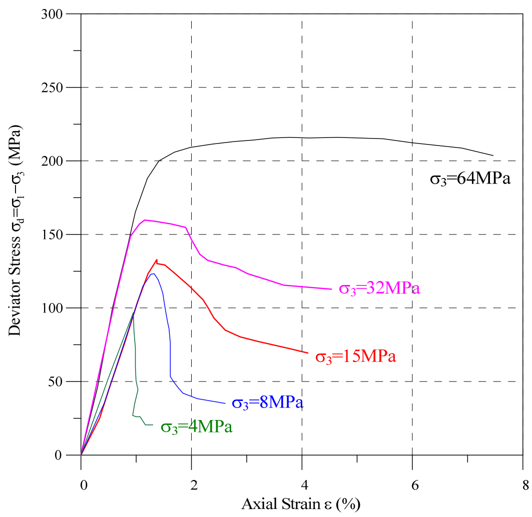

Figure 6 Typical stress-strain curves under different confining stresses on marbles in eastern Taiwan (Test No.1 in Table 1)

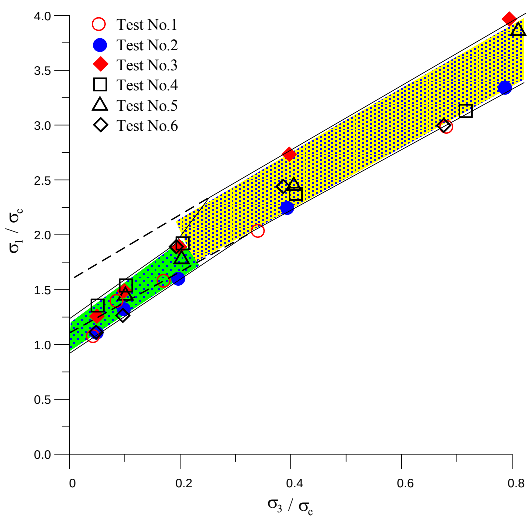

Figure 7 Relationship between normalization of maximum principal stress and minimum principal stress for the laboratory tests on marble in eastern Taiwan

stress release rapidly after excavation, therefore, the rock strength for deep tunneling may over-estimate by using traditional shear failure criteria. According to test results performed in the paper, the possible over-estimated value would reach to 1.2-1.6 times of rock strength when the confining stress equals zero, as illustrated in Figure 7.

3. POST-PEAK STRENGTH ESTIMATION

As Hoek and Marinos (2009) has pointed out that it is nearly impossible to prevent failure initiating from tunnelling in highly stressed rock. The aim of the design of reinforcement or support is to control the propagation of the failure and to retain the profile of the tunnel. Therefore, understanding both the peak and the post-peak strength are necessary for the valid design and stability assessment of deep tunnelling.

The evaluation of rock strength around an underground opening is actually a difficult task. Many methods have been proposed on the determination of peak strength of rock, however only few attempts have been made to estimate the post-peak strength of rock. Actually, there is no a generally acceptable method in estimating the post-peak strength degradation up to now. Experiences from case studies or results from material tests are the usually adopted method. For instance, Read and Chandler (1997) proposed using the constant values of s=0.16, m_b=28, s_r=0.01 and m_r=1 to optimize the tunnel shape on granite in Canada, where s and m_b are the Hoek-Brown peak strength parameters and the subscript ”r” indicates residual values. The relations of s_r=0.04s and m_r=0.65m_b were suggested to estimate the post-peak strength of jointed rock masses by Ribacchi (2000). Furthermore, the relations of s_r=s and m_r=0.1m_b were presented to portray the post-peak strength of muscovite schist by Crowder et al. (2006), which is obtained from the back analysis of underground mining. The Mohr-Coulomb parameters of c_r=0.1c and ϕ_r=0.9ϕ_b were adopted to simulate the post-peak strength of sandstone in a hydraulic tunnel by Kumar et al. (2008). These aforementioned suggestions are essentially obtained from specific rock type or individual case. The appropriateness for different rock type or for different site needs to be re-examined.

Although the Hoek-Brown failure criterion has been widely used in the estimation of rock strength and adopted into many professional software for rock engineering analysis (e.g. PHASE, FLAC, etc.), the post-peak strength degradation due to brittle failure was not considered. Carranza-Torres *et al.,* (2002) presented a solution to quantify the strength degradation in terms of the

Hoek-Brown failure criterion by introducing a strength loss parameter (β), as shown in Eq. (1):

$$\sigma_1 = [1+(1-\beta)(k_\phi -1)]\sigma_3 + (1-\beta)\sigma_c \quad (1)$$

where $k_\phi = (1+\sin\phi)/(1-\sin\phi)$ and ϕ is the internal angle of friction of rock mass, expressed in radians. The strength loss parameter varies as $0\le\beta\le1$, such that $\sigma_1=k_\phi\sigma_3+\sigma_c$ (β=0) for no strength loss and $\sigma_1=\sigma_3$ (β=1) for residual strength condition. Cundall *et al*., (2003) further proposed a similar post-peak form of the Hoek-Brown failure criterion based on the Eq. (1) and the concept of strength loss parameter (β), which is expressed as

$$\sigma_1 = \sigma_3 + \sigma_c^R\left[m_b^R\frac{\sigma_3}{\sigma_c^R}+s\right]^a \quad (2)$$

where the post-peak parameters, σ_c^R and m_b^R are defined as

$$\sigma_c^R = (1-\beta)\sigma_c \quad (3)$$

$$m_b^R = (1-\beta)m_b \quad (4)$$

Substitute Eq. (3) and Eq. (4) into Eq. (2), the following equation can be obtained.

$$\sigma_1 = \sigma_3 + (1-\beta)\sigma_c\left[m_b\frac{\sigma_3}{\sigma_c}+s\right]^a \quad (5)$$

The Hoek-brown parameter of m_b is related to the friction component of material. The amount of change is dependent on the rock mass and type of failure. For example, in massive rock mass that fails in a brittle manner, the value of m_b should experience a large reduction, whereas very weak rock that behaves in a ductile manner should experience very low or no reduction of m_b. Another Hoek-brown parameter s is related to the cohesive component of material. This parameter is basically expected to decrease after failure. However, the value of s does not change when the rock mass fails in Eq. (5) because the cohesion loss is concealed in the reduction of the unconfined compressive strength σ_c (Hsiao *et al*., 2011). The post-peak values for σ_c and m_b can be assessed by multiplying the peak values by the factor (1-β).

Overall, the post-peak form of the Hoek-Brown failure criterion proposed by Cundall *et al*., in 2003 is defined distinctly. The conception of strength loss parameter has been adopted in the evaluation of material softening in the Hoek-Brown model in FLAC since 2005. However, the validity of the strength loss parameter for different rock type or in different confining pressure condition is still unknown. The reasonable estimate method for the strength loss parameter is the upmost important issue.

A valid process, so-called strength loss experiment method, proposed by Hsiao et al. (2012), would establish the relationship between strength loss parameter and confining stress by using Eq. (5) and the parameters of peak strength of intact rock. The accuracy of the strength loss experiment method was verified by a performance of the axisymmetric numerical modelling. In this paper, the strength loss experiment method was adopted to evaluate the β value for each test showed in Table 1. Figure 8 illustrates the assessment results of the Test No.1 marble specimen. The estimation procedures are described briefly as follows.

(1) The uniaxial compressive strength of rock (σ_c), the GSI value of 100 for intact specimen and the m_i value suggested by Hoek (2002) are adopted to plot the Hoek-Brown failure envelope of peak strength. The solid line in Figure 8 is the failure envelope of the peak strength for Test No.1 marble specimen, which uniaxial compressive strength is 94 MPa and the Hoek-Brown parameter of m_b is 6.0.

(2) Based on the parameters of peak strength and the Eq. (5), the post-peak form of the Hoek-Brown failure envelopes with different strength loss parameters (β) can be plotted, such as the dotted lines in Figure 8. The line with β equals 0 represents no strength loss. With the increasing of β, the post-peak strength of rock would reduce progressively. Eventually, the line with β equals 1 means the lowest residual strength. The post-peak strength is closely related to confining stress so that each confining stress would exist a corresponding value of the strength loss parameter. It was found that the β values of 0.05, 0.30, 0.48, 0.70 and 0.80 would respectively represent the post-peak condition as the confining stress is 64, 32, 16, 8 and 4 MPa, as shown in Figure 8.

(3) The post-peak strength for different confining stress condition can be calculated by using the β value estimated and the Eq. (3) and (4). The results of the calculation showed that the strength loss parameter (β) is 0.05 and the post-peak strength parameters of σ_c^R and m_b^R are 89 MPa and 5.7 when the confining stress of 64 MPa was adopted. Moreover, the strength loss parameter (β) is 0.8 and the post-peak strength parameters of σ_c^R and m_b^R are 19 MPa and 1.2 when the confining stress of 4 MPa was adopted. The other results are summarized in Table 2.

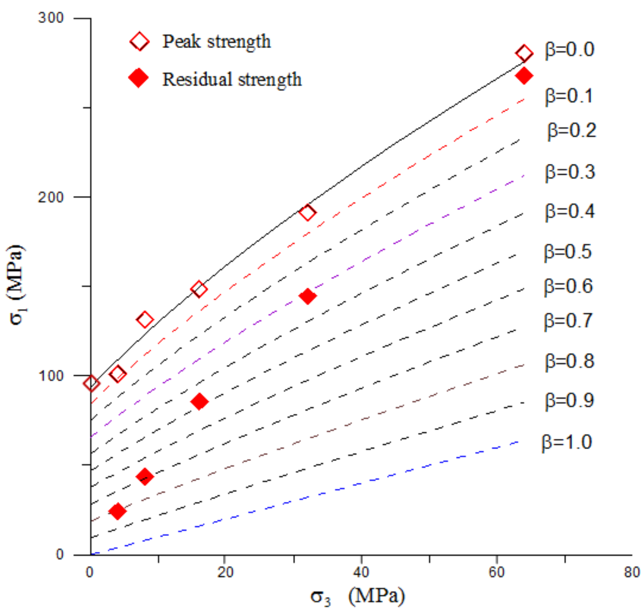

Figure 8 Schematic evaluation of the stress loss parameter by using the post-peak form of the Hoek-Brown failure for the Test No.1 marble specimen

Table 2 Results of post-peak strength estimation for the Test No.1 marble specimen (stress unit: MPa)

	Confining stress (σ_3)	Mechanical parameters						
		β	σ_c	σ_c^R	m_b	m_b^R	s	a
Peak	–	0	94	–	6.0	–	1.0	0.5
Post-peak	64	0.05	–	89	–	5.7	1.0	0.5
	32	0.30	–	66	–	4.2	1.0	0.5
	16	0.48	–	49	–	3.1	1.0	0.5
	8	0.70	–	28	–	1.8	1.0	0.5
	4	0.80	–	19	–	1.2	1.0	0.5

Furthermore, other strength loss parameters under different confining stress condition for the marble summarized in Table 1 (Test No.2~Tsst No.6) would be estimated respectively according to the aforementioned procedure. The relationship between the strength loss parameter and the normalized confining stress (divided by uniaxial compressive strength) for six sets of marble tests was sketched as shown in Figure 9. From this figure, the strength loss parameter is progressively increased with the decreasing of the confining stress and the regression equation can be obtained as

$$\beta = -0.239 \times \ln(\sigma_3 / \sigma_c) + 0.059 \tag{6}$$

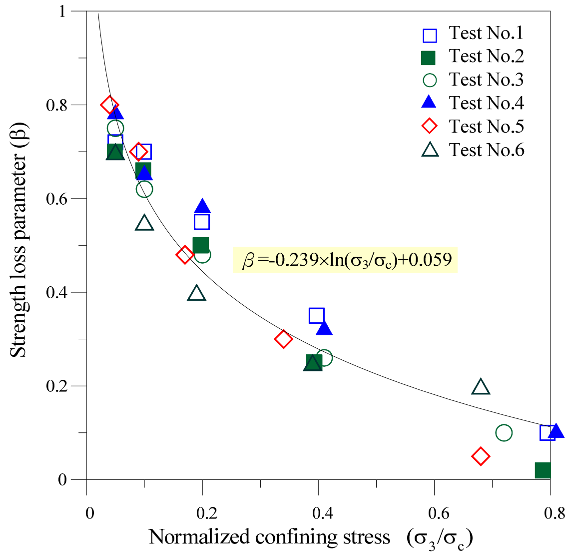

Figure 9 Correlation between strength loss parameter and normalized confining stress on marble

4. NUMERICAL APPORACH

There are an exponential growth in numerical analysis method to assess the effect of brittle failure on tunnel behaviour recently. For example, the strain softening model within FLAC was used by Badr (2003) and Kumar (2008) to discuss the effect of post-peal strength degradation on pillar strength of an underground mining opening and on tunnel deformation of an underground hydropower station. The Hoek-Brown model and the residual strength obtained from back analysis were adopted to predict the excavation damage zone around the tunnel for different rock mass quality under various overburden pressures at Jinping II hydropower station (Zhang, 2011) Furthermore, a progressive residual GSI system was developed to extend the GSI system to estimate the possible residual strength of rock mass (Cai *et al*., 2007) and an application for the construction stability assessment of a hydraulic tunnel in southern Taiwan was performed by Hsiao and Kao (2015).

Due to the rapidly reduction of radial stress near excavation, the rock masses surrounding a tunnel are actually in low confining stress condition. Meanwhile, the reduction magnitude of radial stress varies with the distance away from excavation boundary, as portrayed in Figure 10. However, the influence of the confining stress on strength degradation was not incorporated into the aforementioned analysis methods.

Based on the relation between the post-peak strength and the confining stress proposed by Hsiao *et al.*, (2012), a subroutine of FISH language embedded within the computer program FLAC was developed to incorporate the confining stress dependent post-peak strength when yielding occurred. The stress condition for each element is checked automatically once 10 timesteps during calculation. In case the stress condition beyond the peak strength, the strength loss parameter (β) is evaluated according to Eq. (6) and the calculated confining stress. The residual strength for the yielded element would further obtain from the Eq. (3) and (4) and new balance can be achieved. This numerical analysis model called strength degradation model, "SD model" for short.

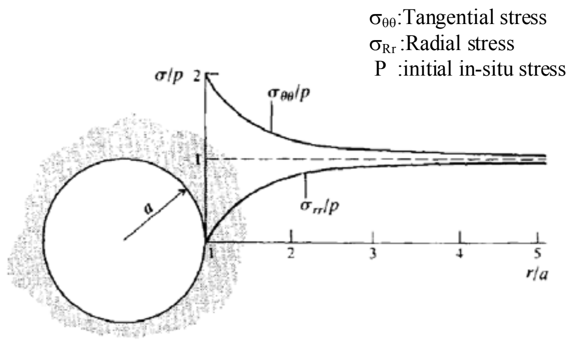

Figure 10 Stress variation tendency around an excavation opening

In order to understand the appropriateness of the strength degradation model used in the modelling of brittle rock excavation, an examination as compared with the empirical relationship proposed by Martin et al. (1999) was conducted. The empirical relationship was installed by a review of available identified eight case histories, as listed in Table 3.

Table 3 Summary of case histories used to establish relationship between depth of brittle failure and in situ stress condition (after Martin et al., 1999)

Rock mass	R_f/a	σ_1/σ_3	σ_3 (MPa)	σ_c (MPa)	Reference
Blocky andesite	1.3	1.92	15.3	100	GRC field notes
	1.5	2.07	14.8	100	(EI Teniente Mine)
	1.4	2.03	14.7	100	
	1.5	2.10	16.3	100	
	1.5	2.03	15.4	100	
	1.6	2.09	15.8	100	
Massive quartzite	1.8	2.15	65	350	Ortlepp and Gay (1984)
	1.7	2.15	65	350	
	1.4	1.86	60	350	
	1.5	1.86	60	350	
Bedded quartzite	1.4	3.39	15.5	250	Stacy and de Jongh (1977)
	1.3	3.39	15.5	250	
Massive granite	1.5	5.36	11	220	Martin et al. (1994)
	1.4	5.36	11	220	
	1.4	5.36	11	220	
	1.3	5.36	11	220	
	1.3	5.36	11	220	
	1.0	3.70	11	220	
Massive granite	1.1	1.31	40	220	Martin (1989)
Interbedded siltstone-mudstone	1.4	2.0	5	36	Pelli et al. (1991)
Bedded limestone	1.1	1.3	12.1	80	Jiayou et al. (1989)
Bedded quartzite	1.0	1.69	21	217	Kirsten and Klokow (1979)
	1.08	1.69	20	151	

The collected information include brittle failure depth, rock type, in situ stress state, intact rock strength, etc. Martin et al. (1999) presented that the depth of brittle failure around a tunnel can be approximated by a linear relationship given as

$$\frac{R_f}{a} = 0.49(\pm 0.1) + 1.25\frac{\sigma_{\max}}{\sigma_c} \tag{7}$$

Where R_f is depth of failure and a is excavation radius. Furthermore, $\sigma_{\max}$ is maximum tangential stress at the boundary of the opening, which may estimate by the equation of $\sigma_{\max}=3\sigma_1-\sigma_3$. The regression relationship, the Eq. (7), and the cases adopted are plotted in Figure 11. The figure shows that the depth of brittle failure is linearly increased with increasing the ratio of maximum tangential stress and uniaxial compressive strength. The empirical relationship has been extensively used in predicting failure depth of brittle rock excavation in many investigations (e.g. Anderson *et al.,* 2009; Cai & Kaiser, 2014; Hoek & Martin, 2014).

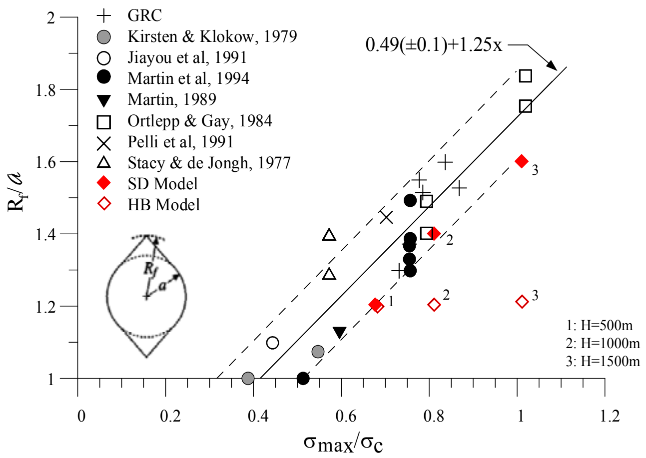

Figure 11 Relationship between depth of failure and the maximum tangential stress at the boundary of the opening

A circular tunnel with an excavation diameter of 10 m was adopted as an example to examine the appropriateness of the strength degradation model. The average uniaxial compressive strength of 80 MPa and the deformation modulus of 7000 MPa for marble were adopted according to the geotechnical survey report of eastern Taiwan (Sinotech, 2011). Assuming the rock mass surrounding the case tunnel is in good quality range (GSI=80). The mechanical parameters, as listed in Table 4, were estimated by the suggestion of Hoek (2002).

Table 4 Mechanical parameters of rock mass for the case tunnel adopted

σ_c (MPa)	Unit weight (t/m^3)	m_b	s	a	Deformation modulus (MPa)
80	2.7	4.406	0.1084	0.5	7000

Both the HB model (Hoek-Brown model without strength degradation) and the SD model (Hoek-Brown model with strength degradation) were used to analyze the effect of the post-peak strength degradation. Various overburden of 500m, 1000m, and 1500m were considered in the analysis. In total six cases for different scenario were performed. The analyzed failure depth for the rock mass around the tunnel and the maximum tangential stress at the tunnel wall are plotted as the rhombus symbol in Figure 11. The solid rhombus data are the analysis results obtained by using the strength degradation model (SD model). The failure depth is clearly increased with the increasing of the maximum tangential stress at the tunnel wall, which is in agreement with the empirical relationship suggested by Martin et al. (1999). It indicates that the strength degradation model could be suitable for modelling the behavior of brittle rock excavation at deeps. Furthermore, the hollow rhombus data in Figure 11 are the results obtained by using the HB model (no strength degradation). The failure depth for the HB model is increased slightly at the depth of 1000-1500m. The failure depth is obviously underestimated when the HB model was used. It indicates that the deep tunneling stability and the relaxation zone around the tunnel would be incorrectly estimated without considering the effect of strength degradation beyond failure.

5. CASE APPLICATION

The effect of post-peak strength degradation on deep underground excavation was applied to predict the tunnel convergence and the failure depth for a highway tunnel. The tunnel is part of the project providing a safe and reliable connecting highway for east and north Taiwan. The case tunnel is a twin-hole tunnel with excavation span of 12.5 m. The pillar width between two tunnels is about 30 m. The top heading method is designed for the tunnel excavation and the cycle length is 3.5 m for good rock mass (GSI=80). The support works used including 8 cm thick steel fiber reinforced shotcrete with systematic rock bolts. The rock bolts installed are of 25 mm ϕ with length of 4 m and spaced of 2.0m×3.0~4.0m. The allowable tunnel deformation is 5 cm in the support system. Two different overburden depths of 500 m and 1000 m were considered in the case study. Vertical stress (P_0) was calculated by dead weight of rock mass as $P_0=\gamma H$, where γ is the unit weight of rock mass and H is the overburden depth. The horizontal stress was estimated with references to the results of in-situ overcoring test in the eastern Taiwan (Hsiao et al. 2006). The maximum horizontal stress is 1.2 times of vertical stress.

Tunnel was excavated since July 2013. The rock mass along the tunnel alignment are marble and a few gneiss. The rock mass revealed at the excavation face is normally in good quality range, as illustrated in Figure 12. The mechanical parameters of rock mass are referred to the aforementioned geotechnical survey report (Sinotech 2011), as shown in Table 4. Furthermore, Eq. (3), Eq. (4) and Eq. (6) were used to estimate the post-peak strength.

Figure 12 Rock mass condition revealed at the station of 4K+407.5 for the northbound of case tunnel

The Hoek-Brown model (no strength degradation, HB model) and the strength degradation model (SD model) were adopted to simulate the tunnelling behaviour. The results of the analyses are displayed in Table 5. At the scenario of the overburden depth of 500 m with the Hoek-Brown model, the tunnel roof settlement of 17 mm, the bench horizontal convergence of 19 mm and the failure depth of 0.5 m at the roof and the sidewall are obtained. The tunnel deformation would increase significantly where the overburden reaches to 1000 m, that is, 38 mm in the roof settlement and 42 mm in the horizontal convergence. And the failure depth surrounding the tunnel would increase to 1.5 m as well. However, the tunnel deformation is still under the designed allowable value of 50 mm even the tunnel depth reaches to 1000 m.

In case the post-peak strength degradation is considered, the roof settlement of 37 mm and the bench horizontal convergence of 44 mm are obtained at the tunnel depth of 500 m for the SD model. And approximate 2.6 times of deformation increased occur as the tunnel depth reaches to 1000 m, that is, 98 mm in roof settlement and 90 mm in horizontal convergence. The tunnel deformation obviously exceeds the designed allowable value and tunnel may need re-mining in deep overburden condition, as shown in Figure 13.

Table 5 Effect of overburden depth and post-peak strength degradation on tunnelling behaviour by using numerical analyses

		Overburden=500m		Overburden=1000m	
		HB model	SD model	HB model	SD model
Tunnel deformation	Roof settlement	17mm	37mm	38mm	98mm
	Horizontal convergence	19mm	44mm	42mm	90mm
Failure depth	Crown	0.5m	1.5m	1.5m	3.5m
	Sidewall	0.5m	1.5m	1.5m	2.5m

The length of about 596 m in tunnel northbound and the length of about 462 m in tunnel southbound have been excavated till now. The maximum overburden for the excavated section is approximate 350 m. The measured roof settlement is in the range of 15-25 mm. Tunnel is in stable condition. However, the maximum overburden in the case tunnel will exceed 1000 m. The above analyses demonstrate that the characteristic of post-peak strength degradation under high overburden stress condition would strongly affect tunnel behaviour. The effect intensity is progressively significant with increasing tunnel depth, as illustrated in Figure 13. There is a significant increase in tunnel deformation at the depth of 1000 m for the SD model. In case additional reinforcement or modified excavation measure is not adopted, severe tunnel deformation may endanger tunnel stability. The strength degradation of marble beyond brittle failure plays an important role in the stability of deep tunnelling.

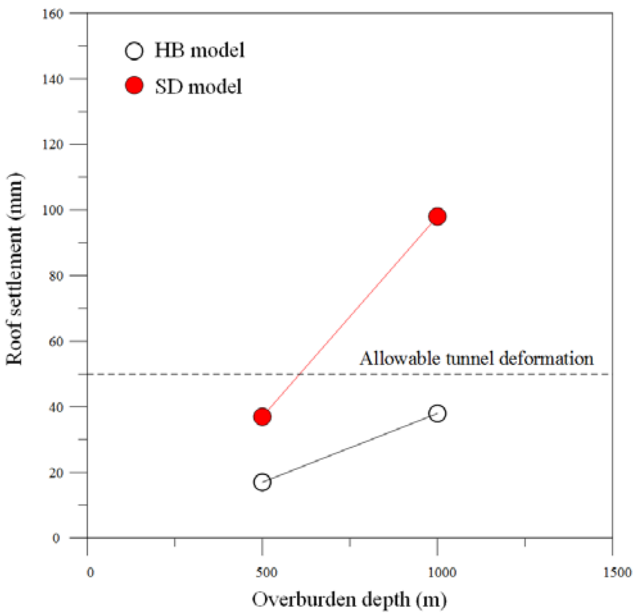

Figure 13 Correlation between roof settlement and overburden depth in the case tunnel

6. CONCLUSION

Tunnelling in hard rock is normally stable at shallow depth except for wedge failure. However, the excavation depth is rapidly increasing recently. Under high overburden stress condition, the integrity of rock would be destroyed with degrading rock strength. Except for violent rock burst, the post-peak strength degradation may endanger the stability of deep tunnelling. With our current understanding, there is generally no acceptable method available to estimate the post-peak strength. The post-peak form of the Hoek-Brown failure criterion by introduced a strength loss parameter may be a feasible method. A series of tri-axial compressive tests of marble, which is the typical metamorphic hard rock in Taiwan, were carried out. The results of the tests show that the post-peak strength is strongly affected by the confining stress. Therefore, the relationship between strength loss parameter and confining stress was established by using the strength loss experiment method. It should be noticed that the post-peak strength estimation method used in the paper is limited to the massive unjointed brittle rock (such as GSI >70) because it was established on the basis of intact rock tests. For jointed rock or other rock type (such as schist, andesite, and so on), the applicability of the method should be subjected to further investigation and re-adjustment.

According to the analysis of actual tunnel case in eastern Taiwan, the effect of post-peak strength degradation should not be disregarded. The tunnel deformation would exceed the design value at deeps. The instability hazard is anticipated and tunnel may require re-mining due to the inadequacy of inner section. Actually, various studies and experiences revealed that brittle failure initiating is extremely difficult to prevent in highly stressed rock. The aim of the design of reinforcement or support is to control the propagation of the brittle failure and to retain the profile of the tunnel. The issue of brittle failure was rarely studied in Taiwan in the past. Therefore, cautious design and systematic monitoring should be fulfilled to overcome the possible problems caused by brittle failure in deep underground excavation in Taiwan.

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