# Effect of brittle failure on deep underground excavation in eastern Taiwan

## Effet de la rupture fragile sur l'excavation souterraine profonde dans l'est de Taiwan

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ABSTRACT: The excavation depth is increasing rapidly in Taiwan. The effect of brittle failure for hard rock was seldom studied in Taiwan in the past. A serial of tri-axial compressive tests of marble were performed in this paper. Relatively small deformation occurs prior to failure during loading, and under continued loading, the integrity of rock is destroyed with degrading rock strength. To reasonably estimate the post-peak strength, the post-peak form of the Hoek-Brown failure criterion and the strength loss experiment method was adopted to establish the relationship between strength loss parameter and confining stress. Furthermore, the numerical modeling of an actual tunnel in eastern Taiwan was conducted to examine the impact of the post-peak strength degradation. The analyses show that the effect of post-peak strength degradation on excavation deformation is progressively significant with increasing the depth. Severe tunnel deformation may endanger the excavation stability in deep overburden. The strength degradation beyond brittle failure plays an important role in the stability of deep underground excavation.

RÉSUMÉ : La profondeur d'excavation augmente rapidement à Taiwan. L'effet de la rupture fragile pour le hard rock a été rarement étudié à Taiwan dans le passé. Série TA de tri-axiales essais de compression de marbre ont été réalisés dans ce document. Déformation relativement faible se produit avant la panne pendant le chargement, et sous une charge continue, l'intégrité de la roche est détruit avec résistance de la roche dégradants. Pour estimer raisonnablement la force post-pic, la forme post-pic du critère de défaillance Hoek-Brown et la méthode de perte de force expérimentation a été adoptée pour établir la relation entre le paramètre de la perte de force et de la contrainte de confinement. En outre, la modélisation numérique d'un tunnel réelle dans l'est de Taiwan a été menée pour examiner l'effet de la dégradation de la résistance post-pic. Les analyses montrent que l'effet de la dégradation de la résistance post-pic sur la déformation de l'excavation est progressivement significative avec l'augmentation de la profondeur. Déformation du tunnel de grande taille peut mettre en danger la stabilité de l'excavation dans les profondeurs. La dégradation de la résistance au-delà de la rupture fragile joue un rôle important dans la stabilité des excavations souterraines profondes.

KEYWORDS: Brittle failure, post-peak strength, numerical modeling , deep underground excavation

## 1 INTRODUCTION

Metamorphic hard rock extensively exists in eastern Taiwan. The uniaxial compressive strength of the metamorphic rock in this region is mainly in the range of 50-200 MPa. In general, underground excavation in hard rock is normally stable at shallow depth except for wedge failure. However, the excavation depth is increasing in Taiwan, e.g. the maximum overburden of 1188 m for Chungren Tunnel in Suhua highway is being constructed. In high overburden stress condition, the integrity of rock would be destroyed with degrading rock strength. The post-peak strength degradation of brittle failure would significantly affect tunnel behavior, including tunnel deformation and the support pressure required, as portrayed in Figure 1. Therefore, understanding both the peak and the post-peak strength are necessary for the stability assessment of deep underground excavation.

Although many researches have been proposed on the determination of peak strength of rock, only few attempts have been made to estimate the post-peak strength of rock. Actually, experiences from case studies or results from material tests are the usually used method. For instance, 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), where $s$ and $m_b$ are the Hoek-Brown peak strength parameters and the subscript "r" indicates residual values. Furthermore, the relations of $s_r=0.04s$ and $m_r=0.65m_b$ 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 $\phi_r=0.9\phi_b$ were adopted to simulate the post-peak strength of sandstone in a hydraulic tunnel by Kumar et al. (2008). However, these suggestions for the post-peak strength estimation are the individual case and the appropriateness on different rock type or in different area needs to be re-examined.

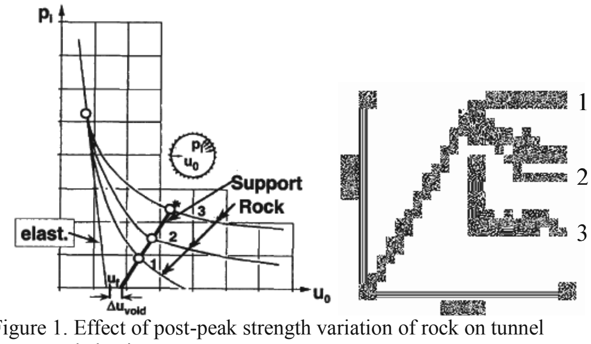


Figure 1. Effect of post-peak strength variation of rock on tunnel behavior

## 2 LABORATORY TESTS

Marble is one of major metamorphic rock types in eastern Taiwan. To understand the post-peak characteristics of marble, six sets of laboratory tests on marble, including uniaxial compressive test and triaxial compressive test, were carried out.

The specimens were circular cylinders having a height to diameter ratio of 2.0 and a diameter of 54 mm. Five different confining stresses from $0.05\sigma_c$ to $0.8\sigma_c$ were applied for each triaxial compressive test, where $\sigma_c$ is the uniaxial compressive strength of rock core.

According to the results of laboratory tests (as summarized in Tab.1), the uniaxial compressive strength of marble is about 61-94 MPa. The typical stress-strain curves under different confining stresses are shown in Figure 2. The characterization of post-peak curves is strongly affected by the confining stress. Under a low confining stress, the curve rapidly drops down to a residual strength after reaching the peak strength, showing a typical brittle behavior. 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 behavior become more and more ductile and eventually acts like an ideally plastic medium under a high confining stress. However, the rock masses surrounding a tunnel are actually in low confinement condition due to the rapidly reduction of radial stress near excavation. Tunneling behaviors are basically dominated by the brittle feature of rocks.

Table 1. Summarized results of laboratory tests on marble in eastern Taiwan

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

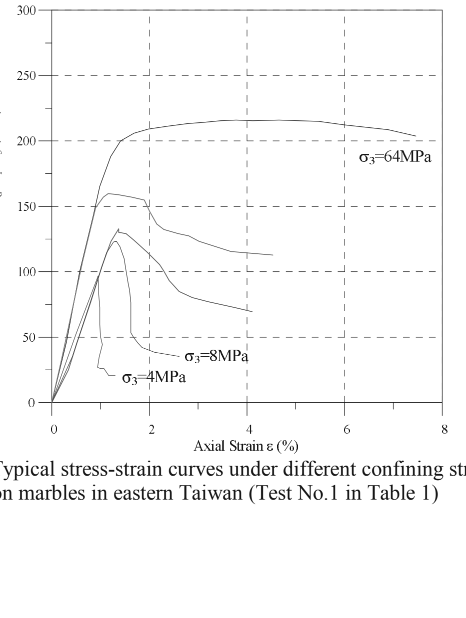


Figure 2. Typical stress-strain curves under different confining stresses on marbles in eastern Taiwan (Test No.1 in Table 1)

## 3 ESTIMATION OF POST-PEAK STRENGTH

### 3.1 Post-peak form of the Hoek-Brown failure criterion

The Hoek-Brown failure criterion is widely used in the estimation of rock strength in rock engineering. However, the phenomenon of post-peak strength degradation in brittle rock was not considered in the criterion. Cundall et al. (2003) presented a solution to quantify the strength degradation in terms of the Mohr-Coulomb failure criterion by introducing a strength loss parameter ($\beta$), as shown in Eq. 1:

$$\sigma_1 = \sigma_3 + \sigma_c^R \left[ m_b^R \frac{\sigma_3}{\sigma_c^R} + s \right]^a \tag{1}$$

where the post-peak parameters, $\sigma_c^R$ and $m_b^R$ are defined as

$$\sigma_c^R = (1-\beta)\sigma_c \tag{2}$$

$$m_b^R = (1-\beta)m_b \tag{3}$$

The strength loss parameter varies as $0 \leq \beta \leq 1$, such that $\beta$=0 for no strength loss and $\beta$=1 for residual strength condition. Substitute Equation (2) and (3) into (1), the following equation can be obtained:

$$\sigma_1 = \sigma_3 + (1-\beta)\sigma_c \left[ m_b \frac{\sigma_3}{\sigma_c} + s \right]^a \tag{4}$$

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 Equation 4 because the cohesion loss is concealed in the reduction of the unconfined compressive strength $\sigma_c$ (Hsiao et al. 2011). The post-peak values for $\sigma_c$ and $m_b$ can be assessed by multiplying the peak values by the factor (1-$\beta$).

### 3.2 Estimation of Strength loss parameter

The post-peak form of the Mohr-Coulomb 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 hoekbrown 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.

A method proposed by Hsiao et al. (2012), so-called strength loss experiment method, would establish the relationship between strength loss parameter and confining stress by using Equation 4 and the parameters of peak strength of intact rock. In this paper, the strength loss experiment method was adopted to evaluate the $\beta$ value for each test showed in Table 1. Figure 3 illustrates the assessment results of the Test No.1 marble specimen. The line with $\beta$ equaled to 0 represents no strength loss. With the increasing the $\beta$, the post-peak strength of rock would degrade progressively. Finally, the line with $\beta$ equaled to 1 means the lowest residual strength. The post-peak strength is closely related to confining stress so that each confining stress would have a corresponding value of the strength loss parameter. It was found the $\beta$ value of 0.05, 0.3, 0.48, 0.7 and 0.8 may represent the post-peak condition as the confining stress was 64, 32, 16, 8 and 4 MPa, respectively. Then the post-peak strength for different confining stress condition can be calculated easily by using Equation (2) and (3). The results of the calculation showed that $\sigma_c^R$ = 89 MPa and $m_b^R$ = 5.7 when $\sigma_3$ = 64 MPa. The other results are summarized in Table 2.

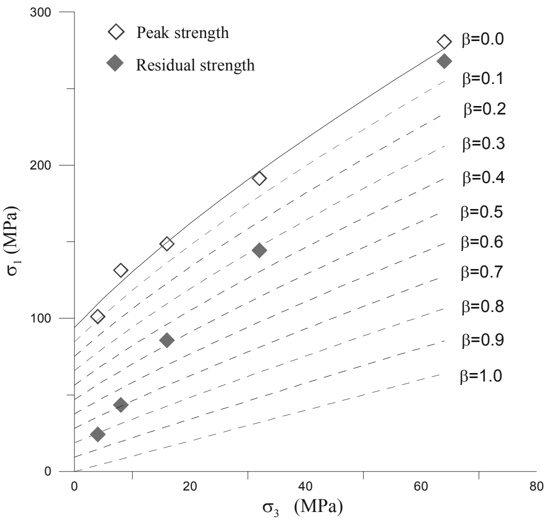


Figure 3. Schematic evaluation of the stress loss parameter by using the post-peak form of the Hoek-Brown failure for the Test No.1 marble specimen

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

| | Confining stress ($\sigma_3$) | Mechanical parameters | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | $\beta$ | $\sigma_c$ | $\sigma_c^R$ | $m_b$ | $m_b^R$ | 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 |

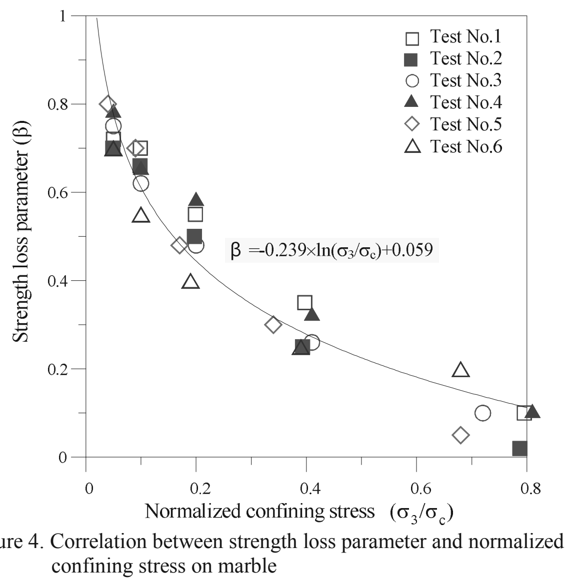


Figure 4. Correlation between strength loss parameter and normalized confining stress on marble

Furthermore, the relationship between the strength loss parameter and the normalized confining stress (divided by uniaxial compressive stress) for the tests showed in Table 1 was sketched (see Figure 4). From the figure, the strength loss parameter is progressively increased with the decreasing of the confining stress and the regression equation can be obtained as

$$\beta = -0.239 \times \ln(\sigma_3 / \sigma_c) + 0.059 \tag{5}$$

## 4 CASE STUDY

The effect of post-peak strength degradation on deep underground excavation was examined by a modeling of a road 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. According to the geotechnical survey report (Sinotech 2011), the main rock type along the tunnel is marble, which average uniaxial compressive strength is around 80 MPa. The strength parameters of rock mass in good quality range (GSI=80) was estimated by using the method suggested by Hoek (2002). The estimation results of peak strength are listed in Table 3. Furthermore, Equation 2, 3 and 5 was adopted to estimate the post-peak strength under various confining stresses. The computer program FLAC was used to simulate the tunnel construction. A subroutine of FISH language embedded within FLAC was developed to input the parameters of post-peak strength depending on the value of confining stress as the element is yielding during period of calculation.

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 $\gamma$ 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.

The Hoek-Brown model (no strength degradation, HB model) and the post-peak strength degradation model (SD model) were adopted to analyze the tunnel excavation behavior. The results of the analyses are displayed in Table 4. When the overburden depth is 500 m with the Hoek-Brown model, the roof settlement of tunnel is 1.7 cm, the horizontal convergence of bench is 1.9 cm and both the relaxation zone thickness at the roof and the sidewall are 0.5 m. Then the tunnel deformation would increase apparently where the overburden reaches to 1000 m, that is, 3.8 cm in the roof settlement and 4.2 cm in the horizontal convergence. And the relaxation zone thickness surrounding the tunnel would increase to 1.5 m as well. However, the tunnel deformation is still under the designed allowable value of 5 cm even the tunnel depth reaches to 1000 m.

In case the post-peak strength degradation is considered, the roof settlement of 3.7 cm and the horizontal convergence of bench of 4.4 cm are obtained at the tunnel depth of 500 m in the SD model. And approximate 2.6 times of deformation increased would occur when the tunnel depth reaches to 1000 m, that is, 9.8 cm in roof settlement and 9.0 cm 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 5.

The above analyses demonstrate that the characteristic of post-peak strength degradation may affect the tunnel behavior. The effect intensity is progressively significant with increasing tunnel depth, as illustrated in Figure 5. There is an outstanding 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 tunneling.

Table 3. Mechanical parameters of rock mass for the case tunnel

| $\sigma_c$ (MPa) | Unit weight (t/m3) | $m_b$ | s | a | Deformation modulus (MPa) |
|---|---|---|---|---|---|
| 80 | 2.7 | 4.406 | 0.1084 | 0.5 | 7000 |

Table 4. Effect of overburden depth and post-peak strength degradation on tunnel excavation behavior by using numerical analyses

| | | Overburden=500m | | Overburden=1000m | |
|---|---|---|---|---|---|
| | | HB model | SD model | HB model | SD model |
| Tunnel deformation | Roof settlement | 1.7 cm | 3.7 cm | 3.8 cm | 9.8 cm |
| | Horizontal convergence | 1.9 cm | 4.4 cm | 4.2 cm | 9.0 cm |
| Loosening zone of rock mass | Crown | 0.5 m | 1.5 m | 1.5 m | 3.5 m |
| | Sidewall | 0.5 m | 1.5 m | 1.5 m | 2.5 m |

* HB model means the Hoek-Brown model ; SD model means the post-peak strength degradation model

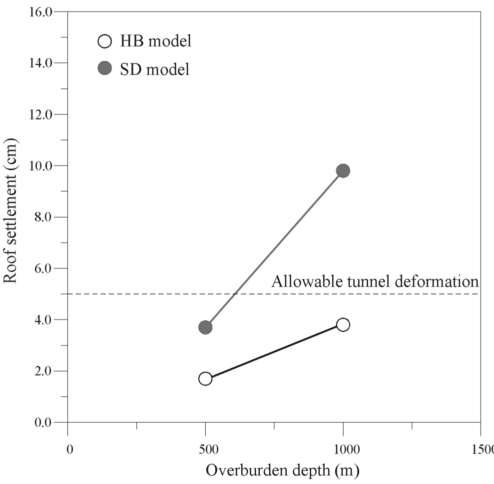


Figure 5. Correlation between roof settlement and overburden depth in the case tunnel

## 5 CONCLUSIONS

Relatively small deformation would occur prior to failure during loading, and under continued loading, the integrity of rock would be destroyed with degrading rock strength. Except for violent rockburst, the post-peak strength degradation may endanger the stability of deep underground excavation. However, there is still not a commonly acceptable method in estimating the post-peak strength up to now. The post-peak form of the Hoek-Brown failure criterion by introduced a strength loss parameter may be a feasible method currently. A serial of tri-axial compressive tests of marble, which is the typical metamorphic rock in eastern Taiwan, were carried out in this paper. 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 estimation measure for post-peak strength used in this 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 analyses of actual tunnel case in eastern Taiwan, the effect of post-peak strength degradation would not be disregarded. The tunnel deformation would exceed the design value in deep overburden. Tunnel requires re-mining due to the inadequacy of inner section and the instability will be anticipated. 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 seldom 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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