MODELING THE POST-PEAK STRENGTH DEGRADATION ON TUNNEL STABILITY

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ABSTRACT

Underground excavation in hard rock is normally stable at shallow depth except for wedge failure. However, the excavation depth is rapidly increasing recently. In high overburden stress condition, the integrity of rock would be destroyed with degrading rock strength. Therefore, understanding both the peak and the post-peak strengths of hard rock are necessary to assess the tunnel stability at depths. The characteristic of the post-peak strength is seldom studied in Taiwan in the past. Various estimation methods for the post-peak strength degradation were briefly reviewed in the paper. Based on the Geological Strength Index (GSI) system is widely used in the determination of the rock mass strength, the quantitative residual GSI system was adopted to estimate the possible strength degradation for a hydraulic tunnel in Taiwan. The Mohr-Coulomb model (no strength degradation) and the post-peak strength degradation model (strain softening model) were used to simulate the excavation of the case tunnel at different depths. The analyses demonstrate that the tunnel is in stable condition at 600m of depth. And the effect of the post-peak strength degradation on tunnel deformation is progressively significant with increasing tunnel depth. Approximate 3-5 times of increase in crown settlement and horizontal convergence would occur when post-peak strength degradation are considered at 2400m of depth. Severe tunnel deformation may endanger the tunnel stability. Modeling results reveal that the post-peak strength degradation has a great effect on tunnel stability under high overburden stress condition, which should not neglect in deep tunnel design.

KEYWORDS

High overburden, Post-peak strength degradation, Residual GSI system, Tunnel stability

INTRODUCTION

Underground excavation in hard rock is normally stable at shallow depth except for wedge failure. However, the excavation depth is rapidly increasing in Taiwan, e.g. the maximum overburden of 1200 m for the Chungren Tunnel in the Suhua Highway is being constructed. In high overburden stress environment, the integrity of hard rock would be destroyed with degrading rock strength (Hsiao et al., 2011). The degradation of post-peak strength may impact tunneling stability. Hoek (2009) noticed that it is seldom possible to prevent failure initiating as tunneling 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 strengths of rock are necessary to assess the tunnel stability at depths.

The determination of rock mass strength around an underground opening is actually a difficult task. Many approaches have been developed to estimate the strength of rock mass over the past decades. However, most of these approaches are focused on the peak strength determination, only few attempts have been made to estimate the post-peak strength of rock. To quantify the strength degradation beyond failure, a modified form of the Hoek-Brown failure criterion was proposed by introducing a strength loss parameter (Cundall et al., 2003). The modified criterion is expressed as

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

Where the strength loss parameter (β) varies from 0 to 1, as the β equal to 0 represents no strength loss and β equal to 1 means the lowest residual strength. The key issue for the application of the modified Hoek-Brown failure criterion is how to reasonably obtain the strength loss parameter. For this purpose, a valid process, so-called the strength loss experiment method, was presented to evaluate the strength loss parameter according to the results of tri-axial compressive tests (Hsiao et al., 2012). It has a great advance to evaluate the strength loss parameter for various confinement by the method because the failure of hard rock is sensitive to confining stress condition. Since the strength loss experiment method was established based on the intact rock tests, the strength parameter obtained would not fully suitable for the jointed rock masses.

Moreover, the Geological Strength Index (GSI) system, emphasized on the geological structure, is a universal rock mass classification method. The GSI system provides a complete set of mechanical properties (Hoek–Brown strength parameters m_b and s, or the equivalent Mohr–Coulomb strength parameters c and ϕ), therefore, it is widely used in the rock engineering design. As well as other methods, the GSI system is basically suitable for the determination of the peak strength of jointed rock mass. A supplementary quantified approach for the GSI system was proposed to reduce the subjectivity influence of the original GSI system, which is evaluated by using a chart of geological description of rock mass (Cai et al., 2004). And 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). The quantification residual GSI system may be a solution to estimate the post-peak strength degradation of rock mass excavated at deeps. However, more practical applications are needed to examine the appropriateness of the residual GSI system in deep tunneling, including the post-peak strength estimation and the strength degradation impact on tunnel stability.

QUANTITATIVE RESIDUAL GSI SYSTEM

The quantified GSI system proposed by Cai et al. in 2004 employs the block volume (V_b) and the joint condition factor (J_c) as quantitative characterization factors, as shown in Figure 1. Considering the facilitation in numerical modeling, the following equation for the calculation of GSI from V_b and J_c was proposed (Cai and Kaiser, 2006)：

$$GSI(V_b, J_c) = \frac{26.5 + 8.79 \ln J_c + 0.9 \ln V_b}{1 + 0.0151 \ln J_c - 0.0253 \ln V_b} \tag{2}$$

Where J_c is a dimensionless factor, and V_b is in cm^3.

In order to extend the GSI system for the rock mass residual strength estimation, an adjustment was developed based on the two major controlling factors, i.e., block volume V_b and joint condition factor J_c, to reach their residual values (Cai et al., 2007). The adjustment on the residual block volume V_b^r and the residual joint condition J_c^r are described briefly as follows.

Residual block volume

Block volume is affected by the joint set spacing and persistence. When the loading beyond the peak strength, rock mass will become less interlocked, and is heavily broken with a mixture of angular and partly rounded rock pieces. The rock will gradually disintegrate into small blocks. However, the reduction of block volume from peak to residual may has an ultimate limitation. According to the field observations, Cai et al. (2007) presented that the failed rock mass blocks are about 1–5 cm in size, i.e., the block volume sizes of the disintegrated rock masses are in the range of 1–27 cm^3, with an average of 10 cm^3. In addition, the residual block volumes would be independent of the original (peak) block volumes, in other words, no matter intact rock, moderately jointed rock mass or heavily jointed rock mass would produces an approximate residual block size after further failure. The residual block volume V_b^r can be evaluated as

$$If \quad V_b > 10\ cm^3 \quad V_b^r = 10\ cm^3 \tag{3a}$$

$$If \quad V_b < 10\ cm^3 \quad V_b^r = V_b \tag{3b}$$

Residual joint condition

In the GSI system, the joint surface condition is defined by the roughness, weathering and infilling condition. The joint roughness is gradually destroyed during the failure process. However, the residual joint roughness would not be destroyed totally unless the joint experiences a very large shearing displacement. The strain levels in most civil and mining engineering are not large so that the residual joint roughness should not reach to zero. As referred to the investigations of joint wall compressive strength by Barton et al. in 1985, the value of the residual joint roughness may be evaluated as

$$If \quad \frac{J_w}{2} < 1, \quad J_w^r = 1 \quad Else \quad J_w^r = \frac{J_w}{2} \tag{4a}$$

$$If \quad \frac{J_s}{2} < 0.75, \quad J_s^r = 0.75 \quad Else \quad J_s^r = \frac{J_s}{2} \tag{4b}$$

Where J_w is the joint large-scale waviness factor and J_s is the small-scale smoothness factor for joint surface. Furthermore, joint alteration is unlikely to occur in a short time period so that the joint alteration factor J_A will be unchanged in most circumstances. That is, no reduction for J_A. However, In case water or clay infill material is involved, the fractured rock surface, of course, needs to reduce. The residual joint surface condition J_c^r can be obtained from

$$J_c^r = J_w^r \times J_s^r / J_A^r \tag{5}$$

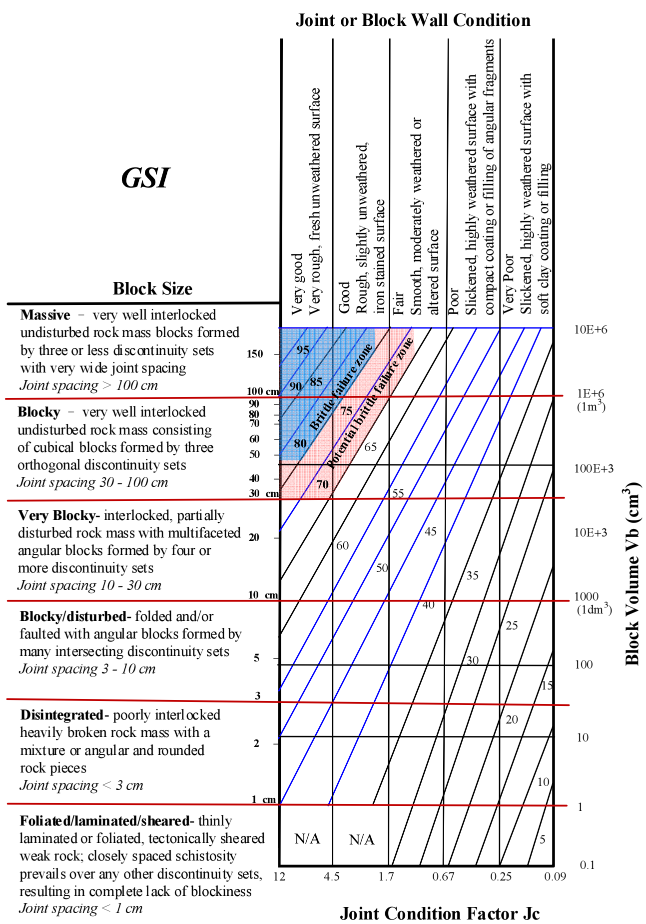


Figure 1 – quantification of GSI chart

Residual GSI estimation

The degradation of the block volume and the joint surface condition for rock mass during tunneling would be graphically demonstrated in Figure 2. According to the logic of the original GSI system, the strength of a rock mass is controlled by its block size and joint surface condition. The same concept is valid for failed rock masses at the residual strength state. In other words, the residual GSI_r is a function of residual J_c^r and V_b^r. Applying the explicitly Equation 2 to rewrite Equation 6 as

$$GSI_r(V_b^r, J_c^r) = \frac{26.5 + 8.79 \ln J_c^r + 0.9 \ln V_b^r}{1 + 0.0151 \ln J_c^r - 0.0253 \ln V_b^r} \tag{6}$$

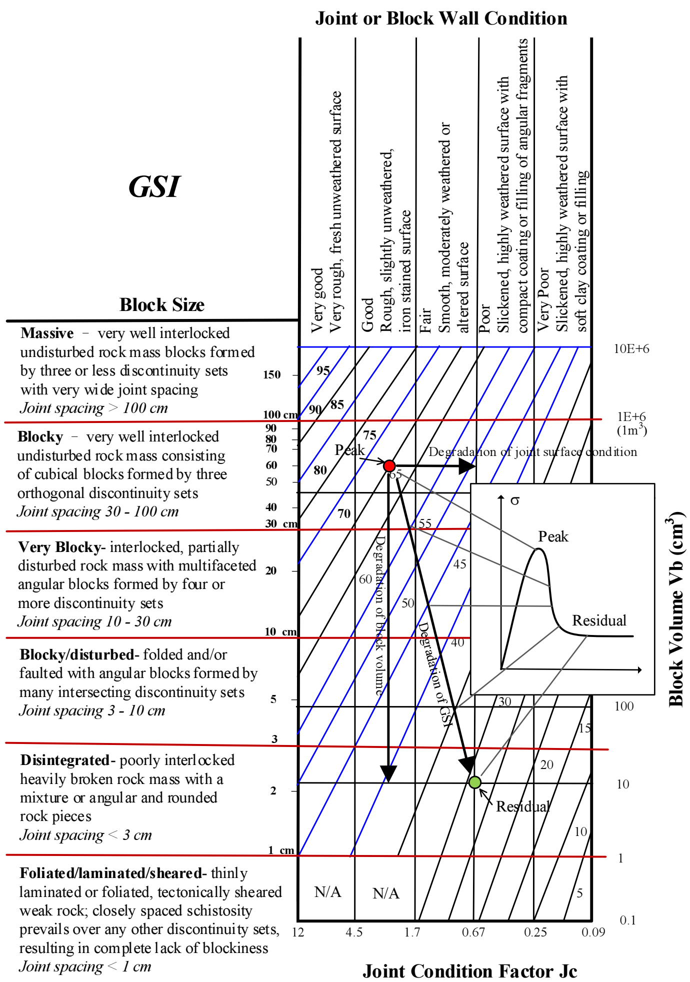
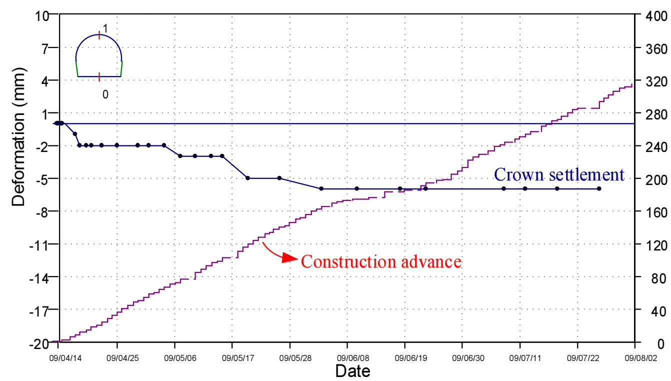


Figure 2 – Degradation of the block volume and joint condition from peak to residual in the GSI chart

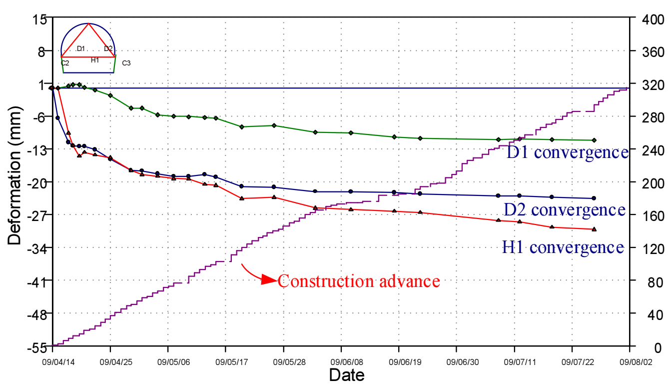
CASE STUDY

A water diversion tunnel in southern Taiwan was adopted as an example to assess the effect of the post-peak strength degradation on deep tunneling stability by using the quantitative residual GSI method. The case tunnel is part of the Tseng-wen Reservoir Transbasin Water Diversion Project, which is to improve the water storage capacity of Tseng-wen Reservoir by diverting surplus water from a neighboring watershed. The tunnel length is total 9.6 km and about 2.9 km long where the rock cover exceeds 800 m. The main rock type along the tunnel is sandstone intercalated by thin layer shale. The uniaxial compressive strength of sandstone is the range of 45 to 100 MPa and the average is around 70 MPa. Bedding planes are the major geological structure, which attitudes are about N20°E/25°NW. The excavation span of the tunnel is 6 m around. The full face D&B method with a cycle length of 2 m was adopted. The support works used including 15 cm thick mesh reinforced shotcrete with systematic rock bolts. The rock bolts are installed in a diameter of 25 mmϕ with the length of 3 m spaced 2.4m×1.5m. About 2500 m long has been excavated for the tunnel and the overburden is around 600 m over there. The monitored crown settlement is about 0.4~0.6 cm and the horizontal convergence is about 2~3 cm, as shown Figure 3. According to the field observation and the monitoring data, tunnel is in stable condition. However, the maximum overburden in the case tunnel will reach to 1300 m, the stability of tunnel construction at depths is concerned.

The evaluation of the peak and the post-peak strength were performed based on the rock mass conditions revealed on the tunnel advancing face, as illustrated in Figure 4. The bedding planes of the sandstone are mostly smooth, have small groundwater infiltration and slightly weathered. No coating and filling are discovered on the surfaces of bedding plane. The average spacing of bedding plane and two major joint sets are 0.75m, 0.5m, and 1.0m, respectively. Accordingly, the block volume V_b is 37500 cm^3, and the joint condition J_c is 0.5. And the peak GSI value is 48 calculated from the Equation 2. The strength parameters estimated from the quantitative GSI system are given in Table 1. Furthermore, the peak GSI value and intact core strength were used to estimate the failure envelope of rock mass and the equivalent c, ϕ values in Mohr-Coulomb criterion. The c value of 2.3 MPa and the ϕ value of 42° are eventually adopted as the peak strength parameters. A numerical simulation using the peak strength was conducted and the results shows that the analytical tunnel crown settlement is about 0.4 cm and the horizontal convergence is about 2.3 cm, both of that are consistent with the monitored results. It is demonstrated that the quantitative GSI system is basically suitable for the case tunnel.



(a) crown settlement



(b) Horizontal convergence

Figure 3 – Monitoring results at the section of 2K+050 for the case tunnel

Figure 4 – Typical rock mass conditions for the case tunnel

Table 1 – Characterization of the rock mass peak and residual strengths for the case tunnel using the GSI system

	Peak	Residual
J_w	1	1
J_s	1	0.75
J_A	2	2
J_c	0.5	0.375
V_b (cm^3)	37500	10
GSI	48	22
C (MPa)	2.3	1.3
ϕ (degree)	42	33

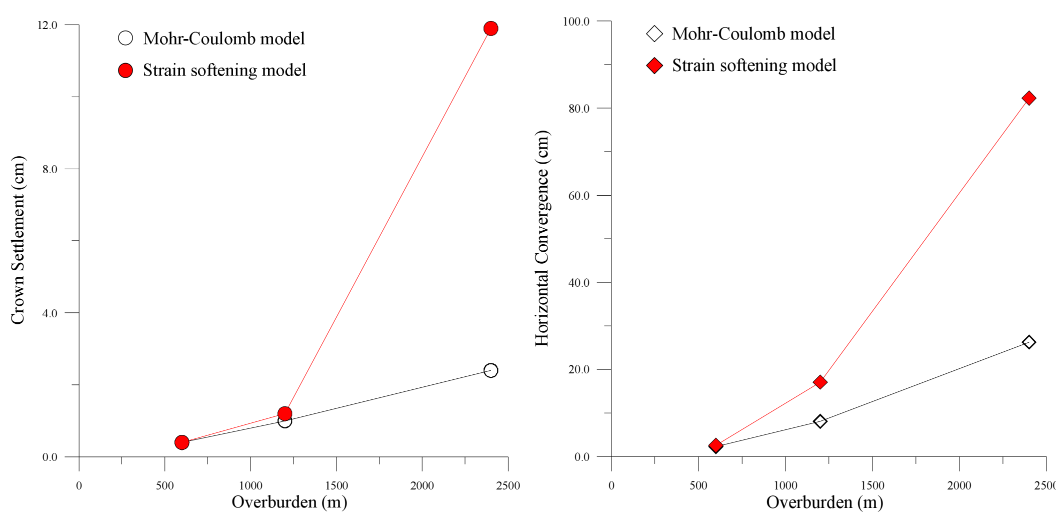
The post-peak strength degradation of the sandstone was further estimated by using the quantitative residual GSI system. The residual block volume of 10 cm^3 is determined from the Equation 3. And the residual joint condition is determined by the possible degradation of joint roughness using the Equation 4 and Equation 5. The results of $J_w{}^r = 1$, and $J_s{}^r = 0.75$ are determined. Which the $J_w{}^r = 1$ instead of $J_w/2 = 0.5$ is because of the minimum constraint on J_w is that it cannot be smaller than 1. The same condition is used for the determination of $J_s{}^r$ value. Moreover, the GSI$_r$ value of 22 is calculated form the Equation 6. Therefore, the equivalent residual Mohr-Coulomb strength parameters, the c_r value of 1.3 MPa and the ϕ_r value of 33°, will be obtained by using the residual GSI$_r$, as shown in Table 1.

To understand the tunneling stability as the depth exceeding 1000 m, a series of numerical analyses, including the Mohr-Coulomb model (no strength degradation, MC model) and the strain softening model (post-peak strength degradation, SS model) within FALC program, were conducted. The residual cohesion and friction angle estimated from the GSI$_r$ value was adopted as the residual strength parameters in the strain softening model. The analysis results are summarized in Table 2. The tunnel deformations, 0.4 cm in crown settlement and 2.6 cm in horizontal convergence, for the case 2 are very close to the conditions with no strength degradation beyond failure. It implies that the post-peak strength degradation is less effect on tunnel deformation at the overburden of 600 m. However, tunneling behavior will be increasingly affected by the strength degradation with increasing tunnel depth. Analytical results shows that approximate 2 times of tunnel deformation increased would occur when the post-peak strength are considered at the depth of 1200 m. Furthermore, approximate 3-5 times of increase in crown settlement and horizontal convergence are found as the SS model compared with the MS model in high overburden stress condition of 2400 m. The dramatic increasing in horizontal convergence might be caused by the horseshoe shape of the case tunnel. Stress concentration will occur at the footing of tunnel sidewall due to the unfavorable excavation shape. In highly overstressed condition, the rock mass at the footing would failed and the post-peak strength degradation would promote the enlargement of horizontal convergence. The correlation between tunnel deformation and overburden depth for the analyses are plotted in Figure 5. From the figure, very small difference in tunnel deformation is found for the MC model and the SS model at the depth of 600 m. However, the effect of the post-peak strength degradation is more and more significant with increasing tunnel depth, especially for the depth exceeding 2000 m. In case the additional reinforcement or the modified excavation measure are not execute, severe tunnel deformation may endanger tunnel stability. The strength degradation beyond failure plays an important role in the stability of deep tunneling.

Table 2 –Effect of overburden depth and post-peak strength degradation on tunnel deformation by using numerical analyses

Case No.	Analysis model	Tunnel overburden (m)	Crown settlement (cm)	Horizontal convergence (cm)
1	MC	600	0.4	2.3
2	SS	600	0.4	2.6
3	MC	1200	1.0	8.1
4	SS	1200	1.2	17.1
5	MC	2400	2.4	26.3
6	SS	2400	11.9	82.3

*：MC model means the Mohr-Coulomb model；SS model means the strain softening model



(a) Crown settlement (b) Horizontal convergence

Figure 5 –Correlation between tunnel deformation and overburden depth for the case tunnel

CONCLUSIONS

Underground excavation in hard rock is normally stable at shallow depth except for wedge failure. However, the excavation depth is rapidly increasing recently. In high overburden stress condition, 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. Based on the Geological Strength Index (GSI) system is widely used in the determination of the rock mass strength, the possible post-peak strength degradation estimated using the quantitative residual GSI system was adopted to assess the construction stability of a deep hydraulic tunnel in Taiwan. The analysis results demonstrate that the tunnel is in stable condition at 600 m of depth. And the effect of the post-peak strength degradation on tunnel deformation is progressively significant with increasing tunnel depth. Approximate 3-5 times of deformation increased would occur when the post-peak strength degradation is initiated at high overburden stress condition of 2400 m. Severe tunnel deformation may endanger the tunnel stability. The modeling shows that the post-peak strength degradation has a great effect on tunnel stability in high overburden stress environment, which should not neglect in deep tunnel design

The issue of rock mass strength degradation beyond failure was seldom studied in Taiwan in the past. The experiences of successful design and effective countermeasure is insufficient for building tunnels in depths of up to 1000m. Therefore, cautious design and systematic monitoring should be fulfilled to overcome the possible problems caused by the post-peak strength degradation in deep underground excavation in Taiwan.

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