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Influence of Rock Properties in Estimating Rock Strength for Shallow Underground Structures in Weak Rocks

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Abstract - Two popular rock strength criteria, the linear Coulomb and non-linear Hoek-Brown, are widely used in underground designs. These two criteria may be applied differently depending on rock conditions. Weak rocks may have different properties compared to hard rocks. Both criteria have been applied in a current research to practically determine the applicability of the criteria in estimating the strength of weak rock masses of five shallow underground structures. Results show that both criteria are able to model the strength of the five weak rock masses, but as expected the criteria provide quite different values for each type of rocks. The strength of rock masses around underground structures depends on uniaxial compressive strength and confinement; but the linear criterion very much depends on shear characteristics of rock materials. Whereas, the non-linear criterion relies on the geological strength index (GSI). Although the GSI may have served practical descriptions for rock masses, some difficulties were found when using the GSI for very weak pyroclastic rocks. The GSI seems to provide underestimated indexes for these rock types. Estimations show that the non-linear criterion may not really exhibit curved strength envelopes rather linear in some sense, for five weak rock masses. Thus in general, when an underground structure is reasonably shallow, has a lack of confinement, and where the shear behaviour dominates rock failures, the linear criterion is more preferable than the non-linear criterion in modelling the strength of weak rock masses.

Keywords: rock property, strength criterion, weak rock, shallow underground structure, shear characteristic

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Introduction

Underground structures require rock strength estimations for stability analyses. The estimations may be applied in practical ways that simple calculations can be easily utilized. Two popular rock strength criteria are commonly applied for the estimations: the Coulomb and Hoek-Brown. However, these two criteria may be used in different rock conditions. The Coulomb is a linear criterion (Labuz and Zang, 2012), mostly applicable for soft, loose, granular rock material. In contrary, the Hoek-Brown is a non-linear criterion (Priest, 2005; Eberhardt, 2012), suitable for jointed hard rock material.

Loose, granular materials, such as soils, usually shear off when they fail (Agustawijaya, 2002). Weak rocks may have similar failure behaviour, which follows a linear envelope of shear strength that depends on cohesion, normal stress, and angle of friction of the Coulomb relation.

According to Agustawijaya (2002), the shearing behaviour of weak rock materials, such as argillaceous rock types, will depend on uniaxial compressive strength and frictional characteristics of each type of rock materials, although for rock masses these characteristics may not solely dictate the strength (Hoek and Brown, 1994; Agustawijaya, 2007). Therefore, any structure design on weak rocks should involve rock properties, particularly when designing an underground structure at very low depths, for those physical properties of rocks may dominate over field stresses. In this paper, the influence of rock properties is investigated for five underground cases by applying two strength criteria.

Review of Strength Criteria

Coulomb Criterion

In terms of major and minor principal stresses, σ_1 and σ_3, the shear strength of weak rocks may be presented as shown in the following linear relation (Agustawijaya, 2011):

$$\sigma_1 = \sigma_{ci} + \sigma_3 \tan^2 \alpha \quad (1)$$

σ_{ci} = uniaxial compressive strength of intact rock material
$\tan\alpha = (1 + \tan^2\phi)^{0.5} + \tan\phi$
ϕ = friction angle

Based on (1), the shearing behaviour of weak rock materials will depend on the parameters of σ_{ci} and φ. For rock masses, not only σ_{ci} and φ influence the strength, but also the size will reduce the strength (Hoek and Brown, 1994; Agustawijaya, 2007). A modified criterion has been, therefore, proposed for determining weak rock mass strength by introducing the empirical constants R and M, representing size and rock material properties, respectively into (1) (Agustawijaya, 2011) as follows:

$$\sigma_1 = R\sigma_{ci} + M\sigma_3 \quad (2)$$

R = ratio of uniaxial compressive strength of rock mass and intact rock (σ_{cm}/σ_{ci})
$M = \dfrac{1 + \sin\phi}{1 - \sin\phi}$
ϕ = friction angle

In Equation 2, the constant R is unity for intact rock, and it should be less than 1 for rock mass, ($\sigma_{cm}/\sigma_{ci}<1$). For disintegrated and decomposed rock masses, the constant R may reduce significantly. Reworked rocks could have an extremely low σ_{cm}, which could be similar to that for dense soils, so that the R parameter could be approaching zero. For weak rocks, the reduction of σ_{ci} could reach over 60% (Agustawijaya, 2007). Thus for rock masses, the R parameter should be less than 0.6, and Agustawijaya (2011) proposed the R values for several limited rock mass conditions, as shown in Table 1.

Table 1. Suggested R values for limited use in weak rocks (Source: Agustawijaya, 2011)

R	Description
1.0	Excellent: Intact rock material
0.2	Good: Massive, few joints or cracks, no significant effect of joints on rock mass
0.02	Poor: Disintegrated, decomposed, intensively weathered rock mass

The parameters R in Table 1 were probably sufficient to model the strength of weak rocks in some conditions (Agustawijaya, 2011); otherwise using Equation 2, the uniaxial compressive strength for rock masses (σ_{cm}) may be obtained by setting the confining stress σ_3 to be zero:

$$\sigma_{cm} = R\sigma_{ci} \quad (3)$$

In a slight different way, the compressive strength of a rock mass can be expressed in terms of frictional parameters, so Equation 1 changes to be, as follows:

$$\sigma_1 = \frac{2c\cos\phi}{1 - \sin\phi} + \frac{1 + \sin\phi}{1 - \sin\phi}\sigma_3 \quad (4)$$

c = cohesion
ϕ = friction angle

Hoek and Brown (1997) adopted the concept of pillar strength, then proposed the concept of a global "rock mass strength" estimated from the Mohr-Coulomb relationship. When the σ_3 in Equation 4 is set to be zero, the σ_{cm} can be, therefore, obtained (Hoek *et al*., 2002), as follows:

$$\sigma_{cm} = \frac{2c \cos\phi}{1 - \sin\phi} \quad (5)$$

The other constant in Equation 2 is M that is the slope of the linear shear strength envelope on the graph of σ_1 and σ_3. This constant may represent intrinsic rock characteristics, as each rock type may have a different M value. Typical M values for weak rocks range from 1.7 for claystone to 4.6 for quartzite (Agustawijaya, 2011), (Table 2).

Table 2. Typical M values for different rock types (Source: Agustawijaya, 2011)

Rock type	M
Claystone	1.7
Mudstone	2.0
Sandstone	2.5
Limestone	3.0
Hard sandstone	3.7
Quartzite	4.6

Hoek and Brown Criterion

According to Hoek and Brown (1994), the failure of hard rock materials follows a non-linear envelope on the graph of major and minor principal stresses:

$$\sigma_1 = \sigma_3 + (m_i\sigma_{ci}\sigma_3 + s_i\sigma_{ci}^2)^{0.5} \quad (6)$$

The constants m_i and s_i represent intact rock characteristics, for which the constant m_i depends upon rock types (Brady and Brown, 1993; Jaiswal and Shrivastva, 2012), and the constant $s_i = 1$ is for intact rock. The constant m_i can be obtained from proper triaxial tests, otherwise using the constant m_i for intact rock in Table 3 (Marinos and Hoek, 2001, 2002).

Table 3. Some m_i values for weak rocks of different rock types (Sources: Marinos and Hoek, 2001, 2002)

Rock type	Group	Rock	m_i
Sedimentary	Clastic	Breccia	19
		Sandstone	17
		Siltstone	7
		Claystone	4
Metamorphic	Foliated	Gneiss	28
		Schists	12
		Phyllites	7
Igneous	Pyroclastic	Breccia	19
		Lapilli	13
		Tuff	8

Equation 6 may be applied for weak rock masses by replacing the constants m_i and s_i with m_b and s (Brady and Brown, 1993; Marinos *et al*., 2005; Hoek and Marinos, 2007):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_2}{\sigma_{ci}} + S\right)^a \quad (7)$$

The strength of rock masses will, therefore, depend upon physical characteristics, such as the degree of weathering, and the structure of rock masses. The constants m_b and s can be estimated from the Geological Strength Index (GSI), valued from 0 to 100, depending on geological rock mass conditions (Brady and Brown, 1993; Marinos *et al*., 2005; Hoek and Marinos, 2007) (Figure 1).

The constants m_b and s calculated from the GSI are, therefore, as follows:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)$$

$$s = \exp\left(\frac{GSI - 100}{9}\right)$$

$$a = 0.65\left(\frac{GSI}{200}\right) \quad (8)$$

The confinement σ_3 plays an important role in deep tunneling. Hoek (2007) suggested the maximum σ_3, (σ_{3max}) for deep tunnels or shallow tunnels where the depth is three times larger than tunnel diameters:

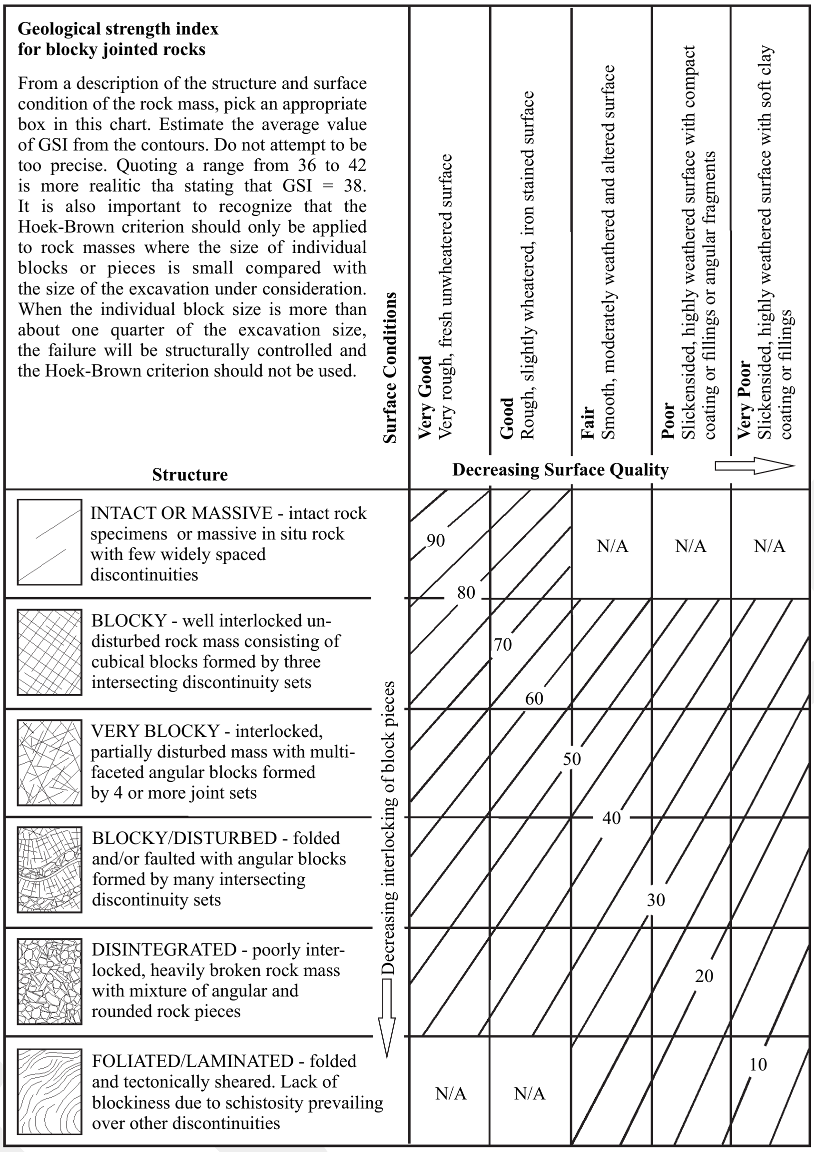

Figure 1. Geological Strength Index (GSI) (Sources: Marinos *et al.*, 2005; Hoek and Marinos, 2007).

$$\sigma_{3max} = \sigma_{cm} 0.47 \left(\frac{\sigma_{cm}}{\gamma H} \right)^{-0.94} \text{ ....} \quad (9)$$

σ_{cm} = uniaxial compressive strength of rock masses
γ = unit weight of rock
H = tunnel depth from the surface

When a tunnel is shallow or near surface, however, shear failure may dominate the behavior of the rock mass around the structure subject to vertical major and horizontal minor principal stresses. Then, shear failure occurs along a plane at an angle of $45^0 + \varphi/2$ to the major principal plane. When the rock mass is assumed to be homogeneous and isotropic, there should develop failure planes in the whole mass equally inclined to the principal planes; subsequently, the minor principal stress may be related to the major principal stress, such that the state of equilibrium is reached when deformation of the mass sufficiently develops (Craig, 1994), as follows:

$$\sigma_3 = (1 + \sin \phi) = \sigma_1 (1 + \sin \phi) - 2c \cos \phi \text{ ... } (10)$$

According to (10), the confinement can, therefore, be calculated:

$$\sigma_3 = \sigma_1 \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) - 2c \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^{-0.5}$$

$$\sigma_3 = \sigma_1 k_a - 2c k_a^{-0.5} \quad \text{.......................................} (11)$$

σ_1 = major principal stress

$k_a = \dfrac{1 - \sin\phi}{1 + \sin\phi}$

c = cohesion

For a shallow tunnel, the major principal stress should be gravity working vertically to the tunnel. So, the major principal stress is a function of unit weight of the rock (γ) and the depth of the tunnel from the surface (H): $\sigma_1 = \gamma H$. As shallow tunnels are often excavated into weak rocks, either sedimentary rocks, or jointed and disintegrated hard rock, the stability analysis of the structures near surface is mostly based on the application of the Coulomb shear strength parameters (Priest, 2005). However, for practical purposes, this paper applies both criteria: linear (Equations 1 - 5), and non-linear (Equations 7 and 8) to examine the applicability of the criteria in estimating the strength of weak rock masses around five shallow underground structures.

Materials and Methods

Two set data of previous underground structures have already been available for analysis from the publications: The Athens Metro tunnel in Greece (Kavvadas *et al*., 1996); and the Desert View Motel in Coober Pedy (Agustawijaya *et al*., 2004). Current investigations have been conducted to three tunnel projects in Lombok Island and Sumbawa Island in Indonesia. Geological surveys and drilling followed by laboratory tests have been conducted to gain rock material properties in regard with the methods suggested by the International Society for Rock Mechanics (1981). Rock descriptions also follow the ISRM suggested methods, and the GSI descriptions are according to Figure 1 (Hoek and Marinos, 2007; Hoek, 2007). Weak rock is subject to the definition given by the ISRM (1981) and Agustawijaya (2007), for which the uniaxial compressive strength of intact rock materials (σ_{ci}) is less than 20 MPa.

Results

Five underground structures were excavated into five different weak rock types. The Athens Metro tunnel project in Greece was excavated into decomposed schist rock (Kavvadas *et al*., 1996). The Desert View underground motel in Coober Pedy, South Australia, was excavated into weathered sandstone rock (Agustawijaya *et al*., 2004). Three current tunnel projects: Pandan Duri in Lombok Island, Mila and Tanju in Sumbawa Island, were still under construction at the time of investigation, and they were excavated into volcanic rock types.

Parameters for rock mass strength estimations were taken from Tables 1 and 2 for Equation 2; while laboratory data of cohesion and friction angle were utilized in Equation 4. The GSI from Figure 1 and parameters m_b, s and a estimated from Equation 8 were applied in Equation 7. Results of rock mass strength estimations of each underground structure are listed in Table 4.

Previous Underground Structures

<u>Athens Metro tunnel in Greece</u>

The Athens Metro tunnel was excavated into poor quality rock masses at shallow depths between 15 and 20 m with a length of 18 km. Rock at this tunnel is completely decomposed schist described as a disintegrated and very poor rock mass. The rock mass is locally known as Athenian schist of Upper Cretaceous flysch-type sediments, which has already been folded and thrusted (Kavvadas *et al*., 1996). The completely decomposed schist may have a GSI value of 20 (Hoek and Brown, 1997).

The schist rock material has cohesion and friction angle in the range of 10 - 60 kPa, and 25 - 280, respectively. The uniaxial compressive

Table 4. Results of rock mass strength estimations for underground structures in five weak rocks

Parameter	Underground structures				
	Athens Metro	**Desert View**	**Pandan Duri**	**Mila**	**Tanju**
Rock	**Decomposed schist**	**Weathered sandstone**	**Volcanic breccia**	**Tuff sandstone**	**Tuff siltstone**
Unit weight, γ (MN/m^3)	0.010	0.013	0.023	0.022	0.015
Depth, H (m)	20	15	22.4	40	20
σ_{ci} (MPa)	10	2.3	2.79	18.7	1.8
σ_3 (MPa)	0.05	0.10	0.13	0.13	0.04
Friction angle, φ^0	28	29	30	35	23
Cohesion, c (MPa)	0.06	0.6	0.04	0.10	0.07
R, (Tabel 1)	0.02	0.2	0.02	0.02	0.02
M, (Tabel 2)	2.0	2.5	3.7	2.5	2.0
M, (2)	2.77	2.88	3.0	3.69	2.28
σ_{cm} (MPa), (3)	0.20	0.46	0.06	0.37	0.04
σ_{cm} (MPa), (5)	0.20	2.04	0.13	0.38	0.21
m_i, (Tabel 3)	12	17	19	13	8
GSI, (Figure 1)	20	45	20	20	20
m_b, (8)	0.69	2.39	1.09	0.75	0.46
s, (8)	0.0001	0.002	0.0001	0.0001	0.0001
a, (8)	0.55	0.43	0.55	0.55	0.55
σ_1 (MPa), (2)	0.29	0.71	0.53	0.71	0.12
σ_1 (MPa), (4)	0.33	2.33	0.51	0.88	0.31
σ_1 (MPa), (7)	0.48	0.99	0.66	1.20	0.20

σ_{ci} = uniaxial compressive strength of rock material; σ_3 = confining stress; R, M = constants; σ_{cm} = uniaxial compressive strength of rock mass; m_i, m, s and a = Hoek-Brown constants; GSI = geological strength index; σ_1 = rock mass strength; (2) - (8) = equation number.

strength of intact rock material (σ_{ci}) ranges from 5 to 10 MPa. Agustawijaya (2011) suggested constants R of 0.02, and M of 2.0 for the decomposed and disintegrated rock mass. Using Equation 2, the estimated rock mass strength is 0.29 MPa; while using Equation 4, the estimated strength is 0.33 MPa. The strength difference of these two estimations is 12%. This may possibly be due to a slight different M value, which are 2.0 for Equation 2 and 2.77 for Equation 4.

Using Equation 7, however, the estimated rock mass strength is 0.48 MPa, which is higher than those obtained from Equations 2 and 4. The parameter m_i of 12 for disintegrated schist may relatively contribute a higher strength calculation compared to those obtained from Equations 2 and 4.

Desert View Motel in Coober Pedy

Agustawijaya *et al*. (2004) reported the geology of Coober Pedy, which comprises the Tertiary-Quaternary Russo Beds and Early Cretaceous marine Bulldog Shale. The Russo Beds are a distinctly weathered, poorly sorted conglomerate; while the Bulldog shale comprises sandstone, siltstone, and claystone. The light brown sandstone of the Bulldog shale is made of fine to medium sand fragments within clay matrix and cement. The Bulldog shale is friable, distinctly weathered, which tends to disintegrate quickly in saturation. The rock formation is generally massive, with minor jointing or faulting.

Results from undrained tests provided the uniaxial compressive strength of intact material of 2.3 MPa, friction angle of 29^0, and cohesion of 0.6 MPa (Agustawijaya *et al*., 2004). According to Tables 1 and 2, parameters R and M are 0.2 and 2.5, respectively. Then, all parameters are put in Equation 2 to estimate rock mass strength of 0.71 MPa for weathered sandstone rock mass. But, when using Equation 4, the strength is almost three times higher than that obtained from Equation 2, for which a relatively high strength value of 2.33 MPa is possibly because of a high σ_{cm} of 2.04 MPa.

The intensively weathered argillaceous sandstone has better structurally rock mass conditions, which has a relatively better GSI of 45. From Table 3, the parameter m_i is 17; then, the estimated parameters m_b, s, and a are 2.38, 0.002, and 0.43, respectively. These parameters are put into the non-linear Equation 7, which estimates reasonably a low rock mass strength of 0.99 MPa, almost 58% lower than that obtained from Equation 4, (2.33 MPa).

Current Tunnel Projects

Pandan Duri Tunnel in Lombok Island

The Pandan Duri tunnel is located in the East Lombok Regency in Lombok Island, Indonesia. Initially, the excavation was to divert river water from the constructed dam to a diversion channel. After construction, the tunnel is used for a water intake conduit for irrigation in the area. The tunnel was excavated into a hill at an average depth of 22.4 m from the top. The excavation has a dimension of width, height, and length: 4.40 x 4.72 x 416.45 m (Figure 2).

Figure 2. Pandan Duri tunnel under construction.

Rock around the tunnel is volcanic breccia of Early Miocene (Mangga *et al*., 1994), comprises andesitic boulder embedded within volcanic mud. The intact rock material has a uniaxial compressive strength of 2.79 MPa, friction angle of 30^0, and cohesion of 0.037 MPa. From Tables 1 and 2, the constants R and M are 0.02 and 3.0, respectively. The σ_{cm} estimated from Equation 5 is 0.13 MPa. The confining pressure working around the tunnel is about 0.17 MPa. By putting all rock parameters into Equations 2 and 4, the estimated strength is, therefore, 0.53 MPa and 0.51 MPa, respectively. Both estimations are very much similar.

The volcanic breccia rock mass has been intensively weathered, and according to Figure 1 it has a low GSI of 20. The parameter m_i for volcanic rock is 19, then the estimated parameters m_b, s and a are 1.09, 0.0001, and 0.55, respectively. All parameters are put into Equation 7 to calculate a rock mass strength of 0.66 MPa. This result shows that, although, the parameter m_i is relatively high for volcanic breccia, the parameter does not really increase the estimated strength, since the rock mass has a low GSI value.

Mila Tunnel in Sumbawa Island

The Mila tunnel was still under construction at the time of investigation (Figure 3). The tunnel is located in the Dompu Regency Sumbawa Island, Indonesia. The tunnel is constructed to connect two dams: the Saneo feeder and the Mila reservoir. This interconnecting tunnel was cut

Figure 3. Mila tunnel under construction.

through a hill at the depths of 15 - 40 m from the top, and has a dimension of width, height, and length: 4.4 x 4.6 x 660 m.

Rock around the tunnel is pyroclastic rocks of Early Miocene (Sudradjat *et al*., 1998), comprise tuff sandstone, and it has a uniaxial compressive strength of 18.7 MPa, friction angle of 35^0, and cohesion of 0.1 MPa. The rock mass constants R and M are 0.02 and 3.69, respectively. The confining pressure working around the tunnel is about 0.17 MPa. Using Equation 2, the rock mass strength is 0.87 MPa. The σ_{cm} estimated from Equation 5 is 0.13 MPa, then using Equation 4 the calculation of rock mass strength results in a value of 0.88 MPa. The tuff sandstone mass has been intensively weathered and disintegrated, and it has a low GSI of 20. The parameter m_i for pyroclastic sandstone (lapilli) is 13 (Table 3); the parameters m_b, s, and a for the rock mass are 0.75, 0.0001, and 0.55, respectively. Equation (7) provides an estimated rock mass strength of 1.20 MPa, which is reasonably high for tuff sandstone.

<u>Tanju Tunnel in Sumbawa Island</u>

The Tanju tunnel was still under preparation for construction (Figure 4). The tunnel is located in the Dompu Regency, Sumbawa Island, Indonesia. Similar function with the Mila tunnel, the Tanju tunnel was designed to have a similar dimension, but with a longer length: 4.4 x 4.6 x 1,700 m.

Rock around the tunnel is tuff siltstone of Early Miocene age (Sudradjat *et al*., 1998). The rock material has a uniaxial compressive strength value of 1.8 MPa, friction angle of 23^0, and cohesion of 0.07 MPa. According to Tables 1 and 2, the rock mass constants R and M are 0.02 and 2.0, respectively. The confining pressure working around the tunnel is about 0.04 MPa. Using Equation 2, the rock mass strength is 0.12 MPa. But, using Equation 5 to gain a σ_{cm} value of 0.21 MPa, the estimated rock mass strength from Equation 4 is 0.31 MPa, which is much higher than that obtained from Equation 2. It seems that a higher σ_{cm} value increases the estimated rock mass strength in Equation 4.

Figure 4. Rock cutting and drilling on the tuff siltstone mass of the outlet face at the Tanju tunnel.

The weathered tuff siltstone mass has already been disintegrated (Figure 4), so the rock mass has only a GSI of 20. The parameter m_i for tuff rock is 8 (Table 3), then the parameters m_b, s and a for the rock mass are 0.46, 0.0001, and 0.55, respectively. These parameters seem to provide a low rock mass strength of 0.20 MPa in Equation 7.

Discussion

From five underground structures, it can be seen that each type of rocks has a different strength value corresponds to rock properties and size. The intact uniaxial compressive strength plays the dominant role in the strength of weak rock masses. Using Equation 4, weathered sandstone at the Desert View Motel has the highest strength value; while tuff siltstone at the Tanju tunnel has the lowest strength value (Figure 5).

Although, the weathered sandstone at the Desert View Motel has low intact uniaxial

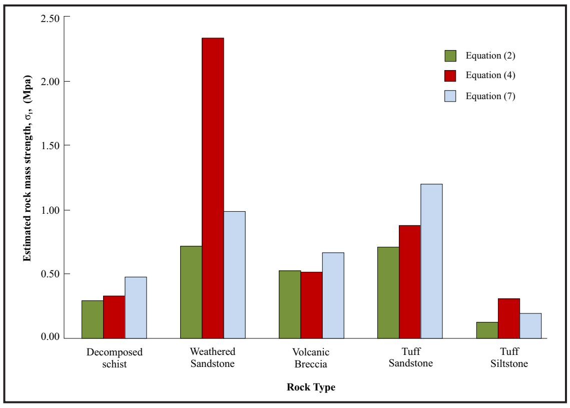

Figure 5. Estimated rock mass strength for various rock types.

compressive strength, this argillaceous rock material has high frictional strength, particularly high cohesion that increases the estimated rock mass strength. This case is different with tuff sandstone and tuff siltstone. Tuff sandstone has the highest σ_{ci}, while tuff siltstone has the lowest σ_{ci}. Comparison between both rocks shows that a difference in σ_{ci} of 17 MPa results in a significant reduce in rock mass strength from 0.31 MPa down to 0.12 MPa, which is about 150%. In terms of strength reduction ratio (σ_{cm}/σ_{ci}), the ratio for four rock types is less than 0.2, except for weathered sandstone, which is 0.86. The σ_{ci} and ratio of σ_{cm}/σ_{ci} certainly play an important role in the estimation of rock mass strength (Yavuz, 2006; Al-Awad, 2012).

The empirical constant M, obtained from typical M values in Table 2, or using frictional estimates of Equation 2, provides a similar result in each estimation. The influence of the parameter M in the shear strength of weak rock around shallow tunnels has been modelled by Stiros and Kontogiani (2009).

Similarly, the function of the parameter m_i in Equation 7 represents rock material characteristics. However, the parameter m_b for rock mass depends on the GSI when it is calculated from the parameter m_i. Figure 1 provides reasonable GSI values, particularly for foliated schist and massive sandstone rocks. Better structurally massive rock will have a higher GSI value that may increase the strength of the rock mass. But, difficulties were found when using the GSI for very weak and sheared rock masses, such as the Athens Schist Formation (Hoek *et al*., 1998). Similar difficulties also arise in the current description of pyroclastic rocks to obtain the GSI values according to Figure 1.

The description of decomposed, disintegrated, blocky, and massive rock masses may have confusing GSI values, they will have low GSI values of below 20 (Figure 6). Volcanic breccia and tuff sandstone are structurally massive, they have the GSI values of 20, instead of >60. Using the GSI value of 20, the rock masses may still have reasonable strength estimation. However, the case of tuff siltstone differs from those rocks. Tuff siltstone at the Tanju tunnel is very soft in drilling, it could have very low rock quality designation (RQD) values (Deere and Miller, 1966; Priest, 1993), even lower than 20%. Laboratory tests also show a very low σ_{ci} value for the rock. The GSI value for this rock mass could as low as 5, but if this GSI value is applied, the estimated rock mass strength will be very low.

Some quantitative approaches have been utilized to gain more exact values by using quantitative methods, such as RQD and block volume (Duran, 2016). Hoek *et al*. (1998) correlated the GSI with cohesion and angle of friction. Such approaches may be valuable as for the case of tuff siltstone at the Tanju tunnel. However, as suggested by Hoek *et al*. (1998) and Marinos *et al*. (2007), the use of the GSI may have some limitations. The GSI will work properly when a rock mass does not have any defining structural feature that controls the behavior of the failure mechanism of the rock (Marinos *et al*., 2005). Thus, engineering judgment at the field is required to put some perspective in rock behavior, particularly for use in underground design. Another way may be to refine the index to gain more representative indexes for pyroclastic rocks, such as methods for flysch rocks proposed by Marinos *et al*. (2007).

However, not only the GSI influences rock mass strength, confinement is also significant when using the non-linear Equation 7. As suggested by Eberhardt (2012), the non-linear

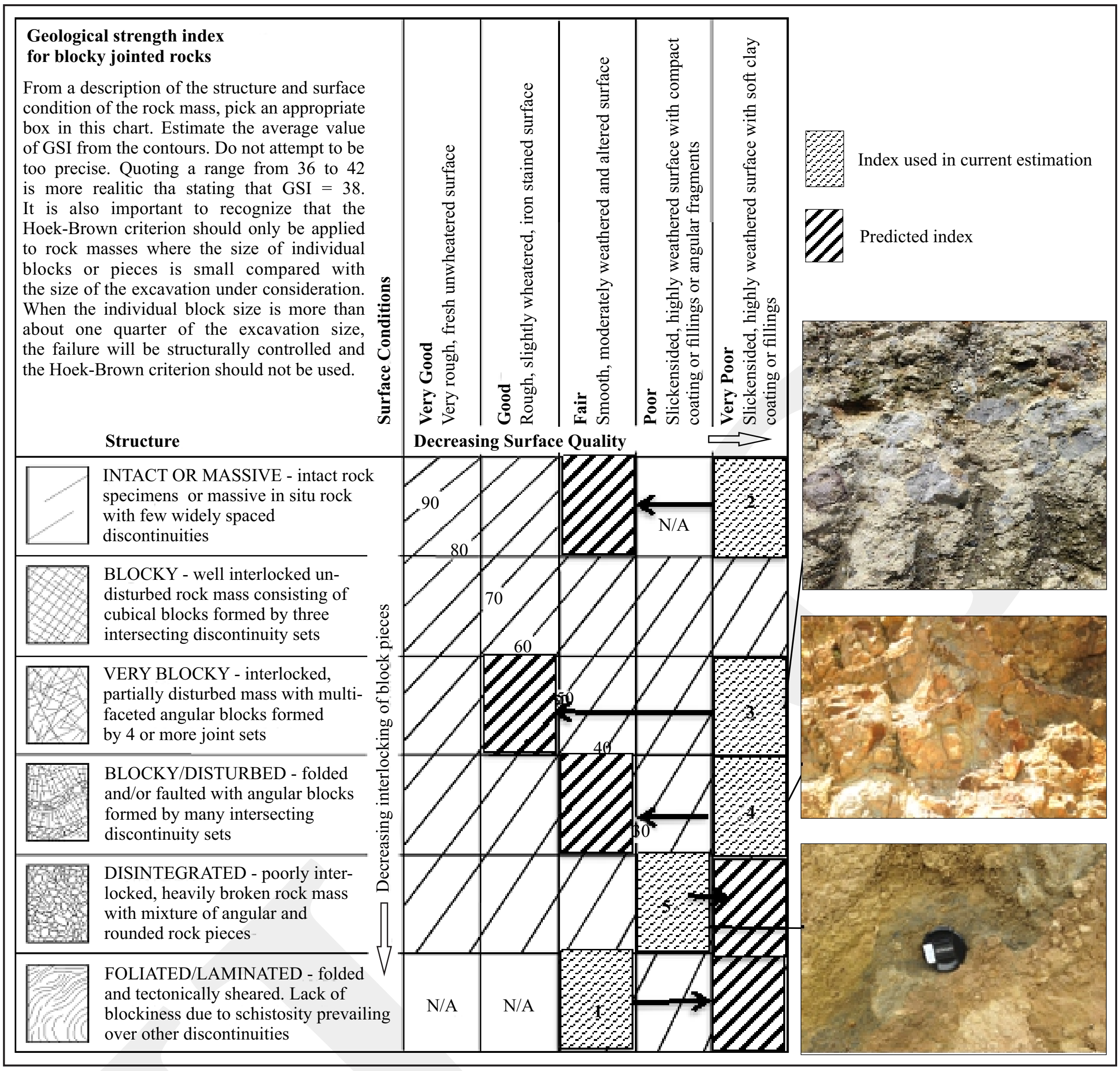

Figure 6. GSI values for five rock types: 1) decomposed schist; 2) weathered sandstone; 3) volcanic breccia; 4) tuff sandstone; 5) tuff siltstone.

Equation 7 is dependent on the confinement, in which the criterion is controlled by the major and minor principal stresses. For example, the Mila tunnel is confined with σ_3 of 0.13 MPa, which results in a strength value of 1.20 MPa. But, under low confinement, the estimated strength data may form linear envelopes on the graph of normalized major and minor stresses (σ_{1n}-σ_{3n}), including data obtained from Equation 7 (Figure 7).

Martin *et al*. (1999a, b) noted when a tunnel in brittle rocks has a lack of confinement, it means that when the difference of far-field maximum and minimum stresses ($3\sigma_v$-σ_3) divided by the uniaxial compressive strength of rock materials (σ_{ci}) is over 0.8, the tunnel will be very hard to support. Although, all five underground structures investigated have the ratio of less than 0.8, the stability of the structures could be subjected to ground subsidence caused by gravity loads. Then, according to Martin *et al*. (2003), when the ratio of σ_{cm}/σ_v is less than 0.25, the plastic yield zone around the tunnel will increase. In this case, the uniaxial compressive strength of the rock mass should be crucial. Thus, in general, the stability of structures near surface will highly depend on rock characteristics.

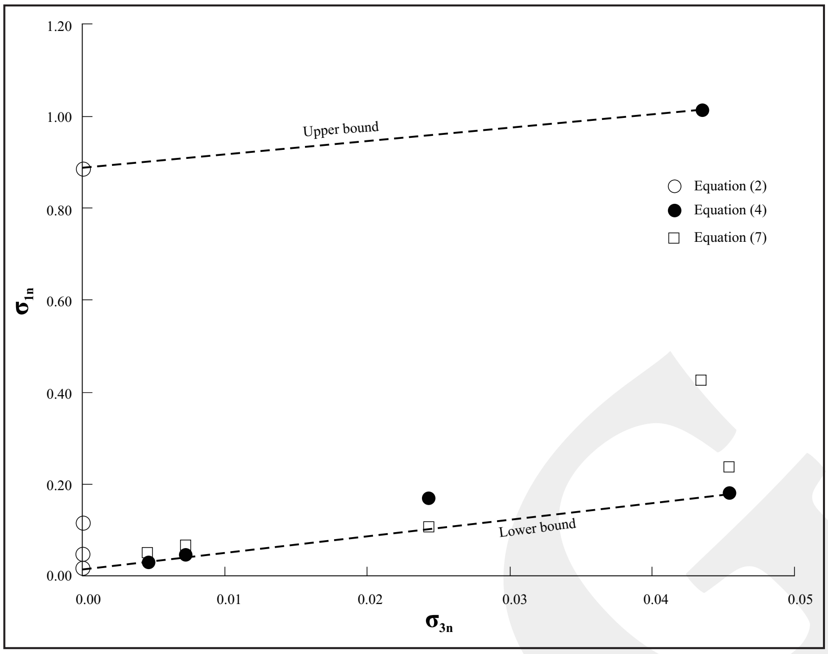

Figure 7. Relation between normalized major and minor stresses.

Conclusion

Two empirical strength criteria, linear and non-linear, provide reasonable different strength values for weak rock masses around shallow underground structures. Although, both criteria are influenced by the uniaxial compressive strength of rock mass (σ_{cm}) and confinement; the linear criterion estimates rock mass strength highly depends on shear characteristics of rock materials; whereas, the non-linear equation depends on the geological strength index (GSI). In general, both strength criteria seem to be sufficiently able to model the strength of weak rock masses. The application at the field of course still requires some engineering judgment for describing the competency of weak rock masses. Particularly, the non-linear equation is much dependent on the index, which in turn the description of rock mass conditions to gain a GSI value is rather subjective, compared to the linear equation depending on laboratory measurements of shear characteristics of rock materials. When an underground structure is excavated into shallow depths, confined with a very low σ_3, and the stability of the structure is due to gravity loads, the linear criterion is, therefore, more suitable, as it facilitates the shear behavior of the rock.

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