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

DIDI S. AGUSTAWIJAYA

Geo-Engineering Research Group, Department of Civil Engineering, Faculty of Engineering, University of Mataram, Mataram 83125, Indonesia

Corresponding author: didiagustawijaya@unram.ac.id
Manuscript received: December 4, 2017; revised: January 10, 2018; approved: March 12, 2018; available online: April 27, 2018

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

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 sheer 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, \( \sigma_1 \) and \( \sigma_3 \), the shear strength of weak rocks may be presented as shown in the following linear relation (Agustawijaya, 2011):

\[
\sigma_1 = \sigma_{ci} + \sigma_3 \tan\alpha \tag{1}
\]

\( \sigma_{ci} \) = uniaxial compressive strength of intact rock material

\( \tan\alpha = (1 + \tan^2\phi)^{0.5} + \tan\phi \)

\( \phi \) = friction angle

Based on (1), the shearing behaviour of weak rock materials will depend on the parameters of \( \sigma_{ci} \) and \( \phi \). For rock masses, not only \( \sigma_{ci} \) and \( \phi \) 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:

\[
\sigma_1 = R\sigma_{ci} + M\sigma_3 \tag{2}
\]

\( R \) = ratio of uniaxial compressive strength of rock mass and intact rock \((\sigma_{cm}/\sigma_{ci})\)

\[
M = \frac{1 + \sin\phi}{1 - \sin\phi}
\]

\( \phi \) = 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 \( \sigma_{cm} \), which could be similar to that for dense soils, so that the \( R \) parameter could be approaching zero. For weak rocks, the reduction of \( \sigma_{ci} \) 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.

| \( R \)   | 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 \((\sigma_{cm})\) may be obtained by setting the confining stress \( \sigma_3 \) to be zero:

\[
\sigma_{cm} = R\sigma_{ci} \tag{3}
\]

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:

\[
\sigma_1 = \frac{2c\cos\phi}{1 - \sin\phi} + \frac{1 + \sin\phi}{1 - \sin\phi} \sigma_3 \tag{4}
\]
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**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 \( \sigma_3 \) in Equation 4 is set to be zero, the \( \sigma_{cm} \) can be, therefore, obtained (Hoek et al., 2002), as follows:

\[
\sigma_{cm} = \frac{2c \cos \phi}{1 - \sin \phi} \quad \text{........................................ (5)}
\]

The other constant in Equation 2 is \( M \) that is the slope of the linear shear strength envelope on the graph of \( \sigma_1 \) and \( \sigma_3 \). 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).

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

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

**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:

\[
\sigma_i = \sigma_1 + (m_i \sigma_0 \sigma_i + s_i \sigma_i^0)^{0.5} \quad \text{.......................... (6)}
\]

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 Shrivastava, 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).

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):

\[
\sigma_i = \sigma_1 + s (m_b \frac{\sigma_1}{\sigma_0} + S)^{0.5} \quad \text{.......................... (7)}
\]

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:

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

\[
s = \exp \left( \frac{\text{GSI} - 100}{9} \right)
\]

\[
a = 0.65 \left( \frac{\text{GSI}}{200} \right) \quad \text{.......................... (8)}
\]

The confinement \( \sigma_3 \) plays an important role in deep tunneling. Hoek (2007) suggested the maximum \( \sigma_3 \) (\( \sigma_{3_{\text{max}}} \)) for deep tunnels or shallow tunnels where the depth is three times larger than tunnel diameters:
From a description of the structure and surface condition of the rock mass, pick an appropriate box in this chart. Estimate the average value of GSI from the contours. Do not attempt to be too precise. Quoting a range from 36 to 42 is more realistic than stating that GSI = 38. It is also important to recognize that the Hoek-Brown criterion should only be applied to rock masses where the size of individual blocks or pieces is small compared with the size of the excavation under consideration. When the individual block size is more than about one quarter of the excavation size, the failure will be structurally controlled and the Hoek-Brown criterion should not be used.

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 $45^\circ + \phi/2$ 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:

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

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

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

$$\sigma_{\text{cm}} = \text{uniaxial compressive strength of rock masses}$$

$$\gamma = \text{unit weight of rock}$$

$$H = \text{tunnel depth from the surface}$$

| Geological strength index for blocky jointed rocks |
|-----------------------------------------------|
| Structure | Surface Conditions | Decreasing Surface Quality |
|----------|-------------------|---------------------------|
| INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities | Very Good | 90 |
| | Very rough | N/A |
| | Rough | N/A |
| | Roughly weathered, iron stained surface | N/A |
| BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets | Fair | 80 |
| | Fairly rough | 70 |
| | Moderately weathered and altered surface | 60 |
| | Very rough | 50 |
| VERY BLOCKY - interlocked, partially disturbed mass with multifaceted angular blocks formed by 4 or more joint sets | Poor | 40 |
| | Poorly weathered | 30 |
| | Highly weathered, iron-stained surface with compact coating or fillings or angular fragments | 20 |
| BLOCKY/DISTURBED - folded and/or failed with angular blocks formed by many intersecting discontinuity sets | Very Poor | N/A |
| | Very rough | N/A |
| | Rough | 10 |
| DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces | Very Poor | N/A |
| | Rough | N/A |
| | Moderate | 10 |
| FOLIATED/LAMINATED - folded and tectonically sheared. Lack of blockiness due to schistosity prevailing over other discontinuities | Poor | N/A |
| | Poor | N/A |
| | Fair | 10 |

Figure 1. Geological Strength Index (GSI) (Sources: Marinos et al., 2005; Hoek and Marinos, 2007).
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\[
\sigma_i = \sigma_i \left( \frac{1 - \sin \phi}{1 + \sin \phi} \right) - 2c \left( \frac{1 - \sin \phi}{1 + \sin \phi} \right)^{0.5} \\
\sigma_i = \sigma_i k_r - 2c k_r^{0.5} \text{ ....................................... (11)}
\]

\[
\sigma_1 = \sigma, \quad k_r = \frac{1 - \sin \phi}{1 + \sin \phi} \quad c = \text{cohesion}
\]

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 (\(\gamma\)) 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 (\(\sigma_{ci}\)) 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**

**Athens Metro tunnel in Greece**

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 (Hock 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...
strength of intact rock material ($\sigma_{ci}$) 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°, 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 $\sigma_{cm}$ of 2.04 MPa.

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

| Parameter          | Athens Metro | Desert View | Pandan Duri | Mila | Tanju |
|--------------------|--------------|-------------|-------------|------|-------|
| Rock               | Decomposed schist | Weathered sandstone | Volcanic breccia | Tuff sandstone | Tuff siltstone |
| Unit weight, $\gamma$ (MN/m$^3$) | 0.010 | 0.013 | 0.023 | 0.022 | 0.015 |
| Depth, H (m)       | 20 | 15 | 22.4 | 40 | 20 |
| $\sigma_{ci}$ (MPa) | 10 | 2.3 | 2.79 | 18.7 | 1.8 |
| $\sigma_3$ (MPa)   | 0.05 | 0.10 | 0.13 | 0.13 | 0.04 |
| Friction angle, $\phi$ | 28 | 29 | 30 | 35 | 23 |
| Cohesion, $c$ (MPa) | 0.06 | 0.6 | 0.04 | 0.10 | 0.07 |
| R, (Table 1)       | 0.02 | 0.2 | 0.02 | 0.02 | 0.02 |
| M, (Table 2)       | 2.0 | 2.5 | 2.5 | 2.5 | 2.5 |
| M, (2)             | 2.77 | 2.88 | 3.0 | 3.69 | 2.28 |
| $\sigma_{cm}$ (MPa), (3) | 0.20 | 0.46 | 0.06 | 0.37 | 0.04 |
| $\sigma_{cm}$ (MPa), (5) | 0.20 | 2.04 | 0.13 | 0.38 | 0.21 |
| $m_i$, (Table 3)   | 12 | 17 | 19 | 13 | 8 |
| GSI, (Figure 1)    | 20 | 45 | 20 | 20 | 20 |
| $m_s$, (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 |
| $\sigma_i$ (MPa), (2) | 0.29 | 0.71 | 0.53 | 0.71 | 0.12 |
| $\sigma_i$ (MPa), (4) | 0.33 | 2.33 | 0.51 | 0.88 | 0.31 |
| $\sigma_i$ (MPa), (7) | 0.48 | 0.99 | 0.66 | 1.20 | 0.20 |

$\sigma_{ci} =$ uniaxial compressive strength of rock material; $\sigma_3 =$ confining stress; R, M = constants; $\sigma_{cm} =$ uniaxial compressive strength of rock mass; $m_i$, m, s and a = Hoek-Brown constants; GSI = geological strength index; $\sigma_i =$ rock mass strength; (2) - (8) = equation number.
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 $m_s$, $s$, 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).

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 $m_s$, $s$ 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...
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°, 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 $\sigma_{cm}$ 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.

Tanju Tunnel in Sumbawa Island

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°, 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 $\sigma_{cm}$ 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 $\sigma_{cm}$ value increases the estimated rock mass strength in Equation 4.

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
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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 $\sigma_{ci}$, while tuff siltstone has the lowest $\sigma_{ci}$. Comparison between both rocks shows that a difference in $\sigma_{ci}$ of 17 MPa results in a significant reduction in rock mass strength from 0.31 MPa down to 0.12 MPa, which is about 150%. In terms of strength reduction ratio ($\sigma_{cm}/\sigma_{ci}$), the ratio for four rock types is less than 0.2, except for weathered sandstone, which is 0.86. The $\sigma_{ci}$ and ratio of $\sigma_{cm}/\sigma_{ci}$ 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 $m_i$ 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 $\sigma_{ci}$ 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...
maximum and minimum stresses ($3\sigma - \sigma_1$) means that when the difference of far-field maximum and minimum stresses ($3\sigma - \sigma_1$) is divided by the uniaxial compressive strength of rock materials ($\sigma_{uc}$) 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 $\sigma_{uc}/\sigma_1$ 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.
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 ($\sigma_{cm}$) 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 $\sigma_3$, 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.

ACKNOWLEDGMENT

The authors would like sincerely to acknowledge for support and access provided by the tunnel contractors: PT Waskita Karya of the Pandan Duri project in East Lombok; and PT Nindya Karya of the Rababaka Kompleks project in Dompu.

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