International Journal of Technology 10(1): 16-26
ISSN 2086-9614

PRACTICAL APPLICATIONS OF STRENGTH CRITERIA IN CIVIL ENGINEERING DESIGNS FOR SHALLOW TUNNELS IN WEAK ROCK

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(Received: January 2017 / Revised: April 2018 / Accepted: December 2018)

ABSTRACT

A number of rock strength criteria are available for use in civil engineering designs. Although considerable uncertainty in geological conditions and variability in rock properties is involved in estimating rock mass strength, it may be necessary to divide the criteria into two categories based on rock failure behavior: linear and non-linear. In this research, both types of criteria are applied to estimate the strength of weak rock masses at five different shallow tunnel sites. The results show varying strength values. The variation in weak rock properties affects the variability of rock strength, depending on the frictional properties for the linear criterion, and on the geological strength index (GSI) for the non-linear criterion. Confinement may also influence both criteria, but the estimated strength of the non-linear criterion is still low for weak rock when the GSI is low. Accordingly, the implication of these variations and uncertainties in rock properties is that the linear criterion may be practically suitable for tunnelling at shallow depths where instability is mostly due to gravity loads. The criterion tends to provide moderate, conservative rock mass strength estimations for this type of tunnel, since shear mechanisms may dominate rock mass failures around it.

Keywords: Shallow tunnel; Shear failure; Strength criterion; Uncertainty; Weak rock

1. INTRODUCTION

A number of strength criteria have been used in civil engineering tunnel design. However, the application of these may be influenced by many uncertainty factors, leading to differences in use. Most uncertainties are caused by the nature of the material (soil and rock), testing conditions and design application (Kulhawy et al., 2001; Prakoso & Kulhawy, 2011; Serra & Miranda, 2013). In current tunnel projects, two rock strength criteria are applied, namely those of Coulomb and Hoek-Brown (Agustawijaya, 2018).

Initially, in 1776 Coulomb proposed a strength criterion based on the shear resistance of masonry and soil (Parry, 1995):

$$S = ca + \frac{1}{n}N \tag{1}$$

where c is cohesion, a is the area of shear plane, N is normal force, and $^1/_n$ is the coefficient of friction. Equation 1 is then changed by a slight mathematical manipulation, changing $^1/_n$ to n = cot ϕ, so the criterion becomes:

$$\tau_f = c + \sigma_n \tan\phi \tag{2}$$

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Permalink/DOI: https://doi.org/10.14716/ijtech.v10i1.2831

Using Equation 2, Mohr gave graphical explanations of stress conditions at failure, although the material in question was steel; Equation 2 is subsequently known as the Mohr-Coulomb criterion in modern geotechnical engineering (Parry, 1995). As the criterion was developed for loose granular materials, the shear strength (τ_f) depends on cohesion (c), normal stress (σ_n) and the friction angle (ϕ). Therefore, the criterion forms a linear envelope on the graph of shear strength against normal stress (Labuz & Zang, 2012; Shen et al., 2012; Hackston & Rutter, 2016).

Agustawijaya et al. (2004), who worked on weak argillaceous rock, indicate that weak rock may have similar failure behavior to soil, in which the shearing behavior of the rock will depend on the frictional characteristics of each type of rock material. The linear criterion is then represented in terms of major and minor principal stresses, σ_1 and σ_3, in order to estimate the strength of weak rock materials:

$$\sigma_1 = \sigma_{ci} + \sigma_3 \tan^2\alpha \tag{3}$$

where σ_{ci} is the uniaxial compressive strength of intact rock material, and $\tan\alpha = (1 + \tan^2\phi)^{0.5} + \tan\phi$. For use in weak rock masses, Equation 3 is modified by introducing the empirical constants ρ and μ (Agustawijaya, 2011):

$$\sigma_1 = \rho\sigma_{ci} + \mu\sigma_3 \tag{4}$$

where ρ is the ratio of the uniaxial compressive strength of the rock mass and intact rock (σ_{cm}/σ_{ci}), $\mu = \frac{1+\sin\phi}{1-\sin\phi}$, and ϕ is the friction angle. In Equation 4, the linear relation between σ_1 and σ_3 will depend on the uniaxial compressive strength ratio ρ, and on the slope μ. The constant ρ is unity for intact rock, and should be less than 1 for rock mass, ($\sigma_{cm}/\sigma_{ci} < 1$). The variability of the constant ρ could be very wide, between 0 and 1, and is scale-dependent. Agustawijaya (2011) studied the variability of the constant ρ for weak rock, and found that for particular massive and jointed or disintegrated soft rock masses, it may have values of 0.2 and 0.02 respectively (Table 1).

Table 1 Suggested ρ values for limited use in weak rock (Agustawijaya, 2011)

ρ	Description
1.0	Intact rock material
0.2	Massive, few joints or cracks, no significant effect of joints on rock mass
0.02	Disintegrated, decomposed, intensively weathered rock mass

However, the application of the values in Table 1 may be limited to rocks that are diametrically opposed in structure: massive and disintegrated. When a rock mass is intensively weathered and disintegrated, the σ_{cm} is extremely low, and could be similar to that of soils. Therefore, the constant ρ may fall significantly, as in the reworked rock mass or residual soils this could approach zero.

The uniaxial compressive strength is clearly influenced by the size of the rock, so the uniaxial compressive strength for rock mass (σ_{cm}) may be obtained from Equation 4 by setting the confining stress σ_3 to zero:

$$\sigma_{cm} = \rho\sigma_{ci} \tag{5}$$

Furthermore, the constant μ in Equation 4 may represent intrinsic rock characteristics. Each rock type may have a different μ value; typical μ values for weak rock were introduced by Agustawijaya (2011). Most weak rock is typically sedimentary; common corresponding μ values range from 1.7 for claystone to 4.6 for quartzite (Table 2).

Table 2 Typical μ values for different rock types (Agustawijaya, 2011)

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

By adopting the Mohr-Coulomb graphical concept, Equation 3 can be expressed in terms of frictional parameters (Parry, 1995), as follows:

$$\sigma_1 = \frac{2c\cos\phi}{1-\sin\phi} + \frac{1+\sin\phi}{1-\sin\phi}\sigma_3 \tag{6}$$

where c is cohesion and ϕ is the friction angle. In the special case where $\sigma_3 = 0$:

$$\sigma_1 = \frac{2c\cos\phi}{1-\sin\phi} \tag{7}$$

where σ_1 is similar to the pillar strength in a tributary area concept (Hoek & Brown, 1997).

The Hoek-Brown criterion is non-linear (Eberhardt, 2012), and is suitable for hard rock material. The failure of hard rock follows a non-linear envelope on the graph of major and minor principal stresses, (σ_1 and σ_3), (Hoek & Brown, 1994):

$$\sigma_1 = \sigma_3 + \left(m_i\sigma_{ci}\sigma_3 + s_i\sigma_{ci}^2\right)^{0.5} \tag{8}$$

The constants m_i and s_i represent rock characteristics. Each type of rock has a different m_i value; harder rock may have a higher m_i value for each different rock type (Brady & Brown, 1993). The constant $s_i = 1$ is for intact rock, and it should be lower than 1 for disturbed or disintegrated rock. If Equation 8 is applied to weak rock masses, the constants m_i and s_i may be replaced by m_b and s (Hoek et al., 2002; Marinos et al., 2005):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b\frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{9}$$

The strength of rock masses will depend on parameters m_b, s and a, which can be obtained from the Geological Strength Index (GSI) proposed by Hoek et al. (2002). The constants m_b and s calculated from the GSI are therefore:

$$\begin{aligned} m_b &= m_i \exp\left(\frac{GSI-100}{28}\right) \\ s &= \exp\left(\frac{GSI-100}{9}\right) \\ a &= 0.65 - \frac{GSI}{200} \end{aligned} \tag{10}$$

When laboratory data for rock constant m_i are not available, the m_i values for typical rock in Table 3 may be used (Hoek et al., 2002).

Table 3 Some m_i values for different weak rock types (Hoek et al., 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

Considering Equation 10, the GSI will control three of the five parameters in Equation 9. In an extreme case when GSI = 0, the parameters will be very low, approaching zero, and parameter a will be 0.65; moreover, parameter m_b will also depend on parameter m_i. The rock mass strength will then depend on σ_3 and σ_{ci}. Thus, in such a situation, confinement σ_3 and σ_{ci} should be important in Equation 9, which could represent the case for tunnelling in weak rock. In particular, for tunnelling at shallow depths, the confinement should be very low; consequently, the tunnel would depend highly on frictional characteristics, and the confinement may be calculated as follows (Agustawijaya, 2018):

$$\sigma_3 = k_a P - 2ck_a^{0.5} \tag{11}$$

where $k_a = \frac{1-\sin\phi}{1+\sin\phi}$, P is overburden stress, and c is cohesion.

Therefore, when designing a shallow tunnel in weak rock, not only uncertainties in rock properties, but also deficient confinement that could trigger instability in the tunnel should be considered. In this way, the estimated rock mass strength could affect the design, so linear Equations 4–7 and non-linear Equations 9–10 are applied in the five cases in this study to identify practical approaches to tunnel design.

2. METHODS

Five tunnel cases were investigated: one case of the Athens Metro tunnel in Greece adopted from Kavvadas et al. (1996), and four cases in Indonesia. The required geological surveys and drilling were conducted at one site on Lombok Island, two sites on Sumbawa Island and one site in Sumatra, followed by laboratory tests to obtain rock material properties. The methods suggested by the International Society for Rock Mechanics (1981) were adopted in the laboratory tests.

The ISRM (1981) provides a definition of weak rock as rock material that has a uniaxial compressive strength (σ_{ci}) of less than 20 MPa. The σ_{ci} of weak rock could fall far below this value, and could be as low as 1.64 MPa (Agustawijaya, 2007). Difficulties may arise in the laboratory testing of such rock, mostly due to laboratory treatment and the nature of the rock (Agustawijaya, 2007; Prakoso & Kulhawy, 2011). Bieniawski (1989) suggests that rock materials that have a σ_{ci} value of below 1 MPa should be treated as soil.

However, since in situ testing of the uniaxial compressive strength of rock masses is difficult to conduct in practice (Hoek & Brown, 1994), and many uncertainties arise in obtaining real values (Prakoso & Kulhawy, 2011), the strength of rock masses is ascertained from modelling based on their σ_{ci} and geological properties (Equations 9 and 10). The GSI, scaled in 10s up to 100, as suggested by Marinos et al. (2005), has considerable potential for use in rock engineering because it permits the manifold aspects of rock to be quantified, enhancing geological logic and reducing engineering uncertainty, particularly for tunnelling in weak rock (Marinos et al., 2006).

3. RESULTS

The Athens Metro tunnel (Case I) was excavated into decomposed schist rock (Kavvadas et al. (1996), while the four current tunnel sites in Indonesia, the Pandan Duri on Lombok Island (Case II), the Mila (Case III) and Tanju (Case IV) on Sumbawa Island, and the Ketaun in Bengkulu (Case V) were excavated into volcanic rock types.

The rock mass strength for all the tunnels was estimated using three different models, two linear and one non-linear. Each of the five tunnel cases has a different rock type, although each of these is categorized as weak rock. The estimated rock mass strength (σ_1) falls into a wide range of values (Table 4 and Figure 1). The lowest σ_1 is 0.12 MPa for tuff siltstone in the Tanju tunnel, while the highest σ_1 is 3.61 MPa for weathered volcanic breccia in the Ketaun tunnel.

Table 4 Results of the rock mass strength of the five tunnels in weak rock

Parameter	Case I	Case III	Case IV	Case V	Case VI
Rock	AM decomposed schist	PD volcanic breccia	ML tuff sandstone	TJ tuff siltstone	KT volcanic breccia
Unit weight, γ (MN/m^3)	0.01	0.023	0.022	0.015	0.023
Depth, H (m)	20	22.4	40	20	68.85
σ_{ci}, (MPa)	10	2.79	18.70	1.80	3.78
σ_3, (MPa)	0.05	0.13	0.13	0.04	0.48
Friction angle, ϕ^o	28	30	35	23	13
Cohesion, c (MPa)	0.06	0.04	0.10	0.07	0.33
ρ (Table 1)	0.02	0.02	0.02	0.02	0.02
μ (Table 2)	2	3.7	2.5	2	3.7
μ (Equation 4)	2.8	3.0	3.7	2.3	1.6
σ_{cm} (Equation 5), (MPa)	0.20	0.06	0.37	0.04	0.08
σ_{cm} (Equation 7), (MPa)	0.20	0.13	0.38	0.21	0.83
m_i (Table 3)	12	19	13	8	19
GSI (Table 4)	20	60	40	20	60
m_b (Equation 10)	0.69	4.55	1.53	0.46	4.55
s (Equation 10)	0.0001	0.012	0.0013	0.0001	0.012
a (Equation 10)	0.55	0.35	0.45	0.55	0.35
σ_1 (Equation 4), (MPa)	0.29	0.53	0.71	0.12	1.84
σ_1 (Equation 6), (MPa)	0.33	0.51	0.88	0.31	1.58
σ_1 (Equation 9), (MPa)	0.48	1.77	2.71	0.20	3.61

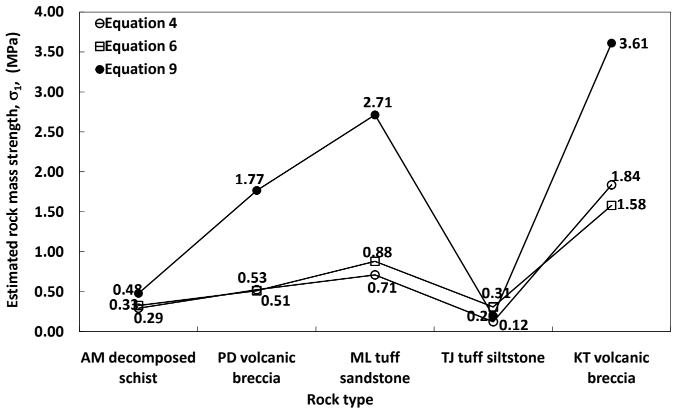

Figure 1 Variations in estimated rock mass strength for five different types of weak rock

A number of parameters influenced the estimated strength. According to Prakoso and Kulhawy (2004), a high number of uncertainties could create variability in estimating rock mass strength. The involvement of each parameter in the current estimations is discussed specifically in each of the following tunnel cases.

Case I

Case I is the Athens Metro tunnel in Greece. The data available from Kavvadas et al. (1996) were analyzed by Hoek and Brown (1997) and Agustawijaya (2011, 2018). The tunnel, with a length of 18 km, was constructed in poor quality decomposed schist rock masses at a depth of 20 m.

According to Kavvadas et al. (1996), the rock material had ϕ of 28^o and c of 0.06 MPa (Table 4). With an intact material parameter σ_{ci} of 10 MPa, and parameters as shown in Tables 1 and 2 (ρ = 0.02, μ = 2.0), using Equation 4 the AM decomposed schist has a σ_1 of only 0.29 MPa, while using Equation 6 the σ_1 is 0.33 MPa. There is only a slight strength difference of 0.04 MPa between the two estimations. Using Equation 9, however, the σ_1 is 0.48 MPa, which is higher than the levels obtained from Equations 4 and 6. It seems that the intact parameter m_i of 12 increases the estimated strength of the rock, although the flysch rock type had a low GSI value of 20 (Kavvadas et al., 1996; Hoek & Brown, 1997).

Case II

Case II is the Pandan Duri tunnel located in the East Lombok District on Lombok Island in Indonesia. The diversion tunnel was excavated into volcanic breccia with an average depth of 22.4 m from the surface, a width of 4.4 m, height of 4.72 m and length of 416.45 m (Figure 2).

Figure 2 Pandan Duri tunnel constructed in volcanic breccia rock mass

The rock around the tunnel is volcanic breccia, with rock parameters σ_{ci} of 2.79 MPa, ϕ of 30^o, and c of 0.04 MPa (Table 3). With rock properties ρ and μ of 0.02 and 3.0 respectively, and using Equations 4 and 6, the σ_1 values are relatively similar, at 0.53 and 0.51 MPa. Estimation using Equation 9 results in a σ_1 of 1.77 MPa, which is higher than those of Equations 4 and 6. This could be due to a GSI value of 60, which increased the σ_1 of Equation 9.

Case III

Case IV is the Mila tunnel located in the Dompu District on Sumbawa Island, Indonesia. The waterway tunnel was excavated into a hill at a maximum depth of 40 m from the top of the hill, with a width of 4.5 m, height of 4.6 m and length of 660 m (Figure 3).

Figure 3 Mila tunnel was excavated into tuff sandstone

Pyroclastic rock is dominant around the tunnel, including tuff sandstone. The rock material has a σ_{ci} of 18.7 MPa, ϕ of 35°, and c of 0.1 MPa (Table 1). Using Equation 4, the σ_1 is 0.71 MPa. The high value of σ_{ci} may result in the relatively high estimated rock mass strength. Equation 6 also results in a high $\sigma_1$of 0.88 MPa, while Equation 9 provides an even higher σ_1 of 2.71 MPa compared to Equations 4 and 6, with the intensively weathered tuff sandstone having a GSI value of 40.

Case IV

Case IV is the Tanju tunnel located in the Dompu District of Sumbawa Island, which has a similar function and dimensions to the Mila tunnel. The tunnel is still under preparation, and it will be constructed in tuff siltstone rock mass at a depth of 20 m. The rock material has a σ_{ci} of 1.8 MPa, ϕ of 23° and c of 0.07 MPa (Table 3), with the lowest σ_{ci} of the six weak rock types. These parameters have consequences for the very low estimated strength rock mass using all three models. Equations 4 and 6 result in σ_1 values of 0.12 MPa and 0.31 MPa, respectively, while Equation 9 also results in a low σ_1 of only 0.20 MPa, which is lower than that of Equation 6. This low strength may be exacerbated by low m_i, GSI and confinement. These parameters do not substantially increase the strength. Thus, the strength may only depend on the frictional characteristics of the rock.

Case V

Case V is a mini-hydro power plant tunnel, which is still at the planning stage for excavation into mountainous terrain in Bengkulu, Sumatra. The rock is part of the Hulusimpang Formation and contains volcanic breccia and tuff sandstone. The tunnel is planned to have dimensions of 4.9×4.85×1900 m^3, constructed at depths of 69-126 m from the surface. The rock is broken volcanic breccia material, with a σ_{ci} of 3.78 MPa, ϕ of 13° and c of 0.33 MPa (Table 4). It has relatively high cohesion compared to the other rock. Since it will be excavated into mountainous terrain, it will be highly confined. Putting these all parameters into the rock strength estimations results in σ_1 values of 1.84 MPa and 1.58 MPa using Equations 4 and 6, respectively. Equation 9 also results in a relatively high σ_1 of 3.61 MPa; however, this value may be sensitive to the GSI, which is 60. The strength of the KT volcanic breccia is the highest among all the five rock types.

4. DISCUSSION

The five tunnel cases show that each type of rock has a different strength value; rock properties, size and confinement contribute to the differences. In the σ_1-σ_3 graph, the estimated strength increases when the confinement increases (Figure 4). The role of confining pressure appears to be significant when using non-linear Equation 9. Taking Case V as an example, the tunnel is confined with 0.48 MPa which results in a σ_1 value of 3.61 MPa, whereas the rock mass around the Tanju tunnel (Case IV) is only confined with 0.04 MPa, which has a σ_1 of only 0.20 MPa. The difference in confinement of 0.43 MPa causes the difference in strength of 3.41 MPa.

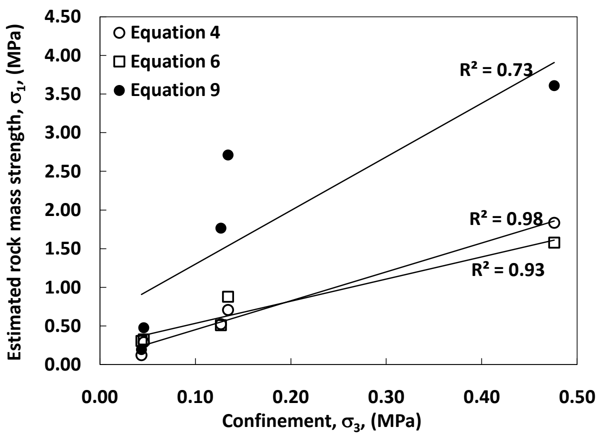

Figure 4 Estimated strength envelopes in the σ_1-σ_3 graph

According to Eberhardt (2012), non-linear Equation 9 is dependent on the confinement, in which the criterion is controlled by the major and minor principal stresses. However, as all five tunnel structures were (or will be) excavated into weak rock at shallow depths, the σ_{ci} plays a dominant role in estimating the strength of the soft rock masses. As for the cases of tuff sandstone in the Mila tunnel and tuff siltstone in the Tanju tunnel, the difference in σ_{ci} of 16.9 MPa between the rock types could result in a significant strength difference of up to 2.52 MPa when using Equation 9.

In terms of strength reduction, the lowering of constant ρ from 0.2 to 0.02 causes considerable strength reduction for weak rock masses. Although the ratio ρ may not provide a reliable estimate, the estimated strength is still reasonably sufficient. The σ_{cm} obtained from the measurement of the cohesion and friction angle seems to provide more reliable results in the estimation (Al-Awad, 2012), particularly when the failure behavior of the rock relies on its shear strength.

The rock material constant μ may represent its intrinsic characteristic. As the parameters μ estimated from Equation 4 for the PD and KT volcanic breccias are significantly different, at 3.0 and 1.6, this could contribute to a strength difference of 61% when using Equation 6, although confinement also contributes to this differentiation. As suggested by Stiros and Kontogiani (2009), parameter μ may represent the shear strength of soft rock material in shallow tunnels.

In terms of rock masses, the GSI may represent rock mass structure conditions. Better structured massive rock will have a higher GSI, which will increase the strength of the rock mass. Marinos et al. (2005) point out that the GSI will work properly when a rock mass does not have any defining structural feature that controls the behavior of its failure mechanism.

However, characterization using the GSI for volcanic rock masses encounters difficulties (Agustawijaya, 2018), similar to those for weak ophiolite rocks (Marinos et al., 2006). Volcanic rock commonly has no significant structure, apart from irregular depositional bedding; boulder fragments are bonded with very weak volcanic mud, causing low strength. Some modification may be required to adapt to structural conditions; for instance, a quantification method may be used (Hong et al., 2017).

As shown in Figure 5, all three equations form different envelopes of the normalized estimated rock mass strength, $\sigma_{1n} = \sigma_1/\sigma_{ci}$, from those depicted in Figure 6. Equation 9 forms a type of non-linear envelope. All the equations provide much better coefficient correlations of R^2 in terms of normalized stresses than those in Figure 6. The standard deviation values for each equation are 19, 15 and 40%, while the coefficients of variation (COV) for Equations 4, 6 and 9 are 0.76, 0.60 and 1.49%, respectively.

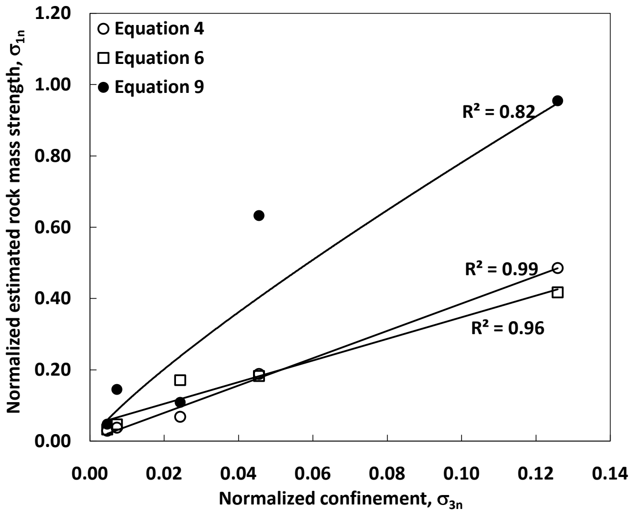

Figure 5 Normalized estimated rock mass strength for the three equations applied

As suggested by Kulhawy et al. (2001) and Prakoso and Kulhawy (2004), the COV values for intact rock indexes will fall over a wide range, particularly for uniaxial compressive strength. Thus, the variability of rock properties greatly affects the estimated rock mass strength, which may then influence the stability analysis of the tunnel design. The instability of shallow tunnels in weak rock may not be only caused by the rock properties, but more importantly also by a lack of confinement, causing ground subsidence due to gravity loads.

In terms of confinement, when the ratio of $(3\sigma_v-\sigma_3)/\sigma_{ci}$ is higher than 0.8, the tunnel will be very hard to support (Martin et al., 1999). Four of the cases in weak rock have ratios of 0.06-0.5, while the Ketaun tunnel (Case V) has a ratio of 1.13. In addition to the ratio, Martin et al. (2003) suggest that the plastic yield zone around the tunnel will increase when the ratio of σ_{cm}/σ_v is less than 0.25; that all five cases have a value higher than this, including the Ketaun tunnel. Thus, regarding these ratios, the stability of all five underground structures is greatly influenced by σ_{cm}, which in turn will depend on the friction characteristics of the rock (Stiros & Kontogianni, 2009). The σ_{cm} is equal to the σ_1 when the σ_3 = 0 MPa, and according to Hoek and Brown (1997), this could be similar to the strength of the pillars in the tributary area concept. Some experimental data show the influence of the intermediate principal stress σ_2 in the strength of rock materials (Priest, 2012); however, in practice the influence of the minor principal stress σ_3 may still be adequate in modelling the strength using the two-dimensional Hoek-Brown criterion of Equation 9.

5. CONCLUSION

Many uncertainties and a high level of variability in rock properties are involved in estimating rock mass strength for underground design. Since in situ estimation of rock mass strength is very difficult to make, modelling consequently relies on intact rock material properties and geological rock mass structures. The estimation of rock mass strength for weak rock using linear and non-linear equations results a wide range of values. The non-linear model is particularly influenced by the GSI, and application in the field requires certain engineering judgment to describe the competency of weak rock masses; otherwise, it may affect the design. For a shallow tunnel design in weak rock, however, the stability of the tunnel may be greatly dependent on the frictional characteristics of the rock. The linear criterion seems to be more suitable from a practical point of view, as it could provide moderate, conservative rock mass strength estimations, and may be less sensitive to subjective indexes.

6. ACKNOWLEDGEMENT

The author acknowledges the support from and access provided by the tunnel project contractors at Pandan Duri (PT. Waskita Karya) and Rababaka Komplek (PT. Nindya Karya). The author also acknowledges the data provided by Dr. Joko Susanto on the Ketaun tunnel.

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