Civil Engineering Dimension, Vol. 13, No. 2, September 2011, 75-81
ISSN 1410-9530 print / ISSN 1979-570X online

The Influence of Rock Properties and Size into Strength Criteria: A Proposed Criterion for Soft Rock Masses

Agustawijaya, D.S.[1]

Abstract: A new modified strength criterion for soft rock masses is proposed in this paper in order to provide a suitable estimation for soft rock mass strength. The new criterion is based upon the current compression test data of soft materials of over 150 samples, and available published data of soft rock strength. It is shown that the proposed criterion estimates reasonable values of soft rock mass strength. Rock properties and size contribute significantly into the strength, represented by friction angle and unconfined compressive strength. Examples exercised reveal that the structure of soft rock masses takes a dominant part in controlling the strength, which then determines the modelled strength of soft rock masses. The results also show that the strength of the proposed equation could relatively be higher three times than the strength of the Hoek-Brown criterion for a massive soft rock mass.

Keywords: Strength criterion, soft rock, rock properties, rock size, model.

Introduction

In engineering design, two popular rock strength criteria are commonly applied in modelling the rock mass behaviour under stresses. The classical Coulomb criterion is usually applied for loose, granular soft materials, which usually shear off when they fail [1]. The shear strength of these materials then forms a linear failure envelope on the graph of shear strength against normal stress, as expressed in the following equation:

$$\tau = c + \sigma_n \tan \phi \tag{1}$$

In this equation, shear strength, τ, depends on cohesion, c, normal stress, σ_n, and friction angle, ϕ. Cohesion and friction angle represent rock characteristics. Granular rock materials tend to have lower cohesion, but higher friction angles than that of clayed rock materials. In terms of major and minor principal stresses, σ_1 and σ_3, the Coulomb criterion can be expressed, as follows [2]:

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

In which:

σ_{ci} = unconfined compressive strength of intact rock material, and

$$\tan\alpha = \left(1 + \tan^2 \phi\right)^{\frac{1}{2}} + \tan\phi$$

[1] Department of Civil Engineering, Faculty of Engineering Mataram University, Jl. Majapahit 62, Mataram-Lombok, INDONESIA
Email: ausaagustawijaya@gmail.com

Note: Discussion is expected before November, 1st 2011, and will be published in the "Civil Engineering Dimension" volume 14, number 1, March 2012.

Received 30 October 2010; revised 11 April 2011; accepted 24 August 2011.

For hard intact rock materials, failure behaviour could be brittle, Hoek and Brown [3] modelled the behaviour as shown in Equation (3). This Hoek-Brown criterion forms a non-linear curve on the graph of major versus minor principal stresses:

$$\sigma_1 = \sigma_3 + \sqrt{m_i \sigma_{ci} \sigma_3 + s_i \sigma_{ci}^2} \tag{3}$$

The constants m_i and s_i depend on rock characteristics. For intact rock, $s_i = 1$, and for aggregate materials $s_i = 0$. The constant m_i varies from rock to rock depending upon rock types [4].

For rock masses, the constants m and s may be obtained from the Rock Mass Rating (RMR) system proposed by Bieniawski [5]. By applying this RMR system, not only rock properties, but also rock size is taken into account in getting m and s values [3]. However, Rock Quality Designation (RQD) and unconfined compressive strength, σ_c, two of the five parameters into the RMR system, are low for soft rock masses, and consequently will result in a low RMR value [6].

For poor weathered rock masses, the RMR system is alternatively replaced with the Geological Strength Index (GSI). This index provides descriptions and indexes for rock mass strength reduction depending upon geological conditions [7]. The estimation of rock mass strength is as follows:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{4}$$

The constants m_b and s can be obtained from the GSI in the following equation:

$$m_b = m_i \exp\left(\frac{GSI-100}{28}\right)$$
$$s = \exp\left(\frac{GSI-100}{9}\right) \quad (5)$$
$$a = 0.65 - \frac{GSI}{200}$$

Based on Equation (5), the strength of rock masses will depend upon physical characteristics, such as the degree of weathering [2], and the structure of rock masses by means the size of rock blocks cutting the rock mass under investigation, represented by the GSI [7]. So, due to the importance of these issues, this paper aims to review the influence of rock properties and size to the strength criteria, and then particularly develop a strength criterion for soft rock masses.

The Influence of Rock Properties

In most engineering cases, rock types are only divided into two groups: soft and hard rocks, based on the value of unconfined compressive strength of 20 MPa [8]. Argillaceous rocks, such as siltstone and sandstone, usually have unconfined compressive strength values below this bench mark [9]. In terms of friction angles, these rocks have commonly a friction angle in the range of 30 – 48^0 [9], which is above that of soils, around 25^0 [10]. In this case, friction angles represent the intrinsic characteristics of the rock. Basic friction angle is different for each type of rocks, depending upon the mineralogy of the rock (Table 1).

Table 1. Basic friction angles for different rock types.

Rock type	Moisture condition	Basic friction angle ($\phi_b{}^0$)
A. Sedimentary rocks		
Sandstone	Dry	26-35
Sandstone	Wet	25-34
Shale	Wet	27
Siltstone	Dry	31-33
Conglomerate	Dry	35
Chalk	Wet	30
Limestone	Dry	31-37
Limestone	Wet	27-35
B. Igneous rocks		
Basalt	Dry	35-38
Basalt	Wet	31-36
Fine-grained granite	Dry	31-35
Fine-grained granite	Wet	29-31
Coarse-grained granite	Dry	31-35
Coarse-grained granite	Wet	31-33
Porphyry	Dry	31
Porphyry	Wet	31
Dolerite	Dry	36
Dolerite	Wet	32
C. Metamorphic rocks		
Amphibolite	Dry	32
Gneiss	Dry	26-29
Gneiss	Wet	23-26
Slate	Dry	25-30
Slate	Wet	21

As can be seen in Table 1, not only the type, but also the moisture content of the rocks influences friction angles. This phenomenon has been recognised for more than three decades [11, 12, 13, 14, 15]. The sensitivity of rock strength to changes in moisture content varies from rock to rock [16, 17, 18]. This influence is frequently associated with capillary suction and crack propagation mechanisms, especially for soft rocks. The critical condition for rocks containing fewer clay minerals is when moisture content increases up to 1%, where a sudden strength loss occurs due to suction, acting as a confining pressure, suddenly disappears in this critical condition. This could be different for rocks that are rich in clay minerals, in which suction disappears gradually up to the degree of saturation of 100% [19].

Clay proportion could increase as the degree of weathering increases. In this relation Johnston and Chiu [20] proposed a criterion for weathered mudstone, which involves two material constants, M and B:

$$\sigma'_{1n} = \left[\frac{M}{B}\sigma'_{3n} + 1\right]^B \quad (6)$$

σ'_{1n} = normalised effective major principal stress (σ'_1/σ'_c)
σ'_{3n} = normalised effective minor principal stress (σ'_3/σ'_c)

Contants M and B are rock material properties that change with water content, w_o, as follows:

$$M = 7.80 - 0.224\,w_o(\%)$$
$$B = 0.772 + 0.0093\,w_o(\%) \quad (7)$$

For soft weathered rock masses, Agustawijaya [2] developed an empirical strength criterion, which was a modification of the Coulomb criterion, as expressed in Equation (8). Weathering parameters are introduced in this modification, represented by a rock mass quality index, CPI, which can be obtained from the soft rock mass classification proposed by Agustawijaya [21]:

$$\sigma_{1n} = R_s + (CPI)\,\sigma_{3n} \quad (8)$$

R_s is the ratio between the unconfined compressive strength for a rock mass and the unconfined compressive strength for an intact rock material being investigated (σ_{cm}/σ_{ci}). The CPI parameter is actually difficult to obtain, as it depends upon field water content affected by environmental conditions [21], although, the parameter might alternatively be replaced by a shrinkage limit parameter [22].

The Influence of Rock Size

Laboratory compression tests are usually conducted on a small sample which may have a diameter of 50 mm, and a length/diameter ratio of at least 2.5:1 [8].

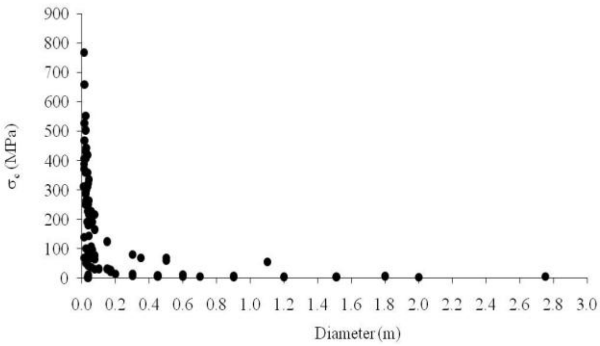

Figure 1. The influence of rock size on unconfined compressive strength.

By applying this ratio, it was found that there was no significant difference in the strength of soft rock materials [19, 23]. Laboratory test results of bigger samples, however, show some differences, as shown in Figure 1 [2]. In this figure, it can be seen that unconfined compressive strength reduces significantly for a diameter of less than 20 cm; but for a diameter of >20 cm, unconfined compressive strength reduces slightly. The maximum reduction could be about 87%. This result is similar to that obtained by Hoek and Brown [7] for sample diameters of 10-200 mm.

Based on Figure 1, the strength of a rock mass could be theoretically lower than that for an intact rock sample, although the reduction may not be linear. The strength will be reduced by the appearance of discontinuities, as they cut apart a rock mass, the rock weakens [6]. In Equations (3) and (4), the reduction is through the constant s that could be down from one to zero for a rock mass. In terms of instantaneous friction angles, the reduction of the constant s could result in the reduction of the friction angle of about 20^0 [24].

Laboratory Tests

Method

Compression tests have been conducted on 153 samples including artificial soft rock materials, soft rocks, and soils (Figure 2). Artificial soft materials were plaster-pumice, and gypsum-pumice mixed materials. The compression tests for soft rocks followed the standard methods given by the International Society for Rock Mechanics [8], and the compression tests for soil materials followed the American Society for Testing and Materials Method [25]. A ratio of length/diameter of 2.5:1 was applied for all samples. Different confining pressures, σ_3, in the range of 0 – 1 MPa, were then applied in triaxial compression tests. Confining pressures were applied gradually, as axial stresses were increased until a hydrostatic pressure was gained, where confining pressures and axial stresses were at the same level. The axial stresses were subsequently increased until the sample failed.

Results and Discussions

Results of unconfined compression tests on artificial soft rock materials show that plaster-gypsum mixed materials had the values of unconfined compressive strength, σ_c, and elastic modulus, E, lower than that of plaster-pumice mixed materials (Tables 2 and 3). The lowest value of unconfined compressive strength (UCS) was 0.06 MPa of gypsum-pumice mixed materials. The average value was about 0.10 MPa, similar to that for soil materials [10]. The highest unconfined compressive value was obtained for plaster – pumice mixed materials, which was 13.54 MPa, similar to that of soft rock materials [9]. The highest elasticity modulus value was obtained for the same materials, which was 3.40 GPa.

Figure 2. Unconfined compression test on plaster-pumice mixed materials

Table 2. Unconfined compression test results for plaster-pumice mixed materials

Material	Sample	UCS (MPa)	*E* Modulus (GPa)
Plaster-pumice	S.100.1	13.54	3.40
Plaster-pumice	S.100.2	7.44	2.23
Plaster-pumice	S.100.3	11.69	3.09
Plaster-pumice	S.50.1	9.74	2.18
Plaster-pumice	S.50.2	8.91	1.60
Plaster-pumice	S.50.3	8.56	1.42
Plaster-pumice	S.40.1	10.31	1.61
Plaster-pumice	S.40.2	5.41	0.89
Plaster-pumice	S.40.3	8.28	1.57
Plaster-pumice	S.30.1	6.00	0.97
Plaster-pumice	S.30.2	6.67	1.40
Plaster-pumice	S.30.3	3.37	0.62
Plaster-pumice	S.20.1	1.19	0.35
Plaster-pumice	S.20.2	0.53	0.15
Plaster-pumice	S.20.3	0.51	0.15
Plaster-pumice	S.10.1	0.13	0.04
Plaster-pumice	S.10.2	0.27	0.09
Plaster-pumice	S.10.3	0.39	0.13

Table 4. Unconfined compression test results for gypsum-pumice mixed materials

Material	Sample	UCS (MPa)	*E* Modulus (GPa)
Gypsum-pumice	GP.100.1	3.35	0.42
Gypsum-pumice	GP.100.2	5.33	0.67
Gypsum-pumice	GP.50.1	0.19	0.03
Gypsum-pumice	GP.50.2	0.27	0.04
Gypsum-pumice	GP.40.1	0.14	0.02
Gypsum-pumice	GP.40.2	0.13	0.02
Gypsum-pumice	GP.30.1	0.07	0.01
Gypsum-pumice	GP.30.2	0.06	0.01
Gypsum-pumice	GP.20.1	0.06	0.01
Gypsum-pumice	GP.20.2	0.20	0.02
Gypsum-pumice	GP.10.1	0.07	0.01
Gypsum-pumice	GP.10.2	0.06	0.01

Results of triaxial compression tests on soft rock materials show that the confining pressures influenced the strength of rock materials. The highest value of triaxial compressive strength (σ_1) of the rock was 15.75 MPa when the confining pressure was 1.5 MPa; and the lowest triaxial compressive value was 0.80 MPa when the confining pressure was 0.0 MPa (Figure 3).

Results of soil tests show the triaxial compressive strength of soil materials reached the highest value of 0.5 MPa when the confining pressure was about 1.33 MPa (Figure 4). The unconfined compressive strength of soil materials was gained at σ_3 equals zero. The unconfined compressive strength values were 0.067 – 0.12 MPa, and the average unconfined compressive strength value was about 0.1 MPa.

Friction angles of soils varied from 20^0 to 27^0, with cohesion values of 0.028 – 0.034 MPa. The average friction angle was 23^0, and the average cohesion value was 0.032 MPa. These values reveal that soils being tested could be classified as silt-sandy soils, as the friction angle was around 25^0, and the cohesion value was relatively low [10].

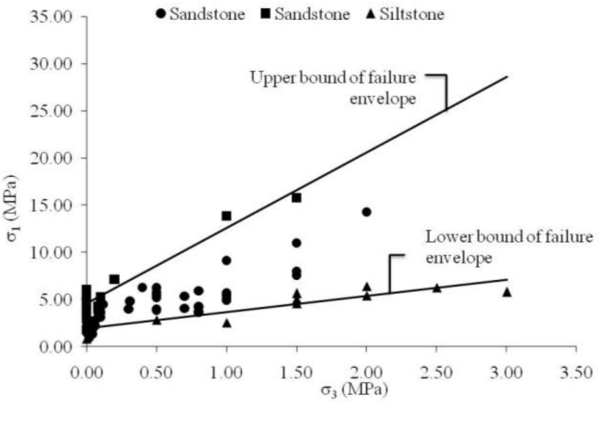

Figure 3. Triaxial compressive strength of soft rocks

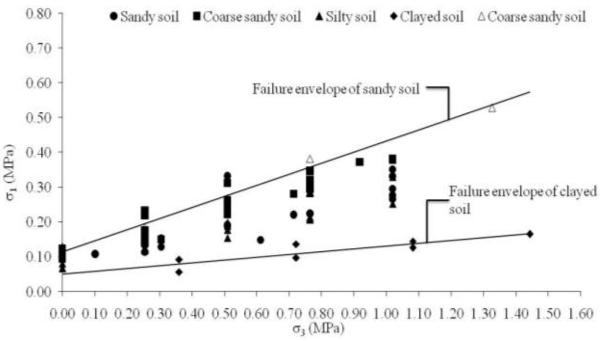

Figure 4. Triaxial compressive strength of soils.

Comparison between data in Figure 3 and Figure 4 shows that both soil and soft rock had similar characteristics in failure, which followed linear curves. The basic difference between soil and soft rock materials is cementation that commonly occurs in rock. Cementation results in higher values of soft rock strength.

Proposed Criterion

As discussed above, material properties and size play an important role in the strength of rock, and they could be well represented in a rock mass strength criterion, such as in Equations (3) and (4). In these equations, the strength of rock masses reduces significantly through the RMR system and the GSI, although, this kind of reduction is not really applicable for soft rock masses. The RMR value for a soft rock mass may be far less than 65, which could result in an inadequate result of strength model [6]. Similarly, the GSI will be below 45 for poor weathered rocks. Thus, soft rocks may need a criterion that represents soft rock properties; and at the same time it should represent the mechanical behaviour of soft rocks that usually performs a soil-liked behaviour, as shown in Figure 3.

In order to develop a strength criterion for soft rocks, it would be appropriate to consider Equation (2). The equation shows linear relation between σ_1 and σ_3, and the slope of the linear envelope is $\tan^2 \alpha$.

$$\sigma_1 = \sigma_{ci} + \mu\sigma_3 \quad (9)$$

σ_{ci} = unconfined compressive strength of intact rock

$$\mu = \tan^2 \alpha = \frac{1+\sin\phi_b}{1-\sin\phi_b}$$

ϕ_b = basic friction angle

Based on this equation, cohesion and friction angle will depend on the $\tan \alpha$ parameter, representing the physical characteristics of the rock. To reduce a wide deviation, Equation (9) may be changed into a normalised form, that all stresses are divided by the unconfined compressive strength of intact rock material (σ_{ci}), as follows:

$$\frac{\sigma_1}{\sigma_{ci}} = \frac{\sigma_{ci}}{\sigma_{ci}} + \mu \frac{\sigma_3}{\sigma_{ci}}$$

$$\sigma_{1n} = 1 + \mu\sigma_{3n} \qquad (10)$$

or

$$\sigma_{1n} = \rho + \mu\sigma_{3n}$$

σ_{1n} = normalised major principal stress
σ_{3n} = normalised minor principal stress
σ_{ci} = unconfined compressive strength
ρ = ratio of unconfined compressive strength

The contant ρ is unity for intact rock, and it should be less than 1 for a rock mass, ($\sigma_{cm}/\sigma_{ci} < 1$). Thus, the constant ρ is scale dependent, as indicated in Figure 1. Based on published data collected by Agustawijaya [26], the ρ values of 0.2 and 0.02 may be adequate for modelling massive and jointed soft rock masses, respectively.

The constant μ represents the basic friction angle, and it should be different for each type of rocks. Typical μ values can be seen in Table 4.

Examples of Application

Hoek and Brown [7] have provided a number of case examples for different types and conditions of rocks. Two of the case examples are used in this paper, which are poor quality rock mass at shallow depth of Athens Metro tunnels, and poor quality rock mass under stress of the Yacambu Quibor tunnel of Venezuela. Another example is provided by Agustawijaya [2] for a siltstone weathered rock mass of an underground motel at Coober Pedy, Australia. Rock properties are listed in Table 5.

In these three examples, Equations (10) and (4) are compared, for which the constants μ and ρ are compared with the GSI for describing rock properties and size. Rock in the first example is completely decomposed schist described as a disintegrated and very poor rock mass. This rock has a friction angle of 22.40, σ_{ci} of 10 MPa, and GSI of 20. Based on the proposed criterion, the rock mass is described as a jointed mass, which has a constant ρ of 0.02.

Table 4. Typical μ values for different rock types

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

Table 5a. Rock parameters for strength criteria

Parameter	Value	Parameter	Value
Rock	Decomposed schist	Rock	Graphitic phyllite
σ_{ci}	10 MPa	σ_{ci}	15 MPa
σ_3 (assumed)	0.046 MPa	σ_3 (assumed)	0.085 MPa
ϕ^0	22.4^0	ϕ^0	24^0
μ	2.2	μ	2.4
ρ jointed	0.02	ρ massive	0.2
m_i	9.6	m_i	10
GSI	20	GSI	24
m	0.55	m	0.66
s	0.0001	s	0.0002
a	0.55	a	0.53
σ_1, Eq (10) jointed	0.30 MPa	σ_1, Eq (10) massive	3.20 MPa
σ1, Eq (4) disintegrated	0.53 MPa	σ_1, Eq (4) disturbed	1.03 MPa

Table 5b. Rock parameters for strength criteria

Parameter	Value
Rock	Weathered siltstone
σ_{ci}	9.02 MPa
σ_3 (assumed)	0.05 MPa
ϕ^0	15^0
μ	1.7
ρ massive	0.2
m_i (assumed)	10
GSI	45
m	1.40
s	0.0022
a	0.43
σ_1, Eq (10) massive	1.98 MPa
σ_1, Eq (4) blocky	0.94 MPa

The compressive strength of this rock mass is estimated to be 0.30 MPa. Using Equation (4), the compressive strength is 0.53 MPa that is for a disitegrated mass. Comparison between these two strength values shows both values below 1 MPa, and these could be due to low values of constants ρ and s of 0.02 and 0.0001, respectively.

Rock in the second example is poor graphitic phyllite (Table 5.a) which had squeezing problems. It seems that this rock has relatively higher strength values than decomposed schist of the first example, as the strengths are above 1 MPa for both Equations (10) and (4). Particularly, the strength of 3.20 MPa is subjected to a relatively high σ_{ci} of 15 MPa and the constant ρ of 0.2 for a massive rock mass. Conversely, a low GSI results in a low strength of Equation (4).

Rock in the third example is massive weathered siltstone which had only few joints. Rock properties of this siltstone mass can be seen in Table 6.b. The compressive strength of this weathered siltstone is around 2.0 MPa estimated from Equation (10). The strength of the third example is still lower compared with the strength of graphitic phyllite mass of the second example, although both rocks are categorised into massive rock masses with a constant ρ of 0.2. The material constant μ of 1.7 and σ_{ci} of 9.02 MPa may contribute to the low strength.

From these three examples, it can be seen that each type of rocks has a different strength value. Rock properties and size have contributed into these differences, and σ_{ci} and ρ seem to play the dominant role in the strength of soft rock masses. Equation (10) estimates reasonable higher values for massive soft rock mass strength compared with the values obtained from Equation (4) for disturbed and blocky rock masses. For completely decomposed schist, the strength behaviour is apparently similar to the strength bahaviour of soils.

In terms of strength reduction, the constant ρ downs from 0.2 to 0.02 causes considerable strength reduction for jointed soft rock masses. Strength reduction from intact to rock mass falls in the range 70-80% when Equation (10) is applied, but it might fall over 90%, when Equation (4) is particularly applied for soft rock masses. However, the strength reduction might not be always the case for every single rock mass, as the strength of rock masses could depend upon the geological conditions of the rock.

Conclusion

A new modified empirical criterion of Coulomb criterion has been proposed in this paper, and the application of the proposed criterion has provided a reasonable result in modelling the strength of soft rock masses. The strength of soft rock masses is significantly influenced by rock properties and size, represented by the constants μ and ρ. The rock size has taken a dominant part in strength reduction, and it is believed that the maximum reduction of around 80% still provides a reasonable value of soft rock mass strength. The application of the proposed criterion at field may, however, still need some engineering judgment for describing the competency of soft rock masses, as in this paper soft rock masses were only grouped into two divisions: massive and jointed.

References

1. Jaeger. J. C. and Cook, N. G. W., *Fundamentals of Rock Mechanics*, 3rd Edition, Chapman and Hall, London, 1979.
2. Agustawijaya, D. S., *The Development of Design Criteria for Underground Excavations in Coober Pedy Arid Soft Rocks*, Ph.D. Dissertation, University of South Australia, 2001.
3. Hoek, E. and Brown, E.T, *Underground Excavations in Rock*, Chapman & Hall, London, 1994.
4. Brady, B. H. G. and Brown, E. T. *Rock Mechanics for Underground Mining*, 2nd Edition, Chapman and Hall, London, 1993.
5. Bieniawski, Z. T., *Engineering Rock Mass Classifications*, John Wiley & Sons, New York, 1989.
6. Priest, S. D., *Discontinuity Analysis for Rock Engineering,* Chapman & Hall, London, 1993.
7. Hoek, E. and Brown, E. T., Practical Estimates of Rock Mass, *International Journal of Rock Mechanics and Mining Science*, 34(8), 1997, pp. 1165-1186.
8. International Society for Rock Mechanics (ISRM), *Rock Characterization, Testing and Monitoring, ISRM Suggested Methods*, Brown, E. T. (Editor), Pergamon Press, Oxford, 1981.
9. Agustawijaya, D. S, Meyers, A., dan Priest, S. D., Engineering Properties of Coober Pedy Rocks, *Australian Geomechanics*, 39(1), 2004, pp. 19-27.
10. Craig, R. F., *Soil Mechanics*, Fifth Edition, Chapman & Hall, London, 1992.
11. Colback, P. S. B and Wiid, B. L., The Influence of Moisture Content on the Compressive Strength of Rock, *Proceedings of the 3rd Canadian Rock Mechanics Symposium*, 1965, pp. 65-83.
12. Chiu, H. K., Johnston, I. W. and Donald, I. B., Appropriate Techniques for Triaxial Testing of Saturated Soft Rock, *International Journal of Rock Mechanics and Mining Science & Geomechanics Abstract*, 20(3), 1983, pp. 107-120.
13. Chiu, H. K. and Johnston, I. W., The Uniaxial Properties of Melbourne Mudstone, *5th Congress of the International Society for Rock Mechanics*, Melbourne, 1983, pp. A209-A214.
14. Hawkins, A. B. and McConnell, B. J., Sensitivity of Sandstone Strength and Deformability to Changes in Moisture Content, *Quarterly Journal of Engineering Geology*, 25, 1992, pp. 115-130.
15. Bell, F. G., *Engineering in Rock Masses*, Butterworth-Heinemann, Oxford, 1994.
16. Schmitt, L., Forsans, T. and Santarelli, F. J., Shale Testing and Capillary Phenomena, *International Journal of Rock Mechanics and Mining Science & Geomechanics Abstract*, 31(5), 1994, pp. 411-427.
17. Dobereiner, L. and DeFreitas, M. H., Geotechnical Properties of Weak Sandstones, *Gêotechnique*, 36(1), 1986, pp. 79-94.
18. Dyke, C. G. and Dobereiner, L., Evaluating the Strength and Deformability of Sandstone, *Quarterly Journal of Engineering Geology*, 24, 1991, pp. 123-134.

19. Agustawijaya, D. S., The Uniaxial Compressive Strength of Soft Rock, *Civil Engineering Dimension*, 9(1), 2007, pp. 9-14.

20. Johnston, I. W. and Chiu, H. K., Strength of Weathered Melbourne Mudstone. *Journal of Geotechnical Engineering*, 110(7), 1984, pp. 875-898.

21. Agustawijaya, D. S., Rock Mass Classification for Soft Rocks, *Journal of Mineral Technology,* JTM, XI(1), 2004, pp. 15-26.

22. Agustawijaya, D. S., Suroso, A. dan Dwiyanti, L., The Quantification of Weathering based on the Shrinkage Limit Parameter of Weathering Products, *Journal of Mineral Technology*, JTM, XIII (1), 2006, pp. 46-57.

23. Matthews, M. C. and Clayton, C. R. I., Influence of Intact Porosity on the Engineering Properties of a Weak Rock, *Geotechnical Engineering of Hard Soils – Soft Rocks*. Anagnostopoulos et al. (Eds.), Balkema, Rotterdam, 1993, pp. 693-701.

24. Hoek, E., Twenty-third Rankine Lecture–Strength of Jointed Rock Masses. *Gêotechnique*, 33(3), 1983, pp. 185-224.

25. ASTM D4767 - 04 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils, 2004.

26. Agustawijaya, D. S., The Development of Strength Criteria of Soft Rocks for Slope Stability Analysis, *Final Research Report*, Directorate of Higher Education, Minister of National Education, 2010.