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NUMERICAL MODELING FOR UNDRAINED SHEAR STRENGTH OF CLAYS SUBJECTED TO DIFFERENT PLASTICITY INDEXES

I-Hsuan Ho [1] and Chung-Ching Hsieh [2]

ABSTRACT

The presence of clay plays a significant role in how it affects the overall engineering behavior of the soil. Hence, the strength of the soil is governed by the clay, especially for the clay with very low strength. Therefore, engineers are required to figure out an appropriate strategy to deal with the engineering problems related to the soil containing the soft clay. It is important to understand the behavior of the clay and make the stress-strain behavior predictable. This paper presents the mechanical behavior of the clay subjected to various plasticity indexes (PI) under different confining pressures. The soil samples were taken from Hsin-Yi district of Taipei city in Taiwan. The Consolidated-Undrained (CU) triaxial tests were performed by controlling the initial void ratio, $e = 1.25$. The three different contents of Bentonite: 3%, 6% and 10% were added in order to change the PI of the clay. Three different confining pressures of 50, 75 and 100 kPa, were applied in the tests. The overconsolidation ratios (OCR = 1.0, 1.25 and 1.5) were also controlled in each test. The undrained shear strength of the clay is therefore addressed based on different PI. Additionally, to investigate the applicable constitutive models for the clay, the modified Cam-Clay (MCC) model and the modified Drucker-Prager/Cap (MDP/Cap) model are employed in the ABAQUS finite element analyses to simulate the stress-strain behavior according to the circumstances created in the laboratory. The results indicate that the MCC model can fit the stress-strain curve better if the PI of the clay is relatively low. For a higher plasticity of clay, the MCC model will overestimate the strength. However, the MDP/Cap model can successfully capture the stress-strain behavior for a higher plasticity of clay. The applicability of these two models used to predict the mechanical behavior of the clay is addressed herein.

Key words: Plasticity index, Modified Cam-Clay model, modified Drucker-Prager model/Cap model, overconsolidation ratio.

1. INTRODUCTION

The weak clay layer was found to play an important role in foundation design. Once clays are present in excess of 10% in the soil, a marked influence on soil's mechanical behavior will be exerted, including the reduction of strength, increase of deformation and the reduction of permeability. Hence, the mechanical behavior of soft clays has to be well understood, particularly for the constructions on the soft clay. In order to explore the undrained behavior of the soft clay, a suitable constitutive model of the soil that can successfully capture the stress-strain behavior has to be employed. Due to the complexity of the soil and the minerology of clay, the elastic soil model is insufficient for explaining its behavior. Thus, many elasto-plastic soil models have been proposed in past decades. Drucker (1956, 1961) proposed an elastic-plastic model for the strain-hardening behavior of a soil. The yield surface and Cap of the proposed model can be extended to capture the strain-hardening portion. Roscoe *et al.* (1958) was thought to be the first to discuss the soil behavior in triaxial tests based on strain-hardening model. In their paper, the state boundary surface (yield surface) and critical state condition are mentioned. Roscoe *et al.* (1963) proposed the intact stress-strain theory for both normally consolidated and slightly overconsolidated soil based on the strain-hardening theory of plasticity. This soil model is the distinguished Cam-Clay model thereafter. Burland (1965) amended the Cam-Clay model solution and proposed the Modified Cam-Clay (MCC) model with Roscoe in 1968. The MCC model can be used to predict the stress-strain behavior of clay on wet side. While the Cam-Clay model overestimated the initial strain of the clay, the MCC model underestimated the initial strain. Vesic and Clough (1968) studied the behavior of granular material under high confining pressure and introduced the critical void ratio concept. When the critical mean stress is reached in shearing, there is no change in volume. The Cam-Clay and MCC models developed at Cambridge University are based on critical state soil mechanics. Because the parameters and calculations are simple, both models are widely used in geotechnical engineering. The critical soil mechanics are developed based on the "wet side" yield surface for normally consolidation or lightly overconsolidated clay (Roscoe and Burland 1968). The yield surface will expand (hardening), accompanied by compression, causing further plastic strains. A new review for the stiffness of natural London Clay was conducted by Gasparre *et al.* (2007). The modified Drucker-Prager/Cap (MDP/Cap) model has been widely used in finite element analysis because of its capability to consider several influencing factors such as the stress history, stress path, dilatancy and the intermediate principal stress. (Helwany 2007). Another new elasto-plastic model call Brick model which can predict some anisotropic behavior was also proposed by Ellison *et al.* (2012).

This paper mainly discusses the undrained shear strength of clays subjected to different plasticity indexes by conducting the conventional triaxial tests and measuring the pore water pressure in shearing. The soil was sampled in the Taipei city area in Taiwan. The soil was classified as the low plasticity clay (CL) ac-

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[1] Clinical Assistant Professor (corresponding author), Department of Civil and Environmental Engineering, Washington State University, Pullman, WA 99164 (e-mail: i-hsuan.ho@wsu.edu).
[2] Engineer, Taiwan Power Company, Kaoshiung, Taiwan.

cording to the Unified Soil Classification System (USCS). In order to change the plasticity index (PI) of the clay, three different percentages of Bentonite (3, 6 and 10%) were added to the clay. The purpose is to compare the stress-strain behavior of Taipei clay subjected to different PI. The conventional triaxial tests as well as isotropic consolidation tests were both conducted where the confining pressures (50, 75 and 100 kPa) and the overconsolidation ratios, OCR (1.0, 1.25 and 1.50) are controlled. The MCC models (ABAQUS 2002) and the MDP/Cap constitutive models are employed to analyze the stress-strain behavior of the clay, as well as compare the numerical analysis results to the conventional triaxial test results to seek the adaptability of these models for these clays. The theory of the numerical models employed and the application of the suitable models to be used for this clay will be addressed as well.

2. TESTING PROCEDURES

In order to obtain the undrained shear strength of clays subjected to different plasticity indexes, the soils were sampled from the Hsin-Yi district in Taipei city and the different contents of Bentonite were added and mixed thoroughly with the clay in measurements of 3, 6 and 10% by weight. The original soil sample is classified as low-plastic clay (CL) in accordance with the USCS. After mixing with Bentonite, the clay exhibits a higher plasticity; the higher the percentage of Bentonite mixed, the higher the PI of the soil will be generated. The clay can therefore be classified as high plastic clay (CH) according to USCS once the Bentonite was added to the original clay. The presence of the Bentonite increases the PI and decreases the undrained shear strength of the clay. The particle size distribution curves of the original clay and the clay mixed with Bentonite are shown in Fig. 1. The index property of these soils and the names of these soils grouped according to the USCS are also summarized in Table 1.

2.1 Sample Preparations

The samples were air-dried and the moisture content was measured between 2 ~ 3%. The different contents of Bentonite were added by weight thereafter. The remold processes was conducted by dividing the sample into five layers to compact. Hence, the initial void ratio could be uniformly controlled in the entire sample. The initial void ratio for all samples tested was controlled at 1.25. The Bentonite and the soil have to be mixed thoroughly in order to generate the uniformity within the plasticity property of the soil sample.

2.2 Consolidated-Undrained Triaxial Test

The Consolidated-Undrained (CU) triaxial tests were conducted using the samples mixed with Bentonite. Three confining pressures: 50, 75 and 100 kPa were applied, respectively. This CU triaxial test is stress-controlled following Tang (2000). The loading rate was controlled as 0.5 kPa per minute and the elapse time (t), deviator stress ($\Delta\sigma_d$), axial strain (ε_a), confining pressure (σ_3), effective confining pressure (σ'_3) and volume change (ε_v) were recorded on the computer.

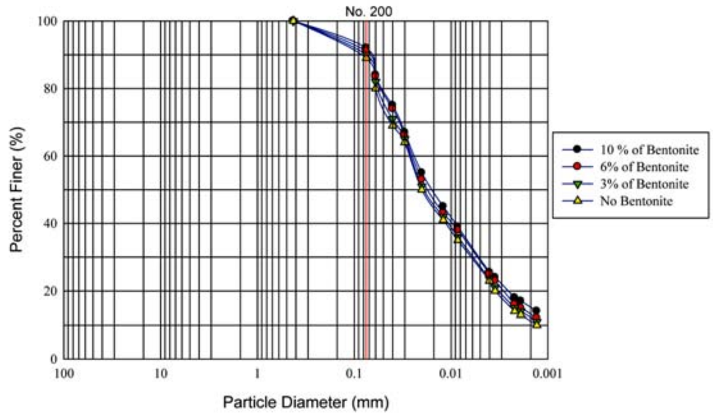

Fig. 1 Particle size distribution curve

Table 1 Physical indexes of the clay containing different amount of Bentonite

Material	LL	PL	PI	USCS
Clay	44.5	25.8	19	CL
Clay + 3% Bentonite	50.5	25.7	25	CH
Clay + 6% Bentonite	56.8	25.6	31	CH
Clay + 10% Bentonite	62	26	36	CH

The specimen must be set up in the device and connected to each LVDT. The device was controlled entirely by the computer program. The back pressure was provided to make the samples fully saturated. The pore pressure parameter, B equal to 0.95 or above, can be regarded as full saturation. The consolidation stage cannot be performed until the soil is fully saturated and usually takes about 12 ~ 24 hours to complete. In the consolidation stage, the isotropic consolidation was conducted; the loading was applied first, then the subsequent unloading and recompression were also performed in order to obtain the parameters λ and κ for the clay. In each loading and unloading stage, the volume change was recorded in one hour intervals. The loading increment has to be adequately selected in order to obtain an uniform and reasonable curve in ν-ln(p') plot. (ν: specific volume; p': mean effective stress)

To control the overconsolidation ratios, OCR equal to 1.0, 1.25 and 1.5, the initial confining pressure has to be applied to the values of 50, 75 and 100 kPa multiplied by the corresponding OCR ratios, and then release to the required confining pressure for testing before the deviator stress is applied.

After the CU tests and the isotropic consolidation tests were conducted, the stress-strain relationship for each test can be obtained. Meanwhile, the material parameters λ and κ to be used in the critical state soil models can be determined as well.

3. TEST RESULTS AND ANALYSES

The experimental results and the selection of the material parameters are discussed, herein. The data analyzed based on the controlled factors such as the Bentonite content, effective confining pressures and the overconsolidation ratios are also presented. In order to find a proper soil model and the suitable material parameters for this clay, the MCC model (ABAQUS 2002), based on critical state theory, as well as the MDP/Cap model were both employed in the finite element analyses. The parameters analyzed based on the MCC model from the experimental results are also summarized in Tables 2 to 5. The critical state soil theory was proposed based on either the normally consolidated or lightly overconsolidated clays by Atkinson (1978). Moreover, the stress strain response of overconsolidated clays depends on their current state and the loading history was observed by Atkinson *et al.* (1990) and Stallebrass and Taylor (1997). Therefore, the OCRs were controlled as 1.0, 1.25 and 1.5, respectively to lower the effects of the loading history.

3.1 Parameters Determination

In order to describe the stress-strain relationships of the soil, three soil parameters are required in the critical state soil models: λ, κ and *M*. The relationship between the deviatoric stress and the mean effective stress is $q_{cs} = Mp'_{cs}$. The relationships between the parameters obtained from the one-dimensional (1-D) consolidation test and the isotropic consolidation test are summarized as follows:

λ: In the ν-ln p' plane, the slope of the normal consolidation curve, the value is about C_c / 2.303, while C_c is the compression index in the conventional 1-D consolidation test.

κ: The slope of the rebound curve in the ν-ln p' plane, the relationship between κ and C_s, $\kappa \approx C_s / 2.303$, while C_s is the swell index from the conventional 1-D consolidation test.

M: The slope of the critical state line, it is associated with the effective internal friction angle of the soil, $M = 6 \sin \phi' / 3 - \sin \phi'$.

3.2 Selection of λ and κ

The determination of λ and κ are based on the isotropic consolidation test. Instead of using conventional 1-D consolidation test, the tests were conducted in triaxial chamber by applying the isotropic confining pressure and releasing the pressure isotropically. A number of loading-unloading stages were performed. The λ and κ are computed from the slopes of the normal consolidation and rebound curves in ν-lnp' plane. The λ values are taken averagely by taking the nonlinear curves shown in the results. One of the examples used to determine λ and κ is illustrated in Fig. 2. The results of the isotropic consolidation tests are plotted by specific volume, v (= 1 + *e*, *e*: void ratio of soil) versus the natural log of the mean effective stress, p' (Fig. 2). Three sets of isotropic consolidation tests must be performed to determine the required λ and κ for the different PI of the clay. The results in Table 2 indicate λ, which describes the plasticity behavior increases along with the Bentonite content mixed in. However, κ describes the elasticity behavior of the clay and decreases with the increase in the PI of the clay. But the discrepancy of either λ or κ in the soils with different PI is very limited. The averaged λ and κ are adopted in the numerical analyses thereafter. The PI of each soil mixed with Bentonite is also presented in Table 2.

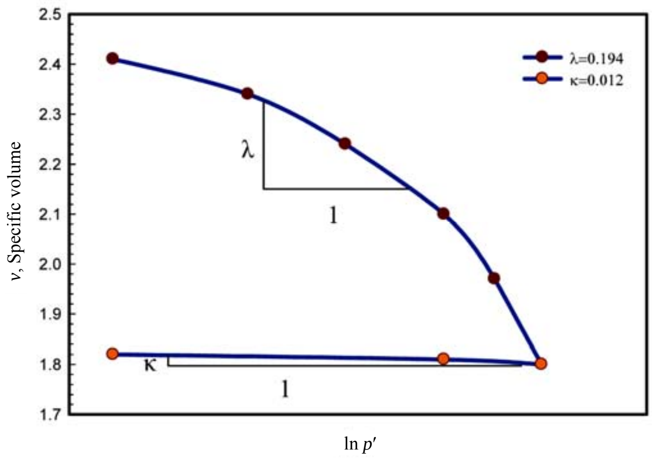

Fig. 2 The isotropic consolidation test used to determine the material parameters on normally consolidated clay (*e* = 1.25, Bentonite = 10%, OCR = 1.00)

Table 2 The isotropic consolidation testing results of λ and κ

Bentonite content	3% (PI = 25)	6% (PI = 31)	10% (PI = 36)
λ	0.1908	0.1923	0.1942
κ	0.0125	0.0123	0.0120

Table 3 The relationships between *M* and effective friction angle ϕ′

Overconsolidation ratio, OCR	1.0								
Void ratio, *e*	1.25								
Bentonite content, %	3			6			10		
Confining pressure, p'_0 (kPa)	50	75	100	50	75	100	50	75	100
M	1.16	1.16	1.15	1.02	1.00	0.99	0.72	0.72	0.71
M_{use}	1.16			1.00			0.72		
Effective friction angle, ϕ′ (°)	29.00			25.5			18.7		
Experimental ϕ′ (axial strain 20%)	29.1			25.4			18.7		
Experimental ϕ′ (axial strain 2.5%)	22.3			19.5			14.2		

* M_{use} is the critical stress ratio used in the numerical analyses

3.3 Determination of the critical stress ratio *M* and internal friction angle ϕ′

From the physical testing of soil, the PI was found to increase subjected to the increase of the Bentonite content in the clay. Moreover, the stress ratio at critical state, *M* and the friction angle, ϕ′ are both related to the strength of soil. The stress ratio defined as $\eta = (q / p')$ attains the critical state, and the parameter η_{cr} is determined as *M*. When the soil in the critical state, there is no plastic volume change and no stress change. By applying the

Table 4 The relationships between *M* and effective friction angle ϕ'

Overconsolidation ratio, OCR	1.25								
Void ratio, *e*	1.25								
Bentonite content, %	3			6			10		
Confining pressure, p'_0 (kPa)	50	75	100	50	75	100	50	75	100
M	1.23	1.22	1.22	1.06	1.03	1.02	0.74	0.74	0.74
M_{use}	1.22			1.04			0.7		
Effective friction angle, ϕ' (°)	30.5			26.2			19.2		
Experimental ϕ' (axial strain 20%)	30.5			26.3			19.2		
Experimental ϕ' (axial strain 2.5%)	23.9			20.7			15.6		

Table 5 The relationships between *M* and effective friction angle ϕ'

Overconsolidation ratio, OCR	1.50								
Void ratio, *e*	1.25								
Bentonite content, %	3			6			10		
Confining pressure, p'_0 (kPa)	50	75	100	50	75	100	50	75	100
M	1.27	1.27	1.26	1.11	1.08	1.08	0.77	0.76	0.75
M_{use}	1.27			1.09			0.76		
Effective friction angle, ϕ' (°)	30.5			27.5			19.8		
Experimental ϕ' (axial strain 20%)	31.9			27.2			19.7		
Experimental ϕ' (axial strain 2.5%)	25.7			21.7			15.7		

deviator stress, the axial strain keeps increasing after the stress reaches the yielding surface (Miura 1984). Due to the tests that were performed as the undrained test, the strength will finally converge to a value, which is regarded as the critical condition being reached. In order to obtain the *M* values directly from the tests, the results plotted in the stress ratio (η) versus the axial strain (ε_a) figures will be used to determine each *M* value corresponding to PI (Bentonite contents) and the overconsolidation ratios (OCR). Figures 3 to 5 present the examples of the three measurements of Bentonite content used in this study for the normally consolidated clay (OCR = 1.0) and determine the critical stress ratio, *M*. Similar tests were also performed to determine the *M* values for the OCR equal to 1.25 and 1.5. The difference observed between each curve by applying different confining pressures (50, 75 and 100 kPa) is very limited and can almost be ignored. The determined critical ratios, *M*, are also summarized in Tables 3 to 5 according to the different conditions applied.

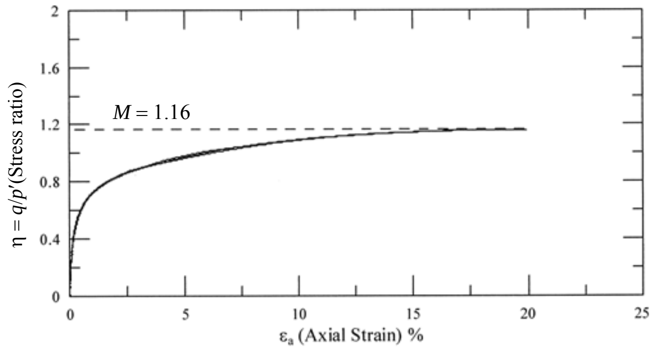

Fig. 3 Relationship between stress ratio η and axial strain ε_a for NC clay obtained from CU triaxial test (e = 1.25, Bentonite = 3%, p' = 50, 75 and 100 kPa)

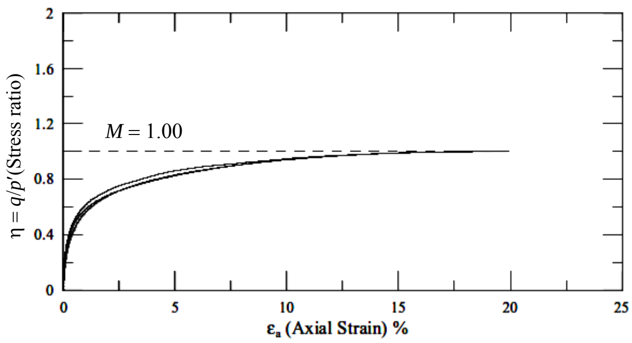

Fig. 4 Relationship between stress ratio η and axial strain ε_a for NC clay obtained from CU triaxial test (e = 1.25, Bentonite = 6%, p' = 50, 75 and 100 kPa)

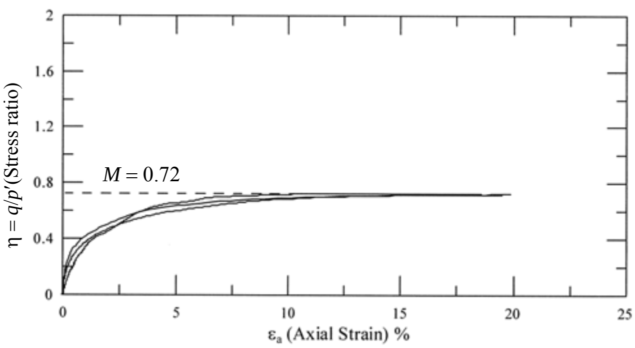

Fig. 5 Relationship between stress ratio η and axial strain ε_a for NC clay obtained from CU triaxial test (e = 1.25, Bentonite = 10%, p' = 50, 75 and 100 kPa)

3.4 Data Analysis

The experimental data are analyzed and summarized in Tables 3, 4 and 5 in accordance with the OCRs of the soil. The effective friction angles are obtained from the back calculations of the critical stress ratio $M_{use.}$ Compared to the experimental friction angles in terms of axial strain 2.5% and 20%, the results show that by selecting the yield point at 20% of strain, the experimental friction angles are very close. If the yield point is selected at 2.5% strain, the effective friction angles from the experiments are 4 ~ 7° lower. The additional strength at the 20% of axial strain can be concluded from the plasticity after yielding.

4. NUMERICAL ANALYSIS USING FINITE ELEMENT METHOD

Besides the laboratory tests that were conducted and the analytical methods used, the finite element methods using ABAQUS were also performed. The analysis results are used to compare with the experimental results. Two types of constitutive models: The MCC (ABAQUS 2002) and the MDP/Cap models are both employed in the numerical analysis. After comparing the results of numerical analysis and the experimental results, the advantages and applicability of each constitutive model to the clay will be discussed.

In the numerical models, a 1 cm by 1 cm axisymmetric element was created following the method proposed by Helwany (2007). In the model (Fig. 6), the y axis is applied the roller to restrict the horizontal movement ($U_x = 0$) and the bottom horizontal surface is restricted in vertical direction ($U_y = 0$). The right-vertical surface and upper-horizontal surface are applied the confining pressures. The porous media element built in the ABAQUS was employed. The elastic portion for both constitutive models is described using the porous elastic built in the ABAQUS. Hence, the material parameter, κ, must be defined in the analysis. The saturated unit weight is set as 20 kN/m^3. The strain rate for the upper-horizontal surface was controlled to be 0.0005 per step in the vertical direction, and 100 sub-steps were generated. The solutions converge at 5% of the vertical strain. The creep of the soil is not considered in either of the constitutive models. The parameters used will be described for each of the models used as follows.

4.1 Modified Cam-Clay Model

The formulations for the MCC model (Roscoe and Burland 1968) are based on the triaxial stress condition that the intermediate and the minor principal stresses are equal ($\sigma_2 = \sigma_3$). The theory was developed at Cambridge University based on the critical state theory of soil mechanics.

The basic characteristics of the model are summarized as follows,

1. The material is isotropic.
2. The yield surface is ellipse and yield behavior is dependent of the mean effective stress, p'.
3. The critical state line separates two regions of the behavior: The dry side for softening behavior and the wet side for hardening behavior.
4. The hardening/softening behavior is a function of the volumetric plastic strain.
5. The yield surface is dependent on the intermediate principal stress.

The yield surface and the critical state line are shown as Fig. 7. The M can be calculated related to the internal friction angle of the soil as the equation mentioned earlier, $M = 6\sin(\phi'/3) - \sin\phi'$. The λ and κ used are summarized in Table 2 according to the different plasticity indexes (Bentonite contents). The Poisson's ratio for this clay regardless of PI is 0.4. The wet yield surface size and the flow stress rate selected in the ABAQUS model are both 1.0. In the p'-q plane, the yield surface of the MCC model is an ellipse

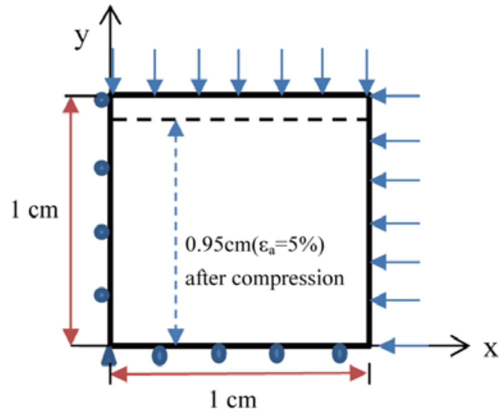

Fig. 6 Finite element model revised from Helwany (2007)

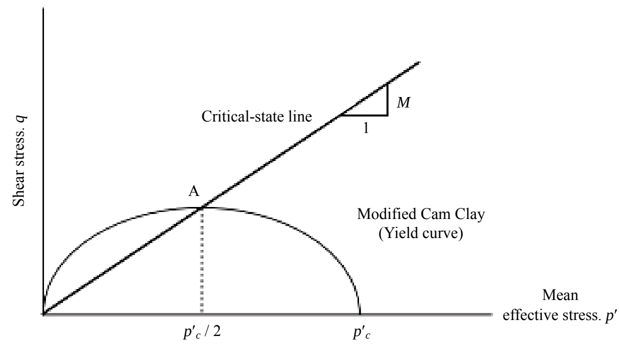

Fig. 7 Yield surface of the modified Cam-Clay model in p'-q plane (Helwany 2007)

and can be expressed as Eq. (1).

$$\frac{q^2}{p'^2} + M^2\left(1 - \frac{p'_c}{p'}\right) = 0 \tag{1}$$

The overconsolidation ratio (OCR) can be expressed in terms of the mean effective stress, p'_0 and yield stress, p'_c (mean preconsolidation pressure) for isotropically consolidated soil as Eq. (2).

$$OCR = \frac{p'_c}{p'_0} \tag{2}$$

4.2 Modified Drucker-Prager/Cap

The Drucker-Prager/Cap model is well known for being used in finite element analysis programs for geotechnical engineering applications. This model is intended to simulate the constitutive response of cohesive geomaterials. A "Cap" yield surface is added to the linear Drucker-Prager model. The two purposes of the Cap are to bind the model in hydrostatic compression as well as help control the volumetric dilatancy. The MDP/Cap model is employed in ABAQUS to simulate the stress-strain behavior of the clay and compare with the results simulated using other soil models such as the MCC model. The failure surface of the MDP/Cap plasticity is composed of two portions: The Drucker-Prager failure surface and the Cap yield surface. The Drucker- Prager failure surface is written as Eq. (3). The yield surface in the *p*-*t* plane is shown in Fig. 8.

$$F_s = t - p\tan\beta - d = 0 \tag{3}$$

where t is the deviatoric stress measure, β is the friction angle of the soil and d is the cohesion in the *p*-*t* plane used in the Drucker-Prager model. For the triaxial stress conditions, the Mohr-Coulomb parameters can be converted to Drucker-Prager parameters using the following two equations.

$$\tan\beta = \frac{6\sin\phi'}{3-\sin\phi'} \tag{4}$$

$$d = \frac{18c'\cos\phi'}{3-\sin\phi'} \tag{5}$$

The MDP/Cap plasticity model is used to simulate geological materials that exhibit pressure-dependent yield. The important parameters defined are simply described as follows:

$$t = \frac{1}{2}q\left[1+\frac{1}{K}-\left(1-\frac{1}{K}\right)\left(\frac{r}{q}\right)^3\right] \tag{6}$$

the equivalent pressure stress $p = -1/3(\sigma_1+\sigma_2+\sigma_3)$

the Mises equivalent stress $q = \sqrt{\frac{3}{2}S:S}$

the third stress invariant $r = \left(\frac{9}{2}S:S\cdot S\right)^{\frac{1}{3}}$

$K = 1.0$, the flow stress ratio to control the shape of the yield surface on the π plane (ABAQUS 2002).

The Cap yield surface function, F_c, and the Drucker-Prager shear failure surface, F_t, are given as the following equations:

Cap yield surface

$$F_c = \sqrt{(p-p_a)^2+\left(\frac{Rt}{1+\alpha-\alpha/\cos\beta}\right)^2} - R(d+p_a\tan\beta) = 0 \tag{7}$$

D-P shear failure surface

$$F_t = \sqrt{(p-p_a)^2+\left[t-\left(1-\frac{\alpha}{\cos\beta}\right)(d+p_a\tan\beta)\right]^2} - \alpha(d+p_a\tan\beta) = 0 \tag{8}$$

where R is a material parameter that controls the shape of the Cap as shown in Fig. 8. α is a small number between 0.01 to 0.05 that defines a smooth transition surface from the Drucker-Prager shear failure surface to the Cap, p_a is an evolution parameter that controls the hardening/softening behavior for the soil and p_b is the mean yield effective stress. The parameter p_a can be calculated related to p_b in the form of the following equation

$$p_a = \frac{p_b - Rd}{1+R\tan\beta} \tag{9}$$

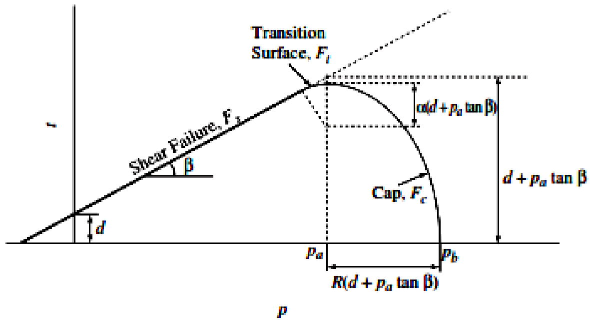

Fig. 8 Yield surface of the modified Cap model in the *p*-*t* plane (ABAQUS 2002)

The function $p_b = p_b(\varepsilon_{vol}^{pl})$ is employed to describe the hardening behavior of the soil. The Cap-hardening curve is obtained from the isotropic consolidation test results and can be calculated as follows:

$$\varepsilon_{vol}^{pl} = \frac{\lambda-\kappa}{1+e_0}\ln\frac{p'}{p'_0} = \frac{C_c - C_s}{2.3(1+e_0)}\ln\frac{p'}{p'_0} \tag{10}$$

In this paper, the results of the isotropic consolidation are summarized in Table 2. The percentage of Bentonite mixed with the soil exhibits a slight difference for the parameters λ and κ. The experimental results of the M values were used to calculate the internal friction angle of the Mohr-Coulomb model using $M = 6\sin(\phi'/3) - \sin\phi'$, and then covert the internal friction angle to the equivalent Drucker-Prager friction angle using Eq. (4). Due to the CU test that was performed, a very small cohesion parameter, d, was used in the numerical analysis. The transition surface radius, α, is 0.05 and the flow stress ratio, K, is 1.0. Under these circumstances, the various confining pressures, 50, 75 and 100 kPa were applied. The material parameter, R, depends on the plasticity of the soil as well as the overconsolidation ratio. As for the plastic potential, it follows the non-associated flow rule, therefore, the unsymmetric solver had to be selected in the numerical analysis.

4.3 Analysis Results

After the MCC and the MDP/Cap plasticity are conducted in the numerical analysis, the stress-strain curves are plotted and analyzed. The analysis results are discussed based on the OCRs (OCR = 1.0, 1.25 and 1.5) as follows.

Normally Consolidated Clay (OCR = 1.0)

The results presented in Fig. 9 are the experimental data obtained by conducting the triaxial compression tests under the confining stresses of 50, 75 and 100 kPa for normally consolidated clay (OCR = 1.0), where the Bentonite content is 3%. Figure 9 also shows the predictions of the same soil (OCR = 1.0 and Bentonite content 3%) using the MCC model and the MDP/Cap model. The results show that the MDP/Cap model predicts the stress-strain behavior of the clay very well using R = 0.8. Meanwhile, the prediction using the MCC model is a little lower than the test data. Figure 10 shows the experimental data for the NC clay (OCR = 1.0) mixed with 6% Bentonite as well as the data from the numerical analyses under the same conditions. The PI of the soil increases to 31, and the ultimate deviatoric stress decreases a little bit compared to the soil mixed with 3% Bentonite. In Fig. 10, it also shows the predictions using different R (0.8 and 1.2) in the numerical analyses and it was found that the $R = 0.8$ will overestimate the strength of the soil, while $R = 1.2$ can make good agreements with the experimental stress-strain curves under the three different confining pressures. In addition, the MCC model has to reduce the stress ratio from the M values around 1.0 to M values equal to 0.80 to better capture the stress-strain curves. Hence, based on $M = 0.8$, the friction angle is back calculated as 20.7° and is about 4.8° lower than the experimental data. The results of triaxial tests for the NC clay mixed with 10% Bentonite are shown in Fig. 11, The NC clay mixed with a higher percentage of Bentonite exhibits a higher PI and a lower strength compared to Figs. 9 and 10. The numerical analysis used $R = 0.8$ and 2.8 in the MDP/Cap model, respectively. While $R = 0.8$ could not make positive agreements with the experimental data, $R = 2.8$ was found to fit the experimental data very well. Moreover, the M values between 0.46 and 0.5 were employed in the MCC model and are presented in Fig. 11 as well. It can be seen that the $R = 2.8$ used in the MDP/Cap model can successfully capture the test results for the NC clay. According to Table 3, the experimental M values are around 0.72 and the corresponding friction angles are about 18.7°. Based on the back calculations for $M = 0.46$ to 0.5, the values can also better fit the curves under the three different confining pressures. The friction angle, ϕ, is back calculated to be about 12.3° to 13.3°, which is 5.3° ~ 6.3° lower than the experimental results.

<u>Overconsolidated Clay (OCR = 1.25)</u>

In the following results, the overconsolidation ratio, OCR controlled in the tests is 1.25; the clay is regarded as slightly overconsolidated. Comparing Fig. 12 to Fig. 8, the deviatoric stresses under these three different confining pressures are a little bit higher than the case of OCR = 1.0 and the strengths are expected to be higher. The $R = 0.8$ adopted in the MDP/Cap model and $M = 1.22$ in the MCC model are both found to successfully capture the stress-strain curves formed in Fig. 12. Similar to that found earlier, Fig. 13 indicates the experimental results and predicted results for the clay mixed with 6% Bentonite, accordingly. The material parameter $R = 1.2$ used in the MDP/Cap model as well as the stress ratios, M, adopted between 0.87 to 0.92 in the MCC model in ABAQUS are also shown in Fig. 13, and are able to fit the stress-strain curves found in the experimental results. The back calculated friction angle based on the numerical analysis results are between 22.3° ~ 23.5°. The numerical analysis results indicate that the range of the friction angle is about 3.9° ~ 2.7° lower than the experimental outcomes. Figure 14 also shows the comparisons of the stress-strain behavior of the overconsolidated clay (OCR = 1.25) that was mixed with 10% Bentonite for both experimental and numerical analysis results. The PI of the soil is 36. The R parameter, which is similar to the results presented in Fig. 11, has to be increased to 2.7 in order to successfully fit the stress-strain curves. The stress ratio, M was adopted ranging from 0.47 to 0.49 to make good agreements with the experimental data. The friction angle was calculated based on the M values between 0.47 and 0.49 are 12.6° to 13.1°, respectively. The difference between the experimental data and the numerical analysis data is 6.1° ~ 6.6°. The numerical analysis results present lower friction angles.

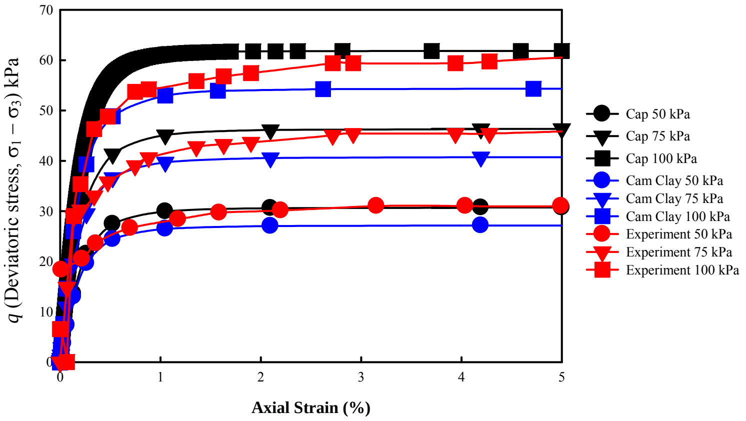

Fig. 9 Stress-strain relationships of CU tests for NC clay (e = 1.25, Bentonite = 3%, OCR = 1.0)

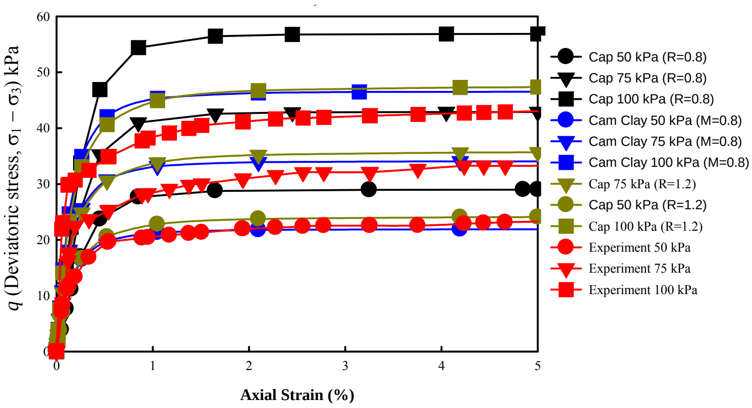

Fig. 10 Stress-strain relationships of CU tests for NC clay (e = 1.25, Bentonite = 6%, OCR = 1.0)

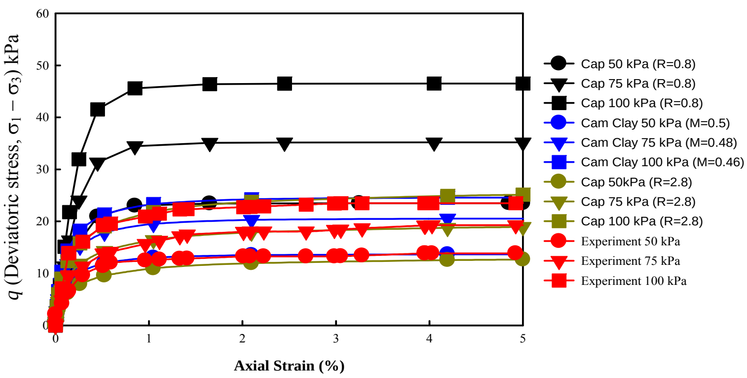

Fig. 11 Stress-strain relationships of CU tests for NC clay (e = 1.25, Bentonite = 10%, OCR = 1.0)

<u>Overconsolidated Clay (OCR = 1.50)</u>

Figure 15 shows that the results from the experiments and the predictions using finite element method for the soil with the OCR = 1.5. When compared to these curves based on the same PI of clays, the experimental results exhibit higher ultimate deviatoric stresses but lower overconsolidation ratios (See Figs. 9 and 12). Similarly, $R = 0.8$ was still used in the MDP/Cap model and

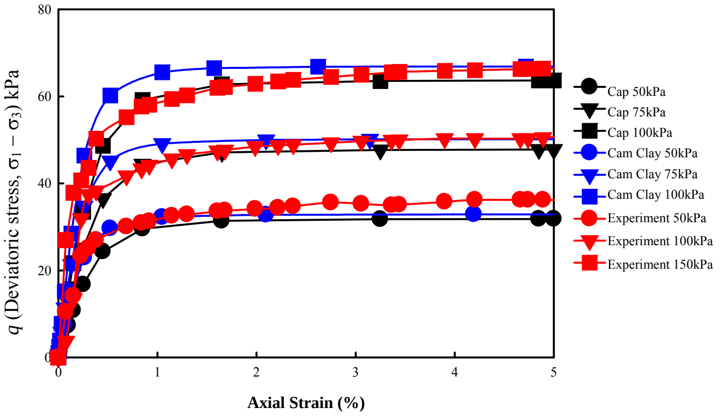

Fig. 12 Stress-strain relationships of CU tests for NC clay (*e* = 1.25, Bentonite = 3%, OCR = 1.25)

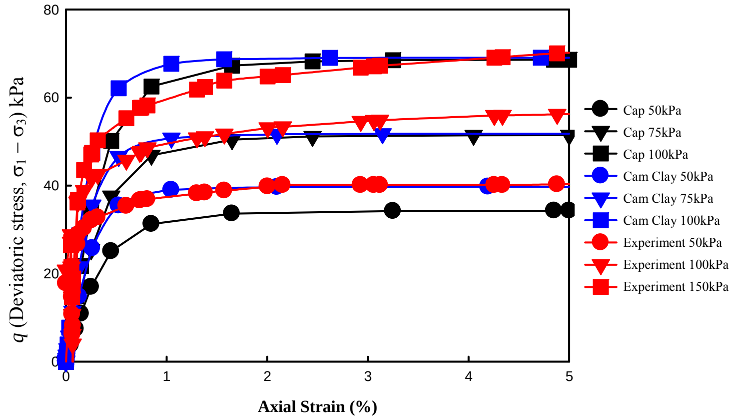

Fig. 15 Stress-strain relationships of CU tests for OC clay (*e* = 1.25, Bentonite = 3%, OCR = 1.50)

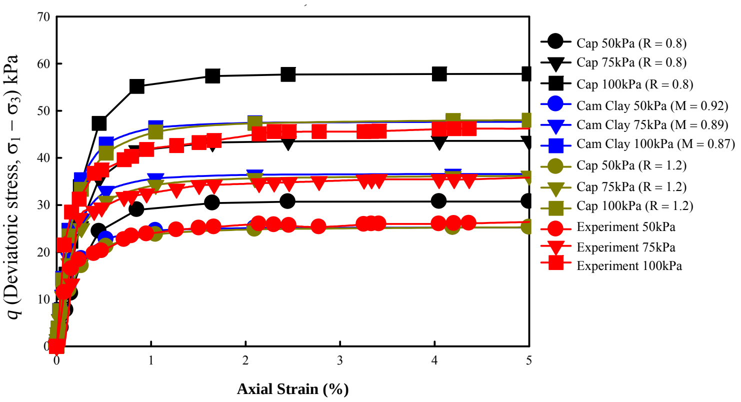

Fig. 13 Stress-strain relationships of CU tests for OC clay (*e* = 1.25, Bentonite = 6%, OCR = 1.25)

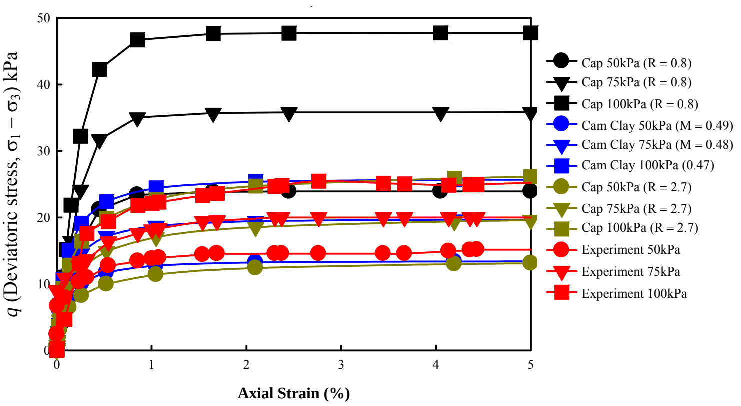

Fig. 14 Stress-strain relationships of CU tests for OC clay (*e* = 1.25, Bentonite = 10%, OCR = 1.25)

M = 1.27 was used in the MCC model. The results indicate that the stress-strain curves predicted by both models make very good agreement with the stress-strain behavior of the experiments. As it can be seen from the results, the Bentonite content increased to 6%, and R = 1.0 had to be revised from the earlier sample in the MDP/Cap model. The M values are still almost exactly the same as the experimental data summarized in Table 5. The experimental results and numerical analysis are shown in Fig. 16. The M values for OCR = 1.5 and 6% Bentonite content are between 1.08 and 1.11. There is not much difference between the M values when the different confining pressures are applied. Hence, the effective internal friction angle (ϕ') of this clay is about 27.4° calculated. In Fig. 17, the R = 2.5 used in the MDP/Cap model is found to best fit the experimental data for the soil mixed with 10% Bentonite. While the stress ratio, M = 0.5 can fit the experimental stress-strain curves well, the M values for the experimental results are between 0.75 and 0.77. A lower stress ratio is found in the numerical analysis results and the effective friction angles calculated are about 13.3°. They are about 6.4° lower than the experimental data.

5. DISCUSSION

The MDP/Cap model is found to predict the stress-strain behavior of the CU test for clayey soil very well. The PI is higher, the material parameter used, R, has to increase. The results indicate that R = 0.8 is applicable to the clay mixed with 3% Bentonite, regardless of the overconsolidation ratio. The R between 1.0 and 1.2 is good for predicting the stress-strain behavior of the clay mixed with 6% Bentonite. As for the clay mixed with 10% Bentonite, the R values between 2.5 and 2.8 are adequate. The higher the overconsolidation ratio of the clay is, the slightly lower R is required. A good constitutive model, the stress-strain relationship has to adequately describe the main characteristics of inelastic behavior as well as provide a stable and unique mathematical formulation. The R is the material parameter that controls the shape of the Cap in the MDP/Cap model and for the numerical model, the shape of the Cap is dependent on the PI of the soil. Based on the results of the numerical analysis, the relationships between R and PI are plotted in Fig. 18. The equations corresponding to the overconsolidation ratios are also presented in the figure. It is concluded that the PI of the soil is relatively higher, therefore, the material parameter, R, applied has to be larger. Moreover, the PI of the soil is higher, thus the required R is slightly lower if the overconsolidation ratio increases.

The MCC model is capable of capturing the lower plasticity of the clay such as the clay mixed with 3% Bentonite in this paper. The critical stress ratio M_{cr} used is nearly consistent in both

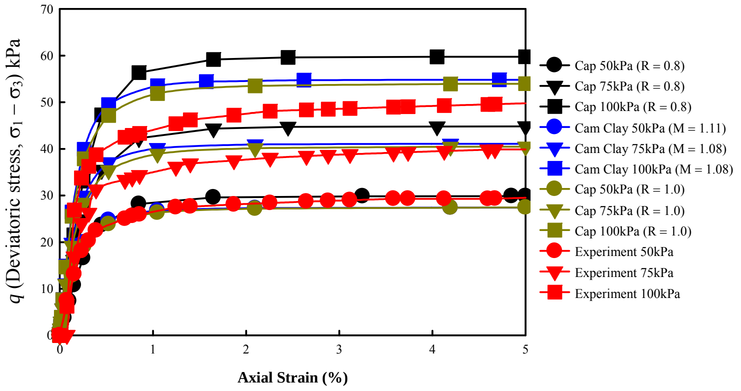

Fig. 16 Stress-strain relationships of CU tests for OC clay (*e* = 1.25, Bentonite = 6%, OCR = 1.50)

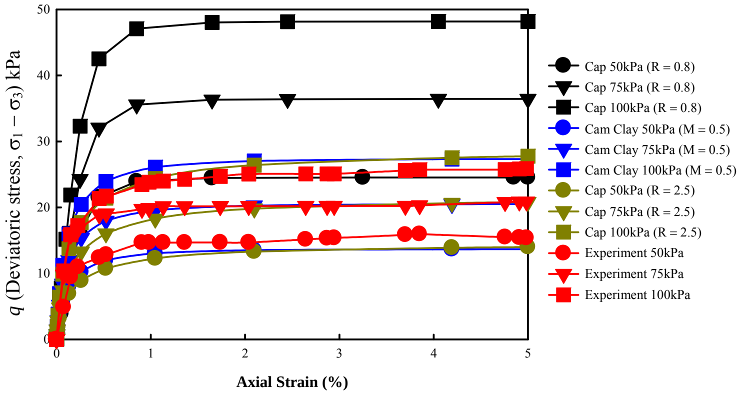

Fig. 17 Stress-strain relationships of CU tests for OC clay (*e* = 1.25, Bentonite = 10%, OCR = 1.50)

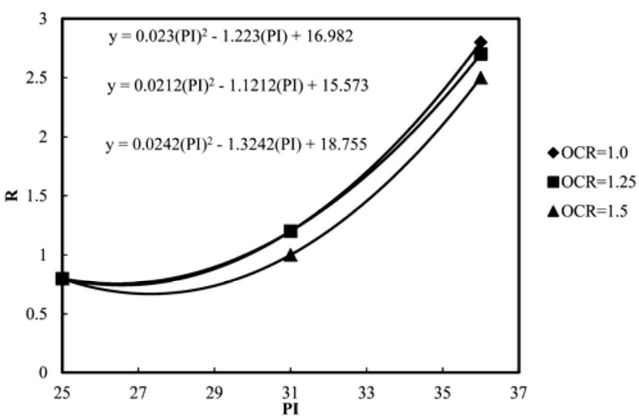

Fig. 18 Relationship between material parameter *R* and plasticity index PI for the clay

the experimental and numerical analysis results. For a higher PI of the soil, such as the clay mixed with 6% or 10% Bentonite, the stress-strain behavior predicted using numerical analysis cannot fit unless each stress ratio is reduced. The internal friction angle calculated is therefore several degrees lower than the experimental results. The reasoning for this can be explained based on the energy dissipation equation assumed for the MCC model (Roscoe and Burland 1968) as follows:

$$dW^p = p\cdot d\varepsilon_v^p + q\cdot d\varepsilon_d^p = \sqrt{(Mpd\varepsilon_d^p)^2 + (pd_v^p)^2} \tag{11}$$

The $d\varepsilon_v^p$ in the energy dissipation equation is one of the influencing variables in the MCC model. Especially in the Conventional Consolidated-Undrained (CU) test, the plastic volumetric strain, $d\varepsilon_v^p$ is equal to zero because there is no water dissipation in this stage. If there is no plastic volume change the unrecoverable energy is governed by the second term $q\cdot d\varepsilon_d^p$ in the equation; therefore the deviator stress q estimated will be higher. In reality, due to the Poisson's ratio that is not likely to be exact 0.5, Hence, the volume could still change slightly. This phenomenon is more obvious for the higher PI of the clay since the Poisson's ratio (ν) is closer to incompressible condition, ν = 0.5. Generally speaking, the MCC model can predict the stress-strain behavior more accurately for the lower plasticity of clay if only one soil model can be selected to simulate the soil behavior for this type of soil. Undoubtedly, the MDP/Cap model is more applicable for describing this type of normally consolidated or lightly consolidated clay.

6. CONCLUSIONS

This paper mainly addresses on the undrained shear strength of various plasticity indexes (PI) for Taipei clay. In order to realize the engineering behavior of the clay subjected to various plasticity indexes, the Bentonite was mixed with the clay to change the PI. In addition, the results of the numerical analyses are compared with the experimental and analytical results. The Modified Cam-Clay (MCC) model and the modified Drucker-Prager/Cap (MDP/Cap) model were both employed in the numerical analysis. Based on these experimental and theoretical results, several conclusions can be drawn and summarized as follows:

1. The MDP/Cap model is found to be useful and is capable of fitting the stress-strain behavior of clays more uniformly for Taipei clay by adjusting the material parameter, *R*. The higher the plasticity index of the clay is, a larger *R* is required. On the other hand, the *R* used is slightly lower for a higher plasticity as well as a higher overconsolidation ratio (OCR) of clay.
2. The MCC model is capable of predicting the stress-strain behavior of a clay more accurately if the clay has a lower plasticity index and is lightly overconsolidated or normally consolidated. On the contrary, if the MCC model is used to simulate the higher plasticity of the soil, the strength of the soil is possibly to be overestimated.
3. The MCC model used to fit the stress-strain curve in the numerical analysis will produce a lower *M* and internal friction angle, ϕ' when compared to the experimental results.
4. The experimental stress-strain curves can be observed as the undrained shear strength of the clay is affected by the plasticity index more than the overconsolidation ratio (OCR). The undrained shear strength of the clay is sensitive to the plasticity index or the Bentonite content that controls the plasticity index of the clay in this paper.
5. Based on the energy dissipation theory of the MCC model, it is deficient in predicting the soil without a volume change if

the undrained test (CU) is performed. This characteristic will be more prominent for a higher plasticity index of clay because the higher the plasticity a clay has, the higher the Poisson's ratio, ν, is. Consequently, the MCC model can be concluded to be not as good when used for predicting a lower plasticity index of clay.

6. This paper concludes that the MDP/Cap constitutive model is more applicable to be employed in numerical analyses for this type of clay in normally consolidated or lightly consolidated conditions.
7. In the future, the research can be conducted to explore the stress-strain behavior of the clay with the initial void ratio smaller than 1.25 that means the soil is in a denser condition.

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