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A Review of Critical State Elasto-Plastic Proposed Constitutive Models to Predict the Behavior of Clay Soils

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***Abstract*- Soils behavior are naturally complex and it can be said that soil reveals the most unknown behavior among all the materials found in nature; therefore, separate interpretations of soil behavior should be linked to interpret and predict the exact behavior and thereby the basis for stating soil behavior emerge. Elastoplastic behavior models that are expanded widely in recent years and they are presented in the framework of critical state model (CSM), provided this possibility. In fact, A constitutive model describes the soils behavior as simple and ideal. When the possibility of soil tests to identify and characterize the site does not exist, or when we have to predict soil behavior in response to changes in loading, in this case, CSM provides a tool for estimating soil response. In the literature, several constitutive models for soils are presented until now in them the critical state is considered as the initial basis. Three cases of old and new constitutive models to simulate the behavior of clay soils are proposed which include S-Clay1, MCC and SANICLAY. In this paper it is tried to investigate the differences in the formulation of these models by introduction of the formulation and basic foundations of critical state model; then, by modeling of MCC model with a computer program, it will be used for simulation the mechanical behavior of a clay soil. Finally, by examining the results of the simulations, it was indicated that this behavior model has relatively acceptable ability to simulate the mechanical behavior of clays and then weaknesses and strengths of this model are fully described.**

***Keywords*- Constitutive Model, Elastoplastic, Critical State Clay.**

I. Introduction

Due to lack of proper land for constructing soil structures, in many civil projects around the world, the foundation of structures are built on clay soils. Due to the complexity of the behavior of clay soils and the influence of various factors such as the sedimentation patterns, erosion, consolidation and pre-loading and etc. on their behavior, development and application of advanced constitutive models to predict the behavior of these soils by geotechnical engineers are getting more important day to day.

If the soil be consolidated by the stress greater than existing tension, its shear strength will increase. However, this increase is dependent on the soil type, loading conditions (drained and not drained condition) and stress pathways. Therefore, separate images of soil behavior should be connected to interpret and predict the exact behavior of soils and then separate images require puzzle or linking to create a basis for the expression of soil behavior. The purpose of this linking is unifying the consolidating and resistance behavior of soil. Of course, real soils demand a complex puzzle not only because the soils are complex natural materials, but also because their loading ways can't be accurately predicted. But theoretical constitutive models that have been proposed based on the critical state soil mechanics, have created this possibility. A critical state model (CSM) actually describes simplification and idealization the soils behavior. Of course, CSM covers the soil behavior that has great importance for the geotechnical engineering. The basic idea of CSM is that all soils will be disjointed in a single rupture surface in the stress space of void ratio (q, p, e). Thus, CSM also included volume changes in its rupture criteria unlike the Mohr-Coulomb rupture criterion that defines rupture as attainment of maximum tilt tension.

When, we don't ability to perform adequate tests to identify and characterize the soil in workshop, or when we have to predict the response of soil behavior against changes caused by loading during construction or after that, in this case CSM provides a tool for estimating soil responses. In the literature, several constitutive models for soils have been proposed in them the critical state is as the basis and initial nucleus of all of them.

Hence, complete understanding of constitutive models principles in one hand and finding the strengths and weaknesses of existing models to develop new constitutive models on the other hand are always an important issue in soil mechanics. Among the existing constitutive models to predict the behavior of clay, modified Cam-Clay model, (MCC) [1] and more recently constitutive models S-CLAY1 [2], [3] and SANICLAY [5], [4] with the aim of eliminating the shortcomings the previous model are the most popular squeezed constitutive models to predict the behavior of clay soils in the analytical methods. Although each of these constitutive models somehow managed to fix the problems of previous models, but still a thorough understanding of precise formulations and their differences and also their defects to accurate simulation of soil behavior made their design controversial.

Hence, in this paper, we introduce three constitutive models to assess strengths and weaknesses and also compare the efficacy of these models (MCC) in predicting clay soils behavior. For this purpose, available experimental real data results on normally consolidated clay soil samples Lower Corner Till (LCT) [6] or different values of strengthening stress ratio (OCR) are used in cases of compression and lateral (tensile).

II. General Formulation in Constitutive Model of Soils

A. General formulation of elastoplastic critical state models

In conventional, in soil mechanics, compressive stresses mark are positive and tension stresses are considered to be effective tension. In other words, volume reduction is appeared with positive sign and dilation is appeared with negative sign. When mechanical behavior of soil is assumed to be fully elastic, the following equation is used to express the relationship between stress and corresponding strain rate:

$$\dot{\sigma} = D\dot{\varepsilon}^e \qquad (1)$$

In that, $\dot{\boldsymbol{\sigma}}$ and $\dot{\boldsymbol{\varepsilon}}$ are second order stress and strain tensor, respectively, and superscript e indicates the elastic strain. D is also represents the fourth order elastic tensor and Isotropic Linear Elasticity is defined:

$$D_{ijkl} = (K - \frac{2}{3}G)\delta_{ij}\delta_{kl} + G(\delta_{ik}\delta_{jl} + \delta_{il}\delta_{jk}) \qquad (2)$$

In the above equation, K and G are respectively Bulk or volumetric modulus and shear elastic and $\boldsymbol{\delta_{ij}}$ is Kronecker's Delta Function in that for $\boldsymbol{i = j}$ is equal to 1 and in otherwise zero will be considered. In triaxial conditions, the mean effective stress (p), shear stress (q), and the volumetric strain (ε_v) and shear (ε_q), are defined by the following relations:

$$p = \frac{1}{3}(\sigma_1 + 2\sigma_3) \quad ; \quad q = \sigma_1 - \sigma_3 \qquad (3)$$

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \quad ; \quad \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) \qquad (4)$$

Based on the above equations, linear elastic and adjusted theory at the triaxial stress can be expressed by the following equation:

$$\begin{bmatrix} \dot{p} \\ \dot{q} \end{bmatrix} = \begin{bmatrix} K & 0 \\ 0 & 3G \end{bmatrix} \begin{bmatrix} \dot{\varepsilon}_v \\ \dot{\varepsilon}_q \end{bmatrix} \qquad (5)$$

In this relationship K and G values in terms of Poisson's ratio (V) have changed and respectively are determined by the following relations:

$$K = \frac{p(1+e_{in})}{\kappa} \quad ; \quad G = \frac{3}{2}K\left(\frac{1-2\nu}{1+\nu}\right) \qquad (6)$$

where, "e_{in}" is the initial value of void ratio, "k" is the slope of unloading-reloading line measured in e-ln$p$ plane, and "ν" is the Poisson's ratio. In most of the proposed models for soil constitutive is assumed on the basis of classical elastoplastic theory, the total strain growth is the sum of elastic and plastic strain growth and is expressed in equation (7):

$$\dot{\varepsilon}_v = \dot{\varepsilon}_v^e + \dot{\varepsilon}_v^p \quad ; \quad \dot{\varepsilon}_q = \dot{\varepsilon}_q^e + \dot{\varepsilon}_q^p \qquad (7)$$

$\dot{\varepsilon}_v, \dot{\varepsilon}_q$ Represent respectively the measures volumetric and shear strain development in the triaxial space. e and p are respectively the elastic and plastic strain parts and Symbol point on each parameter represents the rate of change of strain. When tension stress has been located in yield surface, mentioned models predict soil behaves as an elastic.

Elastic component of the strain rate changes are consistent with the assumption of elastic behavior according to the following equations:

$$\dot{\varepsilon}_v^e = \frac{\dot{p}}{K} \quad ; \quad \dot{\varepsilon}_q^e = \frac{\dot{q}}{3G} \qquad (8)$$

p and q are respectively the mean principal effective stress, and deviator stress measured in triaxial space. space, K and G as previously presented are determined according to equation (5).

With increasing stress field components, the stress placed on the surrender surface (this will be explained further below). In this case, plastic strains will be developed along with the elastic strain. Using plastic theory and normal function determination of the plastic potential function (the normality principle), the growth volumetric and shear plastic strains are obtained according to the following equation:

$$\dot{\varepsilon}_v^p = \langle L \rangle \frac{\partial g(p,q,\alpha,p_\alpha)}{\partial p} \quad ; \quad \dot{\varepsilon}_q^p = \langle L \rangle \frac{\partial g(p,q,\alpha,p_\alpha)}{\partial q} \qquad (9)$$

$\boldsymbol{\alpha}$ is a parameter which applied the anisotropy by changing the placement of the plastic potential function with respect to p-axis

$\boldsymbol{P_a}$ parameter also specifies the size of the plastic potential function and it is equal to the mean value of Mean Confining Stress on plastic potential function.

B. Yield Surface

The basic concept of CSM is a unique rupture surface in the space (q, p, e) that defines the rupture independent of load history or next stress pathways. Rupture is synonym with the critical condition. Critical condition is a state of constant tension which is specified with continuous shear deformation at constant volume. In stress space of (q, p), the CSL line is a straight line with slope M = Mc for pressure and M = Me for lateral extension or stretch (Figure 1). It should be noted, in this paper stretch is not the known stretch, but it shows the state in that lateral stress is greater than the vertical stress.

"L" is loading index whose mathematical expression is presented later, and are Macaulay brackets. For scalar parameter "x", = x if x>0, and zero otherwise.

C. Yield Surface

As previously mentioned, in tension space, there is a surrender surface that dictates the elastic behavior on soil responses and separates it from a tension situation that causes plastic behavior. Usually, to express the surrender surface, stress space (q, p) in the case of triaxial is used instead of (σ_1, σ_3) tension space. Usually in constitutive models surrender surface is assumed as oval and its initial size or larger axis (greater tension) is determined with strengthening stress (maximum soil tension that has endured in its loading history). Many empirical evidences (Wong & Mitchell, 1975) indicate that the surrender surface of an elliptical approximation is usually acceptable for soils, especially clay soils.

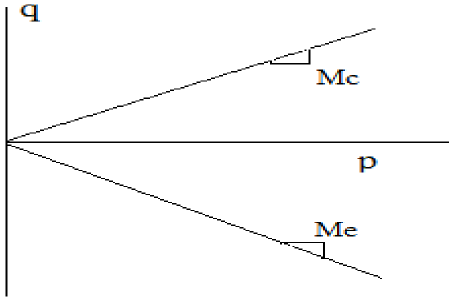

Figure 1: Behavior of clay soils, CSL: the critical state line; NCL: normal consolidation line; URL : unloading-reloading line [11]

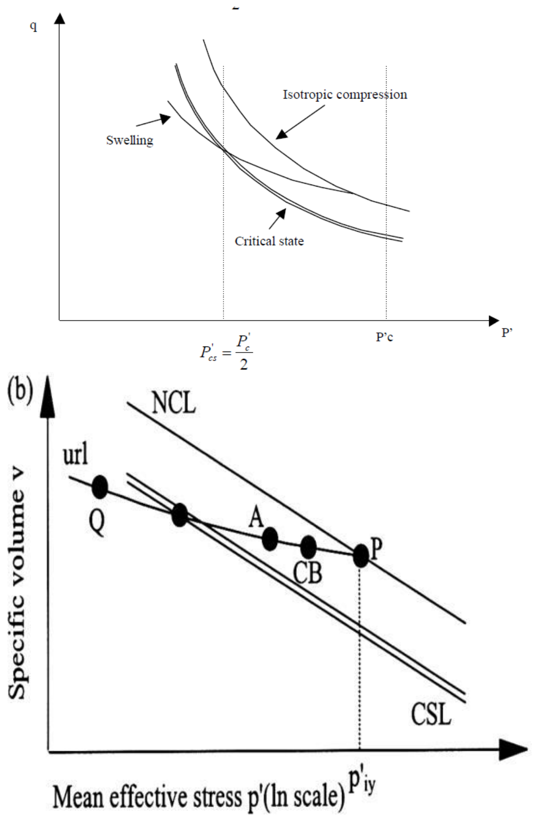

Figure 2.Initial estimates of the form of yield surface for different soils based on experimental and analytical results on different clay soils [12]

It should be noted that if over-consolidation is more, initial ellipse will be bigger. Generally, yield surface is considered for compression, but it is the same for tension except smaller axis (smaller stress) of ellipse yield surface be smaller in tension in comparison with compression. All combinations of p and q are located inside yield surface, as point A (Fig. 3) cause soil to have elastic behavior. If a combination of q and p locates on yield surface, soil will yield just like steel bars. Finally, any combinations of q and p will set to out of initial yield surface with outgrow of initial yield surface in which the process is called hardening; while plastic loading, stress point is outgrow with point C (Fig. 3) on yield surface not out of the surface. BC stress paths (Fig. 3) causes that soil have Elastic-plastic behavior. If soil is unloaded in a situation lower than failure surface, will act like an elastic material and by outgrowing of yield surface, elastic area will be bigger.

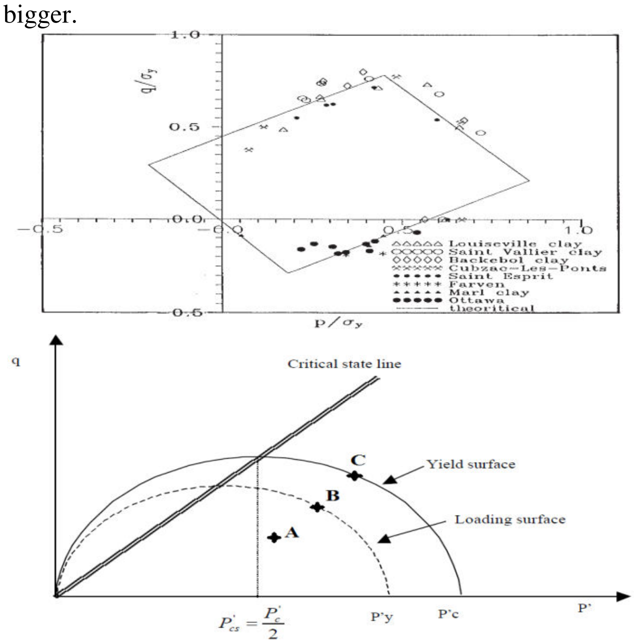

Figure 3. The concept of yield surface in stress space

Yield surface is defined by f (p, q, β, p0), where, β is rotational hardening parameter, and submit non-isotropic effect by deviation of yield surface axis to p-axis. P0 also is the second hardening parameter and submit isotropic hardening to p. P0 also is the second hardening parameter and consider isotropic hardening and mean value on yield surface is q=βp.

D. Flow rule

If plastic potential function and yield surface are defined by one term (f = g), flow law will be considered dependent and otherwise (f ≠ g) not dependent. By the independent law, it is possible to choose different yield surface and plastic potential function in accordance with non-isotropic soil behavior. Rate of change of three parameters α, β and p0 which change by production of volumetric plastic strain and displacement of yielding and potential plastic functions:

$$\dot{\alpha} = < L > \bar{\alpha}; \quad \dot{\beta} = < L > \bar{\beta}; \quad \dot{p}_0 = < L > \bar{p}_0 \quad (10)$$

According above formulations, loading index (L), controls growth of hardening parameters. $\bar{\alpha}, \bar{\beta}$ and $\bar{P}_0$ also are hardening factors which will be introduced separately in the next chapter. By including compatibility state to yield surface, loading index is calculated by equation (6), where K_p is plastic module and according equation (7) can be achieved.

$$L = \frac{1}{K_P}\left(\frac{\partial f(p,q,\beta,p_0)}{\partial p}\dot{p} + \frac{\partial f(p,q,\beta,p_0)}{\partial q}\dot{q}\right) \quad (11)$$

$$K_p = -\left(\frac{\partial f(p,q,\beta,p_0)}{\partial p_0}\bar{p}_0 + \frac{\partial f(p,q,\beta,p_0)}{\partial \beta}\bar{\beta}\right) \quad (12)$$

With respect above equations, it seems that loading index and plastic hardening module depend on β and p_0 changes and yield functions forms and chosen plastic potential. Equations that have been already mentioned, related Cam-Clay family formulation and lots of models have been proposed by them. What distinguishes these models, on the surface of proposed yield surface, are flaw law and hardening parameters. Furthermore, at first, details of formulation of some of constitutive models is described for expressing clays and then, MCC model is evaluated to predict true behavior of clays.

III. Constitutive Models For Clay Soils

A. *MCC models*

Generally, Cam-clay model [4] is the first critical states model to predict and stimulate clay mechanical behavior. It should be noted that this model which first introduced by Cambridge university, was more considered as a training goal and then by distracting shortages, modified model was introduced with the name of MCC which had great capability to eliminate previous model shortages and despite acceptable simplicities, this was able to predict volumetric changing behavior and strength of clay soils. This caused that the model develop quickly and in many analytical problems, used for stimulating clay soils behavior.

In this model, yield surface shape is like an ellipse (Fig. 4), which outgrow with load increase. In the other hand, mentioned yield surface, has enlarging feature, and can be enlarge isotropic by considering hardening and predict clay behavior correctly. Furthermore, dependent flow law and (f=g) and isotropic hardening are used in this model, which result in easier calculations. Proposed yield surface in this model is according equation (13):

$$f(p,q,p_0) = q^2 - M^2(pp_0 - p^2) = 0 \tag{13}$$

It should be noted, this model benefit assumptions, which are listed below:

- ✓ In accordance dependent flow law (f=g) in this model, and
- ✓ With respect to proposed yield function like ellipse, in this model, yield surface does not have rotation and distortion property and just can harden or soften by load increasing. Hence, changing rate of hardening parameters α and β is considered to be zero and the model just has got changing rate of hardening parameter p_0.

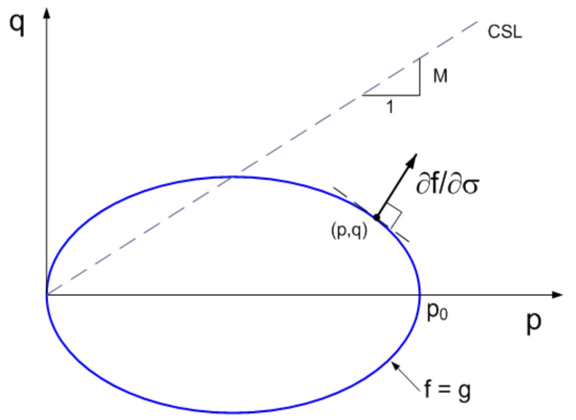

Figure 4. The yield surface in MCC model

In equation (13), p_0 is hardening parameter and M is proportion of critical stress state which indicates slope of critical state in 3-axis (q-p) space and its value varies in compression (Mc), extension and 3-axis tension (Me) mode. It should be explained, value of yield function is defined by p0 parameter in this model which its value changes due to hardening factor.

$$\bar{p}_0 = \left(\frac{1+e_{in}}{\lambda - k}\right) p_0 \frac{\partial(p,q,p_0)}{\partial p} \tag{14}$$

where, "e_{in}" is the initial value of void ratio, "κ" is the slope of unloading-reloading line measured in e-lnp plane, "λ" is the slope of the normal compression line in e-lnp plane.

Generally, in most of models, and at the center of them, MCC model have 5 fixed parameters (λ، κ، ν ،Mc,Me) which obtain from typical 3-axis and consolidation tests. Next, determination of these parameters has been studied by experimental data to evaluate clay samples by MCC model.

So far, MCC model has been used by many numerical geotechnical software for analyzing earth structures. However, it has got fundamental problems. As one of MCC problems, it can be mention; overestimation of shear strength of over-consolidated clays and dilatory behavior which happens because dependent flow law is used. Another shortcoming of this method is its incapability to reasonably predict elastic behavior of non-isotropic over-consolidated samples. Hence, after complete introduction of the model and expressing used properties and assumptions, in the following chapters is tried to stimulate mechanical behavior of clays in different loading conditions and then study advantageous and disadvantageous of the model.

B. *S-CLAY1 model*

The S-Clay1 model [2], [3], is a generalized model of MCC which is presented by Wheeler et al and unlike MCC, possibility of rotational hardening is also defined for yield surface. This feature provides capability of stimulating non-isotropic over-consolidated clays. In this model like MCC, dependent flow law (f=g) is used. Proposed yield surface in this model (Fig. 5) is defined as equation (15):

$$f(p,q,\alpha,p_0) = (q-\beta p)^2 - (M^2-\beta^2)(pp_0 - p^2) = 0 \tag{15}$$

Where, β and p_0 are hardening parameters. By definition of Karstunen et al [3], hardening equation (10) factors obtain from following equations:

$$\bar{p}_0 = \left(\frac{1+e_{in}}{\lambda - \kappa}\right) p_0 \frac{\partial f(p,q,p_0)}{\partial p} \tag{16}$$

$$\bar{\beta} = x_1\left[\left(\frac{3}{4}\eta - \beta\right)\langle\frac{\partial f(p,q,\beta,p_0)}{\partial p}\rangle + x_2\left(\frac{1}{3} - \beta\right)\left|\frac{\partial f(p,q,\beta.p_0)}{\partial q}\right|\right] \tag{17}$$

In this model, rather existence constants in MCC, x1 and x2 also are new constants which can be easily determined by test-error method.

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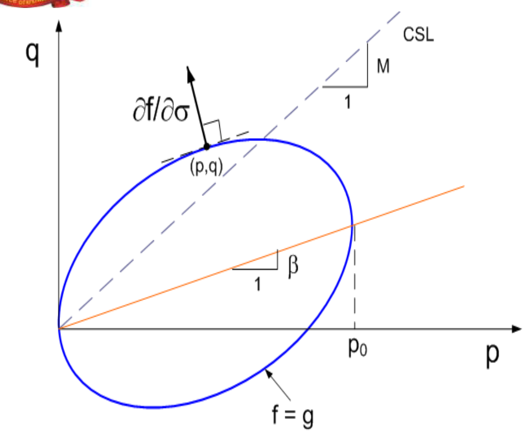


Figure 5.The yield surface in S-CLAY1

C. SANICLAY model

Dafalias [4] showed that how non-isotropic behavior can be seen in modified model by developing MCC model and introducing volumetric and shear plastic work rate. The Saniclay model is a developed model of MCC which is presented in limited plastic surface theory by Dafalias et al [5]. In this model, ellipse yield surface may be rotated and deteriorated due to isotropic hardening and non-isotropies because of applying stress on soil in 3-axis space of q-p. Quantity of rotation and deterioration describe degree of non-isotropies and shown by α.

Saniclay model predicts relatively accurately behavior of normally consolidated clays and stimulates suitably over-consolidated clay. The required costs for stimulating in this model in comparison with MCC, just is three more constant parameter (N, C and x) which can be easily determined by 3-axis experiments. Unlike previous two described models, independent flow law (f≠g) is used in this model. But like previous model, main disadvantage of the model, is using decreasing elastic theory which causes overestimation of shear strength of highly over-consolidated clay samples. In the model, any of functions or potential yield and plastic surfaces (Fig. 6) with ellipse shape with rotational and distortional property is separately defined and expressed by the following equations:

$$g(p,q,\alpha,p_\alpha) = (q-\alpha p)^2 - (M^2-\alpha^2)(pp_\alpha - p^2) = 0 \quad (18)$$

$$f(p,q,\beta,p_0) = (q-\beta p)^2 - (N^2-\beta^2)(pp_0 - p^2) = 0 \quad (19)$$

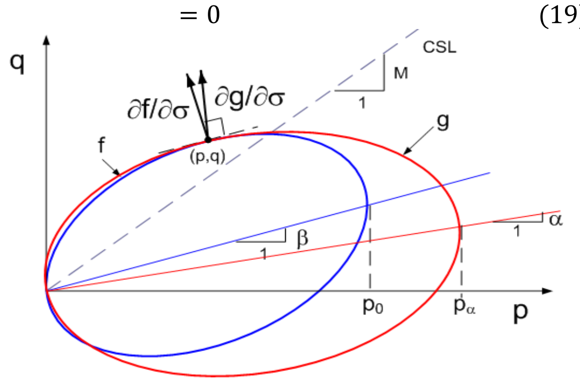


Figure 6.The yield and plastic potential surface in the constitutive model of SANICLAY

where, "N" and "M" are the model parameters. "M" is the slope of critical state line measured in triaxial q-p plane. In definition of the yield function, "N" plays the same role as that of "M" in plastic potential function.

However N is selected except M in the same compression and tension. Also it should be noted that hardening factors are expressed in this model with following equations:

$$\bar{\beta} = C\left(\frac{1+e_{in}}{\lambda-\kappa}\right)\left(\frac{p}{p_0}\right)^2 \left|\frac{\partial g(p,q,\alpha,p_\alpha)}{\partial p}\right| \left|\eta-\beta\right|(\beta^b-\beta) \quad (21)$$

$$\bar{p}_0 = \left(\frac{1+e_{in}}{\lambda-\kappa}\right) p_0 \frac{\partial g(p,q,\alpha,p_\alpha)}{\partial p} \quad (22)$$

In above equations, C and x are parameters and η are stress proportion. when $\left(\frac{\eta}{x}\right) > \alpha$, α^b is equal Mc and otherwise is equal -Me. also if $\eta > \beta$, β^b parameter is equal N and otherwise is equal –N [5].

IV. Simulating The Behavior of Clay Soils Using MCC

A. Process of Modeling using MATLAB software

At this stage, after complete description of the constitutive model formulation of MCC, stress-strain behavior of clay samples simulated by using MATLAB software & coding. For modeling at first, in accordance with what was explained in previous parts, the formulation of MCC model is divided into the two elastic and the plastic section. The basic assumptions in critical state soil mechanics [1] is that before the surrender, the tension in the elastic region and all the volumetric and shear strain are in elastic strain form. The concept of elastic strain is that by removing the elastic strain or stress applied on the soil, the deformation occurring in the samples is removed and they back to their original scales. At this case, by rearranging the equation (8) and giving strain step or rate of change of shear strain, shear (q) and bulk (p) stresses are determined in terms of shear (G) and bulk (K) modulus.

$$\dot{p} = K \times \dot{\varepsilon}_v^e \quad ; \quad \dot{q} = 3G \times \dot{\varepsilon}_q^e \quad (23)$$

As previously mentioned the shear and bulk modulus or shear and bulk stiffness of soils, according to the relations given in equation (10), was determined in terms of the parameters such as initial void ratio, slope of Load-Reload line in e-lnp space and Poisson's ratio.

According to lab results, after reaching the yield level, the behavior of samples are shaken and go to plastic region. It means that by removing load or stress applied on samples, deformations doesn't back into original scale and Amount of residual strain remain in the samples. In this case, resistance and residual stresses in the samples are known as ultimate strength of specimens. Therefore, by definition of yield level according to original MCC model (equ. 13), the two elastic and plastic region have been separated. After getting into the plastic zone, by definition of yield level and hardening parameters are fully described in previous formulations, bulk and shear stress values in the ultimate state (ultimate strength) of samples were determined in terms of different values of over consolidation ratio (OCR).

At the next, the required parameters for simulating the behavior of clay soils by the MCC are provided in Table 1. In addition to, the simulated behavior of clay samples under different experimental conditions and undrained loading were compared with actual behavior.

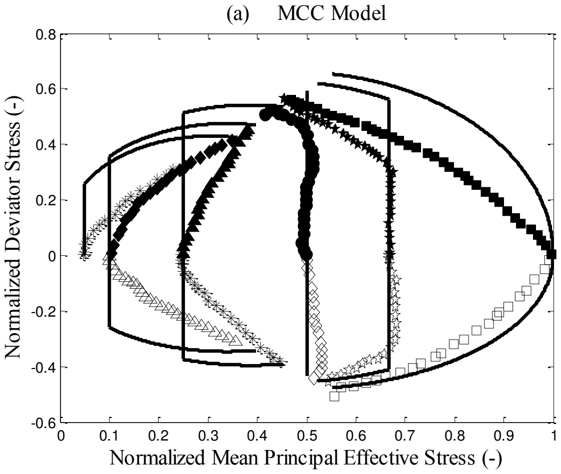
Table 1.Constant parameters of theMCC Constitutive Model for predicting the behavior of clay soils

Parameter	MCC				
	M_c	M_e	λ	κ	υ
In LCT clay soil	1.18	0.86	.0630	.0090	0.20

In figure (7), prediction of the behavior of undrained LCT clays under identical consolidation, compared with experimental data for OCR=1, 1.5, 2, 4, 10, 20 by using MCC constitutive model.

It is noticeable that all stresses as effective stress and prediction of soil behavior is normalized in tri-axial stress paths and deviatoric stress vs axial strain is presented . As shown in figure (7), by using of reduced elastic theory in MCC model, predicted behavior for all samples is estimated in undrained stress paths for initial value of p and case of purely elastic. This estimation is contrary to the actual soils behavior. This simulated behavior will continue until the stress reaches the yield level. According to the figure (7), MCC model is unable to predict properly the behavior of over consolidated (OCR>2) clays using identical consolidation. Also, according to figure (7-b), although MCC model is able to predict properly the strain behavior (bulk change) of most clay samples, but not able to predict behavior of samples with high values of OCR. In addition to, there is a large of difference between Prediction of MCC model and actual clays behavior under unloading at stretching mode. According to this results, in analytical methods avoid using MCC model for prediction of behavior of clays under disparate consolidation is more required.

(a) MCC Model



(b) MCC Model

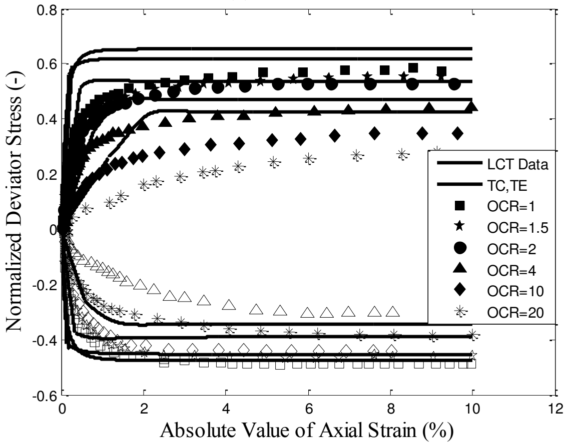


Figure 7.Comparison of predictions Undrained clay behavior LCT (Lower Corner Till) under isotropic consolidation by the MCC model

Totally, by considering simulation of MCC model for various cases of undrained loading, it is observed that in comparison with the actual experimental data, constitutive model is able to predict properly behavior of normal consolidated and little more consolidated samples (OCR<2), but not able to accurately simulate over consolidated samples and in comparison with actual experimental behavior estimate stiffer response. The main causes of disability of this model in accurate prediction of behavior of clay samples are such as selected empirical yield function in this model, Disregarding rotation and distortion property in yield function, dependent flow law, ignoring soil dissimilar behavior and most importantly using reduced elastic theory and ignoring satisfaction of thermodynamic laws.

V. Conclusion

In this research, evaluate the ability of clay soil behavior simulation using MCC constitutive model. In order to this, we assess that after defining details of elasto-plastic models formulation at critical point, present the differences between three constitutive models ie. MCC, S-CLAY1 and SANICLAY. Then, calibration of required constants for MCC model evaluated according to triaxial experiment for LCT clay samples and their weaknesses and profiteers is checked. Although, The constitutive MCC model is applied widely in commercial analytic softwares, it is not able predict properly the behavior of samples under similar consolidation with high value of OCR. In other words, it's prediction has noticeable difference with actual behavior of clays. Such observations can challenge safety of designed soil structures.

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