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A Review on the performance of Modified Cam Clay Model for fine grained soil

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Abstract : **The Modified Cam Clay (MCC) is one of the most widely used soil models. It was developed by the researchers at Cambridge University, UK. The MCC model works very well for predicting the behavior of normally consolidated clays, but it cannot predict many important features of the behavior of over consolidated clays. The original cam clay model was first introduced by Roscoe and Schofield. But this model was found some deficient in some aspects namely yield surface and the predicted value of Ko (co-efficient earth pressure at rest). Later on MCC model was developed by Roscoe and Burland to solve these problems. The MCC is an elastic-plastic strain hardening model and is based on the critical state theory. The MCC model is used to predict the behaviour of locally available clayey soil and validated with the experimental results. A series of Triaxial tests (drained and undrained conditions) and Consolidation tests have been conducted of two samples. A comparison is made between the predictions given by MCC model with experimental data of different soil samples. From the above comparison it is observed that the model predicted results match well with experimental values under drained condition for all the samples tested, but in undrained condition substantive deviations are observed.**

Key Words: Modified Cam Clay, Normally Consolidated Clay, Tri-axial Test, Consolidation Test

1. INTRODUCTION

The Modified Cam Clay (MCC) is most widely used constitutive model in this world. This model is well accepted among the researchers in the field of geotechnical engineering. The MCC model gives satisfactory results for normally consolidated clay.

The first critical state model for describing the behavior of soft soils such as the cam clay (CC) and modified cam clay (MCC) which developed by researchers at Cambridge University. The Original Cam-Clay model is one type of CSSM model which was developed by (Roscoe and Schofield 1963). Professor John Burland (1965) suggested the modification of cam clay model and modified cam clay was developed by (Roscoe and Burland 1968).

Collins (2005) reviewed some recent advances in the constitutive modelling of soils, sands and other granular materials. These new ideas are based on the use of the concepts of the modern theory of the thermomechanics of continua. Most existing engineering theories of soil behaviour are in the nature of ‘‘recipes’’, providing rules by which yield loci, flow and hardening rules and failure lines may be constructed to provide models which predict the response and failure of granular materials in a limited set of laboratory experiments. These models rarely have any firm physical basis.

Eko (2005) put emphasis on a better understanding of soil mechanical properties which is needed to assess soil compaction in clay soil. To fill that need, a research program was undertaken at Laval University, Quebec city, Canada to ultimately find better solutions for managing Sainte-Rosalie clay compacted by liquid manure spreaders. The first phase of this program comprised laboratory tests aimed at studying the mechanical behaviour of the soil and deriving a simple critical state model. In the second phase, the numerical form of the derived critical state model was validated then used to simulate loading paths which occur in the field during compaction of Sainte-Rosalie clay.

Gens et al (2006) reviewed constitutive modeling of unsaturated soils. After a brief historical perspective, a number of existing constitutive models are classified and discussed according to type of stress variables adopted in their formulation. Afterwards, attention is given to recent developments in the proposal of coupled hydraulic-mechanical models and the possibility of casting them in a sound thermo dynamical framework. Finally a double structure model for expansive soil is described. The incorporation of micro structural considerations and its use as a platform for incorporating the influence of new variables are highlighted.

Sheng (2011) discussed an unsaturated soil which is a state of the soil. All soils can be partially saturated with water. Therefore, constitutive models for soils should ideally represent the soil behaviour over entire ranges of possible pore pressure and stress values and allow arbitrary stress and hydraulic paths within these ranges. The last two decades or so have seen significant advances in modelling unsaturated soil behaviour. This paper presents a review of constitutive models for unsaturated soils.

Hattab (2011) explained the aim of the experimental study to identify the local deformation properties in a clayey material which can be activated at the macroscopic ultimate state known by critical state. The approach consists of an extensive study, based on a Scanning Electron Microscope (SEM) picture analysis, of the orientation of the clay particles characterized in the last stages of triaxial loading.

Oqueno (2011) discussed the ability of grains to rotate which can play a crucial role on the collective behavior of granular media. It has been observed in computer simulations that imposing a torque at the contacts modifies the force chains, making support chains less important. The effect of a gradual hindering of the grains rotations on the so-called critical state of soil mechanics was investigated. The critical state is an asymptotic state independent of the initial solid fraction where deformations occur at a constant shear strength and compactness.

Lashkari (2012) introduced the concept of independent stress state variables to consider the impact of unsaturated conditions, an elasto-plastic critical state constitutive model for saturated and unsaturated interfaces. The proposed model is capable of predicting many characteristics of unsaturated interface behavior, such as the dependence of initial tangent modulus, peak shear stress, dilatancy, and ultimate strength on matric suction, net normal stress, and the interface state measured with respect to the critical state line. To this aim, two distinct yield mechanisms are employed in the model.

In this paper, experiments have been carried out on both drained and undrained conditions. The MCC model was employed to understand the applicability of MCC to analyse the experimental data. The present research proposed is aiming towards development of a constitutive MCC model of soil in the framework of critical state soil mechanics incorporating the positive aspects of the existing models.

2. MODIFIED CAM CLAY MODEL

In the critical state soil mechanics, the state of a soil sample is characterized by three parameters, the effective mean stress (p'), the deviatoric or (shear stress q) and the Specific volume (v).

3. YIELD FUNCTIONS

The yield functions of MCC model determined from the following equation:

$$\frac{q^2}{p'^2} + M^2\left(1 - \frac{p_0'}{p'}\right) = 0$$

In p' − q space, the CC yield surface is a logarithmic curve while the MCC yield surface plots as an elliptical curve.The parameter p_o' (yield stress or pre-consolidation pressure) which controls the size of the yield surface. The parameter M is the slope of the CSL in p' − q space. The CSL is that it intersects the yield curve at the point at which the maximum value of q is attained. The stress states that lie inside the yield surface cause the soil to behave elastically. The stress states that lie on the yield surface cause the soil to yield. The stress states that lie outside the yield surface cause the soil to behave elasto-plastically.

4. SPECIFICATION FOR MODIFIED CAM CLAY MODELS

Specification of Modified Cam-Clay models requires five material parameters. These parameters are:

1. λ – the slope of the normal compression line and critical state line(CSL) in v − ln p ' space
2. κ – the slope of a swelling line in v − ln p ' space
3. M – the slope of the CSL in q − p ' space
4. N – the specific volume of the normal compression line at unit pressure
5. µ – Poisson's ratio or
   G - shear modulus

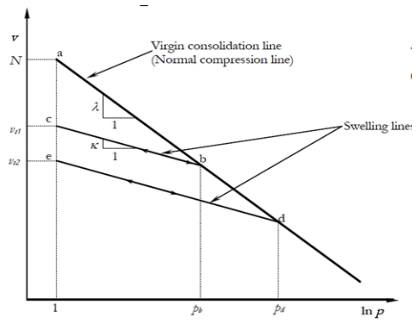

Figure 1: Behavior of soil sample under isotropic compression

5. PREDICTION OF STRESS-STRAIN RESPONSE OF MCC FOR LIGHTLY OVERCONSOLIDATED SOIL UNDER DRAINED CONDITION:

The soil sample in a consolidated drained test is isotropically consolidated and then axial loads or displacements are applied, keeping the cell pressure constant. If the soil sample consolidate up to a maximum mean effective stress P'c , and then unload to a mean effective stress P'o such that Ro= P'c/P'o <2, then we can formulate the MCC model by following these steps.

Step 1: Slope of the critical state line

$$M = \frac{6\sin\emptyset}{3 - \sin\emptyset}$$

Step 2:

For CD test, mean effective stress P'_y =

$$\frac{(M^2 p_c' + 18p_0') + \sqrt{(M^2 p_c' + 18p_0')^2 2 - 36(M^2+9)(p_0')^2}}{2(M^2+9)}$$

And deviatoric stress $q_y = 3(p_y^{'} - p_0^{'})$

Step 3: mean effective stress at failure

$$p_f^{'} = \frac{3p_0^{'}}{3-M}$$

Deviatoric stress at failure

$$q_f = Mp_f^{'}$$

Step 4: $[p_{av} = \frac{p_0^{'}+p_y^{'}}{2}]$.

$$G = \frac{3p_{av}(1+e_0)(1-2v^{'})}{2k(1+v^{'})}$$

Step 5:

Elastic shear strain $(\Delta\epsilon_q^e)initial = \frac{\Delta q}{3G}$ and Elastic volumetric strain $(\Delta\epsilon_p^e)initial = \frac{k}{1+e_0} ln\frac{p_y^{'}}{p_0^{'}}$

Step 6:

The preconsolidation stress, $p_c^{'}$, for each increment; that is $\Delta p^{'}$ = small increment in $p^{'}$ and $\Delta q = 3X\Delta p^{'}$

$$p_c^{'} = p^{'} + \frac{q^2}{M^2 p^{'}}$$

Step 7: Volumetric strain

$$\Delta\epsilon_p = \frac{k}{1+e_0}\{(\lambda - k)ln\frac{p_{c1}^{'}}{p_c^{'}} + kln\frac{p_{y1}^{'}}{p_y^{'}}\}$$

Step 8: Volumetric plastic strain

$$\Delta\epsilon_p^p = \frac{\lambda - k}{1+e_0} ln\frac{p_{c1}^{'}}{p_c^{'}}$$

Step 9:plastic shear strain

$$\Delta\epsilon_q^p = \Delta\epsilon_p^p \frac{q}{M^2(p^{'} - p_c^{'}/2)}$$

Step 10: elastic shear strain

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G}$$

Step 11: Total volumetric strain

$$\Delta\epsilon_q = \Delta\epsilon_q^e + \Delta\epsilon_q^p$$

$$\epsilon_p = (\Delta\epsilon_p^e)inital + \Delta\epsilon_p$$

$$\epsilon_q = (\Delta\epsilon_q^e)initial + \Delta\epsilon_q$$

Step 10:Axial strain

$$\epsilon_1 = \epsilon_q + \epsilon_p/3$$

6. PREDICTION OF STRESS-STRAIN RESPONSE OF MCC FOR LIGHTLY OVERCONSOLIDATED SOIL UNDER UNDRAINED CONDITION:

The MCC model has the following steps.

Step 1: deviatoric stress at initial yield

$$q_y = Mp_0^{'}\sqrt{\frac{p_c^{'}}{p_0^{'}} - 1}$$

Step 2: Failure mean effective stress $p_f^{'} = \exp\left(\frac{e_t - e_0}{\lambda}\right)$ and deviatoric stress at failure

$$q_f = Mp_f^{'}$$

$$G = \frac{3p_0^{'}(1+e_0)(1-2v^{'})}{2k(1+v^{'})}$$

$p_0^{'}$= current mean effective stress

Step 3: elastic shear strain

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G}$$

Step 4: preconsolidation stress

$$p_c^{'} = (p_c^{'})_{prev}\left(\frac{p_{prev}^{'}}{p^{'}}\right)^{k/\lambda - k}$$

Step 5:deviatoric stress

$$q = Mp^{'}\sqrt{\frac{p_c^{'}}{p^{'}} - 1}$$

Step 6: Volumetric plastic strain

$$\Delta\epsilon_p^p = \Delta\epsilon_p^e = \frac{k}{1+e_0} ln\frac{p_{prev}^{'}}{p^{'}}$$

Step 7: plastic shear strain

$$\Delta\epsilon_q^p = \Delta\epsilon_p^p \frac{q}{M^2(p^{'} - p_c^{'}/2)}$$

Step 8: Volumetric elastic strain

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G}$$

Total volumetric strain $\Delta\epsilon_q = \Delta\epsilon_q^e + \Delta\epsilon_q^p$

Step 9:

Axial strain $\epsilon_1 = \epsilon_d = \Delta\epsilon_{q\,ini}^e + \Delta\epsilon_q$

Step 10: Total mean stress

$$p = p_0^{'} + {}^{q}/_{3}$$

Step 11: Excess pore water pressure

$$\Delta u = p - p'$$

7. METHODOLOGY AND EXPERIMENTAL INVESTIGATION:

The entire study has been conducted on two samples collected from different places of Burla. Initially experiments were conducted to find out different properties of soil such as index properties, grain size distribution. Then Consolidation test and Tri-axial test were conducted with drained and undrained conditions to investigate stress- strain response with respect to different drainage conditions. Two samples of triaxial tests were performed during this study with different drainage conditions under confining pressure 100kpa.

8. MCC PREDICTION AND COMPARISON TRI-AXIAL CONSOLIDATED UNDRAINED TEST, SAMPLE – 1

Table – 1(i) (MCC Parameters) 100kpa

1. λ=0.29, k= 0.05, M=1.22, N= 3.3, μ=0.3

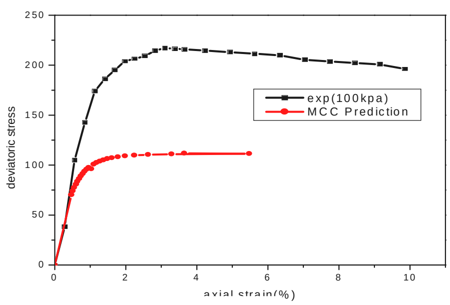

Fig-1(a)

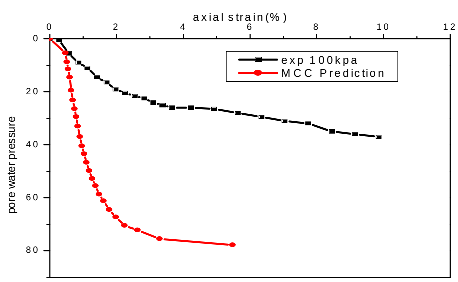

Fig – 1(b)

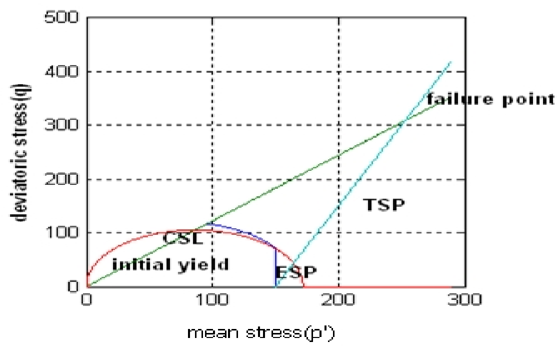

Fig-1(c)

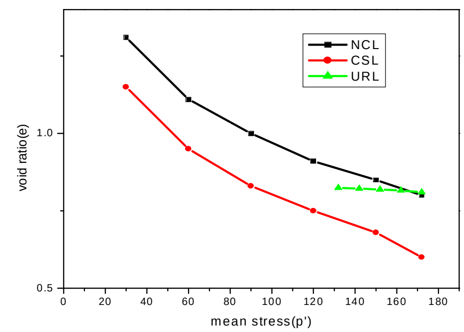

Fig-1(d)

Sample –2

Table – 2(i) (MCC Parameters) 100kpa

2. λ=0.21, k= 0.04, M=1, N= 2.65, μ=0.3

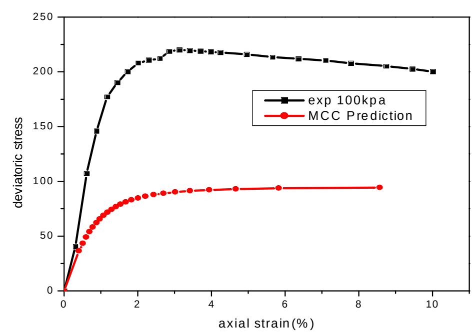

Fig-2(a)

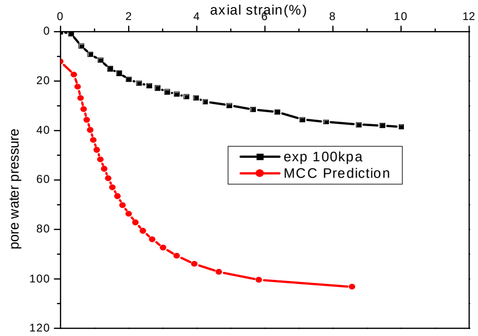

Fig –2(b)

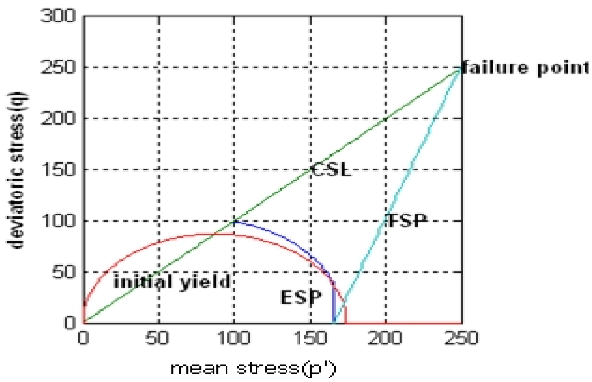

Fig-2(c)

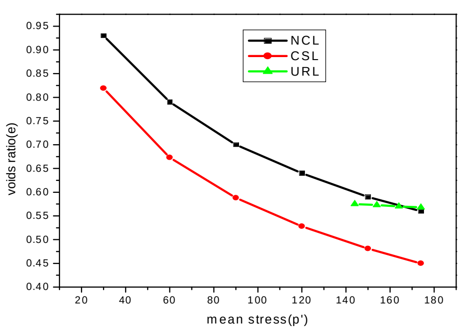

Fig –2(d)

Tri-axial consolidated drained
Sample -1,Table-3(i)(MCC Parameters)100 kpa
5.λ=0.17, k= 0.03, M=1.156, N= 2.69, μ=0.

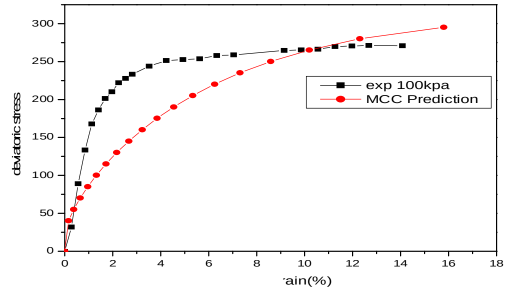

Fig-3(a)

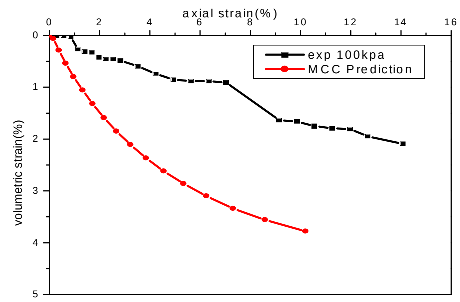

Fig-3(b)

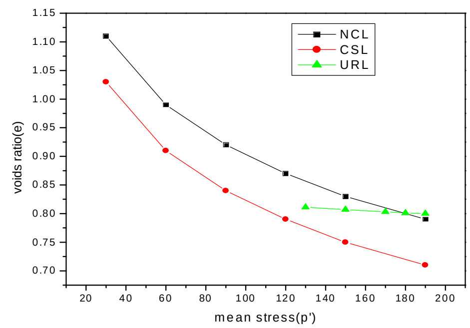

Fig-3(c)

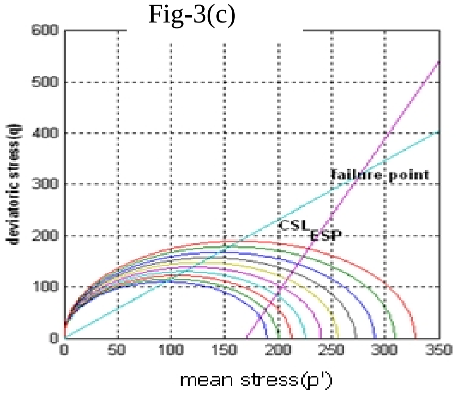

Fig-3(d)

Sample – 2

Table – 4(i) (MCC Parameters) 100kpa

7. λ=0.07, k= 0.01, M=0.94, N= 1.91, μ=0.3

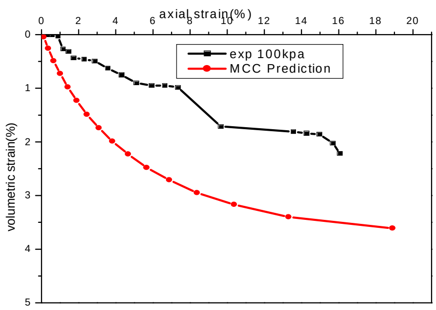

Fig-4(a)

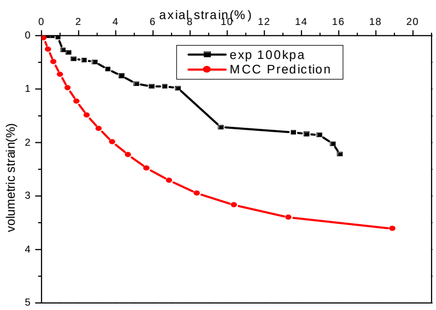

Fig-4(b)

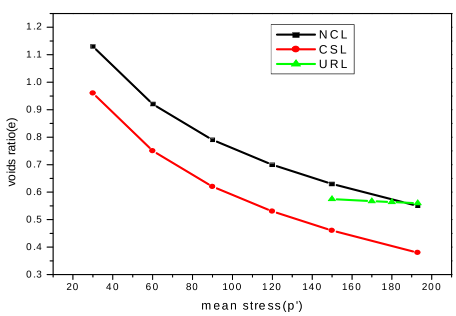

Fig-4(c)

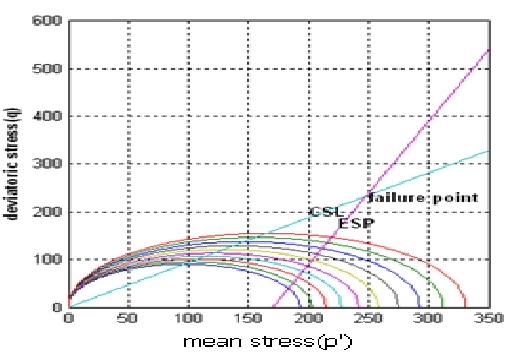

Fig-4(d)

9. RESULTS AND DISCUSSION:

A series of triaxial tests have been conducted in both drained and undrained conditions of two samples. Triaxial tests have been performed to determine the MCC model parameters and to know the stress-strain behavior of soil and provide data for validation of the MCC model to predict the behavior of soil under loading and unloading conditions. All the soils are normally consolidated and lightly over consolidated soil. The yield surface of every soil sample with critical state line and stress path have been plotted by using MATLAB code to know the failure point based on MCC model. The model parameters have been found out from (e, p') space graph from fig.1(d), fig.2(d) under undrained conditions and from fig.7(c), fig.3(c), fig.4(c) under drained conditions that is from critical state line, normal consolidation line and swelling line.

CONCLUSIONS

A series of triaxial tests of two samples in both drained and undrained conditions have been made in this project work to explore the theoretical model of MCC in comparison with experiment and response of soil based on MCC model. From the results and discussions the following conclusions have been made.

1. These test data were plotted in terms of the stress-strain invariants defined by the MCCM.A comparison is made between the predictions given by MCC model with experimental data of different soil samples. From the above comparison it is observed that the model predicted results match well with experimental values under drained condition for all the samples tested, but in undrained condition substantive deviations are observed.
2. In drained case it was noted that the MCCM adequately predict the shear strain response based on the shear strain versus deviatoric stress plots.
3. The model parameters was found out from the graph of (e-p') space. Critical State Soil Mechanics has been applied to analyse this model at failure.

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