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Study of Camclay Models in Predicting the Behaviour of Residual Soils

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Abstract: *Of the Civil Engineering construction materials, soil is unique being a natural material, most often engineered, as it exists, unlike other processed or manufactured material like concrete or steel. Therefore it becomes necessary to characterize the soil appropriately based on factual data available at discrete locations. The discrete locations are the places where samples are extracted and in-situ tests are performed, these locations are chosen based on possible variations in soil conditions in the proposed area of construction of a civil engineering structure. Soil conditions between such discrete locations can be deduced by scientific principles. Understanding the behaviour of the soil is improved if rational simplications of realistic situations are made and analyses are performed. Such models are considered in the present investigation. The models include Cam-clay; Modified Cam-clay and wheeler .These models are essentially based on critical state framework. The critical state framework unifies stress-strain-volume/pore pressure characteristics so that the behaviour of soil under different loading conditions can be comprehensively understood. For the purpose of analysis, three types of model tests have been conducted. These tests include a series of undrained test conducted on soils with different liquid limits at the constant initial volume. Another series of undrained tests were performed by keeping the initial mean principle stress constant. Further a series of "constant p" test were also performed at constant initial volume as well as constant initial stress .From out of these model tests a relative comparison of the behaviour is presented which clearly brings out of the distinguish differences in the behaviour. An attempt has been made to understand the applicability of these models in predicting the behaviour of Residual soils of Tirupathi region .The data on tests conducted on 5 different soils have been analyzed for the purpose of understanding the applicability .The model parameters have been determined after careful analysis of the experimental results. A comparative study of the model predictions has been presented and the deviations explained.*

Keywords: *Characterization of the soil, Critical State framework, Cam-clay, Residual soils, Model Parameters*

I. Introduction

Advanced geotechnical design on soft clays has often been based on finite element analysis using isotropic elasto-plastic soil models, such as Modified Cam clay (Roscoe & Burland 1968). Natural soil deposits, however, tend to be highly anisotropic, due to the deposition process and subsequent loading history. Neglecting the anisotropy of soil behavior may lead to highly inaccurate predictions of soil response under loading.

During the last decade several anisotropic elasto-plastic soil models have been proposed (e.g. Banerjee & Yousif 1986, Dafalias 1987, Whittle & Kavvadas 1994, Newson 1997). Unfortunately some of these models predict unrealistic behavior for certain stress paths. Others are relatively complex and difficult for practicing engineers to understand or the determination of the model input parameters may require non-standard laboratory tests. As a result, the application of these models to practical geotechnical design is not common.

To make the adoption of anisotropic models for geotechnical design more feasible, an alternative elasto-plastic model for soft clayey soils was proposed by Wheeler (1997) and subsequently slightly modified by (Nää tänen et al. (1999). The main objective in developing the model was to provide a realistic representation of the influence of plastic anisotropy whilst still keeping the model relatively simple, so that there would be a realistic chance of widespread application in geotechnical design.

1.1 General Soil Types Encountered In the Region

The properties of residual soils have received increasing attention from geotechnical engineers in recent years. In particular, the extent to which conventional soil mechanics concepts are applicable to residual soils have been addressed by a number of workers in this field. There appears to be a widely held view that the direct applications of such concepts to residual soils is likely lead to misleading conclusions about the properties of at least some of these soils. The following quotations illustrate these views. The development of the 'classical' concepts of soil mechanics has been based almost exclusively on the investigation of sedimentary deposits of soil. These concepts have been found almost universally inapplicable to the behavior of residual soil, and misleading if inadvertently applied (Vaughan 1988).

1.2 RESIDUAL SOILS VERSUS TRANSPORTED SOILS

Residual soils are formed by the in-situ physical and chemical weathering of underlying rock, while sedimentary soils are formed by a process of erosion and transportation followed by deposition and consolidation under their own weight. In addition, the latter may undergo further alteration after deposition due to processes such as secondary consolidation, leaching and thixotropic effects (Bjerrum 1967 a). Unloading processes may produce over consolidated clays. Sedimentary soils may also be subjected to the development of inter particle bonds as well as other post deposited effects (Bjerrum 1967b). As bonds develop with time in residual soils, hardening occurs. The reverse will be normal trend, as bonds and cementation are broken down by the weathering process

II. Need for Predictive Models

2.1 It has been the fervent desire of geotechnical engineers to find simpler and quicker methods of testing, using the data of which the above questions can be satisfactorily answered. Any attempt to interpret, and predict soil behavior will go a long way to:

- Form an independent check on the laboratory investigations.
- Increase the level of confidence to handle extensive test data
- Optimize time and cost to obtain acceptable engineering parameters
- Monitor progressive changes in ground engineering and
- Resort to observational approach in geotechnical engineering more effectively

2.2 To achieve these objectives the two approaches pursed by researches and engineers are:

- An engineering approach in which empirical methods of correlations .essentially based on experimental observations are developed.
- Micro-mechanistic approaches which have a strong bias towards the basic sciences and consider the nature and equilibrium of forces at micro level and associated interaction between different phases of multiphase systems

2.3 A large number of scientific publications of the recent past testify the involvement in the above approaches. Although both the above paths have made great strides independently, they have not been always complimentary to each other .In the first approach of property correlations though empirical means, a scientific function basis cannot always be found. In the later approach, macrobehaviour characterized in terms of micro parameters are not always amenable for measurement. Perhaps the most desirable situation is

- To identify the basic mechanisms responsible for the observed behavior
- To link the empirical and basically approaches, both the behavioral and parametric and
- To formulate appropriate physical models for prediction of engineering properties within the limits of accuracy acceptable at engineering level.

Such an approach may be desirable than to attempt exact micro-analysis with the parameters which are difficult to measure and link with micro-behavior. The cardinal aim could be to establish new meaningful connections between scattered facts and hence uncover new patterns so as to result in a holistic approach of analysis and prediction.

2.4 The geotechnical engineering, is pursuing his field of endeavor has many diverse encounters with rocks and soils, being the materials of geological origin. In nature, rock formation and soil deposits of widely varying characteristics can very well be expected. This is due to inherent nature and diversity of geological processes in such formations. As such a geotechnical engineer is confronted with greater degree of uncertainty in assigning the most probable engineering properties to these geological materials, than of other processed and manufactured construction materials.

III. Cam-Clay Models

3.1Original Cam-clay Model

The basic Cam-clay model was developed for axisymmetric coordinates, i.e., $\sigma_2^{'} = \sigma_3^{'}$ with spherical and deviatoric components given by the invariants of the from:

$$p^{'} = \left(\sigma_1^{'} + 2\sigma_3^{'}\right)/3 \text{ and } q = \left(\sigma_1 - \sigma_3\right) \qquad 1$$

And corresponding components of strain

$$\varepsilon_v = (\varepsilon_1 + 2\varepsilon_3) \text{ and } \varepsilon_s = 2/3(\varepsilon_1 - \varepsilon_3) \qquad 2$$

And the volumetric state of the soil is defined in terms of the specific volume $v = 1 + e$, where e is the void ratio.

The formulations have been extended to plain strain and general stress conditions also. The original Cam-clay had an yield surface (Roscoe surface) in the shape of bullet, derived from the work done or energy consideration, with an assumption that the plastic energy dissipation is by pure friction following the law:

$$W = MP^{'}\left|\varepsilon_s^p\right| \qquad 3$$

Where W is the plastic work done ε_s^p is the plastic shear strain and M is the friction factor. They used the normality rule and a failure criterion of extended vonmises type. The yield surface was assumed to expand isotropically and the hardening rule was obtained from the isotropic consolidation behavior of the soil, noting that each point on the v vs lnp' curve corresponds to state with q=0, on successively hardening yield surface. Linearization of consolidation and rebound behavior has been assumed in v-lnp` space. (Figure 1)

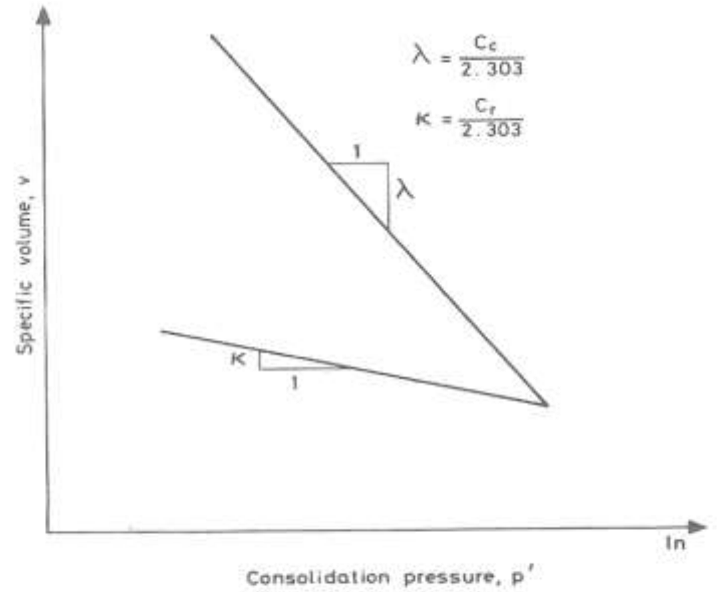

Fig.1 Representation of compression and swelling paths

The failure states defined by the critical state line was shown to be a unique curve in p' – q – v space. The projection of the critical state line on p'-v plane and p'-q plane indicated the following relationship respectively.

$$v = \Gamma - \lambda \ln p_f^{'} \qquad 4$$

$$q_f = Mp_f^{'} \qquad 5$$

Where Γ and λ are the intercept at unit pressure and slope of the critical state line respectively, M is the slope of this line in $q - p^{'}$ space. This implies that the projection of critical state line on $p^{'} - v$ plane is a line parallel to normal consolidation line (i.e., $v = N - \lambda \ln p^{'}$) and in $p^{'} - q$ plane it is a line passing through origin. The practical implications of these equations are that, if for a soil the initial state $(p^{'}, q, v)_f$ are uniquely determined for any stress path. The compacting or dilating state was identified based on:

For $(q / p^{'}) < M$, compression (wet state and weak at yielding)

For $(q / p^{'}) > M$, compression (dry state and strong at yielding).

The model assumed pure elastic behavior for stress states within the yield surface, and also assumed that elastic shear strain is zero.The elastic bulk modulus was obtained from the rebound part of the consolidation curve as $K = vp^{'} / k$ where k is the slope of the rebound line in v vs ln p' plot.

The derived yield curve was of the form:

$$q/Mp^{'} + \ln(p^{'}/p_x^{'}) = 1 \qquad 6$$

Where $p_x^{'}$ is the value of $p^{'}$ at the intersection of the yield curve with critical state line. A family of all such yield curves at different specific volumes forms a yield surface in $p^{'} - q - v$ space. The final expression for the yield surface is of the form:

$$q = Mp^{'}\left[\Gamma + (\lambda - k) - v - \lambda \ln p^{'}\right]/(\lambda - k) \qquad 7$$

On the dry side of critical state, i.e. for overconsolidated states, the yield surface was slightly modified later (Atkinson and Bransby, 1978) to match better the experimental results, by a planar surface of the form of Hvorslev's failure envelope. The complete yield surface is sometimes termed the 'state boundary surface' since it encloses all possible state of existence of a soil. Fig. 2 schematically shows the yield surface in $p^{'}$-q-v space, the Roscoe and Hvorslev surfaces on either side of the critical state line.

A differentiation of equation (3) with respect to v and separation of elastic strain ,would results in an expression for plastic volumetric strain increment in the form:

$$d\varepsilon_v^p = (\lambda - k)\left((M - q/p^{'})dp^{'} + dv\right)/Mp^{'}v \qquad 8$$

Applying normality rule, the shear strain component would be

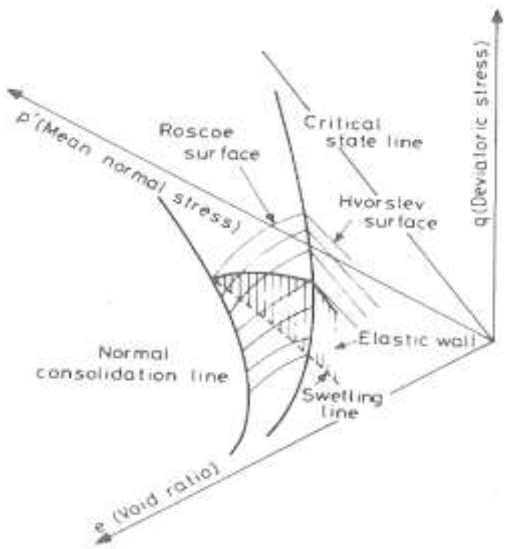

Fig.2 . Three dimensional representation of cam clay yield surface(Roscoe and Hvorslev surfaces)

$$d\varepsilon_s^p = d\varepsilon_v^p/(M - q/p^{'}) \qquad 9$$

Adapting an incremental working procedure, using equations (8) & (9), together with the elastic volumetric strain component, $\varepsilon_v^e = kdp^{'}/p^{'}v$,the entire stress strain behaviour for any loading path can be obtained. However the model predicts larger shear strains than the observed values at small shear stress level. To overcome this limitation, a modified version of the Cam-clay model, with a different energy dissipating mechanism was suggested. This is referred to as 'Modified Cam-clay model' (Roscoe and Burland, 1968).

3.2 Modified Cam-Clay Model

To account for the energy dissipated in plastic volume changes also, a slightly different mode of plastic energy dissipation was assumed as:

$$W/v = p^{'}\left[(dv^p/v)^2 + (Md\varepsilon_s^2)\right]^{1/2} \qquad 10$$

This form, on application of normality rule, resulted in the yield curve:

$$(p^{'} - p_x^2)^2 + q^2/M^2 = (p_x^{'})^2 \qquad 11$$

Where $p_x^{'} = p_0^{'}/2$. 12

The hardening rule was obtained from the consolidation behaviour as follows:

$$d\varepsilon_v^p = (\lambda - k)dp^{'}/vp^{'} \qquad 13$$

$$dp'_x = dp'_0 = vp' d\varepsilon_v^p / (\lambda - k) \qquad 14$$

As can be seen, from the above expressions the only soil parameters appearing are λ, k, and M .Fig. 3 represents the original and modified Cam-clay yield surfaces in triaxial plane.

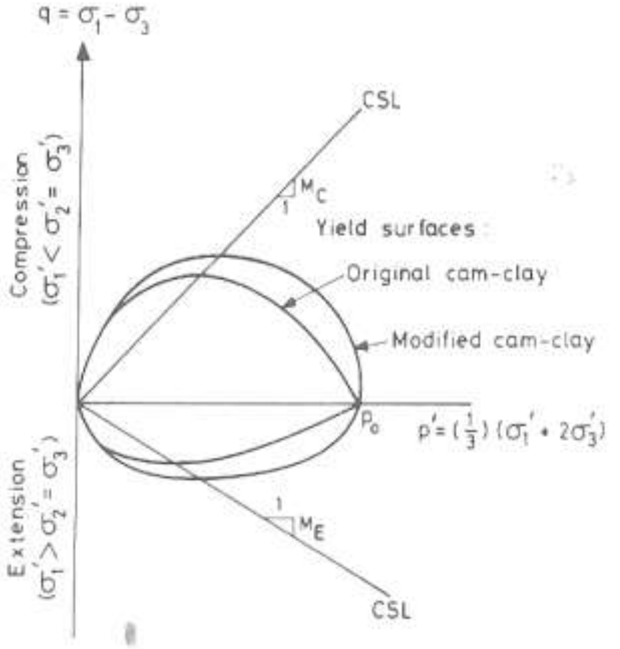

Fig. 3 Cam clay yield surfaces in p'-q plane

3.3 Wheeler Model

3.3.1 Model Formulation

The model is an extension of the critical state models, with anisotropy of plastic behavior represented through a rotational component of hardening. For the sake of simplicity, the model is presented here for the simplified stress space of the triaxial test, although it has already been extended to general three-dimensional stress space.

The model is applicable to tropical residual soils, where plastic deformations dominate. For simplicity, isotropy of elastic behavior is therefore assumed, and hence the elastic increments of volumetric and deviatoric strains are calculated as

$$d\varepsilon_v^e = \frac{\kappa dp'}{vp'}, \qquad d\varepsilon_v^e = \frac{dq}{vG'} \qquad 15$$

where κ is the slope of the swelling line, v is specific volume, G' is the elastic shear modulus and p' and q are the mean effective stress and deviatoric stress respectively.

The yield curve is sheared ellipse, as proposed by Dafalias (1987) and Korhonen & Lojander (1987), defined by $f = (q - \alpha p')^2 - (M^2 - \alpha^2)(p'_m - p')p' = 0$ 16

where M is the critical state value of stress ratio α (where α = q/p') and the parameters p_m' and α define the size and the inclination of the yield curve respectively (see Figure 3.7). The parameter α is a measure of the degree of plastic anisotropy of the soil. For the case of isotropy (α = 0), Equation 3.18 reduces to the Modified Cam clay yield curve.

In the interests of simplicity, an associated flow rule is assumed, and hence:

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2(\eta - \alpha)}{M^2 - \eta^2} \qquad 17$$

The greatest advantage of assuming an associated flow rule is that numerical implementation of the model is far simpler than with a non-associated flow rule. Experimental evidence by Graham *et al.*, (1983) and Korhonen & Lojander (1987) suggests that this assumption is reasonable for many residual soils.

The model incorporates two hardening laws. The first one describes changes in size of the yield curve and it is similar to that of Modified Cam clay:

$$dp'_m = \frac{vp'_m d\varepsilon_v^p}{\lambda - \kappa} \qquad 18$$

The second hardening rule predicts the change of inclination of the yield curve produced by plastic straining, representing the development or erasure of anisotropy with plastic strains. It is assumed that plastic volumetric strain attempts to drag the value of η towards an instantaneous target value χd(α) that is dependent on the current value of xv(η)whereas plastic shear strain is simultaneously attempting to drag $\chi_d(\eta)$ towards a different instantaneous target value $\chi_d(\alpha)$ (also dependent on α). The rotational hardening law is therefore (Näätä nen *et al.*,):

$$d\alpha = \mu[(xv\ (\eta) - \alpha)\ d\varepsilon v^p + \beta(x_d\ (\eta) - \alpha)d\varepsilon_d^p\ ] \qquad 19$$

The overall current target value for α will lie between χv(η) and $\chi_d(\eta)$ Constants μand β control, respectively, the absolute rate at which η heads towards its current target value and the relative effectiveness of plastic shear strains and plastic volumetric strains in determining the current target value.
Based on initial yield curve (Figure 3.7) Näätänen *et al.*, (1999) proposed the following expressions for χv(η) and $\chi_d(\eta)$:

$$\chi_v(\eta) = \frac{3\eta}{4} \qquad 20$$

$$\chi_d(\eta) = \frac{\eta}{3} \qquad 21$$

In practice the expression for χv(η) in Equation (20) means that plastic volumetric strains attempt to align the yield curve approximately about the current stress point (see Wheeler, 1997). The proposal for $\chi_d(\eta)$ in Equation (21) corresponds to a significant degree of anisotropy at critical states (α = M/3 at η= M), as suggested by Näätä nen *et al.,* (1999).

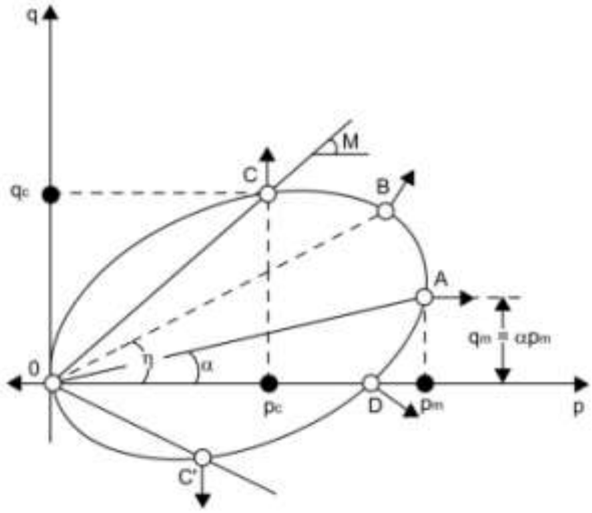

Fig.4 Initial yield curve

IV. Prediction of Soil Behavior

4.1 Introduction

The Cam clay models describe the Stress-Strain-Volume change behavior in a united and coherent manner. The original Cam clay model generally over predicts the shear strains where as the predictions based on the Modified Cam clay model are found to agree with the experimental results quite closely particularly of normally consolidated clays. A brief description of evaluation of plastic strain from original Cam clay and Modified Cam clay models is presented in the following sections.

4.2.1 Procedure for Determination of Critical State Model Parameters:

The critical state parameters viz. N, λ, κ, Γ and M can be evaluated based on two test results. Isotropic compression and swelling paths give rise to λ and κ respectively. The specific volumes corresponding to 1 kPa on normal compression line (Figure 3.1) and critical state line in v-lnp space indicate N and Γ respectively. The value of M can be obtained as a ratio of deviatoric stress to mean principal stress at critical state in retrospect (Figure 3.2), only two test results are required to evaluate the critical state parameters which are useful in understanding the soil behavior under different loading conditions.

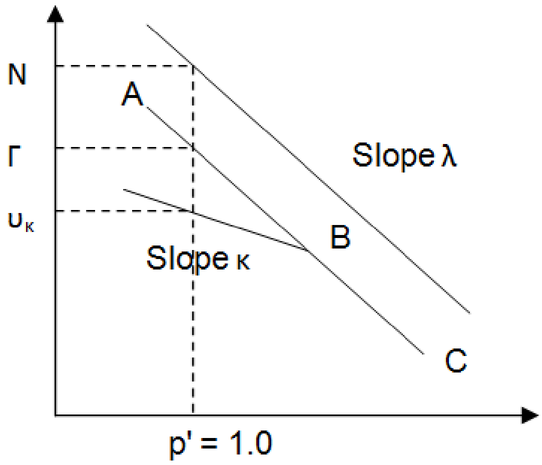

Fig.5 Critical state parameter κ, N, Γ and λ

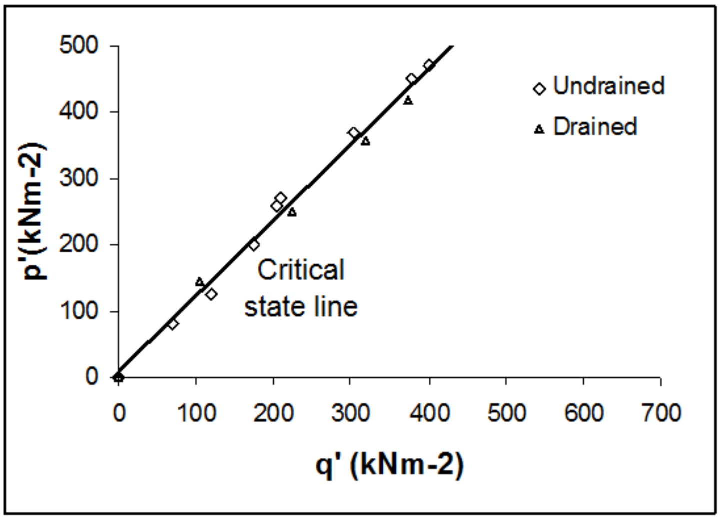

Fig.6 Critical state parameter M

4.2.2 Calculations of Plastic Strains of Cam clay model

From Cam clay model

The flow rule is given by

$$\frac{\delta\varepsilon_v^p}{\delta\varepsilon_s^p} = M - \frac{q'}{p'} \tag{22}$$

The yield curve equation

$$\frac{q'}{Mp'} + \ln\left(\frac{p'}{p'_x}\right) = 1, \tag{23}$$

Finally,

$$\delta v = -\lambda\frac{\delta p'}{p'} - \frac{(\lambda-\kappa)\delta q'}{Mp'} + \frac{(\lambda-\kappa)q'\delta q'}{Mp'^2} \tag{24}$$

We know that

$$\delta v^p = \delta v - \delta v^e \tag{25}$$

$$\delta v_e = -\kappa\left(\frac{\delta p'}{p'}\right) \tag{26}$$

$$\delta v^p = -\frac{(\lambda-\kappa)}{Mp'}\left[\left(M - \frac{q'}{p'}\right)\delta p' + \delta q'\right] \tag{27}$$

Plastic volumetric strain is

$$\delta\varepsilon_v^p = -\frac{\delta v^p}{v} = \frac{(\lambda-\kappa)}{Mp'}\left[\left(M - \frac{q'}{p'}\right)\delta p' + \delta q'\right] \quad (28)$$

Plastic shear strain is

$$\delta\varepsilon_s^p = \frac{1}{M - \dfrac{q'}{p'}}\delta\varepsilon_v^p \quad (29)$$

By calculating the stresses and strains incrementally, based the above expressions, for a given stress path the stress strain behavior can be predicted (See appendix-1).

4.3 Modified Cam Clay Model

In the modified Cam clay model ,the dissipated energy dW is assumed as

$$dW = p\sqrt{\left(d\varepsilon_v^p\right)^2 + M^2\left(d\varepsilon_s^p\right)^2} \quad (30)$$

This leads to

$$\frac{d\varepsilon_s^p}{d\varepsilon_v^p} = \frac{2\eta}{M^2 - \eta^2} \quad (31)$$

Therefore,

$$\psi_{cm} = \frac{M^2 - \eta^2}{2\eta} \quad (32)$$

Here the subscript cm denotes modified Cam clay model

Once ψ is known, the yield locus for the modified Cam clay model can found by integrating Eq(11-13a).that is,

$$\int_0^\eta \frac{2\eta d\eta}{M^2 + \eta^2} = -\int_{p_0}^{p} \frac{dp}{p} \quad (33)$$

$$\ln\left(M^2 + \eta^2\right) - \ln\left(M^2\right) = -\ln p + \ln p_0 \quad (34)$$

Simplified leads to Or

$$\frac{M^2 + \eta^2}{M^2} = \frac{p_0}{p} \quad (35)$$

$$M^2 p^2 - M^2 p_0 p + q^2 = 0 \quad (36)$$

This is the equation of an ellipse on q-p plot. Substituting the value of ψ_{cm} from 4.11 , One can write the following expression for incremental quantities based on the modified Cam clay model:

$$d\varepsilon_v^p = \frac{\lambda - k}{1+e}\left(\frac{dp}{p} + \frac{2\eta d\eta}{M^2 + \eta^2}\right) \quad (37)$$

$$d\varepsilon_v = \frac{\lambda}{1+e}\left[\frac{dp}{p} + \left(1 - \frac{k}{\lambda}\right)\frac{2\eta d\eta}{M^2 + \eta^2}\right] \quad (38)$$

$$d\varepsilon_s^p = \frac{\lambda - k}{1+e}\left(\frac{dp}{p} + \frac{2\eta d\eta}{M^2 + \eta^2}\right)\frac{2\eta}{M^2 - \eta^2} \quad (39)$$

4.4 Wheeler Model

The model is an extension of the critical state models, with anisotropy of plastic behavior represented through a rotational component of hardening. For the sake of simplicity, the model is presented here for the simplified stress space of the triaxial test, although it has already been extended to general three-dimensional stress space.

The model is applicable to tropical residual soils, where plastic deformations dominate. For simplicity, isotropy of elastic behavior is therefore assumed, and hence the elastic increments of volumetric and deviatoric strains are calculated as

$$d\varepsilon_v^e = \frac{\kappa dp'}{vp'}, \qquad d\varepsilon_v^e = \frac{dq}{vG'} \tag{40}$$

where κ is the slope of the swelling line, v is specific volume, G' is the elastic shear modulus and p' and q are the mean effective stress and deviatoric stress respectively.
The yield curve is sheared ellipse, as proposed by Dafalias (1987) and Korhonen & Lojander (1987), defined by

$$f = (q - \alpha p')^2 - (M^2 - \alpha^2)(p'_m - p')p' = 0 \tag{41}$$

where M is the critical state value of stress ratio α (where α = q/p') and the parameters p_m' and α define the size and the inclination of the yield curve respectively (see Figure 5.1). The parameter α is a measure of the degree of plastic anisotropy of the soil. For the case of isotropy (α = 0), Equation 5.2 reduces to the Modified Cam clay yield curve.
In the interests of simplicity, an associated flow rule is assumed, and hence:

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2(\eta - \alpha)}{M^2 - \eta^2} \tag{42}$$

The model incorporates two hardening laws. The first one describes changes in size of the yield curve and it is similar to that of Modified Cam clay:

$$dp'_m = \frac{vp'_m d\varepsilon_v^p}{\lambda - \kappa} \tag{43}$$

The second hardening rule predicts the change of inclination of the yield curve produced by plastic straining, representing the development or erasure of anisotropy with plastic strains. The rotational hardening law is therefore (Näätänen *et al.*,):

$$d\alpha = \mu[(xv(\eta) - \alpha)\, d\varepsilon v^p + \beta(x_d(\eta) - \alpha) d\varepsilon_d^p\,] \tag{44}$$

The overall current target value for α will lie between χv(η) and χ_d(η). Constants μ and β control, respectively, the absolute rate at which α heads towards its current target value and the relative effectiveness of plastic shear strains and plastic volumetric strains in determining the current target value.
Based on initial yield curve (Figure 5.1) Näätänen *et al.,* (1999) proposed the following expressions for χ_v χv(η) and χ_d(η):

$$\chi_v(\eta) = \frac{3\eta}{4} \tag{46}$$

$$\chi_d(\eta) = \frac{\eta}{3} \tag{47}$$

In practice the expression for χv(η)in Equation 5.6 means that plastic volumetric strains attempt to align the yield curve approximately about the current stress point (see Wheeler, 1997). The proposal for χ_d(η) in Equation 5.7 corresponds to a significant degree of anisotropy at critical states (α = M/3 at η= M), as suggested by Näätä nen *et al.,* (1999).

V. Evaluation of Model Parameters

The model involves 7 soil constants: 5 conventional parameters from Modified Cam clay λ, κ, M, G' (or ν') and Γ and two additional parameters relating to the rotational hardening (α and μ). In addition, the initial state of the soil is defined by the stress state and the initial values of the parameters p_m' and α defining the initial size and inclination of the yield curve. If the initial value of specific volume v is also defined, this replaces the requirement to define a value for the parameter α (the intercept of the critical state line in the v: ln p' plane).
Values of the soil constants λ, κ, M, Γ and G' can be measured in laboratory tests using relatively standard procedures. This section therefore concentrates on procedures for evaluating the remaining two soil constants (β and μ) and the initial values of the parameters p_m' and α.

5.1 Initial inclination of yield surface (α)

Initial values of the parameters p_m' and α, defining the size and inclination of the yield curve resulting from the previous stress and strain history of the soil deposit, should ideally be determined by conducting triaxial stress probes on several identical soil samples along different stress paths, in order to identify a number of points on the yield curve. Unfortunately, in practice, this would often be unfeasible or unduly time-consuming. There is, however, a simpler method of estimating an initial value of α, if it can be assumed that the previous history of the soil deposit is restricted to simple one-dimensional (K_o) loading, and possible unloading, to a normally consolidated or lightly overconsolidated state.
If the normally consolidated value of K_o can be measured or estimated, perhaps by using Jaky's simplified formula ($K_o \approx 1 - \sin\Phi'$), this can be used to calculate a corresponding value of stress ratio η_{Ko}. The model

predicts that only one value of yield curve inclination α would produce one-dimensional straining for continuous loading at this stress ratio η_{Ko}, and this therefore provides an estimate for the initial value of α.
Assuming that elastic strains are much smaller than plastic strains, which is often true for very soft soils, one-dimensional straining (zero lateral strain) corresponds to:

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2}{3} \tag{48}$$

Combining Equation 5.8 with the flow rule of Equation 5.3, the yield curve inclination corresponding to one-dimensional consolidation α_{Ko} is given by:

$$\alpha_{K_o} = \frac{\eta_{K_o}^2 + 3\eta_{K_o} - M^2}{3} \tag{49}$$

5.2 Initial size of yield curve (p'$_m$)

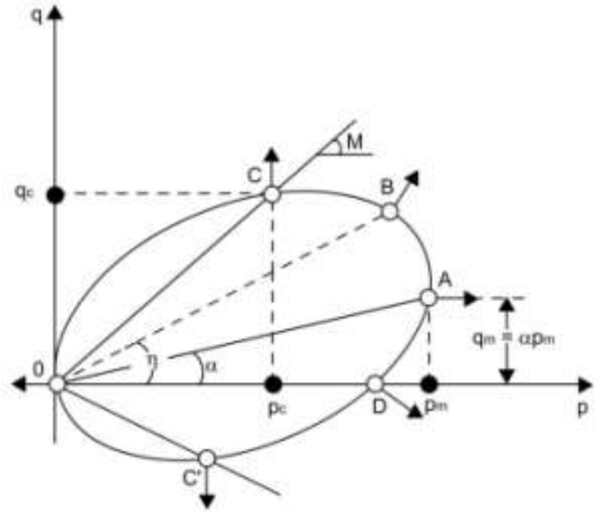

Fig.7 Initial yield curve

If the initial inclination α of the yield curve can be estimated, using the procedure outlined above, then only one point on the yield curve is required in order to calculate an initial value for the parameter p'$_m$, which defines the size of the curve. Ideally this, single yield point would be identified by either isotropic or K_o consolidation in a triaxial apparatus. Alternatively, one-dimensional consolidation in an oedometer would be possible, but this would require either measurement or estimation of radial stress, in order to fully define the stress state. Estimation of a point on the yield curve without the performance of any laboratory tests, from a knowledge of the maximum overburden stress applied to the soil deposit, would rarely be satisfactory, because of uncertainties about the depositionsal history and because of the possibility of an increase in the yield stress above the maximum pressure previously applied, due to the effects of ageing or inter-particle bonding (Burland, 1990).

5.3 Soil constants β

The model parameter β defines the relative effectiveness of plastic shear strains and plastic volumetric strains in rotating the yield curve. For plastic loading along any constant β stress path, α will ultimately tend to a final equilibrium value, which can be found by setting dα = 0 in Equation 5.5 and combining with Equation 5.3. This leads to a quadratic equation for α for a stress path with any constant value of η (see Näätänen *et al*., 1999):

$$(\frac{3}{4}\eta - \alpha)(M^2 - \eta\alpha^2) = -2\beta(\frac{1}{3}\eta - \alpha)(\eta - \alpha) \tag{50}$$

For loading at the K_o stress ratio η_{Ko}, Equation 5.10 shows that only one value of β will result in a value of α corresponding to α_{Ko} from Equation 5.9. Combining Equation 5.9 and Equation 5.10 gives the following expression for the required value of β:

$$\beta = \frac{3(M^2 - \eta_{K_0}^2 - 3\eta_{K_0}/4)}{2(\eta_{K_0}^2 - M^2 + 2\eta_{K_0})} \tag{51}$$

Knowledge of the value of K_o (and hence η_{Ko}) can therefore be used to determine an appropriate value for β, using Equation 4.22.

5.4 Soil constant μ

The model parameter μ controls the rate, at which α tends towards its current target value. It is difficult to devise a simple and direct method for experimentally determining the value of μ for a given soil. The only solution would appear to be to conduct model simulations with several different values of μ and then to compare these simulations with observed behavior in order to select the most appropriate value for μ. The type of experimental test required would be one involving significant rotation of the yield curve. Comparisons of observed and predicted behavior could then be made in terms of both the degree of rotation of the yield curve (identified experimentally by unloading and then reloading along a different stress path) and the observed pattern of straining.

In practice, performing suitable laboratory tests and then undertaking model simulations with different values of μ may not be feasible in a practical design scenario. In such a situation, the best course of action may be simply to select a standard default value for μ. This is considered below.

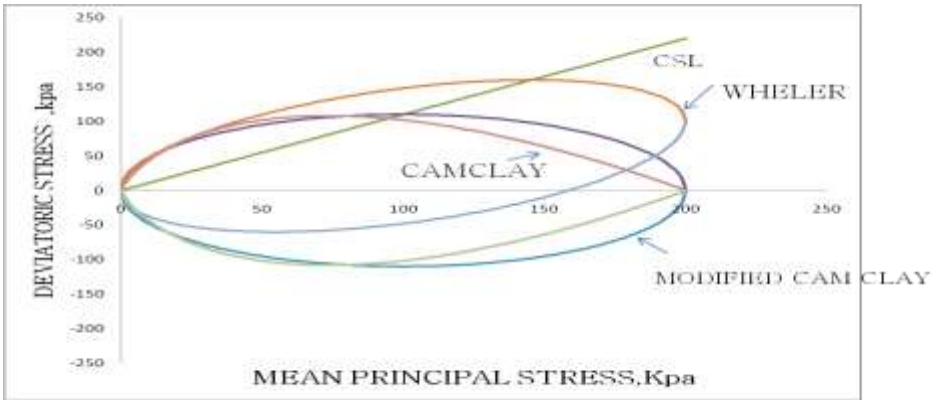

Fig.8 Camclay Models and Wheeler Model Yield Surface

VI. Model Tests

Based on the features explained the model tests have been simulated to bring out the comparison of stress strain behavior under different loading conditions. A series of undrained have been simulated by keeping the initial volume of the tests for different soils. Another series of undrained tests have been conducted for which the initial normal stress has been kept constant. In another series of tests similar test conditions have been applied by allowing the drainage. For bringing out further comparison, constant 'p' tests have been conducted for similar initial test conditions as before.

6.1 Undrained Tests

6.1.1 Undrained Tests for the samples with same initial volume

Soils with different liquid limits are considered. The range of liquid limit considered is 40-80. The normal compression lines are shown in Fig.9

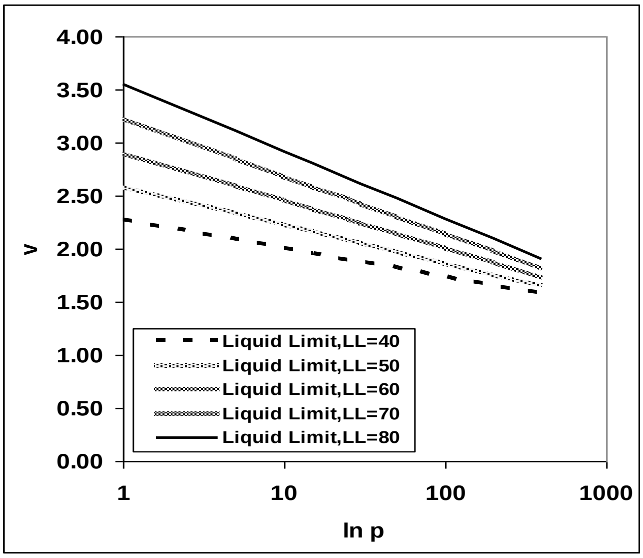

Fig.9 Compression paths in v-lnp plot

The slopes of the compression lines for different liquid limit values are given in Table 1.The values of M are so chosen that the value decreases with increase in liquid limit.

Table 1 Critical state constants for the range of liquid limit considered for the model tests

Serial Number	Liquid Limit, W_L %	Compression Index, C_c	Slope of compression line in specific volume-ln(p) plot, λ	Slope of Swell line in specific volume-ln(p) plot, k	M	N	Γ
1	40	0.27	0.117238	0.02931	1.3	2.272	2.184
2	50	0.36	0.156318	0.039079	1.2	2.59	2.473
3	60	0.45	0.195397	0.048849	1.1	2.908	2.761
4	70	0.54	0.234477	0.058619	0.9	3.226	3.050
5	80	0.63	0.273556	0.068389	0.85	3.544	3.339

As reported earlier, three series of model tests have been performed to analyse the test results obtained by conventional Cam clay models. In the first series undrained tests were conducted by keeping the volume constant for all the soil samples. The values of normal stress for the same initial specific volume as obtained from the Fig. 9 are shown in the following Table 1.

Table 2

Volume,v_0	2.0	2.0	2.0	2.0	2.0
Mean Principal Stress,p_0	10	40	100	198	298
Liquid Limit,%	40	50	60	70	80

It may be seen that as the liquid limit increases the mean principal stress required holding the sample at same volume increases. The strength of the soil increases with liquid limit, if the soil sample of higher liquid limit is to be held at same void ratio of soil sample of lower liquid limit. Further the pore water pressure for soil sample of higher liquid limit would be higher and is found to increase at higher rate with increase in liquid limit.

The model tests performed for modified Cam Clay for the same initial conditions as used for Cam clay yield stress-strain response as shown in **Fig.10**. It may be seen that the strain experienced by the sample for reaching the same stress values is significantly lower compared to the Cam clay. The Deviatoric stress at failure is relatively higher than the deviatoric stress at failure for the Cam clay. Further the pore pressures developed are lower than the Cam clay model.

The predictions form Wheeler model are found to be at variance in comparison to the predictions obtained from the other models. The strain softening may be noticed in the stress-strain response. The softening is noticed at lower strains for soils of higher liquid limit, as may be seen from the **Fig.15**

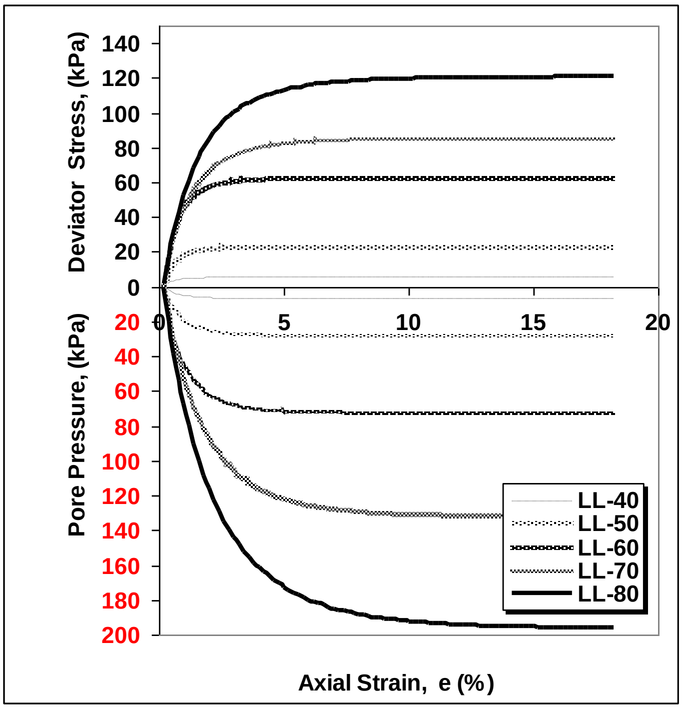

Fig.10 Stress-strain-pore pressure response for Cam Clay

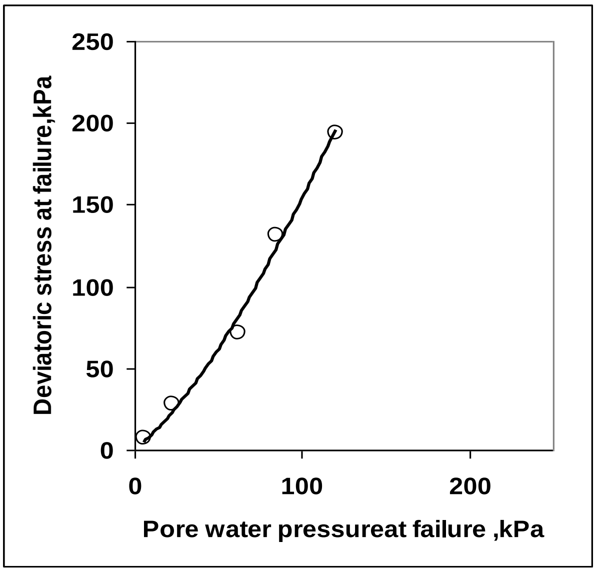

Fig.11 Deviatoric stress and pore pressure at failure

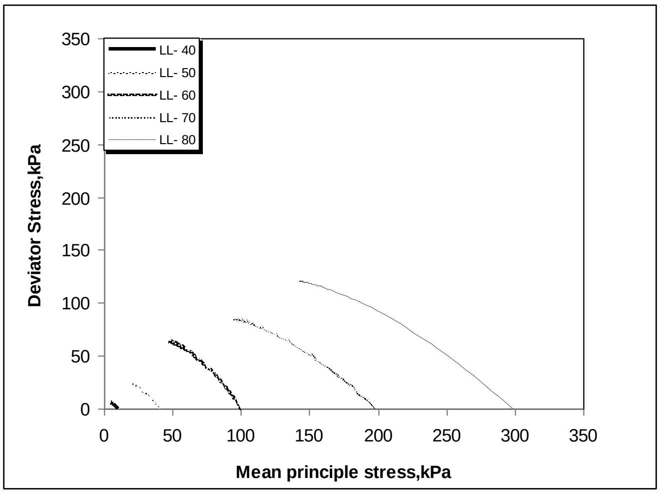

Fig.12 Stress Paths for different liquid limit values from Cam clay

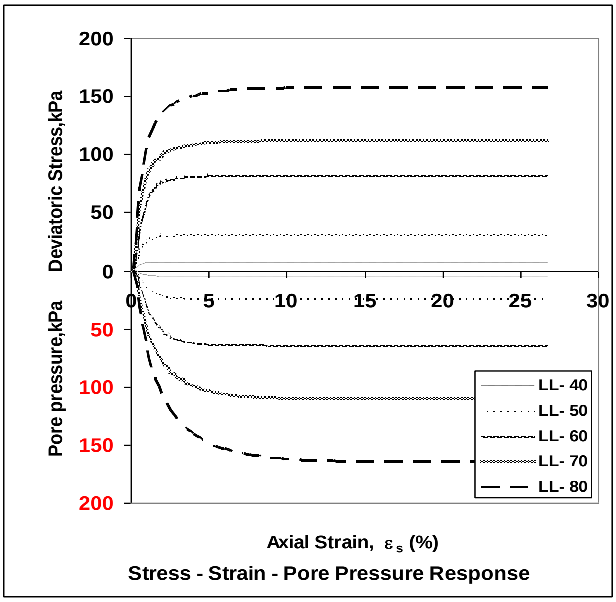

Fig.13 Modified Cam clay predictions for constant initial volume

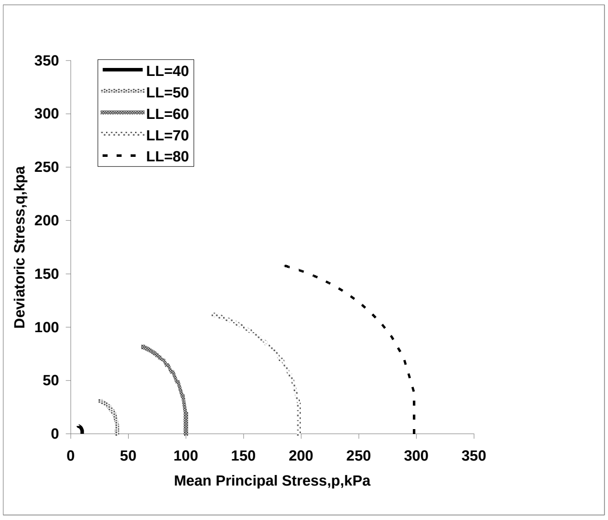

Fig.14 Stress Paths for different liquid limit values from modified Cam clay

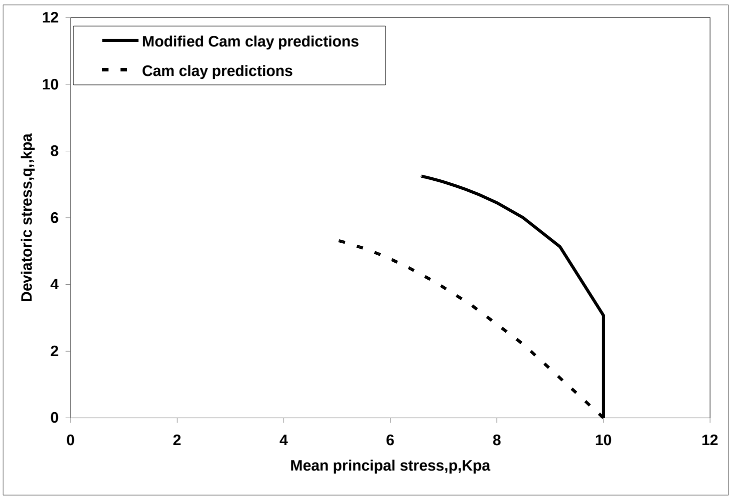

Fig.15 Comparison of predictions of Cam clay and Modified cam clay for constant volume tests

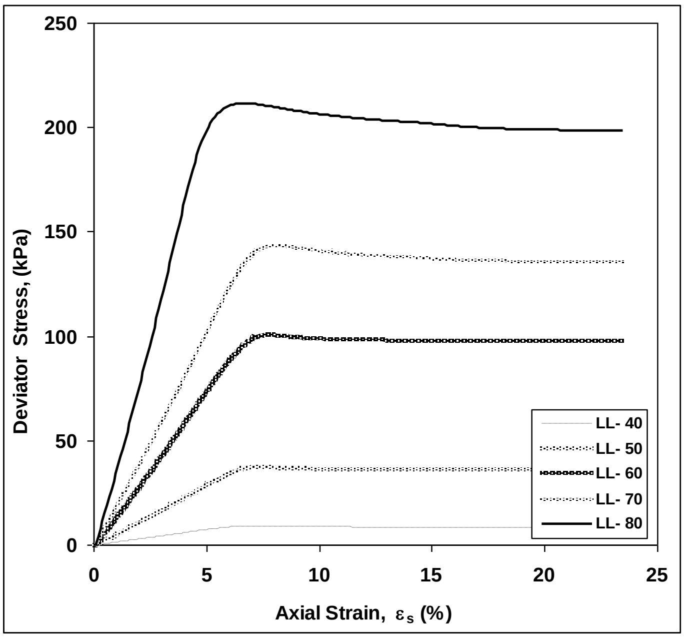

Fig.16 Prediction of Stress-strain response from Wheeler Model

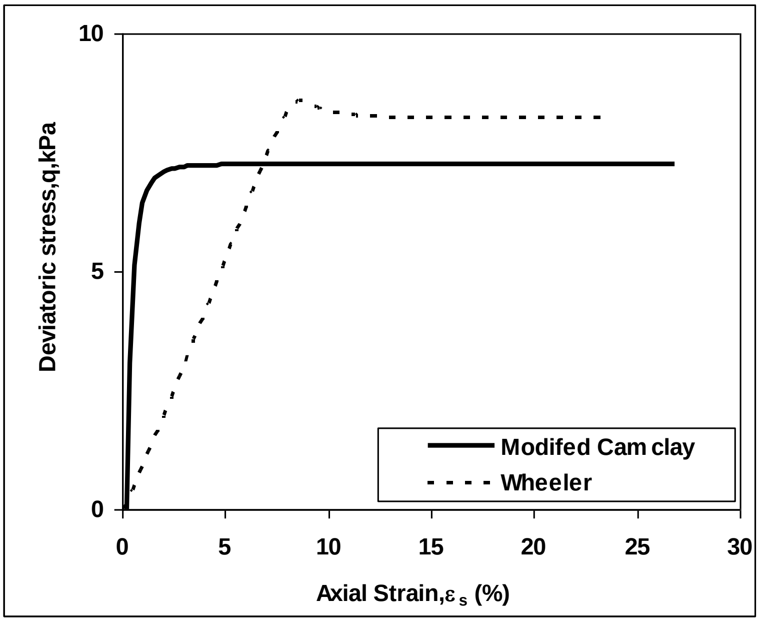

Fig.17 Comparison of Stress-Strain response

6.1.2 Undrained Tests for the samples with same initial mean principal stress

A series of model tests were conducted to predict the undrained shear response for different soils, when tested with same initial mean principal stress. This enables the comparative study to understand the effect of normal stress on the stress stain response of different clays.

The stress state for this condition can be seen in the Table 3

Table 3 conditions for the same initial normal stress

Volume,v_0	1.73	1.87	2.01	2.15	2.28
Mean Principal Stress,p_0	100	100	100	100	100
Liquid Limit,%	40	50	60	70	80

It may be seen from the table that

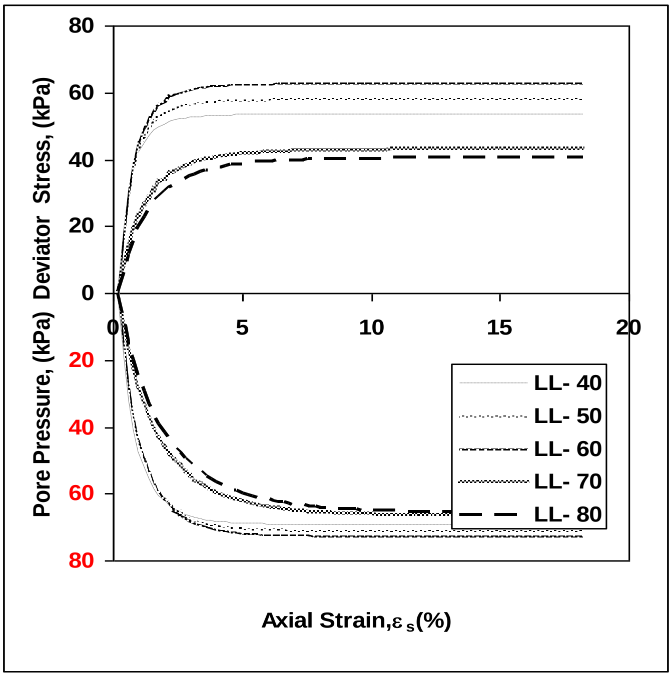

Fig.18 Stress–strain–pore pressure response for the soil samples of constant initial p test

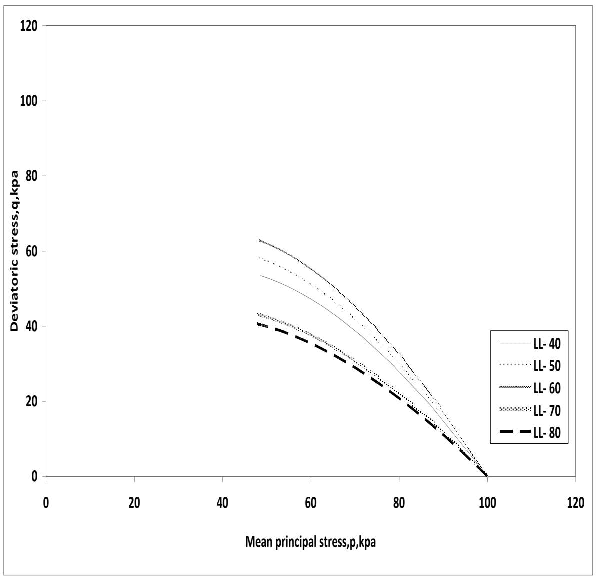

Fig.19 stress paths in undrained shear for the soil samples of constant initial p test

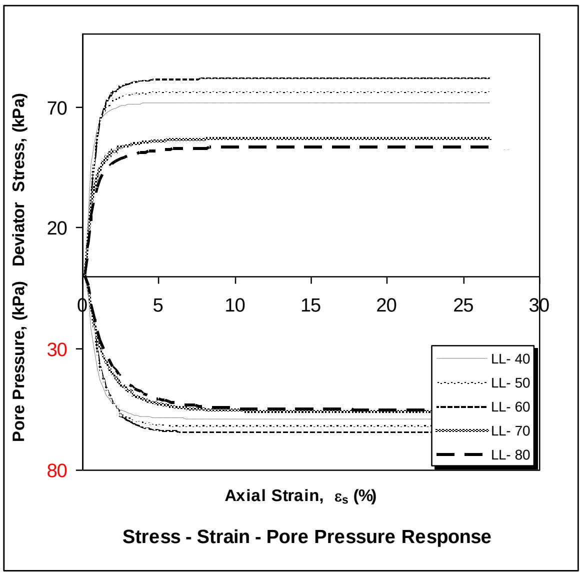

Fig.20 Modified Cam clay Stress–strain–pore pressure response for the soil samples of constant initial p test

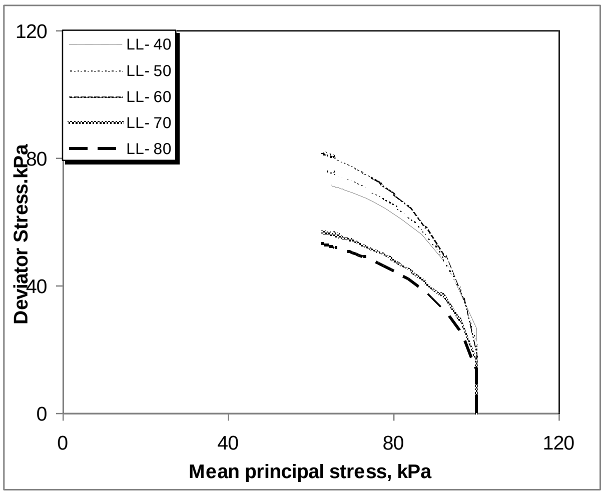

Fig.21 Modified Cam clay stress paths in undrained shear for the soil samples of constant initial p test

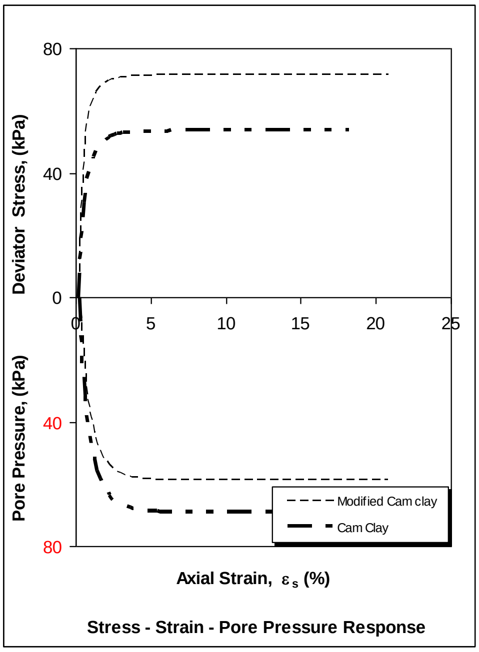

Fig.22 Comparison of Stress-Strain response

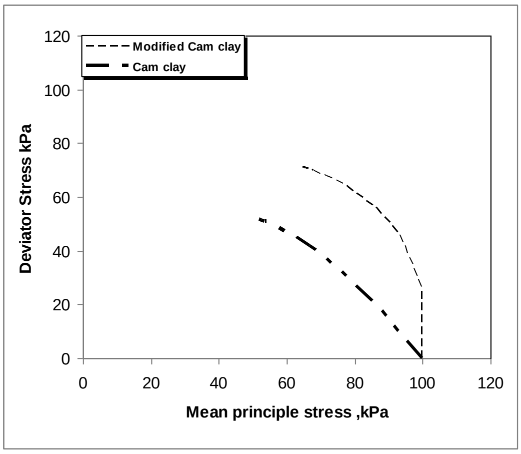

Fig.23 Comparison of predictions of Cam clay and Modified cam clay for constant Mean principle stress tests

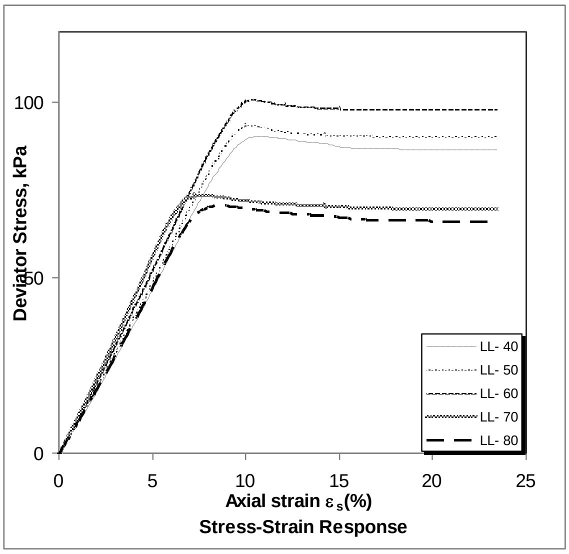

Fig.24 Prediction of Stress-strain response from Wheeler Model

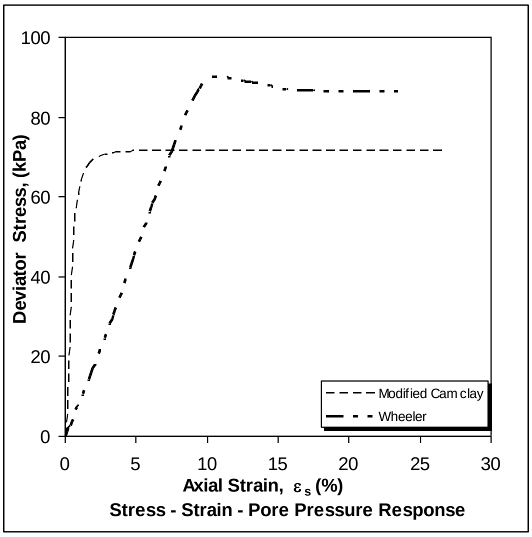

Fig.25 Comparison of Stress-Strain response

6.2 Drained Tests

6.2.1 Drained Tests for the samples with same initial volume

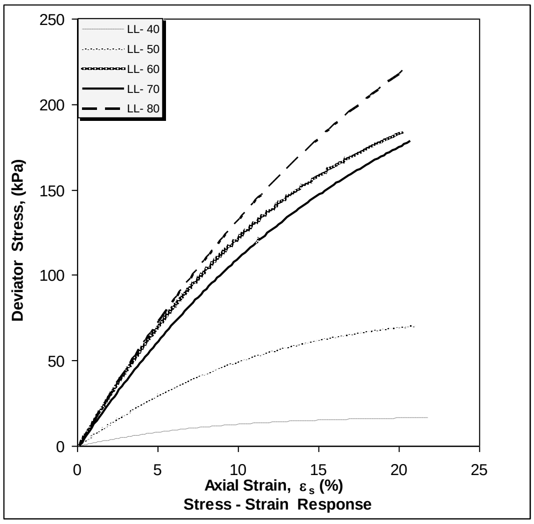

Fig.26 Cam clay predictions for constant initial volume

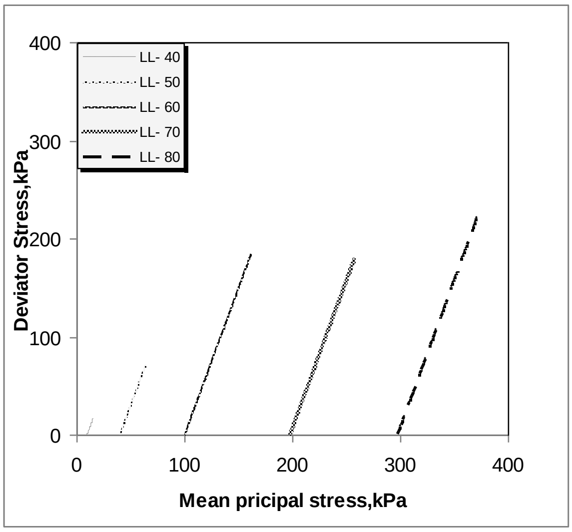

Fig.27 Stress Paths for different liquid limit values from Cam clay

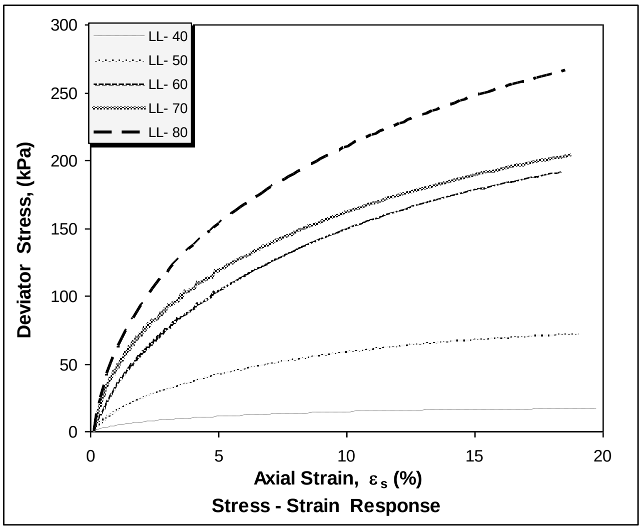

Fig.28 Modified Cam clay predictions for constant initial volume

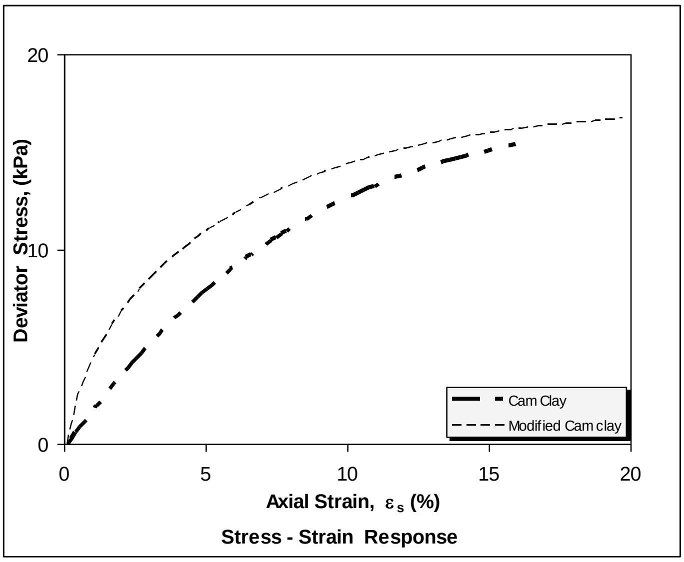

Fig.29 Comparison of Stress-Strain response

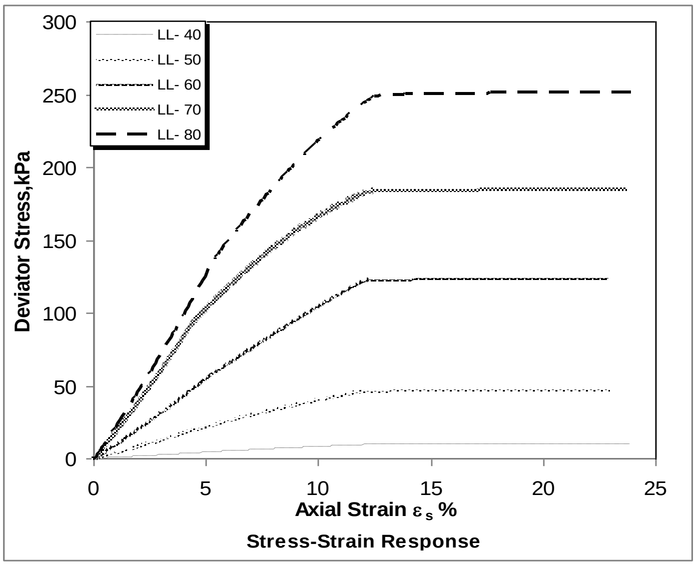

Fig.30 Wheeler predictions for constant initial volume

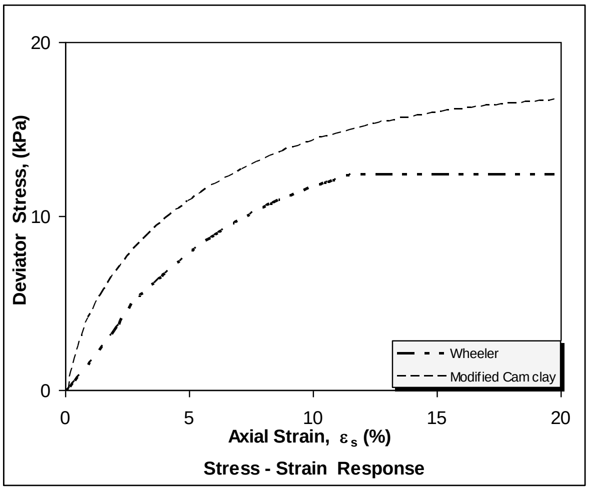

Fig.31 Comparison of Stress-Strain response

6.2.2 Drained Tests for the samples with same initial mean principal stress

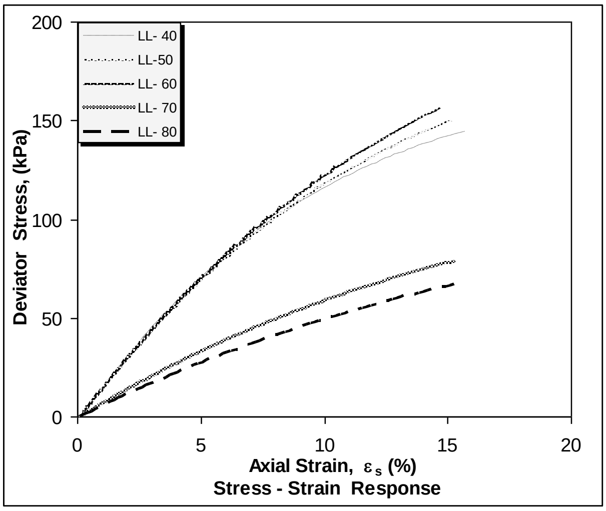

Fig.32 Cam clay predictions for constant Mean principle stress

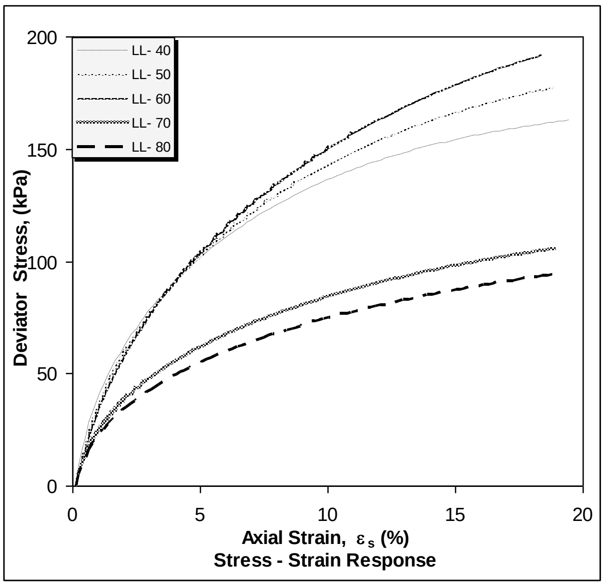

Fig.33 Modified Cam clay predictions for constant Mean principle stress

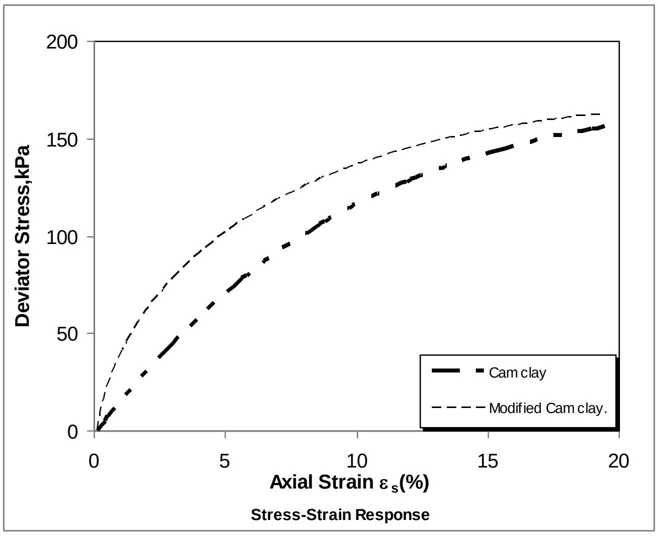

Fig.34 Comparison of Stress-Strain response

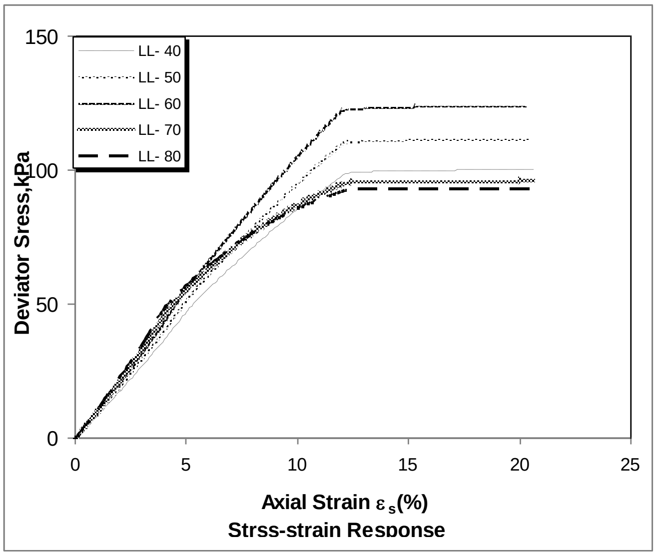

Fig.35 Wheeler predictions for constant Mean principle stress

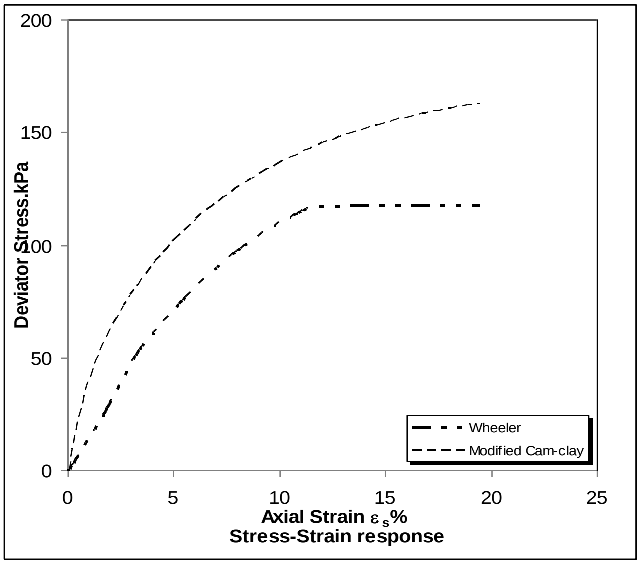

Fig.36 Comparison of Stress-Strain response

6.3 Constant 'p' test

6.3.1Constant' p' Tests for the samples with same initial volume

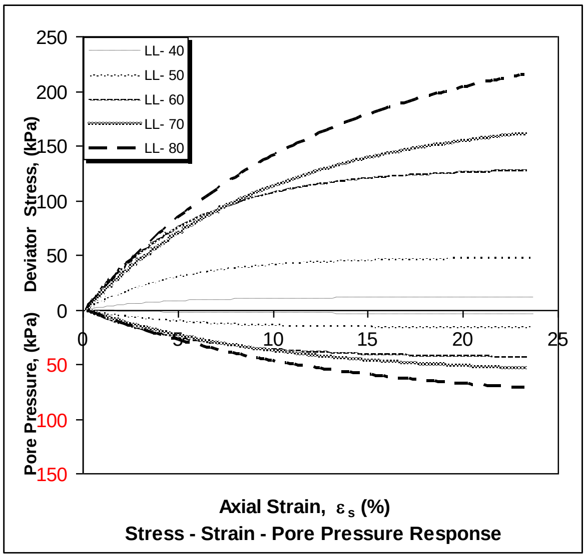

Fig.37 Cam clay predictions for constant Volume

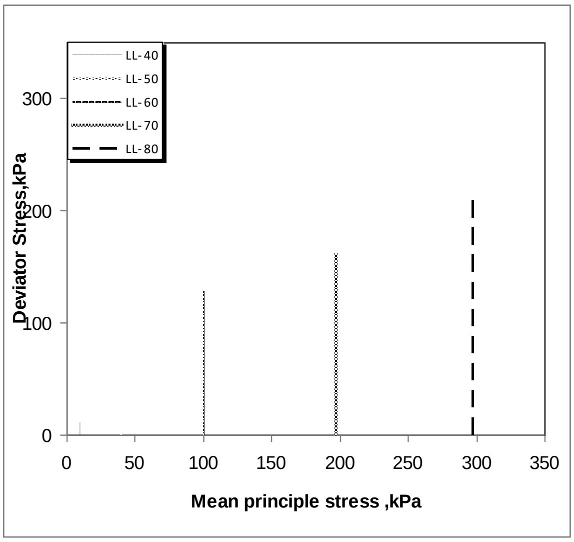

Fig.38 Stress Paths for different liquid limit values from Cam clay

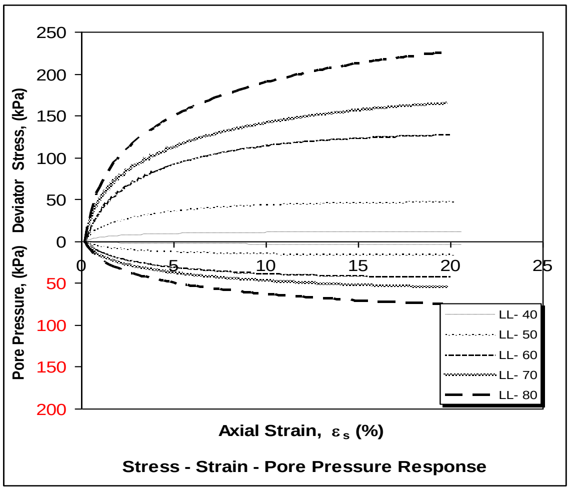

Fig.39 Modified Cam clay predictions for constant Volume

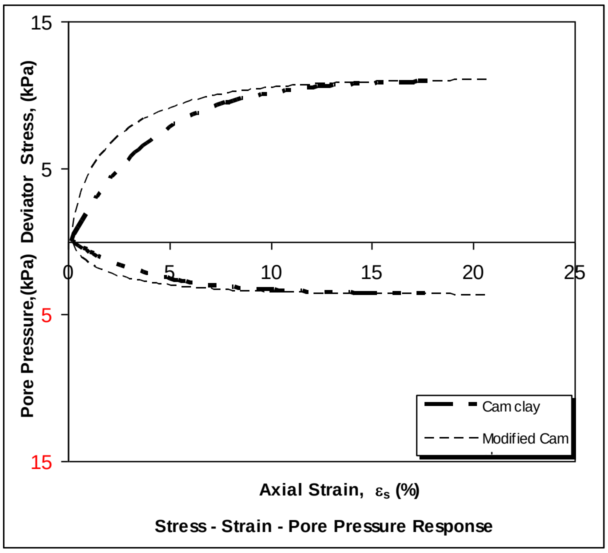

Fig.40 Comparison of Stress-Strain response

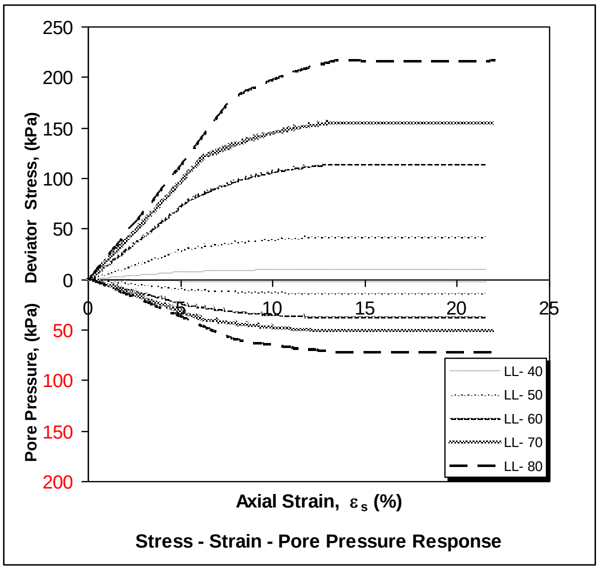

Fig.41 Wheeler predictions for constant Volume

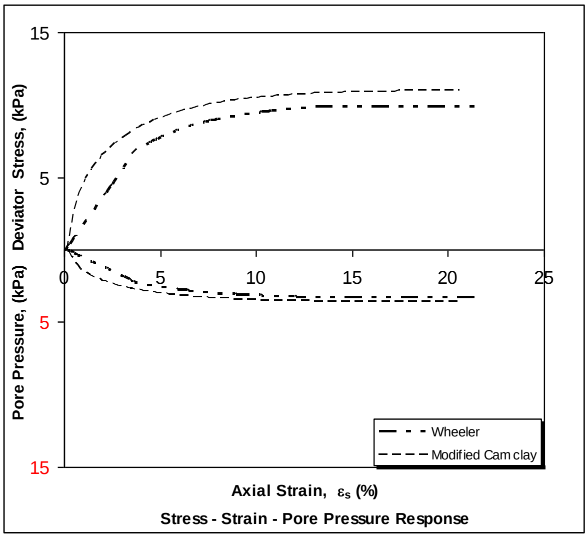

Fig.42 Comparison of Stress-Strain response

6.3.2 Constant' p' Tests for the samples with same initial Mean principle stress

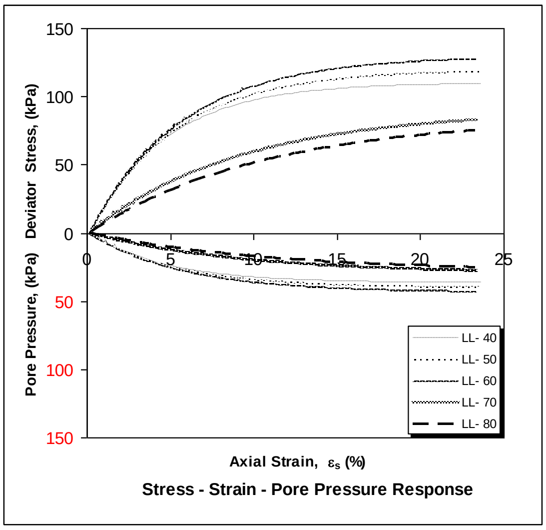

Fig.43 Cam clay predictions for constant Mean principle stress

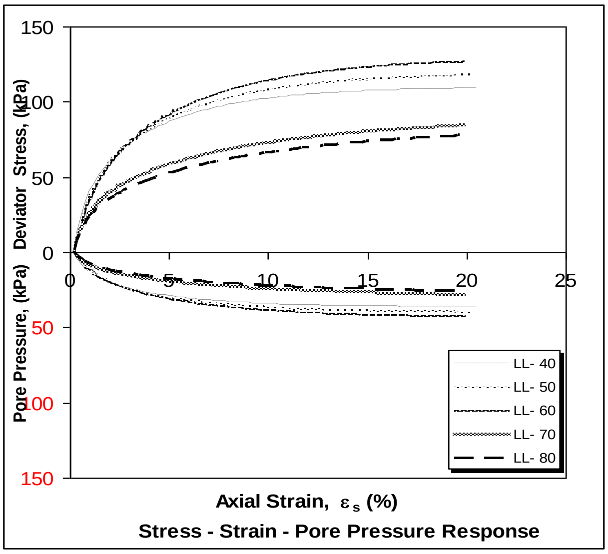

Fig.44 Modified Cam clay predictions for constant Mean principle stress

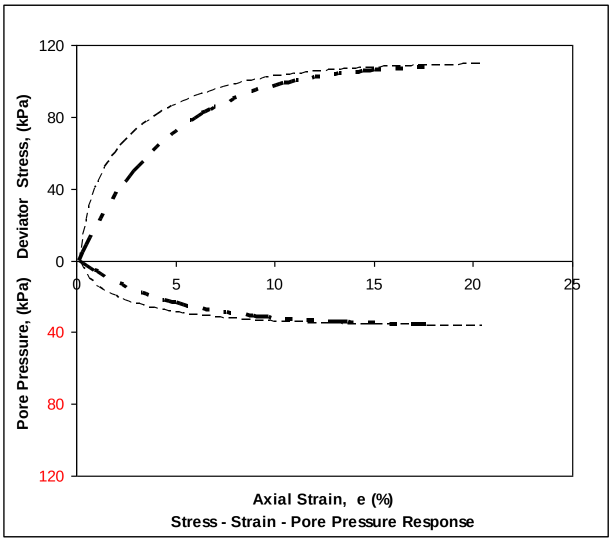

Fig.45 Comparison of Stress-Strain response

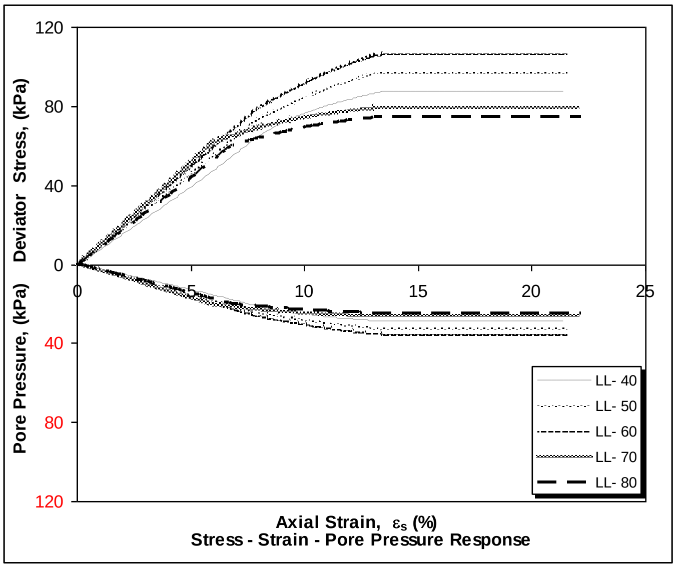

Fig.46 Wheeler predictions for constant Mean principle stress

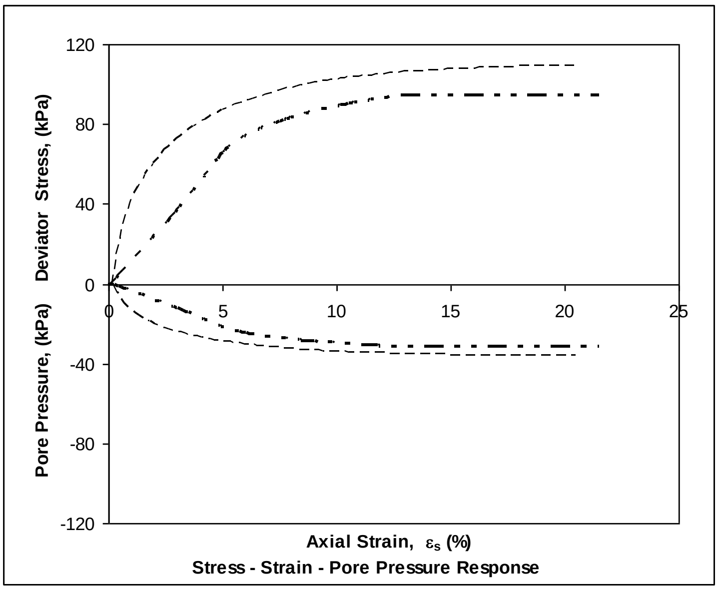

Fig.47 Comparison of Stress-Strain response

Concluding Remarks

1. It may be seen that as the liquid limit increases the mean principal stress required holding the sample at same volume increases.
2. The strength of the soil increases with liquid limit, as the soil sample of higher liquid limit is to be held at same void ratio of soil sample of lower liquid limit.
3. The pore water pressure for soil sample of higher liquid limit would be higher and is found to increase at higher rate with increase in liquid limit.
4. The model tests performed for modified Cam Clay for the same initial conditions as used for Cam clay yield stress-strain response where the strain experienced by the sample for reaching the same stress values is significantly lower compared to the Cam clay.
5. The Deviatoric stress at failure is relatively higher than the Deviatoric stress at failure for the Cam clay. Further the pore pressures developed are lower than the cam clay model
6. The predictions form Wheeler model are found to be at variance in comparison to the predictions obtained from the other models.

7. The strain softening may be noticed in the stress-strain response. The softening is noticed at lower strains for soils of higher liquid limit

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