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Prediction of Soil Behaviour Part III—Cemented Saturated Soils

by

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Introduction

Often environmental factors cause chemical bonding between soil particles which is an additional complexity in soils. These cementation bonds markedly influence the behaviour of such soils. In general, cemented soils exhibit very low compressibility and high strengths or a nearly rigid non particulate response within the yield stress, and markedly high compressibility but still high shear strength compared to that of uncemented soil, beyond this yield stress. It has not been possible many a time, to even explain the behaviour of sensitive soils, let alone predict it. This is so because it has not been possible to uniquely relate the state of cemented soils to the effective stress as could be done in the case of uncemented soils OR in other words the true effective stress responsible for deformations or shearing resistance are not known.

Unfortunately, for quite sometime, there has not been much study on the behaviour of remoulded states of soils devoid of stress history and cementation bonding because it was thought useless, as one rarely finds soils in that state in actual field and what is needed is the behaviour of the soil in its natural state. But it now appears that an ideal remoulded condition serves as a reference state in relation to which the behaviour of other complex states can be explained and even quantified. This paper is an attempt to examine the possibility of considering the stress carrying capacity of cemented soils as a superposition of two components:

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(*a*) resistance due to uncemented state.

(*b*) resistance due to cementation bonds.

This helps in identifying the true effective stress for such soils, which is required in predicting their behaviour.

Cementation bonds can develop in soils when they are in normally consolidated state or overconsolidated state. In the former case, the soil will become soft sensitive and in the latter, stiff cemented. Soft clays are more common because cementation usually occurs under marine environments and hence in recent marine deposits. And also, stiff cemented soils are not normally problematic as they generally have sufficiently high strength and low compressibility. Hence the discussions in this paper will mostly refer to soft soils but the same logic can be extended to over consolidated stiff clays.

Meta stable state of Sensitive Soils

The chemistry or mechanisms involved in the formation of cemented soils is not within the scope of this paper. This aspect has been studied by several researchers (Skempton and Northey 1953, Kenney *et al* 1967, Kenney 1968, Houston and Mitchell 1969 and others). In brief and in very general terms, cementation bonds between particles/or their aggregates are developed under saline environments acting over long periods of time as solid amorphous links of precipitates of calcite, iron oxides, alumina and other inorganic and organic matter. These bonds impart additional rigidity to the soil skeleton against compression. As long as these amorphous bonds are present, the initial void ratio prevails even under stresses greater than the initial overburden pressure. This results in higher equilibrium void ratios than those of the uncemented soil under corresponding overburden pressures *i.e* the void ratio-pressure relation of the soil gets altered due to the contribution of the bonds to the resistance. This state which is not a true reflection of the external stresses acting is usually termed as the 'Meta stable state'. Metastable in the sense, under any stress level, if the bonds are leached or dissipated, the soil has to collapse in an effort to reach the state of uncemented soil under the same stress.

Analysis of Compressibility Behaviour

Figure 21 schematically shows the typical compression path of a soft cemented soil. The compression is negligible upto a certain stress level beyond which there is a sudden compression of relatively high magnitude indicated by a steep slope. This stress level which marks the beginning of yielding is sometimes termed as the pseudo or quasi pre-consolidation pressure. After the initial steep compressions, upon further loading, the rate of change of void ratio with pressure decreases again and reaches

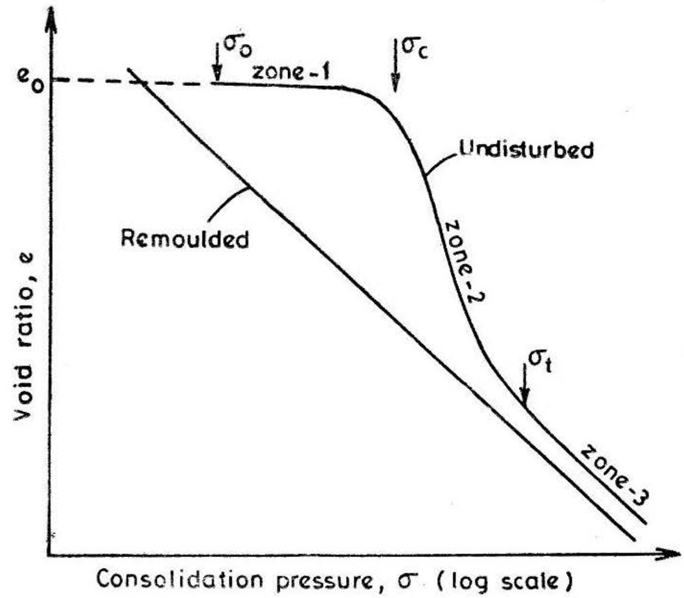


FIGURE 21 Typical e-log σ plot for a soft and sensitive clay

nearly a constant value beyond a stress level (say σ_t). This results in a characteristic inverse *S*—shaped curve with three distinct zones. Zone 1 upto the yield stress σ_c, zone 2 between σ_c and σ_t and zone 3 beyond σ_t. This behaviour was being explained as follows. In zone 1, the soil offers resistance to compression with nearly rigid non particulate response due to cementation bonds. Beyond the yield stress there is gradual breakdown of bonds resulting in high compressions. Upon reaching σ_t, the bonds are completely broken and the soil response is the same as the remoulded soil and hence the constant slope.

But a re-examination in relation to the compression curve of the remoulded state reveals a totally different picture. At any given void ratio, *e*, the equilibrium stress σ is in excess of the stress σ_R on the remoulded state by an amount $\sigma_b = \sigma - \sigma_R$, which may be considered to be the contribution from cementation bonds. *i.e.*, the applied load is resisted both by the bonds and the remoulded state. The applied pressure is resisted by bonds in zone 1, as long as it is less than their capacity. When the maximum capacity of the bonds is reached at σ_b, further increments of load has to be carried by the soil as in an uncemented condition and this can happen only with compatible reductions in void ratio. It appears from the very few available data from literature that the bonds resistance σ_b remains constant (or even slightly increases) but never decreases with increase in stress level. It is not known at this stage whether this implies that the bonds are not broken in isotropic compression or whether they do break but the number of bonds per unit volume still remains the same as it was intially. However the sensitivity of the soil (ratio of undisturbed strength to remoulded strength) will decrease with increase in stress level because of the drastic decrease in void ratio and hence increase in the remoulded strength, the cementation component remaining nearly constant.

$$i.e \quad S_t = \frac{q_{undisturbed}}{q_{remoulded}} = \frac{q_R + q_b}{q_R} \tag{39}$$

which decreases as q_R increases q_b being nearly constant.

A load *increment ratio of 1 at the level of applied stress may actually* be a much higher ratio of increment at the level of remoulded component and hence causes greater compressions which is generally noted as a higher C_c (or 2) for the actual soil. To clarify this further, consider the data of Louiseville clay (Lapierre *et al*, 1989) At a pressure of 200 KPa, the remoulded resistance is about 60 KPa and hence the cementation component is 200 — 35 = 165 KPa, the remoulded component is 200 which means the cementation component is about 200 KPa. This indicates that the bond resistance has remained nearly constant while the pressure increment on the remoulded state is much more than unity *i.e.*, from 35 to 200 KPa. This is responsible for the high compressions and hence the reported high C_c values. In other words, the deformation of the soil is entirely due to the remoulded component of stresses or the true effective stress re ponible for deformation is the remoulded component and hence can be written a

$$\sigma'' = \sigma_R = (R—A) = \sigma - \sigma_b - u \tag{40}$$

and the stress state relation would be the same as for uncemented soils

$$e = a - b \log \sigma_R \tag{41}$$

An excellent support for the state of cemented soil being the reflection of only the remoulded component of stress, is the pore size distribution data of Delage and Lefebrve 1984 on undisturbed and remoulded states of St. Marcel clay which shows that the microstructure is very much the same for both the conditions.

With these definitions it is apparent that it is possible to predict the response of the soil if the two components σ_R and σ_b can be predicted separately.

Prediction of Bond Strength, σ_b

In the case of soft sensitive soils with liquidity index greater than unity, the remoulded resistance σ_R is negligible at the insitu void ratio and hence the bond strength σ_b is same as the yield stress, σ_c.

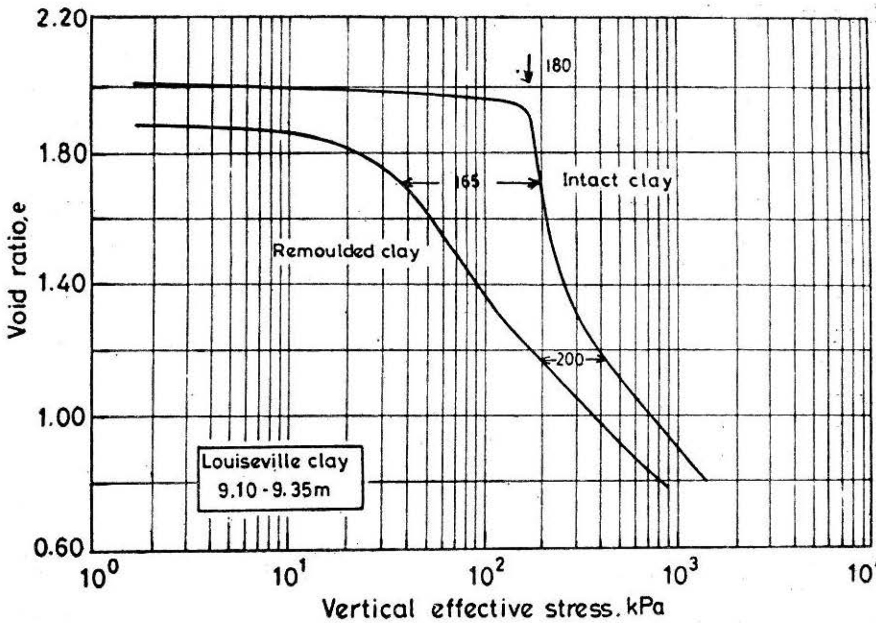


FIGURE 22 e-log σ paths of undisturbed and remoulded state of Louiseville clay

The usual way to obtain the yield stress is to run consolidation tests on undisturbed samples and to determine σ_c by Casagrande's or other methods as we do with stress dependent over consolidated soils. But the problem with sensitive soils is that they are highly susceptible for mechanical disturbance during sampling and handling and hence the laboratory results invariably deviate from the true field behaviour. Schmertmann (1953) has proposed an empirical method for modifying the laboratory compression curve to obtain the true compression behaviour in field. But Nagaraj *et.al* (1990) have shown that this approach is not tenable or applicable for all sensitive soils.

In the following section two simple methods for predicting the field yield stress are discussed. They are:

(*a*) by modifying the laboratory compression curve.

(*b*) by using the field vane strength data.

Effect of Sample Disturbance on e—log σ path

Several researchers (Milovic 1970, Lo 1972, Holtz *et.al* 1986) have analysed the effects of sample disturbance on e—log σ relations of sensitive soils. From these studies it is evident that with increase in degree of sample disturbance, the magnitude of experimentally determined yield stress decreases whereas the strain up to this stress level increases. Form the results of

Holtz *et.al* (1986) it can reasonably be assumed that the locus of σ_c for different degrees of disturbance (Fig 23) is a straight line perpendicular to each of the e—log σ curves at their points of maximum curvature. In the limiting condition this line may be perpendicular to the remoulded curve (which correspond to a state of 100 percent or complete disturbance).

Secondly, there are indications by a few insitu load settlement results (Pelletier *et.al* 1979, Folks and Crooks 1985) that the settlement is negligible upto the level of yield stress. With these two findings, the most probable value of field yield stress, σ_{ct} can now be predicted from the laboratory curve as the point of intersection of a horizontal line from σ_o (assuming zero compression upto σ_{ct}) and a line normal to the laboratory e vs log σ_{ct} curve at the point of maximum curvature. Figure 24 shows the predicted values of σ_{ct} by the above approach which compare well with those obtained by fitting the field behaviour as reported by Pelletier *et.al*.

Yield Stress using Field Vane Strength

Alternately, in the absence of the laboratory compression curve, the field yield stress can be determined using the field vane strength data, which may be an useful method since it is a routine practice to determine the vane strength of the soil during site investigations and hence this data will be always available. The magnitude of yield stress σ_c depends on the degree of cementation (or the strength of bonds). This is equally true of the undisturbed undrained strength of the soil. Both these properties are

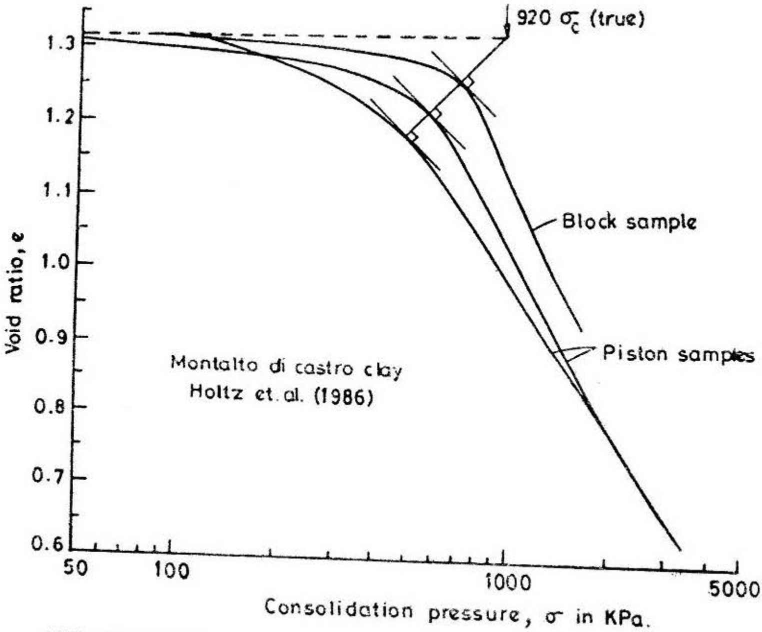


FIGURE 23 Effect of sample disturbance on e-log σ curves

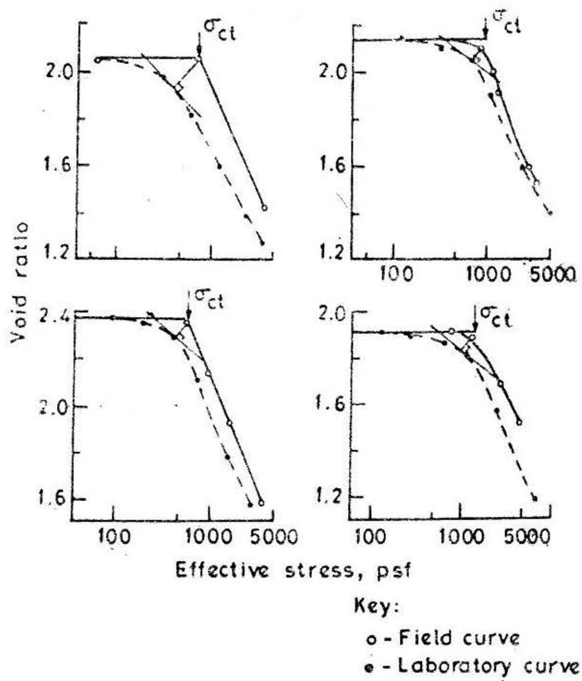


FIGURE 24 Comparison of predicted σ_{ct} with field values (pelletier et al 1979, Fig. 10)

affected by the same factors and hence it appears logical to relate these two parameters. Detailed analysis (Nagaraj *et al* 1990) of published data of some canadian sensitive soils has resulted in a functional relationship between σ_{ct} and s_u of the form

$$\sigma_{ct} = 3.78\ S_u + 7 \tag{42}$$

with a correlation coefficient of 0.994 and a standard error of estimate of 8 KPa. In the above analysis σ_{ct} has been obtained by modifying the experimental σ_c as discussed earlier. Predicted values of field yield stress from the above equation for several sensitive soils have been shown (Nagaraj *et al* 1990) to compare well with those predicted by the earlier method using the laboratory compression curve.

In the case of an overconsolidated cemented soil with natural water content much less than liquid limit, the remoulded component of resistance can be considerable and hence the observed yield stress σ_c, will not directly give the bond stress σ_b. This σ_b will have to be obtained by subtracting the remoulded component σ_R at that void ratio from the observed yield stress value. This case will be discussed a little later.

Prediction of Field Compression Curve

Once the cementation component of resistance, σ_b is known, the field compression curve can be predicted by simply superposing this over the remoulded component at that state. The method to predict the remoulded component has already been dealt in the companion paper 2. This requires only the liquid limit, if the soil is soft sensitive, and in addition the stress dependent pre-consolidation pressure, if the soil is overconsolidated and cemented. The needed equations for computation would be

$$\sigma = \sigma_R + \sigma_b \tag{43}$$

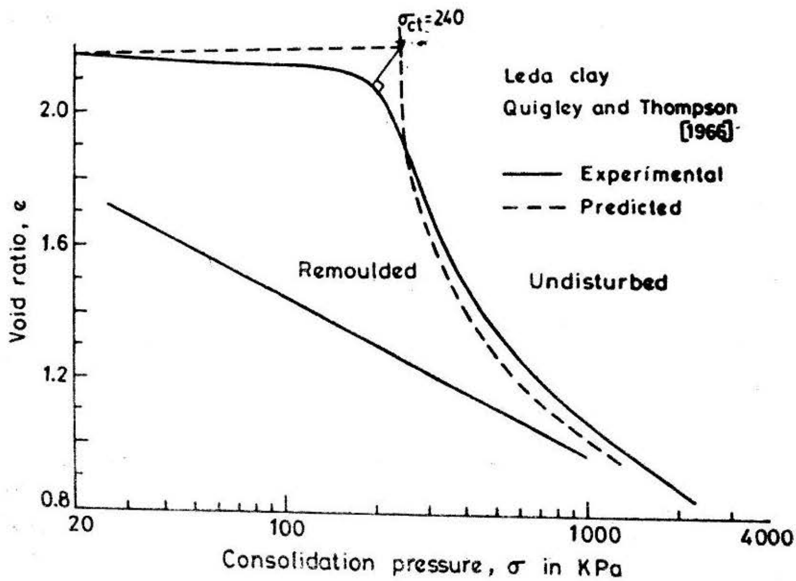
$$\frac{e}{e_L} = 1.122 - 0.234 \log \sigma_R \text{ (for soft sensitive soils)} \tag{44}$$

or

$$\frac{e}{e_L} = 1.122 - 0.188 \log p_c - 0.0463 \log \sigma_R \tag{45}$$

(for stiff and cemented)

Figures 25 and 26 show the predicted field compression paths for two soils for which the remoulded path was available by simply adding the cementation component which is same as σ_c at each level.



FIGURES 25 Predicted e-logσ field curve from remoulded path using σ_{ct} = 240 kPa

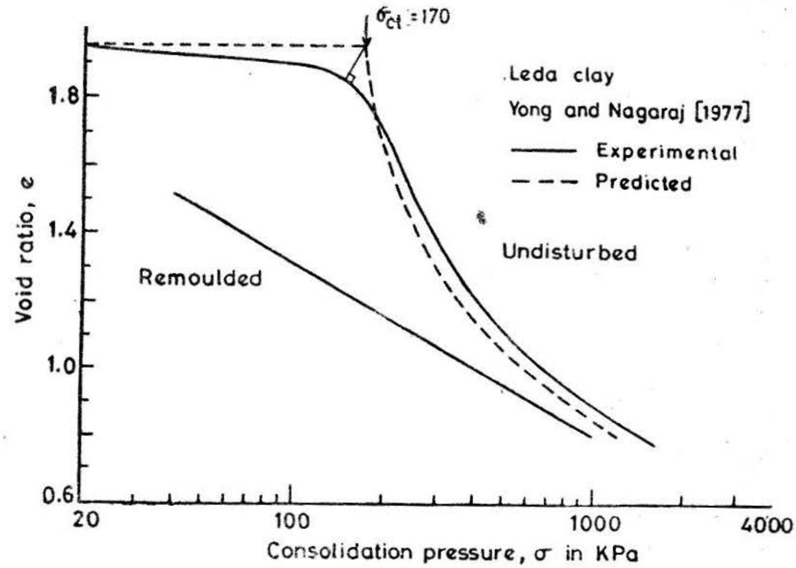


FIGURE 26 Predicted e.—log σ field curve from remoulded path using σ_{ct} = 170 kPa

Method for Prediction of Behaviour under Shear

At the outset, the shearing behaviour of sensitive soils appears to be similar to that of overconsolidated soils. They also exhibit a steep rise in strength or a high tangent modulus at low strains and a strain softening beyond the peak strength *i.e.* in comparison to a remoulded normally consolidated soil under similar condition, sensitive soil shows a higher strength at early strains but gradually approaches the remoulded strength at very large strains. But a closer study reveals that the two are totally different. In sensitive soils invariably strain softening is observed in undrained shearing but in drained shearing softening may occur only at very low confining stress. Whereas overconsolidated soils never exhibit a pronounced peak in undrained test but only in drained shear and that too for high overconsolidation ratios, say greater than 5 to 6. Strain softening in undrained shear on sensitive soils occurs under all confining pressures even far greater than the yield stress itself. This clearly shows that the behaviour is not similar to that of normally consolidated soil even beyond the level of yield stress. This is so since there is an additional component of resistance from bonds. Further, softening in overconsolidated soils is associated with volumetric dilation or negative pore pressure, whereas in sensitive soils continued volumetric compression (much greater than that for a remoulded state) or positive pore pressure has been observed. This behaviour is totally against Drucker's stability criteria and hence the plasticity models cannot be applied directly to these soils.

All the above behaviour can be explained by considering, as evinced from the consolidation behaviour, the yielding or deformation to be entirely due to remoulded component of stresses, and actual shearing resistance of the soil at any stage to be the sum of remoulded resistance and cementation bond resistance, *i.e.* at any strain level

$$q = q_R + q_b \tag{46}$$

This mode of superposition was suggested earlier by Conlon (1966) and then by Feda (1982). The remoulded component, q_R increases hyperbolically with strain as for an uncemented normally consolidated soil. The mobilization of this component can be obtained using the Cam-clay or any other model as usual. The cementation component q_b mobilizes to its peak value at very low strains (usually less than 1 to 1.5%). With continued shearing, the bonds will be gradually broken resulting in a reduction of this component (A break down of bonds during shearing may mean a reduction in the number of bonds per unit volume). Thus the total strength q, which is the sum of these two components, may or may not show a peak and strain softening, depending on whether or not the rate of increase of the remoulded component is less than the rate of decrease of cementation component at any strain level. And finally, at very large strains, when the bonds are completely broken, q_b might reduce to zero and hence the strength of the soil might reduce to the same as that of the remoulded soil.

In general, for highly sensitive soils, with natural water content close to or even higher than liquid limit, q_R will be negligible and hence the peak strength will be practically equal to the bond strength. Further, since the void ratio is nearly of the same under all confining pressures less than the yield stress, the undrained shear strength of the soil will be the same under any confining stress in this range. At confining pressures greater than yield stress, the value of peak strength will be in excess of the bond strength by an amount equal to the remoulded component mobilized at the strain level (Fig. 26*a,c*). Hence in undrained tests, the peak strength increases with increase in stress level and simultaneously, the strain corresponding to the peak strength may also increase. In drained tests, the remoulded component, though small at early strains, progressively increases with continued compression, to make up the loss in strength due to breakdown of bonds until at very large strains, q, p and e will be compatible to each other as for an uncemented soil, q_b being reduced to zero. In any case the equation,

$$\frac{e_f}{e_L} = a - b \log q_f \tag{47}$$

will still hold good for the ultimate strength.

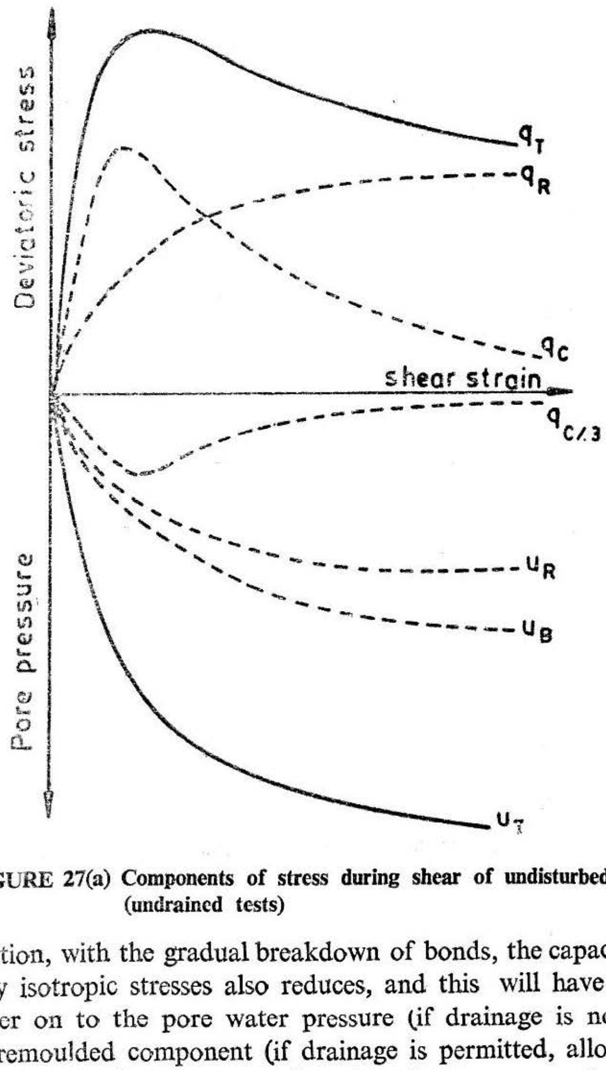


FIGURE 27(a) Components of stress during shear of undisturbed soil (undrained tests)

In addition, with the gradual breakdown of bonds, the capacity of bonds, p_b to carry isotropic stresses also reduces, and this will have to be transferred either on to the pore water pressure (if drainage is not permitted) or to the remoulded component (if drainage is permitted, allowing further compression). This is the reason for observing greater volumetric compression or positive pore pressures than for a remoulded soil under the same conditions. Thus the actual description of the complete stress strain behaviour calls for an incremental modelling taking into the effects of gradual breakdown of bonds and then superposing the stresses carried by the remaining or intact bonds. The adaption of the Cam-clay model to sensitive soil cannot be discussed in greater detail in this paper, that being a major aspect by itself. This aspect has been discussed elsewhere (Vatsala 1988). It is only emphasized here that it is possible to describe the complete behaviour, using Cam-clay model knowing the remoulded soil parameters λ, k, Γ and M as usual and in addition the magnitude and rate of loss of cementation bond strength q_b with shear strain. Determination of q_b requires a typical undrained shear test on the undisturbed sample.

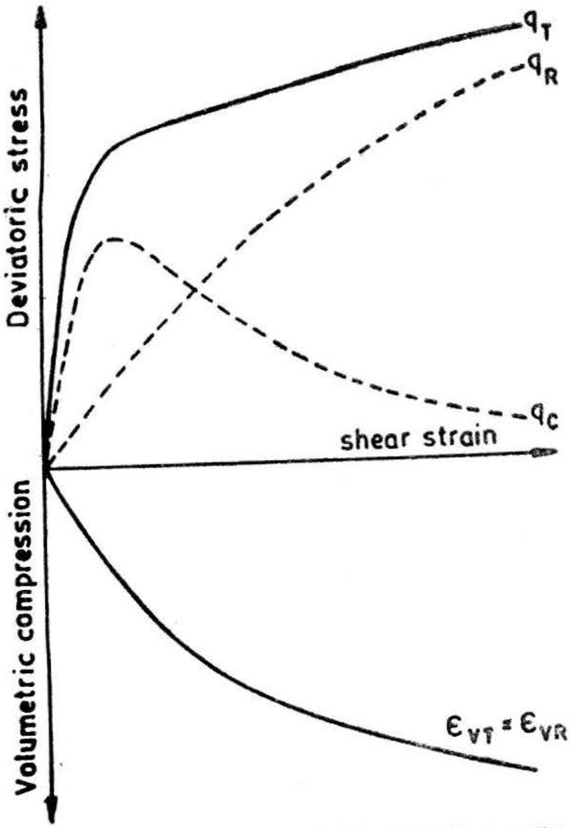


FIGURE 27(b) Components of stress during shear of an undisturbed soil (Drained tests)

Estimation of Sample Disturbance

It is a known fact that it is impossible to obtain a truly undisturbed sample because stress release during sampling itself is a disturbance. Hence a perfect sample is often defined as one which has not been subjected to any disturbance other than stress release. Mechanical disturbance, which may in effect be equal to partial remoulding, may be caused at various stages during advancement of bore hole, insertion of sampling tools, handling and testing of specimen. It is necessary to estimate the degree of such disturbance in order to assess the reliability of laboratory test results and if possible, to account for the effects of these disturbances on the measured values. As has been seen earlier, the response of the soil within the yield stress in zone 1 is essentially due to the cementation component (remoulded component being negligible here) and hence are directly affected by sample disturbance. As such, properties like undrained shear strength, tangent modulus etc for tests with confining stresses within the yield stress can be corrected if the degree of disturbance, is known whereas beyond yield stress, the mechanical properties are not affected one to one by sample disturbance because they include remoulded component also which are not affected by disturbance. Hence it may not be possible to apply reasonable corrections to the measured values under these stress levels.

In this section a simple method (Nagaraj *et. al.* 1990) to estimate the degree of disturbance and to modify the experimental test results to account for such disturbances has been discussed.

The magnitude of yield stress σ_b is directly affected by sampling disturbance and hence extent of disturbance can be defined as

$$SD\ \% \quad \frac{\sigma_{ct} - \sigma_c}{\sigma_{ct}}\ X\ 100 \tag{48}$$

Which can be evaluated by knowing the laboratory determined σ_c and the true undisturbed value σ_{ct} in field. Using similar equations, the laboratory measured values of undrained strength and tangent modulus can be corrected to obtain the possible insitu values as

$$q_{ut} = \frac{q_u}{1-(SD/100)} \quad E_{ut} = \frac{E_u}{1-(SD/100)} \tag{49}$$

To check the validity of the above proposition has been verified (Nagaraj *et. al* 1990) using Milovic's data on two soils, for samples obtained from three different techniques.

Stiff Cemented Clays

The determination of bond stress and the separation of two stress components is not very easy with stiff cemented soils. This is because, the insitu void ratio being considerably low, the remoulded component of stress σ_R or shearing resistance q_R is not negligible as it was with soft sensitive soils. To know the magnitude of σR for the overconsolidated state the stress dependent pre consolidation pressure σ_c is to be known. But the magnitude of σ_c depends on the actual sequence of loading, unloading and cementation bonds development. For example the current state of the soil may be at *E*, the soil might have acquired this state in several ways (Fig. 28):

(1) It might have been subjected to a loading from *A* to *C*, then unloaded to *D*, at which stage cementation bonds might have developed. Further loading from *D* to *E* might have occurred without appreciable compression since the bonds resist compression. In this case, stress dependent σc of the soil would be the pressure at *C*.

OR

(2) The soil might have developed bonding after a loading from *A* to *B*, and then a stress release at this stage take place without marked

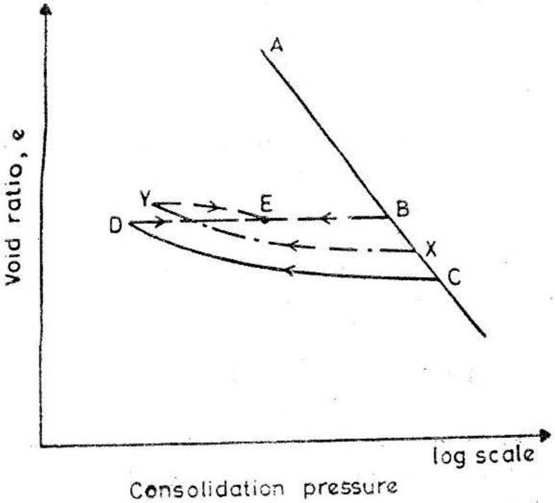


FIGURE 28 Possible sequence of loading and cementation in the formation of stiff clays

change in void ratio, since bonds resist dilation also, and σc in this case may be the pressure at B itself.

(3) Alternately, the soil might have undergone a loading sequence from A to X, a stress release X—Y, reloading from Y to E and then bonds might have formed, in which case σ_c would correspond to the stress at X.

The compression curve for each of the above cases would follow different paths (as per eqn 13 σ_c using different for each case). It is obvious that the behaviour cannot be predicted with just index properties and insitu condition unless we know the sequence of loading. We may require to run an actual compression test to arrive at the unknown quantities σ_c and σ_b. Of course the complete compression curve with load increment ratio of unity at the end of 24 hours may not be needed but the data of equilibriym states at two sufficiently high stress levels X and Y (Fig 29) will be required; that is, stress level Y high enough to ensure that the uncemented component $(\sigma_R)^2y$ has crossed the preconsolidation range or has reached the normally consolidated range, and stress level X, at which the uncemented component is still in the overconsolidated range, but is high enough to ensure that the bond strength is utilized to its maximum capacity σ_b.

From the data of void ratio and pressure at Y, σ_b can be computed since the uncemented component $(\sigma_R)y$ corresponding to this void ratio is known from equation 12 *i.e.* $\sigma_b = (\sigma)y - (\sigma_R)y$. Once σ_b is known, the uncemented component $(\sigma_R)X$ corresponding to the void ratio at X can be determined as $(\sigma_R)X = \sigma X - \sigma_b$. And then the stress dependent pre-consoli-

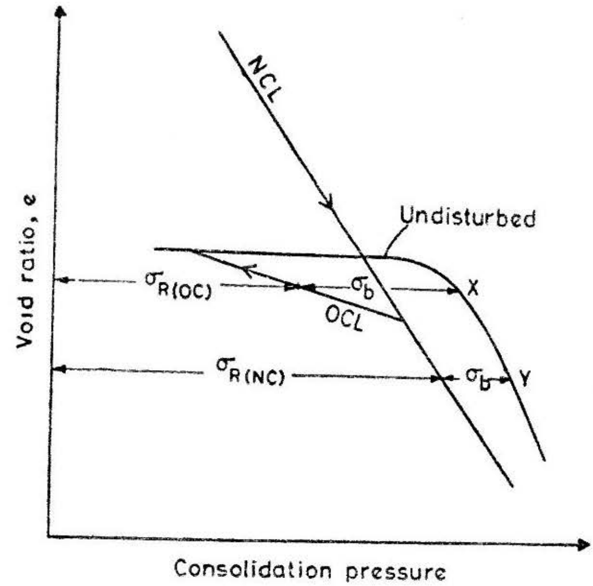


FIGURE 29 Schematic representation of suggested procedure to predict e-log σ path and stress dependent σ_c from estimated cementation bond strength σ_b

dation pressure, σc for the uncemented state can be easily computed using equation 13.

With σ_c and σ_b known, we have all the data required, and the compression or the shearing behaviour of the soil can be predicted by superposing the effects of remoulded (which is an overconsolidated state in this case) and cementation bond component as for a sensitive soil.

The above propositions for stiff clays are yet to be verified experimentally which requires well planned carefully carried out test data.

Effect of Decementation

Another important aspect with cemented soils may be the effect of decementation which may be caused by leaching or by any other process. There can be two different cases of decementation.

(*a*) the process simultaneously may cause a change in the physico-chemical environment of the soil.

(*b*) the process may not alter the environment of the soil.

It is possible to predict the changes in the soil state due to decementation of the later type. In the former case, the potential of the soil itself may get altered and the prediction of such changes requires a complete understanding of the involved chemical processes and their effects. The soil loses part of its stress carrying capacity, *i.e.* the contribution of bonds e_b or q_b, and if the soil is soft sensitive it will collapse, until it reaches a state of equilibrium for the remoulded soil under this stress condition. This is equivalent to applying a load increment of e_b q_b (which was being carried by the bonds so far) on the current state of remoulded soil and the response can be estimated knowing the basic properties of the remoulded soil.

But in a stiff cemented clay of the type where a stress release has taken place after the development of cementation bonds, the effect can be quite opposite, *i.e.*, the soil may expand upon decementation. In such cases, the presence of bonds would not have allowed the soil to expand compatible with the amount of stress release on the remoulded components because the bonds can resist (most likely) tensile stresses. For clarity consider the case of a cemented state at which $e = 100$ units, $e_R = 60$ units and $e_b = 40$ units. If now the soil is unloaded to 10 units level (assuming a rigid response for the cementation component) we must have an expansion corresponding to the stress release on the remoulded component *i.e.* from 60 to 10 units. But if the bonds can resist tensile loads, the soil would be in equilibrium at $\sigma = 10$ units, with $\sigma_R = 50$ units and $\sigma_b = —40$ units. And hence the soil would expand coriesponding to a release of only 10 units (from 60 to 50) of pressure. This is the reason for invariably observing very flat rebound paths in cemented soils. If now the soil at 10 units pressure level undergoes a process of decementation, it would lose its bond strength (i.e.—40 units in this case) and this will be applied on to remoulded state (e_R, $\sigma_R = 50$). Hence the oil would expand corresponding to a stress release of 40 units $= \sigma_b$ (from 50 to 10) on the remoulded soil which can be esaily predicted. Once the equilibrium has been established after the removal of bonds, the soil behaviour is the same as that of the remoulded soil and can be predicted as already discussed. Experimental data by Griffiths *et al* (1988) where the cementation bonds in a stiff clay have been removed by a process of EDTA flushing very well reflect these behaviour in general, although further experimental verification is required to establish quantitative predictions of the amount of swell.

Concluding Remarks

To predict behaviour of cemented soils it is essential to assess true effective stresses. From the consolidation test results on remoulded and undisturbed sensitive soils it has been shown that the total resistance can be considered to be a sum of two components, viz. the cementation component and the remoulded component. Yielding or deformation has been shown to be entirely associated with the remoulded component and hence this component can be identified as the true effective stress.

With this background the prediction of behaviour of sensitive soils reduces to the prediction of the two components independantly and superposing then. The remoulded component can be predicted using the method described in the companion paper 2. The cementation component can be estimated from an appropriate typical test.

The above approach has been logically extended to stiff cemented clays and possibly in developing the prediction methods has been indicated.

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