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YIELDING OF WEATHERED BANGKOK CLAY

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ABSTRACT

A comprehensive series of triaxial compression tests was carried out on undisturbed samples of Weathered Bangkok Clay. These tests included (i) anisotropic consolidation under different stress ratios (ii) undrained tests and (iii) drained tests.

A yield locus for volumetric strain is established for the Weathered Clay which separates the states for which the volumetric strain is small from those for which the volumetric strain is large. The strain paths followed during anisotropic consolidation are made up of two straight lines with different slopes. One corresponds to the case inside the yield locus and the other corresponds to the case outside. $(dv/d\varepsilon)_{\eta}$ vs η relationships are found to be different for both cases.

From a series of undrained tests carried out on samples consolidated to pre-shear consolidation pressures less than the maximum past pressure, the effective stress paths in (q, p) plot are found to be approximately parallel to the q-axis and the constant shear strain contours are found to be nearly parallel to the p-axis. The results of undrained and fully drained tests on Weathered Clay consolidated to the normally consolidated states are presented and discussed.

Key words : clay, deformation, drained shear, failure, strain, stress-strain curve, triaxial compression test
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INTRODUCTION

The two most important problems concerning soil engineering are the settlement and the stability of a structure. In the current methods of "stability" analysis, the soil is assumed to be a non-dilatant material. It is considered to be rigid up to failure and thereafter it behaves as if it has got a constant angle of internal friction and a constant cohesion. In these analysis the deformation of the soil prior to failure is ignored, and after failure the deformation is considered to be very large.

In contrast to the stability problems, the problem of settlement calculation has always been confined to stress levels which are much below the values of the stress at failure. Also, in most cases the settlement of a structure is evaluated on the assumption that the deformation is one dimensional. However, it is well recognised that the conditions of stress in a soil layer due to external loadings hardly correspond to those experienced by a laboratory sample under one dimensional consolidation.

The aim of the work presented in this paper is to study the deformation characteristics of Weathered Nong Ngoo Hao Clay under a variety of stress paths. In most cases the

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specimens of clay will be taken up to failure. It is generally accepted that the Weathered Nong Ngoo Hao Clay exhibits a certain degree of apparent overconsolidation. Hence, the behaviour of this clay at low stress levels will correspond to that of overconsolidated clay. However, large stresses imposed in directions other than those corresponding to failure would cause this clay to change from the overconsolidated state to the normally consolidated state. The change in voids ratio and the corresponding volumetric strain experienced by a clay in the overconsolidated state is considerably smaller than the corresponding changes in the normally consolidated state.

Mitchell (1970) used the word "Yield Point" to define the stress point at which the behaviour of the clay changes from the overconsolidated behaviour to the normally consolidated behaviour. For stress conditions corresponding to those of one dimensional consolidation, the vertical stress at the yield point in an oedometer test will correspond to the maximum past pressure. By carrying out triaxial tests at different values of stress ratio, Mitchell (1970) was able to establish a yield curve which would separate the states of stress for which the volumetric deformation will be small when compared with those states of stress for which the authors propose to establish a yield curve for Weathered Nong Ngoo Hao Clay.

BASIC CONCEPTS AND DEFINITIONS

The stress parameters p and q are defined by

$$p=(\sigma_1'+2\sigma_3')/3 \text{ and } q=\sigma_1'-\sigma_3'$$

where σ_1', σ_2' and σ_3' are the principal effective compressive stresses, and $\sigma_2'=\sigma_3'$ under the triaxial stress system. Similarly, the incremental strain parameters dv and $d\varepsilon$ are given by

$$dv=d\varepsilon_1+2d\varepsilon_3 \text{ and } d\varepsilon=2(d\varepsilon_1-d\varepsilon_3)/3$$

where $d\varepsilon_1$, $d\varepsilon_2$ and $d\varepsilon_3$ are the principal incremental compressive strains and $d\varepsilon_2$ is equal to $d\varepsilon_3$ under the triaxial stress system. The stress ratio, η, is equal to q/p.

PROPERTIES OF WEATHERED BANGKOK CLAY

The unconsolidated deposits over the extensive Chao Phraya Plain have probably resulted from marine sedimentations, created in a large depression of the earth's crust. According to Muktabhant (1967), these structural changes took place during the late Tertiary Period leaving the shore line possibly as far north as Uttradit, 250 km away from its present position. Before this upheaval, the ancient rivers of the plain deposited the alluvial layers of sand and gravel which are found beneath the Stiff Clay. After the tertiary movements had occurred, sedimentation of the Stiff Clay took place during the late Recent Period, in deep water, when the sea was perhaps 50 m higher than present. Eide (1968) suggested that the overlying Soft Clay was deposited when the sea level rose a second time after a period of recession, the water level finally falling to leave the surface of the Soft Clay exposed. Weathering, consisting of a process of alternate wetting and drying of the clay surface and other physical and chemical process, resulted in the hard, shallow crust which covers the whole area of the Chao Phraya Plain (see Moh, Nelson and Brand, 1969).

The subsoil at Nong Ngoo Hao is essentially the same as that already presented for Bangkok Clay. The Weathered zone is about 4 m thick. Observation during sampling process indicated a marked effect of weathering process in this zone. In the upper part, the soil was found to be dark grey in colour. Light brown horizontal silt seams were observed at different depths in the upper 2 m layer. Small holes were also found in this portion, probably created by earthworms or decayed roots. Between 2 m and 2.5 m, a layer of light grey colour was noted having traces of dark coloured decayed organic matter. Below a depth of 2.5 m, the soil became more uniform in colour; the holes were no longer apparent. The samples between depths of 2.5 m and 3.0 m contained traces of silt seams and lenses of fine sand.

Laboratory tests were carried out on samples of Weathered Nong Ngoo Hao Clay to determine natural water content, Atterberg Limits, specific gravity and particle size distribution. These results are given below.

Table 1.

Natural water content %	133±5	Soluble salt content g/l	7.0
Natural voids ratio	3.86±0.15	Organic matter %	4
Degree of saturation %	95±2	pH	8.5
Specific gravity	2.73	Grain size distribution	
Liquid limit %	123±2	sand %	7.5
Plastic limit %	41±2	silt %	23.5
Plasticity Index %	82±4	clay %	69
Dry density (lb/ft^3)	36±2	colour	Dark Grey

SAMPLE PREPARATION AND TESTING PROCEDURE

After extruding from the sampling tubes, 1.4 inch diameter by 2.8 inch high specimens were trimmed using a standard sample trimmer and a wire saw. Because the Weathered Clay was generally fissured, it was necessary to trim the samples from both ends towards the middle of the sample.

The saturation and initial consolidation of the sample were carried out simultaneously. The sample was allowed to consolidate for a period of 1 to 5 days to ensure that 95% of consolidation has taken place.

The load increment and the increment in cell pressure were calculated for any anisotropic consolidation test, prior to commencing the actual test. The stress increments were selected in such a way that sufficient points could be obtained on the (e, log p) characteristic of each test, on either side of the p value demarking the range of overconsolidated states from the normally consolidated states. For every stress increment, in these tests, the increment in axial stress and the increment in cell pressure were applied simultaneously. All specimens were sheared under stress controlled condition.

ANISOTROPIC CONSOLIDATION TEST RESULTS

In this series, one isotropic consolidation test and four anisotropic consolidation tests were carried out. These tests corresponded to stress ratios η of 0, 0.16, 0.43, 0.6 and 0.75. The Weathered Nong Ngoo Hao Clay was isotropically consolidated to 2 lb/in^2. Thereafter the specimens were sheared under the fully drained condition until the relevant stress ratio was reached. Fig. 1 illustrates the stress paths followed during anisotropic consolidation.

Figs. 2(a) and (b) illustrate the variations of the voids ratio with respect to mean normal stress during anisotropic consolidation test. Using the (e, log p) characteristics, the maximum p experienced by the specimen along each of the anisotropic consolidation lines were estimated in a manner similar to that adopted by Casagrande for the estimation of maximum past pressure under one dimensional consolidation. Then, using

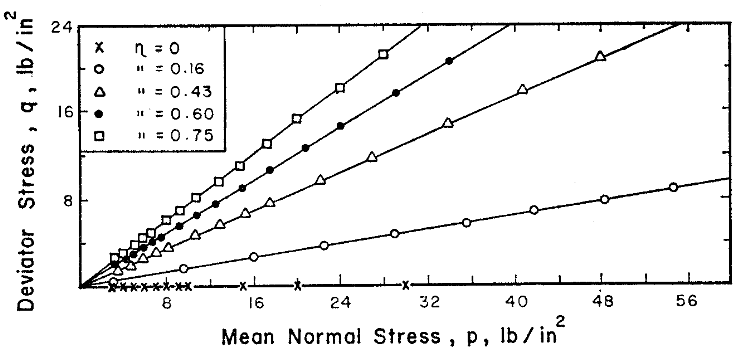

Fig. 1. Stress paths followed during anisotropic consolidation

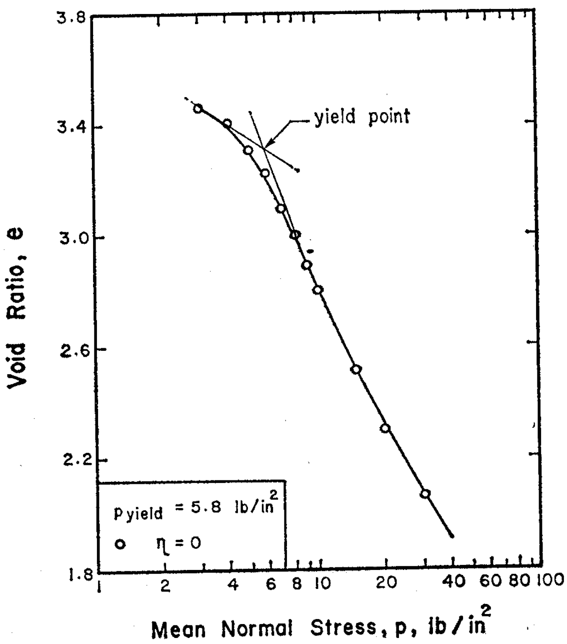

Fig. 2 (a). (*e*, log *p*) characteristic during isotropic consolidation

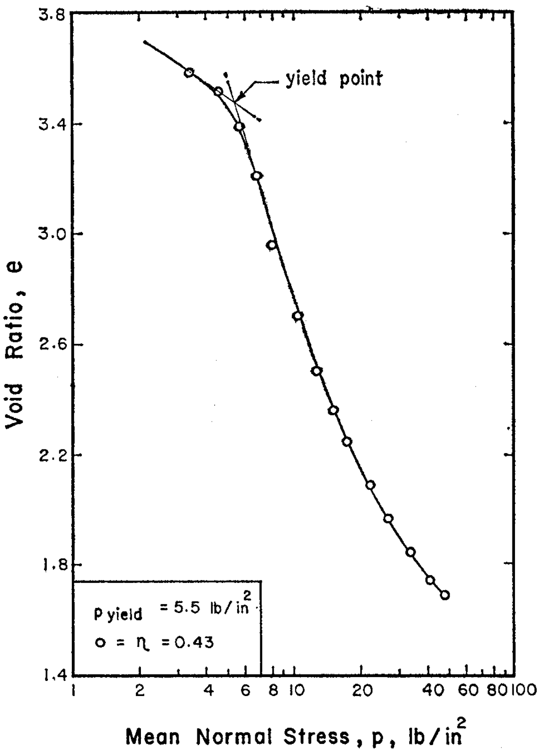

Fig. 2 (b). Anisotropic consolidation

the constant stress ratio lines in Fig. 1, the values of q corresponding to these p values could be obtained. The values of q and p thus obtained on each of the anisotropic consolidation line corresponding to the yield points are plotted in Fig. 3. The dashed line shown in Fig. 3 will be discussed subsequently.

It is evident that the locus of these yield points lie on a curve which is very similar to the undrained stress path on normally consolidated specimens of Weathered Clay. For all conditions of stress inside this yield locus, the volumetric strain experienced by the specimen would be small. However, for conditions of stresses outside this yield locus the volumetric strain would be large. Fig. 4 illustrates the (e, log p) characteristics for all the specimens tested under anisotropic consolidation.

The strain paths followed by the specimens during anisotropic consolidation are shown in Figs. 5(a) and (b). It appears that for any particular stress ratio, the strain path consists of nearly two straight lines.

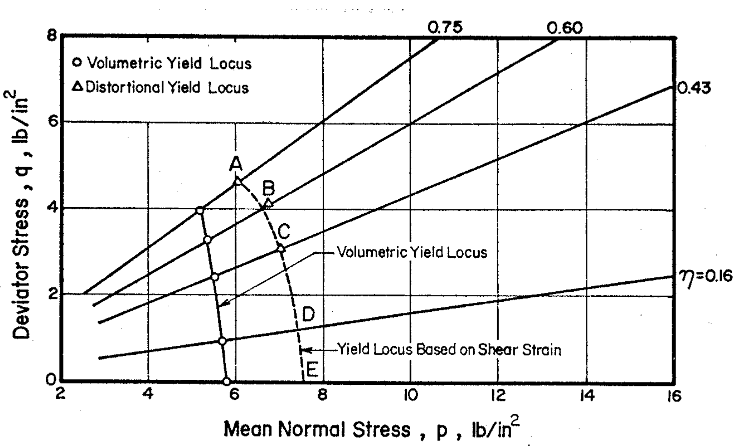

Fig. 3. Yield loci obtained from anisotropic consolidation tests

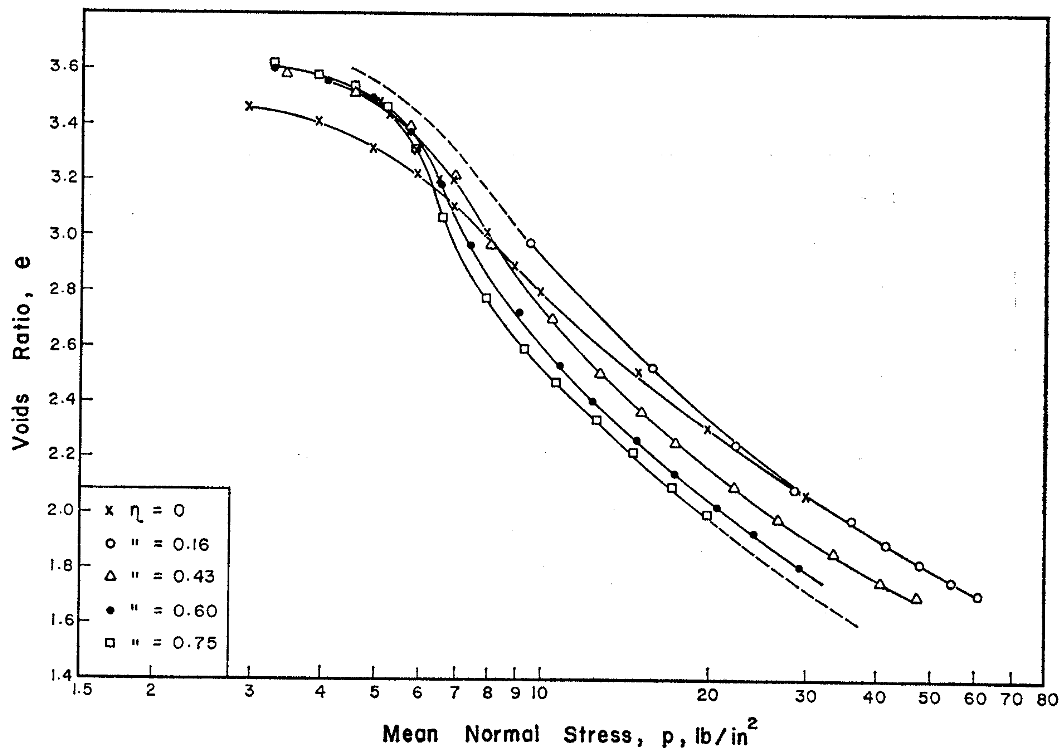

Fig. 4. (e, **log** p) **characteristics of all specimens tested under anisotropic consolidation**

The intersection of these two straight lines would represent a point which corresponds to the boundary which separates large shear strains from small shear strains. In that sense this boundary would represent a form of yield locus for shear strains associated with the volumetric strains. This boundary is the one shown by dashed line in Fig. 3.

The yield locus corresponding to the shear strain appears to lie outside the yield locus derived from the volumetric strain. It therefore seems that Weathered Nong Ngoo Hao Clay is capable of undergoing volumetric yielding at a lower stress boundary than the stress boundary required for shear distortion. Since the two types of curves are similar and are only separated by a small distance in the stress space, for all

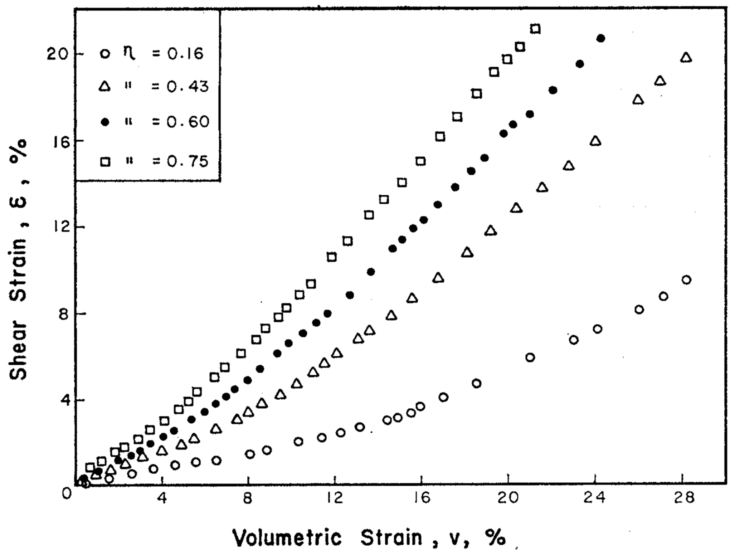

Fig. 5 (a). Strain path followed during anisotropic consolidation

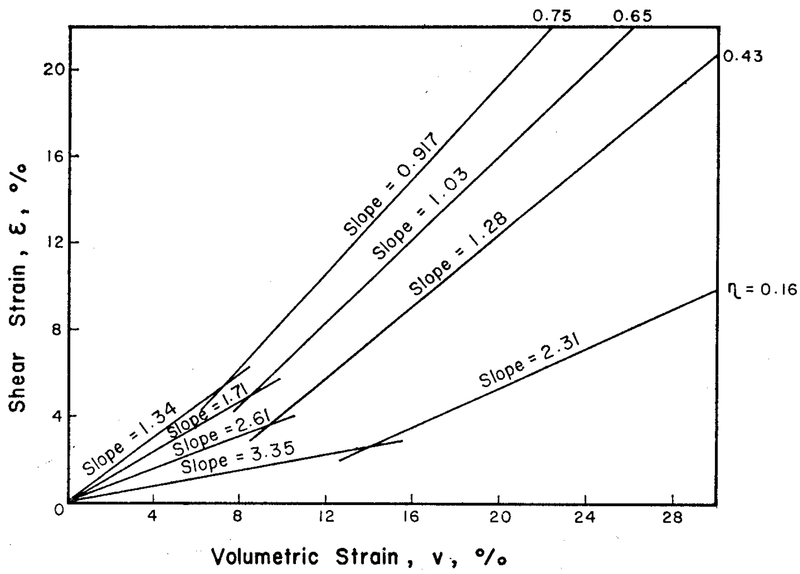

Fig. 5 (b). Idealised strain paths

practical purposes, the inner boundary could be considered as a volumetric yield locus. Associated with the volumetric yielding would be the yielding in shear distortion.

The strain increment ratio $(dv/d\varepsilon)_\eta$ during anisotropic consolidation will be considered in two parts. One part will correspond to the inside of the boundary ABCDE of Fig. 3 and the other will correspond to the case outside the said boundary (shown by dashed line).

In Fig. 6, $(dv/d\varepsilon)_\eta$ is plotted with respect to η for the two cases. The dotted curve in Fig. 6 represents the strain increment ratio $(dv/d\varepsilon)_\eta$ inside the boundary and would be applicable for all stress corresponding to the overconsolidated state, whereas the full line curve corresponds to $(dv/d\varepsilon)_\eta$ outside the yield boundary. The application of these curves in the calculation of shear strains in drained tests will be discussed in a later section.

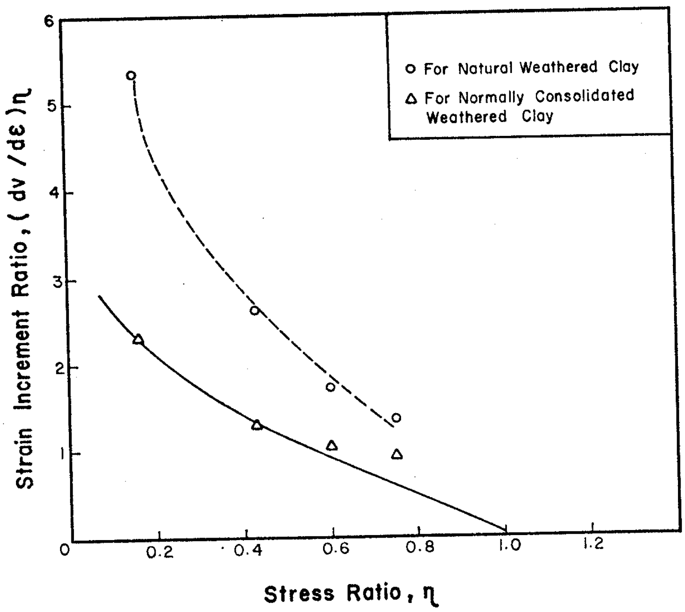

Fig. 6. Variation of $(dv/d\varepsilon)_\eta$ with η

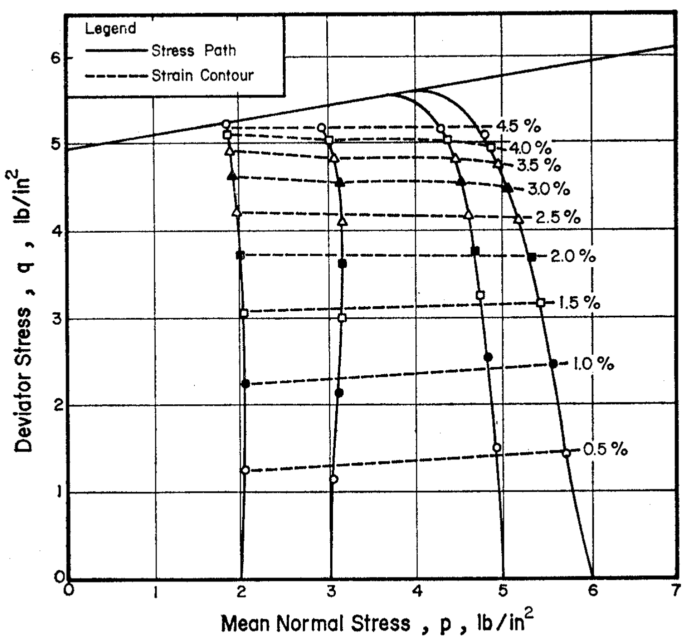

Fig. 7. Effective stress paths followed during undrained tests of Weathered Clay tested at low consolidation pressures

UNDRAINED TESTS AT LOW PRE-SHEAR CONSOLIDATION PRESSURES

Stress Path

In this series of tests, specimens were sheared under undrained conditions from isotropic stresses of 2, 3, 5 and 6 lb/in^2. Fig. 7 illustrates the effective stress paths followed by the specimens in the q, p plot. The end points of the specimens are also shown in this plot. In Fig. 7, the stress paths are found to be approximatery parallel to q-axis indicating that the mean normal stress p did not vary during shear. These results are in good agreement with the elastic wall concept of the stress-strain theories developed at Cambridge (see Calladine, 1963; Roscoe, Schofield and Thurairajah, 1963; Schofield and Wroth, 1968). According to these theories only elastic volumetric strains take place inside the state boundary surface and these volumetric strains are only dependent on the normal stress. Since the volumetric strains are zero in undrained tests, there would not be any change in the mean normal stress, thus the stress paths rise vertically in the (q, p) plot until failure is reached.

The shear strain contours corresponding to values of 0.5, 1, 1.5, 2.0, 2.5, 3.0, 4 and 4.5% are also shown in Fig. 7. The shear strain contours are found to be approximately parallel and nearly horizontal. This would indicate that the magnitude of the shear strain is independent of the magnitude of the mean normal stress. A similar observation was noted by Roscoe and Burland (1968) when they replotted the data of the shear strain contours obtained by Wroth and Loudon (1967). These constant shear strain contours were called by Roscoe and Burland (1968) as the constant q yield loci and were used in the prediction of shear strains on specimens of clay sheared along stress paths which lie below the state boundary surface.

WEATHERED CLAY TESTED IN THE NORMALLY CONSOLIDATED RANGE

In this series of tests, four samples were sheared from pre-shear consolidation pressures of 15, 30, 40 and 50 psi under the undrained condition with constant cell pressure. Also, three specimens were sheared under the drained condition with constant cell pressure from pre-shear consolidation pressures of 15, 30 and 40 psi.

Undrained Test Series

The effective stress paths followed by the specimens are shown in Fig. 8(a). The end points of the specimens are also shown in this plot. The effective stress paths are found to be similar for all the four specimens sheared under different pre-shear consolidation pressures. The effective stress paths are therefore normalised using the stress parameters $(q/p_e, p/p_e)$. These parameters transfer the state boundary surface for normally consolidated clays in (e, p, q) space to a two dimensional curve (see Roscoe, Schofield and Wroth, 1958; Roscoe and Poorooshasb, 1963). p_e in the case of an undrained test will be the same as the pre-shear consolidation pressure. The state paths followed by all the four specimens in the $(q/p_e, p/p_e)$ plots are shown in Fig. 8(b).

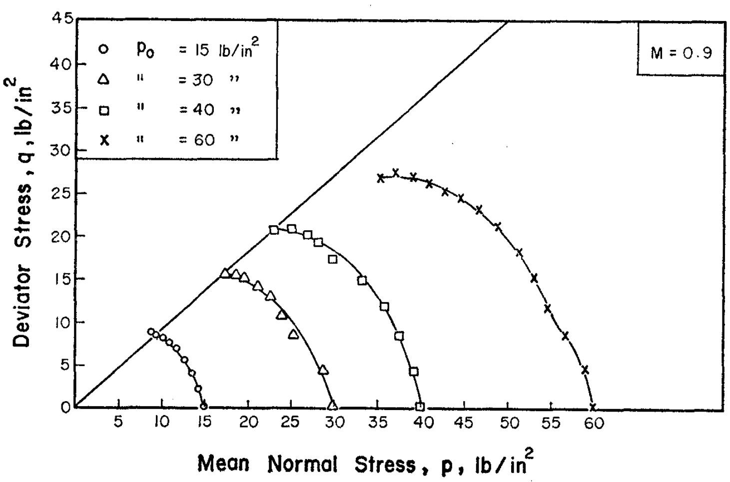

Fig. 8 (a). Effective stress paths followed by Weathered Clay under undrained condition from normally consolidated states

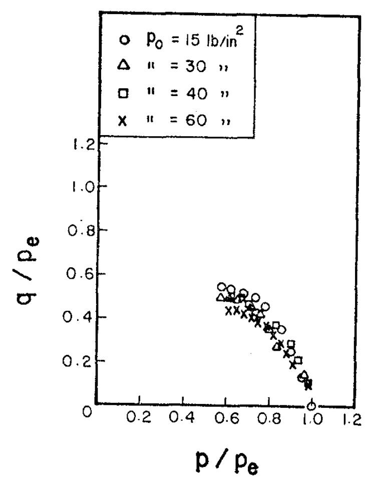

Fig. 8 (b). State paths followed by Weathered Clay tested in the normally consolidated states

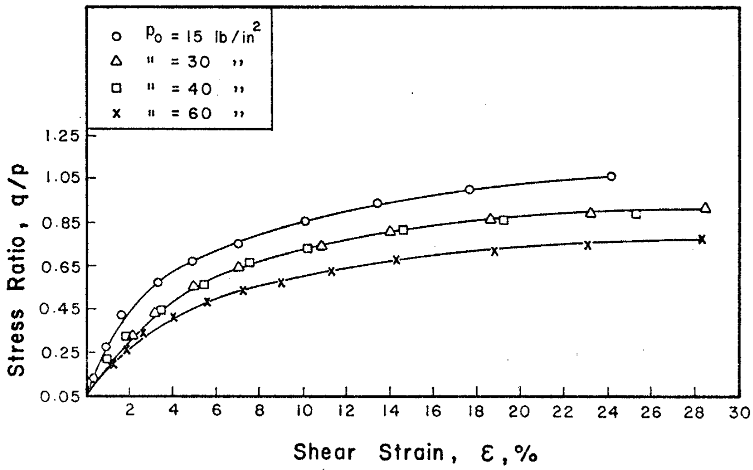

Fig. 9. Variation of ε with q/p for specimens sheared under undrained condition in the normally consolidated state

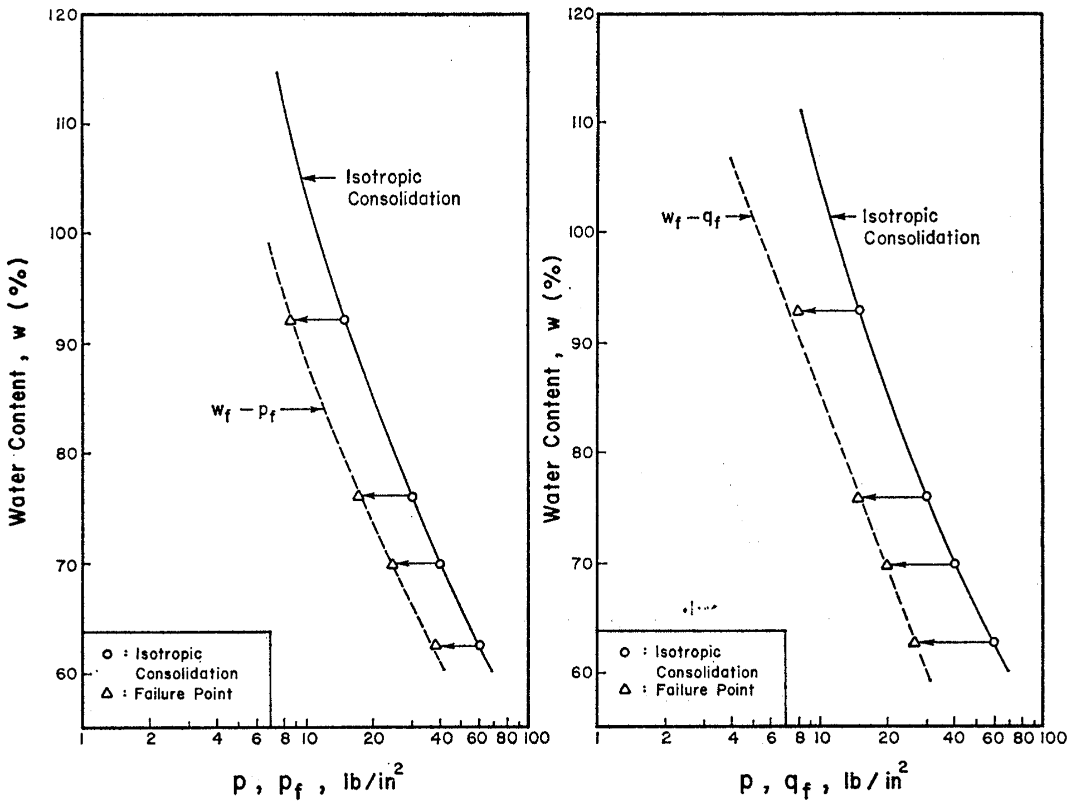

Fig. 10. $(w, \log p)$ and $(w, \log q)$ characteristics at failure of specimens sheared under undrained condition in the normally consolidated state

The state paths followed by the specimens are found to be unique in this plot, thus confirming the similarity of undrained effective stress paths and the uniqueness of the state boundary surface. A similar conclusion was reached by Balasubramaniam (1974), when normally consolidated specimens of Kaolin were sheared under undrained conditions in the conventional triaxial apparatus.

The end points of the specimens sheared under undrained condition are shown in Fig. 8(a) in the (q, p) plot. They are found to lie on a straight line which pass through the origin. The critical state parameter M is equal to 0.9. The corresponding value of the angle of internal friction $\bar{\phi}$ is 22.2°.

Fig. 9 illustrates the variation of the shear strain, ε, with respect to the stress ratio q/p. It is noted that the shear strain-stress ratio relationship is approximately the same for all the samples sheared under different pre-shear consolidation pressures. However, the sample sheared from 15 lb/in^2 isotropic stress seems to have a lower shear strain than the other samples. It is possible that this specimen which was consolidated to the least isotropic stress may still possess a slight degree of overconsolidation.

Fig. 10 illustrates the states of the sample at the end of isotropic consolidation and also at the end of shear plotted on a water content logarithm of mean normal stress plot. The initial states of the samples are adjusted to lie on the isotropic consolidation line. The end points of the specimens are again found to lie on a line parallel to the isotropic consolidation line in the (w, log p) and (w, log q) plot. These observations are in agreement with the statements made by Roscoe, Schofield and Wroth (1958) about the critical state line.

Drained Test Results

The stress paths followed by the specimens in drained tests should be linear in the (q, p) plot with $dq/dp=3$. The stress and state paths followed by all the three specimens (sheared from pre-shear consolidation pressures of 15, 30 and 40 lb/in^2) are shown in

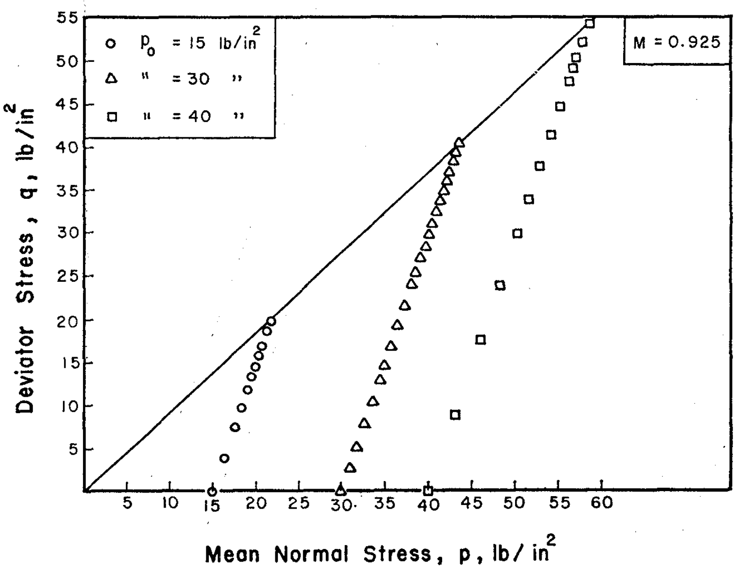

Fig. 11 (a). Stress paths followed during drained tests

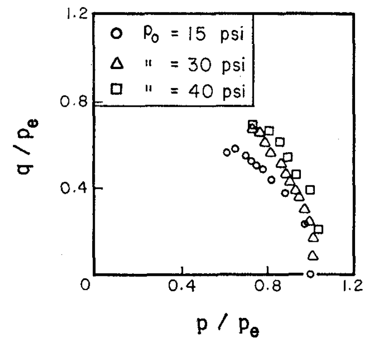

Fig. 11 (b). State path followed during drained tests

Figs. 11(a) and (b). In Fig. 11(b), both the deviator stress and the mean normal stress are normalised by p_e, where

$$p_e=p_0\exp\{(e_0-e)/\lambda\}$$

p_e=mean equivalent pressure.

p_0=the mean normal stress on the isotropic consolidation line corresponding to the pre-shear void ratio e_0.

λ=the slope of the isotropic consolidation line in the (e, log p) plot.

The state paths followed by the specimens during drained test are approximately the same.

The peak stress conditions of the specimens are also shown in Fig. 11(a). The critical state line is found to be a straight line which passes through the origin. The critical state parameter M is 0.93 and the corresponding value of the angle of internal friction $\bar{\phi}$ is 23.5°.

For undrained tests on normally consolidated specimens of Weathered Clay, a unique

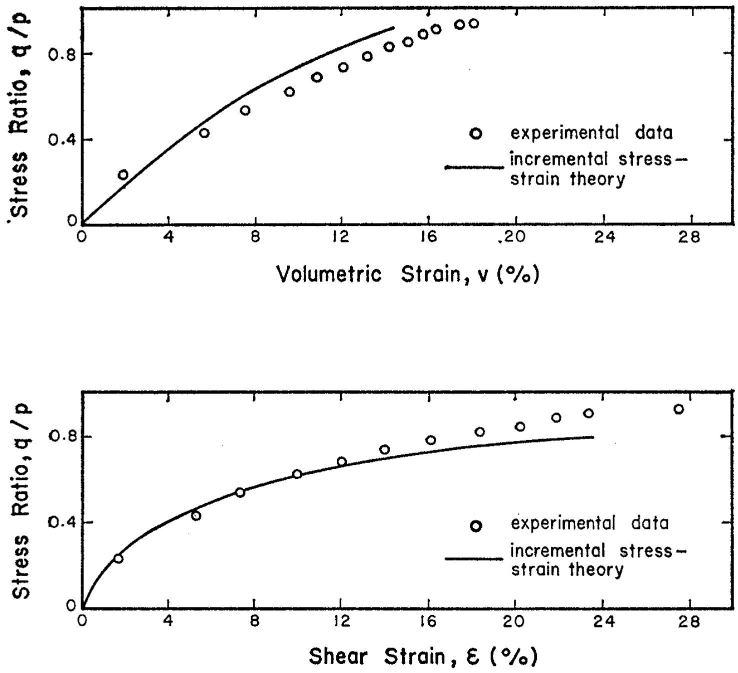

Fig. 12 (a). Observed strains and predicted strains for specimens sheared from 15 lb/in² isotropic stress under drained condition

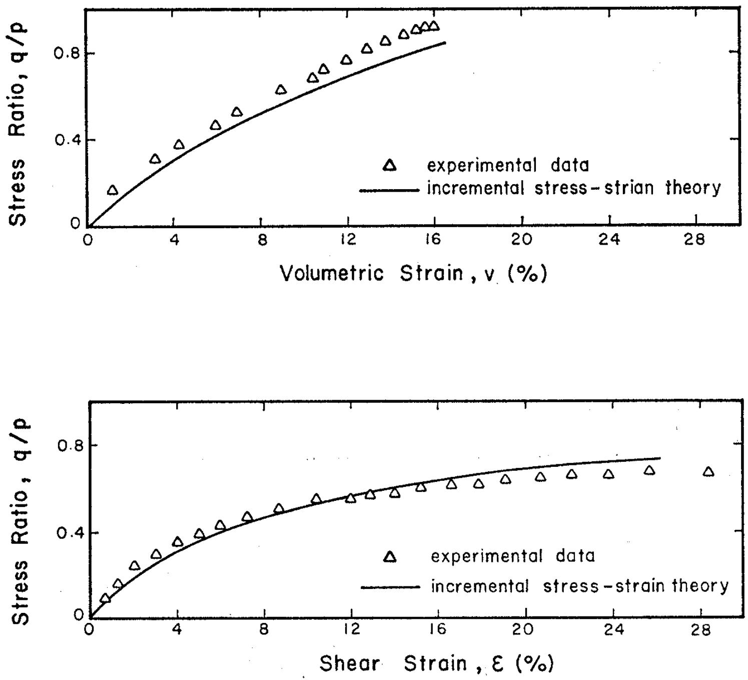

Fig. 12 (b). Observed strains and predicted strains for specimens sheared from 30 lb/in² isotropic stress under drained condition

relationship is observed between the stress ratio, q/p, and the shear strain, ε. Similar unique relationships between q/p and ε, and q/p and v are also noted for the drained tests.

Theoretical Prediction of Strains in Drained Tests

In this section the prediction of strains in drained tests for specimens sheared from

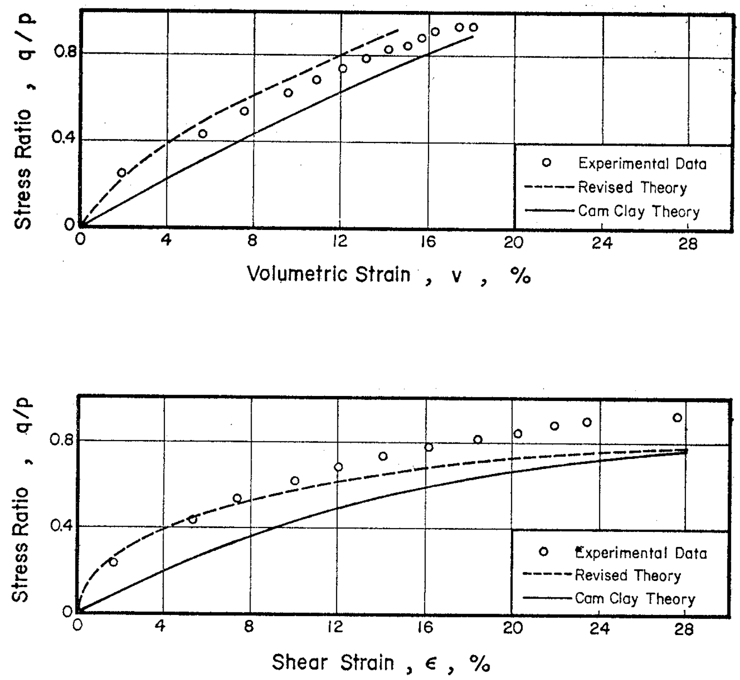

Fig. 13 (a). Observed strains and predicted strains using critical state theories (consolidation stress 15 lb/in^2)

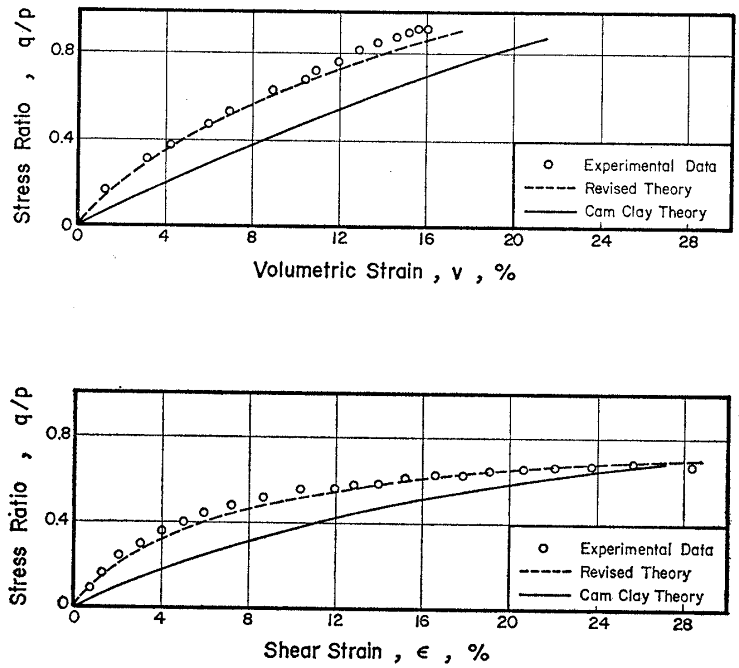

Fig. 13 (b). Observed strains and predicted strains using critical state theories (consolidation stress 30 lb/in^2)

pre-shear consolidation pressures of 15, and 30 lb/in^2 is made using the Incremental Stress-Strain Theory and the Critical State Theories.

An incremental stress-strain theory was proposed by Roscoe and Poorooshasb (1963) for the prediction of shear strain in drained tests with any applied stress path. In this theory, the volumetric strains in drained tests are determined using the state boundary surface obtained from undrained tests. Then the shear strains in the drained tests are calculated using the undrained shear strain, the slopes of anisotropic consolidation in (v, ε) space, and, the volumetric strains determined from the state boundary surface. The value of λ used in the prediction of volumetric strain is 0.51 and is obtained from isotropic and anisotropic consolidation tests data. The procedure for calculating the volumetric strain and the shear strain is described in detail in Balasubramaniam (1975). Figs. 12(a) and (b) illustrate the predicted strains in relation to the experimentally observed strains. The predictions from the incremental stress-strain theory are found to be excellent.

The critical state theories used in the prediction of strains are the Cam Clay theory (see Roscoe, Schofield and Thurairajah, 1963; Schofield and Wroth, 1968) and the Revised Theory (see Roscoe and Burland, 1968). Details of the derivation of these theories and their implications will not be given here. The fundamental soil parameters used in the Critical State Theories are λ, k and M. λ is the slope of the isotropic consolidation line in the $(e, \log p)$ plot and is equal to 0.51 for normally consolidated Weathered Nong Ngoo Hao Clay. k is the slope of the isotropic swelling line in the $(e, \log p)$ plot and is equal to 0.12. M is the slope of the critical state line in (q, p) plot and is 0.93.

In the Revised Theory of Roscoe and Burland, corrections are made for the shear strain from the contributions due to the constant q yield loci. The contributions from the constant q yield loci were approximately the same as the shear strains obtained from undrained tests in the $(q/p, \varepsilon)$ plot. Figs. 13(a) and (b) show the predicted strains from the two theories and the experimentally observed strains. It is noted that the Cam Clay Theory overpredicts the volumetric strain and the shear strain in all the tests. The Revised Theory successfully predicts the strains in all tests.

CONCLUSIONS

Based on the data from the different series of tests and the theoretical predictions, the following conclusions are reached.

(1) A yield locus for volumetric strain is established for the Weathered Clay which separates the states for which the volumetric strain is small from those for which the volumetric strain is large. This yield locus is very similar to the undrained stress paths on normally consolidated specimens of Weathered Clay.

(2) A yield locus for shear strains associated with the volumetric strains is also established. However, this yield locus appears to lie outside the yield locus for volumetric strains. Thus yielding in volumetric strain begins to take place at a lower stress level, than the yielding in shear distortion.

(3) The strain paths followed during anisotropic consolidation are made up of two straight lines with different slopes. One corresponds to the overconsolidated range and the other corresponds to the normally consolidated range. The strain increment ratio during anisotropic consolidation can also be considered in two parts. The $(dv/d\varepsilon)_\eta$ vs η relationships for both parts are found to be different.

For Weathered Clay tested in the undrained condition under pre-shear consolidation pressure less than the maximum past pressure, the following conclusions are reached.

(i) The effective stress paths are found to be approximately parallel to the q-axis in the (q, p) plot.

(ii) The constant shear strain contours in the (q, p) plot are found to be nearly parallel to the p-axis. Thus the shear strain is only dependent on the deviator stress.

For Weathered Clay tested in the normally consolidated range under undrained conditions the following conclusions are reached.

(i) The effective stress paths are found to be geometrically similar and the state paths followed by the specimens are unique in the $(q/p_e, p/p_e)$ plots.

(ii) The end points of the specimens are found to lie on a straight line in (q, p) plot. The slope of the critical state line is 0.93. The corresponding value of the angle of internal friction $\bar{\phi}$ is 22.2°. Unique water content strength relationships are observed, which are similar to the critical state projections for Soft Clay.

(iii) The stress ratio-shear strain relationship is unique and is independent of the pre-shear consolidation pressure.

For Weathered Clay tested in the normally consolidated range under drained conditions, the following conclusions are reached.

(i) The state paths in the $(q/p_e, p/p_e)$ plots are approximately unique and are the same as the state paths followed by undrained test specimens.

(ii) The critical state line is found to be a straight line in (q, p) plot. The value of M being 0.93. The corresponding angle of internal friction $\bar{\phi}$ is 22.3°.

(iii) Unique relationships are observed between the stress ratio, q/p, and the strains ε and v. Also, unique relationships are observed between the normalised deviator stress and the strains.

(iv) The strains in fully drained tests are successfully predicted by the incremental stress-strain theory of Roscoe and Poorooshasb (1963) and the stress-strain theory of Roscoe and Burland (1968).

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NOTATIONS

e = voids ratio
e_0 = voids ratio corresponding to pre-shear consolidation pressure p
k = slope of isotropic swelling line in $(e, \log p)$ plot
M = slope of critical state line in (q, p) plot
p = mean normal stress
p_0 = pre-shear isotropic consolidation stress
p_e = mean equivalent pressure
q = deviator stress
w = water content
$\sigma_1', \sigma_2', \sigma_3'$ = principal effective compressive stresses
$d\varepsilon_1, d\varepsilon_2, d\varepsilon_3$ = principal incremental compressive strains
dv = incremental volumetric strain
$d\varepsilon$ = incremental shear strain

$\left(\frac{dv}{d\varepsilon}\right)_\eta$ = value of $(dv/d\varepsilon)$ at constant η

λ = slope of isotropic consolidation line in $(e, \log p)$ plot

$\bar{\phi}$ = angle of internal friction

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