Geotechnical Testing Journal, Vol. 26, No. 4
Paper ID GTJ8933_264
Available online at: www.astm.org

T. H. Seah[1] *and K. C. Lai*[2]

Strength and Deformation Behavior of Soft Bangkok Clay

ABSTRACT: The influence of stress history and stress state during consolidation and shearing were studied experimentally under undrained triaxial compression and extension modes, by using the Recompression and SHANSEP techniques. In the SHANSEP tests, the samples were consolidated to a vertical consolidation stress of about 1.5 to 2 times of the preconsolidation pressure under K_o consolidation condition, followed by swelling to required overconsolidation ratio before subjecting to undrained shear. Whereas in the Recompression technique, the samples were directly consolidation to in situ effective stress before undrained shearing, the volumetric strains of the reconsolidation were less than 2%, indicating relatively reasonable sampling quality. From the undrained shear results, the Recompression tests give higher undrained shear strength than estimated from SHANSEP tests by about 28%, indicating that some destructuring of soil structure may have occurred in the SHANSEP tests due to large vertical deformation during consolidation. Consequently, the undrained shear strength and the undrained modulus obtained from the SHANSEP tests were much lower.

KEYWORDS: Anisotropy, Clays, Consolidation, Shear strength, Stiffness, Deformation

Introduction

Problems in soil engineering can be divided generally into two categories: one is the prediction of failure of the soil mass and the other concerns deformations. Predictions of stability and deformations required suitable soil models and soil parameters, which must reflect the actual soil behavior. Developments in geotechnical engineering are concerned with the development of theories for stability and deformation predictions, and the development of laboratory and field techniques for obtaining suitable soil parameters, which greatly contribute to the understanding of soil behavior.

Two techniques of shearing are commonly used in practice: they are Recompression and SHANSEP (Stress History and Normalized Soil Engineering Properties). The Recompression technique was developed at the Norwegian Geotechnical Institute (NGI), and the SHANSEP technique was first introduced by Ladd and Foott (1974). Test specimens following the Recompression techniques are reconsolidated to the effective overburden stress (σ'_{vo}), then the specimens are sheared under undrained condition. As for SHANSEP tests, the specimens are consolidated to stress level of 1.5–2 times beyond the preconsolidation pressure, and unloaded to required preshear consolidation stresses as described by Ladd et al. (1977).

The objective of this paper is to investigate the stress-strain-strength characteristics of soft Bangkok clay by using the Recompression and SHANSEP techniques under undrained shear condition. Two series of triaxial tests were performed by means of these techniques. In each series of tests, the soft Bangkok clay samples were subjected to undrained shear in both extension and compression modes. The clay samples used in the research were obtained from the Asian Institute of Technology (AIT) campus at depths ranging 2.5–7.5 m.

Received September 30, 1999; accepted for publication October 31, 2002; published November 17, 2003.

[1] Geotechnical Engineer, MAA Geotechnics Company, 39/165-7 Moo 13 Lad Phrao Road, Bangkok 10230, Thailand.

[2] Former Graduate Student of Asian Institute of Technology, 39/165-7 Moo 13 Lad Phrao Road, Bangkok 10230, Thailand.

Recompression Technique

Recompression technique has been used by NGI to predict in situ behavior. Bjerrum (1973) mentioned that provided the swelling of the soil sample that occurred before testing is so small or it is still of an elastic nature, and provided the mechanical disturbance is relatively small, then the detrimental effect of swelling can be eliminated if the sample is reconsolidated at exactly the same pressure as it is in the field before testing. It is duly beneficial such that, firstly, the field stresses are replaced with an identical set of effective stresses in the laboratory, and secondly, the adsorbed water since sampling is squeezed out of the sample again. The recompression technique has been successfully applied for triaxial tests performed on fixed piston samples of several low *OCR* clays as reported by Berre and Bjerrum (1973). The low plastic clay samples were found to be somewhat more disturbed than high plastic clay samples, and a reduced water content caused by sampling disturbance would be accompanied by a gain in strength. On the contrary, increasing disturbance will produce a reduction in strength and an increase in strain at failure for the reconsolidated specimen. Recompression technique has also been applied to highly structured Canadian clays by La Rochelle et al. (1981).

The applications of Recompression technique are as follows:

1. It is more appropriate with highly structured, brittle clays such as high sensitivity clay, cemented soils, weathered crusts, and heavily overconsolidated deposits, etc.
2. It is not recommended to be used in truly normally consolidated deposits because reconsolidation to $\sigma'_{vo} = \sigma'_p$ may overestimate the in situ c_u.
3. The test should always be accompanied by an evaluation of the in situ stress history to check the validity of the measured c_u values and to extrapolate/interpolate the discrete c_u data.

SHANSEP Technique

SHANSEP represents a design procedure for clay exhibiting normalized behavior. The concept is based on a laboratory testing procedure, which attempts to reproduce the conditions of in situ soils more systematically than in routine tests. It provides a way of taking into account the previous stress history of the soil, especially with regard to overconsolidation. From previous studies on the undrained shear strength of cohesive soils, there is a well-established relationship between normalized shear strength (c_u/σ'_{vo}) and overconsolidation ratio (*OCR*). This method is adopted by many laboratories to minimize the adverse effects of sample disturbance.

Fundamental to the SHANSEP approach is the determination of the overconsolidation ratio of the clay. The preconsolidation pressure (σ'_p) can be derived from carefully conducted oedometer test, or from the compression curve of a K_o-consolidated triaxial test.

The SHANSEP procedure can be outlined as follows:

1. Perform high quality oedometer consolidation test to determine σ'_p. The determination of σ'_p using the oedometer is not so sensitive to sample disturbance as the determination of the shear strength by a UU triaxial compression test. The value of σ'_p depends primarily on the memory or structure of the soil skeleton, rather than on the water content or effective stress after sampling.
2. Consolidate the triaxial specimens under K_o conditions in order to achieve a vertical stress of 1.5–4 times of the in situ σ'_p.
3. For overconsolidated conditions, the specimens are allowed to swell one-dimensionally to a vertical effective stress (σ'_{vc}) at the desired *OCR*.
4. Subsequently, conduct undrained triaxial compression test. A clay exhibiting normalized behavior will yield, for a certain value of *OCR*, a constant value of c_u/σ'_{vc}.
5. Repeat Steps 2–4 to obtain c_u/σ'_{vc} for each required *OCR*.

Advantages of SHANSEP technique are:

1. This method enables a more reliable evaluation of design parameters to be made by utilizing present understanding of the soil behavior.
2. It also enables realistic stress-deformation properties to be determined within the working range of applied stresses for use in design analyses.
3. It provides a clear picture of the soil profile by means of a correlation between stress history and variation of strength with depth.
4. It provides a slightly conservative indication of stability that is more reliable than the relatively erratic predictions from traditional design methods.
5. The method is well proven for embankments or foundation loading on many types of clay deposits.
6. Effects of sample disturbance on the test results are substantially overcome.
7. Only a relatively small number of carefully conducted effective stress tests is needed to confirm that normalized properties followed a regular trend.
8. Assumptions made regarding normalized behavior can be verified in the laboratory, and the effects of other assumptions on the design can be assessed.
9. The method gives more data than those from traditional testing but at little extra cost, and the data can provide additional information on undrained shear strength profiles.
10. The method enables prediction of the undrained strength of clay following changes in field conditions, e.g., softening due to swelling at the base of the excavation.

Disadvantages and limitations of the SHANSEP procedure are:

1. The SHANSEP method can be applied only to fairly uniform clay deposits and is unsuitable for random deposits, especially for those in which deformation alters their structures, such as highly sensitive clays and cemented clays.
2. The method is totally dependent on the knowledge of the stress history and a reliable estimate of the previous preconsolidation pressure.
3. The necessary laboratory tests require procedures that are more complex than those used in traditional testing practice.
4. Samples may need to be consolidated to higher stresses than experienced in normal testing practice.
5. The undrained shear strength derived by the SHANSEP method, though generally more reliable than those obtained from conventional tests, may not be applicable to traditional empirical procedures.

Testing Equipment and Procedure

A comprehensive laboratory testing program was conducted on soft Bangkok clay to compare the stress-strain-strength response of the Recompression and SHANSEP reconsolidation techniques. The method used for triaxial test is described in Germaine and Ladd (1988).

Triaxial Testing Equipment

The triaxial cell accommodating samples of up to 5 cm in diameter was used in this research. Each cell consists of three main components, namely a cell base, a removable perspex cylinder, and a top platen assembly. The cell base consists of a pedestal for the setup of the specimen and three water passages; two are drainage lines and the other is a cell water line. The drainage lines are connected to the top and bottom of the specimen. The drainage lines are also linked to a pore pressure transducer and a burette to measure the pore water pressure during undrained shearing and the volume change during consolidation and drained shear.

The desired stress conditions are imposed on the specimen by a combination of air pressure system and dead weight placed on top of the ram. The air pressure system is connected to a pressure regulator, and the reading can be read manually by a pressure gage. A loading machine is used to apply the vertical load to the sample via the piston under constant speed. A load cell, a displacement transducer, and two pressure transducers are used to measure the quantities in the test, and a data logger is used to record automatically the readings during the test.

Sample Preparation and Saturation

Before setting up the sample, the pressure lines were flushed with de-aired and distilled water to ensure that the system was saturated. Porous stones were cleaned using the ultrasonic cleaner and submerged in distilled–de-aired water prior to use.

During mounting of the trimmed specimen on the pedestal of the cell base, the saturated porous stones and circular filter papers were placed at both ends of the specimen. For compression tests, eight pieces of 8.5-mm-wide filter paper strips were placed vertically around the sample with equal spacing for the compression test. For the extension test, four pieces of 8.5-mm-wide filter papers were spiralled around the specimen surface at approximate 45° inclination; these filter papers only covered about 50 % of the specimen. Both ends of the filter paper strips were extended to the porous stones for easy drainage.

The sample was then enclosed by a rubber membrane, that is, sealed with two O-rings on each end. The sides of the base pedestal were coated with silicone grease in order to prevent water leaking through the seal. The perspex cylinder with top platen assembly was then mounted and fixed to the cell base. The chamber surrounding the specimen was filled with de-aired and distilled water.

Even though the specimen itself may be fully saturated, there is always a possibility that a certain amount of air is entrapped between the rubber membrane and the sample, and perhaps in the previously saturated porous stones and filter papers. A back pressure of 200 kPa is applied to ensure that the specimens would be fully saturated. The cell pressure and back pressure were gradually increased with the pressure increment of 25 kPa. During the application of pressures, the cell pressure was maintained higher than the back pressure at the value equal to the initial effective stress of the sample (15 ~ 18 kPa). When the back pressure reached 200 kPa, the specimen was left for at least 24 h. After 24 h of saturation, the degree of saturation was checked by the pore pressure response. For most of the specimens, a B value of over 98 % was achieved in less than one minute.

Consolidation Procedure

For Recompression technique, the consolidation procedure is summarized below:

1. The Recompression technique requires predetermination of in situ effective vertical stress (σ'_{vo}) of the test specimen. The determination of σ'_{vo} is accomplished by using data related to the sample depth, bulk densities, and thickness of all overlying strata and piezometric pressure conditions.
2. After the saturation is completed, the sample is consolidated under K_o condition. Initially, an additional deviator stress, $\Delta\sigma'_v$, is imposed on the sample through the loading platen. In this study, the first increment of $\Delta\sigma_v$ is taken as $0.2\sigma'_i$ for K of 0.833 (where σ'_i is the initial effective stress, which is in the range of 15–18 kPa for the tested soil).
3. When the primary consolidation is achieved (i.e., the time of t_{100} is achieved), the incremental axial strain ($\Delta\varepsilon_a$) and the incremental volumetric strain ($\Delta\varepsilon_{vol}$) are calculated from the vertical displacement transducer and volume burette readings. In the case that the full consolidation cannot be reached, the degree of consolidation can be checked by closing the drainage periodically.
4. For each loading increment, the K value is adjusted such that $\Delta\varepsilon_a$ is equal to $\Delta\varepsilon_{vol}$.
5. Steps 2–4 are repeated until $\sigma'_v = \sigma'_{vo}$ condition is reached.
6. When the K_o consolidation is completed, the sample is sheared under the undrained shearing condition.

As for SHANSEP technique, the consolidation procedure is as follows:

1. The SHANSEP technique also follows Steps 1–5 as in the Recompression technique.
2. The sample is consolidated to a maximum vertical consolidation stress (σ'_{vm}), which is about 1.5–2 times of its preconsolidation pressure (σ'_p), ensuring that the material behaved similarly to the normally consolidated soil. In this case, the σ'_{vm} is about 160 kPa.
3. If the required overconsolidation ratio is greater than 1, then the sample is allowed to swell to the required vertical stress. In this research, the maximum *OCR* used is 8 with corresponding swelling increments at *OCR* of 1, 1.2, 1.5, 2, 3, 4, 6, and 8.
4. When the sample is at the desired *OCR* value, the t_{100} is determined from the time curve, then the shearing is performed under the undrained condition, as described in the following section.

Shearing Procedure

There are two modes of shearing in triaxial tests: compression and extension. For the compression mode, the test is conducted by maintaining constant cell pressure (i.e., σ_3 = constant) and increasing the vertical load gradually. For the extension mode, the test is performed with the cell pressure remaining constant, but reducing the vertical load. The undrained shearing procedure is as follows:

1. Once the consolidation process is completed, the drainage lines are closed and the piston ram is fixed to keep the prevailing stress condition.
2. The base platen is moved up to transfer the load to the mounted load cell.
3. The strain rate of 0.4 %/h is selected for undrained shear test.
4. In the compression test, the sample is loaded by driving the base platen upwards, and hence, the sample is being compressed, whereas in extension test, the vertical load is unloaded by pulling the piston ram up.
5. The sample is sheared until it fails. The sample is then removed, and the final water content is obtained.

Tested Material and Testing Program

The tested material is soft Bangkok clay, which is a marine silty clay lying beneath the low flat plains of the central area of Thailand. This research focuses on this clay because of its high compressibility and low shear strength. The upper part of the soil profile is the weathered crust, which has been altered by fluctuation of the groundwater and desiccation. The crust contains some fissures that are resulted from alternate cycles of desiccation; therefore, the properties of this layer vary significantly from location to location. The shear strength characteristics of the weathered zone are much more variable and are dependent upon the duration and nature of the environmental conditions after deposition, as described by Moh et al. (1969). The water table fluctuates from 1 to 2 m below the ground level, depending on the season. Directly below the crust, there are layers of soft compressible clay and stiff clay, which are usually dark gray in color. The thickness of the three uppermost layers in the AIT campus consists of 2 to 3-m-thick weathered clay crust, followed by a soft clay layer of about 6 m, and underlain by a layer of stiff clay about 5 m thick.

Sampling

Undisturbed samples were collected by means of piston samplers of 7.5 cm diameter, 90 cm long. Augers were used to bore the holes for successive sampling process. The soft clay samples were taken continuously down to the depth of 8 m.

Once the undisturbed samples were withdrawn from the borehole, then two ends of the thin wall sampler were immediately sealed with paraffin to prevent any loss of moisture before transporting back to the laboratory humid room for storage. The sample extruded from the sampling tube was trimmed into a 12-cm-long piece and immediately covered with paraffin wax. The wax was heated to a temperature just above the melting point to prevent overheating, which might cause brittleness, thus impairing its sealing properties. The unprotected sample was carefully immersed in molten wax to prevent formation of any cavities. Then the sample was stored in a humid room at constant temperature and humidity.

Each piece of clay sample was trimmed to the required nominal dimensions of 5-cm diameter and 10 cm in height. A small hand operated soil lathe and a wire saw were used to trim the sample into a cylindrical shape. The initial water content of the specimen was determined from the trimmings.

Testing Program

A total of ten SHANSEP triaxial tests were conducted at various overconsolidation ratios, ranging from 1 to 8. The soft clay specimens for SHANSEP tests were obtained from depths between 3 and 5 m, in which clay in the zone is fairly uniform. All the samples were consolidated to the required stress levels before being subjected to undrained shear. Five samples were sheared in compression mode, and the rest were extension tests, so that the anisotropic behavior could be studied. Two tests on normally consolidated clay were performed to check the consistency of the test results, as well as to quantify the variability of the sample.

Twelve Recompression triaxial tests were performed on undisturbed clay samples taken from depths of 2.5–7.5 m. These tests were performed to determine the variation in the stress-strain-strength relationship of the soft clay with depth. Six samples were sheared in compression mode, and the other were sheared in extension mode, so that appropriate comparison could be made.

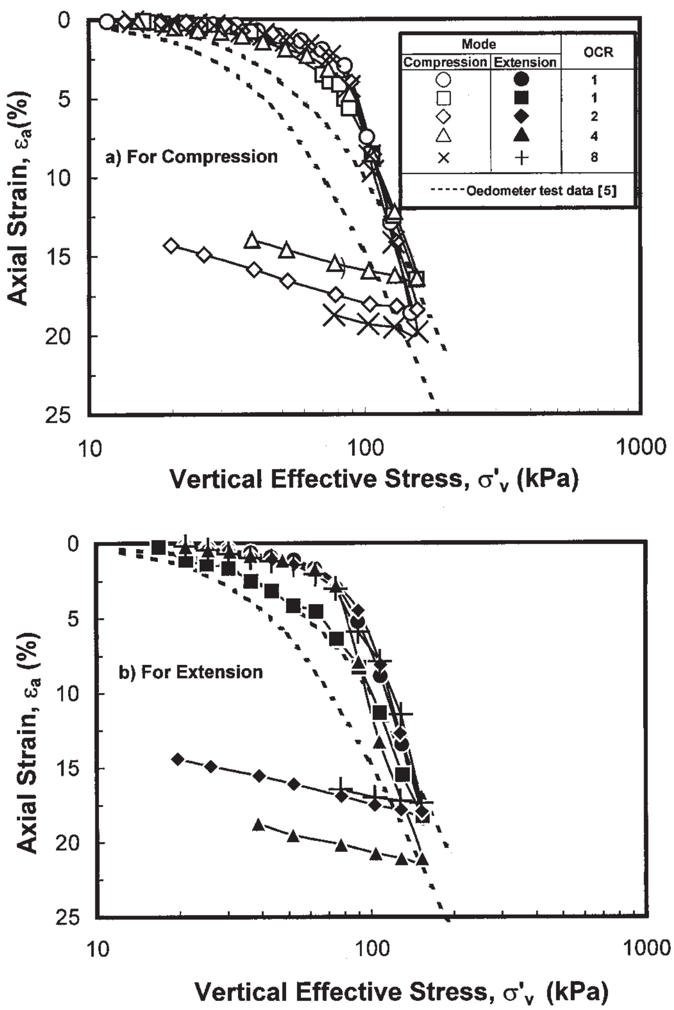

FIG. 1—•••

Results and Discussion

Consolidation Behavior

In the SHANSEP technique, the clay specimens were consolidated to 1.5–2 times beyond their preconsolidation pressures; this is performed to reduce the effect of sample disturbance. Since the clay samples used in these tests were taken from almost the same depth, the preconsolidation pressure and consolidation characteristics obtained from these tests can be compared directly with oedometer tests too. The results shown in Fig. 1 indicated relatively small variations in the preconsolidation pressure, as well as the compression ratios. Based on these results, the consolidation procedure used in triaxial test has proven to be appropriate, since the results match well with the conventional oedometer tests reported by Hassan (1993).

In the SHANSEP tests, it is also possible to obtain a relationship between K_o and OCR (Fig. 2), since the samples are allowed to swell to different OCRs. A typical relationship is as follows:

$$\left(\frac{K_o}{\sigma'_{vc}}\right)_{OC} = \left(\frac{K_o}{\sigma'_{vc}}\right)_{NC} (OCR)^{\alpha}, \text{ (Ladd et al. 1977)} \qquad (1)$$

or

$$K_o = (1 - \sin\phi')OCR^{\sin\phi'}, \text{ (Mayne and Kulhawy 1982)} \qquad (2)$$

From linear regression analysis on the test data as presented in Fig. 2, the $(K_o)_{NC}$ value for normally consolidated clay and the α value are 0.607 and 0.333, respectively. These values are compared

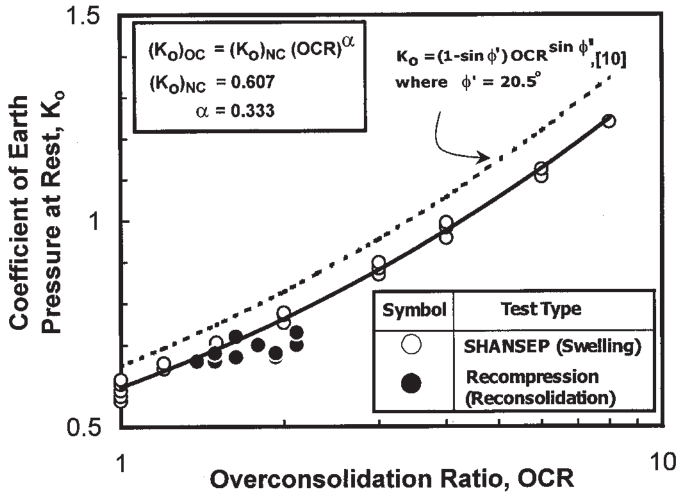

FIG. 2—•••

with those reported by Ladd et al. (1977). In this case, the friction angle obtained from the compression test is about 20.5°, hence the measured $(K_o)_{NC}$ agrees well with the Jaky equation, i.e., $K_o = (1 - \sin \phi')$. As for the α value, the α of 0.333 for Bangkok clay falls close to the value of $\sin \phi'$ as indicated in Fig. 2.

As for Recompression tests, the compression curves are presented in Fig. 3 with the specimens consolidated to in situ effective pressure only. The amount of volumetric strain developed at effective overburden stress can be used as an indication of the degree of sample disturbance. The magnitude of volumetric strain decreases with increasing *OCR*, as shown in Fig. 4, implying that the normally consolidated clay is more susceptible to disturbance. Most of the Recompression tests performed have volumetric strain ranging from 1 to 2 % at effective overburden stress. According to Lacasse and Berre (1988), the samples were considered to be of reasonable quality with low degree of sample disturbance; therefore, the undrained shear strength obtained from the Recompression test should be close to the in situ value.

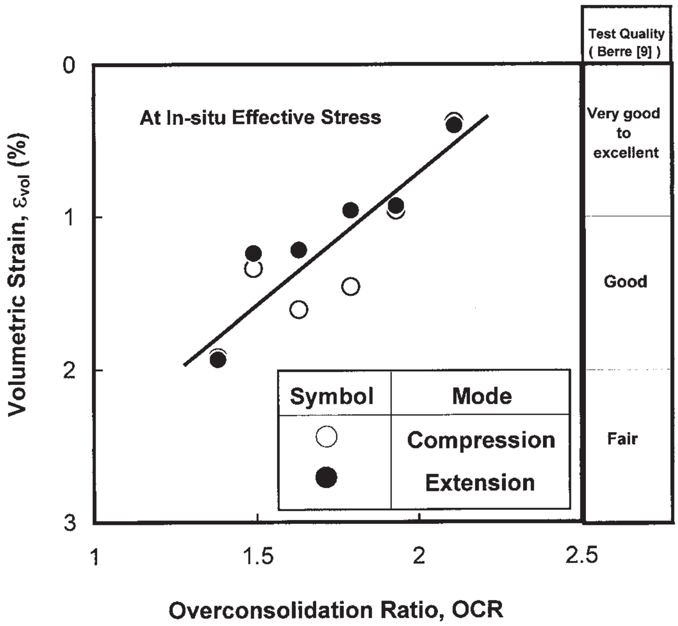

FIG. 4—•••

Shearing Behavior

In triaxial testing, membrane resistance causes a considerable influence on axial stress of very soft to soft clay; therefore, appropriate corrections have to be made. The other crucial influence on axial stress is the cross-sectional area of the sample. The axial stress is computed by dividing the piston force (with corrections for friction uplift, membrane resistance, etc.) by the effective area. Due to substantial deformation caused during both consolidation and shear, it is necessary to compute the appropriate area of the specimen. During K_o consolidation, the specimen is considered to deform as a right cylinder. In the process of shearing, the mid-plane section can undergo various types of deformation. In this study, the deformation has been considered as parabolic nature.

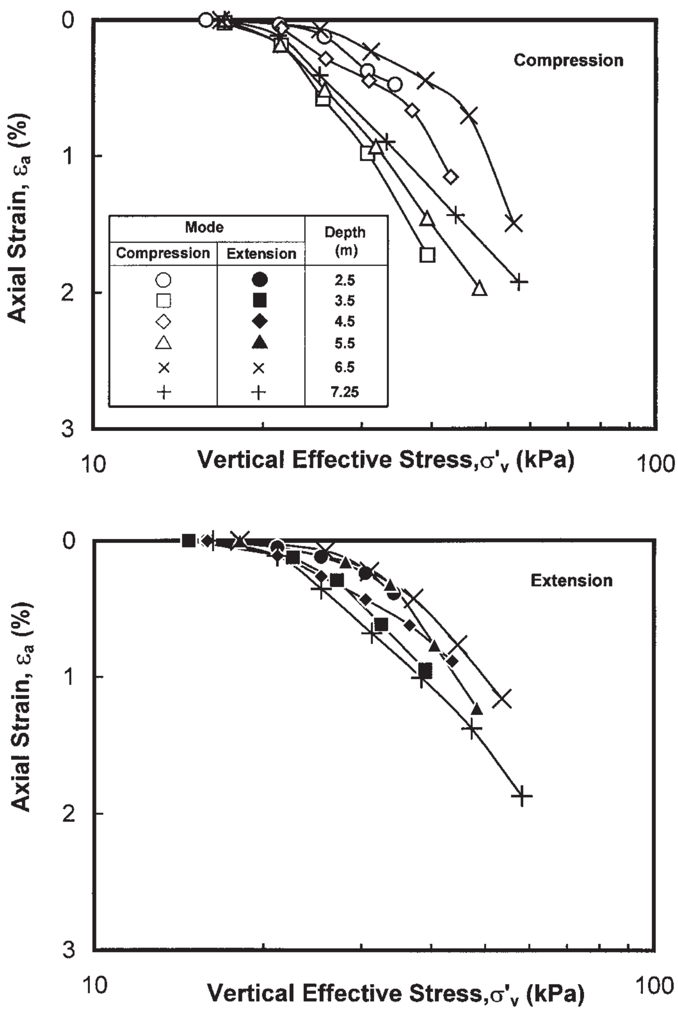

FIG. 3—•••

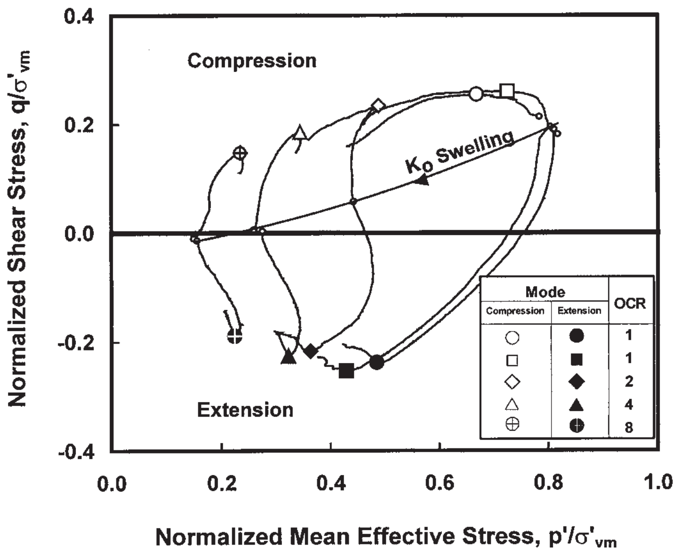

FIG. 5—•••

SHANSEP Test Results

1. Stress path: The normalized effective stress paths of SHANSEP tests are presented in Fig. 5. There is some varia-

tion in the effective stress paths of the two normally consolidated samples, partly because of the difference in the preshear horizontal consolidation stress or K_c value. Two undrained triaxial extension tests were also performed on normally consolidated samples; one test had a drop in the mean effective stress at the beginning of shearing that is probably attributed to the transferring of load from the dead weight to the load cell. The normalized effective stress paths of the other three tests at $OCR = 2$, 4, and 8 show that the initial portions of the paths are inside the yield surface; the peaks of these tests form the yield envelope for this material.
2. Stress-strain relationship: Fig. 6 shows plots of q/σ'_{vm} versus axial strain; the peak value of q/σ'_{vm} decreases with the increase of *OCR*, and the axial strain at failure increases with increasing *OCR*. All stress-strain curves indicated strain-softening behavior in compression mode, and strain-hardening behavior in extension mode. Normalized shear modulus (G/σ'_{vc}) is plotted with the shear strain as shown in Fig. 7, indicating increase in modulus with *OCR*. The difference is more profound in compression mode than in extension mode. The variation in normalized shear modulus with *OCR* is relatively small in extension mode.
3. Pore pressure response: A plot of normalized pore pressure versus axial strain is shown in Fig. 8, indicating the higher pore pressure with decreasing *OCR* for compression tests. In the extension tests, the pore pressure reaches a minimum value and decreases gradually with increasing strain.

Recompression Test Results

1. Stress path: Twelve recompression tests were conducted in both compression and extension modes, as shown in Fig. 9. For the compression tests, all the stress paths give similar form; there is very little change in the mean effective stress up to the peak strength. Once the peak value is reached, the stress path moves towards the original. In extension tests, the mean effective stress decreases gradually during shearing and reaches a peak value in about the same stress ratio, that is, on the same failure line.
2. Stress-strain relationship: The stress-strain curves of the Recompression tests are presented in Fig. 10, showing that the axial strain at failure is almost the same for compression and extension tests for samples taken from same depth. Most samples exhibit strain-softening behavior, except for deeper samples. Relationship between normalized shear modulus and shear strain for both compression and extension tests are shown in Fig. 11; the curves fall within a very narrow band for all the tests regardless of the depth and shearing mode. There seems to be a unique relationship between the normalized shear modulus and the shear strain for this layer of soft clay.
3. Pore Pressure Response: In the compression tests, the pore pressure increases with increasing in axial strain, and it remains constant beyond the peak strength as shown in Fig. 12. In the extension tests, the pore pressures decrease gradually until the peak strength, thereafter, remains constant.

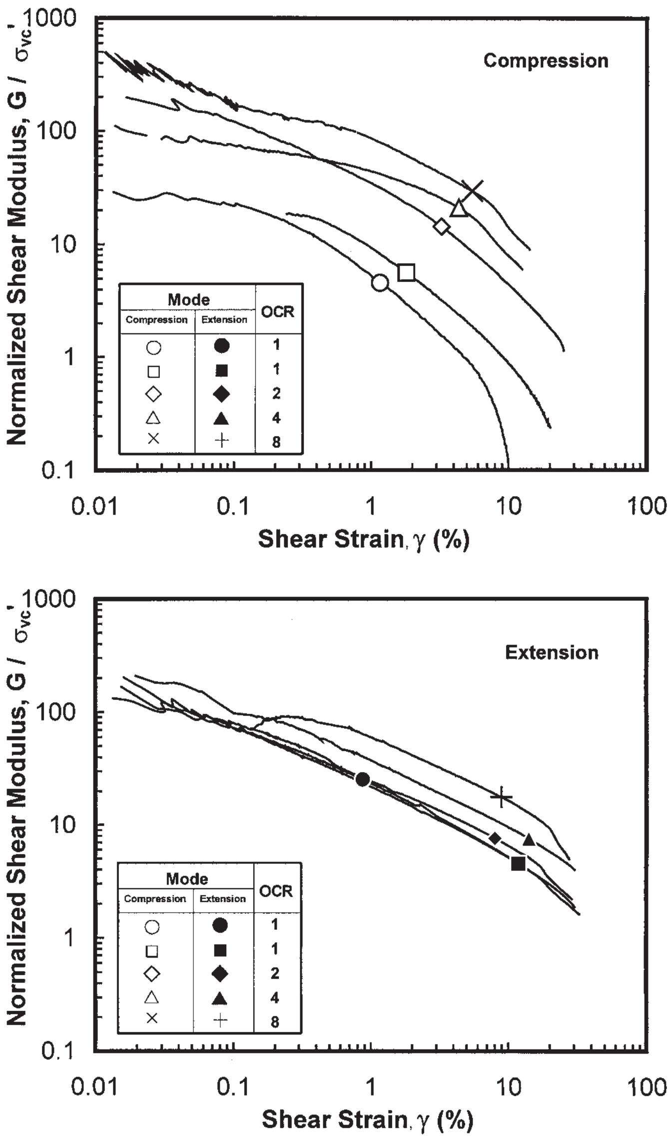

FIG. 7—•••

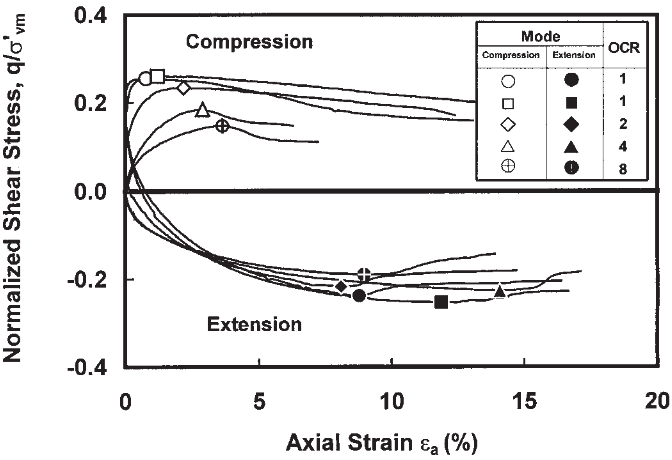

FIG. 6—•••

Result Comparisons

The SHANSEP test results are usually expressed in terms of normalized undrained shear strength $(c_u/\sigma'_{vc})_{OC}$ and the overcon-

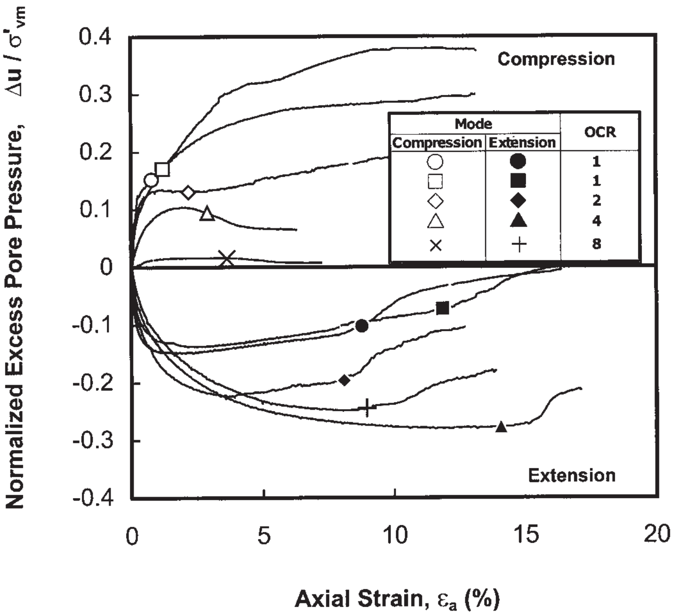

FIG. 8—•••

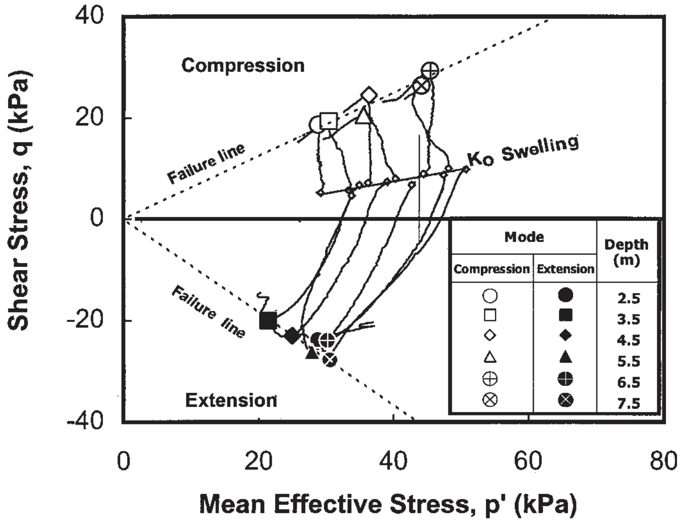

FIG. 9—•••

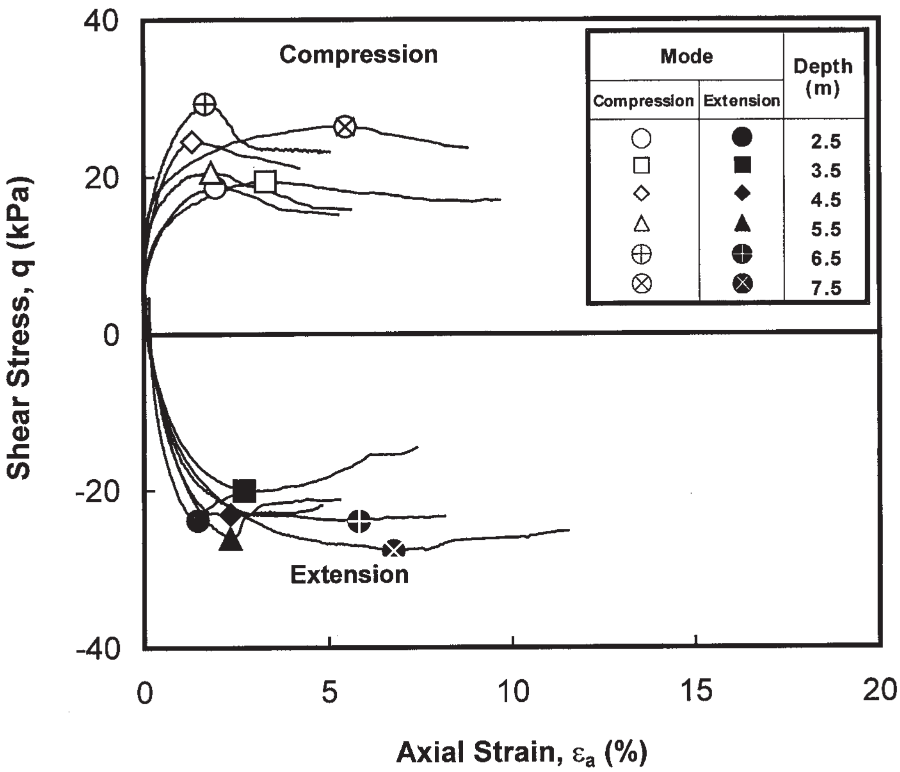

FIG. 10—•••

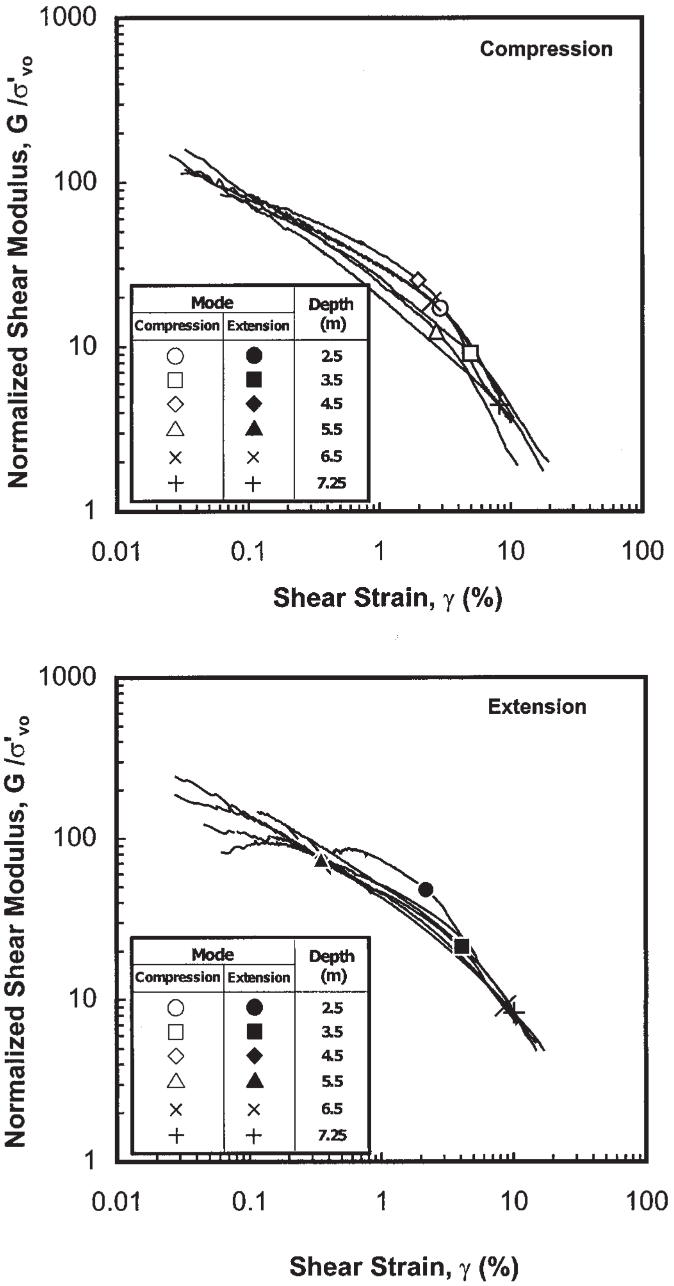

FIG. 11—•••

solidation ratio, having a relationship given below:

$$\left(\frac{c_u}{\sigma'_{vc}}\right)_{OC} = \left(\frac{c_u}{\sigma'_{vc}}\right)_{NC}(OCR)^m \tag{3}$$

The results of SHANSEP tests are summarized in Fig. 13, giving $(c_u/\sigma'_{vc})_{NC}$ values for compression and extension tests of 0.265 and 0.245, respectively. The magnitude of m represents the rate of strength increase with the overconsolidation ratio, having values of 0.735 and 0.89 in compression and extension modes. This implies that the rate of increase in the undrained shear strength in extension mode is greater than the rate in the compression mode. The ratio of $(c_u/\sigma'_{vc})_{NC}$ in extension to $(c_u/\sigma'_{vc})_{NC}$ in compression is defined as the anisotropic undrained strength ratio (K_s), for this clay, with a value of 0.92 for normally consolidated clay. This K_s value increases with the overconsolidation ratio, as shown in Fig. 14. At OCR less than 1.73, the undrained shear strength in compression test is larger than the value in extension test, but at OCR greater than this value, the extension strength becomes greater.

Ladd and Edgers (1972) reported the $(\tau_{max}/\sigma'_{vc})_{NC}$ value from the direct simple shear test (DSS) for normally consolidated Bangkok clay with I_p of 41 ± 20 % = 0.265 and the reported m value = 0.75. Due to anisotropic nature of the clay, the normalized undrained shear strength in the DSS mode is usually less than the triaxial com-

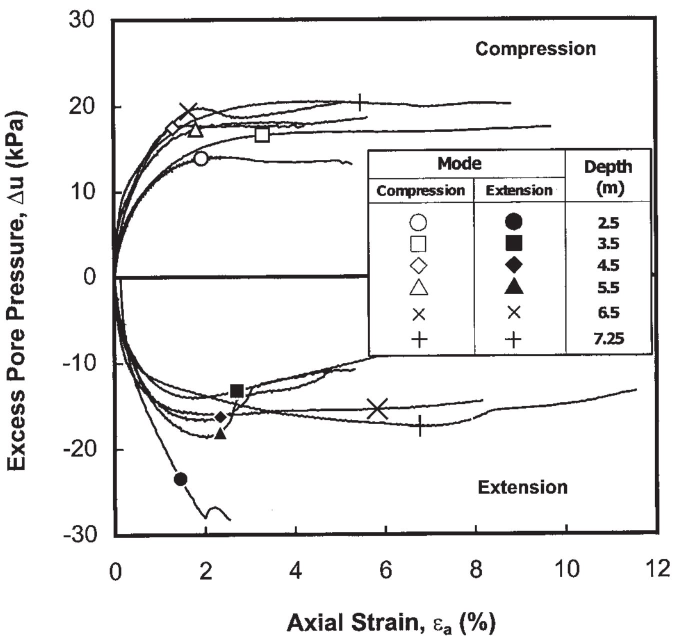

FIG. 12—•••

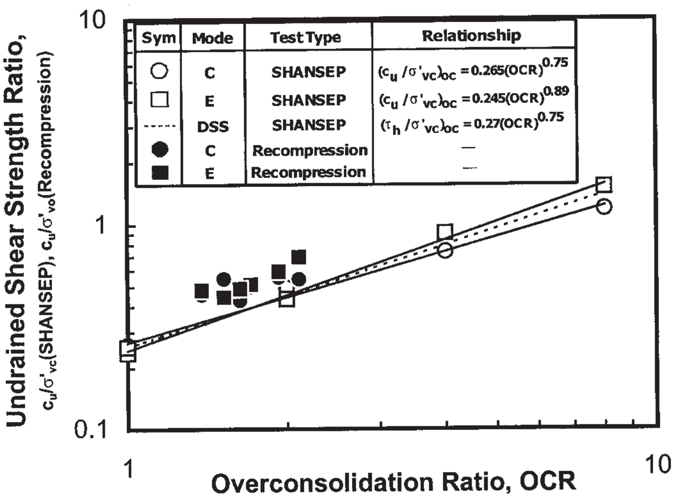

FIG. 13—•••

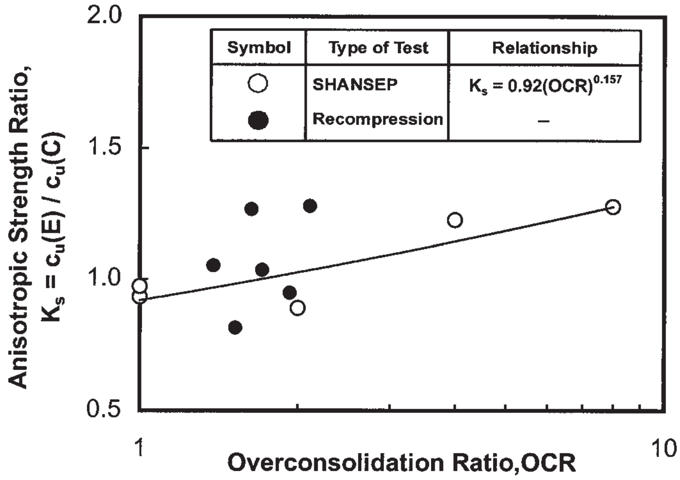

FIG. 14—•••

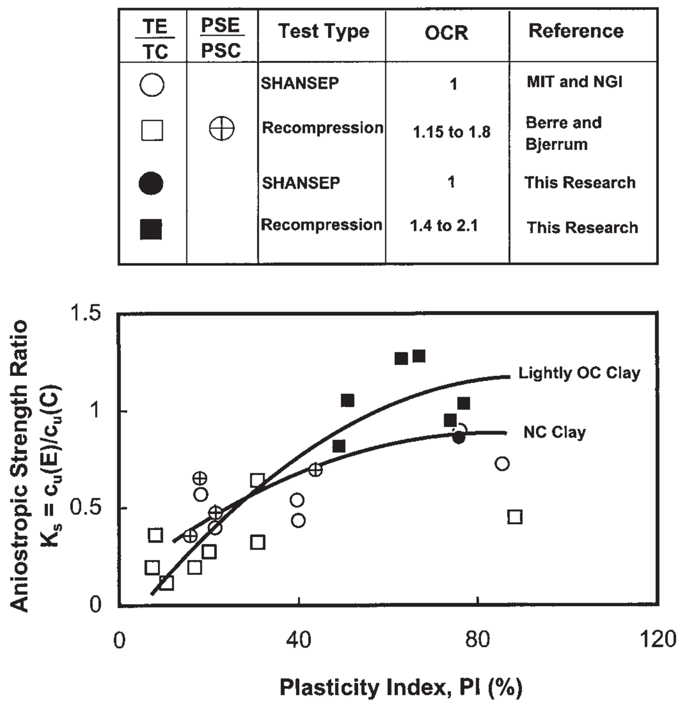

FIG. 15—•••

pression strength. The normalized shear strength in compression is found to be almost the same as the value from the DSS test. The difference in values for different shearing modes is found to be small, implying that Bangkok clay is relatively isotropic.

The anisotropic strength ratios obtained from this research are compared with those from previous tests, indicating a fairly consistent trend as shown in Fig. 15. Higher anisotropic strength ratio is observed for lightly overconsolidation ratio. As for normally consolidated clay, the ratio increases with increasing plasticity index with value approaching unity at higher index.

For the recompression tests, the normalized undrained shear strengths are plotted along with their *OCRs*. The *OCR* values are estimated based on the preconsolidation pressures obtained from oedometer tests, as shown in Fig. 13. The results indicated that the normalized undrained shear strengths for the Recompression tests are higher than the SHANSEP tests. Within this range of *OCR*, the undrained shear strengths in extension tests are somewhat higher than the values in compression tests.

From the stress-strain curves of SHANSEP tests, a relationship between the normalized shear modulus (G_{50}/σ'_{vc}) and the overconsolidation ratio is presented in Fig. 16, having the equation in the following form:

$$\text{For compression mode, } \left(\frac{G_{50}}{\sigma'_{vc}}\right)_{OC} = 13(OCR)^{0.867} \quad (4)$$

$$\text{For extension mode, } \left(\frac{G_{50}}{\sigma'_{vc}}\right)_{OC} = 8.1(OCR)^{0.708} \quad (5)$$

The normalized shear modulus (G_{50}/σ'_{vc}) increases with increasing *OCR*, with compression modulus greater than extension modulus. The normalized shear moduli of the Recompression tests are also plotted in the same figure, showing much stiffer response than the SHANSEP tests by about 2–3 times.

The pore pressure parameter at failure, A_f, decreases with increasing *OCR*, as shown in Fig. 17. For overconsolidated clay, the A_f value obtained from the compression test is lower than the extension test, except at *OCR* of 1. In compression test, the A_f value varies from 1.2–0.23 for *OCR*, ranging from 1 to 8. In extension test, the A_f value decreases from 0.90 to 0.31 for*OCR*of 1–8. There seems to be a good match for the A_f values from SHANSEP and Recompression tests.

Strength Profile

Based on the stress history obtained from oedometer tests, the undrained shear strengths at different depths can be estimated by the SHANSEP equations for both compression and extension modes. The results of undrained shear strength from Recompression and SHANSEP tests are compared as presented in Fig. 18, in-

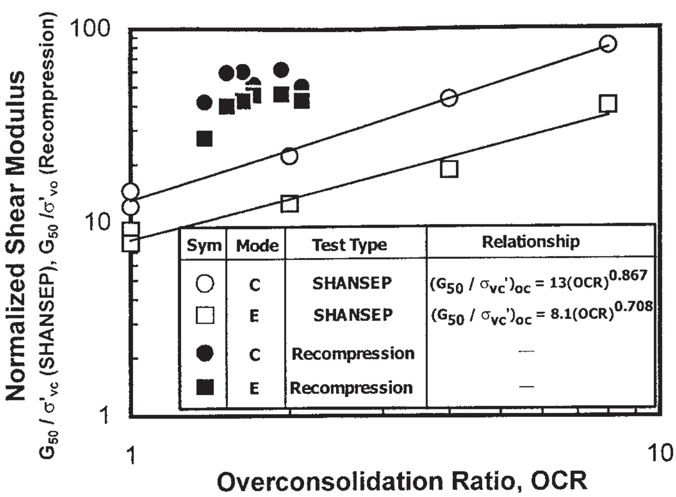

FIG. 16—•••

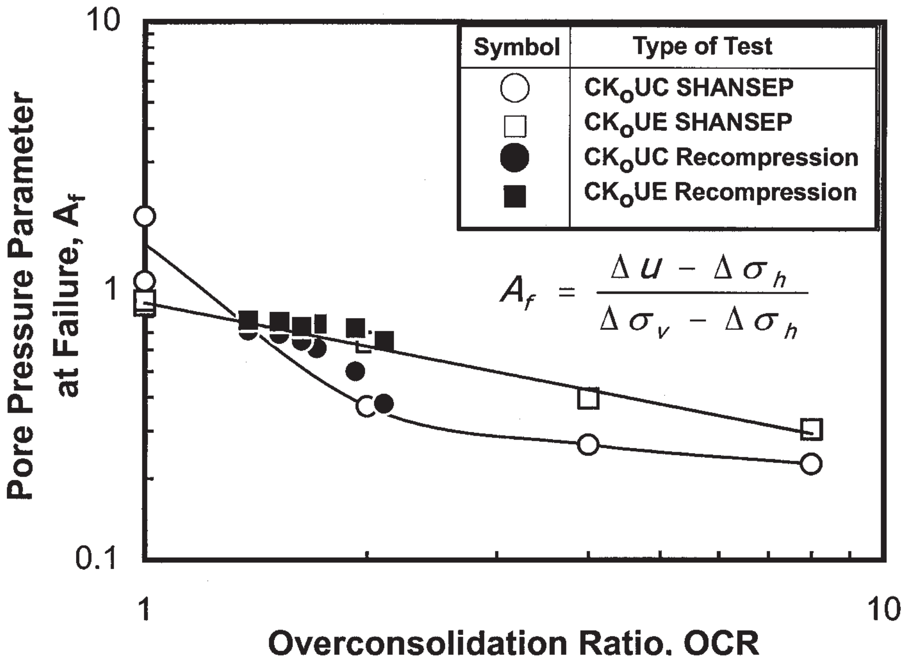

FIG. 17—•••

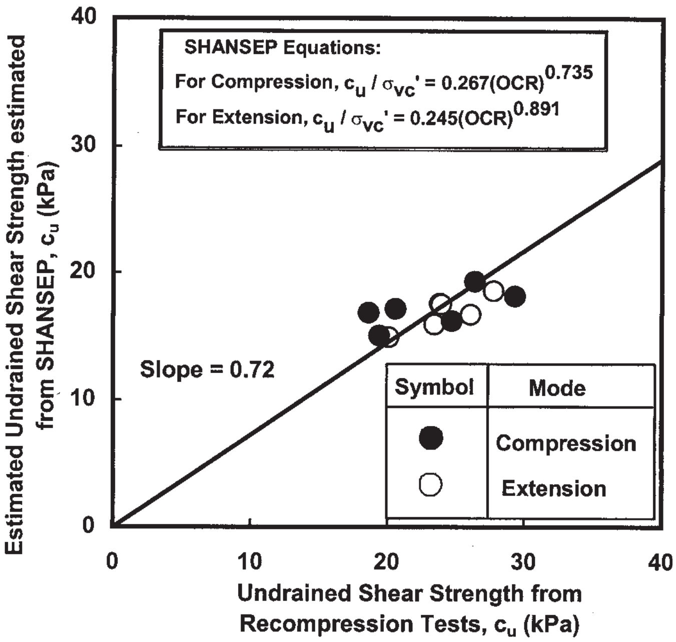

FIG. 18—•••

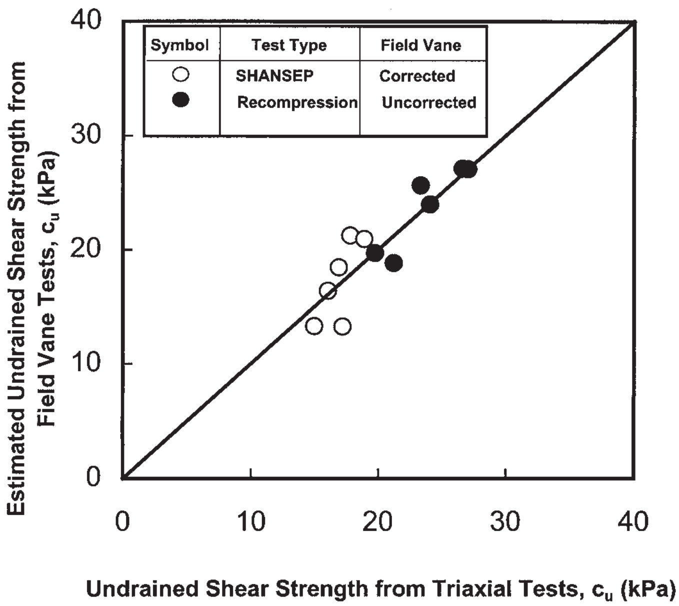

FIG. 19—•••

dicating that the SHANSEP tests underpredict the undrained shear strength by about 28 %. The triaxial test results are also compared with field vane tests performed at the same site, with data presented in Fig. 19. Since the physical properties of the soft clay are known, the μ correction factor for the field vane strength is also estimated. The results shown in Fig. 19 indicated that the Recompression triaxial strength matches well with uncorrected field vane test data, whereas the SHANSEP triaxial strength agrees well with corrected field vane strength.

Profiles of undrained shear strength and undrained modulus from SHANSEP and Recompression techniques are compared, as shown in Fig. 20.

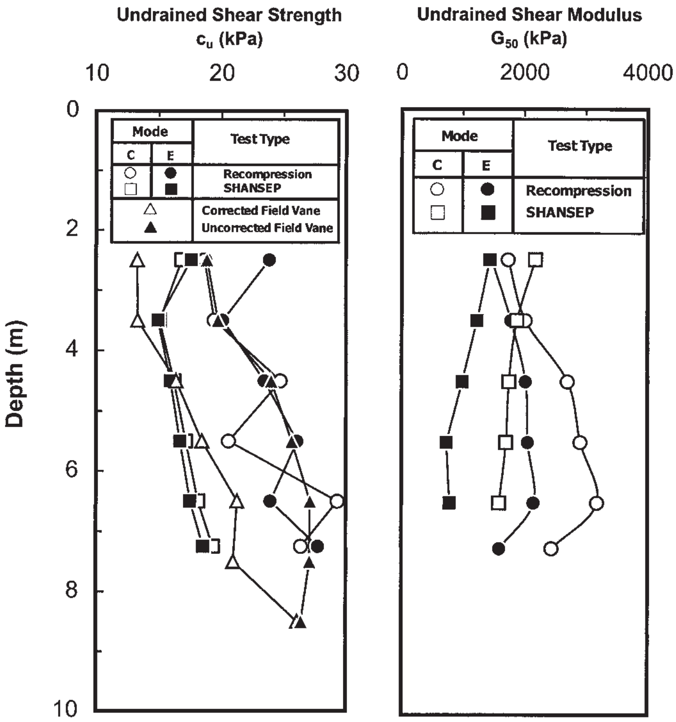

FIG. 20—•••

Conclusions

The undrained shear behavior of soft Bangkok clay was investigated by using the triaxial apparatus with emphasis on different reconsolidation techniques. The consolidation technique adopted gives compression curves and preconsolidation pressures similar to those from oedometer tests, indicating good testing procedure as recommended by Germaine and Ladd (1988). The SHANSEP technique provides a consistent K_o-OCR relationship, as compared with other published data. As for the Recompression tests, the magnitude of the reconsolidation volumetric strain is relatively small, having value of 1–2 %. This implies that the sampling is of reasonable quality, and the strength obtained from the test should be representative of the in situ value.

Based on the results of SHANSEP tests, the anisotropic strength ratio (K_s) for normally consolidated Bangkok clay is found to be 0.92 and increases with increasing OCR, implying that this clay is fairly isotropic in terms of strength. As for shear modulus, the value in compression mode is about 50 % higher than in extension mode.

The results also indicated that the undrained shear strength obtained from the Recompression test is about 28 % higher than the value from SHANSEP test. This difference in the c_u value could be due to the destructuring of clay in the SHANSEP with consolidation strain up to 17 %. This effect also decreases the stiffness of the clay; hence, the shear modulus obtained from the Recompression test is found to be 2–3 times higher that obtained from the SHANSEP test. The results from the triaxial tests are compared with the field vane tests, indicating that the uncorrected field vane strength agrees well with the Recompression triaxial strength and the corrected field vane strength matches well with the SHANSEP triaxial data.

Acknowledgements

The authors acknowledge technical assistance and review provided by Professor Balasubramaniam and Professor Honjo. They are indebted to Mr. Tangthansup Burut for preparing the final manuscript.

References

Berre, T. and Bjerrum, L., 1973, *Proceedings of the Eighth International Conference of Soil Mechanics and Foundation Engineering*, Moscow, Vol.1, Part 1, pp. 39–49.

Bjerrum, L., 1973, “Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils,” *Proceedings, Eighth International Conference of Soil Mechanics and Foundation Engineering*, Moscow, Vol. 3, pp. 111–159.

Germaine, J. T. and Ladd, C.C., 1988, “Triaxial Testing of Saturated Cohesive Soils,” ASTM STP 977, pp. 421–459.

Hassan, S. I., 1993, “Consolidation Behaviour of Bangkok Clay using a Constant Rate of Strain Consolidometer,” Master Thesis Number GT-92-13, Asian Institute of Technology, Bangkok, Thailand.

La Rochelle, P., Sarrailh, J., Tavenas, F., Roy, M., and Leroueil, S., 1981, “Causes of Sampling Disturbance and Design of a New Sampler for Sensitive Soils,” *Canadian Geotechnical Journal*, Vol. 18, No. 1, pp. 52–66.

Lacasse, S. and Berre, T., 1988, “Triaxial Testing Methods for Soils,” *Advanced Triaxial Testing of Soil and Rock, ASTM STP 977*, Robert T. Donaghe, Ronald C. Chaney, and Marshall L. Silver, Eds., ASTM International, West Conshohocken, PA, pp. 264–289.

Ladd, C. C. and Edgers, L., 1972, “Consolidated-Undrained Direct Simple Shear Tests on Saturated Clays,” MIT Research Report R72-82, Number 284, Department of Civil Engineering, MIT, Cambridge, MA.

Ladd, C. C. and Foott, R., 1974, “New Design Procedure for Stability of Soft Clays,” *Journal of the Geotechnical Engineering Division*, Vol. 100, No. GT 7, pp. 763–786.

Ladd, C. C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H. G., 1977, “Stress Deformation and Strength Characteristic SOA Report,” *Proceedings of the Ninth International Conference of Soil Mechanics and Foundation Engineering*, Tokyo, Vol. 2, pp. 421–494.

Mayne, P. W. and Kulhawy, F. H., 1982, “Ko-OCR Relationships in Soil,” *Journal of Geotechnical Engineering*, Vol. 108, No. GT6, pp. 851–872.

Moh, Z. C., Nelson, J. D., and Brand, E. W., 1969, “Strength and Deformation Behavior of Bangkok Clay,” *Proceedings of the Seventh International Conference of Soil Mechanics and Foundation Engineering*, Mexico, Vol. 1, pp. 287–296.