

NEW DESIGN PROCEDURE FOR
STABILITY OF SOFT CLAYS

by

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INTRODUCTION

Research during the last decade has clearly demonstrated that the undrained shear behavior of soft clay is much more complex than was previously thought. It has shown that the present design practice widely used to determine the stability of clay foundations is highly empirical and of indeterminate accuracy. Because of this empiricism, it is very difficult to incorporate the results of the research into the present design process. Accordingly a new method for evaluating the undrained strength of clay foundations is presented. The method better utilizes the present understanding of clay behavior and allows a more thorough and reliable evaluation of the important design parameters.

PRESENT DESIGN PRACTICE

With the exception of very stiff, highly overconsolidated clay deposits, the $\phi = 0$ method of analysis is used to investigate the stability of saturated clay foundations. With drainage the foundation becomes stronger and so this analysis considers the most critical condition of no drainage. In the case of very stiff deposits, the foundation strength can be reduced by drainage and so long term, drained, effective stress analyses should be performed.

The theoretical basis for the $\phi = 0$ method of analysis was presented by Skempton (1948). It is assumed that no drainage occurs during the loading period and that the undrained strength (s_u) of a clay remains constant irrespective of the applied stresses. Thus the usual limiting equilibrium methods of stability analysis are used with $\phi = 0$ and $c = s_u$, where c is the cohesion intercept and ϕ is the friction angle. At that time the value of s_u for a soil was considered to be more-or-less a unique function of its water content. Therefore it could be accurately

determined by any shear test which was performed at the situ water content. Bishop and Bjerrum (1960) analyzed and reviewed numerous end-of-construction failures involving foundation clays. They concluded that s_u could be reliably determined from field vane (FV) tests or from the widely used laboratory unconsolidated - undrained triaxial compression or unconfined compression tests (designated UU and U respectively).

Most of the present-day practice in the U.S. uses the $\phi = 0$ method of analysis in conjunction with s_u data from FV, UU and/or U tests. Usually this results in a safe design. However, during the last decade much basic research has been performed which collectively allows a more thorough appraisal of the undrained strength characteristics of clay soils.

RECENT RESEARCH

Four of the most important results of recent research into the undrained strength and deformation characteristics of clay soils are the findings relating to sample disturbance, strength and stress-strain anisotropy, strain-rate effects and normalized behavior. These findings are described briefly below and in more detail in Ladd (1971).

Sample Disturbance

This represents the inevitable disturbance of the soil structure (Lambe, 1958) during the sampling process. It can be minimized by good sampling techniques but not totally prevented. Sample disturbance has been discussed in detail by Ladd and Lambe (1963), Skempton and Sowa (1963), Seed et al. (1964), Davis and Poulos (1967), ASTM (1970) and IGOSS (1971).

A major source of sample disturbance is the stress relief involved in taking a sample from deep in the ground. Since swelling is prevented, negative pore pressures are developed in the sample. Ladd and Lambe (1963) suggested an evaluation of the degree of sample disturbance by comparing the negative pore pressure in the sample with that which would occur in a

perfectly undisturbed sample. For tube samples taken from depths of more than several meters, they found the measured values to be typically $20 \pm 20\%$ of the "perfect sample" values. Thus sample disturbance causes a major reduction in the effective stress in the sample compared to that in situ. This in turn results in a decrease in s_u , the reduction typically ranging from 20 to 50% of the "perfect sample" strength.

Strength and Stress-Strain Anisotropy

The assumption that s_u is a unique function of water content has been disproved by the measurement of significant strength anisotropy in clay deposits. Strength anisotropy was predicted from theoretical considerations by Hansen and Gibson in 1948. However, it was the result of attempts to relate s_u values from different types of shear tests and the development of sophisticated testing equipment that lead to a realization of its practical significance. The majority of this work was performed in the 1960's with the development of plane strain devices (e.g. Duncan and Seed, 1966; Henkel and Wade, 1966), the direct-simple shear device⁽³⁾ (Bjerrum and Landva, 1966), the Cambridge simple shear apparatus (Roscoe, 1970) and various types of "true" triaxial devices (e.g. Shibata and Karube, 1965; Hambly, 1969).

Undrained strength anisotropy can be divided into two major components. The first, called inherent anisotropy, is the result of major differences in soil structure per se which occurred during formation of

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A cylindrical sample with a nominal height of two cm and an area of 50 cm^2 is enclosed in a wire reinforced rubber membrane which prevents lateral deformation during consolidation. The sample is sheared by moving the top cap laterally while keeping the sample height (and thus volume) constant by adjusting the vertical load.

the soil. Varved clays, which have alternate layers of "silt" and "clay", have a high degree of inherent anisotropy. The second component is a stress induced anisotropy that results from rotation of principal stresses during shear and variations in the intermediate principal stress. For practical purposes the effect of these components can be considered collectively by modelling the stress system and sample orientation that will exist in situ. This is demonstrated in Fig. 1 where the stress systems which would apply along typical failure surfaces are shown for several practical situations. The direction of the major principal stress at failure (σ_{1f}) is also shown.

For homogeneous, non-layered clays sheared under conditions such as those shown in Fig. 1, it is typically found that s_u from plane strain active (PSA) tests is greater than the value from direct-simple shear (DSS) tests, which in turn is greater than the plane strain passive (PSP) value. Triaxial compression (TC) strengths are generally very close to PSA values, while limited data suggest that triaxial extension (TE) strengths will be 10 to 25% lower than those from PSP tests.

Table 1 presents the results obtained from these various strength tests on normally consolidated Boston Blue Clay, a fairly sensitive marine illitic clay. The undrained strengths are all from K_0 (coefficient of lateral earth pressure at rest) consolidated, undrained tests with pore pressure measurements ($\overline{CK_0U}$), and they are presented as a ratio of the vertical consolidation stress ($\overline{\sigma_{vc}}$). A significant decrease in s_u can be seen for the three tests involving rotation of principal planes (DSS, PSP, TE). This is accompanied by an increase in the strain at failure. These effects are more significant with lean sensitive clays than with highly plastic clays of low sensitivity, but a factor of 1.5 to 2.5 in the range of s_u values obtained from the various tests is not uncommon.

Strain-Rate Effects

Work by Casagrande and Wilson (1951), Taylor (1955), Bjerrum et al. (1958), Crawford (1959), Richardson and Whitman (1963), Bjerrum (1971) and others has shown the strength obtained from laboratory tests to vary with the strain rate used. For TC tests, each log cycle decrease in strain rate is typically accompanied by a $10 \pm 5\%$ decrease in s_u , the exact variation being a function of the plasticity and creep susceptibility of the soil. The effect is due to undrained creep which occurs during shear in the sample, giving increased pore pressures, decreased effective stresses and decreased strengths. The slower the strain rate, the more time there is for this creep to occur and the lower the s_u obtained. This can be a significant practical consideration. For highly plastic, creep susceptible clays, the TC strength obtained from consolidated samples failed at an axial strain rate of 60%/hr (typical for U and UU tests) can be 1.2 to 1.3 times the strength obtained at 0.5%/hr (typical for consolidated tests with pore pressure measurements).

Bjerrum (1972) presented a design method based on FV tests. In this the measured FV strength data are "corrected" using factors obtained from a comparison of measured strength values with those required to give a factor of safety of 1.0 for observed failures. It is assumed that the discrepancy in measured values is primarily due to strain rate effects.

Normalized Behavior

In work at Imperial College using remolded clays (Henkel, 1960; Parry, 1960) and work at M.I.T. on a wide range of clays, it has been observed that the results of laboratory tests on clay samples with the same overconsolidation ratio (OCR), but different consolidation stresses and therefore different maximum past pressures ($\bar{\sigma}_{vm}$), exhibit very similar strength and stress-strain

characteristics when normalized with respect to the consolidation stress. This is demonstrated by the plot in Fig. 2 which shows idealized stress-strain curves for isotropically consolidated TC tests on a normally consolidated clay with consolidation stresses ($\bar{\sigma}_c$) of 200 and 400 kN/m². When replotted using $(\sigma_1 - \sigma_3)/\bar{\sigma}_c$, the two curves plot on the same line. This normalized plot can then be used to represent the behavior of other normally consolidated samples with different consolidation stresses, when sheared in the same type of test. Similar normalized behavior is observed in pore pressure data from laboratory tests.

In practice, normalized behavior is not as perfect as that shown in Fig. 2. There is usually some divergence in the normalized plots obtained for different consolidation stresses and also some due to heterogeneity in the soil deposit. The inevitable minor variations in procedure from one test to another can also cause divergence. Figure 3 indicates, by means of three \overline{CK}_0U DSS tests on normally consolidated Maine organic clay, the divergence associated with variation in natural water content and consolidation stress. Figure 4 shows the range of normalized curves obtained from three identical, isotropically consolidated, TC tests on normally consolidated Atchafalaya backswamp clay. The variation in the results of these three tests is indicative of sample heterogeneity.

The normalized method of plotting data has been used extensively at M.I.T. during the past ten years. Normalized plots of data from cohesive soils representing a wide range of properties have shown similar or only slightly more divergence than that shown in Fig. 3 and 4. These divergences are quite small and the normalized strengths, for example, are generally found to be within 10% of the mean. Similarly, only small divergences have been found in normalized plots of overconsolidated samples with the

same OCR. For practical purposes, therefore, normalized behavior has been found to apply to a wide range of cohesive soils.

It should be noted that tests on quick clays and on naturally cemented clays which have a high degree of structure, will not exhibit normalized behavior. This is because the structure is significantly altered during consolidation to higher stresses.

DISCUSSION OF THE PRESENT DESIGN PRACTICE AND THE NEED FOR A NEW APPROACH

The results of recent research detailed above have shown that major variations in strength can be caused by sample disturbance, strength anisotropy and strain rate effects. None of these effects is explicitly included in the present design practice. The reason that the present method generally works is that the variations frequently tend to be self compensating. The commonly used U and UU strength data tend to give high values of s_u due to the TC stress system and the high strain-rate used (60%/hour). Sample disturbance, on the other hand, tends to give low values of s_u provided that drying of the sample is avoided. These effects may compensate each other and yield a reasonable average design strength. However, the method is highly empirical and these compensating factors are in no way controlled or controllable. It is therefore quite possible for the resulting design to be either unsafe or highly overconservative, particularly in view of the large scatter often found in U and UU data. The situation is further confused by the tendency for sample disturbance effects to increase with depth and to obscure strength variations in the profile.

The use of FV data can avoid some of the difficulties associated with the use of U and UU data, for the test is much less affected by

sample disturbance. However, interpretation of the test is highly uncertain and its use is very empirical. Also, agreement between FV and U and UU data is very poor for some soils. Bjerrum (1972) presented correction factors for use with FV data. However results presented later in this paper will show that FV data can overestimate the in situ strength by up to 100%, which is considerably more than indicated by these reduction factors. Therefore the reliable use of FV data is limited to deposits for which the appropriate correction factor has been empirically selected from case studies of failures. For such deposits, it is a very valuable technique.

Thus the present design practice is highly empirical and although the use of safety factors generally gives a safe design, the true areas of certainty and doubt in the analyses are obscured to the point where all that can be said is that collectively the method usually works.

Occasionally the combination of compensating inaccuracies and safety factor are not sufficient to give an adequate design. In these situations a more sophisticated procedure is essential, one which includes the detailed characteristics of the soil within the framework of a more rational approach to design. Such a method should be equally valuable for use with more straight forward designs. By avoiding much of the gross empiricism of the present method, the engineer will have a considerably greater control over his design, which should allow him to avoid overconservatism. This area of soil mechanics is one in which research has moved far ahead of practice. A new design method is needed to bring practice up to date.

The design procedure described in the paper was developed in the mid 1960's from the Normalized Soil Parameter (NSP) Concept. It has since

been used successfully on a variety of different soils and practical techniques for the application of the method have been developed.

THE NORMALIZED SOIL PARAMETER (NSP) CONCEPT

The NSP concept derives directly from the observation of normalized behavior discussed above. For a soil which exhibits normalized behavior, it is possible to run laboratory tests at various OCRs and develop normalized plots for each OCR. NSPs can then be obtained from these plots and applied to a wide range of in situ stress conditions.

The most frequently used NSP and the one used for stability analyses is $s_u/\bar{\sigma}_{vo}$, where $\bar{\sigma}_{vo}$ is the in situ vertical effective stress. This is equivalent to the "c/p" ratio so often quoted in the soil mechanics literature. It is modelled as $s_u/\bar{\sigma}_{vc}$ in laboratory tests. Figure 5 shows the variation in $s_u/\bar{\sigma}_{vc}$ with OCR from \overline{CK}_o UDSS tests on five cohesive soils. As can be seen the trend with increasing OCR is very similar for each soil. The index properties of the five soils cover a wide range of values and it is considered that \overline{CK}_o UDSS data for other clay deposits would probably follow the same pattern. These same trends are observed in the results of other types of shear test on the soils, but the values of $s_u/\bar{\sigma}_{vc}$ are changed due to the strength anisotropy of the soils. Other NSPs used in soil mechanics are E_u/s_u , K_o and pore pressure parameters.

A large variation in $s_u/\bar{\sigma}_{vc}$ with change in OCR can be seen in Fig.5. Therefore in evaluating the normalized strength, it is extremely important to know the precise OCR of the sample. This calls for special laboratory testing techniques as discussed below.

LABORATORY TESTING TECHNIQUES

The main laboratory testing technique developed for use with the NSP Concept involves consolidation to stresses in excess of those in situ in

order to overcome sample disturbance effects and give a sample of known OCR. This is used in conjunction with the NSP Concept to yield NSP values.

The effect of sample disturbance is shown in the idealized void ratio vs. log effective stress plot of Fig. 6. The virgin compression relationship shown is typically a unique relationship for a specific clay, time of consolidation, and type of consolidation stress system. If a sample becomes overconsolidated, its effective stress is reduced and it swells, typically following a relationship such as line a in the figure. With reconsolidation the relationship will follow line b back to the virgin compression line. Since the changes in void ratio associated with soil swelling are much smaller than those associated with virgin compression, overconsolidated soils will always plot below the virgin compression line. Now an "undisturbed" sample will typically suffer a decrease in effective stress during sampling even though the water content may be kept virtually constant. Thus an in situ normally consolidated sample at point 1 in Fig. 6 might plot at point 2 after sampling and be similar to an overconsolidated sample. With reconsolidation it will follow some path back to the virgin compression line such as the one shown. It follows that a test performed at conditions corresponding to any point on this line prior to its reaching the virgin compression line has an uncertain OCR. Therefore meaningful NSPs cannot be obtained from it. A sample which has been consolidated back to the virgin compression line, however, has a clearly known OCR of one. This sample will give NSPs which, assuming the concept holds for the soil, are equally applicable to all normally consolidated samples. If NSPs for overconsolidated samples are required, these can be obtained at known OCRs

by consolidating the sample back to the virgin compression line and then reducing the effective stress to give the required OCR. This is shown in Fig. 6 as consolidation from point 2 to point 3, followed by unloading to point 4 to give a sample of known OCR.

Thus the testing procedure developed to yield NSPs requires that the samples be consolidated back to the virgin compression line before testing. Consolidation to $\bar{\sigma}_{vc}$ levels greater than 1.5 to 2 times the in situ $\bar{\sigma}_{vm}$ is usually required. To standardize the effects of secondary compression, the last consolidation increment should generally be left on for about one log cycle of secondary compression.

It follows that use of this procedure requires a knowledge of the in situ stresses and $\bar{\sigma}_{vm}$ values, and high quality oedometer tests are essential. It has been found that plotting the test results as strain (rather than void ratio) vs $\log \bar{\sigma}_{vc}$ at the end of primary consolidation, instead of at the end of the 24-hour standard load increment period, yields more reliable values of $\bar{\sigma}_{vm}$. Good undisturbed samples are, of course, a major requirement. Fortunately, however, oedometer tests do not seem to be so highly sensitive to sample disturbance as are U and UU tests.

SHANSEP - THE METHOD OF DESIGN

SHANSEP stands for Stress History and Normalized Soil Engineering Properties. It is the basis of the new method of design. It consists of evaluating the stress history of the clay deposit by evaluating the $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$ profiles to determine the OCR variation through the deposit, and then applying the appropriate NSPs to give the representation of the foundation properties required for design. The basic steps are as follows:

- (1) Examine and subdivide the soil profile into component deposits on the basis of boring logs, FV data, visual classifications, etc.
- (2) Obtain good "undisturbed" samples and investigate the stress history of the soil profile using a program of total unit weight, $\bar{\sigma}_{vm}$ and pore pressure measurements.
- (3) Decide which shear strength tests best model the situation under consideration and the range of OCRs for which data are required (from 2)
- (4) Perform the tests selected in 3. First reconsolidate back to the virgin compression line and then reduce the stresses to give the required OCR as described above. Obtain the required NSPs from these tests.
- (5) Apply these NSPs to the soil profile data from 1 and 2 to give the distribution of strength through the foundation.

Steps 1 and 2 apply standard techniques widely used in practice, although a more thorough investigation than customarily performed might be necessary. Also, as noted above, high quality oedometer tests are required. Step 3 depends on the type of analysis to be made. The procedure which should be employed in this step consists of examining the design being performed and selecting the laboratory tests most appropriate to that design. Thus for a design where a PSA failure is likely to occur (see Fig. 1) normalized strength data from BSA tests or the similar values from TC tests should be used in the stability analyses. Similarly, for a design involving a passive loading of the clay, strength data from PSP or TE tests should be used. For circular arc or sliding wedge analyses involving the full range of stress conditions from active to passive (see Fig. 1), the average normalized strengths from PSA and PSP tests or TC and

TE tests can generally be used for non-layered clays. ⁽⁴⁾ Alternatively, research at M.I.T. and Bjerrum (1972) have shown the values from DSS tests to be similar to or slightly lower than these average values. Thus use of DSS data should give good to somewhat conservative strength values for use in these analyses. In lieu of DSS tests, constant volume direct (box) shear tests could be used (see Taylor, 1952; O'Neill, 1962). All of these tests should be performed using the $\overline{CK}_O U$ procedure. Recommended strain rates are 0.5 to 1% axial strain/hr. for triaxial tests and 5% shear strain/hr. for DSS and constant volume direct shear tests.

Step 4 consists of performing the tests selected in step 3 utilizing the testing procedures described above. Finally step 5 is the application of the resulting strength NSPs to the evaluated soil profile of step 2 to give a model of the soil strength profile which can be analyzed. This is a very simple process. From step 2 the values of $\overline{\sigma}_{vo}$ and $\overline{\sigma}_{vm}$ are known throughout the profile. These allow calculation of OCR at various elevations. The corresponding $s_u / \overline{\sigma}_{vc}$ can then be obtained from curves such as those in Fig. 5 and applied to the $\overline{\sigma}_{vo}$ value to give the appropriate value of s_u . Table 2 shows typical calculations of s_u for a soil profile using the DSS strength parameters for Maine Organic Clay.

Having obtained a strength model of the clay deposit, the presently available methods of analysis are used to select a stable design.

(4)

See Davis and Christian (1971) for a discussion of the effects of the variation in s_u with the direction of σ_{1f} on the undrained bearing capacity of anisotropic soils.

CASE STUDIES INVOLVING THE USE OF SHANSEP

The SHANSEP method has been used successfully for designs involving a variety of different soils over the past six years. These cases have provided the practical applications and the opportunity to check the method that is essential for the development of virtually all new methods of design in soil mechanics, as well as providing a comparison with the present day practice. The results of four of these case studies are summarized below.

Embankment on Sensitive Marine Clay

This case study involved the rapid construction to failure of a long 20 ft (6.1m) high sand-fill test embankment to provide data for the design of approach embankments for a new interchange at I-95 in Portsmouth, New Hampshire. It is described in detail by Ladd (1972). The foundation conditions are shown in Fig. 7. Beneath a weathered drying crust of several feet, the marine illitic clay, which had been leached, had a natural water content (w_n) of $50 \pm 5\%$, liquid limit (w_l) of $35 \pm 5\%$ and plastic limit (w_p) of $20 \pm 2\%$. Figure 7 shows the $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$ stress profiles, unconfined compression data, and the strength profiles obtained from FV tests⁽⁵⁾ and from SHANSEP using \overline{CK}_O UDSS data. The \overline{CK}_O UDSS test results equalled the average of the \overline{CK}_O UPSA and PSP values. The SHANSEP strength profile is presented as a band of values corresponding to the range of $\bar{\sigma}_{vm}$ data.

This case illustrates the significant change in $\bar{\sigma}_{vm}$ values which is obtained with some soils by plotting the oedometer data at the end of

(5)

Three Geonor field₂ vane holes yielded values of s_u within 10 to 40 psf (0.48 to 1.92 kN/m²) of the average line shown in the figure.

primary consolidation instead of at the end of the load increment period. The good agreement between SHANSEP and FV data over much of the deposit should also be noted, along with the large scatter in U data and their general tendency to be considerably lower than the SHANSEP and FV values.

The failure section of the embankment was analyzed using the $\phi = 0$ circular arc method. The resulting critical arcs showed excellent agreement with the measured failure surface. The factors of safety equalled 0.83 to 1.08 for the band of SHANSEP strengths, with a value close to 1.0 being obtained for the average strengths. The factor of safety obtained with FV strengths was 0.88 and that for U data was in the region of 0.7.

As a result of this test section, the I-95 embankments were designed using jetted sand drains and stage construction on the basis of \overline{CK}_0 UDSS data using SHANSEP. They performed well (Ladd, Rixner and Gifford, 1972). Had present practice been used, a conservative design would have been obtained, especially based on the average unconfined strengths.

MIT-MDPW Embankment Test Section on Boston Blue Clay

This study also involved I-95 embankments, this time a few miles north of Boston. A 40 ft (12.2m) high test section was constructed for the Massachusetts Department of Public Works (MDPW), but not to failure. The project is described by D'Appolonia, Lambe and Poulos (1971), from which paper Fig. 8 has been taken to show the soil profile, stress history and s_u data. The $\overline{\sigma}_{vm}$ data were obtained from oedometer test results plotted at the end of each consolidation increment, but the values obtained by plotting these results at the end of primary consolidation show little change with this soil.

Figure 8 shows U, UU, FV and SHANSEP strength data. Two SHANSEP strength profiles are shown, one based on \overline{CK}_0 UPSA data and the other

based on \overline{CK}_O UPSP data. The FV data are seen to be similar to the PSP strength while the U and UU data are considerably lower.

Stability analyses using the $\phi = 0$ circular arc method yielded a minimum factor of safety of 0.73 for the average of the U and UU strength data, 1.18 for the average FV data, and 1.50 for the average of \overline{CK}_O U PSA and PSP data. The test section behaved well, and so its actual factor of safety is not known. However, the small deformations and overall good performance strongly suggest that the SHANSEP factor of safety is the most suitable.

The extent of local yield beneath the embankment was evaluated using pore pressure observations and the procedure of detecting a break in the pore pressure - embankment height relationship suggested by Hoeg. et al (1969). With a knowledge of the yielded area, an estimate of the in situ strength can be made. A significant area of local yield beneath the embankment centerline was detected and very good agreement was found between the corresponding in situ strength and that calculated using SHANSEP with \overline{CK}_O UPSA strengths. This is appropriate because PSA stress conditions should apply beneath the centerline.

An overconservative design would have resulted from the use of present design practice for this embankment.

Embankment Test Section on Organic Clay

This experimental test section was built in a tidal mud flat area of Fore River in Portland, Maine, to provide data for the design of I-295 embankments. It is described in detail by Ladd, Aldrich and Johnson (1969).

Figure 9 shows the typical soil profile at the site along with stress history and strength data. The $\overline{\sigma}_{vm}$ values were obtained by plotting

oedometer test data at the end of primary consolidation. The average strength of the deposit was fairly uniform at all elevations due to the almost constant $\bar{\sigma}_{vm}$ with depth, but considerable scatter in the FV data was observed. This was probably due to the inclusion of shells, organic matter and sand lenses in the deposit, although it is also indicative of the heterogeneity of the soil. Scatter is also noticeable in the U data but it is not so extreme. The FV data are seen to be very much higher than the SHANSEP strength data based on \overline{CK}_o UDSS tests, whereas the U data agree with the SHANSEP profile.

After an unexpected failure of the test section, $\phi = 0$ circular arc analyses were used to calculate an average in situ s_u of 255 ± 20 psf (12.21 ± 0.96 kN/m²). The average strengths from the data of Fig. 9 were 525 psf (25.15 kN/m²) for the field vane, 210 psf (10.06 kN/m²) for U tests and 210 psf (10.06 kN/m²) for the SHANSEP profile. UU tests run on samples taken after the failure yielded an average s_u of 280 psf (13.41 kN/m²). Thus the FV data were much too high, UU strengths were slightly too high, and the U and SHANSEP strengths were slightly too low.

The embankment failure provided a calibration factor for the field vane of $255/525 = 0.485$. Several miles of embankment were designed using average field vane strengths times 0.485. Strength increases during stage construction were then computed using the SHANSEP approach. Stress-strain data from \overline{CK}_o UDSS tests were also used to predict lateral deformation during construction. The embankments performed well.

Levee Test Sections on Atchafalaya Backswamp Deposits

In 1964/65 the U.S. Army Corps of Engineers built three levee test sections on highly plastic, creep susceptible backswamp deposits in the Atchafalaya Basin, Louisiana. The performance of these test section is described by Kaufman and Weaver (1967) and the behavior during construction of the two main test sections, numbers II and III, has been extensively

studied at M.I.T. using SHANSEP (Foott and Ladd, 1973). Figure 10 (a) shows Test Section III and the original foundation cross section. Test Section II had the same foundation and was very similar to Test Section III, but with slightly smaller berms.

These test sections were built over an existing levee and adjacent to an existing excavated waterway. Therefore, unlike the previous case studies, the soil profile was not constant throughout the site, having been affected by previous construction. The site was extensively investigated by means of a grid of borings at the levee centerline and at 105 and 180 ft (32.0 and 54.9m) offsets to each side of the centerline. U, UU and FV strength determinations were made at each of these offsets. In addition, the stress history at each offset was evaluated and a SHANSEP strength profile calculated using \overline{CK}_0 UDSS data. Figure 10(b) shows a comparison of the resulting strength profiles from three of these offsets. As can be seen, both the average of U and UU data and the average FV data tend to be higher than the SHANSEP profile. However, it should be noted that although the FV profiles have much higher values than the SHANSEP profiles, they exhibit very similar variations with depth. This is not the case with the U and UU data, demonstrating the tendency with these tests for sample disturbance to obscure strength variations in the profile. However, with FV tests the disturbance is less and the variations are still apparent.

Circular arc $\phi = 0$ analyses of Test Section III gave approximate factors of safety for the floodwayside slope of 1.1 for the SHANSEP strength profiles, 1.4 for average U and UU strength values and 2.0 for the FV average values. After application of Bjerrum's (1972) correction factor, the FV factor of safety became 1.4. The observed

behavior of the test section was marked by excessive lateral deformations and resultant crest settlement, and the SHANSEP factor of safety is considered the most appropriate.

Circular arc $\phi = 0$ analyses were also made for Test Section II, but only for the SHANSEP and FV strength profiles. The approximate floodway-side factors of safety were 1.0 for the SHANSEP strengths and 1.8 for the FV strengths, becoming 1.3 with application of Bjerrum's correction factor. A few months after the end of construction, the floodwayside of this test section was considered to be on the verge of failure as indicated by excessive lateral deformations and the presence of cracks in the embankment crest. Therefore, the factor of safety from the SHANSEP profile again seems most suitable.

For this case, design using SHANSEP strengths obtained from \overline{CK}_0 UDSS tests would seem to give good to slightly conservative results. The use of average U and UU data appears to overestimate the in situ strength, as does the use of FV data. Application of Bjerrum's (1972) correction factor still leads to overestimation of the in situ strength. However, the use of a correction factor of $0.50 \pm .05$ with the FV data, would have given reasonable agreement with the SHANSEP results. It is possible that such a value could be empirically developed for use with these soil deposits.

SHANSEP IN PRACTICE

The case studies described above and summarized in Table 3 demonstrate many of the soil behavior and testing procedure factors discussed earlier. In all cases the SHANSEP approach provided a good or slightly conservative indication of stability, whereas the present practice was seen to be erratic in its prediction of stability and frequently would have

led to inadequate designs of either an unsafe or overconservative nature. The SHANSEP approach was initially applied in a research context, but confidence in the method developed as the number of successful applications increased. It is now considered to be sufficiently well proven to be used in general practice for most situations involving embankment or foundation loadings of a clay deposit. Except when an active or passive failure will clearly be critical, the procedure detailed previously of using DSS, the average of TC and TE or PSA and PSP, or direct (box) shear, strength parameters is recommended. The resulting strengths should be used in conjunction with the present $\phi = 0$ methods of analysis.

The strength parameters obtained from the testing program should be compared to parameters obtained for other cohesive soils, such as those shown in Fig. 5. In view of the regularity of the trends shown in this figure, no more than five or six good \overline{CK}_0 UDSS tests should be required to confidently define a similar curve for a new soil exhibiting the same trend.

In addition, it is recommended that a comprehensive series of FV tests be included in the program. These will indicate pockets of weak material which the SHANSEP method would probably not detect. The resulting data can also be calibrated against the SHANSEP strength profile. As indicated in the Portland, Maine and Atchafalaya case studies, this can lead to the empirical development of a correction factor for the deposit, which can be applied to FV data and the resulting strengths used for design.

The SHANSEP method does not work well near the top of a highly desiccated "drying crust" (see Fig. 7) due to weathering and the difficulty involved in determining the OCR in that region. The use of FV data

corrected on the basis of correlation between SHANSEP and FV values from lower in the profile is recommended in such situations. Alternatively, use of U and UU data (which are often not too seriously affected by sample disturbance near the surface) or Bjerrum's (1972) FV correction factors, may be adequate, depending on the importance of the crust to the overall stability.

A good oedometer testing program is necessary to reliably indicate the stress history of the profile. This might involve an increase in the number of tests compared to the present practice, but with good techniques and a regular deposit, this should not be excessive. A knowledge of the geology of the site can be very useful in planning this program and in interpreting the results.

DISCUSSION

The major assumption of SHANSEP is that the NSP Concept can be applied to the soil. This assumption can be checked fairly easily in the laboratory. Beyond that the method includes the recent developments in understanding clay behavior and uses a series of logical and defined steps to give a clear indication of the soil profile and a framework for analysis. Assumptions may well be required as part of these steps, but they are explicitly made and the engineer can assess their probable impact on the design. The method yields far more data than present practice and can readily be used to give information in addition to the basic strength profiles (e.g. moduli variations for use in finite element analyses, D'Appolonia, Poulos and Ladd, 1971).

SHANSEP can be used at two levels. It gives a good conceptual picture of the soil profile by providing a correlation between stress history and strength variations with depth. When the present design practice is used, SHANSEP should be employed at this conceptual level to examine the

reasonableness of the $s_u/\bar{\sigma}_{vo}$ values obtained in the light of published data and the likely stress history. In addition, it is a reasonably well developed practical method for use in design.

A major advantage of SHANSEP is that the NSF component of the method will progressively produce more and more data. These can be used in plots such as Fig. 5 to provide a check on the parameters obtained for new soils and increasingly reduce the amount of testing required. The method can be expected to reinforce itself with use.

There are, however, disadvantages and problems with the use of SHANSEP. A major limitation is that the method can only be applied to fairly regular deposits for which a well defined stress history can be obtained. Clays often do tend to occur in quite regular deposits, but if a random deposit is encountered, the method is useless. In the application of SHANSEP, difficulties may be encountered in determining the stress history and the regularity of the deposits, particularly if high quality samples cannot be obtained. The method is totally dependent on a good knowledge of the stress history, and high quality $\bar{\sigma}_{vm}$ data are essential. Also, the laboratory testing in general requires more sophisticated techniques than are commonly used in practice.

Although the method has been used successfully on several jobs, it is relatively new and complex and so should be used with care. Movement away from highly empirical methods of design to more theoretically sound and comprehensive methods should be undertaken cautiously. Also every aspect of soil behavior is not yet understood and the highly empirical methods may, in certain cases, be compensating for factors of at present unrecognized importance.

It should also be noted that some methods of analysis are themselves highly empirical and have been developed for use with certain standard

types of design (e.g. the relationship for design lateral stresses on braced excavations, Article 48 of Terzaghi and Peck, 1967). In these cases the use of the test is a part of the method of analysis. Therefore the method of analysis itself must be re-evaluated if different soil parameter values are used.

One of the most contentious aspects of the SHANSEP method is the procedure of consolidating samples well beyond the in situ stresses. The possibility exists that this will destroy some important aspect of soil structure that has developed during and after formation of the clay deposit. As noted earlier, this is clearly true with highly structured "quick" clays and with naturally cemented deposits. With other soils, however, no evidence of a special structure has been found. Also, normalized strength parameters obtained from UU strength data corrected for sample disturbance using the procedure of Ladd and Lambe (1963) agree well with strengths obtained from samples consolidated well beyond the in situ stresses. In addition, the practical applications of SHANSEP have shown the procedure to be acceptably accurate. Therefore it seems that the recommended procedure of reconsolidating samples with one log cycle of secondary compression will yield a soil structure with similar behavior to the in situ clay.

It should also be noted that the practice of reconsolidating samples to the in situ stresses should generally not be used. Reference to Fig. 6 indicates that sample disturbance effects will not be overcome by this procedure since the sample will be significantly below the in situ compression curve. The resulting s_u will therefore be too high and the strength NSP will be larger than appropriate for the in situ OCR. With quick clays and cemented soils, this procedure may be unavoidable. In such a case, with the use of block samples or special sampling procedures and extremely

careful handling, it may be possible to reduce sample disturbance to within acceptable limits (Berre and Bjerrum, 1973). However, in general this procedure is not recommended.

Finally, the cost of using SHANSEP is probably such that for small jobs the present methods are preferable. For larger jobs involving a greater and more sophisticated engineering effort, SHANSEP could well be competitive or cheaper than the present methods, particularly as additional NSPs are published and as the required testing procedures become more widely available.

SUMMARY

When evaluated in the light of the results of recent research, the present design practice widely used to determine the stability of soft clay deposits is found to be highly empirical and of indeterminate and varying accuracy. Using the normalized behavior observed with many clays, a new method of design is presented, called SHANSEP. This procedure evaluates normalized strength parameters for the soil as a function of OCR and stress system. These are applied to the stress history of the foundation to give a strength profile for use in design.

SHANSEP avoids much of the empiricism of the present method and gives the engineer a far greater control over his design. The procedure has been successfully applied with a wide range of cohesive soils and practical methods for its use have been developed. For the more complex engineering designs, its cost is considered to be no more and potential considerably less than the present practice.

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APPENDIX - NOTATION

c	Cohesion intercept in terms of total stresses
\overline{CK}_oU	K_o consolidated-undrained shear test with measurement of pore pressures.
DSS	Direct-simple shear test
E_u	Undrained Young's modulus
FV	Field vane test
K_o	Coefficient of earth pressure at rest
NSP	Normalized Soil Parameter
OCR	Overconsolidation Ratio = $\bar{\sigma}_{vm} / \bar{\sigma}_{vo}$
PSA	Plane strain active shear test
PSP	Plane strain passive shear test
SHANSEP	Stress History and Normalized Soil Engineering Properties
s_u	Undrained shear strength
TC	Triaxial compression shear test
TE	Triaxial extension shear test
U	Unconfined compression shear test
UU	Unconsolidated-undrained triaxial compression test
w_l	Liquid limit
w_n	Natural water content
w_p	Plastic limit
γ_f	Shear strain at failure
$\bar{\sigma}_c$	Isotopic consolidation pressure
$\bar{\sigma}_{vc}$	Vertical consolidation stress (laboratory)
$\bar{\sigma}_{vm}$	Maximum past pressure
$\bar{\sigma}_{vo}$	In situ vertical effective stress
σ_1	Major principal total stress

σ_3	Minor principal total stress
σ_{1f}	σ_1 at failure
τ_h	Horizontal shear stress in DSS test
ϕ	Friction angle in terms of total stress

TABLE 1 UNDRAINED STRENGTH ANISOTROPY OF
NORMALLY CONSOLIDATED BOSTON BLUE CLAY

<u>TYPE OF TEST</u>	$\bar{s}_u / \bar{\sigma}_{vc}$	γ_f (%) ⁽¹⁾	\bar{s}_u / \bar{s}_u (TC)
$\overline{CK}_o U$ Plane Strain Active (PSA)	0.34	0.8	1.03
$\overline{CK}_o U$ Triaxial Compression (TC)	0.33	0.5	1.00
$\overline{CK}_o U$ Direct-Simple Shear (DSS)	0.20	6	0.61
$\overline{CK}_o U$ Plane Strain Passive (PSP)	0.19	8.5	0.57
$\overline{CK}_o U$ Triaxial Extension (TE)	0.155	15	0.47

(1)
 γ_f = shear strain at failure

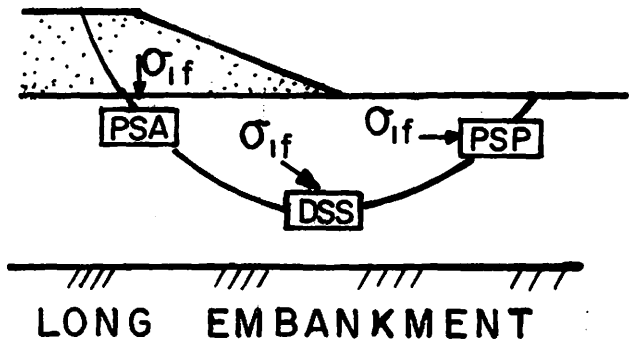
TABLE 2 COMPUTATION OF IN SITU \bar{s}_u FROM
 $\bar{s}_u / \bar{\sigma}_{vm}$ vs. OCR USING DSS STRENGTH
PARAMETERS FOR MAINE ORGANIC CLAY

Elevation, Ft.	$\bar{\sigma}_{vo}$, psf	$\bar{\sigma}_{vm}$, psf	OCR	$\bar{s}_u / \bar{\sigma}_{vc}$	\bar{s}_u , psf
- 10	185	1,000	5.4	1.14	210
- 15	315	800	2.55	0.65	205
- 20	450	740	1.65	0.45	200
- 25	590	740	1.25	0.35	205
- 30	730	765	1.05	0.30	215

Note: $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$ data are from the Portland, Maine, Test Section,
Fig. 9, and $\bar{s}_u / \bar{\sigma}_{vc}$ from Fig. 5.

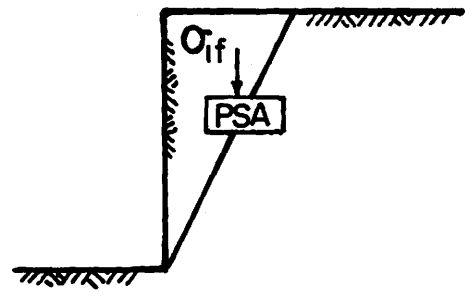
TABLE 3 SUMMARY OF FACTORS OF SAFETY FROM CASE STUDIES

Case Study	Factors of Safety				Comments
	Actual	SHANSEP S_u	Field Vane S_u	U & UU S_u	
Embankment Failure on Sensitive Marine Clay	1.00	1.01 (0.83-1.08)	0.88	0.7 ±	Range in SHANSEP F.S. based on range in estimated $\bar{\sigma}_{vm}$
Embankment Failure on Organic Clay	1.00	0.82	2.06	0.82 (U) 1.10 (UU)	UU tests run on clay after failure occurred
Atchafalaya Test Section II	1.0 +	1.0	1.8	—	Large lateral deformations with cracks in crest, on verge of failure
Atchafalaya Test Section III	>1	1.1	2.0	1.4	Large lateral deformation, poor performance
M.I.T. - MDPW Test Section on Boston Blue Clay	>1	1.50	1.18	0.73	Embankment performed well

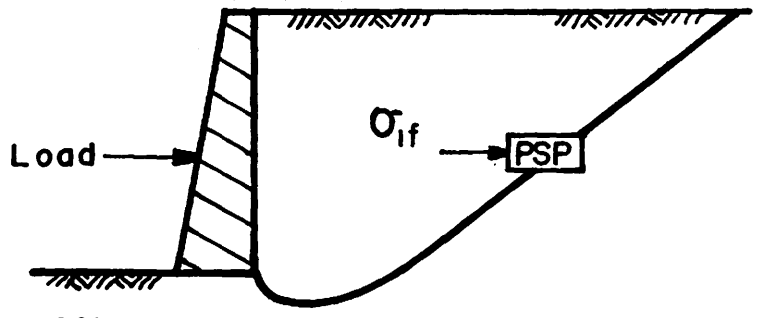


PSA - Plane Strain Active
 PSP - Plane Strain Passive
 DSS - Direct Simple Shear

LONG EMBANKMENT

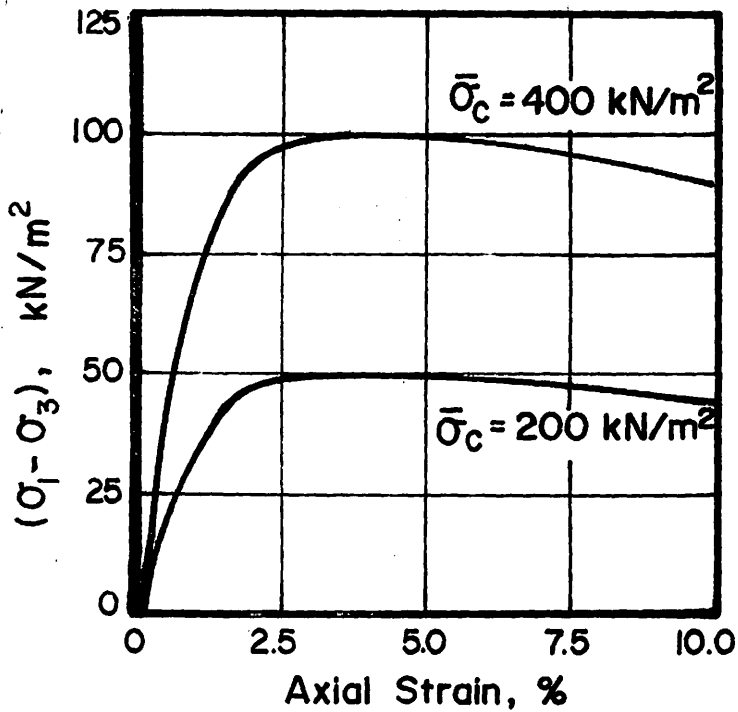


LONG VERTICAL CUT

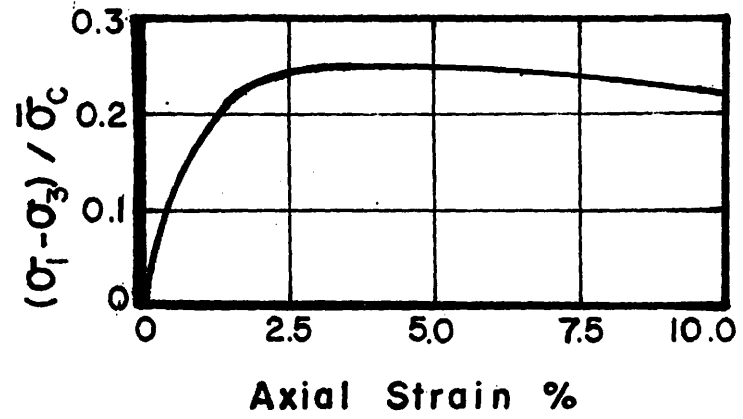


LONG LOADED RETAINING WALL

Figure No. 1 Stress Systems for Typical in Situ Modes of Failure



(a.) TRIAXIAL COMPRESSION TEST DATA FOR $\bar{\sigma}_c = 200$ & 400 kN/m^2



(b.) NORMALIZED PLOT OF TRIAXIAL TEST DATA

Figure No. 2

Example of Normalized Behavior Using Idealized Triaxial Compression Test Data for a Homogeneous Clay

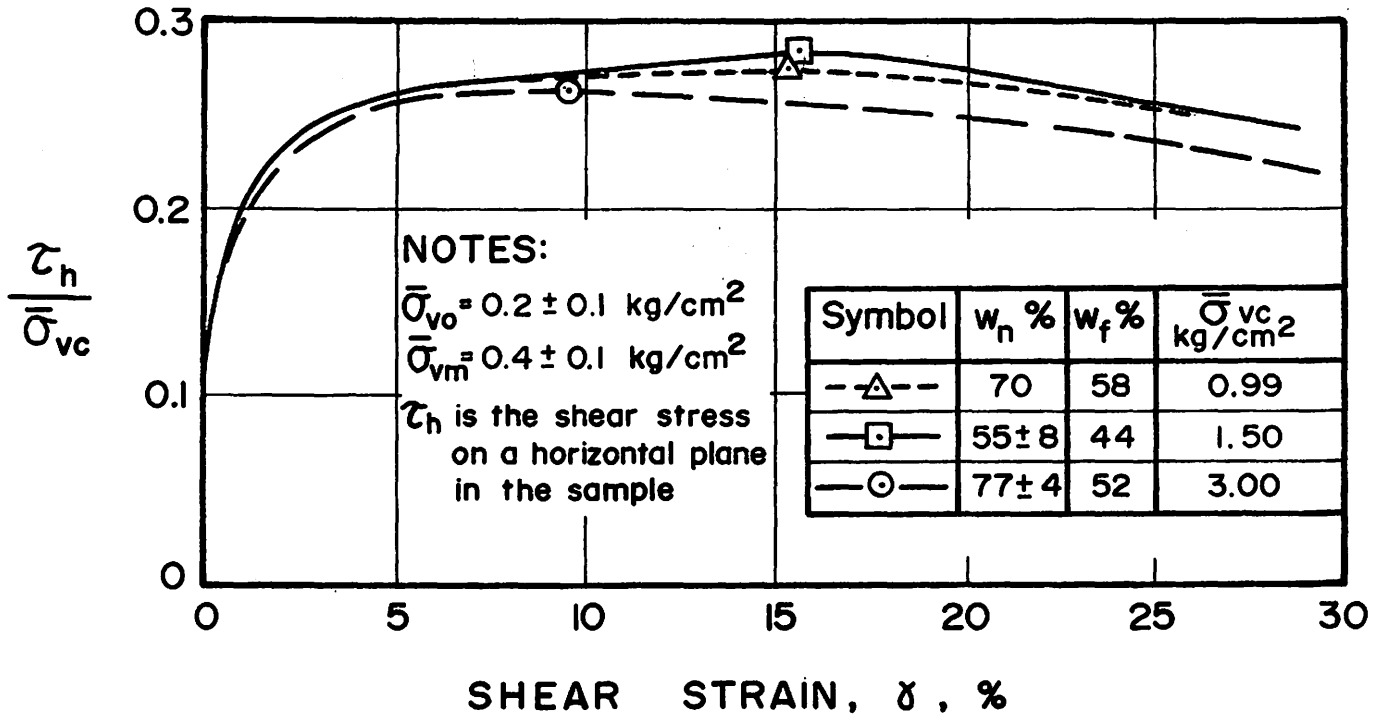


Figure No. 3 Normalized Direct-Simple Shear Test Data for Normally Consolidated Maine Organic Clay

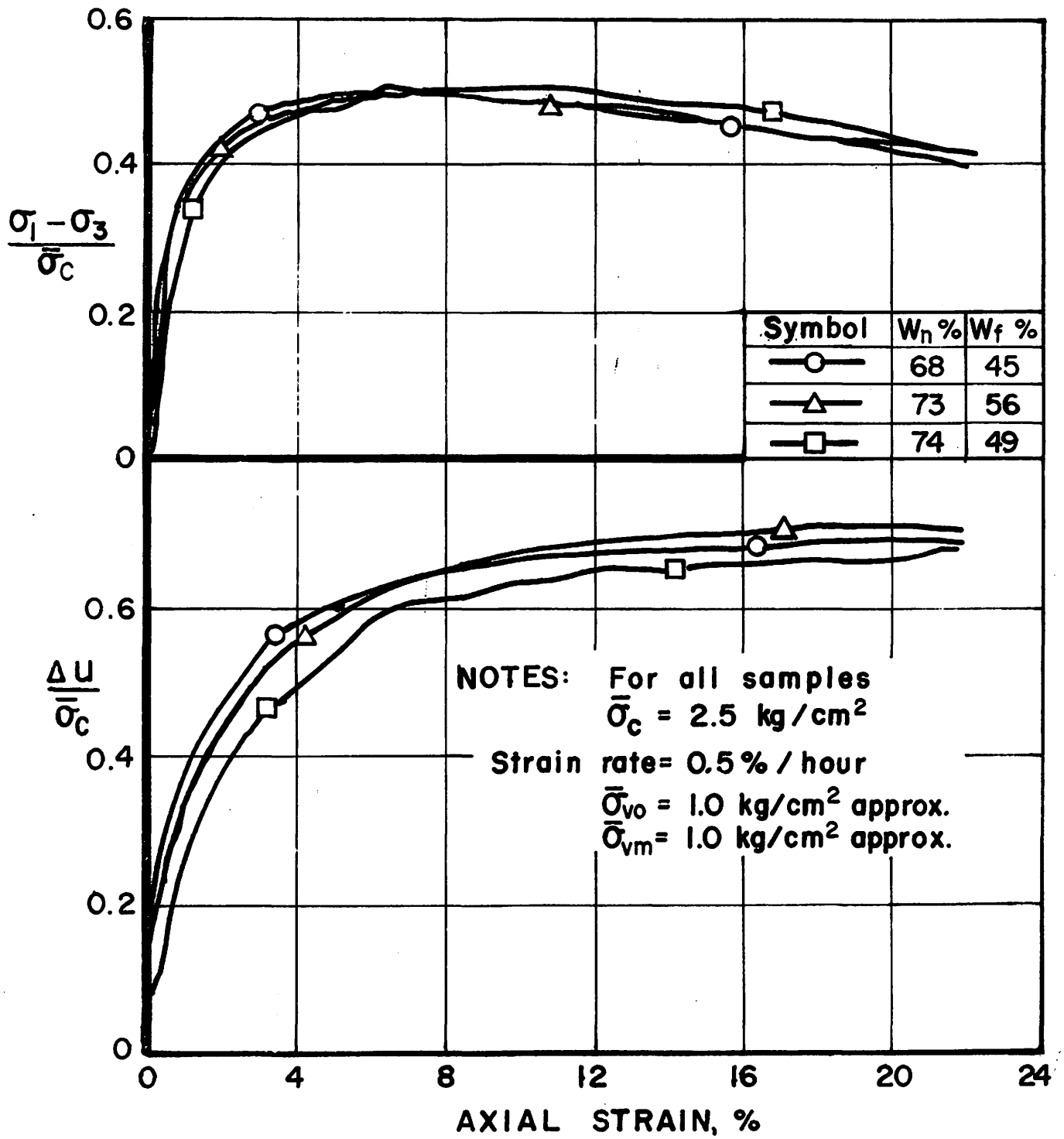


Figure No. 4 Isotropically Consolidated Triaxial Compression Test
 Results for Normally Consolidated Atchafalaya Clay

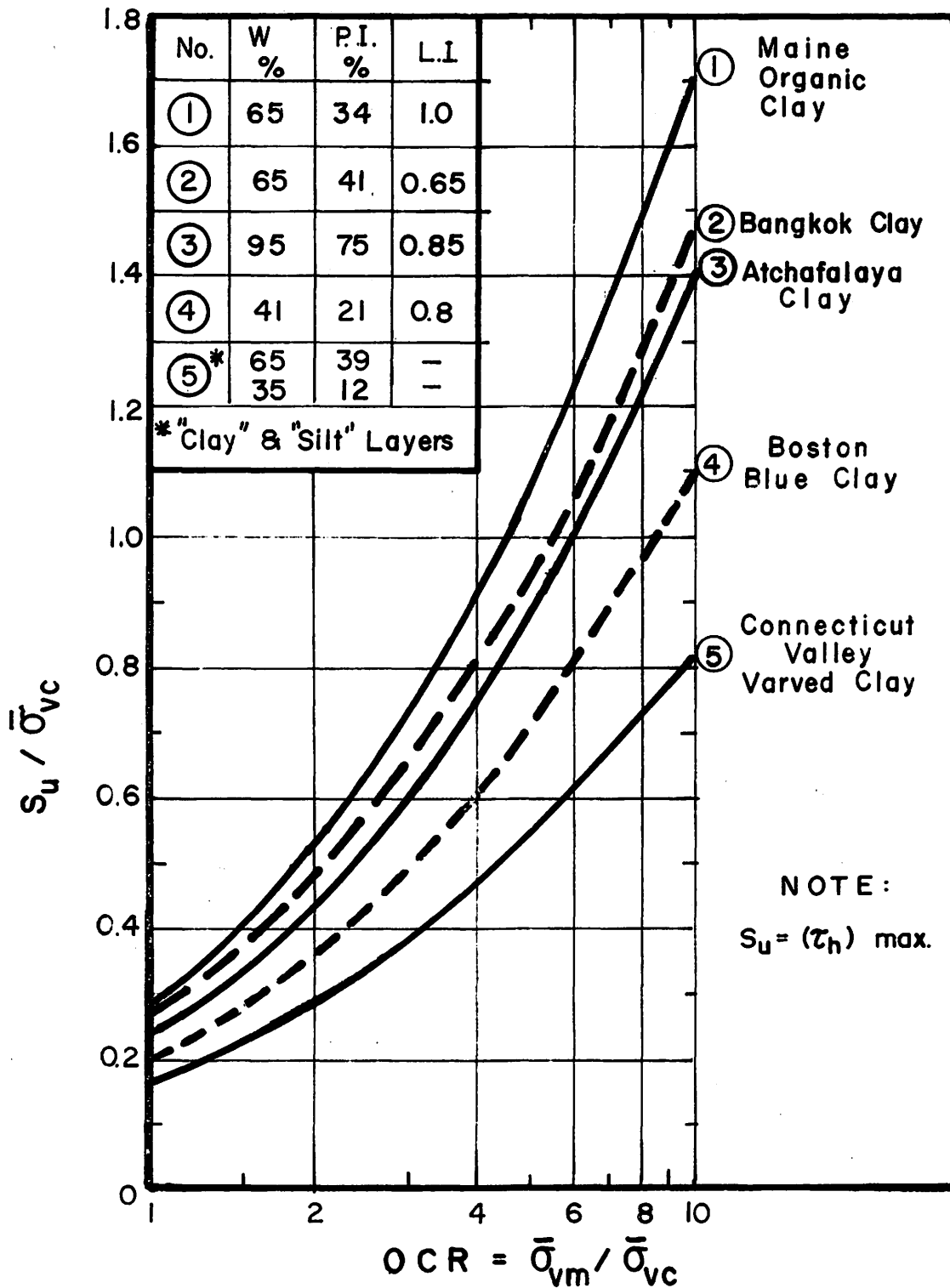


Figure No. 5 Variation of Normalized $\overline{CK UDSS}$ Strength Parameter with OCR for Five Soils

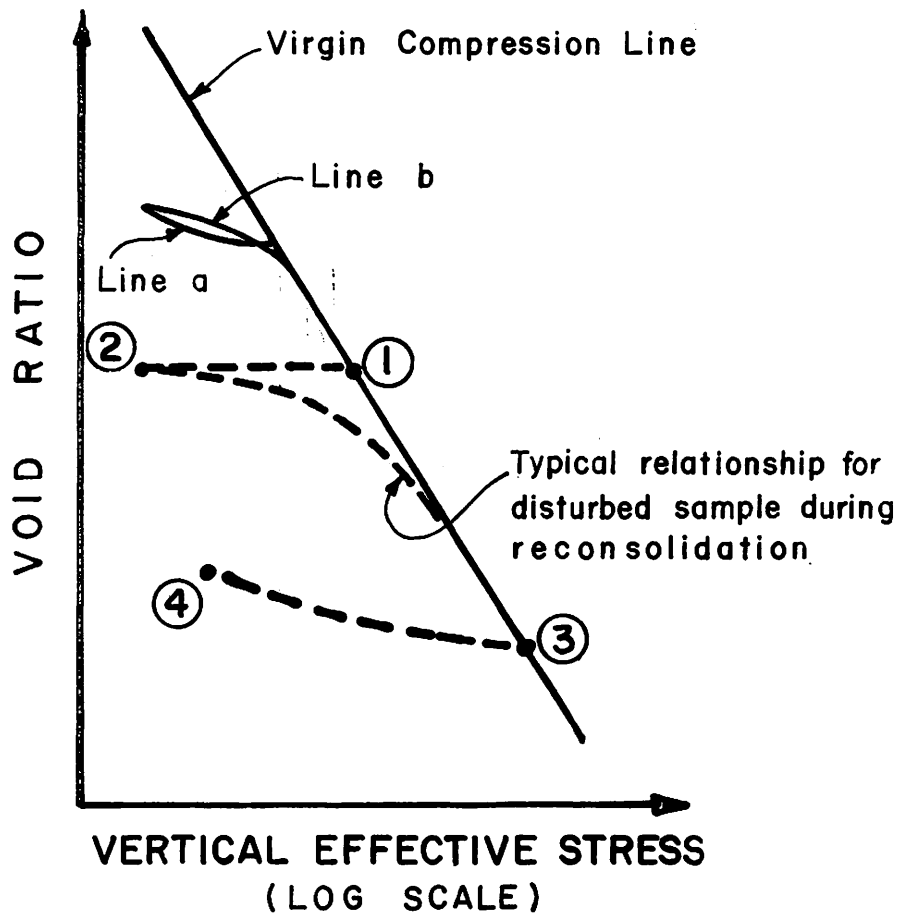


Figure No. 6 Idealized Plot Showing Effect of Sample Disturbance

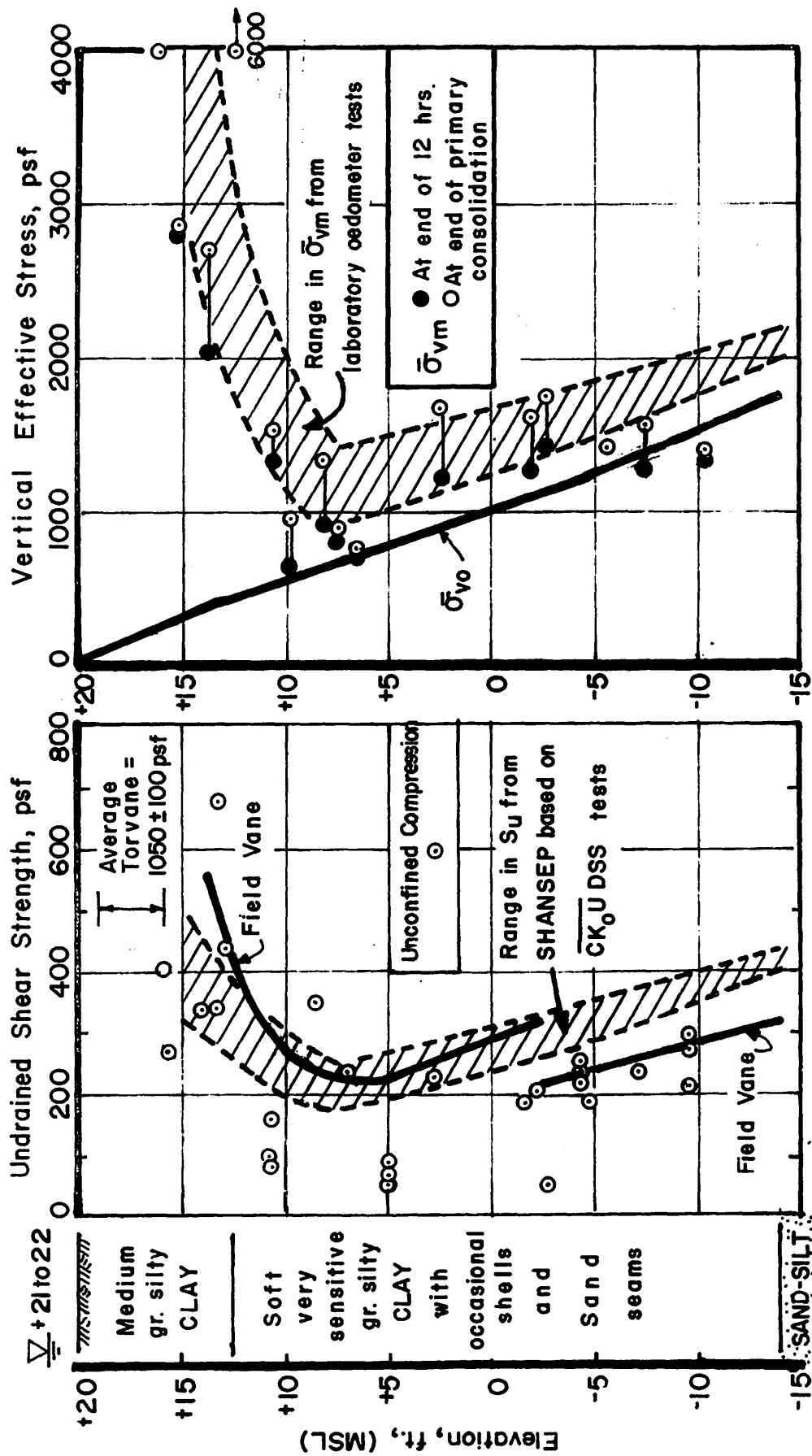
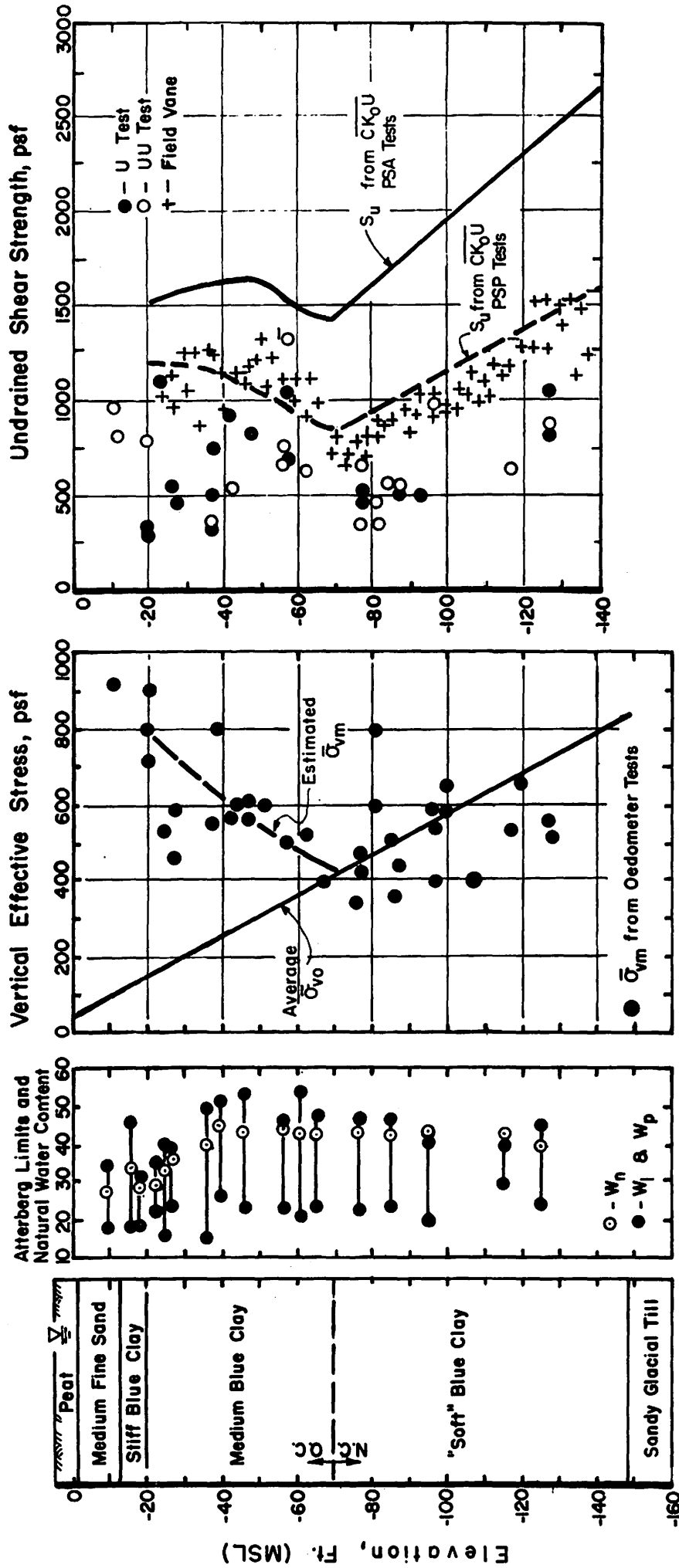


Figure No. 7 Soil Conditions at Portsmouth, New Hampshire, Test Embankments



From D'Appolonia, Lambe and Poulos (1971)

Figure No. 8 Soil Conditions at the M.I.T. - M.D.P.W. Test Section

