CONE PENETRATION AND ENGINEERING PROPERTIES OF THE SOFT ORINOCO CLAY

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SUMMARY

The following program was conducted at two widely separated (120km) borings in a 40m thick deposit of soft, plastic "Orinoco Clay" that covers vast areas offshore Venezuela: (1) "conventional" onboard strength-index tests; (2) cone resistance and excess pore pressures during in situ cone penetration; (3) compositional analyses and extensive consolidation and SHANSEP type strength testing. The conventional strength testing exhibited considerable scatter compared to well defined, uniform profiles of maximum past pressures and undrained shear strength that were generally confirmed by the in situ cone penetration data. The paper discusses the relative advantages of in situ versus conventional and "sophisticated" laboratory testing for design of offshore structures.

1. INTRODUCTION

All major offshore facilities rely upon the soils underlying the sea floor to support gravity loads and to resist horizontal forces caused by waves, currents, and seismic activity. The structure and foundation soils must withstand these forces without deforming in excess of operating limits and hence engineers must conduct offshore exploration and testing programs to obtain information regarding the stratigraphy and pertinent engineering properties of the foundation soils as part of the overall design process.

Procedures commonly employed in the design of offshore structures generally have a high degree of empirical content, both regarding methods for evaluating soil properties and techniques of analysis. In areas such as the Gulf of Mexico, certain "standard" procedures have evolved over the past 30 years that generally yield satisfactory designs since the soil conditions are relatively uniform (deep deposits of "soft" plastic clays) and most of the structures are similar in nature (pile supported platforms). Moreover, test procedures and methods of analysis were altered as appropriate based on experiences gained from hundreds of pile supported platforms and some pile load tests.

Yet even in the Gulf of Mexico with its extensive experience, at great water depths and with other types of structures (e.g.,guyed towers and tension leg platforms), the reliability of these design procedures must be questioned. Moreover, the same practice may be unsatisfactory when applied to different soil conditions encountered at new sites, such as within the North Sea and offshore California. Furthermore, any exploration and testing program that relies heavily on basically empirical procedures for evaluating soil parameters involves several inherent limitations:

1. Empirical correlations, such as those used to relate "strength index" test data to design parameters, vary with the precise sampling and testing procedures, the soil type and the depth of sampling;

2. The data obtained frequently exhibit excessive scatter, hence giving a misleading indication of the actual in situ variability in properties, and may even give erroneous trends with depth;

3. One develops little insight regarding the true nature of the foundation conditions, e.g. how basic properties should vary laterally and with depth, and hence each new facility requires

additional (and costly) site specific exploration and testing programs.

The Instituto Tecnologico Venezolano del Petroleo, INTEVEP, faced the difficult task of assessing the foundation conditions at several "virgin" offshore sites involving soil deposits varying from interlayered stiff clays and dense sands to a very thick stratum of soft plastic clay encountered throughout vast areas of the Gulf of Paria and the Orinoco Delta located offshore eastern Venezuela. Recognizing the inherent limitations imposed by relying solely on using "conventional" exploration and testing procedures, INTEVEP sponsored a comprehensive multi year effort aimed at determining the foundation conditions and pertinent soil engineering properties which consisted of:

1. Extensive geophysical exploration programs to establish the bathymetry, shallow stratigraphy (e.g., thickness of mud or loose soils) and unstable sea floor conditions as well as any geologic hazards such as active faults, paleochannels and gas pockets.

2. Comprehensive geotechnical investigation studies consisting of: gravity core sampling to determine the sedimentation pattern and sea

floor soil conditions; soil borings with high quality undisturbed sampling; and in situ testing involving the electric cone penetrometer as well as the piezometer probe and field vane at selected locations.

3. "Sophisticated" laboratory testing program in order to establish more reliable soil parameters for comparison with those obtained via "standard" practice.

4. Analyses to attempt to correlate the results of the in situ tests with the soil properties established from the laboratory testing programs.

This paper presents results for the thick Orinoco Clay deposit encountered offshore the eastern Venezuelan coast (see Fig. 1). This soft plastic clay was formed from Orinoco River sediments deposited during the Holocene Transgression as the ocean level rose from 400 ft below its present level. The paper specifically focuses on borings E1 and F1 located 120 km apart within the Gulf of Paria and Orinoco Delta, respectively.

2. SOIL EXPLORATION PROGRAM

The major geotechnical exploration program conducted at sites E1 and F1 during the summer of 1979 consisted of:

1. one boring to determine soil stratigraphy and to obtain soil samples;

2. a second to perform cone penetrometer tests using the electric Dutch cone (de Ruiter, 1971) to measure cone resistance and sleeve friction;

3. a third boring to measure pore pressures developed during cone penetration using the piezometer probe (Wissa et al., 1975 and Baligh et al., 1978); and

4. in situ field vane tests to a depth of 50 ft at site E1. This was performed in a separate boring adjacent to the cone penetrometer boring.

The drilling, sampling and in situ tests were conducted from the M/V SURVEYOR by Fugro Gulf, Inc., acting as the geotechnical consultants. MIT supervised the piezometer probe testing program, which used a device manufactured by Geotechnique Int., Inc. and represents the first time in situ penetration pore pressures were measured offshore. The borings were made using power tong rotary drilling techniques, a 5 inch OD IF drill pipe with an open-ended drill bit and appropriate drilling subs and drill collars to enable push sampling and in situ testing (Fugro, 1979 a and b). A heave compensator controlled the vertical motion of the drill string during drilling, sampling, and in situ testing.

Cone penetration tests were conducted by means of the Fugro/Wison equipment which is capable of pushing electrical cones and MIT's piezometer probe at a constant rate of penetration of approximately 2 cm/sec. The Wison uses the weight of the drill string and/or a drill string anchor (packer) to provide the necessary reaction.

Soil samples were taken at intervals ranging from 3 to 10 ft using a 3 inch OD thin-walled tube sampler. Except in the shallow very soft clay where hammmered samples were recovered, samples were obtained using Fugro's "WIP" sampling equipment (Fugro, 1979a). The WIP sampler, a hydraulically operated downhole tool, pushes a 2 or 3 ft long thin-walled tube, with or without a plastic liner, into the soil at a constant rate of penetration of about 2 cm/sec. Most of the soil samples were extruded from the sampler, examined and visually classified. Conventional strength index tests (e.g., miniature vane, Torvane, pocket penetrometer and fall cone), some UU triaxial compression tests and unit weight, water content, etc. determinations were performed onboard the vessel during drilling and sampling operations. Those samples designated for testing at MIT were sealed inside the tubes for eventual shipment.

Figure 2 gives the water depths at sites E1 and F1 and the locations of the 17 Orinoco Clay samples tested at MIT and considered in this paper, broken down as follows: 6 samples from boring E1 obtained at depths ranging from 37 to 135 ft; and 11 samples from boring F1 from depths 25 to 130 ft. Orinoco Clay samples from two other borings (F4 and D2, Fig.1) were tested at MIT, but the results are not included because no piezometer probe tests were performed at these locations.

3. SCOPE AND RESULTS OF LABORATORY TESTING PROGRAM

The program conducted at MIT involved: (1) compositional analyses in order to identify unusual clay minerals and potential testing problems; and (2) SHANSEP (An acronym for Stress History And Normalized Soil Engineering Properties) type testing (Ladd and Foott, 1974) to establish the stress history of the clay deposit, its consolidation characteristics and "normalized soil parameters" (NSP) for estimating the in situ stress-strain-strength properties. Specifically, the program consisted of:

1. index - classification tests (natural water content, Atterberg limits, Torvane, etc), tests to determine the salt content and organic matter, and oedometer tests on all samples;

2. mineralogical and compositional analyses and SHANSEP type K_o consolidated-undrained (CK_oU) tests using the direct simple shear device (Bjerrum and Landva, 1966) on representative samples;

3. SHANSEP type CK_oU triaxial compression and extension tests on a few samples to investigate strength anisotropy; and

4. Measurement of the preshear effective stress in UU triaxial compression test specimens to assess sample disturbance effects in boring F1 and one test to meassure K_o during one-dimensional consolidation.

Day (1980), Ladd et al. (1980) and Urdaneta (1980) present detailed information regarding the above program.

Prior to performing any tests, all sample tubes were radiographed at MIT in order to assess their quality. After several trials, the procedure adopted consisted of placing the sample about 6 ft from the X-ray head, exposing it for 5 minutes using a 160 kV input voltage with a current of 3.9 mA, and developing the film for about 15 minutes. Each tube was radiographed in 10 inch sections.

Figures 3a and b present two radiographs shown as positives made from the X-ray negatives. The first (Fig. 3a) shows the upper portion of a tube sample obtained from a depth of 127 ft from boring F1. The white color next to marking E at the top of the tube indicates a very low density material, which is the top wax seal, and the contorted "structure" and white zones between markings T to E suggest highly disturbed clay with substantial voids. These features were confirmed upon sample extrusion and subsequent Torvane testing. Figure 3b shows the middle portion of a sample obtained from a depth of 98 ft from boring E1. Here the white marks near the top indicate the existence of horizontal cracks which probably resulted from gas coming out of solution during the substantial decrease in confining pressure associated with sampling. The crack density decreases with depth and good to excellent Orinoco Clay is indicated by the radiograph between markings E through I.

Radiography was especially successful in detecting gas pockets or cracks, clay cuttings and zones of excessive sample disturbance. Such information proved essential in estimating the amount of suitable material in each tube and in selecting the best portions for the consolidation/strength tests.

3.1 Index-Classification and Compositional Analyses

Figure 2 presents the natural water content, w_N, and Atterberg limit data for the Orinoco Clay from tests run at MIT, by Fugro onboard ship and by Universidad Catolica Andres Bello, UCAB, in Venezuela. The Orinoco Clay samples tested at MIT were generally quite uniform (unless affected by gas) with very few shells or sand-silt seams. The results show that:

1. The Orinoco Clay is a fairly uniform CH-OH clay with a plasticity index, P.I., ranging from 35 to 65%.

2. The clay at boring F1 has a higher water content and is more plastic (higher P.I.) than the E1 deposit.

3. The clay at F1 appears to be slightly more plastic with increasing depth, while the clay at E1 does not indicate any significant difference between the upper and lower portion of the deposit. As will be shown, fairly marked changes with depth in the undrained strength parameters and in penetration pore pressures were measured at both E1 and F1. Such changes might be expected at F1, since the plasticity characteristics also change, but not at E1 where the average plasticity remains constant with depth.

4. The liquidity index of the Orinoco Clay decreases with depth, as would be expected if the deposit has a fairly constant, low overconsolidation ratio, OCR, throughout (as supported by the stress history data to be

presented).

The pore fluid salt concentrations were determined using electrical resistivity according to the procedure developed by Martin (1970). The results in Fig. 2 show that the salt concentration decreases with depth, having a value of about 32 g/l (normal sea water) at depth of 30 ft and decreases to approximately 25 $\pm$ 5 g/l at 140 ft. No variation can be detected in the salt concentration from one boring to the other. MIT found no effect of pore fluid salt concentration on the liquid limit and concluded that ordinary tap water could be used when performing consolidation and strength tests.

Tests to determine organic matter were performed on oven-dried samples using the Modified Walkley-Black Method (Allison, 1965). Results indicate that the organic contents fall within a narrow band (2 $\pm$ 0.7%) and show no particular trend with depth or boring location. Dr. R. Torrence Martin performed X-ray diffraction analyses on 11 samples, subjected to various types of pretreatment, e.g., bulk versus minus 2 μm fraction, various exchangeable cations, heating from 110° C to 550° C, glycerol hydrations, etc. The analyses indicated the same basic mineralogical composition for all samples, the principal clay minerals being kaolinite, illite and an interstratified phase ranging from chlorite (non-swelling) to smectite (highly swelling). Some of the samples contained calcite, the deeper ones contained a greater proportion of swelling minerals and all had substantial quartz and feldspar contents.

3.2 Stress History and Consolidation Properties

Extensive efforts were devoted to establishing the stress history, e.g., the effective overburden stress, $\bar{\sigma}_{vo}$, and the degree of overconsolidation expressed as OCR = $\bar{\sigma}_{vm}/\bar{\sigma}_{vo}$, where $\bar{\sigma}_{vm}$ = maximum past pressure. The $\bar{\sigma}_{vo}$ values were computed using the measured natural water content and specific gravity values, and assuming 100% saturation and hydrostatic in situ pore pressures. Seventeen incremental oedometer tests were performed using testing procedures which followed standard practice as described by Lambe (1951) except that: load increments were often applied for time intervals only of sufficient duration to enable determination of the end of primary consolidation; and load increments ratios of less than unity were used in the vicinity of $\bar{\sigma}_{vm}$ to obtain a compression curve with a better defined minimum radius of curvature. Casagrande's construction technique was applied to "end-of-primary" compression curves to estimate the in situ $\bar{\sigma}_{vm}$.

Effect of Disturbance

Figure 4 illustrates the effects of disturbance on oedometer test data. Both test specimens came from the same tube (128 ft depth at boring F1) having a computed $\bar{\sigma}_{vo} = 2.30\ kg/cm^2$. Test No.12 yielded a quite reasonable looking compression curve having a well defined minimum radius, but the measured $\bar{\sigma}_{vm}$ of only 1.35 kg/cm^2 indicated an "underconsolidated" clay. Test No. 18, run on material only two inches deeper (but having a substantially higher Torvane strength), showed less strain during recompression, a significantly higher virgin compression ratio (CR = 0.36 vs. 0.25) and most importantly a value of $\bar{\sigma}_{vm}$ which now indicated a slightly overconsolidated deposit.

To better judge the quality of the oedometer test data, several disturbance indices were developed and used to classify the data as "excellent-good" or "fair-poor" based on the following criteria:

1. The ratio of the Torvane strength measured directly above the oedometer specimen prior to its extrusion to that measured at the bottom of the tube onboard ship TV(Oed)/(Boat). It was generally observed throughout this study that "excellent-good" quality tests had

TV(Oed)/(Boat) ratios equal to or greater than unity. For example, oedometer tests No. 12 and 18 in Fig. 4 had Torvane ratios of 0.63 and 1.06, respectively.

2. The amount of computed vertical strain, ε_v, at the vertical overburden stress, $\bar{\sigma}_{vo}$. Disturbed samples experience more strain at $\bar{\sigma}_{vo}$ than good quality samples.

Based on these criteria, 12 out of the 17 tests performed at MIT fall into the "excellent-good" category.

Figure 5 presents typical compression curves from "excellent-good" oedometer tests performed on Orinoco Clay at five representative depths. The virgin compression curves of several of the deeper samples have a slightly concave upward slope, which further indicates very good quality samples. Also the curves shift to the right, with correspondingly higher $\bar{\sigma}_{vm}$ values, with increasing sample depth, which is consistent with the developed stress history.

Stress History

Figure 6 presents a plot of the vertical effective overburden stress at borings E1 and F1 computed assuming saturated clay and hydrostatic pore pressures. Also plotted in Fig. 6 are the maximum past pressure values obtained from the oedometer tests. The results in this figure show the following:

1. Due to differences in natural water content (see Fig. 2), the effective overburden stress at boring E1 is higher than that of boring F1.

2. In spite of differences in $\bar{\sigma}_{vo}$, the oedometer test data at the two locations indicate essentially the same maximum past pressure profile. The dashed lines in Fig. 6 give the best linear fit through the data classified "excellent-good", along with the $\pm$ one standard deviation bands.

3. Considering only the "excellent-good" data, the Orinoco Clay is normally to slightly overconsolidated with an OCR equal to 1.15 $\pm$ 0.15.

No fundamental reasons exist based on the geologic history of the Orinoco Clay deposit or on basic soil behavior principles to suggest that $\bar{\sigma}_{vm}$ should vary linearly with depth and have the same magnitude at the two boring locations, especially in view of the fairly significant differences in the estimated effective overburden stress. Thus, although the linear regression line plotted in Fig. 6 provides a very reasonable representation of the measured data, the actual $\bar{\sigma}_{vm}$ profiles at the two sites no doubt deviate somewhat from this simplified interpretation of the in situ stress history. Nevertheless, there can be little doubt that the Orinoco Clay deposit is essentially normally consolidated at boring E1 and slightly overconsolidated at boring F1.

The following discusses briefly, and in a highly speculative manner, possible mechanisms leading to the observed stress history of the Orinoco Clay. The essentially normally consolidated condition at boring E1 is what one would probably expect in a recent marine sediment assuming:

1. hydrostatic in situ pore pressure, i.e., full dissipation of pore pressures generated during deposition;

2. that the surface of the deposit always existed below ocean level (or at least no significant desiccation occurred);

3. no significant prior erosion of overburden nor natural cementation;

4. little or no aging (secondary compression) because of very recent depositional history; and

5. little or no "precompression" caused by wave action (Madsen, 1978; Urzua et al., 1978).

The geologic history of the Orinoco Clay suggests that the above conditions probably apply within the Gulf of Paria and hence the essentially normally consolidated stress history at boring E1 is consistent with geological considerations. In particular, deposition occurred at a relatively slow rate (estimated at 0.5 cm/yr.) and is still in progress, there is no evidence of significant desiccation or erosion of prior overburden and wave action should always have been minimal. However, except for increased wave action, essentially the same geologic conditions are also thought to apply to the Orinoco Delta. It is therefore difficult to explain why the clay at boring F1 is precompressed, though the overconsolidation ratio is quite small (OCR $\leq$ 1.3). Wave action, which produces cyclic shear stresses within clay sediments, may provide a partial answer since Madsen (1978) suggests that wave-induced cyclic stresses can cause the upper portion of an otherwise normally consolidated deposit to behave as an overconsolidated clay. But it is difficult to see how wave action would produce a constant OCR throughout a 130 ft thick deposit.

In conclusion, the oedometer test program yielded the same $\bar{\sigma}_{vm}$ profile at borings E1 and F1(as did tests at boring F4, Fig. 1). This profile is extremely consistent and well defined compared to what is usually obtained from offshore exploration programs (and even onshore deposits). The overconsolidation ratio varies at the two locations, because of differences in the estimated $\bar{\sigma}_{vo}$, and it is not clear how the clay at F1 became precompressed. Nevertheless, the well defined stress history at these two locations means that in situ properties can be established far more reliably than from a conventional testing program. Moreover, if the same $\bar{\sigma}_{vm}$ profile exists throughout most of the Orinoco Clay, which would appear to be a reasonable assumption for locations between and around borings E1 and F1, then the in situ properties should also be reasonably constant.

The consolidation characteristics of the Orinoco Clay are discussed by Ladd et. al. (1980). Finally, an oedometer test was performed in a special MIT K_o - cell (designed by Geotechnique Int., Inc.) to investigate the variation of the lateral stress ratio, K_o, with OCR. The results for a deep F1 sample measured during two unloading cycles (from stresses of 4 and 8 kg/cm^2) to OCR = 10 $\pm$ 2 can be described by the relationship:

$$K_o = 0.63\ (OCR)^{0.35}$$

3.3 SHANSEP Test Program And Results

The SHANSEP design method (Ladd and Foott, 1974) takes advantage of the well recognized fact that the in situ stress-strain-strength properties of most cohesive sediments are primarily controlled by the stress history of the deposit. Furthermore, many cohesive soils exhibit "normalized behavior", at least reasonably so from a practical design viewpoint, such that normalized soil parameters (NSP) like $s_u / \bar{\sigma}_{vo}$ can be uniquely related to OCR independent of the actual values of $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$. Ladd and Foott(1974) recommended that NSP be measured on test specimens one-dimensionally (K_o) reconsolidated to $\bar{\sigma}_{vc}$ values greater than the in situ $\bar{\sigma}_{vm}$ in order to minimize the effects of sample disturbance. Subsequent experience at MIT suggests that this procedure yields: (1) much more reliable results than reconsolidation to the in situ $\bar{\sigma}_{vo}$ when testing tube samples of low OCR clays, especially those typically obtained offshore; and (2) reasonable estimates of the in situ $s_u/\bar{\sigma}_{vo}$ versus OCR relationship (less true for undrained modulus) for those sedimentary deposits which are not highly sensitive (i.e., naturally cemented and/or leached clays possessing a high liquidity index) such that reconsolidation beyond the in situ $\bar{\sigma}_{vm}$ will obviously alter the natural clay structure.

The SHANSEP strength testing program relied mainly on K_o consolidated-undrained direct simple shear (CK_oUDSS) tests, rather than CK_oU triaxial tests, since DSS tests: (1) require less soil (often a very important consideration with offshore samples) and are simpler to perform than triaxial tests; and (2) yield stress-strain-strength data suitable for undrained stability and deformation analyses (Ladd et. al., 1977). Randolph and Wroth (1981) also suggest that the failure state in the DSS test may be quite similar to that occurring along the shaft of friction piles. The DSS program included nine tests on normally consolidated specimens to investigate the influence of sample location and the value of $\bar{\sigma}_{vc}$(lab)/$\bar{\sigma}_{vm}$(in situ) on normalized parameters and six tests to evaluate the effect of OCR (mainly for comparison with other clay types). A few CK_oU triaxial compression and extension tests were also performed to investigate anisotropy and to obtain a check on the normally consolidated friction angle ($\bar{\phi}$).

The stress-strain data plotted in Fig. 7 from the nine DSS tests show that:

1. The stress-strain curves fall into two groups, but within each group there is relatively little scatter.

2. The ratio of the laboratory $\bar{\sigma}_{vc}$ to the in situ $\bar{\sigma}_{vm}$ appears to have little effect on the measured results (especially compare the two tests from the same tube at z=56 ft), thus the Orinoco Clay exhibits reasonable normalized behavior.

3. All tests had a large strain at failure, γ_f = 11 $\pm$ 4%, and showed relatively little strain softening (except for one test).

4. The undrained strength ratio, $s_u/\bar{\sigma}_{vc}$, varies with depth, equalling 0.235 $\pm$ 0.01 for z less than 75 ft and 0.200 $\pm$ 0.005 for z greater than 75 ft.

The "abrupt" decrease in $s_u/\bar{\sigma}_{vc}$ at about 75 ft was entirely unexpected since it was not accompanied by significant changes in the plasticity index or mineralogy of the clay. Also, the normalized strength ratio for the samples below 75 ft plot well below the $s_u/\bar{\sigma}_{vc}$ vs. P.I. relationship obtained on a variety of clays (Ladd, 1981).

The normalized modulus E_u/s_u (E_u is the secant Young's modulus) data obtained from the N.C. DSS tests are quite consistent, with E_u/s_u decreasing from 650 $\pm$ 200 to about 120 $\pm$ 20 as the stress level increases from 20 to 80 percent of the undrained shear strength. Tests performed on clays above 75 ft yielded somewhat higher normalized moduli.

Six CK_oUDSS tests were performed on deep samples (z > 75 ft) at nominal OCR values of 2,4 and 8. The results show that the normalized strength ratio, $s_u/\bar{\sigma}_{vc}$, increases in proportion to $(OCR)^{0.70}$ which is consistent with trends observed for other clay deposits (Ladd et al. , 1977). Similarly, the variation of E_u/s_u with OCR is very consistent and follows a pattern similar to that exhibited by other soft plastic clays (Day, 1980).

The following compares the mean results of SHANSEP type K_o-consolidated undrained triaxial compression, extension, and direct simple shear tests run on normally consolidated samples of the deep Orinoco Clay:

Test	$\beta°$	$s_u/\bar{\sigma}_{vc}$	E_{50}/s_u	Max. $\bar{\phi}°$	$\gamma_f(\%)$	Remarks
TC	0	0.23	390	25	7	3 Tests
DSS	30–60	0.20	260	—	11	4 Tests
TE	90	≃0.16	130	20(?)	>10	1 Test (Peak not reached)

where:

β = the angle between the major principal stress direction at failure and the vertical, and $s_u = 0.5 (\sigma_1 - \sigma_3)_f \cos \bar{\phi}$ for TC and TE tests, and $= (\tau_h)$ max. in the DSS tests.

The results indicate little undrained strength anisotropy.

4. COMPARISON OF UNDRAINED STRENGTH DATA

Figures 8 and 9 plot undrained shear strength, s_u, values versus depth, at the E1 and F1 boring locations respectively, obtained via "conventional" practice and from the SHANSEP design method. The conventional s_u values in these two figures were obtained by Fugro onboard ship using laboratory vane, Torvane and unconsolidated undrained triaxial compression, UUC, tests and by MIT in its laboratory using Torvane and UUC tests. The mean SHANSEP direct simple shear undrained strength profiles were computed using the relationship:

$$s_u = \bar{\sigma}_{vo} S \, (OCR)^m$$

$$\begin{aligned} \text{where } S &= s_u(DSS)/\bar{\sigma}_{vc} \quad \text{for N.C. clay} \\ &= 0.235 \text{ for } z < 75 \text{ ft} \\ &= 0.200 \text{ for } z > 75 \text{ ft} \\ OCR &= \bar{\sigma}_{vm}/\bar{\sigma}_{vo} \text{ using the profiles in Fig. 6} \\ m &= 0.70 \text{ from Section 3.3} \end{aligned}$$

The $\pm$ one standard deviations about the mean were computed according to the following simplified procedure suggested by Baecher (1982):

$$Cov^2[s_u] = Cov^2[S] + m^2 \, Cov^2[\bar{\sigma}_{vm}]$$

where Cov is the coefficient of variation (standard deviation/mean) resulting from the scatter in the normally consolidated $s_u(DSS)/\bar{\sigma}_{vc}$ and in $\bar{\sigma}_{vm}$ (both m and $\bar{\sigma}_{vo}$ were considered deterministic).

Observing the results in Figs. 8 and 9, the following remarks can be made:

1. The "conventional" mean s_u values undergo an abrupt increase at z = 60 ft for boring E1 and z = 40 ft at F1. This increase is believed to be associated with the change in sampling techniques from hammering to pushing (denoted by WIP samples).

2. A large scatter exists in the "conventional" s_u values (often by a factor of two or more) at essentially all depths. This is particularly the case for the UUC data at boring E1 below z = 80 ft and those measured by the lab vane and Torvane tests. As stated by Ladd et al., 1977, the s_u values obtained from these tests can be significantly affected by sample disturbance, applied strain rate (or time to failure) and strength anisotropy. Sample disturbance can greatly reduce the measured value of s_u. On the other hand, the high strain rate used in the vane and UUC tests and the fact that the latter measures s_u for vertical loading tend to overestimate the in situ strength. Therefore,

with very high quality samples, the measured s_u values from conventional tests can be too high, whereas gross sample disturbance will yield strengths which are much too low. As a result, a large scatter such as encountered here is often observed in conventional s_u data, especially offshore due to the highly variable degree of sample disturbance.

3. Comparison of the conventional test results with the SHANSEP strength profiles shows that about one third to one half of the conventional data plot below the average s_u minus one standard deviation line, and hence must have suffered from excessive sample disturbance. The results also show that a substantial fraction of the conventional strengths (especially from the laboratory vane and UUC tests) greatly exceed the average s_u plus one standard deviation line below z = 60 ft at boring E1. The same is also true of most of the MIT "corrected" UUC strengths at boring F1. These corrected strengths were obtained based on measurements, in a special triaxial cell, of the preshear effective stress and the semi-empirical procedure developed by Ladd and Lambe, 1963 (see Ladd et al., 1980; and

Urdaneta, 1980, for details of the testing program and results). Hence, conventional practice can also yield unconservative strength values.

4. Good agreement exists between the UUC strength values at boring F1 measured by Fugro and MIT (except in a very disturbed MIT sample at a depth of 127.6 ft).

Selecting design strength from the conventional data shown in Figs. 8 and 9 no doubt varies depending upon the particular engineering organization and the nature of the proposed offshore structure. But clearly any values selected would be subject to considerable uncertainty given the extreme scatter in the measured data. By contrast, the SHANSEP s_u profiles should provide a highly reliable estimate of strength variations with depth, even considering the scatter reflected by the $\pm$ one standard deviation bands. These strength profiles and the associated stress histories can be correlated to pile load tests in order to develop more reliable design procedures. They also should represent a good estimate of the in situ s_u appropriate for bearing capacity problems. As an example, the SHANSEP strength profiles were used to predict the leg penetration of a jack-up platform to be placed about 5 km away from boring E1. The predicted penetration of 19 $\pm$ 2 m after preload was close to the measured value of 22.5m.

5. CONE PENETRATION RESULTS

This section presents the cone penetration data at sites E1 and F1 obtained by means of the electric Dutch cone penetrometer (de Ruiter, 1971) and the relatively new conical piezometer probes (Wissa et al., 1975; and Baligh et al., 1978). INTEVEP included both devices in its investigation programs with the objectives of: 1) obtaining continuous measurements which would hopefully lead to better identification of soil stratifications and; 2) establishing correlations between the cone penetration data and the engineering soil properties from laboratory tests, thereby reducing the need for future extensive laboratory exprimental programs.

Figures 10a and b show the cone resistance, q_c, and penetration pore pressure, u, measured during cone penetration at sites E1 and F1, respectively. These plots represent the steady penetration data obtained after deleting the initial segments of the penetration records corresponding to unsteady and/or uncertain locations at the beginning of each push. The complete penetration records are given in Baligh et al. (1980). The q_c data were obtained using a cone with an area of 10 cm^2 and the u data were obtained at the tip of a 60° pore pressure probe. The results in Fig. 10a show that:

1. The values of q_c are only slightly higher than u, which suggests, based on experience with other deposits (Baligh et al., 1978), that the upper 151 ft at site E1 contain soft clays.

2. The transition between the soft clays and the underlying soil at depth 151 ft is clearly marked by a sudden increase in q_c and decrease in u.

3. Based on the classification tests performed onboard the vessel and supplemented by tests at Universidad Catolica Andres Bello, UCAB, in Venezuela, the upper 151 ft at site E1 are described as very soft to stiff grey clay. However, in view of the penetration data in Fig. 10a, this deposit can be possibly divided into 4 sublayers:

   a) Sublayer B, at z = 60 to 65 ft, has a smaller penetration pore pressure in comparison with the layer above and below and is also marked by a very slight increase in q_c. These features suggest that sublayer B is a thin crust. Shallow seismic geophysical data indicate the presence of a reflector at z ≃ 60 ft.

   b) Sublayer D, z = 90 to 151 ft, has a

slightly different rate of increase of u with depth compared to layers A and C. As mentioned previously, laboratory test results presented in Section 3 show different DSS normalized strength characteristics between D and the upper layers.

Similarly, the cone penetration results obtained at site F1 and illustrated in Fig. 10b show the following:

1. the q_c values in the upper 134 ft are only slightly larger than the corresponding u values. As mentioned previously, this is characteristic of soft clays; and

2. The top 134 ft, which are described as soft to stiff clay based on the onboard classification tests, can be further divided into three sublayers:

   a) sublayer B, z = 75 to 130 ft, has a significantly different rate of increase of u with depth compared to A and C. This is consistent with the SHANSEP strength variation with depth.

   b) sublayer C, z = 130 to 134 ft, is characterized by a clear drop in u which is accompanied by a very slight increase in q_c.

A transition sublayer between 75 and 85 ft could also have been added.

The above results show that the q_c and u data are very valuable in detecting the presence of thin layers which might affect stability and drainage, as well as in providing a good detailed picture of soil variability.

The results of the extensive laboratory testing program presented in Section 3 of this paper show that, for all practical purposes, the Orinoco Clay at sites E1 and F1 has similar in situ engineering properties and that those properties vary with depth. The remainder of this section will now attempt to compare the penetration data obtained at the two sites and establish correlations between such data and the laboratory test results.

In order to be able to compare the penetration pore pressure data at the two sites having different water depths and initial overburden stress, Fig. 11 plots the ratio $(u-u_o)/\bar{\sigma}_{vo}$ versus depth from which we note:

1. With the exception of sublayer B, the $(u-u_o)/\bar{\sigma}_{vo}$ values at sites E1 are essentially constant with depth and are in the range of 2.5 to 3. For sublayer B, this ratio is between 1 to 2.5.

2. On the other hand, at site F1, significant variations in $(u-u_o)/\bar{\sigma}_{vo}$ with depth can be observed. For z < 75 ft, this ratio is in the range of 3 to 3.5, except from z = 25 to 35 ft where it ranges from 3.5 to 6. For z = 75 to 125 ft, $(u-u_o)/\bar{\sigma}_{vo}$ ranges between 2.5 to 3.

3. The $(u-u_o)/\bar{\sigma}_{vo}$ values below a depth of 75 ft from both sites are more or less equal, which confirms that the clays at the two sites are similar. The same, however, could not be established for the clays above 75 ft (especially for z = 20-40 ft and 60 - 65 ft).

In summary, the results at site F1 show a significant variation in properties between the "upper" and "lower" clays as detected by the difference in the $(u-u_o)/\bar{\sigma}_{vo}$ values. This was also confirmed by the measured differences in the rate of pore pressure increase with depth (see Fig. 10b). At site E1, although a difference in the rate of pore pressure increase with depth can be identified, such a trend could not be substantiated by the normalized pore pressure data.

Finally, the cone factors, N_c, were computed for the q_c data at the two sites. N_c is defined as:

$$N_c = \frac{q_c - \sigma_{vo}}{s_u(DSS)}$$

where:

σ_{vo} = initial total vertical stress.

$s_u(DSS)$ = mean SHANSEP s_u profiles plotted in Figs. 8 and 9.

At site E1, N_c is essentially constant with depth at about 12 $\pm$ 2. At F1, N_c is equal to 15 $\pm$ 3 down to z 110 ft and decreases to about 13 $\pm$ 1 below.

6. SUMMARY AND CONCLUSIONS

This paper presents results from a geotechnical investigation of the soft, plastic 30 to 40m thick Orinoco Clay covering vast areas offshore eastern Venezuela. Special features of the test program performed at two sites 120 km apart (borings E1 and F1, Fig. 1) included in situ electric cone penetration and piezometer probe tests, compositional analyses and extensive consolidation and SHANSEP type strength tests.

Principle conclusions from the laboratory test program conducted on high quality "push" samples are as follows:

1. Generally similar mineralogical compositions at both locations, thus indicating reasonably similar source materials and depositional environments.

2. Essentially identical, well defined profiles of measured maximum past pressures ($\overline{\sigma}_{vm}$) throughout the deposit, this showing the material at Gulf of Paria to be normally consolidated and that in the Orinoco Delta to be slightly overconsolidated (OCR = $\overline{\sigma}_{vm}$ / $\overline{\sigma}_{vo} \simeq$ 1.2).

3. Excellent agreement between the normalized undrained stress-strain parameters (e.g. $s_u/\bar{\sigma}_{vc}$, E_u/s_u) measured via K_o consolidated-undrained direct simple shear (CK_oUDSS) tests run on normally consolidated specimens from both sites. However, tests at both locations showed a higher normallly consolidated $s_u/\bar{\sigma}_{vc}$ in the upper portion of the deposit (0.235 $\pm$ 0.01) than at great depths (0.200 $\pm$ 0.005).

4. Essentially identical profiles of s_u(DSS) versus depth computed by the SHANSEP design method at the two boring locations.

5. SHANSEP provides a much more reliable estimate of the in situ properties of the Orinoco Clay than obtained from conventional "strength index" tests (e.g. lab vane, Torvane, UUC, etc.), which exhibited extreme scatter.

The cone resistance and pore pressure profiles measured during cone penetration yield a more detailed picture of soil stratification than possible with undisturbed sampling and laboratory testing. The penetration pore pressures confirmed the smaller SHANSEP s_u(DSS)/$\bar{\sigma}_{vc}$ in the lower portion of the deposit at boring F1, but not at boring E1. Values of the cone factor derived from the s_u(DSS) profiles generally showed $N_c = 15 \pm 4$. These results indicate that future in situ testing should use the newly developed piezocone (Baligh et al., 1981) for simultaneous measurements of cone resistance and penetration pore pressures.

The nearly identical results obtained from both the in situ and SHANSEP test programs conducted at widely spaced borings and geological considerations strongly suggest that vast areas of the Orinoco Clay should have very similar engineering properties. This important conclusion should enable substantial cost savings since future site specific exploration programs within the deposit can mainly utilize relatively inexpensive piezocone testing.

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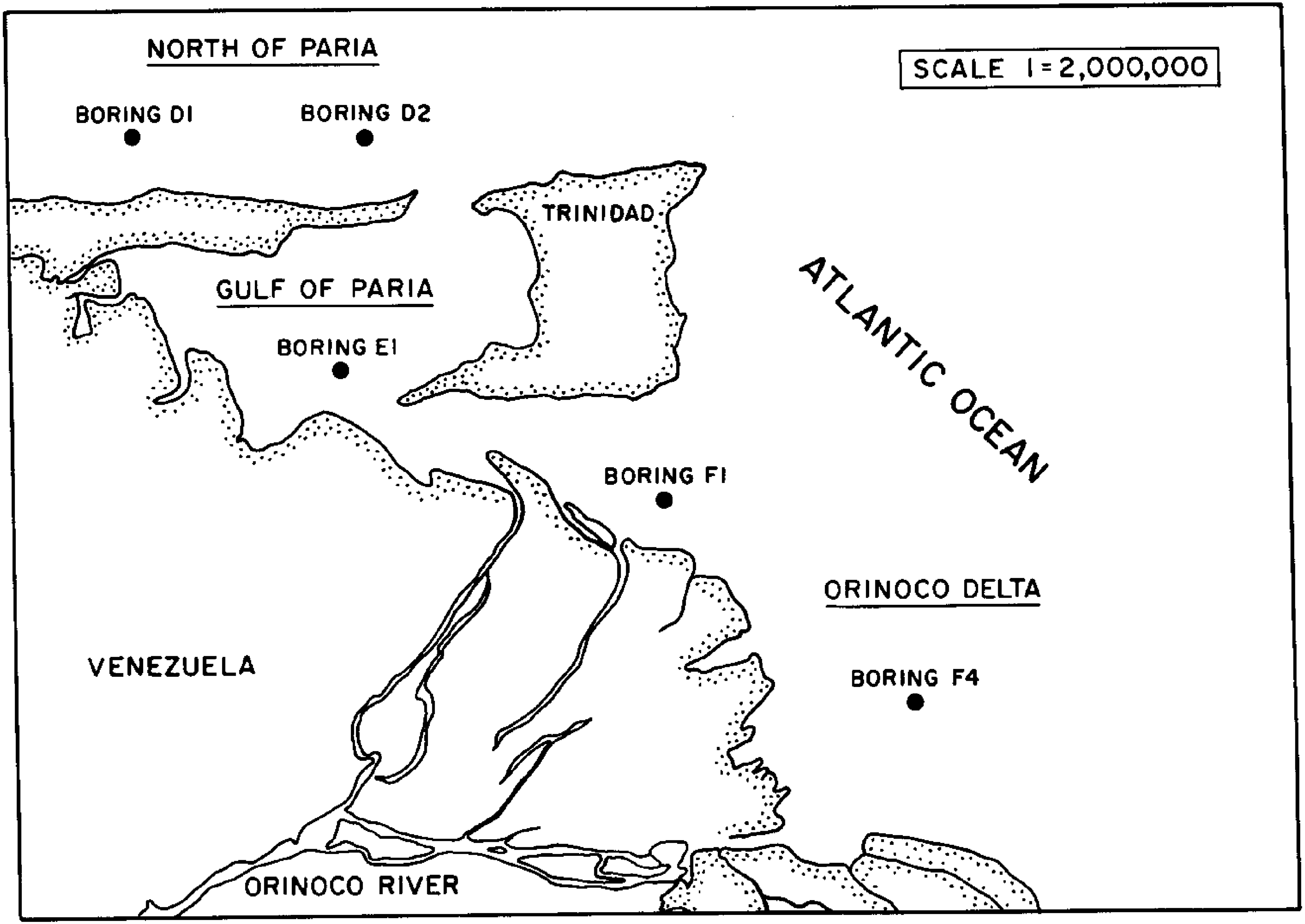


FIGURE 1. BORING LOCATIONS

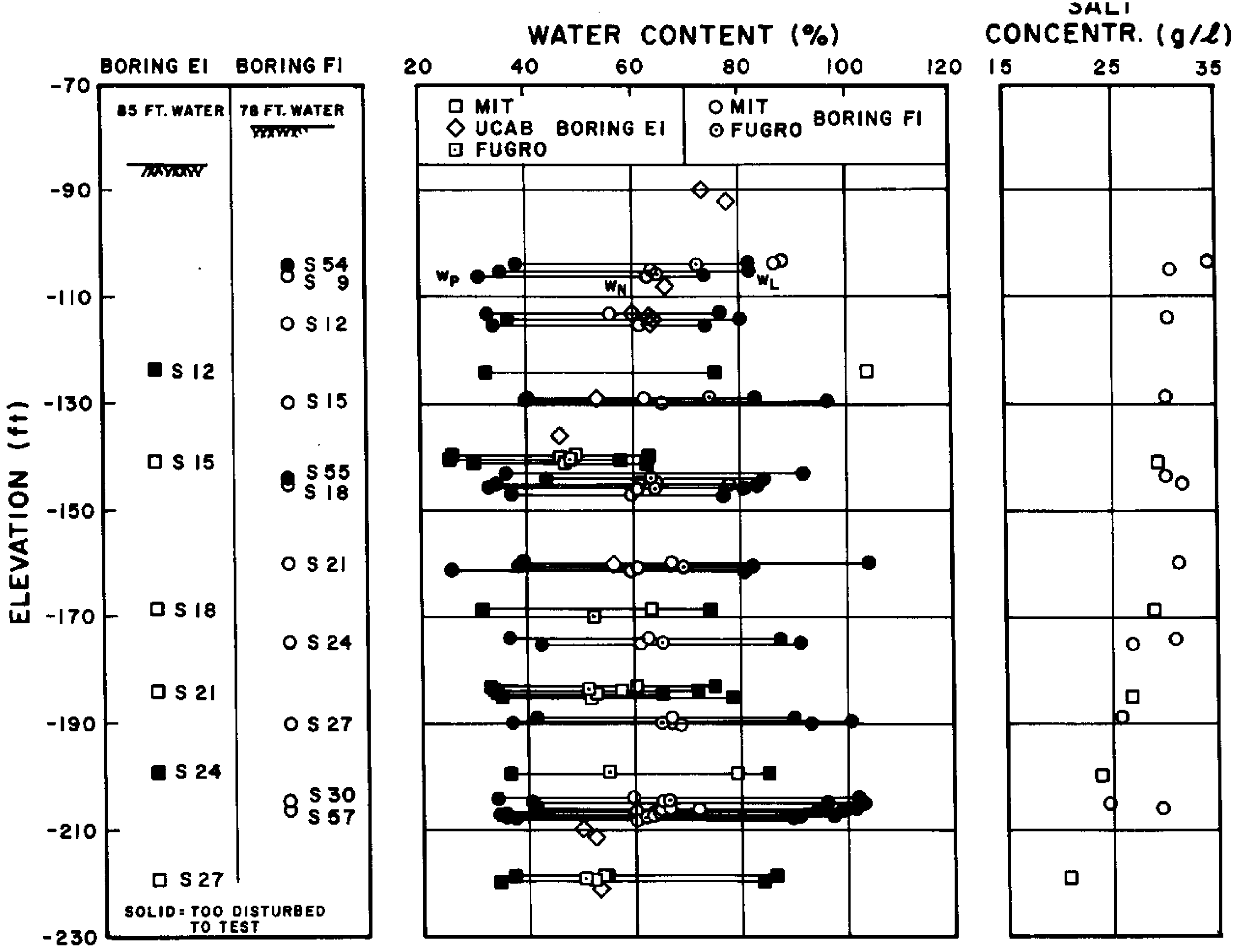
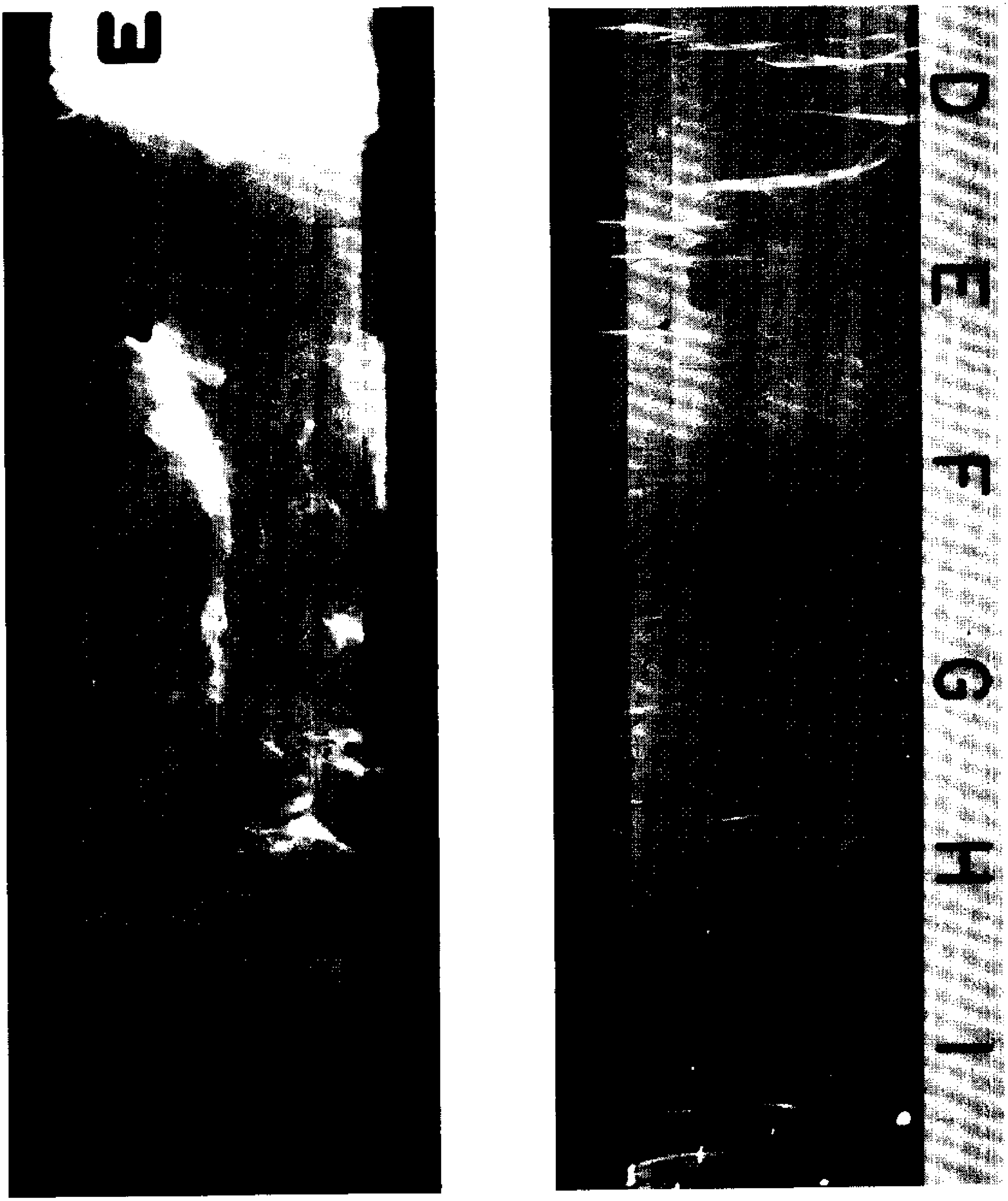


FIGURE 2. MIT SAMPLE LOCATIONS AND INDEX-CLASSIFICATION DATA



(a) (b)

FIGURE 3. RODIOGRAPH PRINTS OF ORINOCO CLAY SAMPLES

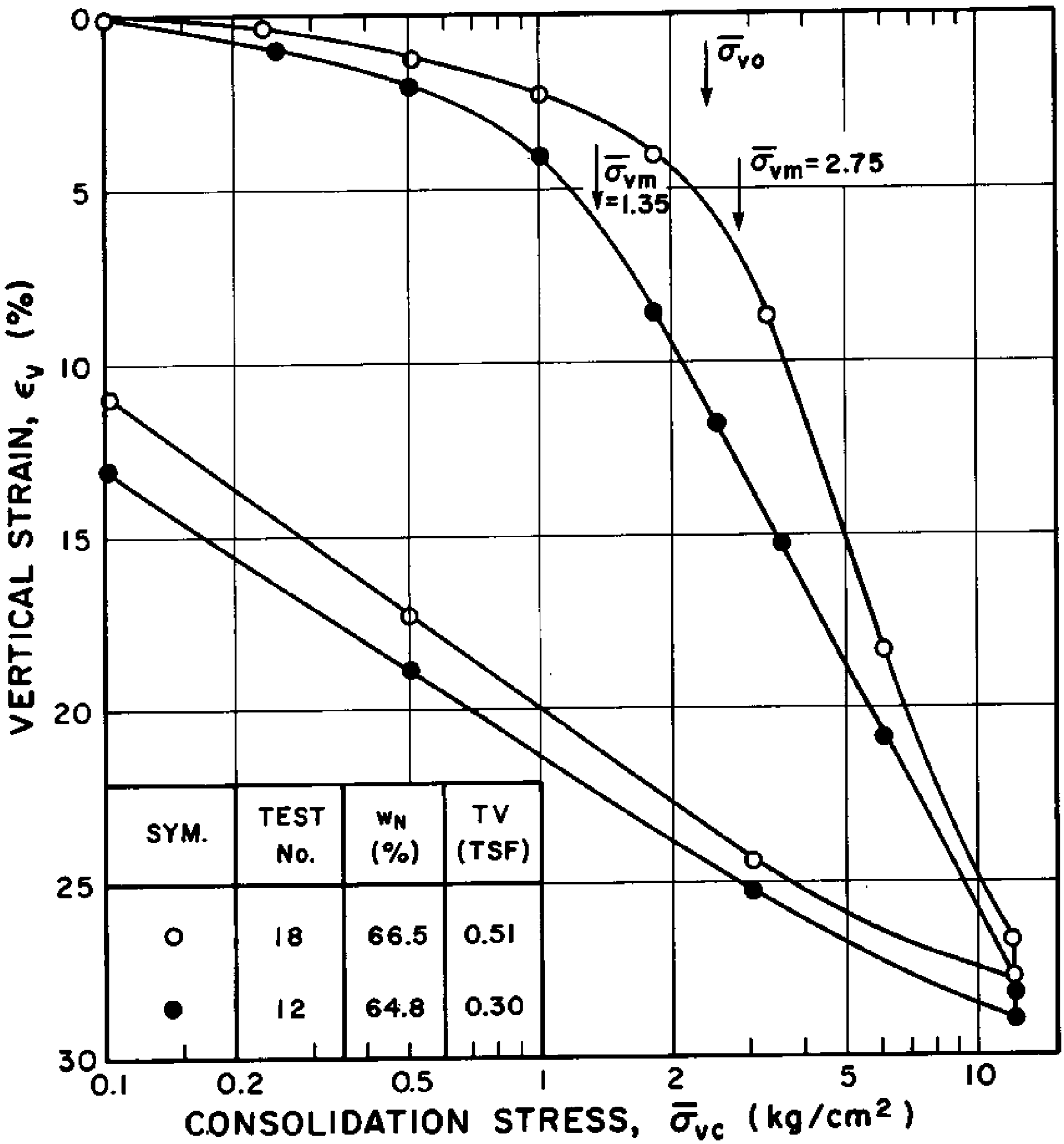
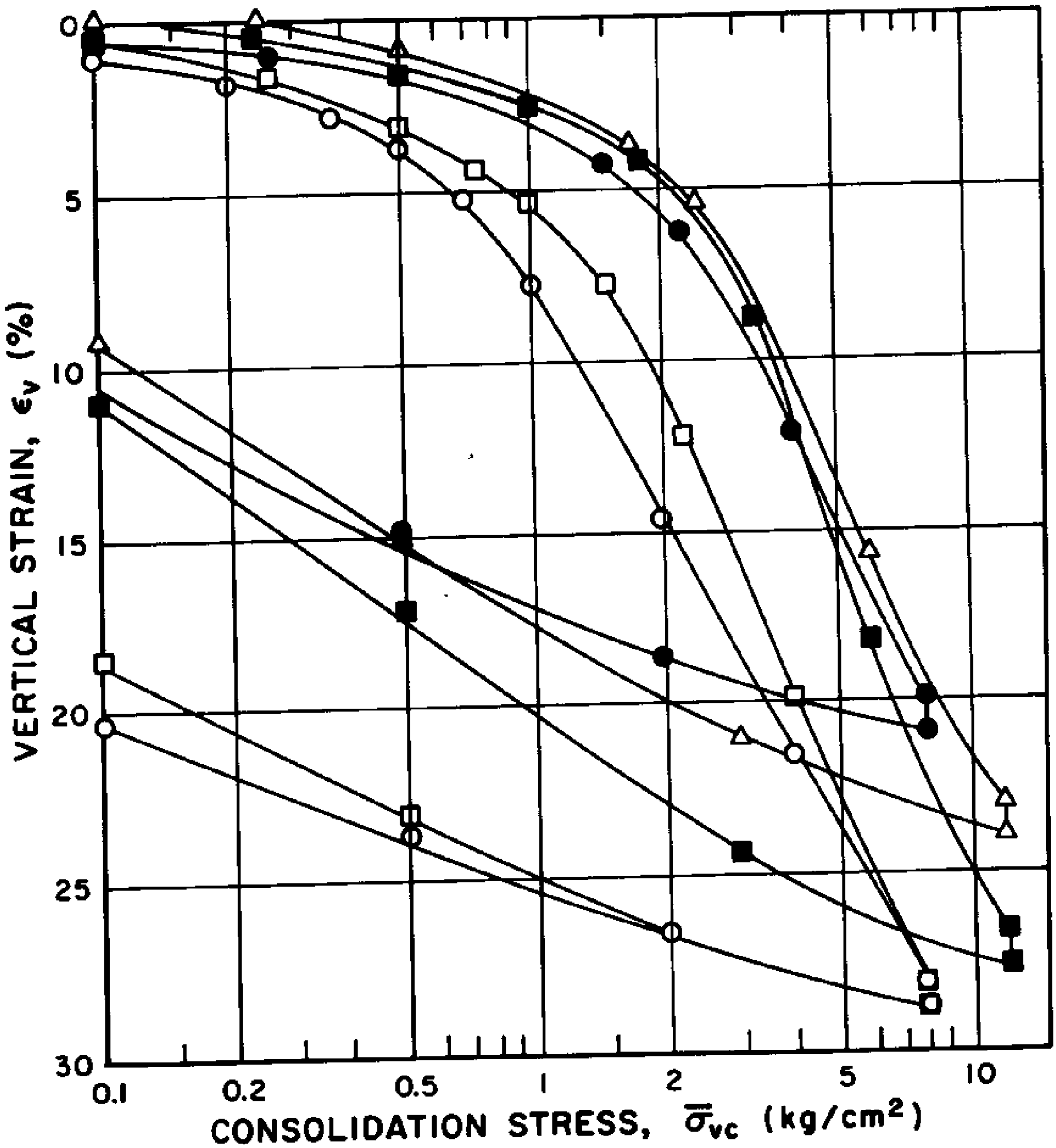


FIGURE 4. EFFECTS OF DISTURBANCE ON OEDOMETER TEST DATA: SAMPLE FIS57



SYMBOL	DEPTH (ft)	SAMPLE No.	$\bar{\sigma}_{vo}$	$\bar{\sigma}_{vm}$
○	36.6	FIS12	0.61	0.75
□	66.7	FIS18	1.17	1.50
●	96.3	FIS24	1.72	2.40
■	128.0	FIS57	2.31	2.75
△	133.9	EIS27	2.74	2.80

FIGURE 5. TYPICAL "EXCELLENT-GOOD" OEDOMETER COMPRESSION CURVES: ORINOCO CLAY

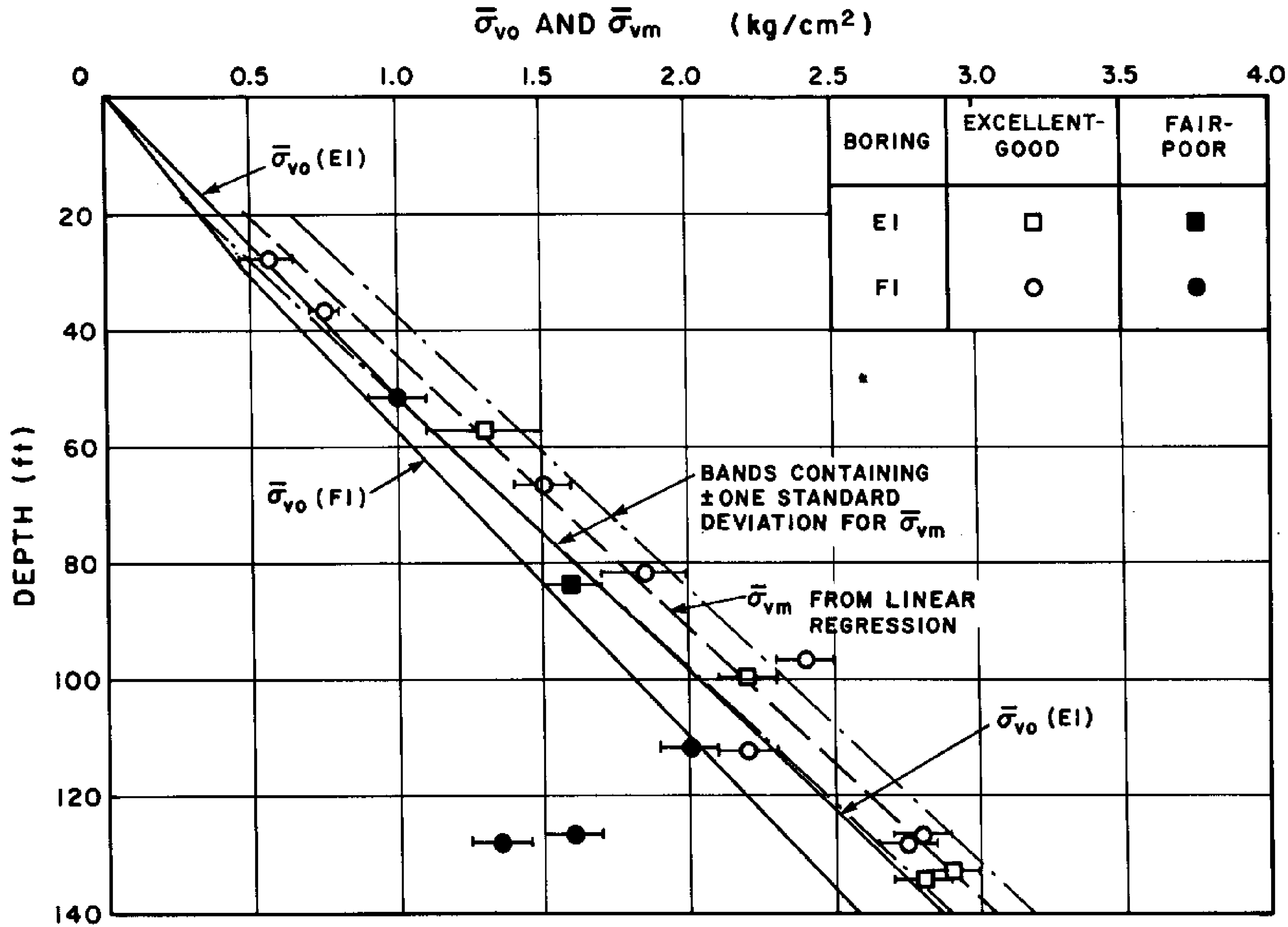
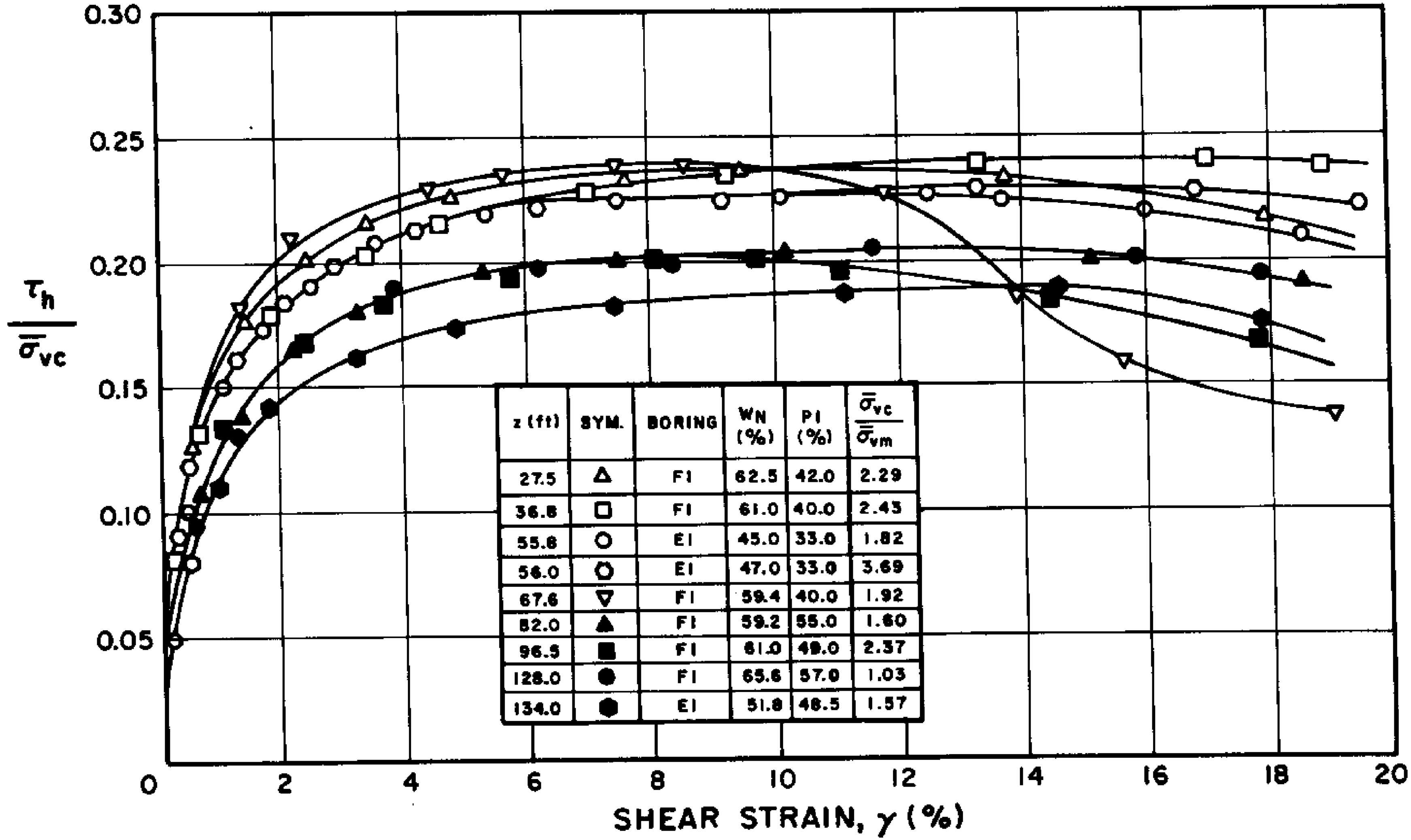


FIGURE 6. STRESS HISTORY OF THE ORINOCO CLAY



z (ft)	SYM.	BORING	W_N (%)	PI (%)	$\frac{\bar{\sigma}_{vc}}{\bar{\sigma}_{vm}}$
27.5	△	FI	62.5	42.0	2.29
36.8	□	FI	61.0	40.0	2.43
55.8	○	EI	45.0	33.0	1.82
56.0	⬡	EI	47.0	33.0	3.69
67.6	▽	FI	59.4	40.0	1.92
82.0	▲	FI	59.2	55.0	1.60
96.5	■	FI	61.0	49.0	2.37
128.0	●	FI	65.6	57.0	1.03
134.0	⬢	EI	51.8	48.5	1.57

FIGURE 7. NORMALIZED STRESS VERSUS STRAIN FROM CK_0UDSS TESTS: N.C. ORINOCO CLAY

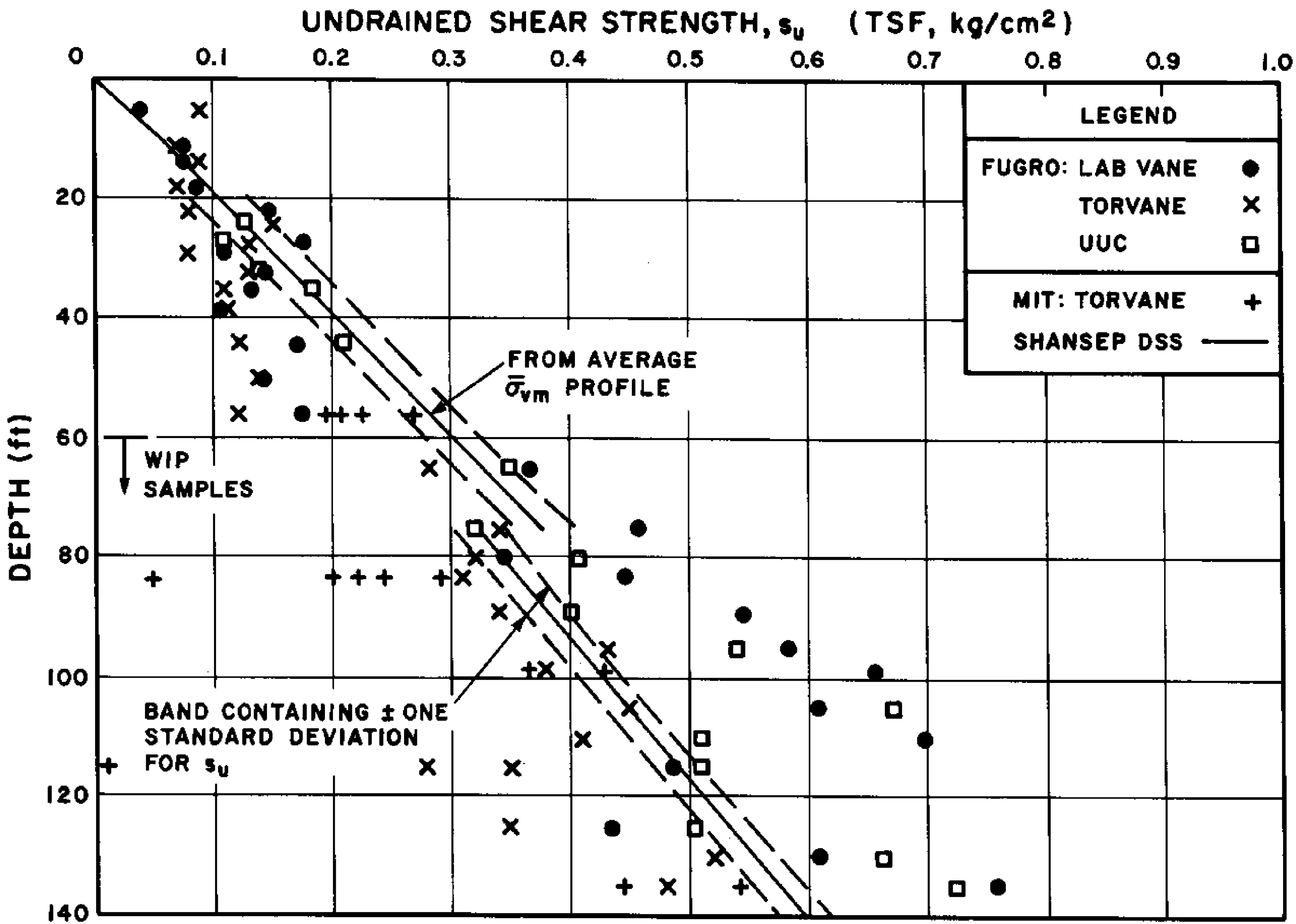


FIGURE 8. COMPARISON OF UNDRAINED STRENGTH DATA: BORING E1

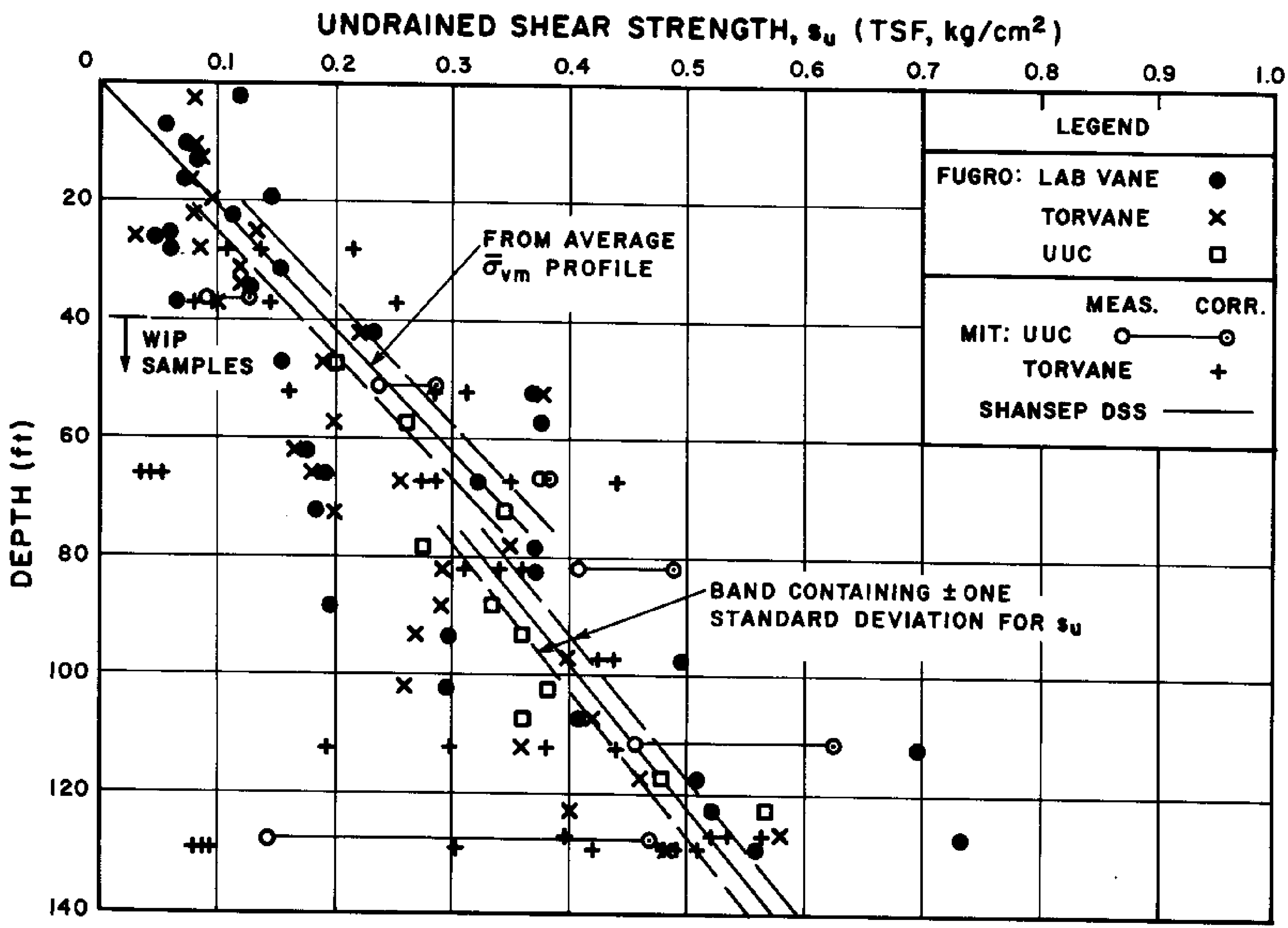


FIGURE 9. COMPARISON OF UNDRAINED STRENGTH DATA: BORING F1

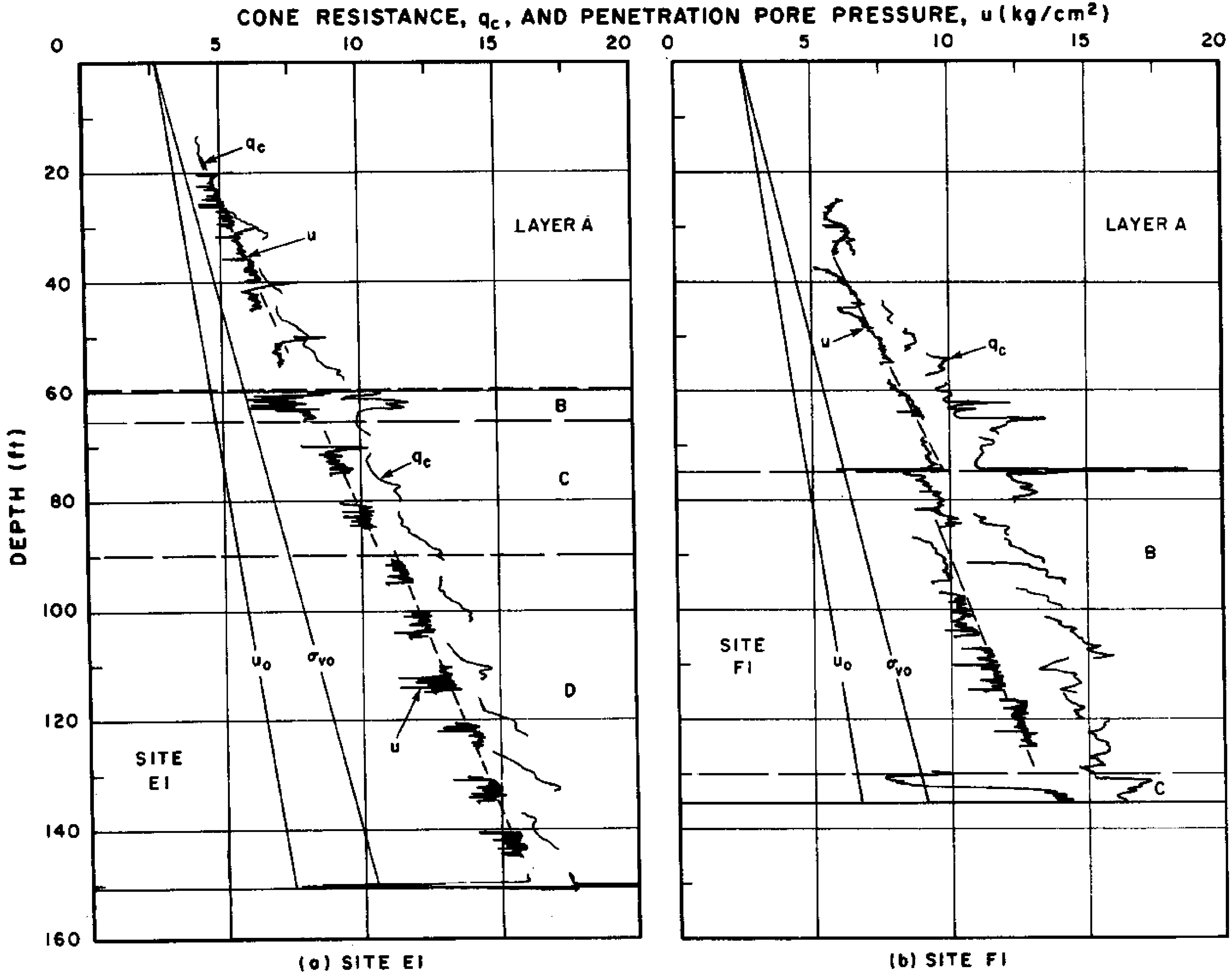


FIGURE 10. CONE RESISTANCE AND PENETRATION PORE PRESSURE AT SITES E1 AND F1

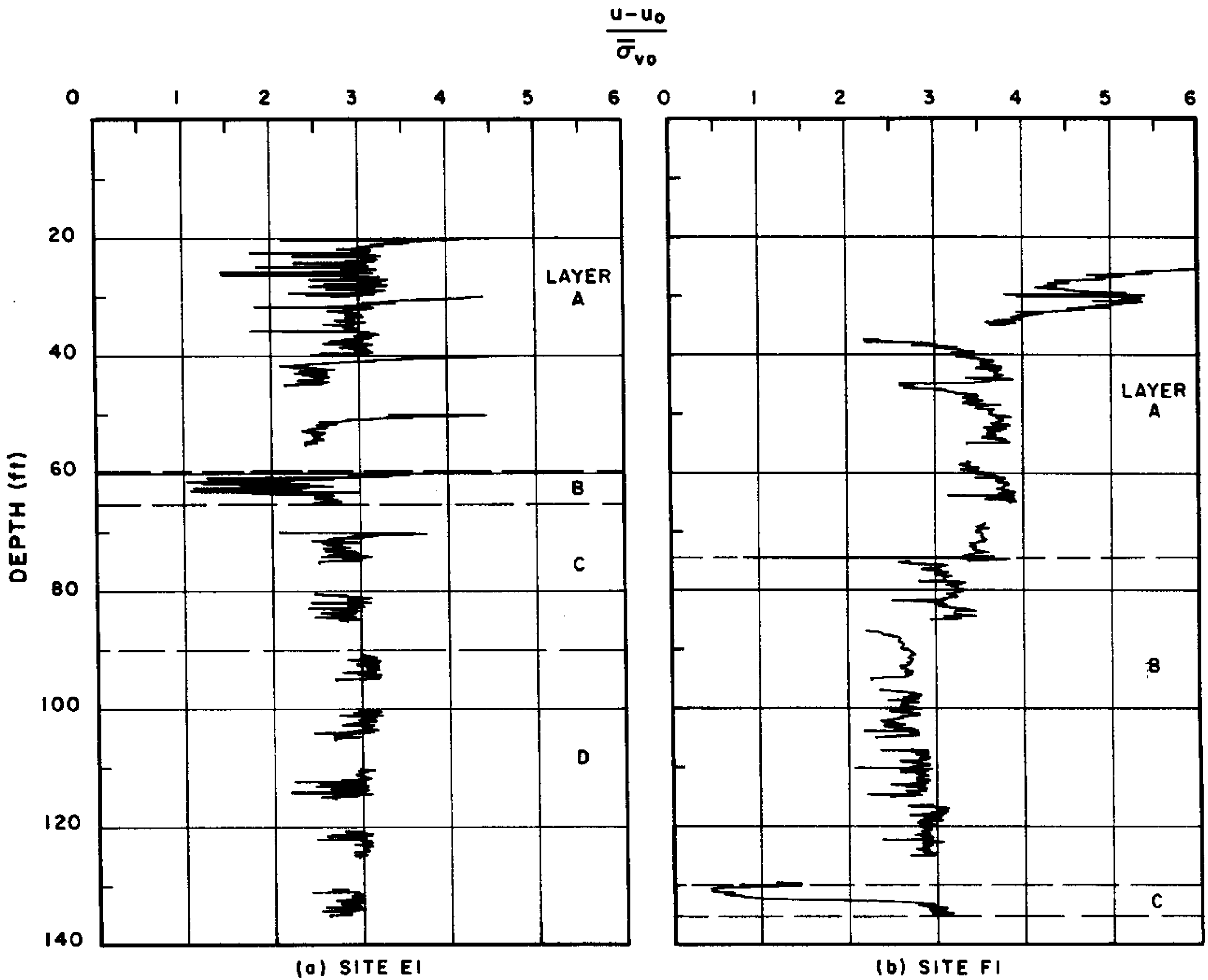


FIGURE 11. NORMALIZED PORE PRESSURE DATA: ORINOCO CLAY