Louisiana Highway Research

PRESSUREMETER CORRELATION STUDY

Louisiana Department of Highways – Research & Development Section

PRESSUREMETER CORRELATION STUDY

by

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Research Report No. 31

Research Project No. 66-2S
Louisiana HPR 1(5)

Conducted by
LOUISIANA DEPARTMENT OF HIGHWAYS
Research and Development Section
In Cooperation with
U. S. Department of Transportation
Federal Highway Administration
BUREAU OF PUBLIC ROADS

October, 1968.

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ACKNOWLEDGMENT

Sincere appreciation is extended to Dr. William G. McIntire, Director, Coastal Studies Institute, and Mr. Rodney D. Adams, Research Instructor for preparation and assistance in interpretation of the Radiographs used in this report.

Sincere appreciation is also extended to Mr. Dave Gedney of the Bureau of Public Roads for his advice and assistance.

SYNOPSIS

This study is an evaluation of the Menard Pressuremeter System for determining in-situ soil strengths for cohesive soils at construction sites.

Results obtained using the pressuremeter were compared to results obtained from use of the unconfined compressive strength test and the consolidated undrained triaxial test.

Data indicated these conclusions:

1. For very soft cohesive soils the correlation between pressuremeter-derived shear strengths and conventional shear strengths for this study was good. It is substantiated by the results at the Houma and Plaquemine test sites and has been noted in investigations by other researchers.

2. At the Sorrento test site there was a not completely-explained increase (not present for conventional tests) in pressuremeter shear strengths with depth. This trend has been noted by other investigators of the pressuremeter; and it is probable that the pressuremeter shear strengths may be closer to the true in-situ strength of the soil than those obtained by conventional methods using open drive sampling techniques.

3. At the Perkins Road and Lake Charles test sites there were considerable differences between pressuremeter-derived shear strengths and conventionally obtained shear strengths, with the pressuremeter results being consistently higher. When the in-place conditions of the soil (as shown by Radiographs, Photographs and Remolded Samples) are considered, it seems highly probable that the pressuremeter shear strengths may be more representative of true in-situ conditions than those shear strengths obtained by conventional tests. It should be noted, however, that for soils in which small strata of a different material, whatever its nature, are interspersed within a core, the strengths estimated probably lie somewhere between the strength of the weakest material and the strength of the strongest material within the test segment. Particular attention should be paid to this fact in any situation where stability against sliding is critical since slip planes could conceivably form along the strata of weak material.

4. Testing of non-cohesive soils with the pressuremeter requires equipment not normally used by the Louisiana Department of Highways. Constant caving of these materials caused difficulties so that no acceptable tests of this material were made during this study. This would probably be a fruitful field for further research with this device.

INTRODUCTION

The determination of in-situ soil strengths at construction sites has long been a problem that has caused considerable difficulty for Construction Engineers. A variety of tests have been developed over a period of time to attempt to obtain an estimate of these conditions. Among these tests are unconfined compression, triaxial, vane shear, and direct shear tests.

This project was initiated as an investigation of the Menard Pressuremeter System. This system has several inherent advantages over conventional undisturbed sampling and testing methods. These are first; that the test is run in the ground on a portion of the soil in-place and, at least in theory, is therefore less susceptible to disturbance in sampling and testing; second, the equipment is portable and gives access to areas not easily sampled by conventional undisturbed sampling equipment; and third, less time is required to obtain results, since the strengths are calculated from data accumulated at the test site. In addition, less total manhours are expended in sampling and testing. Previous studies have indicated that over all costs are reduced. (5)*

* Numbers in parentheses refer to list of references.

PURPOSE

The purpose of this study was to attempt to correlate shear strengths obtained by pressuremeter tests with those obtained for the same soils using conventional methods.

SCOPE

This project was designed to attempt to correlate shear strengths obtained from samples taken by Materials and Tests crews in the normal manner and tested by conventional methods (such as unconfined compression, triaxial, vane shear) with those obtained by use of the Menard Pressuremeter System at the same locations. Where possible the holes from which the samples were taken were utilized for taking the pressuremeter tests.

These correlation tests were made on five projects encompassing a fairly good variety of cohesive soils. The pressuremeter system in theory may be used to determine various other soil properties. However, this project was limited to shear strength correlation.

METHOD OF PROCEDURE

Equipment

The Menard Pressuremeter consists of two main portions, a probe and a pressure-volumeter. (See Figure 1) These two components are connected by plastic tubes through which water and gas pressure are applied.

The probe is a cylindrical metal assembly with rubber membranes attached in such a manner as to effectively form three independent cells. The central measuring cell contains water under gas pressure (CO_2) so that the increase in volume of this cell is measured by the lowering of the water level in the volumeter at the surface. The upper and lower cells are known as guard cells and expand under equal gas pressure (CO_2) from the surface. The purpose of these cells is to minimize the effects of end restraint on the measuring cell.

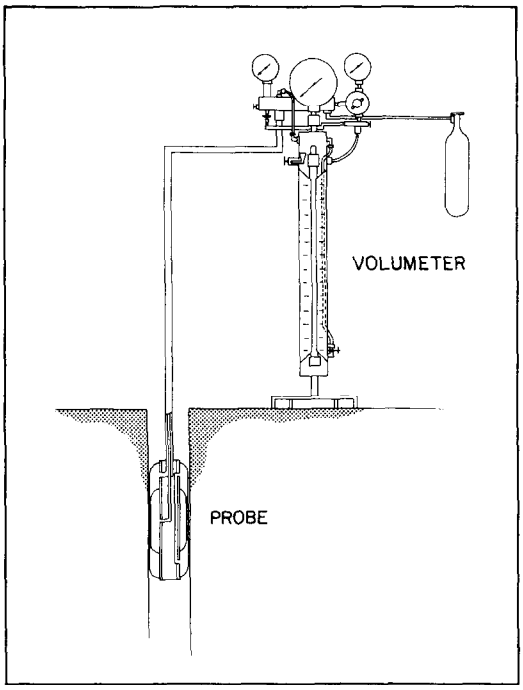


Figure 1

The volumeter is so equipped that a monitored gas pressure can be used to force water into the measuring cell. In addition, a measured gas pressure is applied to the guard cells.

The plastic tube going to the measuring cell is enclosed inside the larger plastic tube going to the guard cells in order to minimize and virtually eliminate expansion of the tube going to the measuring cell. Any expansion of this tube would, of course, lead to erroneous estimates as to the amount of water being forced into the measuring cell.

The pressuremeter probe applies a radial pressure to the soil around the probe. The pressure gauge and volumeter allow simultaneous measurement of pressure and volume change of the probe.

Operational Concept

Data for analysis is obtained by applying increments of pressure to the water in the measuring cell of the pressuremeter and recording the corresponding changes in volume at 15, 30, and 60 seconds after application of the pressure. Tests are taken throughout the depth of the hole with a common interval between tests of three feet.

The information obtained from these tests is plotted on a graph of pressure versus volume and ideally yields curves similar to those shown in Figure 2.

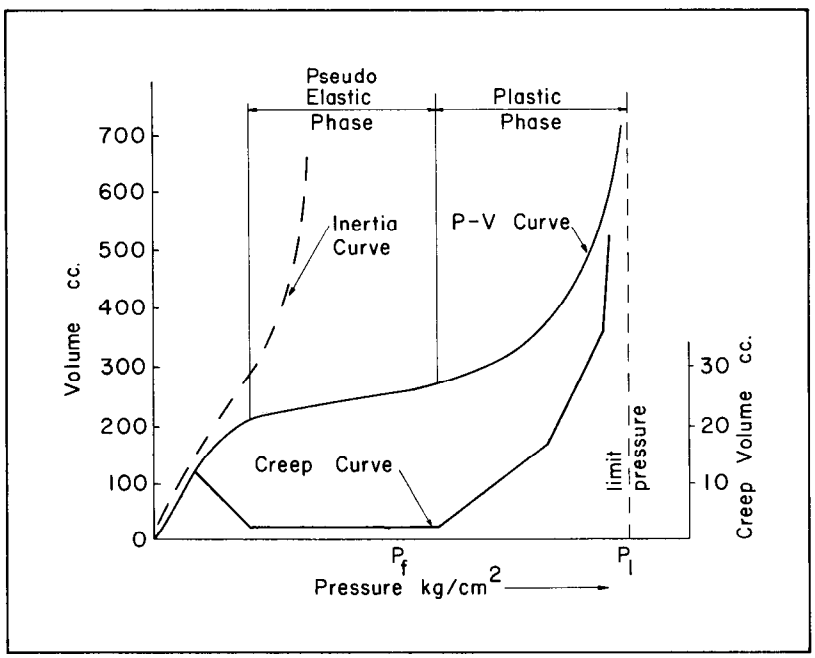


Figure 2

The upper solid curve is a plot of the 60 second readings versus the volume as read on the volumeter. You will note that there is a sloping lead in phase where the earth pressure is being restored to that which existed prior to the removal of the soil from the hole. This is followed by a linear or nearly linear phase. From this portion of the curve which would be an elastic stage if soil were an elastic isotropic material an approximation of the Young's Modulus of the material is obtained. Following this pseudo-elastic portion of the curve is an area where the volume change increases rapidly, and the curve asymptotically approaches a vertical line. It is at this point that the so called limit pressure (P_1) is reached. This gives a direct indication of the bearing capacity of the soil.

There are several factors that must be considered, however, in determining a final limit pressure. First, the resistance of the probe itself must be accounted for. This is done by running the same type of curve on the probe alone without insertion in the ground. (dashed curve Figure 2) The pressure required for this expansion of the probe may then be obtained for any volume and applied as a correction to the observed pressure. Second, the pressure due to the head of water at the particular depth must be added where necessary. Third, a correction for the expansion of the water tube must be made if necessary. For normal cohesive soils this correction may be neglected due to the construction of the tube previously explained under the heading Equipment. With these corrections a corrected pressure versus volume curve may be plotted which more nearly indicates the true pressure volume ratios.

The lower solid curve on Figure 2 is a plot of the volume change between the 30 second and 60 second readings. This plot shows the tendency of the material to deform with time. The point where this curve makes a definite upward break is designated as the creep pressure (P_f) and usually correlates fairly closely with the end of the pseudo-elastic portion of the pressure volume curve.

Calculations

The method of calculation of the shear strength of soils from data obtained by the pressuremeter test is explained by Menard in a paper entitled "Rules for the Calculation and Design of Foundation Elements on the Basis of Pressuremeter Investigation of the Ground." The formula for shear strength as postulated by Menard in this paper is:

$$S_o = \frac{P_l - P_o}{2K_B}$$

where = P_l = the limit pressure as obtained from the test

P_o = the lateral pressure at rest at the depth of the test

K_B = a coefficient dependent on the stiffness $\frac{E}{P_l}$ of the soil and the soil structure.

The calculations involved in obtaining E (approximate Young's Modulus) are explained in a paper by Menard entitled "The Geocel Pressuremeter - Interpretation of a Pressuremeter Test." A good discussion of the theoretical considerations involved in the pressuremeter test is given by Gibson and Anderson in reference 3.

Field Data Collection

Most of the data obtained were taken with an NX size probe inserted into the hole from which the Shelby tube sample had been extracted. Readings were taken at the approximate center of the increment of hole from which the sample was taken. (Figure 3). These tests were made immediately after the removal of the sample by the Shelby tube and not after the entire hole had been sampled.

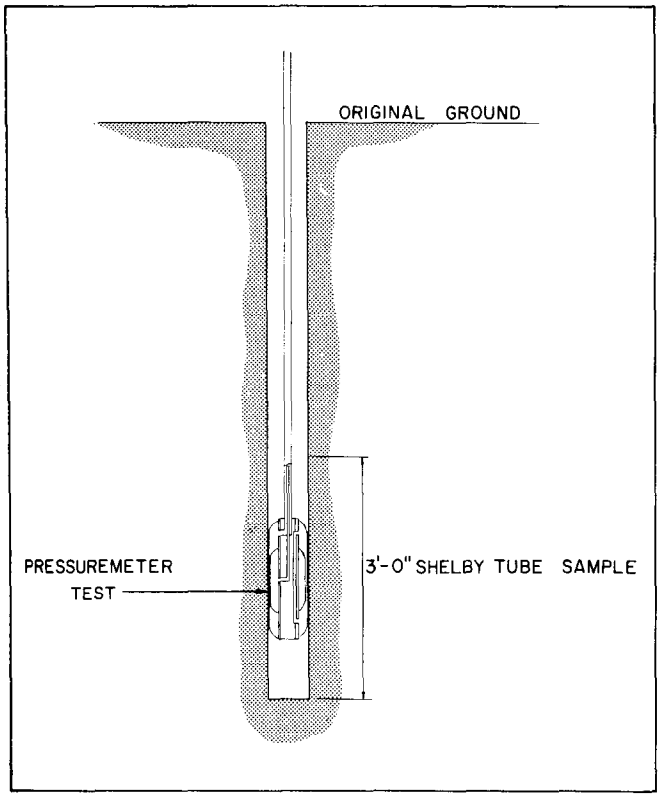


Figure 3

A limited number of samples were taken from augered holes with cores taken from holes as close as practicable to the augered holes for conventional testing. Data were obtained from projects consisting of a variety of cohesive soils.

During the collection of these data several peculiarities of the device or problems with the device were noted. These are as follows:

1. The probe is extremely sensitive to hole size. A hole which is too large does not allow sufficient pressure to be applied to the probe to estimate the limit pressure before all the water in the volumeter is forced into the probe. Conversely if the hole is too small and the probe is forced into it, the soil around the hole is compressed and may cause misleading readings when pressure is applied, especially in the lower range of pressures.

2. When the probe is used in a material which contains angular aggregate particles it is necessary to apply a heavy plastic tape to the probe to keep it from bursting. Inertia curves with the taped probe must be run to apply the proper correction to the measured pressure-volume curve. It was originally recommended by the lessor of the pressuremeter that a slotted steel casing be placed over the probe to protect it. This was not found to be a practical field method, however, especially when used in the relatively soft soils tested in this study. The casing tended to cause excessive disturbance to the soil during placement, and to require so large a hole that a sufficient volume of water was not available in the volumeter to get an accurate estimate of the limit pressure. In addition, the casing acquired a permanent outward bulge after several tests which tended to aggravate these difficulties. It is probable that these difficulties might not be so pronounced in harder soil or rock.

3. In very soft material where the inertia of the probe approaches the strength of the soil the probe must be used without the outer rubber sheath; and, in addition, a softer rubber membrane may be used for the measuring cell.

4. In some soils the holes tended to decrease in size especially at the greater depths causing difficulty in inserting the probe and leading to difficulties as explained in item (1) above.

5. Due to the sensitivity to hole size of the pressuremeter, the disturbance created by taking more than one core before testing with the pressuremeter was often sufficient to enlarge the hole so as to invalidate the information obtained by the pressuremeter at the upper sampling and testing levels.

6. For many of the tests made, a sufficient quantity of water was not available in the volumeter to reach the actual limit pressure. The original method recommended by the lessor to overcome this difficulty was to extrapolate the curved portion of the pressure-volume line to estimate this value. This method was of course very susceptible to error and involved a large amount of the "human factor." Two other problems also occurred. These were first, that many curves did not exhibit the typical lead in phase on the P-V plot and second, that the linear portion of some curves was hard to delineate. In order to eliminate these problems it was suggested by the lessor in a letter on February 24, 1967, that a different approach be used. The pertinent portion of this letter is quoted below:

"For determining the limit pressure (P_l) we have adopted a method which assumes that the plastic phase of the P-V plot is hyperbolic. This method was derived from the paper "Hyperbolic Stress-Strain Response: Cohesive Soils" by Robert L. Kondner presented in the February, 1963 Journal of the Soil Mechanics and Foundations Division of A. S. C. E. We have modified the method proposed by Mr. Kondner to fit our need.

The step by step procedure used to determine P_l is given below:

1. Plot the corrected P-V curve.

2. Extend the linear portion (pseudo-elastic phase) of the plot to P=o. Assume that the intercept on the volume axis is the corrected origin (V=o).

3. Determine the coordinates of several points on the corrected P-V curve with respect to the corrected origin.

4. Prepare a plot of V versus V/P for the points in Step 3 ($\frac{V}{P}$ as the ordinate and V as the abscissa).

Points from the linear or pseudo-elastic portion of the corrected P-V curve should plot as a horizontal line. If the plastic phase of the P-V plot is truly hyperbolic it should plot as a sloping straight line. Whether or not the points plot as a straight line is a check of the validity of this method. Normally in the transition zone between the pseudo-elastic and plastic phases the points may not fall on either the horizontal or sloping straight line.

If the angle between the abscissa and sloping straight line is designated as x the ultimate limit pressure is equal to $\frac{1}{\tan x}$.

The creep pressure is indicated by the intersection between the horizontal and sloping straight lines. The coordinates of this intersection can be used to calculate the creep pressure. The The ultimate limit pressure is attained after inducing large strains in the soil tests. The magnitude of strain at the ultimate limit pressure is quite large in comparison with the strains induced during conventional laboratory tests. For this reason, Menard has suggested that the limit pressure be determined at

a probe volume equal to twice the initial volume. For the NX probe which you are using the initial volume would equal 808cm^3 plus the volume increase required to make contact with the walls of the borehole. The volume increase required is taken as the intercept with the volume axis when the linear portion of the corrected P V plot is extended to P=o. Since the probe is not expanded to twice its initial volume when the NX probe is used, the test results must be extrapolated. The V versus V/P plot can be used for this purpose. After determining the value of V/P corresponding to a volume equal to the initial volume, a corresponding pressure can be calculated. The value of pressure determined by this means is termed the "Limit pressure" and is used for design and for comparison with laboratory tests."

This method was tried and appeared to give satisfactory results.

7. Tests in sand will require special treatment. Difficulties are encountered in sand due to caving in the holes. Normal procedure for sampling sands by the Department consists of the driving of a 2 inch split tube and use of the number of blows for penetration to estimate the strength of the sand. Caving of the sand upon removal of the sampling device prevented testing by pressuremeter in the 2 inch holes. When the holes were cleaned out and enlarged to 3 inch size it was found that excessive cavitation from previous caving of the holes caused erratic results even if no new caving occurred. It is probable that the caving could be prevented in some manner, perhaps by use of a bentonite slurry. Testing could then be accomplished by the pressuremeter in these soils.

Laboratory Testing

At the outset of the project it was recommended by the lessors of the pressuremeter that shear strengths obtained by the pressuremeter be compared with the results of conventional unconfined compression tests. It was later recommended that the results be compared with consolidated undrained triaxial tests results.

Accordingly some of the test sites were tested only in unconfined compression and some were tested only by consolidated undrained triaxial tests. For some test sites both types of test were run.

A. Houma, Louisiana Area

Figures 4 and 5 shows the results of the pressuremeter tests at various depths and at two locations plotted against the results of unconfined compression tests made in the laboratory. The shear strength of this material estimated by either method is relatively small and it may be noted that for the most part the results by the two methods correlate very well. The soil on this project ranged from a very soft silty clay to a silty loam material with many of the individual samples containing organic material. The organic content ranged from practically zero to as high as 65%. A field description of the soils, the liquid limit, plastic index, organic content and natural moisture content are shown in Table 1. (Appendix)

B. Plaquemine, Louisiana Area

Figure 6 shows a plot of pressuremeter results at this location versus unconfined compressive strength results and Figure 7 shows the same pressuremeter results plotted against the results of vane shear tests.

Again the correlation is fairly good both between the pressuremeter and the unconfined test results and between the pressuremeter tests and vane shear results. In general the results of the pressuremeter fall between the unconfined test results and the vane shear test results and therefore correlate better with either of the two conventional tests than the conventional tests correlate with each other.

The soil at this location ranged from a heavy clay to a silty clay loam. A field description of the soil, the liquid limit, plastic index, organic content and natural moisture content are shown in Table 2. (Appendix)

C. Sorrento, Louisiana Area

Figure 8 shows a plot of pressuremeter test shear strengths versus shear strengths obtained by unconfined compressive strength tests. It may be noted from this curve that the unconfined compressive strength results are equal to or greater than those for the pressuremeter near the surface but become less than the pressuremeter test results as the depth increases.

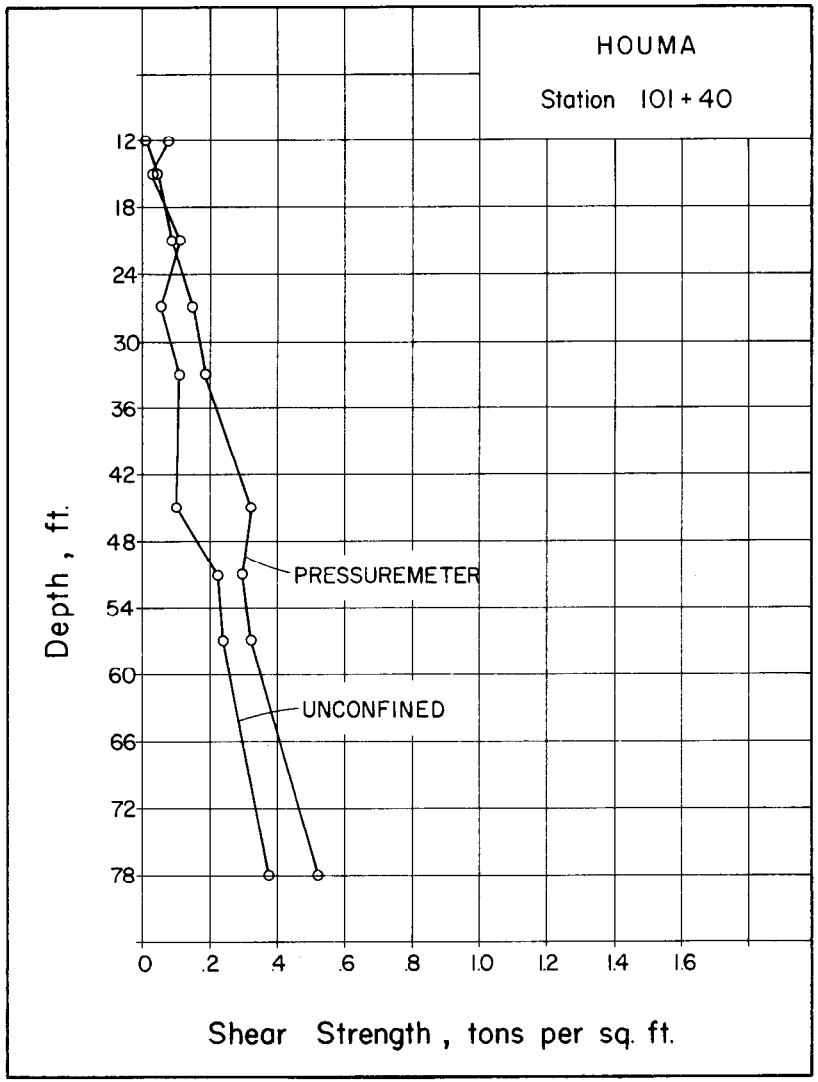


Figure 4

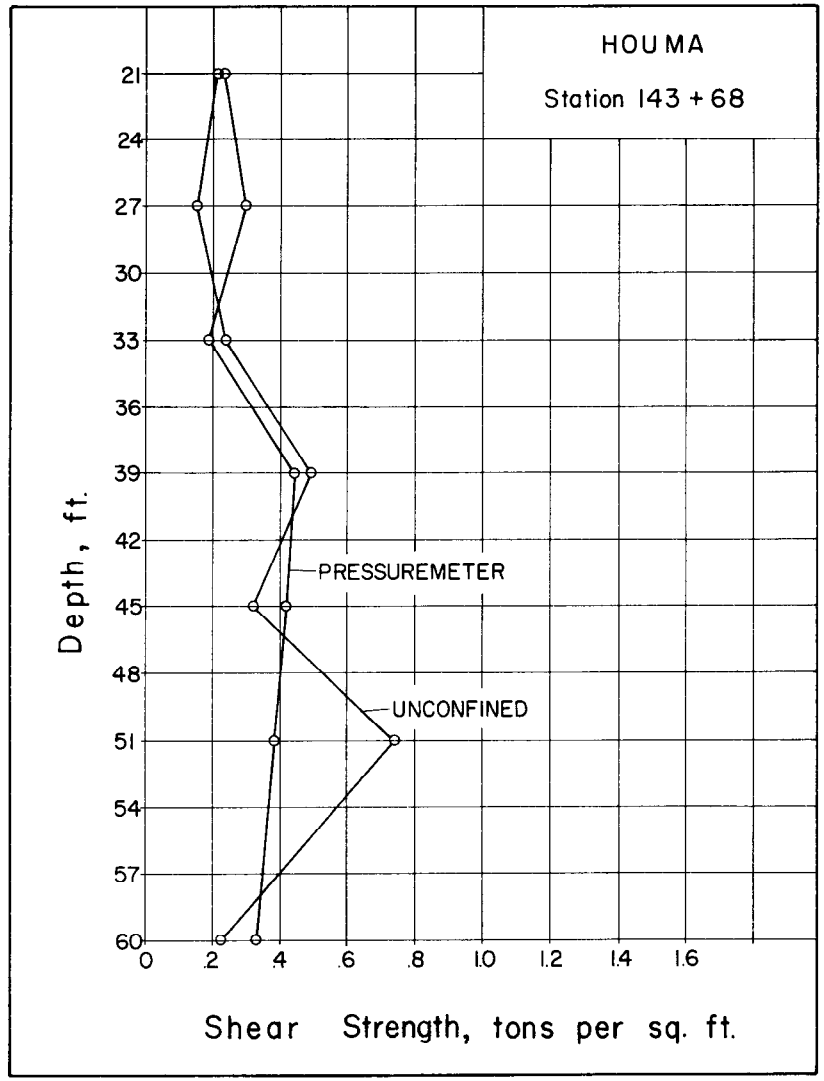


Figure 5

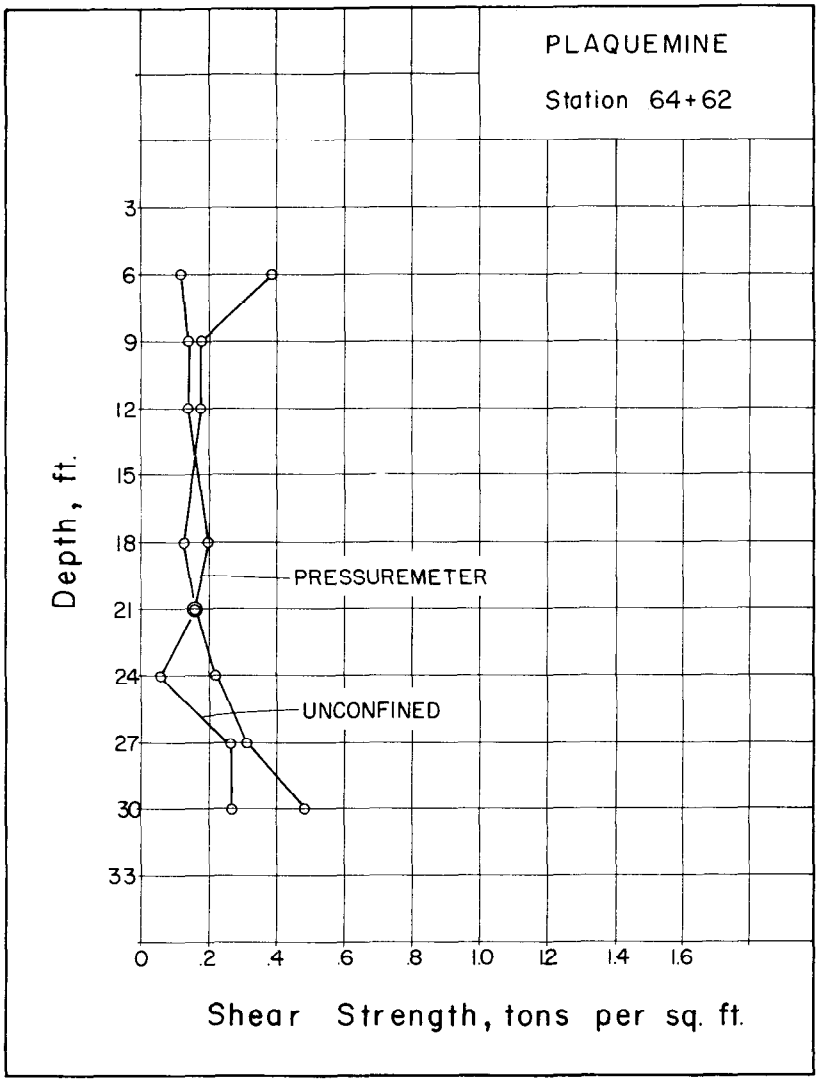


Figure 6

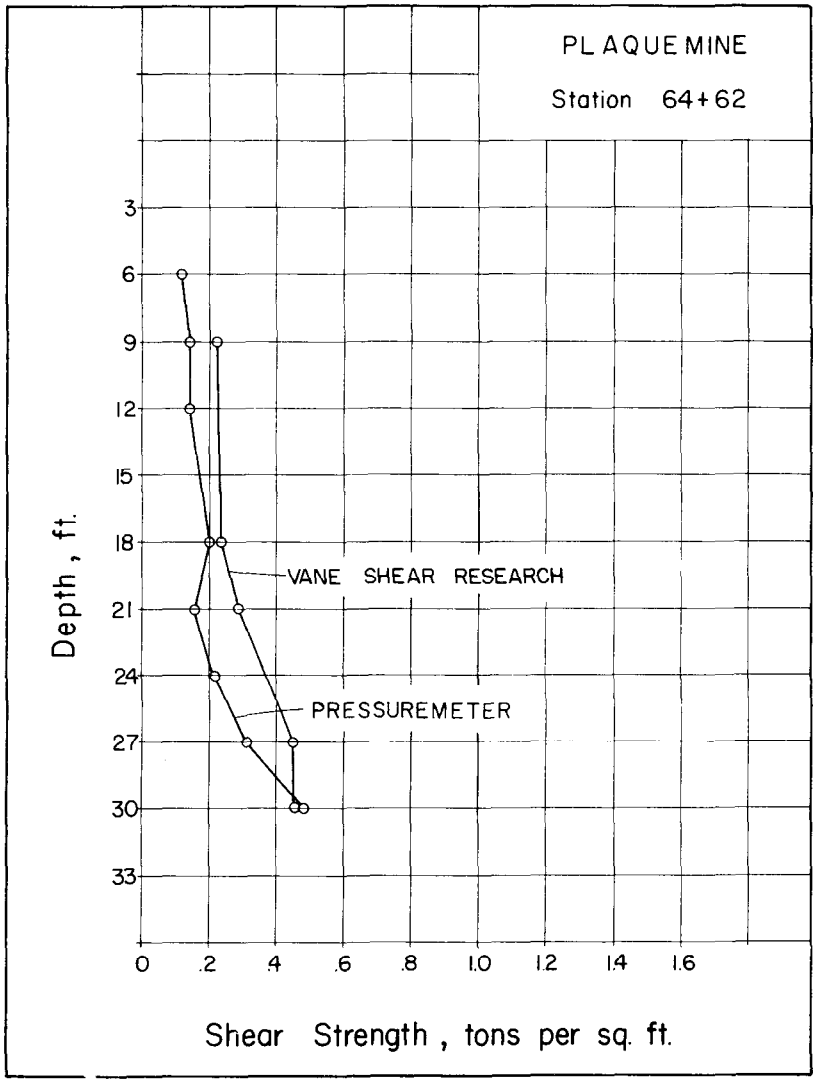


Figure 7

Figure 9 shows pressuremeter tests at the same location plotted against the consolidated undrained triaxial test results. As would be expected, the triaxial results are higher than the results of the unconfined tests. However, the triaxial results are still somewhat lower than the pressuremeter results at the lower elevations.

For purposes of comparison the shear strength results as obtained on successive days by the pressuremeter in holes approximately six feet apart are shown in Figure 10. These tests were taken near the location of the tests shown in Figures 8 and 9. Field soil descriptions, natural moisture contents, liquid limits and plasticity indices are shown in Table 3. (Appendix)

D. Perkins Road, Baton Rouge, Louisiana

Figure 11 is a plot of shear strengths obtained by the pressuremeter versus shear strengths obtained by consolidated undrained triaxial testing. The trend for the two curves is very similar with each increasing and decreasing at the same points. There is, however, a considerable disparity in the actual strengths recorded with the pressuremeter strengths, on the average, about 1½ times as great as the triaxial strengths. It is likely that the difference in magnitude of shear strength estimates by the two methods may be due to a number of factors. Since this same phenomonen is encountered to a certain extent at the Sorrento test site and at the Lake Charles test site, the results of which will be presented next, a discussion of the possible causitive factors will be delayed until the data for the Lake Charles section are presented. The soils for these tests ranged from medium stiff silty clay to stiff silty clay. The natural moisture contents averaged somewhat lower than those for the projects previously discussed. Field soil descriptions, natural moisture contents, liquid limits and plasticity indices are shown in Table 4. (Appendix)

E. Lake Charles, Louisiana Area

Figure 12 shows a plot of shear strengths obtained by the pressuremeter versus those obtained by unconfined compression testing and Figure 13 shows pressuremeter test results plotted against consolidated undrained triaxial test results. Here, as at the Perkins Road test site, there is a considerable difference between the shear strengths estimated by pressuremeter test results and those obtained by unconfined or triaxial testing.

As might be expected the triaxial test results are higher than the unconfined test results. They are, however, still well below those determined by the pressuremeter test. Field soil classifications, natural moisture contents, liquid limits and plasticity indices are shown in Table 5. (Appendix)

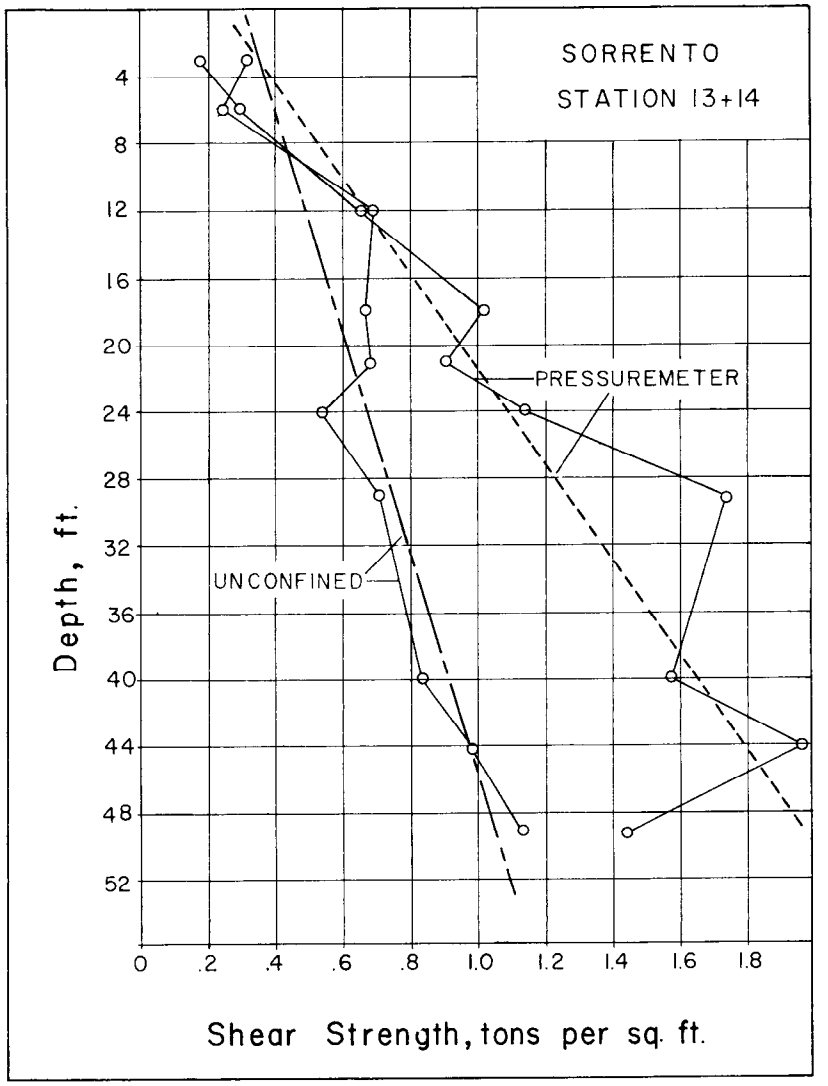


Figure 8

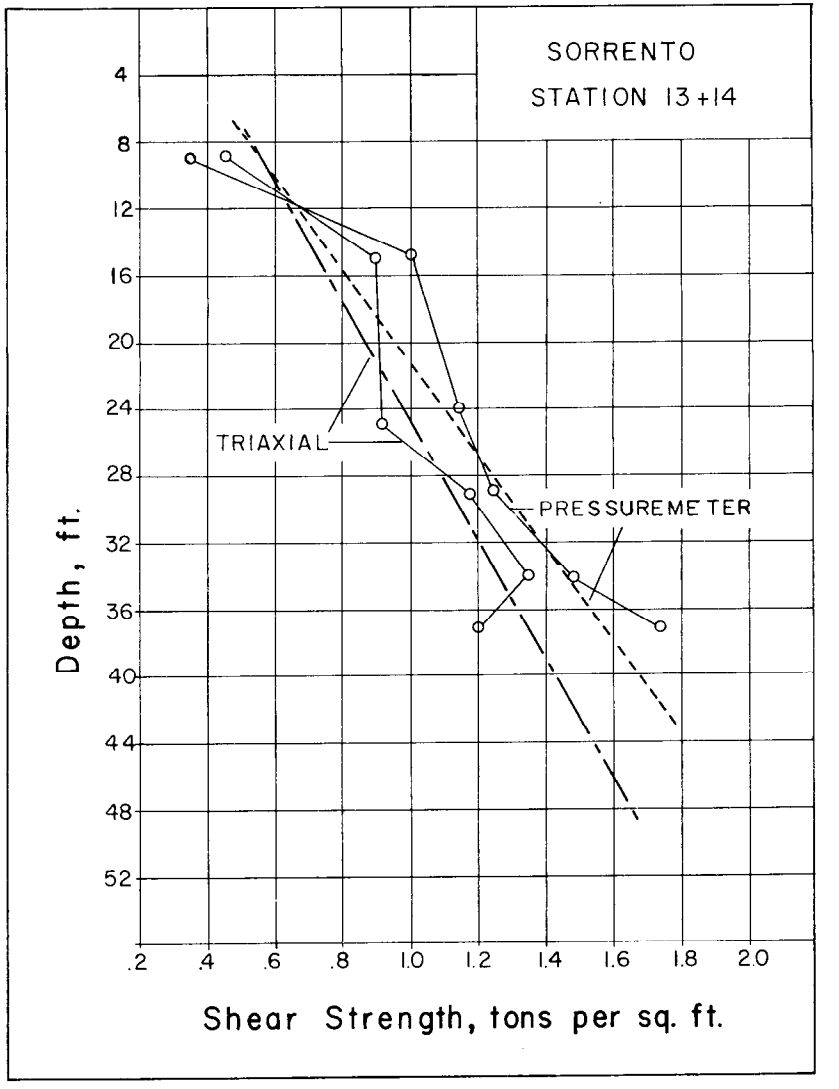


Figure 9

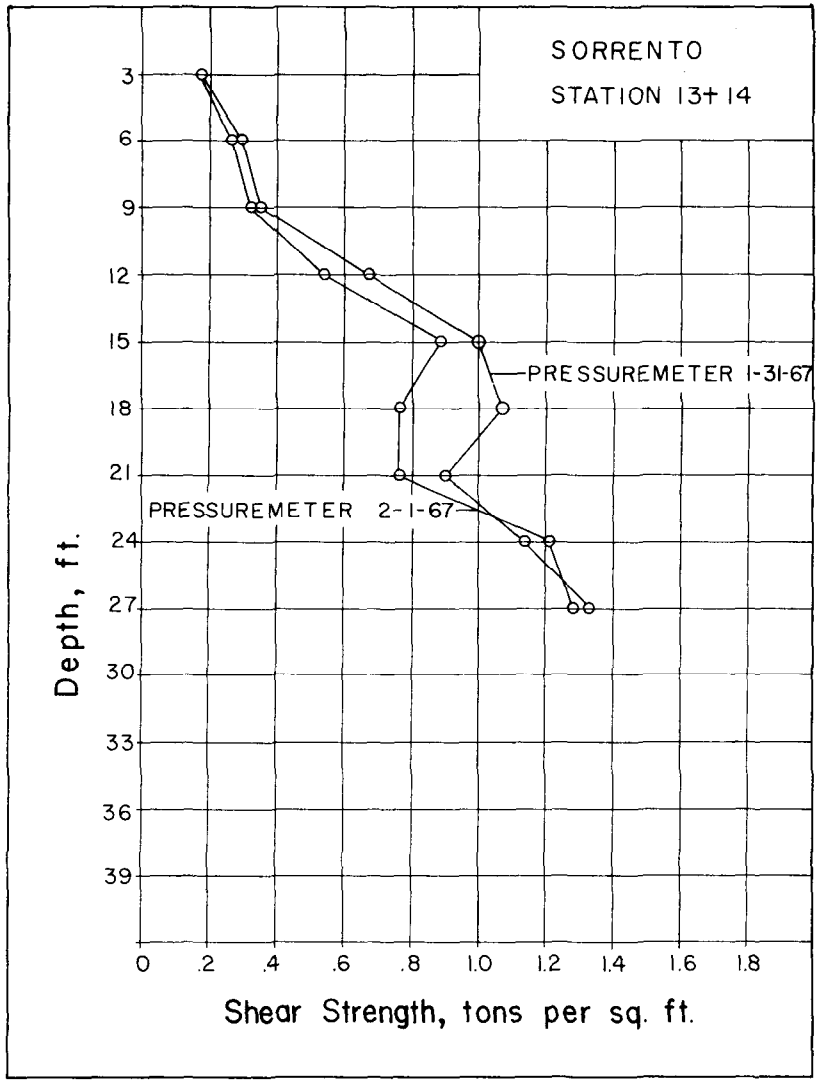


Figure 10

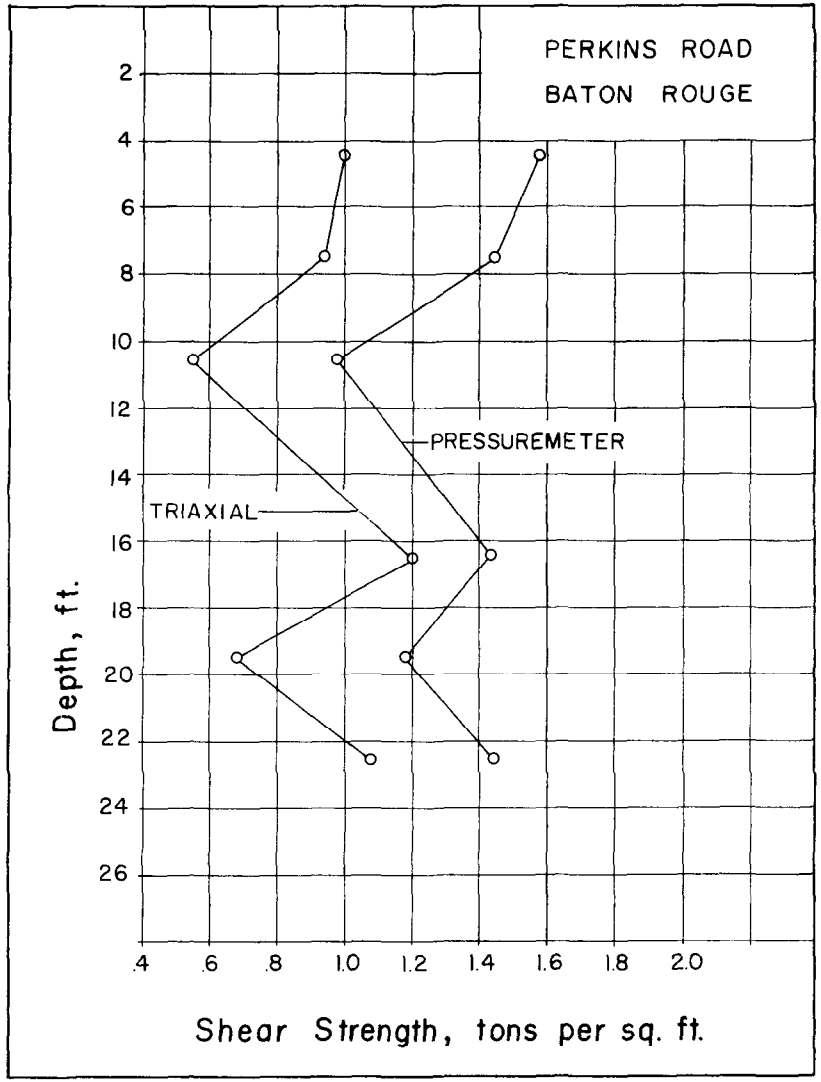


Figure 11

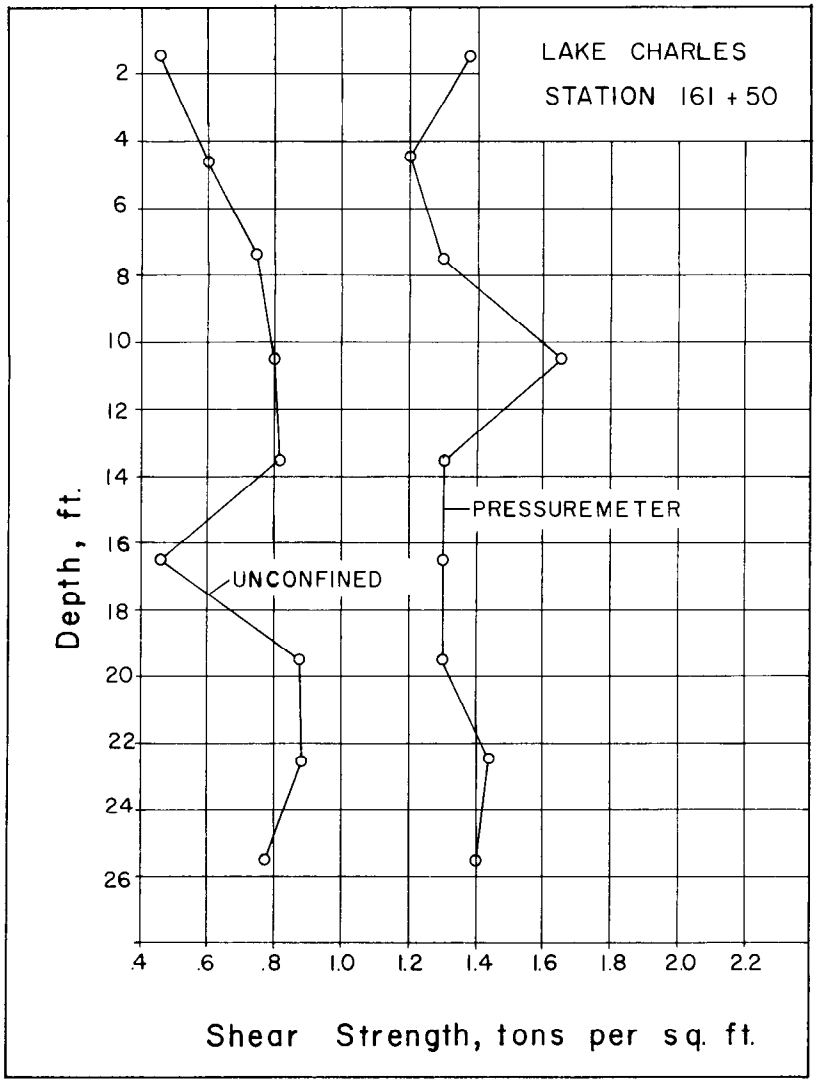


Figure 12

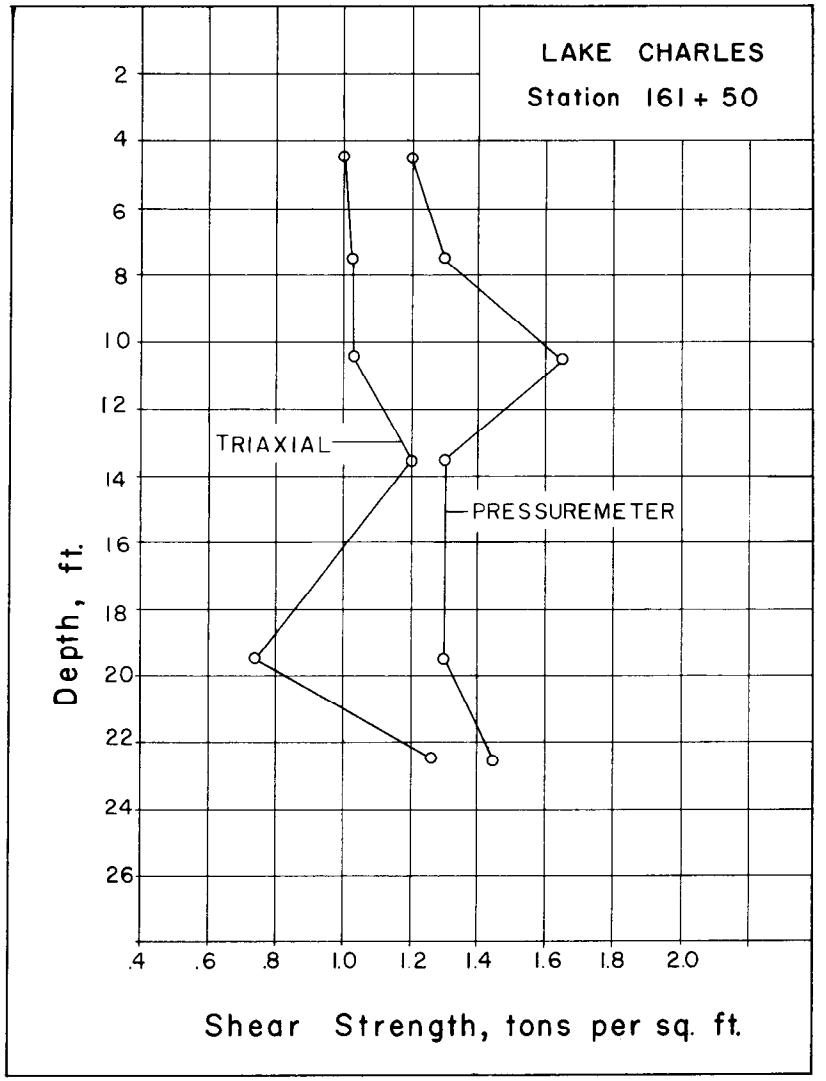


Figure 13

DISCUSSION OF RESULTS

As noted in the presentation of results for the various areas the correlation between pressuremeter results and conventional results for the soft materials at Houma and Plaquemine were very good. However, as stronger soils were encountered a definite disparity in results by the two methods developed with the pressuremeter showing greater strengths. This difference was noticeable at larger depths in the Sorrento area and at all depths in the Perkins Roads and Lake Charles areas.

In order to attempt to establish the reason or reasons for this difference, samples of the soils at two elevations at the Lake Charles test site and at two elevations at the Perkins Road test site were secured. This material was broken down to a maximum No. 4 sieve size and remolded at the natural moisture content and density of the cores previously tested from these elevations.

As an additional check on the cause of the variance between the pressuremeter shear strengths and conventional shear strengths, thin slices were taken from the middle of several cores from the Houma investigation, the Sorrento investigation, the Perkins Road investigation, and the Lake Charles investigation. Through the cooperation of the Coastal Studies Institute of Louisiana State University, radiographs were made of these slices. In addition, photographs were made of these thin sections.

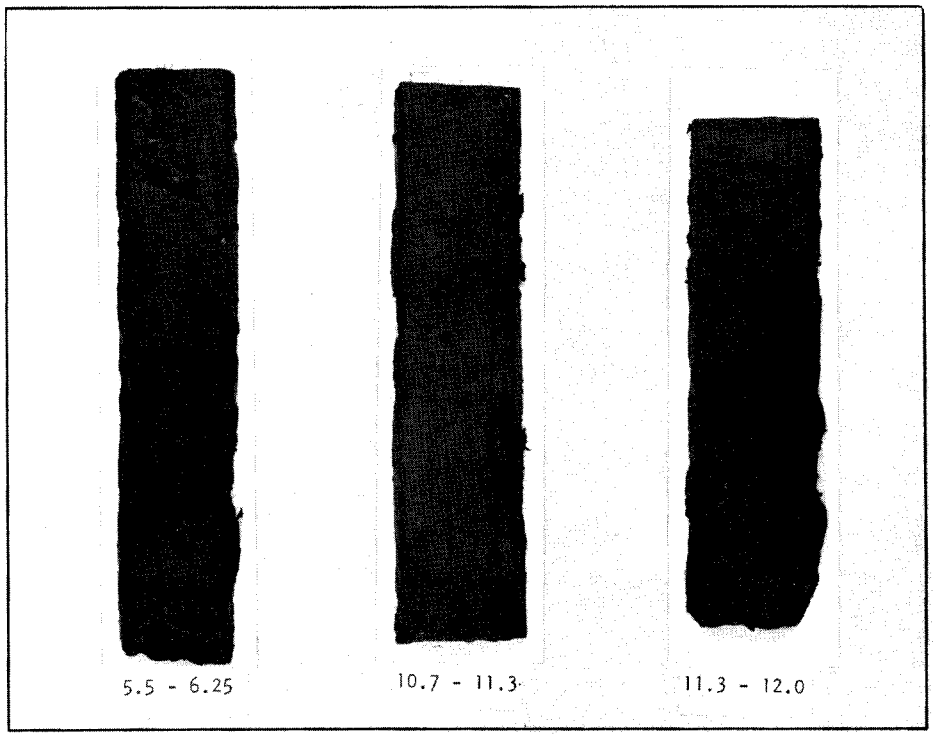
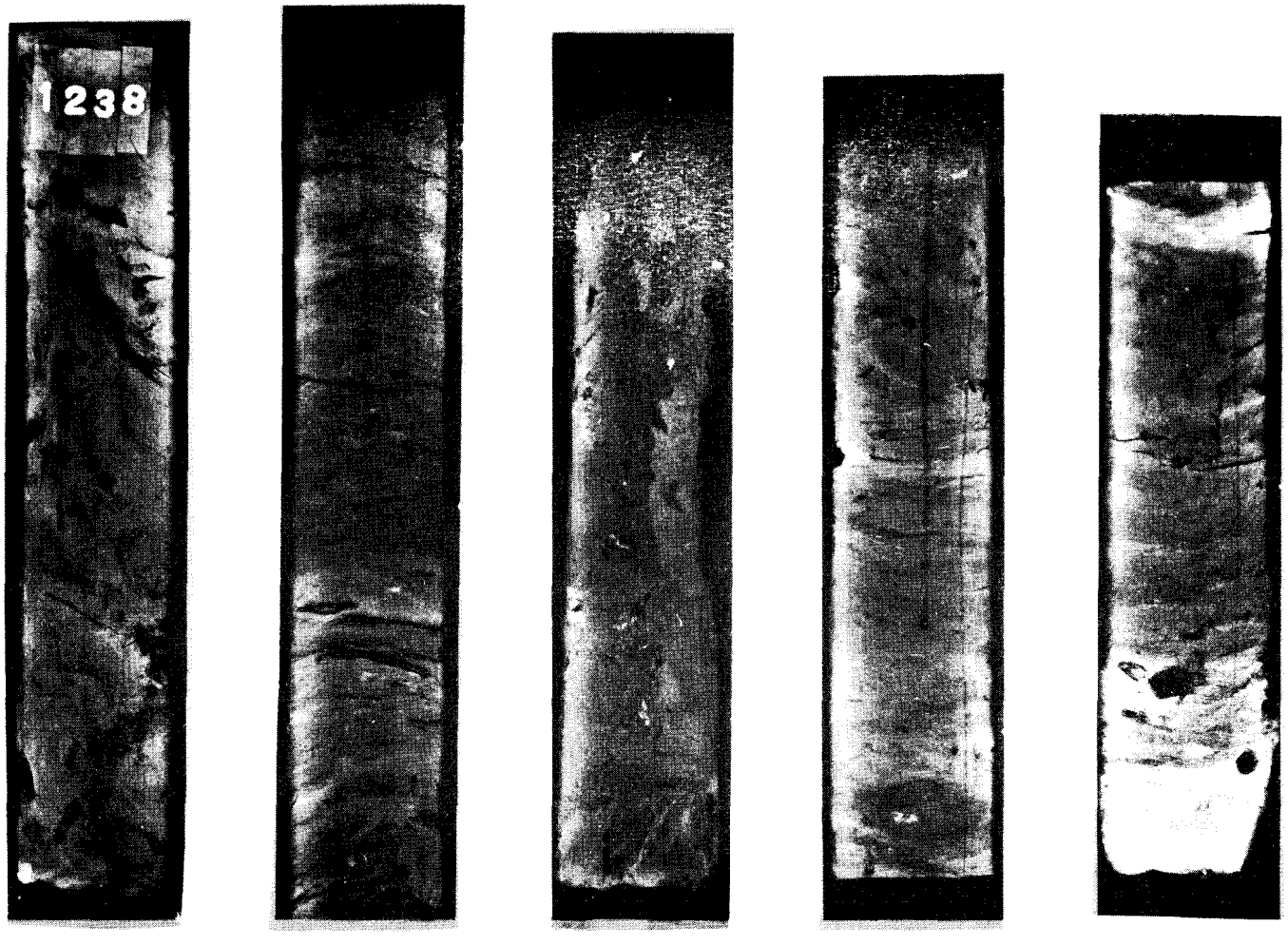


Figure 14 - Houma Photographs

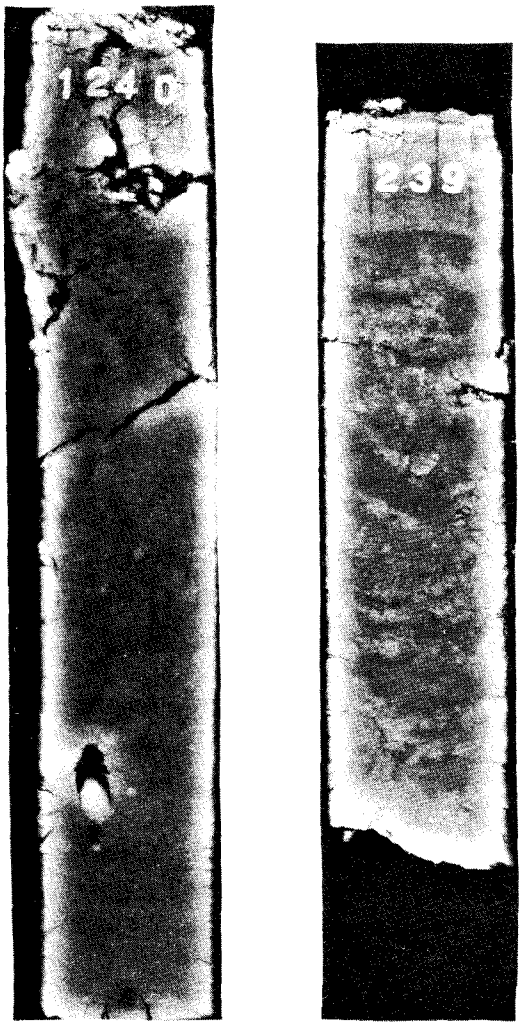


5.3' - 6.3' 6.3' - 7.9' 10.0' - 10.7' 10.7' - 11.3' 11.3' - 12.0

Figure 15 - Houma Radiographs

The photographs and visual observation of the cores from the Houma area indicate a fairly homogeneous mixture of clay and organic matter, (Figure 14). The radiographs, however, reveal that there are spot concentrations of heavier organic material and some wood fragments within the material (Figure 15). The concentrations are not generally layers, but rather represent a random orientation of highly organic material.

It might be expected that these conditions would result in a sample that, on the average, would yield very similar results from any segment tested within the three foot sample obtained; and that pressuremeter shear strengths, which are measured over an 18 inch test length, should correlate well with conventional tests results taken on a smaller segment of the sample.



13.0' - 15.0' *22.0' - 25.0'*

Figure 16 - Sorrento Radiographs

Figures 4 and 5 indicate that this supposition is correct since a reasonable correlation is established between pressuremeter estimated shear strengths and those estimated by unconfined compressiv strength tests.

The soils tested in the Plaquemine area are very similar to those tested at Houma and again show a reasonable correlation between pressuremeter shear strengths and unconfined shear strengths.

Figure 16 together with visual observation indicates the presence of concentrations of material consisting primarily of calcareous material and iron oxide with some manganese oxide also present at the Sorrento test site. These concentrations occur both as nodules and as large conglomerate concentrations. The more extensive concentrations are especially prevalent in the cores at elevations below about 15 feet. (See Figure 16)

Another characteristic of the soils being tested which affects the overall strength pattern is the presence of pockets of soil of a different nature than that of the predominent material. Figure 16 shows a silt concentration near the top of a core

which is a predominently silty clay material. Figure 8 indicates that the pressuremeter tests results correlate very well with the results of unconfined compressive results to a depth of about 12 feet. From this point downward the shear strengths start to deviate from each other with the pressuremeter reflecting consistently higher shear strengths. This pattern is repeated in Figure 9 which is a plot of consolidated undrained triaxial test results versus pressuremeter test results. Average curves fitted by computer demonstrate this trend for both pressuremeter versus unconfined and pressuremeter versus consolidated undrained triaxial tests. These curves are plotted as dashed lines on Figures 8 and 9.

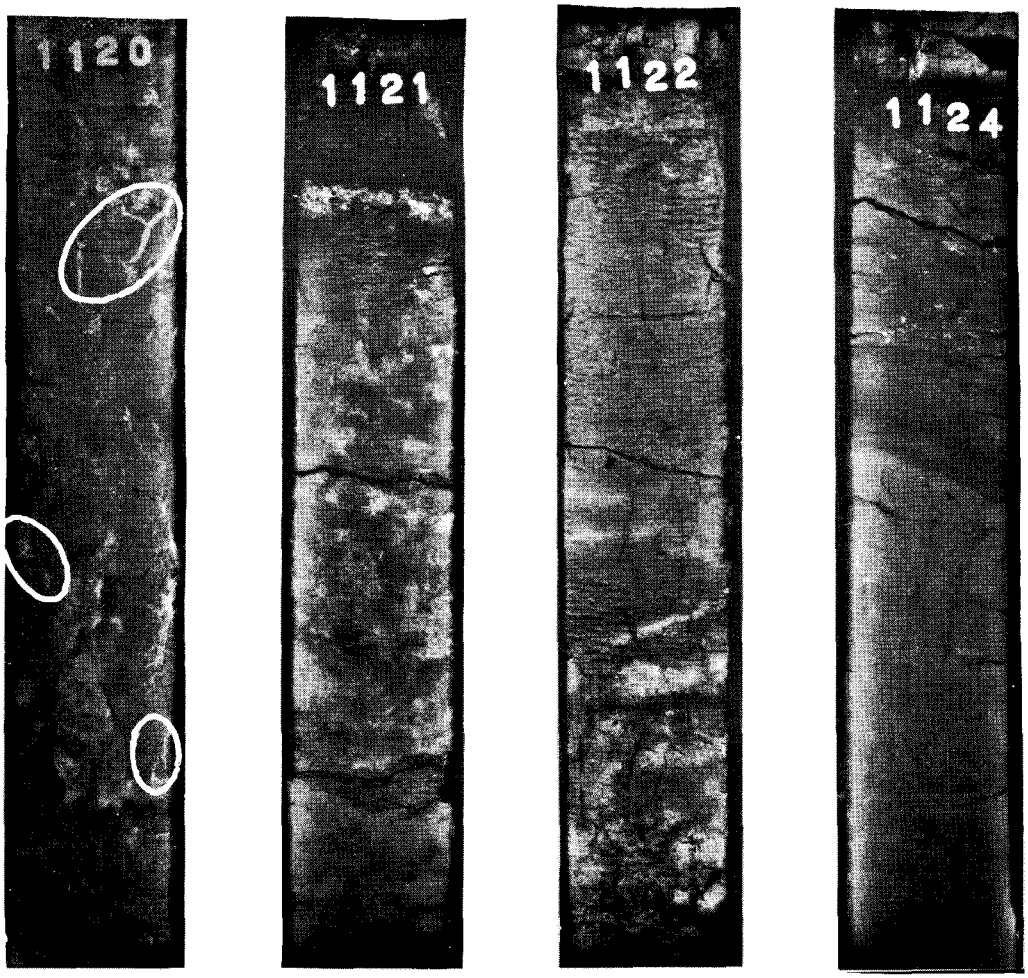
This trend has been noted in previous tests with the pressuremeter. This difference in strengths has not been fully explained, but is attributed in part to excessive sample disturbance at greater depths using open drive sampling, and the fact that shear strength built up on a vertical plane, as is the case in the pressuremeter test, is greater than that on planes inclined at 45 degrees as in the unconfined and triaxial tests. (2)

It is probable that for the particular set of conditions at this location additional contributions were made to this deviation by the presence of the nodules and concentrations of iron oxide at the greater depths and by the presence of small concentrations of soil within the core different than the majority of the material in the core.

As shown in Figure 11 there is a considerable difference between pressuremeter shear strengths and those obtained by triaxial testing throughout the depth of the test hole at the Perkins Road site. This is in contrast to the results just described at the Sorrento test site where the deviation tended to increase with depth.

Examination of the radiographs of the soil at the Perkins Road test site (Figure 17) reveals a basic difference between the condition of the soils at this test site and those at the Sorrento test site. There are present at the upper elevations and diminishing as depth increases, many hairline cracks throughout the core samples. That this condition is not due to drying out of the materials after the samples were taken, is evidenced by the presence of iron oxide concentrations in the cracks (circled areas Figure 17). In a paper entitled "Selection of Analytical Methods and Strength Parameters for Slope Stability Investigation in Cohesive Soils" presented to the Highway Research Board in January, 1968, Robert L. Schuster, Professor and Head of the Department of Civil Engineering at the University of Idaho, pointed out some of the problems associated with estimation of shear strengths in fissured clays. One of the problems that he pointed out in his paper is that where triaxial tests on small samples of these clays are run, falsely high estimations of shear strength may be obtained. One

probable reason noted for this condition is that these fissures may not be present in the particular core sample tested. This situation has also been noted by other researchers.



16.3' - 17.2' *17.2' - 18.0'* *19.0' - 19.8'* *20.1' - 20.9'*

Figure 17 - Perkins Road Radiographs

At the Perkins Road test site the cracking noted is localized, due to some extent to root channels, but probably primarily due to dessication of the material. It seems likely that since the cracking is not in the form of large or extensive fissures or cracks that exactly the reverse effect of that described by Schuster might apply; that is, the effect of the cracking would be to cause incipient failure planes in the small test specimens while not lowering the overall strength of the confined in-place material tested by the pressuremeter by an equal amount. The radiographs and photographs (Figures 17 and 18) at the Perkins Road site also reveal the presence of considerable accumulations of iron oxide (some manganese) mostly in the form of stratified concentrations with some of

the strata vertical and some horizontal in the sample. These concentrations are in some cases hard material but generally are quite soft.

In addition, the radiographs and photographs show small but distinct strata of soil different than that of the majority of the soil interspersed within the cores. This is especially obvious for the sample from 20. 1 feet to 20. 9 feet (Figure 18).

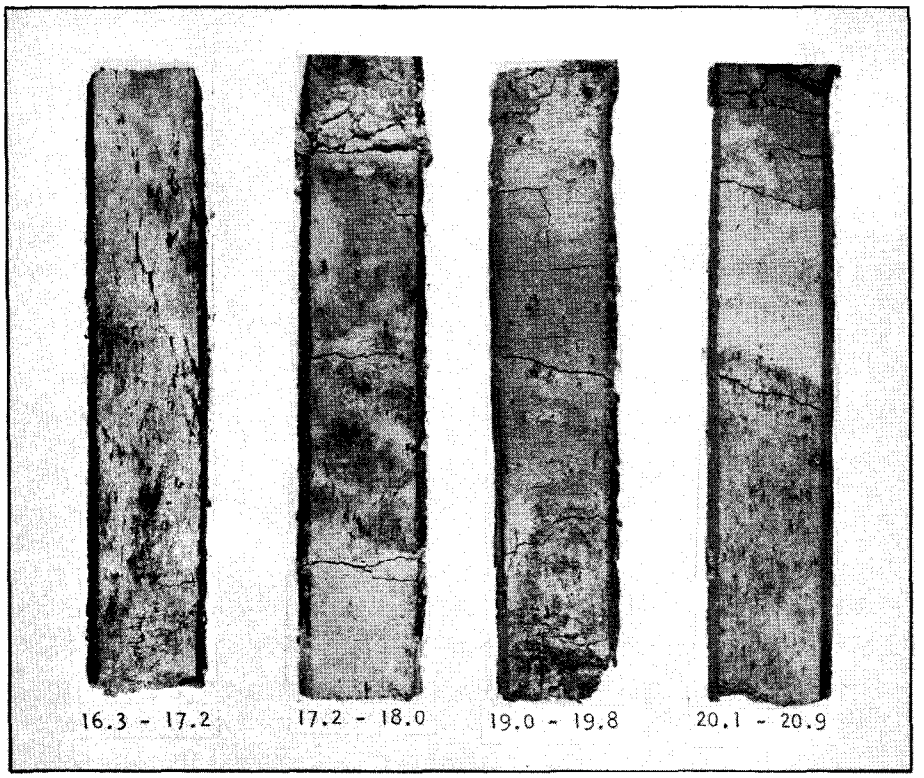


Figure 18 - Perkins Road Photographs

It may also be noted from Figure 11 that this particular sample showed an even wider variance between the pressuremeter derived shear strength and the triaxial shear strength than most of the other samples from this hole.

It seems probable that these areas of different material within the cores, whether primarily iron oxide or different soil, are the principal cause of the large deviations between the pressuremeter derived shear strengths and those obtained by conventional testing. In testing soils either by unconfined compression or triaxial testing the sample most probably will fail at the weakest point within the core. The point of weakness may be caused by a weak stratum of soil within the specimen, soft deposits of iron or manganese oxide, or hard rocklike

aggregations of the iron oxide. In any event the failure is likely to occur at these points of abrupt change.

The pressuremeter, on the other hand, tests the in-place soil over an area about 18 inches in height. Even though the testing cell of the probe is fairly flexible in order to conform to the configuration of the surrounding soil, it is extremely unlikely that enough of a balloon-like effect could be created in these small areas of weakness to show an apparent failure on a plot of volume versus pressure based solely on the yielding of these small areas.

It is much more probable that the failure point (limit pressure) observed by the pressuremeter test is based on the average strength of the 18 inches of material tested. This is not to imply a straight line average but that the in-place strength measurement lies somewhere between the strength of the weakest strata and the strength of the majority of the material in the test area. The amount of influence of the weaker material would, of course, depend on the thickness of the stratum or strata involved. Since most of the concentrations of iron oxide and softer soil are very small when compared to the 18 inch test size of the pressuremeter, it is probable that the strength measured by the pressuremeter is nearer to the strength of the majority of the material in the test area.

Samples of the material from two levels at the Perkins Road test site were remolded at natural moisture content and tested triaxially. One of the samples (15-18 feet) showed a drop in strength for the remolded sample when compared with the results obtained from the original core while the other sample (19-21 feet) showed a small increase for the remolded sample (Table 4). It should be noted (Figure 11) that for the elevation where the remolded strength dropped the variation between pressuremeter estimated shear strength and triaxial test shear strength was not as great as for most of the other specimens tested. This elevation was picked for remolded testing because of this fact.

The other sample (18-21 feet) was selected for remolding because one of the greatest variances between conventional shear strengths and pressuremeter shear strengths occurred at this elevation. As noted there was a slight increase of strength for the remolded sample (Table 4).

In order to examine the pertinence of the data concerning the remolded soils, let us consider the "sensitivity" of soils to remolding or disturbance.

Assuming that a specimen taken for conventional testing is undisturbed, then the amount of the strength drop from undisturbed to remolded is a measure of the sensitivity of the soil. The larger the drop from undisturbed to remolded the more sensitive the soil is considered.

Conversely though, the fact that no decrease in strength occurs upon remolding or that the strength actually increases for the remolded sample may be an indication that the "undisturbed" sample is in fact disturbed.

As previously mentioned one of the remolded cores (15-18 feet) exhibited a slight strength loss when compared to the original test on the undisturbed core, while the other remolded sample (19-21 feet) exhibited a slight strength gain. These results would seem to indicate either an insensitive soil or badly disturbed original cores.

The radiographs and photographs (Figure 17 and 18) do not indicate a great deal of the distortion commonly associated with badly disturbed cores, and the most likely conclusion seems that the cores were not greatly disturbed.

Considering the factors presented by the remolded samples in conjunction with the information furnished by the radiographs, photographs, and visual observation; it seems probable, that though part of the difference between pressuremeter shear strengths and conventional shear strengths may be ascribed to disturbance during sampling, the major portion of the variation is due to the concentrations of different material occurring within the cores.

This supposition is, of course, reinforced by the fact that the photographs and radiographs of the sample from 20-21 feet show a definite layer of a different material within the core and that this is an area where a large difference between pressuremeter results and triaxial results were noted.

The situation for the core samples examined at Lake Charles, was very similar to that at the Perkins Road test site except that the nodules and concentrations of material were primarily of a calcareous nature (Figures 19 and 20). The concentrations of this calcareous material are somewhat more numerous at the Lake Charles test site than were the iron oxide concentrations at Perkins Road. The presence of layers of soil different than the majority of soil as definite strata within the core is somewhat less noticeable. The overall effect, however, is very similar to that at Perkins Road and as might be expected Figures 12 and 13 show much the same pattern as that noted at the Perkins Road site. The conventional test shear strengths and pressuremeter shear strengths are considerably different at all levels, with pressuremeter results higher than conventional test results (Figures 12 and 13).

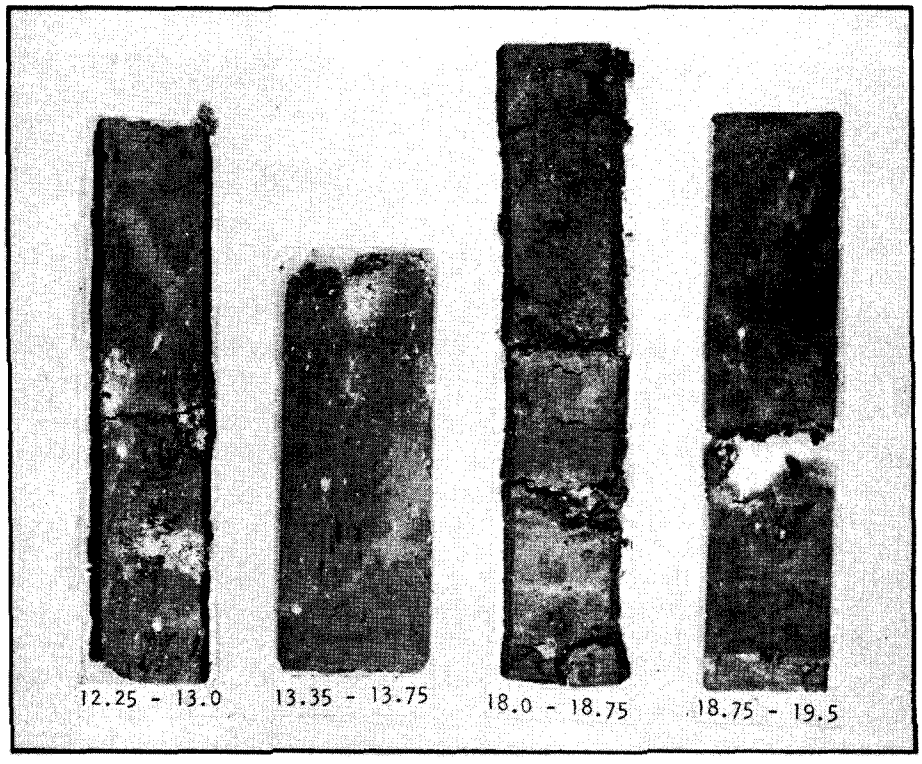
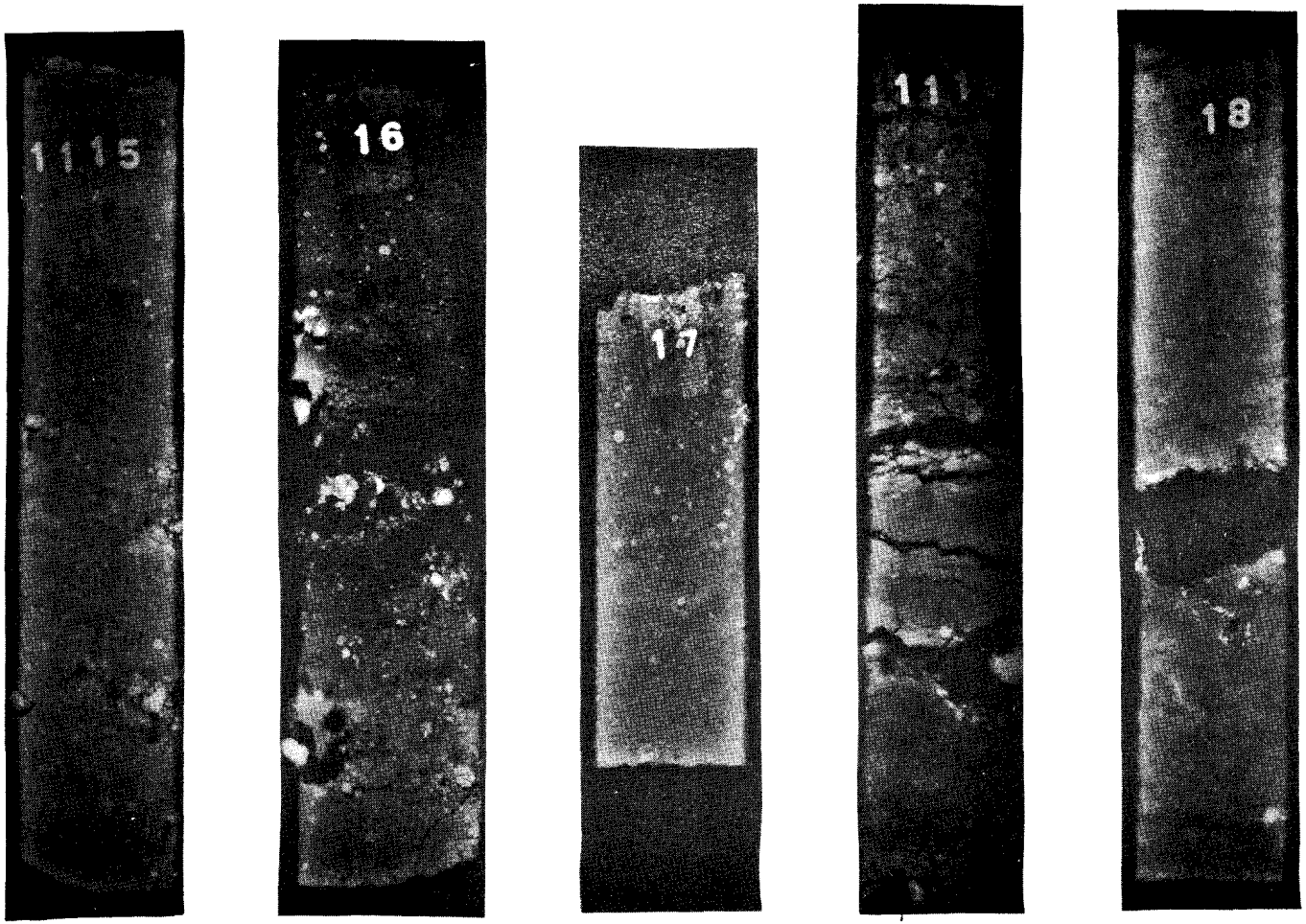


Figure 19 - Lake Charles Photographs

As may be noted from Table 5, the remolded samples from the Lake Charles test site exhibited increases in strength over those of the "undisturbed" cores which were tested. This is true for both the cores tested in unconfined compression and those tested triaxially.

Considering these points of similarity it is probable that the same arguments previously presented at the Perkins Road test site would apply to this case.



12.2' - 13.0 *12.2' - 13.0'* *13.3' - 13.7'* *18.0 - 18.7'* *18.7' - 19.5'*

Figure 20 - Lake Charles Radiographs

CONCLUSIONS

1. For very soft cohesive soils the correlation between pressuremeter derived shear strengths and conventional shear strengths for this study was good. This conclusion is substantiated by the results at the Houma and Plaquemine test sites, and has been noted in investigations by other researchers.

2. At the Sorrento test site the results indicate a not completely explained increase in pressuremeter shear strengths with depth which is not present for conventional tests. This trend has been noted by other investigators of the pressuremeter. It is probable that the pressuremeter shear strengths may be closer to the true in-situ strength of the soil than those obtained by conventional methods using open drive sampling techniques.

3. At Perkins Road and Lake Charles test sites there are considerable differences between pressuremeter derived shear strengths and conventionally obtained shear strengths, with the pressuremeter results consistently higher. When the in-place conditions of the soil as shown by radiographs, photographs and remolded samples are considered, it seems highly probable that the pressuremeter shear strengths may be more representative of true in-situ conditions than those shear strengths obtained by conventional tests. It should be noted, however, that for soils in which small strata of a different material, whatever its nature, are interspersed within a core, the strengths estimated probably lie somewhere between the strength of the weakest material and that of the strongest material within the test segment. Particular attention should be paid to this fact in any situation where stability against sliding is critical since slip planes could conceivably form along the strata of weak material.

4. Testing on non-cohesive soils with the pressuremeter requires equipment not normally used by the Louisiana Department of Highways. Constant caving of these materials caused difficulties such that no acceptable tests of this material were made during this study. This would probably be a fruitful field for further research with this device.

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APPENDIX

TABLE 1
SOIL PROPERTIES
HOUMA TEST SITE

Depth (ft)	Shear Strength (TSF) Pressure-Meter	Unconfined	Triaxial	Remolded Triaxial	Field Soil Classification	Natural Moisture (%)	Organic Content (%)	Atterberg Limits Liquid Limit	Plastic Limit	Plastic Index
9-12	0.0005	0.075			Very soft gray silty clay & traces of organic	-	25	82	56	26
12-15	0.0388	0.025			Very soft gray silty clay	-	-	36	21	15
18-21	0.0815	0.105			Very soft gray silty clay & sandy silt stratas	-	-	45	23	22
24-27	0.141	0.050			Very soft gray silty clay & traces of shell & organic	-	14	53	20	33
30-33	0.185	0.105			Soft gray silty clay & traces of organic	88	15	92	32	60
42-45	0.317	0.09			Soft to medium gray silty clay & traces of shell & silt lenses	43	-	49	21	28
48-51	0.289	0.22			Medium gray silty clay & very light traces of organic	46	11	74	31	43
54-57	0.319	0.23			Medium gray silty clay & silt lenses	59	-	79	23	56
75-78	0.519	0.37			Fairly stiff gray silty clay & silt loam	64	-	95	29	66
18-21	0.224	0.21			Soft gray silty clay & traces of organic	70	10	85	31	54
24-27	0.295	0.15			Soft gray silty clay with traces of organic & silty loam	51	9	44	28	16
30-33	0.185	0.24			Soft gray silty clay	54	-	68	22	46
36-39	0.438	0.49			Silty clay & organic	147	39	-	-	-
42-45	0.415	0.32			Soft gray silty clay with shell & organic	53	11	78	22	56
48-51	0.386	0.75			Medium silty clay & organic	198	56	-	-	-
57-60	0.328	0.23			Silty clay to sand	34	-		N. P.	

TABLE 2
SOIL PROPERTIES
PLAQUEMINE TEST SITE

Depth (ft)	Shear Strength (TSF) Pressure-Meter	Unconfined	Triaxial	Remolded Triaxial	Field Soil Classification	Natural Moisture (%)	Organic Content (%)	Atterberg Limits Liquid Limit	Plastic Limit	Plastic Index
3-6	0.120	0.39			Medium blue gray silty clay & traces of rotten wood		9.0	70	29	41
6-9	0.139	0.18	0.22		Blue gray silty clay	44.0	7.8	32	21	11
9-12	0.139	0.18			Blue gray silty clay	44.0		57	23	34
15-18	0.202	0.13	0.23		Soft blue gray silty clay & traces of rotten wood		13.9	65	32	33
18-21	0.156	0.16	0.28		Blue gray silty clay & rotten wood	53.0	9.4	38	22	16
21-24	0.222	0.06			Soft blue gray silty & silty loam	33.0	4.0		N.P.	
24-27	0.307	0.26	0.45		Medium blue gray silty clay & rotten wood	66.0	11.6	66	25	41
27-30	0.483	0.27	0.45		Medium blue gray silty clay & traces of organic	62.0	14.0	99	29	70

TABLE 3
SOIL PROPERTIES
SORRENTO TEST SITE

Depth (ft)	Shear Strength (TSF) Pressure-Meter	Unconfined	Triaxial	Remolded Triaxial	Field Soil Classification	Natural Moisture (%)	Organic Content (%)	Atterberg Limits Liquid Limit	Plastic Limit	Plastic Index
0-3	0.170	0.31			Medium to fairly stiff gray & brown silty clay	33.3		71	22	49
3-6	0.288	0.24			Soft to medium yellow, brown & gray silty clay	27.5		38	19	19
6-9	0.340		0.43		Medium to fairly stiff yellow, brown & gray silty clay	23.6		40	15	25
9-12	0.66	0.69			Fairly stiff yellow, brown & gray silty clay	20.9		39	14	25
12-15	0.99		0.90		Stiff yellow, brown & gray silty clay	24.5		61	22	39
15-18	1.07	0.66			Stiff yellow, brown & gray clay - light alternate stratas medium brown & gray crumbly silty clay	28.6		91	30	61
18-21	0.90	0.68				38.0		94	24	70
21-24	1.14	0.53	0.92		Stiff yellow, brown & gray clay - light traces of iron oxide	29.4		70	22	48
26-29	1.24		1.18		Stiff gray & brown clay with alternate stratas stiff gray & brown clay & fairly stiff silty clay, light alternate sand stratas	38.7		89 39	21 19	68 20
29-31	1.74	0.71			Stiff brown & gray clay - very light silt lens-light trace iron oxide	30.5		85	24	61
31-34	1.48		1.35		Stiff brown & gray clay-light trace of concretion & iron oxide	43.6		98	32	66
34-36	1.74		1.20		Stiff brown & gray clay - very light silt lense, alternate stratas stiff gray brown clay & fairly stiff gray silty clay with loam	28.0		48	20	28

TABLE 4
SOIL PROPERTIES
PERKINS ROAD TEST SITE

Depth (ft)	Shear Strength (TSF) Pressure-Meter	Unconfined	Triaxial	Remolded Triaxial	Field Soil Classification	Natural Moisture (%)	Organic Content (%)	Atterberg Limits Liquid Limit	Plastic Limit	Plastic Index
3-6	1.58	1.00			Dry gray silty clay	16.7		42	18	24
6-9	1.42		0.94		Stiff gray silty clay	19.2		44	17	27
9-12	0.98		0.51		Stiff gray silty clay	28.9		74	28	46
15-18	1.43		1.21	0.77	Stiff gray heavy clay	30.9		73	26	47
18-21	1.18		0.68	0.71	Medium stiff gray heavy clay	35.8		80	26	54
21-24	1.44		1.08		Stiff gray silty clay	25.2		58	19	39

NOTES:

12-15 Very difficult to prepare - Small dry silt stringer & crumbly core.

15-18 Base of core wetter than top.

18-21 One core had 39.3% and the other 32.3% natural moisture content.

TABLE 5
SOIL PROPERTIES
LAKE CHARLES TEST SITE

Depth (ft)	Shear Strength (TSF) Pressure-Meter	Unconfined	Triaxial	Remolded Triaxial	Field Soil Classification	Natural Moisture (%)	Organic Content (%)	Atterberg Limits Liquid Limit	Plastic Limit	Plastic Index
0-3	1.38	0.46			Packed dark gray silty loam with shell & gravel					
3-6	1.20	0.60	1.01		Soft dark gray-brown silty clay	16.3				
6-9	1.30	0.75	1.03		Stiff red silty clay	17.6				
9-12	1.65	0.80	1.03		Brown very stiff silty clay with concretions	21.6		57	16	41
12-15	1.30	0.81	1.20	1.23	Stiff reddish brown silty clay with concretions	22.8		58	19	39
15-18	1.30	0.46			Packed light brown silty clay	24.8		72	21	51
18-21	1.30	0.88	0.74	0.84	Very stiff brown silty clay with concretions	28.3		-	-	-
21-24	1.44	0.88	1.26		Silty clay, sandy loam	28.7		80 65	21 20	59 45
24-27	1.40	0.78			Sandy loam to sandy clay	30.0		44	17	27

NOTE:

12-15
18-21 Cores for triaxial extremely difficult to prepare because of concretions in soil.

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