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ANALYSIS:
PREDICTION OF PILE CAPACITY USING THE CONE PENETRATION TEST

BY

WILLIAM M. CORSON

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I wish to dedicate this research report to my family:

my beautiful and loving wife, Ginger,

and

my darling new daughter, Amey Christiana.

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I especially thank my family: my wife Ginger, my new daughter Amey, my parents Jean and Bill, my brother Carl, and my grandparents for always having such confidence in my abilities and providing me with the inspiration to always do my best and "give 'em both barrels." Ginger and our daughter Amey have made many sacrifices and made every effort to help me complete my degree requirements in minimum time. Little Amey's smile has frequently uplifted my spirit, and helped me keep things in proper perspective. Ginger has always been my foundation and faithful and selfless assistant; especially during my tenure at the University of Florida. I love her and thank her with all my heart.

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ANALYSIS: PREDICTION OF PILE CAPACITY USING THE CONE PENETRATION TEST

By

WILLIAM M. CORSON

December 1989

Chairman: Dr. Frank C. Townsend
Major Department: Civil Engineering

The purpose of this research report was to evaluate the accuracy of the electrical and mechanical cone penetration test for predicting pile capacity when compared to observed pile capacity results from nearby pile load tests. The research was accomplished by finding construction sites in Knox's 1989 PhD dissertation data base which had pile load test, electrical cone penetration test, mechanical cone penetration test, and boring log data all located within a close proximity of one another. Data was gathered by both the University of Florida and the Florida Department of Transportation. Eight sites containing all or most of the aforesaid data were discovered. The sites were all located on the Florida coast with three sites each at Apalachicola and Choctawhatchee Bay on the Gulf Coast and two sites at Port Orange on central Florida's Atlantic Coast.

Pile capacities were predicted using the electrical and mechanical cone pentration test sounding data. Then the predicted pile capacities were compared to the observed pile capacity determined by nearby pile load tests. As a natural consequence of performing the latter analysis, comparisons could also be made between the results of electrical and mechanical cone penetration tests at a given site. Soil layer divisions along with the average friction ratio and end bearing resistance measurements in each layer were identified and compared from electrical and mechanical cone penetration test sounding data.

Computer programs, designated MCPTUFR and PLAID, were developed by the Geotechnical Engineering Department at the University ~~of Florida~~ to predict pile capacity using conventional methods from mechanical and electrical cone penetration tests respectively. These programs, developed for the mechanical and electrical penetration tests repectively, were used in all cone penetration test pile capacity predictions examined in this report.

Standard penetration test results contained in the boring logs were also essential to successful completion of this report. The ratio of average end bearing resistance to average N value (Qc/N) within the critical depth region for end bearing was used to identify soil layers containing cemented sands.

This report concluded that the electrical and mechanical cone penetration tests were fairly accurate at predicting ultimate pile capacity (within + or - 30%) compared to observed pile load test results, except when cemented sands were present within the critical depth region for end bearing. Cemented sand regions were identified by a Qc/N ratio of about 8 or higher. In addition, better ultimate pile capacity predictions were made by the cone penetration tests when the soundings were very close to the pile load test. The report also concluded that the electrical cone penetration test was fairly accurate at making load-settlement predictions, except when cemented sands were present within the critical depth region for end bearing.

Other conclusions made in this report were related to comparisons between the electrical and mechanical cone penetration tests. Both tests were accurate at detecting the depth divisions between cohesive and cohesionless soil layers when compared with nearby boring log results. The mechanical cone penetration test was generally better than the electric cone penetration test at predicting soil layer classifications due to seemingly more accurate friction ratio determinations.

CHAPTER 1

INTRODUCTION

Piles were successfully used as deep foundations for structural support as early as Roman times (Peck, 1974a). When soils at or near the surface are found to be too weak to provide adequate support for conventional shallow foundations, piles are often called upon to reach stronger supporting soils located at greater depths. Piles are structural members with a small cross-sectional area compared to their length. They are normally installed dynamically with a driving apparatus usually consisting of a hammer. The key to the successful use of a pile or pile group is finding a soil layer of sufficient strength and thickness to support the loads to be carried by the pile(s) with minimal likelihood of settlement. Consequently, field testing that can identify these strong soil layers can be invaluable to an engineer to assist in determining optimum locations for pile placement. The cone penetration test (CPT) is a field test that may fit the bill.

The nature of the cone penetration test is such that it has been used to predict the load bearing capacity of pile foundations. The accuracy of cone penetration test predictions of pile capacity is the subject of this report. Actual pile load tests were performed during the construction of various Florida highway structures. Mechanical and electrical cone penetration tests were performed in close proximity to the pile load tests. Therefore, a comparative analysis of predicted and observed pile capacity was made possible.

Purpose and Scope of Research

The purpose of this research report was to evaluate the electric cone penetration test (ECPT) and mechanical cone penetration test (MCPT) for estimating ultimate pile capacity for a given pile when compared to pile load test (PLT) results. Pile capacity predictions based on separate ECPT and MCPT results for a given pile at a given site were compared to actual PLT pile capacity results using the same pile at the same site. A natural result of the comparative analysis of the predicted and observed pile capacities was a comparison of the ECPT and the MCPT. The ability to perform this research was made possible by the data base gathered by Knox (1989a) for his PhD dissertation.

Research Methodology

The initial phase of the research report required gathering sufficient data to accomplish the stated purpose of the project. A time consuming portion of the initial research phase was checking the ECPT and MCPT sounding data for discontinuities, ensuring the data was sound enough for analysis, and making necessary corrections to the data. In addition, some of the MCPT and PLT data necessary to conduct the research was not in the data base developed by Knox and had to be obtained from other sources. Then all of the test sites with good ECPT, MCPT, and PLT tests located within a reasonable proximity of one another were identified. The following locations in Florida were determined to have test sites meeting most of the requirements to perform a thorough analysis: Apalachicola Bay and River Bridges, Port Orange, and the Choctawhatchee Bay Bridge.

The second phase of research involved the "by eye" analysis and interpretation of the ECPT and MCPT soundings. Soundings also had to be adjusted to match PLT conditions. For instance, ECPT and MCPT soundings required adjustments for excavation, predrilling, and slurrying performed prior to pile load tests at the three sites at the Choctawhatchee Bay Bridge. The cohesive and cohesionless layers were then established for each site's soil profile using ECPT and MCPT soil classification systems with assistance from corresponding boring log information. Also, all parameters necessary for successful operation of the ECPT and MCPT pile capacity analysis were gathered.

After becoming fairly proficient in the use of the ECPT and MCPT programs used for pile capacity analysis, the next phase of research entailed the running of the programs for each test site at each location. The pile geometry data used in the programs matched the actual pile geometry data for the piles driven in the PLTs. Once all of the ECPT and MCPT pile capacity predictions were completed, they were all analyzed and compared to the PLT pile capacity results. In the course of completing the comparative analyses, standard penetration test (SPT) data from the boring logs was also analyzed and used for completion of the research.

CHAPTER 2

CONE PENETRATION TESTING REVIEW

History

Subsurface soil testing has made great advances in the past century. Subsurface testing has made much progress since deep soils were examined by drilling holes and washing the loosened soil to the surface as employed in wash boring. Field tests were required that would measure subsurface soil properties while minimizing soil disturbance to ensure accuracy of the soil property measurements. Sounding tools were developed to fill the aforesaid need, and have been used since the Swedish State Railways first employed them in 1917 (Terzaghi, 1967). Since 1917, numerous modifications and advancements have been made in sounding tool development. Pioneering development of the modern Dutch Cone method of soil penetration was accomplished in the early 1930's by Buisman and Barentsen at the Technical University of the Netherlands. From the 1940's to the 1960's, penetration rigs of progressively greater capacity were developed. Hand operated penetrometer rigs evolved into motorized rigs by 1960. Modern rig capacities are typically 10,000 to 20,000 kg. Begemann developed a penetrometer tip in the early 1960's which not only measured end bearing capacity, but also local lateral friction.

Various methods of cone penetration were developed which achieved penetration by steady pushing, hammering, and screwing. Various penetration tips have also been developed. The variety in methods and equipment have been the cause of some frustration as a goal of eventual standardization is sought by the geotechnical industry. Nevertheless, variety in penetration instruments has led to various improvements, innovations, and modifications which have greatly expanded the capabilities of cone penetration testing.

The widely used Begemann tip is a mechanical tip used in the mechanical cone penetration test (MCPT). Begemann introduced his popular mechanical tip in Indonesia in 1953 (Meigh, 1987a). The loads required to overcome both end bearing on the cone at the end of the mechanical tip and side friction along the side of the tip are measured at the ground surface by a proving ring or load cell.

One of the most important improvements in cone testing capabilities began in 1948 with the development of the first electric cone which had a vibrating wire measuring unit. In 1965, a consortium of the Dutch consulting engineering firm Fugro, the Phillips Company, and a Dutch government research institute at Delft, developed an electric tip for cone penetration testing.

In electric friction cone testing, strain gauges within the tip continuously measure the applied loads on the cone and sleeve during penetration. The strain gauge measurements are then relayed to read-out equipment at the surface by electric cable producing output on a chart recorder.

Combining cone bearing capacity measurements with frictional resistance measurements, many theoretical and empirical correlations have been developed to determine various geotechnical parameters. Cone penetration test results have been commonly used to determine such parameters as soil classification, friction angle, undrained shear strength, relative density, bearing capacity, settlements, and the driveability and bearing capacity of piles. Cone penetration testing has also been used to locate stiff soil layers, cavities, and other subsurface discontinuities; to identify soil layer classifications; and, to determine the stratigraphy of layers and their homogeneity over a site. In comparison to other insitu test methods, the cone penetration test often provides cheaper, faster, more detailed, and more precise data for gathering preliminary design data and defining soil stratigraphy.

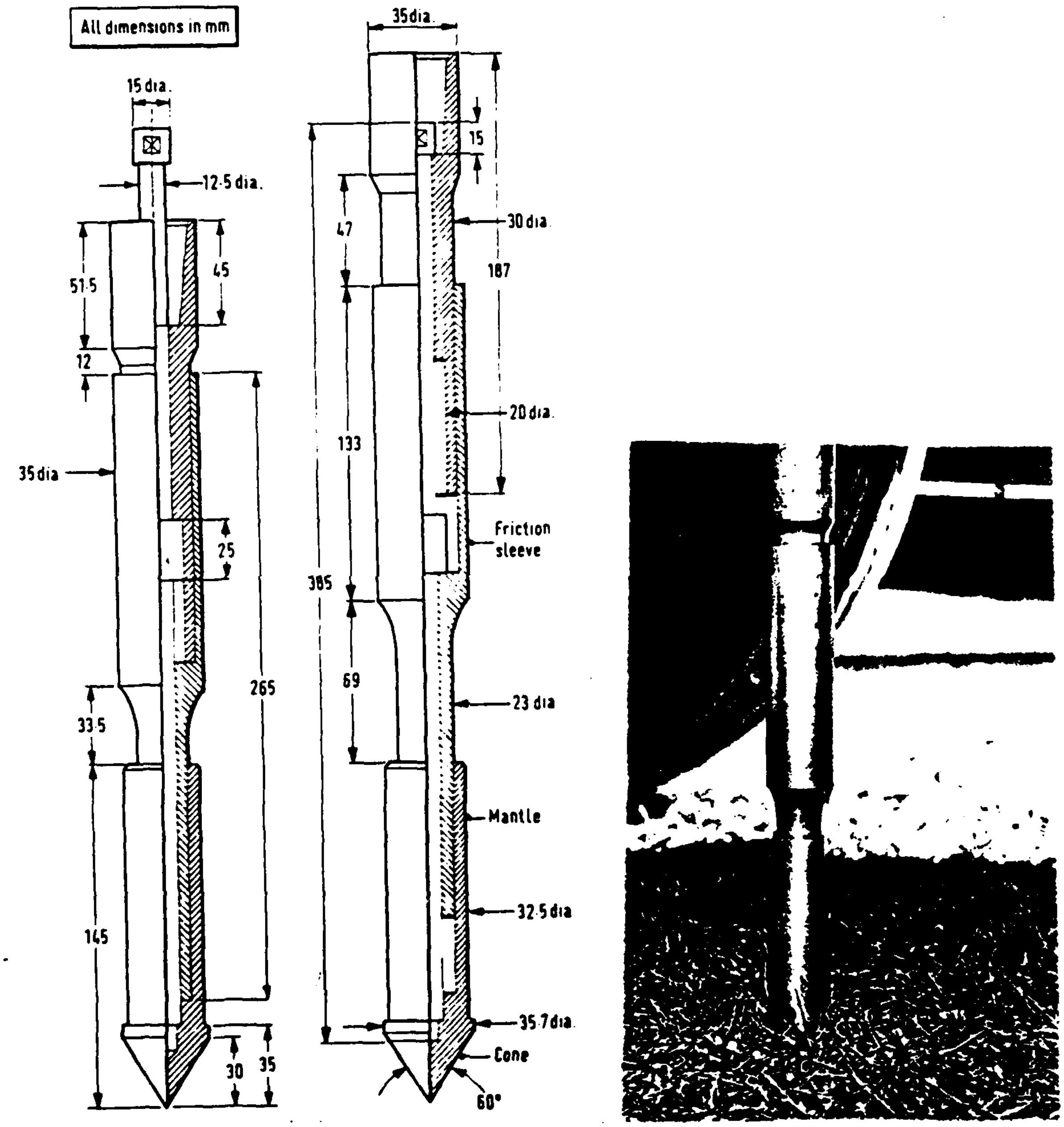
Basic Principles

In cone penetration testing, a cone on the end of a series of rods is pushed below the ground surface at a constant rate. Measurements of resistance to penetration of the cone are made continuously or at regular intervals. Typically, measurements are also made of either the resistance of a surface sleeve as in the MCPT soundings used in this report, or the combined penetration resistance of the cone and outer surface of the rods as in the ECPT soundings used in this report.

Both the electric and mechanical cone penetration tests obtained three important values essential to the successful completion of this report. The first of these values was the cone resistance, which is also called end bearing or bearing capacity. The latter value is represented by the symbol Qc through the remainder of the report. Qc is the soil resistance felt by the cone tip divided by the projected area of the cone tip. The second measurement was the friction resistance, which is also referred to as unit friction or local friction. The symbol for friction resistance used in this report is Fs. The third value was the friction ratio, which is represented by the symbol FR through the remainder of the report. The friction ratio, expressed as a percentage, is simply the measured friction resistance divided by the cone resistance, with the result multiplied by 100. For example, if a particular sand soil had a Qc value of 100 tsf and an Fs of 0.2 tsf, then the resulting FR value is 0.2%. The friction ratio for a given soil is considered to be a measure of the soil's ductility. This report only used Qc and FR values for making comparisons between the ECPT and MCPT, since these two measurements were useful for making soil classification determinations. Additionally, the MCPT sounding logs only plotted Qc and FR values.

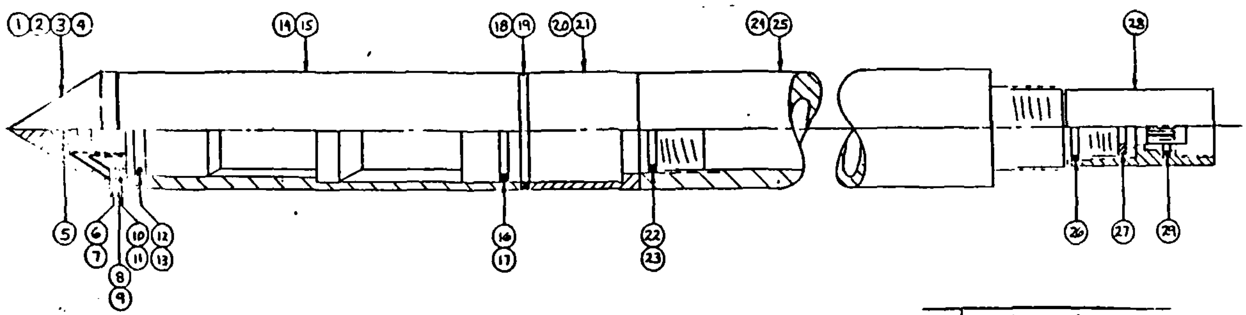
The two cone tips typically used in the ECPT and MCPT soundings are shown in Figures 2-1 and 2-2 respectively. Figure 2-1 shows the typical MCPT Begemann tip, which is also called the Dutch friction sleeve penetrometer tip. The Begemann tip makes separate measurements of cone and friction resistance every 20 cm of penetration. Figure 2-2 shows the typical ECPT subtraction type friction cone. The latter is used by UF and is manufactured by Hogentogler and Company, Inc. In the subtraction type friction cone, cone resistance is measured by compression in the cone load cell. The cone and friction resistance are both measured in the rear strain gauge bridge. The friction resistance is then obtained by the subtraction of the two load cell readings, which is accomplished electronically. Readings are typically recorded for every 5 cm of penetration.

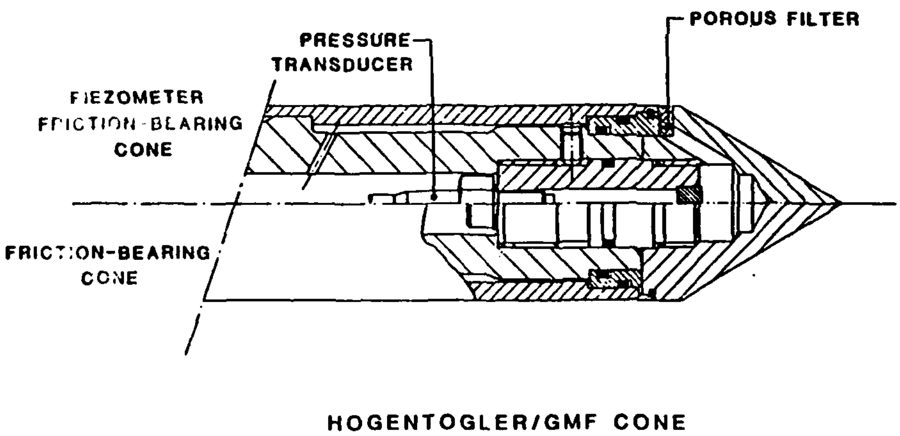
Tests with both tips are considered quasi-static, friction-cone penetration tests. The quasi-static nature of the test is because the penetration rate is approximately 2 cm/sec with pauses every meter for adding penetration rods. Both the MCPT and ECPT used in this report have a typical end cone surface area of 10 cm^2.



(a) Collapsed (b) Extended (c) Penetrating Ground Surface

Figure 2-1. Typical MCPT Begemann or Dutch Friction Sleeve Penetrometer Tip





Item	Description
1	Tip
2	Tip, Piezo
3	Tip
4	Tip, Piezo
5	Piezo Plug
6	Pore Pressure Element
7	Pore Pressure Element
8	Pore Pressure Ring
9	Pore Pressure Ring
10	O'Ring
11	O'Ring
12	O'Ring
13	O'Ring
14	Friction Sleeve
15	Friction Sleeve
16	O'Ring

Item	Description
17	O'Ring
18	Quad Ring
19	Quad Ring
20	Collar
21	Collar
22	O'Ring
23	O'Ring
24	Electronic Housing
25	Electronic Housing
26	O'Ring
27	Split Ring
28	Lemo Housing
29	O'Ring

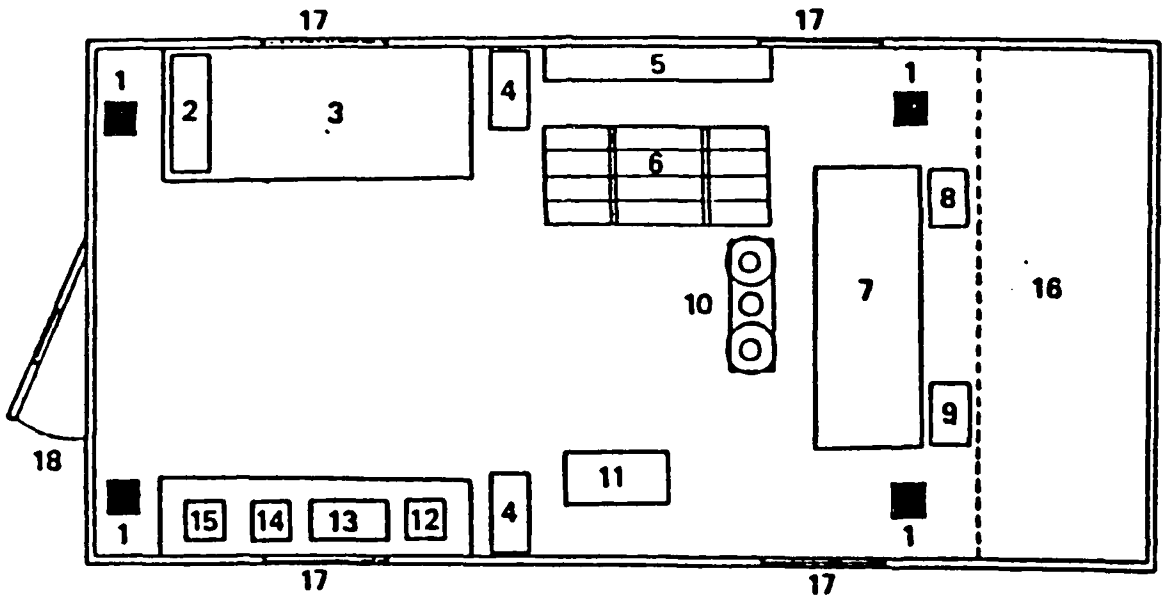
Figure 2-2. Typical ECPT Subtraction Type Friction Cone Penetrometer Tip

In addition to the penetrometer, both tests also require a thrust machine with reaction system and measurement and recording equipment. Typically, trucks with hydraulic ram systems are used to conduct cone penetration testing. The UF ECPT truck is shown in Figure 2-3 with a diagram of the truck's interior layout shown in Figure 2-4. The system is discussed and described in detail in Davidson and Bloomquist (1986).

ECPT and MCPT Capabilities and Comparisons

Two common disadvantages of the ECPT and the MCPT are the tests do not supply an actual soil sample and penetration into stiff strata is limited. Another disadvantage of the ECPT is the high initial cost. The advantages of the ECPT, however, are numerous: the rapid test procedure; continuous recording capability; the potential for automatic data logging, reduction, and plotting; high repeatability and accuracy; and, the capability of using additional sensors, such as for pore pressure and temperature measurements, chemical or radio-active material detection, ion detectors, geophones, and other devices. As outlined in ASTM D 3441, the ECPT has been found to have a standard of deviation of 5% for end bearing capacity and 10% for sleeve friction. On the other hand, the MCPT has been found to have a standard of deviation of 10% for end bearing capacity and 20% for sleeve friction (1989). Compared to the ECPT, the MCPT

Figure 2-3. University of Florida ECPT Truck



Legend:

1	Hydraulic Leveling Rams	10	Hydraulic Rams
2	Electric Cone Calibrator	11	Hydraulic Control Panel
3	Work Bench with Vise	12	Computer Terminal
4	Air Conditioning Units	13	Data Acquisition/Computer
5	Dilatometer Rods Rack	14	Dot Matrix Printer
6	Electric Cone Rods Rack	15	Graphics Pen Plotter
7	Hydraulic Fluid Reservoir	16	Storage Area
8	Battery Charger	17	Windows
9	Inverter	18	Access Door with Window

Figure 2-4. University of Florida ECPT Truck Interior Layout

has an initial low cost; it also has a number of disadvantages other than less accuracy when compared to the ECPT. The MCPT is a slow test procedure, is ineffective in very soft soils, requires moving parts which can be affected by soil particles, and requires very labor intensive data reduction and presentation.

With both the ECPT and MCPT, boundaries between soil layers are identified by looking for distinct changes in Qc and/or FR with depth. Some difficulty in precisely identifying interfaces between soil layers can be encountered due to the layer interface effect. The interface effect occurs because there is a short distance over which the penetrating cone is affected by both an underlying layer before penetrating it and an overlying layer after penetration into the soil layer below it.

Soil classification based on ECPT or MCPT results is difficult or impossible with only a soil profile plot of Qc with depth. Knowledge of local geology is an invaluable aid to Qc data interpretation. In general, sands have higher Qc values than clays; however, some overlap exists between loose sands and highly overconsolidated clays. In addition, the plotted and connected Qc value points in a sand profile are distinctly jagged. The jagged profile results from both the way sand fails under the cone tip pressure and the natural layering of sand deposits. Even under controlled chamber tests on sands, data interpretation from only Qc values was difficult, because Qc was found to vary with both vertical and radial effective stresses. The separation of density and stress effects is one of the great difficulties in Qc value interpretation.

Use of both the Qc and FR plots to identify soil types at different depths is a valuable method for soil classification. In general, sands have low FR values, clays have high FR values, and FR values for silts lay somewhere between sands and clays. When using the Begemann tip, some of the thrust attributed to sleeve friction is actually required to overcome soil bearing on the bottom bevel of the friction sleeve. The bevel effect is generally neglected for clay soils, but in sands it may amount to 50% or more of the measured thrust or friction resistance. Therefore, the actual FR value for sands may be only half of the FR measured using the Begemann tip. The electric friction cone penetrometer tip is smooth-sided above the cone without the bevel effects characteristic of the Begemann tip. The electric friction cone penetrometer tip typically measures close to the actual friction resistance of a sand, which is half of that measured by the MCPT Begemann tip.

Another problem with FR values garnered by the Begemann tip may be encountered in very soft and sensitive clays where the sensitivity can artificially increase the FR value. The problem arises from the cone penetrating undisturbed material while the friction jacket subsequently passes through remolded material. Also, sensitive and cemented soils may act in a similar manner because cone resistance is increased, but the sleeve friction does not increase.

Other Qc and FR interpretation problems may arise using both the ECPT and MCPT. Faulty interpretations might be made of soils consisting of widely dissimilar materials, such as gravel and clay. Well graded, closely graded, or gap graded soils may all be similarly interpreted if they all have the same soil particle diameters. In addition, soil may partially liquefy during cone penetration resulting in low Qc values. In the latter case, by the time the friction sleeve on a Begemann tip reaches the soil the cone had liquefied, excess pore pressures may have dissipated resulting in a falsely high FR.

Despite the specific case problems cited above, the use of Qc and FR values to identify soil types has been generally successful. For the MCPT Begemann tip, an unpublished chart developed by Schmertmann in 1969 is shown in Figure 2-5, and provides good guidance in classifying soils according to a plot of Qc vs. FR (1978a). A similar soil classification chart was published by Robertson and Campanella for electric friction cone results and is shown in Figure 2-6 (1984a). The latter chart is used for soil classification in the PLAID program developed by UF for ECPT data

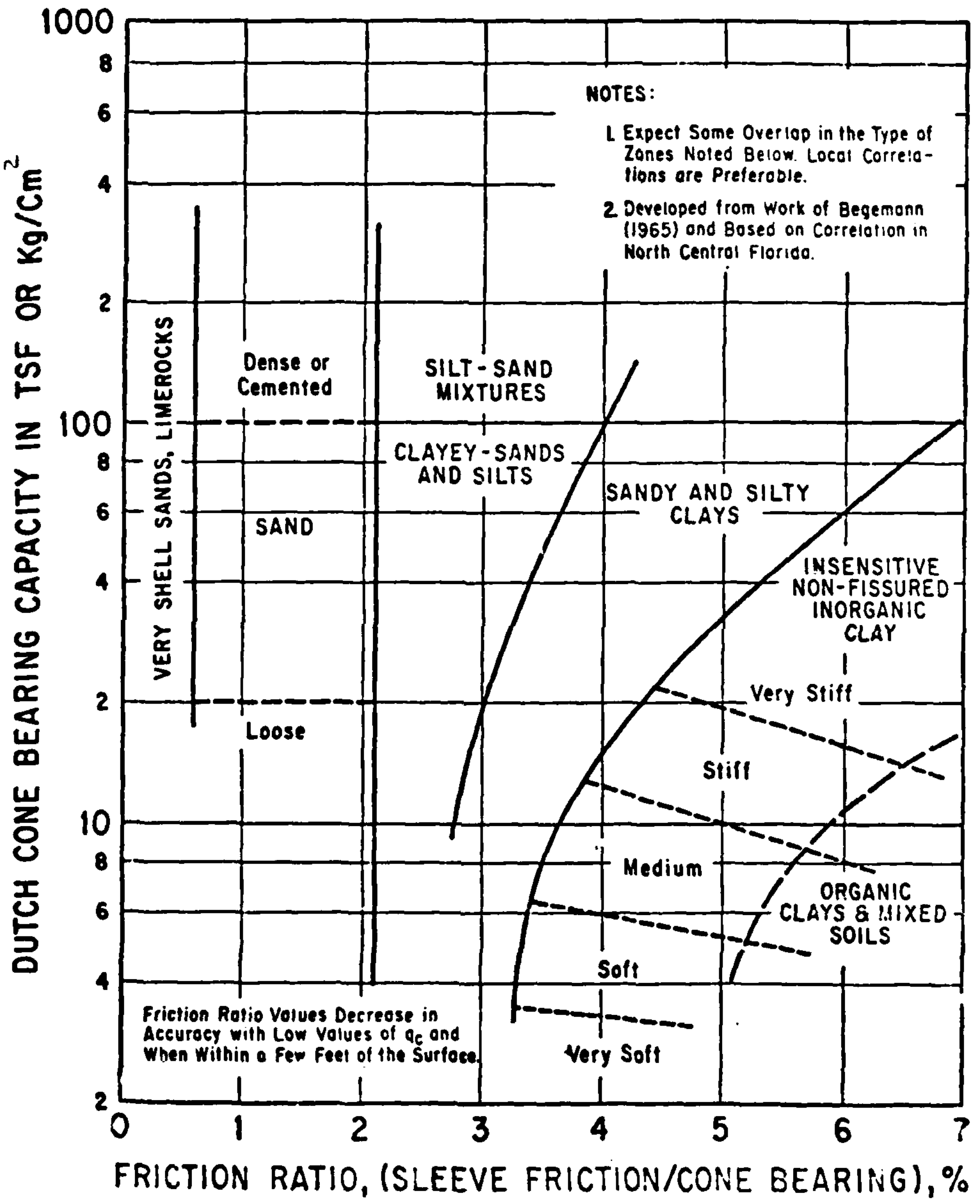
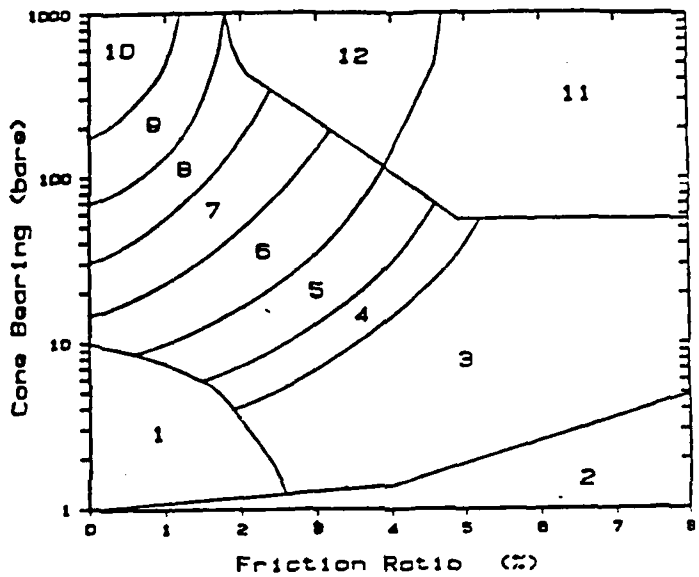


Figure 2-5. Schmertmann's Soil Classification Guide Chart of Bearing Capacity vs. Friction Ratio for Begemann Tip for MCPT



PL-AID Program

Code Identification	Zone	Soil Behavior Type
A	1	Sensitive fine grained
B	2	Organic material
C	3	Clay
D	4	Silty clay to clay
E	5	Clayey silt to silty clay
F	6	Sandy silt to clayey silt
G	7	Silty sand to sandy silt
H	8	Sand to silty sand
I	9	Sand
J	10	Gravelly sand to sand
K	11	Very stiff fine grained (*)
L	12	Sand to clayey sand (*)

(*) overconsolidated or cemented

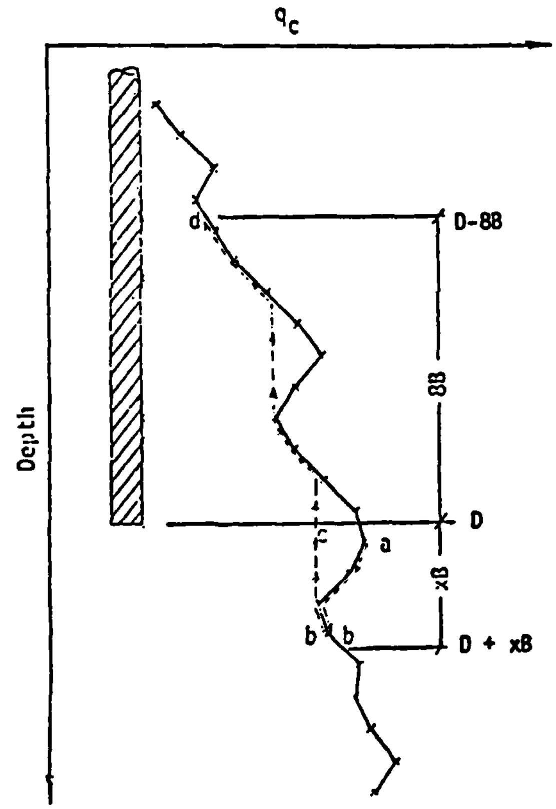
Figure 2-6. Robertson and Campanella's Soil Classification Guide Chart of Bearing Capacity vs. Friction Ratio for Electric Cone Penetrometer Tip interpretation, pile capacity prediction, and other geotechnical applications. Figures 2-5 and 2-6 were used for MCPT and ECPT soil classifications in this report.

ECPT and MCPT Pile Capacity Determination

One of the most useful applications of cone penetration test results is for the prediction of pile capacity. The cone penetration test is well suited for pile capacity predictions since the penetrometer actually is like a small scale pile undergoing the same end resistance and side forces as a real pile except on a smaller scale. Schmertmann, Heijnen, Nottingham, and others developed procedures to determine the load capacity of driven displacement piles using cone penetration test data.

The ultimate bearing capacity of a pile is the sum of the ultimate end bearing capacity and the ultimate shaft or friction resistance. Factors of safety are applied to each of the latter two values to determine appropriate design capacities. Meigh wrote that, in general, end bearing capacity is the dominant factor in sands and friction resistance is the dominant factor in clays (1987b).

The identical procedure was used for making both ECPT and MCPT pile capacity predictions. The Begemann procedure, also termed the minimum path method, shown in Figure 2-7 was used to estimate the ultimate unit pile



$$q_p = \frac{q_{c1} + q_{c2}}{2}$$

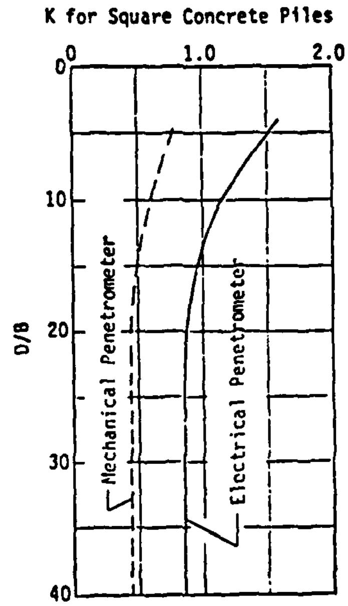
q_{c1} = Average q_c over a distance of xB below the pile tip (path a-b-c). Sum q_c values in both the downward (path a-b) and upward (path b-c) directions. Use actual q_c values along path a-b and the minimum path rule along path b-c. Compute q_{c1} for x-values from 0.7 to 3.75 and use the minimum q_{c1} value obtained.

q_{c2} = Average q_c over a distance of $8B$ above the pile tip (path c-d). Use the minimum path rule as for path b-c in the q_{c1} computations.

BEGEMANN PROCEDURE FOR PREDICTING PILE TIP CAPACITY

Figure 2-7. Equation and Example Profile for Predicting Pile End Bearing Capacity tip bearing capacity in both sands and clays (Schmertmann, 1978b). When the mechanical penetrometer is used in clay soil, the ultimate unit tip bearing capacity may be overestimated due to friction on the mantle of the tip. Consequently, the ultimate unit pile tip bearing capacity is multiplied by a factor of 0.6 to account for the added mantle friction forces. All pile tips at the test sites used in this report were located in cohesionless soil. Since single, very low Qc values may drastically affect the results of the procedure shown in Figure 2-7, such values are discarded and replaced with an average of the Qc values measured directly above and below it unless it is thought the low Qc actually represented a weak soil layer. Upper limits commonly used for ultimate unit pile tip bearing capacity are that no individual Qc value may exceed 300 tsf and the maximum allowable pile tip bearing capacity is 150 tsf. These limits are not truly restrictive because piles are not normally driven to such high tip resistance values. The unit pile tip bearing capacity is then multiplied by the end area of the pile tip to find the total end bearing capacity.

After calculating the end bearing capacity, the remaining component for determining the ultimate pile capacity is the sleeve or skin friction or friction resistance. The equation and accompanying design curve developed by Nottingham for calculating the friction resistance of square concrete piles in sand are shown in Figure 2-8 (Schmertmann, 1978c). The equation shown in Figure 2-8 assumes the sleeve friction resistance does not vary significantly with depth. In the case of multiple sand layers, the equation in Figure 2-8 is applied to each layer individually. The K value in the design curve in Figure 2-8 is based on both the ratio between the total embedded pile length and the pile width and whether or not the cone penetrometer tip is mechanical or electrical. The mantle effect on the mechanical tip results in a K factor about 50% of the K factor for the electrical tip. In a multiple sand layer system, the K value remains the same as it is applied to the friction resistance calculation for each sand layer. The equation and accompanying design curve developed by Tomlinson and Schmertmann for calculating the friction resistance of square concrete piles in clay are shown in Figure 2-9 (Schmertmann, 1978d). The equation shown in Figure 2-9 was based on the assumption that the Fs value measured by the electric or mechanical cone penetrometer is an accurate estimation of the undrained shear strength of a clay. The skin friction resistance for a given pile is the sum of each soil layer's induced friction resistance multiplied by the perimeter area of the pile contained in the soil layer. Then the total friction resistance is added to the total end bearing capacity which results in the ultimate pile capacity. A factor of



$$Q_s = K \left[\sum_{d=0}^{8B} \frac{d}{8B} f_c A'_s + \sum_{d=8B}^{D} f_c A'_s \right]$$

where Q_s = total ultimate side friction resistance

K = ratio of unit pile friction to unit sleeve friction in cohesionless soil (from above)

d = depth to the f_s value being considered

B = pile width or diameter

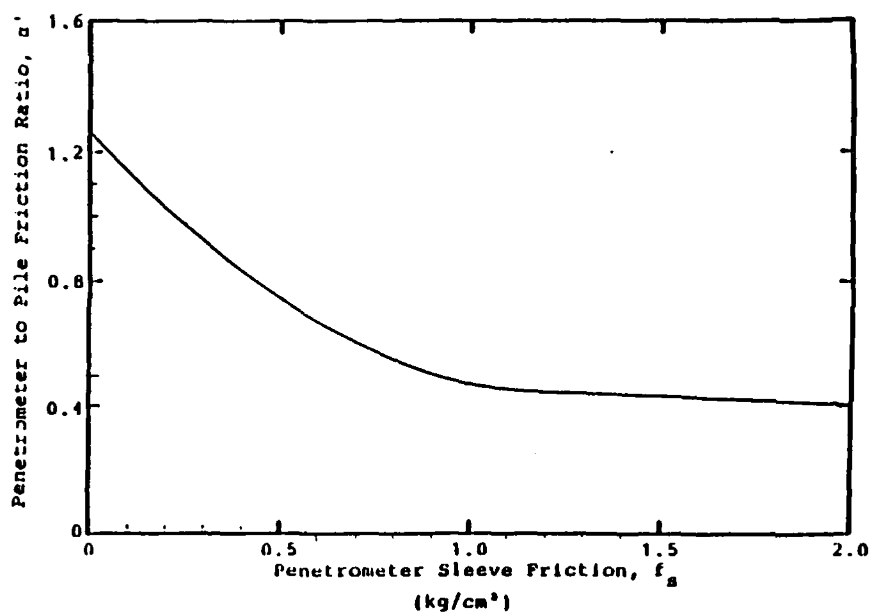
f_c = unit friction sleeve resistance

$A's$ = pile contact area per f_s depth interval

D = total embedded pile length

The K term is a constant that is found from above plot using the appropriate D/B and is a ratio between sleeve friction and unit pile friction.

Figure 2-8. Equation and Design Curve for Determining Pile Skin Friction Resistance in Sand



$$Q_S = \alpha' \bar{f}_C A_S$$

where α' = ratio of unit pile friction to unit sleeve friction in cohesive soil (from above plot)

$\bar{f}_C$ = average undrained sleeve friction

A_S = total pile-soil contact area

Figure 2-9. Equation and Design Curve for Determining Pile Skin Friction Resistance in Clay safety of 3 is commonly applied to the total end bearing capacity, while a factor of safety of 2 is commonly applied to the total friction resistance, and the sum of the two is used as the design pile capacity.

Many other methods and procedures have been developed for making pile capacity predictions using cone penetration data. The goal of researchers, however, has been to keep the methods as simple as possible without losing accuracy. The method described above was used to predict pile capacity in this report.

CHAPTER 3

DATA BASE AND TESTS

Chapter 3 is devoted to a general description of the data base, field tests performed, computer programs used, and problems encountered in pile capacity analysis.

Test Sites

Three general locations, with a total of eight separate test sites, were used in the pile capacity analysis. The locations were the Choctawhatchee Bay bridge, the Apalachicola Bay and River bridges, and a bridge over the Halifax River at Port Orange. The first two locations are on the Gulf coast of Florida's panhandle region, and Port Orange is on the Atlantic coast in central Florida. Locations are shown in Figure 3-1. These locations were the only three from Knox's 1989 PhD dissertation data base with pile load test, ECPT, MCPT, and boring data located within a close proximity of one another.

The Apalachicola Bay and River bridges were FDOT replacement structures for old bridges located on U.S. Highway 98 in Apalachicola. The river bridge is a 3783 ft long structure with a roughly east-west orientation and a turn northward at the western end of

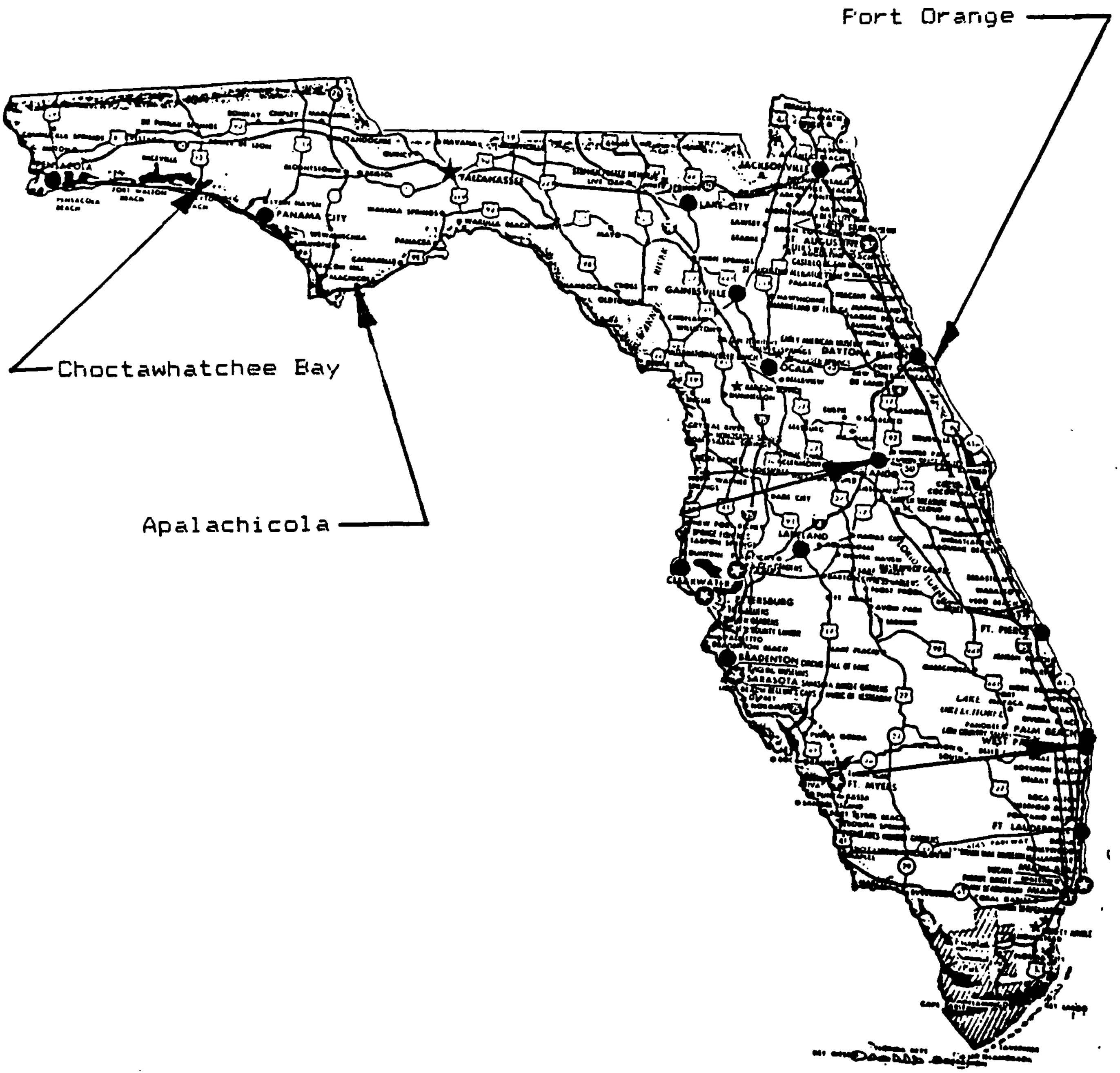


Figure 3-1. Location of Florida Test Sites Used in Pile Capacity Analysis

the structure. Site 1 used in this report was located at the river bridge. The bay bridge is a 14,175 ft structure with an east-west orientation. Sites 2 and 3 used in this report were located at the bay bridge. The soils at both sites were characterized by Knox as sands, clays and clay/sand mixtures (1989b).

The Port Orange site was an FDOT bridge on State Road A1A which crossed the Halifax River. Driven piles were used for the foundations on the bridge approaches, and drilled shafts were used under the main spans of the bridge. Sites 13 and 14 in this report were located at Port Orange. Soil at Port Orange was characterized by Knox as shelly sand and sandy silt from the ground surface to about 65 ft in depth which encompassed the soil region used in the site pile analysis (1989c).

The Choctawhatchee Bay bridge was an FDOT replacement structure for an old bridge on State Road 83 (U.S. Highway 331). The bridge is 7534 ft long with a north-south orientation. Sites 19, 20, and 22 used in this report were located at the Choctawhatchee Bay bridge. Knox characterized soils at this location as mostly sand overlying some clays and clayey sand on the bridge's southern approach with clays increasing northward across the bridge (1989d).

Pile Load Tests

All of the pile load test (PLT) information for the Apalachicola Bay, Apalachicola River, and Port Orange came from data on file in the University of Florida (UF) Geotechnical Engineering Department which was obtained from firms contracted by the Florida Department of Transportation (FDOT) to perform and analyze the tests. The Choctawhatchee Bay PLT information was obtained from Schmertmann and Crapps, Inc. through John Shoucair, a former employee of Schmertmann and Crapps, Inc. and witness to most of the PLTs contained in this report. Comparison of Choctawhatchee Bay PLT results with ECPT and MCPT pile capacity predictions was complicated by excavation, predrilling, and slurrying prior to the PLT at all three sites. Adjustments to the ECPT and MCPT to account for the latter complications are outlined in the individual site discussions in the next chapter. The final report on the Choctawhatchee Bay PLT results is expected to be complete and available for further information within the next year. The Young's Modulus (E) was back calculated from submitted PLT results at all sites except Choctawhatchee Bay. At the latter site, the E was assumed from commonly used American Concrete Inst. criteria. The equation used for the latter was:

$$E = 57,000 \, (SQRT \, (f'c))$$

where f'c was the specified compressive strength of the concrete and was 6000 psi (Troxell, 1968). All of the

PLT load-settlement plots contained in the report were produced by the firms responsible for the pile load testing.

Borings

All of the boring data, with soil profile classifications and standard penetration test blow counts, were taken from field and contract documents produced by FDOT for the purpose of contracting the PLT work at each of the sites. One of the Apalachicola sites used in the report did not have any nearby boring (site 2 at the river bridge), and another only had a boring located about 800 ft from the PLT (site 3 at the bay bridge). The boring for Choctawhatchee Bay site 21 was located 266 ft from the site PLT. The rest of the test sites had borings located reasonably close to the PLT.

Mechanical Cone Penetration Tests

All eight test sites in the report had an MCPT sounding within 80 ft of the PLT at each site; however, the two MCPT soundings close to the PLT at site 3 at Apalachicola Bay were not deep enough to perform comparative analysis. All of the MCPT sounding data were taken directly from the FDOT sounding data included in the plans for contracting the PLT work at each of the sites. Detailed information about the sounding at each site is contained in the following chapter. Much of the

MCPT sounding data in Knox's data base which was transferred to floppy discs was found to be unuseable due to some inaccurate readings and unit conversion problems, so the sounding data was taken directly from the plotted sounding logs. The FDOT sounding logs were plotted with only cone resistance and friction ratio values in units of tsf and % respectively. Knox did not use the MCPT data in his 1989 PhD research, and he warned that the MCPT data base would need review and editing. Very little spike editing was found necessary for the MCPT soundings.

Electric Cone Penetration Tests

All eight test sites in the report had an ECPT sounding within 74 ft of the PLT at each site. The ECPT soundings at Port Orange were performed by FDOT, while the soundings at all other sites were performed with UF equipment and personnel. The FDOT Port Orange sounding data was compiled in 25 cm depth increments, whereas the UF sounding data at the other sites was compiled in normal 5 cm depth increments. Detailed information about the ECPT sounding at each site is contained in the following chapter. The main problem encountered with the UF ECPT sounding data was with the negative friction resistance values which were encountered at sites 19 and 20. The method for handling these negative values is explained in the individual discussion of each of the sites in the next chapter. In addition, a 4.45 m gap in the ECPT sounding data for site 3 required adjustment of depth readings and a 0 value for cone resistance, friction resistance, and friction ratio readings in the gap. The gap corresponded to a soft clay region identified in the corresponding boring log.

All of the ECPT soundings using UF equipment were performed with a 10 ton penetrometer tip. For cone resistance measurements greater than 105 tsf using the 10 ton penetrometer tip, Knox found the Qc values to be generally within 1% of actual values. Qc values below 105 tsf were generally within 4% of actual values. Friction resistance measurements were within 1 to 3% of actual values. The Qc noise rate was within acceptable limits at 0.00046 MPa/kPa (Knox, 1989e). No base line drift or inclination readings were available for the FDOT ECPT readings at Port Orange.

Of the six UF-tested ECPT soundings at Choctawhatchee and Apalachicola, only one site (site 1) exhibited a problem with excessive base line drift, and none appeared to have excessive inclination. Mr. Auxt at Hogentogler and Company, Inc. said that a tolerable difference in before and after base line drift measurements was 1 to 1.5% of the full scale reading. The latter amounted to 1.5 MPa (15.7 tsf) for cone resistance measurements and 15 kPa (0.157 tsf) for friction resistance measurements. Site 1 at the

Apalachicola River bridge had a 32 kPa difference between before and after base line friction resistance readings which was more than desired. Inclination readings for the Apalachicola and Choctawhatchee ECPT soundings to the depths used for pile capacity analysis were less than the 1 degree per meter limit recommended by Robertson and Campanella (1984b). However, the inclinations at sites 3 and 21 were shown as 0 degrees which made it possible the inclinometer was not functioning for these soundings. No pore pressure readings were used in this report, as per Knox's recommendation, due to troubles he experienced with pore pressure readings while performing the UF ECPT soundings.

Computer Software

The computer program used for making MCPT pile capacity predictions was a modified version of the UF Geotechnical Engineering Department's computer program designated MCPTUFR. Dr. McVay modified the program so it would predict pile capacity based on MCPT sounding data which only included cone resistance and friction ratio values. The primary disadvantage of the modified program was that a data file could not be read in to the program for making a pile capacity prediction. Each individual cone resistance and friction ratio value for each 20 cm depth increment had to be entered in to the program interactively which made each pile capacity prediction a fairly lengthy process. Some pile capacity predictions made by the modified program were checked by hand and found to be accurate. The hand calculation of pile capacity determined by the MCPT is shown in the Appendix and is compared to the MCPTUFR prediction for the Port Orange Bent 2 site.

The computer program for ECPT pile capacity predictions was recently developed by the UF Geotechnical Engineering Department and was designated PLAID. A manual outlining the capabilities of the program was published for a July 1989 workshop held by the UF Department of Civil Engineering. The program was essential to the completion of this report as its broad capabilities, rapid data interpretation and analysis, plotting capability, and user friendly design made ECPT pile capacity predictions a fairly quick and easy proposition.

Only two minor difficulties were encountered in the use of the PLAID program. The first was in the pile capacity output portion of the program. The average cone resistance and friction resistance values identified for a given soil layer were both shown as being in units of tons per square foot (tsf). The average cone resistance value shown in the output is actually in MPa, and the average friction resistance value is actually in kPa. The latter is a minor problem which can be corrected, but is important to know when analyzing average cone resistance and friction resistance values for a particular soil layer. The second difficulty was also minor, but since it caused confusion with fellow students using the PLAID program it bore mentioning. When pile geometry data was put into the program, the following three values were required input at the end of the pile geometry data set: the ground surface elevation, depth to water table, and depth at start of test. Confusion about the inputs required for each of these values was best alleviated by simply remembering the first of these three inputs requires an elevation, while the remaining two inputs are depths. For example, a site with a ground surface elevation of +6 ft, with a water table at an elevation of 1 ft (5 ft below the ground surface), and an ECPT sounding that was started at the ground surface would require input values of 6 ft and 5 ft and 0 ft respectively for ground surface elevation, depth to water table, and depth at start of test. A pile capacity prediction was made by hand using the same determination procedures employed in PLAID to verify the accuracy of the PLAID program. The hand solution and PLAID solution results were nearly identical. The hand calculation of pile capacity determined by the ECPT is shown in the Appendix and is compared to the PLAID prediction for the Port Orange Bent 2 site.

Computer Program Defaults and Limits

Various limits and default values were contained in both computer programs. The limits were adjusted on the MCPTUFR program to ensure they matched the limits used in the ECPT PLAID program. Most importantly, the maximum values for allowable pile tip capacity, individual cone resistance, and individual friction resistance were set at 150 tsf, 300 tsf, and 1 tsf respectively for both computer programs. Both programs evaluated the design end bearing pile capacity from between 8B above and 3.75B below the pile tip depth. In determining the ultimate pile capacity, both programs used a factor of safety of 3 for design end bearing and a factor of safety of 2 for design side friction. In addition, a unit weight of 100 psf was assumed for clay and cohesive materials and 110 psf was assumed for sand and cohesionless materials. In the PLAID program, a cone bearing capacity factor of 15 and an area correction factor for piezocone of 0.82 were also assumed for ECPT analysis.

CHAPTER 4

INDIVIDUAL SITE TEST RESULTS AND DISCUSSION

Choctawhatchee Bay Bridge - Flat Slab Bent 3

Load Test

Choctawhatchee Bay Bridge Flat Slab Bent 3 (FSB 3) was designated as site 19 in Knox's 1989 PhD dissertation. The pile load test (PLT), designated P019, was performed Dec 88 on pile 2 at station 111 + 11.8 and 64 ft left of the centerline of the old existing bridge. The ground surface elevation (GSE) was +2.00 ft after 4 ft of surface excavation, and the pile tip elevation (PTE) was -75.62 ft. The length of the pile below ground surface after excavation was 77.70 ft including about an inch of pile tip movement during the first load test. The first load test was the one used for analysis. The total length of the 24 inch square, solid, prestressed concrete pile was 83.91 ft. The top 5 ft of the pile was plugged solid for driving. Prior to load testing, the pile hole was bored and slurried to an elevation of -25 ft to bypass a layer of very soft, loose, silty clayey sand material. The boring log summary, shown in the contract drawings as Hole No. 1 (Figure 4-1) at station 111 + 00 and 20 ft left of existing bridge centerline confirms the presence of the previously described

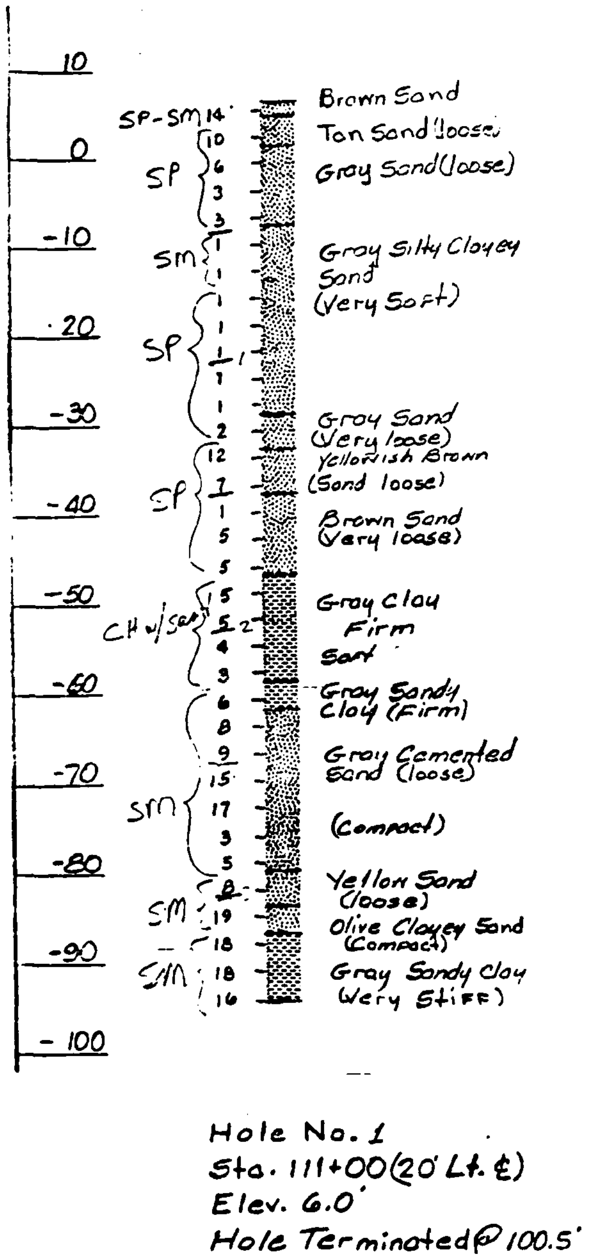


Figure 4-1. Boring Log Summary, Choctawhatchee Bay Bridge FSB 3 (Site 19)

material. The boring was about 46 ft from the PLT. As a result of the predrilling and slurrying, the pile capacity determined by the PLT was not exactly comparable to pile capacities predicted by normal ECPT and MCPT analysis since there was no side friction on the 25 ft of pile below the ground surface and surrounded by the slurry. Final PLT pile capacity analysis was not available yet from Schmertmann and Crapps, Inc. However, the PLT load and deflection data were plotted in Figure 4-2. A Young's Modulus (E) of 4,415,000 psi was assumed as per American Concrete Institute (ACI) criteria. The ultimate pile capacity was then determined to be 248 tons using the conservative Davisson's criteria.

ECPT

The ECPT sounding used for comparison to the result of the pile load test, designated C019D, was located at station 111 + 00 and 25 ft left of the existing bridge centerline. The sounding was 40.7 ft from the PLT, and it was 101.7 ft deep. The sounding was performed 29 June 88 using a ten ton tip with the UF Geotechnical Engineering Department's ECPT truck. Both Qc and Fs base line readings before and after the test were within tolerable limits. The difference between the Qc base line readings before and after the test was 0 MPa, and the difference between the Fs base line readings before and after the test was 5 kPa. Inclination of the rods to

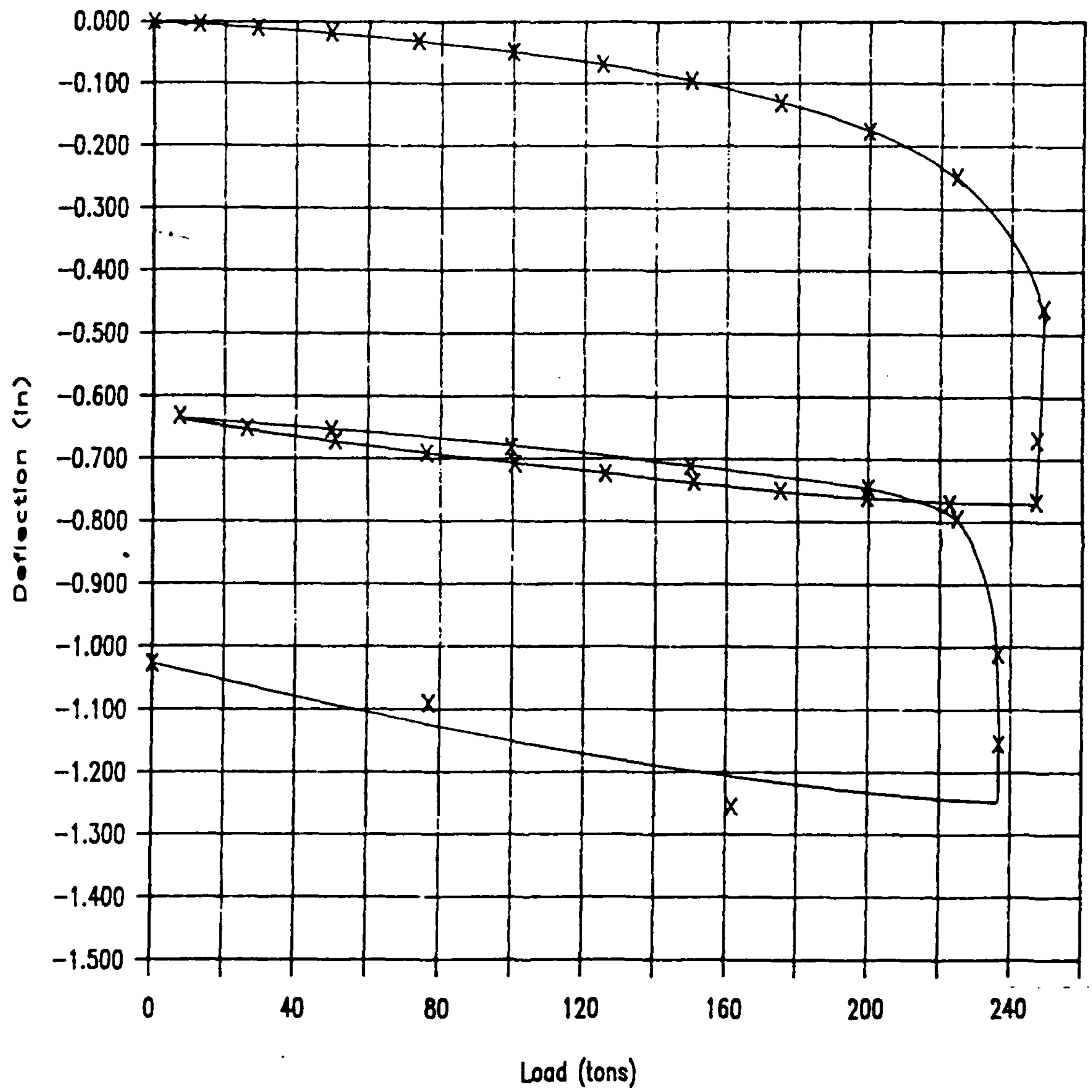


Figure 4-2. Pile Load Test, Load-Settlement Plot, Choctawhatchee Bay Bridge FSB 3 (Site 19)

the depth required for pile analysis was 3.2 degrees. Qc and Fs values from the ECPT surface elevation of +6.0 ft to the slurried depth of -25 ft elevation were adjusted to 0 so the sounding would be truly comparable to the conditions of the PLT. Sounding C019D contained many small, negative Fs readings between -1 and -10 kPa. Therefore, 10 kPa was added to all Fs values below an elevation of -25 ft. Consequently, a -10 kPa reading became 0 kPa and so on to the depth required for pile analysis. The sounding depth required for analysis was 89.2 ft which went from the true GSE of +6.0 ft to the -75.7 ft elevation and included the length 3.75B beyond the pile tip (B was the pile width). After interpreting the sounding with PLAID, a number of spikes for Qc, Fs, and FR were edited between 51 and 79 ft (15.6 and 24 meters). A summary of the edited sounding data used for the pile analysis was shown in Figure 4-3. After analyzing the sounding's PLAID program soil classification using Figures 2-6 and 4-3; a cohesionless layer was identified between 0 and 52.2 ft, a cohesive layer from 52.2 to 62.7 ft, and a cohesionless layer from 62.2 to 88.9 ft. According to the PLAID analysis, the middle layer identified as cohesive soil was on the borderline between cohesive and cohesionless soil. Figure 4-3 showed the soil in the middle layer ranged from silty sand to clayey silt. The decision to label the soil layer as cohesive was finalized after looking at

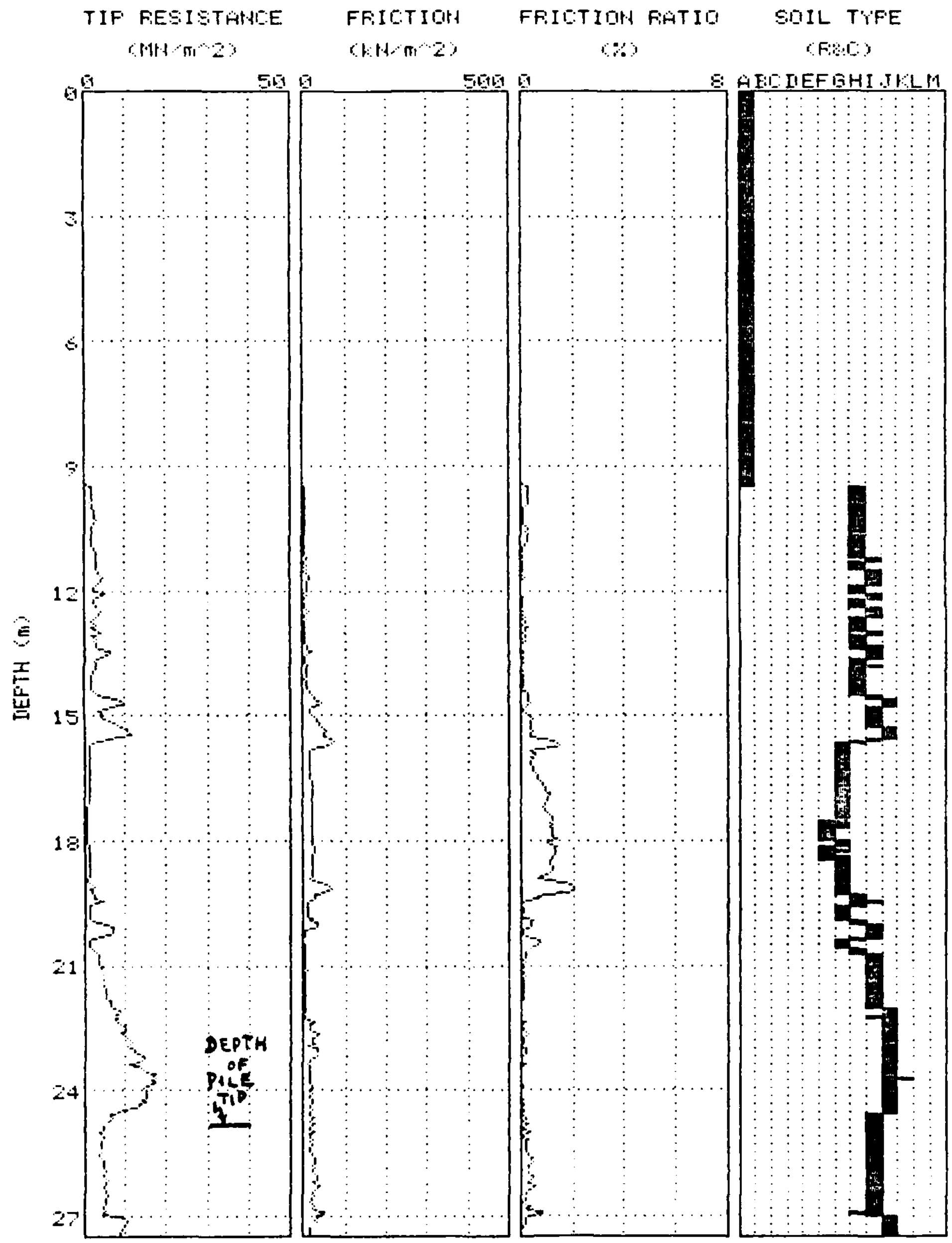


Figure 4-3. ECPT Sounding Data Summary, Choctawhatchee Bay Bridge FSB 3 (Site 19)

the boring log which identified soil in the same area as clay. The depths dividing the three soil layers compared favorably with the layer division depths identified by the nearby boring. Before the PLAID analysis, the pile geometry data used for ECPT analysis was changed slightly from that shown for the PLT. In order to ensure the pile tip in the ECPT analysis was at the same elevation as in the PLT, the pile length below the ground surface for the ECPT analysis was 81.7 ft (PLT pile length below ground surface was 77.7 ft) because the sounding was performed with a GSE of +6 ft vs. the PLT's post-excavation GSE of +2 ft. After entering the pile geometry data into PLAID, the ultimate pile capacity for the identical pile used in the PLT was 215 tons. The ECPT pile capacity analysis was also performed for the actual soil profile without the adjustment of the sounding Qc, Fs, and FR values to account for the slurry. Initially, it was surprising to see the ultimate pile capacity for the latter analysis was only 6% higher than the ultimate pile capacity determined by the analysis which included the adjustments for the slurry. However, recalling the boring log data, further review showed the majority of the soil replaced by the slurry was very loose sand and soft clay. The boring log identified cemented sand between the elevations of -64 and -80 ft, which might have been expected to cause an overprediction of design end bearing and pile capacity with the ECPT analysis. However, the latter was not the case.

MCPT

The MCPT sounding, designated M019D and used for comparison with the PLT and ECPT, was located at station 111 + 00 and 22 ft left of the existing bridge centerline. The test was 43.6 ft from the PLT, 3 ft from the ECPT, and it was 98.4 ft deep. The test was conducted by FDOT on 31 Jan 85. The test was designated as sounding 13 in the contract plans and was shown in Figure 4-4. This particular sounding was not contained in the data base compiled by Knox. As with the rest of the MCPT soundings contained in this report, only the Qc and FR values were shown in the sounding log.

As with all MCPT soundings in this report, designation of cohesive or cohesionless layers was done using Schmertmann's guide chart (Figure 2-5) in combination with the boring logs. Using only Schmertmann's chart and without the added enlightenment of the boring log, the soil stratigraphy for M019D to a depth of 88.6 ft appeared to contain the following five soil layers: cohesionless soil from 0 to 15.1 ft, cohesive soil from 15.1 to 43.3 ft, cohesionless between 43.3 and 49.9 ft, cohesive between 49.9 and 67.6 ft, and cohesionless from 67.6 to 89.2 ft. Use of the boring log however, confirmed the soil between 15.1 and 43.3 ft was

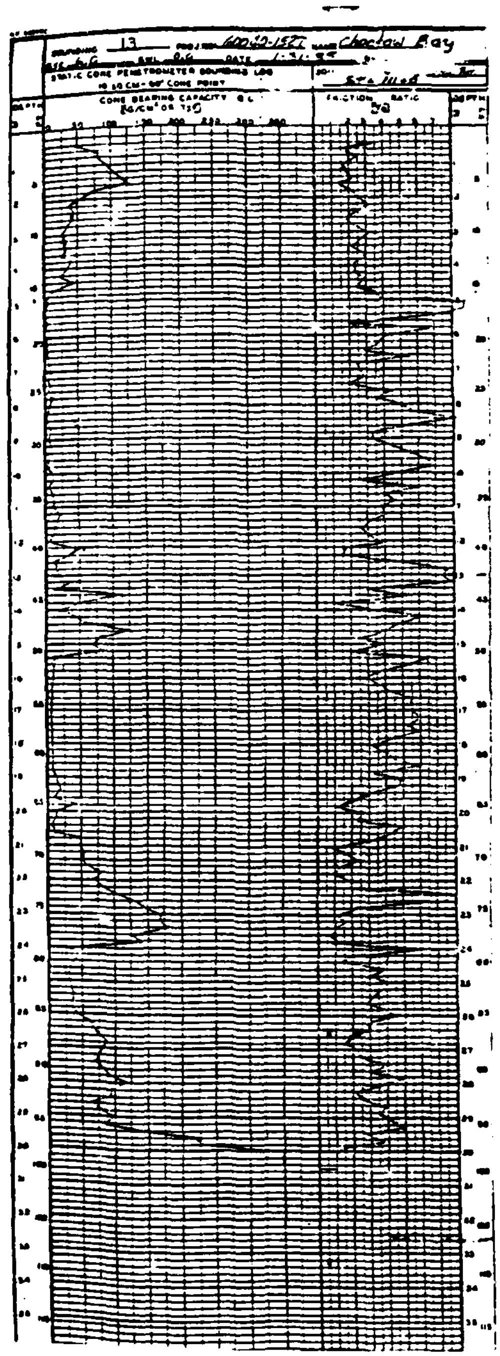


Figure 4-4. MCPT Sounding Data Summary, Choctawhatchee Bay Bridge FSB 3 (Site 19)

actually cohesionless reducing the soil profile to a three layer system. Nevertheless, it was significant the Qc and FR readings between 15 and 43 ft classified the soil as behaving like soft and organic clay and mixed soils as per Schmertmann's Guide Chart when it was classified as very soft, clayey silty sand and loose sand in the boring log.

The identical pile used in the PLT was used to determine the comparative pile capacity as determined by the MCPTUFR program. As with the ECPT, the pile length below ground surface used in the analysis was 81.7 ft since the GSE when the MCPT sounding was performed was +6 ft vs. the post-excavation GSE of +2 ft for the PLT. Qc and FR values were changed to 0.1 tsf and 0.1% respectively (values of 0 crashed the program) from the GSE at +6.0 ft to the slurried depth of -25 ft. Therefore, the first soil layer had no bearing on the MCPT pile prediction, and the second layer contributed minimally. The ultimate pile capacity determined for the three layer soil profile described in the preceding paragraph was 238 tons. The MCPT ultimate pile capacity prediction was 10 tons less than the PLT and 23 tons more than the ECPT ultimate pile capacity prediction.

Predicted vs. Observed Pile Capacity

For the sake of comparison, the pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-1.

Table 4-1 - Choctawhatchee Bay Bridge FSB 3, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P019	248				
ECPT C019D	215	82	31	51	- 13%
MCPT M019D	238	101	65	36	- 4%

The ECPT underpredicted the ultimate pile capacity determined by the PLT by 13%, while the MCPT underpredicted it by 4%. Upon initial inspection, the results compared very favorably with the PLT results. The effect of the cemented sand layer identified in the boring log between 64 and 80 ft might have been expected to cause overpredictions by both penetration tests. More detailed analysis of the ECPT and MCPT results was necessary to determine if there were any other reasons for disparities in predicted pile capacities.

Further comparison of the ECPT and the MCPT pile capacity results showed an 11% higher ultimate and 23% higher design capacity predicted by the MCPT. These differences were accompanied by a rather large disparity between the ECPT and MCPT design end bearing and design side friction predictions. Comparing the design side friction and end bearing values shown in Table 4-1 revealed the disparities. End bearing contributed to only 36% of the design pile capacity using MCPT pile analysis results, while it contributed 62% using ECPT results. Side friction contributed to 64% of the design pile capacity using the MCPT results, while it only contributed to 38% using ECPT results. The reason for the difference was found by analyzing the representative soil classification in each layer of cohesive or cohesionless soil delineated for both the MCPT and ECPT tests. Table 4-2 showed the difference in classifications for each layer and other pertinent data.

Even though the layer depth divisions corresponded well between the two tests, Table 4-2 clearly showed each layer using the MCPT was judged to contain more fines and was classified as more cohesive (based on average Qc and FR values for each layer) than the ECPT's corresponding layers. MCPT sounding average FR values were far higher than average ECPT sounding average FR values throughout the soil profile. Therefore, design side friction was in fact higher for the MCPT pile analysis. MCPT sleeve friction effects were known to cause greater Fs and FR values compared to ECPT values, but certainly not on the order exhibited in comparison of soundings M019D (MCPT). and C019D (ECPT). MCPT friction ratios in cohesionless

Table 4-2 - Detailed Comparison of MCPT and ECPT Soundings
Choctawhatchee Bay Bridge FSB 3

Test	MCPT	ECPT
Layer 1 Depth (w/out slurry)	0 to 49.9 ft	0 to 52.2 ft
Soil Class.	Sandy to Silty Clay	Sand to Silty Sand
Avg Qc (tsf)	30	36
Avg FR (%)	3.75	0.24
Layer 1 Depth (w/ slurry)	0 to 49.9 ft	0 to 52.2 ft
Soil Class.	Slurry	Slurry
Avg Qc (tsf)	13	15
Avg FR (%)	1.70	0.04
Layer 2 Depth	49.9 to 67.6 ft	52.2 to 62.7 ft
Soil Class.	Medium Clay	Sandy to Clayey Silt
Avg Qc (tsf)	10	12
Avg FR (%)	4.29	1.01
Layer 3 Depth	67.6 to 89.2 ft	62.7 to 88.9 ft
Soil Class.	Clayey Sands and Silts	Sand to Silty Sand
Avg Qc (tsf)	74	73
Avg FR (%)	2.87	0.22

soils have been found to be up to about twice the value of ECPT friction ratios. Evidence of the latter is shown in the design curve in Figure 2-8 where the K term value for the MCPT is about one-half of the K term value for the ECPT (Schmertmann, 1978e). However, the ratio of MCPT to ECPT average FR values for this and the following seven sites was far higher than 2 to 1 for most given cohesionless soil layers.

The average Qc values in the second and third soil layers were nearly the same for the ECPT and MCPT as shown in Table 4-2. But the third layer of soil, which contained the much higher averge Qc, was nearly 5 ft thicker in the ECPT and accounted for the greater design end bearing in the ECPT pile analysis. Although the third soil layer did not play a part in the end bearing determination for either test, it was significant the average Qc values in the first soil layer were also very comparable for the ECPT and MCPT.

The similarity between the ECPT and MCPT determinations of the depth boundaries and Qc values did not extend to FR values and soil classification. The average FR in each of the layers was far higher for the MCPT analysis. The MCPT's higher FR and accompanyingly higher Fs, since the Qc values were nearly equal, resulted in soil classifications indicating the presence of more fines than found in the ECPT analysis. The latter may be seen in the soil classifications shown in

Table 4-2. The more cohesive classification of the soil layers by the MCPT was indicative of the higher design side friction shown in the MCPT pile capacity prediction in Table 4-1. The classification of layer 2 as a clay by the MCPT was more in line with the boring findings than the ECPT classification. The ECPT sounding showed a trend toward cohesiveness in layer 2 as shown in Figure 4-3, but did not have a high enough FR in layer 2 to classify the layer as a clay. On the other hand, classifications of layers 1 and 3 by the ECPT were more in line with the boring data than the MCPT classification of these layers. Using the boring log for comparison, the MCPT appeared to be better at the classification of the cohesive soil layer, but the ECPT was more accurate in the classification of the cohesionless soil layers.

A final point worth mentioning in the comparison of the MCPT and ECPT soundings was the ECPT sounding was apparently performed about 3 ft from the MCPT. Three and one-half years elapsed between the two soundings, but the earlier MCPT may have had some minor effects on the later ECPT. However, these effects were probably negligible.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-2) with ECPT predicted results (Figure 4-5) using the PLAID program. The PLT load-settlement plot was also shown in Figure 4-5 for ease of comparison. As shown in Figure 4-5, the ECPT load-settlement analysis was very conservative in

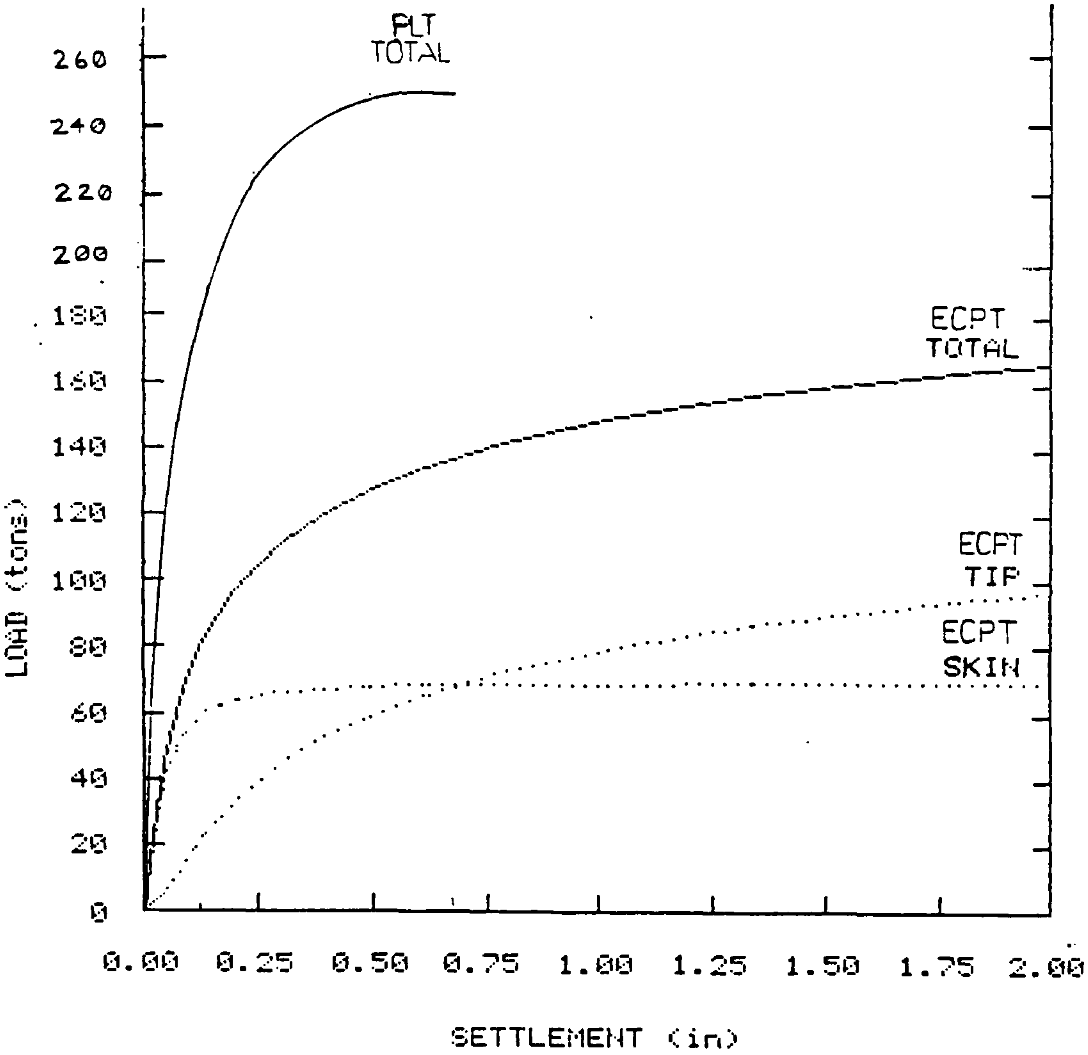


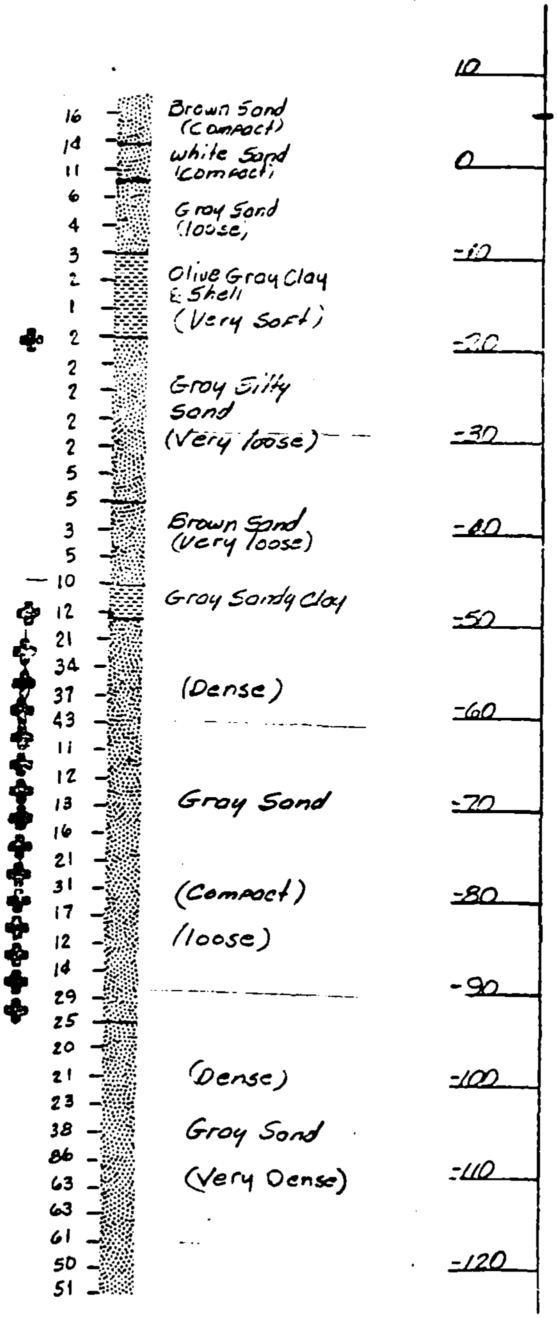
Figure 4-5. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Choctawhatchee Bay Bridge FSB 3 (Site 19)

comparison with the PLT results. End bearing at the pile tip appeared to mobilize faster in the PLT than predicted by the ECPT. The PLT values were extrapolated from the curve shown in Figure 4-2, because the test pile was unloaded and reloaded after 0.68 inches of settlement.

Choctawhatchee Bay Bridge - Pier 5

Load Test

Choctawhatchee Bay Bridge Pier 5 was identified as site 20 in Knox's dissertation. The pile load test was performed in Dec 88 on an out-of-position pile at station 120 + 69.8 and 62 ft left of the old existing bridge centerline. The GSE was -1.80 ft as there was excavation of 8.80 feet of surface soil before the PLT. The PTE was -58.8 ft. The pile length below the ground surface was 57.10 ft including an inch of tip movement that occurred during the test. The total length of the 30 inch square prestressed concrete pile with an 18 inch diameter hollow was 71.06 ft. The top 5 ft of the pile was plugged solid for driving. Prior to load testing the test pile hole was bored and slurried to an elevation of -30 ft. The preboring and slurry was necessary to bypass the same type of soil bypassed at the P019 site at FSB 3. A boring was located about 76 ft from the PLT. The boring log summary, shown in the contract drawings as Hole No. 4 (Figure 4-6) at station 120 + 00 and 33 ft left of the



Hole No. 4
Sta. 120+00 (33' Lt. ℄)
Elev. 7.0'
Hole Terminated @ 130.5'

Figure 4-6. Boring Log Summary, Choctawhatchee Bay Bridge Pier 5 (Site 20) existing bridge centerline, showed the very soft clay and loose sand bypassed with the preboring and slurry. Thus, the ultimate pile capacity determined by the PLT was not exactly comparable to capacities determined by normal ECPT and MCPT analysis since there was no skin friction on the first 30 ft of pile below the ground surface. In addition, the first 8.8 ft of both the ECPT and MCPT soundings was not useable in pile capacity analysis since that much soil was excavated prior to the PLT. Final pile capacity analysis was not available from Schmertmann and Crapps, Inc. However, the PLT load and deflection data were plotted in Figure 4-7. An E of 4,415,000 psi was assumed as per ACI criteria. Using Davisson's criteria, the ultimate pile capacity was determined to be 626 tons.

ECPT

The ECPT sounding chosen for analysis was designated C020B at station 120 + 00 and 38 ft left of the existing bridge centerline. The sounding was located 73.8 ft from the PLT, and it was 128.3 ft deep. The sounding was accomplished 27 Sept 87 using a ten ton tip with the UF ECPT truck. Both Qc and Fs baseline readings were within tolerable limits. The difference between the Qc base line readings before and after the test was 0.07 MPa, and the difference between the Fs base line readings before and after the test was 2 kPa. Inclination of the rods to

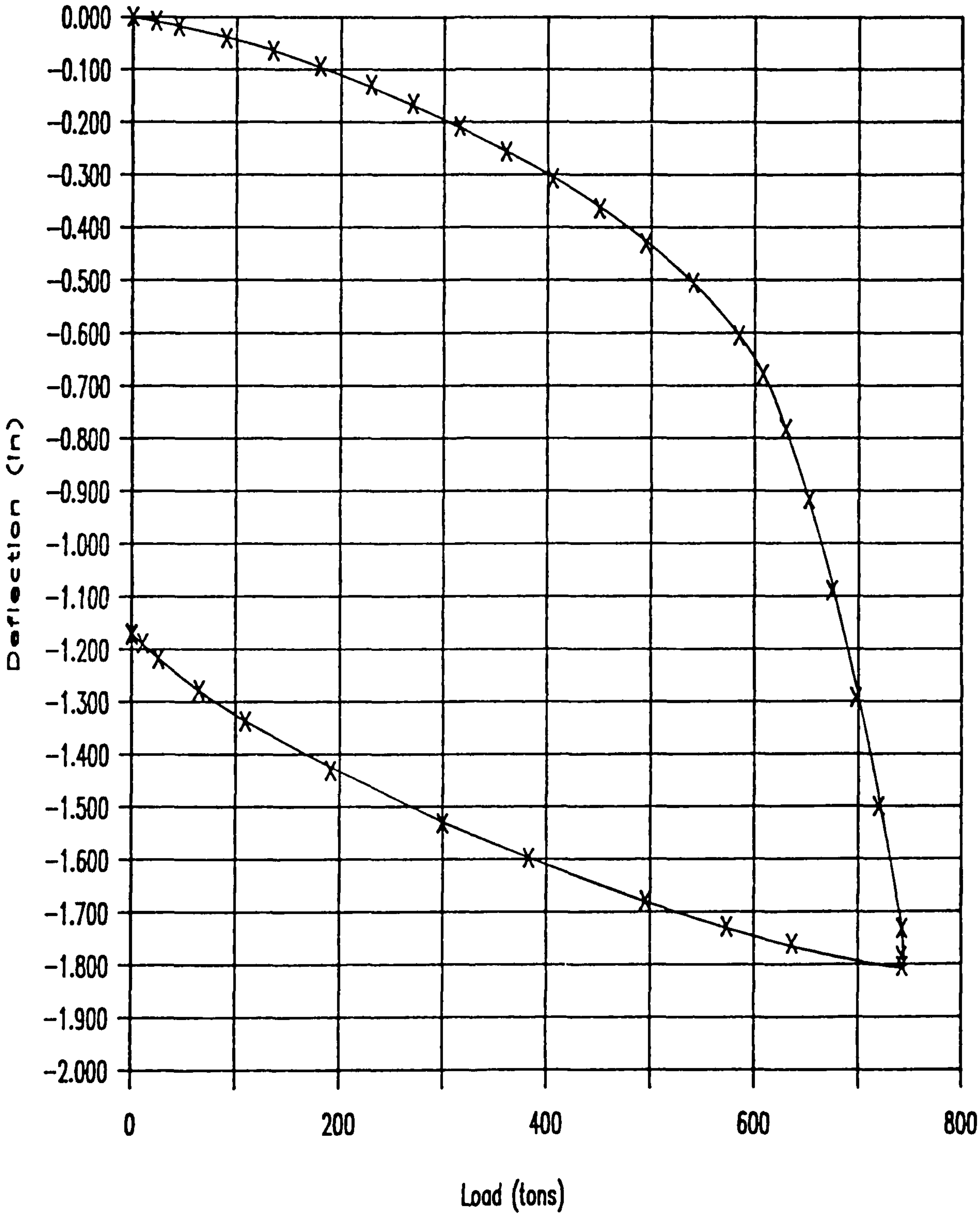


Figure 4-7. Pile Load Test, Load-Settlement Plot, Choctawhatchee Bay Bridge Pier 5 (Site 20)

the 75 ft depth necessary for pile analysis was 0.4 degrees. Sounding C020B contained many negative Fs values between -1 and -17 kPa particularly through the very soft and very loose soils encountered between 11.6 and 46.1 ft. Therefore, 15 kPa was added to all Fs values which changed a -15 kPa reading to 0 kPa and so on through the sounding to the depth required for pile analysis. There were a multitude of -15 kPa readings, so the latter value was judged to actually represent 0 kPa. The few -16 and -17 kPa readings were also changed to 0 kPa. Spikes in Qc, Fs, and FR values were edited with the PLAID program between 7.9 and 8.5 ft and also 54.8 and 55.4 ft. A summary of the edited sounding data used for pile analysis was shown in Figure 4-8. In order to make the sounding pile capacity prediction comparable to the actual PLT; the sounding's Qc, Fs, and FR values were all changed to 0 for the first 37 ft to account for the excavated and slurried soil down to an elevation of -30 ft. After analyzing the PLAID program's soil classifications; a cohesionless layer was identified between 0 and 17 ft, a cohesive layer from 17 to 37.5 ft, and a cohesionless layer from 37.5 to 78.5 ft. In general, the soil profiles identified by the ECPT and the boring were similar. The boring was only 5 ft from the ECPT sounding. The boring (Figure 4-6) showed cohesionless soil on either side of a cohesive layer between 17 and 27 ft below the surface, which agreed well

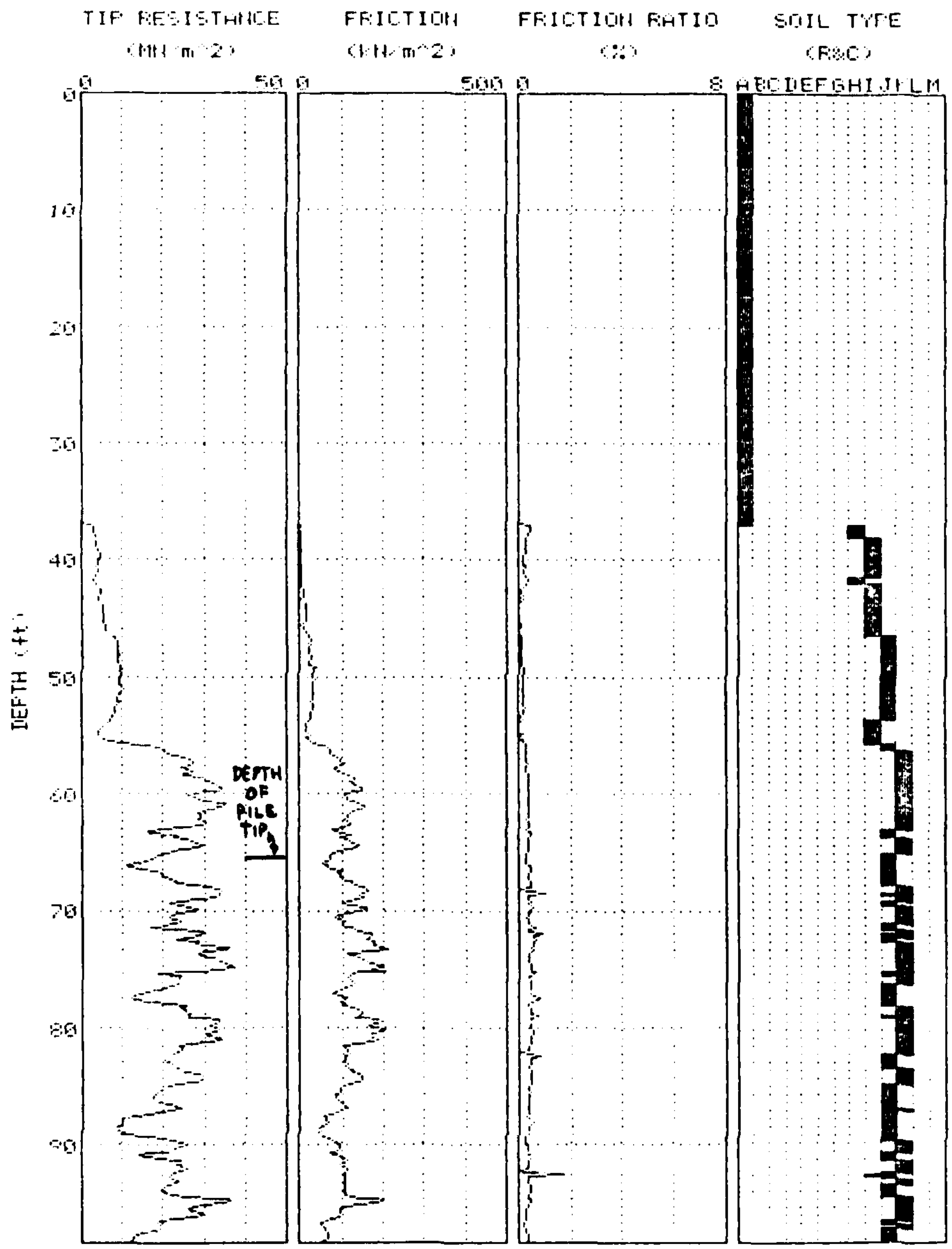


Figure 4-8. ECPT Sounding Data Summary, Choctawhatchee Bay Bridge Pier 5 (Site 20)

with the sounding results except the cohesive layer was 10 ft thicker in the ECPT sounding. The boring also showed a thin clay layer between 54 and 57 ft below the surface which was not classified as cohesive in the sounding although the tendency toward more fines at that depth was in evidence in the sounding classification plot in Figure 4-8. The first two soil layers were within the depth range of the assigned 0 values for Qc, Fs, and FR due to the slurry. Therefore, the first two soil layers were not factors in the sounding's pile capacity determination. A pile length below ground surface of 65.9 ft was used in the ECPT pile analysis so the pile tip would lie at the same depth as that used in the PLT. The pile length below ground surface was 8.8 ft longer than that shown for the PLT because the GSE for the sounding was 7 ft vs. the post-excavation GSE of -1.8 ft for the PLT. After entering the pile geometry data into PLAID, the ultimate pile capacity for a pile identical to the pile used in the PLT resulted in an ultimate pile capacity of 951 tons. Similar to site 19, the ECPT pile analysis using the actual soil profile with no adjustments for the slurry resulted in only a slightly higher (2%) ultimate pile capacity of 968 tons. The dense and possibly cemented sand located between the -50 and -61 ft elevation may have caused the substantial overprediction of the ultimate pile capacity determined by the ECPT analysis compared to the PLT. Unlike site 19 where the ECPT underpredicted the ultimate pile capacity, the pile tip in site 20 was located directly in the region of possibly cemented sand.

MCPT

The MCPT sounding, designated M020B and used for comparison with the PLT and ECPT, was located at station 120 + 00 and 22 ft left of the existing bridge centerline. The sounding was 80.4 ft from the PLT, 16 ft from the ECPT, and was 107.6 ft deep. The test was conducted by FDOT on 4 June 1985. M020B was designated as sounding 47 in the contract plans and was shown in Figure 4-9. Only Qc and FR values were recorded in the sounding plot. The soil profile represented by M020B consisted of three layers which almost exactly matched the profile determined by the ECPT (C020B). The top layer of soil was cohesionless and went from 0 to 17.1 ft followed by a cohesive layer between 17.1 and 36.8 ft. Finally, the third layer was identified between 36.8 and 78.7 ft where deeper analysis was unnecessary beyond the critical depth below the pile tip which was at 75.3 ft. The layer separations agreed well with the boring and ECPT layer divisions although, like the ECPT, the cohesive layer was a few feet thicker in the MCPT sounding than in the boring. In addition, the thin layer of clay shown in the boring at -54 to -57 ft also appeared in the three MCPT soundings between 55.1 and

56.43 ft (16.8 and 17.2 meters) as evidenced by the Qc and FR values (Figure 4-9). The very thin clay layer was not considered in the layer determination for the sounding, since its thinness rendered it insignificant. Leaving the thin clay layer as insignificant also made the MCPT sounding more comparable with the ECPT sounding for analysis purposes. As with C020B, the first two soil layers made no contribution to the pile capacity prediction, because they were within the depth range of the slurry used in the PLT.

The identical pile used in the PLT was used to find the comparative pile capacity using sounding M020B in the MCPTUFR program. However as in the ECPT, the pile length below ground surface used in the analysis was 65.9 ft to account for the fact the sounding was performed with a GSE of 7 ft without the 8.8 ft of excavation performed prior to the PLT. The ultimate pile capacity predicted by the MCPT was 1050.5 tons which exceeded the PLT ultimate pile capacity by 424.5 tons and the ECPT prediction by 99.5 tons.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-3.

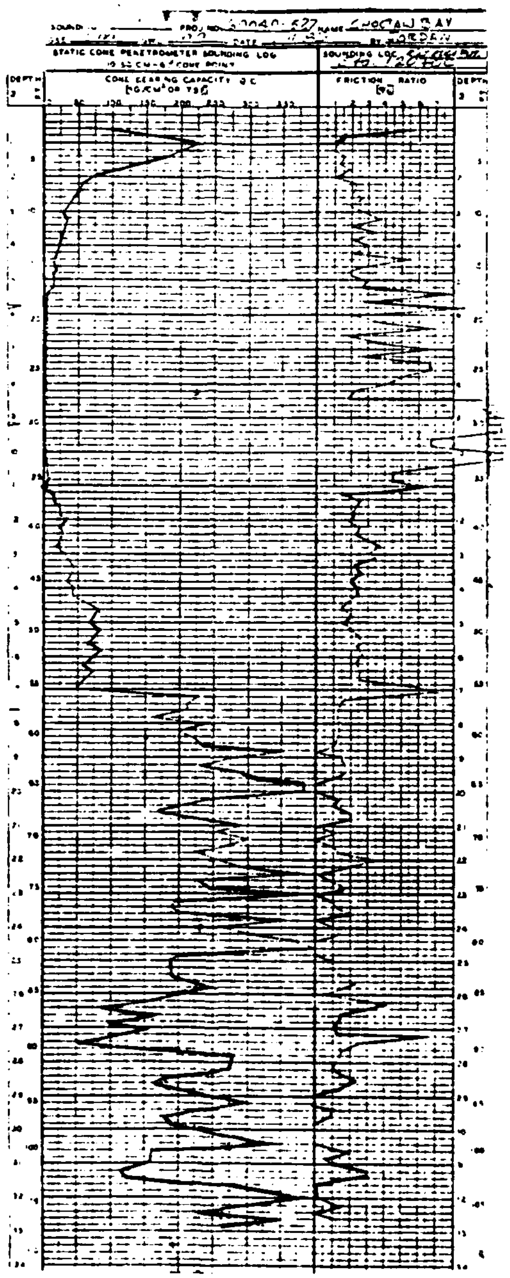


Figure 4-9. MCPT Sounding Data Summary, Choctawhatchee Bay Bridge Pier 5 (Site 20)

Table 4-3 - Choctawhatchee Bay Bridge Pier 5, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P020	626				
ECPT C020B	951	335	55.5	280	+ 52%
MCPT M020B	1050.5	369	56.5	312.5	+ 68%

The ECPT and MCPT each overpredicted the ultimate pile capacity determined by the PLT by 52% and 68% respectively. These overpredictions occurred despite the adjustment of Qc, Fs, and FR values to 0 (0.1 for MCPT) over the first 37 ft of both soundings to account for excavation and slurry used in the PLT. Without the 37 ft of 0 values; the design side friction, design pile capacity, and ultimate pile capacity predictions would have been even higher for both the ECPT and MCPT soundings. The latter was proven for the ECPT analysis as mentioned in the ECPT section for this site. The large design end bearing prediction for both cone penetration tests was the main contributor to the pile capacity overprediction compared to the PLT as shown in Table 4-3. The design end bearing prediction of 312.5 tons for the MCPT prediction was the maximum for the pile analyzed. The high Qc values, both 3.75B below (B = 2.5

ft pile width) and 8B above the pile tip, produced the maximum pile tip capacity of 150 tsf. When the 150 tsf was multiplied by the tip area of the 2.5 ft square pile and divided by the factor of safety of three, the design end bearing value of 312.5 tons resulted. The ECPT Qc values near the the pile tip were also very high resulting in the design end bearing of 280 tons. The dense and possibly cemented sand in the vicinity of the pile tip produced the high Qc values which resulted in the high design end bearing for both the ECPT and the MCPT pile capacity predictions. More detailed analysis of the two soundings was required to find any other possible reasons for the pile capacity overprediction compared to the PLT.

The MCPT analysis predicted a 10% higher ultimate and 10% higher design capacity compared to the ECPT analysis. The design side friction values were nearly equal for the two tests. Comparing design side friction and end bearing values showed the side friction contributed to 15% of the design pile capacity using the MCPT, while it contributed to 17% of the ECPT design pile capacity. Thus, design end bearing contributed to 85% of the design pile capacity using the MCPT and it contributed to 83% of the ECPT design pile capacity. Therefore, in terms of percentages, side friction and end bearing contributions to the design and ultimate pile capacities predicted by both the MCPT and the ECPT were nearly the same. The magnitude of the end bearing values using the MCPT were simply larger than values found with the ECPT. The reason for the latter can be seen in Table 4-4.

Taking into account the 30 ft of slurry used in the PLT, the third soil layer was the only soil affecting the pile capacity predictions by both MCPT and ECPT analysis. The design end bearing for the MCPT was 312.5 tons which was about 12% higher than the ECPT design end bearing of 278 tons. The design end bearing relationship seemed incongruous with the data since the ECPT average Qc was actually greater than the MCPT average Qc. The soil depths lying from 3.75B below the pile tip to the terminal depth of analysis (75.3 to 78.7 ft) were analyzed to see if Qc values within the latter region had skewed the average Qc values for both tests. More specifically, the analysis was performed to determine whether the MCPT average Qc was lowered by Qc values below the 3.75B depth at 75.3 ft and if the ECPT average Qc was raised by Qc values below the 3.75B depth. If the latter were true, then the design end bearing disparity could be easily explained. However, the latter was not the case. The MCPT average Qc from 3.75B below the pile tip (75.3 ft) to the analysis termination depth of 78.7 ft was 261 tsf. The ECPT average Qc over the same depth range was 211 tsf. So both tests had their average Qc values in layer three raised by the values below 75.3 ft, and the MCPT even more than the ECPT.

Further analysis showed the reason for the difference in design end bearing between the two cone penetration tests as the average Qc 1.64 ft (0.5 m) above and below the pile tip at 65.9 ft was 280 tsf for the MCPT and 211 tsf for the ECPT. In the same region close to the pile tip, the minimum MCPT Qc value was 172 tsf compared to 116 tsf for the minimum ECPT Qc value. Using the minimum path method for design end bearing determination, the Qc values close to the tip were critical. So the lower ECPT Qc values near the tip led to the lower design end bearing in the ECPT pile analysis in comparison to the MCPT pile analysis.

The reasons for the greater design end bearing value for the MCPT pile analysis compared to the ECPT analysis were outlined above. Both tests' overprediction of pile capacity compared to the PLT had to have been due to overprediction of the end bearing capacity of the sand in the third soil layer. Side friction contributions were minimal in comparison to end bearing due to the slurry effects and the cohesionless nature of the third soil layer. In addition, the design side friction values from each test were almost equal. Looking at the classifications of the third layer of soil by both the MCPT and ECPT shown in Table 4-4, it was certainly concievable that cementation of sand in the third soil

Table 4-4 - Detailed Comparison of MCPT and ECPT Soundings
Choctawhatchee Bay Bridge Pier 5

Test	MCPT	ECPT
Layer 1 Depth (w/out slurry)	0 to 17.1 ft	0 to 17.0 ft
Soil Class.	Clayey Sands and Silts	Sand
Avg Qc (tsf)	81	96
Avg FR (%)	2.36	0.18
Layer 1 Depth (w/ slurry)	0 to 17.1 ft	0 to 17.0 ft
Soil Class.	Slurry	Slurry
Avg Qc (tsf)	0	0
Avg FR (%)	0.10	0
Layer 2 Depth (w/out slurry)	17.1 to 36.8 ft	17.0 to 37.5 ft
Soil Class.	Organic Clays/Mixed Soils	Clayey Silt to Silty Clay
Avg Qc (tsf)	3	10
Avg FR (%)	5.66	0.93
Layer 2 Depth (w/ slurry)	17.1 to 36.8 ft	17.0 to 37.5 ft
Soil Class.	Slurry	Slurry
Avg Qc (tsf)	0	0
Avg FR (%)	0.10	0
Layer 3 Depth	36.8 to 78.7 ft	37.5 to 78.5 ft
Soil Class.	Dense or Cemented Sand	Sand to Gravelly Sand
Avg Qc (tsf)	169	176
Avg FR (%)	1.93	0.30

layer may have caused the end bearing and pile capacity overpredictions compared to the PLT.

Because of the excavation and slurrying, the first two soil layers did not contribute to the pile capacity analysis. But comparison of the average Qc, FR, and resulting soil classifications in the top two soil layers shed more light on the comparability of the MCPT and ECPT. Table 4-4 showed that in both soil layers the average Qc was lower and the average FR was higher for the MCPT compared to the ECPT. The average Qc values were very comparable, but the average FR values were drastically different. As a result, the MCPT soil classifications in both layers reflected a greater fines content compared to the ECPT soil classification. The MCPT classified the first layer as a clayey sand or silt, while the ECPT classified the layer as only sand. The boring log identified the first soil layer as sand as well. The ECPT classification agreed more with the boring log in its soil classification of the cohesionless first soil layer. The MCPT classification of the second layer as organic clay and/or mixed soils agreed with the boring log's description of soil in the same region. The boring log classified the same soil layer as clay and silty sand. The ECPT classified the second layer as clayey silt to silty clay. Each test's classification of the second soil layer could be considered accurate. But the MCPT definitely characterized the soil as containing more fines than the soil classification identified by the ECPT. The tremendous difference in FR values between the MCPT and ECPT was the main cause of the disparity in soil classifications. The MCPT FR appeared to be more accurate for the cohesive second soil layer, while the ECPT was more accurate for the cohesionless first soil layer. But the MCPT's characterization of the third soil layer as a dense or cemented sand appeared to be more on target than the ECPT, which had too low of an average FR value to suggest the presence of cementation.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-7) with the ECPT predicted results (Figure 4-10). The PLT load-settlement plot was also shown in Figure 4-10 for ease of comparison. As shown in Figure 4-10, the ECPT load-settlement analysis was very conservative compared to the PLT results. The end bearing appeared to have mobilized faster with the PLT than predicted by the ECPT.

Choctawhatchee Bay Bridge - Flat Slab Bent 26

Load Test

Choctawhatchee Bay Bridge Flat Slab Bent 26 (FSB 26) was designated as site 21 in Knox's 1989 PhD dissertation. The pile load test (PLT), designated P021, was performed 2 Dec 88 on pile 3 at station 183 + 15.8 and 54 ft left of the centerline of the old existing

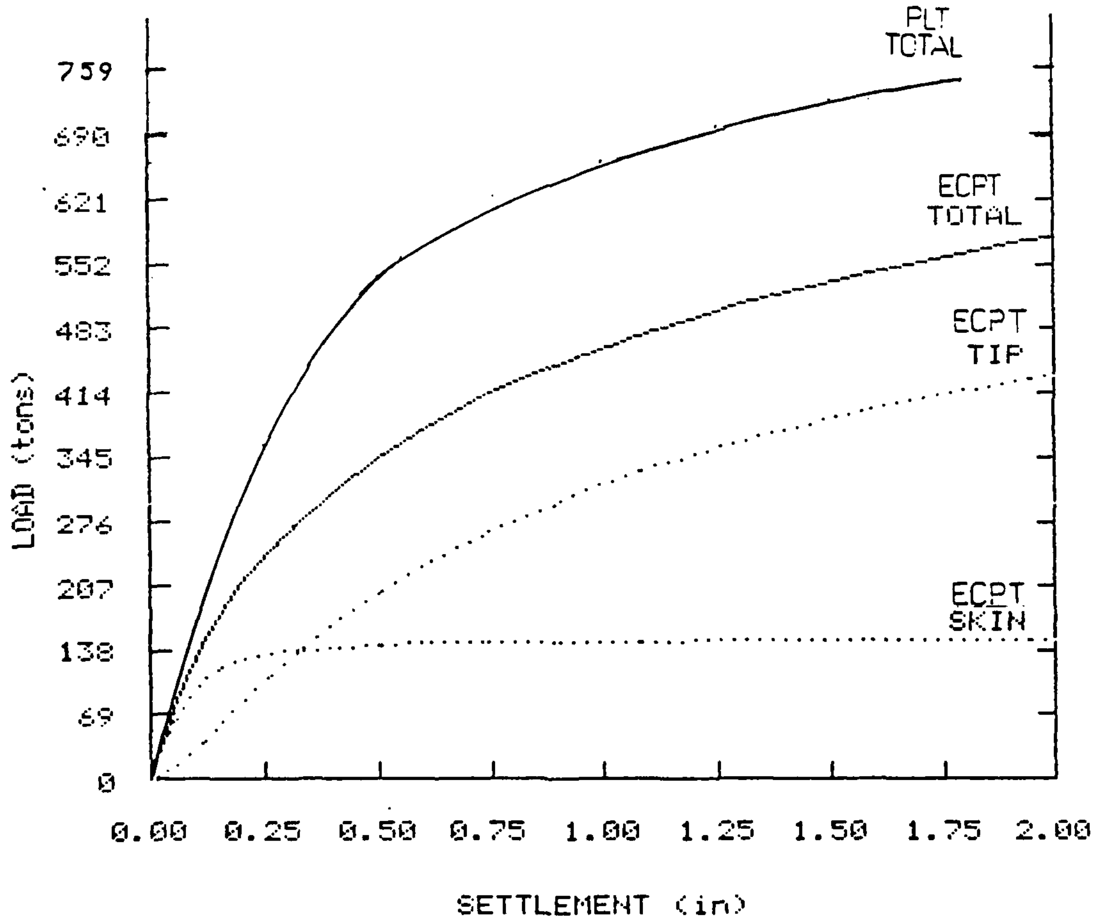


Figure 4-10. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Choctawhatchee Bay Bridge Pier 5 (Site 20)

bridge. The 54 ft is actually an approximation as the old and new bridge centerlines began curving closer together right near station 183 + 00, but 54 ft left of centerline was a good approximation. The ground surface elevation was +2.00 ft after 4 ft of surface excavation, and the pile tip elevation was -62.84 ft. The length of the test pile below ground surface after excavation was 64.88 ft with about a half inch of pile tip movement during the first load test. After having its original pile length of 84 ft cut before the test for easier handling, the total length of the test pile was 69 ft. The prestressed concrete pile was 30 inches square with an 18 inch diameter hollow. The top 5 ft of the pile was plugged solid for driving. Prior to load testing, the pile hole was bored and slurried to an elevation of -60 ft to prevent driving the test pile through a layer of very soft, loose, silty clayey sand material. The boring log summary, shown in the contract drawings as Hole No. 27 (Figure 4-11) at station 180 + 50 and 50 ft left of existing bridge centerline confirmed the presence of the latter material. Some compact sand and soft clay was also identified by the boring within the depth range of the slurry, but the boring was 266 ft from the PLT so some difference in the soil profile was expected. As a consequence of the deep predrilling and slurrying, the pile capacity determined by this PLT was not exactly comparable to pile capacities predicted by normal ECPT

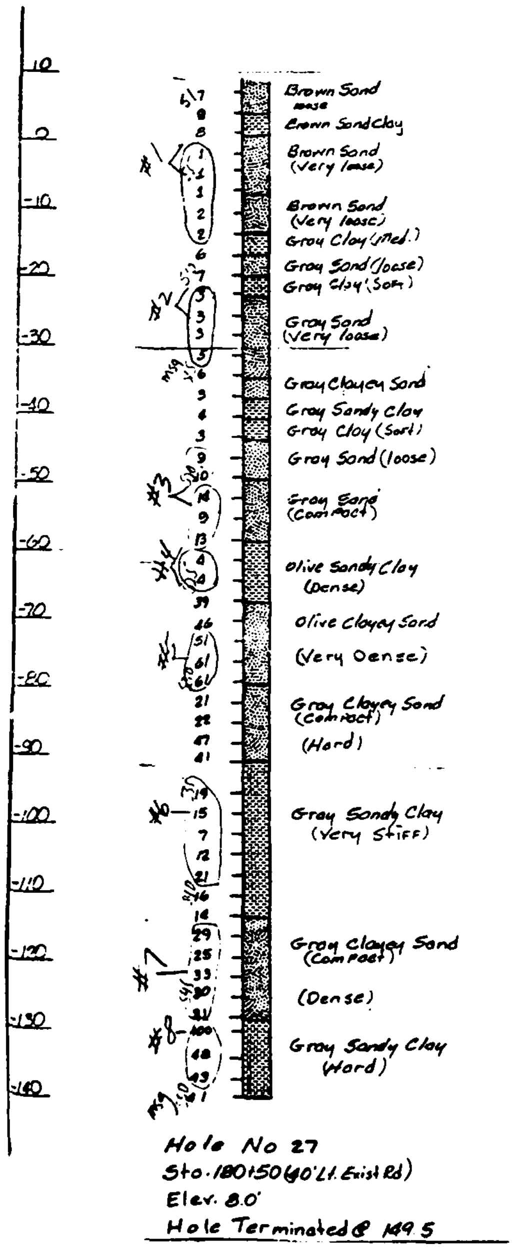


Figure 4-11. Boring Log Summary, Choctawhatchee Bay Bridge FSB 26 (Site 21)

and MCPT analysis since there would be no side friction on the 62 ft of test pile below the ground surface in the slurry. Final PLT pile capacity analysis was not available yet from Schmertmann and Crapps, Inc. However, the PLT load and deflection data were plotted in Figure 4-12. A Young's Modulus (E) of 4,415,000 psi was assumed as per American Concrete Institute criteria. From the load-settlement plot in Figure 4-12, the ultimate pile capacity was determined to be 481 tons using the conservative Davisson's criteria.

ECPT

The ECPT sounding used for comparison to the result of the PLT and designated C021A, was located at station 183 + 16 and 26 ft left of the existing bridge centerline. The sounding was 28 ft from the PLT, and it was 80.7 ft deep. The sounding was performed 28 Sept 88 using a ten ton tip with the UF Geotechnical Engineering Department's ECPT truck. Both Qc and Fs base line readings before and after the test were within tolerable limits. The difference between the Qc base line readings before and after the test was 0.40 MPa, and the difference between the Fs base line readings before and after the test was 9 kPa. The sounding log showed no inclination of the rods to the depth required for pile analysis. There were no negative Fs values in the sounding. A summary of the sounding data was shown in

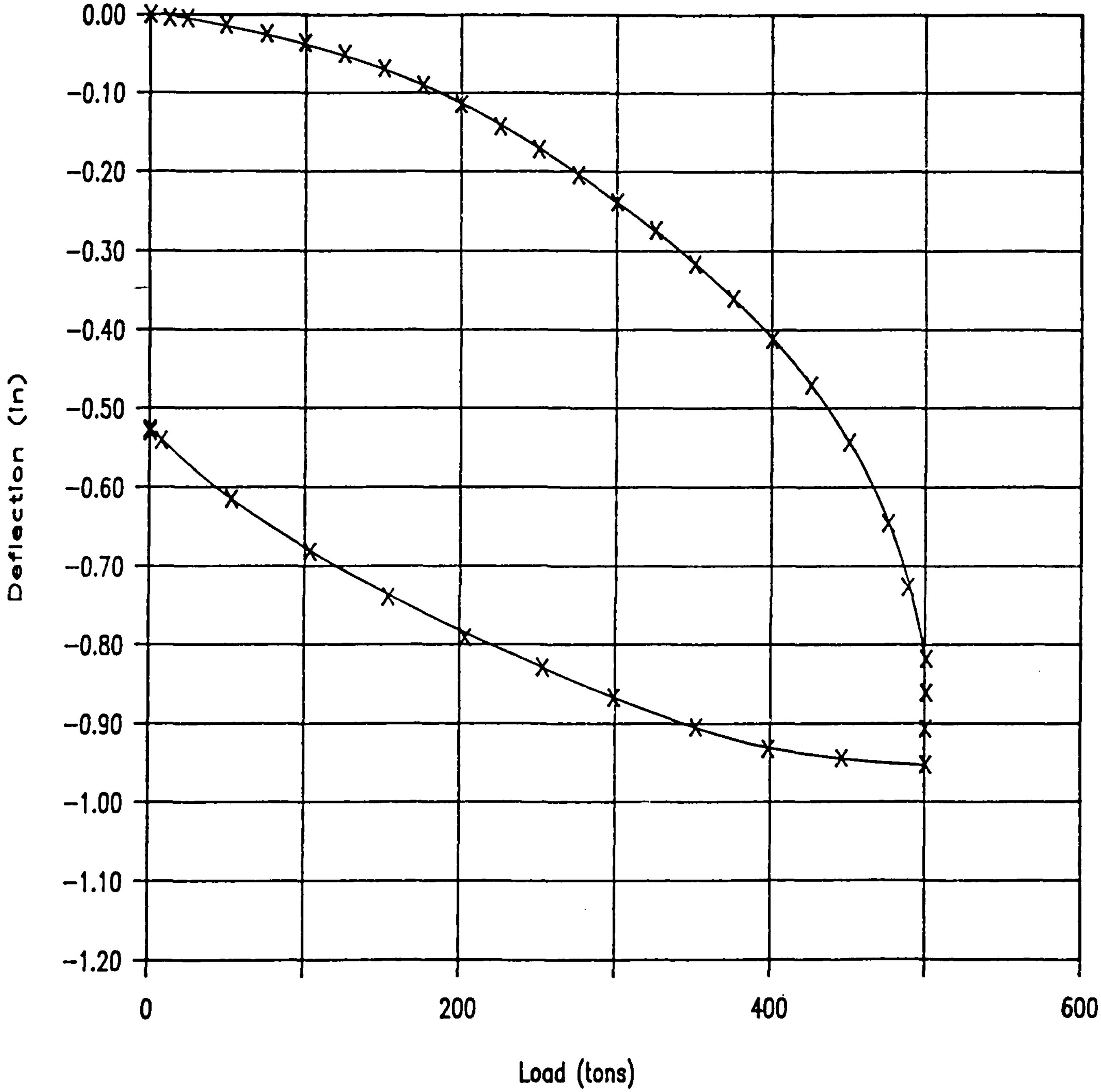


Figure 4-12. Pile Load Test, Load-Settlement Plot, Choctawhatchee Bay Bridge FSB 26 (Site 21)

Figure 4-13. Layer determinations in the slurried region were not critical for comparative pile analysis. Nevertheless, careful layer determinations were made so they could be compared with MCPT layer determinations. A five layer soil profile was identified with a cohesionless layer from the ground surface to 9.1 ft below the ground surface, a cohesive layer from 9.1 to 52.4 ft, a cohesionless layer from 52.4 to 57.2 ft, a cohesive layer from 57.2 to 64.4 ft, and a cohesionless layer from 64.4 to 80.6 ft. In order to ensure the pile tip in the ECPT analysis was at the same elevation as in the PLT, the pile length below the ground surface for the ECPT analysis was 68.88 ft (PLT pile length below ground surface was 64.88 ft) since the sounding was performed with a GSE of +6 ft vs. the PLT's post-excavation GSE of +2 ft.

An attempt was made to edit the sounding data to account for the excavation and slurry in the PLT. Qc and Fs values from the ECPT surface elevation of +6.0 ft to the slurried depth of -60 ft elevation were adjusted to 0 so the soundin~ would be truly representative of the conditions of the PLT. But when the PLAID program was run for pile capacity analysis with the 66 ft of 0 values for Qc, Fs, and FR; the program crashed. Consequently, it was decided to run the actual, unedited sounding data for the pile analysis with only the design end bearing determination representing the design pile capacity.

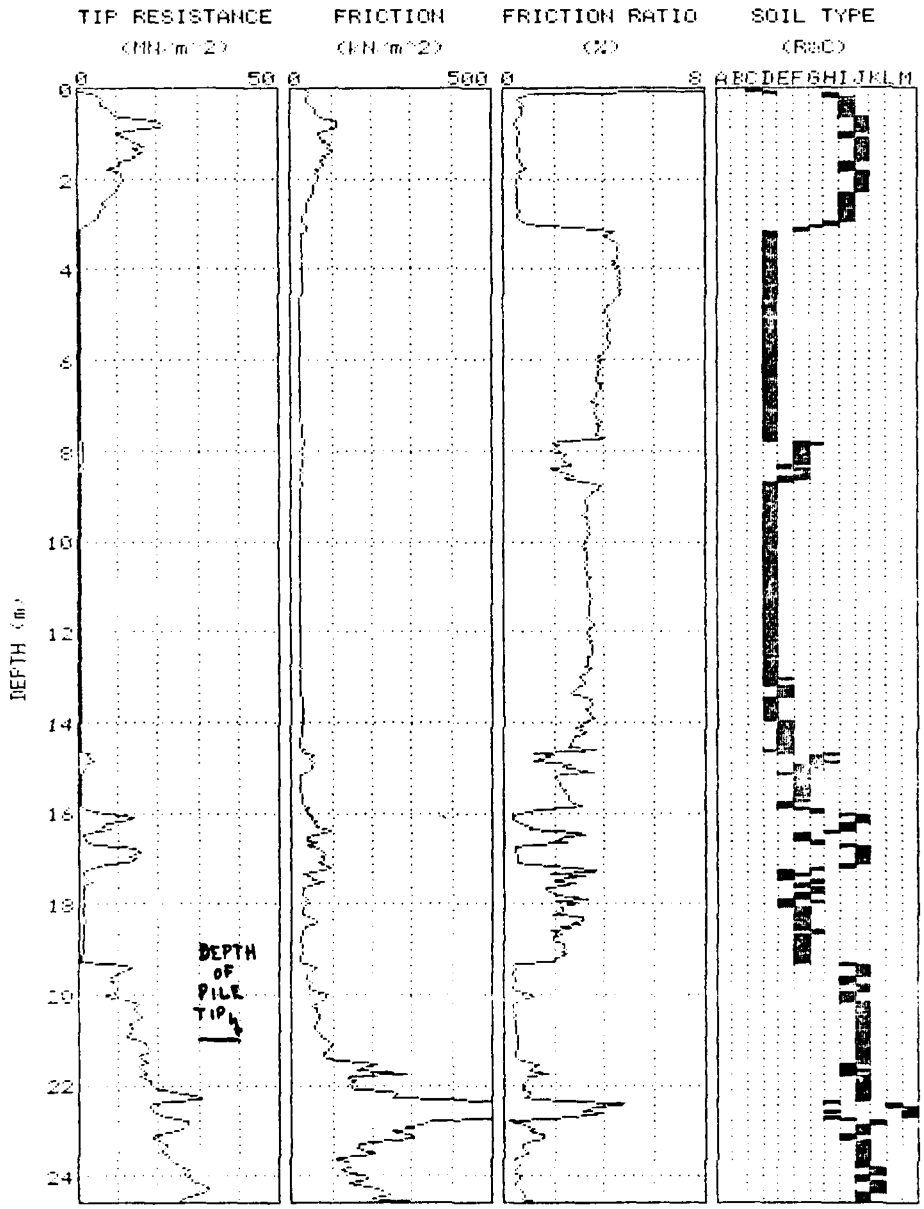


Figure 4-13. ECPT Sounding Data Summary, Choctawhatchee Bay Bridge FSB 26 (Site 21)

Since the only portion of the pile not surrounded by slurry was from the pile tip to 2.88 ft above the tip of the test pile, the actual design side friction contribution to the design pile capacity was considered negligible. The design end bearing, design pile capacity, and the ultimate pile capacity from the latter ECPT analysis was still destined to exceed the test pile's PLT ultimate pile capacity, because there was very little end bearing contribution above the pile tip in the PLT due to the slurry. Because of the presence of the slurry, only the soil region 2.88 ft above the tip influenced end bearing rather than 20 ft (8B) above the tip. After entering the pile geometry data into PLAID, the ultimate pile capacity for the identical pile used in the PLT was 818 tons. Therefore the ECPT predicted ultimate pile capacity did exceed the ultimate pile capacity determined by the PLT. In fact, the ECPT ultimate pile capacity was nearly double the PLT ultimate pile capacity. However, only the design end bearing and the ultimate capacity from only end bearing were to be compared with the PLT ultimate capacity because of the deep slurry. The ECPT predicted design end bearing was 208 tons. Multiplying the latter by the factor of safety of 3, the ECPT predicted ultimate pile capacity was 624 tons. The 624 tons was, as predicted, still greater than the PLT ultimate pile capacity; because the deep slurry caused a lack of end bearing from the pile area above the pile tip.

MCPT

The MCPT sounding, designated M021A and used for comparison with the PLT and ECPT, was located at station 183 + 00 and 30 ft left of the existing bridge centerline. The test was 29 ft from the PLT, 16.5 ft from the ECPT, and it was 78.1 ft deep. The test was conducted by FDOT on 18 March 85. The test was designated as sounding 2 in the contract plans and was shown in Figure 4-14. Only the Qc and FR values were shown in the sounding log. The soil profile represented by M021A was determined to be a five layer system. A cohesionless soil layer was identified from the ground surface to 47.3 ft below the ground surface, a cohesive layer from 47.3 to 55.8 ft, a thin cohesionless layer from 55.8 to 57.8 ft, another cohesive layer from 57.8 to 66.3 ft, and a cohesionless layer from 66.3 to 78.0 ft at the end of the sounding. The layer identification by the MCPT agreed with the ECPT except in the region between 9.1 and 47.3 ft below the ground surface. The latter region was identified as cohesionless by the MCPT vs. cohesive by the ECPT. The boring identified all the soil in the same region as a number of alternating layers of both soft clay and very loose sand. Therefore, each test's different identification of the soil region between 9.1

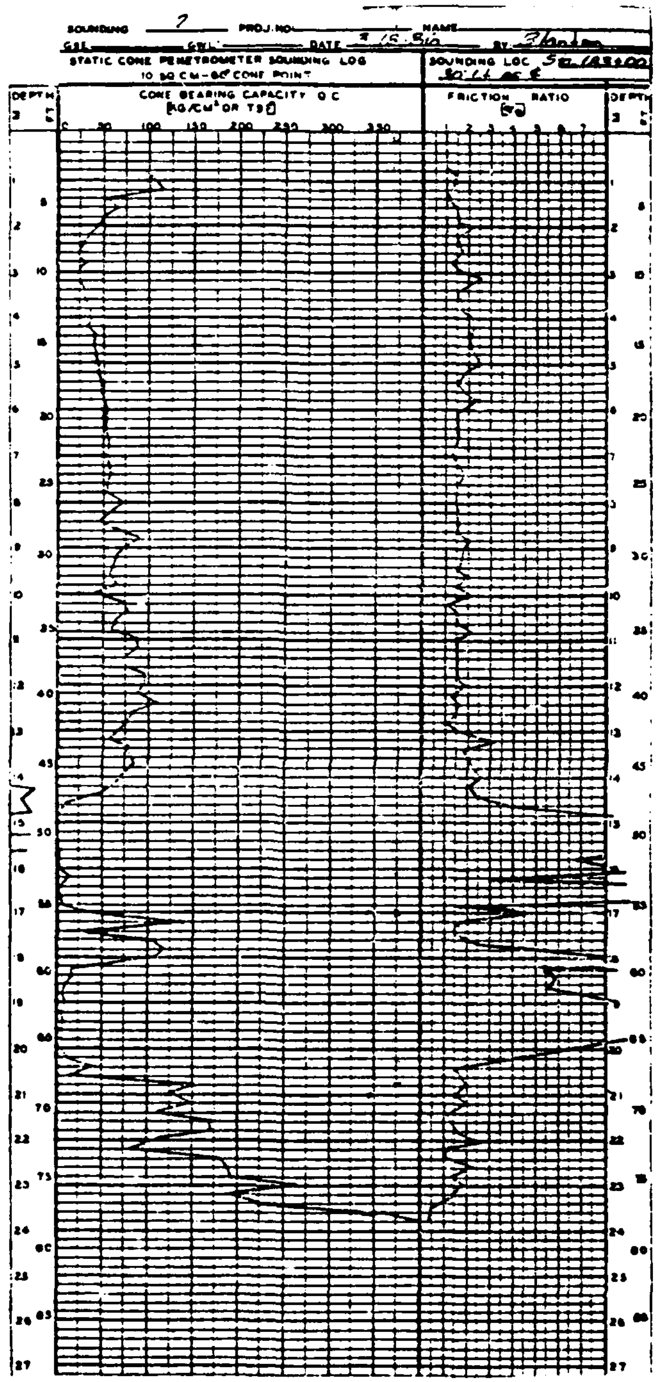


Figure 4-14. MCPT Sounding Data Summary, Choctawhatchee Bay Bridge FSB 26 (Site 21)

and 47.3 ft was understandable. Since the region was slurried, the identification of the layer was not necessary for pile capacity predictions but was of interest for comparing the ECPT and MCPT soil profiles.

The identical pile used in the PLT was used to find the comparative pile capacity using sounding M021A in the MCPTUFR program. In order to ensure the pile tip in the ECPT analysis was at the same elevation as in the PLT, the pile length below the ground surface for the ECPT analysis was 68.88 ft (PLT pile length below ground surface was 64.88 ft) since the sounding was performed with a GSE of +6 ft vs. the PLT's post-excavation GSE of +2 ft. The ultimate pile capacity predicted by the MCPT was 773 tons. However, ignoring the design side friction as done with the ECPT pile capacity prediction because of the deep slurry, the ultimate pile capacity predicted by the MCPT analysis was 561 tons which exceeded the PLT ultimate pile capacity by 80 tons and was 63 tons less than the ECPT prediction.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-5.

Table 4-5 - Choctawhatchee Bay Bridge FSB 26, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P021	481				
ECPT C021A	624	208	N/A	208	+ 30%
MCPT M021A	561	187	N/A	187	+ 17%

The ECPT and MCPT each overpredicted the ultimate pile capacity determined by the PLT by 30% and 17% respectively. Yet overpredictions were expected for both tests, because they both had the benefit of a full 20 ft (8B) length of soil above the pile tip contributing to the design end bearing and design and ultimate pile capacity. The test pile in the PLT had slurry all the way down to just 2.88 ft above the pile tip. Nevertheless, further comparison and analysis of the ECPT and MCPT were warranted to determine if there were any other reasons for the pile capacity overprediction compared to the PLT. Table 4-6 was created to conduct the comparison.

Only the fifth soil layer should have had bearing upon the pile capacity results useful for comparison to those obtained by the PLT. Due to computer program limitations in this analysis, however, the soil up to

48.88 ft which was 20 ft above the pile tip was included in the end bearing determination. In fact, there was essentially only slurry above the pile tip in the PLT. The 11% greater ultimate pile capacity predicted by the ECPT compared to the MCPT was understandable after examining the average Qc values for each soil layer shown in Table 4-6. In every soil layer except the very thin third layer, the ECPT average Qc was greater than the MCPT average Qc. Both tests' overprediction of the PLT pile capacity was probably due to cementation of the sand near the pile tip. The MCPT identified the soil near the tip as dense or cemented sand. As encountered at the other two Choctawhatchee Bay sites discussed previously, the ECPT classification of the fifth soil layer was only as a sand because the FR value was too low for the soil to have been cemented sand. Nevertheless, like the MCPT, the ECPT registered a very high average Qc for the fifth soil layer which may have led to the pile capacity overprediction compared to the PLT.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-12) with the ECPT predicted results (Figure 4-15). The PLT load-settlement plot was also shown in Figure 4-15 for easier comparison. The curves designated "ECPT TOTAL" and "ECPT SKIN" in Figure 4-15 should be ignored because of the excavation and 60 ft of slurrying performed prior to the PLT. The "ECPT TIP" load-setttlement curve in Figure 4-15 was

Table 4-6 - Detailed Comparison of MCPT and ECPT Soundings
Choctawhatchee Bay Bridge FSB 26

Test	MCPT	ECPT
Layer 1 Depth	0 to 47.3 ft	0 to 9.1 ft
Soil Class.	Sand	Sand
Avg Qc (tsf)	60	110
Avg FR (%)	1.76	0.94
Layer 2 Depth	47.3 to 55.8 ft	9.1 to 52.4 ft
Soil Class.	Organic Clays/ Mixed Soils	Silty Clay to Clay
Avg Qc (tsf)	6	9
Avg FR (%)	6.72	3.36
Layer 3 Depth	55.8 to 57.8 ft	52.4 to 57.2 ft
Soil Class.	Sand	Sand to Silty Sand
Avg Qc (tsf)	91	81
Avg FR (%)	1.67	1.42
Layer 4 Depth	57.8 to 66.3 ft	57.2 to 64.4 ft
Soil Class.	Very Stiff Clay	Sandy to Clayey Silt
Avg Qc (tsf)	25	31
Avg FR (%)	6.85	2.03
Layer 5 Depth	66.3 to 78.0 ft	64.4 to 80.6 ft
Soil Class.	Dense or Cemented Sand	Sand
Avg Qc (tsf)	173	204
Avg FR (%)	1.49	0.97

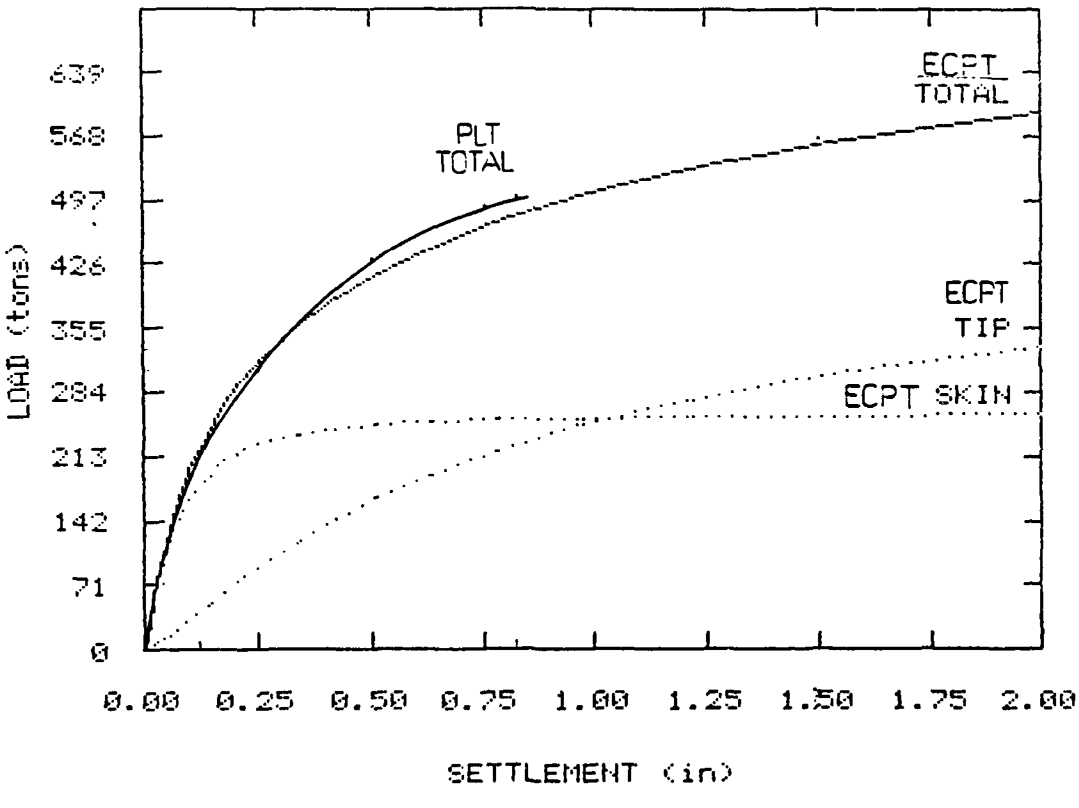


Figure 4-15. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Choctawhatchee Bay Bridge FSB 26 (Site 21)

comparable to the PLT curve since only end bearing was a factor in the pile load test due to the aforementioned slurry. As shown in Figure 4-15, the ECPT load-settlement prediction (the "ECPT TIP" curve) was conservative compared to the PLT results. The ECPT predicted a load of 337 tons with 2 inches of settlement, while the PLT reached 500 tons with only 0.8 inches of settlement before the pile was unloaded. The end bearing resistance appeared to have mobilized much faster with the PLT than predicted by the ECPT.

Port Orange - Bent 19

Load Test

Port Orange Bent 19 was designated as site 13 in Knox's 1989 PhD dissertation. The pile load test designated P013 was performed Jan 88 on pile 9 at station 226 + 01 and 44 ft right of centerline. The ground surface elevation was +4.20 ft, and the pile tip elevation was -26.68 ft. The water table was about 5 ft below the ground surface. The length of the pile below the ground surface was 30.88 ft. The total length of the 18 inch square, solid, prestressed concrete pile was 34.25 ft. The pile was jetted to the -2.5 ft elevation. A boring was performed at station 226 + 00 and 17.5 ft right of centerline. The boring was about 27 ft from the PLT. The boring log shown in Figure 4-16 identified the soil from the surface to the depth of 51.5 ft as sand from 0 to 10 ft and silty sand from 10 to 51.5 ft. The PLT load and deflection data were plotted in Figure 4-17. The ultimate pile capacity was 101.5 tons in the pile's first load cycle. The E value used for the ultimate pile capacity determination was back calculated from the plot in Figure 4-17 and was equal to 4,643,437 psi.

ECPT

The ECPT sounding used for comparison to the result of the PLT, designated C013A, was located at station 226 + 01 and 26 ft left of centerline. The sounding was 69.9

FORM 657-12
7-72

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD BORING LOG

PROJECT NO. 79180-3514 NAME "Bent 19" COUNTY Volusia DISTRICT 5
LOCATION 226+00 17.5 RT ₵ CONST. TOWNSHIP ____ RANGE ____ SECTION ____
ROAD NUMBER A 1A Dunlawton Ave SURFACE ELEVATION ____
EQUIPMENT TYPE CME 55 RIG NO. 7478 BORING NO. 4
DATE STARTED 3-2-88 COMPLETED 3-3-88 DRILLED BY Cherry
LOGGED BY Dawson BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, ____
WATER TABLE: 0 HR. 6.8 TIDAL 24 HRS. ____ HRS. ____ CASED, UNCASED, DRILLING MUD, ____

SAMPLE CONDITIONS: 1 DISTURBED, 2 GOOD, 3 LOST, 4 CORE SAMPLE
SAMPLE TYPES: A: AUGER; SB: SPLIT BARREL; S: SHELBY TUBE; RC: ROCK CORE ____ SIZE
TESTS: W.C.: WATER CONTENT (%); T: TORVANE (TSF); V: IN-SITU VANE TEST (TSF)

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES CON.	SAMPLES NO. TYPE	SAMPLES REC. (%)	TESTS	REMARKS
	0.0	5, 7, 8	Gray to pale brown sand, some shell to shelly, loose to compact & moist	2	1SB	70		
	5.0	4, 4, 3		2	2SB	70		
	10.0	1, 1, 1	10.0 Gray silty sand with trace of clay & some shell, loose & moist 12.0	2	3SB	50		
			Light gray to gray slightly silty sand, trace of shell to shelly & moist compact to very dense					
	15.0	7, 15, 23		2	4SB	50		
								← CASED 18.5
	20.0		12' to 51.5					

Figure 4-16. Boring Log Summary, Port Orange Bent 19 (Site 13)

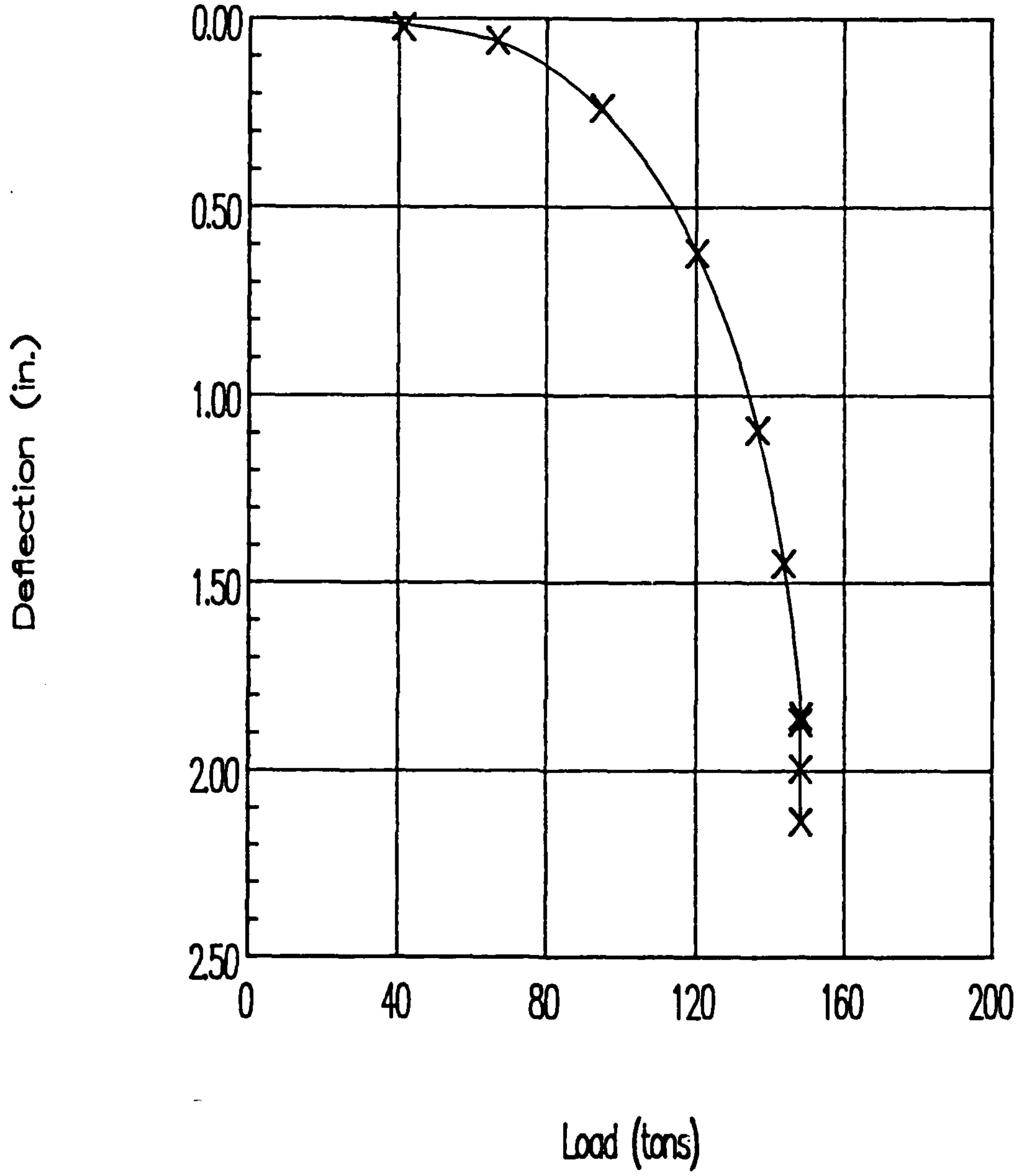


Figure 4-17. Pile Load Test, Load-Settlement Plot, Port Orange Bent 19 (Site 13)

ft from the PLT, and it was 85.3 ft deep. The sounding was performed 21 Oct 87 by FDOT, and the sounding log was compiled in depth increments of 25 cm. Baseline and inclination information were not available. The sounding log showed no negative Fs readings in the sounding. Spike editing was unnecessary to the depth required for pile analysis which was at 36.5 ft (3.75B below the pile tip). A summary of the soil data used for ECPT pile analysis is shown in Figure 4-18. All of the soil from the surface to a depth of 40.1 was classified as cohesionless. The ECPT and boring log compared favorably with one another in identifying the soil at the site. Figure 4-18 showed some comparatively finer grained soil from 10.2 ft to 13.5 ft which agreed with the boring log which identified the 10 to 12 ft region as silty sand with traces of clay and shell. The thin soil layer was also identified as silty sand to sandy silt by the ECPT sounding. Along with the rest of the soil in the sounding to at least a depth of 40.1 ft, the thin soil layer was cohesionless soil. After entering the PLT pile geometry data into PLAID, the ultimate pile capacity from ECPT analysis was 415.5 tons. The reason for the ECPT pile capacity overprediction when compared to the PLT results may have been due to the sand being so very dense and compact as noted in the boring log. The sand may well have been cemented.

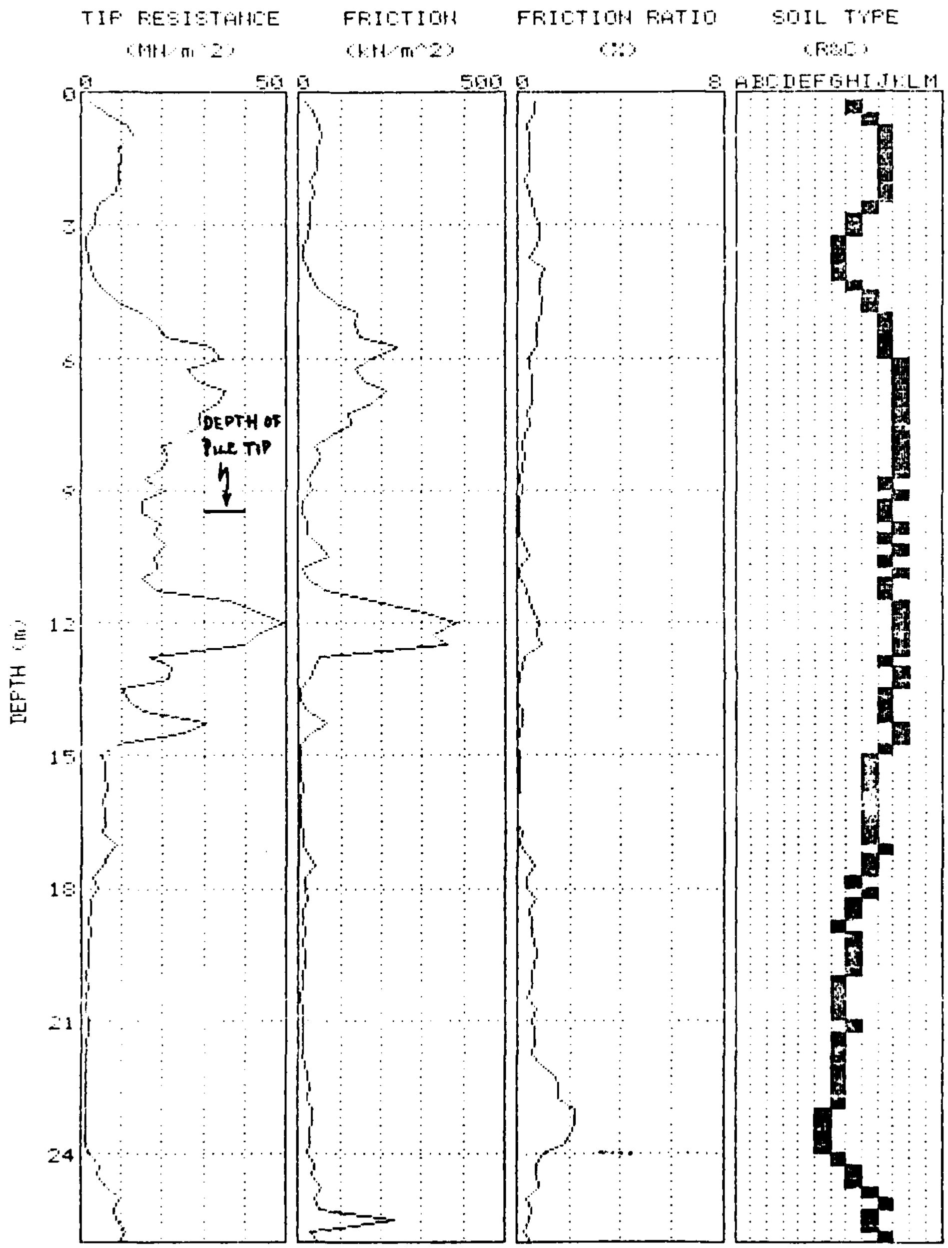


Figure 4-18. ECPT Sounding Data Summary, Port Orange Bent 19 (Site 13)

MCPT

The MCPT sounding, designated M013A and used for comparison with the PLT and ECPT at this site, was located at station 225 + 41 and 18 ft right of center-line. The MCPT sounding was 65.4 ft from the PLT, 74.4 ft from the ECPT, and it was 44.6 ft deep. The FDOT performed the sounding on 5 Aug 85, and it was identified as sounding 7 in their contract plans. Only Qc and FR values were plotted for the sounding as shown in Figure 4-19.

Initial analysis of the sounding alone suggested a three layer soil system between the ground surface and a depth of 38.7 ft with a cohesionless layer from 0 to 9.2 ft, a cohesive layer from 9.2 to 14.4 ft, and a cohesionless layer from 14.4 to 38.7 ft. The boring log disagreed with the latter assessment as it identified all of the soil from the surface to beyond 38.7 ft as cohesionless soil. Table 4-7 showed the analysis of the MCPT sounding to a depth of 38.7 ft for both the one and three layer soil systems.

There was very little difference in the pile capacity results using either the three or single layer soil system. The ultimate pile capacity determined with the three layer soil system was 336 tons vs. 332 tons for the single layer soil system. The important information garnered from the three layer soil system analysis was the fact the majority of soil in the pile profile,

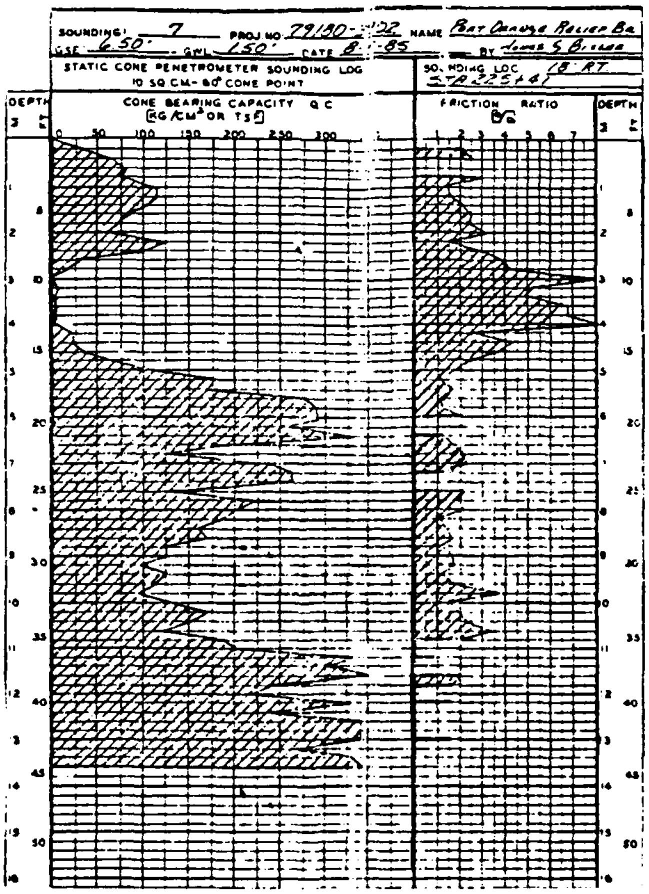


Figure 4-19. MCPT Sounding Data Summary, Port Orange Bent 19 (Site 13)

Table 4-7 - Comparison of MCPT Sounding M013A Single and Three Layer Soil Systems at Port Orange Bent 19

	Sounding M013A Three Layer	Sounding M013A Single Layer
Layer 1 Depth	0 to 9.2 ft	0 to 38.7 ft
Soil Class.	Clayey Sands and Silts	Silt-Sand Mixture
Avg Qc (tsf)	80	139
Avg FR (%)	2.26	2.57
Layer 2 Depth	9.2 to 14.4 ft	
Soil Class.	Medium to Stiff Clay	
Avg Qc (tsf)	9	
Avg FR (%)	5.62	
Layer 3 Depth	14.4 to 38.7 ft	
Soil Class.	Dense or Cemented Sand	
Avg Qc (tsf)	190	
Avg FR (%)	1.76	

located in the third soil layer, was classified as a dense or cemented sand due to the very high average Qc values between 14.4 and 38.7 ft. Cemented sand in the latter soil region may well have been responsible for the very large overprediction of pile capacity using the MCPT when compared to the ultimate pile capacity determined by the PLT.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-8 for comparison.

Table 4-8 - Port Orange Bent 19, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P013	101.5				
ECPT C013A	415.5	151.5	39	112.5	+309%
MCPT M013A	332	120	28	92	+227%

Both the ECPT and MCPT overpredicted the ultimate pile capacity determined by the PLT by a large percentage, 309% and 227% respectively. The dense and possibly cemented sand below the depth of 14 ft, described as compact to very dense in the boring log, may have been the cause for the overpredictions. A closer look at the ECPT and MCPT results was necessary to determine if any other factors may have caused the overpredictions.

Comparison of the two cone penetration tests revealed they were actually fairly comparable although the ECPT produced higher pile capacity results than the MCPT. The design end bearing in the ECPT constituted 74% of the ECPT total design pile capacity (151.5 tons), while the MCPT design end bearing was 77% of the total design pile capacity (120 tons) determined by the MCPT. Therefore, the design side frictions for the ECPT and MCPT also constituted nearly equal percentages of the design pile capacity, 26% and 23% respectively. The ECPT total design pile capacity was 26% higher than that found with the MCPT. The reason for the high pile capacity determinations with both penetration tests compared to the PLT can be seen in Table 4-9 as both tests had very high average Qc values.

Table 4-9 - Detailed Comparison of Port Orange Bent 19 MCPT and ECPT Soundings as Single Layer Soil Systems

Test	MCPT	ECPT
Layer Depth	0 to 38.7 ft	0 to 40.1 ft
Soil Class.	Silt-Sand Mixture	Sand
Avg Qc (tsf)	139	126
Avg FR (%)	2.57	0.55

Since the MCPT had the higher average Qc value and seemingly more friction resistance with the much higher average FR value, the pile capacities determined from the

MCPT would have been expected to be higher than those determined fom the ECPT. Since the latter was not the case as shown in Table 4-8, the 18.9 to 36.9 ft layer of soil was analyzed for both tests as shown in Table 4-10 to try and shed more light on their capacity determinations. Within the 18.9 to 36.9 ft depth region was the critical area for end bearing determination which was from 8B above the pile tip at 18.9 ft below the ground surface to 3.75B below the pile tip at 36.5 ft below the ground surface (tip located 30.88 ft below ground surface).

Table 4-10 - Detailed Comparison of Port Orange Bent 19 MCPT and ECPT Soundings in Critical Depth Region for End Bearing

Test	MCPT	ECPT
Crit. Depth Range	18.9 to 36.9 ft	18.9 to 36.9 ft
Soil Class.	Dense or Cemented Sand	Gravelly Sand to Sand
Avg Qc (tsf)	189	240
Avg FR (%)	1.76	0.31

Comparison of the average Qc values in Table 4-10 plainly showed the 27% higher average Qc for the ECPT in the critical depth area for end bearing determination. The ECPT design end bearing was actually the maximum allowable for the 18 inch square pile analyzed. The MCPT had higher Qc values than the ECPT from the ground surface to 18.9 ft which resulted in the misleading higher average Qc value in Table 4-9. The 0 to 18.9 ft depth range had no influence on end bearing determination. So the higher end bearing and pile capacity determination by the ECPT shown in Table 4-8 was reasonable. As seen with other FR comparisons in this report, the average FR for the ECPT was far lower than the MCPT average FR. But friction resistance contributions to the design and ultimate pile capacity were far less than end bearing, so further analysis was not performed. It was apparent that both the ECPT and MCPT overpredicted pile capacity compared to the PLT because both cone penetration tests represented the soil as having a far greater end bearing capability than it actually possessed. Cementation suggested by the boring and cone penetration data was most likely the cause for overprediction of pile capacities by the ECPT and MCPT.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-17) with the ECPT predicted results (Figure 4-20) using the PLAID program. The PLT load-settlement plot was shown in Figure 4-20 for easier comparison. Comparison of the ECPT vs. the PLT load-settlement plot plainly showed the unconservative nature of the ECPT load-settlement prediction for this particular site. The ECPT plot of the tip and skin load-settlement curves alone were each more comparable with the total load-settlement plot from the PLT.

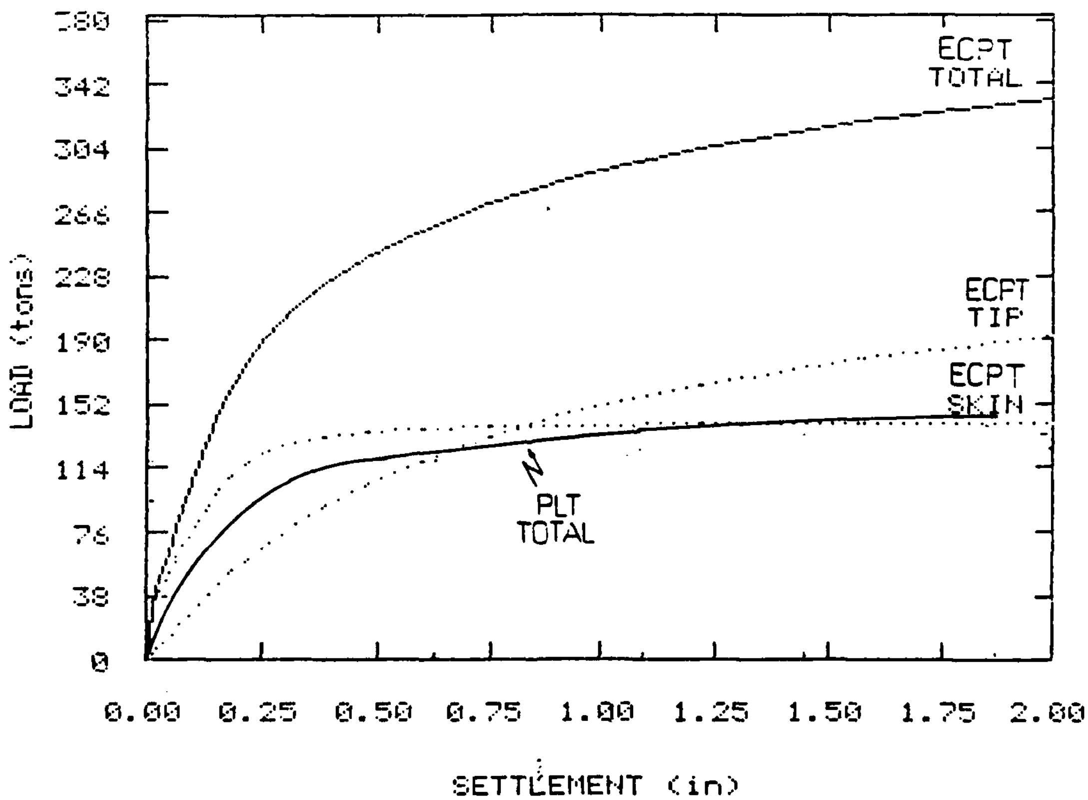


Figure 4-20. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Port Orange Bent 19 (Site 13)

Port Orange - Bent 2

Load Test

Port Orange Bent 2 was designated as site 14 in Knox's 1989 PhD dissertation. The pile load test designated P014 was performed Jan 88 on pile 6 located at station 221 + 25 and 11 ft right of centerline. The ground surface elevation was +6.4 ft, and the pile tip elevation was +23.61 ft. The water table was about 5 ft below the ground surface. The length of the test pile below the ground surface was 30.01 ft. The total length of the 18 inch square, solid, prestressed concrete pile was 32.78 ft. The pile was jetted to the -2.5 ft elevation. A boring was performed at station 221 + 90 and 20 ft left of centerline which was 72 ft from the PLT. The boring log shown in Figure 4-21 identified the soil from 0 to 16 ft below the ground surface as shelly sand and from 16 to 48 ft below the surface as compact to dense sand. The PLT load and deflection data were plotted in Figure 4-22. The ultimate pile capacity was 139 tons. The E used for the ultimate pile capacity determination was back calculated from the plot in Figure 4-22 and was equal to 3,434,387 psi.

ECPT

The ECPT sounding used for comparison with P014, designated C014A, was located at station 221 + 53 and 27 ft left of centerline. The sounding was 47 ft from the

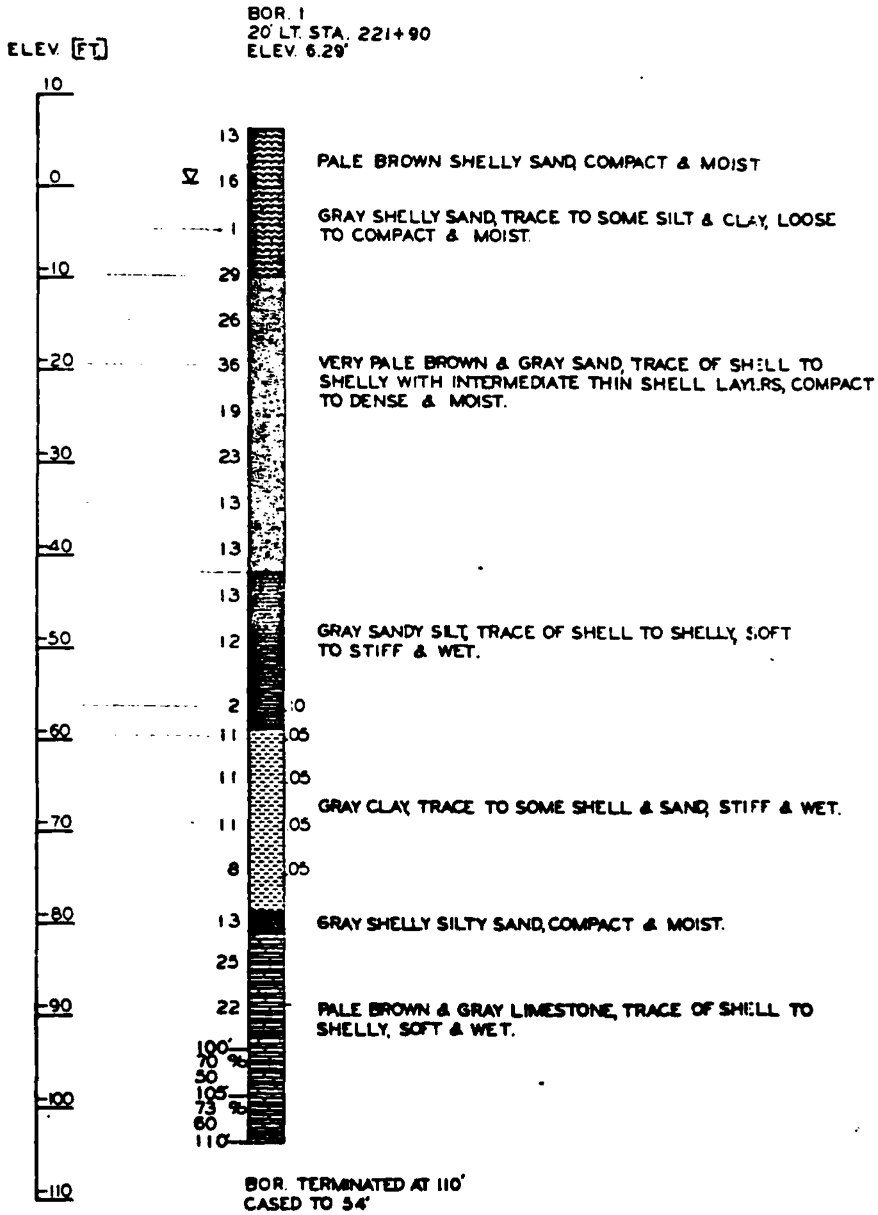


Figure 4-21. Boring Log Summary, Port Orange Bent 2 (Site 14)

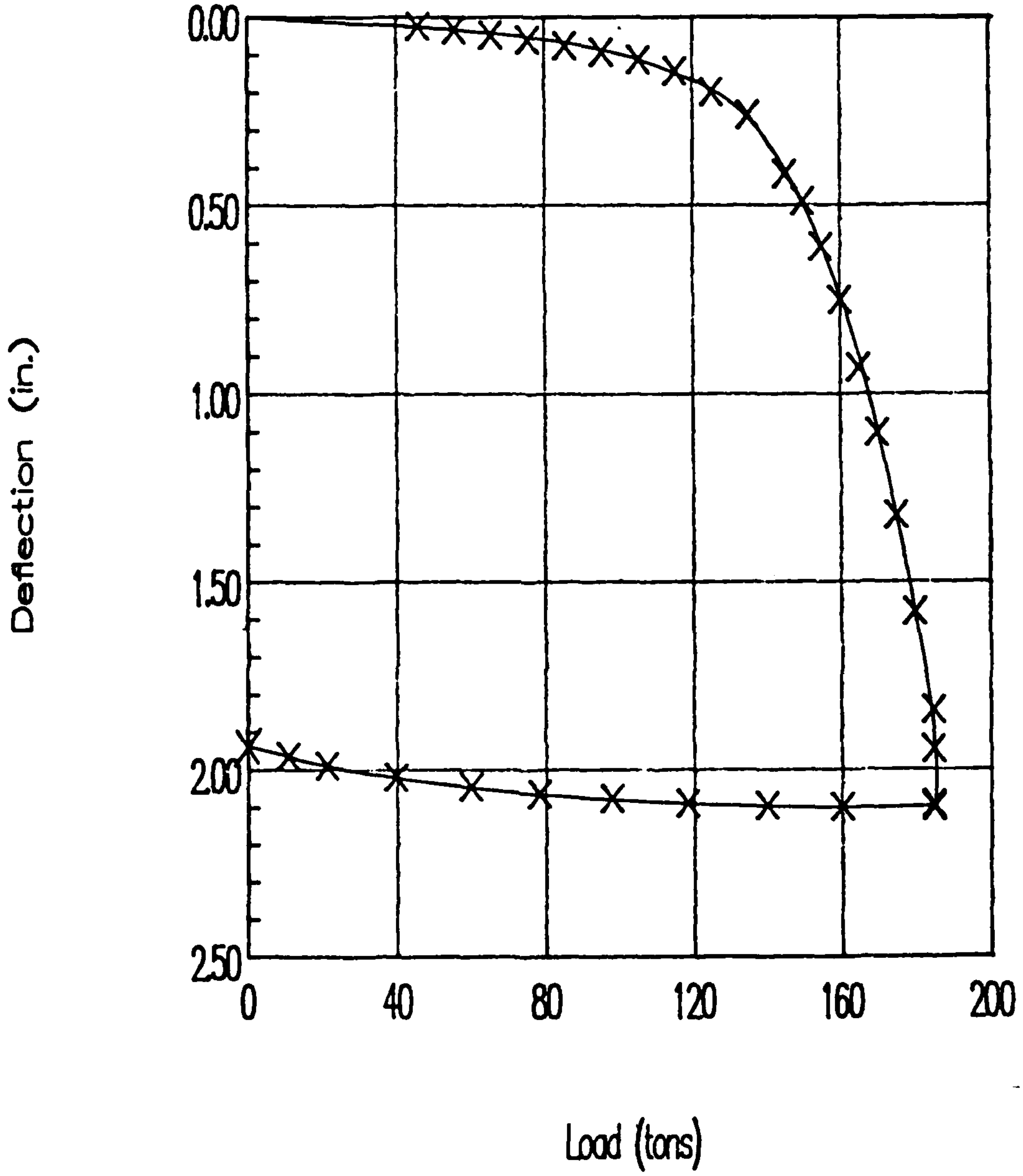


Figure 4-22. Pile Load Test, Load-Settlement Plot, Port Orange Bent 2 (Site 14)

PLT, and it was 86.9 ft deep. The FDOT performed the sounding on 22 Oct 87, and their sounding log was compiled in depth increments of 25 cm. Baseline and inclination information were not available. The sounding log showed no negative Fs readings in the sounding. Spike editing was unnecessary to the depth required for pile analysis which was at 35.6 ft (3.75B below the pile tip). A summary of the ECPT data used for pile analysis was shown in Figure 4-23. All of the soil from the surface to the 40.9 ft depth was classified as cohesionless. The ECPT and boring log compared favorably with one another in identifying the soil at the site. The soil was classified as sand from the ground surface to at least the depth required for pile analysis. After entering the PLT pile geometry data into PLAID, the ultimate pile capacity from ECPT analysis was 397.5 tons. The reason for the ECPT pile capacity overprediction when compared to the PLT results may have been due to the sand being so compact to dense as noted in the boring log. The sand may well have been cemented.

MCPT

The MCPT sounding, designated M014B and used for comparison with the PLT and ECPT at this site, was located at station 222 + 00 and 18 ft right of centerline. The MCPT sounding was 75.0 ft from the PLT, 65.0 ft from the ECPT, and it was 88.59 ft deep. The FDOT

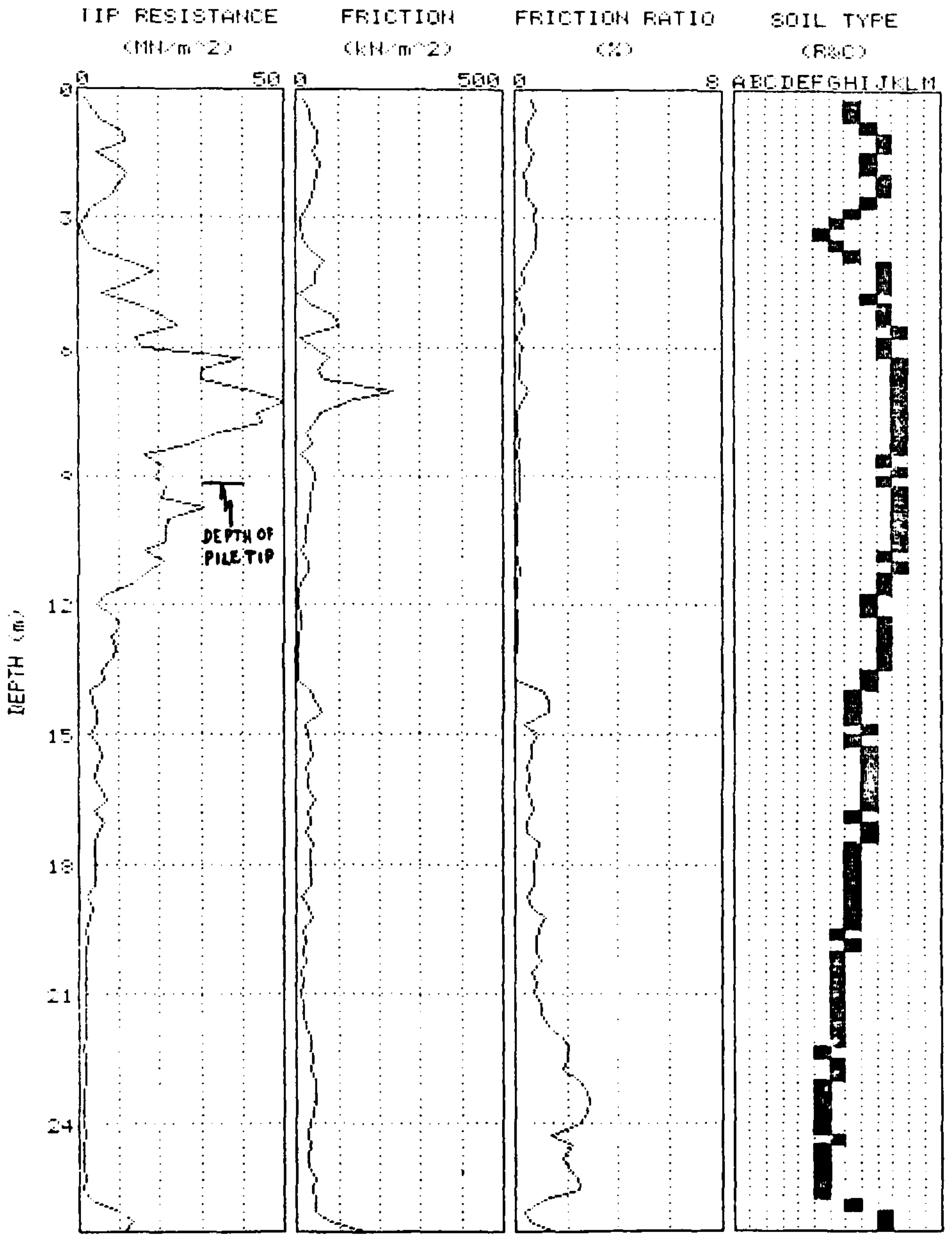


Figure 4-23. ECPT Sounding Data Summary, Port Orange Bent 2 (Site 14)

performed the sounding on 9 Jul 85, and it was identified as sounding 2 in their contract plans. Only Qc and FR values were plotted for the sounding as shown in Figure 4-24.

Initial analysis of M014B without the benefit of the boring log suggested there was some cohesive material between 5.3 and 13.1 ft as the average Qc in the latter depth range was comparatively low at 68 tsf along with a high average FR of 4.33%. But after consulting the boring log and recalling the similar situation at Bent 19 (site 13), it was determined there was a negligible amount of cohesive material present. Consequently, all soil in the profile was labeled cohesionless. The ultimate pile capacity determined using MCPTUFR was 386.5 tons. The very high Qc values encountered in much of the soil profile may have been an indication of cemented sand which would have explained the large overprediction of pile capacity by the MCPT when compared to the PLT. Further analysis and comparisons helped determine the possible reasons for the large overprediction.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-11 for comparison.

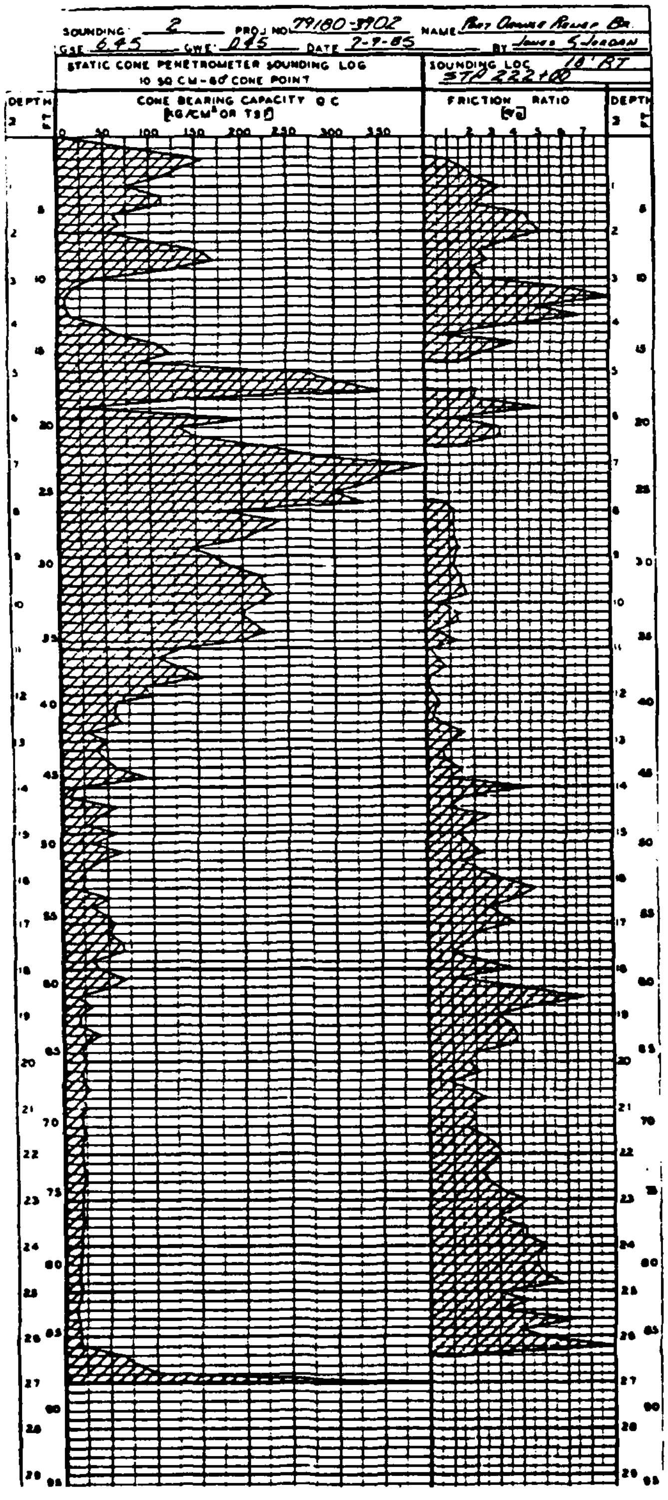


Figure 4-24. MCPT Sounding Data Summary, Port Orange Bent 2 (Site 14)

Table 4-11 - Port Orange Bent 2, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P014	139				
ECPT C014A	397.5	142.5	30	112.5	+186%
MCPT M014B	386.5	137	24.5	112.5	+178%

Both the ECPT and MCPT overpredicted the ultimate pile capacity determined by the PLT by a large percentage, 186% and 178% respectively. The compact to dense and possibly cemented sand below 16 ft beneath the ground surface was the most likely cause for the overpredictions by both cone penetration tests. A closer look at the ECPT and MCPT results was necessary to determine if any other factors may have caused the overpredictions.

Comparison of the ECPT and MCPT revealed they were very comparable although the ECPT produced slightly higher pile capacity results than the MCPT. The design end bearing for both tests was the maximum allowable for the 18 inch square pile (112.5 tons). The design end bearing in the ECPT constituted 79% of the ECPT total design pile capacity (141.5 tons), while the MCPT design end bearing was 82% of the total design pile capacity (137 tons) determined by the MCPT. Therefore, the design side frictions for the ECPT and MCPT also constituted nearly equal percentages of the design pile capacity, 21% and 18% respectively. The ECPT total design pile capacity was only 4% higher than that found with the MCPT. The ECPT ultimate design capacity was only 3% higher than the MCPT. The reason for the high pile capacity determinations with both tests was apparent in Table 4-12 as both tests had very high average Qc values.

Table 4-12 - Detailed Comparison of Port Orange Bent 2 MCPT and ECPT Soundings

Test	MCPT	ECPT
Layer Depth	0 to 40.0 ft	0 to 40.9 ft
Soil Class.	Dense or Cemented Sand	Sand
Avg Qc (tsf)	151	107
Avg FR (%)	1.88	0.74

Since the MCPT had the higher average Qc and seemingly more friction resistance with the much higher average FR, the pile capacities determined from the MCPT would have been expected to be higher than those determined fom the ECPT. Since the latter was not the case as shown in Table 4-11, the 18.0 to 36.0 ft layer of soil was analyzed for both tests as shown in Table 4-13 to try and shed more light on their capacity determinations. Within the 18.0 to 36.0 ft depth lay the critical area for end bearing determination which was from 8B above the pile tip at 18.0 ft below the ground surface to 3.75B below the pile tip at 35.6 ft below the ground surface (tip located 30.01 ft below ground surface).

Table 4-13 - Detailed Comparison of Port Orange Bent 2 MCPT and ECPT Soundings in Critical Depth Region for End Bearing

Test	MCPT	ECPT
Crit. Depth Range	18.0 to 36.0 ft	18.0 to 36.0 ft
Soil Class.	Dense or Cemented Sand	Gravelly Sand to Sand
Avg Qc (tsf)	220	284
Avg FR (%)	1.33	0.18

Comparison of the average Qc values in Table 4-13 plainly showed the 29% higher average Qc for the ECPT in the critical depth area for end bearing determination. As mentioned previously, both the ECPT and MCPT design end bearings were the maximum allowable for the 18 inch square pile analyzed. The MCPT had higher Qc values than the ECPT from the ground surface to 18.0 ft which resulted in the misleading higher average Qc value in Table 4-12. The 0 to 18.0 ft depth range had no influence on end bearing determination. So the higher end bearing and pile capacity determination by the ECPT shown in Table 4-11 was reasonable. As seen with other FR comparisons in this report, the average FR for the ECPT was far lower than the MCPT average FR. But friction resistance contributions to the design and ultimate pile capacity were far less than end bearing, so friction resistance predictions were less critical. It was apparent both the ECPT and MCPT overpredicted pile capacity compared to the PLT because both cone penetration tests represented the soil as having a far greater end bearing capability than it actually possessed. The very compact and possibly cemented nature of the subsurface sands in the vicinity of the pile tip as evidenced by the boring and cone penetration data was the most likely cause for overprediction of pile capacities by the ECPT and MCPT.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-22) with the ECPT predicted results (Figure 4-25) using the PLAID program. The PLT load-settlement plot was also shown in Figure 4-25 for easier comparison. Comparison of the ECPT vs. the PLT load-settlement plot plainly showed the unconservative nature of the ECPT load-settlement prediction for this particular site. The ECPT plot of the tip load-settlement curve alone was more comparable with the total load-settlement plot from the PLT.

Apalachicola River Bridge - Pier 3

Load Test

Apalachicola River Bridge Pier 3 was designated as site 1 in Knox's 1989 PhD dissertation. The pile load test designated P001 was performed 4 Sept 86 on pile 7 at station 1095 + 58.75 and on the centerline of the Market

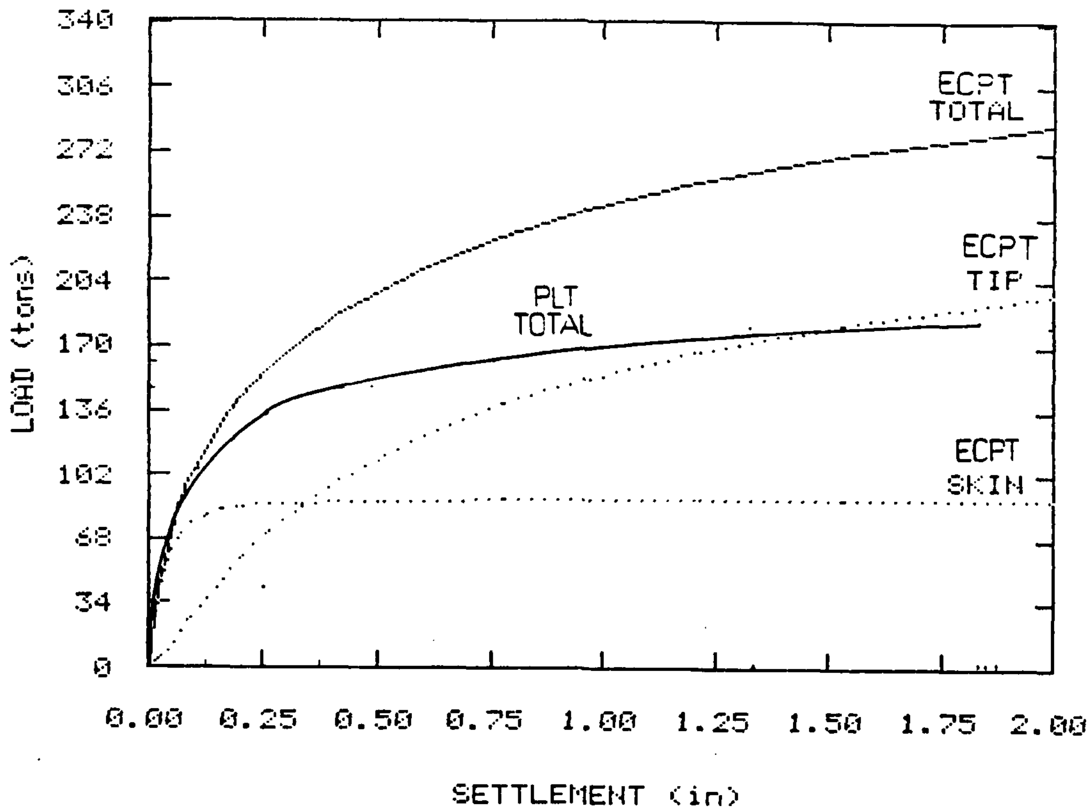


Figure 4-25. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Port Orange Bent 2 (Site 14)

Street bridge. The ground surface elevation was +5.20 ft, and the pile tip elevation was -85.4 ft. The water table was near the ground surface. The length of the pile below the ground surface was 90.60 ft. The total length of the 24 inch square, prestressed concrete pile with a 12 inch diameter void was 98 ft. A boring was performed at station 1095 + 00 and 27 ft left of centerline. The boring was about 65 ft from the PLT. The boring log shown in Figure 4-26 identified the soil as sand from the surface to 16.5 ft below the ground surface, soft clay from 16.5 to 41 ft, loose sand from 41 to 50 ft, stiff clay from 50 to 60 ft, and compact sand in the remaining soil profile to a depth of 100 ft. The PLT load and deflection data were plotted in Figure 4-27. The ultimate pile capacity was 479 tons. The E used for the ultimate pile capacity determination was back calculated from the plot in Figure 4-27 and was equal to 3,973,968 psi.

ECPT

The ECPT sounding, designated C001A and used for comparison to the result of the PLT, was located at station 1095 + 75 on the bridge centerline. The sounding was 16 ft from the PLT, and it was 99.1 ft deep. The sounding was performed 20 June 88 with the UF Geotechnical Engineering Department's ECPT truck using the ten ton tip. The difference between the Qc base line

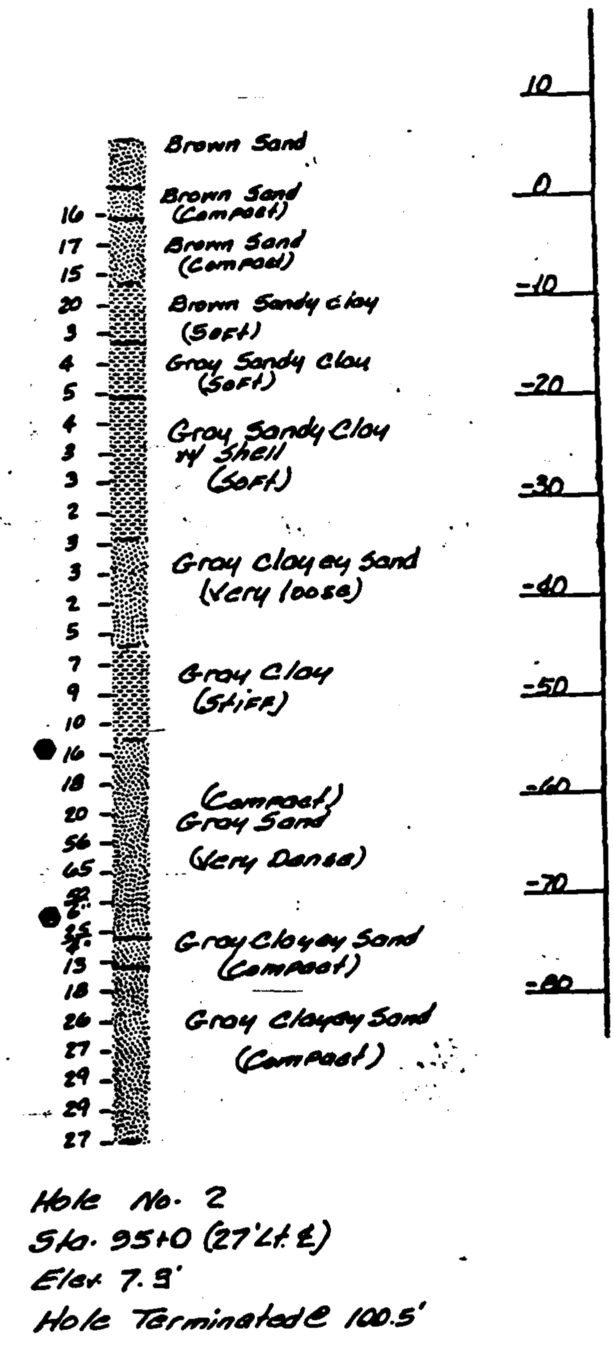


Figure 4-26. Boring Log Summary, Apalachicola River Bridge Pier 3 (Site 1)

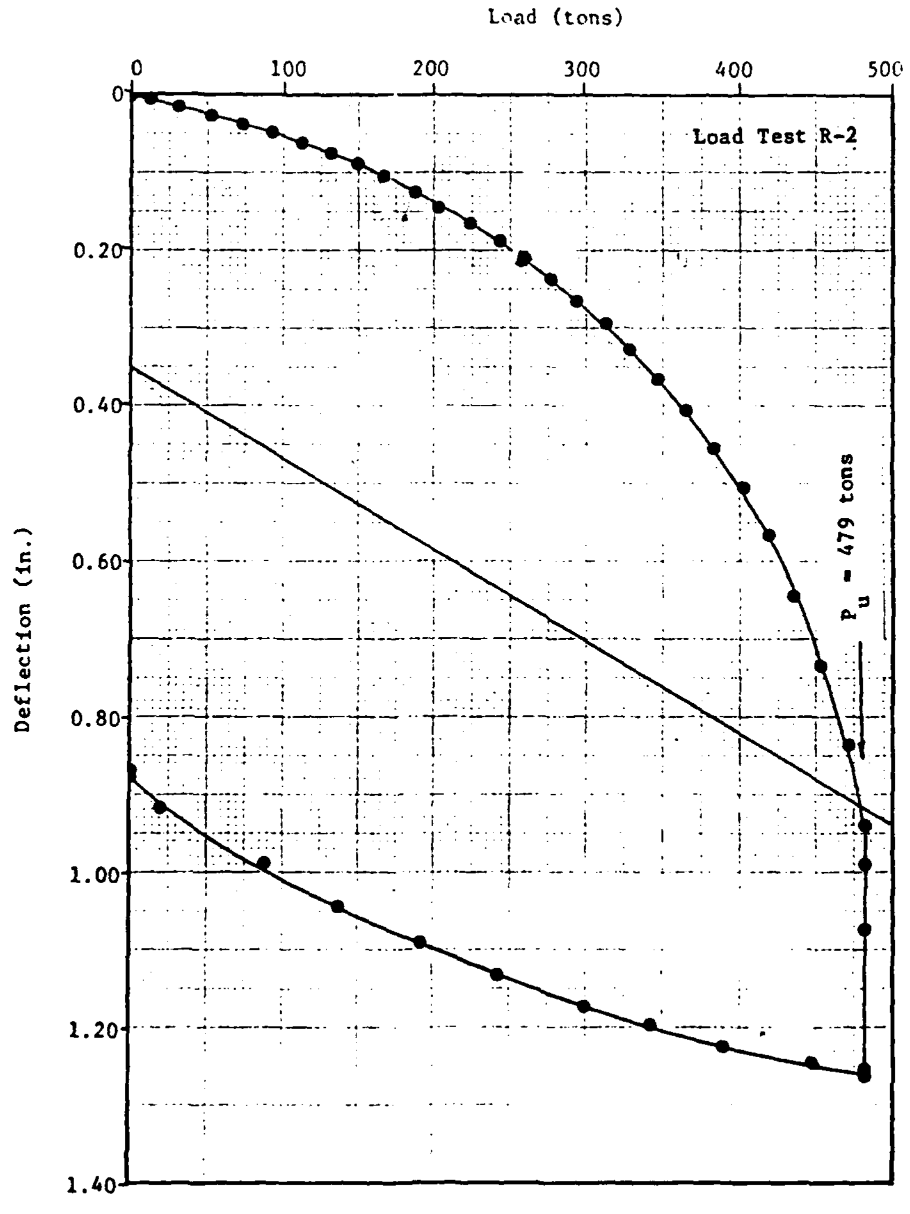


Figure 4-27. Pile Load Test, Load-Settlement Plot, Apalachicola River Bridge Pier 3 (Site 1)

readings before and after the test was an acceptable -.53 MPa, but the Fs base line difference of 32 kPa was a little higher than desired. Rod inclination to the depth necessary for pile analysis was only 1.0 degree. There were no negative Fs readings in the sounding. A spike for the Qc, Fs, and FR values at 11.5 ft below the ground surface was edited with the PLAID program. A summary of the soil data used for ECPT pile analysis was shown in Figure 4-28. Identification of the cohesive and cohesionless soil layers resulted in a three layer soil profile. The boring, located 80 ft away from the sounding, identified a similar profile as the ECPT. However, the cohesive layer in the middle of the profile identified by the ECPT was identified by the boring as two cohesive layers separated by a layer of loose sand. The ECPT also classified the middle soil layer as sandy to clayey silt, which was like a blend of the boring's identification of the same soil region as two clay layers with a layer of loose sand between them. The top and bottom layers of soil were sands, and they were at similar depth ranges in both the boring and the ECPT. The differences between the boring and ECPT soil classifications were not surprising, since there were large layer thickness differences for the middle soil layers in adjacent borings as well. After entering the PLT pile geometry data into PLAID, the ultimate pile capacity from ECPT analysis was 514 tons. The reason for

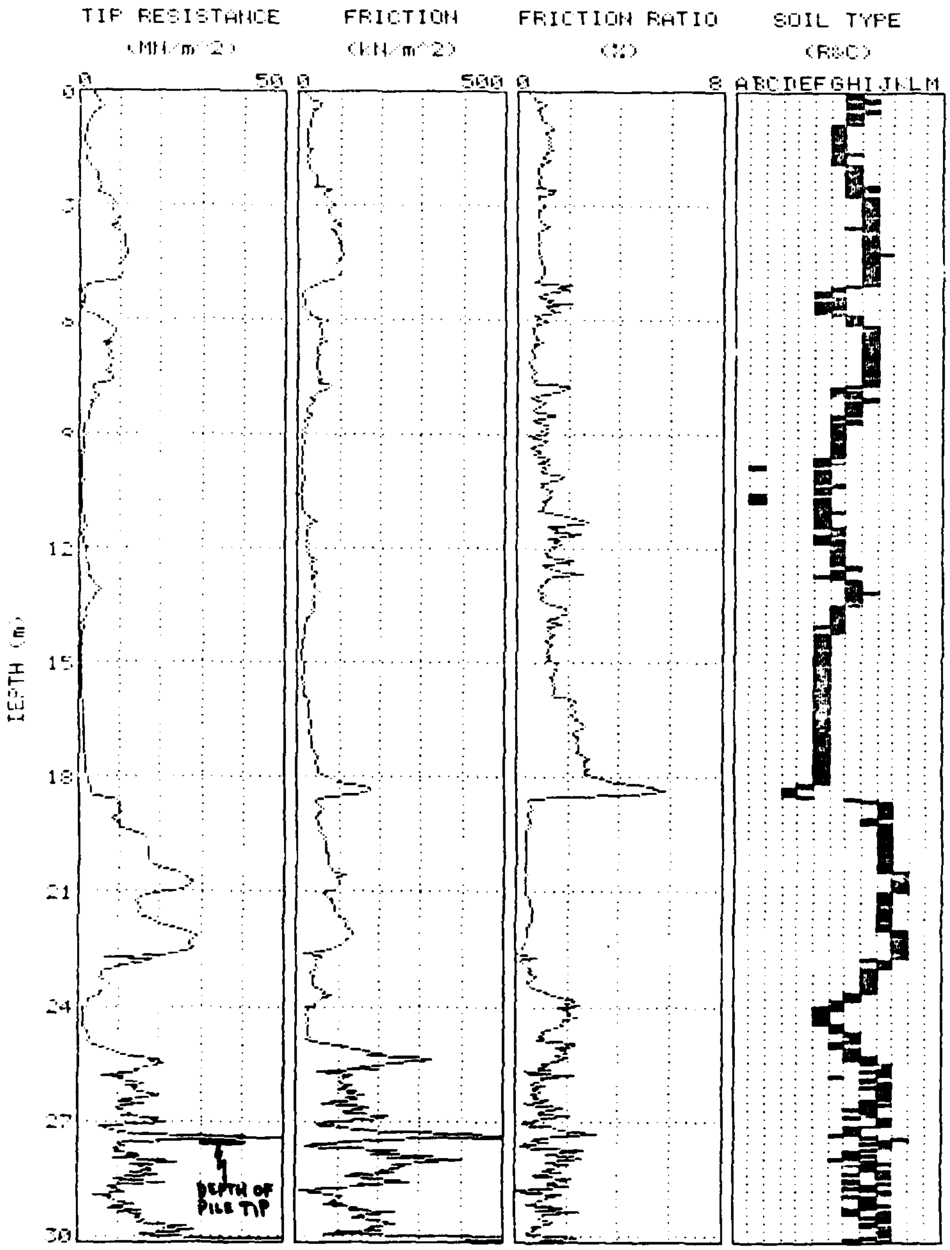


Figure 4-28. ECPT Sounding Data Summary, Apalachicola River Bridge Pier 3 (Site 1)

the ECPT pile capacity overprediction when compared to the PLT results may well have been due to the sand being very dense and compact as noted in the boring log. The sand may well have been cemented.

MCPT

The MCPT soundings close to the PLT that could have been used for comparative pile capacity analysis were not deep enough to allow proper analysis. MCPT sounding M001B was only 14 ft from the PLT, but it was only 83.3 ft deep. The depth required for pile capacity analysis was 98.1 ft (pile length below ground surface plus 3.75B). Sounding M001A was 94 ft from the PLT, but it terminated at even a shallower depth than M001B.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT are shown in Table 4-14 for comparison.

Table 4-14 - Apalachicola River Bridge Pier 3, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P001	479				
ECPT C001A	542	230	148	82	+13%

The ECPT overpredicted the ultimate pile capacity determined by the PLT by 13%. The dense and possibly cemented sand below the depth of 14 ft, described as compact to very dense in the boring log, may have been the cause for the overprediction. A closer look at the ECPT pile capacity analysis results in Table 4-15 was necessary to determine if any other factors may have caused the ECPT overprediction compared to the PLT.

Table 4-15 - ECPT Sounding C001A Soil Layer Characteristics from ECPT Pile Capacity Analysis

	Sounding C001A
Layer 1 Depth	0 to 31.8 ft
Soil Class.	Sand to Silty Sand
Avg Qc (tsf)	55
Avg FR (%)	0.97
Layer 2 Depth	31.8 to 61.3 ft
Soil Class.	Sandy to Clayey Silt
Avg Qc (tsf)	18
Avg FR (%)	1.80
Layer 3 Depth	61.3 to 99.0 ft
Soil Class.	Sand
Avg Qc (tsf)	151
Avg FR (%)	0.99

The ultimate pile capacity overprediction by the ECPT in comparison with the PLT was only 13%. Nevertheless, it was important to examine possible causes of the overprediction. The high average Qc in the third soil layer where the tip of the pile was located may have been partially responsible for the ECPT analysis overprediction; however, unlike the sites studied previously in this chapter, design side friction was the primary contributor to the pile capacity as shown in Table 4-14. It was possible the middle layer of soil identified by the ECPT as cohesive (albeit on the borderline between cohesive and cohesionless soil) may have contained the loose sand layer identified in the boring. In the latter case, there would have been less design side friction than predicted by the ECPT. The failure of the ECPT to identify loose sand in the midst of clay soil was suggested in analysis of the Choctawhatchee Bay soundings.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-27) with the ECPT predicted results (Figure 4-29) using the PLAID program. The PLT load-settlement plot was also shown in Figure 4-29 for easier comparison. Comparison of the ECPT vs. the PLT load-settlement plot plainly showed the two plots were nearly identical up to 1 inch of settlement at which point the PLT plot levelled off while the ECPT plot continued to gradually increase up to 578 tons at 2 inches of settlement. The PLT was terminated to begin redrive after about 1.25 inches of settlement with a load of 479 tons.

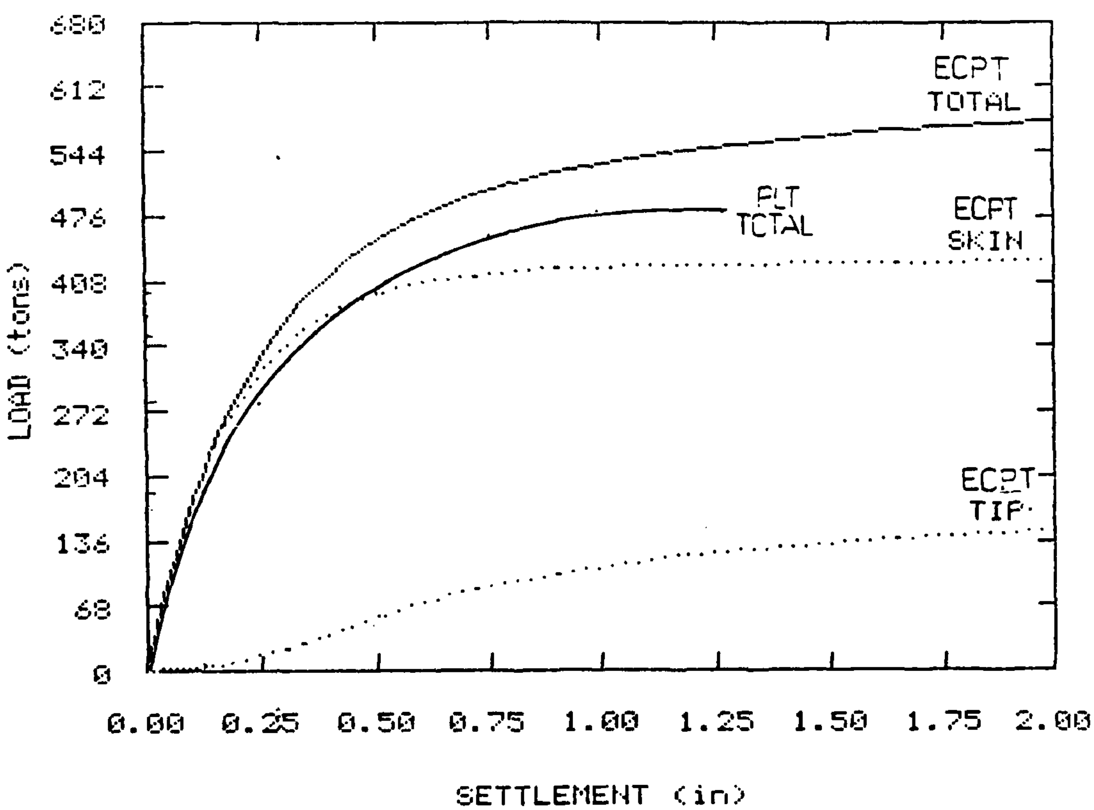


Figure 4-29. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Apalachicola River Bridge Pier 3 (Site 1)

Apalachicola River Bridge - FSB 16

Load Test

Apalachicola River Bridge FSB 16 was designated as site 2 in Knox's 1989 PhD dissertation. The pile load test designated P002 was conducted 13 Oct 86 on pile 3 at station 132 + 09 on the centerline of the Market Street bridge at the east end of the bridge. The ground surface elevation was +6.70 ft, and the pile tip elevation was -54.3 ft. The water table was near the ground surface. The length of the pile below the ground surface was 61.00 ft. The total length of the 18 inch square, solid, prestressed concrete pile was 68 ft. No borings were located close to the PLT or the cone penetration tests. The PLT load and deflection data were plotted in Figure 4-30. The ultimate pile capacity was 165 tons. The E used for the ultimate pile capacity determination was back calculated from the plot in Figure 4-30 and was equal to 3,923,977 psi.

ECPT

The ECPT sounding, designated C002A, used for comparison to the result of the PLT was located 28 ft southeast of the PLT, and it was 87.4 ft deep. The sounding was performed 20 June 88 with the UF Geotechnical Engineering Department's ECPT truck using the ten ton tip. The difference between the Qc and Fs base line readings before and after the test were within

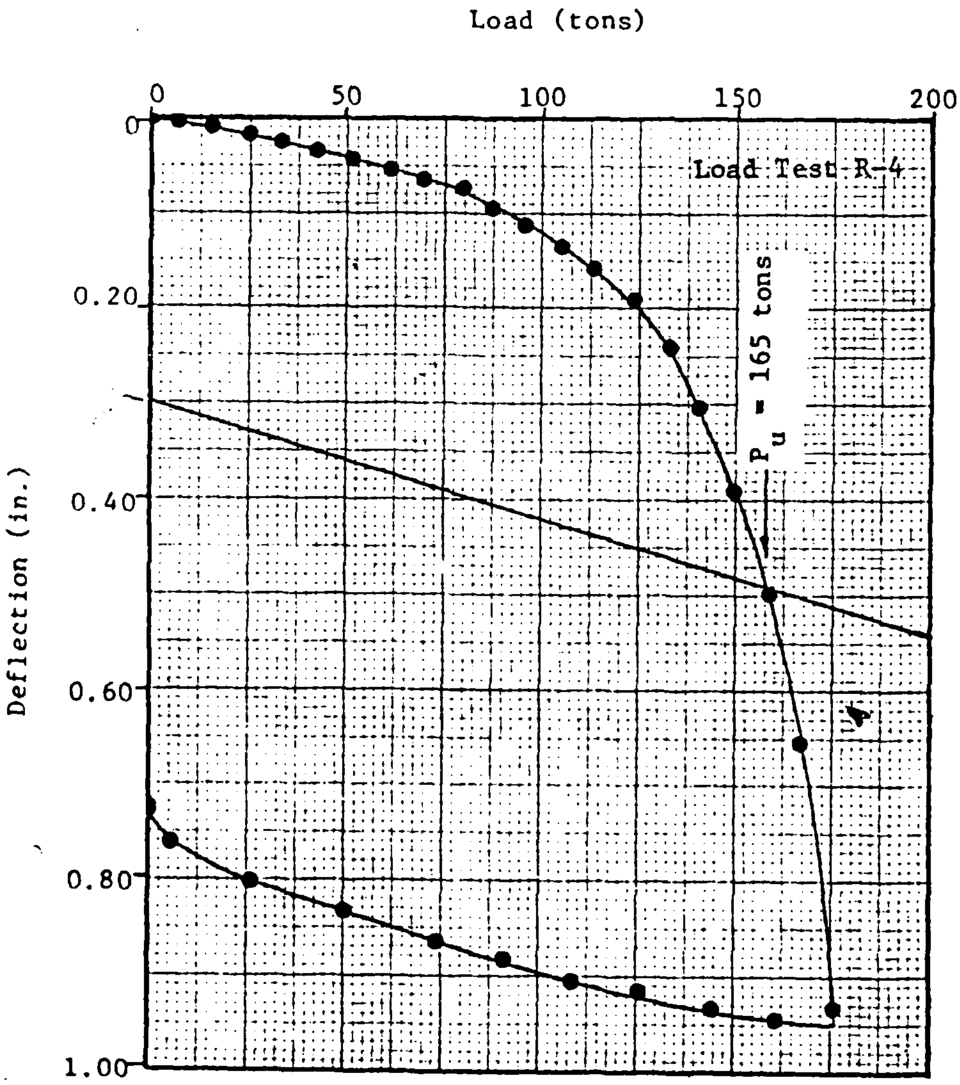


Figure 4-30. Pile Load Test, Load-Settlement Plot, Apalachicola River Bridge FSB 16 (Site 2)

tolerable limits. The difference between the Qc base line readings before and after the test was 0.14 MPa, and the difference between the Fs base line readings before and after the test was 0 kPa. Rod inclination to the depth necessary for pile analysis was only 1.3 degrees. There were no negative Fs readings in the sounding.

Cohesionless and cohesive layer classifications were identified by the ECPT without the benefit of a nearby boring to assist in identification. A summary of the pertinent sounding data for C002A is shown in Figure 4-31. The following layers were identified: a cohesionless layer from the ground surface to 17.7 ft below the ground surface, a cohesive layer from 17.7 to 25.5 ft, a thin cohesionless layer between 25.5 and 30.7 ft, a cohesive layer between 30.7 and 48.9 ft, and a cohesionless layer from 48.9 ft to the analysis termination depth of 75.5 ft. The test pile geometry data was entered in the PLAID program, and the ultimate pile capacity predicted by ECPT analysis was 403 tons which exceeded the PLT ultimate pile capacity determination by 238 tons.

MCPT

The MCPT sounding, designated M002A and used for comparison with the PLT and ECPT at this site, was located at station 132 + 00 and 10 ft to the right of the edge of the road pavement. The MCPT sounding was 31 ft

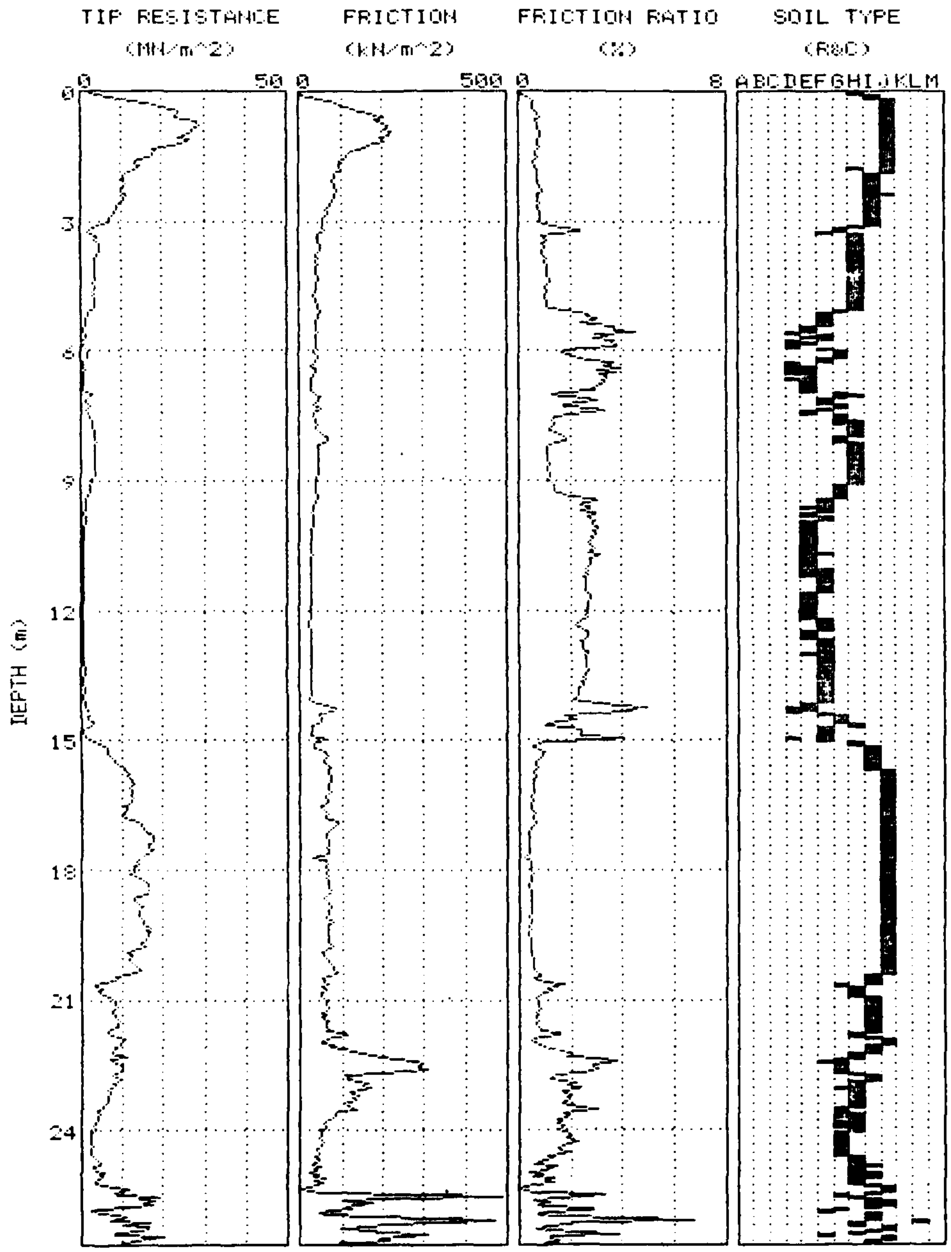


Figure 4-31. ECPT Sounding Data Summary, Apalachicola River Bridge FSB 16 (Site 2)

from the PLT (30 ft south and then 10 ft west of the PLT), 9 ft west of the ECPT, and it was 93.8 ft deep. The FDOT performed the sounding on 6 Dec 84, and it was identified as sounding 3 in their contract plans. Only Qc and FR values were plotted for the sounding as shown in Figure 4-32.

Analysis of the MCPT sounding without the benefit of a nearby boring produced the following cohesionless and cohesive layer divisions: a cohesionless soil layer from the ground surface to 18.4 ft below the ground surface, a cohesive layer from 18.4 to 26.9 ft, a thin cohesionless layer from 26.9 to 30.8 ft, another cohesive layer between 30.8 and 51.2 ft, and a cohesionless layer from 51.2 to an analysis termination depth of 75.5 ft. The second layer of cohesive soil between 30.8 and 51.2 ft was classified as cohesive mainly because of the cohesive classification rendered by the ECPT in this same soil region. Classification with the MCPT results for the latter soil layer was impossible because few or no values were recorded for Qc and FR in this region. Adjacent MCPT soundings also failed to register anything but 0 for Qc and FR within the same depth region. The closest available boring information, 10,000 ft away from M002A, showed both soft clay and very loose sand in this depth region. Based on the ECPT sounding (C002A) and a written interpretation of the layer by Dr. Schmertmann; the layer was classified cohesive with the understanding that,

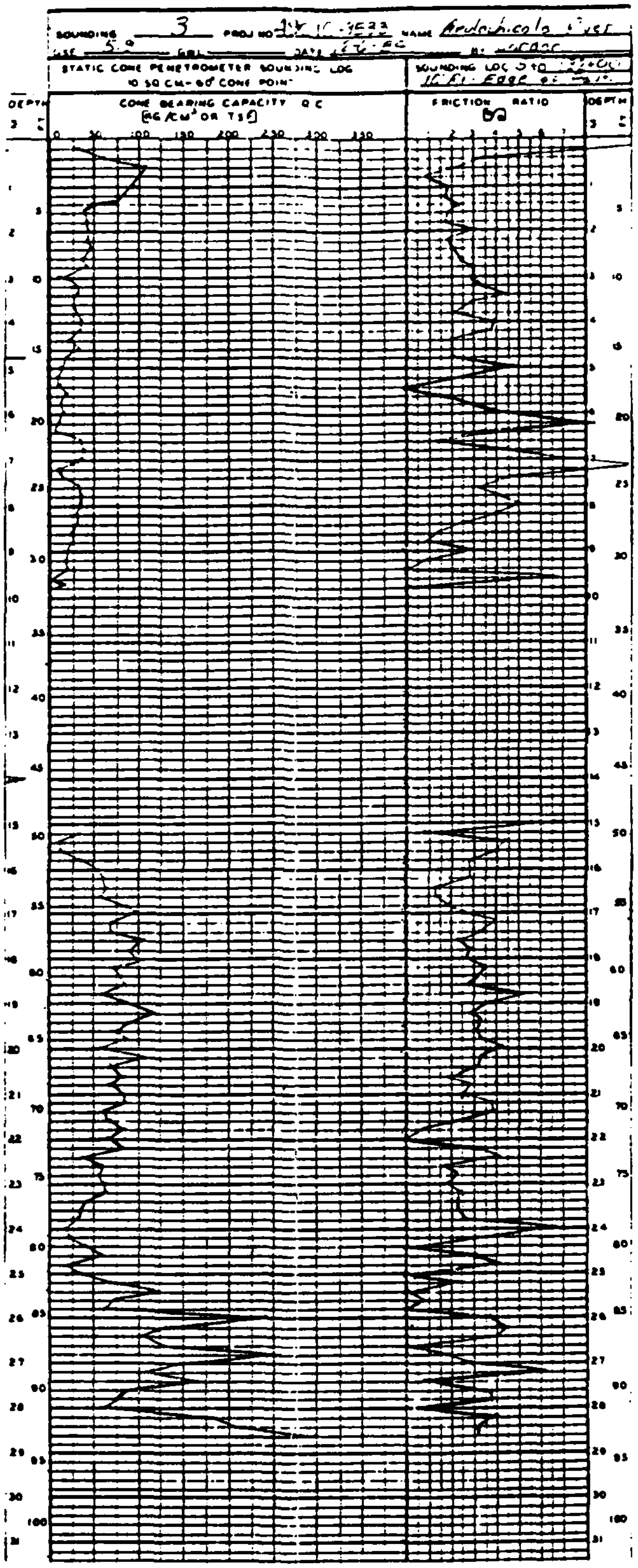


Figure 4-32. MCPT Sounding Data Summary, Apalachicola River Bridge FSB 16 (Site 2)

regardless of classification, it would contribute essentially nothing to the predicted pile capacity. In running the MCPTUFR program, the Qc and FR values for the fourth soil layer were entered as 0.1 tsf and 0% respectively. The predicted MCPT ultimate pile capacity for the sounding was 203 tons, which was 38 tons more than the PLT and 200 tons less than the ECPT predicted ultimate pile capacity.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-16 for comparison.

Table 4-16 - Apalachicola River Bridge FSB 16, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P002	165				
ECPT C002A	403	157	68	89	+144%
MCPT M002A	203	80.5	38.5	42	+23%

Both the ECPT and MCPT overpredicted the ultimate pile capacity determined by the PLT by 144% and 23% respectively. The design and ultimate pile capacities predicted by the ECPT were almost double those predicted by the MCPT. The ECPT design pile capacity was composed of 43% design side friction and 57% design end bearing.

Similarly, the MCPT design pile capacity was composed of 47% design side friction and 53% design end bearing. The ECPT ultimate pile capacity was composed of 34% design side friction and 66% design end bearing. Similarly, the MCPT ultimate pile capacity was composed of 38% design side friction and 62% design end bearing. In an interpretation of the PLT performed by Dr. Schmertmann, he listed the maximum pile load as 177 tons with end bearing constituting 43% of the load and side friction making up 57% of the load. These latter percentages were fairly comparable to those listed above for the ECPT and MCPT. Further analysis of the two soundings was necessary to try and pinpoint the reasons for overprediction by both tests when compred to the PLT results. Table 4-17 was constructed to try and shed light on the disparities.

The reason for the ECPT pile capacity prediction being far higher than the MCPT prediction was clear from the above data. The ECPT average Qc in the bottom layer of soil, which was the soil layer with the most influence on design end bearing, was 36% greater than the MCPT average Qc. The overprediction by both tests compared to the PLT did not appear to be due to any cemented sand as neither sounding suggested cemented sand anywhere. The possibility of weak soils farther below the pile tip similar to the soil in the fourth layer was considered. A weak soil layer below the pile tip might have

Table 4-17 - Detailed Comparison of Apalachicola River Bridge FSB 16 MCPT and ECPT Soundings

Test	MCPT	ECPT
Layer 1 Depth	0 to 18.4 ft	0 to 17.7 ft
Soil Class.	Clayey Sand and Silt	Sand to Silty Sand
Avg Qc (tsf)	40	108
Avg FR (%)	2.73	1.00
Layer 2 Depth	18.4 to 26.9 ft	17.7 to 25.5 ft
Soil Class.	Very Stiff Clay	Clayey Silt to Silty Clay
Avg Qc (tsf)	22	16
Avg FR (%)	4.91	2.75
Layer 3 Depth	26.9 to 30.8 ft	25.5 to 30.7 ft
Soil Class.	Clayey Sands and Silts	Silty Sand to Sandy Silt
Avg Qc (tsf)	25	37
Avg FR (%)	2.17	1.30
Layer 4 Depth	30.8 to 51.2 ft	30.7 to 48.9 ft
Soil Class.	Soft Clay/Loose Sand Guessed from distant boring results.	Clayey Silt to Silty Clay
Avg Qc (tsf)	0	12
Avg FR (%)	0.00	2.65
Layer 5 Depth	51.2 to 75.5 ft	48.9 to 75.5 ft
Soil Class.	Clayey Sand and Silt	Sand to Silty Sand
Avg Qc (tsf)	77	105
Avg FR (%)	2.97	1.07

influenced and caused a lower pile capacity in the PLT but may not have influenced the cone penetration tests. The only weak soil encountered by both the MCPT and ECPT to their terminal test depths was in the 77 to 80 ft depth range where the average Qc was between 34 and 38 tsf for the two tests. But the latter depth range was 13 ft below the 66.63 ft depth which was the point 3.75B below the pile tip, so it was well below the zone of end bearing influence. The end bearing predicted by the two penetration tests must have been due to their overrepresenting the soil strength in the end bearing influence zone 8B above and 3.75B below the pile tip. The latter zone was almost completely in the fifth soil layer for both tests. It was possible the friction resistance predicted by the two penetration tests was also higher than that experienced by the test pile. But no clear reason was apparent for the overprediction of the pile capacity by the two penetration tests.

Comparing the soil layer determinations made by the MCPT and ECPT shown in Table 4-17, it was readily apparent that the layer divisions identified by the two tests were very comparable. Regarding soil classification, the MCPT again classified the soil layers as containing more fines than the ECPT's classification of these layers. Average Qc values for both tests for the three middle soil layers were very comparable; however, the MCPT average Qc values in the soil profile's top and bottom sand layers were far lower than the ECPT average Qc values in these layers. As seen at previously discussed sites, the MCPT average FR values were considerably higher than the ECPT average FR values.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-30) with the ECPT predicted results (Figure 4-33) using the PLAID program. The PLT load-settlement plot was also shown in Figure 4-33 for easier comparison. Comparison of the ECPT vs. the PLT load-settlement plot plainly showed the unconservative nature of the ECPT load-settlement prediction for this particular site. The ECPT plot of the tip and skin load-settlement curves by themselves were each more comparable with the total load-settlement plot from the PLT, although it is apparent the cone resistance mobilized quicker in the PLT than predicted by the ECPT.

Apalachicola Bay Bridge - FSB 22

Load Test

Apalachicola River Bridge FSB 22 was designated as site 3 in Knox's 1989 PhD dissertation. The pile load test designated P003 was conducted 4 Sept 86 on pile 4 at station 316 + 07 and 8.5 ft right of the bridge centerline. The ground surface elevation was +4.70 ft, and the pile tip elevation was -59.32 ft. The water

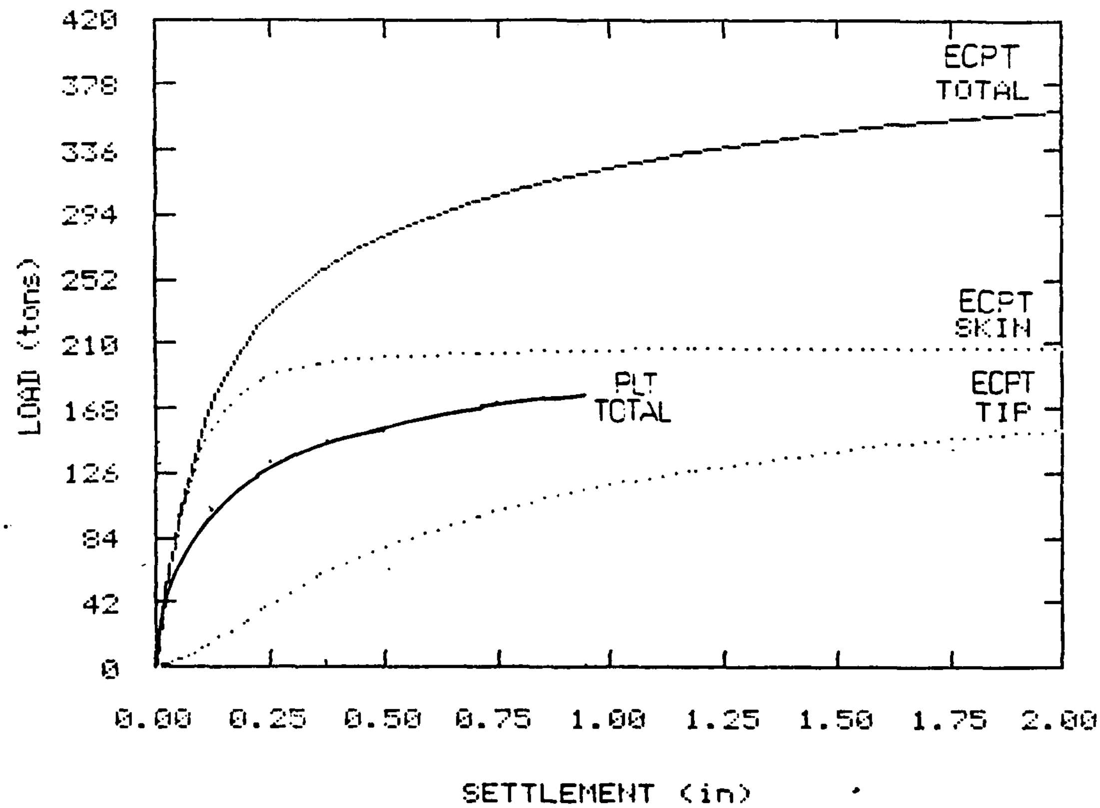


Figure 4-33. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Apalachicola River Bridge FSB 16 (Site 2)

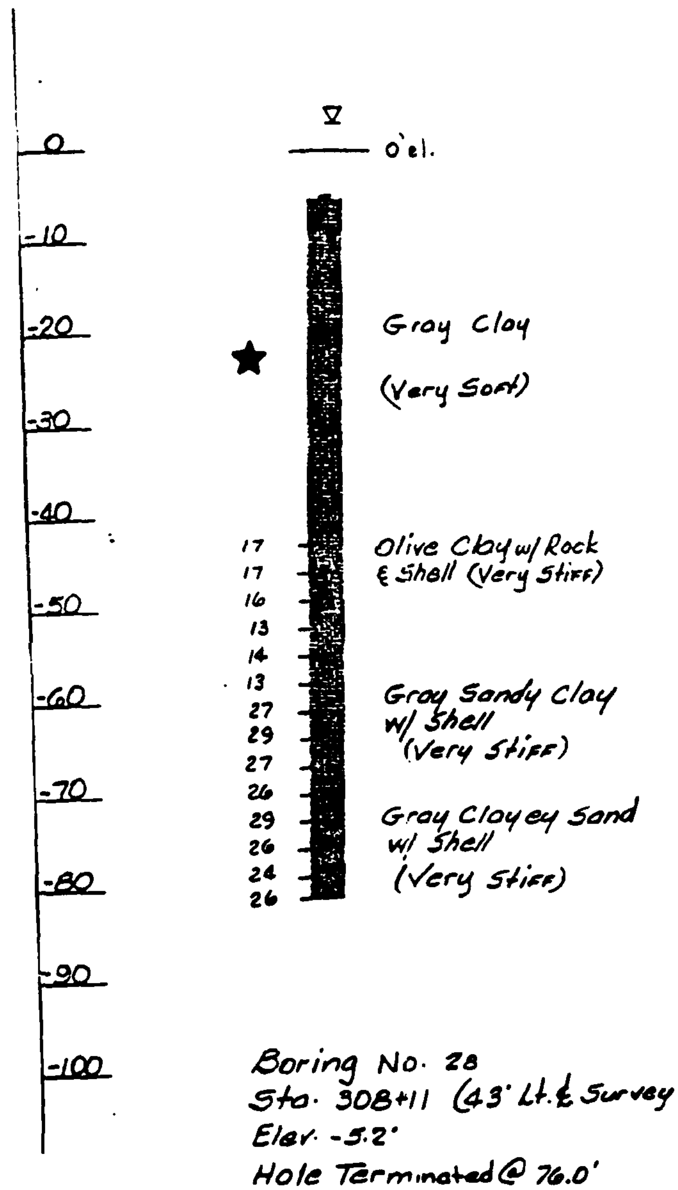


Figure 4-34. Boring Log Summary, Apalachicola Bay Bridge FSB 22 (Site 3)

table was near the ground surface. The length of the pile below the ground surface was 64.02 ft. The total length of the 18 inch square, solid, prestressed concrete pile was 72 ft. No borings were located close to the PLT or the cone penetration tests. The closest boring, shown in Figure 4-34, was over 800 ft away located at station 308 + 11 and 43 ft left of the centerline. The boring, performed below the water surface, identified soft clay between an elevation of -6 and -42 ft, then very stiff clay down to -72 ft, and clayey sand to the boring termination at -82 ft. The PLT load and deflection data were plotted in Figure 4-35. The ultimate pile capacity was 213 tons. The E used for the ultimate pile capacity determination was back calculated from the plot in Figure 4-35 and was equal to 3,769,005 psi.

ECPT

The ECPT sounding, designated C003B and used for comparison to the PLT, was located 53 ft north of the PLT. The sounding was performed 21 June 88 with the UF Geotechnical Engineering Department's ECPT truck using the ten ton tip. The difference between the Qc and Fs base line readings before and after the test were within tolerable limits. The difference between the Qc base line readings before and after the test was 0.14 MPa, and the difference between the Fs base line readings before and after the test was 2 kPa. No rod inclination was

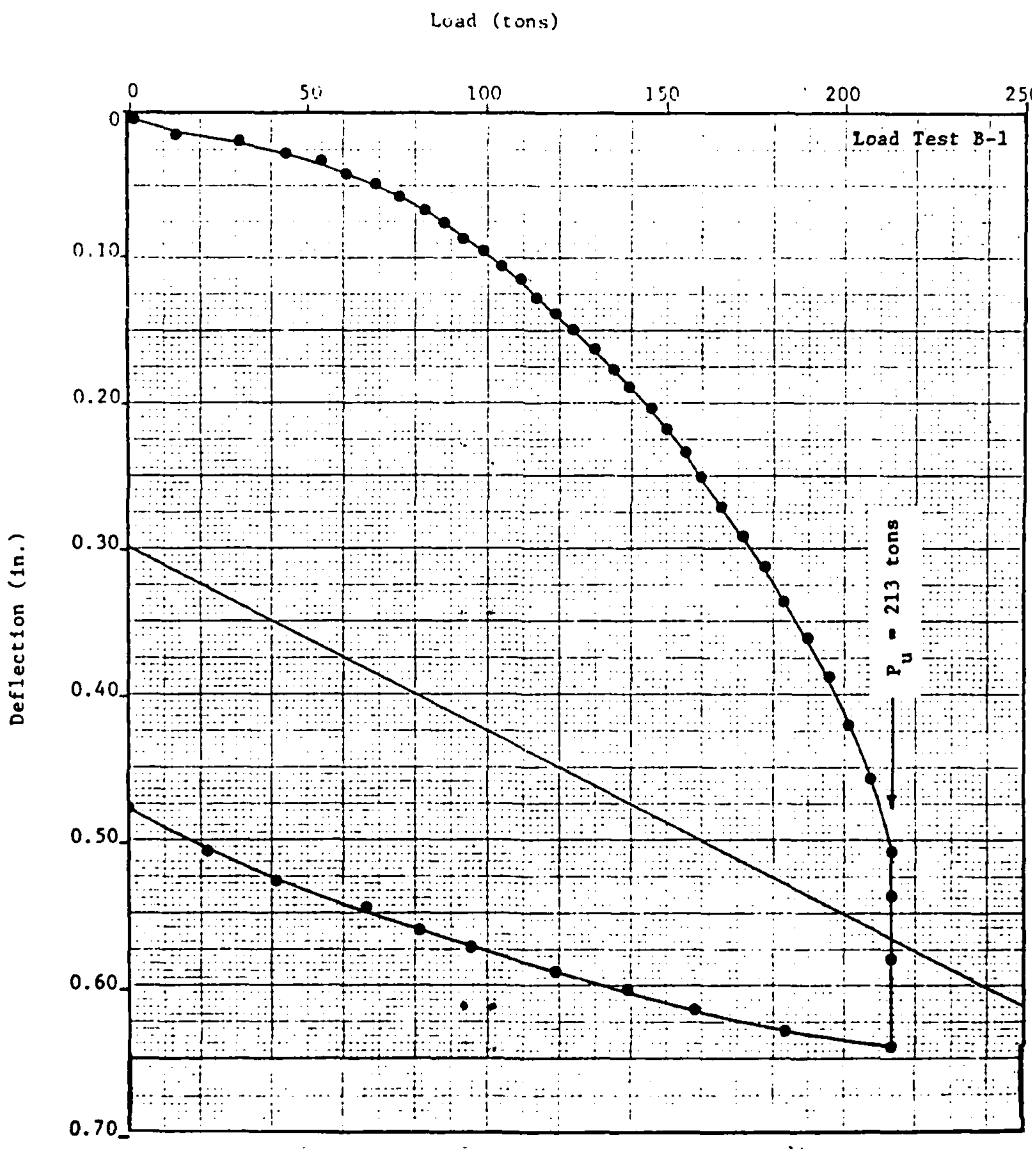


Figure 4-35. Pile Load Test, Load-Settlement Plot, Apalachicola Bay Bridge FSB 22 (Site 3)

recorded to the depth necessary for pile analysis, although there was up to 0.3 degrees of inclination at shallower depths. There were no negative Fs readings in the sounding; however, there was a region between 19.7 and 34.1 ft where no values were recorded for Qc, Fs, and FR. The sounding log showed no data through the latter soil region because apparently the rods just went through this region without registering any Qc, Fs, or FR values due to the extremely penetrable nature of the soil. As a result, the sounding log for C003B had a 14.6 ft (4.45m) offset for all depths and accompanying soil parameter measurements from 6.05 meters on through the termination depth of the sounding. Therefore, the 6.05 m depth and accompanying measurements were actually at the 10.50 m depth.

Cohesionless and cohesive layer classifications were identified by ECPT without the benefit of a nearby boring to assist in identification. A summary of the pertinent sounding data for C003B was shown in Figure 4-36. The following three layers were identified in the soil profile: a cohesionless layer from the ground surface to 19.5 ft below the ground surface, a cohesive layer from 19.5 to 35.1 ft, and a cohesionless layer from 35.1 to 72.9 ft. The middle layer was identified as cohesive as it was thought to be the same soft clay as seen in the boring in Figure 4-34. The test pile geometry data was entered in the PLAID program, and the ultimate pile

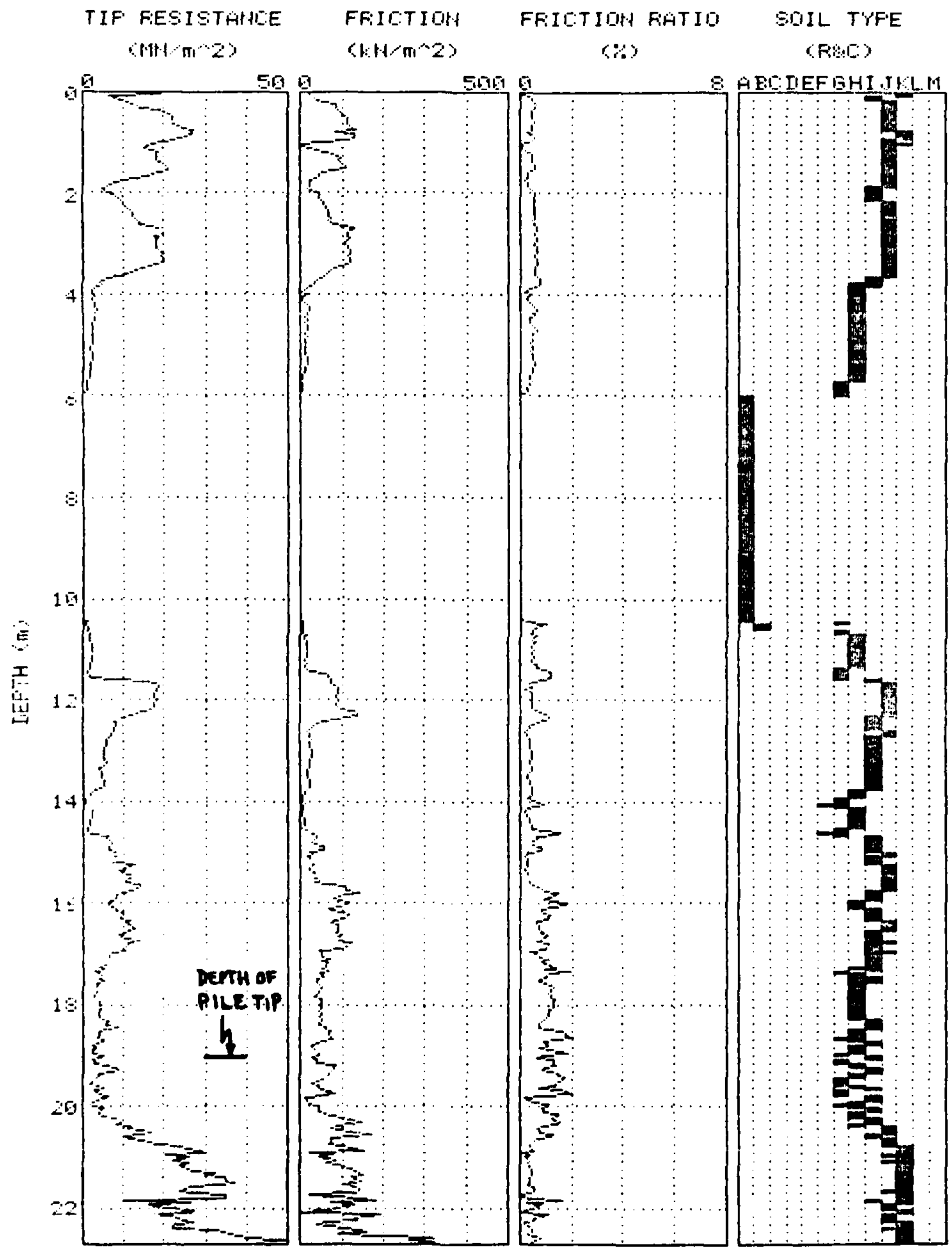


Figure 4-36. ECPT Sounding Data Summary, Apalachicola Bay Bridge FSB 22 (Site 3)

capacity predicted by ECPT analysis was 162 tons which was 51 tons less than the PLT ultimate pile capacity determination.

MCPT

The MCPT sounding, designated M003C and used for comparison with the PLT and ECPT at this site, was located at station 316 + 00 and 10 ft to the right of the paved road edge. The MCPT sounding was 7 ft west of the PLT, and 53.5 (53 ft south and then 7 ft west) from the PLT. The sounding was 71.5 ft deep. The FDOT performed the sounding on 13 Dec 84, and it was identified as sounding 9 in the contract plans. Only Qc and FR values were plotted for this sounding as shown in Figure 4-37. Similar to the ECPT sounding at this site and the MCPT sounding at the previous site, Qc and FR values were almost all 0 between 18 and 37 ft below the ground surface. Presumably, the latter was caused by very soft clay in the latter depth region.

Analysis of the MCPT sounding without the benefit of a nearby boring produced the following cohesionless and cohesive layer divisions: a cohesionless soil layer from the ground surface to 17.7 ft below the ground surface, a cohesive layer from 17.7 to 36.8 ft, and a cohesionless layer from 36.8 to 71.5 ft where sounding analysis was terminated. In running the MCPTUFR program, the Qc and FR values in the second soil layer were entered as 0.1

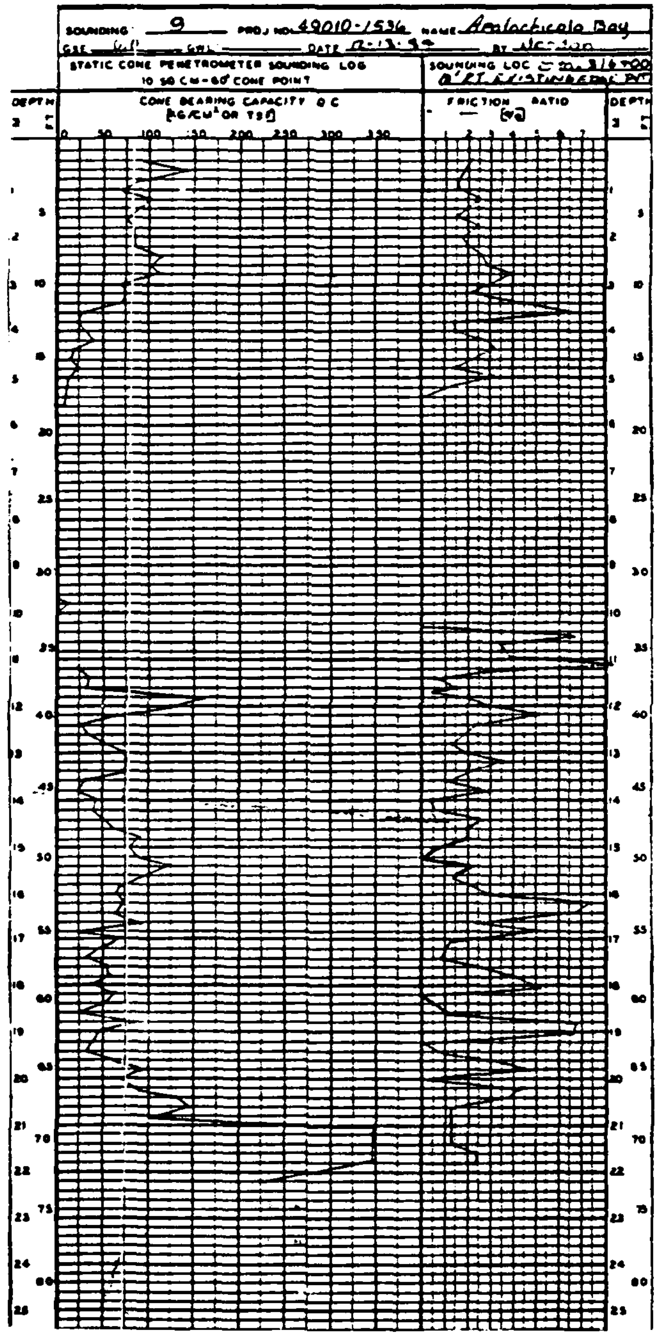


Figure 4-37. MCPT Sounding Data Summary, Apalachicola Bay Bridge FSB 22 (Site 3)

tsf and 0% respectively. The predicted MCPT ultimate pile capacity for the sounding was 195 tons, which was 18 tons less than the PLT and 33 tons more than the ECPT predicted ultimate pile capacity.

Predicted vs. Observed Pile Capacity

Pile capacities determined by the PLT, ECPT, and MCPT were shown in Table 4-18 for comparison.

Table 4-18 - Apalachicola Bay Bridge FSB 22, Pile Capacity Comparisons

Test and Code Name	Ultimate Pile Capacity (tons)	Design Pile Capacity (tons)	Design Side Friction (tons)	Design End Bearing (tons)	% Above/ Below PLT
PLT P003	213				
ECPT C003B	162	71	51	20	-24%
MCPT M003C	195	78	39	39	- 8%

Both the ECPT and MCPT underpredicted the ultimate pile capacity determined by the PLT by 24% and 8% respectively. The design and ultimate pile capacities predicted by the ECPT were very comparable to those predicted by the MCPT. However, the MCPT design pile capacity was the result of equal contributions made by design side friction and design end bearing as shown in Table 4-18. Whereas, the ECPT design pile capacity was 72% design side friction and 28% design end bearing. Further analysis of the two soundings was necessary to try and pinpoint the reasons for the difference between the two tests' predictions and the underprediction of the PLT ultimate pile capacity by both tests. Table 4-19 was constructed to try and shed light on the disparities.

Table 4-19 - Detailed Comparison of Apalachicola Bay Bridge FSB 22 MCPT and ECPT Soundings

Test	MCPT	ECPT
Layer 1 Depth	0 to 17.7 ft	0 to 19.5 ft
Soil Class.	Clayey Sands and Silts	Sand
Avg Qc (tsf)	66	115
Avg FR (%)	2.44	0.48
Layer 2 Depth	17.7 to 36.8 ft	19.5 to 35.1 ft
Soil Class.	Soft Clay	Soft Clay
Avg Qc (tsf)	2	0
Avg FR (%)	0.89	0.03
Layer 3 Depth	36.8 to 71.5 ft	35.1 to 72.9 ft
Soil Class.	Clayey Sands and Silts	Sand
Avg Qc (tsf)	93	111
Avg FR (%)	2.43	0.74

The reason for the ECPT ultimate and design pile capacity predictions being less than the MCPT predictions was not readily apparent after reviewing Table 4-19. The ECPT average Qc in the bottom layer of soil which had the most influence on design end bearing was 19% greater than the MCPT average Qc. Yet, in a seeming contradiction, the design end bearing prediction for the ECPT was only half of the MCPT design end bearing prediction as shown in Table 4-18. The underprediction by both tests compared to the PLT also seemed surprising since the high average Qc values in the third layer, which contained the zone of end bearing influence, had led to pile capacity overpredictions at sites discussed previously in this chapter. However, further analysis of the soundings showed that for both the ECPT and MCPT the lowest Qc values in the third soil layer were located right around the pile tip. The MCPT sounding had low Qc values between 30 and 40 tsf located within 3 ft of the pile tip. Similarly, the ECPT sounding had low Qc values between 22 and 47 tsf within 1 ft of the pile tip with the lowest Qc value right at the pile tip location. These low Qc values were prevalent in the same area in both soundings, and they were over too large a depth range to be considered low spikes in the sounding data. Consequently, using the minimum path method in the end bearing analysis for both tests, the predicted design end bearings were lower than would have been expected for a soil layer with such a relatively high average Qc. The ECPT had lower individual Qc values than the MCPT, so it resulted in a lower design end bearing for the ECPT analysis. The test pile in the PLT must not have been affected by the lower Qc values measured by the soundings, or the ECPT and MCPT measured values underrepresented the actual end bearing capability of the soil in layer three.

Analysis of the average Qc values within the critical region for end bearing, 8B above and 3.75B below the pile tip, more simply explained the reason for the MCPT's higher design end bearing compared to the ECPT prediction shown back in Table 4-18. Table 4-20 showed the higher average Qc value within the critical depth region for end bearing belonged to the MCPT analysis

Table 4-20 - Comparison of Apalachicola Bay Bridge FSB 22 Average Qc Values from MCPT and ECPT Soundings in Critical Depth Region for End Bearing

Test	MCPT	ECPT
Crit. Depth Range	52.0 to 69.7 ft	51.8 to 70.2 ft
Avg Qc (tsf)	96	89

The soil layer determinations by the ECPT and MCPT were very comparable. The sensitive nature of the second soil layer was experienced by both tests. The MCPT average Qc in both the top and bottom cohesionless soil layers was significantly lower than the ECPT average Qc. As seen in the rest of the sites examined in this chapter, the MCPT average FR was considerably higher than the ECPT average FR. Consequently, the MCPT classification of the soils in the first and third layers suggested a higher fines content and more cohesive soil than determined by ECPT classification.

A load-settlement analysis was performed to compare the actual PLT results (Figure 4-35) with the ECPT predicted results (Figure 4-38) using the PLAID program.

The PLT load-settlement plot was also shown in Figure 4-38 for simpler comparison. Comparison of the ECPT vs. the PLT load-settlement plot plainly showed the accurate yet conservative nature of the ECPT load-settlement prediction for this particular site.

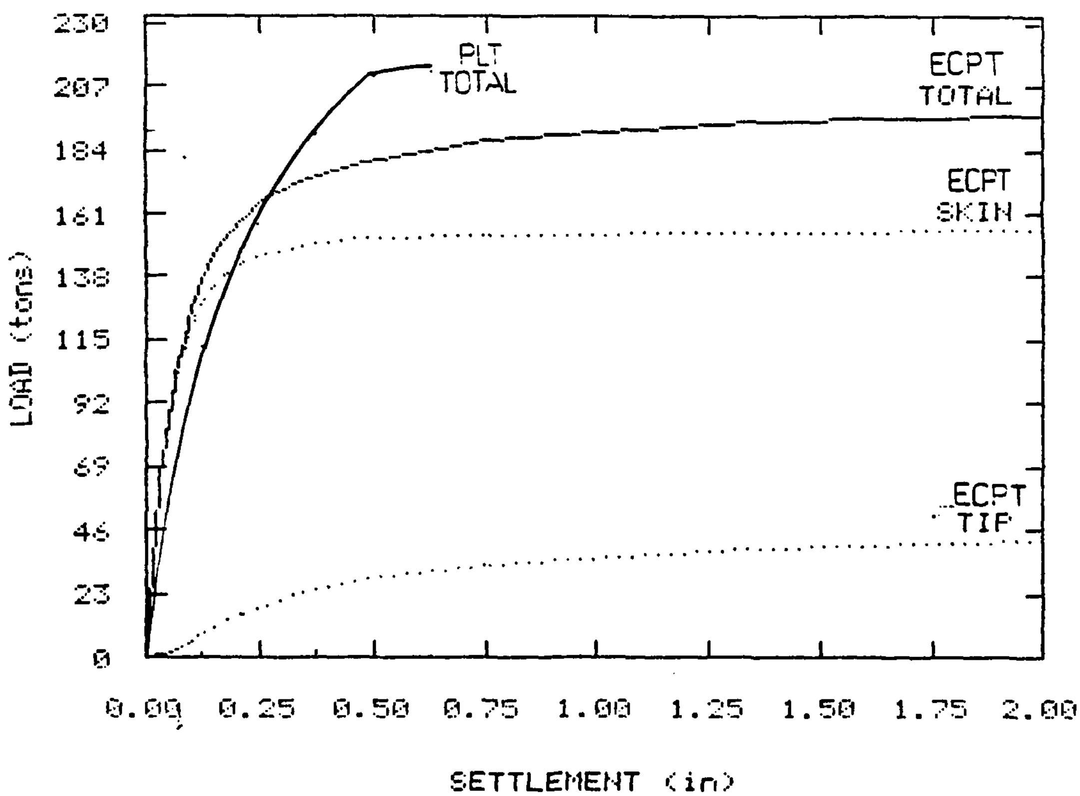


Figure 4-38. Plot and Comparison of Predicted ECPT and Observed PLT Load-Settlement Results, Apalachicola Bay Bridge FSB 22 (Site 3)

CHAPTER 5

OVERALL TEST RESULTS AND DISCUSSION

Predicted vs. Observed Pile Capacity Results

The test results and discussion of each individual test site in the previous chapter showed the ECPT and MCPT pile capacity predictions varied from observed PLT pile capacity results. Additionally, there appeared to be a wide range in the degree of variance for predicted vs. observed pile capacities. However, the large overpredictions of pile capacity by the ECPT and MCPT seemed to occur when the presence of cemented sands was suggested by analysis of the soundings. Robertson and Campanella found that compressibility was reduced, and consequently Qc was increased, when there was cementation between sand particles (1984c). Meigh related how investigations in the Rankine field offshore of Western Australia found a close correlation between cone resistance and cementation. These Western Australia soils were calcareous containing a lot of shells and other marine life remains. Site specific conclusions of the Western Australia investigations were that cemented conditions were characterized by high cone resistance and relatively low friction ratios (1987c).

Without the benefit of lab testing results to confirm the presence of cemented sands, another method was sought to try and verify the presence of cementation. Studies by Knox suggested the Qc/N ratio for a given soil may identify Florida cemented sands where, over a given soil depth region, Qc was the cone resistance or end bearing in tsf and N was the standard penetration test blow count for the last 12 inches of 18 inches of total penetration. Knox found typical Florida sands have a Qc/N ratio between 4.5 and 7.4. He suggested cemented sands have a Qc/N ratio between 8 and 20 (1989f). Fortunately, the borings identified for each site in the previous chapter contained the SPT N values recorded for each soil profile. Tables 5-1 and 5-2 were compiled to identify the average Qc and average N values in the critical depth region for end bearing, 8B above and 3.75B below the pile tip, at each site. The Qc/N values for both the ECPT and MCPT at each site were shown in Table 5-2. The only site ignored in the compilation was site 2 at Apalachicola River Bridge FSB 16, because no boring was located anywhere near the PLT at this particular site.

Review of Table 5-2 revealed Qc/N ratios between 6.5 and 11.1 which suggested the presence of cemented sands at sites 13, 14, 20, and possibly 19. Sites 1, 3, and 21 definitely did not suggest cemented sands with their much lower Qc/N ratios (between 3.6 and 4.5), which were all

Table 5-1 - Critical Depth Regions for End Bearing and Corresponding Average Qc Values for Each Test Location

Location and Site Number	Critical Depth Region 8B Above to 3.75B Below Pile Tip	Avg Qc in Critical Depth Region ECPT (tsf)	Avg Qc in Critical Depth Region MCPT (tsf)
Apalach River Pier 3 Site 1	74.6 to 98.1 ft	116	N/A
Apalach Bay FSB 22 Site 3	52.0 to 69.7 ft	88	95
Port Orange Bent 19 Site 13	18.9 to 36.5 ft	240	189
Port Orange Bent 2 Site 14	18.0 to 35.6 ft	284	220
Choctaw Bay FSB 3 Site 19	65.7 to 89.2 ft	79	65
Choctaw Bay Pier 5 Site 20	45.9 to 75.3 ft	208	193
Choctaw Bay FSB 26 Site 21	48.9 to 78.3 ft	82	68

Table 5-2 - Average SPT N Values in the Critical Depth Region for End Bearing and Corresponding Qc/N Ratios for Each Test Location

Location and Site Number	Avg N Value in Crit Depth Region	Qc/N Ratio ECPT	Qc/N Ratio MCPT
Apalach River Pier 3 Site 1	31.3	3.7	N/A
Apalach Bay FSB 22 Site 3	21.3	4.2	4.5
Port Orange Bent 19 Site 13	27.4	8.8	6.9
Port Orange Bent 2 Site 14	26.6	10.7	8.3
Choctaw Bay FSB 3 Site 19	10.0	7.9	6.5
Choctaw Bay Pier 5 Site 20	18.8	11.1	10.3
Choctaw Bay FSB 26 Site 21	18.6	4.4	3.6

closer to typical Qc/N values for fine to medium sands as per Peck (1974b). Within the critical depth region for end bearing at site 21, the Qc and N values that were within the slurried region were changed to 0 to obtain realistic average Qc and N values. The slurried regions at both sites 19 and 20 were above the top of the critical depth region for end bearing. Assuming that Qc/N ratios between 8 and 20 truly indicated the presence of cemented sand, Table 5-3 was assembled to determine whether or not a correlation existed between ECPT and MCPT pile capacity overpredictions and the existence of cemented sands.

The large ECPT and MCPT ultimate pile capacity overpredictions at sites 13, 14, and 20 were accompanied by Qc/N ratios greater than 8 as shown clearly in Table 5-3. The only exception was at site 13 where the large pile capacity overprediction was accompanied by a Qc/N ratio of 6.9, which was still greater than the typical sand Qc/N ratio between 3 and 6 that was found by Peck (1974c). The remaining sites had Qc/N values less than 8, and they had ECPT and MCPT ultimate pile capacity predictions within 30% of the PLT ultimate pile capacity. The ECPT and MCPT ultimate pile capacity overpredictions at site 21 were expected due to the nature of the deeply slurried PLT, but they were still reasonably close to the PLT-determined ultimate pile capacity. Table 5-3 showed a valuable correlation between cone penetration test pile

Table 5-3 - Relationship Between ECPT and MCPT Predicted Ultimate Pile Capacities and Qc/N Ratios

Location and Site Number	% ECPT Above/Below PLT Pile Cap	ECPT Qc/N Ratio	% MCPT Above/Below PLT Pile Cap	MCPT Qc/N Ratio
Apalach River Pier 3 Site 1	+13	3.7	N/A	N/A
Apalach Bay FSB 22 Site 3	-24	4.2	- 8	4.5
Port Orange Bent 19 Site 13	+309	8.8	+227	6.9
Port Orange Bent 2 Site 14	+186	10.7	+178	8.3
Choctaw Bay FSB 3 Site 19	-13	7.9	- 4	6.5
Choctaw Bay Pier 5 Site 20	+52	11.1	+68	10.3
Choctaw Bay FSB 26 Site 21	+30	4.4	+17	3.6

capacity overpredictions and high Qc/N ratios, typically Qc/N greater than 8. As mentioned previously, a Qc/N ratio greater than 8 has been suggested as an indicator of the presence of cemented sand. Therefore, cemented sands may have caused the ultimate pile capacity overpredictions by both the ECPT and MCPT.

The reason for cemented sand causing cone penetration test ultimate pile capacity overpredictions must be examined to prevent faulty future designs using conventional processes. The penetration of the mechanical and electrical tips exerted much smaller forces over a much smaller area than the driven test pile and may not have been great enough to break up the bonds joining cemented sand particles. On the other hand, the large forces and vibrations accompanying the driving of the test pile during the PLT may have broken the bonds that gave the cemented sands the high end bearing capability detected by the cone penetration tests. Consequently upon reaching the region of cemented sand with broken bonds, the test pile was not in soil with the same characteristics measured previously by the cone penetration tests. In fact, the test pile was now in a soil with a much lower end bearing capability than had been measured by the ECPT and/or MCPT. The lower end bearing capability was a direct result of the destroyed bonds between particles that were now uncemented due to the effects of driving. As a result, the lower PLT ultimate pile capacity compared to the pile capacity predicted by the ECPT or MCPT was not surprising. The pushed penetration of the cone penetrometer tip can easily be imagined as much less "traumatic" for cemented sand as opposed to the driven penetration of the test pile. Studies by Clough on naturally and artificially cemented sands in California concluded that sand cementation had a number of effects. The latter studies concluded that a cohesion intercept and a tensile strength were added to the sand resulting in an increased stiffness, but the friction angle of the sand remained essentially unchanged. In addition, the studies found that the stiffness, tensile strength, and cohesion intercept were sensitive to the amount and nature of the cementing agent (Clough, 1981a). Therefore, the greater end bearing capability recorded by the cone penetration tests was a result of the increased stiffness due to cementation. Presumably, the increased stiffness due to cementation was eliminated by the driving force and accompanying vibrations associated with the PLT.

The ideas expressed in the previous paragraph did not necessarily explain why the pile capacity overprediction at site 20 was not nearly as high as the overpredictions at sites 13 and 14 even though site 20 had the greater Qc/N ratio and presumably the stronger cementation. However, one fact that may have caused the latter was that the pile tip was nearly twice as deep at site 20 (57.1 ft below the ground surface) compared to the pile tip depth at sites 13 and 14 (between 30 and 31 ft below ground surface). With less overburden to confine and possibly dampen the pile driving effects at sites 13 and 14, the cemented sands at these sites may have suffered far greater bond breakage and resulting strength loss during pile driving than the deeper lying cemented sand at site 20. However, the latter may be too simplistic an explanation. Clough found that, in addition to the type and degree of cementation, other factors played important parts in cemented sand behavior. They also cited grain size distribution, density, and grain arrangements as contributing factors in the behavior of cemented sand (1981b).

Spatial Variability

In order to determine whether or not spatial variability had anything to do with the accuracy or inaccuracy of the ECPT and MCPT ultimate pile capacity predictions, Table 5-4 was constructed.

As shown in Table 5-4, the separation distances between the PLT and the ECPT or MCPT were all 80 ft or less. The average distance between the ECPT tests and their corresponding PLT was 45 ft, and the average distance between the MCPT tests and their corresponding PLT was 47 ft. With the exception of the ECPT ultimate pile capacity prediction at site 2; the highest

Table 5-4 - Relationship Between ECPT and MCPT Predicted Ultimate Pile Capacities and Separation Distance from PLT

Location and Site Number	% ECPT Above/Below PLT Pile Cap	ECPT to PLT (ft)	% MCPT Above/Below PLT Pile Cap	MCPT to PLT (ft)
Apalach River Pier 3 Site 1	+13	16	N/A	N/A
Apalach River FSB 16 Site 2	+144	28	+23	31
Apalach Bay FSB 22 Site 3	-24	53	- 8	7
Port Orange Bent 19 Site 13	+309	70	+227	65
Port Orange Bent 2 Site 14	+186	47	+178	75
Choctaw Bay FSB 3 Site 19	-13	41	- 4	44
Choctaw Bay Pier 5 Site 20	+52	74	+68	80
Choctaw Bay FSB 26 Site 21	+30	28	+17	29

overpredictions at sites 13, 14, and 20 did coincide with the farthest horizontal distance separations between the given PLT and coinciding ECPT or MCPT. At the other end of the spectrum, two of the three closest predictions, the ECPT at site 1 and the MCPT at site 3, were made with the least separation distance between the cone sounding and the PLT. Careful review of the remaining tests in Table 5-4 showed the wide range of underpredictions and overpredictions did not truly seem to coincide with separation distance. Within a separation distance of 28 to 53 ft, there was a wide range of predicted pile capacities between -24% and +144% of the observed PLT ultimate pile capacity. The latter would suggest weakness in any proposed direct correlation between the accuracy of ECPT and MCPT ultimate pile capacity predictions and the separation distance between the cone penetration tests and the corresponding PLT. However, it was significant that the best predictions generally corresponded to the smallest separation distances, and the worst predictions generally corresponded to the largest separation distances. Cone penetration tests closest to their corresponding PLT were expected to make the best pile capacity predictions, because the soil profile probed by the cone would more closely match the soil profile corresponding to the PLT soil profile. Conversely, cone penetration tests farthest from their corresponding PLT were expected to make the worst pile capacity predictions, because the soil profile probed by the cone would not correspond as well as closer soundings would with the actual PLT soil profile.

Comparison of the ECPT and MCPT Soundings and Their Predicted Pile Capacities

A further in-depth comparison of the MCPT and ECPT results was not performed beyond the individual site comparisons in the previous chapter and the preceding analysis in this chapter. Nonetheless, significant observations were easily made in the course of the preceding analyses.

First of all, the depths of soil layer divisions determined by the ECPT and the MCPT were very comparable at all of the sites except at both site 1, where no MCPT close to the PLT was deep enough for pile analysis, and site 21. Coincidentally, site 21 was where 60 ft of slurry was used which minimized the importance of soil layer divisions through most of the soil profile. None of the ECPT and MCPT layer depth divisions were forced to coincide with one another for easier comparative analysis. A few very thin layers that were detected above the critical depth region for end bearing within the slurried depths at the Choctawhatchee Bay Bridge sites 19-21 were judged to be inconsequential and were ignored. The soil layer division depths determined by the ECPT and MCPT also corresponded well with the boring log soil layer divisions at the sites with nearby borings.

In the area of soil classification, the MCPT always registered a much higher average FR value than the ECPT for any given site resulting in MCPT soil classifications that showed the presence of more fine material than corresponding ECPT soil classifications. As evidenced by the design curve in Figure 2-8, the MCPT Begemann tip was expected to produce friction resistance and sleeve friction values about twice the value of the smoother penetrating ECPT tip in cohesionless soils (Schmertmann, 1978f); yet nearly all of the sites had MCPT average FR values that were well above double the ECPT average FR values for a given soil layer. The only exception to the latter was the fourth soil layer at site 2 where the MCPT failed to register any Qc or FR values in an approximately 20 ft thick layer of soft clays and very loose sands. With the exception of Port Orange sites 13 and 14, the MCPT was generally more on target in identifying clay soils because the ECPT seldom registered high enough FR values to consider clay classification for a given soil layer. In addition, the latter was due to the fact the ECPT average FR values in a given layer were seldom even above 1%. On the few occasions when the ECPT average FR was over 1% for a given soil layer, the MCPT average FR was usually about three times the ECPT average FR value. Since, very generally speaking, the ECPT and

MCPT average Qc values were fairly comparable for most of the soil layers analyzed in Chapter 4; the great difference in average FR values was due to a large difference in the friction resistance measurements made by the ECPT and MCPT. Robertson and Campanella found that different cone resistance or end bearing, friction resistance, and FR values resulted from cones of slightly different designs. Studies by the latter also found friction resistance values were generally less accurate than cone resistance measurements (1984d).

The MCPT did a better job of identifying cemented sand soil layers. With only one exception at site 2, an MCPT overprediction of ultimate pile capacity was accompanied by an MCPT soil classification of dense or cemented sand in the soil layer around the pile tip. Furthermore, the MCPT ultimate pile capacity overprediction at site 2 was only by 23%. In the case of ECPT ultimate pile capacity overpredictions, the ECPT did not register high enough average FR values to classify soils around the pile tip as cemented sands. Side-by-side comparison of the ECPT and MCPT ultimate pile capacity predictions compared to the PLT was shown in Table 5-5.

Review of Table 5-5 showed that the ECPT and MCPT either both overpredicted or both underpredicted the ultimate pile capacity at any given site. Only site 1 had no MCPT result to compare with the ECPT. Both the

Table 5-5 - Comparison of ECPT and MCPT Predicted Ultimate Pile Capacities in Relation to Observed PLT Ultimate Pile Capacity

Location and Site Number	Observed PLT Ultimate Pile Cap. (tons)	ECPT % Above/Below PLT	MCPT % Above/Below PLT
Apalach River Pier 3 Site 1	479	+13	N/A
Apalach River FSB 16 Site 2	165	+144	+23
Apalach Bay FSB 22 Site 3	213	-24	- 8
Port Orange Bent 19 Site 13	101.5	+309	+227
Port Orange Bent 2 Site 14	139	+186	+178
Choctaw Bay FSB 3 Site 19	248	-13	- 4
Choctaw Bay Pier 5 Site 20	626	+52	+68
Choctaw Bay FSB 26 Site 21	481	+30	+17

ECPT and MCPT pile capacity predictions were fairly comparable to one another. The ECPT had the most extreme pile capacity predictions when compared to the MCPT except at site 20. In other words, the ECPT tended to have more of a pile capacity underprediction or overprediction than the MCPT. The latter was not attributable to ECPT and MCPT separation distance from the PLT, because further analysis of Table 5-4 revealed that that the average separation distances were comparable. The average distance between the ECPT and the PLT was 45 ft, and the average separation between the MCPT and the PLT was 47 ft.

Comparison of the Predicted ECPT and Observed PLT Load-Settlement Results

Review of the individual site load-settlement comparisons in the previous chapter showed the predicted ECPT load-settlement plots were either close to or were conservative compared to the observed PLT load-settlement results if cemented sands were not present at the site. Table 5-6 was constructed to show whether or not an overall ECPT load-settlement prediction for a particular site was conservative, unconservative, or nearly equal to the PLT load-settlement results. As clearly shown in Table 5-6, the only unconservative ECPT predicted load-settlement plots corresponded to the three large ECPT pile capacity overpredictions at sites 2, 13, and

Table 5-6 - Comparison of Predicted ECPT and Observed PLT Load-Settlement Results

Location and Site Number	Predicted ECPT Load-Settlement Conservative, Unconservative, or = to PLT	ECPT % Above/Below PLT
Apalach River Pier 3 Site 1	Nearly Equal	+13
Apalach River FSB 16 Site 2	Unconservative	+144
Apalach Bay FSB 22 Site 3	Nearly Equal	-24
Port Orange Bent 19 Site 13	Unconservative	+309
Port Orange Bent 2 Site 14	Unconservative	+186
Choctaw Bay FSB 3 Site 19	Conservative	-13
Choctaw Bay Pier 5 Site 20	Conservative	+52
Choctaw Bay FSB 26 Site 21	Conservative	+30

14. Recalling previous relationships for the latter sites in this chapter, the unconservative ECPT load-settlement predictions may have resulted from the cemented sand regions around the pile tip and/or the large separation distance between the ECPT and PLT. However, the cemented sand effects were judged to be more influential than separation distance.

CHAPTER 6

CONCLUSIONS

The results presented in this report led to the following conclusions:

1. The ECPT and MCPT were fairly accurate at predicting ultimate pile capacity (within + or -30%) compared to PLT results, except when cemented sands were present within the critical depth region for end bearing.

2. The ECPT was fairly accurate at making load-settlement predictions, except when cemented sands were present within the critical depth region for end bearing.

3. Better ultimate pile capacity predictions were made by the ECPT and MCPT when the soundings were performed very close to the PLT.

4. The MCPT and ECPT were accurate at detecting the depth divisions between cohesive and cohesionless soil layers when compared with nearby boring log results.

5. The MCPT was generally better than the ECPT at predicting soil classification due to seemingly more accurate friction ratio determinations.

The above conclusions showed cone penetration tests, whether ECPT or MCPT, are valuable tools to an engineer trying to predict the pile capacity at a given site.

However, caution must be exercised, because both the ECPT and MCPT overpredict pile capacity when cemented sands are part of the site's soil profile.

MD14B. By Hand — PREDICTED PILE CAPACITY FROM MCPT — WMC

As w/ CO14A w/ ECPT, single cohesionless soil layer from ground surface to at least 40.0 ft

D = Location of Pile Tip = 30.02' = 9.15m

D-8B = Top of Critical Depth Region for End Bearing = 18.01' = 5.49m below ground surface

D+3.75B = Bottom of " " " " " " = 35.63' = 10.86m " " "

END BEARING CALCULATION

D = 9.15m

x	xB (m)	$D + xB$ (m)	q_{c1} calc.	q_{c1} (tsf)
0.7	0.32	9.47	$\frac{1}{4}[2(185)+2(223)] =$	204.0 min. q_{c1}
1.0	0.46	9.61	$\frac{1}{6}[2(185)+4(223)] =$	210.3
1.5	0.69	9.84	$\frac{1}{8}[2(185)+4(223)+2(231)] =$	215.5
2.0	0.91	10.06	$\frac{1}{10}[2(185)+2(223)+231+5(218)] =$	213.7
2.5	1.14	10.29	$\frac{1}{12}[2(185)+2(223)+231+218+6(200)] =$	205.4
3.0	1.37	10.52	$\frac{1}{14}[2(185)+2(223)+231+218+2(210)+6(200)] =$	206.1
3.5	1.60	10.75	$\frac{1}{16}[2(185)+2(223)+231+218+2(210)+2(226)+6(200)] =$	208.6
3.75	1.71	10.86	$\frac{1}{18}[2(185)+2(223)+231+218+210+226+200+9(197)] =$	204.1

$\therefore$ 204.0 = q_{c1} min.

Now find q_{c2} which affects pile from tip up to a depth of 5.49 m

Find critical q_{c2} value looking 1.5 B below pile tip = 30.02' + 2.25' = 32.27' = 9.84 m

Lowest value of q_c between tip & 1.5B below it is the q_c value at the tip = 185 tsf

$q_{c2} = \frac{1}{19}[185 + 170 + 13(148) + 2(128) + 2(25)] = 136.05$ tsf

19 values of q_c between 5.49m and 9.15m

$q_p = \frac{1}{2}(q_{c1} + q_{c2}) = .5(204.0 + 136.05) = 170.03$ tsf, but must use 150 tsf which is maximum allowable tip capacity

$Q_{tip} = 150_{tsf} \times \overset{\text{Area}}{2.25 sf} = 337.5$ tons

Q_{tip} – max. allowable w/ FS = 3 => $\frac{337.5}{3} = 112.50$ tons design end bearing capacity

Now for friction, K term value from Nottingham design curve

1 layer of cohesionless soil: To depth of 33 below ground surface, use:

$F_{s_{0-12'}} = \frac{1}{2} K \bar{f}_s A_s$, $\bar{f}_{s_{0-3.66m}} = \frac{16.5}{18} = .92$ tsf w/ max. of 1 tsf for any individual f_s value

$F_{s_{0-12'}} = .5(.44)(.92)(4 \times 1.5 \times 12) = 14.57$ tons

$F_{s_{12-30.02'}} = K \bar{f}_s A_s$, $\bar{f}_{s_{12-30.02'}} = \frac{21}{28} = 0.75$ tsf

$F_{s_{12-30.02'}} = (.44)(.75)(4 \times 1.5 \times 18.02) = 35.68$ tons

Tot. F_s = 14.57 + 35.68 = 50.25 tons

F_s - max. allowable w/ FS = 2 ⇒ $\frac{50.25}{2}$ = 25.13 tons

Total Ult. Pile Capacity from Hand Calc. = $Q_{tip} + F_s$ = 337.5 + 50.25 = 387.75 tons

" " " " " MCPTUFR = 386.5 tons

By Hand Calc. of Design Capacity = $\frac{Q_{tip}}{3} + \frac{F_s}{2}$ = 112.5 + 25.13 = 137.63

MCPTUFR " " " " = 137.0

Design Pile Capacity, % difference from average, comparing by Hand and MCPTUFR calculations $= \dfrac{137.63 - 137.0}{\frac{137.63 + 137.0}{2}} = \dfrac{0.63}{137.315} \times 100 = 0.46\%$

Ult. Pile Cap., % difference from average, comparing by Hand and MCPTUFR calculations $= \dfrac{387.75 - 386.5}{\frac{387.75 + 386.5}{2}} = \dfrac{1.25}{387.125} \times 100\% = 0.32\%$

Differences between Hand and MCPTUFR Pile Capacity Results are negligible.

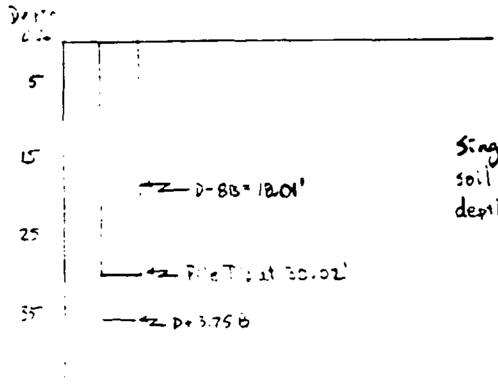
PORT ORANGE BENT 2

PREDICTED PILE CAPACITY
FROM ECPT

WMC 1/2

D-8B = 9.15-3.66 = 5.49 m
= 18.01'

D+3.75B = 9.15+1.71 = 10.86 m
= 35.63'



Single layer of cohesionless soil from ground surface to depth of 71.93'

END BEARING CALCULATION

x	xB (m)	D+xB (m)	q_c, calc.	q_{c1} (tsf)
0.7	0.32	9.47	1/4 [2(18.95) + 2(21.43)] = 20.19 MPa × 10.442 tsf / 1 MPa	210.82
1.0	0.46	9.61	1/6 [2(18.95) + 21.43 + 3(20.17)] = 19.97 MPa	208.53
1.5	0.69	9.84	1/8 [2(18.95) + 21.43 + 3(20.17) + 2(~~30.35~~ 28.73)] = 22.69 MPa (max individual q_c value)	Greater than Above
2.0	0.91	10.06	1/10 [2(18.95) + 21.43 + 3(20.17) + 28.73 + 3(21.97)] = 21.45 MPa (148.57 for 7 values)	" " "
2.5	1.14	10.29	1/12 [148.57 + 21.97(1) + 21.59(4)] = 21.41 MPa	" " "
3.0	1.37	10.52	1/14 [148.57 + 21.97 + 21.59 + 21.42(5)] = 21.37 MPa	" " "
3.5	1.60	10.75	1/16 [18.95 + 21.43 + 20.17 + 28.73 + 21.97 + 21.59 + 21.42 + 16.20(9)] = 18.75 × 10.442 =	195.79
3.75	1.71	10.86	same as above	min. q_{c1} value

watch max

min q_{c1} = 195.79 tsf,

for individual values:

No particular q_c value can exceed 300 tsf

Max. allowable Pile tip Cap. = 150 tsf

Now find q_{c2} which affects pile from tip up to a depth of 5.49 m

Find critical q_{c2} value looking 1.5B below tip = 30.02 + 2.25 = 32.27 = 9.835 m

Lowest q_c value between tip and 1.5B below it is the q_c value at the tip = 18.95 MPa

$q_{c2} = \frac{1}{15}\left[18.95(2) + 20.30(1) + 15.75 + 14.33(2)\right] = 16.56$ MPa × 10.442 tsf/MPa = 172.92 tsf

15 values between 5.49 m and 9.15 m

$q_p = \frac{1}{2}(q_{c1} + q_{c2}) = .5\,(195.79 + 172.92) = 184.36$ tsf, remember max allowable tip cap. = 150 tsf

$Q_{tip} = 150.00 \times 1.5(1.5) = 337.5$ tons

Q_{tip} - max allowable w/ FS = 3 ⇒ $\frac{337.5}{3} = \underline{112.50}$ tons design end bearing capacity

Now for friction, K and ø' values from Nottingham / Tomlinson

none necessary here

1 Layer cohesionless material affecting pile: To depth of 8B, use ($8B = 12' = 3.66$ m), $D/B = \frac{30.02}{1.5} = 20.01 \Rightarrow K = .86$

$F_{s_{0-12'}} = \frac{1}{2} K \bar{f}_s A_s$, $\bar{f}_{s_{0-3.50m}} = 468/14 = 33.43$ kPa × $\frac{0.010442\,tsf}{1\,kPa} = 0.35$ tsf

$F_{s_{0-12'}} = \frac{1}{2}(.86)(.35)(4 \times 1.5 \times 12) = 10.84$ tons

$\bar{f}_{s_{3.5-9.25m}} = 1149/23 = 49.96$ kPa × $\frac{0.010442\,tsf}{1\,kPa} = 0.52$ tsf

incl 9.25 m value

$F_{s_{12'-30.02}} = (0.86)(.52)(4 \times 1.5 \times 18.02)$

$= \underline{48.35}$ tons

Tot F_s = 10.54 + 50.21 = 59.19 tons

F_s - max allowable w/ F.S. = 2 ⇒ $\frac{59.19}{2} = 29.60$ tons

Total Ult. Pile Cap. = $Q_{tip} + F_s$ = 337.5 + 59.19 = 396.69 tons, PLAID predicted 397.50

By Hand Design Cap. = $\frac{Q_{tip}}{3} + \frac{F_s}{2}$ = 112.50 + 29.60 = 142.10 tons

Program Design Cap. from PLAID = 112.50 + 30.00 = 142.50 tons from PLAID Program

Design Capacity, % difference from avg. Comparing By Hand & PLAID calc. - $\frac{142.5 - 142.1}{\frac{142.10 + 142.50}{2}} = \frac{0.4}{142.3} \times 100\% = 0.28\%$

Ult. Cap., % difference from avg. Comparing By Hand & PLAID calc. = $\frac{397.5 - 396.69}{\frac{396.69 + 397.5}{2}} = \frac{0.81}{397.10} \times 100\% = 0.20\%$

Differences between Hand & PLAID Pile Capacity Results are negligible.

BACKCALCULATIONS OF YOUNG'S MODULUS (E)
AT PORT ORANGE AND APALACHICOLA SITES

	PILE LENGTH BELOW SURF (L)	PILE TIP AREA (A)	LOAD (P)	STRAIN (Δ)	$\frac{PL}{\Delta A} = E$
Apalachicola River Pier 3 Site 1	90.6' = 587.2"	462.24 in²	479 tons	(.917 - .35) = .567"	3,973,968 psi
Apalachicola River FSB 16 Site 2	61.0' = 732"	324	165	(.49 - .3) = .19"	3,923,977
Apalachicola Bay FSB 22 Site 3	64.02' = 768.24"	324	213	(.568 - .30) = .268"	3,769,005
Port Orange Bent 19 Site 13	30.88' = 370.56"	324	101.5	(.35 - .3) = .05"	4,643,437
Port Orange Bent 2 Site 14	30.02' = 360.24"	324	139	(.39 - .3) = .09"	3,434,387

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