PIER DEVELOPMENT

ST. PETERSBURG, FLORIDA

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November 10, 1969
File No. 69-716A

TO: Harvard & Jolly
274 Ninth Street, North
St. Petersburg, Florida

SUBJECT: Subsurface Soils Investigation, Analysis and Recommendations, Pier Development, St. Petersburg, Florida.

Gentlemen:

As requested, a subsurface soils investigation was conducted at the subject location to determine the condition and description of these soils. This report includes the results of this investigation, our analysis of the soil conditions, and a recommendation for the foundation system best suited to the proposed structure and these soils. The proposed structure will be supported on four columns, each carrying a load of 3,550 kips. Due to the nature of the project to be constructed on top of the existing pier, ten feet of water and additional 20 feet of loose to very loose sand silt, the only feasible foundation support is piling. High capacity piles can be obtained by driving these into the underlying limerock found 60 feet below the water line. In our study we recommend 100 ton piles to be seated into the limerock.

Two associated major challenges are transporting heavy pile driving equipment to the project site on top of the pier, and the other, the construction of a coffer dam for a tremie concrete pour of the pile cap under water in what we consider tight and cramped conditions.

The effects of driving new piles on the existing ones, and also a re-evaluation of the capacity of the piles in place are discussed in our report.

SCOPE

The scope and extent of this investigation are summarized as follows:

1. Making four structural borings, one at each level of the proposed structures to determine the nature and condition of the foundation soils and geology.
2. Securing undisturbed samples of marginal soils for laboratory tests to determine physical constants of the soil for strength, friction and settlement characteristics.

3. Reviewing structural requirements for wind loads, existing conditions of the pier and recommendations for design and construction.

4. Preparing an engineering report including all laboratory and field data, and recommendations for the most suitable foundation as a guide to the design engineer and architect in the preparation of plans and specifications.

GEOLOGY

General Florida Geology

The Florida peninsula is a southern extension of the North American Continent and separates the Atlantic Ocean from the Gulf of Mexico. The sedimentary section of peninsular Florida is fragmental and marine limestone, sandstone, and shale which reach a known cumulative thickness of more than 18,000 feet. Only a few deep wells in Florida have penetrated crystalline rocks such as granite and hornblende diorite. These rocks are believed to be either Pre-Cambrian or Paleozoic intrusives. The core of the Florida Plateau is Pre-Cambrian.

That portion of the Continent called the Continental Shelf is covered by marine waters which extend from the Florida shoreline to a depth of 300 feet. Florida is a part of the Shelf which alternately rose above and fell below the sea during the Pleistocene epoch. Numerous deltaic deposits were formed during the several stages of the Pleistocene epoch when the level of the sea fluctuated as the glacial ice of the high latitudes developed and subsequently melted. This created a cycle of extreme excavation and sub-aerial erosion followed by a period of intensive alluviation and marine deposition.

The periods when the Shelf was covered with marine waters were of sufficient duration to form seaward-facing escarpments (vertical cuts in the bedrock), broad plains and, most recently, beach ridges. From the early to late stages of the Pleistocene epoch, land was added to the Florida peninsula as each cycle ended. It is known that Florida has experienced five of these cycles from the sediment and terraced surfaces that were formed. A terrace is defined as a flat surface either deposited upon or cut into bedrock and limited by definite escarpments. Erosion and development of the slopes have destroyed many of these escarpments.

The Tampa limestone underlies all of Pinellas County, is encountered at a depth of 70 feet at the Pier Development Project. The surface cover is Pleistocene and Miocene series, consisting of Hawthorn Formation composed of marine sands, clays, marls, all of which are phosphoritic. The underlying Tampa limestone is heterogeneous having many cavities, discontinuities and vertical chimneys. Some of these cavities are filled with silt and other matter, such as muck.

SITE CONDITIONS

The proposed building is to be constructed on the municipal pier located at the foot of 2nd Avenue North in St. Petersburg, Florida. This pier extends 1,825 feet into Tampa Bay and originally supported a casino which has been demolished. The original pier was built in 1925 and is supported on 16 inch x 16 inch precast concrete piles. The length and configuration of these piles are shown in Plate I. These piles were repaired and the surfaces gunited in 1963. The pier deck is reinforced concrete slab and the traffic areas are paved with asphalt. Tampa Bay is approximately 11 feet deep at this location.

FIELD INVESTIGATION

Four test holes were drilled at the locations shown in Figure 1. The borings were drilled with standard penetration equipment (ASTM Designation: D-1586-58T). A two foot long, two inch O.D. split spoon soil sampler was driven into the ground by successive blows of a 140 pound hammer dropping 30 inches. This dynamic method of testing provides information for a conservative evaluation of the bearing capacity of sandy soils and has been widely accepted by foundation design engineers and architects.

A continuous drilling procedure was used. The soil sampler was driven into the ground two feet at a time, then extracted for visual examination and classification of the retained soil sample. The number of blows required for penetration and the texture of the soil were recorded for each foot of drilled depth. Representative samples were taken at every five feet of drilled depth and at each change in soil strata and brought to the laboratory in air-tight jars for further evaluation and testing, if necessary.

Each hole was cased to a depth indicated on Figure 1 to form a rigid entry for the drilling tools and prevent accidental collapse of the sides at the mouth of the test hole. For the rest of the drilled depth, a heavy driller's mud was used to float out the cuttings and hold the fine grains in suspension. In this manner an open wet hole was maintained for accurate, representative sampling.

In addition to the representative split spoon samples, two undisturbed samples of the silty clay layers were secured at a depth of 55' and 60' below grade in Test Holes #2 and #3. The samples were obtained by forcing a 2.37 inch I.D. 16 gauge brass piston sampler into the soil in one continuous, uniform motion with a single hydraulic stroke. After each sample was retrieved, the ends were sealed with paraffin in the field and transported to the laboratory for density, moisture content, void ratio, and specific gravity determinations. In addition to these physical tests of the in-place soil, we made other tests to obtain grain size distribution, compressive strengths and friction values.

SOIL CONDITIONS

The complete soil profiles, showing all pertinent information as to the condition of the subsurface soils, are shown in geological sections A-A & B-B on Figure 1. In general the soil profile, as interpreted from the standard penetration borings, was found to be uniform in texture and stratification, which can be divided and classified as follows:

Depth from Pier Floor - Feet		
From	To	Description
0	9	Height of pier floor above water level.
9	20	Depth of water.
20	21	Dark gray silty sand - very loose.
21	35	Gray to light gray silty sand with brown shell - very loose.
35	51	Gray to brown, slightly silty very fine sand with shell and phosphates - very loose to medium compact.
51	70	Gray silty clay with some consolidation - soft to hard.
70	92+	Light brown decomposed consolidated limerock - dense to very dense.

Test Hole #2 did contain a seam of gray and light gray consolidated calcareous clay with seams of light gray limerock approximately six feet thick. Since this deposit is only located in this hole, it was not separated in the soil profile description above, and as it is very hard, it can be treated as part of the limerock stratum. Also, in Test Hole #3, a two-foot layer of gray plastic silty clay was located seven feet into the limerock. This was the only inconsistency in the continuity of the limerock strata encountered in all of the test borings.

Tides at St. Petersburg vary from minus 1.0 feet to 3.1 feet from datum. Datum for St. Petersburg is 1.2 feet below mean sea level, as established by the U. S. Department of Commerce, Environmental Science Services Administration.

LABORATORY INVESTIGATION

Two undisturbed samples were taken in Stratum #5 as shown in Figure 1. Stratum #5 is a consolidated firm gray silty clay. This is a layer where most of the original piles were believed to be driven into. The prime purpose was to determine the in-place densities, strength and cohesion to evaluate and estimate the capacity of the 16 inch by 16 inch precast piles presently existing to hold the bridge deck. This stratum is a layered clay where its plasticity as well as its in-place density varies considerably. For example, the two tests performed for Atterberg limits indicate that the liquid limit of this soil ranges from 88 to 130, where the plasticity index varies from 36 to 53. The in-place densities are 56 to 66 pcf. The unconfined compressive strength of the undisturbed material was 1.1 tsf. After testing the undisturbed soil, it was remolded to approximately the same density and retested for the unconfined compressive strength. The ratio of the compressive strength of the soil as is in-place, to the compressive strength of the remolded soil at the same density, is designated as sensitivity. For non-cohesive soils, this value is close to one, and for clay soils it varies from two to more than twenty. Clays that are described as "quick clays", this ratio may be higher than 50. For Stratum #5, we found that the sensitivity is approximately 3.8.

The cohesion, which in clay soils determines the ability of the soil to provide friction to the pile, is approximately half of the unconfined compressive strength. The cohesion of Stratum #5 is approximately 0.67 tsf. This and other soil data were used in the determination of the carrying capacity of the existing piles. Plates II and III graphically represent the laboratory test results of the unconfined compressive tests on the undisturbed material from Stratum #5 and also of the remolded sample.

ANALYSIS AND RECOMMENDATIONS

The proposed structure is a five-story inverted obelisk building, supported on four columns. The columns will be reinforced concrete and the building of steel and fabricated metal siding. According to Mr. E. Reiss, Structural Engineer, the building loads including pile caps is 3,550 kips per column. The intent is to use point bearing H-beam piling. The piles are to be imbedded into the cap to be considered fixed.

This project is unique in that it is to be built on an existing pier which previously held a structure. This building has been razed, but its pile foundation is still in place and is not to be utilized in this new building. The new single story outbuildings in this development will be supported on the existing piles. The spacing of these piles and that all foundation construction must take place through the pier floor, economically suggests the smallest foundation plan be utilized. Due to the looseness of the upper soil stratum, a shallow foundation to support this building would require a complete mat. Even if a complete mat is constructed to support the building to dredge line, considerable settlement will be experienced if the silty sand is not artificially densified. In addition to this consideration, more work will be necessary for the con-

struction of a mat foundation, such as the removal of the 3,600 square feet of pier deck, the construction of an interlocking sheet pile coffer dam, maintenance of pumping system for water removal, removal of existing piles above working grade, replacing the pier floor, plus the cost of a mat foundation, indicate the choice of a pile foundation to be the most economical.

Wind Loading:

In the design of the pile foundation and the caps, it is necessary to include the additional loads that are caused by wind forces as horizontal shear and overturning moment. We presumed that the exposed building area will be subjected to a wind pressure of 45 psf. Since the structure is in open water, a shape factor of 1.4 was used to increase the wind pressures exerted on the structure. Estimated total wind force is 367 kips which creates an overturning moment at the mud line of approximately 8,900 ft. kips. This moment will increase the axial load of 3,550 kips by 275 kips, or a total load of 3,820 kips per each of four legs. In our recommendations of the pile foundations, we have assumed that these four columns are rigidly attached to the superstructure and there is sufficient cross bracing or lateral rigidity that the entire structure will act as a monolithic body without causing undue bending in these four pedestals supporting the structure.

New Pile Foundation:

Due to the relatively close quarters that these foundations need to be installed both for economic reasons of construction and also for cutting holes in the existing deck, we suggest point bearing piles with a design capacity of 100 tons each. It appears that each pile cap will require 20 piles driven to the underlying Tampa limestone. It will be necessary to rotate half of these "H" piles in respect to their axis 90 degrees so that an average section modulus can be attained for the entire group. We recommend that a fourteen inch CBP 145 "H" pile weighing 73 pounds per running foot be used for the foundations.

The choice of the kind of piles to be used also becomes a critical item due to the relatively small foundations necessary to support this superstructure. The low displacement "H" piles are recommended due to the minimum amount of soil disturbance or lateral displacements so that they may not upset the stability of the nearby existing piles. If other types of piles are used, their displacement will cause a momentary loss of support for the existing piles by either lateral movement or heave. This is one of the major reasons why steel "H" piles should be used for this project.

Discussion of Pile Foundation:

The piles in this project will dissipate the building load, both in skin friction and end bearing. The penetration resistance of the split spoon sampler in

the test borings can be related to skin friction loading available for a pile driven in this soil. Due to the variations in soil density and type, the frictional resistance will vary with each stratum. In the loose light gray silty sands, Stratum #3, very little load dissipation can be expected due to the continued low hammer count per foot. The underlying more dense, slightly silty sands and harder clays, Strata #4 and #5, will provide most of the frictional load support. The first twenty feet of sands will absorb about fifteen kips load. The next stratum will provide about 600 psf (25 kips in Stratum #4) of frictional resistance and the clays approximately 2,000 psf (140 kips in Stratum #5). Within the soils above the limerock, approximately 90 tons will be absorbed due to friction. The remaining portion of the building load will have to be absorbed by the limerock providing point bearing for the pile. Therefore, to dissipate building loads, we recommend a twenty-pile configuration with each pile having a 100 ton capacity. This value is well within the recommended capacity of a 14 inch CBP-145 , 73 pound "H" beam pile having a unit stress of 12,000 psi.

Figure 2 illustrates the location of existing piles and proposed piles. Also, we have shown in profile the relative depths of existing piles and proposed piles. However, these positions have been determined predominantly from drawings available and there may be slight variations. Also, we were interested in determining the proximity of proposed piles to existing piles, so that the sensitivity of the clay (Stratum #5) as determined in the laboratory, could be evaluated. In Plates II and III curves show the results of unconfined compressive strength tests performed on this clay. Sensitivity was considered since the existing piles terminate in a clay stratum. If this clay had sensitivity of ten or more, the activity of driving new piles adjacent to existing piles could cause this clay to assume a quick condition and result in settlement of in-place piles. The more piles that are driven into a sensitive clay, the wider the area disturbed and the possibility of affecting existing piles increased. The sensitivity of the clay stratum was calculated to be 3.76, which is considered stable, so that a new pile being driven adjacent to an existing pile should not disturb that pile appreciably. However, new piles should be at least two feet from existing piles due to the inherent nature of a clay to form a column about a pile driven into it and become a part of the pile. This build up of soil which adheres to the pile is tear drop in conformation, extending farther from the pile with depth. If a new pile were to be driven through this soil which formed about an existing pile, that pile could be adversely affected.

Another advantage of the "H" beam pile in this situation is that it displaced the least soil for an equal capacity pile of other designs, and therefore causes less lateral movement of soils.

Lateral Capacity of Pile Group:

The piles driven into the soils also have a certain amount of lateral resistance to horizontal loads applied to them such as wind loading in this instance. The modulus of lateral deformation is estimated to be 50 pounds per cubic inch which

vary with depth. A steel "H" pile 14 inches, 73 pounds per lineal foot is capable of resisting a force of 4.4 kips with an over all horizontal deflection of 0.11 inch provided that these piles are presumed to be rigidly fixed in the lower pile cap. The shear to be resisted at each pile cap due to the wind load is estimated to be 87.5 kips. The over all behavior of a group of piles was demonstrated by experiments to equal approximately twice the value of individual piles. Therefore, during hurricane wind loads a total lateral movement of the pile caps of 0.22 inches can be expected. In our opinion, this movement is acceptable and will not damage the structure in any way.

It is important to emphasize that the above numbers assume that the superstructure is completely rigid and the four column legs are not allowed to bend or act as cantilevers that are installed on piles. If sufficient bracing is not provided in the structure, the total movement of the pile cap, coupled with the wind moment will cause the deflections to be much greater. Therefore, it is necessary to provide sufficient cross bracing of the structure to provide a rigid frame to keep bending of piers to a minimum. The other solution will, of course, be to provide batter piles in each of the legs.

The pile group will act as if they are fixed rigidly at 8.1 feet below mud line where the maximum moment occurs due to the horizontal forces. We feel that an additional moment of 200 ft. kips can be absorbed by each pile cap where horizontal deflection will be within 0.25 inches.

Pile Load Test:

Of course, the only true indication of the pile capacity is to subject it to a load test, once it has been driven to its apparent design depth. In the appendix we are including suggested specifications for the pile foundation and a test procedure. We have found in most previous pile specifications considerable difficulty when these conditions became debatable or understandings not clearly spelled out. In these specifications, we have also included a test procedure. The duration of the test is dependent on pile movement rather than an arbitrary time delay between load increments. Experience has shown that this method of applying loads in no way impugns the integrity of the pile, but it does reduce the expense of testing the pile.

In Plate III we have shown the characteristics of the clay within this foundation system. This diagram illustrates the cohesion we can expect from this deposit and indicates that six reaction piles will be required to dissipate the 200 ton test load on the test pile. Should any of the reaction piles show an upward movement, they shall be reseated with the driving hammer after the tests are concluded. Since the test pile will not be laterally supported during the test for a clear height of 20 feet or more, there exists the possibility of buckling. To reduce the possibility of this occurrence, section of "II" beams should be welded along the free standing sides of the test pile five feet below the mud line to act as stiffeners.

Evaluation of Load Carrying Capacity of Existing Pier:

We felt it is important to evaluate the existing pile layout and their carrying capacity for construction purposes where heavy equipment and single story light shops will be placed on them. The existing piles are 16 inches by 16 inches precast concrete units, the lengths of which vary from 53.5 to 57.0 feet as shown in Plate I and Figure 2. From the available soil borings and laboratory tests, we have estimated that the carrying capacity of these piles will be 46 and 50 tons, respectively. The existing pile capacity, without evaluating the strength of the concrete deck, is estimated to be 2,000 psf when it is uniformly loaded. Therefore, we feel some care must be utilized in placement of equipment, stock piling of construction material and other concentrated loads to insure that these piles are not overloaded or the deck slab is not fractured or cracked.

Some of our thoughts are that the "H" beams for piles should not be stacked too high where there is other traffic such as concrete trucks, cranes, etc. We have estimated probably the concrete trucks will have a maximum load of around 27 to 30 tons. Such loads can certainly be carried on the deck provided that the vicinity of the area is not loaded to capacity.

The portion of the deck removed for the installation of cofferdam and "H" piles can be supported on the new pier after it is constructed. A simple sliding joint should be provided to carry the deck loads on the piers so that there will be no horizontal load transfer. The temporary support of the portions of the deck that will be affected by the removal of sections for the piers should be left up to the contractor, but he should be required to submit a detailed procedure of his method to the Architect in advance of construction for his approval.

Cofferdam for Construction of Pile Caps:

We feel the most economical means of constructing the pile cap and the pier is to provide a temporary cofferdam constructed from steel "Z" sections and driven to a depth eight feet below the bottom of the mud line. The procedure recommended below eliminates any additional bracing or stiffening of the cofferdam and also minimizes any work that has to be conducted by divers under water. The following procedure is suggested as a guide to the design engineer.

a. After cutting the portion of the deck to accommodate a 16 foot by 16 foot clear pile cap, the "Z" interlocking steel sections should be driven to eight feet below the mud line. This depth is required to provide sufficient toe resistance to the piles for lateral stability.

b. After each of the four cofferdams are constructed, the bottom should be excavated to three feet below the mud line to provide the necessary cover for the main "H" steel piles for corrosion and scour protection. This excavation can be done by "air-lift" process, and the bottom can be inspected by an Engineer-Diver for compliance.

c. The "H" piles can be driven as specified and the load test performed. The number of piles that need to extend to the top of the cap is only necessary for proper anchorage and dowelling of the superstructure for transfer of horizontal and vertical loads. They can be cut below the pile cap and reinforcing steel can be provided for the structure.

d. The concrete for the cap should be a minimum of nine bags of cement per cubic yard of pump-mix concrete. The tremie should be placed in four equal lifts, providing sufficient time for initial setting and hardening of the concrete between each lift to minimize lateral loads on the sheet pile. Plate IV is prepared to show graphically the suggested method. An alternate method of placing concrete is shown in appendix.

e. All existing piles within the cofferdam areas should be extracted prior to the construction of the cofferdam. It will probably be necessary to jet along the existing piles to loosen the soil for easy removal.

It has been a pleasure to assist you with this project, and if you have questions, please let us know.

Very truly yours,

ARDAMAN & ASSOCIATES, INC.

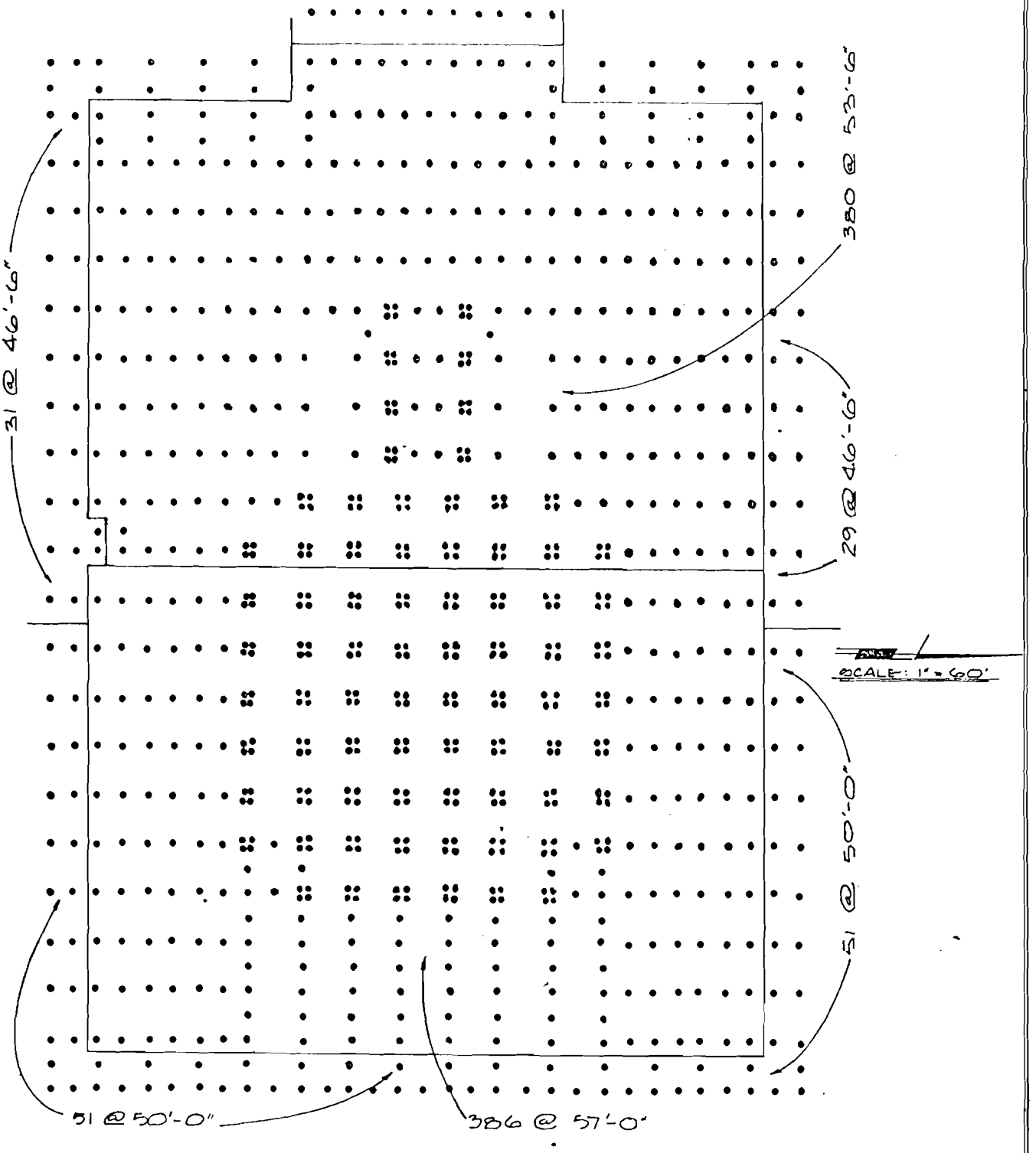
Donald A. Killiany

Donald A. Killiany
Associate Engineer

M. E. Ardaman

M. E. Ardaman, Ph.D., CEC
Engineering Registration No. 5200
State of Florida

DAK:MEA:ja
Enclosures



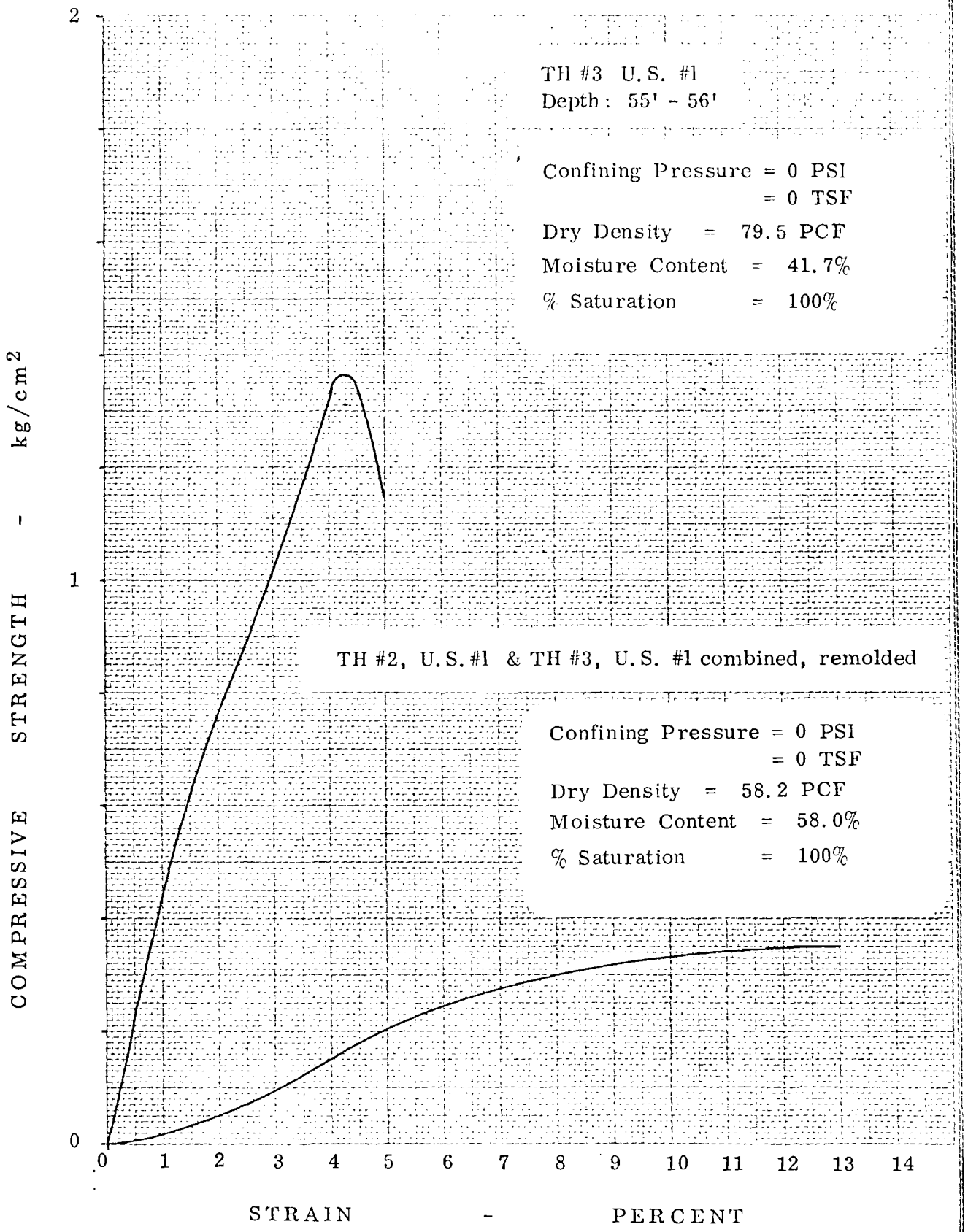
EXISTING PILE LENGTHS AND CONFIGURATIONS
16" x 16" PRECAST RAYMOND
CONCRETE PILES

MUNICIPAL PIER, ST. PETERSBURG, FLORIDA.

File No. 69-716A

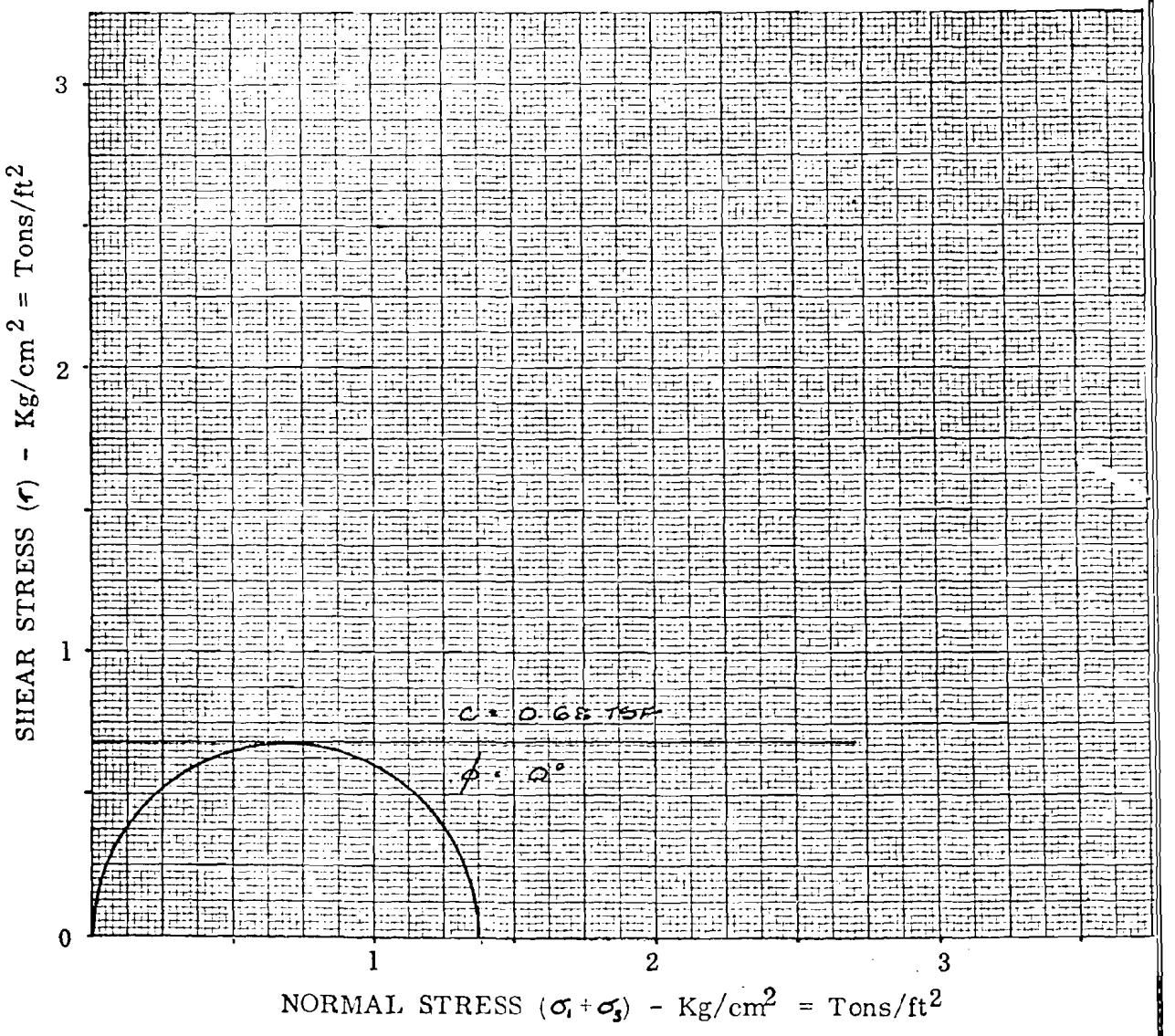
PLATE I

ARDAMAN & ASSOCIATES



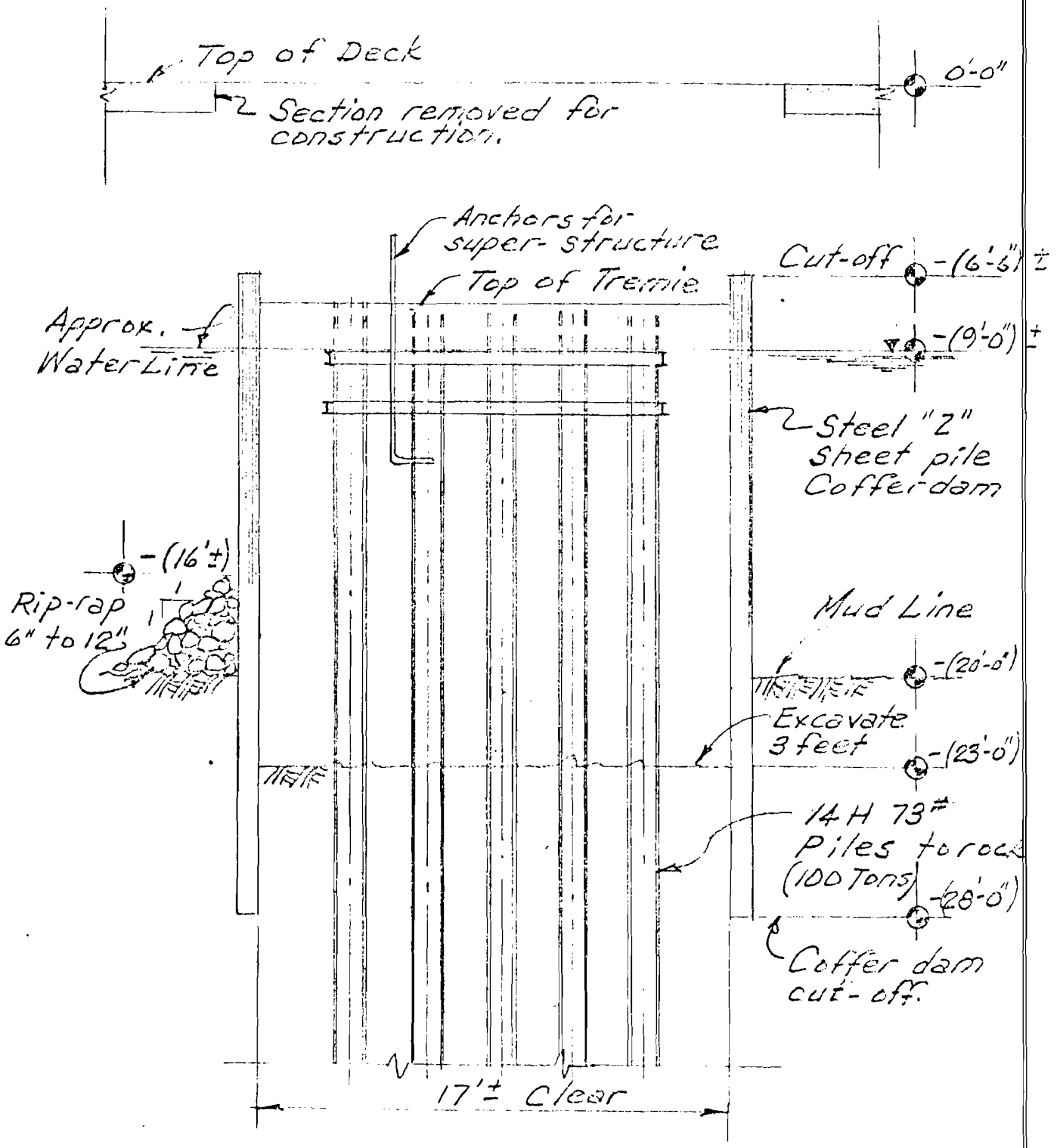
MUNICIPAL PIER
ST. PETERSBURG, FLORIDA

LOCATION: TH #3 U.S. #1
DESCRIPTION: Greenish-gray clay of high elasticity
DEPTH: 55 to 56 feet
L.L.: 88
P.I.: 36



MOHR CIRCLE: STRENGTH RUPTURE ENVELOPE TO DETERMINE THE COHESION (c) AND ANGLE OF INTERNAL FRICTION (ϕ) VALUES FROM UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TESTS

MUNICIPAL PIER
ST. PETERSBURG, FLORIDA



SUGGESTED DETAIL OF COFFERDAM

PIER DEVELOPMENT

ST. PETERSBURG, FLORIDA

69-716A

PLATE IV

ARDAMAN & ASSOCIATES

APPENDIX

PILE SPECIFICATIONS

1. SCOPE

A. Furnish all labor, material, equipment, tools, transportation, necessary supplies, and other costs required to install all steel H-beam piles for the project, as shown on the drawings and in accordance with these specifications, and the terms of the contract.

B. All excavation, including excavation to required levels below pile cut-offs, dewatering and maintenance of embankments will be performed by others. The piling Contractor shall establish and locate all lines and levels and be responsible for the correct location of all piles.

C. All work shall conform with the requirements of the Southern Standard Building Code and the St. Petersburg Building Department, in addition to the requirements or qualifications of the specifications and drawings.

2. SUBSURFACE INFORMATION

A. Any data concerning subsurface soil conditions is provided for the Contractor's information only. If the Contractor desires to obtain additional site information at his own expense, he may make borings or soundings, drive test piles or dig test pits on the site of the work. Permission for admission to the property must first be obtained from the owner.

B. Drawings showing the logs of soil borings made on the site are available for examination by the bidders, but are not to be made as part of the contract. The bidders should ascertain for themselves the soil conditions underlying the surface of the ground. The borings logs are the best information concerning the subsurface conditions in the possession of the architect or owner, but the architect or owner has no responsibility for the correctness of the boring information and makes no representation regarding subsurface conditions.

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C. The attached drawings show the soil profiles and the top of underlying limerock. It is estimated that the piles will develop the necessary driving resistance before they reach the surface of the limerock. These Plates are prepared as a guide to the Pile Contractor in estimating lengths of the piles and showing the nature of geologic formations.

3. GENERAL

A. Before beginning work, the Contractor shall submit details of the pile he proposes to furnish and a discription of the proposed method of installation, all of which shall be subject to the approval of the Engineers. All piles shall be installed by a piling contractor qualified to install the type of pile specified in accordance with the plans and specifications and under the subsurface conditions existing at the site. The pile contractor shall have at least five years experience installing this type of pile and shall present evidence of satisfactory performance on jobs of this type and magnitude and shall present evidence of sufficient financial strength and stability to properly equip and execute this work.

4. EQUIPMENT

A. All piles shall be driven with a power hammer delivering not less than 24,000 foot pounds of energy per blow. The technical specifications of the hammer proposed to be used shall be submitted to the Architect for approval ten days before bid opening in writing. Any interpretation of pile hammers that can be used on this project will be made only by addenda duly issued and copies of such changes will be mailed or delivered to each person receiving a set of such documents.

B. Pile drivers leaders shall be the rigid or fixed type to insure support to the pile during driving. The use of swinging leads will not be permitted.

C. Cushions or capblocks shall consist of one solid block of hardwood, or such cushion as will transmit an equivalent or greater dynamic energy to the head of the pile of proper shape and dimensions to fit the hammer. The grain of the block shall be parallel to the axis of the pile. If laminated materials are used, the strength of such materials shall be equal

to or greater than hardwood. The use of wood chips, rope, small wood blocks, shavings, or any similar materials, shall not be permitted. The continuous or frequent introduction of materials to cushion the hammer blows will be prohibited. Details of capblock arrangement shall be submitted to the Engineer for approval before piles are driven.

5. DRIVING

A. All piles shall be driven by such methods and equipment as will not impair their strength, and as will insure the pile shaft retaining the initial driving resistance and lateral support of the soil.

B. Piles shall be driven to a final resistance of at least 100 tons as determined by the Engineering News Formula.

C. Piles shall be driven with a variation from the vertical of not more than 1/4" per foot of pile length, and a tolerance of three inches from the designed location shall be permitted.

D. Piles damaged or improperly driven due to inadequate equipment or material or unsatisfactory methods shall be replaced at the expense of the pile contractor.

E. There shall be no additional payment for additional work for piling resulting from obstructions of any kind, encountered in the soil during installation of piles. This shall apply to damaged piles, footings, the placement of damaged piles, removal of obstructions by excavation or other means, backfilling such excavations, enlarging or changing the shape of pile caps, relocating piles and all other modifications and additional construction resulting from obstructions. This shall apply to obstructions which occur at any level whether above or below the top of the bearing stratum. Abandoned piles under installations shall be cut off six inches below the bottom of pile caps.

F. If any pile is damaged, mis-located, out of plumb more than two percent of its length, or if its center at the level of the cut off is out of position such a distance that any pile in the group is loaded more than ten percent greater than its design load, the contractor, at his own expense, shall provide and install additional piles as directed so that no pile in the completed group will be loaded more than 10 percent greater than its design load. The cost of checking and redesign, by the designing engineer, at the rate of three times the actual drafting and design cost, shall be borne by the contractor.

6. TEST PILES

A. One test pile to be loaded to twice the design capacity of 100 tons shall be installed. Location of test piles shall be determined by the Engineer. The contractor shall notify the Supervising Engineer or his representative, prior to the installation of the test piles. All information, such as mandrel weight, type of hammer, continuous number of blows, etc., shall be kept and be part of the testing schedule.

B. The contractor shall submit a procedure for testing his piles. This should be approved by the Architect or his representative.

C. The load shall be applied with a claibrated hydraulic jack, equipped with a pressure gauge. Settlement measurement versus load shall be taken for at least two test piles. Additional load tests may be required if the need is shown by the Supervising Engineer. Loading tests shall be performed at least three (3) days after the driving of the piles. The piling contractor shall furnish a hydraulic jack of 200 ton capacity, with a suitable dial and an accompanying calibration chart, showing the accuracy of the jack used. The dial shall be accurate to at least 8000 pounds, or four (4) tons.

D. The load shall be applied in increments of 20 tons, up to the design load of 100 tons. The time elapsed between each increment shall not be less than 10 minutes. Continuous settlement readings shall be recorded at least every two minutes and the next increment shall not be applied if the settlement exceeds 0.005 inch during any two minute interval. The load increment over 100 tons shall be applied in increments of ten tons, at intervals at least 20 minutes after all measurable settlement (less than 0.005 inch) has ceased, due to the previous load or increment. The total load, 200 tons, shall remain in place 24 hours or at least six hours after all measurable settlement has ceased. It shall then be released and the rebound measured after all measurable rebound has taken place. The net settlement, that is, total settlement less rebound, shall not exceed 0.005 inches per ton of test load. All testing shall be supervised by the testing laboratory or the Engineer's representative and the cost of all laboratory supervision shall be paid by the owner. The Engineer may vary the above loading procedure if necessary.

E. Within 48 hours after the test has been completed, the Engineering Laboratory shall submit to the Engineer/Architect a report containing the following information:

(1) A description of soil conditions at the location of the test pile.

(2) A description of the pile and its driving record, including the number of blows per foot throughout the pile length and the final driving resistance in inches per blow for the last ten blows of driving.

(3) A description of the hammer.

(4) A tabulation of loads and settlement readings during the loading and unloading of the test piles.

(5) A graphic representation of the test results in the form of a time-load-settlement curve.

(6) Remarks concerning any unusual occurrences during the testing.

F. Should the results of the tests be unacceptable to the Engineer and/or the local Building Department because of their failure to establish the required pile loading capacity, the contractor shall drive other test piles to a greater resistance than those previously driven and shall repeat the load tests without added cost to the Owner.

7. PAYMENT

A. Payment for work completed in accordance with these specifications shall be made on the basis of the contract lump sum price adjusted from final quantities using the contract unit prices for additions to or deletions from the work.

B. There shall be no payment for damaged piles as defined under Item 5, "DRIVING".

C. The pay length of the pile shall be that length measured from the point to the cut off elevation shown on the plans. No payment will be made for cutting off the piles or for any portion of the pile remaining above the cut off elevation.

D. For contract purposes, the average length of pile shall be assumed to be 75 feet. The lump sum price shall include all labor, materials and equipment to complete the driving of all piles indicated on the drawings and to

the depth indicated, and for two (2) test loads. In the event that final quantities of work completed vary from the quantities indicated on the contract drawings, both with respect to total length of piling, the lump sum price shall be adjusted by the following contract unit prices:

(1) Unit price for one or more additional test loads.

(2) Unit price for each pile of contract length added to the total number of piles indicated on the contract drawings.

(3) Unit price for each pile of contract length deducted from the total number of piles indicated on the contract drawings.

(4) Unit price for each lineal foot of total length of piling added to the total length of piling indicated on the contract drawings.

(5) Unit price for each lineal foot of total length of piling deducted from the total length of piling indicated on the contract drawings.

8. RECORDS, CONTROL, SURVEY

A. Complete driving records of each pile shall be kept by a professional engineer to be retained by the Owner and shall be submitted in quadruplicate to the Architect. The records shall include length, driving resistance for the last three feet of driving, top elevation, results of any redriving, tests or borings, and all other pertinent data relative to the pile driving. The same engineer shall insure and certify that piles are installed in accordance with design, building code, and contract requirements.

B. The contractor shall prepare tracings showing the location of the piles and assign an identification number to each pile, coinciding with the identification number used in driving record of each pile. Prior to driving piles, three prints shall be submitted to the Architect for approval.

C. After pile or piles have been driven, the contractor shall take all adequate precautions and provide all necessary equipment necessary for the protection of adjacent property while driving piles. All work shall be in accordance with local codes and safety regulations. The contractor shall assume full responsibility for repairing or replacing any damage to adjacent property or injury to persons.