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**Title:** Designed, PDA Assisted Load Tested, and Constructed Pile Foundation Provides an Economical Foundation Solution – A Case History

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# DESIGNED, PDA ASSISTED LOAD TESTED, AND CONSTRUCTED PILE FOUNDATION PROVIDES AN ECONOMICAL FOUNDATION SOLUTION – A CASE HISTORY

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A case history of design, prediction, probe pile driving, and construction of a pile foundation for Building 3 of Waterside at Bay Beach Condominiums located in Fort Myers Beach, southwest Florida, is presented. Site geometry constraints, the presence of a 12.0 ft (3.6 m) deep lake occupying the southern one-third portion of the proposed building footprint, and marginal subsoils provided specific challenges for design and construction of the foundation. The subsoil conditions were explored by performing Standard Penetration Test (SPT) borings and a Piezocone Penetration Test (PCPT) Sounding to depths of 100 ft (30.5 m) within the building footprint. Additionally, borings were performed to envelope the building. Deep foundation alternatives included driven concrete, monotube, and steel pipe pile as well as drilled shaft. Following an evaluation of high structural loads, site-specific constraints, foundation test data from buildings 1 and 2, and preference expressed by the project owner, a pre-cast prestressed concrete (PPC) pile-supported foundation system was selected as an economical and effective foundation approach. Use of an augered cast-in-place (ACIP) pile foundation system was precluded by the project structural engineer. This paper deals with the design aspects and methodologies utilized in choosing and economizing the foundation system.

The building foundation was designed for an allowable pile capacity of 40 tons (353 kN) in compression, 1.5 tons (12 kN) in lateral capacity for a deflection of 0.25-in (0.64 cm) and 15 tons (133 kN) in uplift capacity. For the single story extended garage structure, a pile capacity of 27 tons (238 kN) was indicated in compression. As part of the design process, an extensive probe pile driving program was carried out in order to evaluate the pile driving equipment, determine and verify pile capacities, and optimize the production pile lengths. A total of 33, 14-in (356 mm) square, PPC concrete piles were driven within the entire building limits, and dynamically load tested utilizing Pile Driving Analyzer (PDA) as part of the probe pile driving program. Utilization of PDA during probe pile driving helped in predicting pile set-up (i.e., increase in pile resistance with time) as well as reducing the pile lengths by as much as 20 percent. It resulted in variable pile lengths during the production phase. This information was effectively utilized as part of the overall quality control program to install a total of 559 PPC piles (including 33 probe piles) within the Building 3 footprint, including the reclaimed lake area, to depths ranging from 45 to 58 ft (14 to 18 m) below existing ground elevation as part of the foundation support system. Prior to installation of probe or production piles, a termination criterion was established using 1-dimensional wave equation analyses in addition to the design pile embedment requirement. The predicted pile capacities compared reasonably well with the capacities from the CAPWAP analyses for probe piles.

This engineered, monitored and tested foundation solution provided a cost effective and satisfactory basis for completion and approval of the foundation work for the building resulted in cost savings over other foundation alternatives for the owner. The building has since been occupied and a recent reconnaissance has indicated that the building foundations are performing satisfactorily.

## INTRODUCTION

Waterside 3 is a 10-story condominium structure with parking underneath nestled between Ostego Bay and the Gulf of Mexico at the south end of Fort Myers Beach, Lee County, Florida. Two mid-rise condominium structures (Waterside 1 and 2) exist adjacent to the subject structure. The structure consists of a driven pile foundation system, reinforced concrete pile caps, grade beams, concrete columns and shear walls, post-tensioned elevated and ground floor slabs. The geotechnical design for the structure was completed in late 1997 or early 1998. Construction commenced in late 1998 and the building was released for occupancy in late 1999. The project layout plan identifying the building footprint, test-boring locations, as well as the site geometry, are illustrated in Figure 1. A portion of the proposed building footprint was originally occupied by a 12.0 ft (3.6 m) deep lake.

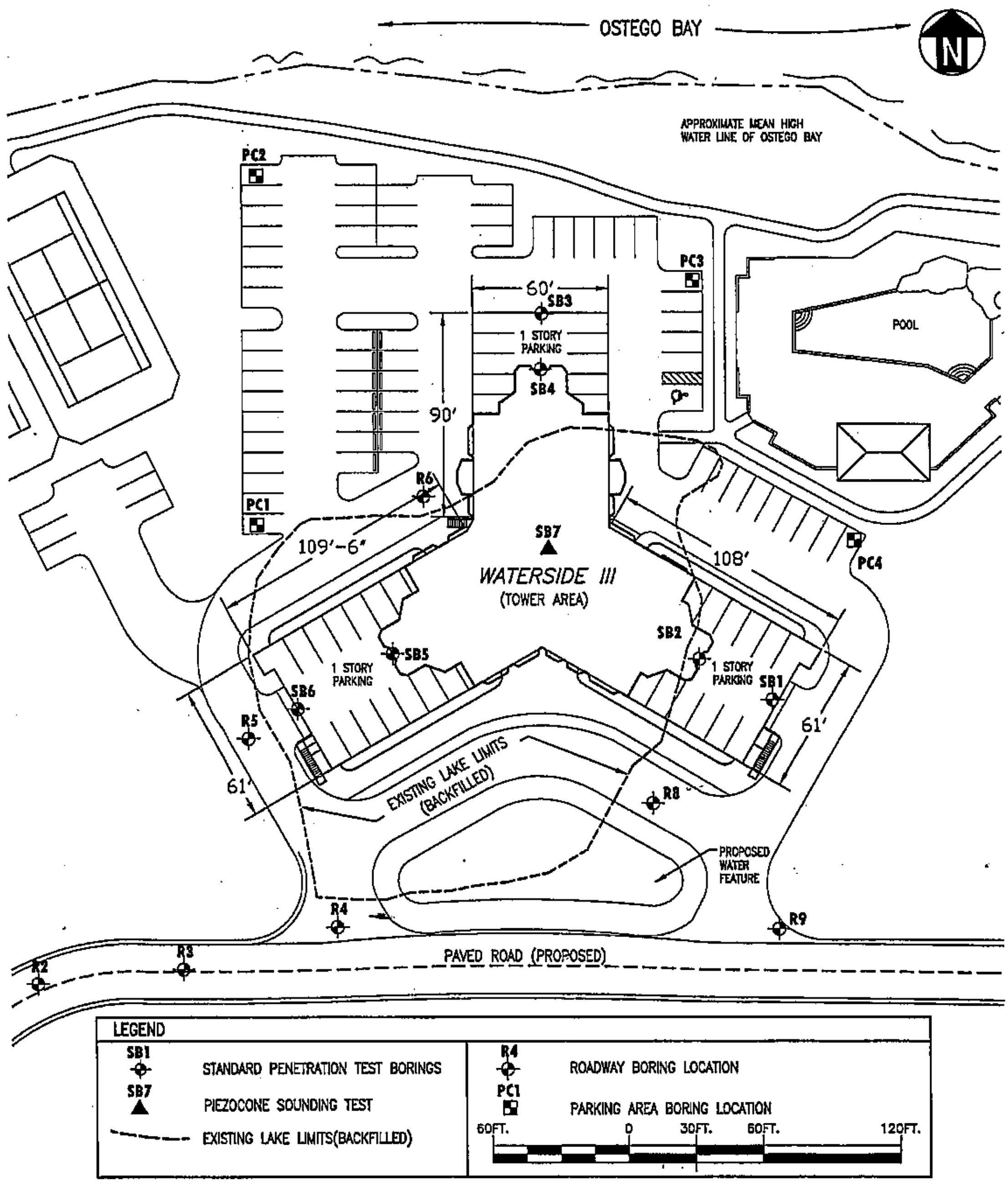



Figure 1.Project and building layout plan with test locations

## SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

Pre-construction project site features included a bay-front tract with generally level topography. A geotechnical program consisting of six (6) Standard Penetration Test (SPT) borings and one (1) Piezocone Penetration Test (PCPT) sounding to a depth of 100 ft (30.5 m) below existing ground surface as well as supplemental borings enveloping the building, was performed. Additionally, previously available subsoil information from nearby buildings 1 and 2 was also included and evaluated.

In general, project site stratigraphy consisted of very loose to medium dense poorly-graded sands (SP) underlain by predominantly marine silty clayey fine sands with intermittent layers of weathered limestone over a depth of 100 ft (30.5 m). Tidal groundwater table was encountered at 6.0 ft (1.8 m) below ground surface. A detailed subsoil profile correlated from SPT borings to depths ranging from 75 to 100 ft (23 to 30.5 m) is illustrated in Figure 2.

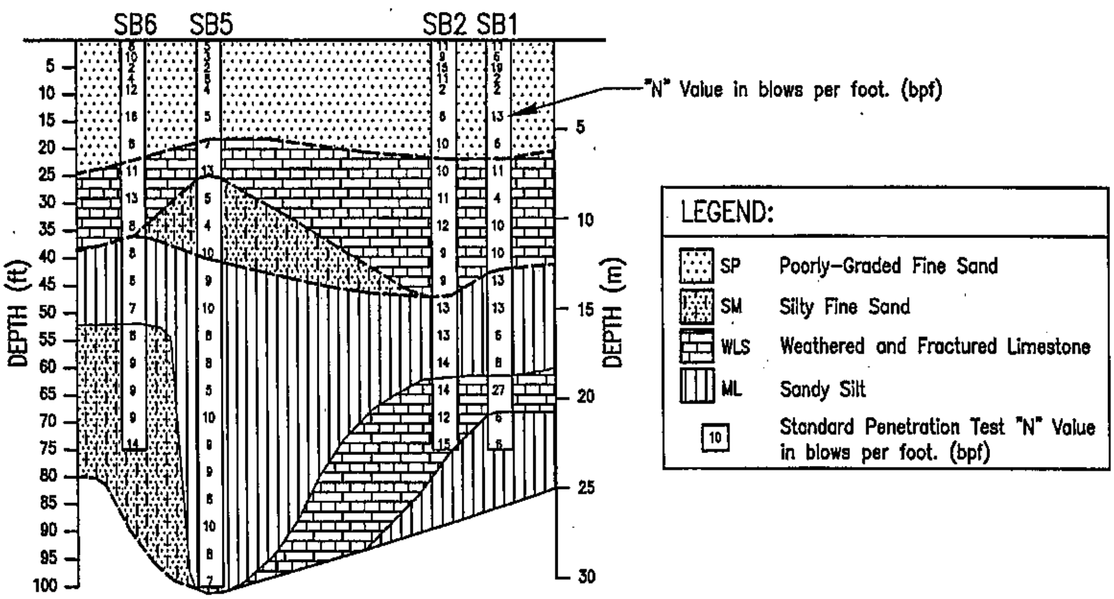



Figure 2. Generalized subsurface profile

## FOUNDATION DESIGN AND ALTERNATIVES

The first step of the design process was to evaluate various foundation systems and associated costs. Several foundation alternatives were considered for the proposed tower structure including: (i) driven precast prestressed concrete (PPC) displacement piling; (ii) drilled shaft; (iii) ground modification using vibro-replacement technique; (iv) monotube with tapered fluted sections; and, (v) steel pipe pile. In view of the owner's accelerated construction schedule and the mandated timeframe, a driven prestressed concrete piling system was finally selected for the tower and underneath parking structure. Piles were designed to penetrate through the weathered limestone layer as composite action piles.

The allowable capacity of piles was estimated using SPT94 software available at the time of completion of foundation design in early 1998. This program, which was developed and used by the Florida Department of Transportation, estimates the axial capacity of a pile based on SPT "N" values. The design method used in SPT94 (and various upgraded versions since then) has been found to be reasonably accurate for driven piles in cohesionless soils (McVay et al). Results indicated that prestressed concrete piles driven into silty sand, clayey sand, weathered limestone and sandy lean clays to depths ranging from 40 to 70 ft (12 to 21 m) below the original ground surface will provide a satisfactory and economical foundation alternative.

Pile capacities of 40 tons (353 kN in compression and 15 tons (135 kN) in tension were estimated for a 60 ft long pile in non-lake area. In the backfilled lake areas the same compression and tension capacities were estimated for a 70 ft long pile. The building contains 1-story parking structures connected to the tower (see Figure 1) which are supported on piles rated for 27 tons (238 kN) in compression.

## PROBE PILE AND DYNAMIC LOAD TESTING (DLT) PROGRAM

As an integral part of the value engineering process and in an effort to optimize penetration length and capacity requirements for piles a DLT program utilizing PDA instrumentation was recommended and performed.

The purpose of the probe pile program for this site was to evaluate the suitability of the contractor's pile driving equipment to provide site-specific pile driving data, determine and verify pile capacities, evaluate set-up for single or cluster piles in a pile cap, provide production pile lengths, and establish driving criteria for use during the production pile driving phase of the project.

### Probe Pile Driving Operations

A total of thirty three 14-in (356 mm) square PPC probe piles were dynamically load tested during the probe pile driving and testing program. The probe piles were driven at production pile locations and incorporated into the foundation system of the tower and parking structure. Variable pile lengths were used depending on whether they were driven in the garage or tower areas and whether those areas were within or outside the lake areas. Additionally, piles were selected in pile caps to evaluate the set-up for completed pile clusters. Various probe piles and DLT locations (such as garage, tower, lake, and non-lake areas) are illustrated in Figure 3.

The pile-driving contractor proposed utilizing an ICE-520S single-acting diesel hammer to install the piles for this project. This hammer has a maximum rated energy of 30,000ft-lbs (4,200 m-kg) and a ram weight of 5,070 lbs (22.5 kN). Prior to initiation of the probe pile driving program, driveability analysis using Goble Rausche Likins Wave Equation Analysis Program (GRLWEAP) was performed in an effort to model the actual driving conditions (Rausche et al., 2000) EOID criteria was determined for the desired capacity using the actual pile and driving equipment information. The results of the GRLWEAP analysis indicated that the ICE-520S hammer was acceptable for use on this project.

Most of the probe piles were instrumented over their entire lengths, while the remaining probe piles were instrumented after 30 to 40 ft (9 to 15 m) of driving. Additionally, all probe piles were instrumented during restrikes. A total of 33 piles were installed and tested. Restrike of all 33 probe piles was performed after approximately 1 to 3 days. Probe piles were driven to depths ranging from 35 to 65 ft (10.6 to 19.8 m). In view of the developer's desire to use these probe piles within the final structure, an attempt was intentionally made not to drive them the entire length in order to leave adequate allowance for cut-off and PDA gage installation.

Dynamic load tests on probe piles confirmed that the subsoil conditions at the project site were generally variable, thus making the site ideally suited for dynamic load testing utilizing a PDA since it appeared to be an appropriate testing method to account for such variability. Being relatively inexpensive and quick, many tests were performed to provide an interpretation of a larger portion of the site. Alternately, many static-load tests would have been required to obtain the quality and quantity of useful information as obtained during dynamic load tests utilizing the PDA and especially in lake and non-lake areas.

PDA predicted allowable pile capacity ranged from 7 to 65 tons (62 to 573 kN) at the end of the initial driving (EOID) and from 40 to 140 tons (364 to 1274 kN) at the beginning of restrike (BOR). Time elapsed between BOR and EOID ranged from 24 to 144 hours. Piles at this site exhibited set-up ranging from 13 to 420 percent. The pile set-up is expressed as a ratio of pile capacity in percent. A summary of PDA results is tabulated in Table 1.

In general a factor of safety of 2.0 was used in accordance with Standard Building Code (SBC) requirements.

While the allowable design capacity was 40 tons (353 kN), some piles in the pile caps were loaded to less than 40 tons (353 kN) as per the actual loads provided by the structural engineer. Due to the actual column loads being lower and due to rounding up the number of piles where calculations indicate a fraction of a pile (i.e., 4.4 pile becomes a 5 pile cap, thereby reducing the actual axial compression pile load), the entire 40 to 45 axial compression capacity was not needed at all pile cap locations.

Table 1. Pile set-up

| PROBE PILE NO./ PRODUCTION PILE NO. | ALLOWABLE PDA CAPACITY AT EOID "a" [1] (tons) (bpf) | ALLOWABLE PDA CAPACITY AT BOR "b" [2] (tons) (bpf) | TIME ELAPSED BETWEEN EOID AND BOR (hrs) | INCREASE ((b-a)/a] x 100 (%) |
|---|---|---|---|---|
| PG1/486 | 42.5 (38) | 79.5 (8/1")[4] | 72 | 87.1 |
| PG2/149 | 20.0 (17) | 54.5 (6/1")[4] | 72 | 172.5 |
| PG3/564 | 20.0 (15) | 40.0 (5/1")[4] | 144 | 100.0 |
| PG4/557 | 15.0 (13) | 43.0 (5/1")[4] | 72 | 186.0 |
| PT6/473 | 27.0 (25) | 84.0 (8/1") | 72 | 211.1 |
| PT7/461 | 37.0 (21) | 61.0 (5/1") | 72 | 64.9 |
| PT8/450 | 55.0 (29) | 80.0 (8/1") | 144 | 45.5 |
| PGL9/527 | 22.5 (22) | 48.0 (7/1")[4] | 72 | 113.3 |
| PGL10/513 | 47.5 (24) | 59.0 (7/1")[4] | 72 | 24.2 |
| PTL11/510 | 52.5 (33) | 72.5 (5/1") | 72 | 38.1 |
| PSWL13/285 | 12.0 (13) | 62.5 (7/1") | 72 | 420.8 |
| PSWL14/356 | 25.0 (16) | 50.0 (18) | 24 | 100.0 |
| PSWL15/296 | 25.0 (9) | 49.0 (7/1") | 72 | 96.0 |
| PSWL16/349 | 7.5 (11) | 65.0 (9/1") | 24 | 766.7 |
| PSWL17/145 | 20.0 (22) | 45.5 (6/1") | 24 | 127.5 |
| PSWL18/55 | 47.5 (48) | 90.0 (11/1") | 72 | 89.5 |
| PC19/153[3] | 30.0 (24) | 112.5 (12/1") | 72 | 275 |
| PC20/155[3] | 21.0 (15) | 100.0 (8/1") | 72 | 376.2 |
| PC21/1 56[3] | 48.5 (26) | 102.0 (28/1") | 24 | 110.3 |
| PC22/154[3] | 44.0 (40) | 92.5 (10/1") | 24 | 110.2 |
| PC23/157[3] | 48.5 (26) | 110.0 (14/1") | 24 | 126.8 |
| PCL24/225[3] | 48.5 (28) | 129.0 (9 /1") | 48 | 63.9 |
| PCL25/229[3] | 65.0 (42) | 143.0 (22/1") | 48 | 120.0 |
| PCL26/567[3] | 45.0 (22) | 144.0 (22/1") | 48 | 220.0 |
| PCL27/228[3] | 55.0 (60) | 132.0 (14/1") | 24 | 140.0 |
| PCL28/227[3] | 46.0 (60) | 112.0 (9/1") | 24 | 143.5 |
| PCL30/497[3] | 57.5 (9/1") | 65.0 (14/1") | 48 | 13.0 |
| PCL31/502[3] | 57.5 (8/1") | 69.5 (24/1") | 48 | 20.9 |
| PCL32/498[3] | 50.0 (18) | 83.0 (11/1") | 72 | 66.0 |
| PCL33/501[35] | 45.0 (28) | 92.0 (11/1") | 48 | 104.4 |
| PCL34/493[3] | 44.0 (24) | 112.0 (11/1") | 48 | 154.5 |

NOTES: (1) EOID = end of initial driving
(2) BOR = beginning of restrike
(3) Pile driven in cluster
(4) Rated allowable capacity of 27 tons for garage area piles(PG, PGL), others at 40 tons

1 ft = 0.3028 m; 1 ton = 8.897kN

bpf = blows per foot on ICE 520S Hammer

## Results of CAPWAP Analyses

Dynamic data obtained in the field was further analyzed according to Case Pile Wave Analysis Program (CAPWAP) for a more comprehensive understanding of the soil and pile behavior during pile driving (ref 2). CAPWAP analyses were performed on a total of 75 blows collected during restrikes of probe files. These analyses provided a better evaluation of total ultimate pile capacity. It is a signal-matching process where a measured signal is matched with a simulated signal. This step provides a refinement of the pile capacity estimated in the field during driving. Additionally, CAPWAP provided a distribution of soil resistance along the embedded pile depth. The distribution of resistance along the pile depth enabled an estimation of resistance at other depths above the pile tip (this type of information is difficult to obtain using conventional static-load tests unless expensive instrumentation such as telltales are installed in the test pile prior to driving). A partial summary of CAPWAP analyses performed on blows collected during restrikes of probe piles are presented in Table 2. It includes a partial summary of static pile capacity, soil resistance distribution along pile shaft and under toe, soil damping and quake (maximum elastic deformation) values, and forces along pile length at ultimate resistance.

Table 2. Partial summary of CAPWAP results from restrikes only

| PROBE PILE NO. | EMBEDMENT DEPTH (ft) | BLOW COUNT (bpi) | ULTIMATE PDA CAPACITY (tons) | ULTIMATE CAPWAP CAPACITY (tons) | | | QUAKE (Q) AND DAMPING (J) | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SIDE FRICTION | END BEARING | TOTAL | $Q_s$ (in) | $Q_t$ (in) | $J_s$ (-) | $J_t$ (-) |
| PG2 | 45.0 | 6 | 109.0 | 81.0 | 24.0 | 105.0 (1) | .051 | .311 | .270 | .128 |
| PG3 | 54.0 | 5 | 80.0 | 37.5 | 31.5 | 69.0 (1) | .050 | .252 | .222 | .181 |
| PT7 | 57.0 | 5 | 122.0 | 85.0 | 30.0 | 115.0 | .040 | .268 | .410 | .197 |
| PGL9 | 57.0 | 7 | 96.0 | 69.5 | 19.5 | 89.0 (1) | .100 | .195 | .470 | .324 |
| PSWL14 | 57.0 | 1.5 | 100.0 | 63.0 | 21.0 | 84.0 | .064 | .325 | .430 | .294 |
| PSWL17 | 51.0 | 6 | 91.0 | 64.0 | 19.5 | 83.5 | .100 | .230 | .736 | .241 |
| PCL32 | 51.0 | 11 | 166.0 | 79.0 | 75.0 | 154.0 | .041 | .176 | .640 | .281 |
| NOTE: (1) Rated allowable capacity of 27 tons for garage area piles(PG, PGL), others at 40 tons | | | | | | | | | | |

## Production Pile Lengths

Following completion of the probe pile and DLT program specific pile lengths and capacity were finalized after taking into consideration the excavated and prepared surface elevations of various areas within the building limits. PDA assisted DLT program resulted in a fully optimized pile layout plan utilizing 5 different pile lengths throughout the tower and underneath parking area. Allowable capacities ranged from 27 tons (120 kN) to 40 tons (355 kN). It resulted in reduced and variable pile lengths. Total of piling estimates was 36,500 ft (11,128 m) and total of piling installed was 30,000 ft (9,146 m) thereby reflecting a saving of 18 percent in length and/or cost.

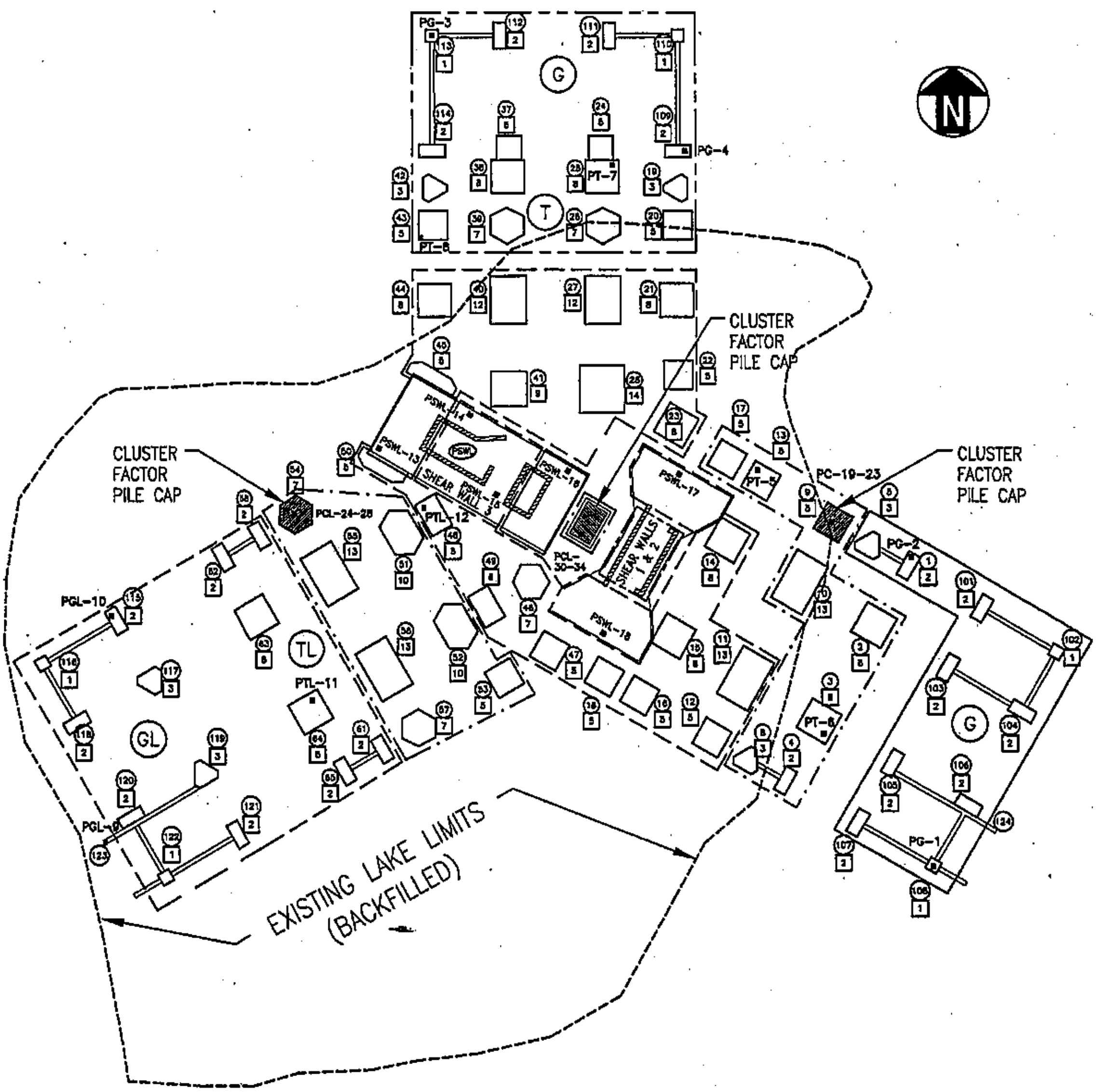
## Pile Installation and Quality Control

Following the geotechnical consultant's review of the pile layout plan, details of pile driving specifications were finalized for the production pile program to proceed. Work commenced following completion of the prepared (i.e. excavated and graded) soil surface, and full-time monitoring and logging of the program was provided. Records of pile number, location, date of installation, length of pile, blows per foot, hammer energy, and unusual occurrences for each pile were kept. The approval for the pile driving termination was given upon review of pile driving records by a representative of the geotechnical consultant.

A total of 568 PPC piles (including 33 probe piles) were installed during the probe and production pile installation phases. For the lower tower and garage area, 14-in (356mm) square piles were delivered to the site in the lengths of 65 ft (19.8m) and were driven to depths ranging from 48 to 55 ft (14.6 to 16.8 m.). A foundation layout of production and probe piles is illustrated in Figure 3.

Probe and production piles were installed with an ICE 520S diesel double-acting hammer rated at 30,000 ft-lbs (4,200 m-kg) maximum rated energy. It was equipped with a bouncing chamber pressure gauge and properly maintained hammer cushion. The hammer was equipped with 3 fuel settings to control the camber pressure and the ram stroke. For this project site the hammer was operated at the highest fuel setting at all times and the ram did not stroke high at the beginning of driving because the soil resistance was low. Special precaution was also taken to maintain proper pile cushion.

During the logging and monitoring of the production pile driving operation 485 PPC piles (for the tower and shearwall area) were rated for 40 tons (363 kN) and 50 (for the garage piles)were rated for 27 tons (245 kN) allowable capacity based on either the end of initial drive (EOID) or beginning of redrive (BOR) results. For piles that met the minimum driving criteria of 20-25 bpf at EOID, a random selection of 1 in 10 was used to retap to verify pile capacity after set-up. Piles that did not meet the minimum driving criteria were all retapped. Minimum retapping criteria for 80 ton ultimate capacity was 6-8 bpi.




| LEGEND | | LINE TYPE CODE | LENGTH (ft) | # OF PILES |
|---|---|---|---|---|
| ⑫ | Column or Pile Cap Number | ——— | 45 | 17 |
| [12] | Number of Piles | — · — · — | 50 | 110 |
| PG-1 | Probe Pile Number | — - — - — | 52 | 12 |
| PG, PT, PTL | Garage, Tower and Lake Area Tower Piles | — — — — | 55 | 279 |
| SWL, C, CL | Shearwall, Pile Clusters in Tower, Lake, and Non Lake Areas | — - - — | 58 | 116 |
| | | TOTAL | | 534 |

Figure 3. Production layout of production and probe piles

Based on continuous and satisfactory monitoring of the pile installation program (e.g., logging the blowcount, bounce chamber pressure, stroke height, and pile embedment) the PPC pile foundation system, as installed, was approved and accepted for the building. The Waterside Building 3 project was completed in late 1999 and released for occupancy. To date the entire structure has performed well.

## OBSERVATIONS, REMARKS, AND CONCLUSIONS

The authors believe that the use of properly installed and effectively monitored and tested precast prestressed concrete piles was economical as compared to the other deep foundation systems and that PDA proved to be a key element of the overall approval and acceptance process.

1) It is important to have an accurate and reliable method of calculating axial pile capacity. Although several methods exist for designing axially loaded piles, every design method should be verified for applicability to a specific site prior to use. The SPT94 software was selected for this project due to its applicability to foundations in Florida soils. SPT 97 was not available in late 1997 when he design was completed.

2) Cost of the selected and installed pile foundation utilizing PDA was generally less than that of other deep foundations such as monotube, steel pipe piles or drilled shaft. It resulted in reduced and variable pile lengths. Total of piling estimated was 36,500 ft (11,128 m) and total of piling installed was 30,000 ft (9,146 m) thereby reflecting a saving of 18 percent in length and/or cost.

3) Longer piles exhibited higher set-up than shorter length piles. This trend can be attributed to the fact that pile set-up is predominantly derived from skin friction rather than end bearing. Longer piles naturally have greater surface area versus shorter piles of the same diameter. (Saxena et al)

4) Pile set-up is directly proportional to the confining pressure of the soil. Longer piles exhibit higher confining pressures due to overburden. This phenomenon has been reported by others as well.

5) The authors believe that further studies involving comparison of CAPWAP-predicted pile capacities at the EOID and BOR will reinforce the belief that set-up is predominantly contributed from the skin friction portion of the pile capacity.

6) In Florida's sandy soils, especially along the gulf coast, pile set up phenomenon has been commonly used for over two decades.

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The case history information from this project provided a useful database for reference on future projects of this type in this area of southwest Florida. The authors also wish to thank Natalie Hall and Jeff Cox for preparation of the text and diagrams presented in this paper.

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