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A NOVEL FOUNDATION PILING SYSTEM - THE SPEAR PILE

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ABSTRACT:

This paper describes the design, installation and field testing of a composite pile system consisting of a 9.1-m long, prestressed concrete bottom section with variable length steel pipe top sections. The bottom 6-m of each concrete section tapers from a 356-mm square top to a 203-mm square tip. The steel pipe is uniform, with an outside diameter of 273 mm. A case history is presented for a 6-story structure and an over-water helipad founded on Spear Piles. Design considerations included the selection of working loads, selection of an appropriate pile driving hammer, and preliminary evaluation of driving procedures. Both static load tests and dynamic measurements and analyses were performed to evaluate pile capacities. The dynamic measurements were also used to evaluate hammer performance and pile stresses during installation. Design loads of up to 890 kN per pile were used at the site. It is concluded that the Spear Pile is an effective and economical deep foundation system when subsurface conditions indicate the potential for large downdrag loads.

INTRODUCTION

Pile foundations are usually employed when shallow foundations will not provide adequate support or protection against excessive settlement. Piles might be timber, concrete, steel or a combination of these materials. Pile shapes available for use include uniform-circular (pipe and concrete), "H" section (steel) or tapered (timber, concrete and metal shell). Composite piles are typically constructed of two different materials such as a timber pile with a concrete section build-up or a concrete pile with a steel "H" or pipe section "stinger". Each of these conventional

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pile types has its advantages and disadvantages (Chellis 1961), and any of them can be an effective foundation solution if used under appropriate conditions.

Pile capacity is derived from a combination of shaft resistance and toe bearing. As the pile is forced down into the soil by the applied load, the soil friction along the sides of the pile is mobilized, as well as the bearing force at the tip of the pile. In cases where compressible soil above the pile tip will continue to consolidate after pile installation, the downward movement of the soil relative to the pile will result in a downdrag load on the pile, reducing its net working load capacity. The downdrag load is caused by negative skin friction acting on the shaft of the pile. Studies and field measurements have shown that the downdrag load magnitudes can be significant (Garlanger and Lambe 1973), and may even be larger than the applied structural loads in some cases. Modern pile design and construction methods incorporate provisions that take the effects of negative skin friction into account (Fellenius 1991). A common construction technique is to apply a bitumen coating along the pile shaft length where negative skin friction is expected (Baligh et al. 1978); however, this is an impractical field operation with questionable effectiveness under many conditions.

This paper presents discussions on design, installation, field testing and application of a new piling system. This new pile is a composite type pile with a tapered prestressed concrete tip section and a steel pipe top section.

THE SPEAR PILE

General Description

The pile type discussed in this paper has been named the "SPEAR PILE" because of the slender, steel pipe top section mated to a tapered concrete bottom section. A steel transfer plate was used to splice the pipe section to the concrete section. Although the taper tip section is limited by the available forms to a length of 6 m, the straight concrete top of the tip section can be as long as 15 m. The typical pile details are shown in Figure 1.

The steel transfer plate is cast in the top of the concrete pile section. The prestress strands taper uniformly from the tip to the top plate. The piles are cast in sets, tip to tip and top to top with a spacer shim system aligning the top plate perpendicular to the pile axis. The taper section tip form drops into the standard 356 mm square section precast bed forms.

Structural Design

The tapered pile tip section was cast using 41 MPa concrete. This section was prestressed with four 9/16 (14.3 mm) low relaxation strands in a square pattern. Each strand was tensioned to 172 kN per strand. The net prestress was about 4.8 MPa after release of the strands from the anchors. The splice plate had a thickness

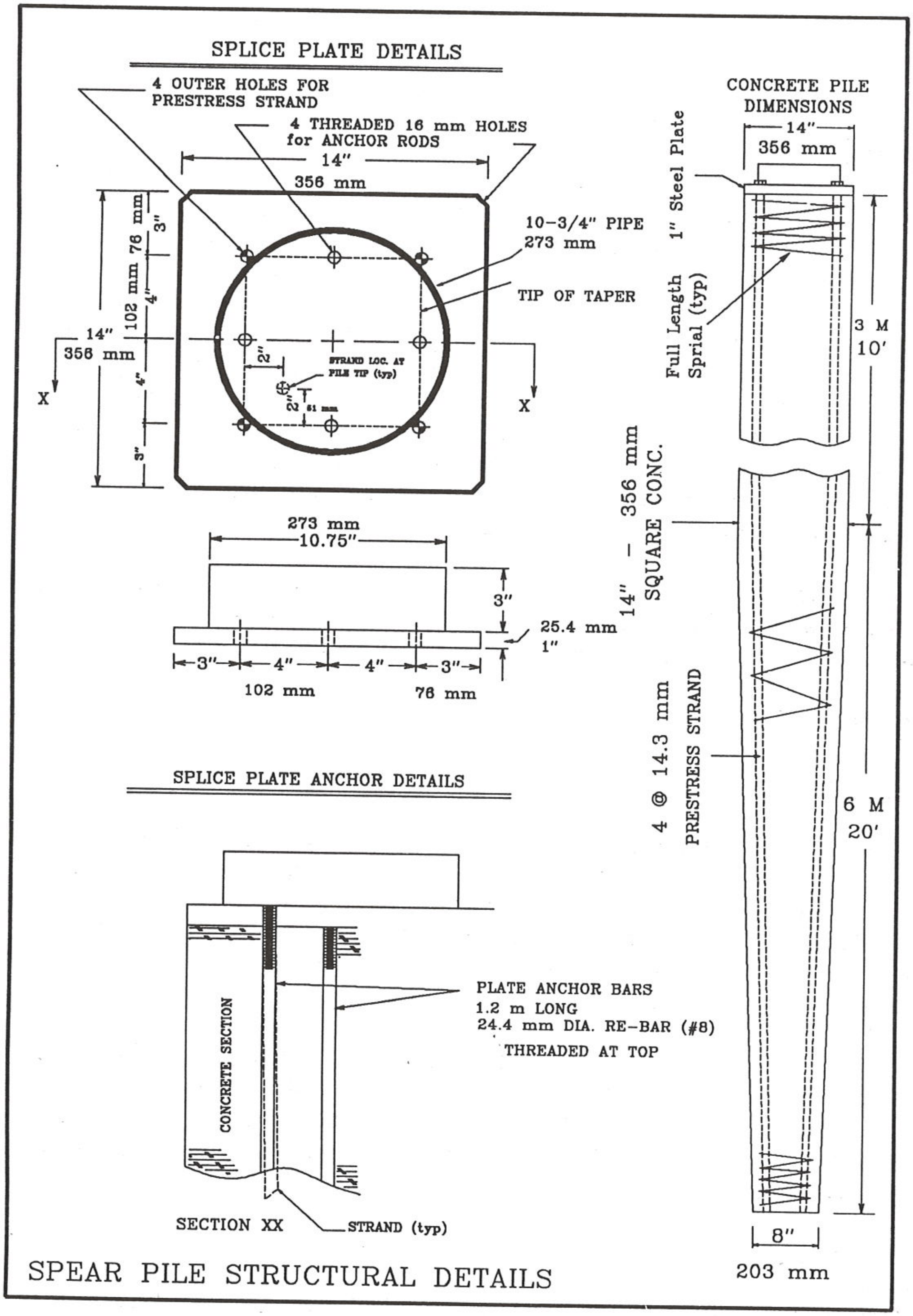

SPLICE PLATE DETAILS
4 OUTER HOLES FOR PRESTRESS STRAND
4 THREADED 16 mm HOLES for ANCHOR RODS
14"
356 mm
10-3/4" PIPE
273 mm
TIP OF TAPER
STRAND LOC. AT PILE TIP (typ)
X
273 mm
10.75"
3"
25.4 mm
1"
102 mm
76 mm
SPLICE PLATE ANCHOR DETAILS
CONCRETE SECTION
PLATE ANCHOR BARS
1.2 m LONG
24.4 mm DIA. RE-BAR (#8)
THREADED AT TOP
SECTION XX
STRAND (typ)
CONCRETE PILE DIMENSIONS
1" Steel Plate
Full Length Sprial (typ)
3 M
10'
14" - 356 mm SQUARE CONC.
4 @ 14.3 mm PRESTRESS STRAND
6 M
20'
8"
203 mm
SPEAR PILE STRUCTURAL DETAILS

FIGURE 1

of 19 mm, and was fixed to the top of the pile with four #8 reinforcing bars 1.2-m long.

A 273-mm diameter, 6.35-mm wall pipe was chosen for this project to maximize the difference between top and bottom section areas. Some 305-mm pipe was used when the supplier ran low on the smaller pipe to maintain progress, but most of the piles on the project were the smaller pipe.

CASE HISTORY

Project Description

Cape Canaveral Hospital is located on a dredged-fill island in the Banana River, west of the cities of Cape Canaveral and Cocoa Beach, and north of SR 520 in Florida, USA. The island is about 212 m long in the N-S direction and 178 m wide. A 105-m by 64-m wide causeway connects the fill island to the causeway fill for SR 520. The Ground surface of the island is at about elevation 1.7 meters above sea level. The site was occupied by two main buildings, the 2-story administrative wing and the 6-story hospital wing. Although the buildings are adjacent to each other and one can be easily accessed from the other, they are structurally separate.

The 6-story building was initially constructed to 4-stories and was founded on step-taper type piles. The building was to be expanded to 10-stories at a later date. Prior to the vertical expansion, the Hospital asked that a review be conducted on the foundation capacity to assure that the vertical expansion was safe. The review revealed that the full expansion could not be constructed because downdrag loading resulting from to the settlement of the site fill had reduced the allowable capacity of the piles. The expansion was limited to two additional stories and the remaining Hospital expansion was reserved for new site areas.

The 2-story building was founded on tapered, fluted metal-shell piles at about depth 13 meters. Because no provision was apparently made for downdrag load on these piles, this building was not scheduled for expansion by the Hospital.

The 1992 Hospital expansion consisted of the addition of in-fill buildings adjacent to and between existing 6-story and 2-story buildings and the addition of a Helicopter Landing Pad (Helo-Pad). Because of the highly limited site area, the decision was made to add the Helo-Pad as an elevated pile-supported platform over water. The Helo-Pad is located at the northeast corner of the site area. Because of the environmental sensitivity of the shallow water surrounding the hospital fill area, the Florida Departments of Natural Resources and Environmental Protection would not allow the use of a barge for construction. All piles had to be driven using a crane located on the shoreline. Since the distance from the shore line was up to 24-m to the pile locations, a very light pile was required.

Subsurface Conditions

The dredge fill is medium dense silty fine sand with some clay lenses extending 3.0 to 4.6 meters below grade. The fill is underlain by soft silty and sandy clay and clayey organic silt to depth 17 m. These strata are underlain by firm sandy clay and loose to medium dense silty sand to depth 22 m. A stratum of firm to stiff sandy-gravelly clay with lenses of sand and limerock was found to extend to a weathered limerock stratum at depth 46 to 52 m. The limerock stratum had refusal Standard Penetration Test resistances of greater than 100 blows per 0.3-m. The typical soil conditions are represented in Figure 2. The area around the island was at about sea level, and exhibited a thin silty sand cap about 1 to 1.5-m in thickness. It appeared that there had been moderate heaving of the soil around the fill island relative the to soil levels observed at the same distance from the State Road fill. Observations around the site structures indicated that at least 600 mm of settlement had occurred since a side walk had been poured around the buildings. Constant maintenance has been required to provide a safe ramp from the entry area to the building due to continued settlement of the fill relative to the buildings.

Foundation Design

The design of the foundation for the proposed site expansion was initiated by concern by the Hospital about the capacity of the original step-taper piles used in the original construction. A review of the available project records revealed that the soil, pile installation and pile load test data were available for the step-taper piles used in the 6-story building, but no data were available on the fluted-taper shell piles used in the 2-story building. The initial load test on a step-taper pile was on a pile installed to depth 27 m. This pile failed at a total load of 800 kN. A second test step-taper pile was installed to depth 41 m. The deeper pile reached a maximum load of 1,690 kN. Although no data were available on the fluted-taper shell pile, it was recalled by the Hospital maintenance staff that the piles were installed to about depth 26 m. The tapered shell pile was described as having a total test load of 1,600 kN.

The review of subsurface data revealed that settlement of fill placed over the thick stratum of soft sediments between depth 4 m and 17 m would result in a potential for large downdrag loads on the piles (800 kN). However, the review of performance data for the piles used on the existing buildings indicated that taper-tip piles could provide a substantial advantage in depth-capacity performance. For this reason, a taper pile was selected for the Hospital expansion. The pile was designed to minimize the downdrag load by reducing the pile section above the anticipated bearing zone. Further, the reduced section above the pile tip would minimize the potential to damage measures such as a plastic wrap, bitumen coating or PVC pipe sleeve that would be used to limit downdrag loads on the piles.

Analyses were run using various methods to evaluate pile capacity and downdrag loads. Pile capacity analyses presented here used the soil strength

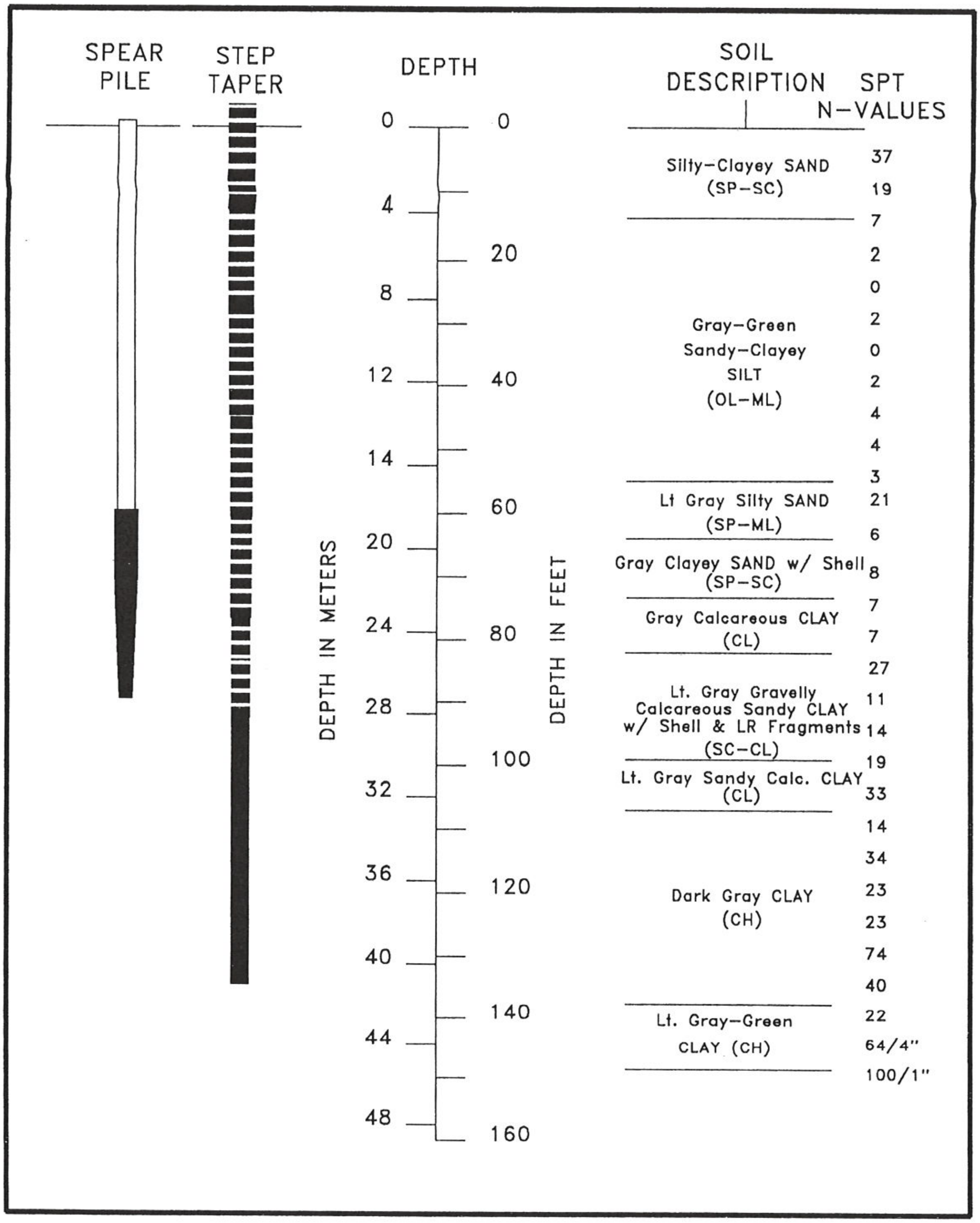


FIGURE 2 SOIL CONDITIONS AT TEST PILE

correlation with SPT N-Value recommended in the Florida Department of Transportation Research Bulletin 121-A (Schmertmann, 1967). A safety factor of 1.5 was used for the negative skin friction, 2.0 on support friction and 3.0 on end bearing. Figure 3 shows that a straight 357 mm square pile develops 768 kN of negative skin friction. The Spear Pile was analyzed using plastic wrap for downdrag protection and an assumption that the SPT N-Values improved by 25 blows in the bearing stratum due to compaction of the loose silty sand by the wedging action of the Spear Pile's taper tip. If the improvement of penetration resistance was not used, the pile capacity demonstrated by load tests could not be duplicated in the analyses. Figure 4 shows that the capacity of the pile for the assumed increase in the bearing stratum N-Values matches the capacity in the load test. This figure shows a net Spear Pile capacity of more than 900 kN below depth 26 meters and only 110 kN of downdrag.

Pile Driving

Because the surface soils varied from loose to dense sand, and were known to contain some debris that could damage the piles, the pile locations were predrilled with a 305 mm diameter auger to depth 12 m. The drill holes stood open, and when the test pile was driven, the steel pipe followed the larger hole left by the drilling and the concrete tip with no observed tendency to cave. The upper 12 m of the pile was wrapped in 3 turns of 6 mil (.15 mm) polyethylene plastic for downdrag protection. The plastic was wrapped with duct tape at intervals along the pile with heavy tape wrap at the bottom to limit the tendency for the soil to peel the plastic from the pipe. No damage or tearing of the plastic was noted on the production piles. The plastic wrap around the steel pipe top of the piles was used only on the heavily loaded piles. Piles loaded to only 267 kN working load, including the off-shore Helo-Pad piles, were not wrapped.

A 6-m section of pipe was welded to the concrete tip prior to the start of pile driving. Figure 5 is a photograph showing the weld of the pipe top section to the steel splice plate at the top of the concrete section. When the pile was driven to about depth 14 m, a 12-m section of pipe, wrapped with the plastic material, was welded to the exposed end of the driven pile and pile driving continued until refusal was achieved. Figure 6 is a photograph of the site showing some Spear Piles stockpiled in preparation for driving. Piles already driven can be seen in the background, including some with the plastic wrap downdrag protection visible.

Two diesel pile driving hammers were used on the project, an open ended Kobe K-13 and a double acting LinkBelt 520. The actual pile driving resulted in resistances of 5 to 20 blows per foot (60 to 2 mm/blow) above depth 18 m. Below depth 18 m, the penetration resistance slowly rose until the pile reached refusal. The average pile takeup for 405 piles was 26 m with the shortest pile reaching refusal at depth 18 m and the deepest pile at 30 m.

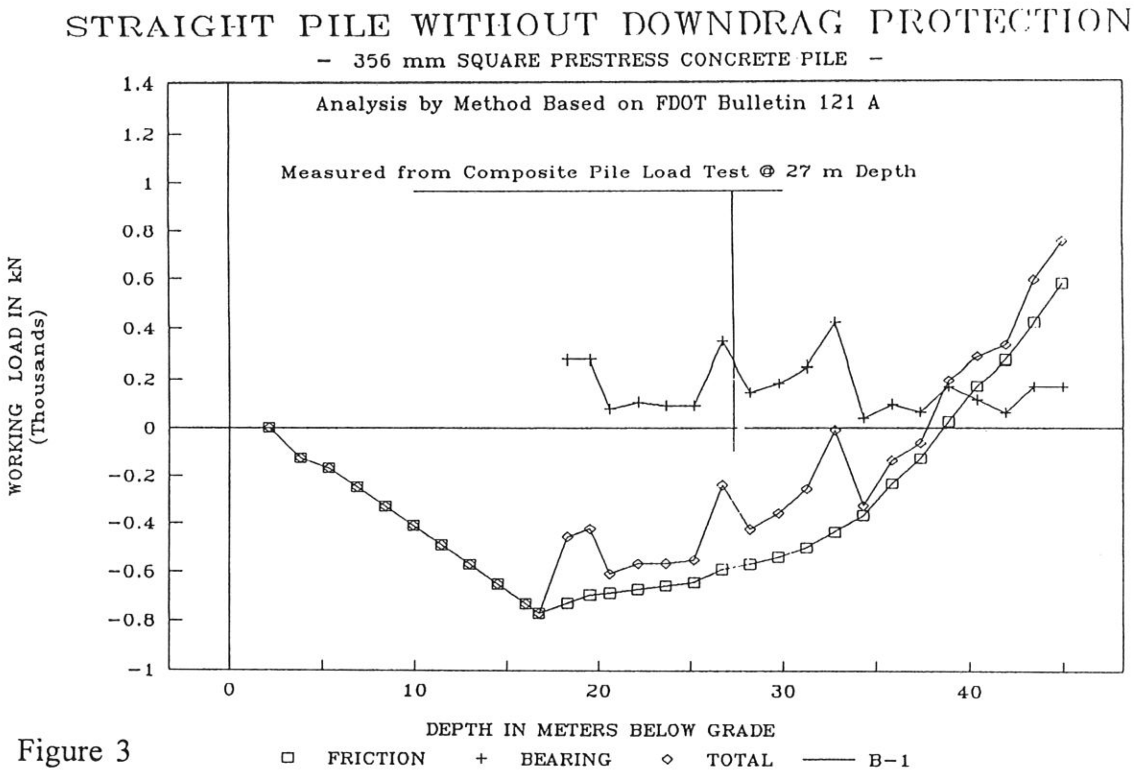


Figure 3

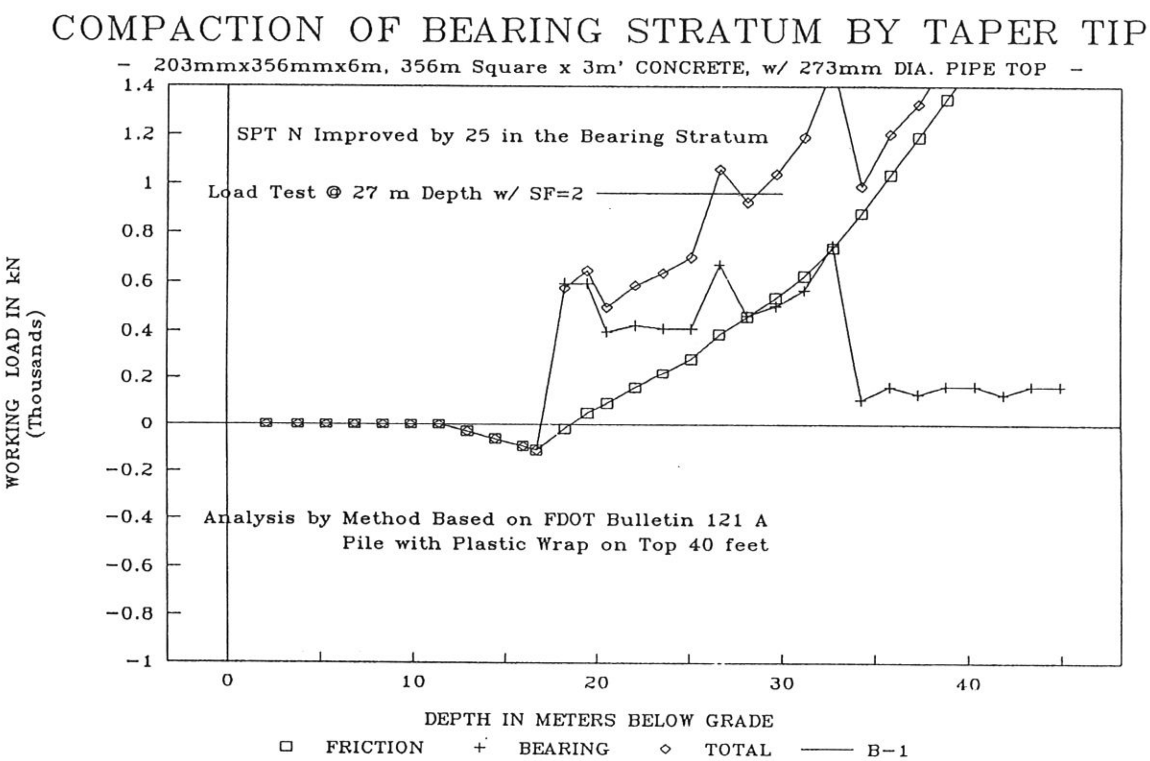


Figure 4

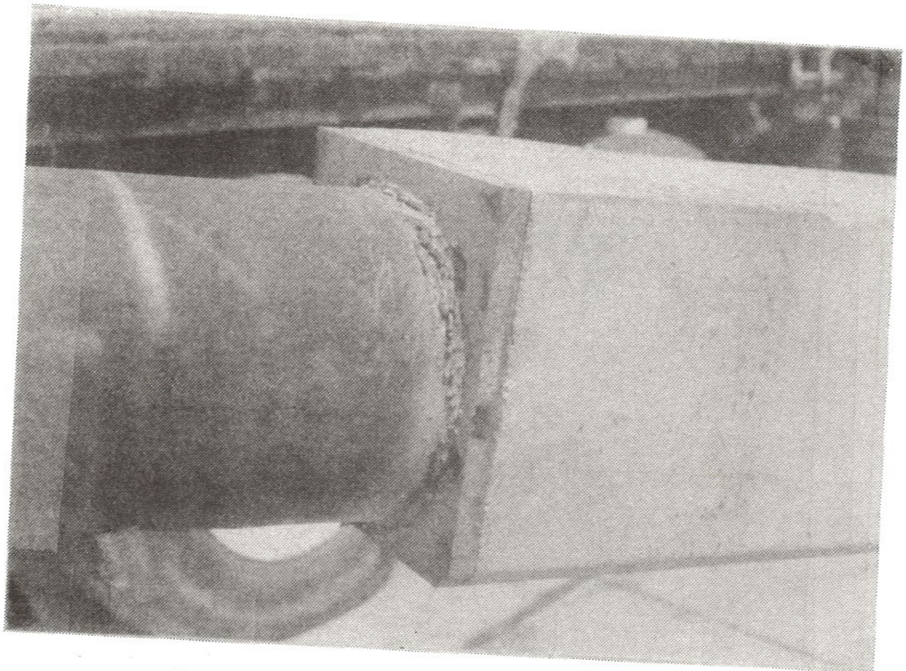


Figure 5 Steel Pipe - Concrete Section Construction

Figure 6 General View of Site and Piles

The Helo-Pad piles were set by hanging the piles from the crane and hand-setting the piles through a steel frame template. After the piles were set, the leads were fixed to the template and the piles were driven using the same techniques that were developed for the land piles. Predrilling was not required for the Helo-Pad piles.

PILE TESTING

Static Pile Load Test

The static load test was run on a pile near boring B-1. Figure 2 shows the pile in relation to the soil profile. The depth of the test pile was 27 m below grade at a penetration resistance of 29 blows per last inch (1 mm per blow) with a Kobe K-13 diesel hammer. The pile was loaded to 1,557 kN, and the load was held for 24 hours. The pile was then unloaded, and reloaded until the jack reached its maximum capacity of 1,931 kN. Figure 7 shows the summary of the load test plotted as settlement versus load. The maximum settlement at the end of the first 24 hour hold was 21 mm. The net settlement was 7 mm following the rebound. Evaluation of the static load test data indicates that the pile did not reach failure, even at 1,931 kN, the limit of the test jack.

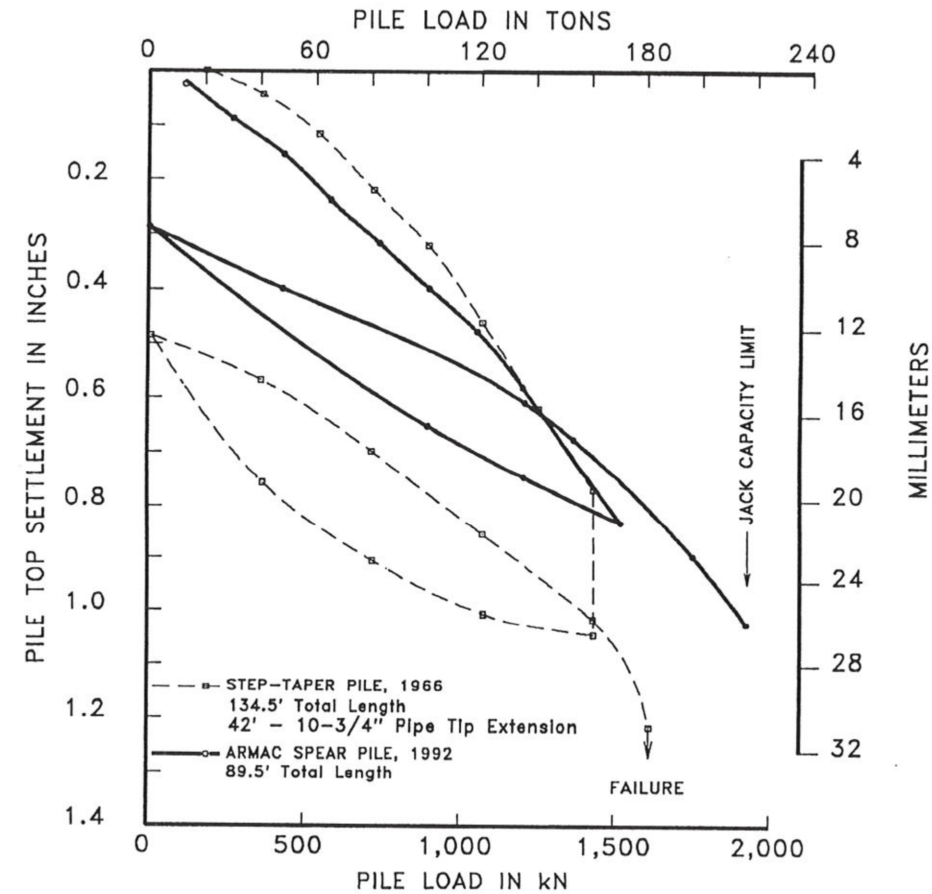


FIGURE 7 STATIC LOAD TEST RESULTS

Figure 7 also shows the results of the 1966 load test on the step-taper pile. This pile reached its original design load of 1,557 kN with the load held for 24 hours. During that time, significant additional settlement (7.5 mm) occurred. The pile failed to sustain additional load at 1,690 kN. The step-taper test pile did not encounter significant driving resistance until depth 36.6 m, about 9 m deeper than the Spear Pile.

Dynamic Pile Testing

Testing and evaluation of the Spear Piles included dynamic pile monitoring during the initial installation of one pile (Test Pile A) and restrike of two other piles (Test Piles B & C). Field testing was performed with a Pile Driving Analyzer™ (PDA) according to the Case Method (Goble and Hussein 1994). Subsequent data analyses were done according to the CAPWAP® Method (Rausche et al. 1994). The primary objectives of the tests were the evaluation of hammer/driving system performance, pile driving stresses and structural integrity, and static pile capacity. The distribution of soil resistance along the pile was also evaluated. This type of pile testing and evaluation is routine in modern deep foundation practice, and is recognized by many standards and specifications (ASTM 1989).

Dynamic measurements of strain and acceleration were taken approximately two feet below the top of each of the three tested piles. Two each strain transducers and accelerometers were bolted on opposite sides of each pile to monitor and average effects of non-uniform hammer impacts. The PDA provided signal conditioning, amplification, filtering, calibration to measured signals and data quality assessment before applying Case Method equations to measured pile records of force and velocity under each hammer impact. Figure 8 presents plots of pile top records of force and velocity obtained under hammer blows during the monitoring of each test pile. Due to the highly non-uniform nature of this pile type, the PDA was primarily used to obtain dynamic pile records. Dynamic data analysis with the CAPWAP program provided much of the information regarding pile and soil behavior. The CAPWAP analysis is done in an interactive environment using measured pile data and wave equation type analysis in a system identification process using signal matching techniques. Results from a CAPWAP analysis included: static pile capacity, soil resistance distribution, soil damping and stiffness values along the pile shaft and under its toe, forces (and stresses) along the pile length at maximum load, and a simulated static loading test showing pile load-movement relationship under static conditions. Figure 9 presents plotted CAPWAP analysis results performed with data representing a hammer blow towards the end of driving of Test Pile A.

All three tested piles were identical with length of 27.43 m and consisting of a 9.14 long concrete section with an 18.29 m long steel pipe (305 mm outside diameter and 6 mm wall thickness. Testing was accomplished using the LB 520 hammer. During initial installation, compression pile stresses were approximately

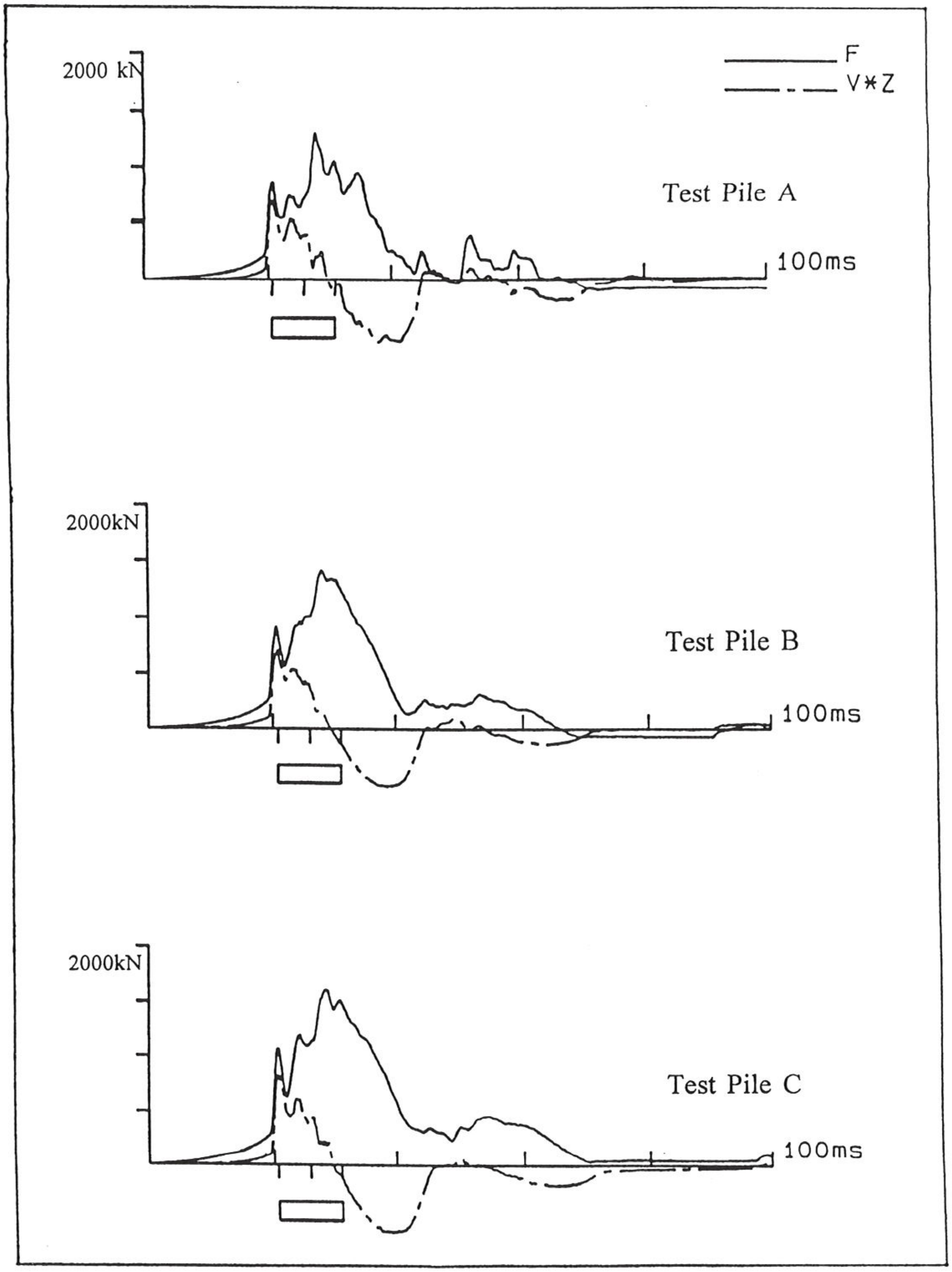
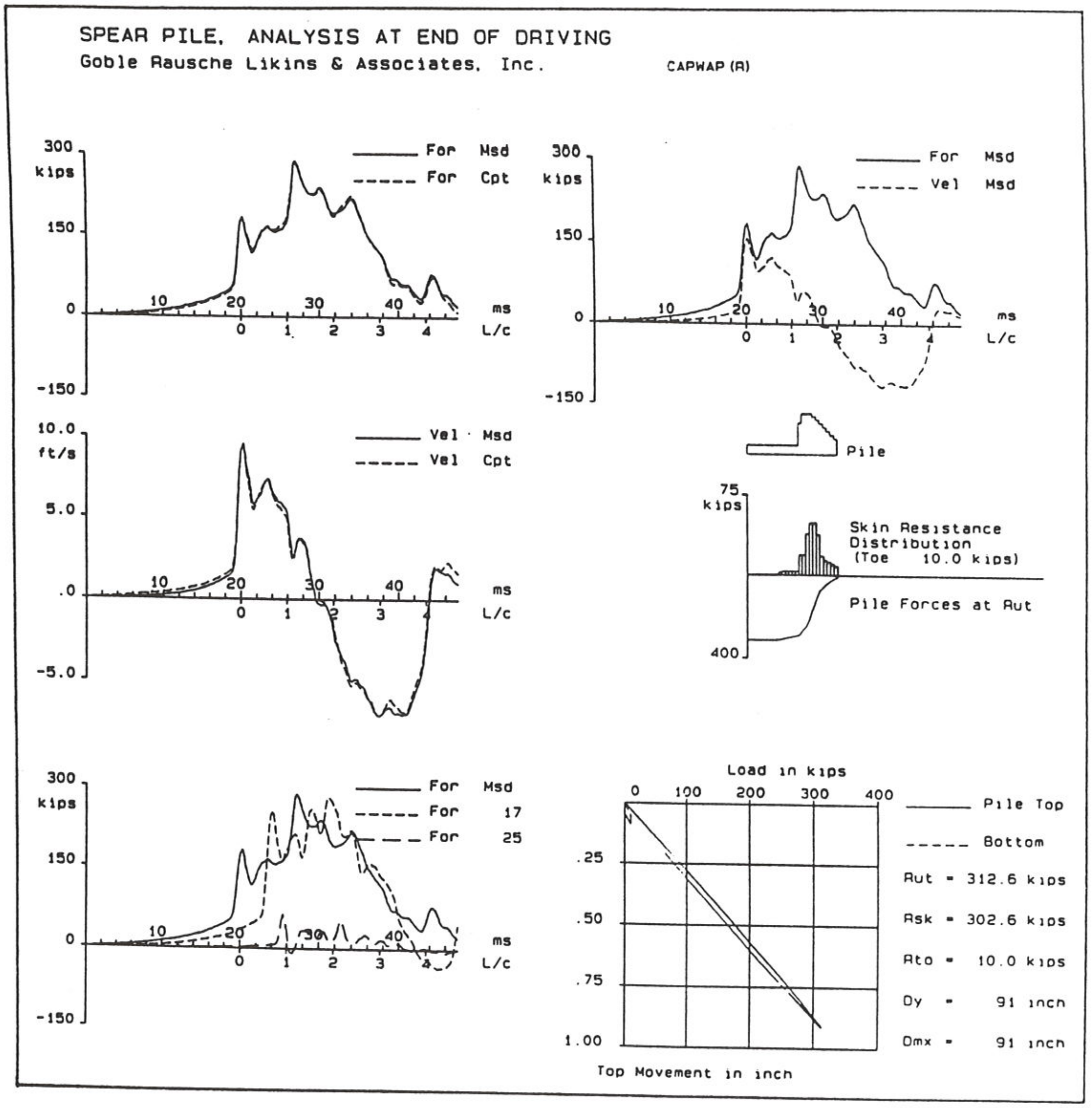


Figure 8: Records of Pile Force and Velocity Histories Under Hammer Impacts.



(1 kip = 4.45 kN, 1 inch = 25.4 mm)

Figure 9: CAPWAP Analysis Results
Test Pile A.

217 MPa in the steel section, 10 MPa in the uniform concrete section and 2.5 MPa near the pile toe. Dynamic pile tension stresses were generally negligible. Towards the end of driving, maximum transferred energy to the pile top averaged approximately 13.6 KJ which translates to 38% transfer ratio when compared to the hammer rated energy of 36 KJ. This is generally considered to be an average hammer performance. Data analysis indicated end of driving mobilized static pile capacity of 1,393 kN. Computations based on field measured dynamic data indicated that no soil resistance was present in the upper 10 m of pile length and that only 107 kN of shaft resistance was coming from the rest of the steel section; 92% of pile capacity was contributed by the concrete section and only 3% was in toe bearing. Capwap analysis with the data representing a hammer blow from the beginning of restrike of Test Pile B indicated a mobilized static pile capacity of 1,722 kN, most of which (more than 80%) was contributed by shaft resistance along the concrete section. Due to the high driving resistance (i.e., low pile set under individual blows) during both initial driving and restrikes, it is believed that the static pile capacities computed represent only lower bound values due to lack of full mobilization of soil resistance. Compressive pile stresses were somewhat higher during restrikes than those encountered during initial driving. Dynamic data did not indicate structural damage in any of the piles tested.

CONCLUSIONS

The Spear Pile proved to be an efficient pile with respect to load capacity versus depth. As demonstrated by the comparison of the static load test on the Spear Pile with the results of the load test on the step-taper pile, the compaction of the bearing stratum resulted in a higher capacity for the shallow depth of Spear Pile in comparison with the deeper step-taper pile.

The Spear pile was easy to handle and install because of its light weight and short segment lengths. The steel pipe top could be easily welded to accommodate the varying lengths of pile encountered on the site. Only two piles were broken of the more than 400 piles handled. There was some problem using a steel pipe with only a 1/4 inch (6.35 mm) wall. Great care had to be used to assure alignment of the hammer and the pile during driving. Otherwise, buckling of the pile top was experienced.

Another advantage of the Spear Pile was the ability to easily reduce negative skin friction. Several options were available, but because of stable ground conditions only wrapping of the upper 40 feet of pipe section in 6 mil (0.15 mm) polyethylene sheet was required. The dynamic analyses demonstrated that the mobilized skin friction above the bearing zone was minimal.

The dynamic testing showed that the pile hammer did not mobilize the full strength of the pile on re-strike. A larger pile hammer would be required to achieve that load. Based on the driving and load test record of the step-taper pile used in the original construction, it was felt that the use of the smaller hammer would prevent excessive shearing of the bearing stratum that could cause a loss of effectiveness in compacting the silty sand bearing stratum.

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