Uplift Capacity of Driven Piles from Static Loading Tests

M. H. Hussein
Goble Rausche Likins and Associates, Inc., Orlando, Florida

J. M. Sheahan
HDR Engineering Inc., Pittsburgh, Pennsylvania

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 1.02

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M. H. Hussein
Partner, Goble Rausche Likins and Associates, Inc., Orlando, Florida

J. M. Sheahan
Senior Geotechnical Engineer, HDR Engineering Inc., Pittsburgh, Pennsylvania

Synopsis: A pile driving and testing program was undertaken to evaluate installation procedures, assess capacity (particularly in uplift) of 24- and 30-inch square, prestressed concrete piles, and provide foundation design parameters for the New Edison Bridge project in Fort Myers, Florida. The subsurface profile generally consisted of three major soil strata: an upper clayey sand and sandy clay layer to a depth of about 45 to 50 feet below mudline, a medium dense to dense silty sand middle layer about 10 feet thick, and a third layer of dense to very dense silty fine sand. Eleven prestressed concrete test piles of varying length were driven at five test sites and uplift tests were performed to allow an independent evaluation of the different soil layers. All piles were dynamically monitored during installation and restrikes to evaluate pile drivability and bearing capacity including time related capacity increases due to soil "set-up." This paper presents descriptions of the pile driving and load testing program along with findings regarding pile uplift capacities from load tests, pile capacities from dynamic testing, soil-pile adhesion values, wave equation factors, soil strength vs time dependency, foundation design and pile driving recommendations.

1. INTRODUCTION

The existing 60 year old, two-lane draw-bridge structure (known as the Edison Bridge) carries Route 41 over the Caloosahatchee River in Fort Myers, Florida and will be replaced by two separate bridges, each approximately one mile in length, for north- and south-bound traffic. The main spans will be 56 feet high to accommodate boat traffic through the relocated river channel. The project cost is estimated at 40 million dollars.

Structural, geotechnical, design ship-impact loading and other considerations required that the new bridges be founded on high-capacity driven piles. Subsurface investigations, geotechnical studies, and a pile load test program were undertaken as part of the foundation design process. A benefit/cost analysis indicated that a load test program was justified to determine pile ultimate uplift capacities and to examine pile drivability. Uplift capacity was an important consideration because of the need to design for ship impact loads on the substructure. Prestressed concrete 24- and 30-inch square piles were driven to varying penetrations at five test sites and uplift load tests were performed to allow an independent evaluation of each of the three major soil layers. Dynamic pile testing was performed during initial installation and restrikes to evaluate hammer performance, pile driving stresses and structural integrity, pile static bearing capacity and time related capacity increases due to soil set-up. This paper presents descriptions of the geotechnical investigations and pile driving and load testing procedures and results. Since the original work was performed using the English units system, the same will be used here. A conversion table to SI units is appended at the end of the paper.

2. GEOTECHNICAL INVESTIGATIONS

Test borings were taken at intervals of about 200 feet along the proposed bridge alignments and at each of the five pile test site locations. The water depth across the river ranged between 4 to 10 feet. Geotechnical investigations included Standard Penetration Testing (SPT) and laboratory analyses, and to a lesser extent Cone Penetration Testing (CPT) and Vane Shear Testing. Interpretation of borings defined four soil layers from mudline to below expected pile toe elevations. The upper most strata contained interbedded layers of fine sand with silt and organic fines and shell fragments, calcareous silty sand, and silty to clayey fine sand. The thickness of this layer varies between 15 and 37 feet with N-values typically less than 10 blows per foot. Ocassionally, a medium dense layer was encountered. The estimated unit weight was 115 pounds per cubic foot (pcf) with no cohesion, and an average angle of internal friction of 27 degrees. The second layer had a thickness of 10 to 30 feet and consisted of sandy, medium to stiff clay with lenses of calcareous clayey fine sand and shell traces. This layer had N-values between 0 and 30 although most were typically below 10, and a unit weight of 115 pcf. Field vane shear tests in this interval showed an average undisturbed shear strength of about 3.10 ksf and an average disturbed shear strength of 1.34 ksf. Unconsolidated, undrained triaxial tests showed an average shear strength of 1.24 ksf with no angle of internal friction. These two upper layers together were typically about 45 to 50 feet thick. The third layer was 5 to 15 ft thick and contained medium dense to dense silty sand with shell and gravel size limestone and cemented sand fragments. It had an estimated unit weight of 125 pcf, no cohesion and an average internal friction angle of 35 degrees based on N-values between 15 and 30. The bottom layer was a dense to very dense silty fine sand with N-values ranging from 30 to more than 100. Most were greater than 50 per foot. The estimated soil parameters for this layer were a unit weight of 125 pcf, an angle of internal friction of 40 degrees, and no cohesion. For the purposes of analyzing the test pile data, layers 1 and 2 were combined. Therefore, subsequent reference to the soil layers will be to Layers I, II, and III as illustrated in the idealized profile presented in Figure 1.

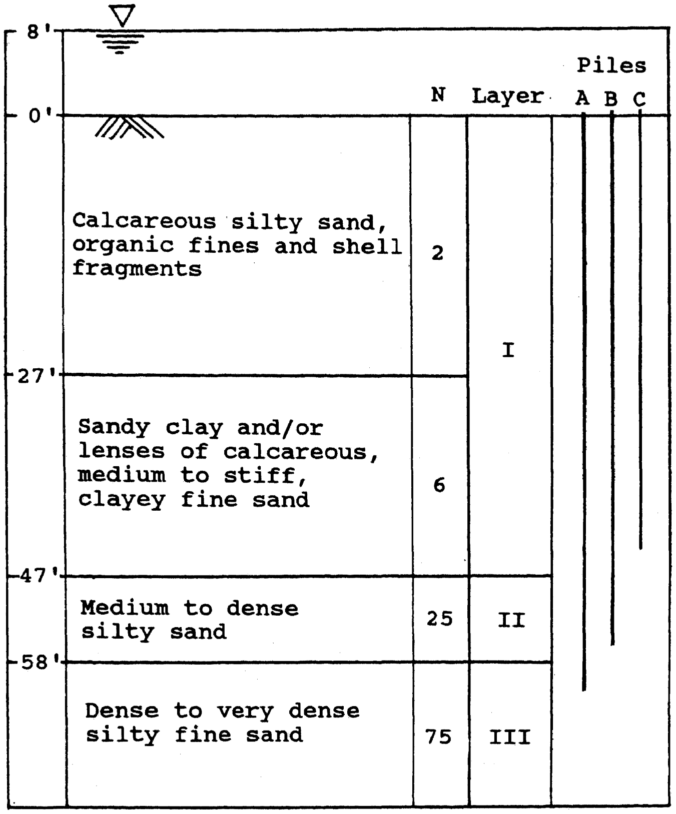


FIGURE 1 Idealized general soil profile

3. PILE INSTALLATION

Five test site locations (Sites 1 through 5) that represent subsurface conditions along the project length were chosen. A total of 11 test piles were driven. Piles whose tips were stopped in soil layers I, II, and III are referred to as C, B, and A piles, respectively as illustrated in Figure 1. Sites 1 and 5 each had one 30-inch square, A-type pile. Sites 2 and 3 each had three 24-inch square piles, one each of the A-, B- and C-type. Site 4 had three 30-inch square piles, one of each type. Piles are distinguished by their site number and pile type (e.g., 1A, 2C, etc.). All piles were initially driven during the first week of November, and some were restruck several times after intervals of up to 13 days thereafter.

The prestressed concrete test piles were cast between October 13 and 24th. The 30-inch piles had 28, 1/2 inch steel strands each tensioned with 28.1 kips and the 24-inch piles had 24, 1/2 inch strands prestressed to 29 kips each, both pile sizes had the same No. 5 gauge spiral ties. Concrete design strength was 6 ksi at 28 days. Sample concrete cylinders made from each pile at the time of casting were tested on the days of pile driving and static load testing. At the day of driving, two to three weeks after casting, concrete strengths were between 6.1 and 7.6 ksi, and the elastic moduli were between 3900 and 4400 ksi. Telltale casings were cast into the piles with the intention of determining load distribution and soil adhesion from static load tests.

Pile driving and restriking were accomplished with a Conmaco 5300-E5 single acting air hammer. The parti ular hammer used had a ram weight of 30 kips and wa fitted with a cam rod allowing it to operate at 2 and 4 strokes, corresponding to rated energies of 60 and 12(kip-ft, respectively. The hammer cushion was six inch of blue polymer and was "cooled" during pile driving w water injected into the upper side of the pile cap. The effect of introducing this "cooling" water on energy transfer will be discussed. New sheets of plywood wi total initial thickness of 6.75 inches and the same size the pile were used in each case. Pile cap weights wer 6.1 and 7.0 kips for the 24- and 30-inch piles, respectively. In some cases, the entire pile installation was accomplished with a 2-foot hammer stroke, while in others, the stroke was changed to 4-foot when driving became relatively hard. Table 1 lists the end of drivin hammer stroke and driving resistance in blows per foo (BPF) for each pile. This table also includes results fro dynamic testing which will be discussed further.

At each site the A-type piles were driven first into Lay III to obtain the project target compressive capacity ar examine pile drivability. Piles of the B- and C-type wer then driven in that order to a specific elevation based (the driving record of the A pile and boring information the site.

4. DYNAMIC TESTING AND ANALYSES

Preliminary wave equation analyses using the GRLWEA program (1), performed prior to mobilization at the site indicated that the proposed Conmaco hammer (with its variable stroke) was capable of driving both pile sizes (all three types to targeted depths and anticipated capa ties without pile overstressing.

The installation and restrikes of all eleven piles were monitored using a Pile Driving Analyzer (PDA) accordin to the Case Method procedures (2). Dynamic measure ments of strain and acceleration were taken four feet below the head of each pile. Two reusable strain trans ducers and two piezoelectric accelerometers were bolt at opposite sides of each pile to monitor and minimize (by averaging) the effects of non-uniform hammer impacts. Strain and acceleration signals were conditione and converted to force and velocity records by the PD Figure 2 shows pile head records of force and velocity (multiplied by pile impedance) under hammer blows fro the end of driving of Piles 2A, 2B, and 2C. Dynamic records from the end of driving of a 30-inch (Pile 1A) a a 24-inch pile (Pile 3A) are presented in Figure 3. Illus trated in Figure 4 are plots of pile head dynamic record from the end of driving and beginning of restrike, five days after installation, of Pile 3B.

In the field, the PDA interpreted measured dynamic dat according to the Case Method equations. The data wa evaluated for pile driving stresses (compressive and

PILE	SIZE (IN)	LENGTH (FT)	PEN (FT)	BLOW COUNT (BL/FT)	STROKE (FT)	EMX (K-FT)	CSX (KSI)	TSX (KSI)	FMX (KIPS)	STATIC CAPACITY TOTAL (KIPS)	SKIN (KIPS)	TOE (KIPS)	SOIL DAMPING SKIN (S/FT)	TOE (S/FT)	SOIL QUAKES SKIN (IN)	TOE (IN)
1A	30	84	63.5	163	4	55.5	2.5	1.4	2250	1330	189	1141	.26	.06	.08	.42
2A	24	80	54.9	110	4	47.5	2.3	0.7	1330	1264	207	1057	.12	.06	.10	.37
3A	24	83	58.1	116	4	57.0	2.9	1.0	1650	1355	242	1112	.10	.05	.12	.41
4A	30	89	65.9	106	4	59.0	2.7	1.2	2450	1376	201	1175	.20	.08	.10	.41
5A	30	87	66.9	91	4	67.0	3.0	1.1	2700	1563	403	1160	.15	.09	.11	.30
2B	24	74	51.9	16	4	55.5	2.6	0.8	1520	640	186	454	.09	.07	.12	.61
3B	24	78	53.8	15	2	33.0	1.8	0.7	1015	354	78	276	.12	.03	.13	.80
4B	30	84	57.9	18	2	30.0	1.6	0.8	1450	346	87	259	.11	.06	.10	.68
2C	24	68	45.1	12	2	37.5	2.2	0.6	1245	240	210	30	.12	.08	.13	.50
3C	24	68	44.3	5	2	31.0	2.0	0.7	1180	75	36	39	.20	.16	.10	.60
4C	30	68	46.1	6	2	26.5	1.4	0.5	1280	92	78	14	.28	.12	.10	.11

PEN ... Pile penetration below mudline
EMX ... Maximum transferred energy to pile head
CSX ... Maximum compressive stress at pile top
TSX ... Maximum pile tension stress throughout pile driving
FMX ... Maximum compressive force at pile top

TABLE 1 Pile driving data and dynamic testing results
(For End of Initial Driving - EOD)

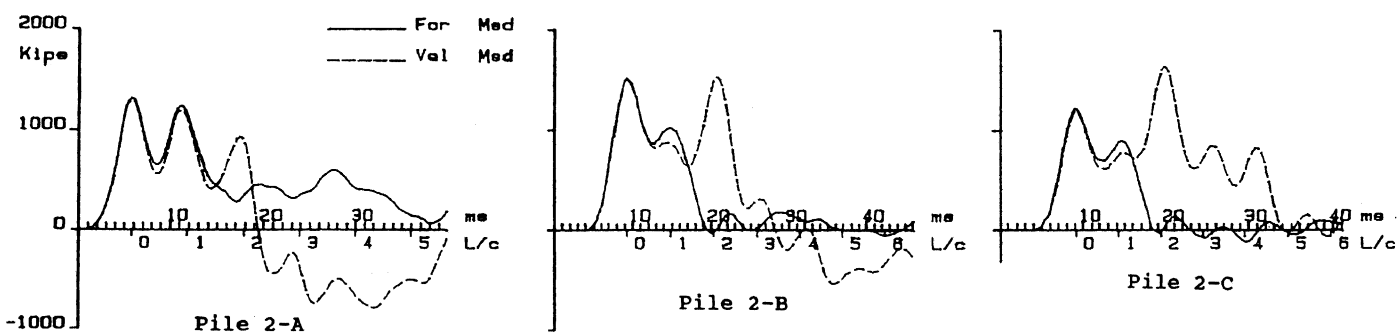
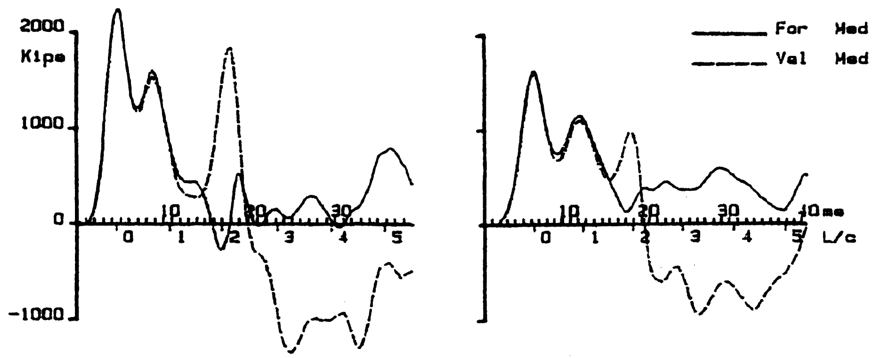


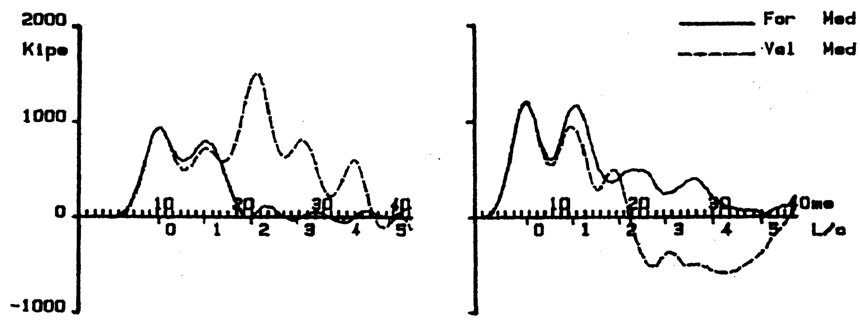
FIGURE 2 Pile head records;
Force vs Velocity, Site 2 @ EOD



Pile 1-A, 30-inch

Pile 3-A, 24-inch

FIGURE 3 Pile head records;
Force vs Velocity, Piles 1A and 3A @ EOD



End of Driving

Beginning of Restrike
(5 Days)

FIGURE 4 Pile head records;
Force vs Velocity, Pile 3B @ EOD and BOR

ensile), structural integrity, and static bearing capacity. ·lammer-driving system performance was also investigat- ·d. Field measured dynamic data was additionally ana- yzed using the CAse Wave Analysis Program (CAPWAP) vhich is an analytical procedure performed interactively ·etween the engineer and the computer program using a nicro-computer (3). This method is used to compute soil esistance forces and their distribution using pile head orce and velocity measurements recorded in the field in ı wave equation type procedure. Results from a CAP- VAP analysis include comparisons of measured with the :orresponding computed pile force/velocity records. Jumerically for each segment (approximately 5 ft) of the ·ile, ultimate static resistance, soil quake and damping actors are tabulated. Also included in the results is a ·ile head and toe load-set relationships computed from ;tatic test simulations. Figure 5 presents a typical CAP- VAP analysis plotted results (end of driving of Pile 1A). \ total of 32 CAPWAP analyses were performed on data rom the end of driving and restrikes of all test piles.)ynamic testing results during pile installations are sum- narized in Table 1 and all dynamic analysis results are liscussed below.

)uring installations, measured pile stress wave speeds ıveraged 12,500 ft/s which corresponds to a dynamic ·lastic modulus of 5055 ksi with a material unit weight ·f 150 pcf. This dynamically determined elastic modulus ınder loading rates between 500 to 1000 kips/ms is an ıverage 20% higher than that determined from concrete :ylinder crushing tests performed at the day of driving ·ach pile.

The maximum compressive stress at pile heads during installation averaged 1.8 and 2.7 ksi with hammer strokes of 2 and 4 ft, respectively while the allowable stress was calculated to be about 3.6 ksi. Pile shaft tension stresses averaged 0.7 ksi with 2 ft ram strokes and 1.1 ksi (but reached 1.4 ksi for Pile 1A) with 4 ft strokes as compared to the calculated allowable stress in tension of about 1.2 ksi. During restrikes, compressive stresses were slightly higher and tension stresses substantially lower than those during initial driving. Dynamic data from all piles did not reveal indications of pile damage below gages, however, minor spalling did occur at the top of several piles.

While on short stroke (2 ft), the hammer-driving system delivered an average of 29 kip-ft of energy to pile heads and with 4 ft stroke it delivered an average transferred energy of 57 kip-ft. These energy transfer values correspond to transfer efficiencies of 48% when compared to the corresponding hammer rated energies. When comparing this energy transfer efficiency value to many others obtained on projects under similar conditions, it indicates a hammer-driving system performance in the top 35% of the sample. It was found that the injection of water into the upper side of the pile cap decreased pile head transferred energies by 5 to 10%, while shutting the water off increased the energy by a similar percentage.

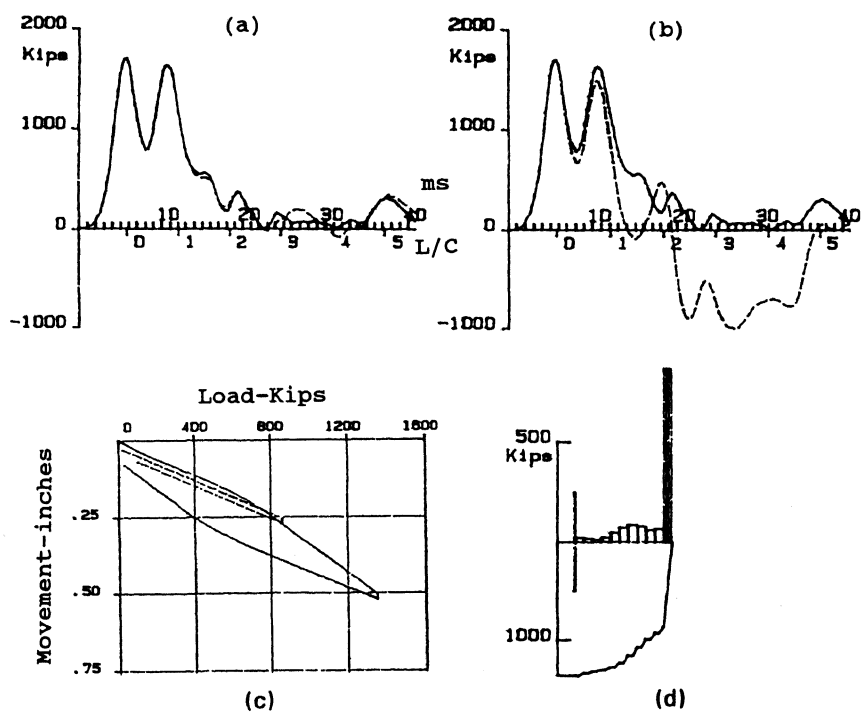


FIGURE 5 Typical CAPWAP analysis output; Pile 1A
(a) Measured and computed forces
(b) Measured force and velocity
(c) Load vs movement at pile head
(d) Resistance distribution and forces in pile shaft at ultimate resistance

For determination of pile static capacities, soil resistance distributions and dynamic variables, CAPWAP analyses were performed with dynamic data representing hammer blows from the end of driving and beginning of restrikes of all piles. For restrikes that consisted of a significant number of blows resulting in notable pile movement. End of restrike data was also analyzed. Comparisons show that PDA field calculated capacities were, on average, within 6% of CAPWAP computed values. Table 1 lists end of driving pile static capacity (total, shaft and toe resistances), soil damping and quake values along pile shafts and under pile toes for each test pile. As expected, A piles had the most end of drive capacity (average 1378 kips) and C piles the least (average 136 kips) with B piles in-between (average 447 kips). Dynamic testing during restrikes were performed to assess time dependent pile capcity changes. In all cases, pile capacities increased with time. End of driving blow counts of the A piles were generally over 100 BPF with higher values encountered at the beginning of restrikes. High pile driving resistance means small pile sets under hammer blows preventing full mobilization of soil resistance. A method of superpositioning, assuming no change in pile toe resistance with time, was employed as illustrated in Figure 6 for Pile 2A. The figure presents the CAPWAP calculated pile static capacity and shaft and toe resistances at the end of driving and during restrikes. At the beginning of each restrike, the hammer was unable to cause sufficient pile movement to mobilize maximum pile resistance, therefore the toe resistance from the previous end of driving analysis was added to beginning of restrikes shaft resistance to compute total pile capacity. Interestingly, at the end of the five day restrike (the restrike consisted of 38 hammer blows), the total pile driving resistance was the same as that at the end of initial driving and pile resistance magnitude and distribution were almost identical (within 2%) to those at the end of driving. Due to the low blow counts at the end of driving and during restrikes of the B and C piles, pile capacities were always mobilized and increases in values were entirely due to increases in shaft resistances.

CAPWAP computed shaft resistances for the B piles are plotted as a function of time (log time scale) along with results (according to Davisson's failure criteria) of uplif static loading tests in Figure 7. Data analysis indicates linear increase in piles shaft resistance and an uplift static capacity that is, on average, 76% of compressio shaft resistance. Due to the very low driving resistanc of the C piles and the fact that static loading tests wer not run to failure on almost all of the A piles, similar comparative analyses were not performed for these pile

5. STATIC LOADING TESTS

5.1 Procedure

All eleven piles were subjected to uplift static loading tests during the month of December. Four, 18-inch diameter steel pipe reaction piles were used at each single test pile site and eight piles were used at each three test pile sites. Test piles were cut off and thread-bars were exposed (4 and 6 for 24- and 30-inch piles, respectively) after the working platform was put into place. The load test frame consisted of a pair of cross beams supported on the reaction piles and twin girders placed onto the cross beams, over the test pile. The threadbars were then extended and connected to the tw 600 kips (1200 kips total) hydraulic jacks which were placed on the test girder. Tension loads were applied in increments of approximately 5% of each pile's estimate (from CAPWAP) capacity. Maximum test loads were based on 80% of the maximum threadbar capacity of each pile size (i.e., 600 kips for 24-inch and 900 kips fo 30-inch piles). Loads were removed in increments of 20% for rebound readings.

Three dial gauges (read to 0.001 inch) were placed at th top of the pile to measure top displacement. Dial gauge were also placed on the steel telltale rods to measure differential pile movements at various levels. A wire lin and mirror system was additionally used to check dial gauge readings, and a remote platform was established to verify that the reference system was unaffected by test loadings.

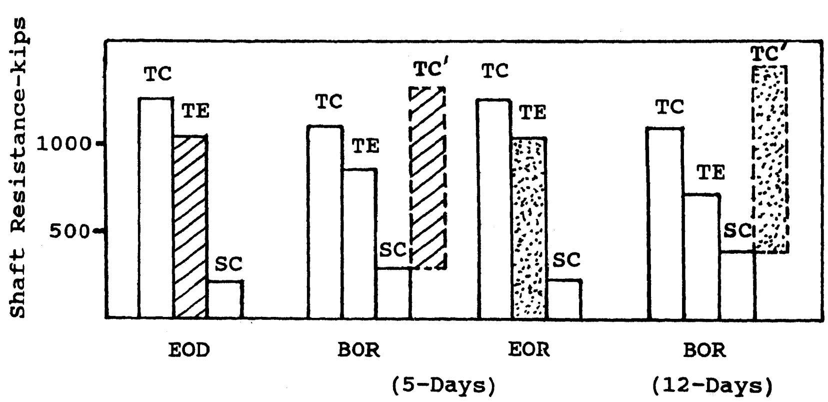


FIGURE 6 Superpositioning of CAPWAP data; Pile 2A where:
TE = Total capacity
TE = Toe capacity
SC = Shaft capacity

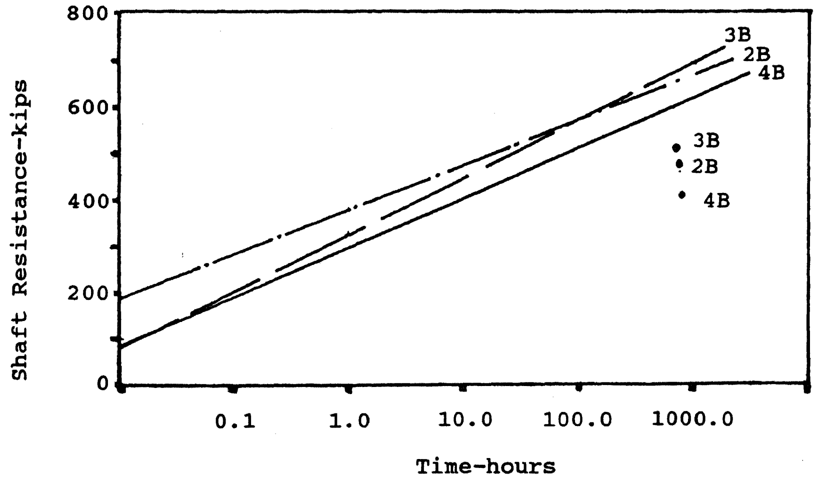


FIGURE 7 Shaft Resistance vs Time plotted with corresponding uplift shaft capacities

5.2 Results

Load versus displacement curves were plotted from which failure loads for each pile could be determined. As an example, Figure 8 presents plots of pile head load vs movement for A-piles,B-piles and C-piles, respectively. Five different methods were utilized to define failure loads. These methods were:

* Davisson's Method [elastic elongation + 0.15" + D/120]
* Canadian Method [elastic elongation + D/30]
* Tangent Intersection Method
* Specific Displacement [elastic elongation + 0.25"]
* Specific Displacement [elastic elongation + 0.10"]

Nhere D is pile side dimension.

\lthough meant to be used for compression static loadng test data interpretation, the methods listed above vere employed unmodified due to the lack of a universaly accepted procedure for uplift static load test results ıssessment. It is recognized in particular, that the "D," actor in the Davisson and the Canadian Methods probaıly should be disregarded. Without that factor, the)avisson failure load would be equal to elastic elongation +.15 inch displacement which is between the two spe:ific displacement methods applied. The Canadian Methıd would not be applicable. With it, the displacement :riteria for Davisson's Method are elastic elongation +.35 inch or .40 inch for 24 inch or 30 inch square piles espectively. For the Canadian Method, displacements re elastic elongation +.80 inch or 1.00 inch for 24 inch nd 30 inch piles respectively. It was decided to apply hese methods in their common form to allow examinaion of a broader range of displacement criteria. With the xception of Pile 2A, all other A-type piles did not fail by ny of the these methods. Load-movement curves for A iles were extrapolated using test data and patterns of isplacements in failed Pile 2A. Figure 9 illustrates the pplication of these methods on the load vs displacement lot for Pile 2A.

The gross failure loads (including pile weights) produced by the above methods are listed in Table 2 along with the number of days elapsed between installation, redriving and testing. The results with Davisson's Method were selected to represent the piles' ultimate capacity in final design. Total pile top displacements associated with the application of this method were between .50 and .70 inch which was believed to be reasonable for an ultimate condition. Table 3 presents net failure loads (excluding pile weights) using this method for all piles. The table shows that the 30-inch A piles had the most capacity and the 24-inch C piles the least; inconsistencies, however, include the facts that the 30-inch B pile had less capacity than the 24-inch B piles, and that 24-inch A and B piles had about the same capacities.

5.3 Data Analysis

Predicted pile capacities were calculated using Nordlund's method for cohesionless soil intervals and the alpha method for cohesive soil intervals (4). These methods include a number of factors which require some judgement, particularly the selection of an alpha factor and the soil shear strengths. The difficulties associated with the selection of an alpha factor for medium to stiff clays are discussed in the literature (4). The shear strengths results from 7 undisturbed, field vane shear tests in the lower portion of Layer I varied between 2.08 and 4.02 ksf with an average value of 3.10 ksf. In those tests the ratio of undisturbed to disturbed strengths was 2.4. The results of thirteen laboratory unconsolidated, undrained triaxial shear strength tests in that same soil interval varied between .44 ksf and 2.5 ksf with an average of 1.24 ksf. Selection of a shear strength in these cohesive soils or the selection of a friction angle in cohesionless soils for evaluation of frictional resistance, introduces some level of probable variation when compared to field load test results. Estimates of pile capacities in Layer 1 were made using an alpha of .35 and an

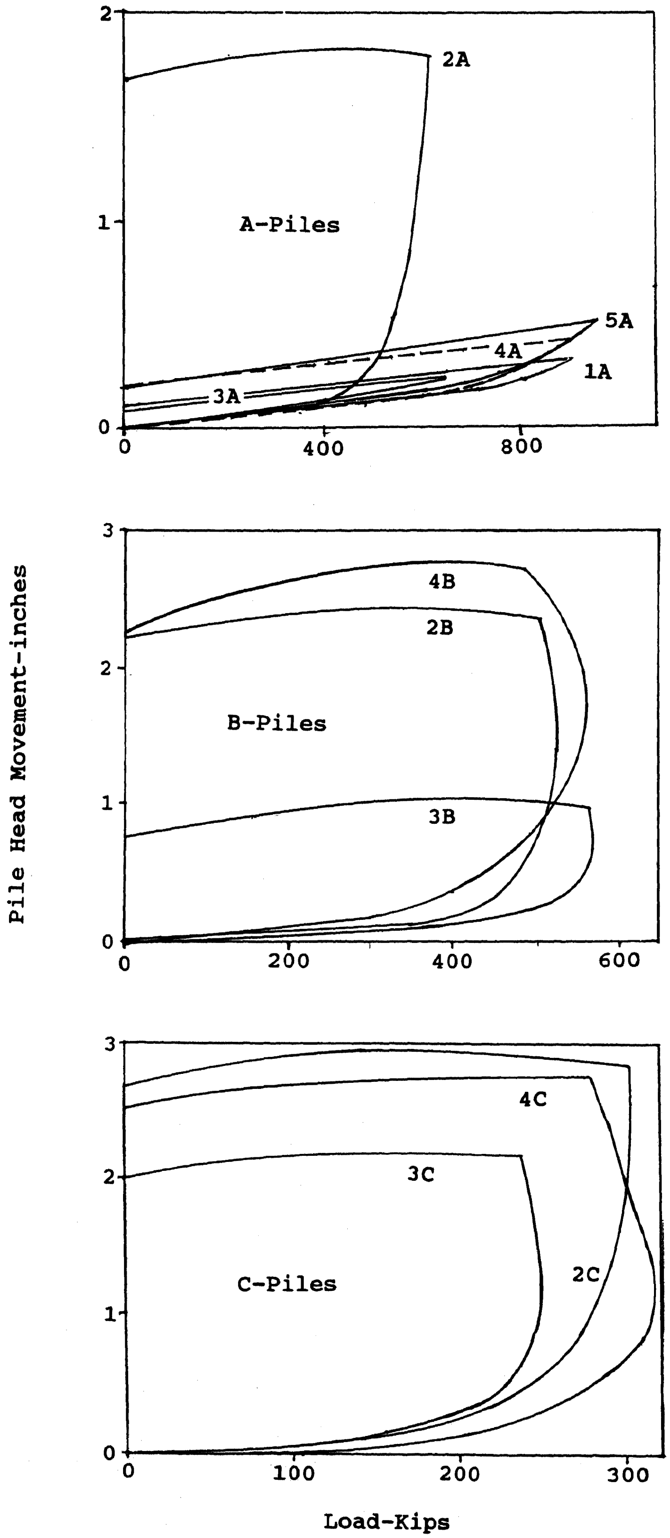


FIGURE 8 Pile Head Displacement vs Applied Load; For A-Piles, B-Piles and C-Piles

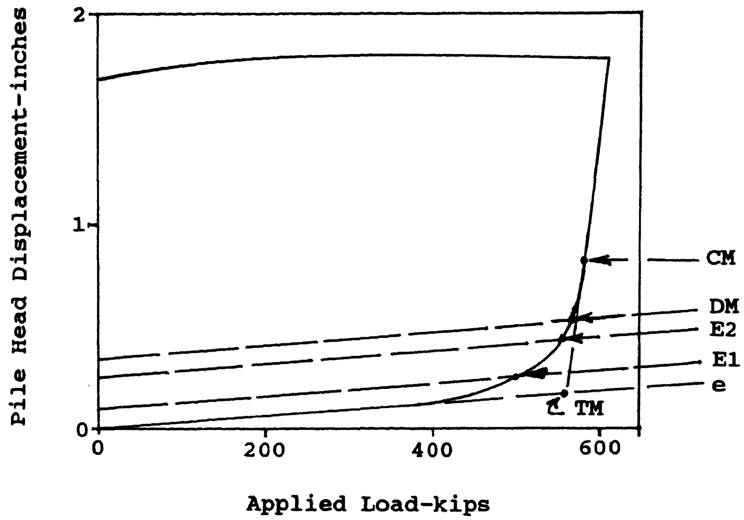


FIGURE 9 Typical Application of Failure Load Methods; Pile 2A, where:
e = Elastic elongation
E1 = e +.10 inch
E2 = e +.25 inch
DM = e +.15 inch + D/120 (Davisson's Method)
CM = e + D/30 (Canadian Method)
TM = Tangent Method

PILE	SIZE (inch)	TIME* (Days)	DAVISSON (Kips)	CANADIAN (Kips)	TANGENT (Kips)	e +.25" (Kips)	e +.10" (Kips)
1A**	30	50/43	1000	1080	1000	950	900
2A	24	36/24	570	590	560	560	500
3A**	24	37/31	610	610	600	590	550
4A**	30	49/36	950	1020	980	900	840
5A**	30	50/43	990	1050	1000	950	850
2B	24	36/24	485	515	490	470	440
3B	24	34/29	560	560	490	560	500
4B	30	49/36	430	530	450	380	360
2C	24	36/25	235	270	245	210	165
3C	24	35/30	220	240	170	200	160
4C	30	49/36	280	320	245	240	210

* Time between initial driving and testing/last restrike and testing
** Estimated failure load
e Elastic elongation

TABLE 2 Gross uplift failure loads by selected methods

PILE	SIZE (Inches)	NET FAILURE LOAD (Kips)
1A	30	950
2A	24	542
3A	24	580
4A	30	902
5A	30	938
2B	24	456
3B	24	532
4B	30	386
2C	24	208
3C	24	196
4C	30	242

TABLE 3 Net failure loads from uplift tests (Davisson's Method)

average shear strength of 3.0 ksf in the cohesive portion and an average friction angle based on Standard Penetration Test (SPT) data in the sandy portion. The same method was used to estimate capacity from Layers II and III since grain size analyses indicated they were also cohesionless. Failure loads for C-piles using the Davisson criteria were, on average, 73% of the computed estimates for pile ultimate frictional resistance. For piles 2B and 3B, the the net failure load was 123% of the computed ultimate resistance. Pile 4B is unusual in that this ratio was 56%. It is suggested that the variation in the C-piles may be attributable to the selection of an alpha factor or a shear strength for computation of a capacity. It is also suggested that for the B-piles, the higher ratio could be attributable to some type of cementation of the calcareous soils in Layers II and III. When these layers were treated as cohesionless soils, computation of their capacity was considerable underpredicted.

Unit soil adhesion values were computed for each soil layer using net failure loads from uplift tests and embedment lenghts in Layers I, II, and III from Piles A, B, and C at Sites 2, 3, and 4. Adhesion values for Layer I was determined from C piles. Unit adhesion for Layer II was calculated by subtracting the product of the unit adhesion and the estimated penetration in Layer I from the failure loads of the B piles. Similarly, calculation of an independent unit adhesion for Layer III from the A piles yielded unreasonable results. This was attributed to the minimal amount of penetration in this very dense layer. A reasonable match was obtained when the penetration lengths in Layers II and III were combined and a corresponding adhesion was determined.

An average adhesion value of 0.56 ksf was obtained for Layer I, and an average adhesion of 4.54 ksf was obtained for pentration into Layers II and III. These average adhesion values were also used to compute uplift capacities of Piles 1A and 5A. Results agreed within 10% of the measured values from the load tests (using Davisson's failure load). These adhesion values appeared to be representative of the soils encountered and were recommended for use as ultimate values in determining estimated uplift pile capacities for final design o the project.

Data from telltale movements was analyzed for computing adhesion forces from differential pile elongations considering applied loads and pile properties. This analy sis yielded unrealistic results that were deemed unusable. It was concluded that the lack of sensitivity of th telltales and the large pile area cross sections were among the reasons for failure of this approach.

6. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions were arrived at from the results of dynamic and static load tests at this site:

a) Dynamic testing can be used to evaluate pile capacity increases in skin friction due to time related increases in pile-soil adhesion.

b) The method of "superpositioning" can be used with CAPWAP results to determine the total capacity on high capacity piles which exhibit little pile toe movement when restruck.

c) The injection of cooling water at the pile hammer cushion during driving reduced energy which would otherwise be delivered to the pile by 5 to 10 percent.

d) Taking into account time factors when determining capacities from the uplift load tests, the ultimate test load on "B" piles determined by Davisson's method as described in this program was, on average, 76 % of the CAPWAP predicted compressive skin friction.

e) Prediction of frictional resistance based on generally accepted computational methods provided erratic and inconsistent results for the "B" and "C" piles. This suggests the need for uplift testing on projects where tension loads are significant in design or the use of appropriate factors of safety in other cases. The use of dynamic testing for this purpose offers potential that should be explored further.

f) For design at this site the recommended ultimate adhesion values for uplift capacity on piles was 0.56 ksf in Layer I and 4.54 ksf in Layers II and III.

g) Telltales did not provide reliable data for use in determining load distribution. This was attributed to several factors particularly,the sensitivity of calculations to the large pile area and the level of sensitivity in telltale data.

7. ACKNOWLEDGMENTS

In addition to their own organizations, the authors would like to acknowledge the encouragement and support of the Florida Department of Transportation (Bartow and Tallahasse Geotechnical Engineers' Offices) and of Misener Marine Construction, Inc. during the field work and in preparing this manuscript.

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. APPENDIX

ft = 0.305 m, 1 inch = 2.54 cm, 1 kip = 4.45 kN, 1 si = 6.89 MPa, 1 kip-ft = 1.36 kJ