PILE FOUNDATION CONSTRUCTION FOR THE BUCKMAN BRIDGE JACKSONVILLE, FLORIDA

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ABSTRACT

This paper describes the geotechnical issues related to pile foundation construction for the widening of the I-295 Buckman Bridge over the St. John's River in Jacksonville, Florida. The project included a total of 3,351 prestressed concrete piles, with sizes up to 30-inches (762-mm) square and design capacities up to 205 tons (1,820 kN). The paper highlights the application of dynamic testing as a routine and essential quality control measure during construction, and presents summaries of the dynamic test data obtained. Issues addressed include increased pile capacity with time after driving, unit friction and end bearing values, hammer efficiency, driving stresses, and dynamic parameters for soil and limerock.

KEYWORDS

Bridges, Construction, Dynamic Testing, Foundations, Limerock, Non-destructive Testing, Piles, Pile Driving, Sand.

INTRODUCTION

The Buckman Bridge, located south of Jacksonville, Florida, carries Interstate I-295 over the St. John's River at a location where the river is approximately three miles (4.8 km) wide. The existing bridge, completed in 1971, consisted of twin bridge structures, each approximately 16,300 feet (4,968 m) long. Each of the existing structures had two travel lanes and two narrow shoulders, providing a total roadway width of 30 feet (9.14 m). To relieve traffic congestion and improve safety on this heavily used crossing, the Florida Department of Transportation (FDOT) implemented a program for widening the bridge. This construction began in February, 1993 and was completed in March, 1997. The widening more than doubled the width of the roadway to 68 feet (20.7 m), and provided each structure with four travel lanes and two 10-foot (3.0-m) wide emergency lanes.

Each of the twin bridges include a total of 84 bents and 19 piers west of the navigation channel, and 94 bents and 19 piers east of the navigation channel. At the navigation channel, the main spans are 250 feet (76.2 m) and provide a vertical clearance of 65 feet (19.8 m). A longitudinal profile along the bridge alignment is shown in Fig. 1.

The 76 new piers are founded on 30-inch (762-mm) square, hollow-core prestressed concrete piles. The hollow core is 14 inches (356-mm) in diameter and extends the full length of the pile. The design capacities of these piles are 200 and 205 tons (1780 and 1820 kN) in compression, and 125 and 145 tons (1110 and 1290 kN) in tension. The high tension loads are the result of ship impact loading criteria. Each pier contains 10 to 40 piles. Figure 2 presents a plan of one of the four pier foundation arrangements.

The 356 new bents are founded on 18-inch (457-mm) and 24-inch (610-mm) square prestressed concrete piles and 30-inch (762-mm) square, hollow-core prestressed concrete piles. The 18-inch (457-mm) piles are used only at the end bents, and the 30-inch (762-mm) piles are used for the bents nearer to the center of the crossing. Each bent consists of four or five piles in a single row. Except for the 18-inch (457-mm) piles, which have a design capacity of 80 tons (710 kN), the piles at the bridge bents have a design compression capacity of 133 tons (1180 kN). No tension loads will be applied to these piles. The piles were installed with a center-to-center spacing of 7.6 to 10.1 feet (2.31 to 3.08 m), and at 6:1 batter.

SUBSURFACE CONDITIONS

Water depths at the crossing generally range from 10 to 23 feet (3 to 7 m). The underlying strata consist of a) soft river bottom deposits (muck), b) sand with interbedded clay layers, and c) limerock. Figure 3 presents a generalized subsurface profile along the bridge alignment.

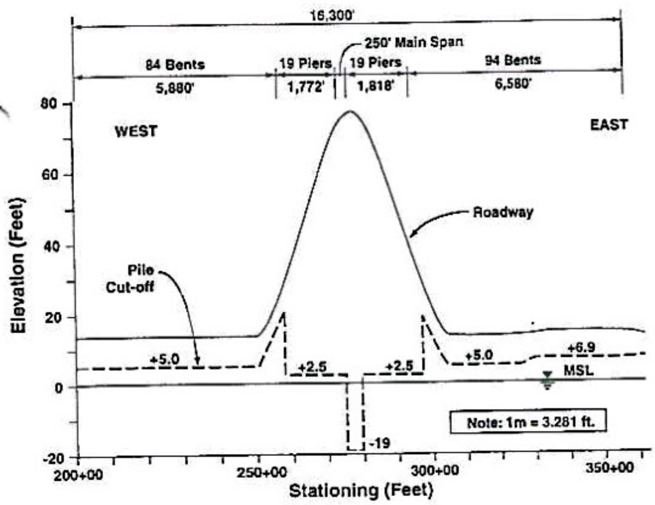


Fig. 1 Bridge profile

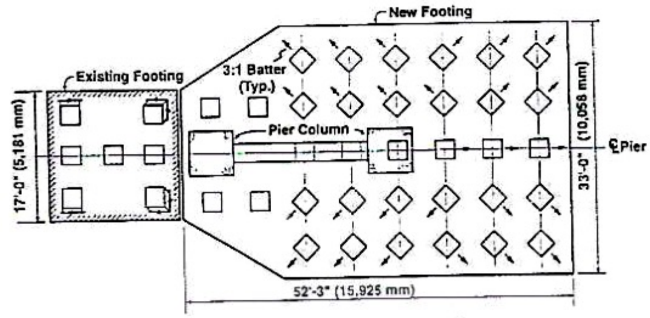


Fig. 2 Foundation plan for one of four pier arrangements

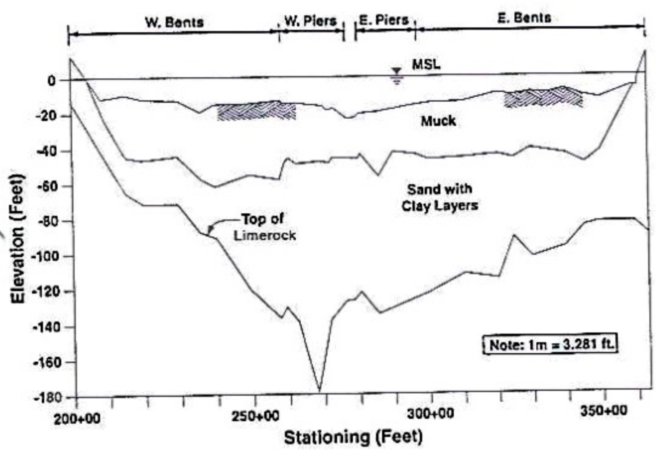


Fig. 3 Subsurface profile along bridge alignment

The soft river bottom deposits consist of organic silts and clays which were typically penetrated by the weight of the sampling hammer. The underlying sand stratum consists of loose to very dense fine sand with a relative density that generally increases with depth. The clay interbeds within the sand stratum are discontinuous, have thickness generally less than 15 feet (5 m), and are medium stiff to very stiff in consistency.

The limerock is generally soft, but contains interbeds of hard limerock and intermittent lenses of sand, shell and clay. Standard penetration test N-values in the limerock generally ranged between 100 blows per foot (per 300 mm) and 100 blows per inch (per 25 mm). The average unconfined compressive strength obtained from tests on intact core samples was 1,200 psi (8,280 kPa).

Table 1 Summary of pile driving equipment

Location/ Pile Size inch (mm)	Hammer Model	Maximum Stroke feet (m)	Max. Rated Energy ft-kips (kN m)
Piers			
30 (762)	Conmaco 300E5	3.5 (1.07)	105.0 (142.4)
30 (762)	Conmaco 300E5	4.0 (1.22)	120.0 (162.7)
Bents			
30 (762)	Conmaco 300E5	3.5 (1.07)	105.0 (142.4)
30 (762)	Delmag D62-22	11.3 (3.44)	165.0 (223.7)
24 (610)	Delmag D46-32	10.6 (3.23)	107.2 (145.4)
18 (457)	Delmag D46-32	10.6 (3.23)	107.2 (145.4)

Note:
The 300E5 used an 8-inch (200-mm) hammer cushion and a 9-inch (225-mm) plywood pile cushion.
The D62-22 used a 3.5-inch (89-mm) aluminum-micarta hammer cushion and a 9-inch (225-mm) pile cushion.
The D46-32 used a 3.5-inch (89-mm) aluminum-micarta hammer cushion and a 12-inch (300-mm) pile cushion.

PILE INSTALLATION CRITERIA

Pile order lengths were determined from static analyses using the available boring data, and were verified from the results of the initial testing program discussed below.

Wave equation analyses were performed to establish the driving resistance criteria for the various driving equipment used by the contractor (see Table 1), the different size and capacity piles, and the typical subsurface conditions along the bridge alignment. Separate criteria were developed for both the end of initial driving and for beginning of restrike.

The installation criteria also required all piles to be jetted to 10 feet (3 m) above the tips of the existing piles to facilitate installation and limit the influence of driving operations on the existing structures.

At the bents, a minimum tip elevation criterion required the new piles to be founded at or below the tip elevation of the lower capacity existing piles to avoid potential post-construction settlement of the new and existing foundations. At the pier foundations, a minimum tip elevation was specified to assure that the piles had sufficient driven length to develop the required tension capacity.

INITIAL TEST PROGRAM

At the beginning of construction a total of 44 index piles were driven using the same driving equipment proposed for installation of the production piles. Dynamic monitoring using the Pile Driving Analyzer® (PDA) was performed during the driving and redriving of all piles to better define pile capacity and to determine appropriate dynamic soil properties for wave equation analyses. The test pile program also included static load tests on four of the 30-inch (762-mm) piles. Following are the primary conclusions obtained from the initial test pile program (Schmertmann & Crapps, Inc., 1993).

- The total pile capacity estimated from the Case Pile Wave Analysis Program (CAPWAP®) compared well with the results of the static load tests. For the four piles tested, the total capacity estimated by CAPWAP using restrike data ranged from 77.3 to 116.5 percent of the static test failure load, with an average of 97.0 percent.

- The soil "freeze", or increase in soil resistance with time after driving, ranged from zero to 94 percent, and averaged about 20 percent, for piles redriven within 12 hours after the end of initial driving, and ranged from zero to 108 percent, and averaged about 31 percent, for piles redriven after more than 12 hours.

- The dynamic soil parameters (damping and quake) estimated from the CAPWAP analyses were slightly higher than the typical values reported in the literature.

PRODUCTION PILES

The 18-inch (457-mm) and 24-inch (610-mm) piles for the west bents were generally driven to end bearing on the limerock layer (see Fig. 3). The remaining piles generally obtained their capacity from a combination of friction and end bearing in the sand stratum.

Table 2 summarizes the average driven pile lengths for the different pile sizes. For the 18 and 24-inch (457 and 610-mm) piles, the lengths were typically shorter on the west side of the channel due to the shallower depth to limerock. The maximum lengths of the 18 and 24-inch (457 and 610-mm) piles were 92 and 119 feet (28.0 and 36.3 m), respectively. For the 30-inch (762-mm) piles the lengths were typically greater on the west side of the channel where the sand layer was thicker (see Fig. 3). The maximum length of the 30-inch (762-mm) piles was 140 feet (42.7 m).

Except for the four piers adjoining the navigation channel, all piles had a cut-off level above the water surface. At these four piers the piles were installed within a cofferdam to a cut-off level at El. -19 feet (-5.8 m) as shown in Fig. 1. A follower, consisting of a steel cylinder with an outside diameter of 22 inches (560 mm) and a wall thickness of 2-1/8 inches (54 mm), was used to drive these piles below water level. The follower length was 20 feet (6.1 m) at one pier and 5.4 feet (1.65 m) at the remaining three piers.

Table 2 Summary of pile quantities

Location/ Pile Size inch (mm)	Number of Piles	Total Driven Length feet (m)	Average Length feet (m)
Piers			
30 (762)	1,808	188,918 (57,579)	104.5 (31.9)
Bents			
30 (762)	360	39,862 (12,149)	110.7 (33.7)
24 (610)	1,160	99,276 (30,258)	85.6 (26.1)
18 (457)	23	1,267 (386)	55.1 (16.8)
TOTAL	3,351	329,323 (100,373)	

DYNAMIC TESTING PROGRAM

Dynamic testing was performed as a quality control measure during production pile driving operations (Hussein and Likens, 1995). The primary purposes of the dynamic testing program were to a) verify pile capacity, b) verify pile driving criteria, c) confirm that driving stresses in the piles remained within allowable limits, and d) assess the performance of the pile driving equipment. These tests were particularly important for the Buckman Bridge Widening project considering the length of the bridge, the wide spacing between existing borings, and the variability in subsurface conditions. Also, since pile driving was performed over a period of more than two years, the dynamic testing program provided a means of assessing modifications to the pile driving equipment and any change in hammer performance with time.

The dynamic testing program included both routinely scheduled tests and contingency tests. The scheduled tests allowed an assessment of changes in subsurface conditions along the length of the bridge, as well as changes in hammer performance occurring as the work proceeded. These scheduled tests included one or more tests at each pier foundation, and one test for approximately every six bents. The contingency tests were performed when needed to evaluate new or modified driving equipment, or to evaluate unanticipated driving conditions. Dynamic testing was performed on 207 production piles, or approximately 6 percent of the total number of piles.

The PDA measured, processed, and recorded the strain and acceleration at gage points located near the top of the pile. Analysis of these data provided a record of driving stresses, estimated total pile capacity, maximum transfer energy to the pile, and an assessment of pile integrity, among other parameters, throughout the length of driving. For selected hammer blows, typically towards the end of initial driving or at the start of a redrive, the test data were further evaluated using the CAPWAP program. Using this program the stress wave characteristics of a computer model are matched with those measured in the field in an iterative process to define the dynamic properties of the soil, and to compute the static soil resistance and its distribution along the pile shaft and at the pile toe. At least one CAPWAP analysis was performed for each pile test.

Figure 4 presents plots of typical PDA dynamic records and CAPWAP analyses. Shown are the results for the end of driving (EOD) and beginning of restrike (BOR) of a 116-foot (35.4-m) long, 30-inch (762-mm) pile at one of the west piers. The restrike was performed 17 hours after initial installation. The pile penetration below mudline was 98 feet (29.9 m), and the driving resistances at the EOD and BOR were 5 and 10 blows per inch (13 and 25 blows per cm), respectively. The CAPWAP computed skin friction values were 147 and 646 kips (654 and 2,875 kN) for the EOD and BOR conditions, respectively. Figure 4 illustrates the change in the dynamic characteristics at the top of the pile, and the amount and distribution of skin friction resistances for the EOD and BOR conditions.

RESULTS OF DYNAMIC PILE TESTING

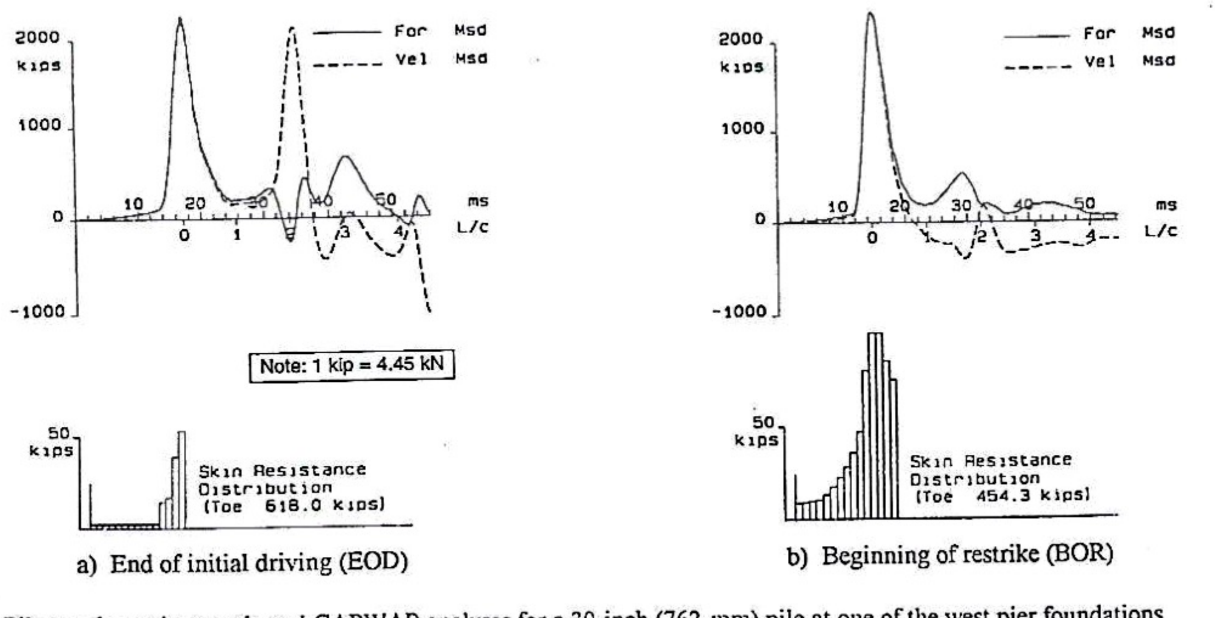
Following are summaries of the extensive test data obtained from the Buckman Bridge Widening project. In addition to documenting the findings for the Buckman Bridge site, these data may also be useful for the design and installation of piles for other projects with similar subsurface conditions.

Soil Freeze

Soil "freeze" is defined as the increase in soil friction along the sides of the pile with time after the end of initial driving. For the Buckman Bridge Widening project a soil freeze factor was computed from the CAPWAP analyses by dividing the total friction resistance at the beginning of redriving by the total friction resistance at the end of initial driving. Figure 5 presents the computed freeze factors plotted versus time.

In evaluating these data it should be noted that the restrike tests were generally performed on piles which did not meet the initial driving criteria. Accordingly, these piles are believed to be founded in looser sand deposits or cohesive soils which would be expected to exhibit a larger magnitude of freeze. With this consideration in mind, Fig. 5 shows that:

- The three piles restruck within about a half hour of the end of initial driving exhibited only a modest increase in friction resistance. The freeze factor for these piles ranged from 1.11 to 1.34, and were comparable to the results obtained from the initial test pile program.
- Piles restruck one or more days after the end of initial driving showed freeze factors generally ranging from 2.0 to 4.4, with an average value of 3.5.



a) End of initial driving (EOD)

b) Beginning of restrike (BOR)

Fig. 4 Pile top dynamic records and CAPWAP analyses for a 30-inch (762-mm) pile at one of the west pier foundations

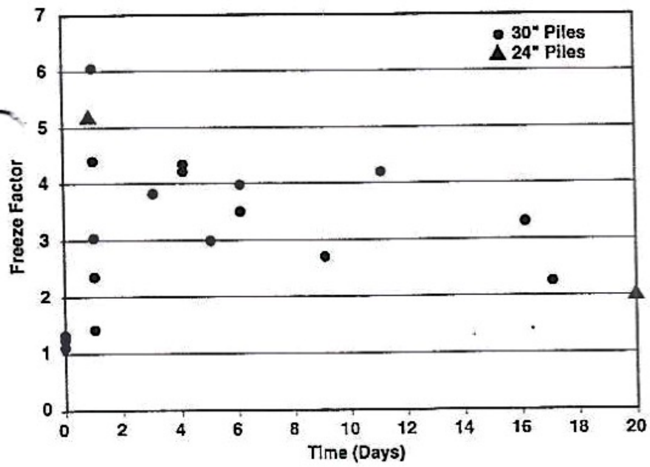


Fig. 5 Freeze Factor versus time after end of initial driving

- The data do not indicate a continued increase in resistance with time beyond one day after completion of initial driving.

The above findings suggest that the increased resistance may be due to the rapid relief of excess pore water pressure in the granular soils surrounding the pile rather than the gradual long term dissipation of excess pore water pressure as might be characteristic of cohesive soils. Also, it is likely that the vibrations and ground displacement caused by driving adjacent piles contributed significantly to the observed increase in friction, particularly within the depth of jetting.

Unit Friction and End Bearing Resistance

Table 3, developed from the CAPWAP analyses, presents a summary of unit friction and end bearing resistance for various foundation locations. Included are data from both the end of initial driving (EOD) and the beginning of restrike (BOR). The maximum friction resistance values were typically determined over a pile segment length of 6 to 8 feet (1.8 to 2.4 m). The unit end bearing resistance for the hollow 30-inch (762-mm) square piles was determined using only the concrete cross-sectional area.

As noted previously, the restrike tests were generally performed on piles founded in looser sand or in cohesive soils. This may explain why the average redrive resistances shown in Table 3 are, in some instances, less than the average resistance values obtained at the end of initial driving.

Following are significant conclusions from these data:

- The maximum unit friction resistance typically occurred at the bottom of the pile. However, some of the piles penetrated through denser soils into looser material which resulted in the maximum unit friction resistance well above the bottom of the pile.

- The maximum unit friction resistance was generally lower for the bent piles since these piles were driven to a lower capacity, and correspondingly less penetration.

- The higher unit friction values shown in the table, typically more than 2.0 ksf (96 kPa), are considered reasonable for the medium dense to dense granular soils. The lower unit friction values, typically in the range of 0.3 to 0.6 ksf (14 to 29 kPa), are considered representative of the loose granular deposits and cohesive soils.

- The largest end bearing resistance values, with an average value of 213 ksf (10,200 kPa), were obtained for the 24-inch (610-mm) piles at the west bents where the piles were driven to bearing on limerock.

- The end bearing resistances of the remaining foundations were erratic, reflecting the varied ground conditions at the pile toe. The higher unit end bearing values, typically more than 100 ksf (4,800 kPa), are considered representative of the medium dense to dense granular soils. The lower values, typically in the range of 30 to 60 ksf (1,400 to 2,900 kPa), are considered representative of the loose granular deposits and cohesive soils.

Hammer Performance

The PDA data were used to compute the energy transfer efficiency for the pile hammers. Energy transfer efficiency is defined as the ratio of maximum energy transferred to the pile to the manufacturers' maximum rated energy for air/steam hammers, or the maximum potential energy for diesel hammers, expressed as a percentage. It should be noted that the actual hammer stroke, and the corresponding potential energy, of the diesel hammers were typically less than the rated stroke and potential energy reported by the hammer manufacturer since these parameters are primarily a function of the available soil resistance. Accordingly, a "field efficiency", computed using the actual hammer stroke, would generally be higher than the energy transfer efficiencies reported below for the diesel hammers.

- For the Delmag D 46-32 diesel hammer driving the 24-inch (610-mm) piles, the energy transfer efficiencies from 39 PDA tests ranged from 23 to 58 percent, with an average of 34.4 percent and a standard deviation of 6.8 percent. This hammer had an average stroke of 9.46 feet (2.88 m), with a standard deviation of 0.65 feet (0.20 m). This average stroke is approximately 89 percent of the maximum stroke for this hammer.

Table 3 Unit friction and end bearing resistance from CAPWAP analyses

Pile Size inches (mm)	Location	Test Type	Number of Piles	Max. Friction, ksf (kPa) Average	Max. Friction, ksf (kPa) Std. Dev.	End Bearing, ksf (kPa) Average	End Bearing, ksf (kPa) Std. Dev.
24 (610)	W. Bents	EOD	15	1.84 (88)	1.19 (57)	213 (10,200)	60 (2,900)
		BOR	0	-	-	-	-
24 (610)	E. Bents	EOD	18	1.12 (54)	0.68 (33)	107 (5,100)	50 (2,400)
		BOR	3	1.65 (79)	0.41 (20)	59 (2,800)	52 (2,500)
30 (762)	W. Bents	EOD	16	1.23 (59)	0.49 (24)	115 (5,500)	47 (2,300)
		BOR	0	-	-	-	-
30 (762)	E. Bents	EOD	5	1.91 (92)	1.05 (50)	41 (2,000)	23 (1,100)
		BOR	1	0.61 (29)	-	66 (3,200)	-
30 (762)	W. Piers	EOD	58	1.84 (88)	0.93 (45)	141 (6,800)	55 (2,600)
		BOR	18	1.90 (91)	0.54 (26)	105 (5,000)	35 (1,700)
30 (762)	E. Piers	EOD	54	1.76 (84)	0.80 (38)	143 (6,900)	53 (2,500)
		BOR	20	1.82 (87)	0.84 (40)	103 (4,900)	45 (2,200)

EOD: Test data from end of initial driving BOR: Test data from beginning of restrike

- For the Delmag D 62-22 diesel hammer driving the 30-inch (762-mm) bent and pier piles, the energy transfer efficiencies from 23 PDA tests ranged from 19 to 38 percent, with an average of 27.9 percent and a standard deviation of 4.9 percent. This hammer had an average stroke of 9.60 feet (2.93 m), with a standard deviation of 0.60 feet (0.18 m). This average stroke is approximately 85 percent of the maximum stroke for this hammer.

- For the Conmaco 300E5 air hammer driving the 30-inch (762-mm) bent and pier piles without a follower, the energy transfer efficiencies from 116 PDA tests ranged from 34 to 86 percent, with an average of 57.8 percent and a standard deviation of 10.5 percent.

- For the Conmaco 300E5 air hammer driving the 30-inch (762-mm) piles with a follower, the energy transfer efficiencies from 34 PDA tests ranged from 33 to 73 percent, with an average of 51.9 percent, and a standard deviation of 10.5 percent. As noted previously, these piles were located at the four piers adjacent to the navigation channel.

Table 4 Allowable driving stresses

Pile Size inches (mm)	Allowable Compressive Stress ksi (MPa)	Allowable Tensile Stress ksi (MPa)
30 (762)	3.64 (25.1)	1.53 (10.56)
24 (610)	3.84 (26.5)	1.25 (8.63)
	3.70 (25.5)	1.44 (9.94)
18 (457)	3.83 (26.4)	1.27 (8.76)

Driving Stresses

In accordance with FDOT specifications, the allowable driving stresses for the prestressed concrete piles were determined from the following equations:

$$S_c = 0.7\, f_c' - 0.75\, f_{pe} \quad \text{in psi, or MPa} \qquad (1)$$

$$S_t = 3.25\, (f_c')^{0.5} + 1.05\, f_{pe} \quad \text{in psi} \qquad (2a)$$

$$= 0.27\, (f_c')^{0.5} + 1.05\, f_{pe} \quad \text{in MPa} \qquad (2b)$$

where: S_c = maximum allowable compressive stress
S_t = maximum allowable tensile stress
f_c' = specified min. compressive strength of concrete = 6,500 psi (44.85 MPa)
f_{pe} = effective prestress (after losses) = 0.8 x prestress = 936 to 1,208 psi (6.46 to 8.34 MPa)

Table 4 summarizes the allowable stresses computed from the above equations. The PDA was used to monitor pile stresses during driving and to help identify the need for remedial measures to maintain driving stresses within the allowable limits. Generally, compressive stresses remained within allowable limits, but high tension stresses were a concern early in the project. The primary measures used to control driving stresses included:

- Modifying the hammer cushion material and thickness of the plywood pile cushion to those noted in Table 1.

- Operating the air hammer with the short stroke to a greater resistance before switching to the full stroke.

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- Furnishing 24-inch (610-mm) piles with greater prestress (see Table 4) to increase the allowable tensile driving stress.

- Avoiding the use of the Delmag D 62-22 hammer at the pier foundations.

Dynamic Soil Parameters

The dynamic soil parameters, including damping and quake, were obtained from CAPWAP analyses for the various foundation locations. These parameters are discussed below and compared with values reported in the literature for similar types of soils. The average values noted below represent averages computed separately for each pile size and for each foundation grouping (i.e. West Bents, West Piers, etc.).

- At the end of initial driving, the average side damping values ranged from 0.15 to 0.18 s/ft (0.49 to 0.59 s/m) for the 24-inch (610-mm) piles, and 0.21 to 0.29 s/ft (0.69 to 0.95 s/m) for the 30-inch (762-mm) piles. At the beginning of restrike the average side damping values generally ranged from 0.35 to 0.41 s/ft (1.15 to 1.35 s/m) for both pile sizes. These values are considerably greater than the value of 0.05 s/ft (0.16 s/m) generally considered representative of non-cohesive soils, but within the range for mixed soils with cohesive components.

- At the end of initial driving, the average toe damping values generally ranged from 0.11 to 0.12 s/ft (0.36 to 0.39 s/m) for the 24-inch (610-mm) piles, and 0.15 to 0.18 s/ft (0.49 to 0.59 s/m) for the 30-inch (762-mm) piles. These results are consistent with the value of 0.15 s/ft (0.49 s/m) generally considered representative for all types of soils. At the beginning of restrike the average toe damping values increased, and ranged from 0.19 to 0.29 s/ft (0.62 to 0.95 s/m).

- During both the end of initial driving and the beginning of restrike, the average side quake values obtained from the CAPWAP analyses ranged from 0.08 to 0.10 inches (2.0 to 2.5 mm). These results are consistent with the value of 0.10 inches (2.5 mm) generally considered representative for all types of soils.

- At the end of initial driving, the average toe quake values obtained from CAPWAP analyses ranged from 0.39 to 0.50 inches (9.9 to 12.7 mm). At the beginning of restrike, the average toe quake values generally ranged from 0.31 to 0.32 inches (7.9 to 8.1 mm). These results are somewhat greater than the values of 0.15 to 0.25 inches (3.8 to 6.4 mm) considered representative for 18 to 30-inch (610 to 762-mm) piles.

CONCLUSION

Dynamic testing was used as an essential quality control measure during pile installation for the Buckman Bridge Widening project. Approximately 6 percent of the 3,351 prestressed concrete piles were tested for the 16,300-foot (4,968-m) long twin structures. The dynamic pile tests verified pile capacity and assured that driving stresses remained below allowable values. It also provided valuable information for identifying deficient piles and confirming the adequacy of remedial measures. Relying on the dynamic pile test data, only 17 piles had to be driven below cut-off level and spliced, and only six piles had to be extracted and replaced with longer piles to achieve the required capacity. No piles were broken during driving operations. The dynamic pile testing program was a primary factor in the successful completion of pile foundation construction.

The results of the project's dynamic pile testing program provided useful information which may be applicable for the design and installation of prestressed concrete piles for other projects with similar subsurface conditions.

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