Test Pile Program to Determine Axial Capacity and Pile Setup for the Biloxi Bay Bridge

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ABSTRACT

Hurricane Katrina caused significant damage to transportation structures on the Gulf coast in August, 2005, including the destruction of the US 90 Bridge over Biloxi Bay, Mississippi. Part of the successful design/build proposal for the replacement bridge included a comprehensive test pile program of indicator piles and load test piles in the geotechnical investigation program. A total of 22 indicator piles were installed on the project using the pile driving analyzer to monitor the pile behavior. Five load tests were performed: two axial Statnamic, two lateral Statnamic, and one static axial. The results of the test pile program established driving criteria for production piles that included end-of-drive blow counts and pile tip elevations with an appropriate allowance for setup. The program also confirmed that the planned installation equipment and techniques of GC Constructors, a joint venture of Massman Construction Co., Kiewit Southern Co., and Traylor Bros., Inc., would install the piles to the design tip elevations before production piles began. An extensive program of restrike measurements at a range of times after initial driving of the indicator piles provided a systematic documentation of pile setup in the soil profiles on the site. These resistance values were used to develop a dimensionless setup factor "S" as the ratio of the restrike resistance to the end of drive resistance. This factor varied from 1.5 for piles penetrating clays and tipped in sand to 3.0 for piles bearing completely in clay and helped establish an end-of drive blow count that included an allowance for setup. The CAPWAP analyses of restrike data with setup and the load test data were used to confirm the design unit side shear and end bearing values developed from the borings and static capacity calculations and to select the design pile lengths.

INTRODUCTION

Hurricane Katrina hit the Mississippi and Louisiana Gulf Coast region on August 29, 2005, causing widespread damage and destruction. Transportation infrastructure all along the coast was destroyed or significantly damaged. The bridge on U.S. 90 over Biloxi Bay between Biloxi and Ocean Springs, Miss. was one of several bridge structures destroyed. Part of the successful design/build proposal submitted by GC Constructors, a joint venture of Massman Construction Co., Kiewit Southern Co., and Traylor Bros., Inc. for the replacement bridge included a comprehensive test pile program of indicator piles and load test piles in the geotechnical investigation program. This paper includes a description of the test pile program, the goals of the program, a summary of the data collected, and the application of the results of the program on the project.

PROJECT DESCRIPTION

The project to replace the bridge included the new main bridge over the bay, changes to the approach grades and alignment, pile supported retaining walls for the approach embankments, and a new bridge over the CSX railroad on the Ocean Springs side of the bay. The new main bridge consists of two structures with three 12-foot (3.7-m) wide lanes each: a north (west bound) bridge and a south (east bound) bridge. The south bridge also includes a combined use pedestrian/bike lane. The inclusion of this lane resulted in higher loads on the piles supporting the south bridge as compared to the north bridge. Each bridge consists of low and high approaches on each side of the 95-foot (30-m) high main channel span.

Prestressed precast concrete pile foundations were selected to support the bridge structures and the abutment retaining walls. The low level approach structures of the main bridge are supported on 24-inch (610 mm) square piles with axial service loads ranging from 320 to 450 kips (1,425 to 2,000 kN) per pile. The high level structures of the main bridge are supported on 30-inch (762 mm) square piles with axial service loads ranging from 420 to 570 kips (1,870 to 2,535 kN) per pile. The remaining land based structures on the project (CSX bridge and abutment retaining walls) are supported by 18-inch (457 mm) square piles designed for axial loads of 120 to 200 kips (535 to 890 kN) per pile. During initial design, both the 24-inch and 30-inch piles were designed as voided piles (cast with a void along the center axis of the pile) to reduce the potential for developing excessive tensile stresses in the piles during driving. The driving stresses recorded during the indicator pile program indicated that solid 24-inch piles would be acceptable. The 18-inch piles were solid piles.

The piles on the main bridge are arranged in waterline pile footings that support the piers. Each footing contains 4 to 36 piles. The design of the pier footings included consideration of wind loads, vessel impact loads, and scour. The vessel impact loads ranged from 200 kips (890 kN) at the low approach piers to 2850 kips (12,680 kN) at the main channel piers. Total scour of 20 to 30 feet (6.1 to 9.1 m) below the existing mudline was included in the design. The piles supporting the CSX bridge are arranged as pile bents while the piles supporting the retaining walls are arranged in a single row beneath the wall footing. Design lateral loads for these land based structures ranged from 10 to 40 kips (45 to 180 kN) per pile.

In addition to the technical details of the project, the design/build team had to meet an aggressive schedule. The cities of Ocean Springs and Biloxi, as well as the surrounding counties, were all anxious to have the bridge open as soon as possible to restore access to the area via US 90. The Mississippi Department of Transportation (MDOT) awarded the contract on June 6, 2006 to GC Constructors (GCC), a joint venture of Massman Construction Co., Kiewit Southern Co., and Traylor Brothers, Inc. GCC was required to have two lanes open to traffic by November 13, 2007 and to have the project complete by April16, 2008.

SUBSURFACE CONDITIONS

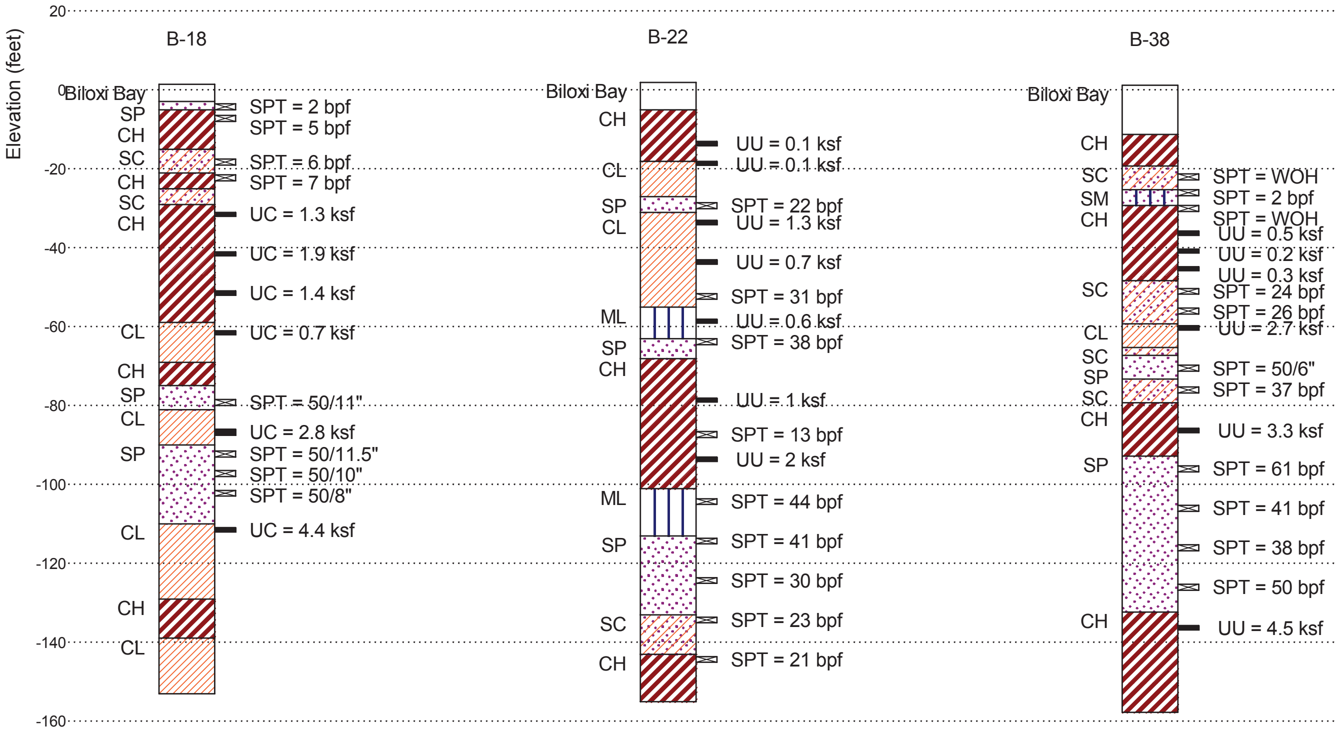
The geotechnical exploration consisted of 62 soil borings, 39 of which were drilled at the main bridge. The main bridge borings were drilled at each abutment, every other approach pier, and at both main channel piers. Additional borings were drilled along retaining wall alignments and the CSX bridge alignment on the east approach. The borings along the main bridge alignment were drilled 150 feet (45.7 m) below the mudline elevation. The borings on the east and west bank approaches were drilled to depths of 100 to 150 feet (30.5 to 45.7 m) below existing ground surface. Standard Penetration Test samples were obtained in non-cohesive soils and relatively undisturbed Shelby tube samples were obtained in the cohesive soils. A laboratory testing program of soil classification, unconfined compressive strength, UU triaxial tests, and consolidation tests was used to provide soil strength and stress history information for foundation design.

The project area was divided into four areas for geotechnical analysis and reporting. These four areas were: East Bank Approach, West Bank Approach, Low Level Structures, and High Level Structure. The East Bank Approach includes the realignment of US 90, retaining walls, and Abutment 1 of the bridge. The West Bank Approach includes realignment of US 90, retaining walls, Abutment 72 of the bridge, and the CSX Bridge. The Low Level Structures include the West Low Level Approach (Piers 2 through 40) and the East Low Level Approach (Piers 65 through 71). The High Level Structure includes Piers 41 through 64, with Piers 52 and 53 spanning the main channel.

In general, the soils at the site consist of sands and clays of Pleistocene or early Recent age. The surface deposits are typically early Recent sands and soft clays. Beneath the sands are Pleistocene deposits of very stiff to stiff clays and medium dense to dense sands. Fig. 1 illustrates the general subsurface conditions along the project alignment through three borings: B-18, B-22, and B-38.

TEST PILE PROGRAM DESCRIPTION

With an aggressive project schedule, the project team structured the test pile program to be focused on "the big picture" while maintaining the details necessary to provide relevant data. The "big picture" was the risk of not meeting the project schedule; therefore, the test pile program was developed to provide information in the design phase that would reduce schedule uncertainties, such as time for pile setup, requirements for restrikes, or requirements



[FIG. 1] Illustration of general subsurface conditions along project alignment

for extra driving of piles beyond planned tip elevations. This information was formulated into an effective set of driving criteria for each pier on the bridge. The driving criteria were developed through indicator piles and load test piles, and included:

- Pile size and type.
- Hammer and cushion information.
- Directives for reduced stroke requirements for controlling driving stresses (e.g., operate at ½ stroke if blow count is less than ***x***).
- Required ultimate axial resistance.
- Minimum tip elevation.
- Recommendations for use of any driving aids above minimum tip elevation.
- Estimated tip elevation.
- Required driving resistance (blow count) at time of initial driving which would allow driving to cease so long as minimum tip is achieved.
- Minimum required driving resistance at estimated tip elevation, *including allowance for pile setup*
- Plan for set checks if minimum required driving resistance was not achieved during initial drive.
- Practical refusal guidelines.

In order to aid in establishing the driving criteria, the specific objectives of the indicator pile program were to:

1. Verify that piles could be installed to minimum tip elevation with the hammers selected;
2. Evaluate need for any driving aids to achieve the first objective;
3. Define the driving resistance level below which reduced stroke is needed to avoid pile damage;
4. Evaluate any changes required to cushions to avoid pile damage; and,
5. Verify or adjust the estimated tip elevations *with an appropriate allowance for setup*.

The geotechnical team anticipated that pile setup, or the increase of pile capacity over time after initial driving, would be significant for most of the pier locations on the project. The team concluded that the end-of-drive blow counts would not be a reliable indicator of ultimate pile capacity and that significant delays to the project schedule would occur if a large number of restrikes were required during production pile driving. It was thus important for the amount of setup to be quantified reliably to aid in the design phase so that setup could be accounted for at the end of driving.

To quantify the setup, a program of restrike measurements at a range of times after initial driving was established to document the setup in a systematic manner. By documenting and accounting for setup, an end-of-drive blow count could be established which included an allowance for setup. With appropriate conservatism in setting estimated tip elevations, the design team could reduce the risk of delays due to questions regarding pile axial resistance and the need for set checks or splices. The MDOT would be assured that piles driven to satisfy the established criteria will ultimately achieve the required ultimate axial resistance, even if the end-of-drive blow count does not indicate that the required resistance has been achieved at the end of initial driving.

A total of 22 indicator piles were driven along the project alignment: 20 at the main bridge and 2 at the CSX bridge. The number of each of the three pile sizes on the project that were driven as indicator piles was based on the relative numbers of piles of each size and the locations along the alignment. The breakdown of indicator piles by pile size was:

- 18-inch (Main Bridge Abutments and CSX Bridge): 4
- 24-inch (Low Level Structures): 11
- 30-inch (High Level Structure): 7

The tip elevations of the indicator piles were estimated by axial analysis using the computer program DRIVEN and other computation techniques based on the soil boring and laboratory data. The axial analysis determined that the piles would derive most of their support from side resistance, with the 18-inch piles on the east and west approaches achieving the required capacity with tips bearing in the shallow sand layer at about elevation -26 feet (-8 m), the 24-inch piles tipping at elevations between -90 and -106 feet (-27 m to -32 m) (mostly in clays, but some in sands), and the 30-inch piles achieving capacity with tips bearing in the sands encountered at about elevation -90 to -95 feet (-27 m to -29 m). To provide data on pile capacity as a function of pile length in the hope of reducing the lengths of the 24-inch piles, the tip elevations of the 24-inch indicator piles were set at a range of elevations between -70 and -105 ft (-21 m and -32 m). The 30-inch and 18-inch indicator piles tip elevations were set to bear in the sand strata identified in their respective analyses.

[FIG. 2] **Conventional Static Load Test**

Response of the 24-inch and 30-inch pile groups to the design lateral loads also contributed to the estimated pile tip elevation analysis. The designers used the computer program FB Pier to analyze the pier responses and determine the pile lengths required to resist the design loads. Soil parameters for input into the FB Pier soil model, including shear strength, shear modulus, subgrade modulus, Poisson's ratio, and unit weight, were determined from the soil boring and laboratory data.

Different hammers were used to drive the indicator piles, depending on the size of the pile. The 18-inch indicator piles installed on land were driven using a Delmag 3032 diesel hammer or a DKH-10U hydraulic hammer, each with a maximum rated energy of about 75,000 ft-lbs (102 kJ). The 24-inch indicator piles were driven using a Conmaco 5200 marine air hammer with a maximum rated energy of 100,000 ft-lbs (136 kJ). The 30-inch indicator piles were driven with a Conmaco 300E marine air hammer with a maximum rated energy of 150,000 ft-lbs (203 kJ). Both Conmaco hammers had a two foot (0.6 m) stroke setting and a five foot (1.5 m) stroke setting.

All of the indicator piles were driven with continuous Pile Driving Analyzer (PDA) monitoring to evaluate driving stresses and indicated axial resistance. The PDA was also used to measure pile resistance during all restrikes. Since the indicator piles and load test piles were installed concurrent with dredging and debris removal, the indicator piles generally had restrikes at one day after driving, at between six and ten days after driving, and at 21 or more days after driving.

The load test program consisted of one axial static, two axial Statnamic© and two lateral Statnamic© tests. Each of the three tests was performed on separate 24-inch piles. Axial Statnamic© and lateral Statnamic© tests were performed on separate 30-inch piles. The axial static test planned for a 30-inch pile was not performed due to schedule issues and the excellent agreement between the static and Statnamic© axial tests on the 24-inch piles. The test pile locations were selected next to planned indicator pile locations based on the soil boring data. A separate pile distinct from the indicator pile at the test location was driven for each axial test. The lateral tests were performed on the indicator piles driven at the test locations. All load tests were performed at least 21 days after installation of the test piles to allow time for setup to occur.

Internal instrumentation for the load test piles was installed into the piles during the casting process. All test piles were instrumented with resistance-type strain gauges mounted on sister bars, four pairs of gauges at various levels within the pile and a set of four gauges two feet (600 mm) above the pile tip. The lateral test piles also had a 2.75-inch (70 mm) diameter inclinometer casing installed in the piles. Additional instrumentation installed during load testing included load cells and accelerometers for the axial Statnamic tests, LVDTs and accelerometers for the lateral Statnamic tests, and dial gauges for the axial static test.

TEST RESULTS AND APPLICATIONS

Axial Resistance from PDA, CAPWAP, and Load Tests

The tables below summarize the data from the test pile program. These tables are:

- Table 1 – Summary of PDA Data
- Table 2 – Summary of Side and End Resistance by CAPWAP
- Table 3 – Summary of Axial Load Test Results

The PDA data table lists the pile number, size pile, and the predominate soil type influencing the pile capacity. The pile resistance at the end of drive (EOD) based on the PDA measurements is listed along with restrike resistance. Damping factors used for the EOD resistance were RX=0.85 for clay and clay/sand soils, and RX=0.7 for sand soils. These factors were selected based on the experience of Eustis Engineering in similar subsurface conditions. The listed restrike resistances are noted as being determined from either PDA only or a CAPWAP analysis of the restrike data in the column labeled “Restrike Capacity Type”. The three piles that were axially load tested are also included and noted as “Load Test” under “Restrike Capacity Type”.

[FIG. 3] Statnamic Load Test

[TABLE 1] Summary of PDA Data

PILE NUMBER	PILE SIZE	PILE TIP EL. (feet)	SOIL TYPE AT TIP	EOD CAPACITY (kips)	RESTRIKE CAPACITY (kips)	RESTRIKE CAPACITY TYPE	"S" Factor	DAYS RESTRIKE
IP-TEST	24-IN.	-105	CLAY/SAND	669	1062	PDA	1.59	1
IP-TEST	24-IN.	-105	CLAY/SAND	669	1408	CAPWAP	2.10	17
IP-TEST	24-IN.	-105	CLAY/SAND	669	1566	CAPWAP	2.34	46
IP-1	18-IN.	-72	CLAY	259	575	PDA	2.22	4
IP-1	18-IN.	-72	CLAY	259	824	CAPWAP	3.18	21
IP-2	24-IN.	-100	CLAY/SAND	835	1197	PDA	1.43	1
IP-2	24-IN.	-100	CLAY/SAND	835	1654	CAPWAP	1.98	33
IP-3	24-IN.	-85	CLAY	388	540	PDA	1.39	1
IP-3	24-IN.	-85	CLAY	388	1298	CAPWAP	3.35	33
IP-3 SLT	24-IN.	-95	CLAY	442	1350	Load Test	3.05	22
IP-4	24-IN.	-73	CLAY/SAND	506	789	PDA	1.56	1
IP-4	24-IN.	-73	CLAY/SAND	506	1300	PDA	2.57	56
IP-5	24-IN.	-85	CLAY	394	608	PDA	1.54	1
IP-5	24-IN.	-85	CLAY	394	887	PDA	2.25	22
IP-6	24-IN.	-85	CLAY	426	906	PDA	2.13	1
IP-6	24-IN.	-85	CLAY	426	1383	CAPWAP	3.25	22
IP-6	24-IN.	-85	CLAY	426	1288	PDA	3.02	61
IP-7	24-IN.	-90	CLAY	364	540	PDA	1.48	2
IP-7	24-IN.	-90	CLAY	364	771	PDA	2.12	13
IP-7	24-IN.	-90	CLAY	364	904	PDA	2.48	21
IP-7	24-IN.	-90	CLAY	364	950	PDA	2.61	62
IP-8 AST	24-IN.	-99	CLAY	354	1390	Load Test	3.93	19
IP-8 LST	24-IN.	-99	CLAY	483	672	PDA	1.39	1
IP-8 LST	24-IN.	-99	CLAY	483	1396	CAPWAP	2.89	29
IP-8 LST	24-IN.	-99	CLAY	483	1047	PDA	2.17	85
IP-9	24-IN.	-90	CLAY	445	948	PDA	2.13	1
IP-9	24-IN.	-90	CLAY	445	1255	CAPWAP	2.82	25
IP-10	24-IN.	-100	CLAY	530	942	PDA	1.78	1
IP-10	24-IN.	-100	CLAY	530	1198	CAPWAP	2.26	10
IP-10	24-IN.	-100	CLAY	530	1450	PDA	2.74	27

[TABLE 2] Summary of Side and End Resistance by CAPWAP

PILE NUMBER	PILE SIZE	PILE TIP EL. (feet)	PREDOMINATE SOIL	SHAFT CAPACITY (kips)	TIP CAPACITY (kips)	TOTAL CAPACITY (kips)	DAYS RESTRIKE
IP-TEST	24-IN.	-105	CLAY	427	388	815	EOD
IP-TEST	24-IN.	-105	CLAY	1070	338	1408	17
IP-TEST	24-IN.	-105	CLAY	1265	301	1566	46
IP-3	24-IN.	-85	CLAY	1138	160	1298	33
IP-6	24-IN.	-95	CLAY	1253	130	1383	22
IP-7	24-IN.	-90	CLAY	765	140	905	21
IP-9	24-IN.	-90	CLAY	1151	104	1255	25
IP-10	24-IN.	-100	CLAY	476	117	593	EOD
IP-10	24-IN.	-100	CLAY	1110	88	1198	10
IP-10	24-IN.	-100	CLAY	1362	88	1450	27
IP-11	30-IN.	-95	SAND	1336	340	1676	8
IP-12	30-IN.	-96	CLAY	1059	457	1516	6
IP-13	30-IN.	-93	SAND	1399	185	1584	21
IP-14 LST	30-IN.	-93	SAND	1042	467	1509	28
IP-15	30-IN.	-90	SAND	1254	169	1423	19
IP-16	30-IN.	-91	CLAY	1271	171	1442	9
IP-17	30-IN.	-96	CLAY	1341	258	1599	23

[TABLE 1 continued] Summary of PDA Data

PILE NUMBER	PILE SIZE	PILE TIP EL. (feet)	SOIL TYPE AT TIP	EOD CAPACITY (kips)	RESTRIKE CAPACITY (kips)	RESTRIKE CAPACITY TYPE	"S" Factor	DAYS RESTRIKE
IP-11	30-IN.	-95	SAND	1243	1300	PDA	1.05	1
IP-11	30-IN.	-95	SAND	1243	1676	CAPWAP	1.35	8
IP-12	30-IN.	-96	CLAY	708	1278	PDA	1.81	1
IP-12	30-IN.	-96	CLAY	708	1516	CAPWAP	2.14	6
IP-13	30-IN.	-93	SAND	1076	1460	PDA	1.36	1
IP-13	30-IN.	-93	SAND	1076	1486	PDA	1.38	10
IP-13	30-IN.	-93	SAND	1076	1584	CAPWAP	1.47	21
IP-13	30-IN.	-93	SAND	1076	1957	PDA	1.82	45
IP-14 AST	30-IN.	-91	SAND	1035	1575	Load Test	1.52	21
IP-14 LST	30-IN.	-93	SAND	1109	1378	PDA	1.24	1
IP-14 LST	30-IN.	-93	SAND	1109	1454	PDA	1.31	8
IP-14 LST	30-IN.	-93	SAND	1109	1509	CAPWAP	1.36	28
IP-14-LST	30-IN.	-93	SAND	1109	1581	PDA	1.43	43
IP-15	30-IN.	-90	SAND	1055	1503	PDA	1.42	2
IP-15	30-IN.	-90	SAND	1055	1557	PDA	1.48	8
IP-15	30-IN.	-90	SAND	1055	1423	CAPWAP	1.35	19
IP-15	30-IN.	-90	SAND	1055	1450	PDA	1.37	43
IP-16	30-IN.	-91	CLAY	613	1239	PDA	2.02	2
IP-16	30-IN.	-91	CLAY	613	1442	CAPWAP	2.35	9
IP-16	30-IN.	-91	CLAY	613	1733	PDA	2.83	34
IP-17	30-IN.	-96	CLAY	1047	1278	PDA	1.22	1
IP-17	30-IN.	-96	CLAY	1047	1492	PDA	1.43	11
IP-17	30-IN.	-96	CLAY	1047	1599	CAPWAP	1.53	23
IP-17 SLT	30-IN.	-90	CLAY/SAND	896	1445	CAPWAP	1.61	24
IP-18	24-IN.	-70	SAND	804	1307	CAPWAP	1.63	2
IP-18	24-IN.	-70	SAND	804	1327	PDA	1.65	38
IP-19	18-IN.	-26	SAND	782	842	CAPWAP	1.08	1
IP-20	18-IN.	-26	SAND	843	959	CAPWAP	1.14	1
IP-21	18-IN.	-26	SAND	882	827	CAPWAP	0.94	1

Pile Setup, End of Drive Criteria, and Pile Tip Elevations

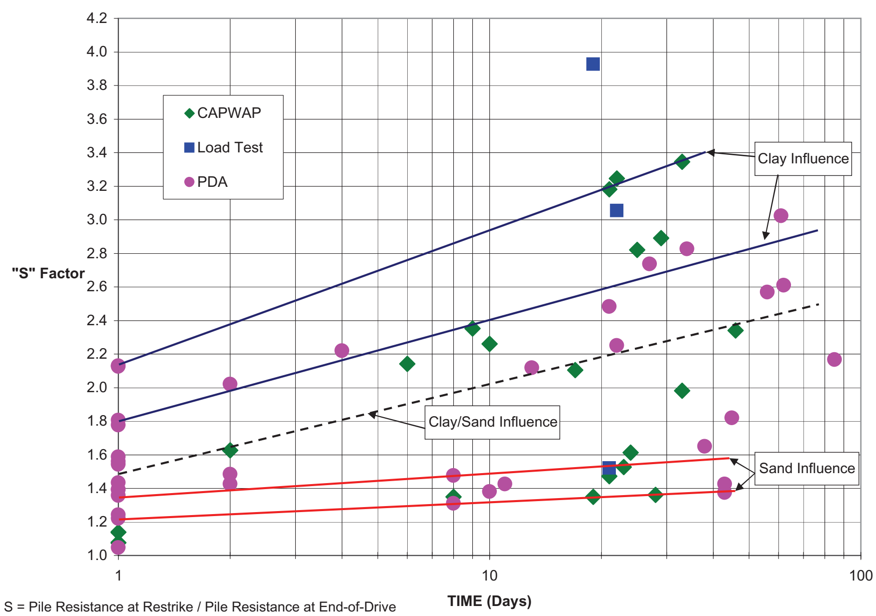
To evaluate pile setup, the total pile resistance at each restrike was divided by the EOD capacity to calculate a setup factor "S" as an indicator of increase in resistance over time, or pile setup. The values for S calculated at the restrikes for each pile were plotted on a semi-log plot of S versus time, similar to Fig. 4 below.

Shown on the graph are groupings of the piles according to the dominate soil influence on pile resistance: clay, sand, or both. Piles in predominately clay soils generally showed setup factors of 1.8 to 3, piles in sand soils 1 to 1.4, and piles in sand/clay profiles 1.5 to 2.5. Fig. 5 shows the data grouped by soil type, plotting the factor "S" against time on an arithmetic scale. This figure illustrates how the pile resistance "levels off" over time based on each soil group.

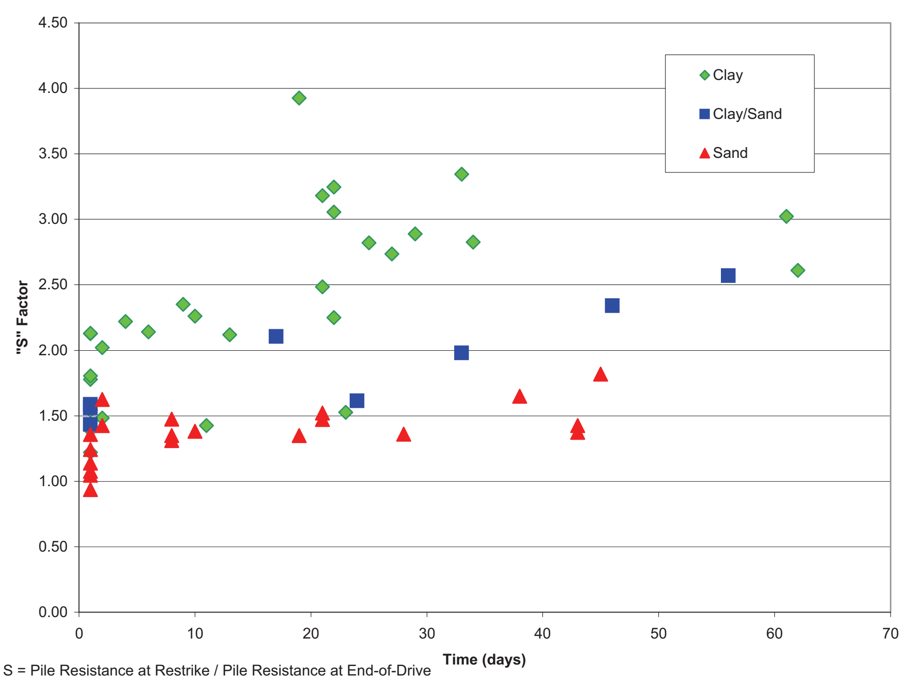
To calculate S, the EOD resistance as determined from the PDA data was used rather than a CAPWAP derived EOD resistance since the PDA resistance estimate would be available in the

[TABLE 3] **Summary of Axial Load Test Results**

Test Pile	Tip Elevation (ft)	Soil at Pile Tip	Load (kips)	Unit Side Shear (ksf)	Unit End Bearing (ksf)
IP-3 SLT	-85	Clay	1398	1.2 – 2.4	13
IP-8 AST	-99	Clay	1390	0.7 – 2.5	44
IP-14 AST	-91	Sand	1575	1.3 – 4.0	79



[FIG. 4] **Pile Setup Factor "S" as a function of time**

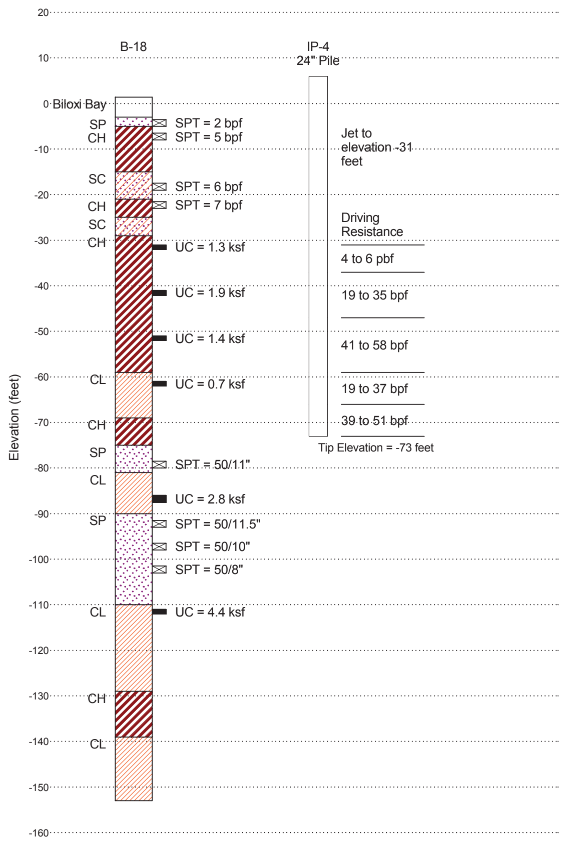


[FIG. 5] **Pile Setup Factor "S" as a function of time grouped by soil type**

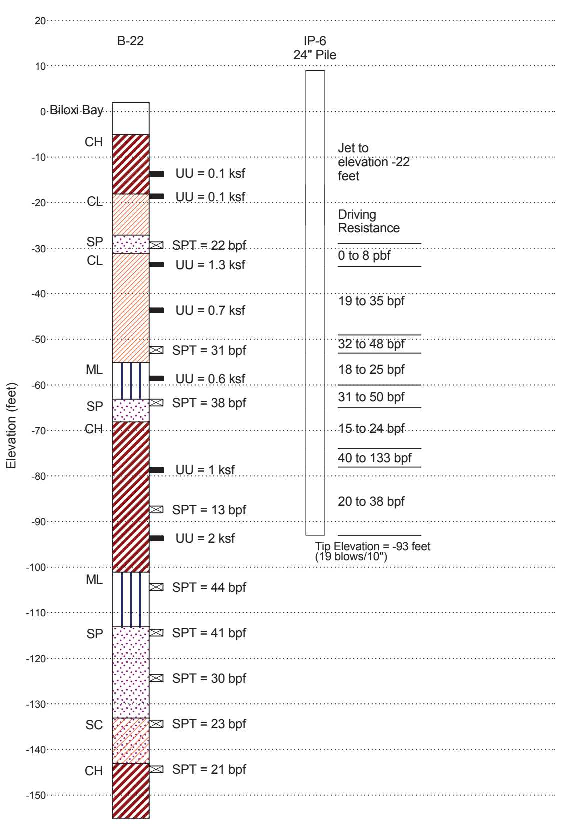
field during production pile installation. Since the indicator piles were driven at every fourth pier location, the project specifications required that the first pile at each pier location be driven with continuous PDA monitoring during production pile installation. Using the PDA on the first pile at each pier allowed for verification of estimated resistances as well as monitoring driving stresses at each pier location. By calculating S based on the PDA EOD resistance of the indicator piles, a consistent basis of comparison could be made during production pile installation.

By evaluating the PDA data from each indicator pile, the load test results, and the pile setup relationships, a reliable set of driving criteria was developed for each pier. The end of drive blow count was based on the blow counts recorded during indicator pile installation. The documentation of the setup as shown in Fig. 4 allowed the end of drive blow count to be established at a threshold value that accounted for the increased pile capacity after driving that was anticipated at each pier or group of piers due to setup. This increased the certainty that piles would reach the required capacity after driving, reducing the need for restrikes. Fig. 6, 7, and 8 illustrate the typical subsurface conditions and initial driving data for an indicator pile in each of the three general soil influence groups.

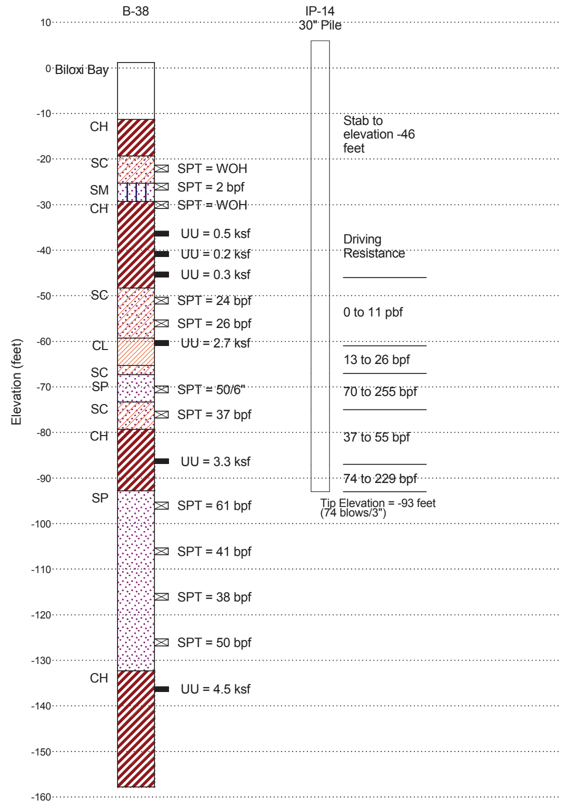
The driving criteria also included instructions for the operators and inspectors concerning pile hammer operation. The PDA data obtained during driving of the indicator piles indicated



[FIG. 6] Subsurface profile and initial driving data for IP-4 (clay/sand influence)



[FIG. 7] Subsurface profile and initial driving data for IP-6 (clay influence)



[FIG. 8] Subsurface profile and initial driving data for IP-14LST (sand influence)

the driving stresses that the piles were subject to under various hammer operating types and conditions. The PDA operator was able to help the pile driver operator determine when to adjust the hammer stroke to avoid damage to the pile, both under easy driving and hard driving conditions. This information was utilized to develop instructions in the driving criteria that included blow counts and penetration rates at which hammer stroke should be increased or decreased. These instructions helped reduce the potential for damaging piles during extreme driving conditions. A sample driving criteria sheet is shown in Fig. 9.

Since the first pile at each bent was driven with PDA monitoring, the EOD resistance at each pier could also be verified. The driving criteria thus included a minimum EOD resistance for each pier. This minimum resistance included an allowance for setup based on the pier location and the established setup relationships. If a pile did not meet the minimum blow count or PDA resistance, the driving data for the pile was evaluated to determine if a restrike test was necessary, or if the pile needed to be driven further to achieve the desired capacity.

STATE OF MISSISSIPPI
REPLACEMENT OF HURRICANE DAMAGED U.S. HIGHWAY 90 BRIDGE
BILOXI AND OCEAN SPRINGS
(HARRISON AND JACKSON COUNTIES), MISSISSIPPI
EUSTIS ENGINEERING PROJECT NO. 19405

DRIVING CRITERIA FOR 30-IN. SQUARE CONCRETE PILES
WEST BOUND LANE ONLY
BENTS 51 THROUGH 54
DESIGN LOAD OF 540 KIPS

The 30-in. square concrete piles are approximately 95 feet in length with a 16.5 inch void in the center of the pile except for a 2.5 foot plug at the tip.

Each pile can be jetted to an elevation of -35.

The piles should be driven with a Conmaco 300E5 air hammer developing a maximum driving energy of 150,000 ft-lbs per blow.

The hammer should have a variable stroke, one at 2 feet and one at 5 feet. In order to reduce the tensile stresses during initial driving, the pile driving should start with a hammer stroke of 2 feet. If the driving resistance exceeds 70 blows per foot, increase the stroke to 5 feet. This may occur at approximately el -70 Continue driving with a 5 ft stroke unless the driving resistance falls below 45 blows per foot at which time the stroke of the hammer should be reduced to 2 feet. Continue changing the stroke as needed until the final tip elevation is obtained. Pile driving operations should start at the center of the footing and drive in a radius outward from the center.

Fourteen inches of new plywood cushion should be used for each pile. Add to the cushion or change the cushion as needed.

Dynamically test the first pile driven from Bents 52 and 54. DPT will be performed on Bents 51 and 53 during the driving of the east bound lane's piles.

The piles should be driven to a minimum tip elevation of -90.

If the dynamic capacity recorded by the PDA is less than 450 kips at the final tip elevation, notify Lloyd Held immediately. If the final blow count is less than 30 blows per foot with a hammer stroke of 2 feet, notify Lloyd Held immediately. This assumes the hammer is operating properly and the pile cushion is adequate.

The pile log should have the same information as the indicator pile log except the station will be with a bent number and pile number. There will be no offset. The pile should be logged for the entire length. Record any events which occur during driving

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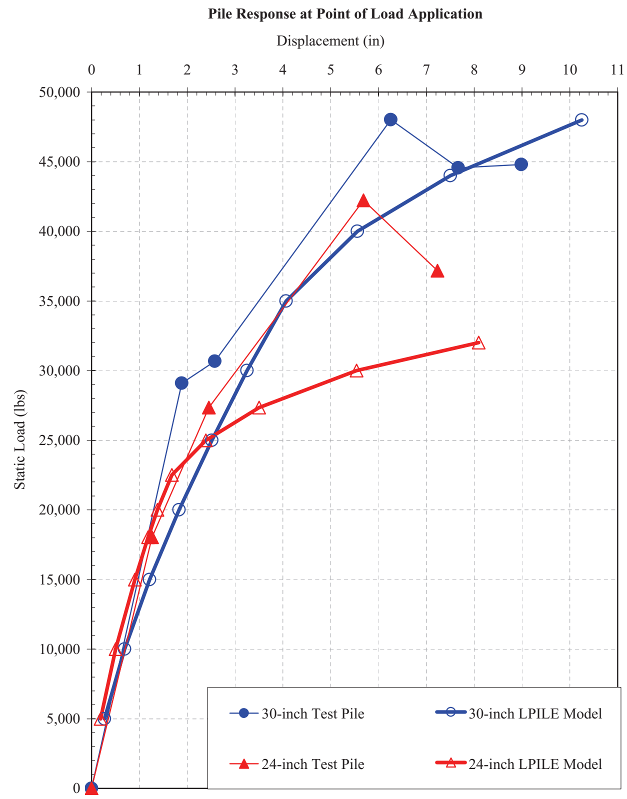
[FIG. 9] Sample Driving Criteria

The minimum tip elevations of the production piles included in the driving criteria were adjusted from the pre-bid estimates using the test pile program data. The load test data showed generally good agreement with the PDA data concerning the total resistance available for the pile, as shown in Table 4 below. The good agreement gave a good level of confidence that the PDA and CAPWAP data could be used to provide reliable estimations of the tip elevations.

[TABLE 4] Comparison of Load Test and PDA Capacities

Test Pile	Test Type	Total Capacity (kips)
IP-3 SLT	Static Load Test	1398
IP-3	CAPWAP	1298
IP-8 AST	Statnamic Load Test	1390
IP-8 LST	CAPWAP	1396
IP-14 AST	Statnamic Load Test	1575
IP-14 LST	CAPWAP	1509

The analysis of the indicator pile and load test data allowed the estimated tip elevations of the 24-inch production piles to be shortened from 5 to 25 feet (1.5 m to 7.6 m), resulting in tip elevations of -65 to -90 feet (-20 m to -27 m). The indicator pile and load test data



[FIG. 10] Derived Static Lateral Load vs. Top of Pile Displacement compared to LPILE model

confirmed the capacity of the 30-inch piles in the dense sands at planned tip elevations of -90 to -95 feet (-27 m to -29 m). The shorter 24-inch piles saved material cost as well as helped reduce the total installation time of the piles. Confirming the capacity of the 30-inch piles reduced uncertainties relating to capacity, greatly reducing the need for future restrikes.

Pile Lateral Resistance Model

The lateral load test results are shown in Fig. 10. This figure shows the derived static load-displacement response curves at the top of the pile for both 24-inch and 30-inch piles. The LPILE model response curves for both piles are also shown for comparison. The LPILE models were generated using the same design soil parameters provided for the FB Pier analysis that was completed before the lateral load tests were performed. This figure illustrates the good agreement between the LPILE soil model and the load test results. Adjustments to the lateral analysis model were thus not required, nor were any adjustments necessary to the pile groups or lengths for lateral resistance.

CONCLUSIONS

An early completion of the pile foundations was attained by incorporating an indicator pile and test pile program into the geotechnical design including a direct assessment of the pile capacity gain with time before the production pile installation began. The data obtained by the pile testing program allowed the design/build team:

- To evaluate the potential for an increase in pile capacity after driving was complete due to setup;
- To establish reasonable driving criteria to reduce the potential for damage to piles while reaching the desired tip elevations;
- To verify and improve the axial design parameters, resulting in reduced pile lengths for some areas of the project; and,
- To verify the design lateral response of the pile groups.

The indicator and test pile program was successful in reducing the uncertainties that could impact the project schedule. The program demonstrated to MDOT that the project design and construction techniques would provide the desired results. The pile installation work started ahead of schedule and stayed ahead of schedule.

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