Design and Load Verification of Driven Pile Foundations

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ABSTRACT

Eleven pile load tests were reviewed and analyzed to document the issues related to the design and load tests of driven pile foundations for a new bridge construction project in Louisiana. This project involves the design and construction of an elevated bridge approximately 17 miles in length. The project area is located in a Coastal Deltaic Plain, Saline Marsh area, underlain by slightly under-consolidated to normally-consolidated weak clays to depths greater than 200 feet. Environmental and site constraints limited the use of pile types and load test methods for this project. The geotechnical investigation methods consist of both cone penetration test and soil borings. Both statnamic load tests and static load tests were used depending upon the magnitude of the test loads. The construction method also dictated the duration of setup allowed for in the performance of the load tests. This paper documents the results of an extensive load test program including the setup behavior of various piles. The results of dynamic monitoring at various times for setup checks with either static or statnamic load tests were also discussed. The Tomlinson α-Method and the Norlund's Method proved to provide excellent predicted pile capacities.

BACKGROUND

The existing LA-1 runs along the bank of Bayou Lafourche and is a primary two-lane transportation route for foreign oil offloaded from ships in the Gulf of Mexico, and also serves as a hurricane evacuation route for the local population of 35,000 plus 6,000 offshore workers. The existing road floods with severe weather. Photograph 1 was taken before a tropical storm Bill landfall on June 30, 2003. At least 10 miles of the southern portion of LA-1 was totally submerged making evacuation through LA-1 very slow and dangerous. The Louisiana Department of Transportation and Development (LADOTD) decided to upgrade the southern 17 miles portion to a four-lane, fully access controlled, elevated bridge. This bridge crosses wetlands, bayous, channels, and flood plains and is located in a sensitive environmental area. It is to consist of low-level and medium-level bridges, two elevated interchanges, and two fixed high-level bridges over navigable waterways: one over Bayou Lafourche and the Boudreaux Canal at Leeville and one over Bollinger Canal. A toll plaza facility and scenic overlook and bird watching area are included in the design. The first phase of the project begins from the intersection of existing LA-1 and LA-3090 to the north connector and is currently under construction.

Photograph 1. Condition of the Existing LA-1 during Tropical Storm Bill, 2003)

The project alignment will be in an environmental sensitive area. Only minimum disturbance to the marsh from pile driving is allowed and no construction traffic is allowed to transverse the marsh. Due to this environmental constraint, the low level bridge will be constructed using the end-on or top-down construction method whereby the bridge will be constructed from the completed portion of the newly constructed bridge. The question of when the construction load can be placed on the recently driven piles becomes very important in setting the project schedule. Due to this constraint, understanding the rate of pile setup becomes extremely important in determining the pile driving sequence and schedule, since the timing of construction loading allowed on the production piles may be the limiting factor. As such, all pile load tests for this project were conducted at multiple time intervals to establish pile setup characteristics to be used for construction.

PILE SETUP

Pile setup phenomenon has been a subject of interest for many geotechnical engineers and researchers (Vesic 1977, Skov and Denver 1988, Fellenius et al 1989, Svinkin, et al. 1994, Komurka and Edil, 2003, Rauche et al. 2004, Bullock 2005a,b). When properly included in the design, the long-term set-up capacity may be used to reduce construction costs or adjust construction schedule. Tavera and Wathugala (1999) estimated a saving of $612,638 or 13% of pile length could be realized for the construction of Bayou Boeuf Bridge Extension in Louisiana. The construction schedule of I-310 Bridge, where end-on construction method was used to protect the sensitive wetland near New Orleans, was dictated by the capacity of the piles which were to support the construction loading during construction. In this case, the construction loading was more critical than the anticipated loading after bridge is in service. Continuous monitoring of the capacity gain using dynamic pile testing was made to ensure a safe construction.

GENERALIZED SOIL CONDITIONS

The average mudline elevation along the majority of the alignment is approximately -1.5 feet, with the exception of the high level bridge crossing Bayou Lafourche where the water depths range from 13 to 40 feet. The water level is roughly at sea level and is impacted by prevailing winds. The soil conditions primarily consist of recent alluvial clays with occasional silty fine sand layers to approximate elevations of -160 feet to -180 feet. Dense to very dense deltaic fine sands underlie the alluvial clays. Based on a series of oedometer consolidation tests performed on the clay soils above the deltaic sands, the overconsolidation ratios (OCR) range from 0.2 to 1.5, with an average value of slightly greater than 0.6. A plot of the OCR profile is shown in Fig. 1. The unconsolidated undrained triaxial tests and cone penetration tests (CPT) indicated an average normalized shear strength parameter (S_u/P') of 0.17. Where S_u is the undrained shear strength and P' is the effective overburden pressure. This is consistent with the findings by McClelland (1956) (S_u/P'=0.15).

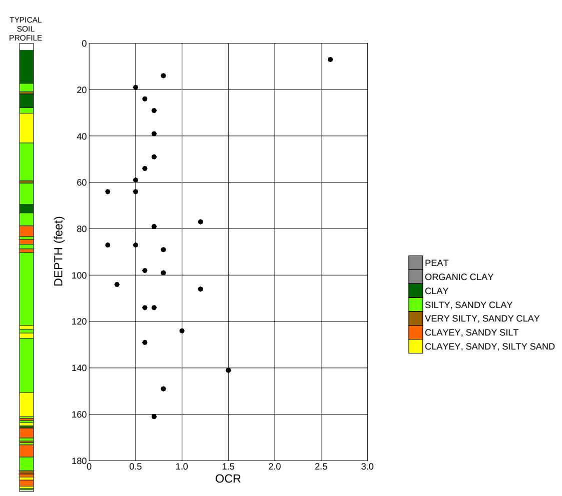

Fig. 1. OCR Profile

Figure 2 presents typical soil conditions showing the results from soil borings and CPT soundings near one of the test piles. The soil generally consists of very weak organic rich clays with a variable thickness sand/silt layer between the depths of about 30 and 40 feet. Frequent silt inclusions are present at the 80-foot depth. Another 10-ft to 20-ft layer of sand/silt was observed at a depth of about 160 feet. These soil conditions are quite consistent throughout the project.

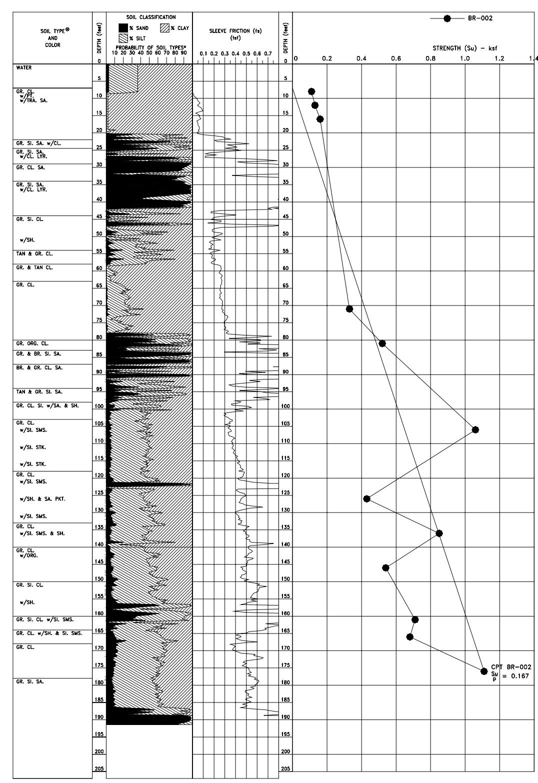

Fig. 2. Typical Soil Conditions

LOAD TESTS

Nine piles were tested during the design stages. The purposes of the testing are threefold: (1) to confirm the calculated pile capacities, (2) to evaluate the drivability, and (3) to evaluate pile setup. Due to problems with limited access to pile driving equipment during the design stage, only four locations were selected for the load tests. The test piles consist of six prestressed precast square concrete (PPC) piles, two 54-inch spin cast cylindrical piles, and one open-ended steel-pipe pile.

Two load tests were conducted at each of the designated locations. At the location of the high level bridge crossing Bayou Lafourche, an additional load test was conducted on one steel pipe pile. Table 1 presents the details of the test piles. These piles were fully instrumented with strain gages throughout the entire piles. A typical instrumentation layout is presented in Figure 4.

Table 1. Test Piles Details

Pile Type*	Location	Pile Length (feet)	Strain Gage Locations+ (feet)
16-in Sq. Conc.	T2	130	6 pairs at 10, 40, 73, 90, 110, 128 ft.
54-in Cyl. Conc.	T2	160	5 pairs at 14, 53, 88, 123, 158 ft.
30-in Sq. Conc.	T3	190	8 pairs at 10, 50, 80, 110, 130, 150, 170, 193 ft.
30-in Steel Pipe	T3	195	9 pairs at 10, 50, 85, 115, 135, 155, 175, 185, 193 ft.
54-in Cyl. Conc.	T3	160	5 pairs at 14, 53, 88 ,123, 158 ft.
24-in Sq. Conc.	T4	210	8 pairs at 10, 50, 80, 113, 135, 160, 185, 208 ft.
24-in Sq. Conc.	T4	160	7 pairs at 10, 50, 72, 100, 120, 140, 158 ft.
24-in Sq. Conc.	T5	170	8 pairs at 10, 40, 65, 93, 110, 130, 150, 168 ft.
24-in Sq. Conc.	T5	145	7 pairs at 10, 35, 60, 85, 105, 125, 143 ft.

Note: *All piles are hollow with the exception of the 16-in. concrete pile at location of T2.
+ Depth below pile top

The strain gages are temperature compensating embedded sisterbar resistive type and are designed specifically to handle the high accelerations associated with pile driving and water pressure at a depth of 200 feet.

In order to study the rate of pile setup, dynamic testing with Pile Driving Analyzer (PDA) was performed on the piles at various time intervals from 2 hours to 5 days after initial driving. Either static load test or statnamic load test were conducted on these piles five to seven days after installation. Due to the high capacities anticipated for the two 54-inch cylinder piles, they were tested with a statnamic device. The remainder of the piles was statically tested in accordance with Section 5.6 of ASTM D1143, "Standard Test Method for Piles under Static Axial Compressive Load" using the quick loading procedure.

STATNAMIC LOAD TEST ANALYSIS PROCEDURE

The statnamic load tests were analyzed using the Segmental Unloading Point (SUP) method as described in Mullins, et al. (2002). This is a refined method over the original Unloading Point Method (UPM) (Middendrop et al, 1992). The shortcoming of UPM is that when wave number is less than 10, there is a phase lag between the pile top and pile bottom movements thus violating the rigid body assumption. The SUP method improves the UPM by discretizing the foundation into smaller segments so that each segment will meet the rigid body assumption. Even with the improvement of the SUP method, judgment is still needed to determine the damping coefficient to remove the dynamic effect. For friction piles, a rate constant is applied. This rate constant was calibrated based only on few load tests and may impact the interpreted capacities. A rate constant of 0.6 was used for the two statnamic tested piles.

LOAD TESTS

Initial pile capacities were predicted using the Tomlinson's α-method and Nordlund method for cohesive soils and coehsionless soils, respectively. Instead of using individual strength tests, a Su/P' ratio of 0.17 was used to predict the pile capacities. Nine piles were tested during the geotechnical investigation stage. The results of the testing program are tabulated in Table 2. Note that the two 54-in cylindrical piles were tested using statnamic load devices. The predicted capacities of six of the nine piles are within 5% of the measured values; two are within 15%; and one is within 25%. These results are presented graphically in Fig. 3. It is worth noting that the two piles at the T-5 location were spaced at a distance of less than 10 feet apart. Yet, the measured capacity for the 170-foot pile was only 35 kips more than the 145-foot pile. The measured unit skin friction for the shorter piles was about 1.8 ksf, much greater that the measured value of 0.6 ksf for the longer pile at the same depth. The spike in the resistance of the shorter pile may be due to the 5-foot layer of sandy soils which was found from a nearby CPT sounding. This sandy layer was also found in the soil boring at this location; however, it is presented as alternate layers of thin sands and clays. The measured skin friction for the longer pile probably reflects the soil conditions from this soil boring. With the exception of the 145-foot pile at this location, the predicted pile capacities using Tomlinson's α-Method and Nordlund Method matched measured capacity quite well.

Table 2. Results of Pile Load Tests and Predicted Pile Capacities

Pile Type	Location	Predicted Capacity (kips)	Load Test Capacities (kips)	Primary Resistance
16-in Sq. Conc.	T2	322	427	Friction
54-in Cyl. Conc.	T2	1299	1295	Friction
30-in Sq. Conc.	T3	1639	1650	Bearing
30-in Steel Pipe	T3	1375	1597	Bearing
54-in Cyl. Conc.	T3	1366	1395	Friction
24-in Sq. Conc.	T4	892	861	Friction
24-in Sq. Conc.	T4	1574	1656	Bearing
24-in Sq. Conc.	T5	790	775	Friction
24-in Sq. Conc.	T5	641	740	Friction

The two piles tested with a statnamic device showed measured values matched the predicted values. Assuming the predicted capacities of these two piles had similar accuracy as the statically tested piles, it may be inferred from the deviations of static load tests to the predicted capacities of other test piles that the statnamic load tests using SUP had a maximum error of less than 25 percent, most likely to be within 15%.

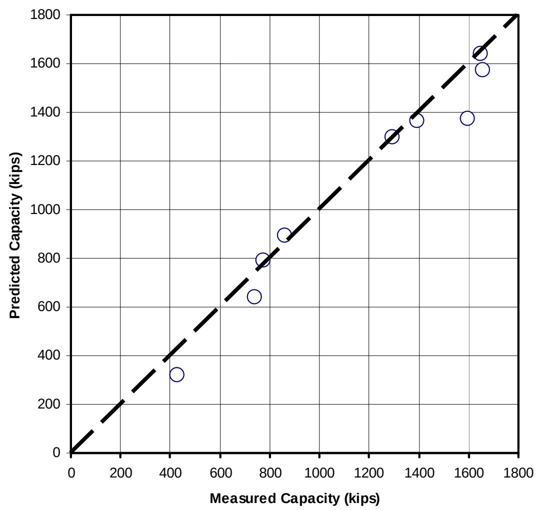

Fig. 3. Measured vs. Predicted Capacities

PILE SETUP

Other than to obtain the final capacities, the primary objective of the load test program was to understand the pile set-up behavior. Due to large amount of the data collected, only detailed results from square concrete piles are presented. The inference drawn from the test program, however, is applicable to all test piles. Tables 3 through 6 show the test results of these piles.

Table 3. Load Test Results – 16-inch, 130-ft Concrete Pile at T2

Testing Event	Elapsed Time (hours)	R_u (kips)	R_s (kips)	R_t (kips)
EOID	0	49	14	35
2-hr Restrike	2.2	178	155	23
4-hr Restrike	3.9	210	176	35
6-hr Restrike	6.0	243	205	38
22-hr Restrike	21.6	383	258	125
55-hr Restrike	56.0	434	311	122
76-hr Restrike	76.9	474	341	134
96-hr Restrike	96.9	473	339	133
Static Load Test	168.0	427	400	27

Note: R_u (ultimate resistance), R_s (skin Friction resistance), and R_t (tip bearing resistance)

Table 4. Load Test Results – 24-inch, 160-ft Concrete Pile at T4

Testing Event	Elapsed Time (hours)	R_u (kips)	R_s (kips)	R_t (kips)
EOID	0			
2-hr Restrike	2.0	389	302	87
4-hr Restrike	3.6	475	381	94
6-hr Restrike	5.8	517	412	105
24-hr Restrike	20.6	625	518	107
48-hr Restrike	44.9	820	666	154
72-hr Restrike	68.5	832	677	155
96-hr Restrike	89.2	880	724	156
Static Load Test	144.0	861	776	85

Note: R_u (ultimate resistance), R_s (skin Friction resistance), and R_t (tip bearing resistance)

Table 5. Load Test Results – 24-inch, 145-ft Concrete Pile at T5

Testing Event	Elapsed Time (hours)	R_u (kips)	R_s (kips)	R_t (kips)
EOID	0	49	14	35
3-hr Restrike	2.6	178	155	23
4-hr Restrike	4.2	210	176	35
24-hr Restrike	21.7	383	258	125
48-hr Restrike	46.6	600	300	299
76-hr Restrike	70.0	654	327	326
96-hr Restrike	90.6	641	314	327
Static Load Test	144.0	739	696	43

Note: R_u (ultimate resistance), R_s (skin Friction resistance), and R_t (tip bearing resistance)

Table 6. Load Test Results – 24-inch, 170-ft Concrete Pile at T5

Testing Event	Elapsed Time (hours)	R_u (kips)	R_s (kips)	R_t (kips)
EOID	0			
3-hr Restrike	3.2	415	225	191
5-hr Restrike	5.3	426	227	199
7-hr Restrike	7.5	469	264	205
24-hr Restrike	23.6	566	361	205
48-hr Restrike	48.1	561	356	205
72-hr Restrike	72.0	748	518	230
96-hr Restrike	92.2	818	598	220
Static Load Test	144.0	769	680	89

Note: R_u (ultimate resistance), R_s (skin Friction resistance), and R_t (tip bearing resistance)

The static load tests show that the measured end bearing values are very close the end bearing values from the end-of-drive capacities. The higher end bearing from the dynamic testing from the longer term restrikes may be a result of low pile set values. Based on that observation, one can assume that the end bearing does not increase with time. Bullock et al. (2005a, 2005b) also observed this phenomenon. It is, therefore, inferred that the time-related capacities are as a result of increasing skin friction with time only.

Many set-up relationships have been proposed by researchers. Most of these relationships assume that pile capacities increase logarithmically with time. The problem with these logarithm of time models is that the ultimate capacity cannot be determined. To overcome this shortcoming, some models fit the set-up curves with hyperbolic relationships. An attempt was made to find a most consistent model that could describe the set-up behavior for this project. This attempt was not successful. It was decided that Skov and Denver (1988) relationship be used. Their empirical relationship can be expressed as

$$\frac{Q_t}{Q_0} - 1 = A\left[\log\left(t/_{t_0} \right)\right] \tag{1}$$

Where Q_t = axial capacity at time t after driving
Q_0 = axial capacity at time t0
A = a constant range from 0.1 to 0.8
t_0 = an empirical value in days

The A constant is a measurement of the rate of capacity increase. Pile size, soil types, and pile types are the dominant factors affecting the magnitude of this constant. The capacities Q_t and Q_0 from the original study refer total capacities. Bullock (2005a, 2005b) suggested that only skin friction component be used for the relationship. The results from this project support Bullock's postulation. As an illustration, the correlation of the 16-inch square concrete test pile is presented in Fig. 4. Note that the elapsed time for the end-of-initial-driving (EOID) is assumed to be about 10 minutes. Bullock et al. used one minute based on load tests conducted in Florida. However, it is found from the load test program at this site that an elapsed time of 10 to 15 minutes for the EOID conditions produced the most reasonable fit as shown in Fig. 4. Note this stage of pile set-up may not be liner with respect to logarithm of time (Komurka and Edil, 2003). Assuming an elapsed time for the end-of-driving condition is to provide a point for extrapolation only and is not a necessary condition to establish the set-up relationship.

Based on the results of this test pile, the A constant is about 0.44 indicating a 44 percent increase in friction resistance with each log cycle of time increase. For example, the project capacity at 10 days would be 44 percent higher than the measured skin resistance at 1 day. Based on the one-day capacity of 276 kips and 27 kips for skin friction and end bearing resistances, respectively, the 10-day capacity would be calculated to be about 397 kips and 27 kips for skin friction and end bearing resistances, respectively.

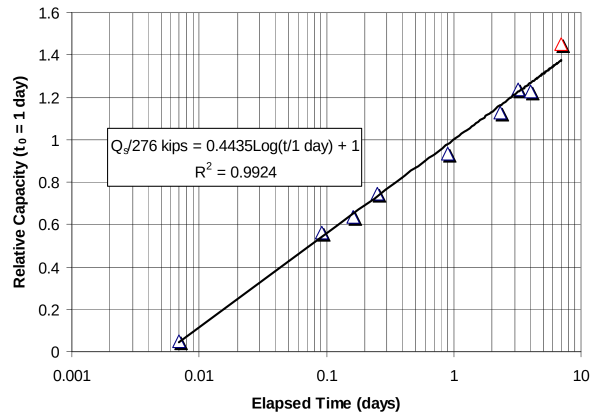

Fig. 4. Pile Set-up Relationship – 16-inch, 130-ft pile at T-2

The set-up parameters from the remainder of the test program are tabulated in Table 7. This table includes two additional load tests (T-1 and T-11) performed during bridge construction and the relatively long term testing of production piles (two at bent NC29). As presented in Tables 3 most static and dynamic load tests were performed within a relatively short period of time, less than 10 days, after initial driving. The inclusion of the 72-day tests for the piles at bent NC29 further improve the understanding of the pile set-up behavior. The piles listed in Table 7 are arranged in a north to south direction so that any trend, if any, can be observed.

Table 7. Set-up Relationships from All Piles

Pile Type	A	Q_0 (kips)	R
T-1, 24-inch, 155 ft	0.28	596	0.98
T-2, 16-inch, 130 ft	0.44	276	0.99
T-2, 54-inch cyl., 160 ft	0.35	824	0.94
T-3, 30-inch , 190 ft	0.25	490	0.84
T-3, 30-inch steel, 195 ft	0.33	699	0.95
T-3, 54-inch cyl., 160 ft	0.31	897	1.00
T-4, 24-inch, 160 ft	0.44	573	0.99
T-4, 24-inch, 210 ft	0.29	618	0.91
T-5, 24-inch, 170 ft	0.59	386	0.85
T-5, 24-inch, 145 ft	0.47	262	0.98
T-11, 24-inch, ft, 155 ft	0.69	429	0.94
NC 29, 24-inch, 125 ft	0.63	206	0.91
Average	0.42		

Note: A – Set-up Constant, Q_0 – 1-day Friction Resistance, and R – Coefficient of Correlation.

The A constant from this project ranges from 0.25 to 0.69. These values are comparable to published case histories and are deem reasonable. By comparing the results with the sizes and length of the piles, it appears that the set-up rate increases southward. Q_0 (1-day capacity) decreases also southward reflecting of the weaker soils toward south. No direct relationship can be established for pile sizes and depths. This trend matches the geological deposition sequence that the soils at the southern end of the alignment are newer deposits and are weaker in general. The coefficients of correlation (R) range from 0.84 to 1.00. Only two of these correlations had R coefficients below 0.9. These are excellent correlations. Based on the test results, a set-up curve was developed for the contract to evaluate the pile capacities at various time increments. A 24-day restrike was specified on all bents so that the pile capacities may be projected.

It was surprising that the only steel pile tested had higher set-up rate, Q_0, and final capacity while having lower pile driving resistances during installation. The strain gage measurements indicated much greater unit skin frictions from the steel pipe pile than those from the concrete piles at the same location.

SUMMARY

The extensive load tests for this project afford the opportunity to verify the calculated capacities and to study the pile set-up phenomenon. The results of the load test program can be summarized below.

- The pile capacities estimated using Tomlinson's α and Norlund's methods are within 20% of the load test values at this site.
- Using a Su/P' ratio of 0.17 for the under-consolidated clays sucessfully predicted the pile capacities.
- Skov and Denver set-up relationship is applicable to the soil conditions similar to this project. The long-term capacity may be used to improve pile designs in geologically similar soils.
- The rates of pile set-up at this site range from 25% to 69% per log cycle.
- Pile sizes and depths do not impact the set-up rate constant (A). The stress history of the soils appears to have a greater impact to the rate constant.
- Piles may still gain capacities even after 72 days at this site.

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