COMPARISON OF DYNAMIC, STATIC, AND STATNAMIC AXIAL LOAD TESTING ON CONCRETE PILES IN SAVANNAH, GA.

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ABSTRACT

An indicator pile program was developed and tested in order to refine the pile design for a new 1,000,000 gallon liquefied natural gas (LNG) tank and supporting facilities in Savannah, GA. A total of over 2,000 production piles were planned for this project. A test pile program, consisting of a total of five (5) concrete piles of various sizes and lengths, was designed and implemented to refine the production pile design. The test piles were subjected to high strain dynamic, static, and STATNAMIC axial load testing over a two (2) week period.

The following paper discusses the development of the test pile program, describes the test piles and testing, and presents the results of the axial load testing. In addition, a comparison of the various type test results is presented and the results are discussed in relation to time dependent pile capacity gain (i.e. "setup" or "freeze"). The load testing program showed that the length of the production piles could be reduced by 3.7m (12 ft), resulting in a substantial savings for the project.

INTRODUCTION

An expansion of the existing liquefied natural gas (LNG) facility on Elba Island, Georgia consisted in a new one million barrel LNG tank, associated containment dike and process facilities and a new slip dock. With the exception of the containment dike, all structures (tank, process equipment, pipe supports and dock) are to be supported on driven pile foundations. Due to the critical nature of the expansion structures and the seismicity of the region, preliminary estimates yielded a total of 1600 piles for the new tank alone and between 2000-3000 for the entire project.

The site is located on Elba Island, which is a former dredge disposal island located in the Savannah River near Savannah, Georgia. A comprehensive geotechnical subsurface exploration and seismic hazard study conducted at the site showed a relatively consistent soil profile across the site. This soil profile is shown graphically in Figure 1 and is summarized in Table 1.

The critical nature of the facility required that the design be subjected to review by the Federal Regulatory Commission (FERC) and meet the stringent federal requirements for the evaluation of seismic risk. Based on the preliminary foundation design conducted by Chicago Bridge & Iron (CB&I), each pile was required to have an ultimate axial compression capacity of 1780kN (400kips) and an allowable axial compression capacity of 890kN (200kips) using a factor of safety of 2.

Table 1. Generalized Subsurface Profile.

Soil Type	Elevation[1] (ft)	Height (ft)	Geologic Period
Hydraulic Fill/ Surface Crust	11 to 7	4	Recent Dredge Spoil
Soft Clay (CH)	7 to -25	32	Pleistocene
Medium Dense Sand (SP)	-25 to -40	15	Pleistocene/ Miocene
Very Stiff Sandy Clay (Marl)	-40 to -95	55	Miocene
Very Dense Clayey Sand (SC)	-95 to -105	10	Miocene
Very Dense Silty Sand (SM)	-105 to -140	35	Miocene
Limestone	-140 to <-370	>230	Oligocene Eocene

NOTES:
1. Referenced from MLW.

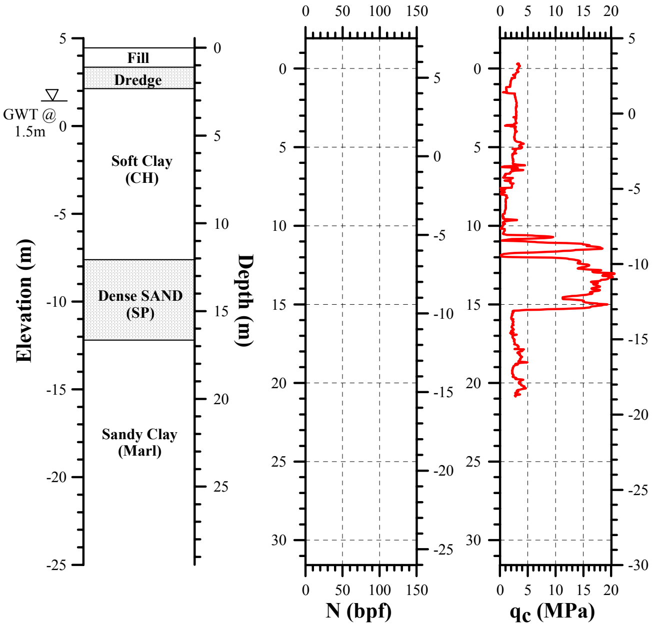

Fig. 1. Site General Soil Profile.

TEST PILE PROGRAM

Due to the large number of piles associated with the project, a test pile program was designed and implemented to optimize pile design and construction planning. The test pile program was designed specifically to achieve the following three objectives:

1. Test axial pile capacity for the determination of the design pile length
2. Study the densification effect from the pile driving by re-evaluating the liquefaction potential after pile driving
3. Test the lateral behavior of the piles for use in the design of pile and pile cap and connection details, particularly the pile behavior under cyclic lateral loads.

This paper details the axial capacity load testing portion of the program. Objective 2 (i.e. re-evaluation of liquefaction potential) is described by Lin et al. (2003), while objective 3 (i.e. the lateral load testing) is detailed by Brown et al. (2003).

The test pile program incorporated a total of five piles driven in a group located near the center of the site. The test piles included two (2) instrumented 18 inch square Pre-cast Pre-stressed Concrete (PPC) piles, two (2) 18 inch square PPC piles without instrumentation, and one 17.7 inch diameter hollow Pre-cast Pre-stressed pile (labeled as ICP after the manufacturer). The two square piles without instrumentation were termed “dummy” piles as they were installed to form a pile group similar to the pile configuration under the tank in order to evaluate the densification effect associated with pile driving. Table 2 provides a summary of the five (5) test piles while Fig. 2 presents a plan view of the test pile layout. All four square PPC piles were manufactured at Standard Concrete Product in Savannah. The ICP pile was manufactured in Malaysia and marketed by Pipe & Piping Concrete USA in Florida.

The test pile group was installed on a test pad of construction fill approximately 12.2m (40ft) square by 1.2m (4ft) high above the existing ground surface. The fill, comprised of fine to coarse sands transported from the existing dredged material containment area (DMCA) located on the north end of the island, was compacted with a dozer in the lower layer and a vibratory roller on the upper layers. The top of test pad has an elevation of 4.46m (14.6ft) MLW, which is within 0.1m (0.3ft) of the design foundation pad elevation. The test pad fill is identical to the fill that would be placed at the site prior to driven pile installation.

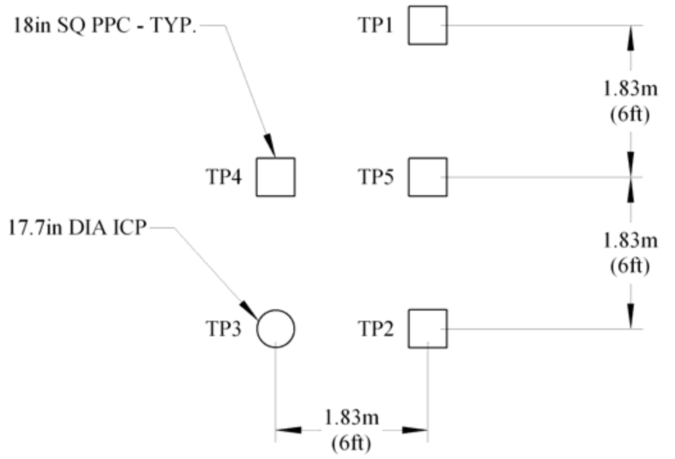

Fig. 2. Test Pile Layout.

Table 2. Pile Details and Instrumentation

Pile No.	Type	L[1] (ft)	Instrumentation		$f'_{c\ 28day}$[4] (ksi)
			SG[2]	Incl.[3]	
TP1	18-in sq PPC	85	Y	Y	10.33
TP2	18-in sq PPC	75	Y		11.98
TP3	17.7 in dia. ICP	98[5]			10.00[5]
TP4	18-in sq PPC	65			10.34
TP5	18-in sq PPC	65			9.94

NOTES:
1. Length.
2. SG = Strain Gages.
3. Incl = Inclinometer casing for lateral load testing.
4. 28 day compressive strength.
5. Total length comprised of 2 49ft sections.

The five (5) test piles were installed using an APE D36-32 single acting open-end diesel hammer, which has a ram weight of 35.3 kN (7.94 kips) and maximum rated energy of 113 kN-m (83.33 kip-feet). The hammer was handled with a Model 222 Manitowoc 100 ton crane with a swing lead. The pile cushion consisted of an oak hardwood and plywood with a total cushion thickness of 152.4mm (6 inches). The 4 day hammer restrikes (4DR) were conducted with the same hammer.

Each pile was installed in a pilot hole made by vibrating a HP14x89 pile between depths of 7.6 to 10.7m (25 to 35 ft) below the top of the test pad. In order to reduce disturbance to the subsurface and help maintain vertical alignment of the pile, each pile was placed in its pilot hole prior to vibrating the remaining pilot holes.

The axial load testing consisted of high strain dynamic, static, and STATNAMIC load tests performed within a 12 day period in September, 2002. A summary of the various axial load tests is provided in Table 3. The restrictive design and construction timetable did not allow for an extended testing period. The dynamic testing was conducted WPC Inc. personnel using the Pile Driving Analyzer (PDA) in accordance with ASTM D4945. The static load testing was conducted by WPC Inc. and Applied Foundation Testing (AFT) personnel using the quick loading method specified in ASTM D1143. The STATNAMIC testing was conducted by AFT personnel using a 4.5MN (500 ton) STATNAMIC device equipped with a hydraulic catching mechanism in accordance with standard testing practice. Refer to Brown (1994) for details concerning STATNAMIC testing.

Table 3. Axial Capacity Testing Program Summary.

Pile No.	Type	EOD[1]	4DR[2]	6D STN[3]	12D SLT[4]
		X	X	X	
TP2	18-in sq PPC	X	X	X	X
TP3	17.7 in dia. ICP	X	X	X	
TP4	18-in sq PPC		Not Tested		
TP5	18-in sq PPC	X	X		

NOTES:
1. EOD = End Of Driving (Dynamic)
2. 4DR = 4 Day Restrike (Dynamic)
3. 6D STN = 6 Day STATNAMIC test
4. 12D SLT = 12 Day Static Load Test

TEST PILE PROGRAM RESULTS

A summary of the test pile program axial capacity results is presented in Table 4. The load-displacement-time results of the 12 day SLT on TP2 are presented in Figure 3. The individual test pile axial capacities with time are presented in Figures 4 through 7 for TP1, TP2, TP3, and TP5, respectively.

Pile capacity from the dynamic load testing was determined using signal-matching program CAPWAP, developed and marketed by GRL Engineers, Inc. of Cleveland, Ohio.

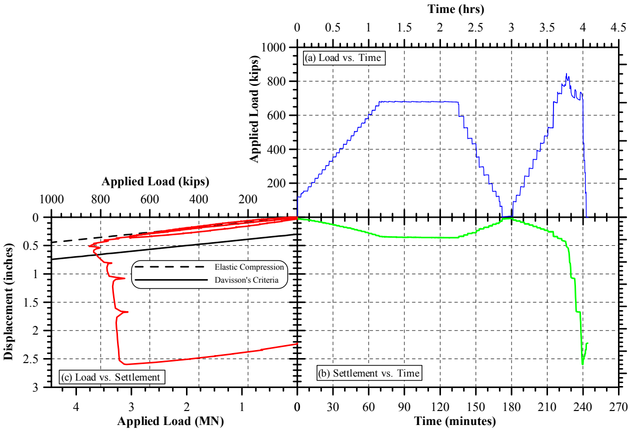

Fig. 3. TP2 12 Day Static Load Test Results.

For the static load test, the offset failure (i.e. Davisson's) criterion (Davisson, 1972) was used to determine the total pile capacity. Davisson's criterion is commonly used in geotechnical practice throughout the US and has been statistically shown to be the best overall method for determining failure of deep foundations and was therefore chosen as the single method to be used when analyzing load-displacement curves for LRFD deep foundation design (Paikowsky and Stenerson, 2000).

The static axial capacities from the STATNAMIC testing for TP1 and TP2 were derived based on the Segmental Unloading Point (SUP) method (Lewis, 1999). To perform this analysis, each pile is initially divided into segments defined by the locations of the embedded strain gauges. The Standard Unloading Point (UPM) method (Middendorp et al. 1992) is then applied to each segment to derive the static response of the pile. Rate of loading (i.e. rate effect) factors (η) of 0.64, 0.92 and 0.92 were used to analyze the clay, sand and marl segments respectively. The rate effect factors are described by Paikowsky et al. (2003). For TP3, the static axial capacity was derived using the Standard Unloading Point (UPM) method was applied to the entire pile length as a single segment. A rate of loading factor was established at 0.91 based on a comparison of the results from the Segmental Unloading Point analysis of TP1 to the Standard Unloading Point analysis of TP1.

Additional analysis of the STATNAMIC test results for the three test piles showed that the final displacements (i.e. permanent set) were insignificant (i.e. < 2 mm). Similar analysis of the 4DR dynamic tests for all 4 tested piles also showed small permanent displacements. Table 5 presents a summary of the individual pile displacement during the STATNAMIC and 4 day dynamic restrikes. Since the permanent displacements for these tests are small, it can be assumed that the piles did not experience failure (i.e. the ultimate pile capacity was not mobilized) and the presented axial capacity in Table 4 represents the capacity mobilized during the individual tests.

Table 4. Axial Capacity Testing Program Summary.

Pile No.	**Type**	**Axial Load Test Results (kN)**			
		EOD[1]	**4DR**[2]	**6D STN**[3]	**12D SLT**[4]
TP1	18-in sq PPC	1646	2847	3959	
TP2	18-in sq PPC	1673	2798	3305	3768
TP3	17.7 in dia. ICP	1094	2856	3812	
TP4	18-in sq PPC	Not Tested			
TP5	18-in sq PPC	1770	2037		

NOTES:
1. EOD = End Of Driving (Dynamic)
2. 4DR = 4 Day Restrike (Dynamic)
3. 6D STN = 6 Day STATNAMIC test
4. 12D SLT = 12 Day Static Load Test

Table 5. STATNAMIC Testing Displacement Summary.

Pile	**Test Type**	**Axial Capacity (kN)**	ρ_{maximum}[1] **(mm)**	ρ_{resdiaul}[2] **(mm)**
TP1	4DR	2847	10.4	0.74
	STN	3959	12.7	0.76
TP2	4DR	2798	9.4	0.58
	STN	3305	12.7	1.52
TP3	4DR	2856	16.3	0.13
	STN	3812	20.8	0.76
TP5	4DR	2037	9.4	2.54

NOTES:
1. Maximum displacement (DMX during dynamic testing)
2. Residual displacement (i.e. permanent set). For dynamic tests, permanent set = 1/blowcount.

To further examine the differences between the static and STATNAMIC load tests, the load-displacement data at the pile top was compared between the two test types for TP2. This data is presented in Figure 8.

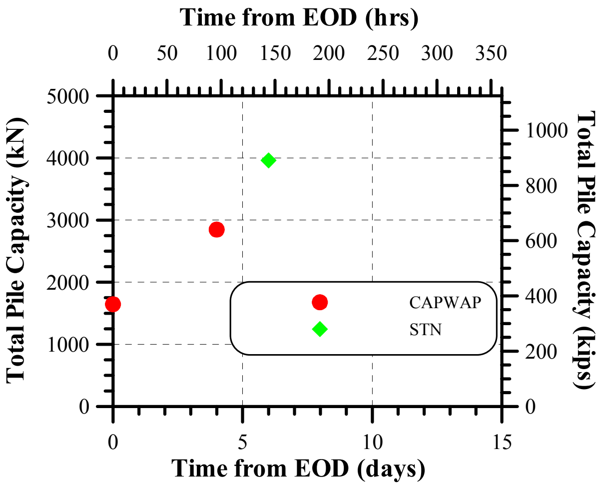

Fig. 4. TP1 Axial Pile Capacity with Time.

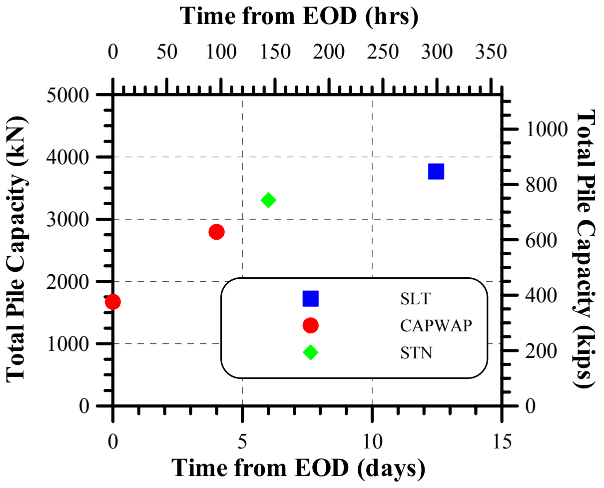

Fig. 5. TP2 Axial Pile Capacity with Time.

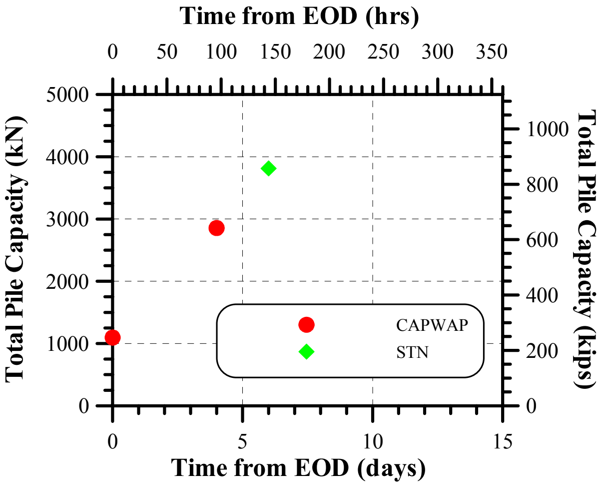

Fig. 6. TP3 Axial Pile Capacity with Time.

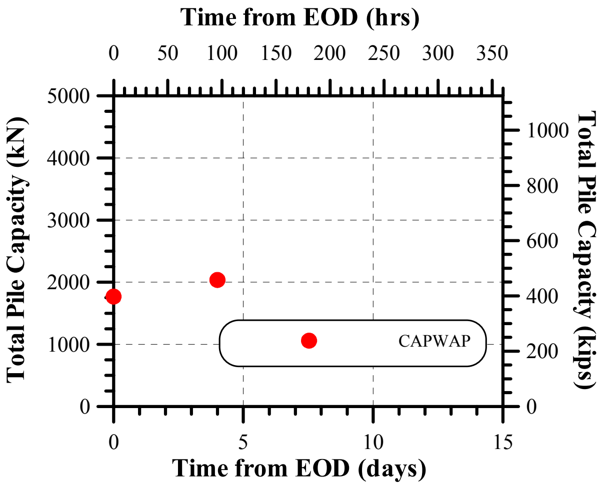

Fig. 7. TP5 Axial Pile Capacity with Time.

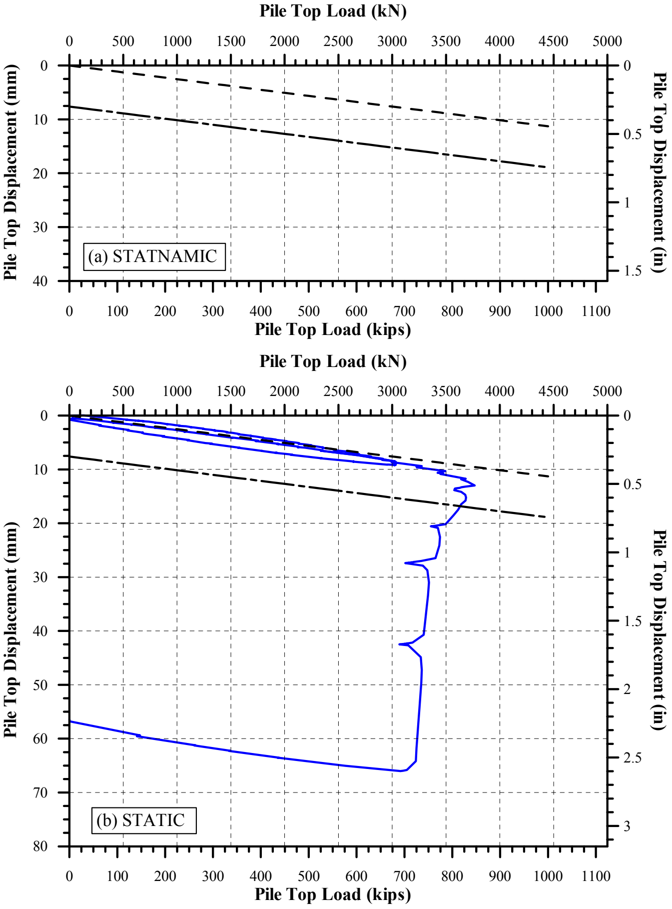

Fig. 8. TP2 STATNAMIC and Static Load test Load-Displacement Results.

As shown in Fig. 8, the STATNAMIC force shows that the pile load- displacement relations during the STATNAMIC tests was primarily elastic, with insignificant permanent set as previously mentioned. The results of the STATNAMIC force-displacement analysis show no indication of failure based on load - displacement relations or as defined by Davisson's criteria. Figure 8 also shows that the derived static - displacement curves followed the same trend as the STATNAMIC force - displacement curve. Similar to the STATNAMIC force-displacement relationship, the derived static analysis show no indication of failure based on load - displacement relations or as defined by Davisson's criteria.

Similar load-displacement results showing primarily elastic deformation with minimal permanent set were observed for STATNAMIC tests in Boston Blue Clay (Hajduk et al. 1998). Similar behavior was also found for STATNAMIC tests in clay soils as reported by McVay et al. (2003). The unfavorable results obtained by Hajduk et al. (1998) suggested an influence of soil viscosity and inertia alongside buildup of pore pressure in fine-grained soils and that current STATNAMIC analysis methods may be inadequate for cohesive soils (Hajduk et al. 1998). However, in the presented data, it appears that not enough force was applied to the pile during STATNAMIC testing to fully mobilize the pile capacity. Therefore, the ability of STATNAMIC testing to independently determine pile capacity at this site could not be verified.

As shown in Table 4, the STATNAMIC results match well with the static load test results for TP2. This match is the result of the application of rate effect factors previously mentioned. Rate effect factors are soil type-dependent multiplier used to reduce the load capacity predicted by analyses when using the various Unloading Point analysis methods (Paikowsky et al. 2003). Essentially, the rate effect factor “fits” the STATNAMIC results to static load test results. The methodology behind the development of the rate effect factors is presented by Paikowsky et al. (2003). Since the STATNAMIC tests showed no clear failure and the rate effect factors “calibrated” the STATNAMIC load results to the static load test results for this project, their use for pile design was limited.

Time Dependent Pile Capacity Gain

As shown in Figures 4 through 7, all four tested piles experienced time dependent pile capacity gain, which is commonly called pile “setup” or “freeze”. This gain was expected, given the cohesive soil deposits located at the site.

As stated previously, the restrictive time schedule prevented long term axial load testing of the five test piles. Furthermore, budgetary concerns prevented multiple load testing of any of the three axial tests (i.e. dynamic, static, and STATNAMIC) within the testing timeframe. Therefore, quantitative time dependent pile capacity gain analysis could not be conducted from the load test program results. However, the following general trends were observed:

- Comparison of the dynamic tests for TP1 and TP5 showed that at the end of driving (EOD), these two piles had approximately the same capacity (see Table 4). However, at the 4 day restrike (4DR), TP1 had 810kN (182 kips) additional capacity than TP5. This can be explained by the 6.1m (20ft) additional embedment of TP1 within the Marl layer.

- With the exception of TP5, the tested piles experienced a similar capacity gain between the EOD and 4DR.
- The STATNAMIC test results correlate well the general trend of time dependent capacity gain for the three (3) piles with STATNAMIC and dynamic load tests. However, since the ultimate pile capacity was not mobilized during either of these tests, it is most likely not representative of the total pile capacity gain.

Use of Load Testing Program Results for Production Pile Design

The final soil strength values used for the design of the production piles were taken from the static load test results on TP2 since the static load test was the only test where failure was experienced. The data from the dynamic and STATNAMIC load tests, which did not experience failure, confirmed the axial capacities of the static load test.

In general, soil strength values derived from this pile test program were generally larger than those used in the pile capacity estimate in the original report. New design charts are developed based on the revised soil strength values, which showed that an 18inch square PPC pile driven to a tip elevation of (-50ft) MLW is required for an 18-inch PSC pile would develop the required 200 kips compression capacity with a factor of safety of 2. This corresponded to a reduction of 3.66m (12ft) in pile length for each 18-inch square PPC pile and a substantial savings to the client.

SUMMARY AND CONCLUSIONS

A load test program was designed and conducted for a LNG plant expansion in Elba Island, Georgia to determine axial pile capacity for use in the production pile design. The program consisted of five (5) test piles subjected to high strain dynamic, static, and STATNAMIC load tests. The results of the static and dynamic load tests showed that the length of the production piles could be reduced by 3.7m (12 ft), resulting in a substantial savings for the project.

Based on the presented data and analysis, the following conclusions were drawn:

- Small permanent displacements of the STATNAMIC and dynamic restrikes indicate that these tests did not fully mobilize the ultimate capacity of the piles. However, the mobilized capacities did confirm the results of the static load tests.
- The primarily elastic deformation of the STATNAMIC tests also suggest that these test did not fully mobilize the ultimate pile capacity. Therefore, the ability of STATNAMIC testing to independently determine pile capacity at this primarily cohesive soil site could not be verified.
- The load test program showed that this site experienced time dependent pile capacity gain. However, the limited testing schedule, coupled with the failure to mobilize ultimate pile capacity from the 4DR and STATNAMIC tests, prevented quantifiable setup analysis.

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