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Instrumented Pile Load Testing Program for a Coal-Fired Power Plant

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ABSTRACT: An extensive preproduction probe pile load testing program was conducted for a coal-fired power plant in New Jersey to minimize pile performance uncertainty resulting from variable subsurface conditions, and allow the selection of the most cost effective and technically suitable of three proposed piling systems for the project. Three types of piles were instrumented and tested: Monotube, closed-end pipe, and step-tapered. This manuscript includes a general description of the site and its geologic conditions, along with subsurface and groundwater conditions in the vicinity of the pile load test areas. A summary of the test pile installation, including driving records and dynamic monitoring, is provided, and results are compared with geotechnical data obtained from borings drilled in the pile load test areas. Compressive static pile load test results are presented, and pile instrumentation results are interpreted. Average unit shaft resistance values are calculated and compared to field-measured shearing strength of site soils. Comparison of pile behavior based on the results of the load test program, along with cost and schedule considerations, served as the basis for the selection of the production piles for the facility.

INTRODUCTION

The power plant is located adjacent to the Delaware River, across the river from Wilmington, Delaware. The 202-megawatt, coal-fired cogeneration facility produces electricity for southern New Jersey and process steam for a neighboring chemical plant. A schematic layout of the main plant (power block area) is shown in Figure 1, which also includes the locations of clusters of preproduction probe piles and boreholes drilled near these clusters.

The preproduction probe pile load testing program described here was conducted to; 1) minimize uncertainty in pile performance (driving characteristics and capacity assumptions) resulting from variable subsurface conditions, 2) to develop production pile driving criteria, 3) to verify compliance with the applicable building code (BOCA 1990) requirements, and 4) to provide data for possible pile design optimization, if allowed by the project schedule. Twenty four preproduction probe piles were driven throughout the power block area. The probe piles were driven at locations T-1 through T-8 shown in Figure 1. Three types of piles were driven at each location: Monotube pile, closed end, straight shaft pipe pile, and step-tapered pile. At each location the three piles were driven in a row with the pipe pile at the center, and the center-to-center spacing between piles was equal to about 1.5 m (> 3.5 pile diameters for the largest pile). No pile cap was used to connect the three piles, and the three piles driven at each of the locations T-1 through T-8 will be referred to as pile clusters. A complete suite of static load testing (compression, tension, and lateral) was performed on each individual pile at cluster T-2 and on the step-tapered piles only at cluster T-8, shown in Figure 1.

This manuscript provides a summary of the site geology and existing subsurface and groundwater conditions in the vicinity of the test pile clusters, with emphasis on T-2 and T-8. Probe pile driving at T-2 and T-8 are discussed, and results are presented. Descriptions of the test pile instrumentation and static compressive load test program at T-2 and T-8 are presented along with an evaluation of the static load test results, including interpretation of instrumentation results. Average unit shaft resistance values are calculated and compared to field-measured shearing strength of fine-grained site soils. Results of tension and lateral load tests are not included here because of space limitations.

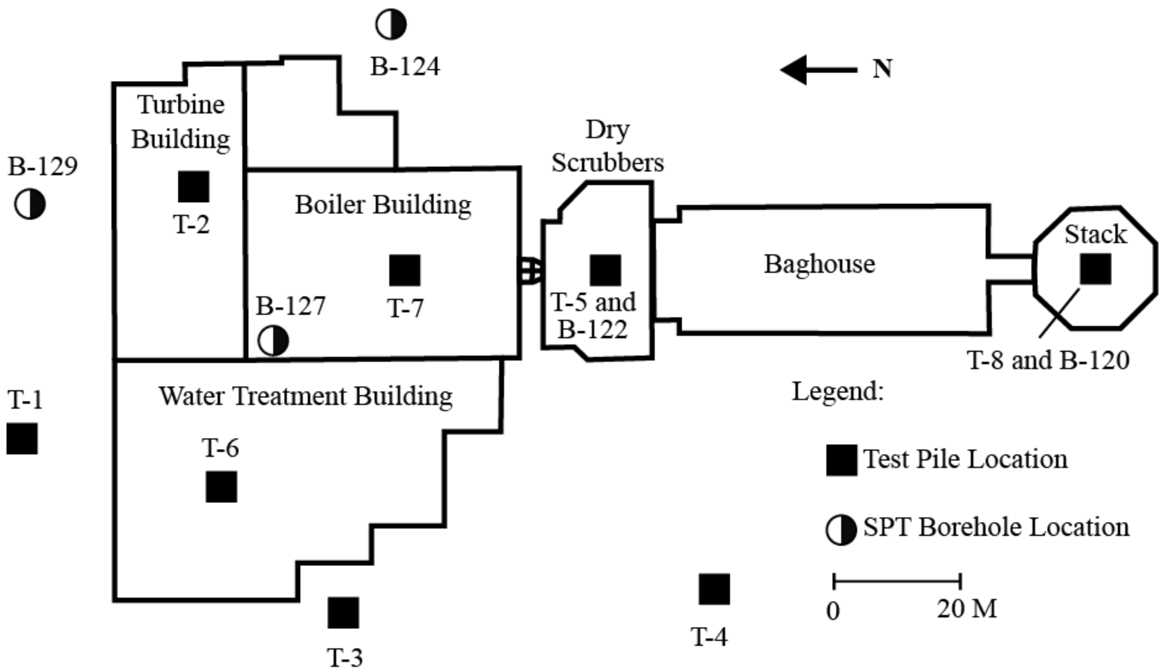

Figure 1. Layout of power block area structures showing locations of borehole and probe pile clusters.

An extensive dynamic testing program consisting of pile driving analyzer (PDA) monitoring with subsequent CAPWAP analyses was performed, and the results are summarized and discussed. Also, noise and ground vibration were monitored during probe pile installation, but results are not included here because of space limitations.

SUBSURFACE AND GROUNDWATER CONDITIONS

The site is located within the Atlantic Coastal Plain Physiographic Province near the southeastern boundary of the Piedmont Physiographic Province. The Fall Line, which defines the boundary between the two provinces, occurs approximately along the northwest bank of the Delaware River, several hundred meters west of the site (Roseneau et al. 1969). The site is underlain by dredged fills and unconsolidated sediments of the Recent, Pleistocene, and Cretaceous age which are, in turn, underlain by consolidated basement rocks of probable Precambrian age.

Fill deposits consisting of dredged spoils from the Delaware River occur at the ground surface in the western portion of the property. The dredged fill consists of very loose to loose, brown silty fine sand. Alluvial deposits, probably of Recent age, overlie the Cape May Formation and consist of loose, fine-grained, gray silty sand (Stratum I) and soft elastic silt/silty clay

(Stratum II). The Cape May Formation, of Pleistocene age, unconformably overlies the Cretaceous formations and consists of gray, medium dense sand (Stratum III). The Magothy and Raritan Formations, of Cretaceous age, unconformably overlie the basement rocks. These formations consist of silt and clay with some lignitic material (Stratum IV) interbedded with dense to very dense sand and gravelly sand (Stratum V). The basement rocks belong to the Wissahickon Formation, which, locally, is comprised of micaceous schist with interbeds of amphibolite and granitoid gneiss (USGS 1967). The basement rocks were encountered in boreholes drilled in the Delaware River at elevations ranging from about -27 m to -35 m; they dip steeply to the southeast toward the power block area but are not reached by boreholes drilled in the power block area.

A subsurface investigation performed at the site included boreholes with Standard penetration test (SPT) (ASTM 2011) and Shelby tube sampling (ASTM 2008a) and the performance of field vane shear testing (ASTM 2008b) in the Stratum II fine-grained soils. Cone Penetrometer Tests (CPT) (ASTM 2012) soundings were also performed throughout the site. Figure 1 shows the locations of boreholes drilled in the vicinity of the test piles discussed in this manuscript. The locations of CPT testing are not shown in Figure 1.

The subsurface profile in the vicinity of probe pile cluster T-2, as interpreted from the boreholes and laboratory test results, is shown in Figure 2. The numbers shown along the depth next to vertical borehole lines represent SPT N-values in blows/300 mm or blows per actual sampler penetration (ASTM 2011). The subsurface conditions disclosed by borehole B-120 (Figure 1) at probe pile cluster T-8 are very similar to those disclosed by borehole B-124, shown in Figure 2.

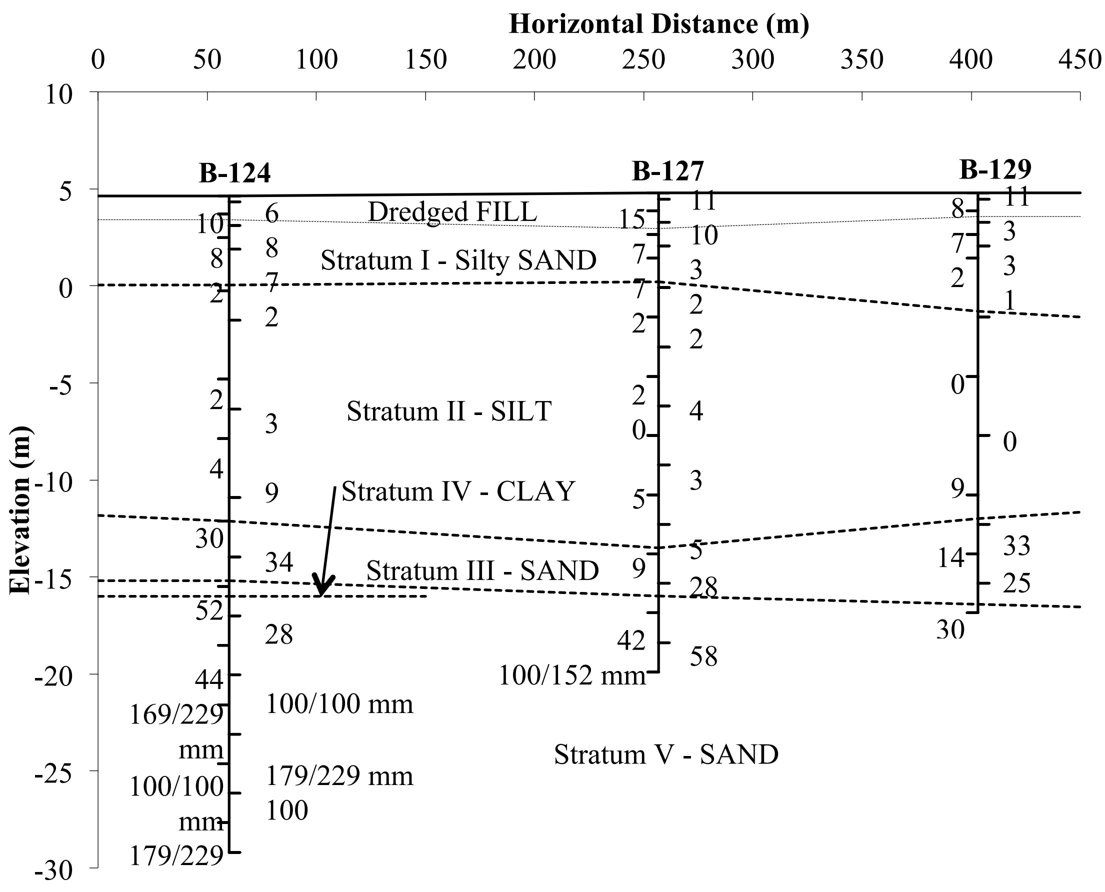

Figure 2. Subsurface profile in the vicinity of probe pile cluster T-2.

The borehole data indicate that fill, consisting of dredged soil from the Delaware River, is present across the power block area. The thickness of the fill ranged from 0.6 m to 1.8 m. The subsurface materials below the fill can be divided into five major strata as previously described. The combined thickness of the upper loose and/or soft Strata I and II ranged between about 15 m and 20 m in the boreholes drilled in the power block area. The pile bearing stratum is the dense to very dense sands and gravels of Stratum V, which is at least 15 m thick in the power block area. Boreholes drilled in the Delaware River to the west of the power block structures shown in Figure 1 encountered weathered rock beneath Stratum V at elevations ranging from about -27 m to -35 m. The top of bedrock surface dipped down significantly toward the east, and boreholes drilled for the power block facilities shown in Figure 1 did not extend to the bedrock.

Field vane shear testing of Stratum II was performed in accordance with ASTM D2573 (ASTM 2008b) at several locations onsite. Both undisturbed and remolded shear strength (s_u) were measured, and the undisturbed shear strength results are shown in Figure 3. The average undisturbed shear strength using all values shown in Figure 3 is about 50 kPa, however variable strengths with depth are noted. These results compared well with s_u-values estimated from CPTs using N_k=15. Shear strength values obtained from triaxial UU and unconfined compression testing of intact Shelby tube samples ranged from about 10 kPa to 53 kPa, with an average of about 32 kPa. Based on all of the data, a value of s_u = 40 kPa was adopted for pile design.

Although shallow groundwater levels were monitored in groundwater observation wells located throughout the site, the design groundwater level was conservatively taken at the ground surface (about El. +4.6 m).

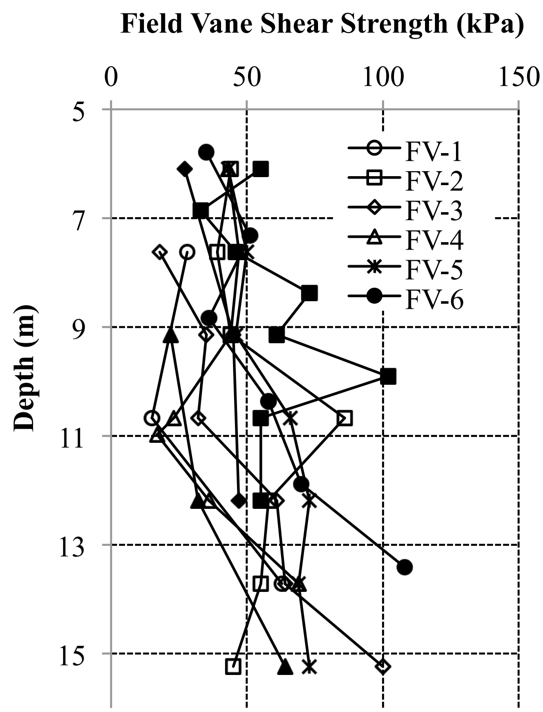

Figure 3. Measured field vane shear strength vs. depth.

The variable thickness of Stratum II, and the variable depth to SPT refusal in Stratum V disclosed by the boreholes is illustrated in Figure 2. This variability in site subsurface conditions created uncertainty related to pile performance (driving characteristics and capacity assumptions). Although the variations were not dramatic, the probe piling and testing program were developed and implemented to minimize this uncertainty prior to installing several hundred piles for the project.

INSTALLATION OF PROBE PILES

Probe piles installed at each of the locations T-1 through T-8 shown in Figure 1 consisted of the following:

- Step-tapered piles – Seven 3.7-m-long sections ranging from section 6 (416-mm diameter) at the top to section 0 (264-mm diameter) at the tip. In addition, a smaller pile was driven at locations T-2 and T-8, consisting of seven 3.7-m-long sections ranging from section 4 (365-mm diameter) at the top to section 000 (219-mm diameter) at the tip. S is used to denote the larger step-tapered piles, and SS is used to denote the smaller step-tapered piles. For example, pile S-2 means the step-tapered pile with the tip diameter of 264-mm, driven at location T-2.
- Pipe pile – 356-mm outside diameter straight shaft, 10-mm wall thickness, with a steel plate welded at the tip. P is used to denote pipe piles.
- Monotube pile – 5 gauge (5-mm wall thickness), 305-mm diameter at the top, tapering down for the last 9.1 m to a 203-mm diameter tip. M is used to denote Monotube piles.

Five pipe piles were driven at location T-9 (not shown in Figure 1) at a center-to-center spacing of 1.5 m (about 4.2 d) to verify whether pile driving would cause ground heave. Less than 20 mm of ground heave was recorded.

Driving Criteria

The piles were designed to support an allowable axial compressive load of 900 kN using BOCA (1990) criteria for allowable load, which is defined as one-half of that load, producing a net pile head settlement equal to 0.03 mm/kN but no more than 19 mm.

The following driving criteria were developed based on pile driving analyses and were generally followed during installation of the probe piles:

- Pipe pile: 10 blows/25 mm for 150 mm with Raymond 2/0 hammer (rated energy of 44 kNm)
- Monotube pile: 8 blows/25 mm for 150 mm with Raymond 80C hammer (rated energy of 33 kNm)
- Step-tapered pile: 6 blows/25 mm for 150 mm with Raymond 80C hammer
- Standard aluminum and micarta discs (combined thickness of about 42 cm) were used for hammer cushion

The pile driving records for probe piles in cluster T-2 and the step-tapered piles in cluster T-8 are shown in Figure 4. The driving behavior of all probe piles shown in Figure 4 (and all other probe piles driven throughout the site) was similar reflecting the observed variable site subsurface conditions. All probe piles were driven into Stratum Vand pile lengths ranged from 22.4 to 24.3 m.

Dynamic Monitoring

Probe Monotube piles M-1 through M-8, and probe pipe piles P-1, P-3, P-4, P-6, P-7, and P-8 were dynamically monitored during driving with a PDA. PDA results indicated mobilized capacities ranging from about 760 kN to 933 kN for the Monotube piles, and from about 1,600 kN to 1,800 kN for the pipe piles. These dynamic test results indicated that the hammer/pile/soil system for the pipe piles was capable of satisfying the design requirements. In the case of the

Monotube piles, this was not the case as the mobilized dynamic capacities were too low. However, considering the high blow counts measured during driving (Figure 4), it was believed that the low mobilized dynamic capacities of the Monotube piles were related to the hammer size used during driving, and it was not considered to be a design or static capacity issue. Static load testing results discussed later in this manuscript confirmed that the Monotube piles were capable of mobilizing sufficient resistance to satisfy the design requirements.

Probe piles P-2, P-8, M-2, and M-8 were also dynamically tested on restrike. There is no exact information regarding the time elapsed between initial driving and the restrike driving. The maximum elapsed time for the pipe piles was 7 days, the maximum elapsed time for the Monotube piles was 1 day, and it seems that pile M-8 was restruck the same day that it was driven. Restriking the piles with the same hammers used during driving resulted in very high blow counts, indicating a condition of practical refusal. The mobilized capacities of these four piles during original and restrike driving are shown in Table 1, along with the ratio of mobilized capacity during restrike over the mobilized capacity during initial driving (Table 1).

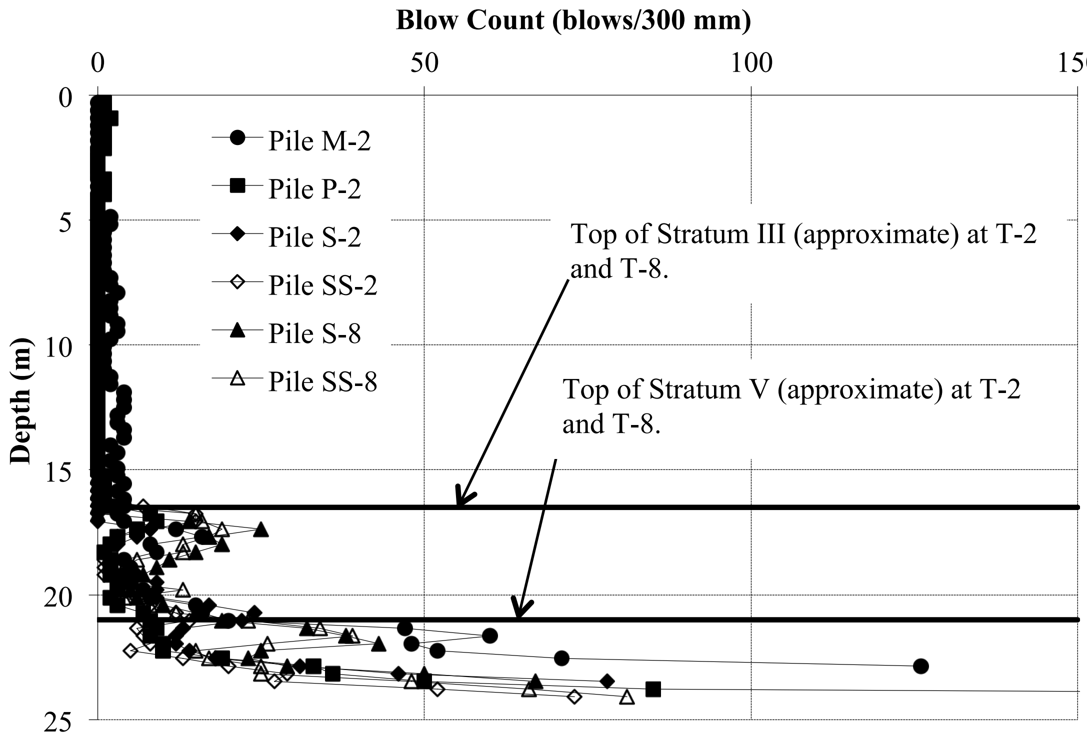

Figure 4. Probe pile driving records at clusters T-2 and T-8.

Table 1. Mobilized capacities measured during dynamic monitoring.

Pile no.	Mobilized capacity (kN) during		Ratio R/I
	Initial driving (I)	Restrike (R)	
M-2	764	947	1.24
M-8	933	822	0.88
P-2	1,780	1,780	1.00
P-8	1,710	1,910	1.12

The results shown in Table 1 suggest the following:

- Some setup effect appears to have taken place for piles M-2 and P-8, despite the short time between original and restrike driving. Pile setup (discussed later in this manuscript) is consistent with anecdotal evidence gathered at the site as production piles were installed.
- The ratio for pile M-8 may reflect measurement variability rather than a decrease in capacity because the restrike apparently took place the same day that the pile was driven.

Test Piles

Step-tapered test piles were inspected and filled with 34 MPa concrete specially formulated for these piles. Pipe and Monotube piles were inspected and filled with regular 34 MPa concrete. All Monotube and step-tapered piles were reinforced to a depth of about 6 m, and all pipe piles were reinforced to a depth of about 2.4 m. Piles selected for tension load testing (not discussed herein) were also provided with two 25-mm diameter high strength steel rods spanning the entire length of the piles.

Telltales consisting of a steel rod free to move vertically inside an oil-filled 25-mm diameter PVC pipe were installed at predetermined depths (discussed in the next section). The PVC pipe was placed inside the pile prior to concreting. Vibrating wire strain gauges were attached to the high-strength steel rods prior to concreting the piles.

STATIC PILE LOAD TEST PROGRAM

General

Since all probe piles drove consistently throughout the site, probe piles in clusters T-2 and T-8 were selected for pile load tests because they are located within the footprint of the proposed turbine building and the stack as shown in Figure 1. In addition, these are the two critical structures from a structural loading and differential settlement standpoint. All piles in cluster T-2, i.e., piles M-2, P-2 and S-2, were load tested. After the tests at cluster T-2 were completed, the step-tapered pile at group T-8, i.e., S-8, was selected for testing. Piles SS-2 and SS-8 were statically load tested to verify their suitability as a more economical alternative to the S-size step-tapered piles. Thus, a total of six piles were statically load tested in compression: M-2, P-2, S-2, SS-2, S-8, and SS-8. As previously stated, tension and lateral load tests were also performed, but those results are not included here because of space limitation.

Testing Equipment and Instrumentation

Compression loads were applied by a calibrated jack, and a calibrated load cell was used to measure the loads applied by the jack. Vertical pile head movements were measured by four dial gauges with 50-mm stem travel and 0.025-mm precision. Vertical pile head movements were also monitored by a scale, wire, and mirror system. The scale had a precision of 0.8 mm. Independently supported reference beams were used during measurement of vertical pile head movements.

All test piles were instrumented with two telltales and three (two in pile S-8) vibrating wire strain gauges.

One telltale was installed near the tip (about 24-m depth) and the second one near the mid-length (about 12-m depth) of the piles. Telltale movements were monitored by individual dial gauges. The telltales measured vertical displacements at the tip and mid-length of the piles under compression loads. These measurements provided qualitative data that are indicative of the load transfer mechanism along the length of the piles.

Strain gauges were located near the tip (about 24-m depth), the mid-length (about 12-m depth) of the pile, and halfway between these two depths (about 18-m depth), except for pile S-8, which had no strain gauge at 18-m depth. Strain gauges were used to measure concrete strains (and subsequent loads) at their locations under the test compression loads.

Test Procedure

Compression load tests were conducted in general accordance with ASTM D1143 Quick Method (ASTM 2007). Piles were loaded in increments of 5% of the maximum test load (twice the design allowable load) and then unloaded in decrements of 25% of the maximum test load to zero load. Piles P-2 and SS-2 were loaded beyond 1,800 kN to investigate the possibility of increasing the allowable load from 900 kN, but this was eventually abandoned because the design revision could not be supported by the project schedule. Pile S-2 was loaded and unloaded in two load cycles. All loads applied to the test piles were maintained for a period of 15 minutes. Table 2 provides a summary of maximum loads applied to the test piles.

Table 2. Maximum loads applied during each cycle of load testing.

Pile	Maximum cycle 1 load (kN)	Maximum cycle 2 load (kN)
M-2	1,800	N/A (only 1 load cycle)
P-2	2,700	N/A (only 1 load cycle)
S-2	1,800	2,670
SS-2	2,100	N/A (only 1 load cycle)
S-8	1,800	N/A (only 1 load cycle)
SS-8	1,800	N/A (only 1 load cycle)

Vertical pile head deflections were measured by four dial gauges positioned vertically on four mutually orthogonal horizontal brackets welded to the sides of the piles. Pile head deflections referred to in this manuscript represent an average of these four measurements.

Test Results

Because of the tight project schedule, load testing of piles took place as soon as the concrete developed sufficient strength but generally between about 13 and 40 days from installation. Compression load test results for the six tested piles are shown in Figure 5. Interpretation of the results in Figure 5 indicates the following:

- Pipe pile P-2 disclosed a stiffer behavior than the Monotube and step-tapered piles, i.e., the pile head settlements for pile P-2, was smaller (about one-half) than for any other pile under the same applied test load. This indicated that the pipe pile could develop a much higher allowable load than required by the pile design. Also, considering that the difference in pile diameters between the pipe pile and the Monotube and step-tapered piles is small, this result suggested that there is no clear benefit from the tapered sections at this site.
- All of the remaining tested piles (M-2, S-2, S-8, SS-2, and SS-8) disclosed similar behavior under load.
- All piles met the design requirement of 900-kN allowable load in accordance with BOCA (1990).

Further discussions in the section on interpretation of pile instrumentation results suggest that the 900-kN allowable load could have been increased. However, as previously stated, attempts at pile design optimization were eventually abandoned. The time required for pile design optimization would impact the overall project schedule in such a way that the cost savings from the optimization was not justified. Thus, production pile selection was based primarily on least cost among the tested piles and installation schedule considerations. This favored the use of the smaller step-tapered pile (SS size) as production piles. Several hundred step-tapered piles were successfully installed onsite for the different structures (Figure 1).

The pile driving records, pile dynamic monitoring, and static pile load tests results were used to develop production pile driving criteria. It was recommended that production piles be driven into the pile-bearing Stratum IV to a minimum depth of 24 m, with 8 blows/25 mm for 300 mm with a Raymond 80C or other equivalent hammer of rated energy equal to about 33kN m. If 6 blows/25 mm was achieved continuously over 600 mm, pile driving could be terminated without meeting the minimum depth requirement of 24 m.

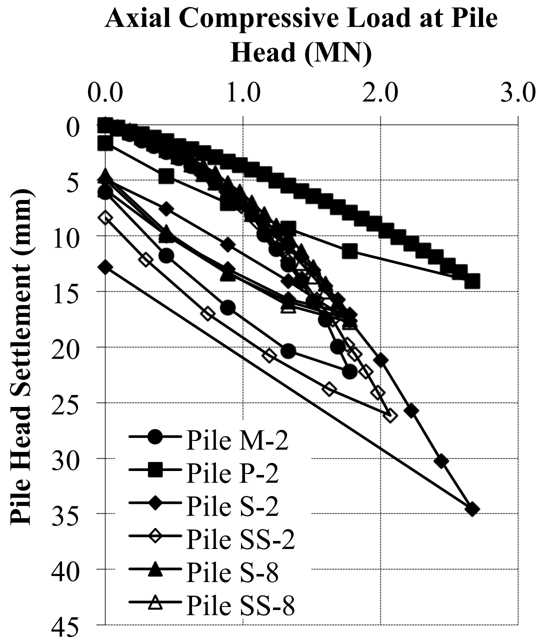

Figure 5. Compression load test results.

INTERPRETATION OF PILE INSTRUMENTATION RESULTS

Approach

Figures 6 through 8 show telltale measurements for piles P-2, S-2, and SS-2. These three piles were loaded to the highest loads(Table 1 and Figure 5) and thus were loaded closer to failure than the other test piles. Figure 7 shows that the bottom telltale was damaged during testing of pile S-2 (further discussion of instrumentation issues is presented later). However, the bottom telltale movements shown in Figures 6 and 8 suggest that piles P-2 and SS-2 were not sufficiently loaded to characterize a pile failure load. For instance, Vesic (1977) postulated that pile shaft resistance is fully mobilized at small pile displacements, but full end-bearing mobilization, i.e., failure, requires displacements of the pile tip on the order of 10% of the pile diameter. Even if pile tip diameters are used, 10% of the pile diameters ranged from 22 mm (pile SS-2) to 36 mm (pile P-2). The maximum tip movement was less than 2 mm for pile P-2, and less than 4 mm for pile SS-2. Thus, no conclusions can be reached with regard to the pile tip resistance. However, the telltale and strain gauge results provide useful data with regard to average unit shaft resistance, f_s.

It is also interesting to note that the pile head settlements are consistently lower than the PL/AE-line elastic compression (assuming zero shaft resistance) for piles P-2 and SS-2. This behavior suggests a significant frictional contribution to pile capacity where shaft resistance reduces the compressive forces in the pile and limits the settlement (Bradshaw & Baxter 2006).

This discussion further supports the argument that higher allowable loads could have been used for the production piles if enough time was available for pile optimization and design revision. Thus, a conservative pile design was adopted that was still cost effective within the project schedule constraints.

Interpretation of telltale measurements was conducted in accordance with the procedure outlined in Fellenius (1980). The unit shaft resistance provided by the predominantly loose/soft soils comprising the dredged fill and Strata I and II soils was considered essentially constant with depth. This assumption was extended to the unit shaft resistance along the full length of the test piles.

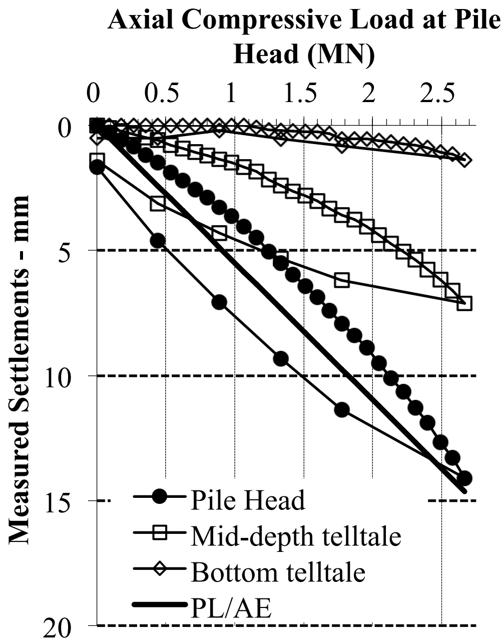

Figure 6. Telltale data for pile P-2.

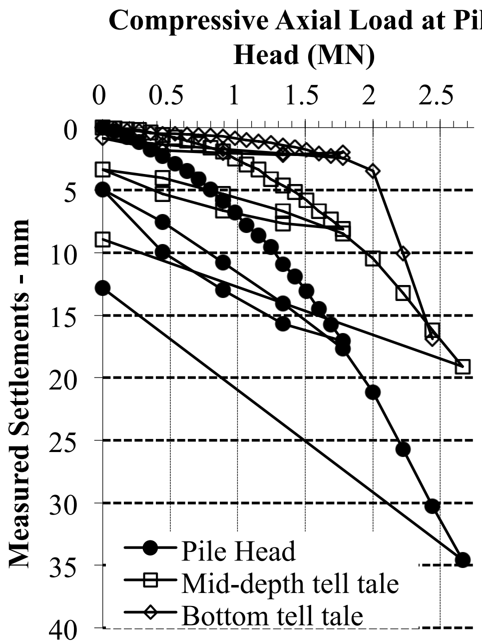

Figure 7. Telltale data for pile S-2.

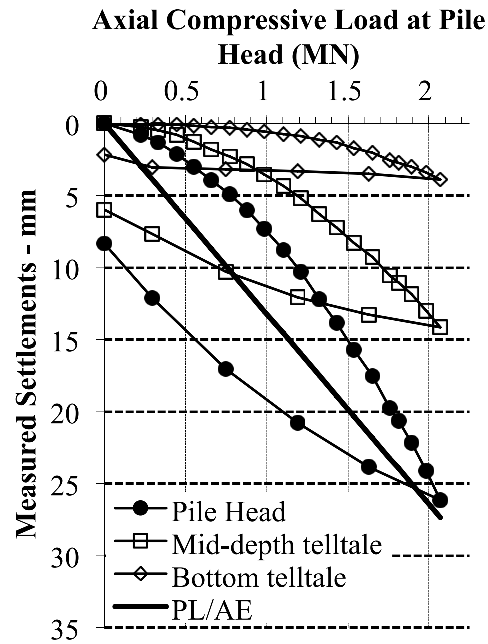

Figure 8. Telltale data for pile SS-2.

If the assumption that unit shaft resistance is constant with depth is made, then

$$P_{avg} = A \cdot E \cdot \Delta L / L \quad (1)$$

where: P_{avg} = average compressive axial load between pile head and telltale level or between telltale levels (kN)
A = cross-sectional area of the pile (m^2)
E = modulus of elasticity of concrete (34,000 kPa)
L = distance between the pile head and telltale level or between telltale levels (m)
ΔL = difference between displacement measured at the pile head and telltale level or between telltale levels (m)

$$P_{tip} = 2 \cdot P_{avg} - P_{head} \quad (2)$$

where: P_{tip} = compressive axial load at the pile tip or at telltale level (kN)
P_{head} = compressive axial load at the pile head (kN)

and

$$P_{skin} = P_{head} - P_{tip} \quad (3)$$

where: P_{skin} = load resisted by shaft resistance between the head pile head and telltale level or between telltale levels (kN)

The average unit shaft resistance, f_s in kPa, is by definition

$$f_s = P_{skin}/A_s \quad (4)$$

where: A_s = pile shaft area between the head pile head and telltale level or between telltale levels (m^2)

Strain gauge measurements were considered representative of the axial strain in the concrete, and the axial compressive load at the strain gauge level was calculated directly from

$$P_{tip} = \varepsilon \cdot E \cdot A \quad (5)$$

where: P_{tip} = compressive axial load at the strain gauge level (kN)
ε = axial strain measured at the strain gauge level (m/m)

Equations (3) and (4) were used to calculate the average unit shaft resistance using the same assumptions that were used for interpretation of telltale measurements.

Results

During the process of interpreting the instrumentation results, it became apparent that a significant amount of data could be used only qualitatively. Application of the data interpretation previously described often resulted in meaningless results. For instance, interpretation of P_{tip} at the mid-depth telltale and at the actual pile tip (bottom telltale) of pile SS-2 is shown in Figure 9. It is clear that that P_{tip} cannot exceed P_{head}, and this is observed for mid-depth telltale for P_{head} larger than about 1,200 kN. Also, the load for the bottom telltale exceeds P_{head} for P_{head} larger than about 2,000 kN. In the cases where $P_{tip} > P_{head}$, values of $f_s < 0$ are obtained. This could suggest negative friction effects, but it is unlikely that this could take place because no site grade raising was performed in conjunction with these tests, and the dredged fill layer was relatively thin and had been in place for more than 30 years. Thus, results such as those shown in Figure 9 were disregarded.

Figure 10 illustrates the application of the data interpretation previously described to pile P-2. In this case P_{tip} at the mid-depth telltale and at the actual pile tip (bottom telltale) are both consistently lower than P_{head}.

Similar interpretation issues were observed with the strain gauge data.

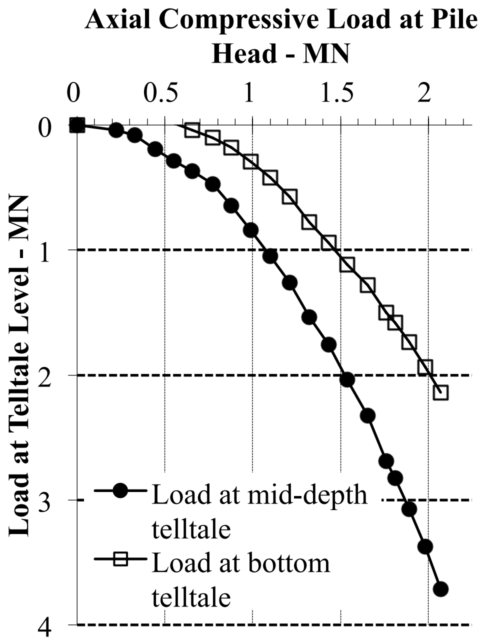

Figure 9. Interpretation of axial compressive loads at the telltale levels for pile SS-2.

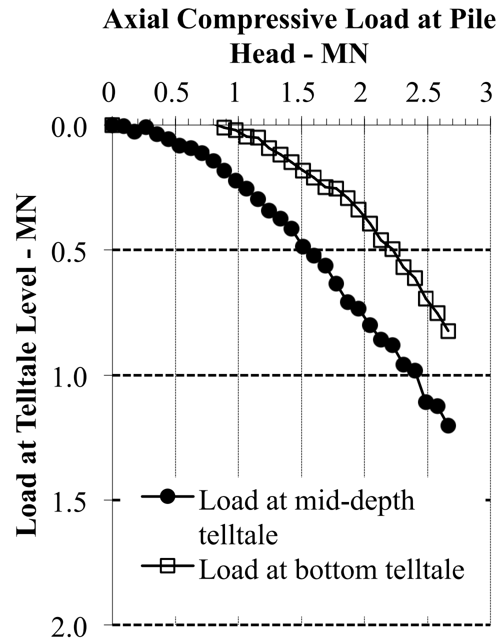

Figure 10. Interpretation of axial compressive loads at the telltale levels for pile P-2.

Figure 11 shows calculated f_s values using instrumentation results considered acceptable. The following can be stated with respect to the results in Figure 11:

- The results shown in Figure 11 include shaft resistance mostly along Stratum II. Thus, direct comparison with the field vane shear test results presented in Figure 3, and other shear strength results discussed previously, is appropriate.
- The f_s curve for pile SS-2 shown in Figure 11 shows a clear peak value of about 44 kPa, which correlates well with the undrained strength of Stratum II (silt) measured by field vane shear and CPT testing (s_u = 50 kPa). This value is also slightly higher than the adopted design value of s_u = 40 kPa. The peak f_s value of about 44 kPa does not seem to reflect any setup effects, if this value is compared to s_u = 50 kPa.
- The f_s curve for pile SS-8 shown in Figure 11 shows a maximum value of about 48 kPa, and it appears to be approaching a peak value. This maximum value also correlates well with the undrained shear strength of Stratum II (silt) measured by field vane shear and CPT testing (s_u = 50 kPa). This value is also slightly higher than the adopted design value of s_u = 40 kPa. The maximum f_s value of about 48 kPa does not seem to reflect any setup effects if this value is compared to s_u = 50 kPa.

- The f_s curve for pile P-2 shown in Figure 11 shows a maximum value of about 107 kPa, and it appears to be approaching a peak value. This maximum value is slightly more than twice the average undrained shear strength of Stratum II (silt) measured by field vane shear and CPT testing. There is anecdotal evidence and also limited dynamic testing evidence performed at the site (previously discussed in this manuscript) of soil freezing (Komurka et al. 2003) at the site. While it cannot be conclusively stated that this large value of average unit shaft resistance is related to soil freezing, Randolph et al. (1979) state that after dissipation of pore pressures caused by pile driving, the undrained shear strength of soil close to the pile can increase by a factor of 1.6 for Boston Blue clay and by a factor ranging from 1.3 to 2 for other soils.
- No benefit from step-tapering in the f_s values for fine-grained soils of Stratum II was expected, and none is evident when the f_s values for piles SS-2 and SS-8 (44 kPa and > 48 kPa) are compared with the undrained shear strength of Stratum II (silt) measured by field vane shear and CPT testing ($s_u = 50$ kPa).

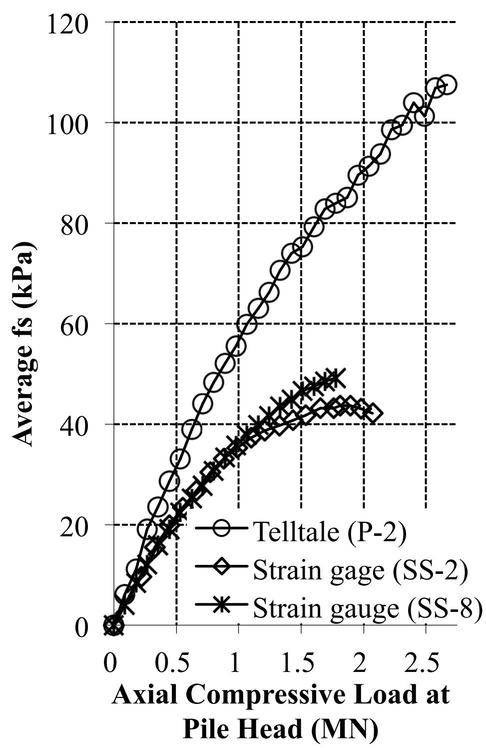

Figure 11. Average f_s for the upper half (about 12 m) of piles.

CONCLUSIONS

An extensive preproduction probe pile load testing program was conducted for a coal-fired power plant in New Jersey. The purpose of probe piling and testing was to minimize pile performance uncertainty resulting from variable subsurface conditions, and allow the selection of the most cost effective and technically suitable of three proposed piling systems for the project. Three types of piles were instrumented and tested: Monotube, closed-end pipe, and step tapered. Despite variable site subsurface and geologic conditions, consistent driving conditions were observed for all installed probe piles, which were all driven to a sand bearing stratum. Dynamic monitoring with a PDA during pipe and Monotube pile driving disclosed adequate behavior of the pipe piles but insufficient mobilized capacity for the Monotube piles. Dynamic monitoring also disclosed possible setup of two piles out of four restruck piles. Compressive static pile load test results indicated that all tested piles (including the Monotube pile) had sufficient capacity to meet the design load criteria, which favored the use of the smaller diameter (SS) step-tapered piles. The compressive static pile load test results also showed a stiffer behavior for the pipe pile than for the Monotube or step-tapered piles, suggesting that there is no clear benefit from the tapered sections at this site. Interpretation of pile instrumentation results indicated larger unit shaft resistance (f_s values) in the fine-grained soils of Stratum II for the straight shaft pipe pile than for two of the step-tapered piles. Also, based on interpretation of instrumentation data, no setup effect in the fine-grained soils of Stratum II could be detected for the step-tapered piles, but possible setup effects could be attributed to the straight shaft pipe pile. Allowable loads higher than the value initially considered in the design could have been used for the production piles. However, pile design optimization and revision would impact the overall project schedule in such a way that the cost savings from the optimization was not justified.

This case history illustrates the benefits of a robust pre-production probe pile and load test program, even for modest variations in site conditions. Generally, the key factor in most projects such as this is schedule control. Having a program that validates design and demonstrates consistent installation conditions across the site reduces the uncertainty in the installation schedule, and is highly recommended.

Comparison of pile behavior based on the results of the load test program, along with cost and schedule considerations, served as the basis for the selection of the small diameter (SS) step-tapered piles as production piles for the facility. Thus, a conservative pile design was adopted that was still cost effective and within the project schedule constraints. Several hundred piles were installed, and the power plant has been successfully in operation since the end of construction and commissioning.

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