Geotechnical Report

Helical Pile Test Program
APE Yard
Kent, Washington

Prepared for
American Piledriving Equipment, Inc.

June 10, 2013

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Contents

Acronyms and Abbreviations

APE	American Piledriving Equipment, Inc.
ASTM	American Society for Testing and Materials
bgs	below ground surface
ksi	kips per square inch
NAD83/91	North American Datum 1983/1991
NAVD88	North American Vertical Datum 1988
OD	outside diameter
pcf	pounds per cubic foot
pci	pounds per cubic inch
psi	pounds per square inch
rpm	revolutions per minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
yd^3	cubic yards

Geotechnical Report for the APE Yard Helical Pile Test Program

1. Introduction

This report provides subsurface information and the results of load tests on test piles installed at the American Piledriving Equipment, Inc. (APE) manufacturing facility located at 7032 South 196th Street in Kent, Washington. CH2M HILL has been contracted by APE to provide observation, analysis, and reporting of load tests on helical piles. CH2M HILL's work included the following:

1. Logging a geotechnical boring at the test site
2. Developing preliminary engineering properties for the site based on the boring and laboratory characterization
3. Observing and documenting installation of helical test piles
4. Observing and documenting static compression and lateral load tests
5. Observing dynamic monitoring
6. Documenting the results of the site conditions and load tests in this preliminary report.

CH2M HILL is acting as an independent reviewer and has no interest in the outcome of the technology testing or development. Review of the structural capacity of the fabricated piling materials was excluded from the scope of this work.

2. Limitations

This report has been prepared for the exclusive use of APE for specific application to the helical pile test program conducted between October 2012 and May 2013 at their Kent manufacturing facility. This report has been prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from a single boring, which indicated conditions only at a specific location and time and only to the depths penetrated. The boring does not necessarily reflect strata variations that might exist beyond its immediate vicinity. Using the boring interpretation for other locations on the site or uses might not be appropriate. In addition, installation of the test piles and reaction piles for the test frames has altered the subsurface so that similar conditions might not be relied upon for future tests within the same test area.

The interpretations and recommendations contained in this report are only for the purpose of documenting helical pile capacity and are not intended for design of structures or other facilities at the site. CH2M HILL is not responsible for any claims, damages, or liability associated with interpretation of subsurface data or reuse of the subsurface data or engineering analyses without the express written authorization of CH2M HILL.

3. Subsurface Conditions

Figure 1 shows the site location. The geology of the area, subsurface exploration, and observed subsurface conditions are described in the subsections below.

3.1 Site Geology

The site is located in the Green River Valley, which was an inlet of the Puget Sound at the conclusion of the most recent glaciation about 12,500 years ago. The marine embayment was gradually displaced by the deposition of alluvial soils by the ancestral and modern Green and White Rivers. The valley floor in Kent is underlain by about 400 feet of alluvial soils. The surface is capped by overbank deposits of the modern Green and White Rivers, consisting of fine sandy silt. Loose, reasonably well-sorted, alluvial sand and silt deposited by the ancestral Green and White Rivers underlie the more recent overbank deposits, which are in turn underlain by a marine sandy silt. The Osceola Mudflow, which originated from Mount Rainier, flowed into the drainage about 5,700 years ago and may interrupt the late Holocene valley fill deposits in some locations. The mudflow deposit is characterized by poor sorting of gravel, sand, silt, and clay with occasional boulders—typically massive texture, random inclusions of wood and organics, and low density—and dominance of angular and volcanic clasts. Large-scale subsurface maps developed from widely spaced well logs project the top surface of the mudflow between elevation -40 and -140 feet in the vicinity of the APE site and suggest that it could be 0 to 40 feet thick (Dragovich, et al., 1994).

3.2 Subsurface Exploration

A 141.5-foot-deep geotechnical boring was drilled in the test pile location in mid-October 2012. The boring was drilled with open-hole mud rotary techniques, and Standard Penetration Test (SPT) samples were driven at 5-foot intervals in general accordance with American Society of Testing and Materials (ASTM) D 1586, except that sample liners were not used. The Brainard Kilman BK81 drill rig was equipped with an automatic trip hammer, which was recently dynamically tested and found to have an average energy transfer ratio (measured energy divided by potential energy) of 54 percent (GeoDesign, 2013). Samples were visually classified and logged in accordance with ASTM D2488. Representative samples were tested in the laboratory for grain size and Atterberg limits. A standpipe piezometer was screened between 20 and 30 feet below the ground surface (bgs). The boring log (boring APE-B-1) and laboratory test results are contained in Appendix A. The depth to groundwater in the piezometer in boring APE-B-1 was measured on October 15 and November 30, 2012; the depth below the original ground surface varied from 7.9 to 8.0 feet.

3.3 Subsurface Conditions

The boring indicates the general stratigraphy listed below at the time of drilling in October 2012. ; descriptions of the materials generally adhere to the ASTM D2488 naming conventions:

- **0 to 3 feet bgs:** Poorly graded gravel with sand and silt (GP-GM), dense site fill consisting of imported angular rock, typically 1.5-inch minus
- **3 to 7 feet bgs**: Silty sand with gravel (SM), medium dense, moist, angular gravel well invaded by primarily fine- and medium-grained sand and silt subgrade
- **7 to 45 feet bgs:** Silty sand (SM) interbedded with silt with sand (ML) and poorly graded sand with silt (SP-SM), loose or soft to firm, wet, primarily fine-grained sand, low plasticity silt with water content near the liquid limit, traces of wood and organics
- **45 to 57 feet bgs:** Poorly graded sand with silt (SP-SM), medium dense, wet, medium- to fine-grained sand, low to nonplastic silt, bedded with slight variation in silt and sand size and composition
- **57 bgs to hole bottom at 142 feet bgs:** Poorly graded sand with silt (SP-SM), as above, but loose with predominantly fine-grained sand
- An additional 12 inches of crushed 1.5-inch-minus rock was placed over the test site after the test piles were installed but before the first static axial load test in early November 2012.

The upper 45 feet of soil, excluding the surface fill, appears to be the modern alluvial overbank deposits, while the materials below 45 feet appear to be earlier Holocene alluvial deposits. The Osceola mudflow deposit does not appear to be present at the boring location or is deeper than the 142-foot boring depth.

4. Interpreted Geotechnical Soil Design Properties

Visual observations of the material, laboratory gradation and plasticity, and SPT blow counts corrected for overburden pressure and hammer energy, $(N_1)_{60}$, or energy only, N_{60}, as appropriate, were used with published correlations and local experience to develop the pile design parameters listed in Table 1. Although SPT hammer energy was not measured directly as part of this test, an energy transfer of 54 percent was measured within 2 months of the sampling date for this drill rig working at another site (GeoDesign, 2013). Some correlations are based on the (N1)60 value (e.g., friction angle) while others are based on the N60 value (e.g., the β value below). In the absence of load test data, the pile design parameters in Table 1 would have been developed from material type and density considerations. All layers are judged to be sufficiently granular or with fines of low enough plasticity to be considered as drained sand for pile design.

The pile design parameters are for use as follows:

Ultimate or Unfactored Pile Shaft Resistance: Rs = Sum(A_{skin} * f)

where,

A_{skin}	=	effective pile diameter * length
f	=	$\beta * \sigma'_v$
σ'_v	=	effective overburden stress at midlength of segment (psf)
β	=	1.5 - 0.135(z)^0.5 for N_{60} >= 15
β	=	(N_{60}/15)*(1.5 - 0.135(z)^0.5) for N_{60}<15
N_{60}	=	SPT N-value corrected normalized to 60-percent hammer efficiency
z	=	depth below ground, at soil layer mid-depth (feet)

Ultimate or Unfactored Pile Toe Resistance: $R_t = A_{toe} * N_t * \sigma'_v$

where,

A_{toe}	=	effective end area of pile toe
N_t	=	toe-bearing capacity coefficient
σ'_v	=	effective vertical overburden stress at toe.

5. Test Pile Program

Six exploratory piles were installed with varying number of helixes and with and without grouting. Figure 2 shows the pile layouts. All piles were 7-inch outside diameter (OD), 0.453-inch wall, API 5CT P-110 tubular steel in 20-foot lengths with threaded 5.5-inch-long couplings. Helix sections were 0.75-inch-thick T-1 100 ksi plate with a single revolution at 5.25-inch pitch. Figure 3 shows the configuration. Where multiple helixes were used, spacing was 7.0 feet along pipe centerline. The piles were installed with APE HD70 rotary hydraulic driver-mounted on Caterpillar® 336E L Hydraulic Excavator. Drill rotation was single speed at 18 to 19 revolutions per minute (rpm).

Table 2 summarizes the pertinent details of pile configuration and depth. Part of the purpose of the test-pile program was to develop methods for grouting. Some methods were less successful than others and did not result in grout at the pile tip or a suitable pile depth to make testing valuable; therefore, only piles 1 and 5 were load-tested. Appendix B includes general observations from installing all piles. It is important to note that substantial amounts of grout came to the surface at all grouted piles. The ground around piles 2 and 4 heaved or bulged vertically up to 6 inches, tapering to match the preinstallation ground elevation at a radius of 5 to 8 feet during grouting. The bulging occurred too gradually to notice in Pile 2, but it clearly occurred when the pile toe was only 2 to 3 feet bgs at Pile 4. In addition, significant amounts of grout came to the surface, creating a wide zone of cemented soil and gravel within the upper several inches of the ground surface around the grouted piles.

Better grout control was maintained during installation of Pile 5, but it was still estimated that approximately 0.5 cubic yard of grout was pooled around the pile head at the completion of installation. The excess grout at the surface surrounding Pile 5 was removed while fluid. At the completion of testing, soil around pile 5 was excavated to a depth of about 10 feet. The upper 10 feet of Pile 5 was surrounded by a relatively uniform grout column approximately 16 inches in diameter. The soil outside the grout column was not cemented.

Grout cubes from the neat cement grout were tested in the laboratory. The 28-day unconfined compressive strength of the grout ranged from 7,000 to 9,500 pounds per square inch (psi). All load tests were conducted at least one month after pile installation, so the 28-day strength is considered appropriate.

TABLE 1
Recommended Soil Design Parameters
Geotechnical Report for the APE Yard Helical Pile Test Program

Soil Unit No.	Depth Range [1] (feet)	Name	Unified Soil Classification Abbreviation	Average $(N_1)_{60}$	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Effective Friction Angle, ϕ' (degrees)	Shallow Foundation Bearing Capacity Factor, Nq [2]	Toe Bearing Capacity Coefficient for Driven Piles, Nt [3]	Bjerrum-Burland Beta Coefficient, β	Coefficient of Lateral Subgrade Reaction, k (pci)	Recommended Soil Model Type for LPILE
1	0 to 7	Site fill	GP-GM and SM	25	130	130	39 to 34	55 to 30	170 to 85	0.60 to 0.35	225 to 70	Sand
2	7 to 45	Firm sandy silt	ML, SM, and SP-SM	8	105	43	34 to 29	30 to 15	85 to 50	0.40 to 0.35	48 to 12	Saturated sand
3	45 to 57	Denser sand	SP-SM	26	124	62	39 to 34	55 to 30	170 to 85	0.55 to 0.40	125 to 48	Saturated sand
4	57 to 142	Looser sand	SP-SM	15	122	60	36 to 31	40 to 20	115 to 65	0.45 to 0.30	76 to 21	Saturated sand

[1] Depths reflect the subsurface conditions at the time of exploratory drilling and pile installation; 1 foot of crushed gravel was placed over the site between the date on which the boring was drilled and the dates of the load tests.

[2] Values from Table 10.6.3.1.2a-1 of American Association of State Highway and Transportation Officials (2010).

[3] Set as 3*Nq as recommended in the *Canadian Foundation Engineering Manual* (Canadian Geotechnical Society, 2006).

pcf pounds per cubic foot
pci pounds per cubic inch

TABLE 2
Summary of Pile Configurations
Geotechnical Report for the APE Yard Helical Pile Test Program

Pile Number	Bottom Helix Depth (2) (feet)	Number of Helixes	Grouted?	Comments
1[1]	77	1	No	
2	17	1	Yes	Ran out of grout.
3	77	4	Partial	Sanded grout. No grout taken below 38 feet.
4	57	1	Partial	Neat cement grout. No grout taken below 50 feet.
5[1]	50	1	Yes	Neat cement grout, full length.
6[1]	50	1	No	Tested laterally only as a replacement for Pile 1

[1] Used in load test program.
[2] Depth below ground surface to middle of bottom helix; bottom of pile is 1-foot deeper.

5.1 Load Testing Procedures

Three piles, Numbers 1, 6, and 5, were load-tested. Pile 1 was tested statically in axial compression on December 5, 2012. The ground around Pile 1 was disturbed by non-test-related activities before it could be tested laterally; a replacement ungrouted pile was installed 20 feet west of the Pile 5 location and tested laterally on May 18, 2013. Pile 5 was tested statically in axial compression on November 9 and 28, 2012, and laterally on November 30, 2012. Appendix C includes the details of the static testing procedures. Dynamic testing was initially performed on Pile 1 on December 8, 2012, using an APE D50-42 open-ended diesel hammer. Pile 5 was tested dynamically on February 8, 2013 with an APE D100-42 open-ended diesel hammer. The hammers have ram weights of 11 and 22 kips and were operated manually (i.e., the ram was lifted manually to the nominal drop height of 5 feet) for the tests. A report with the results of the dynamic testing, conducted by Miner Dynamic Testing of Bainbridge Island, Washington, is included as an appendix to this report.

Because this was the first time that APE had installed and tested high-capacity grouted piles, the testing was a learning experience and there were some test interruptions, including the following:

- Pulling out the reaction piles in the initial testing of the grouted pile, Number 5
- Failure of the hydraulic pump for the jack in the static axial compression test of the ungrouted pile, Number 1
- Inability to unload the jack incrementally on the test of pile Number 1
- Inability to fully mobilize toe bearing on the grouted pile without exceeding the operating range of the dynamic testing equipment during the first attempt at dynamic testing of pile Number 5. The dynamic testing equipment is designed for testing of commonly used piles made from grade 36 or 50 steel instead of grade 110 steel, hence, the strains generated during the dynamic testing exceeded the strains normally seen for lower grade steels, exceeding the range of the strain gages attached to the pile. A successful dynamic test of pile Number 5 was achieved by welding on a 7-inch OD x 1.0-inch thick extension in which the PDA sensors were placed.

5.2 Load Testing Results

Figures 4 through 6 show deformation vs. load plots from the static load tests. The Davisson failure criteria have been plotted on the axial compression test results of Figures 4 and 5.

For the grouted pile, Figure 5 shows two different lines to bound the theoretical elastic shortening of the pile—one based on the assumption that the effective grouted pile diameter was 6 inches and one based on the assumption that the effective grouted pile diameter was 15 inches. Although the methods for measurement were crude and there was wastage in the grout lines and in the grout that came to the ground surface, the volume contained in a uniform 15-inch diameter cylinder around the length of the pile, approximately matches the volume of grout that was pumped to the pile and is in close agreement with the 16 inch diameter observed by excavating the upper 10 feet. While it does bound the problem, the assumption of a uniform column of grout surrounding the pile probably does not reflect reality because of the following:

- Much of the grout appears to have come to the ground surface
- Grout may be in bulbs corresponding to the locations of section joining during installation when pumping was greatest, so is unlikely to be continuous in diameter
- Grout would tend to be larger in diameter near the head of the pile where there is less confining pressure and there is a larger hole annulus due to misalignment and wobbling during installation
- Some grout may have been lost into the formation due to hydraulic fracturing.

The dynamic testing results are provided in Appendix D.

5.3 Comparison of Test Results with Predicted Pile Capacity

5.3.1 Axial Compression

The measured capacities from the axial compression load tests on Piles 1 and 5 were compared with the capacity calculated by the simple equations for toe capacity and skin friction provided in the Interpreted Soil Design Parameters section of the *Canadian Foundation Engineering Manual* (Canadian Geotechnical Society, 2006). Figure 7 shows the calculated capacities based on varying assumptions for the skin resistance coefficient, β, and end bearing coefficient, N_t, compared with the measured capacity from the load test for Pile 1, the ungrouted pile. Figure 8 shows the same comparison for Pile 5, the grouted pile. The plots show calculated ultimate pile capacity for the low and high range of β values shown in Table 1 and for a range of N_t values for driven piles.

"Capacity" of the test piles in axial compression was defined by both the Davisson criteria and 1.0-inch of allowable settlement. For Pile 1, the capacities determined by the two different definitions are quite similar, especially compared with the capacity calculated from driven pile-capacity equations. For Pile 5, with the grouted tip and shaft, the definition of capacity has a much larger effect. Figure 8 shows the capacity as picked by two different assumptions about the effective diameter of grout outside with pile when calculating pile stiffness— one ignoring the presence of grout outside the pile (green line) and one based on the assumption that the grout pumped into the pile produces a uniform grout diameter over the entire length and it acts as an uncracked composite with the steel (orange line). The line shown which does not consider the grout annulus appears to give the most realistic pile stiffness based on Figure 8. Future tests which incorporate telltales or strain gages would help better define the true pile stiffness. Because of this uncertainty, a capacity based on the 1-inch allowable settlement performance criteria might be most appropriate.

The predicted percentage of total capacity attributed to shaft resistance for an ungrouted pile with a 7-inch diameter shaft and an effective toe diameter of 18 inches is approximately 20 percent. Determining skin friction from load vs. deformation plots is a crude approximation, but a portion of the curve for the ungrouted pile in Figure 4 lies on the left side of the elastic pile shortening line, suggesting that the apparent shaft resistance is roughly 30 kips, or approximately 18 percent of the total capacity. The shaft resistance interpreted from dynamic testing is 125 kips, or 74 percent of the total capacity.

For the grouted pile, the actual grout annulus is unknown, so the ratio of shaft resistance to end bearing cannot be accurately determined by calculation or interpretation of the static load vs. deflection plots. If the effective diameter was 7 inches, shaft resistance would comprise slightly less than 10 percent of the calculated capacity of a driven pile with 18-inch end diameter. If the grout annulus was 15 inches, shaft resistance would comprise about 15 to 17 percent of the calculated capacity of a driven pile with 18-inch end diameter.

One crude estimate of the shaft resistance can be made by locating where the theoretical elastic compression curve of the pile is tangent to the field load-compression curve from a static load test. The assumption of negligible grout annulus outside the steel pile provides an apparent shaft resistance of about 330 kips (roughly 50 percent of the capacity) using this approximation, while the assumption of a 15-inch effective grout diameter would indicate that none of the capacity is due to shaft resistance. The shaft resistance interpreted from dynamic testing is about 360 kips, or 55 percent of the total and agrees fairly well with the assumption that there is no grout annulus around the pile which contributes to the pile stiffness. However, based on the static and dynamic tests on the grouted and ungrouted piles, the grout around the pile does contribute greatly to both the shaft resistance and the end bearing of a helical pile.

TABLE 3
Shaft and Toe Resistance Determined from Dynamic Testing and Interpretation of Static Testing
Geotechnical Report for the APE Yard Helical Pile Test Program

			Predicted Capacity from Driven Pile Equations		Dynamic Test Interpretation		Static Load Test Interpretation	
Pile Number	**Installation**	**Pile Length** (feet)	**Shaft Resistance (%)**	**Toe Resistance (%)**	**Shaft Resistance (%)**	**Toe Resistance (%)**	**Shaft Resistance (%)**	**Toe Resistance (%)**
1	Ungrouted, single helix	77	20	80	74	26	18	82
5	Grouted, single helix	48	16	84	55	45	52	48

5.3.2 Lateral Capacity

Lateral load tests were performed on the ungrouted and grouted piles after the axial compression test. Rather than picking a capacity or “failure” criterion for the lateral load case, the measured pile head deformation with load has been plotted relative to the pile head deformation calculated with the inputs in Table 1 using the program LPILE (Ensoft, 2012); Figure 6 shows this comparison. Figure 6 shows that the measured load at 1 inch of pile head deformation, the common “failure” criterion, is about 3.3 times higher than the resistance calculated by LPILE for both the ungrouted pile and the grouted pile with a cracked section modulus. If an uncracked section modulus is assumed for the grouted pile, the measured load at 1 inch of pile head deformation is about 2.5 times higher than the resistance calculated using LPILE.

6. Conclusions

Additional pile testing is required in order to draw conclusions and develop recommendations for applying driven or drilled pile design parameters to the design of grouted and ungrouted helical piles without load testing. Although tests on two piles are insufficient to develop design guidelines, we can conclude that a comparison of calculated vs. measured capacity for these particular piles indicated the following:

1. The measured ultimate axial compressive capacity of the ungrouted pile, Pile 1, approximately matches the calculated ultimate capacity of a driven pile if skin friction is ignored, the low range of N_t based on SPT N_{60}-value and material type correlations is used, **and** the N_t value is divided by 3 (i.e., N_t is set equal to N_q for shallow foundations).
2. The measured ultimate axial compressive capacity of the grouted pile, Pile 5, approximately matches the calculated ultimate capacity of a driven pile if average values for skin friction and N_t in the lower third of the range based on SPT N_{60}-value and material type correlations is used.
3. The measured ultimate compressive capacity of the grouted Pile 5 showed significant improvement in capacity both in skin friction and in end bearing.

4. The lateral capacity of both the ungrouted and grouted piles was far greater than the calculated capacity using a model developed with LPILE.

The improvement in axial capacity with grouting is likely due to formation of a grout bulb near the toe of the pile as well as a grout skin on the pile itself and improvement of the soil near the pile with grout that was injected. This same mechanism would also cause an increase in lateral capacity.

7. Recommendations

We understand that future testing to develop design guidelines for grouted and ungrouted, single and multi-helix piles, is planned. Future testing, in addition to providing a sufficient number of tests to show reproducible results and in a variety of soil conditions and depths, would benefit from minimizing the number of independent variables. In other words, if multiple pile configurations are to be tested, it would be helpful if all the piles were installed to the same depth. If multiple helixes are tested, pick an installation depth where all the helixes can be within a single uniform soil layer.

Consider installation of multiple telltales and/or strain gages welded into the bottom of each of the pile segments for ungrouted piles and attached to a reinforcing bar which can be pushed through the fluid grout to the bottom of the pile for grouted piles in order to be able to isolate toe from skin resistance and estimate the effective pile stiffness. Continue to develop a portable test frame and rapidly deployable thickened pile section suitable for commonly available sensors for dynamic testing. It might also be illustrative to carefully excavate around the reachable portions of grouted piles to observe the actual configuration of the grout bulb with depth, as was done for Pile 5 in this test.

Because lateral capacity of grouted piles is judged to be the significantly impacted by operator technique and local surface conditions, in-cab instrumentation capable of monitoring grout volume and pressure with depth would be beneficial, not only for quality control on the specific project, but for developing design guidance over a range of conditions.

8. References

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Figures

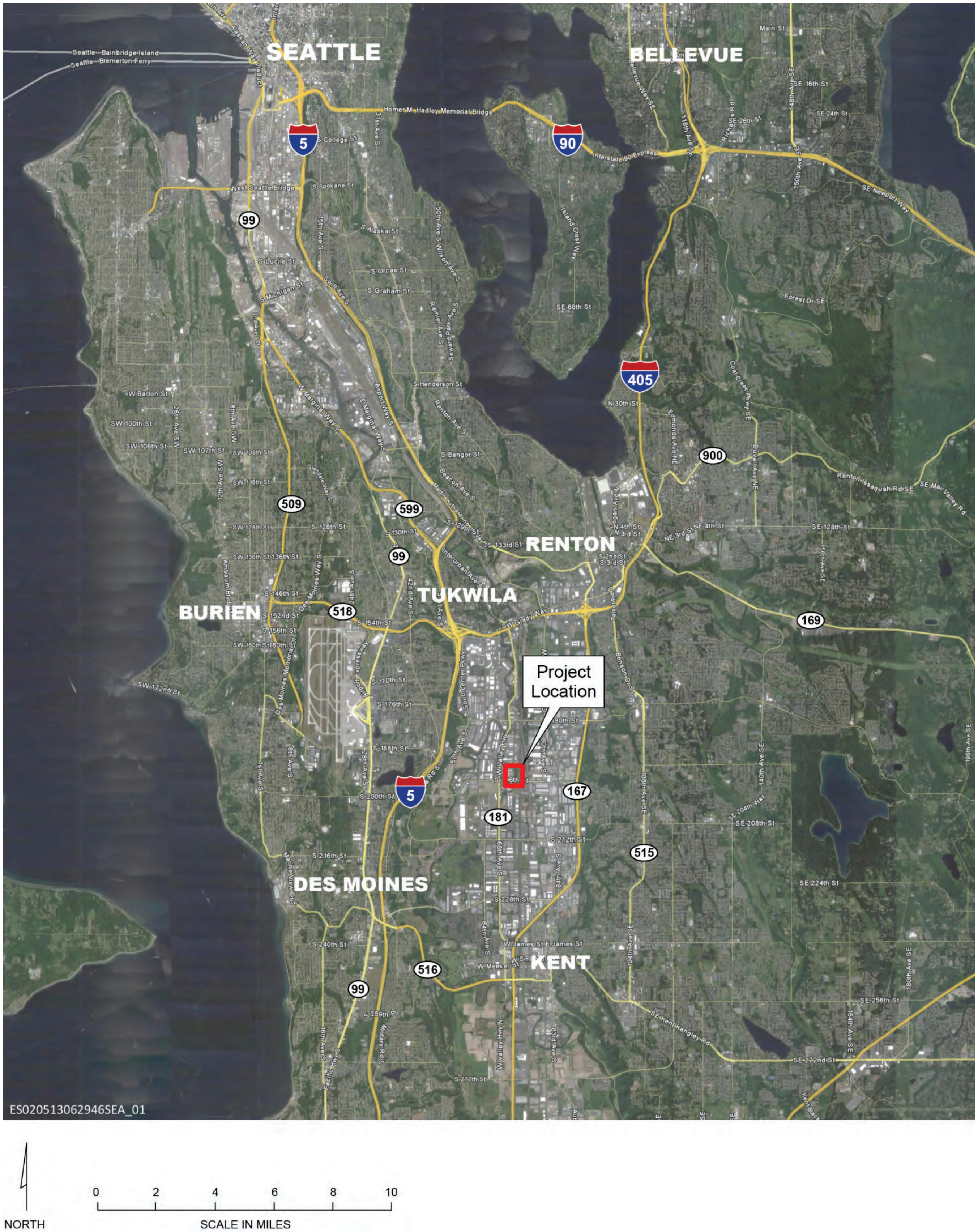


FIGURE 1
Site Location Map

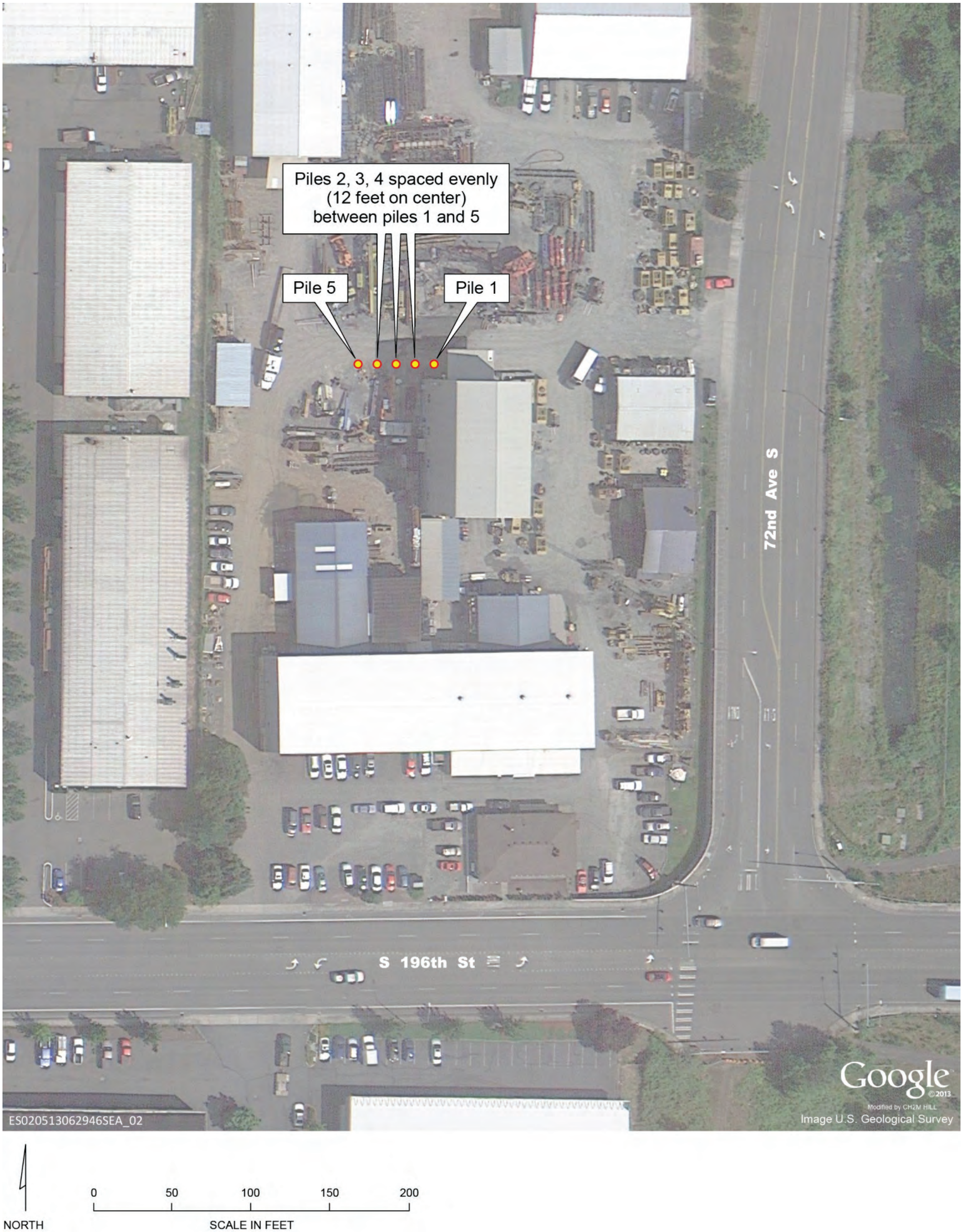
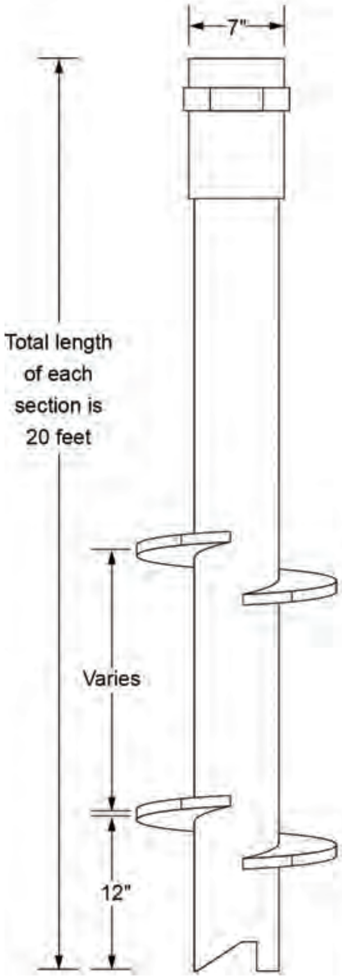


FIGURE 2
Test Site Plan



APE threaded coupler with socket-drive technology, add additional pile sections. Joint strength exceeds pile shaft in section.

Shaft Material

Nominal Diameter (inches)	Outside Diameter (inches)	Designation	Thickness (inches)	Inside Diameter (inches)	A (in^2)	Z (in^3)	I (in^4)
7	7.00	API 32#	0.453	6.094	9.31	14.33	50.17

The API yield strength designation is "P110" – 110,000 pounds per square inch (psi).

NOTE: Material callouts provided by APE.

Number and spacing of additional helixes vary (see test specifics in Appendix B).

18-inch-outside-diameter clockwise helical plate, 0.75-inch thick x 5.25-inch pitch. Material is T-1, yield strength 100,000 psi.

Open-ended for nongrouted; closed-end for grouted. Grouting is through 1 or 2 holes just below bottom helix (configuration varies, see Appendix B).

FIGURE 3
Test Pile Configuration

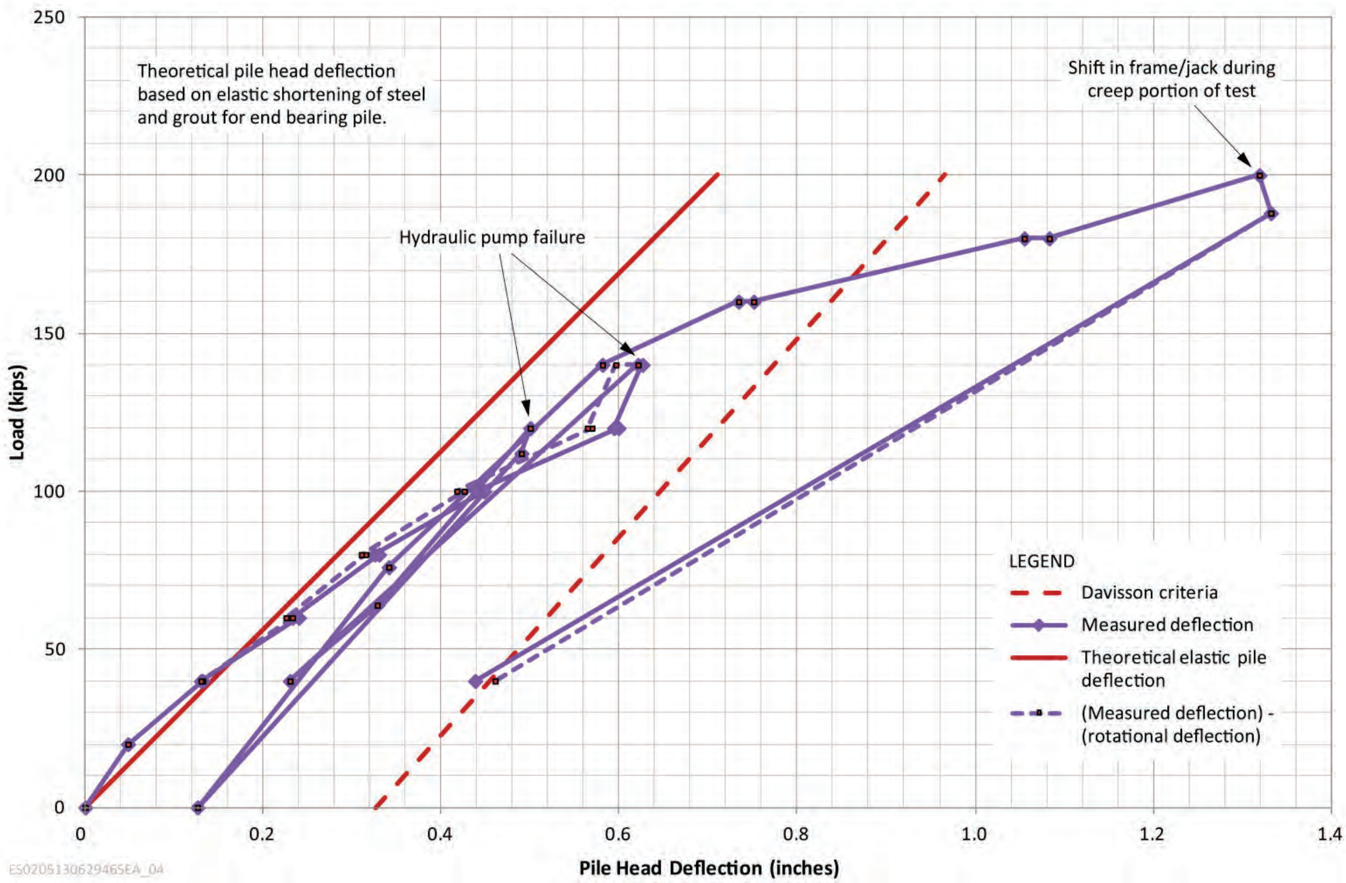


FIGURE 4
Static Axial Compression Load Test Results: Pile 1, Single Helix, Ungrouted

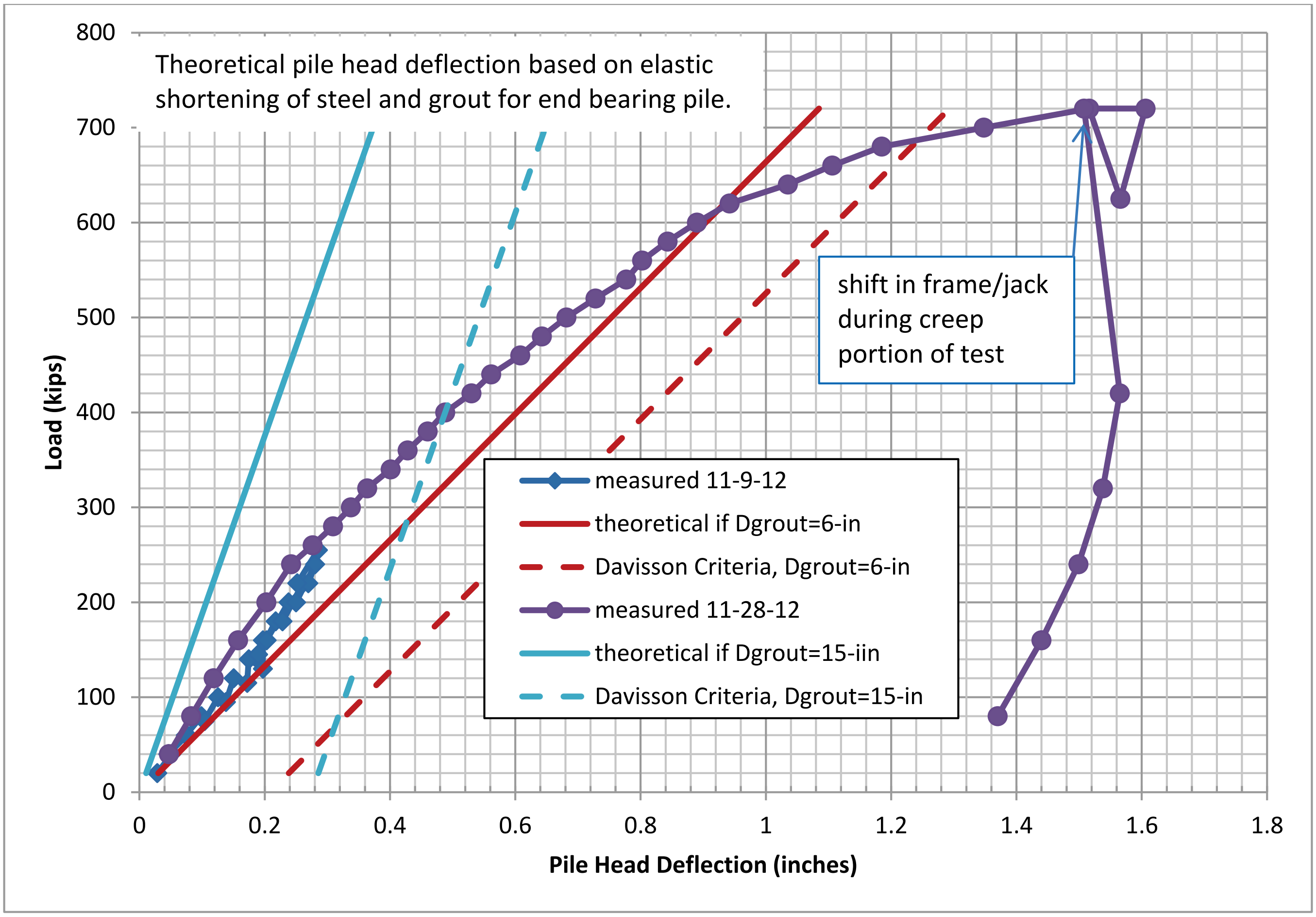


FIGURE 5

Static Axial Compression Load Test Results: Pile 5, Single Helix, Grouted

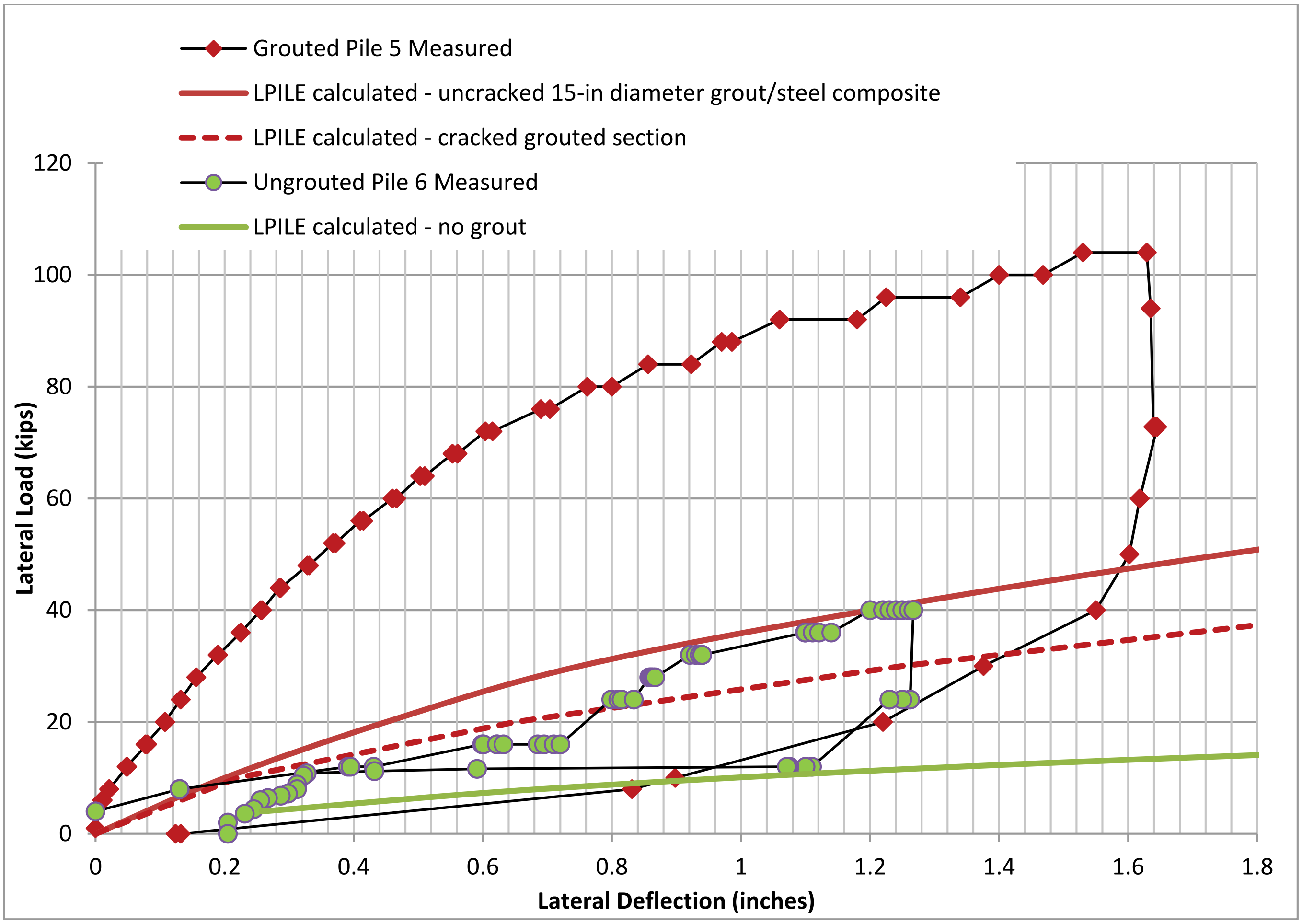


FIGURE 6

Static Lateral Load Test Results and Calculated Lateral Deflection: Pile 5, Single Helix, Grouted

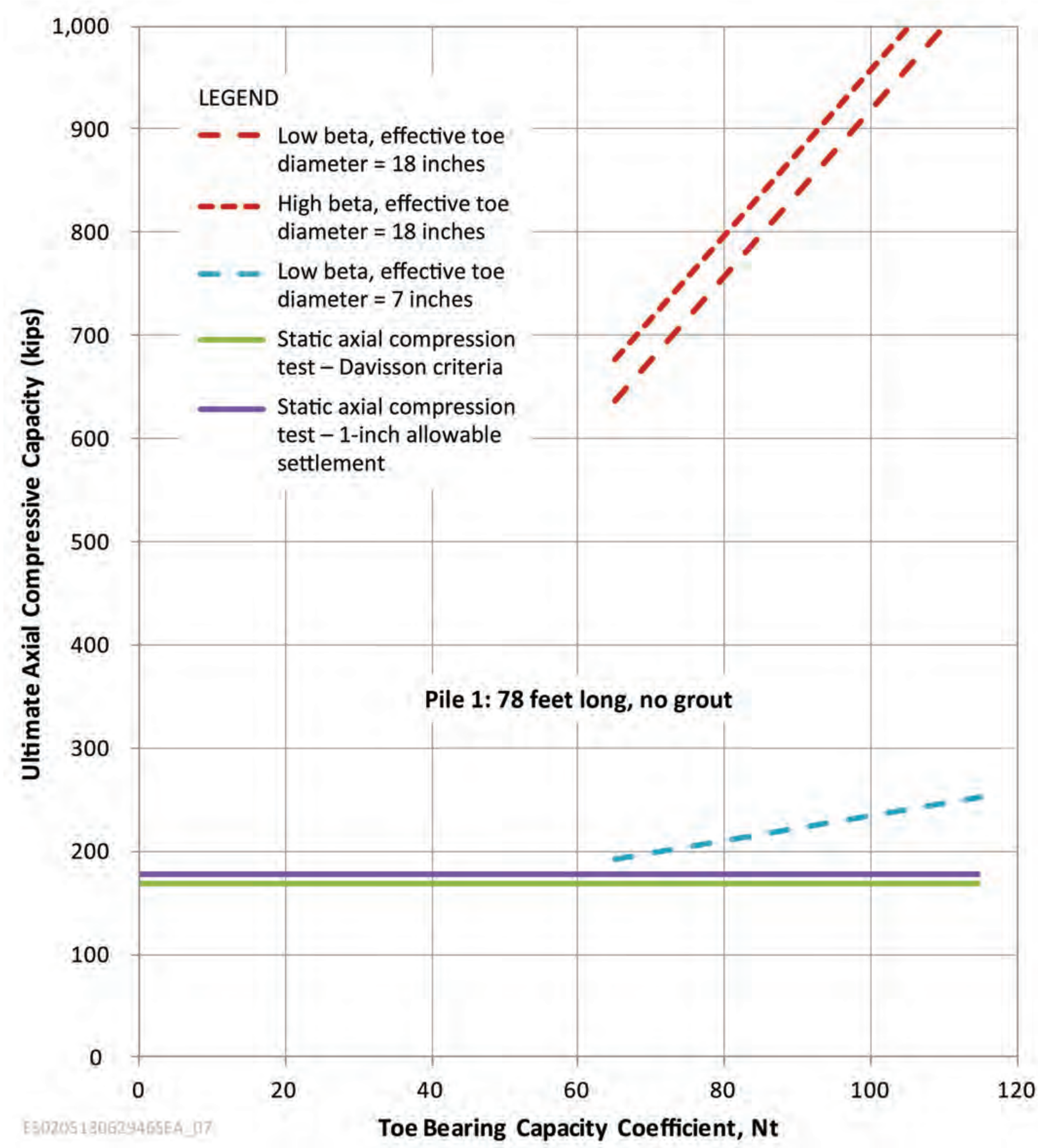


FIGURE 7

Comparison of Calculated and Measured Capacity in Axial Compression: Pile 1

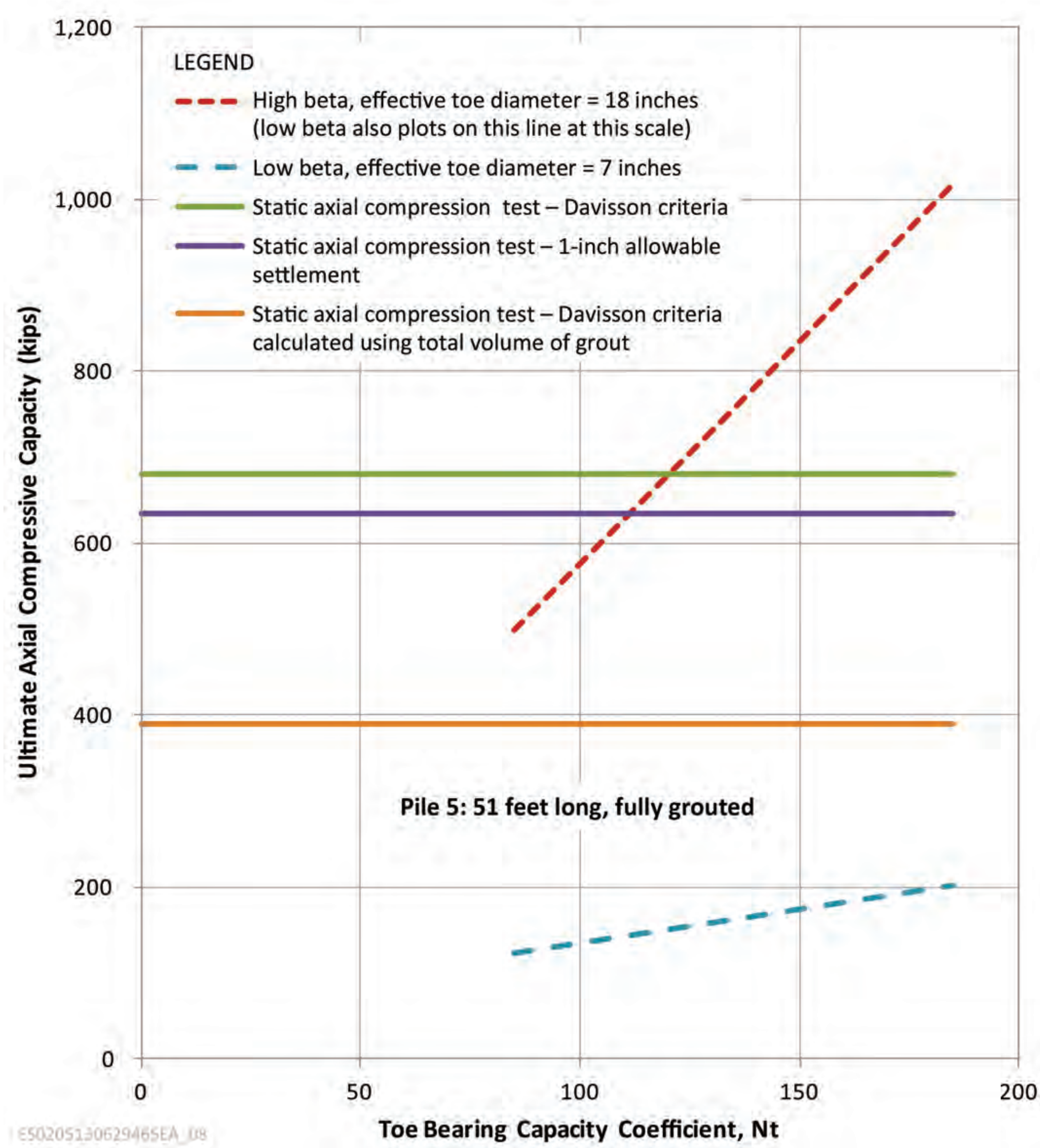


FIGURE 8

Comparison of Calculated and Measured Capacity in Axial Compression: Pile 5

Appendix A
Subsurface Data

APPENDIX A

Subsurface Data

A.1 Explanation of Boring Logs

The section below contains a summary description of the Standard Penetration Test (SPT) method used during drilling, tables defining the descriptions of relative density of coarse-grained and fine-grained soils used in soil descriptions, and information noted on the borings logs, such as the soil classification method and possible abbreviations noted on the boring logs.

A.2 Standard Penetration Test

The SPT is performed by driving a standard split-barrel sampler 18 inches into undisturbed soil at the bottom of the borehole using a 140-pound guided hammer or ram, falling freely from a height of 30 inches. This test is conducted to obtain a measure of the resistance of the soil to the sampler's penetration and to retrieve a disturbed soil sample. The number of blows required to drive the sampler for three 6-inch intervals, for a total of 18 inches, is observed and recorded on the soil boring log. The sum of the number of blows required to drive the sampler the second and third 6-inch intervals is considered the Standard Penetration Resistance or the SPT blowcount, N. If the sampler is driven less than 18 inches, but more than 1 foot, then the SPT blowcount is that for the last 1 foot of penetration. If less than a foot is penetrated, then the number of blows and the fraction of 1 foot penetrated are recorded in the boring logs.

The values of N provide a means for evaluating the relative density of granular (coarse-grained) soils and the consistency of cohesive (fine-grained) soils. Low N-values indicate soft or loose deposits, while high N-values are evidence of hard or dense materials. The criteria used for describing the relative density of coarse-grained soil and the consistency of fine-grained soils based on N-value are presented in Tables A-1 and A-2, respectively. Field classification of the soil, based on these criteria, is incorporated in the boring logs presented at the end of this appendix.

TABLE A-1
Relative Density of Coarse-Grained Soil[1]
Geotechnical Report for the APE Yard Helical Pile Test Program

N (blows per foot)	Relative Density	Field Test
0 to 4	Very loose	Easily penetrated with 0.5-inch steel rod
		Pushed by hand
5 to 10	Loose	Easily penetrated with 0.5-inch steel rod
		Pushed by hand
11 to 30	Medium dense	Easily penetrated with 0.5-inch steel rod
		Driven with 5-pound hammer
31 to 50	Dense	Penetrated 1 foot with 0.5-inch steel rod
		Driven with 5-pound hammer
50 or more	Very dense	Penetrated only a few inches with 0.5-inch
		Steel rod driven with 5-pound hammer

[1]Developed from Sowers (1979)

TABLE A-2
Consistency of Fine-Grained Soil [1]
Geotechnical Report for the APE Yard Helical Pile Test Program

N (blows per foot)	Consistency	Field Test
Fewer than 2	Very soft	Easily penetrated several inches by fist
2 to 4	Soft	Easily penetrated several inches by thumb
5 to 8	Firm	Can be penetrated several inches by thumb with moderate effort
9 to 15	Stiff	Readily indented by thumb, but penetrated only with great effort
16 to 30	Very stiff	Readily indented by thumbnail
30 or more	Hard	Indented with difficulty by thumbnail

[1]Developed from Sowers (1979)

A.3 Test Boring Logs

The boring logs are at the end of this appendix. The soil classifications on the exploration logs are per the American Society for Testing and Materials (ASTM) soil classification, based on the Unified Soil Classification System (USCS). The soil group symbols are marked with parentheses when the classification is based on visual classification alone. When soil classification symbols are separated by commas, then the classification has been confirmed with laboratory testing.

The lines on the boring logs do not define contacts between different soil classifications; the lines are used to separate the descriptions for legibility purposes only. The horizontal datum on the logs is Washington Coordinate System, North Zone North American Datum 1983/1991 (NAD83/91) and vertical datum is North American Vertical Datum 1988 (NAVD88).

Abbreviations listed on borings logs include the following:

bgs   below ground surface

PROJECT NUMBER: 461291	BORING NUMBER: APE-B-1 (unique well No. BHR 7 60)	SHEET 1 OF 5

SOIL BORING LOG

PROJECT: APE Helical Pile Testing — LOCATION: APE @ Kent, WA (LAT 47.4276, LON -122.2453)

ELEVATION: Approx. 27 ft — DRILLING CONTRACTOR: Holocene Drilling Company

DRILLING METHOD AND EQUIPMENT: Truck-mounted Brainard Kilman BK 81, 140-lb auto-hammer with 30-inch drop, Open-Hole Mud Rotary drilling

WATER LEVELS: Approx Elev 19 on 10/15/12 & — START: 10/12/2012 — END: 10/12/2012 — LOGGER: S. Shin

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: SOIL NAME, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
0					Surface Gravels (Fill) Approximately 6 ~ 12 inches.	Begin drilling @ 9:00 am
						DR: ~3' bumpy drilling (gravel)
5	5 - 6.5	9"	SS-1	26-11-8 (19)	**SILTY SAND WITH GRAVEL (SM)** dark gray, medium dense, fine to coarse sand, fine subrounded gravel, est.10~20% fines.	
10	10 - 11.5	13"	SS-2	3-4-5 (9)	**SILTY SAND (SM)** dark gray, loose, fine sand, est.10~20% fines.	
15	15 - 16.5	6"	SS-3	3-2-1 (3)	**SILTY SAND (SM or SP-SM)** dark gray, very loose, fine sand, est.5~15% fines.	
						Wood debris in cutting (17' ~ 29')
20	20 - 21.5	18"	SS-4A SS-4B	2-5-9 (14)	20'~21' **SILT WITH SAND (ML)** gray brown, low plasticity. (SS-4B) 21'~22' **SILTY SAND (SM)** dark gray, medium dense, fine sand, est 10~20% fines (SS-4A)	
25	25 - 26.5	15"	SS-6	5-5-7 (12)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, medium dense, fine to medium sand, est.5~10% fines.	
30						

PROJECT NUMBER: 461291	BORING NUMBER: APE-B-1	SHEET 2 OF 5

SOIL BORING LOG

PROJECT: APE Helical Pile Testing   LOCATION: APE @ Kent, WA (LAT 47.4276, LON -122.2453)

ELEVATION: Approx. 27 ft   DRILLING CONTRACTOR: Holocene Drilling Company

DRILLING METHOD AND EQUIPMENT: Truck-mounted Brainard Kilman BK 81, 140-lb auto-hammer with 30-inch drop, Open-Hole Mud Rotary drilling

WATER LEVELS: Approx Elev 19 on 10/15/12 &   START: 10/12/2012   END: 10/12/2012   LOGGER: S. Shin

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: SOIL NAME, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
30	31.5	14"	SS-6	14-12-11 (23)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above	
35	35 36.5	7"	SS-7	3-2-6 (8)	**POORLY GRADED SAND WITH SILT (ML)** similar to above	wood debris in shoe
40	40 41.5	18"	SS-8	2-2-3 (5)	**SILT WITH SAND (ML)** brown, low plasticity, trace of organic matter.	wood debris in shoe
45	45 46.5	16"	SS-9	15-23-25 (48)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine sand, est.5~10% fines.	
50	50 51.5	16"	SS-10	21-21-23 (44)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above.	
55	55 56.5	18"	SS-11	19-24-23 (47)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above but coaser, fine to medium sand.	
60						

PROJECT NUMBER: 461291	BORING NUMBER: APE-B-1	SHEET 3 OF 5

SOIL BORING LOG

PROJECT: APE Helical Pile Testing   LOCATION: APE @ Kent, WA (LAT 47.4276, LON -122.2453)

ELEVATION: Approx. 27 ft   DRILLING CONTRACTOR: Holocene Drilling Company

DRILLING METHOD AND EQUIPMENT: Truck-mounted Brainard Kilman BK 81, 140-lb auto-hammer with 30-inch drop, Open-Hole Mud Rotary drilling

WATER LEVELS: Approx Elev 19 on 10/15/12 &   START: 10/12/2012   END: 10/12/2012   LOGGER: S. Shin

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: SOIL NAME, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
60	61.5	13"	SS-12	23-19-14 (33)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above but finer, fine to medium sand.	
65	65 / 66.5	14"	SS-13	11-11-20 (31)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above.	
70	70 / 71.5	14"	SS-14	14-13-11 (24)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above.	
75	75 / 76.5	16"	SS-15A SS-15	14-15-21 (36)	**75'~75.5' SILT WITH SAND (ML)** grayish brown, low plasticiy, est.5~10% sand (SS-15A) **75.5'~76.5' POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine to medium sand, est.5~10% fines.	
80	80 / 81.5	15"	SS-16	14-16-24 (40)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine to medium sand, est.5~10% fines.	
85	85 / 86.5	13"	SS-17	10-12-14 (26)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above but medium dense	
90						

PROJECT NUMBER: 461291	BORING NUMBER: APE-B-1	SHEET 4 OF 5

SOIL BORING LOG

PROJECT: APE Helical Pile Testing — LOCATION: APE @ Kent, WA (LAT 47.4276, LON -122.2453)

ELEVATION: Approx. 27 ft — DRILLING CONTRACTOR: Holocene Drilling Company

DRILLING METHOD AND EQUIPMENT: Truck-mounted Brainard Kilman BK 81, 140-lb auto-hammer with 30-inch drop, Open-Hole Mud Rotary drilling

WATER LEVELS: Approx Elev 19 on 10/15/12 & — START: 10/12/2012 — END: 10/12/2012 — LOGGER: S. Shin

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION (SOIL NAME, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY)	COMMENTS (DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION)
90	91.5	16"	SS-18	22-24-26 (50)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine to medium sand, est.5~10% fines.	
95	95 96.5	15"	SS-19	11-11-18 (29)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, medium dense, fine sand, est.5~10% fines.	
100	100 101.5	13"	SS-20	16-16-19 (35)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine to medium sand, est.5~10% fines.	
105	105 106.5	15"	SS-21	10-18-19 (37)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine sand, est.5~10% fines.	
110	110 111.5	14"	SS-22	10-11-14 (25)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, medium dense, fine sand, est.5~10% fines.	
115	115 116.5	9"	SS-23	13-15-13 (28)	**POORLY GRADED SAND WITH SILT (SP-SM)** simlar to above.	
120						

PROJECT NUMBER: 461291	BORING NUMBER: APE-B-1	SHEET 5 OF 5

SOIL BORING LOG

PROJECT: APE Helical Pile Testing
LOCATION: APE @ Kent, WA (LAT 47.4276, LON -122.2453)

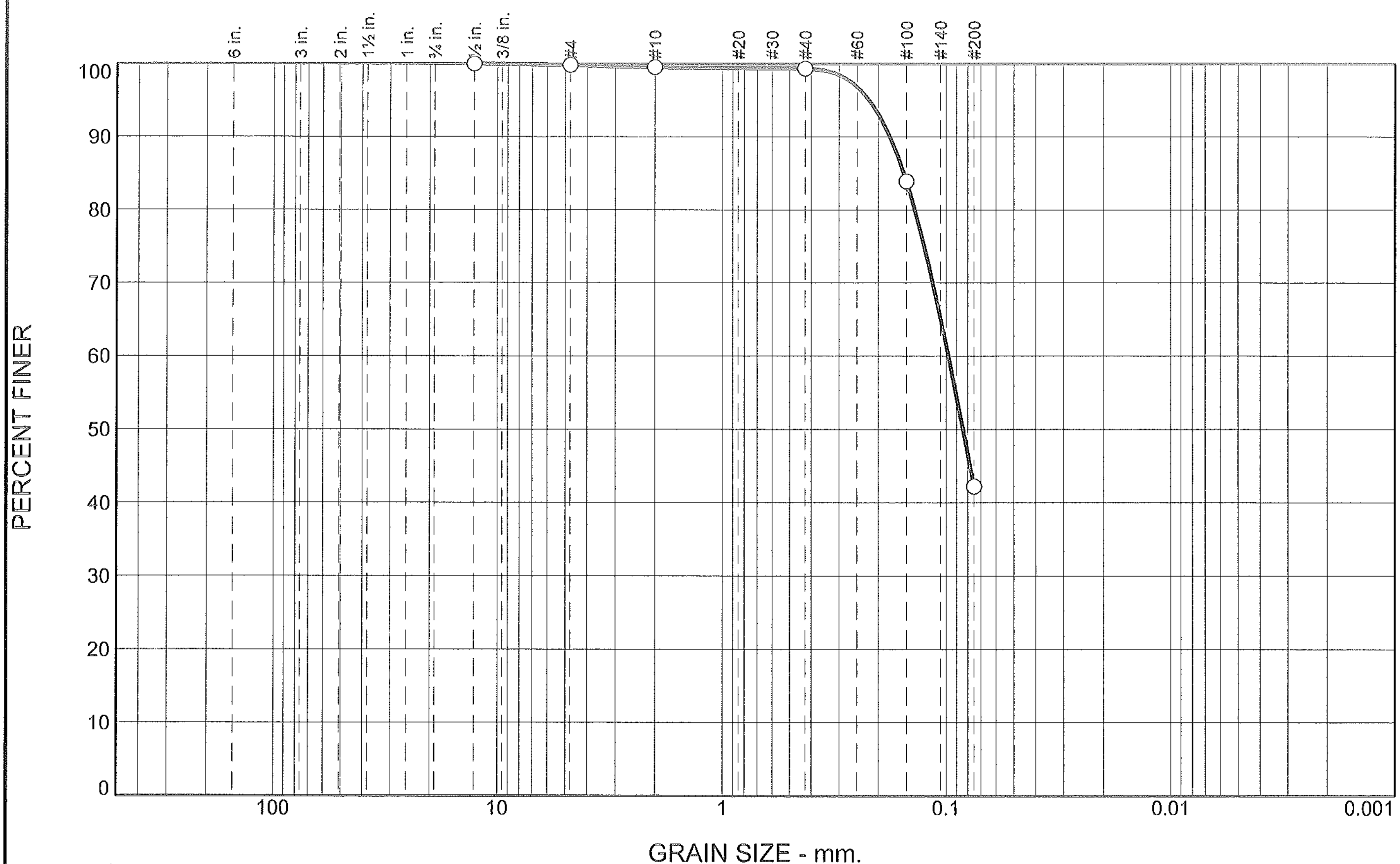
ELEVATION: Approx. 27 ft
DRILLING CONTRACTOR: Holocene Drilling Company

DRILLING METHOD AND EQUIPMENT: Truck-mounted Brainard Kilman BK 81, 140-lb auto-hammer with 30-inch drop, Open-Hole Mud Rotary drilling

WATER LEVELS: Approx Elev 19 on 10/15/12 &
START: 10/12/2012
END: 10/12/2012
LOGGER: S. Shin

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION SOIL NAME, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
120	121.5	15"	SS-24	8-18-19 (37)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, dense, fine to medium sand, est.5~10% fines.	
125	125 / 126.5	10"	SS-25	9-15-19 (34)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above	
130	130 / 131.5	10"	SS-26	10-13-15 (28)	**POORLY GRADED SAND WITH SILT (SP-SM)** dark gray, medium dense, fine to medium sand, est.5~10% fines.	
135	135 / 136.5	8"	SS-27	11-13-13 (26)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above	
140	140 / 141.5	0"	SS-28	6-9-12 (21)	**POORLY GRADED SAND WITH SILT (SP-SM)** similar to above	
					Bottom of Hole @ 141.5'	<u>Well information</u> Well Tag No: 1" PVC Schedule 40 0'~2' Monument 2'~4' Bentonite 4'~20' Sand 20'~30' Screen

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.3	0.1	57.2	42.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
#4	99.8		
#10	99.5		
#40	99.4		
#100	83.9		
#200	42.2		

* (no specification provided)

Material Description

Silty Sand
(lab #7474B)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1777 D_{85}= 0.1540 D_{60}= 0.0975
D_{50}= 0.0840 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: B-1-12 **Depth:** 10'-11.5'
Sample Number: SS-2

Date: 10/23/12

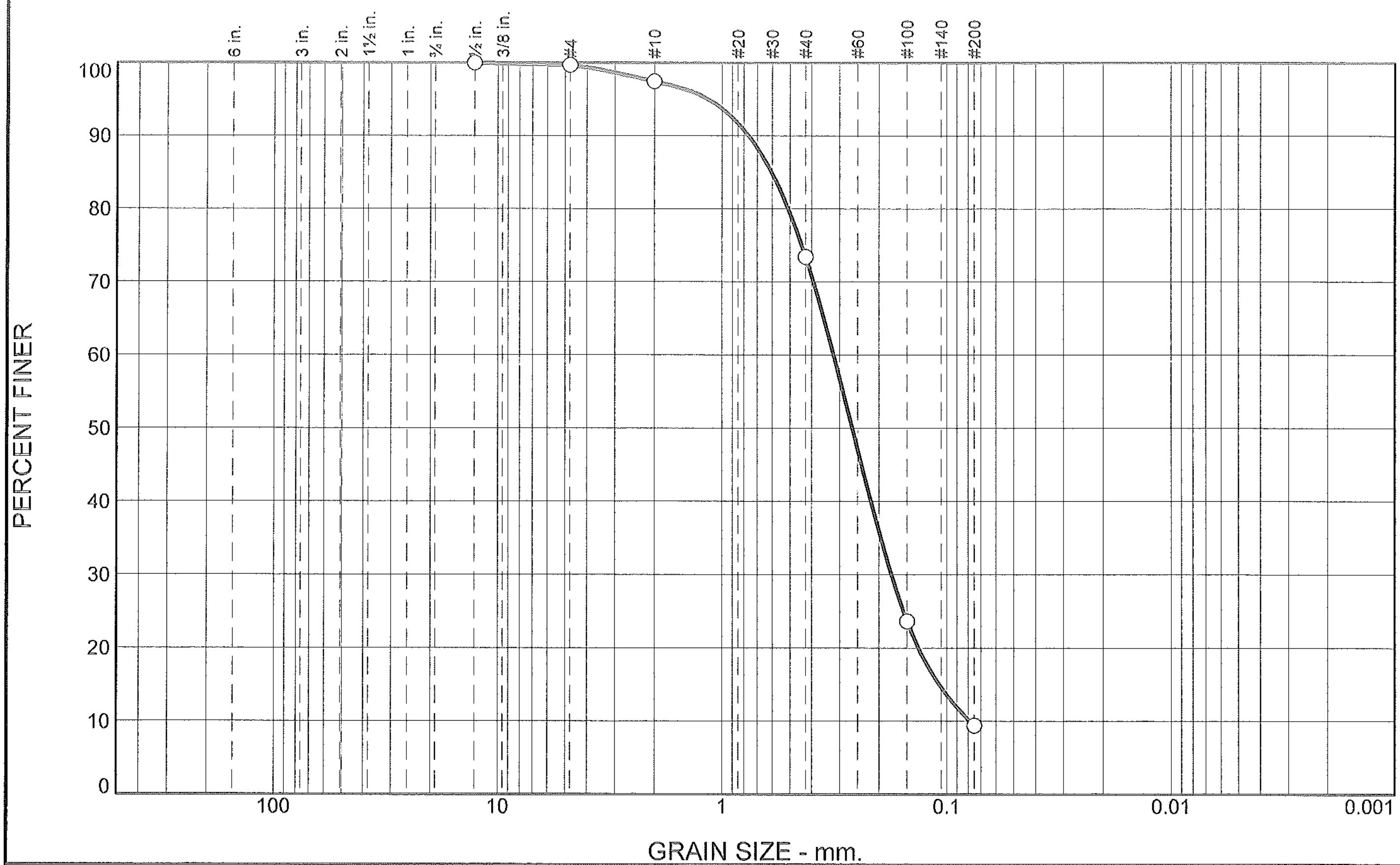
Hayre McElroy & Associates, LLC

Redmond, WA

Client: CH2MHill
Project: American Pile Driving
Project No: 07-103 **Figure**

Tested By: GG **Checked By:** JAM

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	2.2	24.1	64.0	9.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
#4	99.7		
#10	97.5		
#40	73.4		
#100	23.6		
#200	9.4		

* (no specification provided)

Material Description

Sand with Silt
(lab #7474C)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.7617 D_{85}= 0.6035 D_{60}= 0.3193
D_{50}= 0.2637 D_{30}= 0.1762 D_{15}= 0.1080
D_{10}= 0.0785 C_u= 4.07 C_c= 1.24

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: B-1-12 **Depth:** 25'-26.5'
Sample Number: SS-5

Date: 10/23/12

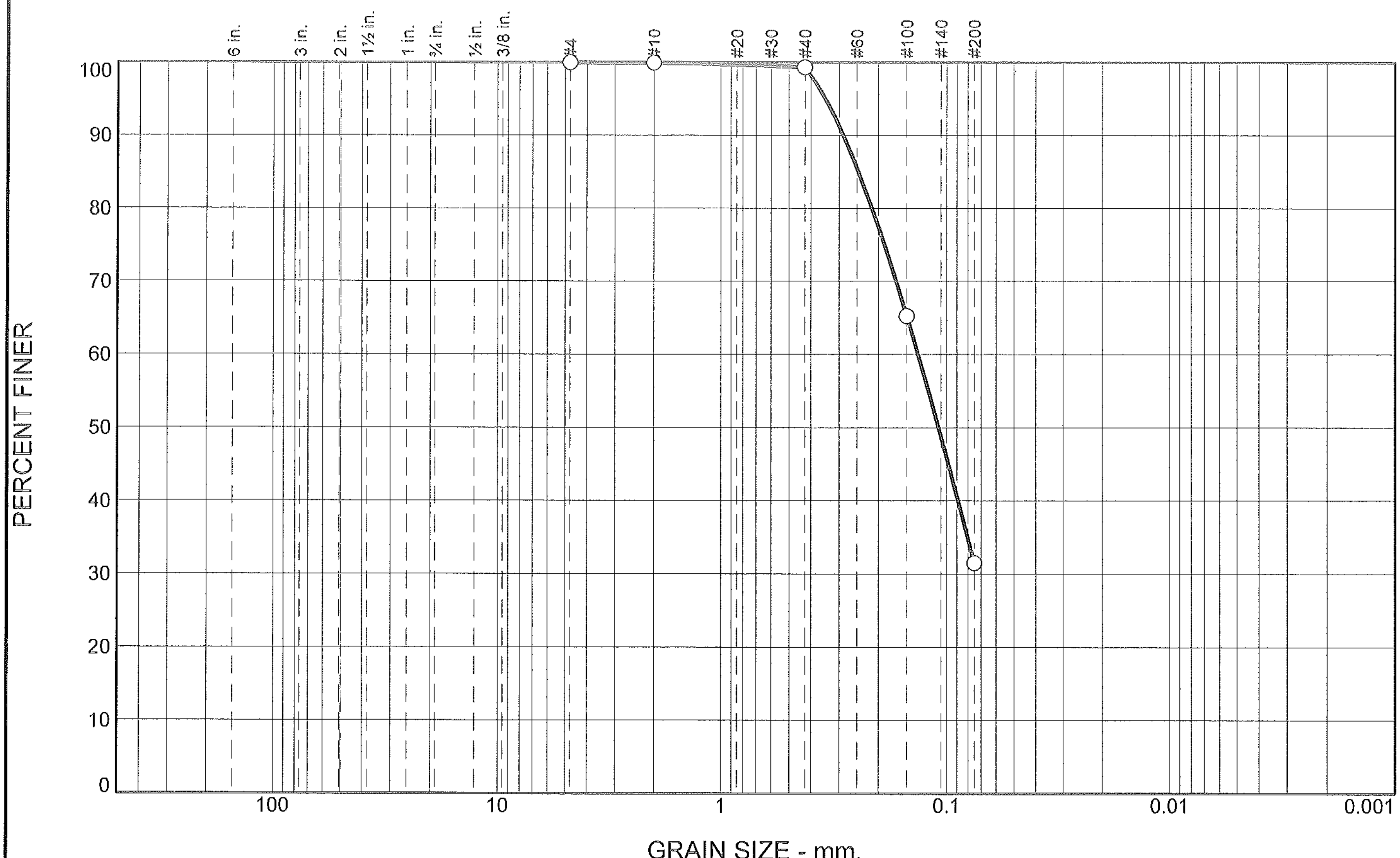
Hayre McElroy & Associates, LLC

Redmond, WA

Client: CH2MHill
Project: American Pile Driving
Project No: 07-103 **Figure**

Tested By: GG **Checked By:** JAM

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	67.9	31.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#40	99.4		
#100	65.2		
#200	31.5		

* (no specification provided)

Material Description

Silty Sand
(lab #7474E)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2866 D_{85}= 0.2455 D_{60}= 0.1338
D_{50}= 0.1086 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: B-1-12 **Depth:** 120'-121.5'
Sample Number: SS-24 **Date:** 10/23/12

Hayre McElroy & Associates, LLC

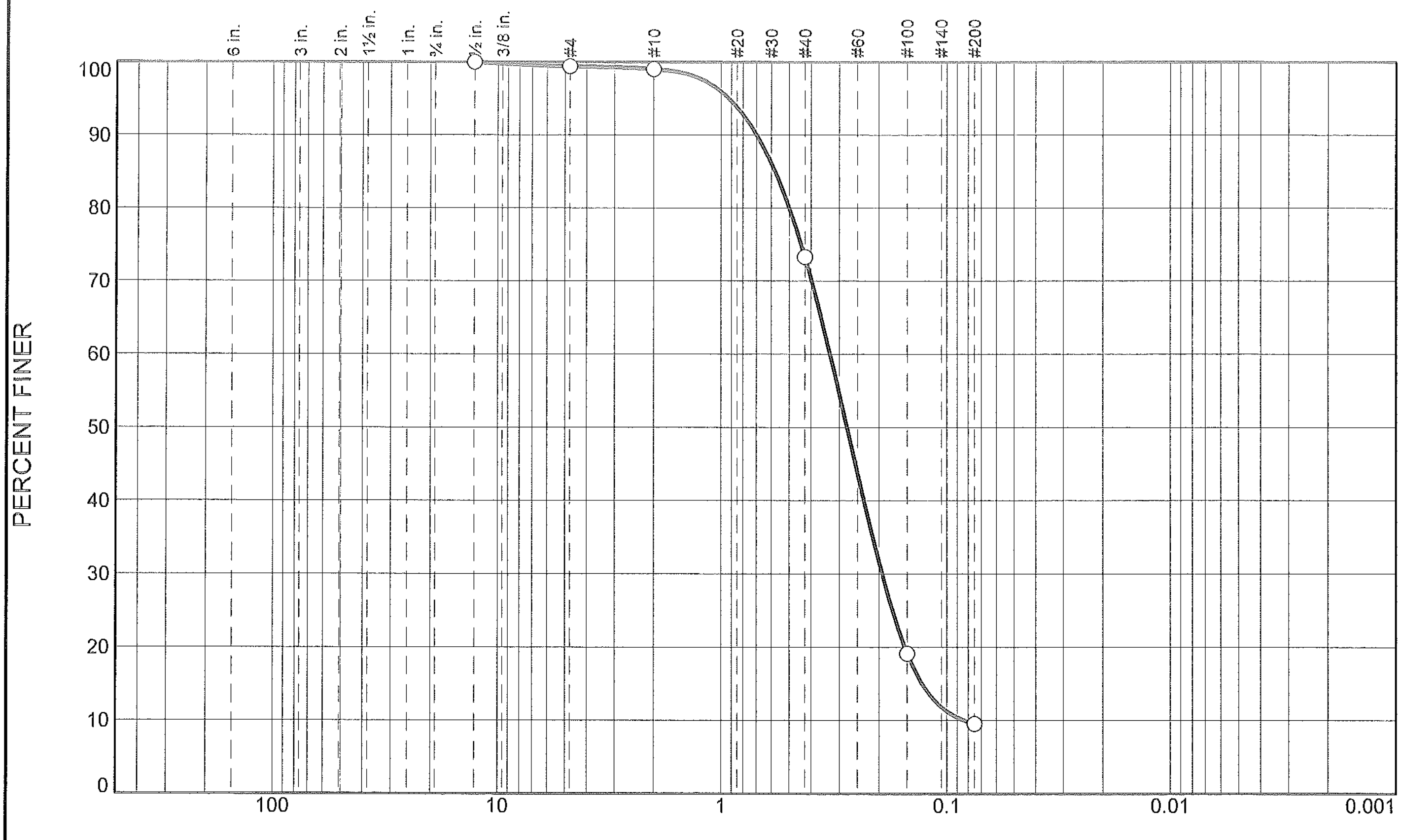
Redmond, WA

Client: CH2MHill
Project: American Pile Driving

Project No: 07-103 **Figure**

Tested By: GG **Checked By:** JAM

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	0.4	25.7	63.8	9.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
#4	99.4		
#10	99.0		
#40	73.3		
#100	19.1		
#200	9.5		

* (no specification provided)

Material Description

Sand with Silt
(lab #7474D)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6949 D_{85}= 0.5764 D_{60}= 0.3298
D_{50}= 0.2776 D_{30}= 0.1939 D_{15}= 0.1288
D_{10}= 0.0831 C_u= 3.97 C_c= 1.37

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: B-1-12 **Depth:** 50'-51.5'
Sample Number: SS-10

Date: 10/23/12

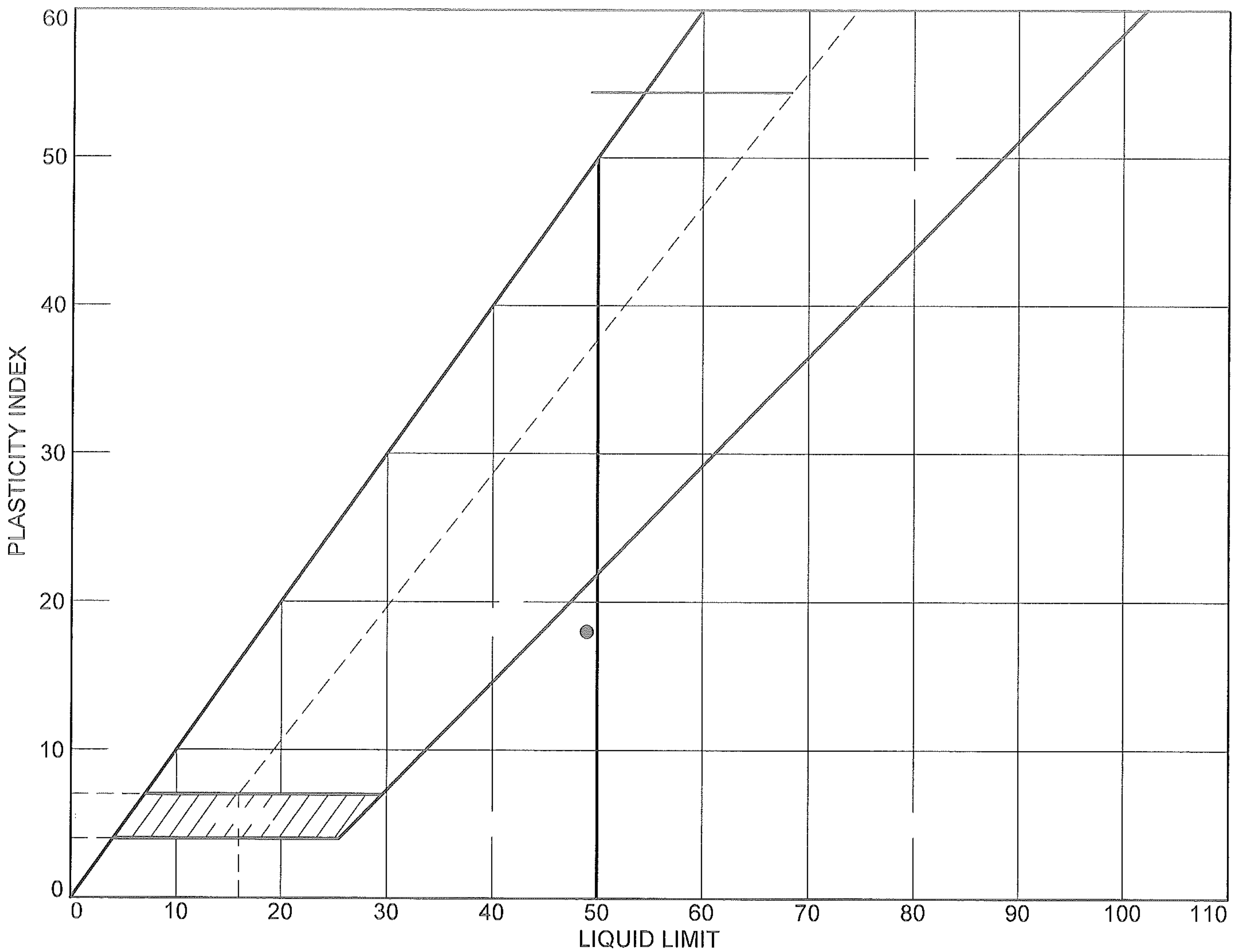
Hayre McElroy & Associates, LLC

Redmond, WA

Client: CH2MHill
Project: American Pile Driving
Project No: 07-103 **Figure**

Tested By: GG **Checked By:** JAM

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1-12	SS-8	40'	48.5	31	49	18	ML

Hayre McElroy & Associates, LLC

Redmond, WA

Client: CH2MHill
Project: American Pile Driving

Project No.: 07-103 **Figure**

Tested By: GG **Checked By:** JAM

Appendix B
Pile Installation Observations

APPENDIX B

Pile Installation Observations

All piles were installed with American Piledriving Equipment, Inc. (APE) HD70 rotary driver mounted on Caterpillar 336E L Excavator. The drill motor was a Poclain MS125. All piles were 7-inch-outside-diameter (OD), 0.453-inch wall, API 5CT P-110 steel tubing in 20-foot lengths with threaded 5.5-inch long couplings. Helix sections were 0.75-inch-thick, T-1 100 ksi plate with a single revolution at 5.25-inch pitch. Where multiple helixes were used, spacing is 7.0 feet along pipe centerline unless otherwise noted. Drill rotation is single speed at 18 to 19 revolutions per minute (rpm).

B.1 Pile No. 1

- Installed October 15, 2012
- Single helix with open end
- No grout
- 80-foot pipe, bottom depth = 78 feet below ground surface (bgs)
- Measure-down inside pipe after installation = 76.3 feet below top of pipe (i.e., 3.7-foot soil plug)
- Installed in 1:20, including time to spot-weld each joint (to allow pipe removal) and with approximately a 15-minute delay to tighten hose.

B.2 Pile No. 2

- Installed October 15, 2012
- Single helix with closed end and 1.5-inch-diameter grout hole located at bottom of leading edge of helical flight (see Photo B-1)
- Neat 5-sack grout
- 20-foot pipe, bottom depth 18 feet bgs
- Grout mixer and pump not calibrated
- General procedure:: Screwed pile 5 feet into ground, then mixed grout and began pumping at approximately 1,000-pounds per square inch (psi) pressure. Truck at 470 pounds with tip at 5-foot depth. Truck at 560 pounds with tip at 18. All 5.5 cubic yards (yd3) of grout pumped. Slight dome evident around pile. Unintended installation procedure.

Photo B-1. *Pile 2 grout hole.*

B.3 Pile No. 3

- Installed October 26, 2012
- Four helixes in bottom 21 feet, 7 inches
- Sanded 7-sack mix
- 80-foot pipe, bottom depth =78 feet bgs
- Grout added for first 38 feet but questionable take thereafter
- General procedure: Pipe screwed 2 feet into ground. Hoses and pipe filled with grout until starting to emerge at ground surface. Continue to screw pipe into ground, filling pipe with addition of each 20-foot section and adding an additional stroke or two during drilling to verify that system was not plugged and pressurizing

system to at least 1,000 psi. Approximately five strokes = 1 cubic foot grout, which fills approximately 5 feet of pipe. Drilled from 18 feet to 38 feet with only 18 feet of grout head in pipe. After installing to 38 feet, backed out pipe to check grout. Filled third section with grout and began drilling, stroking in an additional ten strokes of grout with up to 3,000 psi between 38 feet and 48 feet, then drilling became very slow. No additional grout would leave hole, fourth segment drilled without additional grout (Photo B-2).

Photo B-2. *Picking bottom pile section, Pile 3.*

- Drilling times
  - 2 to 18 feet: 3 minutes
  - 18 to 28 feet: 6 minutes
  - 38 to58 feet: 17 minutes (with high grout pressure)
  - 58 to78 feet: 13 minutes (no or minimal grout pressure)
  - Total duration including pickup, joining, grout filling was 1 hour, 50 minutes

B.4 Pile No. 4

- Installed October 26, 2012
- Single helix at base, closed end, 3-inch by 1-inch grout hole at bottom side of trailing end of pipe (Photo B-3).
- Sanded 7-sack mix
- Begin pumping grout with tip at 9 feet. At 11 strokes and 1,000 psi, ground visibly heaved approximately 3 to 6 inches maximum, affecting 10-foot-diameter area.
- Grout take and drilling penetration rate are variable. Slow drilling and no grout take, then drilling speed up and grout take of 20 strokes with no pressure. Pressurized up to 2,600 psi.
- At depth of 50 feet, very hard drilling. Had to reverse directions multiple times to penetrate. 50 feet to 58 feet in 17 minutes, but no grout take.
- Total duration was 1 hour, 17 minutes for 58 feet

Photo B-3. *Pile 4 grout hole.*

B.5 Pile No. 5

- Installed November 11, 2012
- Single helix at base, closed end, two 1.5-inch diameter grout holes offset 180 degrees—one hole at bottom trailing edge. Each hole has 6-inch by 4-inch protective guard and 1-inch-long nozzle (Photo B-4).
- Neat cement grout with 0.48 w:c.

Photo B-4. *Pile 5 grout holes.*

- Easy drilling from approximately 4 feet to 45 feet except for harder zone (wood?) at about 30 feet. Slow drilling 45 feet to 51 feet.
  - Pumped enough grout to maintain pipe full and provide small flow to surface while drilling from surface to about 45 feet (maximum 500 psi pressure) (Photo B-5). Drilled to 45 feet to 51 feet with slow progress, only pumping grout (approximately 0.1 yd^3) between depth of 49 feet and 51feet at pressures between 700 and 2,000 psi.
  - Average drilling progress 4 feet to 45 feet (excluding slow zone at 30 feet and connection time) was 2.5 feet to 3.0 feet per minutes.
  - Average joining time was approximately 5 minutes
  - Total drilling time for 51 feet was 60 minutes
  - Total grout volume was 3.0 yd^3 (including approximately 0.5 yd^3 in pipes and hoses and approximately 0.5 yd^3 leakage through annulus to ground surface.)

Photo B-5. *Grout returned to ground surface at completion of Pile 5 installation.*

TABLE B-1
Pressure/Torque Readings from Gage in Cab: Pile 5 (Provided by APE)

Section Number	Depth Below Grade (feet)	Pressure Read from Gage (psi)	Pressure Range	Delivered Torque (ft-lb)
1	0 to 18	2,350	Low	14,217
2	18 to 33	2,350 to 2,400	Low	14,217 to 14,544
2	33 to 38	2,700	High	32,724
3	38 to 51	3,200 to 3850	High	38,784 to 46,662

Note: At low pressure, Poclain MS125 drill motor delivers 6,060 ft-lb torque per 1,000 psi drive pressure. At high pressure, it delivers 12,120 ft-lb torque per 1,000 psi drive pressure. Direct drive hydraulic rotary piston motor; pressure readings on drive circuit converted to torque reported accurate to $\pm$ 3%.

Appendix C
Static Load Testing Procedures and Data

APPENDIX C

Static Load Testing Procedures and Data

For this preliminary round of testing, only pile head deflection was measured. The static load frame configuration is shown in Figure C-1 and was used for all axial load tests. A single beam of the load frame, placed on the ground surface and braced against four of the axial load frame piles, was used for the lateral load reaction. Jack calibration data are included at the end of this appendix. In all test cases, the reference beam(s) was supported on wood blocks located at least 6 feet away from the test pile or reaction piles and shielded from direct sunlight during daytime tests.

C.1 Pile 5: Single Helix Fully Grouted—Axial Compression

This was initially tested on November 9, 2013. The reaction piles consisted of 20-foot-long, 7-inch-diameter pipe with a single 18-inch helix and no grout. Figure C-2 shows the initial frame with jack, reference beams, and dial gages. Initially, two reference beams and dial gauges were used. Loads were applied in 20-kip increments at 5-minute intervals. The test had to be terminated at a load of 255 kips because the pile holding the reaction frame had pulled out.

The reaction frame was reconstructed using 40-foot-long piles, each with three helixes spaced 3 feet apart, starting from 1 foot above the pile toe. On November 28, 2013, Pile 5 was reloaded, this time in 40-kip increments with each load held for 5 minutes. A single W18x98 supported on wood blocks at least 6 feet away from the test pile and reaction piles was used as a reference beam. The test was run slightly beyond what would be considered as the Davisson criteria for a grouted pile with 6-inch diameter in order to truly test the pile's capacity. Just before the last load increment, there was a sudden shift in the frame and jack–attributed at the time to readjustment of the load frame, which had become somewhat warped. After the test was completed, it was noted that the pipe wall had begun to bulge just below the loading plate, which may have been the cause of the frame and jack shift.

C.2 Pile 5: Single Helix Fully Grouted—Lateral

A lateral test was conducted on Pile 5 on November 30, 2012. Figure C-3 shows the lateral load frame set-up. After initial seating, loads were initially applied in 4-kip increments every 10 minutes. After 70 minutes, the hold time was decreased to 5 minutes.

C.3 Pile 1: Single Helix Ungrouted—Axial Compression

Pile 1 was tested in axial compression on December 5, 2012. The reaction frame consisted of a 24-foot long W36X231 beam supported by two triple-helix piles on each end. The reaction piles were at least 6.5 feet away from the test pile and the end piles were spaced 2.5 and 3.0 feet apart. The reference beam was a single W18x97 supported on wood blocks. There were problems with the hydraulic pump that operated the loading jack which required restarting the test twice. The pump used to complete the test could not be backed off incrementally, so unloading data was not obtained. Loads were applied in 20-kip increments at 5-minute intervals.

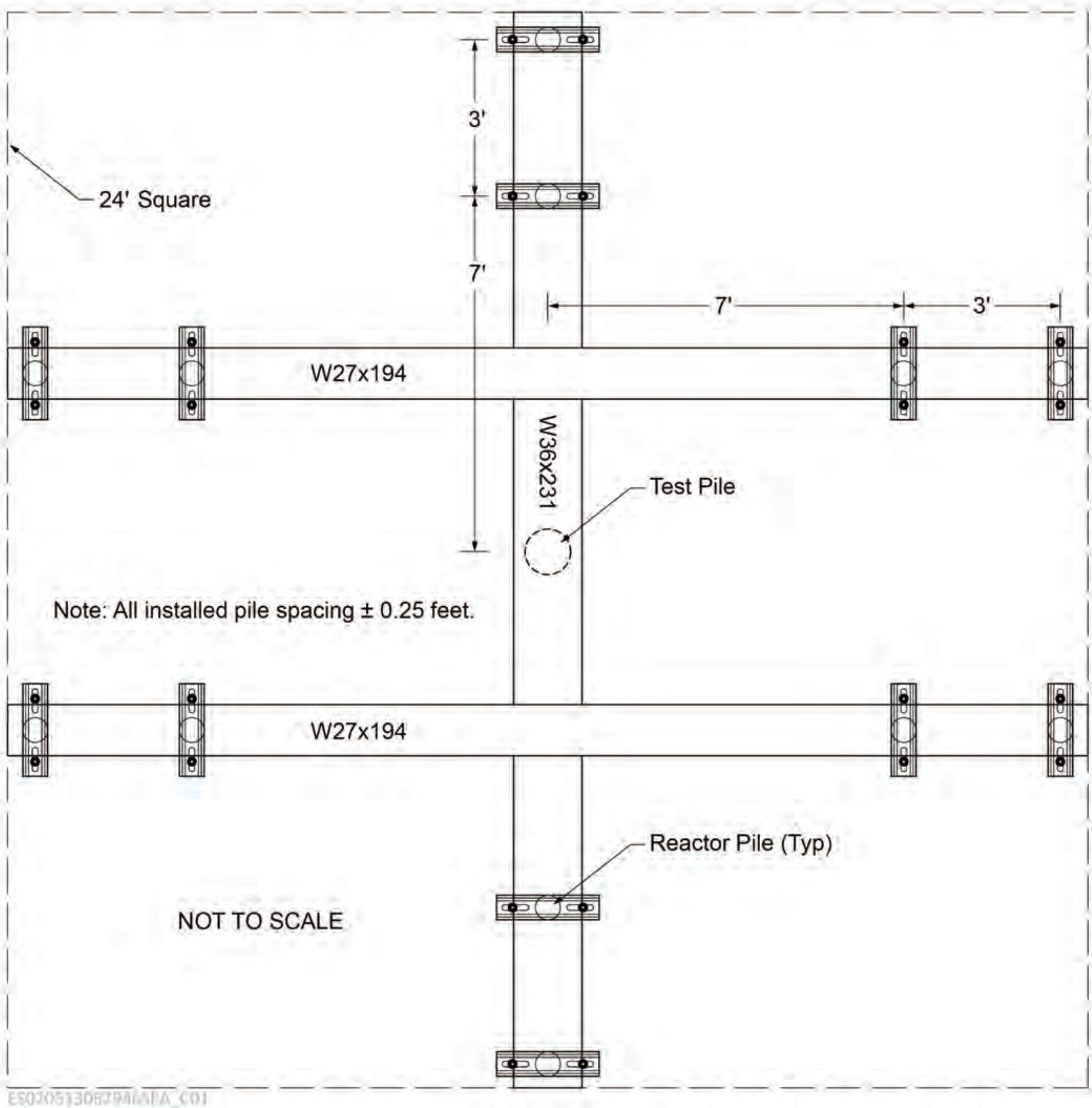


FIGURE C-1
Axial Capacity Load Frame Configuration

FIGURE C-2
Axial Capacity Load Frame Photo

FIGURE C-3
Lateral Load Frame Photo

APE Helical Pile Testing — Lateral Loading

Date: 11/30/2012

Test Pile No. 5

tip=51' bgs

7-inch OD w/ 18-inch single helix — Jack = 200 ton capacity

Grouted full length — Power Team WIKA Model R2006C 9699231

Logger K. Dawson — Serial No. R95 #G100

					Dial Gauge		Comments
			Jack		1		
Test Time	Increment Number	Increm (minute)	Pressure (psi)	Load (kips)	Reading	Deflection (inches)	
	0		25	1	0		seating load
4:05	1	0	150	6	0.01		
4:15	1	10	150	6	0.011		
4:15	2	10	200	8	0.021		
4:25	2	20	200	8	0.021		
4:25	3	20	300	12	0.049		
4:35	3	30	300	12	0.048		
4:35	4	30	400	16	0.08		
4:45	4	40	400	16	0.077		
4:45	5	40	500	20	0.108		
4:55	5	50	500	20	0.107		
4:55	6	50	600	24	0.132		
5:05	6	60	600	24	0.132		
5:05	7	60	700	28	0.156		
5:15	7	70	700	28	0.156		
5:15	8	70	800	32	0.189		
5:20	8	75	800	32	0.19		decreased loading duration time
5:20	9	75	900	36	0.225		
5:25	9	80	900	36	0.225		
5:25	10	80	1000	40	0.256		
5:30	10	85	1000	40	0.258		
5:30	11	85	1100	44	0.285		
5:35	11	90	1100	44	0.287		
5:35	12	90	1200	48	0.328		
5:40	12	95	1200	48	0.331		
5:40	13	95	1300	52	0.368		
5:45	13	100	1300	52	0.372		
5:45	14	100	1400	56	0.41		
5:50	14	105	1400	56	0.415		
5:50	15	105	1500	60	0.46		
5:55	15	110	1500	60	0.466		
5:55	16	110	1600	64	0.503		
6:00	16	115	1600	64	0.51		
6:00	17	115	1700	68	0.553		
6:05	17	120	1700	68	0.561		
6:05	18	120	1800	72	0.604		
6:10	18	125	1800	72	0.615		
6:10	19	125	1900	76	0.69		
6:15	19	130	1900	76	0.704		
6:15	20	130	2000	80	0.762		
6:20	20	135	2000	80	0.8		
6:20	21	135	2100	84	0.856		
6:25	21	140	2100	84	0.923		
6:25	22	140	2200	88	0.97		
6:30	22	145	2200	88	0.986		
6:30	23	145	2300	92	1.06		
6:35	23	150	2300	92	1.18		
6:35	24	150	2400	96	1.225		
6:40	24	155	2400	96	1.34		
6:40	25	155	2500	100	1.4		
6:45	25	160	2500	100	1.468		
6:45	26	160	2600	104	1.53		gap behind pile measured 18" deep (open hole around grout
6:50	26	170	2600	104	1.629		start unload
6:52			2350	94	1.635		
6:55			1820	72.8	1.639		
6:56			1820	72.8	1.642		
6:57			1820	72.8	1.645		
6:58			1500	60	1.618		
7:00			1500	60	1.618		steady between beginning and end of minute interval

APE Helical Pile Testing   Lateral Loading
Date: 11/30/2012
Test Pile No. 5
tip=51' bgs
7-inch OD w/ 18-inch single helix   Jack =   200 ton capacity
Grouted full length   Power Team WIKA   Model R2006C 9699231
Logger   K. Dawson   Serial No. R95 #G100

					Dial Gauge		Comments
			Jack		1		
Test Time	Increment	Increm	Pressure	Load	Reading	Deflection	
	Number	(minute)	(psi)	(kips)		(inches)	
7:00			1250	50	1.602		steady between beginning and end of minute interval
7:01			1000	40	1.55		steady between beginning and end of minute interval
7:02			750	30	1.376		steady between beginning and end of minute interval
7:03			500	20	1.22		steady between beginning and end of minute interval
7:04			250	10	0.898		steady between beginning and end of minute interval
7:05			200	8	0.831		steady between beginning and end of minute interval
7:19			0	0	0.132		had to remove jack to get to 0 pressure
7:26			0	0	0.124		gap in front of pile ~ 12" deep

APE Helical Pile Testing　Lateral Loading　New pile installed at location 20' west of P5

Date: 5/18/2013

Test Pile No. 6

tip=51' bgs

7-inch OD w/ 18-inch single helix　Jack = 200 ton capacity

No Grout　Power Team / WIKA Model EDX 220 6

Logger P. Suver　Serial No. 352962 Test cert Report #10146A Test Date 11-20-2012

					Dial Gauge		Comments
			Jack		1		
Test Time	Increment Number	Increm (minute)	Pressure (psi)	Load (kips)	Reading	Deflection (inches)	Material and depth - 7" OD .408 wall 29# per ft 54ft deep
	0		100	4	0		seating load
4:05	1	0	200	8	0.13		
4:15	1	10	200	8	0.13		
4:15	2	10	200	8	0.13		
4:25	2	20	200	8	0.13		No creep
4:25	3	20	300	12	0.391		
4:35	3	30	300	12	0.395		
4:35	4	30	300	12	0.431		constant slow creep
4:45	4	40	400	16	0.599		
4:45	5	40	400	16	0.601		
4:55	5	50	400	16	0.622		
4:55	6	50	400	16	0.622		
5:05	6	60	400	16	0.632		
5:05	7	60	500	16	0.685		
5:15	7	70	500	16	0.695		
5:15	8	70	500	16	0.71		
5:20	8	75	500	16	0.72		Constant creep
5:20	9	75	600	24	0.799		
5:25	9	80	600	24	0.81		
5:25	10	80	600	24	0.815		
5:30	10	85	600	24	0.834		constant creep
5:30	11	85	700	28	0.858		
5:35	11	90	700	28	0.86		
5:35	12	90	700	28	0.862		
5:40	12	95	700	28	0.864		
5:40	13	95	700	28	0.867		Creep much slower
5:45	13	100	800	32	0.921		
5:45	14	100	800	32	0.93		
5:50	14	105	800	32	0.936		
5:50	15	105	800	32	0.94		
5:55	15	110	800	32	0.94		
5:55	16	110	800	32	0.94		Very slow creep
6:00	16	115	900	36	1.099		
6:00	17	115	900	36	1.1		
6:05	17	120	900	36	1.111		
6:05	18	120	900	36	1.121		
6:10	18	125	900	36	1.14		Very slow creep
6:10	19	125	1000	40	1.2		
6:15	19	130	1000	40	1.22		
6:15	20	130	1000	40	1.23		
6:20	20	135	1000	40	1.24		
6:20	21	135	1000	40	1.25		
6:25	21	140	1000	40	1.26		
6:25	22	140	1000	40	1.267		no creep at load
6:30	22	145	600	24	1.262		
6:30	23	145	600	24	1.25		
6:35	23	150	600	24	1.23		
6:35	24	150	600	24	1.23		very small rebound?
6:40	24	155	300	12	1.11		
6:40	25	155	300	12	1.1		
6:45	25	160	300	12	1.075		
6:45	26	160	300	12	1.072		
6:50	26	170	300	12	1.071		Very small rebound
6:52			290	11.6	0.591		
6:55			280	11.2	0.432		Rebound constant as un loading to zero
6:56			270	10.8	0.327		
6:57			260	10.4	0.322		
6:58			220	8.8	0.312		
7:00			200	8	0.312		Rebound constant as un loading to zero

APE Helical Pile Testing | Lateral Loading | New pile installed at location 20' west of P5

Date: 5/18/2013

Test Pile No. 6

tip=51' bgs

7-inch OD w/ 18-inch single helix | Jack = 200 ton capacity

No Grout | Power Team / WIKA Model EDX 220 6

Logger P. Suver | Serial No. 352962 Test cert Report #10146A Test Date 11-20-2012

					Dial Gauge		Comments
			Jack		1		
Test Time	Increment	Increm	Pressure	Load	Reading	Deflection	Material and depth - 7" OD .408 wall 29# per ft 54ft deep
	Number	(minute)	(psi)	(kips)		(inches)	
7:00			180	7.2	0.298		
7:01			170	6.8	0.287		
7:02			160	6.4	0.267		
7:03			150	6	0.255		Rebound constant as un loading to zero
7:04			110	4.4	0.245		
7:05			90	3.6	0.231		
7:19			50	2	0.205		
7:26			0	0	0.205		Rebound constant as un loading to zero

APE Helical Pile Load Test

Date	12/5/2012	Pile Length	80 feet	D	7	inches
Test Pile No	1			Dr	12	inches
Description	Single helix 77' BGS, no grout			Davisson X	0.255	inches

Jack #13310A: Power Team R2006C RAM ID#R94, Wika Gauge ID# G100, from Jaxx LLC, Seattle, WA

Logger K. Dawson

					Dial Gauge		Rotation							Comments
			Jack		1						Theoretical		Davisson	
Test Time	Increment Number	Increm (minute)	Pressure (psi)	Load (kips)	Reading	Deflection (inches)	Reading (+16th in)	Rotation (degrees)	Calculated Rotational Downward Deflection (inchs)	Measured Deflection minus Rotational Deflection (inches)	Elastic Pile Deflection (inches)	Combined Rotational and Elastic Pile Deflection (inches)	Criteria (without rotation) (inches)	*flexible metal scale on outside of 7" diam pile
2:10	0	0	0	0	0	0	0	0	0.0000	0.0000	0.000	0.000		possitive rotation=clockwise looking down
2:15	1	5	250	20	0.048	0.048	0	0	0.0000	0.0480	0.071	0.071	0.327	
2:20	2	10	500	40	0.132	0.132	0	0	0.0000	0.1320	0.142	0.142	0.398	
2:25	2	15	500	40	0.13	0.13	0	0	0.0000	0.1300	0.142	0.142	0.398	
2:25	3	15	750	60	0.24	0.24	1	0.511569	0.0075	0.2325	0.213	0.221	0.469	
2:30	3	20	750	60	0.233	0.233	1	0.511569	0.0075	0.2255	0.213	0.221	0.469	
2:30	4	20	1000	80	0.33	0.33	2	1.023139	0.0149	0.3151	0.284	0.299	0.540	
2:35	4	30	1000	80	0.325	0.325	2	1.023139	0.0149	0.3101	0.284	0.299	0.540	
2:35	5	30	1250	100	0.448	0.448	3	1.534708	0.0224	0.4256	0.356	0.378	0.611	
2:40	5	40	1250	100	0.44	0.44	3	1.534708	0.0224	0.4176	0.356	0.378	0.611	
2:40	6	40	1500	120	0.599	0.599	4	2.046278	0.0298	0.5692	0.427	0.457	0.682	
2:45	6	50	1500	120	0.594	0.594	4	2.046278	0.0298	0.5642	0.427	0.457	0.682	
2:45	7	50	1750	140	0.626	0.626	4	2.046278	0.0298	0.5962	0.498	0.528	0.753	
2:50	7	60	1750	140	0.621	0.621		0	0.0000	0.6210	0.498	0.498	0.753	out of hydraulic oil; can't get next load
3:35	RL1		500	40	0.23	0.23	0	0	0.0000	0.2300	0.142	0.142	0.398	new hydraulic pump
3:40	RL2		800	64	0.328	0.328	0	0	0.0000	0.3280	0.228	0.228	0.483	
3:55	RL3		1500	120	0.5	0.5	0	0	0.0000	0.5000	0.427	0.427	0.682	
3:57			1400	112	0.49	0.49	0	0	0.0000	0.4900	0.398	0.398	0.654	pump failing
			0	0	0.126	0.126	0	0	0.0000	0.1260	0.000	0.000	0.255	switch out to new hydraulic pump
			950	76	0.341	0.341	0	0	0.0000	0.3410	0.270	0.270	0.526	
			950	76	0.248	0.341	0	0	0.0000	0.3410	0.270	0.270	0.526	no movement - rezeroed gauge which was tilting
4:30			1750	140	0.488	0.581	0	0	0.0000	0.5810	0.498	0.498	0.753	
4:30			2000	160	0.641	0.734	0	0	0.0000	0.7340	0.569	0.569	0.824	
4:35			2000	160	0.658	0.751	0	0	0.0000	0.7510	0.569	0.569	0.824	
4:40			2250	180	0.962	1.055	0	0	0.0000	1.0550	0.640	0.640	0.895	
4:45			2250	180	0.99	1.083	0	0	0.0000	1.0830	0.640	0.640	0.895	
4:45			2500	200	1.226	1.319	0	0	0.0000	1.3190	0.711	0.711	0.967	out of stroke on jack
4:50			2350	188	1.239	1.332	0	0	0.0000	1.3320	0.668	0.668	0.924	
			500	40		0.438	-3	-1.53471	-0.0224	0.4604	0.142	0.120	0.398	hard measurement

APE Helical Pile Load Test

Date 11/9/2012 Pile Length 52 feet D 18 inches

Test Pile No 5 Dr 12 inches

Description Single helix 51' BGS, neat cement grout full length Davisson X 0.283 inches

Note: load frame pre-tensioned to 400 psi.

Jack #13310A: Power Team R2006C RAM ID#R94, Wika Gauge ID# G100, from Jaxx LLC, Seattle, WA

Logger K. Dawson/D. Dailer

					Dial Gauge					Dgrout= pileID	Dgrout= c48	Dgrout= pileID	Dgrout= c48	Comments
			Jack		1		2		Average	Theoretical	Theoretical	Davisson	Davisson	
Test Time	Increment	Increm	Pressure	Load	Reading	Deflection	Reading	Deflection	Deflection	Deflection	Deflection	Criteria	Criteria	
	Number	(minute)	(psi)	(kips)	0.042	(inches)	0.2	(inches)	(inches)	(inches)	(inches)	(inches)	(inches)	
3:10	1	0	500	20	0.069	0.011	0.246	0.046	0.0285	0.030	0.010	0.313	0.292	
3:15	2	0	1000	40	0.005	0.047	0.255	0.055	0.051	0.060	0.019	0.343	0.302	
3:20	2	5	1000	40	0.004	0.046			0.046	0.060	0.019	0.343	0.302	
3:20	3	0	1500	60	0.035	0.077	0.275	0.075	0.076	0.090	0.029	0.373	0.312	
3:25	3	5	1500	60	0.032	0.074			0.074	0.090	0.029	0.373	0.312	
3:25	4	0	2000	80	0.068	0.11	0.288	0.088	0.099	0.121	0.039	0.403	0.321	
3:30	4	5	1900	75	0.062	0.104			0.104	0.113	0.036	0.396	0.319	
3:30	5	0	2500	100	0.102	0.144	0.307	0.107	0.1255	0.151	0.048	0.433	0.331	
3:35	5	5	2300	95	0.096	0.138			0.138	0.143	0.046	0.426	0.328	
3:35	6	0	3000	120	0.134	0.176	0.325	0.125	0.1505	0.181	0.058	0.463	0.340	
3:40	6	5	2800	115	0.13	0.172			0.172	0.173	0.055	0.456	0.338	
3:40	7	0	3500	140	0.16	0.202	0.347	0.147	0.1745	0.211	0.067	0.494	0.350	
3:45	7	5	3300	130	0.155	0.197			0.197	0.196	0.063	0.479	0.345	
3:45	8	0	4000	160	0.181	0.223	0.371	0.171	0.197	0.241	0.077	0.524	0.360	
3:50	8	5	3600	145	0.175	0.217	0.362	0.162	0.1895	0.218	0.070	0.501	0.353	
3:50	9	0	4500	180	0.201	0.243	0.392	0.192	0.2175	0.271	0.087	0.554	0.369	
3:55	9	5	4000	160	0.192	0.234	0.372	0.172	0.203	0.241	0.077	0.524	0.360	
3:55	10	0	5000	200	0.221	0.263	0.413	0.213	0.238	0.301	0.096	0.584	0.379	
4:00	10	5	4500	180	0.212	0.254	0.403	0.203	0.2285	0.271	0.087	0.554	0.369	
4:00	11	0	5500	220	0.243	0.285	0.437	0.237	0.261	0.331	0.106	0.614	0.389	
4:05	11	5	5000	200	0.233	0.275	0.425	0.225	0.25	0.301	0.096	0.584	0.379	
4:05	12	0	6000	240	0.265	0.307	0.454	0.254	0.2805	0.362	0.116	0.644	0.398	
4:10	12	5	5500	220	0.255	0.297	0.442	0.242	0.2695	0.331	0.106	0.614	0.389	
4:10	13	0	6300	255	0.27	0.312	0.458	0.258	0.285	0.384	0.123	0.667	0.406	test ended - max throw of jack at 6"
4:15	13	5	5500	220			0.452	0.252	0.252	0.331	0.106	0.614	0.389	
				720						1.0848526	0.347	1.367519	0.630	

APE Helical Pile Load Test
Date 11/28/2012 Pile Length 52 feet
Test Pile No 5 2nd try (first on 11/9/12)
Description Single helix 51' BGS, neat cement grout full length
Note: continuation of load testing begun on 11-9-12 and halted after reaction frame deformed excessively
Jack #13310A and B in parallel: Power Team R2006C RAM ID#R94, Wika Gauge ID# G100, from Jaxx LLC, Seattle, WA
Logger K. Dawson Dial gages rotating backward

					Dial Gauge					
			Jack		1		2		Average	
Test Time	Increment	Increm	Pressure	Load	Reading	Deflection	Reading	Deflection	Deflection	Comments
2:42	Number	(minute)	(psi)	(kips)	1.1	(inches)	1.1	(inches)	(inches)	
2:42										
2:47	1	5	500	40	1.052	0.048	1.054	0.046	0.047	
2:52	2	10	1000	80	1.017	0.083	1.018	0.082	0.0825	
2:57	3	15	1500	120	0.982	0.118	0.981	0.119	0.1185	
3:02	4	20	2000	160	0.944	0.156	0.941	0.159	0.1575	
3:07	5	25	2500	200	0.9	0.2	0.895	0.205	0.2025	
3:12	6	30	3000	240	0.862	0.238	0.854	0.246	0.242	
3:17	7	35	3250	260	0.829	0.271	0.818	0.282	0.2765	
3:22	8	40	3500	280	0.797	0.303	0.785	0.315	0.309	
3:27	9	45	3750	300	0.77	0.33	0.755	0.345	0.3375	
3:32	10	50	4000	320	0.745	0.355	0.728	0.372	0.3635	
3:37	11	55	4250	340	0.708	0.392	0.69	0.41	0.401	
3:42	12	60	4500	360	0.681	0.419	0.663	0.437	0.428	
3:47	13	65	4750	380	0.65	0.45	0.63	0.47	0.46	
3:52	14	70	5000	400	0.623	0.477	0.601	0.499	0.488	
3:57	15	75	5250	420	0.58	0.52	0.56	0.54	0.53	
4:02	16	80	5500	440	0.549	0.551	0.528	0.572	0.5615	
4:07	17	85	5750	460	0.502	0.598	0.482	0.618	0.608	
4:12	18	90	6000	480	0.469	0.631	0.446	0.654	0.6425	
4:17	19	95	6250	500	0.429	0.671	0.408	0.692	0.6815	
4:22	20	100	6500	520	0.382	0.718	0.362	0.738	0.728	
4:27	21	105	6750	540	0.331	0.769	0.315	0.785	0.777	
4:32	22	110	7000	560	0.308	0.792	0.287	0.813	0.8025	
4:37	23	115	7250	580	0.257	0.843	max out at 0.250		0.843	
4:42	24	120	7500	600	0.21	0.89			0.89	
4:47	25	125	7750	620	0.158	0.942			0.942	
4:52	26	130	8000	640	0.065	1.035			1.035	
4:57	27	135	8250	660	0.994	1.106			1.106	deflection equation reset
5:02	28	140	8500	680	0.915	1.185			1.185	
5:28	29	166	8750	700	0.752	1.348			1.348	
5:39	30	177	9000	720	0.584	1.516			1.516	
5:40			7800	625	0.534	1.566			1.566	frame shifted - reading unclear
5:42		180	9000	720	0.494	1.606			1.606	
5:43			9000	720	0.592	1.508			1.508	unload begin
5:44			5250	420	0.535	1.565			1.565	
5:45			4000	320	0.562	1.538			1.538	
5:46			3000	240	0.601	1.499			1.499	
5:47			2000	160	0.66	1.44			1.44	
5:48			1000	80	0.73	1.37			1.37	
										major shift-weld between pile head and plates broke. Jacks shifted

Appendix D
Dynamic Load Testing Report

APPENDIX D

Dynamic Load Testing Report

Robert Miner Dynamic Testing, Inc.

Dynamic Measurements and Analyses for Deep Foundations

May 20, 2013

Mr. Daniel Collins
American Piledriving Equipment, Inc.
7023 South 196th
Kent, WA 98032-2185

Re: Dynamic Pile Measurements and CAPWAP Analyses
Pile 1, Ungrouted 7.0" OD Helical, December 8, 2012
Pile 5, Grouted 7.0" OD Helical Pile, February, 20, 2013
APE Yard, Kent, WA RMDT Job No. 12F60

Dear Sir,

This report presents results obtained from dynamic pile measurements and CAPWAP analyses completed by Robert Miner Dynamic Testing, Inc. (RMDT) for the project referenced above. The objective of the testing and analysis was evaluation of the soil resistance to pile penetration during restrike.

PROJECT AND TESTING DETAILS

Piles

Restrike tests were completed on two 7.0" OD helical piles installed on the premises of American Pile Driving, Inc. (APE) in Kent WA. Pile 1 was an ungrouted pile installed approximately 77 ft below the adjacent soil. Pile 5 was a grouted pile installed approximately 47.5 ft below the soil line at the time of the test. We understand that both piles had a wall thickness of 0.453", and the bottom of the helix was located approximately 1 ft above pile tip in each case. For the test of Pile 5 on February 8, 2013 a 6 ft long heavy-wall pile extension (7"OD x 1.0" wall) was in place and our PDA sensors were located at the center of this extension. For details on each helix or the pile installation please refer to documents prepared by other project participants.

Hammers

An APE D50–42 and an APE D100-42 open end diesel hammer were used to test Piles P1 and P5, respectively. For the hammer blows used on our analyses these hammers were operated manually using a standard or reduced tripping stroke of approximately 5 to 7 ft. The D50-42 and D100-42 have rams weighing approximately 11 and 22 kips, respectively.

Instrumentation

Dynamic measurements were made with two strain sensors and two accelerometers bolted to the surface of the pile near the pile top. Signals from the sensors were processed and stored by a Pile Driving Analyzer® (PDA). For each hammer blow the PDA displayed the measurements as plots of force and velocity, and computed a variety of results. RMDT's

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Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

engineer reviewed the measurements and the computed results during and after driving. Appendix A contains general information on our methods for measurement and analysis.

Test Sequence

On December 8, 2012 Piles P1 and P5 were tested during brief restrike tests. Pile P5 was also tested on February 20, 2013. Analyses given here for Pile P5 are based on the test of February 20 because the heavy wall upper pile section was necessary for effective dynamic measurements.

PRESENTATION AND DISCUSSION OF RESULTS

Following the field testing RMDT completed CAPWAP® analysis of the soil's resistance to downward pile movement as the pile was struck by the respective impact hammers. CAPWAP analysis is an iterative signal matching method which is based on use of the measured force and velocity as recorded by the Pile Driving Analyzer. Appendix B contains results of the two CAPWAP analyses and Table 1 summarizes the results.

Table 1. Summary of CAPWAP Results.

Pile	Test	Approximate Depth Below Grade (ft)	Computed Ultimate Soil Resistance, kips		
			Total	Shaft	Toe
Pile 1	Restrike, 12/8/12	77	170	125	45
Pile 5	Restrike, 2/20/13	48	660	360	300

The resistance values computed with CAPWAP and given in Table 1 are estimates of the ultimate soil resistance for downward axial pile loading.

Pile 1 was twice struck with the D50-42 ram falling from the tripping stroke of approximately 5.5 ft, and then once with lowest available fuel setting. These three hammer blows apparently caused no net pile advancement. Our CAPWAP analysis yielded an ultimate axial soil resistance of 170 kips, with 125 kips of friction and 25 kips of end bearing. The computed unit friction resistance (kips per lineal ft) increased gradually with depth and was primarily located on the lower 25 ft of the pile.

Pile 5 was struck using only the tripping stroke of the APE D100-42 hammer. Our CAPWAP analysis yielded an axial resistance of 660 kips derived from 360 kips of friction and 300 kips of end bearing. The computed shaft friction was primarily located in the lowest 15 ft of the pile. For Pile 5 the grout surrounding the steel section caused a change in the axial pile stiffness and pile impedance. Uncertainty in the dimensions of the grout column caused some uncertainty in the CAPWAP impedance and soil resistance model. Such uncertainty was expected to have little effect on the computed ultimate resistance, but likely did cause a modest increase in the uncertainty associated with the CAPWAP distribution of shaft friction and the relative balance of friction and end bearing.

Please to not hesitate to contact us if you have questions regarding this transmittal or the work we completed for this project.

Sincerely,

Robert Miner, P.E.

Robert Miner Dynamic Testing, Inc.

May 20, 2013

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing*.

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- <u>Bearing capacity</u> at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- <u>Dynamic pile stresses</u>, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- <u>Pile integrity</u> assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- <u>Hammer performance</u> parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- Bearing capacity i.e. the mobilized capacity present at the time of testing
- Resistance distribution including shaft resistance and end bearing components
- Stresses in pile or shaft calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.
- Shaft impedance vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile
- Dynamic soil parameters for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

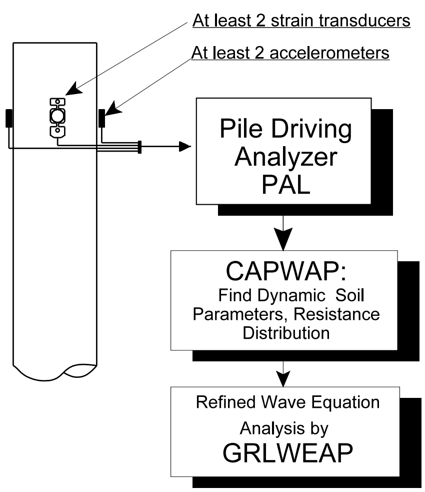
The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or “Pulse-Echo Method” of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called “Transient Response Method” which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the “bearing graph.” Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the “Refined Wave Equation Analysis” or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time $t + 2L/c$
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\, c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \qquad (4)$$

$$W_d = ½[F(t) + Zv(t)] \qquad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \tag{6}$$

with

$$\alpha_i = \tfrac{1}{2}(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \tag{7}$$

where

- W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.
- W_{UD} is the upwards traveling wave due to the damage reflection.
- W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_o^t F(t)v(t)\,dt \tag{8a}$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \tag{8b}$$

where

- E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \tag{9}$$

where

- g is the earth's gravitational acceleration,
- T_B is the time between two hammer blows,
- h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \tag{10}$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \tag{11}$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \tag{12a}$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \tag{12b}$$

or strain

$$\epsilon = v\,/\,c \tag{12c}$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(*B) Relaxation*

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90% of yield strength for steel piles

85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

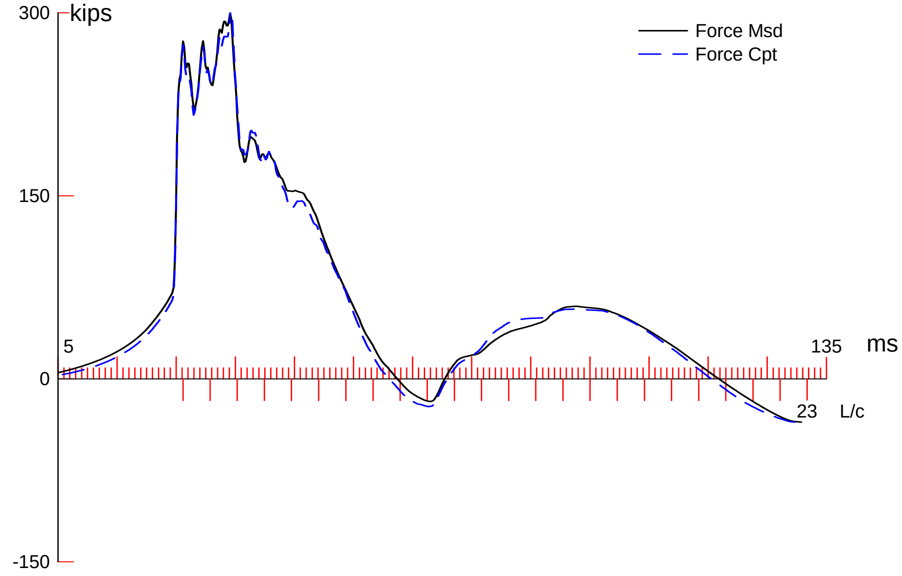
Wave equation analysis results

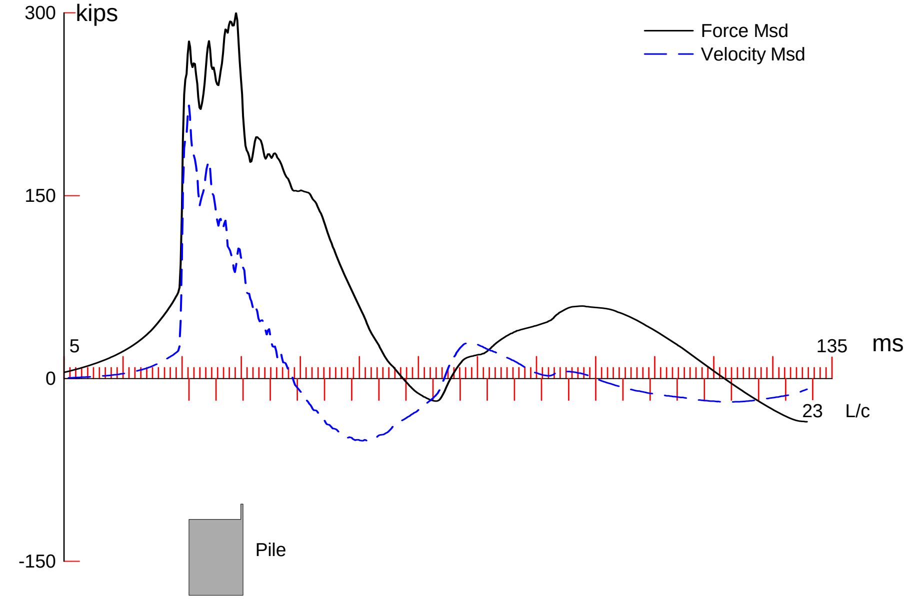
The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

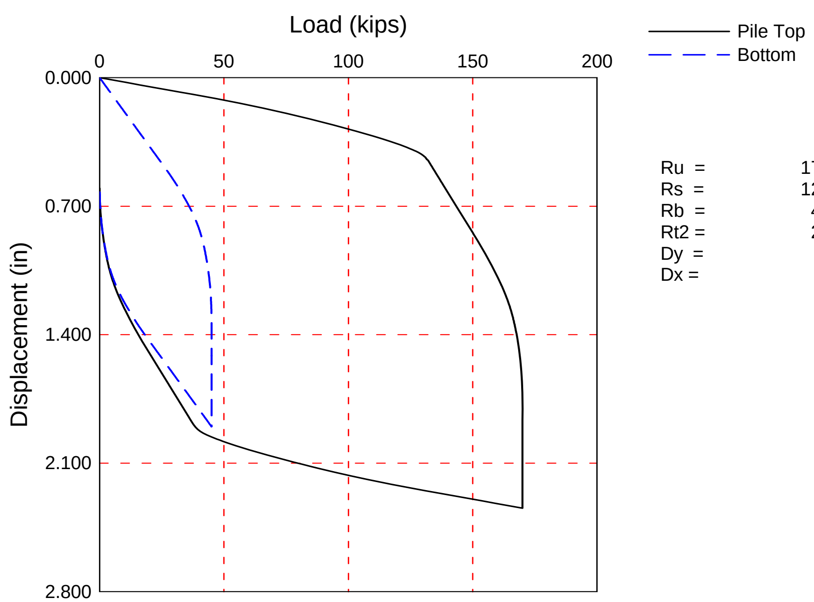
As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

Appendix B

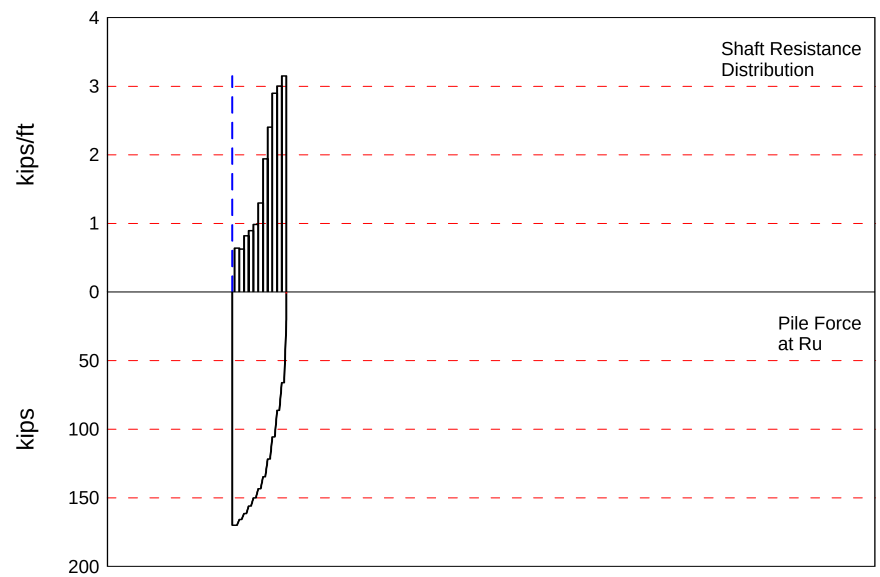
Results of CAPWAP Analysis







Ru =	170.0	kips
Rs =	125.0	kips
Rb =	45.0	kips
Rt2 =	25.0	kips
Dy =	1.75	in
Dx =	2.35	in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 170.0; along Shaft 125.0; at Toe 45.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				170.0					
1	10.0	9.5	4.3	165.7	4.3	0.45	0.25	0.120	0.100
2	16.7	16.2	4.2	161.5	8.5	0.63	0.34	0.120	0.100
3	23.4	22.9	5.5	156.0	14.0	0.82	0.45	0.120	0.100
4	30.1	29.6	6.0	150.0	20.0	0.90	0.49	0.120	0.100
5	36.8	36.3	6.6	143.4	26.6	0.99	0.54	0.120	0.100
6	43.5	43.0	8.7	134.7	35.3	1.30	0.71	0.120	0.100
7	50.2	49.7	13.0	121.7	48.3	1.94	1.06	0.120	0.100
8	56.9	56.4	16.1	105.6	64.4	2.40	1.31	0.120	0.100
9	63.6	63.1	19.4	86.2	83.8	2.90	1.58	0.120	0.100
10	70.3	69.8	20.1	66.1	103.9	3.00	1.64	0.120	0.100
11	77.0	76.5	21.1	45.0	125.0	3.15	1.72	0.120	0.100
2nd	Toe		25.0					0.126	1.000
Avg. Shaft			11.4			1.63	0.89	0.120	0.100
Toe			20.0				74.84	0.150	0.700

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.904	0.181
Damping Type			Smith
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	80	
Soil Plug Weight	(kips)		0.22
Soil Support Dashpot		2.300	0.000
Soil Support Weight	(kips)	1.28	0.00

max. Top Comp. Stress = 31.8 ksi (T= 34.3 ms, max= 1.033 x Top)
max. Comp. Stress = 32.9 ksi (Z= 16.7 ft, T= 33.5 ms)
max. Tens. Stress = -3.91 ksi (Z= 10.0 ft, T= 130.9 ms)
max. Energy (EMX) = 27.5 kip-ft; max. Measured Top Displ. (DMX)= 1.42 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	296.5	-35.9	31.8	-3.85	27.47	13.5	1.490
2	6.7	300.0	-36.2	32.2	-3.88	27.34	13.4	1.469
4	13.4	299.0	-33.1	32.1	-3.55	26.14	13.1	1.426
5	16.7	306.2	-33.4	32.9	-3.58	26.01	12.9	1.405
6	20.1	286.9	-30.2	30.8	-3.24	25.00	12.7	1.383
7	23.4	282.9	-30.4	30.3	-3.26	24.86	12.5	1.361
8	26.8	276.6	-26.1	29.7	-2.80	23.64	12.3	1.340
9	30.1	281.5	-26.3	30.2	-2.82	23.52	12.1	1.319
10	33.5	276.1	-21.4	29.6	-2.30	22.28	11.9	1.300
11	36.8	289.3	-21.6	31.0	-2.32	22.17	11.6	1.281
12	40.2	284.5	-16.2	30.5	-1.74	20.88	11.4	1.262
13	43.5	273.7	-16.3	29.4	-1.75	20.78	11.0	1.244
14	46.9	242.2	-9.3	26.0	-1.00	19.21	10.7	1.227
15	50.2	243.6	-9.3	26.1	-1.00	19.12	10.3	1.209
16	53.6	222.2	-0.3	23.8	-0.03	16.96	10.0	1.194
17	56.9	231.1	-0.3	24.8	-0.04	16.89	9.7	1.179
18	60.3	212.3	0.0	22.8	0.00	14.38	9.5	1.165
19	63.6	218.4	0.0	23.4	0.00	14.33	9.2	1.152
20	67.0	196.4	0.0	21.1	0.00	11.47	8.8	1.141
21	70.3	190.2	0.0	20.4	0.00	11.43	9.5	1.131
22	73.7	145.8	0.0	15.6	0.00	8.55	9.9	1.122
23	77.0	122.9	0.0	11.0	0.00	2.36	10.3	1.115
Absolute	16.7			32.9			(T =	33.5 ms)
	10.0				-3.91		(T =	130.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	312.4	293.6	274.7	255.9	237.0	218.2	199.4	180.5	161.7	142.8
RX	312.4	293.6	274.7	255.9	237.0	223.7	211.5	199.2	187.0	176.7
RU	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

RAU = 45.2 (kips); RA2 = 261.0 (kips)

Current CAPWAP Ru = 170.0 (kips); Corresponding J(RP)= 0.76; matches RX9 within 5%

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
13.50	26.35	224.1	276.7	299.9	1.422	0.690	0.600	27.3	323.6

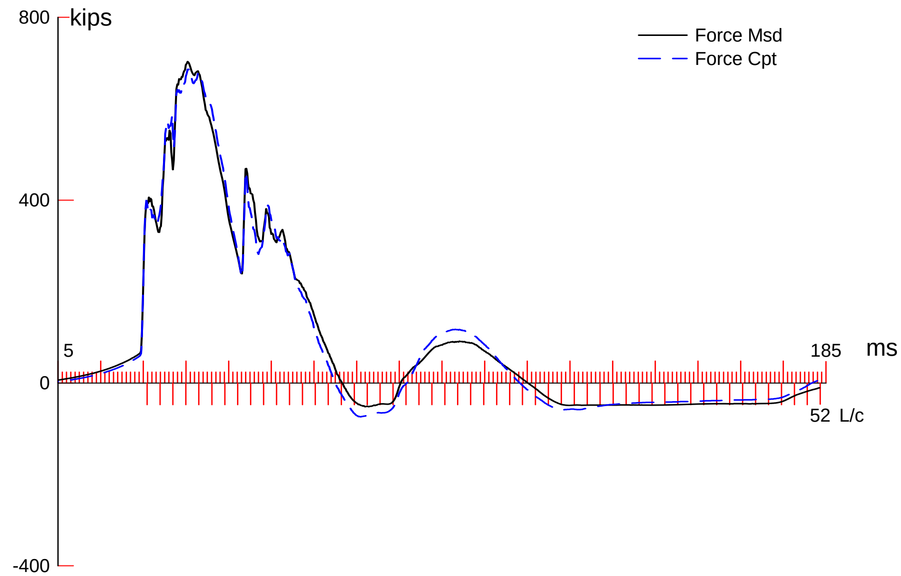
PILE PROFILE AND PILE MODEL

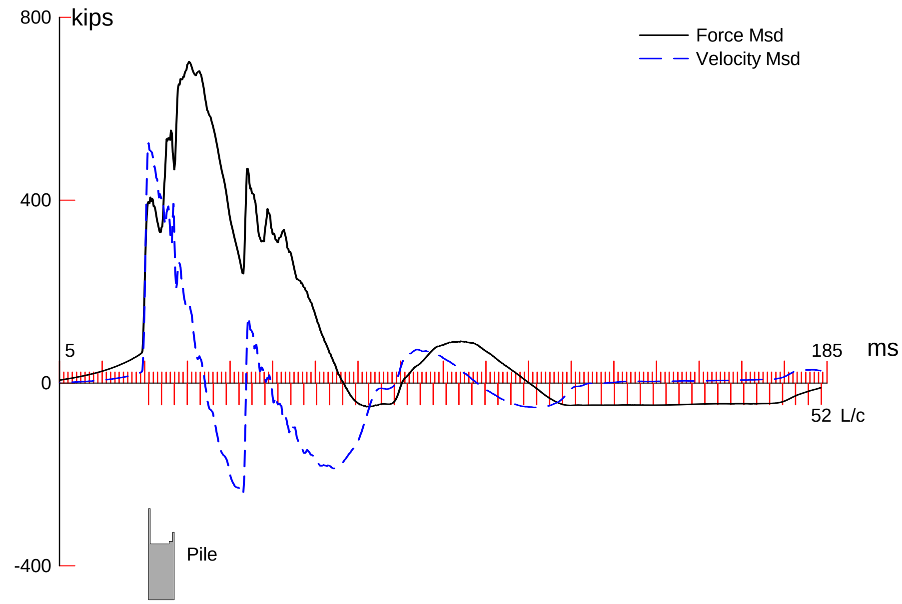
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	9.32	29861.2	492.000	1.833
75.00	9.32	29861.2	492.000	1.833
75.00	15.60	29861.2	492.000	1.833
76.00	15.60	29861.2	492.000	1.833
76.00	9.32	29861.2	492.000	1.833
77.00	9.32	29861.2	492.000	1.833

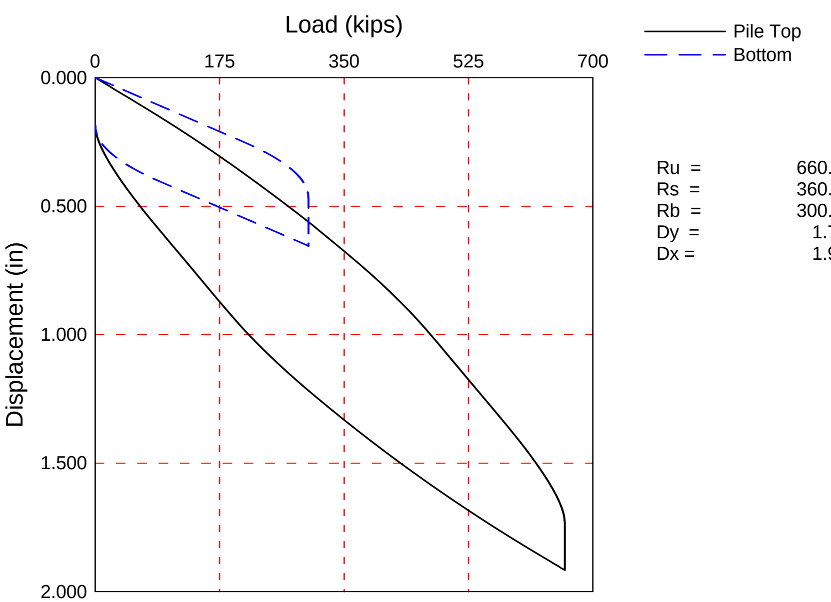
Toe Area 0.267 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.35	16.60	0.00	0.000	0.000	-0.000	0.000	1.833
23	77.00	19.94	0.00	0.000	0.000	-0.000	0.000	1.833

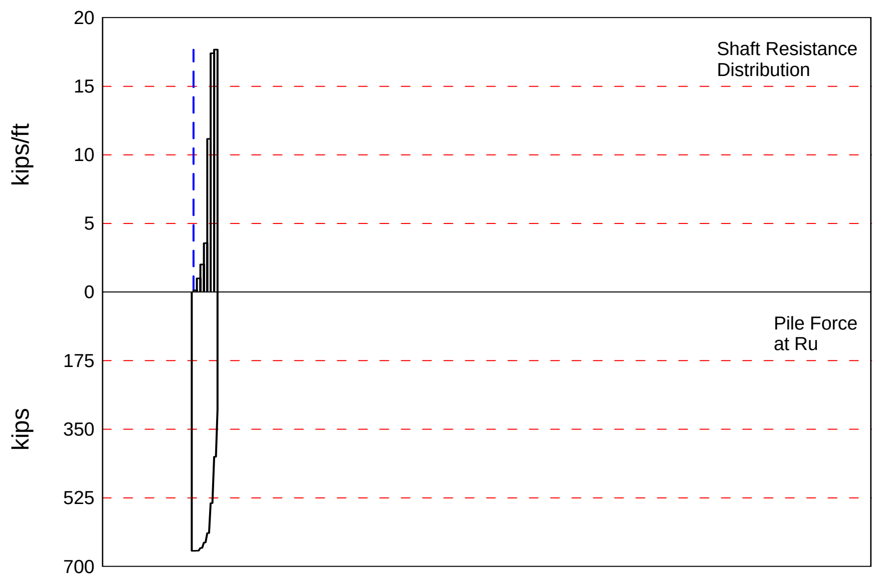
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 16771.1 ft/s, 2L/c 9.2 ms







Ru = 660.4 kips
Rs = 360.3 kips
Rb = 300.1 kips
Dy = 1.73 in
Dx = 1.92 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 660.4; along Shaft 360.3; at Toe 300.1 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				660.4				
1	10.2	6.7	0.9	659.5	0.9	0.13	0.07	0.160
2	17.0	13.5	6.8	652.7	7.7	1.00	0.55	0.160
3	23.8	20.3	13.7	639.0	21.4	2.01	1.10	0.160
4	30.6	27.1	24.3	614.7	45.7	3.57	1.95	0.160
5	37.4	33.9	76.0	538.7	121.7	11.18	6.10	0.160
6	44.2	40.7	118.4	420.3	240.1	17.41	9.50	0.160
7	51.0	47.5	120.2	300.1	360.3	17.68	9.64	0.160
Avg. Shaft			51.5			7.59	4.14	0.160
Toe			300.1				1122.90	0.100

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.360
Case Damping Factor		1.713	0.892
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	20	
Resistance Gap (included in Toe Quake) (in)			0.100
Soil Support Dashpot		4.000	0.000
Soil Support Weight	(kips)	1.30	0.00

max. Top Comp. Stress = 36.8 ksi (T= 36.2 ms, max= 2.088 x Top)
max. Comp. Stress = 76.8 ksi (Z= 23.8 ft, T= 37.2 ms)
max. Tens. Stress = -9.80 ksi (Z= 23.8 ft, T= 77.1 ms)
max. Energy (EMX) = 55.1 kip-ft; max. Measured Top Displ. (DMX)= 1.46 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	693.3	-79.0	36.8	-4.19	55.10	16.0	1.454
2	6.8	698.5	-82.0	75.1	-8.82	52.88	15.9	1.375
3	10.2	703.0	-84.9	75.6	-9.12	50.63	15.7	1.294
4	13.6	708.8	-87.1	76.2	-9.37	48.13	15.4	1.214
5	17.0	714.1	-89.8	76.8	-9.65	45.86	15.0	1.134
6	20.4	710.5	-88.8	76.4	-9.54	42.25	14.4	1.055
7	23.8	714.2	-91.1	76.8	-9.80	40.11	13.8	0.976
8	27.2	700.2	-87.1	75.3	-9.36	35.87	13.0	0.901
9	30.6	703.6	-89.0	75.6	-9.57	33.91	11.8	0.826
10	34.0	676.0	-80.9	72.7	-8.70	29.26	10.3	0.755
11	37.4	679.6	-82.6	73.1	-8.88	27.63	8.6	0.687
12	40.8	604.9	-59.7	65.0	-6.41	21.04	7.2	0.630
13	44.2	606.4	-61.0	65.2	-6.56	20.04	6.4	0.578
14	47.6	508.9	-34.1	54.7	-3.66	13.98	6.4	0.534
15	51.0	509.5	-35.3	45.7	-3.16	8.04	6.1	0.499
Absolute	23.8			76.8			(T =	37.2 ms)
	23.8				-9.80		(T =	77.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	530.4	491.3	452.2	413.1	374.0	334.9	295.7	256.6	217.5	178.4
RX	774.5	760.2	745.8	731.4	717.1	702.7	688.3	676.3	664.5	653.0
RU	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

RAU = 537.3 (kips); RA2 = 746.6 (kips)

Current CAPWAP Ru = 660.4 (kips); Corresponding J(RP)= 0.00; J(RX) = 0.84

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
15.60	26.09	524.8	396.7	703.3	1.465	0.121	0.188	55.6	807.7

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	18.85	29992.2	492.000	1.833
2.50	18.85	29992.2	492.000	1.833
2.50	9.30	29992.2	492.000	1.833
49.00	9.30	29992.2	492.000	1.833
49.00	15.60	29992.2	492.000	1.833

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
50.00	15.60	29992.2	492.000	1.833
50.00	9.30	29992.2	492.000	1.833
51.00	9.30	29992.2	492.000	1.833

Toe Area 0.267 ft^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft
1	3.40	33.65	0.00	0.000	0.000	-0.000	0.000	1.833
2	6.80	20.60	24.10	0.000	0.000	-0.000	0.000	1.833
13	44.20	21.60	30.12	0.000	0.000	-0.000	0.000	1.833
15	51.00	24.91	25.12	0.000	0.000	-0.000	0.000	1.833

Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 16807.9 ft/s, 2L/c 6.1 ms