April 25, 2011

Mr. Joel Halterman
Walsh Construction Company
Cleveland Innerbelt Project
2301 Scranton Road
Cleveland, OH 44113

Re: High Strain Dynamic Load Testing Report
ODOT 3000(10) I-90 WB Innerbelt Project – Pier 9 Testing
Cleveland, OH GRL Job No. 115063-1

Mr. Halterman:

This report summarizes the results from the dynamic pile testing and analyses performed for the above referenced project. Dynamic testing was performed on four HP 18x204 steel H-piles. Three of the piles were tested during initial driving with a Pileco D80-23 diesel hammer and two of the piles were tested with the GRL APPLE IV pile load testing system.

FIELD DETAILS

Monitored Piles

The piles were HP 18x204 steel H-piles approximately 170 ft in total length. The piles were driven without tip reinforcement. We were informed that the piles were fabricated with ASTM A572 Grade 60 steel which has a minimum yield strength of 60 ksi.

Required Bearing Capacity

The plans indicate a factored axial load of 1,747 kips for the Pier 9 piles. We were informed that the required ultimate bearing value to be mobilized through dynamic testing is 1.5 times the factored axial load, or 2,620.5 kips.

Hammer and Driving System

Pileco D80-23

The piles were driven using a Pileco D80-23 open ended diesel hammer. According to the manufacture's literature, this hammer has a ram weight of 18.0 kips and a rated energy of 197.6 kip-ft. At the beginning of driving, the hammer was set on the maximum fuel setting. Near the end of driving, the fuel setting was reduced to fuel setting 3 then 2 to maintain a safe and reasonable stroke since the stroke at times was

near the geometric maximum stroke. The depths where the fuel setting was reduced are shown in Appendix B.

Pileco D80-23 Hammer

APPLE IV Load Testing System

Two piles were tested with the GRL APPLE IV load testing device. This Apple load test device has an 80 kip ram weight and variable drop height. Drop heights used during testing ranged between 0.67 and 7.2 feet.

APPLE IV Load Testing Device

For the first two impacts of pile 166, a helmet was placed over the pile top which consisted of 1.5 inches of steel on top of the pile and 0.75 inch plates on the sides. This helmet was damaged during the second impact (4.0 ft drop). For the remaining impacts, a 0.75 inch thick steel plate was welded to the top of the pile. One sheet of 0.75 inch thick plywood was placed on top of the helmet or steel plate.

Pile Helmet

Plate Welded to Pile

Soil Information

A copy of soil boring B-043-1-10 is included in Appendix E of this report. This soil boring was taken near Pier 9. This boring indicates the overburden soils to be medium to stiff silt and clay to a depth of approximately 105 ft below the bottom of footing elevation. Dense to very dense sand is shown below the clay to a depth of approximately 138 ft depth. An 8 ft layer of hard silt is shown below the dense sand. Severely weathered shale bedrock was indicated for approximately 6 ft. Unweathered to slightly weathered shale bedrock is indicated at approximately 152 ft below the bottom of footing elevation. A detailed soil description is beyond the scope of this report and the interested reader should refer to the proper geotechnical investigation.

Testing Sequence

Pile 178 in the right pile group of Pier 9 was tested during initial driving from approximately 108 ft to 154 ft depth on April 13th, 2011. Pile 164 in the left pile group of

Pier 9 was tested during initial driving from approximately 107 ft to 146.6 ft depth on April 14th, 2011.

GRL returned to the site April 18th, 2011to perform testing using the APPLE IV load testing system. Pile 166 in the left pile group was driven without dynamic testing to a depth of 153.8 ft. Measurements were then collected during 5 impacts using the APPLE load testing system.

Finally, on April 20th 2011, pile 176 was tested during initial driving from approximately 106.5 ft depth to 154.7 ft depth. Measurements were then collected during 3 impacts using the APPLE load testing system.

Instrumentation

Dynamic measurements were obtained with pairs of accelerometers and strain transducers attached at opposite sides of the pile, 36 inches below the pile top.

Wireless Sensors on Pile

Analog signals from the gages were conditioned, digitized, stored and processed with a Model PAX, Pile Driving Analyzer. Selected output from the PDA typically included for each hammer blow among others the following values:

- CSX – the maximum compressive stress at the sensor location, averaged over the pile cross section

- CSB – the calculated compressive stress at the bottom (toe) of the pile, based on pile top measurements
- EMX – the maximum energy transferred to the sensor location at the pile top
- STK – the hammer stroke based on the time between impacts
- RX6 – the Case Method calculated bearing capacity based on a Case damping factor JC = 0.6.

Pile top force and velocity traces were viewed on the PDA's graphic screen during monitoring to evaluate data quality, pile integrity and aspects of soil resistance. Stored dynamic data and the PDA field results were the basis of the analyses presented in this report. A schematic of our equipment setup is shown in Appendix A.

ANALYSIS AND RESULTS

CAPWAP Analysis

CAPWAP analysis was performed on data collected at the end of drive for all three piles tested during initial installation with the diesel hammer. In addition, for piles 178 and 164, another analysis was performed on data collected approximately 2.6 ft prior to reaching refusal (at a blow count of 43 blows/ft) for pile 164 and approximately 1.0 ft prior to reaching refusal (at a blow count of 8 blows/inch) for pile 178.

CAPWAP analysis was performed on data from one impact for each pile tested with the APPLE load testing device. For both piles, the data collected under a 6.0 ft drop was analyzed. The CAPWAP method is described in Appendix A. In general, the method involves a signal matching procedure which yields as a result the shaft resistance and end bearing components in addition to damping factors and soil quakes. CAPWAP results are included in Appendix C and summarized in Table 2.

Discussion of Dynamic Measurement Results

Hammer Performance

Transferred energy calculated from the pile top force and velocity measurements near the end of initial drive when using the Pileco D80-23 hammer ranged from 139.0 to 155.2 kip-ft. These energy transfer levels correspond to energy transfer ratios of 70 to 79% of the rated energy of 197.6 kip-ft. The energy transfer ratio is the measured transferred energy divided by the rated hammer energy. A statistical summary of

transfer ratios for diesel hammers on steel piles at the end of driving is given for comparison in Figure 1. The data in Figure 1 was compiled by GRL Engineers for the end of drive situations and for hammers set to operate at their rated energy levels. Note that the rated energy for the Pileco D80-23 hammer was calculated using a maximum stroke of approximately 11.0 ft. When driving the test piles, the stroke often exceeded this rated stroke and therefore it is expected to have relatively high energy transfer efficiencies.

Pile Stresses and Integrity

The measured compressive stresses, averaged over the cross section at the sensor location at the end on initial installation with the D80-23, ranged between 36.6 and 38.6 ksi at blow counts of 20 blows/inch or higher. CAPWAP analysis evaluates the maximum stress throughout the length of the pile. The CAPWAP results indicated that the maximum stress throughout the length of the piles ranged from 36.8 to 38.9 ksi.

When testing with the APPLE hammer, the maximum measured compressive stresses at the sensor location ranged from 46.2 to 50.2 ksi. CAPWAP computed maximum compressive stresses at other pile locations ranged from 48.3 to 52.4 ksi.

Force and velocity records did not indicate any evidence of detectable pile damage below the sensor location.

Please note that all calculated stresses are averages over the cross section. Non-uniform hammer impacts or a non-uniform resistance at the bottom of the pile may therefore cause additional and potentially damaging stresses. It is always strongly recommended that hammer pile alignment is maintained to avoid high non-uniform stresses.

Pile Capacity

At the end of initial installation with the Pileco D80-23 hammer, CAPWAP analysis mobilized capacities ranged from 1907 to 2050 kips at blow counts in excess of 20 blows/inch. The shaft resistance component ranged from 451 to 780 kips and the end bearing component ranged from 1127 to 1537 kips. The average hammer stroke (averaged over the last inch of driving) ranged from 11.5 to 12.3 ft. At such small pile sets (high blow counts), all of the soil/rock resistance is likely not mobilized, therefore,

the mobilized capacity from dynamic testing is likely a lower bound estimate of the pile capacity at the time of testing.

The APPLE IV load testing device (an 80 kip drop hammer) was used to attempt to mobilize the ultimate capacity at the tip elevation reached with the D80-23 hammer. Pile 166 in the left pile group was reportedly driven to 2 consecutive inches of 20 blows/inch with an average hammer stroke of 11.6 ft over the last two inches of driving (this was measured by GRL onsite using a saximeter). Dynamic testing was not performed on this pile during driving. CAPWAP analysis of data collected under a 6.0 ft drop height indicated a mobilized capacity of 2899 kips with 770 kips in shaft resistance and 2129 kips in end bearing. The measured pile top permanent set under this impact was approximately 0.25 inches, therefore, it is very likely that the full soil and rock resistance was mobilized and the reported capacity can be considered an ultimate pile capacity. The test using the APPLE hammer was performed approximately 3 hours after the completion of driving with the D80-23 hammer.

Pile 176 in the right pile group was also tested using the APPLE load testing system. Data was collected during installation of this pile with the D80-23 hammer. The blow count over the last 2 inches of driving was reportedly 22 and 23 blows/inch with an average hammer stroke of 12.2 ft. CAPWAP analysis indicated a mobilized capacity of 1907 kips with 780 kips in shaft resistance and 1127 kips in end bearing at the end of drive when using the D80-23 hammer. The test using the APPLE load testing system was performed approximately 3.5 hours after the completion of driving with the D80-23 hammer. The data collected during a drop of 6.0 ft height was chosen for CAPWAP analysis since it sustained an observed permanent set of 0.25 inches which is generally considered sufficient to mobilize the ultimate capacity. CAPWAP analysis indicated a total capacity of 2850 kips with 1101 kips in shaft resistance and 1749 kips in end bearing.

Conclusions

The piles at Pier 9 are intended to be driven to refusal into shale bedrock. In an effort to evaluate the bedrock, CAPWAP analysis was performed on data collected at the end of drive and on data collected just before reaching refusal on the apparent bedrock at a blow count of 43 blows/ft for pile 164 and at a blow count of 8 blows/inch for pile 178. The results of these analyses are presented in Table 2. CAPWAP analyses for piles 164 and 178 indicate an increase in end bearing of approximately 679 and 401 kips,

respectively (these increases in end bearing are likely lower bound since the ultimate capacity at the pile toe was likely not fully mobilized at end of drive). These increases in end bearing resistance indicate a significant increase in the strength of the bedrock over the last 1.0 to 2.6 ft of driving. The results from testing with the APPLE hammer on pile 176 indicated an ultimate end bearing resistance of 1749 kips which is 622 kips more than the mobilized end bearing resistance when driving with the D80-23 hammer.

Based on the observed blow counts, CAPWAP results, and the collected force and velocity records, it is very likely that the piles drove several feet into weathered bedrock and reached refusal (20+ blows/inch) in the shale bedrock at the end of driving. Soil boring B-043-1-10 indicates the unweathered to slightly weathered shale to begin at elevation 417 ft (a depth of approximately 152 ft below existing grade); however, because the location of the soil boring is outside the pier footprint, the top of bedrock elevation within the footing may differ from the elevation shown in the boring log.

The pile capacities from data collected when using the APPLE load testing device were 2965 and 2850 kips. These capacities exceed the required capacity of 2620.5 kips (1.5 times the factored axial load). These capacities were measured at the same tip elevations that were reached when driving with the Pileco D80-23 hammer, approximately 3 hours after completion of driving with the D80-23.

Based on the results of the dynamic testing with both the D80-23 hammer and the APPLE IV hammer, GRL suggests that the Pier 9 piles be driven to at least 2 consecutive inches of at least 20 blows/inch with a corresponding minimum average hammer stroke of 11.5 ft with the D80-23 diesel hammer. Dynamic test results indicate that piles driven to these criteria should have ultimate capacities greater than 1.5 times the factored axial load.

LIMITATIONS

Please note the limitations of the analysis and, importantly, the Additional Consideration Section of Appendix A. It is particularly noteworthy that dynamic testing assesses the pile bearing capacity at the time of testing. Time dependent changes of the soil or bedrock cannot be evaluated with end of drive measurements only, and therefore, results obtained at the end of driving, or shortly after, cannot always be used as an indication of the long term bearing capacity of a pile.

We appreciate the opportunity to be of assistance to you on this project. Please contact our office should you have any questions regarding this submittal, or if we may be of further service in any way.

Sincerely,
GRL ENGINEERS, INC.

Benjamin White, P.E.

C. Michael Morgano, P.E.

Table 1: Summary of Case Method Results

ODOT 3000(10) - I-90 Central Viaduct Unit 2

Pile No.	Test Date	Substructure	Test Type [1]	Penetration Depth [2] (ft)	Drop Height (Stroke) (ft)	Observed Set [3] (blow count) (in)	Transf'd Energy (kip-ft)	Max. Compressive Force (kips)	Max. Compressive Stress [4] (ksi)	Case Method Capacity (kips)	CAPWAP Mobilized Capacity (kips)
178	13-Apr-11	Pier 9	EOID	154' - 0"	(12.3)	(20+ / 1")	155.2	2318	38.6	2071	2050
		Right Pile Group									
164	14-Apr-11	Pier 9	EOID	146' - 7"	(11.5)	(25 / 1")	141.3	2195	36.6	1918	1911
		Left Pile Group									
166	18-Apr-11	Pier 9	APPLE	153' - 9"	0.67	n/a	36.8	913	15.2	912	-
		Left Pile Group			4.0	n/a	270.1	2446	40.8	2374	-
					0.67	0.0	16.8	607	10.1	609	-
					2.0	0.0	136.3	1795	29.9	1797	-
					6.0	0.25	471.9	3012	50.2	2998	2899
176	20-Apr-11	Pier 9	EOID	154' - 8"	(12.2)	(23 / 1")	139.0	2209	36.8	1927	1907
		Right Pile Group	APPLE	154' - 8"	1.0	0.0	68.4	1247	20.8	1218	-
					6.0	0.25	467.3	2744	45.7	2757	2850
					7.2	0.5	532.9	2773	46.2	2846	-

Notes:

Information shaded blue was collected while driving with the Pileco D80-23 hammer

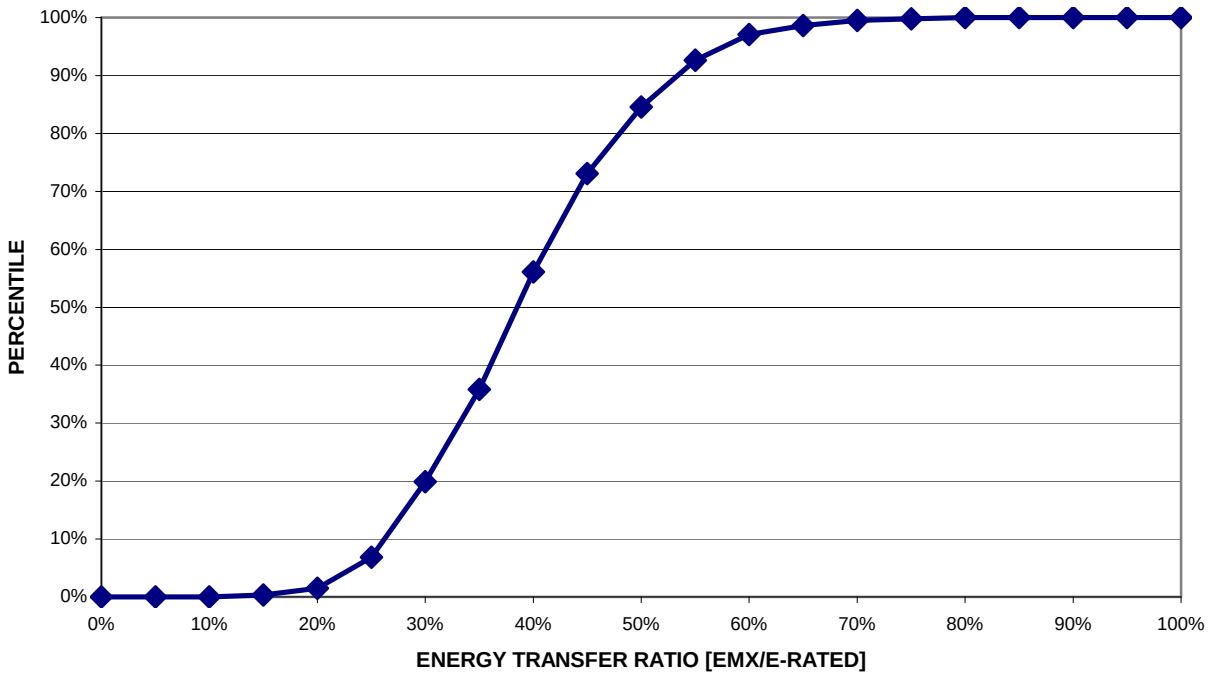
1 - EOID - End of Initial Drive; Tests with APPLE hammer performed approximately 3 hours after driving with production hammer
2 - Depth below existing grade
3 - Set measured with laser projected on pile
4 - Stress from uniform axial average

Table 2: Summary of CAPWAP Results

Pile No.	Test Date	Substructure	Observed Set (blow count	Penetration Depth (ft)	Mobilized Capacity			Soil Damping		Soil Quake	
					Total (kips)	Shaft (kips)	Toe (kips)	Shaft (sec/ft)	Toe (sec/ft)	Shaft (in)	Toe (in)
178	13-Apr-11	Pier 9	(8 / 1")	153.0	1770	635	1135	0.1	0.09	0.30	0.44
			(20+ / 1")	154.0	2050	514	1536	0.15	0.07	0.30	0.30
164	14-Apr-11	Pier 9	(43 / 1')	144.0	1249	468	781	0.15	0.05	0.25	0.52
			(25 / 1")	146.6	1911	451	1460	0.1	0.07	0.3	0.38
166	18-Apr-11	Pier 9	0.25	153.8	2899	770	2129	0.192	0.1	0.22	0.33
176	20-Apr-11	Pier 9	(23 / 1")	154.7	1907	780	1127	0.15	0.07	0.26	0.29
			0.25	154.7	2850	1101	1749	0.18	0.05	0.04	0.39

Results shown shaded blue were from data collected while driving with the Pileco D80-23 hammer

SA DIESEL HAMMERS ON STEEL
N = 1226; MEDIAN = 38.5%



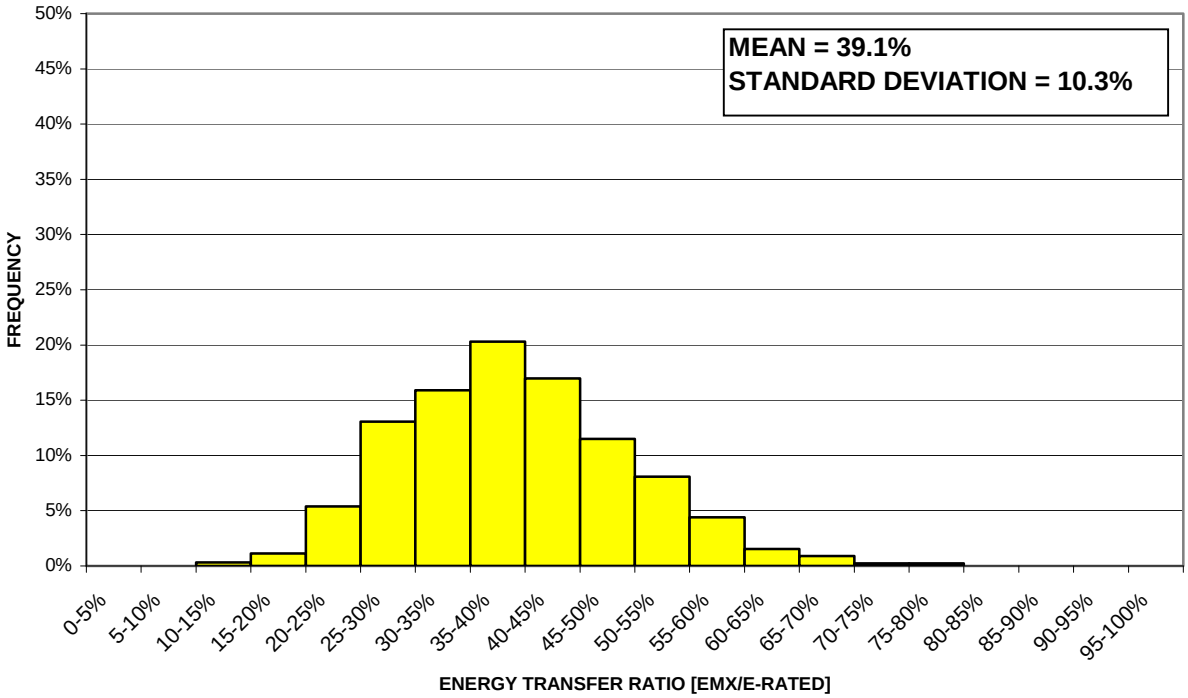


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FIGURE 1

Appendix A

Description of the Dynamic Test Method

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by GRL Engineers, Inc. and may only be copied with its written permission.

1. BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during both preconstruction test programs and 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 (e.g. 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 "High Strain Test" Method of pile testing. However, for the sake of completeness, two types of "Low Strain Tests" are also mentioned: the Pile Integrity Test™ (PIT) and Cross Hole Sonic Logging conducted with the Cross Hole Analyzer (CHA).

2. RESULTS FROM PDA 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 drilled shafts and impact driven piles during restrike.

2.1 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:

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

- *Dynamic pile stresses* 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.

- *Pile integrity* 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.

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

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2.2 DYNAMIC PILE LOAD TESTING

Bearing capacity testing of either driven piles or drilled shafts employs the 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 is 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.)

3. MEASUREMENTS

The following is a general summary of dynamic measurements available to solve typical deep foundation problems.

3.1 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.

3.2 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.

3.3 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.

3.4 PIT

The Pile Integrity Tester™ (PIT) helps in detecting major defects in concrete piles or shafts or assess the length of a variety of deep foundations, except steel piles. PIT performs the so-called “Pulse-Echo Method” which only requires the measurement of motion (e.g., acceleration) at the pile top caused by a light hammer impact. 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. PIT may also be used to evaluate the unknown length of deep foundations under existing structures.

3.5 CHA

This test requires that at least two tubes (typically steel tubes of 50 mm diameter) are installed vertically in the shaft to be tested. A high frequency signal is generated in one of the water filled tubes and received in the other tube. The received signal strength and its First Time of Arrival (FAT) yield important information about the concrete quality between the two tubes. The transmitting and recording of the signal is repeated typically every 50 mm starting at the shaft bottom and all records together establish a log or profile of the concrete quality between the two tubes. The total number of tubes installed depends on the size of the drilled shaft. The more tubes are present the more profiles can be constructed.

4.ANALYTICAL SOLUTIONS

4.1 *BEARING CAPACITY*

4.1.1 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

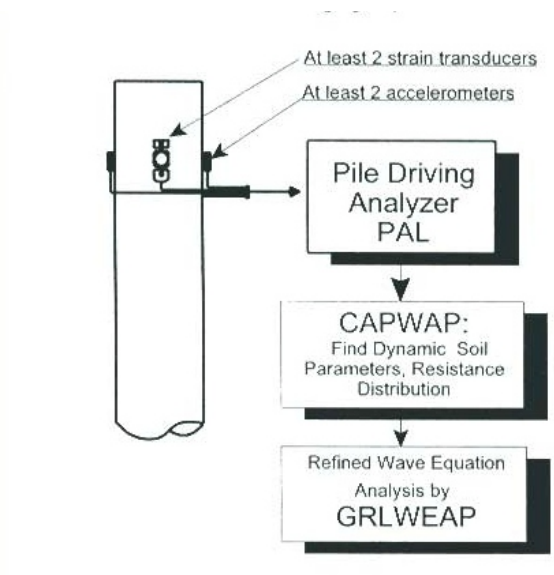


Figure 1. Block Diagram of Refined Wave Equation Analysis

known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements other than blow count. Rather it requires an accurate knowledge of the various parameters describing hammer, driving system, pile and soil. The wave equation is also very useful during the design stage of a project 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 (Figure 1.) is often performed by inputting the PDA and CAPWAP calculated parameters. With many of the dynamic parameters verified by the dynamic tests, it is a more reliable basis for a safe and sufficient driving criterion.

4.1.2 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 (ρc^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 the pile 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 could be evaluated. Most commonly, T 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 for one record. The capacities for other hammer blows are then quickly calculated for the thus selected Case Method and its associated damping factor.

The static resistance calculated by either 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 PDI-PLOT program or formerly in the DOS based PDAPLOT program.

4.1.3 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 stiffness values. 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 based on the dynamic measurements.

4.1.4 Capacity of damaged piles

Occasionally piles are damaged during driving and such damage may be indicated in the PDA collected records, if it occurs below the sensor location. Damage on steel piles is often a broken splice, a collapsed pile bottom section, a ripped of flange on an H-pile or a sharp bend (a gradual dog leg is usually not recognized in the records). For concrete piles, among the problems encountered are cracks, perpendicular due to the pile axis, which deteriorate into a major damage, slabbing (loss of concrete cover) or a compressive failure at the bottom which in effect makes the pile shorter.

Damaged piles, with beta values less than 0.8 should never be evaluated for bearing capacity by the Case Method alone, because these are non-uniform piles which therefore violate the basic premise of the Case Method: a uniform, elastic pile.

Using the CAPWAP program, it is sometimes possible to obtain a reasonable match between computed and measured pile top quantities. In such an analysis the damaged section has to be modeled either by impedance reductions or by slacks. For piles with severe damage along their length it may be necessary to analyze a short pile. It should be born in mind, however, that such an analysis also violates the basic principles of the CAPWAP analysis, namely that the pile is elastic. Also, the nature of the damage is never be known with certainty. For example, a broken splice could be a cracked weld either with the neighboring sections lining up well or shifted laterally. In the former case the stresses would be similar to those in the undamaged pile; in the latter situation, high stress concentrations would develop. A sharp bend or toe damage present equally unpredictable situations under sustained loads which may cause further structural deterioration. If a short pile is analyzed then the lower section of the pile below the damage may offer unreliable end bearing and therefore should be discounted.

It is GRL's position that damaged piling should be replaced. Utilizing the CAPWAP calculated capacities should only be done after a very careful consideration of the effects of a loss of the foundation member while in service. Under no circumstances should the CAPWAP calculated capacity be utilized in the same manner in which the capacity of an undamaged pile be used. Under the best of circumstances the capacity should be used with an increased factor of safety and discounting all questionable capacity components. This evaluation cannot be made by GRL as it involves consideration of the type of structure, its seismic environment, the nature of the loads expected, the corrosiveness of the soil material, considerations of scour on the shortened pile, etc.

4.2 *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, again averaged over the cross section and therefore not including bending stresses, 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.

$$W_u = ½[F(t) - Zv(t)] \quad (4)$$

$$W_d = ½[F(t) + Zv(t)] \quad (5)$$

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.

4.3 *PILE INTEGRITY BY 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 (**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.

$$\beta = (1 - \alpha)/(1 + \alpha) \quad (6)$$

with

$$\alpha = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the damage reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling reflection wave due to the damage.

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.

4.4 *HAMMER PERFORMANCE BY PDA*

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

$$E(t) = \int_0^t F(\tau)v(\tau)\, d\tau \qquad (8a)$$

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

$$e_T = EMX/E_R \qquad (8b)$$

where

E_R is the hammer manufacturer's rated energy value.

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

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

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).

4.5 *DETERMINATION OF WAVE SPEED*

An important facet of dynamic pile testing is an assessment of pile material properties. Since, n most cases 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:

$$c = 2L/T \qquad (10)$$

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

$$E = c^2\rho \qquad (11)$$

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 of the whole pile is lower than the wave speed 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 during the pile installation.

- 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.

5. *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.

5.1 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

$$F = v Z = v (EA/c) \quad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v (E/c) \quad (12b)$$

or strain

$$\epsilon = v / c \quad (12c)$$

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.

5.2 *NUMBER OF SENSORS*

Measurements are always taken at opposite sides of the pile so that the average force and velocity in the pile can be calculated. 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. In that case the averaging of the two strain signals does 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.

6. LIMITATIONS, ADDITIONAL CONSIDERATIONS

6.1 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.

6.2 TIME DEPENDENT SOIL RESISTANCE EFFORTS

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 as a result of soil setup and 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.

6.2.1 SOIL SETUP

Because excess positive pore pressures often develop during pile driving in fine grained soils (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. There are numerous other reasons for soil setup such as realignment of clay particles, arching that reduces effective stresses during pile installation in ver dense sands, soil fatigue in over-consolidated clays etc.

6.2.2 RELAXATION

Relaxation capacity reduction with time has been observed for piles driven into weathered shale, and may take several days to fully develop. Where relaxation occurs, 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 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. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous overpredictions

6.3 CAPACITY RESULTS FOR OPEN PILE PROFILES

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.

6.4 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 signal 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.

6.5 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. The above allowable stresses also apply to those calculated by wave equation.

6.6 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,
- loss of shaft resistance due to scour or other effects,
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads general due not cause buckling even though they may exceed the buckling strength of the pile section), corrosion etc.

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.

6.7 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.

7. FACTORS OF SAFETY

Run to failure, static or dynamic load tests yield an ultimate pile bearing capacity, R_{ult}. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, it is absolutely necessary that the actually applied load, also called the design load, R_d (or working load or safe load), is less than R_{ult}. In most soils, to limit settlements, it is necessary that R_{ult}, is at least 50% higher than R_d. This means that

$$R_{ult} \geq 1.5\ R_d,$$

or the Factor of Safety has to be at least 1.5.

Unfortunately, neither applied loads nor R_{ult} are exactly known. One static load test may be performed at a site, but that would not guarantee that all other piles have the same capacity and it is to be expected that a certain percentage of the production piles have lower capacities, either due to soil variability or due to pile damage. If, for example, dynamic pile tests are performed on piles in shale only a short time after pile installation, then the test capacity may be higher than the long term capacity of the pile. On the other hand, due to soil setup, piles generally gain capacity after installation and since tests are only done a short time after installation, a lower capacity value is ascertained than the capacity that eventually develops.

Not only bearing capacity values of all piles are unknown, even loads vary considerably and occasional overloads must be expected. We would not want a structure to become unserviceable or useless because of either an occasional overload or a few piles with low capacity. For this reason, and to avoid being overly conservative which would mean excessive

cost, modern safety concepts suggest that the overall factor of safety should reflect both the uncertainty in loads and resistance. Thus, if all piles were tested statically and if we carefully controlled the loads, we probably could live with F.S. = 1.5. However, in general, depending on the building type or load combinations and as a function of quality assurance of pile foundations, a variety of Factors of Safety have been proposed.

For example, for highway related loads and based on AASHTO specifications, the Federal Highway Administration proposes the following:

F.S.= 2.00 for static load test with wave equation.

F.S.=2.25 for dynamic testing with wave equation analysis.

F.S.=2.50 for indicator piles with wave equation analysis.

F.S.=2.75 for wave equation analysis.

F.S.=3.00 for Gates or other dynamic formula.

It should be mentioned that all of these methods should always be combined with soil exploration and static pile analysis. Also, specifications of what are occasionally updated and therefore the latest version should be various consulted for the appropriate factors of safety.

Codes, among them PDCA, ASCE, or specifications issued by State Departments of Transportation specify different factors of safety. However, the range of recommended overall factors of safety in the United States varies between 1.9 and 6.

It is the designer's responsibility to identify design loads together with the adopted safety factor concept and associated construction control procedure. The required factors of safety should be included in design drawings or specifications together with the required testing. Only contractors bid for the work and develop the most economical solution. This should include a program of increased testing for lower required pile capacities. This will also help to reduce the confusion that often exists on construction sites as to design loads and require capacities. In any event, it cannot expected that the test engineer is aware of and responsible for the variety of considerations that must be met to find the appropriate factor of safety.

App-A-PDA-9-01

Appendix B

Case Method Results

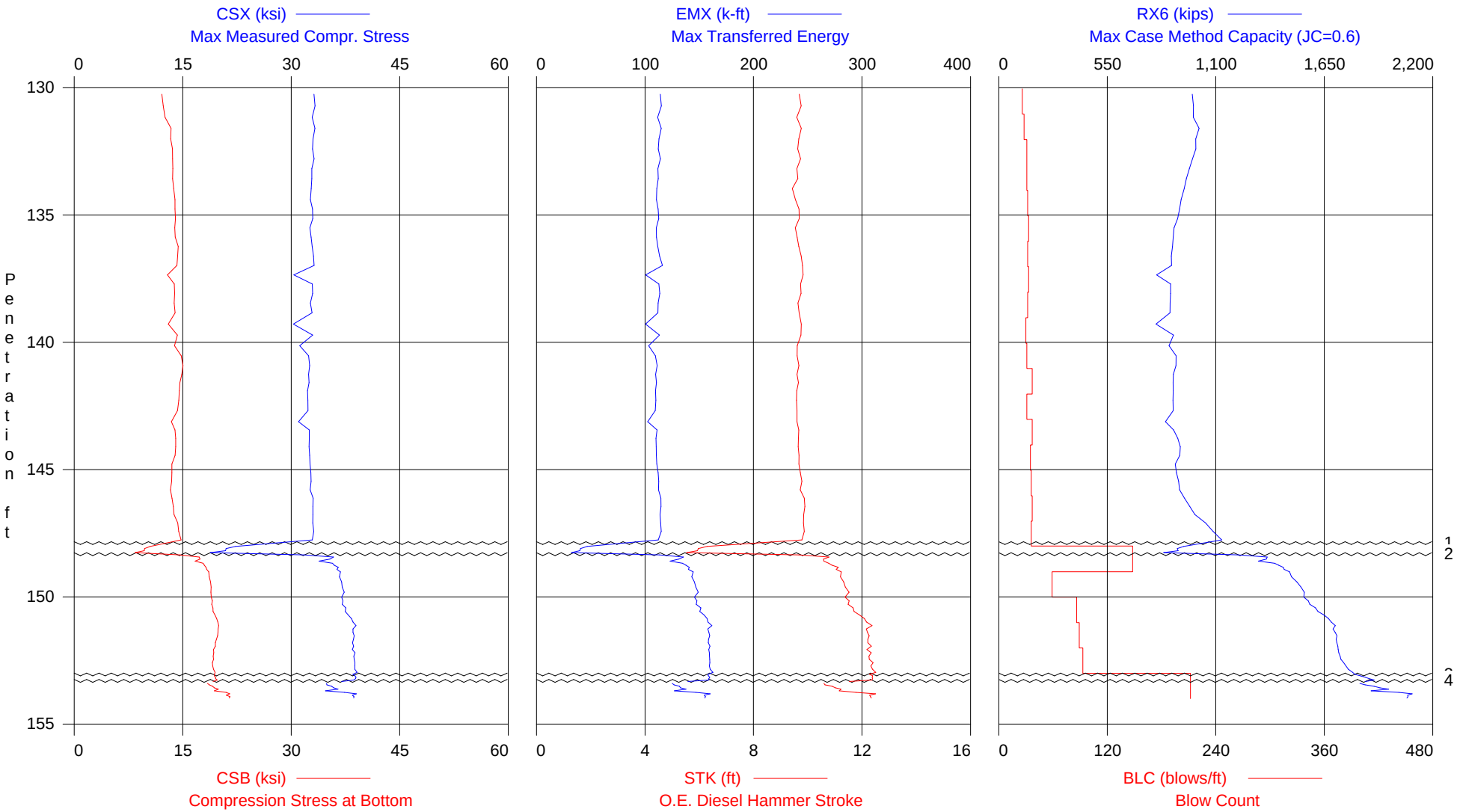
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 14-Apr-2011

Test date: 13-Apr-2011

I-90 Innerbelt CV-2 - Pier 9 Pile 178

HP 18X204



1 - attempt to stop hammer
2 - pause in driving

3 - reduce to fuel setting 3
4 - reduce to fuel setting 2

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OP: BAW

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AR: 60.00 in^2
LE: 167.0 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy

STK: O.E. Diesel Hammer Stroke
FMX: Maximum Force
RX6: Max Case Method Capacity (JC=0.6)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
23	109.00	22	AV22	34.2	34.6	11.0	124.7	10.1	2,053	966
			MAX	38.5	39.2	12.3	160.2	12.0	2,310	1,045
48	110.00	25	AV25	33.6	33.7	11.7	120.8	9.8	2,013	956
			MAX	34.5	34.5	12.5	129.1	10.2	2,071	981
76	111.00	28	AV28	33.0	33.2	11.2	116.1	9.6	1,982	944
			MAX	33.9	34.1	11.7	120.8	9.9	2,036	962
101	112.00	25	AV25	32.9	33.2	10.7	115.3	9.6	1,976	945
			MAX	33.6	33.9	11.2	120.7	9.8	2,015	961
125	113.00	24	AV24	32.9	33.2	10.4	115.3	9.6	1,972	953
			MAX	33.4	33.7	10.8	118.9	9.7	2,002	962
149	114.00	24	AV24	33.0	33.2	10.2	115.7	9.6	1,978	957
			MAX	33.8	34.1	10.6	121.1	10.0	2,029	969
172	115.00	23	AV23	33.1	33.3	10.1	116.1	9.7	1,984	965
			MAX	33.9	34.1	10.7	121.3	10.1	2,033	980
197	116.00	25	AV25	32.9	33.1	10.3	115.0	9.6	1,976	969
			MAX	33.9	34.0	10.7	119.6	10.0	2,036	986
223	117.00	26	AV26	33.1	33.5	10.6	115.4	9.7	1,986	1,002
			MAX	33.8	34.3	11.2	121.9	9.9	2,025	1,020
248	118.00	25	AV25	33.0	33.2	10.5	113.8	9.6	1,977	988
			MAX	33.8	33.9	11.1	119.4	9.9	2,027	1,007
272	119.00	24	AV24	32.6	32.9	10.0	112.2	9.5	1,958	956
			MAX	33.6	33.8	10.5	118.0	9.9	2,017	986
295	120.00	23	AV23	32.8	33.0	9.9	112.9	9.6	1,966	958
			MAX	33.8	34.1	10.3	120.7	10.0	2,030	972
319	121.00	24	AV24	32.8	33.0	9.9	113.2	9.6	1,966	959
			MAX	33.7	34.0	10.4	120.5	9.9	2,022	979
343	122.00	24	AV24	33.0	33.3	10.3	114.9	9.6	1,979	973
			MAX	33.6	34.1	11.1	120.1	9.9	2,014	989
368	123.00	25	AV25	33.1	33.4	10.8	116.3	9.7	1,986	989
			MAX	34.3	34.6	11.5	125.9	10.2	2,056	1,006
394	124.00	26	AV26	33.1	33.4	11.5	115.6	9.7	1,984	1,009
			MAX	33.8	34.3	12.3	121.4	10.1	2,031	1,021
419	125.00	25	AV25	32.9	33.3	11.8	114.3	9.7	1,977	1,005
			MAX	34.1	34.4	12.6	121.7	10.2	2,048	1,029
445	126.00	26	AV26	33.0	33.3	11.7	114.6	9.7	1,977	1,005
			MAX	33.9	34.4	12.4	120.6	10.1	2,035	1,020

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BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
471	127.00	26	AV26	33.2	33.6	11.8	116.9	9.8	1,994	1,005
			MAX	34.3	34.8	12.3	123.8	10.2	2,058	1,020
495	128.00	24	AV24	33.2	33.5	11.8	116.0	9.7	1,991	1,005
			MAX	34.5	34.8	12.7	122.7	10.3	2,073	1,018
521	129.00	26	AV26	33.1	33.3	11.8	115.0	9.7	1,984	989
			MAX	33.8	34.1	12.7	120.5	10.0	2,025	1,006
546	130.00	25	AV25	33.0	33.3	11.8	114.0	9.7	1,982	971
			MAX	34.0	34.5	12.5	121.4	10.1	2,043	986
572	131.00	26	AV26	33.2	33.6	12.3	114.3	9.7	1,990	984
			MAX	34.0	34.4	13.0	119.5	10.1	2,041	997
600	132.00	28	AV28	33.1	33.4	13.1	113.3	9.7	1,985	1,001
			MAX	34.2	34.2	14.1	121.2	10.2	2,050	1,034
631	133.00	31	AV31	33.1	33.2	13.5	113.3	9.7	1,983	992
			MAX	34.0	34.1	14.2	117.7	10.0	2,039	1,010
662	134.00	31	AV31	32.8	33.1	13.7	111.8	9.6	1,970	953
			MAX	33.9	34.3	14.3	118.7	10.1	2,033	977
694	135.00	32	AV30	32.8	33.1	13.9	111.3	9.5	1,968	922
			MAX	33.7	33.9	14.7	117.0	9.9	2,021	946
727	136.00	33	AV33	32.7	32.9	13.9	110.9	9.6	1,963	892
			MAX	34.0	34.2	14.7	120.9	10.1	2,038	915
759	137.00	32	AV32	33.0	33.2	14.3	113.4	9.7	1,983	878
			MAX	34.0	34.5	15.0	120.2	10.1	2,042	898
792	138.00	33	AV33	32.0	33.5	13.5	108.7	9.8	1,919	845
			MAX	34.3	35.3	14.5	120.6	10.2	2,057	892
824	139.00	32	AV31	32.8	33.8	13.9	112.7	9.7	1,971	869
			MAX	34.2	35.0	14.6	120.9	10.3	2,050	885
854	140.00	30	AV28	31.8	34.0	13.7	107.5	9.7	1,907	848
			MAX	33.6	34.8	14.7	119.6	10.0	2,018	909
885	141.00	31	AV30	31.9	33.3	14.5	107.4	9.6	1,916	885
			MAX	33.0	34.1	15.3	115.0	9.9	1,982	919
922	142.00	37	AV37	32.4	33.1	14.7	110.3	9.6	1,944	886
			MAX	33.2	33.9	15.4	117.0	9.9	1,989	906
953	143.00	31	AV28	32.3	32.8	14.3	109.5	9.6	1,936	882
			MAX	32.8	33.3	14.8	113.6	9.8	1,968	906
990	144.00	37	AV37	32.0	33.0	13.8	108.2	9.7	1,922	882
			MAX	33.2	33.6	14.3	113.8	9.9	1,992	922
1025	145.00	35	AV35	32.5	33.0	13.8	110.5	9.7	1,953	910
			MAX	33.1	33.4	14.3	113.3	9.8	1,986	939
1061	146.00	36	AV36	32.7	33.2	13.4	112.6	9.8	1,963	912
			MAX	33.2	33.6	13.9	115.7	9.9	1,992	938
1098	147.00	37	AV37	33.0	33.4	13.7	114.4	9.9	1,979	975
			MAX	33.7	34.1	14.4	118.2	10.2	2,025	1,036

I-90 Innerbelt CV-2 - Pier 9 Pile 178 HP 18X204
OP: BAW Test date: 13-Apr-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
1134	148.00	36	AV36	32.2	32.6	14.2	108.6	9.6	1,931	1,082
			MAX	35.4	35.7	16.3	122.4	10.7	2,122	1,190
1282	149.00	148	AV147	30.8	31.4	14.9	104.0	9.2	1,845	1,227
			MAX	40.8	41.0	19.0	175.5	13.3	2,446	1,480
1341	150.00	59	AV59	37.0	37.8	18.8	146.1	11.4	2,220	1,517
			MAX	38.5	39.1	19.6	157.4	12.2	2,313	1,589
1427	151.00	86	AV86	37.6	38.6	19.3	151.4	11.7	2,255	1,615
			MAX	40.1	40.7	20.4	169.7	12.8	2,404	1,714
1516	152.00	89	AV89	38.6	39.7	19.8	159.2	12.3	2,318	1,708
			MAX	40.9	41.6	21.1	176.1	13.4	2,455	1,763
1609	153.00	93	AV93	38.7	39.7	19.2	159.7	12.3	2,325	1,746
			MAX	41.3	42.1	20.2	178.6	13.6	2,477	1,810
1799	153.90	212	AV184	37.2	38.7	19.7	145.4	11.7	2,235	1,913
			MAX	40.6	42.5	22.1	171.1	13.2	2,435	2,119
1821	154.00	212	AV22	38.6	39.9	21.5	155.2	12.3	2,318	2,071
			MAX	39.5	40.8	22.3	159.8	12.7	2,368	2,103

BL#	depth (ft)	Comments
1130	147.89	attempt to stop hammer
1182	148.32	pause in driving
1620	153.05	reduce to fuel setting 3
1673	153.30	reduce to fuel setting 2

Time Summary

Drive	30 minutes 54 seconds	1:59:24 PM - 2:30:18 PM (4/13/2011) BN 1 - 1182
Stop	7 minutes 16 seconds	2:30:18 PM - 2:37:34 PM
Drive	18 minutes 24 seconds	2:37:34 PM - 2:55:58 PM BN 1183 - 1822

Total time [0:56:34] = (Driving [0:49:18] + Stop [0:07:16])

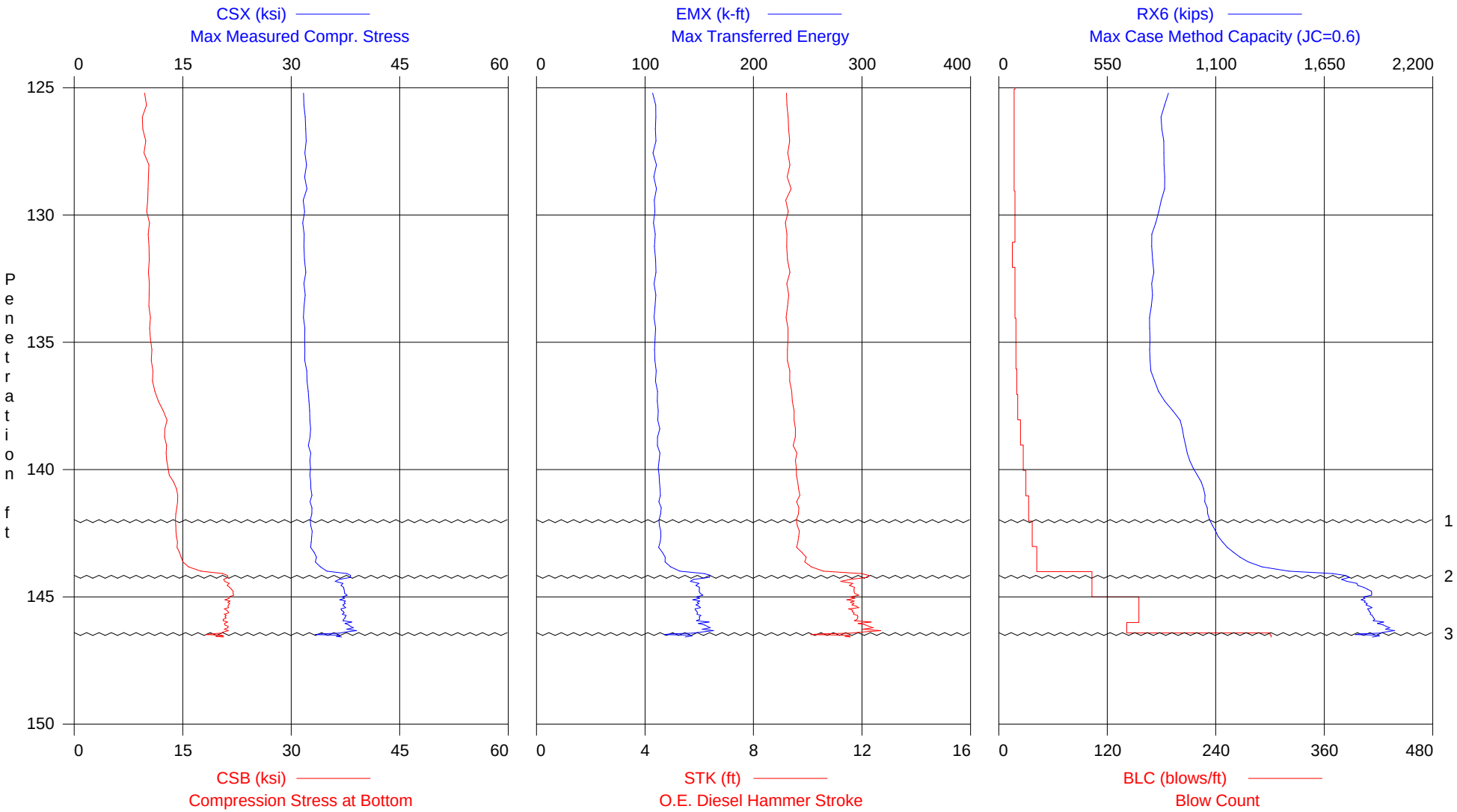
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 14-Apr-2011

Test date: 14-Apr-2011

I-90 Innerbelt CV-2 - Pier 9 Pile 164

HP 18X204



1 - reduce to fuel setting 3

2 - reduce to fuel setting 3

3 - reduce to fuel setting 2

I-90 Innerbelt CV-2 - Pier 9 Pile 164
OP: BAW

HP 18X204
Test date: 14-Apr-2011

AR: 60.00 in^2
LE: 167.0 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy

STK: O.E. Diesel Hammer Stroke
FMX: Maximum Force
RX6: Max Case Method Capacity (JC=0.6)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
14	108.00	13	AV14	31.8	32.3	10.6	113.0	10.6	1,909	791
			MAX	39.1	39.8	13.4	177.0	12.4	2,348	893
33	109.00	19	AV19	33.0	33.4	11.5	120.0	9.7	1,979	760
			MAX	33.9	34.5	12.4	126.8	10.1	2,033	792
45	110.00	13	AV12	28.7	28.7	10.2	95.7	9.1	1,721	671
			MAX	32.3	32.3	12.2	109.4	9.5	1,937	766
58	111.00	13	AV13	30.9	30.9	11.6	101.4	8.9	1,851	695
			MAX	31.6	31.6	12.3	110.8	9.3	1,895	726
71	112.00	13	AV13	30.6	30.6	11.8	99.1	8.8	1,833	738
			MAX	31.4	31.4	12.6	102.0	9.2	1,882	762
84	113.00	13	AV13	31.2	31.2	12.1	102.4	9.1	1,874	762
			MAX	32.1	32.1	13.2	108.7	9.5	1,927	786
87	114.00	4	AV3	30.7	30.7	11.7	96.5	8.9	1,840	747
			MAX	30.8	30.8	12.1	97.2	9.0	1,848	760
91	115.00	4	AV4	31.0	31.0	12.2	102.1	9.0	1,857	745
			MAX	31.4	31.4	12.9	103.4	9.2	1,887	776
95	116.00	4	AV4	30.9	30.9	11.8	100.0	9.0	1,856	751
			MAX	31.2	31.2	12.7	101.1	9.2	1,871	769
99	117.00	4	AV4	31.0	31.0	11.9	101.5	9.1	1,857	757
			MAX	31.3	31.3	12.5	105.0	9.2	1,878	760
110	118.00	18	AV11	32.0	32.5	11.1	109.2	9.3	1,918	808
			MAX	32.6	33.2	12.3	115.1	9.6	1,957	826
126	119.00	16	AV16	31.8	32.5	10.7	110.0	9.2	1,909	817
			MAX	32.6	33.3	11.4	117.1	9.6	1,956	829
143	120.00	17	AV17	31.5	32.0	10.6	106.8	9.1	1,888	827
			MAX	32.9	33.4	11.5	112.5	9.7	1,973	843
162	121.00	19	AV19	31.5	32.1	10.6	107.8	9.2	1,891	832
			MAX	32.4	32.9	11.0	112.9	9.5	1,946	850
178	122.00	16	AV16	31.7	32.0	9.9	107.2	9.2	1,901	822
			MAX	32.4	32.7	10.4	112.5	9.5	1,946	840
195	123.00	17	AV17	31.7	31.7	10.0	108.2	9.2	1,900	813
			MAX	32.3	32.3	10.4	113.1	9.4	1,936	844
212	124.00	17	AV17	31.9	32.0	9.8	108.6	9.3	1,914	827
			MAX	32.8	32.8	10.9	117.3	9.6	1,965	848
230	125.00	18	AV18	31.8	31.9	9.9	107.5	9.2	1,906	845
			MAX	33.0	33.1	10.9	117.7	9.7	1,977	863

I-90 Innerbelt CV-2 - Pier 9 Pile 164
OP: BAW

HP 18X204
Test date: 14-Apr-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
247	126.00	17	AV17	31.7	31.9	9.8	108.6	9.2	1,905	848
			MAX	32.3	32.3	10.5	113.5	9.4	1,936	869
264	127.00	17	AV17	31.9	32.1	9.6	109.5	9.3	1,916	825
			MAX	32.5	32.8	10.2	114.9	9.5	1,950	856
281	128.00	17	AV17	31.9	32.1	9.8	108.4	9.3	1,915	836
			MAX	32.6	32.7	10.8	115.6	9.6	1,958	852
298	129.00	17	AV17	32.1	32.2	10.3	110.3	9.3	1,926	841
			MAX	32.7	33.1	11.2	116.6	9.6	1,962	856
316	130.00	18	AV18	31.8	31.9	10.1	109.0	9.3	1,908	822
			MAX	32.6	32.7	11.0	112.8	9.6	1,957	841
334	131.00	18	AV18	31.7	31.8	10.3	108.7	9.2	1,902	787
			MAX	32.5	32.6	11.1	114.1	9.5	1,951	808
349	132.00	15	AV15	31.8	32.0	10.4	109.4	9.2	1,908	777
			MAX	33.1	33.4	11.4	118.9	9.8	1,988	792
367	133.00	18	AV18	31.9	32.1	10.3	109.5	9.3	1,915	781
			MAX	32.9	33.1	11.6	116.0	9.7	1,976	795
385	134.00	18	AV18	31.8	32.1	10.4	109.0	9.2	1,906	773
			MAX	33.1	33.3	11.2	117.8	9.8	1,988	786
404	135.00	19	AV19	31.9	32.4	10.5	109.5	9.3	1,912	766
			MAX	32.8	33.3	11.3	113.6	9.7	1,967	787
423	136.00	19	AV19	31.9	32.8	10.8	109.1	9.3	1,915	765
			MAX	32.9	34.0	11.7	114.2	9.6	1,972	786
443	137.00	20	AV20	32.2	33.5	10.9	110.4	9.4	1,934	790
			MAX	34.1	35.3	11.8	123.2	10.1	2,048	819
464	138.00	21	AV21	32.5	33.6	12.0	111.7	9.5	1,949	865
			MAX	33.9	35.0	13.1	119.2	10.0	2,032	918
488	139.00	24	AV24	32.6	33.7	12.6	112.2	9.5	1,955	933
			MAX	33.4	34.3	13.2	117.3	9.9	2,002	950
515	140.00	27	AV27	32.6	33.1	12.8	112.8	9.6	1,957	965
			MAX	33.9	34.3	13.2	118.8	10.1	2,032	1,008
545	141.00	30	AV30	32.7	33.0	13.7	113.2	9.6	1,959	1,023
			MAX	33.7	34.4	14.9	120.6	10.1	2,022	1,050
578	142.00	33	AV33	32.8	33.2	14.2	114.0	9.6	1,967	1,055
			MAX	33.7	34.1	14.8	121.4	9.9	2,021	1,072
615	143.00	37	AV37	32.8	33.7	14.1	113.9	9.6	1,966	1,107
			MAX	33.8	34.8	14.6	122.0	10.1	2,029	1,149
657	144.00	42	AV42	33.5	34.3	15.1	119.4	9.9	2,011	1,256
			MAX	35.5	36.4	17.1	135.6	10.7	2,127	1,437
760	145.00	103	AV103	37.2	37.9	21.2	150.2	11.7	2,233	1,802
			MAX	39.7	40.5	23.3	171.8	12.9	2,381	1,923
915	146.00	155	AV155	37.3	38.2	21.1	148.9	11.7	2,236	1,878
			MAX	39.7	40.9	22.8	170.2	13.0	2,379	2,001

I-90 Innerbelt CV-2 - Pier 9 Pile 164 HP 18X204
OP: BAW Test date: 14-Apr-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
974	146.42	142	AV59	38.0	38.5	21.0	155.7	12.2	2,283	1,962
			MAX	40.7	41.2	22.8	177.2	13.6	2,442	2,074
999	146.50	300	AV25	34.7	34.9	19.2	128.8	10.7	2,084	1,858
			MAX	37.4	37.4	20.9	150.7	11.9	2,245	1,959
1024	146.58	301	AV25	36.6	36.8	20.3	141.3	11.5	2,195	1,918
			MAX	37.9	38.1	21.2	149.7	11.9	2,271	1,952

BL#	depth (ft)	Comments
100	117.24	stop to reconnect with radios
579	142.03	reduce to fuel setting 3
679	144.21	reduce to fuel setting 3
988	146.46	reduce to fuel setting 2

Time Summary

Drive 36 minutes 36 seconds 9:41:45 AM - 10:18:21 AM (4/14/2011) BN 1 - 1024

I-90 Innerbelt Project - Pier 9 #2 Pile 166
OP: BAW

HP 18X204 APPLE 4
Test date: 18-Apr-2011

AR: 60.00 in^2
LE: 153.5 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy

USR: User observation
FMX: Maximum Force
DMX: Maximum Displacement
RX6: Max Case Method Capacity (JC=0.6)

BL#	BLC	CSX	CSI	CSB	EMX	USR	FMX	DMX	RX6
	bl/ft	ksi	ksi	ksi	k-ft	[]	kips	in	kips
1	0	15.2	15.5	8.1	36.8	0.7	913	0.9	912
2	0	40.8	41.4	14.0	270.1	4.0	2,446	2.5	2,374
3	0	10.1	10.5	8.1	16.8	0.7	607	0.6	609
4	0	29.9	30.2	10.4	136.3	2.0	1,795	1.7	1,797
5	0	50.2	50.8	11.7	471.9	6.0	3,012	3.2	2,998

Total number of blows analyzed: 5

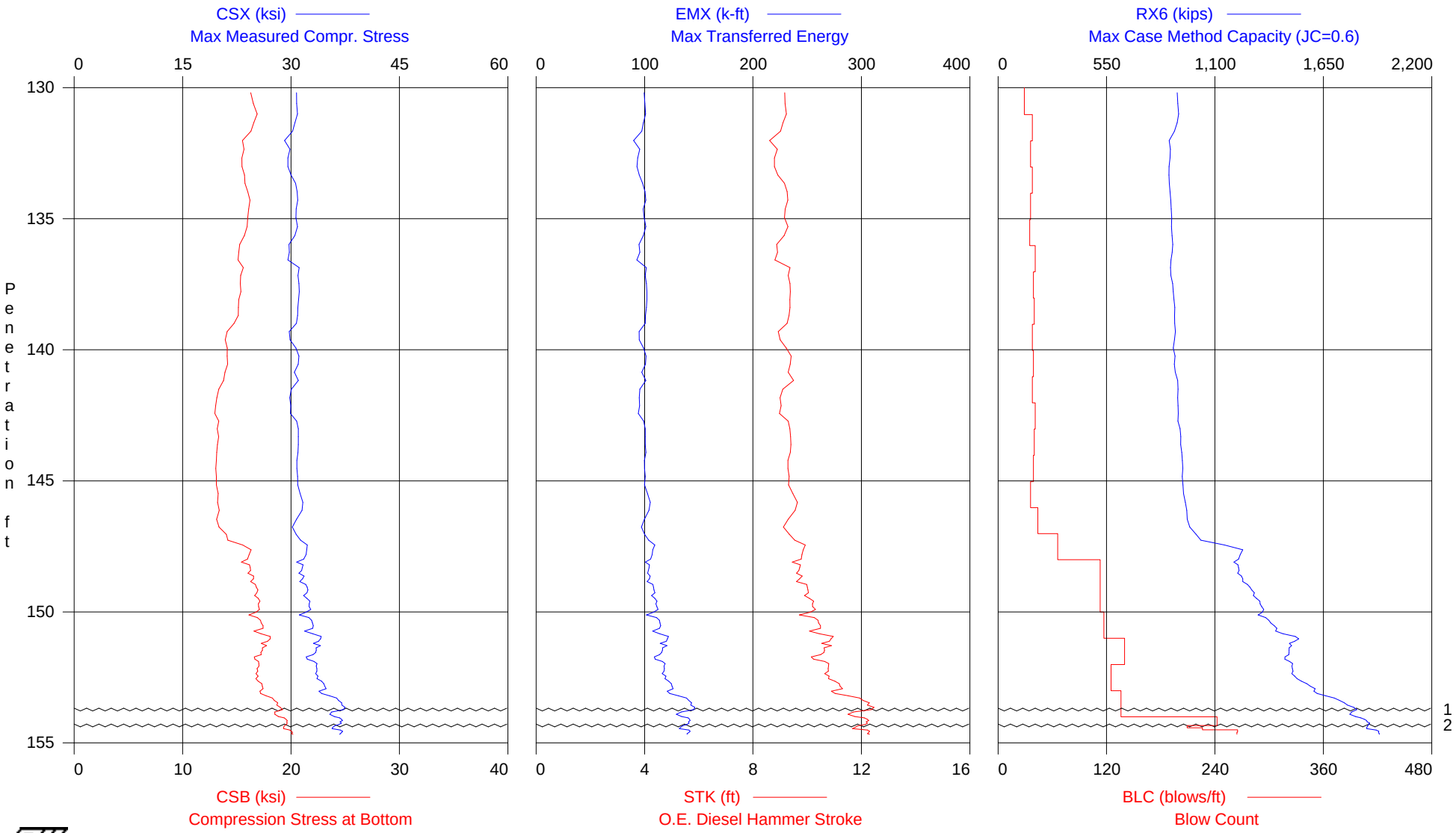
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 21-Apr-2011

Test date: 20-Apr-2011

I-90 Innerbelt Project - Pier 9 Pile 176

HP 18x204



1 - reduce to fuel setting 3

2 - reduce to fuel setting 2

I-90 Innerbelt Project - Pier 9 Pile 176
OP: BAW

HP 18x204
Test date: 20-Apr-2011

AR: 60.00 in^2
LE: 168.0 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy

STK: O.E. Diesel Hammer Stroke
FMX: Maximum Force
RX6: Max Case Method Capacity (JC=0.6)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
12	107.00	23	AV11	30.9	31.1	12.7	103.2	9.2	1,852	777
			MAX	33.5	33.6	13.7	119.7	10.3	2,007	809
35	108.00	23	AV23	30.4	31.5	12.9	98.4	8.9	1,827	731
			MAX	31.3	32.6	13.9	103.5	9.1	1,879	754
58	109.00	23	AV22	29.7	30.9	12.1	92.8	8.6	1,782	693
			MAX	31.0	32.5	13.1	100.9	9.0	1,862	708
82	110.00	23	AV24	29.1	30.1	12.7	90.1	8.5	1,748	696
			MAX	29.8	30.9	13.6	94.9	8.7	1,786	711
107	111.00	25	AV24	29.6	30.7	12.9	92.2	8.6	1,774	708
			MAX	31.1	32.0	13.5	99.8	9.2	1,867	723
132	112.00	25	AV25	29.4	30.4	12.5	91.7	8.6	1,763	712
			MAX	30.1	31.2	13.4	96.1	8.8	1,805	732
158	113.00	25	AV22	29.2	30.4	12.3	88.3	8.5	1,753	717
			MAX	30.0	31.3	13.0	93.5	8.8	1,800	727
183	114.00	25	AV20	29.2	30.3	11.9	89.1	8.5	1,752	728
			MAX	30.2	31.6	12.6	93.7	8.7	1,814	743
209	115.00	25	AV26	29.2	30.5	11.6	89.8	8.6	1,754	737
			MAX	30.3	31.9	12.9	96.5	8.9	1,819	750
234	116.00	25	AV25	29.4	30.6	11.6	90.1	8.6	1,762	753
			MAX	30.5	32.1	12.7	95.2	8.8	1,828	764
259	117.00	25	AV24	29.8	31.3	12.0	93.0	8.8	1,788	784
			MAX	30.9	33.0	13.4	99.3	9.1	1,852	801
285	118.00	25	AV26	29.8	31.8	12.4	92.7	8.8	1,787	794
			MAX	31.0	33.8	13.4	100.7	9.1	1,857	808
310	119.00	25	AV25	30.1	32.0	12.5	93.9	8.9	1,803	796
			MAX	31.1	33.6	13.7	102.3	9.2	1,867	821
336	120.00	25	AV26	30.6	32.2	12.5	97.1	9.0	1,836	803
			MAX	31.7	34.0	14.4	103.1	9.3	1,904	826
364	121.00	28	AV28	29.9	31.2	12.5	93.3	8.9	1,796	812
			MAX	31.6	33.6	14.5	101.8	9.4	1,894	829
392	122.00	28	AV28	30.0	31.1	13.4	95.5	9.0	1,802	847
			MAX	31.3	33.1	14.6	101.7	9.2	1,878	867
420	123.00	28	AV28	30.0	30.4	13.2	94.9	9.0	1,798	847
			MAX	31.2	31.7	14.8	99.5	9.2	1,871	864
448	124.00	28	AV28	30.1	30.3	13.4	95.5	9.0	1,806	855
			MAX	30.6	30.9	14.7	98.7	9.2	1,838	870

I-90 Innerbelt Project - Pier 9 Pile 176
OP: BAW

HP 18x204
Test date: 20-Apr-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
476	125.00	28	AV28	30.0	30.2	13.3	95.5	9.0	1,801	846
			MAX	31.3	31.5	14.5	104.4	9.4	1,879	865
502	126.00	26	AV26	30.2	30.9	13.7	95.7	9.0	1,809	842
			MAX	30.9	31.8	14.8	99.8	9.2	1,852	856
529	127.00	27	AV27	30.4	31.4	13.8	97.3	9.1	1,822	856
			MAX	31.0	32.2	14.8	101.9	9.4	1,860	871
558	128.00	29	AV29	30.5	31.6	14.0	97.2	9.1	1,828	866
			MAX	31.3	32.5	15.4	101.8	9.4	1,878	889
587	129.00	29	AV29	30.7	31.8	15.1	98.8	9.2	1,841	890
			MAX	31.4	32.5	16.2	105.2	9.5	1,887	901
616	130.00	29	AV29	30.8	32.0	15.8	100.2	9.2	1,849	909
			MAX	31.7	33.0	16.9	104.7	9.5	1,900	925
645	131.00	29	AV29	30.8	32.0	16.5	100.3	9.2	1,848	911
			MAX	31.5	32.8	17.4	105.6	9.5	1,890	926
683	132.00	38	AV36	30.3	31.4	16.4	97.3	9.0	1,816	899
			MAX	31.4	32.5	17.0	104.5	9.4	1,885	924
719	133.00	36	AV36	29.5	30.5	15.5	93.4	8.8	1,772	870
			MAX	30.6	31.8	16.1	100.7	9.2	1,838	882
757	134.00	38	AV38	30.3	31.3	15.8	96.7	9.0	1,818	868
			MAX	31.9	33.1	16.7	105.1	9.6	1,912	883
793	135.00	36	AV36	30.8	31.8	16.1	100.1	9.2	1,850	878
			MAX	31.9	32.9	16.9	106.3	9.6	1,912	891
828	136.00	35	AV35	30.5	31.5	15.7	98.9	9.1	1,831	882
			MAX	31.6	32.8	16.5	104.9	9.5	1,898	889
869	137.00	41	AV41	30.1	31.0	15.3	96.5	9.0	1,805	879
			MAX	32.3	33.4	16.5	109.6	9.8	1,940	893
908	138.00	39	AV39	31.1	32.0	15.3	101.6	9.4	1,864	884
			MAX	31.9	32.9	16.0	108.5	9.7	1,915	903
948	139.00	40	AV40	30.9	31.9	15.1	101.4	9.3	1,855	895
			MAX	32.1	33.2	15.8	109.4	9.8	1,926	910
986	140.00	38	AV38	30.1	31.0	14.1	96.6	9.1	1,807	895
			MAX	32.0	32.9	15.2	107.3	9.7	1,918	913
1025	141.00	39	AV39	30.8	31.5	14.1	99.9	9.4	1,851	896
			MAX	32.1	32.9	14.6	108.4	9.9	1,928	909
1063	142.00	38	AV37	30.3	31.2	13.4	97.4	9.2	1,817	912
			MAX	32.7	33.7	14.6	111.6	10.1	1,963	924
1104	143.00	41	AV41	30.3	31.4	13.1	96.7	9.1	1,819	914
			MAX	31.5	32.7	13.8	103.8	9.6	1,892	929
1144	144.00	40	AV40	31.0	32.2	13.2	100.9	9.4	1,861	928
			MAX	31.7	33.0	13.9	105.7	9.7	1,904	948
1183	145.00	39	AV39	30.9	32.2	13.1	100.2	9.3	1,851	935
			MAX	31.7	33.1	14.2	105.1	9.6	1,900	951

I-90 Innerbelt Project - Pier 9 Pile 176
OP: BAW

HP 18x204
Test date: 20-Apr-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	STK ft	FMX kips	RX6 kips
1219	146.00	36	AV36	31.3	32.7	13.2	102.7	9.5	1,877	943
			MAX	32.4	33.9	13.7	109.5	9.9	1,941	956
1263	147.00	44	AV40	30.7	32.1	13.3	99.7	9.3	1,842	967
			MAX	32.3	33.8	14.5	109.2	9.9	1,937	994
1329	148.00	66	AV62	31.9	33.4	15.5	106.4	9.8	1,912	1,152
			MAX	33.1	35.4	17.0	113.6	10.3	1,987	1,264
1442	149.00	113	AV113	31.5	33.0	16.2	104.0	9.7	1,888	1,227
			MAX	33.6	35.2	17.5	120.1	10.6	2,017	1,291
1555	150.00	113	AV113	32.3	33.9	16.9	109.8	10.1	1,941	1,314
			MAX	33.8	35.4	18.0	119.7	10.8	2,025	1,371
1672	151.00	117	AV117	32.7	34.7	17.1	112.2	10.3	1,961	1,399
			MAX	35.2	38.1	18.6	128.8	11.4	2,113	1,536
1812	152.00	140	AV140	33.3	35.7	17.3	116.1	10.6	1,998	1,481
			MAX	37.1	39.6	19.2	135.7	12.3	2,226	1,553
1937	153.00	125	AV125	34.0	36.0	17.0	120.8	10.9	2,038	1,530
			MAX	36.3	38.7	18.4	137.1	11.9	2,176	1,637
2073	154.00	136	AV136	36.0	37.9	18.3	136.0	11.8	2,161	1,738
			MAX	38.4	40.4	19.8	153.4	12.9	2,307	1,840
2155	154.34	209	AV82	36.8	38.4	19.5	140.0	12.1	2,206	1,864
			MAX	38.8	40.6	20.4	152.7	13.2	2,329	1,927
2172	154.42	226	AV17	35.7	37.4	19.2	133.1	11.8	2,145	1,869
			MAX	36.7	38.3	19.6	141.1	12.2	2,202	1,904
2191	154.51	265	AV19	36.1	37.8	19.5	135.8	11.9	2,167	1,887
			MAX	38.9	40.6	21.1	153.6	13.2	2,335	1,968
2213	154.59	264	AV22	37.1	38.7	20.1	142.2	12.3	2,226	1,931
			MAX	38.3	39.9	20.8	149.7	12.8	2,297	1,976
2235	154.67	264	AV22	36.8	38.4	20.1	139.0	12.2	2,209	1,927
			MAX	38.0	39.6	20.7	148.2	12.7	2,279	1,958

BL#	depth (ft)	Comments
2036	153.73	reduce to fuel setting 3
2154	154.33	reduce to fuel setting 2

Time Summary

Drive 58 minutes 50 seconds 10:35:06 AM - 11:33:56 AM (4/20/2011) BN 1 - 2235

I-90 Innerbelt Project - Pier 9 Pile 176
OP: BAW

HP 18X204 APPLE
Test date: 20-Apr-2011

AR: 60.00 in^2
LE: 156.6 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy

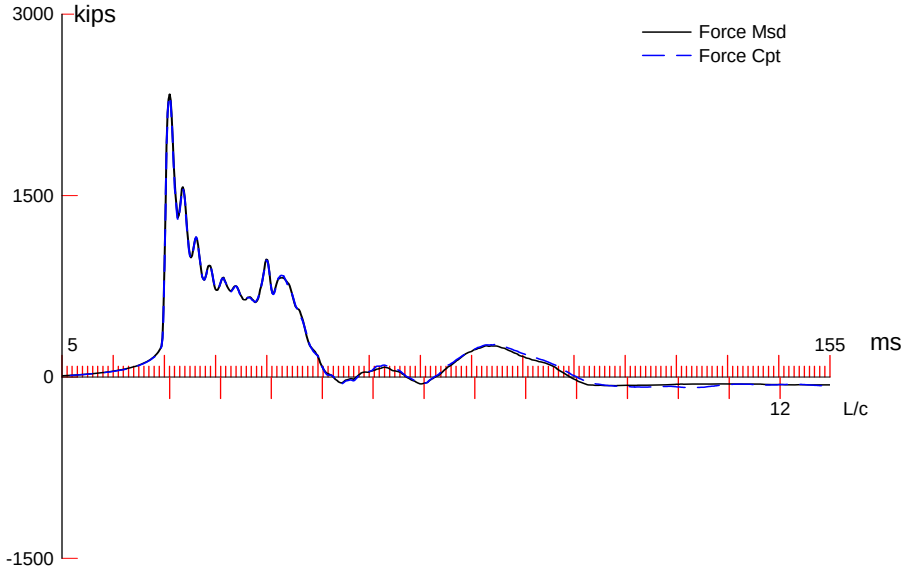
USR: User observation
FMX: Maximum Force
RX6: Max Case Method Capacity (JC=0.6)

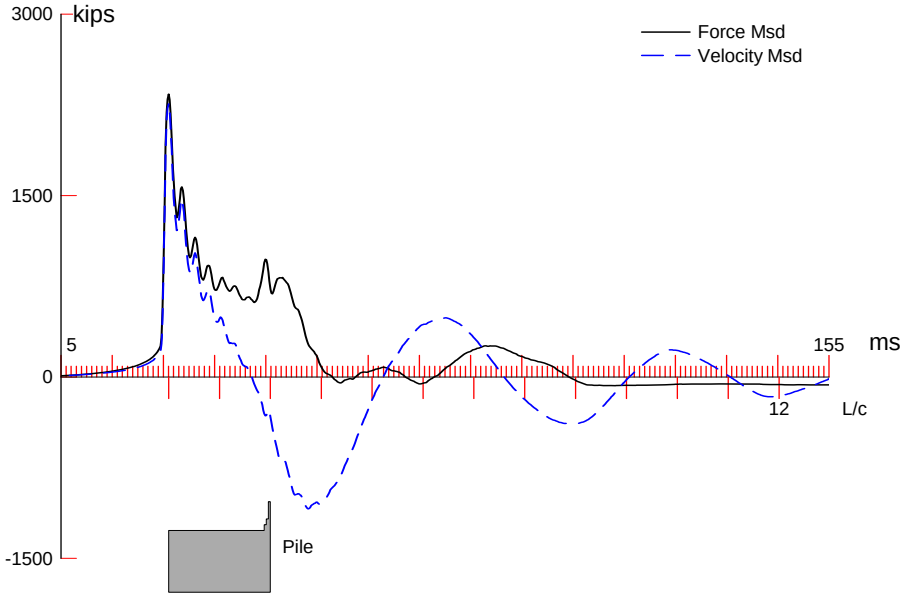
BL#	CSX	CSI	CSB	EMX	USR	FMX	RX6
	ksi	ksi	ksi	k-ft	[]	kips	kips
1	20.8	21.4	7.6	68.4	1.0	1,247	1,218
2	45.7	46.7	13.7	467.3	6.0	2,744	2,757
3	46.2	46.2	14.9	532.9	7.2	2,773	2,846

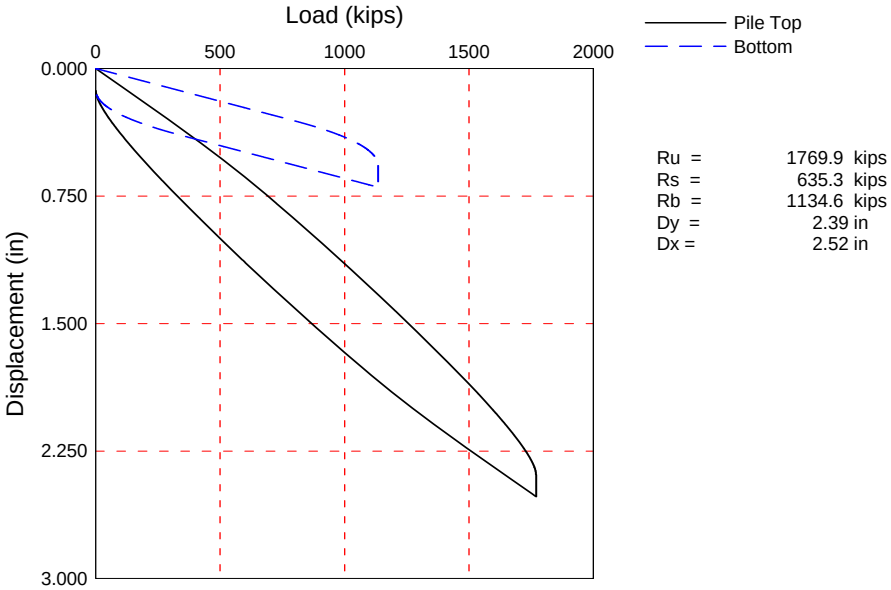
Total number of blows analyzed: 3

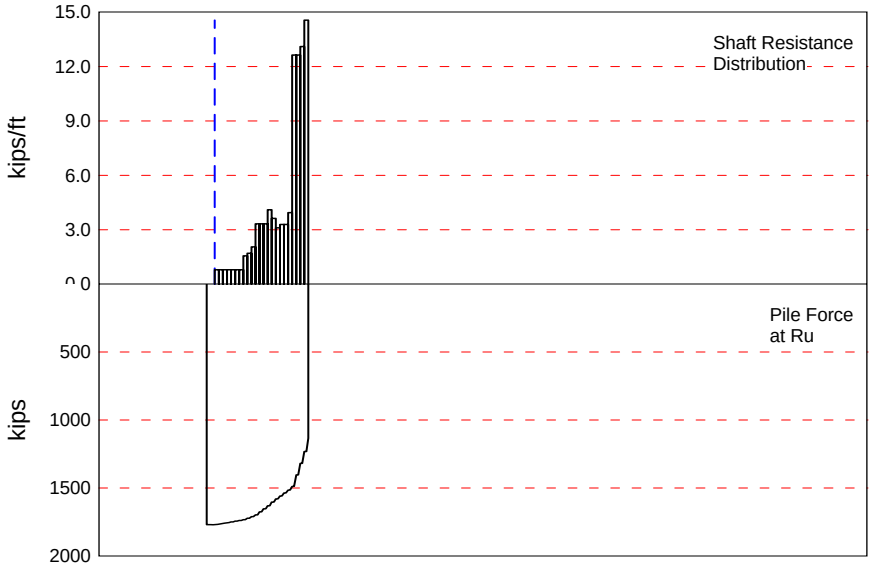
Appendix C

CAPWAP Results









CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1769.9; along Shaft 635.3; at Toe 1134.6 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
				1769.9				
1	20.0	6.0	5.3	1764.6	5.3	0.88	0.14	0.102
2	26.7	12.7	5.3	1759.3	10.6	0.79	0.13	0.102
3	33.4	19.4	5.3	1754.0	15.9	0.79	0.13	0.102
4	40.1	26.1	5.3	1748.7	21.2	0.79	0.13	0.102
5	46.8	32.8	5.3	1743.4	26.5	0.79	0.13	0.102
6	53.4	39.4	5.3	1738.1	31.8	0.79	0.13	0.102
7	60.1	46.1	5.3	1732.8	37.1	0.79	0.13	0.102
8	66.8	52.8	10.4	1722.4	47.5	1.56	0.26	0.102
9	73.5	59.5	11.4	1711.0	58.9	1.71	0.28	0.102
10	80.2	66.2	13.7	1697.3	72.6	2.05	0.34	0.102
11	86.8	72.8	22.2	1675.1	94.8	3.32	0.55	0.102
12	93.5	79.5	22.2	1652.9	117.0	3.32	0.55	0.102
13	100.2	86.2	22.2	1630.7	139.2	3.32	0.55	0.102
14	106.9	92.9	27.4	1603.3	166.6	4.10	0.68	0.102
15	113.6	99.6	24.2	1579.1	190.8	3.62	0.60	0.102
16	120.2	106.2	20.7	1558.4	211.5	3.10	0.51	0.102
17	126.9	112.9	22.0	1536.4	233.5	3.29	0.54	0.102
18	133.6	119.6	22.0	1514.4	255.5	3.29	0.54	0.102
19	140.3	126.3	26.3	1488.1	281.8	3.94	0.65	0.102
20	147.0	133.0	84.4	1403.7	366.2	12.63	2.08	0.102
21	153.6	139.6	84.4	1319.3	450.6	12.63	2.08	0.102
22	160.3	146.3	87.5	1231.8	538.1	13.10	2.16	0.102
23	167.0	153.0	97.2	1134.6	635.3	14.55	2.40	0.102
Avg. Shaft			27.6			4.15	0.68	0.102
Toe			1134.6				493.93	0.085

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.438
Case Damping Factor		0.603	0.900
Unloading Quake	(% of loading quake)	100	73
Reloading Level	(% of Ru)	-99	0
Soil Plug Weight	(kips)		0.18

CAPWAP match quality	= 0.77	(Wave Up Match)	; RSA = 0
Observed: final set	= 0.129 in;	blow count	= 93 b/ft
Computed: final set	= 0.156 in;	blow count	= 77 b/ft

max. Top Comp. Stress	= 38.7 ksi	(T= 26.4 ms, max= 1.007 x Top)
max. Comp. Stress	= 38.9 ksi	(Z= 20.0 ft, T= 27.4 ms)
max. Tens. Stress	= -6.15 ksi	(Z= 126.9 ft, T= 67.6 ms)
max. Energy (EMX)	= 170.4 kip-ft;	max. Measured Top Displ. (DMX)= 1.64 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	2320.9	-91.4	38.7	-1.52	170.42	21.3	1.641
2	6.7	2322.0	-99.5	38.7	-1.66	169.99	21.3	1.626
5	16.7	2331.8	-145.5	38.9	-2.42	168.47	21.1	1.577
8	26.7	2329.1	-178.4	38.8	-2.97	165.52	21.0	1.527
11	36.7	2309.9	-202.8	38.5	-3.38	160.88	20.8	1.471
14	46.8	2308.3	-249.2	38.5	-4.15	156.90	20.7	1.405
17	56.8	2291.6	-263.8	38.2	-4.39	151.79	20.5	1.339
20	66.8	2300.6	-301.4	38.3	-5.02	147.08	20.2	1.263
23	76.8	2266.5	-323.4	37.8	-5.39	139.23	19.9	1.189
26	86.8	2272.0	-352.7	37.9	-5.88	132.92	19.5	1.111
29	96.9	2193.0	-339.9	36.5	-5.66	121.81	19.0	1.036
32	106.9	2184.3	-367.0	36.4	-6.11	114.30	18.5	0.956
35	116.9	2092.7	-340.1	34.9	-5.67	101.68	18.1	0.864
38	126.9	2085.5	-369.0	34.7	-6.15	94.70	17.6	0.780
41	136.9	2031.8	-340.6	33.9	-5.68	84.57	17.1	0.689
44	147.0	2111.6	-318.5	35.2	-5.31	77.13	15.8	0.599
45	150.3	2008.4	-243.3	33.5	-4.05	68.73	15.2	0.569
46	153.6	2124.4	-252.8	35.4	-4.21	66.92	14.3	0.537
47	157.0	2037.6	-206.4	34.0	-3.44	59.64	13.3	0.507
48	160.3	2057.6	-209.1	34.3	-3.48	58.15	13.1	0.481
49	163.7	1818.5	-172.6	30.3	-2.88	52.10	13.6	0.458
50	167.0	1856.6	-173.7	30.9	-2.89	47.82	13.5	0.440
Absolute	20.0			38.9			(T =	27.4 ms)
	126.9				-6.15		(T =	67.6 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	2842.6	2663.7	2484.8	2305.9	2127.0	1948.1	1769.2	1590.2	1411.3	1232.4
RX	2842.6	2663.7	2484.8	2305.9	2127.0	1948.1	1814.3	1736.2	1658.8	1593.1
RU	2867.0	2690.6	2514.1	2337.6	2161.1	1984.7	1808.2	1631.7	1455.3	1278.8

RAU = 1194.6 (kips); RA2 = 1580.1 (kips)

Current CAPWAP Ru = 1769.9 (kips); Corresponding J(RP)= 0.60; J(RX) = 0.66

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
21.28	26.23	2279.2	2352.5	2352.5	1.637	0.128	0.129	171.1	2325.4

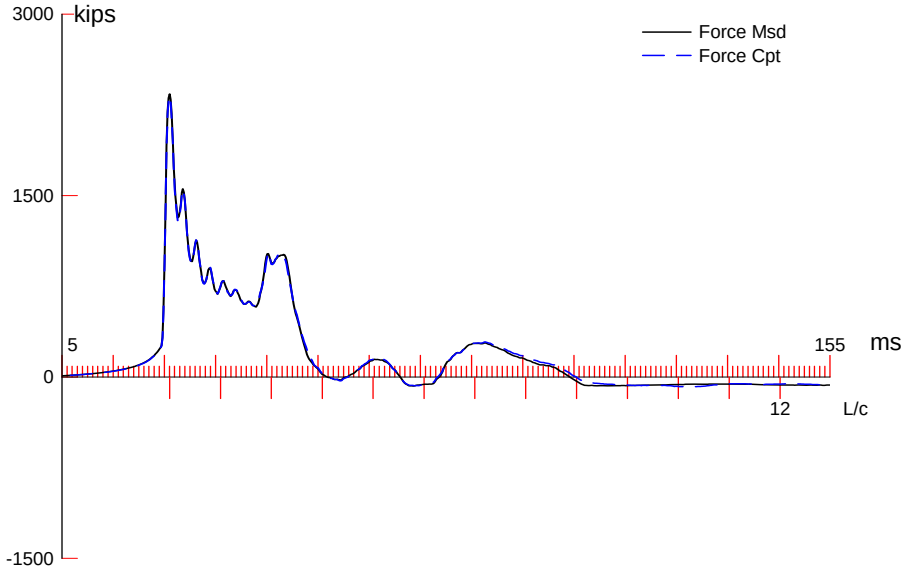
PILE PROFILE AND PILE MODEL

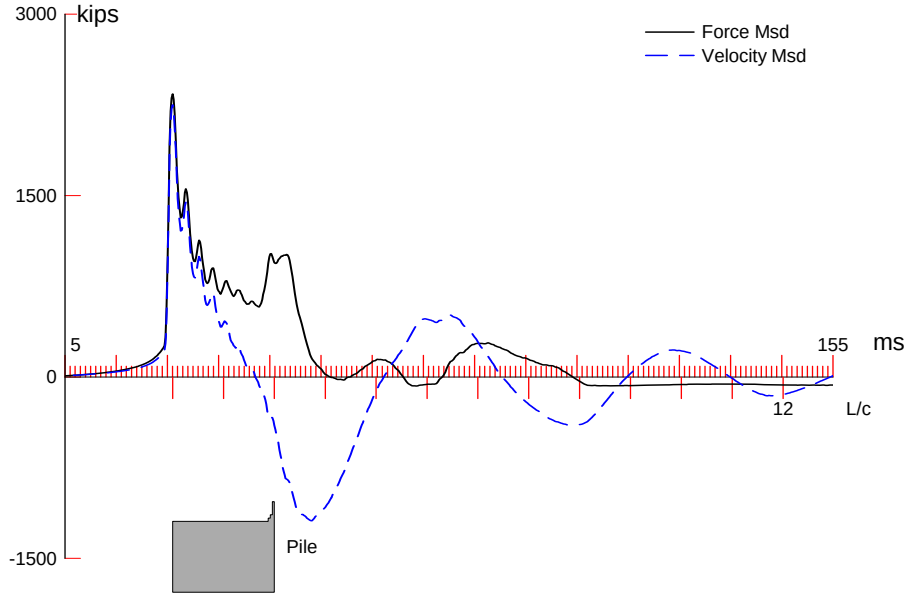
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	60.00	29992.2	492.000	6.063
167.00	60.00	29992.2	492.000	6.063

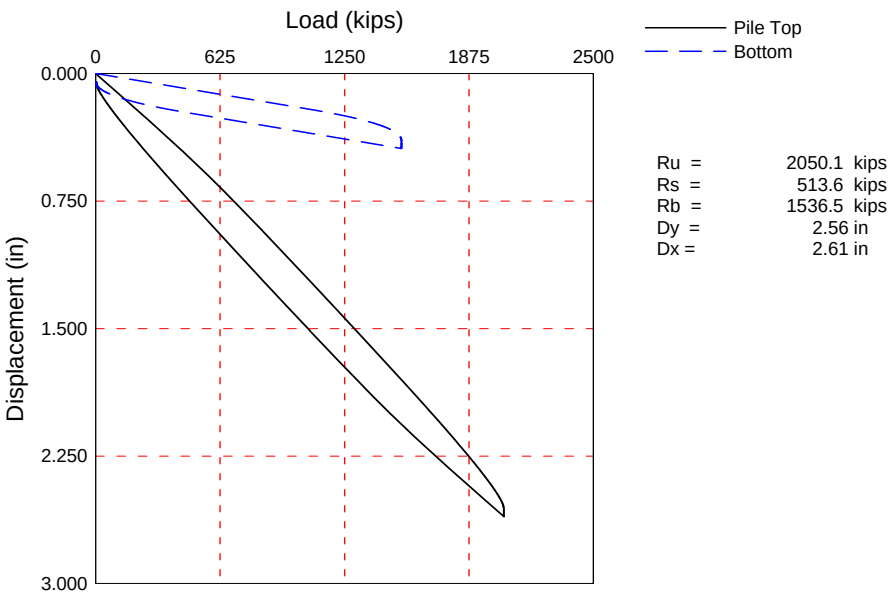
Toe Area 2.297 ft^2

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	3.34	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
48	160.32	117.09	9.34	0.000	0.000	-0.000	0.000	6.063
49	163.66	127.09	18.68	0.000	0.000	-0.000	0.000	6.063
50	167.00	157.09	46.69	0.000	0.000	-0.000	0.000	6.063

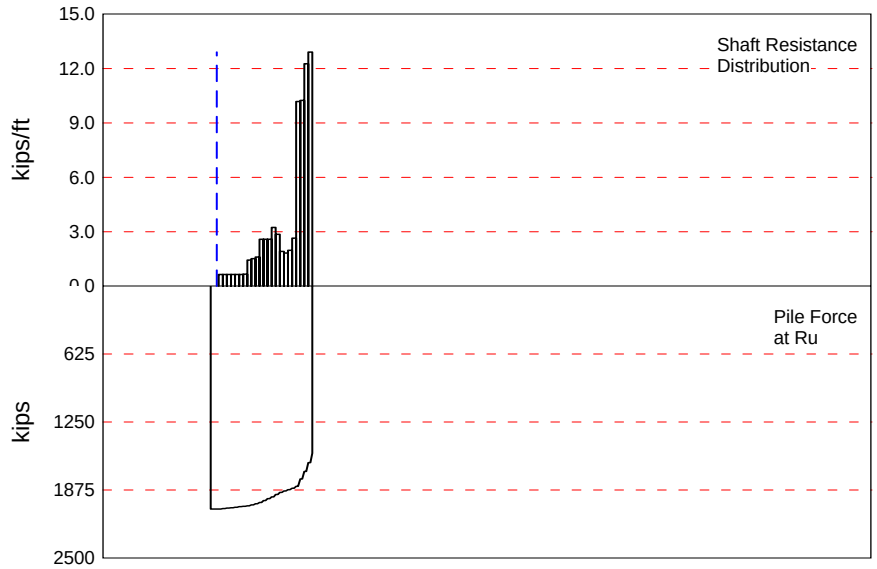
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.9 ms







Ru =	2050.1	kips
Rs =	513.6	kips
Rb =	1536.5	kips
Dy =	2.56	in
Dx =	2.61	in



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2050.1; along Shaft 513.6; at Toe 1536.5 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
				2050.1				
1	20.0	7.0	4.3	2045.8	4.3	0.61	0.10	0.145
2	26.7	13.7	4.3	2041.5	8.6	0.64	0.11	0.145
3	33.4	20.4	4.3	2037.2	12.9	0.64	0.11	0.145
4	40.1	27.1	4.3	2032.9	17.2	0.64	0.11	0.145
5	46.8	33.8	4.3	2028.6	21.5	0.64	0.11	0.145
6	53.4	40.4	4.3	2024.3	25.8	0.64	0.11	0.145
7	60.1	47.1	4.4	2019.9	30.2	0.66	0.11	0.145
8	66.8	53.8	9.6	2010.3	39.8	1.44	0.24	0.145
9	73.5	60.5	10.1	2000.2	49.9	1.51	0.25	0.145
10	80.2	67.2	10.7	1989.5	60.6	1.60	0.26	0.145
11	86.8	73.8	17.3	1972.2	77.9	2.59	0.43	0.145
12	93.5	80.5	17.3	1954.9	95.2	2.59	0.43	0.145
13	100.2	87.2	17.3	1937.6	112.5	2.59	0.43	0.145
14	106.9	93.9	21.6	1916.0	134.1	3.23	0.53	0.145
15	113.6	100.6	19.1	1896.9	153.2	2.86	0.47	0.145
16	120.2	107.2	12.8	1884.1	166.0	1.92	0.32	0.145
17	126.9	113.9	12.2	1871.9	178.2	1.83	0.30	0.145
18	133.6	120.6	13.2	1858.7	191.4	1.98	0.33	0.145
19	140.3	127.3	17.7	1841.0	209.1	2.65	0.44	0.145
20	147.0	134.0	68.0	1773.0	277.1	10.18	1.68	0.145
21	153.6	140.6	68.4	1704.6	345.5	10.24	1.69	0.145
22	160.3	147.3	81.9	1622.7	427.4	12.26	2.02	0.145
23	167.0	154.0	86.2	1536.5	513.6	12.90	2.13	0.145
Avg. Shaft			22.3			3.34	0.55	0.145
Toe			1536.5				668.89	0.074

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.301
Case Damping Factor		0.695	1.060
Unloading Quake	(% of loading quake)	97	110
Reloading Level	(% of Ru)	-97	67
Soil Plug Weight	(kips)		0.45

CAPWAP match quality	= 0.83	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.050 in;	blow count	=	240 b/ft
Computed: final set	= 0.028 in;	blow count	=	425 b/ft

max. Top Comp. Stress	= 38.6 ksi	(T= 26.4 ms, max= 1.007 x Top)
max. Comp. Stress	= 38.9 ksi	(Z= 20.0 ft, T= 27.4 ms)
max. Tens. Stress	= -6.10 ksi	(Z= 133.6 ft, T= 66.6 ms)
max. Energy (EMX)	= 164.8 kip-ft;	max. Measured Top Displ. (DMX)= 1.60 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	2317.1	-85.3	38.6	-1.42	164.79	21.2	1.600
2	6.7	2318.4	-98.5	38.6	-1.64	164.39	21.2	1.586
5	16.7	2329.0	-134.9	38.8	-2.25	163.03	21.1	1.540
8	26.7	2326.5	-159.4	38.8	-2.66	160.28	20.9	1.492
11	36.7	2306.8	-180.7	38.4	-3.01	155.91	20.8	1.438
14	46.8	2305.5	-209.7	38.4	-3.49	152.24	20.6	1.376
17	56.8	2288.7	-224.3	38.1	-3.74	147.48	20.4	1.314
20	66.8	2300.6	-254.3	38.3	-4.24	143.16	20.1	1.242
23	76.8	2258.0	-271.9	37.6	-4.53	134.76	19.8	1.166
26	86.8	2264.5	-325.3	37.7	-5.42	128.73	19.3	1.090
29	96.9	2186.1	-327.2	36.4	-5.45	117.01	18.9	1.004
32	106.9	2179.0	-360.3	36.3	-6.00	108.50	18.3	0.912
35	116.9	2082.4	-338.9	34.7	-5.65	97.23	17.9	0.829
38	126.9	2079.3	-364.4	34.6	-6.07	89.56	17.5	0.732
41	136.9	2051.6	-353.7	34.2	-5.89	78.39	17.0	0.620
44	147.0	2148.4	-357.1	35.8	-5.95	69.89	15.7	0.516
45	150.3	2025.0	-284.0	33.7	-4.73	61.66	15.1	0.480
46	153.6	2128.1	-290.2	35.5	-4.84	58.92	14.3	0.442
47	157.0	2097.5	-232.8	34.9	-3.88	51.78	13.3	0.404
48	160.3	2170.6	-235.8	36.2	-3.93	49.30	12.0	0.369
49	163.7	1999.5	-188.6	33.3	-3.14	43.23	11.9	0.337
50	167.0	2072.7	-188.3	34.5	-3.14	40.09	11.6	0.309
Absolute	20.0			38.9			(T =	27.4 ms)
	133.6				-6.10		(T =	66.6 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	2986.2	2823.3	2660.4	2497.5	2334.5	2171.6	2008.7	1845.8	1682.8	1519.9
RX	2986.2	2823.3	2660.4	2497.5	2334.5	2171.6	2091.0	2033.0	1974.9	1916.8
RU	3008.1	2847.4	2686.6	2525.9	2365.1	2204.4	2043.7	1882.9	1722.2	1561.5

RAU = 1431.2 (kips); RA2 = 1848.5 (kips)

Current CAPWAP Ru = 2050.1 (kips); Corresponding J(RP)= 0.57; J(RX) = 0.67

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
21.16	26.23	2266.3	2349.1	2349.1	1.600	0.049	0.050	165.5	2407.3

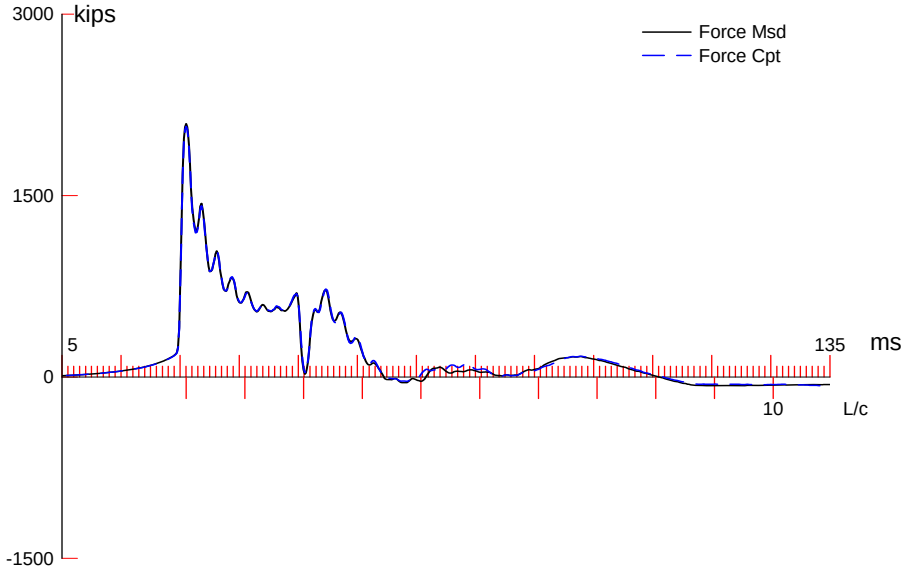
PILE PROFILE AND PILE MODEL

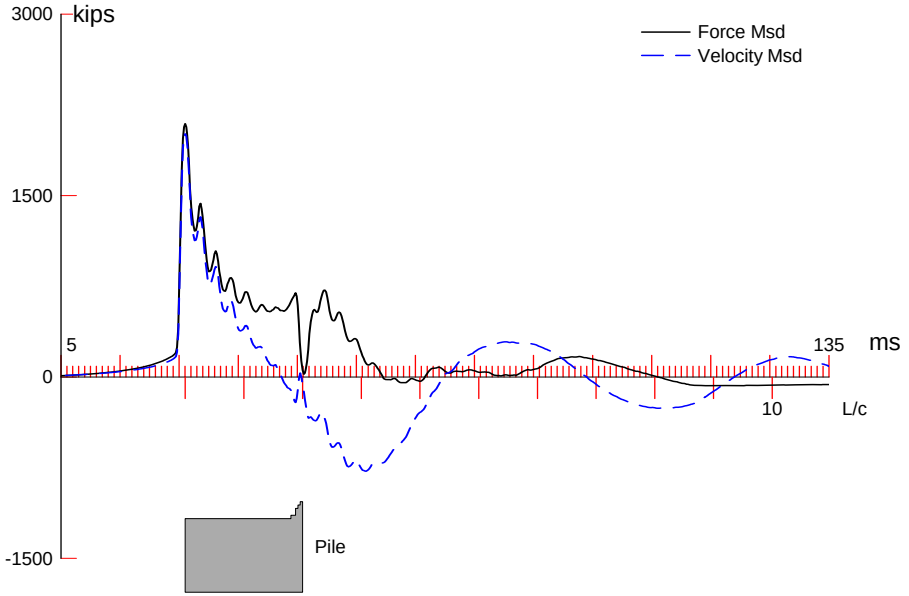
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
167.00	60.00	29992.2	492.000	6.063

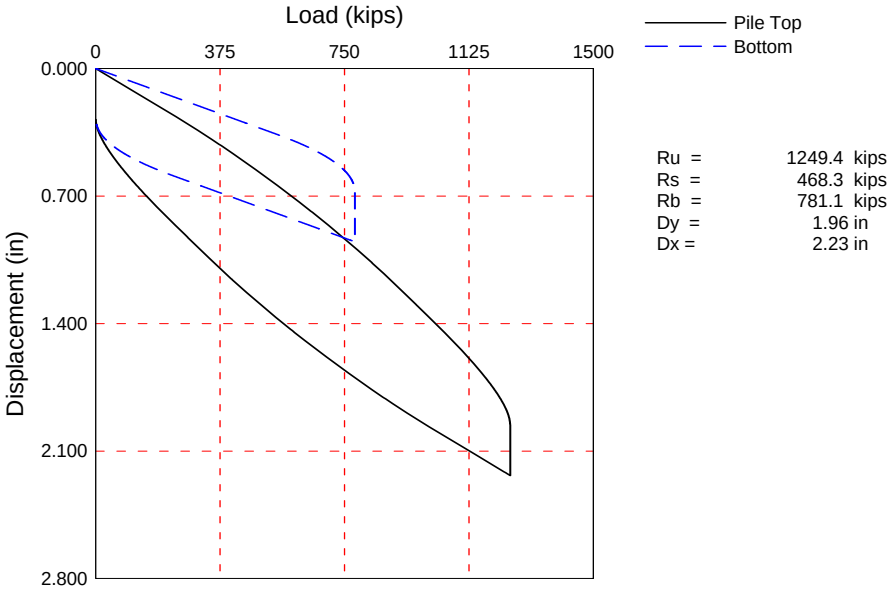
Toe Area 2.297 ft^2

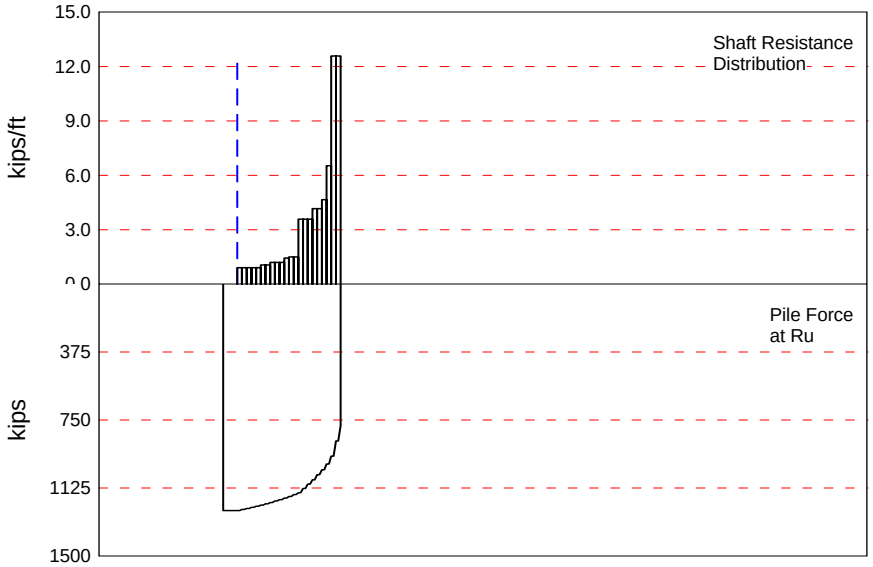
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft
1	3.34	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
48	160.32	112.09	4.67	0.000	0.000	-0.000	0.000	6.063
49	163.66	117.09	9.34	0.000	0.000	-0.000	0.000	6.063
50	167.00	137.09	28.01	0.000	0.000	-0.000	0.000	6.063

Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.9 ms









CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1249.4; along Shaft 468.3; at Toe 781.1 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
				1249.4				
1	26.7	3.7	6.1	1243.3	6.1	1.64	0.27	0.148
2	33.4	10.4	6.1	1237.2	12.2	0.91	0.15	0.148
3	40.1	17.1	6.1	1231.1	18.3	0.91	0.15	0.148
4	46.8	23.8	6.1	1225.0	24.4	0.91	0.15	0.148
5	53.4	30.4	6.1	1218.9	30.5	0.91	0.15	0.148
6	60.1	37.1	7.1	1211.8	37.6	1.06	0.18	0.148
7	66.8	43.8	7.1	1204.7	44.7	1.06	0.18	0.148
8	73.5	50.5	8.0	1196.7	52.7	1.20	0.20	0.148
9	80.2	57.2	8.0	1188.7	60.7	1.20	0.20	0.148
10	86.8	63.8	8.0	1180.7	68.7	1.20	0.20	0.148
11	93.5	70.5	9.6	1171.1	78.3	1.44	0.24	0.148
12	100.2	77.2	10.0	1161.1	88.3	1.50	0.25	0.148
13	106.9	83.9	10.0	1151.1	98.3	1.50	0.25	0.148
14	113.6	90.6	23.9	1127.2	122.2	3.58	0.59	0.148
15	120.2	97.2	23.9	1103.3	146.1	3.58	0.59	0.148
16	126.9	103.9	23.9	1079.4	170.0	3.58	0.59	0.148
17	133.6	110.6	27.8	1051.6	197.8	4.16	0.69	0.148
18	140.3	117.3	27.8	1023.8	225.6	4.16	0.69	0.148
19	147.0	124.0	31.1	992.7	256.7	4.66	0.77	0.148
20	153.6	130.6	43.6	949.1	300.3	6.53	1.08	0.148
21	160.3	137.3	84.0	865.1	384.3	12.57	2.07	0.148
22	167.0	144.0	84.0	781.1	468.3	12.57	2.07	0.148
Avg. Shaft			21.3			3.25	0.54	0.148
Toe			781.1				340.04	0.048

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.247	0.517
Case Damping Factor		0.646	0.350
Unloading Quake	(% of loading quake)	99	67
Reloading Level	(% of Ru)	100	53
Resistance Gap (included in Toe Quake)	(in)		0.001
Soil Plug Weight	(kips)		0.06

CAPWAP match quality	= 1.25	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.279 in;	blow count	=	43 b/ft
Computed: final set	= 0.318 in;	blow count	=	38 b/ft

max. Top Comp. Stress	= 34.5 ksi	(T= 26.4 ms, max= 1.011 x Top)
max. Comp. Stress	= 34.9 ksi	(Z= 26.7 ft, T= 27.8 ms)
max. Tens. Stress	= -4.42 ksi	(Z= 120.2 ft, T= 70.7 ms)
max. Energy (EMX)	= 134.1 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	3.3	2072.8	-69.7	34.5	-1.16	134.10	19.0	1.453
2	6.7	2074.0	-74.6	34.6	-1.24	133.78	19.0	1.440
5	16.7	2078.6	-118.7	34.6	-1.98	132.61	18.9	1.398
8	26.7	2095.4	-136.4	34.9	-2.27	131.35	18.7	1.360
11	36.7	2067.4	-165.0	34.4	-2.75	127.30	18.6	1.326
14	46.8	2062.9	-204.3	34.4	-3.40	124.76	18.4	1.287
17	56.8	2037.0	-227.6	33.9	-3.79	120.71	18.2	1.244
20	66.8	2032.5	-224.2	33.9	-3.74	117.70	17.9	1.198
23	76.8	2000.4	-247.3	33.3	-4.12	112.73	17.7	1.146
26	86.8	1996.1	-236.8	33.3	-3.95	109.32	17.5	1.094
29	96.9	1961.7	-243.1	32.7	-4.05	103.84	17.2	1.037
32	106.9	1964.5	-259.1	32.7	-4.32	99.72	16.8	0.977
35	116.9	1906.8	-246.3	31.8	-4.10	91.55	16.3	0.914
38	126.9	1889.2	-260.8	31.5	-4.35	85.38	15.8	0.852
41	136.9	1789.1	-224.2	29.8	-3.74	75.15	15.2	0.785
44	147.0	1825.0	-205.9	30.4	-3.43	69.19	14.2	0.724
45	150.3	1801.2	-180.4	30.0	-3.01	64.50	13.7	0.702
46	153.6	1870.0	-189.1	31.2	-3.15	63.59	13.0	0.681
47	157.0	1769.0	-156.2	29.5	-2.60	57.83	13.7	0.660
48	160.3	1666.5	-162.2	27.8	-2.70	57.11	15.4	0.641
49	163.7	1207.0	-104.2	20.1	-1.74	47.57	16.2	0.626
50	167.0	1164.0	-105.3	19.4	-1.75	39.69	16.2	0.610
Absolute	26.7			34.9			(T =	27.8 ms)
	120.2				-4.42		(T =	70.7 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	2127.8	1927.8	1727.8	1527.8	1327.8	1127.8	927.8	727.8	527.8	327.8
RX	2127.8	1927.8	1728.3	1545.1	1450.1	1355.1	1260.0	1184.9	1150.6	1123.1
RU	2135.9	1936.7	1737.5	1538.3	1339.2	1140.0	940.8	741.6	542.4	343.2

RAU = 990.0 (kips); RA2 = 1202.9 (kips)

Current CAPWAP Ru = 1249.4 (kips); Corresponding J(RP)= 0.44; J(RX) = 0.61

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
18.90	26.23	2023.8	2104.0	2104.0	1.459	0.279	0.279	134.8	1862.5

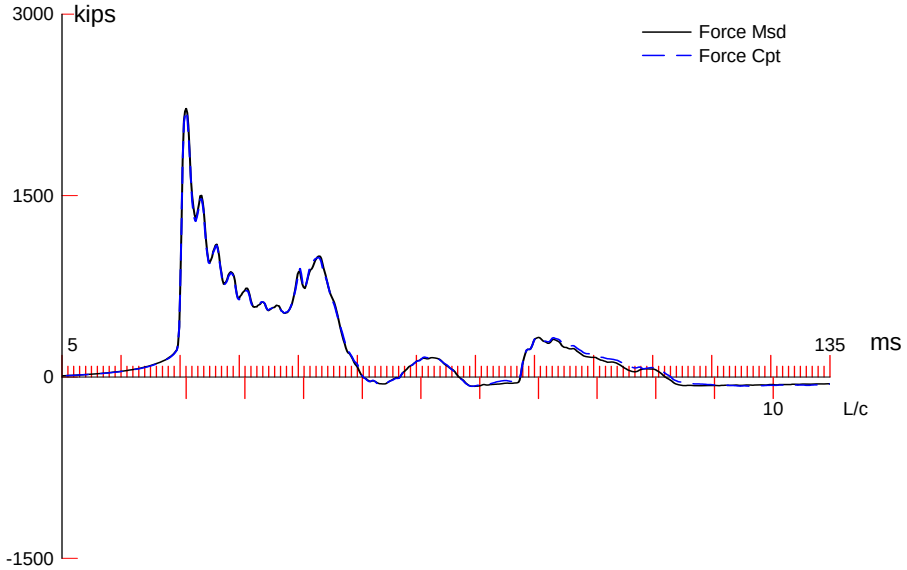
PILE PROFILE AND PILE MODEL

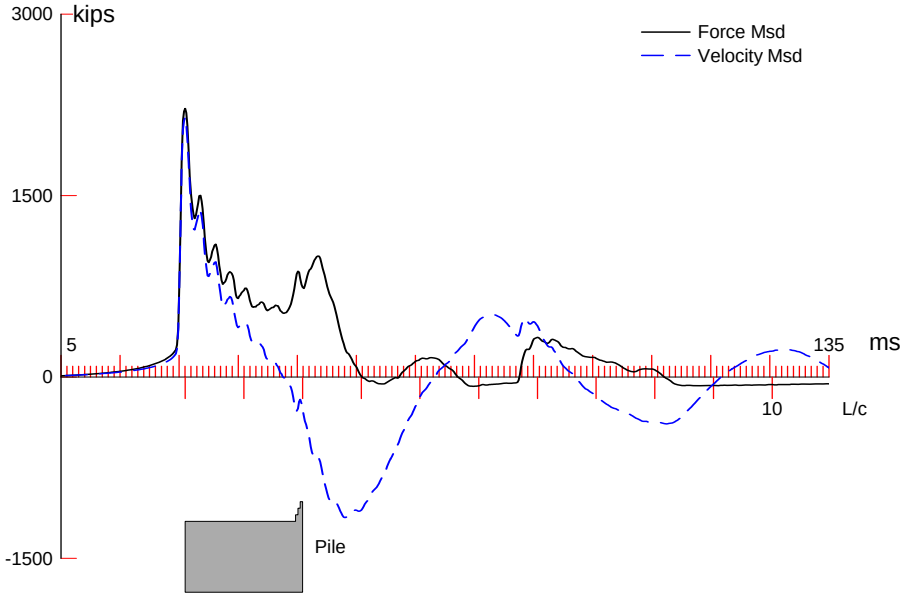
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
167.00	60.00	29992.2	492.000	6.063

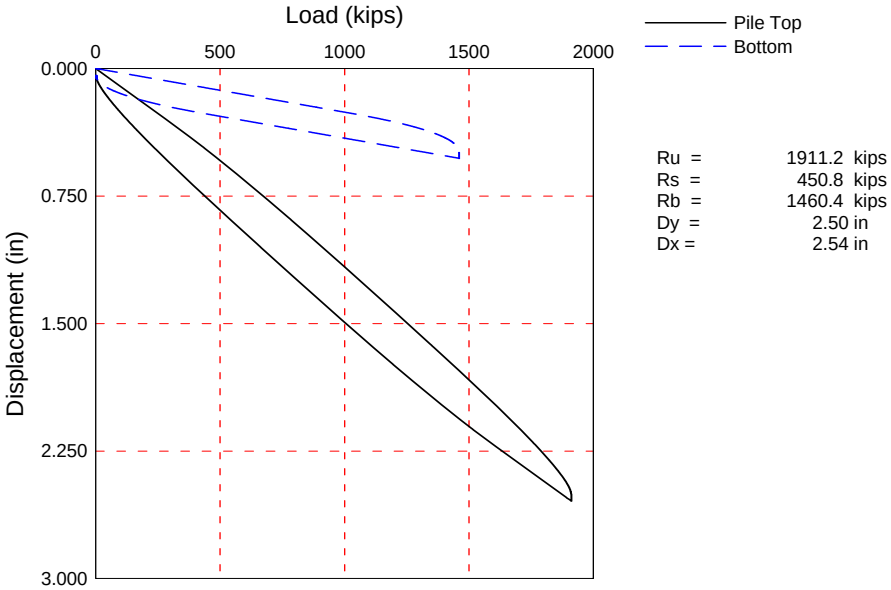
Toe Area 2.297 ft^2

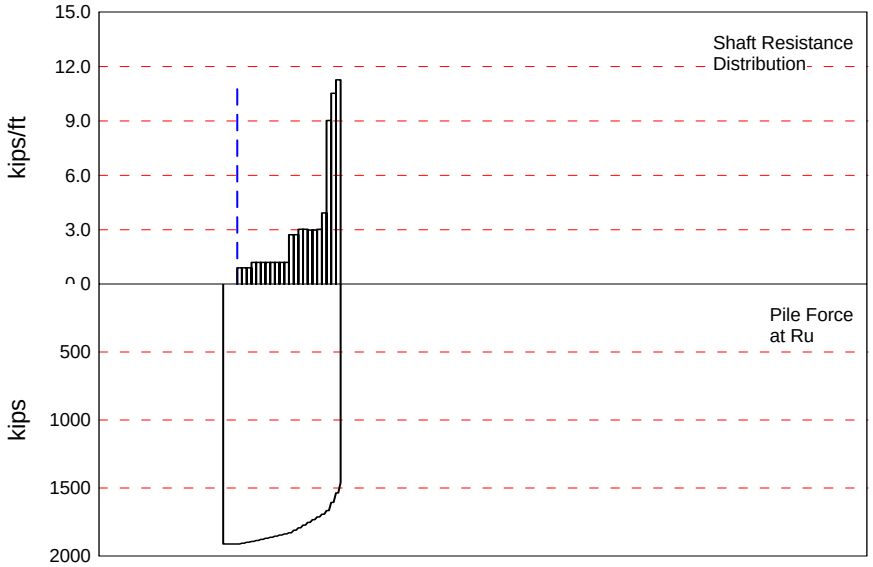
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Slack in	Tension Eff.	Slack in	Compression Eff.	Perim. ft
1	3.34	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
46	153.64	112.09	4.67	0.000	0.000	-0.000	0.000	6.063
48	160.32	122.09	14.01	0.000	0.000	-0.000	0.000	6.063
49	163.66	127.09	18.68	0.000	0.000	-0.000	0.000	6.063
50	167.00	132.09	23.34	0.000	0.000	-0.000	0.000	6.063

Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.9 ms









CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1911.2; along Shaft 450.8; at Toe 1460.4 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
				1911.2				
1	26.7	6.3	6.0	1905.2	6.0	0.95	0.16	0.104
2	33.4	13.0	6.0	1899.2	12.0	0.90	0.15	0.104
3	40.1	19.7	6.0	1893.2	18.0	0.90	0.15	0.104
4	46.8	26.4	8.0	1885.2	26.0	1.20	0.20	0.104
5	53.4	33.0	8.0	1877.2	34.0	1.20	0.20	0.104
6	60.1	39.7	8.0	1869.2	42.0	1.20	0.20	0.104
7	66.8	46.4	8.0	1861.2	50.0	1.20	0.20	0.104
8	73.5	53.1	8.0	1853.2	58.0	1.20	0.20	0.104
9	80.2	59.8	8.0	1845.2	66.0	1.20	0.20	0.104
10	86.8	66.4	8.0	1837.2	74.0	1.20	0.20	0.104
11	93.5	73.1	8.0	1829.2	82.0	1.20	0.20	0.104
12	100.2	79.8	18.2	1811.0	100.2	2.72	0.45	0.104
13	106.9	86.5	18.2	1792.8	118.4	2.72	0.45	0.104
14	113.6	93.2	20.2	1772.6	138.6	3.02	0.50	0.104
15	120.2	99.8	20.2	1752.4	158.8	3.02	0.50	0.104
16	126.9	106.5	19.9	1732.5	178.7	2.98	0.49	0.104
17	133.6	113.2	19.9	1712.6	198.6	2.98	0.49	0.104
18	140.3	119.9	20.2	1692.4	218.8	3.02	0.50	0.104
19	147.0	126.6	26.2	1666.2	245.0	3.92	0.65	0.104
20	153.6	133.2	60.3	1605.9	305.3	9.03	1.49	0.104
21	160.3	139.9	70.3	1535.6	375.6	10.52	1.74	0.104
22	167.0	146.6	75.2	1460.4	450.8	11.26	1.86	0.104
Avg. Shaft			20.5			3.08	0.51	0.104
Toe			1460.4				635.76	0.074

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.375
Case Damping Factor		0.439	1.004
Unloading Quake	(% of loading quake)	92	118
Reloading Level	(% of Ru)	-89	0
Resistance Gap (included in Toe Quake)	(in)		0.000
Soil Plug Weight	(kips)		0.31

CAPWAP match quality	= 1.03	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.040 in;	blow count	=	300 b/ft
Computed: final set	= 0.037 in;	blow count	=	320 b/ft

max. Top Comp. Stress	= 36.7 ksi	(T= 26.4 ms, max= 1.009 x Top)
max. Comp. Stress	= 37.0 ksi	(Z= 26.7 ft, T= 27.8 ms)
max. Tens. Stress	= -5.72 ksi	(Z= 120.2 ft, T= 67.4 ms)
max. Energy (EMX)	= 150.3 kip-ft;	max. Measured Top Displ. (DMX)= 1.54 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	2201.6	-92.6	36.7	-1.54	150.33	20.2	1.548
2	6.7	2202.8	-107.2	36.7	-1.79	150.02	20.2	1.536
5	16.7	2207.1	-146.9	36.8	-2.45	148.93	20.1	1.495
8	26.7	2220.9	-195.6	37.0	-3.26	147.51	20.0	1.448
11	36.7	2199.9	-198.0	36.7	-3.30	143.36	19.8	1.401
14	46.8	2200.6	-218.4	36.7	-3.64	140.13	19.6	1.346
17	56.8	2172.4	-224.9	36.2	-3.75	134.73	19.4	1.290
20	66.8	2168.4	-232.1	36.1	-3.87	130.82	19.3	1.229
23	76.8	2142.2	-243.8	35.7	-4.06	124.70	19.1	1.160
26	86.8	2140.0	-277.4	35.7	-4.62	120.04	18.8	1.088
29	96.9	2126.3	-310.3	35.4	-5.17	114.44	18.5	1.020
32	106.9	2116.1	-335.2	35.3	-5.59	107.69	18.2	0.941
35	116.9	2056.9	-320.3	34.3	-5.34	97.36	17.8	0.852
38	126.9	2047.0	-339.8	34.1	-5.66	90.38	17.4	0.768
41	136.9	1992.5	-303.8	33.2	-5.06	79.69	16.9	0.667
44	147.0	2015.4	-308.8	33.6	-5.14	71.10	16.3	0.563
45	150.3	2032.8	-282.0	33.9	-4.70	66.76	15.7	0.528
46	153.6	2132.0	-292.5	35.5	-4.87	64.29	14.9	0.492
47	157.0	2102.0	-223.5	35.0	-3.72	58.26	14.0	0.457
48	160.3	2163.2	-235.7	36.0	-3.93	56.46	13.2	0.428
49	163.7	1999.5	-184.9	33.3	-3.08	51.34	13.5	0.401
50	167.0	2051.6	-198.2	34.2	-3.30	48.58	13.3	0.375
Absolute	26.7			37.0			(T =	27.8 ms)
	120.2				-5.72		(T =	67.4 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	2711.1	2542.8	2374.6	2206.4	2038.1	1869.9	1701.6	1533.4	1365.2	1196.9
RX	2711.1	2542.8	2374.6	2206.4	2044.0	1981.4	1918.8	1856.1	1793.5	1733.8
RU	2732.3	2566.2	2400.1	2234.0	2067.9	1901.8	1735.7	1569.6	1403.5	1237.4

RAU = 1494.8 (kips); RA2 = 1790.1 (kips)

Current CAPWAP Ru = 1911.2 (kips); Corresponding J(RP)= 0.48; J(RX) = 0.61

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
20.15	26.23	2158.0	2235.4	2235.4	1.535	0.045	0.040	150.8	2297.5

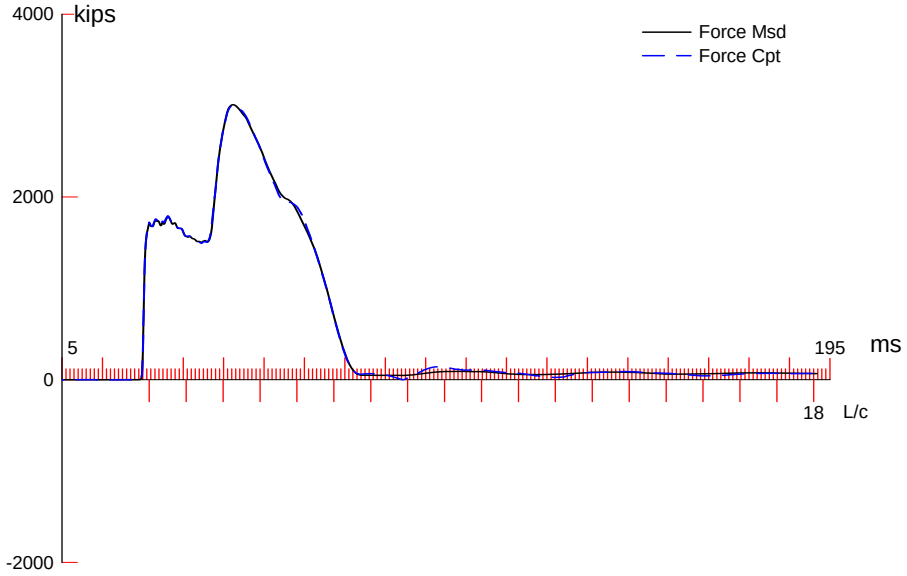
PILE PROFILE AND PILE MODEL

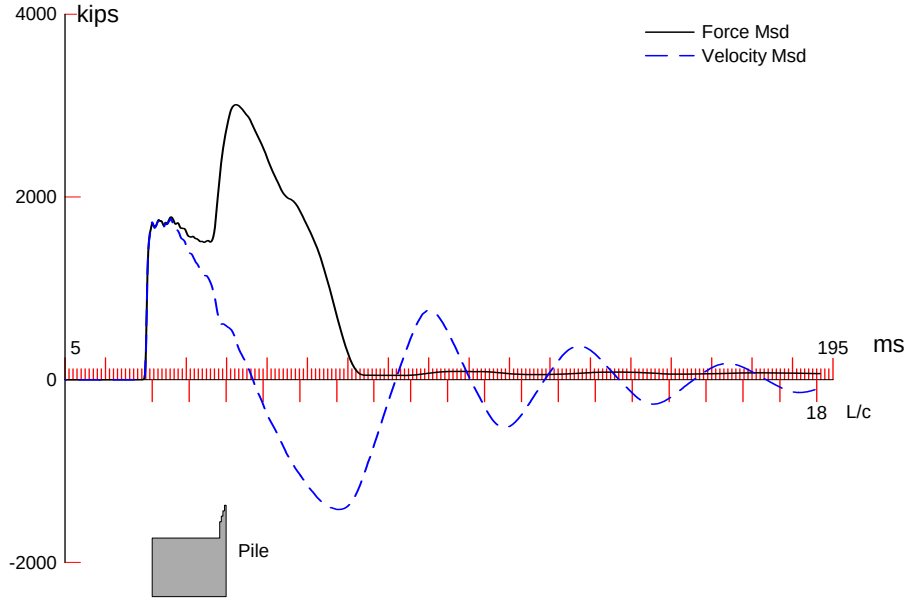
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
167.00	60.00	29992.2	492.000	6.063

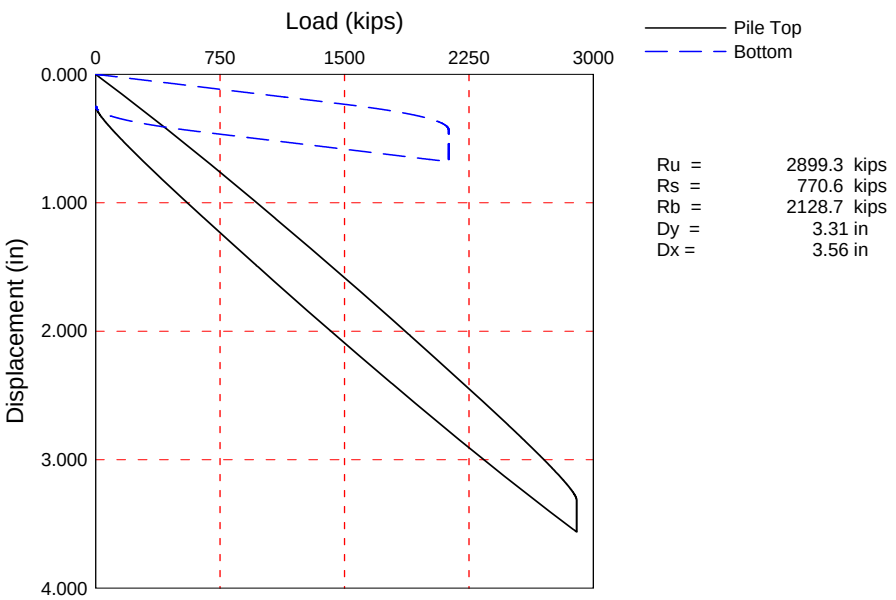
Toe Area 2.297 ft^2

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	3.34	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
48	160.32	117.09	9.34	0.000	0.000	-0.000	0.000	6.063
49	163.66	127.09	18.68	0.000	0.000	-0.000	0.000	6.063
50	167.00	137.09	28.01	0.000	0.000	-0.000	0.000	6.063

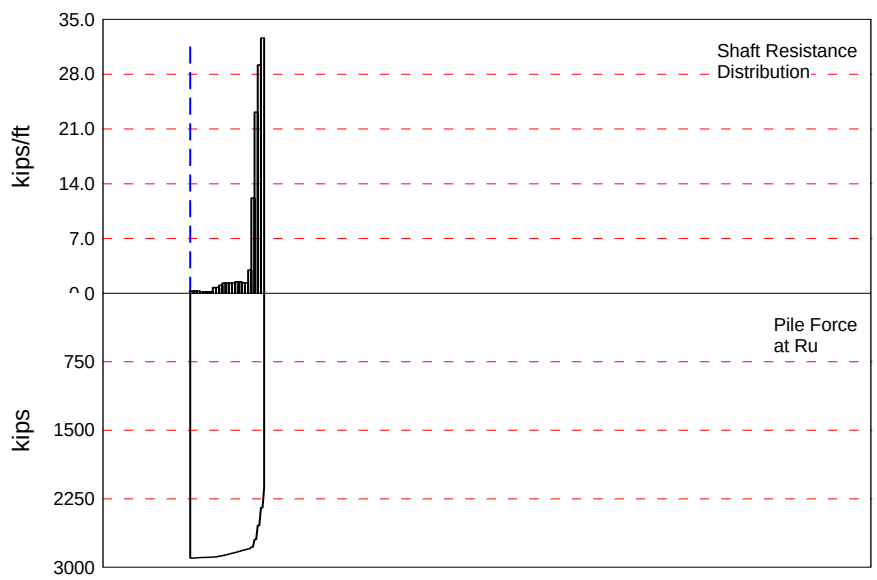
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.9 ms







Ru =	2899.3	kips
Rs =	770.6	kips
Rb =	2128.7	kips
Dy =	3.31	in
Dx =	3.56	in



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2899.3; along Shaft 770.6; at Toe 2128.7 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
				2899.3					
1	6.7	7.0	2.0	2897.3	2.0	0.29	0.05	0.192	0.291
2	13.3	13.6	2.0	2895.3	4.0	0.30	0.05	0.192	0.291
3	20.0	20.3	2.0	2893.3	6.0	0.30	0.05	0.192	0.291
4	26.7	27.0	1.4	2891.9	7.4	0.21	0.03	0.192	0.291
5	33.4	33.7	1.4	2890.5	8.8	0.21	0.03	0.192	0.291
6	40.0	40.3	1.4	2889.1	10.2	0.21	0.03	0.192	0.291
7	46.7	47.0	1.4	2887.7	11.6	0.21	0.03	0.192	0.291
8	53.4	53.7	5.1	2882.6	16.7	0.76	0.13	0.192	0.291
9	60.1	60.4	5.1	2877.5	21.8	0.76	0.13	0.192	0.291
10	66.7	67.0	7.0	2870.5	28.8	1.05	0.17	0.192	0.291
11	73.4	73.7	8.9	2861.6	37.7	1.33	0.22	0.192	0.291
12	80.1	80.4	8.9	2852.7	46.6	1.33	0.22	0.192	0.291
13	86.8	87.1	8.9	2843.8	55.5	1.33	0.22	0.192	0.291
14	93.4	93.7	8.9	2834.9	64.4	1.33	0.22	0.192	0.291
15	100.1	100.4	10.0	2824.9	74.4	1.50	0.25	0.192	0.291
16	106.8	107.1	10.0	2814.9	84.4	1.50	0.25	0.192	0.279
17	113.5	113.8	9.0	2805.9	93.4	1.35	0.22	0.192	0.267
18	120.1	120.4	9.0	2796.9	102.4	1.35	0.22	0.192	0.256
19	126.8	127.1	20.0	2776.9	122.4	3.00	0.49	0.192	0.243
20	133.5	133.8	81.3	2695.6	203.7	12.18	2.01	0.192	0.232
21	140.2	140.5	154.5	2541.1	358.2	23.15	3.82	0.192	0.219
22	146.8	147.1	194.7	2346.4	552.9	29.17	4.81	0.192	0.206
23	153.5	153.8	217.7	2128.7	770.6	32.62	5.38	0.192	0.195
Avg. Shaft			33.5			5.01	0.83	0.192	0.220
Toe			2128.7				926.69	0.097	0.332

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.380	1.927
Unloading Quake	(% of loading quake)	100	34
Reloading Level	(% of Ru)	-99	1
Unloading Level	(% of Ru)	69	
Soil Plug Weight	(kips)		0.48

CAPWAP match quality	= 0.95	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.250 in;	blow count	=	48 b/ft
Computed: final set	= 0.211 in;	blow count	=	57 b/ft

max. Top Comp. Stress	= 50.5 ksi	(T= 47.6 ms, max= 1.038 x Top)
max. Comp. Stress	= 52.4 ksi	(Z= 40.0 ft, T= 49.0 ms)
max. Tens. Stress	= -12.59 ksi	(Z= 126.8 ft, T= 85.6 ms)
max. Energy (EMX)	= 467.1 kip-ft;	max. Measured Top Displ. (DMX)= 3.23 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	3028.4	-23.2	50.5	-0.39	467.10	16.3	3.154
2	6.7	3054.7	-47.5	50.9	-0.79	459.72	16.3	3.091
5	16.7	3096.8	-121.5	51.6	-2.02	434.21	16.2	2.902
8	26.7	3126.7	-197.4	52.1	-3.29	410.71	16.1	2.713
11	36.7	3139.9	-274.0	52.3	-4.57	387.14	16.0	2.527
14	46.7	3142.5	-352.4	52.4	-5.87	365.19	15.8	2.343
17	56.7	3132.8	-427.1	52.2	-7.12	340.86	15.6	2.162
20	66.7	3119.1	-498.8	52.0	-8.31	317.73	15.4	1.981
23	76.8	3081.6	-556.6	51.3	-9.27	288.72	15.2	1.797
26	86.8	3055.5	-619.9	50.9	-10.33	262.71	14.9	1.608
29	96.8	3008.7	-657.7	50.1	-10.96	232.67	14.7	1.415
32	106.8	2996.5	-700.4	49.9	-11.67	205.25	14.5	1.217
35	116.8	3036.3	-727.1	50.6	-12.12	175.40	14.2	1.016
38	126.8	3092.4	-755.4	51.5	-12.59	148.31	12.9	0.814
39	130.1	3085.4	-744.2	51.4	-12.40	137.14	12.2	0.748
40	133.5	3102.7	-751.7	51.7	-12.52	129.19	11.1	0.682
41	136.8	3017.2	-678.0	50.3	-11.30	113.18	9.6	0.619
42	140.2	3030.6	-682.6	50.5	-11.37	106.06	8.5	0.557
43	143.5	2869.0	-543.7	47.8	-9.06	90.10	7.7	0.512
44	146.8	2882.2	-549.0	48.0	-9.15	85.63	6.7	0.470
45	150.2	2668.9	-423.0	44.5	-7.05	71.86	5.9	0.434
46	153.5	2668.9	-426.4	44.5	-7.10	60.66	5.5	0.400
Absolute	40.0			52.4			(T =	49.0 ms)
	126.8				-12.59		(T =	85.6 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	2796.6	2730.8	2665.1	2599.3	2533.6	2467.8	2402.1	2336.3	2270.5	2204.8
RX	3180.5	3147.3	3114.1	3080.9	3047.8	3015.2	2984.8	2957.1	2933.8	2914.2
RU	2796.6	2730.8	2665.1	2599.3	2533.6	2467.8	2402.1	2336.3	2270.5	2204.8

RAU = 2776.2 (kips); RA2 = 2943.4 (kips)

Current CAPWAP Ru = 2899.3 (kips); Corresponding J(RP)= 0.00; 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
16.51	26.80	1730.6	1723.5	3012.1	3.228	0.250	0.250	475.5	3281.4

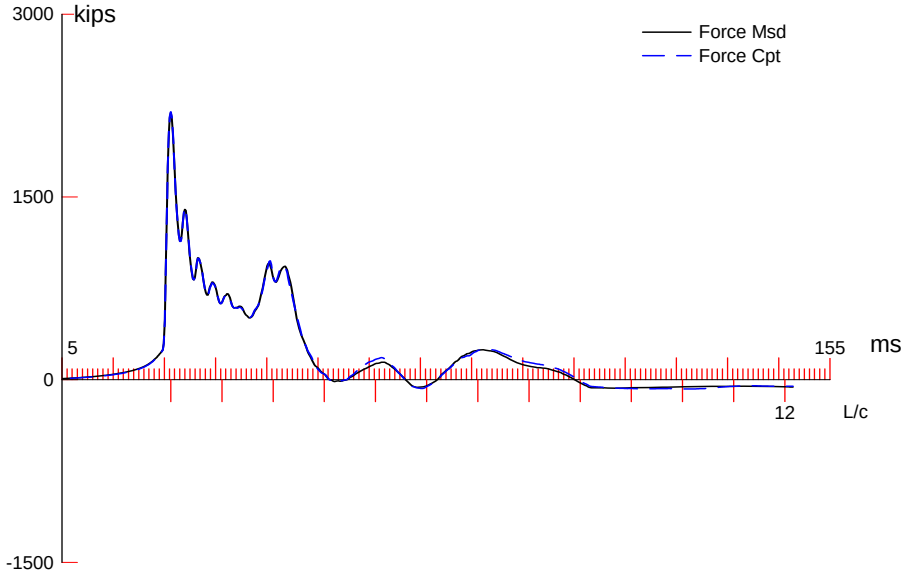
PILE PROFILE AND PILE MODEL

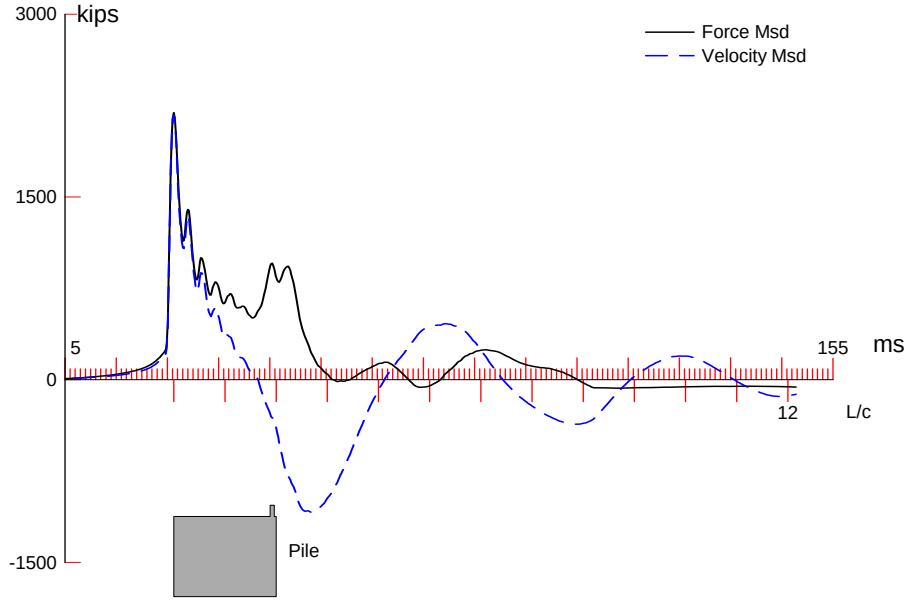
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
153.50	60.00	29992.2	492.000	6.063

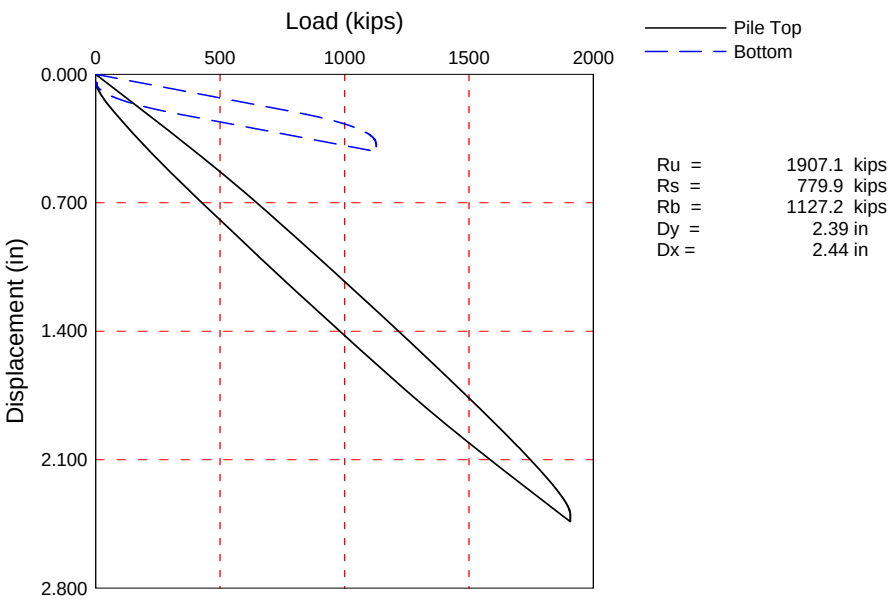
Toe Area 2.297 ft^2

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	3.34	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
43	143.49	137.09	28.01	0.000	0.000	-0.000	0.000	6.063
44	146.83	147.09	37.35	0.000	0.000	-0.000	0.000	6.063
45	150.16	157.09	46.69	0.000	0.000	-0.000	0.000	6.063
46	153.50	167.09	56.03	0.000	0.000	-0.000	0.000	6.063

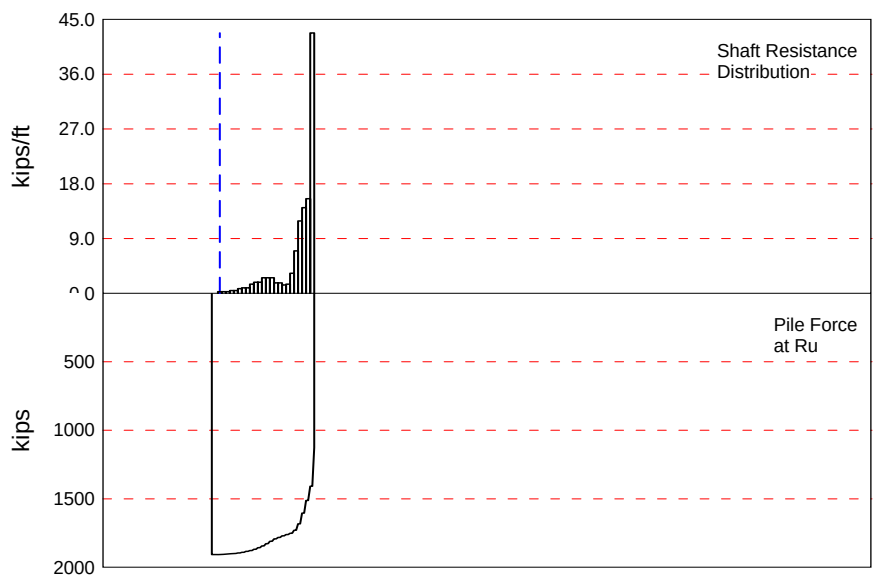
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 18.3 ms







Ru =	1907.1	kips
Rs =	779.9	kips
Rb =	1127.2	kips
Dy =	2.39	in
Dx =	2.44	in



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1907.1; along Shaft 779.9; at Toe 1127.2 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
				1907.1					
1	16.5	3.1	1.9	1905.2	1.9	0.60	0.10	0.147	0.295
2	23.1	9.7	1.9	1903.3	3.8	0.29	0.05	0.147	0.295
3	29.6	16.3	1.9	1901.4	5.7	0.29	0.05	0.147	0.295
4	36.2	22.9	3.1	1898.3	8.8	0.47	0.08	0.147	0.295
5	42.8	29.5	3.1	1895.2	11.9	0.47	0.08	0.147	0.295
6	49.4	36.1	5.2	1890.0	17.1	0.79	0.13	0.147	0.295
7	56.0	42.7	6.1	1883.9	23.2	0.93	0.15	0.147	0.295
8	62.6	49.3	6.1	1877.8	29.3	0.93	0.15	0.147	0.295
9	69.2	55.8	10.0	1867.8	39.3	1.52	0.25	0.147	0.295
10	75.8	62.4	12.1	1855.7	51.4	1.84	0.30	0.147	0.295
11	82.4	69.0	12.1	1843.6	63.5	1.84	0.30	0.147	0.295
12	88.9	75.6	17.0	1826.6	80.5	2.58	0.43	0.147	0.295
13	95.5	82.2	17.0	1809.6	97.5	2.58	0.43	0.147	0.295
14	102.1	88.8	17.0	1792.6	114.5	2.58	0.43	0.147	0.295
15	108.7	95.4	11.4	1781.2	125.9	1.73	0.29	0.147	0.295
16	115.3	102.0	11.4	1769.8	137.3	1.73	0.29	0.147	0.295
17	121.9	108.6	9.2	1760.6	146.5	1.40	0.23	0.147	0.280
18	128.5	115.1	10.0	1750.6	156.5	1.52	0.25	0.147	0.276
19	135.1	121.7	21.9	1728.7	178.4	3.32	0.55	0.147	0.271
20	141.6	128.3	46.0	1682.7	224.4	6.98	1.15	0.147	0.267
21	148.2	134.9	78.5	1604.2	302.9	11.92	1.97	0.147	0.263
22	154.8	141.5	92.8	1511.4	395.7	14.09	2.32	0.147	0.259
23	161.4	148.1	102.5	1408.9	498.2	15.56	2.57	0.147	0.254
24	168.0	154.7	281.7	1127.2	779.9	42.76	7.05	0.147	0.250
Avg. Shaft			32.5			5.04	0.83	0.147	0.263
Toe			1127.2				490.71	0.069	0.290

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.073	0.723
Unloading Quake	(% of loading quake)	100	112
Reloading Level	(% of Ru)	-99	53
Resistance Gap (included in Toe Quake) (in)			0.030

Soil Model Parameters/Extensions	Shaft	Toe
Soil Plug Weight (kips)		0.72

CAPWAP match quality	= 1.10	(Wave Up Match)	; RSA = 0
Observed: final set	= 0.043 in;	blow count	= 276 b/ft
Computed: final set	= 0.032 in;	blow count	= 372 b/ft
max. Top Comp. Stress	= 36.7 ksi	(T= 26.7 ms, max= 1.004 x Top)	
max. Comp. Stress	= 36.8 ksi	(Z= 16.5 ft, T= 27.4 ms)	
max. Tens. Stress	= -7.25 ksi	(Z= 135.1 ft, T= 66.6 ms)	
max. Energy (EMX)	= 142.6 kip-ft;	max. Measured Top Displ. (DMX)= 1.49 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	2200.3	-80.3	36.7	-1.34	142.61	20.2	1.491
2	6.6	2201.4	-87.6	36.7	-1.46	142.32	20.2	1.479
5	16.5	2208.2	-114.6	36.8	-1.91	141.32	20.1	1.440
8	26.4	2200.9	-137.7	36.7	-2.29	138.99	20.0	1.396
11	36.2	2204.4	-165.6	36.7	-2.76	136.83	19.9	1.345
14	46.1	2194.9	-187.0	36.6	-3.12	133.49	19.7	1.293
17	56.0	2194.3	-206.9	36.6	-3.45	129.98	19.5	1.234
20	65.9	2173.2	-222.1	36.2	-3.70	124.26	19.3	1.167
23	75.8	2169.5	-256.1	36.1	-4.27	119.44	18.9	1.102
26	85.6	2120.7	-283.3	35.3	-4.72	111.09	18.5	1.030
29	95.5	2104.5	-327.4	35.1	-5.46	103.60	18.1	0.947
32	105.4	2022.9	-345.8	33.7	-5.76	94.00	17.7	0.871
35	115.3	2015.9	-383.5	33.6	-6.39	88.21	17.4	0.792
38	125.2	1979.6	-405.6	33.0	-6.76	80.50	17.1	0.705
41	135.1	2017.0	-434.9	33.6	-7.25	72.71	16.4	0.603
44	144.9	1964.3	-378.0	32.7	-6.30	61.09	15.1	0.512
47	154.8	1990.1	-313.2	33.2	-5.22	48.83	12.8	0.411
48	158.1	1870.1	-239.9	31.2	-4.00	41.47	12.1	0.378
49	161.4	1971.9	-241.6	32.9	-4.03	39.69	11.1	0.349
50	164.7	1767.2	-185.2	29.4	-3.09	33.75	10.8	0.324
51	168.0	1742.7	-187.7	29.0	-3.13	23.71	11.0	0.294
Absolute	16.5			36.8			(T =	27.4 ms)

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
	135.1				-7.25		(T =	66.6 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	2812.3	2654.8	2497.4	2339.9	2182.4	2024.9	1867.4	1709.9	1552.4	1394.9
RX	2812.3	2654.8	2497.4	2339.9	2182.4	2025.9	1925.4	1874.0	1822.6	1772.6
RU	2834.7	2679.4	2524.2	2368.9	2213.7	2058.4	1903.2	1747.9	1592.6	1437.4

RAU = 1554.0 (kips); RA2 = 1629.4 (kips)

Current CAPWAP Ru = 1907.1 (kips); Corresponding J(RP)= 0.57; J(RX) = 0.64

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
20.39	26.46	2184.1	2203.1	2203.1	1.494	0.043	0.043	143.2	2235.4

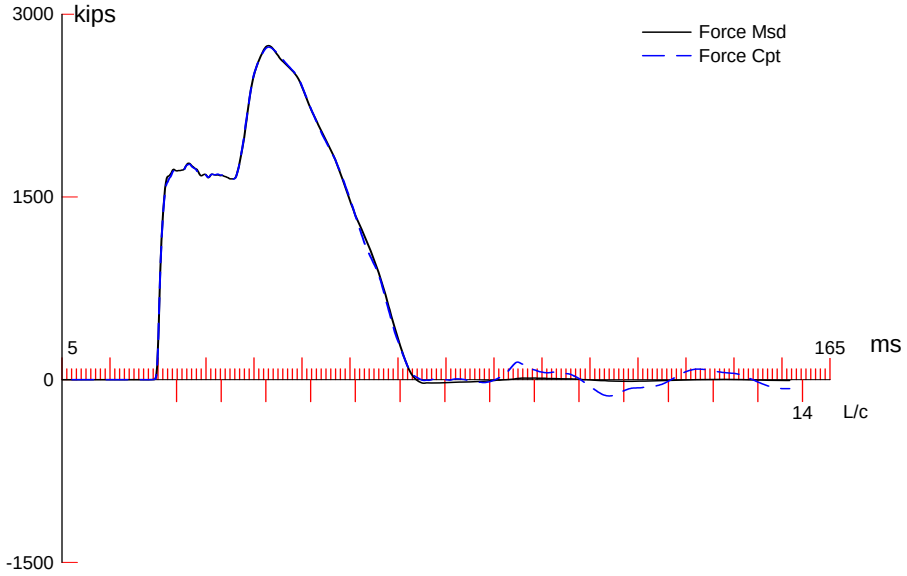
PILE PROFILE AND PILE MODEL

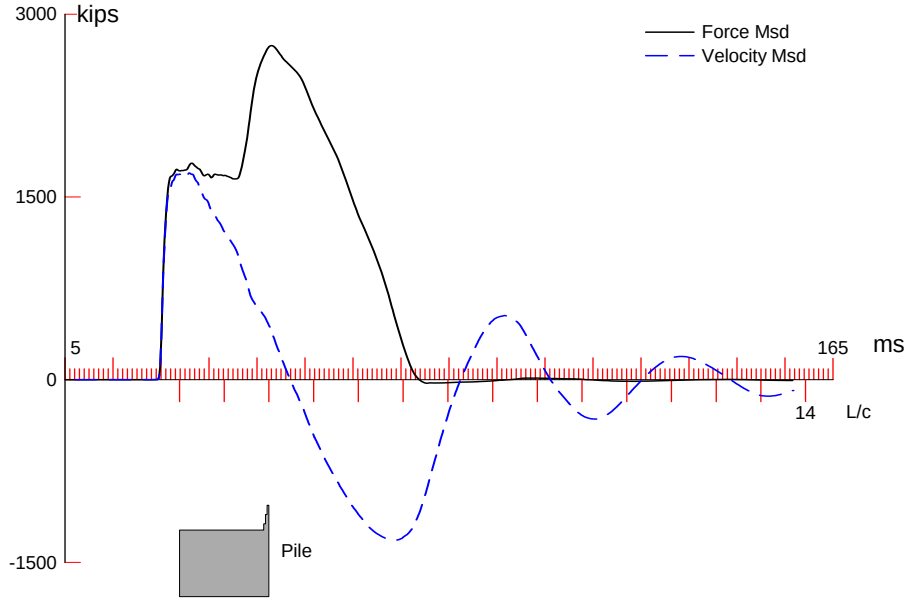
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
168.00	60.00	29992.2	492.000	6.063

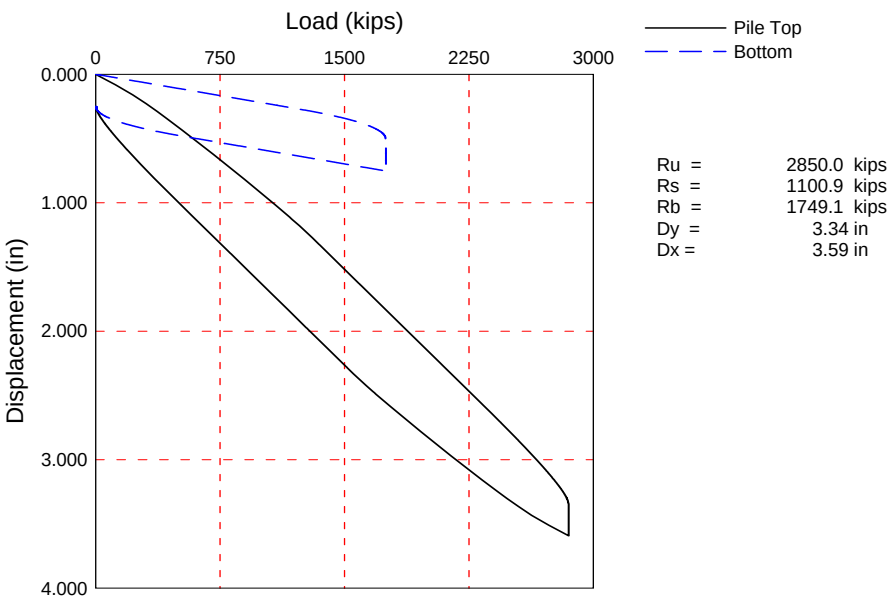
Toe Area 2.297 ft^2

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	3.29	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
49	161.41	122.09	14.01	0.000	0.000	-0.000	0.000	6.063
51	168.00	107.09	0.00	0.000	0.000	-0.000	0.000	6.063

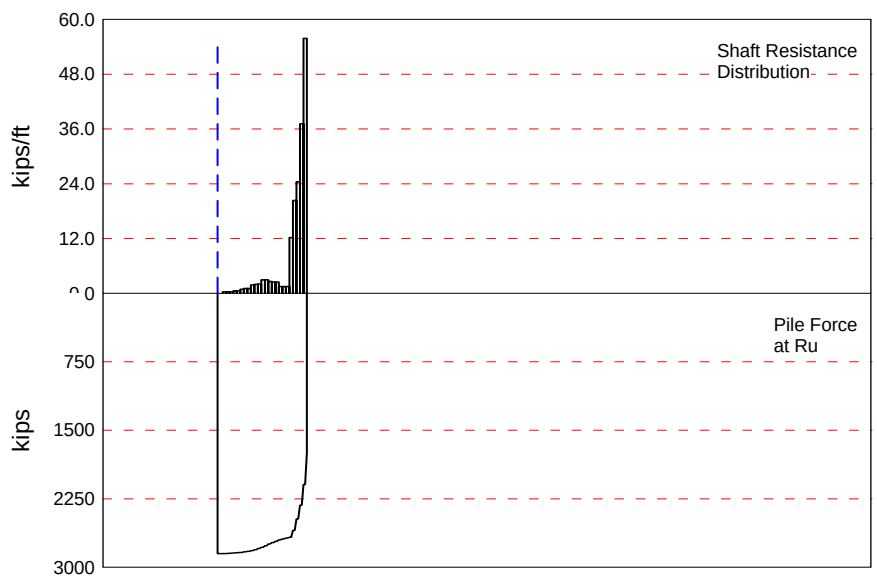
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 16807.9 ft/s, 2L/c 20.0 ms







Ru =	2850.0	kips
Rs =	1100.9	kips
Rb =	1749.1	kips
Dy =	3.34	in
Dx =	3.59	in



CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2850.0; along Shaft 1100.9; at Toe 1749.1 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	Quake in
				2850.0					
1	15.4	13.4	2.2	2847.8	2.2	0.16	0.03	0.178	0.067
2	21.5	19.6	2.2	2845.6	4.4	0.36	0.06	0.178	0.067
3	27.6	25.7	2.2	2843.4	6.6	0.36	0.06	0.178	0.067
4	33.8	31.8	3.4	2840.0	10.0	0.55	0.09	0.178	0.067
5	39.9	38.0	3.4	2836.6	13.4	0.55	0.09	0.178	0.067
6	46.1	44.1	5.6	2831.0	19.0	0.91	0.15	0.178	0.067
7	52.2	50.3	6.7	2824.3	25.7	1.09	0.18	0.178	0.067
8	58.3	56.4	6.7	2817.6	32.4	1.09	0.18	0.178	0.067
9	64.5	62.6	11.3	2806.3	43.7	1.84	0.30	0.178	0.067
10	70.6	68.7	12.3	2794.0	56.0	2.00	0.33	0.178	0.067
11	76.8	74.8	12.4	2781.6	68.4	2.02	0.33	0.178	0.067
12	82.9	81.0	18.4	2763.2	86.8	3.00	0.49	0.178	0.067
13	89.0	87.1	18.4	2744.8	105.2	3.00	0.49	0.178	0.067
14	95.2	93.3	16.0	2728.8	121.2	2.61	0.43	0.178	0.067
15	101.3	99.4	15.3	2713.5	136.5	2.49	0.41	0.178	0.067
16	107.5	105.5	15.3	2698.2	151.8	2.49	0.41	0.178	0.067
17	113.6	111.7	9.3	2688.9	161.1	1.51	0.25	0.178	0.043
18	119.8	117.8	9.3	2679.6	170.4	1.51	0.25	0.178	0.041
19	125.9	124.0	9.3	2670.3	179.7	1.51	0.25	0.178	0.040
20	132.0	130.1	75.1	2595.2	254.8	12.23	2.02	0.178	0.038
21	138.2	136.2	125.1	2470.1	379.9	20.37	3.36	0.178	0.037
22	144.3	142.4	150.2	2319.9	530.1	24.46	4.03	0.178	0.035
23	150.5	148.5	227.9	2092.0	758.0	37.11	6.12	0.178	0.034
24	156.6	154.7	342.9	1749.1	1100.9	55.84	9.21	0.178	0.032
Avg. Shaft			45.9			7.12	1.17	0.178	0.039
Toe			1749.1				761.44	0.052	0.387

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.825	0.843
Unloading Quake	(% of loading quake)	30	35
Reloading Level	(% of Ru)	100	1
Resistance Gap (included in Toe Quake) (in)			0.032

CAPWAP match quality	= 0.92	(Wave Up Match)	; RSA = 0
Observed: final set	= 0.250 in;	blow count	= 48 b/ft
Computed: final set	= 0.289 in;	blow count	= 41 b/ft

max. Top Comp. Stress	= 45.8 ksi	(T= 48.4 ms, max= 1.054 x Top)
max. Comp. Stress	= 48.3 ksi	(Z= 58.3 ft, T= 51.0 ms)
max. Tens. Stress	= -10.90 ksi	(Z= 132.0 ft, T= 87.5 ms)
max. Energy (EMX)	= 463.5 kip-ft;	max. Measured Top Displ. (DMX)= 3.17 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	3.1	2750.7	-138.7	45.8	-2.31	463.46	15.9	3.119
2	6.1	2769.5	-143.7	46.1	-2.39	457.58	15.8	3.065
5	15.4	2816.9	-158.1	46.9	-2.63	439.65	15.8	2.902
8	24.6	2845.7	-194.5	47.4	-3.24	418.11	15.6	2.737
11	33.8	2870.2	-268.7	47.8	-4.48	398.01	15.5	2.570
14	43.0	2885.7	-330.4	48.1	-5.50	374.96	15.2	2.403
17	52.2	2896.6	-386.4	48.3	-6.44	352.96	14.9	2.237
20	61.4	2893.7	-411.6	48.2	-6.86	327.33	14.6	2.071
23	70.6	2888.5	-450.2	48.1	-7.50	303.54	14.2	1.906
26	79.8	2859.8	-472.7	47.7	-7.88	274.53	13.8	1.741
29	89.0	2834.5	-510.3	47.2	-8.50	249.07	13.5	1.575
32	98.3	2791.9	-528.2	46.5	-8.80	219.10	13.2	1.409
35	107.5	2790.1	-571.9	46.5	-9.53	196.00	12.8	1.242
38	116.7	2787.1	-599.4	46.4	-9.99	171.23	12.1	1.074
41	125.9	2803.6	-637.1	46.7	-10.62	150.36	10.8	0.905
44	135.1	2729.3	-586.4	45.5	-9.77	121.10	8.7	0.739
47	144.3	2614.2	-493.7	43.6	-8.23	92.98	6.1	0.583
48	147.4	2459.6	-361.2	41.0	-6.02	77.62	5.3	0.535
49	150.5	2460.1	-368.7	41.0	-6.14	73.50	4.6	0.492
50	153.5	2231.8	-176.4	37.2	-2.94	58.14	4.2	0.457
51	156.6	2231.9	-182.9	37.2	-3.05	39.37	3.9	0.425
Absolute	58.3			48.3			(T =	51.0 ms)
	132.0				-10.90		(T =	87.5 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	2656.8	2594.9	2533.0	2471.1	2409.2	2347.3	2285.4	2223.5	2161.5	2099.6
RX	2972.5	2923.4	2874.7	2826.5	2796.5	2772.4	2754.0	2735.9	2718.0	2700.4
RU	2656.8	2594.9	2533.0	2471.1	2409.2	2347.3	2285.4	2223.5	2161.5	2099.6

RAU = 2676.2 (kips); RA2 = 2887.9 (kips)

Current CAPWAP Ru = 2850.0 (kips); Corresponding J(RP)= 0.00; J(RX) = 0.25

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
15.87	29.05	1605.5	1670.3	2744.1	3.167	0.250	0.250	469.9	3300.3

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	60.00	29992.2	492.000	6.063
156.60	60.00	29992.2	492.000	6.063

Toe Area 2.297 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	3.07	107.09	0.00	0.000	0.000	-0.000	0.000	6.063
49	150.46	117.09	9.34	0.000	0.000	-0.000	0.000	6.063
50	153.53	132.09	23.34	0.000	0.000	-0.000	0.000	6.063
51	156.60	147.09	37.35	0.000	0.000	-0.000	0.000	6.063

Pile Damping 1.0 %, Time Incr 0.183 ms, Wave Speed 16807.9 ft/s, 2L/c 18.6 ms

Appendix D

Pile driving logs

(BR-2-75) Pile Driving Log

WALSH
Walsh Construction

Project No: 10-3000	County-Route-Section: CUY-90-14.90
Bridge No: CUY-90-1651R	
SFN: PIER 9 LEFT DILE GROUP	

Substructure Unit: PIER 9	Date Driven: 4/14/11	Pile Number: 2-164
Pile Type: HP 18x204	Wall Thickness:	Hammer: D-80
Required Ultimate Bearing:	Batter: vertical	Drop Hammer Ram Weight:
Cutoff Elevation:	Ground Elevation:	No. of Splices: 2

Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure
108			142	32				
109	17		143	36				
110	13		144	43				
111	14		145	104				
112	15		146	152				
113	15		1"	14				
114	16		2"	14				
115	17		3"	16				
116	16		4"	14				
117	17		5"	17				
118	16		6"	24				
119	16		* 7"	25	*			
120	18		8"					
121	18		9"					
122	17		10"					
123	17		11"					
124	17							
125	18							
126	17							
127	17							
128	17							
129	17							
130	18							
131	17							
132	17							
133	17							
134	18							
135	19							
136	19							
137	20							
138	21							
139	25							
140	25							
141	31							

TOTAL 146'-7"

Effective Date: 2-14-2011
Revised Date: 2-14-2011

Walsh Form 28

(BR-2-75) Pile Driving Log

WALSH
Walsh Construction

Project No: 10-3000		County-Route-Section: CUY-90-14.90	
Bridge No: CUY-90-1651R			
SFN:			
Total Drive Length: 153'9"			
Substructure Unit: PIER 9	Date Driven: 4/18/1	Pile Number: 2-166 ¥ TEST ¥ APPLE	
Pile Type: HP 18 x 204	Wall Thickness:	Hammer: D-80	
Required Ultimate Bearing:	Batter:	Drop Hammer Ram Weight: 41632	
Cutoff Elevation: +/- 569.58	Ground Elevation: +/- 567.00	No. of Splices: 2	

Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure
100			134	29		# 146'	3	
101			135	29		1"	3	
102			136	30		2"	6	
103			137	30		3"	9	
104			138	30		4"	10	
105			139	33		5"	6	
106			140	34		6"	10	
107			141	39		7"	8	
108			142	47		8"	9	
109			143	56		9"	9	
110	30		144	52		10"	9	
111	30		145	63		11"	9	
112	30		146	90		# 147'	10	
113	30		147			1"	8	
114	31		148			2"	9	
115	29		149			3"	13	
116	32		150			4"	11	
117	32		151			5"	12	
118	34		152			6"	12	
119	30		153			7"	12	
120	33		154			8"	13	
121	31		155			9"	11	
122	31		156			10"	13	
123	33		157			11"	13	
124	33		158			# 148'	13	
125	33		159			1"	13	
126	32		160			2"	12	
127	32		161			3"	12	
128	32		162			4"	10	
129	33		163			5"	12	
130	29		164			6"	11	
131	28		165			7"	11	
132	29		166			8"	12	
133	30		167			9"	11	

Effective Date: 2-14-2011
Revised Date: 2-14-2011

X Danny Bl

Walsh Form 28

(BR-2-75) Pile Driving Log

WALSH
Walsh Construction

Project No: 10-3000	County-Route-Section: CUY-90-14.90
Bridge No: CUY-90-1651R	
SFN:	

Substructure Unit:	Date Driven:	Pile Number: Z-166
Pile Type:	Wall Thickness:	Hammer:
Required Ultimate Bearing:	Batter:	Drop Hammer Ram Weight:
Cutoff Elevation:	Ground Elevation:	No. of Splices:

Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure
10"	11		8"	10		6"		
11"	13		9"	12		7"		
# 149'	11		10"	12		8"		
1"	13		11"	10		9"		
2"	12		# 152'	11		10"		
3"	11		1"	12		11"		
4"	11		2"	9		#		
5"	13		3"	11		1"		
6"	12		4"	14		2"		
7"	10		5"	13		3"		
8"	13		6"	18		4"		
9"	12		7"	12		5"		
10"	8		8"	13		6"		
11"	13		9"	13		7"		
# 150'	12		10"	15		8"		
1"	12		11"	18		9"		
2"	11		# 153'	16		10"		
3"	10		1"	21		11"		
4"	12		2"	18		#		
5"	7		3"	18		1"		
6"	9		4"	17		2"		
7"	7		5"	19		3"		
8"	8		6"	18		4"		
9"	7		7"	16		5"		
10"	7		8"	20		6"		
11"	7		9"	20		7"		
# 151'	8		10"			8"		
1"	8		11"			9"		
2"	9		#			10"		
3"	9		1"			11"		
4"	9		2"			#		
5"	10		3"					
6"	8		4"					
7"	8		5"					

Effective Date: 2-14-2011
Revised Date: 2-14-2011

Walsh Form 28

(BR-2-75) Pile Driving Log

WALSH
Walsh Construction

Project No: 10-3000	County-Route-Section: CUY-90-14.90
Bridge No: CUY-90-1651R	
SFN:	
Total Drive Length:	

Substructure Unit: Pier 9	Date Driven: 4-20-11	Pile Number: 2-170
Pile Type: H 18x204	Wall Thickness:	Hammer: D-80
Required Ultimate Bearing:	Batter: —	Drop Hammer Ram Weight:
Cutoff Elevation:	Ground Elevation:	No. of Splices: 2

	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure
5-20	100	25		134	31		# 148		
6-21	101	22		135	37		1"	6	
7-21	102	24		136	35		2"	8	
8-20	103	27		137	39		3"	11	
9-21	104	26		138	39		4"	9	
	105	22		139	39		5"	9	
	106	22		140	37		6"	9	
55	107	23		141	40		7"	10	
35	108	23		142	39		8"	9	
	109	22		143	39		9"	10	
	110	20		144	40		10"	9	
	111	19		145	38		11"	10	
	112	20		146	36		# 149	9	
	113	21		147	42		1"	9	
	114	22		148	61		2"	9	
	115	26		149			3"	9	
	116	27		150			4"	9	
	117	25		151			5"	9	
	118	27		152			6"	9	
	119	26		153			7"	9	
	120	26		154			8"	8	
	121	29		155			9"	9	
	122	26		156			10"	12	
	123	28		157			11"	11	
	124	29		158			# 150	10	
	125	28		159			1"	10	
	126	30		160			2"	9	
	127	28		161			3"	10	
	128	27		162			4"	9	
	129	29		163			5"	11	
	130	31		164			6"	11	
	131	31		165			7"	13	
	132	38		166			8"	13	
	133	35		167			9"	11	

ive Date: 2-14-2011

Date: 2-14-2011

X

Walsh Form 28

(BR-2-75) Pile Driving Log

Walsh Construction

Project No: 10-3000	County-Route-Section: CUY-90-14.90
Bridge No: CUY-90-1651R	
SFN:	

Substructure Unit:	Date Driven:	Pile Number: 2-176
Pile Type: PIER 9	Wall Thickness:	Hammer: D-80
Required Ultimate Bearing:	Batter:	Drop Hammer Ram Weight:
Cutoff Elevation:	Ground Elevation:	No. of Splices: 2

Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure	Penetration	Blows/Ft	Stroke or Pressure
10"	8		8"	13		6"		
11"	9		9"	11		7"		
# 151	11		10"	15		8"		
1"	13		11"	16		9"		
2"	14		# 154	16		10"		
3"	12		1"	15		11"		
4"	11		2"	16		#		
5"	12		3"	15		1"		
6"	13		4"	16		2"		
7"	13		5"	18		3"		
8"	11		6"	19		4"		
9"	13		7"	22		5"		
10"	12		8"	23		6"		
11"	9		9"			7"		
# 152	13		10"			8"		
1"	11		11"			9"		
2"	9		#			10"		
3"	12		1"			11"		
4"	11		2"			#		
5"	10		3"			1"		
6"	11		4"			2"		
7"	11		5"			3"		
8"	9		6"			4"		
9"	10		7"			5"		
10"	9		8"			6"		
11"	10		9"			7"		
# 153	12		10"			8"		
1"	14		11"			9"		
2"	12		#			10"		
3"	10		1"			11"		
4"	11		2"			#		
5"	12		3"					
6"	10		4"					
7"	9		5"					

Appendix E

Soil Information

PIER 9

PROJECT: CLEVELAND CENTRAL VIADUCT	DRILLING FIRM / OPERATOR: PRO GEOTECH / FAY	DRILL RIG: MOBILE B-57	STATION / OFFSET: 151+80
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&W / TABOR	HAMMER: MOBILE AUTOMATIC	ALIGNMENT: CENTERLINE
PID: BR ID:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 1/14/11	ELEVATION: 582.0 (MSL) EOB: 176.0 ft.
START: 12/7/10 END: 12/13/10	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 81.3	LAT / LONG: Not Recorded

EXPLORATION ID B-043-1-10

PAGE 1 OF 4

MATERIAL DESCRIPTION AND NOTES	ELEV. 582.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	INST.
LOOSE, BLACK, **TOPSOIL**, WITH ROCK FRAGMENTS, MOIST	579.0	2.5	3 2 2	5	33	SS-S-1	-	-	-	-	-	-	-	-	-	-		
LOOSE, BROWN, **COARSE AND FINE SAND**, SILTY, TRACE ORGANICS, MOIST Fill	576.5	5.0	2 2 2	5	56	SS-S-2	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
LOOSE, **GRAVEL AND STONE FRAGMENTS**, ROCK FRAGMENTS, CONCRETE AND GLASS, MOIST Fill	573.5	7.5	2 1 2	4	6	SS-S-3	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
MEDIUM STIFF, DARK GRAY, **SANDY SILT**, WET	571.5	10.0	2 2 4	8	39	SS-S-4	-	-	-	-	-	-	-	-	-	-	A-4a (V)	
SOFT, GRAY, **SILT**, SLIGHLTY CLAYEY TO CLAYEY, SCATTERED WOOD FRAGMENTS, MOIST		W 12.5	1 2 1	4	100	SS-S-5	0.00	-	-	-	-	-	-	-	-	-	A-4b (V)	
		15.0 W	1 1 1	3	100	SS-S-6	0.50	-	-	-	-	-	-	-	-	164	A-4b (V)	
		17.5	1 for 12" 1	1	67	SS-S-7	0.50	-	-	-	-	-	-	-	-	-	A-4b (V)	
		20.0	1 1 1	3	122	SS-S-8	0.00	-	-	-	-	-	-	-	-	-	A-4b (V)	
		22.5																
		25.0	1 1 1	3	122	SS-S-9	0.5 - 1.25	-	-	-	-	-	34	28	6	36	A-4b (V)	
medium stiff beginning at 27 feet		27.5																
		30.0	1 2 2	5	100	SS-S-10	1.50	-	-	-	-	-	-	-	-	-	A-4b (V)	
					25	ST-S-11	1.25 -1.5	-	-	-	-	-	-	-	-	-	A-4b (V)	
	549.0	32.5																
SOFT, GRAY, **SILT**, TRACE TO SLIGHTLY FINE SANDY, WET		35.0	1 for 12" 1	1	100	SS-S-12	-	-	-	-	-	-	-	-	-	32	A-4b (V)	
	543.0	37.5																
LOOSE TO MEDIUM DENSE, DARK GRAY TO GRAY, **COARSE AND FINE SAND**, SLIGHTLY GRAVELLY TO GRAVELLY, TRACE SILT, WET		40.0	1 2 4	8	100	SS-S-13	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		42.5																
	535.0	45.0	6 7 5	16	67	SS-S-14	-	22	36	28	- 14 -		-	-	-	15	A-3a (V)	
STIFF, GRAY, **SILT AND CLAY**, MOIST		47.5																
			3	12	100	SS-S-15	1.25 - 2.0	-	-	-	-	-	-	-	-	-	A-6a (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 3/3/11 10:54 - D:\PWB\PROJECTS\CLEVELAND INNERBELT 21-1-21361\FIELD WORK PHASE_200\GINT PROJECT\21-21361_R2_03031

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 3/3/11 10:54 - D:\PWB\PROJECTS\CLEVELAND INNERBELT 21-1-21361\FIELD WORK PHASE_200\GINT PROJECT\21-21361_R2_03031

PID:	BR ID:	PROJECT: CLEVELAND CENTRAL VIADUCT	STATION / OFFSET: 151+80	START: 12/7/10	END: 12/13/10	PG 2 OF 4	B-043-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	INST.
STIFF, GRAY, **SILT AND CLAY**, MOIST *(continued)*	532.0		3 6															
		52.5																
		55.0	3 5 8	18	100	SS-S-16	1.75 - 4.0	-	-	-	-	-	33	21	12	21	A-6a (V)	
	525.0																	
STIFF TO VERY STIFF, GRAY, **SILTY CLAY**, MOIST		57.5																
		60.0			75	ST-S-17	2.0 -2.25	-	-	-	-	-	-	-	-	-	A-6b (V)	
		62.5																
		65.0	3 5 8	18	72	SS-S-18	0.75 - 1.25	-	-	-	-	-	-	-	-	-	A-6b (V)	
		67.5																
shale fragments from 68.5 feet to 82 feet		70.0			100	ST-S-19	1.25 - 1.5	-	-	-	-	-	-	-	-	-	A-6b (V)	
		72.5																
		75.0	3 4 6	14	100	SS-S-20	1.0 - 1.25	-	-	-	-	-	-	-	-	24	A-6b (V)	
		77.5																
		80.0	3 5 7	16	100	SS-S-21	1.25	-	-	-	-	-	-	-	-	-	A-6b (V)	
		82.5																
		85.0	3 7 10	23	100	SS-S-22	2.0 - 2.5	-	-	-	-	-	-	-	-	-	A-6b (V)	
		87.5																
		90.0	3 5 6	15	100	SS-S-23	1.0 - 1.25	-	-	-	-	-	37	21	16	26	A-6b (V)	
		92.5																
	488.5																	
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**		95.0	3 3 4	9	133	SS-S-24	0.50	-	-	-	-	-	-	-	-	-	A-6a (V)	
					121	ST-S-25	0.50	-	-	-	-	-	-	-	-	-	A-6a (V)	
shale fragments from 97 feet to 102 feet		97.5																
		100.0	3 4 6	14	100	SS-S-26	0.5 - 0.75	-	-	-	-	-	-	-	-	-	A-6a (V)	
		102.5																

PID:	BR ID:	PROJECT: CLEVELAND CENTRAL VIADUCT	STATION / OFFSET: 151+80	START: 12/7/10	END: 12/13/10	PG 3 OF 4	B-043-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV. 478.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	INST.
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY** *(continued)*	475.5	105.0	3 4 5	12	100	SS-S-27	0.5 - 1.5	-	-	-	-	-	32	21	11	28	A-6a (V)	
MEDIUM DENSE, GRAY, **COARSE AND FINE SAND**, SILTY, SLIGHTLY FINE GRAVELLY, SHALE FRAGMENTS, WET	471.0	107.5																
HARD, GRAY, **SILT**, TRACE SAND, MOIST		110.0	4 10 15	34	100	SS-S-28	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
thin silty sand layer at 114 feet	465.0	112.5																
DENSE TO VERY DENSE, GRAY, **COARSE AND FINE SAND**, SILTY, SLIGHTLY GRAVELLY TO GRAVELLY, WITH SCATTERED GRAVEL LAYERS, WET		115.0	15 23 32	75	94	SS-S-29	-	-	-	-	-	-	-	-	-	-	A-4b (V)	
This layer was previously assumed to be CSF. Based on new data, soil at these depths is borderline CSF/CSG. Parameters for very dense CSF and CSG are relatively similar. Therefore, the existing stratigraphy and parameters will not be revised.		117.5																
		120.0	18 50 50	136	89	SS-S-30	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		122.5																
		125.0	17 39 40	107	83	SS-S-31	-	61	18	12	- 9 -		-	-	-	10	A-3a (V)	
		127.5																
		130.0	17 21 21	57	83	SS-S-32	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		132.5																
		135.0	12 17 21	51	83	SS-S-33	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		137.5																
		140.0	13 22 29	69	100	SS-S-34	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		142.5																
		145.0	18 21 21	57	100	SS-S-35	-	14	52	18	- 16 -		-	-	-	15	A-3a (V)	
		147.5																
	431.0	150.0	19 25 35	81	89	SS-S-36	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
HARD, GRAY, **SILT**, CLAYEY, TRACE TO ABUNDANT SHALE FRAGMENTS, MOIST		152.5																
		155.0	11 20 29	66	100	SS-S-37	4.50	-	-	-	-	-	-	-	-	-	A-4b (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 3/3/11 10:54 - D:\PWB\PROJECTS\CLEVELAND INNERBELT 21-1-21361\FIELD WORK PHASE_200\GINT PROJECT\21-21361_R2_03031

PID:	BR ID:	PROJECT: CLEVELAND CENTRAL VIADUCT	STATION / OFFSET: 151+80	START: 12/7/10	END: 12/13/10	PG 4 OF 4	B-043-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	INST.
	424.9																	
	423.3	TR																
SHALE, GRAY, SEVERELY WEATHERED, VERY WEAK, LAMINATED.		157.5	50/3"	-	100	SS-S-38	-	-	-	-	-	-	28	20	8	13	A-4b (V)	
		160.0																
		162.5	50/4"	-	100	SS-S-39	-	-	-	-	-	-	-	-	-	-	Rock (V)	
	417.0																	
SHALE, DARK GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, WEAK TO SLIGHTLY STRONG, THINLY LAMINATED, WIDELY SPACE BEDDING FRACTURES; RQD 27.3%, REC 82.1%.		165.0	0		39	NQ2-1											CORE	
		167.5	9		89	NQ2-2											CORE	
		170.0																
		172.5	52		89	NQ2-3											CORE	
	406.0	175.0 EOB																

This layer was previously described as hard PCS. However, the log has been revised to refer to the layer as severely weathered shale. As the material generally consists of weathered shale fragments in a soil-like matrix, in our opinion, the material will essentially behave as a soil. Therefore, we will not revise our existing stratigraphy and parameters.

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: NOT RECORDED