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To: Mr. Joel Halterman
Of: Walsh Construction
Date: March 9, 2012
From: Benjamin White
GRL Job No. 115058-30
Re: Dynamic Testing Results; ODOT 3000(10) East Bank Bulkhead Walls

Mr. Halterman:

This report summarizes the dynamic testing performed at the above referenced site on February 23, March 1 & 8, 2012. As requested, GRL performed dynamic testing on six piles in the East bank bulkhead walls. Table 1 presents the Case Method results and Table 2 presents the results from CAPWAP analyses for the test piles. The complete Case Method and CAPWAP analyses results are shown in Appendix B and C, respectively.

The tested piles were HP 12x53 steel H-piles. The piles were reportedly fabricated from ASTM A572 Gr.50 steel which has a minimum yield strength of 50 ksi. The plans indicate the following ultimate bearing values.

Type 1 piles (Southern Bulkhead)
Battered Piles – 164 kips in compression
Vertical Piles – 150 kips in uplift resistance

Type 2 Piles (Northern Bulkhead)
Battered Piles – 190 kips in compression
Vertical Piles – 194 kips in uplift resistance

The estimation of uplift resistance through dynamic testing is performed by reducing the shaft resistance calculated from CAPWAP analysis by 20% to account for compression loading and Poisson's ratio effects on the pile. In addition, the resistance from the bottom element of the pile model (approximately 6.0 ft) is removed from the total shaft resistance as the resistance from this element occurs at nearly the same time as the end bearing resistance and it is very difficult to differentiate between the two.

The test piles were driven using an APE D19-42 single-acting diesel hammer. This hammer has a four step fuel pump with 4 being the maximum fuel setting. The piles were driven and restruck with the hammer on fuel setting 4. The energy transferred to the piles near the end of driving ranged from 11.9 to 15.5 kip-ft at average hammer strokes of 6.1 to 6.3 ft. These energy levels correspond to rated transfer efficiencies of 25 to 33% of the maximum rated energy of 47.1 kip-ft.

Measured compressive stresses near the pile top were approximately 18.8 to 25.5 ksi near the end of driving. CAPWAP analysis indicated the maximum compression stresses throughout the pile were approximately 3% higher than the measured stresses at the gage location. The compression stresses were below the recommended stress limits of 90% of the yield strength of the steel or 45 ksi. The force and velocity records did not indicate any detectable damage below the location of the gages.

Piles 151 and 162 in the Northern bulkhead (Type 2 piles) were tested during initial driving on February 23 to depths of 80 and 65 ft, respectively. Piles 188 and 196 in the Southern bulkhead (Type 1 piles) were tested during initial driving on March 1 to depths of 65 and 60 feet, respectively. At the end of drive, the estimated compressive capacities of these piles ranged from 72 to 103 kips. Restrike testing was performed on piles 150 and 169 in the Northern bulkhead and piles 188 and 196 in the Southern bulkhead on March 8, 2012.

<u>Type 1 Piles</u>
CAPWAP analysis of data collected during restrike of 196 indicated a mobilized capacity of 176 kips with 165 kips in shaft resistance and 11 kips in end bearing. This capacity exceeds the required ultimate bearing value

of 164 kips. Based on the results of this test pile, GRL suggests that the battered type 1 piles be driven to at least 23 blows/ft with a corresponding minimum hammer stroke of 6.3 ft. In addition, because these criteria rely on soil set-up to achieve the ultimate bearing value, the piles should be driven to at least 60 ft penetration depth.

CAPWAP analysis of data collected during restrike of 188 indicated a mobilized capacity of 162 kips with 146 kips in shaft resistance and 17 kips in end bearing. To estimate the uplift resistance, the bottom element of resistance from CAPWAP was removed (16.8 kips) from the total shaft resistance. The remaining shaft resistance was reduced by 20% to account for Poisson's ration effects. The total estimated uplift resistance for this pile is 103 kips. This estimate of uplift resistance is below the required uplift resistance of 150 kips.

Type 2 Piles

CAPWAP analysis of data collected during restrike of 169 indicated a mobilized capacity of 194 kips with 186 kips in shaft resistance and 8 kips in end bearing. This capacity exceeds the required ultimate bearing value of 190 kips. Based on the results of this test pile, GRL suggests that the battered type 2 piles be driven to at least 30 blows/ft. No end of drive stroke measurement was provided to GRL, however, if that information is available, it should be used as the minimum stroke criteria. In addition, because these criteria rely on soil set-up to achieve the ultimate bearing value, the piles should be driven to at least 65 ft penetration depth.

CAPWAP analysis of data collected during restrike of 150 indicated a mobilized capacity of 178 kips with 170 kips in shaft resistance and 8 kips in end bearing. To estimate the uplift resistance, the bottom element of resistance from CAPWAP was removed (13.4 kips) from the total shaft resistance. The remaining shaft resistance was reduced by 20% to account for Poisson's ration effects. The total estimated uplift resistance for this pile is 125 kips. This estimate uplift resistance is below the required uplift resistance of 194 kips.

If you have questions or comments please contact us at (216) 292-3076.

Sincerely,
GRL Engineers, Inc.

Benjamin White, P.E.

Brandon Phetteplace, E.I.

Table 1: Summary of Case Method Results

ODOT 3000(10) - Green Bulkhead Walls — Hammer: APE D19-42

Pile No.	Test Date	Pile Orientation [Type Pile]	Test Type [1]	Penetration Depth [2] (ft)	Blow Count [3] blows/set	Hammer Stroke [4] (ft)	Transf'd Energy (kip-ft)	Max. Compressive [5] Force (kips)	Max. Compressive Stress (ksi)	Case Method Capacity (kips)	CAPWAP Mobilized Capacity (kips)	Estimated Tensile Capacity [6] (kips)
150	8-Mar-12	Vertical [2]	BOR	80	5 / 1"	7.3	17.1	452	29.1	209	178	125
151	23-Feb-12	Vertical [2]	EOID	80	26 / 1'	6.3	15.5	396	25.5	103	-	-
162	23-Feb-12	Battered [2]	EOID	65	30 / 1'	6.1	11.9	292	18.8	74	-	-
169	8-Mar-12	Battered [2]	BOR	65	4 / 1"	7.6	14.8	430	27.8	169	194	-
188	1-Mar-12	Vertical [1]	EOID	65	21 / 1'	6.0	15.1	384	24.7	72	-	-
	8-Mar-12		BOR	65	3 / 1"	7.5	12.1	344	27.7	159	162	103
196	1-Mar-12	Battered [1]	EOID	60	23 / 1'	6.3	14.0	359	23.1	90	-	-
	8-Mar-12		BOR	60	3 / 1"	7.5	13.1	393	25.4	172	176	-

Notes:

1 - BOR: beginning of restrike/redrive; EOID: end of initial drive; EOR: end of restrike/redrive
2 - Depth below existing grade
3 - As observed by project inspector
4 - Stroke Calculated based on the time between impacts
5 - Stress from uniform axial average
6 - Estimated from total shaft resistance from CAPWAP minus the bottom element of resistance times 0.8
See report for further description

Table 2: Summary of CAPWAP Results

Pile No.	Test Date	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)
150	8-Mar-12	5 / 1"	80	178	170	8	0.34	0.02	0.30	0.50
169	8-Mar-12	4 / 1"	65	194	186	8	0.26	0.02	0.21	0.49
188	8-Mar-12	3 / 1"	65	162	146	17	0.26	0.02	0.21	0.58
196	8-Mar-12	3 / 1"	60	176	165	11	0.30	0.04	0.30	0.48

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.

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 <u>sufficient waiting time</u> 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

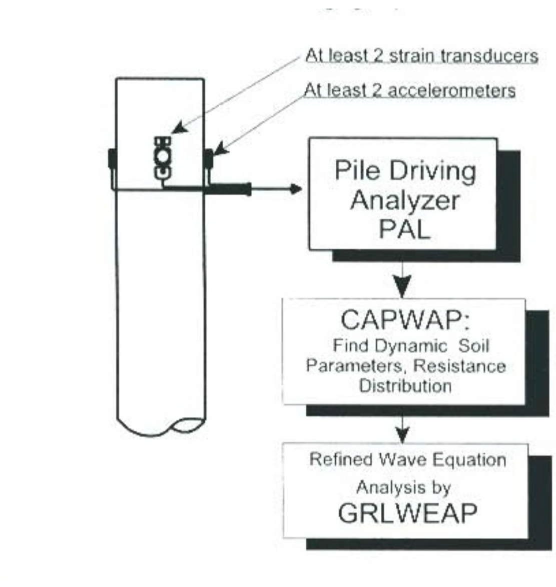


Figure 1. Block Diagram of Refined Wave Equation Analysis

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 ($\rho\, c^2$)
A = pile cross sectional area

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

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

The dynamic component may be computed from a soil damping factor, J, and 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.

$$W_u = ½[F(t) - Zv(t)] \qquad (4)$$

$$W_d = ½[F(t) + Zv(t)] \qquad (5)$$

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

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.

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

with

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

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:

$$E(t) = \int_{o}^{t} F(\tau)v(\tau)\, d\tau \qquad (8a)$$

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

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

This relationship can also be expressed in terms of stress

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

or strain

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

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

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, **<u>restrike testing</u> 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

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 150 Restrike
OP: BAW

HP 12X53
Test date: 8-Mar-2012

AR: 15.50 in^2
LE: 83.0 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
5	80.08	60	AV5	29.1	29.8	24.3	17.1	452	7.3	187
			MAX	31.7	31.9	26.1	20.8	491	7.8	209
9	80.17	48	AV4	31.5	31.8	25.9	20.3	488	7.7	198
			MAX	32.2	32.3	26.3	21.1	499	7.9	201
11	80.25	24	AV2	32.0	32.2	26.1	21.1	496	7.8	192
			MAX	32.3	32.3	26.2	21.2	500	7.8	192
			Average	30.5	30.9	25.2	19.0	473	7.5	192
			Maximum	32.3	32.3	26.3	21.2	500	7.9	209

Total number of blows analyzed: 11

Time Summary

Drive 14 seconds 10:18:30 AM - 10:18:44 AM (3/8/2012) BN 1 - 11

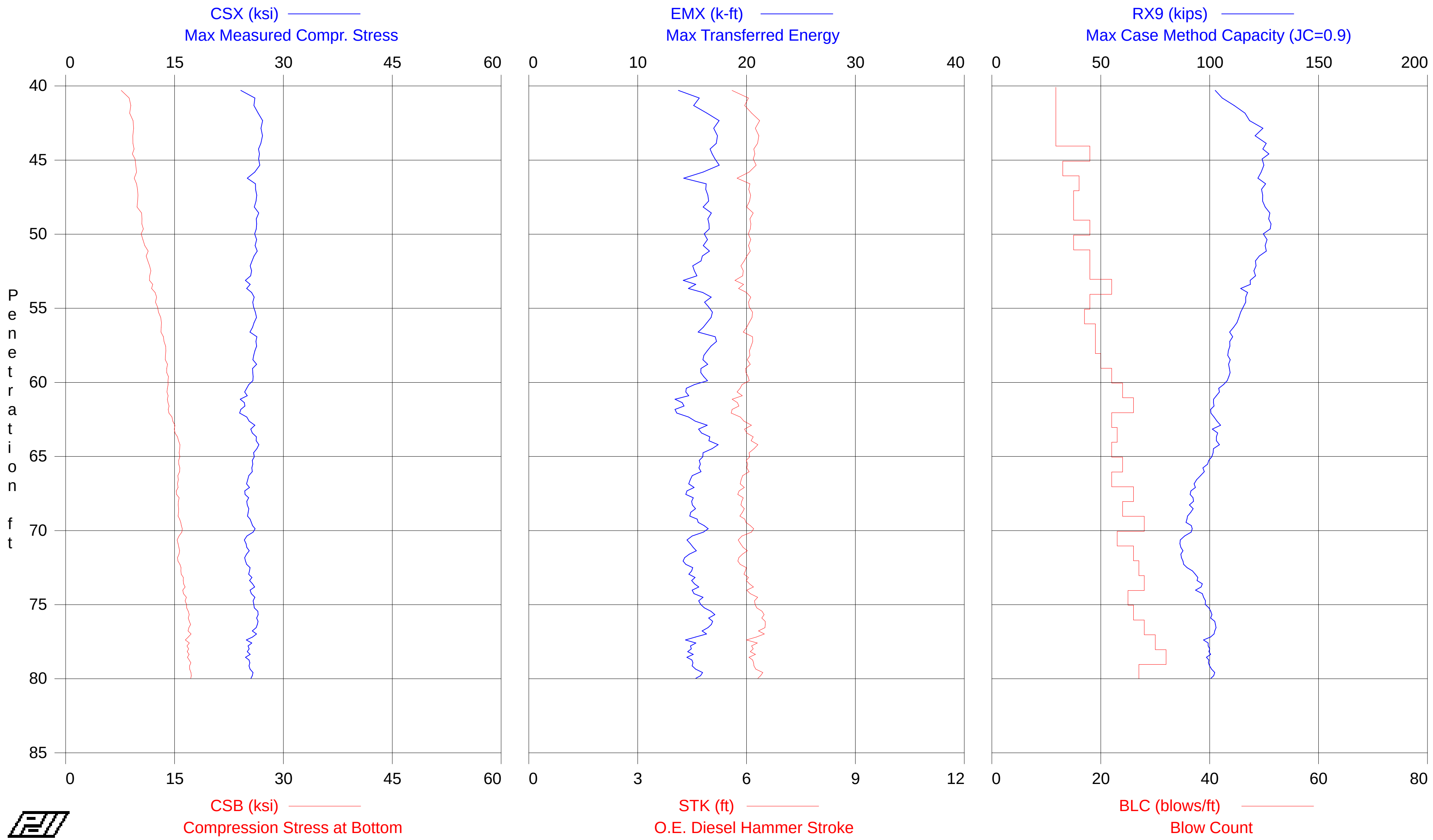
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 9-Mar-2012

Test date: 23-Feb-2012

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 151

HP 12X53



I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 151 HP 12X53
OP: BAW Test date: 23-Feb-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 83.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
12	41.00	12	AV11	24.8	28.0	8.1	14.3	385	5.8	104
			MAX	27.6	31.2	9.4	18.9	428	6.7	114
24	42.00	12	AV12	26.3	29.6	8.9	15.8	408	6.1	114
			MAX	28.1	31.1	9.7	19.1	435	6.6	125
36	43.00	12	AV12	27.0	30.6	9.4	17.3	419	6.3	121
			MAX	27.9	31.5	10.1	18.8	432	6.6	128
48	44.00	12	AV12	27.0	30.4	9.3	17.3	419	6.3	124
			MAX	28.3	31.7	10.1	18.7	439	6.8	130
66	45.00	18	AV18	26.6	29.8	9.3	16.9	413	6.2	126
			MAX	27.7	31.0	10.0	18.7	429	6.5	130
79	46.00	13	AV13	26.4	29.6	9.7	16.7	409	6.2	125
			MAX	27.7	30.9	10.3	18.5	430	6.5	128
95	47.00	16	AV16	25.8	28.8	9.7	15.6	400	6.0	124
			MAX	27.7	30.6	10.6	18.5	429	6.5	129
110	48.00	15	AV15	26.2	29.5	9.9	16.3	406	6.1	125
			MAX	27.0	30.2	10.5	17.5	418	6.3	128
125	49.00	15	AV15	26.3	29.3	10.2	16.4	408	6.1	127
			MAX	27.5	31.2	11.0	18.8	426	6.5	130
143	50.00	18	AV18	26.3	29.5	10.5	16.5	407	6.1	128
			MAX	27.6	30.6	11.4	18.1	428	6.5	131
158	51.00	15	AV15	26.2	29.6	10.8	16.1	405	6.1	126
			MAX	27.1	31.1	11.3	17.1	420	6.4	129
176	52.00	18	AV18	26.0	29.2	11.3	16.1	404	6.0	124
			MAX	27.9	32.0	12.2	18.9	433	6.7	129
194	53.00	18	AV18	25.5	28.5	11.6	15.2	395	5.9	121
			MAX	27.3	30.6	12.3	17.1	423	6.3	126
216	54.00	22	AV22	25.2	28.3	11.9	15.0	390	5.8	117
			MAX	26.8	30.2	13.0	17.7	416	6.4	124
234	55.00	18	AV18	25.8	29.0	12.5	16.4	400	6.1	117
			MAX	27.3	30.8	13.3	18.2	422	6.4	120
251	56.00	17	AV17	26.2	29.4	13.0	16.7	406	6.1	114
			MAX	26.8	30.2	13.4	17.8	416	6.3	118
270	57.00	19	AV19	25.8	29.3	13.2	16.2	400	6.0	111
			MAX	27.9	31.4	14.0	18.9	432	6.6	116
289	58.00	19	AV19	26.2	29.5	13.7	16.8	405	6.1	110
			MAX	27.4	30.8	14.2	18.1	425	6.5	114

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 151 HP 12X53
OP: BAW Test date: 23-Feb-2012

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
309	59.00	20	AV20	26.0	29.6	13.8	16.1	402	6.1	111
			MAX	27.2	30.9	14.5	17.6	421	6.4	113
331	60.00	22	AV22	25.8	29.4	14.0	16.0	400	6.0	110
			MAX	26.6	30.8	14.6	17.5	413	6.3	115
355	61.00	24	AV24	25.0	28.4	14.1	14.7	387	5.8	106
			MAX	26.1	29.7	14.6	15.9	404	6.2	110
381	62.00	26	AV26	24.3	27.1	14.1	13.8	377	5.7	103
			MAX	25.9	28.7	14.5	15.6	402	6.1	105
403	63.00	22	AV22	25.2	27.7	14.7	15.1	390	5.9	105
			MAX	27.0	29.7	15.7	17.2	418	6.4	109
426	64.00	23	AV23	25.9	28.6	15.2	16.1	402	6.1	105
			MAX	27.4	30.3	16.0	17.7	424	6.5	108
448	65.00	22	AV22	26.3	28.9	15.7	16.6	407	6.2	105
			MAX	27.4	30.1	16.2	18.1	425	6.5	109
472	66.00	24	AV24	25.8	28.1	15.7	15.8	399	6.0	101
			MAX	27.0	29.5	16.3	17.1	418	6.3	104
494	67.00	22	AV22	25.2	27.2	15.5	15.0	390	5.9	97
			MAX	26.1	28.3	15.9	15.8	404	6.1	104
520	68.00	26	AV26	24.9	27.1	15.4	14.8	387	5.8	94
			MAX	26.3	29.2	16.2	16.0	408	6.1	98
544	69.00	24	AV24	25.2	26.9	15.6	15.1	390	5.9	94
			MAX	26.0	28.1	15.9	16.3	403	6.2	99
572	70.00	28	AV28	25.6	27.2	15.9	15.8	397	6.0	93
			MAX	26.9	28.4	16.6	17.0	417	6.5	97
595	71.00	23	AV23	25.0	26.7	15.6	15.1	388	5.9	90
			MAX	26.0	27.7	16.0	16.3	403	6.2	96
621	72.00	26	AV26	25.0	26.5	15.6	14.8	387	5.9	89
			MAX	25.9	27.7	16.4	16.3	402	6.2	92
648	73.00	27	AV27	25.1	26.8	15.8	14.7	389	5.9	92
			MAX	26.1	27.9	16.2	16.0	405	6.2	96
676	74.00	28	AV28	25.7	27.4	16.3	15.2	398	6.1	96
			MAX	27.0	28.6	16.9	16.9	419	6.5	99
701	75.00	25	AV25	25.9	27.1	16.5	15.7	401	6.2	99
			MAX	26.7	27.9	17.1	16.7	414	6.5	102
727	76.00	26	AV26	26.3	27.5	16.8	16.5	407	6.4	102
			MAX	27.0	28.3	17.4	17.6	418	6.6	105
755	77.00	28	AV28	26.2	27.6	17.0	16.5	406	6.5	104
			MAX	26.9	28.4	17.6	17.5	416	6.7	107
785	78.00	30	AV30	25.4	26.9	16.8	15.0	394	6.2	101
			MAX	26.8	28.8	17.8	16.8	415	6.7	104
817	79.00	32	AV32	25.2	27.0	17.0	14.9	391	6.2	101
			MAX	26.5	28.3	17.7	16.7	411	6.7	103

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 151 HP 12X53
OP: BAW Test date: 23-Feb-2012

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
844	80.00	27	AV27	25.5	27.4	17.2	15.5	396	6.3	103
			MAX	26.2	28.0	17.6	16.3	407	6.6	107

Time Summary

Drive 17 minutes 45 seconds 8:26:30 AM - 8:44:15 AM (2/23/2012) BN 1 - 849

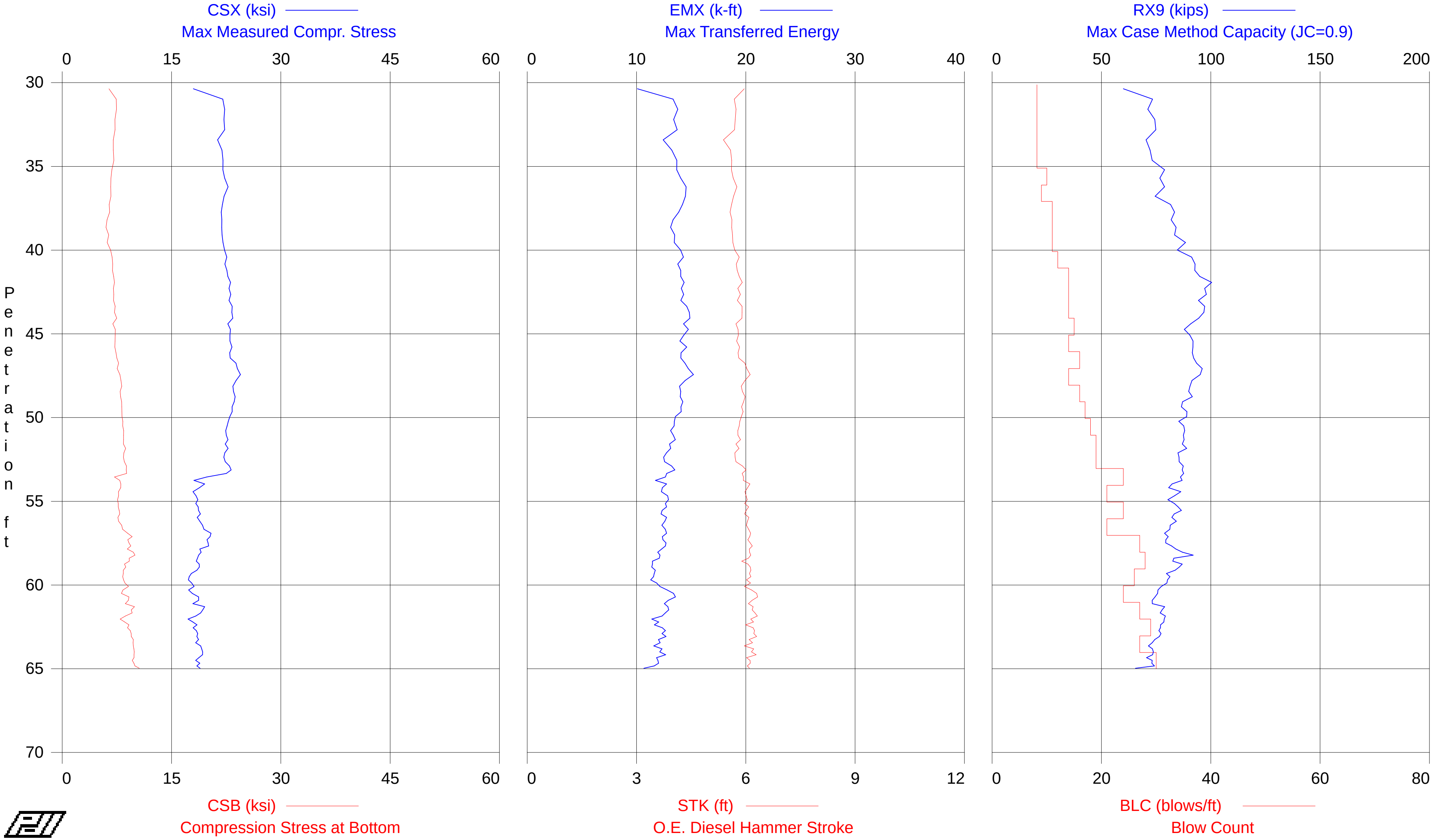
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 9-Mar-2012

Test date: 23-Feb-2012

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 162

HP 12X53



I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 162 HP 12X53
OP: BAW Test date: 23-Feb-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 68.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
9	31.00	8	AV8	19.5	24.4	6.8	11.3	303	5.9	65
			MAX	23.0	26.1	7.8	16.0	357	6.2	77
17	32.00	8	AV8	22.2	24.7	7.4	13.5	344	5.7	72
			MAX	23.2	25.8	7.7	14.4	360	6.0	80
25	33.00	8	AV8	22.1	24.5	7.2	13.4	343	5.7	75
			MAX	23.3	26.1	7.8	14.8	361	6.1	84
33	34.00	8	AV8	21.8	24.1	7.1	13.2	338	5.5	73
			MAX	23.6	26.5	7.6	15.8	365	6.1	81
42	35.00	8	AV9	22.0	24.3	7.0	13.3	341	5.6	75
			MAX	22.9	25.3	7.6	14.5	355	5.9	82
52	36.00	10	AV10	22.3	24.1	6.8	14.1	346	5.7	78
			MAX	23.7	26.3	7.1	15.6	367	6.0	84
61	37.00	9	AV9	22.3	24.1	6.7	14.4	346	5.7	77
			MAX	23.3	25.3	7.1	16.0	361	5.9	85
72	38.00	11	AV11	21.9	23.8	6.4	14.0	339	5.6	82
			MAX	22.7	24.8	6.9	15.1	353	5.8	89
83	39.00	11	AV11	21.9	23.5	6.1	13.2	339	5.6	85
			MAX	22.6	24.2	6.4	14.2	350	5.8	95
94	40.00	11	AV11	22.1	23.9	6.3	13.6	342	5.6	87
			MAX	22.6	24.8	6.7	14.6	350	5.8	93
106	41.00	12	AV12	22.5	23.7	6.9	14.2	349	5.8	92
			MAX	23.1	24.4	7.3	15.0	358	6.0	101
120	42.00	14	AV14	22.8	23.5	7.0	14.2	354	5.8	96
			MAX	23.8	24.7	7.4	15.4	369	6.2	104
134	43.00	14	AV14	22.9	23.2	7.0	14.1	355	5.8	98
			MAX	23.7	24.3	7.3	15.5	367	6.0	106
148	44.00	14	AV14	23.3	23.6	7.2	14.7	361	5.9	97
			MAX	24.1	24.4	7.7	15.6	374	6.1	102
163	45.00	15	AV15	23.1	23.3	7.2	14.7	358	5.8	92
			MAX	23.9	24.2	7.7	15.3	370	6.0	102
177	46.00	14	AV14	23.0	23.3	7.3	14.2	357	5.8	92
			MAX	23.7	23.9	7.8	15.0	367	5.9	94
193	47.00	16	AV16	23.5	23.9	7.5	14.3	364	5.9	94
			MAX	24.5	24.8	8.2	15.2	379	6.2	105
207	48.00	14	AV14	24.1	24.6	7.9	14.8	373	6.0	94
			MAX	25.1	25.8	8.5	16.2	389	6.3	99

I-90 Innerbelt East Bank Bulkhead Walls - Type 2 Pile 162 HP 12X53
OP: BAW Test date: 23-Feb-2012

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
223	49.00	16	AV16	23.5	24.1	8.0	14.0	365	5.9	91
			MAX	24.5	25.0	8.4	14.7	380	6.2	99
240	50.00	17	AV17	23.3	23.5	8.2	14.0	361	5.9	89
			MAX	24.1	24.4	8.8	14.8	373	6.1	94
258	51.00	18	AV18	22.6	22.8	8.3	13.3	350	5.8	88
			MAX	23.4	23.6	8.7	14.1	363	6.1	94
277	52.00	19	AV19	22.6	22.8	8.5	13.2	351	5.8	89
			MAX	23.8	24.1	8.9	14.4	369	6.2	96
296	53.00	19	AV19	22.5	22.6	8.6	12.8	349	5.8	87
			MAX	23.9	24.0	9.3	14.4	370	6.2	91
320	54.00	24	AV24	20.7	21.9	8.2	12.7	321	6.0	89
			MAX	24.0	24.1	9.4	14.7	372	6.3	94
341	55.00	21	AV21	18.5	22.4	7.8	12.6	286	6.0	86
			MAX	19.8	23.8	8.6	13.5	306	6.2	91
365	56.00	24	AV24	18.7	24.2	7.8	12.6	290	6.0	88
			MAX	19.8	25.7	8.3	13.7	307	6.4	97
386	57.00	21	AV21	19.4	24.9	8.2	12.5	301	6.1	86
			MAX	21.2	26.4	9.3	13.7	328	6.4	90
413	58.00	27	AV27	19.8	23.6	9.2	12.4	306	6.1	86
			MAX	21.2	25.3	9.8	13.2	329	6.4	100
441	59.00	28	AV28	18.7	22.3	9.2	11.7	290	6.1	89
			MAX	19.6	24.1	10.4	12.8	303	6.3	105
467	60.00	26	AV26	17.8	20.7	8.5	11.6	275	6.1	84
			MAX	19.4	22.4	9.4	12.5	301	6.3	89
491	61.00	24	AV24	18.1	21.1	8.7	13.0	281	6.2	79
			MAX	19.3	22.8	9.7	14.7	299	6.7	83
518	62.00	27	AV27	18.7	21.8	9.2	12.6	290	6.2	80
			MAX	22.8	24.0	11.4	14.5	353	6.5	84
547	63.00	29	AV29	18.2	21.4	9.0	12.1	282	6.2	80
			MAX	22.1	23.6	11.0	13.8	342	6.5	85
574	64.00	27	AV27	18.7	20.9	9.7	12.1	290	6.1	75
			MAX	21.5	24.7	11.2	13.5	334	6.5	81
604	65.00	30	AV30	18.8	21.8	9.9	11.9	292	6.1	74
			MAX	21.4	25.0	11.4	13.3	331	6.6	79

Time Summary

Drive 12 minutes 31 seconds 10:05:40 AM - 10:18:11 AM (2/23/2012) BN 1 - 604

I-90 Innerbelt East Bank Bulkhead WaWalls - Type 2 Pile 169 Restrike HP 12X53
OP: BAW Test date: 8-Mar-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 68.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
4	65.08	48	AV4	27.8	28.1	20.6	14.8	430	7.6	169
			MAX	30.0	30.1	22.0	17.6	465	7.9	194
7	65.17	36	AV3	29.9	30.1	20.2	17.3	463	7.8	146
			MAX	30.0	30.2	20.6	17.4	464	7.9	156
10	65.25	36	AV3	29.9	30.1	19.7	17.6	463	7.8	138
			MAX	30.2	30.2	20.0	18.1	468	7.9	139
			Average	29.0	29.3	20.2	16.4	450	7.7	153
			Maximum	30.2	30.2	22.0	18.1	468	7.9	194

Total number of blows analyzed: 10

Time Summary

Drive 14 seconds 10:27:08 AM - 10:27:22 AM (3/8/2012) BN 1 - 11

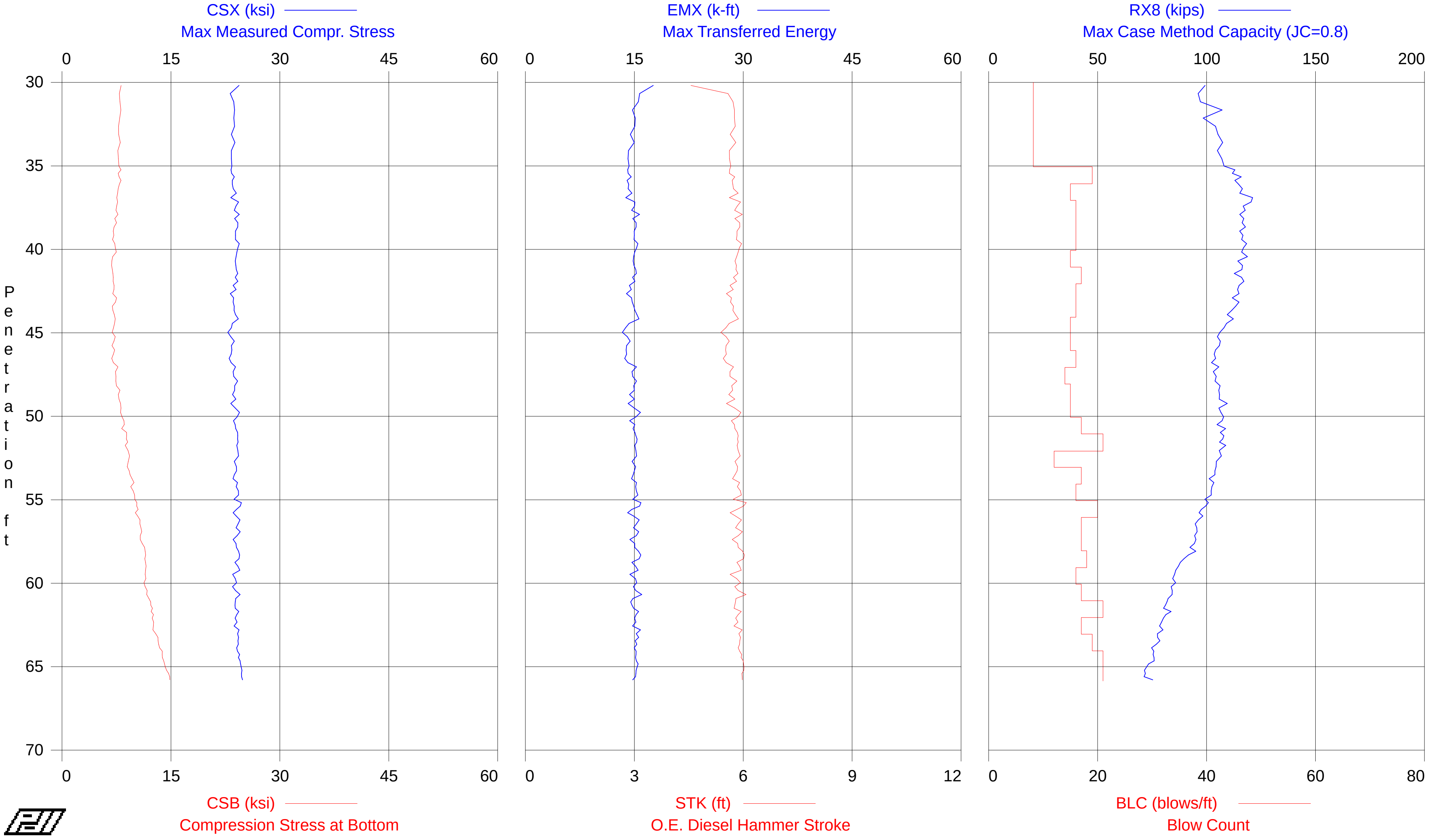
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 9-Mar-2012

Test date: 1-Mar-2012

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 188

12X53 - PLUMB



I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 188 12X53 - PLUMB
OP: TH Test date: 1-Mar-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 68.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
9	31.00	8	AV9	23.7	24.3	8.0	16.5	367	5.8	97
			MAX	26.0	26.9	8.6	19.9	403	6.6	107
17	32.00	8	AV8	23.7	24.1	8.0	15.1	367	5.7	102
			MAX	24.8	25.4	8.5	16.6	385	6.1	117
25	33.00	8	AV8	23.8	24.3	7.8	15.1	369	5.8	103
			MAX	24.7	25.1	8.2	15.8	383	6.1	108
33	34.00	8	AV8	23.5	24.1	7.9	14.5	364	5.7	107
			MAX	24.3	25.0	8.2	15.7	376	6.0	118
42	35.00	8	AV9	23.4	23.9	7.8	14.3	362	5.6	106
			MAX	24.3	24.9	8.1	15.5	376	5.9	113
61	36.00	19	AV19	23.4	24.2	7.9	14.2	363	5.7	113
			MAX	24.6	25.0	8.6	15.6	381	6.0	118
76	37.00	15	AV15	23.6	24.4	7.7	14.3	366	5.7	117
			MAX	24.6	25.6	8.1	16.0	382	6.1	129
92	38.00	16	AV16	24.1	24.9	7.6	15.1	374	5.9	118
			MAX	25.0	25.9	8.3	16.2	388	6.2	127
108	39.00	16	AV16	24.0	24.8	7.3	15.1	372	5.9	117
			MAX	24.8	25.7	7.6	15.9	384	6.1	122
124	40.00	16	AV16	24.1	24.9	7.2	15.2	374	5.9	117
			MAX	25.2	25.8	7.9	16.0	390	6.2	122
139	41.00	15	AV15	23.9	24.6	7.0	14.9	371	5.8	117
			MAX	25.0	25.8	7.7	15.7	388	6.2	121
156	42.00	17	AV17	24.1	24.6	7.0	15.1	373	5.8	115
			MAX	24.8	25.6	7.5	16.1	384	6.0	120
172	43.00	16	AV16	23.6	24.5	7.2	14.4	366	5.6	114
			MAX	24.4	25.5	8.0	15.3	378	5.9	121
188	44.00	16	AV16	23.7	24.3	7.1	15.0	368	5.7	112
			MAX	24.5	25.3	8.0	15.8	379	6.0	116
203	45.00	15	AV15	23.5	23.8	7.2	14.3	364	5.6	109
			MAX	24.5	24.9	7.9	15.9	380	6.0	113
218	46.00	15	AV15	23.4	23.6	7.1	14.1	363	5.5	106
			MAX	24.4	24.5	7.6	15.0	379	5.8	109
234	47.00	16	AV16	23.3	23.8	7.1	14.1	361	5.5	104
			MAX	24.2	24.6	7.8	16.0	376	5.9	107
248	48.00	14	AV14	23.8	24.3	7.5	15.0	369	5.7	104
			MAX	24.7	25.3	7.8	16.0	383	6.0	108

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 188 12X53 - PLUMB
OP: TH Test date: 1-Mar-2012

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
263	49.00	15	AV15	23.7	23.9	7.8	14.7	367	5.7	106
			MAX	24.5	24.7	8.3	15.8	379	5.9	109
278	50.00	15	AV15	24.0	24.3	8.1	15.1	371	5.8	107
			MAX	25.3	25.6	8.7	16.9	393	6.2	111
295	51.00	17	AV17	23.9	24.2	8.4	14.8	370	5.8	107
			MAX	24.7	25.2	9.1	15.8	384	6.0	112
316	52.00	21	AV19	24.2	24.8	8.9	15.3	375	5.9	107
			MAX	24.9	25.8	9.5	16.1	386	6.0	111
328	53.00	12	AV12	24.0	24.3	9.1	15.1	373	5.8	106
			MAX	25.0	25.3	9.7	15.9	387	6.1	109
345	54.00	17	AV17	23.9	24.0	9.4	15.0	370	5.8	103
			MAX	24.7	24.8	9.9	15.7	383	6.0	107
361	55.00	16	AV16	24.1	24.3	9.8	15.2	374	5.9	102
			MAX	25.1	25.3	10.4	16.4	390	6.2	106
381	56.00	20	AV20	24.1	24.4	10.3	15.0	374	5.9	98
			MAX	25.0	25.2	10.7	16.2	387	6.1	102
398	57.00	17	AV17	24.3	24.7	10.8	15.4	377	5.9	96
			MAX	25.2	25.4	11.2	16.4	390	6.2	100
415	58.00	17	AV17	23.9	24.1	11.0	15.0	371	5.8	94
			MAX	24.8	25.0	11.4	16.1	385	6.1	97
433	59.00	18	AV18	24.3	24.6	11.5	15.4	376	6.0	90
			MAX	25.2	25.5	11.8	16.6	391	6.3	98
449	60.00	16	AV16	23.9	24.7	11.5	15.1	371	5.8	85
			MAX	24.8	25.5	11.9	16.2	385	6.1	88
466	61.00	17	AV17	24.0	25.0	11.7	15.3	372	5.9	84
			MAX	24.9	26.0	12.7	16.5	387	6.2	87
487	62.00	21	AV21	24.0	24.8	12.4	15.0	372	5.8	82
			MAX	24.7	25.6	13.0	16.2	382	6.1	86
504	63.00	17	AV17	24.0	24.2	12.6	15.2	373	5.9	79
			MAX	24.6	24.8	13.0	16.1	382	6.0	82
523	64.00	19	AV19	24.2	24.4	13.3	15.2	375	5.9	77
			MAX	24.6	24.9	13.9	15.9	382	6.0	81
544	65.00	21	AV21	24.5	24.7	13.9	15.3	379	6.0	75
			MAX	25.3	26.0	14.4	16.4	392	6.2	79
562	65.86	21	AV18	24.7	27.1	14.6	15.1	384	6.0	72
			MAX	25.5	28.2	15.1	15.8	395	6.1	83

Time Summary

Drive 11 minutes 32 seconds 1:43:58 PM - 1:55:30 PM (3/1/2012) BN 1 - 563

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 188 Restrike HP 12x53
OP: BAW Test date: 8-Mar-2012

AR: 12.40 in^2 SP: 0.492 k/ft3
LE: 68.5 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
3	65.08	36	AV3	27.7	32.0	22.1	12.1	344	7.5	142
			MAX	31.2	36.3	24.5	15.9	387	8.1	159
6	65.17	36	AV3	30.7	34.6	24.0	14.8	381	7.7	141
			MAX	31.0	35.3	24.4	15.2	384	7.7	145
9	65.25	36	AV3	29.6	32.2	22.9	13.9	367	7.2	134
			MAX	30.0	33.0	23.4	14.3	371	7.4	137
			Average	29.3	33.0	23.0	13.6	364	7.4	139
			Maximum	31.2	36.3	24.5	15.9	387	8.1	159

Total number of blows analyzed: 9

Time Summary

Drive 11 seconds 9:48:44 AM - 9:48:55 AM (3/8/2012) BN 1 - 9

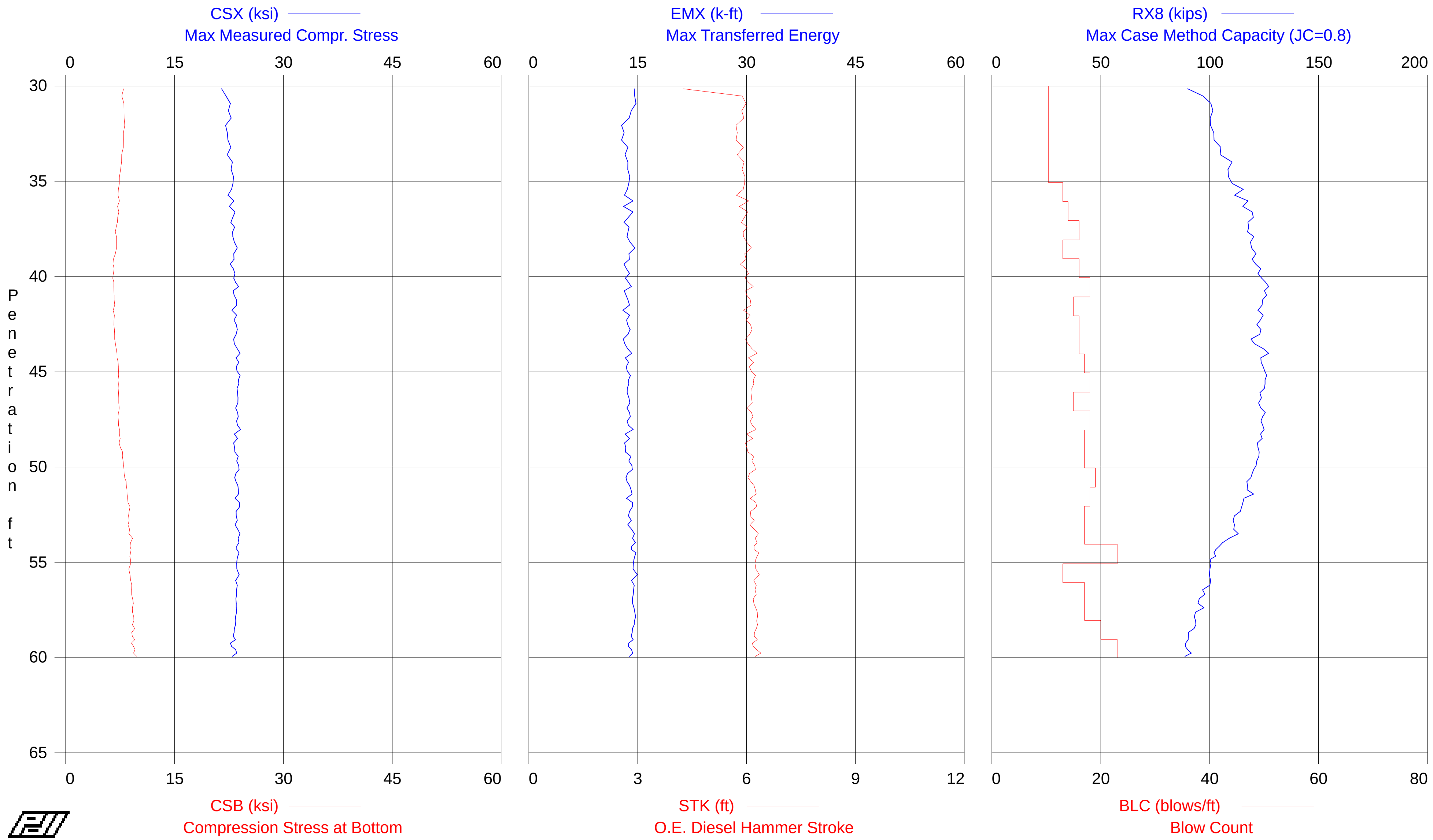
GRL Engineers, Inc. - Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 9-Mar-2012

Test date: 1-Mar-2012

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 196

12X53 - 4TO1 BATTER



I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 196 12X53 - 4TO1 BATTER
OP: TH Test date: 1-Mar-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 63.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.90

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
11	31.00	10	AV11	22.0	23.1	7.9	14.6	341	5.8	95
			MAX	22.8	25.1	8.2	15.9	353	6.0	101
21	32.00	10	AV10	22.6	24.6	8.1	14.0	350	5.9	101
			MAX	23.2	26.1	8.6	15.1	360	6.2	104
32	33.00	10	AV11	22.2	23.6	8.0	12.9	345	5.7	102
			MAX	23.4	24.2	8.5	13.7	362	6.0	105
42	34.00	10	AV10	22.6	24.2	7.8	13.6	351	5.9	106
			MAX	23.5	24.8	8.1	14.2	364	6.0	112
53	35.00	10	AV11	22.9	24.1	7.5	13.7	355	5.9	109
			MAX	23.5	26.7	7.8	14.5	364	6.1	114
66	36.00	13	AV13	22.8	23.7	7.3	13.7	354	5.9	113
			MAX	23.9	25.1	7.6	15.4	371	6.3	119
80	37.00	14	AV14	23.0	24.0	7.3	13.7	356	5.9	118
			MAX	23.7	25.8	7.6	14.5	367	6.1	125
96	38.00	16	AV16	23.0	23.5	7.0	13.5	357	5.9	118
			MAX	23.6	24.2	7.4	14.3	366	6.1	129
109	39.00	13	AV13	23.4	23.8	6.9	14.1	362	6.0	120
			MAX	24.2	24.8	7.4	15.4	375	6.3	125
125	40.00	16	AV16	23.1	23.5	6.6	13.6	358	6.0	122
			MAX	23.7	24.2	7.1	14.7	368	6.2	125
143	41.00	18	AV18	23.3	23.7	6.6	13.5	361	6.0	126
			MAX	24.1	24.5	6.9	14.5	374	6.3	130
158	42.00	15	AV15	23.4	23.8	6.7	13.6	363	6.1	124
			MAX	24.2	24.7	6.9	14.7	375	6.3	127
174	43.00	16	AV16	23.5	23.9	6.7	13.7	364	6.1	123
			MAX	24.1	24.4	6.8	14.5	374	6.3	127
190	44.00	16	AV16	23.4	23.8	6.9	13.4	363	6.1	122
			MAX	24.1	24.5	7.1	14.2	373	6.3	128
207	45.00	17	AV17	23.7	24.2	7.2	13.7	368	6.2	125
			MAX	24.4	24.9	7.5	14.7	378	6.4	130
225	46.00	18	AV18	23.8	24.2	7.3	13.7	368	6.2	125
			MAX	24.4	24.9	7.5	14.6	378	6.3	128
240	47.00	15	AV15	23.7	24.0	7.3	13.7	367	6.1	123
			MAX	24.2	24.6	7.6	14.4	376	6.3	126
258	48.00	18	AV18	23.7	24.0	7.3	13.9	368	6.2	124
			MAX	24.5	24.6	7.5	15.1	379	6.4	129

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 196
OP: TH

12X53 - 4TO1 BATTER
Test date: 1-Mar-2012

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
275	49.00	17	AV17	23.5	23.7	7.4	13.5	363	6.1	123
			MAX	24.2	24.4	7.7	14.6	376	6.4	128
292	50.00	17	AV17	23.6	23.8	7.9	13.8	365	6.1	122
			MAX	24.3	24.5	8.4	14.8	377	6.3	126
311	51.00	19	AV19	23.5	23.9	8.2	13.7	365	6.1	119
			MAX	24.3	24.7	8.7	15.0	377	6.4	123
329	52.00	18	AV18	23.7	24.2	8.5	14.1	368	6.2	117
			MAX	24.6	25.1	9.0	14.9	381	6.5	125
346	53.00	17	AV17	23.6	23.9	8.7	13.9	365	6.1	112
			MAX	24.3	24.9	9.3	15.1	377	6.4	116
363	54.00	17	AV17	23.8	24.0	8.9	14.3	369	6.3	110
			MAX	24.4	24.7	9.6	15.0	378	6.5	116
386	55.00	23	AV23	23.7	24.0	8.9	14.5	367	6.3	102
			MAX	24.5	24.8	9.5	15.6	380	6.5	105
399	56.00	13	AV13	23.6	24.0	8.9	14.5	366	6.3	100
			MAX	24.3	24.8	9.1	15.3	377	6.5	103
416	57.00	17	AV17	23.5	23.9	9.1	14.4	365	6.2	98
			MAX	24.2	24.5	9.4	15.1	375	6.4	104
433	58.00	17	AV17	23.5	24.1	9.3	14.5	364	6.3	95
			MAX	23.8	24.5	9.7	14.9	369	6.4	100
453	59.00	20	AV20	23.3	23.8	9.3	14.3	361	6.3	92
			MAX	23.8	24.6	9.8	15.2	370	6.5	94
476	60.00	23	AV23	23.1	23.8	9.4	14.0	359	6.3	90
			MAX	24.0	24.7	10.2	15.2	372	6.7	93

Time Summary

Drive 10 minutes 1 second 2:38:24 PM - 2:48:25 PM (3/1/2012) BN 1 - 477

I-90 Innerbelt East Bank Bulkhead Walls - Type 1 Pile 196 Restrike HP 12x53
OP: BAW Test date: 8-Mar-2012

AR: 15.50 in^2 SP: 0.492 k/ft3
LE: 63.0 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 1.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
EMX: Max Transferred Energy
FMX: Maximum Force
STK: O.E. Diesel Hammer Stroke
RX8: Max Case Method Capacity (JC=0.8)

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
3	60.08	36	AV3	25.4	28.9	18.1	13.1	393	7.5	145
			MAX	29.3	32.1	19.4	17.4	454	8.1	172
6	60.17	36	AV3	28.8	32.0	18.4	17.4	447	8.0	120
			MAX	29.5	32.2	19.2	17.7	457	8.2	121
8	60.25	24	AV2	29.2	32.4	18.1	17.8	453	7.9	119
			MAX	29.5	32.5	18.1	18.0	457	7.9	121
			Average	27.6	30.9	18.2	15.9	428	7.8	129
			Maximum	29.5	32.5	19.4	18.0	457	8.2	172

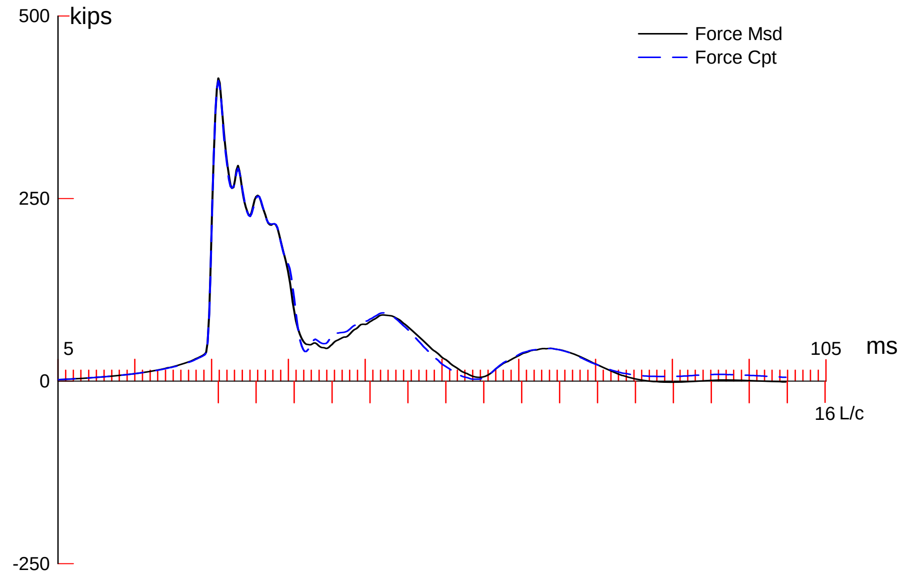
Total number of blows analyzed: 8

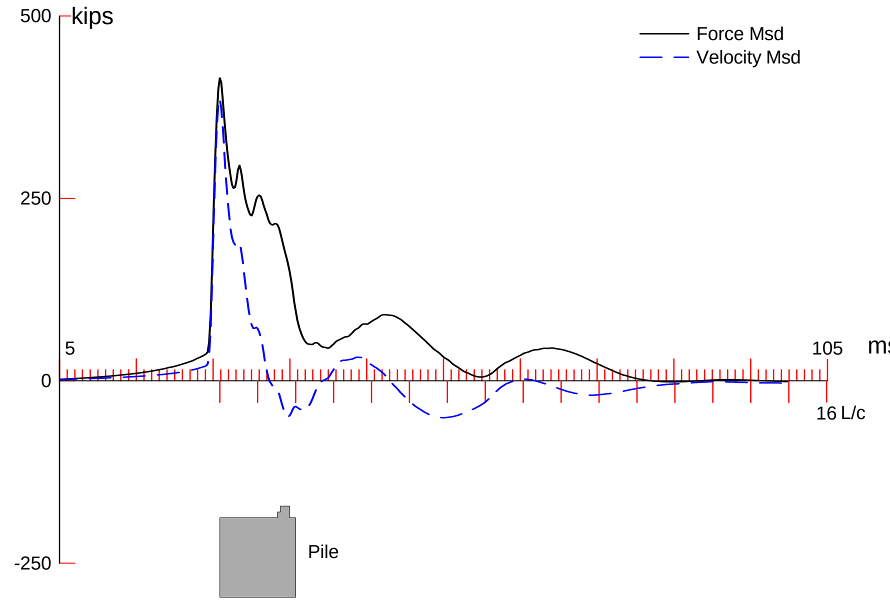
Time Summary

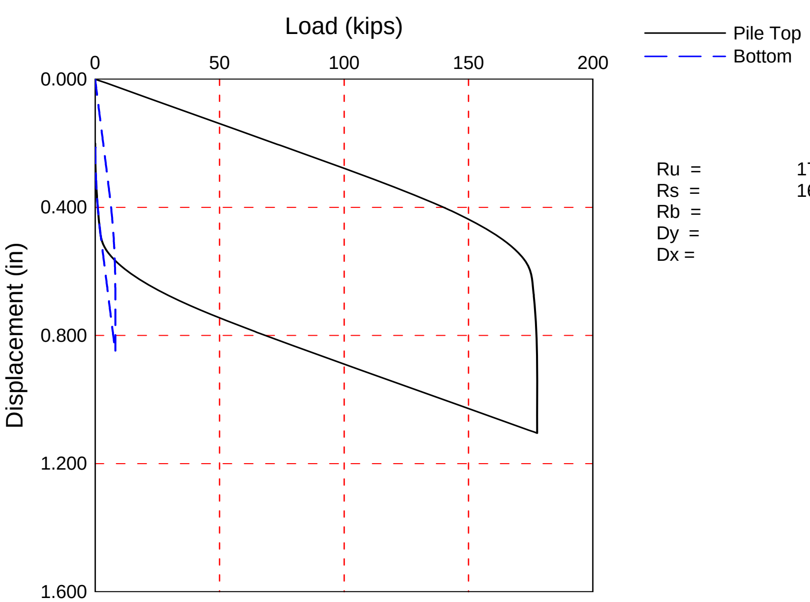
Drive 10 seconds 9:59:30 AM - 9:59:40 AM (3/8/2012) BN 1 - 8

Appendix C

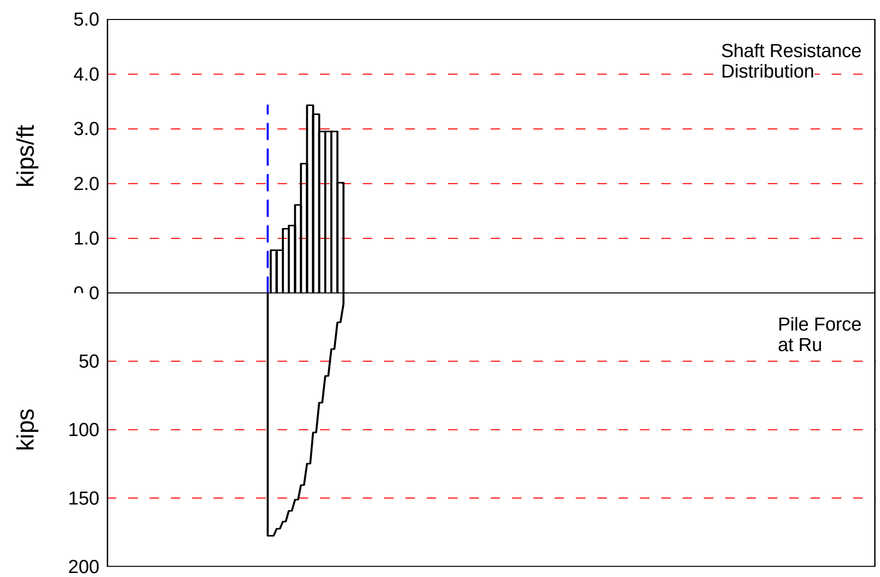
CAPWAP Results







Ru =	177.6	kips
Rs =	169.5	kips
Rb =	8.1	kips
Dy =	0.91	in
Dx =	1.11	in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 177.6; along Shaft 169.5; at Toe 8.1 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
				177.6				
1	10.0	7.0	5.2	172.4	5.2	0.75	0.19	0.337
2	16.6	13.6	5.2	167.2	10.4	0.78	0.20	0.337
3	23.2	20.2	7.8	159.4	18.2	1.17	0.30	0.337
4	29.9	26.9	8.2	151.2	26.4	1.23	0.31	0.337
5	36.5	33.5	10.7	140.5	37.1	1.61	0.41	0.337
6	43.2	40.2	15.7	124.8	52.8	2.36	0.60	0.337
7	49.8	46.8	22.8	102.0	75.6	3.43	0.86	0.337
8	56.4	53.4	21.7	80.3	97.3	3.27	0.82	0.337
9	63.1	60.1	19.6	60.7	116.9	2.95	0.74	0.337
10	69.7	66.7	19.6	41.1	136.5	2.95	0.74	0.337
11	76.4	73.4	19.6	21.5	156.1	2.95	0.74	0.337
12	83.0	80.0	13.4	8.1	169.5	2.02	0.51	0.337
Avg. Shaft			14.1			2.12	0.53	0.337
Toe			8.1				8.22	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.500
Case Damping Factor		2.063	0.007
Unloading Quake	(% of loading quake)	250	35
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	95	
Resistance Gap (included in Toe Quake) (in)			0.000
Soil Plug Weight	(kips)		0.58
Soil Support Dashpot		0.400	0.000
Soil Support Weight	(kips)	2.80	0.00

CAPWAP match quality = 1.99 (Wave Up Match) ; RSA = 0
Observed: final set = 0.200 in; blow count = 60 b/ft
Computed: final set = 0.229 in; blow count = 53 b/ft

max. Top Comp. Stress	= 26.9 ksi	(T= 26.3 ms, max= 1.030 x Top)
max. Comp. Stress	= 27.7 ksi	(Z= 10.0 ft, T= 26.7 ms)
max. Tens. Stress	= -1.34 ksi	(Z= 79.7 ft, T= 32.6 ms)
max. Energy (EMX)	= 14.3 kip-ft;	max. Measured Top Displ. (DMX)= 0.62 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	416.4	0.0	26.9	0.00	14.30	13.9	0.612
2	6.6	422.4	0.0	27.2	0.00	14.11	13.7	0.593
4	13.3	409.4	-0.8	26.4	-0.05	13.04	13.2	0.564
6	19.9	400.8	-2.6	25.9	-0.17	12.29	12.6	0.548
8	26.6	383.1	-4.1	24.7	-0.26	11.33	12.0	0.534
9	29.9	393.1	-4.2	25.4	-0.27	11.30	11.6	0.526
10	33.2	368.8	-5.3	23.8	-0.34	10.43	11.2	0.520
11	36.5	381.4	-5.2	24.6	-0.34	10.41	10.8	0.514
12	39.8	355.0	-6.1	22.9	-0.40	9.43	10.3	0.511
13	43.2	368.9	-6.1	23.8	-0.39	9.41	9.7	0.507
14	46.5	332.4	-5.9	21.4	-0.38	8.19	9.2	0.505
15	49.8	344.0	-5.8	22.2	-0.38	8.18	8.7	0.503
16	53.1	292.8	-3.5	18.9	-0.23	6.67	8.2	0.502
17	56.4	305.6	-3.3	19.7	-0.21	6.67	7.7	0.501
18	59.8	264.4	-5.0	17.1	-0.32	5.30	7.1	0.500
19	63.1	273.7	-3.7	17.7	-0.24	5.29	6.7	0.500
20	66.4	235.9	-6.0	15.2	-0.39	4.03	6.6	0.500
21	69.7	241.4	-3.2	15.6	-0.21	4.03	6.8	0.500
22	73.0	215.9	-4.5	13.9	-0.29	2.72	6.8	0.500
23	76.4	222.7	-1.8	14.4	-0.12	2.72	6.8	0.500
24	79.7	174.8	-20.8	11.3	-1.34	1.37	7.6	0.500
25	83.0	148.6	-9.5	9.6	-0.61	0.21	8.6	0.500
Absolute	10.0			27.7			(T =	26.7 ms)
	79.7				-1.34		(T =	32.6 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	468.9	435.5	402.0	368.5	335.0	301.6	268.1	234.6	201.2	167.7
RX	468.9	435.5	402.0	368.5	335.0	301.6	268.1	234.6	201.2	167.7
RU	512.5	483.4	454.2	425.1	396.0	366.9	337.8	308.7	279.5	250.4

RAU = 59.2 (kips); RA2 = 178.4 (kips)

Current CAPWAP Ru = 177.6 (kips); Corresponding J(RP)= 0.87; J(RX) = 0.87

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
14.07	26.07	389.2	414.4	417.8	0.624	0.200	0.200	14.5	421.6

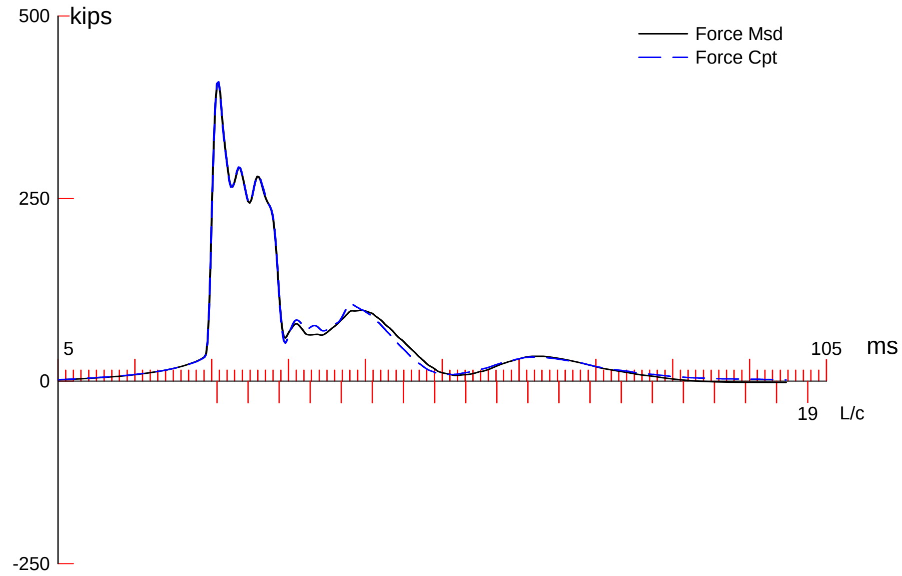
PILE PROFILE AND PILE MODEL

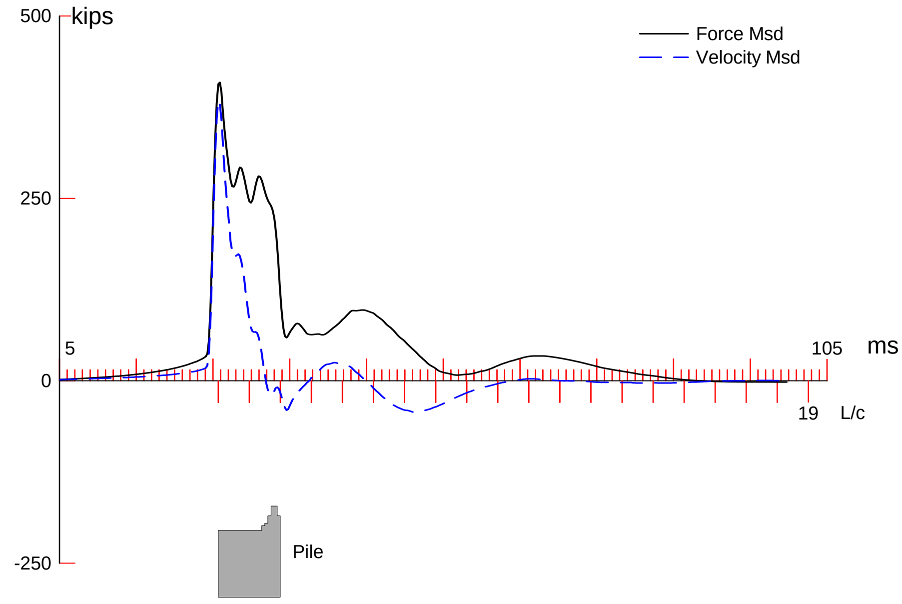
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	15.50	29992.2	492.000	3.971
83.00	15.50	29992.2	492.000	3.971

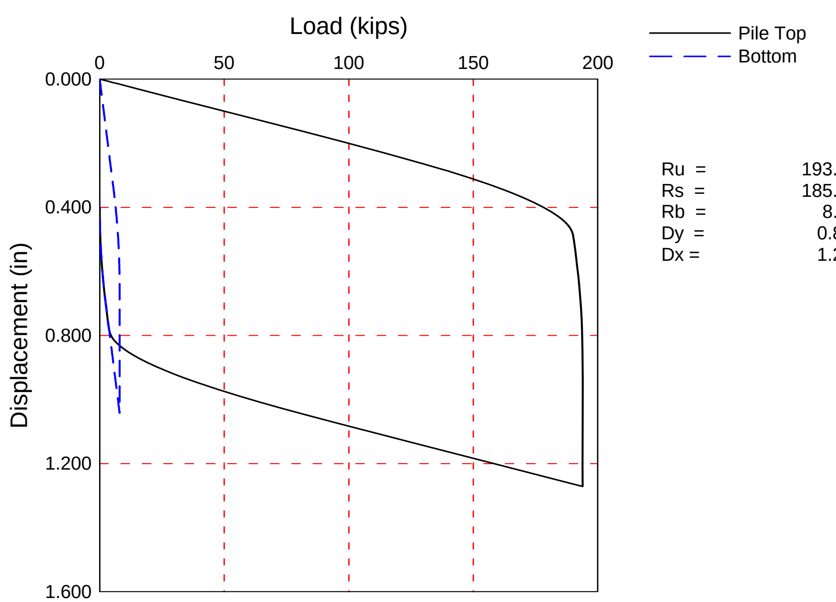
Toe Area 0.985 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Compression Slack	Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.32	27.67	0.00	0.000	0.000	-0.000	0.000	3.971
20	66.40	29.67	7.23	0.000	0.000	-0.000	0.000	3.971
21	69.72	31.67	14.46	0.000	0.000	-0.000	0.000	3.971
24	79.68	27.67	0.00	0.000	0.000	-0.000	0.000	3.971
25	83.00	27.67	0.00	0.000	0.000	-0.000	0.000	3.971

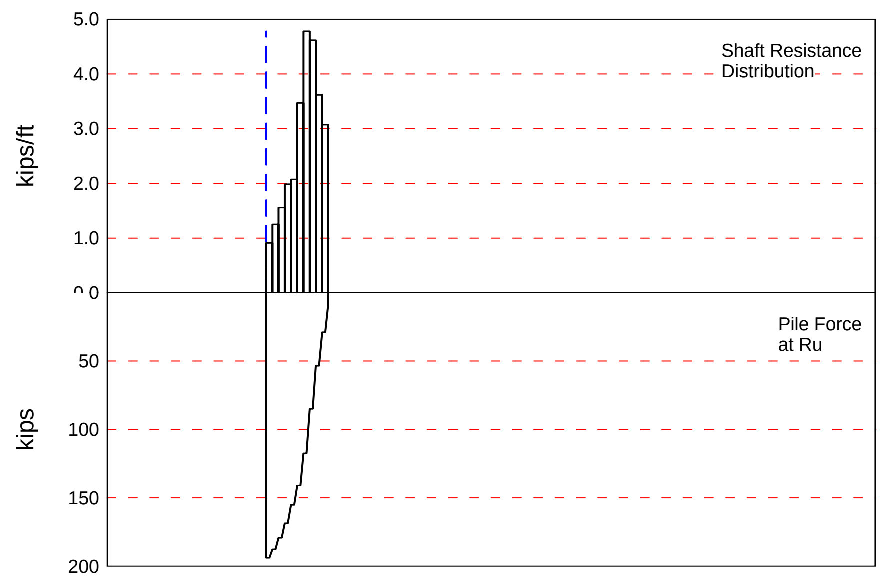
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 9.9 ms







Ru =	193.9 kips
Rs =	185.9 kips
Rb =	8.0 kips
Dy =	0.87 in
Dx =	1.27 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 193.9; along Shaft 185.9; at Toe 8.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				193.9				
1	6.8	3.8	6.2	187.7	6.2	1.63	0.41	0.257
2	13.6	10.6	8.5	179.2	14.7	1.25	0.31	0.257
3	20.4	17.4	10.6	168.6	25.3	1.56	0.39	0.257
4	27.2	24.2	13.5	155.1	38.8	1.99	0.50	0.257
5	34.0	31.0	14.1	141.0	52.9	2.07	0.52	0.257
6	40.8	37.8	23.6	117.4	76.5	3.47	0.87	0.257
7	47.6	44.6	32.5	84.9	109.0	4.78	1.20	0.257
8	54.4	51.4	31.4	53.5	140.4	4.62	1.16	0.257
9	61.2	58.2	24.6	28.9	165.0	3.62	0.91	0.257
10	68.0	65.0	20.9	8.0	185.9	3.07	0.77	0.257
Avg. Shaft			18.6			2.86	0.72	0.257
Toe			8.0				8.12	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.214	0.493
Case Damping Factor		1.724	0.007
Unloading Quake	(% of loading quake)	204	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	83	
Resistance Gap (included in Toe Quake) (in)			0.012
Soil Plug Weight	(kips)		0.27
Soil Support Dashpot		0.500	0.000
Soil Support Weight	(kips)	2.82	0.00

CAPWAP match quality	= 1.57	(Wave Up Match) ; RSA = 0
Observed: final set	= 0.400 in;	blow count = 30 b/ft
Computed: final set	= 0.367 in;	blow count = 33 b/ft
max. Top Comp. Stress	= 26.9 ksi	(T= 26.3 ms, max= 1.019 x Top)
max. Comp. Stress	= 27.4 ksi	(Z= 6.8 ft, T= 26.5 ms)
max. Tens. Stress	= -1.23 ksi	(Z= 57.8 ft, T= 38.2 ms)
max. Energy (EMX)	= 13.7 kip-ft;	max. Measured Top Displ. (DMX)= 0.60 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	3.4	416.5	0.0	26.9	0.00	13.73	13.6	0.567
2	6.8	424.2	0.0	27.4	0.00	13.69	13.3	0.559
3	10.2	407.3	0.0	26.3	0.00	12.93	13.0	0.551
4	13.6	416.4	0.0	26.9	0.00	12.90	12.7	0.543
5	17.0	393.6	-0.1	25.4	-0.00	11.98	12.4	0.534
6	20.4	404.1	-0.1	26.1	-0.00	11.96	12.0	0.527
7	23.8	376.9	-0.6	24.3	-0.04	10.94	11.6	0.520
8	27.2	387.5	-0.6	25.0	-0.04	10.92	11.2	0.514
9	30.6	354.7	-0.9	22.9	-0.06	9.78	10.9	0.509
10	34.0	368.7	-1.1	23.8	-0.07	9.76	10.4	0.504
11	37.4	342.6	-2.6	22.1	-0.17	8.68	9.9	0.500
12	40.8	361.7	-2.7	23.3	-0.18	8.68	9.2	0.497
13	44.2	322.1	-3.5	20.8	-0.22	7.13	8.5	0.494
14	47.6	341.0	-3.5	22.0	-0.23	7.13	7.7	0.492
15	51.0	292.6	-10.9	18.9	-0.70	5.23	7.0	0.491
16	54.4	304.6	-11.2	19.6	-0.72	5.23	6.7	0.491
17	57.8	252.7	-19.0	16.3	-1.23	3.38	7.2	0.492
18	61.2	246.4	-14.5	15.9	-0.93	3.38	7.8	0.492
19	64.6	169.7	-14.0	10.9	-0.90	1.78	8.8	0.493
20	68.0	117.2	-1.3	7.6	-0.08	0.20	9.7	0.494
Absolute	6.8			27.4			(T =	26.5 ms)
	57.8				-1.23		(T =	38.2 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	463.9	430.2	396.4	362.7	328.9	295.2	261.4	227.7	193.9	160.1
RX	463.9	430.2	396.4	362.7	328.9	295.2	261.4	227.7	193.9	160.1
RU	498.6	468.3	438.0	407.7	377.4	347.1	316.8	286.5	256.3	226.0

RAU = 59.9 (kips); RA2 = 213.0 (kips)

Current CAPWAP Ru = 193.9 (kips); Corresponding J(RP)= 0.80; J(RX) = 0.80

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
14.04	25.89	388.4	413.1	413.1	0.598	0.400	0.400	14.1	338.2

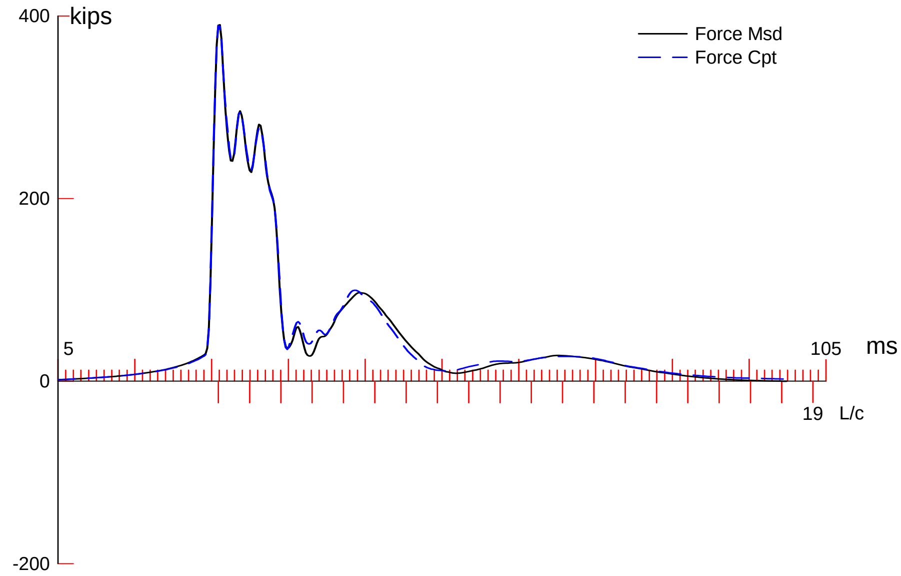
PILE PROFILE AND PILE MODEL

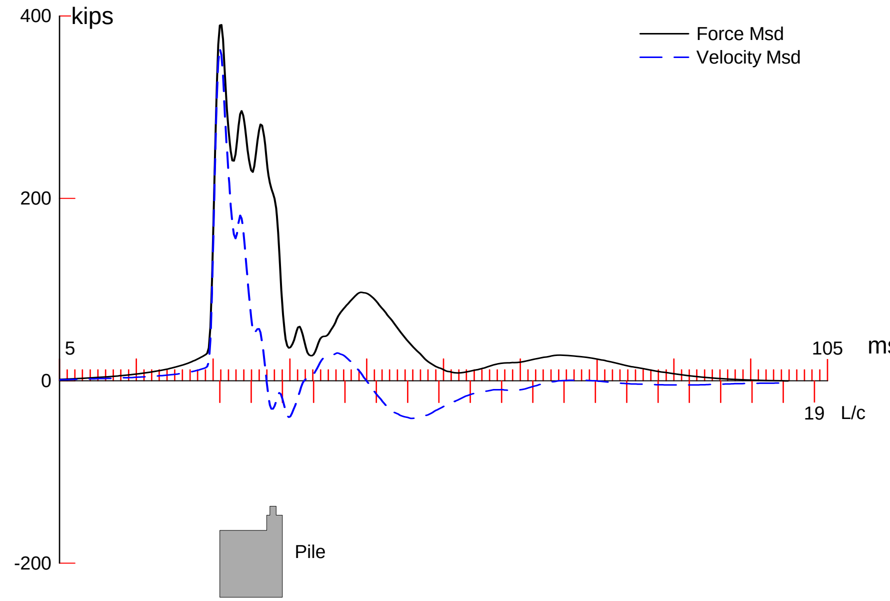
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	15.50	29992.2	492.000	3.971
68.00	15.50	29992.2	492.000	3.971

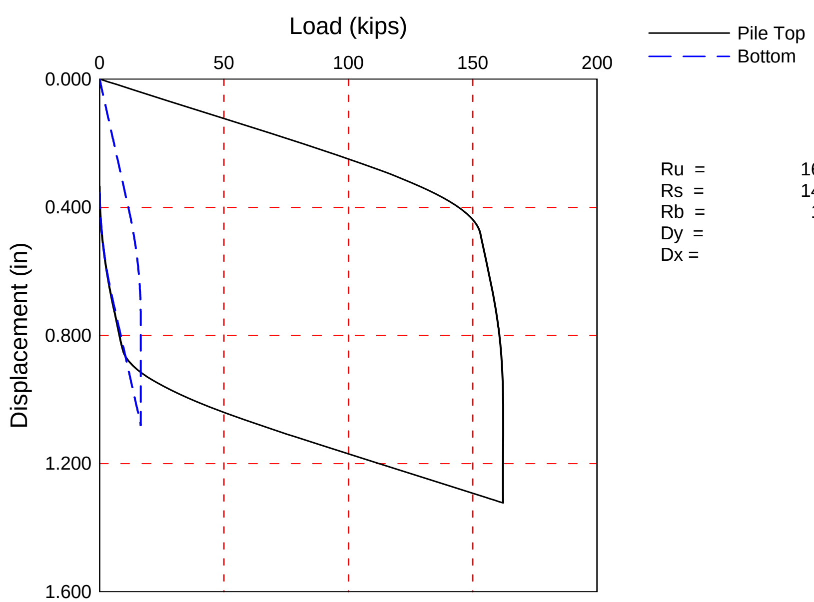
Toe Area 0.985 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.40	27.67	0.00	0.000	0.000	-0.000	0.000	3.971
15	51.00	29.67	7.23	0.000	0.000	-0.000	0.000	3.971
16	54.40	30.67	10.84	0.000	0.000	-0.000	0.000	3.971
17	57.80	33.67	21.69	0.000	0.000	-0.000	0.000	3.971
18	61.20	37.67	36.15	0.000	0.000	-0.000	0.000	3.971
20	68.00	33.67	21.69	0.000	0.000	-0.000	0.000	3.971

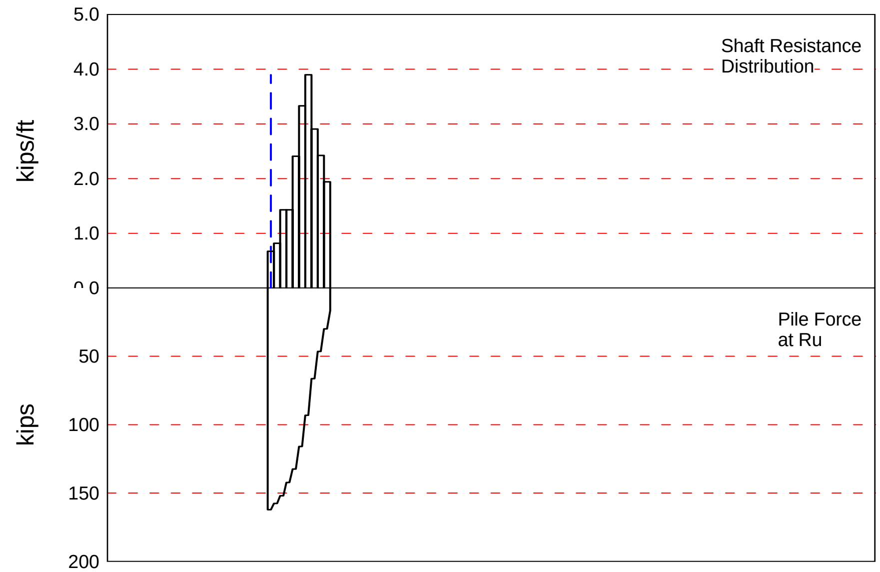
Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 16807.9 ft/s, 2L/c 8.1 ms







Ru =	162.2	kips
Rs =	145.6	kips
Rb =	16.6	kips
Dy =	0.99	in
Dx =	1.32	in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 162.2; along Shaft 145.6; at Toe 16.6 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
				162.2				
1	6.9	3.4	4.6	157.6	4.6	1.37	0.42	0.263
2	13.7	10.2	5.6	152.0	10.2	0.82	0.25	0.263
3	20.6	17.1	9.8	142.2	20.0	1.43	0.43	0.263
4	27.4	23.9	9.8	132.4	29.8	1.43	0.43	0.263
5	34.3	30.8	16.5	115.9	46.3	2.41	0.73	0.263
6	41.1	37.6	22.8	93.1	69.1	3.33	1.01	0.263
7	48.0	44.5	26.7	66.4	95.8	3.90	1.18	0.263
8	54.8	51.3	19.9	46.5	115.7	2.91	0.88	0.263
9	61.7	58.2	16.6	29.9	132.3	2.42	0.74	0.263
10	68.5	65.0	13.3	16.6	145.6	1.94	0.59	0.263
Avg. Shaft			14.6			2.24	0.68	0.263
Toe			16.6				24.46	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.209	0.575
Case Damping Factor		1.731	0.018
Unloading Quake	(% of loading quake)	289	32
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	95	
Soil Plug Weight	(kips)		0.18
Soil Support Dashpot		0.500	0.000
Soil Support Weight	(kips)	2.36	0.00

CAPWAP match quality = 1.69 (Wave Up Match) ; RSA = 0
Observed: final set = 0.333 in; blow count = 36 b/ft
Computed: final set = 0.372 in; blow count = 32 b/ft

max. Top Comp. Stress = 31.8 ksi (T= 26.3 ms, max= 1.016 x Top)
max. Comp. Stress = 32.3 ksi (Z= 6.9 ft, T= 26.5 ms)
max. Tens. Stress = -1.45 ksi (Z= 44.5 ft, T= 37.7 ms)
max. Energy (EMX) = 15.6 kip-ft; max. Measured Top Displ. (DMX)= 0.71 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	394.6	0.0	31.8	0.00	15.58	16.3	0.672
2	6.9	400.9	0.0	32.3	0.00	15.54	15.9	0.662
3	10.3	386.2	0.0	31.1	0.00	14.73	15.6	0.652
4	13.7	395.7	0.0	31.9	0.00	14.69	15.2	0.642
5	17.1	379.7	0.0	30.6	0.00	13.83	14.8	0.632
6	20.6	389.4	0.0	31.4	0.00	13.80	14.3	0.624
7	24.0	358.8	-0.1	28.9	-0.01	12.52	13.9	0.615
8	27.4	371.8	-0.1	30.0	-0.01	12.49	13.3	0.608
9	30.8	348.1	-0.8	28.1	-0.06	11.34	12.7	0.601
10	34.3	363.3	-0.8	29.3	-0.07	11.32	11.9	0.596
11	37.7	323.4	-9.4	26.1	-0.76	9.68	11.2	0.593
12	41.1	341.2	-12.3	27.5	-0.99	9.67	10.4	0.590
13	44.5	297.7	-18.0	24.0	-1.45	7.65	9.5	0.590
14	48.0	315.6	-14.5	25.4	-1.17	7.65	8.6	0.592
15	51.4	264.2	-13.7	21.3	-1.10	5.42	8.6	0.593
16	54.8	268.6	-11.3	21.7	-0.91	5.41	8.2	0.594
17	58.2	226.1	-14.7	18.2	-1.19	3.63	8.4	0.594
18	61.7	216.0	-10.1	17.4	-0.81	3.63	9.2	0.594
19	65.1	149.2	-8.6	12.0	-0.69	2.03	10.7	0.594
20	68.5	99.0	0.0	8.0	0.00	0.55	11.7	0.594
Absolute	6.9			32.3			(T =	26.5 ms)
	44.5				-1.45		(T =	37.7 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	429.1	396.0	363.0	329.9	296.9	263.8	230.7	197.7	164.6	131.5
RX	429.1	396.0	363.0	329.9	296.9	263.8	230.7	197.7	164.6	131.5
RU	461.6	431.8	402.0	372.1	342.3	312.5	282.7	252.9	223.1	193.3

RAU = 50.1 (kips); RA2 = 166.3 (kips)

Current CAPWAP Ru = 162.2 (kips); Corresponding J(RP)= 0.81; J(RX) = 0.81

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
16.61	26.08	367.7	392.0	395.1	0.711	0.333	0.333	15.9	365.9

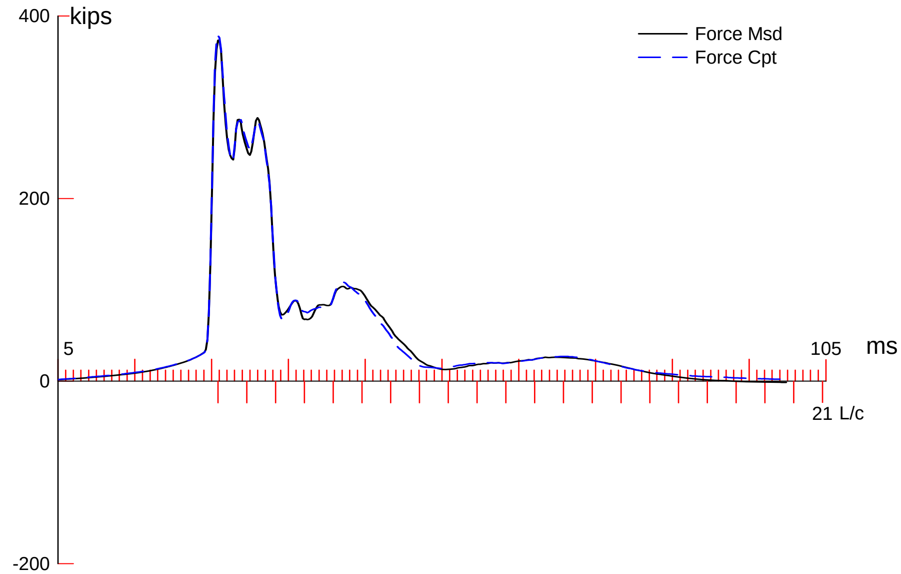
PILE PROFILE AND PILE MODEL

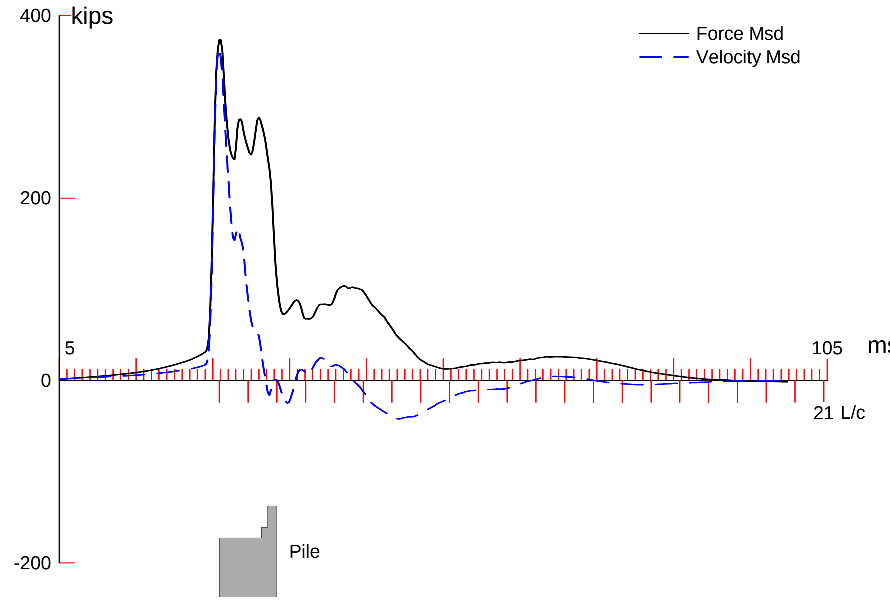
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	12.40	29992.2	492.000	3.296
68.50	12.40	29992.2	492.000	3.296

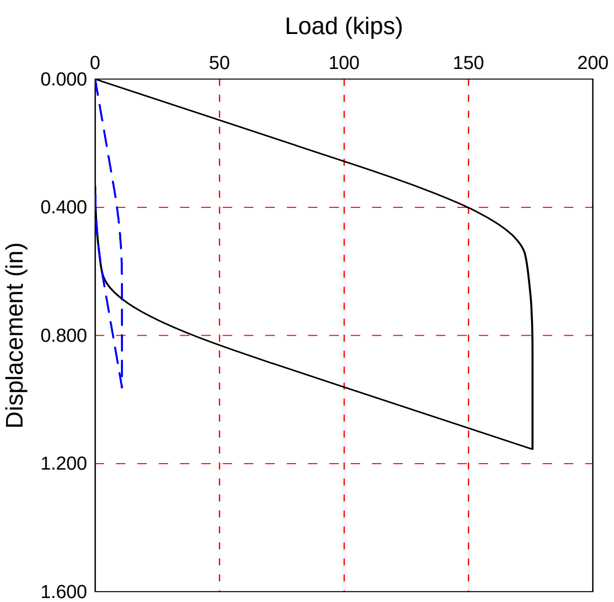
Toe Area 0.679 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Compression Slack	Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.43	22.13	0.00	0.000	0.000	-0.000	0.000	3.296
16	54.80	27.13	22.59	0.000	0.000	-0.000	0.000	3.296
17	58.23	30.13	36.15	0.000	0.000	-0.000	0.000	3.296
19	65.08	27.13	22.59	0.000	0.000	-0.000	0.000	3.296
20	68.50	27.13	22.59	0.000	0.000	-0.000	0.000	3.296

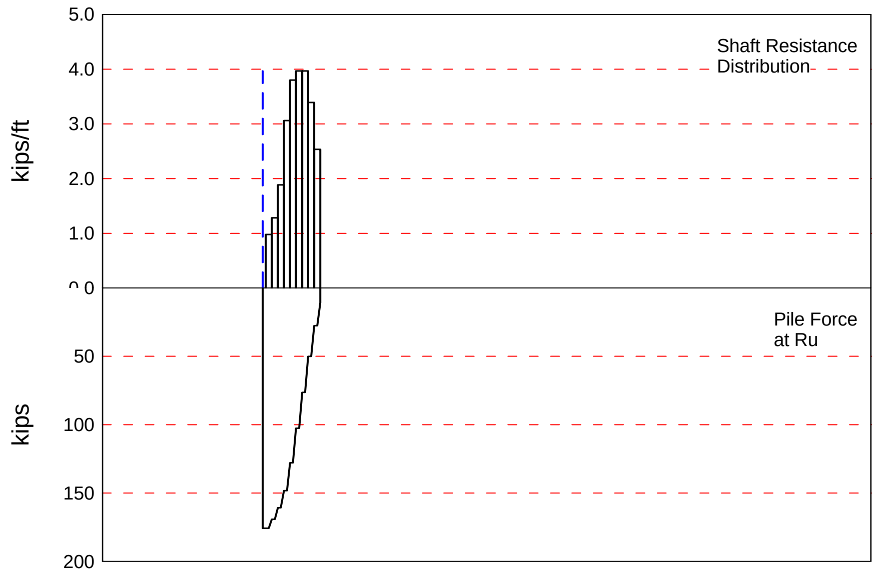
Pile Damping 1.0 %, Time Incr 0.204 ms, Wave Speed 16807.9 ft/s, 2L/c 8.2 ms







Ru = 175.7 kips
Rs = 164.9 kips
Rb = 10.8 kips
Dy = 0.82 in
Dx = 1.16 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 175.7; along Shaft 164.9; at Toe 10.8 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
				175.7				
1	9.9	6.9	6.5	169.2	6.5	0.94	0.24	0.297
2	16.6	13.6	8.5	160.7	15.0	1.28	0.32	0.297
3	23.2	20.2	12.5	148.2	27.5	1.88	0.47	0.297
4	29.8	26.8	20.3	127.9	47.8	3.06	0.77	0.297
5	36.5	33.5	25.2	102.7	73.0	3.80	0.96	0.297
6	43.1	40.1	26.3	76.4	99.3	3.97	1.00	0.297
7	49.7	46.7	26.3	50.1	125.6	3.97	1.00	0.297
8	56.4	53.4	22.5	27.6	148.1	3.39	0.85	0.297
9	63.0	60.0	16.8	10.8	164.9	2.53	0.64	0.297
Avg. Shaft			18.3			2.75	0.69	0.297
Toe			10.8				10.96	0.036

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.484
Case Damping Factor		1.772	0.014
Damping Type			Smith
Unloading Quake	(% of loading quake)	108	58
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	87	
Resistance Gap (included in Toe Quake) (in)			0.000
Soil Plug Weight	(kips)		0.23
Soil Support Dashpot		0.700	0.000
Soil Support Weight	(kips)	2.75	0.00

CAPWAP match quality = 1.40 (Wave Up Match) ; RSA = 0
Observed: final set = 0.333 in; blow count = 36 b/ft
Computed: final set = 0.357 in; blow count = 34 b/ft

max. Top Comp. Stress = 24.8 ksi (T= 26.2 ms, max= 1.037 x Top)
max. Comp. Stress = 25.7 ksi (Z= 9.9 ft, T= 26.6 ms)
max. Tens. Stress = -0.46 ksi (Z= 53.1 ft, T= 37.5 ms)
max. Energy (EMX) = 12.5 kip-ft; max. Measured Top Displ. (DMX)= 0.59 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	383.8	0.0	24.8	0.00	12.49	12.8	0.567
2	6.6	390.4	0.0	25.2	0.00	12.45	12.5	0.558
3	9.9	398.1	0.0	25.7	0.00	12.41	12.2	0.550
4	13.3	380.6	0.0	24.5	0.00	11.66	11.9	0.541
5	16.6	391.5	0.0	25.3	0.00	11.62	11.6	0.533
6	19.9	371.8	0.0	24.0	0.00	10.75	11.2	0.526
7	23.2	386.5	0.0	24.9	0.00	10.73	10.7	0.519
8	26.5	358.9	0.0	23.1	0.00	9.63	10.1	0.513
9	29.8	374.1	0.0	24.1	0.00	9.61	9.6	0.507
10	33.2	329.4	0.0	21.2	0.00	8.10	9.0	0.502
11	36.5	345.9	0.0	22.3	0.00	8.09	8.5	0.497
12	39.8	302.2	0.0	19.5	0.00	6.43	7.8	0.494
13	43.1	320.0	0.0	20.6	0.00	6.42	7.1	0.490
14	46.4	283.5	-1.7	18.3	-0.11	4.78	6.9	0.488
15	49.7	295.9	0.0	19.1	0.00	4.78	6.7	0.486
16	53.1	250.4	-7.2	16.1	-0.46	3.13	7.2	0.484
17	56.4	235.8	0.0	15.2	0.00	3.12	7.9	0.484
18	59.7	160.0	-0.8	10.3	-0.05	1.57	8.6	0.484
19	63.0	95.9	0.0	6.2	0.00	0.25	9.0	0.484
Absolute	9.9			25.7			(T =	26.6 ms)
	53.1				-0.46		(T =	37.5 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	425.8	394.4	363.1	331.7	300.3	269.0	237.6	206.3	174.9	143.5
RX	425.8	394.4	363.1	331.7	300.3	269.0	237.6	206.3	174.9	143.5
RU	447.4	418.2	389.0	359.8	330.6	301.4	272.2	243.0	213.8	184.6

RAU = 79.1 (kips); RA2 = 209.4 (kips)

Current CAPWAP Ru = 175.7 (kips); Corresponding J(RP)= 0.80; J(RX) = 0.80

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
13.23	26.04	365.9	373.5	375.6	0.592	0.333	0.333	12.6	327.7

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	15.50	29992.2	492.000	3.971
63.00	15.50	29992.2	492.000	3.971

Toe Area 0.985 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.32	27.67	0.00	0.000	0.000	-0.000	0.000	3.971
15	49.74	32.67	18.07	0.000	0.000	-0.000	0.000	3.971
17	56.37	42.67	54.22	0.000	0.000	-0.000	0.000	3.971
19	63.00	42.67	54.22	0.000	0.000	-0.000	0.000	3.971

Pile Damping 1.0 %, Time Incr 0.197 ms, Wave Speed 16807.9 ft/s, 2L/c 7.5 ms