GRL engineers, inc. Dynamic Measurements and Analyses

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To: Mr. Joel Halterman

Of: Walsh Construction Date: December 19, 2011

From: Ben White GRL Job No. 115058-23

Re: Dynamic Testing Results; ODOT 3000(10) Bridge 6 Monument Piles and Forward Abutment

Mr. Halterman:

This report summarizes the dynamic testing performed at the above referenced site on December 13 and 19, 2011. As requested, GRL performed dynamic testing on one monument pile during initial driving and restrike and one pile in the forward abutment of bridge 6 during restrike. Table 1 presents the Case Method results and Table 2 presents the results from CAPWAP analyses from all piles tested at this bridge. The complete Case Method and CAPWAP analyses results are shown in Appendix B and C, respectively.

The pile tested for the monument was a 14" O.D. closed end pipe pile with a wall thickness of 0.312 inches. The abutment piles were HP 16x88 steel H-piles. The pipe piles were fabricated from ASTM A-252 Grade 3 steel which has a minimum yield strength of 45 ksi and the H-piles were fabricated from ASTM A-572 Grade 50 steel which has a minimum yield strength of 50 ksi.

The piles were driven using a Pileco D30-32 single-acting diesel hammer. This hammer has a four step fuel pump with 4 being the maximum fuel setting. During initial driving of the monument pile, the hammer was operated at fuel setting 2. During restrike, the hammer was operated at fuel setting 3. The abutment pile was tested during restrike with the hammer set at the maximum fuel setting of 4. The energy transferred to the monument pile near the end of driving 28.4 kip-ft at an average hammer stroke of 7.3 ft. This energy transfer level corresponds to a rated transfer efficiency of 41% of the maximum rated energy of 70.1 kip-ft.

Measured average compressive stresses near the pile top were approximately 28.4 ksi near the end of driving for the monument pile. CAPWAP analysis indicated that the stresses below the gage location were approximately 10% higher than the measured compressive stress at the gage location. The compression stresses were below the recommended stress limit of 90% of the yield strength of the steel or 45.0 ksi. No detectable pile damage below the gage location was observed in the force and velocity records; however, due to high bending stress during restrike, the pile top was damaged. Evaluation of the restrike was performed on data collected before the pile top damage occurred.

Forward Abutment Monument Piles

The required ultimate bearing value of the monument piles is shown as 91.7 tons (183.4 kips) in the plans. However, we were informed that additional capacity was added to overcome the potential down drag forces from settlement. The additional capacity was reported to be 160 kips, therefore the total required ultimate bearing value is 343.4 kips for the forward abutment monument piles.

At the end of initial driving, CAPWAP analysis indicated a total capacity of 254 kips at 85 ft penetration depth, a blow count of 23 blows/ft and an average hammer stroke of 7.3 ft. Restrike testing was performed approximately 6 days later. CAPWAP analysis from data collected during restrike indicated a mobilized capacity of 503 kips. Significant soil set-up had occurred during the waiting period. In addition, the full soil resistance in the bottom approximately 15 ft of the pile as well as the end bearing resistance were not fully mobilized, therefore, the reported capacity can be used as a lower bound estimate of the full pile capacity.

We understand that the lowest blow count of the four monument piles at the forward abutment was 19 blows/ft. Based on the very high mobilized capacity of the monument test pile driven to 23 blows/ft, it is reasonable to assume that all of the monument piles have capacities that exceed the required ultimate bearing value of 343.4 kips.

Forward Abutment Piles

Dynamic testing was previously performed during initial drive and restrike on two piles in the forward abutment. Refer to our report under GRL job number 115058-20 dated December 1, 2011 for further information. From previous test results, GRL suggested the forward abutment piles be driven to at least 29 blows/ft with a corresponding minimum hammer stroke of 9.4 ft and a minimum penetration depth of 87 ft. We were informed that some piles did not achieve these criteria. It was requested that another restrike test is performed on the pile with the lowest blow count and stroke in the abutment. Pile 29 was reportedly driven on December 12, 2011 to a blow count of 23 blows/ft with a corresponding hammer stroke of 9.1 ft. This pile was tested during restrike approximately 7 days after initial driving. CAPWAP analysis performed on data collected during the restrike indicated a total mobilized capacity of 477 kips, which exceeds the required ultimate bearing value of 383.2 kips. Based on this restrike results, GRL suggests that the driving criteria for the abutment piles be revised to a minimum of 23 blows/ft with a minimum corresponding average hammer stroke of 9.1 ft and a minimum penetration depth of 87 ft below existing grade.

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

Sincerely,
GRL Engineers, Inc.

Benjamin White, P.E.

Table 1: Summary of Case Method Results

ODOT 3000(10) - Bridge 6 - East 14th St Ramp to I-90 | Hammer: Pileco D30-32 Diesel

Pile No.	Test Date	Substructure	Test [1] Type	Penetration [2] Depth (ft)	Blow [3] Count blows/set	Hammer [4] Stroke (ft)	Transf'd Energy (kip-ft)	Max. Compressive [5] Force (kips)	Max. Compressive [5] Stress (ksi)	Case Method Capacity (kips)	CAPWAP Mobilized Capacity (kips)
31	22-Nov-11	Forward Abut	EOID	87' - 0"	29 / 1'	9.5	37.9	814	31.6	236	234
	29-Nov-11		BOR	87' - 0"	6 / 1"	10.7	40.7	861	33.4	463	400
42	22-Nov-11	Forward Abut	EOID	87' - 0"	30 / 1'	9.4	38.3	780	30.2	259	-
	29-Nov-11		BOR	87' - 0"	11 / 1"	9.8	36.4	807.0	31.3	451	450
59	13-Dec-11	Monument	EOID	85' - 0"	23 / 1'	7.3	28.4	371.0	27.6	267	254
	19-Dec-11		BOR	85' - 0"	10 / 1"	8.5	20.2	395.0	29.4	459	503
29	19-Dec-11	Forward Abut	BOR	87' - 0"	5 / 1"	10.0	37.9	825.0	32	587	477

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 or GRL personnel
4 - Stroke Calculated based on the time between impacts
5 - Stress from uniform axial average

Table 2: Summary of CAPWAP Results

Pile No.	Test Date	Substructure	Test Type	Blow Count	Penetration Depth (ft)	Mobilized Capacity Total (kips)	Mobilized Capacity Shaft (kips)	Mobilized Capacity Toe (kips)	Soil Damping Shaft (sec/ft)	Soil Damping Toe (sec/ft)	Soil Quake Shaft (in)	Soil Quake Toe (in)
31	22-Nov-11	Forward Abut	EOID	29 / 1'	87' - 0"	234	196	38	0.39	0.11	0.25	0.63
	29-Nov-11		BOR		87' - 0"	400	356	44	0.34	0.10	0.30	0.42
42	29-Nov-11	Forward Abut	BOR	11 / 1'	87' - 0"	450	408	42	0.35	0.04	0.30	0.36
59	13-Dec-11	Monument	EOID	23 / 1'	85' - 0"	254	189	65	0.16	0.04	0.08	0.73
	19-Dec-11		BOR	10 / 1"	85' - 0"	503	493	10	0.15	0.02	0.04	0.04
29	19-Dec-11	Forward Abut	BOR	5 / 1"	87' - 0"	477	434	43	0.21	0.21	0.04	0.35

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 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.

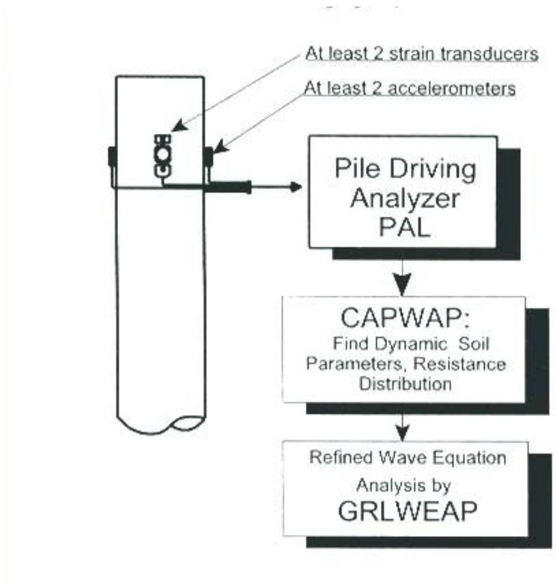


Figure 1. Block Diagram of Refined Wave Equation Analysis

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:

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

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

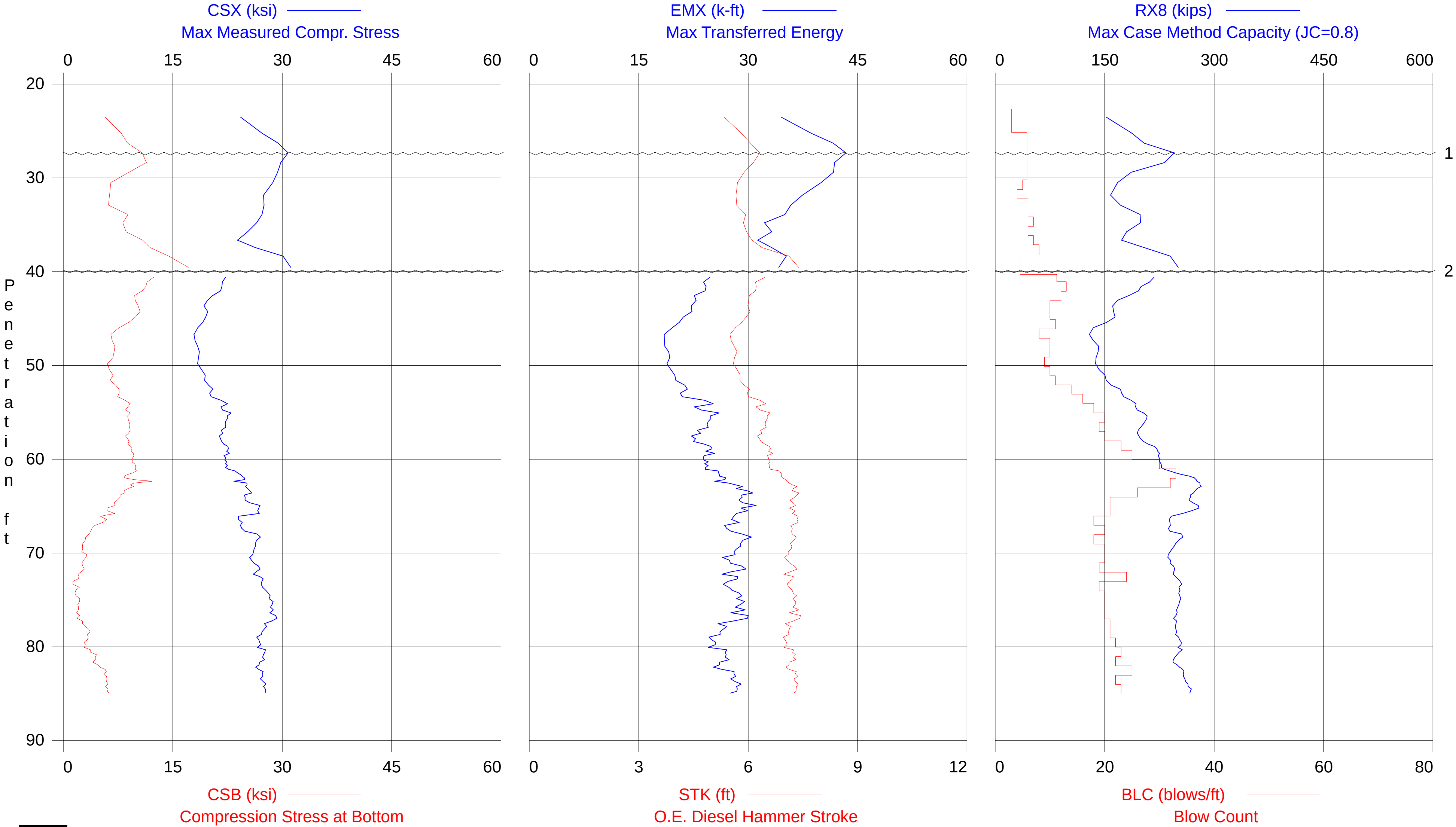
Case Method Results

PDIPLOT Ver. 2010.2 - Printed: 19-Dec-2011

Test date: 13-Dec-2011

ODOT 3000(10) Bridge 6 - Monument Pile 59

14" x 0.312" Grade 3 Closed End Pipe



1 - very high bending - poor data quality

2 - splice pile

ODOT 3000(10) Bridge 6 - Monument Pile 59 | 14" x 0.312" Grade 3 Closed End Pipe
OP: BAW | Test date: 13-Dec-2011

AR: 13.42 in^2 | SP: 0.492 k/ft3
LE: 42.7 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
47	23.00	3	AV2	19.8	37.5	2.8	22.8	265	4.5	114
			MAX	20.1	37.9	3.0	24.2	270	4.5	122
50	24.00	3	AV3	26.4	51.2	6.8	40.2	354	5.7	164
			MAX	27.2	52.8	7.9	47.0	365	6.0	179
53	25.00	3	AV3	27.2	51.5	8.2	41.3	366	5.9	189
			MAX	27.9	52.3	8.4	42.0	374	6.0	192
58	26.00	6	AV5	27.3	51.3	8.1	37.4	367	5.8	187
			MAX	28.3	53.7	9.0	39.3	379	5.9	189
64	27.00	6	AV6	30.0	56.3	9.0	43.0	403	6.2	212
			MAX	31.3	58.4	10.3	46.3	420	6.5	235
70	28.00	6	AV6	30.6	57.3	11.0	42.9	411	6.3	247
			MAX	31.5	58.3	11.5	44.5	423	6.4	251
76	29.00	6	AV6	29.8	56.3	11.2	41.4	400	6.1	227
			MAX	30.5	57.1	11.9	44.3	409	6.3	241
82	30.00	6	AV6	29.1	55.0	8.5	42.1	391	5.9	178
			MAX	30.7	57.2	9.7	45.4	412	6.2	194
87	31.00	5	AV5	28.7	54.3	6.3	39.3	385	5.7	169
			MAX	30.0	57.0	6.8	46.4	402	6.0	174
91	32.00	4	AV4	27.5	52.8	6.4	37.8	369	5.7	158
			MAX	28.5	54.3	6.7	42.0	383	5.9	162
97	33.00	6	AV6	27.3	52.3	5.9	35.1	366	5.6	164
			MAX	28.9	55.0	7.2	44.3	387	6.1	171
103	34.00	6	AV6	27.6	53.1	8.2	36.9	371	5.9	193
			MAX	28.8	55.2	9.8	43.2	387	6.1	205
110	35.00	7	AV7	26.8	50.1	8.6	33.1	360	5.9	199
			MAX	28.1	52.3	10.3	37.3	377	6.2	205
116	36.00	6	AV6	24.9	44.6	8.0	31.1	334	5.8	185
			MAX	26.8	48.5	10.1	38.3	360	6.2	198
123	37.00	7	AV7	24.4	45.0	10.9	32.8	327	6.2	173
			MAX	27.2	48.4	12.3	41.7	365	6.5	198
131	38.00	8	AV8	27.1	49.2	12.1	33.5	363	6.5	212
			MAX	30.8	54.5	13.8	36.3	413	7.2	239
135	39.00	5	AV4	30.5	54.2	15.2	35.7	409	7.2	242
			MAX	31.6	56.2	16.3	38.4	424	7.6	257
140	40.00	5	AV5	31.2	53.3	17.1	34.2	418	7.4	251
			MAX	31.9	54.2	18.3	38.6	428	7.6	256

ODOT 3000(10) Bridge 6 - Monument Pile 59 14" x 0.312" Grade 3 Closed End Pipe
OP: BAW Test date: 13-Dec-2011

BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
150	41.00	11	AV8	22.2	24.0	12.3	24.7	298	6.4	217
			MAX	25.0	28.3	13.9	31.8	336	7.4	228
163	42.00	13	AV13	21.7	22.6	11.1	24.0	291	6.2	202
			MAX	23.4	23.8	12.0	27.4	314	6.6	211
175	43.00	12	AV12	20.5	21.4	10.0	23.1	275	6.1	183
			MAX	21.7	22.8	11.1	25.1	290	6.3	200
185	44.00	10	AV10	19.4	20.1	10.3	22.4	260	6.0	162
			MAX	20.3	21.1	11.0	24.3	272	6.2	166
195	45.00	10	AV10	19.7	20.0	10.2	21.5	264	6.0	164
			MAX	21.3	21.6	11.3	25.4	286	6.4	168
206	46.00	11	AV11	19.1	20.1	8.6	20.6	256	5.8	147
			MAX	20.2	20.9	10.0	22.6	271	6.1	162
214	47.00	8	AV8	17.9	19.3	6.7	18.5	240	5.5	130
			MAX	19.0	20.7	7.4	20.5	255	5.8	134
224	48.00	10	AV10	18.3	19.8	6.8	18.6	245	5.6	137
			MAX	19.8	21.2	7.6	22.1	265	5.9	147
234	49.00	10	AV10	18.6	20.3	7.0	19.2	249	5.7	141
			MAX	20.1	21.9	7.7	22.0	270	6.0	145
243	50.00	9	AV9	18.4	20.5	6.4	18.9	247	5.6	138
			MAX	19.5	21.6	7.3	21.4	262	5.8	141
253	51.00	10	AV10	18.8	21.0	6.4	19.0	253	5.7	143
			MAX	21.0	23.4	7.4	24.4	282	6.2	151
264	52.00	11	AV11	19.7	22.3	6.7	21.0	265	5.9	153
			MAX	21.1	24.0	7.6	24.7	283	6.2	157
278	53.00	14	AV14	20.2	22.8	7.5	21.1	270	6.0	169
			MAX	22.2	25.2	8.2	25.4	297	6.5	176
294	54.00	16	AV16	21.0	23.4	8.1	22.7	282	6.2	182
			MAX	22.9	25.6	10.2	26.5	307	6.5	193
312	55.00	18	AV18	22.0	25.0	8.8	23.8	295	6.3	194
			MAX	23.5	27.0	9.8	27.0	315	6.7	201
332	56.00	20	AV20	22.6	25.8	9.0	25.1	303	6.5	207
			MAX	24.2	28.0	9.8	29.4	325	7.0	211
351	57.00	19	AV19	22.1	24.8	9.1	24.1	296	6.4	200
			MAX	22.8	25.9	9.9	26.3	306	6.7	205
371	58.00	20	AV20	21.6	24.1	8.8	22.9	290	6.3	197
			MAX	23.2	25.5	9.8	25.9	311	6.7	202
394	59.00	23	AV23	22.2	24.9	9.1	24.1	298	6.5	214
			MAX	23.8	26.4	10.1	26.6	319	6.9	223
419	60.00	25	AV25	22.4	24.7	9.5	24.3	300	6.6	224
			MAX	23.9	25.9	10.4	27.5	321	7.0	230
449	61.00	30	AV30	22.3	24.6	9.7	24.3	299	6.6	227
			MAX	23.0	25.2	10.6	25.9	309	6.8	235

ODOT 3000(10) Bridge 6 - Monument Pile 59
OP: BAW
14" x 0.312" Grade 3 Closed End Pipe
Test date: 13-Dec-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
482	62.00	33	AV33	23.9	25.7	9.2	25.8	321	6.9	251
			MAX	25.4	28.0	10.6	29.4	340	7.2	277
514	63.00	32	AV32	24.7	26.7	9.9	27.3	331	7.1	278
			MAX	26.6	28.8	15.2	30.7	357	7.6	288
540	64.00	26	AV26	25.3	27.9	8.3	29.5	340	7.3	272
			MAX	27.2	30.5	12.5	33.6	365	7.8	281
561	65.00	21	AV21	25.6	30.3	7.3	29.5	343	7.2	270
			MAX	27.7	33.4	8.4	32.8	371	7.5	281
582	66.00	21	AV21	26.5	32.2	6.2	29.0	356	7.2	267
			MAX	28.1	33.5	10.6	32.4	378	7.6	291
600	67.00	18	AV18	24.3	30.3	5.4	28.1	325	7.3	240
			MAX	25.3	31.6	6.8	30.3	339	7.6	244
620	68.00	20	AV20	24.9	30.6	3.9	27.6	334	7.2	241
			MAX	28.4	33.0	4.8	33.1	380	7.7	259
638	69.00	18	AV18	26.7	31.5	3.0	29.8	358	7.3	253
			MAX	27.6	33.0	4.1	31.9	370	7.5	260
658	70.00	20	AV20	26.1	30.9	2.6	28.4	351	7.1	243
			MAX	27.1	32.1	3.4	30.9	364	7.5	249
678	71.00	20	AV20	25.8	30.1	3.1	27.5	347	7.1	238
			MAX	27.4	31.4	4.1	31.1	368	7.5	244
697	72.00	19	AV19	26.7	31.1	2.7	28.9	358	7.3	244
			MAX	27.4	32.2	4.0	31.4	368	7.6	248
721	73.00	24	AV24	26.7	30.9	2.0	27.5	359	7.1	248
			MAX	28.2	32.4	3.3	30.2	379	7.4	254
740	74.00	19	AV19	27.4	31.1	1.7	27.4	368	7.1	254
			MAX	28.2	32.2	3.1	30.1	379	7.5	256
760	75.00	20	AV20	28.1	31.9	1.9	28.5	377	7.2	253
			MAX	29.0	32.7	2.6	31.1	389	7.6	256
780	76.00	20	AV20	28.6	33.0	2.1	29.0	384	7.3	251
			MAX	29.4	34.2	3.3	31.4	394	7.6	254
800	77.00	20	AV20	28.8	33.8	2.0	29.2	387	7.3	248
			MAX	29.7	35.2	3.3	31.8	398	7.8	254
821	78.00	21	AV21	28.1	32.4	2.7	27.3	378	7.2	247
			MAX	29.2	33.8	3.8	30.2	391	7.5	252
842	79.00	21	AV21	27.0	31.1	3.5	25.6	362	7.0	249
			MAX	28.8	32.5	4.4	30.2	387	7.5	258
864	80.00	22	AV22	27.0	30.7	3.1	25.5	362	7.1	253
			MAX	28.0	32.6	4.3	29.4	375	7.7	269
887	81.00	23	AV23	27.4	30.2	3.8	26.5	367	7.2	252
			MAX	28.7	31.6	5.1	30.1	384	7.6	266
909	82.00	22	AV22	27.1	30.0	4.4	26.5	364	7.2	246
			MAX	28.4	32.0	6.1	29.8	381	7.7	255

ODOT 3000(10) Bridge 6 - Monument Pile 59 14" x 0.312" Grade 3 Closed End Pipe
OP: BAW Test date: 13-Dec-2011

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	CSB ksi	EMX k-ft	FMX kips	STK ft	RX8 kips
934	83.00	25	AV25	26.9	31.7	5.6	26.9	362	7.2	256
			MAX	28.0	33.8	6.7	29.9	376	7.6	266
956	84.00	22	AV22	27.3	34.2	6.0	28.2	366	7.3	260
			MAX	28.0	35.3	6.7	30.5	376	7.6	268
979	85.00	23	AV23	27.6	36.7	6.0	28.4	371	7.3	267
			MAX	28.9	38.5	7.1	31.0	387	7.6	277

BL#	depth (ft)	Comments
67	27.41	very high bending - poor data quality
140	39.98	splice pile
141	40.20	alignment improved after splice - good quality data

Time Summary

Drive	9 minutes 17 seconds	10:11:33 AM - 10:20:50 AM (12/13/2011) BN 1 - 140
Stop	1 hour 55 seconds	10:20:50 AM - 11:21:45 AM
Drive	18 minutes 34 seconds	11:21:45 AM - 11:40:19 AM BN 141 - 979

Total time [1:28:46] = (Driving [0:27:51] + Stop [1:00:55])

ODOT 3000(10) Bridge 6 - Monument Pile 59 Restrike
OP: BAW

14" x 0.312" Grade 3 Closed End Pipe
Test date: 19-Dec-2011

AR: 13.42 in^2
LE: 87.7 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#	depth	BLC	CSX	CSI	CSB	EMX	FMX	STK	RX8
	ft	bl/ft	ksi	ksi	ksi	k-ft	kips	ft	kips
1	85.00	120	25.1	32.4	3.5	10.7	337	0.0	365
2	85.01	120	34.7	39.0	0.0	14.8	466	6.2	459
5	85.03	120	29.9	36.8	1.9	13.0	401	9.1	392
6	85.04	120	34.7	39.3	7.9	28.8	465	8.4	494
7	85.05	120	34.3	37.3	10.1	32.1	461	9.1	438
8	85.06	120	24.4	30.7	6.9	20.9	328	9.3	327
9	85.07	120	23.5	24.6	3.1	17.8	316	8.7	354
10	85.07	120	29.3	31.9	2.3	22.0	393	8.7	443
11	85.08	120	28.7	31.1	8.2	21.7	385	8.5	441
		Average	29.4	33.7	4.9	20.2	395	8.5	412
		Maximum	34.7	39.3	10.1	32.1	466	9.3	494

Total number of blows analyzed: 9

BL#	depth (ft)	Comments
3	85.02	high stress - unreliable data
4	85.02	high stress - unreliable data
5	85.03	pile top damage

Time Summary

Drive 15 seconds 10:00:45 AM - 10:01:00 AM (12/19/2011) BN 1 - 11

Bridge 6 E 14 Ramp to I-90 - Forward Abutment Pile 29 Restrike HP 16x88
OP: BAW Test date: 19-Dec-2011

AR: 25.80 in^2 SP: 0.492 k/ft3
LE: 90.0 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)

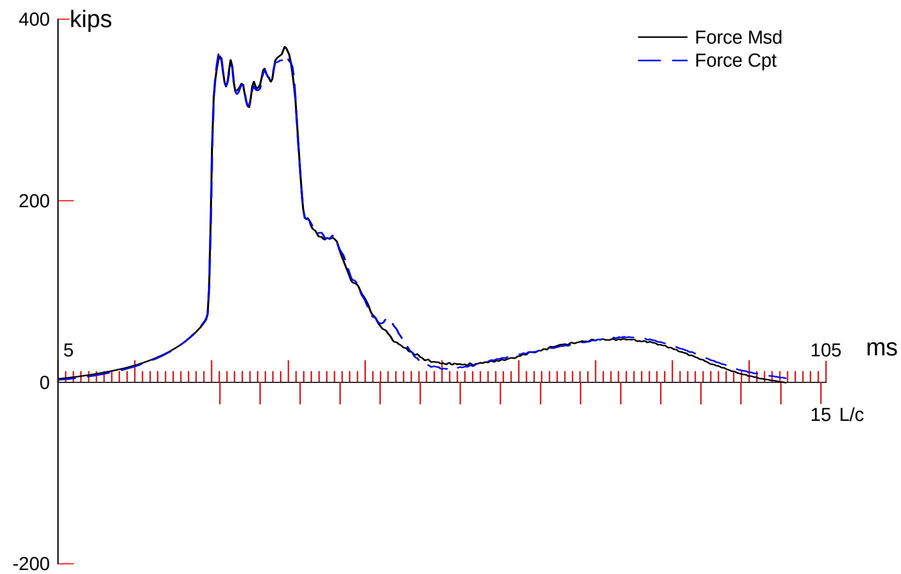
BL#	depth	BLC	TYPE	CSX	CSI	CSB	EMX	FMX	STK	RX8
end	ft	bl/ft		ksi	ksi	ksi	k-ft	kips	ft	kips
6	87.08	60	AV5	32.0	32.2	16.3	37.9	825	10.0	587
			MAX	33.5	34.2	17.3	40.1	864	10.7	607
12	87.17	72	AV6	31.5	31.8	17.1	37.1	814	9.8	594
			MAX	32.3	32.4	17.3	38.4	833	10.2	610
20	87.25	96	AV8	31.7	31.9	17.3	37.4	817	9.9	577
			MAX	32.6	32.9	18.2	40.3	841	10.4	595

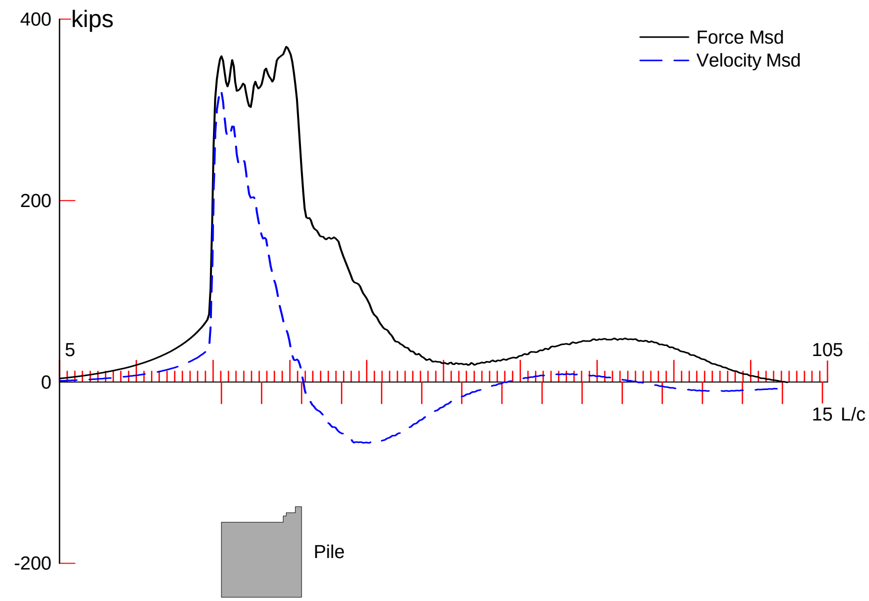
Time Summary

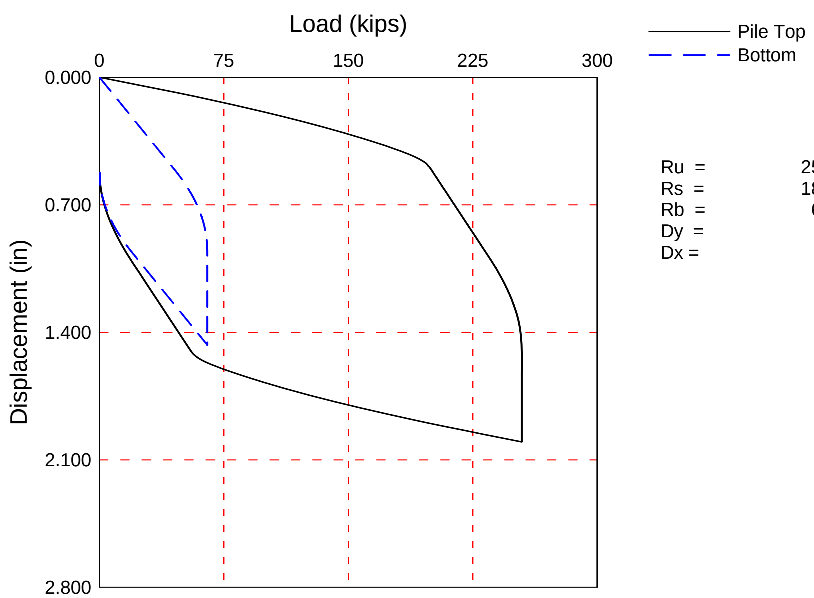
Drive 30 seconds 10:11:14 AM - 10:11:44 AM (12/19/2011) BN 1 - 20

Appendix C

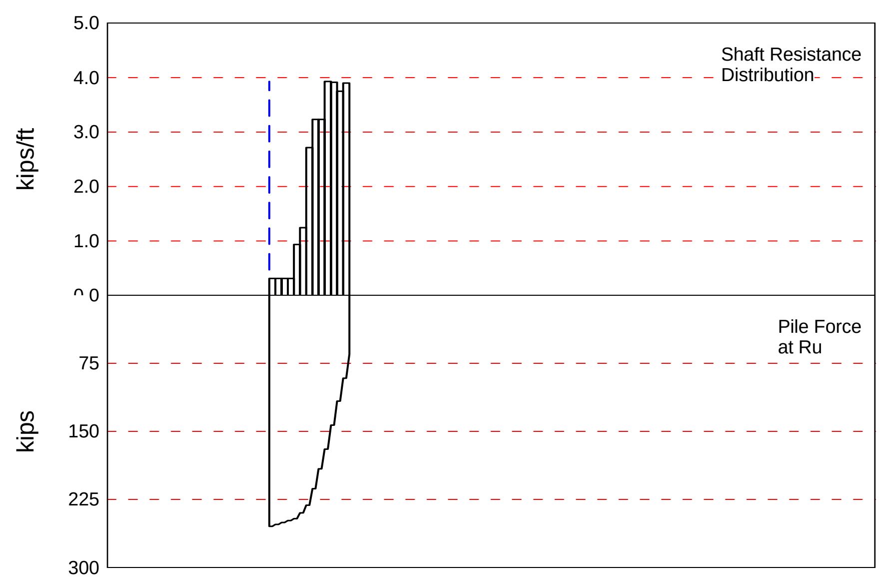
CAPWAP Results







Ru = 254.5 kips
Rs = 189.5 kips
Rb = 65.0 kips
Dy = 1.48 in
Dx = 2.00 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 254.5; along Shaft 189.5; at Toe 65.0 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
				254.5				
1	6.7	4.0	2.1	252.4	2.1	0.52	0.14	0.159
2	13.5	10.8	2.1	250.3	4.2	0.31	0.08	0.159
3	20.2	17.5	2.1	248.2	6.3	0.31	0.08	0.159
4	27.0	24.3	2.1	246.1	8.4	0.31	0.08	0.159
5	33.7	31.0	6.3	239.8	14.7	0.93	0.25	0.159
6	40.5	37.8	8.4	231.4	23.1	1.25	0.34	0.159
7	47.2	44.5	18.3	213.1	41.4	2.71	0.74	0.159
8	54.0	51.3	21.8	191.3	63.2	3.23	0.88	0.159
9	60.7	58.0	21.8	169.5	85.0	3.23	0.88	0.159
10	67.5	64.8	26.5	143.0	111.5	3.93	1.07	0.159
11	74.2	71.5	26.4	116.6	137.9	3.91	1.07	0.159
12	81.0	78.3	25.3	91.3	163.2	3.75	1.02	0.159
13	87.7	85.0	26.3	65.0	189.5	3.90	1.06	0.159
Avg. Shaft			14.6			2.23	0.61	0.159
Toe			65.0				60.80	0.038

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.080	0.728
Case Damping Factor		1.260	0.102
Damping Type			Smith
Unloading Quake	(% of loading quake)	100	53
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	5	
Resistance Gap (included in Toe Quake) (in)			0.028
Soil Plug Weight	(kips)		0.43

CAPWAP match quality	= 1.60	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.522 in;	blow count	=	23 b/ft
Computed: final set	= 0.561 in;	blow count	=	21 b/ft

max. Top Comp. Stress	=	27.3 ksi	(T= 26.5 ms, max= 1.102 x Top)
max. Comp. Stress	=	30.1 ksi	(Z= 30.4 ft, T= 33.9 ms)
max. Tens. Stress	=	0.00 ksi	(Z= 3.4 ft, T= 0.0 ms)
max. Energy (EMX)	=	28.3 kip-ft;	max. Measured Top Displ. (DMX)= 1.12 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	366.5	0.0	27.3	0.00	28.34	13.1	1.123
2	6.7	376.4	0.0	28.0	0.00	28.17	13.0	1.103
4	13.5	380.5	0.0	28.4	0.00	27.40	12.8	1.063
6	20.2	386.8	0.0	28.8	0.00	26.67	12.6	1.023
8	27.0	396.6	0.0	29.6	0.00	25.94	12.3	0.982
10	33.7	398.9	0.0	29.7	0.00	25.23	11.8	0.941
11	37.1	391.4	0.0	29.2	0.00	24.05	11.6	0.920
12	40.5	401.8	0.0	29.9	0.00	23.88	11.2	0.900
13	43.9	391.7	0.0	29.2	0.00	22.50	10.8	0.881
14	47.2	391.5	0.0	29.2	0.00	22.34	10.3	0.861
15	50.6	367.1	0.0	27.4	0.00	19.77	9.8	0.844
16	54.0	379.5	0.0	28.3	0.00	19.66	9.4	0.827
17	57.3	341.1	0.0	25.4	0.00	16.86	9.0	0.813
18	60.7	343.7	0.0	25.6	0.00	16.77	8.3	0.798
19	64.1	317.9	0.0	23.7	0.00	14.15	7.8	0.786
20	67.5	324.0	0.0	24.1	0.00	14.10	7.9	0.775
21	70.8	263.3	0.0	19.6	0.00	11.10	7.9	0.767
22	74.2	267.7	0.0	19.9	0.00	11.07	8.1	0.760
23	77.6	235.3	0.0	17.5	0.00	8.14	8.7	0.755
24	81.0	234.1	0.0	17.4	0.00	8.13	8.7	0.750
25	84.3	182.5	0.0	13.6	0.00	5.33	8.4	0.746
26	87.7	153.6	0.0	11.4	0.00	2.33	8.8	0.742
Absolute	30.4			30.1			(T =	33.9 ms)
	3.4				0.00		(T =	0.0 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	462.0	439.7	417.4	395.1	372.8	350.4	328.1	305.8	283.5	261.2
RX	462.0	439.7	417.4	395.1	372.8	350.4	328.1	305.8	283.5	261.2
RU	462.0	439.7	417.4	395.1	372.8	350.4	328.1	305.8	283.5	261.2

RAU = 62.0 (kips); RA2 = 354.6 (kips)

Current CAPWAP Ru = 254.5 (kips); Corresponding J(RP)= 1.00; 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
13.60	26.29	325.7	359.4	372.3	1.124	0.522	0.522	28.4	414.0

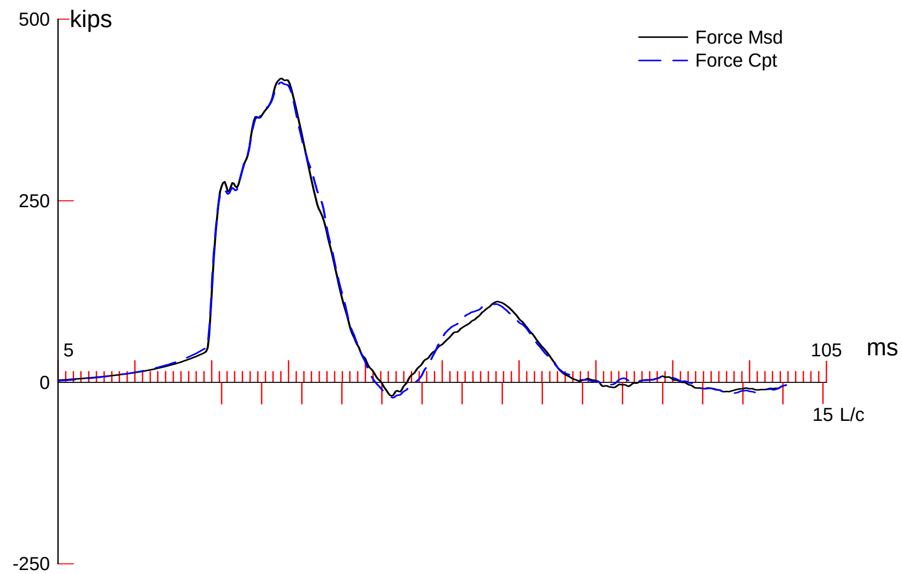
PILE PROFILE AND PILE MODEL

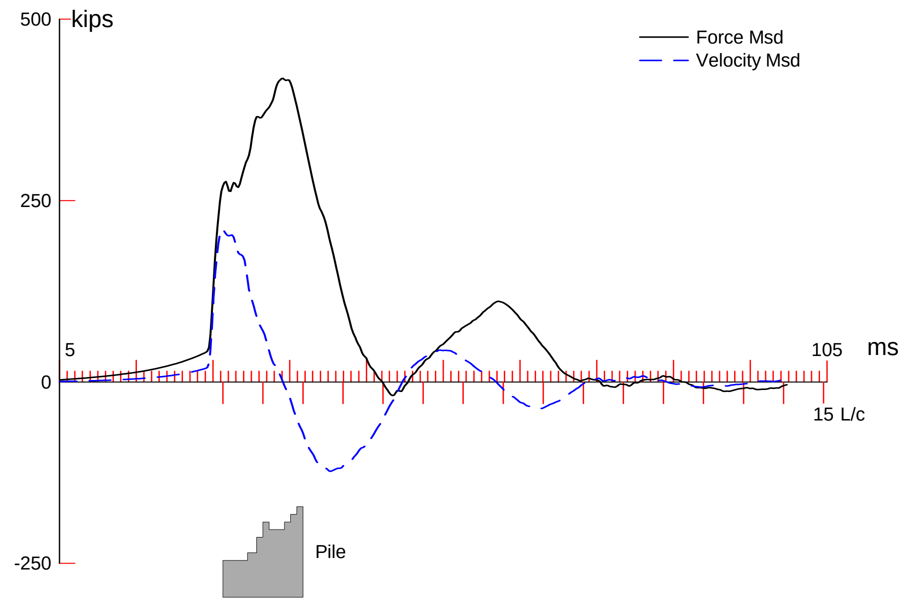
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	13.42	29992.2	492.000	3.665
87.70	13.42	29992.2	492.000	3.665

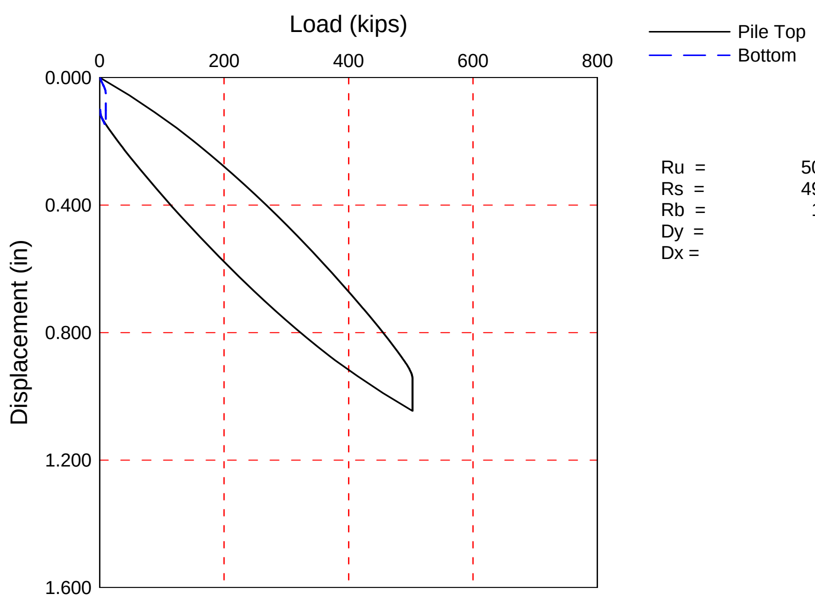
Toe Area 1.069 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.37	23.95	0.00	0.000	0.000	-0.000	0.000	3.665
21	70.83	25.95	8.35	0.000	0.000	-0.000	0.000	3.665
22	74.21	26.95	12.53	0.000	0.000	-0.000	0.000	3.665
25	84.33	28.95	20.88	0.000	0.000	-0.000	0.000	3.665
26	87.70	28.95	20.88	0.000	0.000	-0.000	0.000	3.665

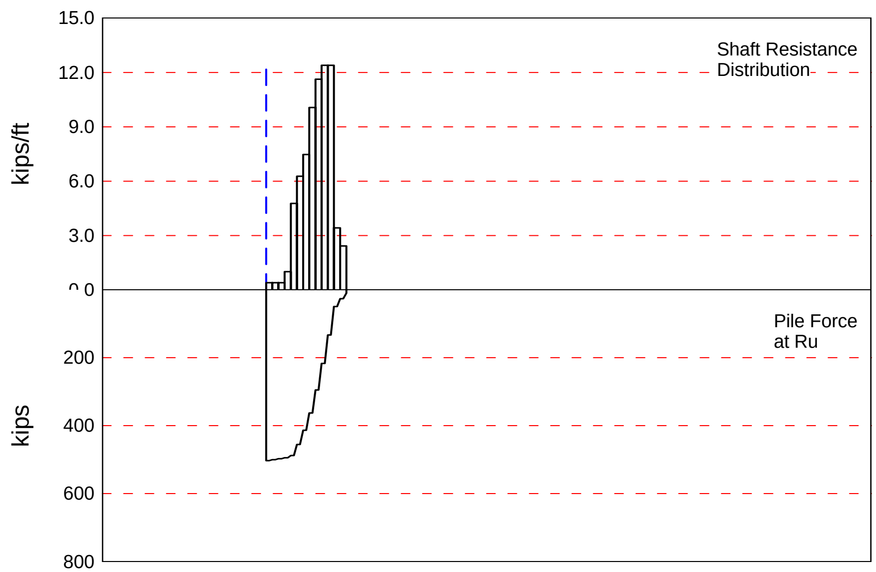
Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 16807.9 ft/s, 2L/c 10.4 ms







Ru =	502.8	kips
Rs =	492.8	kips
Rb =	10.0	kips
Dy =	0.95	in
Dx =	1.05	in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 502.8; along Shaft 492.8; at Toe 10.0 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
				502.8				
1	6.7	4.0	2.7	500.1	2.7	0.67	0.18	0.150
2	13.5	10.8	2.7	497.4	5.4	0.40	0.11	0.150
3	20.2	17.5	2.7	494.7	8.1	0.40	0.11	0.150
4	27.0	24.3	6.8	487.9	14.9	1.01	0.28	0.150
5	33.7	31.0	32.2	455.7	47.1	4.77	1.30	0.150
6	40.5	37.8	42.3	413.4	89.4	6.27	1.71	0.150
7	47.2	44.5	50.4	363.0	139.8	7.47	2.04	0.150
8	54.0	51.3	67.9	295.1	207.7	10.07	2.75	0.150
9	60.7	58.0	78.4	216.7	286.1	11.62	3.17	0.150
10	67.5	64.8	83.6	133.1	369.7	12.39	3.38	0.150
11	74.2	71.5	83.6	49.5	453.3	12.39	3.38	0.150
12	81.0	78.3	23.1	26.4	476.4	3.42	0.93	0.150
13	87.7	85.0	16.4	10.0	492.8	2.43	0.66	0.150
Avg. Shaft			37.9			5.80	1.58	0.150
Toe			10.0				9.35	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.039	0.041
Case Damping Factor		3.096	0.010
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	33
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	55	
Resistance Gap (included in Toe Quake) (in)			0.000
Soil Plug Weight	(kips)		1.14

CAPWAP match quality	= 2.71	(Wave Up Match)	; RSA = 0	
Observed: final set	= 0.100 in;	blow count	=	120 b/ft
Computed: final set	= 0.062 in;	blow count	=	195 b/ft

max. Top Comp. Stress	=	31.1 ksi	(T= 34.3 ms, max= 1.018 x Top)
max. Comp. Stress	=	31.7 ksi	(Z= 13.5 ft, T= 34.7 ms)
max. Tens. Stress	=	-3.16 ksi	(Z= 33.7 ft, T= 50.2 ms)
max. Energy (EMX)	=	12.9 kip-ft;	max. Measured Top Displ. (DMX)= 0.63 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	417.4	-26.3	31.1	-1.96	12.85	9.0	0.588
2	6.7	421.1	-29.7	31.4	-2.21	12.11	8.9	0.546
4	13.5	424.8	-33.8	31.7	-2.52	10.38	8.6	0.460
6	20.2	422.1	-37.2	31.5	-2.77	8.78	7.8	0.377
8	27.0	420.5	-41.3	31.3	-3.08	7.29	6.6	0.296
10	33.7	421.8	-42.4	31.4	-3.16	5.92	5.2	0.229
11	37.1	394.2	-27.2	29.4	-2.03	4.63	4.4	0.198
12	40.5	400.0	-28.3	29.8	-2.11	4.26	4.0	0.174
13	43.9	359.2	-8.5	26.8	-0.63	3.23	3.7	0.152
14	47.2	355.0	-10.0	26.5	-0.75	2.99	3.3	0.135
15	50.6	312.4	0.0	23.3	0.00	2.21	3.0	0.121
16	54.0	316.6	0.0	23.6	0.00	2.01	2.6	0.105
17	57.3	247.4	0.0	18.4	0.00	1.34	2.3	0.092
18	60.7	244.5	0.0	18.2	0.00	1.21	2.0	0.078
19	64.1	181.3	0.0	13.5	0.00	0.73	1.8	0.069
20	67.5	181.0	0.0	13.5	0.00	0.66	1.5	0.059
21	70.8	113.4	0.0	8.4	0.00	0.36	1.4	0.054
22	74.2	115.7	0.0	8.6	0.00	0.34	1.2	0.048
23	77.6	74.7	-2.7	5.6	-0.20	0.13	1.2	0.047
24	81.0	83.0	0.0	6.2	0.00	0.13	1.2	0.046
25	84.3	76.3	-10.8	5.7	-0.80	0.08	1.4	0.045
26	87.7	65.2	-3.7	4.9	-0.27	0.02	1.6	0.045
Absolute	13.5			31.7			(T =	34.7 ms)
	33.7				-3.16		(T =	50.2 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	446.5	443.2	439.8	436.5	433.1	429.8	426.4	423.1	419.7	416.4
RX	446.7	443.2	439.8	436.5	433.1	429.8	426.4	423.1	419.7	416.4
RU	471.2	470.3	469.4	468.5	467.6	466.8	465.9	465.0	464.1	463.2

RAU = 0.0 (kips); RA2 = 446.5 (kips)

Current CAPWAP Ru = 502.8 (kips); Corresponding J(RP)= 0.00; J(RX) = 0.00

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
8.81	26.49	211.0	269.0	419.6	0.632	0.100	0.100	13.5	442.0

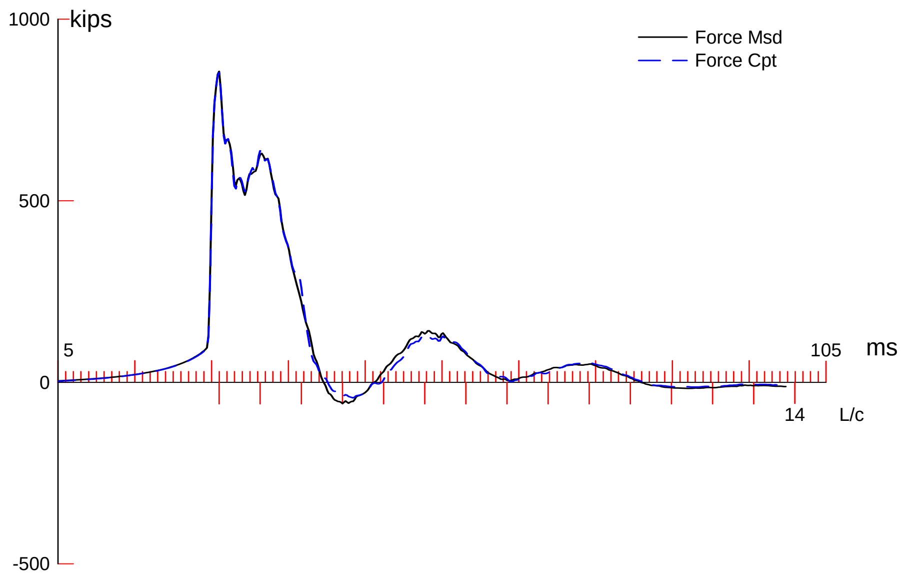
PILE PROFILE AND PILE MODEL

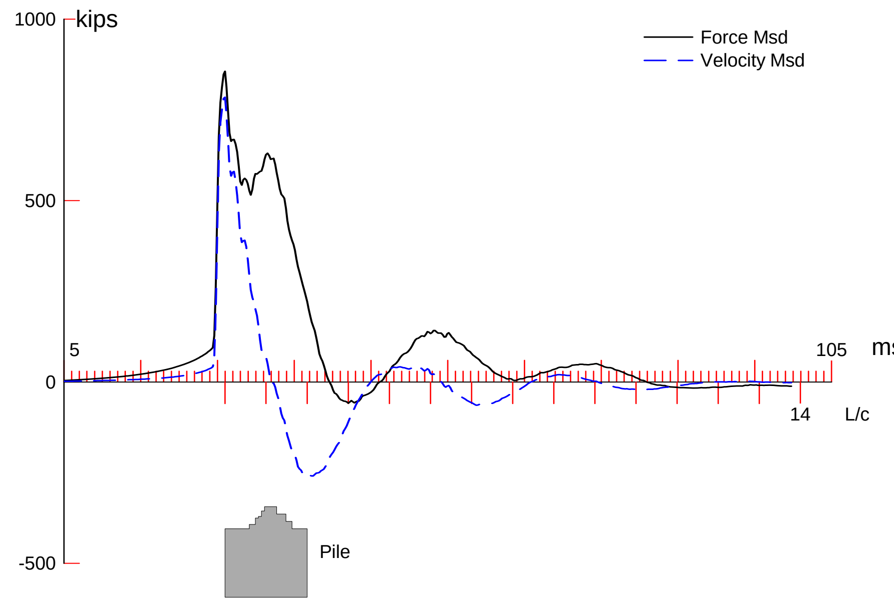
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	13.42	29992.2	492.000	3.665
87.70	13.42	29992.2	492.000	3.665

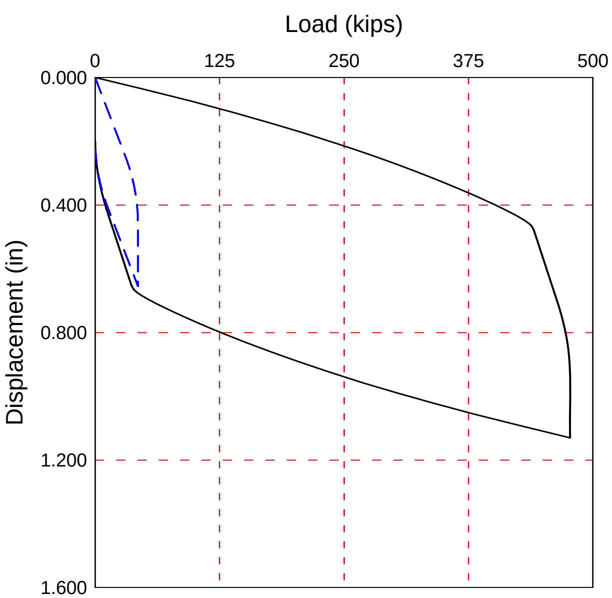
Toe Area 1.069 ft^2

Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.37	23.95	0.00	0.000	0.000	-0.000	0.000	3.665
9	30.36	28.95	20.88	0.000	0.000	-0.000	0.000	3.665
12	40.48	38.95	62.64	0.000	0.000	-0.000	0.000	3.665
14	47.22	48.95	104.40	0.000	0.000	-0.000	0.000	3.665
16	53.97	43.95	83.52	0.000	0.000	-0.000	0.000	3.665
21	70.83	48.95	104.40	0.000	0.000	-0.000	0.000	3.665
23	77.58	53.95	125.28	0.000	0.000	-0.000	0.000	3.665
25	84.33	58.95	146.16	0.000	0.000	-0.000	0.000	3.665
26	87.70	58.95	146.16	0.000	0.000	-0.000	0.000	3.665

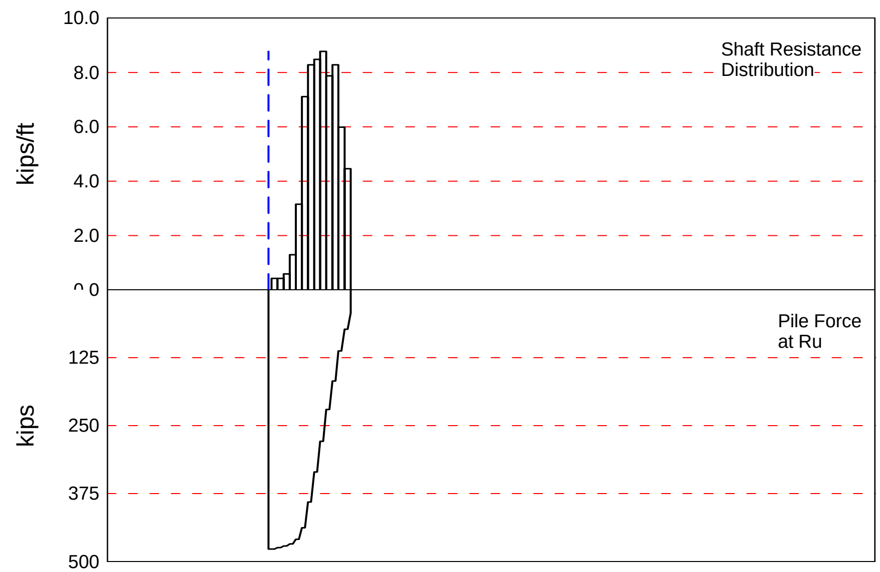
Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 16807.9 ft/s, 2L/c 10.4 ms







Ru =	477.0	kips
Rs =	434.0	kips
Rb =	43.0	kips
Dy =	0.93	in
Dx =	1.13	in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 477.0; along Shaft 434.0; at Toe 43.0 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
				477.0				
1	10.0	7.0	2.8	474.2	2.8	0.40	0.08	0.210
2	16.7	13.7	2.8	471.4	5.6	0.42	0.08	0.210
3	23.3	20.3	3.9	467.5	9.5	0.59	0.11	0.210
4	30.0	27.0	8.6	458.9	18.1	1.29	0.25	0.210
5	36.7	33.7	21.0	437.9	39.1	3.15	0.61	0.210
6	43.3	40.3	47.4	390.5	86.5	7.11	1.38	0.210
7	50.0	47.0	55.2	335.3	141.7	8.28	1.60	0.210
8	56.7	53.7	56.5	278.8	198.2	8.48	1.64	0.210
9	63.3	60.3	58.5	220.3	256.7	8.78	1.70	0.210
10	70.0	67.0	52.5	167.8	309.2	7.88	1.52	0.210
11	76.7	73.7	55.2	112.6	364.4	8.28	1.60	0.210
12	83.3	80.3	39.9	72.7	404.3	5.99	1.16	0.210
13	90.0	87.0	29.7	43.0	434.0	4.46	0.86	0.210
Avg. Shaft			33.4			4.99	0.97	0.210
Toe			43.0				25.78	0.207

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.040	0.350
Case Damping Factor		1.979	0.193
Unloading Quake	(% of loading quake)	15	40
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kips)		0.83

CAPWAP match quality = 1.91 (Wave Up Match) ; RSA = 0
Observed: final set = 0.200 in; blow count = 60 b/ft
Computed: final set = 0.205 in; blow count = 58 b/ft

max. Top Comp. Stress = 33.3 ksi (T= 26.4 ms, max= 1.185 x Top)
max. Comp. Stress = 39.4 ksi (Z= 43.3 ft, T= 28.8 ms)
max. Tens. Stress = -4.55 ksi (Z= 50.0 ft, T= 44.2 ms)
max. Energy (EMX) = 39.1 kip-ft; max. Measured Top Displ. (DMX)= 0.78 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	858.1	-52.5	33.3	-2.03	39.07	17.0	0.750
2	6.7	862.0	-61.1	33.4	-2.37	38.23	16.9	0.718
4	13.3	859.2	-79.0	33.3	-3.06	36.11	16.7	0.654
6	20.0	876.8	-94.0	34.0	-3.64	34.21	16.1	0.594
8	26.7	910.3	-107.2	35.3	-4.15	32.29	15.1	0.536
10	33.3	948.7	-110.5	36.8	-4.28	30.24	13.7	0.481
12	40.0	960.4	-107.5	37.2	-4.16	28.01	12.0	0.455
13	43.3	1016.7	-115.2	39.4	-4.46	27.98	11.2	0.444
14	46.7	906.3	-109.2	35.1	-4.23	23.96	10.3	0.435
15	50.0	937.7	-117.5	36.3	-4.55	23.94	9.8	0.426
16	53.3	806.4	-103.9	31.2	-4.03	19.88	9.2	0.418
17	56.7	839.1	-104.6	32.5	-4.05	19.85	8.7	0.410
18	60.0	710.4	-87.4	27.5	-3.39	16.09	8.3	0.403
19	63.3	730.3	-81.8	28.3	-3.17	16.06	8.0	0.394
20	66.7	595.5	-64.4	23.1	-2.50	12.49	7.7	0.388
21	70.0	618.3	-64.9	24.0	-2.52	12.47	7.5	0.381
22	73.3	512.0	-47.7	19.8	-1.85	9.40	7.2	0.376
23	76.7	529.2	-46.5	20.5	-1.80	9.39	6.8	0.370
24	80.0	442.3	-26.0	17.1	-1.01	6.27	6.7	0.366
25	83.3	486.2	-23.6	18.8	-0.91	6.26	7.5	0.362
26	86.7	369.3	-11.8	14.3	-0.46	4.00	7.8	0.360
27	90.0	305.9	-22.9	11.9	-0.89	2.05	8.5	0.358
Absolute	43.3			39.4			(T =	28.8 ms)
	50.0				-4.55		(T =	44.2 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	1066.7	1007.6	948.6	889.5	830.4	771.3	712.3	653.2	594.1	535.1
RX	1066.7	1007.6	948.6	889.5	830.4	771.3	712.3	653.2	594.1	535.1
RU	1260.2	1220.5	1180.8	1141.0	1101.3	1061.6	1021.9	982.1	942.4	902.7

RAU = 0.0 (kips); RA2 = 460.9 (kips)

Current CAPWAP Ru = 477.0 (kips); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
17.22	26.18	793.0	864.4	864.4	0.777	0.200	0.200	40.1	984.3

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	25.80	29992.2	492.000	5.166
90.00	25.80	29992.2	492.000	5.166

Toe Area 1.668 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	Soil Plug kips
1	3.33	46.05	0.00	0.000	0.000	-0.000	0.000	5.166	0.00
9	30.00	49.05	6.51	0.000	0.000	-0.000	0.000	5.166	0.00
11	36.67	53.27	15.68	0.000	0.000	-0.000	0.000	5.166	0.00
12	40.00	54.38	18.09	0.000	0.000	-0.000	0.000	5.166	0.00
13	43.33	58.05	26.06	0.000	0.000	-0.000	0.000	5.166	0.00
14	46.67	61.05	32.57	0.000	0.000	-0.000	0.000	5.166	0.00
16	53.33	61.05	32.57	0.000	0.000	-0.000	0.000	5.166	0.05
17	56.67	61.05	32.57	0.000	0.000	-0.000	0.000	5.166	0.06
18	60.00	56.05	21.72	0.000	0.000	-0.000	0.000	5.166	0.08
19	63.33	56.05	21.72	0.000	0.000	-0.000	0.000	5.166	0.10
21	70.00	51.05	10.86	0.000	0.000	-0.000	0.000	5.166	0.10
22	73.33	51.05	10.86	0.000	0.000	-0.000	0.000	5.166	0.09
23	76.67	46.05	0.00	0.000	0.000	-0.000	0.000	5.166	0.08
24	80.00	46.05	0.00	0.000	0.000	-0.000	0.000	5.166	0.06
25	83.33	46.05	0.00	0.000	0.000	-0.000	0.000	5.166	0.05

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
27	90.00	46.05	0.00	0.000	0.000	-0.000	0.000	5.166	0.05

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 10.7 ms