

APPLICATIONS AND CORRELATIONS OF THE WAVE EQUATION ANALYSIS PROGRAM GRLWEAP

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ABSTRACT

Since its introduction by E.A.L. Smith in the 1950s, the wave equation analysis of pile driving has enjoyed increasing popularity among geotechnical engineers and foundation contractors. This numerical analysis approach for the simulation of the pile installation process has also undergone repeated modifications, expansions and developments. It is for this reason that the wave equation analysis features should be summarized, the program capabilities and limitations described and the realism of its answers investigated.

Using the GRLWEAP™ program as an example, the paper describes how today wave equation analysis is generally performed, what options are available and where the shortcomings and strengths of the wave equation approach may be found. Furthermore, since existing correlations are not available for the most recently developed program version, measured stresses, capacities and hammer performance parameters are compared with calculated values. The cases selected for these studies are taken from typical construction sites rather than for well controlled pile test programs. This method of correlation better assures that the typical program performance for the typical construction site can be assessed. Also, since wave equation analyses are more and more frequently done as driveability studies, i.e. the calculation of blow counts given a static soil analysis, the reliability of that approach is discussed.

KEYWORDS: deep foundations, driven piles, bearing capacity, driveability, wave equation, pile stresses, residual stresses

1. INTRODUCTION

Since the 1940s the wave equation approach of simulating the pile driving process has enjoyed greater and greater acceptance and it is now a rather mature tool for the deep foundation industry. While the basic one-dimensional Smith approach is most widely accepted, there are some details that several academic researchers and software developers have formulated in different ways. For example, instead of Smith's lumped mass pile model the characteristics approach could be used as in CAPWAP® (PDI, 2000) and instead of using the partially non-dimensionalized Smith soil damping definition, Randolph (1992) proposes viscous damping factors, based on the soil's shear modulus rather than its grain size. Most importantly, however, modeling the soil's effect on pile driving as a soil-pile interface force, dependent on the pile's displacement and velocity, is still the preferred theory. Axisymmetric analyses of the pile and the soil's half-space have not been accepted in practice.

While there are several different codes available to solve the wave equation, the following paper will only investigate the performance of the GRLWEAP program. This program had been commissioned by and was delivered to the Federal Highway Administration (FHWA) in 1976 (Goble & Rausche, 1976)

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and named WEAP. Later after conversion as a PC software it was renamed GRLWEAP and is now maintained on a proprietary basis (PDI, 2003).

2. THE MATHEMATICAL/MECHANICAL MODEL

The program actually models several subsystems: hammer, driving system (cushions, helmet), pile and soil (Figure 1). For the hammer, the program provides a large data file which is a compilation of basic properties of many impact hammers, powered by either air, steam, hydraulic pressure, or diesel combustion. The program also provides for the analysis of vibratory hammers. Driving system data is provided in data files that have been provided by courtesy of the hammer manufacturers or their representatives. The analyst can even find information on a variety of pile types, sizes and their material properties. Finally, there is some help for static soil resistance analysis and dynamic soil resistance parameters.

The analyst selects the applicable basic data and the program then sets up a model that consists primarily of springs, masses and dashpots. More detail on these models can be found in a variety of publications.

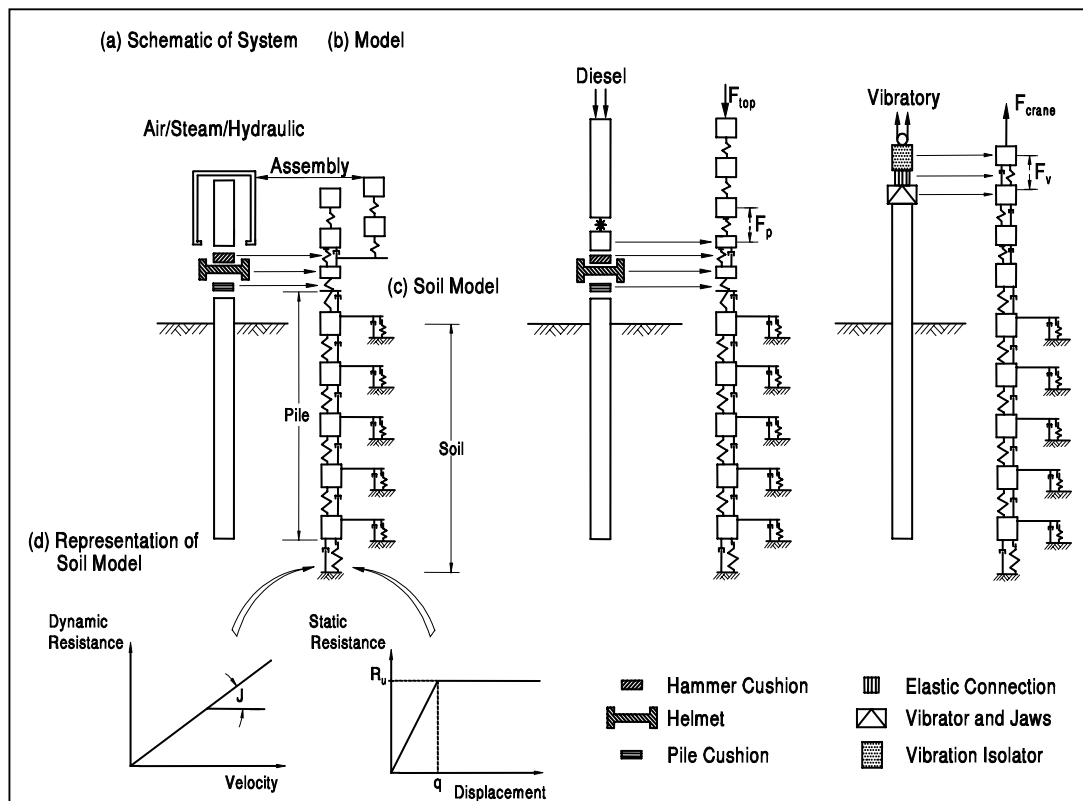


Figure 1. GRLWEAP Models

It may be surprising to some analysts that the program is still based on the very basic and simple lumped mass model that Smith had originally developed. Comparisons with the performance of the method of characteristics which is used for example in the CAPWAP program has shown that the Smith model provides for much greater flexibility and realism with the analysis of non-linearly behaving components such as cushions or splices. Sufficient accuracy can be easily achieved by choosing small model segments as discussed below.

3. ANALYSIS OPTIONS AND APPLICATIONS

In the past, the wave equation approach has been frequently described and tests of its realism have already been performed in the 1970s when the US Federal Highway Administration (FHWA) began its acceptance and dissemination of this technology (Hirsch et al 1976; Goble & Rausche 1976). Those tests were primarily concerned with a check on the bearing capacity prediction based on blow count and the wave equation calculated bearing graph. Similar correlations were shown in the documentation of the 1976 WEAP program and it was referenced by subsequent program updates.

Since the 1970s computer hardware and software systems rapidly developed, new hammers have been introduced and pile driving applications have been expanded to include very large and long piles or those with splices. These developments required program updating and improvement at regular time intervals, eventually leading to today's proprietary computer program and its support data files (PDI, 2003).

Today's foundation engineers solve the following problems with the wave equation analysis.

- As with energy formulas, for bearing capacity assessment based on the observed blow count of a driven pile, a bearing graph is generated.
- For a required capacity, the bearing graph yields the associated necessary blow count and pile stress extrema.
- During the design process a check is made on the feasibility of installing a particular pile type and size to the design depth. In this analysis an assumption is made on what driving equipment would be available. The results would indicate the anticipated blow count and stress maxima as a function of pile penetration. This analysis is called driveability check and requires an accurate geotechnical analysis.
- For construction control and a particular required capacity, GRLWEAP provides a so-called Inspector's Chart. It gives for a range of hammer strokes or energy levels a minimum required blow count and the associated stress level.

Following the recommendations of the FHWA, (Hannigan et al., 1996), pile driving projects often involve a pre-construction test program which requires Pile Driving Analyzer® measurements during pile installation and restrike testing and then a CAPWAP analysis. During the pre-construction tests, the original wave equation assumptions are verified, the pile stresses and the pile bearing capacity are computed. Finally, based on the test results a pile installation criterion is formulated, typically in terms of a minimum installation depth and a blow count.

Occasionally, a static load test is also performed during the pre-construction test, particularly where little experience exists with the reliability of dynamic testing or with the dynamic behavior of the soil.

Based on the CAPWAP analysis, a so-called refined wave equation analysis may be done which relies for its input on measured and calculated rather than assumed dynamic soil resistance and hammer performance parameters. In that case improved accuracy of the bearing graph or inspector's chart is the objective of the testing and analysis effort.

4. EXAMPLES OF INFREQUENTLY USED OPTIONS

It has been mentioned that the basic Smith model requires a limited pile segment length for accuracy and a check on the sensitivity to that parameter is therefore demonstrated. Another interesting pile model feature concerns splices and slacks. Also interesting and infrequently checked is the residual stress analysis which is more rational than the standard blow count calculation method originally proposed by Smith. Particularly for long or flexible piles this option makes sense and its effects on the results will be demonstrated. Finally, for several years already the vibratory hammer analysis exists in GRLWEAP, but it is rarely executed because of difficulties with correctly assessing the soil resistance during the driving

process. However, an interesting problem is that of hammer/pile resonance and it appears that this is an easily examined phenomenon.

4.1 Pile Model – Number of Segments

Smith performed manual calculations, with relatively long segments (e.g. 2.4 m) to reduce calculation effort. The initial WEAP model was based on 1.5 m segments and today segment lengths are normally 1 m. Shorter pile segments cost more computer time (today generally not a problem) and generate more accurate results when the frequency content of the impact event is high like for uncushioned hammers. As an example, two analyses were made for an uncushioned IHC hammer, one with a 2 m segment length, the other one with the standard 1 m default value. Comparing the proportionality of the calculated pile top force and velocity at impact is a good check on the accuracy of the pile model. Figures 2 a and b show these calculated curves and indicate that the non-standard 2 m segment length produced a poor proportionality while the standard 1 m segment gave an improved proportionality impact record.

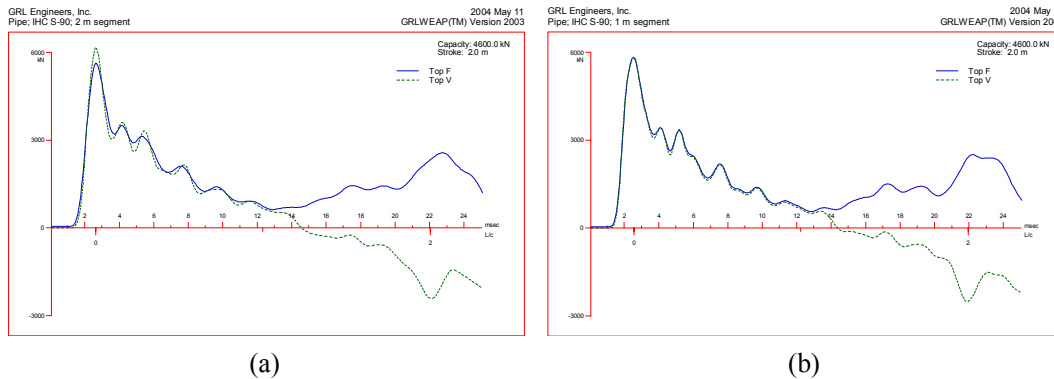


Figure 2. Proportionality Check: (a) 2 m (b) 1 m Pile Segment

Figure 3 shows the influence of segment length on pile top stress of a large and long pipe driven by an uncushioned hydraulic hammer. Differences are relative to the smallest analyzed segment length of 0.25 m. The figure also shows that a segment length greater than 1 m would lead to excessive errors of blow count. However, the values were chosen from an analysis with a relatively high resistance leading to blow counts in the 400 blows/m range;

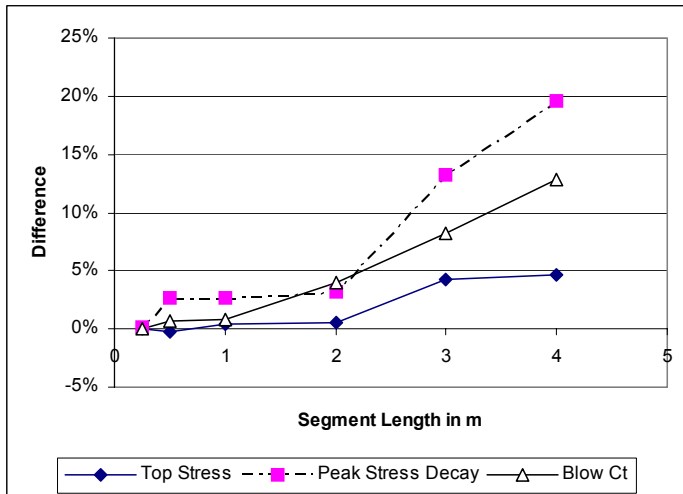


Figure 3. Influence of Segment Length on Stress at Pile Top, Stress Decay and Blow Count

for lower blow counts (e.g., in the 60 blows/m range) the effect of segment length would be negligible, because peak stress would then not be as important for an effective pile driving. Finally the figure shows how the stress peaks vary along the 100 m long pile whose resistance is concentrated at its bottom. This stress wave decay is defined as the difference between top and mid-length stress, relative to the top stress. Obviously, 2 m or fewer segment lengths produce perfectly satisfactory results.

4.2 Pile Model - Splices

Mechanical splices in segmental precast concrete piles are often subjected to high driving stresses and measurements may reveal a reflection from the splice which may be caused either by a natural flexibility of the splice or by a crack near or damage of the splice. Such a situation can be modeled with GRLWEAP and the seriousness of the reflection investigated. As an example consider the results obtained when a 20 m concrete pile composed of 2 sections of 10 m length each is spliced. Let us assume that a 5 ton Banut hammer drives this pile. The splice input consists of a compressive slack and a tensile slack plus a coefficient of restitution for the splice. The compressive slack is a distance over which the pile stiffness increases linearly to the full nominal value. The tension slack is a distance that the splice can expand without transferring any tension. It is the compressive slack that produces a reflection when the compressive stress wave passes the splice. Figure 4a shows a calculated force and velocity record together for a 1 mm compressive slack. Figure 4b shows the equivalent calculated record for a 3 mm compressive slack. Probably a 3 mm slack would be close to an actual damage and it shows a much greater reflection while the 1 mm slack is more likely to represent the undamaged slack. To correctly model the splice it is recommended to take measurements during the early driving when the splice is definitely undamaged. Then modeling and matching the force-velocity record, in particular the splice reflection, will yield more accurate splice parameters for the simulation of further pile driving.

For this example the two bearing graphs have also been plotted in Figure 5 using the GRLWEAP output option. The graph shows very clearly that the larger slack reduces the tension stresses in the concrete pile. It actually filters the tension stresses. On the other hand, the relationship between blow count and capacity is little affected.

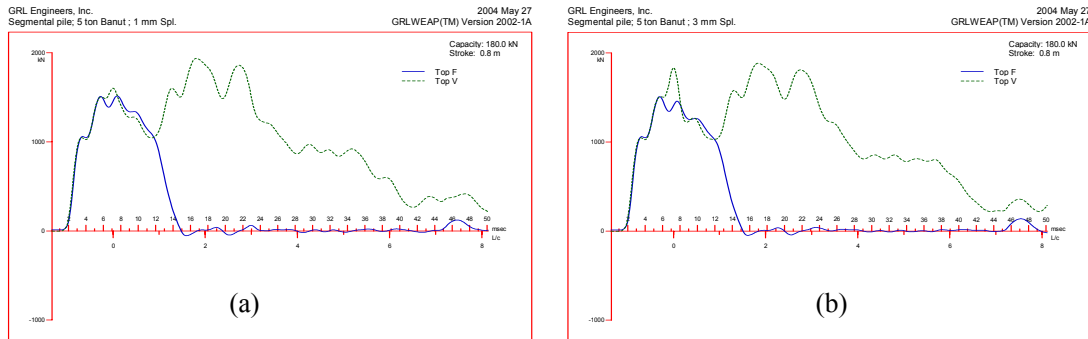


Figure 4. Calculated Splice Effect: (a) Small Slack (b) 3 mm Slack

4.3 Residual Stresses

It has been known for quite some time that the consideration of residual stresses in a wave equation analysis can have significant effects on calculated stresses and blow counts (Goble et al. 1983). It is also known that the flexibility of the pile and differences between toe and shaft quakes cause residual stresses. This is because the pile's shaft resistance tends to restrict the pile's rebound at the end of the impact event. When the next blow occurs, pile and soil are prestressed and therefore less energy is needed to compress and move the pile through the soil.

- GRL Engineers, Inc.
- 2-Seg. pile; 3 mm C-splice; 5 ton Banut
- 2-Seg. pile; 1 mm C-splice; 5 ton Banut

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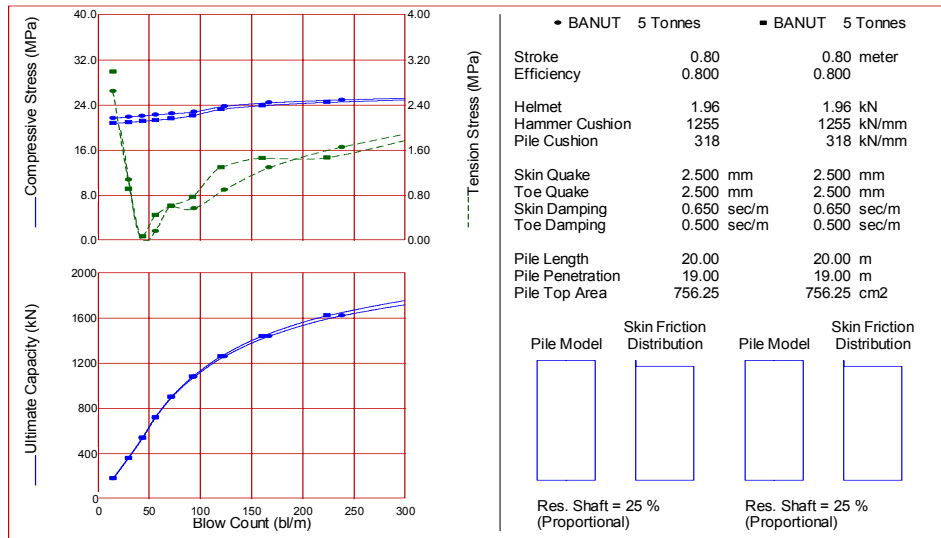


Figure 5. Effect of Increased Splice Slack on Blow Count and Stresses

Starting with the 2002 version, the residual stress analysis in GRLWEAP has been greatly improved, particularly as far as the convergence for long piles is concerned. In fact, the program now not only checks consecutive “hammer blows” but even groups of successive blows for convergence of pile sets. This is important when piles with very deep penetrations are analyzed. Of course, the new algorithm requires more iterations and therefore longer computation times. Fortunately, the cost of these computation times has become immaterial.

Table 1 summarizes three different calculation examples. In Case 1 (Figure 6), the intent was to show the influence of the penetration on the results obtained with and without Residual Stress Analysis (RSA) on a typical offshore open ended steel pipe pile. Obviously, the greater the penetration the greater the potential for energy stored in the pile. However, the shaft resistance percentage was left the same 90% for all investigated penetrations.

Case No.	Pile Type*, Size	Pile Length	Hammer Type*	Penetration	Quakes Shaft/Toe	Damping Shaft/Toe
	mmxmm	m		m	mm / mm	s/m / s/m
1	OE Pipe 1220x25	100	OE Diesel Wr 98 kN Er 360 kJ eff. 0.80	10 25 50 75 100	2.5/2.5	0.65/0.50
2	OE Pipe 1220x25	100	ECH Wr133 kN Er 203 kJ eff. 0.67	30	2.5/2.5	0.16/0.50
3	Square Conc. 610x610	30	ECH Wr 74 kN Er 149 kJ eff. 0.95	5 to 100	2.5/10	0.16/0.50

Table 1: Investigated residual stress cases

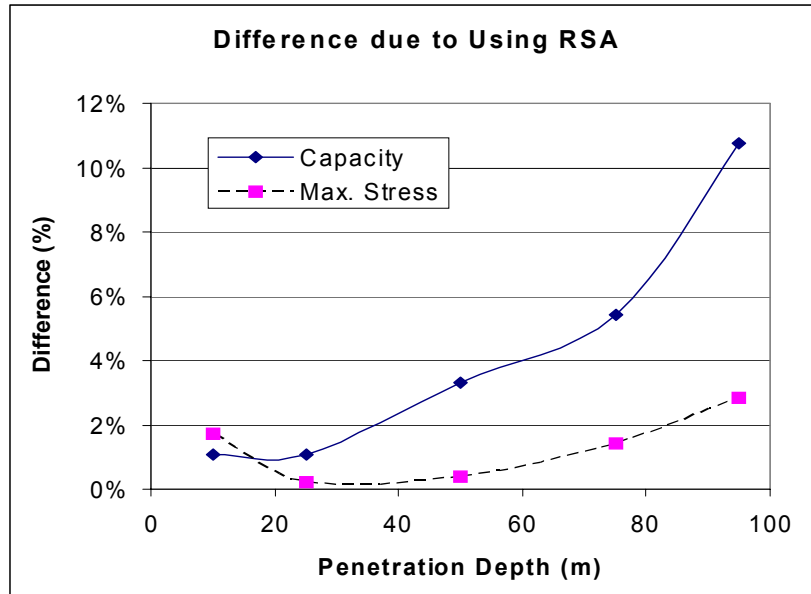


Figure 6. Effect of RSA on Blow Count and Maximum Pile Stress of a Long Steel Pile

* OE ... Open End; Wr ... Ram Weight; Er ... Rated Energy; eff. ... efficiency

The driveability analysis of Case 2 (Figure 7), also very clearly demonstrates the influence of penetration on the blow count results. In fact, it appears that, at least in part, RSA explains why often blow counts remain relatively constant as piles are driven to great penetrations. While this is primarily attributable to the loss of shaft resistance due to the dynamic effects, the shaft resistance does increase with depth. That

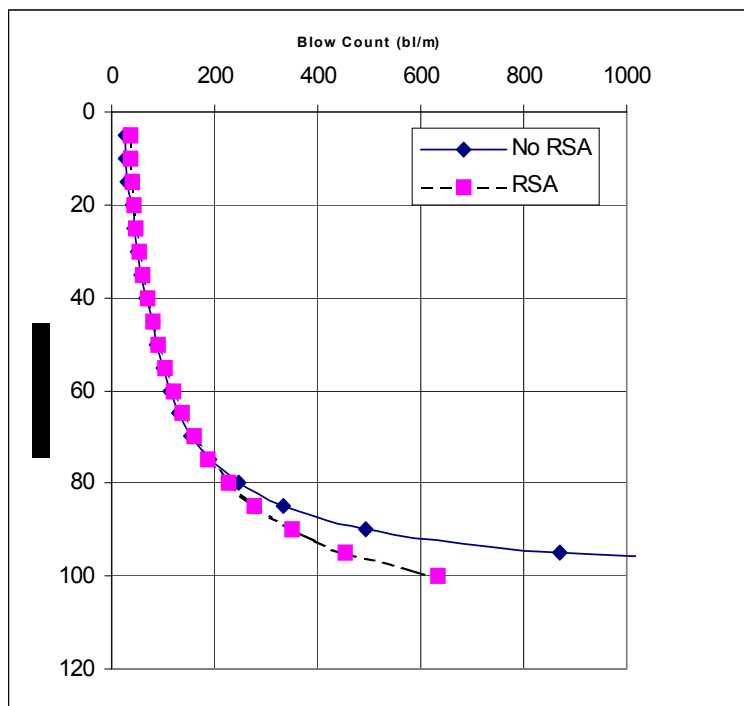


Figure 7. RSA Effect on Blow Count: Driveability Analysis of a Long Pile

the blow count is hardly affected is thanks to the help by the residual stresses.

The third case is a concrete pile. It is much stiffer relative to the soil resistance than a steel pile and it is therefore not surprising that the calculated bearing capacity shows little sensitivity to changes in

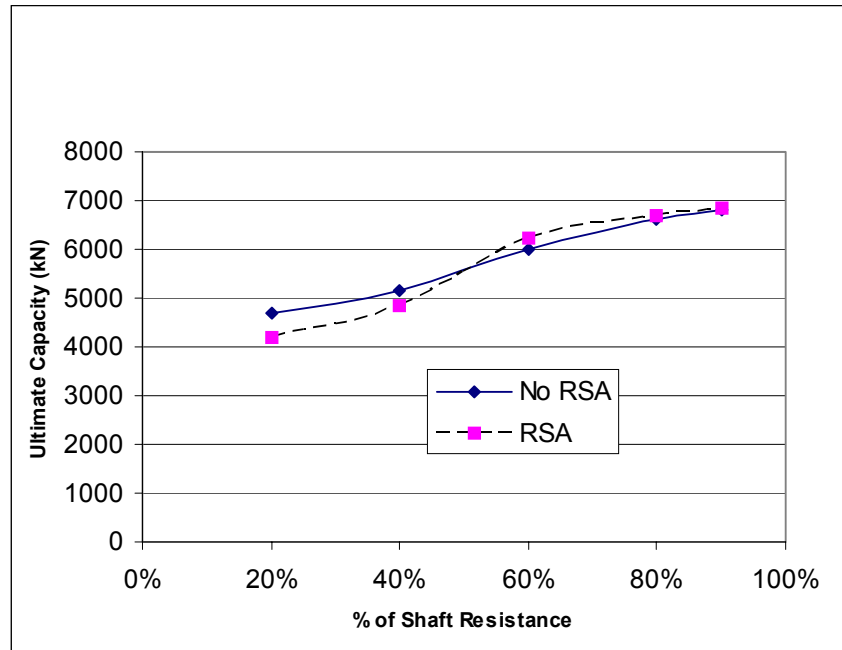


Figure 8: RSA effects on a stiff concrete pile

penetration (Figure 8).

4.4 Vibratory Hammer Analysis

Although still not as well documented as the analysis for impact hammers, the mechanics of vibratory pile driving is quite well adaptable to wave equation analysis (Rausche, 2002). Still much work needs to be done as far as the vibratory modeling of the soil is concerned. However, an example is used here to show the vibratory hammer form of the Inspector's Chart Option of GRLWEAP. The example demonstrated here is a 914x25 mm open ended pipe pile of 52 m length. The pipe is to be driven as far as possible with an APE 400B hammer which has an eccentric moment of 1.675 kN-m. It is assumed that the pile penetrates roughly 12 m below seabed and that it then has an ultimate soil resistance of 1350 kN.

While for impact hammers the Inspector's Chart relates hammer stroke to blow count and pile stresses, for vibratory hammers the stroke is replaced by hammer frequency. This allows for studying potential effects of hammer-pile resonance. Figure 9 shows the current example, that the penetration rate in mm/s reaches a relative maximum at approximately 16 Hz and then again at the maximum hammer frequency of 23.3 Hz (1400 RPM). Because of the centrifugal force increase with the square of the hammer frequency, at 23.3 Hz the theoretical pile top force is 45% greater than at 16 Hz and it is therefore surprising that there is so little difference in pile penetration rate. Interesting is also that the relative stress, defined as the sum of pile tension and compression stress divided by the pile stress corresponding to the hammer centrifugal force has a minimum at 17.5 Hz and a maximum at 20 Hz. The lowest rate of penetration of approximately 3 mm/s occurs at about 18 Hz where the relative pile stress starts to increase. Refusal is generally defined as a penetration rate between 5 and 10 mm/s.

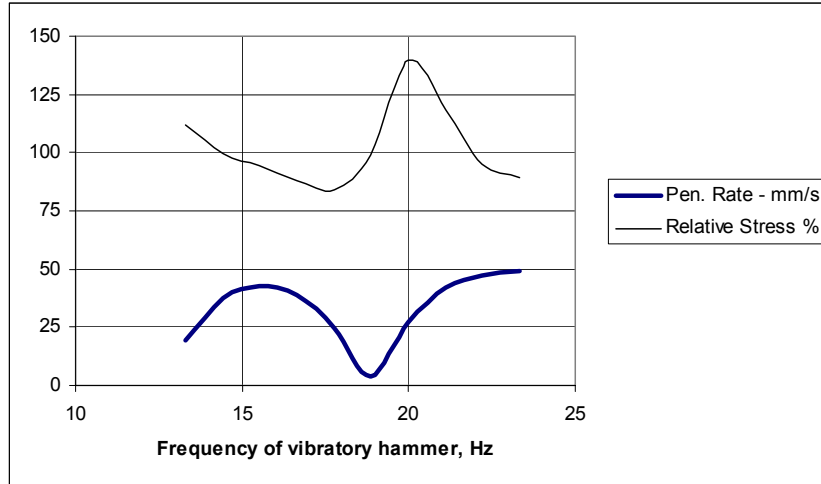


Figure 9. GRLWEAP Calculated Resonance Effects for Vibratory Pile Drive

5. PROGRAM PERFORMANCE

Throughout its development, i.e. since 1976, the GRLWEAP program, its hammer models, hammer data and driving system files have undergone various updates. Correlations were reported in the original research documents (Goble & Rausche 1976) and in later GRLWEAP Background reports. The updates have been necessary because of changes in hammer technology, added knowledge about dynamic pile and soil performance, changes in computer technology (which allowed for greater computational efforts and therefore refined modeling) and other reasons. Because of these software and data file changes a renewed correlation effort was made. The correlations were done with recently collected field data of standard pile driving projects that reflect day-to-day construction site practice.

The most important wave equation results are the pile top stress and the bearing capacity. The energy transferred to the pile is also of interest because it is useful for hammer performance evaluations. Pile top stress, averaged over the pile cross section, is always calculated and displayed by a Pile Driving Analyzer during dynamic pile testing. Bearing capacity correlations ideally would involve comparisons with a static load test. However, since the selected cases were not from special test programs with static load tests, capacities calculated by CAPWAP or CAPWAP correlated Case Method were compared with GRLWEAP results.

For this study a total of 39 cases were investigated of which 21 were steel (Table 2) and 18 square, prestressed concrete (Table 3). It was attempted to represent a variety of hammer types and models and not only different site conditions but also, on each site, several piles. This was possible for the steel piles; unfortunately, for the concrete piles recently tested sites did not include as much a variety of driving systems and pile types as for steel piles.

Table 2: Investigated steel pile cases

Site ID	No. of Piles	Pile Type Size mm	Pile Length m	Soil Shaft	Soil Rock	Hammer Type	Hammer Model
1	3	HP305x79	16.75	SI	Wthd Rock	A/S	CON65E5
2	4	HP305x110	9.15	SA, si	Breccia	A/S	CON100E5
3	1	CE Pipe356 NU	16.75	SA, gr	SA, gr	A/S	Vulcan06
4	4	HP356x132	18.25	SA, si, cl	Wthd Shale	A/S	Vulcan010
5	5	CE Pipe324	27.5	SA, si, cl	Wthd Shale	A/S	Vulcan010
6	2	HP356x132	18.25	SA, si, cl	Wthd Shale	HYDR.	ICE 115
7	4	HP356x132	18.25	SA, si, cl	Wthd Shale	HYDR.	ICE 115
8	3	HP254x63	27.5	SA, si, cl	CL	HYDR.	HER-HMC62
9	2	HP356x174	40	soft soils	Limestone	HYDR.	DAW-HPH6500
10	6	CE Pipe 245	29	SA	Wthd Rock	HYDR.	ICE 160
11	1	HP305x79	6.2	CL	Wthd Shale	CED	ICE 520
12	1	HP254x63	9.25	GR, sa, cl	Limestone	CED	ICE 520
13	2	HP254x85	40	SA	Wthd Rock	OED	DEL D12-32
14	4	HP305x79	15.25	SA, cl, si	SA, si	OED	DEL D30-32
15	4	HP305x110	12.2	SL	Limestone	OED	ICE I30
16	4	CE356 PipeNU	18.25	SA, si, gr	SI,sa	OED	ICE 42-S
17	4	HP356x174	29	SA, cl	Limestone	OED	DEL D46-32
18	3	HP356x152	35.4	CL, si	CL, si	OED	ICE 20S
19	2	HP356x152	15.25	SI, gr	Siltstone	OED	BERMB4505
20	9	HP254x94	17.4	CL, sa	Shale	OED	DEL D16-32
21	9	HP254x94	17.4	CL, sa	Shale	OED	DEL D16-32

Notes: NU - Non-Uniform (Monotube(R)); CE: Closed Ended Pipe - diameter; A/S Air/Steam; HYDR. Hydraulic; CED Closed Ended Diesel; OED: Open Ended Diesel
 CL Clay; SI silt; SA Sand; GR Gravel; SL Slag;
 Capital letters: dominant grain size

Acceptable data sets had to provide soil information, blow counts and, most importantly, end-of-driving PDA results. Restrike information was not considered, since the GRLWEAP correlations primarily concerned the ability of the program to correctly predict stresses, transferred energies and capacities as they would occur during pile driving.

Table 3: Investigated concrete pile cases

Site ID	No. of Piles	Pile Type Size mm	Average Pile Length m	Soil Shaft	Soil Toe	Hammer Type	Hammer Model	Pile Cushion Thickness mm
1	6	PPC-406-sq	16.5	SA, si	SA, si	ECH-Hydr	HMC-62	150
2	3	PPC-355-sq	21.3	SA, si	SA, si	ECH-SA-A/S	Con100E5	175/220
3	2	PPC-406-sq	12.0	SA	SA, CL	OED	DEL D30-32	150
4	3	PPC-460-sq/St.	21.5	SA, SI	Wthd Rock	OED	ICE 42S	150/300
5	1	PPC-455-sq	18.0	SA	SA	OED	ICE 80S	155
6	1	PPC-455-sq	30.5	SA	sa CL	OED	DEL D36-32	228
7	1	PPC-455-sq	25.0	cl SA	SA, si	OED	ICE 120S	203
8	1	PPC-455-sq	22.8	si SA	SA	OED	MKT 70/50	203
9	1	PPC-455-sq	21.0	si SA	Limestone	OED	Berm 450 5	254
10	1	PPC-455-sq	30.0	cl, si SA	CL	OED	ICE 100S	203
11	1	PPC-455-sq	15.8	SA	SA	OED	DEL D-30	203
12	1	PPC-455-sq	35.0	cl, si SA	sa CL	OED	ICE 100S	203
13	1	PPC-610-sq	38.1	cl, si SA	SA	OED	DEL D62	305
14	1	PPC-405-sq	15.2	SA	SA	OED	ICE 60S	152
15	1	PPC-405-sq	18.3	SA, SI	Wthd Limest.	OED	DEL D46-32	152
16	1	PPC-455-sq	21.0	si, sa Limest	Limestone	OED	DEL D36-32	229
17	1	PPC-610-sq	22.9	SA	si SA	OED	ICE 120S	190
18	3	PPC-610-sq	22.9	CL	sa Si	OED	ICE 120S	190

Notes: all piles square, prestressed concrete; St. – pile with steel stinger

The procedure was as follows: after selecting the data from a particular site, several representative test piles were selected spanning a range of blow counts, stresses and capacities. Next, the soil resistance distribution was estimated and then a bearing graph was calculated by GRLWEAP. For these analyses, driving system parameters, hammer efficiencies and all dynamic resistance parameters were chosen as per standard GRLWEAP recommendations. In other words, the correlations would indicate the variability and uncertainty with predictions made prior to pile driving which is the most important application for the wave equation approach. Using the blow counts from end-of-driving, the associated stresses, transferred energies and capacities were then found in the GRLWEAP output. For diesel hammers on steel piles, the stroke was also included in the comparison. Note, that the pile top stresses, not the maximum stresses were selected from the numerical output of GRLWEAP to correlate with the pile top stresses from PDA measurements. Capacities were compared with those from Case Method and CAPWAP where available.

Finally, the ratios of measured to GRLWEAP computed quantities were calculated and statistically summarized in Table 4, showing average as an indication of bias and coefficient of variation. Plots of these ratios are shown for stress and transferred energy in Figures 10 through 13. Note that these results not only provide a view of the GRLWEAP correlation but also on the variability of piles within one site since, for purposes of GRLWEAP prediction, the piles on each site were considered identical.

Table 4: Statistical Summary of GRLWEAP/PDA Measured Ratios

Pile Material Pile Cushion	Quantity	No. of Piles	Average	Coeff. Of Variation
Steel	Pile Top Stress	78	1.03	0.15
	Transfd. Energy	78	0.99	0.21
	Stroke (diesels)	37	0.97	0.06
	Case Capacity	78	0.92	0.14
	CAPWAP Capacity	42	0.91	0.12
Concrete New Cushion	Pile Top Stress	30	0.86	0.33
	Transfd. Energy	30	0.91	0.28
	Case Capacity	30	1.14	0.36
	CAPWAP Capacity	20	1.00	0.36
Concrete Used Cushion	Pile Top Stress	30	1.05	0.26
	Transfd. Energy	30	1.13	0.25
	Case Capacity	30	1.29	0.44
	CAPWAP Capacity	20	0.87	0.31

The spread of energy and stress values within the same site indicates the variability of the hammer and driving system. Furthermore, the much greater scatter of results for concrete than steel piles is clear evidence that the pile cushion introduces great uncertainty in the results. In fact, while for steel piles GRLWEAP slightly overpredicts the stresses and transferred energies, it underpredicts by 14 and 9% the concrete pile stresses and transferred energies. The fact that all cases were selected from the end of driving, when the pile cushions are normally well compressed, explains the underprediction. For that reason, a second set of GRLWEAP analyses was conducted in which a “Used” pile cushion was modeled, i.e. with 2/3 of the nominal thickness and roughly twice the elastic modulus for plywood (300 MPa instead of 211 MPa). Statistical results were again entered in Table 4 and energy and stress ratios were plotted in Figures 14 and 15. Obviously, these results are much more encouraging. Note that for Case 7 the field notes on the driving log indicated a cushion exchange shortly before the end of driving and also a problem with hammer performance.

Finally, blow counts from a few driveability predictions were compared with measurements. Numerically comparing blow counts is an exercise in futility, because slight shifts in soil strata would indicate rather high variations of the calculated blow count at the same depth. An example is given in Figure 16. Note that the GRLWEAP predictions are basically correct, however, the 5 piles for which these predictions were made all reached the high blow counts at different depths and some of these piles encountered harder layers that the others did not “feel”.

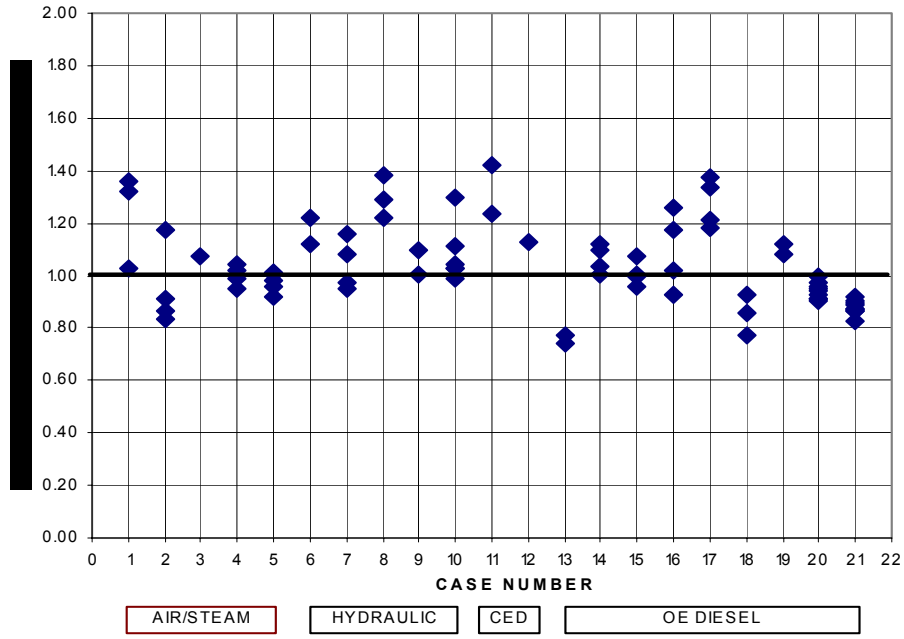


Figure 10. Calculated / Measured Pile Top Stress – Steel Piles

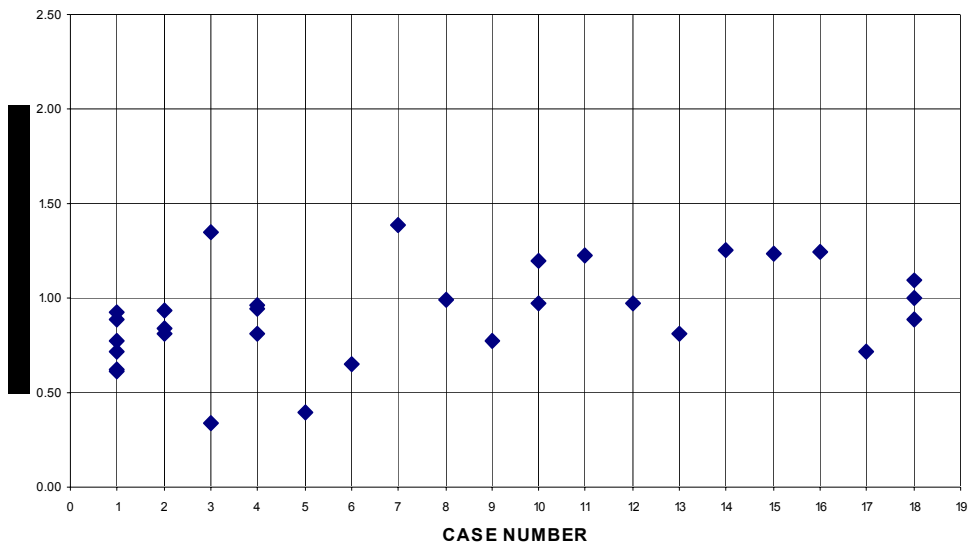


Figure 11. Calculated / Measured Transferred Energy – Steel Piles

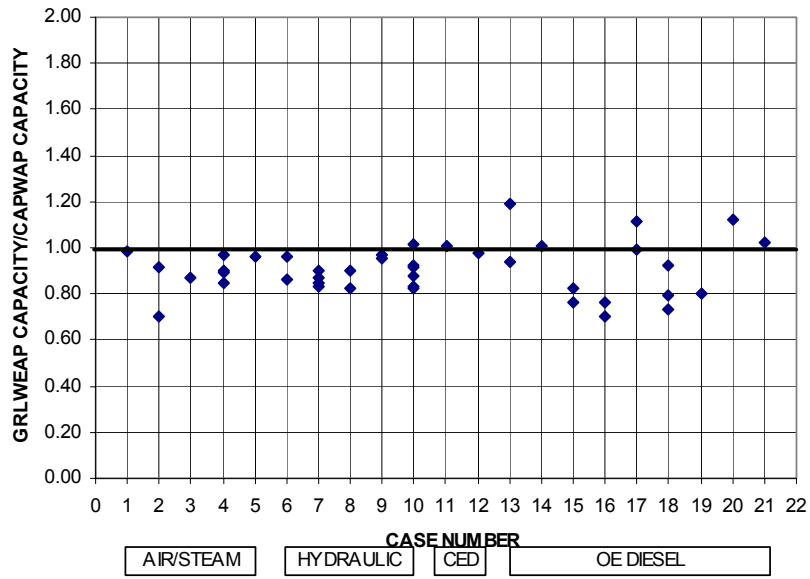


Figure 12. GRLWEAP / CAPWAP Capacity – Steel Piles

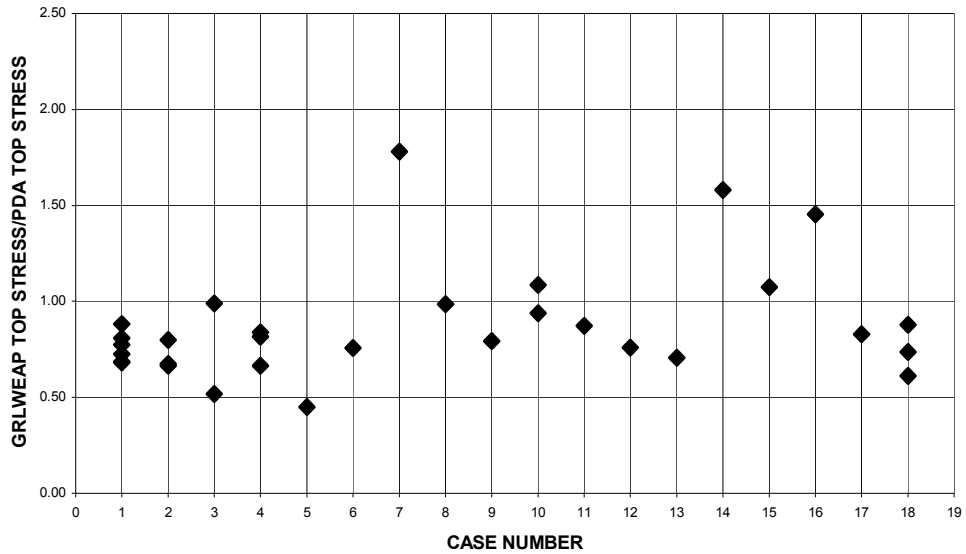


Figure 13. Calculated / Measured Pile Top Stress – New Cushion

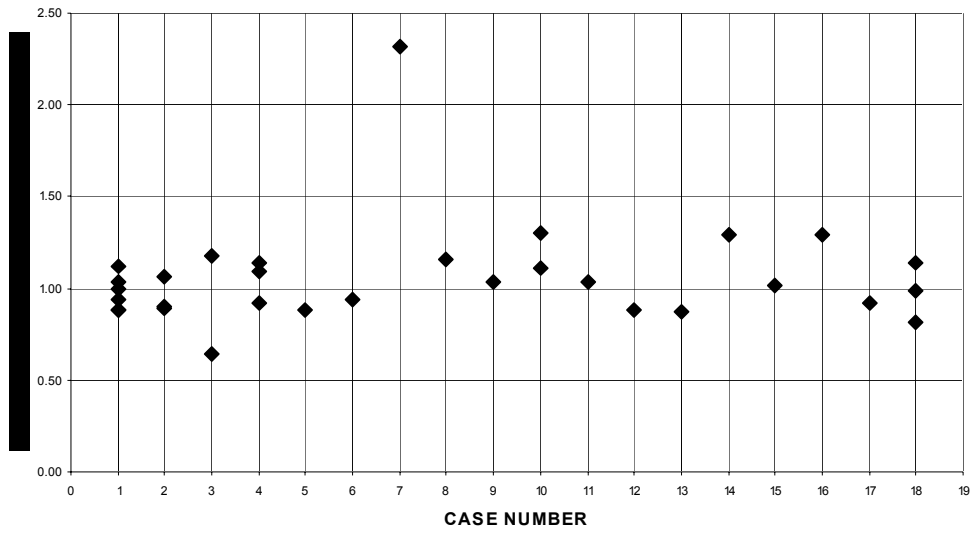


Figure 14. Calculated / Measured Pile Top Stresses – Used Cushion

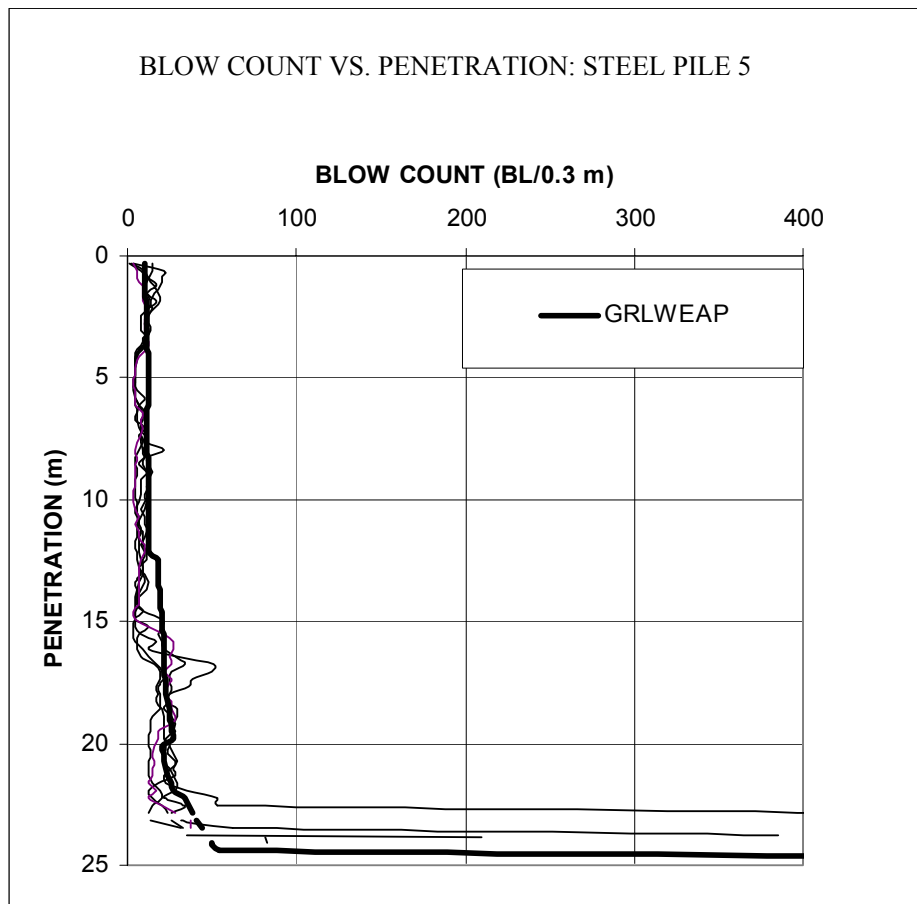


Figure 15. GRLWEAP Predicted and Actual Driving Records

6. SUMMARY AND CONCLUSIONS

The GRLWEAP software offers a variety of model and analysis options. The basic Smith algorithm is sound and the default segment lengths assure good accuracy even for systems with high frequency components (uncushioned hammers).

The software is also flexible enough to realistically model non-linear interfaces and material behavior.

The residual stress analysis option eventually will replace the traditional zero-initial stress analysis. In particular for long steel piles this method appears to explain to a limited degree the constant driving resistance phenomenon for deep penetrations. Considering the fact that the improved RSA algorithm is robust and the computer time cost negligible, there is no more reason to perform the less accurate analysis.

Steel pile predicted stresses and transferred energies compare well with measured values and therefore it can be concluded that the basic mechanical model of the GRLWEAP program is sound. Uncertainties are introduced by the actual performance of hammer and driving system components.

The pile top stress and transferred energy values compared well for the steel piles with less than 4% overprediction. For concrete piles, if the "New Cushion" properties are used for end-of-driving in the driving system model, the pile top stress underprediction in the cases analyzed averages 14%. By using a 2/3 cushion thickness and a doubled elastic modulus to reflect the compressed pile cushion properties, the stresses were in much closer agreement with measurements (overprediction of only 5%). A slight overprediction is conservative and therefore preferable to underprediction.

While the study suggests to input "Used" cushion properties for end-of-driving analyses, the prediction of tension stresses in early, easy driving still should be done with the standard recommendations (nominal thickness and, for plywood, $E = 210$ MPa).

In general, wave equation predictions of capacity compared well for steel piles with dynamic test results. Comparison with actual static load tests would introduce additional scatter. For concrete piles the uncertainty of pile stresses and transferred energies adds an additional element of uncertainty to the capacity predictions.

The correlation study supports the fact that there is little improvement that can be made to the mechanical model of hammer, driving system and pile. Improvements in prediction can only be achieved by performing measurements on production piles to assess actual field conditions.

This paper did not assess the realism of the Smith soil model. Additional work in that area of study is warranted.

REFERENCES

Goble, G.G. and Rausche, F., (1976), "Wave Equation Analysis of Pile Driving-WEAP Program." *Volumes 1 through 4*, FHWA #IP-76-14.1 through #IP-76-14.4.

Goble, G., Rausche, F., and Hery, P., (1983), "Colorado University Modified Wave Equation Analysis of Pile Driving – CUWEAP Program." *Volume 2, Department of Civil, Environmental, and Architectural Engineering*, University of Colorado.

Hannigan, P.J., Goble, G.G., Thendean, G., Likins, G.E., and Rausche, F., (1996), "Design and Construction of Driven Pile Foundations." *Vol. I and II, FHWA HI-96-033*, Washington, DC.

Hirsch, T.J., Carr, L., and Lowery, L.L. (1976), "Pile Driving Analysis – Wave Equation", *Vol 1-4, FHWA HIP-76-13.1 through IP-76-13.4*.

Lowery, L.L., Hirsch, T.J.Jr., and Samson, C.H., (1967), "Pile Driving Analysis - Simulation of Hammers, Cushions, Piles and Soils." *Texas Transportation Institute*, Research Report 33-9.

Pile Dynamics, Inc., (2003), "GRLWEAP Procedures and Models version 2003," 4535 Renaissance Parkway, Cleveland, Ohio 44128, www.pile.com.

Pile Dynamics, Inc., (2000), "CAPWAP for Windows Manual version 2000-1," 4535 Renaissance Parkway, Cleveland, Ohio 44128, www.pile.com.

Randolph, M.F., (1992), "IMPACT – Dynamic Analysis of Pile Driving." *Program Manual*, Department of Civil and Environmental Engineering, The University of Western Australia.

Rausche, F., (2002), "Modeling of Vibrator Driving", "*Proceedings of the International Conference on Vibratory pile Driving and Deep Soil Compaction*", Louvain-La Neuve, Belgium.

Smith, E.A.L., (1960), "Pile Driving Analysis by the Wave Equation." *Journal of the Soil Mechanics and Foundations Division*, ASCE, Volume 86.