Static and Dynamic Models for CAPWAP Signal Matching

Frank Rausche[1], P.E., M.ASCE, Garland Likins[2], P.E., M.ASCE, Liqun Liang[3], P.E., M.ASCE, and Mohamad Hussein[4], P.E., M.ASCE

[1] President, GRL Engineers, Inc., 4535 Renaissance Parkway, Cleveland, OH; frausche@pile.com
[2] President, Pile Dynamics, Inc., 4535 Renaissance Parkway, Cleveland, OH, garland@pile.com
[3] Senior Engineer, Pile Dynamics, Inc., 4535 Renaissance Parkway, Cleveland, OH, Liqun@pile.com
[4] Vice President, GRL Engineers, Inc., 8000 South Orange Ave., Orlando, FL, MHGRL@pile.com

ABSTRACT

Signal matching is the preferred analysis method for Dynamic Load Test (DLT) evaluations. It is applicable to DLT records of driven piles, auger-cast piles, drilled shafts, and even on dynamic penetrometers. Although signal matching is considered standard best-practice, required by many code specifications and therefore routinely used on thousands of deep foundation projects worldwide, and of significant importance to the deep foundation industry, many features of the CAPWAP® signal matching model and procedure are not well known.

CAPWAP's signal matching is possible because of the availability of redundant measurements of load and movement, and it is necessary to determine the unknown boundary conditions. The goal of CAPWAP is the determination of dynamic and static soil resistance parameters of the generally accepted Smith-type pile-soil interface model. However, the classic Smith model cannot explain some of the phenomena that occur during the impact event. For reliable signal matching results, therefore, several modifications of the original Smith model were made. While some modifications fundamentally do not affect the signal match, other more substantial changes are of considerable importance to the reliable determination of the all-important static load bearing capacity result.

Before discussing the CAPWAP procedure and its automatic analysis tools, this paper describes the more unusual CAPWAP pile and soil model parameters and their effects on the final results. Measurement and analysis results from actual projects demonstrate the various features of the program and aspects of the models. The paper includes a summary of recommended limits for model parameters, match qualities, and calculation procedures and a few suggestions for additional research.

INTRODUCTION

Dynamic pile testing methods analyze dynamic force and motion (acceleration, velocity, displacement) recorded during a high-strain impact to determine load bearing capacity. The impact event stresses the pile in a non-uniform manner, e.g., in the first instance the forces are high near the top. At a slightly later time the forces and motions may be highest at pile mid-length and again a short time later the forces and/or motions increase near the pile bottom. Because of this "wave propagation", the bottom soil resistance forces are activated later than those acting higher along the shaft. These effects and the finite sharp rise of the records, typically one to three milliseconds in duration, make it possible to differentiate between end bearing and shaft resistance forces, and even the distribution of shaft resistance along the pile length. Using the basic solution to the one-dimensional wave equation, the simple closed form Case Method equations as well as the CAPWAP numerical solution were derived and correlated starting in the 1960s (Rausche et al., 1972, Goble et al., 1975, Goble et al., 1980, Rausche et al., 1985, Hussein and Camp, 1994, and Likins et al., 2004). These publications show that good correlations can be expected, if

(a) the waiting times after pile installation of the static and dynamic tests are comparable,
(b) sufficient energy is provided during the dynamic test to produce a permanent pile set of at least 2.5 mm, and
(c) the static load test is run to failure and evaluated by the Davisson criterion.

While dynamic measurements of driven piles during installation provide additional important information such as driving stresses, pile integrity and hammer performance, it is often the static bearing capacity which is of primary interest. Fortunately, CAPWAP signal matching provides much more than total bearing capacity; for example, it provides the necessary parameters for a t-z analysis yielding a simulated static pile-top load versus movement curve which can be directly compared with the load-set curve from a static load test. The calculated resistance distribution from end-of-drive and restrike tests can then help in optimizing a foundation design for the greatest economic benefit (Komurka, 2004). CAPWAP results are used for pile acceptance not only for smaller piles on land but also for large diameter pipe piles in the offshore environment even under hard driving conditions (Stevens, 2004).

Originally, CAPWAP used the basic Smith pile and soil models (Smith, 1960), i.e., the pile was represented by springs and masses, and for every mass the soil model included three unknowns: soil stiffness, static capacity and a damping factor. It was later determined that a more precise pile model and a more versatile soil model were needed to more realistically represent the dynamic phenomena. To correct the shortcomings in the pile model, a numerical procedure was adopted (Fischer, 1960), including tensile and compressive slack models. More important, as additional experience was gained, the soil model was expanded to include among other options: residual stress analysis, Smith-Case damping, radiation damping, soil plug and unloading stiffness.

THE BASIC CAPWAP MODEL

Pile Model Details

The original lumped-mass pile model of 1970 (Rausche, 1972), initially proposed by Smith (1960), has the advantage of great flexibility, allowing for easy modeling of splices and non-linear pile material behavior. For example, concrete with slight cracks can have different wave speeds in compression and in tension. For the modeling of individual cracks, it was possible to include non-linearities such as bending cracks open on one pile side and closed on the other. While the lumped-mass model is very flexible, the transmission of the suddenly changing wave is filtered to some degree, depending both on the length of pile segments and computational time intervals. This in turn causes a mismatch of the calculated with the measured response and since the determination of the soil model depends on the differences between the calculated and measured response, even small differences affect the signal matching process, particularly for uncushioned impact systems. For this reason, in the early 1980s a numerical model (Fischer, 1960) was adopted which uses continuous, uniform segments of length ΔL having an impedance $Z = m / \Delta t$ where m is the mass of the segment and Δt is the time that the stress wave travels through that segment ($\Delta t = \Delta L/c$ with c being the stress wave speed). The pile segment lengths are selected so that they have equal travel time Δt.

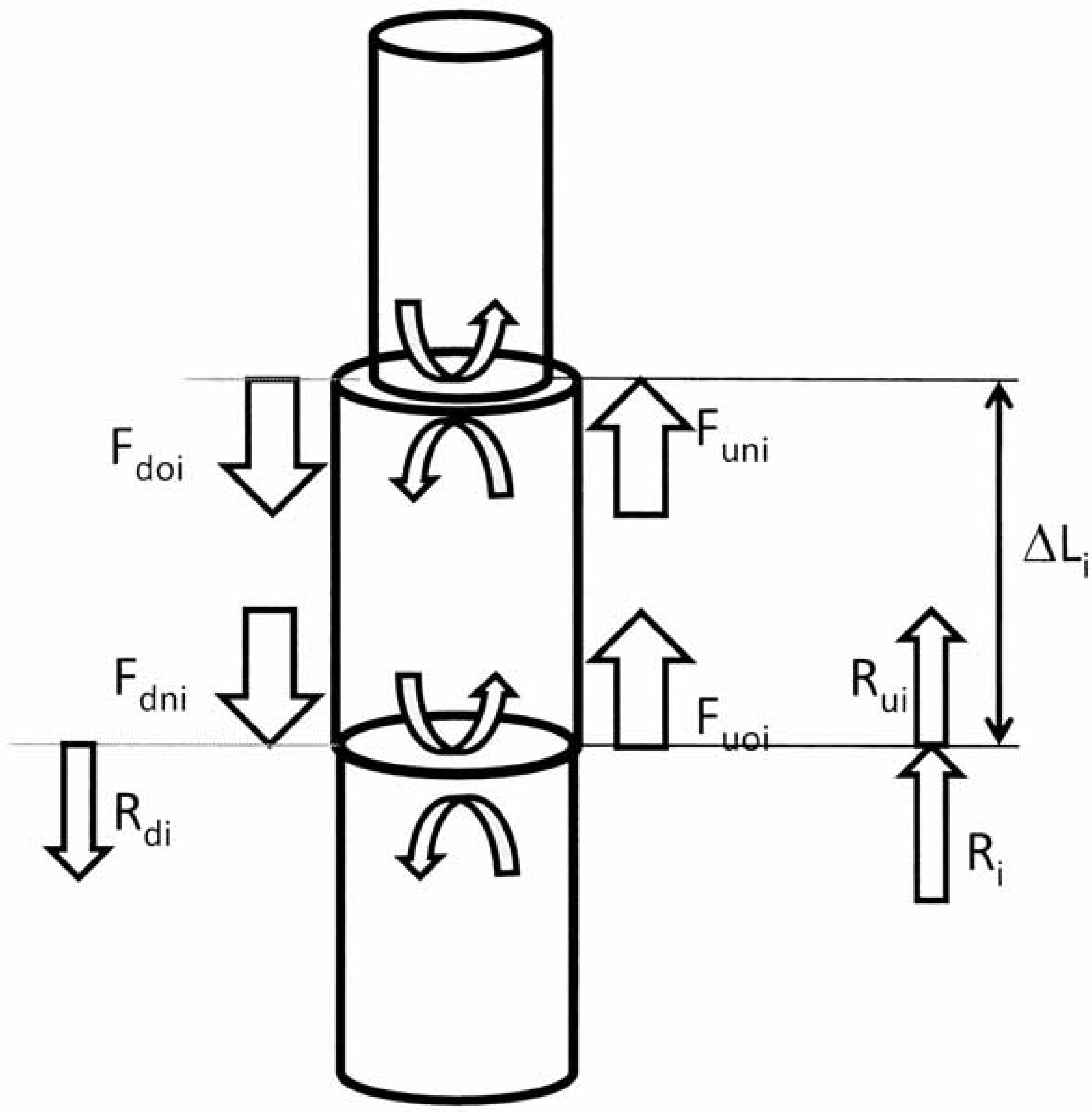

FIG. 1: The CAPWAP pile model

During each time increment in segment i, downward traveling stress waves F_{doi} and an upward traveling stress wave F_{uoi} are modified (Figure 1), becoming F_{dni} and F_{uni} due to reflections from soil resistance and possibly segment impedance changes when the pile is non-uniform. For example, a resistance force, R_i, generates both an upward traveling wave, R_{ui} and a downward wave R_{di}. These waves modify the incident waves by superposition.

A splice with slack or an existing crack presents a more complex condition. A slack is a distance which two neighboring pile sections can separate or compress with zero or only limited tension or compression force transmission. For example, a "Gravity Connector" or a "Can Splice" cannot transmit any tension and, therefore, has an unlimited tension slack. CAPWAP provides two different models for tension and compression slacks: (a) a distance-limited and (b) a force-limited slack.

The distance-limited slack has two parameters: distance and efficiency. For a slack efficiency of 1.0, full reflection happens, while for a slack efficiency zero, the wave is fully transmitted. For example, a stress wave is fully reflected on a distance-limited tension slack with slack efficiency of 1.0 until the two neighboring segments separate the full slack distance; after that full force transmission results. The fully efficient slack model is too simple for a bending crack extending only over a portion of the cross-section. In that case the slack efficiency should be less than 1.

The force-limited slack model is convenient for a concrete pile (reinforced or prestressed) with a crack. Here it is reasonable that a limited force can be transmitted. In the case of a prestressed pile, for example, full wave transmission should happen as long as the tension force is less than the effective prestress level. Higher tension forces open cracks which then reflect the excess force. For a compression slack, equivalent transmission/reflections occur when, for example, the crushing strength of the pile, a splice or a cushion material is exceeded.

An example (see also Rausche et al., 2006) is a concrete pile (420 mm diameter with a center void diameter of 200 mm; length 18 m) driven by a diesel hammer with a steel pipe follower (420x16 mm diameter wall thickness; length 15 m below sensors), separated by both a heavy steel plate under the follower and a softwood cushion (80 mm thick) between steel plate and concrete pile top. Figure 2, center, depicts this pile assembly with the "pile" top (sensor location on the follower) on the left and the tapered bottom of the concrete pile on the right. The pile-top force and velocity records measured with a Pile Driving Analyzer ® (PDA) are shown in the top of Figure 2 at a time scale matching the pile assembly sketch. Clearly a tensile stress wave reflection (compressive force reduction relative to the velocity) occurs at both the follower bottom and at the pile toe.

The "pile" model of the follower bottom/cushion contains a compressive and a tensile slack, both using the force limited model. The force limits were 1800 kN and 10 kN for compression and tension slacks, respectively (the 10 kN tension force limit is practically equivalent to an unlimited tension slack distance). Rather than modeling the cushion with thickness, elastic modulus and density, a 55% impedance reduction was made for the concrete pile top. This model resulted in a very good match (Figure 2, bottom) of the measured pile top reflection, albeit with a slight time shift.

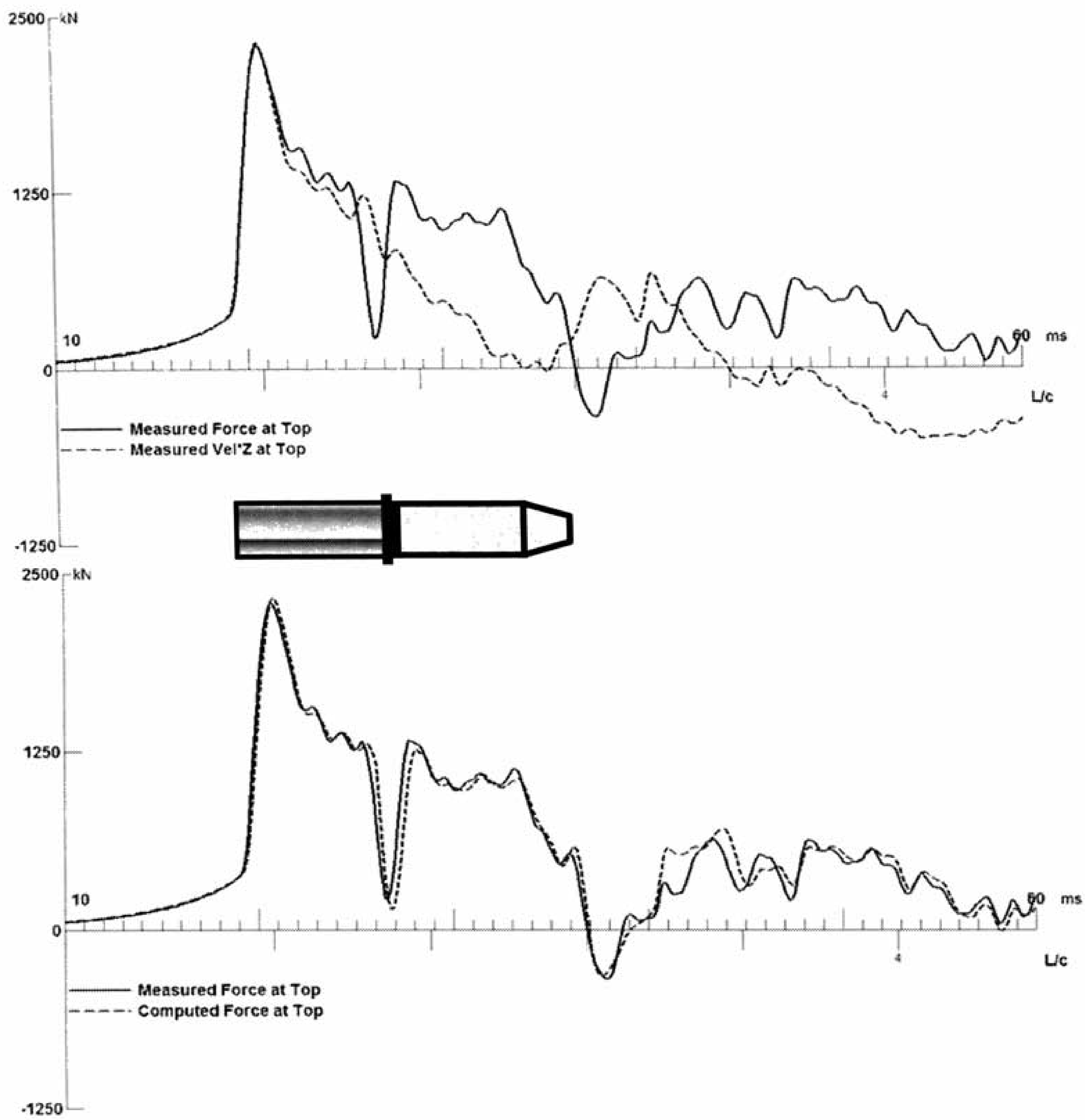

FIG. 2: Measurements, pile assembly sketch and CAPWAP signal match of a steel follower driven, tapered concrete pile.

The Static CAPWAP Soil Model

The basic premise of most soil models of relatively simple one-dimensional dynamic pile analyses is that the resistance acting at the pile-soil interface depends solely on the motion of the pile. Furthermore, it is assumed that the soil resistance, R(t), acting at some location along the pile under impact can be expressed as follows:

$$R(t) = f_{gl}R_s(t) + R_d(t) \tag{1}$$

Where f_{gl} is a gain/loss factor, R_s is the long-term static resistance that is displacement dependent, and R_d is the dynamic or velocity dependent component. The product of R_s and f_{gl} is often referred to as the "Static Resistance to Driving", SRD. For shaft resistance, f_{gl} usually is the inverse of the soil setup factor, f_s. The setup factor expresses the common gain of shaft resistance after initial driving ends and is almost always greater than unity. For end bearing, f_{gl} is the inverse of a relaxation factor. The relaxation factor, f_t, expresses a loss of end bearing after driving and is, therefore, less than unity. Thus, in general, the gain/loss factor reduces shaft resistance and increases end bearing during initial pile installation from the long term expected service conditions. Performing restrike tests and comparing those results with SRD occurring during pile installation helps determine the setup, relaxation and gain/loss factors.

CAPWAP's static soil resistance model, based on Smith (1960), is piecewise linear. Figure 3 demonstrates the basic static resistance force versus pile displacement relationship. As a pile segment moves downward, the resistance may remain zero for a "resistance gap" distance, g, increase linearly over a "quake" distance, q, up to the ultimate resistance level, R_u, and then remain constant until the maximum displacement is reached. Unloading then occurs with a slope defined by the "unloading quake", q_u, to a negative ultimate resistance level, R_{nu}. If a second downward motion occurs during the impact, after some upward slip in the negative region, this reloading first follows the unloading slope and above the reloading level, R_L, increases at the loading slope. Table 1 contains reasonable ranges for these quantities, based on the original Smith recommendations and additional experience. Note that the toe quake (plus the toe gap, if any) is limited by the maximum pile toe displacement u_{px}; otherwise all available resistance is not activated. While the original Smith model is simpler and adequate for a purely analytical approach, CAPWAP must produce a match with actual measurements and thus model extensions such as $q_u \le q$, $R_{un} \ge -R_u$ for the shaft, and $g \ge 0$ for the toe are necessary.

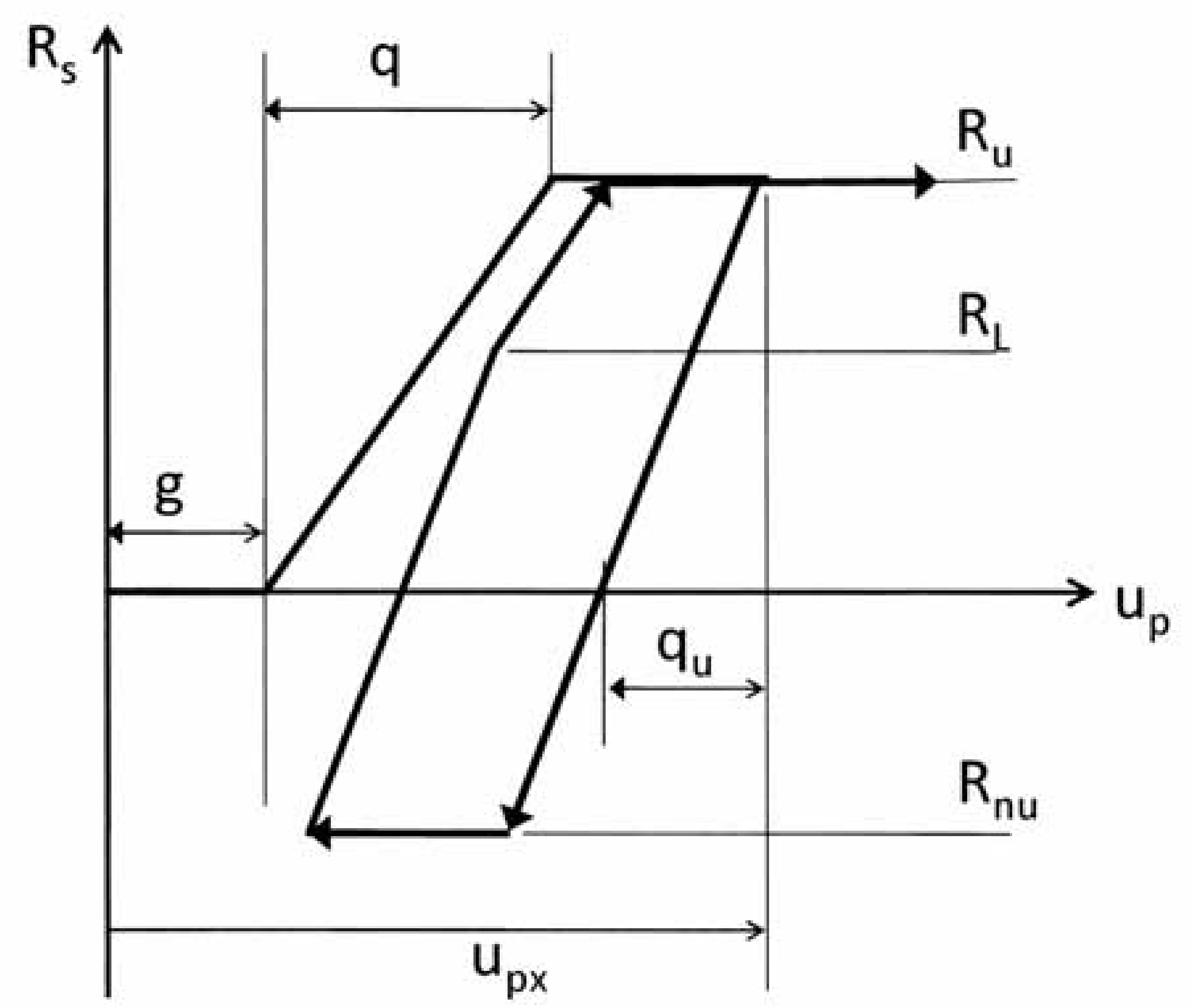

FIG. 3: The static soil resistance, R_s, vs pile displacement, relationship.

Already in the early 1970s CAPWAP signal matching efforts clearly showed that the toe quake is a function of diameter for displacement-type piles. Since Smith's toe quake assumption was a constant 2.5 mm (realistic for the small diameter piles considered by Smith), this finding had important consequences for the analysis of piles with greater bearing areas. Several studies (Authier and Fellenius, 1980, Likins, 1983, and Hannigan and Webster, 1987) led to the recommendation of a toe quake equal to diameter divided by 60 for wave equation analyses.

Table 1: Recommendations for limits of CAPWAP soil resistance parameters

Name	Symbol	Unit	Shaft, min	Shaft, max	Toe, min	Toe, max
Quake	q	mm	1	7.5	1	Max.displacemt-g
Unloading Quake	q_u/q	1	0.3	1.0	0.3	1.0
Neg. ult. Resistance	R_{nu}/R_u	1	0.0	1.0	0.0	0.0
Gap	g	mm	0.0	0.0	0.0	Max.displacemt-q
Reloading	R_L/R_u	1	-1.0	1.0	0.0	1.0
Damping Option		1	0	2	0	2
Damping Factor	j	s/m	0.04	1.4	0.04	1.4

The CAPWAP Soil Damping Model

Much has been published on the shortcomings of the Smith damping model (Gibson, 1968) and a variety of other approaches have been developed (for example, Chin and Seidel (2004) did a very thorough experimental study of damping in clays). However, the simplicity of Smith's approach is still attractive. He defined damping in the pile soil interface as:

$$R_d(t) = j_s\, R_s(t)\, v_p(t) \tag{2a}$$

where j_s is the Smith damping factor and $v_p(t)$ the pile velocity. This basic Smith approach is referred to as Option 1 in CAPWAP. However, since $R_s(t)$ is sometimes very small (e.g., during the unloading portion after the main impact) the Smith approach usually under-dampens the real measured pile response. To improve the match, CAPWAP frequently uses two additional damping options.

Option "0" replaces $R_s(t)$ in Eq. 2a with the constant R_u, producing a truly viscous damping.

$$R_d(t) = j_s\, R_u\, v_p(t) \tag{2b}$$

While Option 0 appears to be the best definition for the shaft, a combination of Eq. 2a and 2b often proves advantageous for the toe response. Option 2 (in this paper referred to as Smith-Case damping) calculates damping as for the Smith Option 1 until the quake is reached and, therefore, the ultimate resistance has been activated, and according to the viscous Option 0 thereafter. This option has, for example, the desirable property of zero dynamic resistance as long as the pile toe moves through the gap distance at no static resistance and is also common for very large quakes.

Recommended limits for the soil damping parameters are shown in Table 1. Obviously, the range of these parameters is large and often consistency has not been observed even at the same site, regardless of the different damping options. Indeed, great variations in calculated damping parameters between end of drive and restrike have been documented (Rausche et al., 2008a). It is suggested, that the CAPWAP analyst attempts to improve consistency by trying different resistance distributions. For example, shifting some of the shaft soil resistance near the toe to end bearing, or vice versa, only marginally affects the signal match, however, the associated changes in j_s values may be significant.

Example of a Toe Resistance with Gap

The following example demonstrates the effect of several unique soil resistance parameters of CAPWAP. A 10.6 m long steel H-pile (10 m below sensors) of type 305x110 (HP 12x74) was driven with a diesel hammer to 9.1 m penetration through compact sands and very dense gravels to sandstone bedrock. The PDA obtained time records (i.e., pile top force, F(t), pile top velocity, v(t), times pile impedance, Z, and wave–up) are included in Figure 4.

The measurements can be represented by their upward and downward traveling components (i.e. wave-up and wave-down) from

$$W_u(t) = ½ [F(t) - Zv(t)] \qquad (3)$$

$$W_d(t) = ½ [F(t) + Zv(t)] \qquad (4)$$

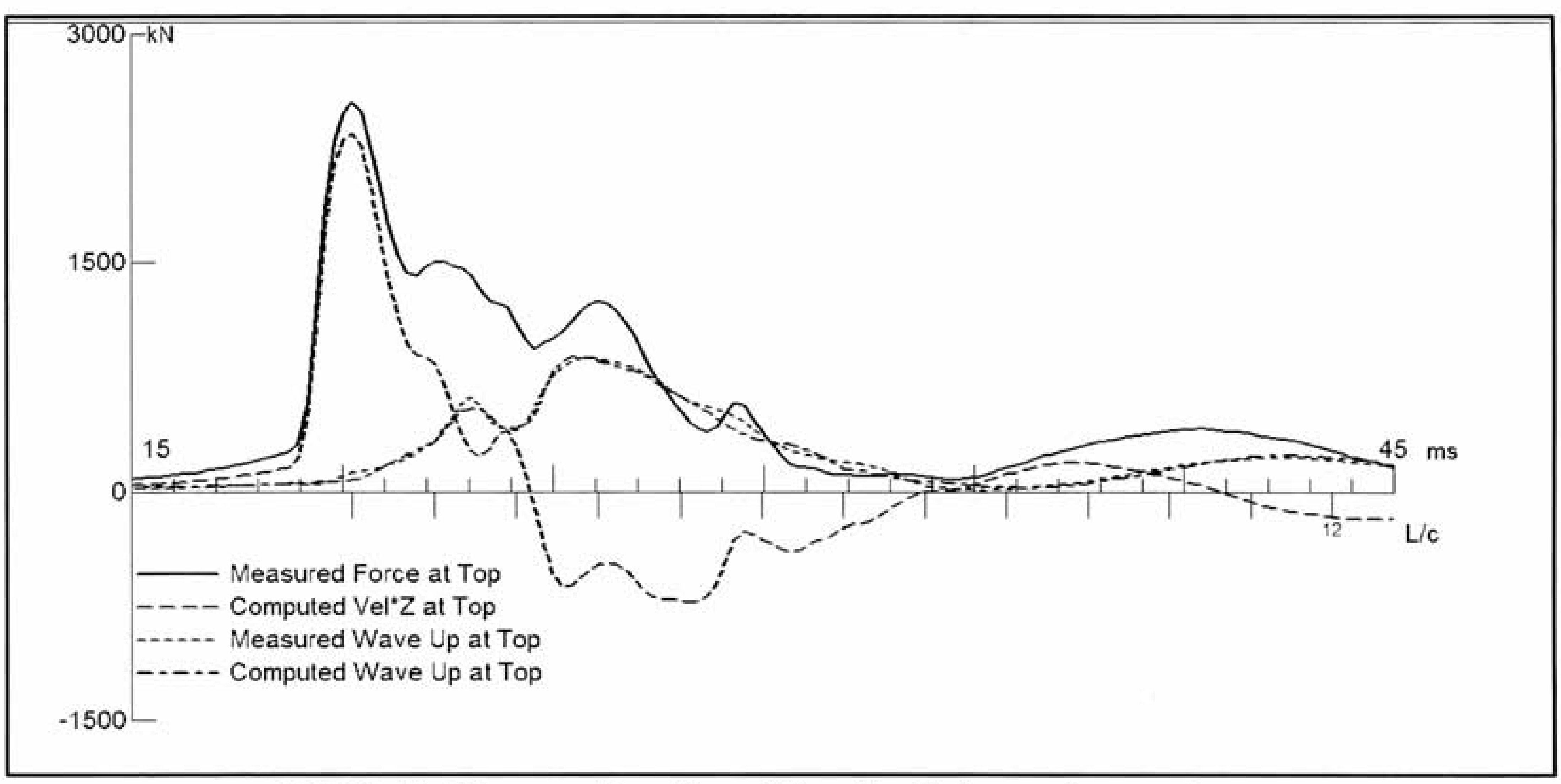

FIG. 4: Example of an H-pile driven to rock

While the wave-down curve primarily represents the input from the hammer system, the wave-up curve includes reflections from soil resistance and pile non-uniformities including the pile toe; prior to the first peak in wave-up, the monotonic increase is caused by the cumulative shaft resistance effect, beginning at the top; there follows a reduction due to the impact wave's return from the pile bottom; and then the next increase in wave-up is due to the strong build-up of toe resistance forces. The plot also shows the calculated wave–up curve which closely matches the measured wave-up curve. Wave-up matching is more precise than either force matching or velocity matching and, therefore, has generally replaced the other two options which were used in the early days of signal matching. Given the calculated wave-up curve, the corresponding, computed force at the sensor location can be computed from

$$F_c(t) = \tfrac{1}{2}\,[F(t) + Zv(t)] + W_{uc}(t) \tag{5}$$

where the c index refers to computed quantities.

The example match was achieved with the major soil resistance parameters shown in Table 2.

Table 2: Parameters for best match of example for end bearing pile

Quantity	Unit	Result
Total calculated capacity	kN	1890
Shaft/Toe capacity		1010/870
Shaft Damping / Factor	s/m	0.70 / Viscous (Option 0)
Toe Damping / Factor	s/m	0.30 / Smith-Case (Option 2)
Shaft/Toe Quakes	Mm	3.3/1.9
Shaft/Toe Gap	Mm	0.0/3.8

The toe gap value is relatively large (3.8 mm) compared to the toe quake (1.9 mm) and that is also demonstrated by the calculated toe resistance versus toe displacement in the upper left hand corner of Figure 5. The resistance vs. displacement curves on the left side include the static component (solid) and the static plus damping component (dashed). Thus, the difference between the dashed and the solid line is the damping resistance. To the right of this graph a sketch shows the components of the corresponding static resistance parameters and on the far upper right the wave-up match, which corresponds to the parameters of Table 2.

The center row of graphs was calculated with the same input parameters except the gap was zero and the toe quake equal to the sum of gap plus toe quake of Table 2 (5.7 mm). This would be the solution with a traditional Smith model. In contrast, the third row demonstrates the difference between Smith-Case and Case damping with otherwise identical parameters as for the best match solution, the corresponding pile toe resistance force versus pile toe penetration is shown in the bottom left portion of Figure 5. The three solutions demonstrate very clearly that, in this case, the total resistance is practically zero as long as the toe displacement is less than 3.8 mm. It seems likely the pile either rebounded off the rock, or the rock is crushed under the toe by each impact.

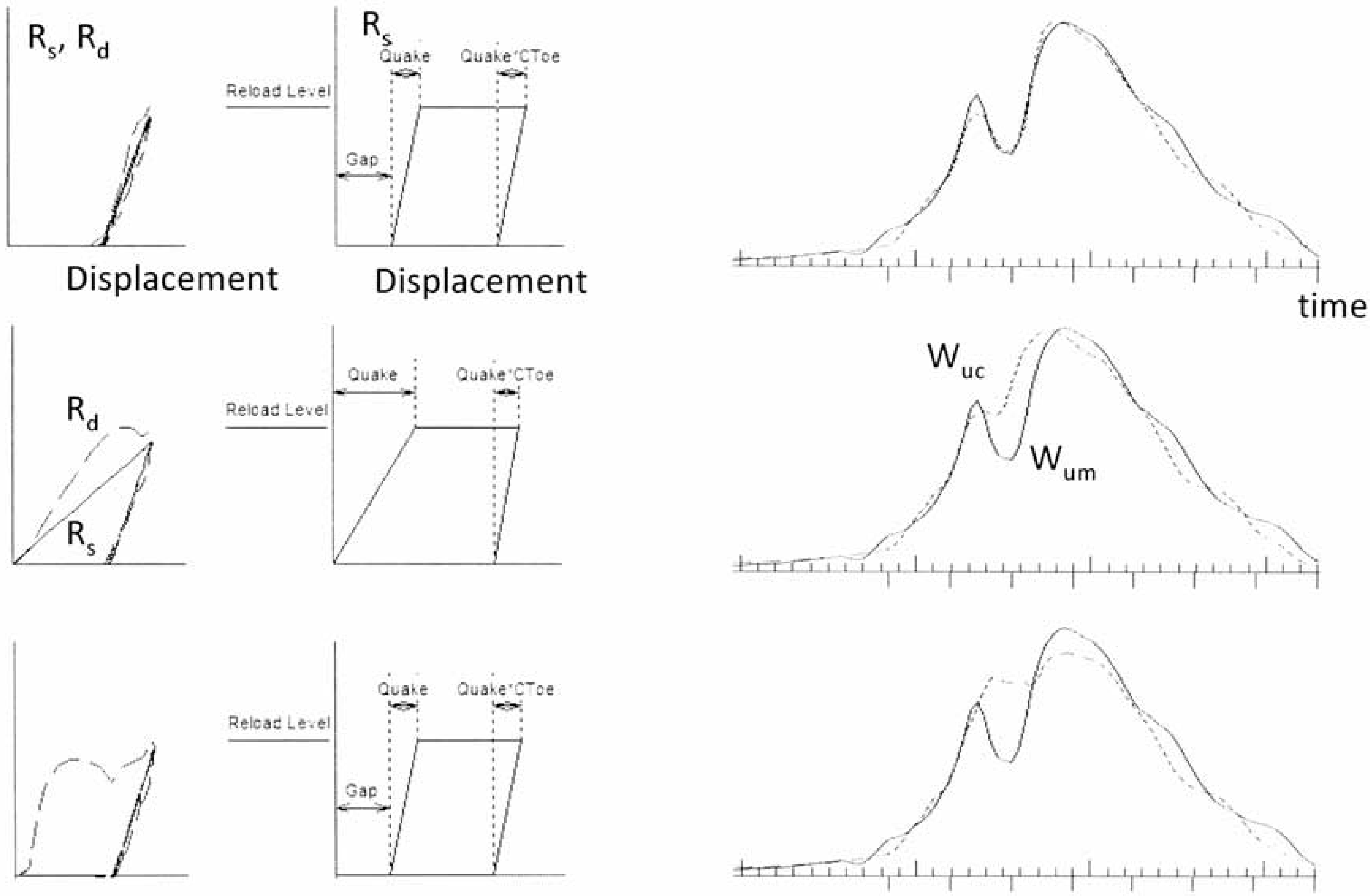

FIG. 5: Pile toe static and dynamic resistance and associated wave up matches, top: gap and Smith-Case damping (best match), center: no gap, bottom: gap and Case damping.

RADIATION DAMPING (RD)

The original Smith and basic CAPWAP soil models represent the resistance forces acting at the pile-soil interface. This model is generally satisfactory, if

- A shear failure occurs along the interface, and
- Once in the plastic range, the static soil resistance component stays constant as long as the pile velocity is positive.

The first condition is generally satisfied as long as the set per blow is greater than 2 mm; however, if the soil resistance is so great that the soil moves with the pile, the resistance will be a function of the relative pile-soil displacement and velocity and not its absolute values. (In the case of auger-cast piles, drilled shafts, or specific driven piles, a very rough surface may cause such a high shaft resistance component.) Also when the pile toe encounters hard rock, an end bearing failure of the rock will not occur and activated resistance will most likely be proportional to the pile toe motion since stress waves will propagate through the underlying material.

The second condition is violated, for example, in an open-end pipe, when the soil resistance at the bottom of the internal soil plug builds up, but begins to diminish when resistance plus inertia exceed the internal friction limit and the plug slips. Although more thorough models have been proposed (e.g., Paikowsky, 2008), they are more difficult to use under standard test conditions where knowledge about the soils is limited. Another example of a violation of the second condition is a soil resistance which decreases to a lower residual value after overcoming its peak strength.

Likins et al. (1996) introduced and correlated the CAPWAP RD model which, in an admittedly highly simplified manner, represents the soil surrounding the pile soil interface with a mass and a dashpot (Figure 6). The mass is a function of the pile perimeter and segment length. The dashpot is given in terms of pile impedance. Based on the correlation study, Likins et al. (1996) provided recommended values. Caution is, however, advised because the RD model generally produces higher predicted static capacities than the standard soil model.

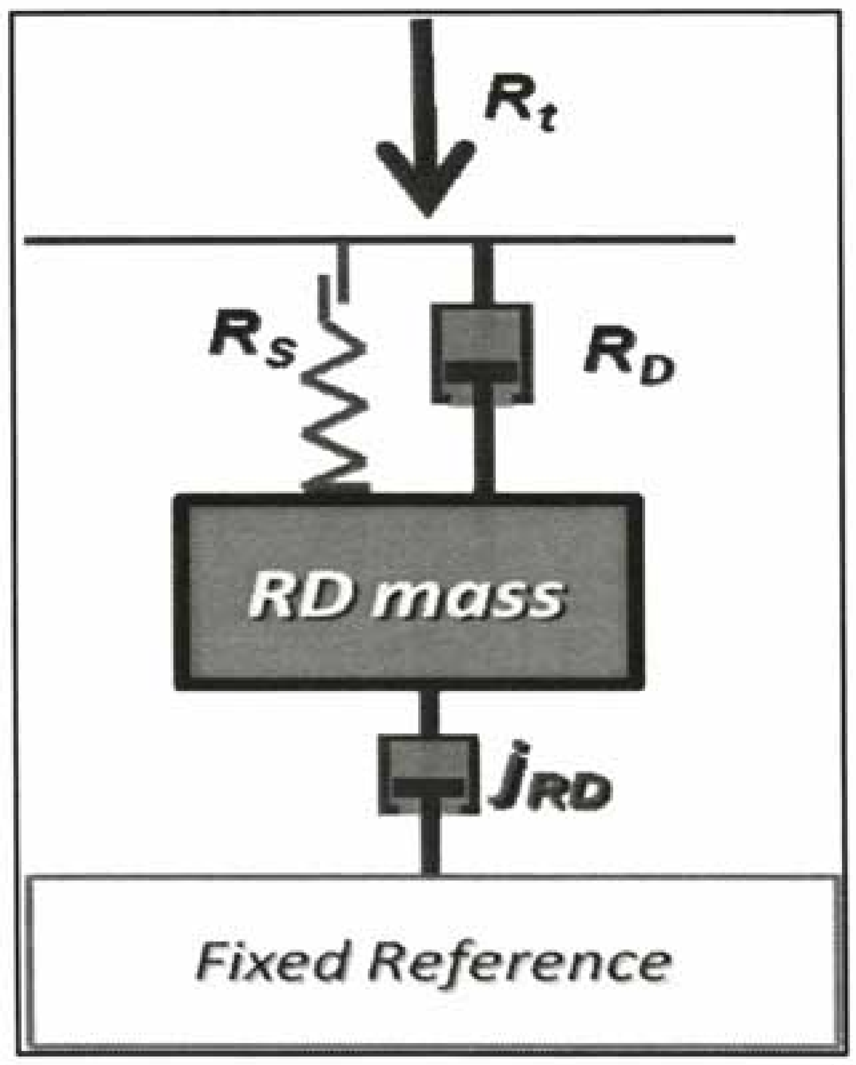

FIG. 6: Smith model with CAPWAP Radiation Damping

RESIDUAL STRESS ANALYSIS (RSA)

Residual stresses remaining in pile and soil after the impact event has occurred (usually due to an incomplete rebound) have long been recognized as a source for inaccurate interpretation of pile test results (Fellenius, 2002). Holloway (1979) was the first to propose an algorithm for a wave equation analysis with consideration of residual stresses. It requires a standard wave equation analysis and then a static equilibrium check followed by repeated cycles of dynamic and static analyses until convergence was achieved. Each dynamic analysis begins with the initial stresses and forces calculated by the previous static analysis. Convergence is achieved when two consecutive dynamic analyses have essentially the same final pile compression. The static analysis is relatively straight forward for the simple elasto-plastic Smith model.

It was soon recognized for long piles that the blow count calculation based on either pile top, pile toe, or an average final displacement could never be accurate if residual stresses were neglected. Fortunately, in CAPWAP the blow count is not as important a quantity for capacity calculation as it is for the wave equation. However, ignoring residual stresses may produce an erroneous resistance distribution profile. An option was, therefore, added which considers the residual stresses. Because of its complex soil model, CAPWAP replaces the static analysis with a dynamic analysis long enough in duration to assure that all pile segments practically reach zero velocity. A static equilibrium is then achieved and the next dynamic analysis cycle begins with the previous set of final stresses and forces as initial values. Convergence is obtained when the final pile compression of two consecutive analyses is similar, which typically happens after 3 or 4 cycles.

Residual stresses can only then occur in pile and soil if the shaft resistance develops negative resistance (R_{nu} in Table 1 and Figure 3) at the end of the analysis. While non-RSA analyses frequently produce a satisfactory signal match with R_{nu} of zero, it is generally agreed that RSA should be done with an R_{nu} of at least 20% of the positive ultimate capacity value (R_u).

On occasion, residual stress consideration in CAPWAP not only produces improved resistance distribution results, it can also produce more realistic capacity predictions on long slender piles with high resistance due to the stored energy "help" which the downward directed residual resistance forces provide in overcoming high resistance forces near and at the bottom of the pile.

The question has been asked whether or not both RD and RSA may be used in the same analysis. The main reason why this is discouraged is the contradiction which exists between the simplified RD and RSA models. RSA relies on the assumption that the soil surrounding the pile-soil interface is at rest while RD implies that the soil mass moves. However, RD does not accurately calculate the soil displacements and since small differences in displacements can cause relatively large changes in residual stresses, results may be less reliable when analyzing with both model options.

QUANTIFICATION OF THE MATCH QUALITY (MQ)

The quality of results from a CAPWAP analysis depends on what is considered the "Best Match", defined by good agreement between the measured and the computed quantity. The "Match Quality" (MQ) is quantified by calculating the sum of the absolute values of the differences between calculated and measured quantity (normally wave-up) in four time periods of Table 3 and Figure 7.

Table 3: Record time periods for match quality determination

Time Period	Time Period Description
I:	First 2L/c time when the shaft resistance is being activated
II:	Following Period I when the toe resistance is being activated
III:	Following Period I when the total capacity has its greatest effect
IV:	Following Period I for a longer time when the unloading occurs

To normalize MQ, the sum of differences is divided by the maximum measured pile top force. Periods II through IV overlap in that record portion where the total capacity has its effect and, therefore, after adding the signal match differences in all four periods, the total capacity determination is over-weighted in the MQ-value assuring that spurious match variations do not unduly affect the total capacity determination. Furthermore, Period I is normalized so the resistance distribution of long piles does not cause an excessive influence on the match quality.

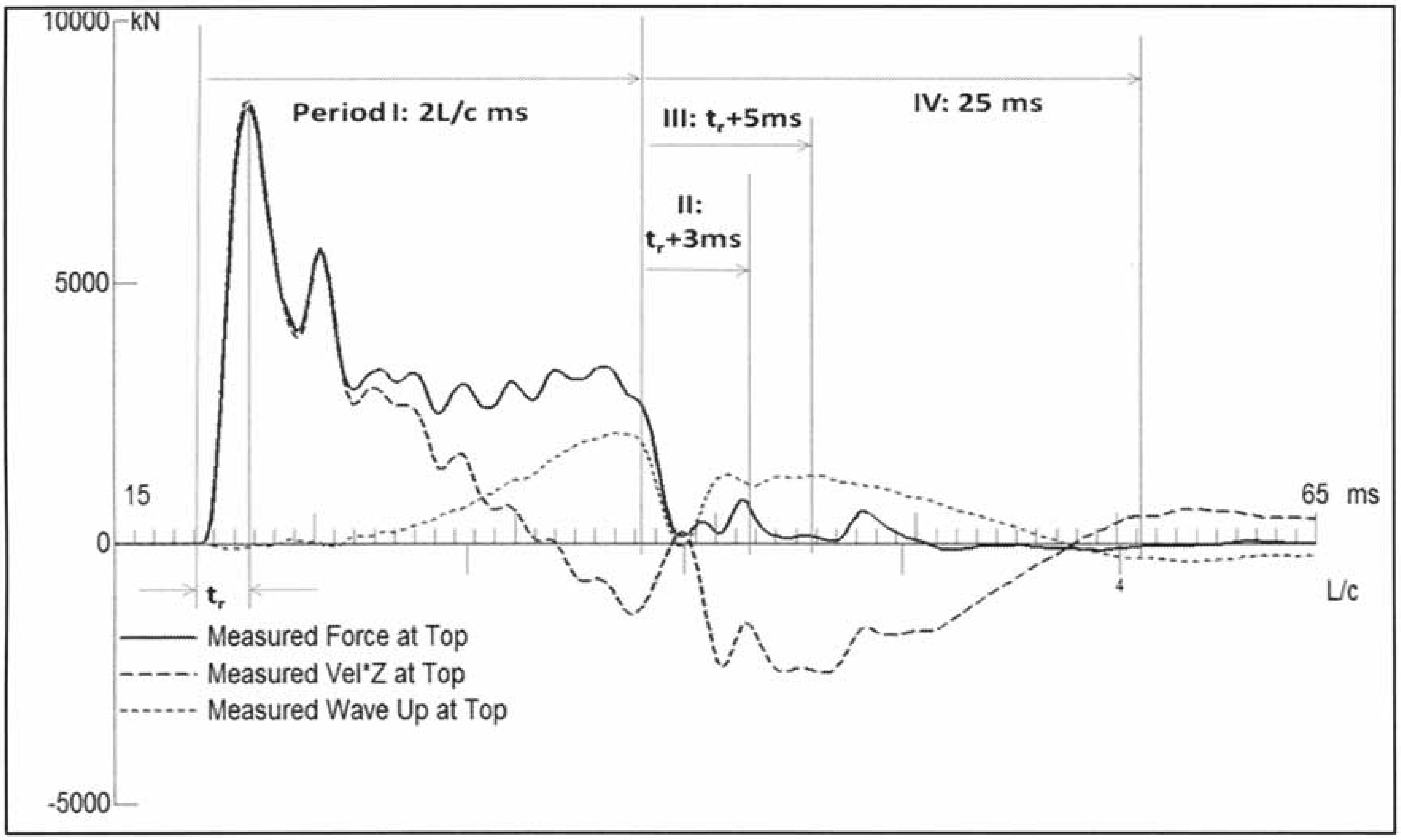

FIG. 7: Record divisions for match quality calculation

The calculated blow count is not strongly influenced by static capacity or resistance distribution; primarily because the measured velocity record defines the calculated final set at the pile top. The final set values of other depths along the pile are determined by the analysis procedure. Actually, to more correctly calculate the pile set and thus the blow count, RSA should be performed., Nevertheless correlations with static tests are improved (Rausche et al., 2000) if the computed and measured blow counts match. The MQ-value is increased by the set mismatch (in mm) minus 1, if this mismatch is greater than 1 mm. The non-RSA blow count is calculated as the inverse of the average of the final sets of all pile segments. It should be noted that generally there is uncertainty in knowing the exact pile set per blow under any individual hammer impact.

The CAPWAP definition of MQ is somewhat arbitrary and may have a significant effect on the prediction of bearing capacity and resistance distribution. Experience suggests that the current definition is reasonable and leads to reliable results. Correlations published to date (Likins et al., 1996, Likins et al., 2004) were achieved by generating the lowest possible MQ ("Best Match") under this definition. There is no definite upper or lower MQ limit for an acceptable CAPWAP result. The value that would be acceptable in any given case will depend on the particulars of the situation. Analysis of data from driven piles with well defined properties produce MQ typically between 2 and 4. Smaller values are uncommon and MQ less than 0.5 is nearly impossible. Greater MQ values occasionally occur with acceptable analysis results, for example, when tension cracks are modeled, or for mostly end-bearing resistance with non-linear behavior situations. However, MQ values greater than 6 may indicate incomplete analysis, poor data or a severe pile integrity problem, in which case the results could be unreliable. Hannigan and Webster (1987) presented a correlation study where one case did not provide a satisfactory agreement with the static load test and that was the only case with a poor match quality.

CAPWAP PROCEDURE

CAPWAP requires five distinctly different processes:

a. Records selection and data checking
b. Pile model generation
c. Signal matching
d. Result checking
e. Output

Steps (a) and (d) are as important as the others; they have to be performed by the experienced analyst. Rausche et al. (2008b) discuss certain principles of record selection. For non-uniform piles, even step (b) requires an engineer's careful input.

Step (d) includes attempting several different solutions, maybe with a view of more or less well known soil properties, or experience in similar soils. Ultimately, the best match solution which satisfies certain constraints (e.g., Table 1) on the various model parameters should be selected.

The signal matching process, Step (c), has been simplified with powerful automatic search procedures. The engineer first selects a total capacity with an initial resistance distribution, and an initial set of damping and quake parameters, then automatically, CAPWAP determines all remaining soil model parameters. The total capacity and resistance distribution can then be automatically improved by seeking the lowest MQ, or "Best Match". After a best match is found, the engineer may consider potential modifications of the pile model (e.g. the stress wave speed), or the adoption of a more complex soil model (e.g., RSA, RD).

Two different types of automated procedures exist in CAPWAP; the first type calculates improvements from certain match properties. For example, the resistance distribution is improved by direct calculation of the mismatch over the first time period. The second type simply minimizes the MQ value by a grid search of the relevant soil model variables which includes the total capacity. Both types of searches can be applied and are helpful in the matching process to achieve the final solution.

EXAMPLE PIPE PILE

The following example compares results from RSA and RD analyses with the standard CAPWAP model. A 762x19 mm open-end pipe pile was tested during initial driving and restrike with a hydraulic hammer. The total penetration of 50 m included various layers of sand and clay into a sand bearing layer. Such a long, slender and relatively flexible pile justifies RSA as more realistic than the standard model. During the restrike test the pile was at practical refusal (1 mm/blow). Figure 8 (top) shows the force and velocity data. Indeed, good RSA convergence was achieved (2.6% at 4 cycles) with a 25% negative limit resistance (R_{nu}) for RSA. The CAPWAP results for both RSA and standard model are depicted at the bottom of Figure 8, both in the form of "unit shaft resistance" and "force in pile" (as would be measured in the pile at the predicted ultimate capacity). MQ values for the two solutions were similar (1.65 and 1.66 for Standard and RSA, respectively). RSA indicated a roughly 11% higher capacity than the standard, non-RSA model. Such differences are expected for the wave equation approach, but are less common for CAPWAP. The calculated resistance distributions differ primarily over the bottom half of the pile where it would be expected that the mobilized capacity is on the low side because of small pile motions. The RSA results are, therefore, considered more realistic and reliable than those provided by the standard model.

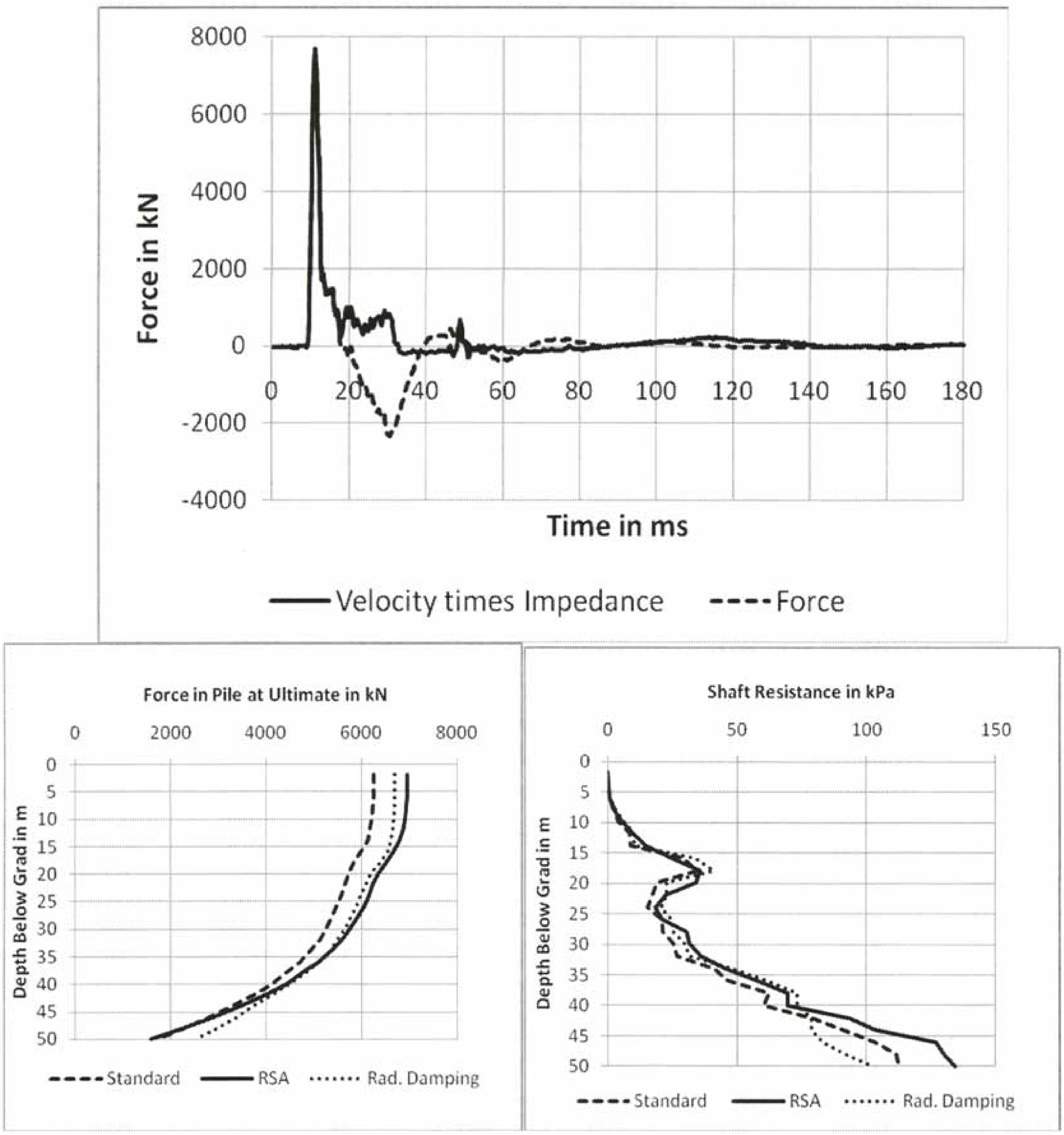

FIG. 8: Example of results with standard, RSA and RD options; top: force and velocity records; bottom: resistance distribution results in terms of pile forces and unit shaft resistance

The RD analysis was also performed with both shaft and toe RD models. The reason why RD would be appropriate for the shaft was the very small set per blow. Also, the pipe was open-end and plug slippage could be expected, justifying both shaft and toe RD models. The CAPWAP analysis produced a very good match (MQ=1.29) with RD dashpot values (in pile impedance units) of 1 and 4 for shaft and toe, respectively (recommended ranges 1 to 2 and 3 to 10, respectively; actually, the higher limits are not strictly binding since exceeding the higher limit has minimal effect and would produce more conservative results). The RD model produced a slightly lower capacity than RSA, being 7% higher than the standard result. Having produced the very best match, the RD result is preferable.

Large Drilled Shaft Test Example

Two large diameter shafts were drilled through various layers of clay, sand and silt and socketed into a hard limestone. The two shafts had top diameters of 2300 and 1820 mm, and respective lengths of almost 20 and 24 m. The shafts were drilled through telescoping casings with respective rock sockets of 1980 and 1520 mm diameter. For transducer attachment, the contractor added shaft top extensions of 2130 and 1520 mm diameter, respectively. The two production tests were conducted to check both capacity and integrity and, for that reason, impact velocities were limited to meet these test objectives. Using the APPLE drop weight system (Robinson et al., 2002) with a ram weight of 360 kN, the test engineers applied two ram drops of 600 and 900 mm height to the smaller and larger shafts, respectively. The resulting shaft penetrations per blow were near zero.

Record evaluations were conducted with both the standard and the RD CAPWAP models. The RD model is more appropriate since these two shafts penetrated very little and thus shaft and geomaterial likely moved nearly together. Also the probable roughness of both upper shaft and bottom socket would assure that there was no relative shaft-rock motion. Indeed the RD model produced better MQ values of 3.3 and 4.56 than the standard model (3.45 and 5.38). It is believed the rock socket and end bearing behavior is not linear in the tested load range and, therefore, causing the relatively high MQ values. Figure 9 shows the calculated load-set curves and associated load transfer curves, respectively, for both standard and RD models.

The APPLE loading device activated capacities of nearly 100 times its own weight on the larger pile because of the very stiff shaft-rock response. For the smaller pile, the resistance response was more flexible and for that reason, and due to the lower applied energy, the dynamic test loads mobilized lower capacity values.

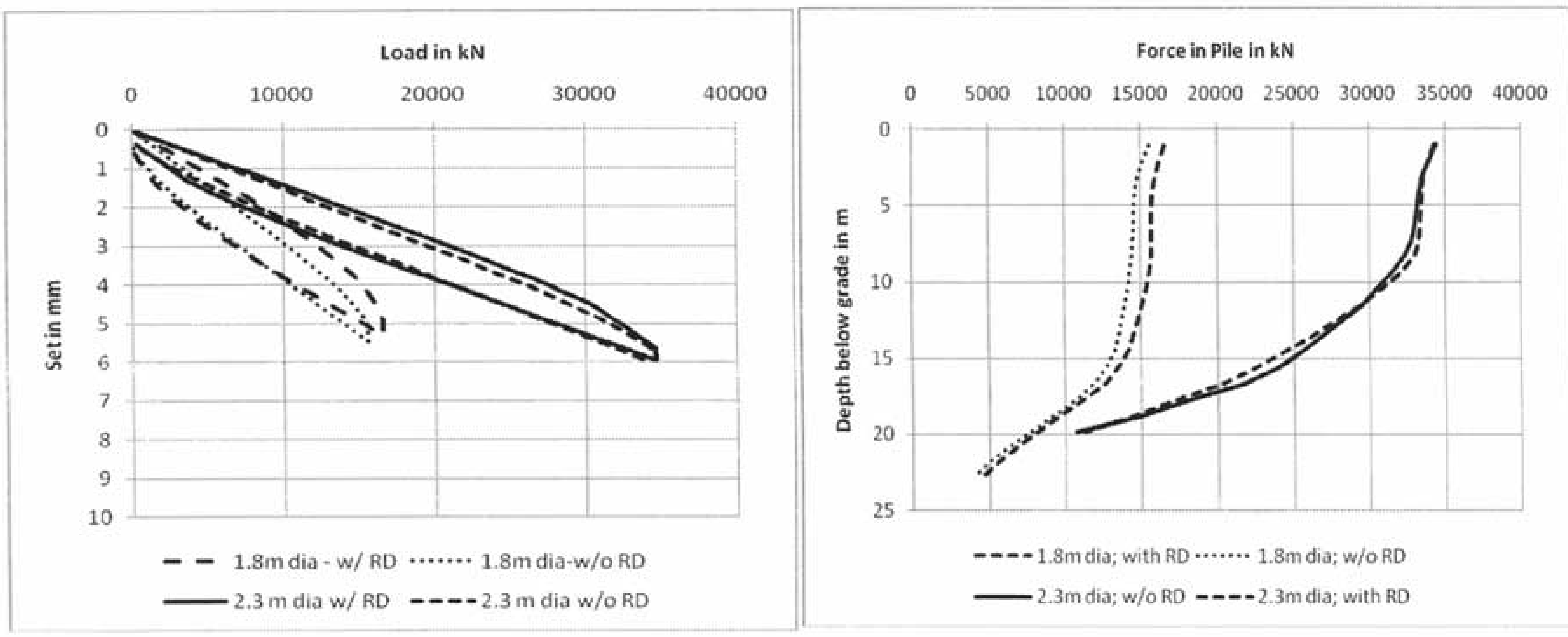

FIG. 9: Calculated results for large shafts, left, load-set curves and, right, resistance distribution effects

RECOMMENDATIONS

Although measurement technology and the maturity of the software have greatly advanced, it is still strongly recommended that persons with dynamic testing experience review and apply the dynamic load testing results. While lack of uniqueness is sometimes considered a major drawback of signal matching, the total bearing capacity has been shown to well defined by the CAPWAP procedure. A more worthwhile area of continued research would be the investigation of soil plugging in non-displacement piles and studies of the end bearing behavior of large diameter displacement piles.

SUMMARY

CAPWAP is a signal matching analysis for the calculation of dynamic load test results performed by dynamic loading on deep foundations. The method requires the input of measured force and velocity, pile geometry and pile material information. Originally based on Smith's pile and soil models, CAPWAP has been further developed during the past 40 years. The pile lumped spring-mass model was replaced by continuous pile segments and tension and compression slacks were added for the modeling of splices, connections and minor tension cracks.

For meaningful results, the complexity of the soil conditions also requires that signal matching is done with a more flexible soil model than provided by Smith. While CAPWAP still works in a one-dimensional mode, i.e. the distributed soil resistance effects are represented by a number of concentrated forces, important additional features were modified: unloading parameters for the static resistance component, modifications of the damping approach, a radiation damping model and residual stress analysis option. Suggested ranges for parameters are given. Both RD and RSA help improve the signal match and correlations with static test results.

The CAPWAP results include total capacity including its distribution along the shaft and end bearing, a load-set curve for the short duration dynamic test at the time of testing, dynamic resistance (damping component), and the stiffness of the static resistance components. As shown in the literature, these results have been shown to compare well with static tests and proven reliable indicators of deep foundation performance. However, additional research on improved models for the dynamic behavior of highly plastic soil types is recommended.

ACKNOWLEDGEMENTS

The development of the dynamic load testing methods and their application to a wide variety of deep foundations would not have been possible without the enthusiasm for innovation, recognition of the importance of testing, and understanding of the potential of Dynamic Load Testing of visionary professionals in the geotechnical community. Foremost among these professionals has always been Mr. Clyde Baker who already during the early development stages of these methods stimulated us to make the dynamic pile and shaft testing methods more generally applicable, reliable and more convenient to use. It was Mr. Baker who suggested developing the large drop weight system for the proof tests described above. The authors thank him for his encouragement and foresight.

REFERENCES

Authier, J. and Fellenius, B.H., (1980). "Quake values determined by dynamic measurements", *Proc. of the 1st Int. Conf. on the Appl. of Stress-Wave Theory to piles*, Stockholm, Sweden, Balkema, pp 197-216.

Chin, V.B.L. and Seidel, J.P., (2004). An experimental study in the viscous damping response of the pile-clay interfaces. *Proceedings of the 7th Int. Conf. on the Appl. of Stress-Wave Theory to Piles,* Kuala Lumpur, Malysia, The Inst. of Eng. of Malaysia, pp 435-446.

Fellenius, B.H., (2002). "Determining the resistance distribution in piles, Part 2: Method for determining the residual load", *Geotechnical News Magazine*, 20(3) pp 25-29.

Fischer, H.,C., (1960). "New graphodynamical pulse method of computing pile-driving processes", *Appl. Sci. Res*. (A 9), pp. 93-139.

Gibson G.C., and Coyle, H.M., (1968). "Soil damping constants related to common soil properties in sands and clays." *Research Report* (No. 125-1), *Texas Transportation Institute*, Texas A&M University, College Station, TX.

Goble, G., Likins, G., Rausche, F., (1975). "Bearing Capacity of Piles from Dynamic Measurements". *Final Report to Ohio DOT, Case Western Reserve University,* Cleveland, OH.

Goble G., Rausche, F., and Likins, G., (1980). "The analysis of pile driving-a state of the art". *Proc. of the 1st Int. Conf. on the Appl. of Stress-Wave Theory to piles, Stockholm*, Sweden, H. Bredenberg, Ed., Balkema, pp 131-161.

Hannigan, P., (1984), "Large quakes developed during driving of low displacement piles", *Proc. of the 2nd Int. Conf. on the Appl. of Stress-Wave Theory to Piles*, Stockholm, Sweden; Balkema, pp118-125.

Hannigan, P., and Webster, S., (1987) "Comparison of static load test and dynamic pile testing results", *Proc. of the 2nd Int. Symposium,* Deep Foundations Institute, Luxembourg, pp 1-24.

Holloway, D.M., Clough, G.W., and Vesic, A.S., (1979), "A Rational Procedure for Evaluating the Behavior of Impact-Driven Piles", *Behavior of Deep Foundations, ASTM STP 670*, American Society for Testing and Materials, pp. 335-357.

Hussein, M., and Camp, W., (1994). "Dynamic and static tests on driven and cast-in-place piles". *Int. Conf. on the Design and Construction of Driven Piles, FHWA*, Orlando, 1103-1117.

Komurka, V., (2004), "Incorporating Set-Up and Support Cost Distributions into Driven Pile Design," *Current Practices and Future Trends in Deep Foundations*, ASCE/Geo-Institute, GSP 125, 0-7844-0743-6, pp. 16-49.

Likins, G., (1983). "Pile Installation Difficulties in Soils with Large Quakes. Dynamic Measurement of Piles and Piers", *ASCE Geotechnical Engineering Division, Symp. 6 at the ASCE Spring Convention,* Philadelphia, PA.

Likins, G., Rausche, F., Thendean, G., Svinkin, M., (1996). "CAPWAP Correlation Studies". *Proc. of the 5th Int. Conf. on the Appl. of Stress-Wave Theory to Piles,* Orlando, FL, pp 447-464.

Likins, G. and Rausche F., (2004). "Correlation of CAPWAP with Static Load Tests"; *Proc. of the 7th Int. Conf. on the Appl. of Stress-Wave Theory to Piles*, Petaling Jaya, Selangor, Malaysia, Institute Engineers Malaysia, pp 153 – 165.

Paikowski, S.G. and Chernauskas, L.R., (2008). "Dynamic Analysis of Open-ended Pipe Piles", *Proc. of the 8th Int. Conf. on the Appl. of Stress-Wave Theory to Piles*, Lisbon, Portugal, IOS Press, pp 59-76.

Rausche, F., Goble, G., and Likins, G., (1985). "Dynamic determination of pile capacity". *Journal of Geotechnical Engineering*, 111, (3), pp 367 – 383.

Rausche, F., Moses, F., and Goble, G., (1972). "Soil resistance predictions from pile dynamics". *ASCE, Journal of the Soil Mechanics and Foundation Division*, 98(SM9), pp 917-937.

Rausche, F., Nagy, M., and Likins, G., (2008a). "Mastering the art of pile testing". *Proc. of the 8th Int. Conf. on the Appl. of Stress-Wave Theory to Piles*, Lisbon, Portugal, IOS Press, pp 19-32.

Rausche, F., Pezzetti, G., Jannacci, C., and Klesney, A., 2006. Dynamic analysis of follower driven piles for the Venice floodgate project, *Proc. of the 10th Int. Conf. on Piling and Deep Foundations*, Deep Foundations Institute, Amsterdam, pp 763-772.

Rausche, F., Robinson, B., and Liang, L., (2000). "Automatic signal matching with CAPWAP". *Proc. of the 6th Int. Conf. on the Appl. of Stress Wave Theory to Piles*, Sao Paulo, Brazil, Balkema, pp 53-58.

Rausche, F., Likins, G., Hussein, M., (2008b). "Analysis of Post-Installation Dynamic Load Test Data for Capacity Evaluation of Deep Foundations". *From Research to Practice in Geotechnical Engineering*, ASCE, Geotechnical Publication No.180, Reston, Virginia, pp 312-330.

Robinson, B., Rausche, F., Likins, G., Ealy, C., (2002). "Dynamic Load Testing of Drilled Shafts at National Geotechnical Experimentation Sites". *Deep Foundations 2002, An Int. Perspective on Theory, Design, Construction, and Performance, Geotechnical Special Publication No. 116,* O'Neill M. W., and Townsend, F. C. Eds., ASCE, Orlando, FL.

Smith, E.A.L., (1960). "Pile driving analysis by the wave equation", *ASCE, Journal of the Soil Mechanics and Foundations Division*, 86.

Stevens, R.F., (2004). "Pile acceptance based on combined CAPWAP analysis", *Proc. of the 6th Int. Conf. on the Appl. of Stress-Wave Theory to Piles*, Sao Paulo, Brazil, Balkema, pp 17-27.