HELPFUL HINTS FOR FIELD TESTING AND DATA INTERPRETATION USING THE PILE DRIVING ANALYZER®

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Table of Contents

INTRODUCTION

Dynamic pile testing has become truly a routine procedure no longer confined to researchers. It has been proven cost effective and reliable, and as a result many contractors, consultants and government agencies have acquired the necessary equipment to service their own needs; others prefer to obtain the testing as a service from those of us offering such testing services.

However, technology changes. Microprocessors have revolutionized our everyday lives from hand calculators to microwave ovens. Computers used to be large, unfriendly, and relatively inaccessible. However, they have been of great benefit to the civil engineer in solving complex analysis problems. For example, it has been noted that the wave equation analysis of pile driving was perhaps the first non-military application of computers to the civil engineering field. The PC version of the PDA has made for a rather user friendly environment.

THE DIGITAL PILE DRIVING ANALYZER

Because of the arrival of powerful PC technology and the growing familiarity by engineers with computers, Pile Dynamics first developed a PC version of our PDA in 1989 (called "GCPC") and the current model (PAK) in 1991. The PAK has several advantages over the previous "knob and switch" model (*i.e.*, the "blue analyzer" Model GCXS).

A) The PAK/GCPC is a fully self-contained unit. The PAK currently has a 486SLC processor with 8MB RAM and a 240 MB hard disk. The PAK during operation displays the data curves on the VGA graphics screen. An external VGA monitor can also be attached. The pile properties are always displayed and labeled with the proper dimensions. The PDA will NOT accept input quantities which are out of reasonable ranges; for example, modulus or specific weight values which are too high or too low are rejected. The most noticeable feature of the PAK is the small size (140 x 280 x 380 mm) which is roughly only 30 percent of the "blue PDA" volume! The PAK looks like a "briefcase" and the soft carry case allows us to hand carry the PAK through airports (PAK weighs only about 9kg - 20 pounds) and check for air flights only one transit case with cables and transducers, etc.

B) The PAK has been designed to operate on either 90 to 250 volt AC power or 12 volt DC. Thus it can be run off the car battery. For concrete or timber piles, battery powered drills are available and the generator is then no longer required.

C) The PAK can store every blow (currently up to 860 blows). The user can then select "after the fact" which blows (or even all blows) to save for permanent history on floppy disks. One 720K 3.5 inch floppy disk will hold 86 blows. One other advantage of this archive storage is that all individual transducer records are retained and can therefore be replayed with only one accelerometer or to investigate bending in the strain record. It also means that the replay of data will always match exactly the original data.

D) An external floppy disk connects through the parallel output port, which makes the system more modular and thus easier to repair or replace. A built-in floppy drive was offered as an alternate to the external floppy.

E) The PAK has a standard built-in membrane keyboard with function keys labeled with the PAK PDA functions and an extra keypad for some of the frequently needed commands. This is more rugged and dust resistant than the standard detachable keyboard. The PAK also accepts a standard detachable keyboard.

F) The signal conditioning now contains 4 channels of strain, 3 piezoelectric accelerometers (for decades our standard accelerometer), 3 new piezoresistive type accelerometers, and four integrators (to get velocity). The third channel of piezoelectric and/or piezoresistive acceleration is for backup purposes; with a little effort, the user could change a jumper to insert the backup signal conditioning. The 4 channels of strain and integration allow us now to simultaneously acquire 4 strain and four velocity signals where extra backup on the pile is requested (such as offshore oil projects) or for drilled shafts.

G) Because the PAK is a complete PC, it can perform a CAPWAP analysis in the field to confirm or correct PDA Case Method estimates. The PAK has standard serial RS232 and parallel ports.

H) The PAK was designed with software control functions given through keyboard entry. Most functions are also logically named; for example, all **P**rinter commands start with the letter **P**, all **S**torage functions begin with **S**, all **R**ecall functions start with **R**... The PAK uses function keys for commonly accessed procedures such as printer on/off, complete printout, transducer selection, or accept/standby. Several functions make use of pop up "windows" which have further help and extra options clearly identified.

I) The calculated values for six (or now optionally nine) selected parameters are displayed and can be automatically saved in an ASCII file (using SQ command) for further summary output (as required by ASTM D-4945) by the PDAPLOT program. Using the PDAPLOT **SQ files**, we no longer need to print since the results are available in electronic form; using the new **RQ review function**, we can review these results inside the PAK program. For diehard printer fans, there is a provision to print these results with an optional external printer (contact PDI for details).

J) The software has also been expanded with new data input "forms," transducer selection and checking, and **"expert advice"** on data quality and result interpretation given as an aid primarily to the new user. The "HELP" function has also been helpful to new users seeking the available commands.

Because most everything is controlled by keyboard and external scopes and tape recorders are no longer required, the training process is simplified and shortened. This translates into hopefully the extra time being used to concentrate on the theory involved and practical applications. It makes introduction to new PDA operators less time consuming. Eliminating the expensive tape recorders (about $8,000) also reduces the overall cost of a system.

TRANSDUCER PERFORMANCE

The first and foremost requirement for any field work is to **acquire quality data** for without it the results are also suspect. This implies that the engineer should place his primary emphasis on the collection of good data. The first step is to **confirm that the transducers are functioning** properly. Test the transducers as described in the PDA manual and observe their performance; replace or repair defective units. Also regularly check the cables and their connections (wiggling the wire near the connector while observing the strain "offset" values. For PAK, set the trigger to the appropriate transducer as the cables and connections are checked as above or again attach output to a scope. Sufficient spare cables and connectors should be available on site to replace defective units as required.

TRANSDUCER CALIBRATION

The field settings of Area, Length, Modulus, Density, (EA/C, MC/L, L/C) DELTA, J, and PEAK selection do not affect the data recorded and can be corrected in reprocess (the field results may be wrong!). The **calibration settings for F1-F2-A1-A2 MUST be correct,** requiring that the calibrations of the transducers be accurately known and that the **correct** wave speed c used; this c value is the wave speed (WS) at the gage location which can be different than the average c value (WC) used in the overall wave return (2 L/c setting) due to cracks, etc. The "Units" (SI, Metric or English) must also be properly chosen. A more detailed discussion is included in Appendix 1 to this paper.

It is unlikely that the two transducer systems (namely strain and acceleration) could produce proportional data if both were not working properly and both were not properly calibrated. If any strain transducer consistently fails to give good proportionality with the velocity, then it also should be recalibrated or replaced. It can be concluded in our study of strain transducer recalibrations that the calibration or accuracy of any individual strain transducer (prior to 1993) is within about five percent. In late 1992, we developed a more precise method of calibration. As this new primary measurement system seems to give repeatability to within one percent, it is **recommended to now recalibrate all older transducers** (ASTM D-4945 requires annual recalibration).

No data is abundantly available to verify the accuracy of the accelerometer calibration, although the accelerometer manufacturers generally quote 5 to 10 percent accuracy figures and the data we do have generally supports this accuracy. However, if proportionality is poor; the strain is more susceptible to bending and local stress concentration influences. PDI developed in 1993 its own system for calibrating accelerometers based on a Hopkinson's Bar test. Equipment (or calibration services) is now available to satisfy yearly recalibration requirements.

To check the PDA calibration itself, the built-in (PAK "CT" function) feature should be used, on a periodic basis, to compare values with known inputs as described in the PDA manual; this takes little time and will detect any problem with the PDA itself. Using this Cal Test feature also helps determine if the transducer is functional.

TRANSDUCER ATTACHMENT

The **transducers should be firmly attached** to steel piles as the high acceleration, could cause slippage, resulting in inconsistent and therefore erroneous data. If the strain transducer is not secured properly, damage to the transducer could also result. For concrete piles, over tightening the transducers may result in pulling of the anchors instead; it is therefore very important to firmly seat the anchors. It is always very important to have **both strain signals working** since bending usually makes the strain signals on opposite sides quite different; if a strain transducer fails partway through a test, it should be replaced immediately with a good working unit. The transducers should be attached diagonally opposite the pile neutral axis so that bending is properly canceled. It should be noted that it is really strain which is being measured. It is further assumed that the pile is of a linear elastic material. In high strain situations, the pile material may go into the plastic range and/or pile top damage may result. This could result in unrealistic and inconsistent forces, energies and capacities in these cases. Since the acceleration signals are very similar, generally one good unit is all that is required for a successful test (unless bending is excessive).

The transducers also should (if at all possible) be attached at least 2 and preferably 3 or more diameters below the pile top to avoid end effects and local contact stresses. In general, the farther from the pile top the gages are attached, the better quality the data becomes (the only difference this makes to the results is that the maximum energy EMX is slightly reduced due to the energy required to compress the pile above the transducers). Also avoid attaching near cross section changes, welds, splices, stiffeners or other nonuniformities. It should be noted that "telltale" pipes cause complications which must be properly accounted (contact PDI for advice). For regularly reinforced concrete piles (*i.e.*, not prestressed) which are common in Europe, the transducers should always be near the pile top to avoid including cracks between the attachment points which could induce serious errors. For large diameter drilled shafts, using 4 strain transducers may produce better data if attaching 2 diameters below the top is impractical.

For all concrete piles, we now use the same (Hi G) accelerometers as for all other pile types and seem to get better data than with the older Low G type, particularly when accelerations are higher due to compressed cushions. (In fact, **we no longer recommend the Low G accelerometer** and have not sold new units for a couple years for any dynamic pile testing). We have recently been investigating another new piezoelectric accelerometer (shear type) which appears to be a significant improvement for use on steel piles; it should be available from PDI in mid 1994.

The new piezoresistive type accelerometers are capable of good data on steel to steel impacts from uncushioned hammers or SPT test situations. It should be noted that this new piezoresistive accelerometer can be used for all pile types and all hammer types and in all cases gives data of at least the quality of the older piezoelectric type accelerometer. Signal conditioning and software in the PAK allow the user to operate with either accelerometer type. (For "blue PDAs" or the GCPC, PDI has developed plug-in replacement signal conditioning boards for this new accelerometer which allows upgrade without having to send the PDA for repair; the entire upgrade can be completed by the average PDA operator in 5 minutes.)

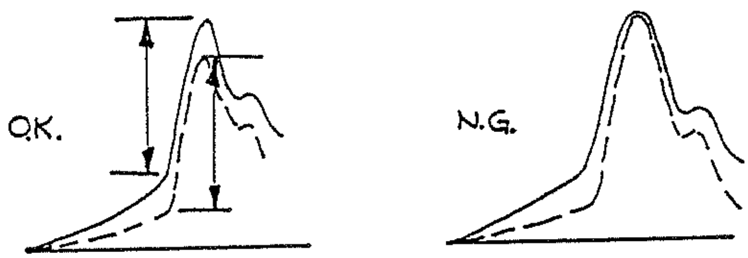
DATA QUALITY CHECKING

To confirm good quality data, put the PAK TS timescale to full and **check for consistency**; for similar hammer blows, the data should also look similar. Each accelerometer can be checked individually (PAK by F9 function key). If one accelerometer is more consistent than the other, it is better to use only that accelerometer than to corrupt your data with a bad signal.

One way to assist the evaluation of velocity quality is to inspect the displacement curve (selected by DPE command on PAK). Better data has the correct final displacement (compared with observed set per blow); questionable data with even negative (upward) final set would require more data adjustment in CAPWAP, and may tend to overpredict capacity and underestimate transferred energy.

In the field, begin with all transducers active (F9 and F10 function keys). By watching the data display, the engineer can select the transducers as described above much better than the PDA. If the velocity is relatively unstable from blow to blow, switch to A1 or A2 in the hope of finding one accelerometer which is functioning properly. I spend by far the largest portion of my time during the hammer operation in observing the PAK graphic screen where I can view the data quality and assess other features such as bending, tension, large quake cases, friction distribution, damage, wave speed, capacity determination methods, etc..

It is very important to **check for proportionality** of the major input rise (the peaks don't have to match). The WU (wave up) should be smooth through the impact time.



For slow rise times, impedance increases just below the transducer location, or gages near the ground elevation with high friction in the upper soil layers, the force can be higher than the velocity at the time of the first peak. For impedance decreases just below the transducers, the velocity can be higher than the force. The transducers also should not be attached near (either just above or below or worse still, straddling) an impedance or cross section area change as the stress path can rapidly change through this area.

The PAK does have "expert advice" which looks at data quality. This feature is accessed using the "FL" command (we suggest FL always be set to "ON"). If problems are detected, a "warning" will be shown. The following are automatically investigated and warnings are displayed in windows for problems such as:

1. proportionality
2. excessive bending
3. accelerometer performance

4. strain "noise" (low frequency drift)
5. strain slippage (offset at end of blow or high frequency ringing causing negative force before 2L/c)

WAVE SPEED DETERMINATION

In all PDA's the force and velocity are normally displayed at a scale proportional to the impedance so that proportionality between velocity and force can be easily evaluated. Some computations (such as capacity and tension stresses) are influenced by wave speed input. **Since the force calibration depends upon the wave speed, it should be obvious that correct determination of wave speed is essential for a good test.**

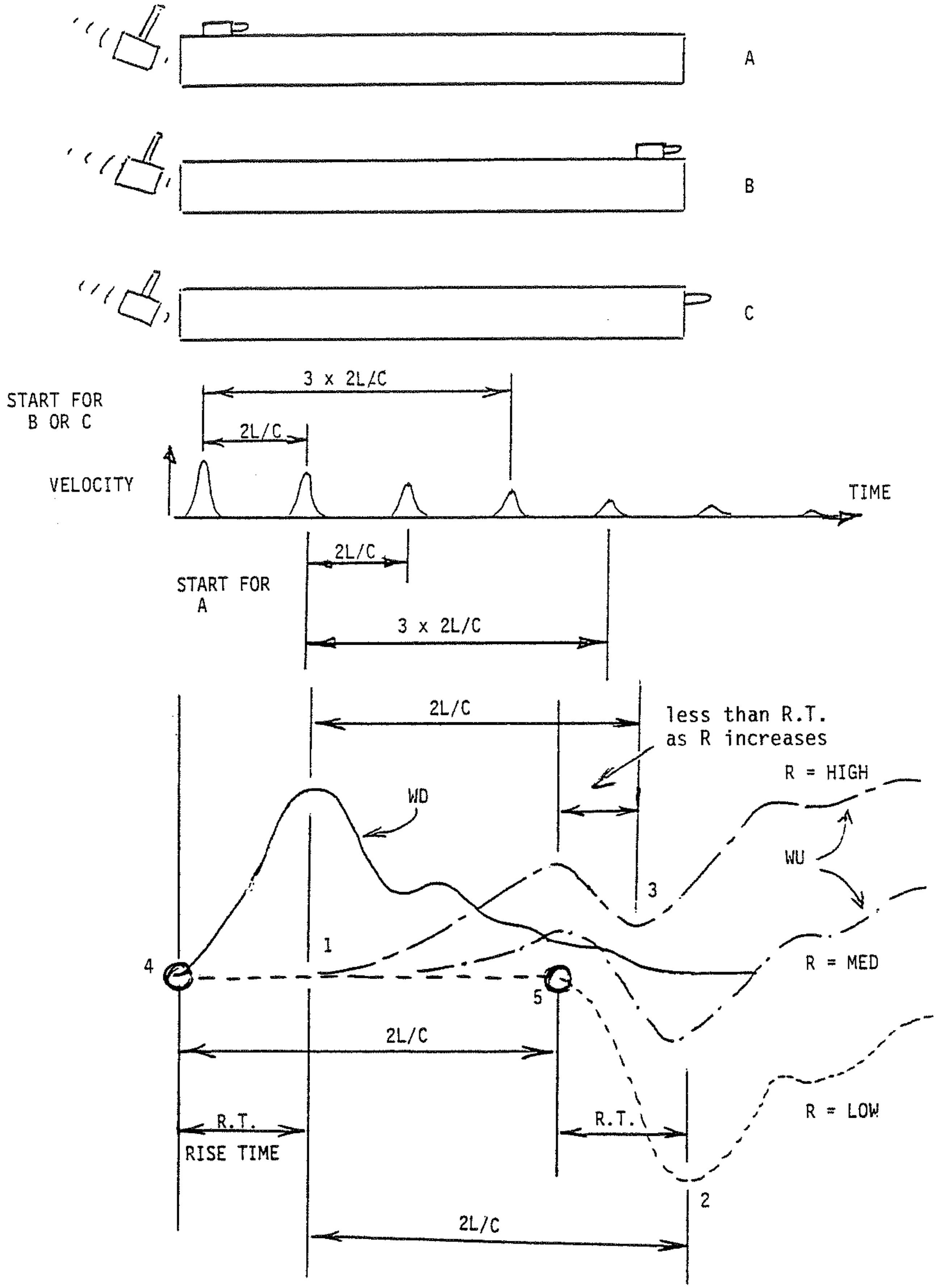
For steel the wave speed is a constant (16,800 ft/sec; 5120 m/sec). However, **for concrete (or timber)** wave speed must be measured as it can be quite variable. The following figure shows two ways to get the wave speed c (WS). The first method is the "free pile test". With the pile laying horizontally, an accelerometer is attached (held by hand pressure is sufficient -- need not be bolted) and the pile struck with a hand held hammer. A "Low G" accelerometer is more sensitive and is preferred to the "High G" type (which will not ordinarily work directly but rather only unless it is tape recorded and played back with high gain).

When attached in the "A" position, the interpretation is made beginning with the second cycle, measuring the total time for one or more cycles, and using the total pile length L (the first cycle should be ignored due to the short length from hammer to accelerometer) to calculate c from 2L/c. The data will probably be improved and the analysis simplified (start at first peak then OK) with the accelerometer in the "B" position. In either the A or B positions, the accelerometer must point away from the hammer and the calibration of the low G accelerometer be assumed to be "1667 g/v" (or higher) so the electronic gain is set to its maximum since the actual signal will be very small. **Our best recommendation is that an "integrity accelerometer" (for our P.I.T. System) can be used in position "C" and should easily trigger the PDA** since its sensitivity is more than 10 times that of the LOW G accelerometer and more than 50 times more sensitive than the HIGH G accelerometer. Also artificially set the calibration to a high value (1000 to 2000 g/v).

The above method will work for timber piles, prestressed concrete piles, and perhaps short reinforced concrete piles. Longer regularly reinforced piles have cracks (due to handling, etc.) which will probably prevent the free pile test from being successful. The second method (see figure) uses the wave down and wave up (WD and WU) as derived from measured F and V), preferably in early easier driving. When the resistance is low, the wave up has a similar shape to the wave down, except it is inverted and delayed by 2L/c (usually true for uncracked prestressed piles but not necessarily for cracked reinforced piles). The wave speed should NOT be taken from the peak WD to the peak (actually valley) WU times (times 1 to 2, and the length below sensors). Because as the resistance increases, the peak (or valley) return time (shown as time 3) will come sooner than time 2 and using a peak-to-peak method will lead to an erroneous (high) wave speed since time 1 to 3 is too short for the true 2 L/c. Fortunately, the "rise-to-rise" method avoids this problem as shown in the figure and therefore **we strongly recommend the "rise-to-rise" method**. The times 4 to 5 as shown are constant and represent the true 2 L/c time regardless of the resistance.

If wave speed is assumed during the test and later determined to be different, corrections can and must be made. A discussion of these corrections is in Appendix 1 to this paper.

"FREE PILE TEST"



MODULUS OF ELASTICITY

The Modulus of Elasticity (E or EM) is used to convert the measured strain to force. The modulus is computed from the wave speed c (WS) and mass density ρ by

$$E = \rho c^2$$

or

$$EM = SP * WS^2/g$$

For steel the specific weight ($SP = \rho g$) is 0.492 k/ft^3 (7.85 T/m^3, 78.5 kN/m^3) and for concrete the typical value is 0.150 k/ft^3 (2.45 T/m^3, 24.5 kN/m^3). **For timber the density must be determined for every pile.** Using the above equation, the modulus for steel can be calculated as 30,000 ksi (2100 T/cm^2, 210,000 MPa) and is for all practical purposes a constant. For concrete or timber, the modulus is calculated (by hand, watch for dimensional analysis units conversions and the value of gravity!) automatically by the PAK with the command "EM0" or ("EMS" for steel). If the EM value (in PAK) does not correspond to the SP and WS as in the above equation, a "*" is shown indicating a potential problem.

Special Note for regularly reinforced concrete piles:

> Since the wave speed is difficult to measure (and perhaps may appear variable due to cracks), generally a predetermined wave speed c(WS) is used for the modulus computation (may vary depending on supplier due to mix design, aggregates, etc.) and a separate speed WC used only for the 2L/c time; WC (includes time delays due to cracks) must always be equal to or lower than WS, the true speed in an uncracked section.

DATA RECORDING

For the PAK, data storage is made directly to hard disk for a user selected blow frequency (SX; if SX = 0, a message warning will be shown when in the ACCEPT mode). All four individual sensor signals are saved independently. For restrikes, we recommend to save every blow. For testing during driving, perhaps saving every fifth blow is adequate. "Permanent" storage is accomplished with the SF command to archive all blows to floppy disk or with the SC command to save specific blows in CAPWAP compatible format (a separate PC DOS program which converts SF (X01) files to SC CAPWAP format is available upon request). In any case, after data is acquired, data MUST be transferred to a PC for permanent storage, BEFORE new data from another pile is collected and overwrites the current data. For restrikes, every blow should be saved; during driving, the number of blows to drive the pile should be estimated and the blow sample frequency chosen accordingly so that only representative blows are actually saved. The advantage of digital recording is that the data collected and saved is exactly that observed in the field rather than filtered (and gain changed and noise added) by the analog recording. Also, those with the CAPWAP capability can also perform this valuable analysis on site.

For the PAK, the user can now easily store his results (like PDA printout) in an electronic (ASCII) file using the SQ command. A new RQ command allows quick review of this data from within the PAK program. The SQ file can then be read by the PC program PDAPLOT which can a) plot the results versus blow number or depth, and b) print the results or averages (and optionally standard deviations, maximum minimum for full ASTM D-4945 compliance) as function of blow number or depth. The PDAPLOT improves reporting to a professional quality result and even speeds report preparation.

Use of SF, SC and SQ are easy and logical. Proper data storage and presentation is improved with minimal effort and (as required by ASTM) is necessary for complete documentation (professional liability) reasons.

With the field data acquisition now covered, let us turn our attention to data interpretation.

DATA INTERPRETATION

STRESSES

We should always be **observe the maximum forces** so that the driving stresses can be determined. The maximum force **FMX is the compression <u>force at the transducers</u>** and needs little explanation; the force could be slightly higher just above a point of high shaft resistance, but this increase would be modest and not likely to cause pile damage compared with pile top damage from local contact stresses or bending stresses due to non perfect alignment. For piles with cross sectional area changes along the length, wave equation studies should be made to find a stress amplification factor.

The input wave is transmitted to the toe and, for piles with little friction, if a stiff end bearing is present, a compression wave will be reflected if this resistance is large relative to the input force. This can potentially result in a doubling of the stress at the pile toe in "fixed condition cases". The PDA can **<u>C</u>alculate the <u>F</u>orce at the <u>B</u>ottom** (CFB) in these cases where blow counts are also likely to be high.

The **tension force is generally of interest for concrete piles** and usually only during easy driving although for large quake soils, high tension stresses can still be a problem even for refusal driving. CTN only considers the maximum net tension returning from the toe (upward tension plus downward compression); CTX also considers the downward tension wave late in the blow and if this is larger is printed as a positive value.

The PAK, since it knows the pile area (AR), can directly calculate the maximum compressive stress (average FMX/AR = CSX; or max of either individual strain, CSI) and the tension stress (CTN/AR = TSX). These quantities (CSX, CSI, TSX) are available as a result for printing and/or saving into a file (done automatically by PAK with the SQ function call) for later plotting by PDAPLOT.

PILE DAMAGE

Pile damage is generally noted by a BTA value less than 100. If the pile is short or the rise time very long, the PDA cannot check for damage and the value 200 is printed to distinguish this condition. If BTA is less than 100, a third time marker is shown the QBTA or "quick beta" function will do always a preliminary scan for damage even if BTA is not selected unless the QBTA is turned off with ALT-F1). If damage is indicated by the PDA, the operator should immediately note its suggested location from the third (middle) time marker, and probably stop the hammer to prevent further possible damage. The first question to ask is if the pile is non-uniform, or if a splice detail is causing a false indication. Earlier blows can be reinvestigated to view if the problem area is getting worse. If the problem is very near the pile bottom, perhaps the indicated length or wave speed is slightly in error; again check earlier blows or pile records. If the pile is a closed end pipe, drop a tape inside to measure the length or a light lowered (if sunshine, reflect from a mirror) will allow visual inspections. Replay the forces and check if bending is severe in the records which could cause a minor disturbance in the data and therefore a false indication.

Remember that the data is only checked for damage between two milliseconds after the peak until one rise time before 2L/c. For short piles, there may not be much of the shaft which remains and is checked. **In reality, the PDA operator should always check the wave up curve himself;** a monotonic increase for 2L/c after the initial rise indicates no damage while a reduction in wave up (relative increase of velocity to force) before 2L/c after the initial rise is likely due to an impedance reduction, crack or gap, particularly if the change is sharp or dramatic.

NOTE:

Sometimes the damage detected by PDA during driving of a pile cannot be seen during the restrike primarily because of a) soft cushion on restrike, b) due to setup and a higher friction, and c) also possibly because cracks in concrete "heal" with time after driving. The high friction essentially prevents motion at the damage location and therefore no reflection waves are generated which would be indicative of the damage.

In order to"see" the damage during restrike, a much sharper impact (less thickness or more stiff cushion; or higher drop if high friction) would be required, however, that may also damage the pile top so care and caution are advised.

It is generally assumed that the restrike tests are not always sufficient to evaluate a pile for integrity, particularly if the soil shows strong setup behavior. For that reasons, it is generally required, for example, in Sweden (where they drive reinforced, jointed concrete piles through clay) that pile integrity tests by the high strain method are conducted at the end of driving. Bearing capacity tests, of course, must be performed after a setup period.

HAMMER PERFORMANCE

Hammer performance should also always be evaluated. **The most important factor is EMX, the maximum energy transferred into the pile.** Comparison of EMX with the potential energy Wh (or, better still, with the manufacturer's rating) is generally all that is necessary. Further analysis is possible by looking at the momentum calculations as given in the PDA manual (MF0 is generally preferred to MW0). If the RAM WT (WR) is input, the maximum velocity of the ram before impact VRI (for ASH hammers only) can be calculated from MFO and can be used to compute the kinetic energy of the ram to investigate if losses are primarily in the hammer or the lower driving assembly. It should be noted that these momentum equations can sometimes lead to unusual answers and, thus, these results should be viewed with caution and for their reasonableness. The force FCP and the stiffness KCP in the hammer cushion can also be computed if both WR and the CAP WT (WH) are input. The hammer cushion stiffness can be further evaluated by plotting the load deflection graph using the program DATPRO. These FCP and KCP values are only correct for, and should be restricted to, air/steam/hydraulic or drop hammers on steel piles. A new output quantity is the time from rise to peak ("TRP") which is a measure of rise time (data samples actually) which in turn depends on cushion stiffness.

Pile Dynamics also offers two other test devices for hammer performance testing. The Hammer Performance Analyzer (HPA) uses radar technology to obtain the ram impact velocity which is particularly useful for air/steam/hydraulic/drop hammers or for SPT energy determination. The Saximeter counts blows, blow rate, and can calculate the stroke for every blow of an open end diesel hammer; results (averages) can be stored in memory for an automatic report presentation (data down loaded to PC in ASCII and printed with Saxprint program).

CAPACITY DETERMINATION

This is usually the biggest challenge facing the engineer, and potentially the most costly if he makes a mistake. However it is also the most rewarding when the PDA can reduce the amount of expensive static testing or determine that the length and therefore the cost of a pile foundation can be significantly reduced.

It is impossible to give guidelines that generally apply to all situations. The PDA operator should carefully read all applicable sections of the PDA manual (especially Chapter 1, the section on J selection, and Appendix A).

All PDA Case Method capacity results assume that the pile is uniform (cross section and modulus versus length). If the pile is not uniform, the PDA capacity result may not be reliable. The CAPWAP program, which can accurately model non-uniform piles, should be considered in these non-uniform pile cases.

It should also be emphasized that **the 2L/c time period and wave speed must be correctly determined** and entered, or else very large errors in some cases (particularly those with clearly distinguishable reflections at 2L/c) may result; the rise to rise method is required (see section on wave speed determination). The wave speed not only influences the 2L/c, but also is contained in the calculation of modulus of elasticity which is used in converting strain to force; a 10% error in wave speed results in a 20% error in modulus and/or force, emphasizing the need for accurate determination of wave speed. The modulus is also linearly dependent upon the pile material density. Although density is reasonably uniform for concrete and steel, the density of timber piles can widely vary and <u>must</u> be measured for <u>each</u> pile to assure accuracy.

A "dynamic formula" method QULT (QULT = 2*EMX/(DMX + SET) can be calculated by hand which uses the EMX and DMX measured values in a "modified Hiley formula with measurements", (where SET is calculated from the blow count as the permanent penetration per blow.) Although we do not recommend these formulas as a general rule, this one at least contains some actual measured values, and may indicate the potential benefit to be gained at the very high blow counts. RLT is the limiting value of QULT for zero set per blow (note that RLT is not the same as RTL which is RSP with a J of zero). We do not recommend the reliance upon this result but offer it only for reference.

It should again be noted that the following Case Method equations assume the pile to be uniform. For non-uniform piles, capacity analysis should be made using CAPWAP which can model all non-uniformities.

The RSP methods (RS1 for peak 1, RS2 for peak 2, and RSM for peak max) are the original methods for which the empirical damping factors based on soil type were determined (implies that the soil is properly identified, preferably by grain size analysis, and that the soil at the boring is similar to that at the pile location). The original study included data primarily from restrike (or end of driving in sands) where the blow counts were at least moderate. Unfortunately, for very low blow counts, this method is also very sensitive and small changes in J can result in large changes in capacity. Also, for large quake soils or high blow counts (small set per blow), the full toe resistance may not be fully active at 2L/c after the first peak; a small delay is often useful for this and for concrete piles, which often have 2 peaks (due to the non-uniform compression of the pile cushion), selection of the second peak almost always leads to a better solution. Viewing the RT-RS curves and adjusting J until a smooth (preferably flat) curve is obtained also may help; however as the shaft friction increases, this technique becomes less reliable. As a small note on shaft friction, the SFT computation makes no allowance for damping.

The RMX method searches over time for the maximum resistance during the entire blow and thus overcomes some of the limitations of the RSP methods for large blow count, small blow count, or high quake situations. Although the temptation exists, do not use damping factors less than 0.4 with this RMX method without substantial proof from CAPWAP or a static test that a lower damping factor correlates well. Higher damping factors may sometimes be necessary, although logic (instinct) implies that J factors above 0.8 are the result of unusual soil conditions (recommending a static test is a good idea). For friction piles in clay (where high J factors are normally appropriate), the full resistance should be active during the first 2L/c cycle anyway (RSP = RMX); in the case of high shaft friction in clay and high end bearing, the friction may only be available after a setup period and seen during restrike, while the end bearing is easiest determined from the end of driving. If and only if the restrike blow count is very high, a summation of these two cases (by CAPWAP solutions) may in some instances be possible and give a better indication of the total capacity available during service conditions. However, great care should be exercised since this addition could also lead to overprediction particularly in relaxation cases; a confirming static test is therefore advised. At low to moderate blow counts (set greater than 3 mm/blow), this superposition of results is definitely not advisable nor necessary as the full resistance has already been mobilized.

For uniform piles with very little skin friction, the RAU method is (theoretically at least) the perfect method as all theories are correct and the method is independent of a damping constant. It makes no difference if it is easy or refusal driving; the key is that the force and velocity must be proportional for the entire first 2L/c (implies good data) and the 2 L/c must be correctly chosen.

The newest method, RA2, has shown considerable promise in determining the ultimate load even for piles with little to moderate shaft friction and this method also does not require the selection of a damping factor. Results are generally in good (not necessarily great) agreement with results from CAPWAP and therefore the method deserves at least a casual consideration on every project and often has proven to be the best method available; if RA2 differs from the results of the damping factor methods, then further investigation is clearly warranted. If the pile is driving through a layered soil, this method has the additional advantage that the damping factor does not need adjustment. Again, 2L/c must be chosen correctly.

For longer piles with very high friction, such that the velocity goes negative prior to 2L/c, most all methods underestimate capacity and the unloading method RSU may be beneficial. However, CAPWAP (or static test) should be performed as soon as possible to verify the correct procedures.

On practically all of our projects, we perform at least one CAPWAP to confirm our field methods; for larger projects, we generally perform CAPWAP for about 20 to 40 percent of tested piles, although usually only for the data at the end of drive and/or begin of restrike. **We always recommend some CAPWAP analysis for confirmation.** Unit friction values and the determination of friction versus end bearing, and analysis of both EOD and BOR cases often result in better recommendations regarding the total capacity, optimum driving criteria or pile length. Further discussion of the great benefits of CAPWAP are beyond the intended scope of this paper except to say that this confirmation by an independent analysis can often detect unusual conditions not noted simply by PDA testing and is therefore usually well worth the investment. With the PAK PDA it is now rare for me to not perform at least a preliminary CAPWAP analysis on the field site. In this way, major surprises after finishing the field work are kept to a minimal occurrence.

If the static load test is not run to failure, then only a lower bound solution can be given; similarly if the blow count is very high (small set per blow, typically less than 3mm/blow or greater than 10 blows per inch) then the dynamic test's ultimate capacity may be low. Unfortunately, it is currently not possible to determine just how low (if any) the capacity may be at these small sets per blow (depends greatly on soil type and end bearing percentage). Ideally a larger hammer, higher drop height, or less cushion could be used to increase energy transferred and hence produce a higher set per blow and thereby mobilize more capacity.

It should be obvious because of soil strength changes as a function of time, that the calculated capacities correspond to the time of testing (*i.e.* EOD or BOR). **Restrike tests are therefore considered very important**, especially if good correlation is to be achieved between the dynamic test and a static load test. The optimum waiting period for the restrike varies depending on the pile type and soil type; some report that testing reveals that capacities are still increasing after more than a year, while in other soils no observable setup is ever noticed. Knowledge of the soil type, whether the soil is saturated, and local experience can be used as guides for how long the period before restrike tests should be performed. As a general rule, a longer wait time before restrike is better. In many cases recommendations of a seven day wait (similar to static test recommendations) result in near full setup. Setup effects often are linear with log time.

Although less common (fortunately), **some soils exhibit relaxation or a reduction of capacity** as a function of time. Some suspected cases are in reality due to hammer performance being poor at the end of driving causing relatively high blow counts, and performing much better during restrike or redrive resulting in relatively lower blow counts; the PDA can easily identify these cases by looking at the hammer performance indicator EMX.

There are cases where relaxation is very real and the PDA User should be very aware of this potential since loss of capacity can result in foundation failure and the ensuing legal implications. Three cases of real relaxation quickly come to mind. First is the case of piles driven into some shale bedrock. If the shale is weathered, the potential is generally greater for relaxation. These shales have been widely identified in many parts of North America and probably also exist in other parts of the world; losses in capacity of up to half the end of drive capacity have been observed! (There goes the typical safety factor used in the USA!) The second case is for displacement piles driven into very dense soils (often silts) with little shaft friction; the high end bearing at the end of driving reduces with time, the soil beneath the toe springs back since the friction is unable to keep the toe in a compressed condition. In a third situation, negative pore water pressure during driving would cause artificially high effective stresses and end bearing. Reductions may also be due to pore pressure changes. In any case, one day restrikes are often not sufficient, especially in shale where the relaxation process often takes a week or more to fully develop. Some later restrike test (with or without PDA testing) should be performed with a very careful measurement of the set for the first few blows which can then be compared with the end of drive blow count with the PDA; **look for the capacity of the <u>first</u> high energy blow.** Capacity often increases rapidly blow by blow and if a "later blow" is chosen, the capacity may overpredict the true static condition. Therefore, (to repeat for emphasis), choose the earliest blow with reasonable energy to produce an acceptably large set per blow.

It is also because of this relaxation potential that adding EOD end bearing to restrike friction (without confirming by static tests) is <u>not always a good idea</u>. It is only in cases of extremely high blow count restrikes that this need be considered since at 2mm set per blow (less than 150 blow/ft) and certainly above 3 mm set per blow (less than 100 blow/ft) the full capacity of the pile is likely to be mobilized.

In cases where the pile is driven to high blow count with significant end bearing and then large additional shaft friction from setup occurs, it may be beneficial to stop one pile just above the bearing layer (low tip resistance) such that on restrike the full friction effect may still be mobilized during restrike.

From the preceding discussion it should be apparent that capacity determination is often a complex problem with many features contributing to the success (or failure) of a good result. Obviously good quality measurements are the basic requirement; if the data is no good, then any analysis of the data is suspect.

Soils are often difficult materials to deal with. The wide variety and variability of soils encountered both horizontally across the site or even vertically along the pile shaft definitely complicates the issues. Clean coarse grained sands are generally well suited to dynamic capacity analysis, often even at the end of driving, since capacity changes with time are usually minimal; however, the end bearing of larger diameter displacement piles may be underpredicted at higher blow counts (which is the largest real problem area for piles driven in clean sands; another problem is change in water table and effective stresses). Similarly, testing of piles driven to a hard competent rock is often quite straightforward.

For friction piles driven in clay, the effects of soil remoulding and pore pressure effects require that restrike tests be performed. The wait period before restrike is dependent upon the soil porosity, etc. but to include the full setup effect often requires several days to even weeks to obtain a good correlation with the long term capacity of the pile soil system. It should be obvious that a one day restrike is not a sufficient wait period for many soils. Comparison of end of drive dynamic tests with static tests performed weeks later will undoubtedly produce disappointing results. These time dependent soil strength changes should be clearly explained and discussed with the client. **The PDA indicates only the <u>mobilized</u> capacity <u>at the time of testing.</u>** Many engineers fail to understand this and compare end of drive PDA results with static tests run two weeks later, with obviously poor correlation. They then improperly blame the PDA when it is really due to the soil changing strength. This is probably the single most common problem we hear.

In between clean sands and pure clays are a wide range of other soils. In the majority of cases, **good** capacity results can usually be obtained by following two simple rules.

1) **Make sure resistance is fully activated.** The hammer energy should be sufficient to produce a pile set per blow greater than about 3 mm/blow (less than 10 blows per inch).

2) **Restrike the pile after a wait period** appropriate for the soils on site. As the soil pressures equalize around the pile (larger hole during driving due to lateral motions or over size pile shoe), as pore pressures decrease, as the soil structural changes after being remoulded by the driving process, and other strength changes (setup increases or relaxation decreases) are best evaluated by restrike testing.

It is only in a small percentage of cases (estimated at less than 5 percent of all sites) that capacity determination is unsuccessful if the dynamic test is performed on restrike (after appropriate wait) and the blow count is reasonable. **CAPWAP confirmation is always recommended** as it better determines soil behavior and identifies unusual soil conditions; when conditionals are unusual, static tests should be recommended.

One suggestion to reduce testing costs is to use a "sister pile" approach. First, have several piles installed before arriving on site; this also assures that the contractor has worked through problems which often occur in setting up his pile driving hammer at the beginning of a project rather than have you needlessly wait around. Then, a few days into the project (or after a weekend), you can test a couple piles during driving and several piles previously installed all in the same day. This gives the maximum information with the minimum testing time for small budget projects. Of course, on many projects the specifications require testing during driving of all test piles, often for hammer performance or driving stress information.

The PDA is obviously a very powerful analysis tool when properly applied. Organizations with PDA's should make every attempt to make measurements on all of their in-house pile projects as it will detect most common problems. **Projects without dynamic testing rely on**

numerous assumptions and therefore generally attract legal problems and litigation. It is much better to avoid court battles through proper documentation (PDA measurements) and solving problems early in the project. On larger projects, periodic testing during production will identify site variations and hammer consistency or identify malfunctions; the recommended testing frequency is generally at least one day every two weeks. Testing relatively early in the project avoids having to perform major corrective action on a large number of already driven production piles which is by far more expensive than the cost of the dynamic testing.

REVIEW OF CAPACITY DETERMINATION

It should be noted that the dynamic testing estimates for the pile capacity 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.

Static pile capacity from dynamic method calculations provides an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **dynamic testing during <u>restrike tests</u> usually yield a better indication of long term pile capacity than a test at the end of pile driving**. The capacity of a pile at the time of driving may often be less than the long term pile capacity particularly for piles driven in fine grained soils (clays, silts and even fine sands). During pile driving, excess positive pore pressures are often generated. 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.

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. 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. Again, restrike tests should be used, with great emphasis on early blows; often a wait period of one or two days is satisfactory but depends on the permeability of the soil.

Numerous other factors are usually considered in pile foundation design. Some of these considerations include additional pile loading from downdrag or negative skin friction, soil setup and relaxation effects, cyclic loading performance, lateral and uplift loading requirements, effective stress changes (due to changes in water table, excavations, fills or other changes in overburden), settlement from underlying weaker layers and pile group effects. These factors have not been evaluated by the PDA and have not been considered in the interpretation of the dynamic testing results. **The foundation designer should determine if any of these considerations are applicable to his project and the foundation design.**

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions.

STATIC TESTS

Sometimes we are informed that our correlation of dynamic tests with a static test is bad. Many people then simply assume that the dynamic test is in error because the static test must be right. I would like to argue this point as I have observed many problems with many different static tests. Many tests are performed by people who only occasionally do static tests when required (and they also are often less than enthusiastic). Often the data collection is performed by technicians, particularly if it is a maintained load test, who may not understand "why" or "how do I" or "what do I do now" and therefore does not realize when something is wrong.

As another example, many tests are still performed without a good, linear, **recently calibrated** load cell. This can result in serious errors, perhaps even an over estimation of the load by 30 percent due to internal pressure in the jack (this does NOT lead to conservative errors). I recall a project where our prediction was very low (compared with loads determined by reading the jack pressure). Further investigation showed that the last time the jack had been calibrated was in 1957! Fortunately, the engineer in charge insisted the jack be recalibrated and the new calibration confirmed then our original dynamic test result. Reading jack pressures only can overestimate the true capacity, particularly if spherical heads are not used to account for non axial loadings which then puts extra pressure on the piston seal. As another point to consider, how accurate do you think the jack pressure is?

I have also observed errors in displacement measurements. Reference beams should be anchored far from the test piles and reaction piles since any ground movement will distort the signal; unfortunately, long reference beams are more susceptible to other problems like temperature effects for example. I would hope that the engineer looks at the stiffness of the static test, since sometimes errors can be detected, particularly where multiple tests are run on multiple piles at the same site.

I recently read a case where the method of reaction caused a great difference in load. On the same pile, a load test using a dead weight gave about twice the load held by the same pile using reaction piles. The offered explanation was changed effective stresses in the soil around the test pile due to the reaction piles. Incidently, if effective stresses change (due to change in overburden or water table) then the pile capacity will also change.

Another concern is in failure load evaluation. Have you ever heard that we loaded the pile to "xxx tons" which seems awfully high to you, and then find that the xxx tons was at a displacement of 6 inches (150 mm)! Such displacements are beyond the serviceability limits of most structures. There are numerous methods for evaluating failure loads: slope methods, stiffness methods, ultimate applied load, some percent of the shaft diameter, differential settlement methods, etc. If the test is essentially linear and then quickly plunges, all methods will give about the same answer. But if the curve only gently rolls over, taking continuously more load at increasing displacement, then a wide range of answers will be result; what is the likelihood of any one answer being absolutely correct?

We have generally tried to use Davisson's method of evaluation since it can be easily described mathematically (although what modulus do you use for concrete as it depends on rate of strain; also watch out for when a steel pipe is later filled with concrete to get the right overall stiffness). The Davisson method is not intended for uplift evaluations. It is also my understanding that this method was intended to be applicable to "quick test loadings" (such as CRP or Texas Quick loading), and further that it gives a load which should not cause excessive creep loading or other serviceability problems at the design or working load level. I should emphasize that the Davisson limit is NOT programmed into the PDA or CAPWAP, but we have only observed that correlations are often better if the static test is evaluated at a reasonable displacement such as is obtained by the Davisson method. A little bit of judgement by the engineer will also be of value. It should further be understood that the dynamic test result, although given as a single value (capacity is "xxx tons"), also has a few considerations (time dependency, reasonable set....) and perhaps a tolerance or "confidence band" of 5 to 10 percent of the impact force is a realistic expectation for accuracy. To blindly compare one number from any dynamic test with another one number from a static test, without reviewing all test information, test details, sequence of installation and all data available, may lead to disappointing results. It is further occasionally encountered where the soil conditions are altered due to drilling or jetting or driving other piles (particularly if the later piles are at a close spacing and extend below the toe of the test pile.

SPECIAL APPLICATIONS

Most piles which are tested are one material and uniform area (such as H piles, steel piles, or prismatic concrete) along their entire length. In these cases, attachment of transducers directly to the pile and the subsequent analysis is rather straightforward (and described in the manual and/or this paper).

There are some situations where the pile is not completely uniform, but testing is still desirable. The first such case is perhaps a **concrete filled steel pipe.** This is maybe uniform along the length but made of two materials. Should we attach the transducers to the steel or cut "windows" in the steel and attach to the concrete?

I shall try to explain what we do and <u>why</u> we do it. The basic premise is that the strain in concrete and strain in steel are equal. If this is not true, then it is really not possible to do any reasonable analysis with confidence. If the pile is properly constructed with concrete and steel being flush and hit uniformly then all should be good (provided concrete quality is reasonable).

If the concrete is under filled with respect to top of steel, a uniform stress is unlikely and probably the bond between steel and concrete will be broken. <u>In this case,</u> we recommend cutting off the excess steel at the top and then hitting the concrete, since the impedance of concrete is usually several times time impedance of the steel (and we generally prefer to hit the concrete through a pile cushion). We then compute an <u>[average SP</u> $= \{(SP*AR)_{steel} + (SP*AR)_{con}\}/AR_{total}]$ and then using a wave speed WS determined from the PDA test (look for toe reflection from rise to rise method) we calculate the modulus $EM_{avg} = SP_{avg} * WS^2_{avg}$ and enter SP, WS and EM as above and the <u>total ARea</u> (concrete plus steel).

If the impedance of the steel is very low (at least an order of magnitude less) compared with the impedance of the concrete, you could remove a "window" in the steel large enough for transducers to be attached to the concrete. As a word of caution, the concrete should be filled to the top of the pipe; hit the concrete through a wood pile top cushion (thin or compressed for restrike tests). Compressing the concrete increases the bond between steel and concrete due to Poisson's effects; hitting the steel first decreases the bond.

If the steel impedance is relatively large, then cutting "windows" could make a stress concentration in the concrete which is not desirable, making the concrete strain too large. Remember we are <u>measuring strain</u>. If the piles are manufactured under good quality control, it may then be better to attach to steel and not cut windows. If proportionality is then good, the bond is probably OK, and data acceptable.

A second special application is a **pile which is non-uniform with length**. This could be a steel pipe of variable wall thickness sections, a composite pile (a concrete section with H pile projection at the bottom), a steel follower driving a concrete pile, or a tapered section (monotube or timber pile).

As a general rule, transducers should be placed some distance away from sudden cross section changes. Stress path changes, "end effects", or two dimensional (out of plane) strain adversely affect the data near the change. For the non-uniform pipe pile, placing the transducers at least one half diameter below the change is preferred, although this is not always possible. For steel pipe piles (particularly spiral welded pipes) place the strain transducers as far from the welds as is practically possible.

Attaching transducers to a steel follower when driving or testing concrete piles has several advantages. Because the steel modulus is known, the strain to force conversion is better defined. Further, the follower has to be prepared only once and then numerous piles can be tested without individually attaching transducers to each pile. A few simple precautions are necessary for a successful test. 1) The impedance (EA/c of the follower should be as close as possible to the impedance of the pile. 2) Do not put transducers very close to bottom of follower; leave at least a half diameter distance. Also stay away from other stress concentrations such as stiffeners. **3) The location of the pile cushion is <u>extremely</u> important.** The amount of cushion material between the follower and pile should be kept to an absolute minimum; the main cushion normally used at the pile top should instead be placed between the hammer and the top of the follower.

For this follower/concrete test, or other composite or tapered piles, there is a question of how to analyze the data. Since the Case Method is not intended for non-uniform piles, it is probably <u>best if one assumes the pile is uniform</u> (EM, WS, SP values at transducer location apply to entire pile) and then analyze the data later by CAPWAP (the entire first element in CAPWAP <u>must</u> also be uniform so either locate transducers at least one meter above non-uniformity, or increase the number of pile segments in CAPWAP, or simply tell CAPWAP that the length to change is at least one meter). The maximum force and energy will be correct, capacity by the Case Method in the PAK will be unreliable but should be made by CAPWAP anyway.

SUGGESTIONS FOR A SUCCESSFUL TEST

Many factors enter into a successful test. **Good communication and cooperation** between all parties (engineer, contractor, owner, and PDA test engineer) is the foundation. By appropriate advance planning, a good test program can be "designed" which should answer the main questions. Depending on project size, soil type and site variability, a test could be performed in one site visit or may need several days and second visits for restrike tests. Ideally sufficient information will be gathered in this process but the effort must be balanced so that the contractor is not overly burdened and the owner's cost is kept reasonable. Further good communication is essential if the PDA engineer is to locate and arrive at the test site at the proper time.

It has been observed that often <u>those who succeed are those who want to succeed</u>. Some have used the expression "technology champion" for this engineer. If the engineer is enthusiastic and has the desire, he will usually find a way to make it work; he will apply the methods in the correct way and interpret results consistently and with reason (making effort to account for capacity changes with time or assuring blow counts are less than refusal). If the engineer is reluctant and only performs the test because someone else ordered him to do it, then chances of a good result are then considerably diminished. It boils down to **having a good attitude**.

Prior to going to the job, the engineer should inventory his equipment. If a transducer is found to give unreliable data, it should be repaired or replaced. (If you have bad data from a transducer on the previous job, at the end of that project do something about it right then since chances are it will not work any better the next time). I recall about 15 years ago when on a site, I was experiencing transducer problems and was told by my client to "keep smiling" when the owner came around; after the job was finished, however, I still had to write the report. With bad data, the writing of any report is difficult. Remember that if the data is bad, the test is worthless.

The engineer must make every effort to arrive at the job site with <u>everything</u> he needs (PDA, cables, transducers, tools, bolts, safety equipment ....). He may rely on the contractor to supply some items such as a generator, but communication is then very important.

The equipment should be assembled and checked early on arrival at the site; if a problem is found or something is missing, you at least then have a chance to find replacements. Attach the transducers to the PAK and verify their performance through tapping them. **If the transducers don't work on the ground, then chances are they won't work attached to the pile either.** This has saved much embarrassment and hours of contractor time. I can assure you that he does not appreciate you sorting out your "problems" while on his time clock. Quality data should be our primary goal.

Prepare the pile in advance. This means drill the pile on the ground and consider your safety while doing this! Recently one of the GRL engineers was asked to stand directly under and drill a large concrete pile while it was lifted only with (no cribbage to support pile) a crane! Think what would happen if the brakes on the crane failed and crushed the engineer! Fortunately, our engineer properly refused. Lifting and setting the pile in position is perhaps the most dangerous activity on the piling site; I have lost track of how many injuries (and even deaths)

I have heard about. Stay clear of objects which can potentially fall on you. I recall a massive piece of steel breaking off a hammer and landing exactly where I had been standing only seconds before. Our engineers have had several close encounters with headache balls that have fallen (and in some cases smashed our equipment cases or briefcase). A little common sense will probably allow you to return home injury free at the end of the day.

We are also occasionally asked to drill the pile after it has been lifted into place and the pile top is then say 50 ft (15 m) up in the air. We now refuse to "drill in the leads". The contractor can drill it himself. We suggest, just waiting and test the next pile which can then be properly prepared on the ground (or use a proper lift basket if it must be tested). Chances are that if you think something is dangerous, OSHA would likely consider it a violation (e.g. it will cost you money if you are caught) and could result in your death. Safety is your first priority. THINK at all times.

We also suggest that you show a member of the piling crew how to bolt your transducers to the pile and let him do all the climbing.

Make sure you get as much site information as possible. What is the hammer? Hammer cushion? Pile top cushion for concrete piles? Obtain copies of appropriate soil borings and site plan. Get copies of the driving logs (blow count records). Collect load test data if available. It is much easier to get all this information before you leave the job site.

One of the most important site parameters is obtaining the set per blow, often counted as "blows per ft, blows per inch, or blows per meter,....set per 10 blows,..." This is perhaps your assurance that the capacity is fully mobilized. It is also a value used as a guide in the CAPWAP matching process. For restrikes, draw a reference line on the pile, drive the pile 5, 10 or 20 blows, draw a second line using the same reference, and finally measure the total permanent pile movement. For restrikes, do not simply get the blow count for the entire first foot of restrike; try to get blows per inch for each inch or total set for a limited number of blows. Assess the accuracy of the set per blow. Is the blow count changing at the end of drive? If a restrike, is the first blow with low energy? or is the capacity changing quickly either increasing or decreasing? How reliable was the displacement measured? How accurately were the "inches" drawn (or were they "guesstimated" as the pile was being driven)?

Measure the pile yourself (length and cross section area) and see if that agrees with the section you are told as the force (and capacity and energy ...) are all directly proportional to the pile cross sectional area. Determine the length below gages accurately as this is essential information also. For concrete or timber piles, determine the wave speed. For timber piles, get the density of each pile tested by actual weight/volume measurements. Calculate the elastic modulus (or have PAK do it for you). Enter all information or PDA settings.

Be careful that data storage is sufficient. Save every blow on restrike, or say every 5th blow during driving. For PAK use the SX command to set this frequency. For the PAK, we strongly also recommend that you save the results in the SQ file; This will allow you to summarize the field data quickly and in a professional output format for your report on the project.

Make sure you get all the PDA data needed (end of drives and restrikes). If possible (and if you have CAPWAP) do at least a preliminary analysis of some data to assure yourself there are no "surprises" later. You should always have a reasonable number of CAPWAP analysis for every job site; we would suggest that as a rule-of-thumb that there be about 2 to 5 CAPWAPs for every 10 piles tested. CAPWAPs should be for important data sets such as restrikes or end of drive, after a splice, or potentially possible bearing layers.

Ideally, everyone should have a good idea of how to proceed before you leave the site. Again, good communication is the key here. Special precautions or instructions for production piles must be clearly stated.

The report should be written as soon as possible. The ASTM D4945 specification lists many items which should be included in this report. Site details are often described for the permanent record. All CAPWAP or PDA results should be clearly and concisely presented.

RECENT PDA PROGRAM FEATURES

If you have an older program version, there are many improvements available to you with the newer versions. It is the responsibility of the PDA User to request these program upgrades from PDI and install the new PC program to keep current.

Please also submit additional suggestions (changes, improvements, ***etc.***) to Pile Dynamics for our consideration; many features (improvements) have resulted from your valued suggestions. Changing the program is as simple as reading a new disk. Check your program version. If your version is older than one year, please contact us so that we may update your PDA. For the PAK, new programs come on floppy disk and are easily installed.

The PDA has been equipped with several features which make automated data processing possible. Digital storage and plotting capabilities (especially the PDAPLOT program to plot results and print summaries) have made life easier. The serious user should read the manuals (or other available material such as Users Day notes) and adapt this technology into his practice.

APPENDIX 1

CONVERSIONS AND CALCULATIONS FOR PDA CALIBRATION FACTORS

I have often received frantic calls from users who have discovered much to their horror that the input they had used on site was in error and then ask what can possibly be done to salvage their test. The signal conditioning is essentially identical for both the blue box PDA models (GA, GB, GC, GCX, GCS) and the newer models PAK/GCPC, so the answers apply equally to both. The blue PDA units, of course, have front panel input dials (potentiometer - "pots") which set the calibrations (A1, A2, F1, F2) and the pile properties (EA/C and L/C); the PAK/GCPC makes all input through the keyboard. The GCPC transducer calibration inputs then electronically adjust internal potentiometer (located on the 8DA31 board) to the same settings as the blue PDA and both are based on the following formulas for data input:

setting = (accelerometer calibration) x 0.6 Equation 1

or

setting = (strain trans calibration) x (wave speed) x (constant) Equation 2

In fact, the GCPC displays these setting values for interested users; the values shown should be less than "998" indicating that valid calibrations were given.

It should also be apparent from the above equations that only the transducer calibrations and the wave speed affect the recorded signals. If errors were made in the field for pile length, modulus, area or other inputs, the data is not affected; replay of this type data (area, length, *etc.*) with the new value is all that is required to obtain the new corrected result.

The blue PDA user should record the A1, A2, F1, and F2 dial settings and really should not change them during the test. The PAK/GCPC user will have all his data automatically stored with the blow; these inputs can be recalled at any time by the RI command. The calibrations can be inspected in the CAPWAP data file (SC command).

The output from the signal conditioning is primarily a velocity signal (obtained from the direct electronic integral of acceleration) and a signal proportional to strain (converted to velocity scale by wave speed in the above equations). This "strain" signal is later converted (numerically for all computations) to force by multiplying by EA/c (note the wave speed in Equation 2 above cancels the c in EA/c - in effect the original strain is multiplied by EA to get the force).

Both of these analog data curves are in a velocity scale equal to 1.0 m/sec/volt (3.28 ft/sec/volt) and are digitized by the PDA such that the integer value 8192 (2^{13}) equals 10 volts or 10 m/sec (32.8 ft/sec). These values are affectionately known at Pile Dynamics as "funny numbers" and are, in fact, the numbers used internally in all PDA computations (kept in PDA memory). The purpose for the funny numbers is to increase the speed of computations and compress the storage size of the data. In the older digital output format (16I5) each value has this (8192 =

10v) interpretation; in the newer "difference format" the first value in each line is calibrated such that 1024 (8 times smaller value) equals 10 volts and the following ten numbers on each line are differences to be added to the previous value to obtain the new value at that point. Of course, these values are transparent to the user who is running PDI/GRL software as the programs naturally convert the data to scaled engineering units for all analyses (*i.e.*, CAPWAP). Only when the user ventures off on his own research interests is this value conversion necessary.

The PAK/GCPC user has one additional input which can affect his data. If he inputs a different area and/or modulus at the gage/sensor/transducer location (AG, EG) than at the pile top (AR, EM), then the PAK/GCPC will compute a force multiplication factor (FM) which will be applied to the data (*i.e.*, funny numbers). The user can, of course, override this value and input any FM (and also VM factor) he so chooses. It is suggested that the user not adjust these values without cause since the easiest CAPWAP analysis will result (and the least confusion generated) when both FM and VM are 1.0. The FM and VM values used are given in the SC CAPWAP file header.

The user should input the proper transducer calibrations. If an error is made and then later detected, however, he may replay the data multiplying by a correction factor of

correction factor = (correct calibration) / (field calibration) Equation 3

In the case of Equation 2, the strain/force calibration is also dependent upon the assumed wave speed c (WS in PAK/GCPC). If the wave speed was later determined to be different, the following equation would apply:

correction factor = (correct wave speed) / (original assumed wave speed) Equation 4

and it should be further noted that both correction factors from Equations 3 and 4 may apply and the result factor would simply then be the product of the two factors.

This correction factor is also in addition to the tape recorder gain factor. (Obviously, one should also know the tape recorders calibration factors; in case they are not known, the best one could then do is to adjust the replay gains until the calculated results (FMX, EMX, and VMX) approximately match the field printout. For example, if the recorder was at 2.5 volt full scale (output scale 1 volt), and the Equation 3 factor was 1.10, then the replay gain would be (2.5 x 1.1 =) 2.75 and would be input times 100 (2.75 x 100 = 275) into either the "F" or "V/A" input dial for the blue boxes, or directly as 275 into the appropriate channel for the PAK/GCPC external input. For data already saved directly by the PAK/GCPC, the replay factors can be individually adjusted (*i.e.*, "RF1.10" would set the channel 2 force calibration temporarily to 1.10 times higher for all computations for this blow. The RU command will then update this new replay calibration automatically for all subsequent blows in the same data set.

Changes of WS **during** a PAK/GCPC test are not a particularly good idea. We would suggest that the wave speed be determined either (a) in advance of the test with free pile solutions, or (b) by inspecting the first few blows during driving. For the latter test, ask the contractor crew to apply say five blows (using an assumed WS value) and then stop. Inspect the curves and then adjust WS as appropriate before continuing the test. Leave the WS at the new fixed value adjusting WC as required, if the wave speed gets slower during the test due to cracking causing the toe reflection to be delayed. The wave speed should never increase (WC should always be less than WS). Remember the WS value affects the strain curve calibration/magnitude, so find the wave speed and stick with it. Blue PDA users need to record on paper everything they do (calibrations, dial settings, and wave speed) as there is no such feature to automatically record this information.

The final question is "is there anything which is completely irreversible?" In general, as long as you record on the tape recorder (watch for the PAUSE button!) and do not clip the signals (use too low a full scale setting for the data), or fail to record the result digitally (blue PDA print selector switch or with the SX command for PAK/GCPC), then you can probably salvage the data as indicated above. Of course, if you have a digital record, you can adjust the calibrations in either the PAK/GCPC using the replay factors RV1, RV2, RF1 and RF2) or in CAPWAP (FCal and VCal). It is then helpful if you know what adjustment is necessary and the best place to begin is with the field documentation of calibrations and wave speed as described above.

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