# Deep Foundations Integrity Testing: Techniques & Case Histories

*Integrity testing can identify defects and serve as a means to evaluate and modify foundation design and construction.*

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Deep foundations integrity testing is employed to assess the soundness of in-place constructed elements. The increased use of these members in the New England area over the past 20 years has resulted in an increased demand for quality control testing. Integrity testing is the process by which the soundness of the inspected object can be determined. Integrity testing of deep foundations has become common due to the combination of construction requirements and technological advances. Growth in the use of in-place constructed foundations (*e.g.*, drilled shafts), along with higher design loads and a more litigious legal climate, have spurred the need for integrity testing. Advances in the areas of instrumentation, data acquisition and signal processing accompanied the increased power of personal computers. These advances enhanced the capabilities and reduced the cost of developing methods for the integrity evaluation of foundations.

Deep foundations integrity testing mostly applies to foundations constructed from concrete/grout — such as drilled shafts, drilled mini-piles, pressure-injected footings and precast concrete piles. Testing is required for quality control during construction to detect flaws in the pile (*e.g.*, necking, cracking, voids, poor quality material, etc.). Such defects are applicable to cast-in-place (or injected in-place) concrete piles and, to a lesser extent, to precast concrete piles. In some cases, the foundation length must be determined. Integrity testing can be performed on any deep foundation type (including timber and steel piles) with some methods capable of determining foundation length even when the foundation is not directly accessible (*e.g.*, structure/cap coverage of the pile's top).[1]

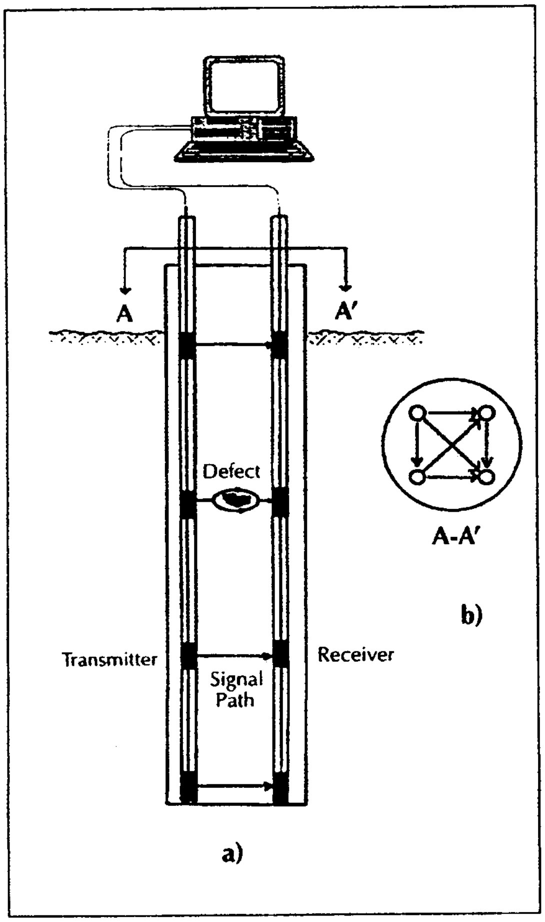


**FIGURE 1. A typical crosshole sonic logging (CSL) test set-up showing a) the transmitter and receiver placed at different depths and b) a plan view of the CSL tubes noting possible test configurations.**

Determining the integrity of a material can be accomplished by either intrusive or non-intrusive methods. Intrusive methods are more conventional and include drilling, coring or penetration via preinstalled conduits. These methods can include destructive testing (*e.g.*, on core samples), which provides direct information about the condition of the structure under consideration. However, the use of intrusive methods may compromise structural integrity once testing is completed. Non-intrusive testing can provide information about the condition of the structure without altering its structural integrity. Integrity testing by non-intrusive methods is often more cost effective, but it requires sophisticated equipment and specialty training to yield meaningful results. A number of sources provide more extensive information and analyses of non-destructive testing (NDT) methods.[2-5]

## Background

Two techniques broadly categorize pile testing: small-strain and high-strain testing. Small-strain testing is aimed at investigating the pile integrity alone and is based on the measurement of sound/stress waves by either direct transmission or reflection. Common direct transmission techniques include:

- Crosshole sonic logging (CSL);
- Single-hole sonic logging (SSL); and,
- Parallel seismic logging.

In these methods a sonic pulse is produced by one transducer (transmitter) and the signal is picked up by another transducer (receiver). The transducers typically consist of a geophone or accelerometer. The methods differ in the location of the transducers and the pulse generation method. Common surface reflection techniques include:

- Pulse echo (or, sonic echo);
- Transient dynamic response (or, impulse response); and,
- Conventional high-strain dynamic testing.

In these methods, the reflections of waves generated at the top of the pile are measured. Since both generated and reflected signals are measured at the same location, more sophisticated instrumentation (typically, accelerometers and strain gages), data acquisition and signal processing procedures must be employed. The major difference among these techniques is whether the generated impact pulse propagates under high-strain or low-strain conditions.

Other common reflection techniques include the use of high-frequency, electromagnetic pulses (such as x-rays or microwaves). These methods are more commonly used for subsurface soil evaluation (*e.g.*, stratification, groundwater and bedrock) and/or concrete slab mapping (*e.g.*, rebar, voids, thickness and condition determination).

## Direct Transmission Techniques

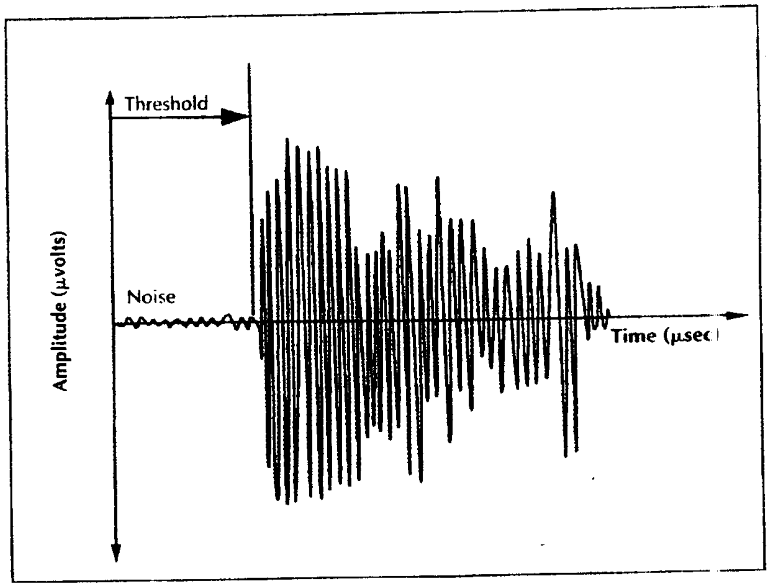


**FIGURE 2. CSL typical testing signal.**

*Crosshole Sonic Logging (CSL) Technique.* The most common direct transmission integrity testing method is CSL (or, in Europe, sonic coring). The method is used to evaluate the condition of the concrete within cast-in-place piles (caissons or drilled shafts) as well as slurry or diaphragm walls. A piezoelectric transducer is used to generate a signal that propagates as a sound (compression) wave within the concrete and another transducer is used to detect the signal. Each transducer is placed into a vertical PVC or steel tube that has been attached to the reinforcement cage and filled with water prior to concrete placement. The water acts as a coupling medium between the transducer and the tube. A typical tube arrangement and testing principles are presented in Figure 1.

The source and receiver transducers are lowered to the bottom of their respective tubes and placed so that they lie in the same horizontal plane. The emitter transducer generates a sonic pulse (on the order of 10 pulses per second), which is detected by the receiver in the adjacent tube. The two transducers are simultaneously raised at a rate of around 1 foot per second until they reach the top of the drilled shaft. Typically, this process is repeated for each possible tube pair combination (perimeter and diagonals). Figure 1b shows the six tube combinations that can be tested (logged) using a configuration of four tubes within a drilled shaft. Increased shaft diameter calls for a larger number of tubes, which increases the number of combinations and, thereby, the resolution of the testing zone.

In homogeneous, good-quality concrete, the stress/sound wave speed, $C$, is typically around 12,000 to 13,000 feet per second and is related to the modulus, $E$, and unit weight, $\gamma$, and the gravitational acceleration, $g$, as follows:

$$C = \sqrt{E \bullet g / \gamma}$$

If for any reason the condition of the concrete is compromised, the wave speed will be reduced relative to the value of the wave speed in sound concrete. Figure 2 presents a typical sonic signal for which the propagation time between the transducers is measured. The vertical axis is the signal amplitude (in microvolts) and the horizontal axis is the time (in microseconds). The point where the amplitude begins to rapidly fluctuate indicates the arrival time of the signal to the receiver (or, the threshold time). Since the distance between the two tubes is known, the wave speed of the concrete between the tubes can be evaluated. The signal arrival times can then be plotted by depth to generate a log for the particular tube combination (see Figure 3). In addition to the threshold times, the energy of each signal may also be plotted by depth. This information can be used to compare signals of one zone to another where lower energy and/or longer arrival times correspond to compromised concrete quality and/or a defect.

Advantages to this method include the direct assessment of pile integrity and the ability to position the transducers in different elevations to create more signals, allowing the development of a tomographic presentation of the

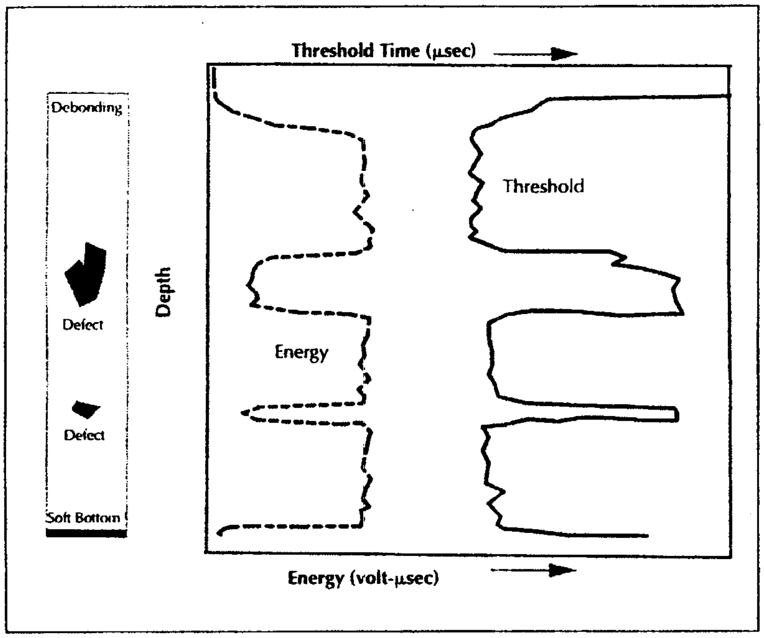


**FIGURE 3. CSL results in the form of threshold time and energy versus depth.**

investigated zone. The limitations of the method include detecting defects only when they exist between the tubes. Testing can be performed only on drilled shafts for which access tubes have been installed. Also, the method can only be used for drilled shafts, since other deep foundations are usually too small or are constructed using methods that do not lend themselves to accommodate the access tubes. Debonding between the tubes and concrete is common if testing occurs long after concrete placement. Testing in fresh concrete is also difficult since certain zones may cure at a slower rate, creating difficulties in the interpretation of the threshold time and energy. These zones may be interpreted, therefore, as poor-quality concrete.

*CSL Case History.* CSL testing was required for over 30 drilled shafts installed for the support of a state highway bridge in New Hampshire. The shafts were constructed using steel casing to the top of the rock. The soil was removed along with the casing and a 6-foot rock socket was advanced below the surface of the rock. The CSL tubes were attached to the reinforcement and placed within the shaft prior to placing the concrete. A plot of threshold time and energy against depth for a typical pair of tubes without defects is shown in Figure 4. The threshold time and calculated energy do not vary with depth, indicating uniform concrete with a wave speed of around 12,900 feet per second (tube spacing of 2 feet divided by average threshold time of 155 μsec). Approximately zero energy and no signal in the upper 6 feet are a result of the tubes sticking up above the top of the shaft.

Figure 5 presents the CSL test results for a pair of tubes indicating a soft bottom condition. The increased threshold time and decreased energy over the lowermost few feet suggest the influence of a slurry cake or a slurry/soil mixed with the concrete at the bottom of the rock socket. The soft bottom conditions were also encountered in several other shafts. Since all shafts had 6-foot rock sockets, their load-carrying ability did not rely on end bearing and the soft bottom conditions were deemed not to affect their performance.

Figure 6 presents the CSL test results for a shaft in which a defective zone was identified in the upper 14 feet. During the placement of the concrete and the pullout of the casing some of the surrounding soil along the upper 26 feet collapsed. As a result, the concrete level dropped 12 feet and a contaminated concrete zone was suspected. The extent of the contamination was not known at the time, and the CSL test log for other tube pair combinations (in addition to the data presented in Figure 6) confirmed that the upper 14 feet consisted of a zone of compromised concrete. This zone was chipped out, removed and replaced with new concrete.

Debonding between the tube and the concrete can create similar signal patterns to those that were detected for the compromised concrete. It is extremely important, therefore, to obtain both the installation log and nearest soil-boring log for each shaft to aid in CSL test inter-

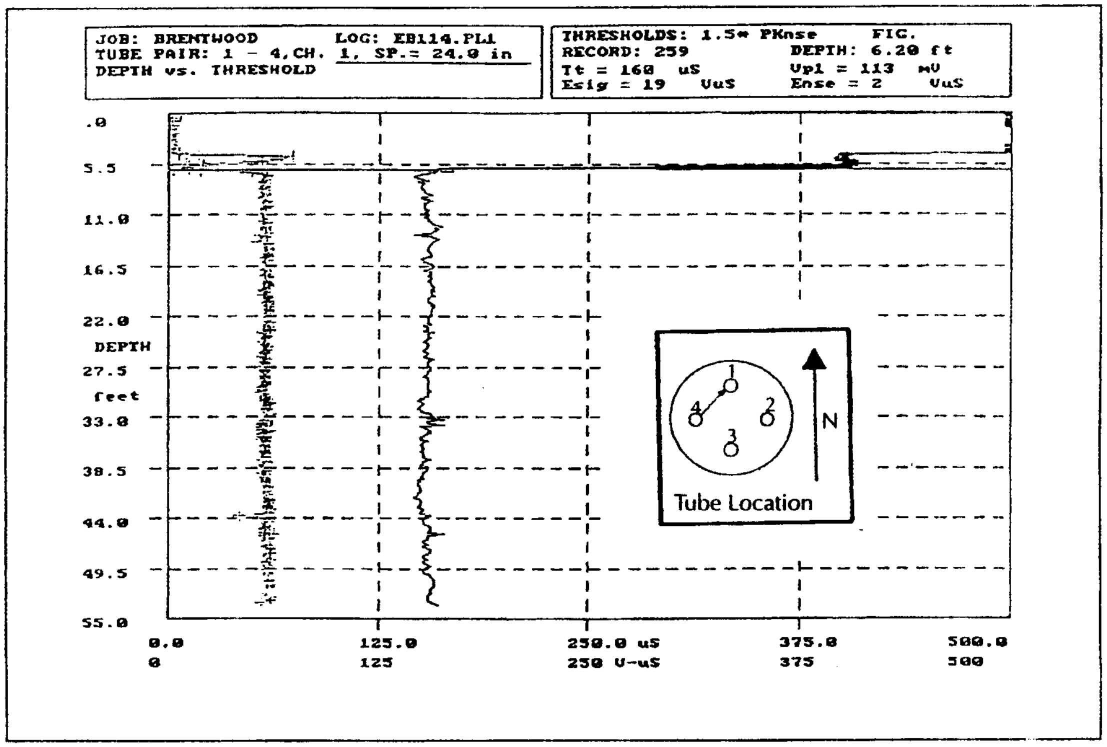


**FIGURE 4. CSL threshold time and energy versus depth (no defects).**

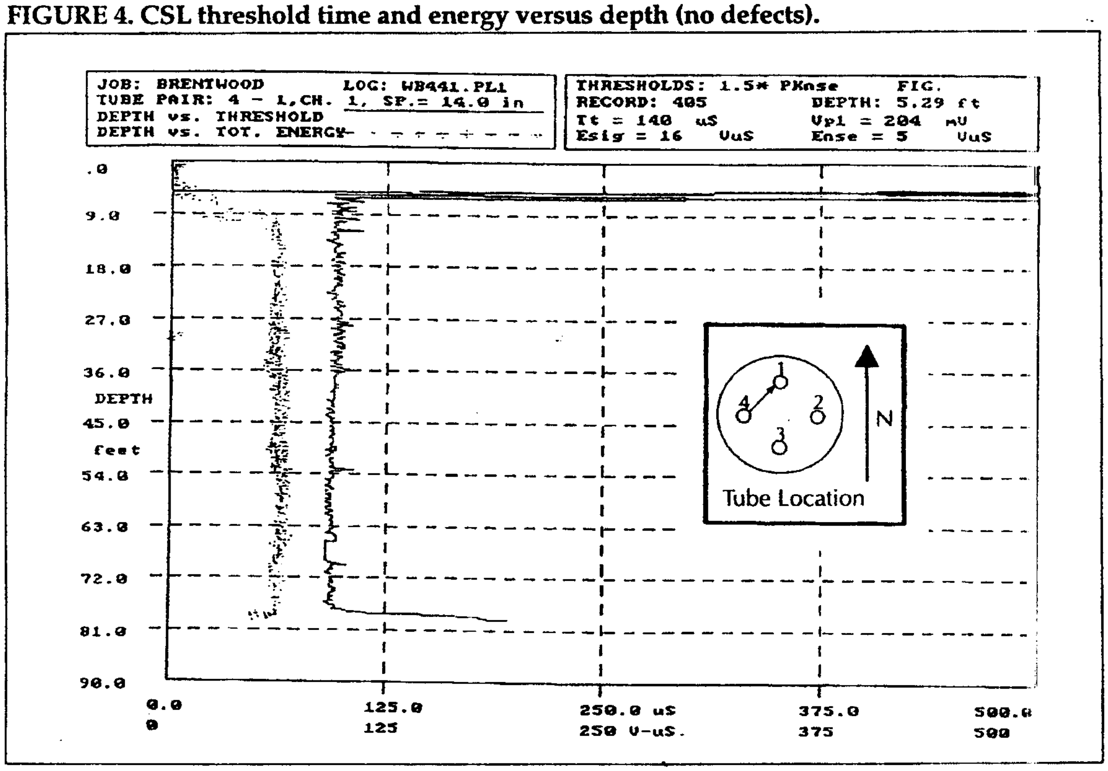


**FIGURE 5. CSL threshold time and energy versus depth (soft bottom condition).**

and word processing). In addition, such systems can easily utilize updated algorithms, for example, for real-time on-screen tomographic presentation.

The first use in the United States of a portable personal computer-based CSL test system was applied for the analysis of some drilled shafts for highway construction near Worcester, Massachusetts. Figure 7 presents the layout of the test system's screen for shaft identification. User-friendly drag-and-drop features allow for a simplified and efficient use. Real-time updated data of the screen are presented in Figure 8. This dual presentation allows observing (as well as scaling, filtering and cutting among other features) the arrival time and energy signal (see Figure 3) simultaneously. The detection of a possible defective zone allows the acquisition of data in different sections by tracking the depth of both transmitter and receiver and its use in tomographic analysis, outlining the defective zone. The test system will probably result in a new generation of NDT equipment that is better suited for versatile testing demands, advanced analyses and field applications.

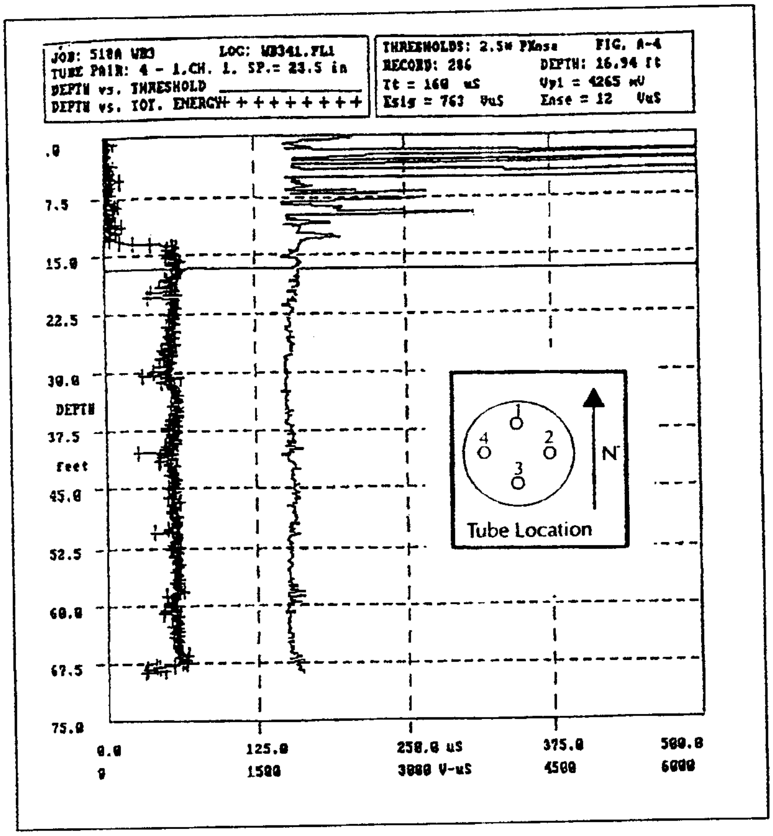


**FIGURE 6. CSL threshold time and energy versus depth (defective zone in upper 14 feet).**

pretation. For example, similar signals that were recorded in the upper few feet of other shafts in the same project were attributed to debonding. The major reason was that those CSL tests were performed almost two months after the concrete placement, thereby allowing sufficient time for the tubes to separate from the surrounding concrete in the upper shaft zone.

*Recent Advances in CSL Testing.* Following recent technological advances, a new concept in NDT equipment has emerged.[6,7] The use of laptop/portable PC-based systems and modular equipment components seem to be taking the place of the current dedicated systems. Naturally, the new concept allows small size, lighter, independent equipment with broader NDT applications. Such equipment has the advantage of employing common operating systems conforming to other requirements (*i.e.*, graphics presentations, spreadsheet, database

## Additional Direct Transmission Techniques

*Single-hole Sonic Logging (SSL).* SSL is a variation of the direct transmission CSL method in which the source and receiver are placed in the same tube and the signal travels in a vertical direction (see Figure 9). The method is limited to defects adjacent to the tube and is usually used only when a drilled shaft requires integrity assessment after construction. Due to high coring costs, a single hole typically is advanced (often down the middle) to the bottom of the shaft or slightly below the depth where a defect is anticipated. It may also be desirable to perform SSL tests during CSL testing to isolate the location of a defect at a certain depth (*i.e.*, determining whether the defect identified by using CSL is adjacent to the tube or in between the tubes). Brettman and Frank describe a comparison between CSL and SSL tests.[8]

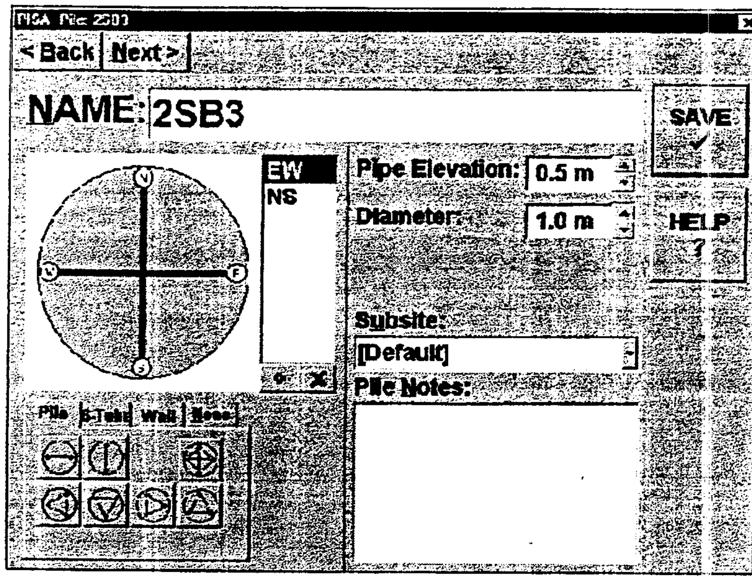


**FIGURE 7. Layout of field screen for shaft identification.**

*Parallel Seismic Logging.* Parallel seismic logging is another direct transmission integrity testing variation of the CSL test. The method is performed primarily for the assessment of the depth of older foundations. Although large voids or bulges can be identified along the deep foundation edge, it is not typically used for identifying defects. Figure 10 presents the procedure in which a boring is drilled in the ground adjacent to the existing deep foundation (usually within 2 to 3 feet of the deep foundation edge). The drilled hole is advanced well beyond the estimated tip elevation to ensure that the entire deep foundation profile can be logged. A capped PVC tube is placed within the drilled hole and surrounded with bentonite slurry/grout that bonds the tube to the edge of the boring.

A receiver transducer is placed at the bottom of the water-filled tube and pulled upwards at intervals of approximately 1 or 2 feet. At each depth interval, the foundation top is struck with an instrumented hammer that sends a pulse down the pile and the soil. This pulse is to be detected by the receiver. A typical profile of the signal arrival time with depth can be logged as shown in Figure 10b. A change in the rate of signal arrival time signifies either a large defect or the end of the pile.

The most attractive feature of this technique is that any deep foundation type can be tested as long as the drilled hole is close to the foundation. (Due to the higher cost associated with drilling, this technique is used to identify foundation depth only when other methods fail.)

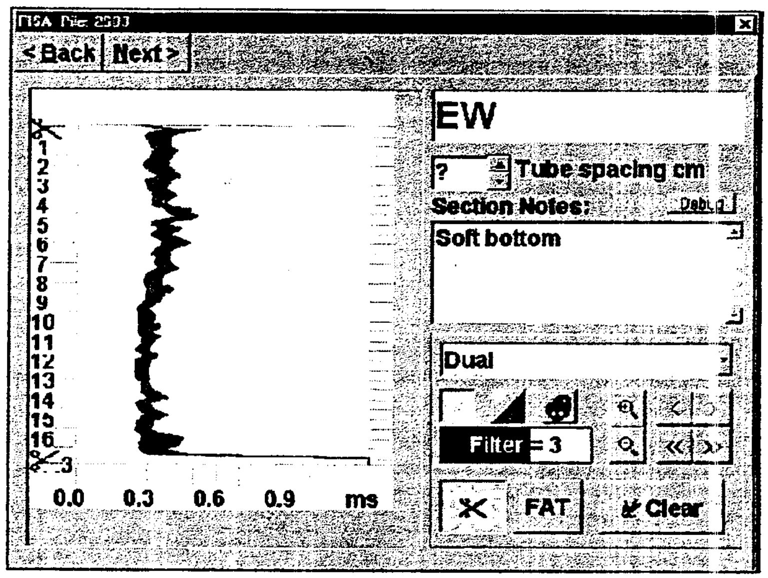


**FIGURE 8. Real-time screen layout presenting arrival time and the energy signal simultaneously.**

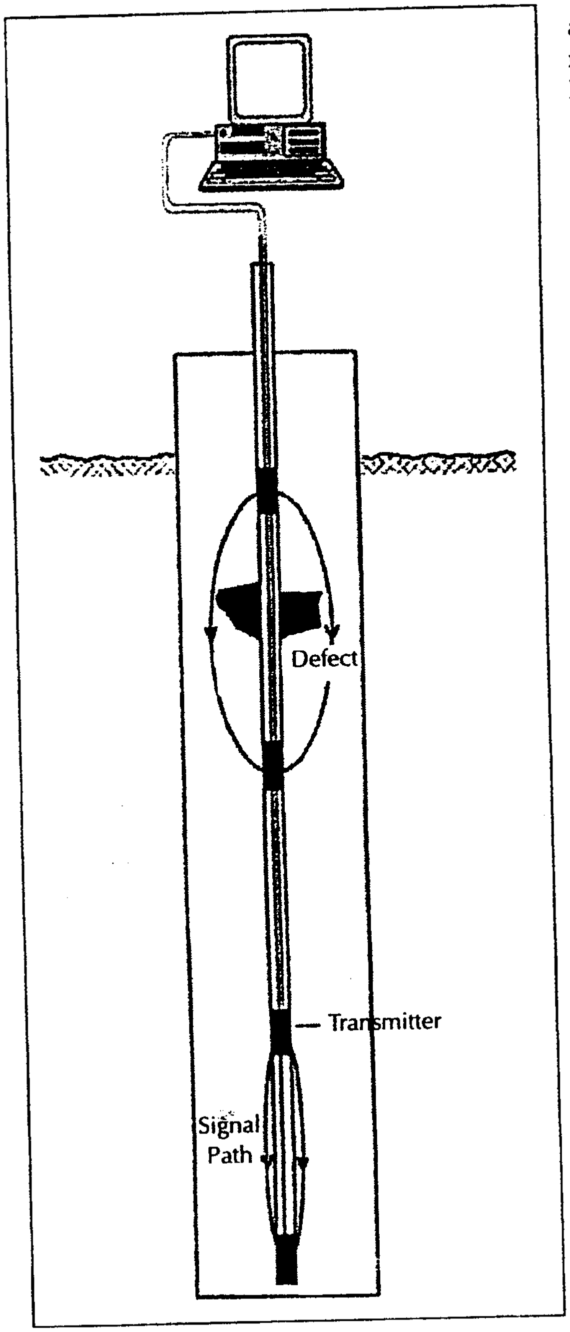


**FIGURE 9. A typical SSL test set-up showing the transmitter and receiver placed at different depths.**

## Surface Reflection Techinques

*Pulse Echo Method (PEM).* PEM (also known as the sonic echo method) is a surface reflection integrity testing technique. A high-frequency accelerometer is attached to the pile top using a mild bonding agent such as petro wax or petroleum jelly. A light-weight hand-held hammer (1 to 3 pounds) is used to strike the pile top and generate a small-strain stress wave. The strains associated with the propagating stress wave are in the order of 1 $\mu\varepsilon$. Figure 11 presents a typical PEM integrity test set-up. The hammer is usually constructed from plastic to minimize the extraneous high frequencies generated by the steel. The accelerometer attached to the pile top measures the acceleration at impact and the reflections arriving at the surface from within the pile. This analog acceleration signal is recorded, digitized and integrated (using a computer) to create a velocity record.

The velocity record indicates the speed at which the pile material (at the point of measurement) moves due to the impact and reflected stress waves created by the hammer. Typical velocity and force records are shown in Figure 12. This record can be further processed using algorithms that enhance the signal through filtering, shifting, pivoting and magnification. This manipulation allows enhancement of the velocity signal for weak toe reflections, reducing the effect of unwanted noise and drifts, thereby aiding in the interpretation of the pile response.

Changes in the pile cross-section, concrete density and/or soil resistance affect the impedance in the direction of the traveling wave and create reflections of the stress wave that propagate back towards the pile top. These reflected stress waves can return in compression or tension, depending on the type of impedance change. The pile properties that define impedance, Z, are expressed as:

$$Z = (E \bullet A)/C$$

where:

$C$ is the speed at which the stress wave propagates;
$E$ is the elastic modulus; and,
$A$ is the cross-sectional area.

Figure 13 illustrates the relationship between the variations in the pile impedance, the traveling wave and the reflections recorded at the surface. A reflected tension wave indicates a decrease in impedance. Conversely, a reflected compression wave indicates an increase in impedance. Combinations of these impedance changes can create complex reflections at the pile top. By inspecting the velocity record for these changes, the approximate location of the impedance change can be determined. At a time of $2L/C$ (where $L$ is the pile length), the pile toe response can be identified by observing a reflected tension wave due to softer soil at the tip (the signal is in the opposite direction of the impact pulse — analogous to free-end conditions) or a reflected compression wave due to denser soil at the pile tip (the signal is in the same direction as the impact pulse — analogous to a fixed-end condition).

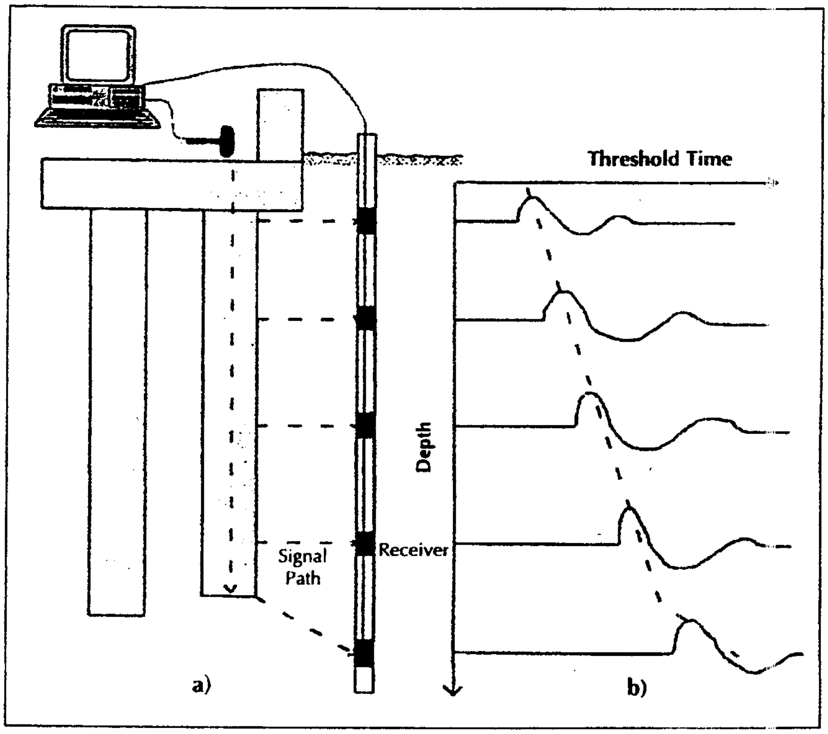


**FIGURE 10. A typical parallel seismic testing arrangement showing a) instrumented hammer and receiver at several depths, and b) threshold time versus depth.**

One of the most difficult tasks in the interpretation of the velocity record is distinguishing between the velocity reflections due to pile defects (*e.g.*, crack, neck, void or poor quality concrete) and velocity reflections due to soil resistance. Detailed quantification of defects is difficult (if not impossible) since the interpretation is based on reflected waves and also relies on an assumed wave speed. The most reliable way to use the method is by comparing the response from a large number of piles at the same site. Piles that indicate a response that is different from the majority should be further investigated.

PEM testing is simple and quick and, hence, can often be performed on all the piles at a site. PEM testing can be carried out on various deep foundation types and materials. Under certain conditions, PEM tests can be performed on piles that have been covered by a cap or grade beam structure. The small-strain PEM technique is generally effective to a depth of 20 to 30 pile diameters, depending on the magnitude and distribution of the frictional soil resistance.

*Transient Dynamic Response (TDR) Method.* The TDR method (also known as the impulse response method) is based on the PEM technique except that an instrumented hammer is used to generate the impact pulse. An accelerometer mounted in the hammer, or a force transducer built into an impulse hammer, permitting the determination of the impact force (using the hammer's mass) in addition to the velocity records obtained by the PEM test (see Figure 12a). Since a force transducer is not attached to the pile, only the impact force is recorded. The force and velocity records can be converted from the time domain to the frequency domain using a Fast Fourier Transform (FFT). The ratio of the velocity spectrum, $V$, over the force spectrum, $F$, yields the mobility spectrum ($V/F$ in the frequency domain, presented in Figure 14), providing an indication of the pile's velocity response due to the induced excitation force.

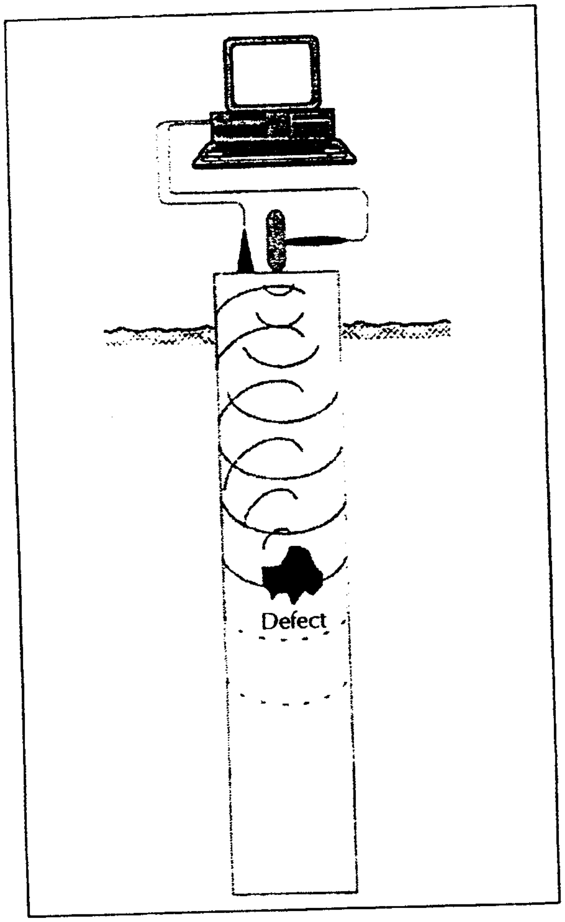


FIGURE 11. A typical PEM integrity test set-up.

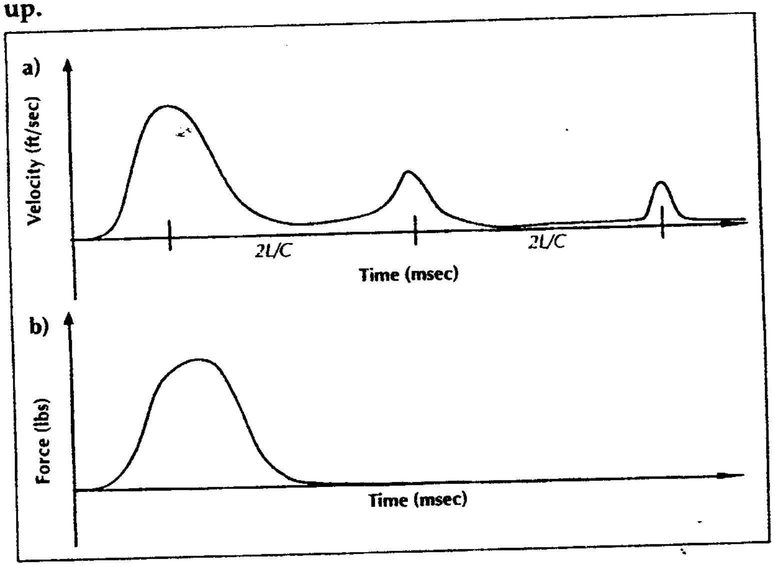


FIGURE 12. Typical PEM a) velocity and b) force records.

The TDR method allows additional insight compared to the PEM interpretation technique. Certain dominant frequencies can be identified and correlated to pile length and distance to variations either in the pile impedance or in the soil. In addition, the low-frequency components (less than 100 Hz) can provide an indication of the dynamic stiffness of the pile. Although low-strain methods permit obtaining an estimate of static pile behavior, they cannot accurately determine the pile bearing capacity. In contrast to dynamic measurements during driving or static load test to failure, these methods do not fully mobilize the pile's resistance.

*PEM/TDR Case History: Pressure-Injected Footings.* Approximately 600 pressure-injected footings (PIFs) were installed as part of the foundation system for a large entertainment complex in Worcester, Massachusetts. Two PIFs were visually observed to contain poor-quality, low-strength concrete reduced to a putty-like consistency near the pile tops. The upper few feet of these PIFs were cut off to remove the material and assess the extent of the defective zone.

Ten PIFs, including the visually observed defective piles, were selected for PEM/TDR integrity testing in order to assess the concrete quality in the shafts. The shafts consisted of corrugated metal shells filled with cast-in-place concrete. Reinforcement steel was installed within the upper 5.5 feet to allow for connection to the pile caps. The subsurface profile in the vicinity of the test area included 5 to 20 feet of granular fill over dense sand and gravel. The PIF bulbs were formed in this denser stratum.

Figure 15a presents the velocity record with pile length for a sound PIF. The signal indicates a decrease in the velocity around 24 feet, signifying a compression wave reflection due to the transformation from the shaft to the bulb, corresponding to an increase in the impedance. The velocity

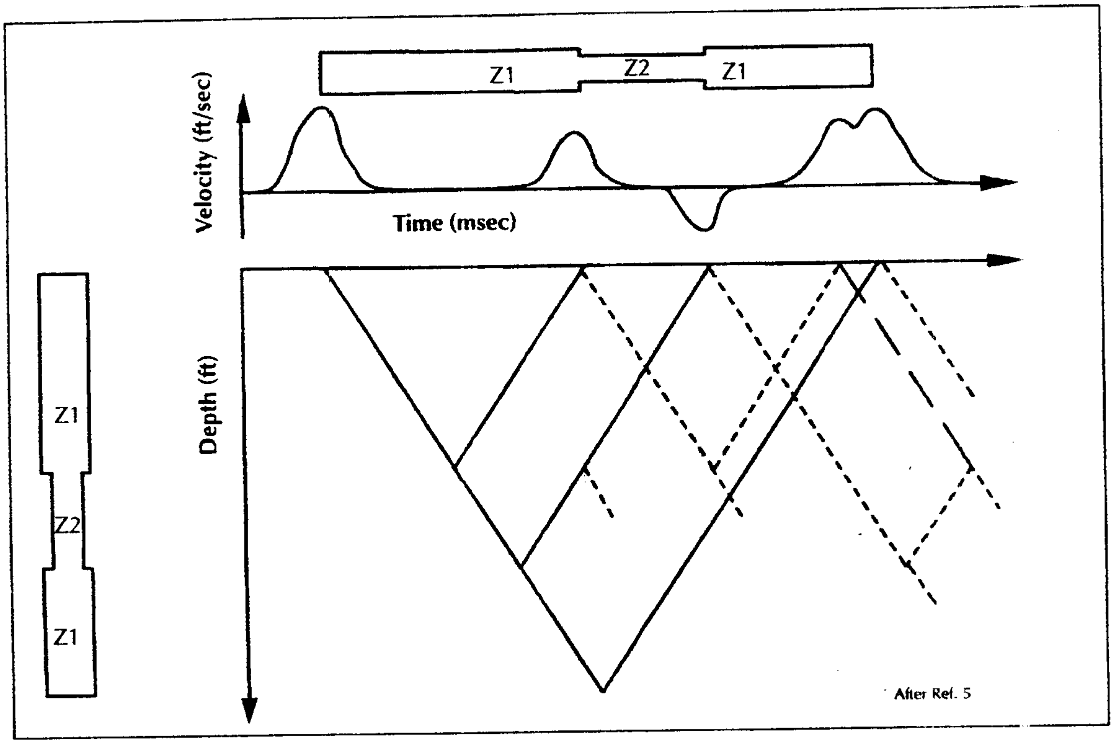


**FIGURE 13. Wave propagation and reflections versus time and depth.**

increases sharply at around 26 feet, due to a tension reflection from the bottom of the bulb (where the impedance decreases when transforming from the concrete bulb to the surrounding sand). The mobility spectrum for this PIF is presented in Figure 16a where peaks about 256 Hz apart appear between approximately 400 and 1600 Hz. This change in frequency, $\Delta f$, corresponds to a length of around 25 feet, based on the following relationship between time ($t$) and frequency ($f$):

$$t = 1/\Delta f$$

The relationship between time and distance (considering reflection) is also applied:

$$L = C \bullet t/2$$

The PIF shaft length was reported to be 23.7 feet, which closely agrees (considering the accuracy of the construction method and the testing procedure) with the above-determined length.

Figure 15b presents the velocity record with pile length for a PIF that was found to have a major defect. The velocity increases sharply at around 7 feet due to a discontinuity associated

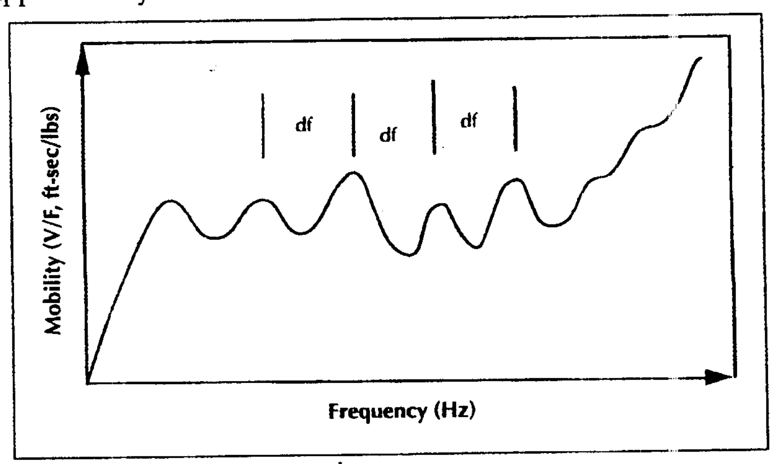


**FIGURE 14. A mobility spectrum (V/F versus frequency) using records obtained by the TDR method.**

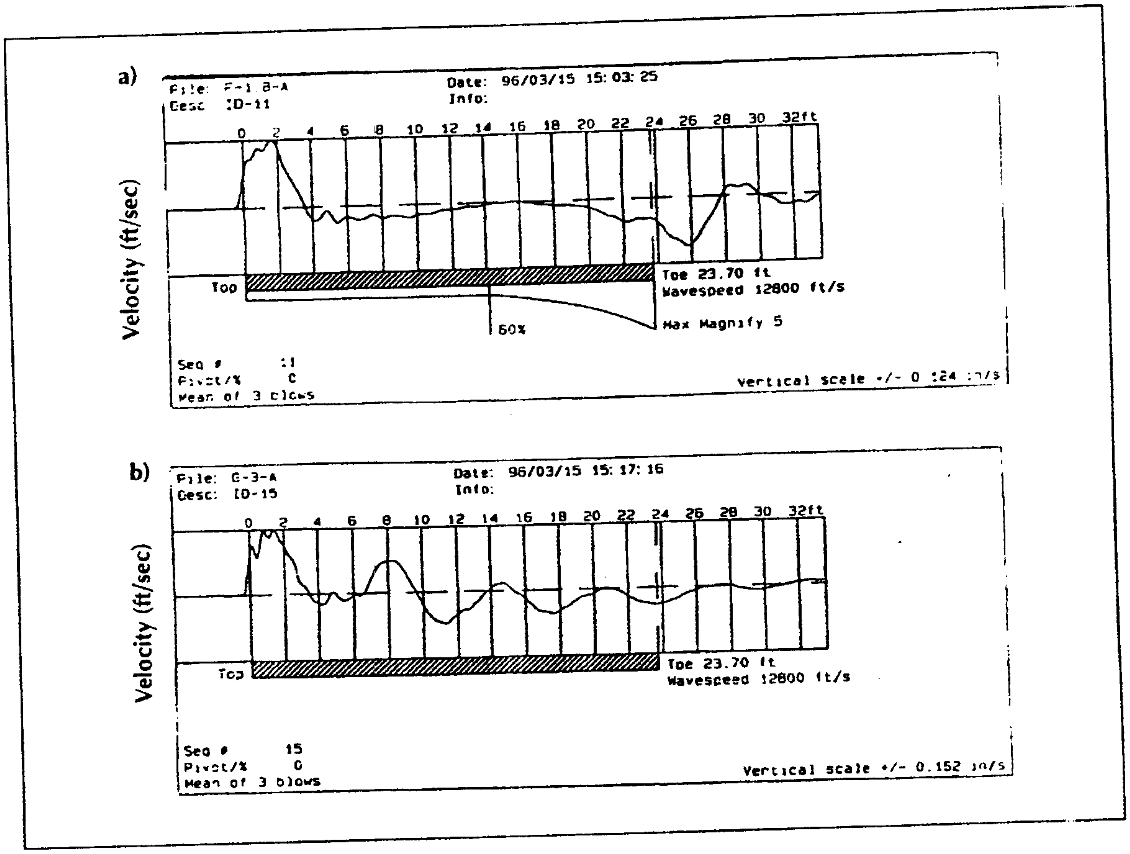


**FIGURE 15. PEM velocity records versus time for a) a sound pile and b) a defective pile (for PIFs).**

with a large reduction in the impedance. In fact, the low-magnitude stress wave could not pass through this defect and the reflections are repeated every 7 feet with the signal dampened with time. Even though the PIF shaft was reported to be 23.7 feet long, the length indicated by the test was around 7 feet since the defect occupied almost the entire cross-section. The mobility spectrum in Figure 16b looks significantly different from that of a sound pile presented in Figure 16a. In this case, the change in frequency is around 928 Hz, which corresponds to a length of 7 feet.

The soil around the "compromised" PIF was excavated to a depth of 10 feet. The corrugated shell was torched off the shaft around 8 feet below the top of the pile. When the shell was removed, the PIF fell over, due to a complete break in cross-section around 7.5 feet. Another PIF evaluated by PEM/TDR testing revealed a defect at around 5 feet. This PIF was also excavated. After its corrugated shell was removed, a large volume of water and putty-like concrete fell out of the shell. An approximate 20 percent reduction in cross-section was observed at a depth of 4 to 5 feet below the top of the PIF. As a result of the integrity testing and subsequent verification in the field, a reduced cross-sectional area was used to reassess the load-carrying ability of the foundations.

*PEM Case History: Precast Concrete-Driven Piles*. Damage in driven piles can be detected while monitoring the pile capacity using high-strain dynamic measurements. These tests are traditionally carried out on a small number of piles even though typical concrete pile breakage during installation is about 5 to 7 percent of the piles installed. Damage during driving or site work following the installation can result in piles with questionable integrity.

Figure 17 presents PEM test results on 14-inch-square concrete piles about 90 feet long that were driven for the support of multi-story buildings in Cambridge, Massachusetts. A re-

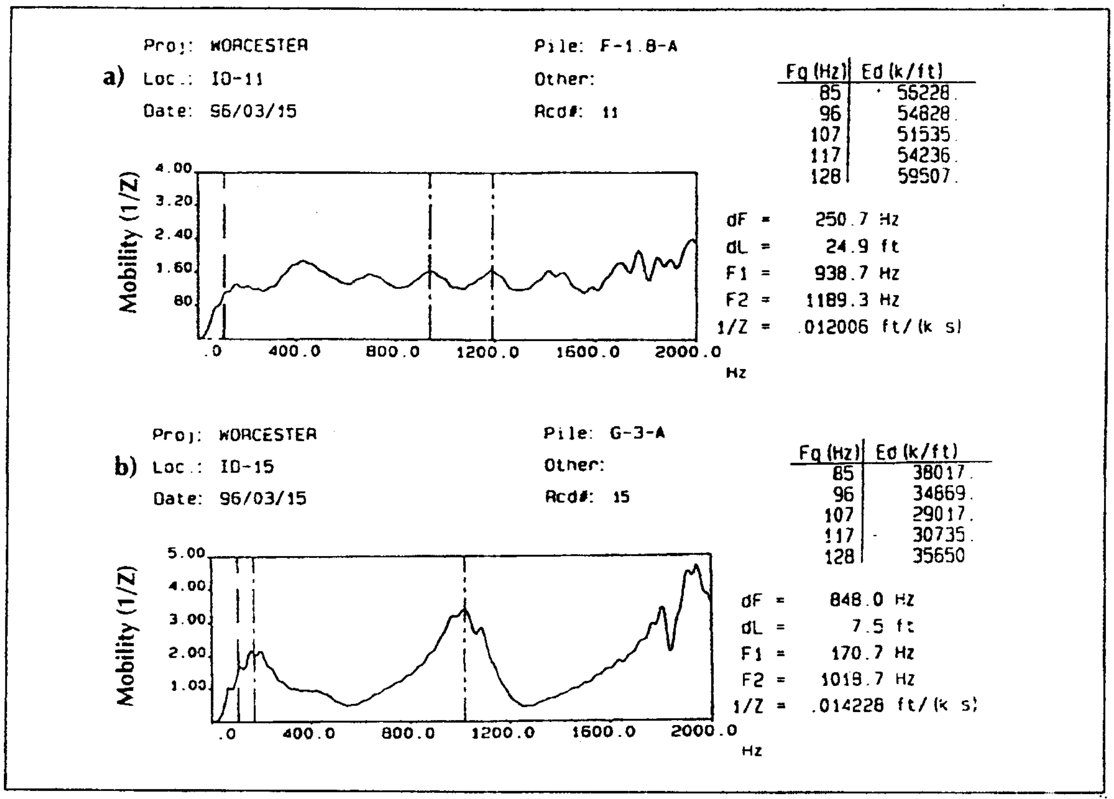


**FIGURE 16. TDR mobility versus frequency responses for a) a sound pile and b) a defective pile (for PIFs).**

petitive increased velocity reflection (at times corresponding to a distance of about 20 feet) is presented in Figure 17a. The repetitive reflection indicates damage extensive enough to prevent the signal from propagating any deeper than the indicated depth. However, no evidence is provided by the test regarding the compression load-bearing capability of the pile. Figure 17b presents the results obtained from a nearby sound pile for which the propagating signal responded to the variation in the soil type (sand layer at about 25 to 30 feet and a till layer at about 60 to 70 feet). The tip response was magnified due to the small energy used in the PEM testing. As a result, the technique's effectiveness at such depths is questionable.

The usefulness of the method with regard to time and cost savings was certainly a big advantage, allowing the identification of a large number of defective piles in a short period of time. However, the limitations regarding the nature of the damage and the structural ramifications need to be recognized as well.

## High-Strain Integrity Testing During Pile Driving

Dynamic pile testing is commonly employed for evaluating the drivability and capacity of driven piles. The same method is also used to assess the capacity of cast-in-place shafts. When a ram strikes the pile head, it initiates a large strain wave that propagates down the pile as illustrated in Figure 18. External soil resistance or changes in the pile's impedance (due to variations in the pile's material or geometry) cause reflection waves that are recorded at the surface in a manner similar to that done for PEM/TDR low-strain methods. Typical dynamic pile testing instrumentation consists of two accelerometers and two strain transducers attached on opposite sides close to the top of the pile. Knowing the material properties and pile geometry at the point of meas-

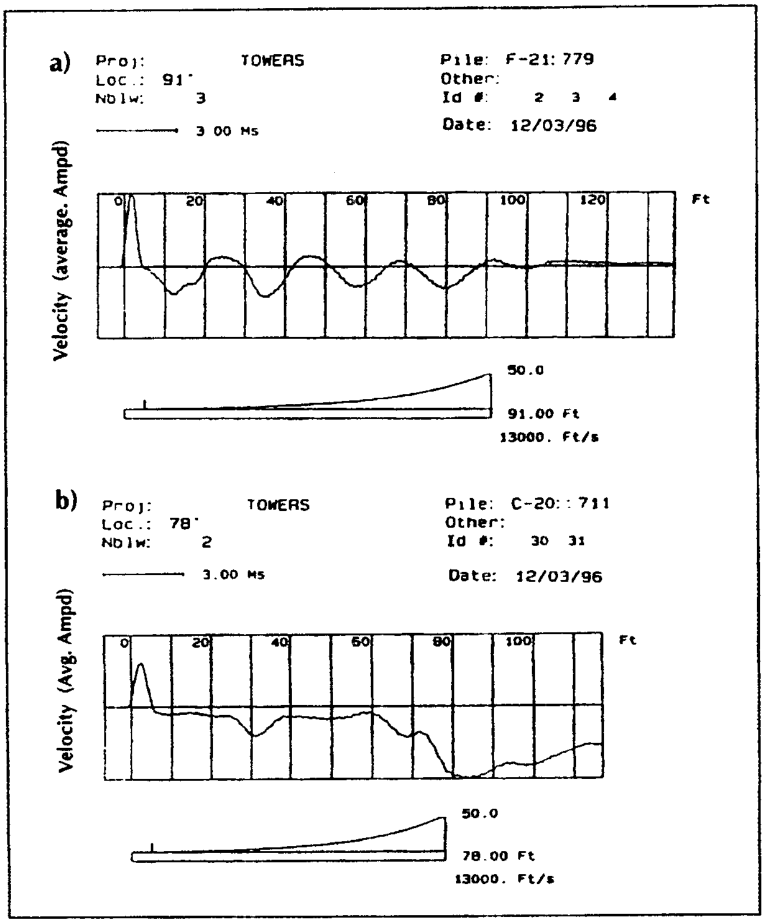


**FIGURE 17. PEM velocity records versus time for a) a defective pile and b) a sound pile (precast concrete driven piles).**

urement, the strain is converted to force, while the acceleration is integrated with time to produce a velocity record. These force and velocity records can be used to evaluate the pile's integrity. As long as there is no change in the pile impedance or as long as external forces (friction) are not activated, the force and velocity remain proportional. Reflections from the tip can be reviewed in light of two classical boundary conditions (see, for example, Timoshenko and Goodyear[9]): free-end and fixed-end conditions. Free-end conditions (analogous to easy driving through soft clay) call for zero stress and no velocity restrictions at the tip, resulting in a compression wave returning as a tension wave and velocity increase (theoretically doubling). Figure 19 presents reflections from a 48-inch-diameter pipe pile driven offshore with an initial penetration of about 3 feet. The downward velocity and compression stress returned from the tip as a tension wave and as increased downward velocity. Fixed-end conditions (analogous to hard driving against bedrock) call for zero velocity and no stress restrictions at the tip, resulting in a compression wave being reflected with a greater magnitude than the incident wave and a tip velocity of approximately zero.

If a pile contains a defect or is damaged during driving, the wave reflecting from the zone of decreased impedance is comparable to free-end conditions. These reflections would arrive at the measuring transducers before the reflections associated with the pile's tip since the damaged zone is located at a point along the pile between the top and the tip. The detection of damage during driving is routine and usually is associated with tension cracking of concrete piles. Other structural damage (*e.g.*, splice breakage) can also be identified. The advantage of high-stress wave propagation testing over small-strain integrity testing is its ability to quantify the structural significance of the discontinuity. While a small-strain wave would indicate a complete discontinuity for any size crack across the pile, the high-strain stress wave would pass through these discontinuities enabling the transformation of compression forces, therefore indicating the adequacy of the structural member.

*Case History.* Several hundred H-piles were installed for the support of an elevated walkway in the Boston area. Dynamic pile testing was specified for capacity monitoring and the driving operation progressed routinely. One of the inspected piles exhibited a clear damage profile during driving. Figure 20 presents the force and velocity records obtained during the driving of that pile. The force and velocity (multiplied by the pile's impedance) signals at the pile top shortly before and after damage detection are depicted in Figures 20a and 20b, respectively. Since the early damage identification was dismissed, driving continued and the dynamic records for the subsequent blows are presented in Figures 20c, 20d and 20e. A clear velocity increase accompanied by a force decrease attests to the development of the damage. The record shown as Figure 20e suggests that the pile essentially "ends" at mid-point, indicating a complete detachment between the upper and lower pile sections. The identified damage was associated with a full penetration weld splice that apparently disintegrated during driving. When the pile was pulled out of the ground only the upper section above the weld was extracted with severe deformations at the weld connection.

## Discussion

A variety of non-destructive, intrusive and non-intrusive deep foundation integrity test-

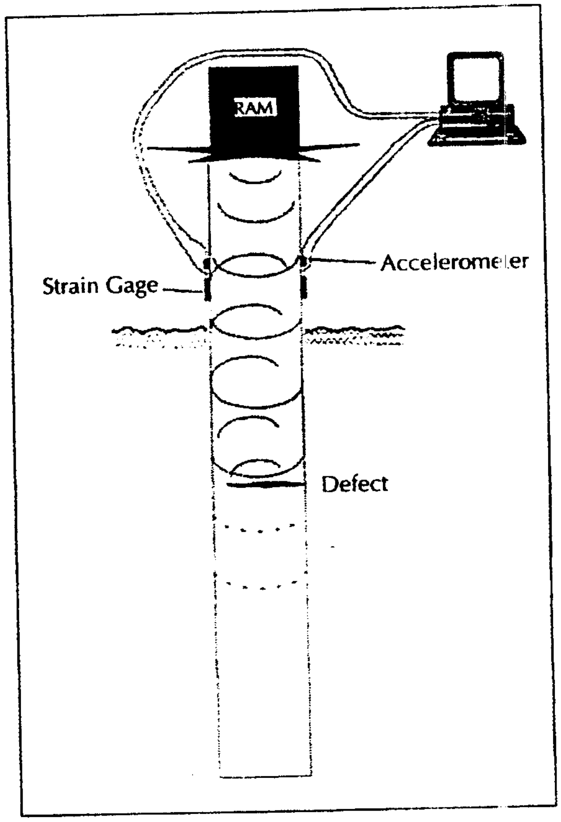


**FIGURE 18. A typical dynamic test set-up.**

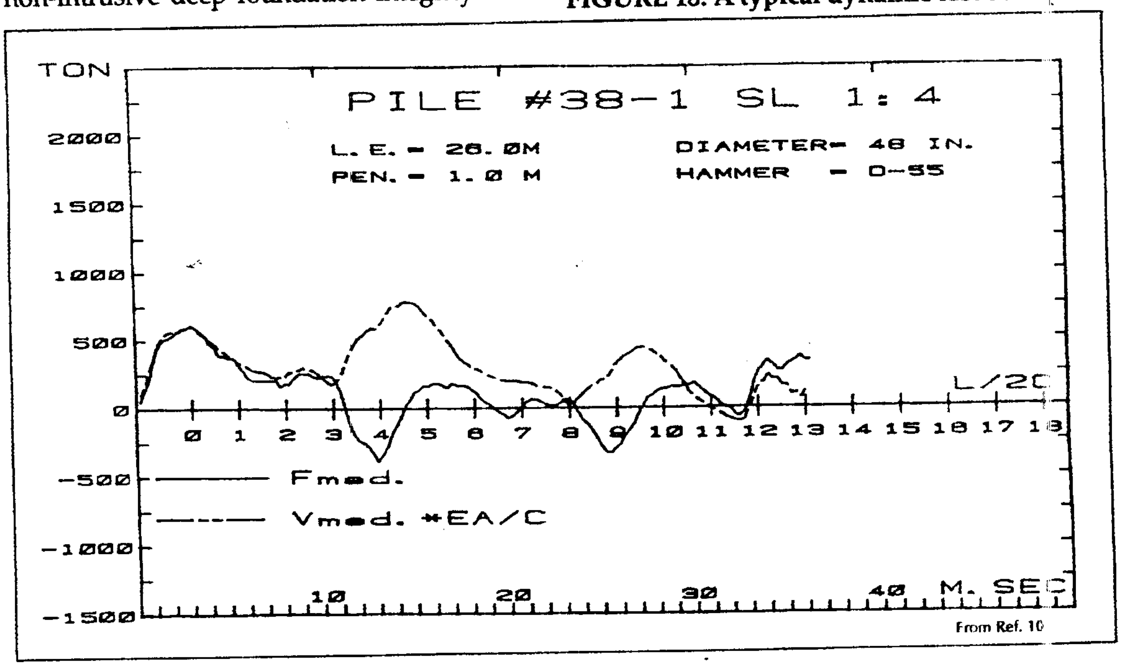


**FIGURE 19. Measured force and velocity (times the pile impedance) at the pile top versus time during initial penetration.**

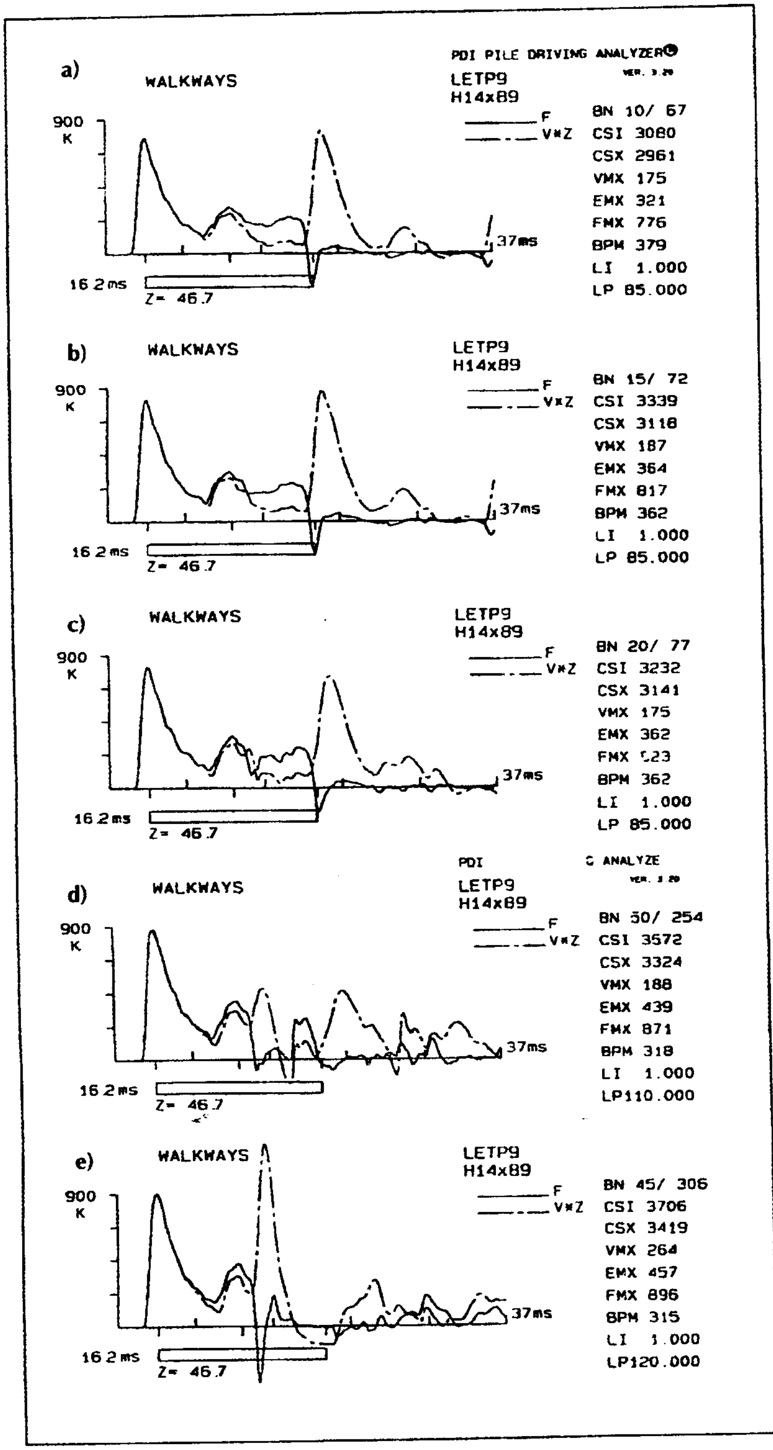


**FIGURE 20. Force and velocity records obtained during the driving of a steel H-pile a) shortly before detecting damage, b) showing initial damage, c) as the damage develops, d) as the damage progressed and e) upon complete discontinuity.**

ing methods provide different benefits and drawbacks. The methods' strengths and limitations are related to their effectiveness, time (in terms of preparation, testing and interpretation) and associated cost. In general, the direct transmission methods necessitate considerable preparation and can provide higher accuracy in the zone bounded by the penetrating sleeves. Surface reflection techniques require only minimal preparation but are limited in their zone of meaningful operation and accuracy. The selected testing method needs to reflect the anticipated result and the associated course of remedial action. When selecting a method, it is best to review the comparative studies of known embedded defects.[2,11]

The ability of a method to detect a certain defect should be examined in light of the defect's influence on foundation serviceability. This course of action leads to the selection of an integrity testing method based on the expected outcome. For example, the possible detailed data provided by the direct transmission methods should not result in rejecting using a caisson just because certain zones suggest a lower quality of concrete. Such decisions need to be associated with the design loads and the load-bearing assessment of the tested caisson. The surface reflection methods, on the other hand, allow extensive testing with the expectation that detailed investigations should be carried out on the suspected caissons only. Choices, therefore, should be made regarding the quantity and quality of the testing program. For example, many piles can be tested with the ability to detect major defects (where possibly undetected defects are not expected to compromise the pile's load-carrying ability), or detailed studies of a smaller number of piles can be performed, or a combination of the two methods can be implemented.

## Conclusions

The reviewed methods and the presented case histories demonstrate that deep foundations integrity testing is useful and has significant importance. PEM/TDR and CSL techniques are routinely used to assess the quality and condition of cast-in-place and driven piles. Conventional dynamic testing is effective in evaluating pile integrity during driving or whenever a driving system is available. In some cases, the results of the integrity testing were used to reject the piles; in other cases, they were used to re-evaluate or redesign the piles. Frequently, integrity testing is used to confirm anticipated defects in the piles. When using an adequate testing method, along with engineering judgment, integrity testing of deep foundations can be employed as an important tool with sound economical justification.

Integrity tests are a useful and important tool — especially true when a match exists between the implemented technique, foundation type, user expertise and the owners' expectations. Solid engineering judgment, analysis and decision-making enhances the ability to utilize the test results and, hence, their usefulness and importance.

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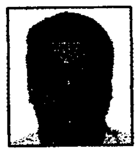

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