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Nondestructive Testing of Unknown Subsurface Bridge Foundations—Results of NCHRP Project 21-5

This NCHRP digest presents the findings of NCHRP Project 21-5, "Determination of Unknown Subsurface Bridge Foundations," which evaluates existing and new technologies for use in determining unknown subsurface bridge foundation characteristics. A continuation project (NCHRP Project 21-5(2) is underway, which will research and develop equipment, field technologies, and analysis methods for those new technologies with the most promising application to nondestructive testing of unknown subsurface bridge foundations. This digest was prepared by the staff of Olson Engineering, Inc. The Principal Investigator for the project is Larry D. Olson of Olson Engineering, Inc.

INTRODUCTION

This digest contains information about the feasibility of using nondestructive test (NDT) methods for the determination of unknown depths of bridge foundations. This will be of interest to bridge and other structural engineers; soils, geology, and foundation engineers; and materials and construction engineers.

Of the approximately 580,000 highway bridges in the National Bridge Inventory, many of the older, non-federal-aid bridges have no design plans available. Therefore, no information is available regarding the type, depth, geometry, or material incorporated in the foundations (Elias, 1992; Watson, 1990; Baguelin, et al. 1980). The current best estimate of the population of bridges over water with unknown foundations is 106,000, with 25,000 of the bridges being on-state systems and 81,000 bridges being off-state systems. These unknown bridge foundations pose a significant problem to the departments of transportation (DOTs) of the various states because the Federal Highway Administration (FHWA) is requiring state DOTs to screen and evaluate all bridges to determine their susceptibility to scour. Foundation depth information, in particular, is needed for performing an accurate scour evaluation at each bridge site, along with as much other information on foundation type, geometry, materials, and subsurface conditions as can be obtained.

NCHRP Project 21-5, "Determination of Unknown Subsurface Bridge Foundations," was introduced to evaluate and develop existing and new technologies that could determine subsurface bridge foundation characteristics, where such information was unavailable. The project was carried out in two stages. The first stage consisted of the review and evaluation of existing and proposed technologies having promise for use in determining unknown subsurface bridge foundation characteristics such as depth, type, geometry, and materials; this was followed by development of a research plan. The second stage of the project consisted of evaluating and testing as many of the recommended concepts, methods, and equipment as was feasible under the remaining project budget.

Nine technologies were selected for the second stage of research work. They included five *surface* techniques (i.e., sonic echo/impulse response, bending [flexural] wave, ultraseismic, spectral analysis of surface waves [SASW], and dynamic foundation response tests) and four *borehole* techniques (i.e., parallel seismic, borehole sonic, borehole radar, and induction field tests). The surface techniques require access to the exposed parts of the bridge substructure elements; the borehole methods require access through a nearby boring. The major objective of the research was to evaluate the capabilities of the various NDT methods that indicate depth and other information on unknown bridge foundation characteristics for widely varying *known* bridge substructure conditions.

Nondestructive tests, coupled with theoretical modeling, were performed at seven bridge sites (four in Colorado, two in Texas, and one in Alabama) under NCHRP Project 21-5. Case histories were also reported from independent NDT consulting investigations to determine unknown foundation conditions.

Since this project was completed, a continuation project, 21-5(2), "Unknown Subsurface Bridge Foundation Testing," has been awarded to Olson Engineering, Inc. That project is for further development, instrumentation, and validation of the surface and borehole NDT technologies with the greatest application range as determined in this research project.

NCHRP PROJECT 21-5 RESEARCH APPROACH AND INTERPRETATION

Discussions of the NDT methods used in the research and their applications follow, along with a summary of the research findings. Data example results are shown below *only for the most promising NDT methods*.

Sonic Echo/Impulse Response Method and Results

The sonic echo/impulse response method was developed for testing the integrity and length of single rodlike, columnar, deep foundations such as drilled shafts and driven piles (Koten and Middendorp, 1981; Davis and Dunn, 1975). This method (Figure 1) involves hitting the top of a deep foundation with a hammer to generate a downward-traveling compressional wave; this compressional wave reflects back to the surface from changes in stiffness, cross-sectional area, and density (i.e., acoustic impedance). The arrival of the reflected compressional wave energy is sensed by a receiver (i.e., an accelerometer or a vertical geophone). Analyses are done in the time domain for the sonic echo test and in the frequency domain (mobility transfer function, that is, velocity/force) for the impulse response test. Where top access is not available, or cap beams or superstructure are present that would greatly complicate the interpretation of reflections, testing can be performed on the sides of accessible substructure.

In the sonic echo test, exponential amplification with time of the receiver trace (in velocity units) is usually required to enhance weak echoes and compensate for the damping of the wave energy as it travels down and back up a foundation. For wavelengths that are long relative to the rodlike foundation diameter, the bar compressional wave velocity, V, is equal to the square root of Young's modulus, E, divided by mass density, ρ ($V = \sqrt{E/\rho}$). The depth of the reflector is calculated as $D = \Delta t \cdot V/2$; where D is the reflector depth and Δt is the time interval between two echoes.

In the impulse response test (also called the transient dynamic response), the transfer and the coherence functions and the spectrum of the receiver are typically recorded to calculate the depth of reflectors. The coherence function is used to judge the quality of data. The depth of reflector is calculated as $D = V/(2 \cdot \Delta f)$; where Δf is the frequency interval between two or more evenly spaced resonant peaks in the transfer function or the auto power spectrum plots.

The sonic echo/impulse response method is most applicable to columnar substructures on drilled shafts or other columnar deep foundations that are exposed above the ground or water. For concrete and steel piles, lengths can be predicted to within 5 to 10 percent. Brooks et al. (1992) estimate that the lengths of timber piles can be conservatively predicted within 15 percent because of the greater variation of compressional wave velocity in wood than in concrete. When embedded length to diameter ratios are greater than 20:1 to 30:1 in stiffer soils, there will be no identifiable bottom echoes because of excessive damping of the compressional wave energy in the sonic echo/impulse response tests. This problem is even worse for steel pipe and H-piles, which have a larger surface area per unit length than solid concrete, and timber piles, which are typically square and round in cross section.

Sonic Echo/Impulse Response Example Results

Figure 2 (top) shows results from sonic echo tests from a timber pile of a Franktown County, Colorado, timber bridge. On the basis of the calculations shown in the figure, a length of 29.8 ft was calculated for this pile, which agreed reasonably well with the 28-ft length from the engineering drawings. The length of the pile is calculated on the basis of a compressional wave velocity of 17,000 ft/sec measured from longitudinal ultrasonic pulse velocity (UPV) tests from a source on the pile top to a receiver on the pile side. The auto power spectrum of the accelerometer receiver used in impulse response testing of the pile indicated a frequency interval of 305 Hz between the peaks as shown at the bottom of Figure 2. This corresponds to a pile length of 27.9 ft, which is also in agreement with the actual length of 28 ft.

The sonic echo method also showed potential for identifying the depths of shallower, more massive abutments and piers, although not as conclusively as the ultraseismic method. Because sonic echo/impulse response testing has been extensively used for evaluating the integrity and length of concrete piles, drilled shafts, and timber piles (which are all columnar, rodlike elements), the test was expected to work best on columnar bridge substructure, and this was generally the case.

Sonic Echo/Impulse Response Theoretical Modeling Summary

Theoretical modeling studies were performed for the NCHRP 21-5 project by Dr. Jose Roesset and his student, Shu-Tao Liao, of the University of Texas at Austin, of sonic echo/impulse response tests of piles using 1-, 2- and 3-dimensional (1-, 2-, and 3-D) finite-element programs to model the application of a vertical force at the top-center of a drilled shaft and the receiver response toward the top-edge of a shaft (Liao, 1994). The basic difference between the three approaches is that the 3-D solution allows the energy to spread out into the larger radius top slab, and changes in impedance (cross-sectional area changes in the Figure 3 models) stand out sharply. Thus, the 3-D axisymmetric model (all elements are circular in cross section) much more accurately reflects the real 3-D wave propagation, and its results are shown here.

Two bridge substructure and superstructure models used for the theoretical modeling of pile systems are shown in Figure 3 (top a and b). Example results of a theoretical sonic echo plot are presented in Figure 3 (bottom a and b) for the case of the top slab having the same diameter as the column (top a) and the case of a 2-m-radius slab on top of a 0.4-m-radius column (top b). Comparison of velocity plots for the two geometries indicates significant differences in the recorded responses.

The Figure 3 bottom a record was theoretically generated (for the straight column case top a) by applying an impact to the top of the drilled shaft (Point A) and recording its response as waves traveled up and down the slab-column-shaft substructure. The letters in Figure 3 refer to wave reflection events from the various impedance changes (i.e., cross-sectional area changes). Examination of the 3-D velocity plot (bottom a) shows the initial downward break at time 0 that becomes positive at the time duration of the impact, *Td* at 1.5 ms. The record is then quiet until the strongest reflection event, ACA, occurs at about 4.2 ms. This is the time required for the generated tensile wave (first particle motion is tension) to travel 8 m upward at an assumed velocity of 3,793 m/sec for concrete to the top of the column, reflect almost 100 percent at the free column end and travel 8 m downward as a compressional wave to the receiver on top of the drilled shaft. The reflection from the bottom of the 12-m-long shaft occurs at about 6.3 ms as indicated by the ADA label. The bottom reflection event started downward as a compressional wave (first particle motion is compression) and was only partially reflected back up the shaft as a tensile wave–some of the energy was transmitted into the soil surrounding the shaft and at the bottom of the shaft by radiation damping.

Examination of the Figure 3 bottom b record (which is the velocity record for the 2-m-radius slab on top of the 0.4-m-radius column) now shows a positive break at ABA, which corresponds to the (reflected) upward traveling tensile wave encountering an increase in impedance, or fixed condition, at the column/slab (now larger than the column) interface. This is followed by a downward break corresponding to the reflection from the slab-air interface. Also of interest is that the bottom reflection is now completely obscured in the 3-D velocity plot at the ADA event. This is because so much energy was reflected back down the column from the larger area slab.

The theoretical finite element modeling clearly shows the complex vibrations that result from column to beam interactions can mask the desired echoes from foundation bottoms in sonic echo tests. Impulse response tests can be even more complicated, particularly if the testing is done in the middle of a column rather than toward the end.

Feasibility of Neural Networks for Sonic Echo/Impulse Response Analyses

Even experts find it can be very difficult, even impossible, to analyze the sonic echo/impulse response test results for most bridge substructures because of the highly variable geometry. Dr. Glenn Rix of Georgia Tech conducted a study (as part of this project) to evaluate the feasibility of using neural networks as a means of analyzing the more complex data produced by sonic echo/impulse response tests.

An artificial neural network is a highly interconnected collection of relatively simple processing elements that excels at pattern recognition tasks, including classification, forecasting, and mapping. In this context, determining the depth of an unknown foundation from nondestructive test results is viewed as a mapping problem—personnel can train a neural network to associate the length of pile with NDT results by presenting it with examples of the relationship. Neural networks are fast in their computations, learn from example and experience, and deal well with noisy input data. The extent to which a neural network will learn is bounded, in a sense, by the breadth of the training examples and how similar a particular input is to those training examples.

Two synthetically trained networks (based on 1-D finite element modeling for raw [i.e., normal] data) and enhanced impulse response mobility plots were used to predict pile lengths for steel H-piles in air and in the pilecap and for timber piles. Good agreement between predicted and actual depths was obtained for the steel pile in air and the timber piles. In both cases, the experimental mobility curves matched the theoretical mobility curves reasonably well. These are also the two cases for which 1-D modeling would predict the response accurately. In more complex cases (e.g., the pile in a pilecap and the pile with a beam on top of it) a 1-D model would not predict the response accurately. Not surprisingly, the neural network did not predict the depths of these piles well.

The study did demonstrate the feasibility of using artificial neural networks in analyzing the sonic echo/impulse response data for simpler rodlike shapes. Solid, broad training is important; for more complex substructure shapes, 3-D modeling is beneficial. Ideally, the best training would be on experimental results obtained from a broad spectrum of bridges with known foundation depths; this would allow experience to be incorporated into the neural network, but large amounts of experimental data would be required and this training is expensive.

Bending Wave Method and Results

The bending wave test is based on the reflection of dispersive (i.e., velocity is a function of wavelength) bending (flexural) waves in a timber pile resulting from the horizontal impacts of small- to large-sized hammers (Douglas and Holt, 1993). The passage of the flexural wave energy up and down the timber piles is monitored by two accelerometer receivers positioned a few feet apart and mounted on the heads of roofing nails driven radially into the pile. The two receivers are also used in determining the bending (flexural) wave velocity propagating up and down the pile. The receivers are in-plane with the hammer blow on the opposite side of the pile as shown in Figure 4.

Douglas and Holt (1993) first conceived of and used the short kernel method to analyze bending wave data for timber piles. The method is similar to narrowband cross-correlation procedures between the input (the hammer blow) and the output (receiver response). However, instead of measuring the hammer blow, a periodic function of 0.5 to 1 or more cycles is used as a "kernel seed," and several seeds of frequencies ranging from 500 to 4,000 Hz may be cross-correlated with the receiver responses. The short kernel method correlation procedure amplifies flexural wave energy responses with the selected seed frequency. The dispersion of the bending wave velocity is thus accounted for by calculating the bending wave velocity for each kernel seed frequency. The kernel frequencies are selected after transforming the receiver outputs to the frequency domain to identify the predominant frequencies in the receiver responses.

Echoes of bending wave energy from the toe and the head of the pile are manifested in the receiver outputs. By knowing the velocity of the bending waves, the length of the pile can be easily calculated (using a procedure similar to that for the sonic echo test) as $D = V_f \bullet \Delta t/2$; where D is the length of pile, V_f is the velocity of flexural (bending) waves at a given seed frequency, and Δt is time interval between two return echoes (or the time between first arrival and first echo with consideration of the receiver location).

Bending Wave Example Results

The same timber pile from the Franktown, Colorado, timber bridge was also tested with the bending wave method. This pile has an exposed surface approximately 9 ft long above the ground surface with a free end at top. Two accelerometers were placed in the transverse direction on the tested pile at distances of 42 in. and 84 in. from the top of the pile. The pile was excited in the transverse direction with a hammer hit at the top and the response of the pile was measured by the two receivers. The time records measured by the two receivers were then optimally cross-correlated with 1 cycle of a 500-Hz-frequency signal to determine the flexural wave velocity at that frequency to account for the dispersive effect. Figure 5 (bottom) shows the response of the receiver located at 42 in. from top with echoes separated by 22 ms. Figure 5 (top) shows the time records of the two receivers after cross-correlating them with the 500-Hz, 1-cycle wave from which a bending wave velocity of 2,480 ft/sec was calculated. A pile length of 27.3 ft was calculated; this compares well with the actual length of the pile (28 ft on the design drawing). The sonic echo results also showed a length of 29.8 ft, and the impulse response results showed a length of 27.9 ft, which are comparable to the value determined by the short kernel method analysis.

Bending Wave Theoretical Modeling Results

Theoretical modeling on propagation of flexural waves was done by Dr. Jose Roesset and his student, Chih-Peng Yu, of the University of Texas at Austin for this project (Olson et al., 1995). Their work indicated that flexural wave reflections do not follow the same pile-end boundary condition rules (i.e., for free to fixed conditions) as body wave reflections (i.e., compressional and shear waves) do. Body waves show the wave form changing sign (i.e., from compression to tension) after reflecting from a free end and show no sign change after reflecting from a fixed end (i.e., from compression to compression). However, flexural waves do not follow these rules. Thus, under some pile-end boundary conditions, picking the same sign peaks in the short kernel method (as recommended in the experimental research of Douglas and Holt [1993]) can introduce error into reflector depth predictions. Using a half-cycle kernel seed helps make the signs of the peaks more obvious so that changes in sign due to differences in end conditions are taken into account. The effect of embedment depths for concrete and timber piles was also examined by Roesset and Yu, and it was found that the bending wave method may be unable to detect lengths of deeply embedded piles. For shallowly embedded timber piles, the reflection from the soil surface may be as significant as that from the bottom, and with the shallow embedment, the reflection signals will overlap and make bottom depth identification difficult.

Ultraseismic Test Method and Results

As described in the previous two sections, the nondestructive sonic echo/impulse response (Davis and Dunn, 1975) and the bending wave methods (Douglas and Holt, 1993) are best used to determine the integrity and/or unknown depths of foundation elements of columnar, rodlike geometries with minimal overlying superstructure. For columnar, complex-shaped bridge substructure elements with large changes in cross-sectional area (impedance boundaries), however, the results of these tests are hard to interpret because of multiple reflections that occur at each impedance boundary.

In response to the difficulties encountered by the sonic echo/impulse response and the bending wave test methods for a large subset of non-columnar bridges or other complex civil structures, the ultraseismic test method[1] was conceived as a broader application of these methods during the NCHRP 21-5 study. The method can be used more broadly on bridge substructures because it is applicable to both columnar and more massive substructures. The ultraseismic test method uses multichannel recording of acoustic data, followed by digital filtering techniques adapted from the seismic exploration method, to process the data. Seismogram records are collected by using impulse hammers or vibrators (as the source) and accelerometers mounted on the accessible structure element at intervals of 1 ft or less (as receivers). The structure element itself is used as the medium for transmitting the seismic energy. All wave modes traveling down or reflected back (i.e., echoes from the bottom) are recorded by this method. For concrete structure elements, useful wave frequencies up to 4 to 5 kHz are commonly recorded.

Multichannel recording of ultraseismic data is used to isolate and enhance "desired signal" in the recorded seismograms. Desired signal often consists of reflection echoes from major defects or the bottom depth of the foundation. Data processing is used to isolate and enhance this signal, using some characteristic property, from other sources of coherent and incoherent (i.e., random noise) energy.

Although certain similarities exist between processing data from extended (i.e., geological) and bounded media, different processing schemes and philosophies must be adopted. For example, because of the access requirements, wave propagation velocity and source wave shape can often be deterministically measured rather than statistically estimated. The source of coherent noise contamination, like multiple reflection from many reflecting boundaries, is far more acute; and normal travel time moveouts are often small.

Moreover, the underlying physical principles for propagating stress waves in bounded media are different; this makes (algorithm) requirements for forward modeling, inversion analyses, and some filter designs different. For a medium with a bounded geometry, such as a bridge column, three types of stress waves are generated. In each type, longitudinal, torsional, and flexural waves are included (Kolsky, 1963). In longitudinal vibration, each element of the column extends and contracts along the direction of wave motion (i.e., along the column axis). In torsional vibration, each transverse section of the column remains in its own plane and rotates about its center. In flexural vibration, the axis of the column moves laterally in a direction perpendicular to the axis of the column. Although each wave type independently can provide information about the depth of the foundation or the presence of significant flaws within the bridge substructure, longitudinal (i.e., P-wave and compressional) and flexural (i.e., bending) waves are much easier to generate than torsional. Consequently, compressional and flexural wave energy is generated by orienting impacts to tested structures vertically and horizontally, respectively.

The following types of ultraseismic test geometries have been developed specifically for this problem:

- For a 1-D image of the foundation depth and for tracking the upgoing and downgoing events, the vertical profiling test method is used. Data appearance is similar to that of the vertical seismic profile method from seismic exploration. In this method (Figure 6), the bridge column or

[1] The ultraseismic method (ultraseismic refers to high-frequency seismic waves) was researched and developed by Farrokh Jalinoos, as co-principal investigator of NCHRP Project 21-5.

abutment is hit from the top or bottom (both vertically and horizontally), and the resulting wave motion is recorded at regular intervals down the bridge substructure element. Typically, three-component recording of the wave field is taken in order to analyze all types of ensuing wave motion. A vertical profiling line can be run in *both* a columnar (e.g., a bridge pier or pile foundation) and a tabular (e.g., a bridge abutment) structure.

• For a 2-D image of the foundation depth, the horizontal profiling test method is used. With this method, data appearance is similar to that of the surface seismic reflection method. In this method (Figure 7), the reflection echoes from the bottom are analyzed to compute the depth of the foundation. The source and receiver or receivers are located on the top or at any accessible length along the side of the substructure element, and a full survey is taken.

An important distinction between the two tests is that, in a vertical profiling test, the target is perpendicular to the survey line; in a horizontal profiling test, the target is parallel to the survey line. The vertical profiling lines are used to differentiate downgoing events from upgoing events on the basis of their characteristic time moveout and to measure their velocity accurately. A vertical profiling line is also used to link reflection events from the bottom to a corresponding horizon in a horizontal profiling section.

In the vertical profiling test (Figure 6), the impact point can be either at the top or the bottom of the receiver line. Vertical impacts to the substructure are comparatively rich in compressional wave energy, although considerable dispersive flexural/Rayleigh surface modes are also generated. Horizontal impacts are rich in flexural wave energy when the impacts generate wavelengths longer than the thickness of the substructure element. Impacts that generate wavelengths shorter than the thickness will propagate with Rayleigh wave velocity, and longer wavelengths will propagate at a slower flexural wave velocity.

In the horizontal profiling test (Figure 7), the source and receiver can be close to each other on the top at an optimum offset to reduce interference from Rayleigh surface waves. The source and receiver are then marched together in order to obtain a common offset dataset. A full survey using all combinations of source and receiver locations can also be taken, but this is more data intensive. Alternative survey lines can also be obtained by moving the source or receiver lines to a different depth within the structure as dictated by accessibility requirements. In the alternative survey line shown in Figure 7, reflection events from the bottom (or top) as well as the backside of the wall can be analyzed to give useful information about the structure.

Vertical Profiling Example Results

Figure 8 shows the source/receiver layout for the vertical profiling testing from a pier (i.e., columnar) foundation of a bridge. The 12-lb hammer source was located at the bottom of the pier near the grade and was struck both vertically and horizontally (i.e., perpendicular to the axis of the column). Two horizontal hammer hits were used from opposite sides of the column. Three-component accelerometers were mounted on the side of the column at 0.5-ft intervals *above* the source location. Thus, a full six-component dataset from two source directions and a three-component recording was obtained. This allows for a near-full study of the dynamics of wave motion for all longitudinal, torsional, and flexural wave modes in this type of semicolumnar geometry.

The field recordings from a horizontal 12-lb hammer hit and horizontal receiver responses (perpendicular to the axis of the column) are shown in Figure 9. Each trace indicates the record from a different survey location on the column—with trace 2 corresponding to 0.5 ft and trace 21 at 10 ft above the source location as indicated in Figure 8. Figure 9 shows the first 15 msec of data at a 4-μsec sampling interval. All data were de-biased (i.e., direct current shift was removed and data were bandpass zero-phase-filtered [using a 0-0.5-3-4 kHz trapezoidal filter] and automatic gain controlled) to enhance weaker arrivals. The following events, similar to a typical vertical seismic profile record, are evident in Figure 9:

1. Upgoing flexural events (including reflections from the footing) with a linear positive

moveout with travel time increasing with distance and

2. Downgoing flexural events (including reflections from the beam) with a linear negative moveout with travel time decreasing with distance.

The reflector event indicates a flexural wave reflection at 4.9 ft below the source point, which corresponds to the (bottom) depth of the pilecap footing. This gives a depth below the top of the beam of 30.2 ft, which compares well with the 31.1-ft distance shown on the bridge plans.

Because of the geometry of the pier at its top, both the downgoing and upgoing events are cyclical (ringy). In the frequency domain, the multiple reflection of the ringy seismic wavelet gives rise to characteristic resonant peaks of a column. By calculating the distance between the resonant peaks (Δf) and knowing the velocity (V_c) of compressional wave motion, the unknown depth of the foundation (d) can be calculated by $d = V_c/(2\Delta f)$. This is the basis for the impulse response NDT method for a depth (length) determination of a column as previously described.

No evidence of the steel piles underlying the concrete pilecap footing was apparent in the vertical profiling results. Vertical profiling test lines can also be run from the side of a massive abutment and other tabular type structures.

Horizontal Profiling Example Results

This method was developed for use on massive abutment and wall substructure elements; they typically have greater widths of top or side surface access, which permits a line of receivers to be placed at the same elevation. The source and receiver locations used on one side of a pier of a Weld County, Colorado, bridge are shown in Figure 10, which shows a wall-type pier substructure on a strip footing on steel H-piles. This pier wall was a good site for the initial research on the horizontal profiling method. An automated and repeatable prototype tool was developed, which allowed for signal averaging of the received signal. The tool consists of an electromechanical solenoid source generating an impulse signal by tapping with a 0.2-lb force transducer. The automated 0.2-lb source was fired horizontally (perpendicular to the wall surface) from one side of the wall (placed at 3.5 ft from the top), and horizontal accelerometers were placed on the opposite side of the wall directly across from the source locations. The source and receivers were then moved simultaneously at 1-ft intervals along the width of the wall foundation.

Figure 11 shows the data for 32 station locations across the pier length for a single-hit source. These data were corrected for direct current bias, bandpass-filtered (0-0.7-4-5 kHz trapezoidal), and automatic gain controlled to aid with the display of the weaker, late (i.e., deeper) arrivals. The approximate locations of the bottom reflections are indicated in the figure. Data quality is comparable to a set obtained with manual (hammer) hitting. The bottom depths in Figure 11 were obtained after comparing the horizontal profiling line with a vertical profiling line (Olson et al., 1995) obtained from the corner of this pier. In a vertical profiling section, events coming from above the survey line (like the superstructure) are distinguished from those coming from below (like the foundation bottom) on the basis of the slope of their time moveout curve. This dataset indicated weak, bottom P-reflections, which seriously reduced the predominance of surface wave energy in the data. Optimum survey geometries and effective means for suppressing the Rayleigh waves are the subject of current continued research; this research may lead to use of techniques different from those employed in surface seismic exploration.

SASW Method and Results

The SASW method, which measures the variation in surface wave velocity with depth in horizontally layered media, was initiated at the University of Texas at Austin in the early 1980s with funding from the Texas Department of Transportation (Heisey et al., 1982; Nazarian and Stokoe, 1984). The method has been successfully applied to determine the shear wave velocity profiles for soils (Stokoe et al., 1988) and pavement systems (Rix, 1988; Aouad, 1993). Testing is per-formed by impacting the surface and recording the passage of predominant Rayleigh (surface) wave energy, using two receivers. A series of receiver spacings is used, and testing is performed in for-ward and reverse directions at each receiver spacing. A plot of the surface wave velocity versus wavelength is called a dispersion curve. Once the dispersion curve is determined, one can obtain the shear wave velocity profile of the structure or soil being tested.

In the application of this test to bridge substructures (Figure 12), one source and two receivers are placed on top of the abutment so that the distance from the source to the first receiver is equal to the distance between the two receivers. A dynamic signal analyzer is used to capture and process the receivers' outputs [denoted by $x(t)$ and $y(t)$ for receivers 1 and 2, respectively]. Then, $x(t)$ and $y(t)$ are transformed to the frequency domain [$X(f)$ and $Y(f)$] through the employment of a Fast Fourier Transform. $X(f)$ and $Y(f)$ are used to calculate the cross power spectrum between the two receivers, denoted by G_{XY} [$G_{XY} = X(f)^* \cdot Y(f)$ where the * symbol denotes the complex conjugate)]. The surface wave velocity and wavelength, $V_R\,(f)$ and $l_R\,(f)$, are determined as: $V_R\,(f) = D/t\,(f)$; and $l_R\,(f) = V_R\,(f)/f$, where $t(f)$ = time delay between receivers as a function of frequency (= phase shift of the cross power spectrum in degrees divided by frequency), D is the distance between the two receivers. A plot of surface wave velocity versus wavelength is called a dispersion curve and is used in determining the depth of the foundation.

SASW Example Results

The SASW can determine the thickness or depth of bridge abutments as long as certain physical conditions are met: the foundation must be massive and must have an exposed, fairly flat ledge or top surface on which impacts are applied and a pair of receivers is placed. As an example of data (Aouad et al., 1996), an SASW test array for the west abutment of a bridge in Connecticut is shown in Figure 13. Three receiver spacings of 3, 6, and 12 ft were used. The composite curve from the three receiver spacings is shown in Figure 14. The dispersion curve shows a two-layer system with two different velocities. The first part, up to a wavelength of 9 ft, shows a surface wave velocity of 7,700 ft/sec which is indicative of concrete velocities. The second part shows velocities of approximately 4,000 ft/sec for wavelength greater than 9 ft, which is indicative of velocities of medium-hard rock. Therefore, it can be inferred from the SASW measurements that the depth of the abutment is approximately 9 ft.

Dynamic Foundation Response Method

This test method (Figure 15) was proposed as a means of differentiating between shallow foundations and foundations with piles (or other deep foundations) underlying the visible bridge substructure. Although the method is unproven for this use in bridges, it is based on the dynamic analysis theory for vibration design of foundations (soil dynamics) and geotechnical analyses of foundations subjected to earthquake loading. The dynamic analysis theory and the geotechnical analyses are based on the theoretical work of Novak (1976) and Novak and Aboul-Ella (1978). Novak analyzed the problem of a simple shallow footing foundation with and without piles for vertical and horizontal modes of vibrations on the ground surface and embedded foundation. Pile foundations underlying a pilecap are more rigid and exhibit higher vibration amplitudes and resonance frequencies than shallow footing foundations in the same soils. In NCHRP Project 21-5, dynamic foundation response measurements to determine resonances were performed on all applicable bridges (i.e., nonvisible pile bridges). Theoretical modeling of the specific vibration response to match field tests as well as free-vibration analyses were performed for selected bridges to be compared with the measured results. Some differences in the dynamic foundation responses were found between the shallow and deep foundations. However, bridge substructure responses were influenced by the effects of the visible substructure and the superstructure. Enough potential was seen that an FHWA research project is being conducted (Olson et al., 1996).

Borehole Parallel Seismic Test Method and Results

The borehole parallel seismic method was researched and developed specifically to determine the depths of unknown foundations by the CEBTP research organization headquartered in Paris, France (Stain, 1982). This method (Figure 16) consists of impacting the exposed foundation top (or other accessible substructure element) either vertically or horizontally with an impulsive hammer source to generate compressional or shear waves that travel down the foundation and are transmitted into the surrounding soil. The wave arrival is tracked at regular intervals by a hydrophone receiver suspended in a water-filled, cased borehole (the past procedure) or by a clamped, three-component, geophone receiver (the new procedure) in a cased or uncased borehole. The boring is drilled typically within 3 to 5 ft of the foundation edge and should extend at least 10 to 15 ft deeper than the anticipated and/or minimum required foundation depth. The foundation bottom, or a major defect in the foundation, will appear as a low velocity zone with reduced signal amplitude. The foundation depth (or defect depth) is determined by plotting initial arrival times of the wave energy to the downhole receiver against test depths and observing the depth where change of slope in the travel time plot occurs.

This procedure works well for purely columnar foundation elements and where the surrounding soil velocity is constant (as is the case in fully saturated soils). For soils with varying velocities, the traces in the section must be statically shifted to correct for the variation in the (measured) soil velocities. Typically, at the tested sites, the soil compressional velocities increased from about 1,000 ft/sec to 5,000 ft/sec (corresponding to 100 percent saturated soil, which is the nominal velocity of water at the water table) as well as at bedrock. Moreover, the bridge foundation shapes were mostly noncolumnar (in cross section), because of the existence of a footing or pilecaps. In these cases, the conventional parallel seismic analyses do not always work.

However, it was observed that the bottom of the foundation can act as a strong (secondary) source of energy, especially in more massive foundations. The foundation tip acts as a point diffractor in emitting both upward and downward traveling waves into the borehole. This diffraction event is best observed by using three-component geophones in a cased, grouted borehole and was not as readily observable in hydrophone sections because of high contamination from the tube waves energy. The diffraction results in a steep, V-shaped, hyperbolic event in the recorded seismic section. The bottom of the foundation is then identified by noting the depth where the peak of the hyperbolic event occurs.

Parallel Seismic Example Results.

As a typical example of the parallel seismic test with geophones, Figures 17 and 18 show test geometry and field data respectively from the concrete pile pier of a highway bridge in Texas. The wave energy generated by the horizontal blow is mostly propagated in the flexural wave mode in the pile caisson; this becomes refracted as a shear wave in the borehole. The SP positions shown across the tops of the figures refer to depths of the records in feet from the top of borehole. Therefore, SP 2 refers to a receiver location of 2 ft below the top of the borehole. At this site, soil shear velocity increased from 850 ft/sec to about 1500 ft/sec at the bedrock. A sharp point diffractor event in the flexural wave energy from the foundation tip is indicated at a depth of 32.5 ft from the top of the borehole; this predicts a depth of 33.2 ft below the pilecap for the pile bottom. It agrees fairly well with the plan depth of 34 ft.

Parallel seismic data were also obtained from the same highway bridge in Texas using hydrophone receivers. Figure 19 shows the hydrophone results after automatic gain controlled amplification. Review of this figure shows a fast velocity (corresponding to the concrete pile below about 16 ft [below the water table, saturated]) and then a slower velocity (corresponding to the shale). The 33-ft depth of the pile below the top of the borehole is clearly apparent in the figure. This predicts a depth of 33.7 ft to the pile tip below the pilecap, which also agrees very well with the plan depth of 34 ft. Note the predominance of both upgoing and downgoing tube wave events (with a velocity of 1,660 ft/sec) in the hydrophone section.

The three-component parallel seismic method provided foundation depth data for the widest range of bridge substructures and geological conditions by using geophone receivers in a grouted, cased boring. By using three-component geophones in cased borings with good contact between the casing and soils, improved quality of parallel seismic results was obtained at sites with variable soil velocity conditions.

Borehole Sonic Test Method

The borehole sonic method was proposed for research as a promising, but unproven, approach for determining unknown bridge foundation depths (and perhaps even foundation geometry). This method ideally involves lowering a source and a receiver unit in the same borehole and measuring the reflections of compressional or shear waves from the side of the bridge substructure foundation using essentially horizontal raypaths (Figure 20). No borehole sonic tool suitable for determining unknown bridge foundation conditions exists; conventional sonic logging tools have limited depth of penetration around the borehole. A three-component downhole source and a three-component geophone receiver for crosshole seismic tests were used in two pairs of separate borings at a wall-shaped caisson foundation in Texas (the largest bridge substructure element tested, and consequently the best target). The results of these tests showed promise for the borehole sonic method because the reflections obtained indicated, by their timing, the presence of the caisson foundation. However, no clear reflections were obtained with a commercial singlehole sonic logging tool for measurement of compressional and shear wave velocities around the borehole. The limited tests showed promise, but considerable research would be required to develop a borehole sonic tool for use in determining unknown foundation conditions.

Borehole Radar Test Method and Results

As with the borehole sonic test, borehole radar had not previously been used to determine unknown foundation conditions. This test uses a transmitter/receiver radar antenna to measure the reflection of radar echoes from the foundation (Figure 21). The radar signal is propagated as an electromagnetic wave in the soil in response to the displacement currents generated from the applied electric field. Displacement currents, generated from intrinsic dipole moment distribution in a dielectric material, are in contrast with conduction currents created from the movement of free electrons in a conducting material. Reflections occur in the subsurface at interfaces with a contrast in dielectric properties (i.e., changes in electrical impedance). Electrical impedance changes are governed by changes in relative permittivity or dielectric constant in the ground. For a vertically incident plane wave impinging on a boundary between two materials of dielectric constant K_1 and K_2, a reflected wave is returned upward with an amplitude governed by the reflection coefficient, *R*, given by: $R = \left(\sqrt{K_1} - \sqrt{K_2}\right) / \left(\sqrt{K_1} + \sqrt{K_2}\right)$. The reflected portion of the signal is recorded by the receiver antenna and processed with various gains and filters.

For the borehole radar test (Figure 21), the reflection from the soil/foundation boundary (i.e., the abutment) is of interest. The soil dielectric constant of a soil is a direct function of the water content—that is, as the water content increases, so does the dielectric constant. This results in a decrease of radar velocity or longer travel paths through the same thickness of soil. The increase of water content also increases the electrical conductivity in soil, which increases the intrinsic attenuation of the radar signal. This results in reduced depth of penetration of the radar signal. Typical radar records include a constant transmit pulse at early parts of the record as well as the deeper reflected events from the subsurface or the bridge abutment. Thus, environmental factors (such as salt water, conductive soils, ground moisture conditions, and buried electrical power lines) can critically limit and/or confuse radar signals.

Borehole Radar Example Results

Borehole radar did not work at sites with conductive clayey soils. For example, Figure 22 shows the field testing geometry of a battered steel BP pile of an Alabama. Figure 23 shows an example borehole radar result from the battered pile. The pile tip and the angled reflection corresponding to the battered pile are relatively clear in Figure 23. Steel is an extremely good reflector of the radar signal; therefore, it serves as an ideal target for this method. The horizontal axis displays depth measured from the top of the boring with depth increasing from right to left. The small tick marks on the top of the figure indicate depth marks in 1-ft increments. In this figure, the depth marks increase from 0 to 50 ft. The vertical axis displays time in nanoseconds (ns), which increase from top to bottom.

Several events are discernible in these types of sections. The first event to note in Figure 23 is a constant amplitude and travel time "transmit pulse" event at about 10 ns, which is the result of the direct electrical coupling between the transmitter and receiver antennae. A second series of events are believed to result from the effect of soil saturation at water-table depth. This ringy signal is indicated at about 17 ns where a linear event is observed, which increases to about 20 ns at 28 ft (the water table) from the top of the boring. The third series of events in this figure results from the normal incidence primary reflection events (and their multiples) from the steel pile, which are passed through the lower-velocity, saturated sediments below the water table and higher-velocity, dry soils above. The slanted reflection event increases in time at shallow depths because of the tilt of the battered pile. The bottom depth of the steel pile is identified where the reflection events are substantially reduced in amplitude at 35 ft from the top of the boring. This would predict a pile length of 31 ft, which is 8 ft shorter than the 39-ft pile lengths from driving records. This effect is more apparent in the color video monitor display in the field or from a color plot section.

Induction Field Method

In the induction field method (Wright, 1979; Beattie, 1982), an alternating current flow is impressed into a steel pile (or the reinforcing bar in a reinforced concrete pile) from which the current couples into the subsurface and finally to a return electrode (Figure 24). The return electrode can be another pile, or it can be a pipe or piece of reinforcing bar driven into the ground. A receiver coil suspended in a nearby boring is then used as a sensor of the magnetic field induced by the alternating current flow in the pile. By plotting the magnitude of the induced voltage against the depth of the search coil, an indication of the length of the pile is provided, because the field is weaker below the piles. The basic limitation of this method is that the foundation substructure must contain electrically continuous steel for its entire length, and the steel used must be accessible at the top to allow the electrical connection. Steel pile and (electrically continuous) reinforced concrete pile depths can be obtained from this type of survey. This method was not examined in this study but will be investigated in the Phase II research.

CONCLUSIONS AND SUGGESTED RESEARCH

The results of this research indicate that the ultraseismic test has the broadest application of the surface tests (no boring required) for determining the depths of unknown bridge foundations. The sonic echo/impulse response tests were also useful for predicting depths of columnar (for example, partially exposed piles) foundation substructure; the sonic echo test was effective on more massive abutments, but was not as conclusive as the ultraseismic test. The bending wave method showed good agreement with sonic echo/impulse response tests of the slender timber piles, but use of the bending wave method is limited to very slender foundations. The dynamic foundation response method showed only minor changes in vibration resonances for footings with and without piles. Another method, SASW, was attempted on Connecticut DOT bridges, where access permitted. This method worked well for these shallow, more massive wall abutments. The major limitation of the surface methods is that none of them can detect the presence of piles underlying a buried pilecap or a more massive abutment or pier wall. Nevertheless, the new ultraseismic method was able to determine the depths to the first significant substructure change (i.e., footing on soils or pilecaps, bottoms of abutments, bottoms of piers), especially for shallow footings. However, no indication of secondary weak echoes from steel pilings underlying a buried pilecap or a more massive abutment or pier wall was observable in the data.

In the case of buried pile foundations, the parallel seismic test was found to have the broadest application for varying substructure and geological conditions, but it requires drilling boreholes. Higher quality data may be obtained by using clamped geophone receivers in grouted, cased borings instead of using only hydrophone receivers that are suspended in a water-filled borehole. The commercial borehole radar tool also worked relatively well at sites where soils were nonconductive. The borehole sonic feasibility tests showed promise, but much research remains before a fully evaluated commercial tool can be developed.

NCHRP Project 21-5 (2)—researching and developing equipment, field techniques, and analysis methods for the technologies with the most promising immediate applications—is underway. The specific NDT methods to be further researched and developed are the surface method of the ultraseismic (including sonic echo/impulse response and the bending wave methods) and the borehole methods of parallel seismic and induction fields. Another FHWA study is being conducted (Olson et al., 1996) in which the modal response of bridge substructures from dynamic excitations is being investigated. This study will build on the dynamic foundation response tests.

Summary evaluations of the promising NDT methods in Phase I are presented in Tables 1 and 2 for the surface and borehole tests, respectively.

ACKNOWLEDGMENTS

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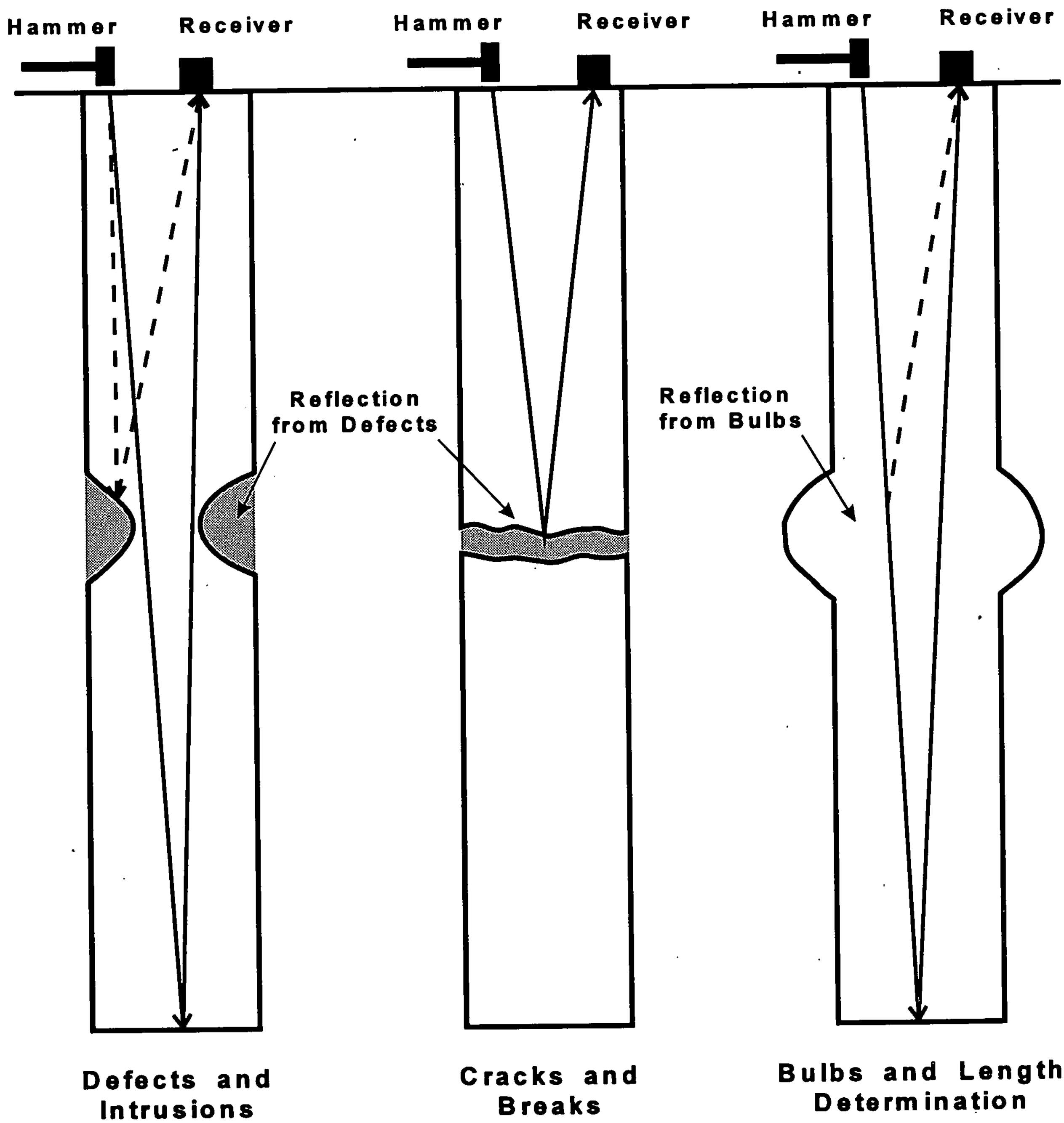


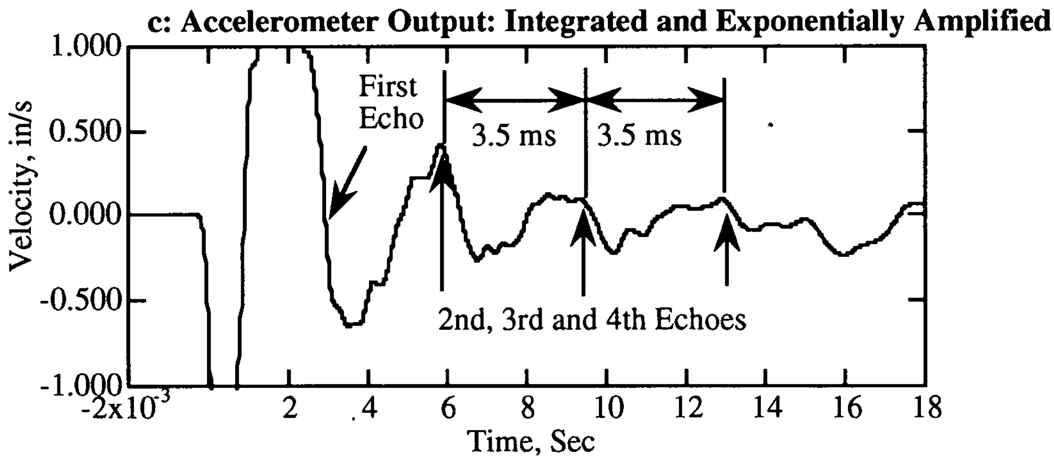
Figure 1. Sonic echo/impulse response test methods.

Length Calculation:

$\Delta t = 3.5$ ms

$V = 17{,}000$ ft/sec

Length of Pile = $V \times \Delta t / 2 = 17{,}000 \times 3.5 \times 10^{-3} / 2 = 29.8$ ft



Length Calculation:

$\Delta f = 305$ Hz

$V = 17{,}000$ ft/sec

Length of Pile = $V / (2 \times \Delta f) = 17{,}000 / (2 \times 305) = 27.9$ ft

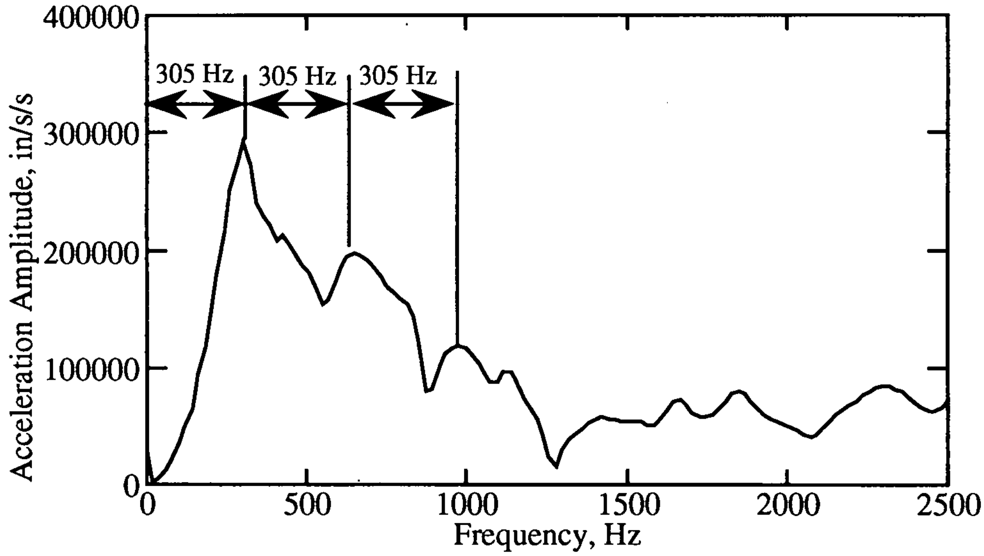


Figure 2. Sonic echo test results (top) after integration and exponential amplification, and auto power spectrum of accelerometer output from impulse response test (bottom) from an exposed timber pile of a bridge in Franktown, Colorado.

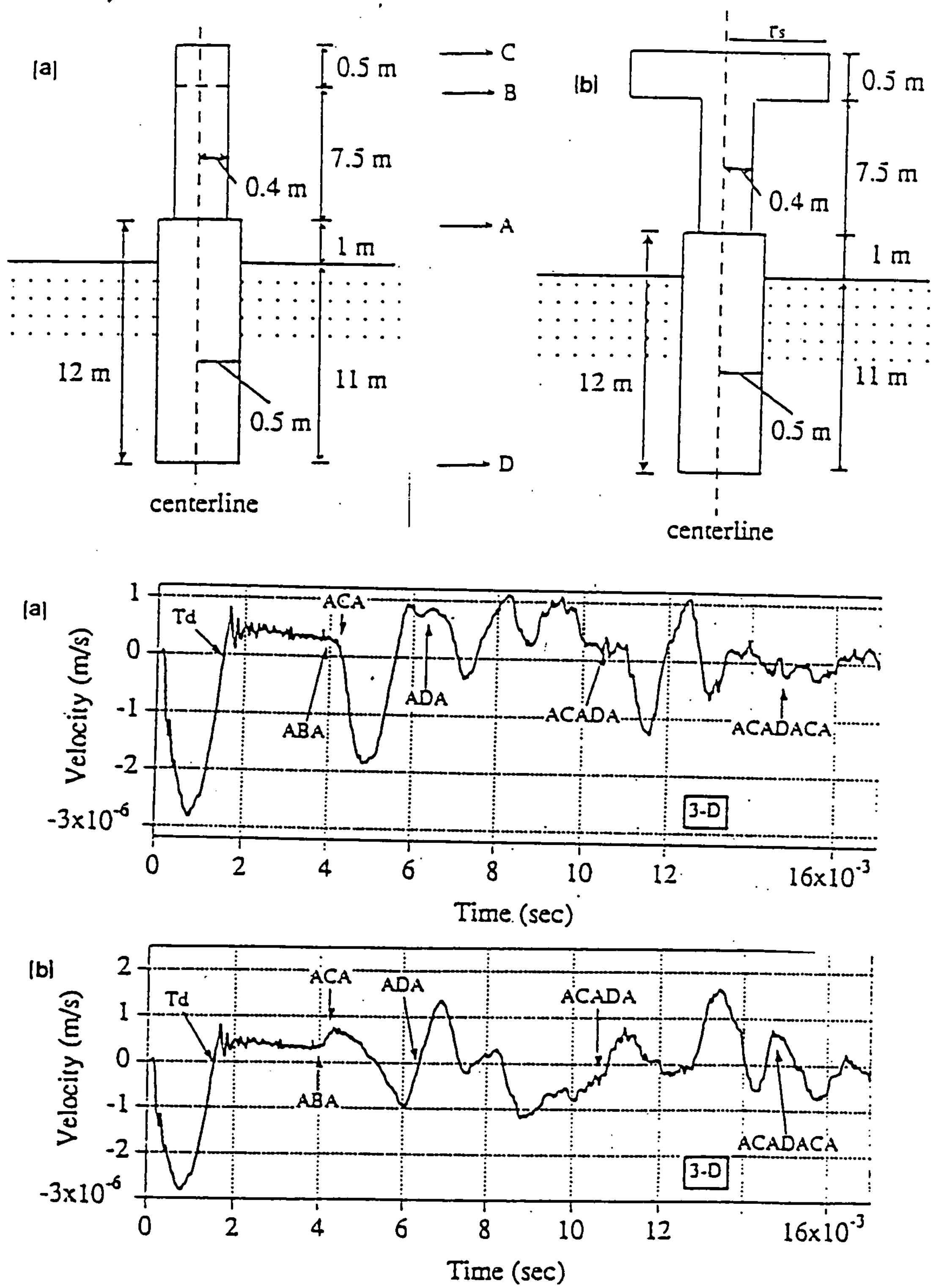


Figure 3. Bridge substructure model for finite element analysis of sonic echo tests from the top of the drilled shaft (top a and b) and theoretical results (bottom a and b).

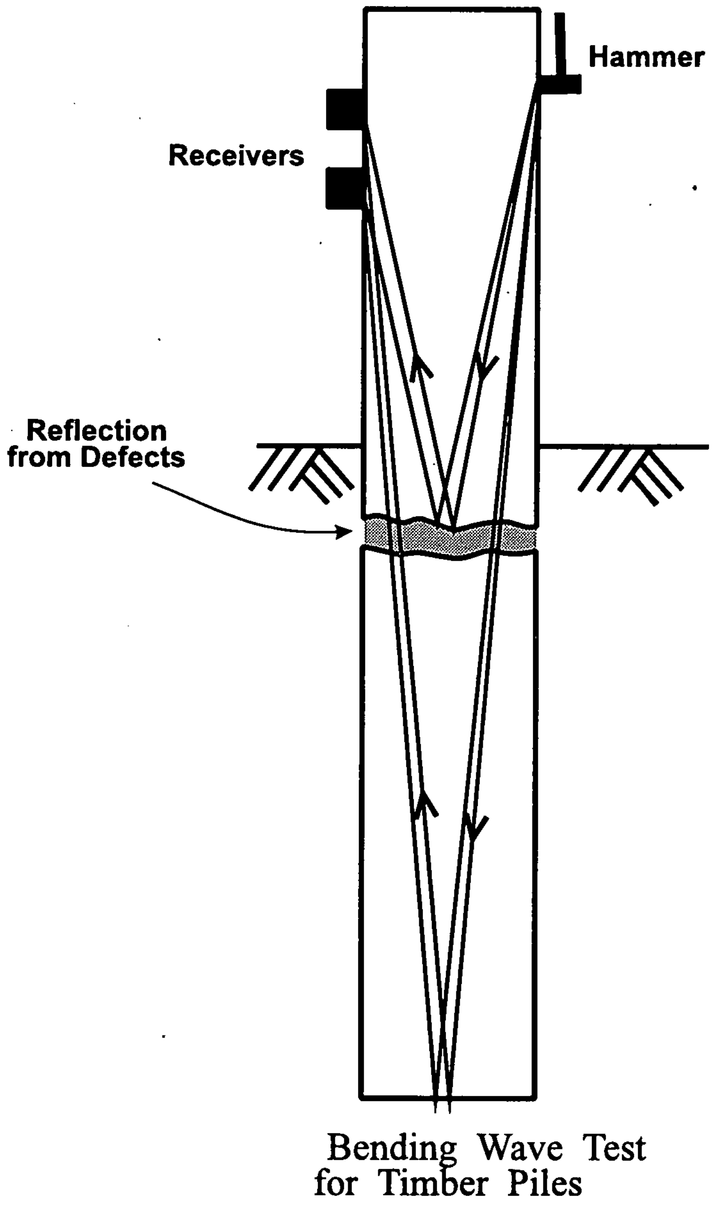
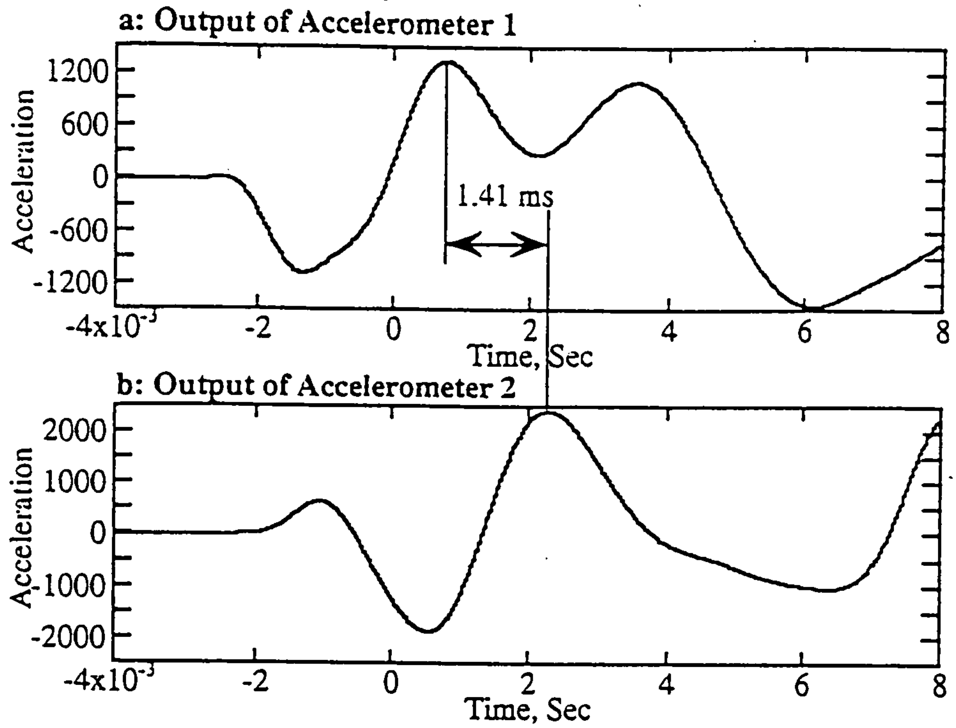


Figure 4. Bending wave test method.

Velocity Calculation:
$\Delta t = 1.41$ ms
R1-R2 = 42 inches = 3.5 ft
Bending Wave Velocity = R1-R2 / Δt = 2,480 ft/sec



Length Calculation:
$\Delta t = 22$ ms
V = 2,480 ft/sec
Length of Pile = V x Δt / 2 = 2,480 x 22 x 10^{-3} /2 = 27.3 ft

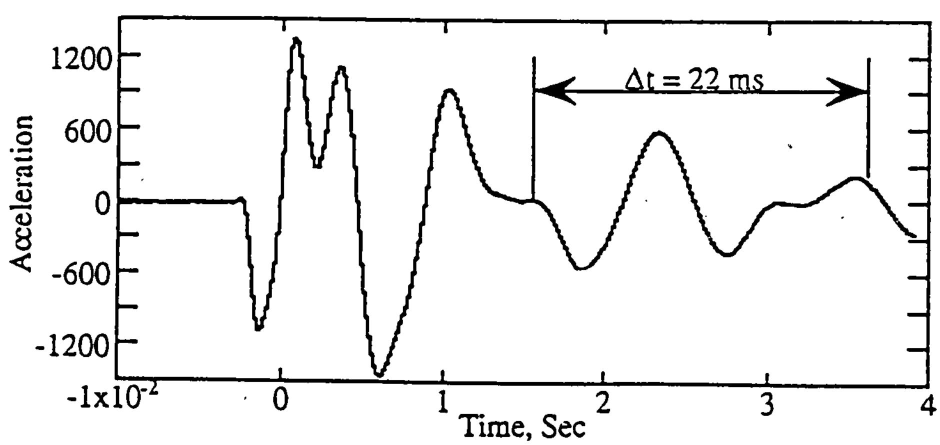


Figure 5. Bending wave pile results for a timber pile of a Franktown County, Colorado, timber bridge.

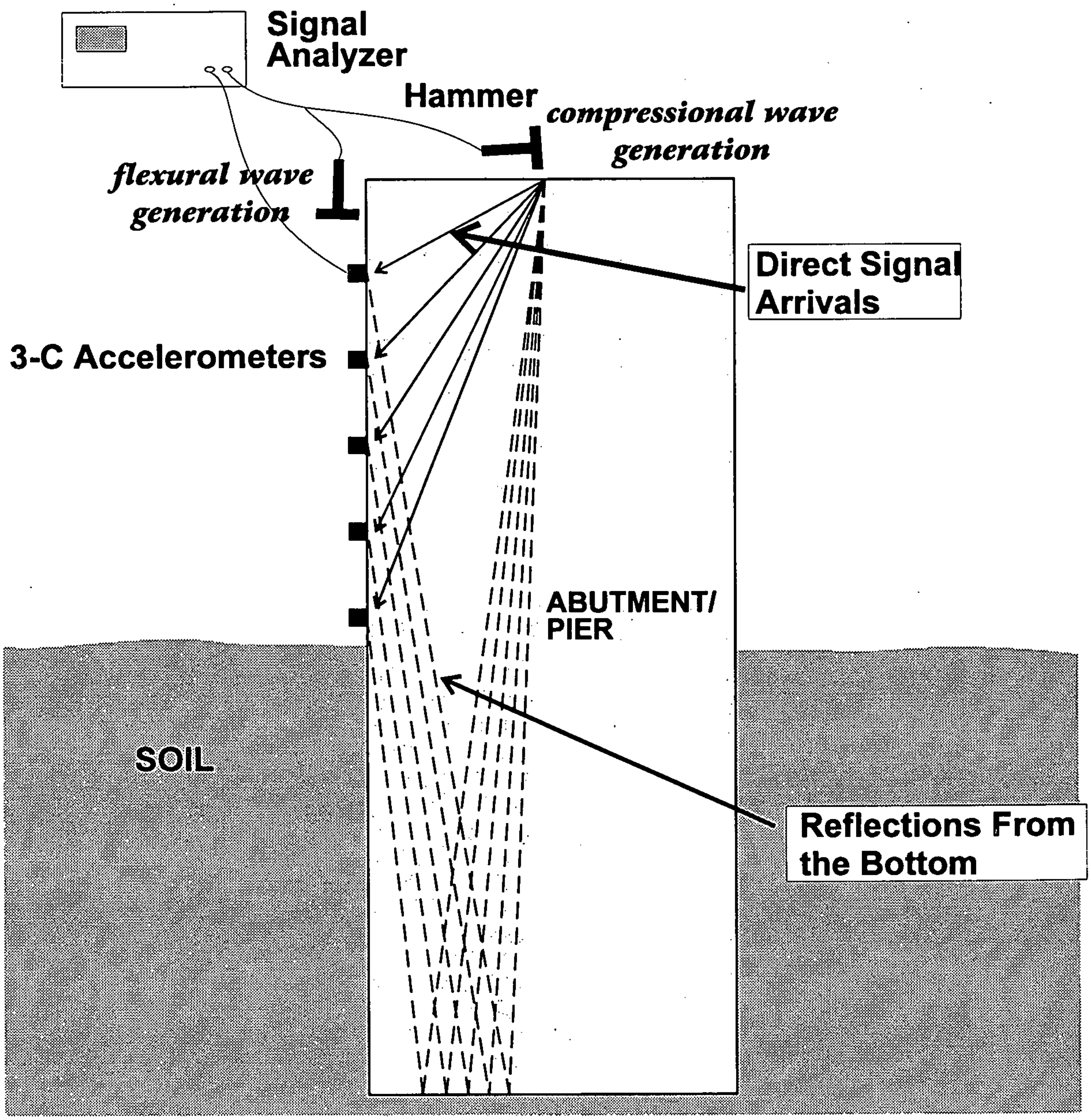


Figure 6. Ultraseismic method; vertical profiling (VP) test.

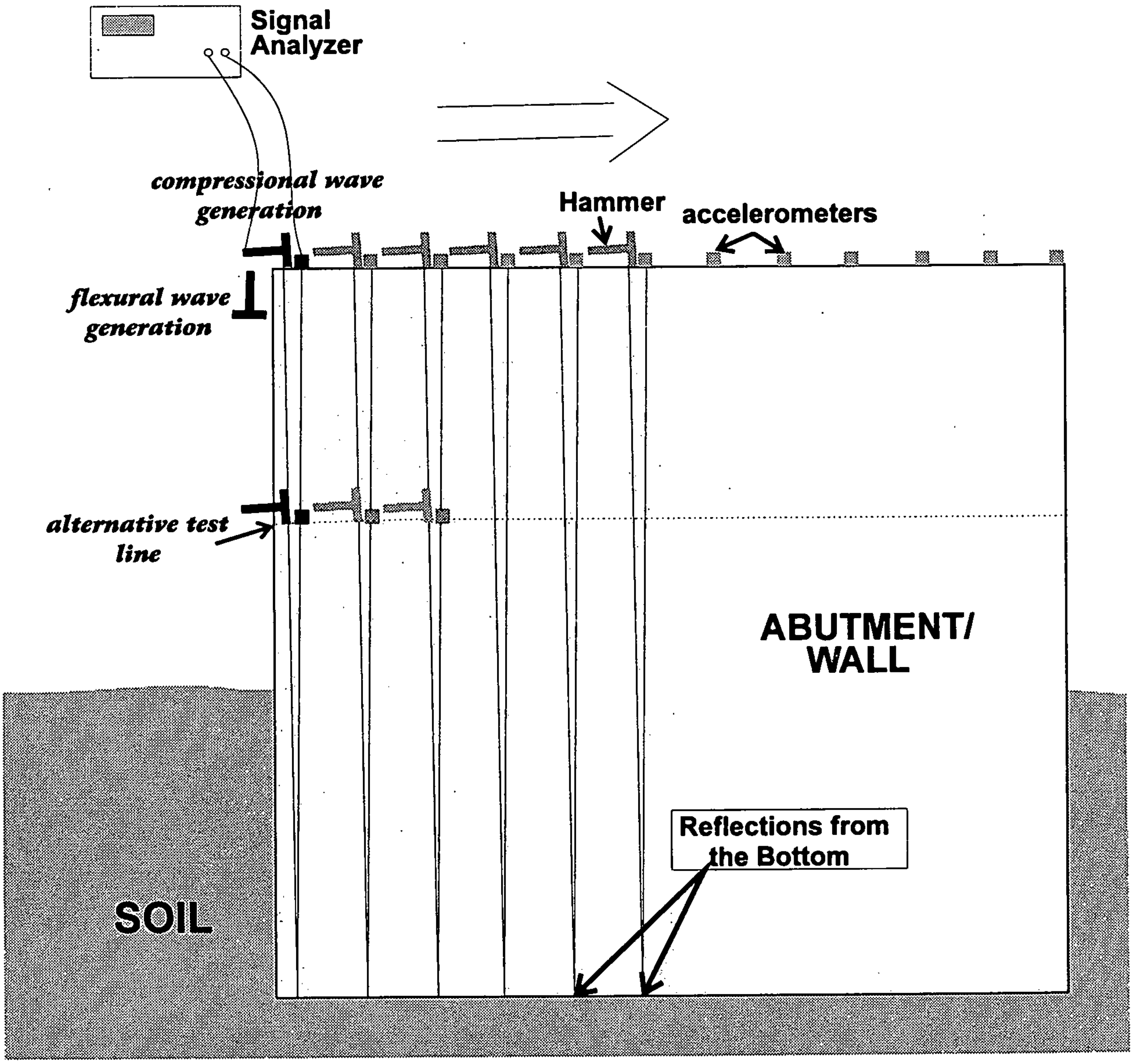


Figure 7. Ultraseismic method; horizontal profiling (HP) test.

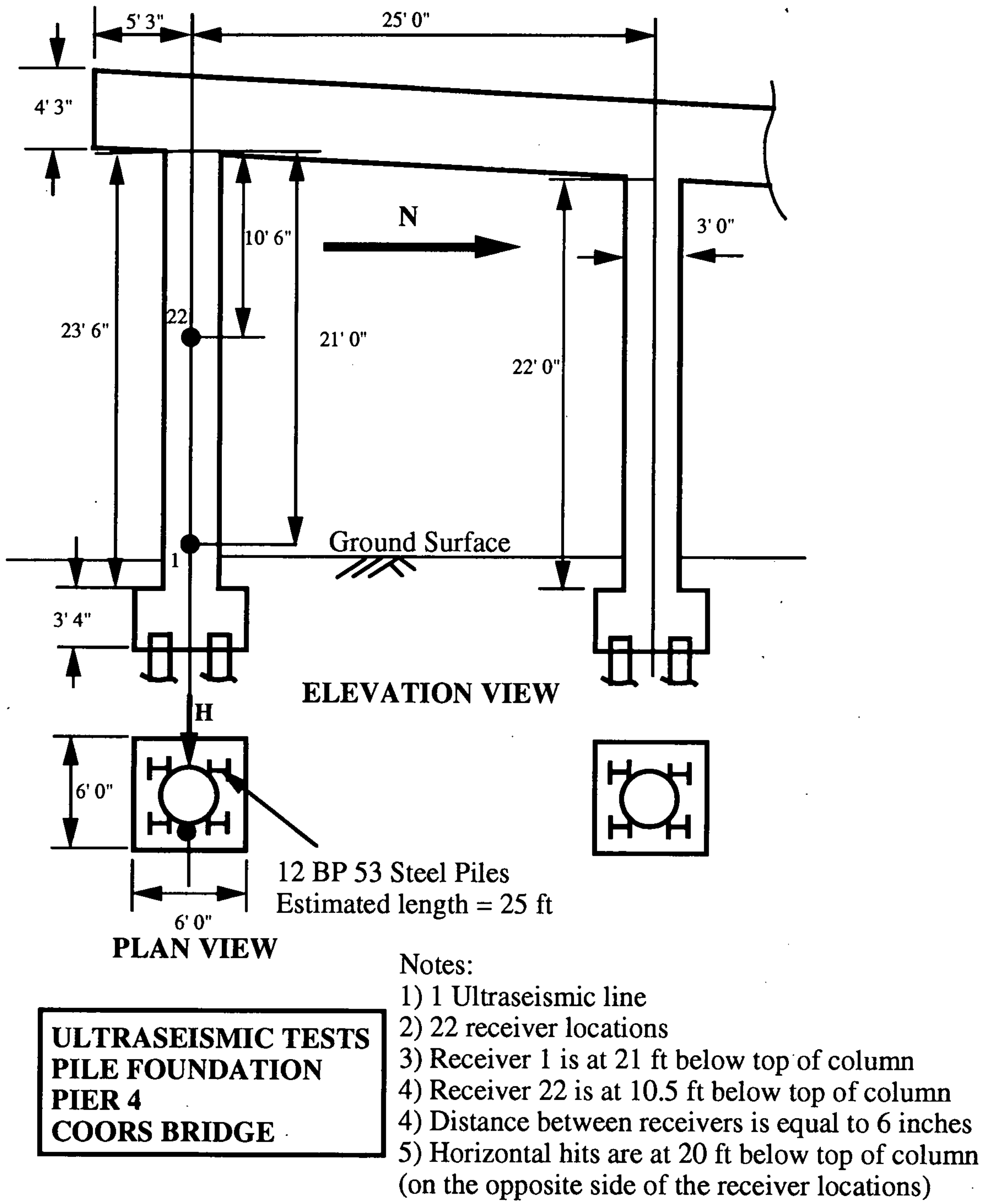


Figure 8. Source/receiver layout for ultraseismic VP tests with impacts at the bottom of a Colorado bridge pier.

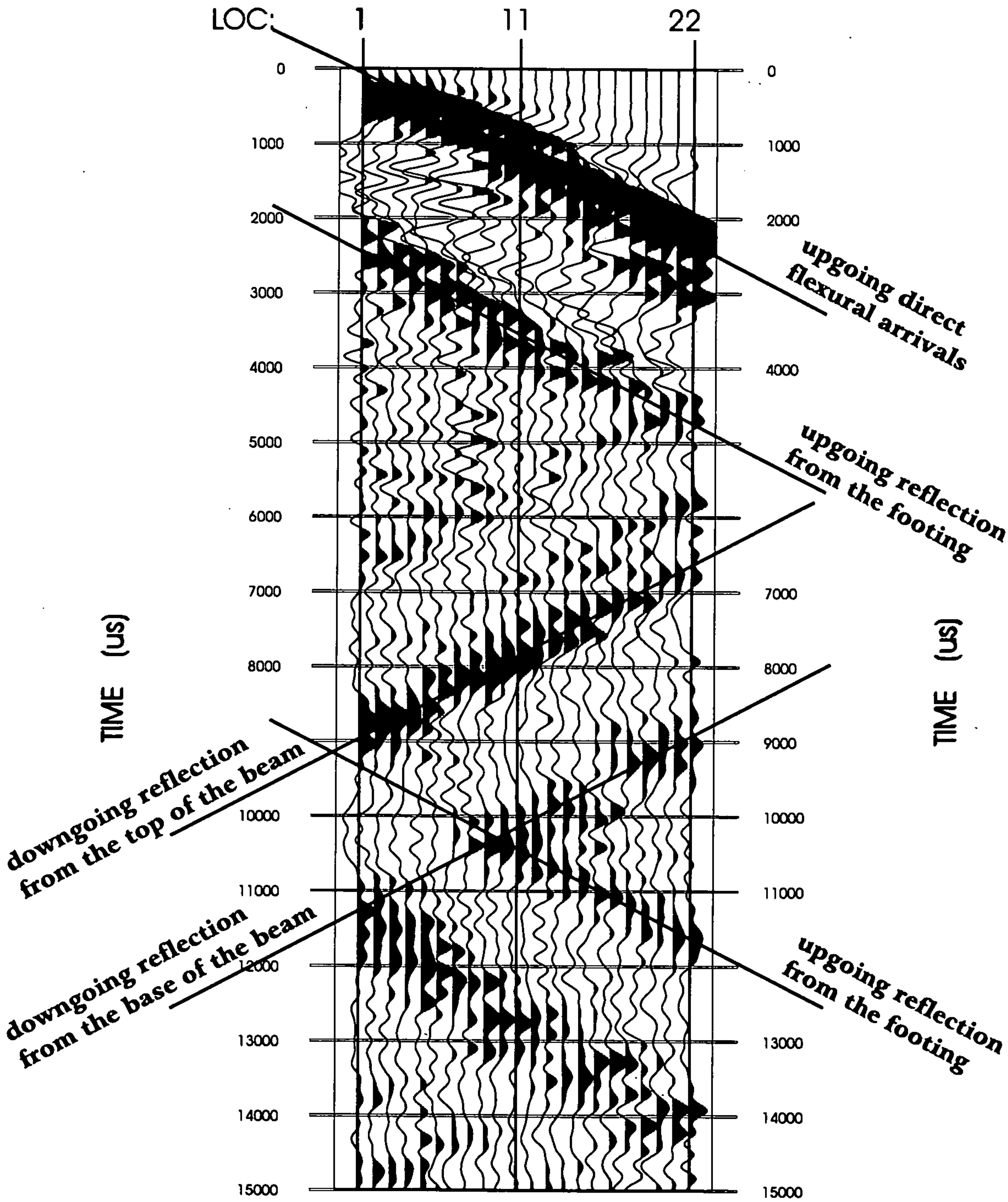


Figure 9. Ultraseismic VP records of flexural waves from horizontal impacts at the top of a bridge pier with a 3-lb hammer and 22 horizontal receiver locations in the same vertical plane on the south column (see Figure 8 for test geometry).

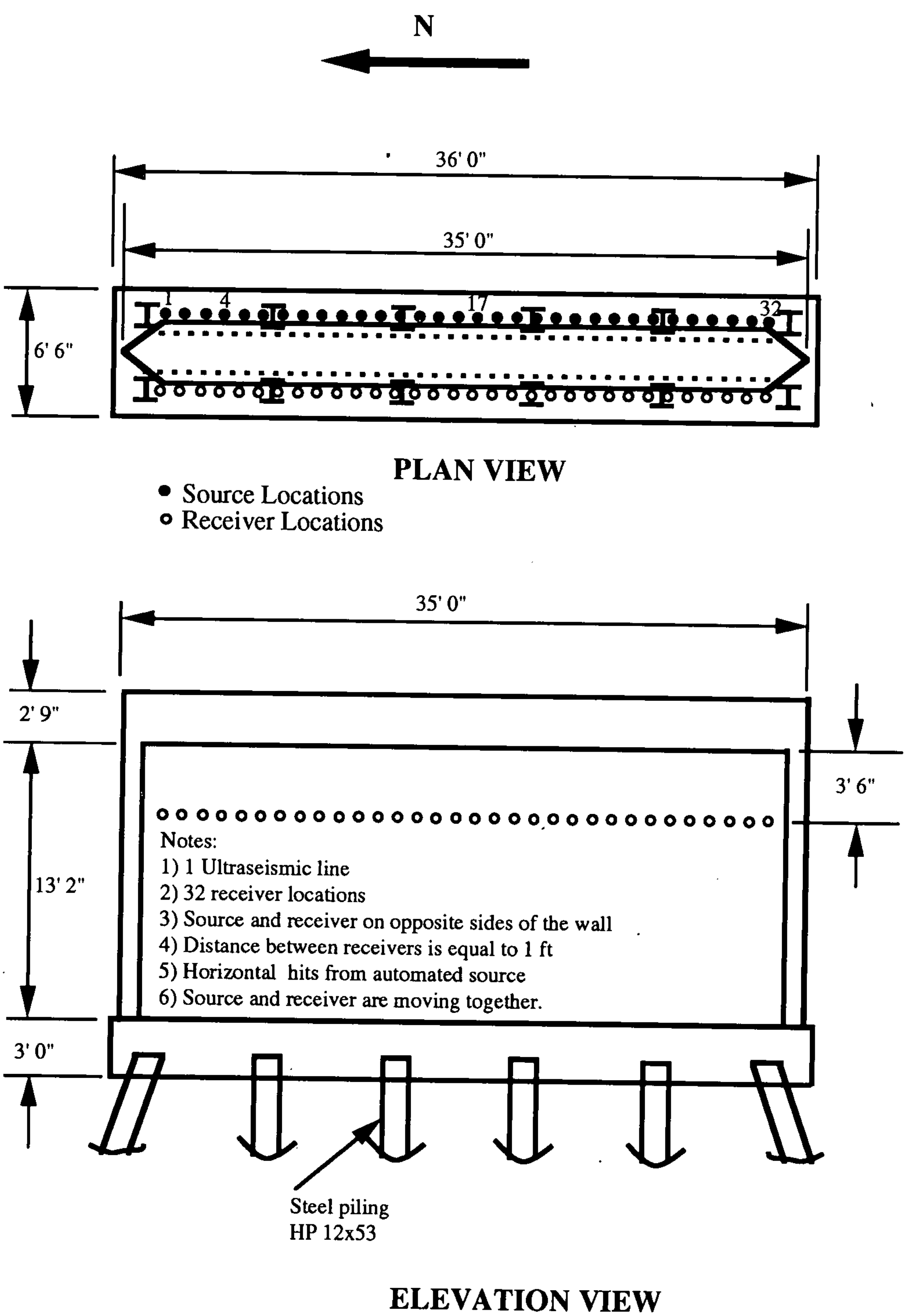


Figure 10. Source/receiver layout for HP test on sides of a pier wall of a Colorado County bridge.

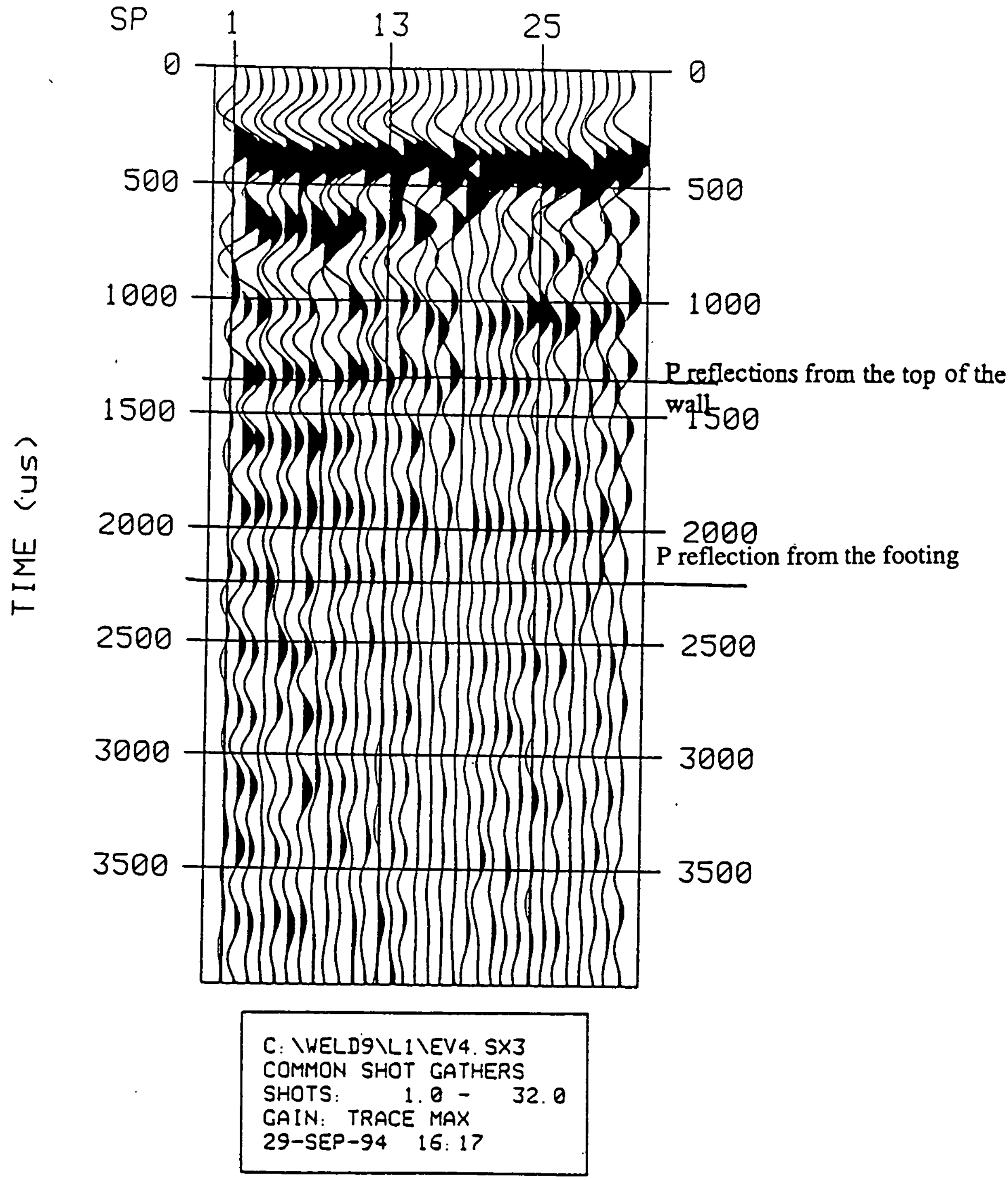


Figure 11. Example data from the HP dataset with the source and receivers moving simultaneously along the width of a Colorado County bridge pier wall at 1-ft intervals.

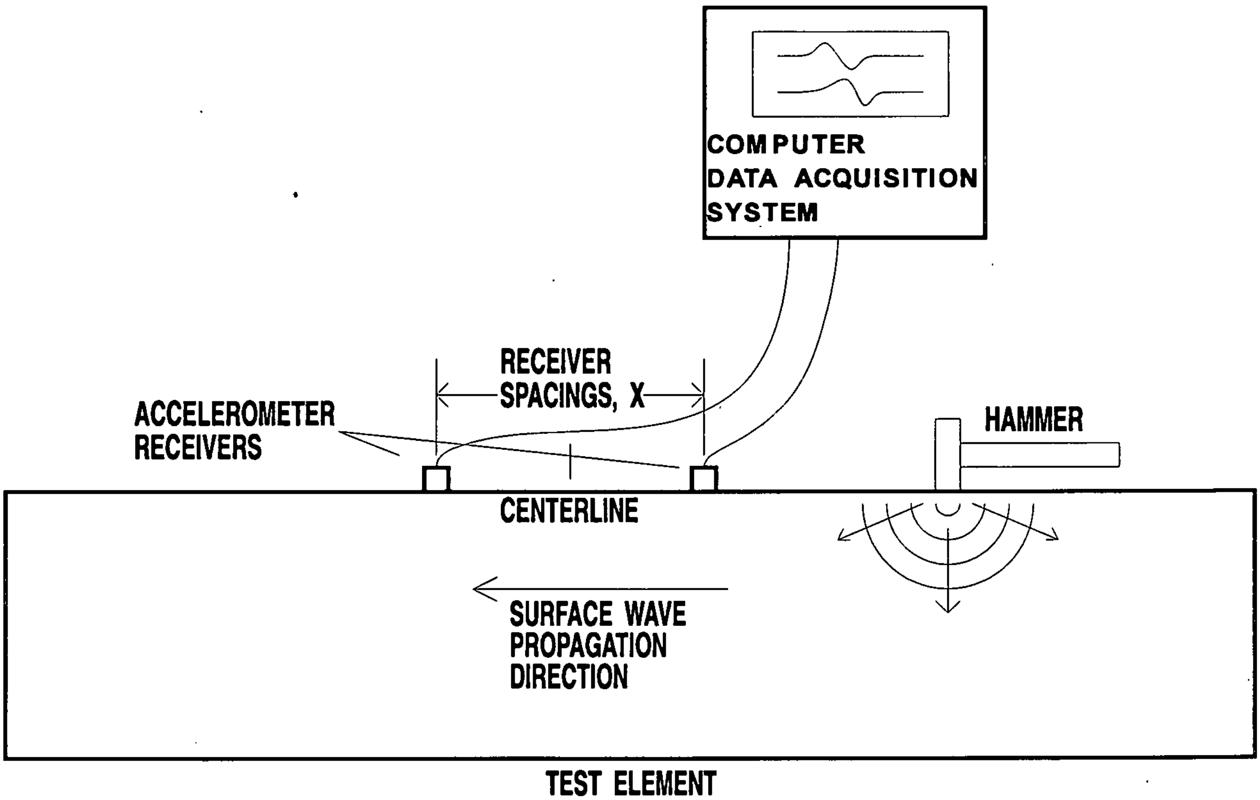


Figure 12. Source/receiver array used in SASW measurements.

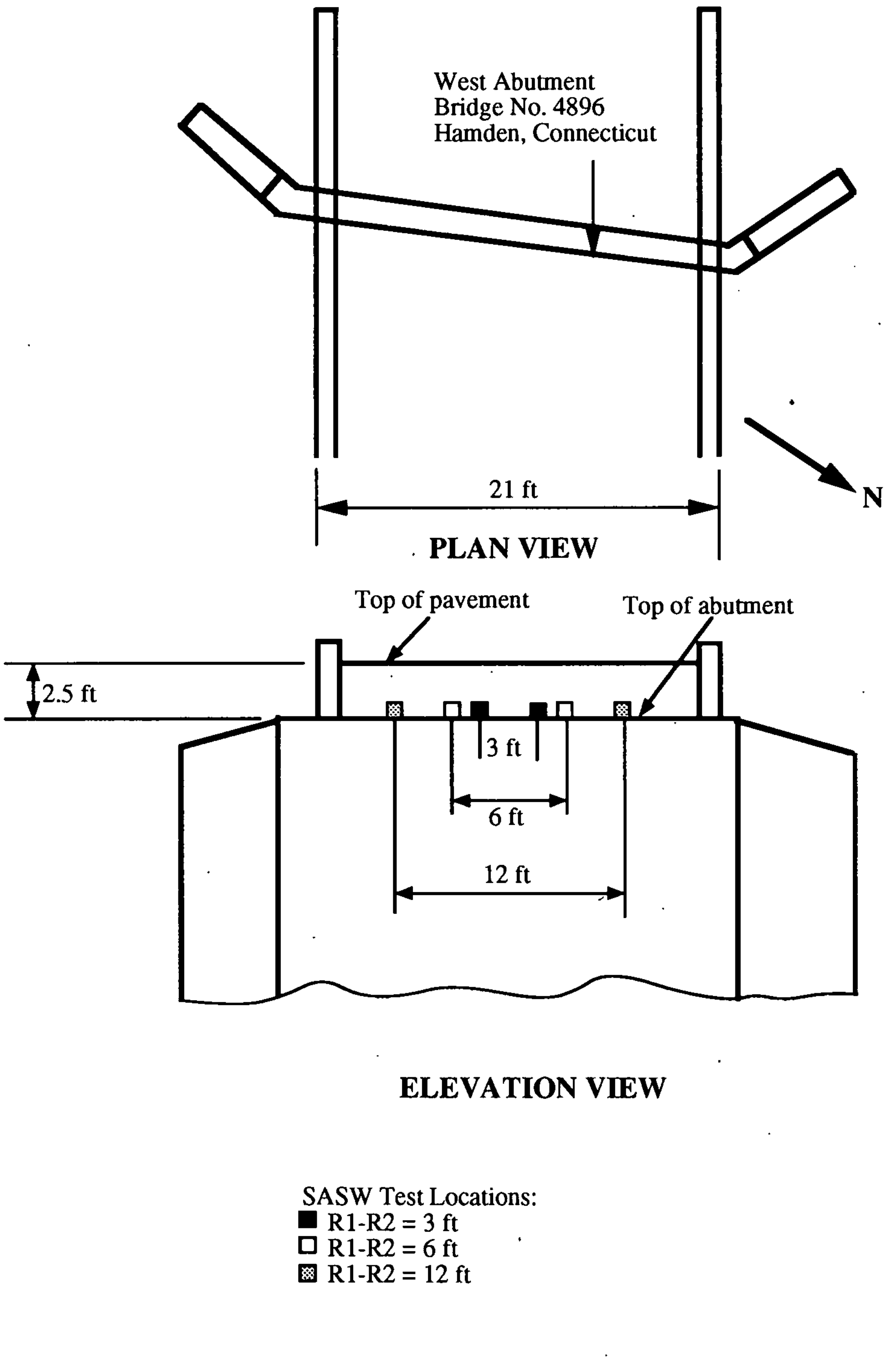


Figure 13. SASW test array from an abutment of a bridge in Connecticut.

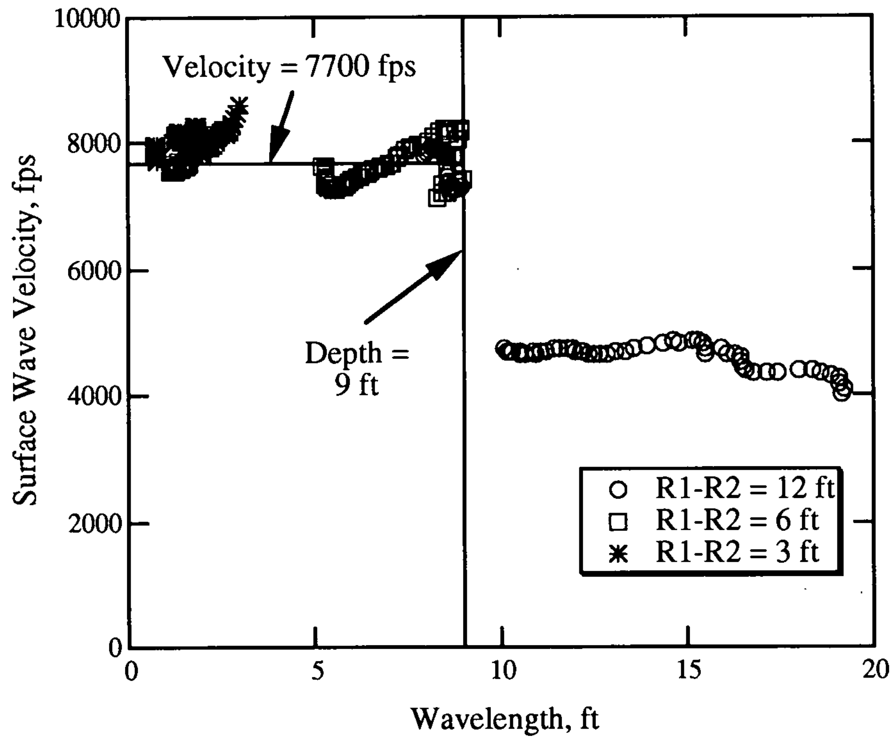


Figure 14. SASW test results from an abutment of a bridge in Connecticut, bottom depth = 9 ft, (reference = top of the abutment).

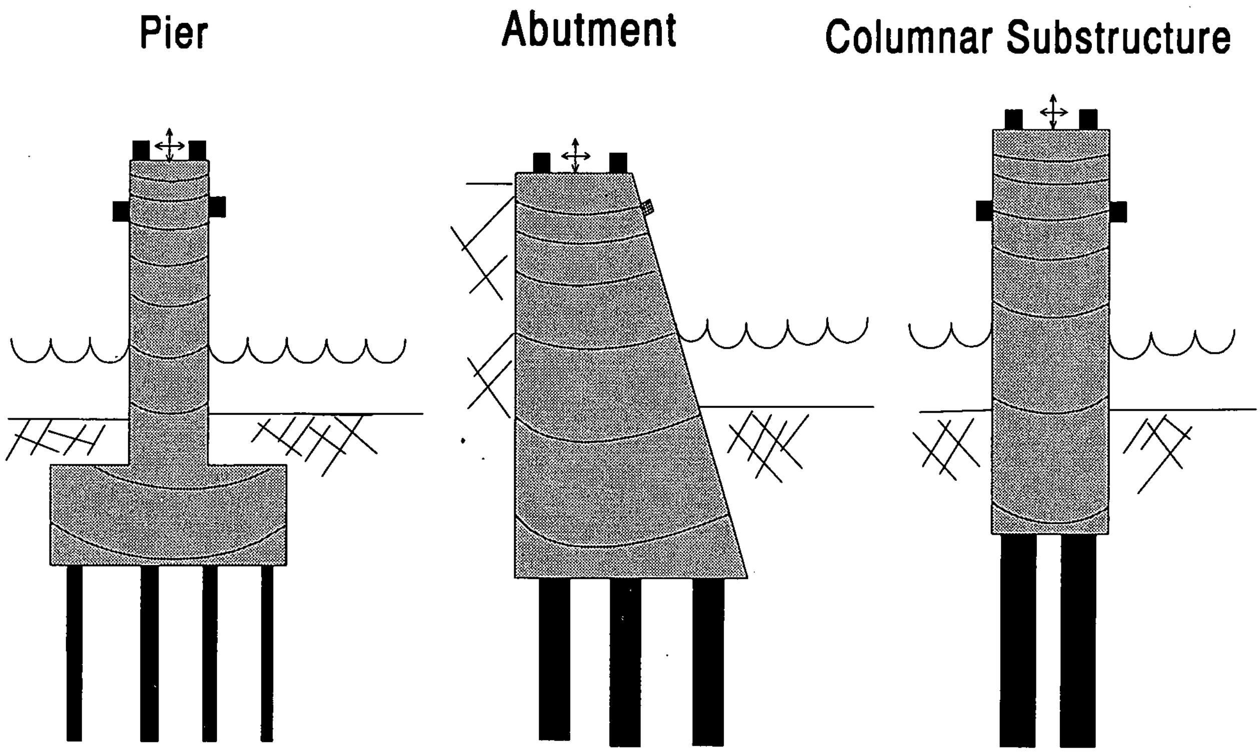


Figure 15. Dynamic Foundation Response (DFR) test method.

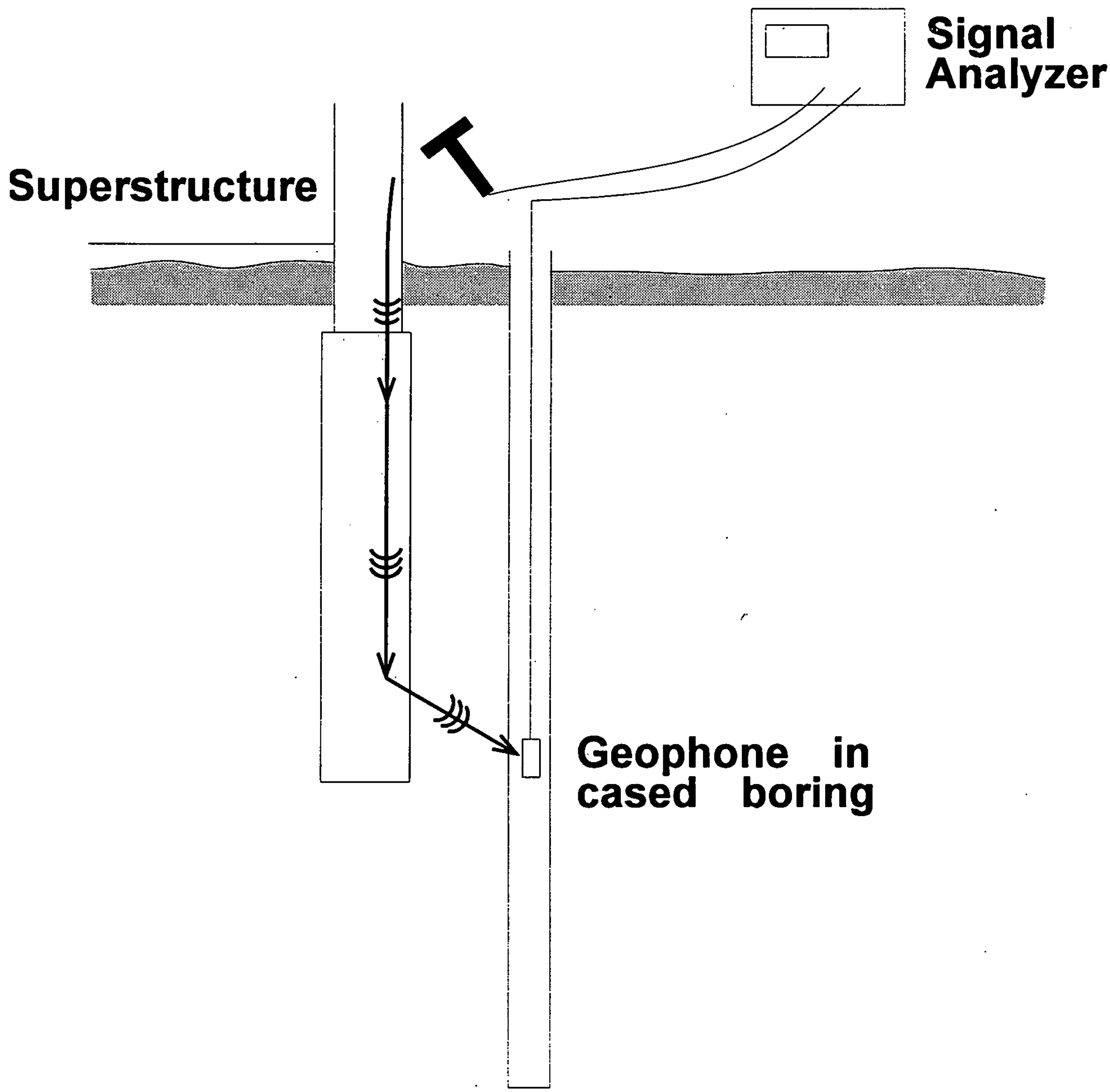


Figure 16. Parallel Seismic (PS) test method.

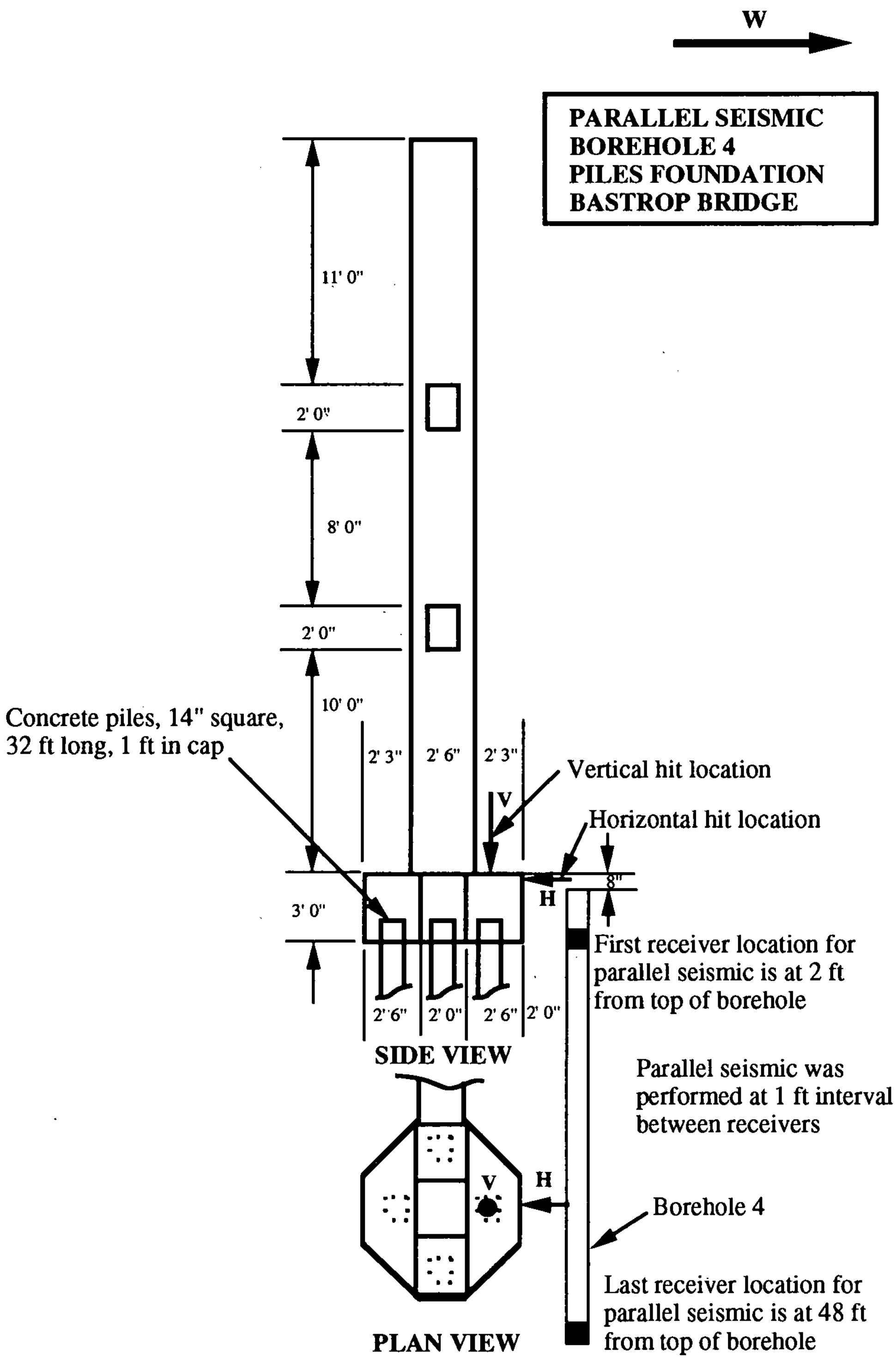


Figure 17. Source/receiver layout for PS test from a highway bridge.

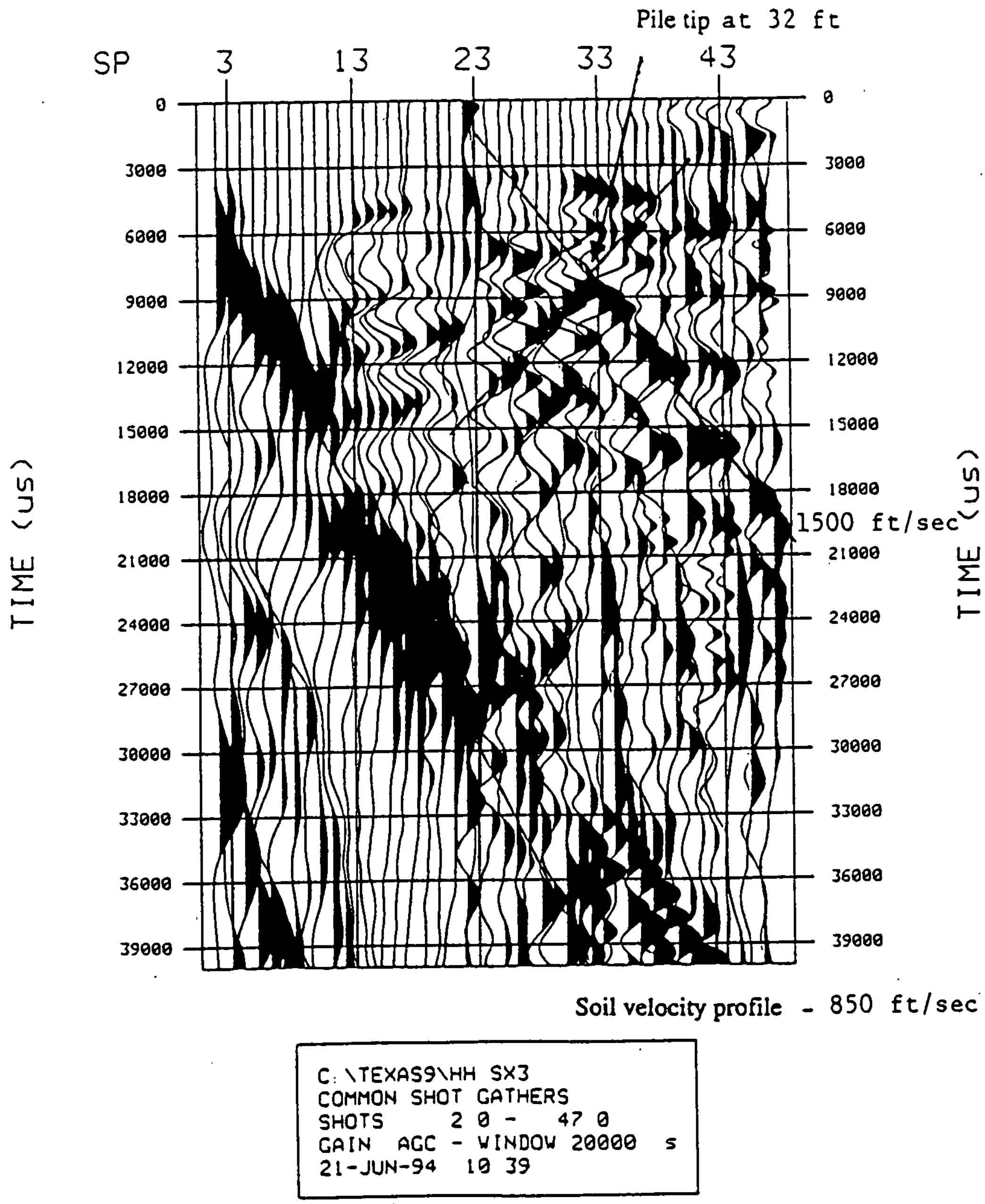


Figure 18. Parallel seismic field records (AGC) from a concrete pile foundation (see Figure 17 for geometry) from a 12-lb horizontal hammer hit and horizontal component geophone recording.

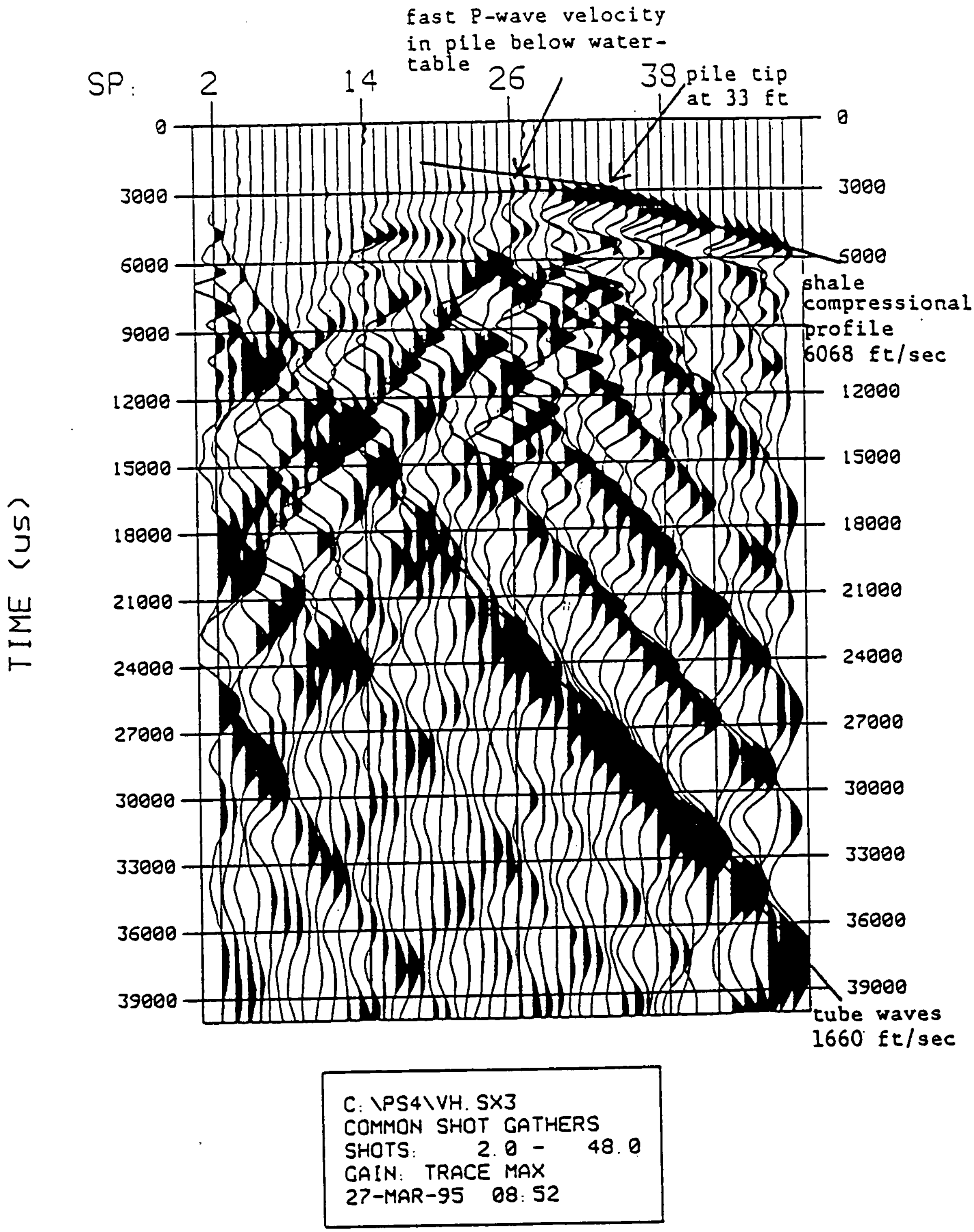


Figure 19. Parallel seismic field records (AGC) from a concrete pile foundation (see Figure 17 for geometry) from a 12-lb. vertical hammer hit and hydrophone receivers.

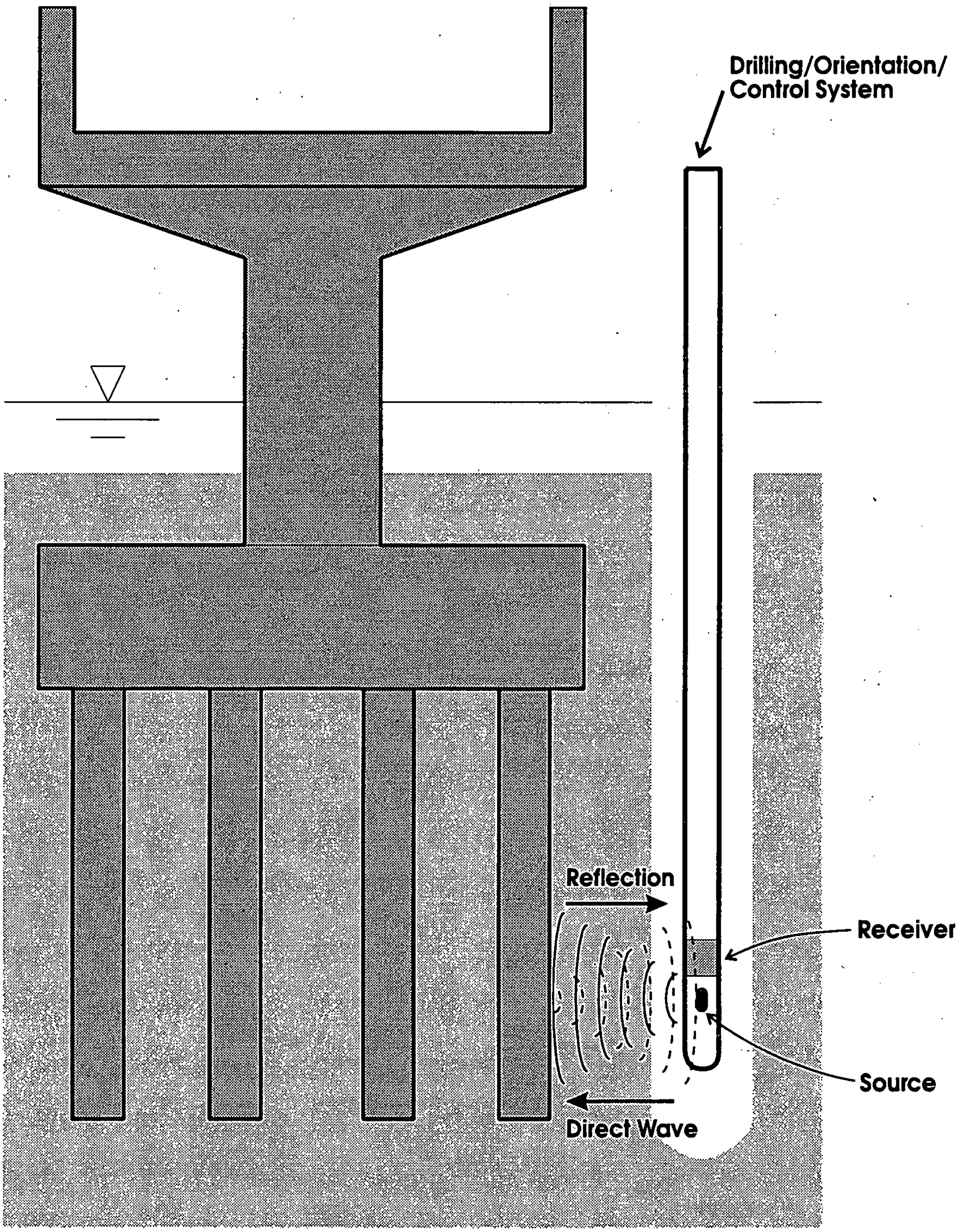


Figure 20. Borehole Sonic (BHS) test method.

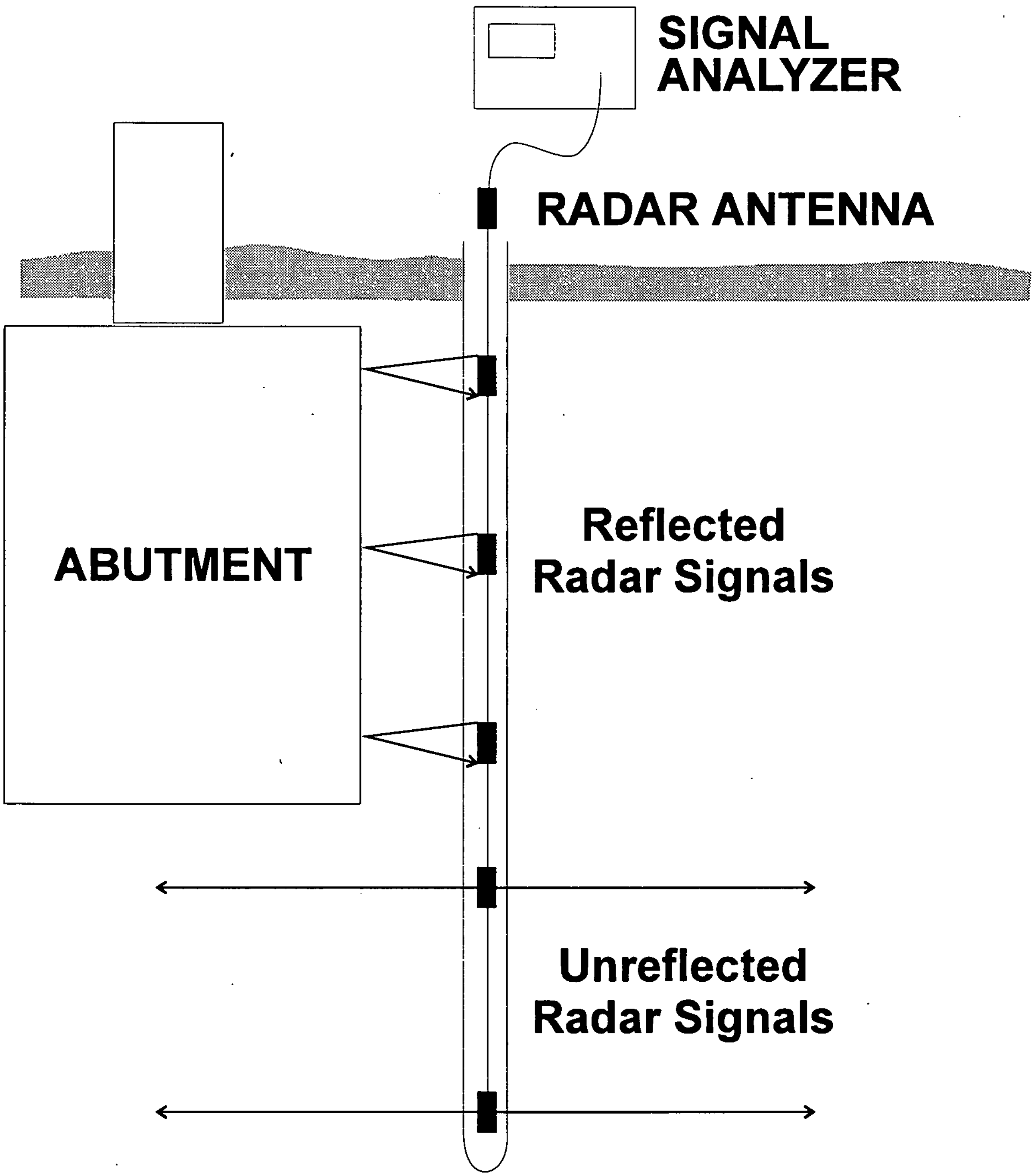


Figure 21. Borehole Radar (BHR) test method.

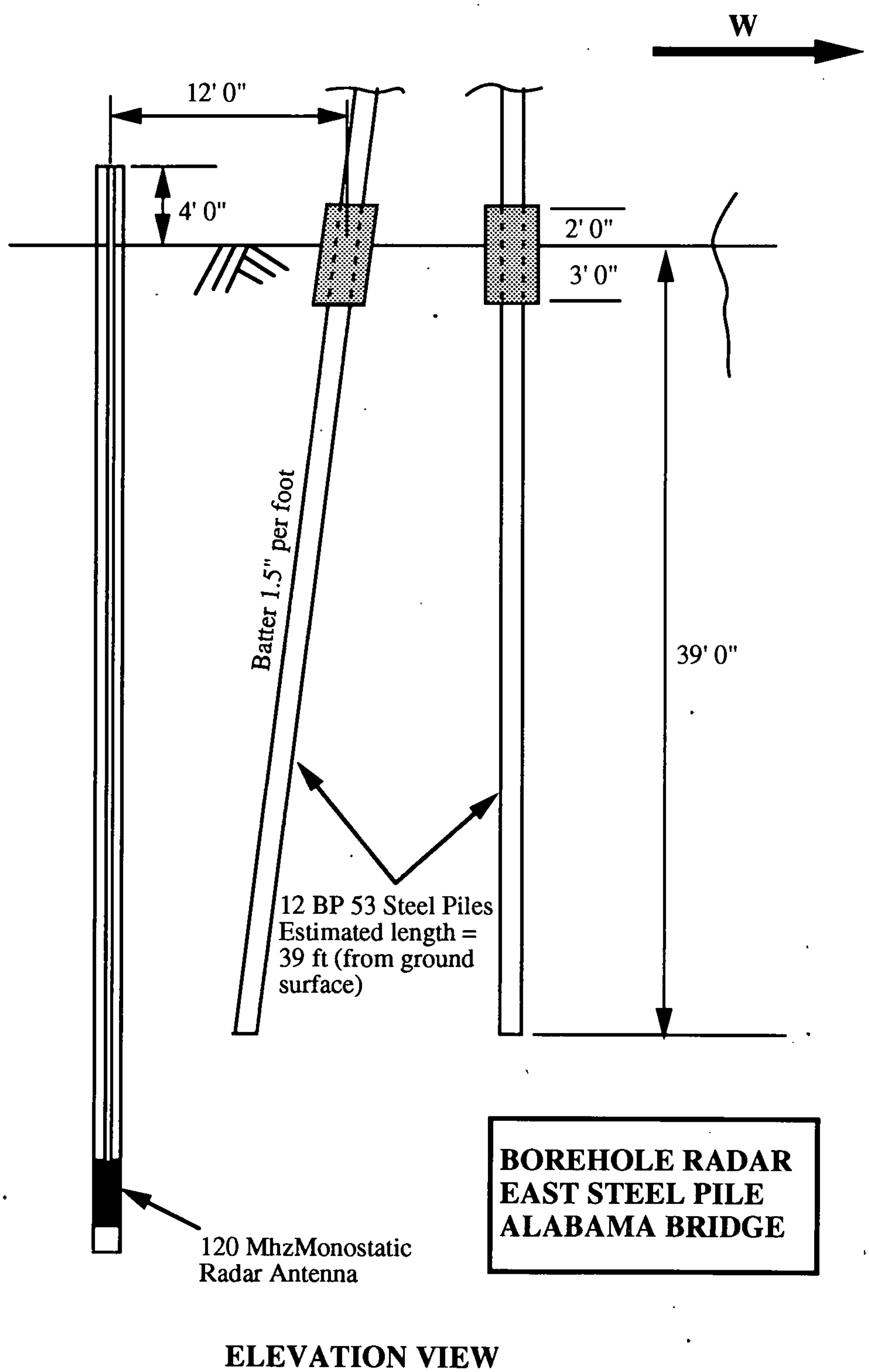


Figure 22. Field test layout for the BHR test from the east battered BP steel pile of a steel bridge in Alabama.

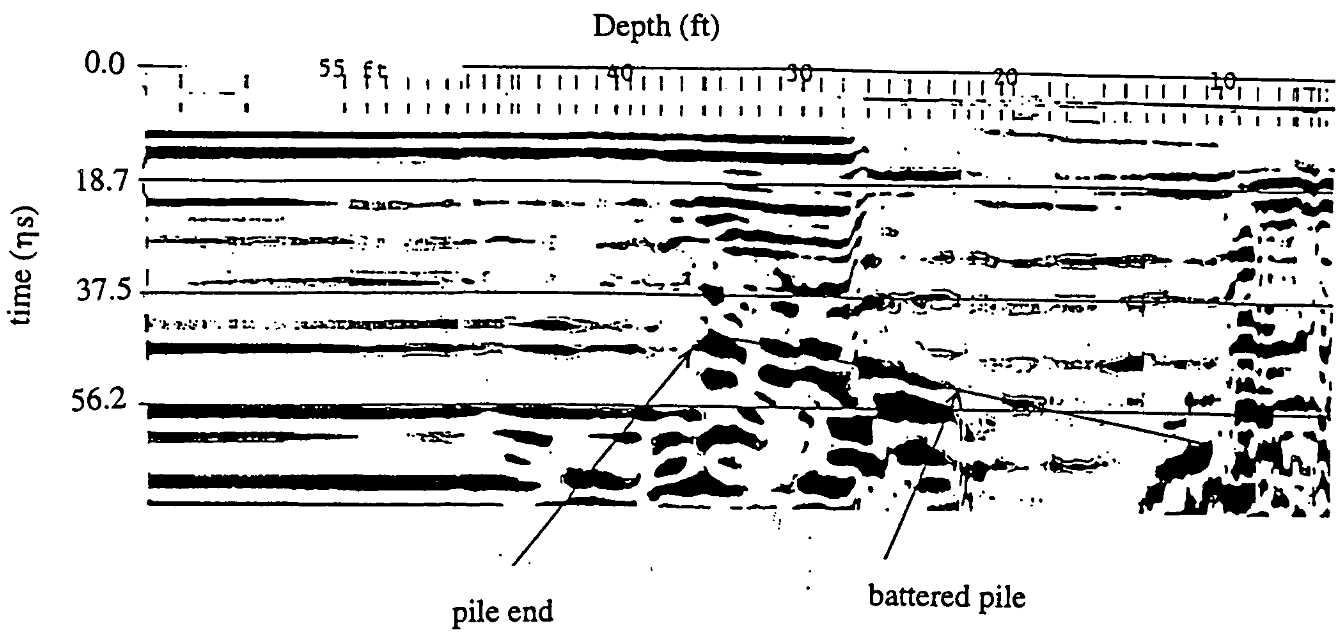


Figure 23. BHR data records from the east battered steel BP pile of a steel bridge in Alabama.

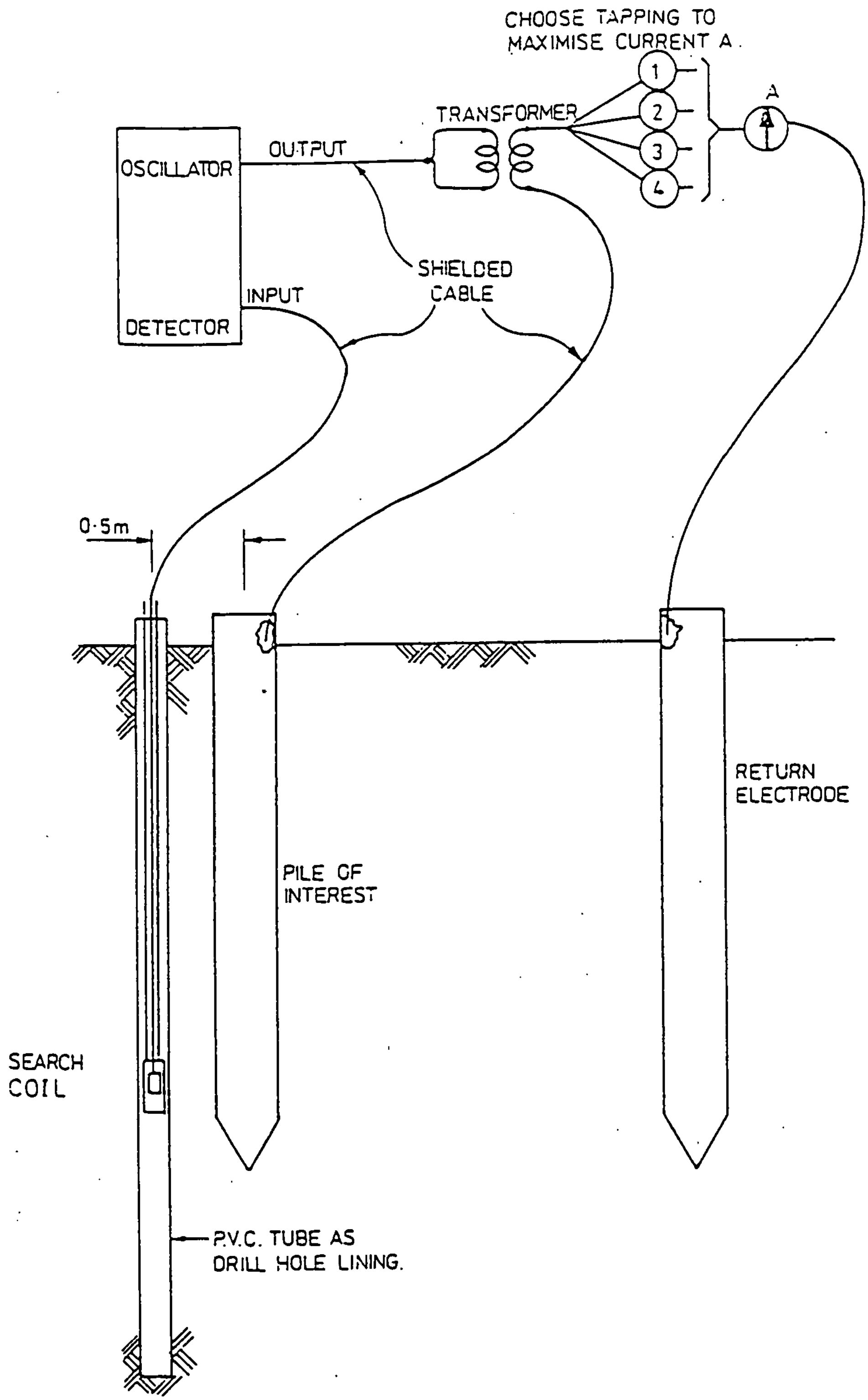


Figure 24. Induction Field (IF) test method (after Beattie, 1982).

TABLE 1 Summary Evaluation of the Applicable Surface NDT Methods

Ability to Identify Foundation Parameters	Sonic Echo (SE)/Impulse Response (IR) Test (Compressional Echo)	Bending Wave (BW) Test (Flexural Echo)
Foundation Parameters:		
Depth of Exposed Piles	Fair-Excellent	Fair-Excellent
Depth of Footing/Cap	Poor-Good	Poor-Fair?
Piles Exist Under Cap?	N/A	N/A
Depth of Pile below Cap?	N/A	N/A
Geometry of Substructure	N/A	N/A
Material Identification	N/A	N/A
Access Requirements:		
Bridge Substructure	Yes	Yes
Borehole	No	No
Subsurface Complications:		
Effect of soils on response	Low-Medium	Medium-High
Relative Cost Range:		
Operational Cost/SSU*	$1,000-$1,500	$1,000-$1,500
Equipment Cost	$15,000-$20,000	$15,000-$20,000
Required expertise:		
Field Acquisition	Technician	Technician
Data Analysis	Engineer	Engineer
Limitations:	Most useful for columnar or tabular structures. Response complicated by bridge superstructure elements. Stiff soils and rock limit penetration.	Only useful for purely columnar substructure. Response complicated by various bridge superstructure elements, and stiff soils may show only depth to stiff soil layer.
Advantages:	Lower cost equipment and inexpensive testing. Data interpretation for pile foundations may be able to be automated using neural network. Theoretical modeling should be used to plan field tests.	Lower cost equipment and inexpensive testing. Theoretical modeling should be used to plan field tests. The horizontal impacts are easy to apply.

*SSU = Substructure Unit cost is for consultant cost only - DOT to supply 1-2 people. N/A=Not Applicable

TABLE 1 Summary Evaluation of the Applicable Surface NDT Methods (Cont.)

Ultraseismic (US) Test (Compressional and Flexural Echo)	Spectral Analysis of Surface Wave (SASW) Test	Surface Ground Penetrating Radar (GPR) Test
Fair-Excellent Fair-Excellent N/A N/A Fair N/A	N/A Fair-Good N/A N/A Poor-Good Good	N/A Poor Fair-Poor Poor Poor-Good Poor-Fair
Yes No	Yes No	Yes No
Low-High	Low	High
\$1,000-\$1,500 \$20,000-\$25,000	\$1,000-\$1,500 \$15,000-\$20,000	\$1,000-\$1,500 \$30,000+
Technician Engineer	Technician-Engineer Engineer	Technician-Engineer Engineer
Cannot image piles below cap. Difficult to obtain foundation bottom reflections in stiff soils.	Cannot image piles below cap. Use restricted to bridges with flat, longer access for testing.	Signal quality is highly controlled by environmental factors. Adjacent substructure reflections complicate data analysis. Higher cost equipment.
Lower equipment and testing costs. Can identify the bottom depth of foundation inexpensively for a large class of bridges. Combines compressional and flexural wave reflection tests for complex substructures.	Lower equipment and testing costs. Also shows variation of bridge material and subsurface velocities (stiffnesses) vs. depth and thicknesses of accessible elements.	Fast testing times. Can indicate geometry of accessible elements and bedrock depths. Lower testing costs.

TABLE 2 Summary Evaluation of the Applicable Borehole NDT Methods

Ability to Identify Foundation Parameters	Parallel Seismic (PS) Test	Borehole Radar (BHR) Test	Induction Field (IF) Test
Foundation Parameters:			
Depth of Exposed piles	Good-Excellent	Poor-Excellent	None-Excellent
Depth of Footing/Cap	Good	Poor-Good	N/A
Piles Exist Under Cap?	Good	Fair-Good	None-Excellent
Depth of Pile below cap	Good-Excellent	Fair-Good	None-Excellent
Geometry of Substructure	Fair	Fair-Excellent	N/A
Material Identification	Poor-Fair	Poor-Fair	Poor-Fair
Access Requirements:			
Bridge Substructure	Yes	No	Yes
Borehole	Yes	Yes	Yes
Subsurface Complications:			
Effect of soils on response	Medium	High	Medium-High
Relative Cost Range:			
Operational Cost/SSU*	$1,000-$1,500	$1,000-$1,500	$1,000-$1,500
Equipment Cost	$15,000-$25,000	$35,000+	$10,000
Required expertise:			
Field Acquisition/SSU*	Technician-Engineer	Engineer	Technician
Data Analysis	Engineer	Engineer	Engineer
Limitations:	Difficult to transmit large amount of seismic energy from pile caps to smaller (area) piles.	Radar response is highly site dependent (very limited response in conductive, clayey, salt-water saturated soils).	It requires the reinforcement in the columns to be electrically connected to the piles underneath the footing. Only applicable to steel or reinforced substructure.
Advantages:	Lower equipment and testing costs. Can detect foundation depths for largest class of bridges and subsurface conditions.	Commercial testing equipment is now becoming available for this purpose. Relatively easy to identify reflections from the foundation; however, imaging requires careful processing.	Low equipment costs and easy to test. Could work well to complement PS tests and help determine pile type.

*SSU = Substructure Unit cost is for consultant cost only - DOT to supply 1-2 people + does not include drilling costs.
N/A = Not Applicable.

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