Stress Wave NDE Methods for Condition Assessment of the Superstructure and Substructure of Concrete Bridges

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1.0 INTRODUCTION

This paper discusses ultrasonic, sonic, vibration and acoustic-based stress wave methods used for condition assessment of the superstructure and substructure of concrete bridges. The basic physics, capabilities, training/experience requirements and example case history results are presented herein for the nondestructive evaluation (NDE) methods. Superstructure NDE methods are discussed first and include chain dragging/acoustic sounding for corrosion of rebar induced delaminations and ultrasonic pulse velocity, impact echo, spectral analysis of surface waves, and slab impulse response for quality assurance and integrity evaluations of decks, girders and exposed substructure. Recent innovations in impact echo scanning technologies for evaluation of internal grout conditions in post-tensioned bridge ducts are also discussed. In terms of buried bridge substructure (foundation systems), stress wave NDE methods discussed herein include: crosshole sonic logging and tomography for quality assurance and 3-D imaging of drilled shaft defects; sonic echo/impulse response for deep foundation integrity; ultraseismic for surface-based evaluation of unknown foundation depths; and, parallel seismic for borehole-based evaluation of unknown foundation depths. Most of the NDE methods discussed herein are also discussed in the two ACI 228 Nondestructive Testing Committee reports (ACI 228.1R-03 and ACI 228.1R-98).

2.0 NDE METHODS FOR CONCRETE BRIDGE SUPERSTRUCTURE

Stress wave NDE methods can be used for quality assurance of new bridge decks and girders, or condition assessment of existing, aging bridge superstructure. The methods can provide data on the following concrete conditions: corrosion induced delaminations of deck and girder elements; thicknesses of concrete decks; fire and frost damage, internal void, cracking and honeycomb; concrete strength; and, void in post-tensioned ducts in girders. The following sections discuss the acoustic, ultrasonic pulse velocity, impact echo, spectral analysis of surface waves, and slab impulse response methods along with example results.

2.1 Acoustic Sounding of Decks and Girders for Corrosion Delaminations

By impacting concrete decks, girders and other elements of a bridge, one can quickly locate shallow flaws such as delaminations of concrete (cracking that is subparallel to the concrete surface) due to corrosion and expansion of reinforcing steel. An ASTM standard covers sounding of decks with automated impact/sounding, chain drag/hammer/bar sounding and rolling impactor methods. The standard is entitled "Measuring Delaminations in Concrete Bridge Decks by Sounding" (ASTM D 4580-02) . Such delaminations are common problems for bridge decks and girders due to the effects of exposure to moisture and chlorides (often from de-icing salts and salt water).

Acoustic sounding of decks is most often done by dragging chains across the deck and listening for the lower frequency, hollow, drummy sounds that are indicative of delaminations. Sound concrete produces clear, high-frequency ringing sounds when acoustically sounded. Other acoustic sounding impact sources include steel hammers, rods, and the Delam 2000 with a pair of rolling steel gears on a T-handle. Microphones can also be used along with recording equipment to improve the identification of delaminations by quantifying the amplitude and frequency of the flexural vibration response of the concrete due to acoustic impacts. A comparison of chain dragging with confirming hammer soundings, impact echo and ground penetrating radar tests was recently reported by Scott et al (2002). Their study found good correlation of carefully done chain drag and impact echo soundings with core results on a delaminated concrete bridge deck in northern Virginia. However, the author's reported on research studies that showed chain dragging varies considerably when done more quickly with a number of state DOT inspectors.

The method is fairly easy to use and easily trained, but does require personnel with good hearing and can be problematic in noisy traffic situations and is not recommended for use on frozen concrete. It also is only sensitive to shallow delaminations located within a few inches of the surface being tested (typically 3-4 inches or shallower in the author's experience).

Chain dragging of a deck is shown in Figure 1 for dragging a loop of heavy chain while walking on the northern Virginia bridge deck by an engineer of the author's firm. Often devices are made with 4 chains attached to a bar that is pulled or rolled across a deck, or can be "danced" on a parking structure deck as shown in Figure 2. The Delam 2000 device is particularly useful for overhead surveys as shown in Figure 3 on the same parking structure. A solenoid based impactor with a microphone is shown in Figure 4 again on the northern Virginia bridge deck. A comparison was made between the chain drag of Figure 1 and the Impact Echo (IE) Scanner with a microphone for acoustic sounding as shown in Figure 4. The results of this showed that the IE Scanner with microphone and amplitude/frequency analysis was more sensitive to near-surface delaminations than the chain drag and human ear approach and this will be reported in the near-future.

Figure 1 - Loop Chain Dragging of a northern Virginia Bridge Deck

Figure 2 - Chain dragging with a rolling bar on a parking structure deck

Figure 3 - Delam 2000 sounding of underside of parking structure deck from underside

Figure 4 - IE Scanner with microphone for acoustic sounding of bridge deck

2.2 Ultrasonic Pulse Velocity (UPV) and Ultrasonic Tomographic Imaging

The UPV method is a direct compression wave velocity measurement method used in structural applications to evaluate the condition of materials such as concrete (ASTM C597 – 97). The method generally requires access to two sides of the test element for direct and semi-direct tests, although indirect surface measurements can provide data on the near-surface concrete condition (see Figure 5). This method allows relative comparisons of concrete strength based on the measured compression wave velocity as well as allowing the location of defects. The UPV method has been traditionally applied at fixed locations. Recent innovations with the method include scanning UPV measurements and Ultrasonic Tomography (UT) to image flaws in concrete with two-sided access (Jalinoos and Olson, 1995). Discussions are presented in the following sections of the UPV method for condition assessment and strength estimation, and the UT method for imaging concrete flaws.

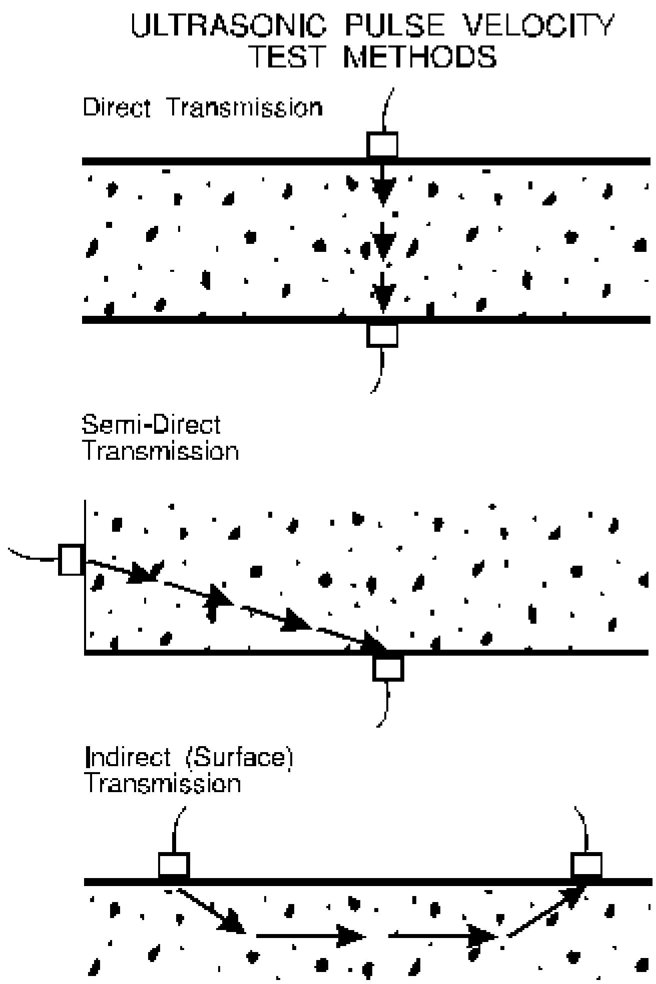


Figure 5 – Direct, Semi-Direct, and Indirect Ultrasonic Pulse Velocity (UPV) Methods

2.2.1 Ultrasonic Pulse Velocity Method for Concrete Condition Assessment

The Ultrasonic Pulse Velocity (UPV) method is based on the speed and amplitude of a compressional wave pulse and is used for determining material velocity and integrity conditions. Ultrasonic source and receiver transducers are placed on opposite ends of a given test path, and a signal is sent between them. The signal travel time and attenuation provide information as to the concrete integrity along that signal path. Since this method uses a source and receiver to pass wave energy through a test member, it requires access to two sides of a member for evaluation of interior material conditions. A faster measured velocity in a given material generally correlates with greater strength and better integrity. The velocity (V) is calculated as the travel path distance (d) divided by time (t) and it is normally the compressional wave velocity, but shear transducers are available if the shear wave velocity is to be measured in the UPV test.

$$V = d/t \tag{1}$$

The UPV method is an ultrasonic test for evaluating concrete quality and integrity, which is shown schematically in Figure 5. UPV test equipment is commercially available and relatively easy to use for lab measurements immediately and for field-testing after a day of training. UPV test equipment is shown in Figures 6a and 6b for a honeycomb/void evaluation on a highway sign column, which included a pair of 54 kHz resonant frequency UPV transducers, and an Olson Instruments Freedom NDT PC for recording the pulse wave energy sensed by the receiver.

Figure 6a - Picture of a 54 kHz UPV transducer being coupled to the column with grease

Figure 6b - Picture showing the Freedom NDT PC for UPV testing

The UPV test involves passing an ultrasonic compressional wave pulse or sound wave a known distance through the concrete from the source to the receiver transducers which are coupled with grease or a non-staining ultrasonic gel couplant to the concrete). The signal is then amplified and filtered and applied to the computer where the signal can be viewed and the travel time and voltage amplitude recorded. The pulse velocity of the concrete is calculated by dividing the pulse travel distance by the travel time. Internal voids, cracks, honeycomb, and other flaws between the source and receiver such as weakly bonded surface patches reduce the UPV velocity and amplitude. Generally, the faster the velocity of concrete the better the concrete quality and the stronger the concrete. A complete air-filled void inside a column, shaft, or pier may result in zero signal transmission or a significant time delay as the signal travels around the void.

Example UPV results for sound and honeycomb/void concrete conditions are shown in Figures 7a and 7b below for the highway sign column, respectively. The strong energy at a time of zero seconds is due to noise from the ultrasonic source pulse at high gains in both figures. The actual signal arrival is clear at around 370 microseconds in Figure 7a which corresponds to a good velocity of about 13,500 fps for the sound concrete. In comparison, a very weak and delayed signal is evident at around 550 microseconds in Figure 7b which reflects the void/honeycomb poor quality condition and corresponds to an average velocity of 9,100 fps through the 5 ft square column. Good velocity for concrete is typically faster than 12,000 feet per second (fps) with questionable quality concrete conditions between 10,000-12,000 fps and poor quality concrete conditions at velocities less than 10,000 fps.

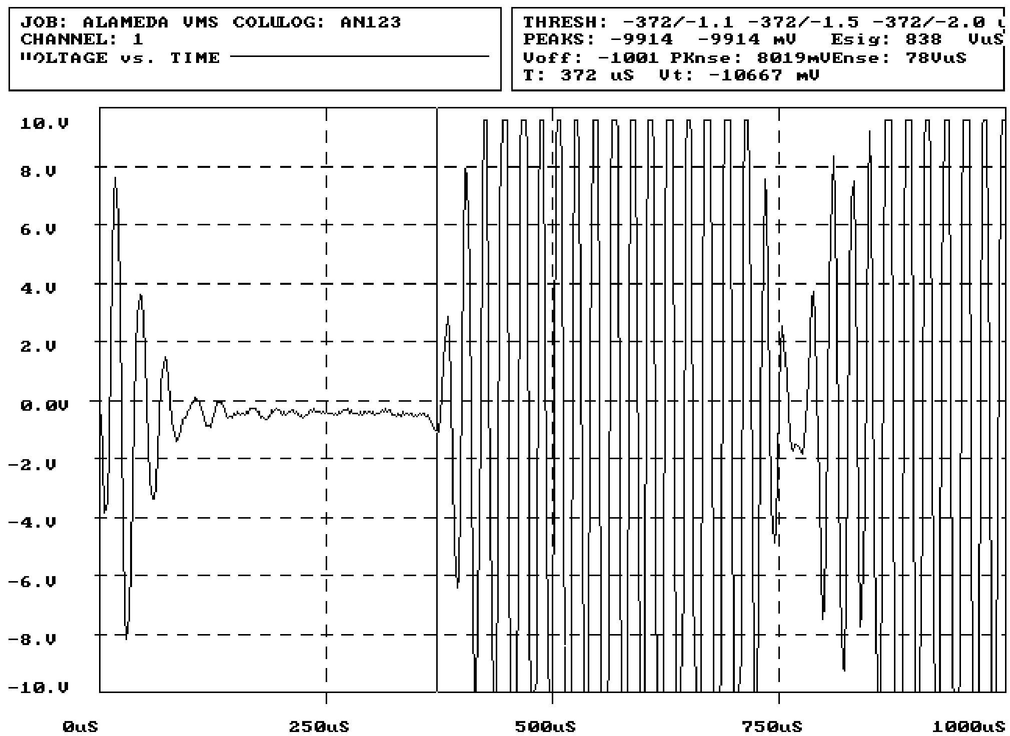


Figure 7a - Good concrete quality example UPV record from Sign Column

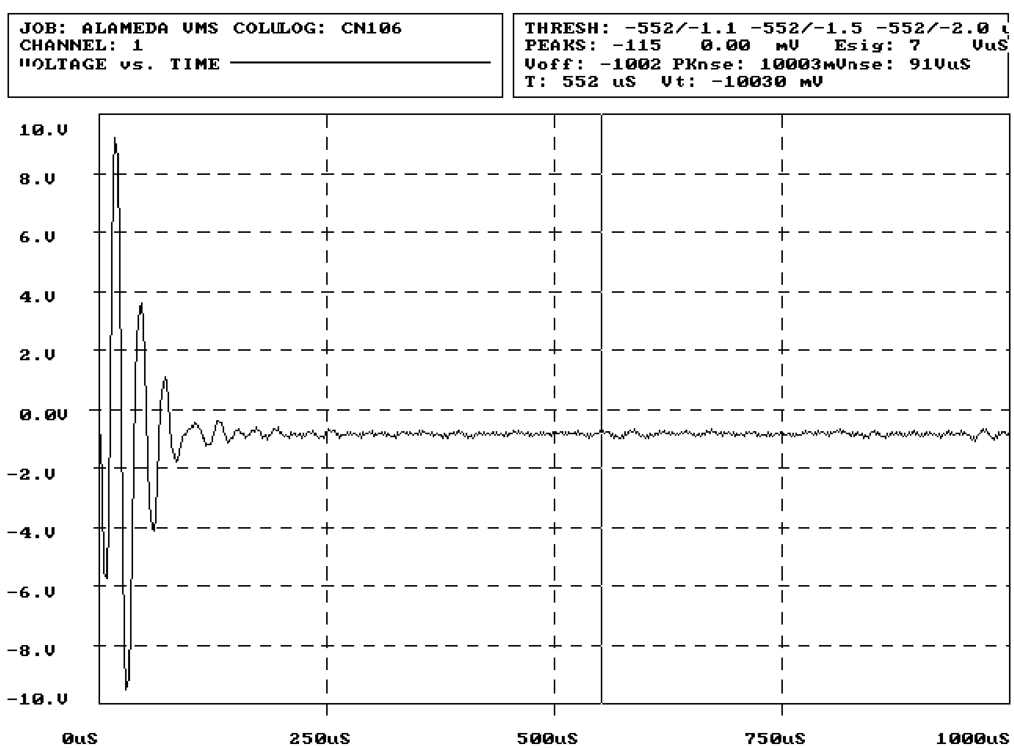


Figure 7b - Poor concrete quality example UPV record from Void/Honeycomb in Sign Column

2.2.2 Ultrasonic Pulse Velocity Method for Concrete Strength Prediction

There are a number of NDT methods that can be correlated with destructive strength tests on cylinders or cores as documented in the American Concrete Institutes Manual of Concrete Practice report ACI 228.1R – 03 NDT Methods for Concrete Strength which has just been updated and will be available in early 2004. These methods include rebound (Schmidt or Swiss Hammer), pin penetration resistance, pull-out (and CAPO of hardened concrete) and Windsor Probe and Maturity tests of the near-surface concrete. Ultrasonic Pulse Velocity (UPV) and other stress wave methods such as Impact Echo (IE) and Spectral Analysis of Surface Waves (SASW) measure deeper in the concrete to get stress wave velocities that relate to strength as. The advantages and disadvantages of these methods are discussed in the ACI 228.1R document, and are not repeated herein. As detailed in ACI 228.1R, UPV measurements of compressional wave velocity (Vp – see Figure 5) are related to Young's Modulus (E) and compressive strength for concrete (f'c) by the approximate relationship of:

$$V^4 \text{ is proportional to } E^2 \text{ is proportional to } f'c$$

The exact relationship is best determined with multiple regression analyses of velocity measurements with compressive strength test results on cylinders (new concrete) or cores (existing concrete).

The author and his firm served as the principal consultant on a National Science Foundation Small Business Innovation Phase I Research project in 1987 to predict concrete strength. This study involved rebound hammer, UPV, and Impact Echo (IE – see Section 2.3) tests of standard 6 x 12 inch concrete cylinders at ages of 3, 7, 14 and 28 days with mix design strengths of 3000, 40000 and 5000 psi. Multiple regression correlation analyses resulted in R^2 values of 0.89, 0.91 and 0.93 for rebound, UPV and IE tests, respectively. IE tests produced the best correlation and had a standard error of estimate of 232 psi over the age and strength range tested.

Strength predictions with UPV and other stress wave measurements have been performed on concrete, grout samples and foamed concrete materials and the author has developed similar high quality correlations with multiple regression analyses for correlation of UPV and strength. An example linear regression correlation of UPV predicted strength is presented in Figure 8 below from tests on low density foamed concrete cylinders taken from an embankment for a DOT highway project.

However, often access to both sides of structures such as bridges and dams (which is required for UPV tests), is impractical and the wave travel path distance is not precisely known. In these instances, we have used IE and Spectral Analysis of Surface Waves tests (SASW – see Section 2.5). The IE tests work well when the section thickness is a constant and tests can be done from one side. If the thickness is not known or varies, or the material strength varies with depth, SASW tests can determine the velocity versus wavelength (depth) with one-sided access like the IE test.

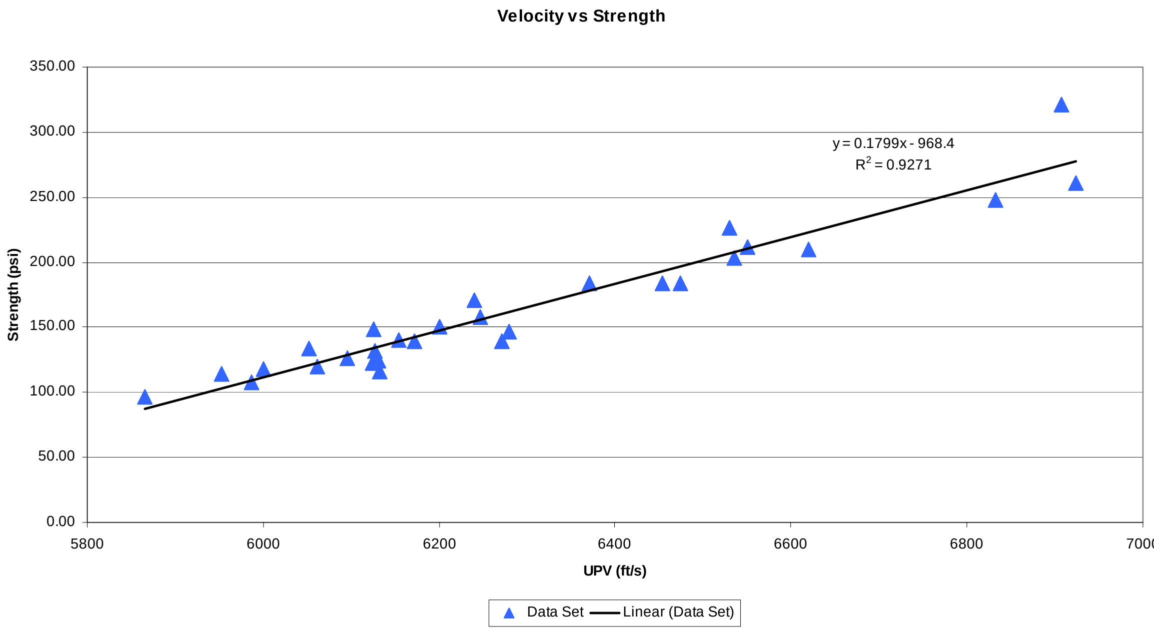


Figure 8 – Correlation of Ultrasonic Pulse velocity and Compressive Strength for Foamed Concrete Embankment 3x6 inch test cylinders

Thus when determination of strength is a concern for large structures and only minimal coring is desired, UPV, IE and SASW tests have been used on a number of consulting projects where strength data was desired and cores were cut for correlation purposes. For example, a combined approach of UPV, IE and SASW tests with cores was taken on a power company dam project as part of an earthquake safety study of an early 1900's dam. Initial coring indicated visibly sound concrete had strengths ranging from 1500 to 7500 psi for the Ambersen dam spillway section (hollow with internal large buttress concrete walls). It was decided to perform IE and SASW tests on interior dam buttresses and correlate the in situ velocities with UPV results on cores to predict concrete strength. An excellent multiple regression correlation was obtained in the UPV/core tests that was used to interpret the in situ SASW velocity results and predict the in situ strength. This allowed dam engineers to evaluate the safety of the dam in the event of earthquakes. The SASW predicted strengths showed the original dam designers had intentionally used leaner and weaker concrete materials in areas of low stress and stronger concrete in areas of high stress. The use of this NDT strength approach minimized damage to the dam by greatly reducing the coring and also provided moduli data for analyses and revealed the strength distribution throughout the dam buttresses.

2.2.3 Ultrasonic Tomography (UT) for Imaging of Concrete Conditions and Flaws

Ultrasonic Tomography (UT) is an imaging method analogous to CAT-scanning in the medical industry but uses acoustic waves rather than X-rays. Olson Engineering served as the prime contractor on a National Science Foundation Small Business Innovation Phase I Research project in 1993 to image flaws in concrete walls using a scanning UPV source and fixed UPV receiver to take the direct and semi-direct (angled) UPV data necessary for UT imaging of internal flaws. UT testing and analysis is typically performed after UPV testing finds potential flaws in order to obtain more information about the size, shape, location, and severity of suspected internal defect(s) in a concrete structure. UT data collection is intense and the procedure is relatively slow compared with UPV. In UT, many test paths are used to produce precise, detailed images of internal concrete conditions. The spatial resolution of UT is much higher than that of UPV and actual 2-D and even 3-D images of the internal concrete structure can be produced. Because of the higher resolution of the UT test, often more anomalies and smaller anomalies are identified in addition to those found with UPV tests.

The UT method uses the same general equipment as the UPV method. For UT testing, acoustic data are collected for many receiver and source combinations at different locations, both direct and semi-direct (angled) tests whereas UPV testing typically incorporates source and receiver positions at the same depth or horizontal plane. For a typical UT data set, sound velocity raypaths are generated for tens to hundreds of source-receiver location combinations. The term "ray coverage" describes the area through which acoustic wave rays travel from the many source-receiver position combinations.

Ultrasonic Tomography is an analytical technique which uses an inversion procedure on the first arrival time data of compressional or shear wave energy that can produce ultrasonic pulse-velocity based images of a 2-D or 3-D concrete zone inside a structure, or of the entire structure. This type of tomography is termed "velocity tomography" and can be used together with amplitude tomography. The test region is first discretized into many cells with assumed slowness values (inverse of velocity) and then the arrival times along the test paths are calculated. The calculated times are compared to the measured travel times and the errors are redistributed along the individual cells using mathematical models. This process is continued until the measured travel times match the assumed travel times within an assumed tolerance. The end result is a 2-D or 3-D velocity image (or contour) of the internal concrete of the structure, revealing sound (fast) versus defective (slow) areas.

An example 2-D compressional wave velocity tomogram from the highway sign column has been included as Figure 9 for the combination of 5 north-south and 5 east-west tests. Examination of the UT velocity tomogram in Figure 9 shows that the velocity is significantly slower in the top half and left side of the column at this elevation (the color velocity scale on the right side is in thousands of feet per second, kfps). This tomogram shows the velocity distribution within the concrete column in a horizontal slice fashion based on UPV arrival time analyses of the 5 north-south and 5 east-west UPV direct tests through the column (the dimensions are in feet and north is at the top of the page). The tests were taken with a relatively wide spacing of 1 ft for the 5 x 5 ft square column section directly through the column at 1 ft elevations so resolution of smaller defects was limited. However, the UT analyses greatly helped to show the internal area of honeycomb/void in the column. State DOT bridge engineers ultimately decided the column would have to be removed and replaced due to the extensive internal void and honeycomb conditions found in the UPV/UT investigation.

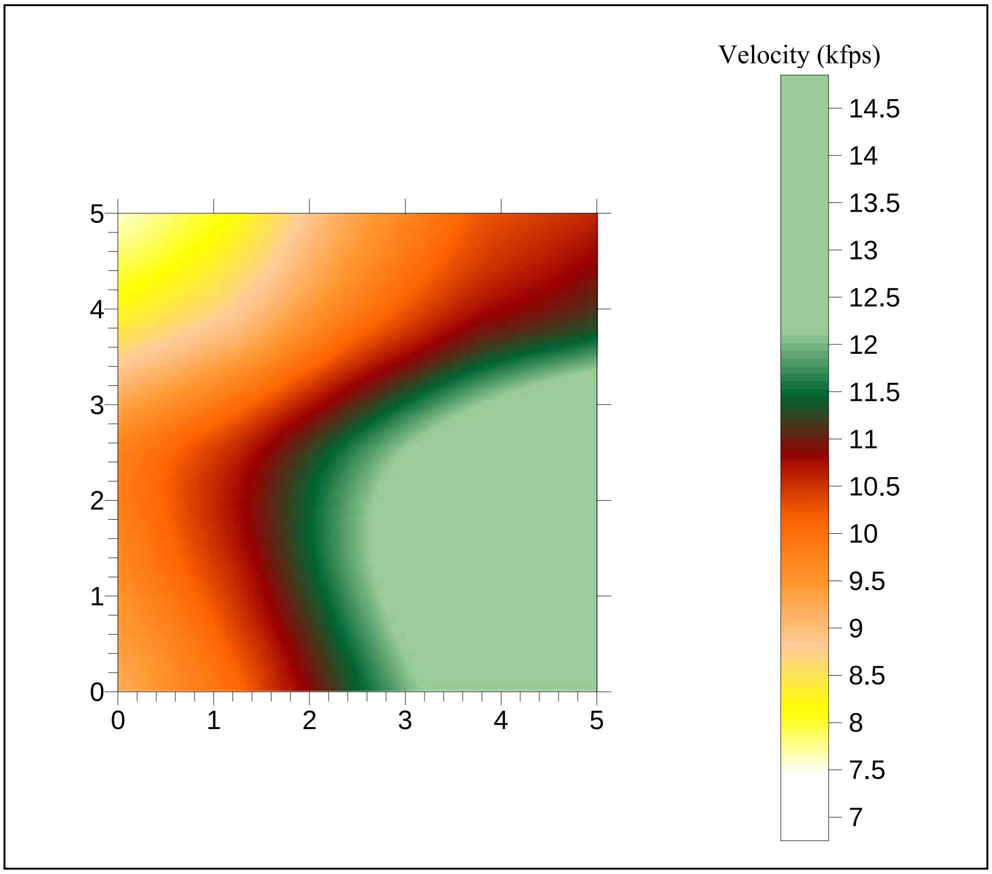


Figure 9 - Example UT tomogram from highway sign column

Training to take UPV data for UT analyses is relatively straightforward. However, UT analyses require specialized equipment, training and experience to obtain 2-D and 3-D tomographic images of internal concrete conditions in bridge columns, piers, abutments and other massive concrete structures.

2.3 Impact Echo (IE) Method for Concrete Thickness and Integrity

The IE method was researched and developed at the National Institute of Standards and Technology (Sansalone and Carino, 1986). There is an ASTM Standard for concrete thickness determination (ASTM C1383-98a). The method as used by the author's firm involves hitting the concrete surface with a small impactor or impulse hammer (0.09 kg (0.2 lb)) and identifying the reflected wave energy with a displacement or accelerometer receiver mounted on the surface about 50 mm (2 in) from the impact point as shown in Fig. 10. An IE test with a built-in electric solenoid impactor/displacement transducer head and Concrete Thickness Gauge (CTG-1TF) by Olson Instruments, Inc. is shown in Fig. 11 in use on a bridge deck. The output force of the hammer (if used) and the resulting displacement, or acceleration, response of the receiver are recorded in time and analyzed in the frequency domain during testing.

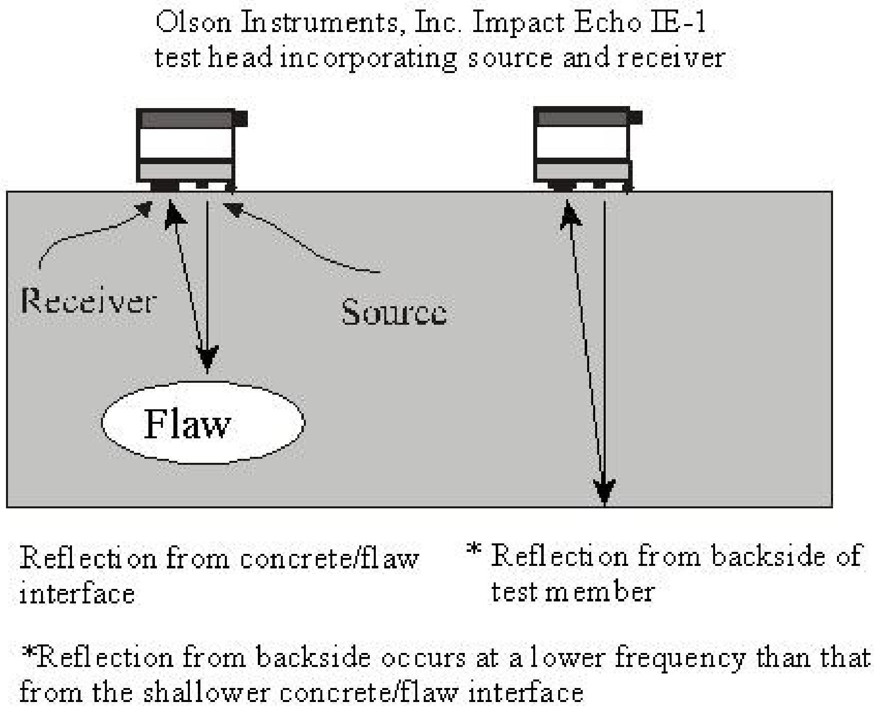


Figure 10 – Impact Echo test method for thickness and flaw evaluation of concrete

Figure 11 – Concrete Thickness Gauge (CTG -2TF) used in Impact Echo (IE) Bridge Deck Tests

The resonant echoes are usually not apparent in the time domain. The resonant echo peak frequencies in concrete slabs are more easily identified in the frequency domain. Consequently, the time domain test data are processed with a Fast Fourier Transform (FFT), which allows easier identification of frequency peaks (echoes). The auto power spectrum of the receiver or the transfer function (receiver output/hammer input vs. frequency) can be used to determine the resonant peaks. If the thickness of the test member is known, the compressional wave velocity (V_P) can be determined by the following equation:

$$V_P = 2*d*f/\beta \qquad (2)$$

where d = member thickness, f = resonant frequency peak, β= shape factor (equal to 0.96 for slabs, Sansalone and Streett, 1997).

The IE method can be used for measuring concrete thickness, evaluating concrete quality, and detecting hidden flaws such as cracks, honeycombs, etc. Concrete quality is related to compression wave velocity and elastic modulus and increases in compression wave velocity generally correlate with increased concrete strength and better concrete quality. Example results from IE tests performed on a concrete bridge deck and on a twin-T web section are presented in Figures 12 and 13, respectively, to illustrate this point. In the example shown, the bridge deck thickness in Fig. 12 was equal to approximately 8.9 inches. A resonant peak of approximately 7,500 Hz was identified in Fig. 12 which translates into a compression wave velocity of 11,600 ft/sec using Equation 1. The IE test was performed at Location 1 of the bridge deck. The web thickness in Fig. 13 was equal to 10.6 inches. A resonant peak of 5,500 Hz was identified in Fig. 13 which translates into a compression wave velocity of 8,500 ft/sec. Looking at the shallow echo at approximately 21,000 Hz this may be indicative of the depth and extent of a surface crack into the web of the twin-T member.

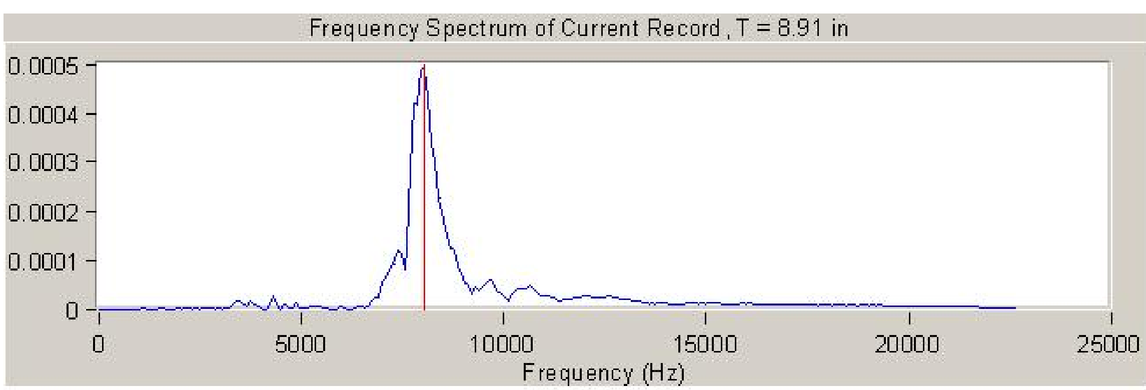


Figure 12 - Impact Echo record from a twin-T bridge deck showing a thickness of 8.91 inches

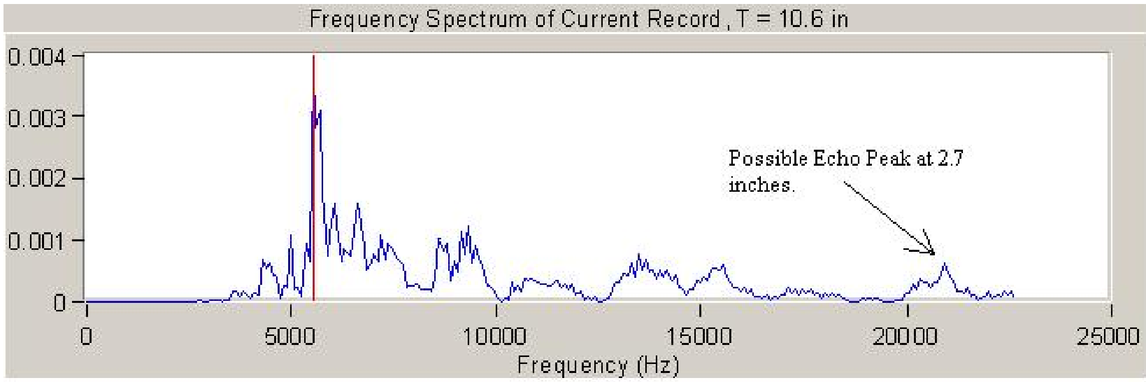


Figure 13 - Impact Echo record from the web section of a twin-T bridge showing a too large thickness of 10.6 inches (due to internal cracking) and a possible crack echo at 2.7 inches

The Impact Echo method can be quickly trained for field testing to predict thicknesses of concrete bridge decks, pavements, and walls. A recent CALTRANS research study reported an accuracy of 0.16 inches on nominally 8 inch thick new concrete pavements on a 6 inch thick concrete base for CTG impact echo tests when correlated to one core thickness for multiple tests for thickness QA (Maser et al, 2002). However, more training and experience are required for analysis of IE results to identify internal concrete flaws such as void, cracking, honeycomb, delaminations, etc., and for testing of columns and beams. Beams, columns and other non-plate shapes have multiple resonances due to their geometry and different shape factors in equation 2 that depend on their aspect depth to width ratio (Sansalone and Streett, 1997).

2.4 Impact Echo Scanning for Post-Tensioned Ducts, Decks and Pavements

The Impact Echo Scanning technology was first developed in the early 1990's by the author's firm and then used as a part of a US Bureau of Reclamation prestressed concrete cylinder pipe research project (Sack and Olson, 1995). This section describes the implementation of an IE Scanner with a rolling displacement transducer and electric solenoid for impact echo scanning of concrete structures. IE scanning for concrete thickness and flaws is analogous to ground penetrating radar scanning for rebar in concrete.

To expedite the IE testing process, an Impact Echo Scanning device was researched and developed with a rolling transducer assembly incorporating six transducers in a rolling sensor wheel which is attached underneath the test unit. When the test unit is rolled across the testing surface, an opto-coupler on the central wheel keeps track of the distance. This unit is calibrated to apply an electric solenoid impact at intervals of about 1 inch. The Freedom Data PC acquisition system with the Impact Echo Scanning unit is shown in Fig. 14.

Figure 14– IE Scanning Unit with Olson Instruments Freedom Data PC Acquisition System

2.4.1 Internal Grout Condition Evaluation with IE on a PT Bridge

Post-tensioned systems are widely used for bridge structures. However, one potential problem in the construction is that the duct may not be fully grouted, leaving voids in some areas. Over the long term, water could enter the tendon ducts through the voids resulting in corrosion of the tendon. Therefore, it is necessary to insure the quality of the grout fill inside the ducts after the grouting process is complete.

The Impact Echo Scanning method is showing considerable promise and increasing use for evaluating the grout condition in PT ducts due several reasons. First, the technique requires on only one-sided access which is practical for testing ducts inside a bridge. Second, the method uses a scanning head, which expedites the test process by allowing near-continuous testing at 1 inch intervals directly along a duct or across several ducts. Third, the results in time and frequency domains can be processed to give a two dimensional intensity display (impactechogram), which yields more details of what is inside the ducts. Fourth, 3-D images of duct conditions from IE are now becoming possible and practical as well (Maierhofer et al 2004).

As mentioned in Section 2.3, IE results are more apparent in the frequency domain. From a Finite Element model, the natural fundamental frequencies of a solid structure are different from those of structure with voids inside (Sansalone and Sansalone and Streett, 1997). In general, voids inside a duct shift the natural fundamental frequencies of the tested member to a lower frequency. In other words, the IE thickness

This segmental bridge wall with 4 steel ducts appears to have two zones of ungrouted ducts: at 2-2.3 feet from the start of the scan, and at 3.2-3.4 feet. Apparent honeycomb/voids are present at 3.5 ft to the end of the scan at 4.5 feet (scanning from bottom to top).

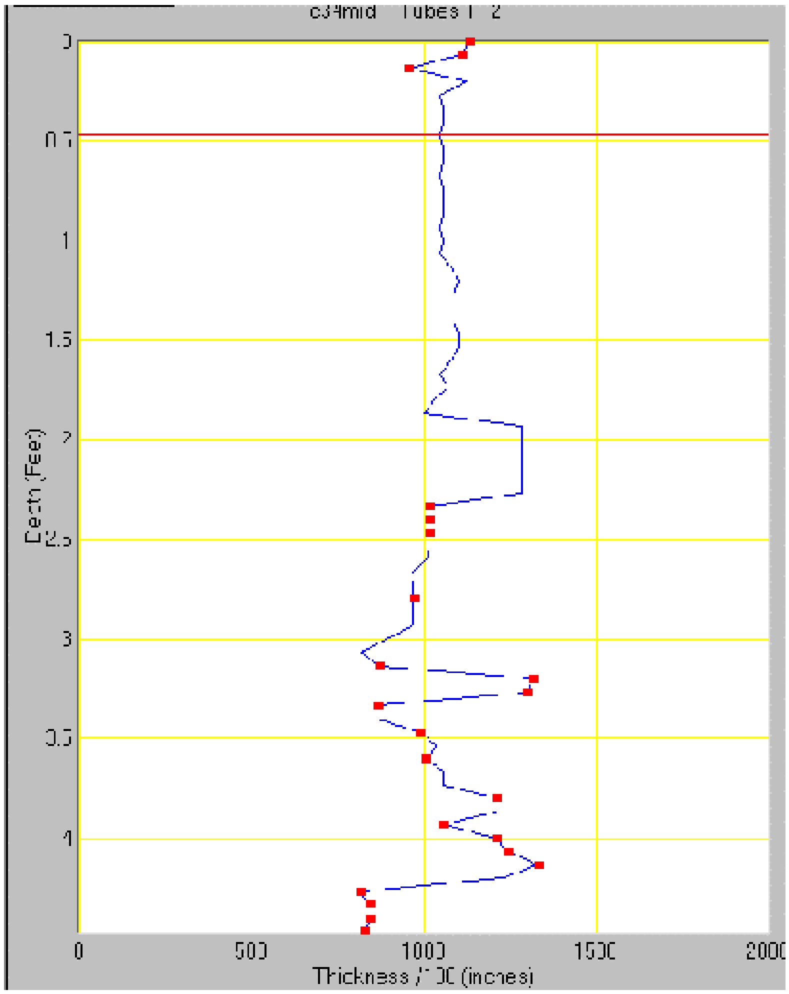


Figure 15 – Impact Echo Scan Results for apparent back wall echo thickness for vertical scan

results on the wall appear to be thicker in the presence of voids (due to the decreased stiffness and resonance of the wall section. Echoes from the depth of the duct may also be detected for void conditions if high frequencies are excited

Impact Echo Scanning was used to locate ungrouted area in a post-tensioned cable stayed bridge. This section gives an example result from a vertical scan across steel ducts in Figure 15 where the vertical axis is depth or distance in ft of the scan from the top down and the horizontal axis is the thickness echo (inches/100). By normalizing the Impact Echo spectral amplitudes and plotting versus a frequency range of 0 to 20,000 Hz on the horizontal axis as shown in Figure 16, one obtains a mirror image of Figure 15. By combining 2-D Impactechograms, one can obtain 3-D images that show thickness echoes as well as echoes from flaws versus echo amplitude depth. The void and honeycomb conditions were confirmed using drilling for video borescope viewing and by special void volume measurements on this project.

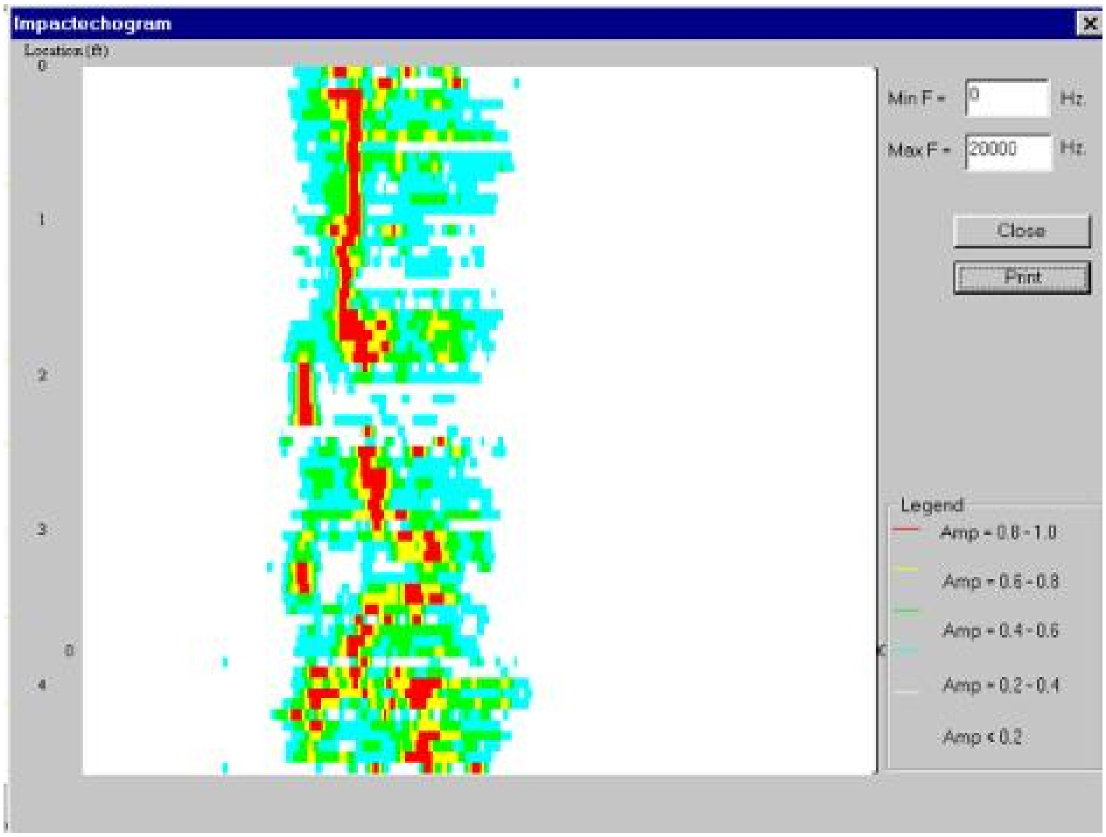


Figure 16 - Impactechogram from a Steel Post-Tensioned Duct Bridge

2.5 Spectral Analysis of Surface Waves (SASW) Method for Concrete Condition

One stress-wave based NDT method, which has recently been applied to structural and geotechnical testing, is the SASW method (Bay and Stokoe, 1990). This is based on a similar procedure used for the determination of shear moduli profiles at soil sites (Stokoe et al, 1988) and Young's moduli profiles at pavement sites (Nazarian and Stokoe, 1985). The method relies on the dispersion characteristics of surface waves.

In SASW tests, two receivers are placed on the ground/pavement/structural member surface to monitor the passage of surface waves due to an impact from a source placed at distance from Receiver 1 equal to the distance between the two receivers. Figure 17 shows a concrete thickness gauge (model CTG-1SW) with an added SASW bar used in investigating concrete slabs, walls, columns, where both IE and SASW data are desired A digital analyzer is used to record the receiver outputs for spectral (frequency) analyses. The result of the analysis is a plot of the phase difference between the two receivers versus frequency. A dispersion curve (surface wave velocity versus wavelength) is calculated from the phase plot using the following equations:

$$t = \phi/360 \quad (3)$$
$$V_R = X/t \quad (4)$$
$$\lambda_R = V_R/f \quad (5)$$

where t = Time, V_R = Surface Wave Velocity, X = distance between receivers, λ_R = Wavelength, and f = Frequency.

Figure 17 - SASW investigation on bridge deck

The final process in SASW testing is the Forward Modeling process to determine the shear wave velocity profile. The forward modeling process is an iterative process, and involves comparing the actual dispersion curve with a theoretical dispersion curve calculated from an assumed shear wave velocity profile.

To illustrate the use of the SASW test method, concrete SASW results from the afore-mentioned bridge deck (where IE tests were also performed above) are presented in Fig. 18. Note that the surface wave velocity at Location 1 is equal to 6,500 ft/sec. For a Poisson's ratio of 0.2, the surface wave velocity is equal to 0.56 * compression wave velocity. Therefore, the compression wave velocity at Location 1 is equal to 11,600 ft/sec, which agree extremely well with values back-calculated from IE tests. Therefore, a combination of IE and SASW tests performed at the same locations can be very useful if the thickness of concrete elements is not known. In this case, the compression wave velocity determined from SASW tests can be used as an input for thickness evaluation from IE tests.

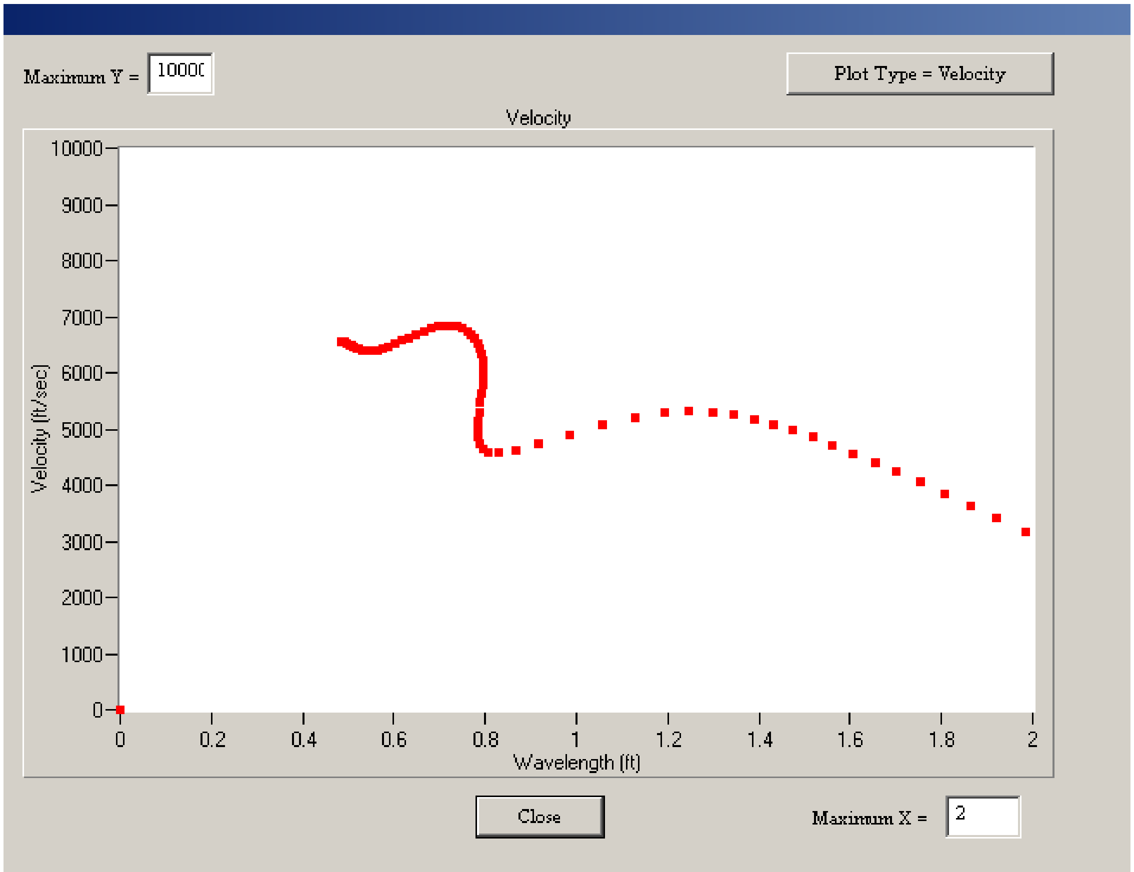


Figure 18 – SASW velocity plot showing approximate thickness = 0.8 ft, surface wave velocity = 6,500 ft/s.

The SASW method is a more complicated method for analysis, but field data acquisition can be straightforward for concrete pavement/bridge deck thickness evaluation when combined with IE tests using the CTG-1SW. The SASW method can uniquely evaluate fire damage, frost damage, perpendicular cracks and concrete quality/strength with 1-sided access (ACI 228.2R-98).

2.7 Slab/Structural Impulse Response (SIR) Method

Subgrade support for slab-on-grades and structural element conditions can be nondestructively evaluated with the Slab/Structural Impulse Response (SIR) method. The SIR tests can detect and define the extent of good versus void/poor support/concrete conditions for thin sections, honeycomb, void, etc. The method was developed from a force-response modal vibration test for investigating the integrity of deep foundations and was originally adapted for slabs by a European group as discussed in Section 3 herein.

The SIR tests are usually conducted from the surface of the slab/structure. Test equipment includes an impulse hammer, geophone receiver, and a PC-based signal analyzer. The tests involved hitting the slab to generate vibration energy in the slab. The

3-lb impulse hammer has a built-in load cell with a plastic head to measure the force of the impact. The vibration response of the slab to the impact is measured with the geophone held in contact with the concrete close to the point of impact, as illustrated in Fig. 19. The analyzer performs Fast Fourier Transform (FFT) operations on the time domain data to produce the mobility plots.

Figure 19 - Slab Impulse Response on underside of pre-stressed box girder bridge showing 3 lb impulse hammer and geophone

Support condition evaluation includes two primary measurement parameters. The slope of the initial straight-line portion of the mobility plot indicates the quasi-static flexibility of the system. The low frequency flexibility provides a general indication of the slab stiffness since the inverse of flexibility is dynamic stiffness. The steeper the slope of the line, the more flexible and less stiff the system is. Dynamic stiffness can be correlated to static stiffness. In a simple sense then, the dynamic SIR test is analogous to placing a weight on the slab and measuring the deflection of the slab to calculate the low strain static stiffness (pounds force per inch displacement). The shape and/or magnitude of the mobility at frequencies above the initial straight-line portion of the curve is the second indicator of support conditions. The response curve is more irregular and has a greater mobility for flaws such as thin sections in a box girder versus good thick concrete conditions due to the decreased damping of the vibration response for a thin member. The flexibility is also greater which corresponds to reduced stiffness of the thin box girder slab.

Example SIR data for sound and thin conditions are presented for box girder bottom slabs in Figures 20 and 21, respectively. The upper graphs in Figs. 20 and 21 represent the coherence functions for each test point data set. A value of coherence close to 1 indicates good quality data. The lower graphs in Figs. 20 and 21are the mobility plots in unit of velocity / force (inches per second / pounds force). Note that both graphs are plotted at the same scale to allow visual comparison of the mobility levels. Note the irregular mobility plot versus frequency and the high mobility values (poor) in Fig. 21 as compared to the smooth shape of the mobility plot and the low mobility values (sound) in Fig. 20. The average mobility between 100 and 800 Hz for the sound, 6.7 inch thick concrete in Fig. 20 is 1.7 E-6 in/sec/lbf, while the average value for the thin location, 2.6 inch thick concrete in Fig. 21 is 6.5 E-6 in/sec/lbf (the thicknesses were determined with IE tests). Similarly, the initial slope for the thin location is almost 5 times higher than that for the sound location which means the thin bottom slab is 1/5 as stiff as the normal thickness slab. Off-center SIR tests at this bridge verified the ability of the SIR to detect off-center voids or unconsolidated concrete.

The SIR test requires training and experience, but is not as complex as the SASW test in terms of data analysis. It is most sensitive to near-surface concrete conditions.

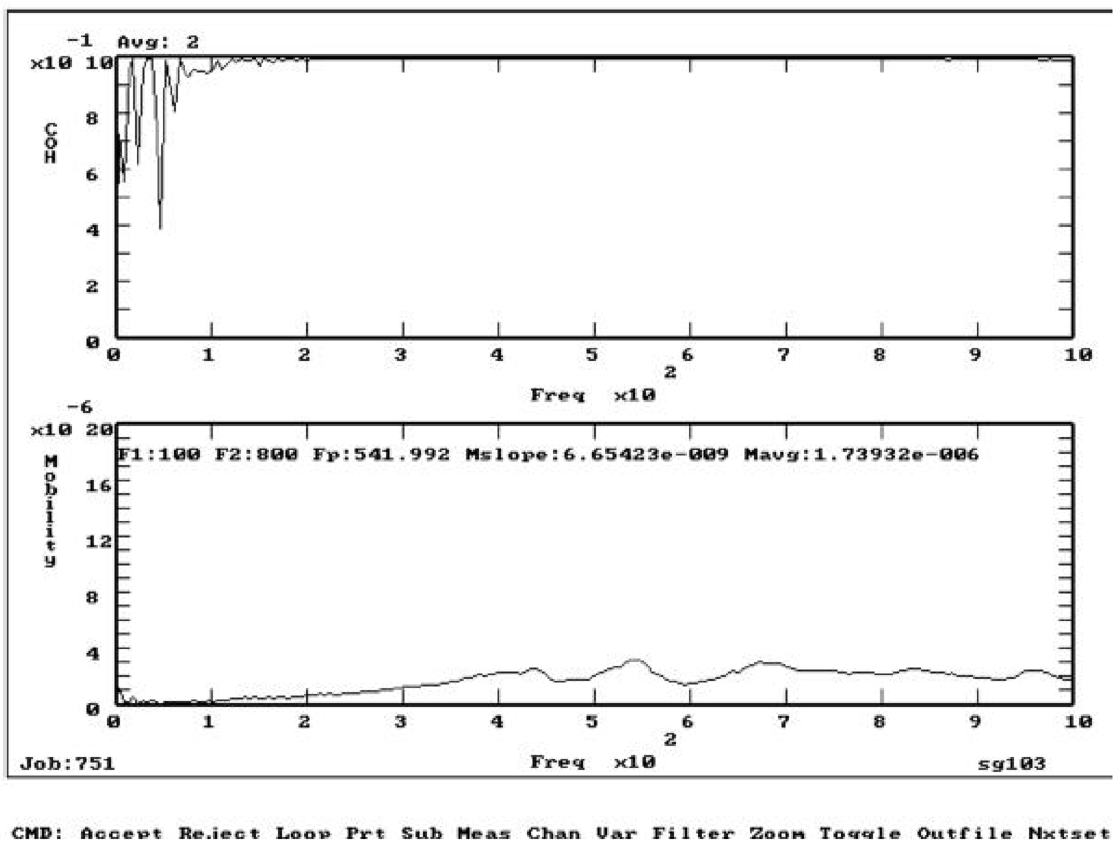


Figure 20 - Example Slab Impulse Response record showing normal concrete on a freight rail bridge

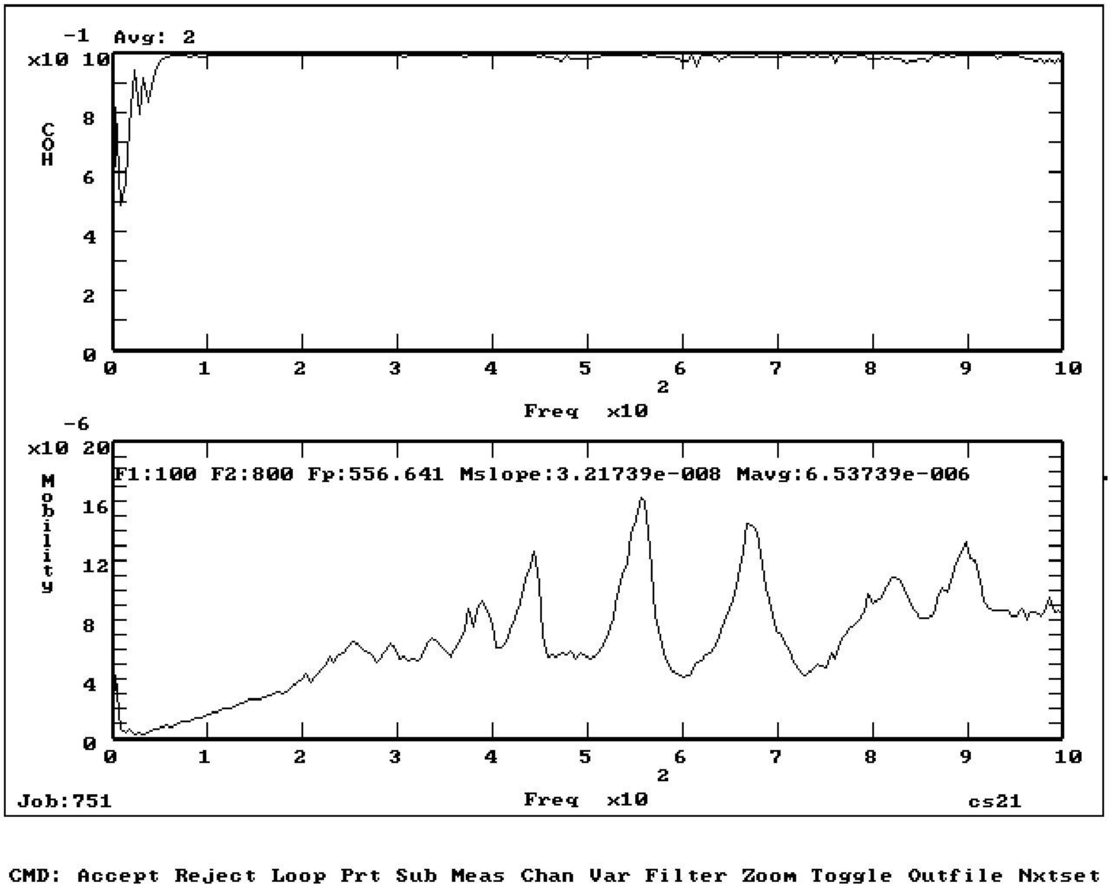


Figure 21 - Example Slab Impulse Response record showing thin concrete on a light rail bridge

3.0 NDE METHODS FOR CONCRETE BRIDGE SUBSTRUCTURE

Bridge foundations are commonly tested with NDE methods for quality assurance, trouble-shooting and investigation of unknown depths and conditions for scour/earthquake/performance evaluation purposes. The crosshole sonic logging, crosshole tomography, sonic echo/impulse, ultraseismic and parallel seismic NDE methods are discussed below. Most of these methods are discussed in ACI228.2R-98.

3.1 Crosshole Sonic Logging Method

The CSL method was adopted in the U.S. in the mid 1980's for quality assurance of drilled shaft foundations, diaphragm and barrette walls and seal footings. The CSL method relies on direct transmission of sonic/ultrasonic waves between 2 or more access tubes placed in a drilled shaft prior to concrete placement (Figure 22) and is detailed in ASTM D6760-02. The number of access tubes per drilled shaft is dependent on the diameter of the shaft, typically 1 tube per 0.3 m (1 ft) of diameter, and the tubes are installed around the perimeter of the shaft and tied to the inside (or outside) of the cage of the shaft. The tubes are usually 38 to 50 mm (1.5 to 2.0 in.) inside diameter schedule 40 steel or PVC pipe. Tube debonding from the surrounding concrete can occur at an earlier time in PVC tubes as compared to steel tubes. Most state DOT's specify that CSL tests be performed in 10 days or less after concrete placement for PVC tubes and in 45 days or less for steel tubes to avoid problems associated with tube debonding.

To perform a CSL test, two probes (hydrophones) are lowered to the bottom of two access tubes, and are retrieved to the top of the shaft while CSL measurements are taken approximately every 50 mm (2 in.). The ultrasonic wave pulser is controlled by a distance wheel to trigger the transmission of waves at pre-selected vertical intervals. Automatic scanning of the collected records produces two plots, time (or velocity) and energy, versus depth. Anomalies and defects between tested tubes are manifested by time delays (or velocity decreases) and energy drops in the scanned CSL plot. Concrete velocities are calculated by simply dividing the distance between the two tubes by the time required for the wave to travel from the source hydrophone to the receiver hydrophone. CSL tests are typically performed between all perimeter tubes to evaluate the concrete conditions of the outer part of the shaft and between major diagonal tubes to evaluate the concrete conditions of the inner part of the shaft. Typical CSL test equipment is shown in Figure 23 and Figure 24 presents a CSL log.

Desirable results show consistent pulse arrival times with corresponding compression wave velocities that are reasonable for concrete. Defects such as contaminated, weak concrete and soil intrusions will result in delayed arrivals (slower velocity) or no arrivals in the defect zone. The signal energy level is a secondary indicator of concrete quality with low energy indicating poorer quality concrete. The wave velocity increases with time in concrete as it matures, particularly in the first few days of curing. A CSL log for a sound shaft foundation showing the first arrival time and energy plots has been included as Figure 24.

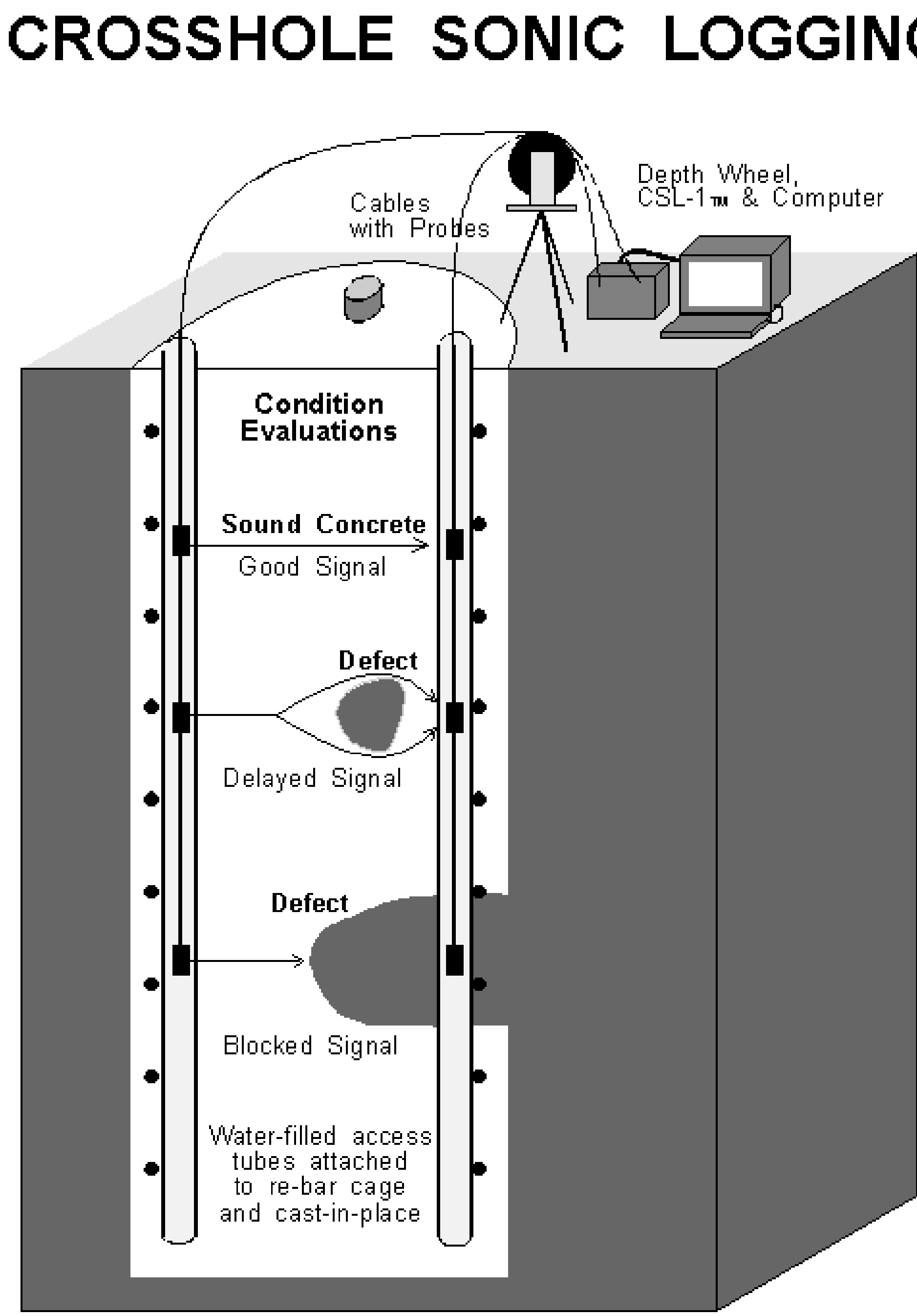


Figure 22 - Crosshole Sonic Logging Test Method

Figure 23 - Crosshole Sonnic Logging Typical Test Setup

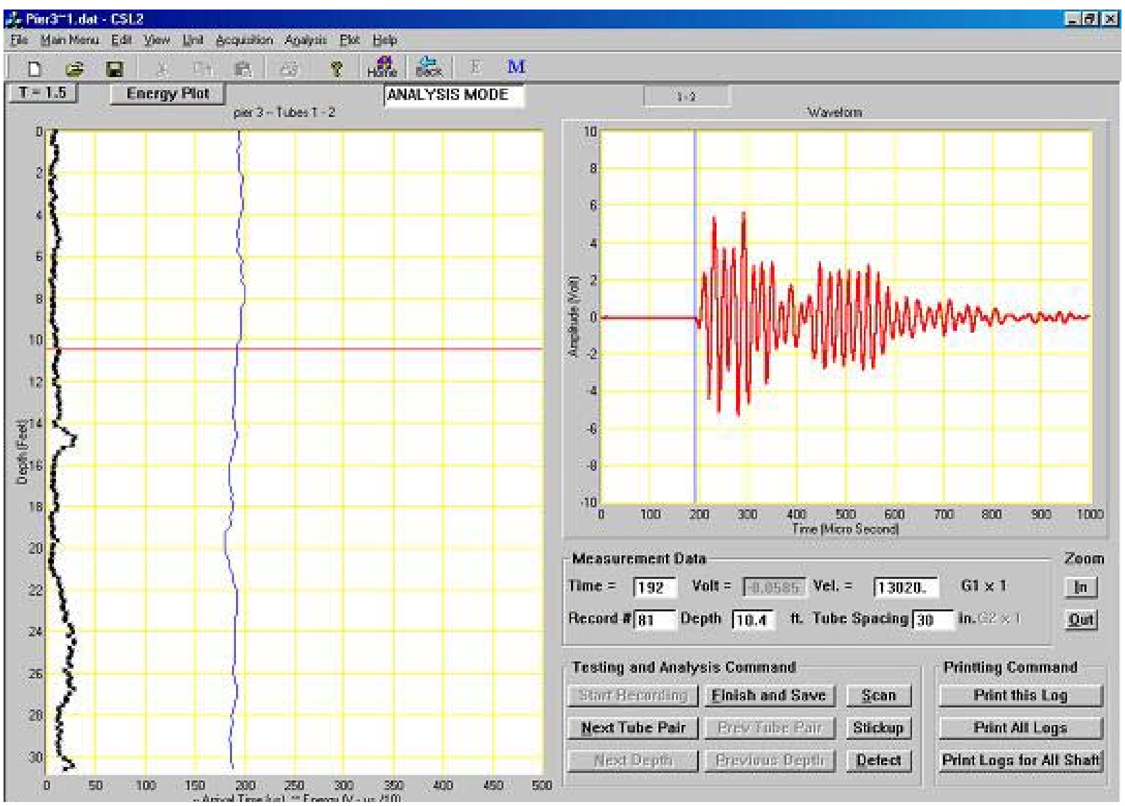


Figure 24 - CSL log for a SOUND shaft foundation (left) showing the first arrival time (blue) and energy plots (black asterisks). A single recorded time domain signal is shown in red (right) for the red cursor depth position in the log.

Initial compressional wave (P wave) arrival times are automatically picked by the CSL software program or manually picked by the user. The arrival times are then plotted versus depth to produce a CSL log like that shown in Fig. 24 in what is known as a FAT plot (First Arrival Time). Fig. 24 shows the CSL log and a single time domain signal for a sound (no anomalies) concrete shaft foundation, 32 ft long, from a consulting project. First arrival times are plotted in blue (light line) and receiver output energy is plotted in black asterisks. The time domain signal recorded at a depth of 10.4 ft below the top of the shaft is displayed on the right side of the screen in red and the first arrival time is marked with the vertical cursor. The tubes used for the CSL test recorded in Fig. 24 were 30 inches apart. At a depth of 10.4 ft, the velocity of the concrete, V_c, can be determined by:

$$V_c = D / t_p = 30 \text{ inches} / 192 * 10^{-6} \text{ s} = 13020 \text{ ft/s} \quad (1)$$

where D is equal to the measured tube spacing at the surface in inches and t_p equals the first arrival of the compressional wave energy (P wave).

3.2 Crosshole Tomography Method

The Crosshole Tomography method uses the same equipment as the CSL method with more tests being collected (Figure 25 shows many angled and horizontal source and receiver locations). Once a defect is identified in CSL tests, CT tests can be performed to produce an image of the defect between the test tubes.

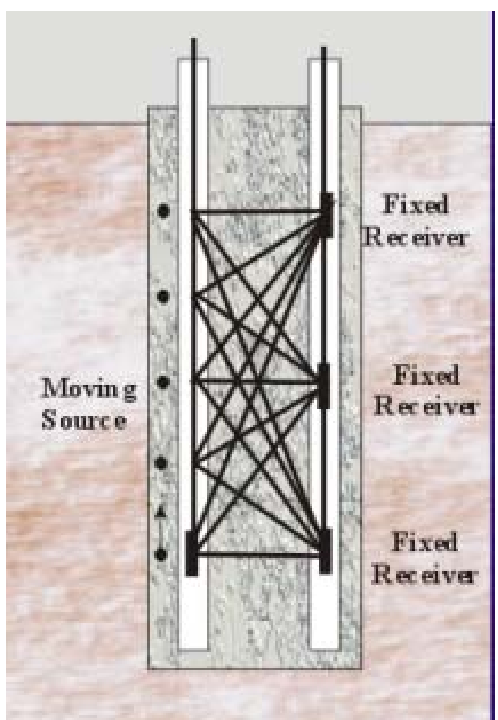


Figure 25 - Crosshole Tomography (CT) test schematic for concrete foundation quality assurance

The CT method uses the same equipment and access tubes as the CSL method. For CT testing, acoustic data are collected for many receiver and source combinations at different depths (Fig. 25) whereas CSL testing is for source and receiver positions at the same depth or horizontal plane. For a typical CT data set, thousands of raypaths are generated for hundreds or thousands of source-receiver location combinations. For CT testing, the receiver is fixed at a given depth and the source is pulled from the tomogram bottom extent to top and generates sound wave energy at 0.2 ft vertical intervals (Figure 26). The source is typically pulled from shaft bottom to top to ensure proper ray coverage (± 45^{o} from horizontal) and to simplify field testing procedures.

Figure 26 – Crosshole Tomography (CT) on a bridge pier

Tomography is an inversion procedure that can provide for ultrasonic images of a concrete zone from the observation of transmitted compression or shear first arrival energy. The CT data is used to obtain an image of the defect. The test region is first discretized into many cells with assumed slowness values (inverse of velocity) and then the time arrivals along the test paths are calculated. The calculated times are compared to the measured travel times and the errors are redistributed along the individual cells using mathematical models. This process is continued until the measured travel times match the assumed travel times within an assumed tolerance. Tomographic analysis can be performed using series expansion algorithms with a curved ray analysis from geotomography. Tomography is time consuming and its use is justified for critical shaft defects. For more details on how tomography is applied for imaging defects, the reader is referred to Olson et al (1993) and Hollema and Olson (2002).

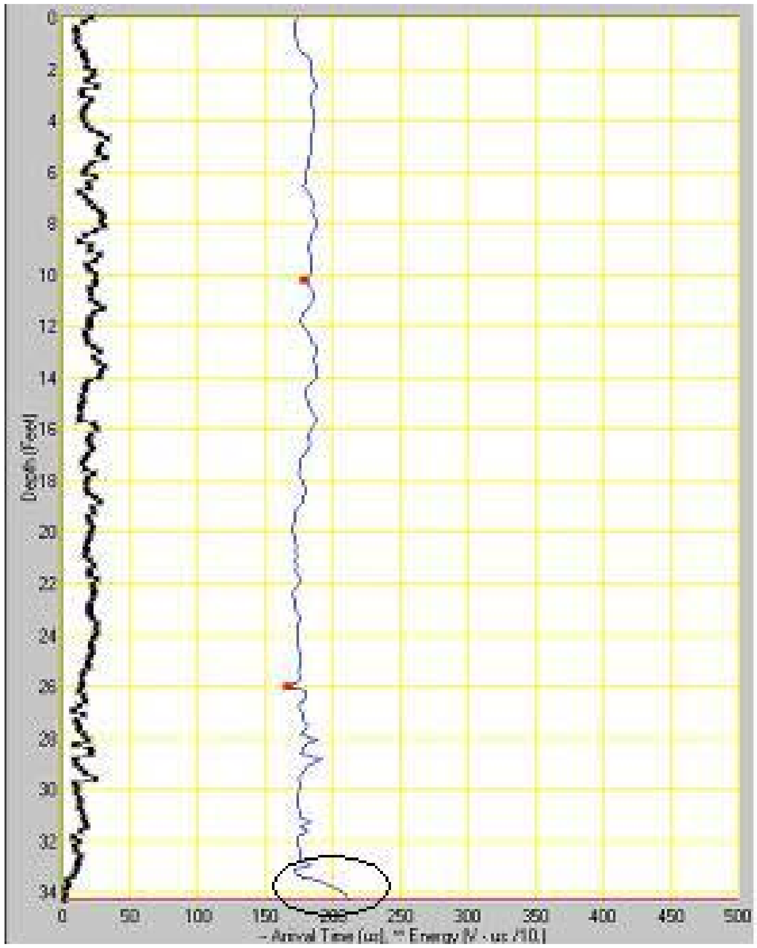


Figure 27 - CSL log showing defect in bottom of shaft, CT will confirm severity of defect

Crosshole Tomography (CT) testing was performed on 6 drilled shaft bridge foundations after CSL results indicated partial defects between 6 tubes in the shafts. The shafts tested were foundations for a future highway overpass bridge. The CT testing was performed to image the size, shape, and exact location of possible defects discovered during previous Crosshole Sonic Logging (CSL) testing by another firm and to determine the severity of the anomalies by 2-D and 3-D velocity tomograms. Limited CSL testing was performed prior to CT testing by Olson Engineering to confirm the anomalies previously discovered. The CSL and CT results for 1 tested foundation are presented and discussed below. In addition to the 2-D tests, 3-D CT was performed as well for the 6 perimeter and 3 diagonal tube pairs. CT data were collected in 9 tube pairs (panels), 1-2, 2-3, 3-4, 4-5, 5-6, 6-1, 1-4, 2-5, and 3-6, from approximate depths of 34.5 ft (shaft bottom) to 25.0 ft. This survey was designed to image anomalies discovered with the CSL method in all tube pairs near the shaft bottom. Receiver vertical increments were 0.5 ft and source energy was generated at 0.2 ft vertical increments. The 3D data set was iteratively inverted for velocities with a 3D SIRT routine. Fig. 27 presents the CSL results for tube pair 3-4, revealing a soft bottom anomaly. This type of anomaly is often found with the CSL method in wet-hole placed foundations. From the CSL results, it is not possible to pin-point the exact size or location of the 2 anomalies. We can tell that 1 anomaly is occupying a portion of the travel path between tubes 3 and 4 and another occupies a portion of the travel path between tubes 4 and 5. CT analysis is necessary to obtain size and location information of this anomaly.

The 3D velocity tomography results are presented in the form of tomogram compilations in Figs. 28 and 29 below. Fig. 28 shows a compilation of vertical tomograms with the defects lying at tubes 1, 3 and 5 at around 34 ft deep. The perimeter tomograms are organized in an "orange peel" arrangement, as if the shaft were sliced along tube 1 and peeled open laterally. Additional diagonal tomograms (across shaft center) are shown in Fig. 28 as well. The legend at the figure bottom explains the color assignments. Shaft voxels with less than a 10% velocity reduction from the mean foundation velocity (11.7 kilofeet per second, kfps) are plotted in light green and voxels with velocity reductions greater than or equal to 10% of the mean foundation velocity are shown in varying shades of green, orange, yellow, and white. Three anomalies with major velocity reductions are visible in both the perimeter and diagonal compilations, near tubes 1, 3, and 5, labeled A, B, and C, respectively. Fig. 29 shows a horizontal slice through the shaft at a depth 33.5 ft. The color assignments are the same as those of Fig. 28. Anomalies A, B, and C are clearly evident in the horizontal slice format as well and show the plan view extent of the defects at tubes 1, 3 and 5 at a depth of about 34 ft. The 3D tomographic results in Figs. 28 and 29 accurately quantified the size, shape, severity, depth, and location of shaft defects. The bridge engineers ultimately decided that given the smaller size of the defects and the higher than specified strength of the concrete that the shafts would perform satisfactorily without costly repairs.

The use of Crosshole Sonic Logging (CSL) to identify concrete defects in drilled shafts for wet holes has become a proven QA method for most DOT's in the U.S. Now, 2-D and 3-D Crosshole Tomograms (CT) are practical and powerful for use in imaging CSL anomalies to characterize the size, shape, extent, and severity of potential defects. The CSL and CT methods have proven to give accurate and reliable results when performed to locate known defects in the research arena. The high level of proven confidence in these imaging methods are extended to the shaft contractor and/or design engineer for reliable QA of foundations in the real-world arena.

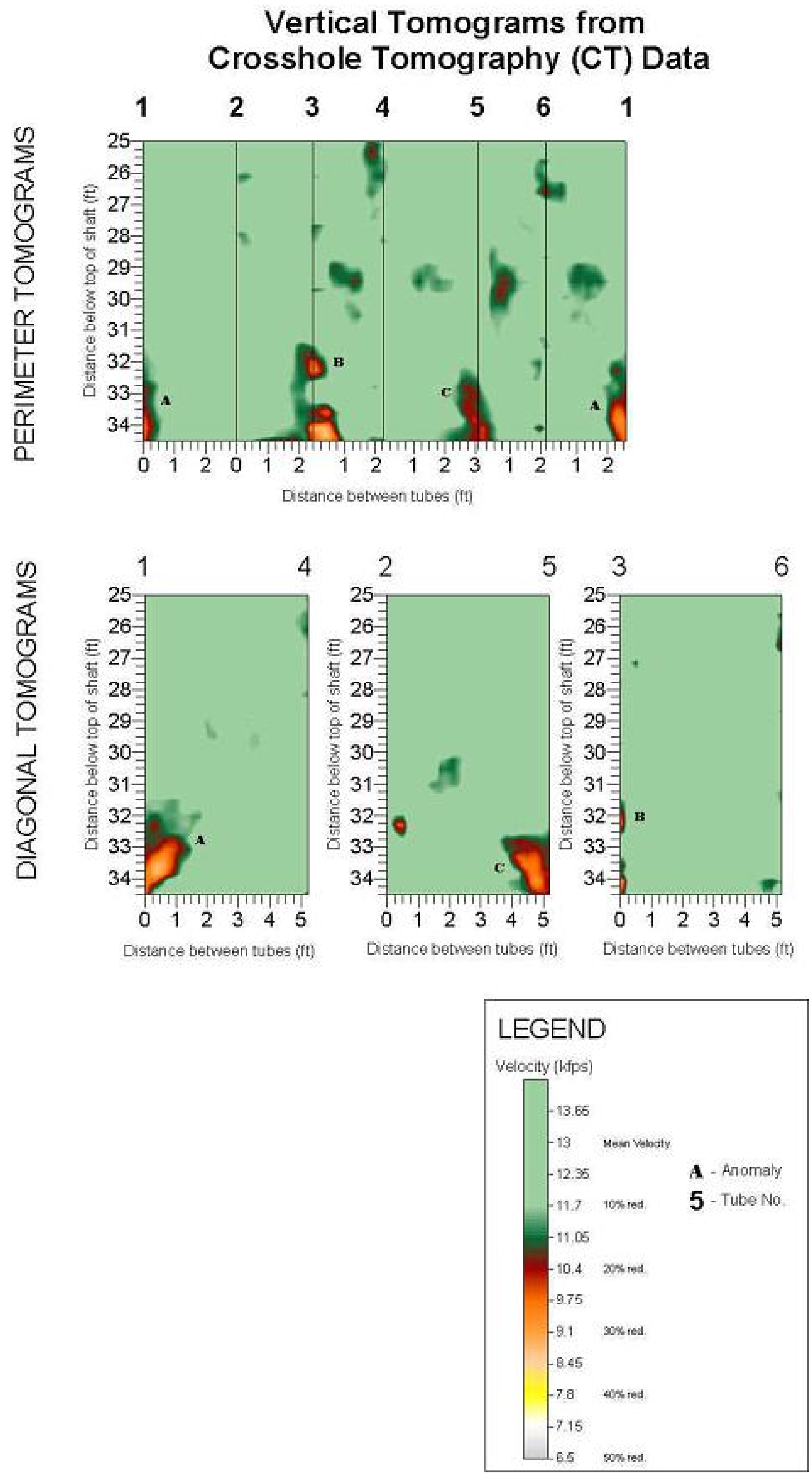


Figure 28 - Sample tomograms of a 6-tube shaft confirming defect regions in the bottom of the shaft as indicated by CSL above in Figure 29

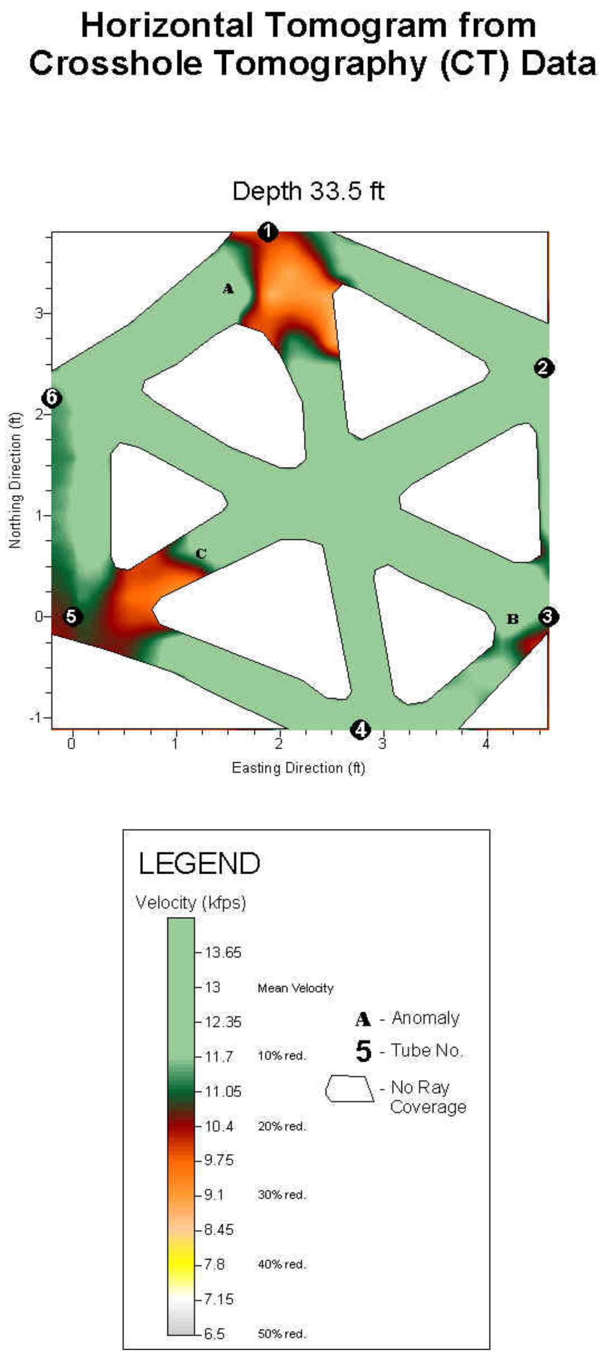


Figure 29 - Horizontal tomogram compilation of 3D CT data for Shaft

3.3 Sonic Echo/Impulse Response (SE/IR) Low-strain Pile Integrity Method

The Sonic Echo (SE) method is a low strain integrity test conducted from the surface (ASTM D5882-00). Test equipment includes an impulse hammer (optional, an ordinary plastic tipped hammer) and an accelerometer (or geophone) on the shaft. The impulse hammer has a built-in load cell that can measure the force and duration of the impact (needed for IR tests). The test involves hitting the foundation top with the hammer to generate wave energy that travels down the foundation. The wave reflects off irregularities and/or the bottom of the foundation and travels up the foundation to the foundation top. The receiver measures the vibration response of the foundation to each impact. The signal analyzer or PC processes and displays the hammer and receiver outputs. Foundation integrity is evaluated by identifying and analyzing the arrival times, direction, and amplitude of reflections measured by the receiver in time. The receiver output is usually integrated (if an accelerometer is used) and exponentially amplified with time (Koten and Middendorp, 1981) to enhance weak reflections. Digital filtering with a low-pass filter of about 2,000 Hz is usually applied to eliminate high frequency noise. In some cases, where reflections are difficult to identify, an impedance imaging procedure is used to obtain a 2-D image of the shaft (Paquet, 1991).

The Impulse Response (IR) method is also an echo test and uses the same test equipment as the SE method. The test procedures are similar to the SE test procedures, but the data processing is different. The IR method involves frequency domain data processing, i.e., the vibrations of the foundation measured by the receiver are processed with Fast Fourier Transform (FFT) algorithms to generate transfer functions for analyses (Davis and Dunn, 1974). The coherence of the impulse hammer impact and accelerometer receiver response data versus frequency is calculated to indicate the data quality. A coherence near 1.0 indicates good quality data. In the IR records the linear transfer function amplitude is in velocity/force on the vertical axis (mobility) and frequency in Hz on the horizontal axis. The SE/IR method is illustrated in Figure 30 and further discussed in ACI 228.2R-98.

Example SE/IR records from bridge drilled shafts are presented in Figures 31 and 32, respectively. The lower SE trace shown in Fig. 31 is the same data as the upper acceleration response trace after it has been integrated and exponentially amplified. The echo times are at 6 and 12 milliseconds and the calculations of the necking anomaly defect depth are given belowl

T_1 = 6 ms T_2 = 12 ms
Compression Wave Velocity = 3658 m/sec (12,000 ft/sec)
Depth of Reflector = V_C * T1/2 = 3658 * 0.006/2 = 11 m (36 ft)

The reflection was identified as being from a necking defect anomaly located at about 11 m below the shaft top.

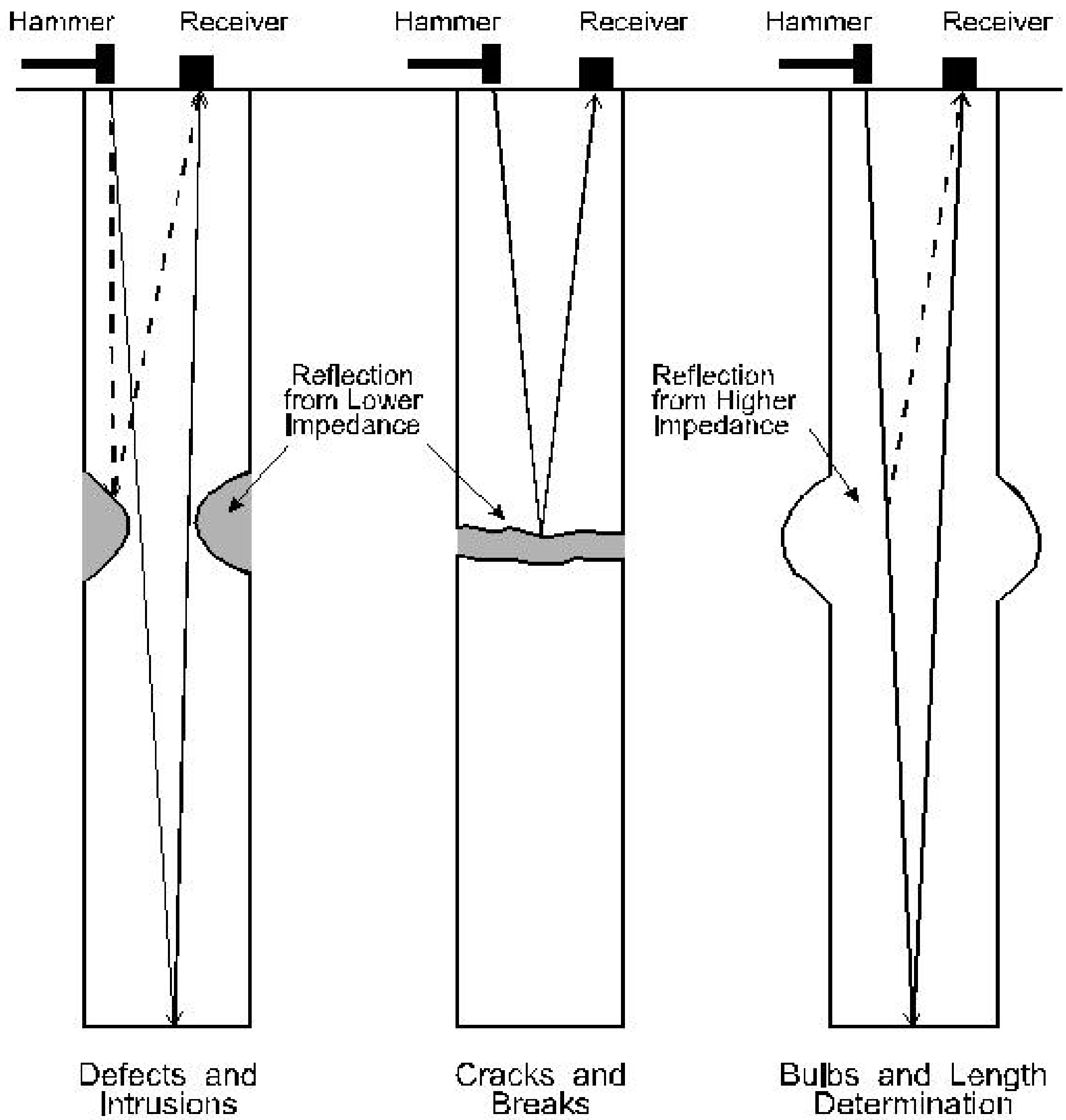


Figure 30 – Sonic Echo/Impulse Response Test Method for Deep Foundations

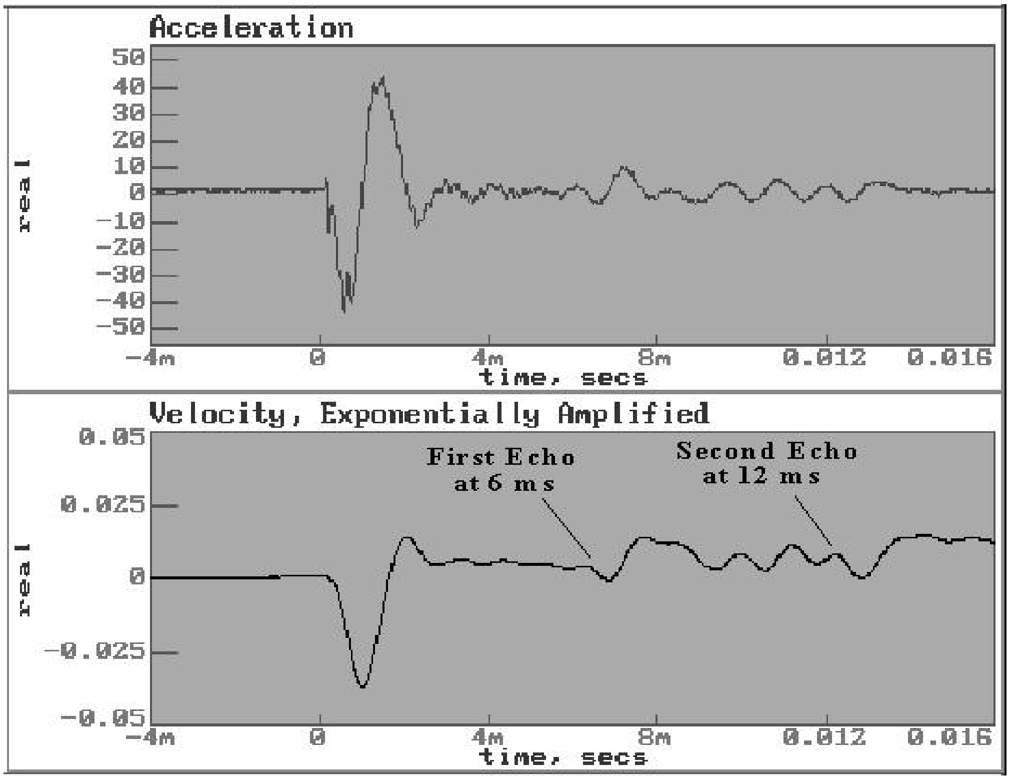


Figure 31 – SE result illustrating an anomaly at approximately 11m in a drilled shaft

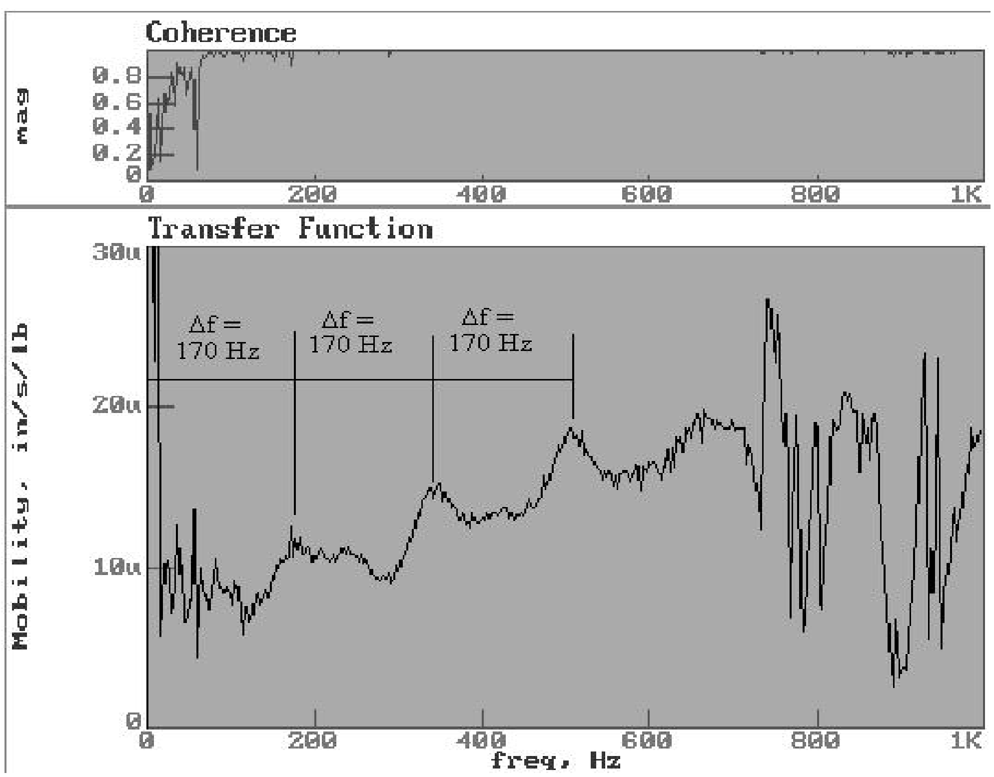


Figure 32 - IR result illustrating an anomaly at approximately 11m in a drilled shaft

The IR result for the upper trace shown in Fig. 32 is the coherence function to reflect data quality with a value near 1.0 being good quality data. The lower trace is the mobility function used to obtain reflector depths based on the spacing between resonant peaks in the plot of mobility versus frequency as calculated below.

$\Delta f = 170$ Hz
Compression Wave Velocity = 3658 m/sec (12,000 ft/sec)
Depth of Reflector = $V_C/\Delta f * 2 = 3658/170 * 2 = 10.8$ m (35.3 ft)

The reflection is again seen to be from a necking anomaly located at about 11 m below the shaft to which agrees with the SE result. IR data can also be analyzed to determine the shaft flexibility/stiffness at its head and theoretical mobility to further indicate the severity of a defect (ACI 228.2R-98).

The SE/IR method is commonly used for quality assurance of concrete drilled shafts and driven piles. Equipment field use and training is relatively straight forward. Interpretation of the results for partial area defects generally requires at least a 25% area defect and experience to interpret the data. Impedance imaging is offering promise to ease the interpretation. Foundations with attached substructure can be tested for length and integrity if pile caps are not too thick relative to the pile diameter, or if the piles are exposed. However the ultraseismic and parallel seismic methods discussed below may need to be used instead for more complicated substructure systems.

3.4 Ultraseismic (US) Method for Unknown Substructure/Foundation Depths

The Ultraseismic (*US)* test involves impacting exposed substructure to generate and record the travel of compression or flexural waves down and up substructure at multiple receiver locations on the substructure as shown in Fig. 33. This test combines the capabilities of the SE/IR and Bending Wave (BW) measurements with geophysical processing to separate reflections of wave energy coming from foundation elements versus reflections from the top of exposed substructure and came out of the research by the author for the NCHRP 21-5 and 21-5(2) (Olson and Aouad, 2000) projects for determination of unknown bridge foundation depths for scour safety evaluation purposes. The US method was found to be more accurate and applicable than the SE/IR or BW tests in this research and a guideline for the US and parallel seismic (PS) methods was also prepared (Olson, 2001). A recent discussion of the NCHRP 21-5 research was given by Olson (2003).

The Ultraseismic method is a sonic reflection technique that uses geophysical digital data processing techniques. With the US method, one analyzes the propagation of induced compression and flexural waves as they reflect from foundation substructure boundaries (impedance changes, i.e. changes in the multiple of wave velocity x density x cross-sectional area). This is the same principle that the Sonic Echo/Impulse Response and Bending Wave methods rely on as well. However, the data acquisition and processing for the US method involves recording and display of multiple channels of data to better track the reflections from foundation element interfaces and bottoms as discussed below. The Ultraseismic Vertical Profiling method is shown above in Fig. 33.

The Ultraseismic method was researched and developed by the authors to overcome the difficulties encountered by the Sonic Echo/Impulse Response method and the Bending Wave method tests on non-columnar and complex columnar bridge substructures. The Ultraseismic method is a broad application of geophysical processing to both the Sonic Echo/Impulse Response and Bending Wave tests in that the initial arrivals of both compression and bending waves and their subsequent reflections can be analyzed to predict unknown foundation depths.

Two types of Ultraseismic test geometries have been specifically introduced for this problem: US Vertical Profiling and US Horizontal Profiling. For a one dimensional imaging of the foundation depth and tracking the upgoing and downgoing events, the term Vertical Profiling (VP) test method is used as shown in Figure 33 below. In this method, the bridge column or 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 wavefield is taken in order to analyze all types of ensuing wave motion. A VP line can be run in both a columnar (a bridge pier or pile foundation) and a tabular (a bridge abutment) structure.

For two-dimensional imaging of the foundation depth, the term Horizontal Profiling (HP) test geometry is used where there is flat, horizontal access for testing such as the top of an accessible pier or abutment. In this less commonly applicable US method, the reflection echoes from the bottom are analyzed to compute the depth of the foundation. The source and receiver(s) are located horizontally along the top of accessible substructure, or any accessible face along the side of the substructure element, and a full survey is taken along the top of the element.

The main objective of Ultraseismic tests is to determine the unknown depth of the foundation. Example US results from the NCHRP research are presented herein to illustrate the use of the method and the interpretation of the results. The source/receiver layout for Ultraseismic Vertical Profiling tests on one of the pier columns of Bridge No. 5188, Minnesota Highway 58 in Zumbrota is shown in Fig. 34. The source point was able to be located vertically on top of the concrete beam. Thus, impacts were applied with downward vertical impacts to the top of the beam over the column using a 1.4 kg (3 lb) impulse hammer with a hard plastic tip. A 3-component accelerometer was mounted on the side of the exposed portion of the column at intervals of 0.15 m (0.5 ft) from 1.5 m (5 ft) below the top of the beam to near the ground surface.

Field data for a Vertical Profiling test done to measure flexural waves is shown in Fig. 35. The depth shown in Fig. 35 represents the depth below the top of the concrete beam. All data were debiased to remove any DC shift and f-k filtered to enhance upgoing waves. After this processing, two clear reflection events are apparent in the data. The first reflector corresponds to a depth of 2 m (6.5 ft) below Receiver 24 at a depth of 5 m

(16.5 ft) below the top of the concrete beam as shown in Fig. 35. The flexural wave velocity of 1,770 m/sec (5,800 ft/sec) was determined from the slope of the initial downgoing and reflected upgoing waves. This reflection corresponds to a depth of 7 m (23 ft) from the top of the concrete beam. The actual depth of the foundation was equal to 9.4 m (31 ft).

Further examination of Fig. 35 shows there is a second reflector at a depth of 4.3 m (14 ft) below Receiver 24 at a depth of 5 m (16.5 ft) below top of concrete beam. This reflection corresponds to a depth of 9.3 m (30.5 ft) from the top of the concrete beam. The actual depth of the foundation was equal to 9.4 m (31 ft). The shallower reflection is most likely due to a change in the subsurface material properties at this depth which is below water. The most important thing to note in Fig. 35 is that the flexural wave energy is first tracked down the bridge substructure, and that upgoing events correspond to reflectors whose depth can be predicted with geometry as shown. Downgoing events from the bridge structure above the impact point can be eliminated in the US data with the use of f-k filtering.

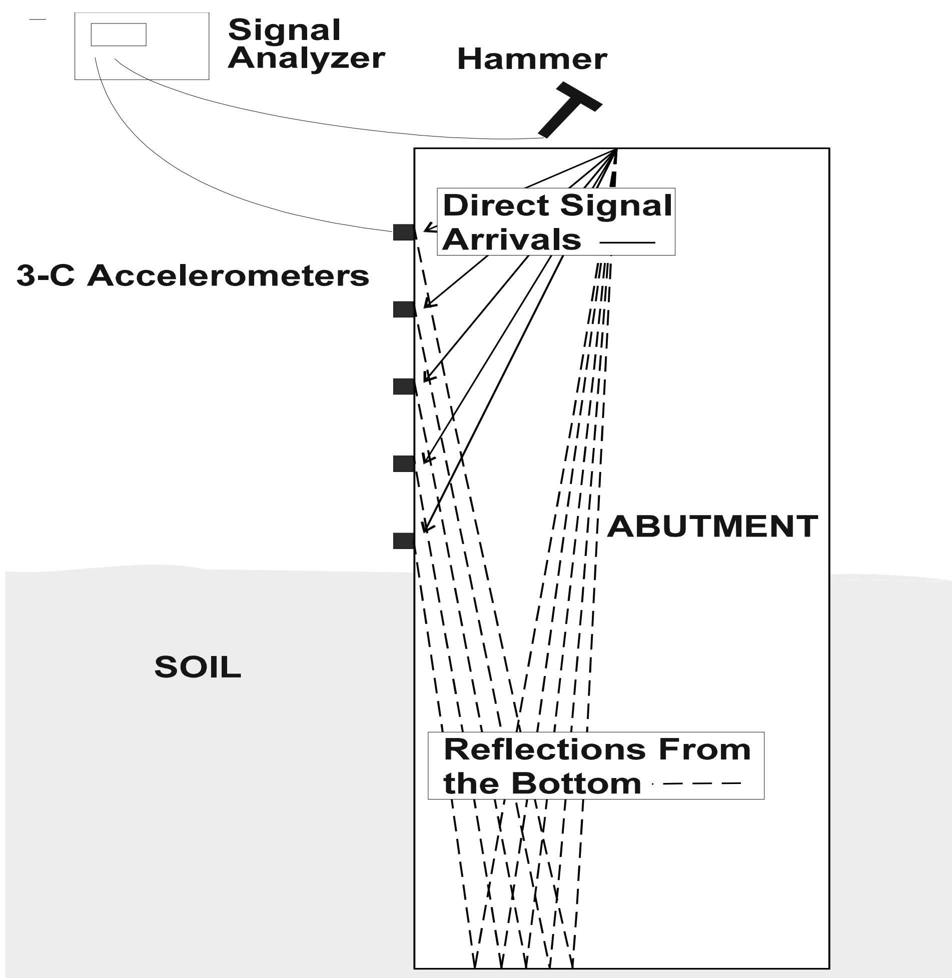
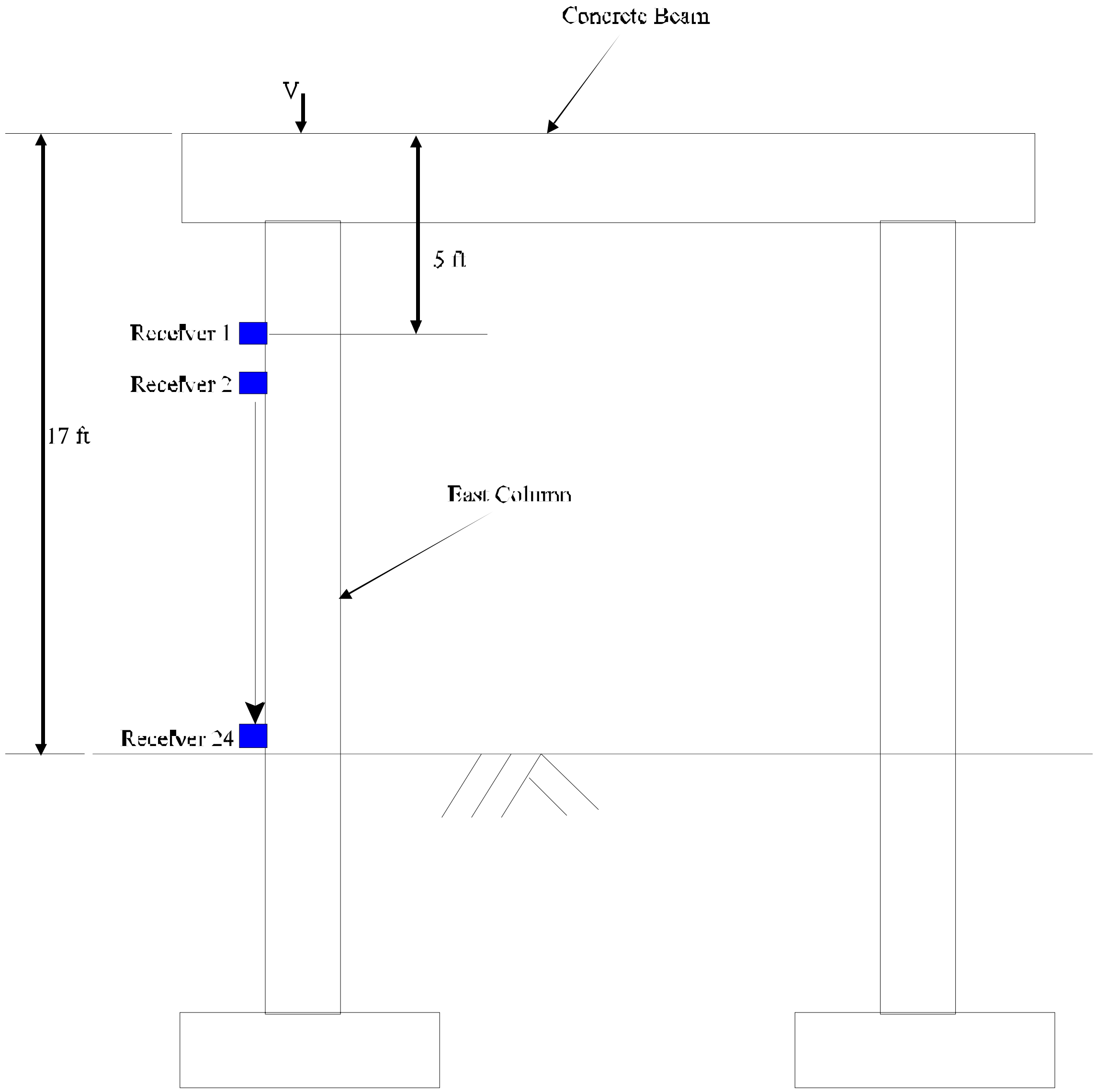
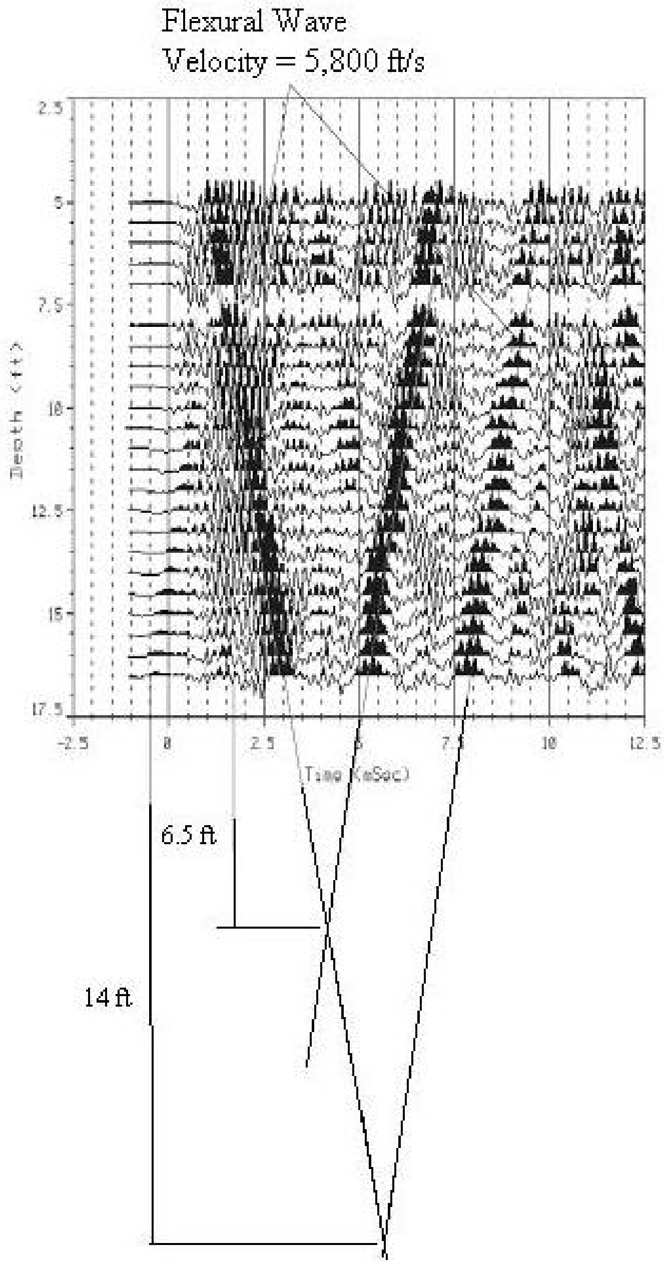


Figure 33 - Ultraseismic Test Method



V = Vertical Hammer Hit Location
24 Accelerometer Receiver Locations at Intervals of 0.5 ft

Figure 34 - Source/Receiver Layout for Ultraseismic Tests Performed at Bridge No. 5188 Minnesota Hwy 58, Zumbrota, Minnesota



Flexural Velocity at top arrows equals 5,800 ft/sec
Depth shown in Figure is depth below top of pier
Depth of first reflector = 16.5 + 6.5 = 23 ft (reference is top of pier)
Depth of second reflector = 16.5 + 14 = 30.5 ft (reference is top of pier)

Figure 35 - Ultraseismic Data from a 3-lb Vertical Hammer Hit
and Radial Component Recording f-k Filtered to Enhance Upgoing Waves
Bridge No. 5188, Minnesota Highway 58, Zumbrota, Minnesota

The Ultraseismic method requires a strong geophysical, geological and/or NDE background for data analysis. Data collection is more straightforward, but still requires specialized equipment and training to be done successfully. The US method is a powerful tool though to increase the accuracy and reliability of stress wave reflections to indicate the depth of buried piers/abutments, footings/pilecaps and exposed bridge piles by tracking wave travel down and back up substructure elements. It allows one to discriminate from false, misleading reflection events from the attached bridge substructure/superstructure as opposed to desired reflections from the bottoms of piers, abutments, footings and piles. The US method is applicable to steel, concrete, and timber piles. One is limited to embedded pile length/diameter ratios of 20:1 to 30:1 in stiffer soils. Also, while the depth of a buried pilecap/footing can usually be determined with US tests on the exposed portion of the bridge substructure, the presence and lengths of any piles below the pilecap cannot be determined without excavating to expose the pile. The borehole-based PS method is a better approach for this and all other substructures in general, as discussed below.

3.5 Parallel Seismic (PS) Method for Unknown Substructure/Foundation Depths

The Parallel Seismic (PS) test consists of impacting exposed foundation substructure either vertically or horizontally with an impulse hammer to generate compression or flexural waves which travel down the foundation and are transmitted into the surrounding soil as shown in Fig. 36. The refracted compression (or shear) wave arrival is tracked at regular intervals by a hydrophone receiver suspended in a water-filled cased borehole (original PS procedure) or by a clamped three-component geophone receiver (new procedure-better for shear wave arrivals) in a cased or uncased borehole (if it stands open without caving). The depth of a foundation is typically indicated by a weaker and slower signal arrival below the tip of the foundation. Diffraction of wave energy from the foundation bottom was also found to be indicative of its depth in PS tests as well. The PS test was found to the most accurate and widely applicable NDE method for determination of unknown bridge foundation depths of all tested NDE methods in the NCHRP research referenced above for the US method (Olson and Aouad, 2000). The PS method is also discussed in ACI 228.2R-98 and was originated by Paquet of CEBTP in Paris, France.

The main objective of Parallel Seismic tests is to determine the depth of the unknown foundations. Based on the NCHRP 21-5 and 21-5(2) research results, several criteria were established for determining the foundation depths based on Parallel Seismic data as follows:

1. Breaks in the slope of the lines in a plot of depth versus recorded time
2. Drop in energy amplitude below the bottom of the foundation, and
3. Diffraction of wave energy at the bottom of the foundation.

Examination of Figure 37 shows an example PS result for the case where subsurface conditions are uniform with depth (this usually means saturated soil conditions where the compression wave velocity is that of water, i.e. about 1500 m/s or 4900 ft/s). This allows one to determine the velocity of the foundation element, and to clearly see the foundation bottom as the point where the wave velocity is slower and the amplitude is weaker in the soil below the bottom of this timber pile. The foundation bottom is then taken as the intersection of the foundation velocity line with the soil velocity line as shown in Fig. 36 by the arrow pointing to a depth of 22 ft. The vertical axis in Figure 36 represent depth below the top of the casing. The horizontal axis represents recorded time in milliseconds (1 mSec = 0.001 seconds).

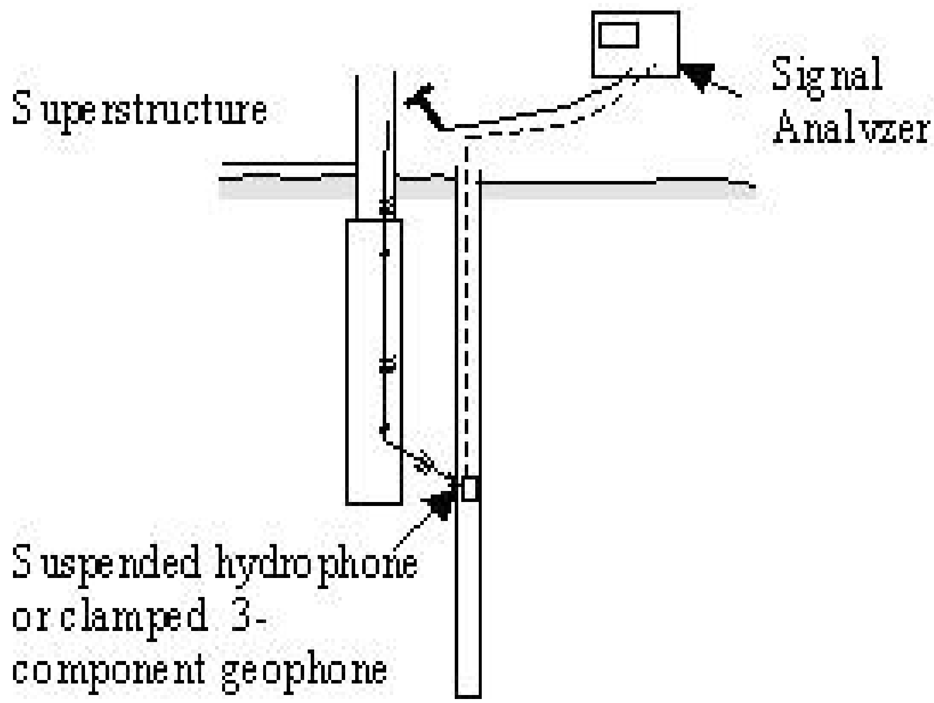


Figure 36 - Parallel Seismic Test Method

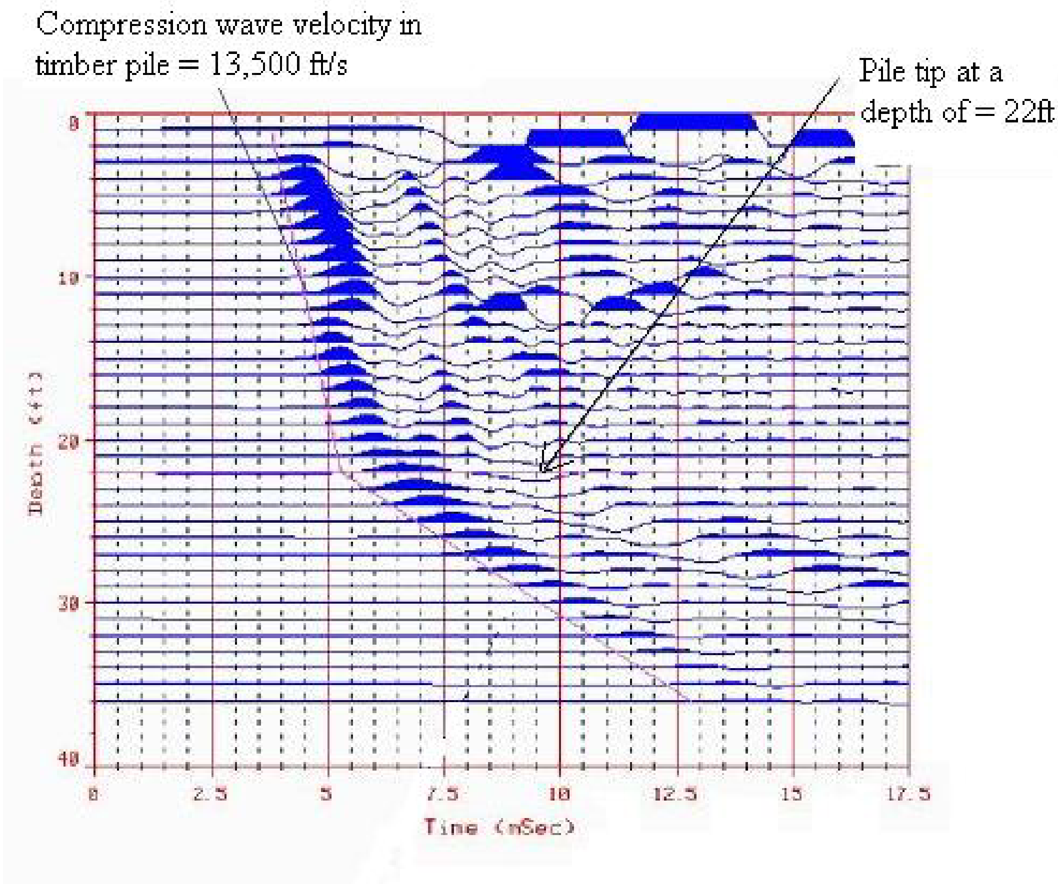


Figure 37 - PS Result showing pile tip at 22 ft

The Parallel Seismic (PS) method requires a strong geophysical, geological and/or NDE background for data analysis. Data collection is more straightforward, but still requires specialized equipment and training to be done successfully. The method has been used on concrete shafts below massive bridge piers to depths of over 100 ft for unknown bridge foundations with good success. It is applicable to steel, concrete, and timber piles and is the single best method for determining unknown foundation depths. PS data is more complicated for piles below pilecaps, steel H-piles, and partially saturated sites above the water table. However, pile depths have been accurately determined even for these complicated cases.

4.0 SUMMARY

The paper has covered many applications of stress wave methods for superstructure and substructure of bridges. The physics of stress waves interact with the mechanical properties of concrete, and other materials such as masonry, steel and wood to provide accurate data on unknown geometry, strength and internal conditions of bridges. Research and consulting experience have shown high correlation between destructive results (coring or drilling) with stress wave NDE investigations. Imaging and scanning technologies show considerable promise for improving quality assurance of new construction, troubleshooting of problems and condition assessment of aging bridge structures. In short, stress wave NDE methods will plan an important role in maintaining our nation's transportation infrastructure as evidenced by the ACI and ASTM publications on stress wave NDE.

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