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Non-Destructive Evaluation of Installed Soil Nails

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16. Abstract::
Recent forensic studies that investigated failure of several Texas Department of Transportation (TxDOT) soil nailed retaining walls revealed that incomplete grouting of soil nails was a very common problem. This finding has raised serious questions about the adequacy of QC/QA procedures currently used by TxDOT for verifying the integrity of installed soil nails. The research study described in this report attempted to address these concerns by: (a) developing of a non destructive test method to check the integrity of soil nails, (b) identifying the construction variables that have the most dominant influence on grout integrity and developing guidelines to control them. Sonic Echo method was selected, further refined and "customized" for non-destructive evaluation of soil nail grout columns. Optimum test parameter combination to be used with Sonic Echo testing was determined through several cycles of NDT testing that were conducted at an experimental soil wall that was specially built for this research. Among the various construction variables examined, grout rheology (or flowability) and tremie insertion length were found to be the most important parameters that influence integrity of soil nail grout columns. A new grout consistency requirement is proposed based on the findings from the research.

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CHAPTER I

INTRODUCTION

1.1 Overview

Soil nailed retaining walls are used extensively by the Texas Department of Transportation (TxDOT) in both temporary and permanent applications. Although most of these soil nailed walls have performed satisfactorily, occasional failures have occurred indicating that there are some unresolved issues related to the construction of this type of retaining wall. In a recent forensic study that investigated failure of TxDOT soil nailed retaining walls, it was found that incomplete grouting of soil nails was a common problem. Thus, the ability to assess the grout condition is vital for the safe performance of soil nailed retaining structures. The current TxDOT QC/QA procedure consists of installing two test soil nails and then load-testing them to verify the effectiveness of the contractor's installation procedure. The QC/QA of production nails is limited to monitoring the installation process through visual inspection. TxDOT experience has shown that this procedure is ineffective and does not answer all questions about the production nails that are not load-tested. This report presents the findings from a research study that was undertaken by Texas Tech University under TxDOT sponsorship to address concerns related to integrity of soil nails.

In this research project, the soil nail grout integrity problem was examined from two different viewpoints. The first involved the development of a non-destructive test technique that could be used to detect any defects that may be present in a grouted soil nail. Since non-destructive evaluation techniques have been used for many years to provide quality control of installed drilled shafts and driven concrete piles, it is prudent to investigate the possibility of developing a similar test method for the evaluation of soil nail quality. The second involved a study of construction variables (such as grout consistency, tremie length, borehole diameter, soil nail angle) to determine which of those variables have the most dominant influence on integrity of the grout column. Appropriate QC/QA procedures can then be implemented to have better control of these variables in the field.

1.2 Development of a Non-Destructive Test (NDT) Procedure

In this project, the development of an NDT procedure was accomplished as a collaborative effort between two research agencies; Texas Tech University and Olson Engineering, Inc., a company that specializes in NDT testing and equipment development. The objective in this first phase of the research was to develop a non-destructive test method that would meet the following requirements.

(a) The test method should be capable of detecting voids/cavities within the grout column with good reliability.

(b) It should be capable of verifying the grouted length of tendons with accuracy.

(c) The test method should be easy for TxDOT field personnel to use. Conducting the test procedure and interpreting the data should not require high levels of expertise or skill.

(d) It should be possible to complete the test procedure in a reasonable amount of time, e.g. 15 minutes for routine Q&A tests and 1-2 hours for forensic investigations.

(e) The test equipment should be appropriate for field use; rugged and portable.

It is also important to point out that, in this research, the primary interest was on an NDT technique that can detect *major defects* in soil nails in a consistent and reliable manner. The sensitivity of the new NDT test and its ability to detect *minor defects* was of secondary interest.

Figure 1.1 illustrates the general approach used in the development of the new NDT method. As a first step, candidate NDT methods that show the greatest promise for successful application in the evaluation of soil nails were identified. This was based on information available in literature as well as prior experience with the use of different NDT methods on structures that are similar to soil nails. Soil nails are rod-like structures that are embedded in soil. They are very similar to piles and drilled shafts. Therefore, NDT methods that have been successfully used for non-destructive evaluation of piles and drilled shafts are logical candidates for use in this application as well. These test methods include: Sonic Echo, Impulse Response, Impedance Logging, Impact Echo, Parallel Seismic and Cross-hole Sonic Logging. Once the candidate NDT methods had been identified they were used in a series of preliminary tests conducted on a specially-constructed experimental soil nail wall. The results from different candidate NDT methods were compared and those techniques that were found to be most effective were selected for further evaluation during subsequent rounds of testing. After each round of testing, necessary improvements were made in the testing procedure, data processing methods and hardware. This iterative procedure of testing and test method refinement was continued until the new test method provided reliable and consistent results. This report includes a detailed review of background literature, description of laboratory and field testing conducted and general recommendations concerning NDT methods that work best for soil nail testing. The design configuration for the proposed NDT method, hardware specifications and the user manual are presented in Appendices of this report.

1.3 Evaluation of Construction Variables

As mentioned previously, the research also included a study of construction variables (such as grout consistency, tremie length, borehole diameter, soil nail angle) to determine which of those variables have the most dominant influence on integrity of the grout column. Once the more critical variables have been identified, appropriate QC/QA procedures can be implemented to achieve better control of these construction parameters in the field.

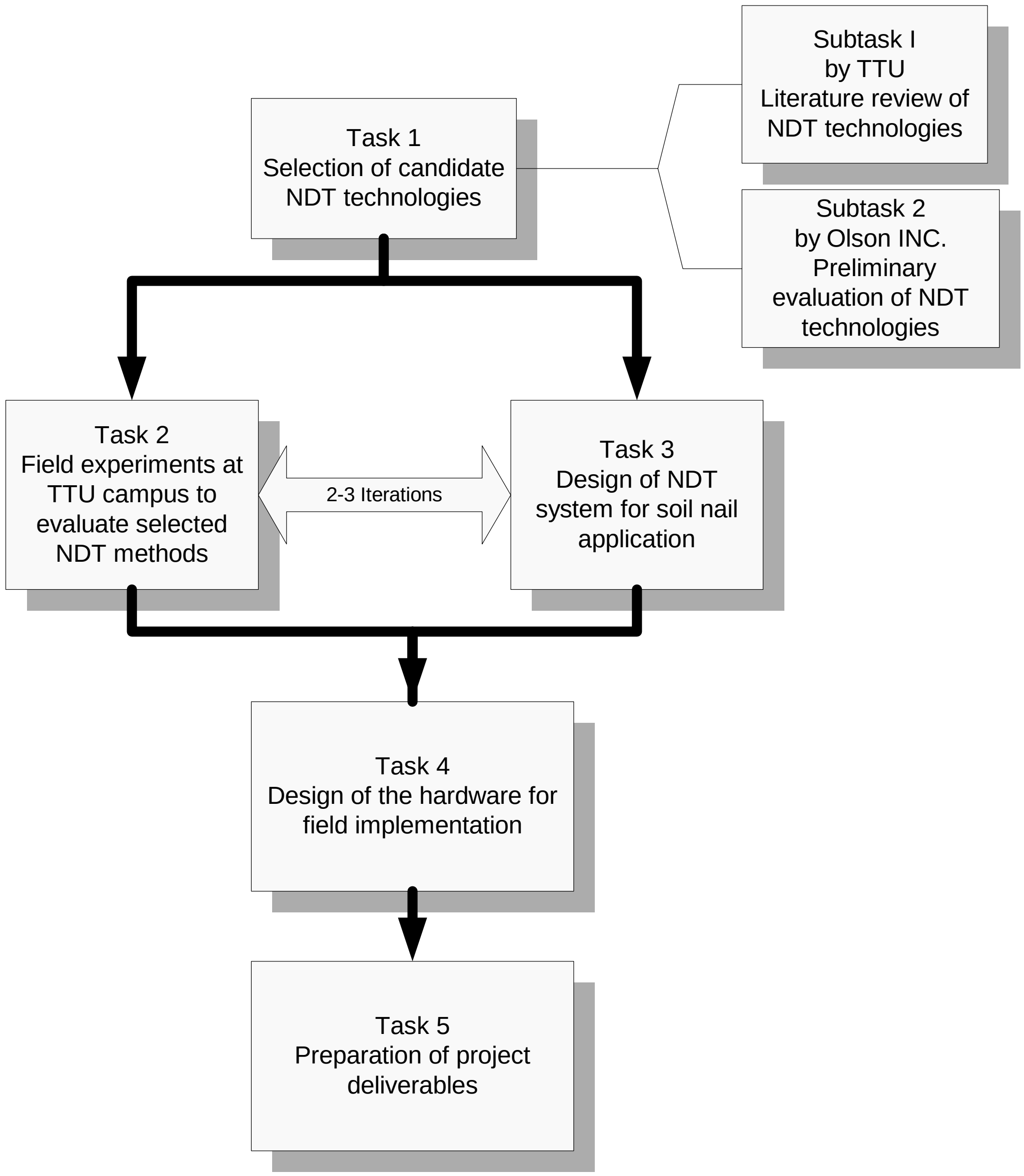


Figure 1.1 General Approach Used in the Development of the NDT Procedure for Soil Nail Testing

To examine the influence of construction variables on the quality of grout column, a series of experiments was conducted by pumping grout into 6-in diameter, 20-ft long PVC tubes. These PVC tubes were placed on a specially prepared ramp and covered with loose fill. Prior to placement, each tube was cut lengthwise into two halves. The inside of the tubes was roughened by spraying with a solvent based adhesive and then sprinkling PVC shavings and gravel/soil particles on to the adhesive. Then the top and bottom halves of the tubes were reassembled and clamped together with self-locking nylon ties. Then No. 6 steel rebars fitted with split PVC centralizers were inserted into these tubes and the annulus filled with grout. Different combinations of grout mix designs and other construction variables were used in different tubes. After the grout had hardened sufficiently, the tubes were opened to examine the condition of the grout columns.

1.4 Report Organization

As explained in preceding sections, this research project consisted of two separate phases: Phase I dealt with the development of an NDT method for soil nail evaluation and Phase II dealt with the evaluation of construction variables such as grout mixture design and placement methods to determine their influence on soil nail integrity. Chapter II provides a detailed review of background information on soil nailed walls and their construction. Chapters III through V document the research work completed in Phase I, selection of candidate NDT methods, field non-destructive testing and the results obtained. Chapter VI describes the experimental work conducted in connection with the evaluation of grout mixture design and placement procedures. The last chapter, Chapter VII summarizes the findings from both phases of research and the recommendations made based on the research findings.

CHAPTER II

REVIEW OF BACKGROUND INFORMATION
SOIL NAILED WALLS

2.1 Overview

Soil nailing, as a method of stabilizing excavations, originated in France in the early 1970s [1]. In this method of ground stabilization, soil is effectively reinforced by installing closely spaced grouted steel bars, called "nails," into a slope or excavation as construction proceeds from top down. The stabilization of the soil is achieved by two mechanisms: (a) increase in the normal force and hence the soil shear resistance along potential slip surfaces in frictional soils and; (b) reduction of the driving force along potential slip surfaces in both frictional and cohesive soils.

A typical soil nail wall consists of three basic components as shown in Figure 2.1: soil nails, drainage elements, and structural wall facings [2]. Soil nails are structural elements that provide load-transfer to the ground through shear resistance mobilized at the interface between the cement grout and soil. Generally, soil nails are not pre-tensioned and therefore are considered "passive" inclusions. Nevertheless, as excavation of soil is continued, the ground deforms in response to loss of lateral support and nail bars are forced into tension. As a result, the nails increase the overall shear strength of the in-situ soil and restrain its displacements during and after excavation. A typical soil nail consists of a deformed steel reinforcing bar (generally Grade 60), also called a tendon, which is inserted into a predrilled, straight-shafted drillhole usually ranging from 4 to 12 inches in diameter. The steel tendons extend the entire length of the holes. They are provided with plastic centralizers to ensure they stay close to the center of the holes and to provide a minimum specified grout cover. After the nail tendon is inserted, the drillhole is completely filled with structural grout pumped under low-pressure via a tremie pipe. The grout "bonds" the nail tendon to the surrounding ground. This procedure is often referred to as an "open hole" installation.

Stabilization of excavation with soil nailing offers many advantages over other methods. These include:

a) Ability to follow outline of existing structures easily
b) Suitability for small construction equipment
c) Suitability for special applications and remedial work
d) Ability to mobilize to a site quickly
e) Elimination of the need for soldier piles (required with tieback walls)
f) Flexibility to allow for modifications during construction
g) Compatibility with the usual constraints of operation in urban environments
h) Adaptability of structure elements and installation methods

The following soil conditions are considered favorable for soil nailing: naturally cohesive materials, naturally cemented or dense sands and gravels with some real cohesion or

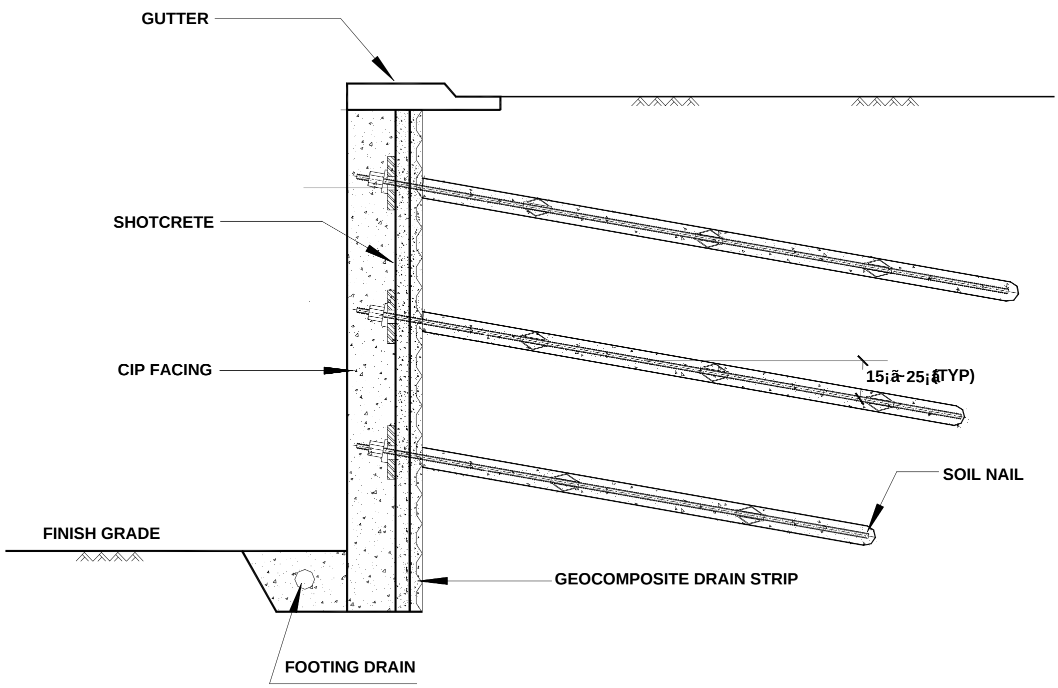


Figure 2.1 Basic Components of a Soil Nailed Wall

apparent cohesion, and weathered rock. However, in some situations, the soil nailing construction method is not a good choice. These include sites where groundwater is a problem, sites with soil ravelling in cohesionless sands and gravels, sites with heavy concentrations of utilities, vaults or other underground obstructions behind the wall, and sites with expansive or highly frost-susceptible soils.

2.2 Construction of Soil Nailed Walls

The construction of a soil nail retaining wall typically involves six steps, as shown in Figure 2.2.

Step 1. Excavate a small height cut.
Step 2. Drill hole for nail.
Step 3. Install and grout soil nail tendon.
Step 4. Place geocomposite drain strips, initial shotcrete layer, and install bearing plates and nuts.
Step 5. Repeat process to final grade, and
Step 6. Place final facing on permanent walls.

Note that Steps 3 and 4, order of nail and shotcrete installation, may be reversed.

The epoxy coated steel tendons (typically ¾-1 inch diameter bars) used in TxDOT construction are shown in Figure 2.3. The most commonly used holes are 6 inches in diameter. The length of the holes varies from approximately 8 to 30ft, but more commonly between 15 and 25ft. The holes may be drilled horizontally or with a 10°-15° inclination (See Figure 2.4). There are two common types of centralizers available in the market: wheel style and split PVC style. Wheel style centralizers are not allowed to be used in TxDOT soil nail wall construction projects. Figure 2.5 shows both wheel style and split PVC style centralizers.

For drainage elements, face drainage is the most commonly used drainage element. It usually consists of prefabricated vertical geocomposite drainage strips installed from the top to the bottom as the excavation proceeds downward (See Figure 2.6). A surface water collector ditch is usually placed behind the top of the wall to prevent surface runoff.

Structural wall facings are required for face confinement, protection of the retained soil from weathering and erosion, and resisting lateral earth pressures. Three kinds of wall facings are commonly used in soil nail wall construction. They are temporary shotcrete facing, permanent wall facing, and architectural fascias and face treatments. Temporary shotcrete facing typically consists of 3 to 4 inches of shotcrete reinforced with a single layer of welded wire mesh. The temporary shotcrete facing is placed concurrently with each excavation lift. Permanent walls may consist of full-thickness shotcrete, CIP concrete over temporary shotcrete, or precast concrete panels over shotcrete.

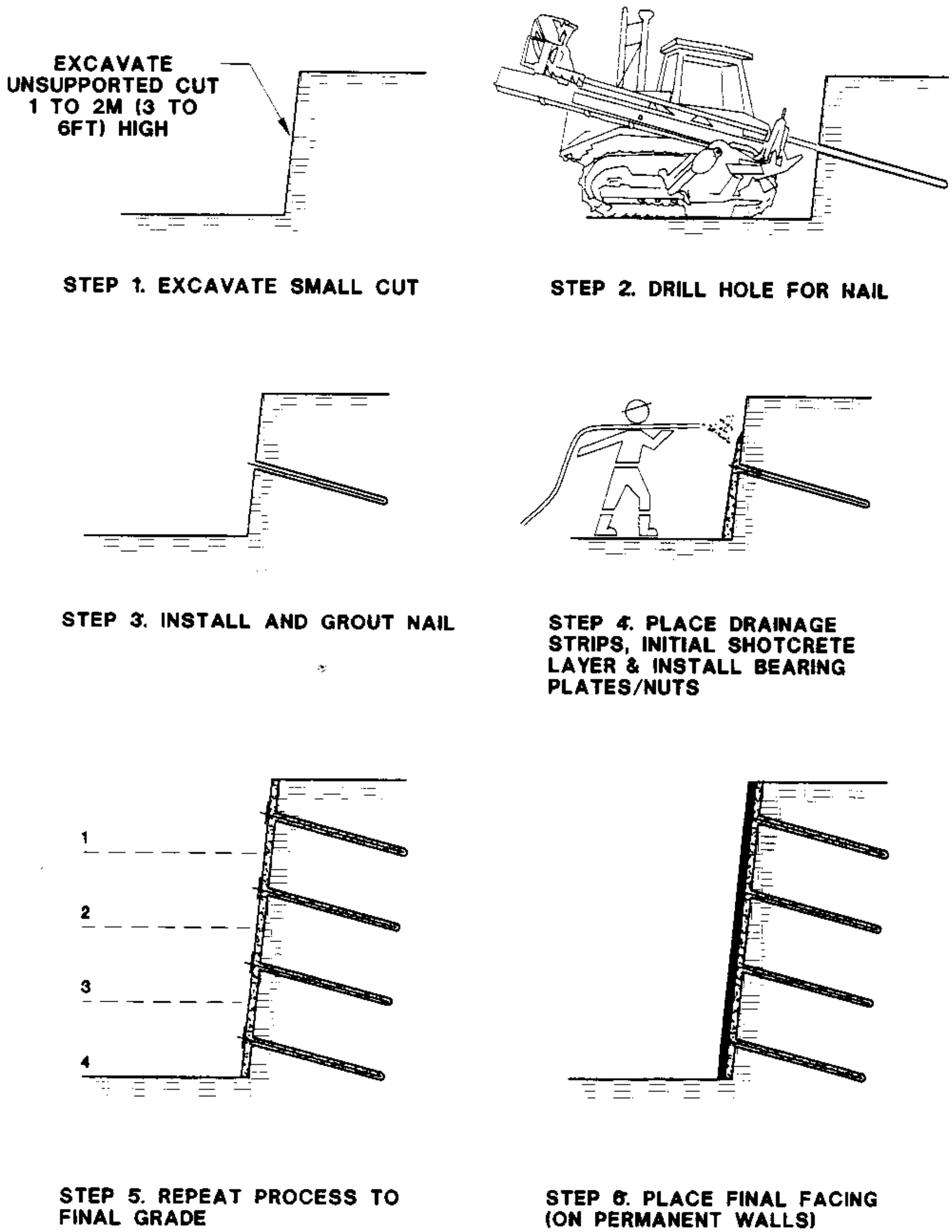


Figure 2.2 Typical Construction Sequence Used in Soil Nailing
Source: FHWA-SA-93-068: *Soil Nailing Field Inspectors Manual* [2]

Figure 2.3 Epoxy-Coated Steel Tendons Used in TxDOT Installations

Figure 2.4 Drilling Hole for Soil Nail Installation

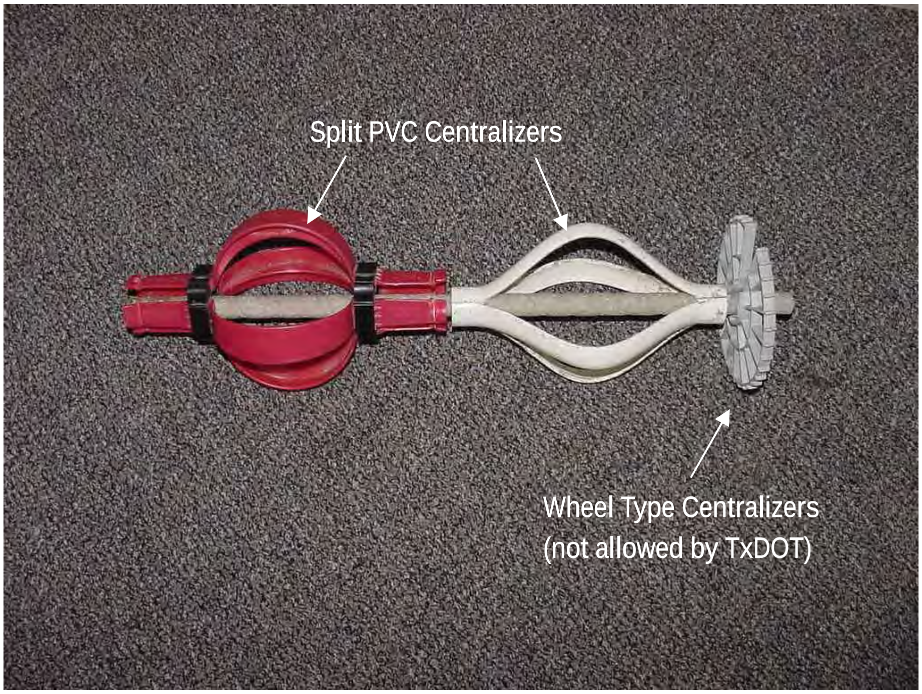


Figure 2.5 Split PVC and Wheel Type Centralizers Used in Soil Nail Installations

Figure 2.6 Use of Prefabricated Vertical Geocomposite Drainage Strips

Permanent shotcrete walls are constructed with either the full-thickness shotcrete placed concurrently with each excavation lift, or with a second full-height shotcrete layer placed over the initial shotcrete layer following the excavation to full depth. This type of wall generally has a total thickness of 6 to 12 inches, and is reinforced with reinforcing bars or welded wire mesh.

2.3 Construction Quality Control Procedures for Soil Nailed Structures

Construction quality control of soil nailed structures is commonly implemented in two separate stages; (a) monitoring of the construction procedure and (b) load testing of selected soil nails. Only a small percentage of the nails will be load-tested and therefore it is important to make sure that the construction methods are similar to those used for test soil nails. Recommendations for procedures to be followed during construction monitoring are found in FHWA Report SA-93-068: *Soil Nailing Field Inspectors Manual* [2].

A commonly used method to inspect the soil nail hole after drilling utilizes a mirror or high intensity light to inspect the hole for cleanliness. This needs to be done before tendon insertion. The mirror only works on bright sunny days; however, a high intensity light (500,000 to 1,000,000 candlepower) works well anytime. Excessive slough should be removed, either by redrilling the hole, or by cleaning with a tool, if possible.

FHWA Report SA-93-068 recommends that each tendon be checked by an inspector to ensure that the length, diameter, steel grade, centralizers, and corrosion protection are in accordance with the plans and specifications. The tendon must be inserted into the hole to the minimum specified length. Centralizers are also important. They should be stiff, and large enough to provide space for the minimum specified grout cover. Centralizers should be spaced closely enough to each other to keep the tendon from sagging and touching the bottom of the hole, but should not impede the free flow of tremied grout up the hole. Openings between the centralizer support arms should not be obstructed by materials used to secure the centralizer to the tendon. In the event the nail tendon has not been inserted completely into the drillhole, the tendon should never be allowed to be driven or pushed beyond the drilled length, or cut off.

Two other items that deserve special attention during grouting inspection are the length and the pullout rate of the tremie pipe. The inspection manual indicates that the depth the tremie pipe reaches and its withdrawal rate have a great impact on the integrity of grout. The manual requires the tremie pipe be inserted to the bottom of the drillhole and the grout should flow continuously as the tremie pipe is withdrawn. That is, the withdrawal rate should be controlled to ensure that the end of the tremie pipe is always below the grout surface. Another way a contractor may ensure full grouting of the drillhole is accomplished is by calculating the actual volume of grout and comparing it with the volume of the drillhole.

While the quality control methods described above may be effective, there are some difficulties associated with their implementation in the field. First of all, they require a field inspector or inspectors to be present at the construction site throughout the entire construction process. TxDOT does not have adequate workforce that would allow an inspector to be assigned full time to each construction site. Secondly, some of the guidelines are very qualitative and therefore, their implementation in the field is subjective.

The second quality control procedure that is commonly used to ensure satisfactory performance of soil nailed walls involves load testing of selected soil nails. In this method, a few selected test soil nails are subjected to load tests in the field to verify that the nails are capable of carrying their design loads without excessive movements and with an adequate safety factor. Testing is also used to verify the adequacy of a contractor's drilling, installation, and grouting procedures. A typical test setup used in load testing conducted by TxDOT is shown in Figure 2.7.

There are four types of loading tests that can be performed on installed soil nails. They are: (a) ultimate tests, (b) verification tests, (c) proof tests, and (d) creep tests. Ultimate tests and verification tests are typically performed on "sacrificial" test nails. Ultimate tests are performed by loading the soil nail until pullout failure takes place along the grout-soil interface (pullout failure occurs when the soil nail can no longer maintain constant test load without excessive movement). Verification tests are conducted to verify that installation methods will provide a soil nail capable of achieving the specified design adhesion capacity with a specified safety factor. Proof tests are typically performed on a specified number (typically up to 5 percent) of the total number of constructed soil nails. The proof test is a single cycle test in which the load is applied in increments until a maximum test load, typically 125 to 150 percent of the design adhesion capacity, is reached. Creep tests are typically performed as part of the ultimate verification, or as proof tests. Creep testing is conducted at a specified, constant test load, with movement recorded at specified time intervals. Detailed interpretation of each test can be found in FHWA Report SA-93-068: *Soil Nailing Field Inspectors Manual* [2]. A general guideline that is applicable for all four types of tests is that the soil nail tendon should not be stressed to more than 80 percent of its minimum ultimate tensile strength for 150 grade steel, or more than 90 percent of the minimum yield strength for 60 grade steel.

While these tests do provide useful information about construction quality, there are only a limited number of nails tested in each project. It is not uncommon to find a large amount of variance within a single project. Therefore, the limited number of samples is not enough to assure evaluation of the conditions of all the nails. Moreover, performing tests on "sacrificial" nails always entails high cost. Accordingly, a new testing protocol, especially one based on non-destructive testing will greatly enhance quality control of this type of wall.

Figure 2.7 Test Setup Used for Soil Nail Load Testing

2.4 Soil Nailed Wall Failures

Soil nailed walls are used quite extensively both in short- and long-term earth retaining applications. The vast majority of these soil nailed walls have performed satisfactorily. However, over the past years several failures have occurred in this type of earth retaining structure. Investigation of these failures reveals that these failures are largely due to poor construction rather than deficiencies in the design. Table 2.1 below summarizes construction problems commonly encountered in the field and the stability problems they have caused.

Table 2.1 Stability Issues Related to Construction

Undesirable Construction Practice	**Resulting Stability Problem/s**
Unnailed slopes exposed for a long period	Face sloughing; Face instability
Water drainage	Washout zones; Face instability
Incorrect drillhole diameter	Nail pullout; Wall instability
Improper grouting of holes	Nail pullout; Wall instability

Among these, TxDOT has identified the following construction problems as the most important:

a) Failure to install test nails properly and perform tests correctly
b) Performing excavation too far in advance of nailing and shotcreting
c) Failure to completely grout soil nails

The third problem occurs when grout mixes are too stiff, grout pipe is not used or is incorrectly used, and too few centralizers are used.

At this time, TxDOT relies upon construction drawings, soil nail anchor specifications, and inspections to assure construction quality of soil nailing structures. TxDOT Bridge Division also assists in nail installation, and in nail testing. However, the Department desires to make further improvements in these quality control measures. Safe performance of soil nailing structures can be greatly enhanced by implementing a new, systematic QC/QA test protocol.

Figures 2.8 through 2.23 graphically illustrate the common problems found in the construction of soil nailed walls. These examples come from actual construction projects, the majority of which is in Texas, and field studies conducted as part of this research.

Figure 2.8 Face Sloughing Caused by Unnailed Cut Left Exposed for a Long Period of Time

Figure 2.9 Face Instability Caused by Unnailed Cuts Left Exposed for a Long Period of Time

Figure 2.10 Long-Term Face Instability Caused by Drainage

Figure 2.11 Washout Zone Caused by Poor Installation of Drain Elements

Figure 2.12 Failure of Wall Caused by Incomplete Grouting

Figure 2.13 Failure of Wall Caused by Incomplete Grouting and Drainage

Figure 2.14 Pullout Caused by Incomplete Grouting

Figure 2.15 Pullout Caused by Drillholes Half Filled with Grout

Figure 2.16 Low Pullout Strength Due to Incomplete Grouting

Figure 2.17 Off-Center Tendon Due to the Use of Too Few Centralizers

Figure 2.18 Stiff Grout Causes Incomplete Grouting Problem

Figure 2.19 Excessively Fluid Mixes Cause Incomplete Grouting of Nail Head (Bird's Mouth Defects)

Figure 2.20 Bird Mouth Defect Caused by Poor Grouting

Figure 2.21 Insufficient Tendon Length Caused by Nail Hole Caving

Figure 2.22 Void at the End Caused by Incomplete Grouting

Figure 2.23 Inspection of Soil Nail Installation

CHAPTER III

EVALUATION OF CANDIDATE NDT TECHNOLOGIES

3.1 <u>Overview</u>

Non-Destructive Testing (NDT) techniques have been used for many years to provide quality control of drilled shaft foundations and driven concrete piles. Although soil nails have characteristics that are similar to drilled shafts and piles, there has been little or no previous work related to the use of NDT methods for soil nail quality control. Nearly all NDT methods for testing deep foundations have been developed by applying the principle of stress wave propagation or electromagnetic wave propagation through materials such as concrete, steel, and wood. The stress wave methods include Sonic Echo, Impulse Response, Impedance Logging, Impact Echo, Parallel Seismic, and Cross-hole Sonic Logging. These methods can be further classified into a number of categories as shown in Table 3.1. The electromagnetic wave propagation methods in use at the present time are: Time Domain Reflectometry, Magnetic Flux Leakage, Electromagnetic Impedance Spectrometry, and Ground Penetrating Radar. A description of each method is presented in this chapter. Conclusions are then made regarding the suitability of these NDT methods as candidates for soil nail testing.

3.2 <u>Ultrasonic Pulse Velocity, UPV</u>

Ultrasonic Pulse Velocity (UPV) is one of the oldest NDT methods used to test concrete. It is based on measuring the travel time over a known path length of a pulse of ultrasonic compressional waves. This method is also known as *ultrasonic through transmission*. Generally, UPV tests are performed to assess the conditions of structural members with two-sided access such as slabs, beams, and columns. Voids, honeycomb, cracks, delaminations, and other damage in concrete, wood, stone, and masonry materials can be determined with this method. UPV tests are also performed to predict strength of early age concrete.

3.2.1 Ultrasonic Pulse Velocity; Principle of Operation

The speed of propagation of stress waves depends on the density and the elastic constants of the solid. In a concrete member, variations in density can arise from non-uniform consolidation, and variations in elastic properties can occur due to variations in materials, mix proportions, or curing. Thus, by determining the wave speed at different locations in a structure, it is possible to make inferences about the uniformity of the concrete. The compressional wave speed is determined by measuring the travel time of the stress pulse over a known distance. A comparison of the wave speeds at different test points can indicate anomalies within the member. It may also be possible to use signal attenuation as an indicator of relative quality of concrete, but this requires special care to ensure consistent coupling of the transducers at all test points.

Table 3.1 Classification of NDT Methods Based on Stress Wave Propagation

Classification Standards	**Methods**	**Characteristics**
Wave Frequency Content	Ultrasonic Pulse	> 20 KHz
	Sonic Pulse	<20 KHz
Application Field	Ultrasonic Pulse Velocity	Structure
	Impact Echo	Structure
	Spectral Analysis of Surface Wave	Structure
	Sonic Echo	(Geotechnical) Deep Foundation
	Impulse Response	(Geotechnical) Deep Foundation
	Impedance Logging	(Geotechnical) Deep Foundation
	Parallel Seismic	(Geotechnical) Deep Foundation
	Crosshole Seismic	(Geotechnical) Deep Foundation
	Ultraseismic	(Geotechnical) Deep Foundation
	Crosshole Logging	(Geotechnical) Deep Foundation
Sensor Location	Sonic Logging	At two different ends
	Sonic Echo	At one end
Analysis Technique	Sonic Echo	Response analyzed in time domain
	Impulse Response	Force and Response analyzed in frequency domain
	Impact Echo	Response analyzed in frequency domain
	Impedance Logging	Inverse Technique

3.2.2 Ultrasonic Pulse Velocity; Test Equipment

The main components of test devices for measuring ultrasonic pulse velocity are shown in Figure 3.1. A transmitting transducer is positioned on one face of the member and a receiving transducer is positioned on the opposite face. In some test configurations, the two transducers are used on the same surface. This approach has been suggested for measuring the depth of a fire-damaged surface layer having a lower wave speed than the underlying sound concrete and for measuring the depth of concrete damaged by freezing.

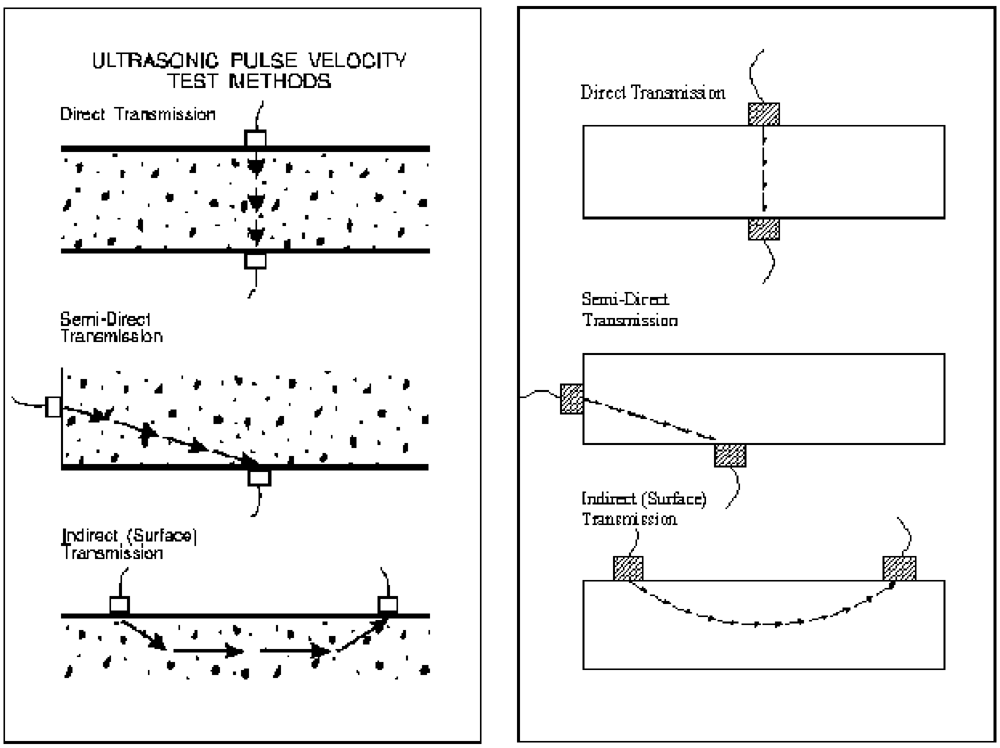


Figure 3.1 Ultrasonic Pulse Velocity Test Configuration

Source: www.olsonengineering.com (Retrieved on 04/14/2005)

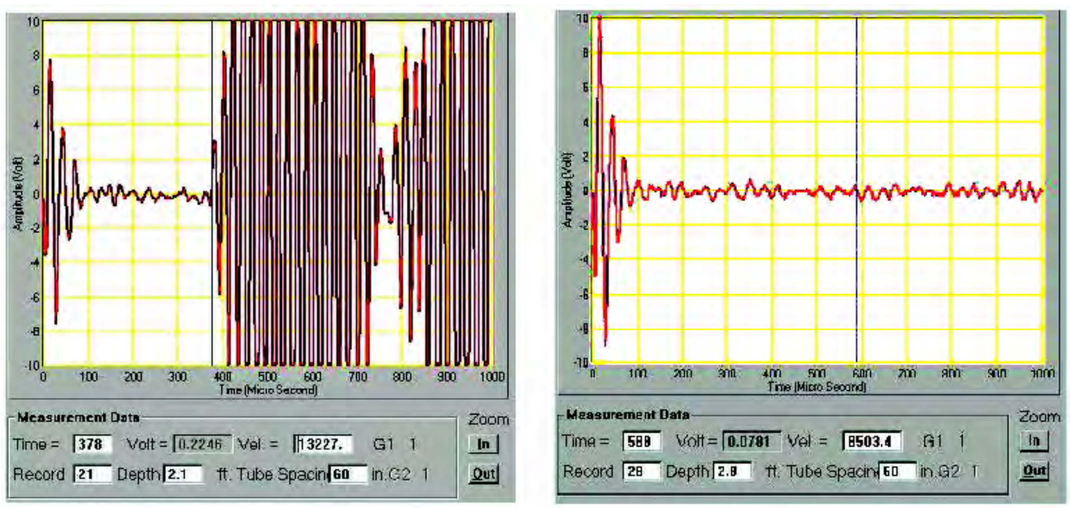
3.2.3 Ultrasonic Pulse Velocity; Testing Procedure

In a UPV test, a piezoceramic source is electrically pulsed to generate ultrasonic waves which travel in the structural element, and are sensed by the matching receiver on the opposite side or the same side of the test member. Knowing the travel distance and travel time, the ultrasonic compression wave velocity is calculated.

3.2.4 Ultrasonic Pulse Velocity; Interpretation of Data

The receiver output is recorded by a digital oscilloscope card in a PC. Three parameters are used in the interpretation of data: 1) arrival of compression waves, 2) signal strength, and 3) distortion of the transmitted signal. In defect areas, the compression wave velocity is slower than in sound areas. In some defect areas, such as honeycombs, the compression wave velocity may be almost the same as in sound areas, but distortion of the signal (filtering of high frequencies) may be used as an indication of a honeycomb defect.

Figure 3.2 shows example results from tests on a square concrete column. The left hand figure (a) shows a strong signal where testing was performed through sound concrete. The right hand figure (b) shows a very weak signal indicative of poorly consolidated concrete or wave travel through void conditions.



(a) (b)

Figure 3.2 UPV Test Results on (a) a Sound and on (b) a Poor Quality Concrete Beam

3.2.5 Ultrasonic Pulse Velocity; Limitations

UPV usually requires access to two surfaces, preferably two parallel surfaces such as the top and bottom surfaces of a slab or the inside and outside surfaces of a wall. In many testing conditions, it is not possible to access two surfaces for testing. As a result, the applications of this method are limited primarily to laboratory rather than field testing.

3.3 <u>Impact Echo, IE</u>

Impact-echo is a method for nondestructive testing of concrete and masonry structures, based on the use of impact-generated stress (sound) waves that propagate through concrete and masonry and are reflected by internal flaws and external surfaces. It can be used to determine the location and extent of flaws such as cracks, delaminations, voids, honeycombing, and debonding in plain, reinforced, and post-tensioned concrete structures, including plates (slabs, pavements, walls, decks), layered plates (including concrete with asphalt overlays), columns and beams (round, square, rectangular and many I and T cross-sections), and hollow cylinders (pipes, tunnels, mine shaft liners, tanks).

3.3.1 Impact Echo; Principle of Operation

Impact-echo is based on the use of transient stress waves generated by elastic impact. A diagram of the method is shown in Figure 3.3. A transient stress pulse is introduced into a test object by mechanical impact on the surface. As shown in Figure 3.4, the P- and S-waves produced by the stress pulse propagate into the object along hemispherical wavefronts. In addition, a surface wave travels along the surface away from the impact point. The waves are reflected by internal interfaces or external boundaries. Surface displacements caused by the arrival of reflected waves at the impact surface are recorded by a transducer, which produces an analog signal of voltage vs. time, called a waveform.

Interpretation of waveforms in the time domain has been successful in seismic-echo applications involving long slender structural members, such as piles and drilled shafts. In such cases, there is sufficient time between the generation of the stress pulse and the reception of the wave reflected from the bottom surface, or from an inclusion or other flaw, so that the arrival time of the reflected wave is generally easy to determine even if long-duration impacts produced by hammers are used.

For relatively thin structural members such as slabs and walls, time-domain analysis is feasible if short-duration impacts are used, but it is time-consuming and can be difficult depending on the geometry of the structure [3]. The preferred approach, which is much quicker and simpler, is frequency analysis of displacement waveforms. In frequency analysis, the time-domain signal is transformed into the frequency domain using the fast Fourier transform technique. The result is an amplitude spectrum that indicates the amplitude of the various frequency components in the waveforms. Peaks in the spectrum identify the dominant frequencies in the waveform, which are used to calculate thickness and/or the depths of flaws.

3.3.2 Impact Echo; Test Equipment

A typical impact-echo field test system has five main components, as shown in Figure 3.5:

a) A hand-held transducer unit that produces a voltage signal in response to surface displacements caused by reflected stress wave
b) A set of small hardened steel spheres called impactors or calibrated hammers, for producing impact-generated stress waves
c) A high-speed, analog-to-digital data acquisition system that receives and digitizes the signal from the transducer and transfers it to the computer memory
d) A notebook computer that receives, processes the digitized signal from the data acquisition system, and displays the results in numerical and graphical form
e) A software program that monitors and controls each test, and processes the data to produce output displays that provide information about the structure being tested

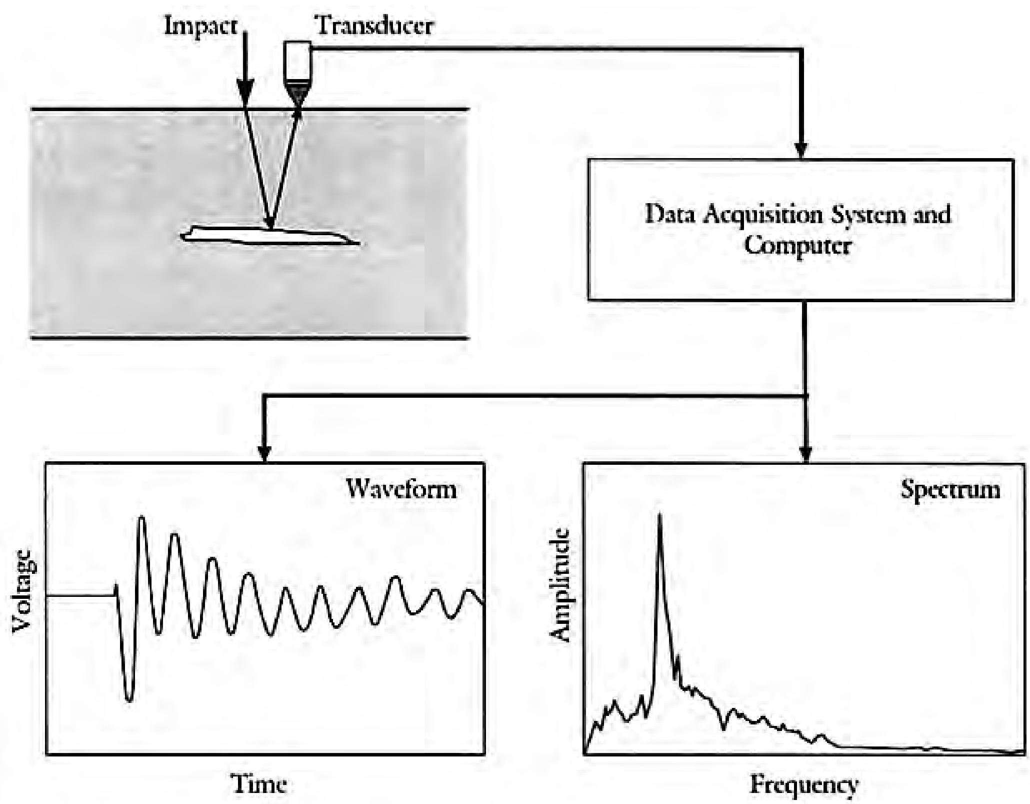


Figure 3.3 Schematic Diagram of the Impact-echo Method
Source: Sansalone. 1997[4]

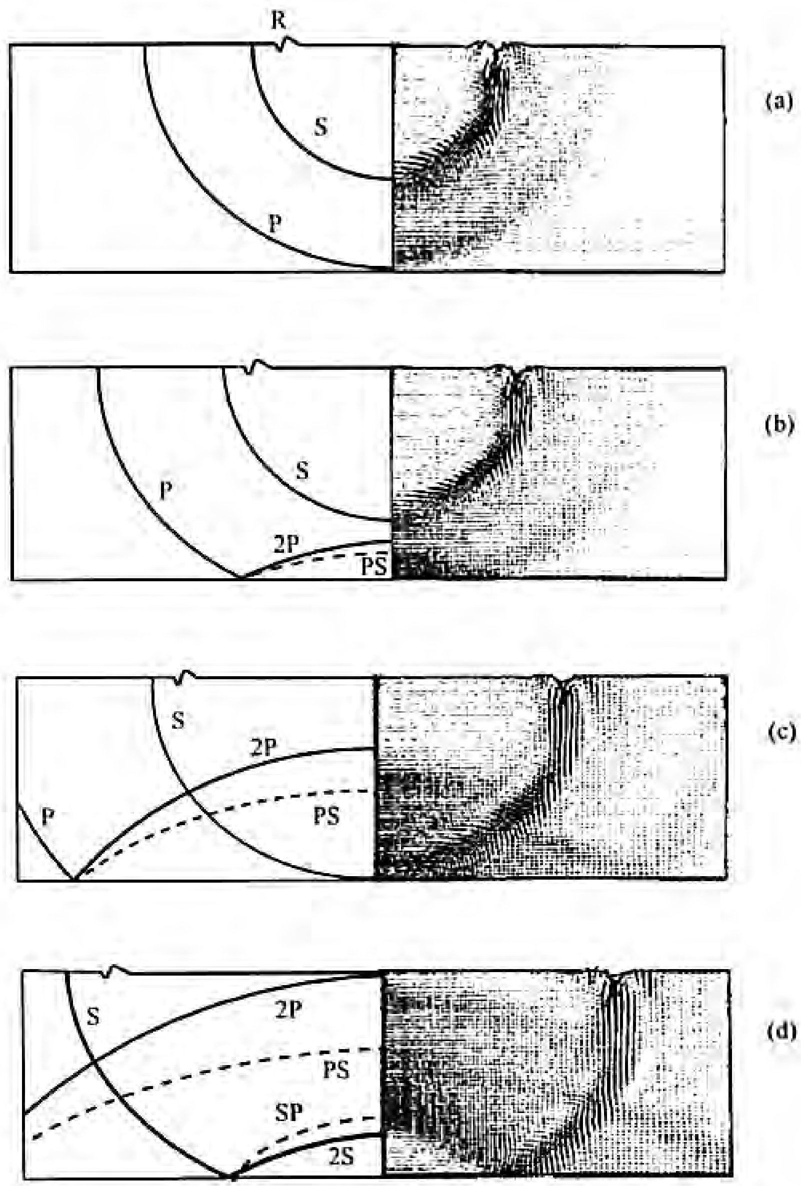


Figure 3.4 Reflections of Spherical Wavefront in a Plate (Theoretical Diagram vs. Finite Element Simulation)
Source: Sansalone. 1997[4]

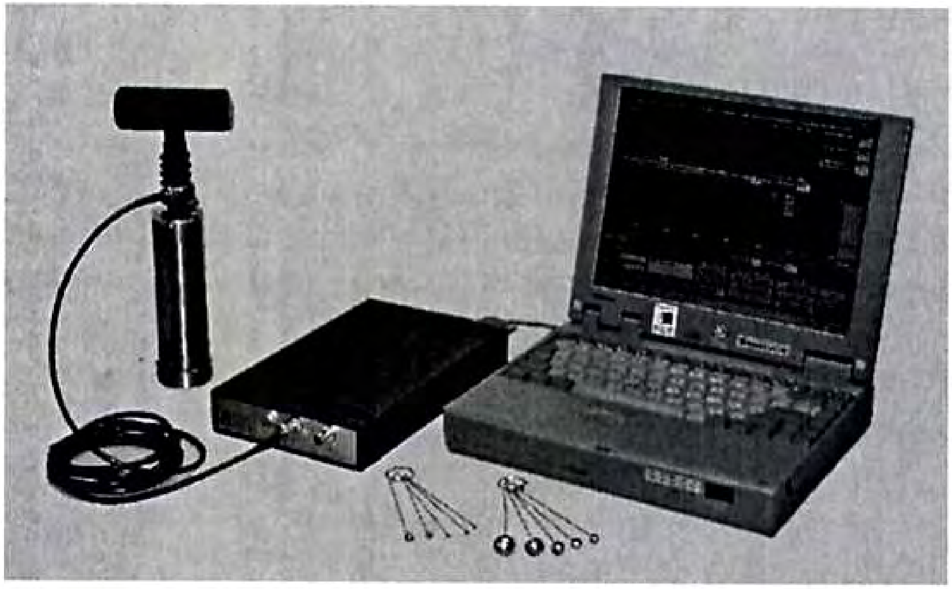


Figure 3.5 A Typical Impact-echo Test System

The selection of the impact source is a critical aspect of a successful impact-echo test system. The impact duration determines the frequency content of the stress pulse generated by the impact, and determines the minimum flaw depth that can be determined. In evaluation of piles, hammers are used that produce energetic impacts with long contact times (greater than 1 ms) suitable for testing long, slender structural members. Impact sources with shorter duration impacts (20-80 μs), such as spring-loaded spherically-tipped impactors, have been used for detecting flaws within structural members less than 1 m thick.

In regard to receivers in the evaluation of piles, geophones (velocity transducers) or accelerometers have been used as the receiving transducer. For impact-echo testing of slabs, walls, beams, and columns, a broad-band, conically-tipped, pizzoelestric transducer that responds to surface displacement has been used as the receiver [3]. Small accelerometers have also been used as receivers. In this case, additional signal processing is carried out in the frequency domain to obtain the appropriate amplitude spectrum. Such accelerometers must have resonant frequencies well above the anticipated thickness frequencies to be measured.

3.3.3 Impact Echo; Testing Procedure

A short duration mechanical impact is produced by tapping a small steel sphere or hammer against a concrete or masonry surface. As a result, low-frequency (70 kHz or less) stress waves propagate into the structure and are reflected by flaws and/or external surfaces. The surface response of the structure is monitored by a transducer.

3.3.4 Impact Echo; Interpretation of Data

The waveform is the raw response of an impact-echo test. It contains all the information provided by the test, but in a form that often makes it difficult to extract key features of the response, such as the transient resonant frequencies associated with multiple P-wave reflections. Nevertheless, it is the features of the waveform that allow one to determine when a recorded test is valid. A common mistake by new users of impact-echo is to ignore the waveform altogether, and to base an interpretation solely on the spectrum. The waveform always contains useful information, and it should be examined as part of the interpretation of each test. The first step in the interpretation of data is to distinguish valid waveforms from invalid waveforms. The following discussion is also very useful for interpretation of data in other non-destructive methods such as sonic echo and spectral analysis of surface wave.

A valid waveform as shown in Figure 3.6, indicating a successful test, will consist of a horizontal or zero voltage segment at the beginning, followed by a distinct R-wave (except in the case of surface-opening cracks) and a periodic displacement pattern caused by multiple reflections of stress waves.

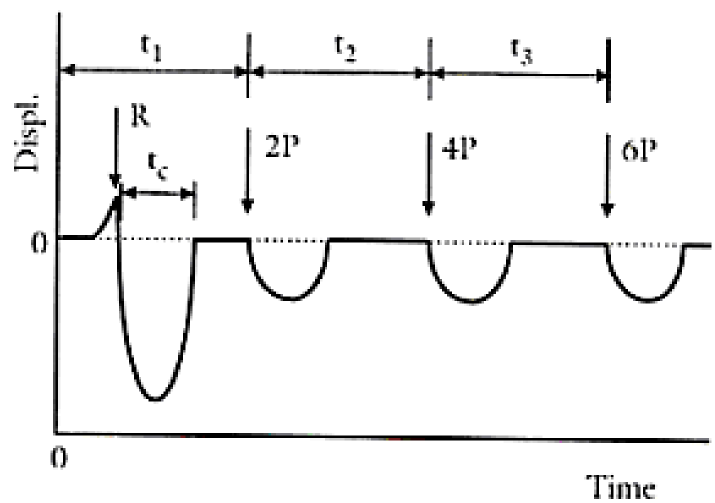


Figure 3.6 A Typical Valid Waveform

Invalid waveforms can result from rough surfaces on the concrete, dirt, or other foreign material on the surface, premature triggering, loss of contact between the transducer and the surface, accidental movement of the transducer during the test, or a host of other causes. After checking the validity of the waveform, the interpretation of waveform in impact-echo is generally done in the frequency domain. The highest amplitude frequency peak is the main indicator of a reflector depth (thickness echo). The presence of additional echo peaks can also be significant, indicating the presence of possible defects or other interfaces in the concrete. The approximate relationship between the distance D to the reflecting interface, the P-wave speed C_p and the thickness frequency, f is as follows:

$$D = \frac{C_p}{2f} \quad (3.1)$$

3.3.5 Impact Echo; Limitations

Frequency analysis of signals obtained from impact-echo tests on bar-like structural elements, such as reinforced concrete beams and columns, bridge piers, and similar members, is more complicated than the case of slab-like structural members. The presence of the side boundaries gives rise to transverse modes of vibration of the cross section. Thus, prior to attempting to interpret test results, the characteristic frequencies associated with the transverse modes of vibration of a solid structural member have to be determined. Insufficient penetration depth is another limitation associated with this method. So far, current instrumentation is limited to testing members less than 2 m thick.

3.4 Spectral Analysis of Surface Waves, SASW

The Spectral Analysis of Surface Waves (SASW) has been used successfully to determine the stiffness profiles of soil sites, asphalt and concrete pavement systems, and concrete structural members. The method has been extended to the measurement of changes in the elastic properties of concrete slabs during curing, the detection of voids, and assessment of damage.

3.4.1 Spectral Analysis of Surface Waves; Principle of Operation

The SASW method uses the dispersive characteristics of surface waves (R-wave) to determine the variation of the shear wave velocity (stiffness) of layered systems with depth. The general test configuration is illustrated in Figure 3.7.

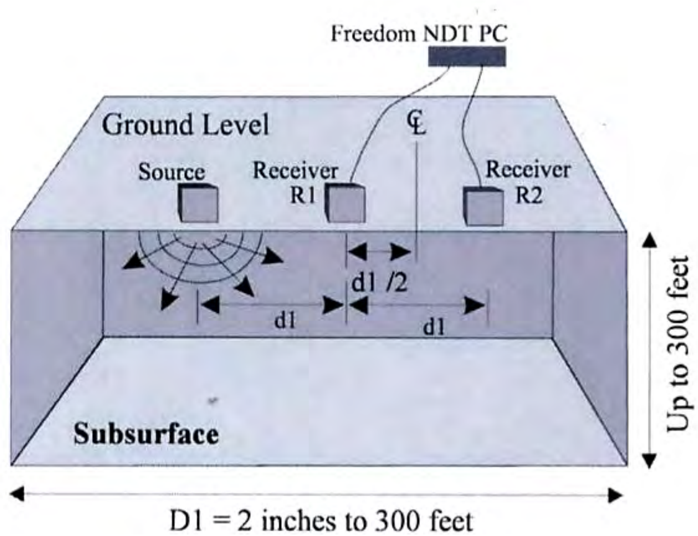


Figure 3.7 Field Setup for SASW Tests

Just as the stress pulse from impact contains a range of frequency components, the R-wave also contains a range of components of different frequencies or wavelengths. (The product of frequency and wavelength equals wave speed). This range depends on the contact time of the impact; a shorter contact time results in a broader range. The longer wavelength (lower-frequency) components penetrate more deeply, and this is the key to using the R-wave to gain information about the properties of the underlying layers. In a layered system, the propagation speed of these different components is affected by the wave speed in those layers through which the components propagate. A layered system is a dispersive medium for R-waves, which means that different frequency components of the R-wave propagate with different speeds, which are called phase velocities.

Phase velocities are calculated by determining the time it takes for each frequency (or wavelength) component to travel between the two receivers. These travel times are determined from the phase difference of the frequency components arriving at the receivers. The phase differences are obtained by computing the cross-power spectrum of the signals recorded by the two receivers. The phase portion of the cross-power spectrum gives phase differences (in degrees) as a function of frequency. The phase velocities are determined as follows:

$$C_{R(f)} = X \frac{360}{\phi_f} f \quad \text{(3.2)}$$

where

$C_{R(f)}$ = surface wave speed of component with frequency, f

X = distance between receivers

ϕ_f = phase angle of component with frequency, f

The wave length λ_f, corresponding to a component frequency, is calculated using the following equation

$$\lambda_f = X \frac{360}{\phi_f} \quad \text{(3.3)}$$

By repeating calculations for each component frequency, a plot of phase velocity versus wavelength is obtained. Such a plot is called a dispersion curve. The shear wave velocity profiles are determined from the experimental dispersion curves through a process called forward modeling or through an inversion process. Both processes will be discussed in the section on interpretation of data.

3.4.2 Spectral Analysis of Surface Waves; Test Equipment

As shown in Figure 3.7, there are three components to an SASW test system: the energy source is usually a hammer but may be a vibrator with variable frequency excitation;

two receivers that are geophones (velocity transducers) or accelerometers; and a two-channel spectral analyzer for recording and processing the waveforms. The required characteristics of the impact source depend on the stiffness of the layers, the distance between the two receivers, and the depth to be investigated (Nazarian et al., 1983). When investigating concrete pavements and structural members, the receivers are located relatively close together. In this case, a small hammer (or even smaller impactor/vibrator) is required so that a short-duration pulse is produced with sufficient energy at frequencies up to about 50 to 100 kHz. As the depth to be investigated increases, the distance between receivers is increased, and an impact that generates a pulse with greater energy at lower frequencies is required. Thus, heavier hammers, such as a sledge hammer, are used.

The two receiving transducers measure vertical surface velocity or acceleration. The selection of transducer type depends, in part, on the test site (Nazarian and Stokoe, 1986a). For tests where deep layers are to be investigated and larger receiver spacings are required, geophones are generally used because of their superior low-frequency sensitivity. For tests of concrete pavements, the receivers must provide accurate measurements at higher frequencies. Thus, for pavements a combination of geophones and accelerometers is often used. For concrete structural members, small accelerometers and small impactors or high-frequency vibrators are typically used (Bay and Stokoe, 1990). The receivers are first located close together, and the spacing is increased by a factor of two for subsequent tests. As a check on the measured phase information for each receiver spacing, a second series of tests is carried out by reversing the position of the source. Typically, five receiver spacings are used at each test site. For tests of concrete pavements, the closest spacing is usually about 0.15 m (Nazarian and Stokoe 1986b).

3.4.3 Spectral Analysis of Surface Waves; Testing Procedures

In SASW tests, two receivers are placed on the surface, and a hammer is used to generate the wave energy. Other sources used in SASW measurements include solenoid-operated impactors and V-meters (high frequency sources) and large drop weights and bulldozers (low-frequency sources). Short receiver (typically accelerometers) spacings are used to sample the shallow layers while long receiver (typically velocity transducers) spacings are used in sampling the deep materials. Two profiles, a forward profile and a reverse profile, are typically obtained in SASW measurements where the accessible surface is struck by a hammer on two opposite sides of the receivers. A signal analyzer is used to collect and transform the receiver outputs to the frequency domain. Two functions in the frequency domain are of great importance in SASW tests:

(1) the cross power spectrum between the two receivers (used in the preparation of the experimental dispersion curve) and

(2) the coherence function (used to ensure the high signal-to-noise ratio data is being collected,

3.4.4 Spectral Analysis of Surface Waves; Interpretation of Data

The experimental dispersion curve as shown in Figure 3.8 can be used to determine the thickness and stiffness of the top uniform layer such as the asphalt concrete layer in a pavement system. The thickness and stiffness of the underlain layers are determined from the experimental dispersion curve through the forward modeling or inversion process. Even if the forward modeling process is not performed, a comparison between experimental dispersion curves of different sites gives relatively valuable information about the existing conditions, but not absolute values of the stiffness or thickness. Exponential windowing can help in the interpretation of the SASW results as unwanted reflections from nearby boundaries are reduced due to the windowing process. Averaging of SASW data from the forward and reverse profiles can also help in the interpretation of data.

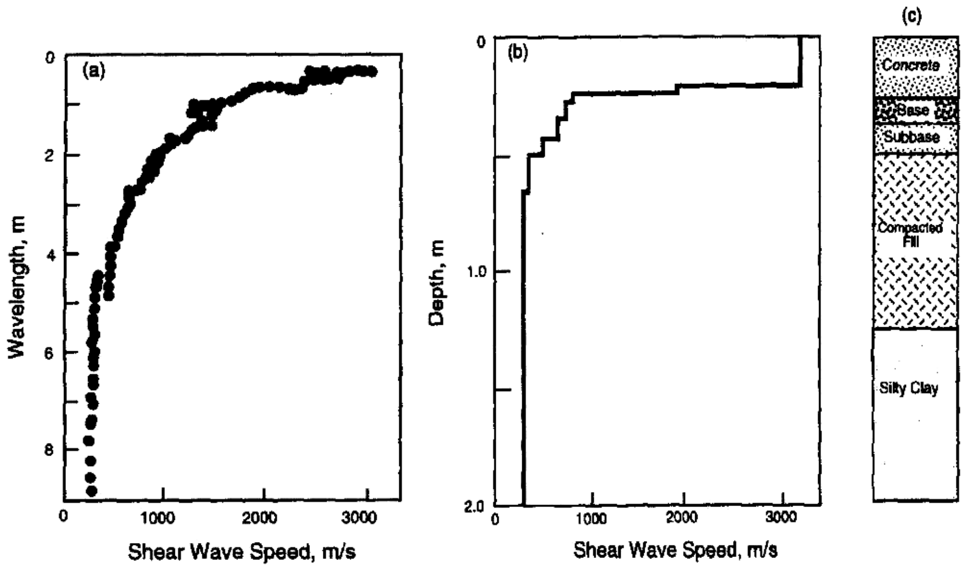


Figure 3.8 (a) Dispersion Curve Obtained from SASW Testing of Concrete Pavement; (b) S-wave Speed Obtained from Inversion of Experimental Dispersion Curve; (c) Soil Profile Based on Boring

Generally inversion is used to obtain the approximate stiffness profile at the test site from the experimental dispersion curve. The test site is modeled as layers of varying thickness. Each layer is assigned a density and elastic constants. Using this information, the solution for surface wave propagation in a layered system is obtained and a theoretical dispersion curve is calculated for the assumed layered system. The theoretical curve is compared with the experimental dispersion curve. If the curves match, the problem is solved and the assumed stiffness profile is correct. If there are significant discrepancies, the assumed layered system is changed or refined and a new theoretical curve is calculated. This process continues until there is good agreement between the theoretical and experimental curves.

3.4.5 Spectral Analysis of Surface Waves; Limitations

SASW measurements require one surface to be accessible for testing. The depth that can be tested by SASW measurements is sometimes controlled by the accessible surface extent. A thin layer of slow velocity material lying between two thick high velocity layers cannot be identified, particularly if this layer is deep.

3.5 Sonic Echo, SE

The sonic echo test is a stress wave reflection method which relies on the measurement of compression wave velocities to verify structural integrity.

3.5.1 Sonic Echo; Principle of Operation

The Sonic Echo test was developed by the Dutch in the 1970s as a means to provide quality control for precast driven concrete piles. Due to the straight-sided shafts of precast piles, sonic echo tests could be used with greater confidence.

The test relies on the reflection of compression waves (fastest of all waves) from the bottom of the tested structural element or from a discontinuity such as a crack or a soil intrusion. The compression wave is generated by means of an impact on the head of the structure. In simple terms, the generated wave from an impulse hammer travels down in a shaft or a pile until a change in impedance is encountered where the wave reflects back and is measured by a receiver placed next to the impact point. Analysis of the Sonic Echo data is performed in the time domain. A typical plot of a sonic echo test result on a concrete drilled shaft is shown in Figure 3.9. The figure shows the processed shaft head velocity versus time. The velocity is obtained by integrating the shaft head acceleration signal measured by accelerometers.

3.5.2 Sonic Echo; Test Equipment

Figure 3.10 shows the typical test set up for the sonic echo test. The sonic echo test equipment consists of a hammer with a triggering device and a vertical accelerometer attached to a portable PC. Geophones can also be used in place of accelerometers. Accelerometers have high frequency sensitivity but it is necessary to integrate the acceleration signal to get the velocity. Geophones measure velocity directly, and are generally considered low frequency transducers. Geophones can measure frequencies above 2 kHz. The portable PC contains a data acquisition card and a signal conditioning unit that, once triggered by an impact, records the response of the structure via accelerometers or geophones.

3.5.3 Sonic Echo; Testing Procedure

A blow on the shaft head by a small sledgehammer equipped with a load cell generates a stress wave with a wide frequency content, which can vary from 0 to 1000 Hz for soft rubber-tipped hammers and from 0 to 3000 Hz for metal-tipped hammers. The vertical

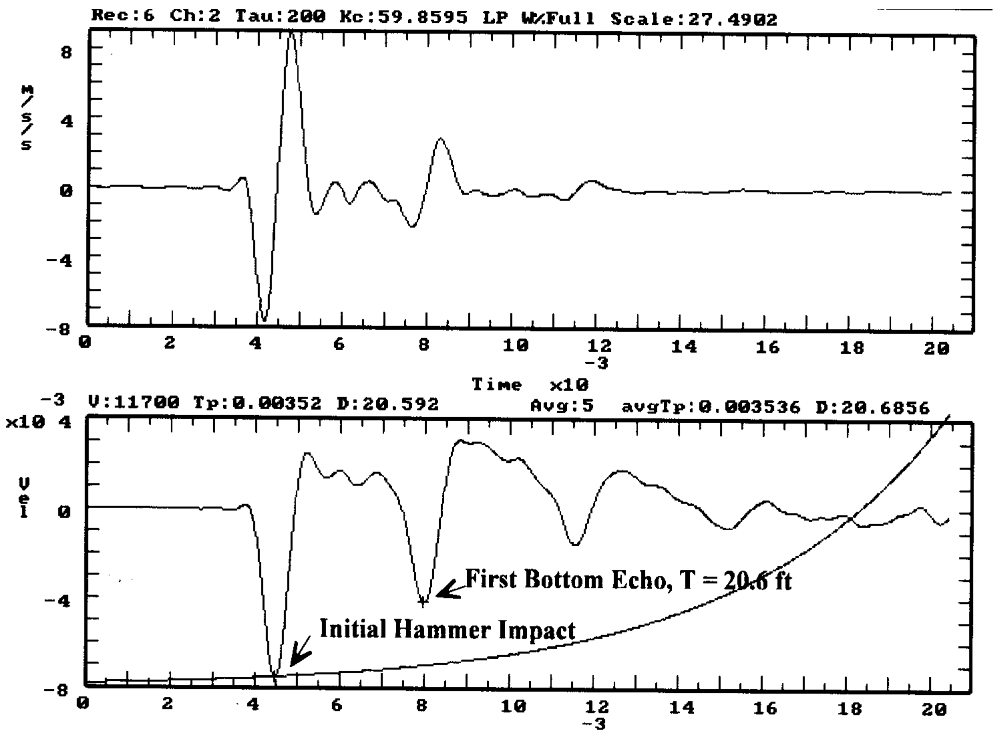


Figure 3.9 Typical Results from Sonic Echo Test

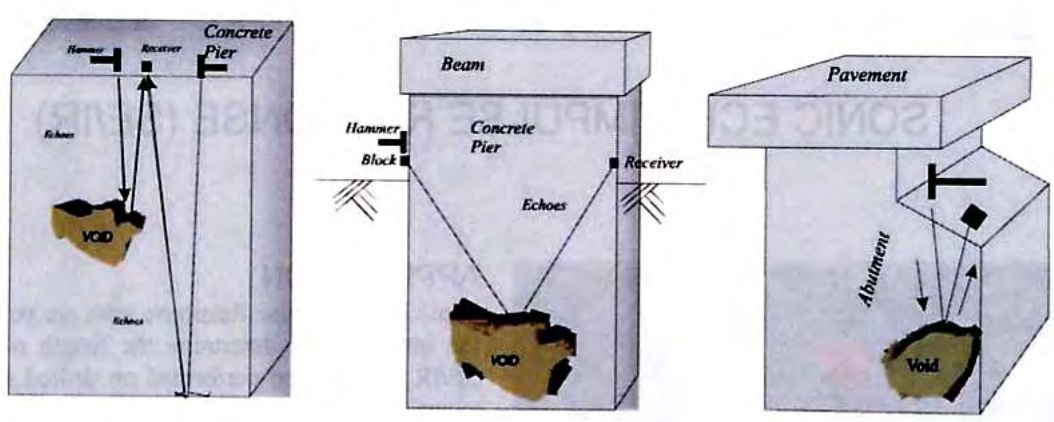


Figure 3.10 Sonic Echo Test Equipment

response of the shaft head is monitored by a geophone.

3.5.4 Sonic Echo; Interpretation of Data

The Sonic Echo data is used to determine the depth of the foundation based on the time separation between the first arrival and the first reflection events, or between any two consecutive reflection events (t_p) according to the following equation:

$$l = \frac{v_p t_p}{2} \quad \text{(3.4)}$$

where l is the reflector depth and v_p is the velocity of compression waves.

Generally the reflection planes in a sound, straight-sided shaft or pile are the concrete-soil interface at the toe and the concrete-air interface at the head. For the concrete-soil interface, some of the energy is reflected and some is transmitted into soil. For the concrete-air interface, almost all energy is reflected back down the shaft. Any defects in the shaft will also cause the energy to reflect. So a reflector can be the bottom of the foundation or any discontinuity along the embedded part of the foundation. If the depth of the deep foundation is known and the transmission time for the stress wave to return to the transducer is measured, its velocity can be calculated. Since the velocity of the stress wave is primarily a function of the dynamic elastic modulus and density of the concrete, the calculated velocity can provide information on concrete quality. So the Sonic Echo data can be used to determine the existence of a bulb or a neck in a shaft or the end conditions of the shaft based on the polarity of the reflection events.

Empirical data has shown that a typical range of values for wave velocity, v_p in concrete can be assumed where 3800 m/s to 4000 m/s (12500 to 13200 ft/sec) would be indicative of good quality concrete, with a crushing strength of the order of 30-35 N/mm^2 (4500-5250 psi). The actual correlation will vary according to aggregate type and mix, and these estimates should be used only as a broad guide to concrete.

When the length of the shaft or the depth of the foundation is known, an early arrival of the reflected wave means it has encountered an obstacle other than the toe of the shaft. This may be a break in the shaft, a significant change in the shaft cross section, or the point at which the shaft is restrained by a stiffer soil layer. In certain cases, the polarity of the reflected wave (whether positive or negative, with respect to the initial impact) can indicate whether the apparent defect is from an increase or decrease of support at the reflective point.

For some defects, the toe may be undetectable due to severe attenuation that can occur, preventing the signal from reaching the toe. Attenuation of the compression wave is a major problem with the surface reflection tests. The impact to the shaft head induces small strains relative to those needed to mobilize the shaft capacity. To increase sensitivity, exponential amplification of the signal has been used to progressively increase the amplitude of the reflected signal in a similar fashion to that in which it was attenuated. Special care must be taken in the amplification process, however, to ensure that the echo is being amplified and not background noise.

Depending on the stiffness of the surrounding soil and the diameter of the nail, there is a maximum length, beyond which all the wave energy is dissipated and no response is detected at the shaft head. In this situation the only information that can be derived is that there are no significant defects in the upper portion of the shaft, since any defect closer to the head than the critical L/D ratio would reflect part of the wave. The limiting L/D ratio varies depending on the adjacent soils, with a typical value for medium stiff clays of 30/1. In soft deposits such as those found in estuary environments, good results can be obtained for L/D ratios of 50:1.

3.5.5 Sonic Echo; Limitations

The sonic echo test is more useful for determining linear continuity of a shaft, but less effective for determining change in shaft cross-section or behavior under load. The concrete wave speed is predicted with a 10% variance allowing an approximation of the concrete quality to be made. If the test is conducted without embedded receivers, only the uppermost defect can be reliably detected. If embedded receivers are used, they must be carefully installed as the shaft is being constructed. To locate a defect, a reflection must be identifiable. Higher frequency compression waves can be generated to detect smaller concrete defects, but the wavelengths cannot decrease to much less than the shaft diameter. Otherwise, the shaft will not behave as a rod-like structure, but as an elastic medium where compression wave reflections will occur from all shaft boundaries [5]. It can be difficult to locate defects present near the toe of the shaft. These defects may produce sufficient reflection that could be easily interpreted as the toe itself. This problem is further complicated by the uncertainties associated with knowing the exact value of the compression wave velocity. Moreover, if the actual length of the shaft is unknown, it is difficult to draw a distinction between the toe and a defect near the toe.

Another problem with the sonic echo test is the noise generated by the hammer impact. This impact causes Rayleigh waves to propagate along the shaft structure, which causes a noisy environment, especially in the top 10 feet of the shaft and results in problems detecting defects close to the shaft head [5]. Accurate determination of the source of a reflection is limited by the types of defects present in the shaft. For example, necking and poor concrete both produce reductions in impedance and distinguishing between the two can be complicated. Also, a defect with a gradual decrease in cross-sectional area may go undetected because a distinct reflection may not be generated. On the other hand, if a bulb (a local increase in cross-sectional area) is present in a shaft, the reflection generated may cause the shaft to be considered defective, when in actuality a bulb does not decrease the shaft integrity. In addition, very stiff soil will also produce reflections similar to those of increases in impedance, causing additional uncertainty in shaft integrity analysis. Surface waves also create problems for interpretation of impulse response signals, especially for tests made under inaccessible-head conditions. The large amplitude of the surface waves can mask the lower amplitude reflections from the toe and defects within the shaft.

A major restriction is the limiting length-to-diameter ratio (L/D) which is further impacted by the soil conditions surrounding the concrete shaft. Highly attenuating soils, such as stiff soils, contribute to attenuation of the compression waves for shafts with high L/D ratios. As the critical L/D ratio is approached, the resolution of the mobility curve, defined by the ratio of P/Q, i.e., the maximum and minimum mobilities, will approach 1.0 as a result of the peaks flattening out to a point where resonant peaks are not discernable due to attenuation effects. Thus, one can not expect to be able to locate the bottom of a drilled shaft under those conditions. The limiting L/D ratio depends upon the surrounding soils, but typically the value in a medium stiff clay is about 30:1. The amplitude of the reflection is also dependent on the impedance contrast at the toe of the shaft between the concrete and the soil/rock bearing strata. The greater the similarity between the toe material and the shaft concrete, the smaller the reflected wave amplitude.

In general, the method is best suited for checking precast and permanently cased piles due to the straight-sided shafts these structures provide. It is not as suitable for drilled shafts due to variations in cross-section that often exist causing multiple reflections.

3.6 Impulse Response, IR

The impulse response test is a stress wave reflection method that, similar to the sonic method, relies on the measurement of compression wave reflections. Additionally, it measures the low-strain hammer impact force.

3.6.1 Impulse Response; Principle of Operation

The impulse response test was developed in France in the late 1970's as an extension of the vibration test. The vibration test was developed by Paquet in 1968 to provide a measure of quality control for the numerous drilled shafts constructed in France.

In the IR test, the vibration test procedure has been replaced by impacting the shaft head with a hammer that induces transient vibrations with frequencies as high as 2000 Hz and measuring the shaft response in the time domain. The signal is digitally converted to the frequency domain. This process can be compared to the human ear which hears sound waves in the time domain and immediately converts them to frequency.

Moreover, this test was found to be more informative than the sonic echo test which does not perform as well with the irregular profiles of drilled shafts. In traditional ultrasonic testing, primary interest has centered about the correlation between the amplitude of the received signals and the size of the discontinuities giving rise to these signals. In this approach it is assumed that the amplitude of the flaw-induced echo is proportional to the size of the flaw. The inadequacy of this approach is that flaw orientation, attenuation, composition, etc. also have drastic effects on signal amplitude. Attempts to correct these shortcomings involved such concepts as the use of multiple transducer (or geophone) locations for detecting the flaw from more than one direction. This in turn led to difficult problems in interpretation. In addition, amplitude information alone is not capable of providing information about flaw shape, which may be of paramount importance, since for certain applications the shape of the discontinuity may determine whether or not its presence will compromise the integrity of the sample. One possible solution to this impasse was to use an ultrasonic pulse with a broad band of frequencies and analysis of the frequency spectrum from a void. At this point frequency analysis has become a viable research tool for supplementing conventional pulse-echo analysis.

3.6.2 Impulse Response; Test Equipment

The test equipment for impulse response test is almost the same as the sonic echo test. Something unique for the impulse response test is that the hammer has a load cell built into it that measures the impact force with time.

3.6.3 Impulse Response; Testing Procedure

The test procedure is same to the sonic echo method. It is very important for this method that the hammer strikes the shaft head squarely to ensure proper force measurement.

3.6.4 Impulse Response; Interpretation of Data

The interpretation of data is what separates this test from the sonic echo method. The impulse response method provides a measure of the homogeneity of concrete in the shaft and also gives information about the shaft performance. In addition to providing length measurements, a stiffness value is obtained which provides useful information about the shaft performance.

In the test, the force and velocity time-base signals are recorded by a digital data acquisition device, and then processed by computer using the Fast Fourier Transform (FFT) algorithm to convert the data to the frequency domain. Velocity is then divided by force to provide the unit response, or transfer function, which is displayed as a graph of shaft mobility against frequency. It is illustrated in Figure 3.11.

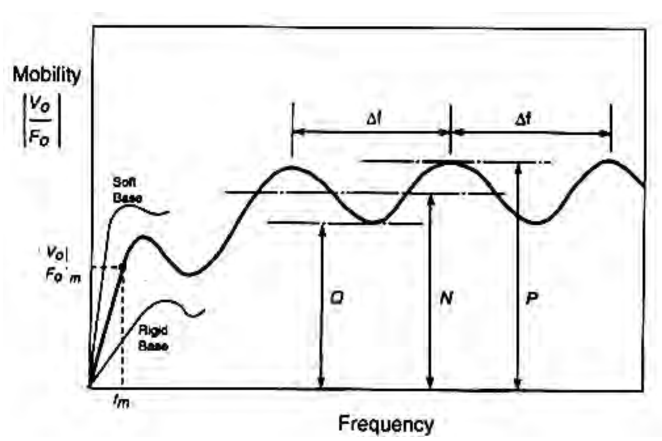


Figure 3.11 Theoretical Mobility Plot for Impulse-response Test of Perfect Pile in Homogeneous Soil

In the graph, the geometric mean value of mobility in the resonant portion of the plot is known as the average mobility, *N*. *N* is given by:

$$N = \frac{1}{\rho_c C_b A_c} \quad \text{...(3.5)}$$

where A_c is the pile cross-sectional area.

This is the inverse of the pile impedance. Also the denser the soil and the longer the pile, the greater is the attenuation, with increasing reduction in the difference between the maximum and minimum amplitudes. A soil damping factor σ can be given by:

$$\sigma = \frac{\rho_s \beta_s}{\rho_c C_b r} \quad \text{...(3.6)}$$

where

ρ_s = soil density

ρ_c = concrete density

β_s = lateral soil shear wave velocity at the pile/soil

r = pile radius

The maximum and the minimum amplitudes *P* and *Q* provide a measure of the soil damping effect from the relationships.

$$P = N \coth(\sigma L)$$

$$Q = N \tanh(\sigma L) \quad \text{...(3.7)}$$

This can also be expressed as:

$$N = \sqrt{PQ} \quad \text{...(3.8)}$$

and σL can be calculated from:

$$\coth(\sigma L) = \sqrt{PQ} \quad \text{...(3.9)}$$

The mass of the pile M_p is calculated as follows

$$M_p = L \rho_c A_c = \frac{1}{2 \Delta f N} \quad \text{...(3.10)}$$

Moreover, this response curve consists of two major portions which contain the following information:

(1) At low frequencies (<100 Hz) the lack of inertia effects cause the pile/soil complex to behave as a spring, which is shown as a linear increase in amplitude from zero with increasing frequency. The slope of this portion of the graph is known as the compliance, the inverse of which is dynamic stiffness. The dynamic stiffness is a property of

the shaft/soil complex and can be used to assess a shaft population on a comparative basis, either to establish uniformity or as an aid to selecting a representative shaft for full scale load testing either by static or dynamic means. For example, for a very compressible base, the measurement of dynamic stiffness is not precise, but by comparing stiffness values for similar sized shafts, it can indicate which shafts warrant further investigation. Lower stiffness values are obtained for shafts founded in soft or loose soils and in shafts containing soil inclusions, necks, and breaks. Mathematically, dynamic stiffness k_d is defined as:

$$k_d = \frac{2\pi f_m}{\left(\frac{V_0}{F_0}\right)_m} \qquad \text{...(3.11)}$$

where f_m is the frequency corresponding to the end of the initial linear portion of the mobility plot. Generally, the stiffness value provides a good indication of the low-stain, soil-foundation interaction. For a very compressible base, the initial slope will be high, hence giving a low stiffness value. Conversely, for a rigid base, the initial slope will be low, giving a high stiffness value.

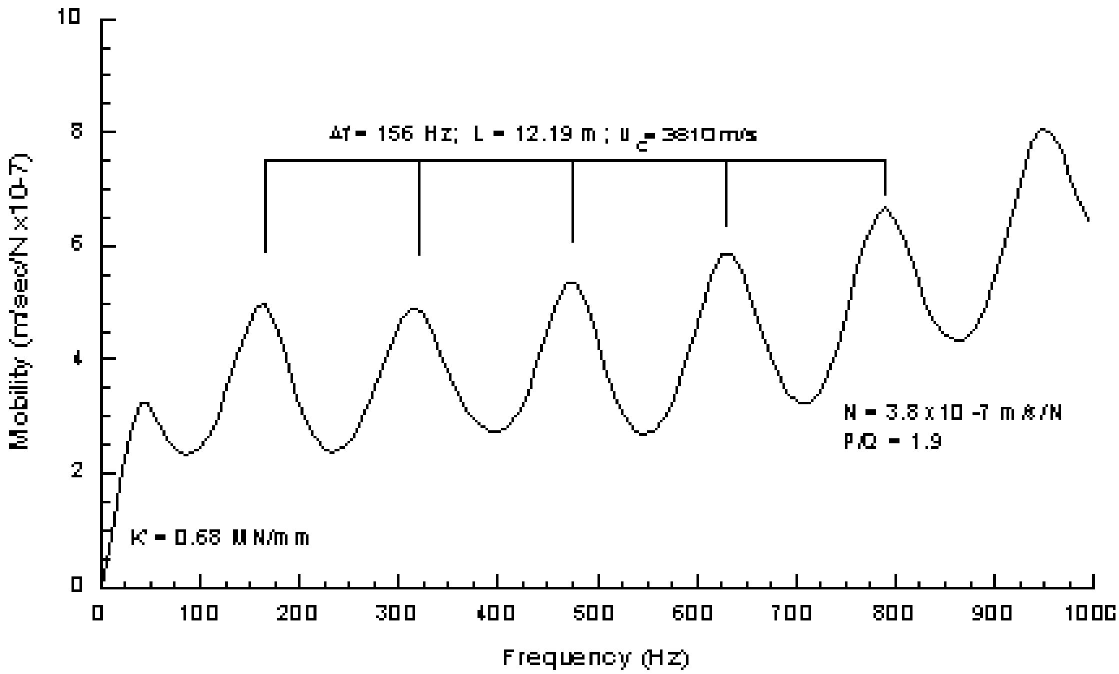
(2) The higher frequency portion of the mobility curve represents the resonance of the shaft. The frequencies of these resonances are a function of the shaft length and the degree of shaft toe anchorage, and their relative amplitude is a function of the lateral soil damping. The mean amplitude of this resonating portion of the curve is a function of the impedance of the pile shaft, which in turn depends on the shaft cross-section, concrete density, and stress wave propagation velocity, v_c.

However, the mobility response curves obtained from real piles are seldom as simple as the theoretical curve for a perfect pile in homogeneous soil as shown in Figure 3.11. It will look like those shown Figure 3.12. This deviation would be caused by several factors. The most common factors are variations in pile diameter, variations in the pile concrete quality with pile length, variations in the lateral soil stiffness and the top section of pile being exposed above ground level.

By measuring the frequency change between these peaks, a shaft length calculation can be made from the following equation.

$$L = \frac{v_c}{2\nabla f} \qquad \text{..(3.12)}$$

As with the sonic echo test, when the shaft length is known, a shorter length measurement will indicate the presence of an anomaly, as demonstrated in Figure 3.12(b). The additional information available from the mobility curve, such as the cross section and dynamic stiffness, can help in differentiating between an increase or reduction in the cross section, even in relatively complex soils.



(a)

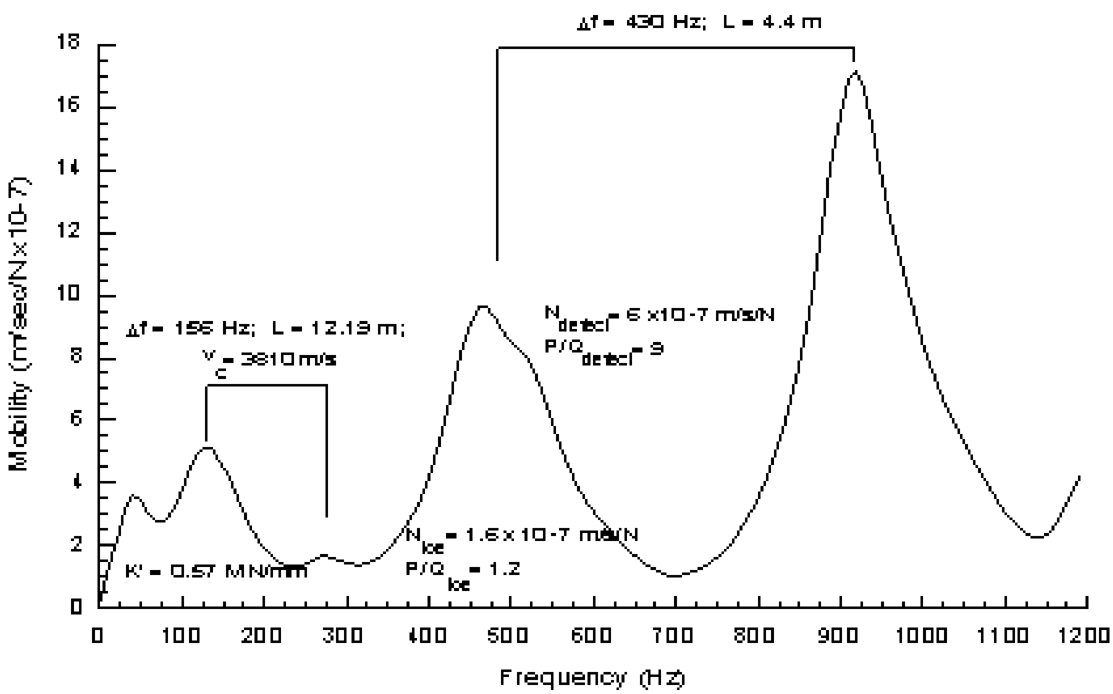


Figure 3.12. Mobility Response Curves for (a) Integral Shaft versus (b) Broken Drilled Shafts

The response curve also contains information on the phase of the reflected signals, shown as a shift of peak frequencies along the frequency axis. The time-based sonic echo results give a signal phase only as positive or negative, with no graduation. The impulse response test makes it possible to quantify the phase shift caused by a change in support conditions, providing information on the quality of contact between the shaft and lateral soil.

3.6.5 Impulse Response; Limitations

The impulse response method shares most of the same limitations associated with the sonic echo method because they are both surface reflection methods and rely on measuring reflected responses at the surface of a structure. Generally the impulse response method works best for columnar type foundations such as piles and drilled shafts. Reflection events are clearest if there is nothing on top of the foundations. In cases where the superstructure is in place, the impulse response data becomes more difficult to interpret because of the many reflecting boundaries or more receivers should be used to track reflections.

3.7 Impedance Logging, IL

A recent approach to interpreting the responses from a combination of both Sonic Echo and Impulse Response surface reflection methods is called Impedance Logging. In this approach the information from the amplified time domain response from the velocity transducer is combined with the characteristic impedance of the shaft measured with the IR test.

3.7.1 Impedance Logging; Principle of Operation

Even though the force applied to the head of the shaft by the surface reflection methods is transient, the wave generated by the blow is not. This wave contains information about changes in shaft impedance as it proceeds downward, and this information is reflected back to the shaft head. The reflectogram so obtained in the sonic-echo test can not be quantified. However, it is possible with modern recording equipment to sample both wave reflection and impedance properties of tested shafts. Measurements of force and velocity response are stored as time-base data, with a very wide band-pass filter and rapid sampling. Resolution of both weak and strong response levels is thus favored. In the reflectogram, a complete shaft defect (zero impedance) is equivalent to 100 percent reflection, while an infinitely long shaft with no defects would give zero reflection. If either a defect or the shaft tip is at a considerable distance from the shaft head, the reflected amplitude is reduced by damping within the shaft. With uniform lateral soil conditions, this damping function has the form $e^{-\sigma L}$, where L is the shaft length and σ is the damping factor, and the reflectogram can be corrected using such an amplification function to yield a strong response over the total shaft length, as is frequently done in the treatment of sonic-echo data. The frequency-domain (impedance) analysis obtained from the impulse-response test confirms shaft length and gives the shaft dynamic stiffness and characteristic impedance.

$$I = \rho_c A_c C_b \quad \text{...} (3.13)$$

where

ρ_c = concrete density

A_c = shaft cross-sectional area, and

C_b = concrete bar wave velocity

In addition, simulation of the tested shaft and its surrounding soil can be carried out most efficiently in the frequency domain. The reflectogram and the characteristic impedance can then be combined to give dimensions to the reflectogram to produce a trace referred to as the impedance log (Figure 3.13). The output of this analysis is in the form of a vertical section through the shaft, giving a calculated visual representation of the pile shape. The final result can be adjusted to eliminate varying soil reflections by use of the simulation technique.

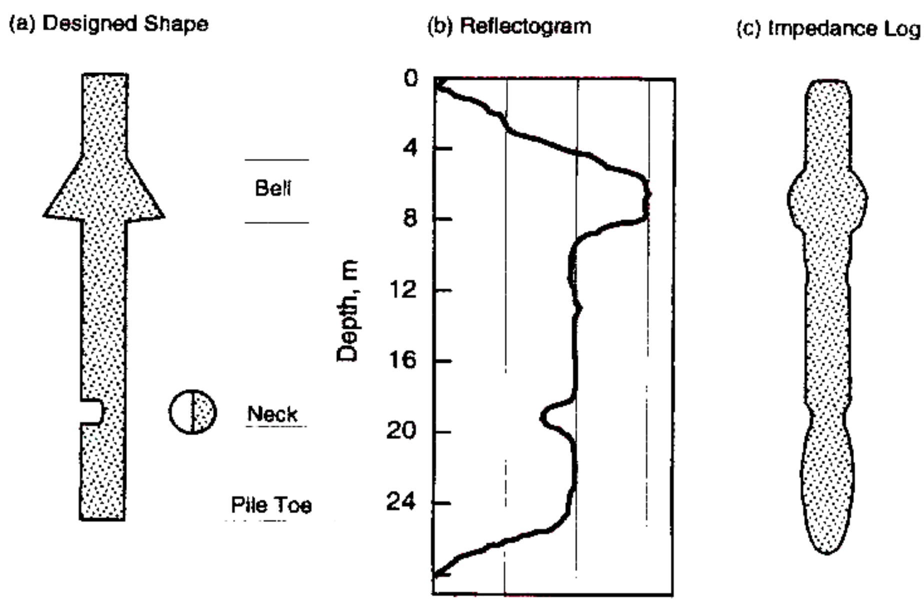


Figure 3.13. (a) Planned Defects in Experimental Pile; (b) Reflectogram Obtained by Signal Processing of Sonic Data; (c) Impedance Log Obtained by Combining Information from Reflectogram and Characteristic Impedance Obtained from Impulse-response Analysis

3.7.2 Impedance Logging; Test Equipment

Field testing equipment must have the following requirements:

a) Hammer load cell and the velocity transducer or accelerometer must have been correctly calibrated (within the six months prior to testing);

b) Data acquisition and storage must be digital, for future analysis; and

c) Both time and frequency-domain test responses must be stored.

3.7.3 Impedance Logging; Testing Procedure

Its testing procedure is same as the sonic echo/impulse response test.

3.7.4 Impedance Logging; Interpretation of Data

Early simulation models used an analogy between mechanical wave propagation and electric transmission line theory. A more recently developed technique for simulation of shaft response to a hammer blow models the impedance characteristics of the pile-soil system only [6]. The pile is divided into as many as ten segments and the pile base. Each pile segment is assigned a length l, diameter d, concrete density ρ_c and concrete bar stress wave velocity v_c (a function of the concrete modulus and density). The soil surrounding the pile shaft is given a shear wave velocity β and density ρ_s. The useful frequency range for most IR drilled shaft testing is 0-2000 Hz. The first step in the model is to calculate the impedance for the pile base for the prescribed frequency range from the equation for the stiffness, k_b of a spring on an elastic base:

$$k_b = 1.84r\left[E_b / (1 - v_b^{\ 2})\right] \qquad (3.14)$$

where r is the pile radius, E_b and v_b are the Young's modulus and Poisson's ratio for the soil at the base.

For each of the pile segments from the top of the shaft downwards, the following parameters are calculated cumulatively:

(a) Body and soil damping coefficient, σ;

(b) Segment impedance, taking into account the geometric and damping properties;

(c) Variation of impedance with frequency.

At the bottom of each segment, the effect of its impedance is added to the impedance from the previous calculations, in the form of a complex array representing the variation of impedance with frequency. The inverse of the magnitude of the complex entry of this array when summed at the end of the cumulative calculation is the simulated mobility for the pile as a function of frequency. By remaining in the frequency mode, magnitudes of force and velocity do not have to be known or assumed for simulation. In this way, changes in shaft and soil properties can be assigned to successive shaft segments. The relatively large number of variables means that several simulation solutions are available. In particular, the selection of the β value is important. This value can range between 50 m/s (very soft clays) to over 300 m/s (rock sockets).

3.7.5 Impedance Logging; Limitations

The result of this processing is an impedance log, which will provide a clear indication of average shaft diameter versus depth. Impedance Log testing has the same general limitations of the previous two tests, but is less prone to false positive results.

3.8 Cross-Hole Sonic Logging, CSL

Cross-Hole Sonic Logging tests are performed to check the concrete integrity of newly placed drilled shafts, seal footings and slurry or diaphragm walls. The testing can be performed on any concrete foundation provided that two or more access tubes or coreholes capable of holding water are present in the foundation. CSL can also be used to check the integrity of underwater concrete piers and foundations by strapping access tubes to the sides. Cross-Hole Tomography can be performed to image critical anomalies found in CSL tests as discussed below.

A companion of the CSL test is the Singlehole Sonic Logging (SSL) test which can be performed in one access tube or corehole to check the integrity of the concrete foundation around the tube in a fashion similar to Gamma-Gamma nuclear density tests.

3.8.1 Cross-Hole Sonic Logging; Principle of Operation

The CSL test relies on propagation of ultrasonic waves between two or more access tubes to measure the velocity and signal strength of the propagated waves.

3.8.2 Cross-Hole Sonic Logging; Test Equipment

Figure 3.14 shows field setups for Cross-Hole Sonic Logging and Single-Hole Sonic Logging tests. Also shown in Figure 3.14 is a sketch of how to obtain data for Cross-Hole Tomography analysis which can be used to better define the location, shape and size of a defect located by CSL. Figure 3.15 shows the test equipment for Cross-Hole Sonic Logging.

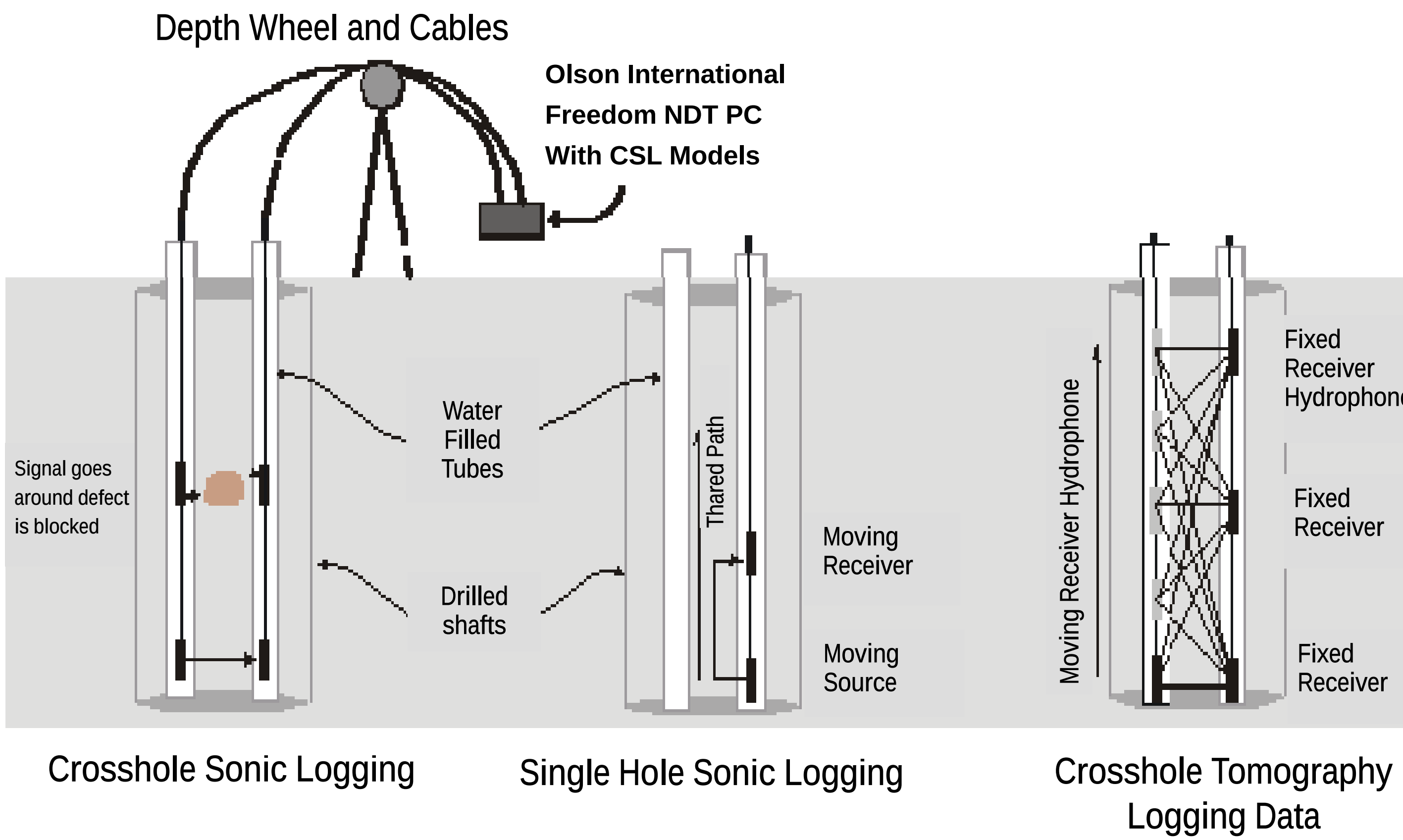


Figure 3.14 Field Setups for Cross-Hole Sonic Logging and Single-Hole Sonic Logging

Access tubes must be installed before the construction of the drilled shaft for quality assurance purposes, unless coreholes are to be drilled in a forensic case. PVC or black steel tubes (U.S. schedule 40) are typically used. The tubes are 1.5 (steel tubes only) to 2 inches (38 to 50 mm) in diameter, and are typically tied to the inside of the rebar cage to ensure close to vertical positions of the tubes. The tubes must extend about 3 feet (1 m) above the top of the shaft to compensate for the water displaced by the source, receiver, and cables. Tubes must be bonded to the concrete for good test results. In order to minimize debonding of tubes, water should be added immediately prior to or after concrete placement and the tubes should not be mechanically disturbed. At least two tubes are needed to perform the CSL test.

Figure 3.15 Test Equipment

The concrete in the shaft should normally be allowed at least 1-2 days to cure to hardened concrete prior to testing. If PVC tubes are used, testing should be done within 10 days after the placement of concrete due to possible tube-concrete debonding. If steel tubes are used, the testing can be done within 45 days after concrete placement as the steel tubes bond better than PVC tubes over a longer time.

3.8.3 Cross-Hole Sonic Logging; Testing Procedure

In a CSL test, the source is lowered to the bottom of one of the tubes and the receiver is lowered to the bottom of another tube. The source and receiver are pulled simultaneously to allow the horizontal ultrasonic pulse velocity to be measured. A depth wheel controls the resolution of the collected data. Typically, the source is excited every 0.2 ft (6 cm) vertically and a measurement is taken. The source and receiver are pulled to the top of each shaft, thus giving a complete assessment of the concrete quality between the two tubes. CSL tests are typically performed between all the perimeter tubes to check the perimeter of the shaft. Additional opposing diagonal CSL tests are also performed to check the integrity of the inner core of the shaft. If there are more than 4 tubes and an anomaly is identified, CSL tests may be performed of subdiagonal tube pairs to further define an anomaly. A pair of tubes can be logged and the results typically displayed on the PC screen in less than 5 minutes.

3.8.4 Cross-Hole Sonic Logging; Interpretation of Data

The data collected from CSL measurements between two tubes at all depths are saved in one file. The file is scanned to determine first wave arrival times and energy levels at all depths. A CSL log shows both the arrival time (or velocity) and signal energy plots vs. depth. In uniform, good quality concrete, the travel time between vertical equi-distant tubes will be relatively constant and correspond to a reasonable concrete pulse velocity from the bottom to the top of the foundation. The CSL test will also produce records with good signal amplitude and energy in good quality concrete. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, void, honeycomb and soil intrusions. In some severe defects, the signal may be completely lost.

Moreover, Ultrasonic Tomography analyses can now be done to better characterize the shape of an anomaly lying between a tube pair. As a result of a National Science Foundation Phase I and II Small Business Innovation Research grant for imaging of flaws in concrete, this research was adapted to image flaws identified by Cross-hole Sonic Logging (CSL). Extensive data is acquired by testing all the angles between a tube pair. The tomographic analysis is then performed on picked travel times to delineate areas of slower velocity, poorer quality concrete. The tomogram, shown in Figure 3.16, is from a drilled shaft foundation of a highway bridge. The results of CSL tests showed an anomaly between 15 to 17 feet deep in this shaft. In order to better image the defect between the tubes, a tomographic dataset was obtained between tubes 1 and 4 (semi-diagonal tube pair) which showed the slowest CSL velocity. In these tests, the source was pulled starting from a point located at 43 feet below the shaft head to the top of the concrete with the receiver suspended at a given location. The receiver was fixed at 49 locations separated by 2.25 inch intervals with the first receiver location also at 43 feet deep.

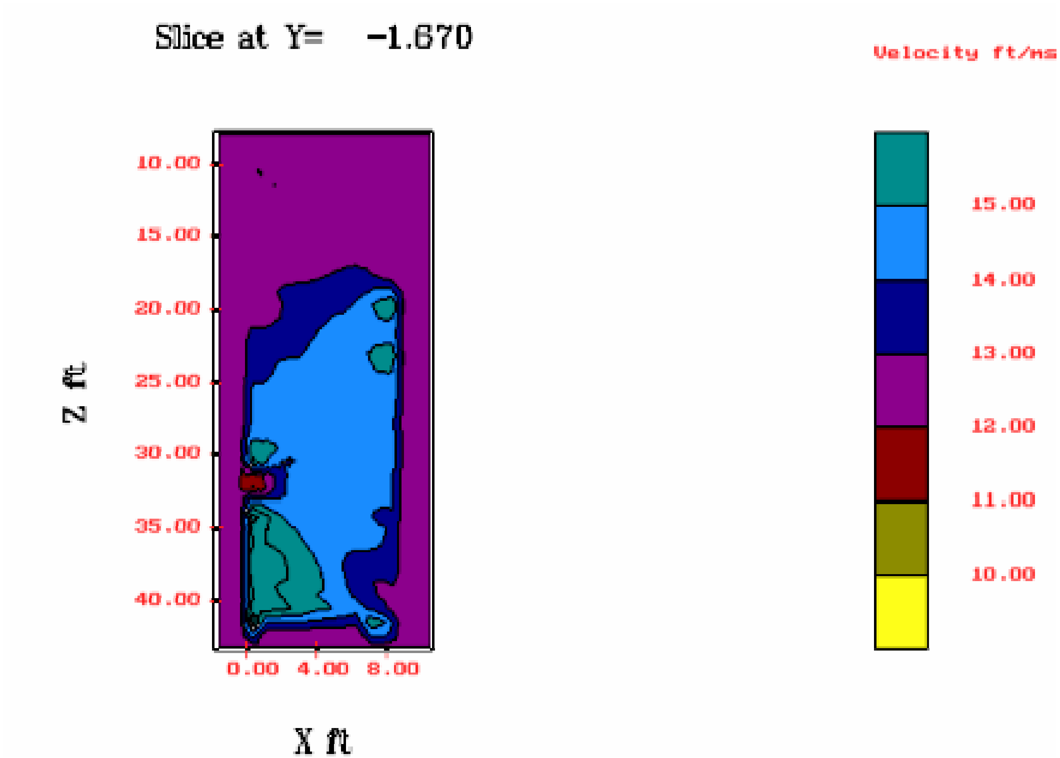


Figure 3.16 Velocity Tomogram on a Drilled Shaft of a Highway Bridge

3.8.5 Cross-Hole Sonic Logging; Limitations

The access tubes must be installed prior to concrete placement. For existing shafts or other concrete members, coreholes or drillholes must be drilled to allow access for the source and receiver hydrophones. CSL is best used for quality assurance. Tubes must be bonded to the concrete for good test results. In order to minimize debonding of tubes, water should be added immediately prior to or after concrete placement and the tubes should not be mechanically disturbed.

The CSL method is the most accurate quality assurance method for defect identification in drilled shafts. CSL testing provides assurance that the foundation concrete is sound and also hardened as velocity to the 4th power is proportional to concrete strength. One of the advantages of the CSL method over the surface Sonic Echo/Impulse Response method is that multiple defects can be identified in the same shaft using CSL which may not be possible with the SE/IR method. In addition, the extent, and the location of the defect can be determined with the CSL method as compared to only the depth of the defect from the SE/IR method. Finally, the CSL method is sensitive to smaller defects and yields more accurate depth information.

3.9 Parallel Seismic, PS

The parallel seismic method is a direct transmission method developed in France in the mid 1970's to evaluate the integrity of drilled shafts and piles under existing structures.

The Parallel Seismic method is applied to determine the lengths of deep foundations when foundation tops are not accessible, or when the piles are too long and slender (such as H piles or driven piles) to be testable by echo techniques. In addition, the PS method can provide information about the soil below the foundation bottom.

3.9.1 Parallel Seismic; Principle of Operation

The PS method involves hitting any part of the structure that is connected to the foundation (or hitting the foundation itself, if accessible) and receiving compression and/or shear waves traveling down the foundation by a hydrophone or a geophone receiver. The receiver is placed in a cased borehole drilled adjacent to the foundation. Analysis of the PS data is performed in the time domain. In PS tests, one relies on identifying direct arrival times of compression and shear waves at the receiver locations, as well as the wave amplitudes. The PS tests are performed at 1-2 ft vertical receiver intervals in the borehole.

3.9.2 Parallel Seismic; Test Equipment

A typical testing arrangement for parallel seismic testing is shown in Figure 3.17. Figure 3.18 shows the test equipment used in parallel seismic method.

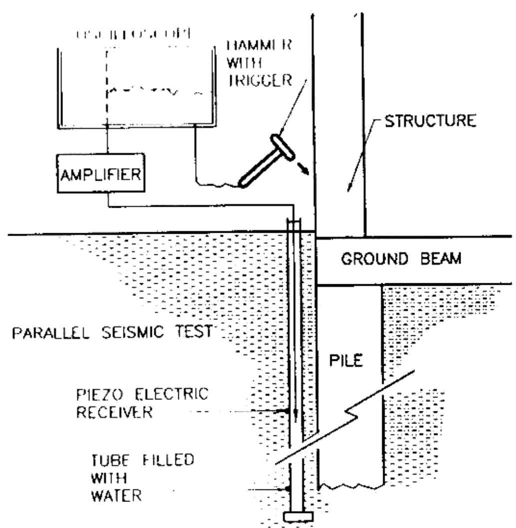


Figure 3.17 Test Layout Parallel Seismic Testing

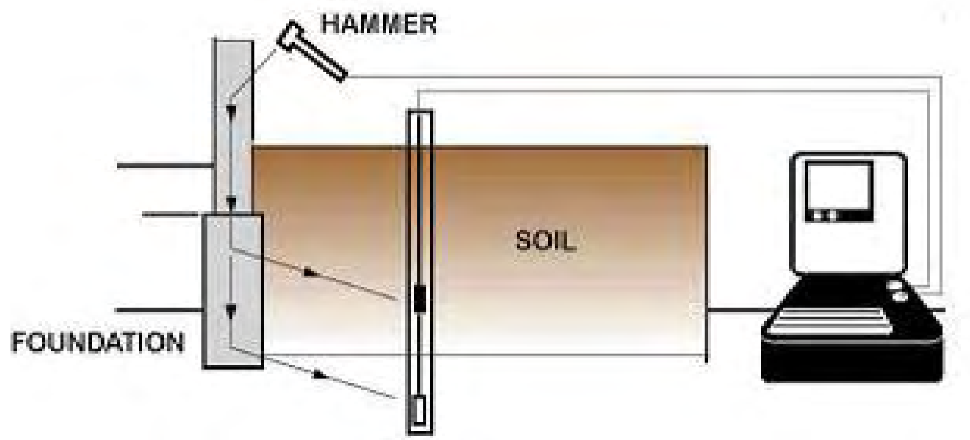


Figure 3.18 Test Equipment for Parallel Seismic Method

The equipment required for the parallel seismic test includes an impulse hammer (typically a 4 or 12 lb impulse hammer), a hydrophone receiver, and a portable computer with appropriate analytical software. The impulse hammer and the hydrophone receiver are connected to a data acquisition card installed in the computer. To signal the data acquisition cycle to begin upon impact, the hammer is outfitted with a trigger. Once impact is made the hydrophone receives the signal and it is recorded by the computer where it can be viewed and stored in the field.

3.9.3 Parallel Seismic; Testing Procedure

To perform the test, a bore hole adjacent to and slightly deeper than the shaft must be drilled. Then the exposed structure is struck with a hammer close to the foundation to generate stress wave energy, some of which travels down the shaft and through the soil where the compression wave passage is monitored by a hydrophone in an adjacent water-filled bore hole. The transit time of the stress wave is measured between the point of impact and the receiver for each probe location. The probe is initially located at the bottom of the borehole, and is raised a short distance after each hammer strike until the entire depth has been sensed.

3.9.4 Parallel Seismic; Interpretation of Data

Hydrophone Data: The time arrival of compression waves is picked from the data for all receiver locations. A plot of the time arrival versus depth is prepared. For uniform soil conditions, two lines are identified in the plot. The slope of the upper line is indicative of the velocity of the tested foundation, and the second line is indicative of the velocity of the soil below the bottom of the foundation. The intersection of the two lines gives the depth of the foundation. For non-uniform soil conditions, the interpretation of data from hydrophone use can be difficult due to the nonlinearity of the first time arrival.

Geophone Data: For uniform soil conditions, the geophone data can be interpreted in a way similar to the hydrophone data. When variable soil velocity conditions exist, an alternative to the first arrival time in data interpretation is used. All the traces are stacked and a V-shape is searched for in the data because the bottom of the foundation acts as a strong source of energy (a point diffractor and a reflector) which produces upward and downward traveling waves. When a geophone is used, the borehole is generally not filled with water. As a result, tube waves are minimized so that later arrival of reflected and diffracted shear and compression waves can be identified.

Figure 3.19 shows PS results from tests performed on a sheet pile in saturated soils. The bottom of the sheet pile is identified at 27.9 ft where the compression wave velocity changes from 17,000 ft/sec (velocity of steel) to a velocity of 5000 ft/sec (velocity of water). Note the clear PS data due to the favorable surrounding soil conditions due to saturation. In these cases, it is very easy to interpret the PS results. PS results from a test performed on a concrete shaft with variable soil conditions are shown in Figure 3.20. The PS results could not be interpreted based on the first arrival of the compression waves. However, the tip of the shaft acted as a source of energy that sent shear waves propagating in the soil below and above the tip of the shaft. The depth of the shaft was interpreted at 36 feet for this test. Note that for sites with variable soil conditions, experience in interpreting the PS data is required. Figure 3.21 shows change in rate of time increase indicates defect at 6.0-6.5m. Moreover, the test result also can used to identify the defect in deep foundations.

3.9.5 Parallel Seismic; Limitations

A major disadvantage of the parallel seismic and downhole tests is the cost of coring and installing the access hole. Another disadvantage is determining the type of defect encountered when a slope change of arrival times occurs. All that is known is that the arrival time increased as a result of a lower propagation velocity. This could be caused by several factors including changes in concrete quality, cracks, and soil inclusion. Also, little

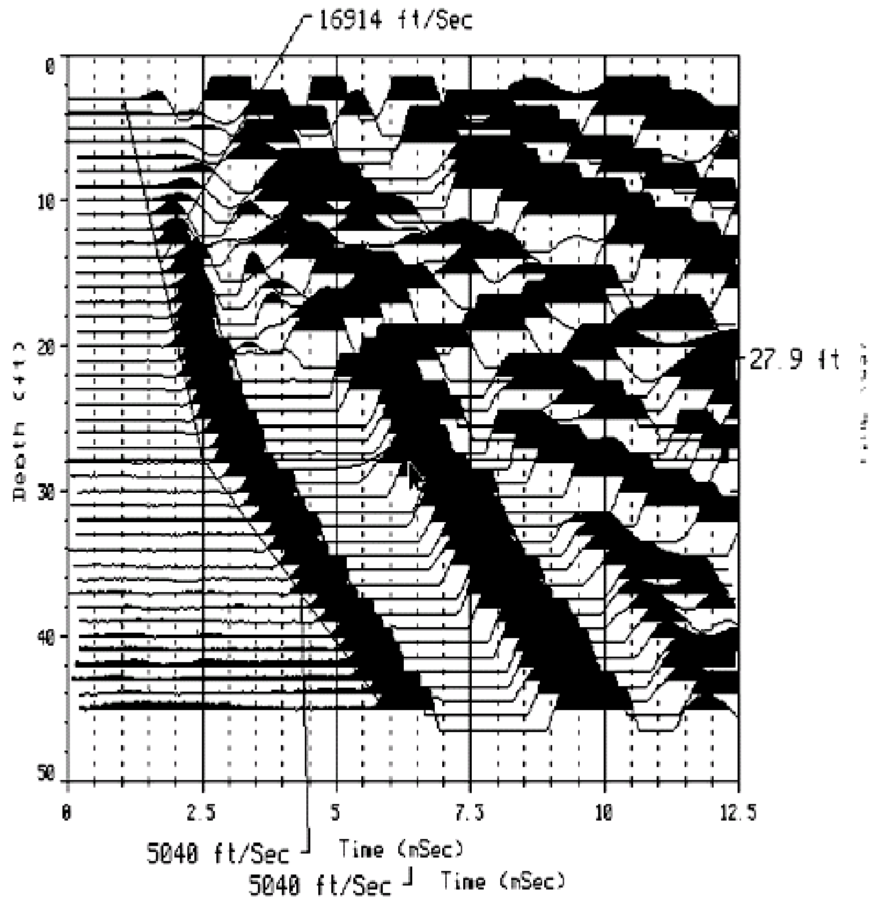


Figure 3.19 Parallel Seismic Test Results for Uniform Soil Conditions

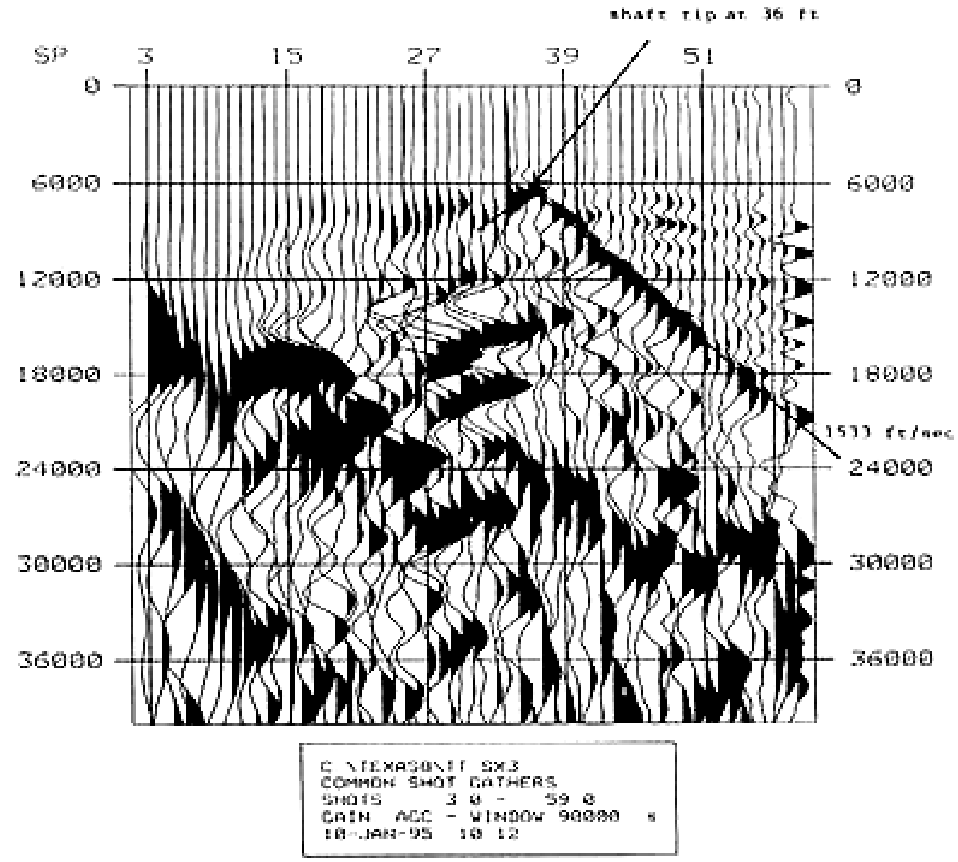


Figure 3.20 Parallel Seismic Test Results for Variable Soil Conditions

has been studied about obtaining information from the tube wave, which theoretically should be able to yield information on the concrete quality.

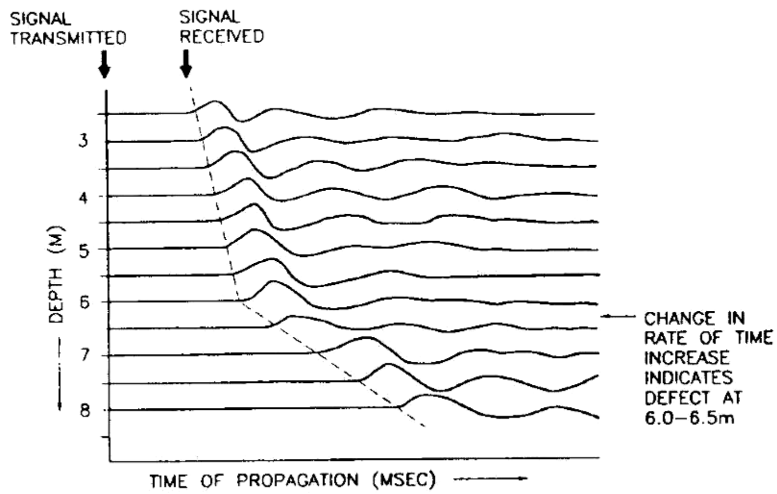


Figure 3.21 Parallel Seismic Test Results for a Pile Defect

3.10 <u>Cross-Hole Seismic, CS</u>

Crosshole Seismic (CS) tests are performed to provide information on dynamic soil and rock properties for earthquake design analyses for structures, liquefaction potential studies, site development and dynamic machine foundation designs. The test determines shear and compression wave velocity profiles vs. depth. Other parameters, such as Poisson's ratios and modules, can be easily determined from the measured shear and compression wave velocities. In addition, the material damping can be determined from CS tests. A companion of the CS test is the Downhole Seismic (DS) test which requires only one borehole.

3.10.1 Cross-Hole Seismic; Principle of Operation

The Cross-hole Seismic method is a downhole method for the determination of material properties of soil and rock. A source capable of generating shear and compression waves is lowered in one of the boreholes, and a pair of matching 3 component geophone receivers are lowered to the same depth in two additional boreholes set at evenly spaced increments (typically 10 and 20 feet from the source borehole) in a line, as shown in Figures 3.22 and 3.23. The receivers are clamped to the side of the borehole casing to allow detection of the passage of shear and compression waves.

As compared to surface methods, the CS method is the most accurate method for determining material properties of rock and soil sites. Thin low-velocity layers lying between high velocity layers can be detected with the method, which may not be possible

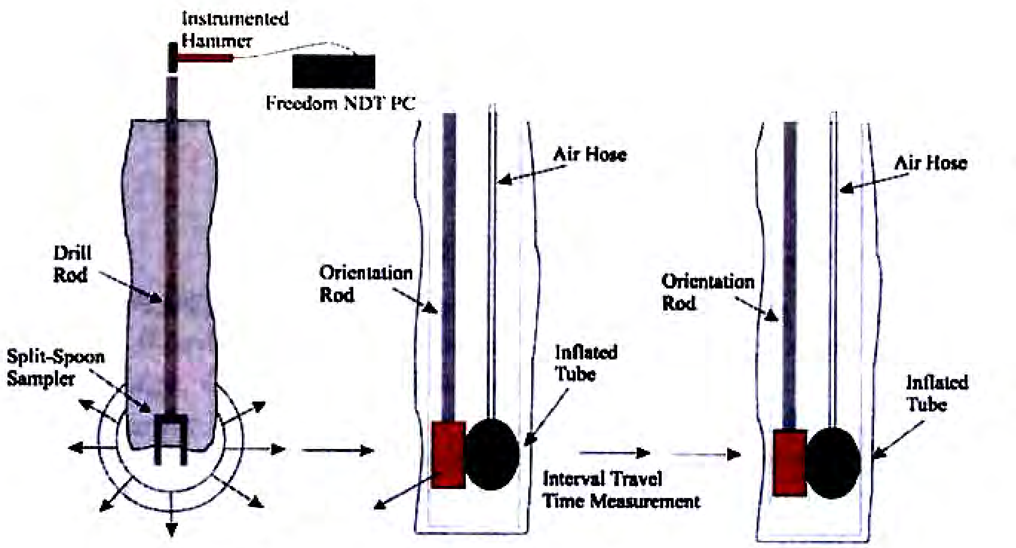


Figure 3.22 CS Tests with Surface Hammer Sources and Orientation Rods

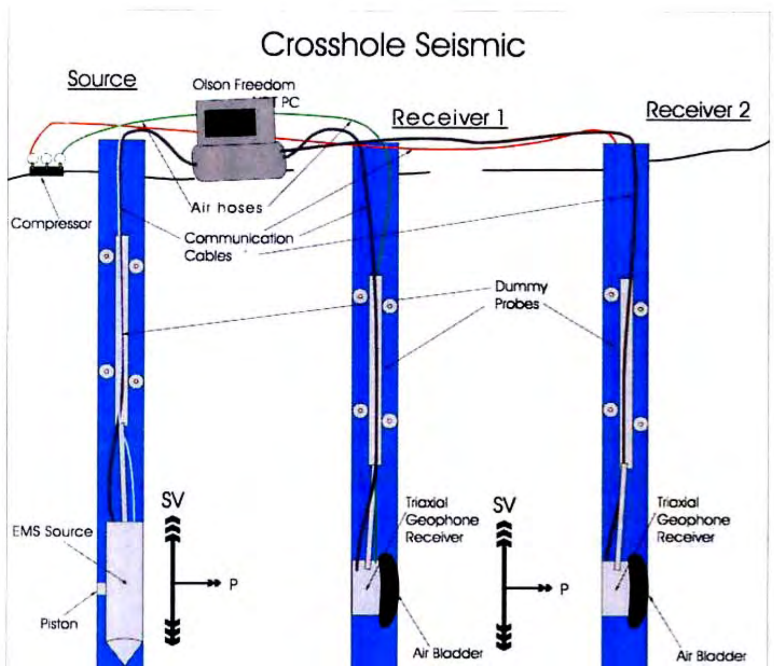


Figure 3.23 CS Tests with Downhole Source and Inclinometer Casing

with surface methods such as Spectral Analysis of Surface Waves (SASW) or Diffraction Survey tests. In addition, the accuracy and resolution of the CS method is constant for all test depths, whereas the accuracy and resolution of the surface methods decreases with depth.

3.10.2 Cross-Hole Seismic; Test Equipment

A typical testing arrangement for parallel seismic testing is shown in Figure 3.22 and Figure 3.23. The CS tests require drilling of two or more (typically three) boreholes. The boreholes are typically 3-4 inches in diameter, PVC cased and grouted to ensure good transmission of the wave energy. The testing is simplified if inclinometer casing is used rather than normal PVC pipe. Typical distances between adjacent boreholes are in the order of 10 feet. Figure 3.22 shows a field setup for CS measurements. The receiver boreholes are drilled to the total investigation depth. For tests using the split spoon as a source (Figure 3.22), the source borehole is advanced during testing at intervals equal to the measurement intervals required (2-5 feet). If a source containing an impactor that can be clamped to the borehole wall is used (Figure 3.22), then the source borehole can be drilled to the total investigation depth prior to testing.

3.10.3 Cross-Hole Seismic; Testing Procedure

In a CS test, the source is lowered to the measurement depth in the incrementally advanced borehole and one or two receivers are lowered to the same depth in the other boreholes. Orientation rods are attached to the source and receiver as shown Figure 3.22, unless inclinometer casing is used. The top of the source rods are struck by an instrumental hammer (or the downhole source is triggered) to generate shear and /or compression wave energy. The vertical component of the receiver is used to capture the vertically propagating shear waves (SV). The radial component senses (Figure 3.22) CS Tests with Surface Hammer Source and Orientation Rods the propagating compression waves (P) and the tangential component senses the horizontally propagating shear waves (SH). The hammer input and the receiver outputs are recorded by our Freedom NDT PC. The source borehole is advanced to the next measurement depth (or the downhole source is lowered to the next depth) and the process is continued until all desired measurements are taken.

3.10.4 Cross-Hole Seismic; Interpretation of Data

If one receiver borehole is used, the travel time from source to receiver is measured. This is referred to as direct travel time measurements. If two receiver boreholes are used, the travel time between the receivers is measured. This is referred to as interval travel time measurements. The wave velocities at the measurement depth are simply calculated by dividing the travel distances by the measured travel time. The travel distances are determined after the verticality of the boreholes is evaluated (inclinometers are typically used). Note that interval travel times are normally more accurate than direct travel times, and thus the three hole test configuration is preferred.

The Poisson's ratio, as well as shear and constrained moduli, can be determined from the shear and compression wave velocities using the following equations:

$$G = \rho \cdot V_s^2$$

$$M = \rho \cdot V_p^2$$

$$\nu = [0.5 \cdot (V_p / V_s)^2 - 1] / [(V_p / V_s)^2 - 1] \quad (3.15)$$

where G is the shear modulus, ρ is the mass density, V_s is the shear wave velocity, M is the constrained modulus, V_p is the compression wave velocity, and ν is the Poisson's ratio.

To illustrate the concepts of the CS test, example results from CS tests on a soil site are presented below (see Figure 3.24).

3.10.5 Cross-Hole Seismic; Limitations

Two, or preferably three, boreholes are required to perform the test. In rock site investigations, the boreholes may be uncased, but in most of the soil site investigations, the borehole should be cased (preferably with inclinometer casing) and grouted.

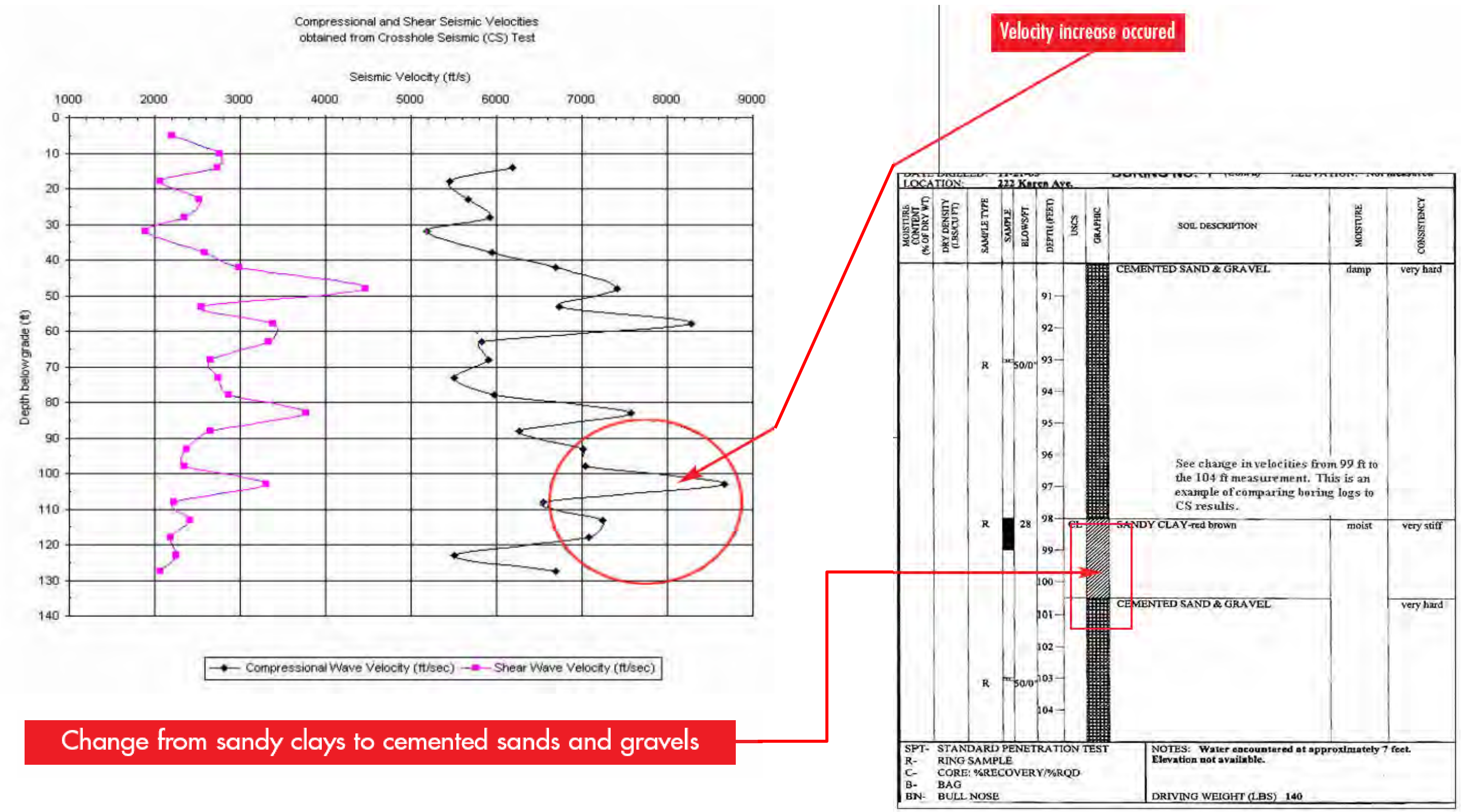


Figure 3.24 Typical CS Test Results

3.11 Ultraseismic, US

Ultraseismic tests are performed to evaluate the integrity and determine the length of shallow and deep foundations. US tests can be performed on drilled shafts and driven or auger-cast piles. The test can also be performed on shallow wall-shaped substructures such as an abutment or a wall pier of a bridge, provided at least 5 to 6 feet of the side of the structural element are exposed for testing. The method is particularly useful in testing abutments and wall piers of bridges because of the relatively large exposed areas available for testing.

3.11.1 Ultraseismic; Principle of Operation

The ultraseismic method represents a more sophisticated approach to the Sonic Echo/Impulse Response method (for compression waves) and the Short Kernel method (for flexural waves). The method was developed as a response to encountered difficulties with the SE/IR and SKM methods when many reflecting boundaries are present. Ultraseismic tests can be performed on concrete, masonry, stone and wood foundations. Steel pile foundations can also be tested, but damping of the energy in this case is much greater than that of concrete and wood due to the large surface areas and small cross-sectional areas of steel piles. Ultraseismic tests can determine the depth of the foundation within 5% accuracy.

3.11.2 Ultraseismic; Test Equipment

The method requires at least 5 to 6 ft be exposed for receiver attachments (see Figure 3.25). The larger the exposed area, the better the definition of the reflected events.

3.11.3 Ultraseismic; Testing Procedure

In an ultraseismic test, the foundation top is struck by a hammer (both vertically and horizontally) and the response of the foundation is monitored by a 3-component receiver. The hammer input and the receiver outputs are recorded by a digital oscilloscope. The vertical hits are used to generate compression waves while the horizontal hits are used to generate flexural waves. The receivers are moved along the exposed surface with intervals of 0.5 to 1 ft depending on the extent of the exposed surfaces.

3.11.4 Ultraseismic; Interpretation of Data

The recorded receiver outputs from the many receiver locations are stacked together much like stacking of geophysical data. The stacking of many traces allows for better tracking of the reflected waves. In addition, the slope of coherent events in the stacked records determines the velocity of the direct and reflected waves to be used in the depth calculation. The confidence in the interpretation of the ultraseismic data is higher than in the SE/IR and SKM test data because of the use of many receiver locations.

In addition to stacking the data from ultraseismic tests, other geophysical data processing techniques can be used. Applications of digital filters and Auto Gain Controlled

(AGC) techniques to the data enhance weak echoes. The separation of downgoing events from upgoing events also enhances the weak echoes coming from the bottom of the foundation or any discontinuity along the buried length of the foundation.

3.11.5 Ultraseismic; Limitations

The ultraseismic method requires at least 5 to 6 ft of the structural member to be exposed which is not always possible. For very deep foundations, echoes from the bottom may not be obtained because of the attenuation of energy in the surrounding soil. The ultraseismic method is not capable of determining depths of buried piles underneath a buried pilecap.

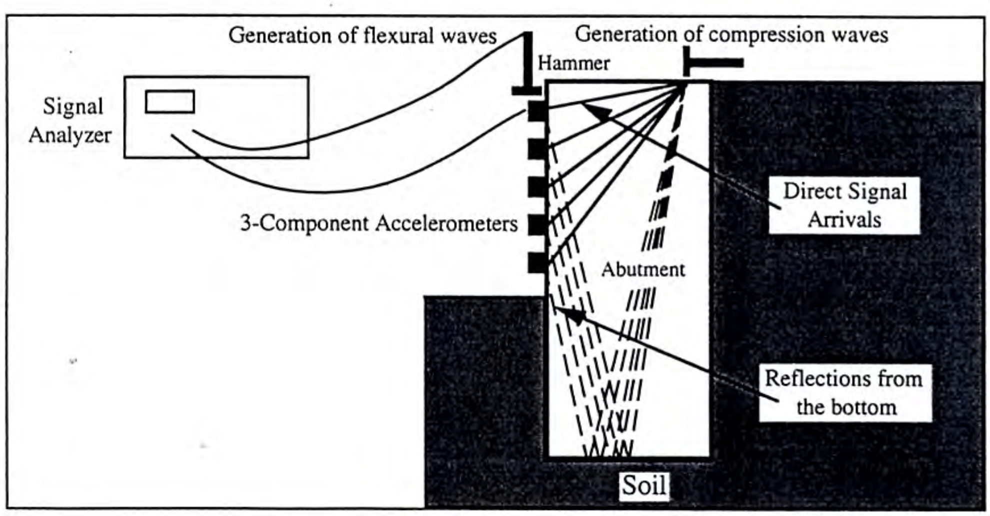


Figure 3.25 Source and Receiver Locations in an Ultraseismic Test (Large Exposed Surface Required)

3.12 <u>Time Domain Reflectometry, TDR</u>

Time Domain Reflectometry (TDR) is a well-established technique in the field of electrical engineering that has been used for many years to detect faults in transmission lines. TDR has also been used in other fields, such as geotechnical engineering and mining. And recently, some researchers have extended the application field of TDR to corrosion detection or void detection of grouted post-tension cable.

3.12.1 Time Domain Reflectometry; Principle of Operation

TDR involves sending an electrical pulse along the transmission line and using an oscilloscope to observe the echoes. Any discontinuity will cause a reflection. From the transit time, magnitude, and polarity of the reflection, it is possible to determine the spatial location and nature of the discontinuity.

A transmission line is a wave guiding system along which electromagnetic waves can travel. It typically has at least two parallel conductors. Examples are telephone lines and television cables. The key difference between transmission lines and conventional circuits is the size. A transmission line can be miles long. Therefore, it is long compared to the signal wavelength. As a result, signals cannot travel instantaneously from one end to the other, as there will be a propagation delay. For a thorough analysis of the wave propagation in a transmission line, one needs to solve Maxwell's equations with boundary conditions imposed by the physical nature of the system under investigation. It is also possible to represent a line by the distributed parameter equivalent circuit and discuss wave propagation in terms of voltage and current, as shown in Figure 3.26.

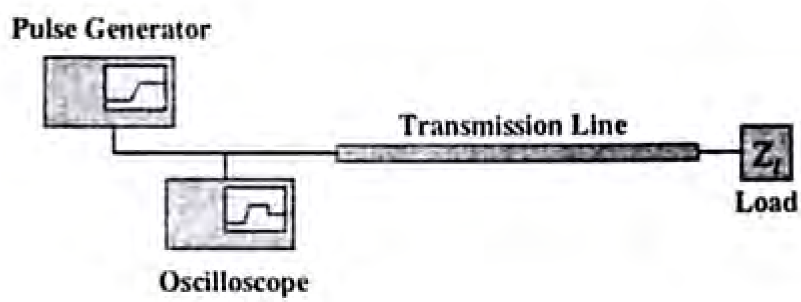


Figure 3.26 Functional Block Diagram for Typical Time Domain Reflectometry

A distributed parameter model is used to study the wave propagation in this transmission line. The distributed parameter equivalent circuit is shown in Figure 3.27.

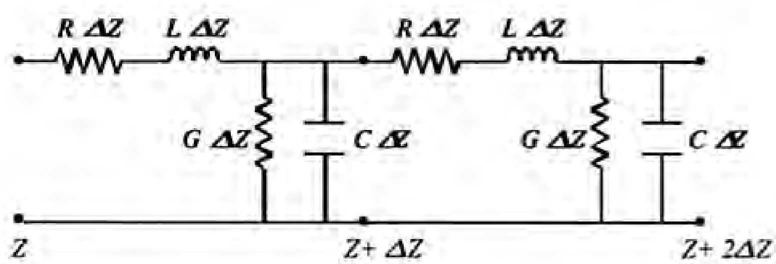


Figure 3.27 Distributed Parameter Equivalent Circuit for the Steel Cable Transmission Line

It possesses a uniformly distributed series resistance *R*, series inductance *L*, shunt capacitance *C*, and shunt conductance *G*. (*R*, *L*, *C*, and *G* are defined per unit length). By studying this equivalent circuit, several characteristics of the transmission line can be determined. By applying Kirchhoff's voltage and current laws to the distributed equivalent circuit, the characteristic impedance of the line can be given by:

$$Z_0 = \sqrt{\frac{R + j\varpi L}{G + j\varpi C}} \qquad (3.16)$$

At high frequencies, the characteristic impedance is given to a high degree of accuracy by the simplified expression

$$Z_0 = \sqrt{\frac{R + j\varpi L}{G + j\varpi C}} \approx \sqrt{\frac{L}{C}} \qquad (3.17)$$

The distributed parameters of the steel strand transmission line are calculated from the geometry and material parameters of the strand. The capacitance per unit length is calculated by considering the electric field of two parallel infinitely long straight line charges of equal and opposite uniform charge densities. The equal-potential surfaces are cylinders with axes parallel to the line charges. The capacitance per unit length of the line is obtained by placing the two conductors in two equal-potential surfaces, and calculating the potential difference. The inductance per unit length is calculated similarly. The resistance per unit length includes the resistance of the strand and sensor wire. To calculate the resistance at high frequency, skin effects must be taken into account.

When the wave travels down the transmission line at v_p, the velocity of propagation, at every point that the excitation crosses, the transmission line equations must be obeyed. For a line terminated by a load Z_1, if Z_1 is different from Z_0, the transmission line equations are not satisfied unless a second wave is considered to originate at the load and propagate back up the line, i.e., a reflection is generated at this point. The ratio of reflected voltage to the incident voltage is denned as voltage reflection coefficient, Γ and is related to Z_1 and Z_0 by

$$\Gamma = \frac{V_r}{V_i} = \frac{Z_1 - Z_0}{Z_1 + Z_0} \qquad (3.18)$$

The reflected wave is superimposed on the incident wave. However, they are separated in time. This time, T, is the transit time from the monitoring point to the mismatch and back again. Therefore, the distance from the monitoring point to the mismatch is calculated to be $D = v_P T/2$.

3.12.2 Time Domain Reflectometry; Test Equipment

A time domain reflectometry is usually configured as shown in Figure 3.25.

3.12.3 Time Domain Reflectometry; Testing Procedure

The testing procedure for TDR is very simple. It involves sending an electrical pulse along a transmission line and using an oscilloscope to observe the echoes returning back from the system being tested.

3.12.4 Time Domain Reflectometry; Interpretation of Data

A typical TDR waveform is shown as in Figure 3.28. The figure shows the TDR reflection from a 3-m steel rebar sample. This sample has one simulated 50% pitting corrosion site 1.55 m from the front end. In the figure, the first step in the waveform corresponds to the generation of the step wave (A). The wave is launched into a coaxial cable, which is used to connect the sample to the measuring system. The characteristic impedance of this coaxial cable is 50Ω. However, the sample has higher impedance. As a result, there is a positive reflection at the beginning of the sample (B). At the end of the sample, the wave goes up, because the line is terminated by an open circuit (D). In the middle of the sample there is a simulated corrosion site. A positive reflection from that site is observed at location (C). The time interval between points B and D is 23.0 ns, which gives a propagation velocity of 2.61x10^8 m/s, i.e., about 87% of the speed of light. The location of the damage site is accurately determined as 1.58 m from point B, because $T_C - T_B$ = 12.1 ns. The accuracy of the distance measurement can be further improved with better coaxial cable-to-specimen connections.

3.12.5 Time Domain Reflectometry; Limitations

Generally for TDR, a silver monitoring wire running parallel to the element is required. In some situations, this is not possible.

Figure 3.28. TDR Returns from 3-m Reinforcing Steel Sample

3.13 Recommendations About Candidate NDT Methods

Section 3.2 through Section 3.12 reviewed a number of NDT techniques that are widely used in the evaluations of structural components and deep foundations. In this section, recommendations concerning candidate NDT methods for the evaluation of installed soil nails are presented based on the above review.

The number of NDT techniques available for evaluating particular structural conditions is rapidly expanding. Based on reliability, simplicity, and cost, some methods or techniques are preferable over others. NDT methods are valuable tools, but there is no one NDT technique that will work equally well for a broad range of applications. Often times, the most sensible and effective approach is to use a variety of nondestructive testing techniques to obtain mutually complementary information. A general guideline for selecting candidate methods for certain type of problems could be found in ACI 228.2R-98, Nondestructive Test Methods for Evaluation of Concrete in Structures [7].

The primary purpose of this research study is to find NDT techniques which can reliably determine the integrity of the grout column and the length of nails. As mentioned in Chapter II, there are several different types of defects commonly found in installed soil nails. It is true that none of the currently available NDT techniques is sensitive to all types of defects, which makes it necessary to select different candidate NDT methods according to the specification of defects.

Bird's mouth is a defect that occurs at the nail head. After shotcrete of wall surface is placed, the bird's mouth is buried and presents itself as a shallow defect. As discussed in Section 3.2, the special challenge in finding shallow defects lies in the fact that there is no sufficient time between the generation of stress pulses and the reception of waves reflected from shallow defects, which makes it very difficult to determine the arrival time of reflected waves. Hence, the NDT methods in which data analyses are performed in the time domain are not suitable for detecting this kind of defect. Even some NDT methods that interpret test results in the frequency domain (e.g. impulse response, impedance logging etc.) are not best suited because they use low-frequency wave sources that significantly reduce their sensitivity to minor defects. Impact echo method is always used to determine the thickness of thin structures based on its high frequency wave sources and frequency domain data analysis. Considering the specification of bird's mouth, we believe that Impact Echo technique is the most promising test method for detecting bird's mouth.

Deep defects, such as soil cave-in, air voids at the middle or end of soil nails, and insufficient grout length, present as an abrupt impedance change in installed soil nails. The NDT methods selected to detect these defects must be capable of penetrating deep into soil nails and the reflected waves must be strong enough to be picked up by the sensors. Among the NDT methods reviewed, Sonic Echo/Impulse Response, Cross-hole Sonic Logging, and Parallel Seismic methods could meet this requirement. The direct transmission methods such as Cross-hole Sonic Logging and Parallel Seismic methods have advantages of ease of interpretation and lower susceptibility to attenuation effects; while the surface methods such as Sonic Echo/Impulse Response methods have the advantages of low cost and simple implementation procedures. In this research study, the small diameter of soil nails makes it very difficult to install access tubes inside soil nails for Cross-hole Sonic Logging method. The final choice of candidate NDT methods for detecting deep defects is Sonic Echo/Impulse Response, and Parallel Seismic techniques.

The length of steel tendon nail is another concern of this project. The steel tendons used in construction projects usually have a small diameter and large L/D ratio. In order to detect the end of such a steel tendon, wave sources with high frequency contents are preferred over low frequency waves. However, high frequency waves tend to experience severe attenuations, which makes them unlikely to have large propagation distance. Hence current stress wave based methods are not able to reliably detect the end of long steel tendons. However, Time Domain Reflectometry does not have this kind of limitation. In a typical soil nail, the steel tendon goes all the way to the end of borehole and it is a good analog of cable. The end of steel tendons represents abrupt characteristic impedance change which can be detected by Time Domain Reflectometry methods.

The maximum length of soil nail/grout column which can be probed by the selected NDT-method is also of interest in this research study. In stress wave based NDT methods, this value depends on the attenuation of stress waves during propagation, which makes it necessary to study the property of surrounding soil. The damping effect of surrounding soil is the key parameter to estimate the wave attenuation effect. Spectral Analysis of Surface Wave and Cross-hole Seismic methods are proposed to implement at the experiment site to evaluate soil damping effects.

Table 3.2 summarizes the selected candidate NDT methods for specific problems occurring in installed soil nails.

Table 3.2 Selected Candidate NDT Methods

Type of Defect	**Selected candidate NDT methods**
Bird's mouth	Impact Echo
Void at the middle or end of grout column, half fill	Sonic Echo, Impulse Response, Parallel Seismic
Length of steel tendons	Time Domain Reflectometry
Surrounding soil properties	Spectral Analysis of Surface Wave, Cross-hole Seismic
Stress wave velocity in Grout	Ultrasonic Pulse Velocity

CHAPTER IV

DESIGN AND CONSTRUCTION OF AN EXPERIMENTAL SOIL NAILED WALL ON TTU CAMPUS

4.1 Overview

The preceding chapter, Chapter III, provided a detailed description of the review process used in the evaluation and selection of candidate NDT techniques for soil nail testing. As shown in Figure 1.1, the next task involved further evaluation of these selected NDT methods in a series of field tests on actual soil nails. For this purpose, a 6.5-ft tall, 150-ft long retaining wall with 32 test soil nails was especially built at the Texas Tech University field research site. The test nails included "defect-free nails" as well as "nails with intentional defects." The nail lengths, grout mixture, and type of defect were varied so that the capabilities of each NDT method under a range of conditions could be evaluated.

The construction of the experimental soil nailed wall was accomplished as a collaborative effort between Texas Tech University and the Granite Construction Company. The Granite Construction Company was selected because of the company's extensive experience as a soil nailed wall contractor for TxDOT. The design of the wall (nail lengths, grout types, and defect types) as well as preparation of tendons with defects mounted on desired locations, and sampling and testing of grout samples were done by Texas Tech researchers. The excavation of the site to create the 6.5-ft tall vertical cut, drilling, nail installation and grouting was done by the Granite Construction Company. Soil nail wall construction was completed in two separate phases. The first phase consisted of a retaining wall embedded with 24 soil nails and a 25-ft reserve panel with no soil nails. However, after the first two series of NDT testing were conducted, unplanned defects were found in some of the soil nails. Most of these unplanned defects appeared to have been caused by the use of inappropriate construction procedures. Therefore, a second phase of construction was undertaken so that problems found in the first phase of construction could be overcome. During the second phase of wall construction, 8 new soil nails were added to the reserve panel bringing the total number of soil nails to 32.

In this chapter, details related to the construction of the experimental soil nailed wall, soil nail specifications and defect simulations are presented.

4.2 Site Selection

The initial planning for the construction of the test wall was conducted by the TTU research team with input from Granite Construction as well as Olson Engineering Company. The preliminary planning involved the selection of a suitable site and the design of the wall. As a part of the wall design, it was necessary to determine the major test variables to be included, the number of nails representing each parameter combination and nail configuration (or layout). Construction plans were then prepared based on the selected design.

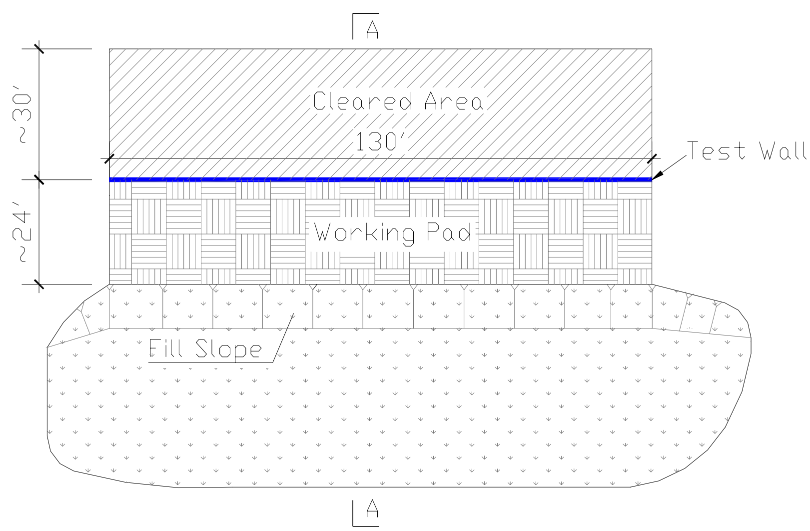
Figure 4.1 shows the site selected for the construction of the experimental soil nailed wall. This site was selected based on following considerations:

(a) Easy access for construction equipment,
(b) Sufficient space for maneuvering drilling equipment,
(c) Ability to accommodate a soil nailed wall up to 130-ft length,
(d) Ability to accommodate soil nails of lengths up to 30-ft that were to be later exhumed for direct observation
(e) Availability of water and electricity supply.

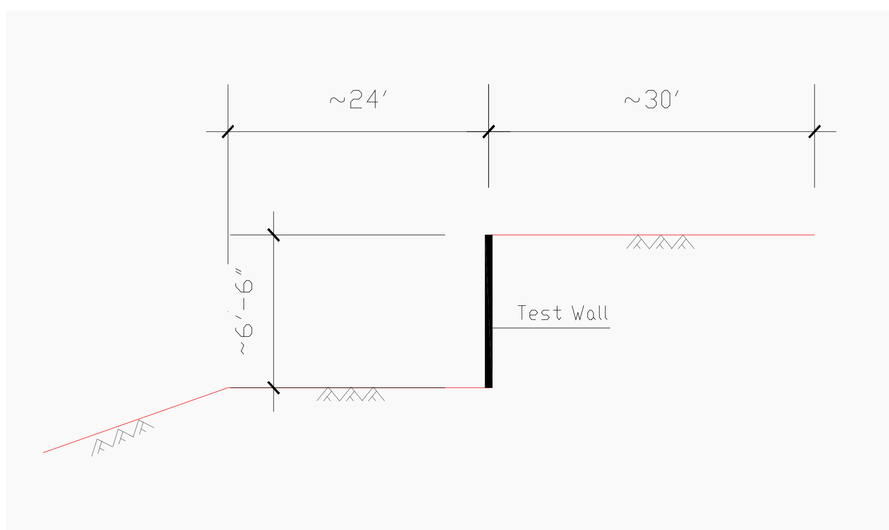
Figure 4.1 Site Selected for the Construction of the Test Wall

4.3 <u>Design of the Soil Nailed Wall</u>

The major variables that were included in the test soil nailed wall design are: (a) nail length, (b) grout mixture design, and (c) type of defect. Nail length is an important parameter because the length of nail that could be probed by a given NDT technique is limited when the cross sectional area of the nail is fixed. Therefore, it was important to find out from the experiments conducted in this study the limits of capability for each NDT method examined. Secondly, there is considerable variation in the consistency of the grout mixtures used in TxDOT soil nailed wall construction. Therefore, in the design of the test wall two mixtures were used; (a) a sand-cement mixture to represent stiff grouts with low flowability, and (b) a neat mixture to represent lean grouts with high flowability. Thirdly, the wall design included the following defect conditions: (a) no defects, (b) void at the end of the nail, (c) void at the middle of the nail, and (d) bird's beak (or bird's mouth) defect. The Phase 1 wall construction involved a 100-ft long, 6.5-ft high working panel that accommodated 24 soil nails and a 25-ft long reserve panel. All of the nails were installed in a single row placed at a depth of 3 ft from the top of the wall. The nails were spaced 4 ft from one another. The nail length, type of defect, and grout mixture design were varied so that a broad range of conditions were represented. Figures 4.2 (a) and (b) show the plan view of the test wall and a vertical section respectively. Figures 4.3 and 4.4 show the details of soil nail layout used in Phase I construction.

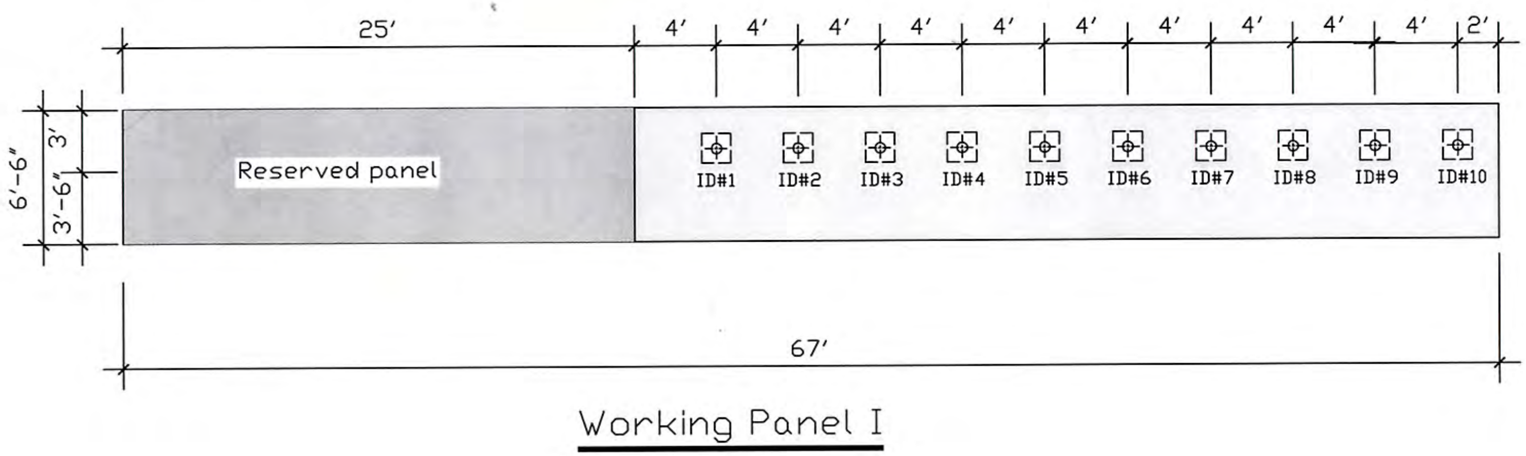


(a)



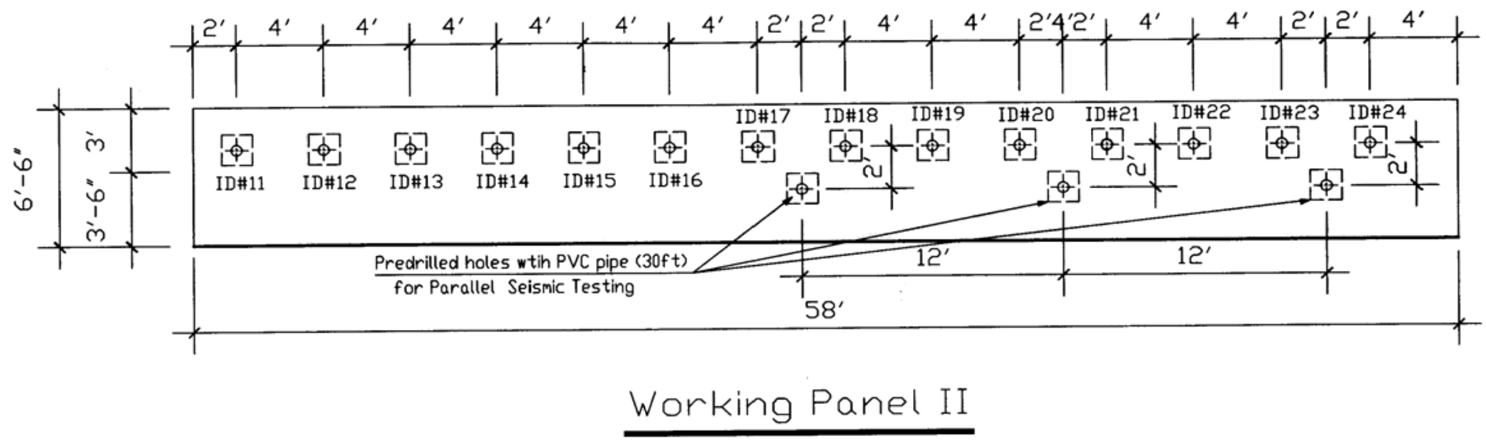
(b)

Figure 4.2 (a) Plan View of Test Wall, (b) Section A-A through Test Wall



Working Panel I (Shaded Area)	Grout Mixture Type	Length of Nails				
		5'	10'	15'	20'	25'
Soil nail with no defect	Sand Cement Grout	ID#1	ID#4	ID#7	ID#13	ID#19
Soil nail with no defect	Neat Cement Grout	ID#2	ID#5	ID#8	ID#14	ID#20
Defect at the end of soil nail	Sand Cement Grout	ID#3	ID#6	ID#9	ID#15	ID#21
Defect at the middle of soil nail	Sand Cement Grout			ID#10	ID#16	ID#22
Bird mouth	Sand Cement Grout			ID#11	ID#17	ID#23
Defect at the middle of soil nail	Neat Cement Grout			ID#12	ID#18	ID#24

Figure 4.3 Working Panel I in Phase 1 Wall Construction



Working Panel II (Shaded Area)	Grout Mixture Type	Length of Nails				
		5'	10'	15'	20'	25'
Soil nail with no defect	Sand Cement Grout	ID#1	ID#4	ID#7	ID#13	ID#19
Soil nail with no defect	Neat Cement Grout	ID#2	ID#5	ID#8	ID#14	ID#20
Defect at the end of soil nail	Sand Cement Grout	ID#3	ID#6	ID#9	ID#15	ID#21
Defect at the middle of soil nail	Sand Cement Grout			ID#10	ID#16	ID#22
Bird mouth	Sand Cement Grout			ID#11	ID#17	ID#23
Defect at the middle of soil nail	Neat Cement Grout			ID#12	ID#18	ID#24

Figure 4.4 Working Panel II in Phase 1 Wall Construction

As described in Chapter III, the candidate NDT techniques selected in this research for further study included Parallel Seismic and Cross-hole Seismic methods. In order to implement these two testing methods, it was necessary to drill additional bore holes parallel to the test soil nails to serve as access holes for NDT instrumentation. These additional drillholes are shown adjacent to Nails 17 and 18, Nails 20 and 21 and Nails 23 and 24 (See Figure 4.4). 30-ft long, 2-in diameter PVC pipes were inserted into these drillholes and the annular space sealed with bentonite grout.

In the design of the experimental soil nail wall, the nail diameter and the angle of inclination of the nails were kept constant. The nail diameter was not varied because 6-in diameter drillholes are used in all TxDOT construction projects almost exclusively. The angle of nail installation is typically maintained at 10°-15° to the horizontal. The nails are sometimes installed at shallower angles when obstructions such as underground utility lines or other buried structures prevent their installation at the desired batter. In the construction of the experimental soil nail wall, a nail inclination angle of 10° to the horizontal was used.

The length of the proposed soil nails varied from 5 ft to 25 ft, representing the full range of lengths commonly used in field construction. As mentioned earlier, it was important to test nails of different lengths because this would provide data on the maximum length that could be tested with each candidate NDT method. The test nails represented four defect conditions; no defects, voids at the end of soil nails, voids at the middle of soil nails and bird's mouth. Grout materials used were neat cement or sand cement mixtures. Both are widely used in soil nail construction projects in Texas. Table 4.1 provides a summary of the parameter combinations used in Phase 1wall design.

Table 4.1 Phase 1 Test Wall Design

	Grout Mixture Type	**Length of Nail**				
		5ft	**10ft**	**15ft**	**20ft**	**25ft**
Soil nail with no defect	Sand Cement	ID#1	ID#4	ID#7	ID#13	ID#19
Soil nail with no defect	Neat Cement	ID#2	ID#5	ID#8	ID#14	ID#20
Defect at the end of the nail	Sand Cement	ID#3	ID#6	ID#9	ID#15	ID#21
Defect at the middle of the soil	Sand Cement			ID#10	ID#16	ID#22
Bird's mouth	Sand Cement			ID#11	ID#17	ID#23
Defect at the middle of the nail	Neat Cement			ID#12	ID#18	ID#24

4.4 Simulation of Soil Nail Defects

According to standard TxDOT specifications, 1-inch diameter, Grade 60 epoxy coated steel tendons were used in the test nails. The centralizers used were of the split-PVC style. As shown in Figure 4.6, the centralizers were fastened to the tendon with duct tape.

The objective of this phase of the research was to determine whether the selected NDT methods were capable of detecting various types of defects that are commonly found in actual soil nail construction projects. The defects that are most common and therefore, of greatest interest to this study were; voids found at the far end of a nail, reduced cross section of the grout column at the middle of the nail, and bird's mouth type defects near the nail head. To study how each candidate NDT method would behave under different defect conditions, it was necessary to intentionally introduce defects in the grout columns of test nails.

Figure 4.6 1-inch, Grade 60 Epoxy Coated Steel Tendon with Split PVC Style Centralizer

To create voids in the grout column, a closed-cell, polyethylene foam was used. The foam was available in 1-inch thick sheets. Donut shaped disks were cut from the polyethylene sheet and sufficient number of donuts were mounted on the tendon until a foam column equal to the desired length of the void was formed. Figures 4.7 through 4.9 illustrate this process. The foam columns prevent the grout materials from penetrating through creating a defect in the grout column at desired locations. The stress wave used in the NDT tests would reflect back upon reaching the interface between the grout material and the foam. Figures 4.10 through 4.13 are construction plans representing soil nails with the four different defect conditions.

Figure 4.7 Polyethylene Foam Disks Used in Defect Simulation

Figure 4.8 Mounting Foam Disks on Steel Tendons

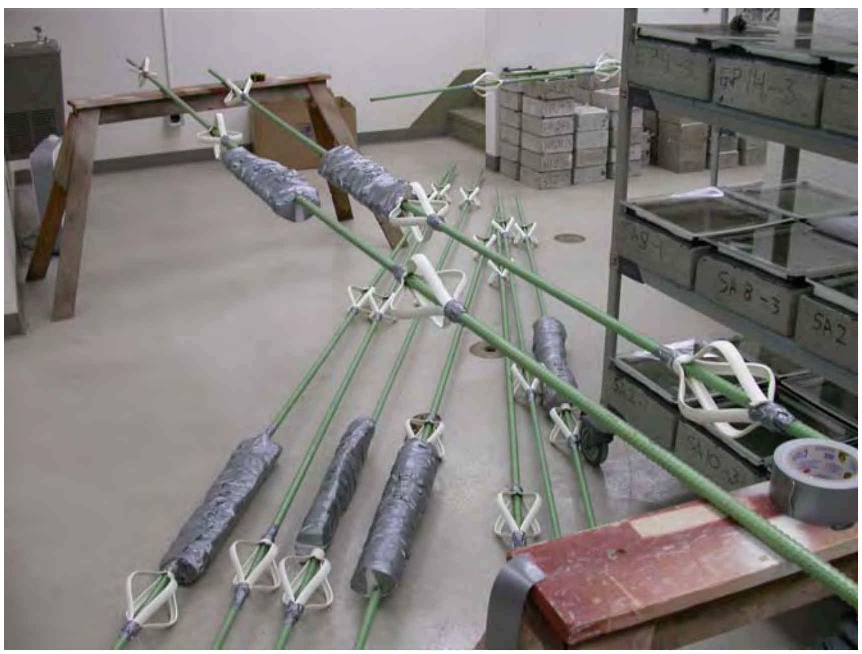
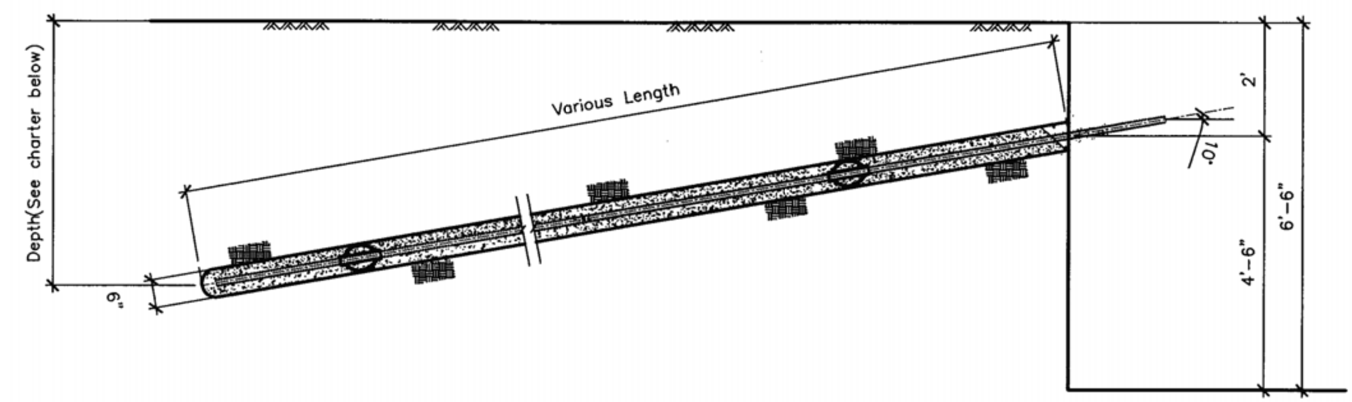
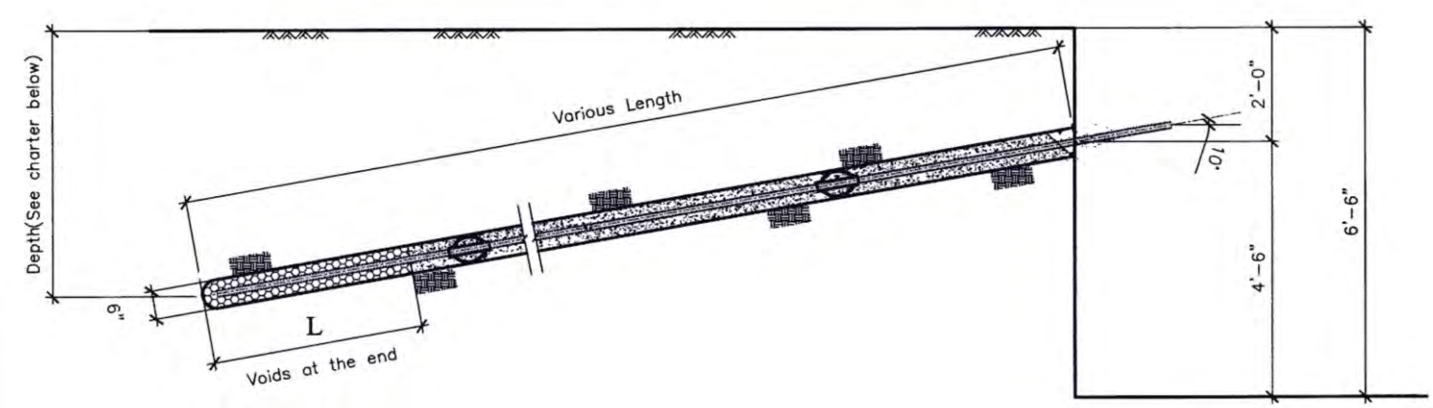


Figure 4.9 Soil Nail Tendons with Foam Disks Mounted to Simulate Voids at the Middle of the Nail



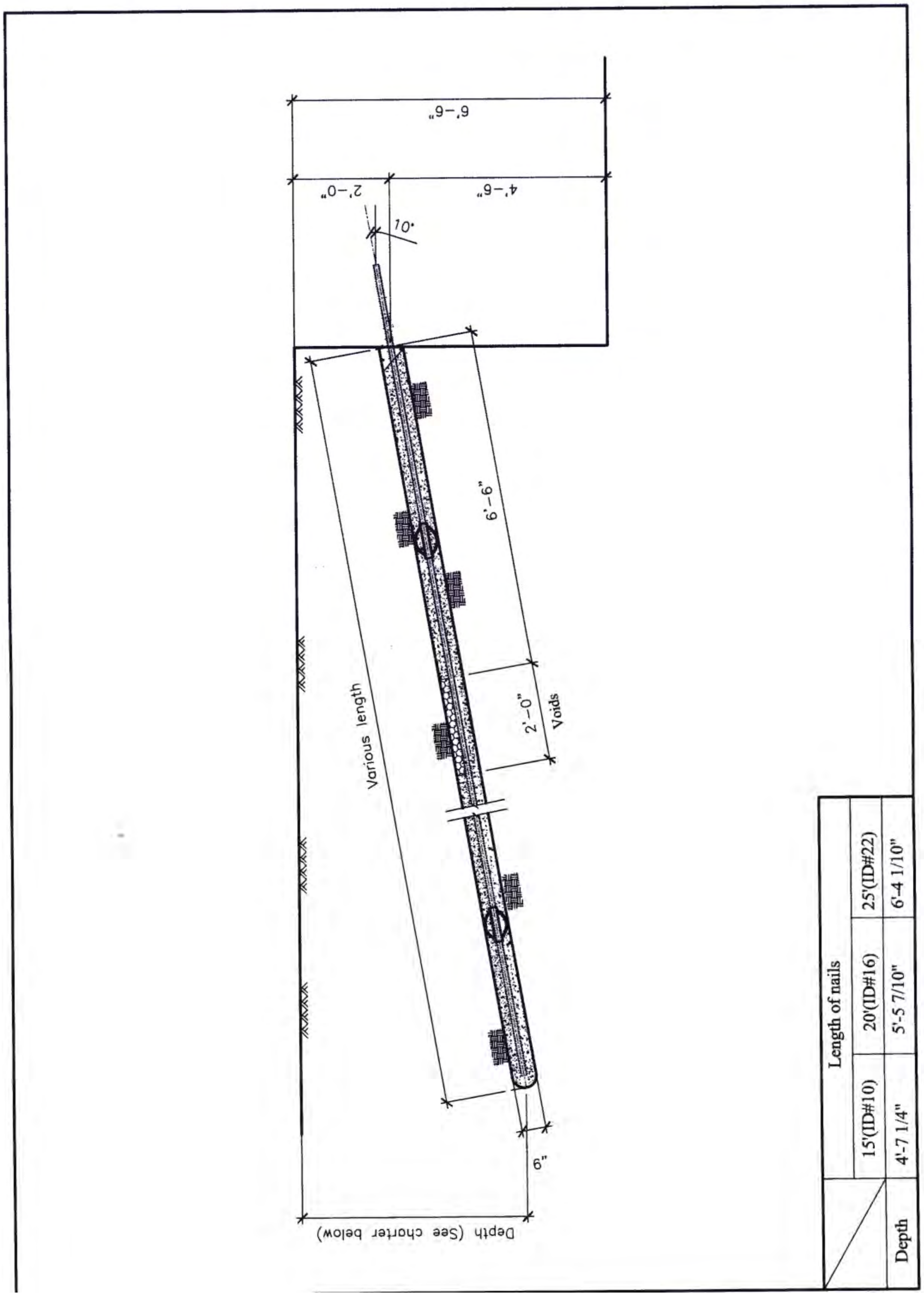
	Length of nails				
	5'(ID#1,2)	10'(ID#4,5)	15'(ID#7,8)	20'(ID#13,14)	25'(ID#19,20)
Depth	2'-10 3/8"	3'-8 13/16"	4'-7 1/4"	5'-5 7/10"	6'-4 1/10"

Figure 4.10 Soil Nails without Defects; Phase 1 Construction



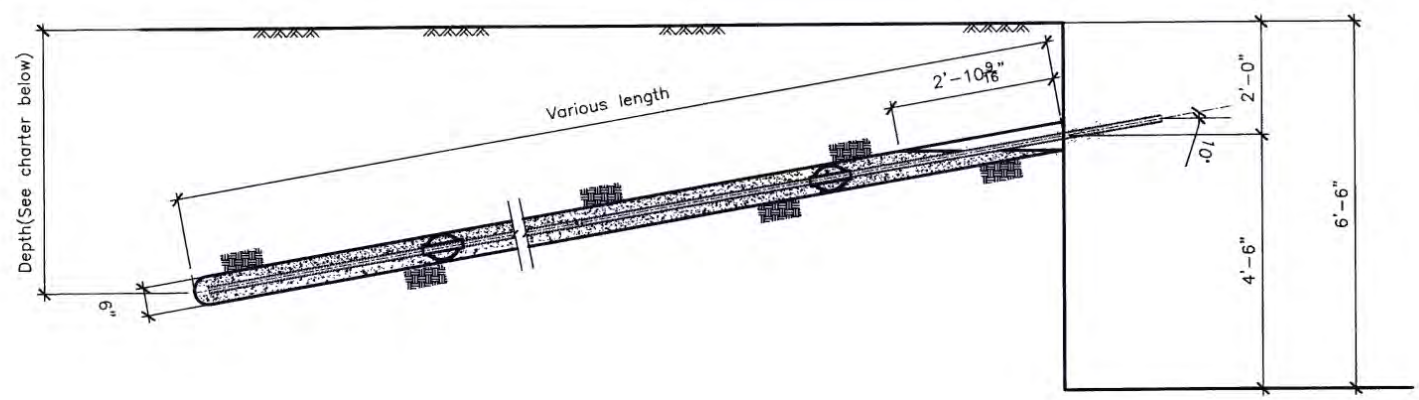
	Length of nails				
	5'(ID#3)	10'(ID#6)	15'(ID#9)	20'(ID#15)	25'(ID#21)
Depth	2'-10 3/8"	3'-8 13/16"	4'-7 1/4"	5'-5 7/10"	6'-4 1/10"
L	1'-6"	2'-6"	3'-6"	4'-6"	5'-6"

Figure 4.11 Soil Nails with Defects at the End; Phase 1 Construction



Depth \ Length of nails	15'(ID#10)	20'(ID#16)	25'(ID#22)
Depth	4'-7 1/4"	5'-5 7/10"	6'-4 1/10"

Figure 4.12 Soil Nails with Defects at the Middle; Phase 1 Construction



	Length of nails		
	15'(ID#11)	20'(ID#17)	25'(ID#23)
Depth	4'-7 1/4"	5'-5 7/10"	6'-4 1/10"

Figure 4.13 Soil Nails with Bird's Mouth Defect; Phase 1 Construction

4.5 Field Construction; Phase I

Phase I field construction took place during the months of July through November, 2003. Cleaning the site in preparation for construction began on July 19, 2003. Construction involved soil excavation to create a 6.5-ft tall vertical cut, drilling holes for soil nail installation, inserting soil nail tendons with centralizers and foam columns that simulate defects and finally, grouting of soil nails. Phase I construction of the experimental soil nail wall was completed on Nov. 15, 2003. The photographs shown in Figures 4.14 through 4.27 document various stages of construction of the soil nail wall in Phase I.

Figure 4.14 Site Cleaning and Soil Excavation

Figure 4.15 View of the Construction Site after Completion of the Excavation

Figure 4.16 Drilling Holes for Installation of Test Soil Nails

Figure 4.17 View of the Wall Face with Drillholes

Figure 4.18 Insertion of Steel Tendons into Drillholes

Figure 4.19 Grout Pump Used in Test Wall Construction

Figure 4.20 Grouting Truck and Grout Pump

Figure 4.21 Sampling Grout for Testing

Figure 4.22 Soil Nail Grouting Using Tremie Pipe

Figure 4.23 Grout Overflow at the Completion of Grouting

Figure 4.24 PVC Pipes Used as Access Tubes for NDT Instrumentation
(Parallel & Cross-Hole Seismic Tests)

Figure 4.25 Flexible Hose Used for Placing Bentonite Grout Around PVC Access Tubes
(Parallel & Cross-Hole Seismic Tests)

Figure 4.26 PVC Access Tubes after Annulus had been Grouted with Bentonite

Figure 4.27 Completed Test Soil Nail Wall

4.6 Problems Encountered in Phase 1 Construction

At the completion of Phase I construction, non-destructive testing of the 24 test soil nails started. Two construction problems that may limit the effectiveness of NDT testing were recognized during these tests. One of them involved the poor condition of the nail head and the other was the actual grout length.

Since shotcrete facing had not been placed prior to testing, the nail head conditions could be easily observed prior to NDT testing. In many cases, the condition of the nail heads was found to be less than optimum for NDT testing. Figure 4.28 shows an example of such a nail head. The transducers or geophones used in NDT testing are mounted on the exposed surface of the nail head. When the nail head is incompletely grouted or when the nail head surface too rough or irregular, good contact between the grout column and the sensors is not achieved. The poor condition of the nail head can also affect the quality of the impact generated. The first series of NDT testing was conducted with the nail heads in the same condition they were found after Phase I construction. Before the second series of testing, nail heads were re-grouted, considerably improving the nail head conditions.

Figure 4.28 Nail Head in Poor Condition

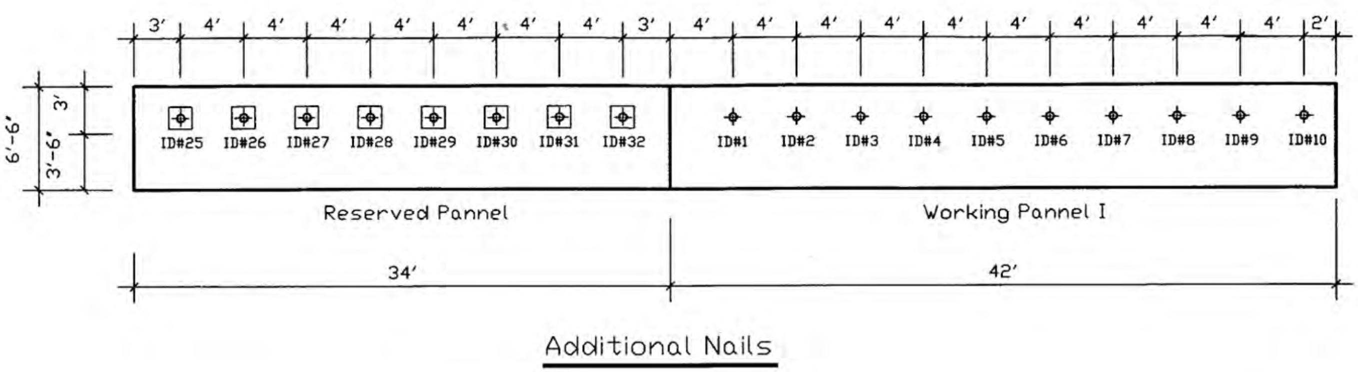
The second problem associated with Phase I construction involved the actual length of grout column. It was discovered that the actual lengths were different from those specified in the construction plans. The results from the first two series of NDT tests indicated that there were discrepancies between the NDT-predicted lengths and design lengths of grout columns. Such discrepancies were found to be most common among the long soil nails. In order to verify that the actual grout column lengths matched the design lengths, a number of soil nails (Nails No. 19 through No.24) were exhumed. The findings revealed that the actual grout lengths agreed with the grout lengths predicted by NDT rather than with design lengths. For example, Soil nail No. 23 is designed as a 25-ft long nail with an intentional bird's mouth defect. NDT results predicted its length to be 14.2 ft, and the actual length determined by exhumation is 14 ft (See Figure 4.29). This discovery prompted the researchers to conduct a more complete evaluation of the construction quality, especially the actually grouted soil nail lengths. Based on this evaluation, it was concluded that use of the improper length of tremie pipe resulted in incomplete grouting of the holes. The length of tremie pipe used by the contractor was 12-ft and evidently, this length was not sufficient for grouting nails that were longer than 15ft. The problem of incomplete grouting, however, was confined to sand-cement grout and not the neat cement grout.

Figure 4.29 Incomplete Grouting of Test Nail No. 23

Due to the construction problems described above, Phase 1 construction did not provide an adequate number of "long nails" for validation of NDT test results. The nails affected were in 20-ft and 25-ft nail length categories. Therefore, a second phase of construction was undertaken to install 8 additional nails on the reserve panel. Figure 4.30 shows the soil nail layout used in Phase 2 construction on the reserve panel. Table 4.2 provides a summary of the parameter combinations used in Phase 2 wall design.

Table 4.2 Phase 2 Test Wall Design

	Grout Mixture Type	**Length of Nail**				
		5ft	**10ft**	**15ft**	**20ft**	**25ft**
Soil nail with no defect	Neat Cement			ID#27	ID#25	ID#28
Soil nail with no defect	Sand Cement				ID#26	ID#32
Defect at the middle of the soil	Neat Cement					ID#30
Defect at the middle of the soil	Sand Cement					ID#31
Defect at the end of the nail	Sand Cement					ID#29



Reserved Panel (Shaded Area)	Grout Mixture Type	Length of Nails				
		5'	10'	15'	20'	25'
Soil nail with no defect	Sand Cement Grout	ID#1	ID#4	ID#7	ID#13	ID#19
Soil nail with no defect	Neat Cement Grout	ID#2	ID#5	ID#8	ID#14	ID#20
Defect at the end of soil nail	Sand Cement Grout	ID#3	ID#6	ID#9	ID#15	ID#21
Defect at the middle of soil nail	Sand Cement Grout			ID#10	ID#16	ID#22
Bird mouth	Sand Cement Grout			ID#11	ID#17	ID#23
Defect at the middle of soil nail	Neat Cement Grout			ID#12	ID#18	ID#24
Soil nail with no defect	Neat Cement Grout			ID#27	ID#25	ID#28
Soil nail with no defect	Sand Cement Grout				ID#26	ID#32
Defect at the middle of soil nail	Neat Cement Grout					ID#30
Defect at the middle of soil nail	Sand Cement Grout					ID#31
Defect at the end of soil nail	Sand Cement Grout					ID#29

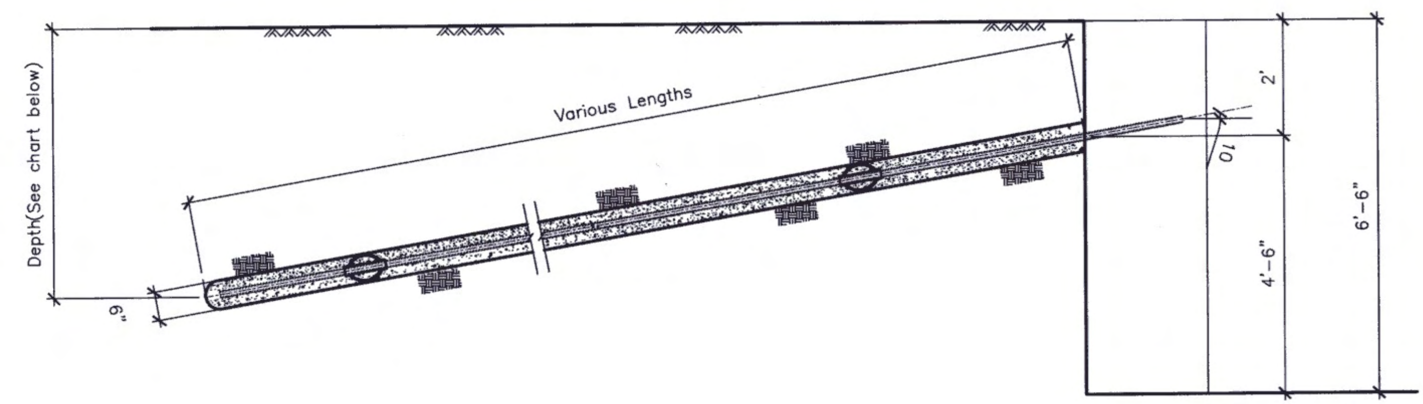
Figure 4.30 Reserve Panel in Phase 2 Construction

4.7 Field Construction; Phase 2

The second phase of construction involved the installation of 8 additional long nails (Nail Nos. 25 through 32) on the reserve panel. The specifications of the 8 additional nails were almost identical to those used in Phase 1 construction. Figures 4.31, 4.32 and 4.33 present the construction plans used in Phase 2 construction.

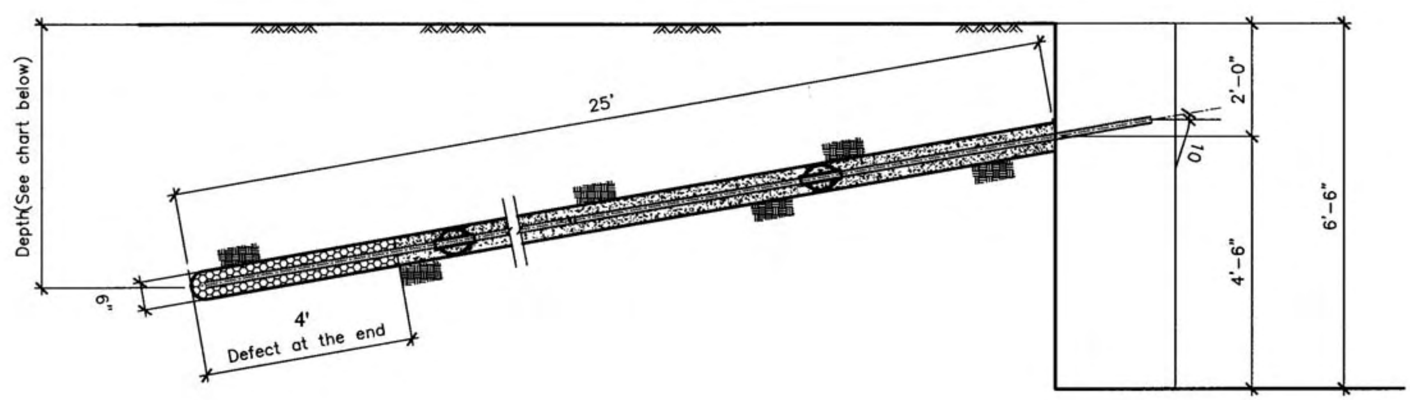
Phase 2 construction was performed on Jan 11, 2003. The drilling procedure followed the same guideline as the first construction phase. The grouting methods were modified in order to avoid the problems found in Phase 1construction. To ensure that complete grouting of the hole is achieved, 5 of the 8 holes were partially filled with grout before inserting the steel tendons. The remaining 3 nails had intentional defects at specified locations. For these nails, the contractor planned to use a 20 feet long tremie pipe. However, the tremie pipe diameter was found to be too small for pumping the stiff sand-cement grout. Accordingly, only Nail No. 29 was installed using this tremie pipe (Figure 5.40). Nail nos. 30 and 31 were installed by pumping grout from the top of the hole while continuously shaking the steel tendon to help the grout flow to the far end of the hole. Special care was taken so that each nail head will have a smooth, flat surface.

Figures 4.34 through 4.37 document the new grouting procedures used in Phase 2 construction.



	Length of nails	
	20'(ID#25-NC,ID#26-SC)	25'(ID#28-NC,ID#32-SC)
Depth	5'-5 1/2"	6'-4"

Figure 4.31 Soil Nails Without Defects in the Second Construction



	Length of nails
	25'(ID#29-SC)
Depth	6'-4"
L	5'-6"

Figure 4.32 Soil Nails with Defects at the End in the Second Construction

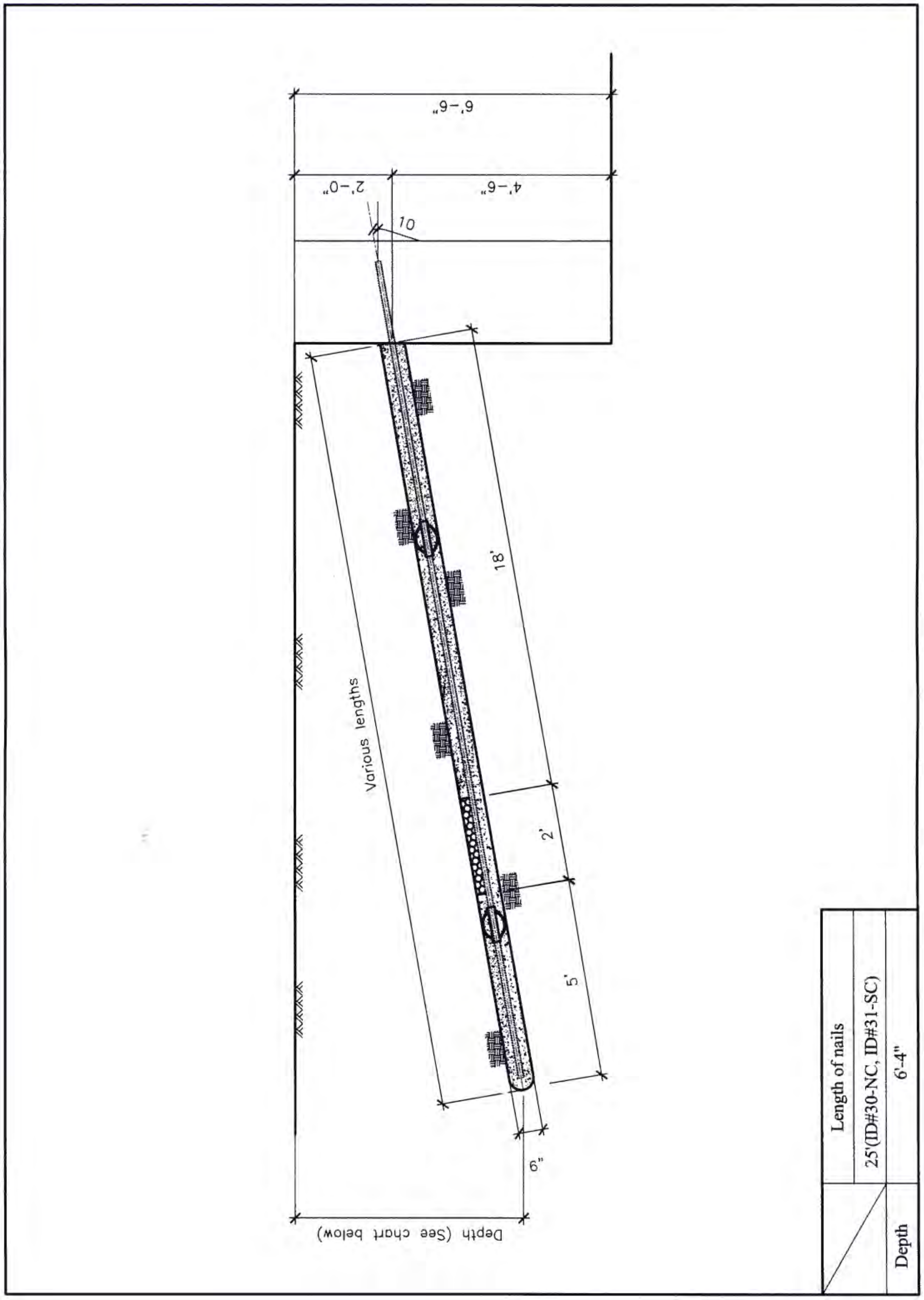


Figure 4.33 Soil Nails with Defects at the Middle in the Second Construction

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Figure 4.34 Grouting Drillholes before Inserting Steel Tendons

Figure 4.35 Inserting Steel Tendons after Partial Grouting of the Hole

Figure 4.36 The 20 feet long PVC Tremie Pipe Used in Phase 2

Figure 4.37 Grouting Nail No. 29 with the 20 feet Long Tremie Pipe

CHAPTER V

NON-DESTRUCTIVE TESTING OF SOIL NAILS AT THE TTU EXPERIMENTAL SITE

5.1 Overview

The field test program used in the evaluation of the selected NDT methods consisted of 3 separate cycles of testing. The first cycle of testing included all candidate NDT technologies that were selected based on the preliminary evaluation described in Chapter III. However, not all of these candidate NDT methods proved to be effective in the evaluation of installed soil nails. Therefore, the NDT methods that did not produce satisfactory results were removed from further consideration after the first round of NDT tests. Subsequent tests focused on the remaining candidate NDT methods. During each cycle of testing specific limitations were identified in each of the selected NDT methods. Necessary improvements and refinements were then made in hardware used as well as in the data processing techniques before the next round of testing was undertaken. This chapter describes the field non destructive test procedures used and the results obtained.

5.2 Description of Field Testing Plan

As mentioned in Chapter III, six NDT methods including Sonic Echo, Impulse Response, Impact Echo, Parallel Seismic, Cross-hole Seismic and Time Domain Reflectometry were selected as candidate methods for evaluating the integrity of installed soil nails. All of these six methods were evaluated during the first series of NDT investigations. The tests done during the first cycle showed that, in some soil nails the nail head condition was less than satisfactory and therefore, optimum results from NDT tests could not be obtained. Therefore, these nail heads were regrouted and the second series of NDT testing conducted using only those methods that proved to be most effective in the first round. The third series of NDT investigations were implemented in order to validate a new approach of transducer attachment and to determine the benefits from the use of new and improved test equipment. Upon the completion of three series of NDT investigations, all the installed soil nails were exhumed so that actual grout conditions of the nails could be directly observed and documented. The actual conditions were then compared with NDT predictions to determine the reliability of NDT predictions. Figure 5.1 summarizes the overall plan used in field NDT testing.

5.3 Experimental Setup for Non-Destructive Testing

As mentioned above, six NDT methods including Sonic Echo, Impulse Response, Impact Echo, Parallel Seismic, Cross-hole Seismic and Time Domain Reflectometry were evaluated at the TTU campus site. In addition, Ultrasonic Pulse Velocity test was used to determine pulse velocities for the grout cylinders. The experimental setups used in some of the test methods were very similar. For example, Sonic Echo, Impulse Response, and Impact Echo methods shared almost the same experimental setup except that Impulse Response uses a larger sledge hammer for generating low frequencies. For this reason, the following description of experimental setups is organized into 5 separate groups.

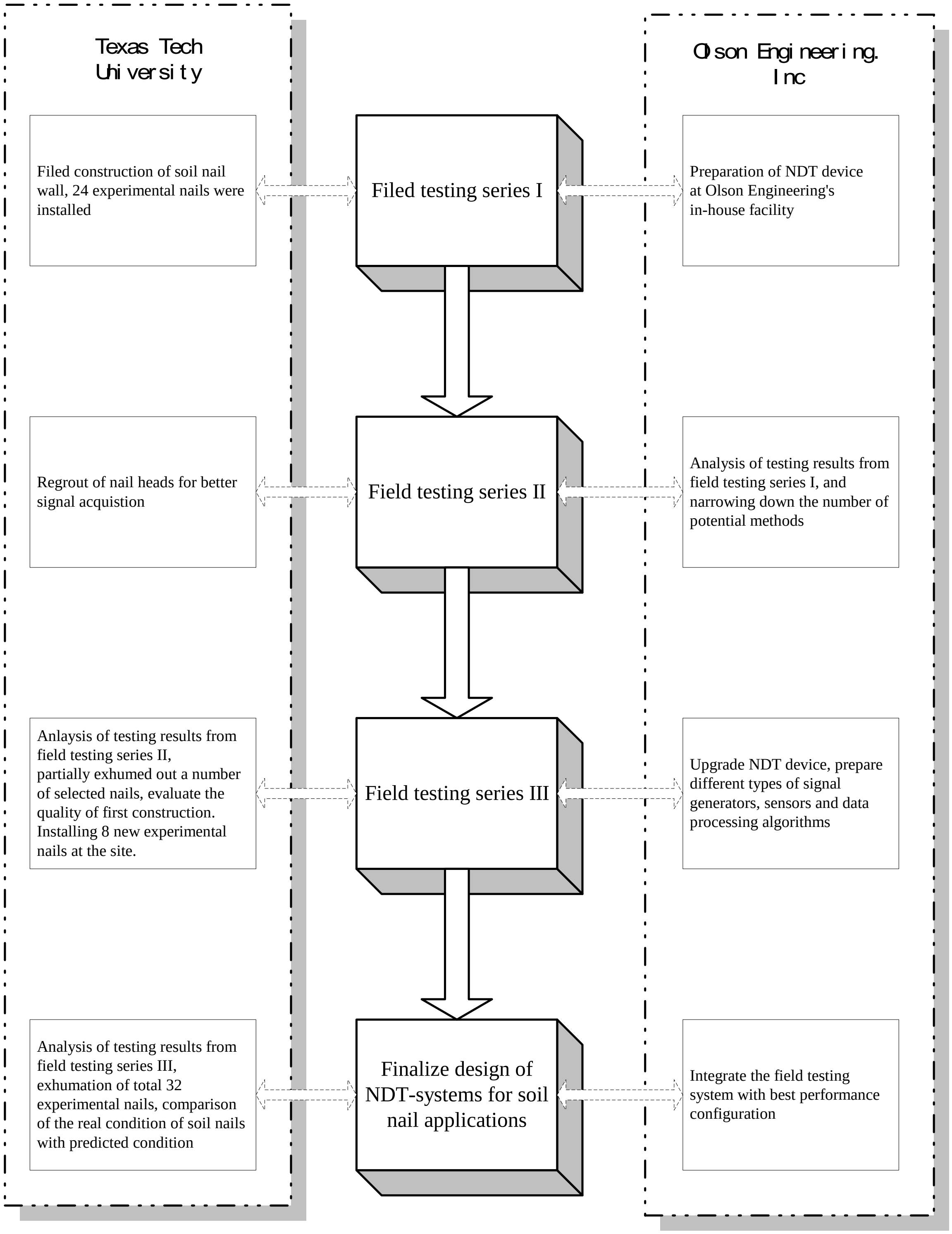


Figure 5.1 NDT Field Testing Plan

5.3.1 Experimental Setup for Sonic Echo, Impulse Response and Impact Echo Testing

The hardware system of Sonic Echo, Impulse Response and Impact Echo consists of an impacting device, a signal detector and data collection equipment. In this research study, modulated PCB hammers were used as the impacting device (See Figure 5.2). The small hammer with two different types of tips, aluminum and plastic, was used for Sonic Echo and Impact Echo methods. The two different tips produce stress waves with slightly different frequencies. When compared with the plastic tip, the alumina tip generates higher frequency vibrations that yield lower penetration depth but greater sensitivity to minor defects. Accelerometers were used to record the surface response (Figure 5.3). The acceleration is then integrated to produce the velocity history. In this manner part of the noise found in the signal can be filtered out. This provides a better quality data than that obtained with geophones which record surface velocity directly. A rugged Freedom Data PC along with a data collection and analysis software was used for data aquisition (Figure 5.4). Typically, the Freedom Data PC includes multiple channels so that it may collect data from multiple sensors at the same time. In addition, the Freedom Data PC can be used to perform different NDT methods including Sonic Echo, Impulse Response, Impact Echo, Parallel Seismic, Crosshole Seismic and Ultrasonic Pulse Velocity. The software used in this NDT program is WinTFS. This data acquisition program can perform data analysis in both frequency and time domains.

Figure 5.2 Modulated PCB Hammers with Alumina and Plastic Tips

The test configurations used in Sonic Echo and Impact Echo testing were varied based on four different variables. These variables were: type of accelerometer, position of accelerometer, position of impact, and type of hammer tips (Table 5.1). The various combinations of these variables gave a total of eight test configurations in the first and second series testing and sixteen test configuration combinations in the third series testing.

Impulse response test has only two combinations in the first and second series of testing and three in the third series of testing because the large hammer does not have multiple tips and it could only be used to hit on the grout. Figures 5.5 through 5.8 show the details of these different test configurations.

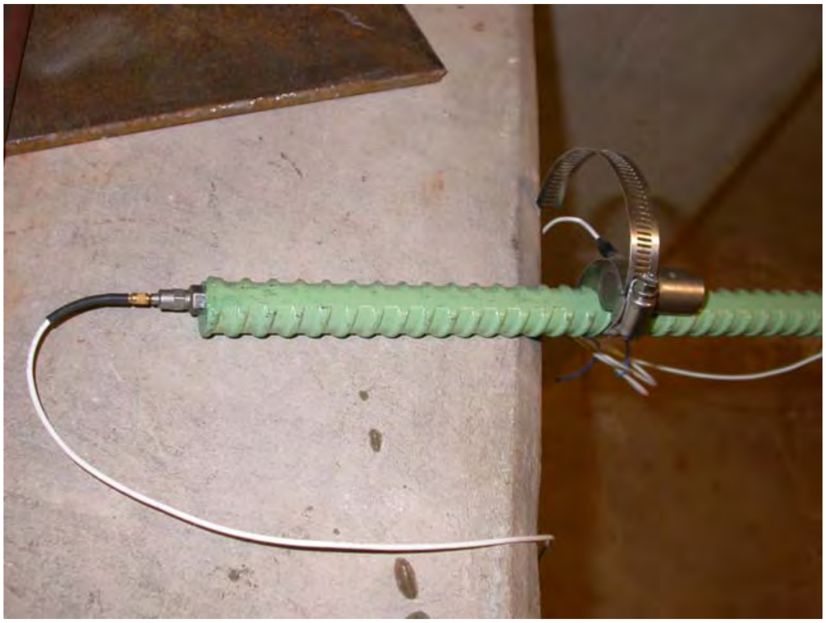


Figure 5.3 An Accelerometer being Used for Receiving Feedback Signal

Figure 5.4   Freedom Data PC Used for Data Acquisition

Table 5.1 Test Configuration Variables

Variable	Available Options	
Position of accelerometers	On the grout surface	On the steel tendon
Position of impact	On the grout surface	On the steel tendon
Type of hammer tips	Aluminum	Plastic
Type of accelerometers (only available in third series of testing)	Normal frequency range	Low frequency preferable

Figure 5.5 Freedom Data PC Connected to Sensors and Impactor

The contact between accelerometers and test object surface is vital to the quality of data acquisition. Therefore, a steel washer was mounted on the front face of the grout column and steel tendon and then the sensor was attached to the washer. In the first and second series of testing, grease was used as the coupling material between the washers and the grout or steel (Figure 5.9).   In the third series, epoxy glue was used instead of grease (Figure 5.10). This provided better test results.

Figure 5.6 Sonic Echo and Impact Echo Test Configuration: Accelerometer Mounted on the Grout, Impact on the Steel Tendon with the Alumina Tip

Figure 5.7 Sonic Echo and Impact Echo Test Configuration: Accelerometer Mounted on the Grout, Impact on the Grout with the Alumina Tip

Figure 5.8 Sonic Echo and Impact Echo Test Configuration: Accelerometer Mounted on the Steel Tendon, Impact on the Steel Tendon with the Plastic Tip

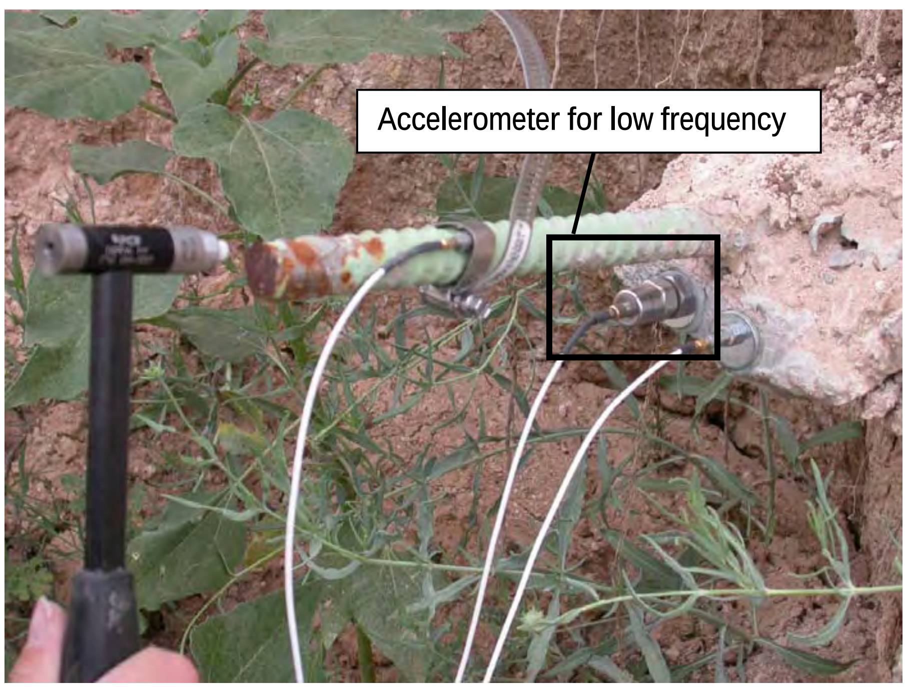


Figure 5.9 Sonic Echo and Impact Echo Test Configuration: Accelerometers Mounted on the Steel Tendon and the Grout, Impact on the Steel Tendon with the Plastic Tip

Figure 5.10 Impulse Response Configuration: Accelerometers Mounted on the Grout and the Steel Tendon, Impact on the Grout

Figure 5.11 Grease was Used to Glue the Washer with the Grout

Figure 5.12 Epoxy Glue was Used to Glue the Washer with the Grout

5.3.2 Experimental Setup for Ultrasonic Pulse Velocity

The Ultrasonic Pulse Velocity method was not used in the field testing for defect detection of installed soiled nails. Instead, this test was conducted in the lab for calculating the stress wave velocity in the grout and steel tendon. The test was performed on the grout cylinders that were sampled from the grout batches used for grouting field experimental nails (Figure 5.11).

Figure 5.13 Grout Cylinder Used for Wave Velocity Calculation

The hardware system of Ultrasonic Pulse Velocity in this research consisted of a Freedom Data PC, two types of transducers and a calibration sample. As shown in Figure 5.12, the larger transducers operate at 54 KHz while the smaller ones operate at 150 KHz transducer. The basic test configuration is that a transmitting transducer is positioned on one face of the test object and a second, receiving transducer is positioned on the opposite face (Figure 5.13).

Figure 5.14 Two Types of Transducers

Figure 5.15 Ultrasonic Pulse Velocity Testing Configuration

5.3.3 Experimental Setup for Parallel Seismic and Cross-hole Seismic Tests

To perform Parallel and Cross-hole Seismic tests, three 30 foot long holes were drilled parallel to the test soil nails. Three PVC pipes were inserted into the boreholes, and annulus grouted with bentonite. The pipes were then filled with water. These PVC tubes served as access tubes for instrumentation used in the Parallel Seismic method (Figures 5.14 and 5.15). The three boreholes were also deployed 12 feet apart from each other for the Cross-hole Seismic method.

Figure 5.16 Three Boreholes Embedded with PVC Pipes

Figure 5.17 The PVC Pipe was Filled with Water

The hardware systems of Parallel Seismic and Cross-hole Seismic methods share some common elements. In Parallel Seismic test, the impact is generated by the modulated sledge hammer and the wave response is recorded by the sensor which is placed within the adjacent PVC pipe. The sensor is moved from the bottom of the bore hole to the top of bore hole at a rate of 3 feet up per hammer hit (Figure 5.16); while in Cross-hole Seismic test the wave is generated by DS-V downhole sources. The sensors in either method are 3-component Geophones. Since the Cross-hole Seismic method requires more detection sensors, it uses more channels than the Parallel Seismic methods. A typical hardware system for both methods is shown in Figure 5.17. The actual system used in this research is shown in Figure 5.18.

Figure 5.18 Parallel Seismic Test Configuration

Figure 5.19 Hardware System for Parallel Seismic and Cross-hole Seismic Methods

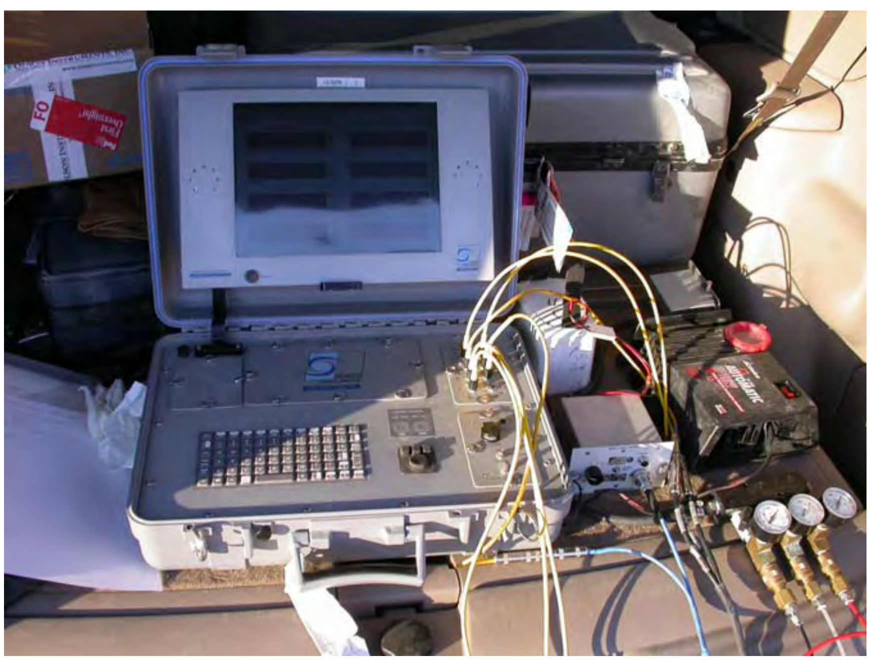


Figure 5.20 The Hardware System Used in Field Testing

5.3.4 Experimental Setup for Time Domain Reflectometry

Time Domain Reflectometry system used in the research consisted of a rugged field TDR unit and two cables, one used for sending and the other for receiving electrical current. Before attaching the clamps of cable on the steel tendon, a hand grinder was used to remove part of the epoxy on the steel tendon for better electricity contact (Figure 5.19). The test device and configuration are shown in Figures 5.20 and 5.21.

Figure 5.21 Grinding the Steel Tendon to Achieve Electrical Contact

Figure 5.22 Attaching Cable on Steel Tendon

Figure 5.23 Rugged Field TDR System

5.4 Exhuming Soil Nails

Upon completion of all 3 cycles of field NDT evaluation, the test nails were exhumed to observe actual grout condition. Initial excavation was done using a backhoe. The steps used in the exhumation are as follows: use the backhoe to remove the soil from the top of nails (Figure 5.22); remove the soil in between nails (Figures 5.23 and 5.24); use hand shoveling to remove the remaining soil (Figure 5.25); document the actual condition of exposed nails (Figure 5.26).

Figure 5.24 Remove Soil from the Top of Nails

Figure 5.25 Remove Soil from the Sides of Nails

Figure 5.26 Backhoe Excavation Closer to the Nails

Figure 5.27 Using Hand Shovel for Final Cleanup

Figure 5.28 Measuring and Recording Actual Nail Grout Condition

5.5 Results from Non-Destructive Testing in the Laboratory

There were several reasons for conducting laboratory tests prior to the beginning of field NDT testing. They were as follows: (a) calibrating the test device, (b) obtaining necessary reference data for subsequent field testing, and (c) measuring the wave velocity in sand cement, neat cement and steel. These wave velocities were used later when calculating defect depth and grout length. Table 5.2 summarizes the pulse velocity results from UPV tests conducted on the grout cylinders.

Table 5.2 Summary of UPV Test on Grout Cylinders

Grout cylinder	Condition	Height (inch)	Wave travel time (µS)	Velocity (feet/sec)
1	Dry neat cement	7.625	49.5	12836
2	Dry sand cement	7.813	46.3	14091
3	Wet neat cement	7.475	48.5	12843
4	Wet neat cement	7.625	49.5	12843
5	Wet neat cement	7.375	48.2	12750
6	Wet sand cement	7.750	47.0	13741
7	Wet sand cement	7.750	46.2	13979
8	Wet sand cement	7.750	46.7	13829

The average wave velocity in grout was calculated by averaging the above eight wave velocities. This yielded a wave velocity of 13,364 feet/sec for the grout. In addition, a number of Sonic Echo tests were performed on a standalone steel tendon with known length to calculate the wave velocity in steel tendon (Figure 5.28). The result of tests indicated the wave propagation velocity in the steel tendon is approximately 20,100 feet/sec. However, in an actual soil nail, the steel tendon is embedded in grout. Therefore, when an impact is applied at the head of a soil nail, either on the steel tendon or on the grout, stress waves propagate along the grout and steel tendon at the same time. Although stress waves travel faster in steel tendons than in the grout, when the grout and the tendon are bonded together well, the stress waves travel through the soil nail as a composite. A theoretical composite wave can be calculated based on an average steel and grout velocity using correct volume of grout and tendons. However, the theoretical calculation was not performed in this case. Instead, the composite velocity was obtained from a direct calibration on an actual soil nail (Nail 1) with an assumption that Nail 1 was in sound condition. The calibration yielded a composite wave velocity of 16,000 – 16,500 ft/sec.

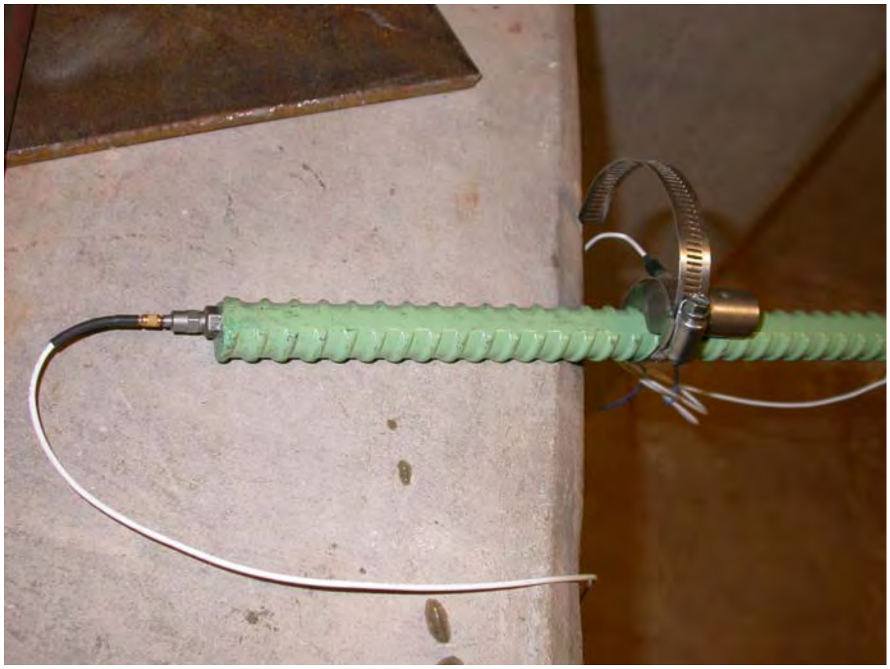


Figure 5.29 Laboratory Sonic Echo Tests on Standalone Steel Tendon

5.6 Results from Non-Destructive Testing in the Field

The detailed results from NDT testing conducted on each of the 32 test soil nails are presented in Appendix A. Appendix A also provides information on the comparison between the nail condition predicted based on NDT results and the actual nail condition. This comparison is used as the basis for evaluating the effectiveness of different NDT methods. The following sub-sections discuss the findings from each of the six NDT methods.

5.6.1 Results from Sonic Echo Method

The Sonic Echo (SE) test was performed on all 32 experimental nails. The comparison between predicted and actual nail conditions show that, out of all NDT techniques examined, the Sonic Echo test is the most effective and reliable method for evaluating length and grout integrity. Figures 5.29 through 5.52 present examples of Sonic Echo results for several selected soil nails. The results obtained for the remaining soil nails are found in Appendix A.

Nail No. 26 was designed as a defect-free soil nail with a length of 20 ft. The SE results obtained from the 3rd series of test for Nail No. 26 is presented in Figure 5.29. Figure 5.29 shows how the strength of the return signal varies with time. The Y-axis on this plot represents the amplitude of the return signal measured in volts by the accelerometer. The X-axis is the time measured in micro-seconds. The first sharp "dip" in the curve appearing at approximately 3000μs represents the initial impact from the hammer. The second (and more gentle) dip represents the first reflected wave arriving at the accelerometer. The vertical line shown is a cursor that can be moved to any desired location so that X-Y coordinates can be read accurately. In the example shown, the time of arrival of the first reflected wave is read by the cursor as 5.48ms (or 5,480 μs). The time interval between the two dips in the curve can be used to calculate travel distance when the composite wave velocity is known. The second dip seen in Figure 5.29 is referred to as a "neck echo." This type of echo indicates either a decrease in nail diameter (or defect) or an end of the nail. Using a composite velocity of 16,000 ft/sec, the distance to this discontinuity is calculated as 20.4ft. The actual observations of the exhumed nail (Figures 5.30 and 5.31) show that the nail is sound (constructed as planned) with a length of 21.4 ft. Thus, the results from SE show good agreement (within a foot) with the actual nail condition.

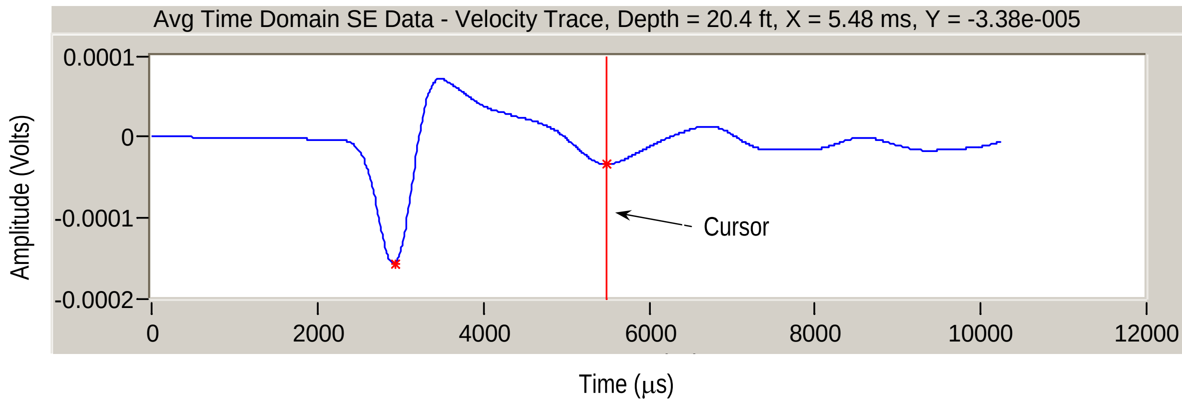


Figure 5.30 Sonic Echo Test Result from Test Nail No. 26

Figure 5.31 Exhumed Nail No.26

Figure 5.32 Grouted Length of Nail No.26 = 21 ft 5in

The SE results obtained for Nail No. 27 are presented in Figures 5.32(a) and 5.32(b). In these examples, the signal amplitude has been multiplied by an amplification factor that varies in an exponential manner. In other words, the tail end of the signal has received much higher amplification than its front end. This is done so that the echo can be seen better. The second curve shown on the plot represents the above amplification function. Review of Figure 5.32(a) shows a bulb echo at 17.8 ft and Figure 5.32(b) shows a neck echo at 26 ft. Once again these lengths are calculated using the average composite wave velocity of 16,000 ft/sec. A bulb echo indicates an increase in the diameter of the nail. Nail No. 27 was also designed as a defect-free nails with a length of 20 ft. However, the exhumation records (Figure 5.33) show that Nail No. 27 joined with Nail No. 28 at a length of 18.3 ft and the grout ended at 27 ft. The SE results show good agreement with the actual condition of the nail within one foot accuracy.

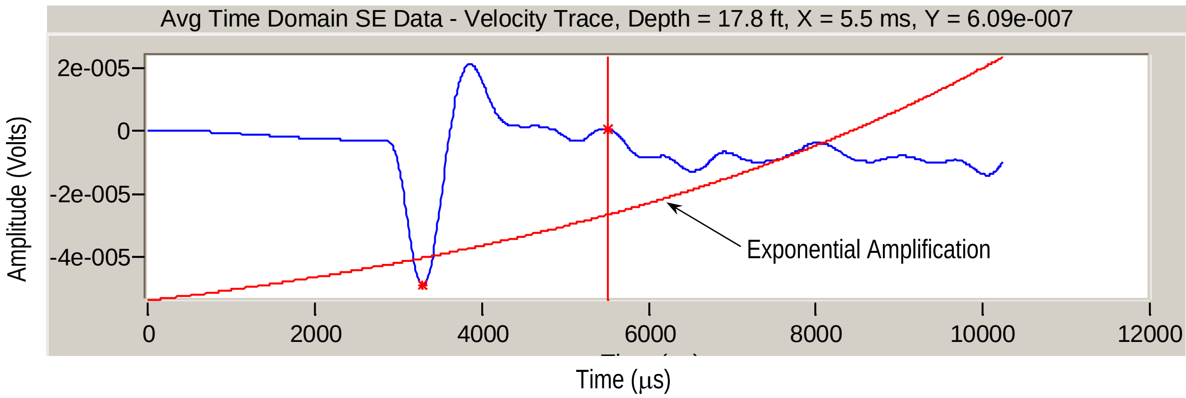


Figure 5.33a - Sonic Echo Test Result for Nail No. 27 Showing a Bulb Echo (an Increase in Cross-Section) at a Length of 17.8 ft

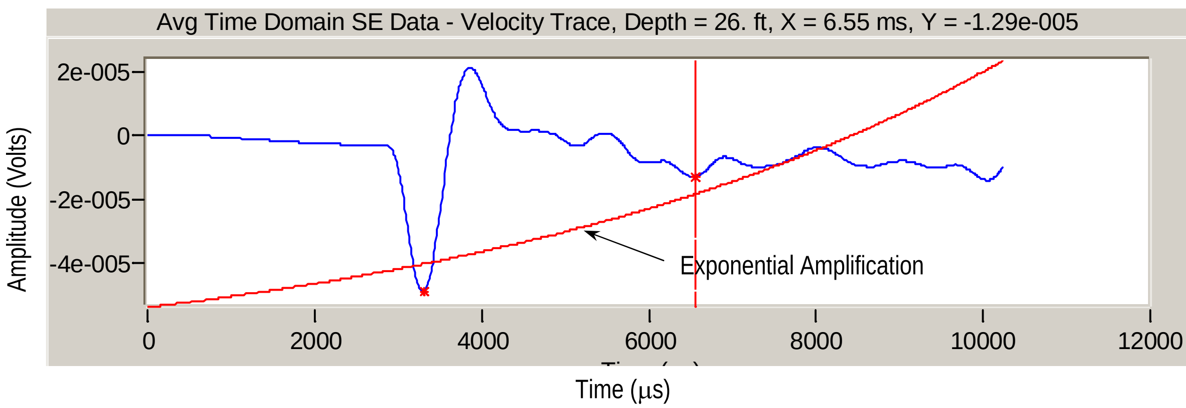


Figure 5.33b - Sonic Echo Test Result for Nail No. 27 Showing a Neck Echo (I.E. Reduction in Cross-Section) at a Length of 26 ft

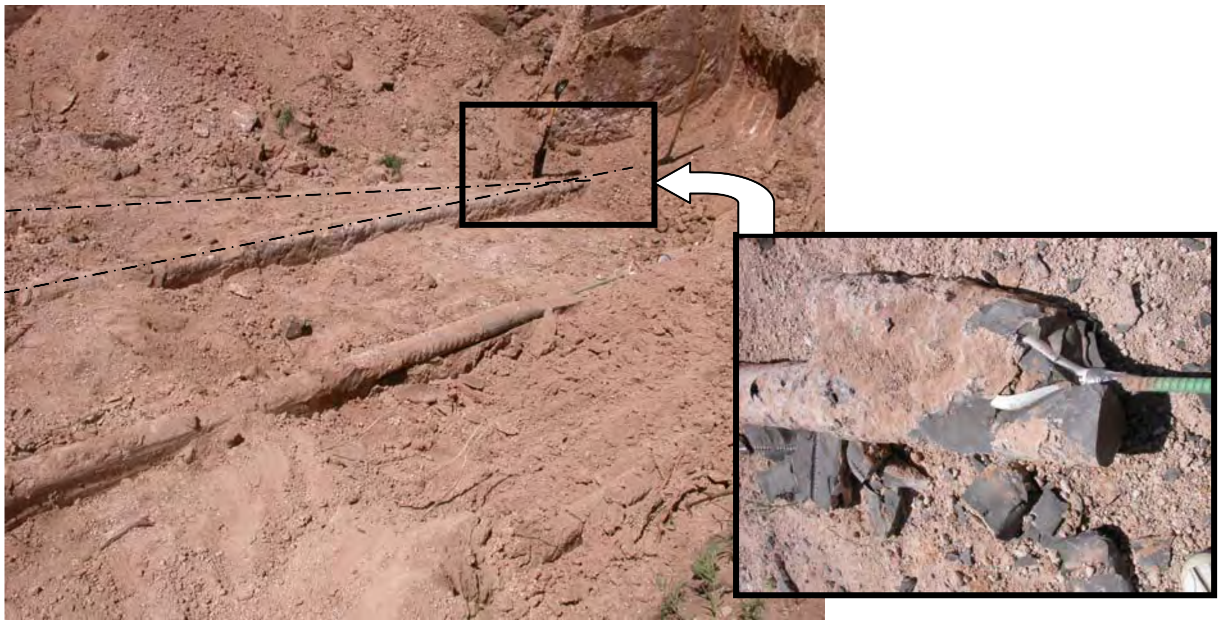


Figure 5.34 - Nail #27 and Nail #28 Join Each Other at 18ft 4in

Figure 5.34 shows the SE result for Nail No. 29. Review of Figure 5.34 shows double neck echoes at a length of 15.2 ft. Nail No. 29 was designed as a 25-ft long nail that has an end defect at 20 ft. The measurements made after exhuming the nail (Figures 5.35 and 5.36) confirmed that the grout ended at a length of 15.3 ft as predicted by the SE method. Accordingly, the actual nail condition was different from the designed conditions and the SE testing was able to detect this change and predict nail length within 0.1ft.

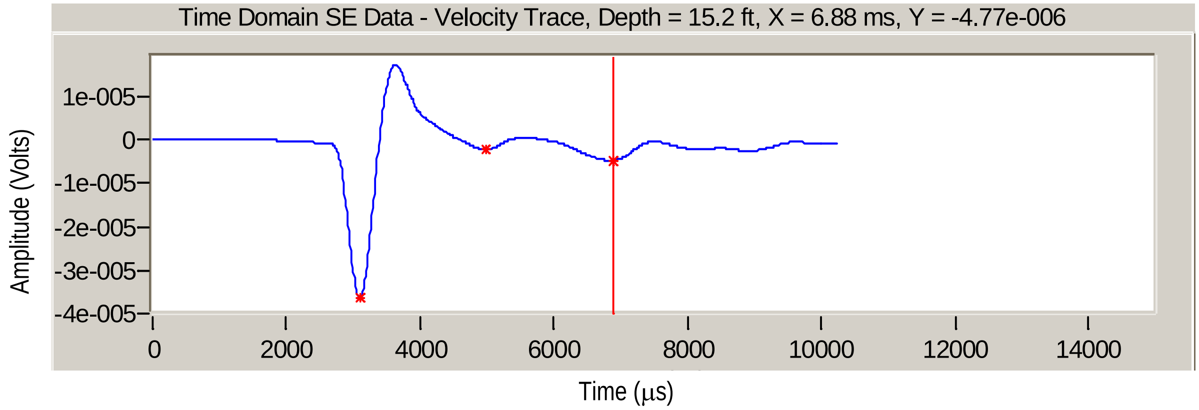


Figure 5.35 Sonic Echo Test Result for Nail No. 29 Showing Two Neck Echoes (Double Echoes) at a Length of 15.2 ft

Figure 5.36 Exposed Nail #29 (Grout Did Not Reach Foam Obstruction)

Figure 5.37 Nail 29 - Grout Stops at 15.3ft

The SE result from Nail No. 7 (from the third testing series of) is presented in Figure 5.37. The Figure shows a neck echo indicating a decrease in the diameter of the nail at a length of 10.4 ft. The nail was designed to be used as one of the sound nails with a length of 15 ft. However, the exhumation records (Figures 5.38 and 5.39) show that the grout tapered off at a length of 10.0 ft and ended at a length of 13.6 ft. Once again, in this case also the SE test predicted the location of grout reduction accurately.

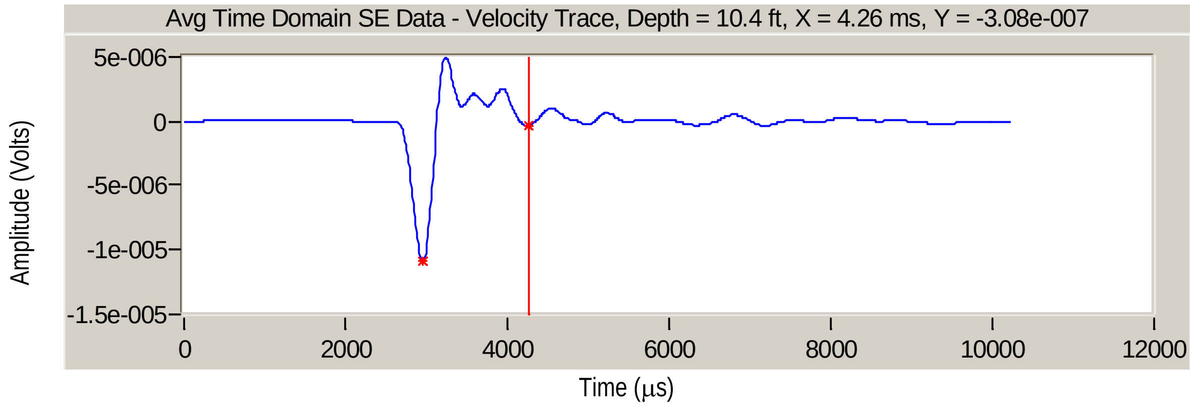


Figure 5.38 Sonic Echo Test Results for Nail No.7 (Shows a Neck Echo at a Length of 10.4 ft)

Figure 5.39 Nail No. 7 – Grout Ends at 13.6ft

Figure 5.40 Nail No. 7 – Grout Tapers off at a Length of 10ft

Figures 5.40a and 5.40b display the Sonic Echo test results of Nail No.16 from the third series of testing. According to construction plan, the nail should have foam defect at 8' and the grout should pass underneath the foam to the maximum length of 20 feet. Sonic Echo tests indicate that defects are detected at 8.7 ft and 15.1 ft. Exhumation records (Figures 5.41 and 5.42) show the grout passed underneath the middle foam and ends at 15.6 ft. This agrees well with the prediction from the Sonic Echo test. The construction error could be considered as a typical construction problem "defect at the middle of nails resulting from incomplete grouting".

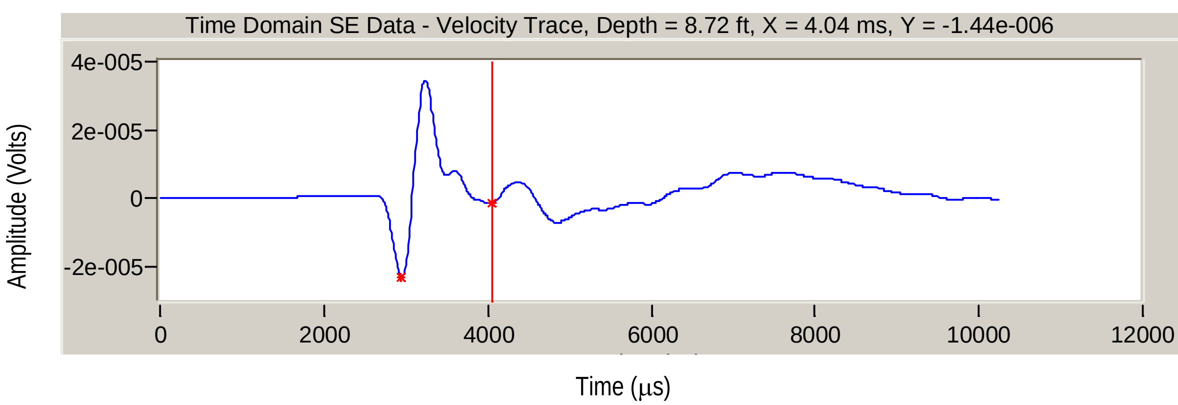


Figure 5.41a Sonic Echo Test Results from Nail No.16 (Shows a Neck Echo Indicating a Decrease in Diameter at a Length of 8.7 ft)

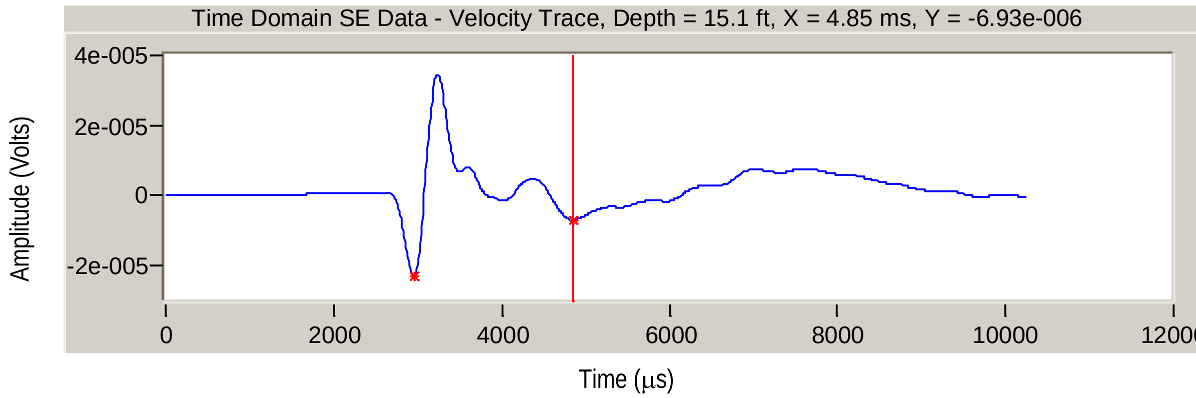


Figure 5.41b Sonic Echo Test Results from Nail No.16 (Shows a Neck Echo Indicating that Grout Ends at a Length of 15.1ft)

Figure 5.42 Exhumed Nail No.16

Figure 5.43 Nail No.16 Grout Ends at 15ft 8in

Figure 5.44 Nail No.16 Grout Meets the Foam at 8 feet and Passes Underneath

Figure 5.45 Shallow Defects Found in Nail No.16

The test results of Nail No.16 show that Sonic Echo can detect multiple defects, especially when the first defect is not severe enough to block wave propagation. However, it can be noted that the Sonic Echo method tends to overlook shallow defects when the defects are minor. Tables 5.3, 5.4 and 5.5 provide summaries of results obtained from the Sonic Echo tests from Test Series 1, 2 and 3 respectively.

Table 5.3 Summary of Results from Sonic Echo Tests Conducted during Test Series No.1

Nail ID	**Designed condition**	**Actual condition**	**Sonic Echo prediction**
1	NL=5 ft, SC, no defect	As designed condition	Grout ends at 5 ft
2	NL=5 ft, NC, no defect	Grout ends at 5.5 ft, bad head condition	N/A – Poor data quality
3	NL=5 ft, SC, defect @ end (from 2.5 ft)	As designed condition	Defect at 2.64 ft
4	NL=10 ft, SC, no defect	Grout ends at 10.1 ft, bad head condition	N/A – Poor data quality
5	NL=10 ft, NC, no defect	Grout ends at 10 ft, bad head condition, defects at 7 and 9 ft	Defects at 2.3 feet.
6	NL=10 ft, SC, defect @ end (from 7.8 ft)	Grout ends at 7 ft	N/A – Poor data quality
7	NL=15 ft, SC, no defect	Grout starts reducing from 10 ft and ends at 13.7 ft	Grout ends at 14 ft
8	NL=15 ft, NC, no defect	Defect at 3 and 4 ft, grout ends at 14.7 ft	Grout ends at 15.1 ft
9	NL=15 ft, SC, defect @ end (from 10.5 ft)	Grout ends at 10 ft, defects at 5.4 and 8.2 ft	Grout ends at 11.6 feet

Table 5.3 Summary of Results from Sonic Echo Tests Conducted during Test Series No.1 (continued from previous page)

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
10	NL=15 ft, SC, defect @ middle (from 6.2 to 8.2 ft)	Grout goes underneath middle defect, ends at 11.3 ft	Defect at 7.92 ft
11	NL=15 ft, SC, bird's mouth	Bird's mouth, defect at 11.2 ft, grout ends at 14.4 ft	N/A – Poor data quality
12	NL=15 ft, NC, defect @ middle (from 5.9 to 7.9 ft)	Grout goes underneath foam, ends at 15.2 ft, defects at 5.9, 8.7 and 11 ft.	Defect at 8.74 feet
13	NL=20 ft, SC, no defect	Grout starts reducing from 13 ft and ends at 14.75 ft, defect at 3.25-4.7 ft.	Defect at 13.1 ft
14	NL=20 ft, NC, no defect	Grout ends at 20.75 ft, multiple minor defects at 1.2, 2.4, 10.7, 11.7, 12.8, 13.7, 14.75 and 17.2 ft.	Defect at 17.3 ft
15	NL=20 ft, SC, defect @ end (from 14.5 ft)	As designed condition	N/A – Poor data quality
16	NL=20 ft, SC, defect @ middle (from 6.5 to 8.5 ft)	Middle defect starts from 8.5-11.5 ft. Grout passes underneath middle defect and ends at 15.7 ft. Shallow defect at 1 ft.	N/A – Poor data quality
17	NL=20 ft, SC, bird's mouth	Bird's mouth, grout ends at 13.8 ft	N/A - Poor data quality
18	NL=20 ft, NC, defect @ middle (from 6.0 to 8.0 ft)	Middle defect as designed condition; Minor defect at 12.4 ft; Grout passes underneath middle defect and ends at 20.3 ft.	Defect at 8 ft and
19	NL=25 ft, SC, no defect	Grout ends at 14.7 ft	Grout ends at 16.4 ft
20	NL=25 ft, NC, no defect	Grout ends at 28.7 ft, defect at 11.5 ft	N/A – Poor data quality due to bad access
21	NL=25 ft, SC, defect @ end (from 18.5 ft)	Grout ends at 10.6 ft	N/A – Poor data quality
22	NL=25 ft, SC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect starts from 10.5-12.5 ft. Grout ends at 10.5 ft.	N/A – Poor data quality
23	NL=25 ft, SC, bird's mouth	Bird's mouth, grout ends at 13 ft, defect at 2.3 ft.	N/A – Poor data quality
24	NL=25 ft, NC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect as designed condition; Minor defects at 8 and 16 ft; Grout passes underneath middle defect and ends at 27.6 ft.	N/A – Poor data quality

Table 5.4 Summary of Results from Sonic Echo Tests Conducted during Test Series No.2

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
1	NL=5 ft, SC, no defect	As designed condition	N/A – Poor data quality
2	NL=5 ft, NC, no defect	Grout ends at 5.5 ft, bad head condition	Grout ends at 7.6 ft
3	NL=5 ft, SC, defect @ end (from 2.5 ft)	As designed condition	Defect at 2.69 ft
4	NL=10 ft, SC, no defect	Grout ends at 10.1 ft, bad head condition	Grout ends at 9.8 ft
5	NL=10 ft, NC, no defect	Grout ends at 10 ft, bad head condition, defects at 7 and 9 ft	Defects at 7.9 feet.
6	NL=10 ft, SC, defect @ end (from 7.8 ft)	Grout ends at 7 ft	N/A – Poor data quality
7	NL=15 ft, SC, no defect	Grout starts reducing from 10 ft and ends at 13.7 ft	Grout ends at 12 ft
8	NL=15 ft, NC, no defect	Defect at 3 and 4 ft, grout ends at 14.7 ft	Grout ends at 15.7 ft
9	NL=15 ft, SC, defect @ end (from 10.5 ft)	Grout ends at 10 ft, defects at 5.4 and 8.2 ft	Grout ends at 10.5 feet
10	NL=15 ft, SC, defect @ middle (from 6.2 to 8.2 ft)	Grout goes underneath middle defect, ends at 11.3 ft	Defect at 8.56 ft
11	NL=15 ft, SC, bird’s mouth	Bird’s mouth, defect at 11.2 ft, grout ends at 14.4 ft	Defect at 3.48 ft
12	NL=15 ft, NC, defect @ middle (from 5.9 to 7.9 ft)	Grout goes underneath foam, ends at 15.2 ft, defects at 5.9, 8.7 and 11 ft.	Defect at 5.48 feet
13	NL=20 ft, SC, no defect	Grout starts reducing from 13 ft and ends at 14.75 ft, defect at 3.25-4.7 ft.	Defect at 2.08 ft
14	NL=20 ft, NC, no defect	Grout ends at 20.75 ft, multiple minor defects at 1.2, 2.4, 10.7, 11.7, 12.8, 13.7, 14.75 and 17.2 ft.	Defect at 10.5 ft
15	NL=20 ft, SC, defect @ end (from 14.5 ft)	As designed condition	Defect at 13 ft
16	NL=20 ft, SC, defect @ middle (from 6.5 to 8.5 ft)	Middle defect starts from 8.5-11.5 ft. Grout passes underneath middle defect and ends at 15.7 ft. Shallow defect at 1 ft.	Defect at 9.24 ft
17	NL=20 ft, SC, bird’s mouth	Bird’s mouth, grout ends at 13.8 ft	N/A - Poor data quality
18	NL=20 ft, NC, defect @ middle (from 6.0 to 8.0 ft)	Middle defect as designed condition; Minor defect at 12.4 ft; Grout passes underneath middle defect and ends at 20.3 ft.	Defect at 7 ft and
19	NL=25 ft, SC, no defect	Grout ends at 14.7 ft	N/A – No clear echo
20	NL=25 ft, NC, no defect	Grout ends at 28.7 ft, defect at 11.5 ft	N/A – Poor data quality due to bad access

Table 5.4 Summary of Results from Sonic Echo Tests Conducted during Test Series No.2 (continued from previous page)

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
21	NL=25 ft, SC, defect @ end (from 18.5 ft)	Grout ends at 10.6 ft	Defect at 8.36 ft
22	NL=25 ft, SC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect starts from 10.5-12.5 ft. Grout ends at 10.5 ft.	N/A – Poor data quality
23	NL=25 ft, SC, bird's mouth	Bird's mouth, grout ends at 13 ft, defect at 2.3 ft.	Defect at 4.47 ft
24	NL=25 ft, NC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect as designed condition; Minor defects at 8 and 16 ft; Grout passes underneath middle defect and ends at 27.6 ft.	Defect at 8.5 ft

Table 5.5 Summary of Results from Sonic Echo Tests Conducted during Test Series No.3

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
1	NL=5 ft, SC, no defect	As designed condition	N/A – Not tested
2	NL=5 ft, NC, no defect	Grout ends at 5.5 ft, bad head condition	Grout ends at 5.28 ft
3	NL=5 ft, SC, defect @ end (from 2.5 ft)	As designed condition	Defect at 2.8 ft
4	NL=10 ft, SC, no defect	Grout ends at 10.1 ft, bad head condition	Grout ends at 10.2 ft
5	NL=10 ft, NC, no defect	Grout ends at 10 ft, bad head condition, defects at 7 and 9 ft	Defect at 6.08 ft
6	NL=10 ft, SC, defect @ end (from 7.8 ft)	Grout ends at 7 ft	Grout ends at 7.28 ft
7	NL=15 ft, SC, no defect	Grout starts reducing from 10 ft and ends at 13.7 ft	Grout ends at 14 ft
8	NL=15 ft, NC, no defect	Defect at 3 and 4 ft, grout ends at 14.7 ft	Defect at 4 ft and Grout ends at 15.1 ft
9	NL=15 ft, SC, defect @ end (from 10.5 ft)	Grout ends at 10 ft, defects at 5.4 and 8.2 ft	Grout ends at 10.9 feet
10	NL=15 ft, SC, defect @ middle (from 6.2 to 8.2 ft)	Grout goes underneath middle defect, ends at 11.3 ft	Defect at 7.12 ft
11	NL=15 ft, SC, bird's mouth	Bird's mouth, defect at 11.2 ft, grout ends at 14.4 ft	Defect at 4.56
12	NL=15 ft, NC, defect @ middle (from 5.9 to 7.9 ft)	Grout goes underneath foam, ends at 15.2 ft, defects at 5.9, 8.7 and 11 ft.	Increase in diameter at 11.5 ft
13	NL=20 ft, SC, no defect	Grout starts reducing from 13 ft and ends at 14.75 ft, defect at 3.25-4.7 ft.	Defect at 2.96 ft
14	NL=20 ft, NC, no defect	Grout ends at 20.75 ft, multiple minor defects at 1.2, 2.4, 10.7, 11.7, 12.8, 13.7, 14.75 and 17.2 ft.	N/A – No clear echo

Table 5.5 Summary of Results from Sonic Echo Tests Conducted during Test Series No.3 (continued from previous page)

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
15	NL=20 ft, SC, defect @ end (from 14.5 ft)	As designed condition	N/A – No clear echo
16	NL=20 ft, SC, defect @ middle (from 6.5 to 8.5 ft)	Middle defect starts from 8.5-11.5 ft. Grout passes underneath middle defect and ends at 15.7 ft. Shallow defect at 1 ft.	Defect at 8.7 ft and grout ends at 15.1 ft
17	NL=20 ft, SC, bird's mouth	Bird's mouth, grout ends at 13.8 ft	N/A – No clear echo
18	NL=20 ft, NC, defect @ middle (from 6.0 to 8.0 ft)	Middle defect as designed condition; Minor defect at 12.4 ft; Grout passes underneath middle defect and ends at 20.3 ft.	Increase in diameter at 13 ft
19	NL=25 ft, SC, no defect	Grout ends at 14.7 ft	Increase in diameter at 14.2 ft
20	NL=25 ft, NC, no defect	Grout ends at 28.7 ft, defect at 11.5 ft	N/A – Not tested due to bad access
21	NL=25 ft, SC, defect @ end (from 18.5 ft)	Grout ends at 10.6 ft	Defect at 8.32 ft
22	NL=25 ft, SC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect starts from 10.5-12.5 ft. Grout ends at 10.5 ft.	Grout ends at 9.9 ft
23	NL=25 ft, SC, bird's mouth	Bird's mouth, grout ends at 13 ft, defect at 2.3 ft.	Defect at 4.8 ft
24	NL=25 ft, NC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect as designed condition; Minor defects at 8 and 16 ft; Grout passes underneath middle defect and ends at 27.6 ft.	Defect at 13.8 ft
25	NL=20 ft, NC, no defect	Grout ends at 20.6 ft, minor defect at 12 ft.	N/A – No clear echo
26	NL=20 ft, SC, no defect	As designed condition, grout ends at 20.4 ft	Grout ends at 20.4 ft.
27	NL=20 ft, NC, no defect	Grout joins with the grout of nail no 28 at 18.3 ft	Increase in diameter at 17.8 ft and grout ends at 26 ft
28	NL=25 ft, NC, no defect	Grout joins with the grout of nail no 27 at 18.3 ft. grout ends at 27 feet	SE tests detect defect at 17 feet. No end of grout detected.
29	NL=25 ft, SC, defect @ end (from 20 ft)	Grout stops at 15.3 ft.	Grout ends at 15.1 ft
30	NL=25 ft, NC, defect @ middle (from 17 to 19 ft)	Middle defect as designed condition; Minor defect at 3.7 and 15.4 ft; Grout passes underneath middle defect and ends at 26.4 ft	defect at 17 ft
31	NL=25 ft, SC, defect @ middle (from 17 to 19 ft)	Middle defect as designed condition; Lost most of grout from 13.75 to 15.75 ft; Nail was fully grouted from 0 to 13.75 ft and from 15.75 to 17 ft. Grout didn't pass the middle defect	Defect at 9.1 ft

Table 5.5 Summary of Results from Sonic Echo Tests Conducted during Test Series No.3 (continued from previous page)

Nail ID	Designed condition	Actual condition	Sonic Echo prediction
32	NL=25 ft, SC, no defect	For the whole length, only the following part was fully grouted:0-2.3ft, 8-9.75ft. And the nail lost 50% of cross-section at 9.75 ft and 75% of cross-section at 12.3 ft.	Increase in diameter at 6 ft

5.6.2 Maximum Detectable Nail Length

The nail lengths in this field experiment varied from 5 feet to 25 feet. This broad range of nail lengths was used so that the largest penetration depth of the selected NDT can be determined. Based on data collected in this study, the maximum nail length which could be detected by the Sonic Echo method is 26 feet. This length was recorded for Test Nail No.27. Figure 5.32 shows the Sonic Echo test results for Nail No.27. According to construction plans, the nail should have no defects and should end at 20 feet. In this case, the Sonic Echo tests indicated that a bulb or an increase in the diameter occurred at 17.8 feet and the grout ended at 26 feet. Exhumation records show the nail joined with the adjacent nail at 18.3 feet and ended at 27 feet confirming that the prediction from the Sonic Echo test were correct. Another good example for determining the maximum detectable length for the Sonic Echo test is the SE test data collected for Nail No.26. The SE data clearly showed that the grout ended at 20 feet.

However, in some cases the Sonic Echo test could not reach a depth of 20 feet. An example is Test Nail No.14. This nail was designed as a defect free nail with length of 20-ft. The SE test did not produce any clear echoes for this nail. The exhumed nail had multiple shallow defects but the grout ended at 20.8ft (see Table 5.5). This shows that the maximum detectable nail length depends on other factors such as the presence of minor defects. Table 5.6 summarizes the grout condition of Test Nails 14, 27 and 26. From this table, we can conclude that the most preferable conditions for Sonic Echo that would provide the largest depth of penetration are as follows:

(a) The tested soil nail is grouted with neat cement
(b) The tested soil nail has no shallow defect
(c) A flat and well grouted head surface is available
(d) Impact on the grout column, accelerometer mounted on the grout and either plastic or aluminum hammer tip can be selected as the test configuration

All of these features contribute to less wave attenuation, so the wave could propagate a longer distance. Neat cement is known to have more shrinkage than sand cement, which probably creates poor bonding between soil and grout, causing less wave attenuation.

Table 5.6 Comparison of SE Test Conditions for Nail Nos. 14, 27, and 26

	Nail No.14	**Nail No.27**	**Nail No.26**
Grout type	Neat cement	Neat cement	Sand cement
Severity of defects	Bad and multiple defects	Join with an adjacent nail	No defect
Location of end based on SE prediction	No end detected	26 feet	20 feet
SE Test configuration	Any configuration can not detect the end of grout	Impact on grout; small PCB hammer with plastic tip; accelerometer on grout	Impact on grout; small PCB hammer with both plastic and aluminum; accelerometer on grout

Defects are the targets that we seek to identify inside the test object. They actually cause multiple wave reflections. The wave is gradually attenuated along the wave reflection path giving the opportunity for defects to hide each other. In some cases, the wave is almost all blocked by defects (as in Nail No.14) and when this happens the wave carrying the information about the end of grout is too weak to be identified. A flat and well grouted head surface provides the most favorable environment for generating a solid and well-coordinated impact, producing a wave that is initiated at a proper energy level and at its shortest propagation path. Hitting the grout column using a hammer with a plastic tip generates a wave which has a relatively low frequency content. Stress waves with low frequency contents have a lower attenuation rate than high frequency waves, and therefore they have a larger propagation distance.

Based on the records from our field testing, we believe that under the condition of the testing nail having no defect Sonic Echo test could reliably detect the end of grout up to 26 feet. If any defect presents, the detectable length from Sonic Echo will likely decrease depending on the severity and number of defects.

5.6.3 Defect Sensitivity Studies

The sensitivity of the Sonic Echo test to grout column defects can be evaluated in two ways, based on type of defect and size of defects.

Review of information summarized in Tables 5.3 – 5.5, one can conclude that the Sonic Echo test does not always correctly identify shallow defects located at depths less than 5 feet. As an example, consider the SE data obtained during the Test Series No.1. The SE tests were able to identify shallow defects in Nail Nos. 3 and 5 but failed to detect shallow defects in Nail Nos. 8 and 13 (minor defects). The 2nd series of SE tests were able to detect shallow defects on Nail Nos. 3, 11 and 13, but failed to identify shallow defects on Nail No. 8. The 3rd series of SE tests were able to detect shallow defects on Nail Nos. 3, 8, 11 and 13; however the SE tests were unable to identify a bird's mouth condition on Nail No. 23.

Figure 5.46 Bird's Mouth Defects in Nails 2, 20 and 4 that Limit the Proper Access

In summary, Sonic Echo is not sensitive to shallow defects when the nail head conditions are poor. Therefore, the SE test is not suitable as a NDT method for detecting shallow defects especially bird's mouth type defects.

Defects at the middle and the end of soil nails are the other two major construction problems encountered in real projects. In this research study, six experimental nails had intentional "defects at the end of the nail", eight experimental nails had intentional "middle of the nail defects." Defects at the end of the nail represented 100% reduction in cross section while defects at the middle of soil nails represented a 50% cross-section area reduction in the grout column. In addition these "intentional defects," unplanned defects were discovered in some of the test nails after they were exhumed. Review of Table 5.3 reveals that, in Test Series No.1, the SE test was able to correctly identify the defect locations of Nails 1, 2, 3, 5, 7, 8, 9, 10, 13 and 18 within 1.0-ft accuracy. Review of Table 5.4 (Test Series No.2) shows that the SE test correctly identified the defect locations of Nails 3, 4, 5, 7, 9, 11, 12, 13, 14, 16 and 18 (within 1.0-ft accuracy). Review of Table 5.5 (Test Series No.3) shows that the SE test correctly identified the defect locations of Nails 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 13, 16, 22, 26, 27, 29 and 30. In summary, the Sonic Echo method proved to be very effective in detecting major defects in the soil nail provided that the nail head condition is satisfactory.

In addition to these planned defects, the SE test was able to detect major unplanned defects as well. Unplanned minor defects such as cracks, small voids (less than 20% area reduction) were observed in exhumed nails, and some of these defects could be also identified in the recorded waveforms. Figures 5.46 through 5.48 demonstrate the detection of a small void in Nail No.5. Figure 5.49 demonstrate the detection of crack and minor defects in Nail No.5. Although the presence of these defects may not impair nail serviceability, they can cause reflection of part of the wave energy and thereby reduce the effectiveness of the Sonic Echo test. However, based on the test results presented in Appendix A, it was found that different testing configurations, different positions of impact and accelerometer on grout can be used to detect different types of minor defects in the same nail. Accordingly, Sonic Echo would still be effective in detecting minor defects, especially when the test is launched with an impact containing high frequency contents provided that there are no major defects ahead of these minor defects.

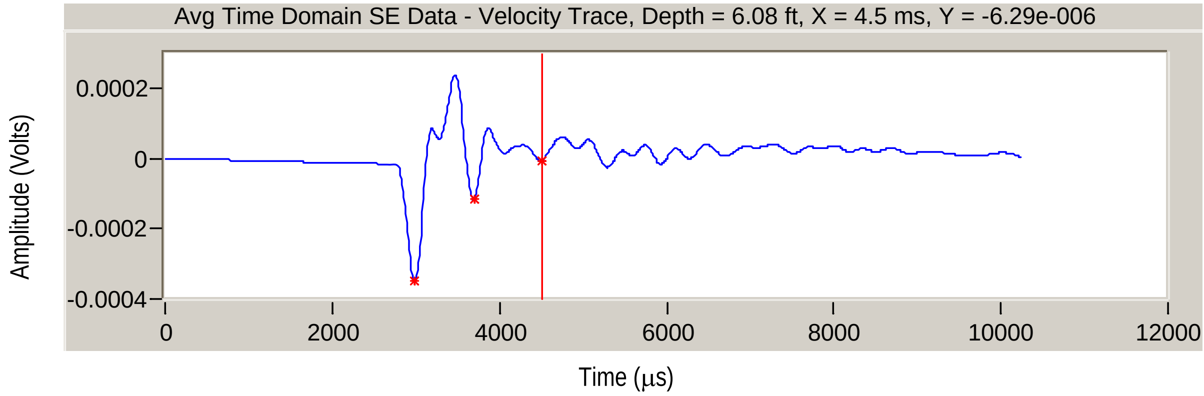


Figure 5.47 Sonic Echo Test results for Test Nail No.5

Figure 5.48 Exhumed Nail No.5

Figure 5.49 A minor defect on Nail 5 at 7 ft

Figure 5.50 A Crack on Nail No. 5 at 9ft

Figure 5.51 Nail No.5 Grout Column Ends at 10ft

5.6.4 Sonic Echo Test; Ease of Use and Testing Time

The Sonic Echo method is relatively simple to perform when compared with other more sophisticated stress wave methods such as Parallel Seismic and Cross-hole Seismic methods. However, the test operator must receive some training on the proper use of the equipment as well as on the interpretation of the data collected.

The time for testing a soil nail using Sonic Echo method varies from one nail to another. Generally, time required for testing a nail depends on the proficiency of the operator, number of test combinations for each nail, and number of channels available in the data acquisition system for simultaneous data collection. The average test duration for each nail was documented during the third series of testing and that information is summarized here to serve as a reference.   In the third series of testing, the Sonic Echo hardware system used three types of accelerometers and four channels, three for the accelerometers and one for the impact force. This means that, with each hammer hit, three test combinations can be done. The average time of data acquisition for each nail was about 7 minutes. Test preparation, i.e. mixing epoxy glue to attach washers on grout would require about 5 minutes. Accordingly, the total duration of Sonic Echo test in the third series of testing was about 12 minutes/per nail. Within this 12 minute time period, 16 test combinations were done and 6 data sets were acquired for each test combination.   This included a total of 96 effective data sets. Based on the above statistical information from the third series of testing, Sonic Echo is efficient in terms of data acquisition density and time spent per nail.

5.6.5 Sonic Echo; Equipment Cost

The system used for the Sonic Echo test included a Freedom Data PC for data acquisition, an accelerometer and a PCB instrumented hammer. The Freedom Data PC can be used to perform different NDT methods that utilize stress waves. The Freedom Data PC came with different plug-ins for different tests. The cost of the Freedom Data PC with a

Sonic Echo test module is around $17,000 per unit. This unit includes an accelerometer and a PCB hammer with different types of tips. However, a future dedicated system can also be manufactured for the SE tests on soil nails. It is estimated that this system will cost around $8,000 - $10,000 per unit.

5.6.6 Results from Impact Echo Method

Based on the field NDT tests conducted on the test soil nails, the Impact Echo was selected as the best candidate for detecting the shallow defects in soil nails. The shallow defects consisted of "bird's mouth (or bird's beak) defects" as well as some unplanned shallow defects that were discovered after the soil nails were exhumed. As explained previously, most of these shallow defects are not detected by Sonic Echo test because the Sonic Echo test had been fine tuned to enhance its capability to detect major defects that are found at greater depth. The de-noising techniques used in Sonic Echo test, (i.e. use of low pass filter) gets rid of early echoes arising from shallow defects. In the Impact Echo Method the entire response record (which in this study was a velocity response) is transformed into the frequency domain and the dominant frequency peak is identified. The echoes from the shallow defects always tend to be the strongest and most periodic, and can be best identified be identified by examining the frequency plots. This process is illustrated below using several examples.

Figure 5.51 shows Test Nail No.11 which was designed as a 15 feet long with a bird's mouth defect. The Impact Echo test results for this test nail are given in Figure 5.52. Accordingly, the Impact Echo tests predicted a shallow defect at 0.54-4.57ft. This observation matched with the actual observations.

Figure 5.52 Bird's Mouth Defect Observed in Test Nail No.11

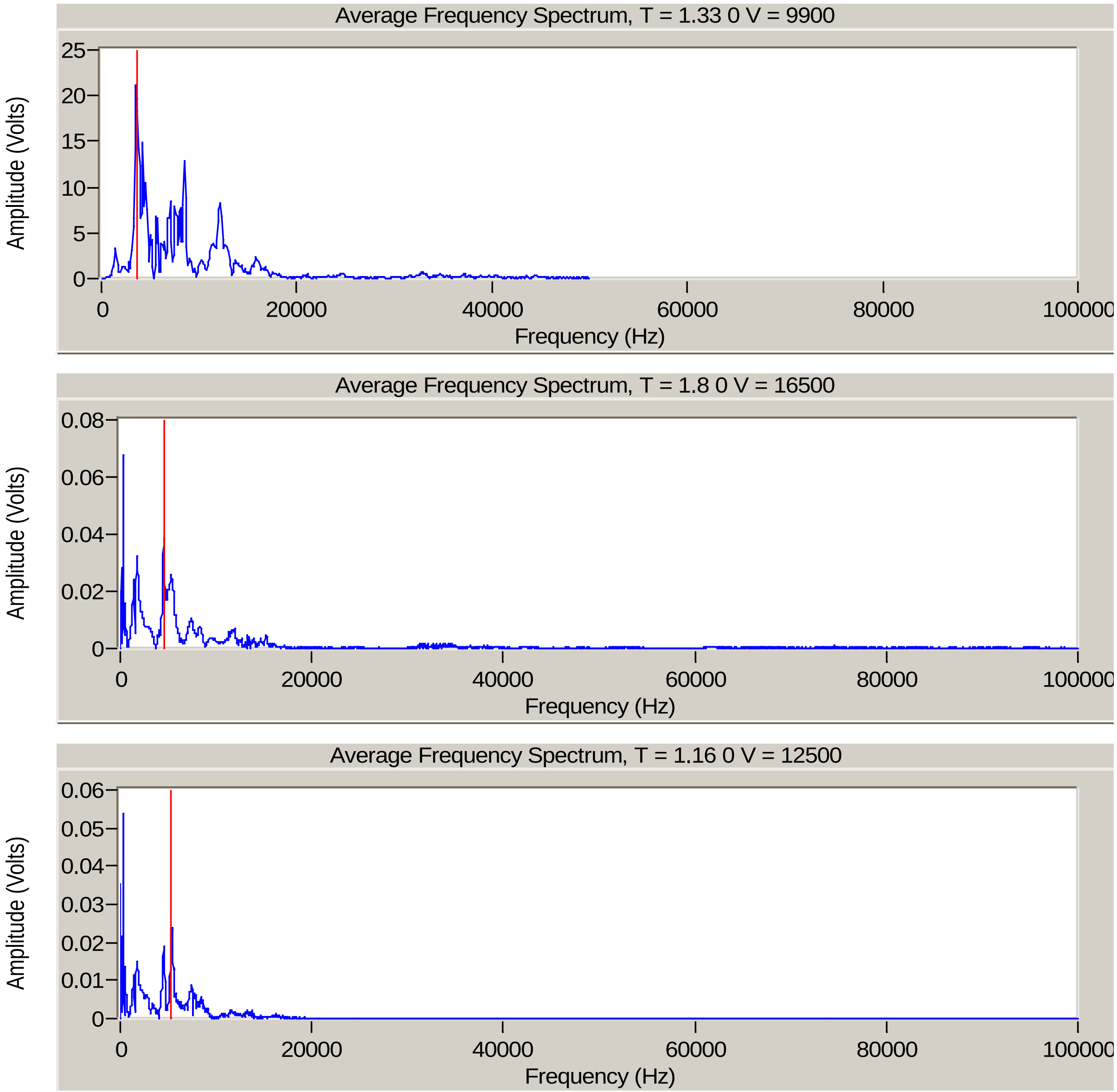


Figure 5.53 Impact Echo Test Results for Test Nail No.11

Similarly, Figure 5.53 displays the Impact Echo test results for Nail No.17. According to the construction plans, the nail should be 20 feet long with bird's mouth defect. Impact Echo tests detected shallow defects at 1.91-2.15ft which agreed with the bird's mouth defect observed after the nail had been exhumed (See Figure 5.54).

Figure 5.55 represents the Impact Echo test results for Nail No.23. This nail was designed as a 25 feet long with bird's mouth defect. The Impact Echo tests detected shallow defects at 0.982-2.54. This prediction matched the bird's mouth defect observed in nail exhumation (Figures 5.56). Table 5.5 is a summary of results from Impact Echo tests performed on 32 experimental nails at TTU campus site.

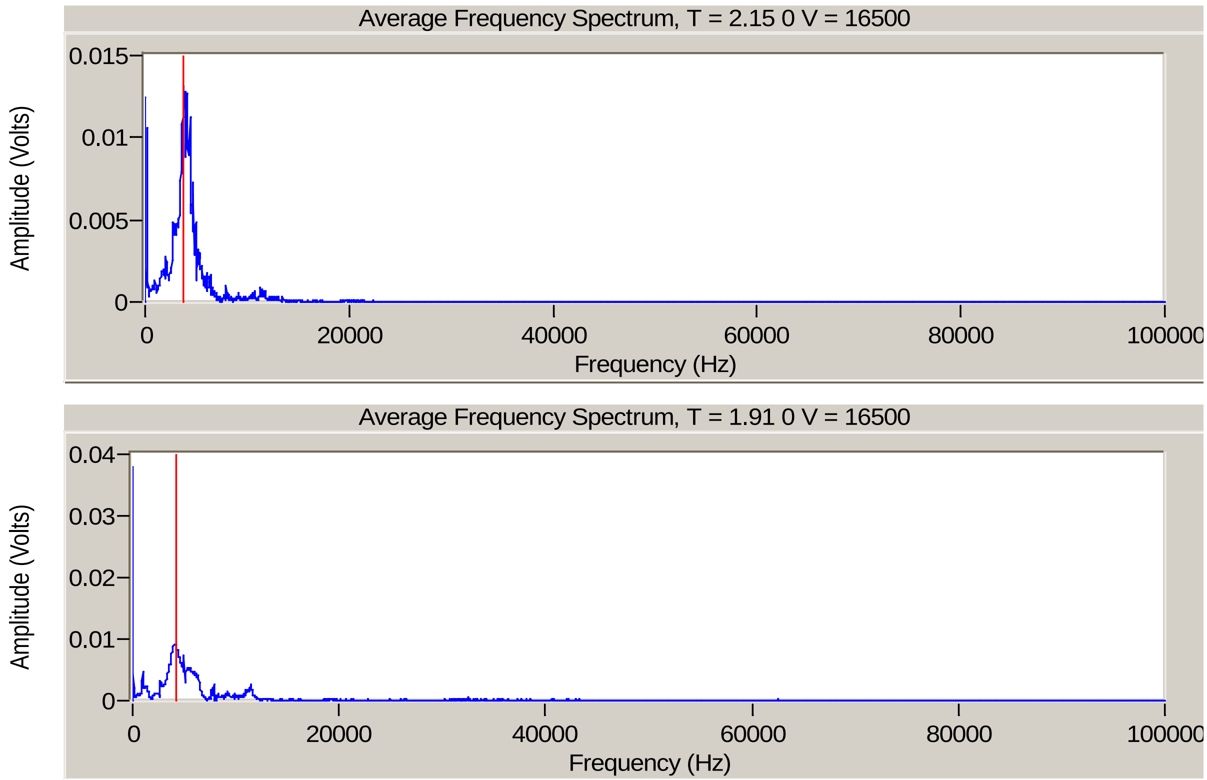


Figure 5.54 Impact Echo Test Results for Test Nail No.17

Figure 5.55 Bird's mouth Observed in Nail No.17

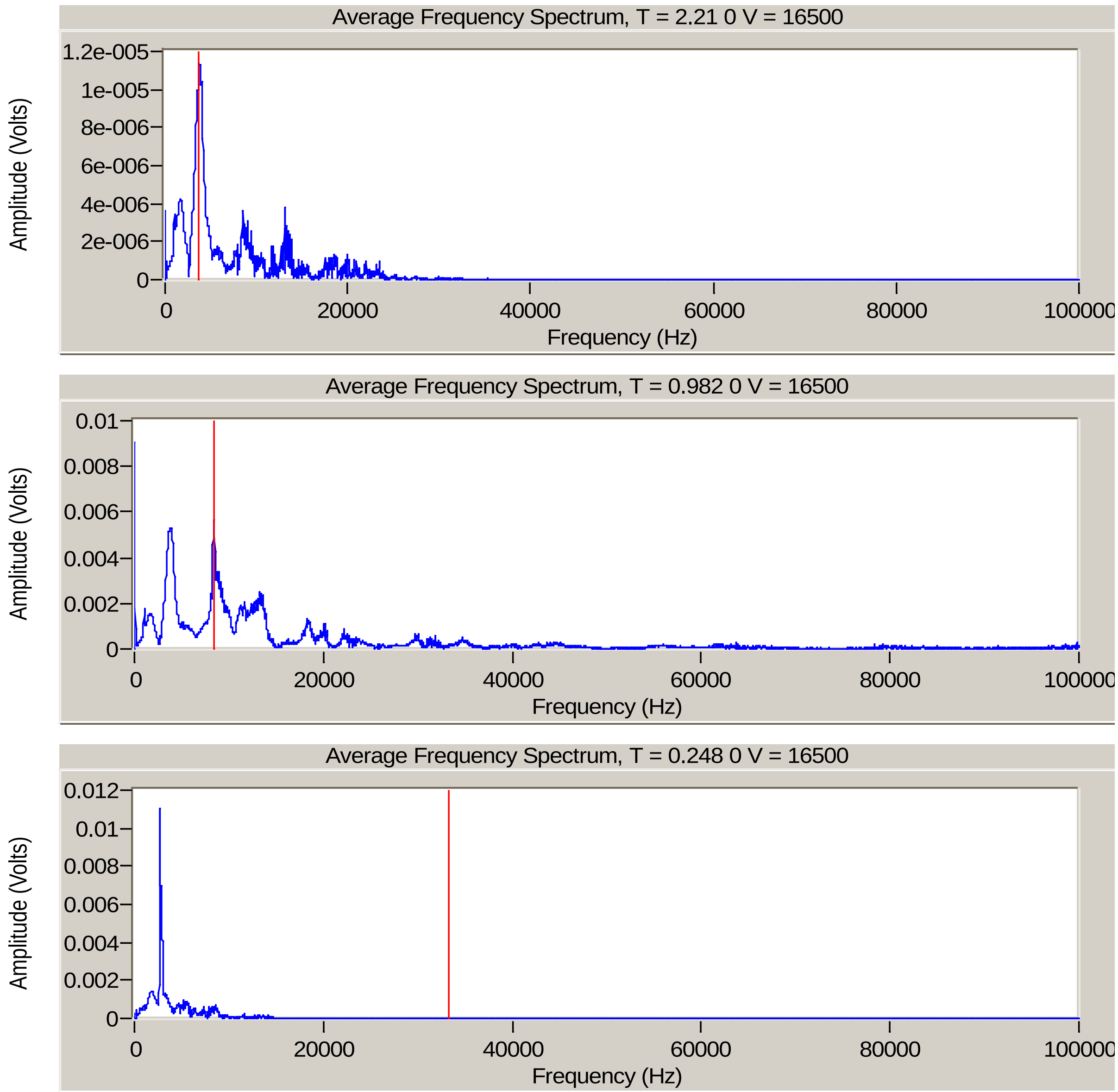


Figure 5.56 Impact Echo Test Results for Nail No.23

Figure 5.57 Bird's Mouth Observed in Nail No.23

It can be noted from Table 5.7 that Impact Echo is very reliable in detecting bird's mouth defect, however, in terms of successful prediction percentage for unplanned shallow defects, the performance of Impact Echo method is not very satisfying. Impact Echo method shares the same hardware system with Sonic Echo method, the only difference between them in this study is that the test data are acquired through accelerometers and the analyses are conducted in the frequency domain. Accordingly, conducting the Impact Echo test in the field is just as convenient as the Sonic Echo test. Further more, because Impact Echo also shares the same groups of data with Sonic Echo, there is no additional test time needed for the Impact Echo.

Table 5.7 Summary of NDT Results from Impact Echo Test

Nail ID	Designed condition	Actual condition	Impact Echo prediction
1	NL=5 ft, SC, no defect	As designed condition	Shallow defects at 2.7 ft
2	NL=5 ft, NC, no defect	Grout ends at 5.5 ft, bad head condition	Grout ends at 4.42-4.69 ft, Shallow defects at 0.76 ft
3	NL=5 ft, SC, defect @ end (from 2.5 ft)	As designed condition	Shallow defect at 2.26 ft
4	NL=10 ft, SC, no defect	Grout ends at 10.1 ft, bad head condition	Grout ends at 10.2 ft
5	NL=10 ft, NC, no defect	Grout ends at 10 ft, bad head condition, defects at 7 and 9 ft	Shallow defect at 2.3 ft.
6	NL=10 ft, SC, defect @ end (from 7.8 ft)	Grout ends at 7 ft	No shallow defect detected
7	NL=15 ft, SC, no defect	Grout starts reducing from 10 ft and ends at 13.7 ft	Shallow defect at 3.04 ft
8	NL=15 ft, NC, no defect	Defect at 3 and 4 ft, grout ends at 14.7 ft	Shallow defect at 2.17 ft
9	NL=15 ft, SC, defect @ end (from 10.5 ft)	Grout ends at 10 ft, defects at 5.4 and 8.2 ft	Shallow defect at 2.47 feet
10	NL=15 ft, SC, defect @ middle (from 6.2 to 8.2 ft)	Grout goes underneath middle defect, ends at 11.3 ft	No shallow defect detected
11	NL=15 ft, SC, bird’s mouth	Bird’s mouth, defect at 11.2 ft, grout ends at 14.4 ft	Shallow defect at 0.54-4.57 feet
12	NL=15 ft, NC, defect @ middle (from 5.9 to 7.9 ft)	Grout goes underneath foam, ends at 15.2 ft, defects at 5.9, 8.7 and 11 ft.	No shallow defect detected
13	NL=20 ft, SC, no defect	Grout starts reducing from 13 ft and ends at 14.75 ft, defect at 3.25-4.7 ft.	Shallow defect at 2.4 ft.
14	NL=20 ft, SC, no defect	Grout ends at 20.75 ft, multiple minor defects at 1.2, 2.4, 10.7, 11.7, 12.8, 13.7, 14.75 and 17.2 ft.	Shallow defect at 1 and 2.17 feet
15	NL=20 ft, SC, defect @ end (from 14.5 ft)	As designed condition	No shallow defect detected
16	NL=20 ft, SC, defect @ middle (from 6.5 to 8.5 ft)	Middle defect starts from 8.5-11.5 ft. Grout passes underneath middle defect and ends at 15.7 ft. Shallow defect at 1 ft.	No shallow defect detected
17	NL=20 ft, SC, bird’s mouth	Bird’s mouth, grout ends at 13.8 ft	Shallow defect at 1.91-2.15 ft
18	NL=20 ft, NC, defect @ middle (from 6.0 to 8.0 ft)	Middle defect as designed condition; Minor defect at 12.4 ft; Grout passes underneath middle defect and ends at 20.3 ft.	No shallow defect detected
19	NL=25 ft, SC, no defect	Grout ends at 14.7 ft	No shallow defect detected
20	NL=25 ft, NC, no defect	Grout ends at 28.7 ft, defect at 11.5 ft	No shallow defect detected
21	NL=25 ft, SC, defect @ end (from 18.5 ft)	Grout ends at 10.6 ft	No shallow defect detected

Table 5.7 Summary of NDT Results from Impact Echo Test
(continued from previous page)

Nail ID	Designed condition	Actual condition	Impact Echo prediction
22	NL=25 ft, SC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect starts from 10.5-12.5 ft. Grout ends at 10.5 ft.	No shallow defect detected
23	NL=25 ft, SC, bird's mouth	Bird's mouth, grout ends at 13 ft, defect at 2.3 ft.	Shallow defect at 0.982-2.54 feet
24	NL=25 ft, NC, defect @ middle (from 5.5 to 7.5 ft)	Middle defect as designed condition; Minor defects at 8 and 16 ft; Grout passes underneath middle defect and ends at 27.6 ft.	No shallow defect detected
25	NL=20 ft, NC, no defect	Grout ends at 20.6 ft, minor defect at 12 ft.	No shallow defect detected
26	NL=20 ft, SC, no defect	As designed condition, grout ends at 20.4 ft	No shallow defect detected
27	NL=20 ft, NC, no defect	Grout joins with the grout of nail no 28 at 18.3 ft	No shallow defect detected
28	NL=25 ft, NC, no defect	Grout joins with the grout of nail no 27 at 18.3 ft. grout ends at 27 feet	No shallow defect detected
29	NL=25 ft, SC, defect @ end (from 20 ft)	Grout stops at 15.3 ft.	No shallow defect detected
30	NL=25 ft, NC, defect @ middle (from 17 to 19 ft)	Middle defect as designed condition; Minor defect at 3.7 and 15.4 ft; Grout passes underneath middle defect and ends at 26.4 ft	No shallow defect detected
31	NL=25 ft, SC, defect @ middle (from 17 to 19 ft)	Middle defect as designed condition; Lost most of grout from 13.75 to 15.75 ft; Nail was fully grouted from 0 to 13.75 ft and from 15.75 to 17 ft. Grout didn't pass the middle defect	No shallow defect detected
32	NL=25 ft, SC, no defect	For the whole length, only the following part was fully grouted:0-2.3ft, 8-9.75ft. And the nail lost 50% of cross-section at 9.75 ft and 75% of cross-section at 12.3 ft.	No shallow defect detected

5.6.7 Test Configurations for Sonic Echo and Impact Echo Methods

Sonic Echo and Impact Echo tests can be performed under different configurations based on where the impact is made (grout versus steel tendon), where the sensor is mounted (on the grout versus steel tendon) and what type of tip is used in the impact (aluminum versus plastic). Based on the analysis of test records, it appears that different test configurations have different advantages in detecting different types of defects. Eight major Sonic Echo test configurations are discussed in this section.

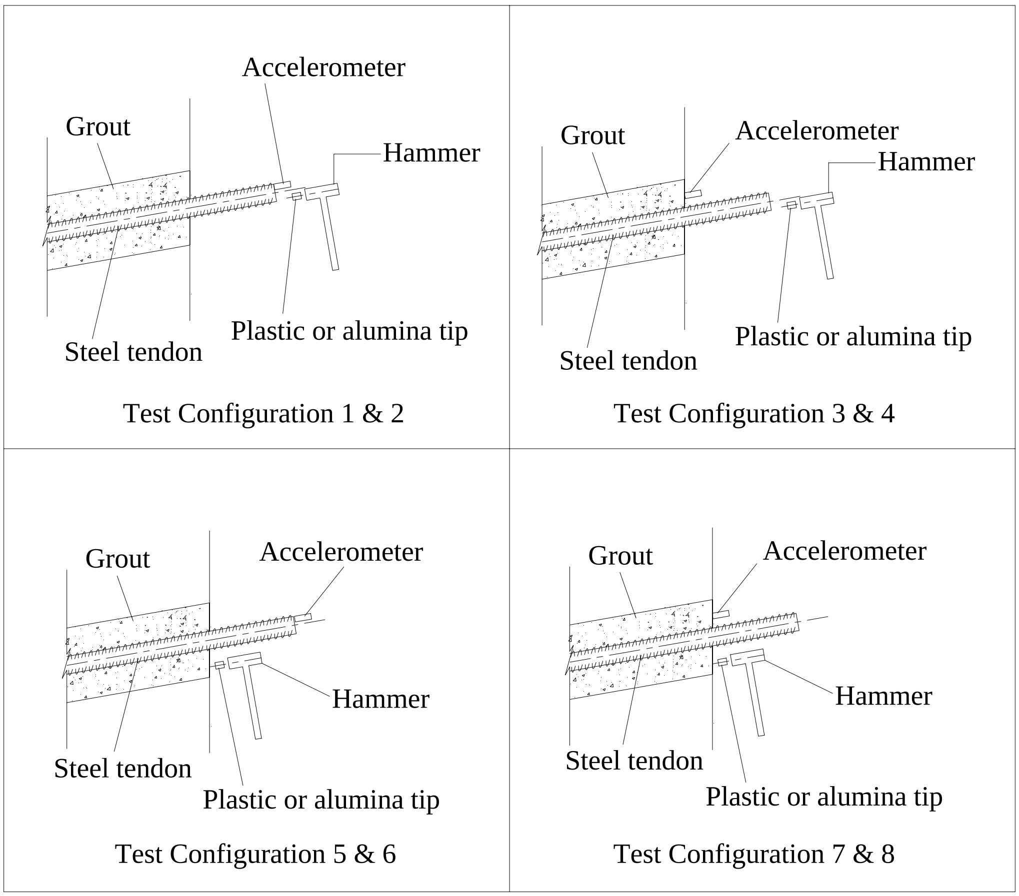


Figure 5.58 Different Test Configurations for Sonic Echo

The choice between plastic tip and alumina tip affects the frequency content in the impact generated. Generally an impact generated by a alumina tip hammer has short contact time, thus the waves from the impact having higher frequency content than the waves from a plastic tip hammer. Waves with high frequency content are attenuated faster than those with low frequency content. Therefore, waves with low frequency content propagate a longer distance, thus can detect deep defects. On the other hand, waves with high frequency content have shorter wavelengths and have better capability to detect smaller defects than waves with low frequency content. A combination of the two types of hammer tips can be used to achieve greater penetration depth as well as superior scan resolution.

Test Configurations 1 and 2 - Impactor and Accelerometer on Steel Tendon

Test configurations 1 & 2 usually did not yield useful results when the nail length exceeded 15 feet. Depending on the quality of the impact, they sometimes yielded good results for defects or length of nail within the top 15 feet (e.g. Nail No.5). The waveforms recorded from test configurations 1 and 2 were usually too noisy because impacts on steel tendon appeared to create transverse vibrations (i.e. so called "ringing effect") and the limited space available on the steel head around the accelerometers made it difficult to apply a high quality impact. It is recommended these two test configurations not be used unless the grout is in very poor condition near the nail head so that accelerometers cannot be mounted on it. In the event that this test configuration is used in QC/QA testing, the inspector must understand that data collected will be applicable to nail condition within the top 15 ft only.

Test Configurations 3 and 4 - Impactor on Steel Tendon; Accelerometer on Grout

Test configurations 3 & 4 provide the best quality impacts and the most reliable response detection because steel tendons used in construction projects tend to be defect free and extend all the way to the end of nails. This made it easier to excite the entire nail with an impact upon the tendon. This test configuration allowed the Sonic Echo method to detect the ends of grout behind severe middle defects (e.g. Nail no.18), while in other test configurations Sonic Echo method tended to be "blind" to any object beyond the middle defects. It appears that, in these cases, the large middle defect blocked most of wave energy and prevented it from penetrating to greater depths.

Test Configurations 5 and 6 - Impactor on Grout; Accelerometer on Steel Tendon

The test configurations 3 & 4 proved to be satisfactory as far as Sonic Echo test's ability to detect any major defects. More importantly, they were found to be the most efficient configuration for Impact Echo tests in terms of data quality and ease of defect peak identification. For a nail with bird's mouth defect, applying impact on grout induces the nail head to behave more like a thin plate. And surface response recorded by accelerometers mounted on steel tendons would be less noisy because any disturbance from Rayleigh waves would be less pronounced.

Test Configurations 7 and 8 - Impactor and Accelerometer on Grout

Although test configurations 7 & 8 have the strictest requirements for the nail head conditions, these configurations proved to be the most effective in detecting defects and determining defect locations in soil nails. However, a good nail head is essential to acquire high quality data through these two test configurations. Between these two configurations, the test configuration with plastic tip generates waves with lowest frequency content or the longest wavelengths when compared to any other test configuration, yielding the deepest penetration as discussed in the previous section. This test configuration tends to overlook minor defects due to the long wavelengths. The other test configuration with aluminum tip hammer has finer resolution than the test configuration with plastic tip hammer, enabling it to detect smaller defects at the expense of penetration depth. Thus, the two configurations overcome limitations of each other and the combination works better than any one of the two.

5.6.8 Results from Parallel Seismic and Cross-hole Seismic Methods

Cross-hole Seismic and Parallel Seismic methods require special access tubes for NDT instrumentation. In this research project, 30 feet long PVC pipes were used for this purpose. The PVC tubes were installed inside boreholes that were drilled parallel to the test soil nails. The gap around the PVC pipes was grouted using Bentonite. In the first series of NDT investigations, Parallel Seismic and Cross-hole Seismic methods were conducted at the test soil nail wall. Unfortunately, very little useful test data was collected from these tests. It was concluded that poor contact between surrounding soil and PVC pipes prevented the wave energy from being transmitted into the soil. The only good data acquired was from Parallel Seismic test on Nail No.18 (Figure 5.58). The test result clearly indicates the nail length is 19.8 feet which compares well with the actual length of 20.3ft. However, the test result did not give any information about the intentionally created middle defect that was in Nail No.18.

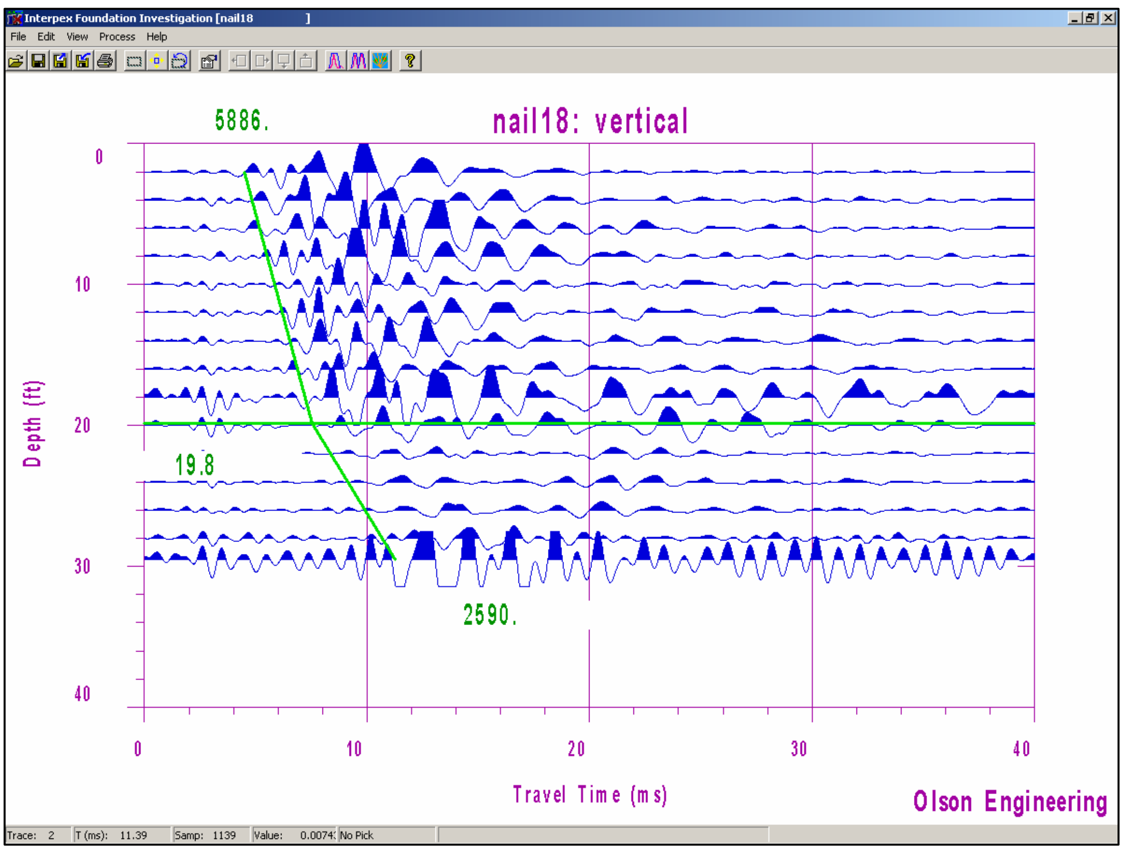


Figure 5.59 Results from Parallel Seismic Tests on Nail No.18

It is quite possible that Parallel Seismic method can be used to detect the nail length. However, further validation is needed before any definite conclusions can be drawn.

Because of the need for special access tubes parallel to the test nails, Parallel Seismic and Cross-hole Seismic methods are not suitable for use in routine QC/QA of field installed soil nails.

5.6.9 Time Domain Reflectometry Tests

Time Domain Reflectometry (TDR) was selected as a candidate method for steel tendon length detection because of the recent reports that this NDT technique had been successfully used to evaluate the continuity of pre-stressed cables used in bridge girders.

Two series of TDR tests were performed on the experimental nails at TTU campus site. Figure 5.59 shows typical results obtained from TDR tests conducted on Test Nail No.20.   In the figure, the reflection is at approximately 39.4 ft, and this includes an extension cable that is approximately 10.0 ft in length. The 60.2 ft displayed is the range the TDR is searching for an echo.

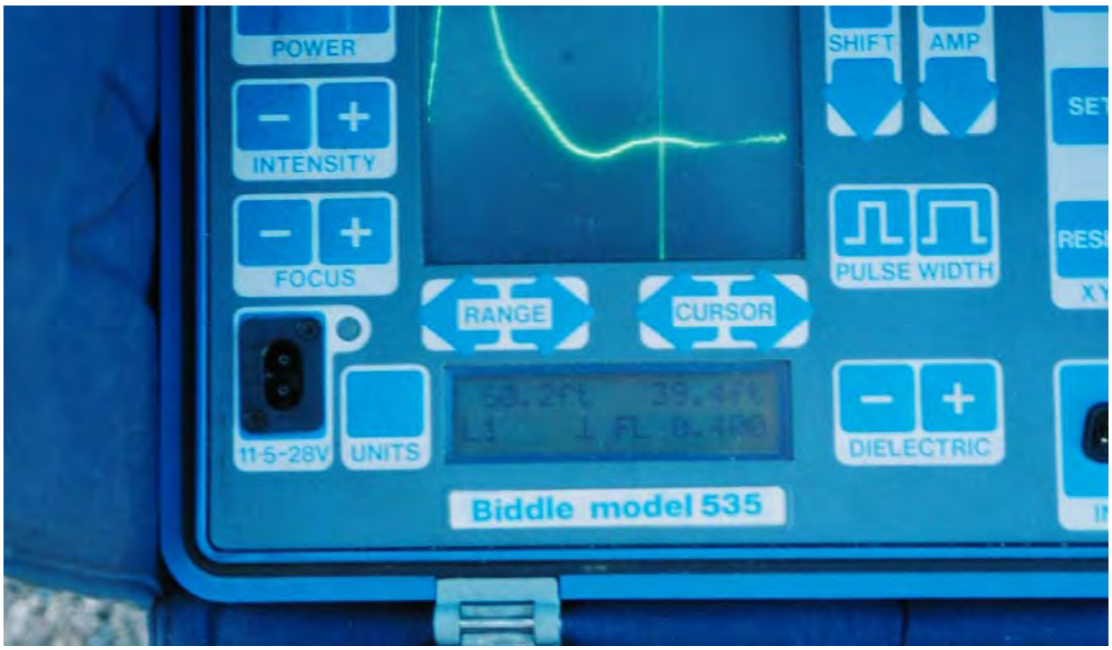


Figure 5.60 Typical Result from TDR Tests Conducted on Test Nail No. 20

Although TDR was expected to work well in the determination of the actual length of steel tendon, the overall results from TDR tests were disappointing. The method appeared to work reasonably well with long nails, especially those that were 25 feet long. The tests performed on shorter nails did not produce good results.

With a long history of application in the detection of cable breaks and recent applications in civil engineering projects, TDR was expected to be a possible solution to tendon length detection. However, the results obtained in this research study, did not support that viewpoint.

CHAPTER VI

EXPERIMENTAL STUDY TO DETERMINE OPTIMUM GROUT CONSISTENCY AND PLACEMENT PARAMETERS

6.1 Overview

As described in Chapter I, research project 0-4484 examined the soil nail grout integrity problem from two different perspectives. The first involved the development of a non-destructive test technique that would detect any defects that may be present in a grouted soil nail. The second involved a study of construction variables (such as grout consistency, length of tremie, borehole diameter, soil nail angle) to determine the optimum conditions for achieving defect-free grouting. The previous chapters, i.e. Chapters III, IV and V described research tasks that were concerned with the development of a NDT test method. This chapter describes the research tasks that investigated the effects of grout rheology and placement parameters on the integrity of grout column.

To examine the influence of construction variables on the quality of grout column, a series of experiments was conducted by pumping grout into 6-in diameter, 20-ft long PVC tubes. These PVC tubes were placed on sloping ground (specially prepared ramp) and covered with loose fill. Prior to placement, each tube was cut lengthwise into two halves. The inside of the tubes was roughened by spraying with a solvent based adhesive and then sprinkling PVC shavings and soil particles on to the adhesive. Then the top and bottom halves of the tubes were reassembled and clamped together with self-locking nylon ties. No. 6 steel rebars fitted with split PVC centralizers were then inserted into these tubes and the annulus filled with grout. Different combinations of grout mix designs and other construction variables were used in different tubes. After the grout had hardened sufficiently, the tubes were opened up to examine the condition of the grout columns. The following sections explain various phases of the study of influence of construction variables on the grout column integrity and its findings in complete detail.

6.2 Contractor Survey

The first task in this phase of research study involved a phone survey among selected soil nail contractors. The contractors were selected by the TxDOT project monitoring committee (PMC). All of the selected contractors had significant experience in building soil nail retaining walls as a part of TxDOT construction projects. The participating contractors are listed in Table 6.1.

Each company was contacted by phone and information related to specific construction procedures, material specifications and construction equipment were requested. Although a standard survey questionnaire was not used, specific items of interest were used as a baseline for discussion. In addition to the information collected via telephone survey, information about these contractors' practices was also taken from printed materials and websites of the respective soil nail contractors. Printed sales and design materials provided by *Dywidag*, a supplier of soil nail tendons and centralizers, were also used as reference in compiling information as apart of this survey.

Table 6.1 – Participating Soil Nail Contractors

1. Craig Olden, Inc., Little Elm, TX
2. Schnabel Foundation Company, Houston, TX
3. Sanders & Associates Geostructural Engineering, Inc., Granite Bay, CA
4. Bencor Corp of America, Dallas, TX
5. H. B. Zachry Company, Dallas, TX
6. Granite Construction Company, Lubbock, TX

6.2.1 Grout Mix

The majority of contractors identified neat cement grout (cement and water) as their first preference. Some indicated that they use neat cement grout exclusively. Three contractors also used a sand-cement or sand-cement-gravel grout where conditions permitted. The reasons identified for using neat cement grout included high early strength, excellent bond and filling, and ease of handling, ability to pump with a grout pump. It was noted by one contractor that neat cement grout met grout strength requirements more consistently and in their view, repetitive strength testing were not necessary when this grout mix is used. Also, they believe that there will be less risk of installing under-strength grout when neat cement grout is used. The use of other mixes (those containing small to medium-size aggregates) was found to be more common among general construction contractors, rather than specialty soil nail contractors. The reasons for using such mixtures ranged from supplier preference and familiarity, to lessening of the "bird's beak" at the open end of the borehole. The use of a standard concrete pump allows high-rate placement of grout and the use of aggregate, while grout pumps typically allow only cement-water mixes and pump at lower volumetric flow rates.

Portland cement (Type I or I-II) is used unless a very high early strength is desired, such as where a wall will be placed and the soil nails loaded soon after completion; in those instances, Type II or bagged, specialty grouts are used. Type II is also specified where chloride or sulfate attack is anticipated. Cost was not mentioned as a factor except by one contractor, who stated that the cost margin on each nail is such that using aggregate provides a necessary cost savings over neat cement grout.

6.2.2 Tendons

The survey findings suggest that it is standard practice to use plain steel tendons for temporary installations and epoxy-coated tendons for permanent installations. Only one contractor identified the use of pregrouted tendons (in PVC jacket). Another contractor had previously used pregrouted, duct-enclosed tendons, but found that defects in the inner grout were unacceptably frequent. The contractor investigated this with the supplier and found that outside air temperatures at the manufacturing location were too cold to achieve proper curing, and that this and other factors made the pregrouted tendons prone to defect.

6.2.3 Tendon Insertion and Tremie Use

Tremie type varied among contractors, but the most commonly used type of tremie pipes were either a ½ to 1 inch diameter plastic hose or 1 to 1-1/2 inch diameter plastic pipe. Those contractors who use a grout pump typically use smaller diameter tubes or pipes, while the aggregate grouts are always placed using larger (1-1/2 inch or larger) tremie pipes. All contractors contacted used pump pressure rather than gravity for grout installation (the tremie is connected directly to the pump hose with no air gap). However, none of the contractors indicated that they used high pressure grouting.

Only one contractor noted that they placed grouts before inserting the tendon, stating that the majority of their work was in saturated clay and sandy-clay soils. The use of neat cement grout is required where the tendon is inserted after grouting as a practical matter of being able to insert the bar with affixed centralizers. None of the contractors reported grouting or inserting the tendon through an open stem auger as common practice, although this alternative is used in some of the TxDOT soil nail projects constructed in accordance with Special Specifications.

6.2.4 Borehole Diameter

The borehole diameter used in soil nail installations range from four to eight inches, with six-inch being the most common size. One contractor discussed this at some length, explaining that the use of four inch soil nails is often adequate to meet the shear strength, pullout, and other requirements. This contractor stated that the additional cost of six inch nails versus four inch nails may be warranted because of the likelihood of under-strength soil nails occurring due to poor grouting, soil collapse or inclusions and other factors. This contractor noted that a failure of a single test nail could require a 50-100% increase in the number of nails, which translates to a doubling of the installation time and other costs that can be entirely avoided by using six inch nails. It was also noted that grout placement is far more difficult around a tendon in a four inch diameter boring. As shown in Figure 6.1 below, there is insufficient diameter available to tremie any grouts using low flow pumps other than the most fluid grouts.

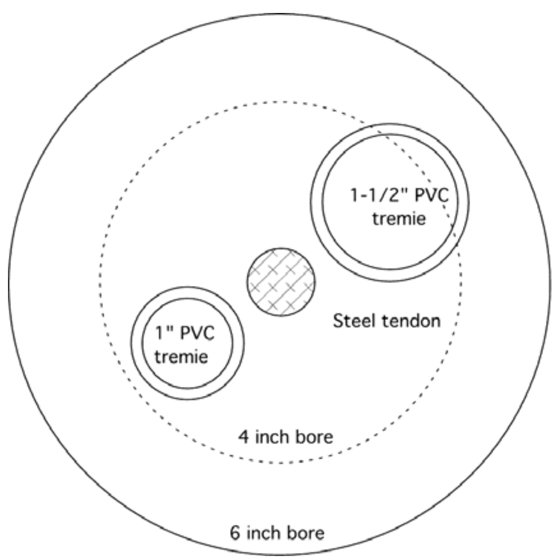


Figure 6.1 Cross Section of a Soil Nail, Comparing Size of the Borehole, Tremie and Tendon

6.2.5 Construction Crew

Most contractors emphasized the need to have special crews who are experienced in soil nailing. They stated that unfamiliar crews generally are not responsible for such installations without proper training and supervision.

6.3 Field Experiments to Evaluate Factors Affecting Grout Column

A series of field tests was performed in order to determine the optimum grout consistency and grout placement conditions that would yield the best quality soil nails. These tests were designed to examine the contribution of each factor that may influence the effective filling of soil nail drillhole. The parameters that were postulated to be the most critical were grout flow character ("rheology") and depth of insertion of the tremie tube or pipe. Design parameters (such as angle of the drillhole), soil properties (such as cohesiveness), or the presence of moisture in the borehole, may also affect the final installed quality of the soil nail. In these field experiments, however, all of the simulated soil nails were installed at a five degrees angle (5°) from horizontal. It was understood that this low angle was not representative of a typical field installed soil nail. In the field, most soil nails are installed at a greater angle (10-15° from horizontal) although, in some field installations, nails are placed at shallower angles to avoid underground obstructions such as utility lines. It is intuitive that grout materials would flow better when drillhole angles are steeper. Therefore, the 5° nail angle used in experimentation can be considered to be representative of the worst-case condition. Another parameter that was maintained constant in these tests was the diameter of the borehole. All of these tests were conducted using 6-inch diameter boreholes because this diameter is used as a standard in all TxDOT construction projects. Among the two types of centralizers commonly used, i.e. split PVC tube and disk, only the split PVC tube centralizers are allowed in TxDOT construction. Therefore, the research did not investigate the use of disk type centralizers. Other factors which were held constant in this experiment were soil type and soil moisture content.

Three primary variables that could influence reliable grout placement in soil nail installations were considered during the design of this experiment. The first was the grout consistency which is measured by ASTM C143 (Slump test) and several other methods. The second was the depth of insertion of the tremie pipe or tube. The third and the final variable involved grout mix design. In addition to neat cement grout, several other sand and gravel grout mix designs that are representative of grouts that are used in the field to reduce shrinkage, reduce cost, and improve crack distribution were also used.

6.3.1 Grout Consistency

Grout flow character, or rheology, is important as the grout must have the ability to flow into the annulus between the tendon and the surrounding soil mass. The grout must fully embed the tendon, flow around the centralizers, and conform to all surfaces intimately to assure proper bonding of the soil nail. Where grout is more fluid, expression of water or water-cement mixtures into the surrounding soil can increase, and subsidence may occur as a result of soil absorbing water. There is also a direct correlation between water content and strength, so using more water to create a more fluid grout may adversely affect the grout strength; a higher water content in grout also increases volume shrinkage during cure (particularly in dry soils). For these reasons, the experiment sought to determine the stiffest, or least fluid grout which could be installed reliably. The method selected for this test was the ASTM C143 slump test. A neat cement grout, consisting of water and Portland cement (Type I or I-II), mixed at a water-cement weight ratio of 0.36 to 0.50 was considered the baseline grout, representative of a majority of soil nail installations. Throughout this range of water content, the grout is very fluid, yet has a compressive strength which is consistently above the specification minimum at 7-day of 3000 psi (20 MPa.)

6.3.2 Tremie Length

Specifications generally require the tremie to extend the full length of the tendon. To comply with this requirement, the contractor must select a tremie pipe that easily fits within the narrow space between the tendon and centralizers. At the same time, the tremie pipe diameter should be large enough so that the grout will flow through it to the end of the hole. Where the tremie is of reasonably smooth pipe or tubing, the force required to insert or withdraw the tremie is minimal so long as the borehole walls are relatively straight and smooth. TxDOT reports that the only catastrophic failure of a soil nail installation placed under TxDOT oversight occurred when the installer did not properly tremie the grout. Examination of the failure showed incomplete grout filling of the boreholes throughout their length.

6.3.3 Aggregates in Grout

Some vendors either currently use, or expressed interest in using, grouts containing sand or larger aggregates. FHWA notes that such additions to the grout may be desirable and TxDOT generally permits the use of grouts containing aggregate. With a typical “neat” cement-water grout, the cement will shrink during set and cure. The use of non-shrink grouts offsets this effect, but increases the cost per soil nail; expanding grouts (which swell as they set and cure) may also be used with success, however they also result in higher cost. The addition of sand and gravel in various proportions can be used to impart certain desirable characteristics such as toughness, but they also reduce the cement content, cost and potential for shrinkage. Three notional grout mixtures were developed to cover a range of potential field conditions.

In addition to the neat cement grout discussed in Section 6.2.1, a sand-cement grout was designed, along with a sand-gravel-cement grout. The sand-cement grout (“sand grout”) would be a mortar mix which has a rich, Portland cement content and proportionally higher water content. Compressive strength would be maintained by limiting the water cement ratio to 0.50 or less. The sand-gravel-cement grout (“gravel grout”) would be similar to a typical transit mix, with a higher cement content than typical, and a water cement ratio in the range of 0.4 to 0.5.

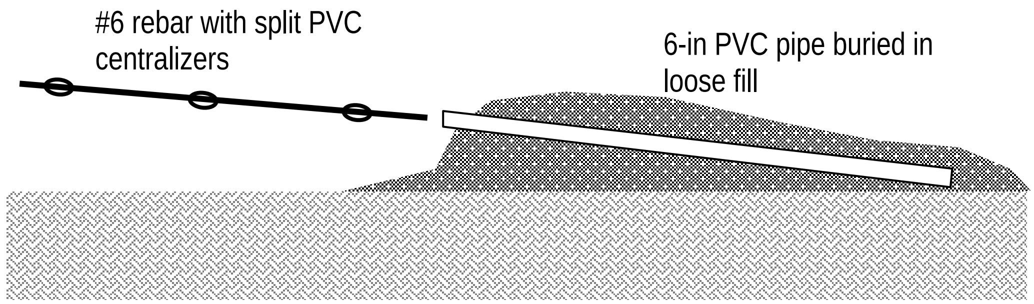
6.3.4 Testbed Preparation

The testbed designed for this experiment consisted of fifteen PVC tubes, each twenty feet in length. The tubes were nominal six inch diameter, schedule 40 PVC pipe obtained from a local supplier. Each tube was cut lengthwise into approximately upper and lower halves. The inside of these tubes were sprayed with solvent-based adhesive, into which PVC shavings and local soil were imbedded. The smooth interior surface of the PVC pipe was roughened in this manner so that the tube would better simulate the surface of a typical soil nail boring. No attempt was made to simulate larger irregularities in the surface of the borehole, such as those created by augers as they encounter difficult or loose soil. The tubes were reassembled using fiber reinforced tape (common "duct tape") and further strengthened with self-locking, nylon ties ("cable ties”). Figure 6.2 is a close up view of the surface coating applied to the inside of the tubes. Figure 6.3 shows the interior of a tube after the tendons had been placed inside the tube.

Next, a soil area was excavated at an angle of approximately five degrees from horizontal (the “ramp”) as depicted in Figure 6.4. The tubes were placed on the ramp and the sides and ends of the tubes were backfilled; the downhill end was compacted so that soil extended a short way into each tube to act as a plug to prevent escape of grout. The backfill provided sufficient confinement to prevent lateral and vertical movement of the tubes.

Figure 6.2 Roughened Interior of PVC Tube Prior to Reassembly

Figure 6.3 – Inside of the PVC Tube with Tendon and Centralizers in Place



(a)

(b)

Figure 6.4 Test Bed Used to Investigate the Influence of Grout and Placement Parameters: (a) End View, (b) Front View

One #6 (3/4 inch) common steel reinforcing bar (rebar) was fitted with three split PVC centralizers and placed into each of the tubes. The centralizers were installed at approximately four feet from each end and at the middle of each bar, using either duct tape or nylon ties to secure the centralizer. Common practice is to place centralizers not more than 8 feet apart along the tendon. The rebar extended a short distance out of the open tube end and down to the soil plug in the distant end. Figure 6.5 shows common rebars with centralizers attached and ready to be inserted into the testbed.

Figure 6.5 – Common #6 Rebar with Centralizers

6.3.5 Test Variables

Tremie Length: Three representative tremie insertion depths were selected: full depth (about 19 feet), half depth (at approximately the middle centralizer), and minimal depth (near the first centralizer). The tests with full tremie length were to determine if stiff grouts would fill the tube completely and consolidate well if tremied properly. It was postulated that under a full head (on this test, about 24 inches) the stiff grout would consolidate well. The partial tremie lengths were selected to test whether grout would plug at centralizers, or will flow downhole when it has sufficient pressure head and/or when it has sufficient flowability. By placing the tremie a short distance into the tube on the minimal tremie cases, it could be observed whether grout tended to flow under minimal head, and whether plugging occurred at the centralizers or elsewhere. The middle position tremie tests, when compared with the minimal tremie tests, would indicate whether additional head increased downhole grout flow.

Grout Consistency: Rigorous control of flowability of the grout is generally not possible under operational field conditions, and therefore only basic parametric checking should be implemented for field use. Secondly, better compliance can be achieved if the equipment used for the measurement of flowability is readily available and the average contractor is already familiar with it. To this end, grout specification for each test was based on ASTM C143 *Standard Test Method for Slump of Hydraulic Cement Concrete.* This assured that the grout supplier could understand and provide an appropriate mix using locally available materials.

Once delivered at the site, grout consistency was measured using two test methods: ASTM C143 slump, and a "v-funnel" test which is commonly used to measure flow of self-consolidating concrete. In addition to the measurement of slump, the resulting diameter of slumped grout was evaluated; this is sometimes referred to as *puddle diameter* or *slump flow*. The average diameter of the slumped grout after the ASTM slump cone was removed was measured; this result is reported herein as slump flow. Grout slump, as measured by the ASTM method, was planned over the range of 6-7 inches up to 11 inches. This would represent a range from the limit of a typical trailer-mounted concrete pump ("unacceptably stiff"), to the practical limitation of the ASTM C143 method, which cannot measure highly fluid mixtures. The flowability of very fluid, neat cement grout was measured using the v-funnel and flow cone methods. The v-funnel test was predicted to be useful as a "go/no go" method, since it provides very limited, quantitative measurements which would be difficult to implement in the field with good repeatability.

Grout constituents were selected to represent the likely materials provided by a typical concrete supplier, and were chosen based on a local supplier experience database; grout design variation and testing would allow refinement of the grouts prior to use in an actual installation. Locally available material and limitations of the pumping equipment dictated the grout designs. Five grouts were selected: neat cement, sand-cement low slump ("Sand 1"), sand-cement high slump ("Sand 2"), pea gravel low slump ("Gravel 1") and pea gravel high slump ("Gravel 2"). Table 6.2 shows the grout constituents.

Table 6.2 – Grout Mix Designs

Grout	**Water: cement ratio**	**Target ASTM slump**	**Constituent Actual quantity mixed**
Sand 1	0.4	8	Sand: 5131 lbs. Cement[1]: 1505 lbs. Fly ash: 295 lbs. Water: 528 lbs. Additive: 300R[2]: 72 oz.
Sand 2	0.5	11	Same as above, remove ½ yard, then add Water:~40 lbs.
Gravel 1	0.5	10.5	Gravel (½" max): 1200 lbs. Sand: 4766 lbs. Cement*: 1140 lbs. Fly ash: 264 lbs. Water: 324 lbs. Additive: Polyheed[3]: 52 oz.
Gravel 2	0.4	8.5	Gravel (½" max): 1240 lbs. Sand: 4720 lbs. Cement[1]: 1070 lbs. Fly ash: 265 lbs. Water: 180 lbs. Additive: Polyheed[3]: 52 oz.
Neat Cement	0.4	NA	Cement[1]: 2090 lbs. Water: 754 lbs. Additive: 300R[2]: 20 oz.

[1]Portland Type I or I-II
[2]Degussa Pozzolith 300R water reducer-set retarder
[3]Degussa Polyheed water reducer-plasticizer

6.3.6 Testbed Installation

On the day of the test, weather was clear and mostly sunny. Temperatures were in the 80's, with low humidity. The tubes were inspected for debris and were found to be unobstructed and ready for grout. Grout placement began shortly after noon, finishing about four hours after starting. Samples of each grout mix were placed into plastic, 4 inch by 8 inch, cylindrical sample containers.

The sand cement grout was initially provided by the transit mix supplier at an ASTM C143 slump of 8 inches. This material was installed into three tubes, one at each tremie position. The same mixture was re-tempered with the addition of water to the mix truck; ASTM slump was measured to be 11 inches. The next set of four tubes was filled with this grout using the three tremie positions. In each case, the grout was pumped as the tremie was withdrawn. This was continued until the end of the tremie was at the end of the tube and the grout flowed from the open end. This was the process used in all cases. Figure 6.6 shows the slump test of the first sand cement mix; Figure 6.7 shows the puddle of the 11 inch slump sand cement grout.

Figure 6.6 ASTM Slump Test of Sand-Cement Grout in Progress

Figure 6.7 Puddle Remaining After Slump Test of 11-inch Slump Sand-Cement Grout

There was no attempt to avoid bird's beak defects at the open end, however, with the exception of the neat cement grout, the grouts had sufficient body to retain their fill without significant repose. Table 6.3 shows the field test results for the measurements taken at the time of placement.

Table 6.3 Grout Flowability

Grout	**Slump (inch)**	**V-funnel (sec)**	**Notes**
Sand 1	8	No flow	
Sand 2	11	~2	Slump flow: 20 inches
Gravel 1	10.5	~4	Slump flow: 16 inches
Gravel 2	8.5	No flow	
Neat Cement	NA	~2.5	½ inch flow cone plugged by unmixed material and gravel

The pea gravel grout arrived at a slump of 10-1/2 inches, and was placed into three tubes using the three tremie positions. A second batch was rejected at the site as it arrived at greater than 10 inch slump. The final batch of gravel grout tested at a slump of 8-1/2 inches; this was placed into the two remaining tubes using full tremie and minimal tremie only. The transit ticket from the supplier showed that an inferior grade of gravel was used for these gravel mixes. It was decided to go ahead with the tests in spite of the risk that this grout would have compressive strength below the required 3000 psi at seven days. This decision would not compromise the column-filling aspects of the tests, but the mix design would not be suitable for actual soil nail installations.

Figure 6.8 is a view of the cross section of grout samples taken for strength testing. The samples were prepared by crosscutting with a wet saw to insure that the end surfaces were flat and perpendicular to the long axis.

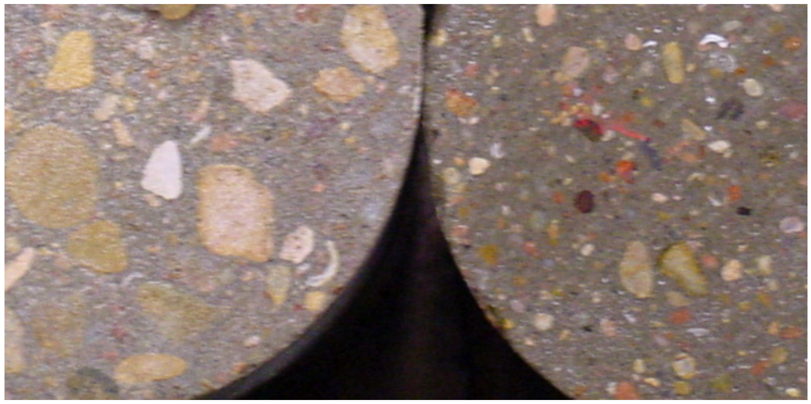


Figure 6.8 – Cut Section Of Grout Samples Showing Gravel Grout (Left) And Sand Grout (right).

The final experiment used a mid-range water cement ratio for the neat cement grout, which yielded a very fluid grout with relatively low shrinkage. The water-cement ratio was 0.37, and the grout was slightly viscous, yet very fluid. This grout was placed into three tubes. The first of these was placed using full tremie; the second was placed using a minimal tremie. The tendon was removed from the third tube, the grout was placed using a full length tremie, then the tendon with the centralizers was inserted into the grouted tube. Only minimal resistance was experienced when inserting the tendon in this manner.

V-funnel testing showed that only the most fluid grouts would flow through the 2.5 inch square opening. It was also noted that the high slump sand- and gravel-containing grouts which did flow in the v-funnel test did leave a substantial amount of material clinging inside the apparatus. The neat cement grout took longer to clear this apparatus as more material exited under gravity without a break in the discharge stream. Figure 6.9 shows the v-funnel apparatus.

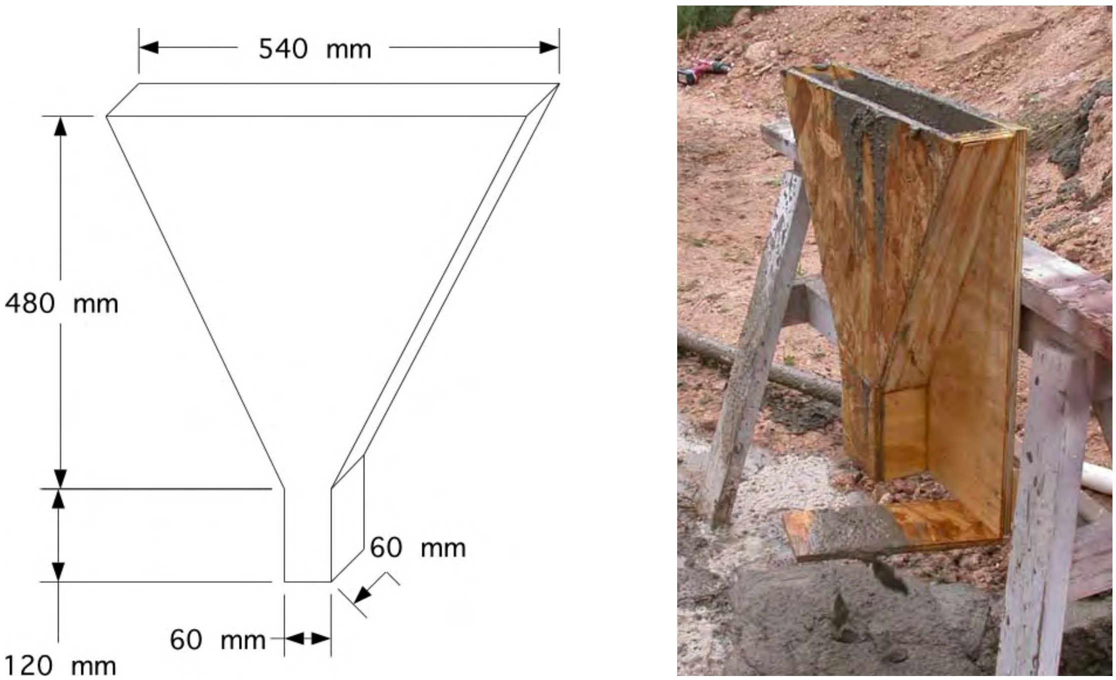


Figure 6.9 V-funnel Apparatus; Dimensions (left) and Test Apparatus in Use (right)

6.3.7 Results from Field Experiments

Two days following the placement of the grout, the testbed was excavated and the PVC tubes were opened up to expose the grout columns. The observations made at the time of exhumation are summarized in Table 6.4. One observation that could be readily made was that, in all cases, when fully tremied, the grout filled the simulated soil nail borehole and fully embedded the tendon. In all cases, when the tremie was not fully inserted, the grout did not fill the tube completely.

All grout mixtures of similar slump show similar embedment and grout column integrity. The grout column of neat cement grout appears to be in every way similar to high slump sand cement and pea gravel grouts. The distant end of the neat cement grout column did show signs of gravity flow as evidenced by a tapering slope (Figure 6.10). This seems to indicate that a greater water content and/or a greater incline might have resulted in a more completely filled tube.

Where the grout did not completely fill the tube, the sand and gravel grouts had a rounded "plug", usually at or just beyond the first centralizer beyond the tremie end. Lower slump grouts had smaller, visible defects in the grout column and tended to fill for a shorter distance beyond the end of the tremie or stopped nearer a centralizer. Figure 6.11 shows plugging of the grout flow at a centralizer.

Table 6.4 Visual Evaluation of Grout Columns

Tube	Grout	Tremie	Result
1	Neat	Minimal	Grouted 2/3 full
2	Neat	Full	Fully grouted
3	Neat	Full	Fully grouted
4	Sand 2	Full	Fully grouted
5	Sand 2	Half	Grouted to distant centralizer
6	Sand 2	Minimal	Grouted 2/3 full
7	Sand 1	Full	Fully grouted
8	Sand 1	Half	Grouted to middle centralizer
9	Sand 1	Minimal	Grouted to 8’
10	Gravel 2	Full	Fully grouted; small voids
11	Gravel 2	Half	Grouted 2/3 full
12	Gravel 2	Minimal	Grouted to middle centralizer
13	Sand 2	Full	Fully grouted
14	Gravel 1	Full	Fully grouted
15	Gravel 1	Minimal	Grouted to middle centralizer

Figure 6.10 Reposed Tail on the Downhole End of the Minimally Tremied Neat Cement Grout Tube

Figure 6.11 Sand 1 Grout in Tube 8, Showing Middle Centralizer and Distant End of Grout Column

Several types of defects were noted as the tubes were exhumed. Figures 6.12 shows an air pocket defect in a neat cement column and Figures 6.13 and 6.14 show cylindrical voids. Each of these was apparently caused when the tremie was withdrawn quicker than the tube filled with grout, thereby trapping air and leaving a defect.

Figure 6.12 Neat Cement Grout In Tube 1, Showing A Void Across About ½ Of The Grout Column Diameter

Figure 6.13 Sand Grout Column With A Defect Caused By Withdrawal Of The Tremie From The Surface Of The Grout As It Was Being Pumped

Figure 6.14 Sand Grout Column with a Defect Caused as an air Pocket was Formed during Grouting

6.4 **Laboratory Testing of Grout Cylinders**

Cylindrical samples of grout were made at the time of their placement into testbed tubes. These samples were lime water cured according to standard practice (TxDOT material testing procedure TEX-447-A). Subsequently, they were tested for compressive strength using a Gilson MC-600 compression testing apparatus according to TxDOT material testing procedure TEX-418-A. A summary of the results is shown in Table 6.5. The slightly low compression strength of the gravel samples may be attributed to the low quality local gravel used in the production of the gravel grout. This material was permitted for use in this experiment because the primary parameter of interest was the flow character of the grout. The similarity in strength between batches, in spite of different water-cement ratios, indicates aggregates likely controlled the strength. It was also noted during the testing that some of the gravel broke apart during the tests.

Table 6.5 - Compressive Strength of Grout Samples from Testbed Experiments

Grout	W/C ratio	Slump (inch)	Strength min (psi)	Number of samples
Neat	0.36	N/A (<11.5”)	5700	2
Sand Cement 1	0.30	8	4600	2
Sand Cement 2	0.36	11	3500	2
Gravel 1	0.23	10.5	2500	2
Gravel 2	0.20	8.5	2500	2

6.5 Interpretation of Testbed Results

From the observations made during the field experiments described in this chapter, several conclusions can be reached. First of all, the results indicate that the single most important factor in assuring proper grout placement is full insertion of the tremie pipe or grout tube. Insertion of the tremie to full depth of the soil nail is required by FHWA and TxDOT, and is standard practice in the industry. However, compliance with this requirement in the field is not done consistently. It appears that any grout that is capable of being pumped through general purpose concrete or grout pumps, will fill the annular space in the soil nail boring if the grout is properly placed through a full length tremie. It was also observed that improperly placed grout of any type, even those that have high fluidity, may not completely fill a soil nail boring when the available gravitational head is not sufficient. There is insufficient evidence from these tests to determine whether highly fluid grouts will completely fill higher angle (10-15 degree) soil nail borings when pumped through short tremies. There does appear to be a consistent finding of defects in soil nails which are not tremied correctly, without regard to grout type or flow character.

The use of highly fluid grouts, similar to those used in pre-placed aggregate concrete or post-tensioning ducts, is common practice in soil nails installation. The use of such grouts does not necessarily correct for improper installation techniques, nor is such highly fluid grout required to reliably create good quality soil nails. It was noted that the voids observed in the low angle test nails with all grouts, seemed to correspond to withdrawal of the tremie while grout was not flowing (between pump strokes) or where the grout tube was withdrawn faster than the borehole filled.

In actual installations, with highly fluid, neat cement grout placed into higher angle bores, the drilled soil nail cavity will likely fill in spite of poor placement practices. The gravity flow of grout beyond the tremie length may take some time to occur even with highly fluid grouts. Where full tremie is used, a highly fluid grout may make the installation less difficult and may be more forgiving of various placement errors. However, with stiffer grouts, the formation of defects at the open end of the borehole are more easily controlled; with highly fluid grouts, it is much more difficult to prevent a “bird’s beak” defect from forming.

It appears that grouts of similar slump will fill soil nail borings equally well. The presence or absence of aggregates up to 1/2 inch does not seem to affect consolidation to any significant degree. The use of highly fluid grouts with or without aggregates should be generally acceptable unless testing shows that excessive crack development due to shrinkage will reduce corrosion protection. The use of stiffer grouts also appears to be suitable so long as good installation technique is followed. Visual examination of grout removed from samples showed no significant voids and showed continuous contact between the grout and tendon. It appears that grout composition has little effect on grout-tendon embedment.

CHAPTER VII
CONCLUSIONS AND RECOMMENDATIONS

7.1 Overview

Incomplete grouting of installed soils nails has been found to be the primary cause for the poor performance and the eventual failure of several TxDOT soil nail walls. It has been further observed that the use of grout mixes with improper consistency and/or poor installation practices are among the major factors contributing to this problem. The QC/QA procedure that is currently used by TxDOT consists of installation of two sacrificial test soil nails and load testing them to determine their ultimate load capacity. While this procedure is useful in verifying the assumptions made during wall design and the adequacy of the contractor's installation method, it does not necessarily guarantee good quality installation of all production nails. Therefore, this research was initiated by TxDOT with the primary objective of developing appropriate tools and construction specifications that can be used to achieve improved quality in field installed soil nails.

This research examined the soil nail grout quality problem from two different perspectives. The first involved the development of a non-destructive test technique that could be used to detect defects or voids that may be present in the grout column of an installed soil nail. The second involved a study of construction variables (such as grout consistency, tremie length, borehole diameter, soil nail angle) to determine which of those variables have the most dominant influence on the integrity of the grout column. Appropriate QC/QA procedures can then be put in place to have better control of those variables in the field.

The development of an NDT method for verifying the quality of soil nail grout columns involved identification of candidate NDT techniques that have been used in similar applications and then testing them on field installed soil nails. Three iterations of NDT testing were conducted, so that limitations in the selected NDT methods could be identified and necessary refinements implemented after each round of testing. To enable NDT testing under controlled conditions, a 125-ft long, 6.5-ft high experimental soil nail wall was constructed on the Texas Tech University campus. The test wall accommodated 32 soil nails that were placed in a single row with a nail spacing of 4 ft. The design lengths of nails varied from 5-ft to 25-ft. The soil nails were 6 inches in diameter and installed at a 10° downward angle with nail heads located 3 ft from the top of the test wall. The center steel tendons consisted of epoxy coated Grade 60 steel bars with a diameter of 1 inch. Split PVC style centralizers were used because TxDOT specifications only allow the use of this type of centralizer. Two types of grout, a sand-cement-water mixture ("sand cement grout") and cement-water mixture ("neat cement grout") were used in these soil nails. Three types of artificial defects, including voids at the end of nails, voids in the middle of nails, and bird's mouth, were simulated at specific locations by mounting closed cell polyethylene foam on the steel tendon. Once all of the necessary non-destructive testing was completed, the nails were exhumed and the grouted condition of each nail was directly observed and recorded. This information was then used to determine the validity of NDT predictions made previously.

To examine the influence of construction variables on the quality of grout column, a series of experiments was conducted by pumping grout into 6-in diameter, 20-ft long PVC tubes. These PVC tubes were placed on sloping ground (especially prepared ramp) and covered with loose fill. Prior to placement, each tube was cut lengthwise into two halves. The inside of the tubes was roughened by spraying with a solvent based adhesive and then sprinkling PVC shavings and soil particles onto the adhesive. Then the top and bottom halves of the tubes were reassembled and clamped together with self-locking nylon ties. Then No. 6 steel rebars fitted with split PVC centralizers were inserted into these tubes and the annulus filled with grout. Different combinations of grout mix designs and other construction variables were used in different tubes. After the grout had hardened sufficiently, the tubes were opened up to examine the condition of the grout columns.

The following sections present the conclusions and recommendations from this research.

7.2 Conclusions

The preliminary evaluation of available NDT methods revealed that the Sonic Echo method has the greatest promise for use in soil nail grout integrity testing. Therefore, this test method was adopted, further refined and "customized" for soil nail application through NDT testing conducted at the Texas Tech experimental soil wall. Data collected from three rounds of NDT testing were used to identify the optimum test parameter combination to be used with Sonic Echo testing. Among the various impact sources that generate waves with different frequency contents, 0.2-lb modal hammer that integrates a force sensor into the head to allow measurement of the impact time and waveform was selected for soil nail testing. The hammer can be used either with aluminum or plastic tip. The proposed test procedure requires six repetitions of the test; three repetitions using the hammer with the aluminum tip and three additional repetitions using the hammer with the plastic tip. Accelerometers with frequency range between 1.0 to 10,000 Hz and with magnetic base were chosen to receive the feedback signal. The sonic echo test can be performed under different configurations based on where the impact is made (grout versus steel tendon) and where the receiver is mounted (grout versus steel tendon). Optimum conditions were achieved when both the impact and the receiver were on the grout column. However, a good nail head is essential to obtain good quality data through this test configuration. When the nail head condition is poor, the impact can be made on the steel tendon and the reflected signal received with an accelerometer mounted on the grout column. To attach the accelerometers, steel washers are first affixed to the front face of the grout column with hard set epoxy and after the epoxy has hardened, the accelerometers with magnetic base are mounted to the washers.

The Sonic Echo NDT System recommended for the integrity testing of installed soil nails shall consist of the following essential components: (a) a data collection computer with signal conditioning, digitizing and processing components, (b) accelerometer capable of being mounted via magnetic base and a glue-on washer to the end of the soil nail, and (c) a 0.2-lb modal hammer equipped with replaceable steel or hard plastic tips. A complete and more detailed description of the NDT system is found in a companion report entitled *Non-destructive Testing of Installed Soil Nails Using Sonic Echo Test Method; Hardware Specifications*. Work on a hand-held Sonic Echo System that meets the requirements of this application is now nearing completion at Olson Engineering. The cost of the unit is estimated to be between $8,000-$11,000 depending on the specifications.

The results collected from NDT testing conducted in this research show that the quality of Sonic Echo test data and the SE predictive capability are greatly enhanced when the nail head conditions are good. This is clearly evident when the results from NDT test series No.1 are compared with those from test series Nos.2 and 3. Test series No.1 was conducted shortly after the experimental soil nail wall had been constructed. The nail head conditions of a large percentage of soil nails were found to be poor after their initial installation. They were repaired before the remaining NDT tests were conducted. For this reason, the results from test series No.1 were not considered in the evaluation of Sonic Echo test data to determine the predictive capability of the test method.

The Sonic Echo test data also showed that the accuracy of prediction is better when the defects in the grout column were more isolated. In other words, SE test was able to detect the grout column length with better accuracy when the grout column were either defect free or when it had a major defect but otherwise defect free. These conditions were found in the following soil nails: Nail Nos. 2 through 10, Nail No. 16, Nail No. 18, Nail No. 22 and Nail Nos. 26 through 29. It must be noted that not all of these soil nails were found to be in “as-designed” condition. But whenever unintentional defects occurred, SE test was able to detect their location correctly. Examples of this include: Soil Nail Nos. 5, 9, 27 and 29. In some cases, minor defects found at shallow depth did not impair the test method’s ability to detect major defects found at larger depths. Nail Nos. 8, 9 and 16 are examples of soil nails with minor defects at shallow depth that yielded good SE results. However, this situation was an exception rather than the rule. More commonly, poor head condition and/or multiple defects at shallow depth significantly impaired SE test’s ability to probe the grout column to greater depths. Soil Nail Nos. 11, 13 and 14 are some examples of this condition.

Another important variable that must be examined when evaluating the selected NDT method is the maximum length of nail that could be probed by the test method effectively. This was the reason for including a range of nail lengths, 5-ft through 25-ft in the research plan. In general, agreement between predicted and measured grout column lengths of up to 15ft was quite good. As far as longer nails are concerned, very few soil nails with design lengths of 20-ft and 25-ft were found to be in “as-designed” condition. This was particularly true for the soil nails that were installed in Phase I wall construction. In 20-ft and 25-ft long soil nails that used sand-cement grout, the grout often stopped at a distance of about 14-15ft (e.g. Nail Nos. 13, 17, 19 and 23). In nails that used neat-cement grout, the grout filled the full length of the hole, but multiple minor defects were found in the grout column. Phase II construction was undertaken to specifically address this problem. Accordingly, all soil nails that were installed in Phase II were either 20-ft or 25-ft long. Among these, Nail No.26 was found to be in “as-designed” condition with a length of 20-ft and was completely free of defects. In this sand-cement grouted nail, the predicted length of grout column matched exactly with the actual measured length. Its neat cement counterpart, Nail No. 25, was grouted to full length but had an unintentional defect in the middle. This nail did not produce a clear echo in the Sonic Echo test. An accurate length prediction was also made for Nail No. 27. This neat-cement soil nail, joined with the adjacent soil nail (i.e. Nail No. 28) because of a drill rig misalignment. The nail was in otherwise defect-free condition. The Sonic Echo test correctly identified the location where the two grout columns joined and also accurately predicted the total length of 26-ft. These findings suggest that the Sonic Echo test has the capability measure grout column lengths of up to 25-ft under ideal conditions but this capability is greatly diminished when multiple defects are present at shallow depth or when the head conditions are poor.

Many useful conclusions could also be drawn from the research that examined the impact of grout consistency and placement parameters on the integrity of grout column. The findings from this phase of research highlighted the importance of having a construction crew knowledgeable on the use of proper construction procedures, equipment and material in soil nail installation projects. In this regard, specialty soil nail contractors can be expected to have an advantage over general contractors. The majority of contractors surveyed indicated that they use neat-cement grout for soil nail installation while others indicated preference for sand-cement grout. The findings from this phase of research showed that successful nail installation can be achieved with either type of grout. The following paragraphs summarize the key findings from phase two research.

Placement of Grout before Tendon: When neat-cement grout is used, the grout can be placed in the drillhole before the tendon with centralizers is inserted. When the grout is placed in the drillhole first, there is very little chance of having voids at the end of the hole or in and around centralizers. This option for placement of the grout may not be practical with sand-cement grouts that are too stiff.

Grout Tube or Pipe: When the tendon (with centralizers) is inserted into the drillhole first, it is necessary to use a tremie pipe to place the grout in the hole. The findings from this research highlight the importance of having full insertion of the tremie during grouting. In other words, complete filling of the borehole can be ensured only if the tremie pipe is inserted all the way to the bottom of the hole and is then withdrawn gradually as the hole is filled with grout. Tremie insertion depth becomes even more important as the grout becomes stiffer and nail angles become smaller. The stiffer grouts require larger diameter tremie pipes that may be more difficult to insert and withdraw through the limited space available. Stiffer grouts are also more likely to create voids in the grout column if the tremie pipe is withdrawn too fast.

Grout Pump: This research also showed that the selection of the grout pump must be made based on the consistency of the grout. Grouts containing aggregate will require the use of a standard concrete pump. This type of pump allows a high rate of grout placement. However, when neat cement grouts are used pumps that provide fast volumetric flow rates may cause numerous air voids to form in the grout column. Therefore, grout pumps with slower volumetric flow rates must be used with neat cement grouts.

Bird’s Beak Defects: Bird’s beak defects are more difficult to avoid when neat cement grouts are used. Bird’s beak defects become a specially challenging problem when the nail angles are small. Some contractors cited this as the primary reason for their preference for sand-cement grouts. Special procedures are necessary to ensure that the drillholes are completely grouted near the nail head. Such procedures may include: grouting in multiple stages, use of packers or soil dam temporarily to retain grout until it has set.

Grout/Tendon Bond and Grout Column Integrity: Findings from the exhumation of soil nails showed that the sand-cement grout provided a better grout-tendon bond. Also, the sand-cement grout columns held together better as the surrounding soil was removed and the nails were pulled out than neat cement grout columns. It will be interesting to find out whether the weaker grout-tendon bond strength and grout column strength found in neat cement grouts result in a decrease in nail pull-out capacity. However, this research plan did not include any soil nail pull out tests and, therefore, no conclusions can be drawn regarding pull capacities obtained from the two types of grout.

Measurement of Grout Consistency/Flowability: As a general rule, soil nail grout mix specifications do not include test standards that measure fluidity of the grout. Instead, the specifications rely on advance approval of the contractor's grout mix design used by the engineer. Nevertheless, the findings from this research suggest that there is merit in introducing a requirement based on grout fluidity. The objective of this requirement is to avoid the use of grout mixes that are very stiff. A slump flow of 20-in diameter is proposed as a suitable threshold to be used initially for sand-cement grout mixes.

7.3 Recommendations for Implementation

The implementation of the findings from this research project should focus on the following three areas: (a) Providing necessary training to TxDOT inspectors and construction crews, (b) Non-destructive test protocol to evaluate soil field installed nails and appropriate acceptance/rejection criteria, (c) A test procedure to determine fluidity of grout mixes and minimum/maximum acceptance limits. Although this research project laid much of the necessary groundwork towards achieving these goals, the researchers feel that it will be desirable to initiate an implementation project involving several pilot projects before final implementation.

The proposed scope for such an implementation project is as follows. At the outset, 8-10 TxDOT soil nail wall construction projects will be selected to be included in the implementation project as pilot projects. These projects should include soil nail walls to be constructed by specialty contractors as well as general contractors. Consideration should also be given to the type of grout mix used by each contractor. In other words, the entire range of grout mixes used by TxDOT soil nail wall contractors, from very stiff to very flowable, should be represented in the implementation project. Grout mix designs, strength and fluidity measurements and the variability of these properties (from one grout batch to another) must be recorded. The findings from this research study favor the use of a grout fluidity measurement test method such as slump flow (ASTM C-1611) or flow cylinder (ASTM D-6103). A threshold value of slump flow of 20-in based on ASTM C-1611 is recommended for initial implementation. As a part of the proposed implementation project, however, the measurement of grout fluidity must be made using more than one test procedure (slump, slump flow, flow cone and V-funnel efflux time etc). Details of construction procedures and equipment used (grout pump, tremie pipe) will be documented. The entire construction procedure will be video taped with the objective of developing a training video for future use.

Sonic Echo tests will be conducted on test nails that are installed at each jobsite as well as on randomly selected production nails. These tests will be conducted by consultants from Olson Engineering Company. These NDT test series will serve the dual purpose of training TxDOT engineers and inspectors and, at the same time, help build a database that is necessary for establishing criteria for acceptance and rejection of soil nails. Nails that are deemed to be of questionable quality would be load tested to determine their load carrying capacity and compared with capacities of test soil nails. If the results show that nail loads do not meet design values then the contractor would be required to install additional nails.

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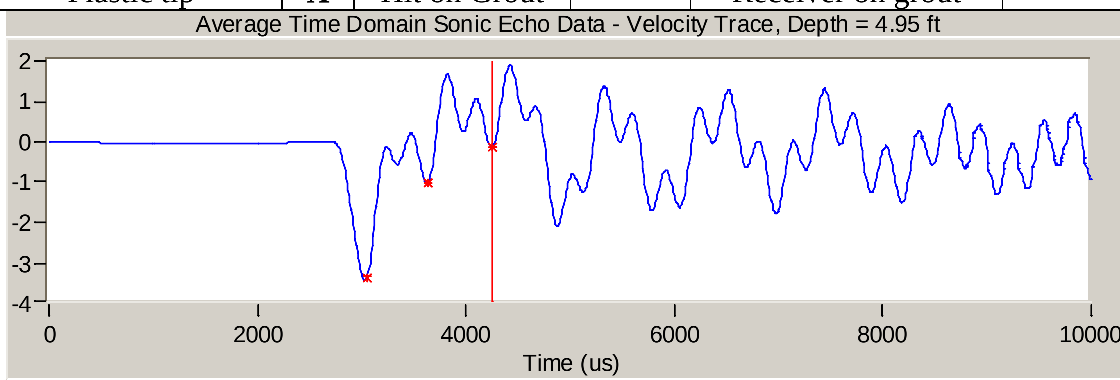
APPENDIX

Table A.1--Nail No.1

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
5	Sand Cement	No defect	N/A

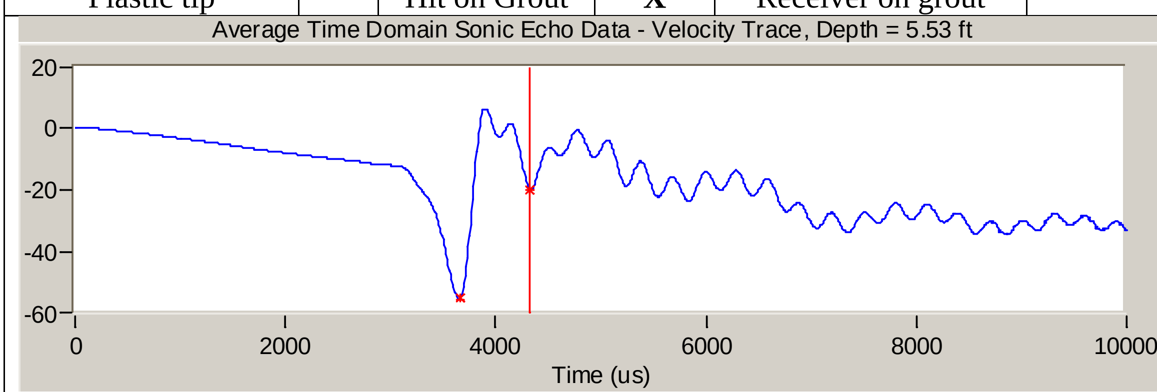
Head Condition		
First series of testing	Second series of testing	Third series of testing – not performed due to access
		

Field Testing Results: Sonic Echo—1st testing series					
Metal tip		Hit on bar	**X**	Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout		Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 4.95 ft

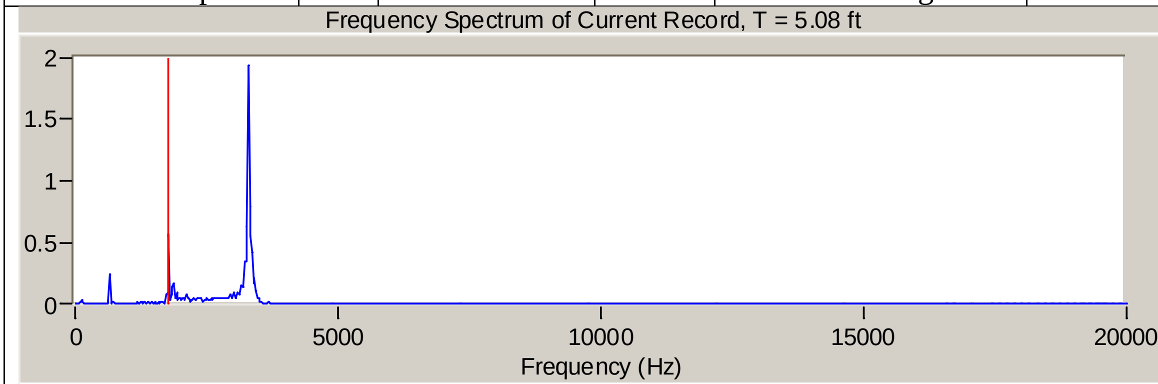
Sonic Echo—1st testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 5.53 ft

Impact Echo—1st testing series

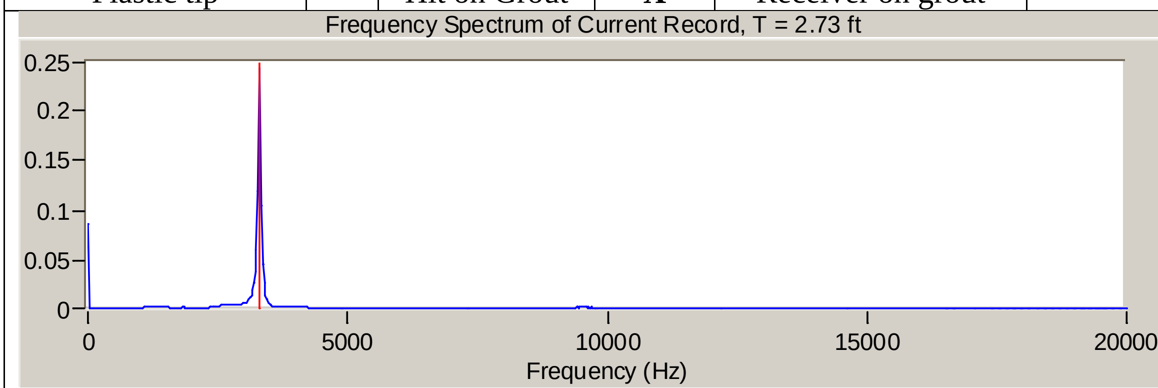
Metal tip		Hit on bar	X	Receiver on bar	**X**
Plastic tip	X	Hit on Grout		Receiver on grout	



Wave Velocity=16500 ft/sec, peak at 2.75 and 5.08 ft

Impact Echo—1st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, peak at 2.73 ft

NDT conclusion

IE and SE tests can detect the end of the nail of Nail 1

Note: Check for possibly minor void at 2.7 feet – detected in IE data

Exhumation Results

Exposed Nail #1
Nail ends at 5 feet

Table A.2--Nail No.2

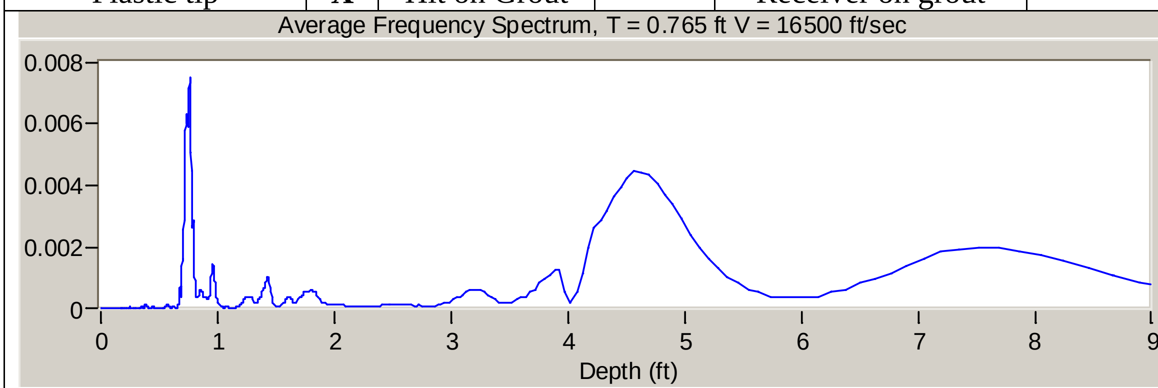
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
5	Neat Cement	No defect	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Impact Echo—1st testing series

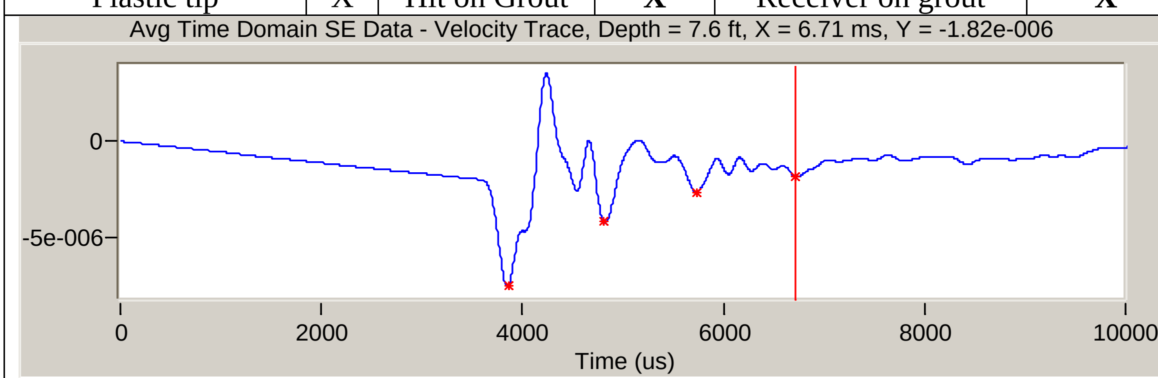
Metal tip		Hit on bar	X	Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout		Receiver on grout	



Wave Velocity=16500 ft/sec, peak at 4.69 ft

Sonic Echo – 2nd Series

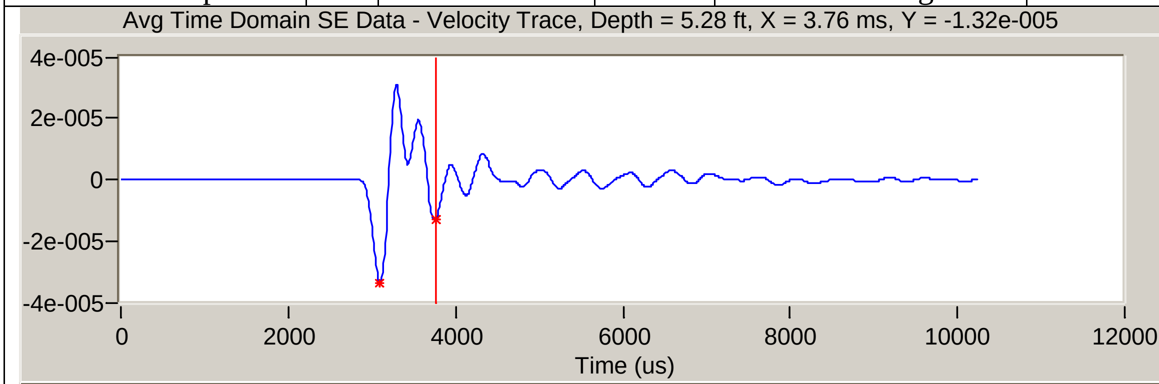
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 7.6 ft

Sonic Echo – 3rd Series

Metal tip	X	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	X	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 5.28 ft

NDT conclusion

SE tests detect the nail end at 5.28 feet
2nd Series SE tests detect the nail end at 7.6 ft
IE detect shallow defect at 0.76 ft and the end at 4.41 ft

Exhumation Results

Exposed Nail #15
Nail ends at 5.5 feet, bad head condition

Table A.3--Nail No.3

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
5	Sand Cement	@ End	2.7-4.2

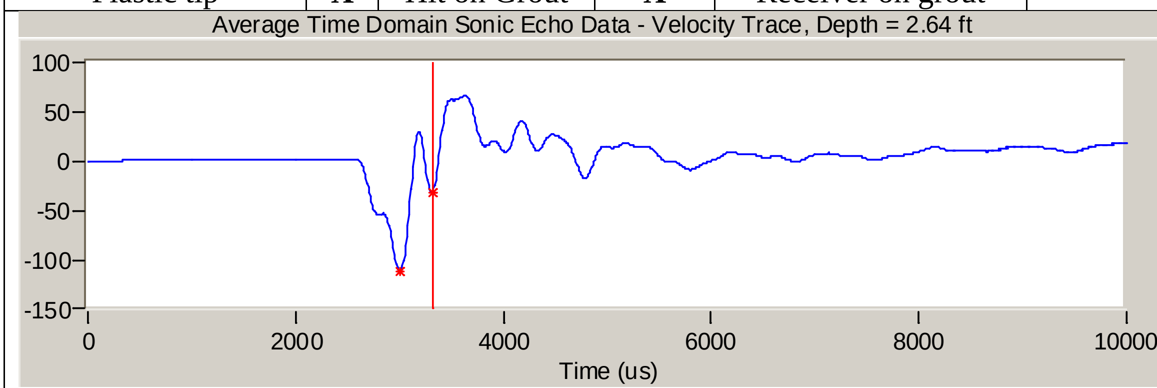
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Sonic Echo—1st testing series

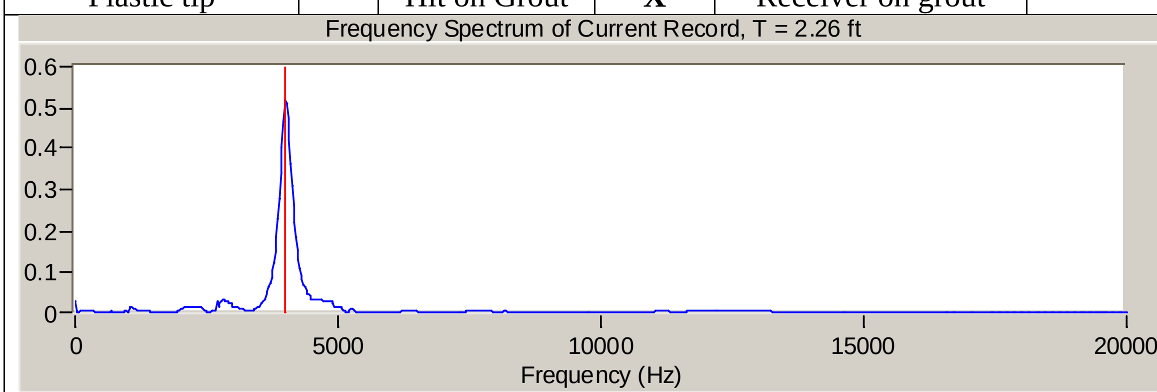
Metal tip		Hit on bar		Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 2.64 ft

Impact Echo—1st testing series

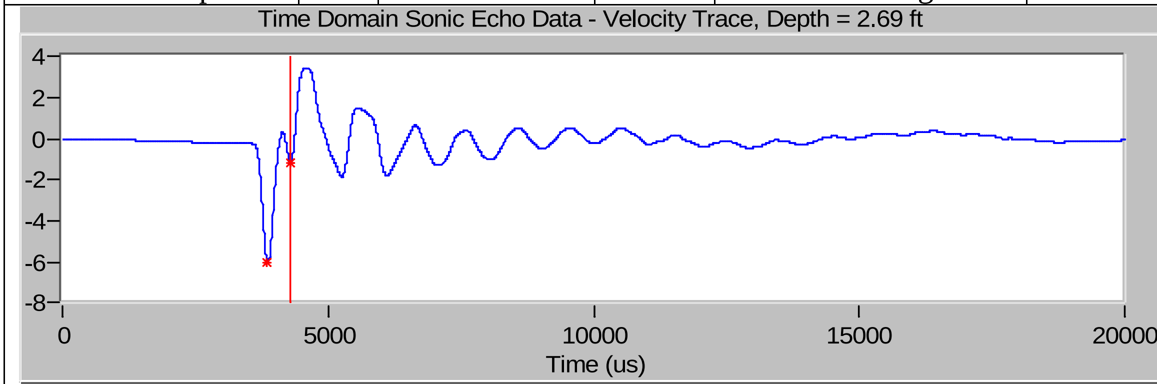
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 2.26 ft

Sonic Echo—2st testing series

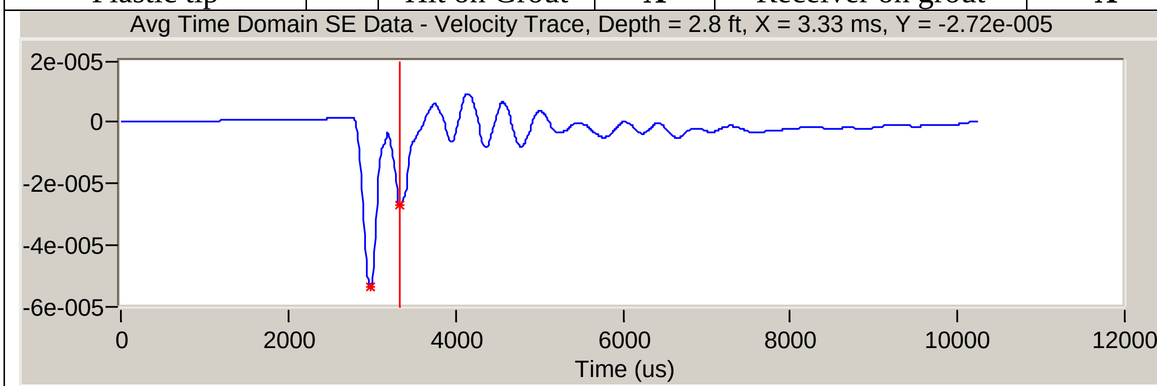
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	X	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 2.69 ft

Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16500 ft/sec, echo at 2.8 ft

NDT conclusion

SE tests detect defect at 2.64-2.8 feet, IE test detects defect at 2.26 feet

Exhumation Results

Exposed Nail #3

Nail as designed condition, grout fill to middle foam at 2.5 feet

Table A.4--Nail No.4

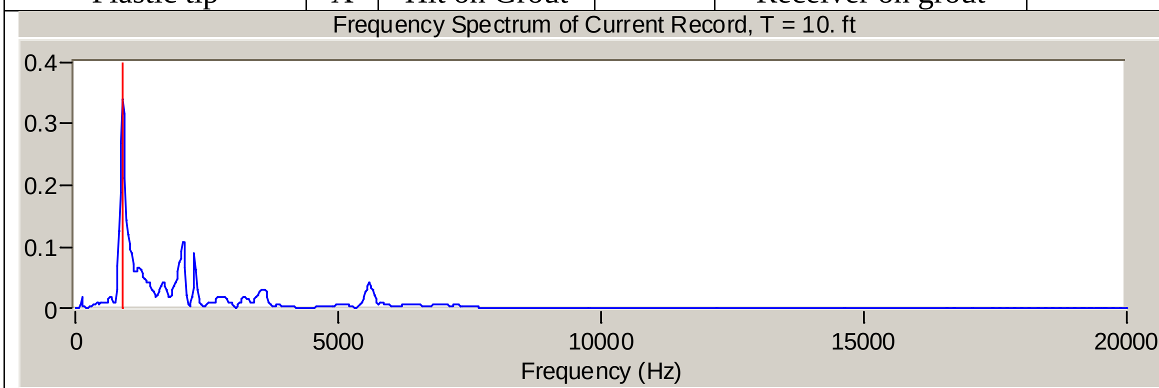
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
10	Sand Cement	No defect	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Impact Echo—1st testing series

Metal tip		Hit on bar	X	Receiver on bar	**X**
Plastic tip	X	Hit on Grout		Receiver on grout	

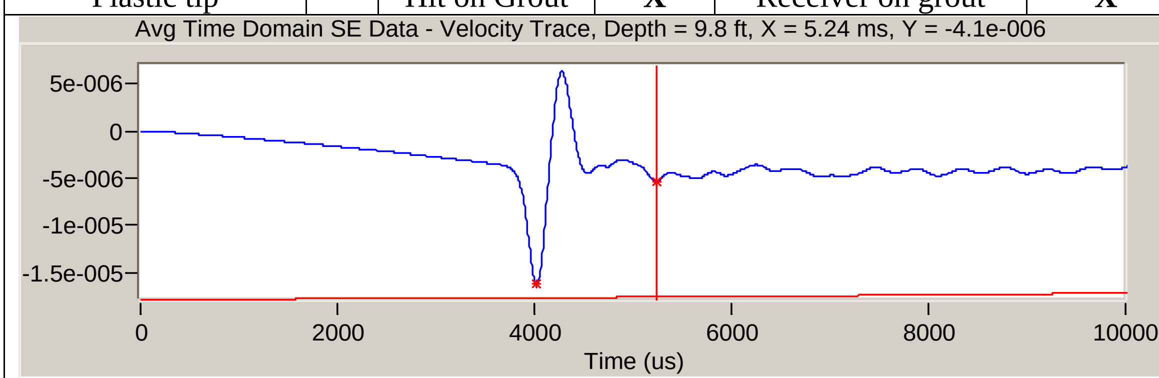


Wave Velocity=16500 ft/sec, echo at 10.0 ft

Field Test Results

Sonic Echo—2nd testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**

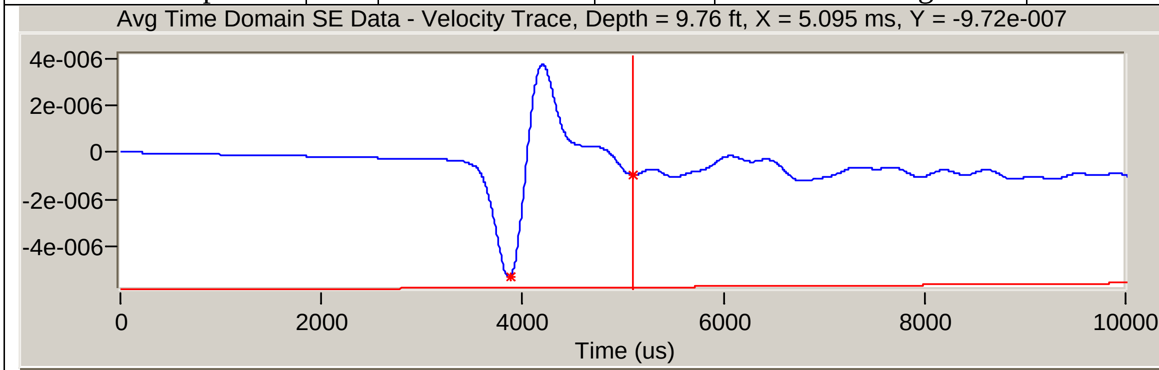


Wave Velocity=16000 ft/sec, echo at 9.8 ft

Sonic Echo—2nd testing series

Metal tip		Hit on bar		Receiver on bar	

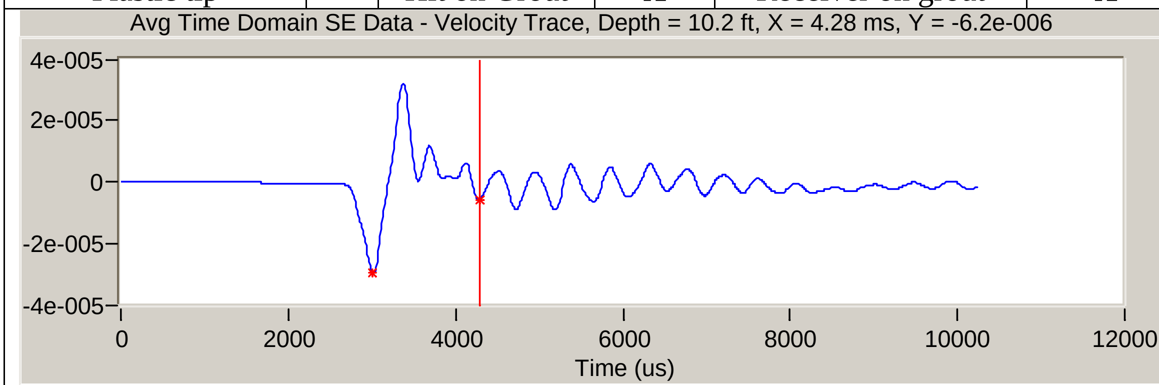
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 9.76 ft

Sonic Echo—3rd testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 10.2 ft

NDT Conclusion

2nd SE tests detected the end of nail at 9.8 feet
3rd SE tests detected the end of nail at 10.2 feet
IE test detect the end of nail at 10.0 ft

Exhumation Results

Exposed Nail #4
Bad head condition, only 2/3 of nail head is grouted

Nail ends at 10’2”

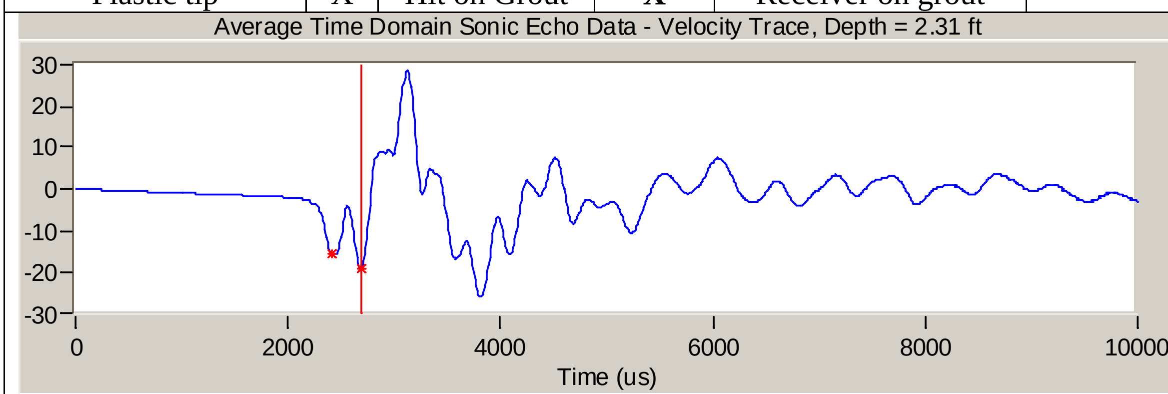
Table A.5--Nail No.5

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
10	Neat Cement	No defect	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results
Sonic Echo—1st testing series

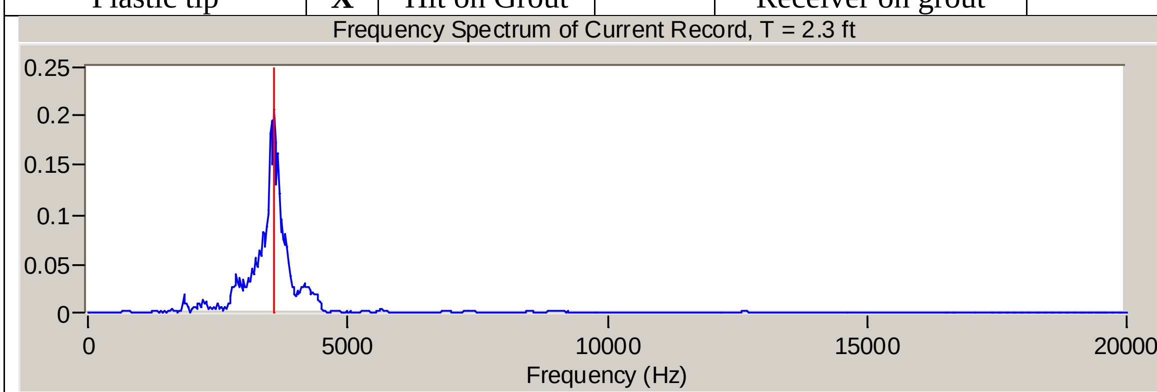
Metal tip		Hit on bar		Receiver on bar	**X**
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 2.31 ft

Field Testing Results
Impact Echo—1st testing series

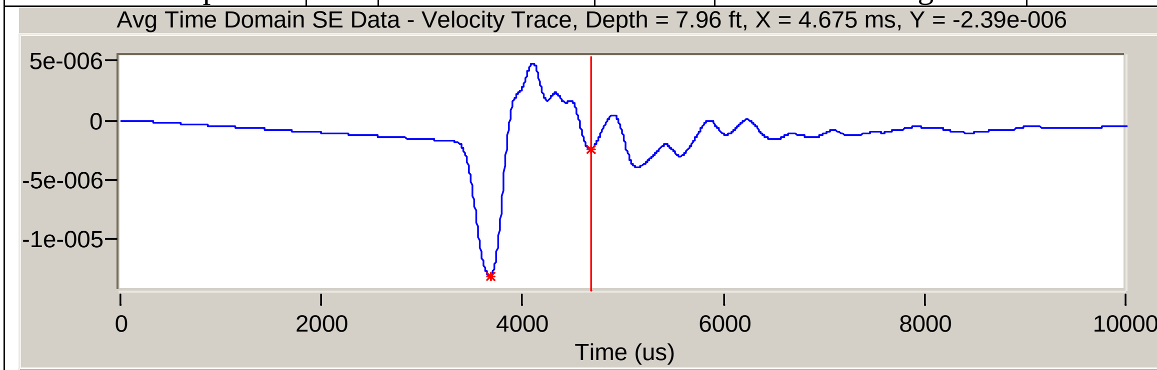
Metal tip		Hit on bar	X	Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout		Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 2.3 ft

Sonic Echo—2nd testing series

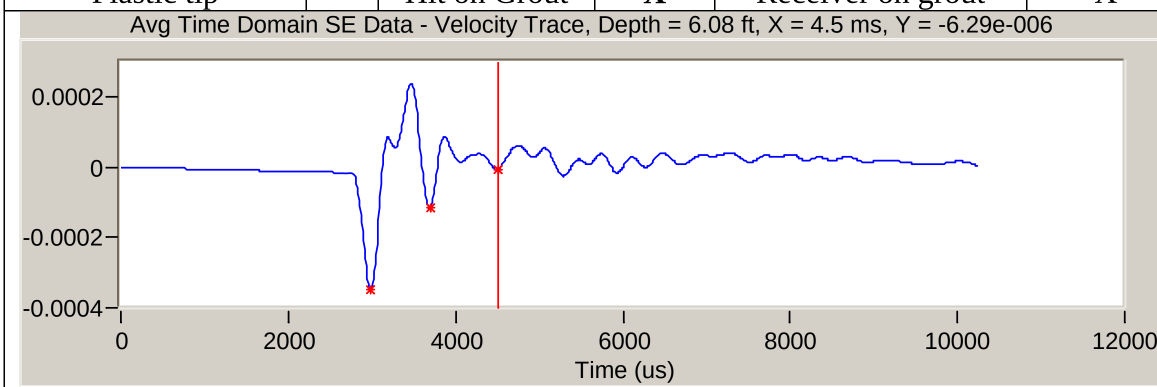
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 7.96 ft

Sonic Echo—3rd testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 6.08 ft

NDT conclusion

SE Test from the 1st series detected a shallow defect at 2.3 ft due to a bad head condition. Then the re-grouting process was performed to fill up the unplanned birdsmouth

SE test from the 2nd series detected a neck at 7.96 ft

SE tests from the 3rd series detected a defect at 6.08 ft

Exhumation Results	
 Exposed Nail #5 Bad head condition, only 1/2 of nail head is grouted	 Nail ends at 10’
 Minor Defect (area reduction) at 7’	 Crack at 9’1’’

Table A.6--Nail No.6

Designed Condition

Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
10	Sand Cement	@end	7.8-10.3

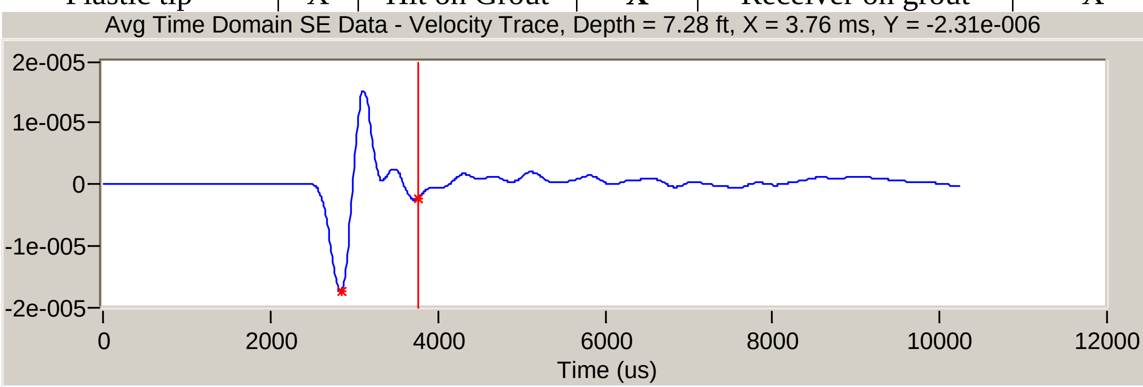
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 7.28 ft

NDT conclusion

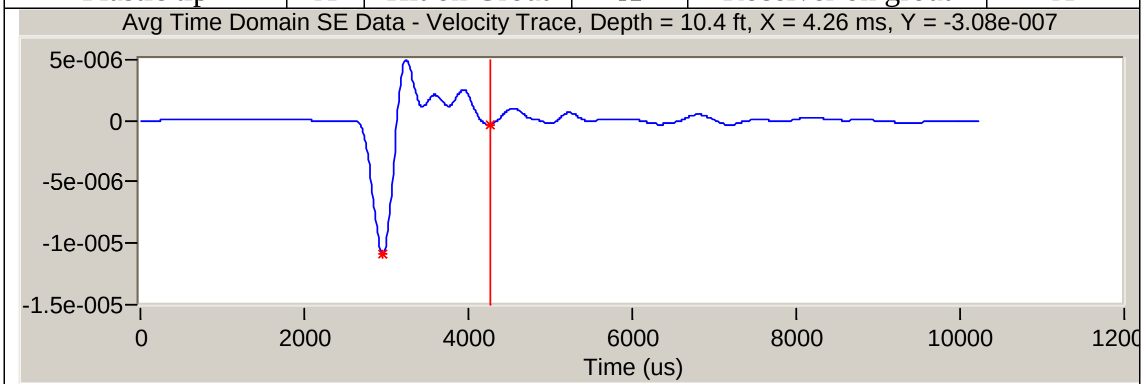
SE tests detect defect at 7.28 feet,

Exhumation Results

	
Exposed Nail #6	Grout ends at 7'

Table A.7--Nail No.7

Sonic Echo—3st testing series					
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 10.4 ft

NDT conclusion

1st SE tests detected the end of nail at 14 ft
2nd SE tests detected the end of nail at 12 ft
3rd SE tests detected the end of nail end or defect at 10.4feet.

Exhumation Results

 Exposed Nail #7	 Grout ends at 13’8”
 Grout actually starts reducing from 10’ and ends at 13’8”	

Table A.8--Nail No.8

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
15	Neat Cement	No defect	N/A

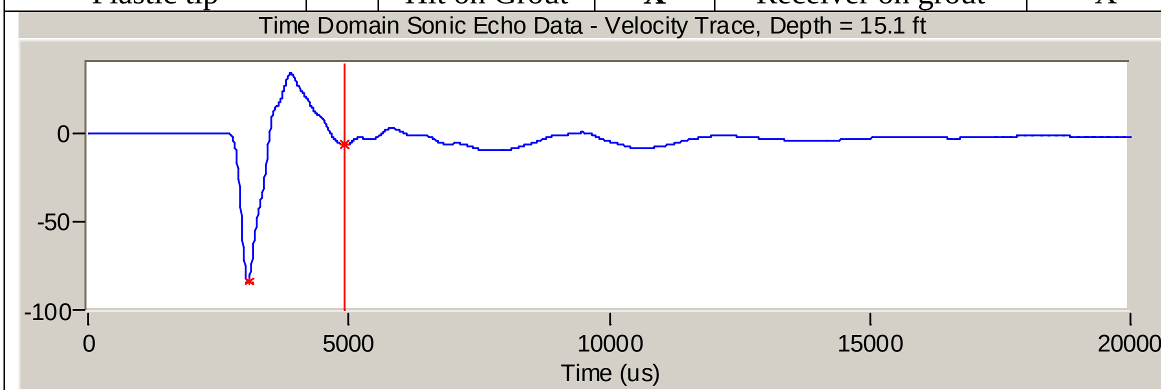
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—1st testing series

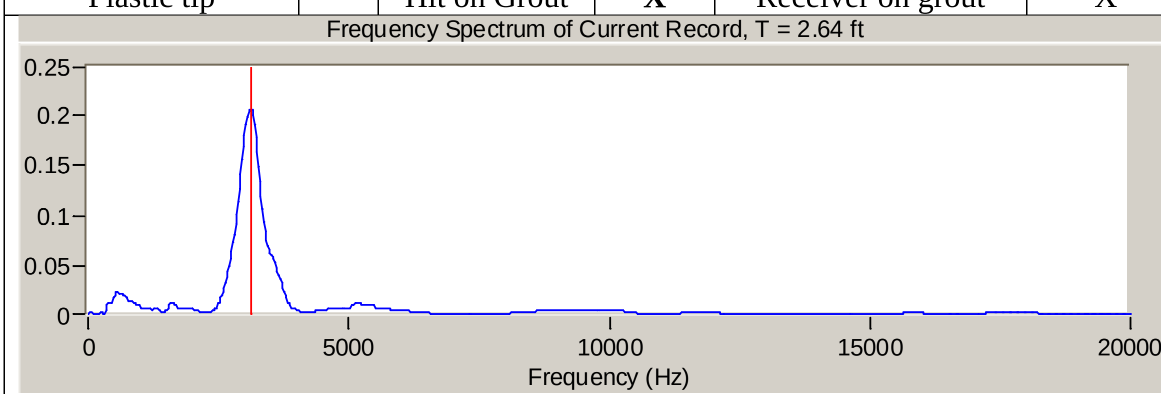
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 15.1 ft

Impact Echo—1st testing series

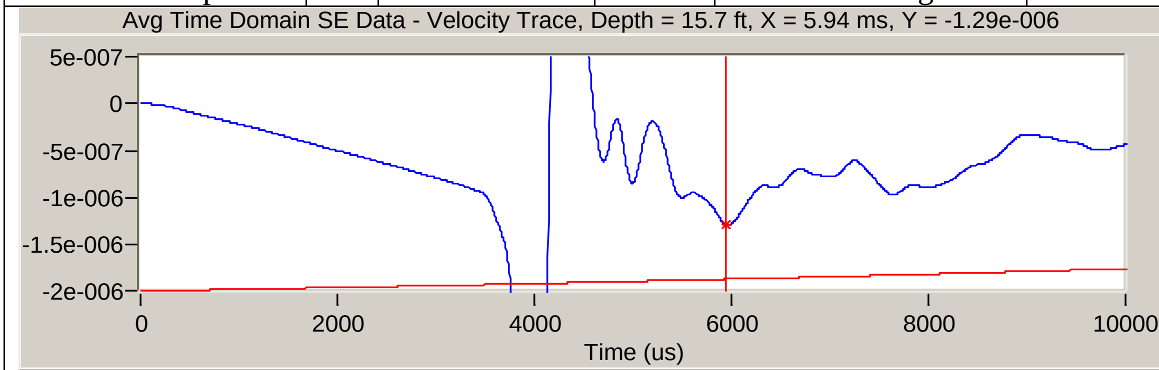
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 2.64 ft

Sonic Echo—2nd testing series

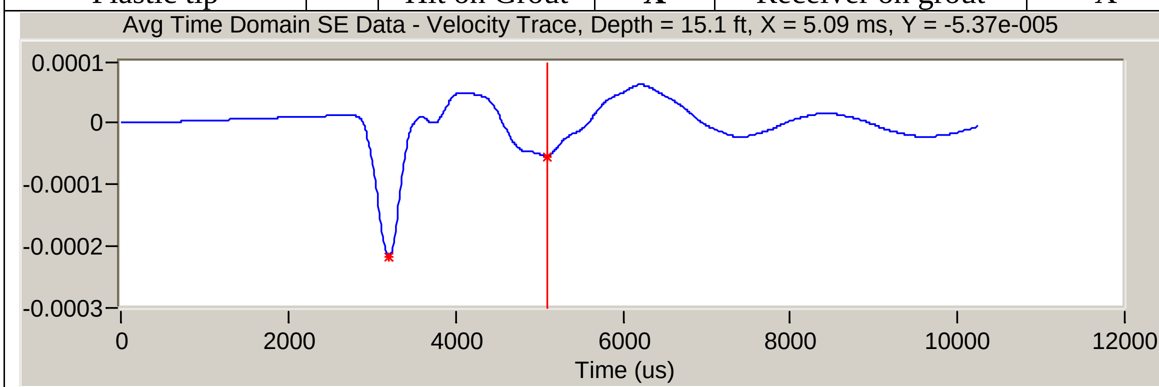
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 15.7 ft

Sonic Echo—3st testing series

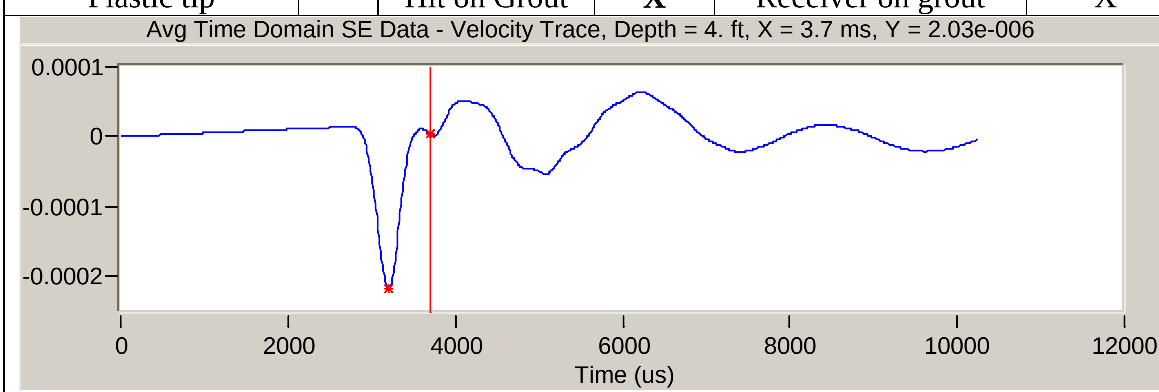
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 15.1 ft

Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 4.0 ft

NDT conclusion

1st SE detect the end at 15.1 ft and the IE detect a shallow defect at 2.64 ft
2nd SE tests detect a weak end at 15.7 ft
3rd SE tests detect a minor defect at 4.0 ft and an end at 15.1 ft

Exhumation Results

Exposed Nail #8

Minor defect at 3 ft

Minor defect around 4 ft

Grout ends at 14.7 ft

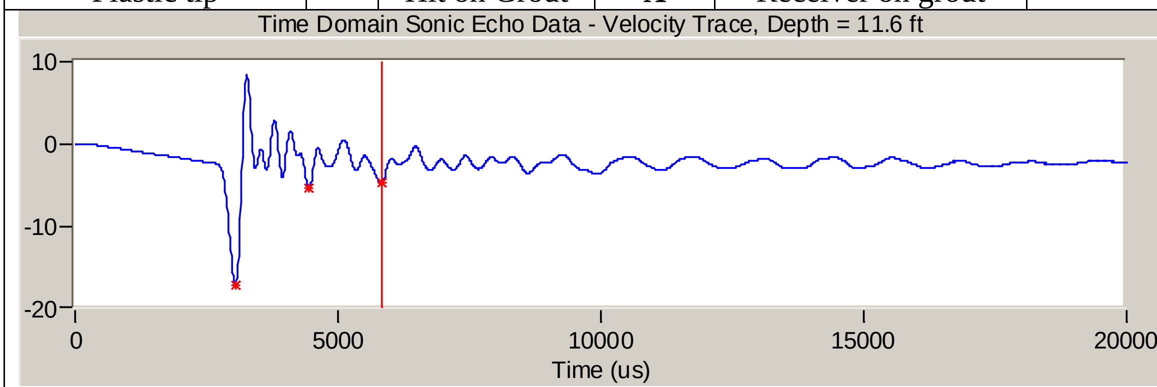
Table A.9--Nail No.9

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
15	Sand Cement	@end	10.5-14

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Sonic Echo—1st testing series

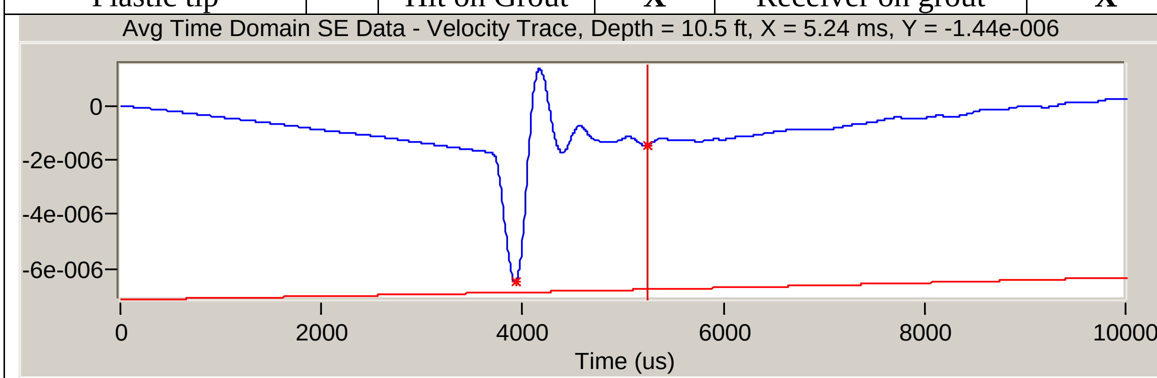
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 11.6 ft

Sonic Echo—2nd testing series

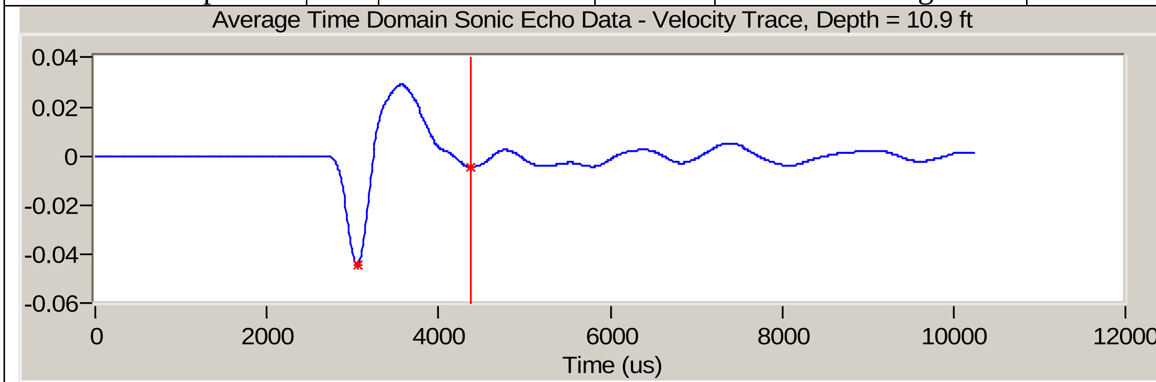
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 10.5 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	X	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 10.9 ft

NDT conclusion

SE tests detect defect at 10.5-11.6 feet

Exhumation Results


Exposed Nail #9

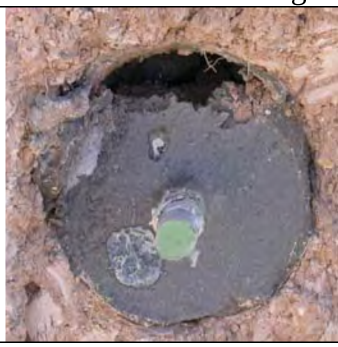

Grout ends at 10’8”


Minor defect at 5’5”


Minor defect at 8’2”

Table A.10--Nail No.10

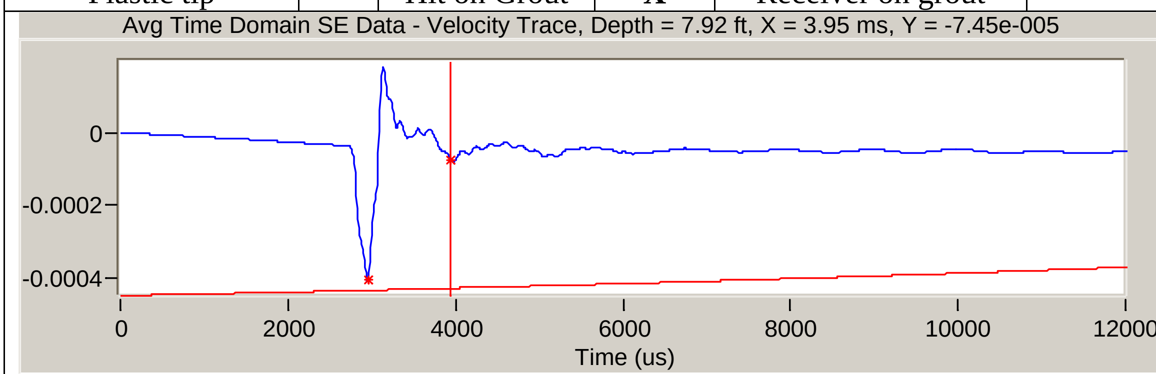
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
15’	Sand Cement	@middle	6.2-8.2

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—1st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	

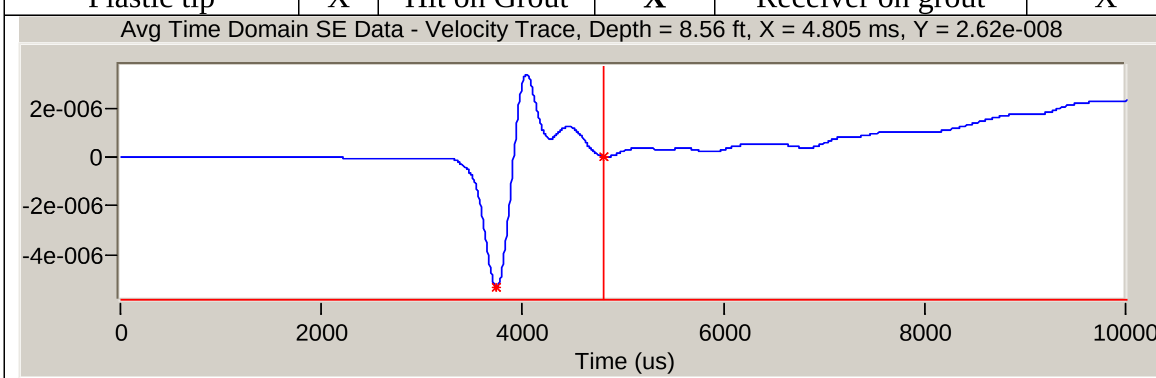


Wave Velocity=16000 ft/sec, weak echo at 7.92 ft

Field Test Results

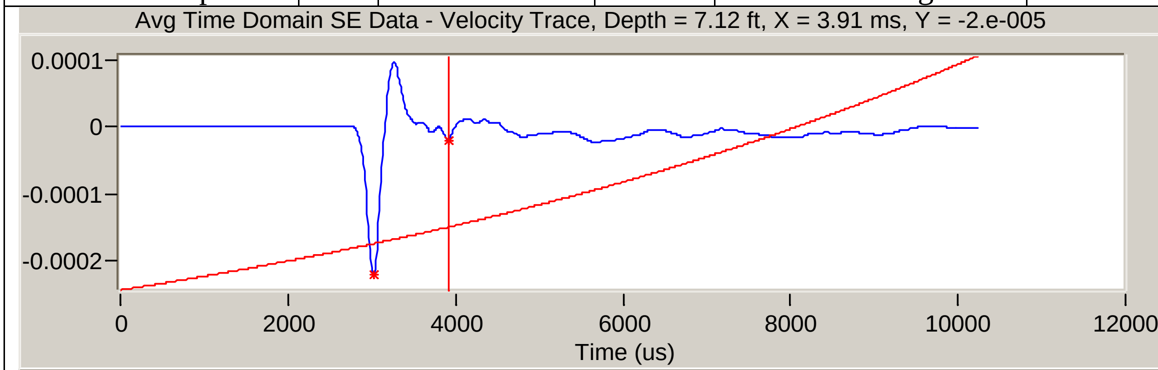
Sonic Echo—2nd testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, weak echo at 8.56 ft

Field Test Results **Sonic Echo—3rd testing series**					
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, weak echo at 7.12 ft

NDT Conclusion
SE tests detected defects at 7.12 – 7.92 ft 2nd SE tests detected defects/end at 8.56 ft

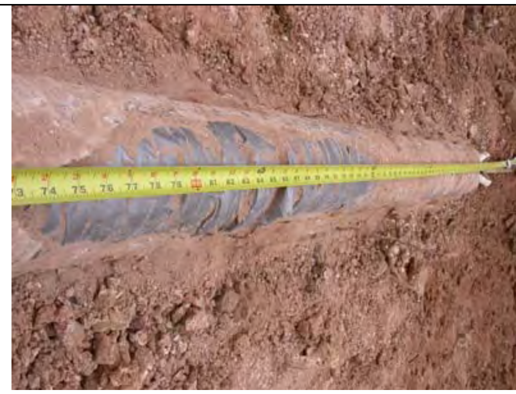
Exhumation Results	
 Exposed Nail #10	 Grout goes through the middle defect, ends at 11’4’’
 Grout meets foam at 6’	

Table A.11--Nail No.11

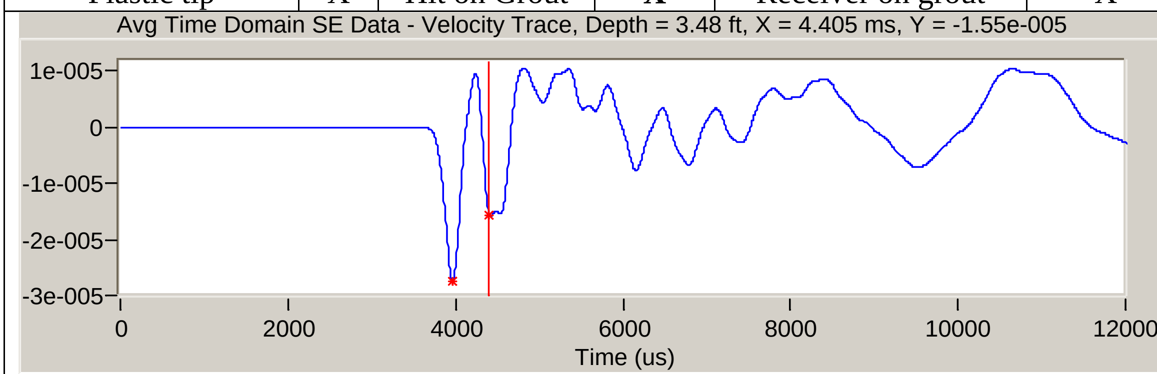
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
15	Sand Cement	Bird mouth	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—2nd testing series

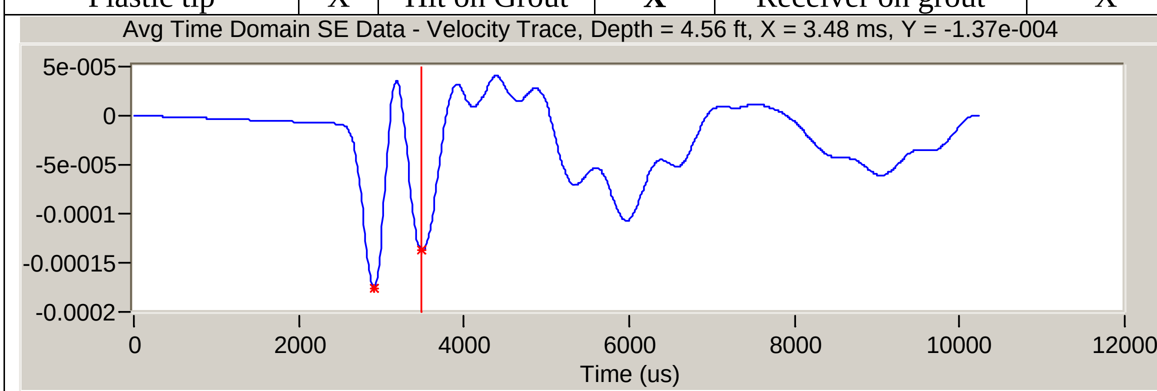
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 3.48 ft

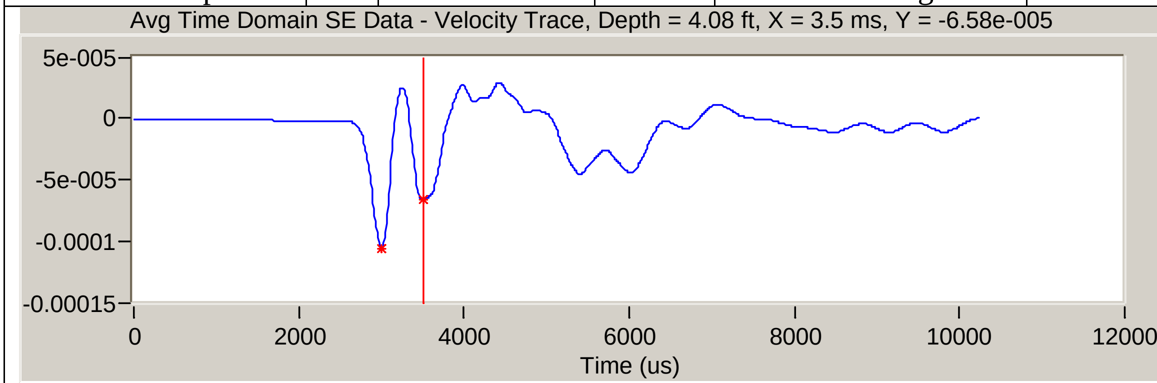
Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 4.56 ft

Sonic Echo—3st testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 4.08 ft

NDT conclusion
1st SE test detect end at 16.7 ft 2nd SE test detected a neck at 3.48 ft 3rd SE tests detect defect at 4.56 ft

Exhumation Results	
 Exposed Nail #11	 Bird mouth
 Big cave-in from 11’2” to 12’	 Grout ends at 14’5”

Table A.12--Nail No.12

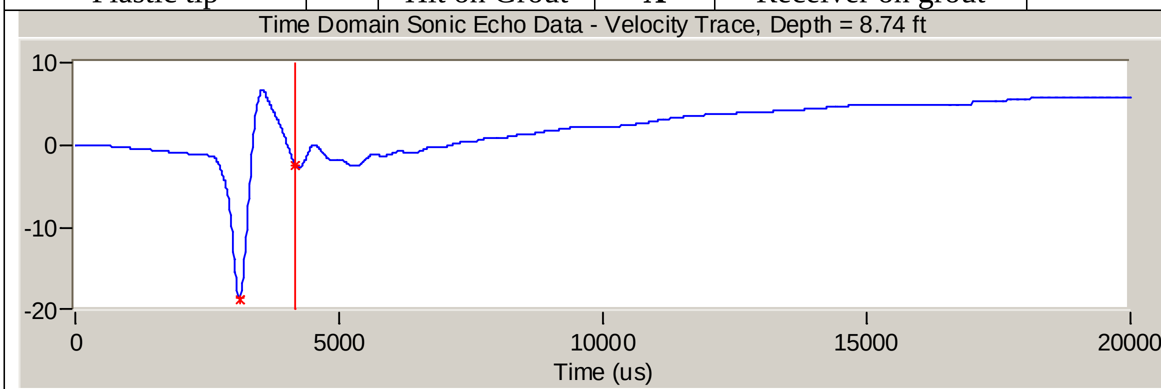
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
15	Neat Cement	@middle	5.9-7.9

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—1st testing series

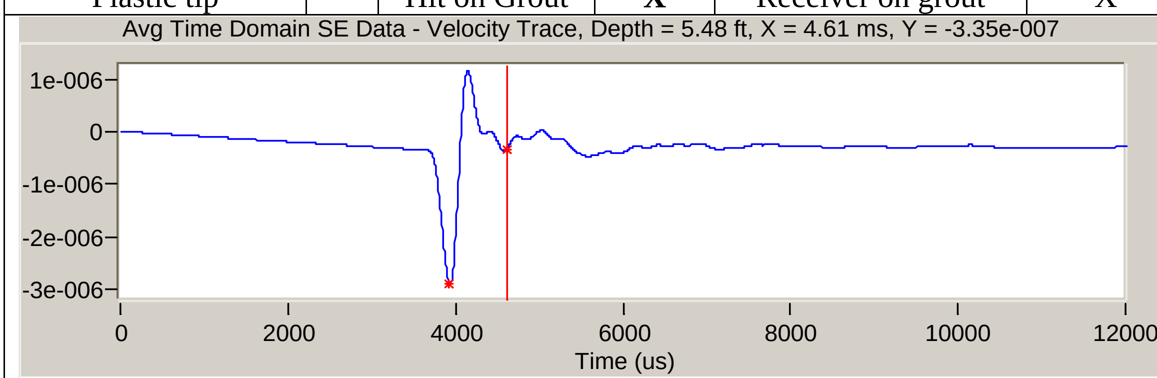
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



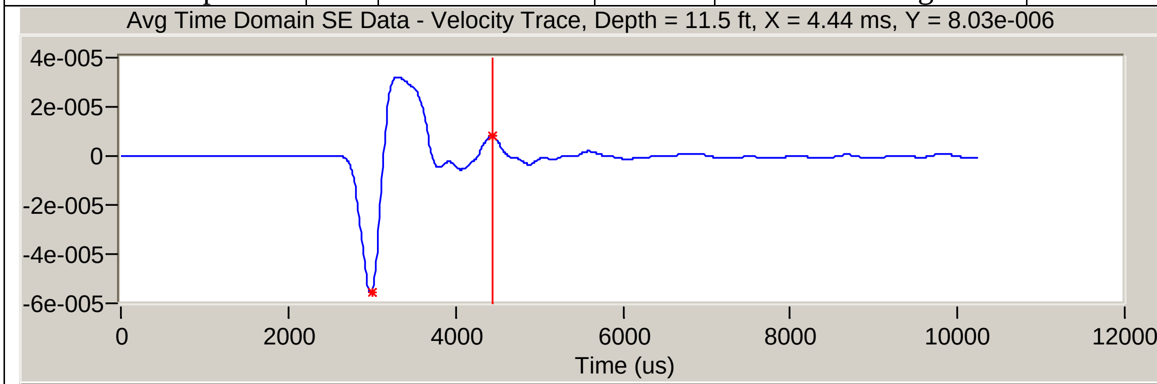
Echo at 8.74 ft

Sonic Echo—2nd testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Neck at 5.48ft

<table>
<tr><th colspan="6">Sonic Echo—3st testing series</th></tr>
<tr><td>Metal tip</td><td>X</td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td></td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6">

Bulb at 11.5ft</td></tr>
<tr><th colspan="6">NDT conclusion</th></tr>
<tr><td colspan="6">1st SE test detected a neck at 8.74 ft
2nd SE test detected a neck at 5.48 ft
3rd SE tests detect a bulb (increase in diameter) at 11.5 ft</td></tr>
</table>

Exhumation Results	
 Exposed Nail #12	 Grout passes through the middle foam, ends at 15’2”
 Minor defect at 11’1”	 Minor defect at 8’8”
 Grout passes through foam	

Table A.13--Nail No.13

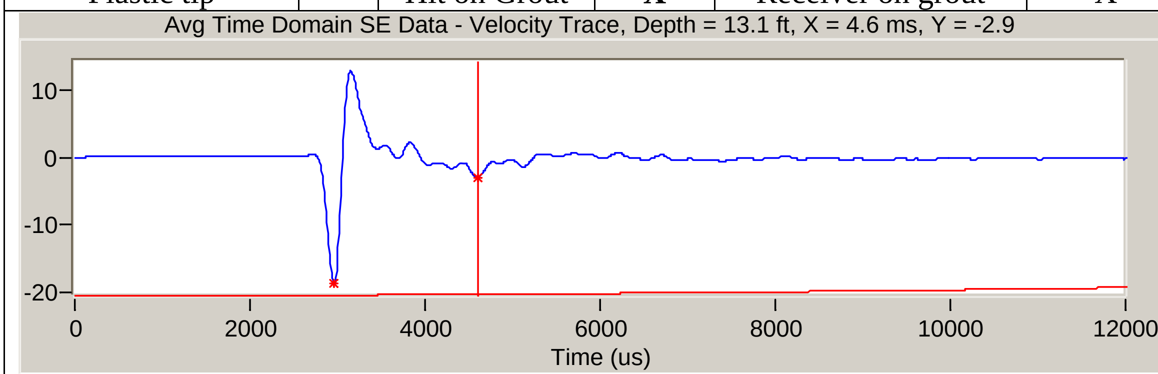
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Sand Cement	No defect	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—1st testing series

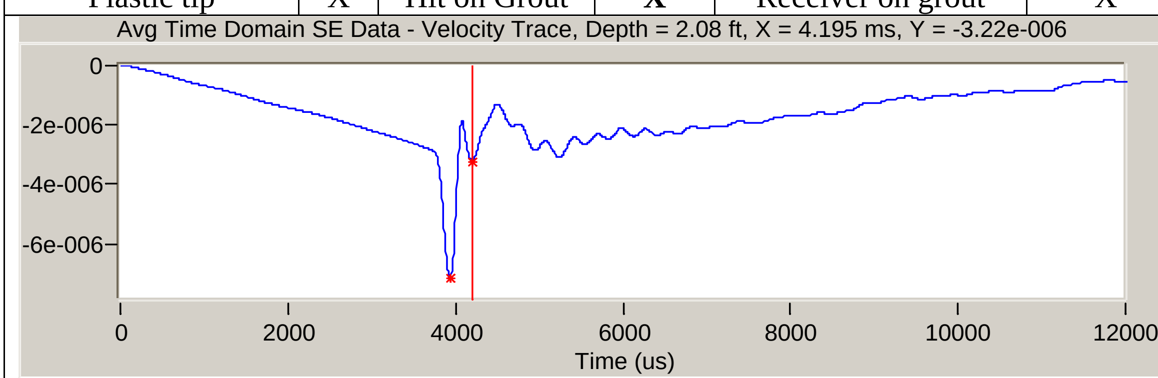
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, neck echo at 13.1 ft

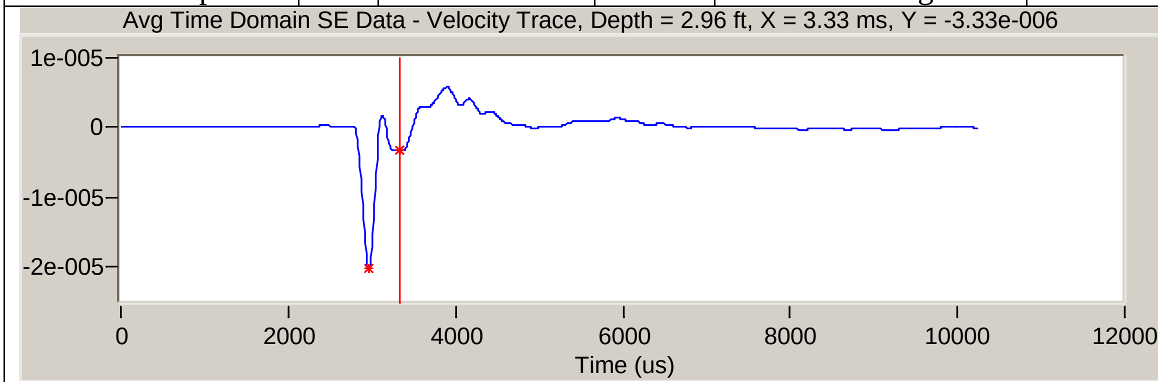
Sonic Echo—2nd testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, neck echo at 2.08 ft

Sonic Echo—3rd testing series					
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, neck echo at 2.96 ft

NDT conclusion

1st SE tests detected defect/end at 13.1 ft
3rd SE tests detect defect at 2.96 ft

Exhumation Results


Exposed Nail #13


Grout ends at 14’9”


Minor defect around 3’3”-4’8” area


The cross section starts reduction at 13’

Table A.14--Nail No.14

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Neat Cement	No defect	N/A

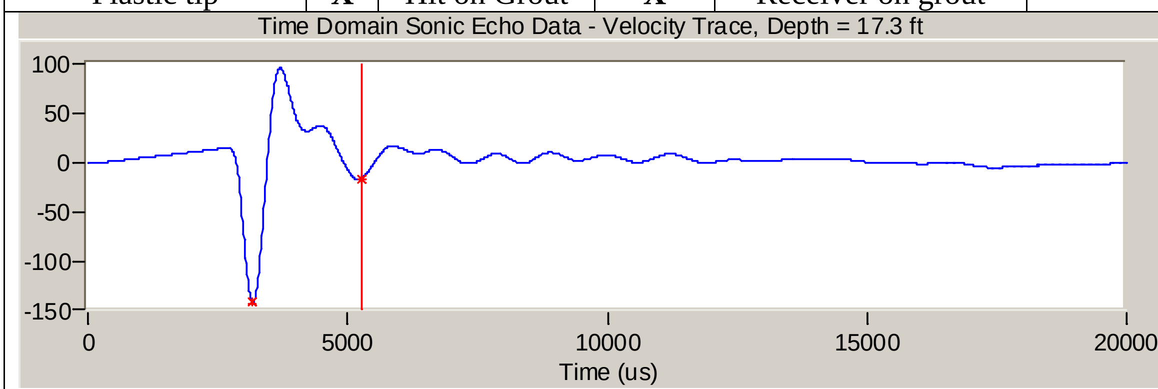
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Sonic Echo—1st testing series

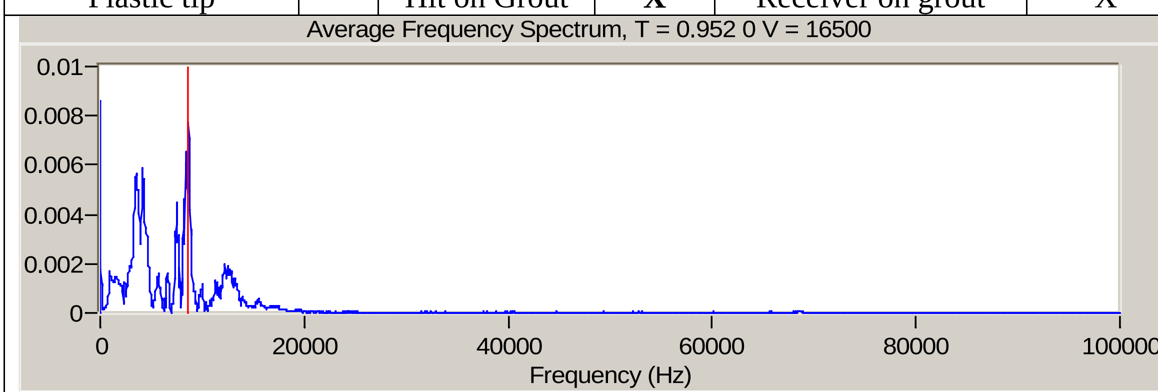
Metal tip		Hit on bar		Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 17.3 ft

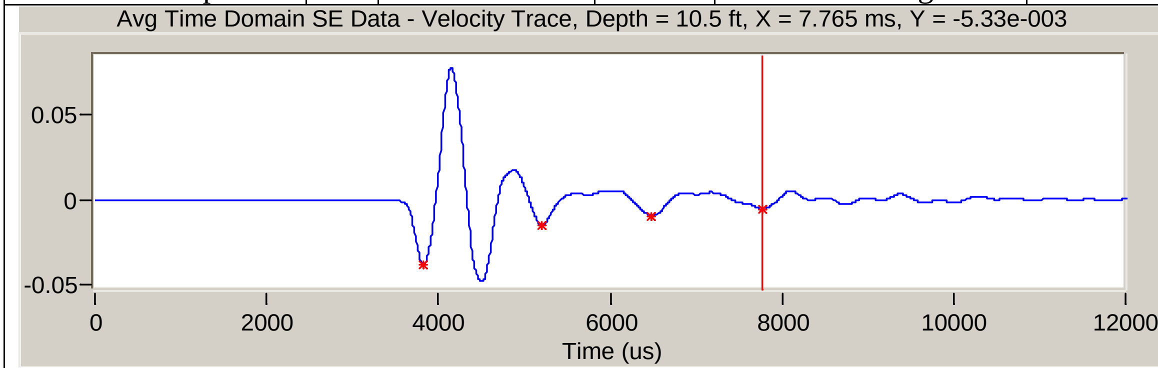
Impact Echo—1st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



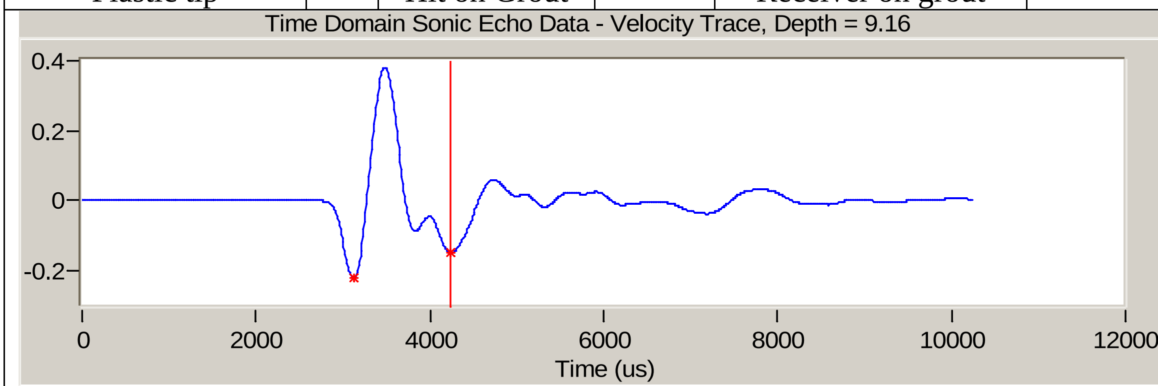
Wave Velocity=16500 ft/sec, peak at 0.92 and 2.17 ft

Sonic Echo—2nd testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 10.5 ft

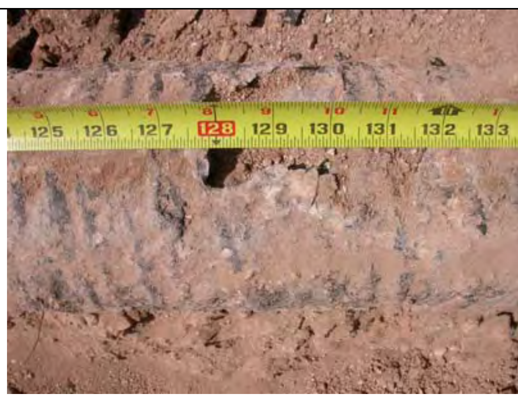
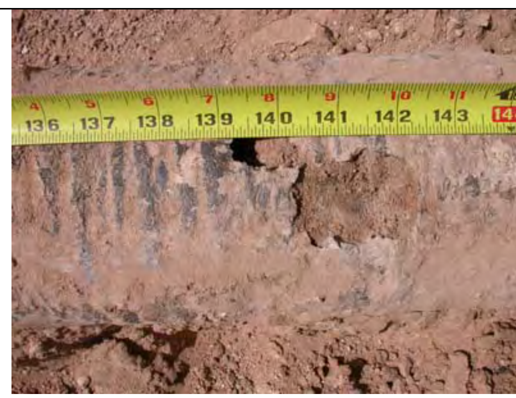
Sonic Echo—3st testing series					
Metal tip	**X**	Hit on bar	**X**	Receiver on bar	**X**
Plastic tip		Hit on Grout		Receiver on grout	



Wave Velocity=16000 ft/sec, echo at 9.16 ft

NDT conclusion
SE tests detect multiple defects at 8 feet, 9 feet, 10 feet, 12 feet and 17feet. 2nd SE test detected an end at 21.1 ft IE tests detect shallow defects at 1 and 2.17 feet.

Nail No.14

Exhumation Results	
 Exposed Nail #14	 Grout ends at 20’9”
 Minor defect at 1’2”	 Minor defect at 2’5”
 Minor defect at 10’8”	 Minor defect at 11’8

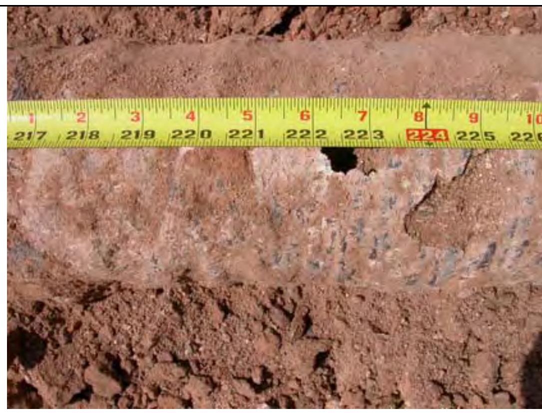
Nail No.14 (cont.)	
 Minor defect at 12’10”	 Minor defect at 13’8”
 Minor defect at 14’9”	 Minor defect at 17’2”

Table A.15-Nail No.15

Designed Condition

Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Sand Cement	@end	14.5-19.0

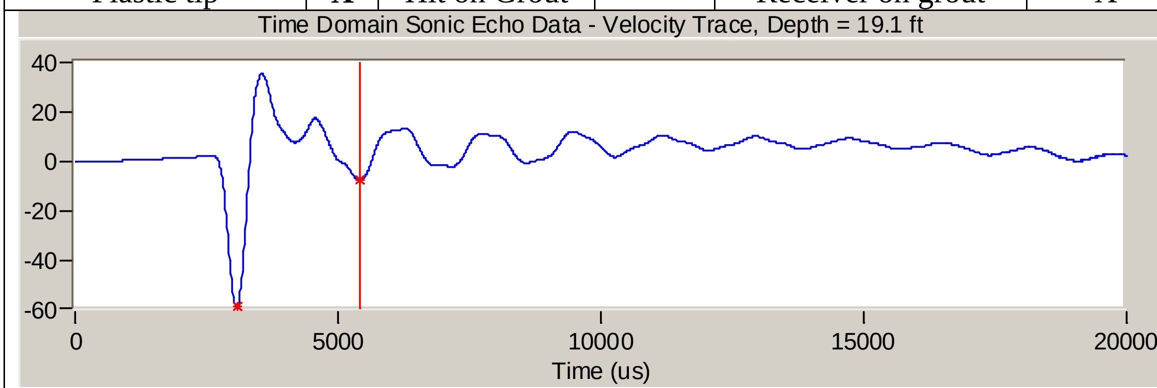
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Sonic Echo—1st testing series

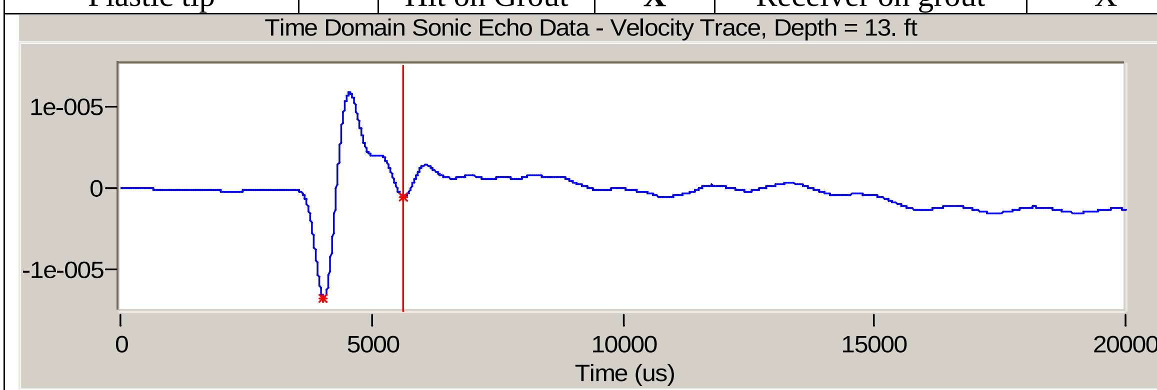
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	**X**	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 19.1 ft

Sonic Echo—2st testing series

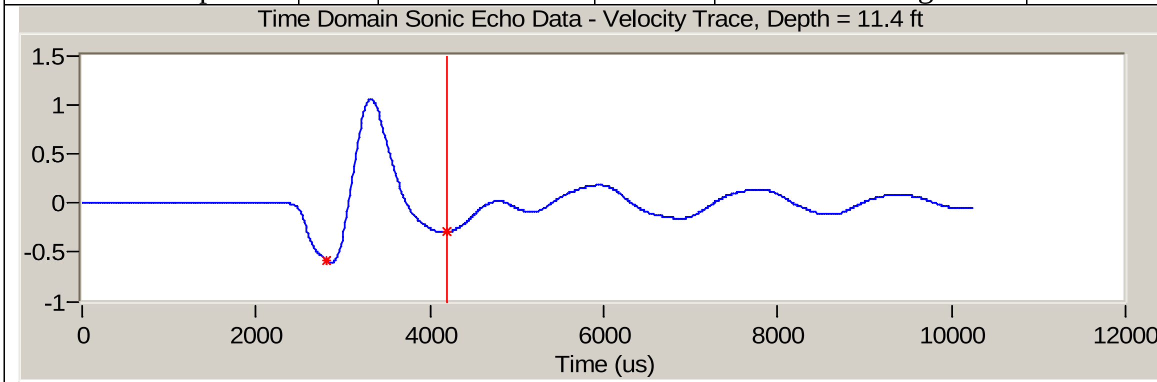
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 13 ft

Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 11.4 ft

NDT conclusion

SE tests detect defect at 13-16.3 feet.

Exhumation Results


Exposed Nail #15


Grout fill to foam, ends at 14’5”

Table A.16--Nail No.16

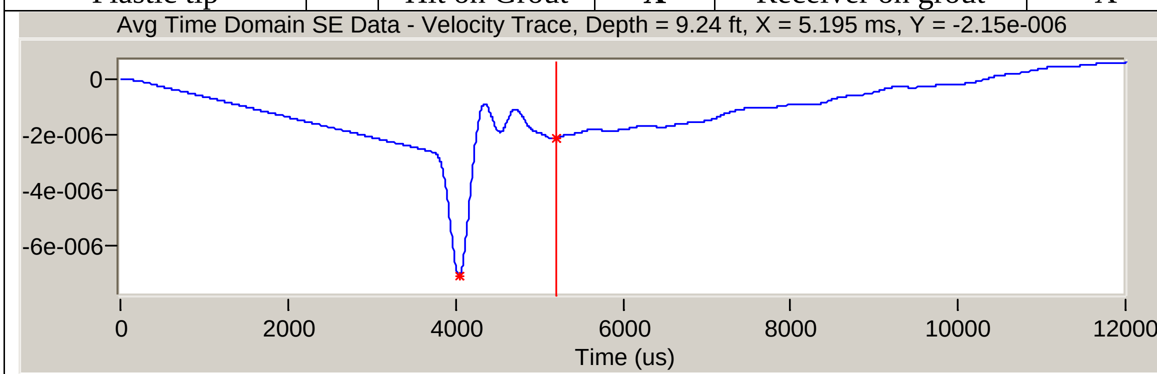
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Sand Cement	@middle	8.5-10.5

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—3st testing series

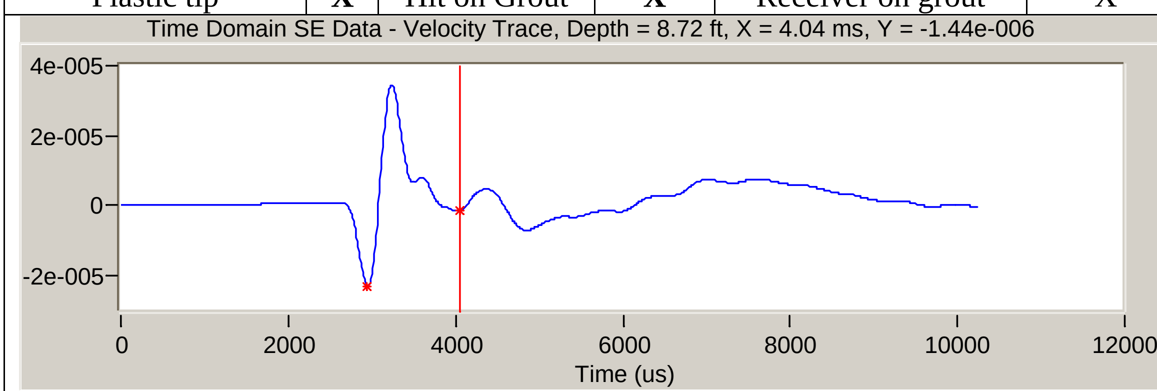
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



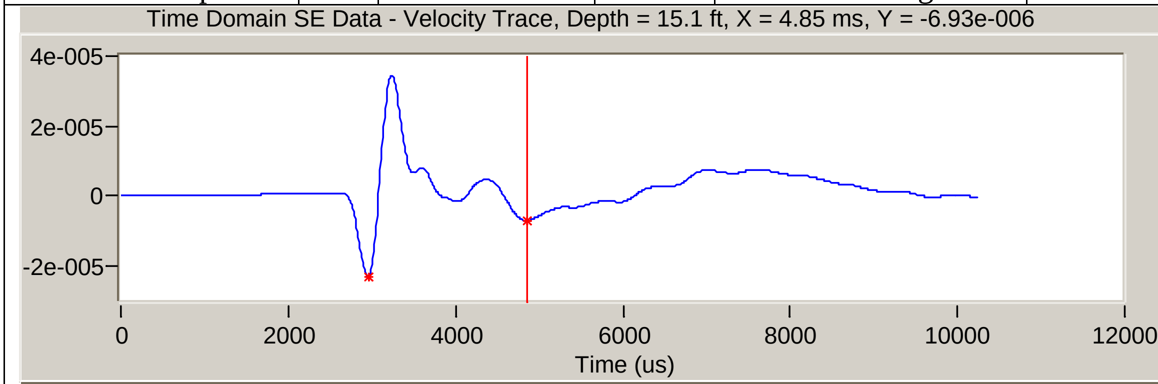
Wave Velocity=16000 ft/sec, echo at 9.24 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 8.72 ft

<table>
<tr><th colspan="6">Sonic Echo—3st testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6">
</td></tr>
<tr><td colspan="6">Wave Velocity=16000 ft/sec, echo at 15.1 ft</td></tr>
<tr><td colspan="6">NDT Conclusion</td></tr>
<tr><td colspan="6">SE tests detect a minor defect at 8.7 feet and the end of grout at 15.1feet
2nd test detected a defect at 9.24 ft</td></tr>
</table>

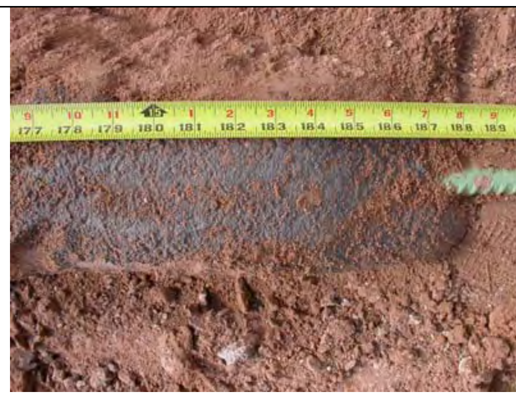
<table>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #16</td><td>
Grout passes through the middle foam, ends at 15’8”</td></tr>
<tr><td>
Minor defect at 9”-1’2”</td><td>
Grout meets the foam at 8’</td></tr>
</table>

Table A.17--Nail No.17

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Sand Cement	Bird Mouth	N/A

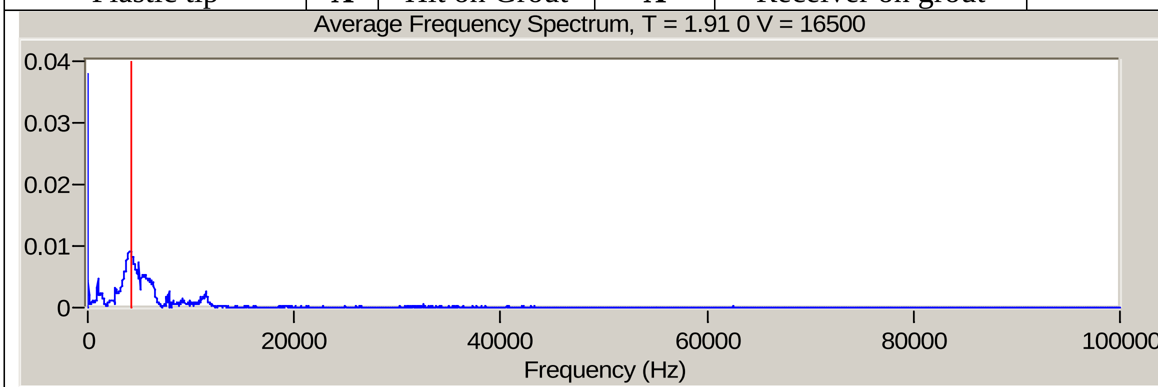
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Impact Echo—2nd testing series

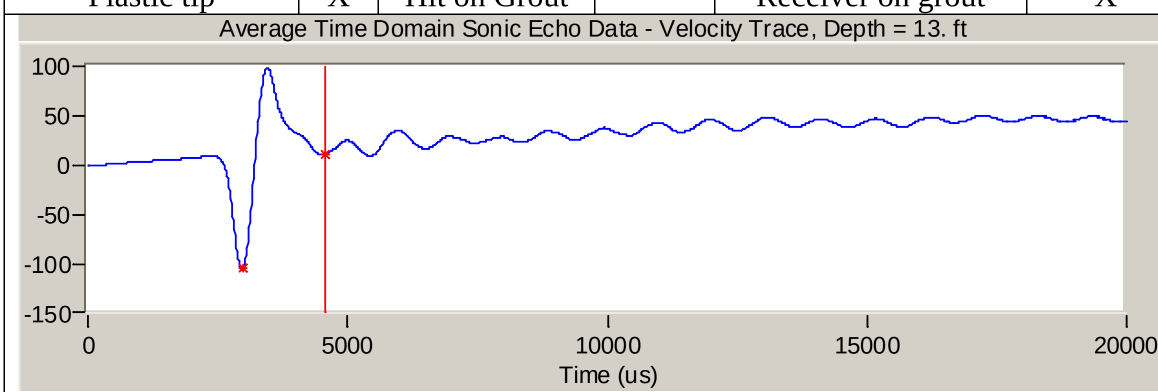
Metal tip		Hit on bar		Receiver on bar	**X**
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, peak at 1.91 ft (TTU Analysis)

Sonic Echo—1st testing series

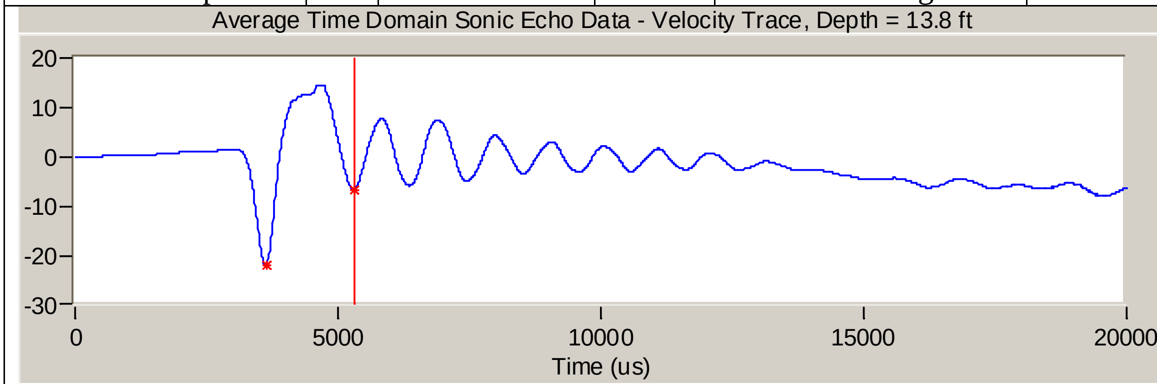
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	X	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 13 ft (TTU Analysis)

Sonic Echo—1st testing series

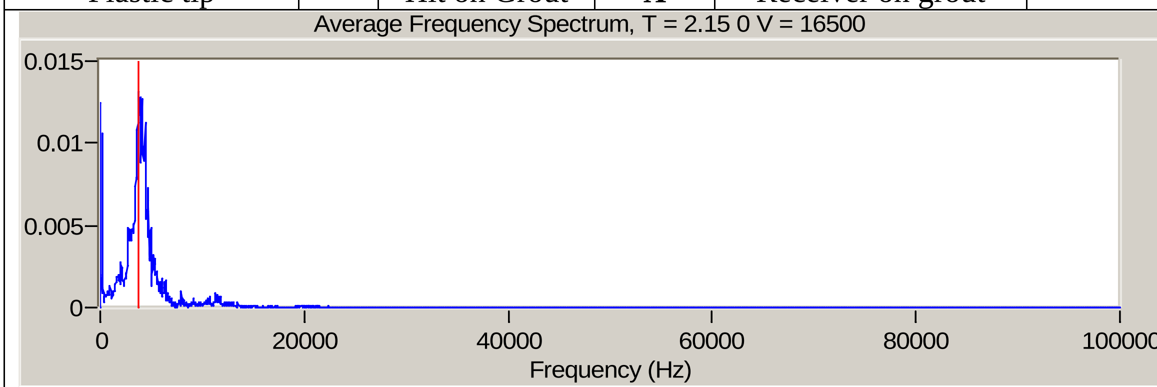
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16500 ft/sec, echo at 13.8 ft (TTU Analysis)

Impact Echo—2st testing series

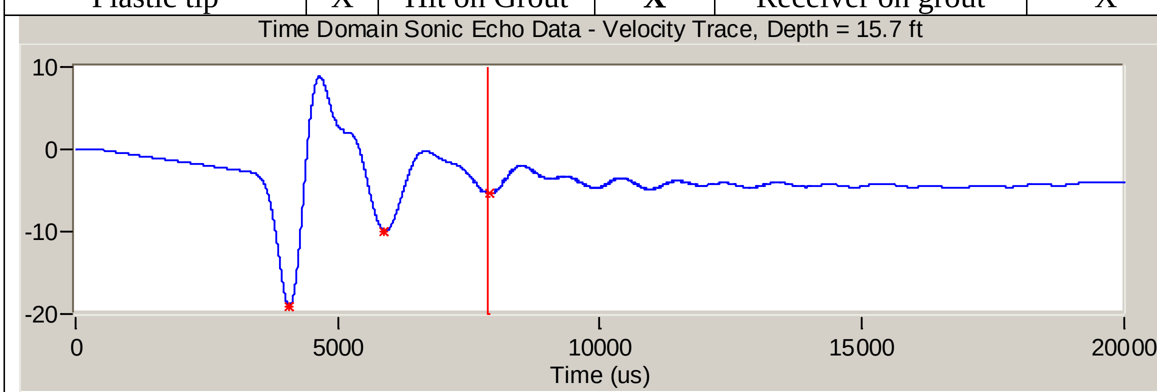
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, peak at 2.15 ft (TTU Analysis)

Sonic Echo—2st testing series

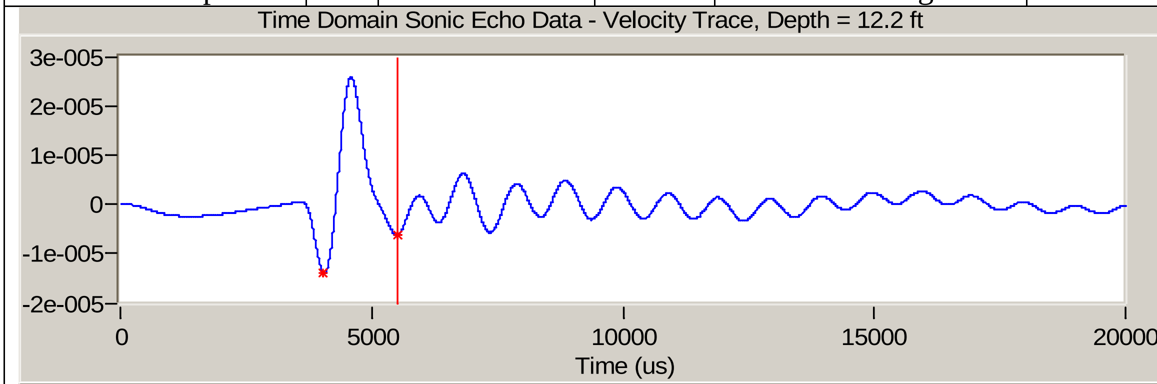
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 15.7 ft (TTU Analysis)

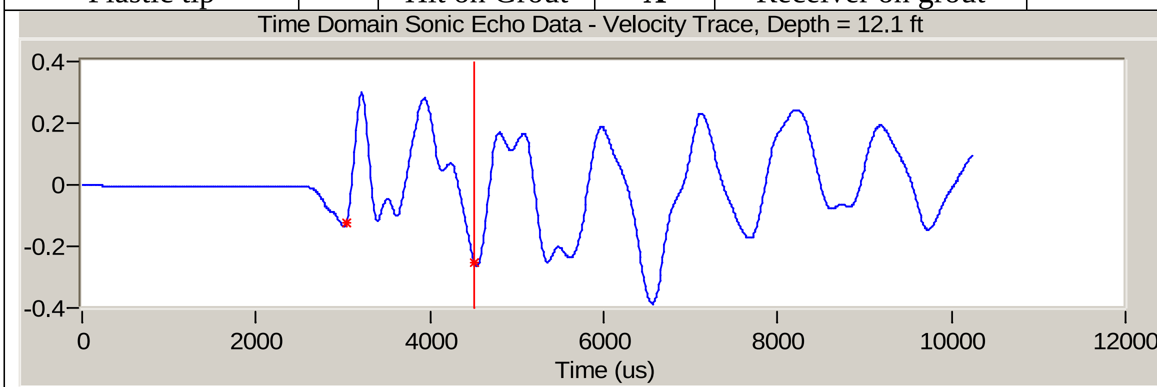
Continue Table A.17

Sonic Echo—2^{st} testing series					
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	**X**	Hit on Grout		Receiver on grout	X



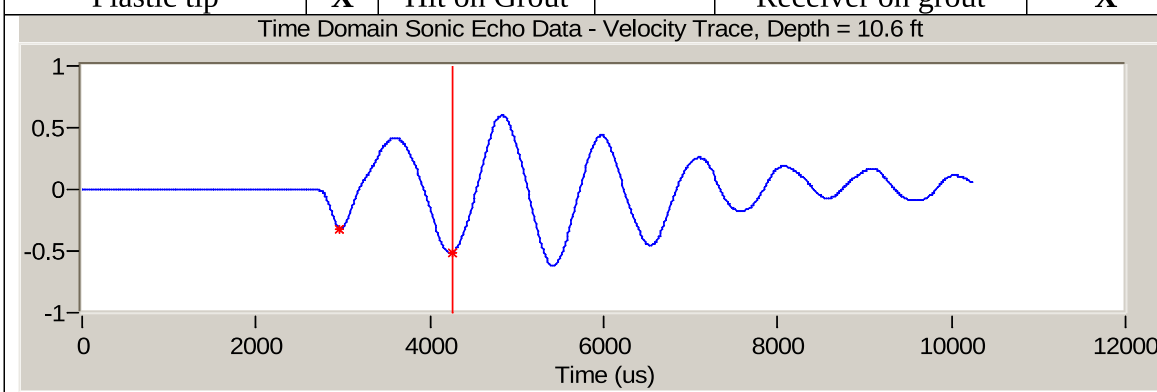
Wave Velocity=16500 ft/sec, echo at 12.2 ft (TTU Analysis)

Sonic Echo—3^{st} testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	**X**
Plastic tip		Hit on Grout	**X**	Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 14.7 ft (TTU Analysis)

Sonic Echo—3^{st} testing series					
Metal tip		Hit on bar	**X**	Receiver on bar	
Plastic tip	**X**	Hit on Grout		Receiver on grout	**X**

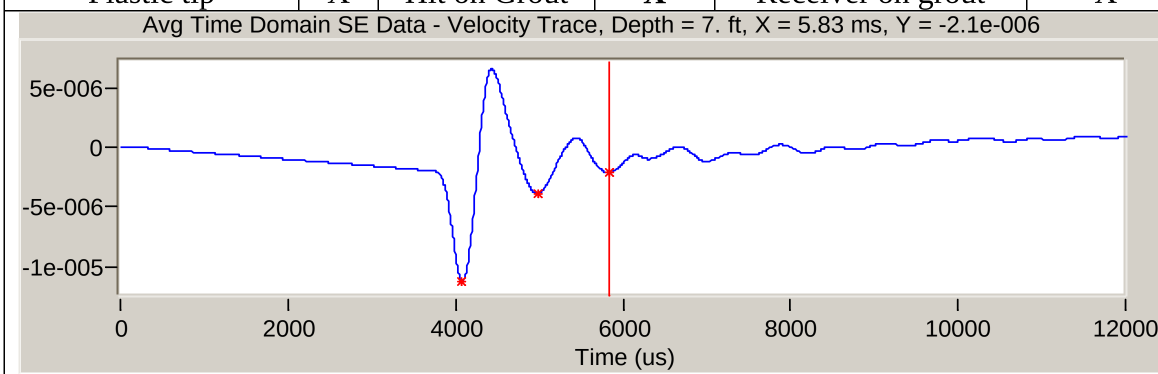


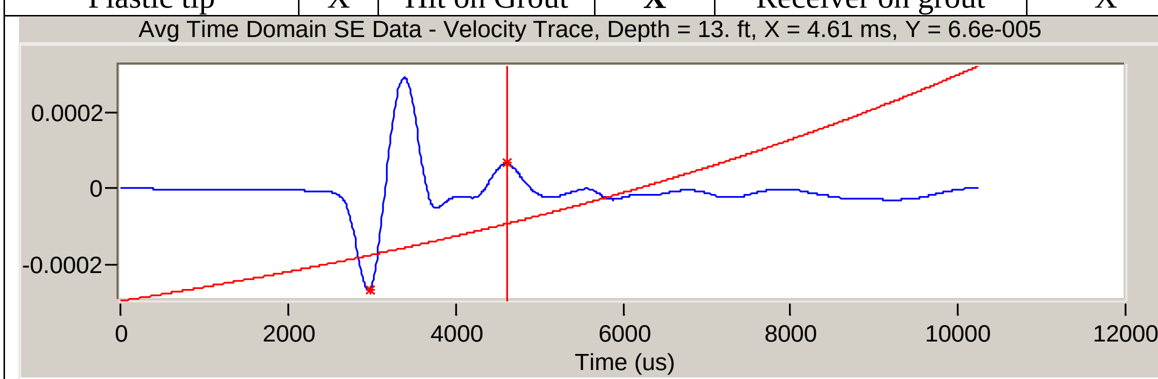
Wave Velocity=16500 ft/sec, echo at 10.6 ft (TTU Analysis)

Continue Table A.17

NDT conclusion	
SE tests detect defect or the end of nail at 10.6-15.7 feet. IE tests detect shallow defect at 1.91-2.15 feet	
Exhumation Results	
 Exposed Nail #17	 Grout ends at 13’8”
 Bird’s mouth	 Nail head exposed

Table A.18--Nail No.18

<table>
<tr><th colspan="6">Designed Condition</th></tr>
<tr><td>Nominal Length (feet)</td><td colspan="2">Grout Type</td><td colspan="2">Defect Type</td><td>Defect Dimension (feet)</td></tr>
<tr><td>20</td><td colspan="2">Neat Cement</td><td colspan="2">@middle</td><td>6.0-8.0</td></tr>
<tr><th colspan="6">Head Condition</th></tr>
<tr><td colspan="2">First series of testing</td><td colspan="2">Second series of testing</td><td colspan="2">Third series of testing</td></tr>
<tr><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><th colspan="6">Sonic Echo—2nd testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6">

Wave Velocity=16000 ft/sec, neck echo at 7 ft</td></tr>
<tr><th colspan="6">Sonic Echo—3st testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6">

Wave Velocity=16000 ft/sec, bulb echo at 13 ft</td></tr>
</table>

Continue Table A.18

<table>
<tr><th colspan="2">NDT Conclusion</th></tr>
<tr><td colspan="2">SE test detect an increase in diameter at 13 ft</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #18</td><td>
Grout passes through the middle foam</td></tr>
<tr><td>

Grout ends at 20’4”</td><td>

Minor defect at 12’5”</td></tr>
</table>

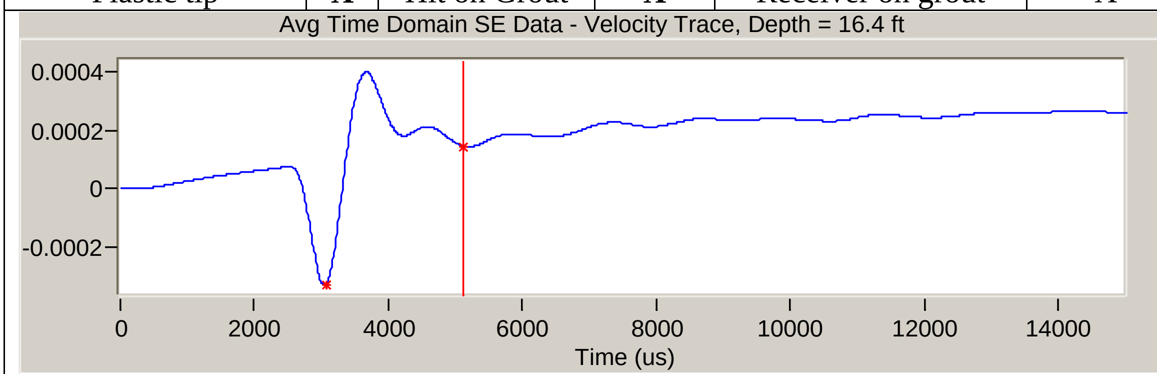
Table A.19--Nail No.19

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Sand Cement	No defect	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Sonic Echo—3rd testing series

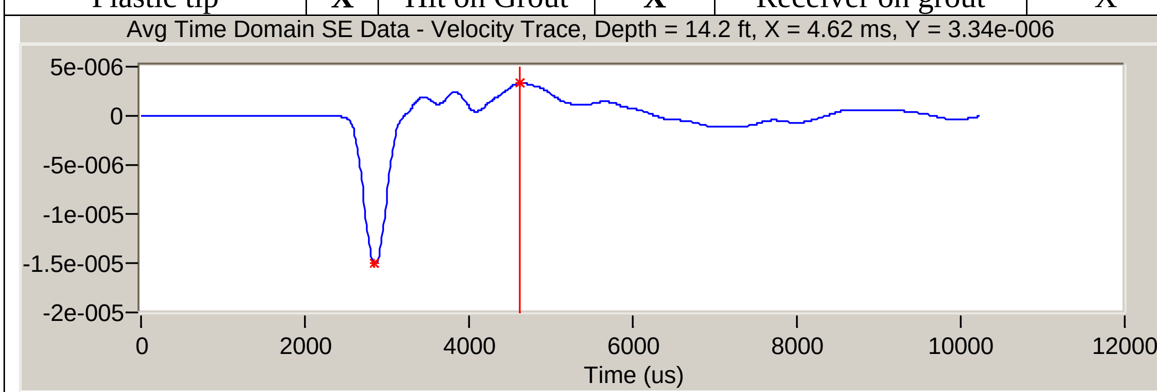
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, bulb echo at 16.4 ft

Sonic Echo—3rd testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, bulb echo at 14.2 ft

<table>
<tr><th colspan="2">NDT conclusion</th></tr>
<tr><td colspan="2">1st SE tests detected a neck at 16.4 ft
3rd SE tests detect an increase in diameter at 14.2 ft</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #19</td><td>
Grout ends at 14’8”</td></tr>
</table>

Table A.20--Nail No.20

Designed Condition

Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Neat Cement	No defect	N/A

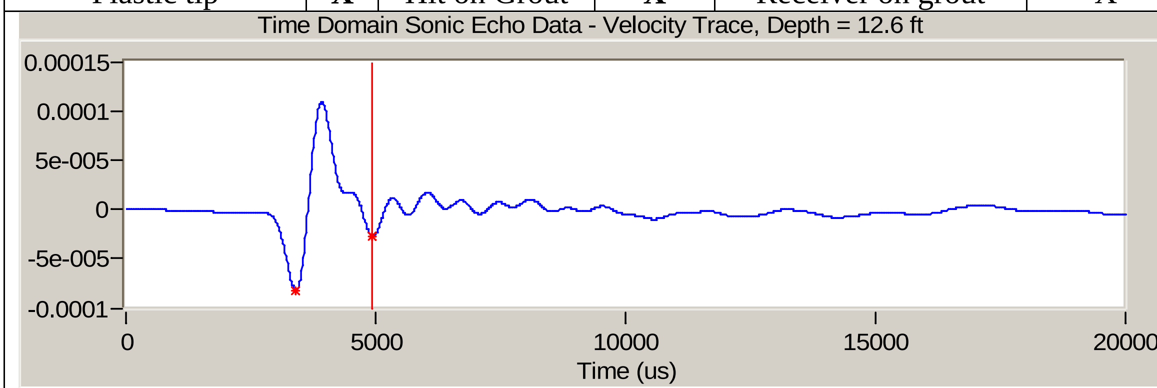
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Testing Results

Sonic Echo—1st testing series

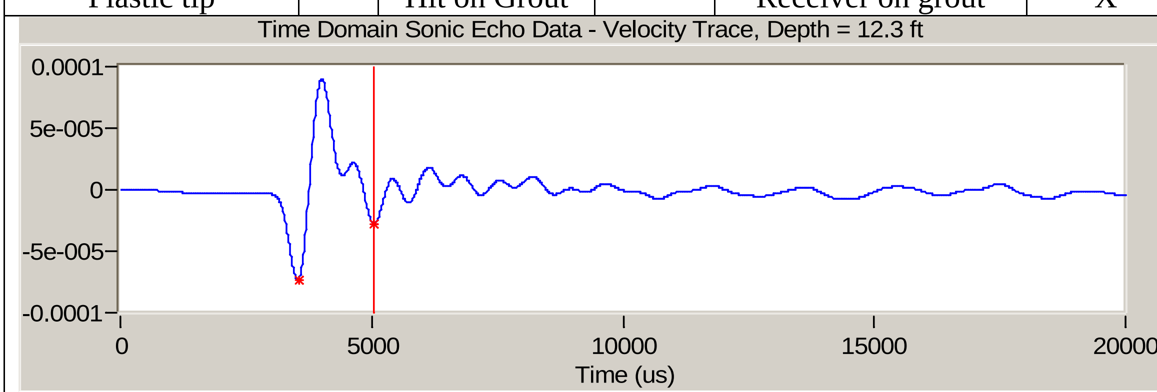
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 12.6 ft (TTU Analysis)

Sonic Echo—1st testing series

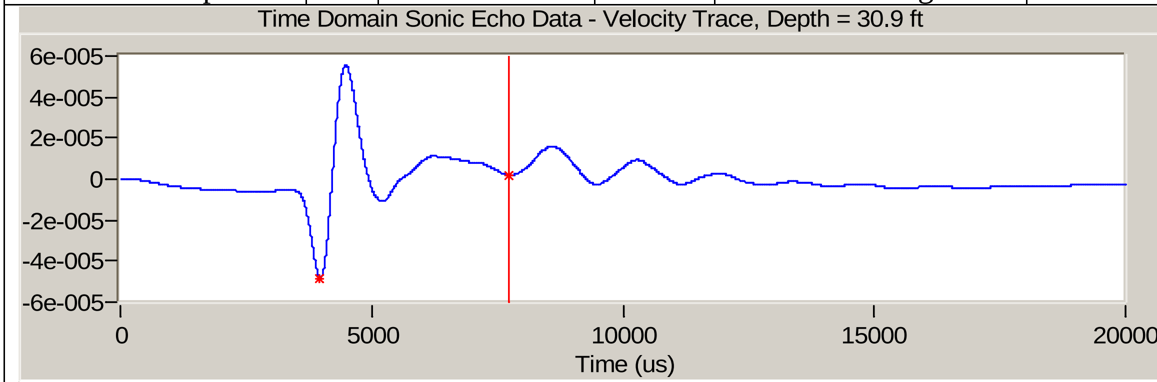
Metal tip	**X**	Hit on bar	X	Receiver on bar	
Plastic tip		Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 12.3 ft (TTU Analysis)

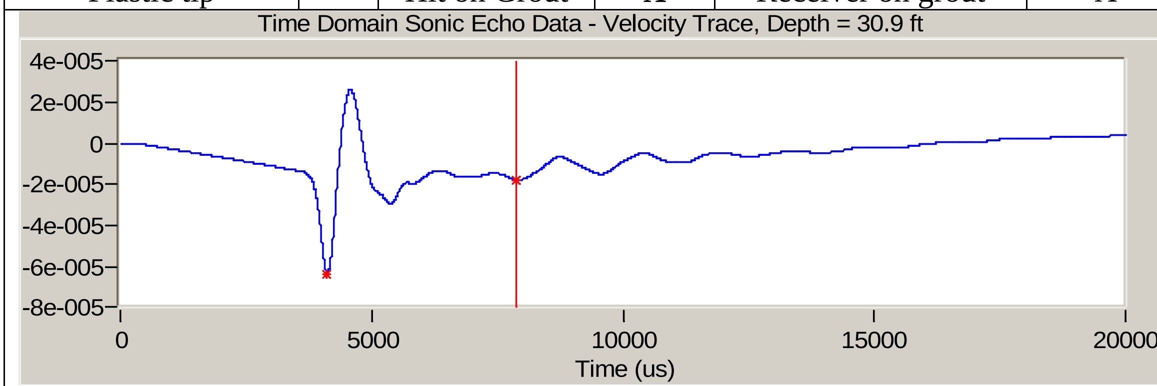
Continue Table A.20

Sonic Echo—2st testing series					
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	**X**



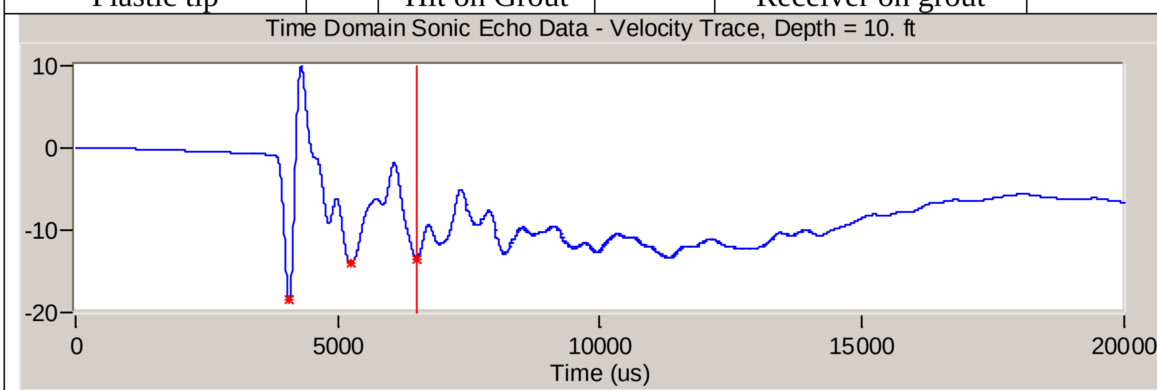
Wave Velocity=16500 ft/sec, echo at 30.9 ft (TTU Analysis)

Sonic Echo—2st testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 30.9 ft (TTU Analysis)

Sonic Echo—2st testing series					
Metal tip	**X**	Hit on bar	**X**	Receiver on bar	**X**
Plastic tip		Hit on Grout		Receiver on grout	



Wave Velocity=16500 ft/sec, echo at 13.9 ft (TTU Analysis)

Continue Table A.20

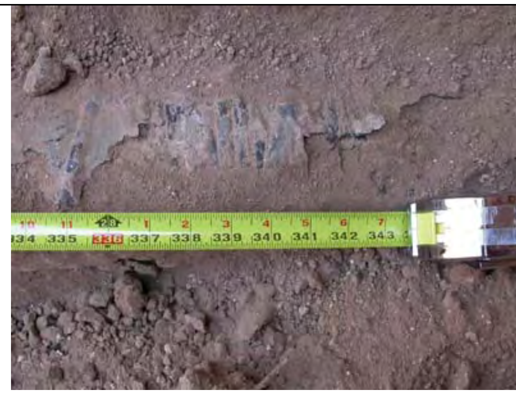
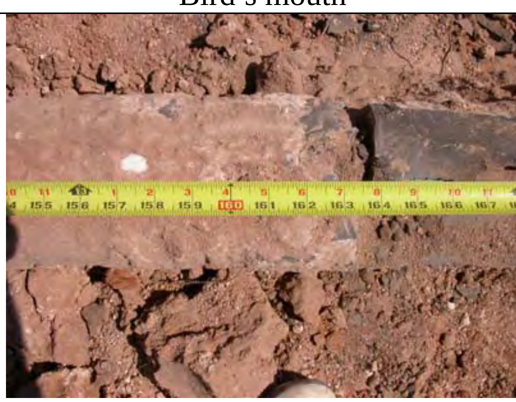
NDT conclusion	
SE tests detect defect at 10-12.6 feet and detect the nail end at 30 feet	
Exhumation Results	
 Exposed Nail #20	 Grout ends at 28’7”
 Bird’s mouth	 Minor defect at 11’6”
 Part of the nail was damaged by the first exhumation	 Grout was broken at the end (25’)

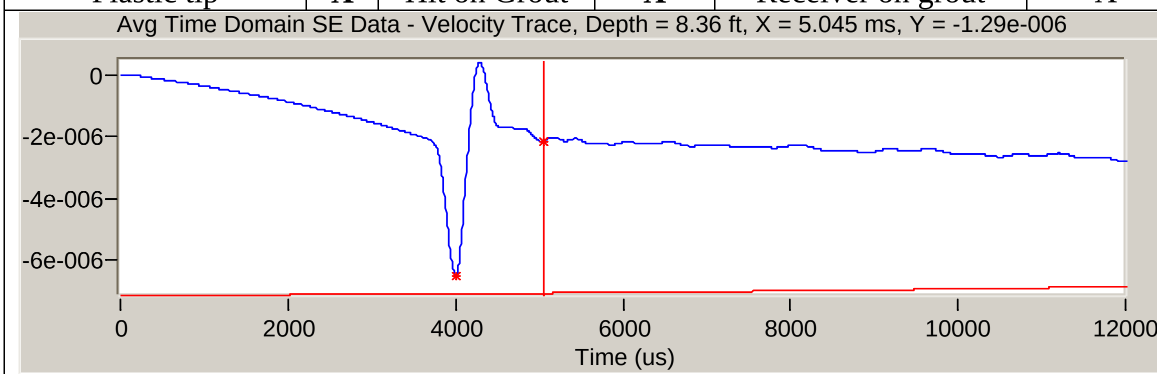
Table A.21--Nail No.21

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Sand Cement	@end	18.5-24

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Sonic Echo—2nd testing series

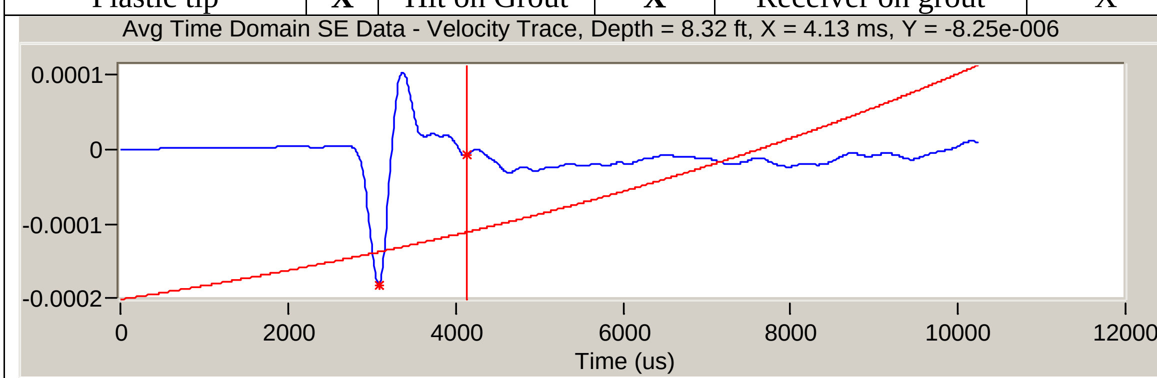
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 8.36 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 8.32 ft

<table>
<tr><th colspan="2">NDT conclusion</th></tr>
<tr><td colspan="2">SE tests detect the end of nail or defect at 8.32 – 8.36 ft</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #21</td><td>

Grout ends at 10’7”</td></tr>
</table>

Table A.22--Nail No.22

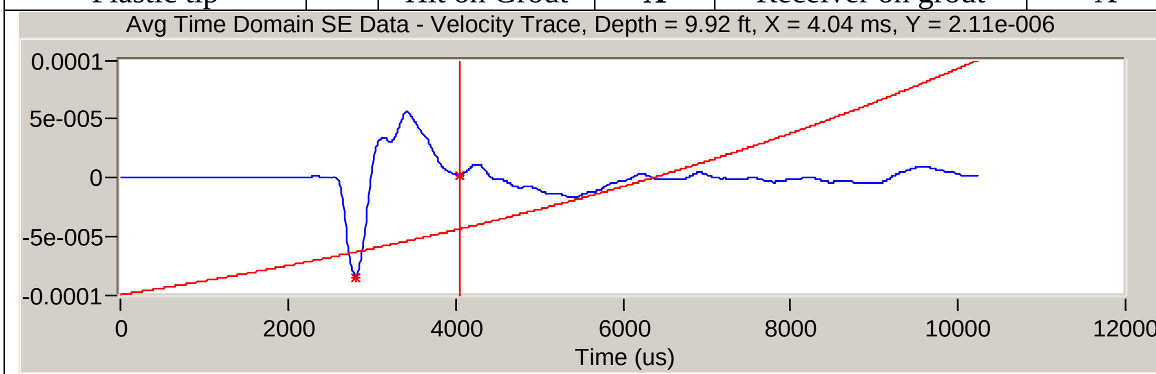
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Sand Cement	@middle	5.5-7.5

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—3rd testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 9.92 ft

NDT conclusion
SE tests detect defect or the end of grout at 9.92 feet

Exhumation Results	
 Exposed Nail #22	 Grout stops at the middle foam, ends at 10'5"

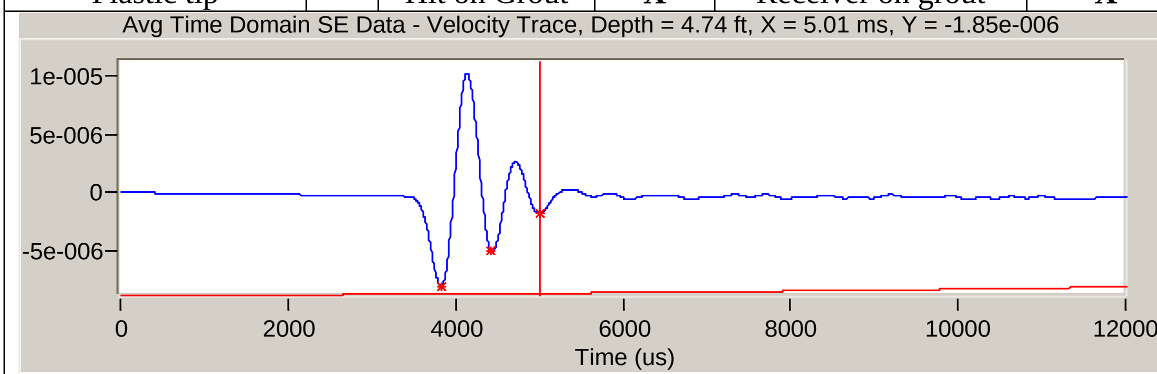
Table A.23--Nail No.23

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Sand Cement	Bird's mouth	N/A

Head Condition		
First series of testing	Second series of testing	Third series of testing
		

Sonic Echo—2nd testing series

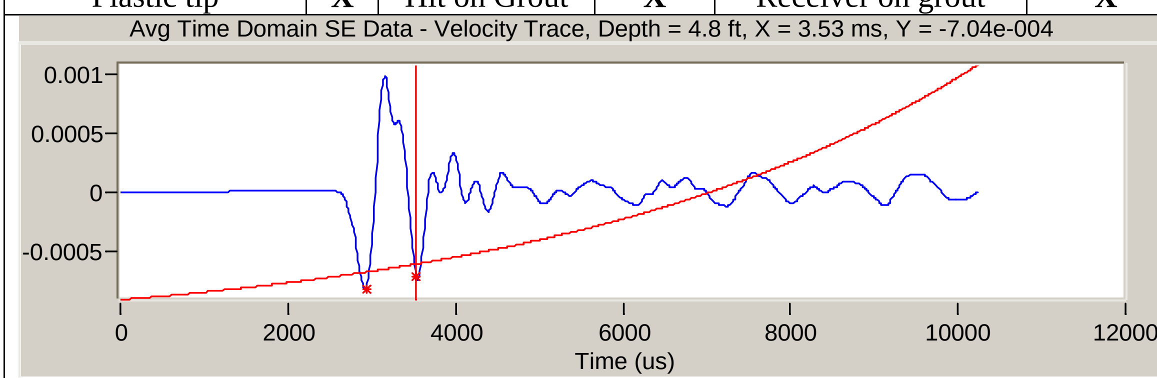
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 4.47 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	**X**	Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16000 ft/sec, echo at 4.8 ft

Continue Table A.23

<table>
<tr><th colspan="2">NDT conclusion</th></tr>
<tr><td colspan="2">SE tests detect defect or end of grout 4.47 - 4.8 ft</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #23</td><td>
Bird’s mouth</td></tr>
<tr><td>

Minor defect at 2’3”</td><td>

Grout ends at 13’</td></tr>
</table>

Table A.24--Nail No.24

Designed Condition			
Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Neat Cement	@middle	5.5-7.5

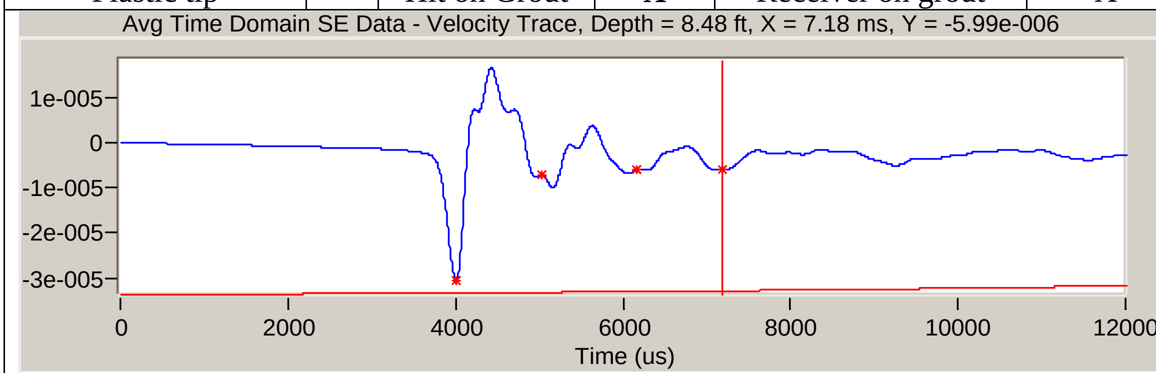
Head Condition

First series of testing	Second series of testing	Third series of testing
		

Field Test Results

Sonic Echo—2nd testing series

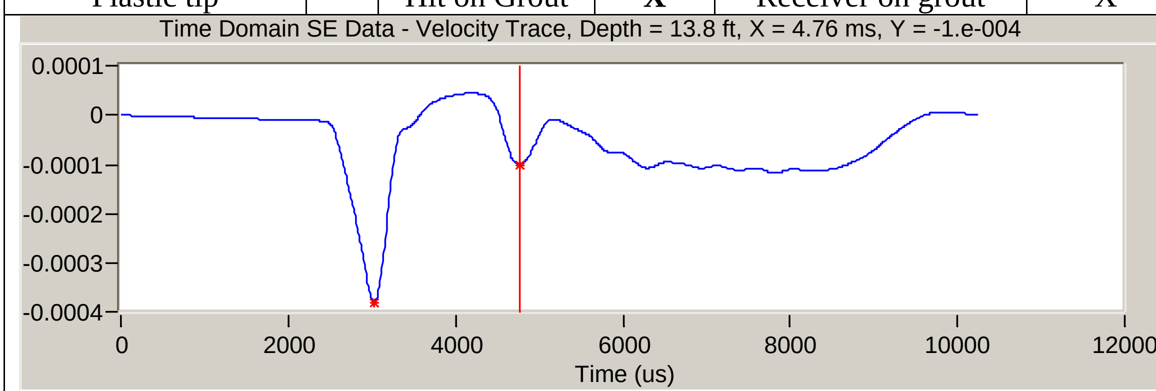
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, neck echo at 8.5 ft

Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X

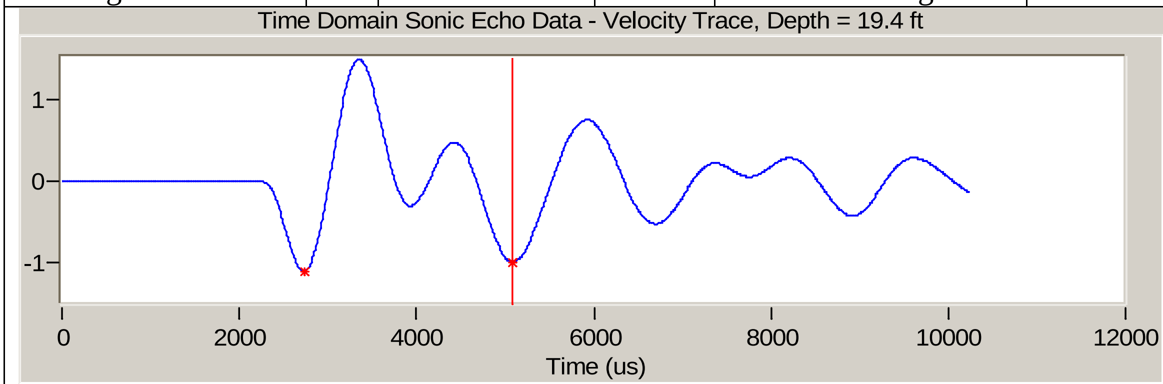
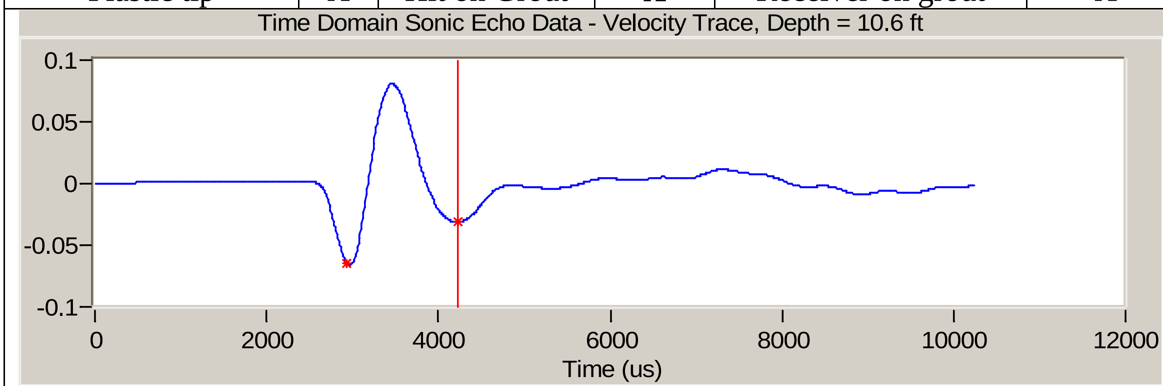


Wave Velocity=16000 ft/sec, neck echo at 13.8 ft

Continue Table A.24

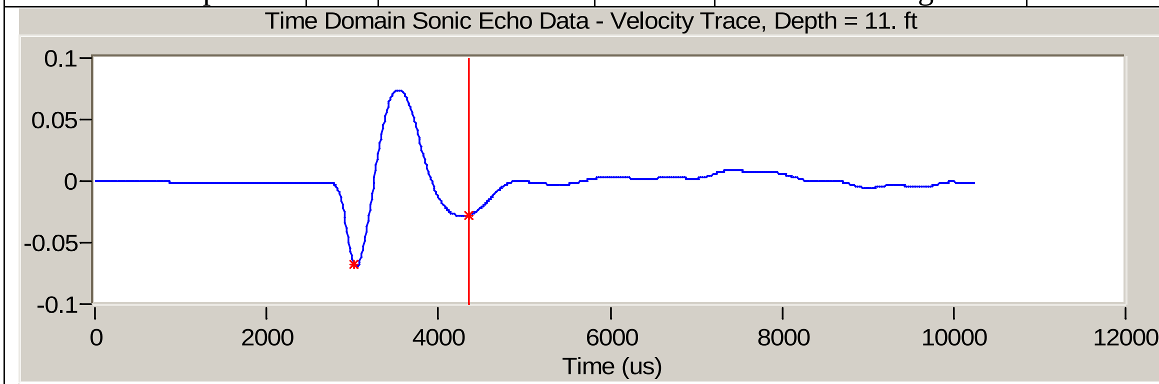
NDT conclusion	
2nd SE test detected a neck at 8.5 ft 3rd SE tests detect defect at 13.8 ft	
Exhumation Results	
 Exposed Nail #24	 Grout passes through the middle foam, ends at 27’7”
 Minor defect at 16’	 Minor defect at 8’

Table A.25--Nail No.25

<table>
<tr><th colspan="6">Designed Condition</th></tr>
<tr><td>Nominal Length (feet)</td><td colspan="2">Grout Type</td><td colspan="2">Defect Type</td><td>Defect Dimension (feet)</td></tr>
<tr><td>20</td><td colspan="2">Neat Cement</td><td colspan="2">No defect</td><td>N/A</td></tr>
<tr><th colspan="6">Head Condition</th></tr>
<tr><td colspan="6"></td></tr>
<tr><th colspan="6">Field Testing Results
Sonic Echo—3rd testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td>X</td></tr>
<tr><td>Big hammer</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td></td></tr>
<tr><td colspan="6"></td></tr>
<tr><td colspan="6">Wave Velocity=16500 ft/sec, echo at 19.4 ft (TTU Analysis)</td></tr>
<tr><th colspan="6">Sonic Echo—3st testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6"></td></tr>
<tr><td colspan="6">Wave Velocity=16500 ft/sec, echo at 10.6 ft (TTU Analysis)</td></tr>
</table>

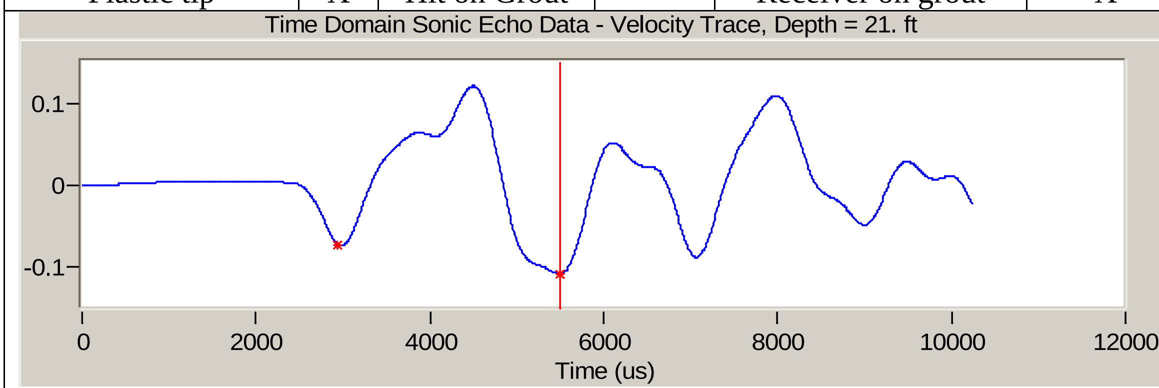
Continue Table A.25

Sonic Echo—3st testing series					
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	**X**



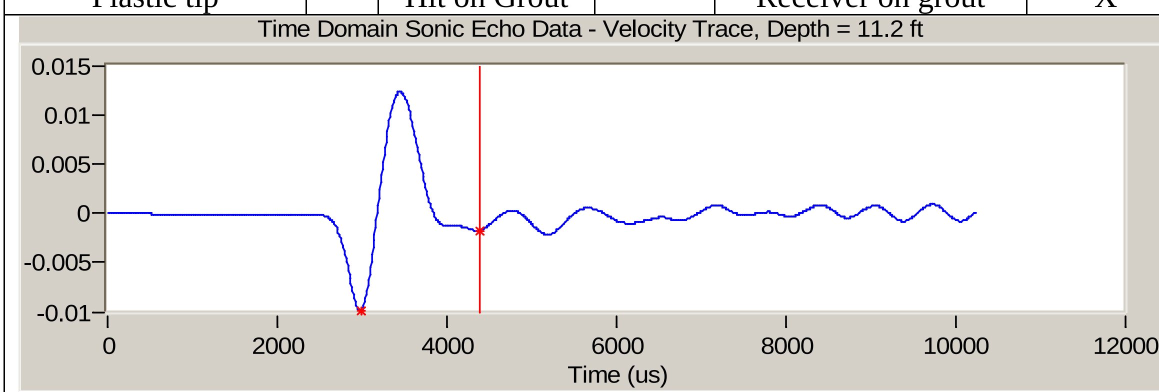
Wave Velocity=16500 ft/sec, echo at 11 ft (TTU Analysis)

Sonic Echo—3st testing series					
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	X	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 21 ft (TTU Analysis)

Sonic Echo—3st testing series					
Metal tip	**X**	Hit on bar	**X**	Receiver on bar	
Plastic tip		Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 11.2 ft

Continue Table A.25

<table>
<tr><th colspan="2">NDT Conclusion</th></tr>
<tr><td colspan="2">SE tests detect defect at 10.6-11.2 feet and the end of grout at 19.4-21 feet
(TTU Analysis)</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Grout ends at 20’7”</td><td>
Minor defect at 12’</td></tr>
</table>

Table A.26--Nail No.26

Designed Condition

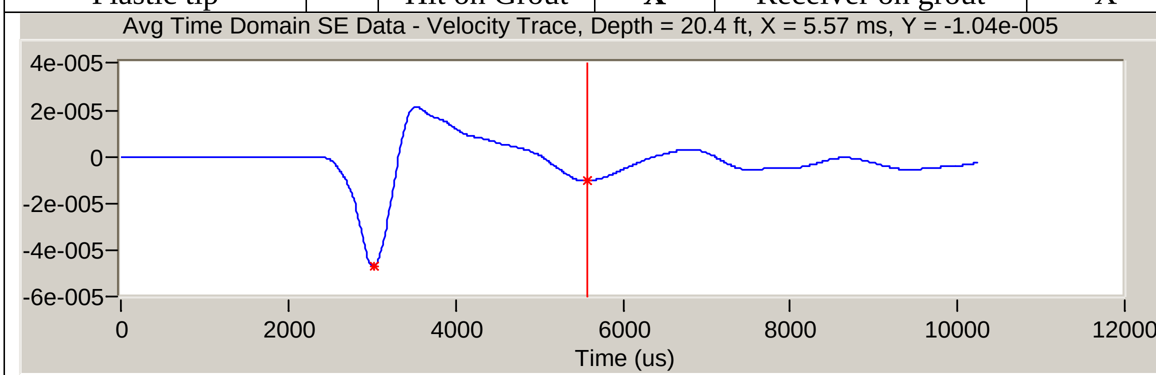
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Sand Cement	No defect	N/A

Head Condition

Field Testing Results

Sonic Echo—3st testing series

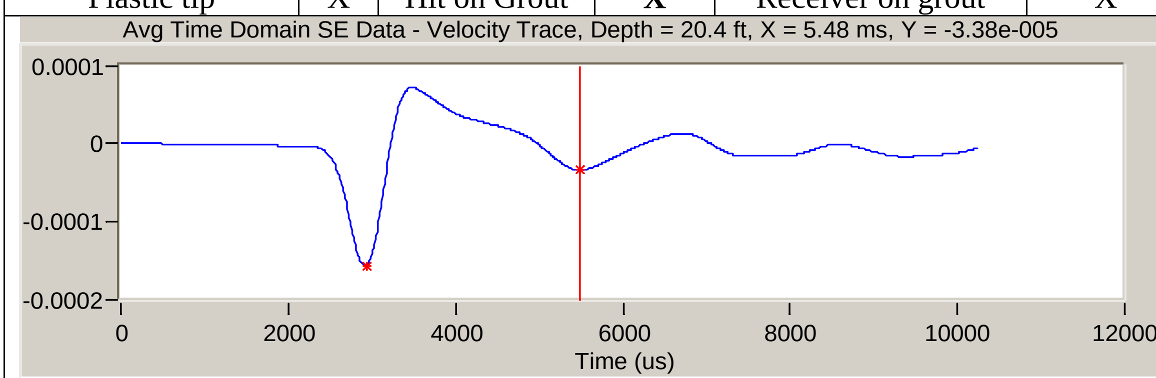
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 20.4 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 20.4 ft

Continue Table A.26

NDT conclusion	
No defect detected, nail ends at 20.4 feet	
Exhumation Results	
	
Exposed Nail #26, no defect	Grout ends at 20’5”

Table A.27--Nail No.27

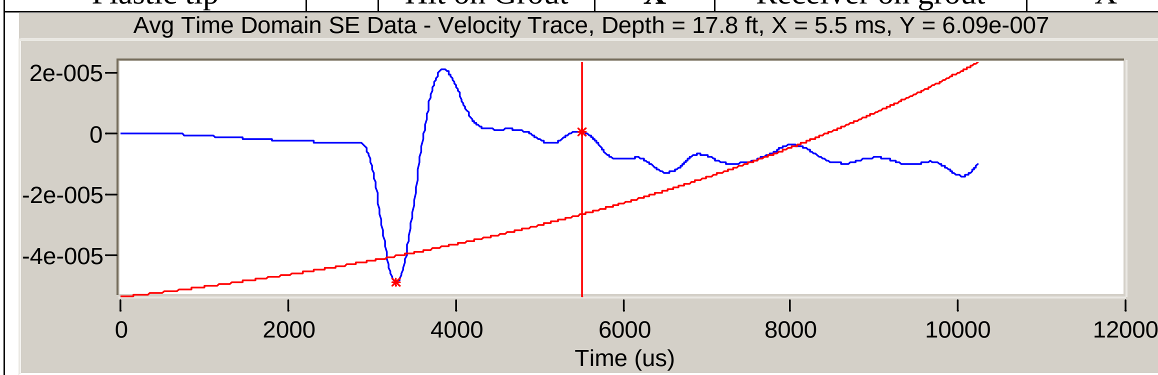
Designed Condition			
Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
20	Neat Cement	No defect	N/A

Head Condition

Field Testing Results

Sonic Echo—3st testing series

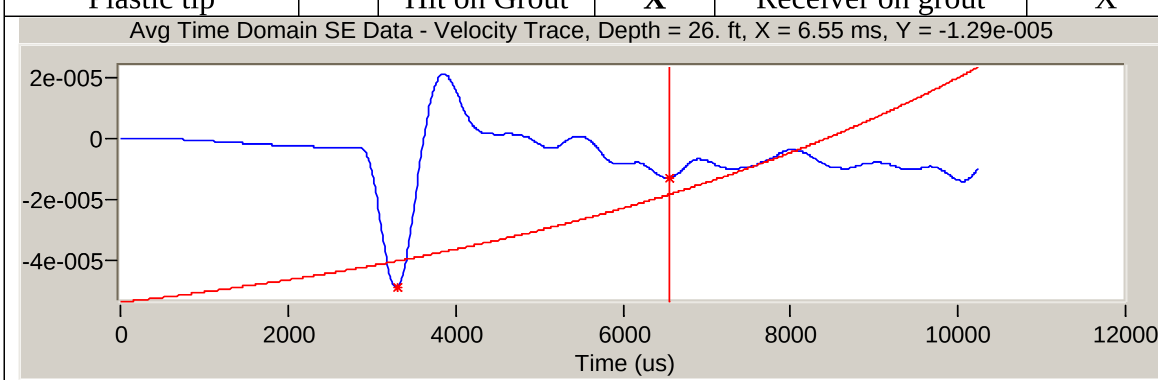
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, bulb echo at 17.8 ft

Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X

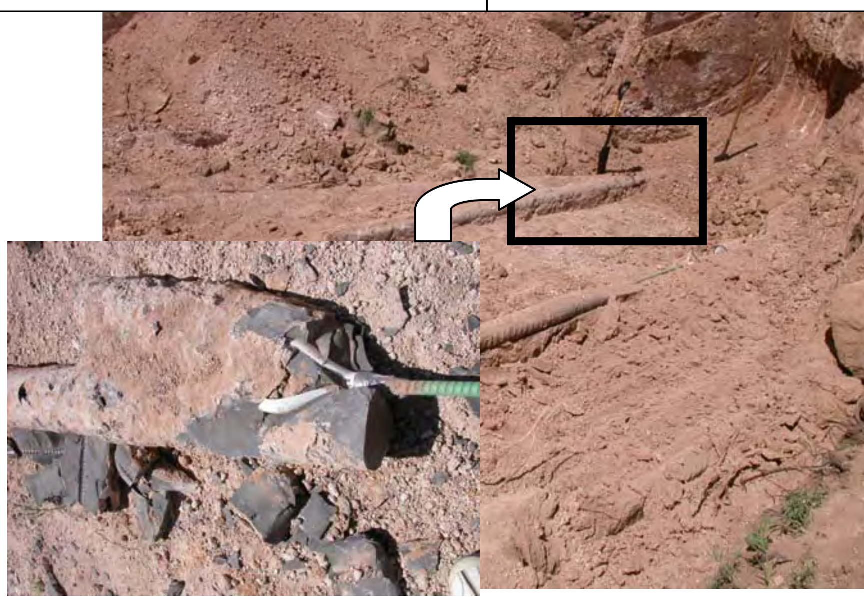


Wave Velocity=16000 ft/sec, end echo at 26.0 ft

NDT Conclusion
SE tests detect an increase in diameter at 17.8 feet and an end at 26 ft

Exhumation Results

Exposed Nail #27 (left) joined with Nail #28 at 18'4" and grout ends at 27'



Joint of Nail #27 and Nail #28 at 18'4"

Table A.28--Nail No.28

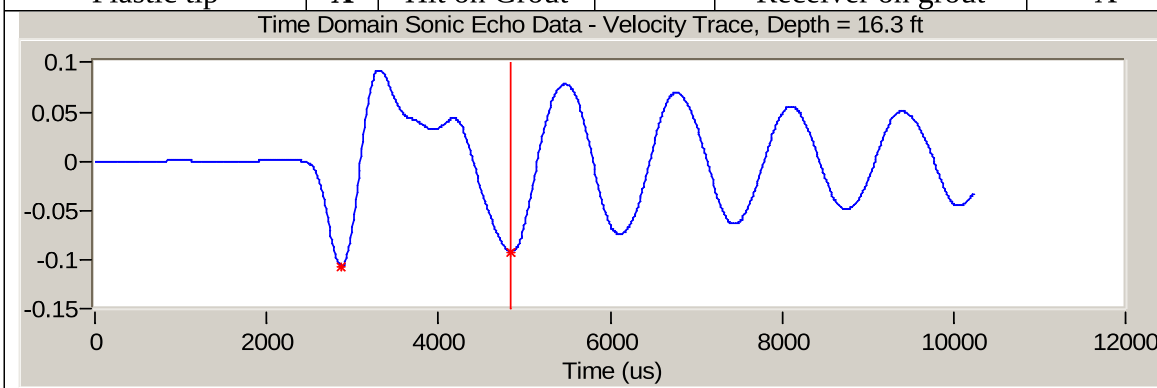
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Neat Cement	No defect	N/A

Head Condition

Field Testing Results

Sonic Echo—3st testing series

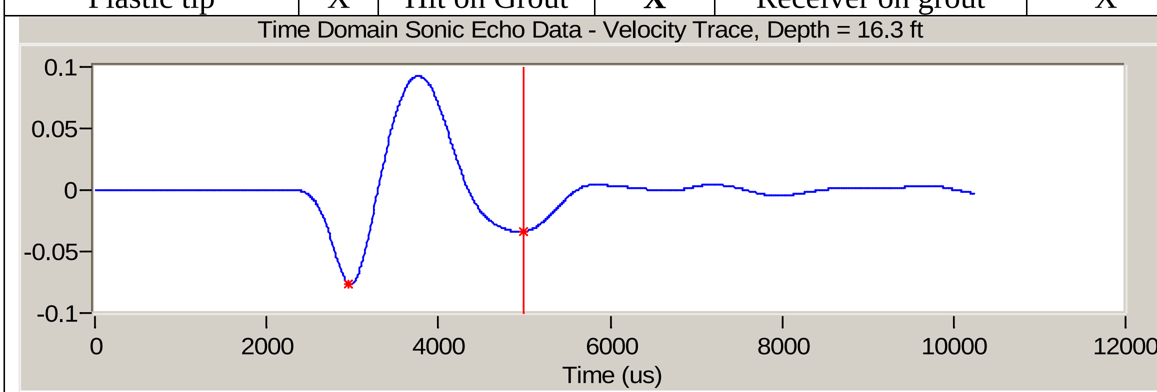
Metal tip		Hit on bar	X	Receiver on bar	
Plastic tip	**X**	Hit on Grout		Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 16.3 ft

Sonic Echo—3st testing series

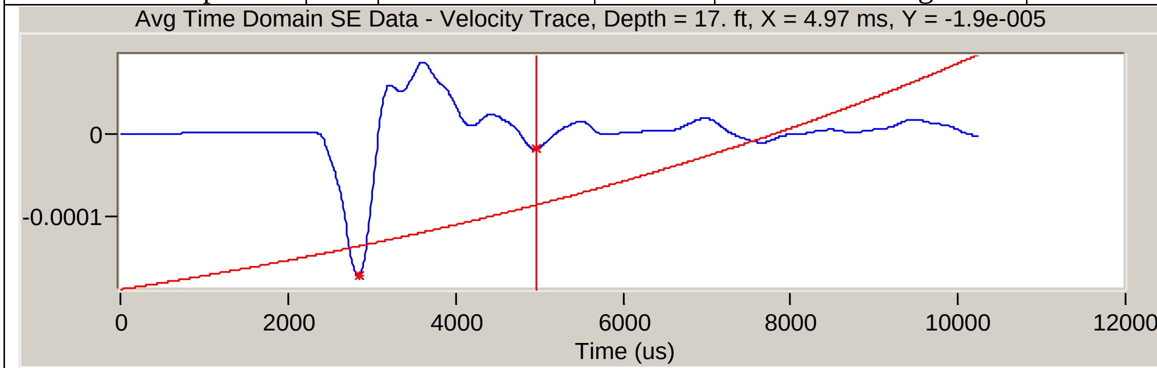
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16500 ft/sec, echo at 16.3 ft

Continue Table A.28

Sonic Echo—3st testing series					
Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	**X**



Wave Velocity=16500 ft/sec, neck echo at 17 ft

NDT conclusion

SE tests detect defect at 17 feet

Exhumation Results

Exposed Nail #28 (middle) joined with Nail #27at 18’4”, and the grout goes to 27’

Joint of Nail #27 and Nail #28 at 18’4”

Table A.29-Nail No.29

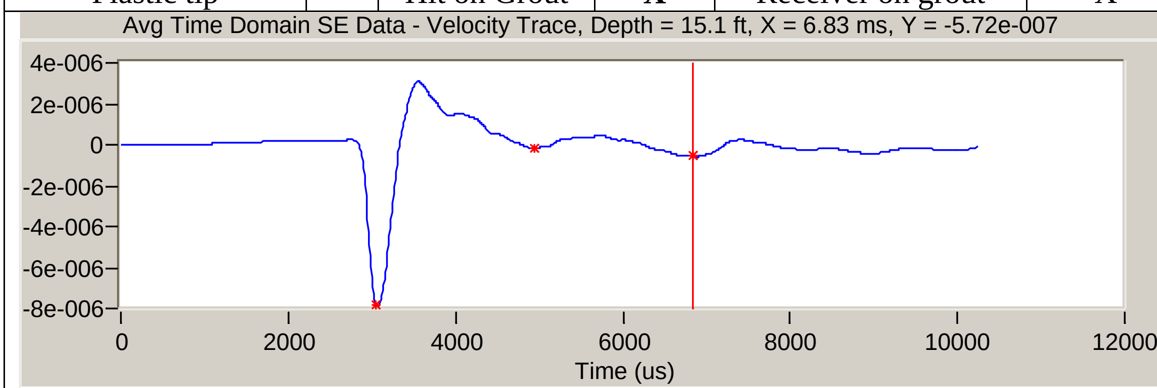
Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Sand Cement	Defect @end	20-end

Head Condition

Field Testing Results

Sonic Echo—3st testing series

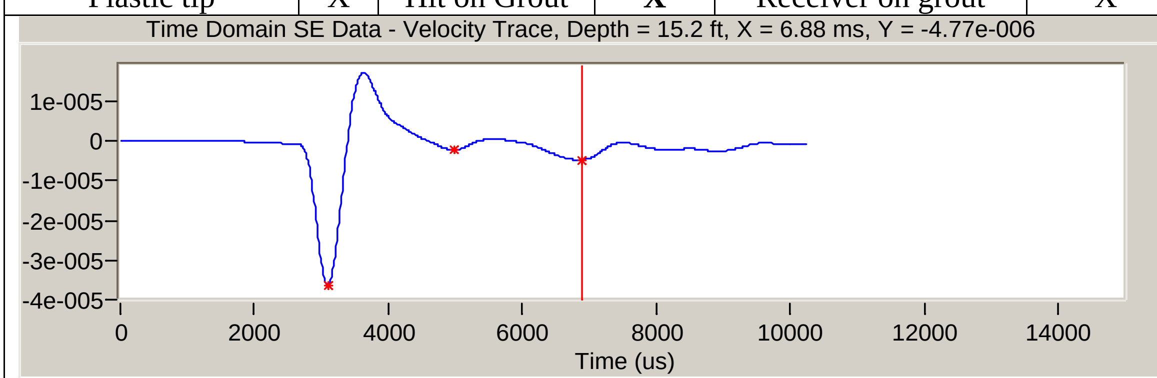
Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, neck echo at 15.1 ft

Sonic Echo—3st testing series

Metal tip		Hit on bar		Receiver on bar	
Plastic tip	X	Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 15.2 ft

Continue Table A.29

<table>
<tr><th colspan="2">NDT conclusion</th></tr>
<tr><td colspan="2">SE tests detect defect or the end of grout at 15.1 – 15.2 feet</td></tr>
<tr><th colspan="2">Exhumation Results</th></tr>
<tr><td>
Exposed Nail #29, grout doesn’t reach foam</td><td>
Grout stops at 15’4”</td></tr>
</table>

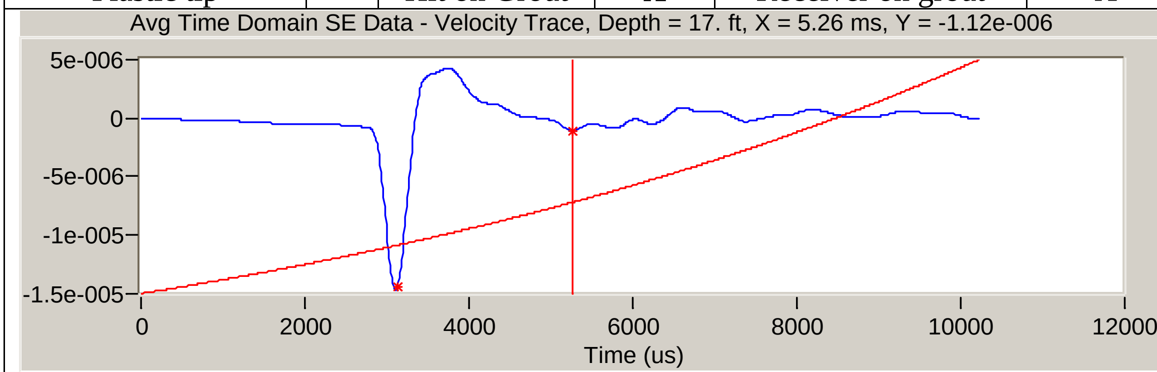
Table A.30--Nail No.30

Designed Condition			
Nominal Length (feet)	Grout Type	Defect Type	Defect Dimension (feet)
25	Neat Cement	Defect @middle	17-19

Head Condition

Field Testing Results
Sonic Echo—3st testing series

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, weak echo at 17.0 ft

NDT conclusion

SE tests detect defect at 17.0 feet

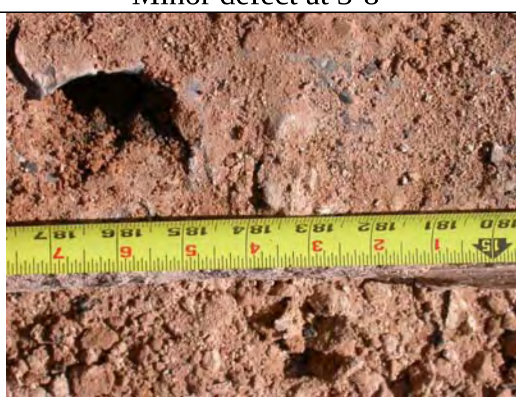
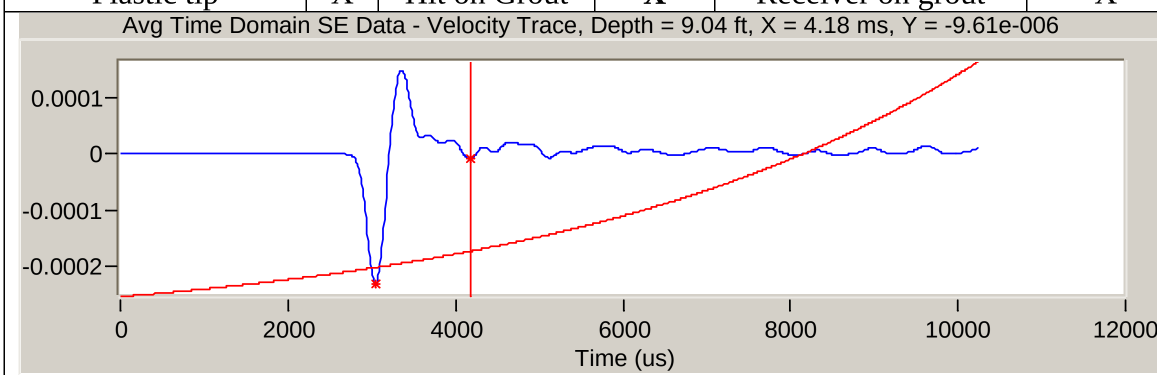
Exhumation Results	
 Exposed Nail #30	 Grout goes through foam and ends at 26’5”
 Minor defect at 3’8”	 Grout goes through foam
 Minor defect at 15’5”	 Foam ends at 19’5”

Table A.31--Nail No.31

<table>
<tr><th colspan="6">Designed Condition</th></tr>
<tr><td>Nominal Length (feet)</td><td colspan="2">Grout Type</td><td>Defect Type</td><td colspan="2">Defect Dimension (feet)</td></tr>
<tr><td>25</td><td colspan="2">Sand Cement</td><td>Defect @middle</td><td colspan="2">17-19</td></tr>
<tr><th colspan="6">Head Condition</th></tr>
<tr><td colspan="6"></td></tr>
<tr><th colspan="6">Field Testing Results
Sonic Echo—3st testing series</th></tr>
<tr><td>Metal tip</td><td></td><td>Hit on bar</td><td></td><td>Receiver on bar</td><td></td></tr>
<tr><td>Plastic tip</td><td>X</td><td>Hit on Grout</td><td>X</td><td>Receiver on grout</td><td>X</td></tr>
<tr><td colspan="6">

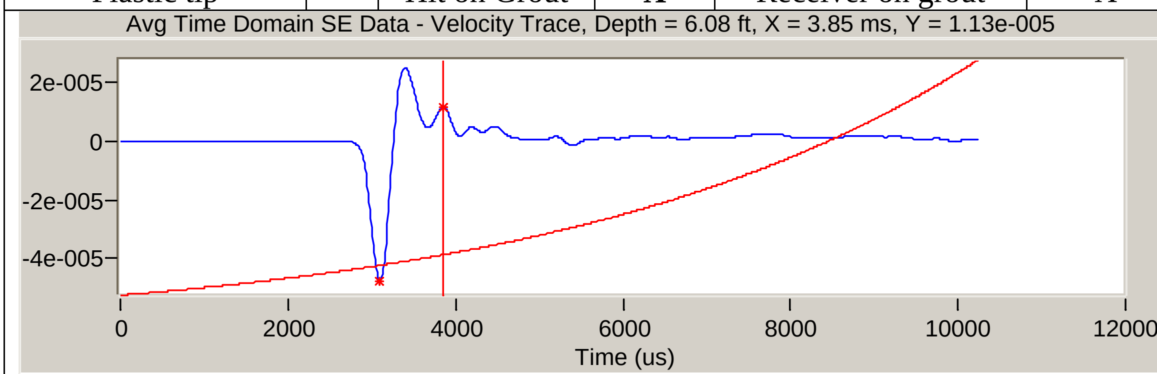
Wave Velocity=16000 ft/sec, echo at 9.04 ft</td></tr>
<tr><th colspan="6">NDT conclusion</th></tr>
<tr><td colspan="6">SE tests detect defect at 9.04 ft</td></tr>
</table>

Exhumation Results	
 Exposed Nail #31	 Grout lost half cross section at 14’
 Grout discontinuity	 0~13’9” of the hole was fully grouted, and 13’9” ~15’9” of the hole was partially grouted, 15’9” to foam of the hole was fully grouted

Table A.32--Nail No.32

<table>
<tr><th colspan="4">Designed Condition</th></tr>
<tr><td>Nominal Length (feet)</td><td>Grout Type</td><td>Defect Type</td><td>Defect Dimension (feet)</td></tr>
<tr><td>25</td><td>Sand Cement</td><td>No defect</td><td>N/A</td></tr>
<tr><th colspan="4">Head Condition</th></tr>
<tr><td colspan="4"></td></tr>
<tr><th colspan="4">Field Testing Results
Sonic Echo—3st testing series</th></tr>
</table>

Metal tip	**X**	Hit on bar		Receiver on bar	
Plastic tip		Hit on Grout	**X**	Receiver on grout	X



Wave Velocity=16000 ft/sec, echo at 6.08 ft

NDT conclusion
SE tests detect defect at an increase in diameter at 6 ft

Exhumation Results	
 Exposed Nail #32	 The soil nail was inconsistently grouted For the whole length, only the following part was fully grouted: 0-2’4”, 8’-9’9”. And the nail lost 50% of cross-section at 9’9” and 75% of cross-section at 12’3”
 Partially grouted nail	 Partially grouted nail

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