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DETERMINING PILE LENGTH FOR IN-PLACE BRIDGE FOUNDATIONS

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1. Introduction and Objectives

Problem Statement

The need exists to re-evaluate the performance criteria for foundation structures supporting existing bridges, both within the State of Alabama, and in a broader sense, within the entire United States. A large number of bridges exist within this country for which no data are available regarding construction records and original design plans. The necessity of determining the details of unknown subsurface foundation structural elements stems primarily from the following two reasons:

- to reevaluate the structural load bearing capacity of the bridge, which is directly related to the load bearing capacity of the foundation. If the nature and design of the foundation elements themselves are unknown, there is no practical means of judging the performance criteria of the bridge as a whole.
- to determine embedded lengths for foundations which have been subject to scour and thus may not be in a position to perform according to the original design requirements.

Project Objectives

The current HRC funded research project has been established to achieve the following objectives:

1. Identify test concepts, methods and equipment to enable the determination of unknown subsurface bridge foundation characteristics. In particular, the work would focus upon the problem of determination of embedded pile lengths for foundations supporting existing bridge structures. This portion of the work would include a comprehensive review of the state of the art as regards Non Destructive Testing (NDT), which might conceivably be applied to the current problem. This work would also include an evaluation of the applicability of the Pile Integrity

Testing Equipment (PIT) to the specific problem of determination of unknown pile lengths. This has been highlighted as a special point of interest because the Alabama Highway Department currently possesses such a PIT test setup.

2. Conduct field trials to evaluate the performance of the various methods proposed within the first objective.

3. Recommend procedures which may be used on a routine basis to solve the problem of unknown subsurface foundation conditions.

2. Testing Program

The unknown bridge foundation problem can broadly be classified into the following 2 categories:

1. Problems where the foundation type is known (either from original records or from direct observations) but the embedded length is unknown. This may the case for a typical foundation located in moving water which has been subjected to scour.
2. Both the foundation type and the length is unknown. For example, such a situation might arise in the case of a bridge bent supported upon concrete column(s), which might in turn be transferring its load to a shallow foundation, or to a pile cap resting on an unknown number of piles.

As has already been mentioned in the previous chapter, the primary objective of this study will be to provide simple rational method(s) for the **determination of unknown embedded lengths** for foundations supporting existing bridges. In particular, the major thrust of this study has been directed towards bridge foundations comprised entirely of piles as this type of foundation has been used extensively in the past to support bridges built in Alabama. Limited research has been conducted to date which relates directly to the solution of such a problem. However, a study of current literature has indicated several methods which might possibly be of significance in solving this problem of unknown embedded pile length. Also invaluable in this regard has been the personal communication (Olson and Jalinoos, 1994) carried out with the personnel of Olson Engineering of Golden, Colorado. Broadly, these methods have been classified into the following five categories, and detailed descriptions of each of the methods have been provided in the following chapter of this report.

1. Sonic Echo/Impulse Response (SE/IR) techniques which utilize the concepts of conventional wave propagation analysis in both the time and the frequency domains (this includes the setup used by the Pile Integrity Tester).

2. Microseismic (MS) techniques which utilize the data acquisition and processing methods used commonly by geophysical surveyors.
3. Parallel Seismic (PS) methods which are based upon conventional cross-hole and down-hole methods used by civil engineers to determine dynamic soil properties.
4. Ground Penetrating Radar (GPR) which is a relatively new test concept that utilizes modern radar detection technology to "see" or locate structures (both man made as well as geological) within the shallow subsurface.
5. Dynamic Foundation Response (DFR) methods which are based upon conventional modal analysis techniques.

Figure 2.1 is a schematic representation of how such a test setup might be utilized in the field, for purposes of evaluation of unknown bridge foundations.

2.1 Test Equipment

A major portion of the test equipment which was targeted for use with this study was purchased from Olson Engineering of Golden, Colorado, a firm which specializes in non-destructive testing and evaluation of a wide variety of structures. The equipment purchased consists primarily of data acquisition hardware and software, and includes:

- A computerscope card with upto 16 channels of data acquisition capability. The card includes software for time vs. voltage data acquisition with a maximum sampling rate of 1 MHz (recording 1 channel of data) and with a 12 bit ADC. The card is manufactured by RC Electronics Inc. of Santa Barbara, California, and can be used with almost any personal computer running Microsoft DOS with at least one full length open card slot (8 or 16 bit).
- A signal conditioner box with 3 receiver/amplifier channels with selectable gain and highpass and bandpass filters. This can process upto 3 channels of geophone/accelerometer analog input, plus a fourth input to condition an impulse hammer (dynamic force transducer).
- A 12 lb Impulse Sledge (dynamic force transducer) to act as the source, and accelerometers having 2 different sensitivity levels (receivers) manufactured by PCB electronics.

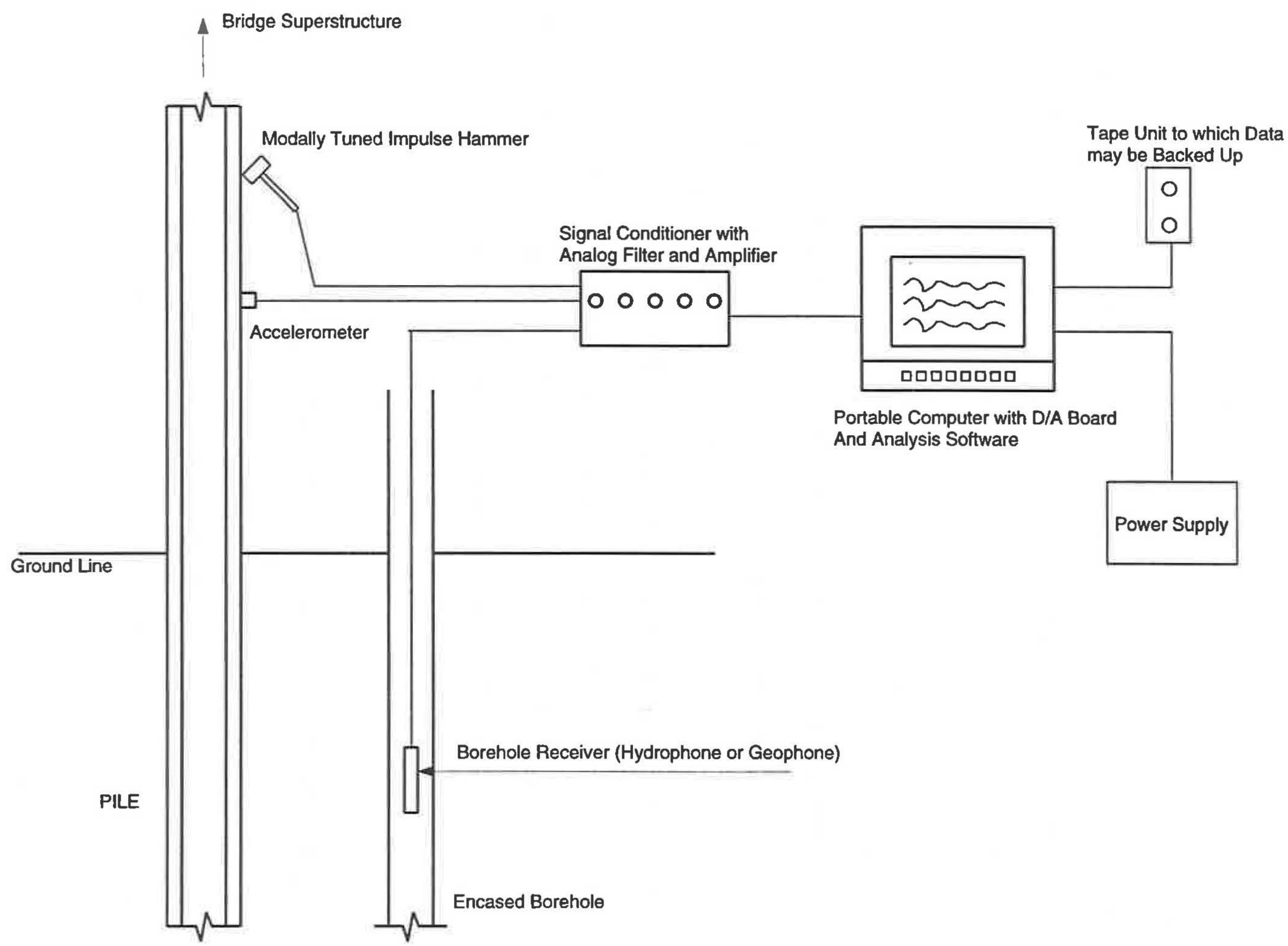


Figure 2.1 : Generalized Test Setup and Equipment

- Software for the analysis of time, frequency and transfer function data (more software has been developed as part of this study).
- Two 3-component Waterproof Geophones (velocity transducers).
- Three Standard source/receiver hydrophone probes.
- Portable computer system (486DX2-66MHz) to house the data acquisition card and for data analysis.

Some equipment which was used as part of the initial study (like the Ground Penetrating Radar) was provided by Geophysical Survey Systems, Inc. (GSSI) and by Olson Engineering.

2.2 Test Sites

The foundation elements of 4 bridges have been tested as part of this study. Two of the bridges were supported by steel-H piles which extended from the bottom of the bent structural member (reinforced concrete beam) to the ground level, and were continued on under the ground surface to form the foundation. One of the bridges was supported on timber piles, and the last bridge was supported on steel-H piles also, but in this case the piles were encased in concrete from the level of the bent to a certain depth beneath the ground. Specific details regarding the construction of the different foundations tested has been provided in the following pages.

2.2.1 US Highway 31, Bridge over the Alabama River

The bridge over the Alabama River (US Highway 31, northbound lane) was selected as the test site for conducting the primary series of field trials for the test methods and equipment mentioned earlier. The field trials were carried out as a cooperative effort between Olson Engineering and the Auburn University research team. These preliminary efforts concentrated upon the determination of embedded lengths of the steel H piles supporting Bent #4, northbound lane, located upon the south bank of the Alabama river. The structural details of the bridge bent along with the recorded soil conditions at the site have been provided in the following pages. Methods 1 through 4 (SE/IR, MS, PS, GPR) described earlier were used to obtain test data. A comprehensive analysis of this volume of data is presented in the next chapter of the report.

Structural Details of Test Foundation

The foundation elements tested were located below the bridge over the Alabama River on US Route 31 (northbound lane), between Montgomery and Prattville. The details of the existing bridge structure were provided by the Alabama Department of Transportation. These consist of the plan and elevation drawings of the entire bridge (north-bound lane), and included structural details of the pile superstructure that exist under some of the bents. After a study of the plans and a visit to the proposed test site, Bent #4, located upon the South bank of the river, was chosen for testing purposes. The reasons behind this choice were as follows:

- easy access to the foundation elements
- approximately 30 feet of headroom, which would allow the placement of casings within the earth and located very close to the selected piles with conventional drilling equipment.

Figure 2.2 provides a schematic diagram of the structural elevation at Bent #4 (facing North), with piles being numbered 1 through 5 from left to right. A two-story sway-bracing system used to provide additional stiffness has also been included in the same figure. At the top, the piles are connected directly to the concrete structural member comprising the Bent. At the ground level, the piles are encased in concrete (1.25 × 1.25 feet square) for a total depth of 5 ft. (2 ft above and 3 ft below the existing ground line). Figure 2.3 shows the plan view of the same bent.

Soil Conditions

The test site is located on the south bank of the Alabama River. The soil stratification (borehole log) has been presented in Table 2.1. The table represents the data logged for the Borehole E (Figure 2.3), at a distance of about 10 feet from the outermost pile on Bent #4. The values of blowcount (N) from the Standard Penetration Test are presented in Table 2.2.

Table 2.1 Description of Soil Type for Borehole "E"			
Layer	Depth (feet)		Soil Type
	From	To	
1	0.0	9.4	Stiff to moist brown Clay
2	9.4	34.0	Brown/Tan Clay with Sand
3	34.0	40.3	Dense brown Sand with Silty Clay
4	40.3	60.0	Very dense brown Sand with some silty Clay

Table 2.2 SPT Blowcount Records for Borehole "E"					
From (ft)	To (ft)	Penetration (ft)			Blowcount (N)
		0.5	1.0	1.5	
9.0	10.5	9	10	12	22
19.0	20.5	5	7	9	16
29.0	30.5	7	10	17	27
39.0	40.5	17	24	26	50
49.0	50.4	13	25	25/4	50/0.9
59.0	60.0	22	50		50

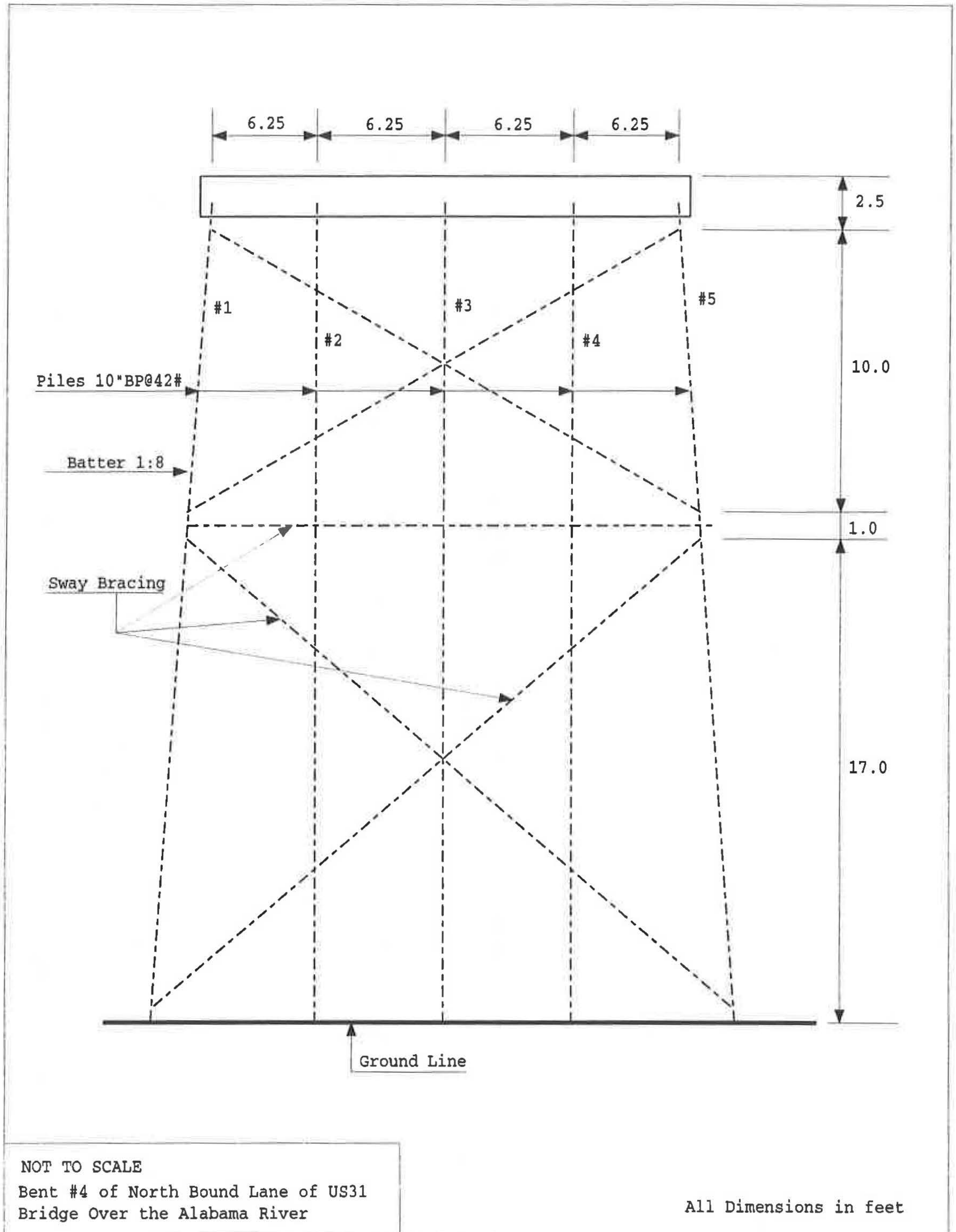
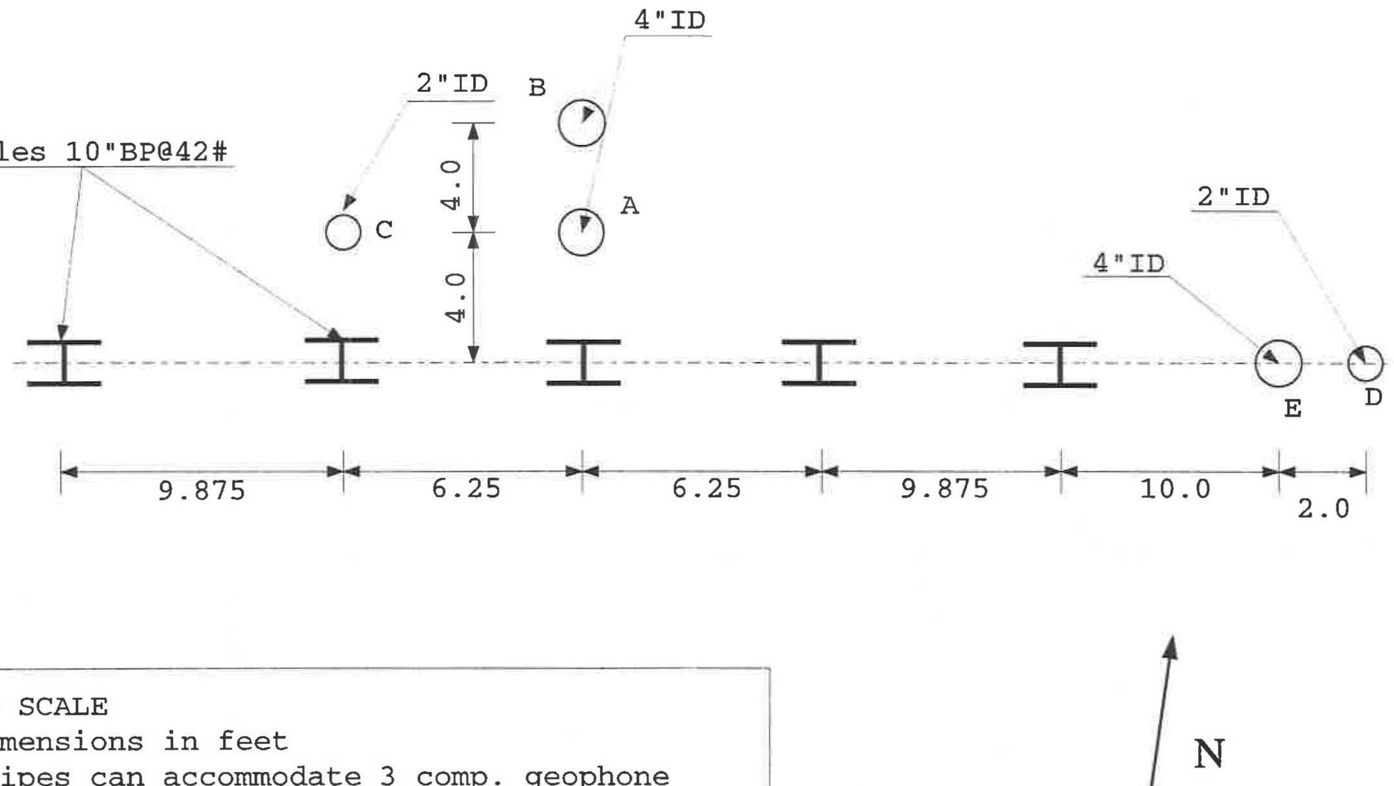


Figure 2.2 : C/S Detail of Bent #4 (facing North) Showing Sway Bracing



NOT TO SCALE
All dimensions in feet
4"ID pipes can accommodate 3 comp. geophone
2"ID pipes can accommodate hydrophone

Figure 2.3 : Plan View of Bent #4 at GL showing Borehole Locations

2.2.2 US Highway 80, Bridge over Chubahatchee Creek

The bridge over the Chubahatchee Creek on the US Highway 80 is located in Macon County, Alabama, slightly west of Shorter near mile marker 59. The bridge is supported on steel-H piles which also extend from the bottom of the bent to beneath the ground surface and form the primary foundation elements. In particular, the efforts of the research team were directed toward the determination of embedded pile lengths supporting Bent #3, which is located on the western side of Chubahatchee Creek. The clearance from the ground surface to the bottom of the bent structural member was about 18 feet, so only the microseismic profiling technique could be used, as the placement of encased boreholes using a conventional drill rig would require more head room, or else drilling from the bridge deck. Thus, in essence, all the tests conducted at this site were truly non-invasive in nature.

The plan and elevation drawings of Bent #3 were provided by the Alabama Department of Transportation. A two-storey sway bracing system was used to provide additional stiffness for the part of the foundation that exists between the ground surface and the bridge bent. The details of the bent are shown schematically in Figure 2.4. A 16 inch square concrete collar was also placed around each of the piles at the level of the ground.

A record of the soil conditions at the site was also provided along with the structural details of the bent. The surface layer is a Brown Silty Sand approximately 10 feet deep. This is underlain by a layer of Grey Sand about 4 feet deep, followed by a layer of Gravel about 3 feet in thickness. The bottommost layer is Grey Marl.

2.2.3 Bridge on Conecuh County Road 42, Evergreen, Alabama

This particular site consisted of a bridge spanning a small stream along Conecuh County Road 42 in Evergreen, Alabama. The bridge deck was supported on two intermediate bents, each resting on 4 timber piles which were all approximately 12 inches in diameter. The distance from the bottom of the bent to the mud line was approximately 8 feet, and thus only the microseismic technique was used to determine the embedded

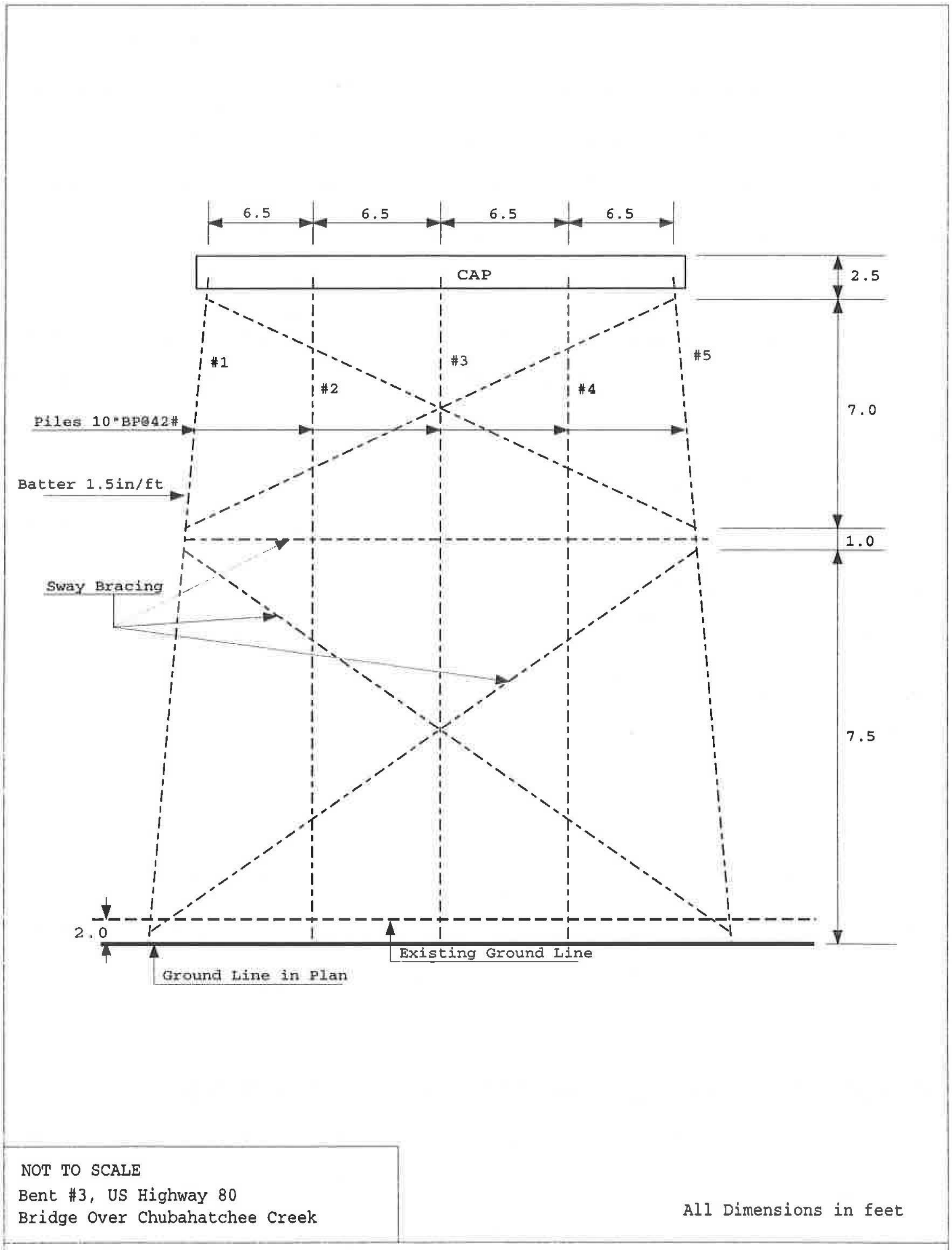
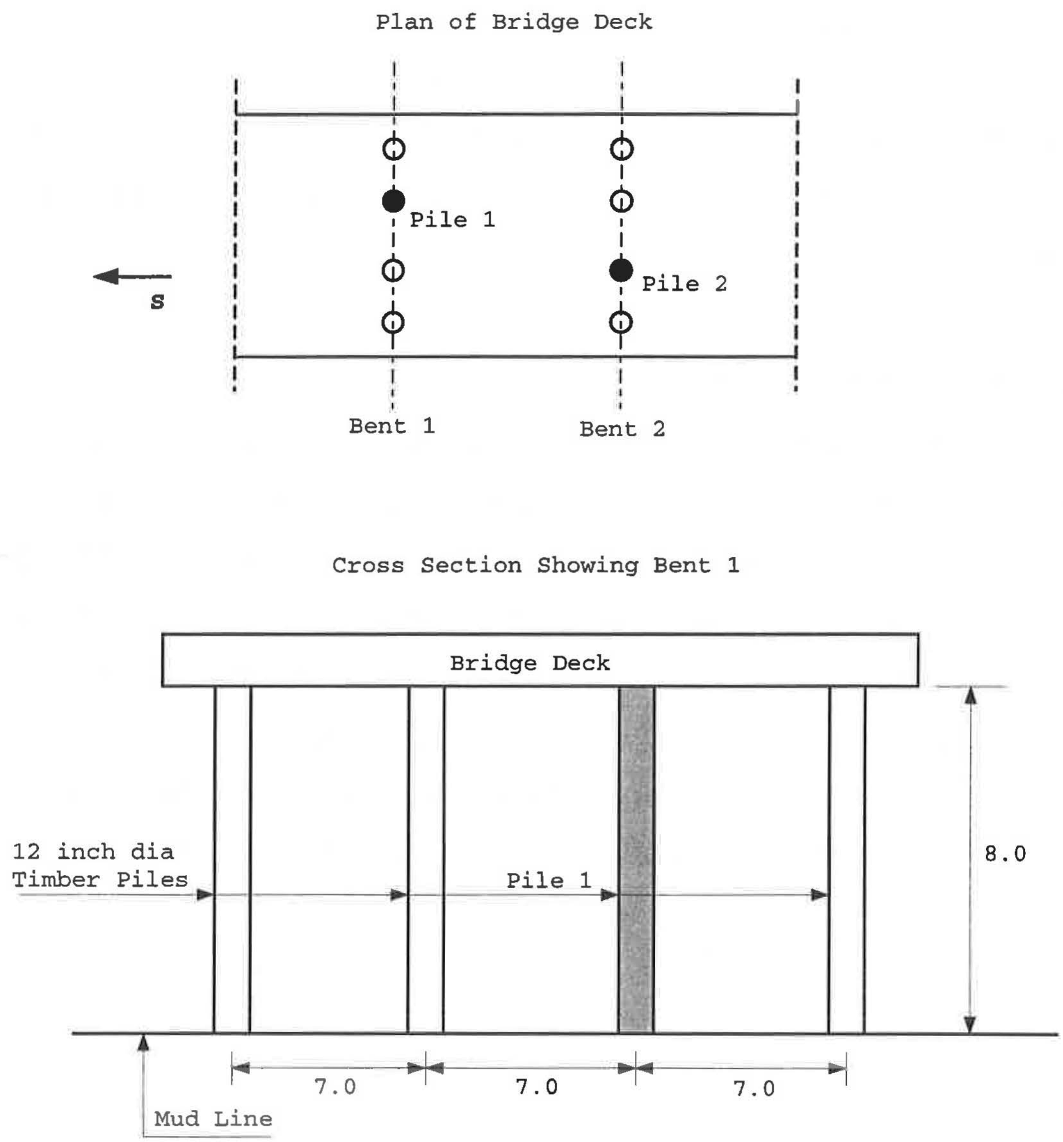


Figure 2.4 : Schematic of Bent #3, Bridge Over the Chubahatchee Creek, US80



NOT TO SCALE
Plan and Elevation
Bridge On Conecuh County Rd42

All Dimensions in feet

Figure 2.5 : Bridge on Conecuh County Road 42, Evergreen, Alabama

lengths of the timber piles. The plan and elevation of the bridge deck and the foundation is shown in Figure 2.5. Two piles (marked 1 and 2 in the figure) were actually tested. This site proved to be of particular significance because the State DOT has recently torn down the old bridge and replaced it with a new one. The old piles that were tested could thus be pulled out of the ground, and provided a means of directly verifying the testing procedure. The soil stratification data at the site was not available at the time of testing, and was not considered to be of any great significance as these timber piles were not expected to have been placed very deep.

2.2.4 Bridge on Conecuh County Road 43, Evergreen, Alabama

This particular bridge is located on Conecuh County Road 43 in Evergreen, Alabama, and is used to span a small stream along the route. The bridge is supported at 6 intermediate bents, each resting on a foundation of 3 concrete encased steel-H piles. The headroom available beneath the bridge deck varied from about 8 to 10 feet, and thus again, the Microseismic Profiling technique was the only practical solution. The plan and elevation of the bridge deck and the foundation is shown in Figure 2.6. Two piles (marked 1 and 2 in the figure) on Bent 5 were actually tested. The soil stratification at the bridge site and the actual building records (bridge and pile driving data) were also not available.

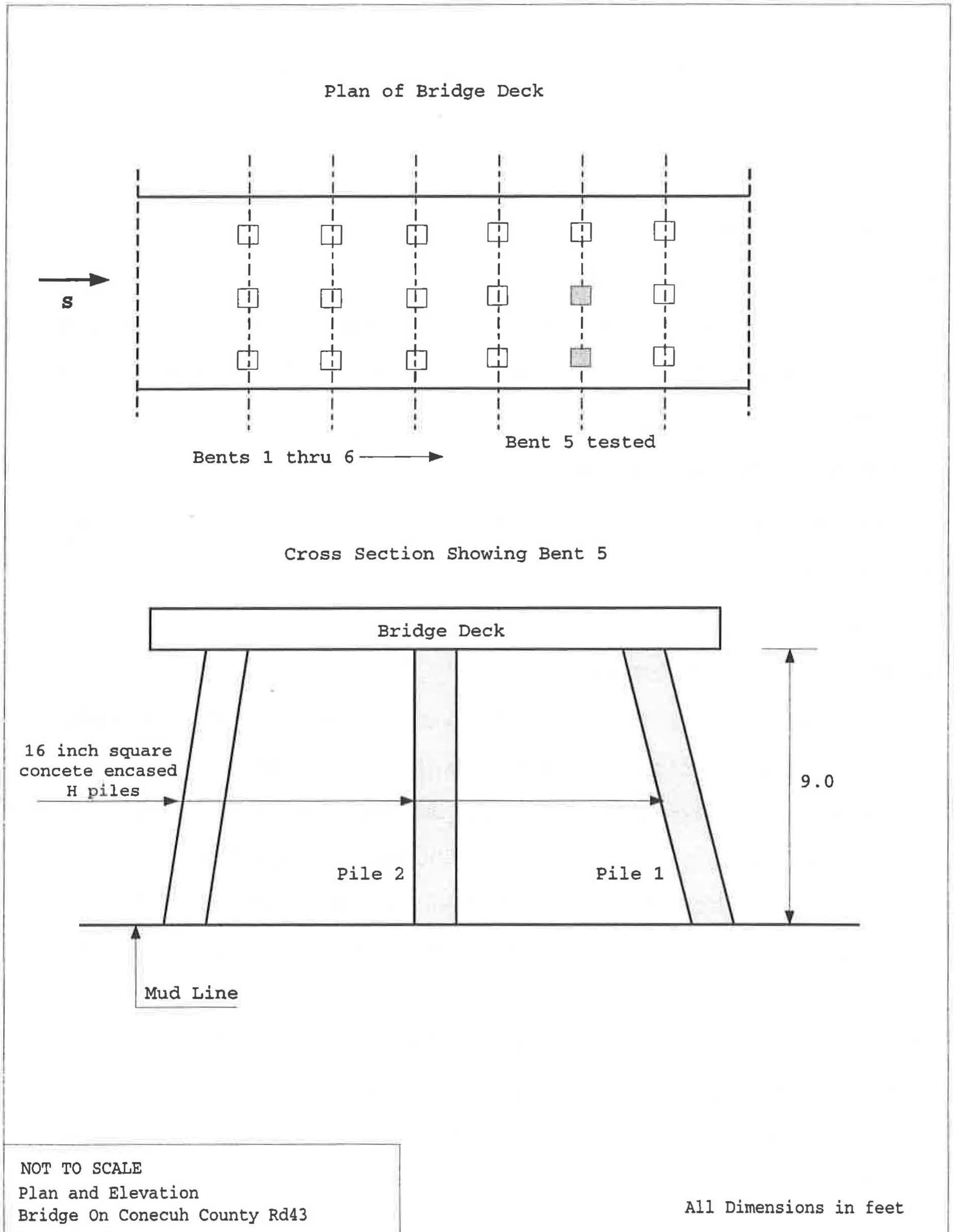


Figure 2.6 : Bridge on Conecuh County Road 43, Evergreen, Alabama

3. Test Procedures and Results from Field Trials

This chapter of the report provides:

a. a description of the test procedures adopted

b. a summary of the results obtained from the field trials

As was noted in the previous chapter, a total of 5 methods were identified from a study of existing literature and selected for rigorous field evaluation. The 5 methods are as follows:

1. Sonic Echo/Impulse Response
2. Microseismic Profiling
3. Parallel Seismic
4. Ground Penetrating Radar
5. Dynamic Foundation Response

Methods 1 through 3 are "low strain dynamic testing methods" performed primarily for checking integrity of existing foundations. Method 4 does not use the input of any externally applied forcing function to "excite" the foundation; it simply uses modern radar detection technology to map the presence of buried structures in the shallow subsurface. Method 5 involves the excitation of the foundation and the associated superstructure over a broad range of frequencies; the measured structure/foundation response is used to infer the dynamic foundation properties. As such, this method may involve the generation of large strains/displacements at or near the resonant frequency peaks of the structure/foundation system. Due to this reason, no actual tests were performed. However, a brief description of the method has been provided as it might prove to be of significance in certain types of testing.

3.1 Sonic Echo/Impulse Response Techniques (SE/IR)

The SE/IR methods provide a simple non-invasive means to determine embedded foundation lengths for existing bridge foundations. Both methods are based upon conventional wave propagation theory and differ only in the domain within which analysis is carried out (time or frequency). The Sonic Echo method analyzes foundation response in the time domain by identifying reflectors in the recorded velocity time histories. The Impulse Response technique transforms the time domain response to the frequency domain, which data can then be used to infer dominant foundation lengths at resonance.

The SE/IR field trials can be carried out using the data acquisition and processing capabilities of the Pile Integrity Testing Equipment (PIT). Tests have been carried out at the Alabama River test site using both the PIT equipment (made available by the Alabama Department of Transportation), as well as the more general purpose data acquisition hardware purchased for the current project by Auburn University. SE/IR results have been obtained from the PIT data, and are presented in this section (3.1) of the report. The more detailed information available from the generalized data acquisition system have been analyzed and presented in section 3.2 (Microseismic).

The use of the Pile Integrity Tester is based upon the recovery of depth dependent information from time dependent single point measurements at or near the pile head. For many existing structures, information regarding foundation conditions may not be available. PIT measurements can be obtained even in cases where access to the foundation elements themselves are very restricted.

3.1.1 Theory and Test Procedure

The procedure consists of the application of a sharp or well defined force pulse at or near the pile head. An instrumented hand held hammer is most commonly used for this purpose. Such an impact typically produces strains within the pile material of the order of 2-10 μstr (Holeyman, 1992). The nature of the pulse generated within the pile depends upon both the mass and the stiffness characteristics of the hammer head and the velocity with which the hammer impacted the pile surface, as well as the manner in which the operator imparts the blow. Heavier hammers tend to produce pulses which contain lower frequency energy and are thus less dispersive in nature. Heavy hammers are thus better suited for the purpose of producing more prominent "toe" reflections. Lighter hammers produce pulses which contain relatively more high frequency dispersive energy, and thus provide better resolution at shallower depths.

A longitudinal or compressive wave is generated within the pile if the pile head or pile surface is impacted vertically. This wave propagates down the shaft with a velocity 'c', which is described by the following equation:

$$c = \sqrt{\frac{E}{\rho}}$$

where,

c = compressive wave propagation velocity within shaft

E = modulus of elasticity of the pile material

ρ = mass density of the pile material

The dynamic response of the pile can be interpreted in terms of conventional time history analysis. However, elastic wave propagation theory is a much simpler and more practical tool for analyzing the measured response at the pile head. The acceleration response at the pile head (or at the surface of the pile when the head is not accessible) is monitored and recorded using accelerometers. The PIT equipment uses a field data acquisition system to convert the analog signal(s) (acceleration and possibly hammer force) to digital format. The acceleration record is integrated to produce a velocity signal, in which format further analysis is usually carried out. Several such velocity records (produced by repeated hits at the same location) may also be averaged, which decreases the effects of random noise on the signal to be interpreted. This practice of averaging several signals to highlight common or repeated features within each trace is known as stacking, and its origin lies in the seismic data processing techniques commonly used for geophysical mapping. The hammer force may or may not be recorded; analysis may be carried out using just the velocity records.

Interpretation of velocity records is based on the premise that changes in pile impedance and soil resistance down the length of the shaft produce predictable wave reflections at the pile head (Hussein et al, 1992). The pile impedance Z may be defined as the ratio of the force in the shaft to the velocity of propagation of the stress wave :

$$Z = \frac{EA}{c}$$

where A represents the c/s area of the shaft. If an impedance change from Z_1 to Z_2 is encountered by the impact wave at any cross section, a part of the energy is reflected back up (F_u) while another part continues downward (F_d). Such a division of energy in the form of 2 waves travelling in opposite directions satisfies both continuity and equilibrium conditions.

$$F_d = F_i\,[2Z_2/(Z_2+Z_1)]$$

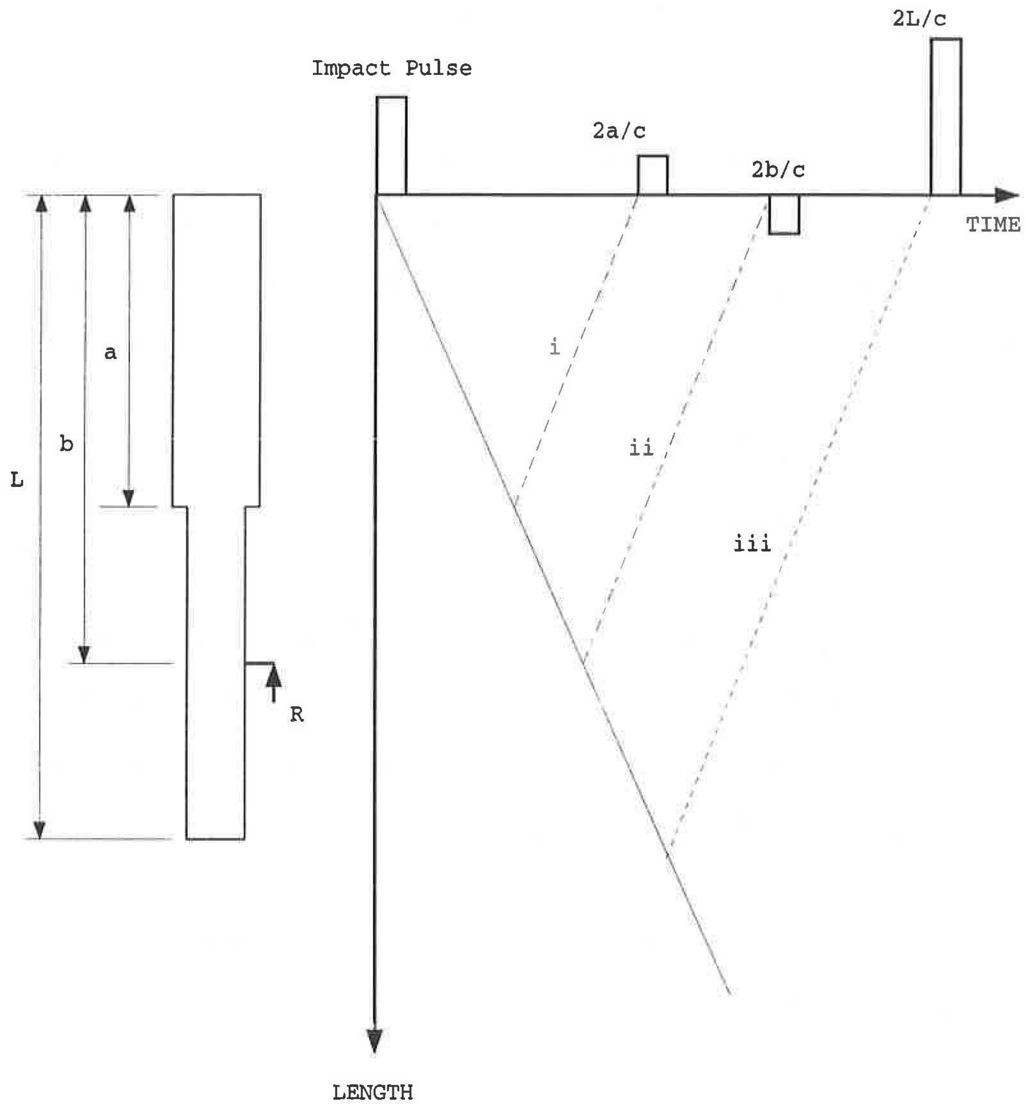
$$F_u = F_i\,[(Z_2-Z_1)/(Z_2+Z_1)]$$

where F_i represents the original incident wave/force pulse. At the pile toe, the impact wave is either wholly or partially reflected back. A fixed end produces a reflected compression wave whereas a free end produces a reflected tension wave. The phenomenon of wave reflections within and at the end of the pile is shown schematically in Figure 3.1.1. An impedance change is encountered at a depth 'a', whereas an abrupt soil resistance is encountered at depth 'b'. The abscissa represents the time axis with the relative magnitudes and signs of the impact and reflected waves produced at the pile head. The ordinate represents vertical distance down the shaft.

3.1.2 Determination of Unknown Pile Length

The velocity records may be used to determine the embedded pile length. This can be done in the following 2 ways:

- In the **Time Domain** : It is usually possible to "see" the toe reflection (pulse reflected from the toe) by closely studying the velocity time history at the pile head. In some cases, specially with long piles with high slenderness ratios and with high values of soil resistance along the shaft, this toe reflection may not be apparent from the velocity record. In such cases, the velocity record may be amplified



Reflections from :
(i) impedance reduction
(ii) passive resistance R (velocity proportional)
(iii) pile toe

Figure 3.1.1 : Impact and Wave Reflections at Pile Head (Hussein et al., 1992)

exponentially with time, which enhances and helps to identify weak signals with delayed arrived times. The PIT postprocessor program possesses such an exponential amplification windowing capability. Once the reflection from the toe has been identified, the depth to the toe may be computed as:

$$d = t \times c \div 2$$

where, d = depth to toe (from point of measurement)

c = compression wave velocity within pile material

t = time interval between successive crossings at the accelerometer

- In the **Frequency Domain** : Such analysis may be carried out using either the velocity record alone, or both the force and velocity records. In the case of velocity only, the Fast Fourier Transform may be used to convert the velocity time history to a velocity spectrum (amplitude vs. frequency). This spectrum may be used to identify resonant frequencies of vibration by establishing successive amplitude peaks and measuring the frequency difference between them. Depending upon the boundary conditions imposed upon the problem (fixed/free ends), closed form solutions for the resonant frequencies may then be used to estimate the "resonant" length (Timoshenko and Weaver, 1990). The PIT postprocessor provides such a tool, which performs a 2nd FFT on the velocity spectrum and indicates "velocity reflectors" as a function of length.

When dealing with both velocity and force records, the PIT may be used to generate a mobility spectrum. Mobility is defined as the inverse of impedance and has the units of velocity/force. The mobility spectrum, generated by performing a FFT on the ratio of the measured velocity to force, may then be analyzed in exactly the same manner as the velocity spectrum (discussed previously) by studying mobility reflectors which indicate dominant acoustic lengths. The mobility spectrum may also be used to determine the dynamic elastic spring stiffness of the pile as well as the characteristic impedance down the shaft.

Figures 3.1.2, 3.1.3 and 3.1.4 provide an example of analysis in both the time and the frequency domains. Figure 3.1.2 shows the velocity record for an "ideal" case. This pile was in fact tested before installation as part of another study, using the PIT equipment. A very clear reflection from the toe is evident from the figure, which corresponds to a pile length of 19.0 ft. Figure 3.1.3 represents the velocity spectrum, with very clearly defined amplitude peaks indicating the predominant vibration frequencies, and corresponding acoustic lengths. Figure 3.1.4 is a plot of the velocity reflectors, which simply confirms the conclusions that can be arrived at by studying the velocity spectrum. Here too, a clearly defined reflector indicates the presence of the pile toe at a "depth" of 19.0 ft.

3.1.3 Test Results

PIT test data for pile #3, Bent #4 of the Alabama River Bridge was obtained by the Auburn University research team using the ALDOT equipment. The data provided was in the following format :

- File ATEST : consisting of both measured velocity and hammer force
- File DTEST : consisting of a measured velocity trace only

In both cases, the pile was impacted vertically at the top surface of the concrete casing, which is placed at an elevation of 2 feet above the existing ground surface. The vertical acceleration response was also monitored at the same location by fixing the accelerometer to the top of the casing surface.

The bottom of the pile tested (#3) is believed to lie at a depth of 39.0 feet below the ground surface. This information has been recovered from the original pile driving records. Thus the distance from the receiver to the pile toe is 41 feet (=39+2). The cross sectional area of the pile is 12.4 in^2, and the longitudinal wavespeed is estimated to be 16800 ft/sec.

Analysis of ATEST data

a. Velocity Data Only :

Figure 3.1.5 indicates the velocity and acceleration time history, exponentially amplified with a factor of 8.0 at a time corresponding to 2L/c, where L and c indicate the pile length and longitudinal wavespeed respectively. Figure 3.1.6 shows the velocity time history in somewhat greater detail. The velocity traces in this figure have been shown both with (bottom trace) and without amplification. The beginning of the toe reflection is not sharply defined, even with an amplification factor of 8.0. However, the data appear to suggest the existence of the toe at a distance between 38 and 42 feet from the receiver. This corresponds to a range of about 36 to 40 feet for embedded length (receiver placed 2 feet above the ground surface), which is in good agreement with the actual embedded length of 39 feet. The velocity data also suggest the existence of increasing side friction from the soil from a depth of roughly 10 feet.

Figure 3.1.7 is a plot of the velocity spectrum, obtained by transforming the velocity data in the time domain to the frequency domain, using the Fast Fourier Transform. The frequency separation between the first 3 dominant peaks is roughly 53 Hz. Dominant resonant lengths may be calculated in the following manner:

- for a free-free or fixed-fixed boundary at each end, the acoustic length will be given by

$$L = \frac{Nc}{2F_N}$$

  where, N is the mode of vibration being considered and F_n is the frequency separation between successive peaks. This equation gives us an acoustic length of 158 feet for the pile for the first mode, which clearly cannot be the case.

- for a fixed-free boundary condition, the acoustic length is calculated as

$$L = \frac{1}{4}(2N-1)\frac{c}{F_N}$$

  The second equation translates to an acoustic length of 79 feet for the first mode, which is closer to the actual value of 69 feet (the distance from the pile toe to the bent cap), but still not sufficiently accurate. The velocity spectrum possesses peaks which indicate a dominant resonant length of 29 feet. This might correspond to the length of the pile above the ground line (roughly 29 feet from ground surface to bent cap), which should contain the major portion of the vibration energy. The velocity reflectors derived from a FFT on the velocity spectrum, also do not indicate any dominant resonant lengths.

b. Velocity and Force Data combined :

Figure 3.1.8 includes plots of the velocity, force, and velocity minus force (V-F) data. The V-F data clearly indicates the existence of the toe signal at a distance of 40 feet from the receiver. This leads to a value of 38 feet for the embedded length, which is in very close agreement to the recorded value of 39 feet. It should be noted that the V-F signal presented was plotted with an amplification factor of 10.0.

Figures 3.1.9 and 3.1.10 represent the mobility spectra and mobility reflectors respectively. A frequency analysis leads to the same results as were obtained from the velocity spectra, and no dominant acoustic length could be estimated from this information.

Analysis of DTEST Data

The DTEST data consisted of just the recorded velocity trace (no force record). The amplified velocity and acceleration have been presented in Figure 3.1.11, and the velocity data alone has been shown in somewhat greater detail in Figure 3.1.12. The record is seen to contain quite a bit of high frequency vibration, possibly caused by vehicle movement upon the bridge. This higher frequency noise can be removed by processing the signal through a low pass digital filter; however, the PIT postprocessor does not include such a feature. A number of arrival pulses can be viewed in the velocity record, and it would be hard to discriminate the toe signal without foreknowledge. In this case a clear arrival is evident at a distance of 41 feet from the receiver, which can be matched to the toe reflection from the same depth.

The velocity spectrum and velocity reflectors are presented in Figures 3.1.13 and 3.1.14 respectively. At least 3 consecutive well defined peaks need to be present for interpretation of acoustic length, and this is not so for the given spectrum. The velocity reflectors also do not impart any information concerning what the actual length of pile might be.

3.1.4 SE/IR Conclusions and Recommendations

The quantity of data that was available for the PIT test, as applied to the determination of unknown pile lengths under existing bridge structures, is at this point of time extremely limited. Based upon such limited test data, no definite conclusions can really be made. However, certain trends are apparent from a close study of the available data.

- From the point of view of estimating unknown pile lengths from PIT records, no clear-cut strategy exists, and interpretation is largely based upon the experience of the operator. It is perhaps best to "look" for wave reflections from the pile toe within the amplified velocity time history. Multiple reflections from the foundation superstructure supporting the bridge bent can make this a task which cannot be accomplished with any great degree of certainty. However, reflection patterns from the superstructure may be estimated and identified within the velocity record, which might make the identification of the toe reflection easier.
- Analysis in the frequency domain did not produce any results that were indicative of embedded pile length. It can thus be concluded that the simple closed form solutions that exist for longitudinal vibration of "short" prismatic bars do not model the actual vibration characteristics of piles embedded in the ground, which at the top are attached to the bridge superstructure. This is specially true in the case of "long" (read high slenderness ratio) steel piles, where the ratio of the soil-pile interface area to the pile cross-section is proportionally much greater than in the case of reinforced concrete piles. This leads to a much greater attenuation of the signal down the shaft, which in effect damps out the vibration in the fundamental mode i.e. longitudinal vibration of the pile as one unit.

- For this study, force records were available only in one case. For that case, the velocity-force trace did indicate a prominent signal arrival from what is believed to be the toe.

The only type of pile analyzed in this study was a steel pile with a relatively high slenderness ratio. More work is needed to determine if the PIT method works better in the case of concrete and possibly timber piles, as logic suggests it should. The study also indicates the need for development of discrete-parameter models, which could be used to predict or match the pile response in the frequency domain more accurately. The latter, however, is beyond the scope of the discussion on the PIT method, and can be developed only using the data recorded by the more general data acquisition system.

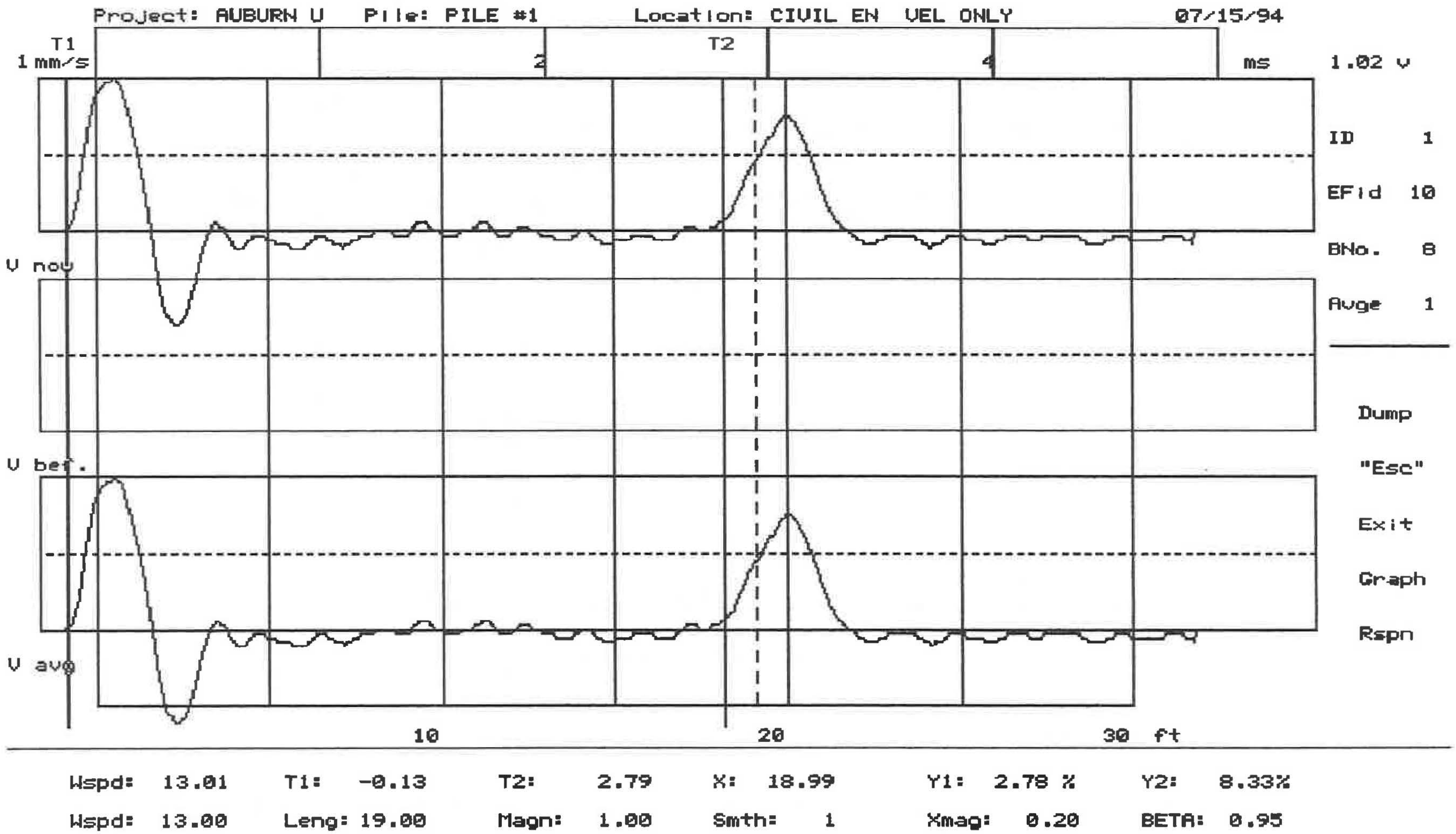


Figure 3.1.2: Velocity Record for Ideal Case

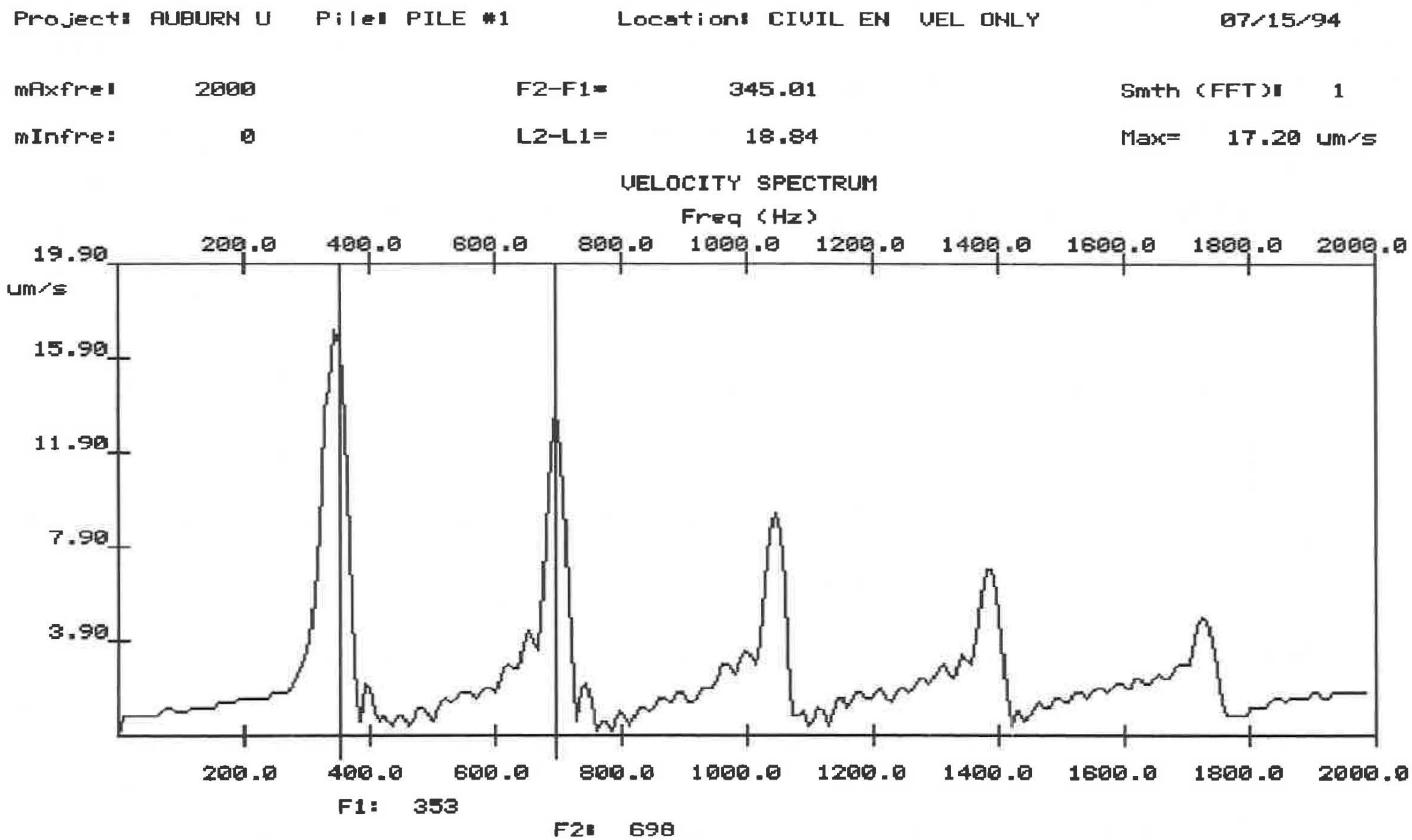


Figure 3.1.3: Velocity Spectrum for Ideal Case

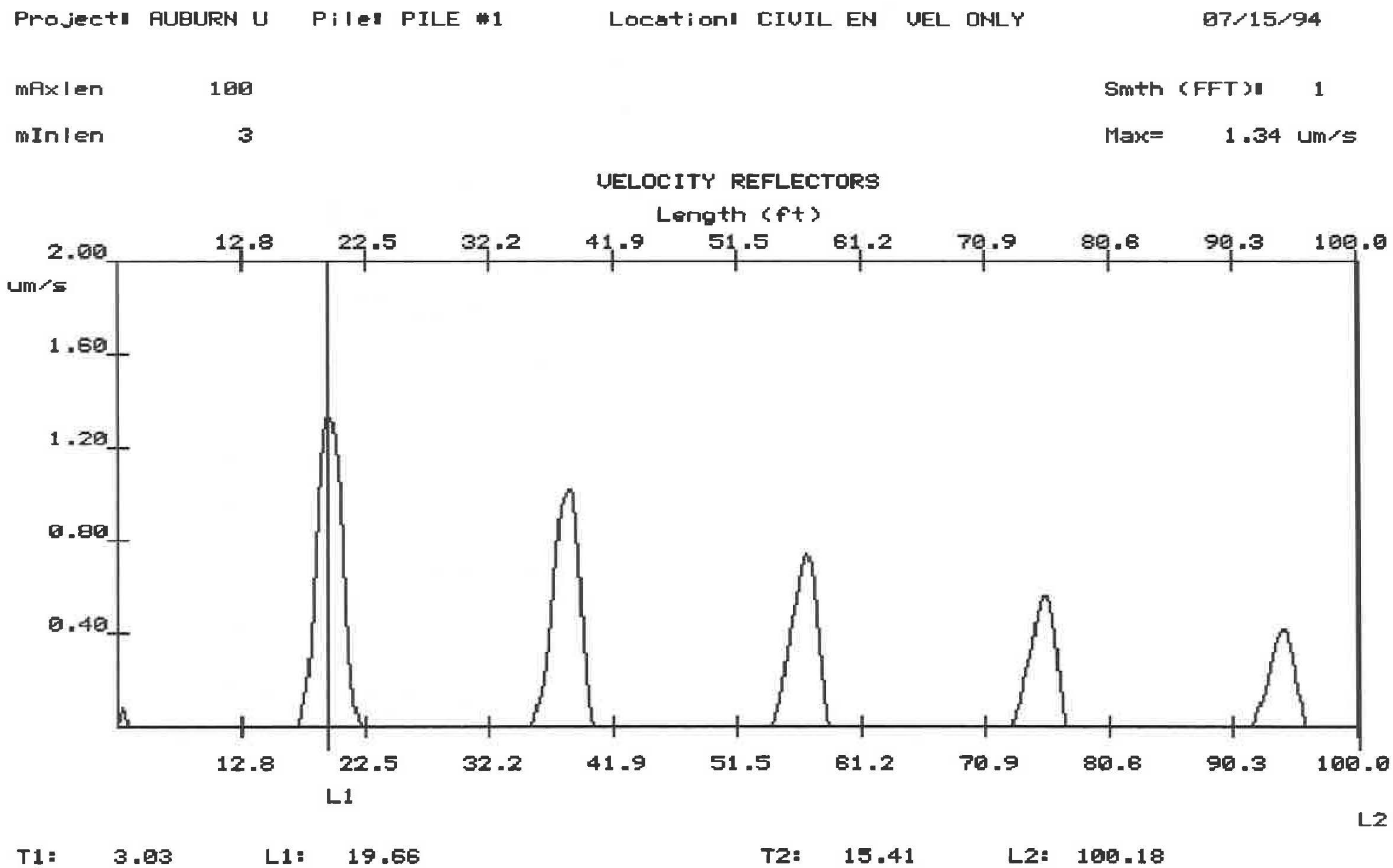


Figure 3.1.4: Velocity Reflectors for Ideal Case

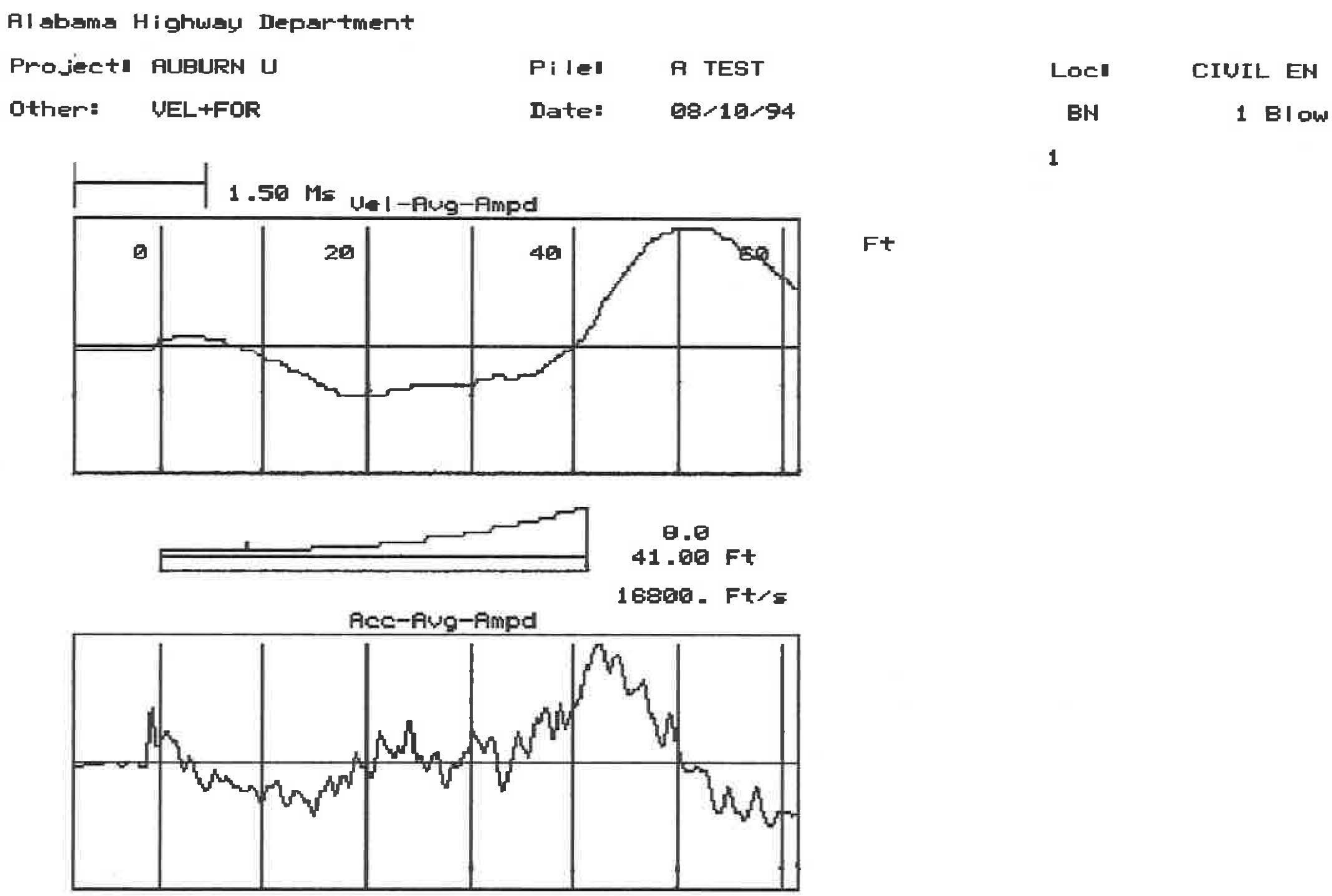


Figure 3.1.5: Velocity and Acceleration Time History (Atest data)

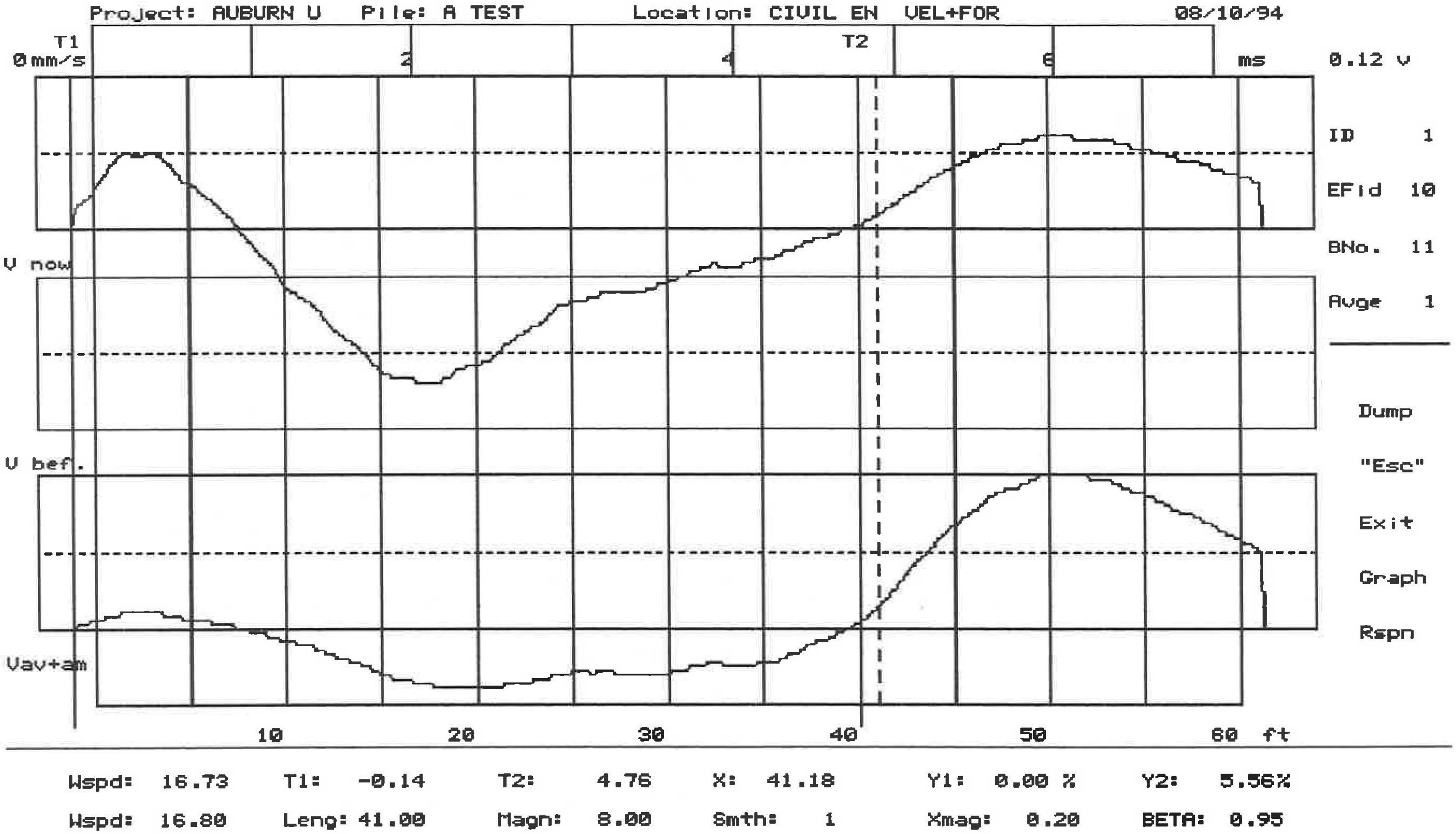


Figure 3.1.6: Amplified Velocity Time History (Atest data)

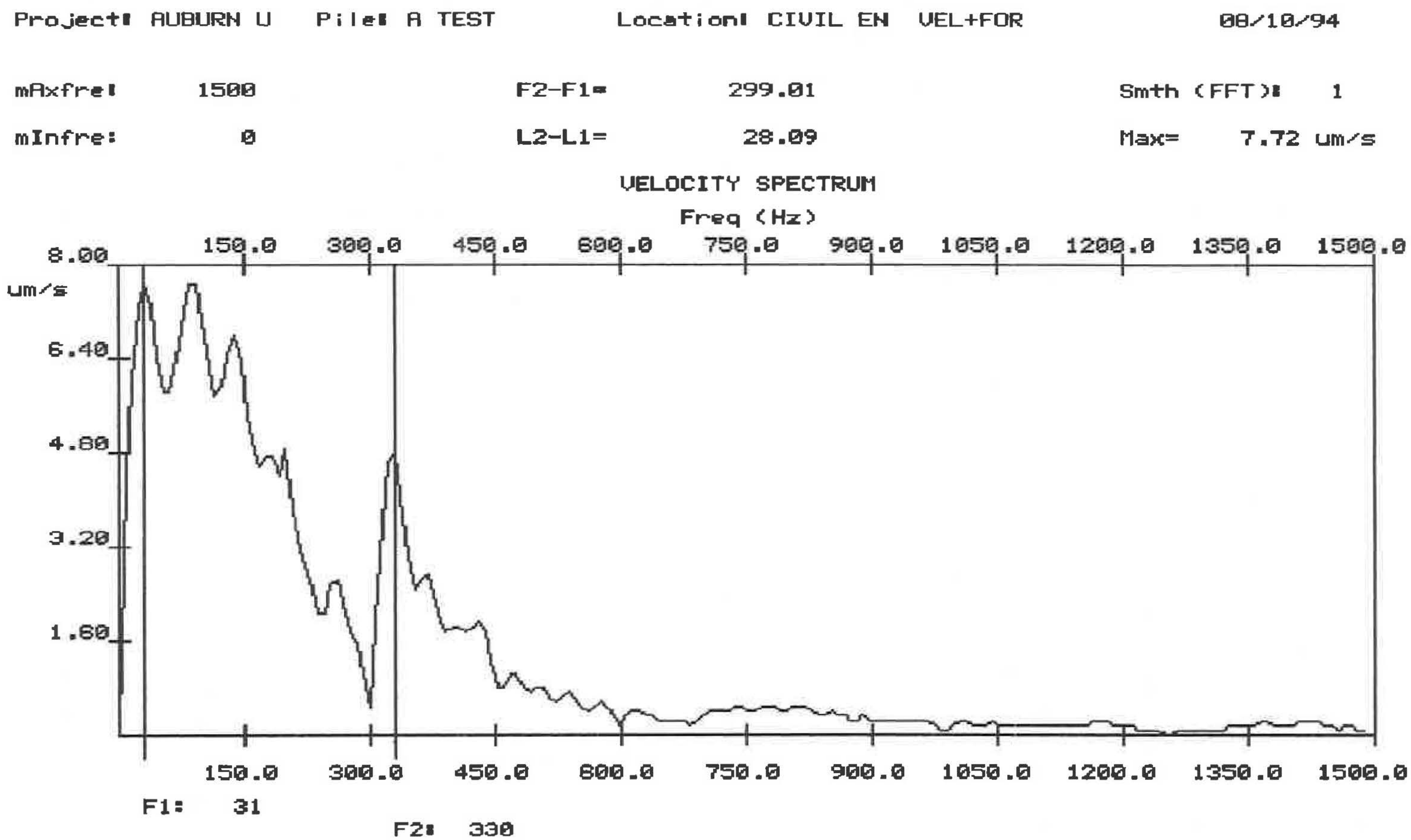


Figure 3.1.7: Velocity Spectrum (Atest data)

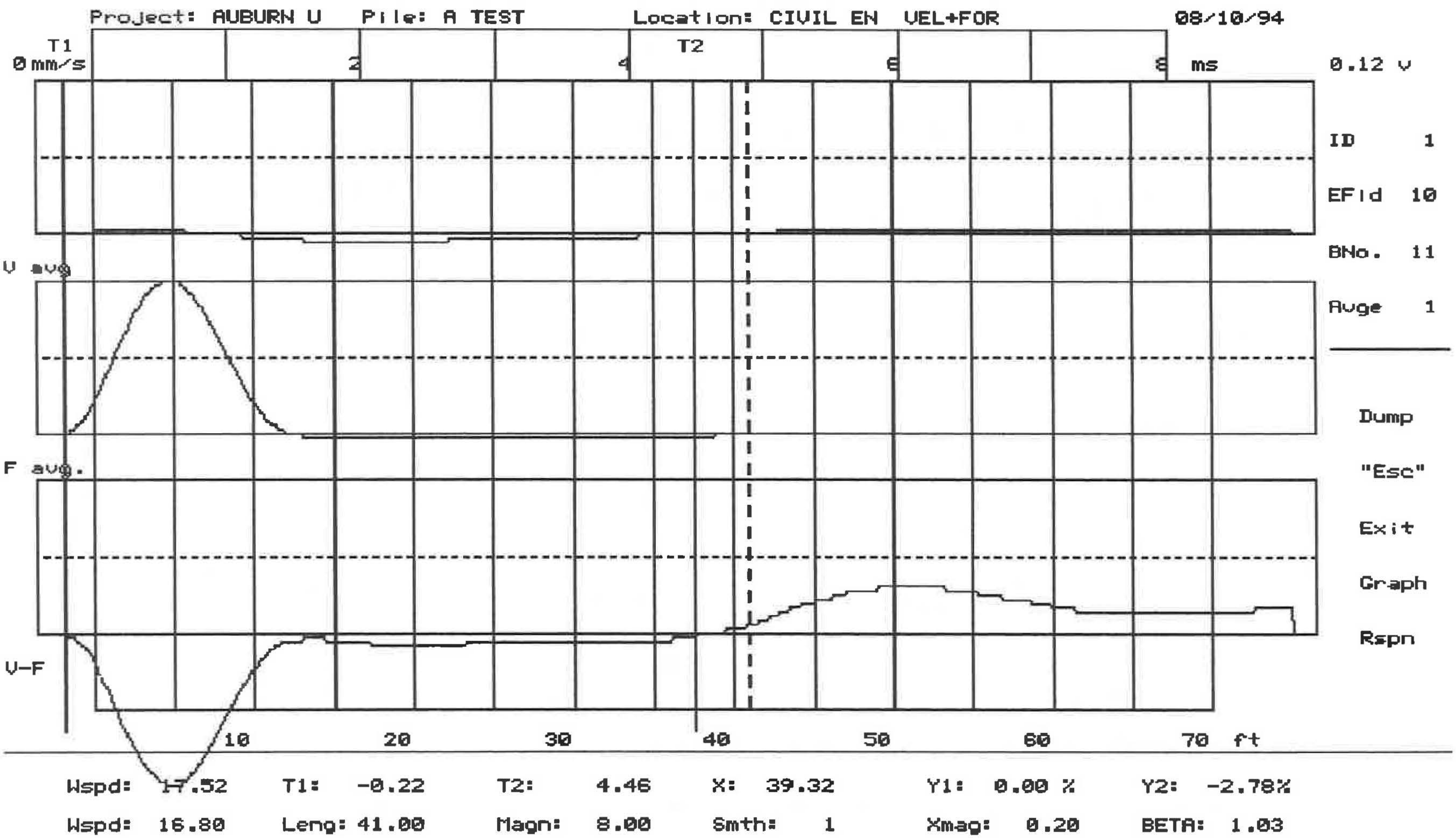


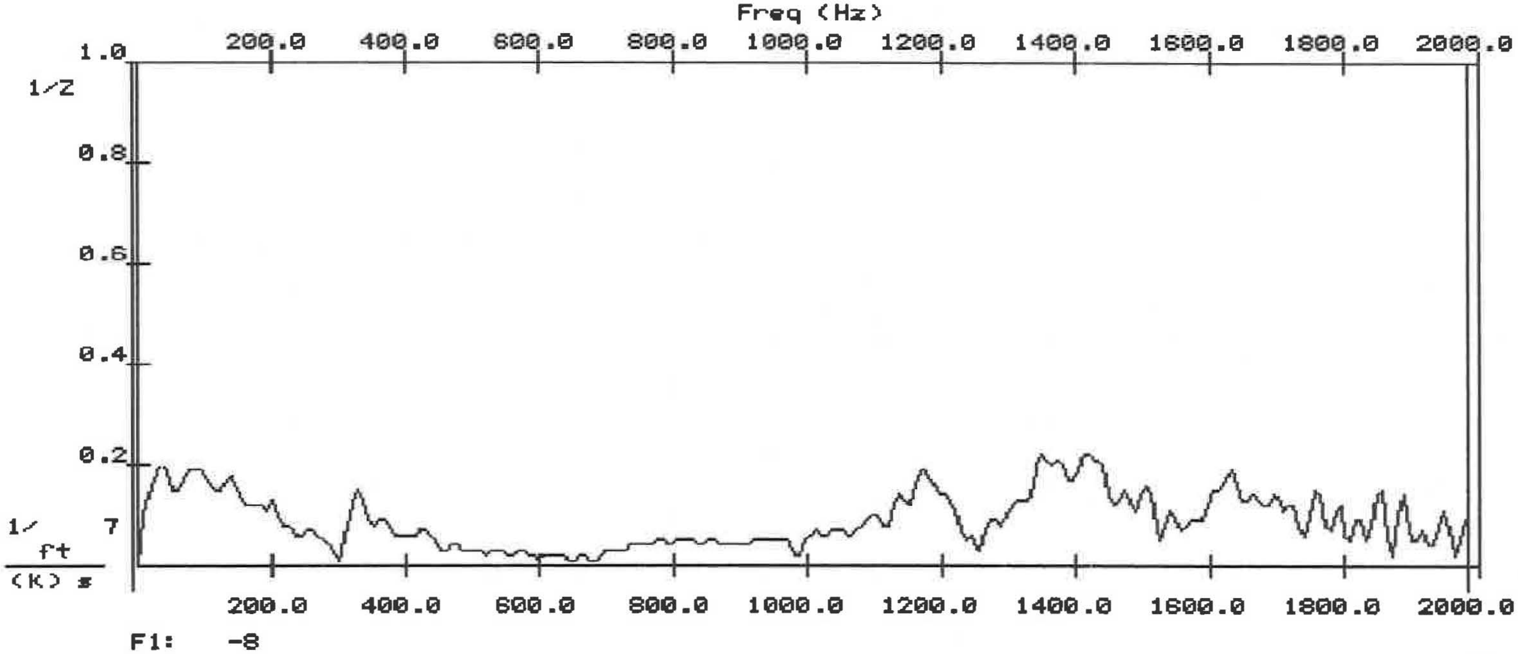
Figure 3.1.8: Velocity, Force, and Velocity - Force (Atest data)

Project: AUBURN U   Pile: A TEST   Location: CIVIL EN   VEL+FOR   08/10/94

mAxfre: 2000   F2-F1= 1993.42   Smth (FFT): 1

mInfre: 0   L2-L1= 4.21   Max= 0.0329 ft/(K) s

MOBILITY



F2: 1986

Freq(Hz)	15	46	77	107	138
Ed(k/ft)	4599	11177	17337	28395	33368

Figure 3.1.9: Mobility Spectra (Atest data)

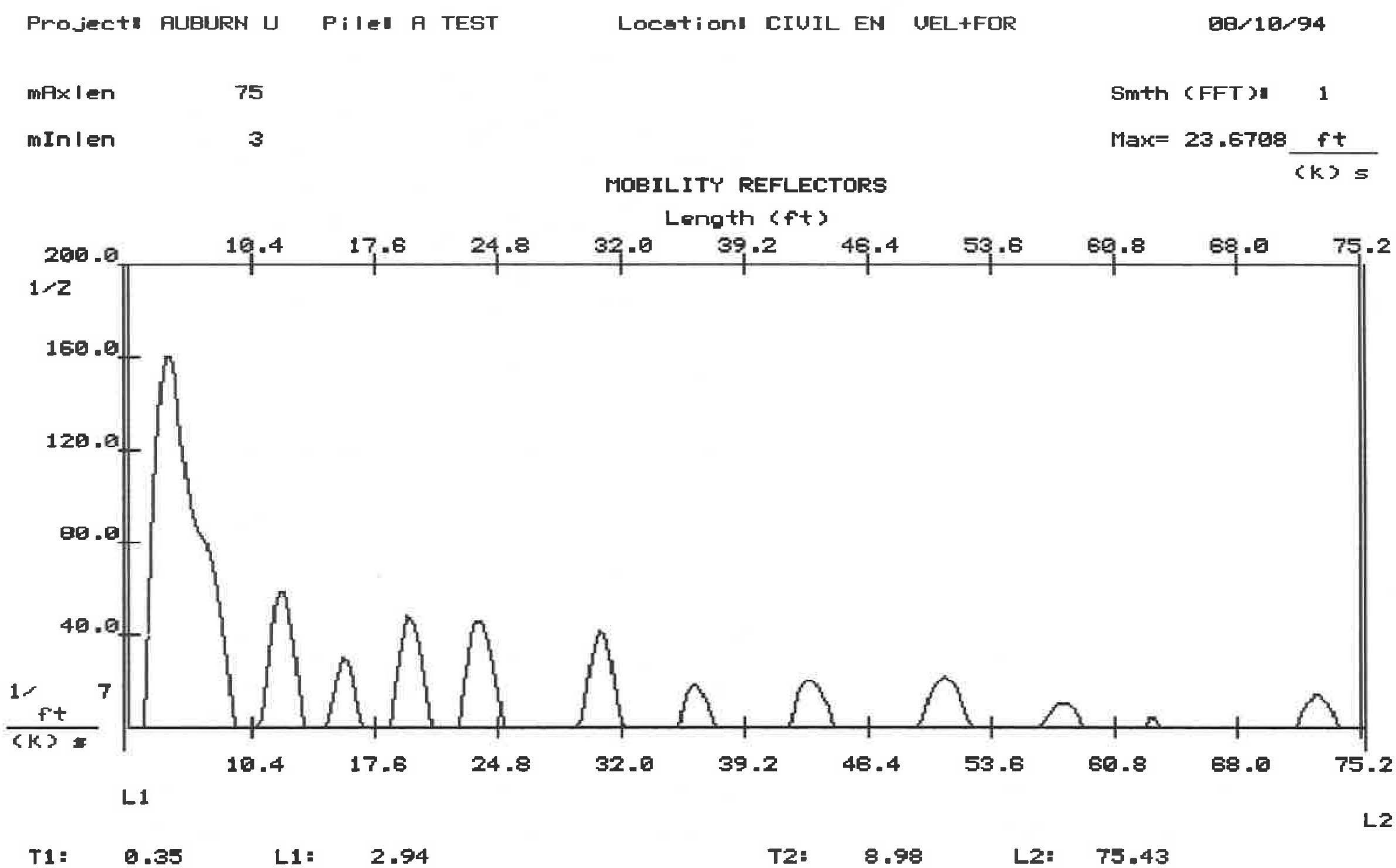


Figure 3.1.10: Mobility Reflectors (Atest data)

Alabama Highway Department

Project# AUBURN U　　Pile# D TEST　　Loc# CIVIL EN

Other: ACC ONLY　　Date: 08/10/94　　BN　　1 Blow

1

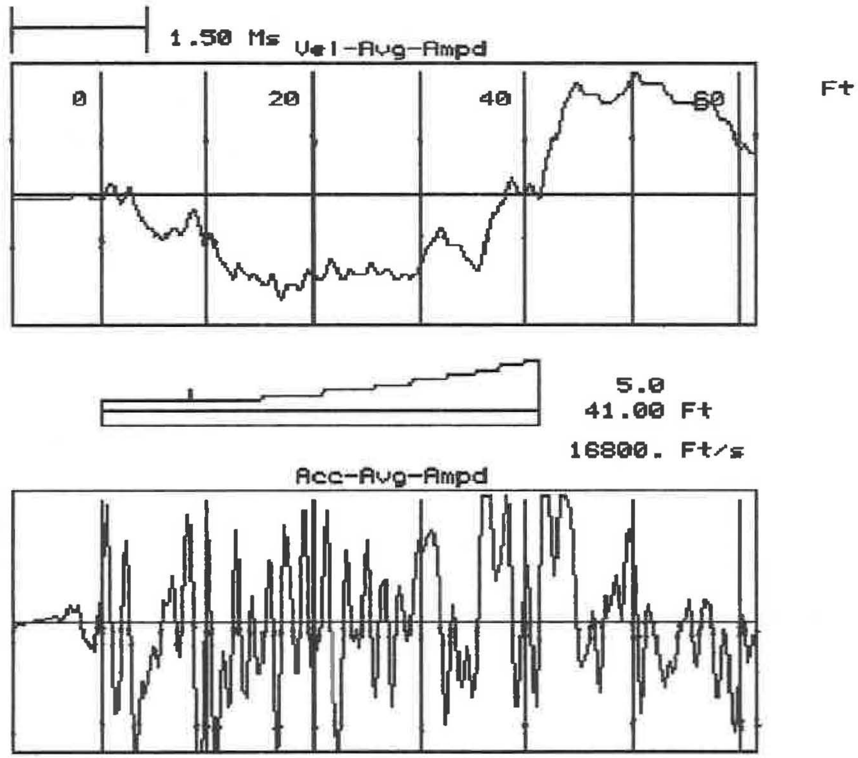


Figure 3.1.11: Amplified Velocity and Acceleration (Atest data)

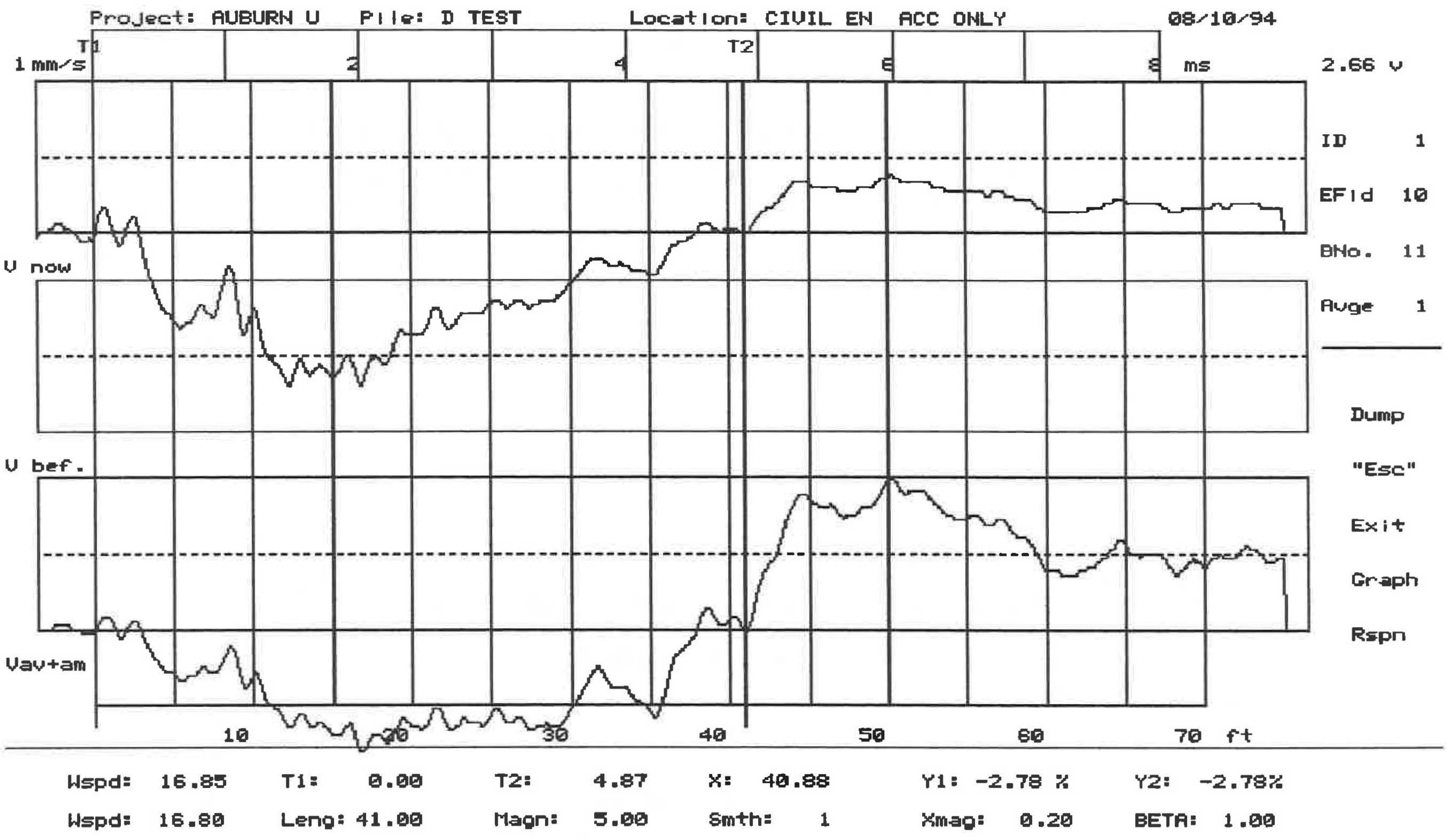


Figure 3.1.12: Amplified Velocity Time History (Dtest data)

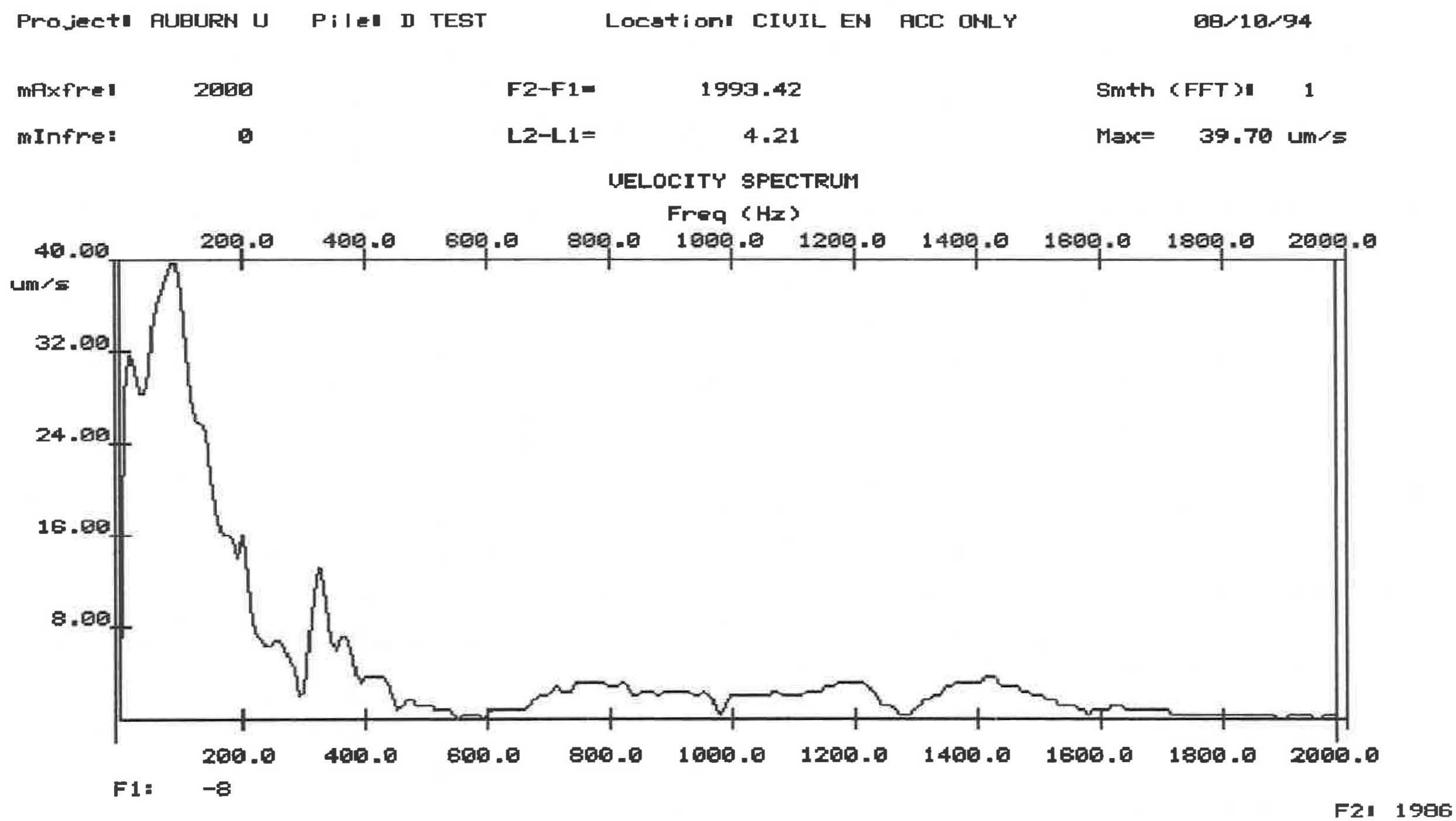


Figure 3.1.13: Velocity Spectrum (Dtest data)

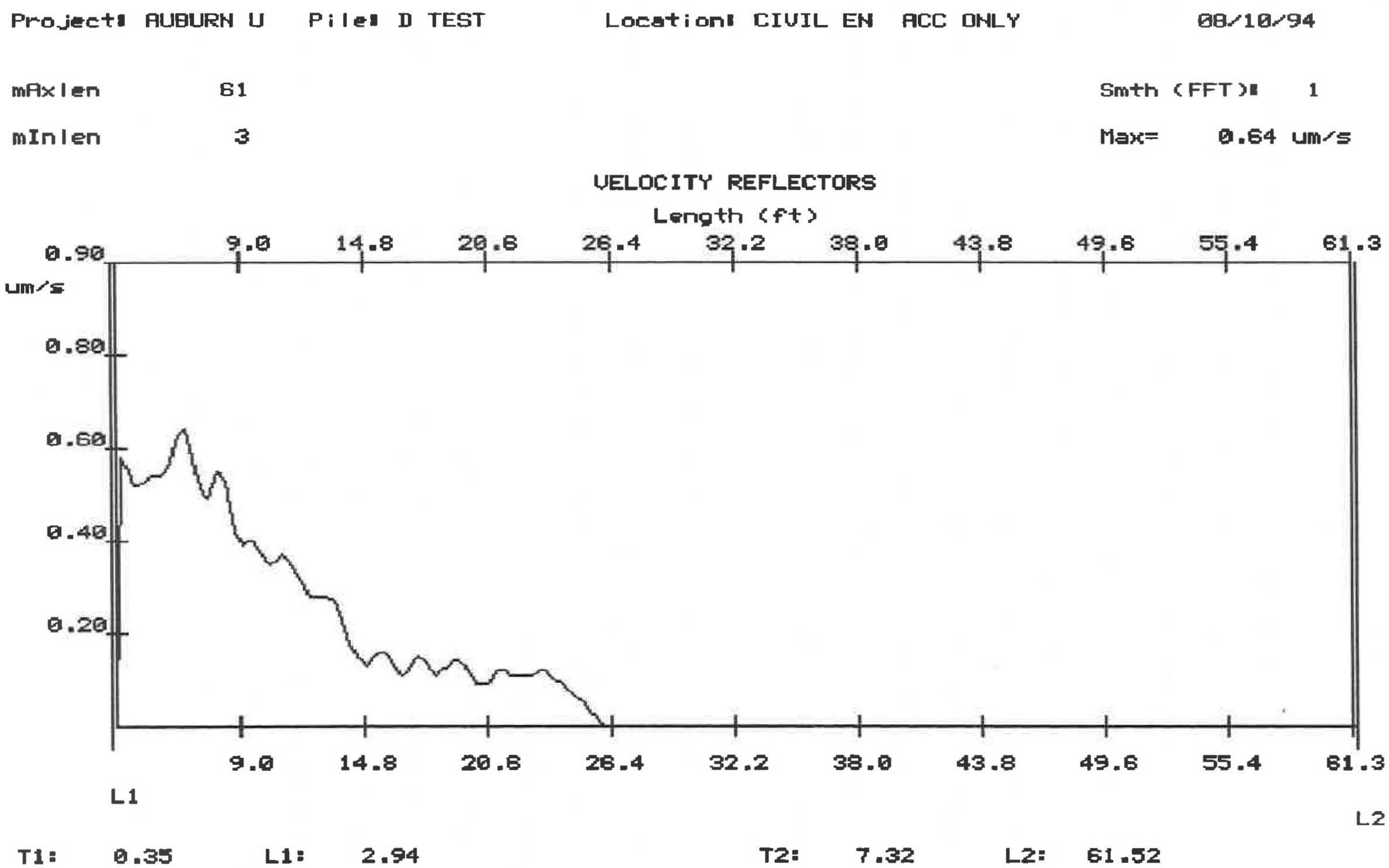


Figure 3.1.14: Velocity Reflectors (Dtest data)

3.2 Microseismic Methods (MS)

The microseismic method (Olson and Jalinoos, 1994) has been specifically developed for the determination of unknown depths of bridge foundations, and is an extension of the Sonic Echo/Impulse Response method described earlier. The principles of geophysical surveying and data processing are utilized to obtain an enhanced image of the subsurface foundation condition. Using the bridge substructure element as the medium for transmission of seismic energy, seismogram records i.e. acceleration response is obtained at various points on the accessible bridge substructure. An impulse hammer is used to input a force pulse at the top (or some other convenient location) of the substructure. The acceleration response is then monitored at closely spaced points upon the surface of the substructure (pile). The recorded acceleration data are converted to velocity, digitally processed, stacked or arranged in sequential order, and displayed graphically. This graphic plot can then be studied to infer the relationships between the recorded signal patterns. Values of time lag which decrease or increase systematically within the plot may be used to infer the depth to subsurface discontinuities for the foundation element.

Microseismic surveys may broadly be classified into the following two categories:

- Transillumination Profiling (TP) : this method is particularly suitable for near columnar shaped bridge substructure elements, and will be studied in more detail as part of the current project.
- Reflection Profiling (RP) : this method may potentially be used in the testing of more massive abutment and wall substructure type elements with surface access. Its applications lie beyond the scope of the current study.

The primary advantage of the Microseismic method is that it is truly noninvasive in nature, just like the Sonic Echo/Impulse Response techniques described in the previous section. But unlike the SE/IR methods, the Microseismic procedure can clearly identify the source of the reflections, be it from the superstructure or from a substructure element located beneath the ground surface. It is for this reason that a greater degree of emphasis was placed on the Microseismic technique, which procedure was used to determine embedded pile lengths at all 4 bridge sites tested.

3.2.1 Digital Signal Processing

Of particular significance in the processing of Microseismic and Parallel Seismic data is the use of digital signal processing. As has been stated previously in this report, the Microseismic profiling techniques are based upon the concepts of geophysical signal processing, which basically involve the processing, display and interpretation of large quantities of data gathered during geophysical seismic surveys. The original signals in these surveys are collected using analog devices (geophones). Some preconditioning of the signals (like filtering and amplification) is then carried out using analog systems known as signal conditioners (hardware). The signals are then converted to a digital format using the sampling capabilities of an analog to digital convertor. This sampled data may be stored in a digital format on any permanent storage device like computer hard disks and backup tapes. Such information may readily be retrieved at any desired time for purpose of analysis, which is usually carried out using specialized software. Such a program, "seis_pro", dedicated to the processing of Microseismic and Parallel Seismic data has been developed by the research team at Auburn University as part of this project. "Seis_pro" is based upon the principles of geophysics and digital signal processing, and has the capacity to perform the following functions. A brief write-up on "seis_pro" along with a copy of the original code is provided in Appendix A.

a. **Smoothing or filtering of recorded signal** : A comprehensive discussion on the use of digital filters in seismic data processing may be obtained from Holloway (1958). Seismic signals are often found to be rich in energy which encompasses a broad band of frequencies, amplitudes and phases. No matter how complicated, such a signal can always be expressed as a sum or superposition of a series of simple sinusoidal wave forms, and this can be done using the Fourier theorem. The purpose of **time smoothing** of the recorded signal is to reduce the amplitudes of the high frequency waves without causing any significant change in the low frequency components. The high frequency oscillations may be thought of as "noise" as they tend to mask or obscure the behavior of the lower frequency components which are usually of primary interest in seismic signals. Smoothing is in a sense analogous to "low pass filtering". Depending upon the smoothing scheme adopted, all energy above a certain frequency are seen to be attenuated for all practical purposes.

Smoothing is usually implemented by means of some kind of averaging scheme, where the recorded value of the signal at any particular index is altered to be equal to a weighted average of a certain number of data points in its vicinity on both sides. In particular, the smoothing scheme implemented in "seis_pro" involves one or more passes of a "Hanning Window", which is mathematically described as:

$$X_n = 0.25X_{n-1} + 0.5X_n + 0.25X_{n+1}$$

where X_n represents the recorded value of the signal at index *'n'*.

b. Integration of acceleration to velocity : It is often most convenient to monitor the response at the pile surface using accelerometers, which tend to be small in size or mass, and thus do not significantly affect the response being studied. Velocity transducers tend to be bulkier and are more difficult to "attach" to the pile surface. These are the reasons as to why Microseismic data is usually recorded as an acceleration signal. However, interpretation of the Sonic Echo and Microseismic data is best done using a velocity record, and thus the recorded acceleration signal has to be integrated to a velocity signal for analysis purposes. The integration may be carried out numerically using the simple formula presented next:

$$v_1 = v_0 + 0.5\,h\,(a_0 + a_1)$$

where, v_1 and v_0 denote values of velocity at times t and $(t+h)$ respectively, a_1 and a_0 denote the recorded accelerations, and h is the sampling interval, usually in microseconds. It is assumed that the acceleration varies linearly with time within the time interval h, and this is reasonable because of the extremely small values of sampling period that are adopted for these tests, usually in the range of 5 to 20 microseconds.

The problem with this procedure arises from the fact that the velocity time histories obtained by such integration drift with increasing time, and result in an unreasonably high end time velocity. This defies logic, as one might expect the end time velocity to be very small, as most of the energy of vibration has been dissipated in this time. Such a discrepancy is commonly caused by small errors in the acceleration data, which are introduced either at the time of recording, or during subsequent manipulations. The base line or the "zero acceleration", if shifted only very slightly from the true zero value can lead to such gross errors in the integrated velocity. It is generally impossible to determine the true base line. However, several mathematical techniques exist which may be used to successfully correct or approximate the base line. One such technique is the **"Least Mean Square Velocity Technique"** that has been described by Berg and Housner (1961). The technique adds a time-dependent second order polynomial $(C_0 + C_1t + C_2t^2)$ to the acceleration, which in effect compensates for the errors in the recorded base line. Such a technique has been implemented in the "seis_pro" software for integration of acceleration records to the velocity format.

c. **Exponential amplification of the velocity record :** It has been observed that as a wavefront propagates up or down a pile embedded in the ground, it attenuates as a function of time. Thus a reflection from the pile bottom might be significantly reduced in amplitude by the time it reaches the monitoring station located above the ground level. This problem is particularly acute in the case of long and slender steel-H piles, in which the wave energy is dissipated at a higher rate than in more massive foundation types like concrete piles and drilled shafts. In such cases, it is often helpful to amplify the recorded signal as a function of time, and an exponential amplification function is seen to best compensate for the exponentially decaying wave energy propagating down the pile. A typical amplification function would be represented by

$$v = v\, e^{[\ln X \frac{t}{T}]}$$

where, v = amplitude of signal at time t

T = total time window of the signal

X = value of amplification factor at time T

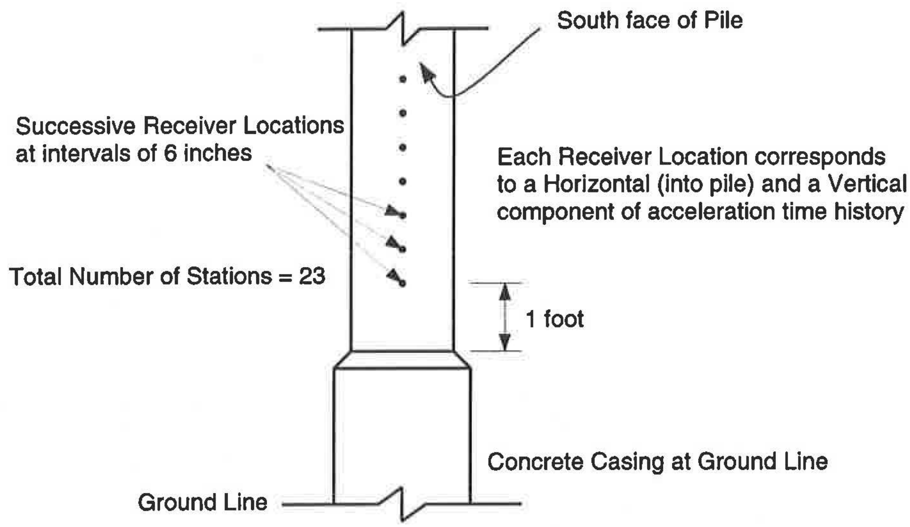
Such an exponential amplification scheme has been used in the "seis_pro" program, where the maximum value of the amplification factor, X is selected by the user and may be changed depending upon need.

3.2.2 Microseismic Test Results

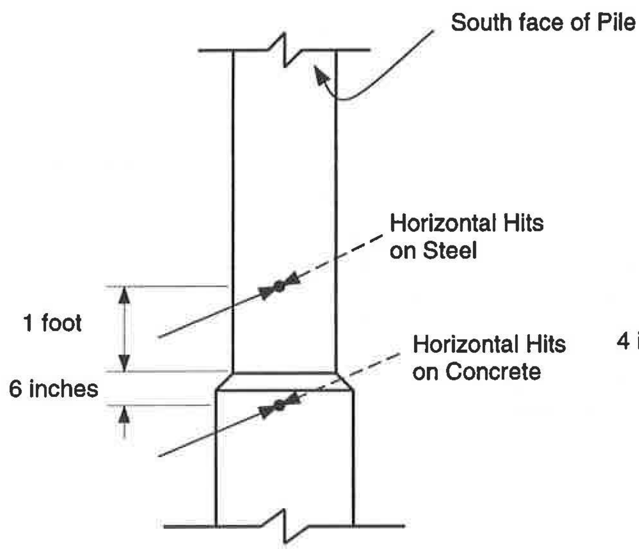
Alabama River Bridge, US Highway 31

The microseismic method was used to determine the embedded length of pile #3, the middle pile. The pile was impacted both horizontally and vertically at different locations (on the steel pile as well as the concrete casing at the ground level) using both a 3 lb and a 12 lb hammer. Acceleration response was monitored at the pile surface at intervals of 0.5 feet for a total of 23 stations or records, starting from an elevation of 3 feet above the ground level and moving the receiver up the face of the pile for successive records. Vertical and horizontal components of acceleration were obtained at each station, which were integrated to velocity and scaled and stacked in sequential order for purposes of analysis. This was done using the program "seis_pro" which has been specifically developed for such a purpose. The test setup for pile #3 showing multiple source and receiver locations is illustrated in Figure 3.2.1. A total of 24 sets of stacked velocity records were produced by various combinations of input signals and receiver alignments. The pile was impacted at 6 different locations, four of which were aligned horizontally and 2 vertically, and included impacts upon the concrete casing at the ground level as well as the steel pile itself. The nature of the stress wave generated in the pile was also varied by using different hammers. The lighter hammer (3 lb) characteristically generates a stress wave which is richer in higher frequency energy as compared to the heavier 12 lb hammer.

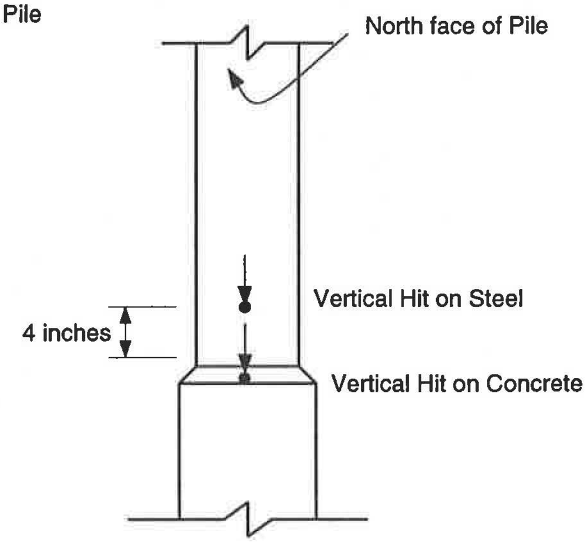
Figure 3.2.2 shows the set of velocity records produced by horizontally aligned hammer impacts upon the concrete casing at a location approximately 6 inches below the top of the casing. These were produced using the 12 lb hammer and the signals record the horizontal velocity response at the face of the pile. Figure 3.2.3 shows a similar set of records which were produced in exactly the same manner but using the 3 lb hammer. It should be noted that each set of records in Figures 3.2.2 and 3.2.3 mentioned above has been amplified exponentially with time, with an amplification factor of 3.0 at a time of 40 milliseconds. The velocity of propagation of the flexural wave in the steel pile was



Receiver Locations for Microseismic Test on Pile #3



Source : Horizontal Hits



Source : Vertical Hits

Figure 3.2.1 : Microseismic Test Setup for Pile #3

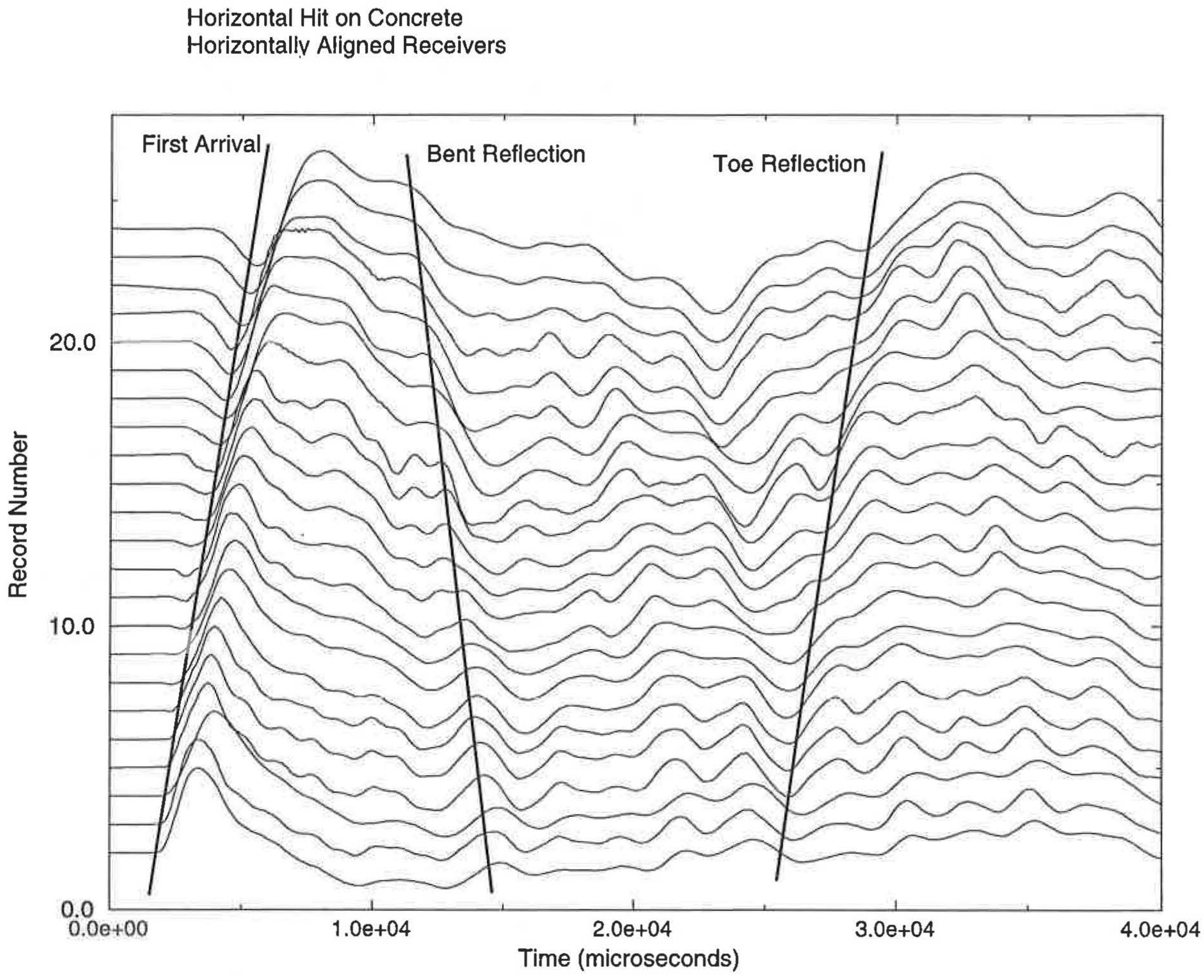


Figure 3.2.2 : Microseismic Records, Pile #3, 12 lb Hammer (BB3)

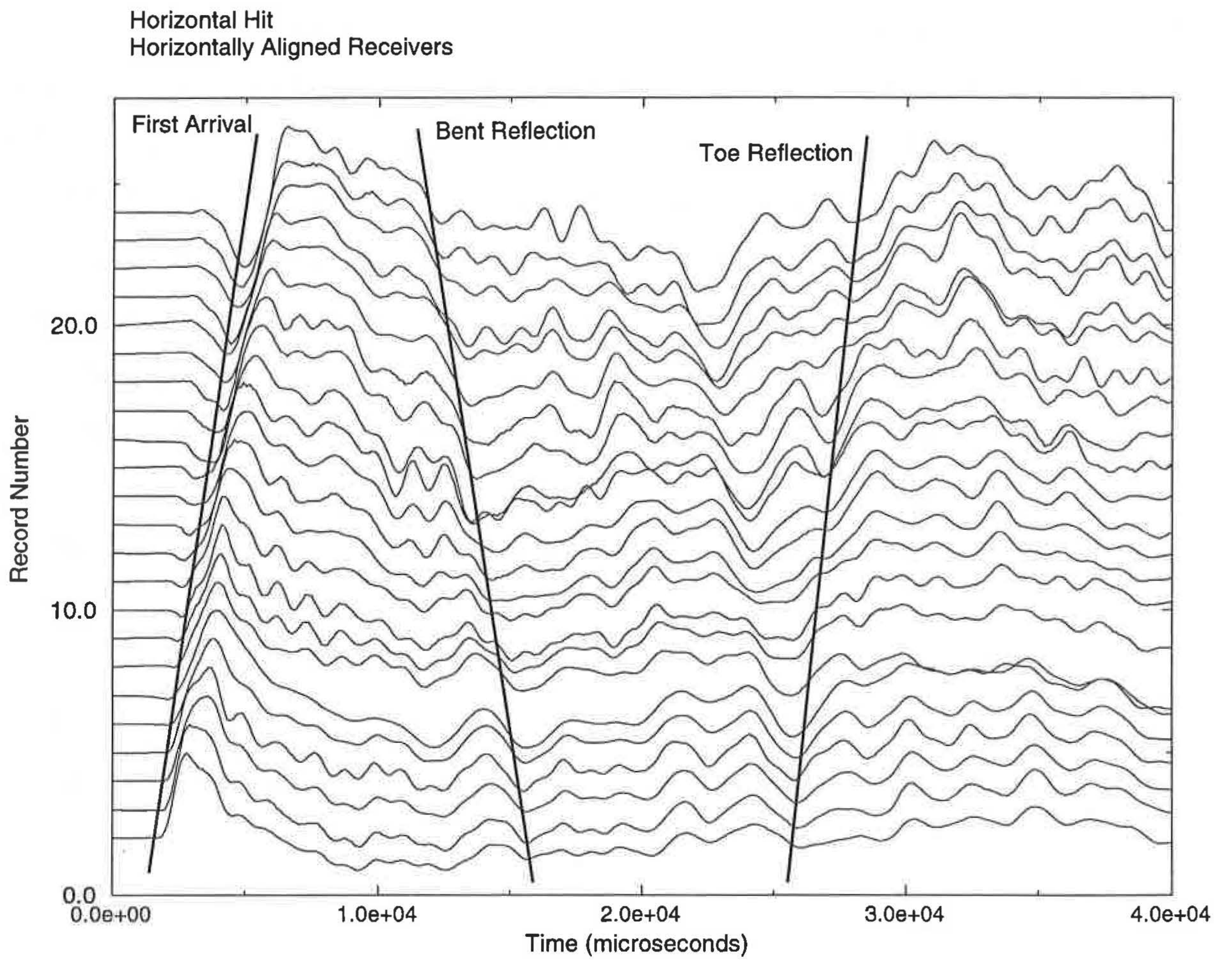


Figure 3.2.3 : Microseismic Test Records, Pile #3, 3 lb Hammer (HH3)

determined to be roughly 2900 feet/sec from a study of the linear sequence of first arrivals within each record. In a similar fashion, the compressive wave velocity was found to be about 16800 feet/sec from a study of the vertical component of velocity response. The set of reflections from the point of fixity of the pile head to the bridge bent is evident within both Figures 3.2.2 and 3.2.3. The slopes of the lines corresponding to these sets of reflected pulses indicates that the distance to this particular reflector decreases with each receiver location. Multiples of this reflected pulse (from the bent) can also be seen within the records. Another set of reflections from what is believed to be the pile toe can also be distinguished within the stacked data of Figures 3.2.2 and 3.2.3. These reflections are delayed in a linear fashion for each successive trace, which indicates that the reflector exists somewhere beneath the ground surface, and which corresponds to a value of roughly 36 feet for embedded pile length. This is somewhat less than the actual recorded value of 39 feet. No inference could be made regarding pile length from a study of the velocity records which were aligned vertically and hence picked up compressive stress pulses within the pile. This could possibly arise from the fact that compressive stress pulses propagate at a much faster rate within the pile material as compared to bending waves, and thus provide much less resolution within the time histories from which individual reflection events may be determined. Another factor that might account for the difference in the nature of the response patterns is the fact that a certain reflector might affect a bending wave pulse in a manner different from its effect on a compressive wave pulse.

Chubahatchee Creek Bridge, US Highway 80

The foundation tested at this site was very similar in nature to the Alabama River site. Piles 2 and 3 (steel-H) on Bent #3 on the west side of the Chubahatchee Creek were selected for testing. Although the Microseismic procedure was used at both sites, several significant refinements had been made to the test procedure, and these have been noted below:

a. The accelerometers were mounted onto the surface of the steel pile using magnetic mounting bases, whereas at the Alabama River site the accelerometers were mounted on the pile using a thin layer of grease. It is believed that the use of the magnetic mounting bases produces potentially better results, as the receivers are coupled to the pile much more securely. In fact, the Shorter piles had been tested initially without the magnetic bases, and the nature of the stacked data and the interpretation of significant reflection patterns were greatly enhanced the second time around (with magnetic bases).

b. The signals recorded with the Alabama River piles were obtained with one pickup/accelerometer station for each hit of the impulse hammer. The technique was modified somewhat with the Chubahatchee Creek piles, where each hit was recorded simultaneously by 2 or 3 accelerometers placed along a line and kept a certain distance apart. This way, for any particular hit, all 2 or 3 of the receivers picked up exactly the same waveform as it moved up or down the length of the pile. This improved the coherence or similarity between successive signal traces within the stacked data, which made it easier to identify significant reflection patterns. In an ideal case, the response at all the desired points on the surface of the pile could be monitored simultaneously with a single hit, but this would require the use of 12 to 24 accelerometers and a signal conditioner that could handle the same number of data channels at once.

As was seen from the analysis of the test data at the Alabama River site, compressive or longitudinal waves were not very effective in the determination of embedded pile length. Thus only bending or flexural waves were used with the piles at the Chubahatchee Creek bridge. Figure 3.2.4 illustrates the test setup for piles 2 and 3. Both piles were encased with a concrete collar at the level of the ground surface. The piles were impacted horizontally with an instrumented 12 lb sledge hammer 1 foot above the level of the casing, both on the flange and on the web. With impacts on the flange, the receiver stations were placed 6 inches apart, with the first station being located 2 feet above the level of the casing and along the centerline of the flange. With hits on the web, the recording stations were similarly located along the centerline of the web. Horizontal

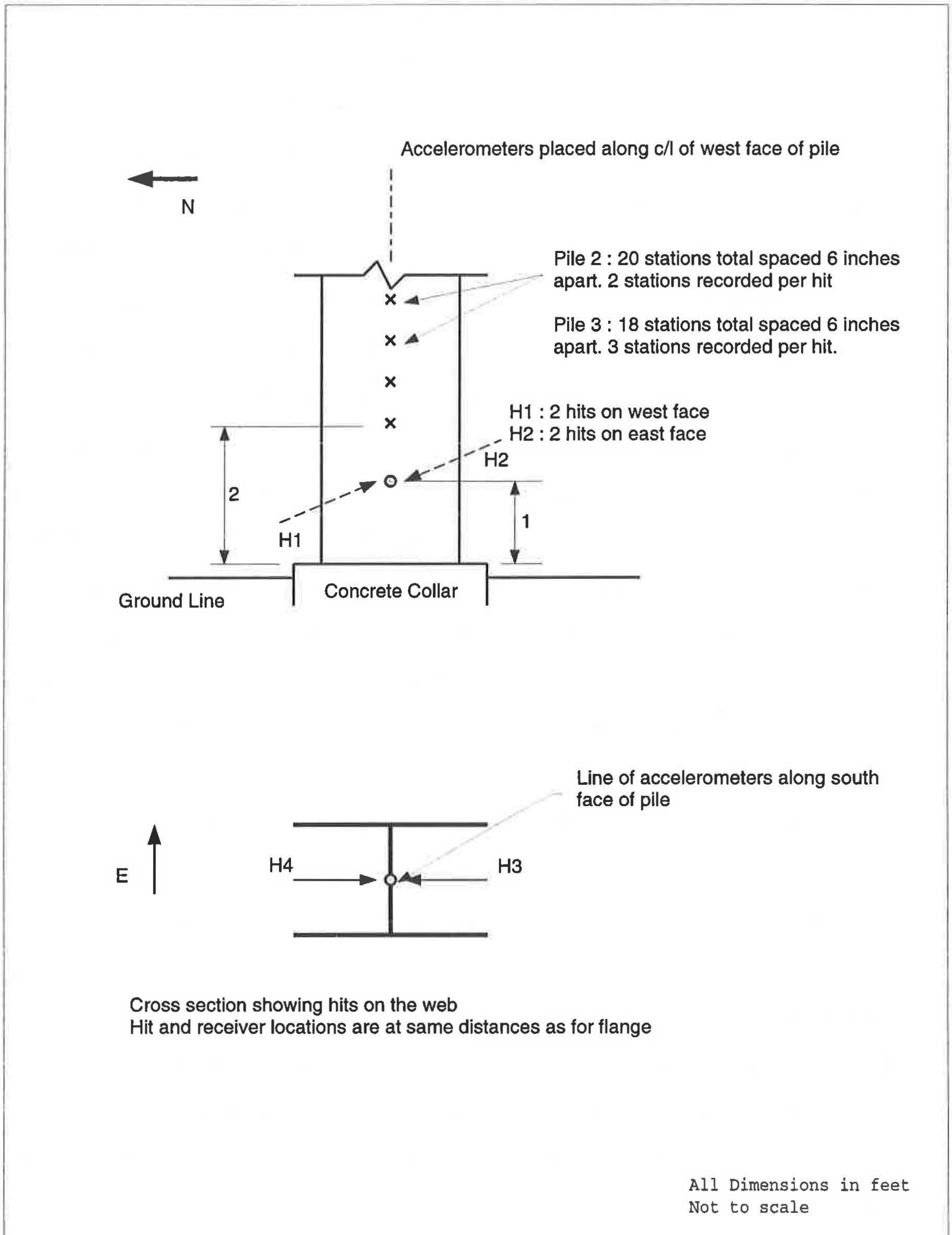


Figure 3.2.4 : Test Setup, Piles 2 and 3, Bent 3, Chubahatchee Creek Bridge

hits were made along both directions so as to produce signals with both direct and reversed polarities. Also, 2 hits were recorded at each location to enable the averaging of data. A total of 20 stations were recorded at pile #2, with 2 accelerometers recording a single hit (6 inch spacing between 2 successive receiver locations). A total of 18 stations were recorded with pile #3, with 3 receivers recording a single hit (again 6 inches apart).

Figure 3.2.5 shows a set of stacked velocity records, which were produced by horizontal hits on the western face of the web of pile #3. Figure 3.2.6 shows the stacked data records produced by hits on the eastern face of the same pile with the same receiver locations, but the waveforms are seen to have their polarities reversed. From a study of the slope of the first arrival patterns within each set of data, the bending wave velocity was estimated to lie between 2800 and 2900 feet/sec. It was hard to get an exact value as the computed velocity depended strongly upon the selection of a discrete point which defined the arrival of a specific waveform. Our calculations were based upon the time difference at the middle of the peaks (or the troughs) in the first arrivals at the first and the last receiver stations, which were located a known distance apart. Reflections from what is believed to be the bottom of the pile were also identified within the stacked data sets. Figure 3.2.7 shows the 2 sets of stacked data with reversed polarities which have been superimposed on the same plot, and the reflection from the pile toe becomes much more obvious when this is done. The embedded lengths of both piles 2 and 3 were computed to lie between 30 and 32 feet from the top of the concrete collar. The original pile driving records were then checked; the length of pile #2 from the top of the collar was stated to be 26.5 feet, and for pile #3 was about 28 feet. Our tests thus overestimated pile lengths by 10 to 15 % at this test site. It is believed that this discrepancy might have arisen due to the following reasons:

1. an inaccurate velocity measurement
2. the difficulty in selecting the precise arrival time for the toe reflection

Both of these are due to a lack of sufficient resolution or lack of sharp well defined wave patterns, which are caused by the dispersive nature of the bending wave. The dispersive property causes the different frequency components of the wave to move with slightly

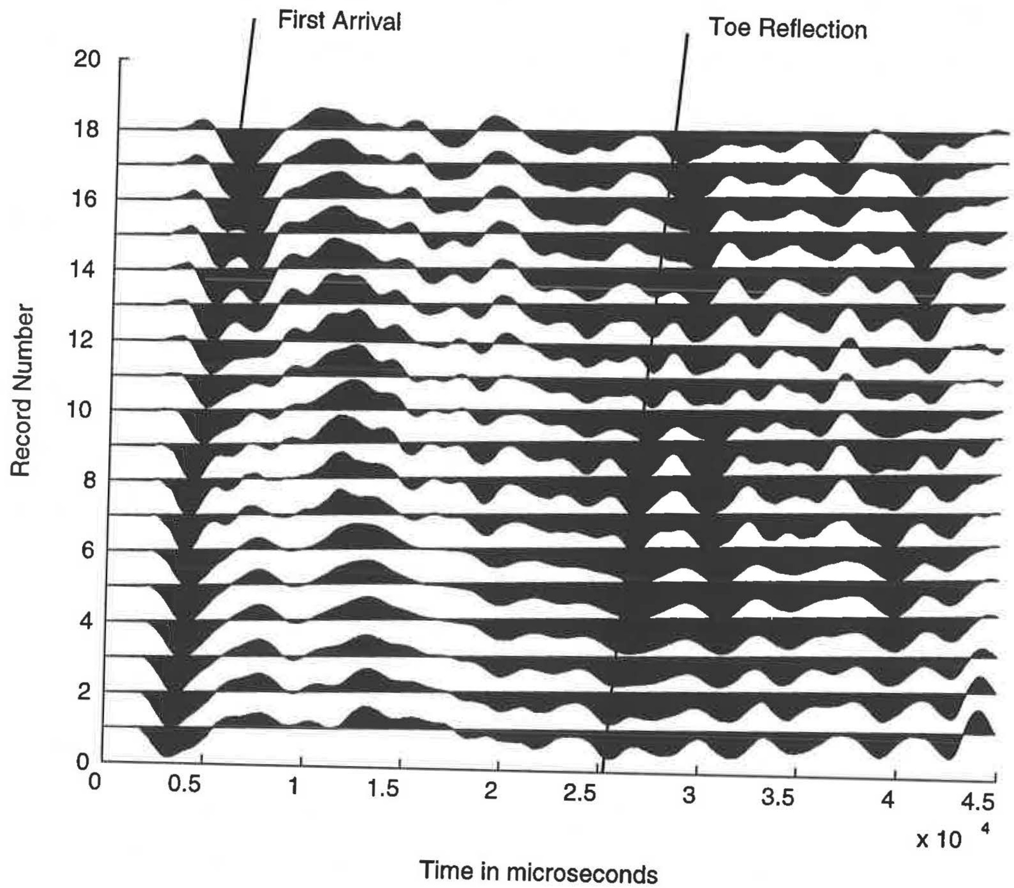


Figure 3.2.5 : Velocity Records, Pile 3, Chubahatchee Creek Bridge (bb1)

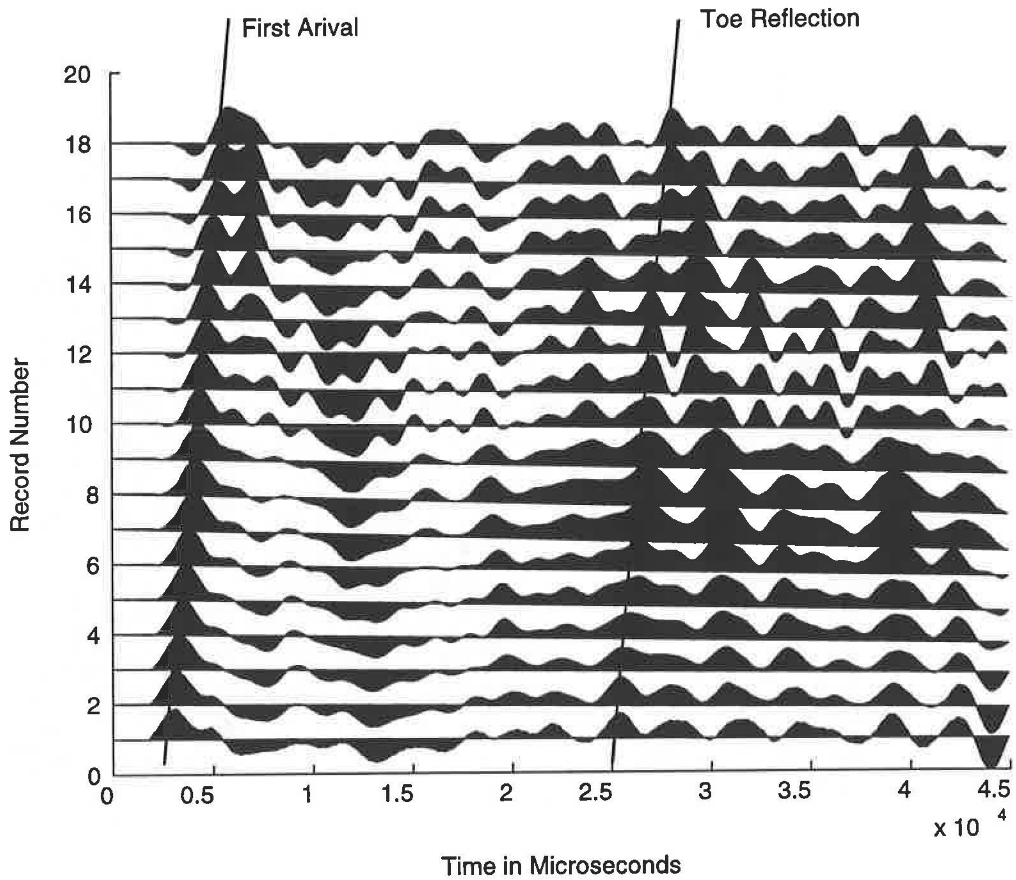


Figure 3.2.6 : Velocity Records, Pile 3, Chubahatchee Creek Bridge (cc1)

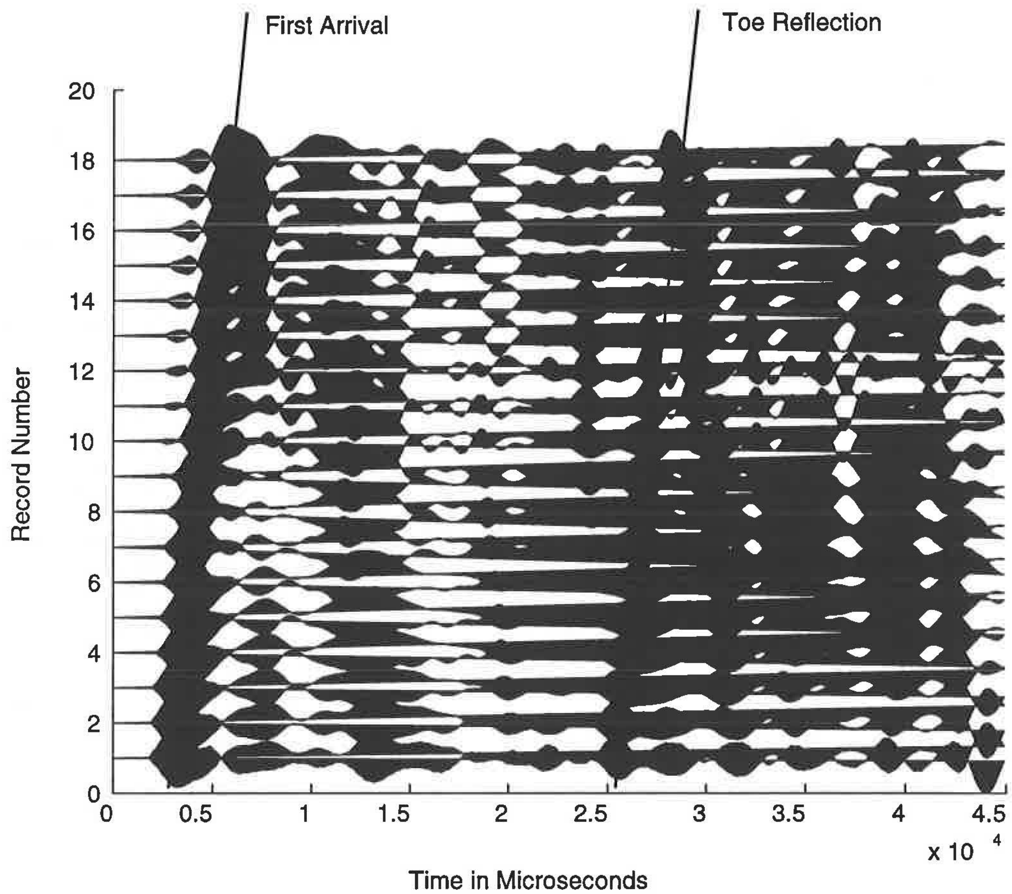


Figure 3.2.7 : Velocity Records Superimposed, Pile 3, Chubahatchee Creek Bridge

different velocities, which in effect tends to smudge out the initial input wave pulse which has very well defined start and end points on the time scale.

The toe reflection pattern was also better defined with pile #3 than with pile #2. This is probably due to the fact that 3 receiver stations per hit were used with the former as opposed to 2 per hit with the latter, which produced signals with enhanced coherence properties.

Bridge on Conecuh County Road 42

This was a small bridge that had been used to span a stream along Conecuh County Road 42 in Evergreen, Alabama. The 2 bents were supported on timber piles, and piles 1 and 2 as marked in Figure 2.5 were chosen for testing. Both piles were timber, approximately 12 inches in diameter, and each extended approximately 8 feet from the mud line to the bent. The test setup has been illustrated in Figure 3.2.8.

The piles were impacted horizontally using an instrumented 12 lb sledge at points located 2 feet above the mud line. The receiver stations were placed 6 inches apart and included a total of 12 stations at pile 1 and 13 stations at pile 2. 3 components of acceleration were recorded at each receiver location:

1. Radial or into the pile
2. Tangential to the pile surface
3. Vertical

The acceleration signals were processed digitally, integrated to velocity and stacked in sequential order using the program "seis_pro". The wave velocity in the pile (obtained from an analysis of the first arrivals) was found to range between 2800 and 3200 ft/sec.

Each of the separate sequences of stacked signals were analyzed in an attempt to determine the toe reflection, and the reflected signal from the toe was not obvious from any any of the individual records. However, when all the separate stacks were looked at together, a consistent pattern was noted within each. This wave corresponded to a reflection at a depth of roughly 8 to 10 feet beneath the ground, which was believed to be the bottom of the pile. Figure 3.2.9 shows a set of stacked microseismic velocity traces

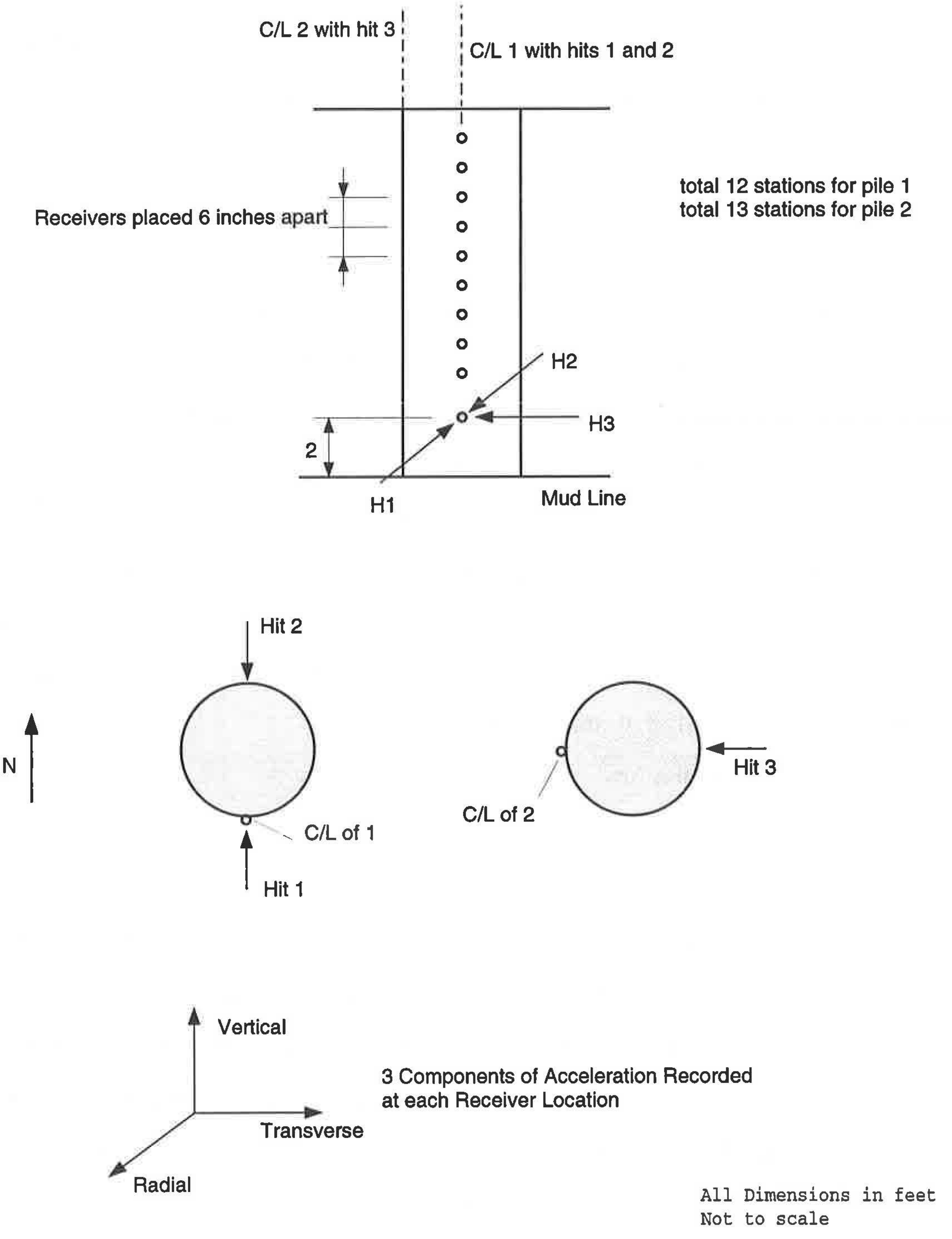


Figure 3.2.8 : Test Setup, Piles 1 and 2, Conecuh Cty Rd 42

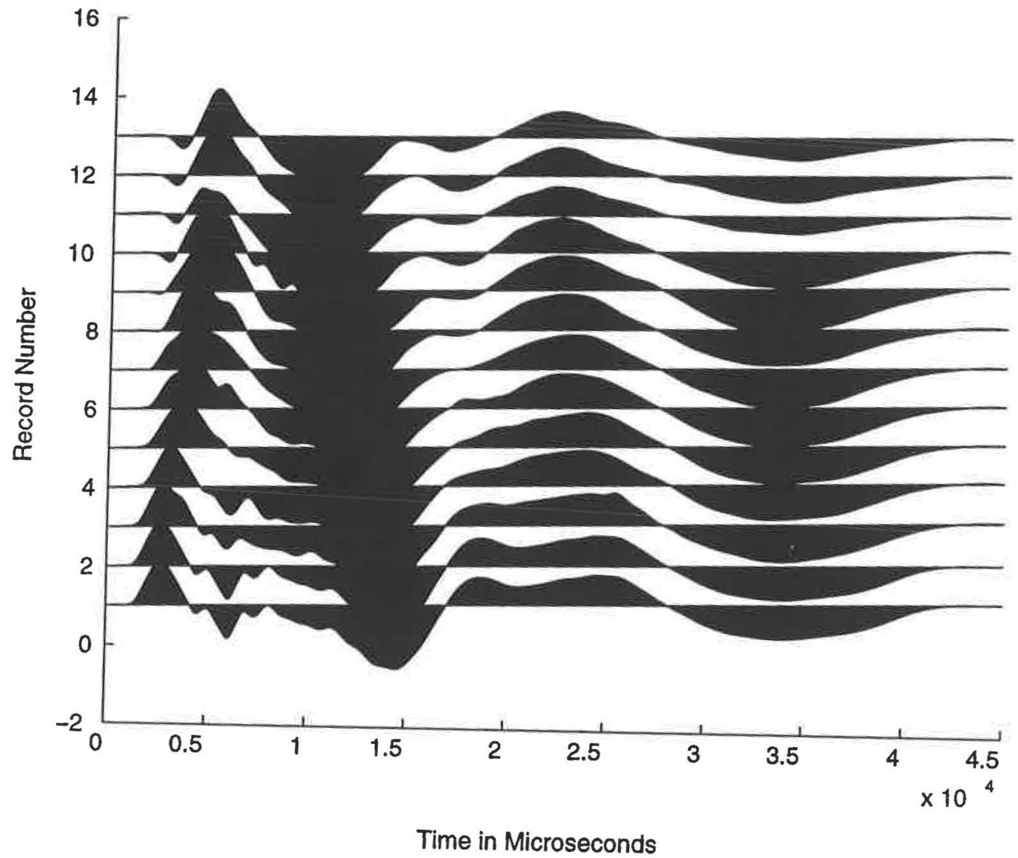


Figure 3.2.9 : Microseismic Records, Pile 2, County Road 42

recorded on pile 2. No driving records were available for this particular bridge; however, it was torn down and replaced with a new bridge shortly after the tests were conducted. The original wooden piles were pulled out of the ground, and their embedded lengths were measured to lie between 12 and 14 feet. Thus the predictions were seen to be a little conservative. Without the direct measurements however, the amount of confidence that could be placed on such predictions was not very high.

Bridge on Conecuh County Road 43

The bridge on County Road 43 is founded on concrete encased steel piles. Pile 1 and 2 (as illustrated in Figure 2.6) were tested using the microseismic procedure. The test setup was exactly similar to that used for testing the timber piles on Conecuh County Road 42. The piles were impacted horizontally 2 feet above the mudline with an instrumented 12 lb sledge. A total of 10 stations were recorded on pile 1, and 13 on pile 2. The acceleration response was monitored at each receiver location using 3 accelerometers mounted on a triaxial mounting adaptor (vertical, radial and tangential to the pile). Figure 3.2.10 illustrates the test setup. The recorded signals were processed digitally and integrated to velocity using the program "seis_pro". The wave velocity in the pile was determined to be approximately 6400 feet/sec from an analysis of the first arrivals at each receiver location. From a study of the stacked data, 2 sets of reflections were identified to be originating from beneath the ground surface. The first reflection corresponded to a depth of approximately 7 to 8 feet beneath the ground, and it is believed that this corresponds to the bottom of the concrete casing. The second reflection was produced at a depth of about 23 to 25 feet beneath the ground, and this probably corresponds to the bottom of the steel pile. Figure 3.2.11 shows a contour plot of the time vs. velocity stacked data. The x-axis represents the record number scale or distance along the pile. The y-axis represents the time scale (700 data points per record). The contour plot is useful in cases where it is hard to tell a reflection pattern from just the stacked velocity record; in a sense, the contour amplifies dominant trends in the data and makes the identification of these trends easier. As to the accuracy of the predictions, it should be noted that the original construction records for this particular bridge foundation

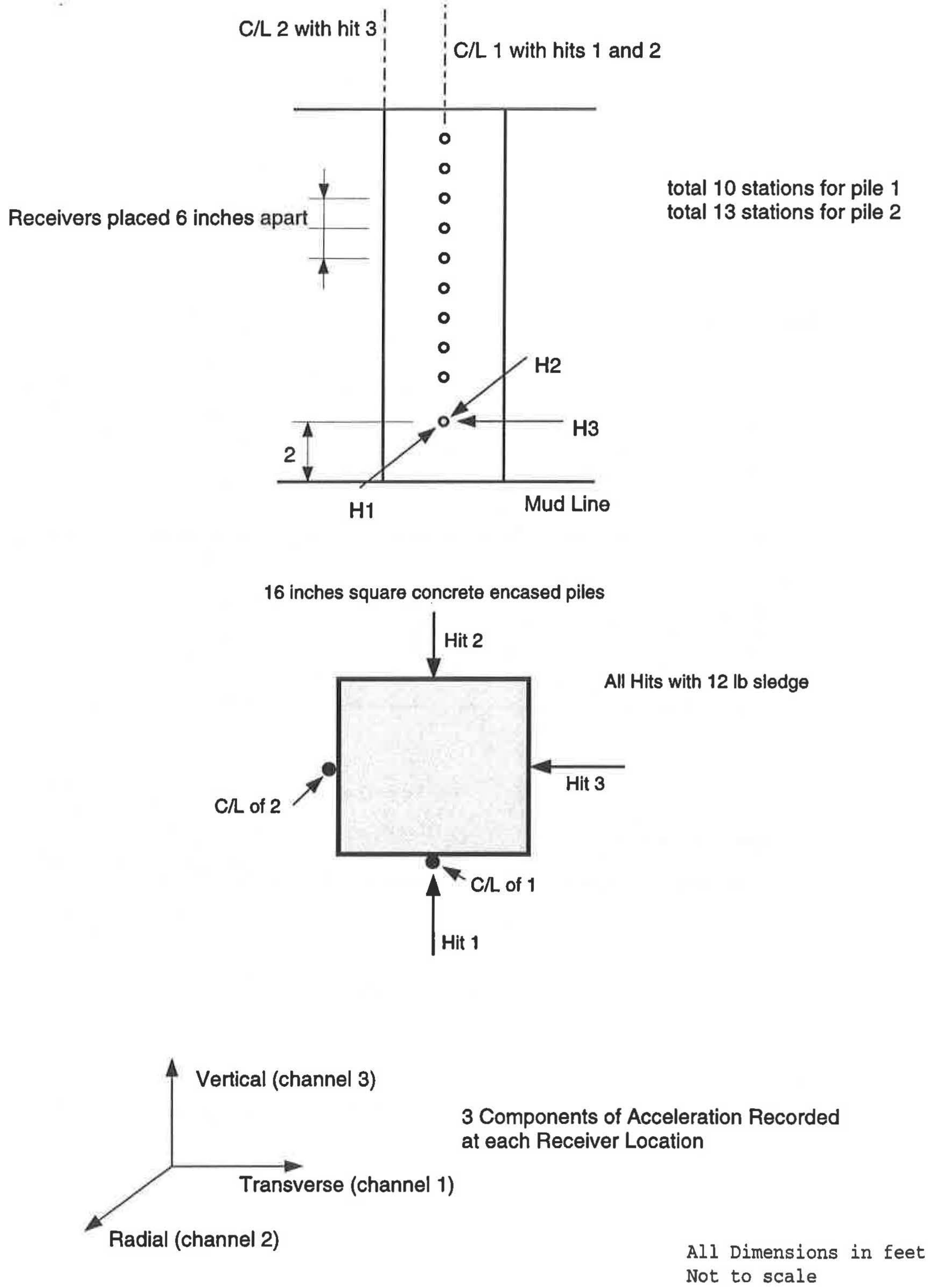


Figure 3.2.10 : Test Setup, Piles 1 and 2, Conecuh Cty Rd 43

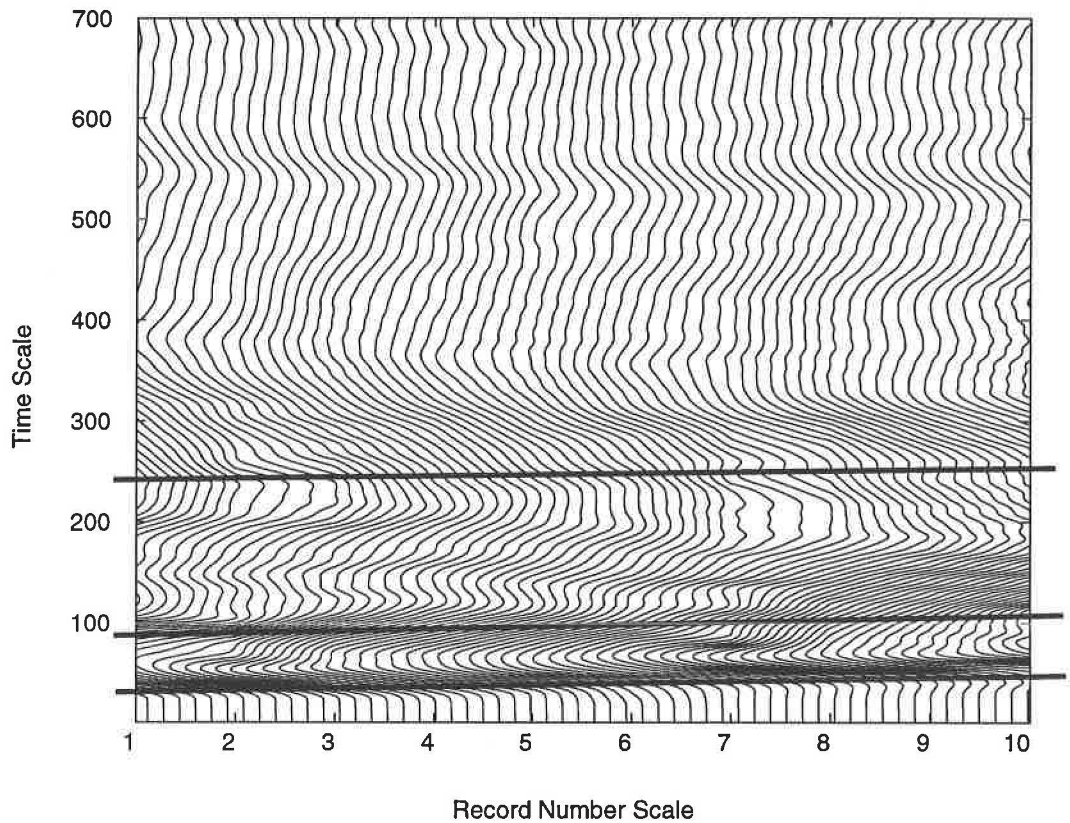


Figure 3.2.11 : Microseismic Records converted to a Pseudocontour Plot, Pile 1, CR43

are not available, and thus there is no easy way of telling how successful the efforts of the research team were in this regard.

3.2.3 Conclusions and Recommendations (Microseismic Method)

From a study of the limited quantity of data that is currently available, the following conclusions have been reached:

- Bending waves are better suited to the determination of unknown embedded lengths of steel piles than compressive waves. Compressive waves were unable to provide even an estimate for embedded length because of very little consistency or repeatability of reflection patterns within the sets of responses recorded at differing elevations. The bending wave patterns on the other hand showed remarkable similarities in the recorded waveforms with reflection patterns advanced or delayed consistently within successive records. Also of significance was the fact that impacts on the steel pile itself produced waves which were somewhat richer in higher frequency components than those produced by impacts upon the concrete collar.
- Currently, the prediction of pile length is also limited by the lack of resolution in the recorded signals, which makes it difficult to pick a single discrete point on the wave to define the arrival. This is not a shortcoming of the hardware or the test procedure, but is caused by the dispersive nature of the flexural wave itself. Thus any real life predictions may be off by as much as 10 to 15 % even when the toe reflection is clearly identifiable.
- It was also noted that the lighter hammer produced sharper and more high frequency pulses than the heavier hammer, and thus generated signals with enhanced resolution characteristics when compared to the latter. This would imply that lighter hammers are more useful for purposes of imaging relatively shallow foundations, whereas heavier hammers would be suited to the determination of reflectors located at greater depths, since higher frequency signals also decay faster.

- The use of magnetic mounting bases on the accelerometers was also seen to significantly improve the quality of the recorded signals due to better coupling between the pile and the receiver.
- Traffic on the bridge was seen to create noise in the recorded signals, and thus blocking traffic while testing is in progress is seen as another way to improve signal quality.
- It was observed that better coherence or repeatability was achieved with the recording of multiple receiver locations with a single input blow. It is believed that if it were possible to record at all receiver locations with a single blow the task of identifying repeated reflection patterns within the stacked data would be simplified and more confidence might be placed in the resulting predictions.
- It was easier to identify the reflection patterns of interest if two sets of stacked data with reversed polarities were superimposed on a single plot. The recordings could be made at the same receiver locations but with hits recorded on opposite faces of the pile.

All in all, the microseismic method when used with stacked displays and superposition of reversed polarity bending waves, appears to be the most reliable non destructive testing method which does not involve drilling into the soil next to the pile. it is relatively simple to carry out the tests although processing and analysis of the recorded data may be more time consuming when compared to the Sonic Echo method.

3.3 Parallel Seismic (PS) Method

The parallel seismic method is based upon conventional crosshole and downhole survey techniques, which are commonly used by civil engineers to determine dynamic soil properties. The procedure consists of the application of a sharp and well defined force pulse somewhere upon the exposed surface of the pile structure present above the ground surface. The primary intention of the test is to measure and record the amplitude and form (frequency content) of the propagated wave energy through the use of receivers placed at regular intervals down the length of a borehole drilled adjacent to the pile. The borehole would have to be installed reasonably close to the pile for the best results. Typical pile-borehole distances might lie between 5 to 10 feet. As the longitudinal wave travels down the length of the pile, a part of the energy is dissipated out to the surrounding soil due to refraction at the pile-soil interface. This refraction is due the acoustic contrast that exists on opposite sides of the interface. The acoustic contrast is defined as the ratio Z_{pile}/Z_{soil}, where Z is the acoustic impedance of the material being considered, and is numerically equal to the product of the mass density and the compression wave velocity for the material. Also, the angle at which the refracted energy or the "head wave" propagates outward from the interface is dictated by Snell's Law, and can be determined from the ratio of the compression wave velocities within the pile and the soil. It is this refracted energy that is recorded by the receiver placed in the borehole.

From a study of the first arrival log generated by a traverse down the borehole, it is often possible to detect the presence of the pile tip. The first arrival log is simply a plot of depth (of receiver in the borehole) vs. arrival time of the first wave energy (the P-wave) at that depth. For soil conditions which remain uniform down the entire length of the embedded pile, the first arrival time data can be fitted to 2 straight lines. The initial slope corresponds to the velocity of propagation of the longitudinal wave within the pile material. The final slope corresponds to the propagation velocity of dilational or compressive waves (P-waves) within the soil, which is usually much smaller than the pile wave velocity. The point of intersection of the 2 lines with different slopes corresponds to the pile bottom, and this break appears at a depth which is slightly greater than the depth to the embedded pile tip. It should be noted that the P-wave is always the first to arrive at any particular receiver location. This is because the P-wave is always the fastest wave to propagate within 3 dimensional space constituted of any material. Also, as the point of measurement is moved to depths beyond the end of the pile, a sharp reduction in amplitude is noticed in the recorded signal. This is due to the fact that the pile itself is much stiffer than the surrounding soil, and acts as a sort of guide or path for the impact energy to follow, which effect is lost once the receiver is placed beyond the pile tip.

Another interesting phenomenon that is usually observed in the stacked data is the presence of a diffraction tail at a depth corresponding to the tip of the pile. The presence of diffraction tails is routine in geophysical reflection surveys, and can be explained by Huygens' Principle (Telford et al., 1980). A scattering of wave energy in 3 dimensional space occurs at any point on an interface separating media with differing acoustic impedances when there is a sharp or abrupt change in curvature at that point. Such an abrupt change occurs at the pile tip, which, as a consequence, acts as a secondary emitter of wave energy. If the data recorded by a geophone traverse is relatively noise-free, the diffraction tail resulting from the pile tip should be visible within the stacked records.

Several combinations of source/receiver locations may be tried to obtain more data for analysis. The bridge substructure may be impacted at several locations, both horizontally and vertically. A horizontal hit produces a wave which is predominantly bending/shear in nature, whereas a vertical hit produces longitudinal waves. Shear waves and longitudinal waves travel down the pile material with different velocities and thus produce different logs of first arrival times at the receiver. Two types of receivers are commonly used in practice for this type of survey:

- hydrophones - which measure ambient water pressure (scalar)
- geophones - which measure velocity along 3 mutually orthogonal directions (one component vertical, the other two horizontal).

Thus for a single hit and a 3 component geophone receiver, 3 independent channels of receiver signal are obtained, which can then be analyzed independently of each other to produce first arrival logs. Greater confidence in the data interpretation can be achieved if the various source/receiver combinations are seen to produce identical results for subsurface foundation profile.

3.3.1 Parallel Seismic Test Results

Parallel seismic testing was carried out on 3 different piles at the Bent #4 of the Alabama River bridge (refer to Figure 2.1 for structural and location details):

- Pile #2 (2nd from the westernmost)
- Pile #3 (middle pile)
- Pile #5 (easternmost pile, tapered)

Both hydrophones and geophones were used for test purposes, with all 3 components of velocity being recorded for the latter. The 3 velocity components were consistently maintained along the following directions :

- vertical
- radial (horizontal into pile)
- tangential (parallel to pile face)

Three different hits of the impulse hammer were used to produce signals at any particular depth. The hits were maintained along the following directions:

- a vertical hit at the top of the concrete casing
- a horizontal hit applied along the pile-borehole line, acting towards the receiver
- a horizontal hit applied along the pile-borehole line, acting away from the receiver

Thus, at the most 9 combinations of signals could be attained for any one traverse of the receiver down the borehole. Also, simultaneous traverses down 2 parallel boreholes could be achieved by using the hydrophones (1 component pressure transducer). This enabled the creation of data which supported a detailed parametric study.

Test Results for Pile #2

Figure 3.3.1 shows the details of the source and receiver locations. Borehole 'C', located at a distance of approximately 4 feet from the face of the pile, had been installed with a 2" diameter PVC case grouted in. This hole allowed only the placement of the slender hydrophones, and thus no geophones could be used with this particular pile. The bottom of the casing was located at a depth of 59 feet from the ground surface. The receiver signals were logged at intervals of 1 feet during the downhole traverse. Three different sources were used at each depth logged. These consisted of a single vertical hit and 2 horizontal hits, the natures of which have been described previously.

The arrival logs produced by the horizontal hits did not indicate any sharp break (refer to figure 3.3.2). Thus no inference pertaining to the embedded pile length could be made from the records produced by the 2 horizontal hits. The log of the first arrival times for the signals generated by the vertical hits shows a distinct break at a depth of about 37 feet below the ground line (Figure 3.3.3). This leads us to infer that the bottom of the pile is located at a depth of slightly less than 37 feet, somewhere within the range of 35 to 36 feet. The actual length of the pile, as obtained from the original pile driving records, is given to be 38 feet. This is seen to be in good agreement with the results obtained.

The data points in Figure 3.3.3 have been fitted to 2 straight line segments. The first segment corresponds roughly to the pile wave velocity in steel (16800 ft/sec). The second segment corresponds to a velocity of approximately 1000 ft/sec, which appears to be the shear wave velocity in the soil. The P-wave velocity should have a value around 5000 ft/sec (V_p in water for saturated soil). Thus it would appear that for points located below the end of the pile, most of the energy being transmitted to the receiver is in the form of shear waves, and the first arrival that is being observed is actually the arrival of the shear wave (the P-wave being undetectable). The shear wave presumably initiates an acoustic wave in the borehole fluid at the borehole-soil interface.

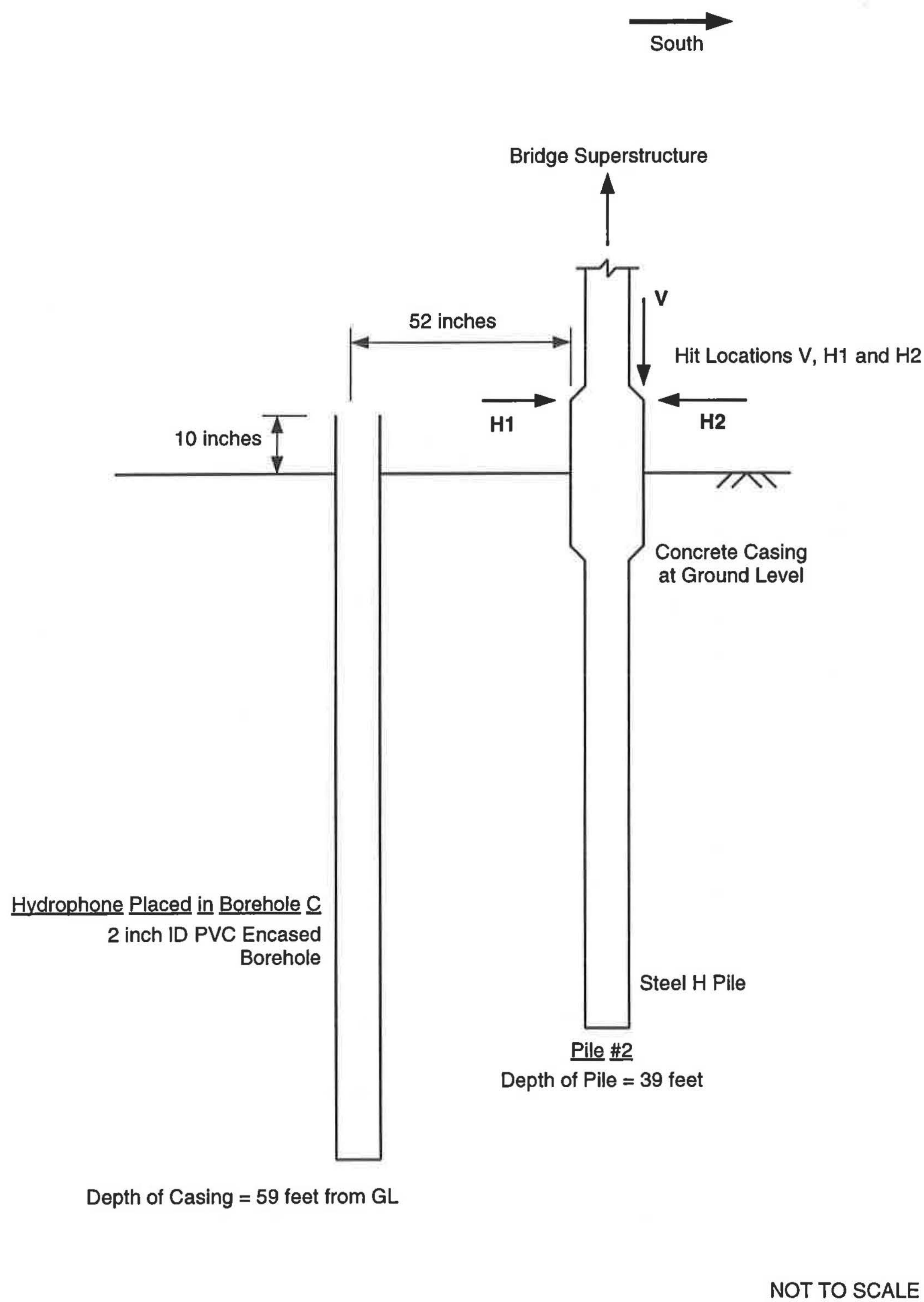


Figure 3.3.1 : Source and Receiver Layout, Pile #2

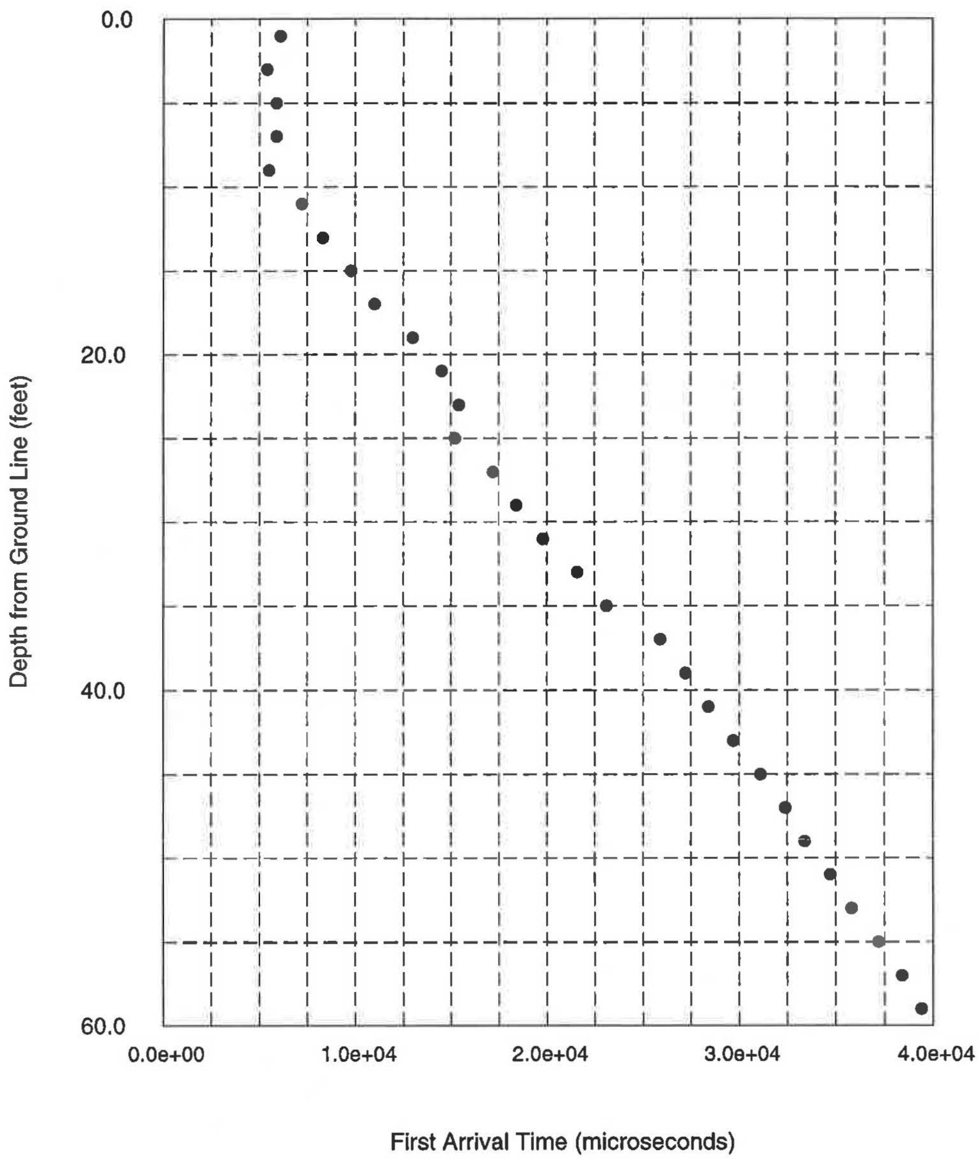


Figure 3.3.2 : Parallel Seismic Survey, Pile #2, Hit H2 (oo3.pl)

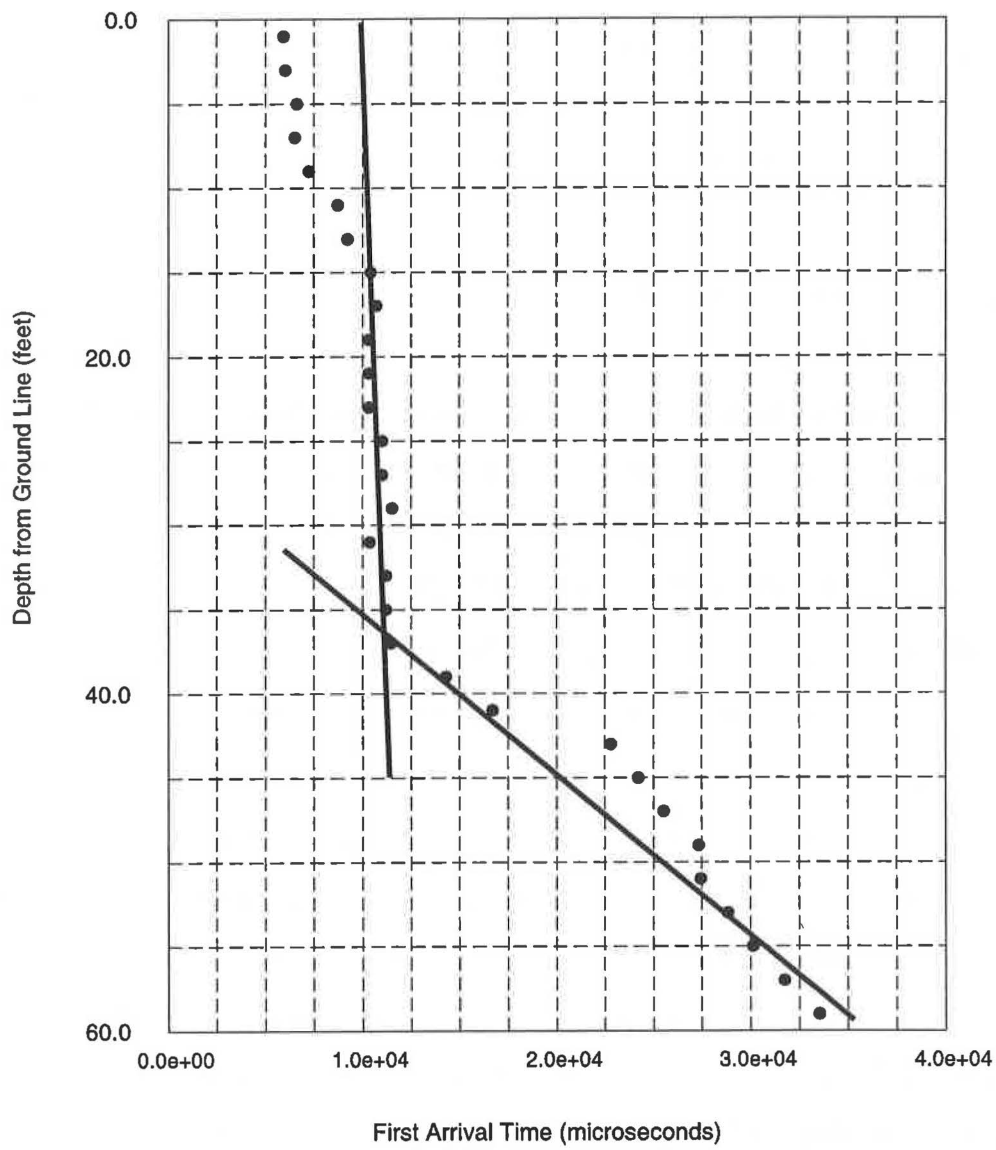


Figure 3.3.3 : Parallel Seismic Survey, Pile #2, Hit V (qq2.pl)

Test Results for Pile #3

Both hydrophones and geophones were used to attempt the determination of the embedded length for this particular pile. In both cases, records for the transmitted energy at each depth (traverse station) were generated by a total of 3 hits on the exposed pile substructure, 1 vertical and 2 horizontal. The test layout is illustrated in Figure 3.3.4, which provides details of the source locations (hammer hits) as well as location of the boreholes used to record the transmitted energy.

Analysis of Hydrophone Data

The data recorded by the hydrophones generally consisted of signals with mostly well defined arrival times, and with little or no high frequency noise which was so characteristic of the signals recorded by the geophones. 3 sets of records were produced by each downward traverse of the hydrophones down the boreholes A and B. An analysis of the arrival times produced the following results:

Hit H1 ⇒ slight break at about 36 feet below the ground line

Hit H2 ⇒ slight break at about 35 feet below the ground line

Hit H3 ⇒ sharp break at about 37 feet below the ground line

This indicates that the embedded length of the pile would lie somewhere in the range of 36 to 37 feet. The pile driving records states the embedded length to be 39.1 feet, which is in pretty good agreement with the results obtained from the hydrophone survey. Also, the best records were obtained when the source was impacted vertically, which produced waves rich in longitudinal components of vibration. Figure 3.3.5 illustrates a typical travel time log, which was produced by hit V with the probe located in Hole A.

Analysis of Geophone Data

The survey using geophones was carried out with the receiver traversing the depth of borehole A, which is located closest to Pile #3. Table 3.3.1 illustrates the results obtained for the various test combinations for source and receiver. It should be noted that the signals produced by hit H1 were of extremely poor quality, containing a lot of high frequency noise and showing a very marked decrease in signal energy with depth. No useful data could be obtained from the analysis carried out on these records.

Table 3.3.1 PS Results for Pile #3 (Geophone traverse)		
Source/Receiver Combination	Breakoff Location (feet)	Comments on data
V1/Vertical	40	very good records with well defined first arrivals (see Figure 3.3.6)
V1/Into Pile	41	good records
V1/Parallel to Pile	41	records not so clear, some high frequency noise present
H2/Vertical	41	good records
H2/Into Pile	40	some noise, decent records
H2/Parallel to Pile	39	very noisy signals, also large signal attenuation with increasing depth

The results indicate that the embedded length of the pile is within the range of 39 to 41 feet, which is in good agreement with the actual recorded depth of 39 feet. The vertical hits were again seen to produce the "cleanest" set of signals with very well defined arrival times. The signals received at shallower depths were seen to contain some high frequency noise, which dropped off rapidly with depth. Figure 3.3.6 illustrates a typical travel time log, which was produced by hit V1 with the probe located in Hole A. The signals were also seen to decrease appreciably in amplitude/energy content when the probe passed the pile tip and progressed further downward. Figure 3.3.6a shows the amplitudes of the signals recorded at depths of 4ft and 44 ft. It is obvious from these traces that the amount of wave energy that actually gets through to the receiver is much reduced with increasing depth. The energy produced by the horizontal hits were also subject to greater attenuation than those produced by the vertical hits.

It should be noted that the hydrophone data predicted an embedded length of 36 to 37 feet for pile #3. For the same pile, the geophone data appeared to indicate an embedded length between 39 to 41 feet, which is closer to the actual value.

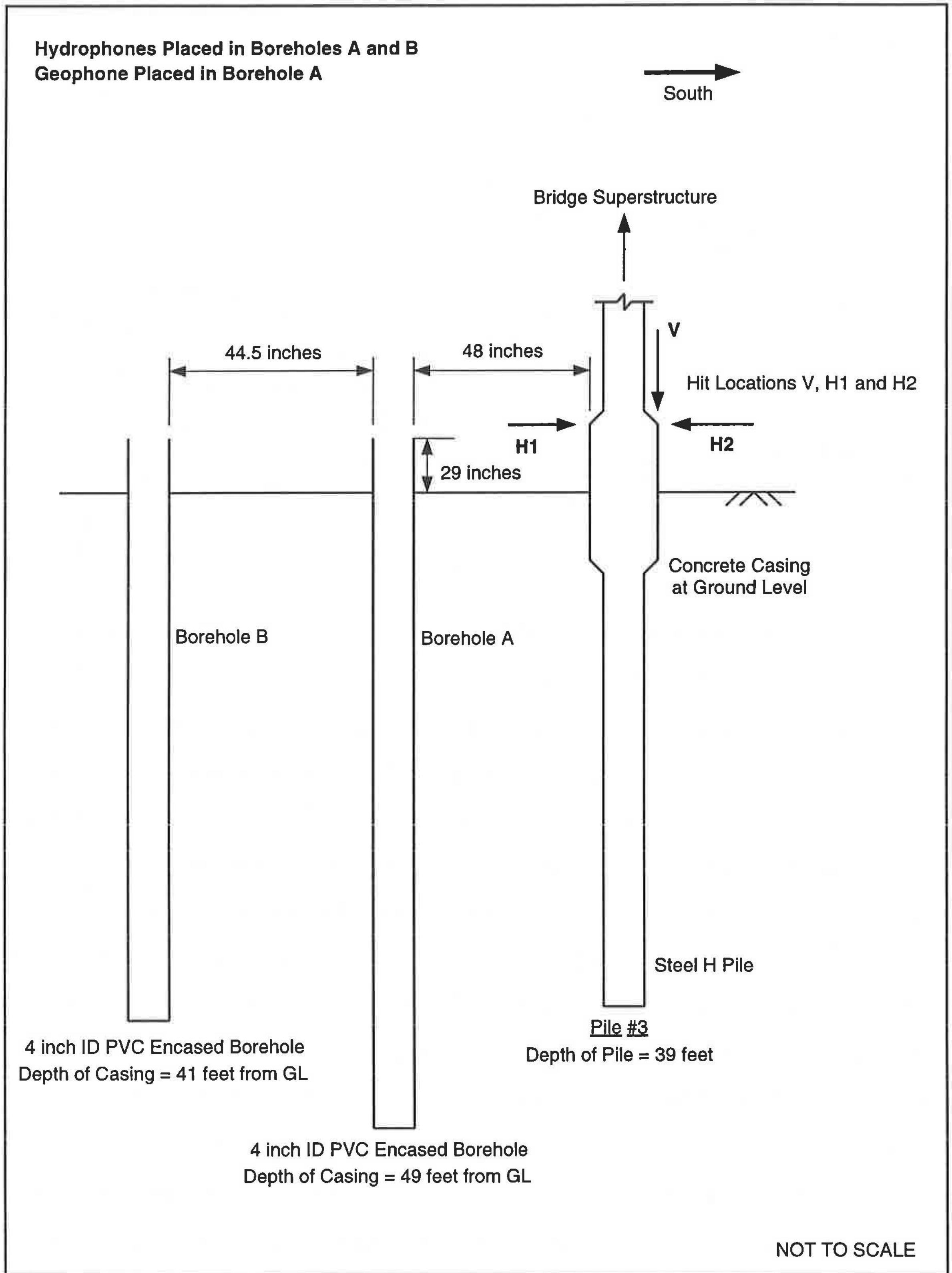


Figure 3.3.4 : Source and Receiver Layout, Pile #3

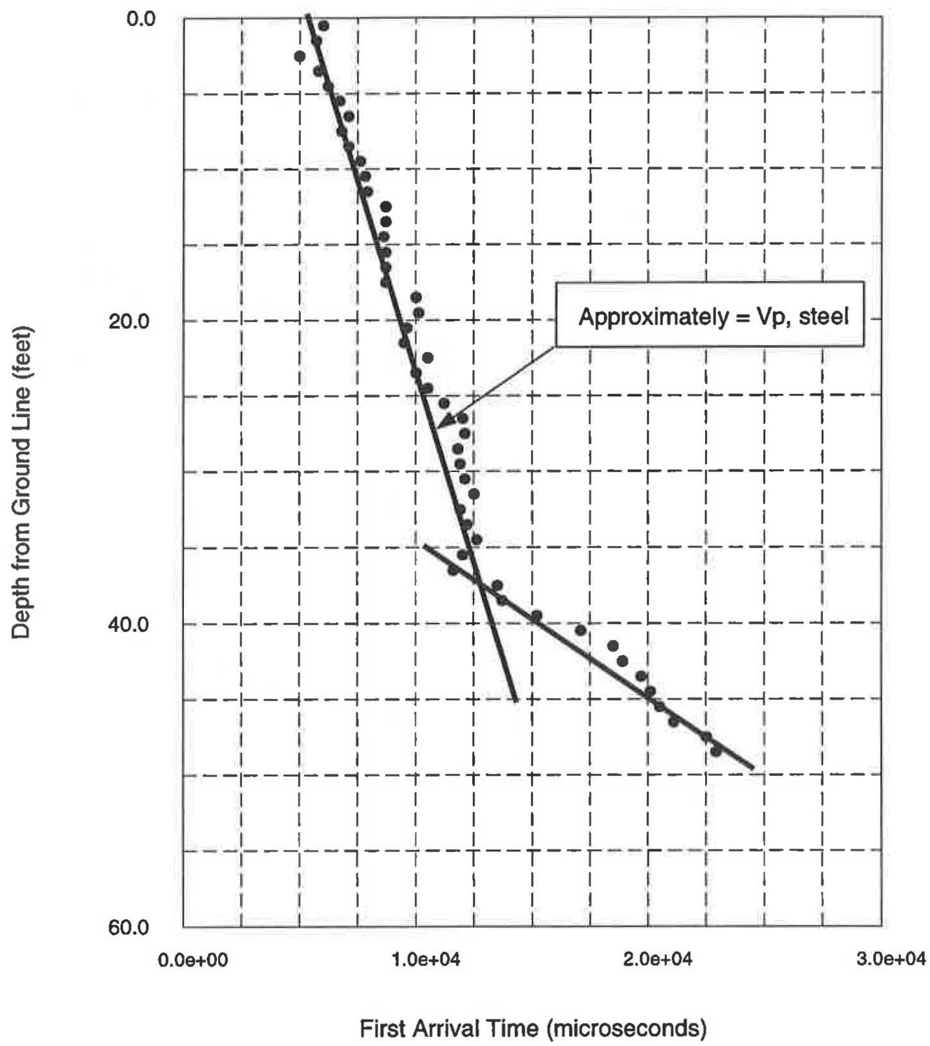


Figure 3.3.5 : Parallel Seismic Survey, Pile #3, Hit V (nn2.pl)

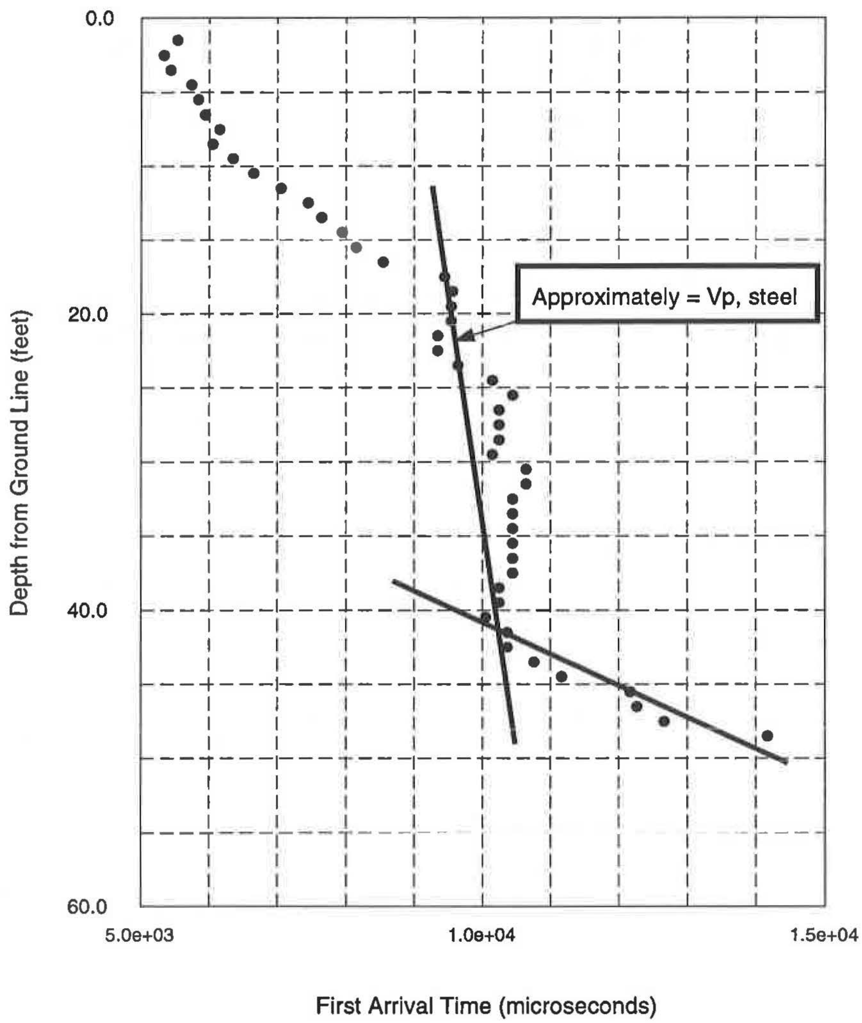


Figure 3.3.6 : Parallel Seismic Survey, Pile #3, Hit V1 (cc2.pl)

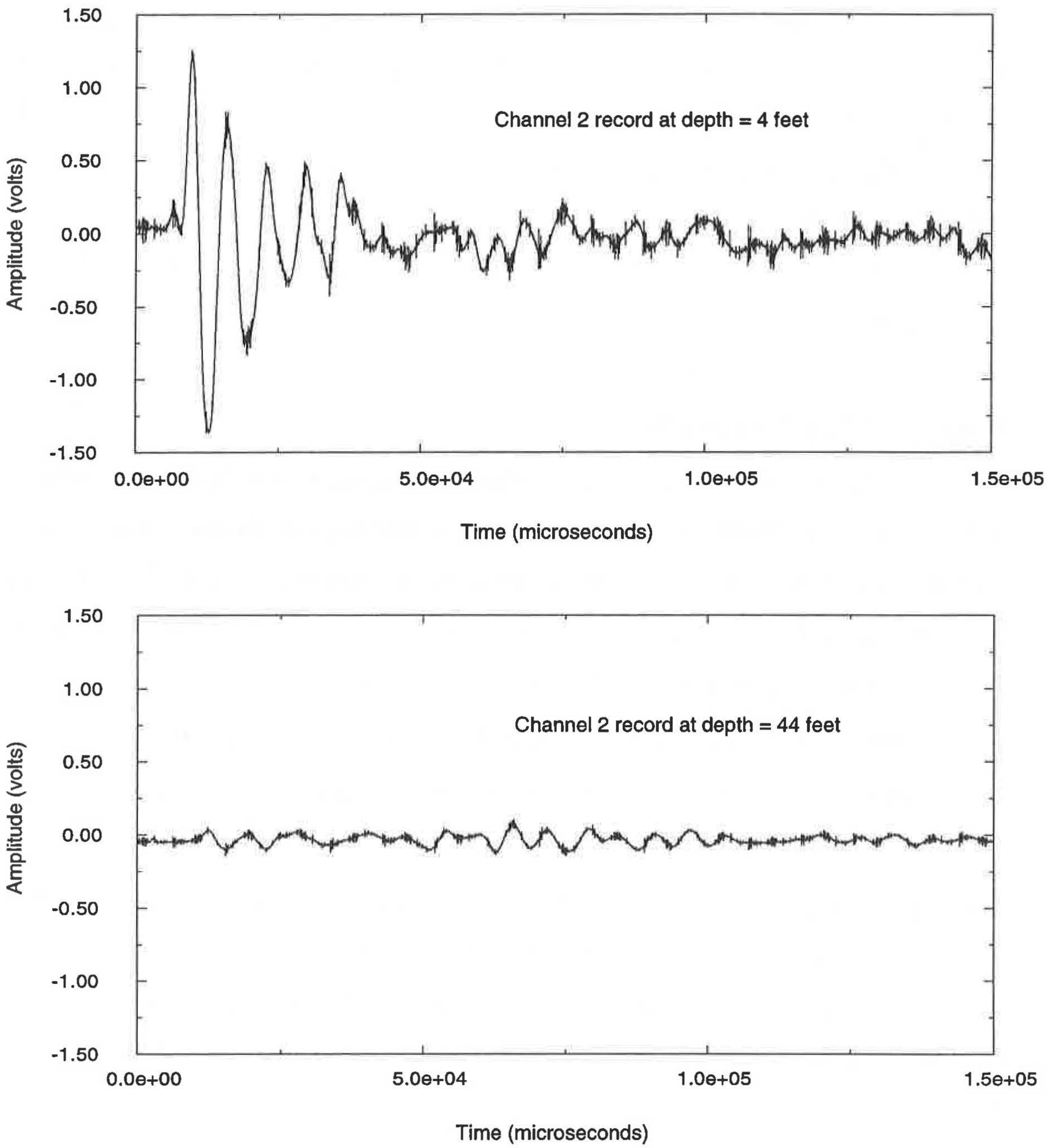


Figure 3.3.6a : Amplitude Difference between Records at 4 ft and at 44 ft.

Test Results for Pile #5 (easternmost)

Unlike Piles 2 and 3, Pile #5 was not vertical and had a batter of 1:8 (H:V). Also different from the 2 previous piles tested, was the fact that the boreholes were located at distances of 12 and 14 feet from the pile surface (at ground level), which was two times or more the corresponding difference for the two previous piles. However, it should be noted that the horizontal distance between the receiver in the borehole and the pile gets smaller with increasing depth, as the pile batter is in the direction of the borehole. As in the previous case, crosshole surveys were conducted using both the hydrophones and the geophones. The results from the analysis of these data are presented in the following paragraphs.

Analysis of Hydrophone Data

The hydrophone survey was conducted by using probes located in boreholes D and E, which are situated about 14 and 12 feet from the pile surface at the ground line. Recordings of the transmitted signal were made simultaneously within the parallel boreholes. As before, the signals corresponding to the vertical hits were seen to produce the best records so far as travel time logs are concerned.

Hit H1 $\Rightarrow$ very slight break at about 40 feet (not very well defined)

Hit H2 $\Rightarrow$ a slight break in the signal is seen at about 40 feet depth

Hit V1 $\Rightarrow$ break in the signal at a depth of about 39 feet

The embedded length indicated by the survey is about 38 to 39 feet, which corresponds well with the actual recorded length of 39 feet. However, it was noted that the breaks were not well defined for the horizontal hits, and the location of the pile end could not be determined with any degree of accuracy from these records generated by the horizontal hits.

Analysis of Geophone Data

The geophone data recorded consisted of a total of 9 records, using different sources and receiver channels. The following table (3.3.2) indicates the results of the analysis:

Table 3.3.2
PS Results for Pile #5 (Geophone traverse)

Source/Receiver Combination	Breakoff Location (feet)	Comments on data
V1/Vertical	41	very good records with well defined first arrivals
V1/Into Pile	40	good records
V1/Parallel to Pile	40	good records, some high frequency noise present
H2/Vertical	39	noisy signals, loss in amplitude at greater depths
H2/Into Pile	40	some noise, decent records
H2/Parallel to Pile	40	large signal attenuation with increasing depth beyond 40 feet
H1/Vertical	39	very noisy signals
H1/Into Pile	39-40	signals at greater depths very bad
H1/Parallel to Pile	39-40	very little energy reaching the receiver beyond 40 feet

As is apparent from the tabulated results, the vertical hits were the most effective in producing good signals with well defined first arrivals. The data indicate that the pile tip is located between 38 to 40 feet below the ground line, which is in good agreement with the actual length. Figure 3.3.7 is a plot of a typical First-Arrival Record, within which the break at a depth of 39 feet can be seen clearly.

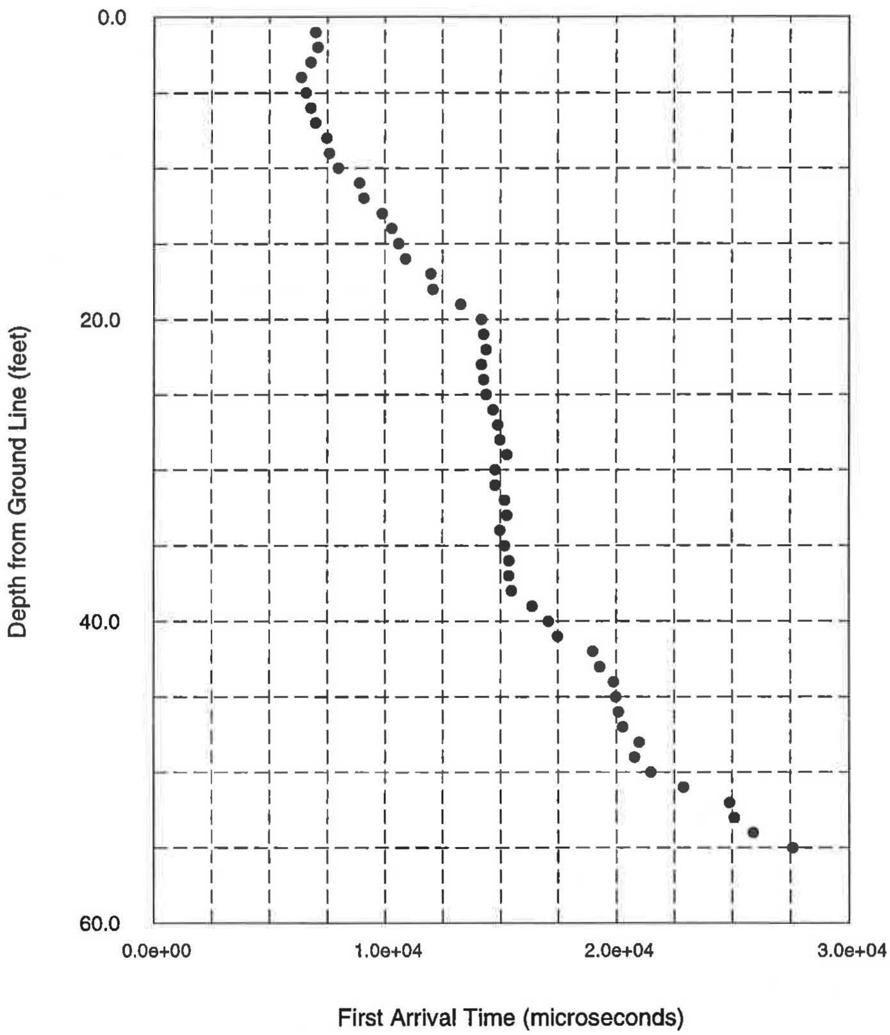


Figure 3.3.7 : Parallel Seismic Survey, Pile #5, Hit H1 (ff3.pl)

3.3.2 Conclusions and Recommendations (PS Method)

The parallel seismic method proved to be particularly well suited to the determination of unknown pile lengths under existing bridge structures. The most important drawback to its use lies in the need for providing boreholes within close proximity of the foundation element being tested. In a lot of cases where headroom is limited by the dimensions of the bridge superstructure, it might not be possible to drill boreholes directly underneath the bridge deck using conventional drilling equipment. In such cases, a PVC casing might conceivably be installed within the ground by using jetting or spudding methods. As to the PS method itself, the following points were noted during the course of this study:

- Vertical hits were seen to provide the best records as far as first arrival logs were concerned. Signals produced by horizontal hits were seen to contain comparatively more high frequency noise, and were subject to greater attenuation losses with increasing depth, when compared to signals produced by vertical hits. This made the choice of first arrival times very difficult for signals produced by the horizontal hits at depths below 40 feet or so, in the case of both hydrophones and the geophones. The smudging over of the 1st arrival pulse due to the dispersive nature of the wave can also lead to inaccuracies in selecting a first arrival time.
- The signals recorded by the hydrophones (pressure transducer) were seen to be cleaner (with respect to high frequency noise) than those recorded by the geophones (velocity transducers). However, first arrival times tended to be better defined within the geophone signals. The presence of some very low frequency noise (roll) with low amplitudes was noted for the hydrophone data, which might have resulted from some ambient groundwater conditions present at that time.
- A possible means of negating the ill effects of signal attenuation with depth might lie in the use of automatic gain control. Prior to display and analysis, the data recorded at greater depths with correspondingly weak signal amplitudes, might be amplified to the same level as the data obtained at shallower depths. However, such a process might also amplify the ambient noise to an extent which would

render meaningful interpretation of the data difficult. Also, this would require the use of sophisticated signal processing software which is not available to the research team at present.

- It is also possible that in certain cases (horizontal hits) it is the relative velocity difference between the bending wave in the pile and the shear wave in the soil that is being observed. In such cases, the velocities are much closer which makes the task of identifying the pile toe even harder from the stacked data.

3.4 Ground Penetrating Radar (GPR)

In terms of use and interpretation, the Ground Penetrating Radar is probably the simplest of the 5 methods that are being studied for purposes of this project. The GPR instrument functions both as an emitter of radio waves, usually in the 1 to 2000 MHz range, and also as a receiver to detect and record the wave energy that is reflected back to the instrument from the surrounding structure. When wave energy travelling through a medium encounters an interface which separates that medium from another with different material properties, a portion of the wave energy is transmitted through to the second medium, while another portion is reflected back into the first. The material properties which determine this distribution of energy at an interface are the material dielectric constants or the electrical impedances. The amplitude of the reflected wave is governed by the reflection coefficient, R, which in the case of normal incidence for a plane wave is given by the following equation:

$$R = (\sqrt{K_1} - \sqrt{K_2}) \div (\sqrt{K_1} + \sqrt{K_2})$$

where,

K_1 = dielectric constant of the first material

K_2 = dielectric constant of the 2nd material

For purposes of determination of unknown pile length, the GPR is lowered down an encased borehole located close to the target pile. The technique has been graphically illustrated in Figure 3.4.1. The instrument acts as a continuous emitter and the corresponding reflection pattern is continuously monitored and recorded as the probe is

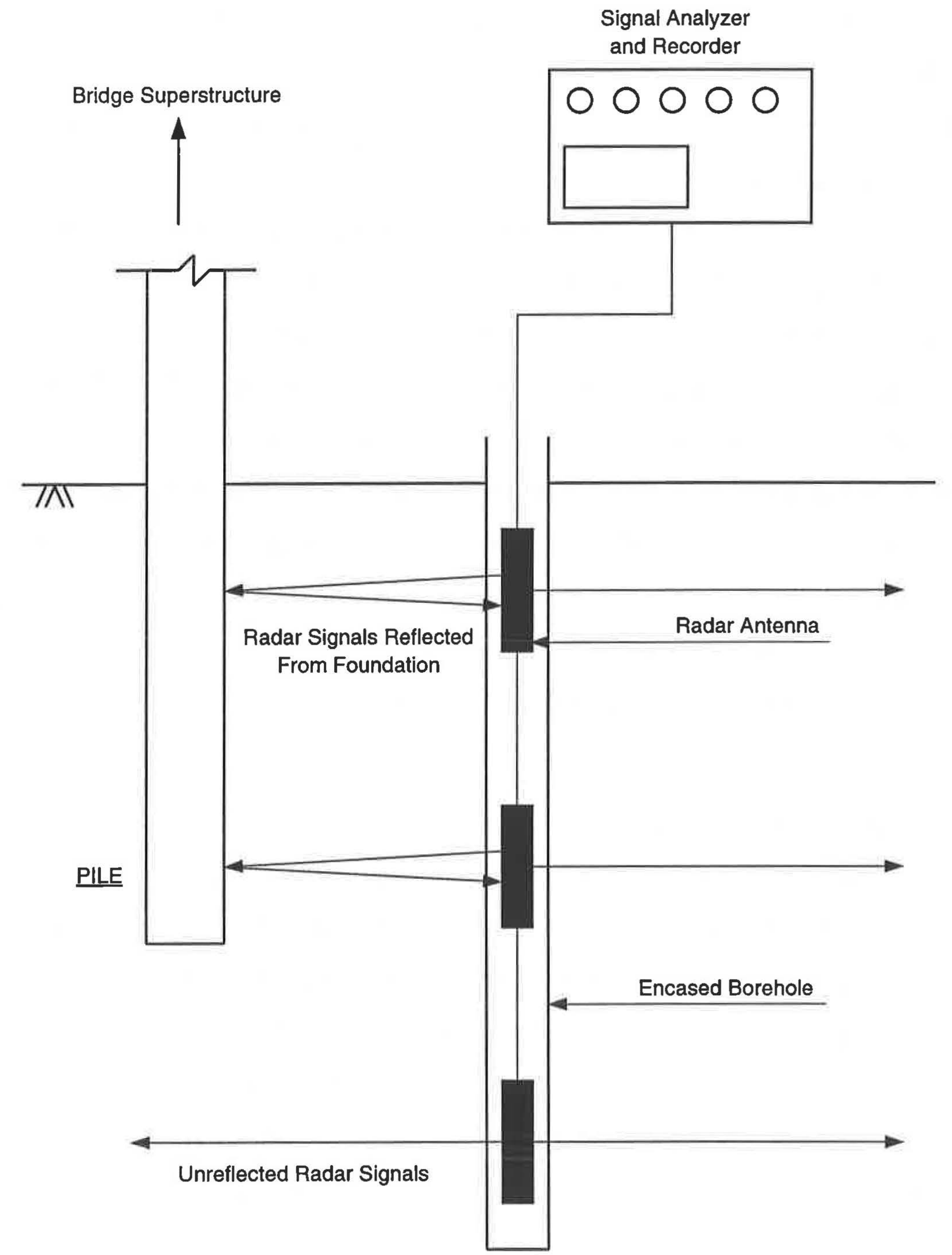


Figure 3.4.1 : Borehole Radar Method

either lowered down the borehole or pulled up at a steady rate. So long as the depth does not exceed the depth of the embedded foundation, a clear reflection from the foundation should be visible within the receiver record (some limitations exist which will be stated later). In this regard, the contrast between the dielectric constant of the foundation and the surrounding soil is important, and for cases where steel is used as the foundation material (H piles), this contrast is substantial. For best results and the clearest resolution the borehole should be as close to the pile as is practically possible. The intensity of the outward directed wave energy is rapidly diminished as the wave propagates in three dimensional space (due to geometrical spreading as well as losses along the path). Therefore, if the pile-borehole distance is large, a reflection signal from the distant pile may not be apparent in the records. The rate of attenuation of the radar signal is a function of several factors. Typically, an increase in the water content of the soil increases the intrinsic attenuation of the wave energy, which results in a reduced depth of penetration of the radar signal. Some other factors which influence attenuation and confuse the records are the conductive properties of the soil, the salinity of the ground water, and the presence of other buried structures, either man-made or natural, which exhibit significant contrast with the natural earth material.

3.4.1 Test Results for GPR

Borehole radar tests were conducted on piles 3 and 5 supporting Bent #4 on the Alabama River bridge. The testing was carried out by Geophysical Survey Systems, Inc. (GSSI), using a 120 MHz, omnidirectional, monostatic borehole radar system. The receiver instrument was approximately 3 inches in diameter and about 3.5 feet in length, which allowed its placement within the 4 inch ID encased boreholes.

In case of both the piles tested using the borehole radar method, a clear reflection from what is believed to be the embedded pile is evident within the record at shallow depths. Unfortunately, the graphic plot which is produced by the GPR recorder is not currently available to the research team, and thus cannot be reproduced within this report. In the case of pile #5 which is the outermost at that bent and battered outward (1.5 inches per foot), the reflection is seen to change position with depth in a manner consistent with the batter angle, which serves to strengthen the belief that it is indeed the reflection form the pile that is visible. However, this prominent reflection dies down in intensity below a depth of approximately 30 feet from the ground level in case of both piles, and the data appear to suggest that the pile tips are located at about this depth (28.1 feet for pile #3 and 31 feet for pile #5). The construction records indicate embedded lengths between 38 and 39 feet for both piles, which is approximately 25% greater than what is indicated from the borehole radar tests. It is unclear as to why no reflection from the pile could be obtained at depths greater than 30 feet, and further tests need to be carried out using the borehole radar to determine conditions under which this instrument might be used with a greater degree of reliability.

3.5 Dynamic Foundation Response (DFR)

The dynamic foundation response method was one of the techniques that was chosen for closer inspection and evaluation for purposes of this study. This method is based upon the theory of conventional **modal analysis** techniques, which are commonly used in the automobile and aerospace industries to determine vibration characteristics of mechanical components, like automobile body shells and aircraft wings. Modal analysis techniques have also been used in the field of Civil Engineering, again with the same purpose of determining system vibration parameters. Of particular interest in the field of dynamic soil-structure interaction is the work done by Novak (1976, 1978), who has formulated simplified closed form solutions to the problem of modeling the dynamic response of deep foundations (piles).

The proposed technique would consist of actual dynamic measurements made in the field, coupled with theoretical modeling of the foundation response in the laboratory. The following steps might be undertaken to achieve the desired results:

1 A signal/force generator may be used in the field to excite the target foundation over a broad range of frequencies. The excitation input might consist of a repeating load signature varying sinusoidally with time. The displacement response of the foundation would have to be monitored and the maximum amplitude of vibration recorded at each input frequency. In this way an actual response spectra might be generated from the field measurements.

2 Once the field response spectra has been obtained, a forward modeling procedure may be utilized to generate theoretical response spectra for a suite of models having different embedded lengths of the foundation elements. This computed response would then be compared to the actual or field data, and estimates of embedded pile length may be made using the theoretical model which best correlates with the field data.

However, several shortcomings of the method proposed above come to mind upon closer inspection of the concepts upon which the technique will be based. One of the foremost concerns with carrying out actual modal testing of a bridge structure is to attain the desired excitation of the foundation elements. To obtain meaningful data, the foundation element(s) and the associated bridge superstructure would have to be excited at very low frequencies, at or very close to the fundamental mode of vibration of the structure. It would be very difficult to energize the lowest modes i.e. impart sufficient energy to the foundation at frequencies which are close to the fundamental frequency using conventional non destructive testing equipment. An impulse hammer in all probability would prove inadequate for this task, even with sophisticated FFT processing of the resulting transient signal. Heavier signal generators utilizing the principle of unbalanced rotating masses would be required for this purpose. Even if it were possible to excite the lowest modes, problems might arise due to excessive amplitudes of vibration of the bridge superstructure and the foundation at or near the resonant peaks. Excessive deformations might lead to unsafe conditions and even cause structural damage to the elements being tested.

Another drawback to the dynamic foundation response method would lie in the theoretical modeling of the foundation structural elements. No closed form solutions are available that take into consideration the various complexities of the soil-structure interaction problem. The simplified closed form solutions proposed by Novak and other researchers would prove inadequate for purposes of realistically predicting the dynamic parameters that govern the response of actual soil-structure interaction systems. One possible solution would lie in the use of numerical models (finite difference or finite elements) to determine the dynamic system parameters, like the generation of synthetic response spectra. Such spectra could be generated for a suite of models with slightly varying embedded lengths, and a good match between a particular spectra and the actual field data might lead to the determination of unknown embedded length. Caution would have to exercised in such a procedure however, as the solution to such forward modeling techniques are usually nonunique. A range of models might lead to the generation of very similar synthetic spectra, but even in such a case one might expect the suite of solutions to be of a bounded nature. At the worst, an estimate for embedded pile length could be specified within a maximum and a minimum value.

Primarily due to the various difficulties identified above, the dynamic foundation response has not been tested in the field as a part of this research. But in spite of the associated complexities, this method shows some promise for the purposes of determination of embedded pile length and deserves a closer study.

4. A Theoretical Finite Element Study

This chapter presents the results obtained from a theoretical study of the Parallel Seismic Testing Procedure using the finite element method. Finite elements have been used in the past to model the wave propagation phenomenon in 1, 2 and 3 dimensions (Laturelle, 1989, Tedesco et al, 1989). Some guidelines as to the modeling procedure is also available in the text by Bathe (1982), which deals with the formulation of both the finite elements that may be used for such analysis, and the various numerical integration schemes that may be used to obtain response in the time domain. The typical wave propagation problem may be described as the transient response of an elastic material to a pressure pulse input at some point upon its surface. For the particular case of the impulse applied to the pile-soil system, the system may be idealized as an elastic pile embedded within an elastic half space (soil). **Linear elastic behavior** may be assumed for both the pile and soil material, as well as at the pile-soil interface. This is because the strain levels generated during a typical nondestructive test like the parallel seismic test are very small, and the material behavior does not deviate significantly from the linear portion of the constitutive stress strain curve. Nor does such a test result in any permanent deformations of the pile (no slip at the pile-soil boundary). Obviously, such an assumption of linear elasticity cannot be used for the wave propagation phenomenon experienced during pile driving, which is associated with extremely large strains and permanent deformations (set).

4.1 Modeling Considerations

The solution to a dynamic problem using finite elements is largely dependent upon 2 factors; namely, the finite element idealization incorporated in the model, and the time integration scheme used to obtain solutions (Bathe, 1982). Both are closely related and have to be chosen depending on the actual physical problem to be analyzed, which may either be a **structural dynamics problem**, or a **wave propagation problem**. The former is essentially set up as a static problem which includes inertia effects. For such a solution, a significant portion of the energy of vibration is contained within the lower modes, and the time of interest and the characteristic duration of load generally exceed several wave travels across the structure. For wave propagation problems however, the time of interest is usually limited to the same order as the typical wave travel time, and a significant energy contribution results from the higher modes, which effectively rules out the use of mode-superposition techniques. Thus, severe limitations are imposed upon the element size to be used as well as the time step to be adopted for wave propagation analysis. It is usually recommended that the "effective" element size be limited to the value L_e, where

$$L_e = c_L \times t_w / n$$

where,

c_L = velocity of wave propagation

t_w = characteristic wave propagation time, which is the time required for one critical wavelength to travel past a point

n = number of time steps necessary to represent the wave length

The corresponding time step to be used for time integration would thus be given by

$$\Delta t = L_e / c_L$$

The effective length and the corresponding time step chosen must be able to represent the complete wave travel accurately, and would thus be a function of the element idealization and the time integration scheme used for analysis. For complex 2 and 3 dimensional problems it is usually quite difficult to determine the values of L_e and Δt which produce the most accurate solution, and these parameters are also dependent upon the time integration scheme used. The use of a **lumped mass** formulation is recommended when an **explicit time integration** scheme is to be used (central difference method). Lower order elements would best serve the purpose of such an analysis; namely 4 and 8 noded elements respectively for a 2 and a 3 dimensional model. When such a formulation is employed, the "effective" element size L_e is given by the smallest distance separating any two nodes within the mesh. An explicit scheme may be used when the time required to generate the solution is an important factor, as large finite element runs on powerful computing systems usually cost a lot of money, and explicit methods prove the least computationally intensive in this regard. Explicit algorithms are based upon the decoupling of the coupled equations of motion by using eigensolution techniques to extract the necessary number of natural frequencies and mode shapes. The decoupled equations are then solved individually and independent of each other, and solutions can be obtained to any degree of accuracy necessary simply by changing the size of the modal transformation matrix. An **implicit algorithm** on the other hand works with the direct integration of the coupled equations of motion, and thus requires the solution of a set of simultaneous algebraic equations at each time step. Since there are as many equations as there are degrees of freedom in the model, the computational effort and hence the cost involved can be substantially more for large models.

The need for a more refined and accurate solution necessitates the use of higher order parabolic or cubic continuum elements. In such a case L_e is no longer dictated by the spacing of the nodes defining any particular element and the time step has to be further reduced. In addition, if structural elements like beams or plates are incorporated in the model, the time step Δt may be governed by the flexural modes of the elements. In such a case, an estimate for the lower bound for the smallest period of the mesh T_n would result from the computation of the smallest period for any individual element used in the mesh, which can usually be obtained in closed form. The time step Δt would then be given by $\Delta t \leq T_n/\pi$.

If an implicit time integration scheme were to be used, the choice of the time step and the element length become considerably simpler. In such a case L_e can be equated to the smallest distance between any two consecutive nodes that lie on the path of wave travel. Implicit schemes are also preferable from the point of view of solution stability. The stability of such a scheme is unconditional and does not depend on the time step adopted. The choice of the time step is effective only in determining the accuracy of the solution. It has been shown that a **consistent mass** idealization used together with an implicit integration method leads to better results in the case of wave propagation through a semi-infinite half space (Laturelle, 1989). The trade-off lies in the use of the generalized (consistent) mass matrix as opposed to the simpler and less time consuming diagonal system matrix generated by a lumped mass formulation. The implicit schemes are more computationally intensive then the explicit schemes; however, a larger time step may be used without stability problems. The methods typically employed for implicit time integration are the Newmark-β family of methods and the Wilson-θ technique. The formulation of such methods are essentially the same, the difference lying within the constants of integration and the interpolation functions that are used for each. The stability and accuracy characteristics of both implicit and explicit methods are described in detail in the text by Bathe (Bathe, 1982).

4.2 The Parallel Seismic Wave Propagation Model

This study attempts to model the wave propagation mechanism associated with the parallel seismic test described in chapter 3.0 of this report. The setup for the physical model has been shown in Figure 4.1, which illustrates the use of 2 domains where the kinematic response of the model will be monitored. The dynamic behavior of Domain A is of primary interest; Domain B has to be included in the model in order to remove the effects of the spurious reflections associated with the model boundaries. The physical system which we are attempting to model does not contain any such boundaries; a finite element model however needs boundaries for its definition. Energy absorbing boundary elements may be used to prevent or at least reduce this problem of unwanted reflections, but such an element formulation is not

DOMAIN A is the primary area of interest and is modelled with a very fine mesh
DOMAIN B is required to prevent the generation of any spurious reflections
from the finite element model boundaries within the time of interest

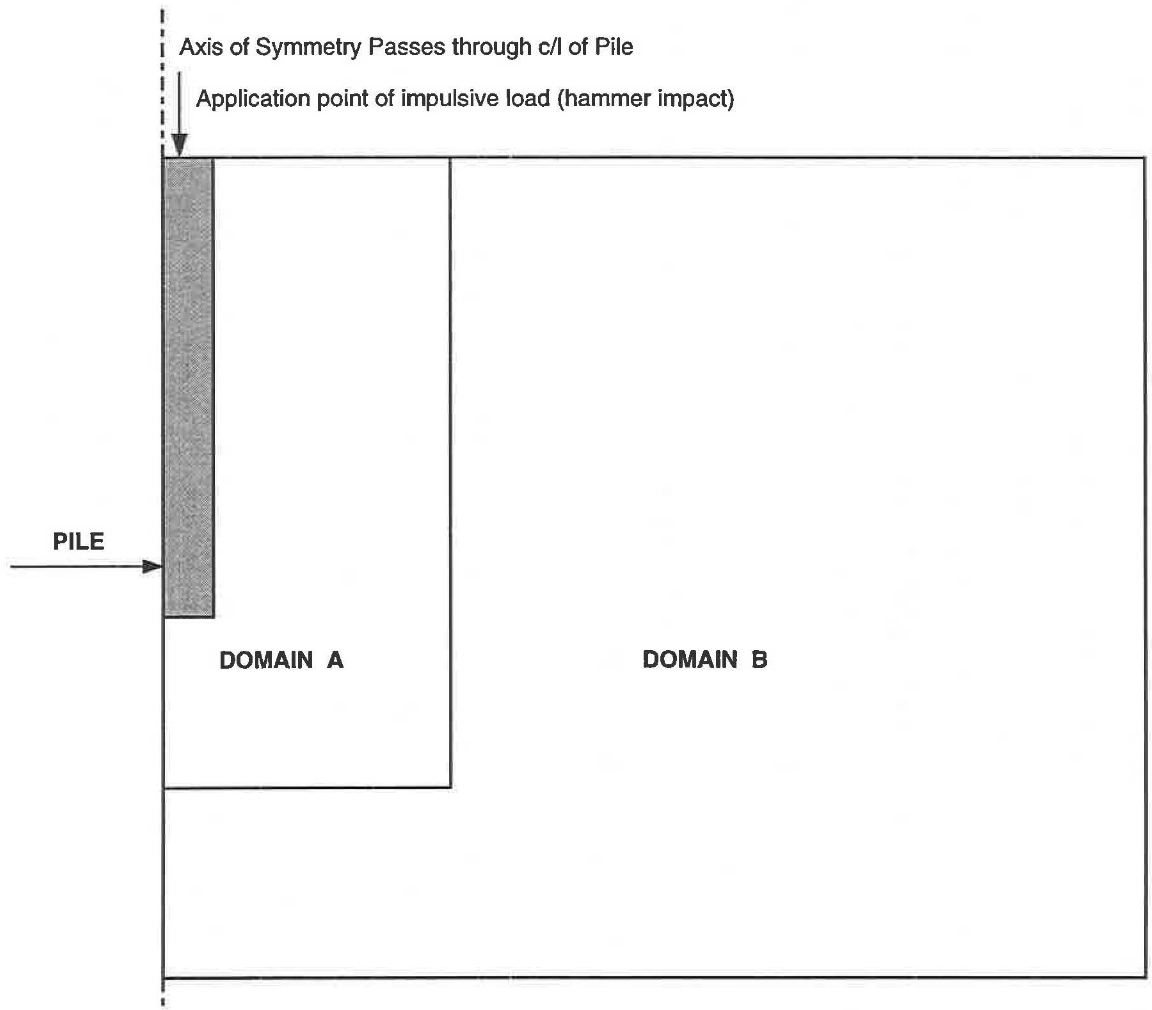
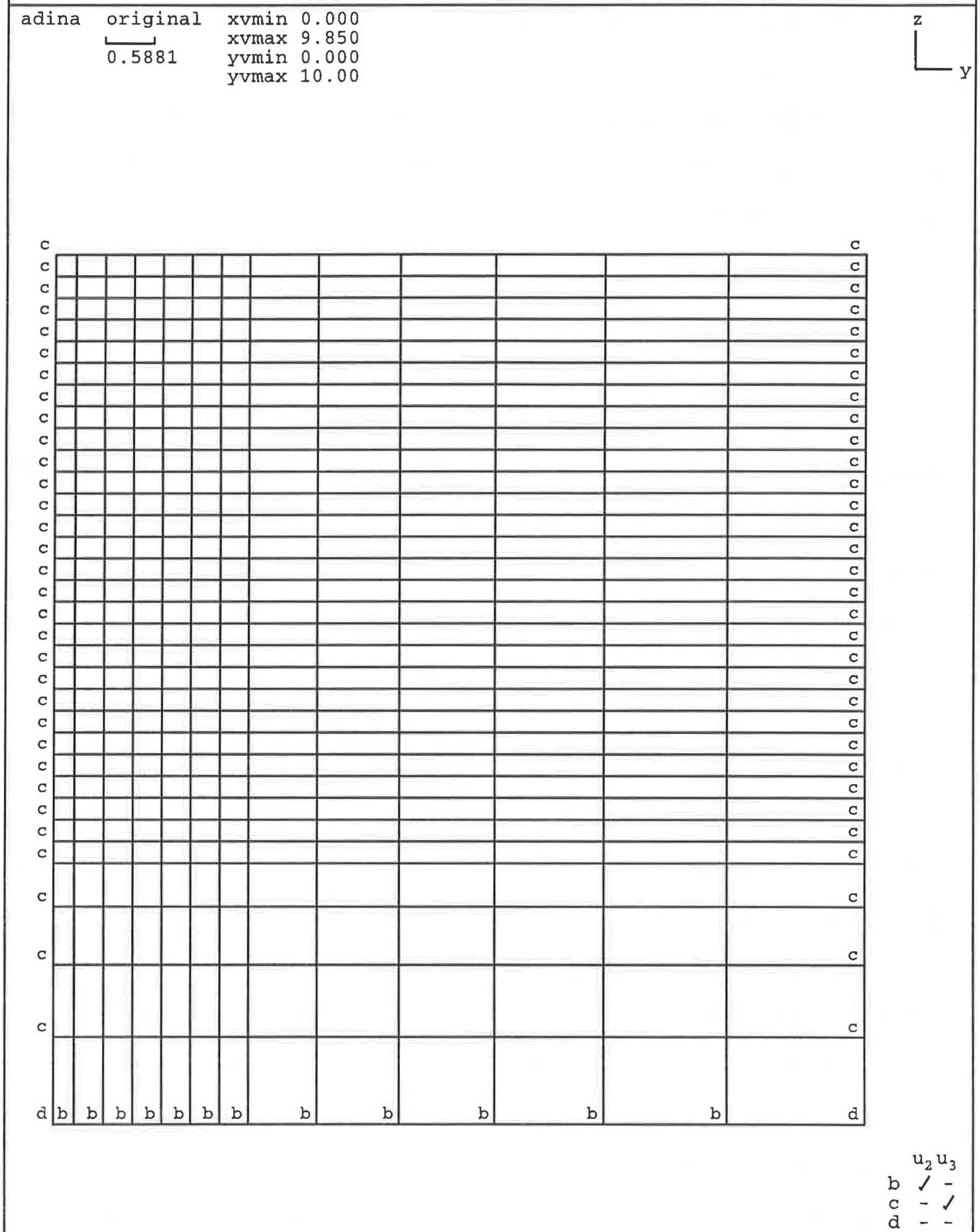


FIGURE 4.1: PHYSICAL MODEL FOR WAVE PROPAGATION ANALYSIS (PS)

Figure 4.2 : Total Finite Element Mesh used for PS Simulation



currently available with the FEM software used. In any case, if the model boundaries are defined at distances far removed from the zone of interest, the undesirable reflections are shifted back in time and effectively do not interfere with the phenomenon of interest. It should also be noted that the top of the pile is located at the upper boundary of the half space (earth). Piles having varying depths of embedment can also be modeled with ease; however, the current study is limited to this single case of total embedment.

4.3 Element and Mesh Generation Parameters

A detailed discussion of the requirements pertaining to mesh size and element density has been provided in the previous section (4.1). As a preliminary model, the mesh illustrated in Figure 4.2 was chosen for analysis purposes. The outer dimensions of the mesh are 9.85m (horizontal) and 10.0m (vertical). As a study of the mesh reveals, the node spacing and element size have not been maintained constant. The mesh has a constant finer density closer to the pile (top left corner of the model) than further away toward the model boundaries.

4.3.1 Finite Element Formulation Used :

All the elements used within the model are *4-noded 2-dimensional axisymmetric* elements. Also, a *lumped mass formulation* was used to represent the mass distribution characteristics of the entire system. As the problem essentially generates very small strains within the system, *no damping* has been included for the model. The effect of damping would perhaps need to be considered if the the model were used to simulate the response of the pile top near the point of impact (sonic echo test).

4.3.2 Pile Dimensions :

The pile dimensions were fixed at a length of *5.0m* with a diameter of *0.5m.* As the system is symmetric about the center-line of the pile, the dimensions of the pile actually modeled are 5.0m (length) by 0.25m (radius). The mesh size used within this domain was *0.25m by 0.25m.*

4.3.3 Domain A :

The dimensions for Domain A were fixed at *2.1m (horizontal) and 7.0m (vertical).* The size of each element in this domain was maintained at *0.25m (vertical) by 0.35m (horizontal).* A detailed representation of this domain has been illustrated in Figure 4.3, which also indicates the node numbering scheme used. The node numbers are important because the kinematic response of the model will be obtained at these nodal locations. Also, a somewhat coarser mesh might have been used to represent the soil, as the compression wave velocity within this medium is much less than the corresponding velocity within the pile material.

4.3.4 Domain B :

Domain B has been modeled using elements whose size progressively increase with increasing distance from the pile. This is permissible because the dynamic behavior of soils far removed from the pile will not contribute significantly to the behavior in domain A, toward which the primary thrust of this study is directed.

4.3.5 Time Integration Scheme Used :

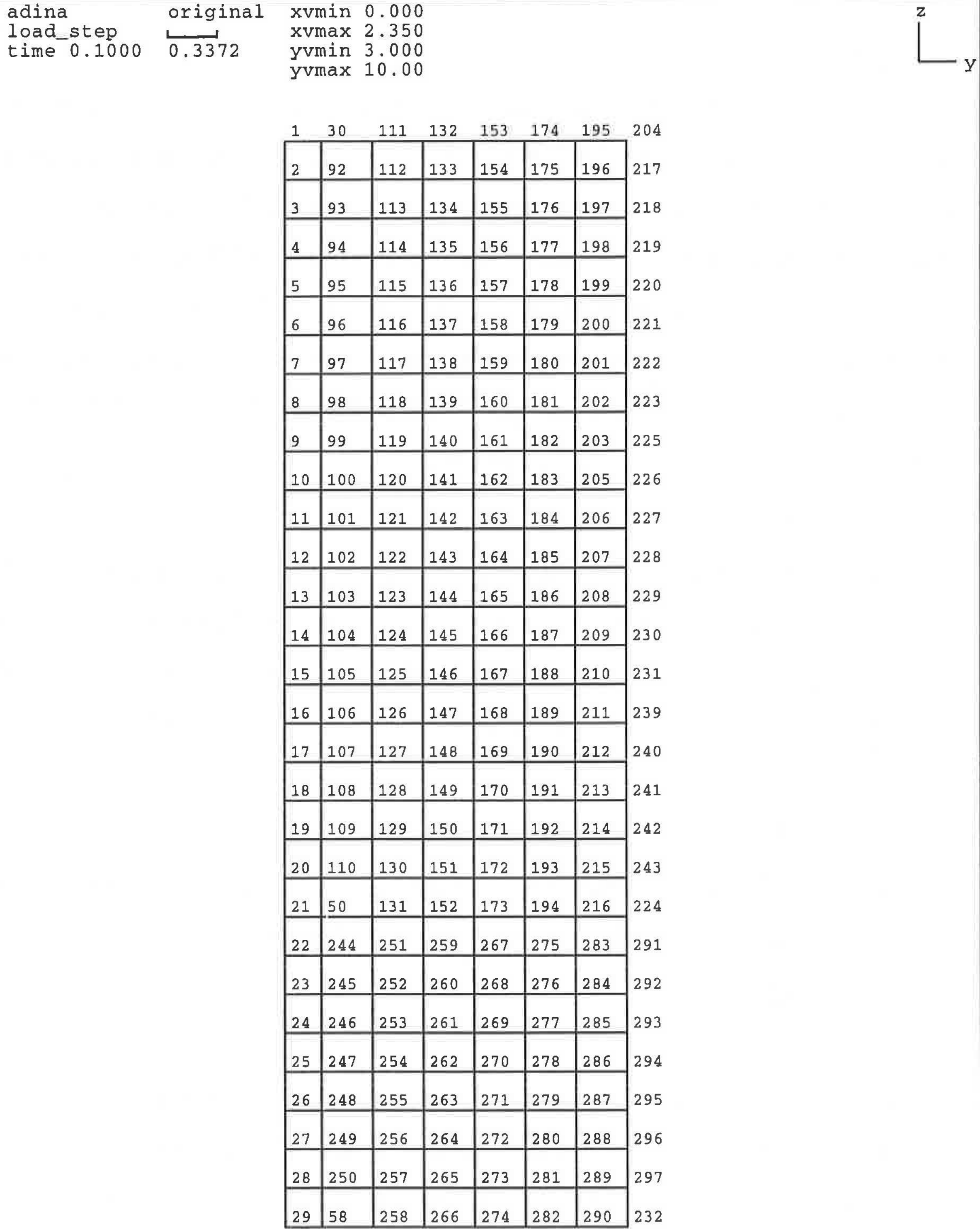
The *Newmark-*β scheme was used to integrate the coupled equations of motion through time. The values of the integration parameters were set at:

$\alpha = 0.25$

$\delta = 0.50$

Analysis was carried out using a time step, Δt= 25E-6 seconds (*25 μsec*). Another run using a larger time step of 50 μsec was also carried out for studying the vibration characteristics of the system for a larger duration of time.

Figure 4.3 : Details of Mesh Within DOMAIN A

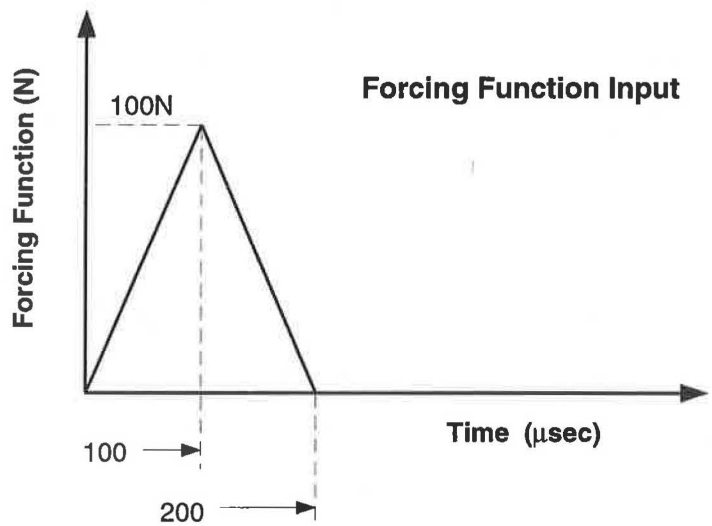


4.3.6 Material Properties :

Material Properties for Pile and Soil		
Property	Pile (concrete)	Soil
E (Pa)	3.31E10	1.80E8
ν	0.2	0.4
ρ (Kg/m^3)	2300.0	1925.0
Vp (m/s)	4000.0	448.0
Vs (m/s)	2450.0	183.0

4.3.7 Forcing Function :

The forcing function used has been shown below. This is a simplified representation of a typical hammer blow applied during the parallel seismic test.



4.3.8 Finite Element SoftWare Used :

ADINA ver6.1 has been used for all the finite element analysis carried out for this study. ADINA (Adina, 1984) is a state of the art FEM program that has been developed under the guidance of Professor Klaus Jurgen Bathe at MIT. Version 6.1 of this code is currently installed on a 2-processor Cray XMPc90 at the Alabama Supercomputing Network located at Huntsville, Alabama. Pre and post processors (Adina-In and Adina-Plot), intended for use with this program are also available at the same location.

4.4 Analysis and Results

Two runs were carried out to obtain the response of the system in the time domain. The first used a time step of 50 μsec and the response was obtained for a total period of 50 msec. The second used a value of Δt=25 μsec for a total time of 20 msec. The velocity response was plotted for node points located within domain A, and first arrival values were scaled directly from these plots. The following figures are the first arrival logs for node points located at a distance X from the centerline of the pile:

Figure 4.4 : Vertical velocity (first arrival), X=0.6 m

Figure 4.5 : Vertical velocity (first arrival), X=2.35 m

Figure 4.6 : Radial velocity (first arrival), X=0.6m

Figure 4.7 : Radial velocity (first arrival), X=2.35m

Graphs of the actual time history response at selected points within domain A have been provided in Appendix B.

4.4.1 Pile Length Predictions

The following table provides the estimated depth to the pile tip from Figures 4.4 through 4.7. Two straight lines have been fitted to each set of first arrival data, and the pile tip is defined at the intersection of the 2 fitted lines. It should be noted that the actual length of the pile incorporated into the finite element model is 5 m

Estimated Pile Length			
Figure	Distance from Pile c/l (pile diameter)	Depth of Pile (meters)	Comments
4.4	1.2	5.25	z-velocity
4.5	4.7	4.95	z-velocity
4.6	1.2	5.25	r-velocity
4.7	4.7	5.6	r-velocity

As can be seen from the figures, the first arrival data obtained at a distance of 0.6m from the pile shows the presence of 2 distinct straight lines (very good linear fit), for

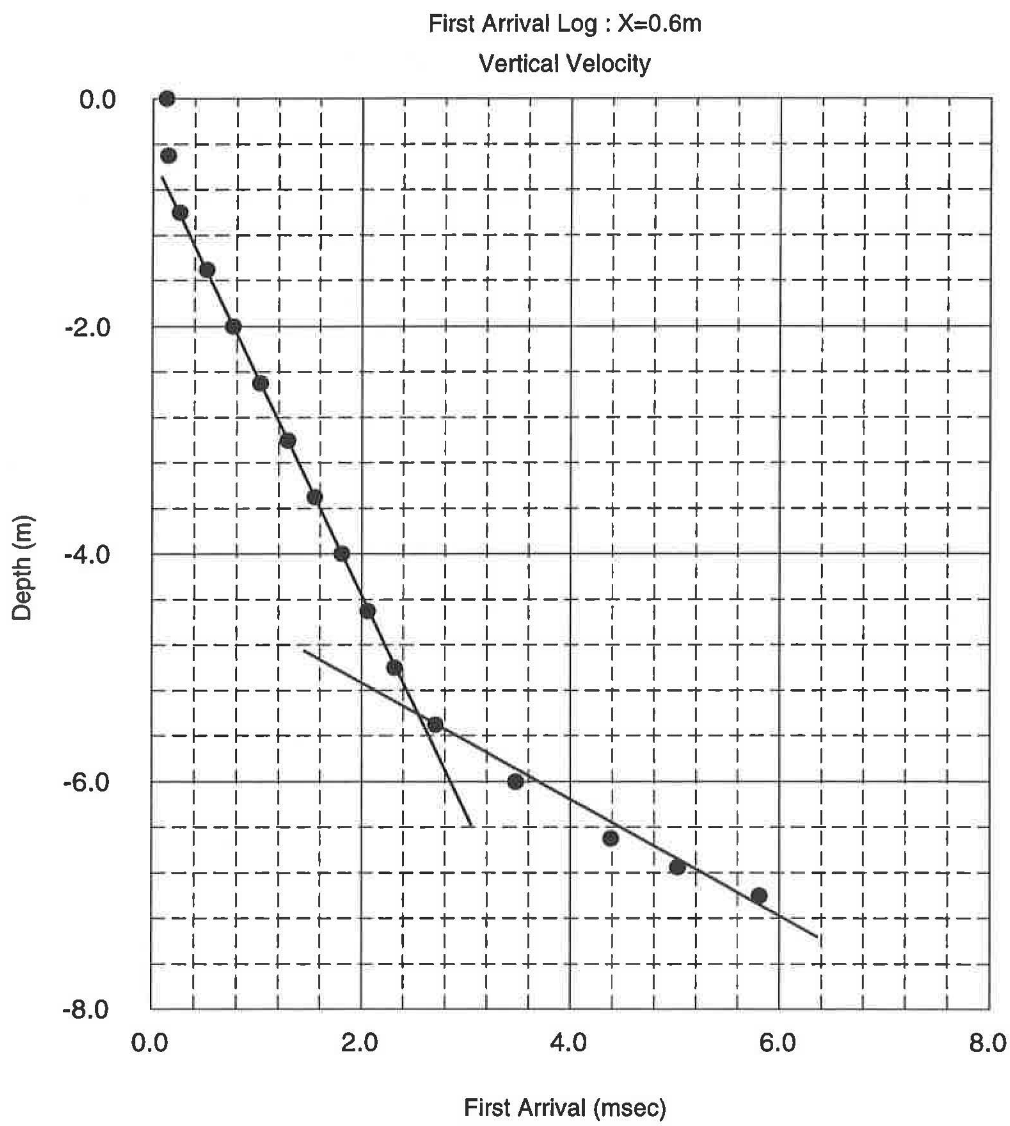


Figure 4.4 : First Arrival Log at X = 0.6m (1.2 pile dia)

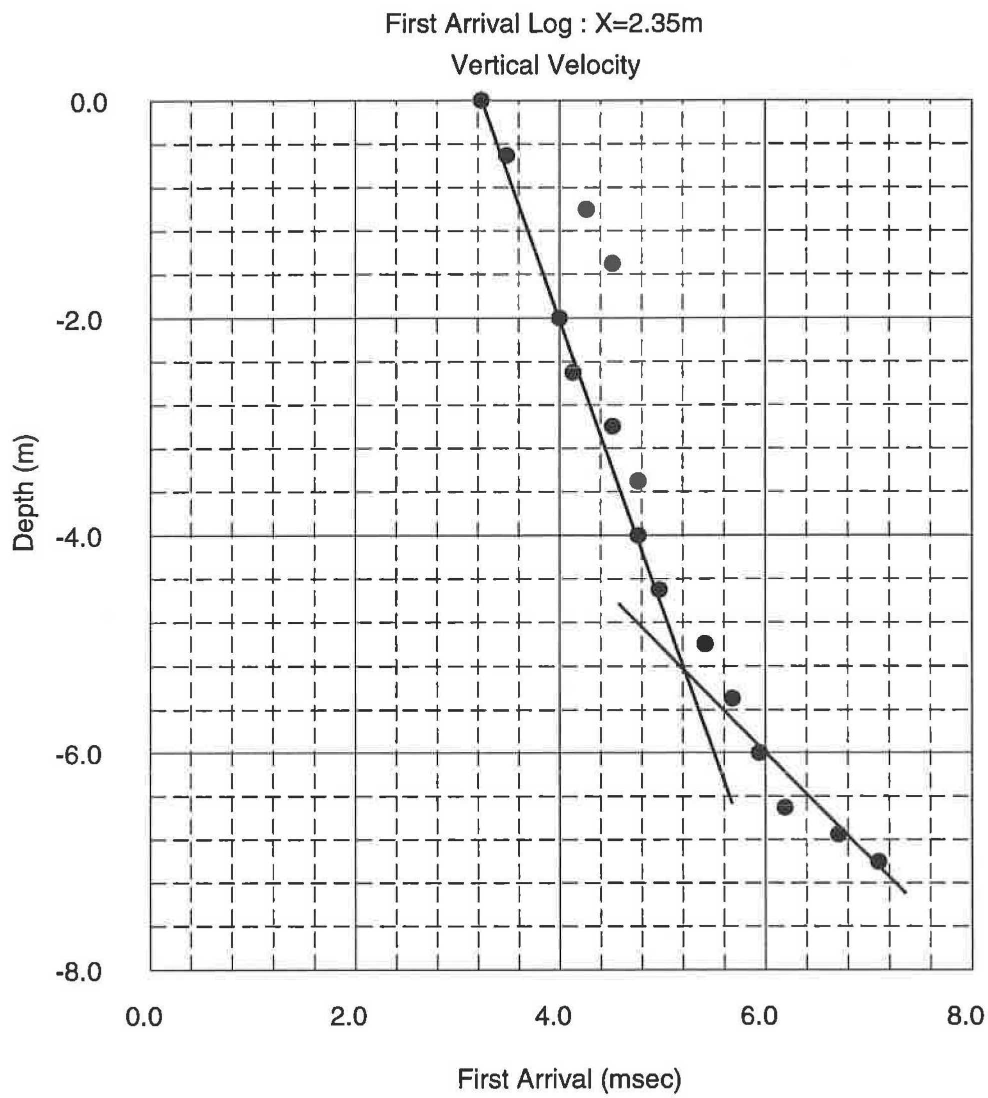


Figure 4.5 : First Arrival Log at X=2.35m (4.7 pile dia)

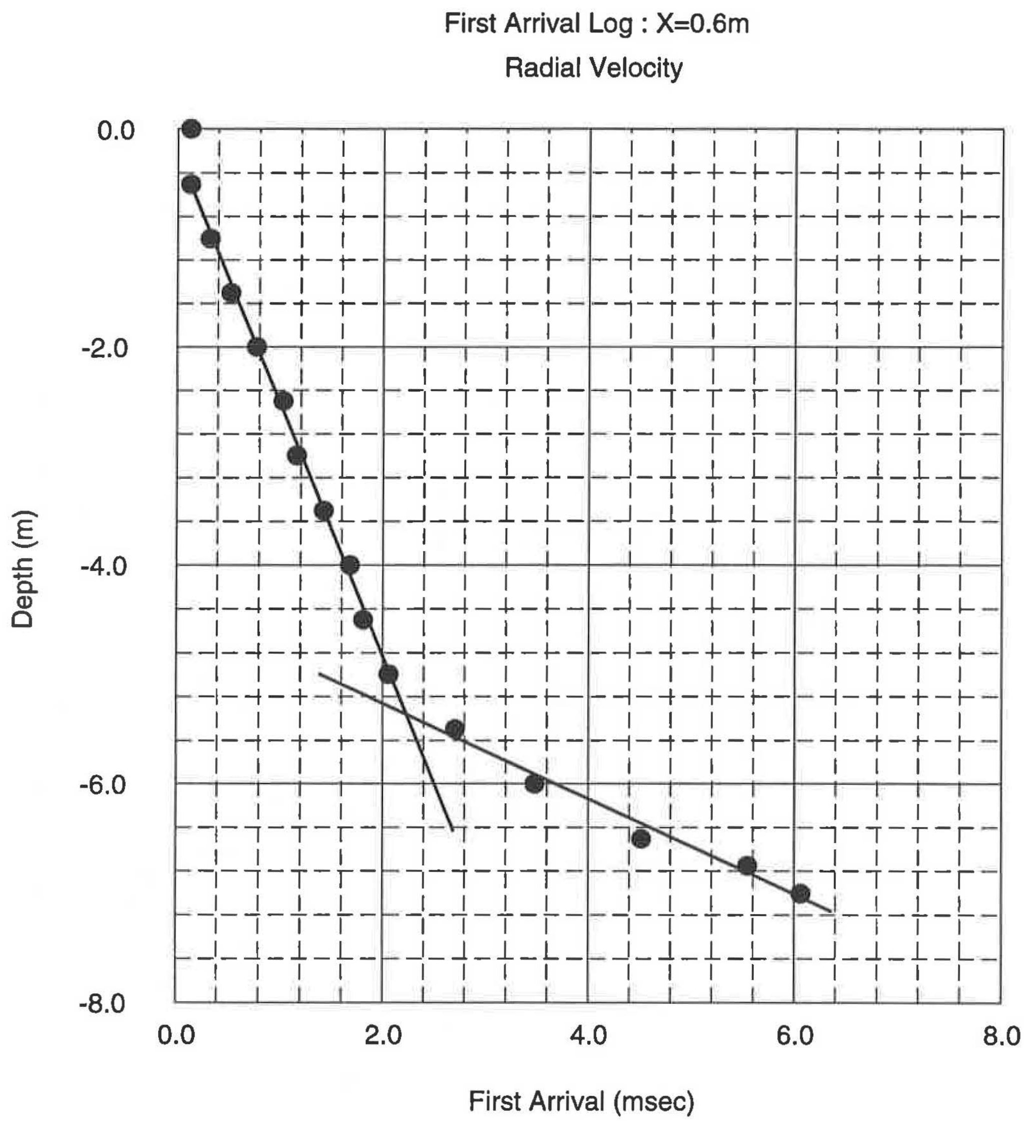


Figure 4.6 : First Arrival Log at X=0.6m (1.2 pile dia)

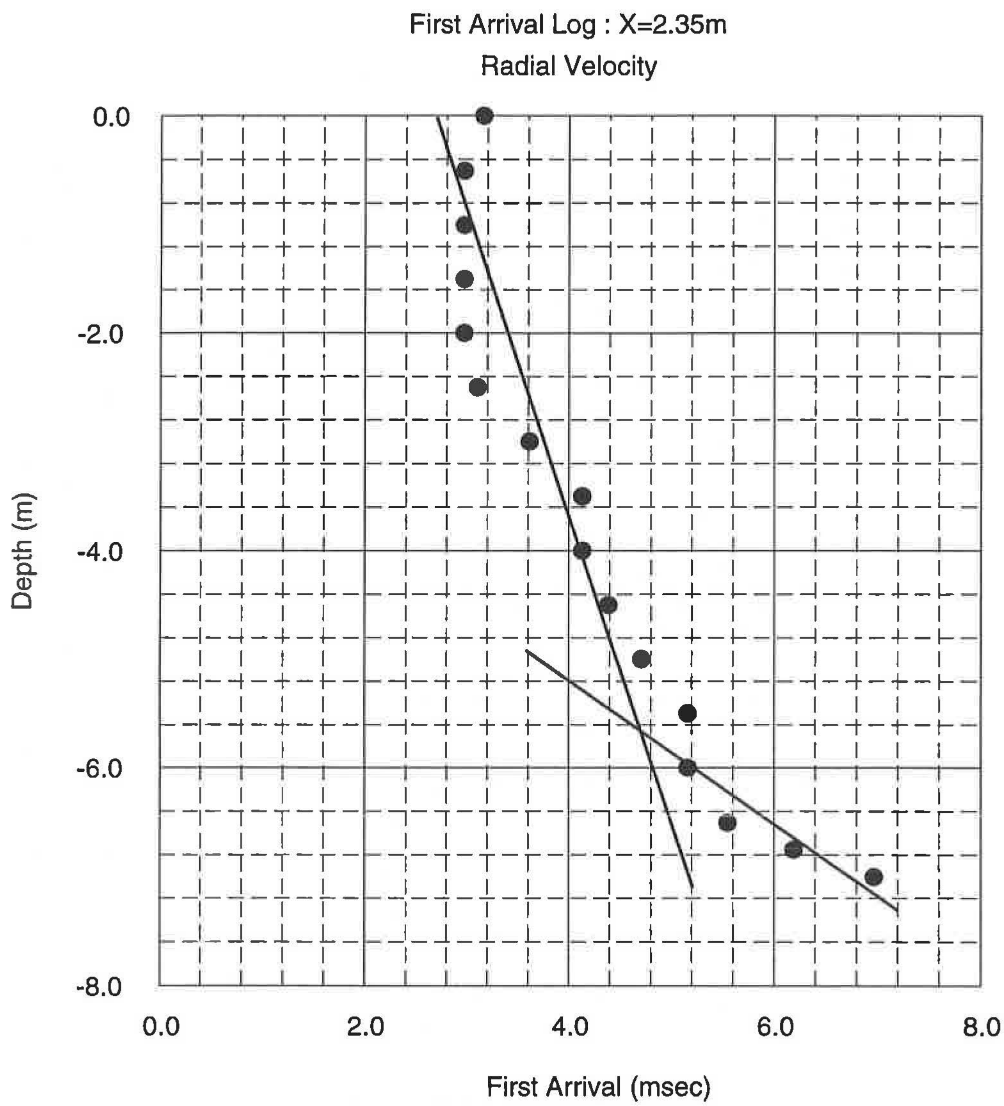


Figure 4.7 : First Arrival Log at X=2.35m (4.7 pile dia)

both the vertical as well as the radial velocity response. As has been mentioned before, these lines correspond to velocities of the P-wave within the pile and the soil material. However, the situation changes drastically when the first arrival log is plotted at a distance of 2.35 m from the pile. First arrival times get harder to determine from the individual velocity time histories. This is because the amplitude of the first arrival energy is significantly reduced, compared to the amplitude of the energy reaching the same point at a later time. Such a drop in the amplitude of the first arrival energy might be due to the inherently dispersive nature of wave propagation, wherein waveforms with different frequency components propagate with slightly different velocities. Again, it should be remembered that for any such wave propagation problem the intensity (energy crossing unit cross sectional area per unit time) of the wave diminishes as the square of the distance from the source, whereas amplitude decreases linearly with distance. At any rate, the data points within the first arrival plot are much more scattered and the task of fitting the required straight lines to this data proves much more difficult. Predictions for depth to the pile tip might vary over a wide range depending upon the personal interpretation of the investigator. The lesson here appears to be that the parallel seismic test is best carried out using receivers placed in boreholes located as close to the pile as is practically possible. Also, the pile length predicted by both the horizontally aligned and the vertically aligned receiver logs is 5.25 m (for X=0.6m), which overestimates the actual length by about 5%.

4.4.2 Nature of Energy Transmitted to Receiver

The form of the velocity response recorded at short radial distances from the pile is seen to be rich in high frequency energy, which quickly attenuates to a much smoother waveform as distance from the pile is increased. This is also the case as one moves deeper below the pile tip. Such a phenomenon is observed due to the fact that the earth acts as a low pass filter, and rates of attenuation are much higher for waves with higher frequencies than corresponding waves with lower frequencies.

Thus we see that our somewhat simplified finite element model is capable of reproducing the attenuation behavior encountered during the course of a real test. At depths smaller than the pile length, the component of the P-wave in the vertical direction is seen to be much weaker than the corresponding component along the horizontal direction. The angle at which the refracted head wave is inclined to the horizontal is given by Snell's Law, and is numerically equal to $\sin^{-1}(448/4000)=6.4°$, where 448 and 4000 are the P-wave velocities in soil and concrete in m/sec respectively. As one progresses below the pile tip however, the direction of wave travel becomes more vertically aligned and the amplitude of the recorded P-wave increases. Correspondingly, the amplitude of the S-wave (which arrives at a slightly later point in time) gets weaker as the observation point is moved downward. For an actual field test, we might thus expect the radially aligned receiver (for the 3 component geophone) to be a better source for first arrival times, for depths which do not exceed the pile length. The vertically aligned receivers should produce stronger first arrivals once the pile tip has been crossed.

5. Summary of Recommended Procedures

This study was conducted in order to come up with procedures that can be used in the field to determine the unknown embedded lengths of pile foundations that support existing bridges. To a large part, the study did succeed in achieving its stated objectives:

- it identified four methods that can be used to test for unknown foundation lengths.
- it suggested changes to both the testing and the data analysis procedures for the four techniques mentioned above, which lead to increased accuracy and confidence in the resulting predictions.

Based upon the results of this study the research team has come up with the following recommendations for those attempting to solve the problem of unknown foundation lengths:

1. The first step would be to conduct microseismic tests on the foundation elements. For most bridges supported by "short" piles, this should be sufficient to draw a reasonable inference as to embedded lengths. The definition of a "short" pile is somewhat hazy; it would depend on both the cross sectional area of the pile (guide area available to the stress wave) and distribution of soil resistance along the length of the pile (attenuation of stress wave).
2. For cases where a direct microseismic test does not yield obvious results, several procedures may be used to enhance the information that is available in the recorded signals. (i) The simplest would be to record multiple hits at the same location and average the signals, thus enhancing the repeated patterns (reflections of interest) and reducing/cancelling out some of the noise. (ii) Recording signals at the same receiver locations with reversed polarities. This can be done by hitting the pile horizontally on opposite faces while keeping the receiver fixed at the same location and aligned in the same direction. Once the 2 sets of signals with reversed polarities are superimposed on the same plot, the significant reflection patterns become clearer. (iii) The use of multiple receivers placed at different recording

stations to simultaneously record the same hit and the same waveform as it moves up and down the pile. This increases the coherence from signal to signal. (iv) Good bonding of the receiver to the surface of the pile. In the case of steel piles, this is achieved by using magnetic mounting bases that screw on to the base of the accelerometers.

3. If the results from a microseismic analysis are still inconclusive, parallel seismic tests need to be conducted. With parallel seismic tests it is important to remember that vertical hits produce receiver signals with better defined first arrivals than horizontal hits. First arrivals also tend to be better defined when using geophones as the downhole receivers as compared to hydrophones.

4. The digital signal processing required for the analysis of microseismic and parallel seismic data may involve a significant amount of time and effort. Several trials may be needed in order to come up with the optimum set of processing parameters like filter and gain control settings, in order to obtain the clearest interpretation. Some degree of skill and experience in data interpretation is also required on the part of the engineer analyzing the data.

Acknowledgments

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Appendix A

Digital Signal Processing Using "Seis_Pro"

The program "seis_pro" was developed at Auburn University as part of the current pile length determination project. This program has been designed to digitally process the data that is collected using the RC Electronics data acquisition card. It may be used with either the Microseismic or the Parallel Seismic analysis. This program essentially does the following:

1. read in the user defined control parameters
2. read in the raw input data generated by the data acquisition system
3. if desired, perform digital filtering to enhance certain waveforms
4. if desired, integrate the time domain signal from acceleration to velocity for the microseismic technique including correcting for baseline drift
5. scale each individual record according to user defined parameters
6. stack the processed data for viewing or interpretation, either on a single plot, or on multiple plots.

The program is currently installed to run on a PC type computer running Microsoft Windows (or Windows 95). The program requires a large amount of memory and will perform fastest with at least 16 Megs of RAM. The program has also been designed to be portable (ANSI standard), so it can be installed on any other type of system with relatively minor changes. A listing of the source code of the main program is provided in the following pages. Some applications may require the use of other specific preprocessors, which have not been listed here.

```
*************************************************************************
*                    SEIS_PRO
*                    ********
*            PROGRAM AUTHOR : Sanjoy Chakraborty
*            DEVELOPED AT : Auburn University, AL
*************************************************************************
*     PROGRAM TO PROCESS THE SEISMIC DATA GENERATED FROM THE
*     FOLLOWING TESTS :
*     1. PARALLEL SEISMIC
*     2. MICROSEISMIC
*     3. TRANSILLUMINATION PROFILING
*************************************************************************
*     THIS PROGRAM USES THE ASCII VERSIONS OF THE DATA FILES
*     GENERATED BY THE ISC-67 OSCILLOSCOPE SOFTWARE
*************************************************************************
*                 INPUT FORMAT
*     ALL INPUT IS IN FREE FORMAT FROM AN INPUT FILE WHICH HAS
*     TO BE SPECIFIED AT THE TIME OF PROGRAM EXECUTION
*
* 1 CARD 1 : nfile1,nfile2,incr,nchanl,const
*    nfile1, nfile2 = starting and ending data file numbers
*                 (AA5 to AA45)  5 and 45
*    incr   = increment for file numbering (aa5,aa6,aa7 = 1)
*    nchanl = number of channels to process (may be less than or
*           equal to the total number of channels recorded)
*    const  = constant term for depth adjustment
*           depth = nfile(i) + const
*
* 2 CARD 2 : fchar
*    fchar = character string describing data files
*         eg. ='aa'
*
* 3 CARD 3 : ndata,ndrop,integrat,sclamp
*    ndata = number of data points in processed output per channel
```

```
*    ndrop = number of intermediate data points to drop for output
*    integrat = 1  for integration from accln to velocity
*             = any other value for NO integration
*    sclamp = for (sclamp.>.1.0) amplify exponentially from
*             amplification factor = 1.0 at time=0.0
*                                  = sclamp at time=T (total processed time)
*
* 4 CARD 4 : npoint,delt,sclmax,nsmooth
*    npoint = total number of data points per channel (original)
*    delt   = original sampling period
*    sclmax = scaling factor for waterfall display
*      peak value in each trace = sclmax*incr
*    nsmooth = number of passes of low-pass Hanning Filter to be
*              applied to each original data channel
*
* 5 CARD 5 : nch(i),i=1,4
*    nch(i) = 1 for processing channel number i
*           = 0 for ignoring channel number i
*
* 6 CARD 6 : fout1
*    fout1 = name of output file for processed channel 1 data
*
* 7 CARD 7 : fout2
*    fout2 = name of output file for processed channel 2 data
*
* 8 CARD 8 : fout3
*    fout3 = name of output file for processed channel 3 data
*
* 9 CARD 9 : fout4
*    fout4 = name of output file for processed channel 4 data
*
***********************************************************************
*     DUE TO MEMORY REQUIREMENTS, THIS PROGRAM HAS TO BE COMPILED
*     TO RUN UNDER MicroSoft Windows (TM). DEPENDING UPON THE PROBLEM
*     SIZE, THE DIMENSIONS OF THE ARRAYS v, y, and area WILL NEED TO
```

```
*     BE ADJUSTED. COMPILE USING (MicroSoft Fortran Compiler 5.0 or later)
*
*     fl -AH -MW seis_asc.for /link /SEG:1000
***********************************************************************

      program seis_asc
      implicit real *8 (a-h,o-z)
      character *12 fout1,fout2,fout3,fout4,fin1,ftemp1
      character *1 fchar
      dimension v(16385,4),y(4000,4,32),area(16385),nch(4)

***********************************************************************
*     OPEN INPUT FILE
*     READ OPERATION PARAMETERS
***********************************************************************

24    write(*,'(///a\)')' Input File : '
      read(*,'(a)')fin1
      open(1,file=fin1,status='old',err=21)
      goto 22
21    write(*,23)fin1
23    format(//,' ** ERROR **',//,' Input File :',a12,
     1 ' Not Found',//)
      goto 24

22    continue
      read(1,*)nfile1,nfile2,incr,nchanl,const
      read(1,'(a1)')fchar
      read(1,*)ndata,ndrop,integrat,sclamp
      read(1,*)npoint,delt,sclmax,nsmooth
      read(1,*)(nch(i),i=1,nchanl)
      if(nch(1).eq.1)read(1,'(a12)')fout1
      if(nch(2).eq.1)read(1,'(a12)')fout2
      if(nch(3).eq.1)read(1,'(a12)')fout3
      if(nch(4).eq.1)read(1,'(a12)')fout4
```

```
************************************************************************
*     INITIAL CALCULATIONS
************************************************************************

10     delt1=float(ndrop+1)*delt
        tim_tot=float(ndata-1)*delt1
        nactual=1+(ndata-1)*(ndrop+1)
        afac=alog(sclamp)*delt/tim_tot

        if(nactual.gt.npoint)then
          write(*,11)npoint,ndata,ndrop,nactual
11        format(//,' *** ERROR ***',//,
     1   ' No. of data points per channel in input (npoint) =',i6,/,
     2   ' No. of data points to be written (ndata)         =',i6,/,
     3   ' No. of intermediate points to be dropped (ndrop) =',i6,/,
     4   ' No. of data points required per channel (nactual)=',i6,//,
     5   ' Try again with a reduced value of ndata/ndrop !!',//,
     6   ' New value of ndata : ',\)
          read(*,*)ndata
          write(*,'(/a\)')' New value of ndrop : '
          read(*,*)ndrop
          goto 10
        endif

        open(12,file='fname.tmp',status='unknown')

************************************************************************
*     ic1=counter for total number of depths at which
*        receiver was placed
*     start of loop for processing each input file in order of depth
************************************************************************

        ic1=1
        do 101 ii=nfile1,nfile2,incr
```

```
      if(ii.lt.10)then
        write(12,102)fchar,ii
102     format(a1,i1,'.asc')
      else
        write(12,103)fchar,ii
103     format(a1,i2,'.asc')
      endif

      rewind(12)
      read(12,'(a12)')ftemp1
      write(*,118)ftemp1
118   format(/,3x,a12)
      rewind(12)
      open(2,file=ftemp1,blocksize=40000,status='old')

      do 105 k=1,5
        read(2,*)
105   continue


      do 110 i=1,npoint
        read(2,*)(v(i,j),j=1,nchanl)
110   continue

      close(2)

*************************************************************************
*     j = counter for each channel within the ic1'th data file
*     START OF LOOP FOR CHANNEL BY CHANNEL PROCESSING
*     PROCESS CHANNEL ONLY IF nch(j)=1
*************************************************************************

      do 115 j=1,nchanl
        if(nch(j).ne.1)goto 115
```

```
      write(*,*)j

************************************************************************
*     FILTER OPTION - HANNING WINDOW
************************************************************************

      if(nsmooth.lt.1)goto 303
      do 302 ipass=1,nsmooth
      do 301 i=2,(npoint-1)
        v(i,j)=0.25*(v(i-1,j)+v(i+1,j))+0.5*v(i,j)
301   continue
302   continue
303   continue

************************************************************************
*     INTEGRATION OF ACCELERATION TO VELOCITY
************************************************************************

      if(integrat.ne.1)goto 252

      area(1)=0.0
      do 250 i=2,npoint
        area(i)=delt/2.*(v(i-1,j)+v(i,j)) + area(i-1)
250   continue

      sum1=0.0
      sum2=0.0
      sum3=0.0
      do 255 i=1,npoint-1
        tval=(float(i-1)+0.5)*delt
        t255=0.5*(area(i)+area(i+1))
        sum1=sum1+t255*tval*delt
        sum2=sum2+t255*tval**2*delt
        sum3=sum3+t255*tval**3*delt
255   continue
```

```
      ttot=float(npoint-1)*delt
      c0=-300.*sum1/ttot**3 +900.*sum2/ttot**4 -630.*sum3/ttot**5
      c1=1800.*sum1/ttot**4 -5760.*sum2/ttot**5
     1  +4200.*sum3/ttot**6
      c2=-1890.*sum1/ttot**5 +6300.*sum2/ttot**6
     1  -4725.*sum3/ttot**7

      do 256 i=1,npoint
       tval=float(i-1)*delt
       v(i,j)=v(i,j)+c0+c1*tval+c2*tval**2
256   continue

      area(1)=0.0
      do 257 i=2,npoint
       area(i)=delt/2.*(v(i-1,j)+v(i,j)) + area(i-1)
257   continue

      do 251 i=1,npoint
       v(i,j)=area(i)
251   continue
252   continue

**********************************************************************
*     EXPONENTIAL AMPLIFICATION OF VELOCITY RECORDS
**********************************************************************

      if(sclamp.le.1.0)goto 401
       do 402 i=1,nactual
                v(i,j)=v(i,j)*exp(afac*float(i-1))
402    continue
401   continue

**********************************************************************
*     SCALE ACCELERATION/VELOCITY/PRESSURE TRACES FOR
*     WATERFALL DISPLAY
```

```
************************************************************************

          pval=abs(v(1,j))
          do 120 i=2,nactual
            if(abs(v(i,j)).gt.pval)pval=abs(v(i,j))
120       continue
          do 121 i=1,nactual
            v(i,j)=v(i,j)/pval*sclmax*float(incr)
            v(i,j)=v(i,j)+float(ii)+const
121       continue

************************************************************************
*     DROP ndrop NUMBER OF INTERMEDIATE POINTS FOR REDUCED
*     DISPLAY STORAGE
************************************************************************

          ic=1
          do 130 i=1,nactual,(ndrop+1)
            y(ic,j,ic1)=v(i,j)
            ic=ic+1
130       continue

115     continue

          ic1=ic1+1

101    continue

        ic1=ic1-1

************************************************************************
*     END OF PROCESSING LOOP (EACH DATA FILE)
************************************************************************
*     OPEN REQUIRED OUTPUT FILES
*     WRITE PROCESSED DATA FOR DISPLAY AT REQUESTED TIME INTERVALS
```

```
****************************************************************************

      write(*,'(///a///)')' Printing Processed Output.....'

      time=0.0

      if(nch(1).eq.1)open(6,file=fout1,status='unknown')
      if(nch(2).eq.1)open(7,file=fout2,status='unknown')
      if(nch(3).eq.1)open(8,file=fout3,status='unknown')
      if(nch(4).eq.1)open(9,file=fout4,status='unknown')

      do 201 i=1,ndata
       if(nch(1).eq.1)write(6,203)time,(y(i,1,k),k=1,ic1)
       if(nch(2).eq.1)write(7,203)time,(y(i,2,k),k=1,ic1)
       if(nch(3).eq.1)write(8,203)time,(y(i,3,k),k=1,ic1)
       if(nch(4).eq.1)write(9,203)time,(y(i,4,k),k=1,ic1)
       time=time+delt1
203    format(f8.0,80f8.3)
201   continue

      close(12,status='delete')

      stop
      end
```