

Florida Department of Transportation

Title

**EFFECTS OF VIBRATION AND SOUND DURING THE
INSTALLATION OF DEEP FOUNDATIONS**

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EXECUTIVE SUMMARY

This project addresses two frequent construction problems associated with deep foundation installation: i) The effect of vibrations induced during the installation of drilled shafts, on "green concrete", defined as freshly placed and maturing concrete within 24 hours after initial placement, and ii) Noise levels during pile driving due to the low threshold of human perception.

The laboratory investigation addressed the determination of the damage threshold particle velocity by shake table vibration testing of concrete cylinders at predetermined time delays (0,1,2,3,4, and 24 hours), ultrasonic testing for compression moduli before compressive strength testing, and testing of cored cylinders from full scale drilled shafts. The principal finding from laboratory testing was that construction vibrations that produce particle velocities less than 8 in./sec would not have any effect on the compressive strength or compression modulus of the concrete aged between the time of placement to the first 24 hour.

In the field investigation, the peak particle velocities during drilled shaft were monitored to determine their effect on "green concrete". The principal findings from the field study are as follows: i) Vibrations, with peak particle velocities of upto 2.5 in./sec do not cause damage to the "green concrete" at a distance of two times the shaft diameter and beyond. ii) In general, a spacing of three times the shaft diameter would be a safe specification to ensure no concrete damage due to shaft vibration.

In Part 2, the propagation of noise generated from the driving of concrete piles was measured and the effect of mitigation through the use of an acoustic curtain evaluated. This project focused on the use of an acoustic curtain constructed of a combination of absorptive and reflective materials to provide a complete enclosure of the source to mitigate the propagation of the inherent noise. In the field, a full-scale experiment was conducted by evaluating noise levels generated from the installation of concrete piles, with and without the aid of the acoustic curtain, a reinforced polyvinyl chloride outer shell for added mass, with a specially developed inner lining of 2in. thick fiber glass acoustic insulation (1.5 lb/ft²). The results indicated that noise levels were reduced by up to 10dB, with the curtain enclosing the diesel hammer. Further research is recommended.

SUMMARY

This project addresses one of the frequent construction problems associated with deep foundation installation. The primary concern is the effect of vibrations on “green” concrete, where, “green” concrete as defined in this paper refers to freshly placed and maturing concrete within 24 hours after initial placement. And the noise levels during pile driving due to the low threshold of human perception.

The project is divided into two parts. Part 1 addresses the " Effects of Vibrations Induced During the Installation of Drilled Shafts" and Part 2, the “ Effects of Sound Induced During the Installation of Piles”. Part 1 comprises Field and Laboratory Investigations. The field work was carried out by Florida Atlantic University, FAU laboratory at four sites-two preliminary tests at FAU, one in Delray Beach, FL, and the other in Clearwater, FL. The laboratory investigation was done by Florida State University, FSU. Part 2, a minor component of the research project, was mainly a field investigation with a component of laboratory work, all of which was carried out by FAU.

In Part 1, a review of the characteristics of construction vibration is followed by a laboratory investigation to determine the peak particle velocity by shake table vibration testing of concrete cylinders at predetermined time delays (0,1,2,3,4, and 24 hours) and their compressive strength testing at 3, 7, 14, and 28 days. The input signals were based on field acceleration records transformed from the time domain to the frequency domain by using the Fast Fourier Transform(FFT). The measurements also included ultrasonic testing for compression moduli before compressive strength testing. Core samples from full-scale drilled shafts were also tested in the laboratory. The field investigation addressed the monitoring of drilled shaft installation-induced vibrations (peak particle

velocities) in the ground and their effect on "green concrete". Sets of two shafts 24" (unreinforced) and 36" (reinforced) were installed. The purpose of the smaller shaft was to determine the effect on the integrity and strength of a shaft within a three diameter distance from the vibrating source. The instrumentation consisted of subsurface triaxial accelerometers attached to the steel casing, surface accelerometers in the ground, and geophones at varying depths in a circular array around the vibrating steel casing. Both control concrete and cored concrete cylinders were tested for compressive strength. The Pile Integrity Testing (PIT)- a combination of the Pulse Echo Method (PEM) and the Transient Response Method (TRM), and Cross-Hole Sonic Logging (CSL) were the Nondestructive Testing Methods used. Additionally, Geophysical Logging was carried out with neutron-neutron (porosity) and gamma-gamma (density) measurements.

Vibrations, with peak particle velocities of up to 2.5in./sec did not cause damage to the "green concrete". The PIT and CSL tests also indicated no damage, although the data at the Clearwater site was incomplete for the upper thirds of the shaft poured. The recommendation is made for using strains, rather than particle velocities, to monitor damage in "green concrete", as the same velocity would produce different strains amplitudes in different materials and in different structures. The principal findings are as follows: 1) a spacing of 3 times the diameter of a shaft, between shafts, seems to be generally conservative, 2) concrete cured upto 24 hours is not damaged due to vibrations due to particle velocities upto 2.5 in/sec.

In Part 2, the propagation of noise generated from the driving of concrete piles was measured and the effect of mitigation though the use of an acoustic curtain evaluated. This project focused on the use of an acoustic curtain constructed of a

evaluated. This project focused on the use of an acoustic curtain constructed of a combination of absorptive and reflective materials to provide a complete enclosure of the source to mitigate the propagation of the inherent noise.

Tests were conducted in a laboratory chamber, with a sample acoustic curtain, indicated noise level reductions more than 16 dB, identifying the strong need for field testing. In the field, a full-scale experiment was conducted by evaluating noise levels generated from the installation of concrete piles, with and without the aid of the acoustic curtain, a reinforced polyvinyl chloride outer shell for added mass with a specially developed inner lining of 2in. thick fiber glass acoustical insulation (1.5 lb/ft²). The results indicated that noise levels were reduced by up to 10dB, with the curtain enclosing the diesel hammer. Further research is recommended.

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PART 1

**EFFECTS OF VIBRATION INDUCED
DURING THE INSTALLATION
OF DRILLED SHAFTS**

CHAPTER I

INTRODUCTION

1.1 STATEMENT OF THE PROBLEM

Construction of drilled shafts often induces ground vibrations with varying intensities. The generation of the vibration is usually caused by the process of driving the exterior casings needed to stabilize the boreholes, in addition to the other construction related vibrations. The casing method of construction is very common in Florida, especially for deep foundations in waterways. The casing method in drilled shaft construction may be used on a temporary or permanent basis. Specifications of the Florida Department of Transportation (FDOT) require that all drilled shaft casings be removed except for those intended to be permanently placed in the boreholes (*FDOT Specifications 455-15.4 and 455-15.5*). If the permanent casing method is specified for certain site conditions, then the deeper shafts have to be constructed to compensate for the less of skin friction due to the presence of the casings. Another situation, where casings may become permanent, occurs due to the difficulty of withdrawing the casing from the excavations. Such situations should be assessed by the project engineer, and the contractors are usually compensated for the extra cost. On the other hand, temporary casings are removed during concrete placement in the shaft, when the concrete cures a workable condition.

Current specifications provide regulatory procedures for protection of existing structures from the drilled shaft construction induced vibrations. Under article *455-1.1*, structures within a distance of ten shaft diameters or the estimated shaft depth, whichever is greater, should be monitored for settlement, and for possible development of

structural cracking. Existing footings within a distance of three times the depth of the excavation, should also be monitored. The vibration monitoring equipment should be capable of detecting velocities of 2.5 mm/sec (0.1 in./sec) or less. It is mandatory that the source of vibrations cease immediately when structural settlement reaches 1.5 mm (0.06 in.) vibration levels reach 13 mm/sec (0.5 in./sec), or damage to existing structures occurs.

However, the current specifications do not provide acceptable levels of construction induced vibrations for drilled shafts with freshly placed concrete. A waiting period of about 12 to 24 hours may be required before construction proceeds. Although the FDOT specifications do not necessitate such a time span for every project, the delay periods are usually set by the project engineers at the sites. The rationale behind these restrictions is to allow additional curing time for the freshly placed concrete to avoid any possibility of changes in the physical or mechanical properties of the concrete.

Despite the fact that uncontrolled vibrations are usually not allowed during concrete placement, such restrictions have been considered by contractors as subjective and unsubstantiated.

1.2 BACKGROUND

In recent years, Florida has undergone a significant growth in its population. Consequently, the infrastructure of the state's roadways has been expanding to accommodate the traffic load accompanying the population growth. Continuous efforts are now underway to increase the vehicular capabilities of the roads. The greatest challenge posed is in the construction of new bridges or lane widening of existing ones or overpasses.

Some of the Florida DOT construction projects use deep foundations because of the following advantages:

- Very high load carrying capacities.
- Applicability to a wide variety of load conditions.
- Large resistance to lateral and uplift forces.
- Significant resistance to scour.

This research project addresses one of the frequent construction problems associated with deep foundation installation. The primary concern is the effect of vibrations on "green" concrete, where, "green" concrete as defined in this report refers to freshly placed and curing concrete within 24 hours after initial placement. The investigation addresses the effect of the stress waves due to the installation process with respect to their frequency and the peak particle velocity, and on the integrity of adjacent foundations.

1-3 SCOPE OF THE WORK

Several concerns have often been raised by engineers and practitioners regarding the effect of construction induced vibration during drilled shaft installation. These concerns can be listed as follows:

- 1 What is the magnitude of construction vibration that can be generated without**

affecting the properties of a freshly poured shafts?

- 2 What is the minimum allowable distance between the shafts that construction induced vibration would not adversely impact the concrete properties?**
- 3 What is the minimum allowable distance between the shafts that construction induced vibration would not adversely impact the general performance of the shafts?**
- 4 If there is any effect of the construction vibration at one site, is it possible that changing the site conditions may negate or augment these effects?**

The extent of these questions has necessitated the current investigation. The effect of construction -induced vibrations on the overall performance of drilled shafts involves full scale load testing and settlement monitoring setups.

To investigate the effect of the induced vibration on drilled shafts with the freshly placed concrete, it was decided that the work be divided into three parts. These parts are comprised of the following:

Laboratory study of the effect of vibration on the compressive strength and the compression modulus of a fresh concrete. This task was needed to predetermine the thresholds of the velocity amplitudes and to assess the time delay required to avoid any changes in the properties of the concrete.

Full scale field testing to determine the nature of the vibrations generated by a driven steel casing and to record the patterns of the propagated waves as the casing penetrates the ground.

Field investigation of a typical simulated construction procedure of full scale drilled shafts subjected to construction vibrations with different time delays.

Although the source of the vibration in drilled shaft installation is limited to certain waveforms, attempts have been made in this study to collect time-history records directly from a full scale casing driven in the ground and to correlate the generated waves to those propagated in the vicinity of the shafts. Since a search of the literature has not revealed any prior work related to the vibration of drilled shaft casing, it was decided in this study

to attach a special type of shock accelerometer to a steel casing to record the generated signals in the longitudinal direction along the shaft length. The accelerometer was positioned at predetermined locations along the casing and records were collected at various depths.

The above stated tasks were distributed between the two the principal investigators, Florida Atlantic University and Florida State University. The laboratory testing was conducted at the Civil Engineering Department at Florida State University, and the full scale field tests were performed by Florida Atlantic University at sites located at the Boca Raton campus, Delray, Miami, and Clearwater. Florida State University participated in all the field testing activities and carried out all the laboratory and field analysis of results. **Figure 1.1** presents a detailed schematic of the scope of work and all the efforts devoted to accomplish this study.

1-4 REPORT ORGANIZATION

The following chapters present in detail all the tasks involved in this study preceded by a thorough literature review describing relevant subject matters such as: (1) general characteristics of construction vibrations, (2) response of cohesionless soil to construction vibrations, and (3) responses of concrete to vibrations. Findings and conclusions of the study are presented in a separate chapter. Recommendations needed for the Florida Department of Transportation's construction vibration specifications are listed at the end of this report.

1.5 OBJECTIVES AND APPROACH

The objectives of the study were the following:

- i. To monitor the vibration levels during the installation of drilled shafts.
- ii. To monitor the settlement of adjacent structures due to installation.
- iii. To evaluate the effect of vibrations on the cracking associated with the setting of concrete.
- iv. To determine the distance and time at which vibratory effects become insignificant.
- v. To perform maturity testing of the concrete.
- vi. To carry out non-destructive testing, pile integrity testing (PIT), and cross-hole sonic logging (CSL) of the drilled shaft concrete.

Ten drilled shafts were designed and constructed according to the Florida Department of Transportation (FDOT) Specifications 455. The selection of the monitoring equipment and instrumentation was chosen primarily as a function of the portability and the robustness.

The scope of the investigation is presented in a flow chart, Fig. 1.1

**Effect of Vibration
During the installation of Deep Foundations.**

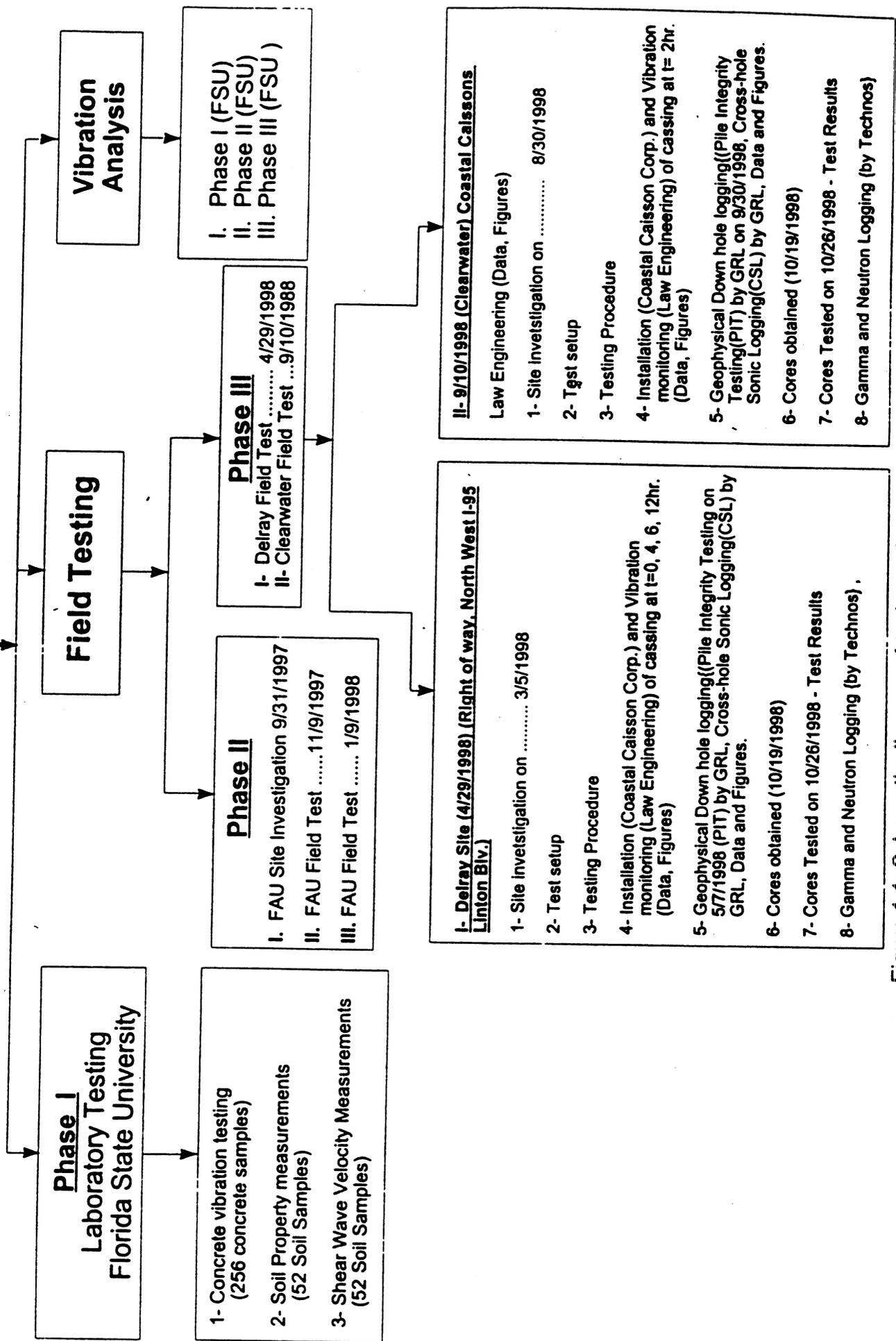


Figure 1.1: Schematic diagram of tasks performed in this study

CHAPTER II

BACKGROUND

2-1 CHARACTERISTICS OF CONSTRUCTION VIBRATIONS

Vibration from drilled shaft construction will generally be temporary, but the disturbance it could cause may result in permanent damage to the surrounding shafts and specifically to the freshly poured ones. This unpredictable damage may lead to restraints on the working method that result in additional costs or even curtailment of activities.

Vibration-induced damage thresholds are usually expressed in terms of peak particle displacement, velocity or acceleration, and sometimes include a frequency-dependent factor. If construction vibrations comprise of harmonic waves, the characteristics of these waves can be described by any two of the following parameters: frequency, peak particle displacement, peak particle velocity, or peak particle acceleration. The peak particle characteristic indicates that it is the maximum value associated with the motion of a particle at a point in the ground (or on structure) that is being considered. Usually the peak particle velocity (**PPV**) is used, because it has been found to be the best correlated with case history data of damage occurrence and because it has a theoretical underpinning in as much as the strain induced in the ground is proportional to the particle velocity. Although **PPV** is widely used to quantify the damaging potential of a vibration, it must be recognized that velocity of itself, cannot induce damaging forces. Such forces are generated in structures by both:

1. *Differential displacements, which give rise to distortion as the structures follows movement of the ground upon which it is found, and*
2. *Change in the ground particle velocity vector (magnitude or direction), which produce internal forces upon the structure.*

In the case of transient vibrations, the peak particle velocity or acceleration would be of a short duration depending on the loading function. Unless the peak value is detected, the prediction of the maximum peaks in random vibrations require statistical analysis which involve the application of the theory of “extrema”. For drilled shaft construction, the generated vibrations are more deterministic harmonic signals than random. Thus, converting motions from acceleration to velocity or displacement can be accomplished through direct integration of the functions. In some cases, however, the generated vibrations do not possess the harmonic waveforms. Such vibrations were recorded in this study and were dealt with based on the assumption of a narrow band stationary Gaussian process. Additional cases were encountered in this study where the maximum values of particle velocities were recorded. Considering such values in deciding about the severance of the vibrations may be questionable because of the short duration of peaks. Guidelines for selecting the maximum particle velocities in such cases do not exist. The common practice is to still consider these peak values.

In practice, any structural element will be subjected to both distortion and internal mechanisms at the same time, and these will be superimposed upon pre-existing stresses and strains from other causes. Damage may occur when the combined effects exceed the tolerance of the structure. In the case of freshly placed concrete in a drilled shaft, construction vibrations may directly or indirectly alter the specified concrete properties. Such changes may occur in the strength of the stiffness properties. Other changes may influence the density or the porosity of the matrix which in turn may change the permeability characteristics and hence the concrete durability.

Construction induced vibrations can be divided into three main types: compressive, *P*, distorsional (shear), *S*, and surface, *R* (Figures 2.1 to 2.3). To describe the vibration response,

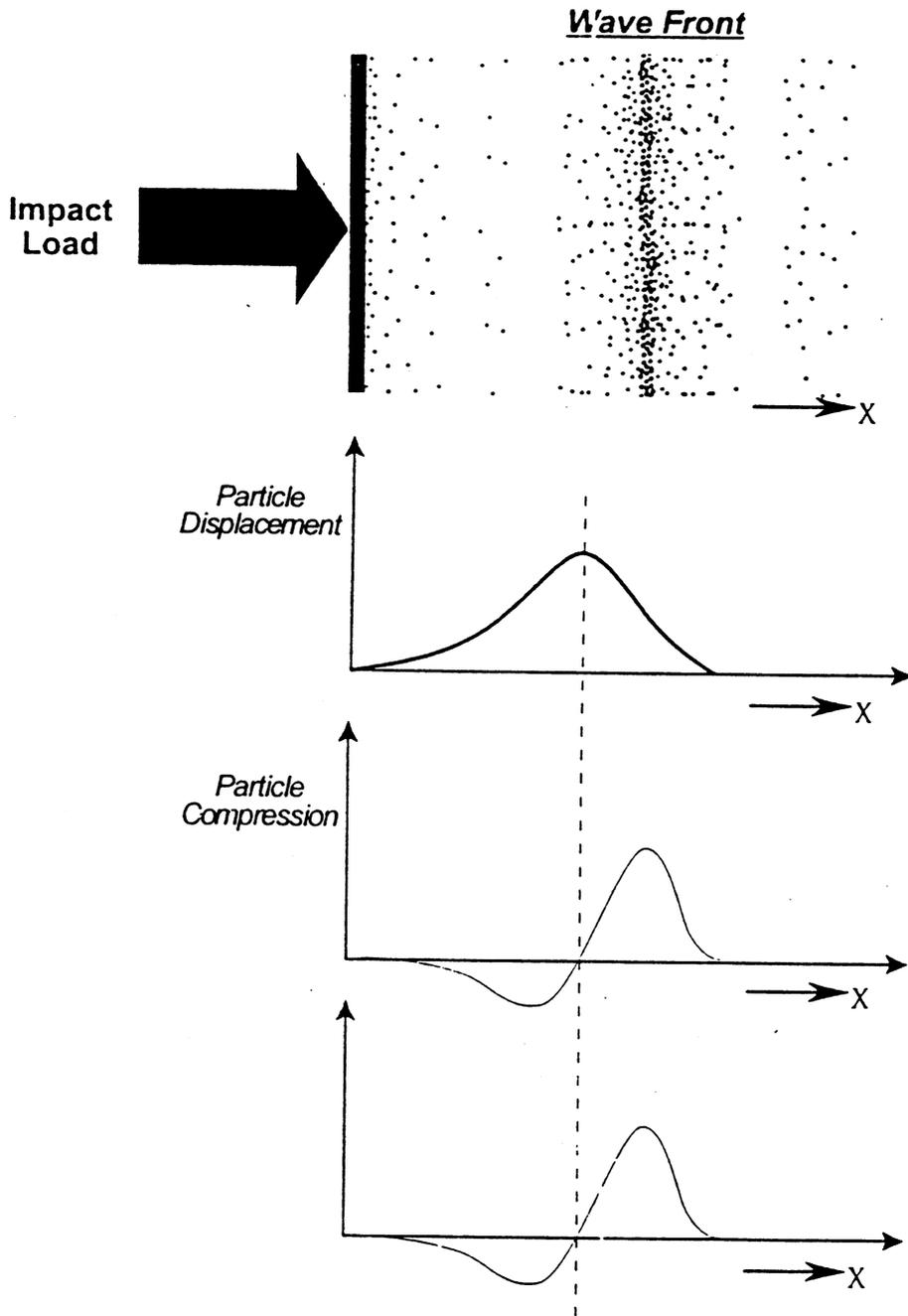


Figure 2.1: Construction vibration generating compression waves.

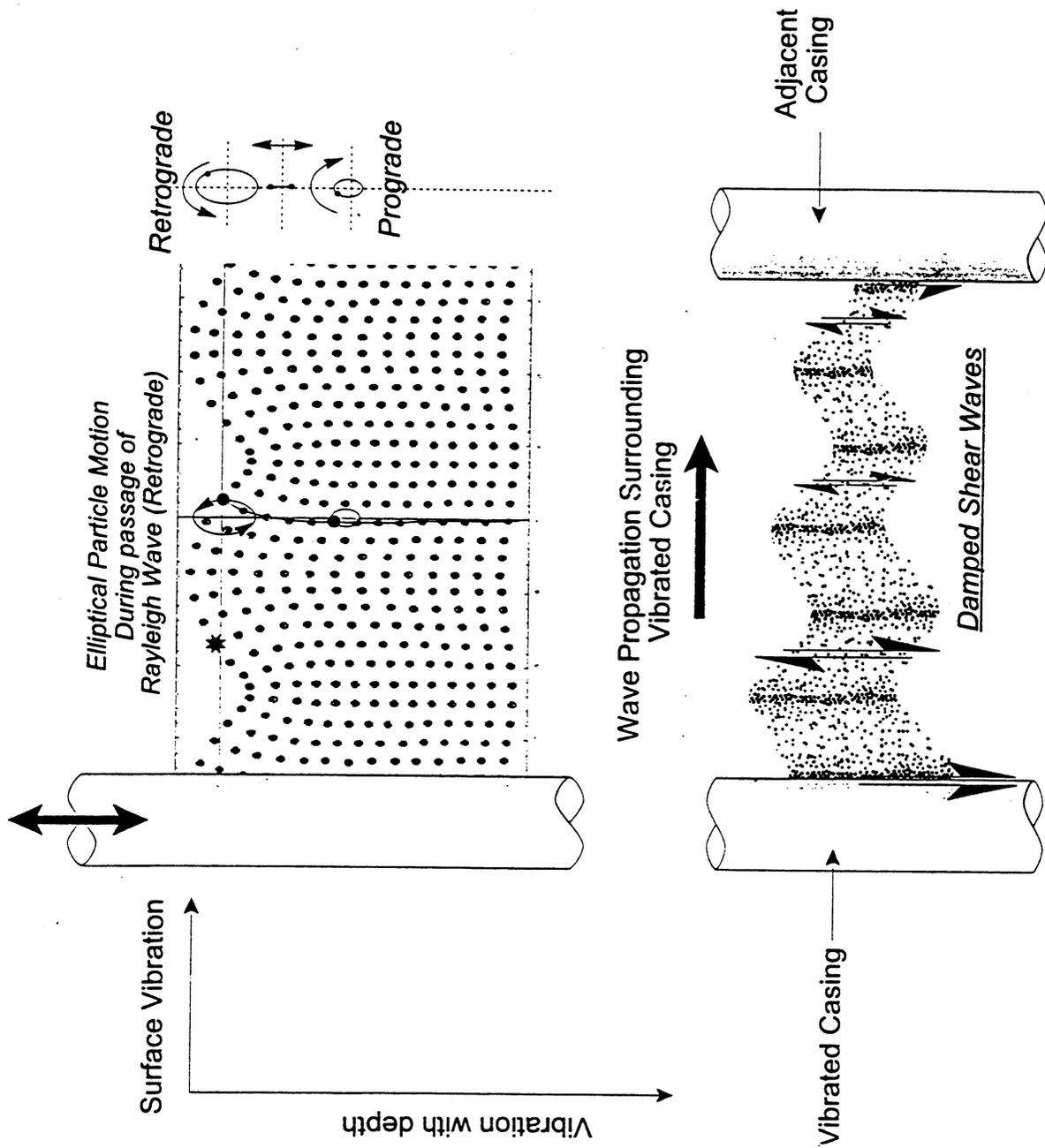


Figure 2.2: Generation and propagation of surface and body shear waves due to a driven casing.

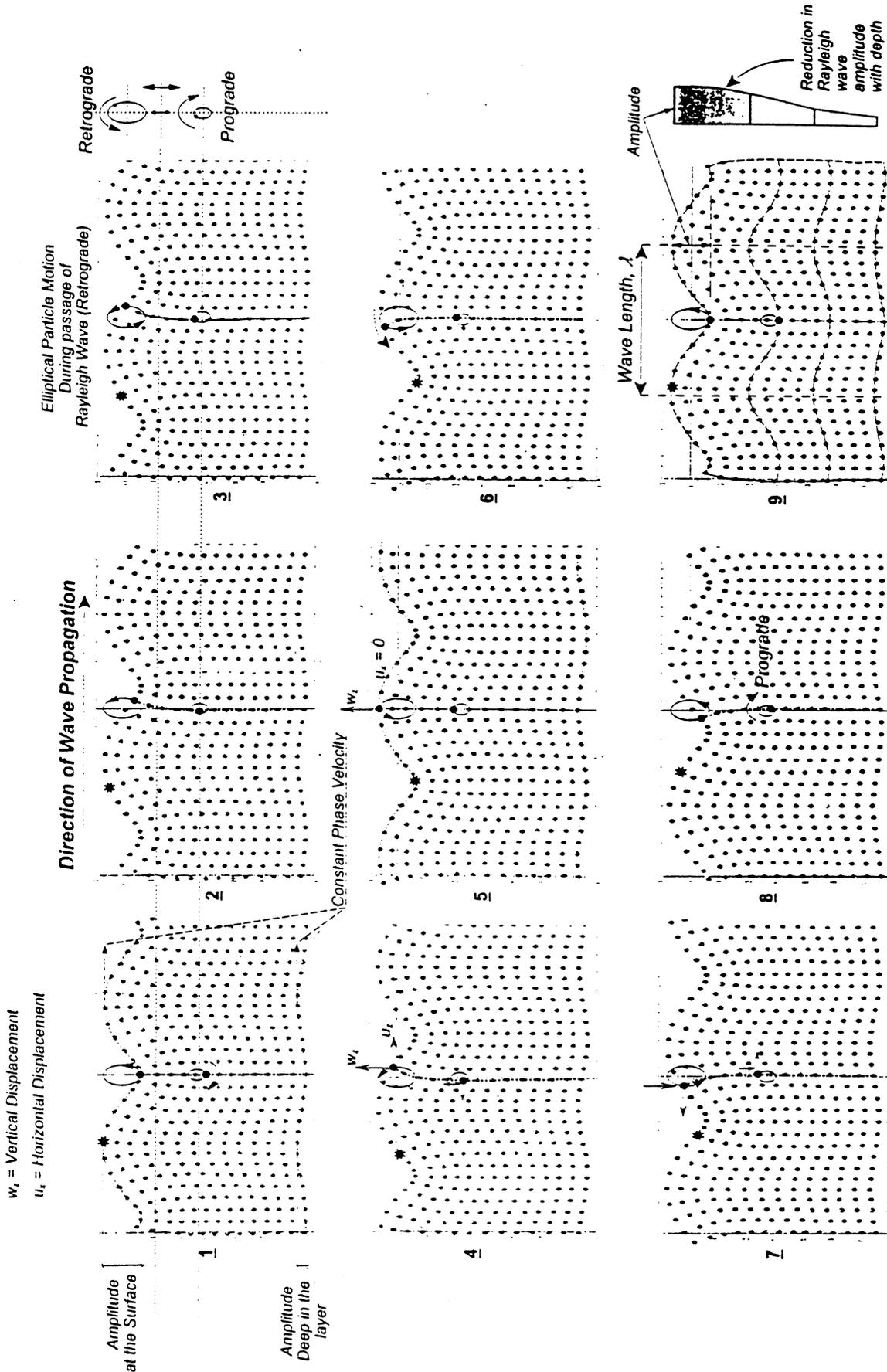


Figure 2.3: Rayleigh waves from construction vibration

three perpendicular components of motions must be measured. The longitudinal component, L is usually oriented along a horizontal radius of the vibration source. It follows, then, that the two perpendicular components will be vertical, V , and transverse, T , to the radial direction (Figure 2.4).

The three main wave types can be encompassed in two different wave categories: body waves which include compressive and shear waves and propagate in the soil, and surface waves, which are transmitted along a surface (usually, the upper ground surface). The surface waves are often described by Rayleigh waves (the most complicated) which produce motions both in the vertical direction and parallel to the direction of propagation. The shear waves in turn are divided into horizontal shear waves, S_H , and vertical shear wave, S_V . The dominance of any mode of vibration, whether it is P , S_H , or S_V , depends on the polarization of the waves and on the source of the vibration.

Transient impact loads below the ground surface produce predominately body waves at small distances. These body waves propagate outward in a spherical manner until they intersect at a boundary such as another layer (rock or soil) or the ground surface. At this intersection, shear and surface waves are produced, and the reflected surface (Rayleigh) waves become important at larger transmission distances. At small distances, all three wave types will arrive together and greatly complicate wave identification, whereas at large distances, the more slowly moving shear and surface waves begin to separate from the compressive wave and allow identification. Most construction impacts produce a single dominant pulse with trailing subsidiary reflections. Drilled shaft construction produces more deterministic harmonic than random vibrations. The vibratory devices that are used to drive the steel casings are operated by hydraulic motors, which means that they have the potential to operate at a wide range of

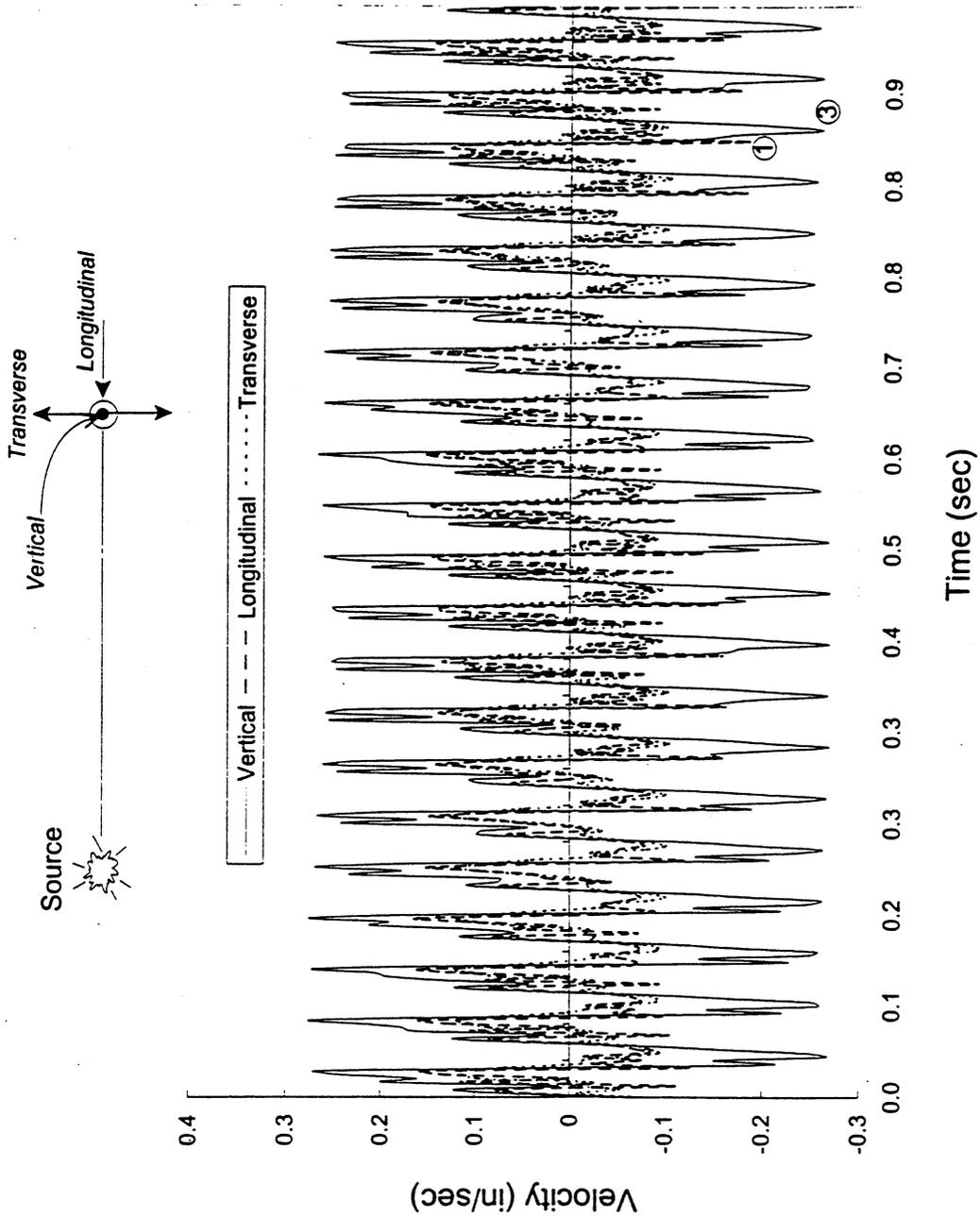


Figure 2.4: Vertical, Longitudinal and Transverse mode of steel casing vibrations

frequencies. As for driven pile, when a steel casing or a pile is driven by a steady-state vibrator, the energy per cycle of vibration may be much less than the energy per blow from an impact load situation. However, it has been observed in practice that the ease or difficulty of vibratory casing driving depends on the characteristics of the soil as well as the characteristics of the casing and the vibratory driver. and is true because two of the three potential resonances that occur during vibratory driving involve soil properties and layer thicknesses. For optimum efficiency, the casing and the soil should not be vibrating "in phase," that is, the casing and the driver must not move together with the surrounding soil, or no penetration occurs.

The following are the three distinct frequencies that affect vibratory casing driving:

1. *Driver-casing resonant frequency, which results in maximum particle velocity at the casing top;*
2. *Soil-casing-driver system resonant frequency, which results in maximum displacement of the soil surrounding the casing; and*
3. *Soil stratum resonant frequency, which is a function of stratum thickness and properties.*

The first is the optimum frequency for driving because the relative motion between the soil and the casing is maximum. The integration of this relative motion gives the casing penetration. The casing is most efficiently driven by vibrations when the combination of mass of driver and casing and the frequency of vibration combine to produce a driver-casing resonance. As long as the frequency of this resonance does not coincide with a resonant frequency of any nearby target or the stratum frequency of the site, large ground vibrations do not occur and damage is unlikely to happen.

Maximum ground vibration amplitude adjacent to the casing during vibratory driving will be encountered when the vibratory casing driver is operating at the resonant frequency of the soil-casing-driver system; this frequency is dependent on properties of the soil stratum that the

casing is penetrating. The amplitude of motion under these conditions will also depend on the force generated by the vibratory driver, the mass of the system, and the stiffness of the soil. However, at this frequency the casing and soil are "in phase or are moving together" and all penetration stops. Also, the ground motion is maximum, exacerbating transmission of vibrations to the neighborhood.

The third resonance involves the soil stratum that the casing is currently penetrating and may change throughout the driving process. At this resonant frequency, the stratum resonates generating large ground motion that very efficiently transmits vibrations throughout the neighborhood. Soil layers or strata will selectively transmit and amplify specific frequencies depending on V_p , V_s , and strata thicknesses H . The resonant frequency of a soil layer can be estimated from Richart et al. (1970) as follows:

$$f = V/4H \quad (1)$$

where f = frequency; V = seismic velocity of layer, P -wave. or S -wave depending on motion observed; and H = layer thickness.

A potential hazard with vibratory drivers exists in the form of matching the frequency of a soil layer. Vibrators often operate in the range of 20 to 30 Hz. For soils with shear wave velocities of 120 to 600 m/sec, this frequency range may spell a hazard for layers between about 1 and 5 m thick, not uncommon in nature. This phenomenon points out the advantage of providing a vibratory hammer with completely variable frequency, and hammers like this are available. Not only can one optimize the driving by adjusting the frequency, but also minimize damage due to accidental resonance of the ground itself. A similar situation may arise for impact-driven casings, but the opportunity for resonance is much less because the impact is not

associated with only a single frequency; and only a few cycles of any given frequency occur so resonance does not develop.

Ideally, it is possible to compute all resonant frequencies, but for real field conditions, it is not realistic to do so. If one were to use a completely frequency-variable vibrator, all frequencies could be determined in situ by starting the vibrator at zero frequency and increasing frequency until all resonances have been identified by observing the amplitude of casing motion and ground motion near (within 1 m) the casing. Usually the stratum frequency will be lower than the system frequency, which is lower than the driver-casing frequency. The stratum frequency would be encountered first in the run-up of frequency from zero, and, if the force amplitude associated with the vibrator is great, large ground motion may be generated and transmitted away from the casing. The effect is similar for the pile-driver system frequency. It would be better if the force amplitude of the vibrator could be varied so that stratum and soil-casing-driver resonances could be approached and passed with low power or force. Then, when the casing-driver system frequency is found, the force level could be increased to optimize driving. This kind of vibrator now exists.

Not only can the state-of-the-art vibrators vary the frequency of operation, but they can vary the force amplitude by changing the static moment. A vibratory driver, that can do this, is shown schematically in **Figure 2.5** (Massarsch and Westerberg, 1995). The two sets of eccentric masses are arranged in two rows. By changing the orientation of the eccentric masses of one row, compared to the other row, the static moment can be changed, thereby changing the force at all frequencies. With this type of vibrator, the first two resonances associated with vibratory driving can be passed through with low static moment and then, when casing-driver resonance is reached, the static moment can be increased, therefore, increasing the force level to

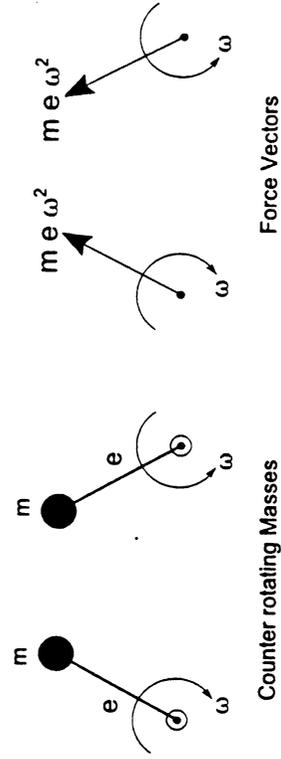
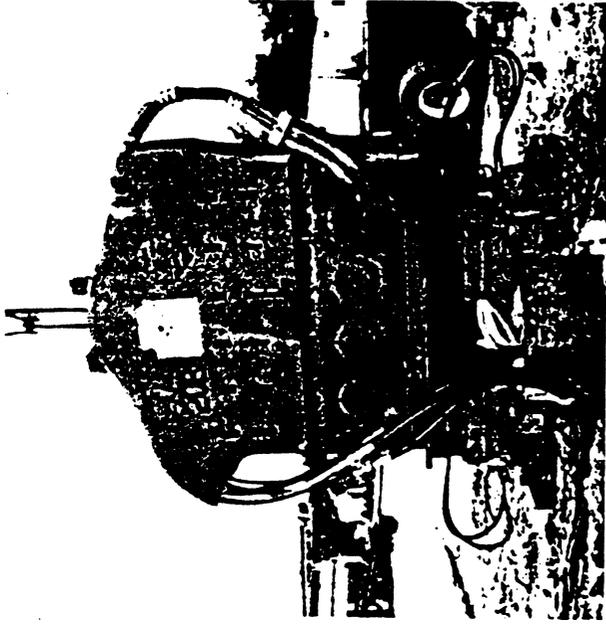
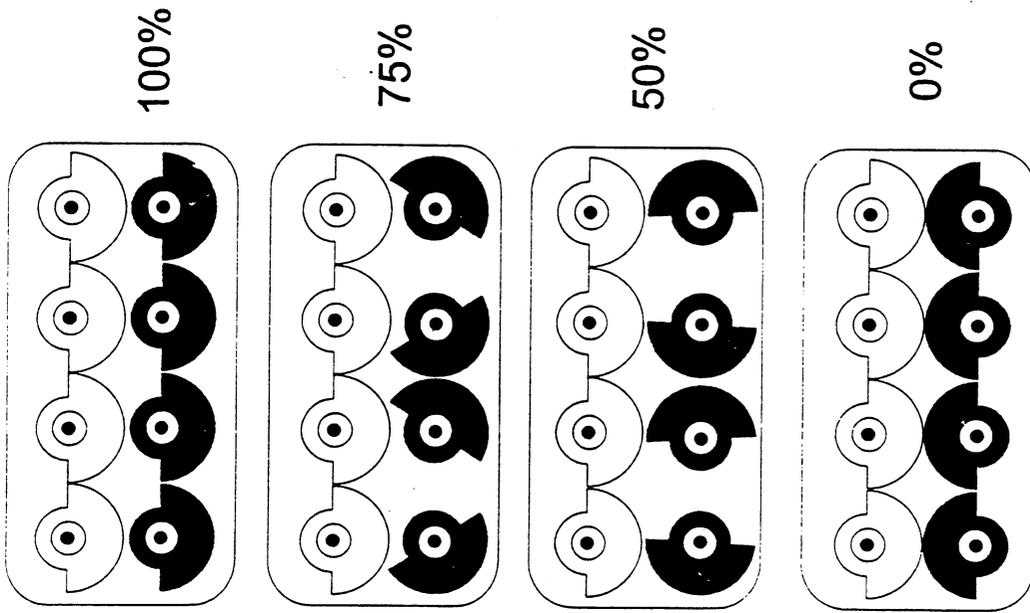
drive the casing efficiently. For some hammers, the optimum static moment can be monitored and adjusted on the basis of the amperage consumed by the electric motors driving the hydraulic pumps.

2-1-1 Peak Component and True Vector Sum

Passage of construction vibrations forces the ground particles to move in an elliptical manner in three dimensions. A two-dimensional representation of displacements produced by pile driving is shown in **Figure 2.6**. While the retrograde motion is symptomatic of a surface wave, the radial motion dominates and gives way to vertical (compression) dominance at a greater distance. To define this motion, three mutually perpendicular components are measured [transverse, longitudinal (radial), and vertical]. Comparison of a typical three-component time history from a transient impact vibration (**Figure 2.4**) shows that:

1. *Many peaks are present in all cases.*
2. *The peak component varies (i.e.. it is radial for one case and transverse for another).*
3. *The peak amplitude in the longitudinal direction, ③, does not occur at the same time as in the transverse direction, ①.*
4. *Shear and compression waves may not be distinguishable from the three components.*

The difference between the three components results from the presence of the various wave types in the vibration wave trains. Variation of motion with each component has led to difficulty in determining which component is more critical. Peak motions should always be reported as either the peak component or the peak true vector sum. Another measure, the maximum vector sum, is frequently reported but is conservative and not directly related to a maximum velocity at a particular time. In general, the maximum of each component is used regardless of the time when it occurs. Also, the maximum particle velocities are utilized in the field assessments without considering the mode of vibration.



Forces produced by two counterrotating masses

Figure 2.5: Operating principle of vibrator with dual rows of eccentric mass, allowing variation of static moment (displacement amplitude) (Massarsch and Westerberg, 1995)

This type of assessment is insufficient to analyze the consequences of the effect of construction vibration. In other words, underground structures are more vulnerable to compressive waves than above ground structures which are susceptible to distorsional waves. Therefore, it is necessary in some applications to distinguish between the modes and the components of the construction vibration.

2-1-2 Propagation Velocity

Propagation velocity is an important factor because it is an indirect measure of soil properties that affect decay of peak particle velocities as well as wavelength. It is important to keep in mind the distinction between the particle and the wave velocities when dealing with seismic wave propagation. The speed of transporting the input energy from point to point in a solid medium is called the wave velocity and it remains the same for the same material. The speed at which a solid particle is being displaced or disturbed at a point is called the particle velocity. Wave velocities are expressed in thousands of meters per second, while particle velocities are expressed in millionths of a meter per second. Particle velocity may be equal to the seismic wave velocity only at a very close distance from the source of the energy.

Geophones on a ground surface record the particle velocities at pertaining points. Knowing the distance between two geophones and the time taken by a seismic wave to travel between the geophones, the wave velocity of the surface layer can be determined. For a decayed vibration in a homogenous elastic solid medium, the velocity of the seismic wave remains unchanged and this corresponds to the stiffness of the material, while particle velocities decrease along the wave propagation path (**Figure 2.2**).

As discussed earlier, the propagation velocities of compressive, shear, and Rayleigh waves vary. Propagation through rock and soil provides a greater wave velocity contrast, as shown by the comparison in **Table 2.1**. The range of velocities for a particular soil type, such as sand, is greater than for rock because of the great variation in stiffness caused by density and overconsolidation ratio for sand and clay, respectively.

Table 2.1 Estimated Propagation Velocities

Material	Wave Velocity (m/s)	
	Compression	Shear
Limestone	2000 - 5900	1000 - 3100
Metamorphic rocks	2100 - 3500	1000 - 1700
Basalt	23000 - 4500	1100 - 2200
Granite	2400 - 5000	1200 - 2500
Sand	500 - 2000	250 - 850
Clay	400-1700	200-500

As shown in **Table 2.1**, the compressive (first arrival) propagation velocity varies greatly for one material type because of differences in densities. In soils the proper propagation velocities should be matched with the appropriate wave type (e.g., compressive with compressive, shear with shear, etc.) to compute frequency and strain. This correlation is important since the shear wave is much more sensitive to changes in soil properties for

propagation below the water table. This sensitivity is a result of the inability of the shear wave to propagate through water. Thus it propagates predominately through the soil matrix.

Below the water table, the compressive wave is much less influenced by the soil density or modulus. because it can propagate through the water itself. The compression modulus values are in turn affected by the presence of the water table and the rate of loading. Higher rates of loading may result in higher moduli values.

2-1-3 Strains From Particle Velocities \dot{u}

Ground strain, possibly one of the most important parameter in construction vibration.. It

can be calculated directly from the mathematical description of the displacement, u , caused by a plane-wave traveling in positive direction, and from the particle velocity, \dot{u} , which is expressed

as

$$\dot{u} \frac{du}{dt} = -cU \cos(x - ct) \quad (2.2)$$

In engineering terms, strain is usually defined as the change in length, Δl , divided by the original length l , or $\Delta l/l$. This ratio is the same as the change in displacement per unit distance. Therefore, in a manner similar to the particle velocity derivation, the first derivative of the displacement, u , with respect to position x yields the strain

$$\varepsilon = \frac{du}{dx} = U \cos(x - ct) \quad (2.3)$$

The strain can be defined in terms of the particle velocity, \dot{u} , and propagation velocity, c , by substituting Equation 3 into the Equation 2.

$$u = -c \cos(x - ct) = -c\varepsilon \text{ and } \varepsilon = \frac{\dot{u}}{-c} \quad (2.4)$$

Positive particle velocities toward the right then produce negative strains. Therefore, negative strains are compressive. Thus for plane waves, ground strains can be calculated directly from the particle velocities if the compressive-wave propagation velocity is known or can be estimated.

2-2 RESPONSE OF COHESIONLESS SOIL DEPOSITS UNDER CONSTRUCTION VIBRATIONS

For ground response involving no residual soil displacements, the response is determined mainly by the moduli values and damping characteristics. In such cases, analyses are often made using the equivalent linear analysis method in which the moduli and damping factors used in the analysis are compatible with the strains developed in the soil deposit or earth structure. Values of soil moduli can be determined using laboratory or field testing techniques. Advantages and disadvantages of these techniques have been reported in the literature (Marshall, 1981; Tawfiq, 1987). Several useful empirical relationships were suggested to obtain shear and compression modulus from physical soil properties of cohesionless soils.

A comprehensive survey of the factors affecting the shear moduli and damping factors of soils and expressions for determining these properties have been presented by Hardin and Drnevich (1970). In these studies it was suggested that the primary factors affecting moduli and damping factors are: strain amplitude, γ , effective mean principal stress, σ'_m , void ratio, e , number of cycles of loading, N . Factors with minor influence include: Octahedral shear stress, σ_{oct} , overconsolidation ratio, OCR, effective stress strength parameters, c' and ϕ' .

Relationships developed by Hardin and Drnevich (1970) showed clearly that modulus values for sands are strongly influenced by the confining pressures, the strain amplitude, and the void ratio (or relative density), but not significantly by variations in grain size characteristics or other factors. The expression suggested for maximum shear modulus was as follows:

$$G_{\max} = 14760 \frac{(2.973-e)^2}{1+e} (OCR)^a (\sigma'_m)^{1/2} \quad (2.5)$$

where G_{\max} = maximum shear modulus, in psf,
 e = void ratio
 OCR = overconsolidation ratio
 a = a parameter that depends on the plasticity index of the soil
 σ'_m = mean principal effective stress in psf.

The value of 'a' can be obtained from the following table:

<u>PI</u>	<u>a</u>
0	0
20	0.18
40	0.30
60	0.41
80	0.48
≥100	0.50

For practical purposes, a convenient relationship between the shear modulus and the confining pressure was provided by the simplified equation (Seed and Idriss, 1970):

$$G = 1000 K_2 (\sigma'_m)^{1/2} \text{ in (lb/ft}^2\text{)} \quad (2.6)$$

where σ'_m = mean principal effective stress in psf.
 K_2 = soil modulus coefficient

The effect of the void ratio and the strain amplitude can be expressed by their influence on the *soil modulus coefficient, K_2* . For any sand, this coefficient has a maximum value $(K_2)_{\max}$ at very low strains of the order of 10^{-4} percent. The influence of other factors such as angle of

friction, ϕ' , effective vertical stress, σ_v' , coefficient of lateral stress at rest, K_o , and void ratio, e , on the computed relationships between K_2 and strain amplitude, γ , can be summarized as follows:

- (a) *At very low strains ($\gamma < 10^{-3}$ percent), K_2 depends only on the void ratio, e .*
- (b) *At intermediate strains ($10^{-3} < \gamma < 10^{-1}$ percent), the variation of K_2 with strain is only slightly influenced by the vertical stress and very slightly by variations in ϕ' and K_o . The values of K_2 are still influenced strongly by the void ratio however.*
- (c) *At very high strains ($\gamma > 10^{-1}$ percent), the values of K_2 are slightly influenced by the vertical stress but they are essentially independent of K_o , ϕ' and e .*

Thus for practical purposes, values of K_2 may be considered to be determined mainly by the void ratio, e , or relative density, D_r , and the strain amplitude, γ . This conclusion has also been shown in studies by Kuribayashi et al. (1974) and Krizek et al. (1974).

Yoshimi et al. (1977) suggested that **Equation (5)** is more applicable for sands with angular grains and proposed the following equation for sands with rounded grains :

$$G_{\max} = \frac{8400(2.17-e)^2}{1+e} (\phi_m)^{1/2} \quad kN / m^2 \quad (2.7)$$

The Yoshimi et al. equation represents the upper bound limit of similar relationships suggested by Hardin and Black (1968). According Hardin and Black, the maximum shear modulus at low amplitudes ($\gamma \leq 1 \times 10^{-4}$) can be determined from:

$$G_{\max} = \frac{6908(2.17-e)^2}{1+e} \sigma_o^{-1/2} \quad kN / m^2 \quad (\text{round - grained}) \quad (2.8)$$

and

$$G_{\max} = \frac{3230(2.97-e)^2}{1+e} \sigma_o^{-1/2} \quad kN / m^2 \quad (\text{angular - grained}) \quad (2.9)$$

where

propagation methods in the field, and then reducing this value for other strain levels in accordance with the results indicated by the average (dashed) line in **Figure 2.8**.

The studies by Prakash and Puri, 1981, using in-situ tests, indicate that for silty sands the modulus attenuation curve may be slightly flatter than that shown in **Figure 2.8**, but the difference is relatively small. It should also be noted that the Hardin-Drnevich equations and the experimental results of Shibata and Soelarno (1975) and Iwasaki et al.(1976) show that the modulus attenuation curve for sands is influenced slightly by the confining pressure. The experimental results of Iwasaki et al. are presented in **Figure 2.9**. Thus, where more refined analyses are required, it may be more appropriate to use a family of curves similar to those shown in **Figure 2.9** to evaluate the response of sand deposits.

2-2-1 Comparison of Values of $(K_2)_{\max}$ for Sands Determined by Laboratory and Field Tests

The values of K_2 shown in **Figure 2.** are based on laboratory tests on sands. It may be seen that for relatively dense samples, the values of $(K_2)_{\max}$ determined at very low strains for laboratory test specimens are typically in the range of 50 to 75. The results of a number of determinations of shear moduli for sands at very low strain levels by means of in-situ shear wave velocity measurements are summarized in **Table 2.2**:

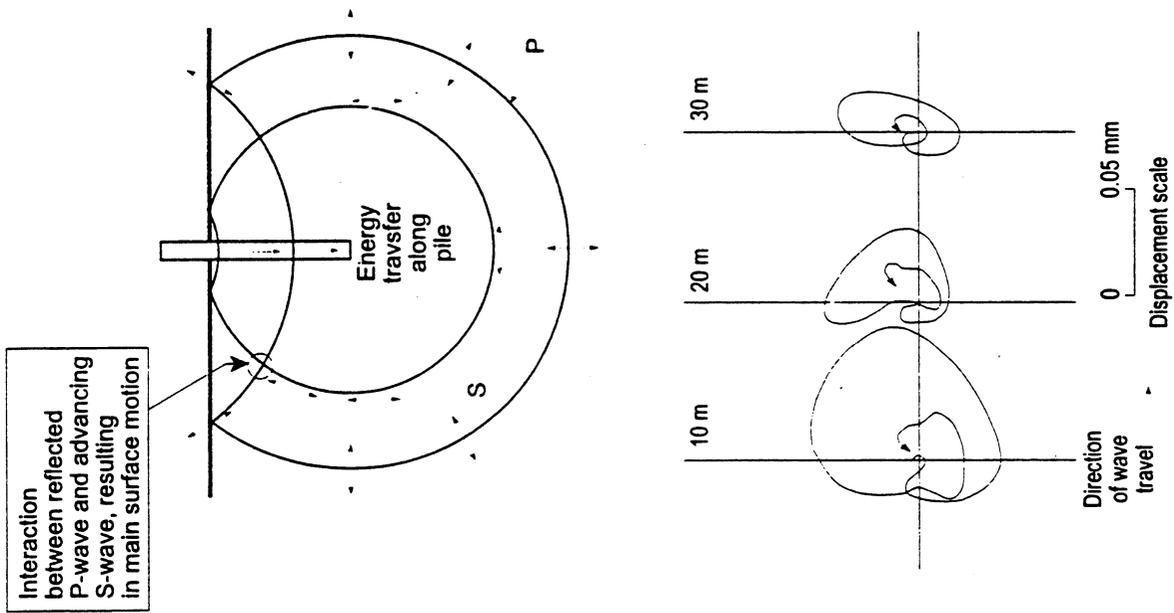


Figure 2.6: Principal wave types and particle displacement paths during passage of surface waves produced by sheath pile driving (Attewell & Farrer (1973)).

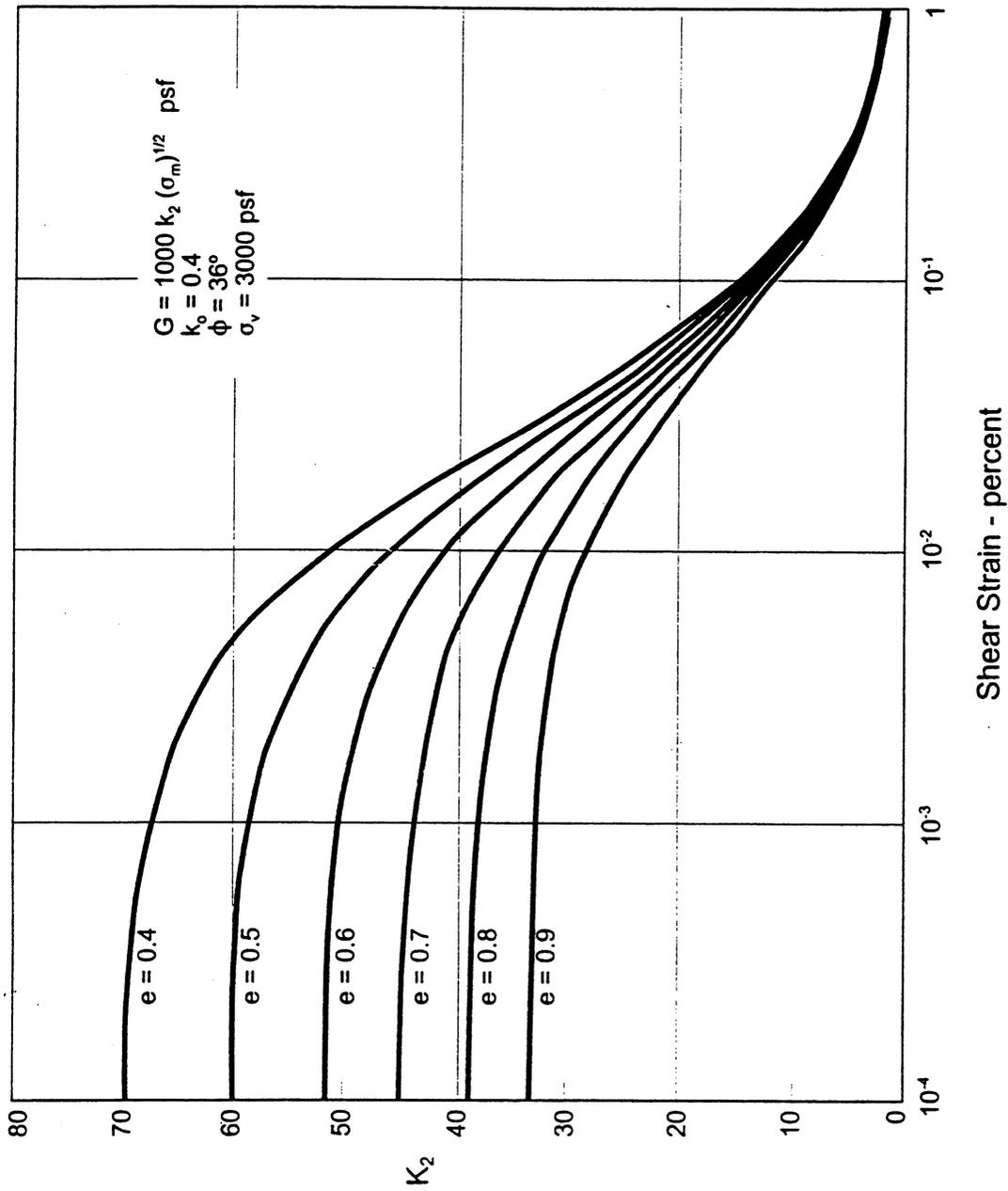


Figure 2.7a: Shear moduli of sands at different void ratios (Based on Hardin - Drnevich expressions)

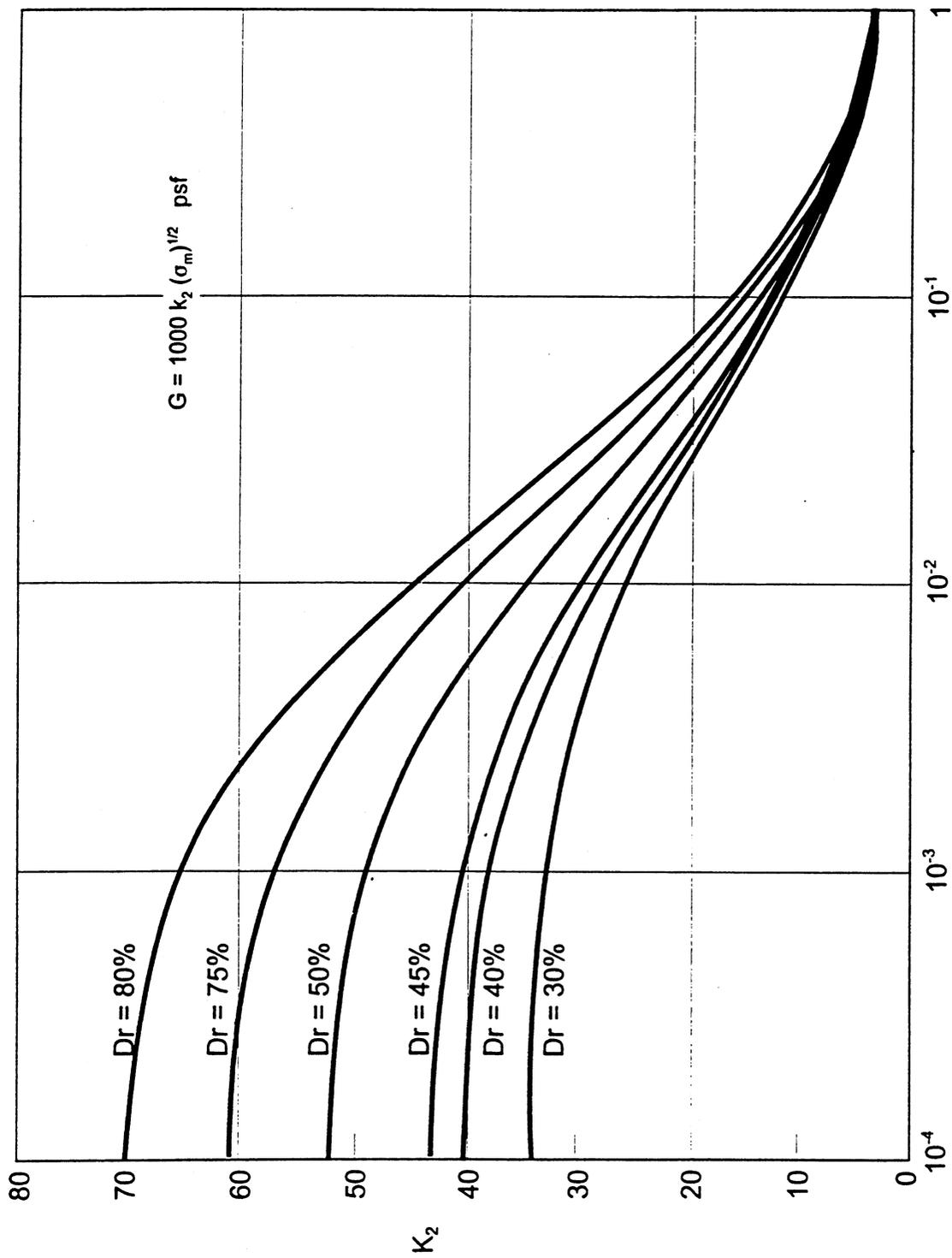


Figure 2.7b: Shear moduli of sands at different relative densities

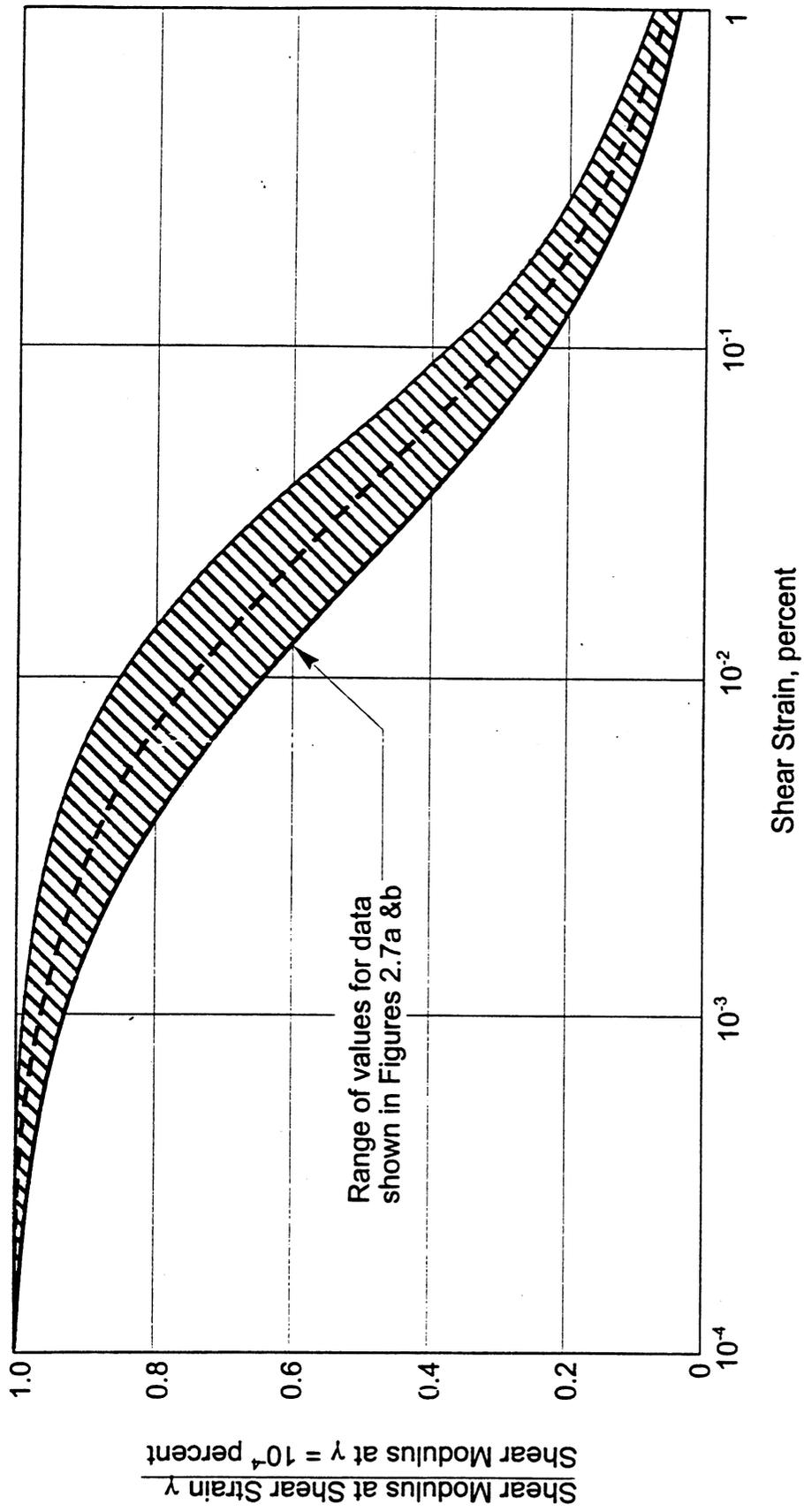


Figure 2.8: Variation of shear modulus with shear strain for sands (Seed and Idriss, 1970)

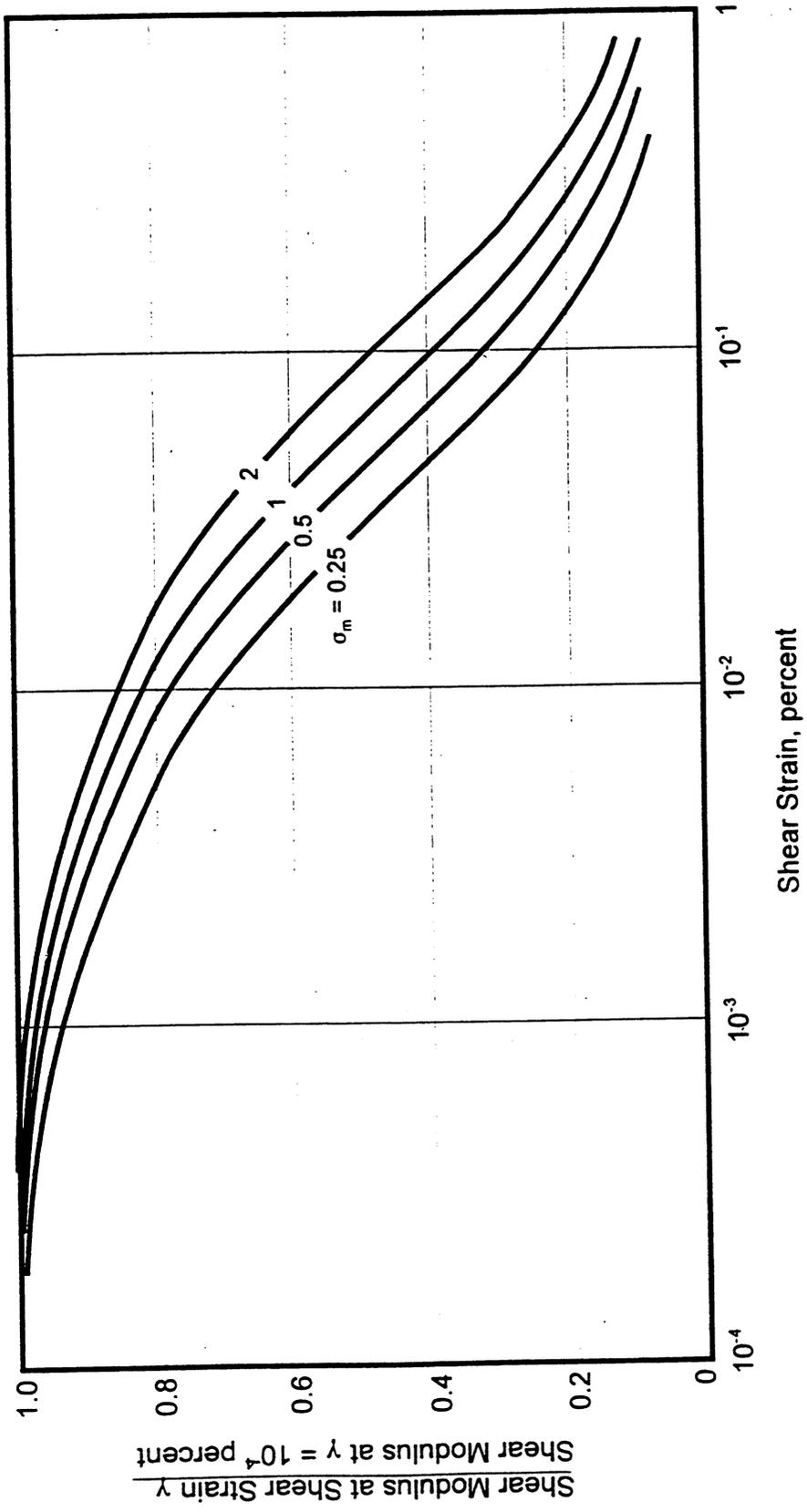


Figure 2.9: Variation of shear modulus with shear strain for sands (Iwasaki, et al., 1976)

Table 2.2 Shear modulus of sand based on field shear wave velocity measurements
(Seed et al., 1984).

Soil	Location	Depth (ft)	k_2
Loose moist sand	Minnesota	10	34
Dense dry sand	Washington	10	44
Dense saturated sand	South California	50	58
Dense saturated sand	Georgia	200	60
Dense saturated silty sand	Georgia	60	65
Dense saturated sand	South California	300	72
Extremely dense silty sand	South California	125	86
Dense dry sand (slightly cemented)	Washington	65	166
Moist clayey sand	Georgia	30	119

The six investigations for dense to extremely dense sands (excluding clayey and partly cemented sands) give values for $(K_2)_{max}$ ranging from 44 to 86. Thus there appears to be good general agreement between the results of laboratory and in-situ investigations.

Further evidence of this result is provided by studies by Ohta and Goto (1976). On the basis of numerous shear wave velocities measured in the field, these investigators presented the following equation:

$$v_s \text{ (m/s)} = 69 N_j^{0.17} D^{0.2} \text{ (m)} \cdot F_1 \cdot F_2 \quad (2.10)$$

where:

N_j = SPT N-value as measured in Japanese practice

D = depth of soil below ground surface

F_1 = a factor, depending on the nature of the soil, having a value of 1 for alluvial deposits and 1.3 for diluvial deposits

F_2 = a factor, depending on the nature of the soil as follows:

<u>Soil Type</u>	<u>Factor F_2</u>
Clay	1.0
Fine sand	1.09
Medium sand	1.07
Coarse sand	1.14
Sandy gravel	1.15

Gravel

1.45

Thus for sands and sandy gravel deposits, the average value of F_1 is about 1.15 and the average value of F_2 is close to 1.1 so that the product of $F_1 F_2$ is typically very close to 1.25.

Converting the results to fps units, **Equation (8)** reduces to

$$v_s = 220 N_{60}^{0.17} D^{0.2} \quad (\text{ft/sec}) \quad (2.11)$$

where:

N_{60} = N-value measured in SPT test delivering 55% of the theoretical free-fall energy to the drill rods and

D = depth of soil in feet.

Actually, due to the small power of N_j in the original equation, the difference in SPT N-values can be neglected for all practical purposes.

$$\text{Since } G_{\max} = \gamma/g (v_s)^2 \quad (2.12)$$

Equation (11) provides a correlation between G_{\max} and SPT N-value, based on field test data as follows. Assuming $\gamma = 120$ psf, substitution of **Equation (11)** in **Equation (12)** leads to:

$$G_{\max} = 180 \cdot 10^3 \cdot (N_{60})^{0.34} \cdot D^{0.4} \quad (2.13)$$

If the water table is at a relatively shallow depth below the ground surface, the effective stress at depths below 10 feet may be expressed approximately by:

$$\sigma_o' \simeq 62.5 D \quad (2.14)$$

$$\text{Thus } D \simeq \sigma_o'/62.5 \quad (2.15)$$

Substituting **Equation (15)** into **Equation (13)** leads to

$$G_{\max} \simeq 35 \cdot 1000 N_{60}^{0.34} (\sigma_o')^{0.4} \quad (2.16)$$

Noting that $N = N_1 / C_N$ leads to

$$G_{\max} \simeq 35 \cdot 1000 (N_1)_{60}^{0.34} \frac{(\sigma_o')^{0.4}}{C_N^{0.34}} \quad (2.17)$$

It can readily be shown that with a high degree of accuracy for effective stresses up to 6000 psf:

$$\frac{(\sigma_o')^{0.4}}{C_N^{0.34}} \simeq 0.47 (\sigma_o')^{0.5} \quad (2.18)$$

Thus, **Equation (17)** may be rewritten as:

$$G_{\max} \simeq 16.5 \cdot 1000 (N_1)_{60}^{0.34} (\sigma_o')^{0.5} \quad (2.19)$$

For normally consolidated deposits, $\sigma_m' \simeq 0.65 \sigma_o'$, therefore

$$G_{\max} \simeq 1000 \cdot 20 (N_1)_{60}^{0.34} (\sigma_m')^{0.5} \quad (2.20)$$

It may be noted that this equation has the same form as **Equation (6)**, provided:

$$(K_2)_{\max} \simeq 20 [(N_1)_{60}]^{1/3} \quad (2.21)$$

Values of $(K_2)_{\max}$ determined from **Equation (21)** for N_1 values ranging from $N_1 = 5$ (loose sands) to $N_1 = 44$ (very dense sands) are as follows:

<u>$(N_1)_{60}$</u>	<u>$(K_2)_{\max}$</u>
5	34
8	40
10	43
18	52
28	61
44	71

These values for $(K_2)_{\max}$ are in the same range as those shown in **Figure 2.7** providing further confirmation of the similarity in values of $(K_2)_{\max}$ for laboratory and field determinations.

2-3 CONCRETE VIBRATION

Importance of vibration of the consolidation of fresh concrete has been covered in many studies since the development of the vibratory machines in 1930's. In 1936, an ACI Committee 609 reported the benefits of vibrators but failed to explain the effect of vibrations on a fresh

concrete. The frequencies of the early vibrators were limited to 3000 - 5000 cycles per min (50-80 Hz). In 1948, L'Hermite and Tournon reported their fundamental research into the mechanism of consolidation. They found that friction between aggregates is the most important factor preventing consolidation (densification) of fresh concrete, but that this friction is practically eliminated when concrete is in a state of vibration. Meissner (1953) summarized previous research studies and reviewed the state of the art on available equipment and its characteristics. The 1960 ACI Committee 609 gave specific recommendations for vibrator characteristics applicable to different types of construction and field practices. Walz (1960) described the various types of vibrators: internal, surface form, and table and their applications. He also showed that the reduction in internal friction is primarily the result of acceleration produced during vibration. Olsen (1987) used accelerometers to measure the rate of movement of fresh concrete and was able to establish the minimum energy level required to achieve a degree of consolidation of 97 percent or more.

In drilled shaft construction, freshly placed concrete can be subjected to either overvibration or revibration. The difference between the two occurrences is the time delay involved in vibrating the concrete. Overvibration of concrete results from subjecting the concrete mix to a long duration of vibration, or due to the use of grossly oversized equipment and vibration the concrete many times over the recommended amount. Revibration occurs by subjecting the concrete to additional vibration cycles at successive time delays.

Accordingly, fresh placed concrete during drilled shaft construction may be subjected to overvibration, if the surrounding vibrations due to construction activities continue to be generated for a long period during concrete placement, or the fresh concrete is exposed to additional cycles of construction vibrations at different time levels.

Overvibration may result in segregation, or sand streaks. In the same shaft, concrete segregation, can produce different densities with the depth. This variation in the densities may adversely affect the design capacity of the drilled shaft and the durability performance of the concrete. Keeping in mind that the unfavorable subsurface conditions in Florida, durability represents one of the major parameters in designing any underground concrete structures. The problem of concrete overvibration has been discussed by Forssblad and Sallstrom (1995), Alemo and Grandas (1993) and Stark (1986). Forssblad and Sallstrom (1995) suggested that the duration vibration in concrete to amount 60 to 70 percent of the total casting time or the vibration effort can be obtained as follows:

$$V_e = 1800/C \quad (2.22)$$

where V_e = vibration effort s/m^3
 C = casting capacity m^3/hr

They also found that the optimum vibration effort ranged between 200 s/m^3 to 325 s/m^3 . Using this relationship, a drilled shaft with 4 ft diameter an 20 ft depth can sustain 50 min of continuous vibration, without inducing any changes in the concrete density or compressive strength.

On the effect of revibration of fresh concrete, literature did not present any conclusive evidence that construction-induced vibration would significantly affect concrete properties. However, Tuthill (1977) reported that revibration may produce benefits, particularly for the wetter mixtures, in eliminating water gain under reinforcing bars, reducing bugholes, especially in the upper portion of deep lifts, all of which increase the strength of the concrete. Simulating a field blast condition, Esteves (1978) conducted laboratory testing in concrete prisms subjected to transient impact loading. At different intervals of curing time, he observed the development of microcracks versus the amplitude of the particle velocities (**Figure 2.10**). Surface cracks were

noticed on concrete prisms subjected to impact compression waves at 10 hr of curing time. The particle velocities that produced these cracks reached a level of 9.8 in/sec (250 mm/sec).

If the longitudinal-wave propagation velocity of fully cured concrete is assumed to be 3,000 m/s (10,000 ft/sec), the plane-wave strain associated with the minimum velocity for cracking will be

$$\varepsilon = \frac{\dot{v}}{c} = \frac{150 \text{ mm/sec}}{3000 \text{ m/sec}} = 50\mu \quad (2.23)$$

Other studies have shown that during curing, the modulus, and, therefore, the compressive-wave velocity, are much lower than the final value. Thus, the strain calculated with this larger propagation velocity is a lower bound.

Esteves (1978) results indicated that there is a period of greater susceptibility to vibration cracking (between 10 and 20 hours); however, the high threshold during this period (150 mm/s) explains why other studies have also shown that there is no loss of final strength from transient vibration (Howes, 1979; Oriard and Coulson, 1980). Hulshizer (1996), suggested some vibration acceptance levels for freshly placed and maturing concrete. In general, Hulshizer found that a value of 5.0 in/sec would represent an average value for an acceptable particle velocity during field construction. This limit may increase or decrease depending on the type of vibration (Impact or harmonic) and on the duration of the vibration (short period or continuous).

Two of the pile vibration tests conducted by the Michigan and California Highway Departments on in-situ curing of concrete are of special interest. The first involved driving through sand within 0.75 m (30 in.) of 5 m (15 ft)-long-cast-in-place piles some 5 to 6 hours after pouring. After 46 days, these piles were extracted and cored to determine the strengths. The

ground motions produced by pile driving show that the vibration levels at these piles may have been as high as 100 mm/s. Also, piles subjected to vibration were statistically stronger than the nonvibrated comparison pile. The second case involved vibrating in-place cylinders by driving two 11 meter Raymond step taper piles over a curing time span similar to that reported by Esteves (1978). Particle velocities at distances of 2.5, 5, 10, 20, and 40 ft from the vibrating piles recorded amplitudes of 3.9, 1.97, 0.5, 0.3, and 0.12 in/s, respectively. Once again, these California results showed that vibratory excitation by adjacent piling, even during the critical 12 to 14 hour period, did not reduce the strength of cast-in-place concrete piles.

It is apparent that enough has been learned about concrete vibration during the past 50 years to insure that low slump concrete can be placed successfully. However, a better understanding of the interaction of vibration and fresh concrete is still desirable. Knowledge gained from past experiences on this subject will be utilized in the current study to investigate the extent of the effect of construction vibration on the concrete performance. The investigation will also address the determination of the minimum distance where vibration in the vicinity of a freshly poured drilled shaft should not be allowed.

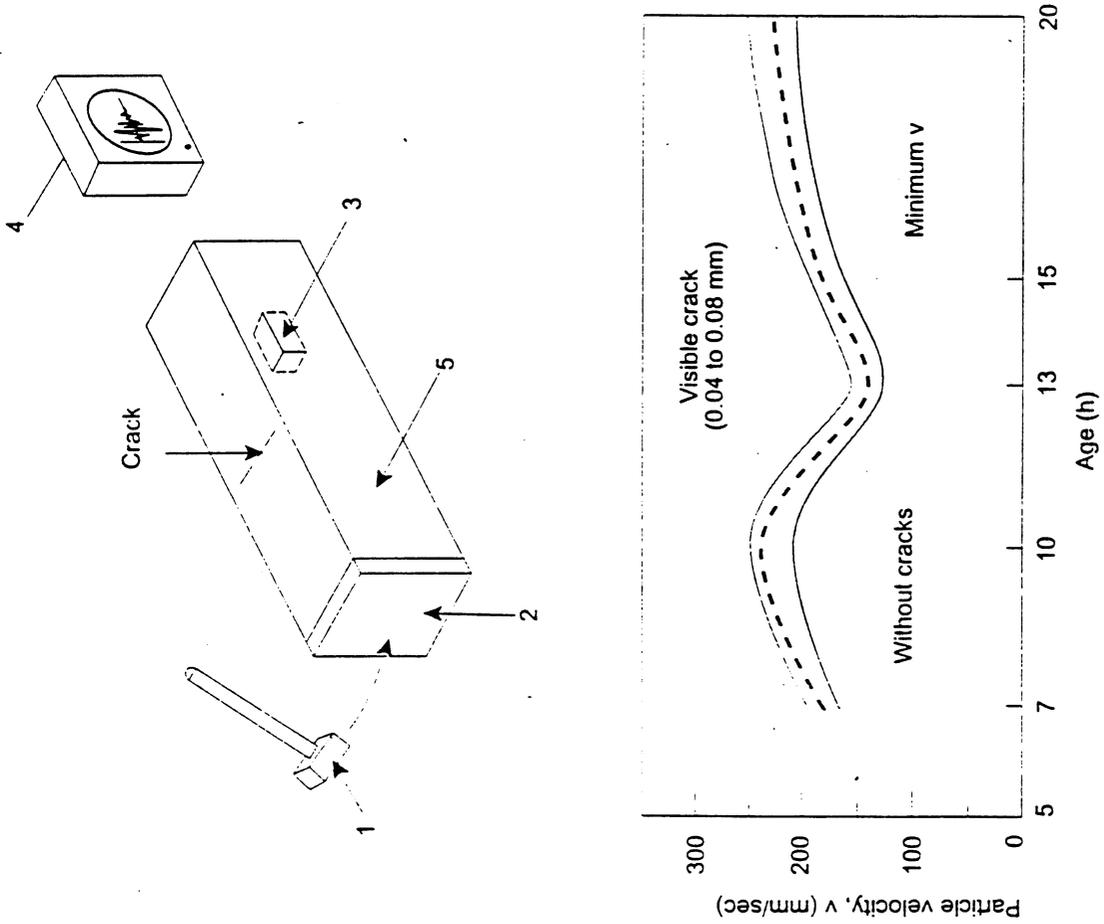


Figure 2.10: Vibratory cracking of curing concrete. (a) Schematic of the test: 1, hammer 6 kg; 2 iron plate; 3 geophone; 4, records; 5, recorder; 5, concrete prism with 0.20 x 0.20 x 0.85 m. (B) Critical velocity level (Esteves, 1978)

CHAPTER III

LABORATORY TESTING

3-1 INTRODUCTION

The test program involved in this study was divided into three phases. These phases were set so that certain parameters such as vibration amplitude and curing time for concrete were first investigated under controlled laboratory conditions. Then results from the first phase were utilized to set up the second and the third phases. Before commencement of the full scale drilled shaft construction and testing, which constituted the third phase, a decision was made to monitor and record simulated construction vibrations produced by a full scale driven casing of 3 foot diameter by 40 foot height. This intermediate stage was considered as the second phase of the test program and was given particular attention, since results from this phase would determine the pattern of instrumentation, and the type of measurements needed for the full scale in-situ testing. This was done because of the lack of any standard testing procedure to conduct such a field test. Most of the published studies on construction vibration were limited to the effect of pile driving, sheet pile construction, and other blast vibrations on adjacent structures or ground settlement. The in-situ test procedure and the instrumentation setup to evaluate the effect of construction vibration on freshly placed concrete have not been covered in the literature. The maximum particle velocities from the second phase were used to arrange the ground and embedded geophones for the full scale drilled shafts. Additional information was gathered from the field testing in the second stage to specify certain driven depths by the steel casing and the time intervals needed to record the sensor outputs.

Various destructive and nondestructive testing methods were used in the test program to obtain strength, stiffness, density, and porosity measurements as related to induced construction vibrations. Laboratory results were used to establish guidelines for the thresholds of vibration amplitudes and permissible curing time spans. In-situ field testing addressed the effect of the severance of construction vibration on the density and porosity of the installed shafts. Additional sets of compression tests were performed on core samples obtained from the full-scale drilled shafts.

Since soil properties were among the parameters considered in this study, the decision was made to limit the investigation to sandy soil layers only. The reason for this selection was: (1) the casing method in drilled shaft construction is typically followed in caving soils, and (2) Soil configurations in Florida are predominantly sandy soils. Also, the presence of shallow ground water tables at the selected testing site provided further information about the effect of effective octahedral stresses (σ_o) on the shear wave velocities (v_s) and on the damping characteristics ($D\%$) of the soil layers. About 60 soil samples were obtained from the Florida Atlantic University (FAU) site were classified and tested for water content, unit weight, and void ratio. Grain size distribution analyses were performed on representative samples obtained from the site.

The following sections describe in detail the steps carried out in the testing program to meet the objectives of this study. Since the laboratory and the full scale field testing were conducted at three different stages, concrete samples obtained for each stage were prepared using the same proportions of materials in each case. Requirements given in *Sections 346 and 455* of the FDOT Standard Specifications for drilled shaft concrete were followed in the testing program.

3-2 TEST METHODOLOGY

A large number of concrete samples were prepared to investigate the effect of vibration amplitudes on the strength and stiffness properties of concrete. Taking into account the reproducibility of the compression test results and the different time delay for vibrating the concrete samples, a test was conducted on a total of 216 concrete cylinders (6 in. x 12 in.). The mixing and curing time was scheduled in a way so that the sequence of vibration testing was started for some sets immediately after pouring the cylinders ($t = 0$).

Three levels of vibration amplitudes were selected for this segment of the testing program. The vibration amplitudes were designated low, medium, and high. Although there were no guidelines for this designation, the amplitudes were found to increase as the frequencies in the readout box increased.

A **FMC Syntron (Model 04A)** vibration table was used in this study. This model has a variable frequency counter (**Figure 3.1**). However, this shaking table, as in any other multipurpose commercial tables, did not have control over the vibration frequency and amplitude at the same time. It was noticed that as the reading on the frequency counter increased, the amplitudes of the vibrations increased, accordingly. Therefore, before the vibration of concrete samples began, the decision was made to identify the signals generated by the shaking table, including waveforms, frequencies and amplitudes. Also, since the electric vibrating motor was mounted at the center of the table, the vibration identification could be conducted at two different locations, the center, and at the edge of the table.

After recording the generated waves using two accelerometers (**PCB, Model 353-B04**) connected to a two channel oscilloscope, the vibration amplitudes increased with the frequency.

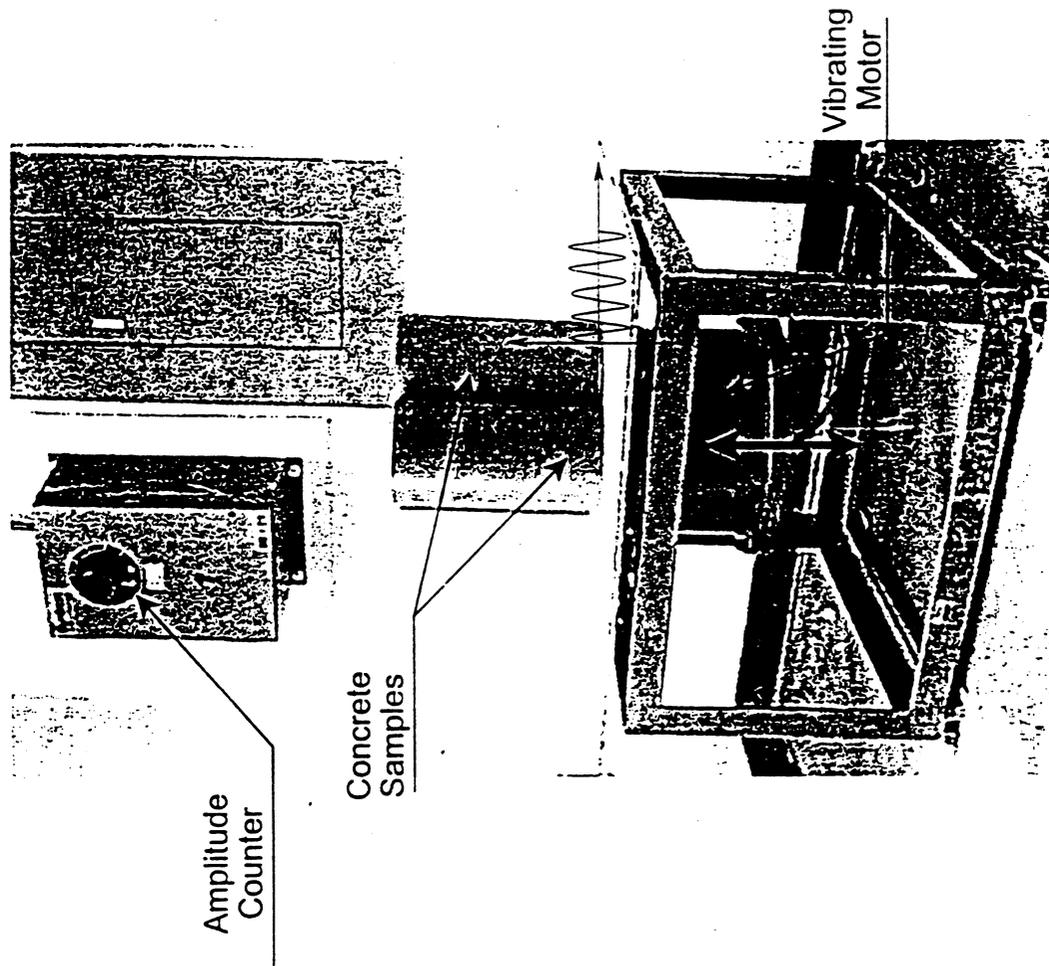


Figure 3.1. Shaking table used in the study.

The frequency increment, however, did not exceed 300 Hz after attaining an acceleration of about 15g.

Using the generated sinusoidal wave forms, the velocity amplitudes selected for this study were 0.128 in./sec, 2.58 in./sec and 8.3 in./sec; these were used for all the tested samples. The concrete samples were subjected to vibration at predetermined time delays. The time delay intervals were 0 hour, 1 hour, 2 hour, 3 hour, 4 hour, and 24 hour. A set of three samples was tested at the same time to obtain the average measurements. Measurements of the compressive strength, f'_c , for the concrete samples were taken at 3 days, 7 days, 14, days, and 28 days.

Before the compression testing, each sample was tested using the ultrasonic testing method (**ASTM C-597**) to determine the compression modulus of the material (**Figure 3.2**). An ultrasonic concrete tester **Model PUNDIT CT-330** was used for the testing. As for the compression test, the ultrasound testing method, necessitates full contact between the exciter and the concrete surface. This contact was achieved by using high vacuum grease to ensure the transfer of the high frequency pulses (54 kHz) through the concrete sample. Any variations in the arrival time of the high frequency signals can drastically affect the determination of the wave velocities and, hence, the compression modulus values.

Of particular concern is that measuring compression modulus using the ultrasound testing method may result in stiffness values that are higher than those obtained from the standard method for determining the static modulus of elasticity (**ASTM C469**). The main reason for this increase in the stiffness values is the very small strain levels produced by the ultrasound pulses. Despite the difference in the compression modulus values, the consistency in the ultrasound testing procedure provided comparable variations in the stiffness values of the concrete samples.

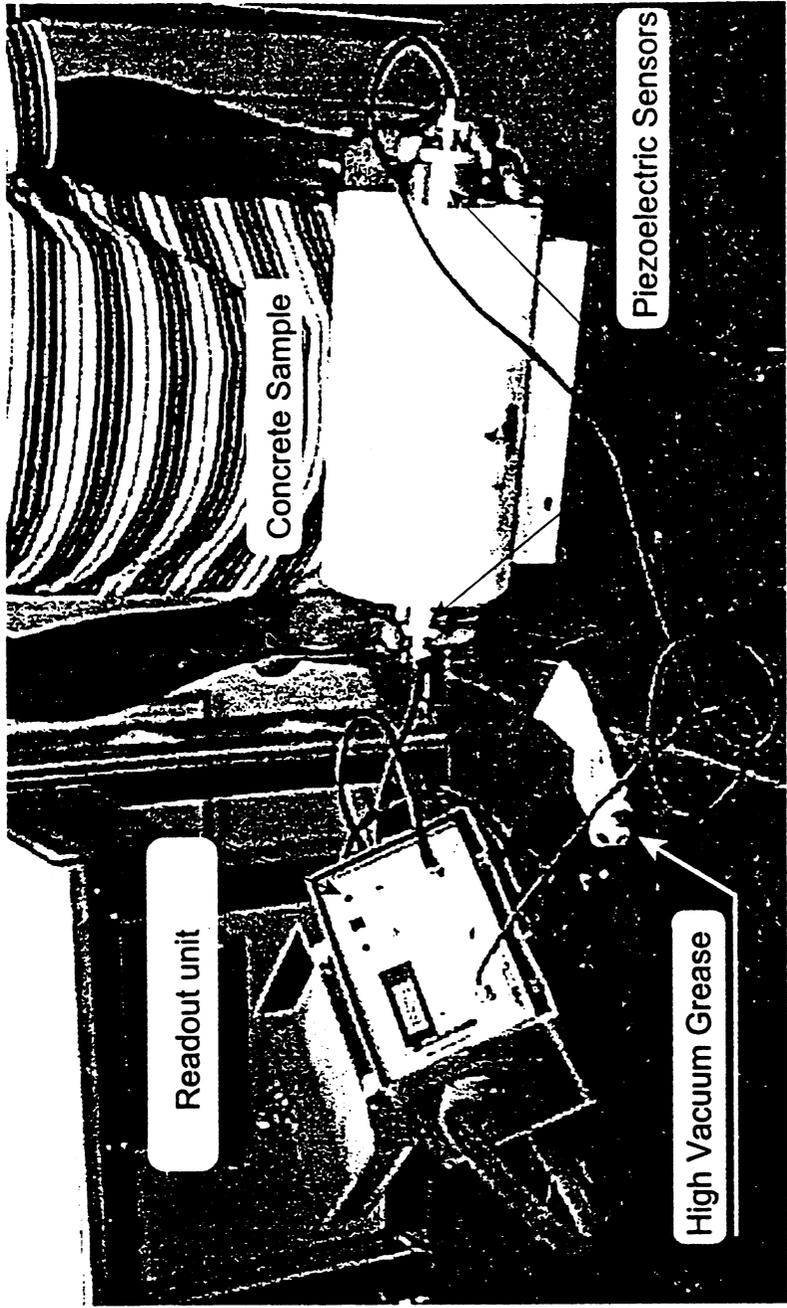


Figure 3.2: Ultrasonic Testing For Concrete Samples

Therefore, the effect of parameters, such as vibration amplitude and time delay from revibration, could be studied from ultrasound testing.

After concluding the stiffness measurements, concrete samples were prepared for the compression testing according to the *ASTM C-39* specifications. All the samples were carefully monitored during load applications for any unusual behavior that the samples could exhibit. The compressive strength tests did not reveal any uncommon patterns of failure associated with vibrations. However, some concrete cylinders exhibited increased surface voids when they were visually examined (**Figure 3.3**). These cylinders were obtained from sets subjected to high vibration amplitudes (8.3 in/sec) after three hours from concrete placement. It was difficult to establish the increased void content at the cut surfaces of the laboratory samples and cores from the field, as compared to those subjected to lower vibration amplitudes. Similar observations were recorded for core samples obtained from the full scale drilled shafts at the Delray Beach and Clearwater sites. These cores were obtained from shafts subjected to vibrations after two to four hours from placement.

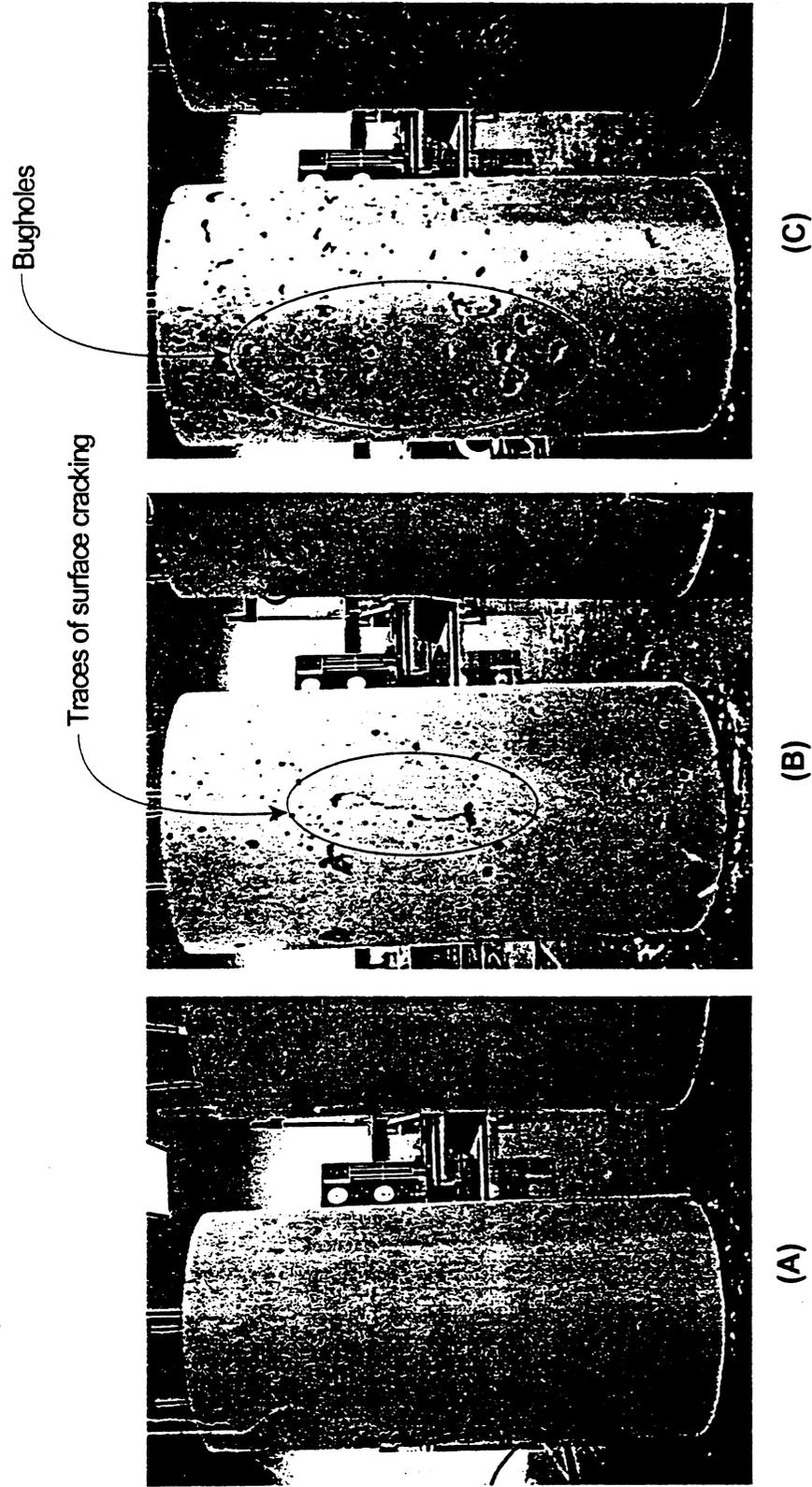


Figure 3.3: Concrete samples subjected to different levels of vibration amplitudes and at various delay time;
 (A) Low vibration amplitude at zero time, (B) High vibration amplitude at two hours,
 (C) High vibration amplitude at three hours.

sites. In each boring, a slurry mix was used to prevent caving in the bored hole. The soil samples obtained with the split-spoon methods at FAU were analyzed for their properties at FAMU/FSU, the details of which are covered later in the chapter on laboratory testing. On-site visual inspection of the soil samples showed some very thin layers of crushed shells; however, these layers of shell were far and in between, and went unnoticed in the SPT blow count records. Soil samples were not taken at the other two sites, rather their existing records were analyzed and compared with the samples taken at the FAU North Campus Site; their similarity did not dictate any further testing.

4.2 FIELD VIBRATION TESTING

4.2.1 FIELD TEST NO. 1 (FAU, BOCA RATON, FL)

In view of the cost of installing deep foundations, it was necessary to conduct a preliminary installation of a steel casing during which the vibration levels in the subsurface would be observed and analyzed. The purpose was to gain an understanding of the dynamic properties of the subsurface and the amplitude of the generating source of vibration that would be used during installation of the drilled shafts.

To gather the data needed to generate a profile of the vibration travel in a 3-D environment, two tests were undertaken at the North Campus of FAU. On December 3, 1997 the profile of the range of displacements at the surface and subsurface was determined in the laboratory to predict the effect of these vibrations on the integrity and strength of the field shafts.

Before starting the field testing, representative soil samples were obtained on August 27, 1997, from two soil borings drilled to about 41 feet. The two boring logs showed that the soil layers at the site were predominantly light tan to dark sandy soil. The obtained samples were then transferred to the FAMU-FSU College of Engineering in Tallahassee for further testing.

During the preliminary investigation, the driving subcontractor vibrated the 36-in. steel casing into the subsurface. The installation was achieved using an MKT vibratory hammer. The steel casing was vibrated using the maximum frequency of the hammer's output. These frequencies ranged between 1800-2000 rpm. The steel casing was introduced into the ground at a steady rate in increments of five feet. This increment was decided based on the need to remount the accelerometer attached to the steel casing, which measured the actual displacement of the casing from a distance of five feet above ground level, until it reached the surface of the soil. In addition to the monitoring of the displacement, surface vibrations were successfully recorded and the nature of these will be discussed later.

4.2.1.1 SITE PREPARATION

The ground water table at the site was at about 6.5 feet from the surface. This shallow water table caused some difficulties in installing the triaxial geophones needed to monitor the propagation of the vibrations. The triaxial geophones (**MARK, Model A020793**) inserted in the designated boreholes were buried in the ground once the casing started to vibrate. Recovering the sensors from the ground resulted in a significant time delay which necessitated the postponement of this segment of testing.

4.2.1.2 INSTRUMENTATION AND VIBRATORY EXCITATION

Only casing and surface vibrations could be monitored and recorded from this trial. The sensor attached to the steel casing was of a special type of shock accelerometers (**PCB Model 370A02**) and was placed at a five foot depth from the ground surface. After driving the 36 inch diameter casing into the ground, the accelerometer was relocated at

the same depth from the surface (**Figure 4.1a**) . To provide a proper coupling, the surface accelerometer (**PCB Model 353 B04**) was attached on a steel rod which was inserted in the ground (**Figure 4.1b**). Vibration measurements were taken at distances of five, ten and ten foot consecutively from the vibrating casing (**Figure 4.1c**). Signals from the accelerometers were captured by a two channel FFT analyzer. **Figures 4.1d and 4.1c**, Measurements were then stored on a floppy disk for further analysis.

Dimensions of the steel casing used during this investigation:

Thickness = 0.5 in.

Length = 40.0 ft.

Diameter = 36.0 in.

4.2.1.3 MONITORING OF VIBRATIONS

During the installation of the steel casing, vibration levels were monitored at three different locations, which are discussed below:

Steel Casing

The steel casing is the medium through which the vibrations are translated into the subsurface. Thus attaching a PCB accelerometer to the casing generated a history of the displacement of the casing through its penetration into the ground. The accelerometer was placed five feet above ground on the steel casing. Once the accelerometer reached ground level the vibration was stopped and the accelerometer relocated to five feet above the ground. The amplitude obtained during the oscillation of the steel casing were used to determine the attenuation of the subsurface material and these records are compared with the laboratory results in Chapter V.

Surface

Motion at ground level has the potential to produce the largest displacements. The surface is a 'free-end', thus its range of motion is not confined as deepening layers. Using a Wilcox Research accelerometer on the surface at varying distances during the installation of the steel casing, the vibration levels in the surface layer were monitored and recorded. In similar fashion, these records were used to determine the attenuation factors in the working soil.

Subsurface

The recordings were limited and for all practical purposes not useful to interpretation. Therefore, the need for a more comprehensive investigation into the subsurface was identified and is addressed in the next section. The limitations imposed were due to an interface problem with the triaxial accelerometer.

4.2.2 FIELD TEST NO. 2 (FAU, BOCA RATON, FL)

In actual field conditions, the generation of vibration during the installation of the steel casing would be significantly similar to a seismic condition generated by blasting or a natural geological event. Seismic methods involve the measurements of the travel-time of seismic waves from the surface, or generating source, to interface geophones. The travel-time would be controlled by the soil conditions, as the seismic velocity in the soil is a function of its elastic constant and density. However, the soil conditions in this investigation were site-specific, and no extensive processing of this data was done beyond what was needed to meet the objectives of the project. When evaluating the concrete strengths, these recordings enabled the correlation of possible damage with the ensuing peak particle velocities. These vibrations are those experienced by a freshly

poured concrete drilled shaft from the vibration due to the installation of a neighboring drilled shaft.

4.2.2.1 SITE PREPARATION, PROCEDURES, AND MONITORING OF GROUND VIBRATION

To reproduce a complete picture of the vibration levels, that would be experienced by a nearby freshly poured shaft during the installation of a neighboring shaft, a 'virtual shaft' was constructed in the field. The 'virtual shaft' enabled the placement of eight geophones at varying depths. The depth of the geophones varied in five feet increments from 0-40 feet in a circular array around the vibrating steel casing at nine feet of $3D$, where D is the diameter of the vibrating steel casing, Figure 4.2. The bored holes were cased with PVC pipes to withstand caving of the surrounding soil mass. The geophones were lowered onto their intended depths and restrained, ensuring proper contact with the surrounding soil.

The same steel casing and vibrating hammer, as used in Field Test No. 1, were utilized in this test, on January 9, 98. The actual location of some of the field geophones did not meet the intended depths due to backwash (approximately five feet) in the PVC pipes at depths greater than 10 feet; however, the variation in location was not believed to be a detriment. The measurement points in the 'virtual shaft' were sufficient to provide an accurate account of the vibrations experienced at the recording depths during the installation of the steel casing. The depth locations, 5 and 40 feet, were measured twice; these two points indicated the upper and lower limits of the location of the geophones. During the installation, the remaining geophones in the circular array provided

intermediate data between the upper and lower limit points. However, the geophones 92 to 97 were at distances greater than 3D for the second drilled shaft (vibrating casing).

The sampling rate was 200 samples per second using the data acquisition system. The recordings were conducted in series, one geophone at a time in one-second intervals. The steel casing was centrally located in the center of a nine feet diameter circle bounded by the eight boring holes. Vibration of the steel casing was increased from 2000rpm at 20 feet to 3000rpm up to a depth of 40 feet. This procedure was repeated twice to validate the data.

Vibrations were recorded on a multichannel seismograph at a rate of sampling equal to 300 sample/seconds. Although it was not particularly high, the rate of sampling and the sequence of signal recording would correspond with the nature of the induced vibrations. The generated waveforms were supposed to be sinusoidal with frequencies ranged from 20 to 40 Hz. The data acquisition system collected a time-history record of one second length for each channel. The recording process was in a sequential order, which means the second time history record for the same channel would be collected after eight seconds. For three consecutive time-history records from each channel, the vibration had to last for about 24 seconds.

4.2.2.2 DRILLED SHAFT CONFIGURATION AND INSTALLATION PROCEDURES

The configurations of the shafts installed are shown in **Figure 4.3**. Two different diameter shafts were used, i.e. 24" and 36". The instrumentation of the shaft would provide a record of the levels of vibrations that the freshly poured drilled shafts

experience. The layout of a single shaft set up is shown in **Figure 4.4**. A set comprised two shafts, one of $\phi 24''$ and the other of $\phi 36''$. The larger diameter shaft $\phi 36''$ was reinforced, and designed to duplicate actual construction conditions. The purpose of the smaller shaft was to determine the vibrations and their possible effect on the integrity and strength of a shaft installed within a $3D$ distance from the vibrating source, where D is the diameter of the vibrating source, in this case the steel casing.

Installation Procedure in the Field:

1. Set up of equipment and instrumentation.
 - a. Geophones.
 - b. Temperature sensor.
 - c. PVC tubes.
 - d. Vinyl ties.
 - e. Protecting Foam.
 - f. # 9 Re-bars.
 - g. # 3 Ties.
2. Construct steel cages.
3. Install instrumentation as specified by design and manufacturer recommendations.

Field Installation of the Steel Casings.

1. Obtain digging permit for the site.
2. Clear surface debris.
3. Record dimensions of the steel casing.
4. Drive steel casings to desired depth.

5. Check levels of the casing.
6. Auger soil from the shafts.
7. Record soil material.
8. Check depth of drilling.
9. Continue to check the shaft for desired length.
10. Remove any foreign material introduced into the shaft.

The steel casing was then extracted from the ground using the same vibration technique. Time-history records were gathered during withdrawal in the same manner as for the driving stage. The extraction process was accomplished by connecting one side of the casing to the vibrator using a steel cable (**Figure 4.4**). Although this method is a common practice when extracting the steel casings, vibrations generated from this operation may not necessarily be similar to those obtained during the driving process. The main reason for this discrepancy is the method by which the casing was driven and extracted. As can be seen from **Figure 4.4**, the casing was pulled out in an asymmetrical manner. This mode of vibration application resulted in generating vibration components that were different from those produced during driving.

4.2.3 FIELD TEST NO. 3 (DELRAY, FL)

A total of eight drilled shafts were installed at the site allocated by FDOT on the right of way of the off-ramp from I-95 onto Linton Blvd heading west. A layout of the shafts is shown in **Figure 4.4**. Installation and instrumentation of the drilled shafts is shown pictorially in **Figures 4.4 to 4.17**.

4.2.4 FIELD TEST NO. 4 (CLEARWATER, FLORIDA)

The 2-hr test data was not monitored at the Delray Site due to equipment problems. Therefore, the testing was done for a single shaft set at the Clearwater site, to save costs, the installation procedures were similar to those carried out in Field Test No. 2 with the use of a single drilled shaft set. The pictorial presentation is in Figures 4.12 to 4.20. This site differed somewhat in terms of geology as demonstrated from the SPT blow counts, however, the difference was minimal and for practical purposes the comparison between vibration levels can be considered reasonable.

4.2.5 VIBRATION DATA

The peak particle velocity(PPV) vs time records were obtained for the Field Tests No.3 and No.4 at Delray Beach and Clearwater, respectively. See Appendix B.

4.3 TESTING OF CONTROL CONCRETE CYLINDERS

To evaluate the effect that vibrations had on the freshly poured concrete, samples were taken from the concrete poured in the field. The cylinders were cured and tested at fourteen and twenty-eight days. The compressive strength results are shown in Appendix C.

4.4 NON - DESTRUCTIVE TESTING

4.4.1 PILE INTEGRITY TESTING (PIT)

The PIT method uses the compressive wave stress signal generated by a hand held hammer striking the top of the drilled shaft to determine the integrity of the shaft. Non-uniformity in the foundation or the shaft toe will reflect the stress wave; this reflection can be observed at the top by using a combination of the Pulse Echo Method (PEM) and Transient Response Method (TRM). For further details on the PIT method of NDT see Appendix D.

The PIT results served to verify the actual concreted depth of the shafts. Although, no cracks or major defects are identified with the PIT, the limitation of the theory does not make this conclusive.

4.4.2 CROSS-HOLE SONIC LOGGING (CSL)

Sonic logging is used to detect defects in concrete structures. Some of the anomalies that can be detected in a structure and crucial in this study are the segregation of concrete, cracks due to shrinkage, and non-uniformity of shaft due to collapsing of the wall after withdrawing of a temporary casing. Pulses generated by the ultrasonic pulse generator are converted into ultrasonic waves by the transmitter probe and received by the receiver probe. After conditioning, signals are digitized and stored. Additional details of the operation and procedures are shown in Appendix E.

4.5 CONCRETE CORING OF THE DRILLED SHAFTS AND COMPRESSION TESTING

One of the additional tests used to determine the integrity of the drilled shafts, was the compression testing of cored concrete samples. The field shafts were cored to obtain samples at various depths by Law Engineering, Miami Lakes, for the Delray Site,

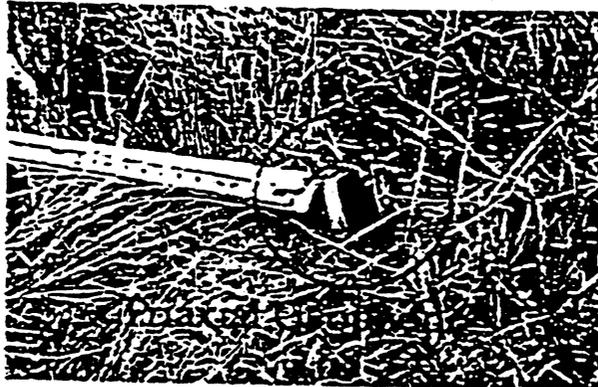


Figure 4.1a. Testing setup for each borehole.

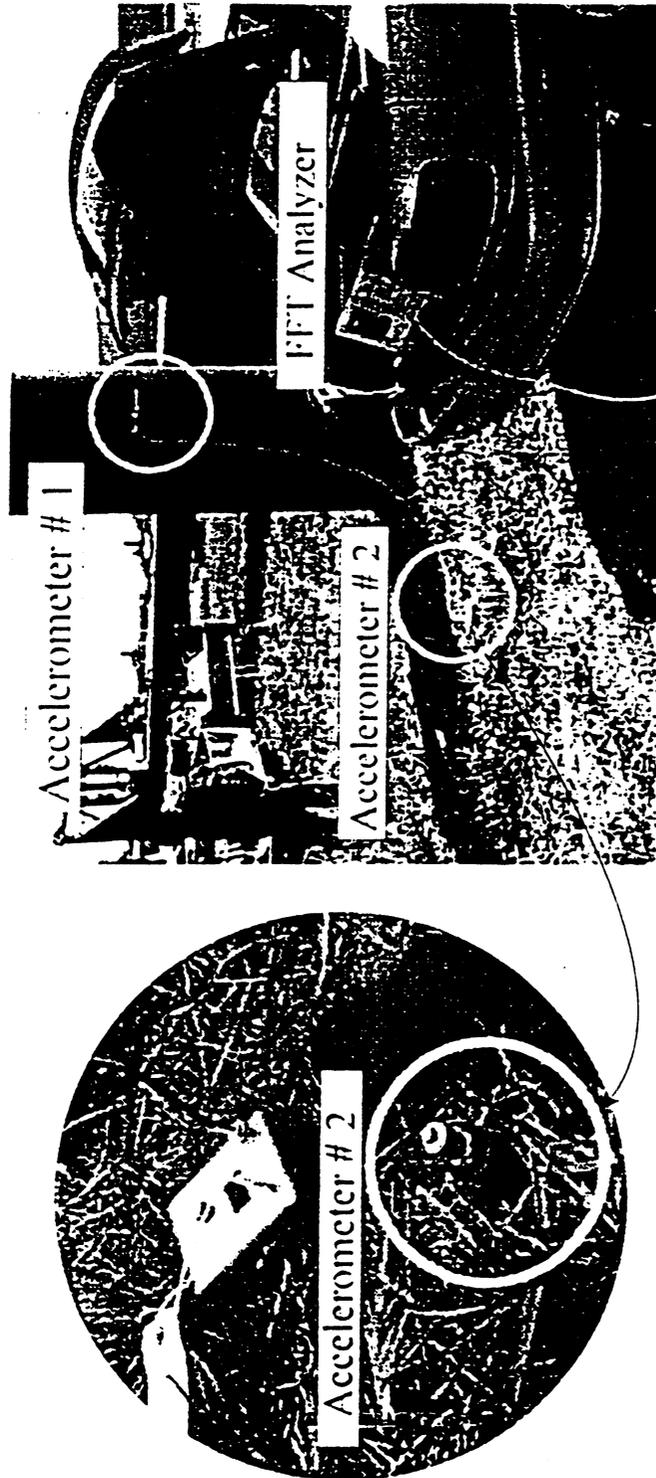


Figure 4.1b. Recording steel casing and ground responses using FFT Analyzer.

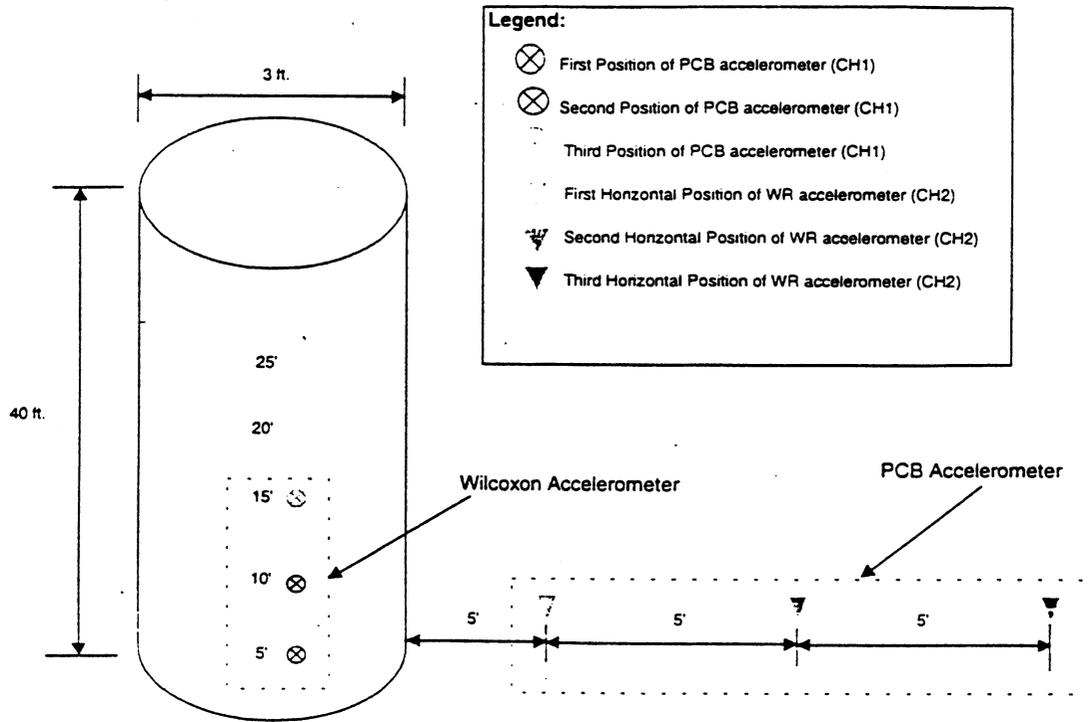


Figure 4.1c Accelerometer Layout

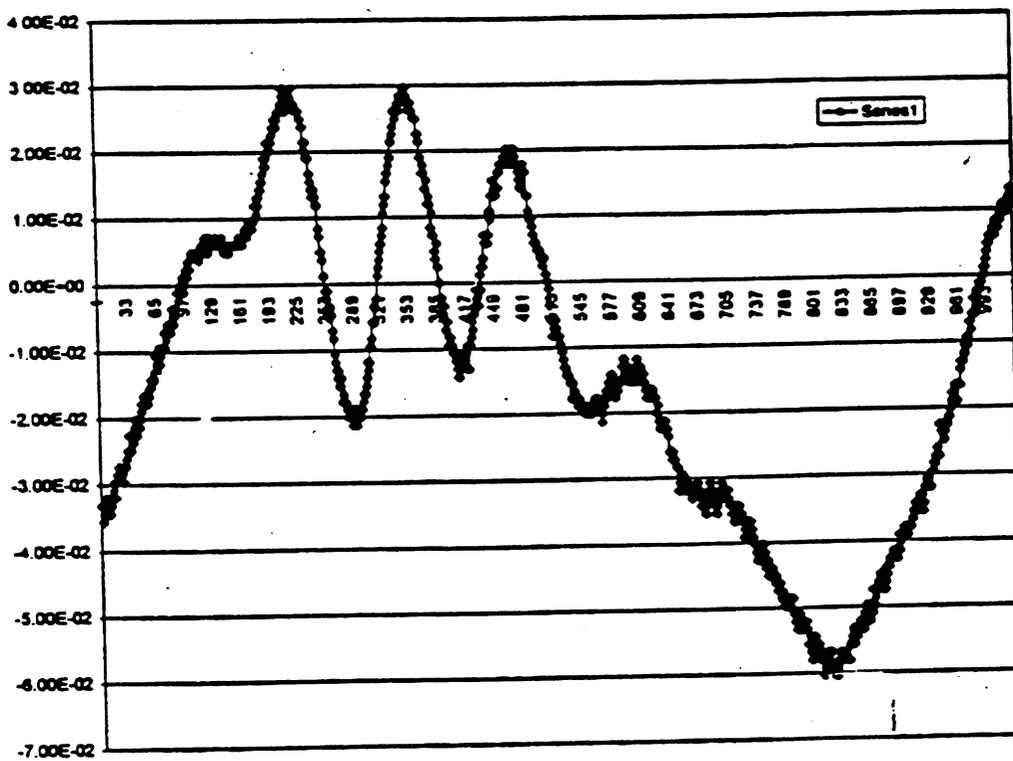
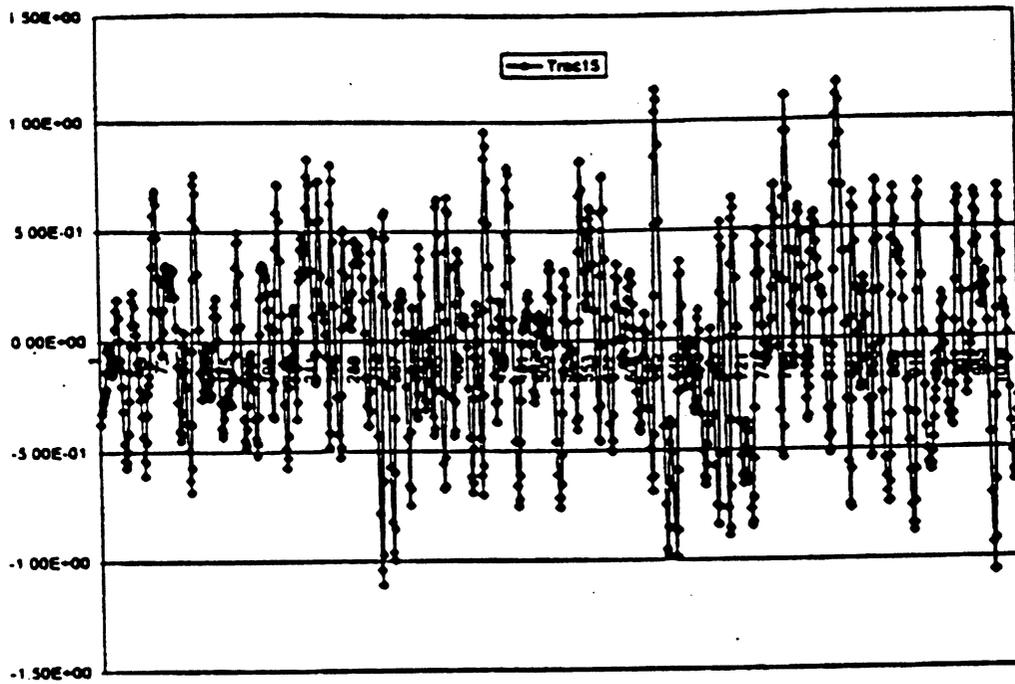


Fig.4.1D. Traces 15 and 16 showing accelerometer #1 for the vibrating casing, and ground surface accelerometer #2 at 15' away from the casing

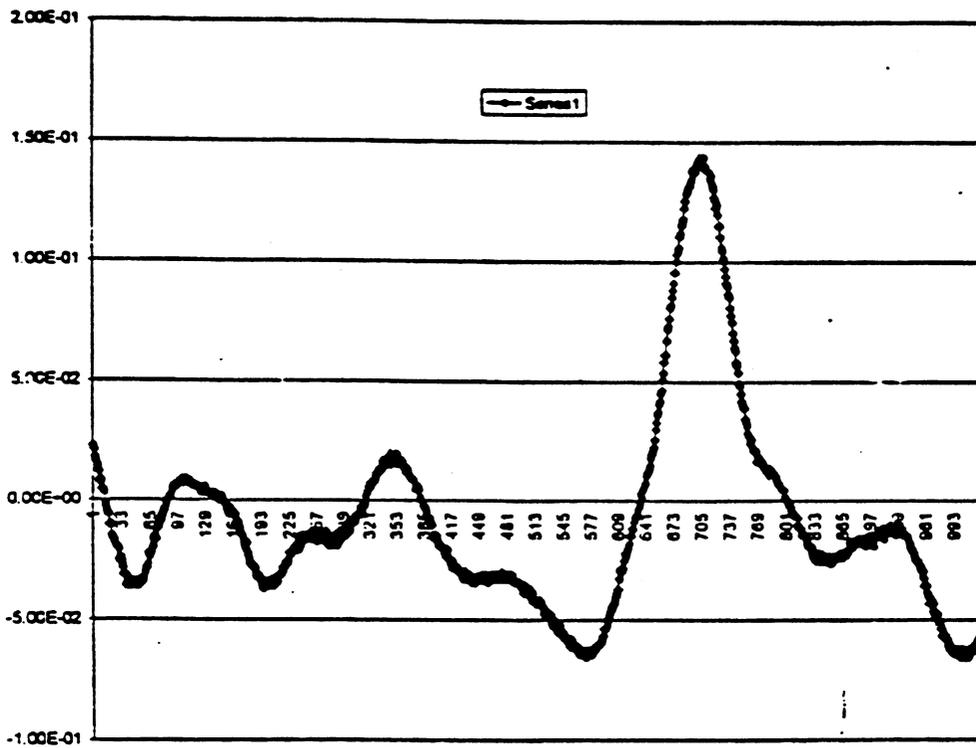
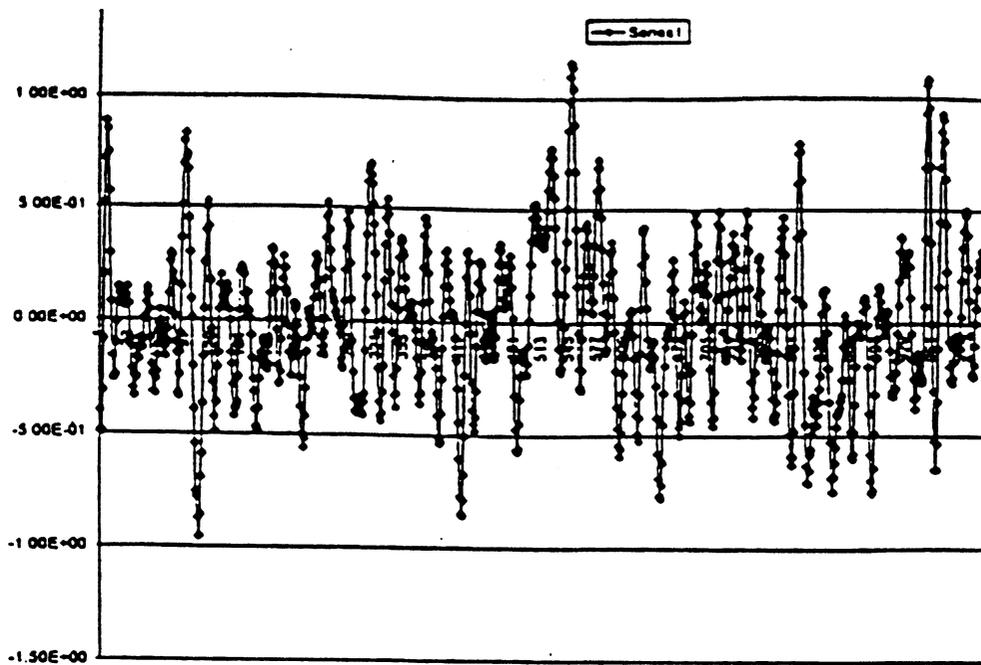


Fig.4.1e. Traces 7 and 8 showing accelerometer #1 on the vibrating casing, and the ground surface accelerometer #2 at 10' away from the casing

II. 1/9/1998 at FAU Boca Campus

1- Site Investigation on 9/31/1997

2- Vibration Testing using 8 Geophones around two steel casing (See layout below)

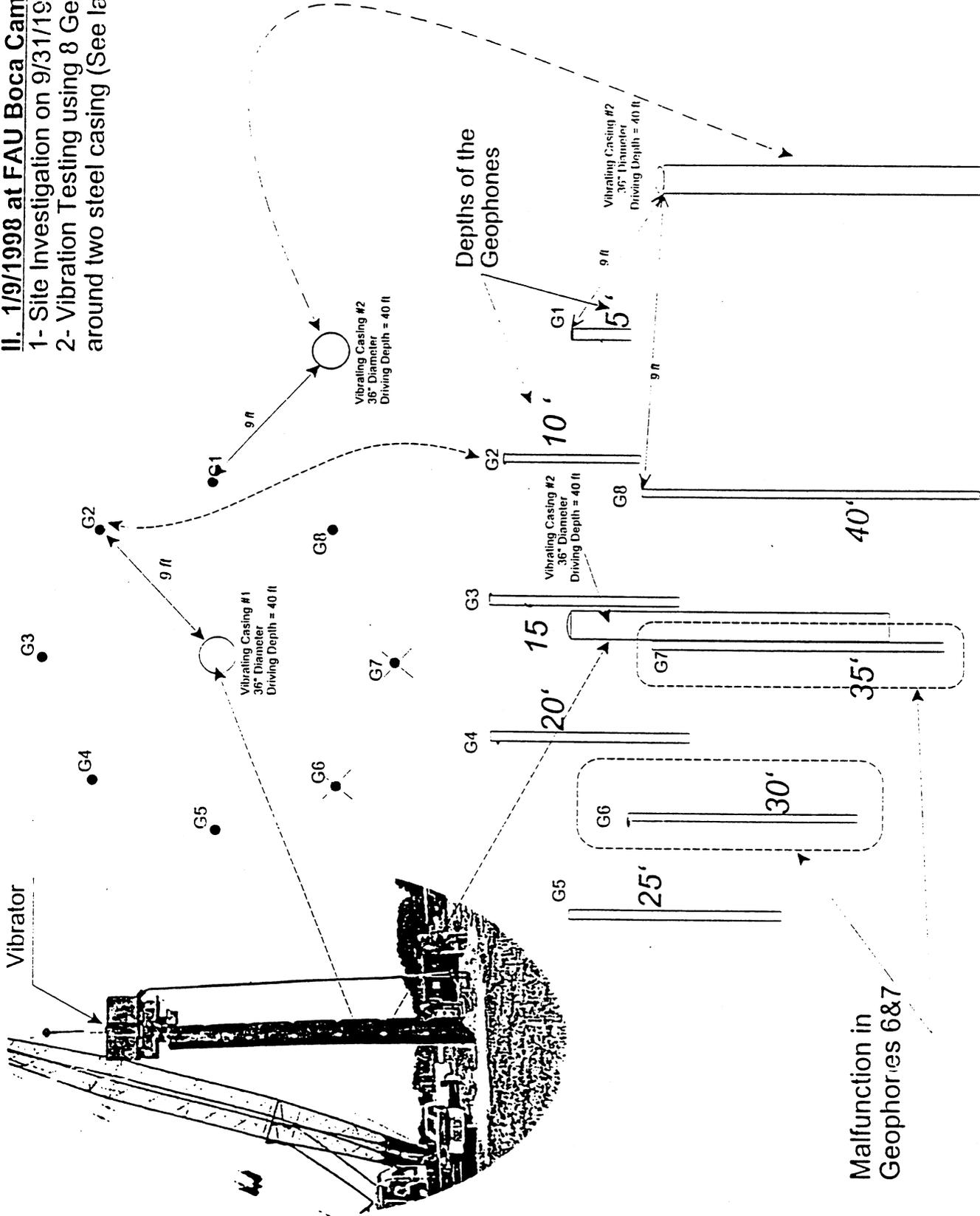
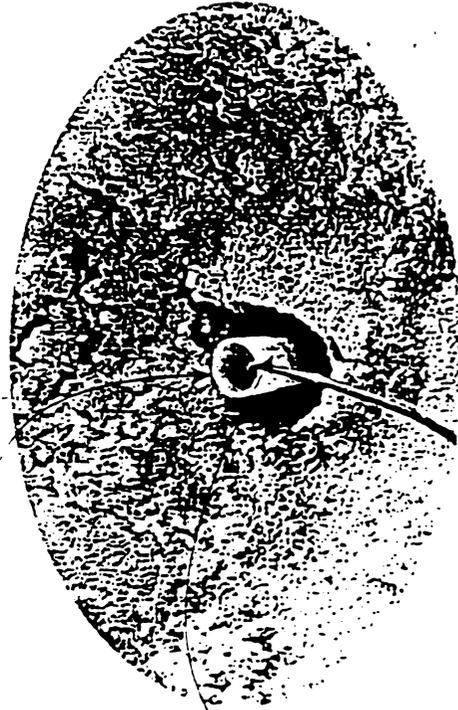
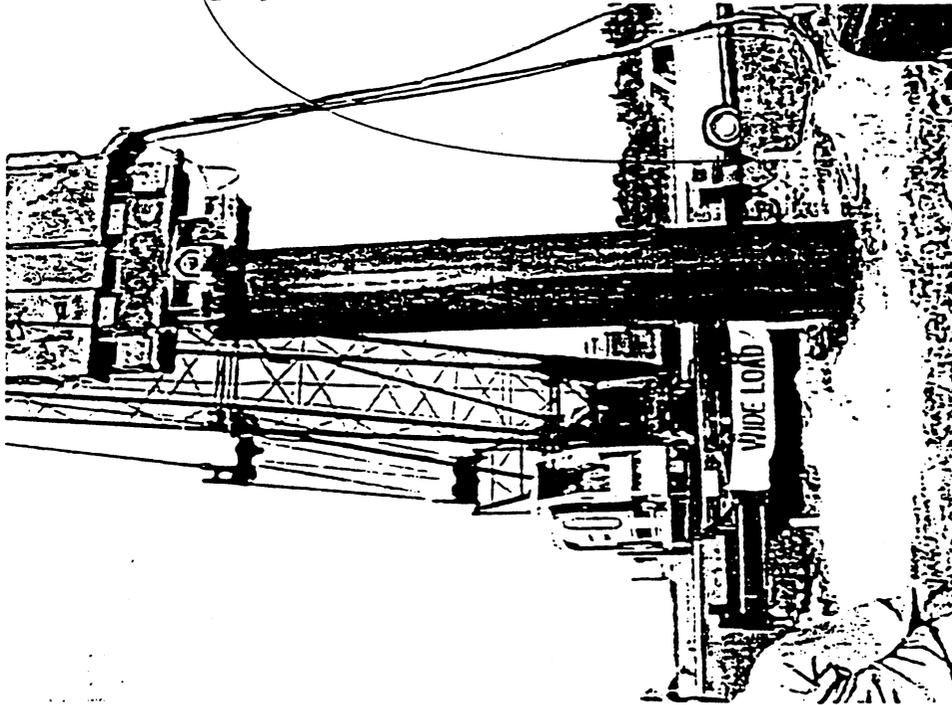


Figure 4.2 FAU Field testing showing the arrangement of the geophones and the driven steel casing.

PVC Tube inserted to a designated depth



Triaxial Geophone used in the study



Figure 4.3. Arrangement of geophones in PVC tubes at the FAU test site.



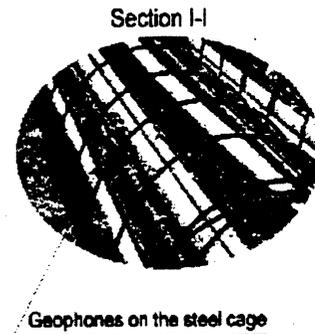
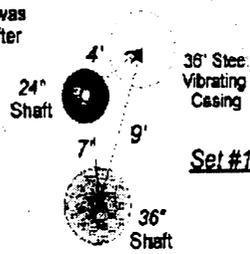
Figure 4.4. Extraction of the steel casing.

III- Delray Site (4/29/1998) (Right of way north west I-95 Linton Blvd.)

Vibrating Time	Set Number
4 hr	3
6 hr	1
12 hr	4
Control Setup	2

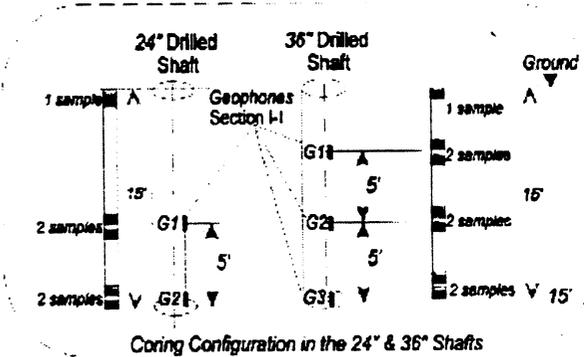
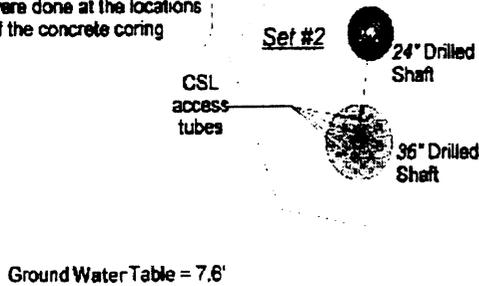
○ = Concrete Coring Location (Diameter = 3.5")
 CSL = Cross-hole Sonic Logging
 PIT = Pile Integrity Test
 Neutron log (For Porosity Measurements)
 Gamma-Gamma log (For Density Measurements)

Vibration was induced after 6 hr



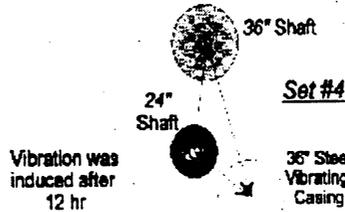
Both the Neutron log & Gamma-Gamma log were done at the locations of the concrete coring

Control Setup Shafts were not exposed to vibration



Ground Water Table = 7.6'

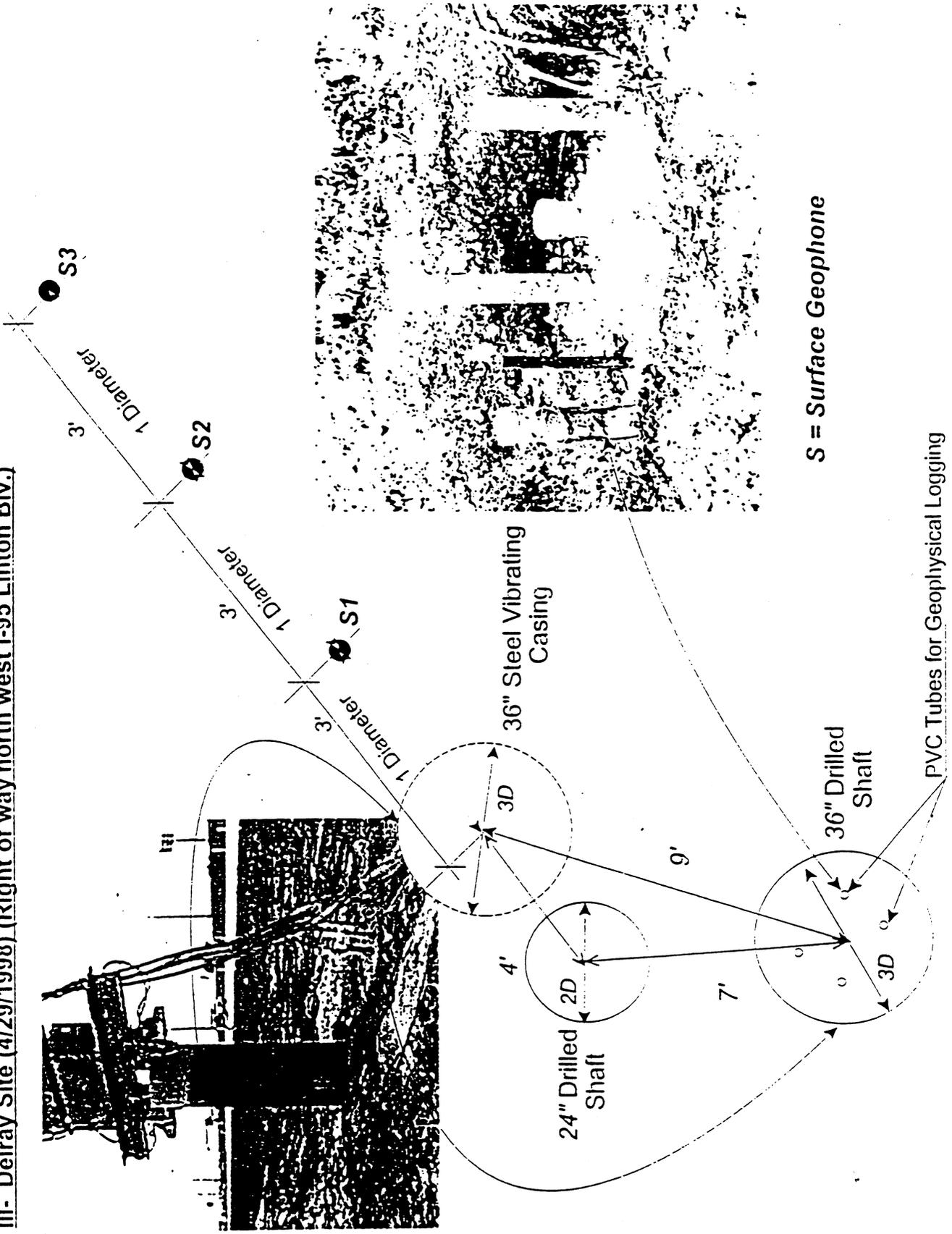
Vibration was induced after 4 hr



Ground Water Table = 4.3

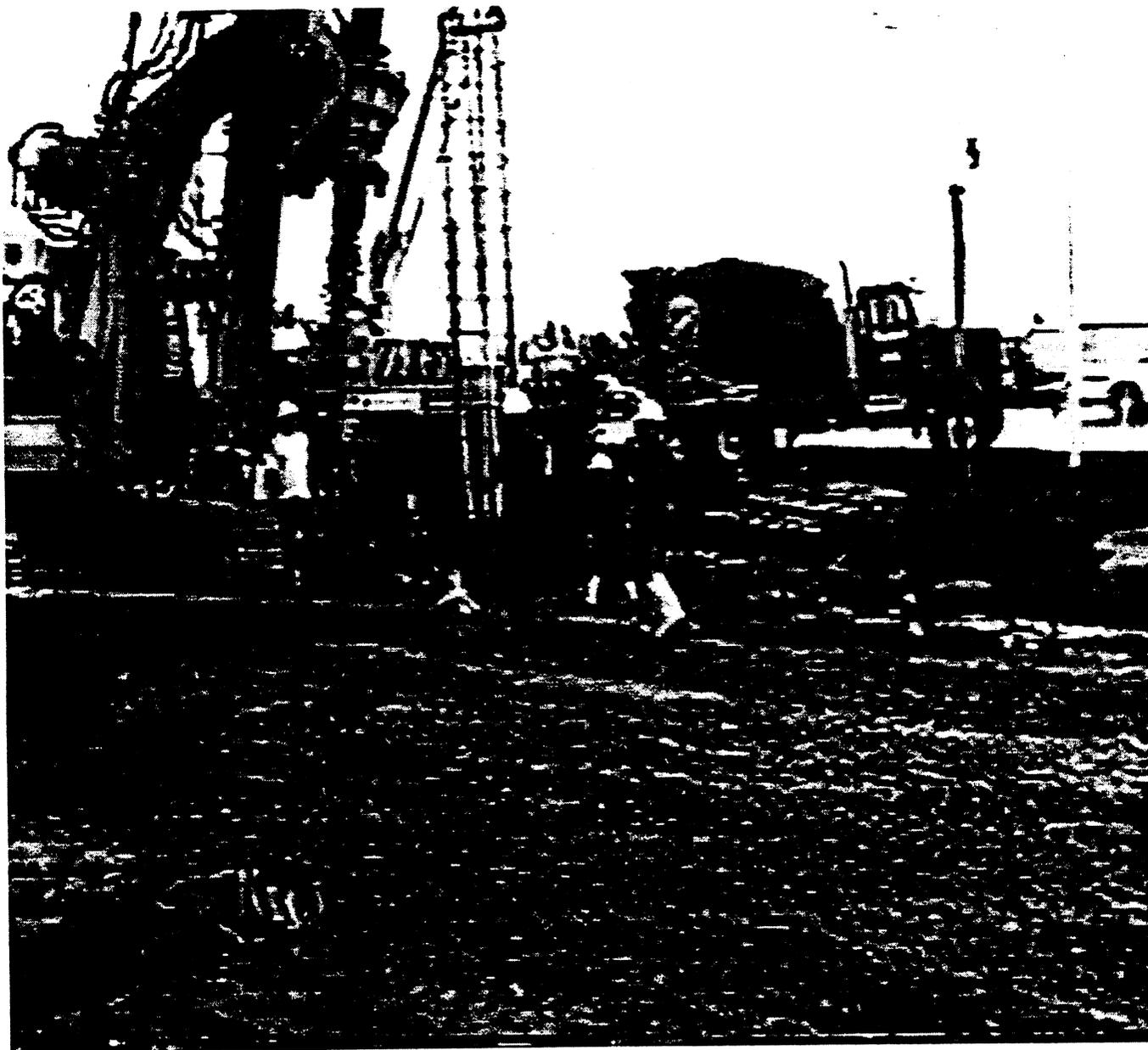
Figure 4.5 a. Configuration and instrumentation of the shaft at the Delray Site

III- Delray Site (4/29/1998) (Right of way north west I-95 Linton Blv.)



S = Surface Geophone

Figure 4.5b. Layout at the Delray Testing Site.



**Figure 4.7. Placing of steel reinforcement with instrumentation
Set no.4 (Delray Beach, FL)**



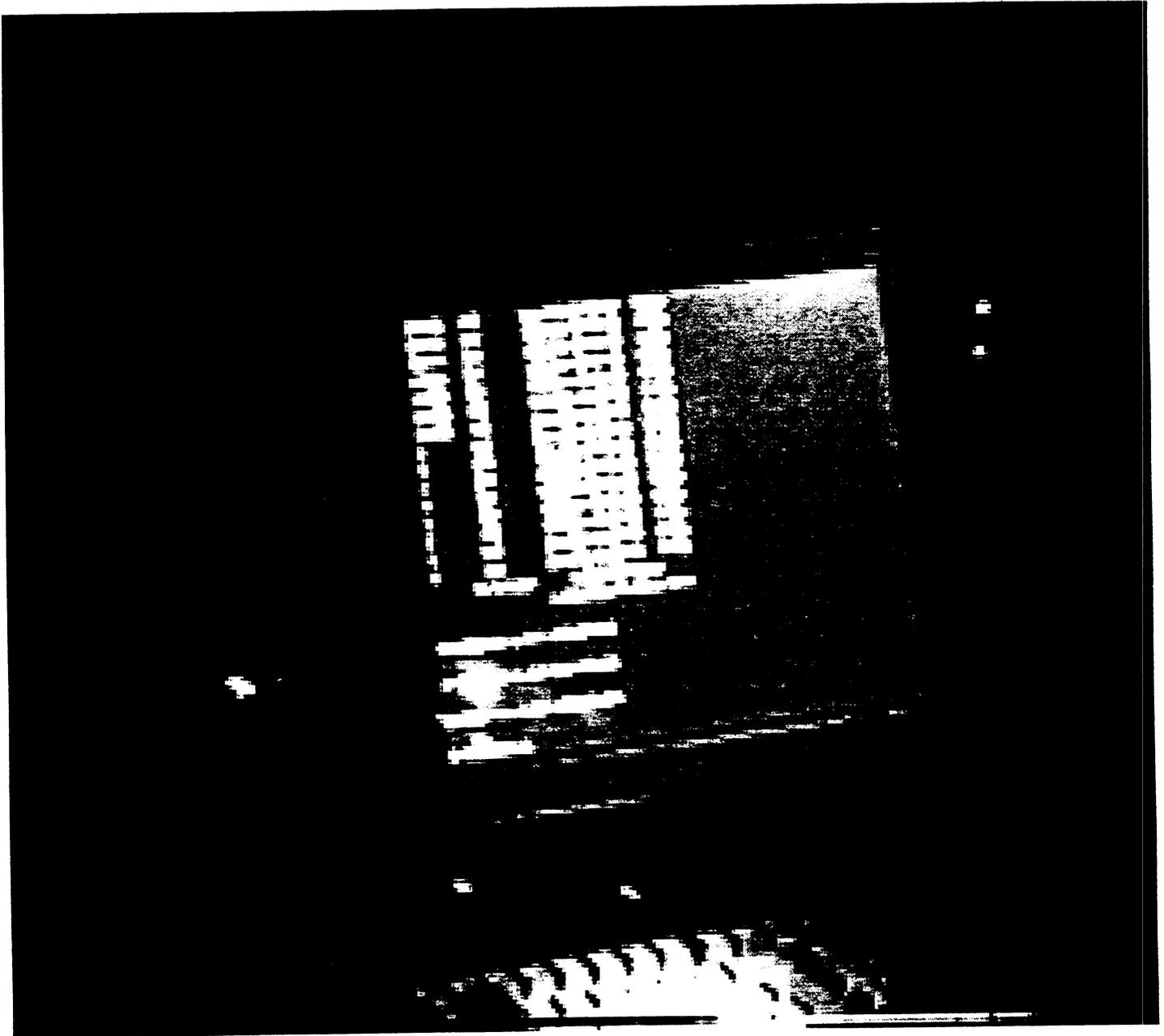
**Figure 4.8. Dropping the tremmie in the hole, Set no. 4
(Delray Beach, FL).**



**Figure 4.9. Pulling of steel casing after pouring of the concrete,
Set no. 4 (Delray Beach, FL)**



**Figure 4.10. CSL tubes and settlement plate, ϕ 36'' drilled shaft,
Set No. 1 (Delray Beach, FL).**



**Figure 4.11. Geophone data acquisition system
(Delray Beach, FL)**

IV- 9/10/1998 (Clearwater) Coastal Caissons

Only one set was tested
 The vibration was induced after 2 hr from
 construction
 two pile were used 3D and 2D

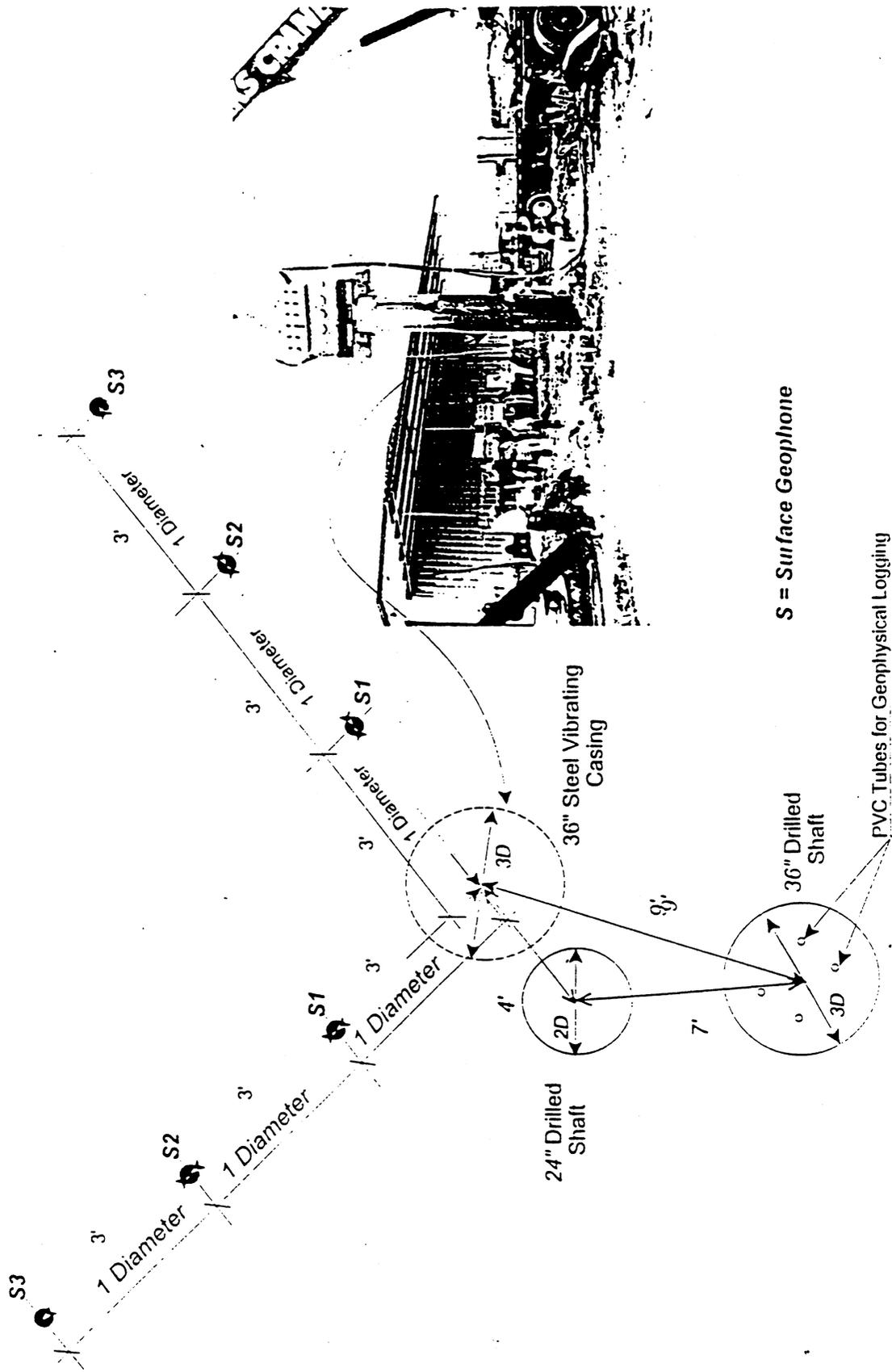


Figure 4.12 Layout at the Clearwater Test Site.



**Figure 4.13. Installation of the 24- inch diameter steel casing
(Clearwater, FL)**



**Figure 4.14. Installation of the 36-inch diameter steel casing
(Clearwater, FL)**

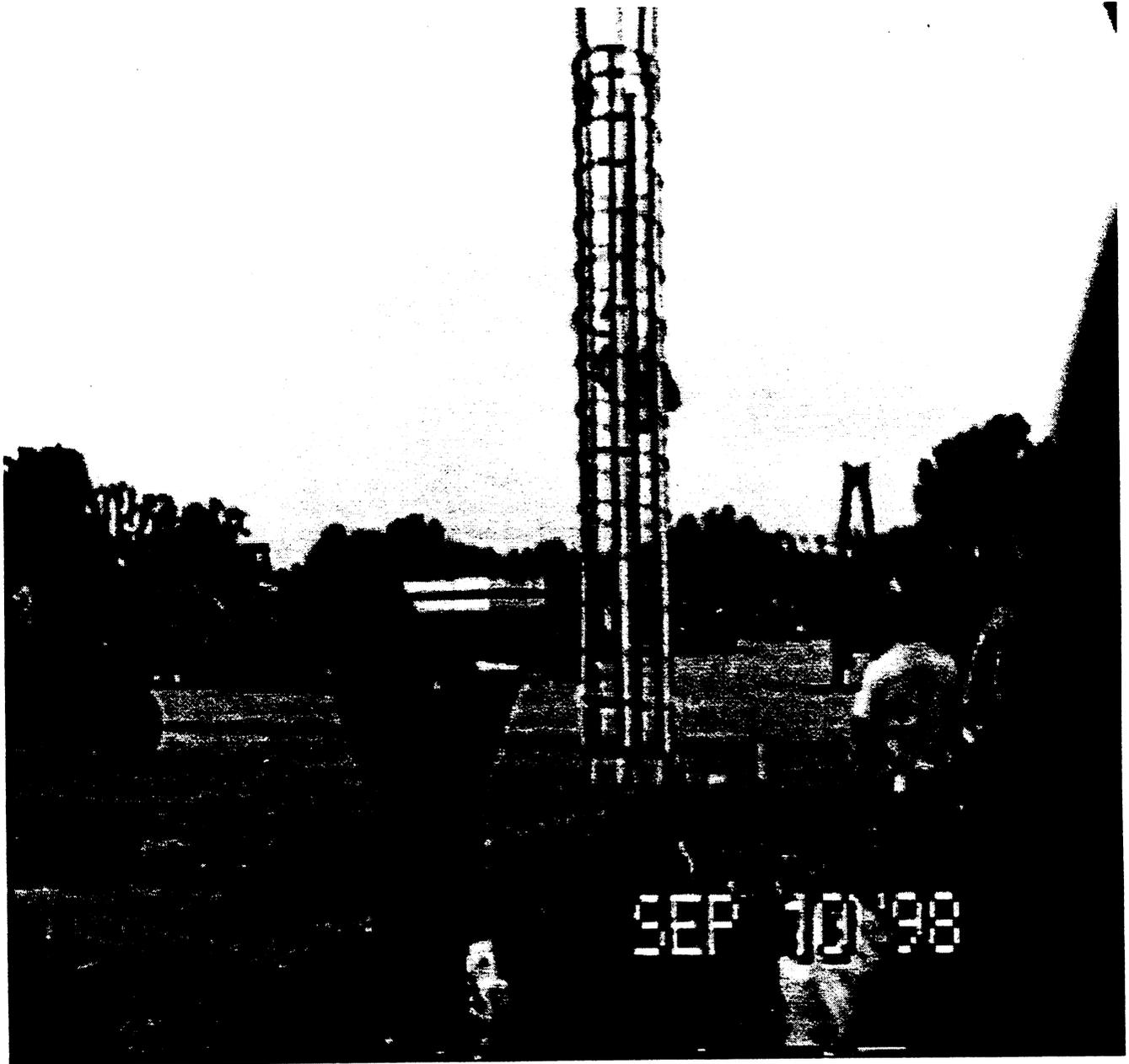


Figure 4.15. Steel cage with geophone and cross-hole sonic logging instrumentation (Clearwater, FL).

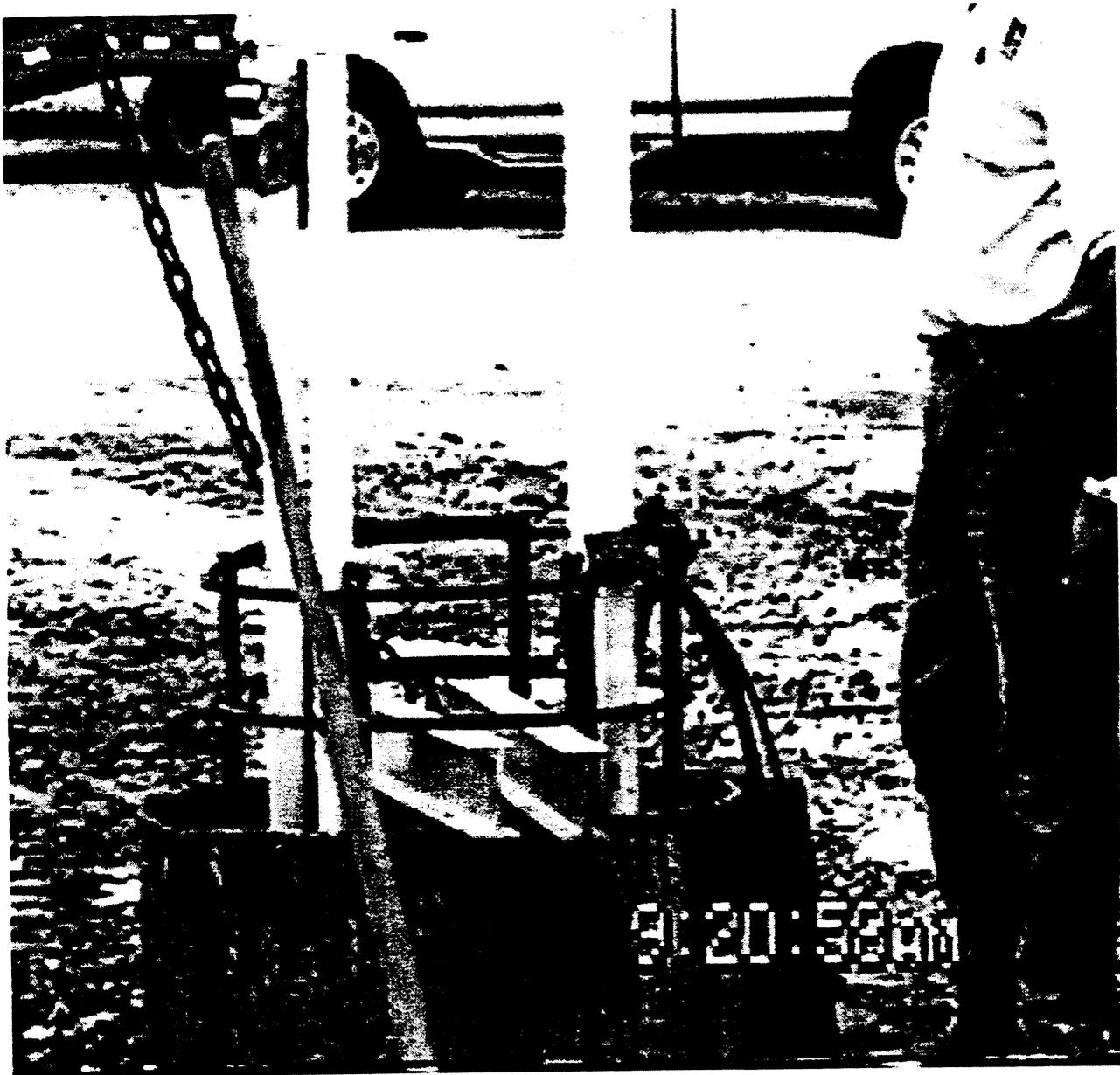
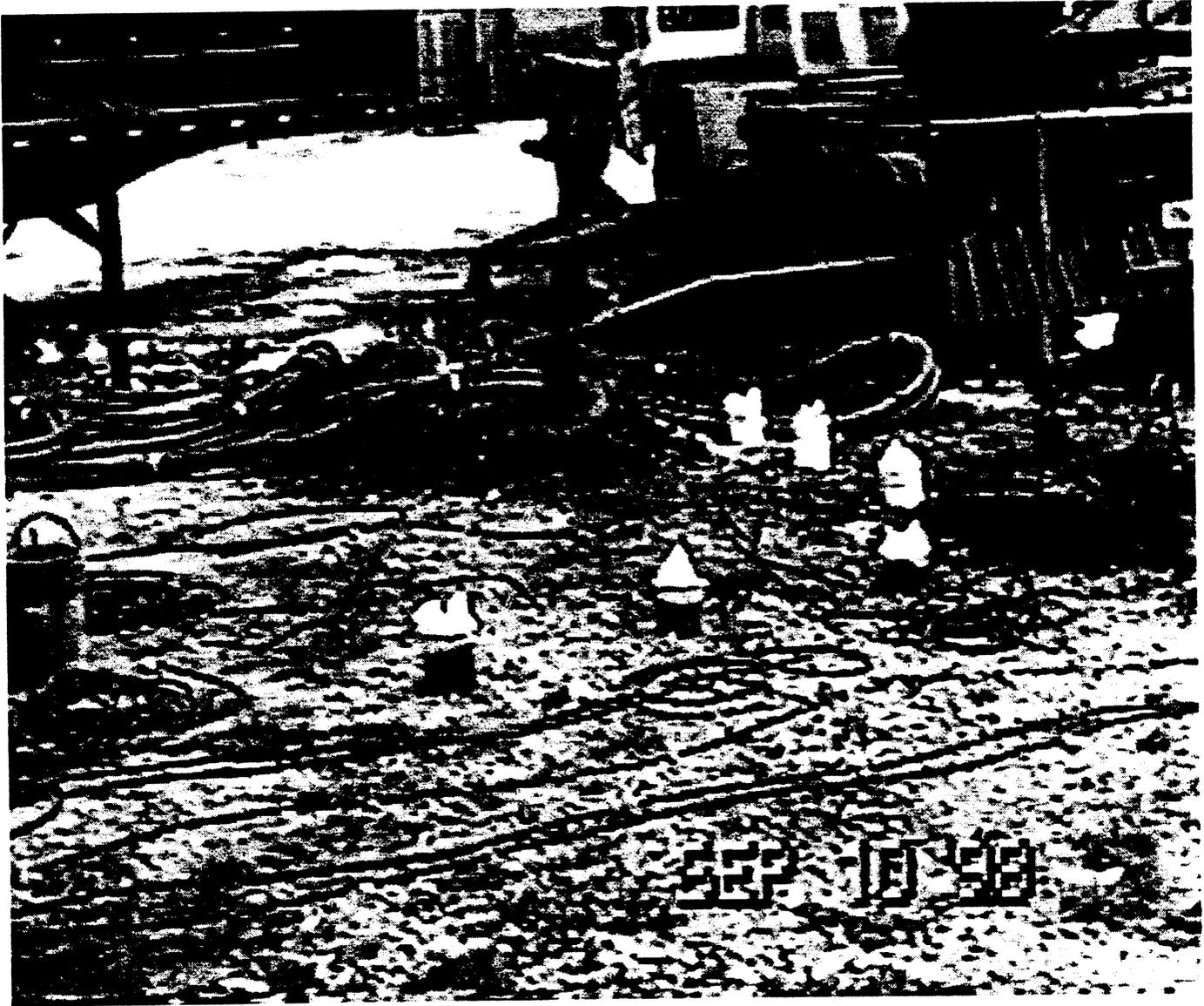


Figure 4.16. Placing the steel cage in the drilled shaft (Clearwater, FL).



**Figure 4.17. Vibration of casing adjacent to freshly poured shafts
(Clearwater, FL).**



**Figure 4.18. Surface placement of cylinders exposed to 2-hr vibration
(Clearwater,FL)**



Figure 4.19 PIT on shaft (Clearwater, FL)

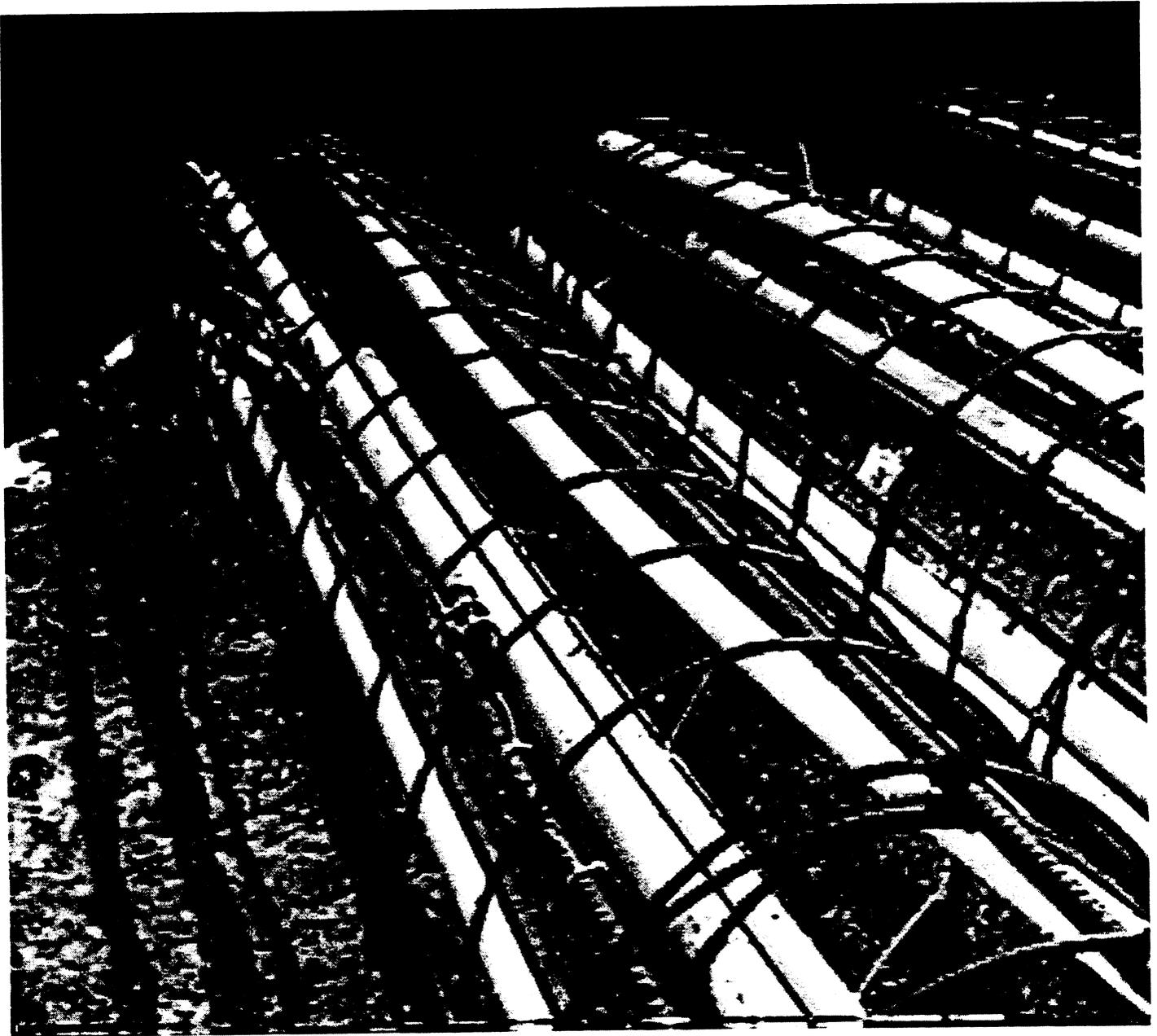


Figure 4.20. CSL tubes and geophones in place on the steel reinforcing cage.



Figure 4.21. CSL recording equipment (Delray Beach, FL).

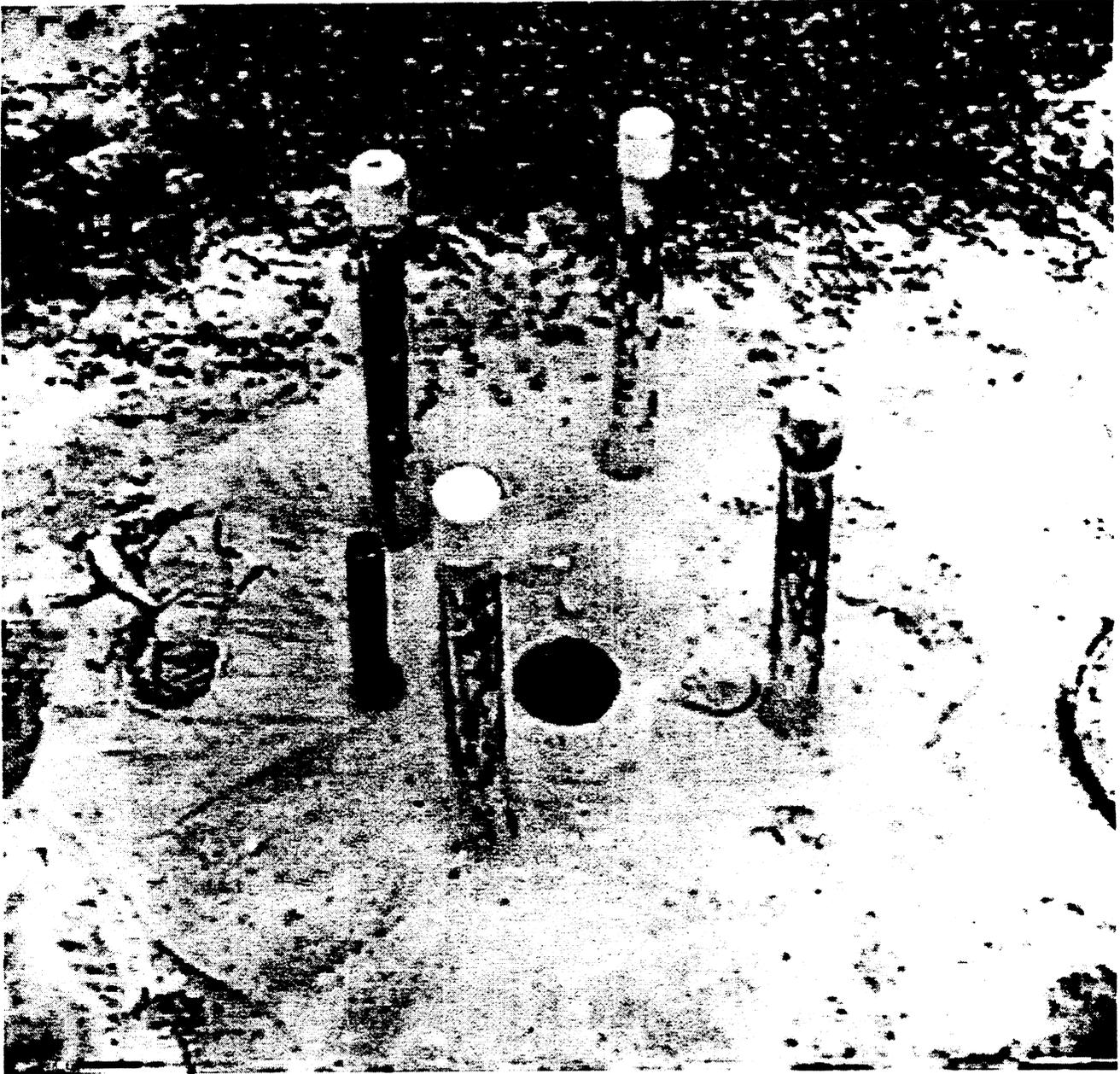


Figure 4.22. Cored drilled shaft (Delray Beach, FL).

CHAPTER V

ANALYSIS OF RESULTS AND DISCUSSION

5-1 Laboratory Test Results

Results from compressive strength tests showed considerable variations (Figures 5.1 to 5.10). These variations may not be significant if the results from one set are compared with those obtained from other sets for the same vibration amplitudes. Concrete samples subjected to vibration amplitudes of 0.5 in./sec did not show any trend when the samples were vibrated, at different time delay levels. As the amplitude exceeded 2.5 in./sec, a slight reduction was noticed in both the compression modulus and the compressive strength (Figures 5.11 to 5.13). The largest reductions in the stiffness and strength values were of concrete samples subjected to velocity amplitudes of about 7 in./sec. Data tables showing all the laboratory measurements of this segment are presented in Appendix A.

On a log-log scale, if tangents of the obtained function are extended, the intersection would produce a threshold velocity at which a noticeable change could be detected in the strength as well as the stiffness values. The threshold for the compressive strength was 6 in./sec and about the same value for the compression modulus. Accordingly, variations of the compressive strength with the velocity amplitude can be estimated using the following relationship:

$$f'_{c_{vel}} = (f'c) \cdot e^{-0.03 v} \quad (5.1)$$

where $f'_{c_{vel}}$ = compressive strength at any velocity amplitude (psi)

$f'c$ = compressive strength at 28 days (psi)

v = velocity amplitude (in./sec)

The following relationship represents a reasonable estimate for the variation in the compression modulus with velocity amplitude:

$$E_{c_{vel}} = (E_c) \cdot e^{-0.02 v} \quad (5.2)$$

where $E_{c_{vel}}$ = compression modulus at any velocity amplitude (psi)

E_c = compressive strength at 28 days (psi)

v = velocity amplitude (in./sec)

However, it must be noted that the wave forms used in this phase of the testing program were compression waves. Thus, the velocity amplitudes in **Equations 5.1 and 5.2** represent the compression wave. In a real field situation, the modes of vibrations generated from a typical drilled shaft construction are predominantly shear waves (**Figure 5.14**). Therefore, the threshold velocities suggested in this study may not be attainable in the field since the generated shear waves are transferred to the adjacent freshly poured concrete through the shear friction displacement of the casing relative to the soil. Even if full contact was assumed between the soil and the adjacent casings, the amplitudes of the shear waves would have damped based on the material and geometrical damping of the soil surrounding the casings. This hypothesis was proved true during field testing on full scale drilled shafts.

5-2 Analysis of Field Results

5-2-1 FAU Field Test # 1

The first field testing at **Florida Atlantic University** took place on **December 3, 1997**. The objective for this segment of field testing was to drive a 36" diameter with 40 foot length steel casing into sandy soil layers in order to monitor the ground response. A total of seven holes

Compressive Strength
Particle Velocity Range = 2.584-3.230 in/sec (20%)

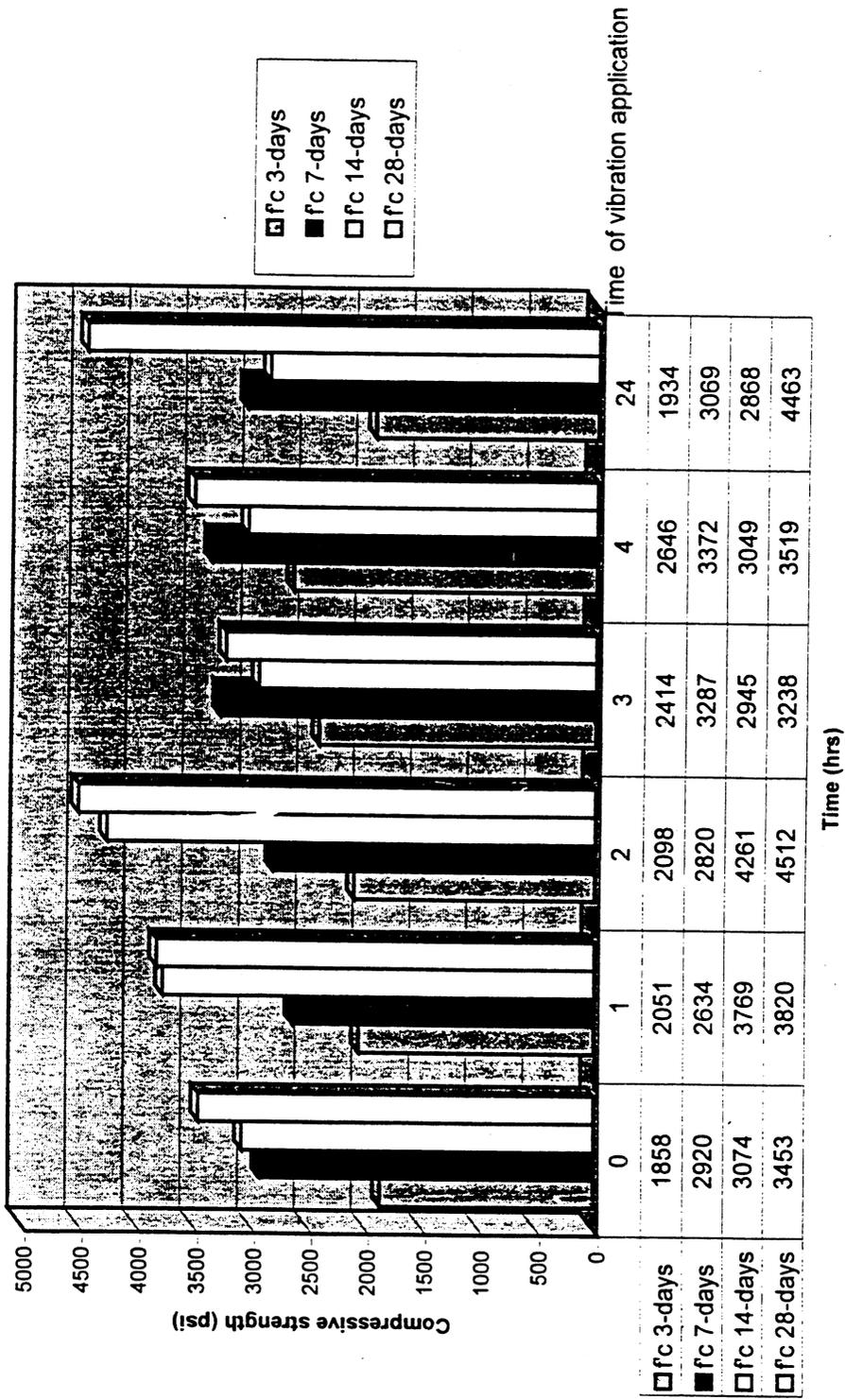


Figure 5.2: Compressive strength of concrete samples subjected to acceleration of 14 g.

**Compressive Strength
Particle Velocity Range = 5.537-8.305 in/sec (40%)**

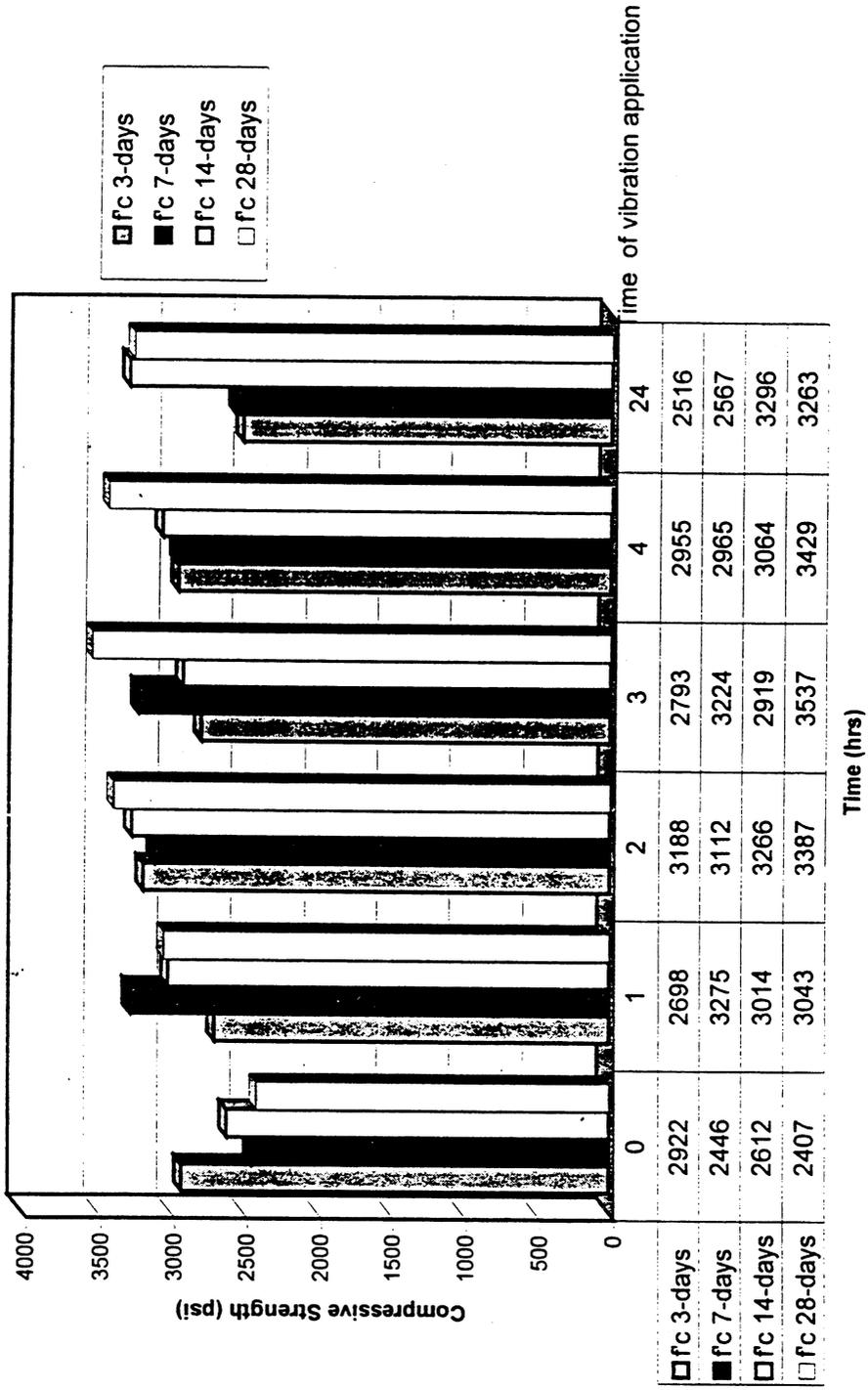


Figure 5.3: Compressive strength of concrete samples subjected to acceleration of 30 g.

Compressive Strength

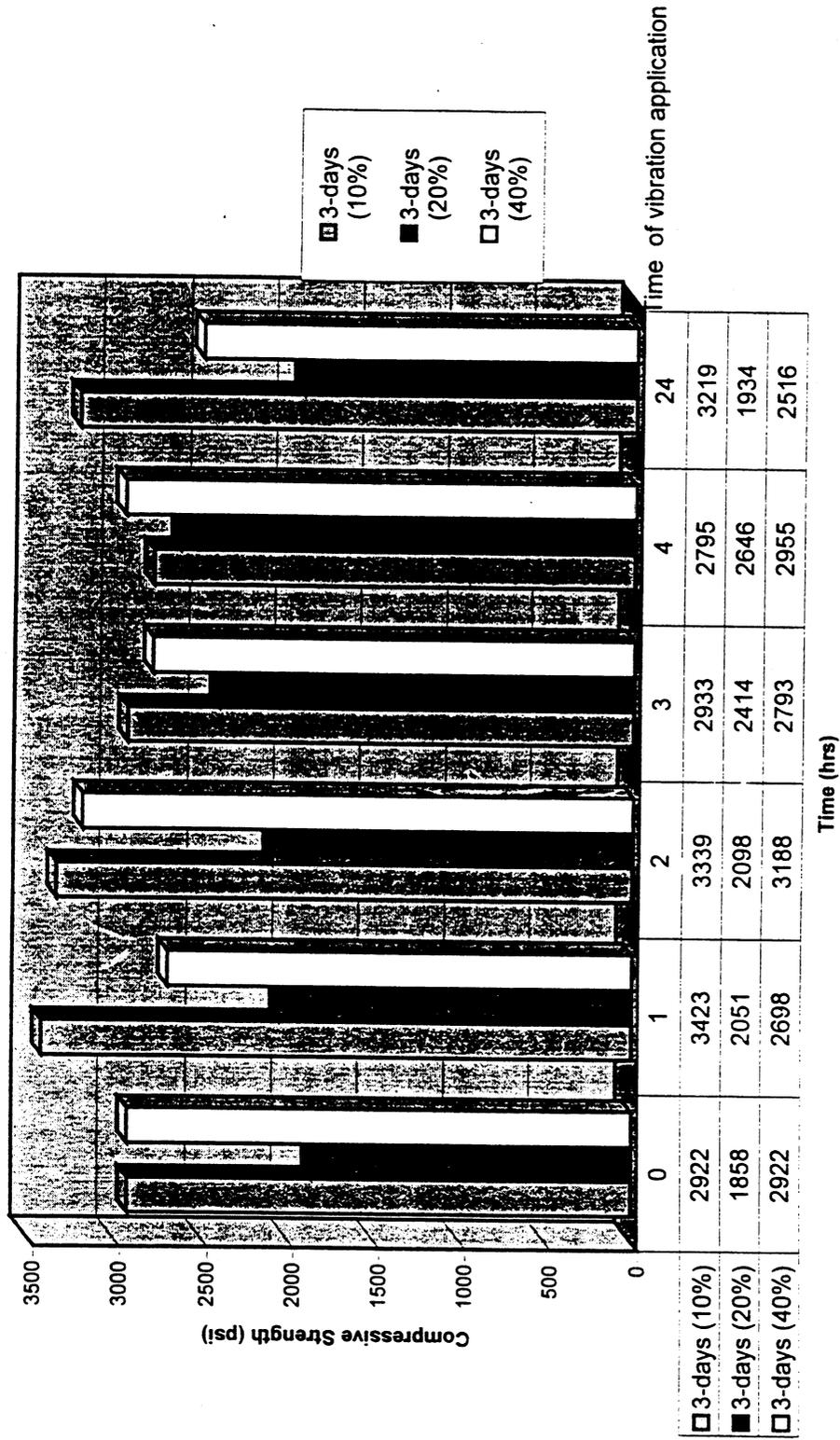


Figure 5.4: Compressive strength of concrete samples after 3 days

Compressive Strength

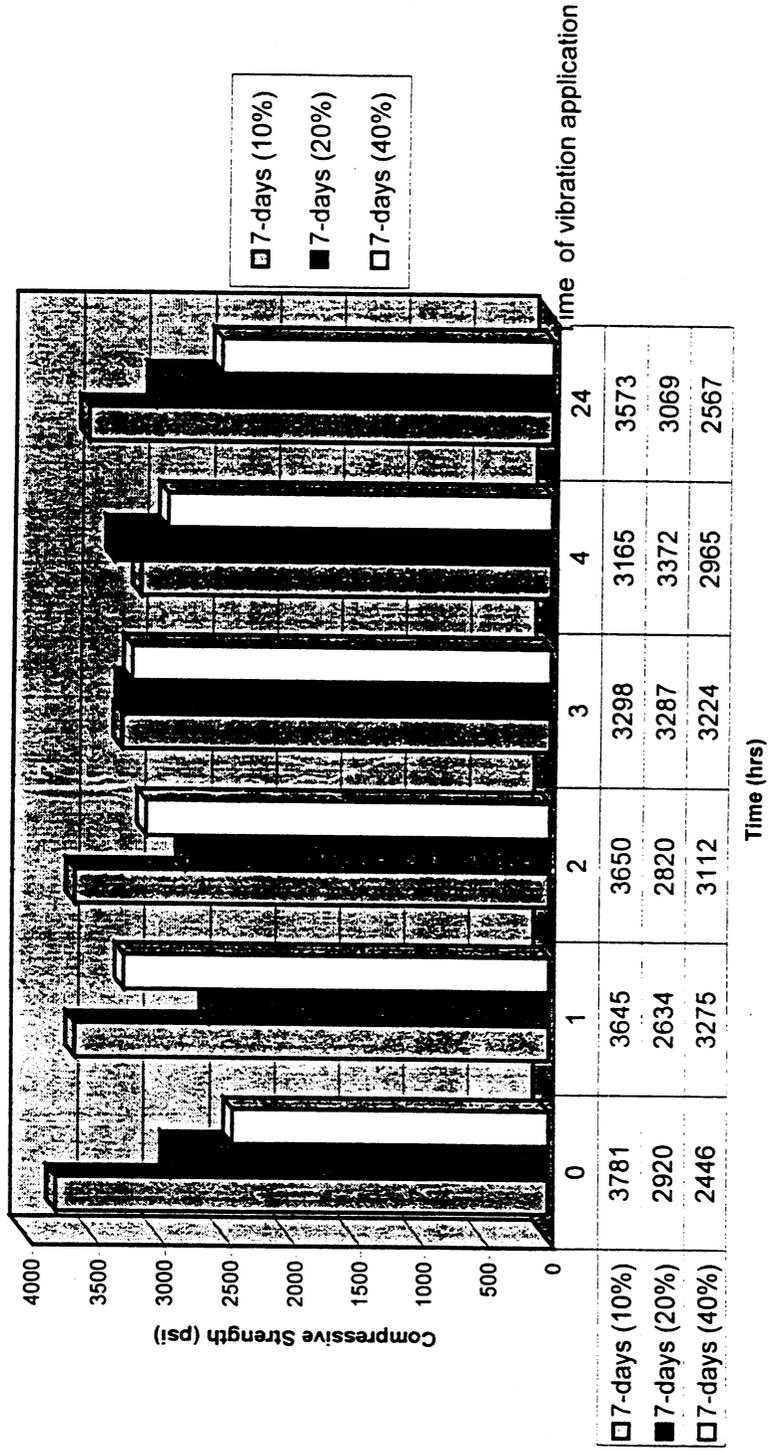


Figure 5.5: Compressive strength of concrete samples after 7 days

Compressive strength

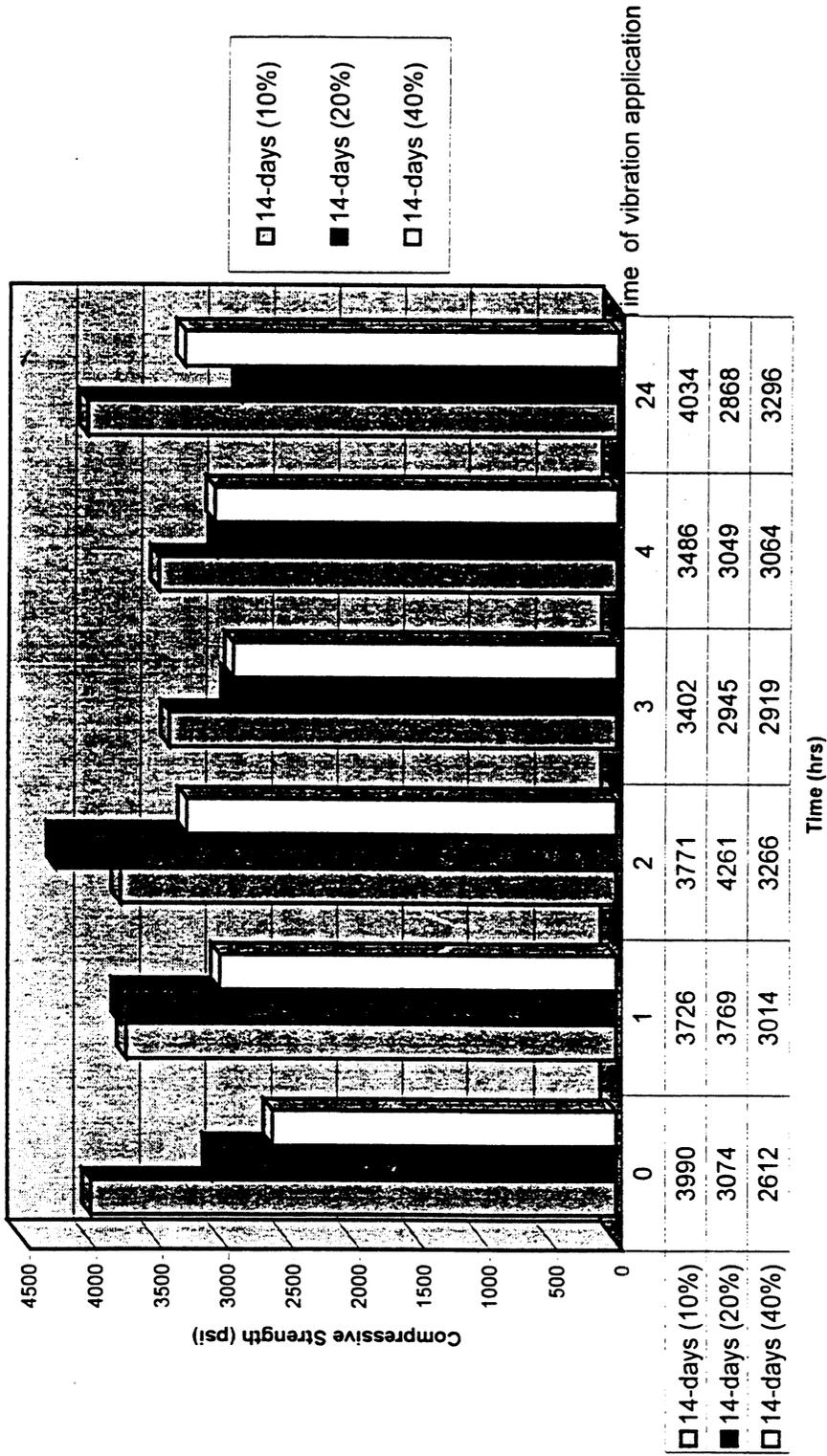


Figure 5.6: Compressive strength of concrete samples after 14 days

Compressive Strength at 3-days

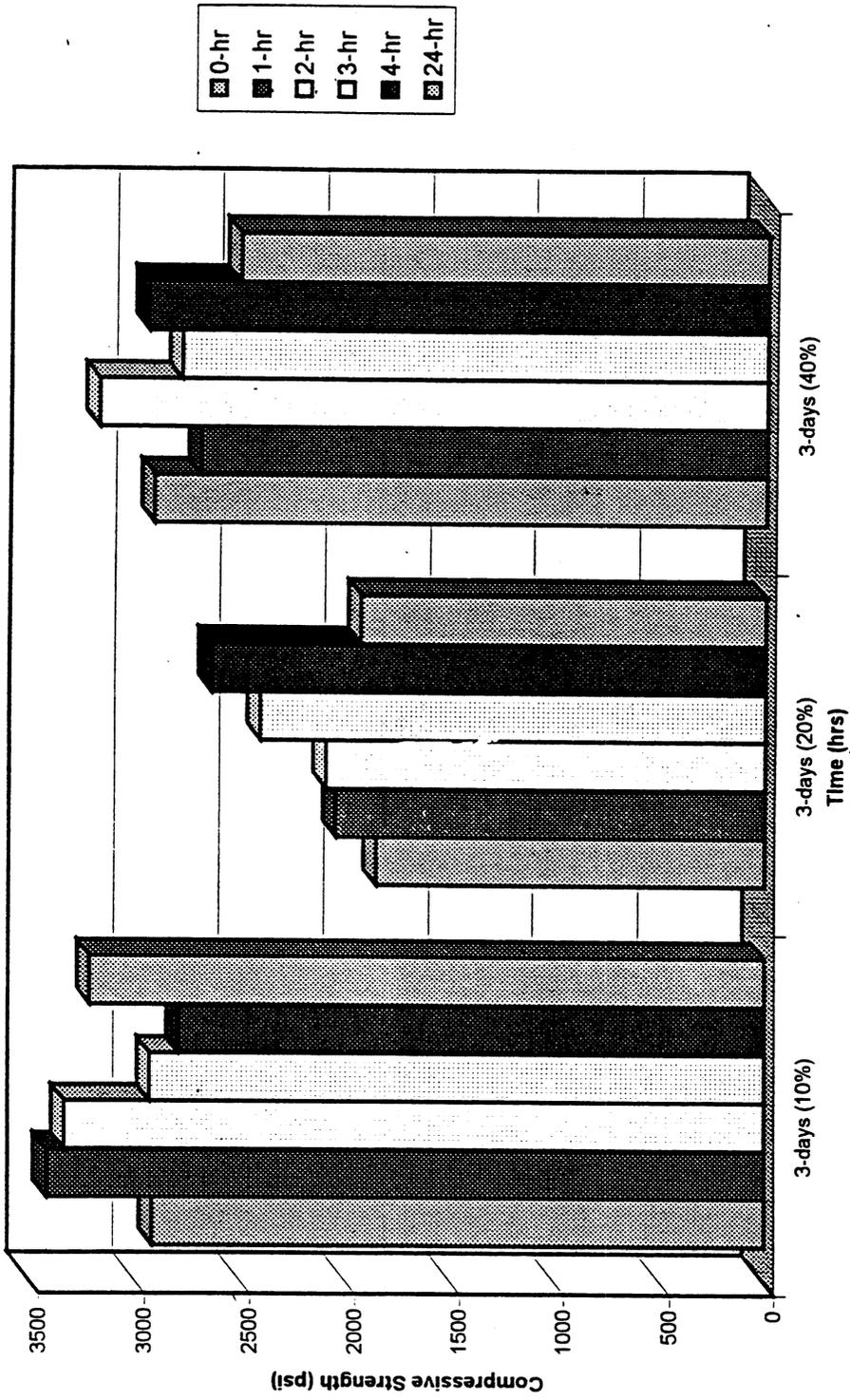


Figure 5.7: Compressive strength of concrete samples after 3 days; vibration amplitude = 30 g.

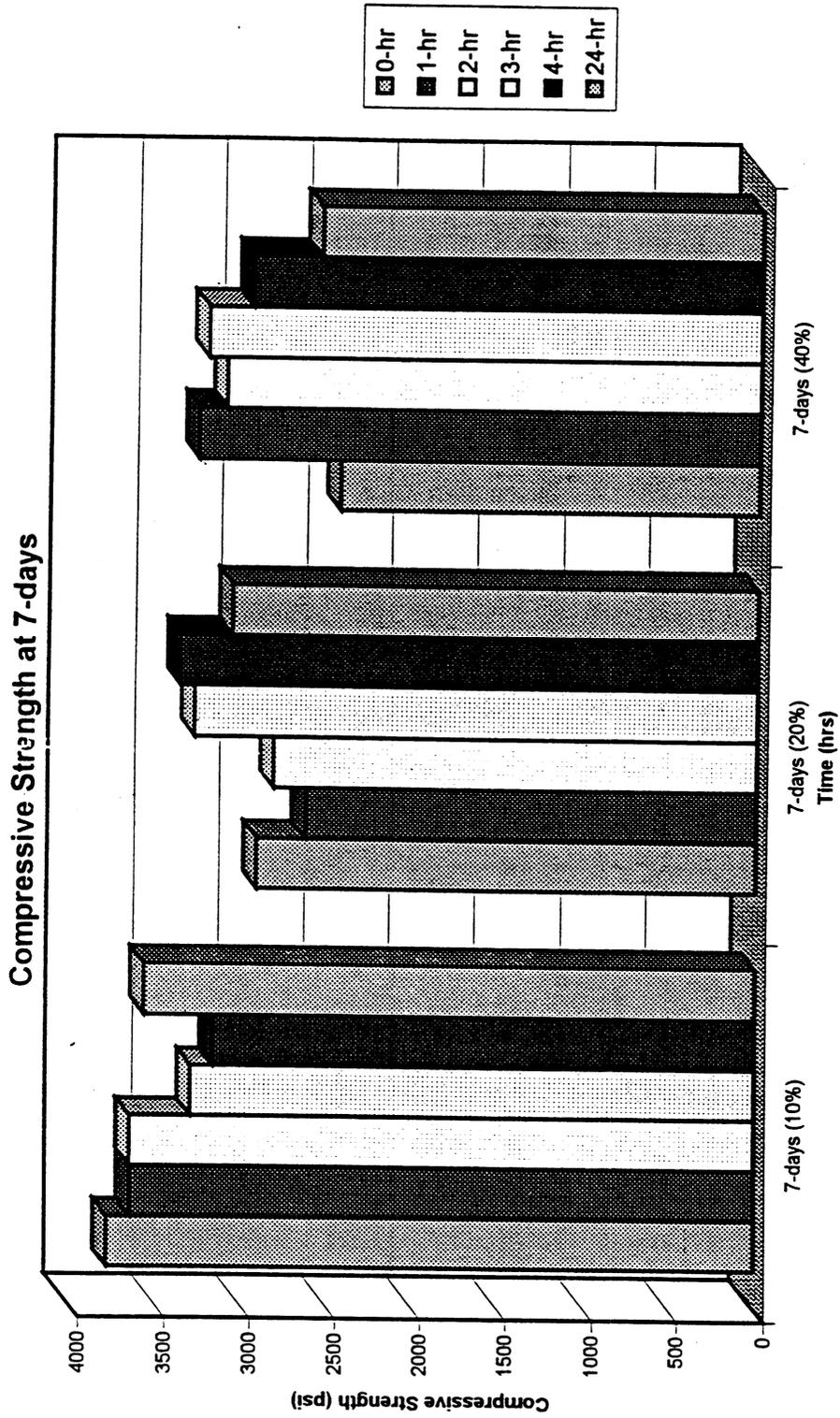


Figure 5.8: Compressive strength of concrete samples after 7 days; vibration amplitude = 30 g.

Compressive Strength at 14-days

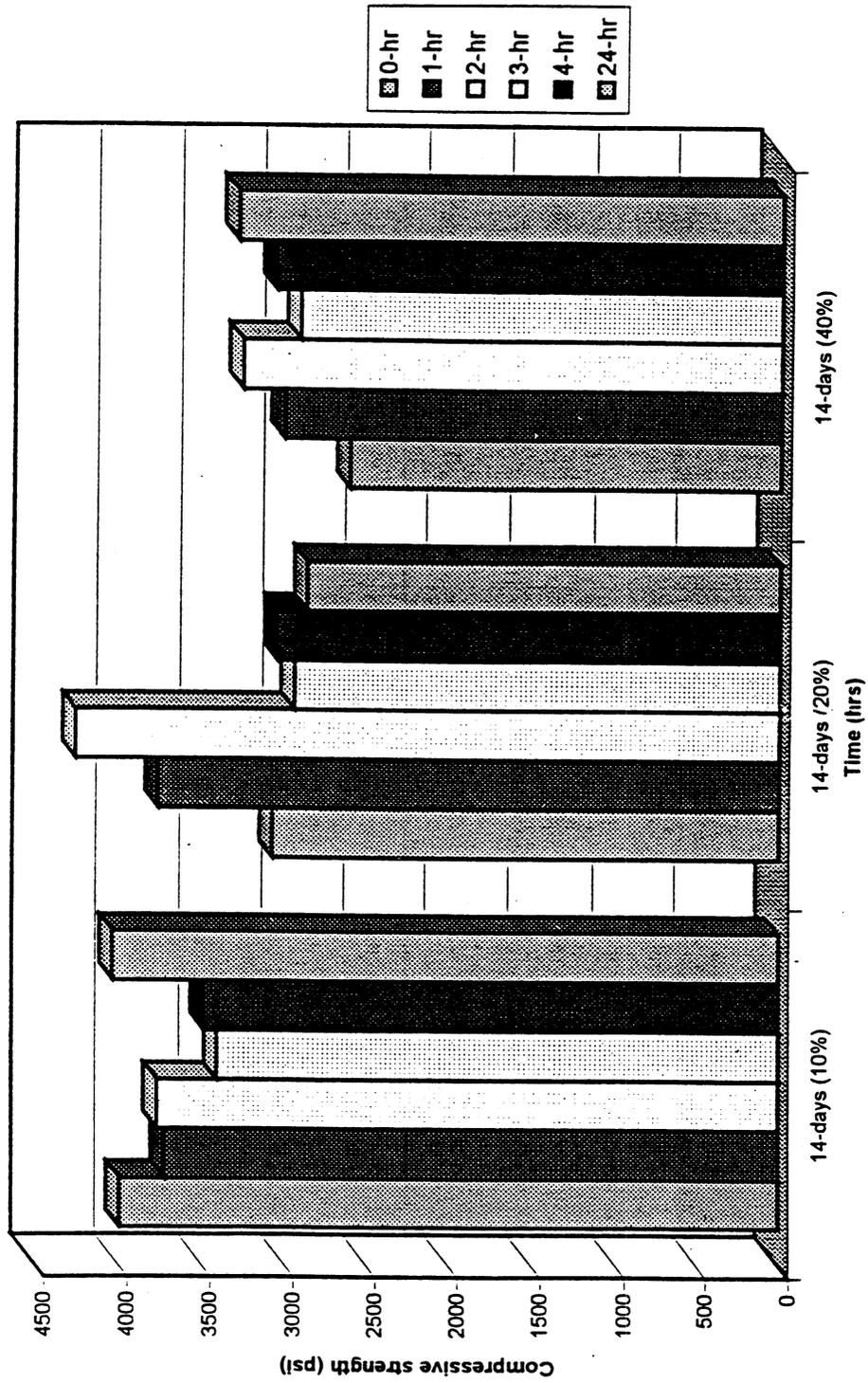


Figure 5.9 : Compressive strength of concrete samples after 14 days; vibration amplitude = 30g.

Compressive Strength at 28-days

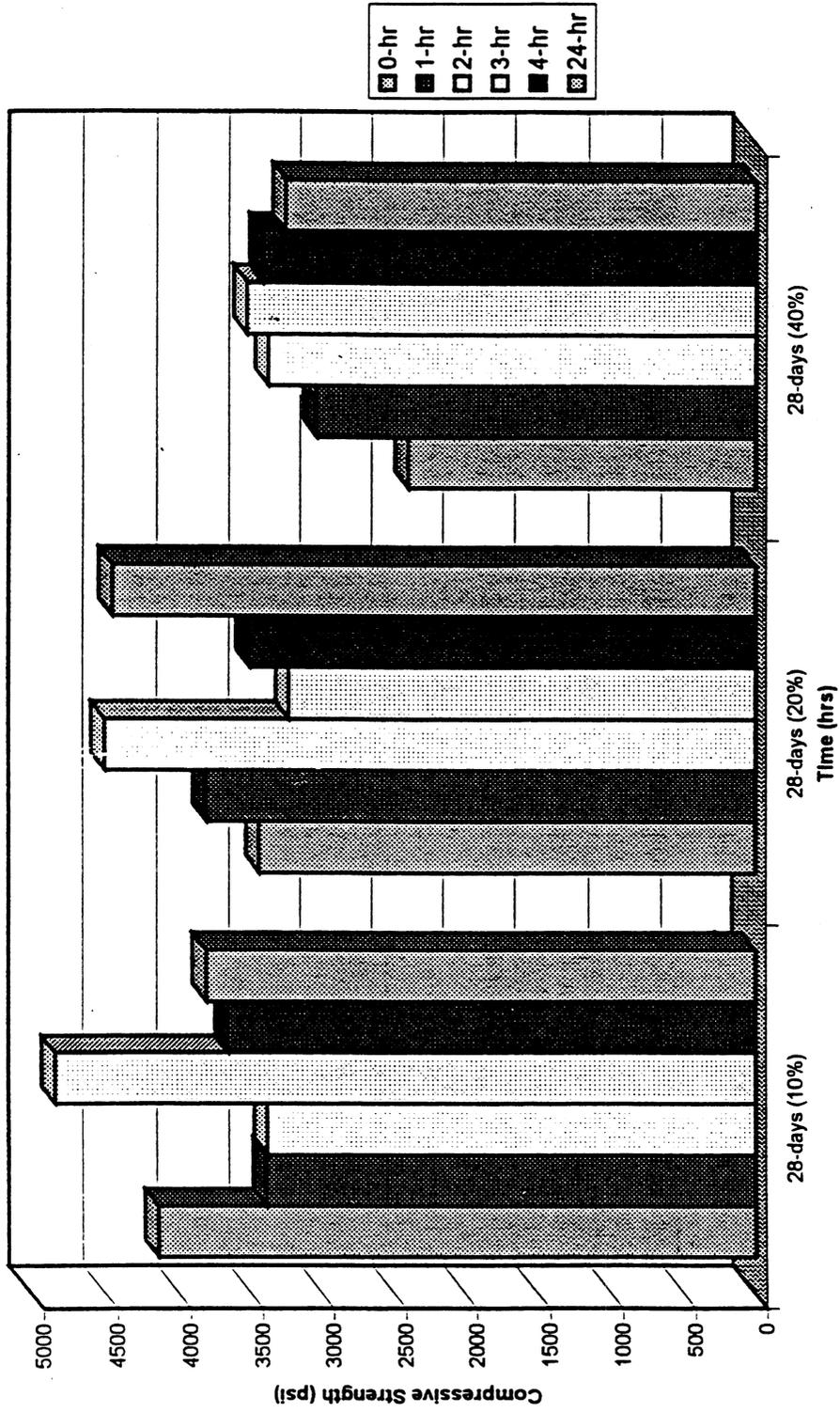


Figure 5.10 : Compressive strength of concrete samples after 28 days; vibration amplitude = 30g.

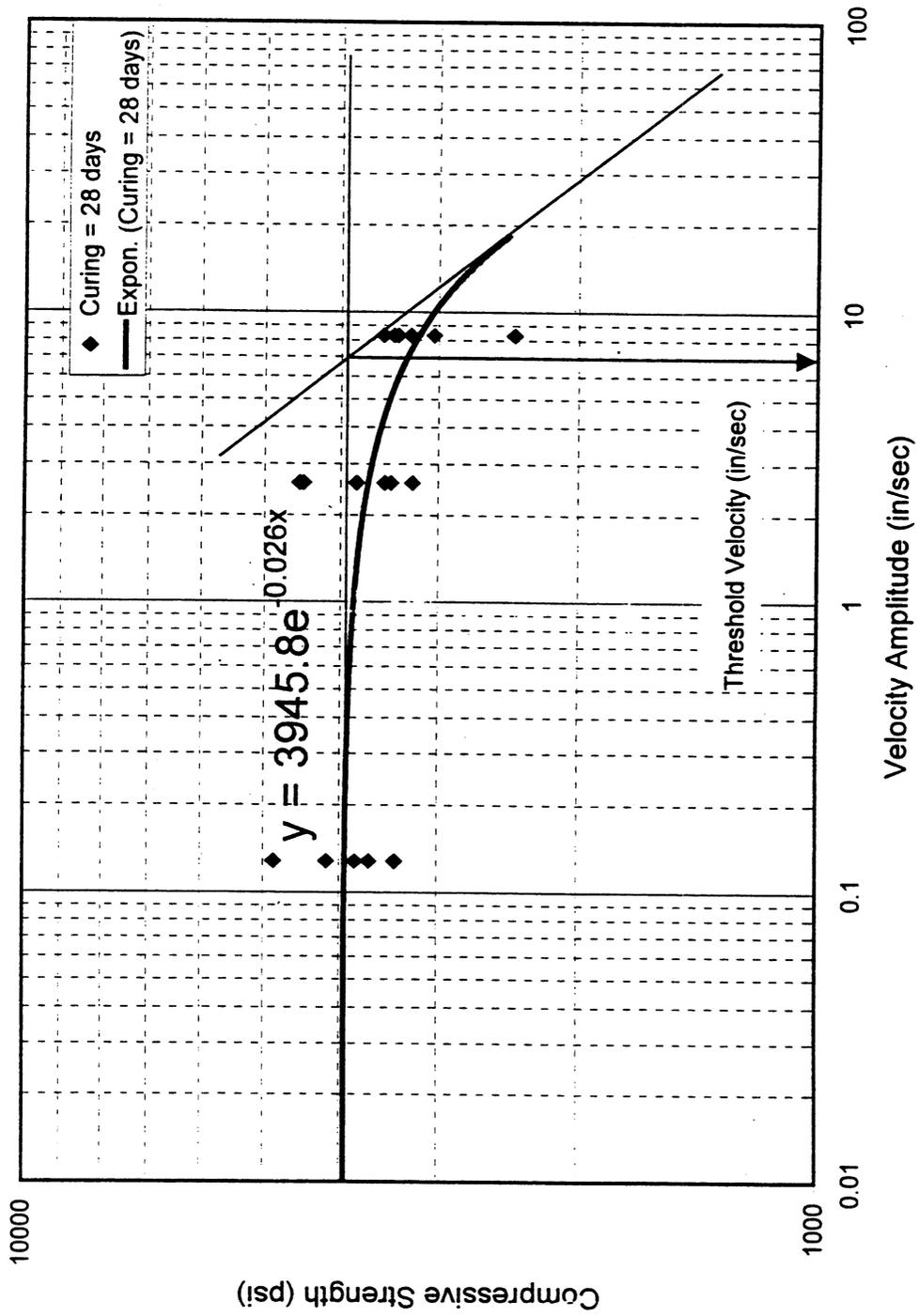


Figure 5.11: Compressive strength of concrete vs. velocity amplitude for all the tested samples

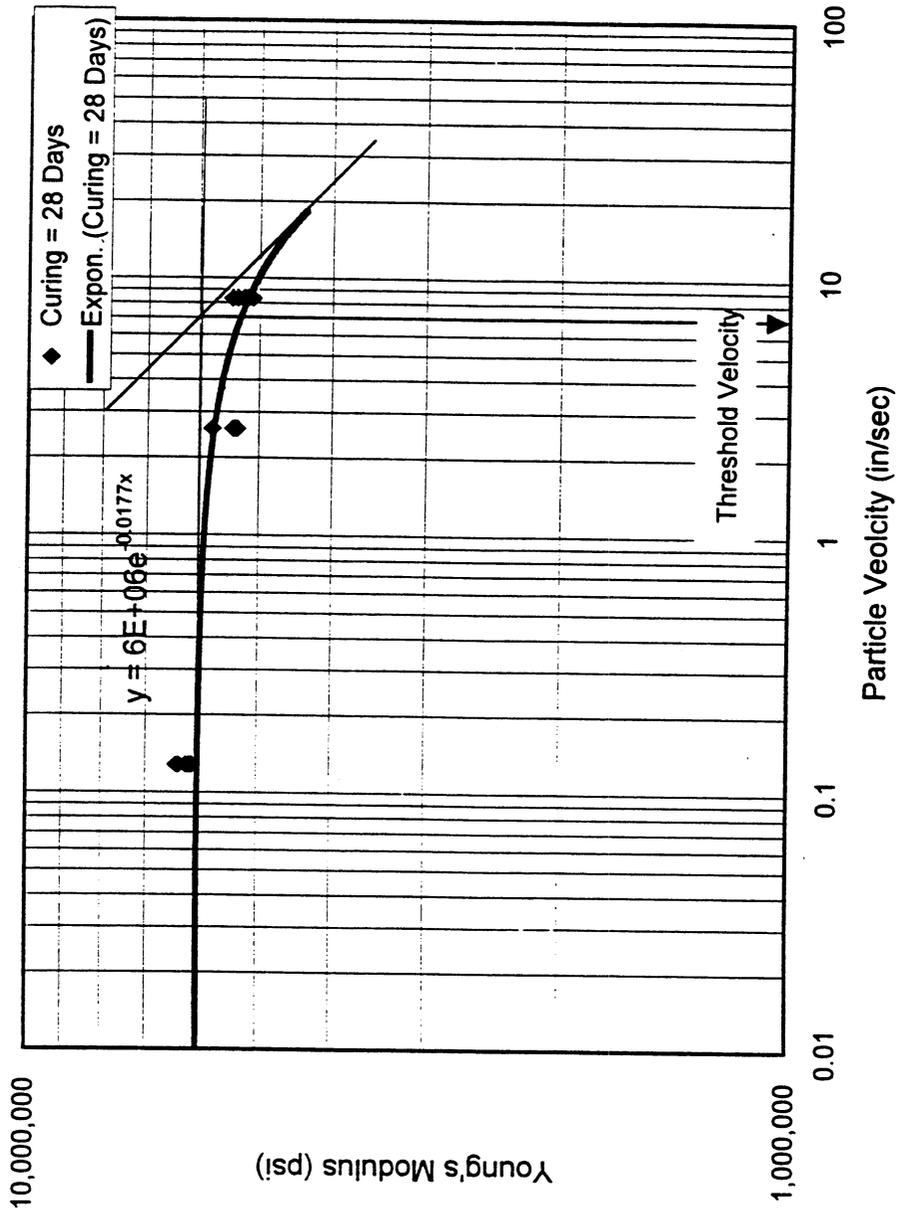


Figure 5.12: Compression modulus of concrete vs. velocity amplitude for all the tested samples

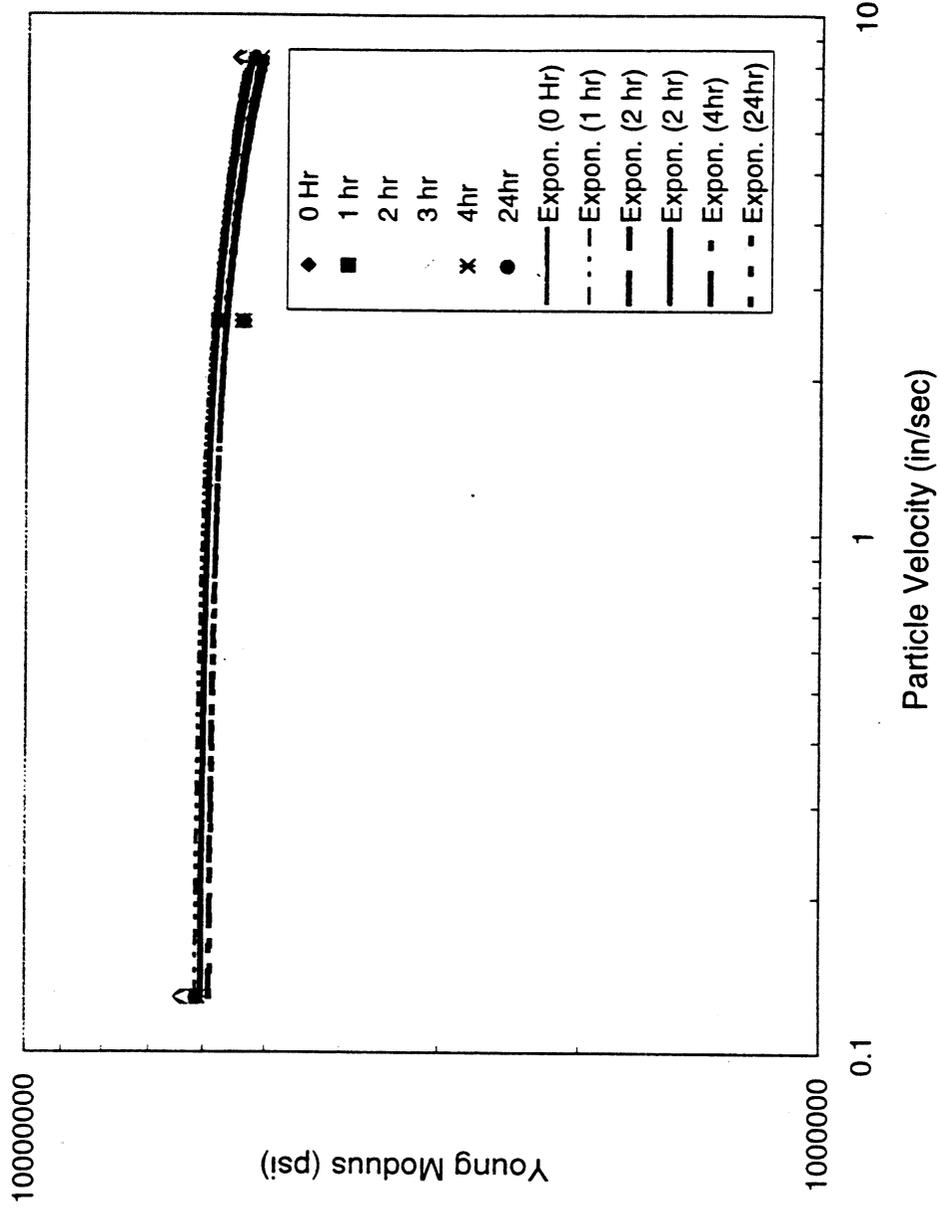


Figure 5.13: Compression modulus of concrete vs. velocity amplitude for the time delay used in testing

were drilled in the ground to monitor the generated vibrations. Two type of sensors were used in this phase. The first set of sensors consisted of two accelerometers. The first accelerometer was a triaxial one mounted on the steel casing directly using a metal stud to affix it firmly to the surface. The other two sensors were placed on 12 inch long steel rods driven into the ground. A wax adhesive was used to affix the accelerometers to the steel rods. Since the FFT analyzer used in the testing was capable of controlling two channels at the same time, the first channel was for the input vibration induced by the steel casing and the second channel for the ground accelerometer. The second accelerometer was located on the ground surface at three different distances from the casing, i.e. 5, 10, and 15 feet respectively. The triaxial accelerometer was first placed at 5 ft above the ground surface. While the casing was penetrating the soil, the accelerometer became closer to the ground. At that time the triaxial accelerometer was removed and placed again at a location of 5 feet above the ground. Thus, the second location was at 10 feet and third one at 15 feet from the tip of the casing (**Table 5.1**).

Results from this segment of testing indicated that velocity amplitudes generated from the driven steel casing were between 2 to 15 in./sec (**Figures 5.15 to 5.22**). The lower bound was for vibratory oscillation of 21Hz (1250 rpm). The upper bound was for 46 Hz (2750 rpm) oscillation. Unexpectedly, the waveforms recorded from the triaxial accelerometer showed that the signals were more random narrow band vibrations than harmonic sinusoidal ones. The reason was not precisely known for not getting typical sinusoidal waveforms despite the fact that the rotated masses inside the vibrator were moving systematically. However, one interpretation for such behavior by the steel casing might be related to the manner in which the vibrator was gripping the steel casing. The vibrator was gripping the casing only at one point causing what might be an uneccentric dynamic loading. This interpretation was confirmed from

Table 5.1: Florida Atlantic University Test Site

PERFORMED BY: FLORIDA STATE UNIVERSITY & LAW ENGINEERING

PART A: Data Organization

CH1: Accelerometer from FAU (PCB) mounted on steel casing

CH2: Accelerometer from DOT (Wilcox Research) mounted on the ground

Casing Depth (Ft.)	HP File name		CH1 Sensor Vertical Dist.	CH2 Sensor Horizontal Dist.	Hammer RPM	Frequency Hz
	CH1 Vertical	CH2 Horizontal				
1.5	Trace1.dat	Trace2.dat	5 ft. Sensor was 5' from edge	5 ft. Sensor was 5' from the casing	1250	21
5	Trace3.dat	Trace4.dat	5 ft. Sensor was 5' from edge	5 ft. Sensor was 5' from the casing	1800	30
8	Trace5.dat	Trace6.dat	10 ft. Sensor was 10' from edge	10 ft. Sensor was 10' from the casing	2000	33
10	Trace7.dat	Trace8.dat	10 ft. Sensor was 10' from edge	10 ft. Sensor was 10' from the casing	2500	42
12	Trace9.dat	Trace10.dat	15 ft. Sensor was 15' from edge	10 ft. Sensor was 10' from the casing	2750	46
13	Trace11.dat	Trace12.dat	15 ft. Sensor was 15' from edge	15 ft. Sensor was 15' from the casing	2750	46
13.75	Trace13.dat	Trace14.dat	15 ft. Sensor was 15' from edge	15 ft. Sensor was 15' from the casing	1500	25
14.75	Trace15.dat	Trace16.dat	15 ft. Sensor was 15' from edge	15 ft. Sensor was 15' from the casing	2000	33

Part B: Signal Analyzer Set-Up

The HP Signal Analyzer was set-up as follows:

- 20 averages using the time domain
- Channel amplification was X10
- The bandwidth for the frequency domain was from 0-6.4 KHz
- The state (set-up) file name is FAU1.sta
- Two channel data acquisition
- Number of Points = 1024 at 400 Resolution
- PCB accelerometer (on Casing) = 100 mVolt/g, Wilcoxon Accelerometer on the ground = 517 mV/g

PART C: Steel Casing Dimensions

The casing dimensions were as follows:

- Thickness = 1/2 inch
- Length = 40 ft.
- Diameter = 36 inches

Table 5.2 Time and Frequency Domain Analysis of Particle velocity

Record # / Record Type	RMS of the Velocity vs. Time	RMS from SDF
Trace 1	1.169	1.07
Trace 5	1.270	1.182
Trace 7	1.758	1.664
Trace 11	3.820	3.556

SDF=Spectral Density Function

Table 5.3 Soil Properties of Samples Obtained From the FAU Site on 7/3
 Tested by: Dr. Kamal Tawfiq

Sample No.	Depth (ft)	γ_{wet} (gm/cm ³)	γ_{dry} (gm/cm ³)	wc (%)	Void Ratio e
1	1.5	1.25	1.25	0	0.289
2	3	1.395	1.388	0.48	0.332
3	4.5	1.29	1.247	3.45	0.298
4	6	1.51	1.319	14.48	0.407
5	8	1.7	1.437	18.27	0.41
6	9	1.6	1.417	12.91	0.364
7	10.5	1.686	1.437	17.3	0.378
8	12	1.694	1.487	13.94	0.352
9	13	1.819	1.493	21.77	0.415
10	15	1.578	1.371	15.13	0.327
11	16.5	1.578	1.289	22.3	0.336
12	19	1.577	1.281	23.06	0.327
13	21.5	1.547	1.271	21.75	0.325
14	24	1.526	1.241	23	0.323
15	26.5	1.491	1.215	22.71	0.324
16	29	1.681	1.366	23.07	0.367
17	31.5	1.615	1.289	25.32	0.356

Soil is mostly loose fine light tan sand to dark tan fine sand

Water Table at 6.7 ft

Boring Termination = 41 ft

Table 5.4: Determination of Damping characteristics (D) in the sandy soil layer in FAU Site
 Samples were tested on 8/15/1997

Depth (ft)	Unit Weight Gm/cm ³	Effective Unit Weight psf	Void Ratio (e)	Vertical Effectiveness Stress (psf)	Shear wave Velocity ft/sec	Shear wave Velocity in./sec
1.5	1.25	78	0.289	117	484.78	11634.72
3	1.395	87.0	0.332	261.14	579.02	13896.55
4.5	1.29	80.5	0.3	362.23	639.30	15343.14
6	1.51	94.2	0.4	565.34	676.42	16234.11
8	1.7	43.7	0.41	349.44	596.39	14313.25
9	1.6	37.4	0.364	336.96	606.40	14553.60
10.5	1.686	42.8	0.378	449.47	646.65	15519.50
12	1.694	43.3	0.352	519.67	680.25	16325.89
13	1.819	51.1	0.415	664.37	698.32	16759.66
15	1.578	36.1	0.27	541.01	718.05	17233.20
16.5	1.578	36.1	0.336	595.11	709.87	17036.96
19	1.578	36.1	0.327	685.28	738.96	17734.99
21.5	1.547	34.1	0.325	733.86	752.53	18060.80
24	1.526	32.8	0.325	787.74	765.98	18383.57
26.5	1.491	30.6	0.324	811.92	772.21	18533.07
29	1.681	42.5	0.367	1232.34	837.19	20092.67
31.5	1.615	38.4	0.356	1208.84	838.25	20117.95

Water table at 5.9 ft. to 6.7 ft.

Hardin and Richard 1963

Shear wave velocity = $v_s =$

$(170 - 78.2e) * \sigma_0^{1/4}$ for $\sigma_0 > 2000$ psf

Shear wave velocity = $v_s =$

$(119 - 56.0e) * \sigma_0^{1/5}$ for $\sigma_0 < 2000$ psf

Hardin and Richart (1963)
For Sandy Soil:

$$v_s = (170 - 78.2 e) \sigma_o^{1/4} \quad \text{For } \sigma_o \geq 2000 \text{ lb/ft}^2$$

$$v_s = (119 - 56 e) \sigma_o^{1/4} \quad \text{For } \sigma_o < 2000 \text{ lb/ft}^2$$

σ_o = effective confining stress
 e = void ratio

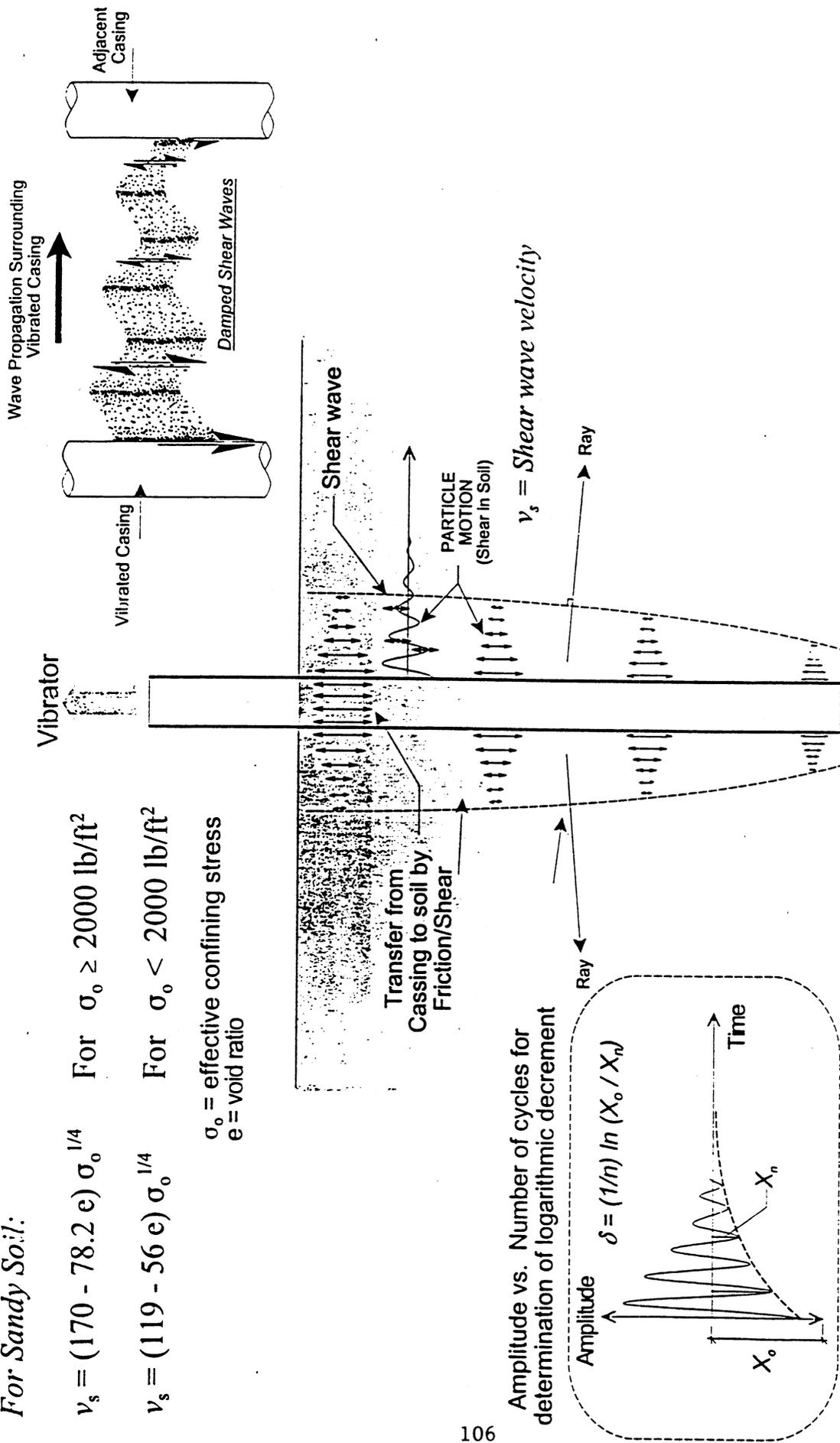


Figure 5.14. Vibratory drilled shaft casing generating shear waves in the surrounding soil.

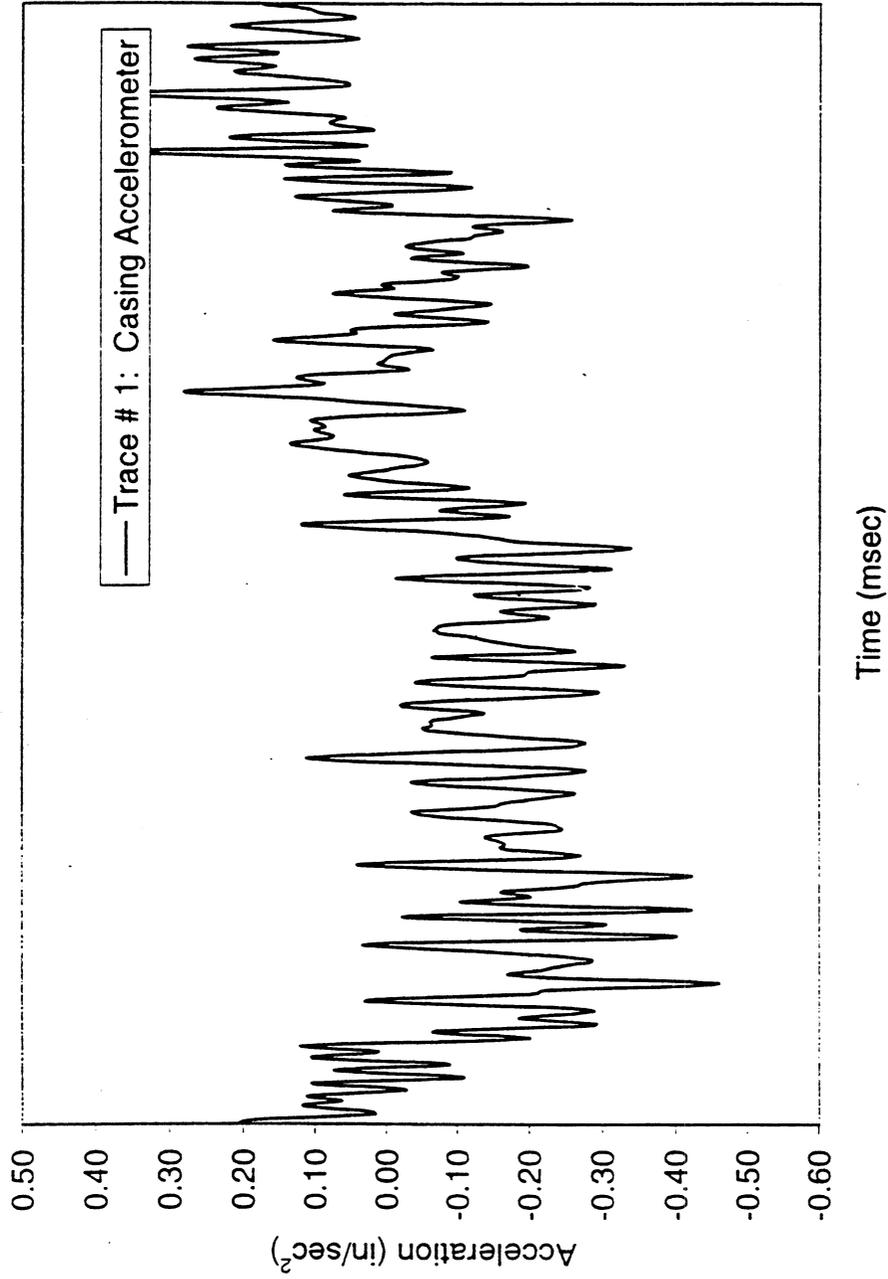


Figure 5.15 Time-history record of the steel casing at the accelerometer 5 ft. from the ground

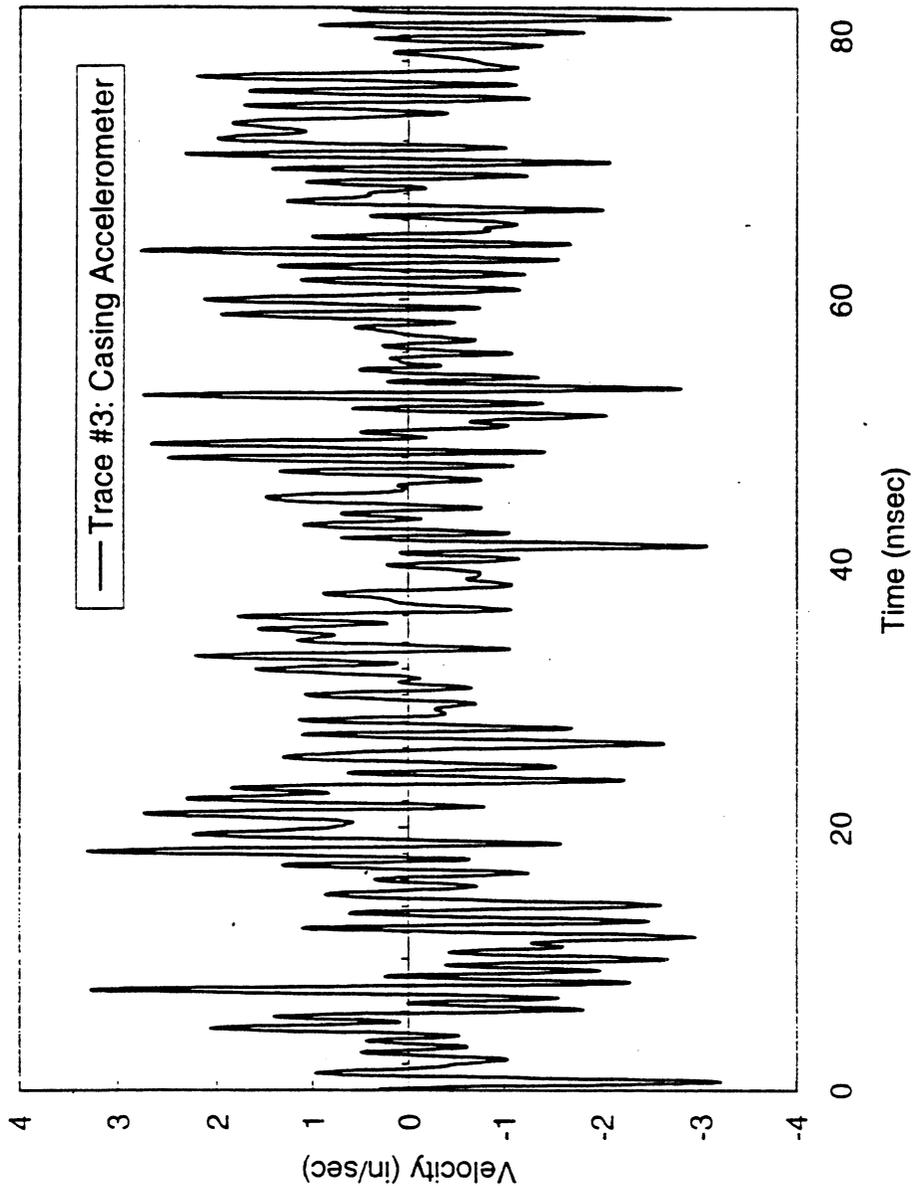


Figure 5.16 Time-history record of the steel casing at the accelerometer 5 ft from the ground

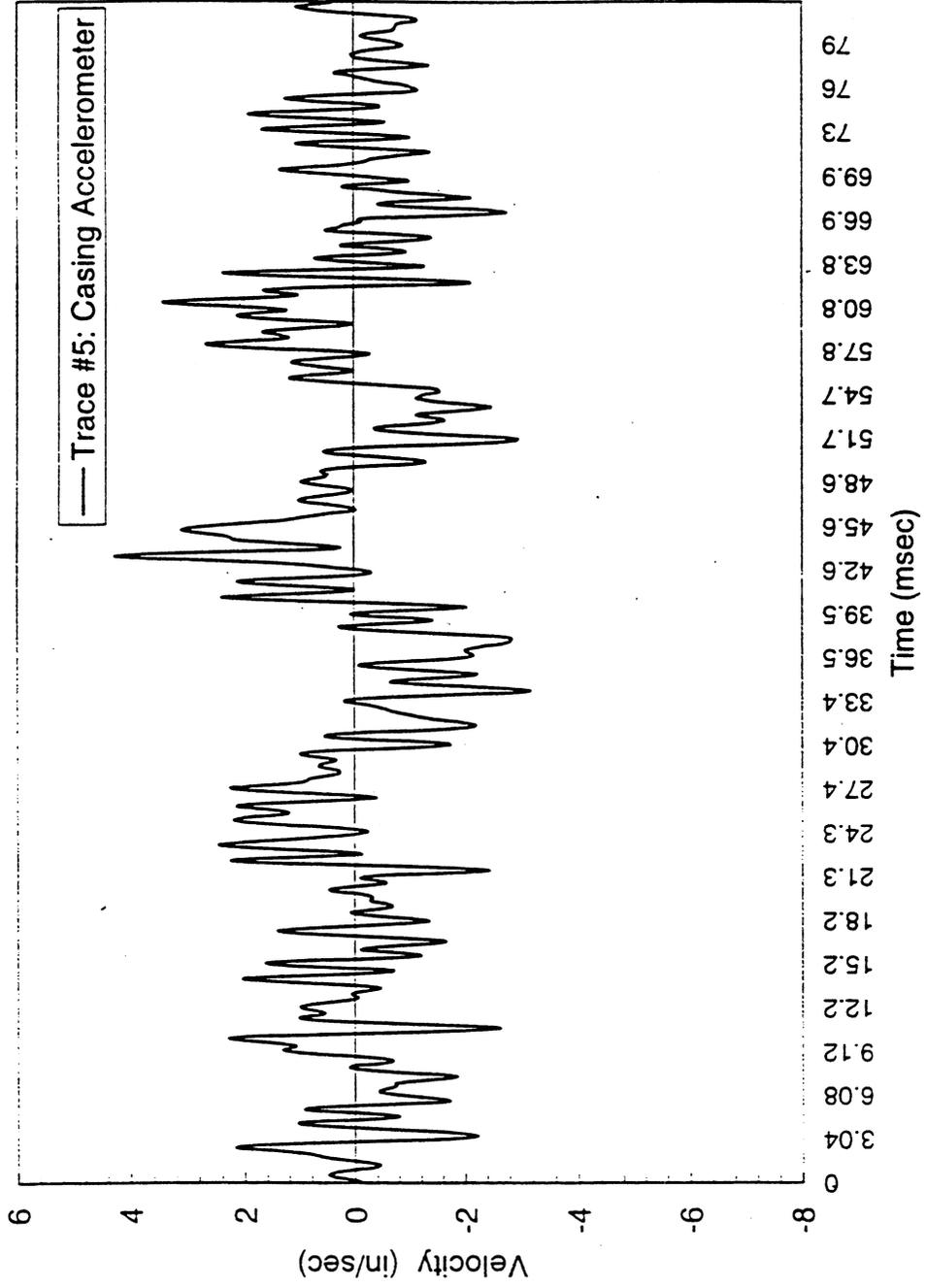


Figure 5.17 Time-history record of the steel casing at the accelerometer 5 ft from the ground

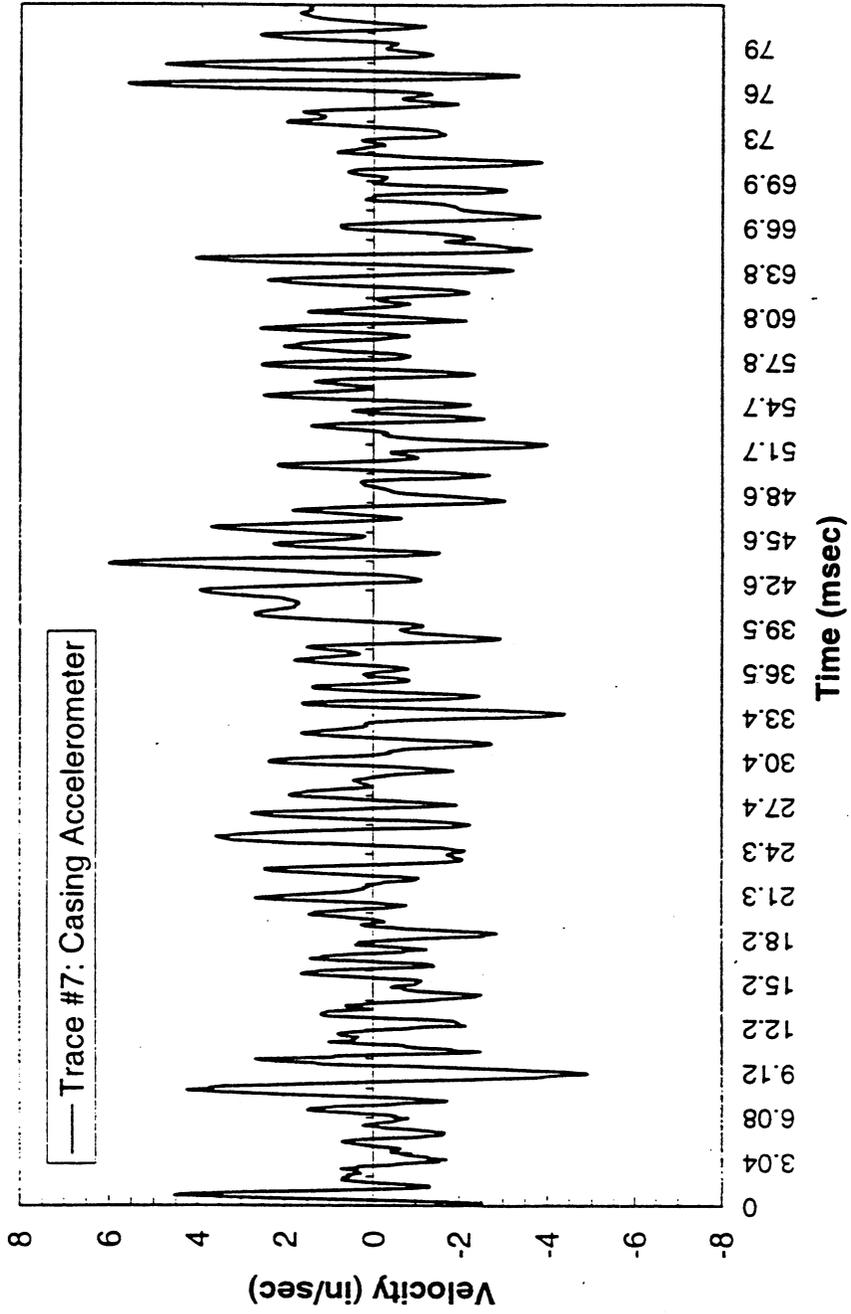


Figure 5.18 Time-history record of the steel casing at the accelerometer 5 ft from the ground

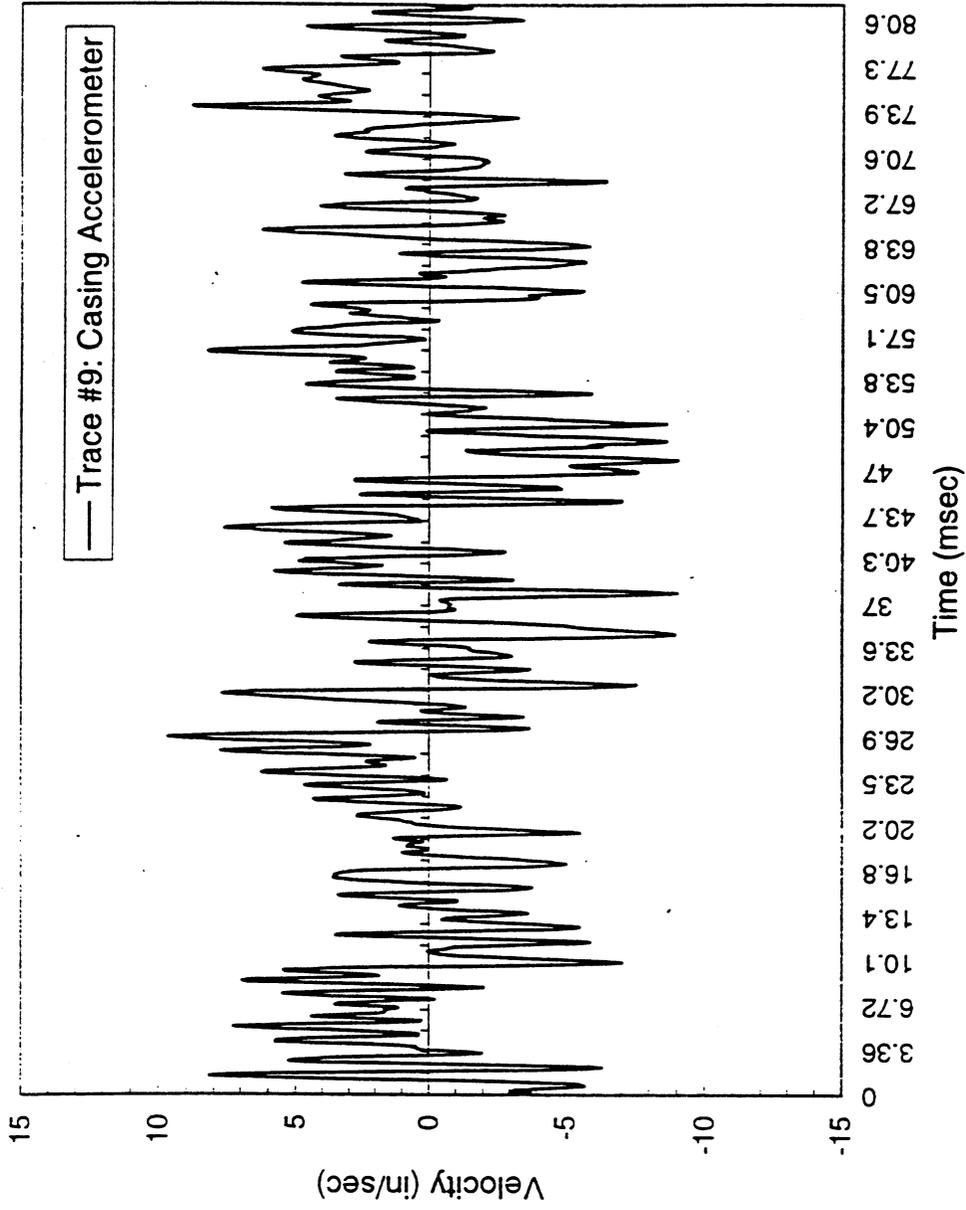


Figure 5.19 Time-history record of the steel casing at the accelerometer 5 ft from the ground

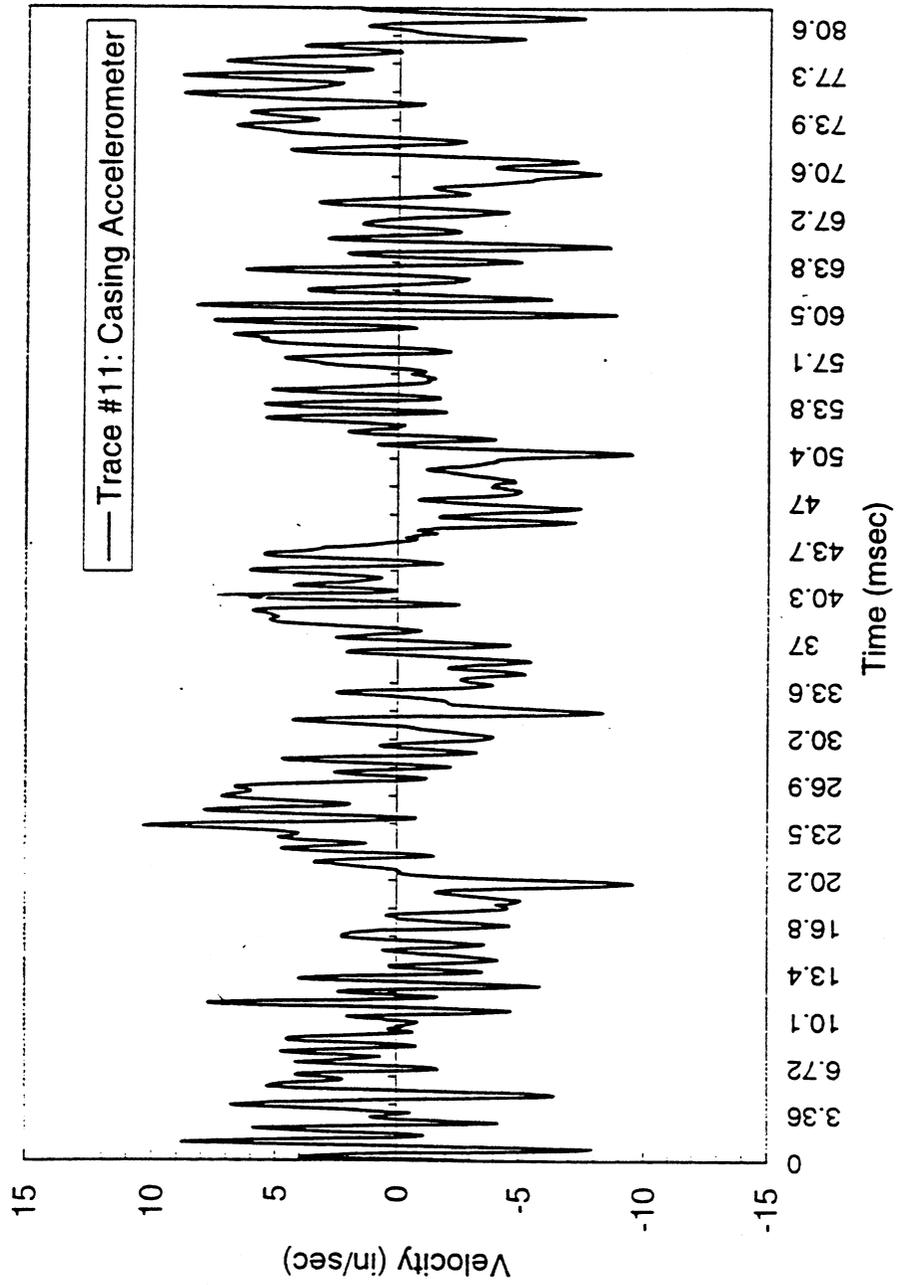


Figure 5.20 Time-history record of the steel casing at the accelerometer 5 ft from the ground

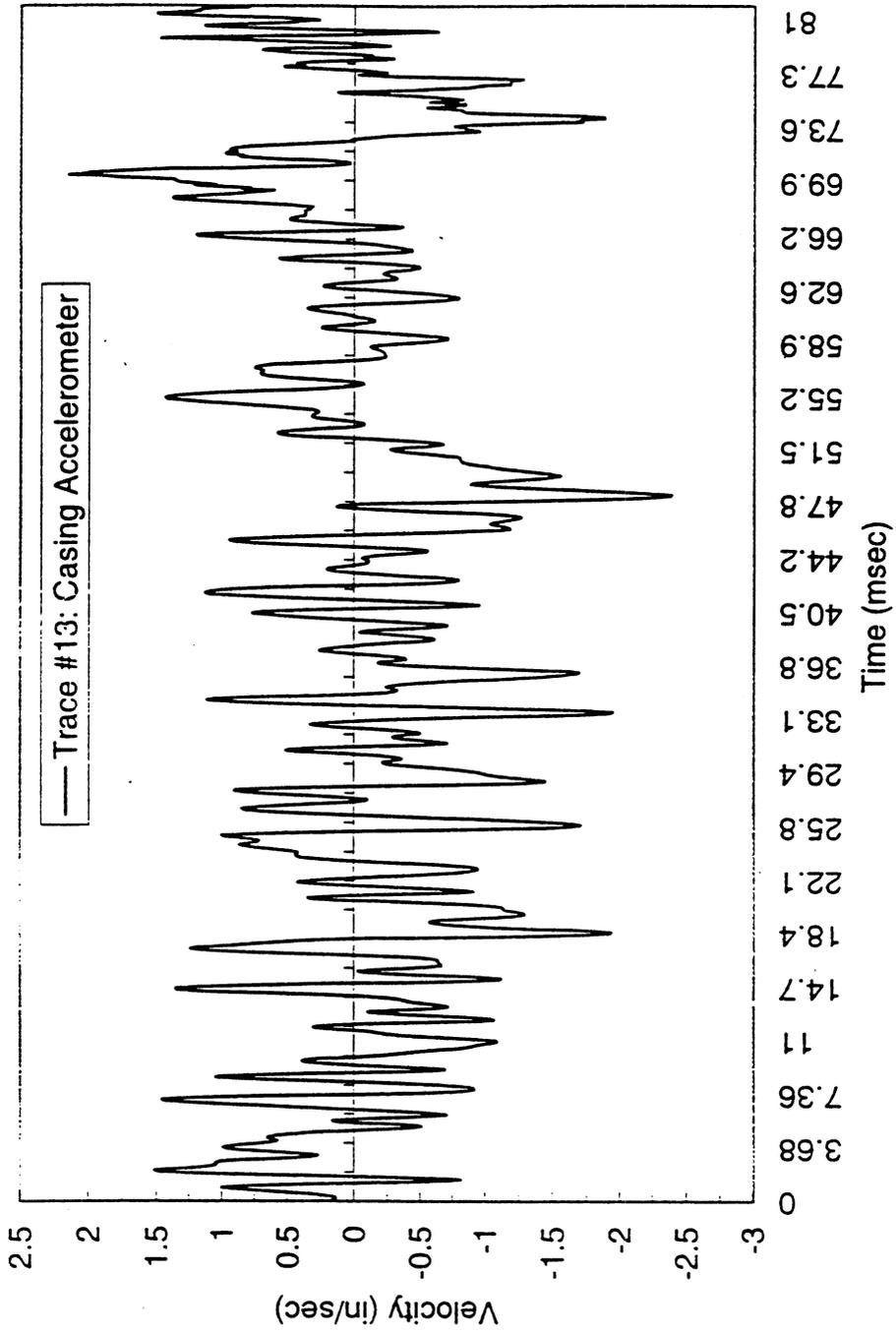


Figure 5.21 Time-history record of the steel casing at the accelerometer 5 ft from the ground

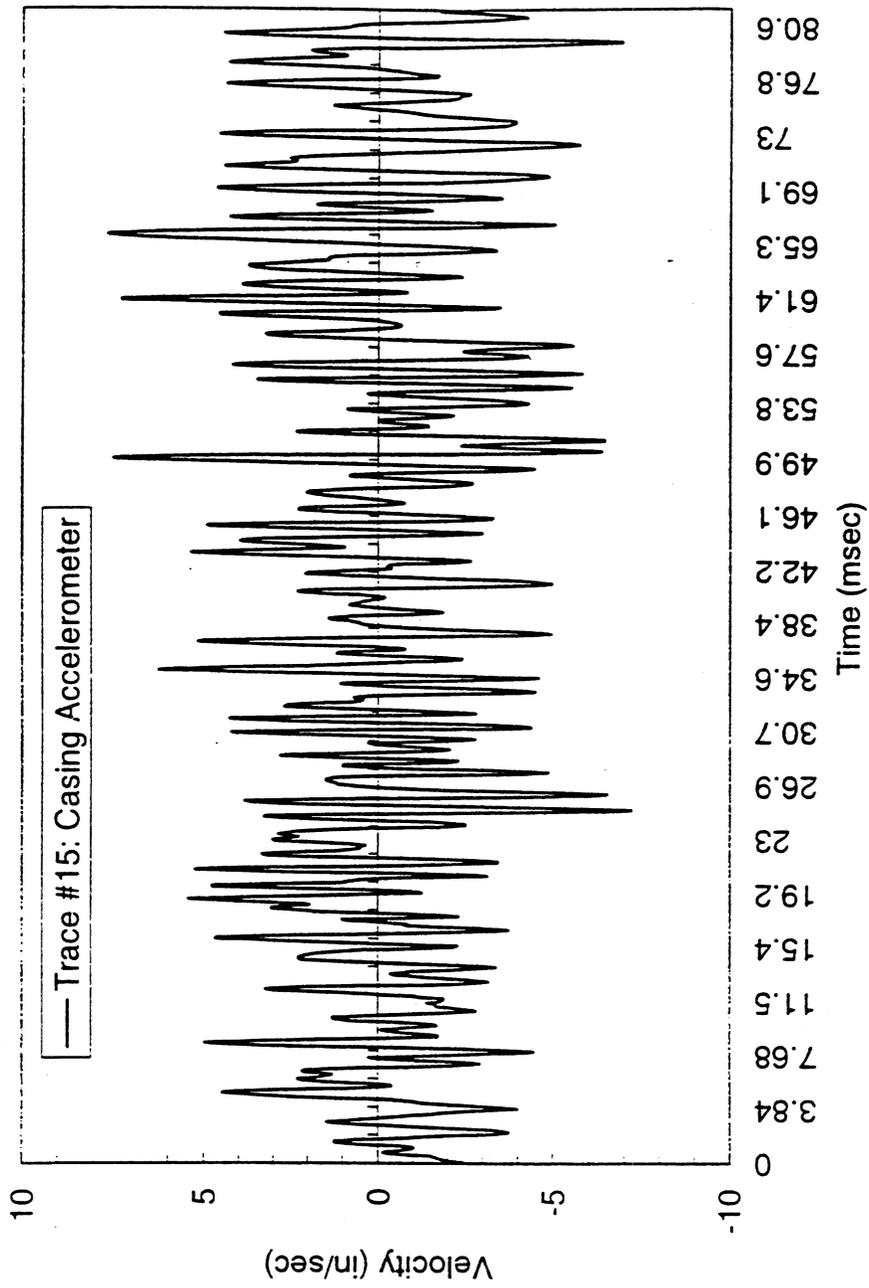


Figure 5.22 Time-history record of the steel casing at the accelerometer 5 ft from the ground

the records in the ground geophones at the second FAU field test site. These records showed vibration components that are larger than the vertical ones, although the intended motion was in the vertical direction.

The time-history records from the accelerometers were converted to velocity by using the direct integration method. If the vibrations are considered as narrow band responses, then the direct method will be inapplicable. In this case another method for determining the RMS velocity should be used. In this study it was decided to implement this technique and the casing vibrations were examined using the frequency response analysis. This analysis could be accomplished by integrating the spectral density function for the acceleration requiring the transformation of the time-history records from the time domain to the frequency domain (**Figures 5.23 to 5.26**). This transformation could be accomplished using the Fast Fourier Transform (FFT) algorithm.

For that purpose, an add-in program called FFTTOOLS was used along with a spreadsheet (Excel 4.0) that contained the acceleration records. Before converting the ASCII files collected during the field testing to acceleration records, it was necessary to establish a macro on the spreadsheet program to arrange the ASCII files and set the tables in a format that the FFTTOOLS could accept. Setting the required parameters for the FFT analysis, it was possible to obtain the frequency response function (FRF) and the spectral density function (SDF). The SDF was used to determine the RMS value of the acceleration and then integrated to obtain the SDF of the velocity. Accordingly, the RMS velocity could be obtained and compared with the RMS values from the direct integration of the time-history records. The RMS values from both methods are presented in **Table 5.2**.

As can be seen from **Table 5.2** the variation between the RMS values of the records using the two methods was expected. However, the small variations were an indication that using the

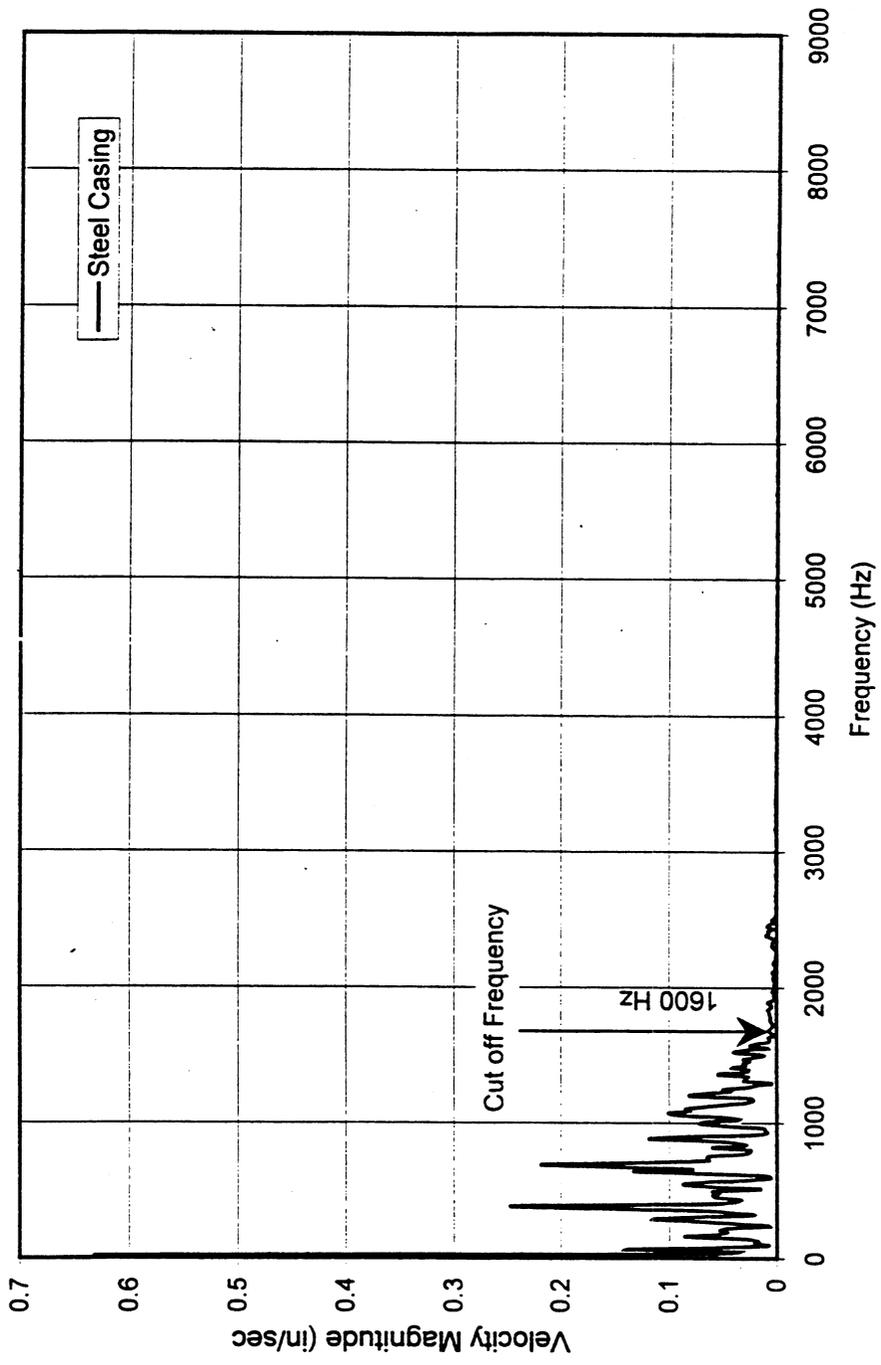


Figure 5.23: Frequency content of a 40 ft drilled shaft steel casing driven to 1.5 ft in sandy soil.

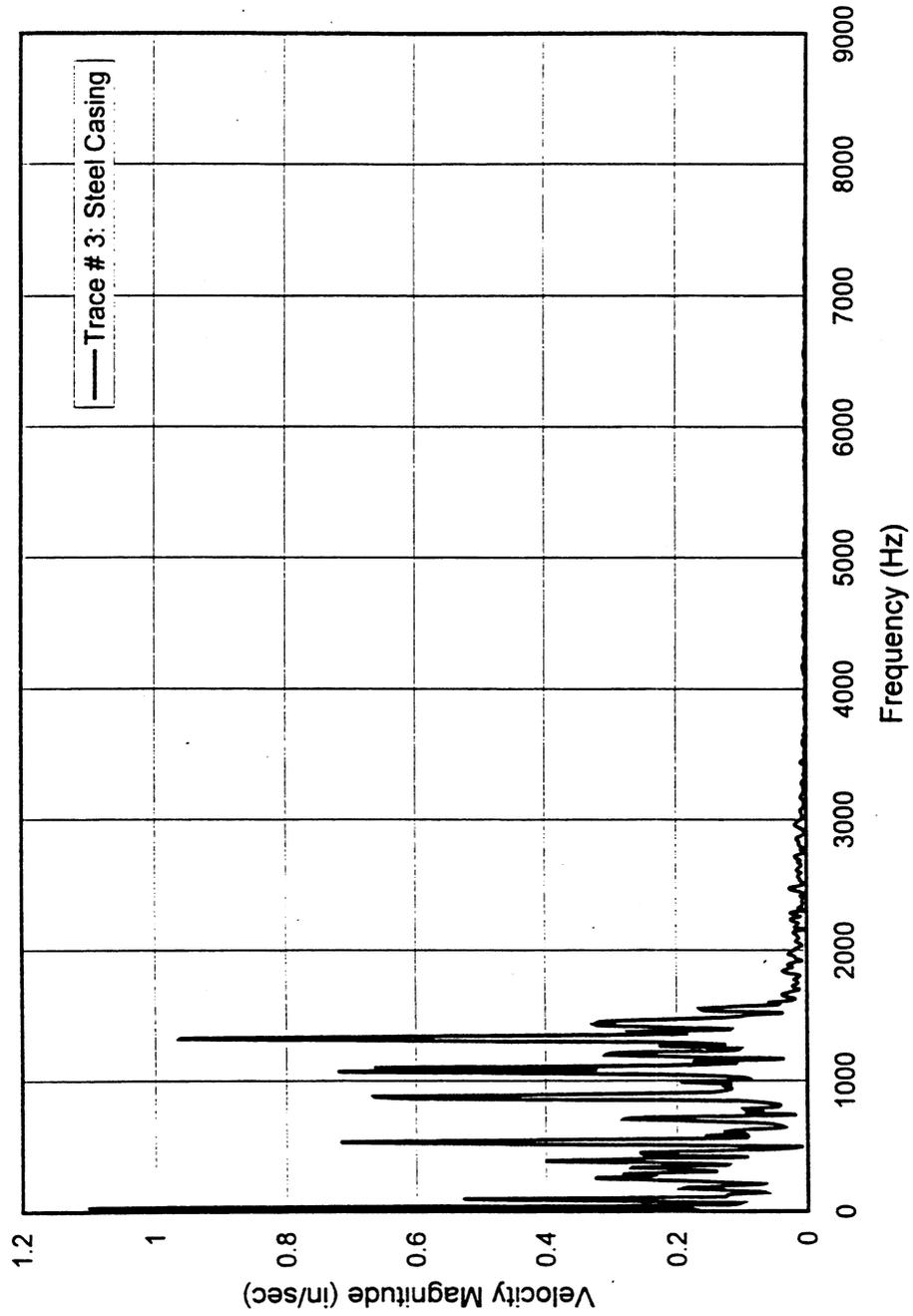


Figure 5.24: Frequency content of a 40 ft drilled shaft steel casing driven to 5 ft in sandy soil

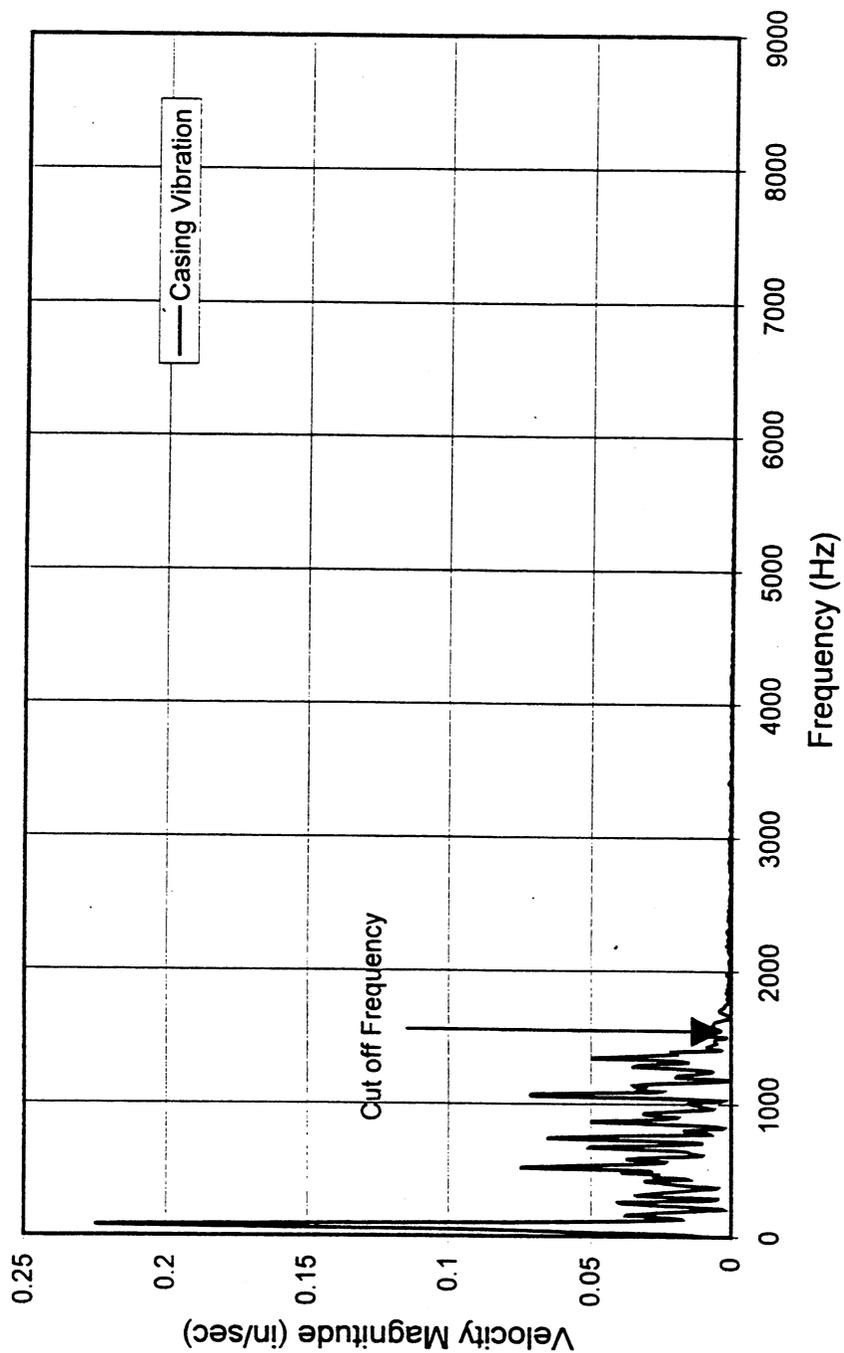


Figure 5.25: Frequency content of a 40 ft drilled shaft steel casing driven to 8 ft in sandy soil.

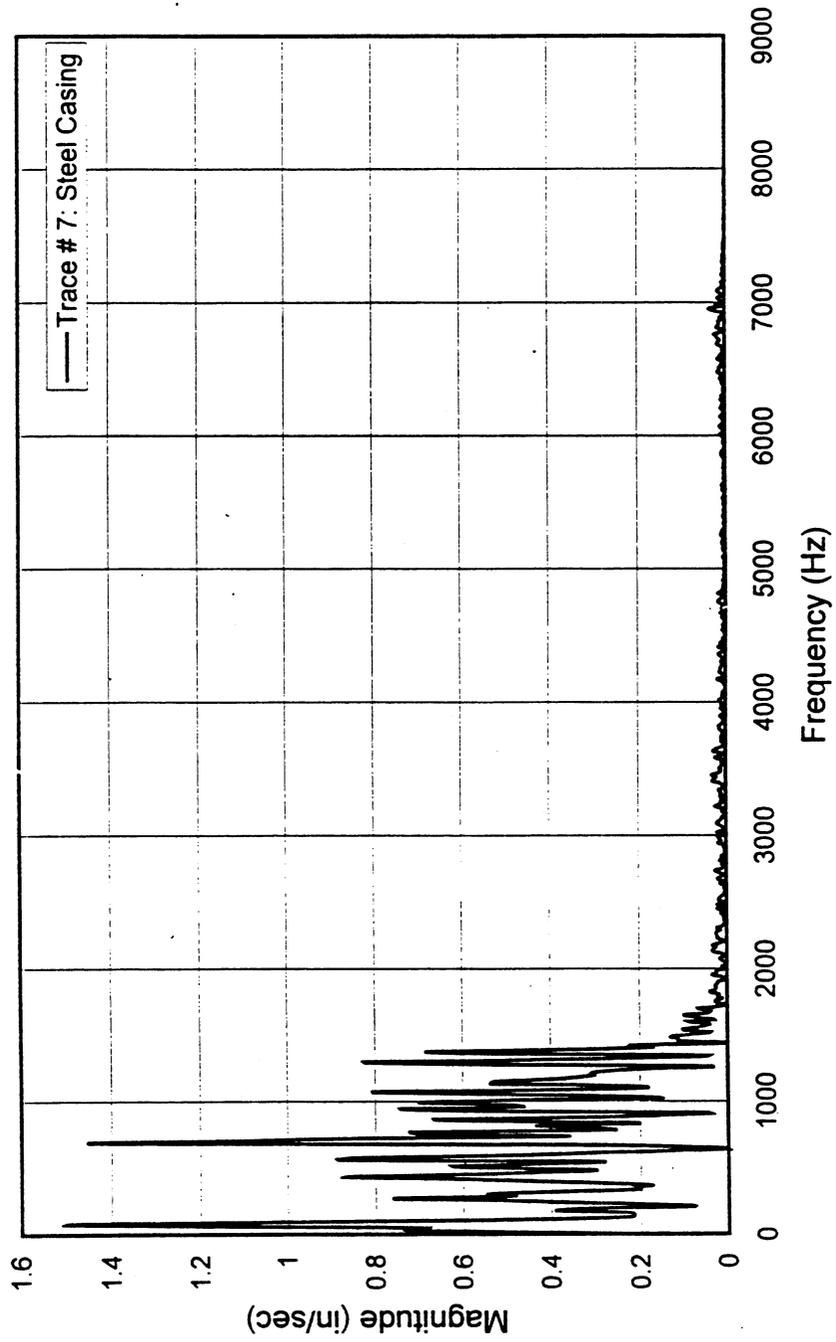


Figure 5.26: Frequency content of the steel casing due to 10 ft penetration.

direct integration method was an acceptable practice as determining the peak particle velocity from random vibration analysis is a laborious process that may deem the procedure impractical. On the other hand, with the advancements in hardware and software technologies make it easier to use the frequency domain analysis. The system parameters of the construction vibrations were determined by real time domain and frequency domain analysis in the present study. This approach, as compared with only frequency domain analysis, may provide field engineers with a broader knowledge of modal testing and analysis.

The frequency content of the casing vibration from the FRF was determined to be about 1500 Hz. When the RPM on the hydraulic pump was increased the frequency content reached 2000 Hz., the dominant frequency ranged between 16 Hz to 75 Hz (**Figures 5.27 to 5.30**). These values corresponded to the increase in the angular velocity of the mass rotation in the vibrator.

The surface accelerometer recorded particle accelerations that ranged from 0.5 in/sec^2 to 0.03 in/sec^2 for distances of 5 and 15 feet, respectively. The peak particle velocities of the ground response obtained by the direct integration method, ranged from 0.3 in/sec to 1 in/sec. It was noticed, that the surface response to the steel casing vibration exhibited higher velocities as the casing penetrated deeper in the ground. This observation was confirmed when the second field test at FAU was conducted with geophones inserted in the ground at certain depths. In general, the frequency content of the ground surface was about 300 Hz with the dominant frequency of about 32 Hz (**Figures 5.31 to 5.35**).

5-2-2 FAU Field Test # 2

The second test at Boca Raton was on January 9, 1998 . The objectives were similar to the first field test, with the exception of that geophones were embedded in the ground and were used

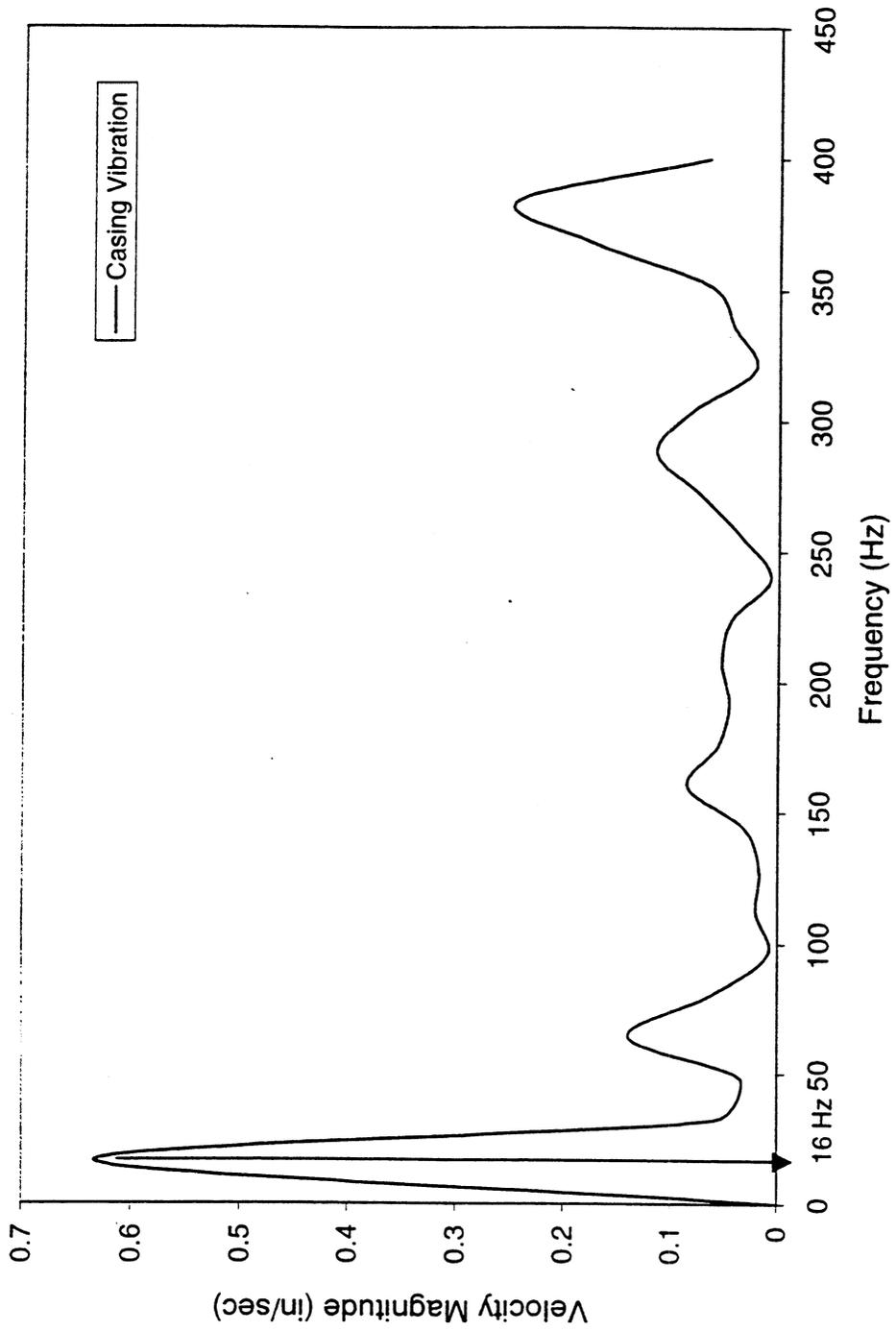


Figure 5.27: Dominant frequency of the steel casing at penetration depth of 1.5 ft

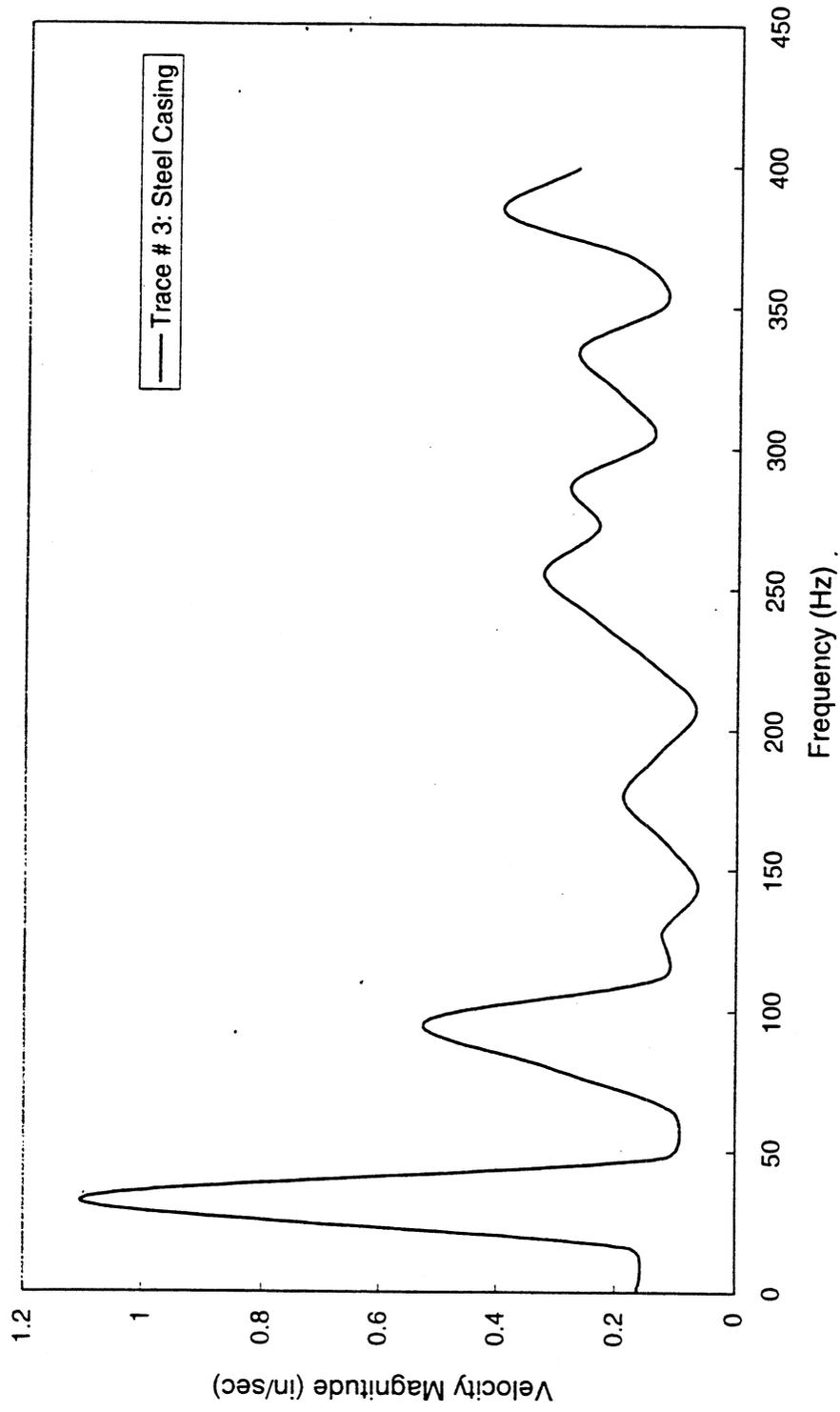


Figure 5.28: Dominant frequency of the steel casing at penetration depth of 5 ft

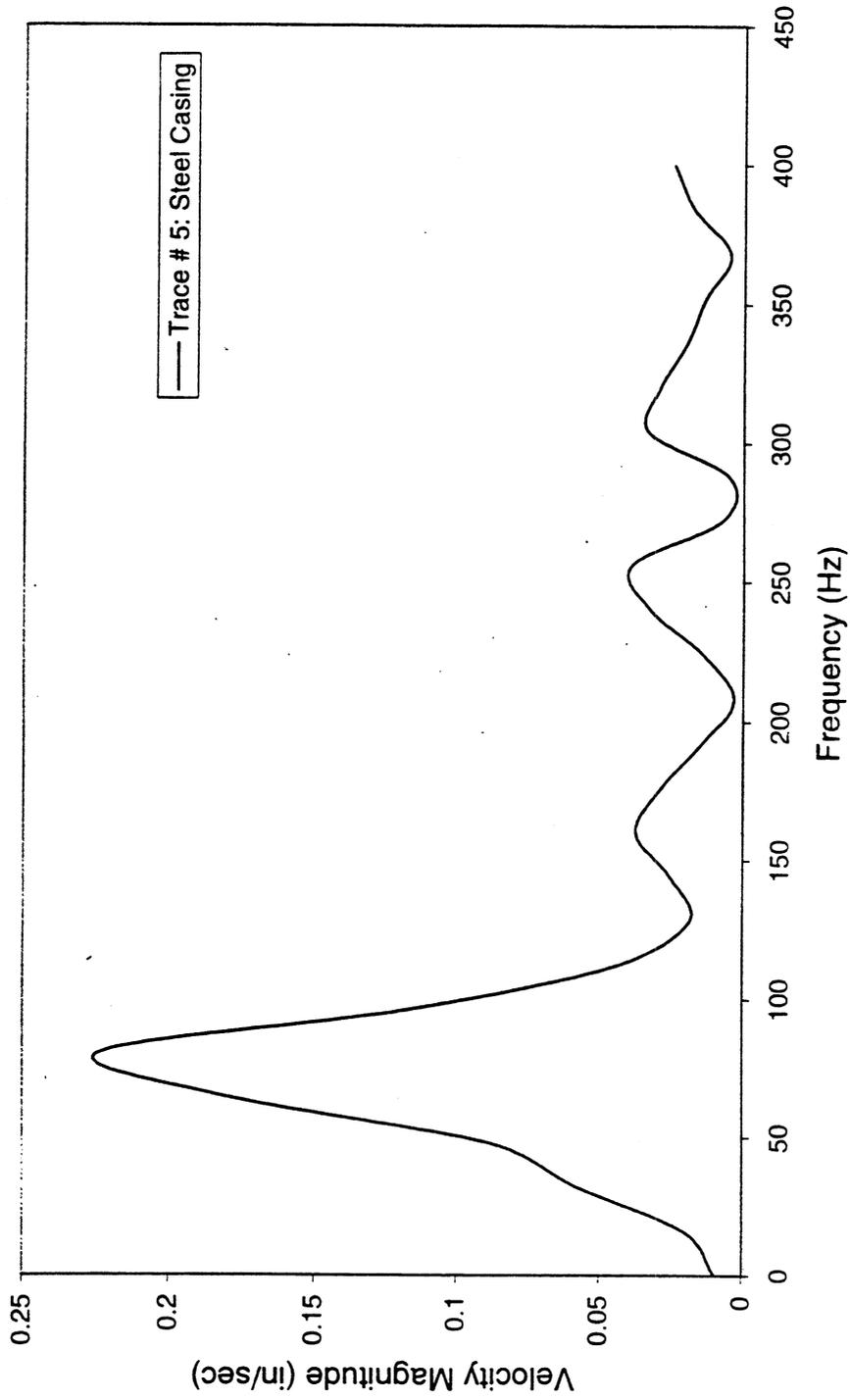


Figure 5.29: Predominant frequency of the steel casing at penetration depth of 8 ft.

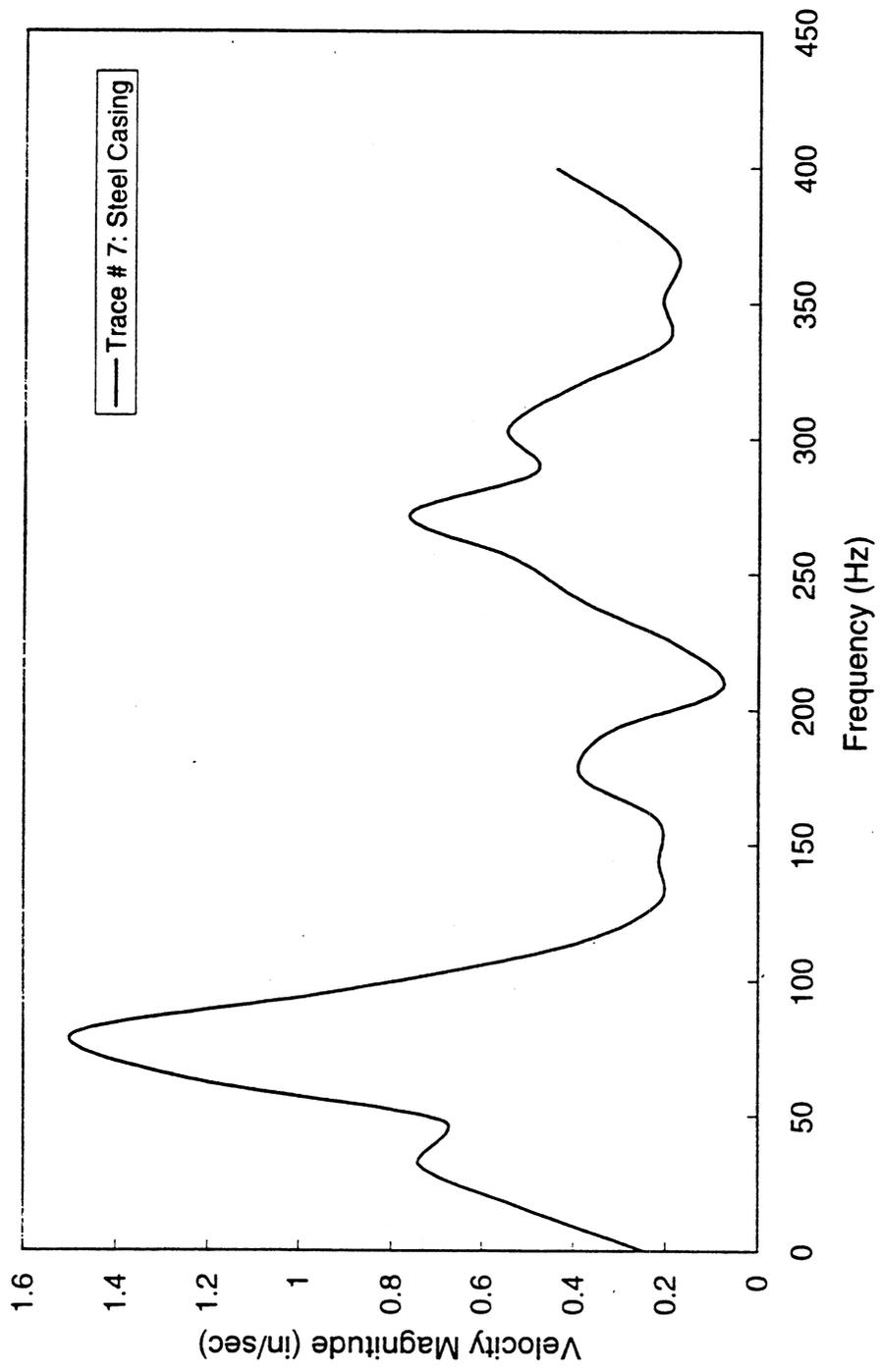


Figure 5.30: Predominant frequency of the steel casing due to a penetration depth of 10 ft in the ground

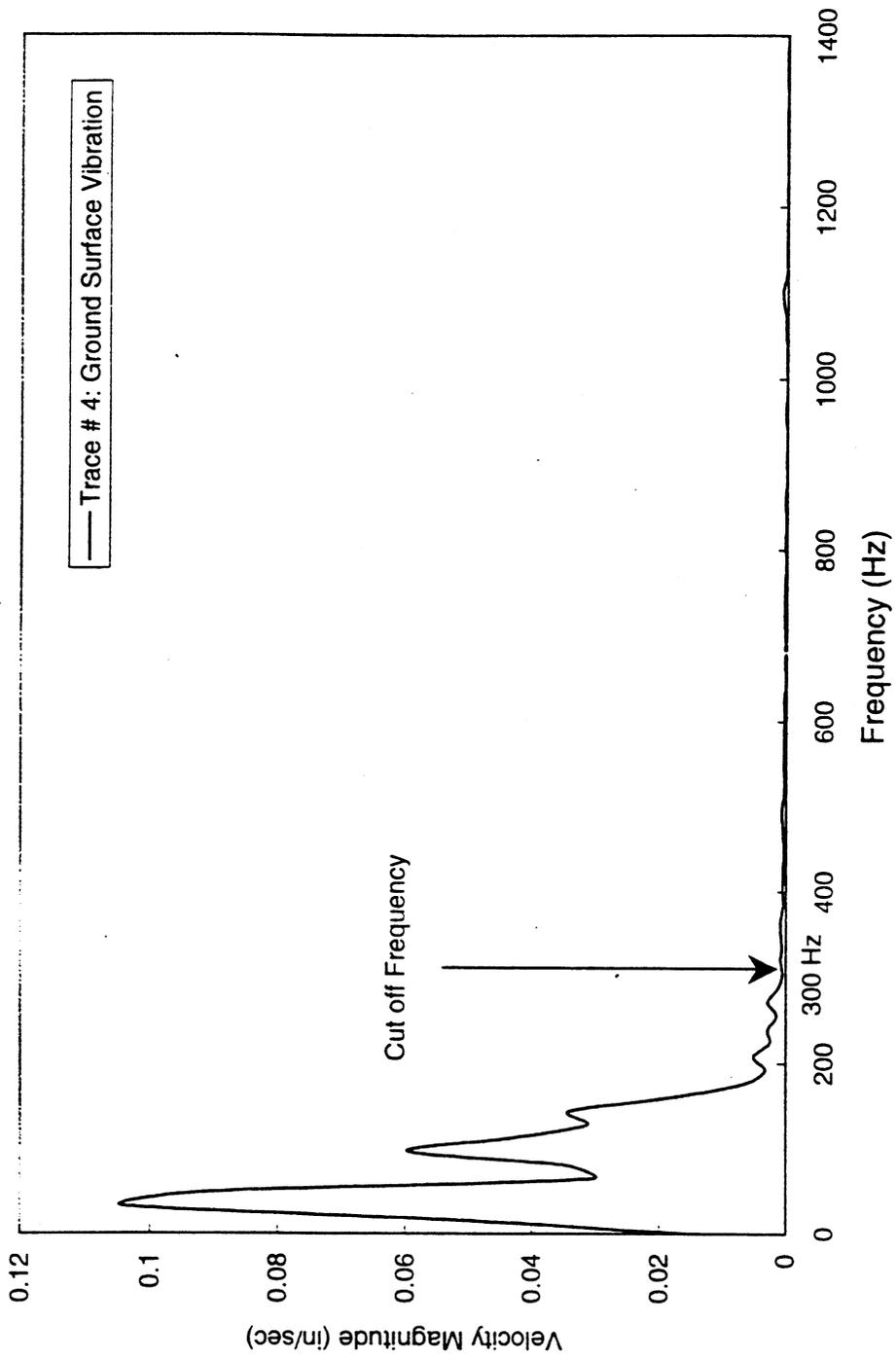


Figure 5.31: Frequency content of the ground surface vibration due to an 8 ft casing penetration

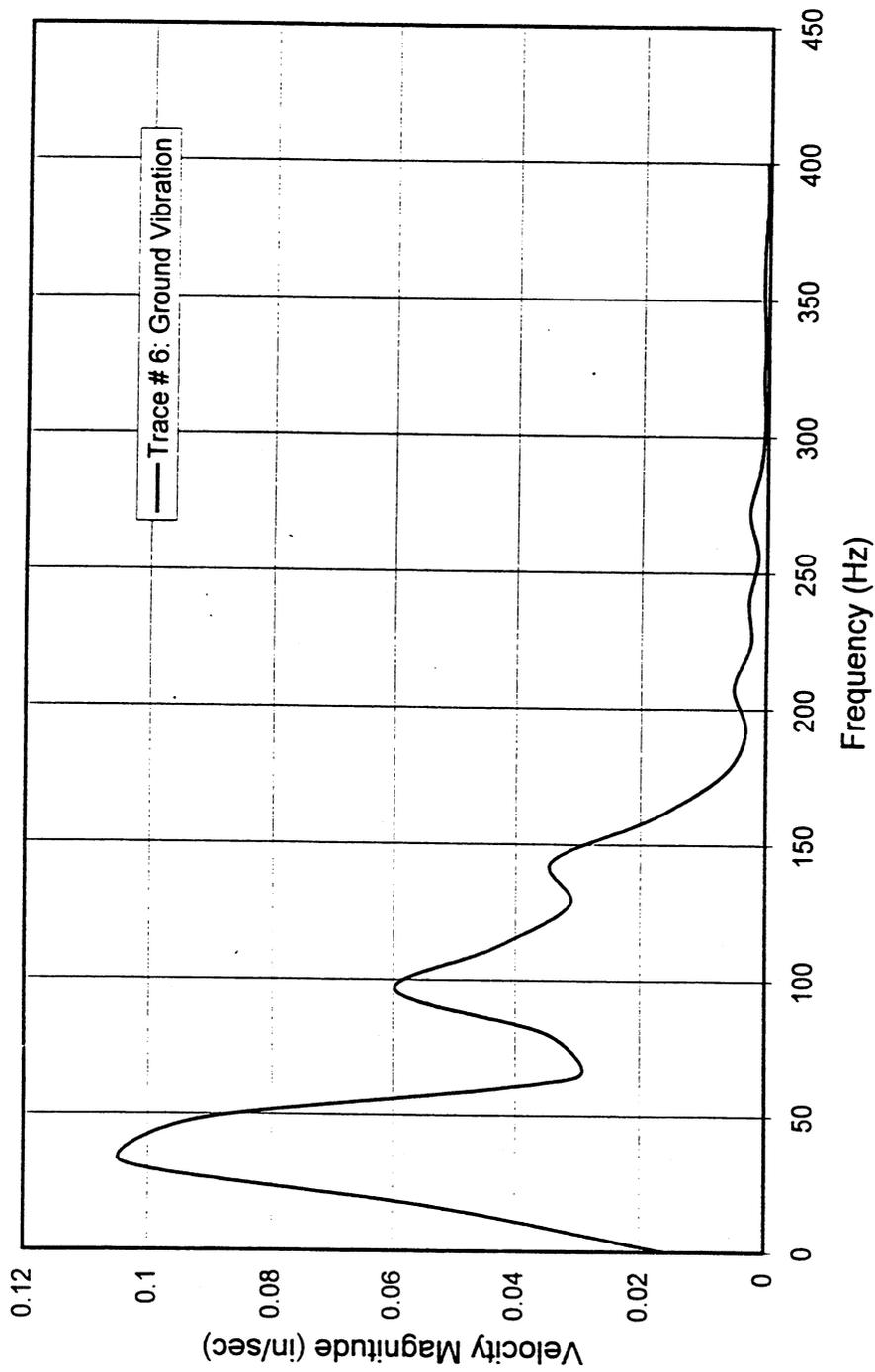


Figure 5.32: Dominant frequency of the ground surface due to 8 ft casing penetration.

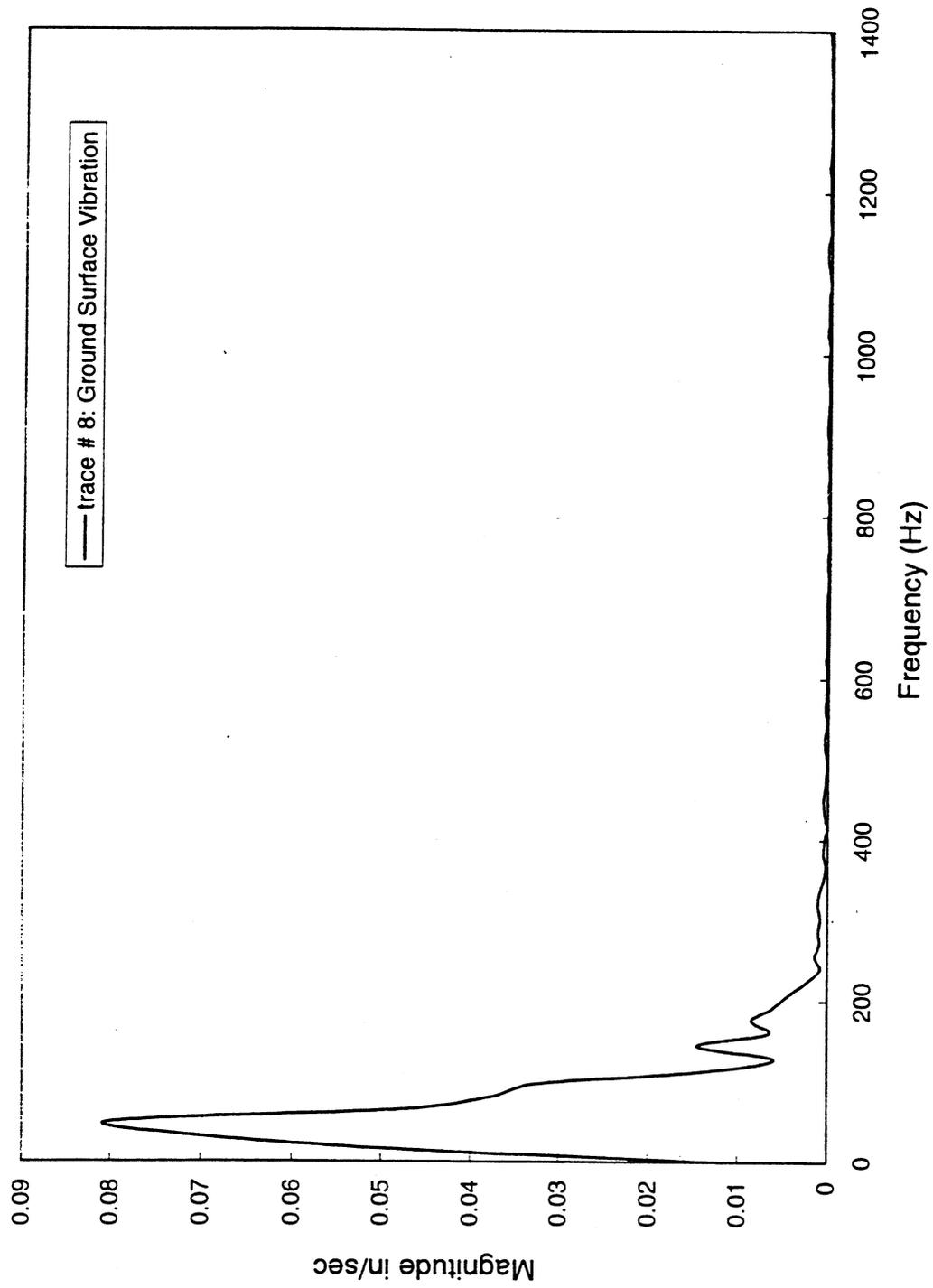


Figure 5.33: Frequency content of the ground surface due to a casing penetration of 10 ft.

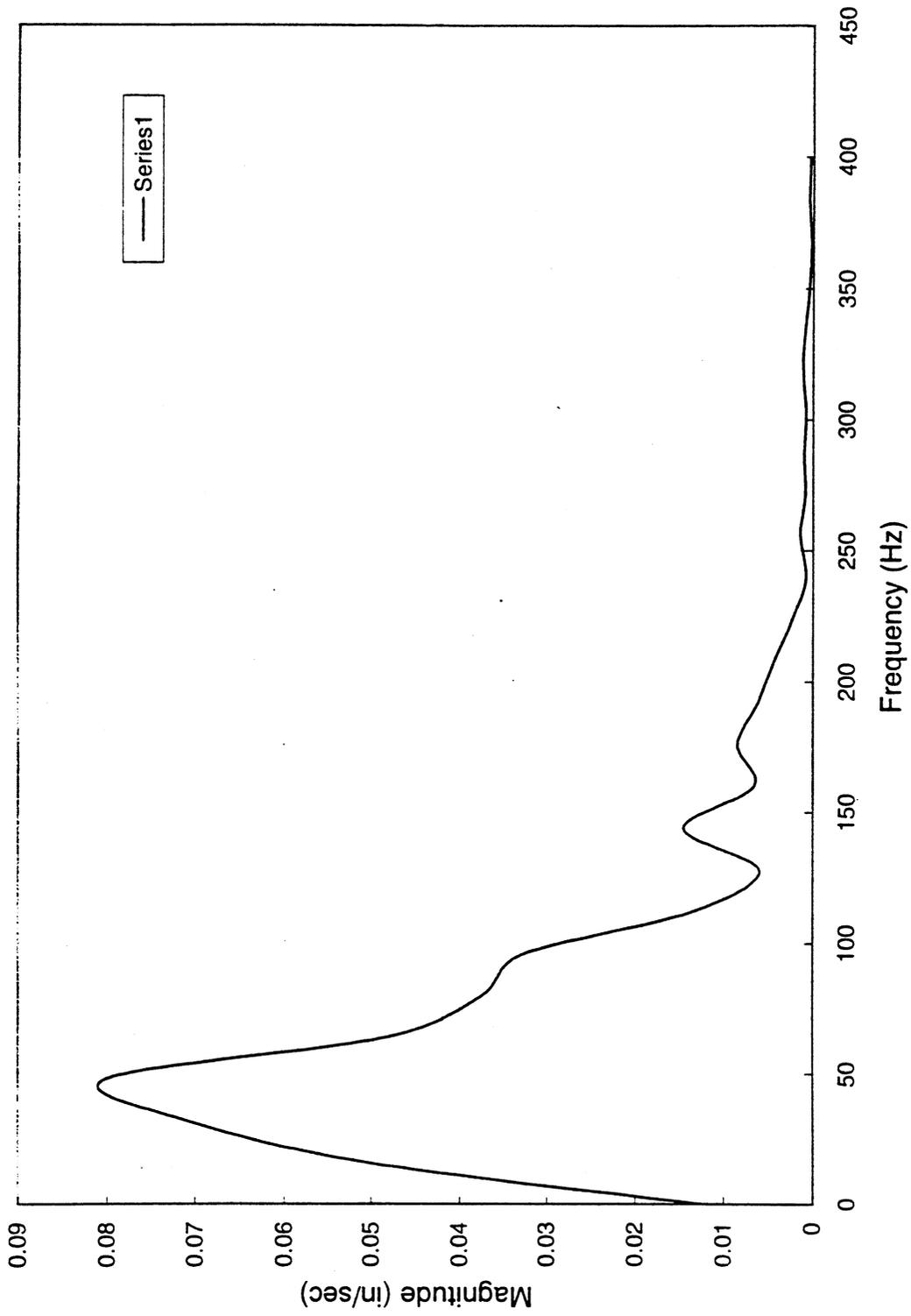


Figure 5.34 : Predominant Frequency of the ground surface due to a casing penetration of 10 ft.

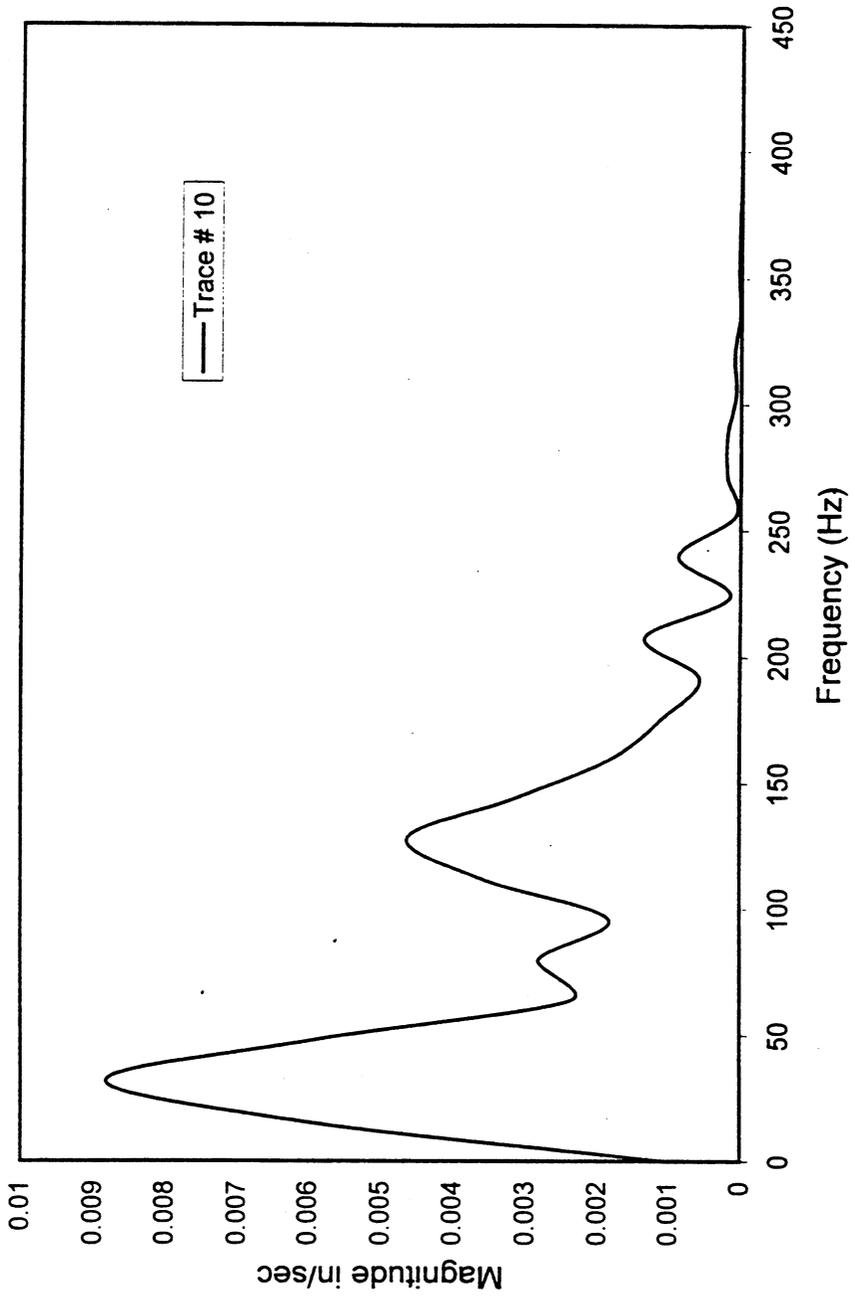
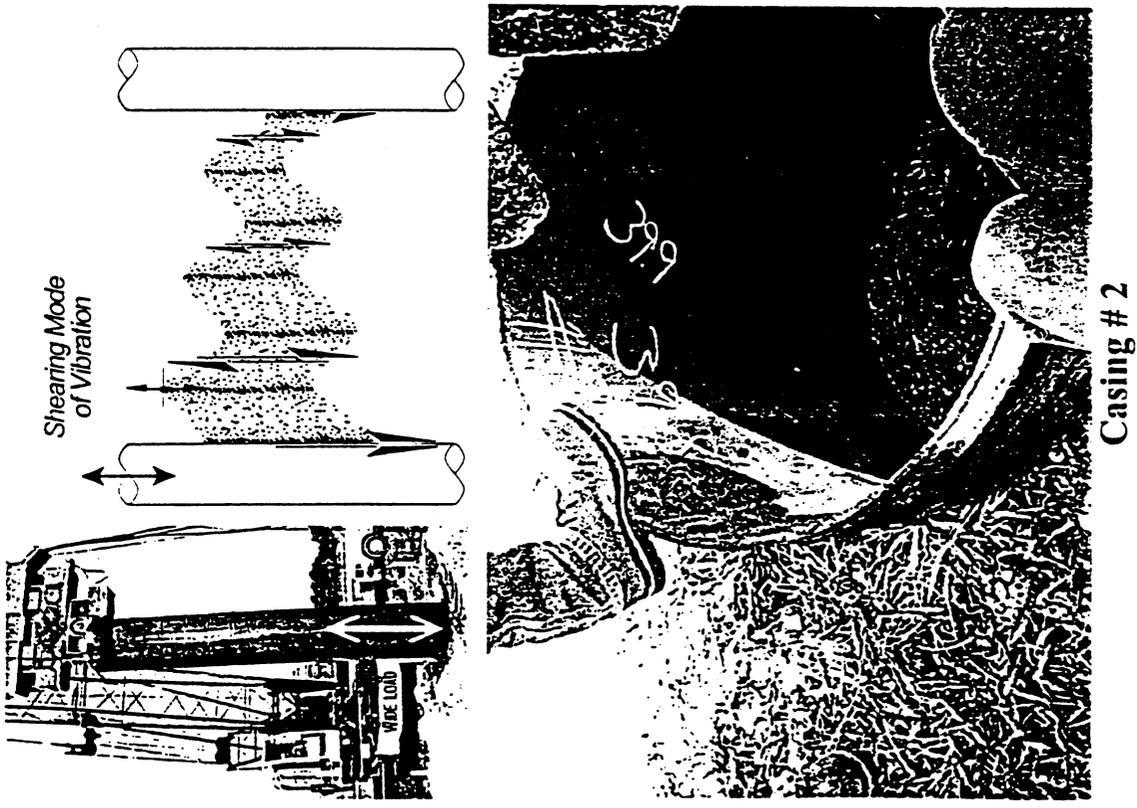


Figure 5.35: Dominant frequency of the ground surface due to 12 ft casing penetration.



Casing # 1

Casing # 2

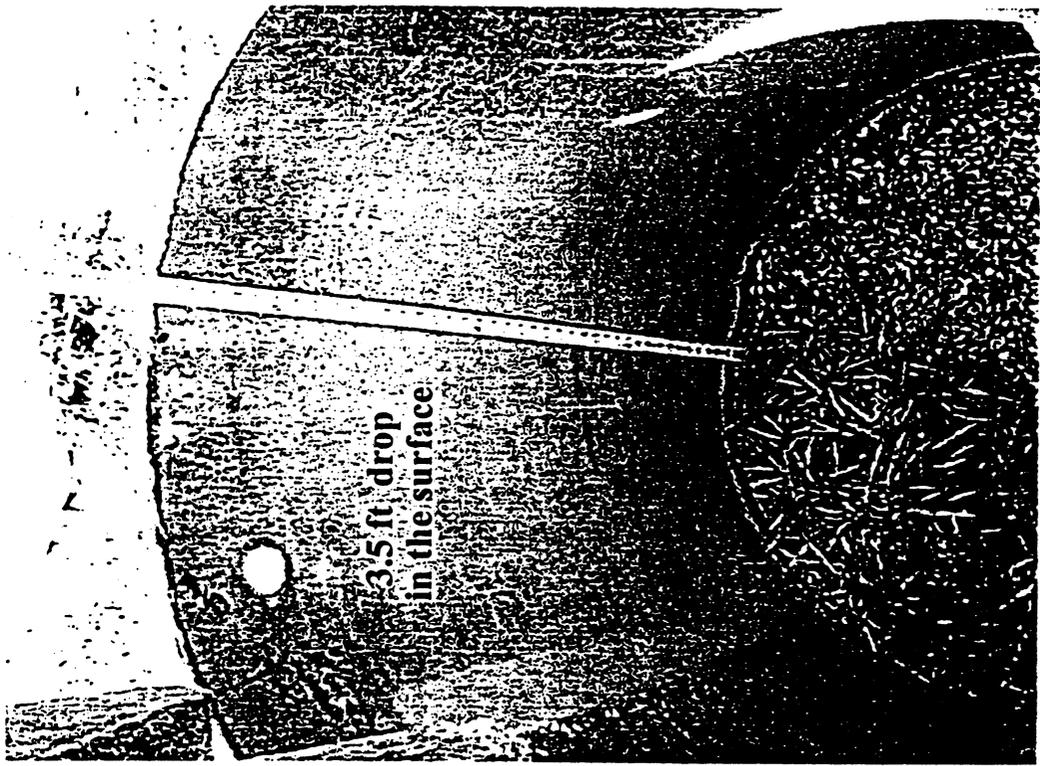


Figure 5.36. Drop in the ground level after completing the driving process.

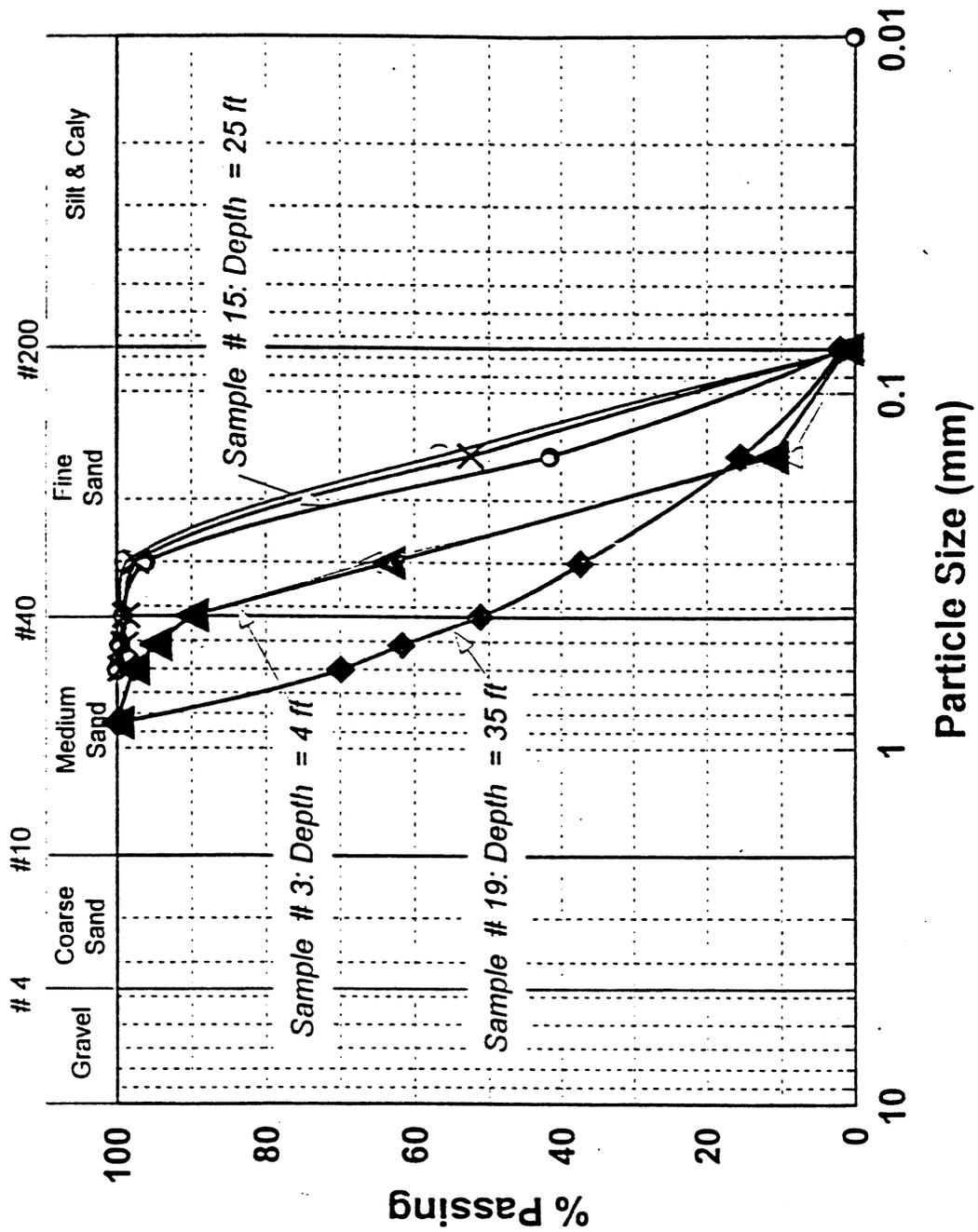


Figure 5.37. Grain size distribution of sand samples obtained from FAU Site.

to determine the peak particle velocities at different depths. Six of the eight geophones gave reasonable vibration measurements. The other two, embedded at 30 feet and 35 feet, malfunctioned. The geophones were placed in a circular pattern with the driven casing in the center and at a 9 feet radius. The needed measurements were collected by Law Engineering and the analysis was performed at Florida State University.

Peak particle velocities obtained from this field test indicated that some vibration components might dominate the maximum values. These components were in the longitudinal and the transverse directions. Most of the measurements, however, indicated that the vertical component prevailed. The reason for the escalation of the longitudinal and the transverse components could be due to the eccentric vibration that was applied on the steel casing by the vibrator. As it was explained in a previous section, the gripping of the steel casing was on one side only. The same was done when the casing was extracted. This eccentricity may have caused side vibrations in the casing which were manifested in large velocities in the L and T directions.

Selecting the vertical component (shear mode) of the peak particle velocity necessitated the re-examination of all the time-history records collected in the field. Such an examination was time consuming and can not be used for regular practice. It is customary to record, as a final result, the peak particle velocity from any of the three orthogonal directions. Any velocity that gives the maximum value is usually reported. Evaluation of the PPV values that are reported during construction vibration monitoring, may not indicate the mode of vibration that dominated the response.

In this study it was important to distinguish between these modes in order to assess the type of vibrations propagated through the system. For example, if the PPV was for the vertical mode then the conclusion is that the type of vibration would mostly be of a shear type. This is only true

for the driven steel casing. The reason is that the generated vibrations in the ground would predominantly be due to the friction between the casing surface and the surrounding soil. However, some kind of soil displacement may occur. This lateral displacement could give rise to the development of longitudinal vibration or compression vibrations. Since the lateral displacement in this case is very low, as compared to the large displacement caused by pile driving, the longitudinal and the transverse modes should be low accordingly.

Additional indication of the effect of vertical vibration on the surrounding soil was noticed in the first and the second field tests. After driving the full length of the casing, it was found that the ground level inside the casing was much lower than the one outside the casing (**Figure 5.36**). The drop between the two levels was about 3.5 feet.

The use of the PPV without knowing the mode of vibration would defeat the purpose of using the concept itself. One of the main objectives of specifying the particle velocities in monitoring construction vibration is to be able to relate to the strain levels that the vibrations induce in the system.. **Equation 2.4 in Chapter 2** showed that the strain amplitude can be determined from the particle velocity and the wave velocity through the system. If the compressive strain is of interest in monitoring the construction vibration, then the compression wave velocity has to be determined; accordingly the compression (V, L, or T) mode of the vibration should be used. If the shear strain is the critical response, then, all the shear components should be determined.

In this regard, soil samples obtained from the boring logs were analyzed to determine the shear wave velocities of the various layers at the site. Water contents, void ratios, and unit weight of about 52 samples were determined in the laboratory. Soil property measurements are presented in **Table 5.3**. Based on the obtained soil properties, the shear wave velocity at each

level was determined using the Hardin and Richart (1963) equations. According to Hardin and Richart, the shear wave velocity, v_s , can be determined as:

$$v_s = (170 - 78.2 e) \sigma_o^{1/4} \quad \text{For } \sigma_o > 2000 \text{ psf} \quad (5.3)$$

$$v_s = (119 - 56 e) \sigma_o^{1/5} \quad \text{For } \sigma_o < 2000 \text{ psf} \quad (5.4)$$

where σ_o = octahedral effective stress

Values of the shear wave velocities are presented in **Table 5.4** and **Figure 5.38**. As can be seen from **Figure 5.38**, the effect of the ground water table was noticeable at a depth of 6.7 ft. Shear wave velocities dropped by 13 percent for the soil layer below the water table. At a depth of 12 ft, the shear wave velocity started to exceed the values at 6.7 ft.

Using the SPT results obtained from the two boring logs (**Figures 5.39 to 5.42**), one can notice the similarity in the variations of the wave velocities and the blow counts with depth. This similarity led to the use of **Equation 2.11** in **Chapter 2** to determine the shear wave velocity from the SPT blow counts. Shear wave velocities of the two boring logs at Delray site are also presented in **Figures 5.43 and 5.44**. Results from similar field studies are shown in **Figures 5.45 to 5.47**.

The peak particle velocities recorded by the geophones have shown that as the casing is driven deeply into the ground, the PPV values increased at all depths. For 5 ft casing penetration, the corresponding geophone at the same level (**G1** see **Figure 4.2**) recorded PPV of about 0.18 in/sec. When the penetration depth of the casing reached 40 ft, readings from that geophone (**G1**) showed 2.0 in/sec. Also, at that penetration depth for the steel casing, the sixth geophone (**G6**) at 40 ft depth indicated a PPV values approximately equal to the values at the geophones above (**Figure 5.41**). **Table 5.5** presents all the measurements obtained from the FAU field test. These

measurements include PPVs during both driving and extracting the shaft casing. Additional readings are presented for **G1** and **G6** due the vibration of the second casing (**Figure 3.6**).

From the obtained soil properties, the vertical effective stresses at various depths were calculated. These stresses were plotted against the PPV recorded from all the geophones. It should be mentioned here that the values of PPV were obtained manually, **Figures 5.50 to 5.52**.

$$v = \alpha e^{-\beta\sigma_0} \quad (5.5)$$

where v = peak particle velocity (in/sec)

α and β = penetration depth factors

σ_0 = Effective vertical stress

$$\alpha = -0.115\ln(D) + 0.7$$

$$\beta = 0.57e^{-0.105 D}$$

in which D = penetration depth (ft)

5.3 FIELD MONITORING OF INDUCED GROUND MOTION

The field work included monitoring of installation induced vibrations in the ground for sites 1 and 2. Assessment of vibration disturbances provides an insight into the source mechanism and the eventual dissipation of the energy transmitted to the subsurface during the installation of a steel casing. The data collected is presented to evidence the events monitored during this attempt to characterize the peak particle velocities of particles in the ground. However, the conditions are specific to this test and no attempt was made to relate the motions evaluated here with other soil types. This test was the first of its kind, and all the work in the tasks at FAU is considered exploratory.

The monitoring of induced vibrations during Field Tests Nos. 1 and 2 provided an insight into the actual inner working of the strata and the source mechanism. The data obtained was stored and analyzed on an HP Fast Fourier Transform Analyzer by LAW Engineering

(Jacksonville, FL). The results of which show a frequency content for the driven casing in the range of 2 - 2.2 kHz. **Figure 5.53** shows typical accelerations of the vibrating casing, with the attached accelerometer placed five feet above the ground.

As expected, given the sandy conditions of the site, the energy dissipation from the driving of the casing significantly diminishes with increasing distance from the vibrating source. To show this energy dissipation, **Figure 5.54** is presented. The absolute values of the acceleration at 15 feet away from the source are significantly greater in magnitude than the values recorded at 10 feet.

More interesting data from this preliminary investigation is operated with the monitoring of vibrations in the subsurface. The virtual shaft discussed in Chapter IV shows what typical vibrations could be expected at varying locations at depth within the shaft. These vibrations are in fact those that can cause detrimental effects on the curing of the concrete in the shaft. With the geophones implanted, the vibrations generated during the installation procedure were recorded. A large volume of data was acquired during this exploratory phase. The sampling rate was 200 samples per second using the data acquisition system. The recordings were conducted in series, one geophone at a time in one-second interval.

As already indicated, a summary of the velocities observed during the installation has been presented in **Table 5.5**. The table shows the geophone numbers, their locations at depths and the average velocities of their recordings at varying driving ranges of the steel casing. The data is interpreted in **Figures 5.53 and 5.54**. The vibration levels at individual geophones during driving ranges is presented in **Figure 5.53**, in which a trend for each individual geophone is plotted. From the graph of the velocities at the location of each geophone during the stages of driving, it can be observed that the velocities increase as the tip of the steel casing penetrates to greater depth. Another presentation of the data is the recording of vibration levels during the installation as a function of the driving and the levels recorded by the geophones. **Figure 5.54** shows the results of the average recorded vibrations for the different stages of driving. These stages are representative of the tip elevation of the steel casing as it penetrated into the subsurface. From the graph it can be shown that for all but the last driving stage (30-40 feet), the vibration of the free end of the subsurface experiences the highest levels of vibrations. In other

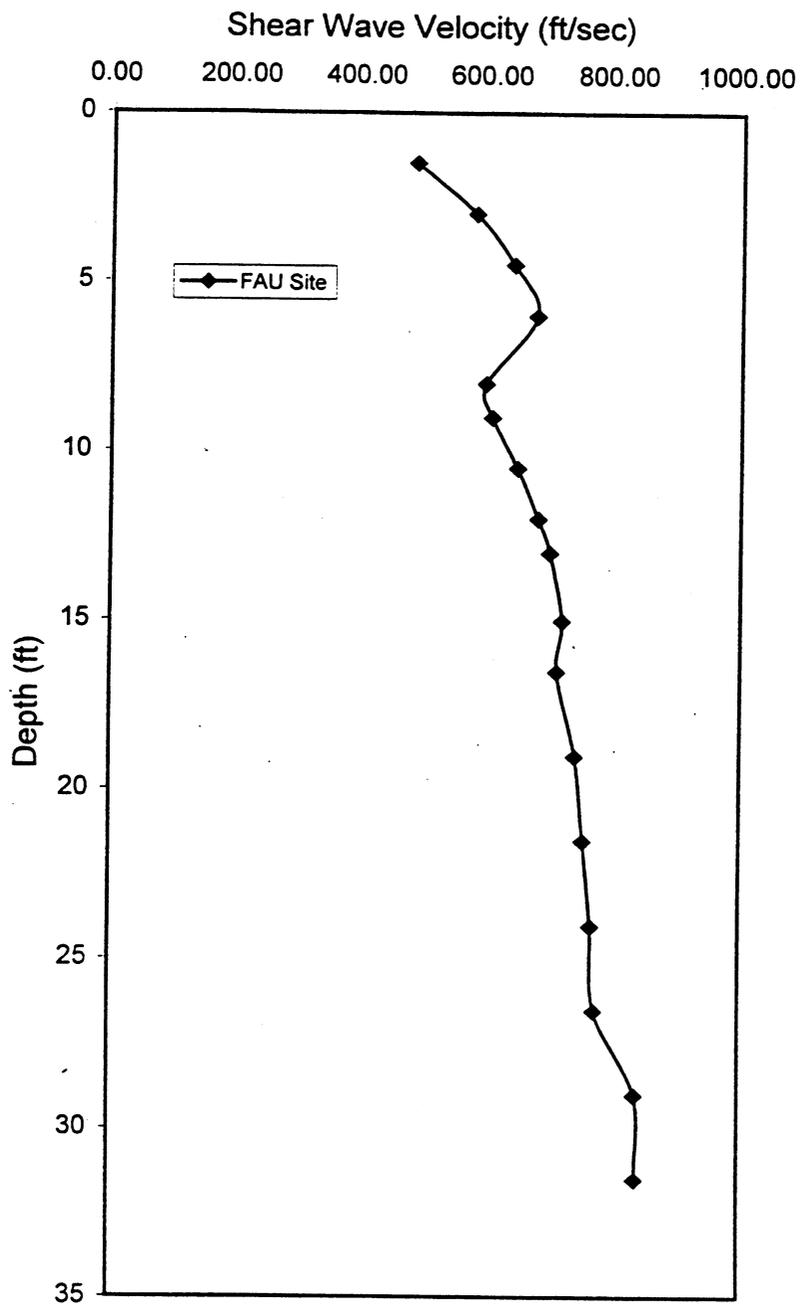


Figure 5.38: Shear wave velocity vs. depth at FAU Site

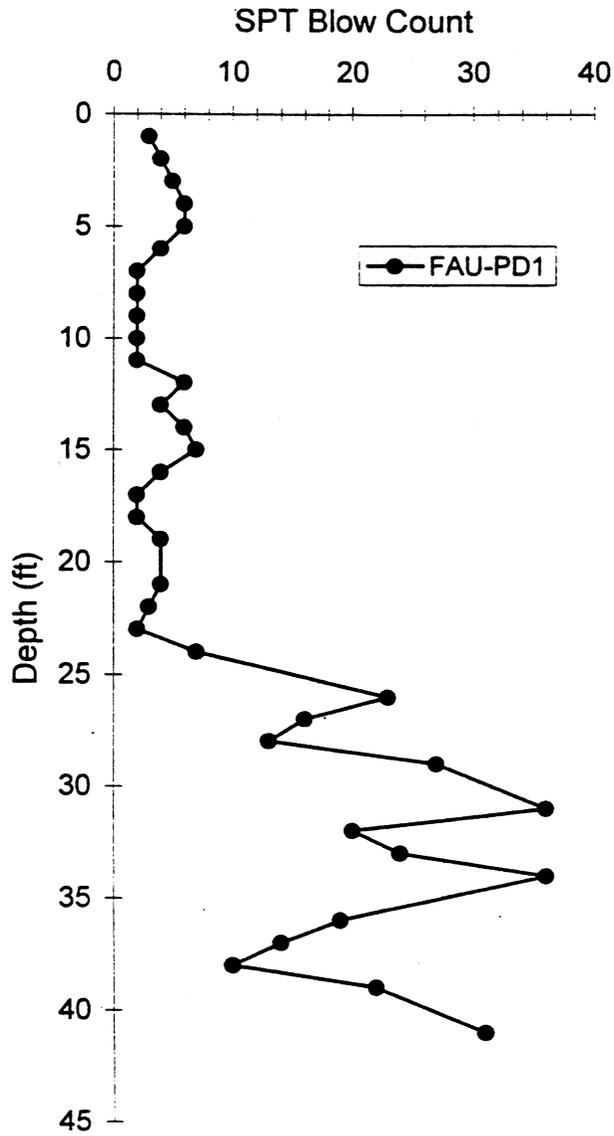


Figure 5.39: Standard Penetration Test results of boring # 1 at FAU site.

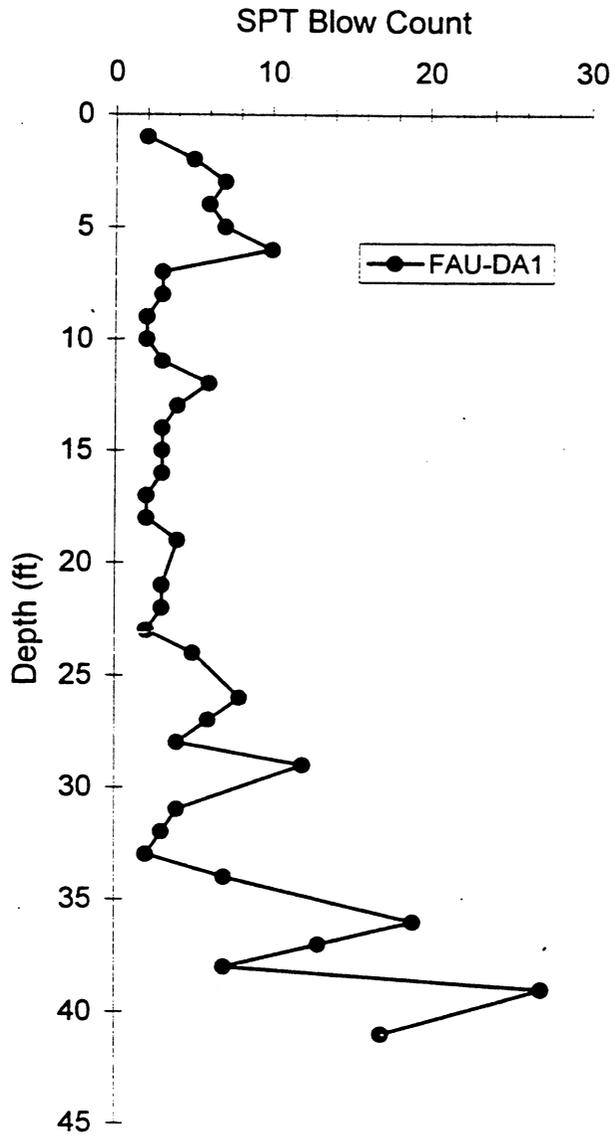


Figure 5.40: Standard Penetration Test results of boring # 2 at FAU site.

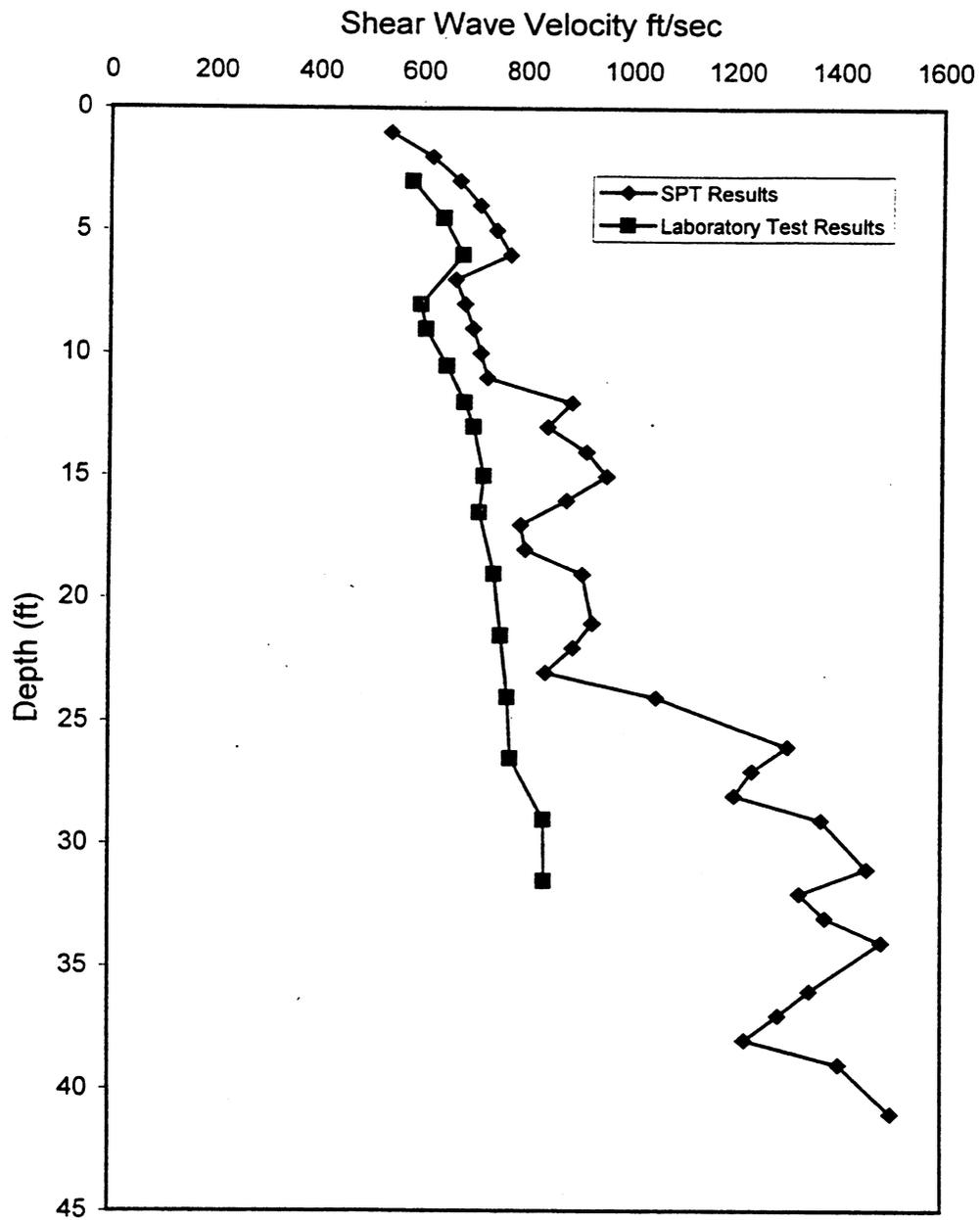


Figure 5.41: Shear wave velocity vs. depth using laboratory test results and SPT blow count values.

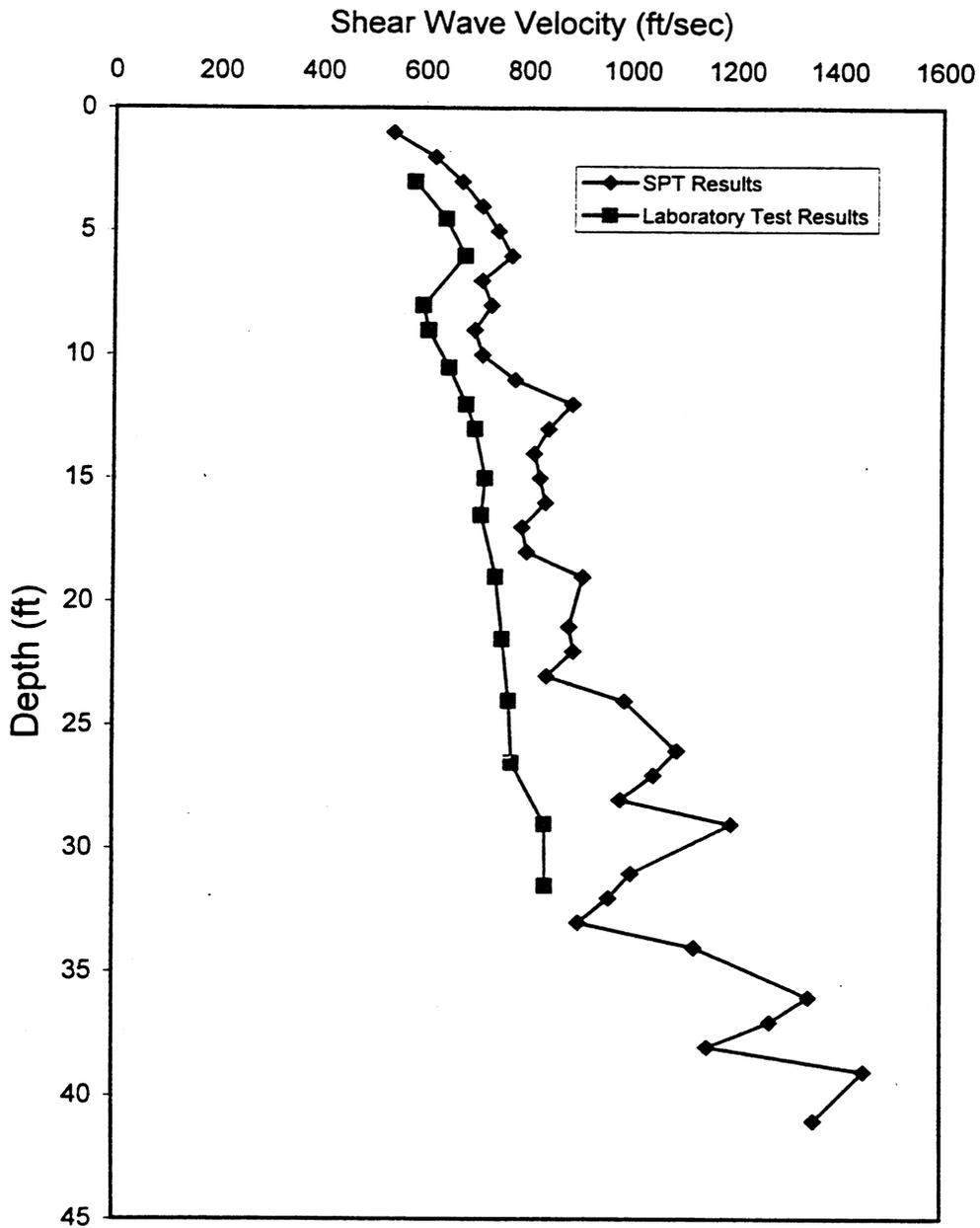


Figure 5.42: Shear wave velocity vs. depth using laboratory test results and SPT blow count values for the second boring log.

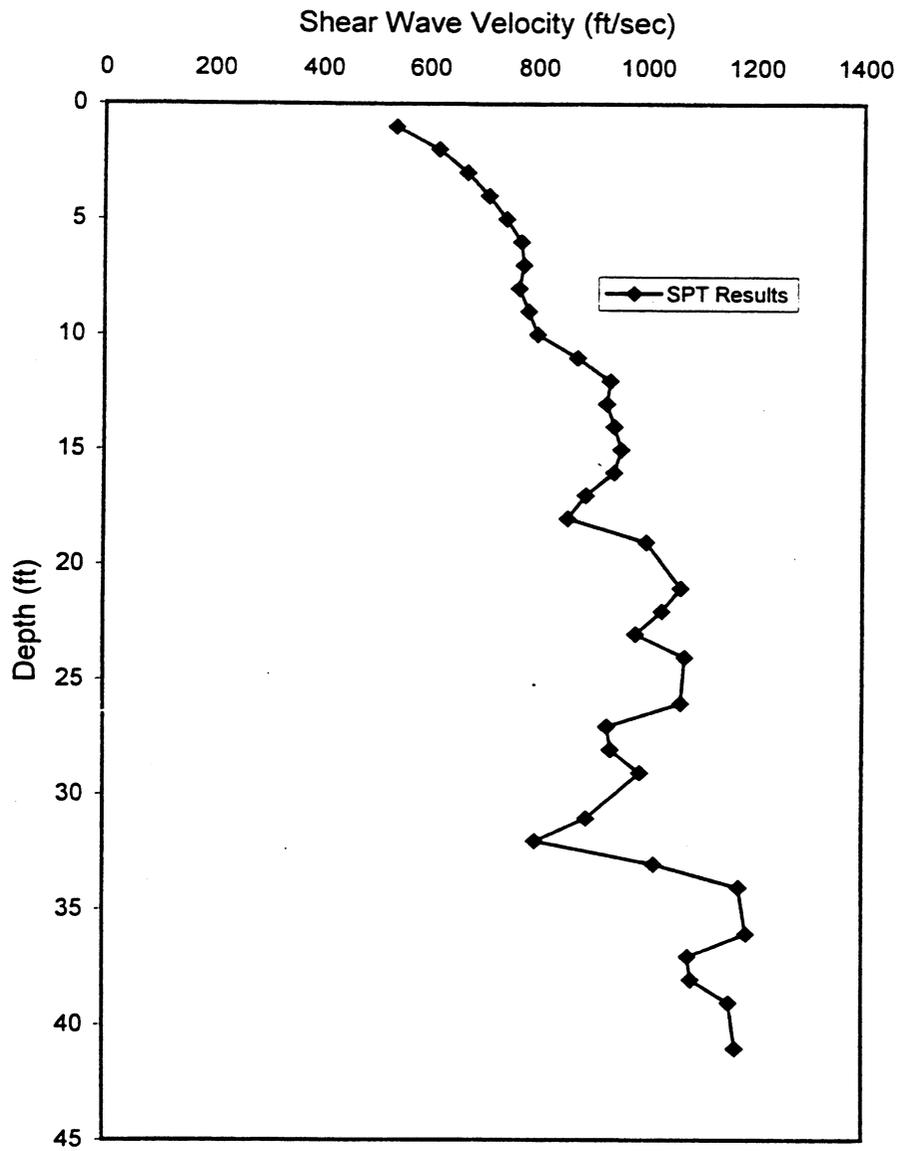


Figure 5.43: Shear wave velocity vs. depth using laboratory test results and SPT blow count values for the Delray Site.

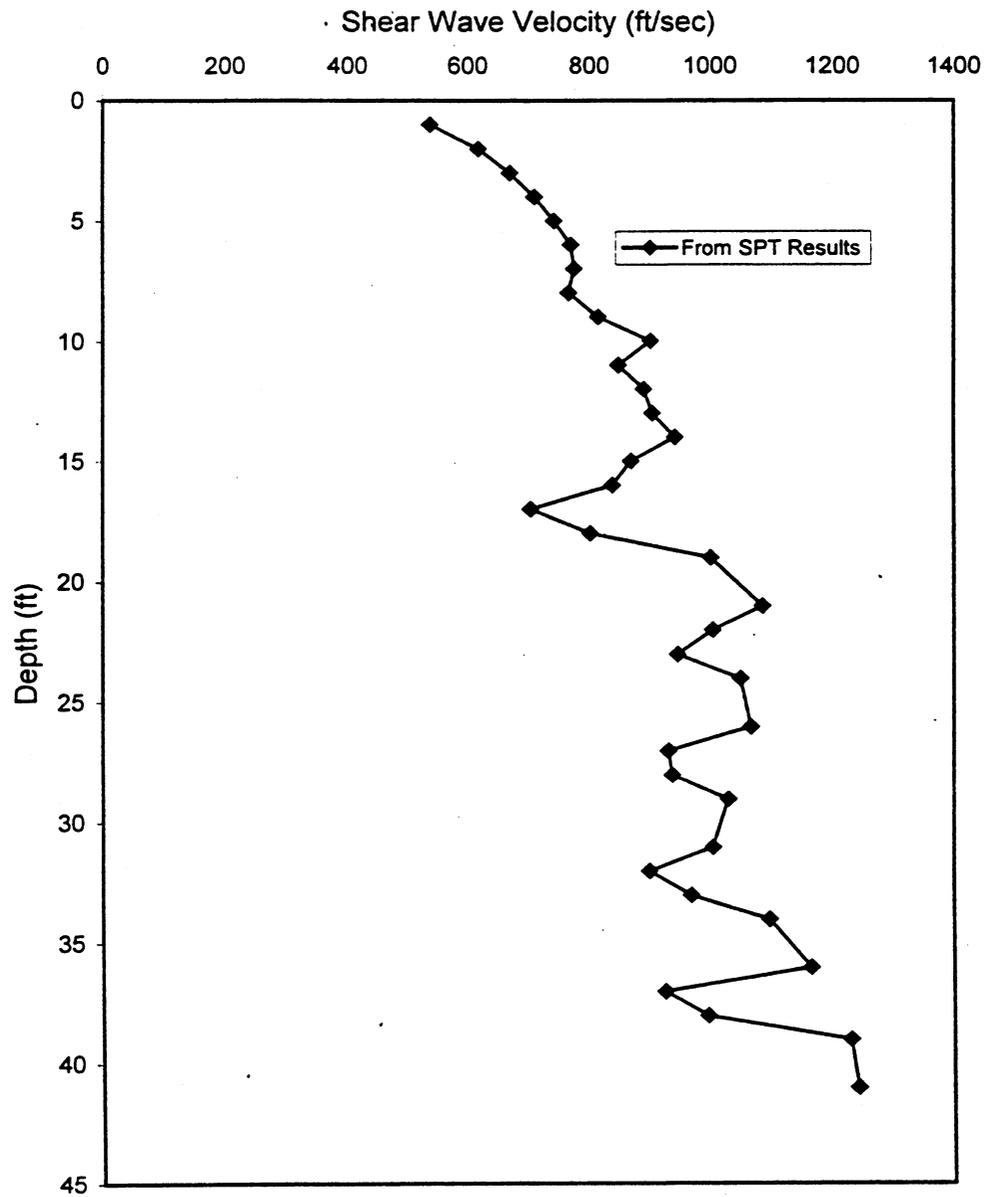


Figure 5.44: Shear wave velocity vs. depth using laboratory test results and SPT blow count values for the Delray Site Boring # 2.

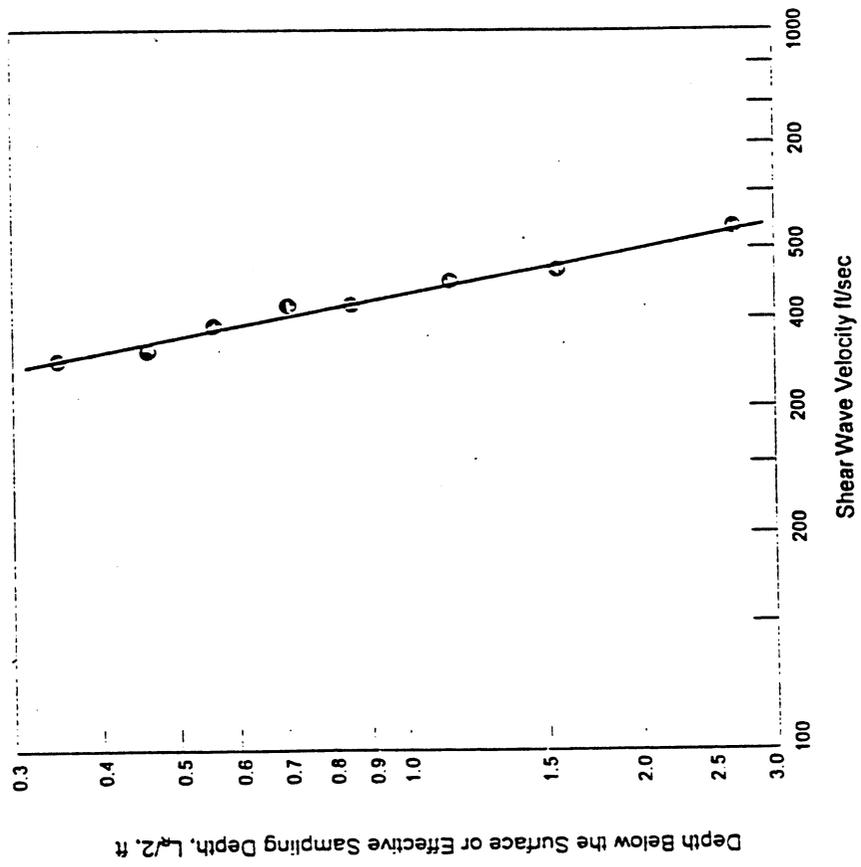


Figure 5.45 Shear wave velocity vs. Depth (Richart et al, 1970)

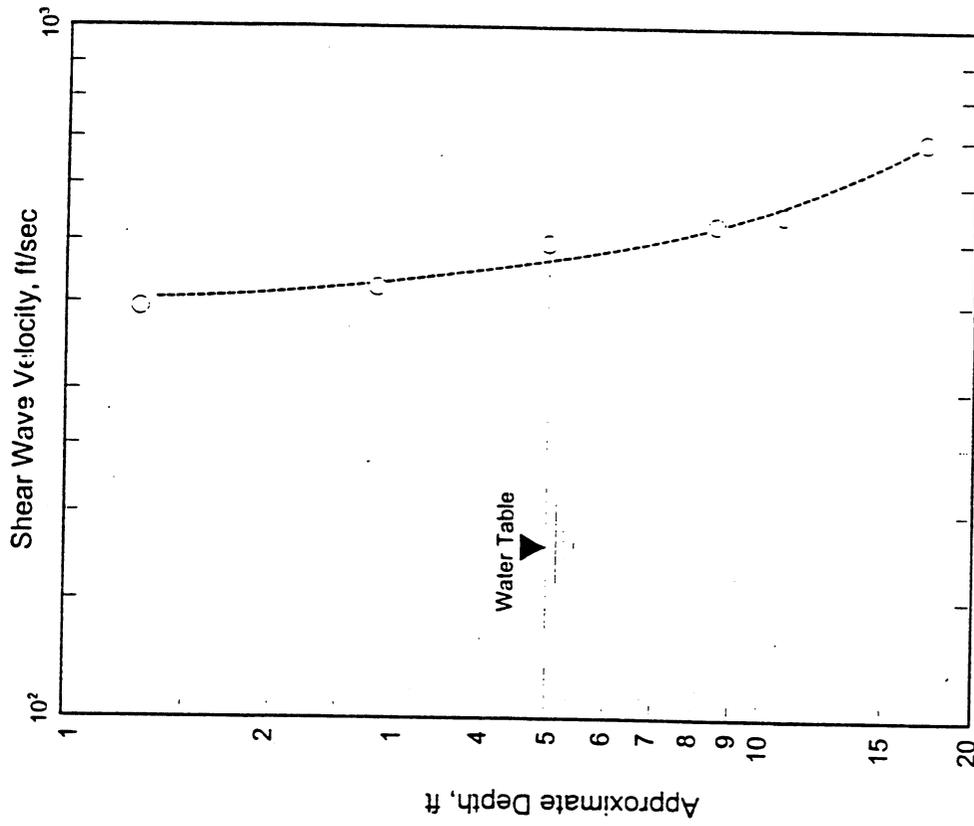


Figure 5.46. Shear wave velocity vs. Depth in uniform, fine sand
(after Ballard and Cassagrande, 1967)

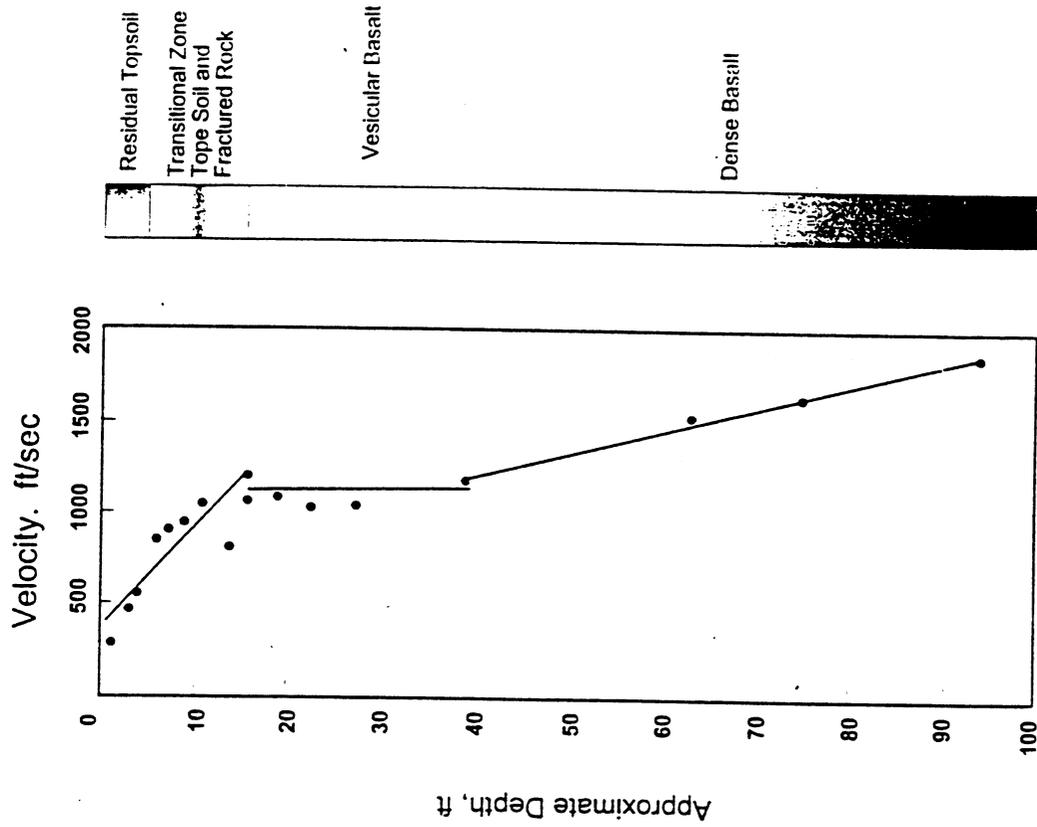


Figure 5.47. Shear wave velocity vs. Depth in rock profile (after Fry, 1965)

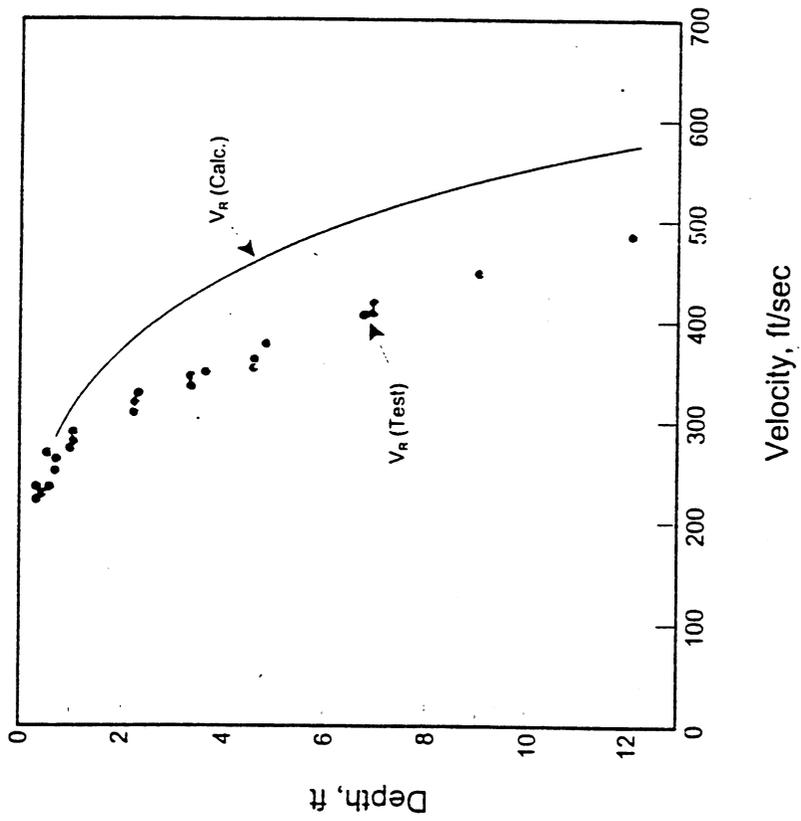


Figure 5.48. Variation of Rayleigh wave velocity with depth-Eglin Field Site
(after Fry, 1963)

Table 5.5: Particle Velocities vs. penetration depth and vertical effective stress

Geophone No	Geophone Depth (ft)	Unit Weight (lbm/cm3)	Effective Unit Weight (lb/ft3)	Effective Vertical Stress (lb/ft2)	Effective Vertical Stress (lbm/ft2)	Approx. Maximum Particle Velocity (ft/sec)	Approx. Maximum Particle Velocity (ft/sec)	Approx. Maximum Particle Velocity (ft/sec)	Penetration Depth
1	5	1.51	94.224	471.12	3.27	0.189	0.023	0.023	0 to 5 ft
2	10	1.686	42.8064	428.064	2.97	0.196	0.025	0.025	
3	15	1.578	36.0672	541.008	3.76	0.180	0.021	0.021	
4	20	1.547	34.1328	682.656	4.74	0.131	0.028	0.028	
5	25	1.526	32.8224	820.56	5.70	0.100	0.008	0.008	
6	40	1.615	38.376	1535.04	10.66	0.021	0.008	0.008	
1	5	1.51	94.224	471.12	3.27	0.23	0.025	0.025	5 to 10 ft
2	10	1.686	42.8064	428.064	2.97	0.22	0.015	0.015	
3	15	1.578	36.0672	541.008	3.76	0.19	0.021	0.021	
4	20	1.547	34.1328	682.656	4.74	0.17	0.014	0.014	
5	25	1.526	32.8224	820.56	5.70	0.13	0.012	0.012	
6	40	1.615	38.376	1535.04	10.66	0.06	0.008	0.008	
1	5	1.51	94.224	471.12	3.27	0.24	0.023	0.023	10 to 15 ft
2	10	1.686	42.8064	428.064	2.97	0.25	0.013	0.013	
3	15	1.578	36.0672	541.008	3.76	0.23	0.013	0.013	
4	20	1.547	34.1328	682.656	4.74	0.18	0.017	0.017	
5	25	1.526	32.8224	820.56	5.70	0.15	0.015	0.015	
6	40	1.615	38.376	1535.04	10.66	0.09	0.008	0.008	
1	5	1.51	94.224	471.12	3.27	0.25	0.025	0.025	15 to 20
2	10	1.686	42.8064	428.064	2.97	0.27	0.016	0.016	
3	15	1.578	36.0672	541.008	3.76	0.23	0.010	0.010	
4	20	1.547	34.1328	682.656	4.74	0.21	0.025	0.025	
5	25	1.526	32.8224	820.56	5.70	0.18	0.017	0.017	
6	40	1.615	38.376	1535.04	10.66	0.13	0.009	0.009	
1	5	1.51	94.224	471.12	3.27	0.29	0.025	0.025	20 to 30
2	10	1.686	42.8064	428.064	2.97	0.30	0.021	0.021	
3	15	1.578	36.0672	541.009	3.76	0.28	0.023	0.023	
4	20	1.547	34.1328	682.656	4.74	0.25	0.017	0.017	
5	25	1.526	32.8224	820.56	5.70	0.22	0.019	0.019	
6	40	1.615	38.376	1535.04	10.66	0.24	0.011	0.011	
1	5	1.51	94.224	471.12	3.27	0.35	0.025	0.025	30 to 40
2	10	1.686	42.8064	428.064	2.97	0.36	0.023	0.023	
3	15	1.578	36.0672	541.008	3.76	0.35	0.021	0.021	
4	20	1.547	34.1328	682.656	4.74	0.34	0.019	0.019	
5	25	1.526	32.8224	820.56	5.70	0.34	0.018	0.018	
6	40	1.615	38.376	1535.04	10.66	0.34	0.010	0.010	
1	5	1.51	94.224	471.12	3.27	0.85	0.010	0.010	Extraction
2	10	1.686	42.8064	428.064	2.97	0.48	0.010	0.010	
3	15	1.578	36.0672	541.008	3.76	0.35	0.010	0.010	
4	20	1.547	34.1328	682.656	4.74	0.7	0.010	0.010	
5	25	1.526	32.8224	820.56	5.70	0.7	0.010	0.010	
6	40	1.615	38.376	1535.04	10.66	0.6	0.010	0.010	
1	5	1.51	94.224	471.12	3.27	0.25	0.010	0.010	2nd Shaft
2	10	1.686	42.8064	428.064	2.97	0.12	0.010	0.010	
3	15	1.578	36.0672	541.008	3.76	0.15	0.010	0.010	
4	20	1.547	34.1328	682.656	4.74	0.15	0.010	0.010	
5	25	1.526	32.8224	820.56	5.70	0.35	0.010	0.010	
6	40	1.615	38.376	1535.04	10.66	0.6	0.010	0.010	

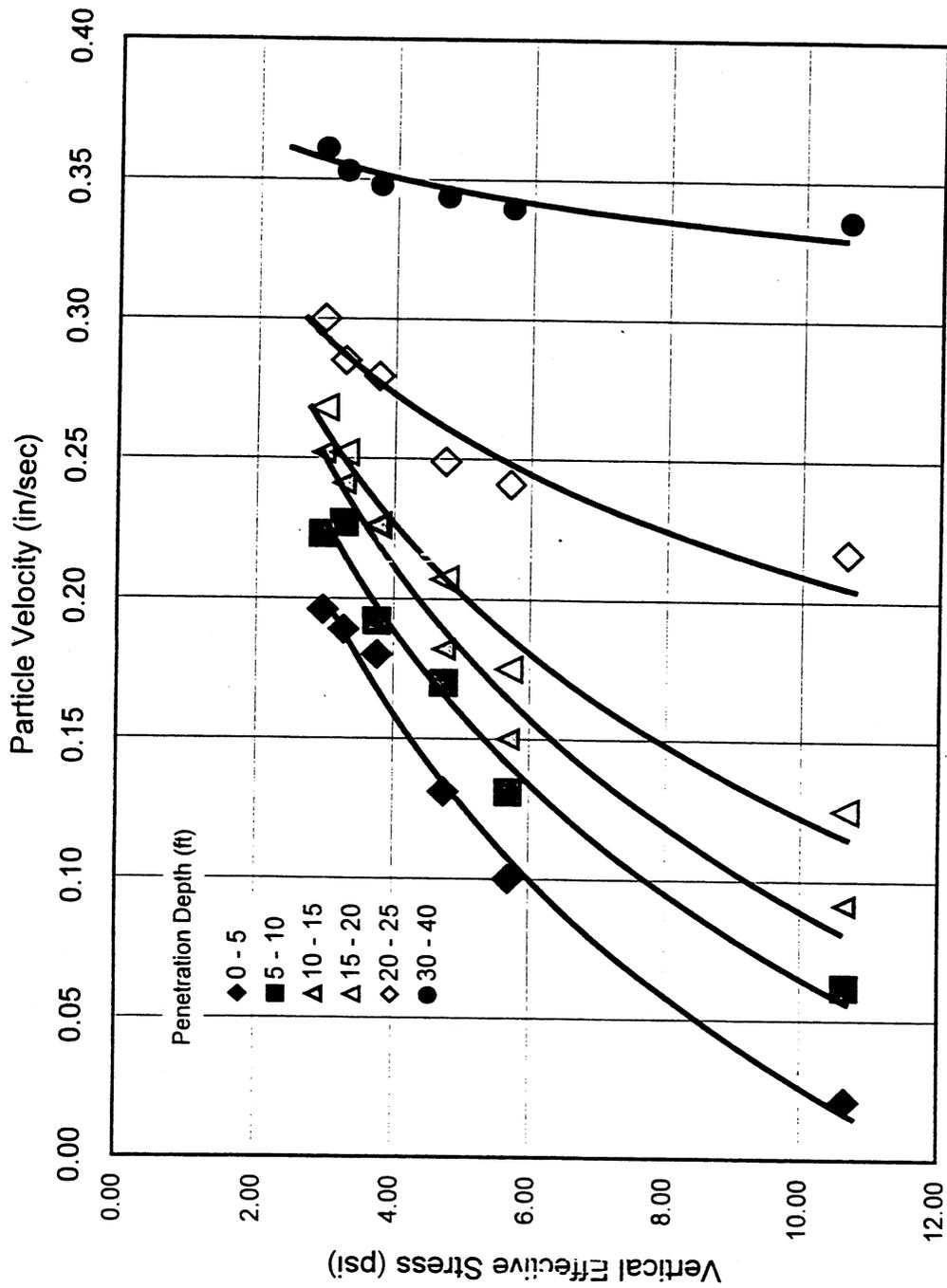


Figure 5.49: Particle velocity vs. Penetration depth of the steel casing

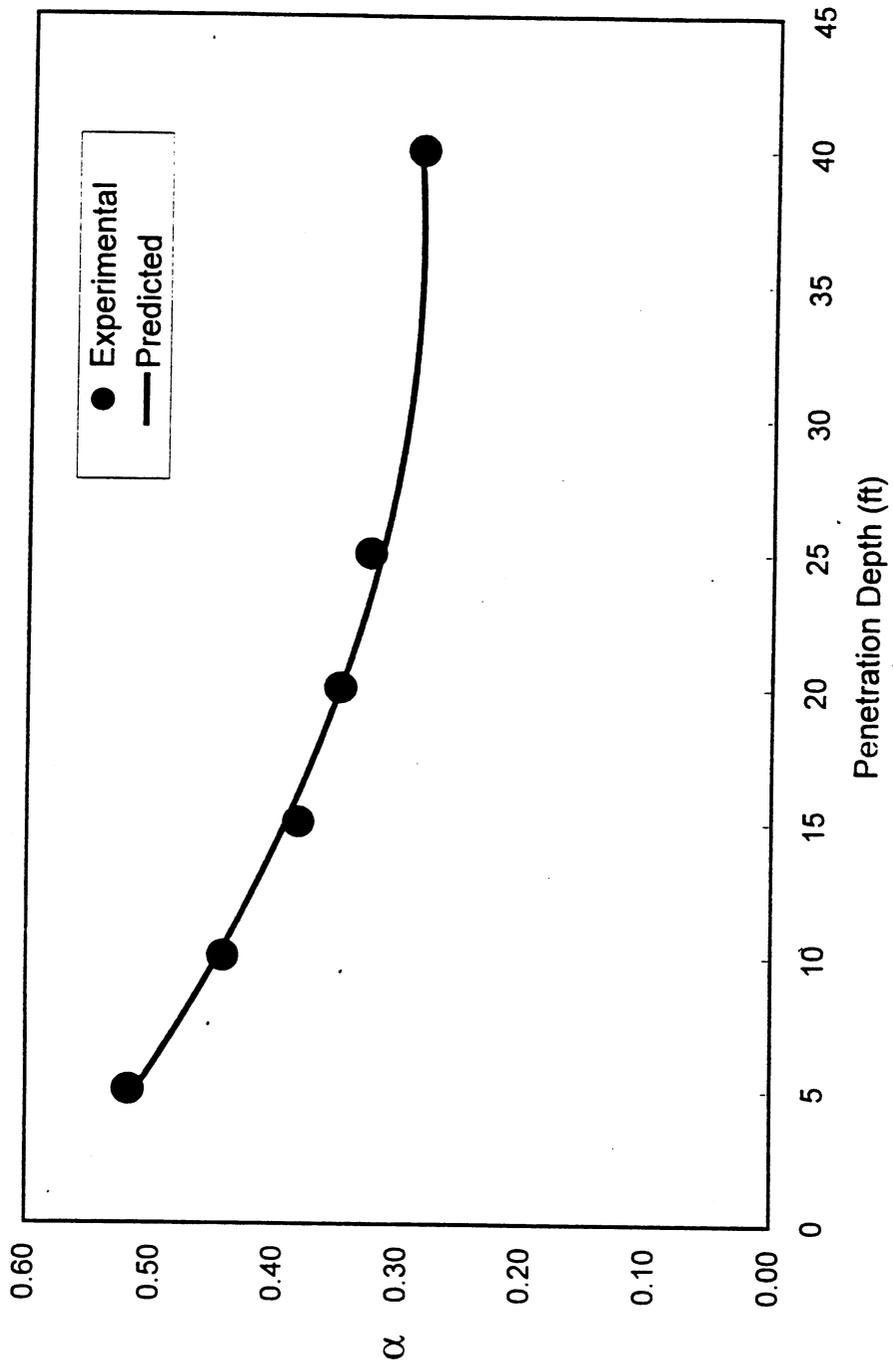


Figure 5.50: Factor α vs. Penetration Depth of the Drilled Shaft Casing

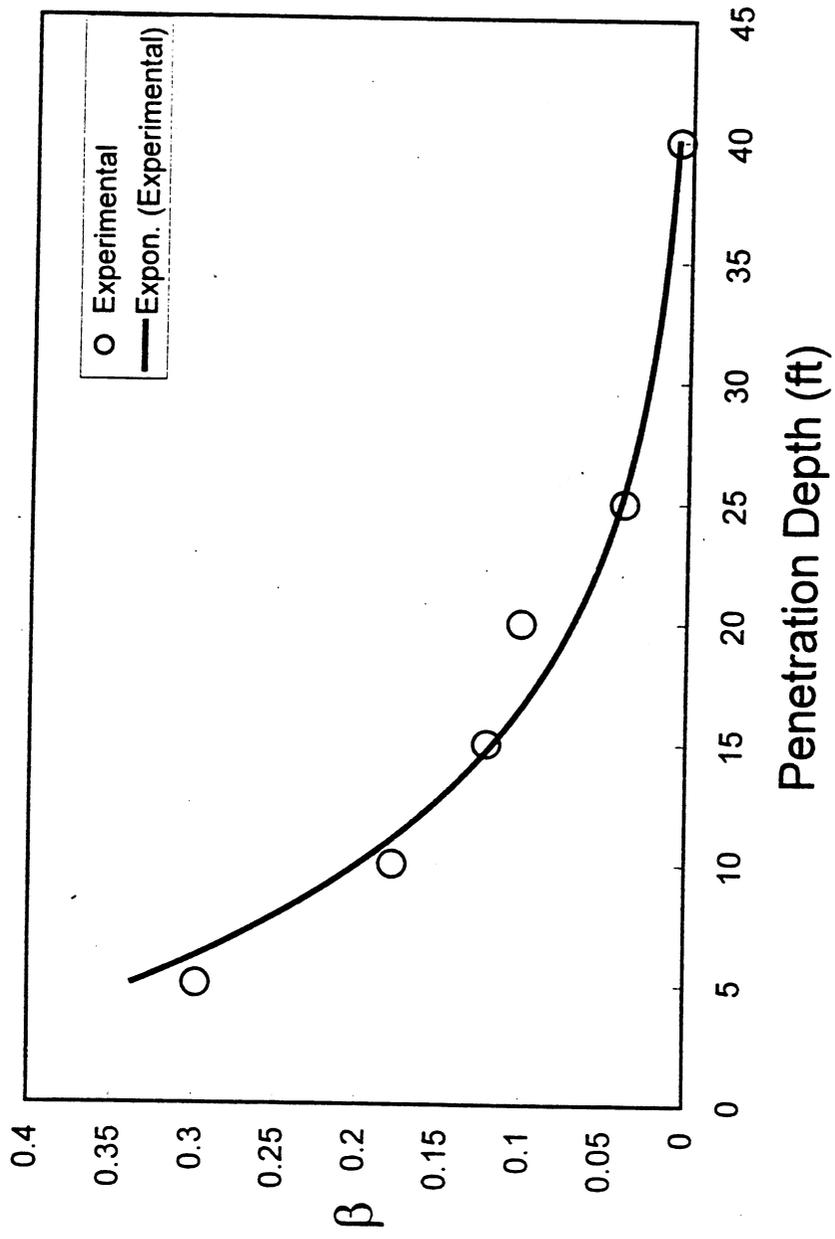


Figure 5.51: Factor β vs. Penetration Depth of the Drilled Shaft Casing

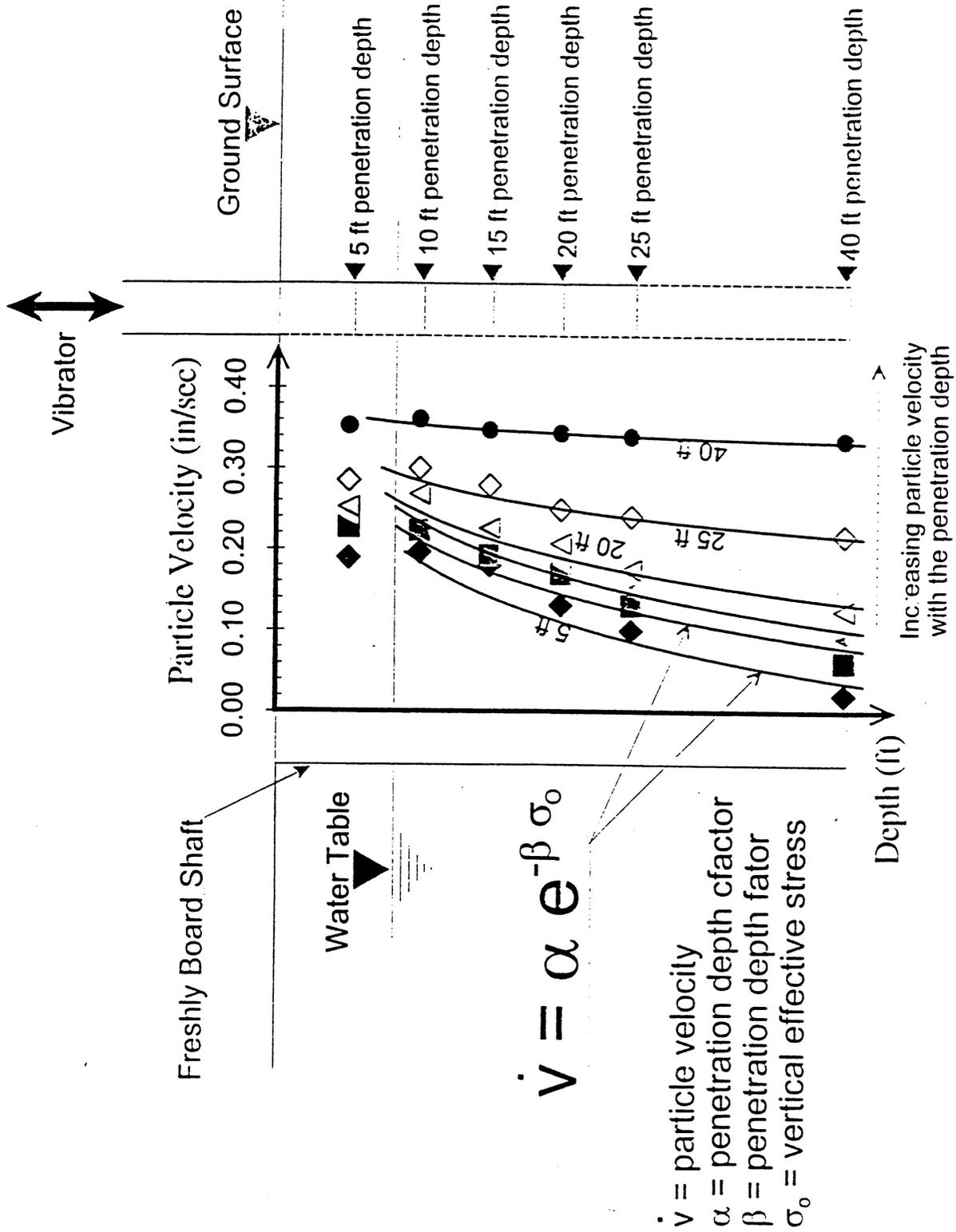


Figure 5.52 Change in the particle velocity with penetration depth of a drilled shaft casing.

VIBRATION LEVELS AT INDIVIDUAL GEOPHONES DURING DRIVING RANGES

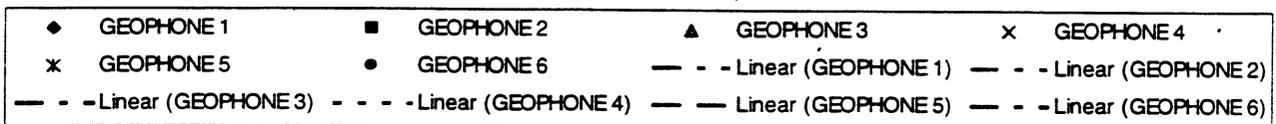
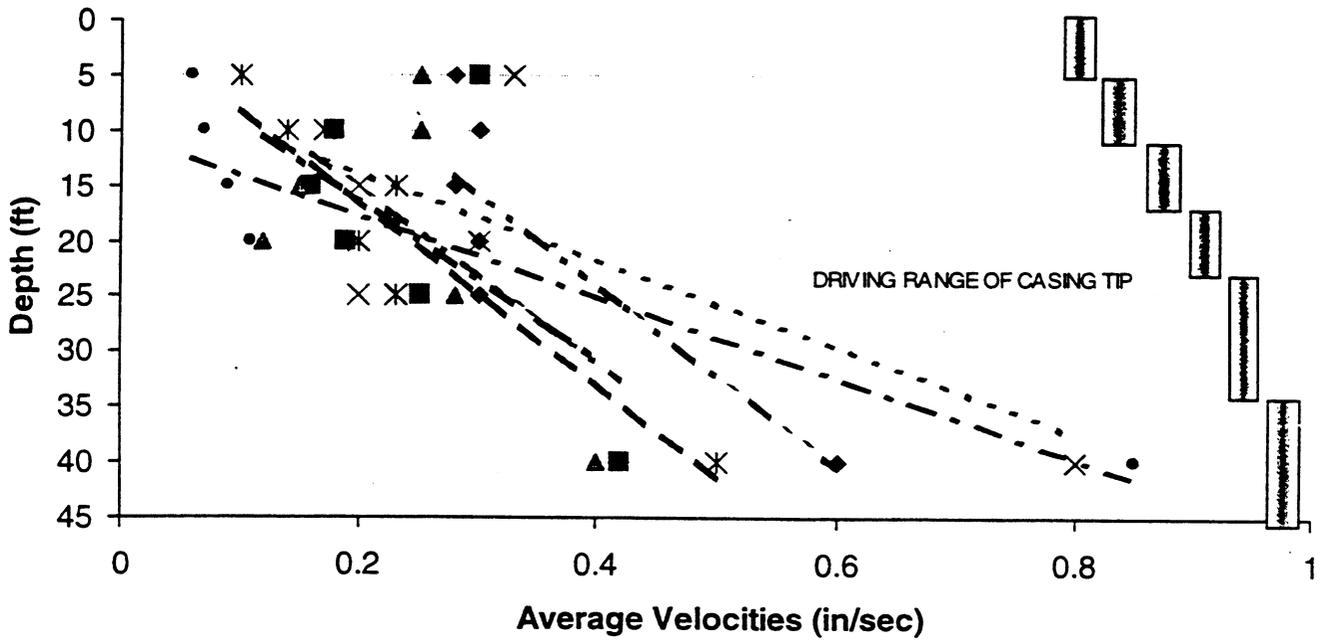


Fig 5.53. Vibration levels at individual geophones during driving ranges

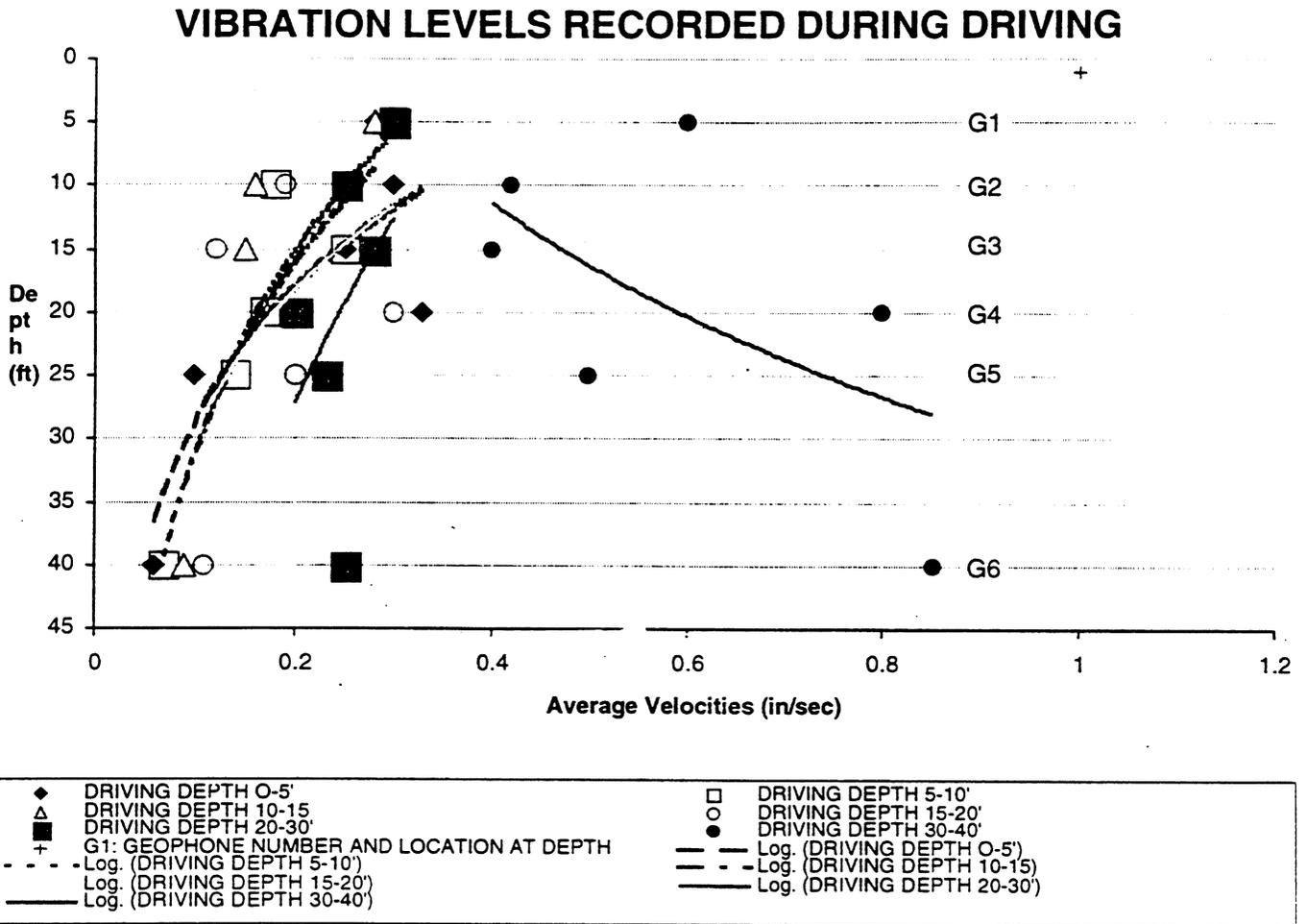


Fig 5.54. Average recorded vibration for different depths of driving

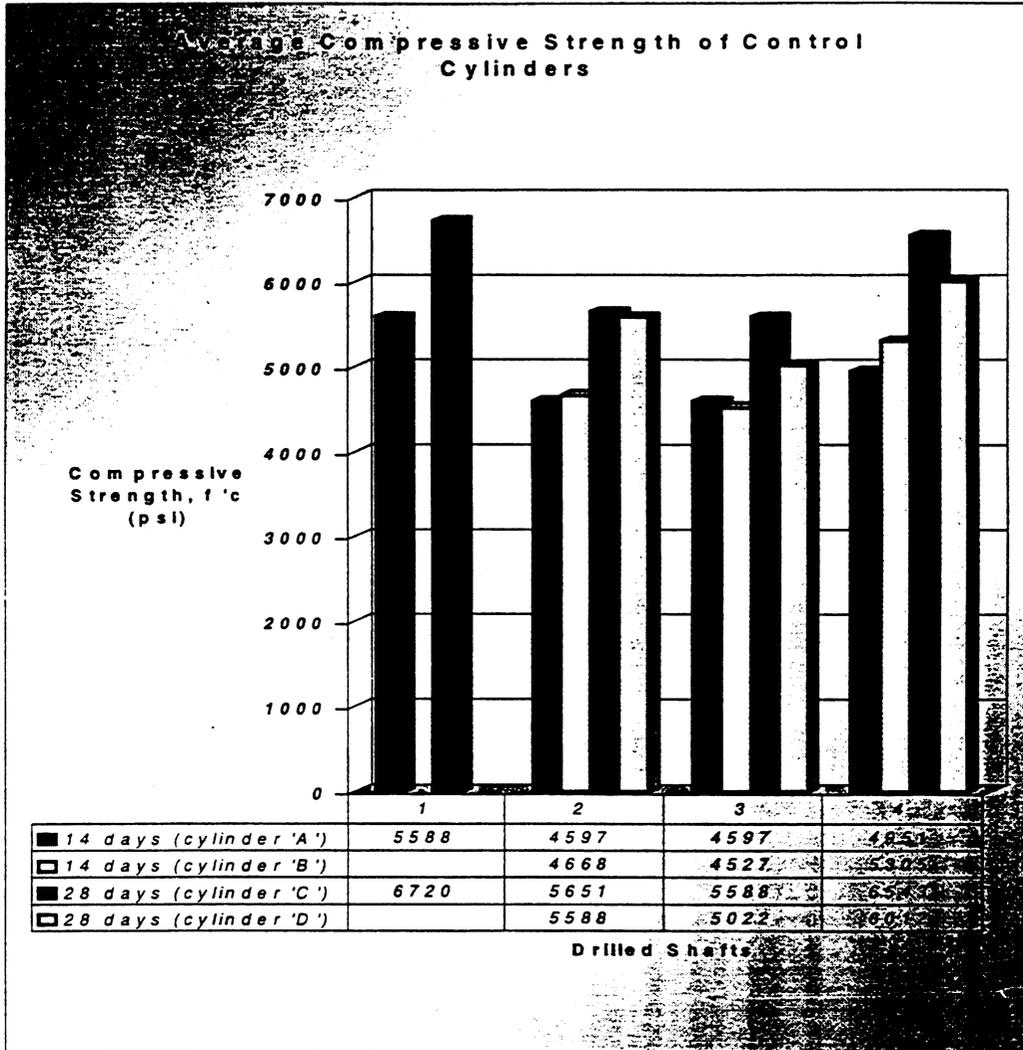
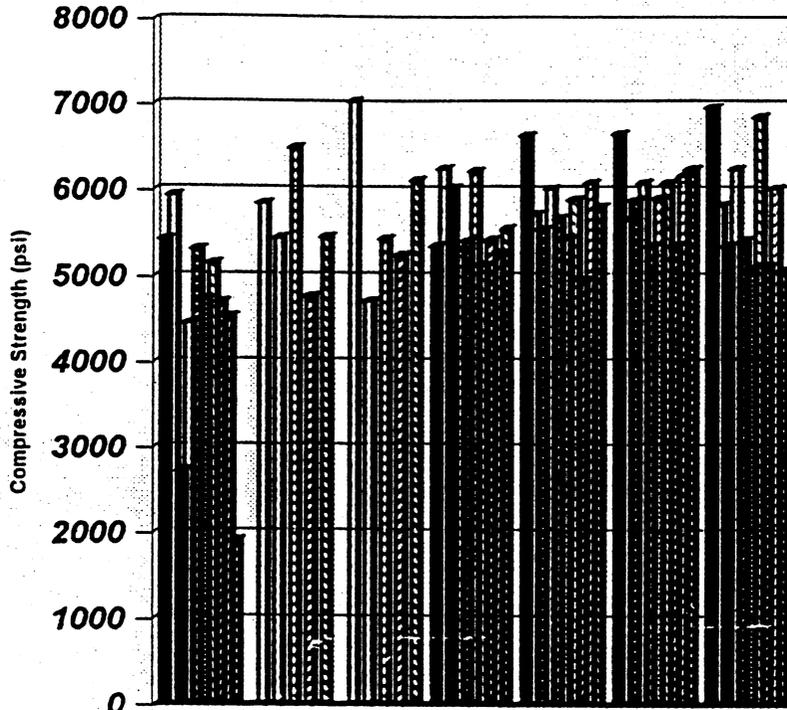


Figure 5.55. Compressive strength of control samples.

COMPRESSION STRENGTH OF CORED SAMPLES



	.25'	4.5'	5.0'	10.0'	10.5'	13.5'	14.5'
DS 1-2	5430	0	0	5350	6630	6650	6960
DS 1-3	5940	5850	7040	6250	5720	5650	5820
DS 2-2	2720	0	0	6030	5540	5880	5350
DS 2-3	4450	5450	4710	5300	6010	6080	6240
DS 3-2	5320	0	0	5410	5670	5350	5420
DS 3-3	4730	6500	5440	6220	5440	5900	5088
DS 4-2	5160	0	0	5130	5890	6080	6850
DS 4-3	4700	4770	5250	5420	4960	5360	5120
DS 5-2	4536	0	0	5269	6078	6159	6023
DS 5-3	1910	5460	6114	5545	5806	6240	5069

Figure 5.56. Compression strength of cored samples.

words, the total amount of vibration exposure is greatest at ground elevation, with the exception of the final driving stage, where the trend indicates that the particles velocities at a point near the final destination of the driven casing is greatest. One more observation that can be made from **Figure 5.54** is that as the steel casing penetrates to greater depth, the percent difference of the readings between the geophones decreases. Notably, the loose sandy soils in which this test was conducted adversely affect the transmission of vibrations.

The peak particle velocities (PPV) recorded are within the ranges expected based upon review of similar data from other projects. However, the maximum PPV values are rather small, due to the nature of the soil. Therefore, it would be reasonable to suggest that the effects of these vibrations would not pose any damage to existing structures.

Laboratory tests on representative concrete samples were performed under controlled environmental and boundary conditions. Samples were prepared from the same mix design, and subjected to predetermined vibration amplitudes at certain frequency contents. The delay time of induced vibration were equal for all the sets. The main advantage of testing concrete samples under such controlled conditions was the reproducibility of the test procedure for a large number of samples. Although some researchers do not consider it as a weakness, the lack of reproducing the actual waveforms produced in the field restricts the material behavior obtained from laboratory testing as a function of the controlled loading conditions. It should be noted herein that the mode of dynamic loading in the laboratory was predominantly compression, while the actual waves propagated in the ground during drilled shaft construction were mainly shear waves. The effect of the mode of vibration was not one of the parameters that were covered in this study. The reason was the difficulty to separate the modes of vibrations in the field. The use of the triaxial geophones indicated that vibrations in the vertical, longitudinal, and transverse directions were recorded at different locations in the ground and in the freshly placed concrete. However, the vertical (shear) mode was, for most of the measurements, higher than the other two modes. At some locations when the steel casing was not fully penetrating the ground, the longitudinal and the transverse modes exhibited larger particle velocities than the vertical mode. An explanation for such behavior was that the interaction between the vibrator and the casing was not perfectly eccentric. Any small eccentricity at the grip between the vibrator and the casing can amplify the transverse or the longitudinal modes causing increased particle motions in these directions. Although too small, soil displacement during casing penetration would produce compression mode of loading in the longitudinal and the transverse directions. To control the predominancy of the vertical vibrations (shear) during casing vibration and at any

penetration depth is unrealistic. Therefore, one should expect that during casing higher particle velocities are recorded in directions other than the vertical direction.

CONCLUSIONS

I) Laboratory Test Results :

Construction vibrations that produce particle velocities less than 8 in./sec would not have any effect on the compressive strength or compression modulus of the concrete aged between the time of placement to the first 24 hours.

- 1) All concrete samples, subjected to vibration amplitudes of 8 in./sec, after three to four hours from placement, showed noticeable surface voids on the concrete surface. Traces of microcracks were also noticed on some samples, subjected to large vibration amplitudes, i.e. 8.3 in./sec. after three to four hours from placement. The presence of the microcracks was not as common as the surface voids in samples of this category. Therefore, a final conclusion on the development of microcracking at this age of concrete is not definitive. Further investigations, using a Scanning Electron Microscope (SEM), may be needed to clarify such occurrences. No voids were induced inside the samples.

II) Field Test Results:

1. The testing conducted to establish the integrity of the shafts included Pile Integrity Testing (PIT), Crosshole Sonic Logging (CSL), Gamma-Gamma, Neutron-Neutron, and coring to obtain an actual indication of the compressive strength. The geophysical methods used, when combined, provided a subsurface profile of the shaft.
2. Crosshole Sonic Logging (CSL) tests showed continuity across the cross-sectional areas of the shafts with the exception of the 2-hour, 36-inch shaft. However, the PVC access tubes were stressed during the passing of a hurricane, which was the most likely reason for the deviation from the other shafts.

3. Pile Integrity Testing (PIT) showed a definite reflection of the transmitted wave from the shaft tip, indicating no structural damage in the shaft length.
4. Gamma-Gamma logging of the drilled shafts showed lower density profiles for the upper parts relative to the bottom ones.
5. Neutron-Neutron logging showed that the porosity within the shafts is relatively unchanged by the vibration.
6. The geophysical data obtained can be compared in relative terms to conclude the following: the upper 1/3 or 5 feet of the shafts showed some relatively reduced concrete properties. The relative weakness of the upper shaft sections was further evidenced by the compressive strengths of the cored concrete samples, Fig 5.56. This can be attributed only to the segregation of the aggregate during vertical pouring of concrete, since the field velocities of 2.5 in./sec are much below the threshold value of 8.3 in./sec that can affect the strength.
7. The geophysical data only served to show relative performance within each individual shaft, and no comparisons were established with the other shafts, as no trend could be established in the correlation of the values between the compressive strength and the geophysical data. Nevertheless, the data shows that shaft integrity was not adversely affected, and though within each shaft relative weakness existed in the upper sections, it was not an indication of poor concrete.
8. These findings however have limitations. Primarily, the data is true only for the subsurface conditions tested, i.e. loose soils. The loose soil condition aided to a great extent in the attenuation of the vibrations induced, and consequently diminished the transfer of the higher peak velocity across the interface between the stratum and shaft.

- 9 Peak particle velocities activated during travel of the stress waves, (i.e. 2.5 in./sec) were well below the threshold values of 8in./sec known to cause damage in concrete structures.

III) Comprehensive Evaluation:

The concept of particle velocity is used to control construction vibrations for almost all types of projects in civil engineering. Although the concept is helpful as a quality control factor during construction, particle velocities should not be used alone to assess the impact of the construction vibration on the surrounding structures. For a meaningful assessment, particle velocities should be translated into strain amplitudes. A value of 8 in/sec particle velocity would produce different strain amplitudes in different materials and in different structures. Even for the same construction material, but with different moduli values, the same particle velocity would result in different strain amplitudes and hence different effects. For example, “green concrete” would have lower compression and shear moduli as compared to aged concrete. Therefore, the effect of PPV on the same material at different time levels may vary since the induced strain amplitudes are different.

It is important to conduct a preliminary assessment of the effect of vibration before construction of deep foundations. Such an assessment requires the following:

- a) **Determining the physical and mechanical properties of the in-situ soil layers**
- b) **Predicting generated waveforms**
- c) **Assessing the PPV based on the generated waveforms**
- d) **Determining the shear or compression wave velocity of the in-situ layers.**
- e) **Field testing for Part (c).**
- f) **Determining the damping ratios of the soil layers**
- g) **Determining the strain amplitudes at critical locations.**

h) Predicting the damage due to the obtained strain amplitudes.

The concept of the PPV is very helpful to estimate the impact of construction vibration. However, the PPV concept is a transposition from mining engineering, where the applications are limited to certain practices. In Civil Engineering applications, the sources of vibrations are numerous. The types of vibrations range from natural to man-made vibrations. Also, the waveforms range from deterministic to non-deterministic.

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PART II

**EFFECTS OF SOUND INDUCED
DURING THE INSTALLATION OF PILES**

PILE DRIVING NOISE MITIGATION THROUGH USE OF AN ACOUSTIC CURTAIN

1.1 Background

During the installation of driven piles, the noise level becomes a critical parameter due to the low threshold of human perception. In particular, construction noises from this type of foundation installation becomes a concern when the site is near office buildings or residential areas. Common pile hammers often exceed the noise limits set by state transportation projects, thus requiring an alternate method of installation to be sought. This project focused on the use of an acoustic curtain constructed of a combination of absorptive and reflective materials to provide a complete enclosure of the source to mitigate the propagation of the inherent noise.

1.2 Laboratory Testing

1.2.1 Test Setup

A reduced scale experiment was conducted to determine the Sound Pressure Level (SPL) reduction through the use of an acoustic curtain in a laboratory chamber, which represented open field conditions with no reverberation. An omni-directional noise to represent the hammering of a shaft was provided by dropping a hammer onto a steel tube of rectangular cross-section, which provided a sharp single point source. The noise was measured as a SPL in dB (ref. 2×10^{-5} N/m²) and was taken as an average of 20 hammer blows. The test setup is shown below and the results indicate a SPL reduction of about 16.5 dB.

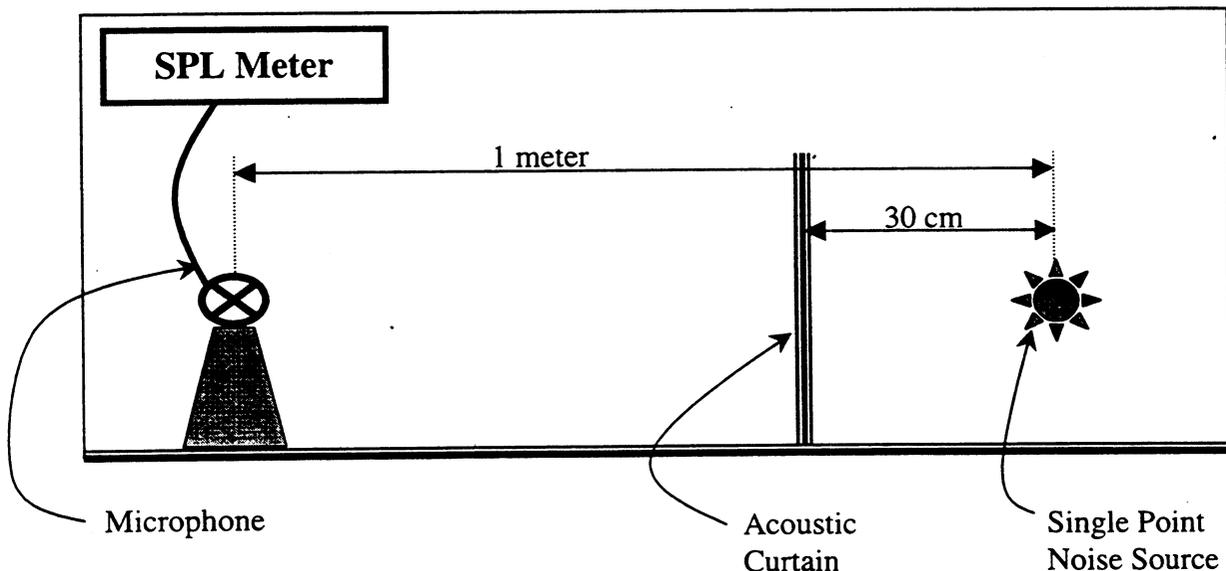


Figure 1. Enclosed chamber with no reverberation

1.2.2 Test Results

- Without curtain - Average SPL = **118.1 dB**
- With curtain - Average SPL = **101.6 dB**
- Noise reduction obtained with curtain = **16.5 dB**

These results indicate that the acoustic curtain provides significant reduction in sound pressure levels when placed around a single-point, Omni-directional noise source. In a practical sense, a 16.5 dB reduction is almost equivalent to heavy construction noises being reduced down to the sound of average street traffic (a 20 dB reduction).

1.3 Field Testing

A full-scale field-test was conducted to determine the actual sound pressure level (SPL) reduction through the use of an acoustic curtain in a true on-site application. An industrial diesel hammer was used to drive 12 inch square concrete piles 20 ft in length to accurately replicate true construction field practice. The noise was measured as a sound pressure level at various heights and distances from the source and an octave band analysis was performed at a specified location to identify the pertinent frequencies. When compared to the lab results, higher SPL reductions (up to 20 dB) were achieved in the field. A detailed description of the companies involved and the experimental test setup is included on the following pages.

1.3.1 Field Test Setup

Commencement of the field test took place on the north of the FAU Boca Raton campus in the field just east of Parking Lot #5 (see Figure 2). The site was chosen to be the same as for the first phase of the project (drilled shafts) because the required permits were still valid through FAU Facilities Planning and the soil type was already known. Standard Penetration Tests (SPT) performed prior to the drilled shaft phase indicated sandy soil conditions down to depths greater than would be reached by the 20' pile. Locally known as "sugar sand," the soil provided ideal pile driving conditions and easy pile extraction. It was also noted that the water table was no more than 5' below the surface and no bedrock or any other underground obstruction was encountered.

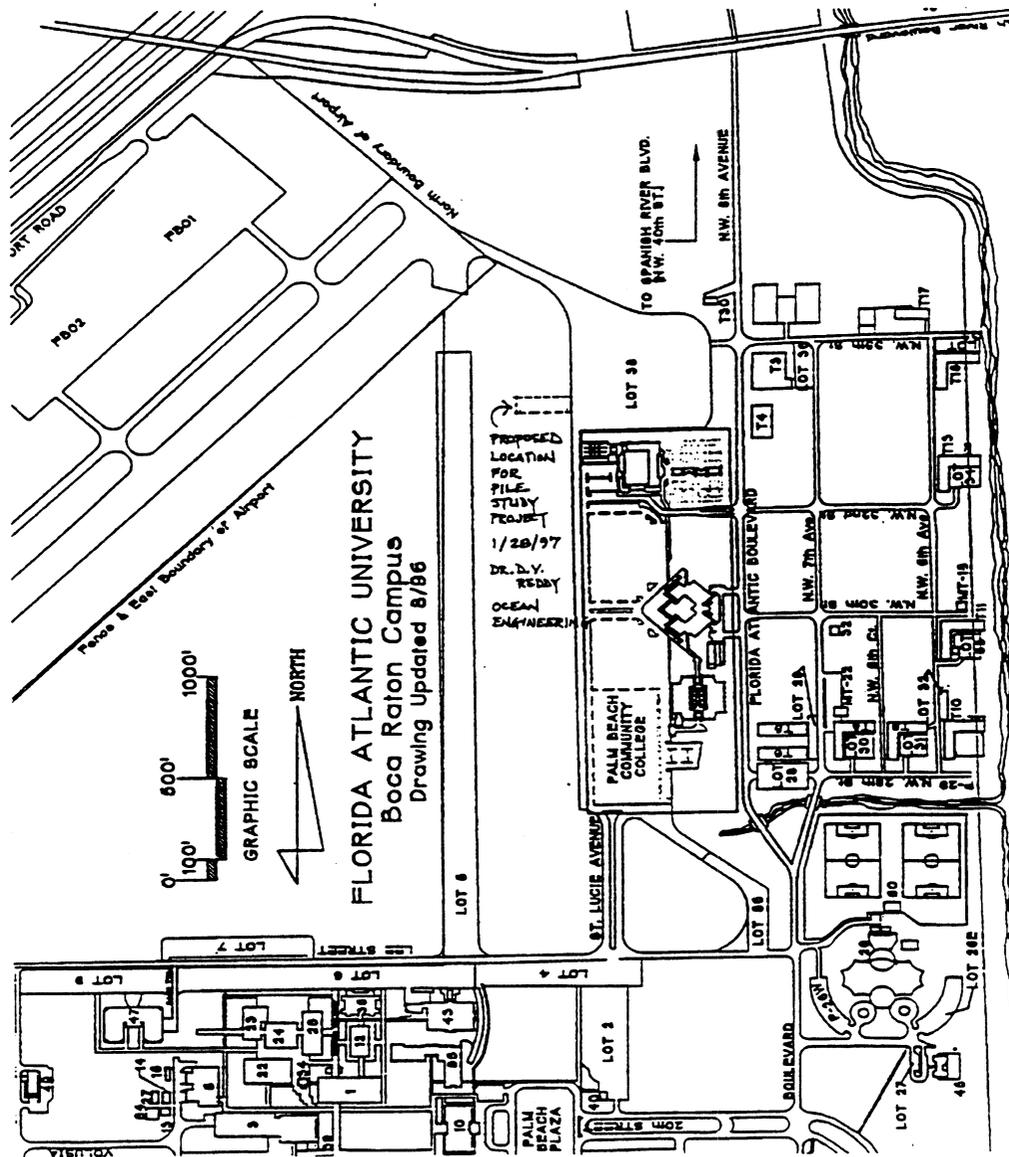


Figure 2. Site selection

The noise measurements were taken at seven distinct recording stations as shown in Figure 3. Three stations provided sound pressure levels at 10' above the ground for radial distances of 50, 100, and 300 ft. Three separate stations were also setup to take measurements at the same radial distances, but only 5' off the ground. As per the recommendation of Dr. Kenneth Campbell, FDOT Noise Specialist, the 5' and 10' measurement poles were set in different radial directions 45° apart to ensure that the meters would measure the true spread of the noise. The seventh station was located at a radial distance of 100 ft and provided a real-time octave band analysis to determine all the inherent sound frequencies at 5' above grade.

In total, four piles were driven and extracted, and noise levels were measured for two piles driven in the standard manner, and for two driven with the acoustic curtain in place. Figure 3 shows a plan view of the field test setup depicting the location of the noise source, the acoustic curtain, and the seven measuring stations.

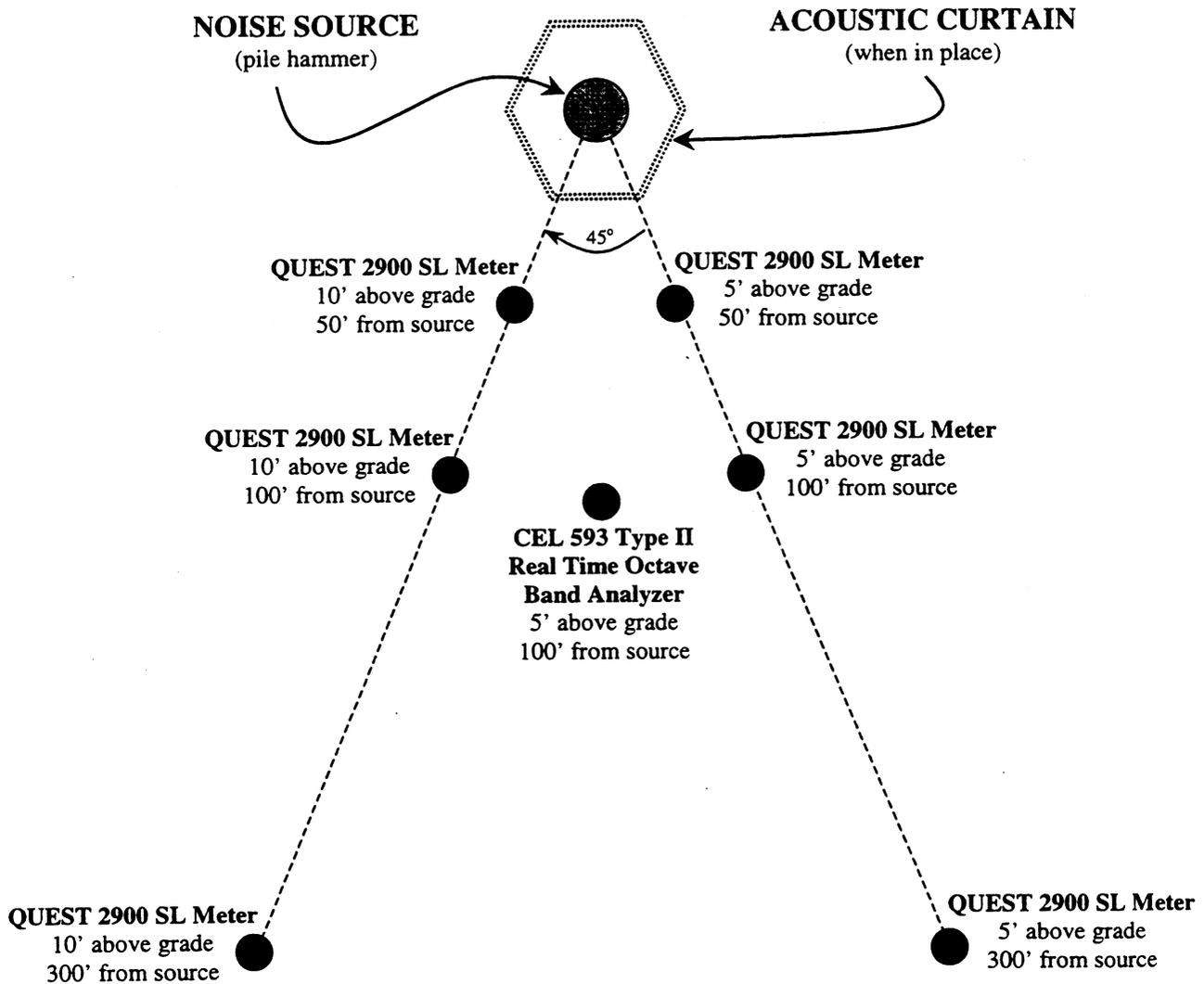


Figure 3. Field test setup

1.3.2 Pile Driving

Curry Industries, a specialist in pile driving and precast concrete, provided all material, labor and equipment necessary to install and extract the concrete piles used in the full-scale field-test. The piles were 12"x12"x20' commercial grade prestressed concrete with a 5000 psi 28-day strength, and were driven with a standard Delmag D-12 diesel pile hammer. In total, the piles were driven and extracted four times, twice in the

standard manner and twice with the acoustic curtain enclosing the hammer. Upon completion of the project, Curry Industries extracted the piles and removed all the equipment, leaving the site in the natural state it was prior to commencement. Figure 4 shows the piles, installation, and extraction.

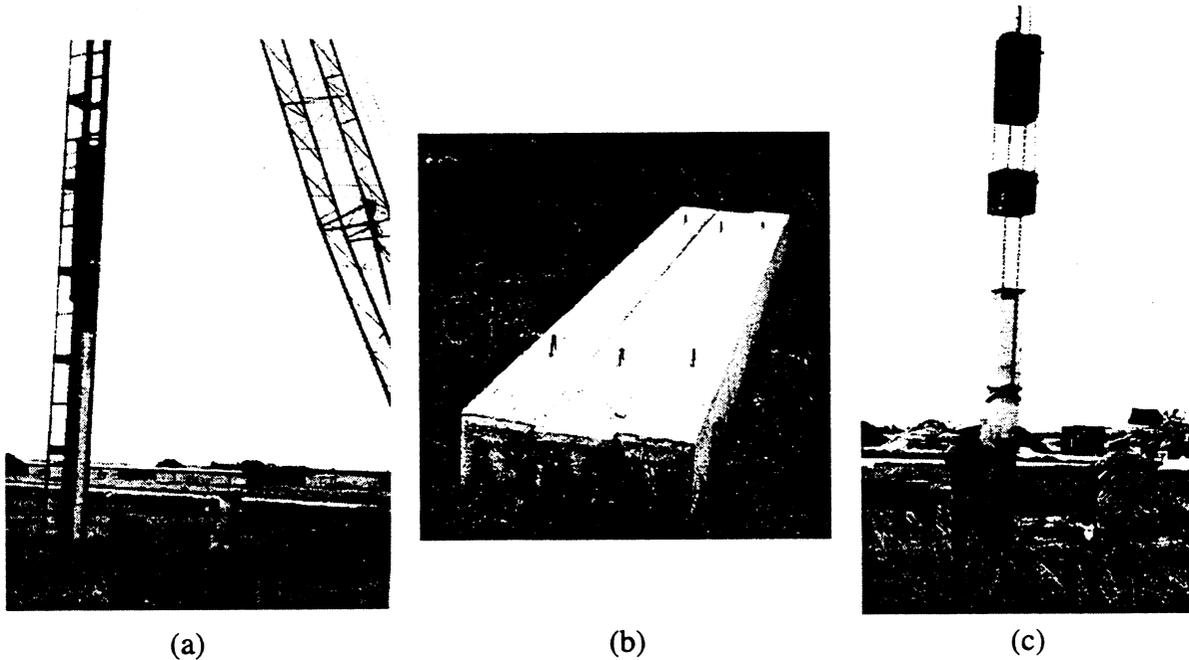


Figure 4. (a) Pile driving, (b) 20 ft concrete piles, and (c) pile extraction

1.3.3 Noise Measurements

A pile driver noise impact and insertion loss analysis was conducted by LAW Engineering & Environmental Services to determine the dB reduction achieved through the use of an acoustic curtain. Six Quest 2900 Type II Sound Level Meters were used to measure the sound pressure levels (in decibels) in conjunction with a CEL 593 Type II Real Time Octave Band Analyzer to identify the frequency ranges omitted by the source and mitigated by the curtain. In total, noise measurements were taken at 7 stations: 3 at 10 ft above the ground for radial distances of 50 ft, 100 ft, and 300 ft from the source, 3 at 5 ft above the ground for the same radial distances, and the last (octave band analysis) at 5 ft above ground 100 ft from the source. Pictures of the sound pressure level measuring devices and setup are shown below in Figure 5. (The Details of report is presented in Appendix-1).

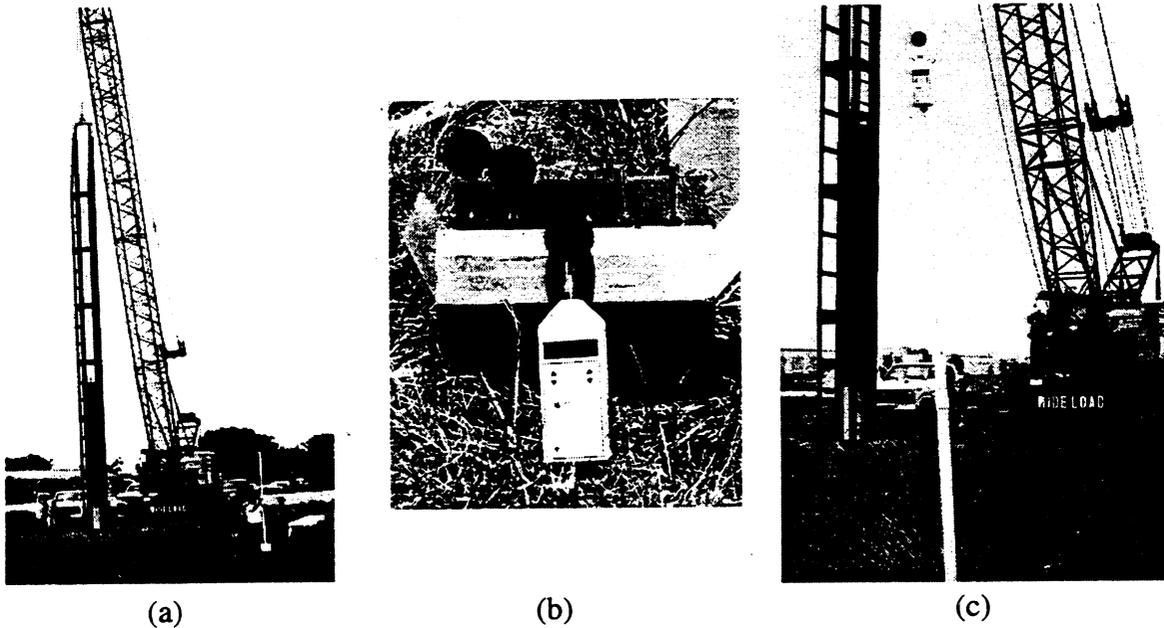


Figure 5. (a) Setting up for noise measurements at 50 ft, (b) Quest 2900 SL meters, (c) 10 ft measurement pole with meter

1.3.4 Acoustic Curtain

The acoustic curtain used in this study was provided by Environmental Noise Control, a Division of Dynamic Services, Inc. and was made of 400 square feet of Acoustical STC-18 High NRC Blanket, which is a combination of absorptive and reflective materials designed for industrial applications. It is made from a (400 ft², 1 lb/ft²) . Reinforced poly-vinyl chloride outs shell for added mass with a specially developed inner lining of 2 in. thick fiber glass acoustical insulation(1.5 lb/ft²). A clear vinyl strip was included as a window for the hammer operator to view the pile's progress. The blanket was connected to the specially designed frame shown in Figure 6, which was then attached to the crane with steel leads and slowly erected into place.

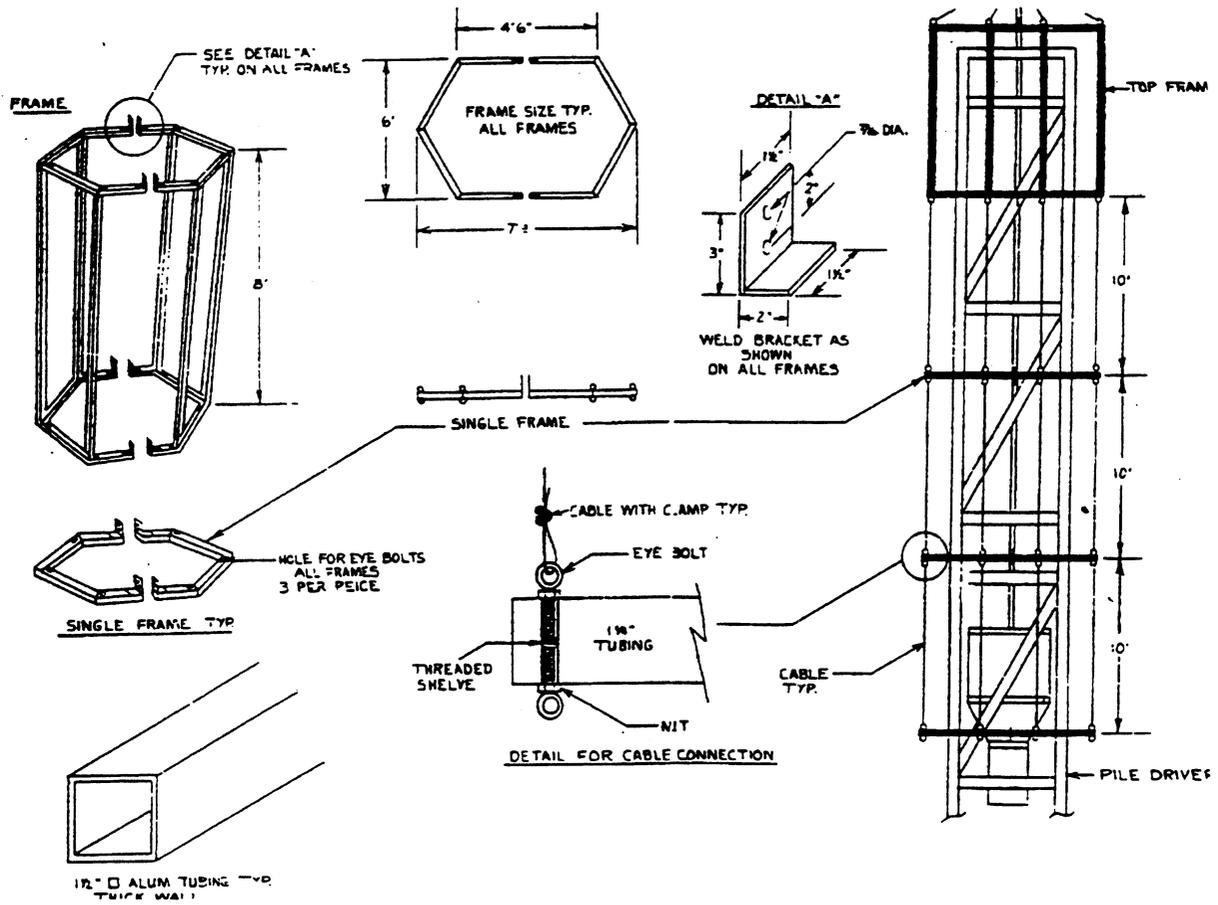


Figure 6. Curtain and frame assembly (courtesy of ENC)

The curtain was allowed to swing open one side so that new piles could be easily slipped in under the pile hammer, and the curtain was pulled closed when the pile was in place to ensure total closure and maximum noise absorption. Figure 7 shows a picture sequence outlining the procedure followed to secure the curtain around the pile driving apparatus.



Figure 7(a). Assembling the curtain from rectangular panels connected together by Velcro and rope

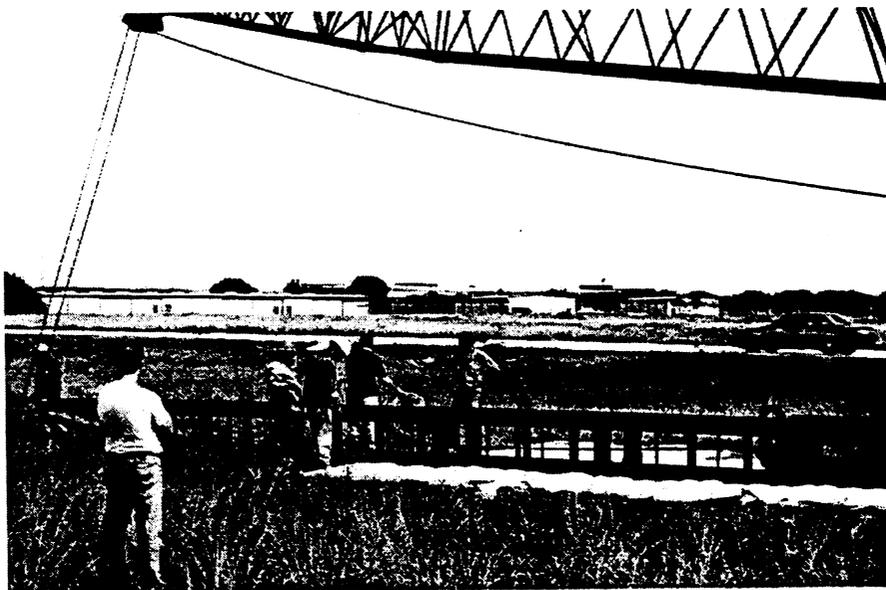


Figure 7(b). Attaching the completed curtain to the crane with steel leads to the frame

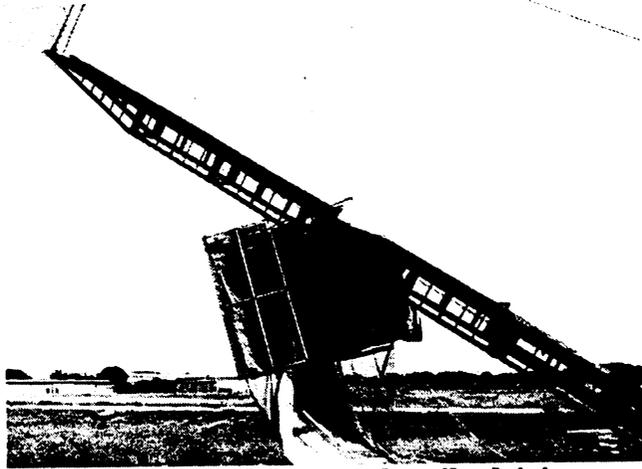


Figure 7(c). Erecting the pile-driving frame with the curtain attached

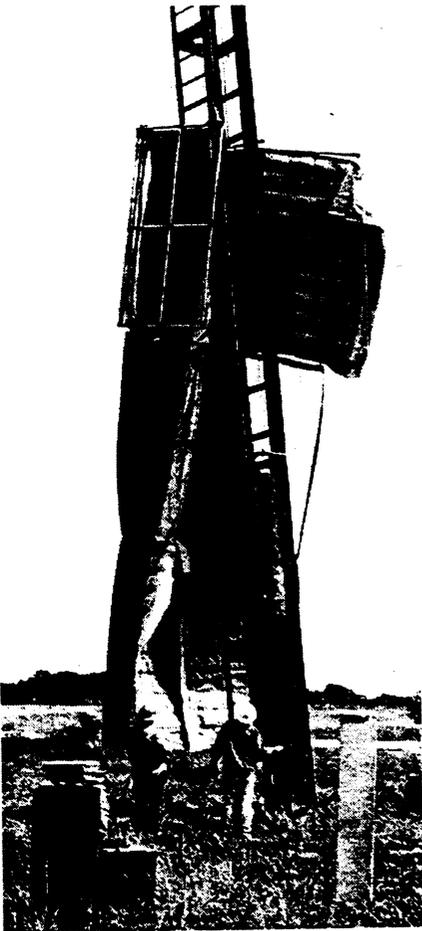


Figure 7(d). Adjusting the frame to ensure the pile will be driven vertically and at the proper location



Figure 7(e). Curtain swung open for pile installation and setup

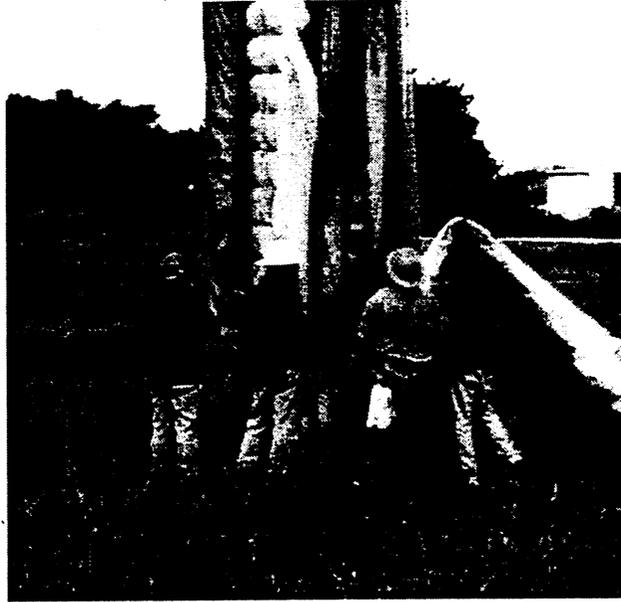


Figure 7(f). Attaching the skirt to the bottom of the curtain to minimize sound propagation along the ground

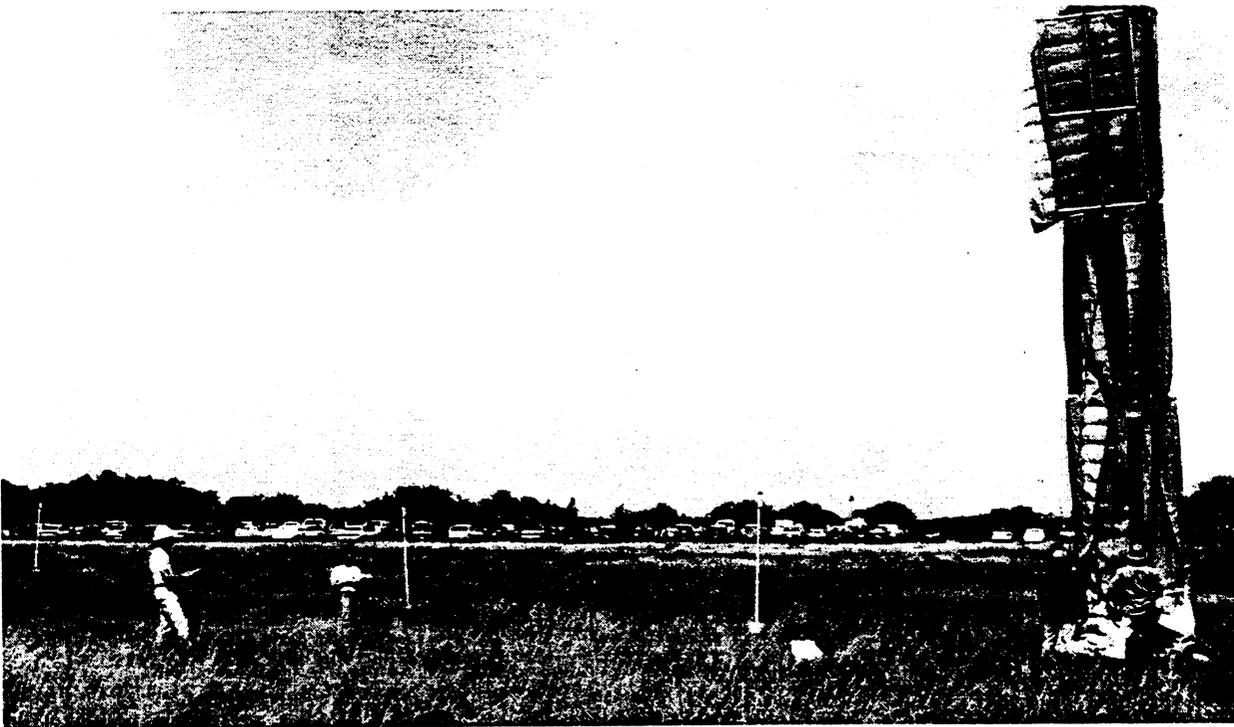


Figure 7(g). Final field-test setup showing noise measurement poles and the acoustic curtain completely enclosing the pile driving apparatus

1.3.5 Discussion and Conclusions

Based on a very limited study, it appears that the acoustic curtain provides approximately a 20 decibel reduction when total enclosure is achieved. This project has opened the doors to new area of research in the field of construction noise reduction. Through the use of practically designed curtains, FDOT can achieve a greater volume of work without adding to noise pollution, which becomes bothersome to local residents as a result of traditional construction practices. Further research should include: driving in varied soil conditions, testing different curtain materials and designs, testing with larger pile hammers and louder noise sources such as steel piles, and providing a practical method of attaching the curtain to the crane, with a design that allows for rapid installment of new piles so that the job schedule is not slowed down.

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Appendix A

Laboratory Compressive Strength Data

Properties of set # 1

Curing Time: (3 Days)

Frequency (20Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,3} , t ₀	11.625	7.70E-05	150974	28.26	43891	4787497	1553.121	4733979	1858.15
S _{2,3} , t ₀	11.688	7.90E-05	147943	28.26	52609	4597197	1861.605		
S _{3,3} , t ₀	11.813	7.60E-05	151442	28.26	61034	4817242	2159.728		
S _{1,3} , t ₁	11.75	7.60E-05	154605	28.26	61293	5020565	2168.9	4899967	2051.28
S _{2,3} , t ₁	11.813	7.70E-05	153409	28.26	59231	4943178	2095.932		
S _{3,3} , t ₁	11.563	7.70E-05	150162	28.26	53384	4736157	1889.019		
S _{1,3} , t ₂	11.688	7.70E-05	151786	28.26	61793	4839114	2186.599	4832707	2098.39
S _{2,3} , t ₂	11.688	7.80E-05	149640	28.26	57636	4715830	2039.507		
S _{3,3} , t ₂	11.813	7.70E-05	153409	28.26	58472	4943178	2069.062		
S _{1,3} , t ₃	11.438	7.80E-05	146635	28.26	53916	4516241	1907.839	4537521	2413.91
S _{2,3} , t ₃	11.625	7.90E-05	147152	28.26	69388	4548161	2455.338		
S _{3,3} , t ₃	11.625	7.90E-05	147152	28.26	81348	4548161	2878.539		
S _{1,3} , t ₄	11.5	7.90E-05	145570	28.26	69672	4450877	2465.393	4501376	2645.66
S _{2,3} , t ₄	11.5	7.90E-05	145570	28.26	72235	4450877	2556.093		
S _{3,3} , t ₄	11.25	7.60E-05	148026	28.26	62392	4602374	2915.499		
S _{1,3} , t ₂₄	11.5	8.00E-05	143750	28.26	40816	4340300	1444.314	4356789	1934.26
S _{2,3} , t ₂₄	11.25	7.70E-05	146104	28.26	65875	4483608	2331.04		
S _{3,3} , t ₂₄	11.375	8.00E-05	142188	28.26	57295	4246459	2027.414		

Properties of set # 1

Curing Time: (7 Days)

Frequency (20Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,7, t₀}	11.75	7.60E-05	154605.3	28.26	81442.56	5020565	2881.9	5074421	2919.97
S _{2,7, t₀}	12	7.70E-05	155844.2	28.26	85619.52	5101350	3029.71		
S _{3,7, t₀}	12	7.70E-05	155844.2	28.26	80493.12	5101350	2848.31		
S _{1,7, t₁}	11.75	7.60E-05	154605.3	28.26	77170.56	5020565	2730.73	5030864	2634.45
S _{2,7, t₁}	11.688	7.50E-05	155833.3	28.26	85144.32	5100641	3012.89		
S _{3,7, t₁}	12	7.80E-05	153846.2	28.26	61033.92	4971384	2159.73		
S _{1,7, t₂}	11.5	7.70E-05	149350.6	28.26	80303.04	4685094	2841.58	4749407	2820.31
S _{2,7, t₂}	11.813	7.80E-05	151442.3	28.26	83530.56	4817242	2955.79		
S _{3,7, t₂}	11.875	7.90E-05	150316.5	28.26	75272.64	4745884	2663.58		
S _{1,7, t₃}	11.75	7.70E-05	152597.4	28.26	90080.64	4891008	3187.57	4883176	3287.18
S _{2,7, t₃}	11.75	7.80E-05	150641	28.26	92548.8	4766401	3274.9		
S _{3,7, t₃}	11.563	7.50E-05	154166.7	28.26	96057.6	4992120	3399.07		
S _{1,7, t₄}	11.5	7.60E-05	151315.8	28.26	101654.4	4809197	3597.11	4879500	3372.21
S _{2,7, t₄}	11.75	7.70E-05	152597.4	28.26	87991.68	4891008	3113.65		
S _{3,7, t₄}	11.5	7.50E-05	153333.3	28.26	86249.6	4938297	3405.86		
S _{1,7, t₂₄}	11.75	7.90E-05	148734.2	28.26	90935.04	4646496	3217.8	4782769	3068.89
S _{2,7, t₂₄}	11.625	7.70E-05	150974	28.26	82392	4787497	2915.5		
S _{3,7, t₂₄}	11.625	7.60E-05	152960.5	28.26	86853.12	4914313	3073.36		

Properties of set # 1

Curing Time: (28 Days)

Frequency (20Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,28} , t ₀	11.875	7.20E-05	164930.6	28.26	97372.8	5713553	3445.61	5434927	3452.67
S _{2,28} , t ₀	11.75	7.40E-05	158783.8	28.26	99379.2	5295614	3516.6		
S _{3,28} , t ₀	11.75	7.40E-05	158783.8	28.26	95965.44	5295614	3395.8		
S _{1,28} , t ₁	11.875	7.50E-05	158333.3	28.26	100704	5265611	3563.48	5247378	3819.96
S _{2,28} , t ₁	11.938	7.50E-05	159166.7	28.26	99571.2	5321184	3523.4		
S _{3,28} , t ₁	11.75	7.50E-05	156666.7	28.26	123580.8	5155339	4372.99		
S _{1,28} , t ₂	11.813	7.50E-05	157500	28.26	124060.8	5210329	4389.98	5342701	4512.05
S _{2,28} , t ₂	11.875	7.40E-05	160473	28.26	136780.8	5408886	4840.08		
S _{3,28} , t ₂	11.875	7.40E-05	160473	28.26	121689.6	5408886	4306.07		
S _{1,28} , t ₃	11.813	7.50E-05	157500	28.26	96249.6	5210329	3405.86	5229339	3237.93
S _{2,28} , t ₃	11.938	7.60E-05	157072.4	28.26	83720.64	5182074	2962.51		
S _{3,28} , t ₃	11.75	7.40E-05	158783.8	28.26	94541.76	5295614	3345.43		
S _{1,28} , t ₄	11.938	7.70E-05	155032.5	28.26	95016	5048349	3362.21	5102342	3518.94
S _{2,28} , t ₄	11.938	7.70E-05	155032.5	28.26	118080	5048349	4178.34		
S _{3,28} , t ₄	11.813	7.50E-05	157500	28.26	85239.36	5210329	3016.25		
S _{1,28} , t ₂₄	11.75	7.60E-05	154605.3	28.26	117321.6	5020565	4151.51	5183929	4462.68
S _{2,28} , t ₂₄	11.875	7.50E-05	158333.3	28.26	125289.6	5265611	4433.46		
S _{3,28} , t ₂₄	11.875	7.50E-05	158333.3	28.26	135734.4	5265611	4803.06		

Properties of set # 1

Curing Time: (14 Days)

Frequency (20Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,14,t₀}	11.688	7.50E-05	155833.3	28.26	90269.76	5100641	3194.26	5137204	3074.47
S _{2,14,t₀}	11.813	7.50E-05	157500	28.26	87043.2	5210329	3080.08		
S _{3,14,t₀}	11.688	7.50E-05	155833.3	28.26	83340.48	5100641	2949.08		
S _{1,14,t₁}	11.938	7.70E-05	155032.5	28.26	95965.44	5048349	3395.8	5139294	3768.59
S _{2,14,t₁}	11.938	7.70E-05	155032.5	28.26	100233.6	5048349	3546.84		
S _{3,14,t₁}	11.938	7.50E-05	159166.7	28.26	123302.4	5321184	4363.14		
S _{1,14,t₂}	11.938	7.50E-05	159166.7	28.26	112195.2	5321184	3970.11	5331283	4261.12
S _{2,14,t₂}	12	7.50E-05	160000	28.26	118550.4	5377049	4194.99		
S _{3,14,t₂}	11.75	7.40E-05	158793.8	28.26	130512	5295614	4618.26		
S _{1,14,t₃}	11.875	7.60E-05	156250	28.26	73819.2	5127954	2512.14	5048048	2945.37
S _{2,14,t₃}	11.875	7.70E-05	154220.8	28.26	93213.12	4995625	3298.41		
S _{3,14,t₃}	11.75	7.60E-05	154605.3	28.26	82676.16	5020565	2925.55		
S _{1,14,t₄}	11.938	7.70E-05	155032.5	28.26	93592.32	5048349	3311.83	5074884	3048.73
S _{2,14,t₄}	11.875	7.60E-05	156250	28.26	84860.16	5127954	3002.84		
S _{3,14,t₄}	11.938	7.70E-05	155032.5	28.26	80018.88	5048349	2831.52		
S _{1,14,t₂₄}	11.875	7.40E-05	160473	28.26	88371.84	5408886	3127.1	5159059	2868.46
S _{2,14,t₂₄}	11.5	7.40E-05	155405.4	28.26	73658.88	5072666	2606.47		
S _{3,14,t₂₄}	11.875	7.70E-05	154220.8	28.26	81157.44	4995625	2871.81		

Properties of set # 2

Curing Time: (3 Days)

Frequency (40Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,3} , t ₀	11.438	7.70E-05	148539	28.26	74703	4634307	2643.431	4776090	2922.21
S _{2,3} , t ₀	11.5	7.60E-05	151316	28.26	84480	4809197	2989.384		
S _{3,3} , t ₀	11.438	7.50E-05	152500	28.26	88562	4884766	3133.826		
S _{1,3} , t ₁	11.813	7.90E-05	149525	28.26	88562	4696059	3133.826	4630239	2698.28
S _{2,3} , t ₁	11.75	7.90E-05	148734	28.26	71191	4646496	2519.134		
S _{3,3} , t ₁	11.625	7.90E-05	147152	28.26	69008	4548161	2441.885		
S _{1,3} , t ₂	11.625	7.80E-05	149038	28.26	91314	4665528	3231.219	4645337	3187.56
S _{2,3} , t ₂	11.625	7.70E-05	150974	28.26	90650	4787497	3207.711		
S _{3,3} , t ₂	11.688	8.00E-05	146094	28.26	88277	4482985	3123.737		
S _{1,3} , t ₃	11.688	8.00E-05	146094	28.26	79544	4482985	2814.709	4483673	2793.44
S _{2,3} , t ₃	11.75	8.10E-05	145062	28.26	71760	4419873	2539.278		
S _{3,3} , t ₃	11.625	7.90E-05	147152	28.26	85524	4548161	3026.344		
S _{1,3} , t ₄	11.75	8.10E-05	145062	28.26	85429	4419873	3022.981	4516451	2954.68
S _{2,3} , t ₄	11.688	8.00E-05	146094	28.26	82771	4482985	2928.917		
S _{3,3} , t ₄	11.75	7.90E-05	148734	28.26	82297	4646496	2912.136		
S _{1,3} , t ₂₄	11.75	8.00E-05	146875	28.26	54922	4531060	1943.439	4557817	2516.45
S _{2,3} , t ₂₄	11.875	8.10E-05	146605	28.26	78120	4514413	2764.331		
S _{3,3} , t ₂₄	11.875	8.00E-05	148438	28.26	80303	4627978	2841.58		

Properties of set # 2
Curing Time: (7 Days) Frequency (40Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,7} , t ₀	11.5	7.60E-05	151315.8	28.26	81632.64	4809197	2888.63	4743483	2445.86
S _{2,7} , t ₀	11.5	7.70E-05	149350.6	28.26	56345.28	4685094	1993.82		
S _{3,7} , t ₀	11.563	7.70E-05	150182.3	28.26	69382.08	4736157	2455.13		
S _{1,7} , t ₁	11.688	7.60E-05	153782.9	28.26	104793.6	4967297	3708.2	4944780	3274.85
S _{2,7} , t ₁	11.688	7.50E-05	155833.3	28.26	69292.8	5100641	2451.97		
S _{3,7} , t ₁	11.75	7.80E-05	150641	28.26	103555.2	4766401	3664.37		
S _{1,7} , t ₂	11.75	7.70E-05	152597.4	28.26	86663.04	4891008	3066.63	4717667	3112.46
S _{2,7} , t ₂	11.563	7.80E-05	148237.2	28.26	98236.8	4615496	3476.18		
S _{3,7} , t ₂	11.75	7.90E-05	148734.2	28.26	78974.4	4646496	2794.56		
S _{1,7} , t ₃	11.688	7.70E-05	151785.7	28.26	90269.76	4839114	3194.26	4885195	3224.49
S _{2,7} , t ₃	11.688	7.80E-05	149839.7	28.26	90459.84	4715830	3200.99		
S _{3,7} , t ₃	11.688	7.50E-05	155833.3	28.26	92642.88	5100641	3278.23		
S _{1,7} , t ₄	11.75	7.60E-05	154605.3	28.26	79259.52	5020565	2804.65	5022304	2964.74
S _{2,7} , t ₄	11.75	7.70E-05	152597.4	28.26	81821.76	4891008	2895.32		
S _{3,7} , t ₄	11.75	7.50E-05	156666.7	28.26	90269.76	5155339	3194.26		
S _{1,7} , t ₂₄	11.813	7.90E-05	149525.3	28.26	65780.16	4696059	2327.68	4886601	2567.28
S _{2,7} , t ₂₄	11.813	7.70E-05	153409.1	28.26	64166.4	4943178	2270.57		
S _{3,7} , t ₂₄	11.75	7.60E-05	154605.3	28.26	87707.52	5020565	3103.59		

Properties of set # 2
Curing Time; (14 Days) Frequency (40Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	'Area' (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,14, t₀}	11.25	6.90E-05	163043.5	28.26	87707.52	5583557	3103.59	5528232	2611.85
S _{2,14, t₀}	11.5	7.00E-05	164285.7	28.26	76886.4	5668963	2720.68		
S _{3,14, t₀}	11.313	7.10E-05	159331	28.26	56838.72	5332177	2011.28		
S _{1,14, t₁}	11.375	7.00E-05	162500	28.26	85808.64	5546395	3036.4	5580111	3014.01
S _{2,14, t₁}	11.313	7.00E-05	161607.1	28.26	90840	5485613	3214.44		
S _{3,14, t₁}	11.375	6.90E-05	164855.1	28.26	78879.36	5708325	2791.2		
S _{1,14, t₂}	11.75	7.40E-05	158783.8	28.26	99136.32	5295814	3508.01	5298850	3266.29
S _{2,14, t₂}	11.375	7.20E-05	157986.1	28.26	71190.72	5242541	2519.13		
S _{3,14, t₂}	11.5	7.20E-05	159722.2	28.26	106588.8	5358395	3771.72		
S _{1,14, t₃}	11.75	7.40E-05	158783.8	28.26	88751.04	5295614	3140.52	5276885	2918.84
S _{2,14, t₃}	11.75	7.40E-05	158783.8	28.26	86853.12	5295614	3073.36		
S _{3,14, t₃}	11.688	7.40E-05	157939.2	28.26	71855.04	5239428	2542.64		
S _{1,14, t₄}	11.875	7.40E-05	160473	28.26	94067.52	5408886	3328.65	5286214	3063.89
S _{2,14, t₄}	11.688	7.40E-05	157939.2	28.26	79543.68	5239428	2814.71		
S _{3,14, t₄}	11.813	7.50E-05	157500	28.26	86145.6	5210329	3048.32		
S _{1,14, t₂₄}	11.75	7.50E-05	156666.7	28.26	93592.32	5155339	3311.83	5155339	3296.14
S _{2,14, t₂₄}	11.75	7.50E-05	156666.7	28.26	87991.68	5155339	3113.65		
S _{3,14, t₂₄}	11.75	7.50E-05	156666.7	28.26	97862.4	5155339	3462.93		

Properties of set # 2

Curing Time: (28 Days)

Frequency (40Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,28, t₀}	11.313	7.00E-05	181807.1	28.26	67774.08	5485613	2398.23	5342377	2406.62
S _{2,28, t₀}	11.625	7.30E-05	159246.6	28.26	0	5326529	0		
S _{3,28, t₀}	11.188	7.10E-05	157570.4	28.26	68248.32	5214990	2415.01		
S _{1,28, t₁}	11.438	6.90E-05	165760.9	28.26	0	5771226	0	5758220	3043.13
S _{2,28, t₁}	11.5	6.90E-05	166666.7	28.26	82866.24	5834472	2932.28		
S _{3,28, t₁}	11.5	7.00E-05	164285.7	28.26	89131.2	5668963	3153.97		
S _{1,28, t₂}	11.688	7.30E-05	160102.7	28.26	93023.04	5383957	3291.69	5403202	3387.33
S _{2,28, t₂}	11.75	7.30E-05	160958.9	28.26	0	5441693	0		
S _{3,28, t₂}	11.688	7.30E-05	160102.7	28.26	98428.8	5383957	3482.97		
S _{1,28, t₃}	11.625	7.30E-05	159246.6	28.26	97286.4	5326529	3442.55	5335367	3536.65
S _{2,28, t₃}	11.688	7.30E-05	160102.7	28.26	102604.8	5383957	3630.74		
S _{3,28, t₃}	11.75	7.40E-05	158783.8	28.26	0	5295614	0		
S _{1,28, t₄}	11.688	7.30E-05	160102.7	28.26	85144.32	5383957	3012.89	5374374	3429.33
S _{2,28, t₄}	11.813	7.30E-05	161815.1	28.26	0	5499738	0		
S _{3,28, t₄}	11.688	7.40E-05	157939.2	28.26	108681.6	5239428	3845.77		
S _{1,28, t₂₄}	11.688	7.30E-05	160102.7	28.26	98236.8	5383957	3476.18	5343891	3263.01
S _{2,28, t₂₄}	11.75	7.40E-05	158783.8	28.26	86188.8	5295614	3049.85		
S _{3,28, t₂₄}	11.813	7.40E-05	159628.4	28.26	0	5352100	0		

Properties of set # 3

Curing Time: (3 Days)

Frequency (10Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,3, t₀}	11.5	7.00E-05	164286	28.26	80873	5668963	2861.758	5616109	2922.2
S _{2,3, t₀}	11.5	7.10E-05	161972	28.26	74988	5510399	2653.486		
S _{3,3, t₀}	11.5	7.00E-05	164286	28.26	91884	5668963	3251.363		
S _{1,3, t₁}	11.563	7.10E-05	162852	28.26	79924	5570457	2828.161	5643657	3422.66
S _{2,3, t₁}	11.5	7.00E-05	164286	28.26	103939	5668963	3677.962		
S _{3,3, t₁}	11.688	7.10E-05	164613	28.26	106310	5691551	3761.868		
S _{1,3, t₂}	11.563	7.00E-05	165179	28.26	84765	5730750	2999.473	5644018	3338.68
S _{2,3, t₂}	11.563	7.10E-05	162852	28.26	102230	5570457	3617.495		
S _{3,3, t₂}	11.625	7.10E-05	163732	28.26	96058	5630841	3399.068		
S _{1,3, t₃}	11.688	7.30E-05	160103	28.26	57996	5383957	2052.246	5414476	2933.34
S _{2,3, t₃}	11.625	7.20E-05	161458	28.26	90555	5475515	3204.348		
S _{3,3, t₃}	11.688	7.30E-05	160103	28.26	100138	5383957	3543.439		
S _{1,3, t₄}	11.625	7.30E-05	159247	28.26	76886	5326529	2720.679	5318640	2794.58
S _{2,3, t₄}	11.5	7.30E-05	157534	28.26	75273	5212595	2663.575		
S _{3,3, t₄}	11.563	7.20E-05	160590	28.26	84765	5416797	2999.473		
S _{1,3, t₂₄}	11.625	7.20E-05	161458	28.26	100522	5475515	3557.028	5414579	3218.9
S _{2,3, t₂₄}	11.625	7.30E-05	159247	28.26	82676	5326529	2925.554		
S _{3,3, t₂₄}	11.75	7.30E-05	160959	28.26	89700	5441693	3174.115		

Properties of set # 3

Curing Time: (7 Days)

Frequency (10Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,7, t₀}	11.5	6.90E-05	166866.7	28.26	105734.4	5834472	3741.49	5765324	3780.78
S _{2,7, t₀}	11.563	7.00E-05	165178.6	28.26	103363.2	5730750	3657.58		
S _{3,7, t₀}	11.563	7.00E-05	165178.6	28.26	111436.8	5730750	3943.27		
S _{1,7, t₁}	11.563	7.20E-05	160590.3	28.26	94161.6	5416797	3331.97	5552073	3645.41
S _{2,7, t₁}	11.563	7.10E-05	162852.1	28.26	98428.8	5570457	3482.97		
S _{3,7, t₁}	11.5	7.00E-05	164285.7	28.26	116467.2	5668963	4121.27		
S _{1,7, t₂}	11.5	7.00E-05	164285.7	28.26	91978.56	5668963	3254.73	5623421	3649.89
S _{2,7, t₂}	11.563	7.10E-05	162852.1	28.26	106022.4	5570457	3751.68		
S _{3,7, t₂}	11.625	7.10E-05	163732.4	28.26	111436.8	5630841	3943.27		
S _{1,7, t₃}	11.5	7.10E-05	161971.8	28.26	63882.24	5510399	2260.52	5530418	3298.33
S _{2,7, t₃}	11.5	7.10E-05	161971.8	28.26	106022.4	5510399	3751.68		
S _{3,7, t₃}	11.563	7.10E-05	162852.1	28.26	109728	5570457	3882.8		
S _{1,7, t₄}	11.625	7.30E-05	159246.6	28.26	78879.36	5326529	2791.2	5269465	3185.17
S _{2,7, t₄}	11.75	7.50E-05	156666.7	28.26	83815.68	5155339	2965.88		
S _{3,7, t₄}	11.625	7.30E-05	159246.6	28.26	105648	5326529	3738.43		
S _{1,7, t₂₄}	11.75	7.30E-05	160958.9	28.26	103171.2	5441693	3650.79	5393000	3572.53
S _{2,7, t₂₄}	11.75	7.30E-05	160958.9	28.26	87138.24	5441693	3083.45		
S _{3,7, t₂₄}	11.75	7.40E-05	158783.8	28.26	112569.6	5295614	3983.35		

Properties of set # 3

Curing Time: (14 Days)

Frequency (10Hz)

Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f_c (psi)	AVG (E)	AVG (f_c)
S _{1,14,t₀}	11.625	6.90E-05	168478.3	28.26	107539.2	5961998	3805.35	5919374	3990.15
S _{2,14,t₀}	11.563	6.90E-05	167572.5	28.26	128803.2	5898062	4557.79		
S _{3,14,t₀}	11.563	6.90E-05	167572.5	28.26	101942.4	5898062	3607.3		
S _{1,14,t₁}	11.5	6.90E-05	166666.7	28.26	92737.92	5834472	3281.6	5820605	3726
S _{2,14,t₁}	11.625	7.00E-05	166071.4	28.26	121113.6	5792872	4285.69		
S _{3,14,t₁}	11.5	6.90E-05	166666.7	28.26	102038.4	5834472	3610.7		
S _{1,14,t₂}	11.5	6.90E-05	166866.7	28.26	106972.8	5834472	3785.31	5806850	3770.8
S _{2,14,t₂}	11.688	7.00E-05	166964.3	28.26	91314.24	5855328	3231.22		
S _{3,14,t₂}	11.563	7.00E-05	165178.6	28.26	121401.6	5730750	4295.88		
S _{1,14,t₃}	11.75	7.10E-05	165493	28.26	87043.2	5752586	3080.08	5800262	3402.45
S _{2,14,t₃}	11.688	7.00E-05	166964.3	28.26	111811.2	5855328	3956.52		
S _{3,14,t₃}	11.625	7.00E-05	166071.4	28.26	89605.44	5792872	3170.75		
S _{1,14,t₄}	11.813	7.20E-05	164082.5	28.26	94541.76	5653569	3345.43	5699452	3486.49
S _{2,14,t₄}	11.625	7.10E-05	163732.4	28.26	111532.8	5630841	3946.67		
S _{3,14,t₄}	11.813	7.10E-05	166373.2	28.26	89510.4	5813946	3167.39		
S _{1,14,t₂₄}	11.75	7.10E-05	165493	28.26	126144	5752586	4463.69	5627012	4033.86
S _{2,14,t₂₄}	11.688	7.20E-05	162326.4	28.26	96912	5534550	3429.3		
S _{3,14,t₂₄}	11.75	7.20E-05	163194.4	28.26	118934.4	5593901	4208.58		

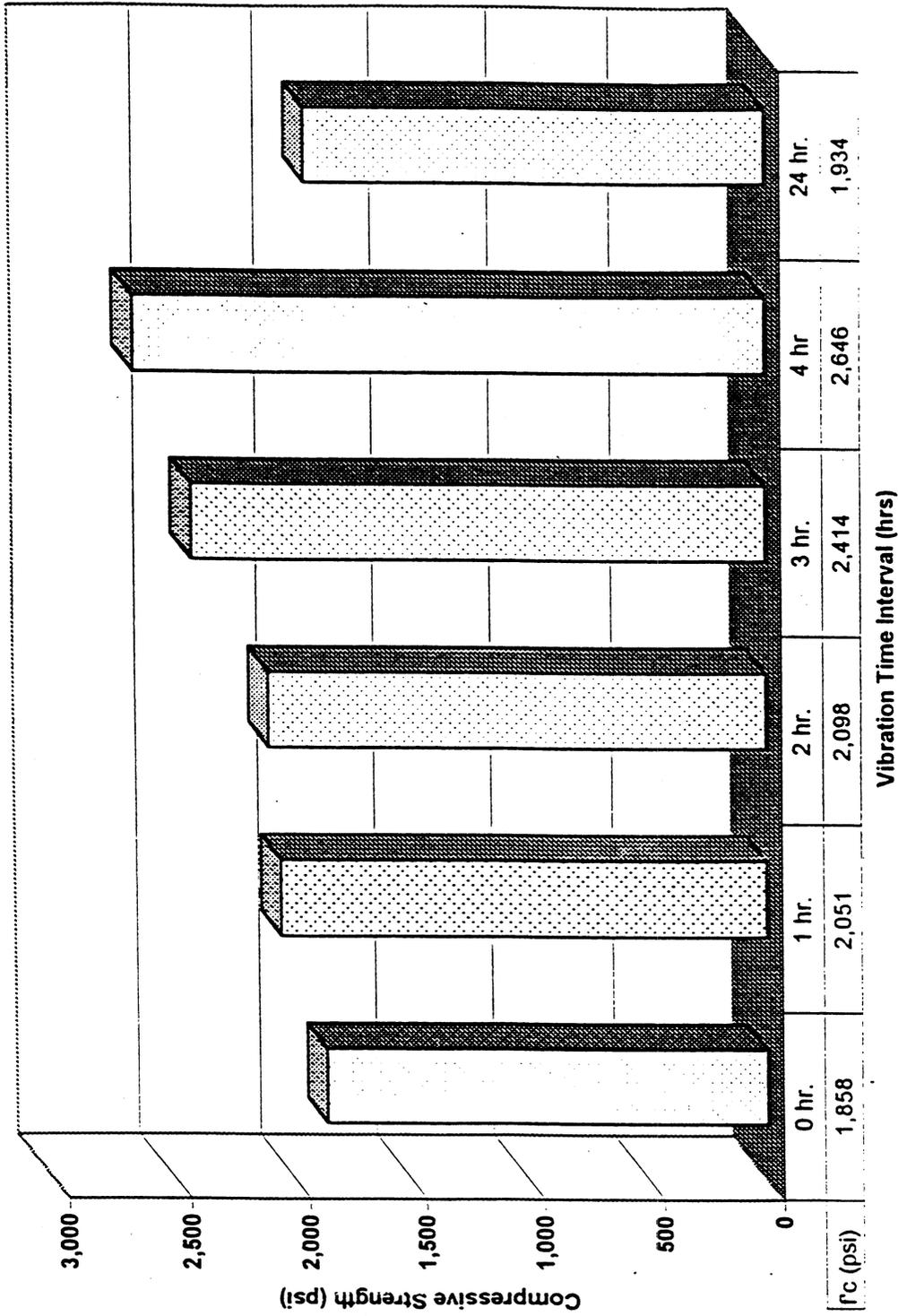
Properties of set # 3

Curring : (28 Days)

Frequency (10Hz)

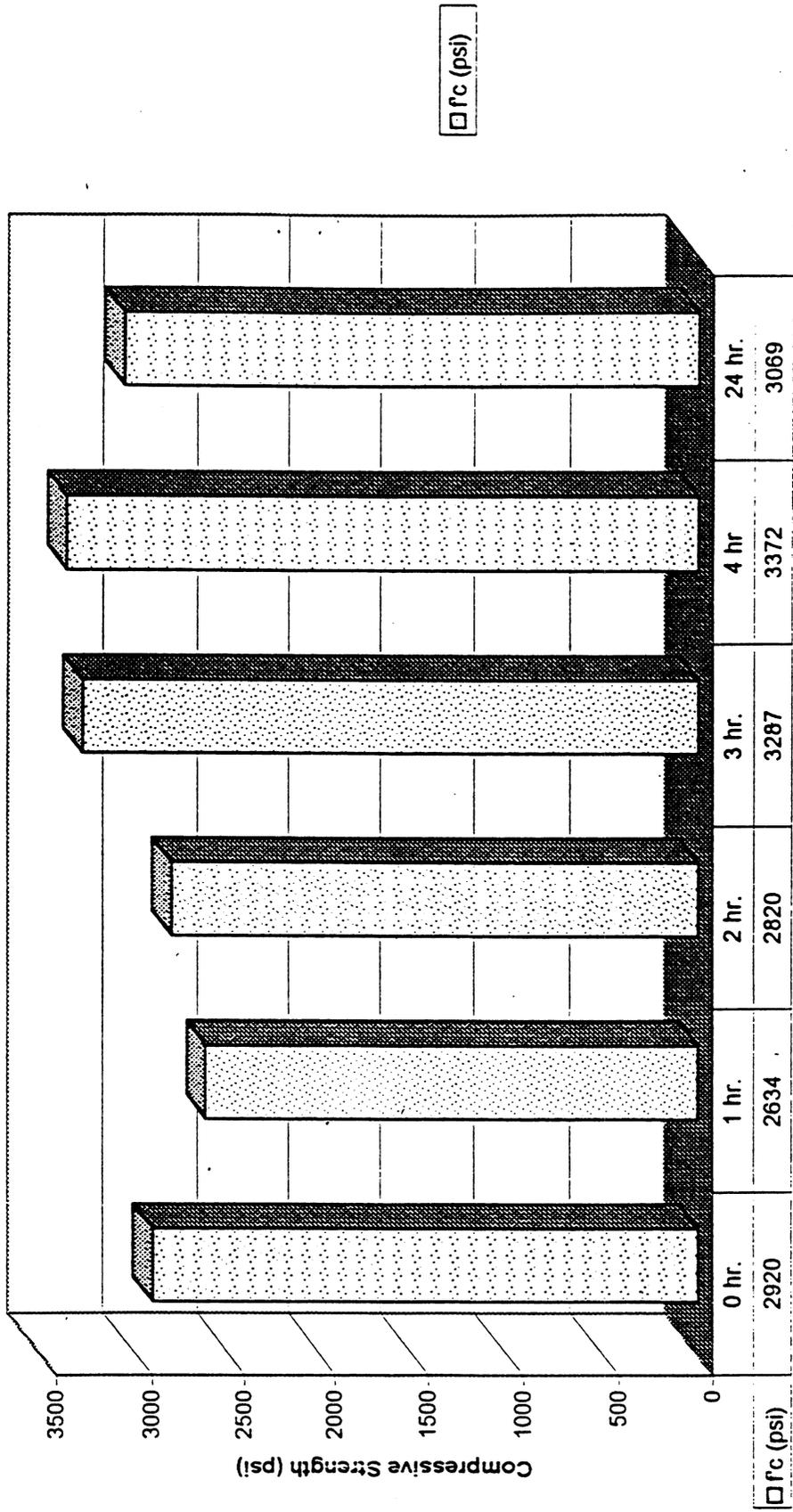
Sample	Height (in)	Time (s)	Velocity (in/s)	Area (in ²)	Force (lb)	E (psi)	f _c (psi)	AVG (E)	AVG (f _c)
S _{1,28} , t ₀	11.625	6.70E-05	173507.5	28.26	0	6323250	0	6359164	4151
S _{2,28} , t ₀	11.563	6.60E-05	175189.4	28.26	108489.6	6446436	3838.98		
S _{3,28} , t ₀	11.438	6.80E-05	173295.5	28.26	126124.8	6307807	4463.01		
S _{1,28} , t ₁	11.625	6.80E-05	170955.9	28.26	84099.84	6138640	2975.93	6280172	3400.83
S _{2,28} , t ₁	11.563	6.70E-05	172574.6	28.26	108115.2	6255441	3825.73		
S _{3,28} , t ₁	11.563	6.60E-05	175189.4	28.26	0	6446436	0		
S _{1,28} , t ₂	11.75	6.80E-05	172794.1	28.26	97104	6271363	3436.09	6328680	3395.8
S _{2,28} , t ₂	11.688	6.70E-05	174440.3	28.26	94826.88	6391425	3355.52		
S _{3,28} , t ₂	11.625	6.70E-05	173507.5	28.26	0	6323250	0		
S _{1,28} , t ₃	11.688	6.80E-05	171875	28.26	0	6204824	0	6160701	4849.94
S _{2,28} , t ₃	11.625	6.80E-05	170955.9	28.26	133641.6	6138640	4729		
S _{3,28} , t ₃	11.625	6.80E-05	170955.9	28.26	140476.8	6138640	4970.37		
S _{1,28} , t ₄	11.625	6.80E-05	170955.9	28.26	117417.6	6138640	4154.9	6048721	3667.87
S _{2,28} , t ₄	11.813	7.00E-05	168750	28.26	0	5981245	0		
S _{3,28} , t ₄	11.688	6.90E-05	169384.1	28.26	89890.56	6028277	3180.84		
S _{1,28} , t ₂₄	11.813	7.00E-05	168750	28.26	96912	5981245	3429.3	6119434	3822.34
S _{2,28} , t ₂₄	11.875	6.90E-05	172101.4	28.26	119126.4	6221185	4215.37		
S _{3,28} , t ₂₄	11.813	6.90E-05	171195.7	28.26	0	6155871	0		

**3-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #1**



Frequency of Vibration: 20% Vibration Time: 15 min.

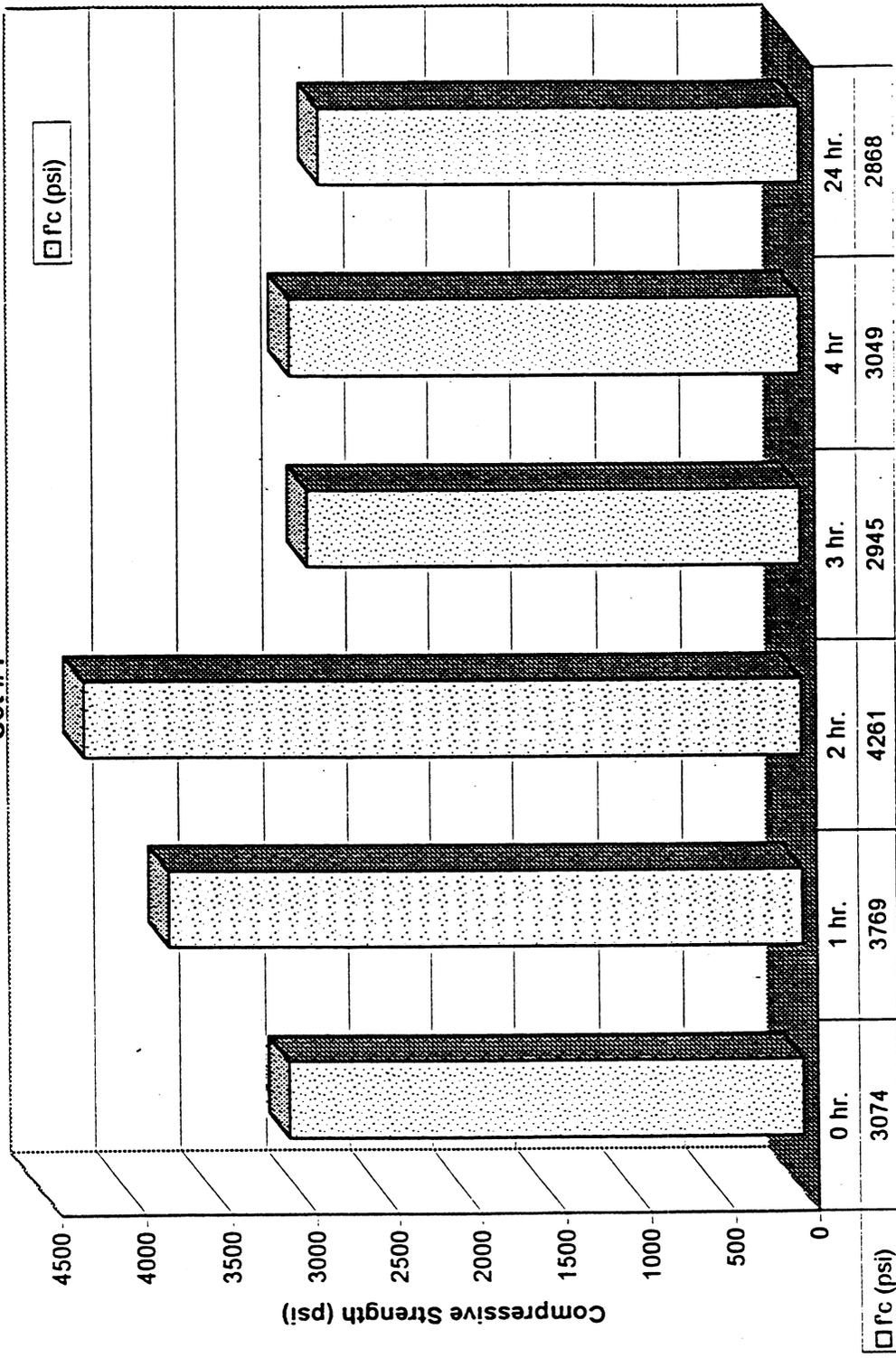
**7-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #1**



Vibration Time Interval (hrs)

Frequency of Vibration: 20% Vibration Time: 15 min.

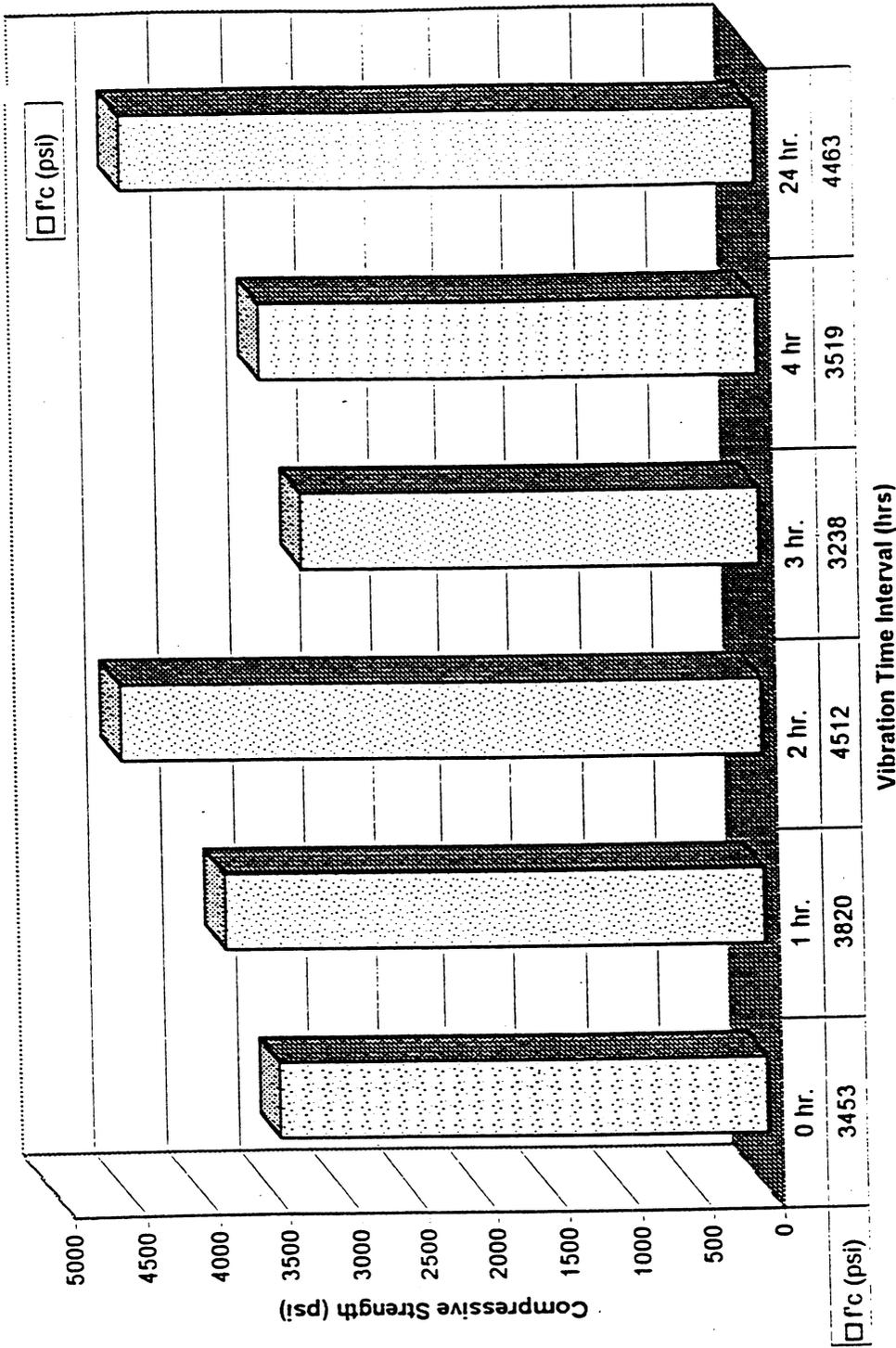
**14-Days Compressive Strength after 15-minutes Vibration
per Time Interval
Set #1**



Vibration Time Interval (hrs)

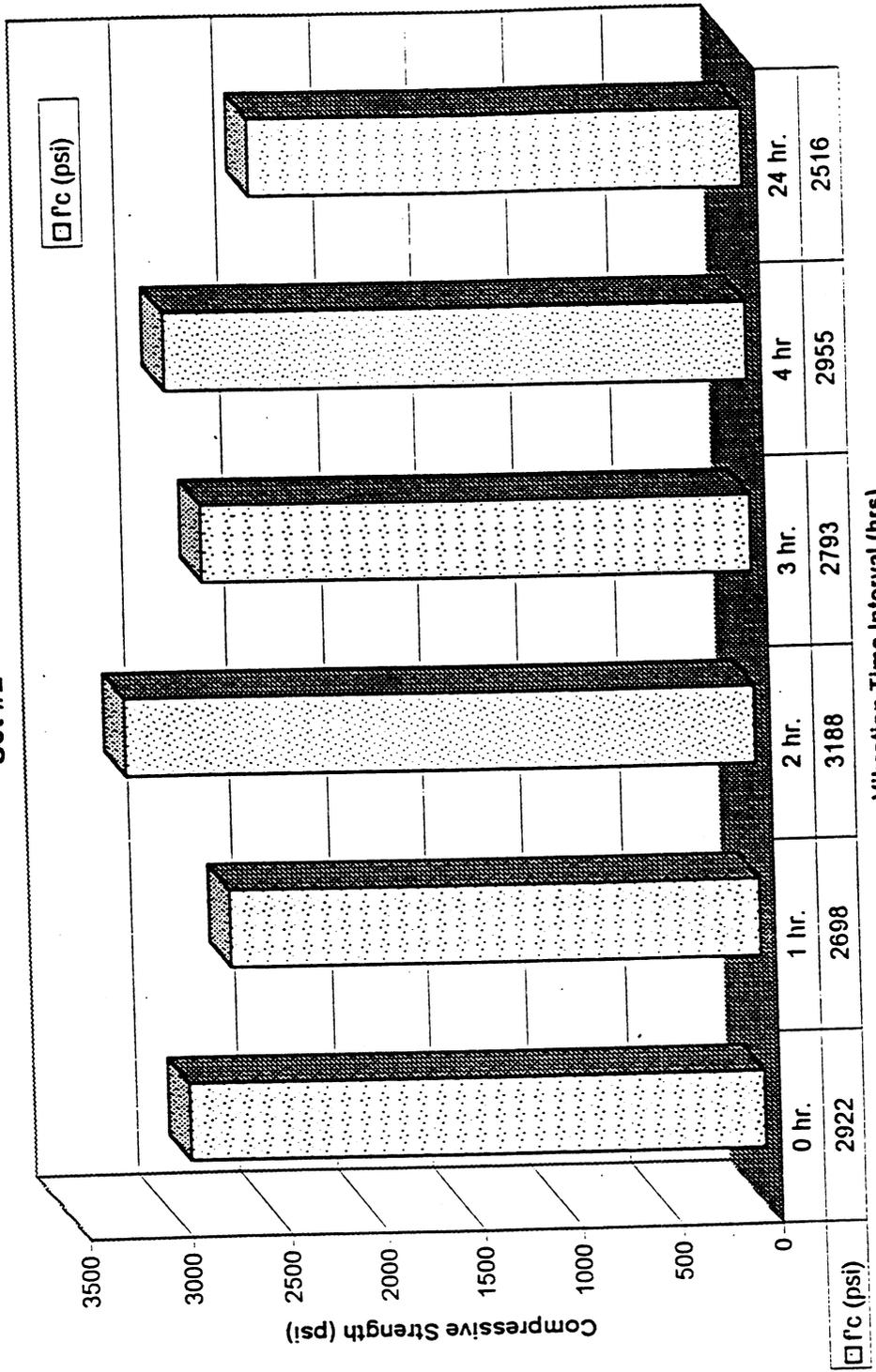
Frequency of Vibration: 20% Vibration Time: 15 min.

**28-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #1**



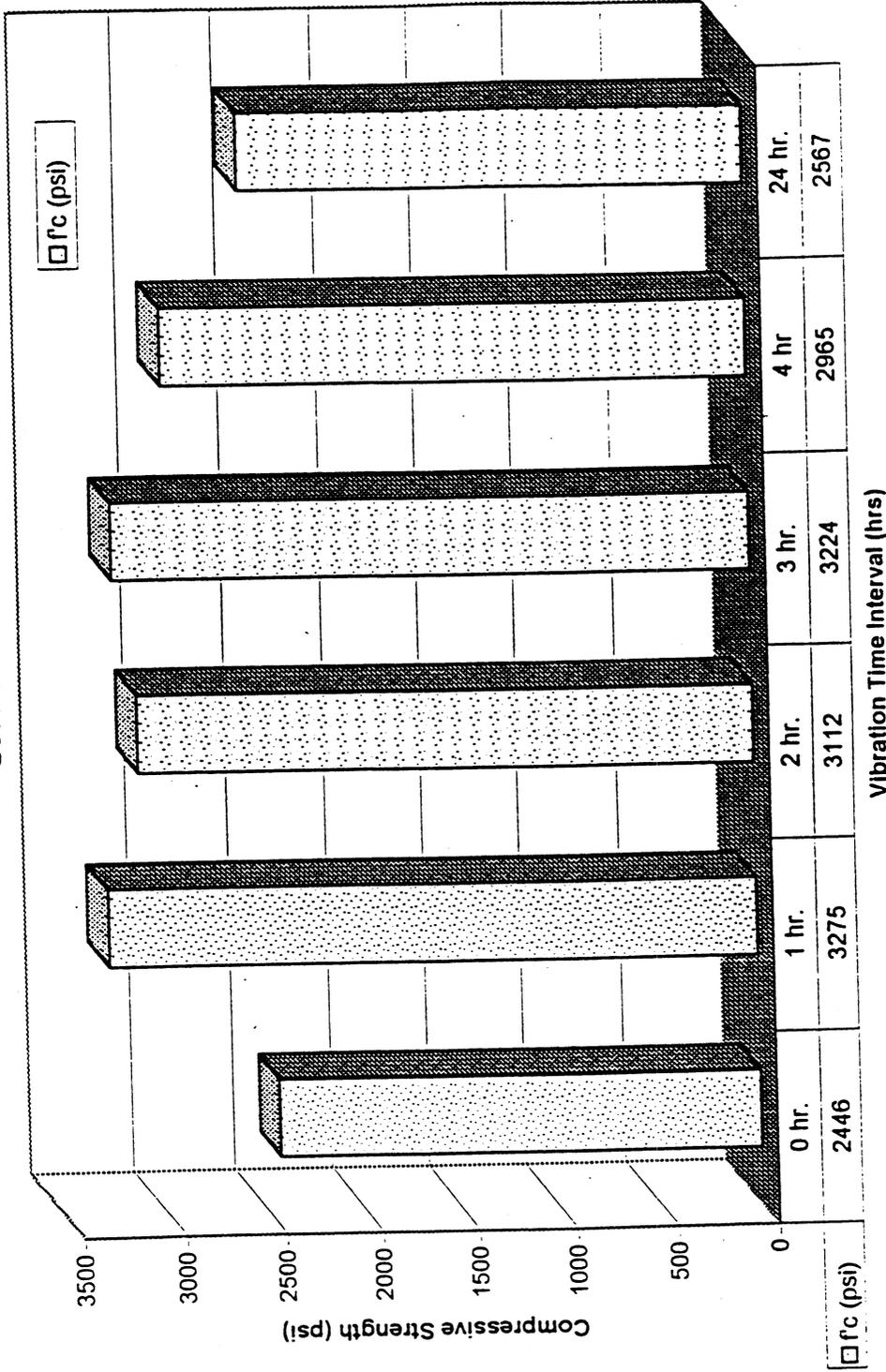
Frequency of Vibration: 20% Vibration Time: 15 min.

**3-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #2**



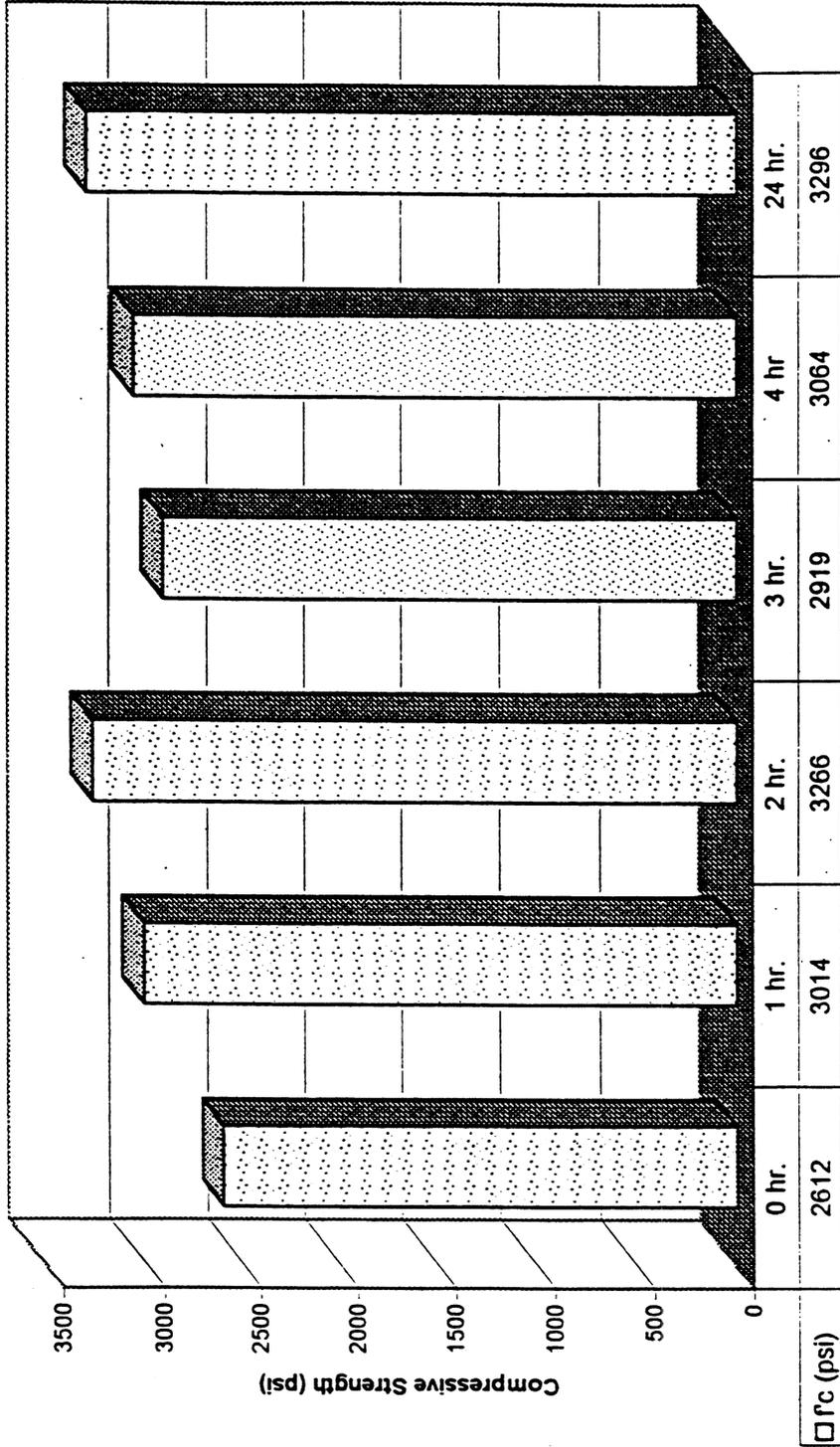
Frequency of Vibration: 40% Vibration Time: 15 min.

**7-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #2**



Frequency of vibration: 40% Vibration Time: 15 min.

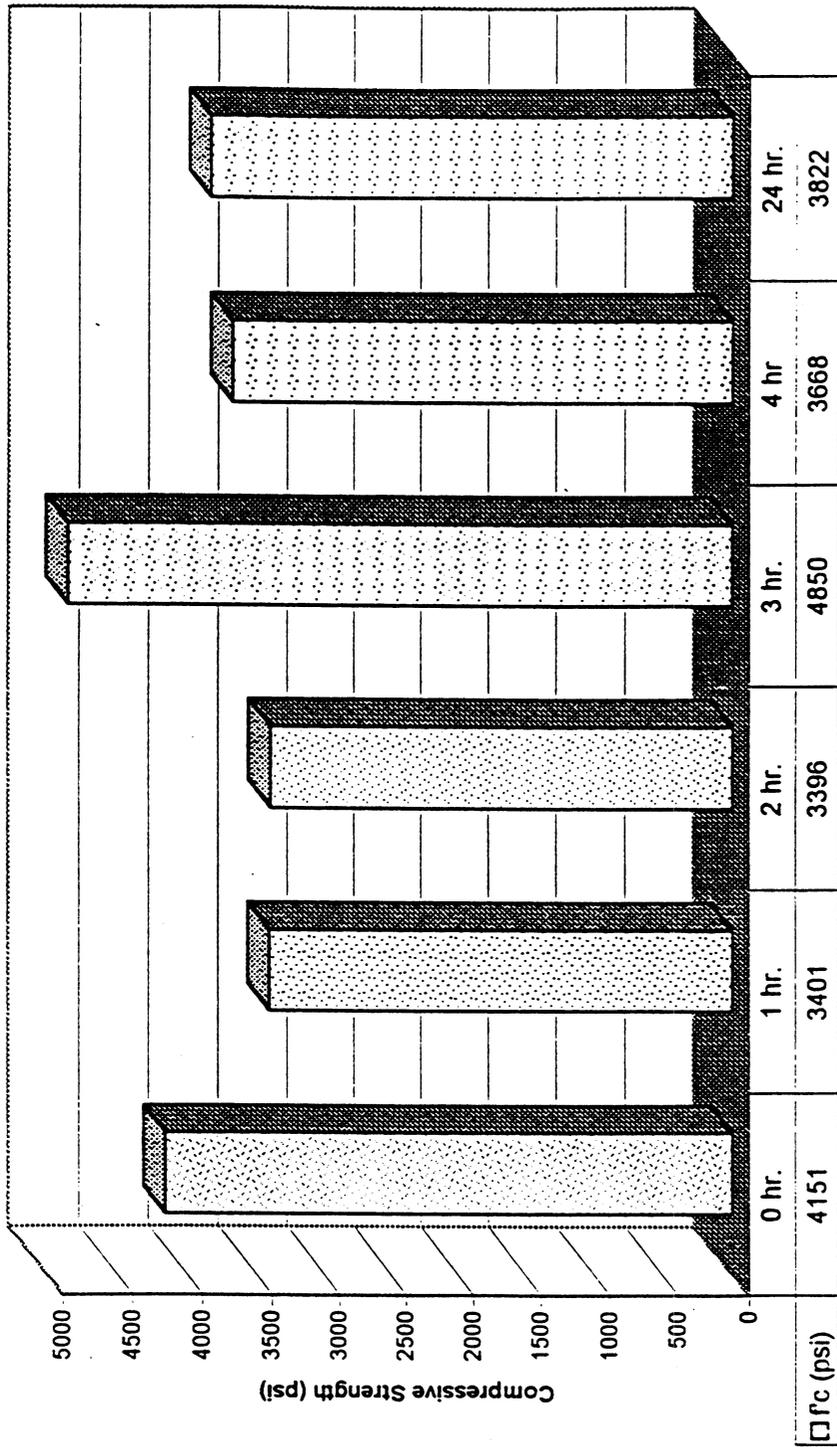
**14-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #2**



Vibration Time Interval (hrs)

Frequency of Vibration: 40% Vibration Time: 15 min.

**28-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #3**



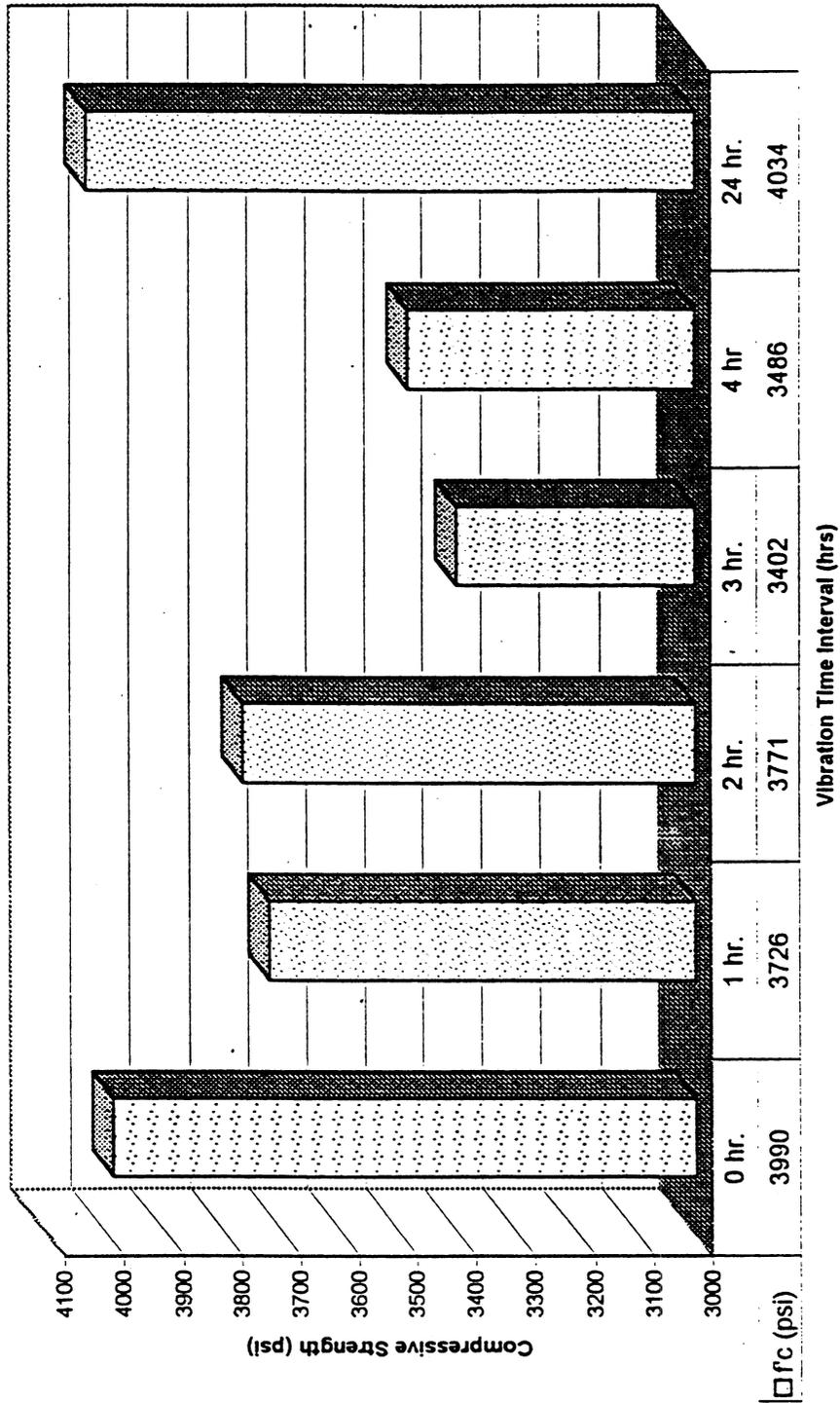
Vibration Time Interval (hrs)

Frequency of Vibration: 10%

Vibration Time: 15 min.

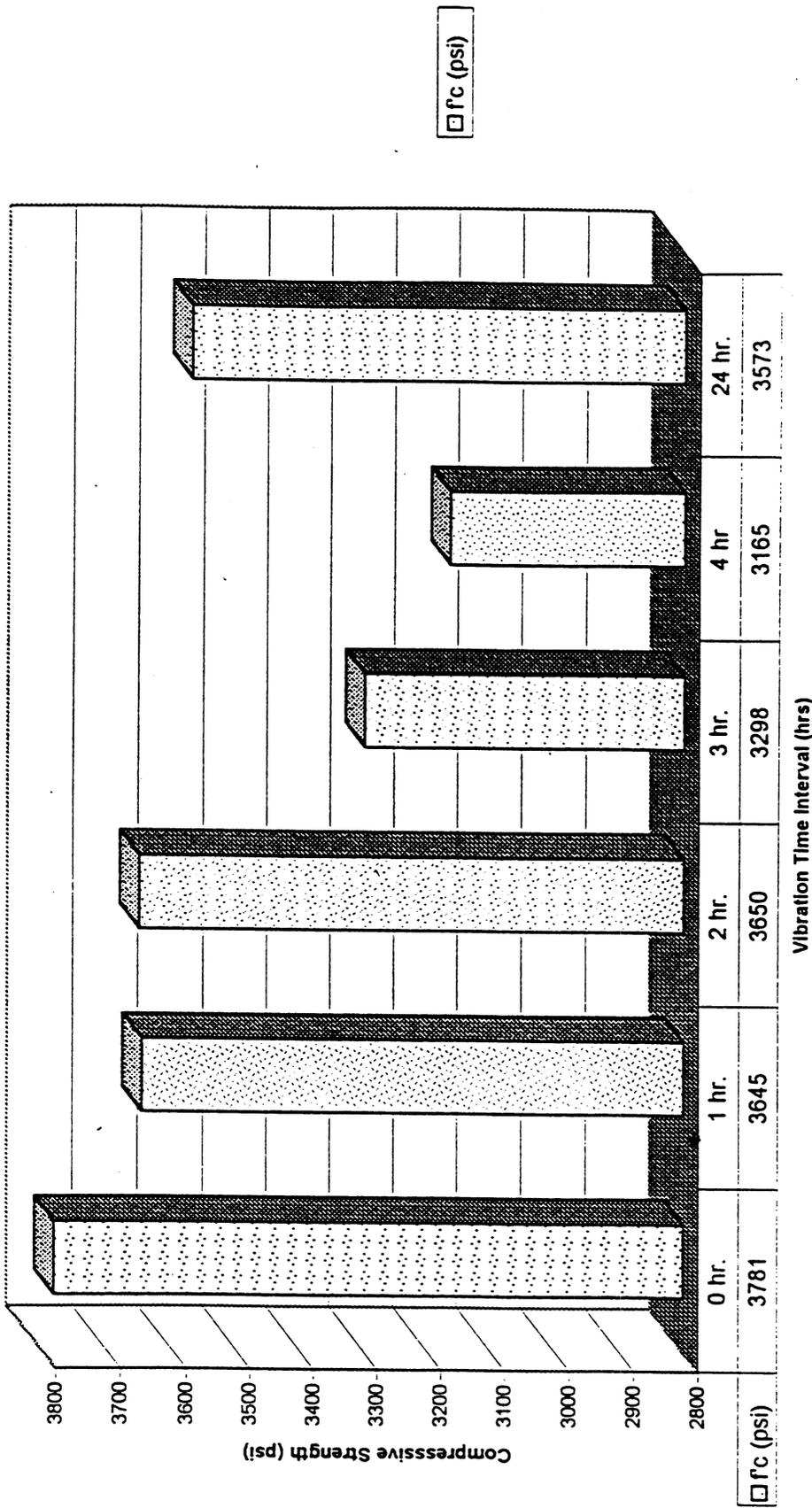
473

**14-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #3**



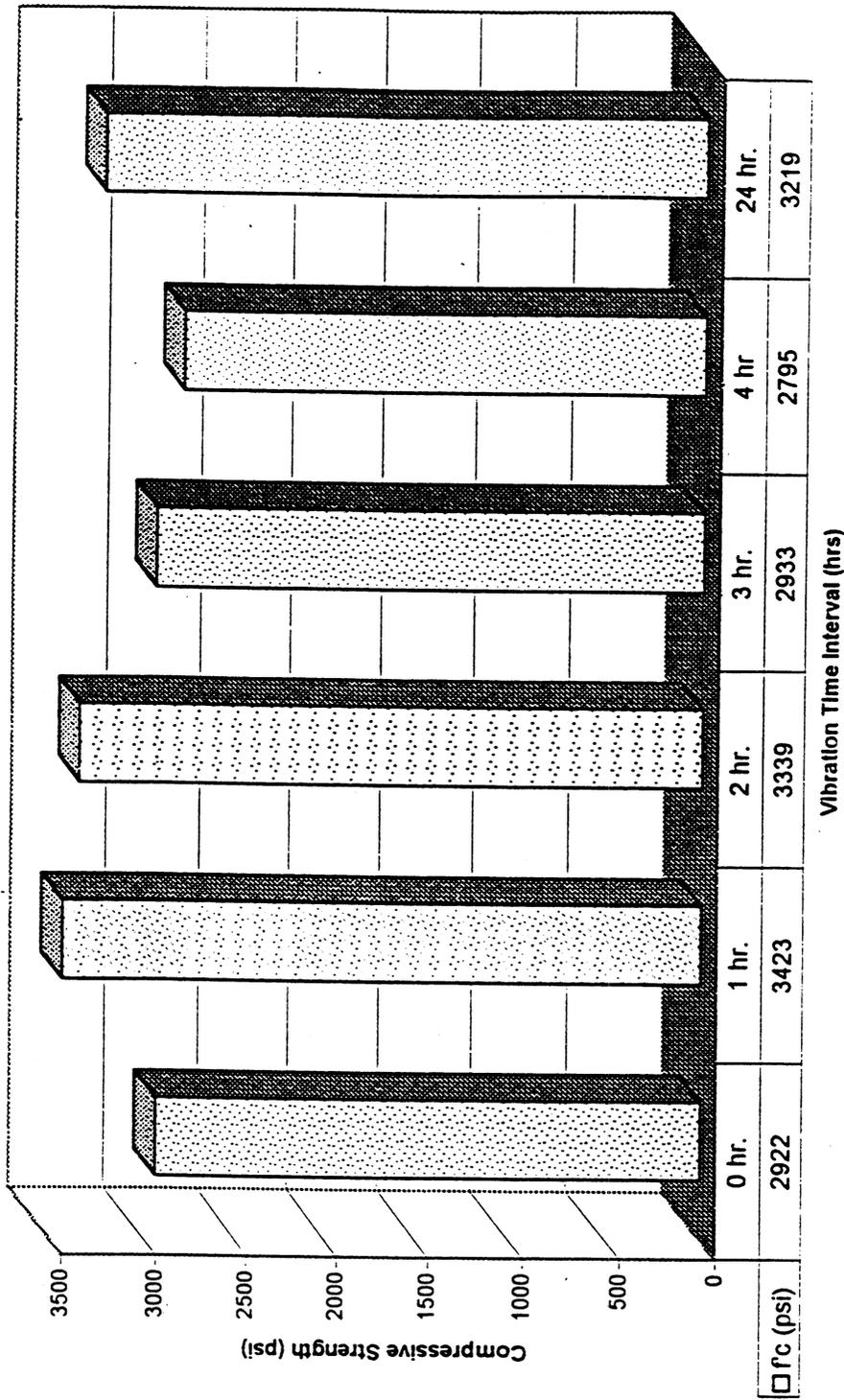
Frequency of Vibration: 10% Vibration Time: 15 min.

**7-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #3**



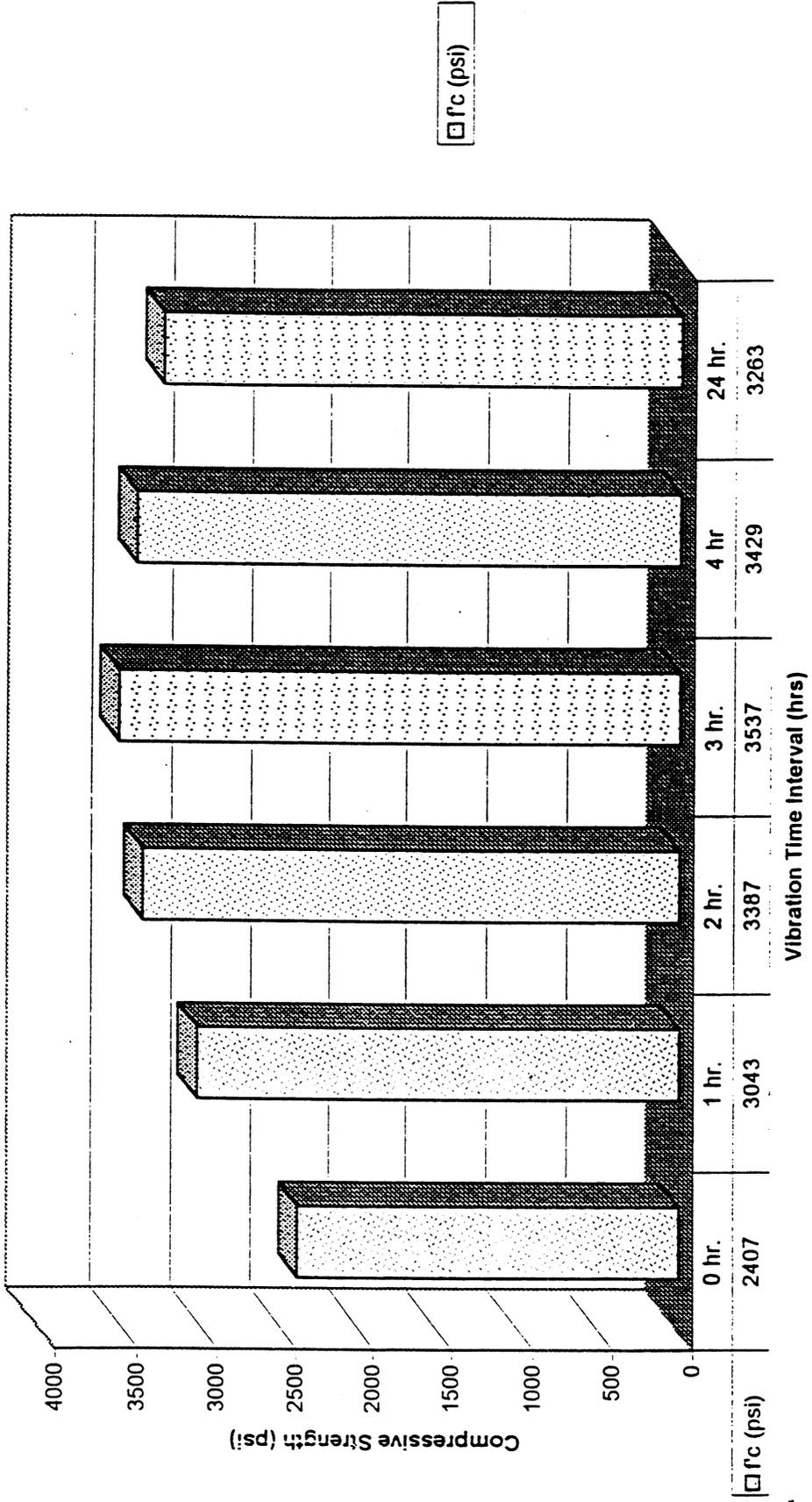
Frequency of Vibration: 10% Vibration Time: 15 min.

**3-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #3**



Frequency of Vibration: 10% Vibration Time: 15 min.

**28-Day Compressive Strength after 15-minutes Vibration
per Time Interval
Set #2**



APPENDIX B

STANDARD PENETRATION TESTS

DELRAY BEACH

FDOT, District IV

FROM: DISTRICT MATERIALS

TO: 4073572789

APR 5, 1998 1:42PM 423 P.

FIELD BORING LOG

SHEET 1 of 3

PROJECT NO. FW Research NAME _____ COUNTY Palm Beach Co DISTRICT 4
 LOCATION _____ TOWNSHIP _____ RANGE _____ SECTION _____
 ROAD NUMBER I-95 to Linton SURFACE ELEVATION _____
 EQUIPMENT TYPE CME-73 RIC NO. DOT 24526 BORING NO. B-1
 DATE STARTED 3/4/93 COMPLETED 3/4/93 DRILLED BY INNUCK, Lewis, Benjamin
 LOGGED BY DIAGNOSE BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, Strength Standard
 WATER TABLE: 0 HR. 7.6' 24 HRS. _____ HRS. _____
 CASED, UNCASED, DRILLING MUD
 SIZE 140 & AUTOMATIC HAMMER.

SAMPLE CONDITIONS:  DISTURBED SAMPLE TYPES: A: AUGER
 GOOD ~~SB: SPLIT BARREL~~
 LOST S: SHELBY TUBE
 CORE SAMPLE RC: ROCK CORE SIZE

TESTS: W.C.: WATER CONTENT (%)
 T: TORVANE (TSP)
 V: IN-SITU VANE TEST (TSP)

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES			TESTS	REMARKS
				CON.	NO. TYPE	REC. (%)		
	1	1	light to dark tan fine sand		1	50		
	2	2	tan fine sand.		2	60		
	3	3			3	70		
	4	4			4	70		
	5	5			5	70		
	6	6			6	70		
	7	7			7	70		
	8	8			8	70		
	9	9			9	70		
	10	10			10	70		
	11	11			11	60		
	12	12			12	50		
	13	13		Dark to light tan fine sand		13	60	
	14	14	light tan fine sand		14	60		
	15	15	light gray fine sand		15			
	16	16			16			
	17	17			17			
	18	18						
	19	19						
	20	20						

RECYCLED PAPER

FROM: DISTRICT MATERIALS

TO: 4873672789

SEP 9. 1998 11:41AM 2483 P.2

FIELD BORING LOG

SHEET 2 OF 3

PROJECT NO. _____ NAME _____ COUNTY _____ DISTRICT _____
 LOCATION _____ TOWNSHIP _____ RANGE _____ SECTION _____
 ROAD NUMBER _____ SURFACE ELEVATION _____
 EQUIPMENT TYPE _____ R/O NO. _____ BORING NO. e-1
 DATE STARTED _____ COMPLETED _____ DRILLED BY _____
 LOGGED BY _____ BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, _____
 WATER TABLE: 0 HR. _____ 24 HRS. _____ HRS _____ CASED, UNCASD, DRILLING MUD. _____

SAMPLE CONDITIONS:  **DISTURBED SAMPLE TYPES:** A: AUGER
 GOOD SB: SPLT BARREL
 LOST S: SHELBY TUBE
 CORE SAMPLE RC: ROCK CORE **TESTS:** W.C.: WATER CONTENT (%)
 T: TORVANE (TSF)
 V: IN-SITU VANE TEST (TSP) **SIZE**

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES			TESTS	REMARKS
				CON.	NO. TYPE	REC. (%)		
	21	4 5 7	light gray fine sand		13	40		
	22							
	23	5 4	" " "		14	40		
	24	4						
	25	3 4			15	40		
	26	2						
	27	1			16	40		
	28	2						
	29	1						
	30	1	light gray fine sand with		17	30		*Advanced 12" under weight of hammer
	31	1	remained sand, and shell fragments.					
	32							
	33	4 5	" " "		18	40		
	34	5						
	35	5			19	30		
	36	4 5						
	37							
	38	3 4			20	30		
	39							
	40							

RECYCLED PAPER

FROM: DISTRICT MATERIALS

TO: 4873672789

APR 9, 1998 1:42PM 2483 P

FIELD BORING LOG

SHEET 1 OF 1

PROJECT NO. FAU - Research NAME _____ COUNTY Polk Beach DISTRICT 4
 LOCATION _____ TOWNSHIP _____ RANGE _____ SECTION _____
 ROAD NUMBER T-9E 3rd Line Rd SURFACE ELEVATION _____
 EQUIPMENT TYPE C.M.C. - 75 RIC NO. 24496 BORING NO. B-2
 DATE STARTED 3-5-98 COMPLETED 3-5-98 DRILLED BY Leland Lorus Pedraza
 LOGGED BY SURASANI BORING TYPE: AUGER WASHED, PERCUSSION, ROTARY, _____
 WATER TABLE: 0 HR. 9.3' 24 HRS. _____ HRS. _____
 SAMPLE CONDITIONS: DISTURBED GOOD LOST CORE SAMPLE
 SAMPLE TYPES: A: AUGER SB: SPLIT BARREL S: SHELBY TUBE RC: ROCK CORE SIZE _____
 TESTS: W.C.: WATER CONTENT (%) T: TORVANE (TSF) V: IN-SITU VANE TEST (TSF)
 140 lb. core Automatic hammer

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES			TESTS	REMARKS
				CON.	NO. TYPE	REC (S)		
	1	1	Fine to coarse		1	60		
	2	2	" "		2	70		
	3	3	" "		3	70		
	4	4	" "		4	60		
	5	5	" "		5	60		
	6	6	" "		6	60		
	7	7	" "		7	60		
	8	8	" "		8	60		
	9	9	" "		9	60		
	10	10	" "		10	60		
	11	11	" "		11	60		
	12	12	" "		12	60		
	13	13	" "		13	60		*Turning to lighter tan / grey
	14	14	" "		14	60		
	15	15	" "		15	60		
	16	16	" "		16	60		
	17	17	" "		17	60		
	18	18	" "		18	60		
	19	19	" "		19	60		
	20	20	" "		20	60		

RECYCLED PAPER

FROM: DISTRICT MATERIALS

TO: 4373672789

APR 9. 1998 1:43PM 0483 P. 0

FIELD BORING LOG

SHEET 2 of 3

PROJECT NO _____ NAME _____ COUNTY _____ DISTRICT _____
 LOCATION _____ TOWNSHIP _____ RANGE _____ SECTION _____
 ROAD NUMBER _____ SURFACE ELEVATION _____
 EQUIPMENT TYPE APIE RIG NO. _____ BORING NO. B-2
 DATE STARTED _____ COMPLETED _____ DRILLED BY _____
 LOGGED BY _____ BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, _____
 WATER TABLE: 0 HRS. _____ 24 HRS. _____ HRS. _____
 CASE# UNCASED, DRILLING MUD. _____

SAMPLE CONDITIONS: DISTURBED GOOD LOST CORE SAMPLE
 SAMPLE TYPES: A: AUGER SB: SPLIT BARREL S: SHELBY TUBE RC: ROCK CORE
 TESTS: W.C.: WATER CONTENT (%) T: TORVANE (TSF) V: IN-SITU VANE TEST (TSF) SIZE _____

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES		TESTS	REMARKS
				CON.	NO. REC. TYPE (%)		
	21	5 5 6	Gray fine sand		13 60		
	22						
	23	7 4	" "		14 60		
	24	3					
	25						
	26	3 4	" "		15 60		
	27						
	28	3	" "		16 60		
	29	3					
	30						
	31	2 2	" "		17 60		
	32						
	33	3 2	Gray sand with cemented sand fragments (traces of shell)		18 60		
	34	2					
	35	4	" "		19 60		
	36	2 2					
	37						
	38	3 6	Gray sand and shell traces of cemented sand		20 50		
	39	2					
	40						

RECYCLED PAPER

APPENDIX C

PEAK PARTICLE VELOCITY V_s.
TIME RECORDS

September 98

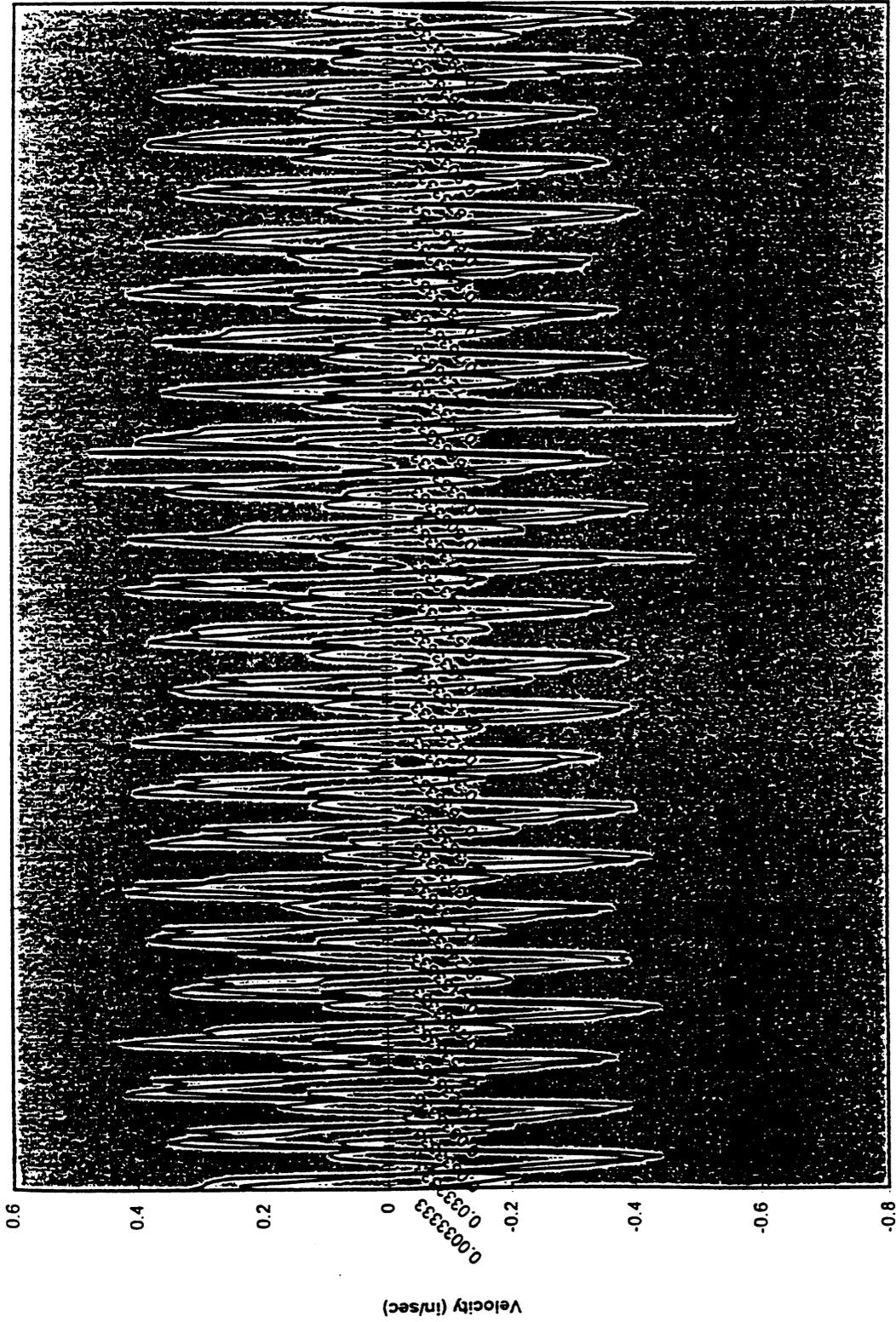
DELRAY BEACH

**LAW ENGINEERING AND ENVIRONMENTAL SERVICES,
INC.**

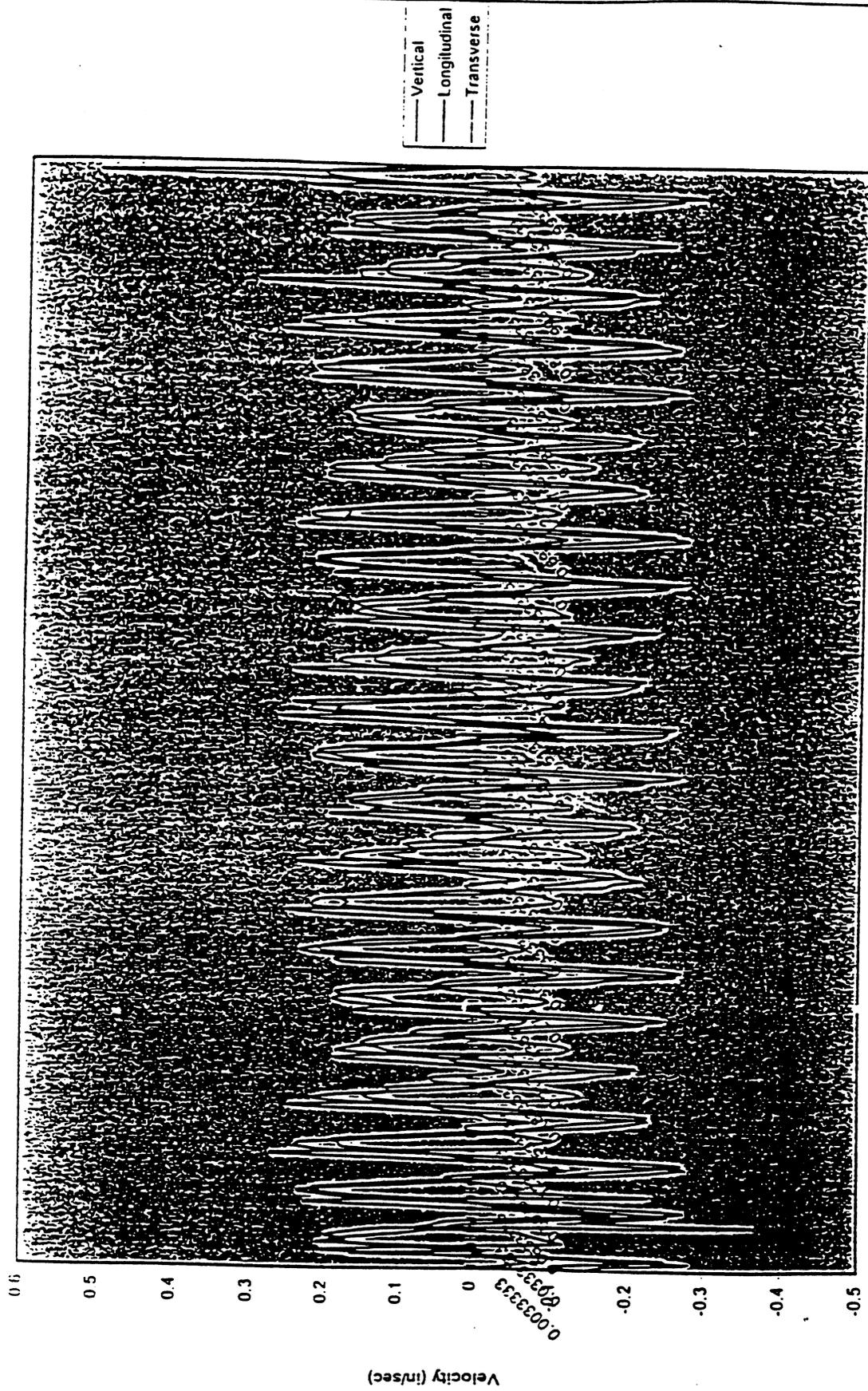
CLEARWATER

**LAW ENGINEERING AND ENVIRONMENTAL SERVICES,
INC.**

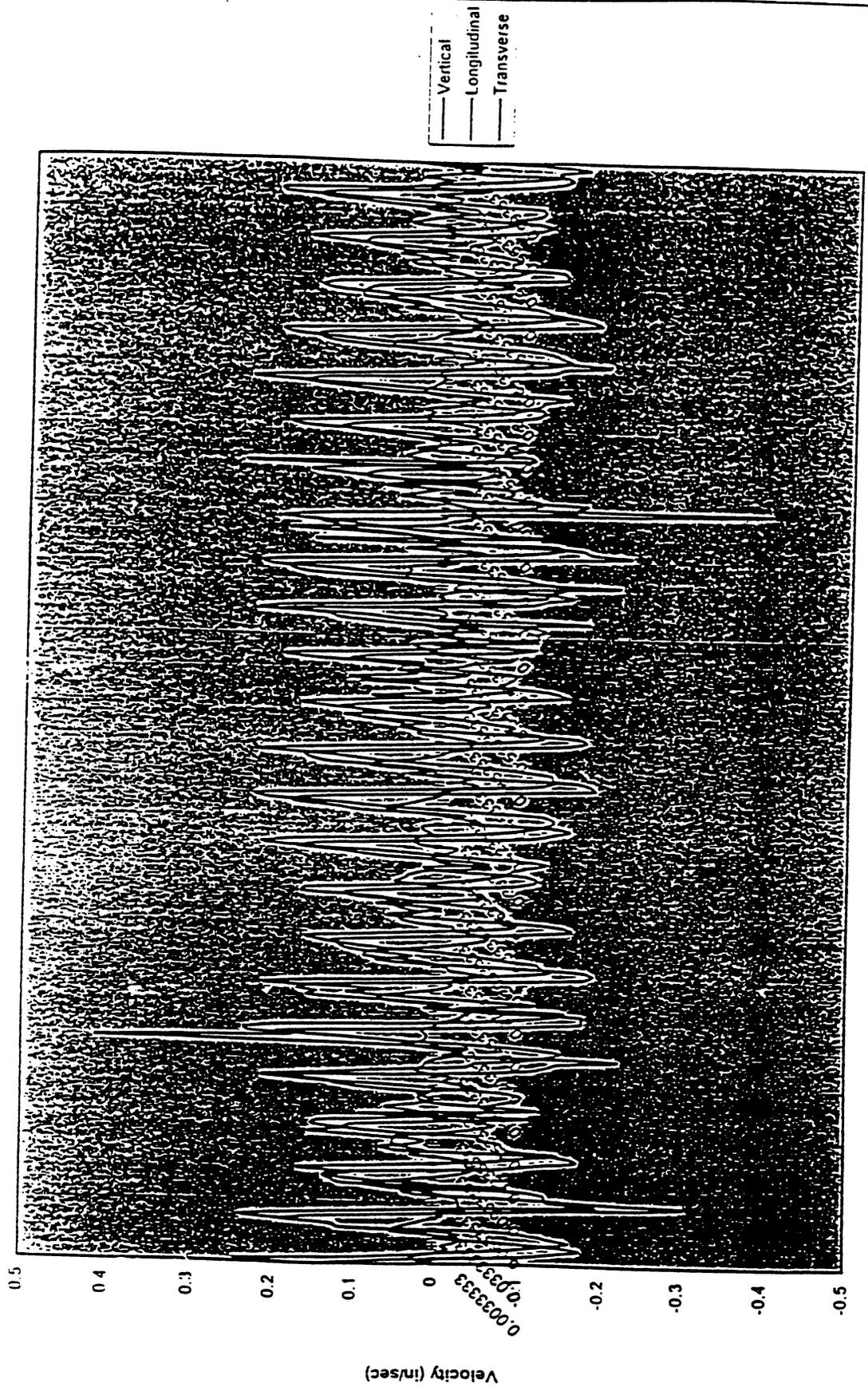
2-Hour Test; Geophone 2.3.1; 0-5 Foot Drive Depth (Interval 9)



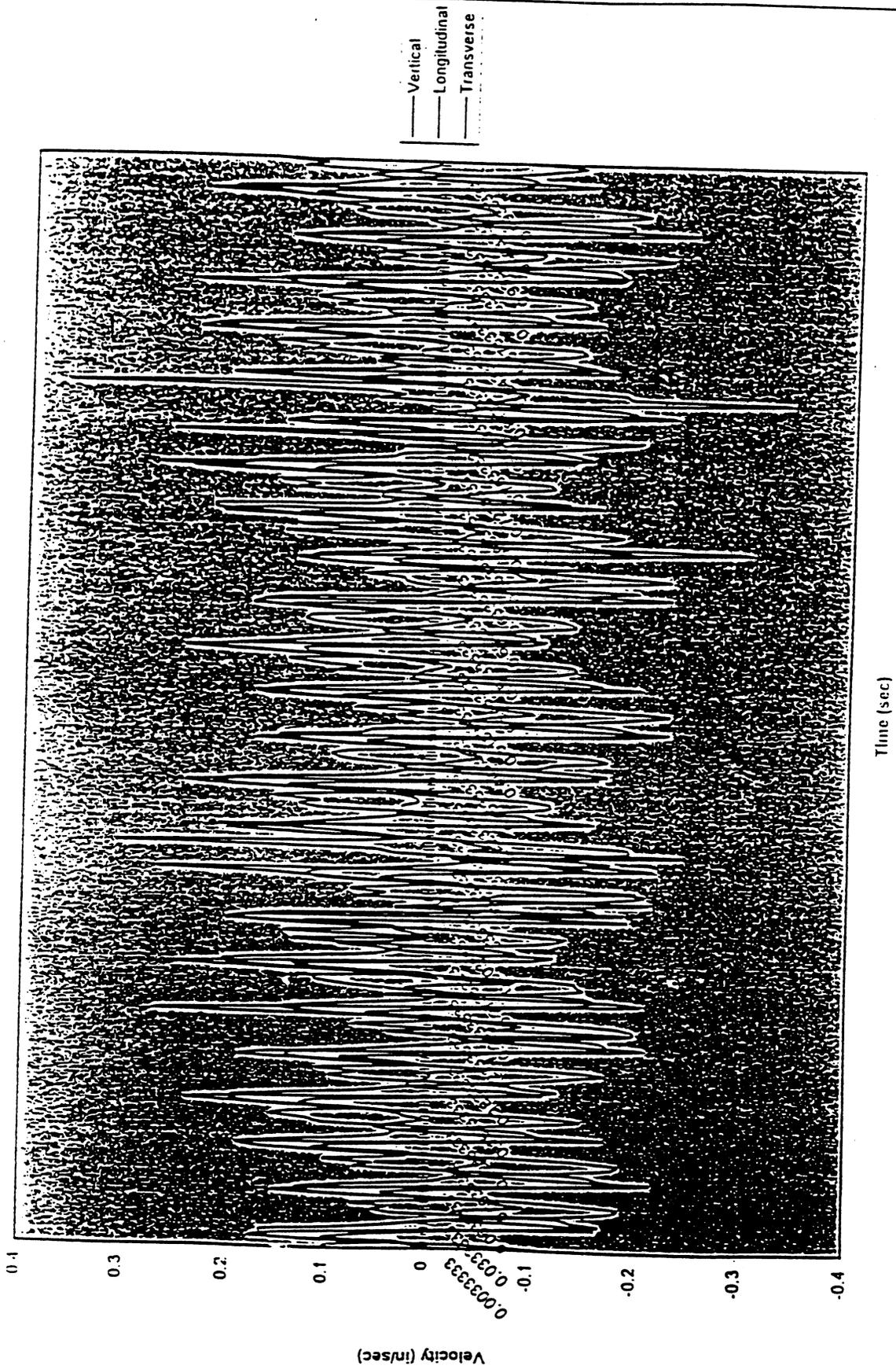
2-Hour Test; Geophone 2.3.4; 10-15 Foot Drive Depth (Interval 11)



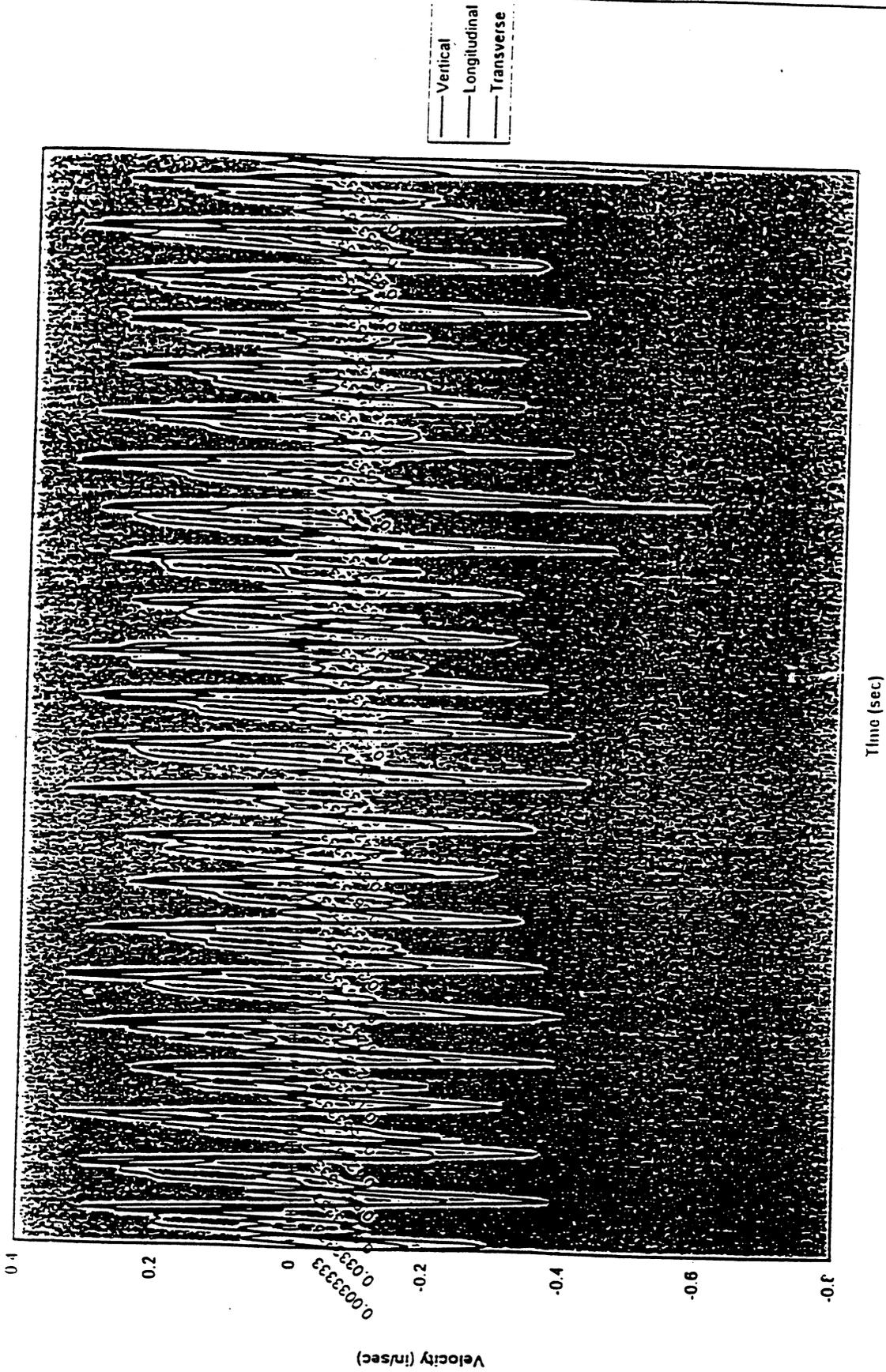
2-Hour Test; Geophone 2.3.1; Shaft Pullout (Interval 6)



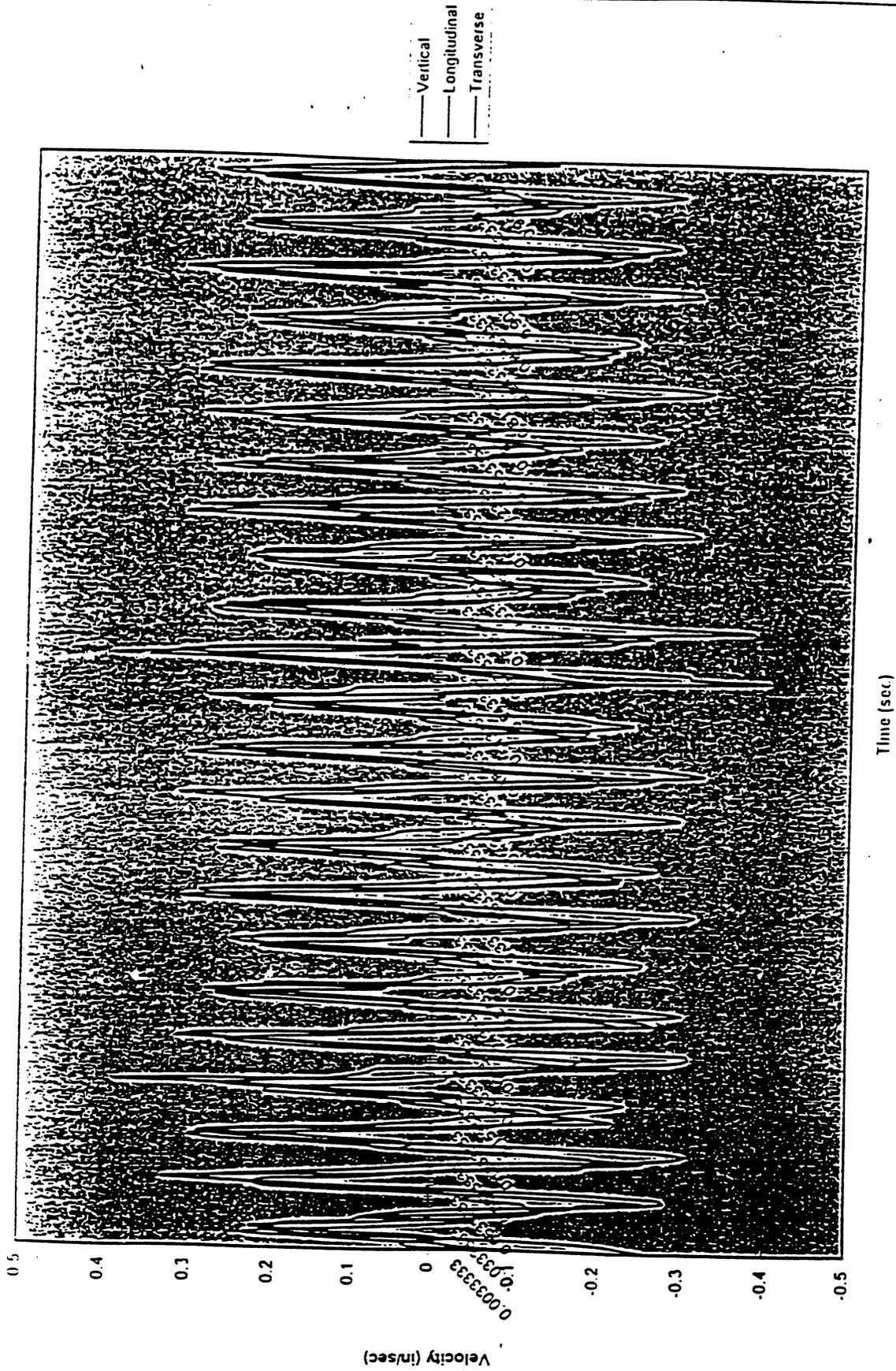
2-Hour Test; Geophone 2.3.2; 0.5 Foot Drive Depth (Interval 10)



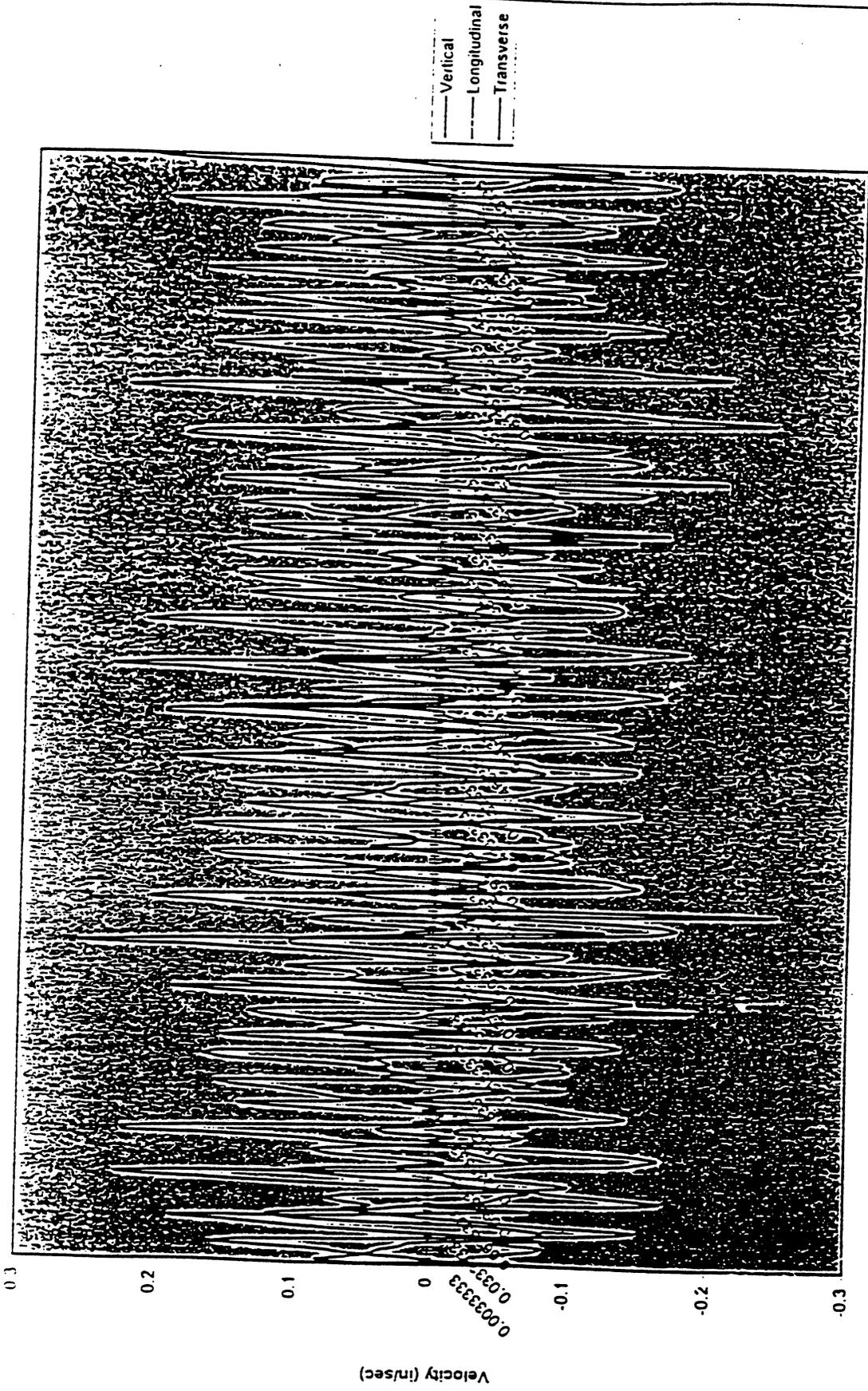
2-Hour Test; Geophone 2.3.2; 5-10 Foot Drive Depth (Interval 5)



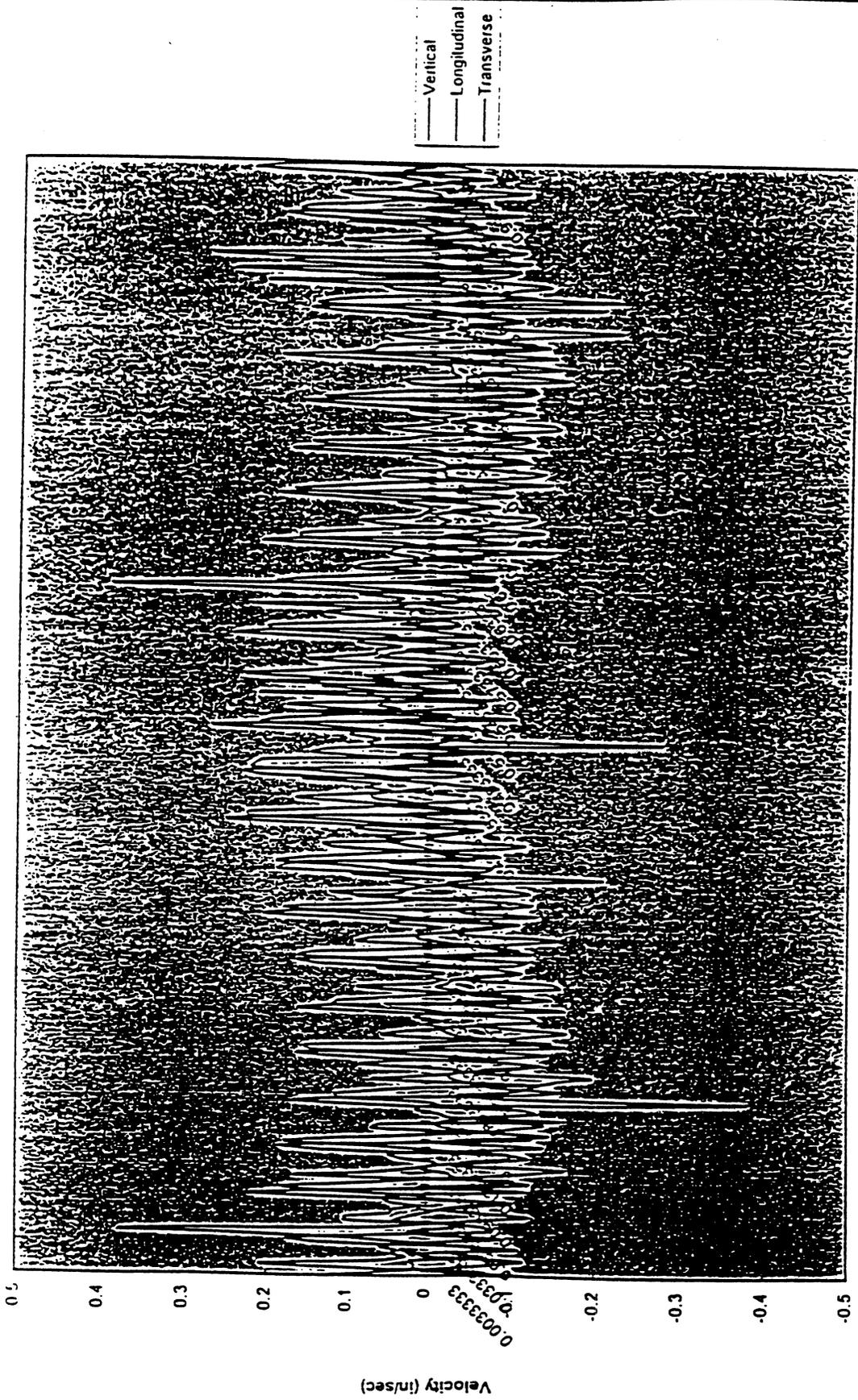
2-1 hour Test; Geophone 2.3.2; 10-15 Foot Drive Depth (Interval 12)



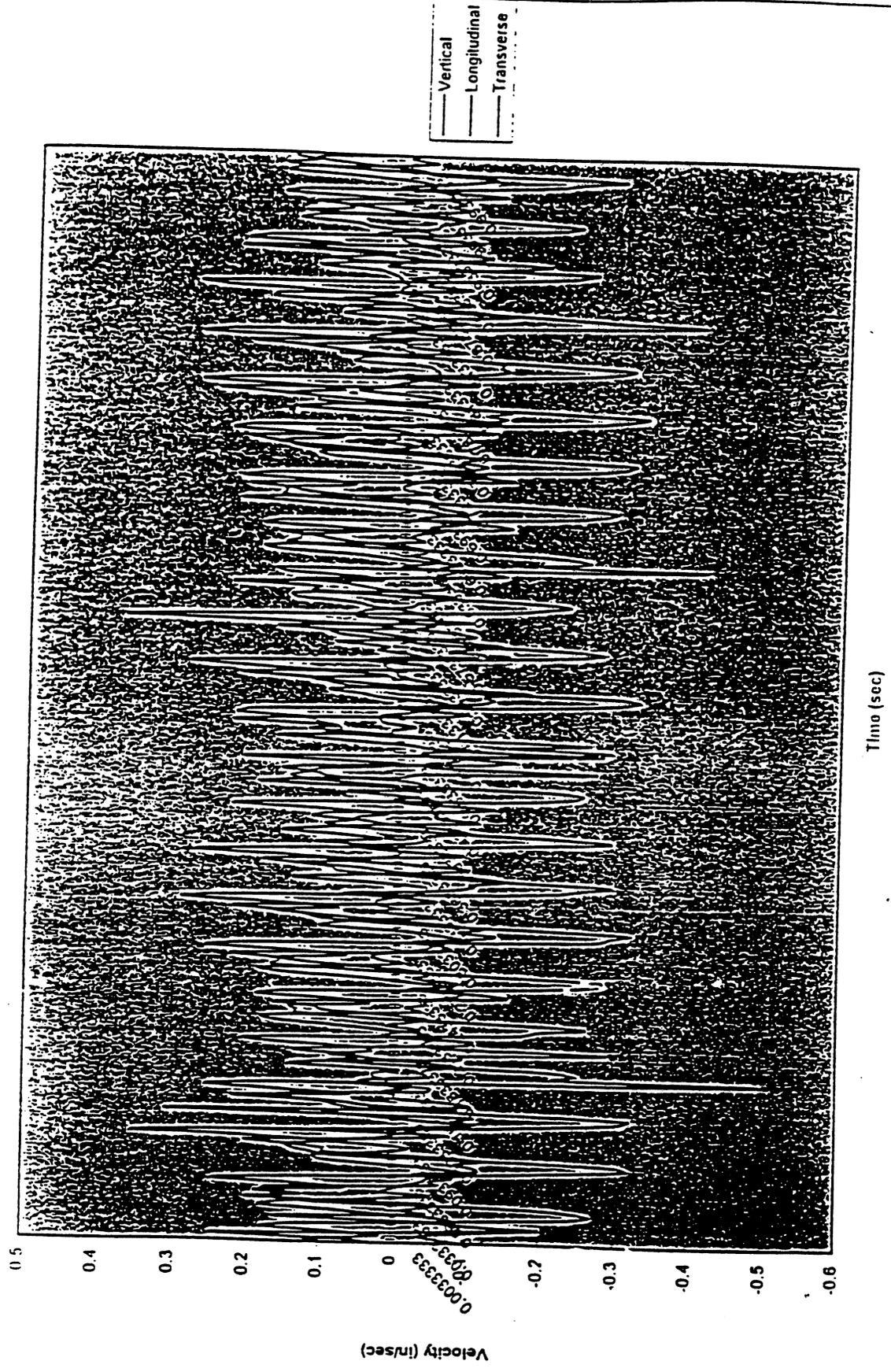
2-Hour Test; Geophone 2.3.2; Shaft Pullout (Interval 7)



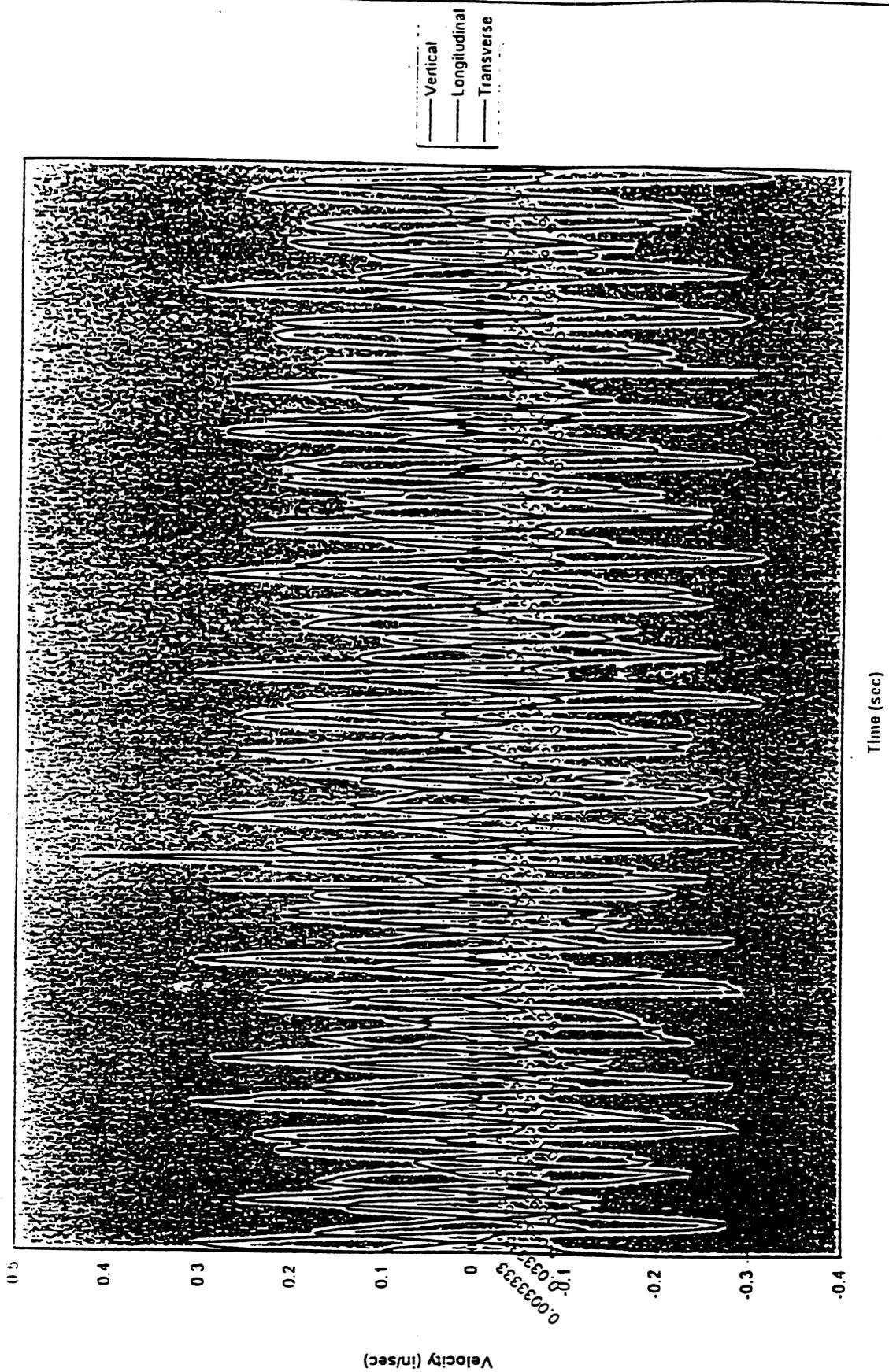
2-Hour Test; Geophone 2.3.3; 6.5 Foot Drive Depth (Interval 5)



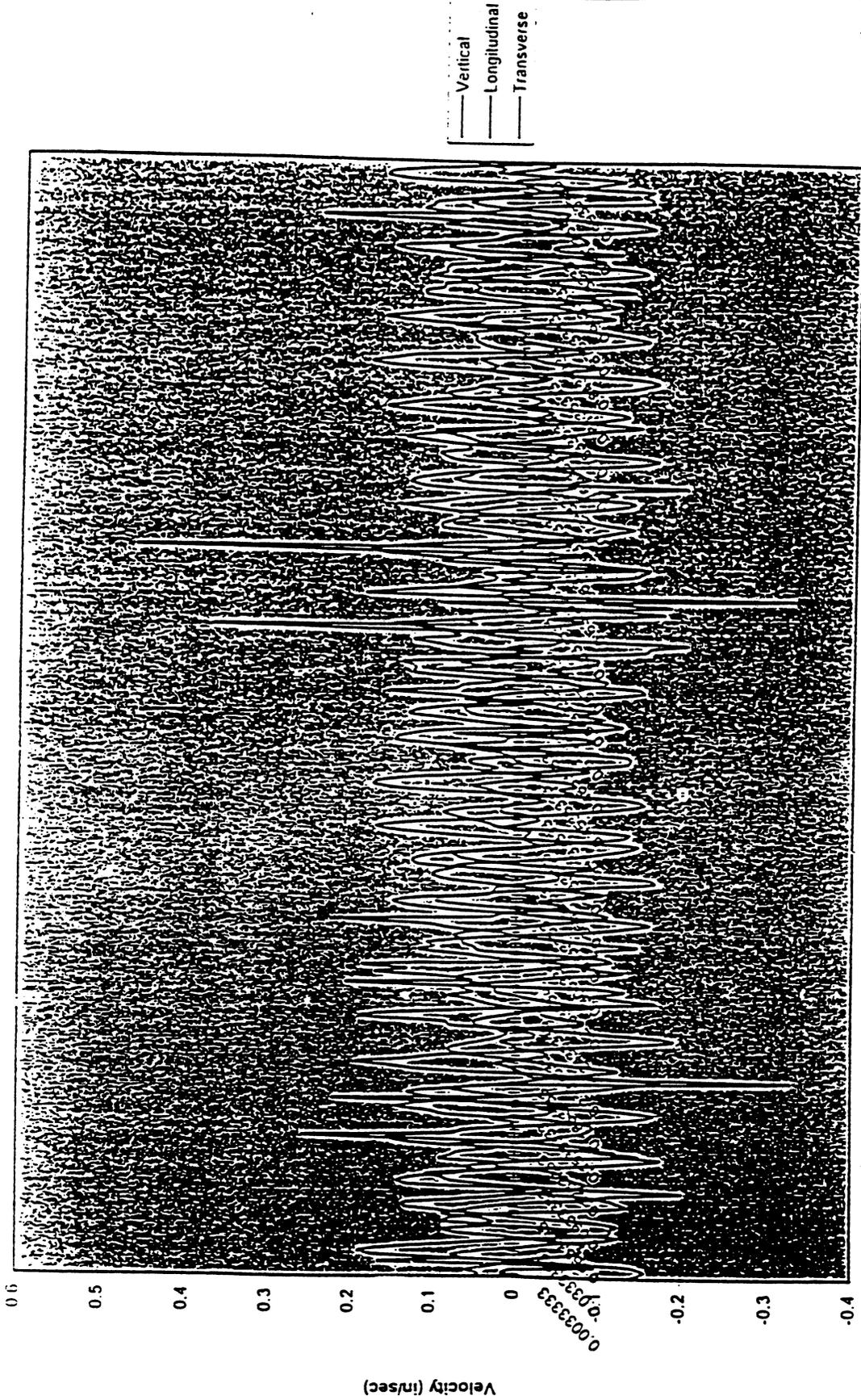
2-Hour Test; Geophone 2.3.3; 5-10 Foot Drive Depth (Interval 5)



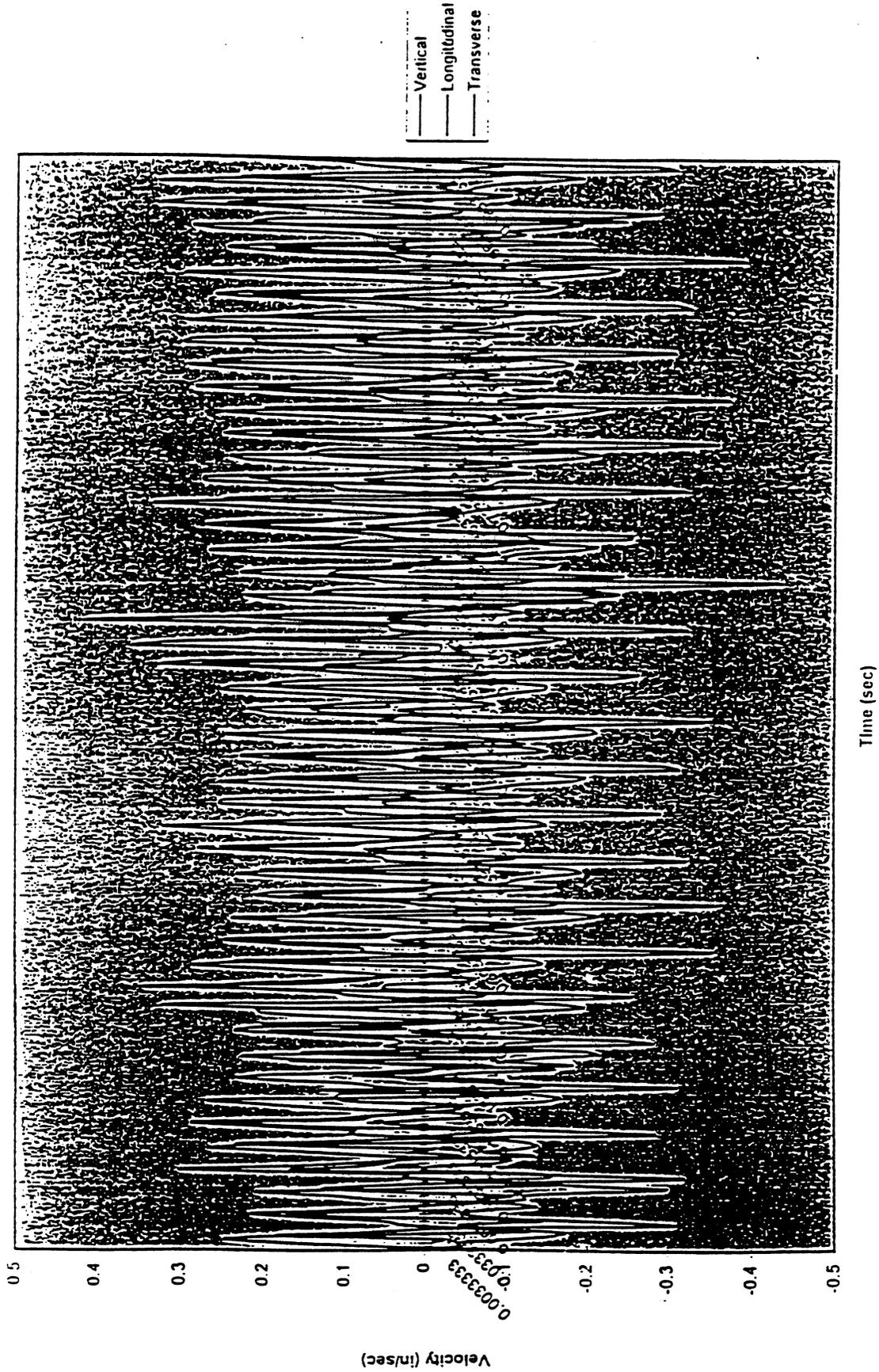
2-Hour Test; Geophone 2.3.3; 10-15 Foot Drive Depth (Interval 11)



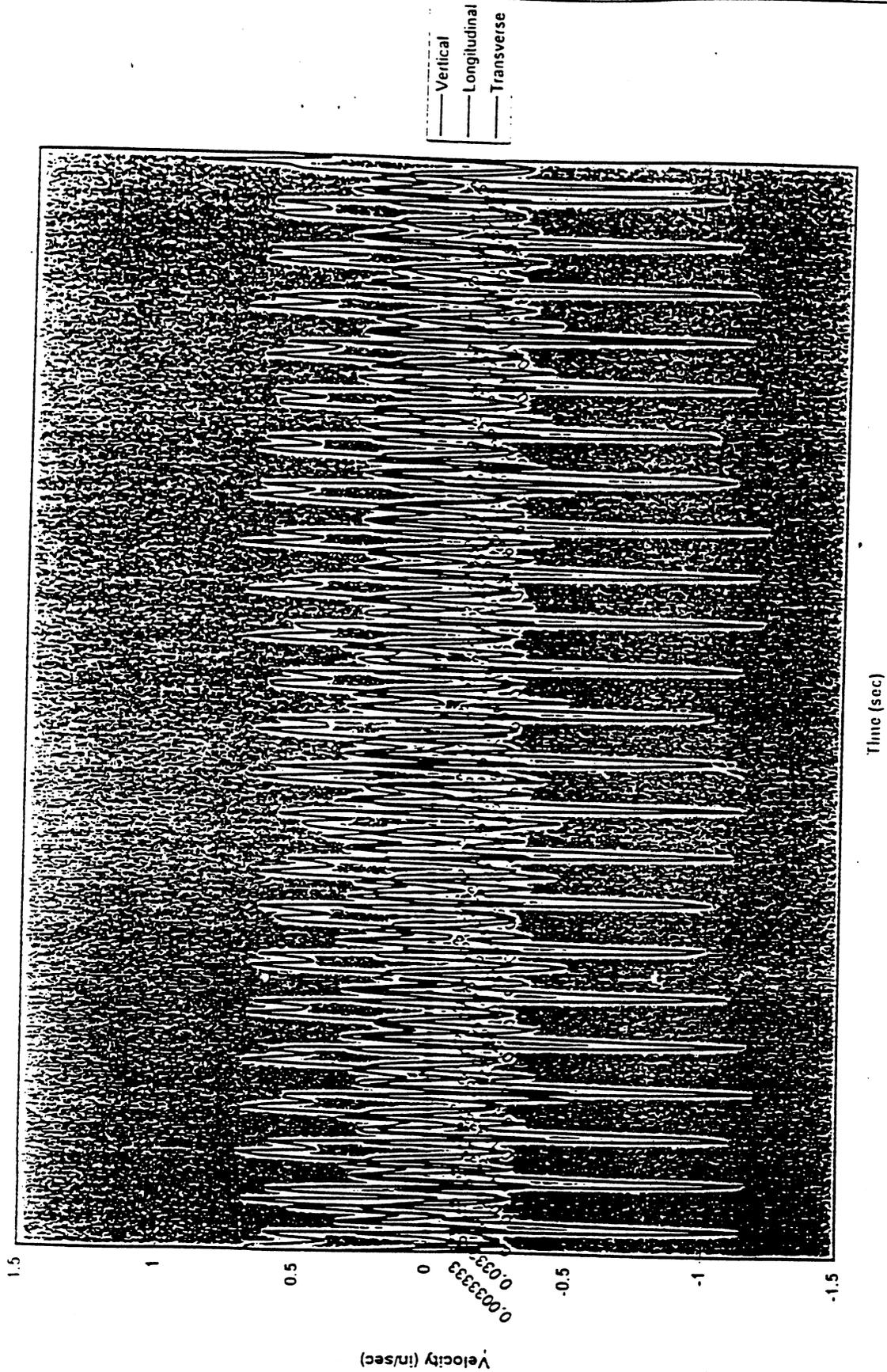
2-Hour Test; Geophone 2.3.3; Shaft Pullout (Interval 7)



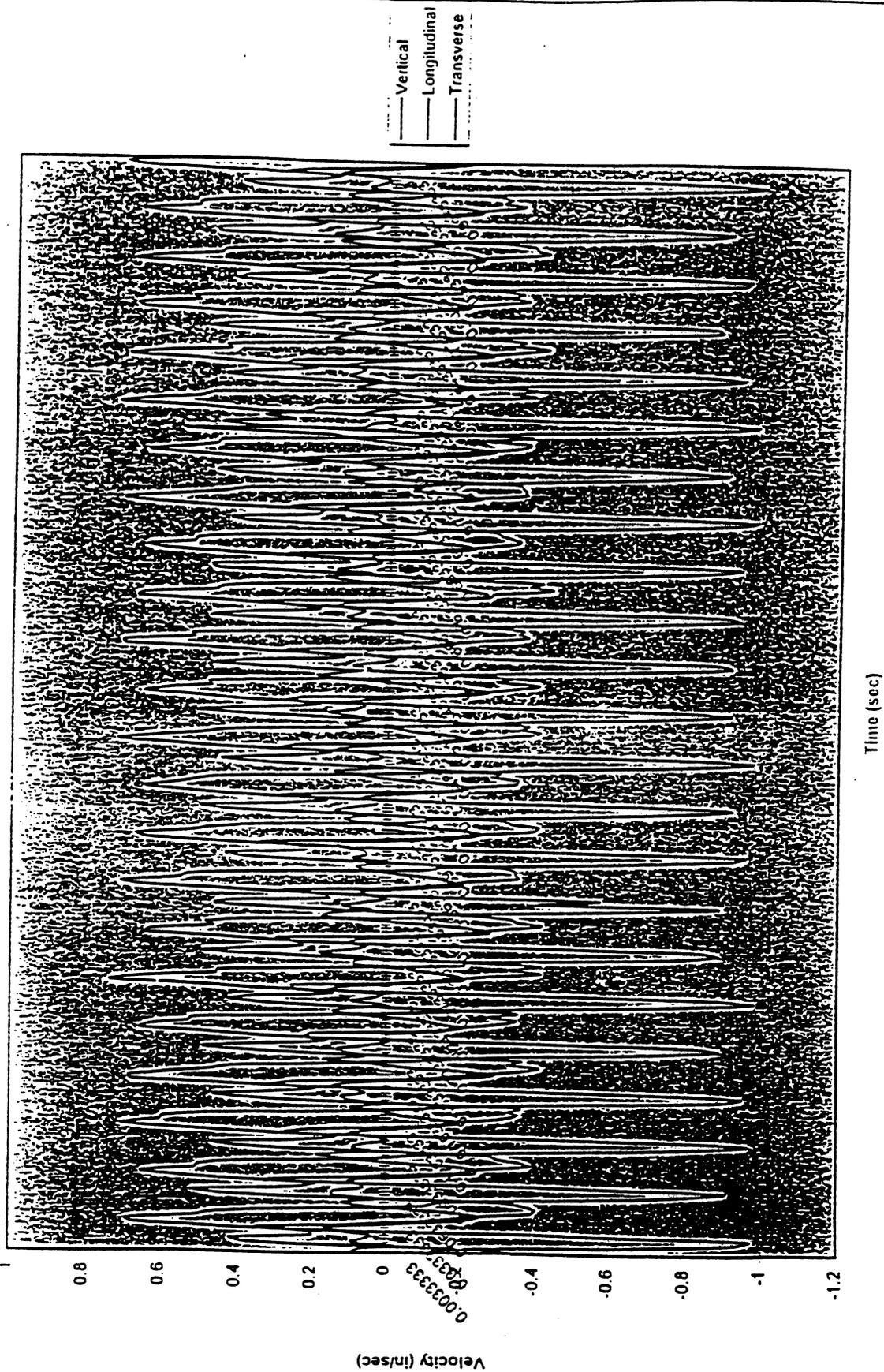
2-Hour Test; Geophone 2.3.1; 0-5 Foot Drive Depth (Interval 9)



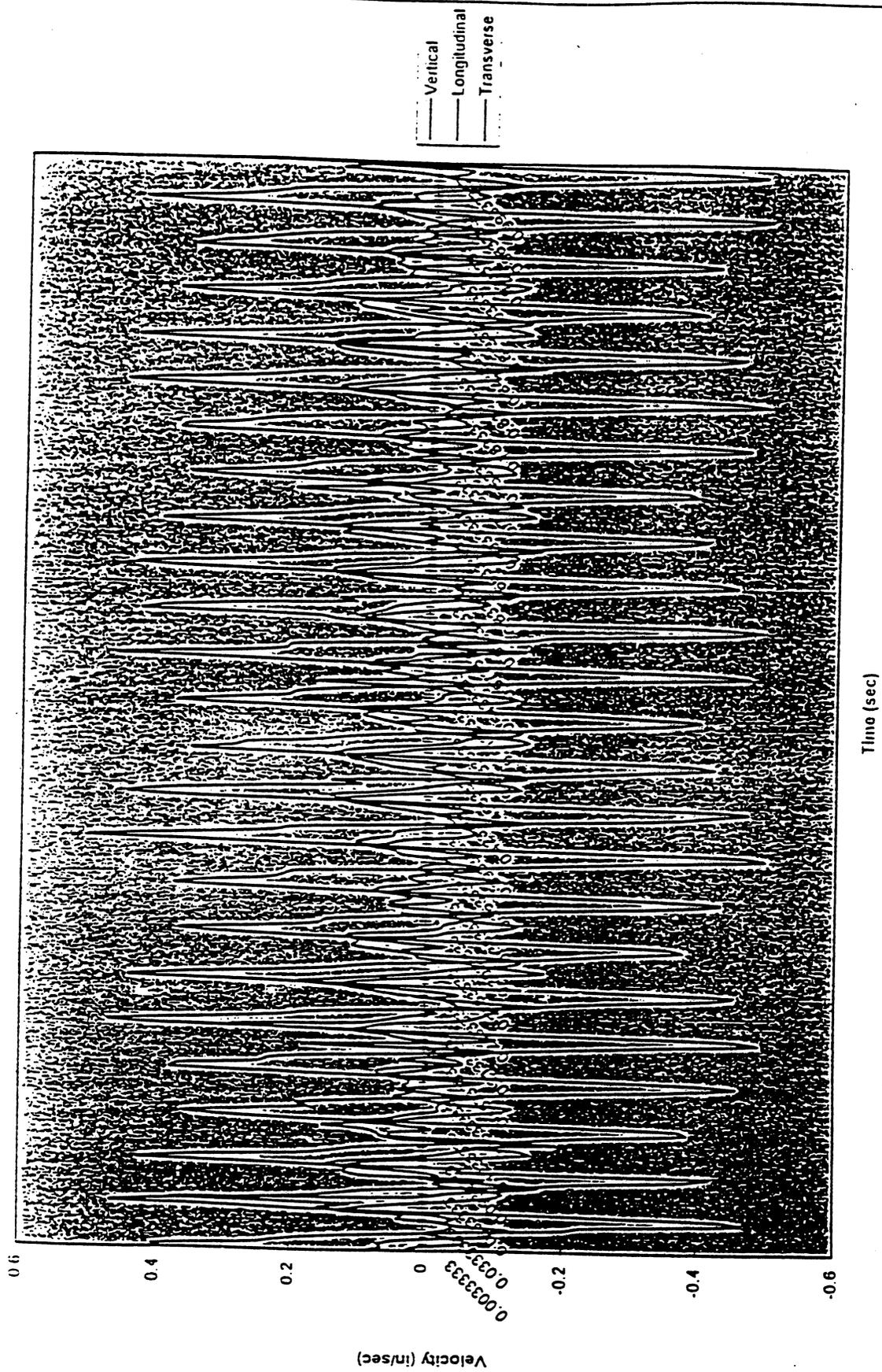
2-Hour Test; Geophone 2.3.1; 5-10 Foot Drive Depth (Interval 9)



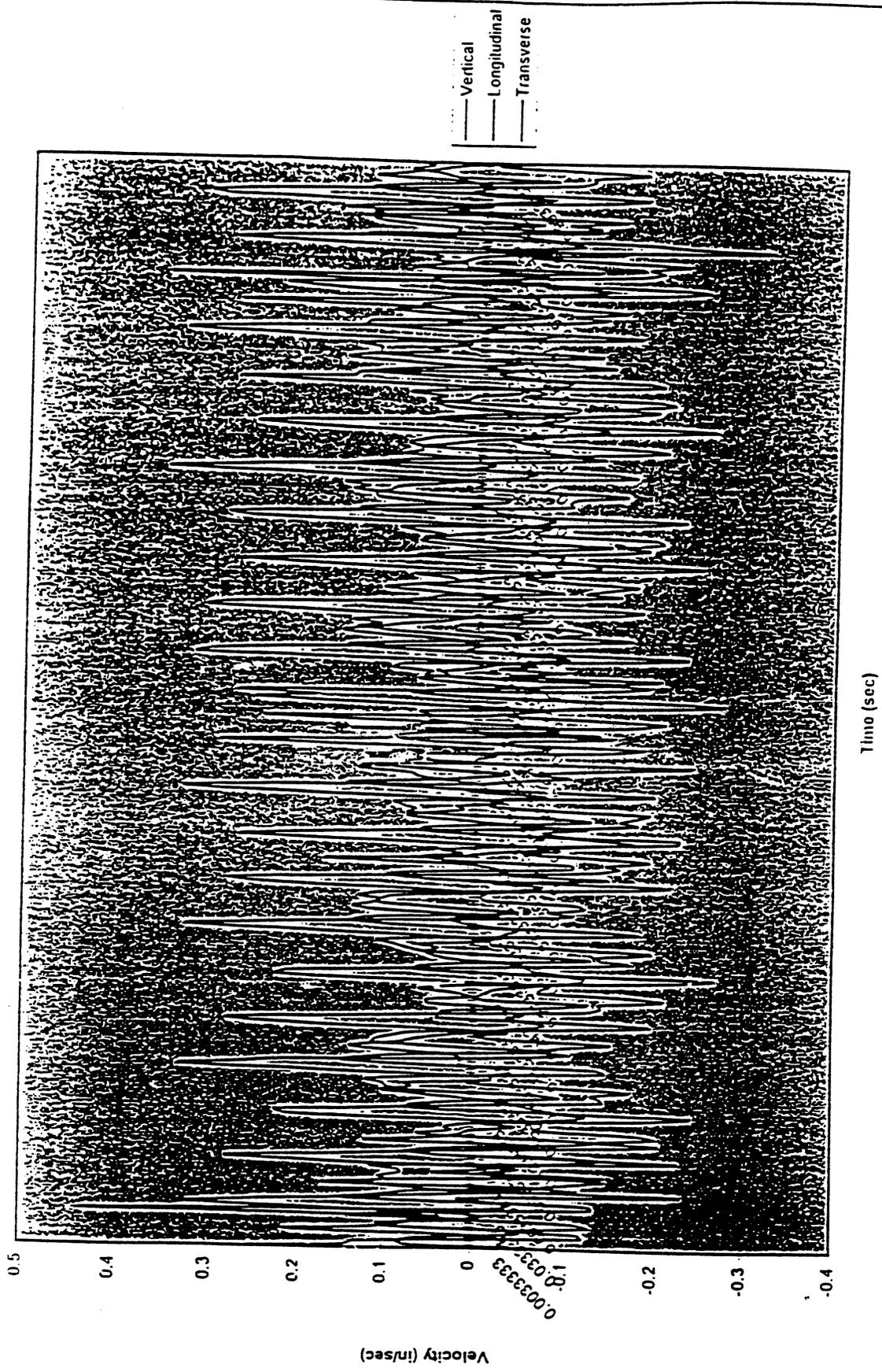
2-Hour Test; Geophone 2.3.1; 10-15 Foot Drive Depth (Interval 11)



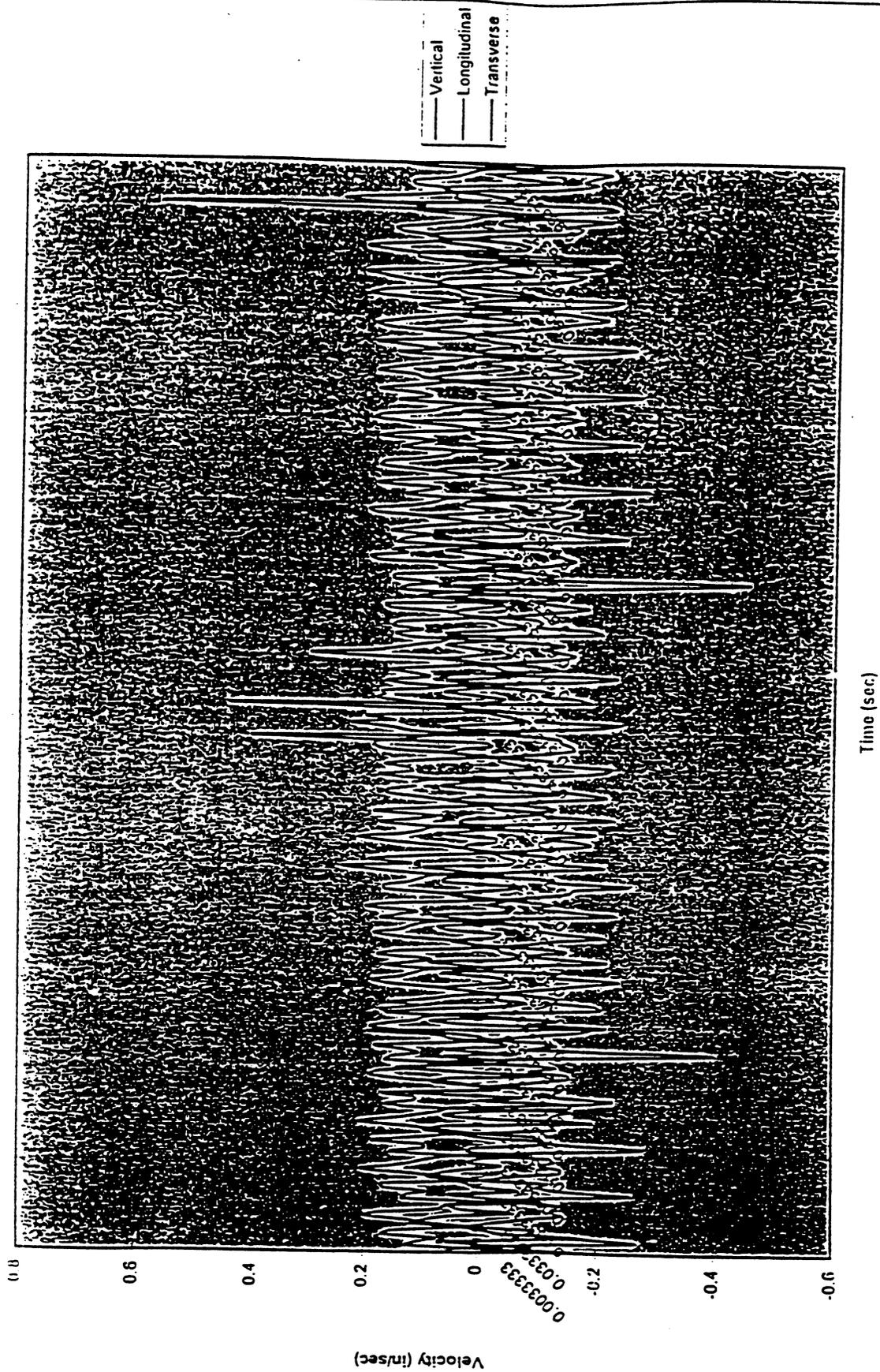
2-Hour Test; Geophone 2.3.1; Shaft Pullout (Interval 7)



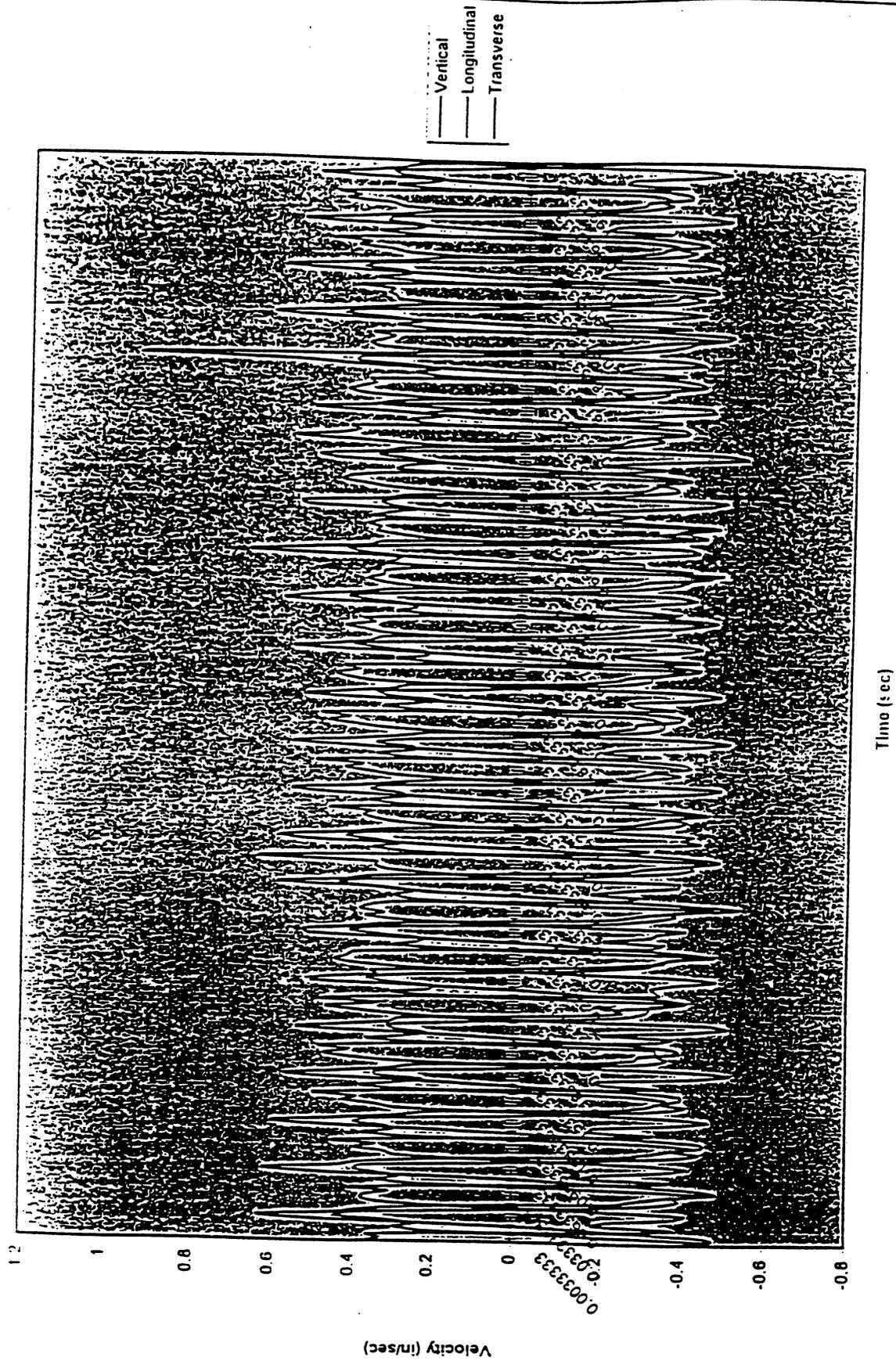
2-Hour Test; Geophone 2.3.2; 0.5 Foot Drive Depth (Interval 9)



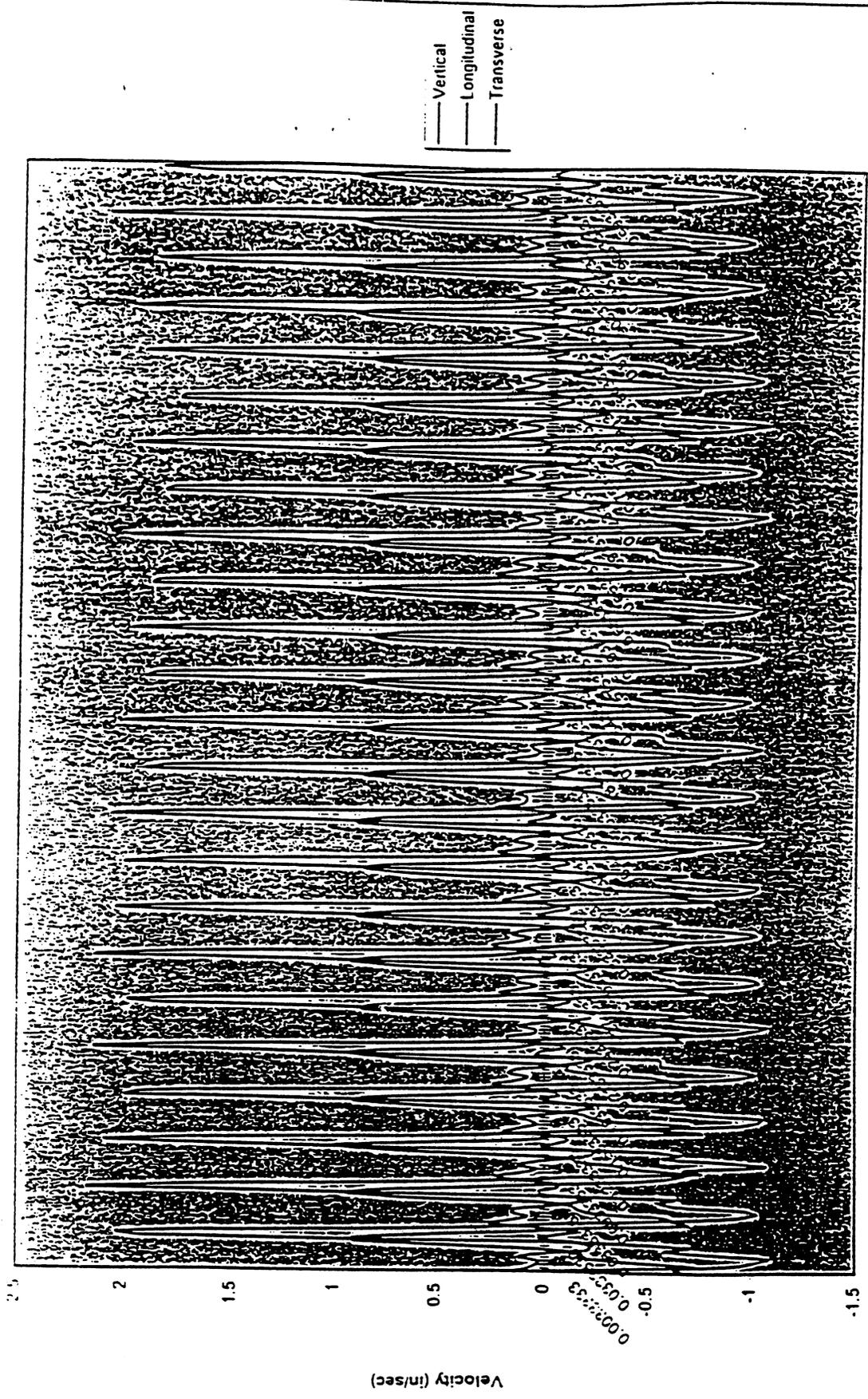
2-Hour Test; Geophone 2.3.2; 5-10 Foot Drive Depth (Interval 9)



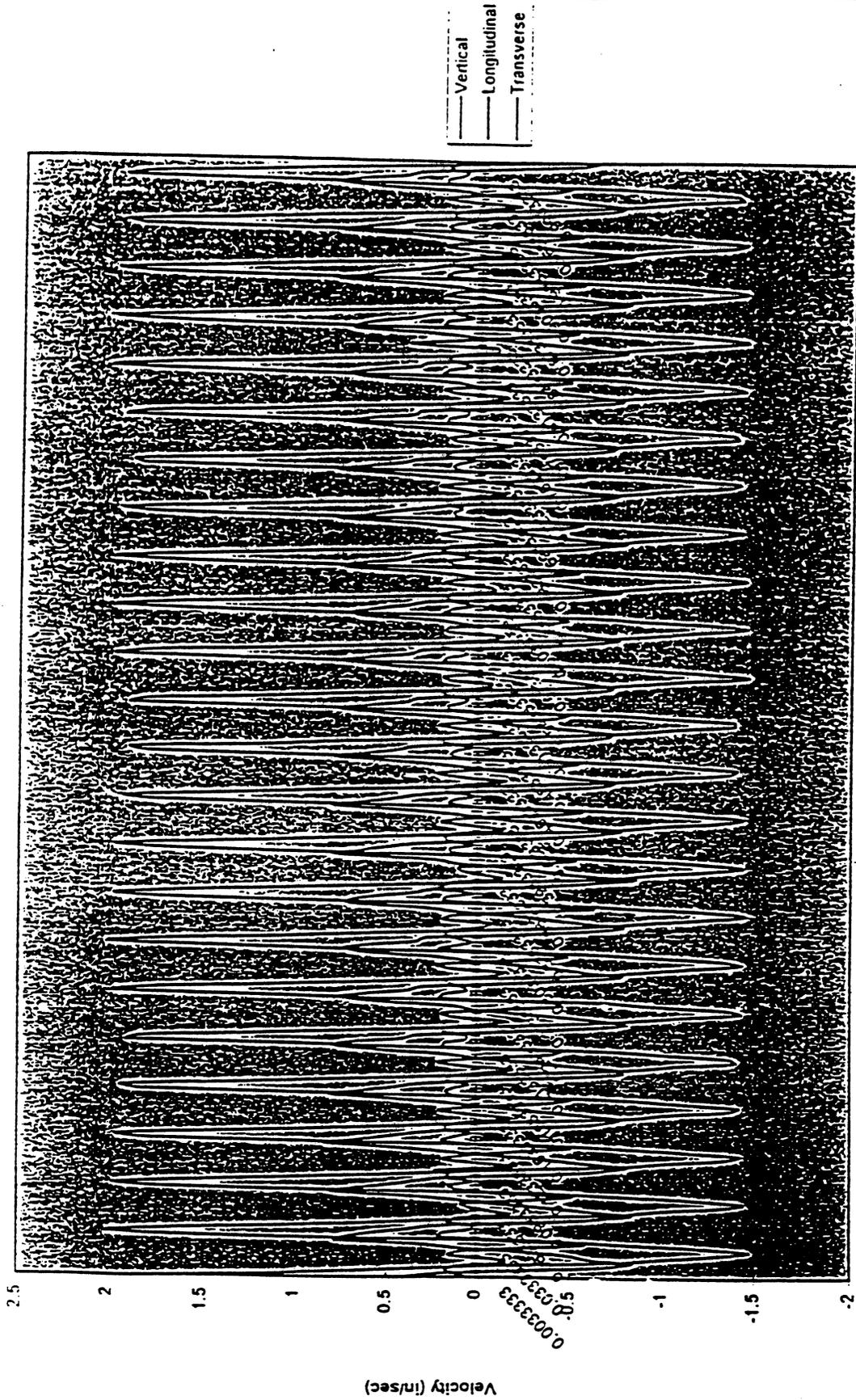
2-Hour Test; Geophone 3.3.2; 10-15 Foot Drive Depth (Interval 10)



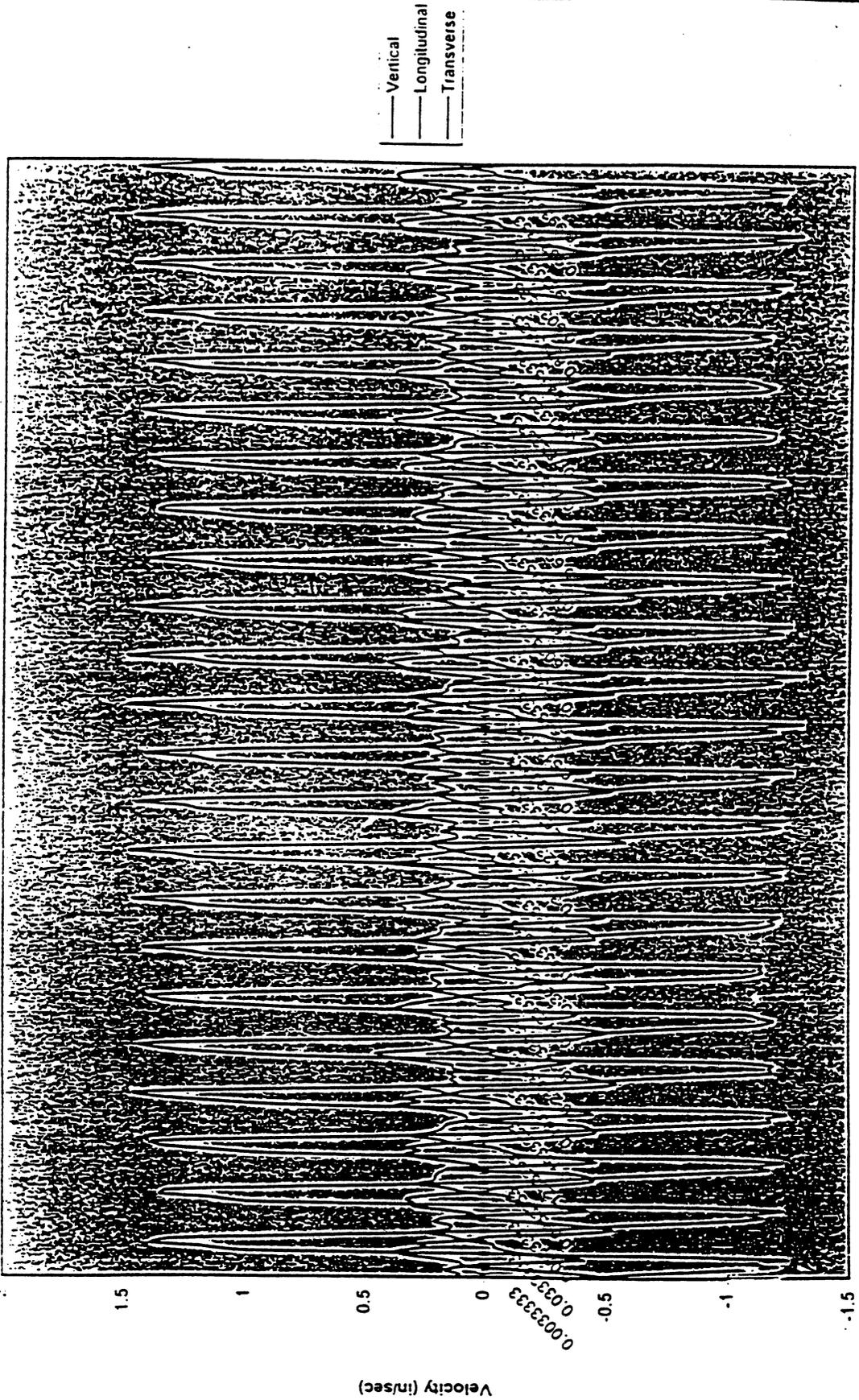
2-Hour Test; Geophone S1; 0.5 Foot Drive Depth (Interval 10)



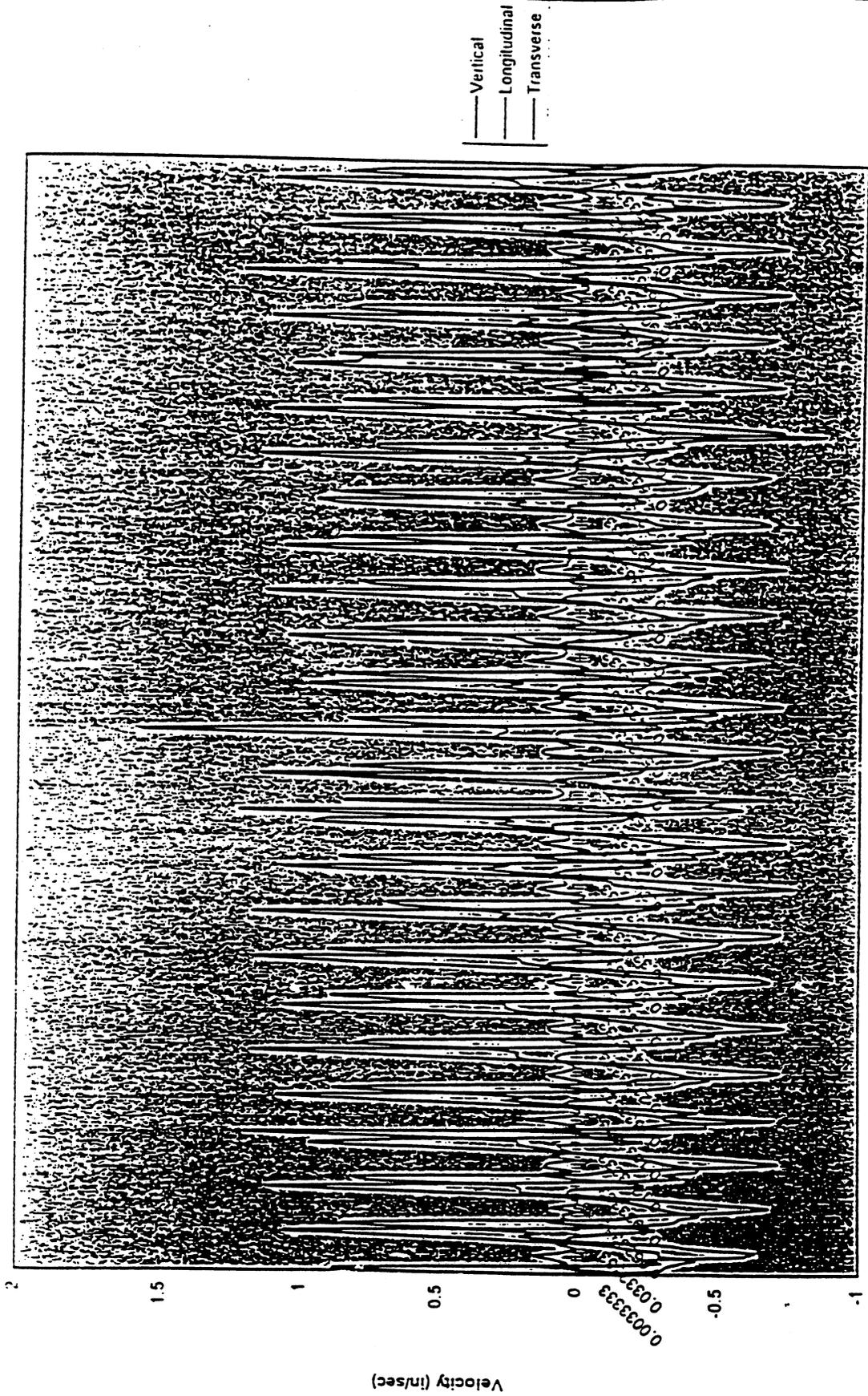
2-1 hour Test; Geophone S1; 5-10 Foot Drive Depth (Interval 9)



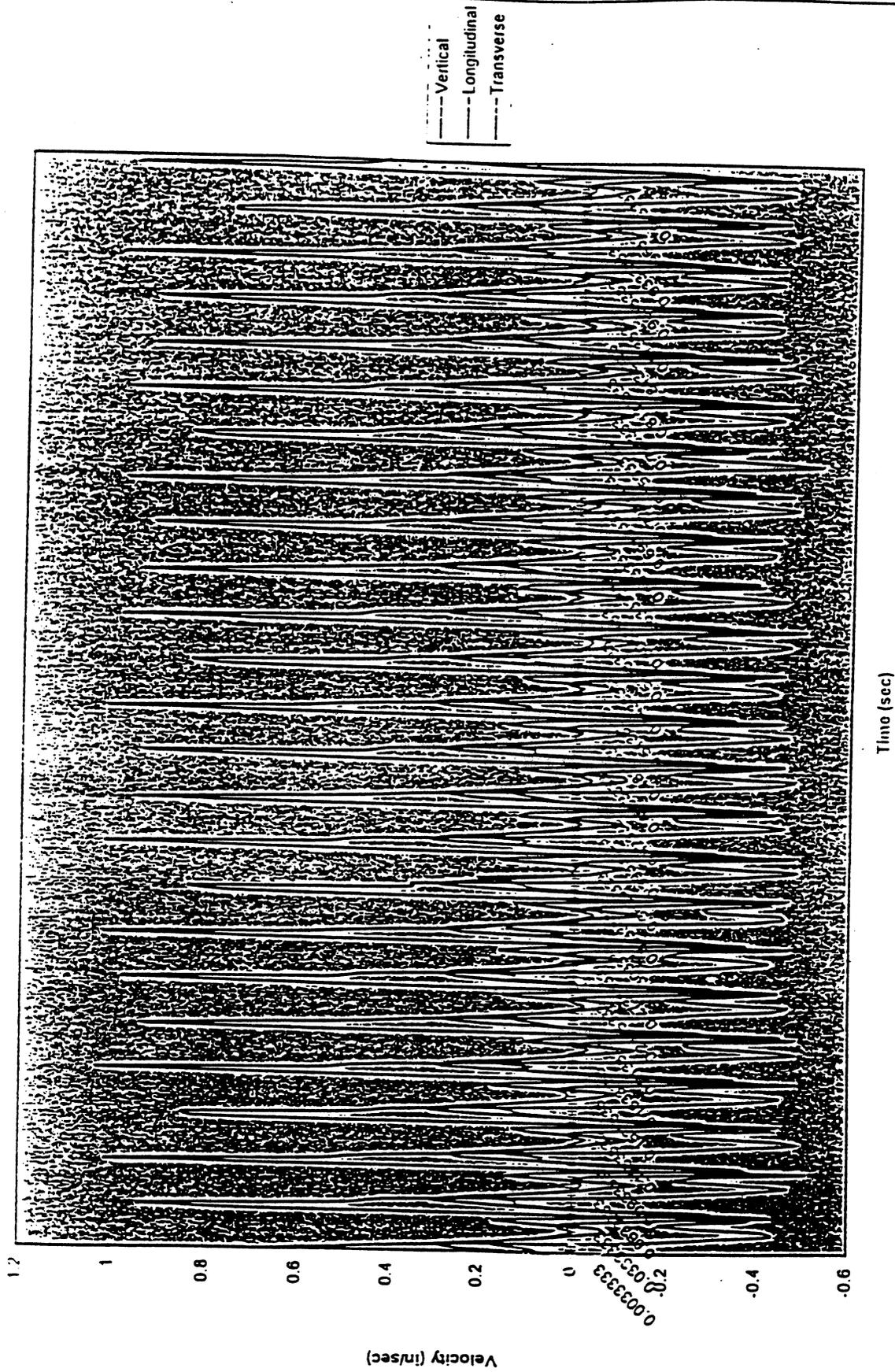
2-Hour Test; Geophone S1; 10-15 Foot Drive Depth (Interval 10)



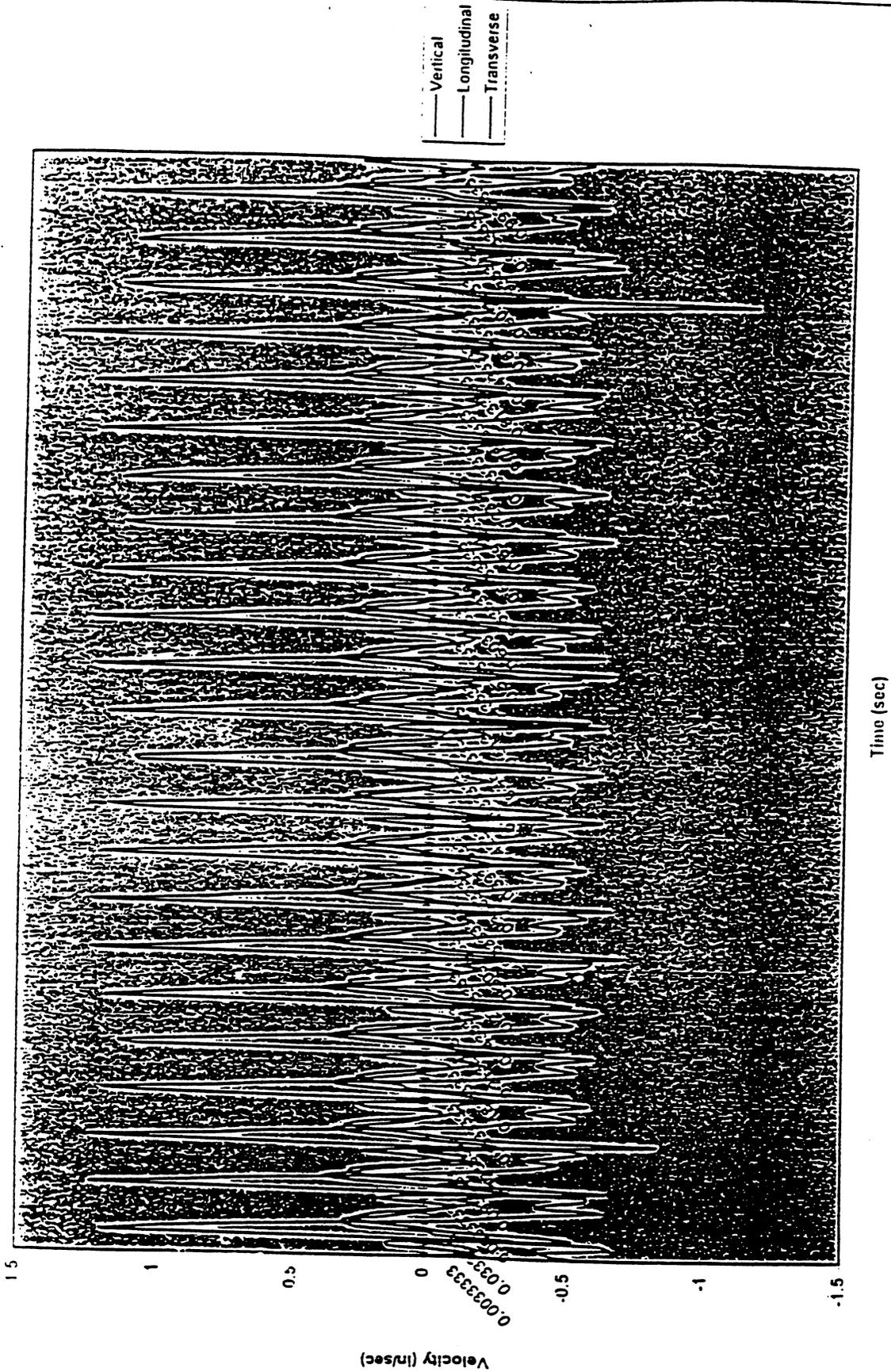
2-Hour Test; Geophone S*; Shaft Pullout (Interval 10)



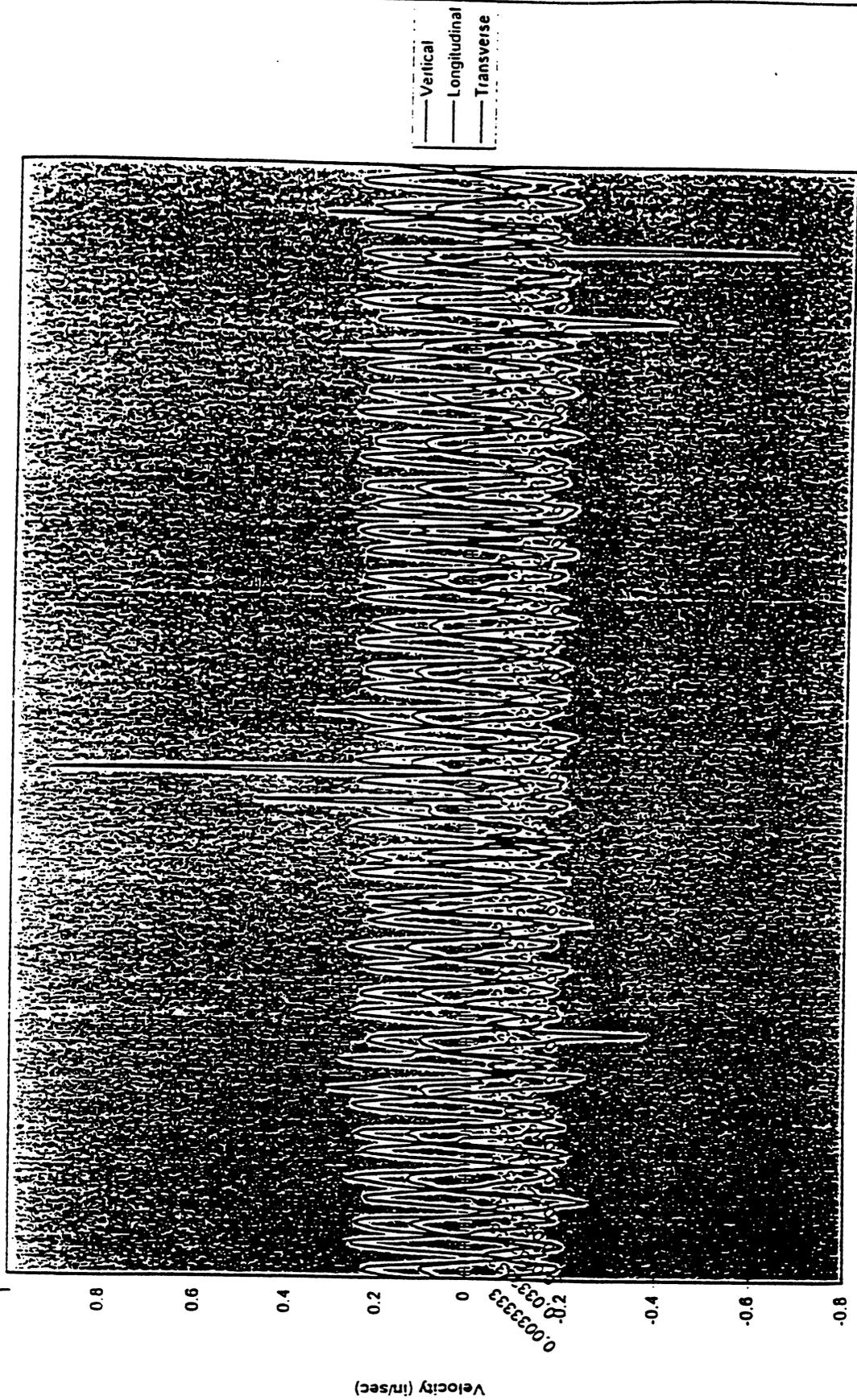
2-Hour Test; Geophone #2; 0-5 Foot Drive Depth (Interval 10)



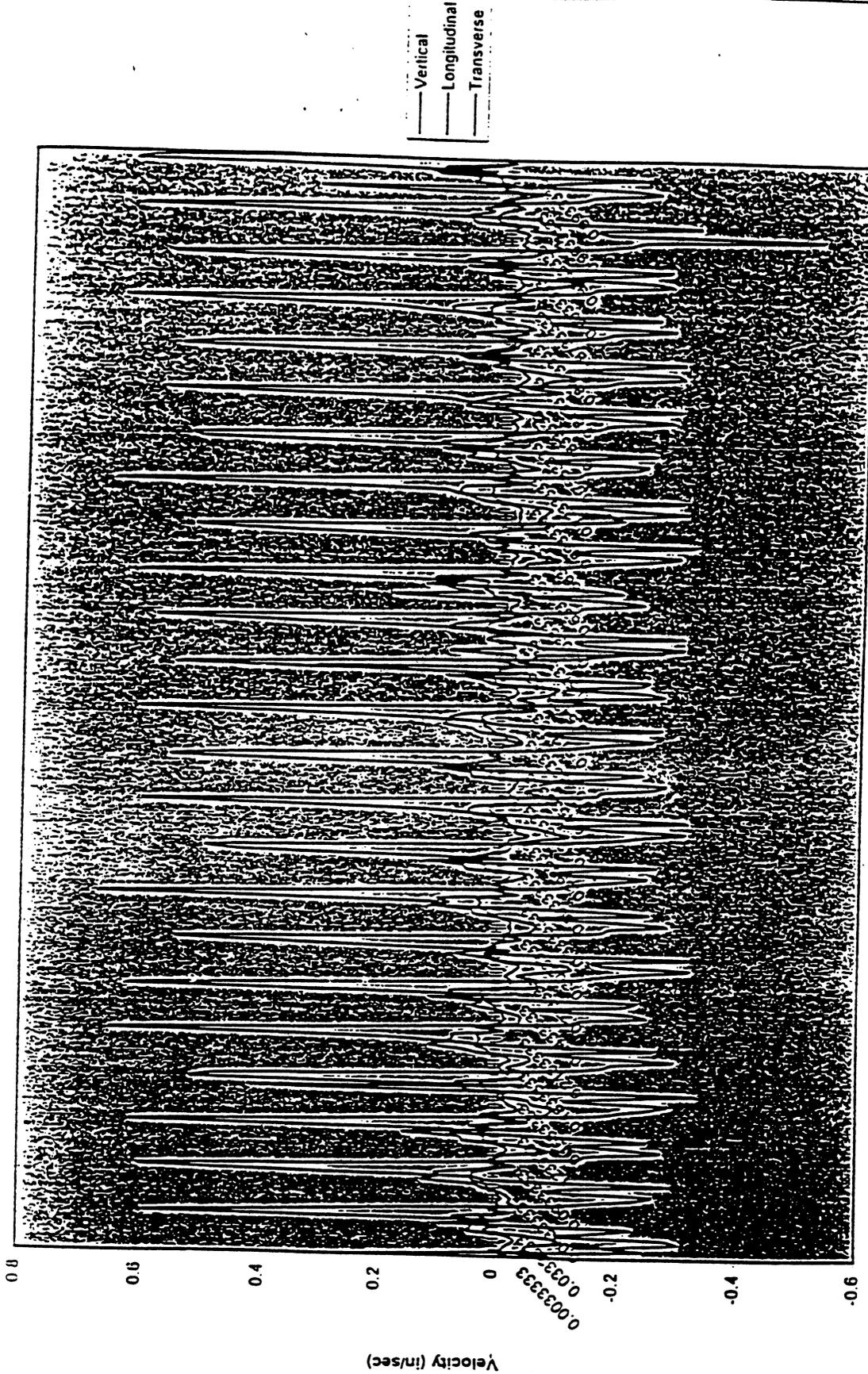
2-Hour Test; Geophone S2; 5-10 Foot Drive Depth (Interval 8)



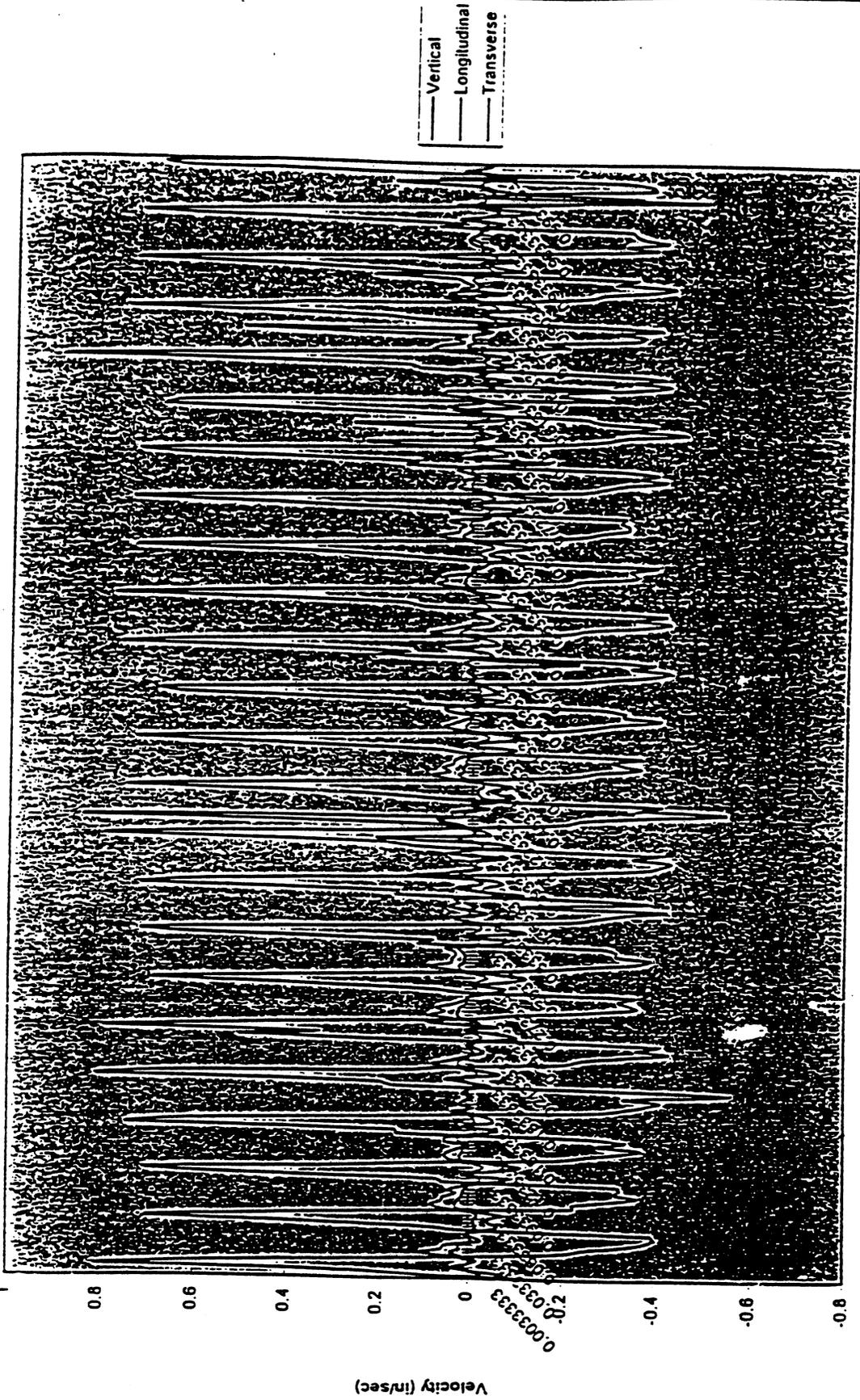
2-Hour Test; Geophone S2; Shaft Pullout (Interval 8)



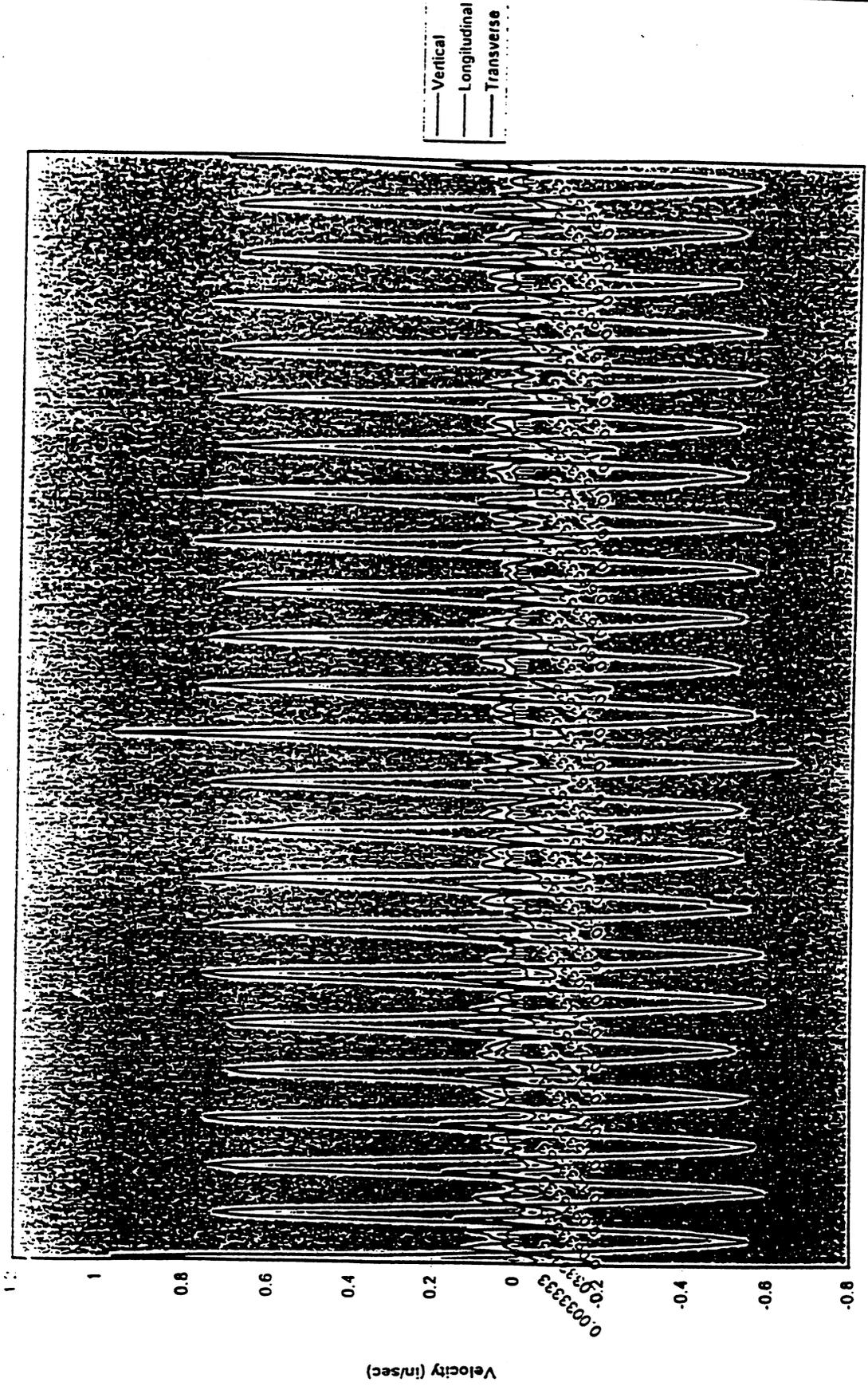
2-Hour Test; Geophone S3; 0-5 Foot Drive Depth (Interval 10)



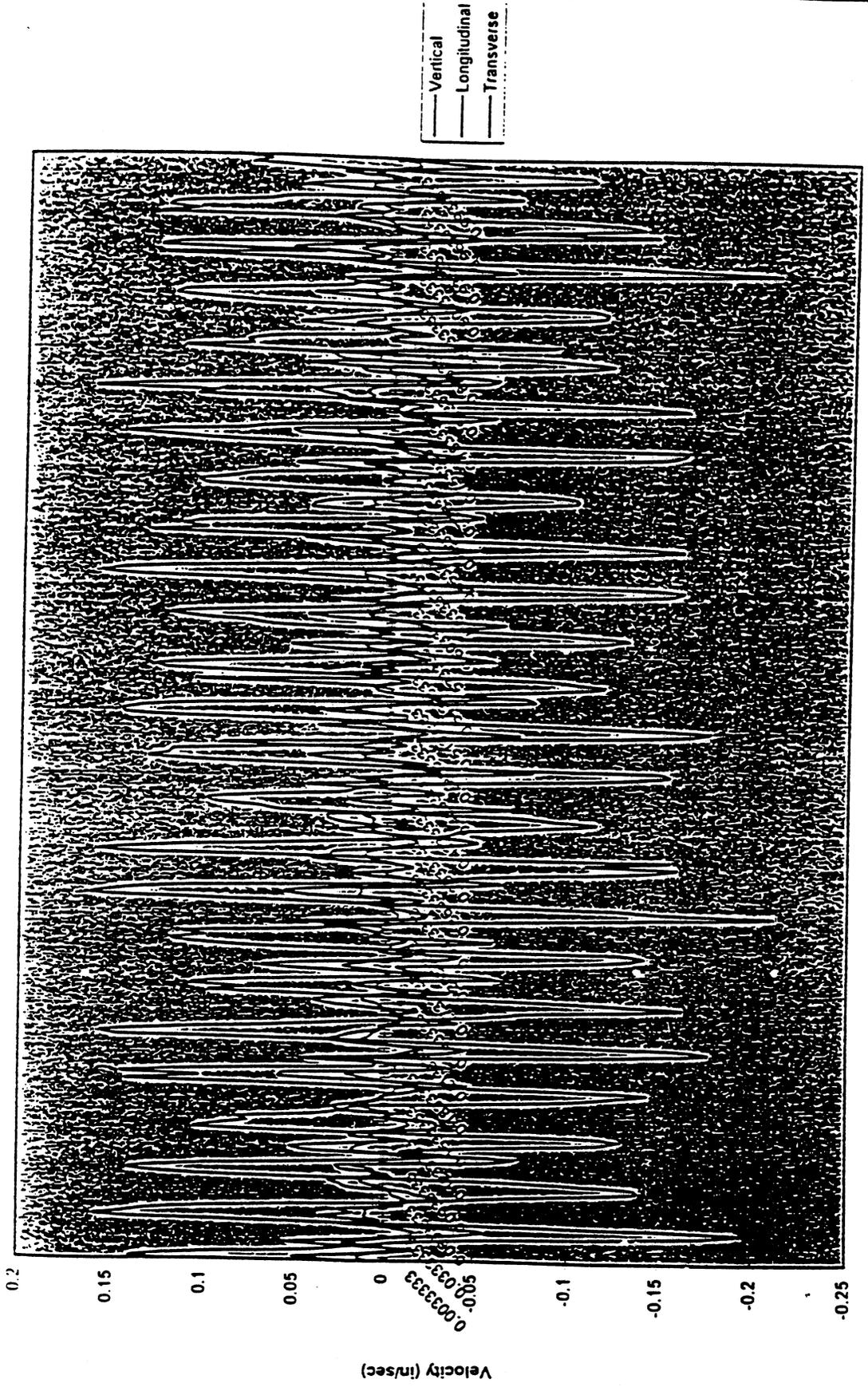
2-Hour Test; Geophone S3; 5-10 Foot Drive Depth (Interval 8)



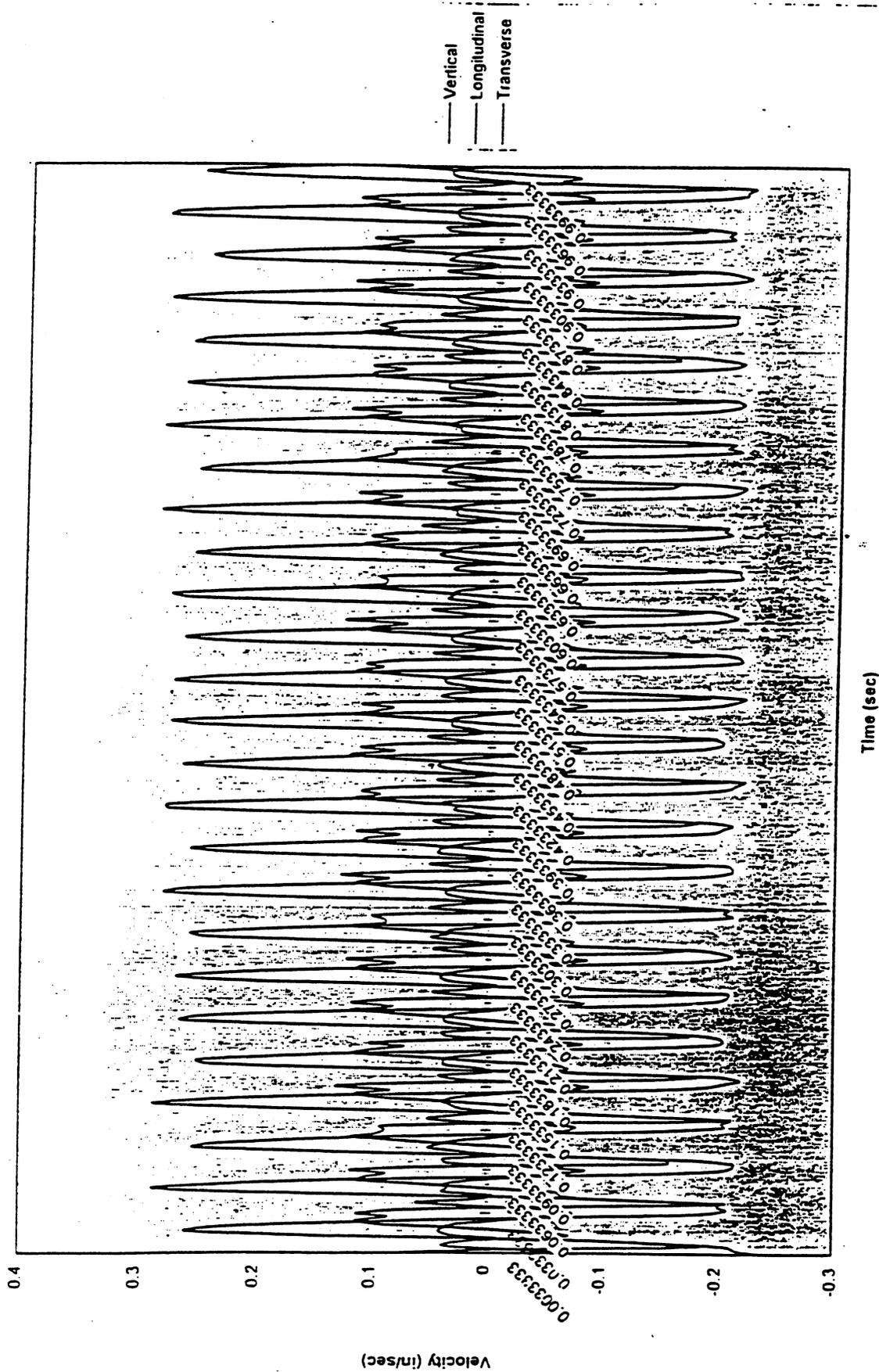
2-1/2 hour Test; Geophone S3; 10-15 Foot Drive Depth (Interval 10)



2-Hour Test; Geophone S3; Shaft Pullout (Interval 7)



4-Hour Test; Geophone 3.3.2; 5-10 Foot Drive Depth (Interval 7)



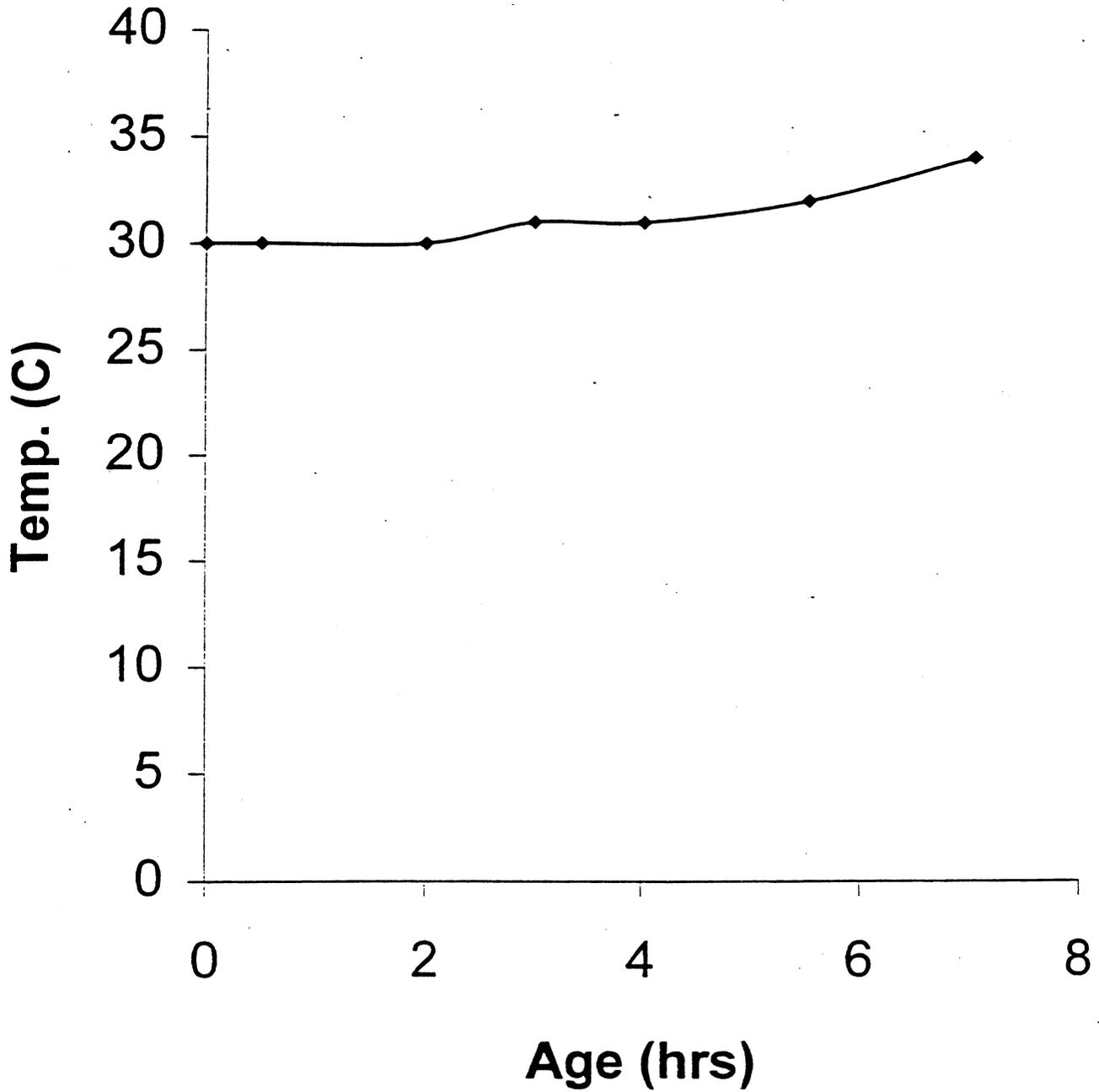
Appendix D

Concrete Test Results – Control Samples

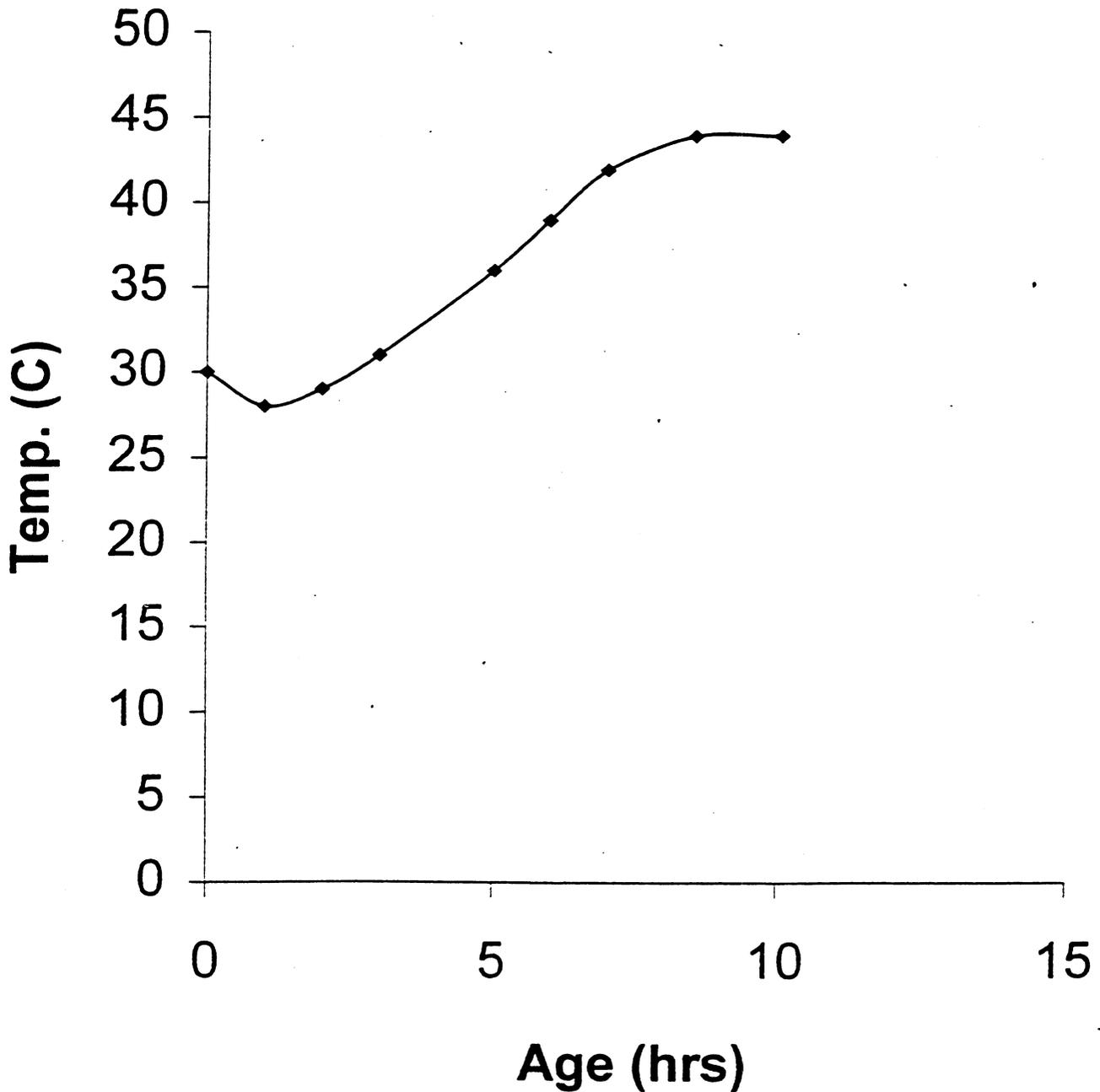
Law Engineering and Environmental
Services, Inc.

April 1998.

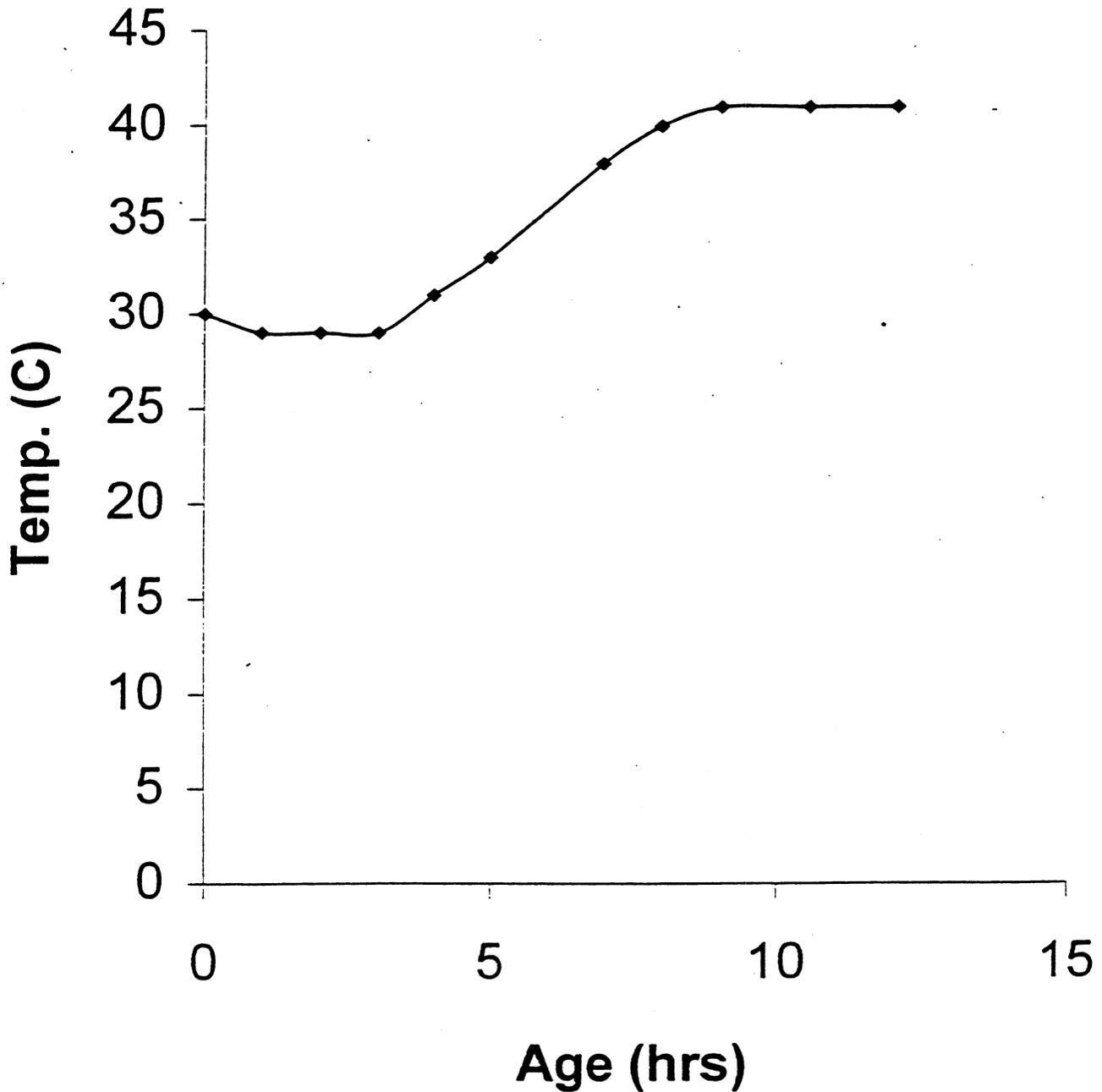
36-Inch Shaft Concrete Temp. vs. Age (Shaft Group 1)



36-Inch Shaft Concrete Temp. vs. Age (Shaft Group 3)



36-Inch Shaft Concrete Temp. vs. Age (Shaft Group 4)



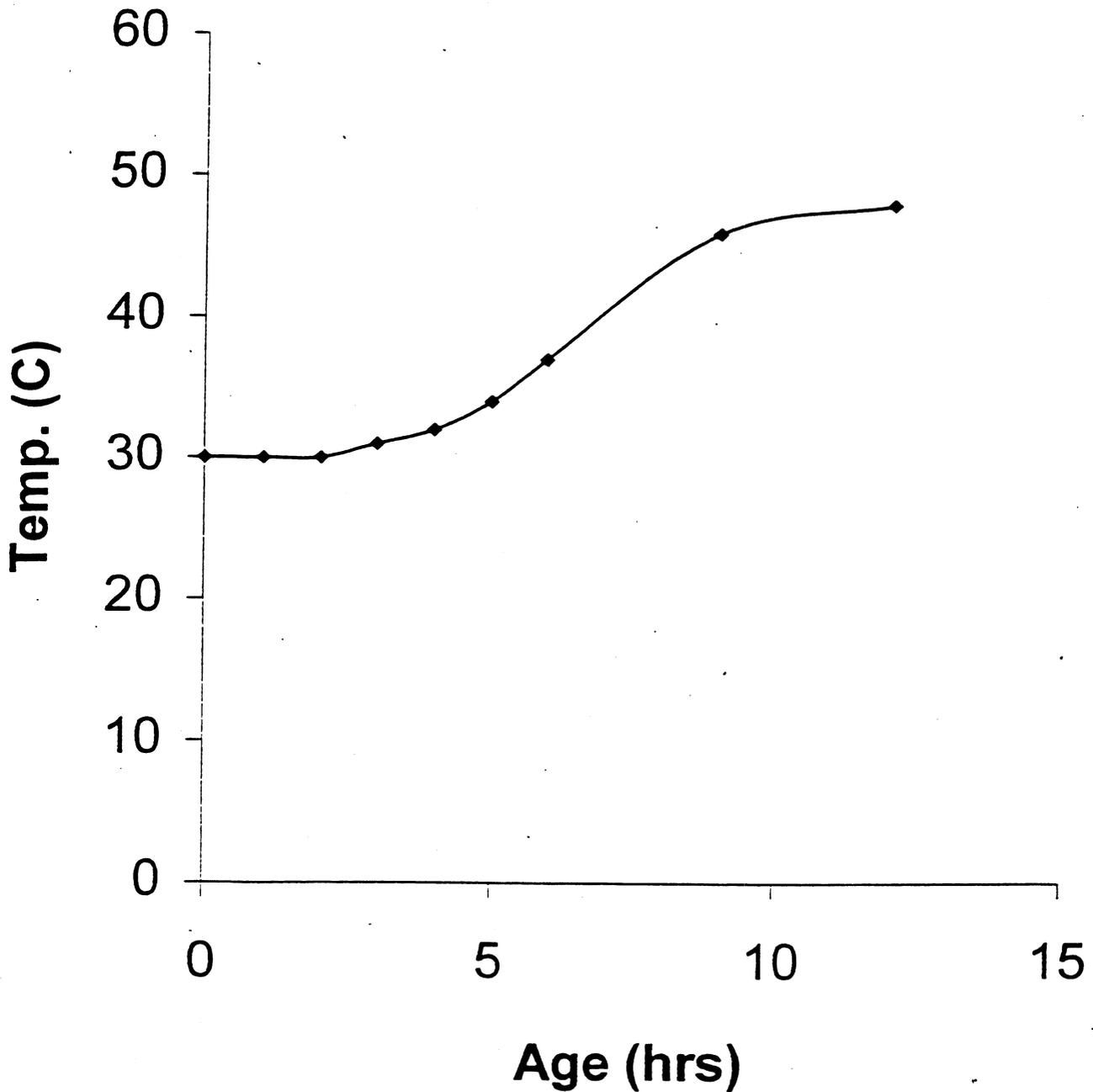
Appendix E

Concrete Test Results – Control Samples

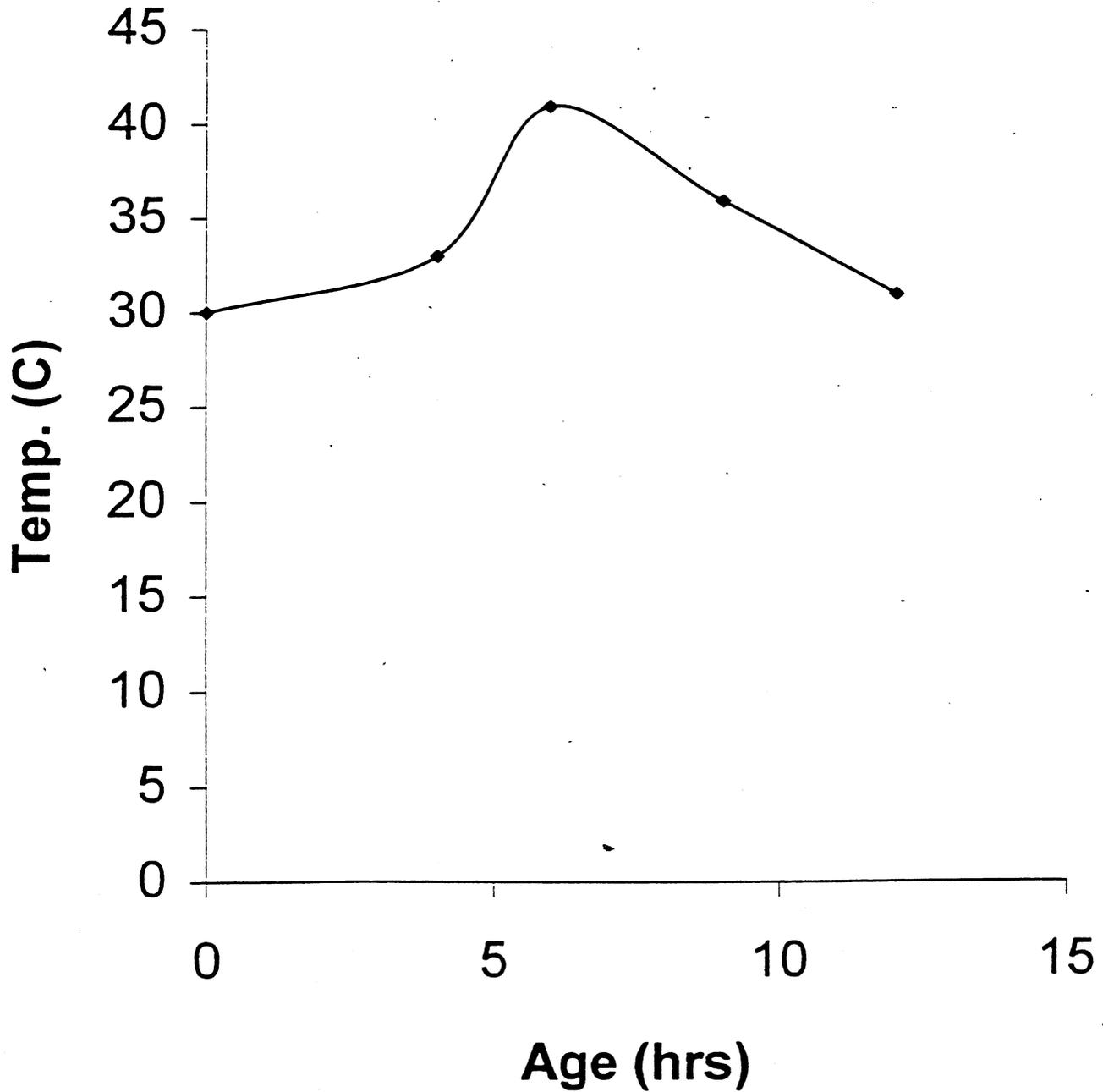
Law Engineering and Environmental
Services, Inc.

September 1998.

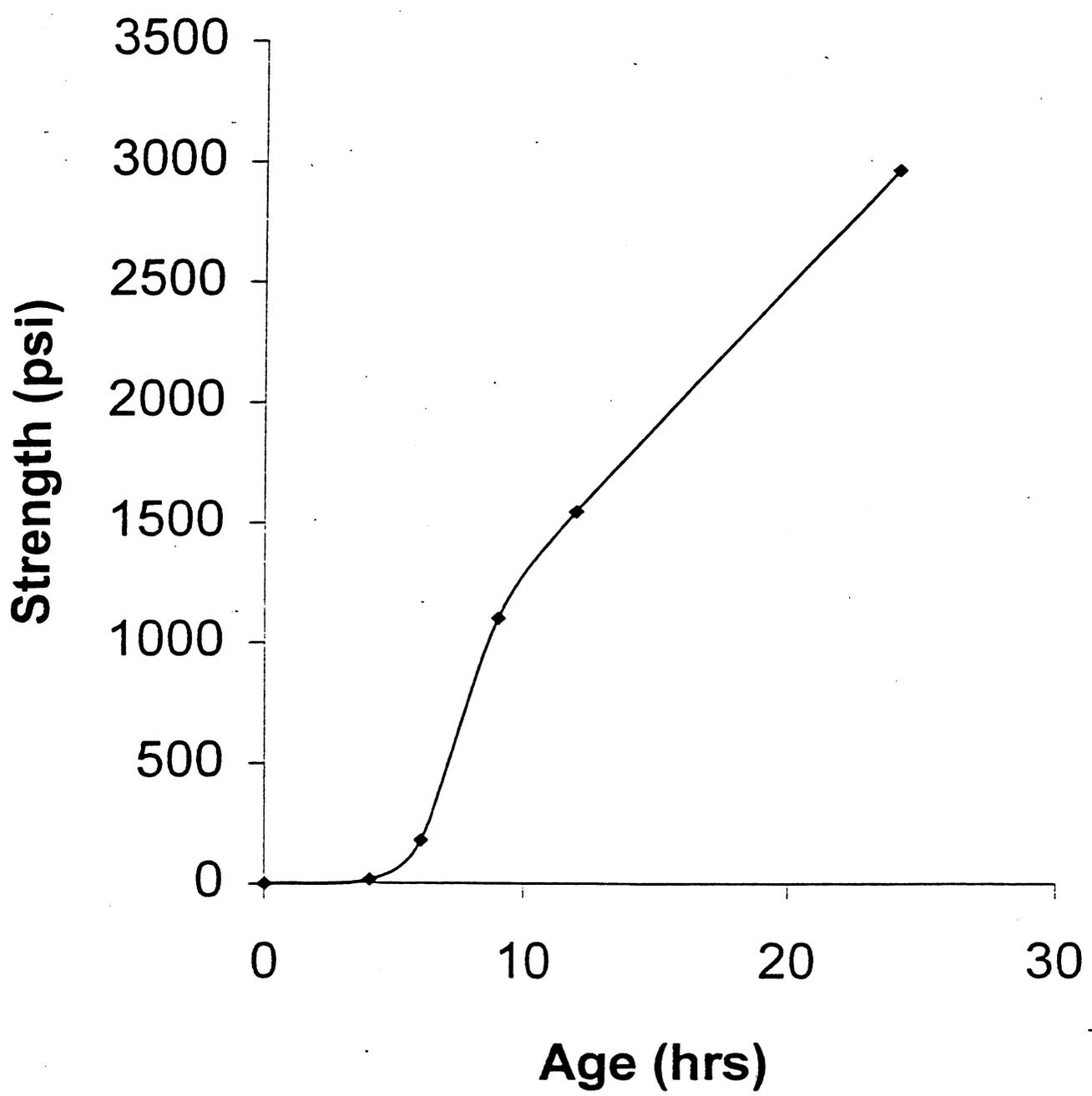
36-Inch Shaft Concrete Temp. vs. Age (Shaft Group 2)



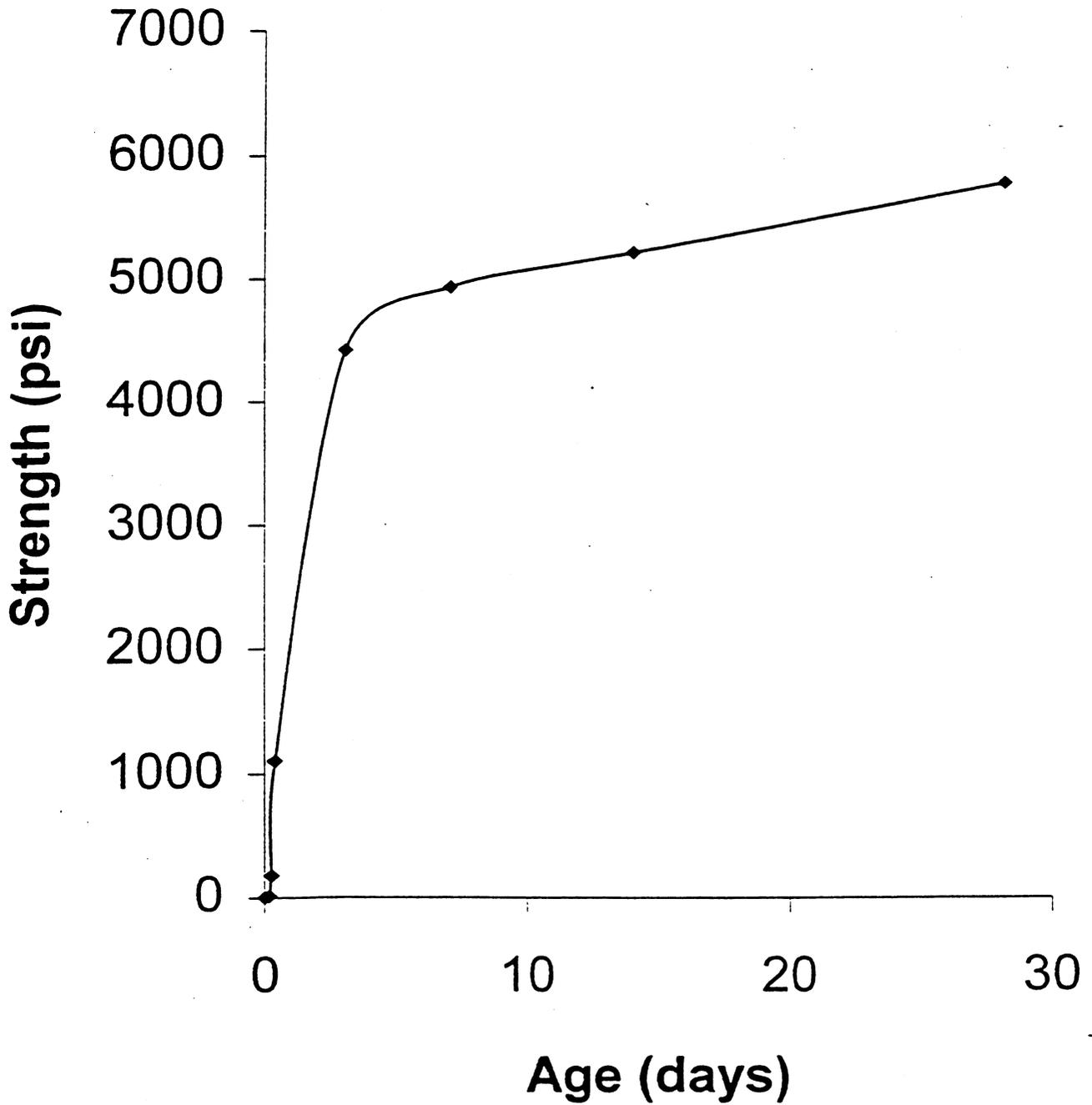
Test Cylinder Concrete Temp. vs. Age (Shaft Group 2)



Test Cylinder Strength vs. Age (Shaft Group 2: 0-24 hrs)



Test Cylinder Strength vs. Age (Shaft Group 2: 0-28 days)





CONCRETE TEST REPORT

CLIENT: FAU / FDOT

JOB NO: 40522-7-7891

PROJECT: FAU Vibration Monitoring

LAB NO:

MIX ID: 51-4709

SPECIFIED STRENGTH: 4,000

MIX DESC: Drilled Shaft

FIELD INFORMATION

FIELD TESTS
(SPECIFIED RANGE)

DATE SAMPLED: 9/10/98 BY: LAW JB SLUMP: 8"
 TIME BATCHED: 10:25 a.m. SAMPLED: 11:15 a.m. PERCENT AIR: N.I.
 BATCH PLANT: Florida Rock UNIT WEIGHT: N.I.
 TRUCK/TICKET: 708/82 DATE RECEIVED: 9/10/98 MIX/AIR TEMP: 30°/28°C

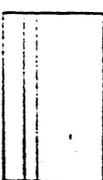
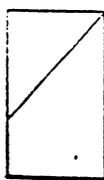
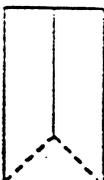
COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
1A	6.00	28.27	9/10/98	4 hours	400	14	
1B	6.00	28.27	9/10/98	4 hours	350	12	
1C	6.00	28.27	9/10/98	4 hours	400	14	
2A	6.00	28.27	9/10/98	6 hours	5500	194	
2B	6.00	28.27	9/10/98	6 hours	4400	155	
2C	6.00	28.27	9/10/98	6 hours	5300	187	
3A	6.00	28.27	9/10/98	9 hours	30000	1061	
3B	6.00	28.27	9/10/98	9 hours	32500	1149	
3C	6.00	28.27	9/10/98	9 hours	31000	1096	
4A	6.00	28.27	9/10/98	12 hours	44000	1556	
4B	6.00	28.27	9/10/98	12 hours	42500	1503	
4C	6.00	28.27	9/10/98	12 hours	45000	1591	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C39 AND ASTM C1231.
 <<< DENOTES CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1231.

POUR LOCATION: 24-inch and 36-inch diameter shafts

REMARKS: 10 gallons water added to 9 c.y.



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CONCRETE TEST REPORT

CLIENT:

JOB NO:

PROJECT:

LAB NO:

MIX ID:

SPECIFIED STRENGTH:

MIX DESC:

FIELD INFORMATION

FIELD TESTS
(SPECIFIED RANGE)

DATE SAMPLED:

BY:

SLUMP:

TIME BATCHED:

SAMPLED:

PERCENT AIR:

PATCH PLANT:

UNIT WEIGHT:

TRUCK/TICKET:

DATE RECEIVED:

MIX/AIR TEMP:

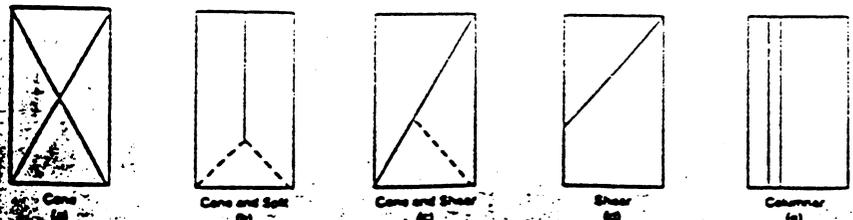
COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
5A	6.00	28.27	9/11/99	1 day	81,000	2865	
5B	6.00	28.27	9/11/99	1 day	87,000	3077	
6A	6.00	28.27	9/13/99	3 days	123,000	4350	
6B	6.00	28.27	9/13/99	3 days	127,000	4492	
7A	6.00	28.27	9/17/99	7 days	139,000	4915	
7B	6.00	28.27	9/17/99	7 days	140,000	4952	
8A	6.00	28.27	9/24/99	14 days	147,500	5217	
8B	6.00	28.27	9/24/99	14 days	148,000	5235	
9A	6.00	28.27	10/08/99	28 days	161,000	5695	
9B	6.00	28.27	10/08/99	28 days	166,000	5871	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C39 AND ASTM C1231.
 NOTES CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1231.

POUR LOCATION:

REMARKS:



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[Handwritten Signature]

CONCRETE TEST REPORT

CLIENT: FAU / FDOT

JOB NO: 40522-7-7891

PROJECT: FAU Vibration Monitoring

LAB NO:

MIX ID: 51-4709

SPECIFIED STRENGTH: 4,000

MIX DESC: Drilled Shaft

FIELD INFORMATION

FIELD TESTS
(SPECIFIED RANGE)

DATE SAMPLED: 9/10/98 BY: LAW JB

SLUMP: 8"

TIME BATCHED: 10:25 a.m. SAMPLED: 11:15 a.m.

PERCENT AIR:

BATCH PLANT:

UNIT WEIGHT:

TRUCK/TICKET: 708/82

DATE RECEIVED: 9/10/98

MIX/AIR TEMP: 30°/28°C

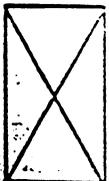
COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
S1.1	6.00	28.27	9/24/99	14	160,000	5659	
S2.1	6.00	28.27	9/24/99	14	162,000	5730	
S3.1	6.00	28.27	9/24/99	14	154,000	5447	
S1.2	6.00	28.27	10/08/99	28	169,000	5978	
S2.2	6.00	28.27	10/08/99	28	172,000	6084	
S3.2	6.00	28.27	10/08/99	28	174,000	6155	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C09 AND ASTM C1231.
 NOTES CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1231.

POUR LOCATION: Surface cylinders

REMARKS: 10 gallons water added to 9 c.y.



Cone (a)



Cone and Split (b)



Cone and Shear (c)



Shear (d)



Columnar (e)

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CONCRETE TEST REPORT

CLIENT:

JOB NO:

PROJECT:

LAB NO:

MIX ID:

SPECIFIED STRENGTH:

MIX DESC:

FIELD INFORMATION

FIELD TESTS
(SPECIFIED RANGE)

DATE SAMPLED:

BY:

SLUMP:

TIME BATCHED:

SAMPLED:

PERCENT AIR:

BATCH PLANT:

UNIT WEIGHT:

TRUCK/TICKET:

DATE RECEIVED:

MIX/AIR TEMP:

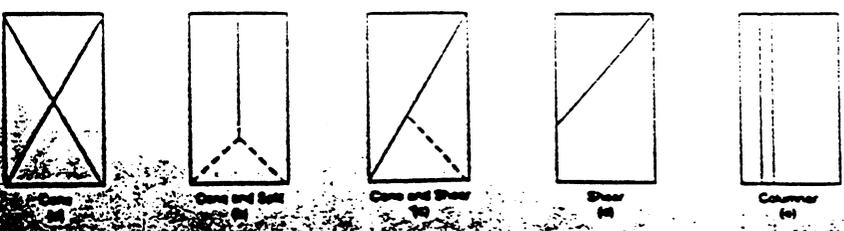
COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
5A	6.00	28.27	9/11/99	1 day	81,000	2865	
5B	6.00	28.27	9/11/99	1 day	87,000	3077	
6A	6.00	28.27	9/13/99	3 days	123,000	4350	
6B	6.00	28.27	9/13/99	3 days	127,000	4492	
7A	6.00	28.27	9/17/99	7 days	139,000	4916	
7B	6.00	28.27	9/17/99	7 days	140,000	4952	
8A	6.00	28.27	9/24/99	14 days	147,500	5217	
8B	6.00	28.27	9/24/99	14 days	148,000	5235	
9A	6.00	28.27	10/08/99	28 days	151,000	5695	
9B	6.00	28.27	10/08/99	28 days	156,000	5871	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C39 AND ASTM C1231.
 <<< DENOTES CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1231.

POUR LOCATION:

REMARKS:



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[Handwritten Signature]



CONCRETE TEST REPORT

CLIENT: FAU / FDOT

JOB NO: 40522-7-7891

PROJECT: FAU Vibration Monitoring

LAB NO:

MIX ID: 51-4709

SPECIFIED STRENGTH: 4,000

MIX DESC: Drilled Shaft

FIELD INFORMATION

FIELD TESTS
(SPECIFIED RANGE)

DATE SAMPLED: 9/10/98 BY: LAW JB SLUMP: 8"
 TIME BATCHED: 10:25 a.m. SAMPLED: 11:15 a.m. PERCENT AIR: N.I.
 BATCH PLANT: Florida Rock UNIT WEIGHT: N.I.
 TRUCK/TICKET: 708/82 DATE RECEIVED: 9/10/98 MIX/AIR TEMP: 30°/28°C

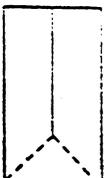
COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
1A	6.00	28.27	9/10/98	4 hours	400	14	
1B	6.00	28.27	9/10/98	4 hours	350	12	
1C	5.00	28.27	9/10/98	4 hours	400	14	
2A	6.00	28.27	9/10/98	6 hours	5500	194	
2B	6.00	28.27	9/10/98	6 hours	4400	155	
2C	6.00	23.27	9/10/98	6 hours	5300	187	
3A	5.00	28.27	9/10/98	9 hours	30000	1061	
3B	5.00	28.27	9/10/98	9 hours	32500	1149	
3C	5.00	28.27	9/10/98	9 hours	31000	1096	
4A	6.00	23.27	9/10/98	12 hours	44000	1556	
4B	6.00	23.27	9/10/98	12 hours	42500	1503	
4C	6.00	28.27	9/10/98	12 hours	45000	1591	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C39 AND ASTM C1231.
 DENOTES CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1231.

POUR LOCATION: 24-inch and 36-inch diameter shafts

REMARKS: 10 gallons water added to 9 c.y.



RESPECTFULLY SUBMITTED



CONCRETE TEST REPORT

CLIENT: FAU / FDOT

JOB NO: 40522-7-7891

PROJECT: FAU Vibration Monitoring

LAB NO:

MIX ID: 51-4709

SPECIFIED STRENGTH: 4,000

MIX DESC: Drilled Shaft

FIELD INFORMATION _____ FIELD TESTS (SPECIFIED RANGE) _____

DATE SAMPLED: 9/10/98 BY: LAW JB SLUMP: 8"
 TIME BATCHED: 10:25 a.m. SAMPLED: 11:15 a.m. PERCENT AIR:
 BATCH PLANT: UNIT WEIGHT:
 TRUCK/TICKET: 708/82 DATE RECEIVED: 9/10/98 MIX/AIR TEMP: 30°/28°C

COMPRESSION TEST RESULTS

CYLINDER NUMBER	DIAMETER (IN.)	AREA (SQ. IN.)	DATE TESTED	TEST AGE (DAYS)	MAX LOAD (Lbs.)	COMP. STRENGTH (psi)	FRACTURE TYPE
S-S1.1	6.00	28.27	9/24/99	14	160,000	5659	
S-S2.1	6.00	28.27	9/24/99	14	162,000	5730	
S-S3.1	6.00	28.27	9/24/99	14	164,000	5447	
S-S1.2	6.00	28.27	10/08/99	28	169,000	5978	
S-S2.2	6.00	28.27	10/08/99	28	172,000	6084	
S-S3.2	6.00	28.27	10/08/99	28	174,000	6153	

UNLESS OTHERWISE INDICATED, ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM C39 AND ASTM C1201.
 CEMENTS CURED IN FIELD <<< DENOTES LOW COMPRESSIVE STRENGTH
 FRACTURE TYPE IS INDICATED BY LETTER: (A) CONE (B) CONE & SPLIT (C) CONE AND SHEAR (D) SHEAR (E) COLUMNAR
 FRACTURE TYPE NOT APPLICABLE WHEN CYLINDERS TESTED IN ACCORDANCE WITH ASTM C1201.

POUR LOCATION: Surface cylinders

REMARKS: 10 gallons water added to 9 c.y.



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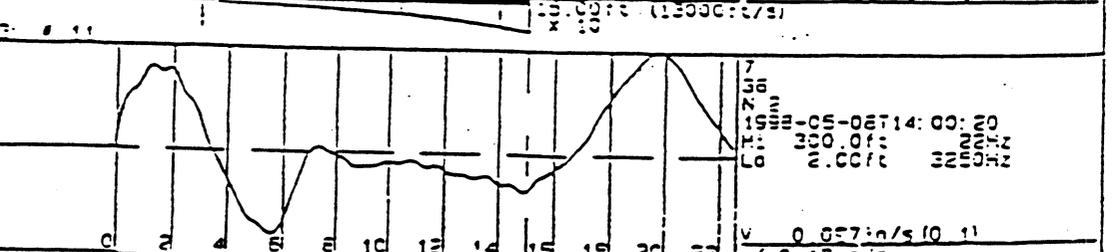
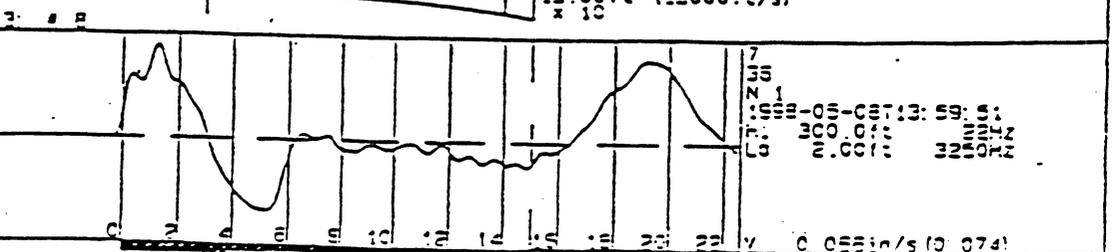
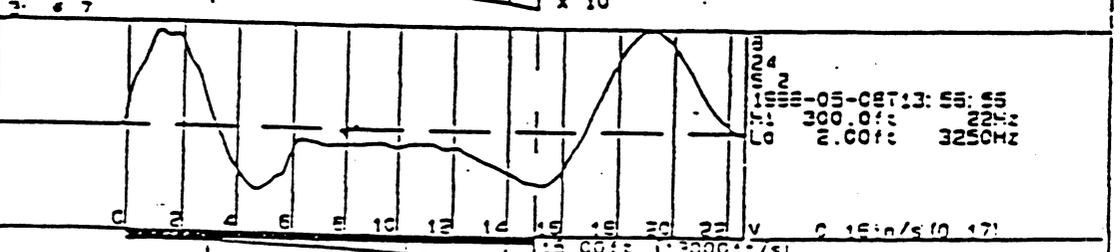
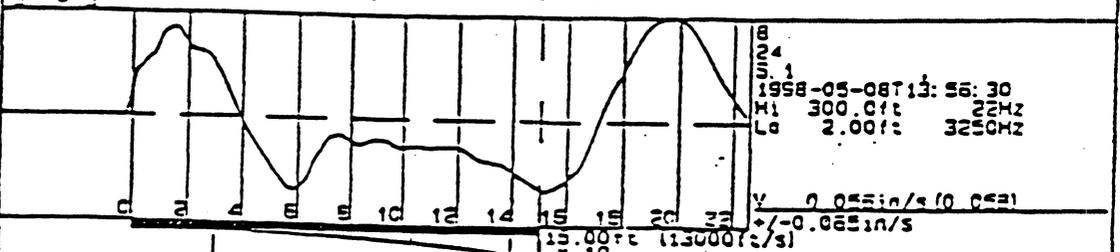
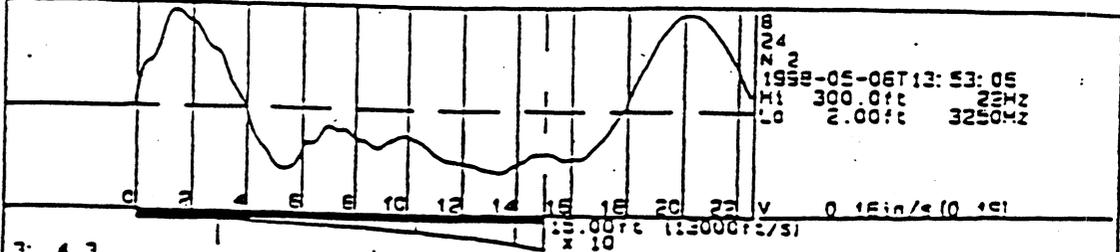
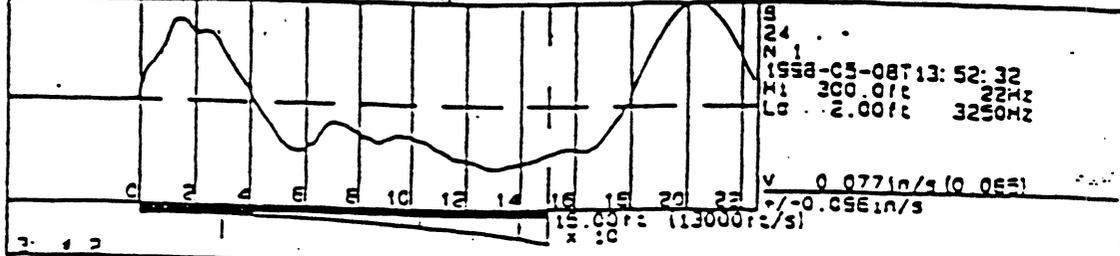
APPENDIX F

P.I.T. METHOD

**GLOBAL RAUSCHE LIKINS AND ASSOCIATES,
INC.(GRL)**

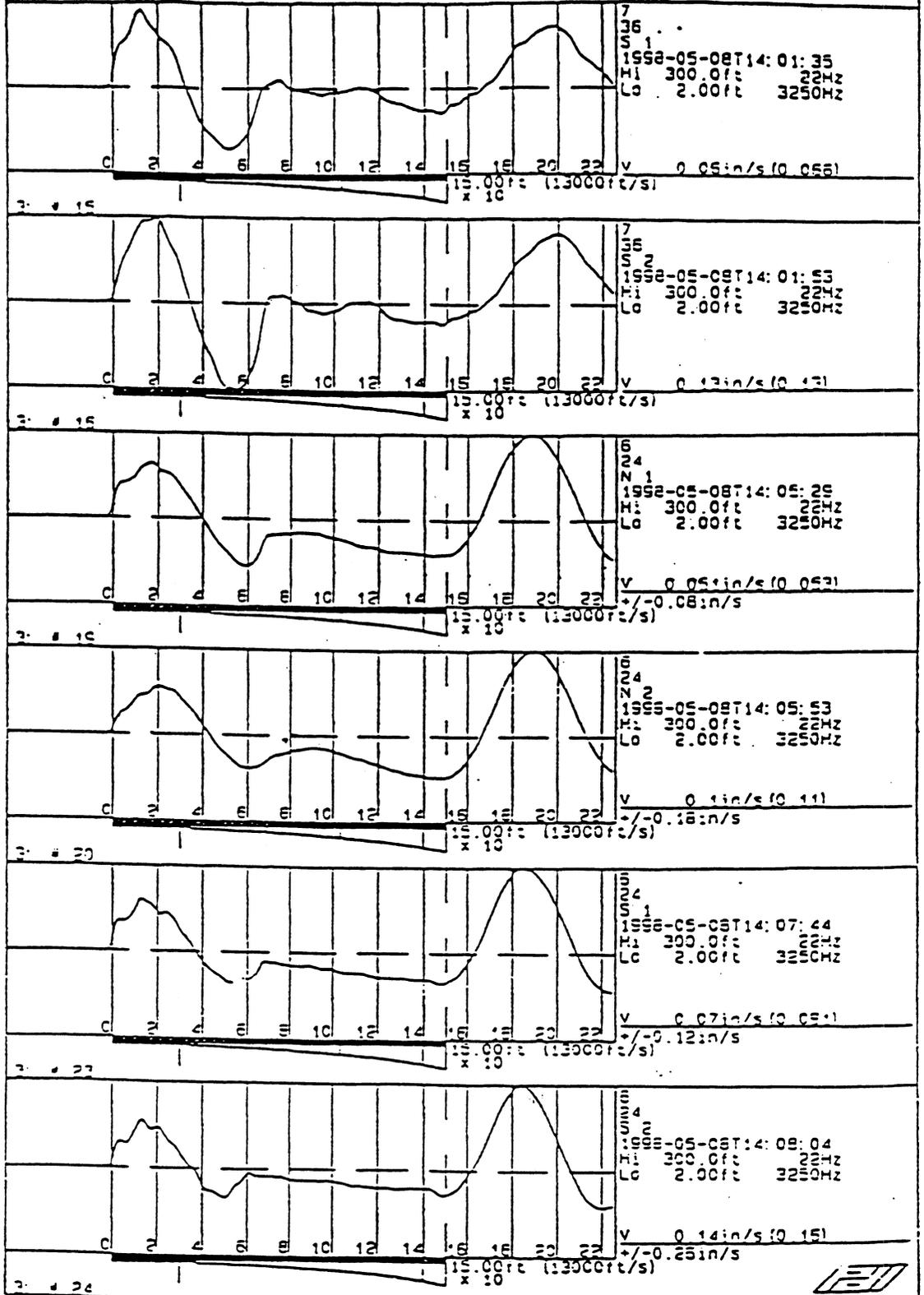
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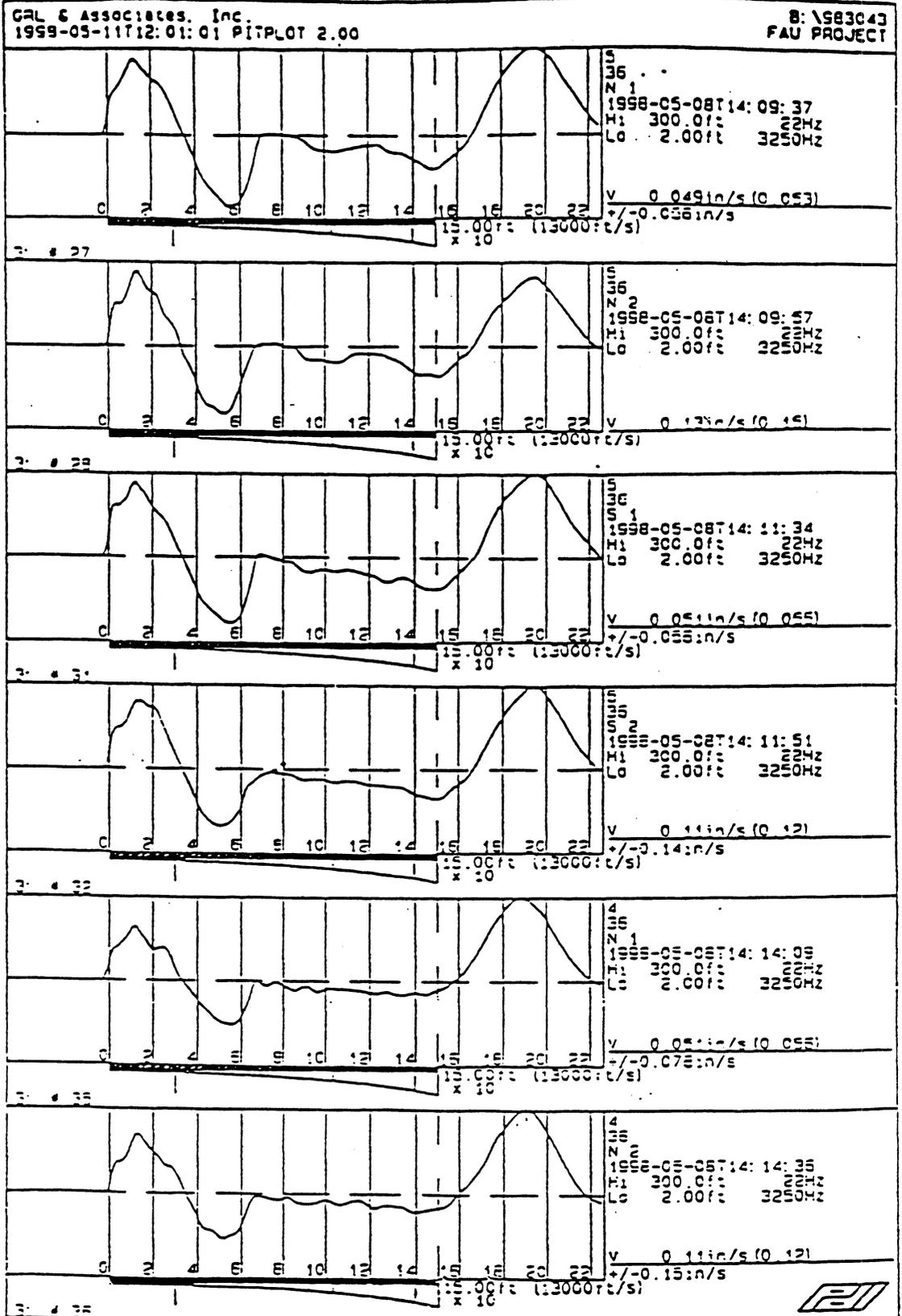
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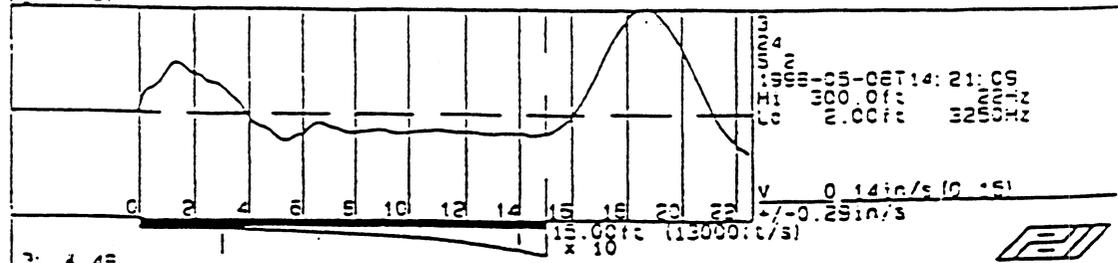
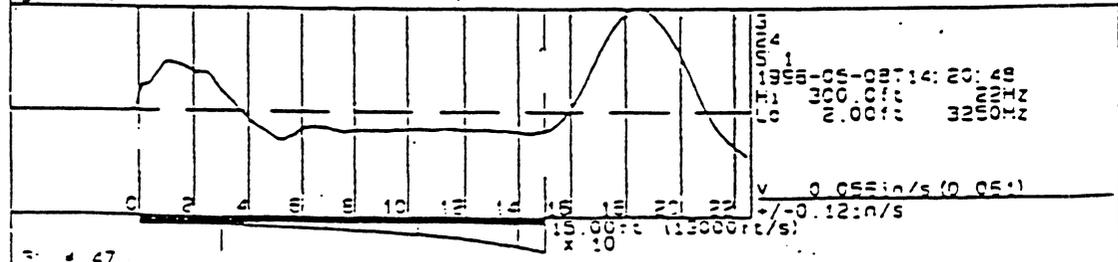
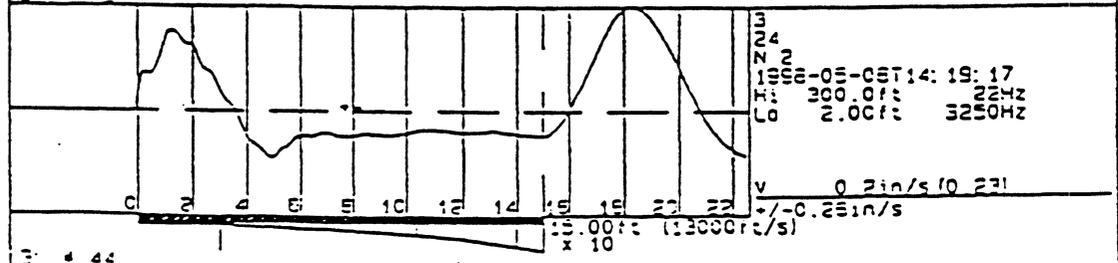
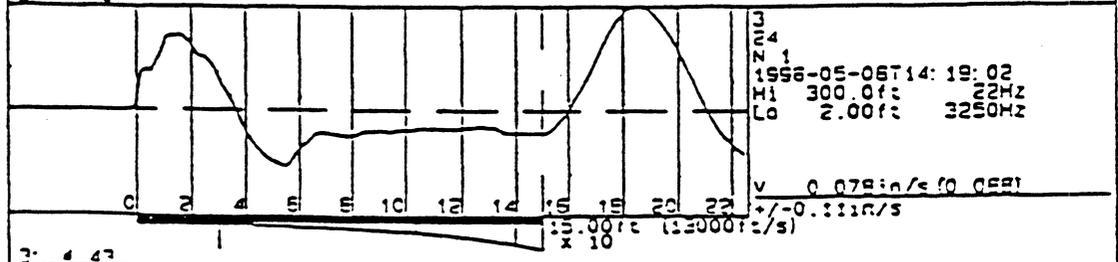
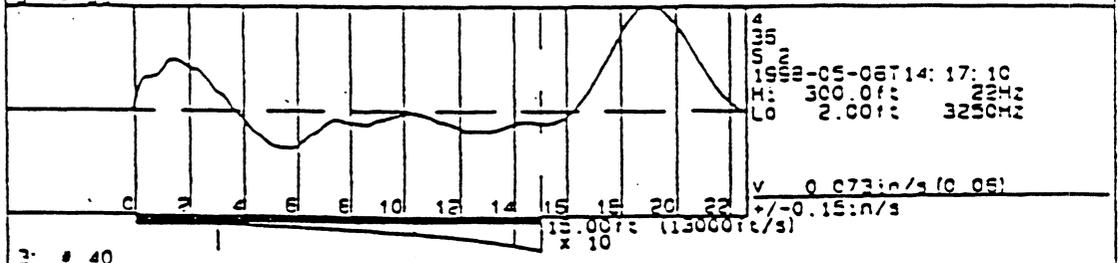
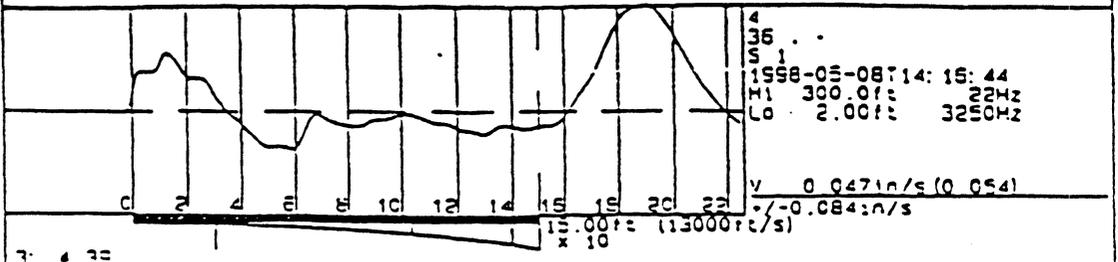


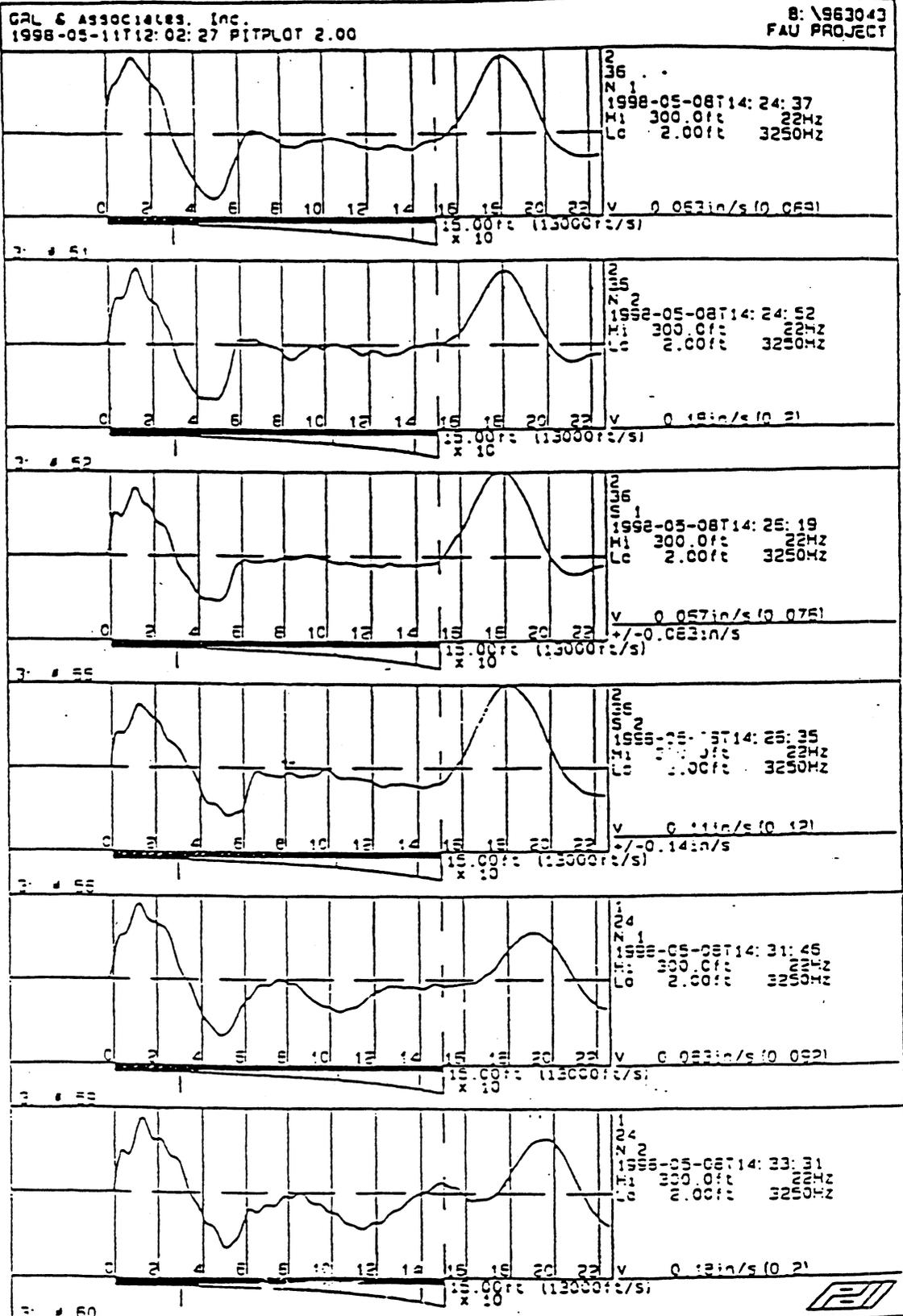


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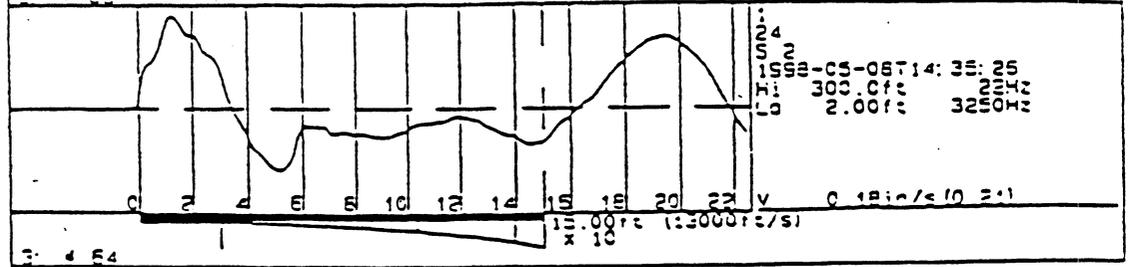
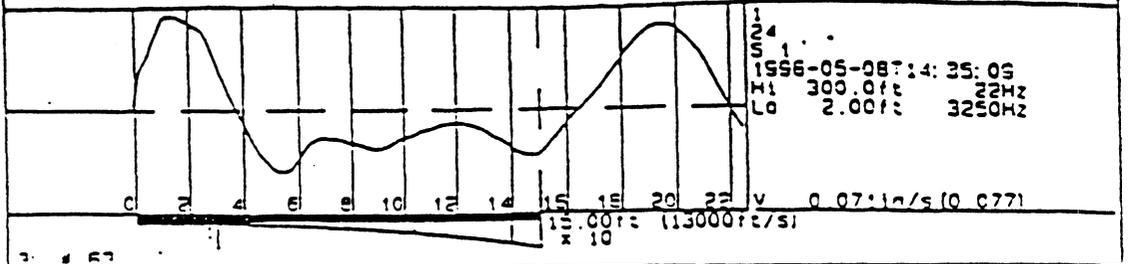




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GRL Cable Rausche Links and Associates, Inc.

APPENDIX A:

THE LOW STRAIN PULSE ECHO AND TRANSIENT RESPONSE METHOD FOR PILE INTEGRITY TESTING

INTRODUCTION

Pile integrity test methods of driven and cast-in-situ piles have been developed by several researchers using differing equipment and methods. GRL's low strain test method is easily applied, quick, inexpensive and, therefore, routinely applied worldwide. Often all piles on a site are tested and problem piles are identified for corrective action.

The method uses signals from a hand held hammer impacting the pile top and generating a compressive stress wave in the pile. Stress wave reflections from non-uniformities or the pile toe are observed at the pile top, processed and interpreted by the experienced test engineer. The Pulse Echo Method (PEM) records the pile top velocity as a function of time. The Transient Response Method (TRM) displays the mobility, *i.e.*, the ratio of frequency spectra of pile top velocity and force. P.I.T. combines both PEM and TRM.

BACKGROUND

When a long rod, such as a precast, driven or drilled shaft or pile, is struck with a small hammer, a stress wave is generated which travels down the shaft to the pile bottom where it is reflected. When the reflected stress wave returns to the pile top, a measurable pile top motion occurs. If this reflection wave occurs at the correct time and if no other earlier reflection is received at the pile top, then the pile shaft is most probably free of major defects.

When a lightweight hand held hammer strikes the pile top, a small pile top motion (velocity) is generated and measured. The associated pile strains are of such a low magnitude that they would be measured in the pile only with great difficulty. This test is therefore also known as low strain integrity test. However, the force applied by the hammer can be easily measured by instrumenting the hammer itself. Primarily, the velocity record and to a lesser degree the force record contain information about the location and magnitude of pile non-uniformities. Under the assumption of proportional force and

velocity records, and for short duration impact pulse the velocity record may be sufficient. One of the distinguishing features of the Transient Response Method (TRM) is that it requires the measurement of both velocity and force while the Pulse Echo Method (PEM) only relies on velocity records. A second difference is the display of the TRM results in the frequency domain. PEM offers powerful record enhancement techniques and presents the resulting curves as a function of time.

STRESS WAVE PROPAGATION IN A PILE

An impact applied to the pile top generates momentary compression and a particle motion of the pile top surface. The compression is related to the force, F ; the motion causes a particle velocity, v . In concrete, the stress wave travels along the pile with a stress wave speed, c , ranging from 10,000 to 15,000 ft/s (3,000 to 4,500 m/s).

$$c = \sqrt{\frac{E}{\rho}} \quad (1)$$

where E is the pile's elastic modulus and ρ its mass density. Figure 1 shows, in the form of a time-depth plot, the path of a stress wave in the pile, illustrating the arrival times of reflection waves at the pile top. Both cross sectional changes and soil resistance forces generate reflection waves. The pile top velocity is affected by and therefore indicates the arrival of both tensile and compressive reflection waves.

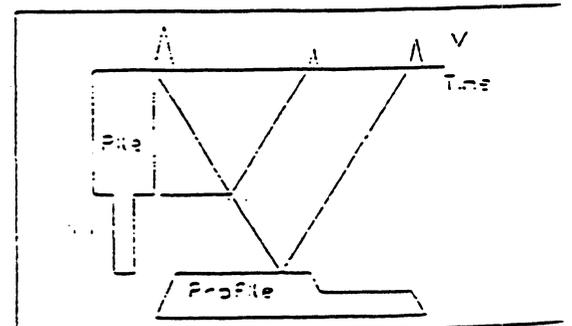


Figure 1: Traveling stress waves and the principle of the pile profile estimate.

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P-13

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the reflection from the pile toe is not readily apparent (as in the example of Figure 3), then the velocity is usually multiplied with an amplification function whose magnitude is unity at impact increasing exponentially with time until it reaches its maximum intensity at time $2L/c$ after impact ($2L/c$ is the time which the stress wave requires to travel the pile length, L , and return). In Figure 4, the upper graph is the same average velocity trace as in Figure 3 but after an amplification of 40 was applied. Note that the pile toe now is clearly identifiable. For longer piles or stronger soils, even higher amplification factors may be used; this process requires signal conditioning and digital processing equipment with very low noise to signal ratios. If the toe signal is apparent, then it is possible to confirm the originally assumed wave speed.

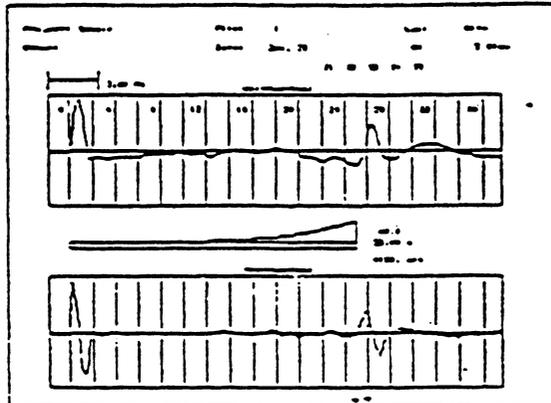


Figure 4: Amplified average velocity and acceleration from the standard P.I.T. output.

Figure 4 is the standard P.I.T. output. It includes the acceleration on the bottom and the exponential amplification function with the multiplier at time $2L/c$. The title section includes the blow numbers averaged. The assumed wave speed and pile length are also included in the output. A clearly indicated toe signal together with a fairly steady velocity trace between the impact and toe signal are signs of a sound pile. Strong velocity variations may be the result of changes in pile cross section, concrete quality or soil resistance. For example, relative increases in pile top velocity may be the result of either a cross sectional decrease or a soft soil layer. In the absence of soil resistance changes, pile top variations are caused by pile impedance changes. The pile impedance is defined as

$$Z = EA/c = A\sqrt{E\rho} \quad (2)$$

with A being the pile's cross sectional area. Thus, impedance reduction can be caused by a decrease either in area, A , or in the concrete's modulus, E , in density, ρ . Since both modulus and density are related to concrete strength, it is fair to say that impedance depends on cross sectional area and concrete quality.

In general, relatively sharply defined reflections are attributed to impedance changes, slowly changing reflections are thought to be caused by the soil. The basic assumption limits damage recognition integrity assessment to situations which occur over distance given by one impact pulse width. Gradual pile tapers therefore cannot be detected. On the other hand, if the effect of soil resistance is known from reference piles, then unusual shafts can be more easily identified. Improved quantitative interpretation may require signal matching, pile profile calculation and comparison with records of other piles at the same site (see section on P.I.T.WAP). But even with these more sophisticated methods, the question of the effect of soil resistance still may not be clear.

The velocity curve can also be transformed to the frequency domain (Figure 5) and interpreted in a manner similar to mobility discussed below. An additional transform of the transformed velocity may be used to extract significant record portions, called "Velocity Reflectors", over a length scale.

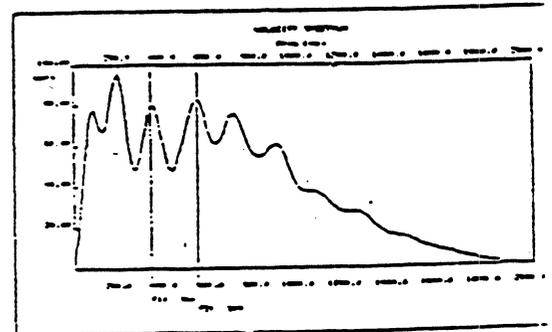


Figure 5: Velocity spectrum

THE TRANSIENT RESPONSE METHOD

The TRM requires that both the pile top motion at the impact force be measured. This concept has been borrowed from standard Non-Destructive Testing technology. In fact, the first applications of NDT to piles required the measurement of force and velocity under a steady state vibrator which could

A-3

apply substantial forces at variable frequencies. Fortunately, the force frequency spectrum of a hand held impact hammer is flat over a relatively wide frequency band. A simple hand held hammer can therefore adequately produce the frequency components necessary to test both well constructed or defective piles with TRM.

The standard result of TRM is a plot of the frequency spectrum of the ratio of velocity to force. This so-called mobility is really the inverse of the impedance and therefore an indication of the pile's velocity, $v(\omega)$, response to a particular excitation force, $F(\omega)$.

$$M(\omega) = v(\omega)/F(\omega) \tag{3}$$

A mobility peak occurs at a frequency indicative of the time when the velocity changes due to a reflection from the pile toe or an intermediate impedance reduction. Mobility peaks occurring at regular intervals are indicative of a dominant frequency Δf . The corresponding depth $(L_2 - L_1)$, in Figure 6) at which the change occurs is calculated from

$$x = c/2\Delta f \tag{4}$$

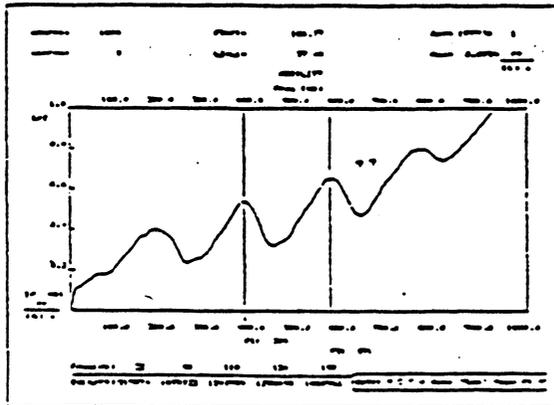


Figure 6: Mobility with dynamic stiffnesses

Furthermore, dividing the velocity by frequency leads to displacement. Dividing force by displacement at a given frequency leads to the so-called dynamic stiffness.

$$E_d = \omega F(\omega)/v(\omega) = 2\pi f/M(\omega) \tag{5}$$

where ω and f are the frequency in radians per second and Hz, respectively. In practice, low frequency values are divided by the associated

mobility yielding a so-called dynamic stiffness, E_d . This quantity increases with decreasing pile toe response. A low pile toe response is often the result of high soil resistance. However, it may also be caused by highly variable pile properties or internal pile damping and is therefore only indirectly related to quantitative pile bearing capacity. However, E_d is calculated since it does provide a quantitative result for the evaluation of pile quality. Figure 6 shows an example of a mobility and, at the bottom, the associated E_d values for several frequencies.

P.I.T.WAP SIMULATED RESPONSE

P.I.T. yields some form of pile top motion curve. Interpretation of these curves is left to the more or less experienced engineer. An invaluable aid in the interpretation effort are similar curves analytically developed. The special purpose computer program P.I.T.WAP™ (Pile Integrity Testing Wave Analysis Program) which is based on CAPWAP® (Case Pile Wave Analysis Program), requires that the description of pile and soil are input, and generates as an output pile top velocity vs. time or the mobility.

A large catalogue of these calculated responses was compiled as a guide for record interpretation. One of many possible demonstration cases is shown in Figure 7. It demonstrates the calculated curves for a pile with sufficient uniform shaft resistance to reduce the pile toe reflection to a small pulse. The pile was assumed to have an enlarged center section which first produces a negative and later a positive velocity response. In the middle of the record, a very slight secondary reflection can be observed. It was caused when the initially negative reflection from the shift increase again reached the enlarged pile section after reflection at the top.

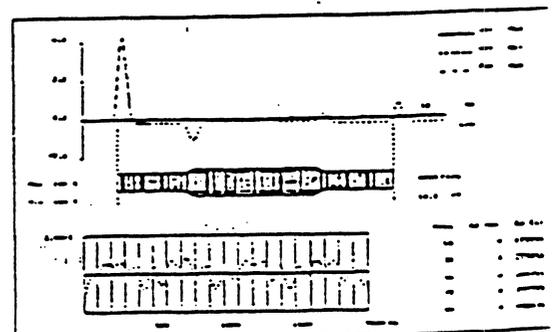


Figure 7: Calculated velocity and mobility for a non-uniform pile.

P.I.T.WAP SIGNAL MATCHING

The P.I.T.WAP program can also be employed in the interpretation of measured P.I.T.-FV velocity or velocity and force records. Suppose that the measured force record is imposed as a pile top boundary condition for an analytical model of pile and soil. One result of this analysis would be the calculated pile top velocity. Comparison of the measured with the computed velocity allows the engineer to gain insight regarding the pile impedance variations. Reanalyzing with a variable impedance pile model should therefore lead to an improved match of measured and calculated pile top velocity. The process of changing impedance and reanalyzing is continued until a good match is achieved. At that point, the most likely pile impedance profile has been determined. Typically, P.I.T.WAP divides the pile in continuous segments of approximately 10 inches (250 mm) length. The program automatically calculates the pile impedance of all segments after the engineer has decided on a soil resistance model. (This soil resistance model bears no relationship with static resistance or with damping magnitudes encountered during pile driving.) Generally, it is possible to extract these relative soil resistance parameters from P.I.T.-FV tests on reference piles. These parameters are then used to analyze neighboring piles with impedance variations.

Figure 8 illustrates the results obtained from P.I.T.-WAP matching. The plot includes the velocity match, the predicted pile impedance variation and the mobility based on both calculated and measured velocity.

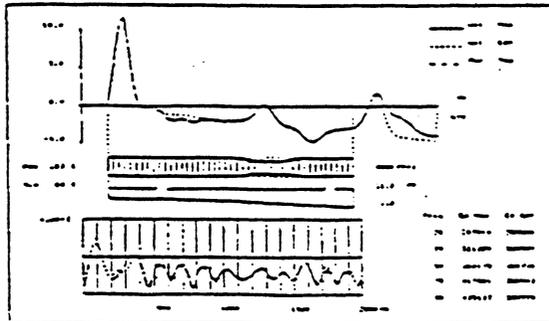


Figure 8: Results from P.I.T.WAP velocity matching, including velocity and mobility match and predicted pile impedance variation.

THE P.I.T.WAP PILE PROFILE

The P.I.T.WAP program contains a second option for pile impedance calculations. It has the advantage of much greater simplicity and the disadvantage that it does not identify effects of secondary reflections. An estimated pile profile is calculated based on the difference between calculated and measured pile top velocity. The basis of the pile profile calculation is the fact that a step-wise change in impedance causes a pulse-like velocity wave effect at the pile top (Figure 1). In other words, the effect of an impedance change has the appearance of the derivative of the impedance change. Inversely, the profile is the integral of the wave effects at the pile top. In the absence of soil resistance effects, the profile therefore has the appearance of the pile top displacement. Again, please note that the P.I.T. Profile option will not identify and correct for secondary reflections. Furthermore, any impedance reduction, whether crack or lack of concrete quality, will be identified in the form of a decreasing cross sectional area.

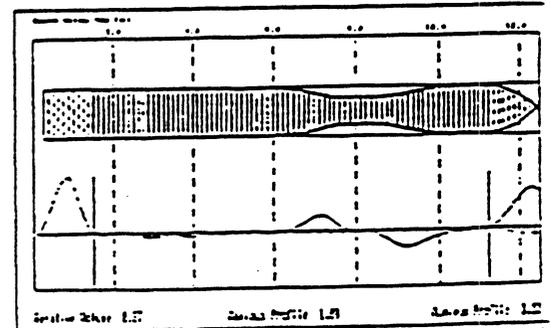


Figure 9: Results from P.I.T.WAP pile profile calculation

The pile profile results is shown in Figure 9 showing the following

- a length scale
- a relative volume calculated from the apparent pile profile (this value may be compared with actual construction records). A relative volume of 1.0 corresponds to the pile top cross-sectional area times the pile length.
- The calculated pile profile graph includes shaded areas which are uncertain due to the width of the input pulse.

- Minimum and maximum impedance values between the shaded areas.
- The measured velocity (solid line), and the calculated velocity (dash) which accounts for soil resistance effects.

CONCLUSIONS

The low strain integrity test, P.I.T., is powerful, quick, inexpensive but, naturally, limited. First, the length information obtained from a toe signal (or a governing frequency) is only as accurate as the wave speed value assumed in the processing of the records. Even at sites where concrete quality is well maintained, wave speed variations of 5% are not uncommon. A pile length calculated from a toe signal is therefore only well known within $\pm 5\%$. Second, certain reflections produce secondary and even tertiary wave reflections. For example, if an impedance reduction occurs in the middle of the pile, then what may appear to be the pile toe response may actually be a secondary reflection of the mid-pile defect. For piles with severe cracks or manufactured mechanical joints, the stress wave will, in general, not be transmitted below the "gap" and therefore the pile below this "defect" cannot be evaluated. Third, piles with multiple or highly variable cross sectional changes are difficult to analyze. Soil effects may severely interfere with pile impedance related signals. In fact, a rule of thumb limits the testable pile length to 30 diameters, however, this is probably only correct for soils with relatively high resistance. For extremely high resistance or strongly variable shaft impedance, the test length may be less than 30 diameters. It is often helpful to identify soil effects by comparing records of a large number of piles or by using the record of a statically loaded (or high strain tested) reference pile.

Piles which are rigidly attached to other parts of a structure can sometimes be tested and analyzed successfully, but often the analysis is more difficult than those with free pile tops.

The additional force measurement of P.I.T.-FV and TRM definitely provides supplemental information of cross sectional changes near the pile top, i.e., within the distance corresponding to the impact signal. The extra expense of the force measurement is, therefore, worthwhile whenever questions arise as to the integrity of upper (say 5 ft. or 1.5 m) pile portion.

The record presentation in the frequency domain may, on occasion be of benefit. For example important record components may be hidden in steady state signal caused by pile top reinforcement vibration. They also provide information on dynamic stiffness although no low strain method can truly give quantitative information on ultimate capacity of a pile. In general, however, interpretation of time records is much simpler than that of frequency records. Time records can also be simply analyzed by the P.I.T.WAP signal matching the pile profile technique for a more objective and very illustrative result compared to the engineer's verbal assessment.

IMPLEMENTATION

There is no point in doing the P.I.T. test if there is no consensus among the owner, engineer and contractor on corrective actions should the tests indicate defects. Also, it must be anticipated that not all tests are conclusive.

- If serious defects are indicated by the method a contingency plan must be in place which allows the engineer(s)-in-charge to choose from a menu of additional tests or corrective measures such as replacement pile, coring, drilling, pressure injection, etc.
- The test may be inconclusive for greater depth due to its limitations. However, it may be sufficient to accept the results for the upper shaft portion where most problems usually occur for drilled shafts.
- Other readily acceptable solutions may include additional testing after removing poor pile top concrete, tests by another method such as a high strain dynamic load test method, excavation around the pile, core drilling, etc.

APPENDIX G

**CROSSHOLE SONIC LOGGING RESULTS
GLOBAL RAUSCHE LIKINS AND ASSOCIATES,
INC. (GRL)**

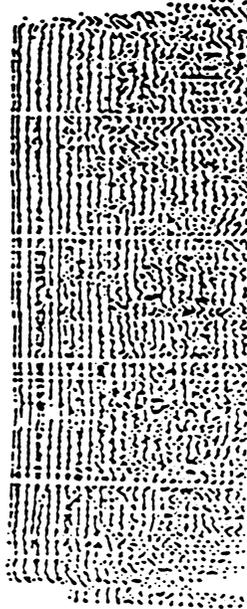
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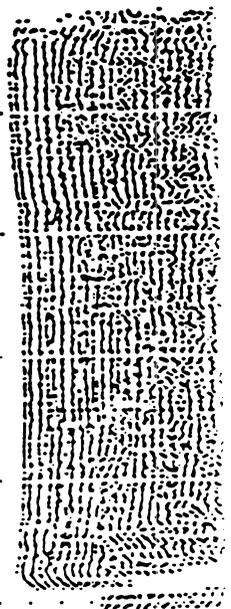
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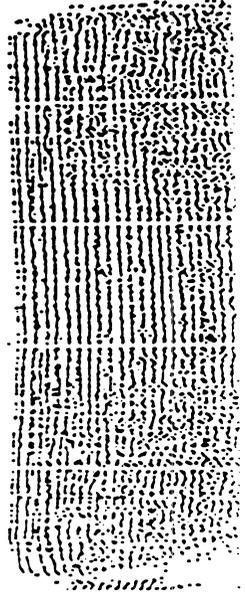
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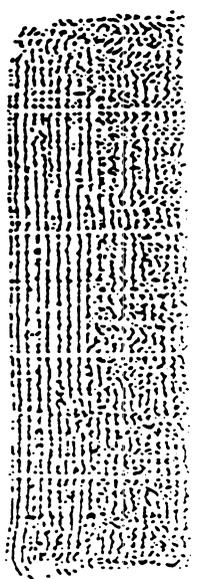
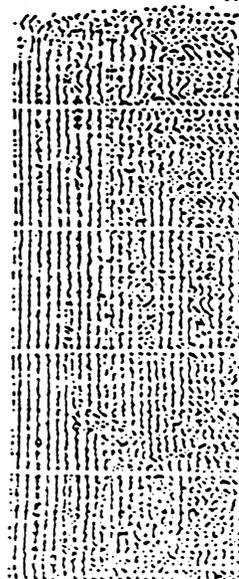
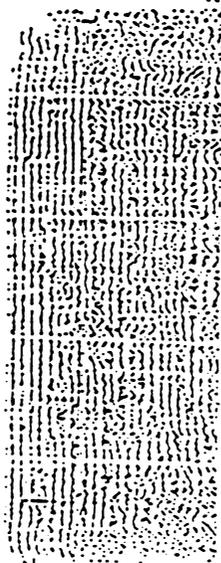
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Y 25

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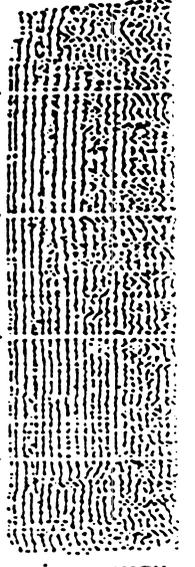
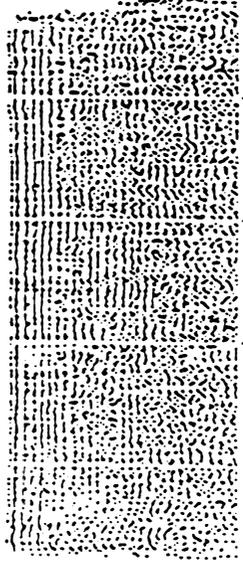
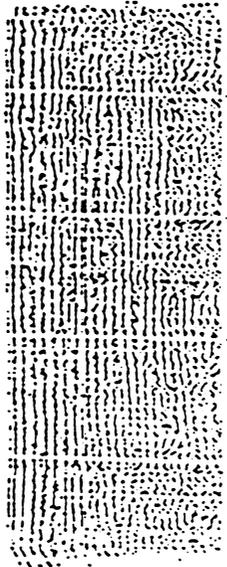
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APPENDIX H

CONCRETE CORE TEST RESULTS

WILLIAMS EARTH SCIENCE INC.

**LAW ENGINEERING &
ENVIRONMENTAL SCIENCE**



PROJECT NAME: FAU DRILLED SHAFT STUDY

PROJECT Nº: J327094B

CLIENT: FLORIDA ATLANTIC UNIV.

BORING Nº: C-1

INSPECTOR: Lewis Grace

ENGINEER Mark Hirsch

START DATE: 03/26/99

START TIME: _____

RIG Nº DR-4

FINISH DATE: 03/26/99

FINISH TIME: _____

BIT TYPE: Core Barrel

METRIC _____

BIT SIZE: 2 7/8

LOCATION Nº N/A

WATER LEVEL: N/A

OFFSET: N/A

WATER DEPTH: N/A

TOTAL CASING: N/A

ELEVATION: N/A

MUDLINE: N/A

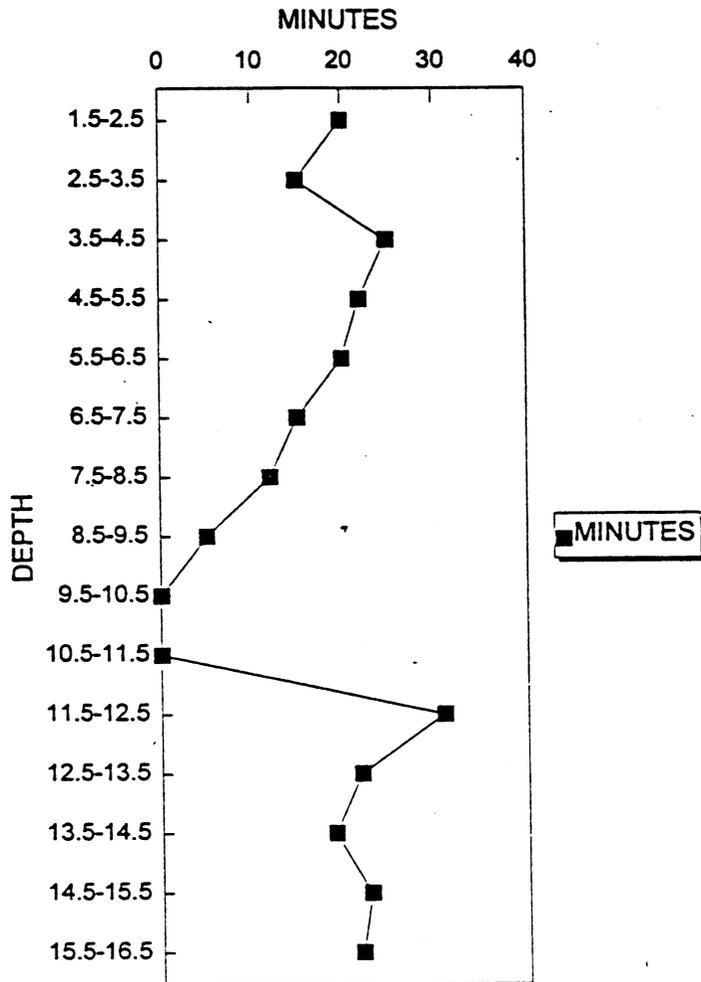
TOTAL GROUT: N/A

GROUNDWATER LEVEL: N/A

GROUT TIME: N/A

Sample		Classification
From	To	
0.0		CONCRETE CUT INTO PILE
	1.5	
1.5		250 P.S.I. = 20 min.
	2.5	
2.5		250 P.S.I. = 15 min.
	3.5	
3.5		250 P.S.I. = 25 min.
	4.5	
4.5		250 P.S.I. = 22 min. (100 % RECOVERY)
	5.5	
5.5		250 P.S.I. = 20 min.
	6.5	
6.5		250 P.S.I. = 15 min.
	7.5	
7.5		250 P.S.I. = 12 min.
	8.5	
8.5		250 P.S.I. = 5 min.
	9.0	Drop into other core hole, Kelly stopped at 11.5ft.
11.5		250 P.S.I. = 31 min.
	12.5	
12.5		250 P.S.I. = 22 min.
	13.5	
13.5		250 P.S.I. = 19 min.
	14.5	
14.5		250 P.S.I. = 23 min.
	15.5	
15.5		250 P.S.I. = 22 min. (75 % RECOVERY)
	16.5	

C-1 SHAFT 5.3
DRILLING TIME





PROJECT NAME: FAU DRILLED SHAFT STUDY

PROJECT N^o: J327094B

CLIENT: FLORIDA ATLANTIC UNIV.

BORING N^o: C-2

INSPECTOR: Lewis Grace

ENGINEER Mark Hirsch

START DATE: 03/29/99

START TIME: _____

RIG N^o DR-4

FINISH DATE: 03/29/99

FINISH TIME: _____

BIT TYPE: Core Barrel

METRIC

BIT SIZE: 2 7/8

STATION N^o N/A

WATER LEVEL: N/A

OFFSET: N/A

WATER DEPTH: N/A

TOTAL CASING: N/A

ELEVATION: N/A

MUDLINE: N/A

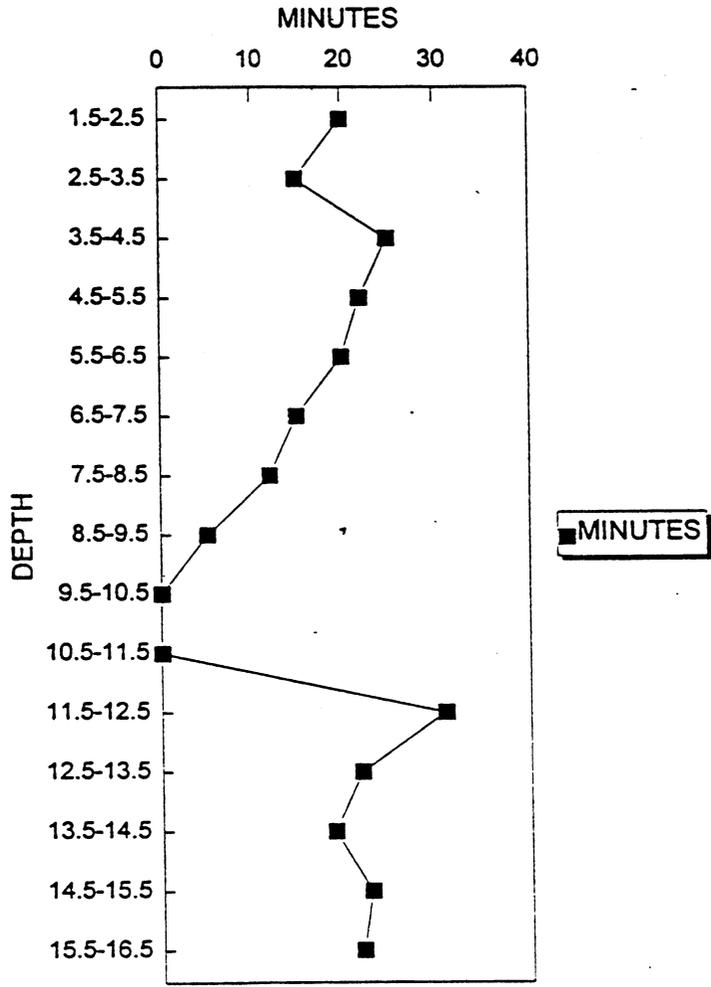
TOTAL GROUT: N/A

GROUNDWATER LEVEL: N/A

GROUT TIME: N/A

Sample		Classification
From	To	
0.0		FILL IN PILE AT 5 ft.
	5.0	
5.0		250 P.S.I. = 29 min.
	6.0	
6.0		250 P.S.I. = 18 min.
	7.0	
7.0		250 P.S.I. = 37 min.
	8.0	
8.0		250 P.S.I. = 26 min.
	9.0	
9.0		250 P.S.I. = 26 min. (100 % RECOVERY)
	10.0	
10.0		250 P.S.I. = 39 min.
	11.0	
11.0		250 P.S.I. = 15 min.
	12.0	
12.0		250 P.S.I. = 23 min.
	13.0	
13.0		250 P.S.I. = 25 min.
	14.0	
14.0		250 P.S.I. = 42 min. (90 % RECOVERY)
	15.0	
		HIT STEEL AT 15ft AND STOPPED

C-2 SHAFT 5.2
DRILLING TIME





CORED SEGMENTS

H-5

LABORATORY MEASUREMENTS

SHAFT 1-2

	<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
Depth, ft.	2.5	12
Diameter, in.	3.276	3.29
Core Length, in.	8.125	10.375
Initial Weight, lbs.	5.596	7.308
Saturated Weight, lbs	5.662	7.4
Dry Weight, lbs.	5.228	6.894
Core Volume, ft ³	0.0396	0.0510
Absorbed Water, ft ³	0.0070	0.0081
Saturated Unit Weight, lbs/ft ³	142.90	145.00
Dry Unit Weight, lbs/ft ³	132.02	135.18
Porosity, %	17.55	15.89

SHAFT 1-3

	<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
Depth, ft.	8	11
Diameter, in.	3.281	3.289
Core Length, in.	8.125	11.875
Initial Weight, lbs.	5.726	8.332
Saturated Weight, lbs	5.8	8.432
Dry Weight, lbs.	5.418	7.856
Core Volume, ft ³	0.0398	0.0584
Absorbed Water, ft ³	0.0061	0.0092
Saturated Unit Weight, lbs/ft ³	14.90	144.40
Dry Unit Weight, lbs/ft ³	136.29	134.55
Porosity, %	15.40	15.81

SHAFT 2-2

	<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
Depth, ft.	2.5	6
Diameter, in.	3.285	3.282
Core Length, in.	8.875	10
Initial Weight, lbs.	6.144	6.826
Saturated Weight, lbs	6.216	6.918
Dry Weight, lbs.	5.716	6.324
Core Volume, ft ³	0.0435	0.0490
Absorbed Water, ft ³	0.0080	0.0095
Saturated Unit Weight, lbs/ft ³	142.80	141.30
Dry Unit Weight, lbs/ft ³	131.31	129.17
Porosity, %	18.41	19.44

SHAFT 4-2

	<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
Depth, ft.	2	9
Diameter, in.	3.266	3.268
Core Length, in.	10	11
Initial Weight, lbs.	6.816	7.542
Saturated Weight, lbs	6.916	7.632
Dry Weight, lbs.	6.36	7.086
Core Volume, ft ³	0.0485	0.0534
Absorbed Water, ft ³	0.0089	0.0087
Saturated Unit Weight, lbs/ft ³	142.70	142.90
Dry Unit Weight, lbs/ft ³	131.18	132.71
Porosity, %	18.38	16.39

SHAFT 4-3

	<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
Depth, ft.	2.5	6.5
Diameter, in.	3.273	3.269
Core Length, in.	9.5	12
Initial Weight, lbs.	6.604	8.046
Saturated Weight, lbs	6.696	8.166
Dry Weight, lbs.	6.172	7.452
Core Volume, ft ³	0.0463	0.0583
Absorbed Water, ft ³	0.0084	0.0114
Saturated Unit Weight, lbs/ft ³	144.80	140.10
Dry Unit Weight, lbs/ft ³	133.43	127.85
Porosity, %	18.15	19.63

ASTM C 642 TEST RESULTS

SAMPLE NUMBER	WEIGHT A	WEIGHT B	WEIGHT C	WEIGHT D	ABSORPTION AFTER IMMERSION	ABSORPTION AFTER IMMERSION & BOILING
A1	785.0	848.4	849.5	468.6	8.08%	8.22%
A2	779.3	830.1	832.4	466.0	6.52%	6.81%
A3	838.6	906.3	907.5	498.0	8.07%	8.22%
A4	845.6	915.8	917.7	504.7	8.30%	8.53%
A5	837.9	897.4	898.5	504.4	7.10%	7.23%

BULK SP GR DRY	BULK SP GR AFTER IMMERSION	BULK SP GR AFTER IMMERSION & BOILING	APPARENT SP GR	VOLUME OF PERMEABLE PORE SPACE (VOIDS)
2.06	2.23	2.23	2.48	16.93%
2.13	2.27	2.27	2.49	14.49%
2.05	2.21	2.22	2.46	16.83%
2.05	2.22	2.22	2.48	17.46%
2.13	2.28	2.28	2.51	15.38%

CORRELATION TABLES

GAMMA-GAMMA COUNTS PER SECOND	COMPRESSIVE STRENGTH (p.s.i.)
5000	5770
5000	5530
4700	5160
4650	6970
4550	3170

GAMMA-GAMMA COUNTS PER SECOND	ABSORPTION PERCENT
5010	8.2
4980	8.2
4700	6.8
4600	8.5
4550	7.2

GAMMA-GAMMA COUNTS PER SECOND	SPECIFIC GRAVITY
5010	2.46
4980	2.48
4700	2.49
4600	2.48
4550	2.51

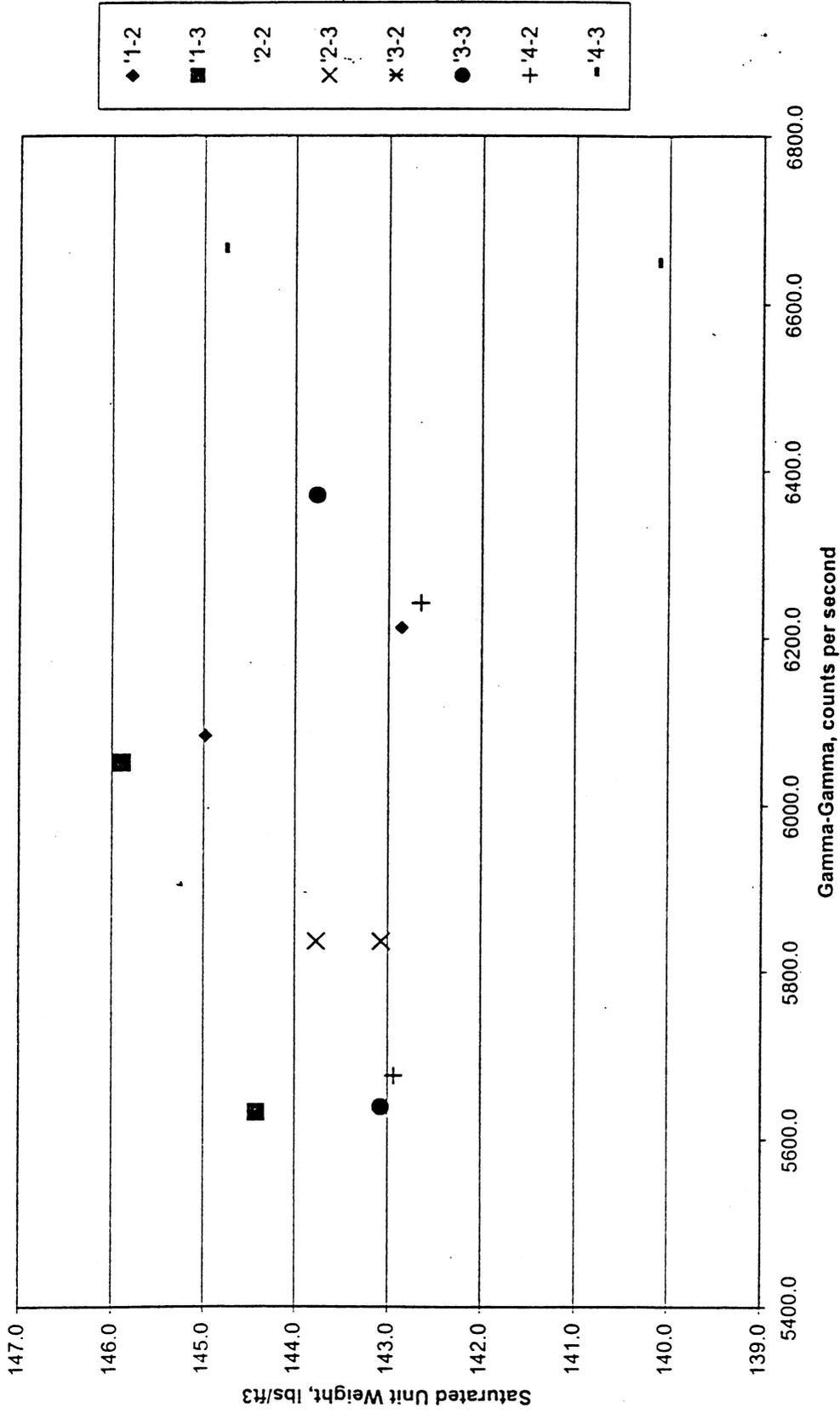
GAMMA-GAMMA COUNTS PER SECOND	PERCENT VOIDS
5010	16.8
4980	16.9
4700	14.5
4600	17.5
4550	15.4

NEUTRON-NEUTRON COUNTS PER SECOND	COMPRESSIVE STRENGTH (p.s.i.)
390	3170
380	6970
360	5530
350	5770
350	5160

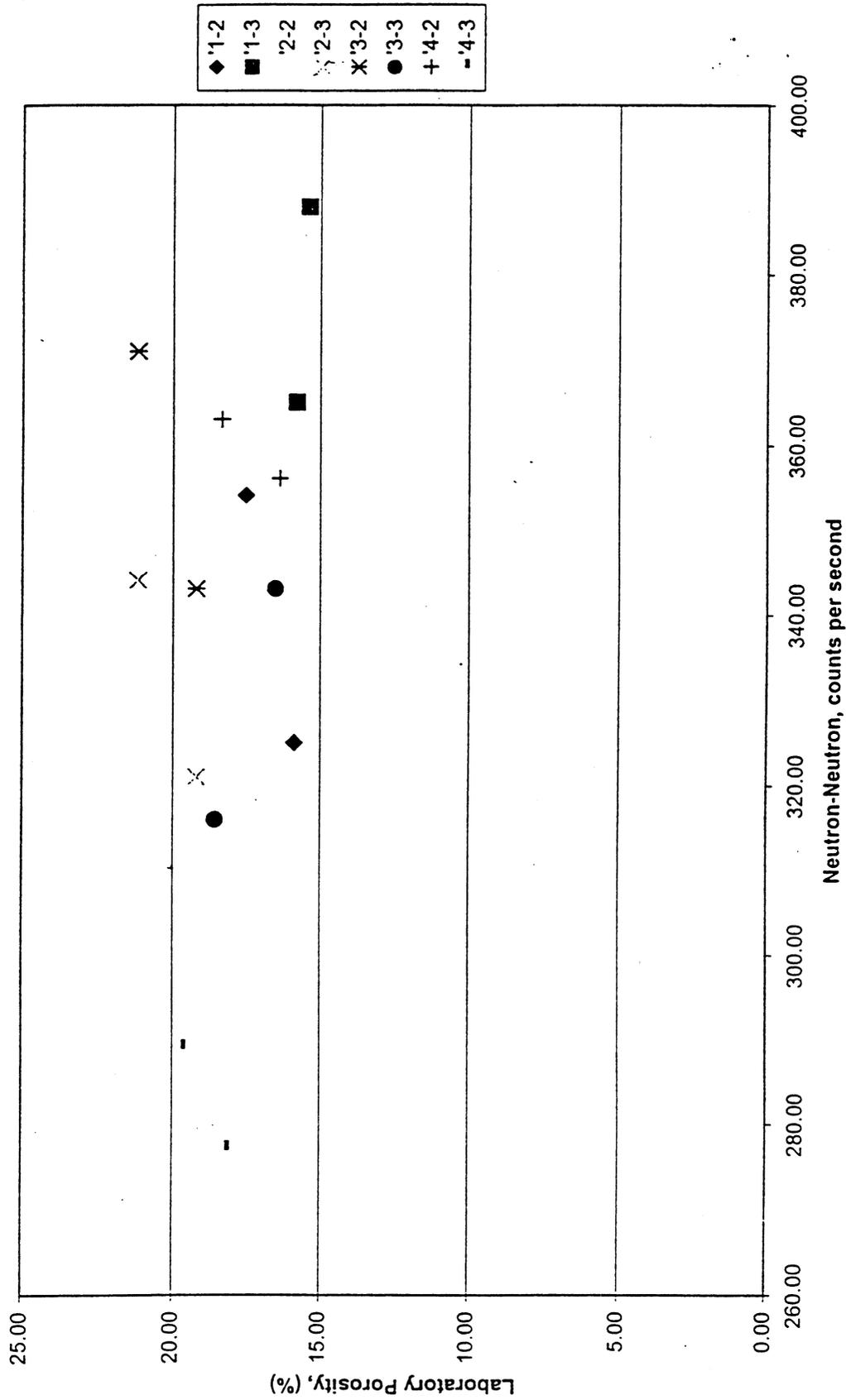
NEUTRON-NEUTRON COUNTS PER SECOND	ABSORPTION PERCENT
400	6.8
400	7.2
360	8.5
330	8.2
200	8.2

NEUTRON-NEUTRON COUNTS PER SECOND	SPECIFIC GRAVITY
400	2.51
400	2.49
360	2.48
330	2.46
200	2.48

NEUTRON-NEUTRON COUNTS PER SECOND	PERCENT VOIDS
400	14.5
400	15.4
360	17.5
330	16.8
200	16.9



GRAPH 1. Laboratory Saturated Unit Weight vs. Gamma-Gamma



GRAPH 2. Laboratory Porosity vs. Neutron-Neutron

SHAFT ID	SAMPLE No.	DEPTH (ft)	LABORATORY SATURATED UNIT WEIGHT (lbm)	GAMMA-GAMMA counts per second	LABORATORY POROSITY (%)	NEUTRON-NEUTRON counts per second
1-2	1	2.5	145.0	6212.0	17.55	354.00
	2	12	145.0	6063.0	15.89	325.00
1-3	1	8	145.9	6051.0	15.40	388.00
	2	11	144.4	5832.0	15.81	365.00
2-2	1	2.5	142.8	6238.0	16.41	350.00
	2	6	141.3	5802.0	19.44	378.00
2-3	1	3.5	141.0	5838.0	19.22	321.00
	2	7.5	139.8	5836.0	21.17	344.00
3-2	1	2.5	143.8	6370.0	18.59	343.00
	2	13	143.1	5839.0	18.54	371.00
3-3	1	3	142.1	6444.0	19.37	318.00
	2	12.5	143.5	5864.0	16.30	343.00
4-2	1	2	142.7	6241.0	18.38	383.00
	2	9	142.9	5678.0	16.38	358.00
4-3	1	2.5	144.8	6662.0	18.15	277.00
	2	6.5	140.1	6846.0	19.63	289.00

TABLE 1. Summary of Laboratory Saturated Unit Weight and Laboratory Porosity Including Gamma-Gamma and Neutron-Neutron field values.

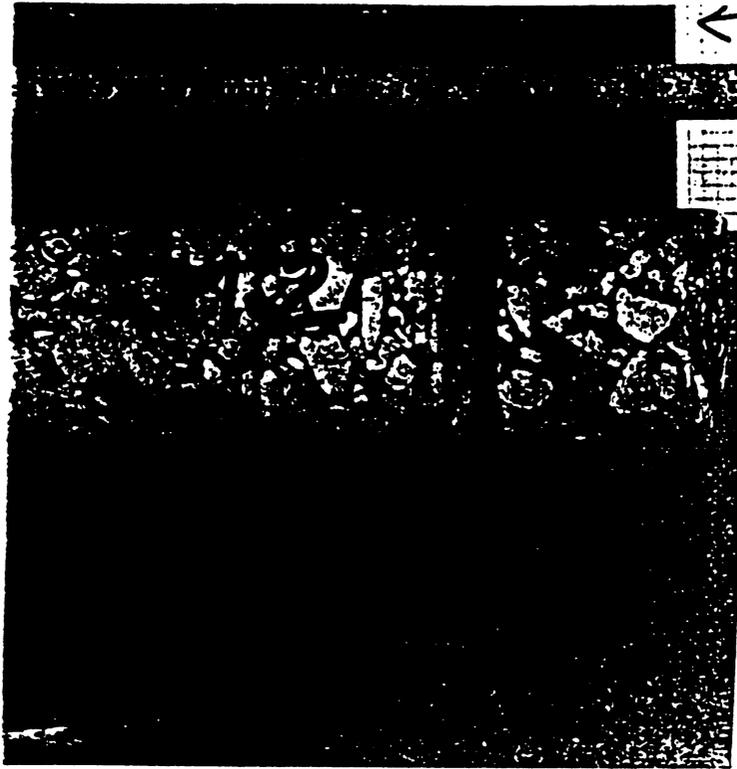


Photo No.3 Drilled Shaft 1-2 (Top Row). Drilled Shaft 1-3 core used for compression test.
Core Depth = 4.5-5 ft.



Photo No.4 Drilled Shaft 1-2 (Top Row), Drilled Shaft 1-3 (Bottom Row). Core Depth = 5-7 ft.

TABLE 4. Visual Observation Drilled Shaft 2-2

DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	Sign of aggregate segregation on the first 4-1/2-5 feet. Air voids are well distributed and ranging in size from 1/16" to 3/8". No sign of honeycomb.
5 - 10	Aggregates are well distributed with some small air void, ranging in size from 1/16" to 1/8". No sign of honeycomb or aggregate segregation.
10 - 15	Aggregates are well distributed with some small air void, ranging in size from 1/16" to 1/8". No sign of honeycomb or aggregate segregation

TABLE 5. Visual Observation Drilled Shaft 2-3

DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	Sign of aggregate segregation on the first 1-2 foot. Presence of honeycombing at a depth of 4-5 feet.
5 - 10	Aggregates appear well distributed. There is well distribution of air voids and no sign of honeycomb or aggregate segregation.
10 - 15	Aggregates are well distributed, no sign of honeycomb. Some air voids ranging in size from 1/16" to 1/8".

TABLE 6. Visual Observation of Drilled Shaft 3-2

DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	Sign of aggregate segregation on the first 1 foot. Sign of honeycomb on the first 2 feet.
5 - 10	Aggregates are well distributed with some small air void, ranging in size from 1/16" to 1/8". Segregation observed within the first two feet.
10 - 15	Aggregates are well distributed with some small air void in the aggregates. No sign of honeycomb or aggregate segregation

TABLE 7. Visual Observation of Drilled Shaft 3-3

DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	No sign of aggregate segregation or honeycomb. Small air voids were observed in the aggregates.
5 - 10	Segregation observed within the first two feet. Aggregates are poorly distributed. Some air voids are present, ranging in size from 1/16" to 1/8". No sign of honeycomb.
10 - 15	Aggregate and air voids are well distributed, no sign of honeycomb or aggregate segregation was observed.

TABLE 8. Visual Observation of Drilled Shaft 4-2

DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	No sign of aggregate segregation or honeycomb. A small amount of air voids up to 1/8" in size were observed.
5 - 10	No sign of aggregate segregation or honeycomb. A small amount of air voids up to 1/8" in size were observed
10 - 15	No sign of aggregate segregation or honeycomb. A small amount of air voids up to 1/8" in size were observed.

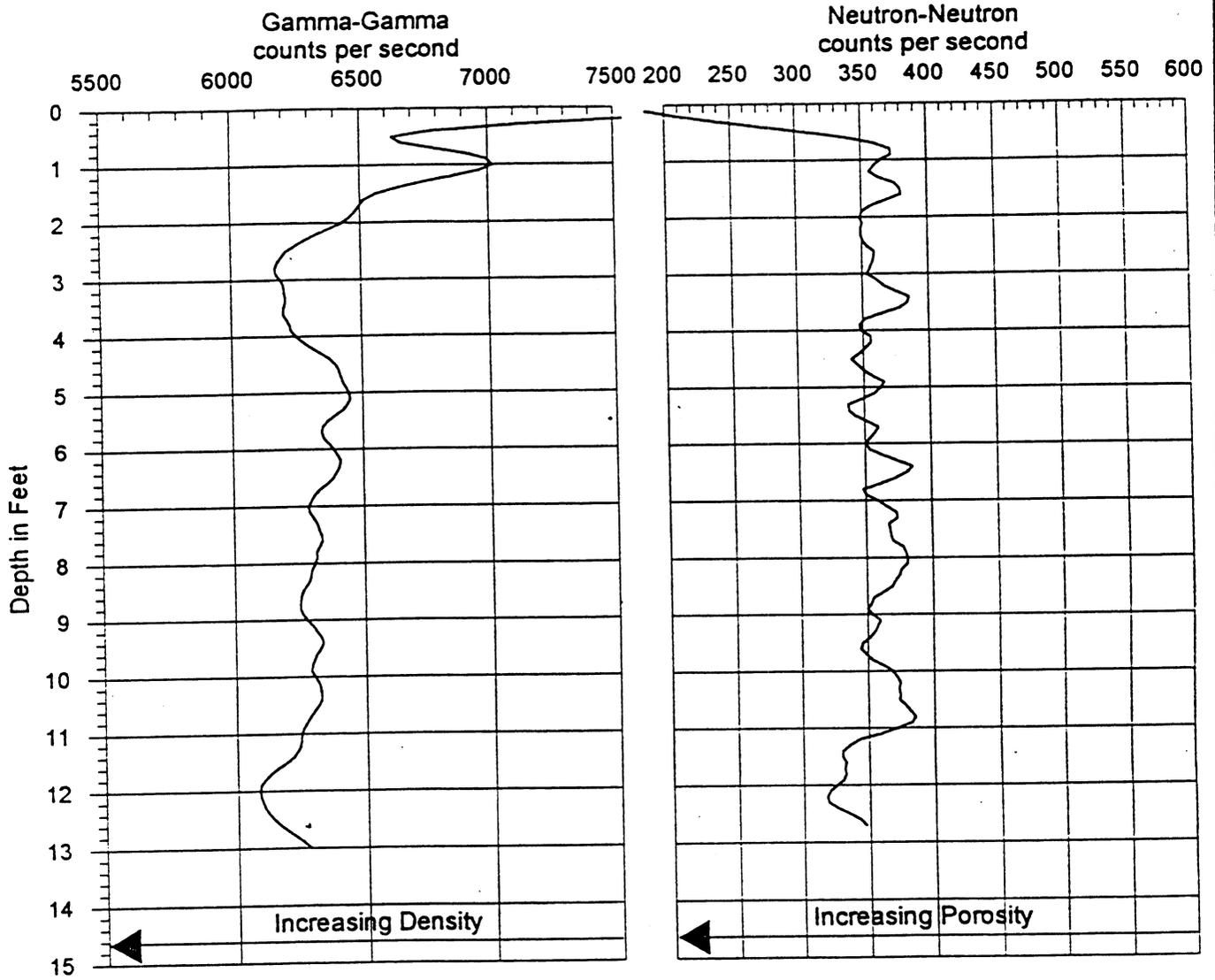
TABLE 9. Visual Observation of Drilled Shaft 4-3

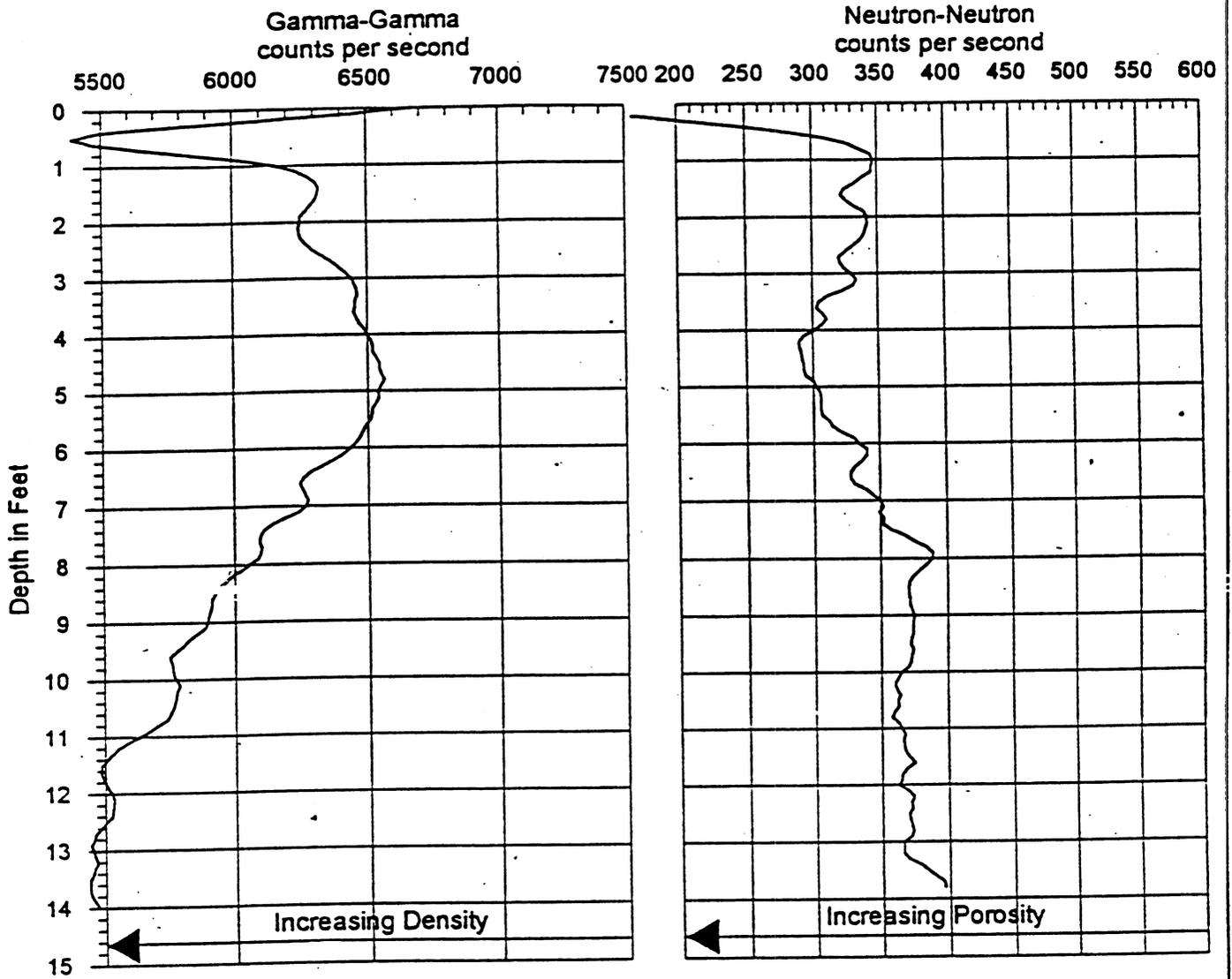
DEPTH (FT)	CONCRETE DESCRIPTION
0 - 5	Sign of aggregate segregation and honeycomb was observed within the first two feet. Air voids range in size from 1/16" to 1/8".
5 - 10	No aggregate segregation was observed. Some air voids are present, ranging in size from 1/16" to 1/8". Presence of honeycomb was not observed.
10 - 15	No aggregate segregation was observed. Some air voids are present, ranging in size from 1/16" to 1/8". Presence of honeycomb was not observed.

APPENDIX I

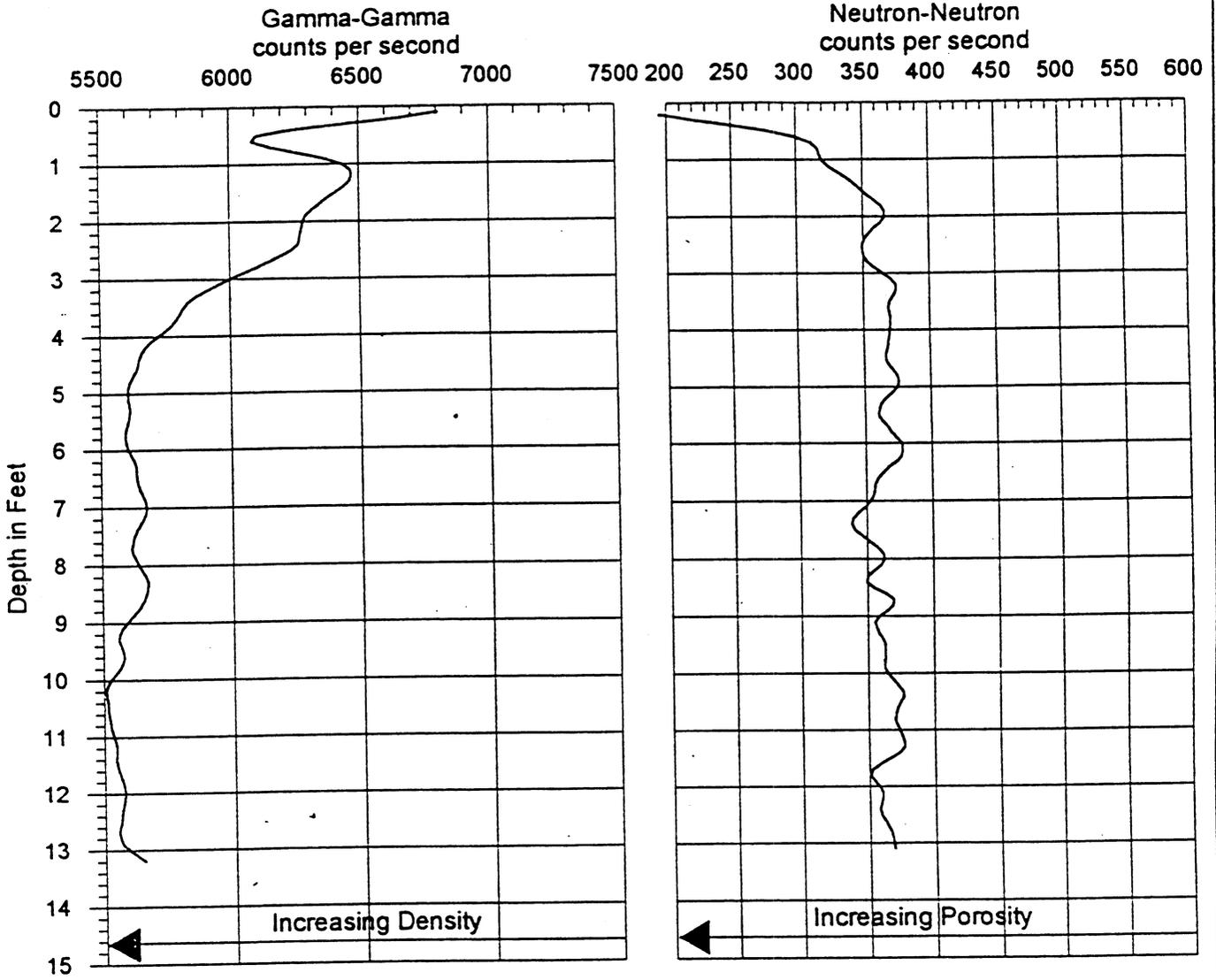
GEOPHYSICAL LOGGING

**γ - γ DENSITY AND NEUTRON -NEUTRON
POROSITY**

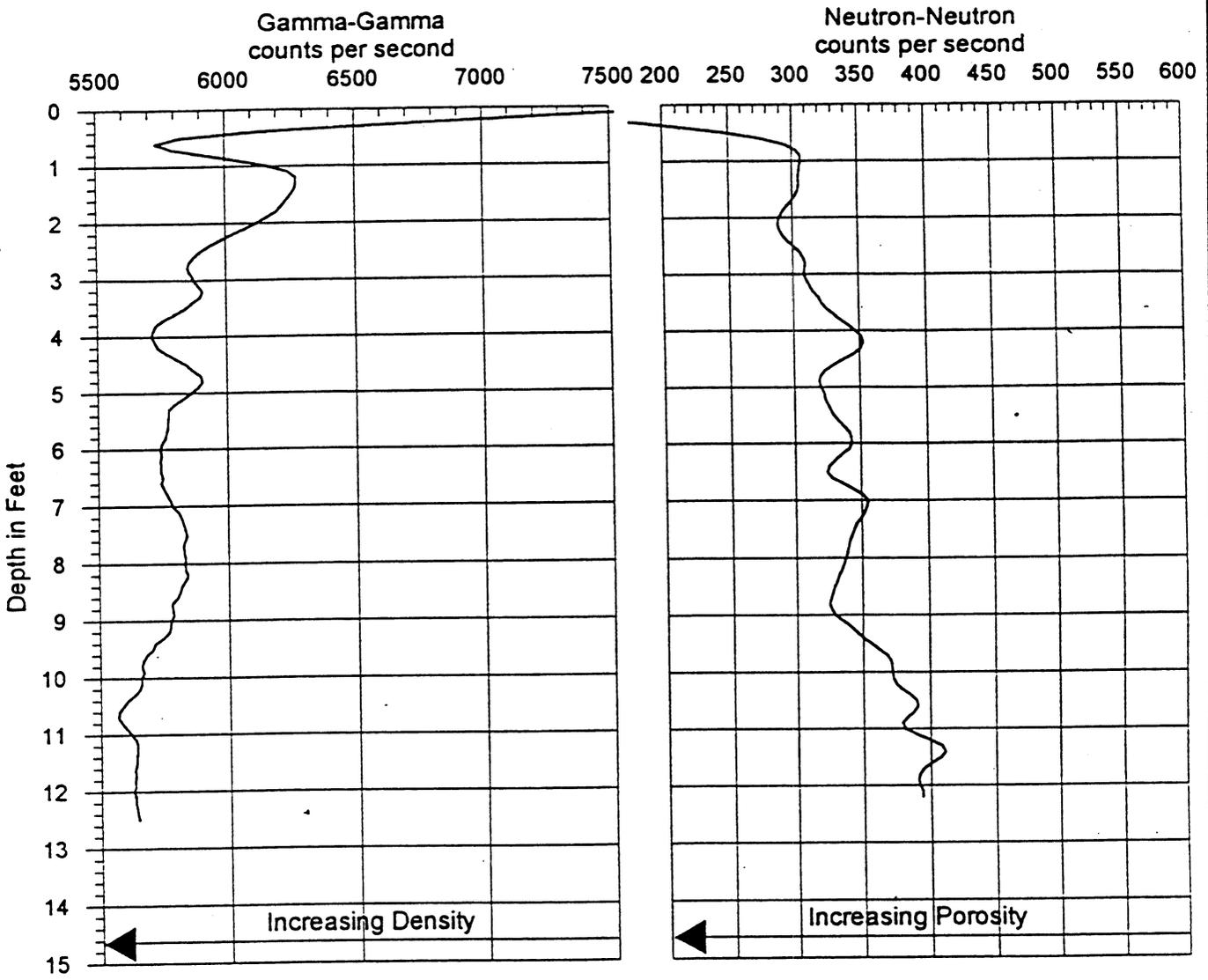




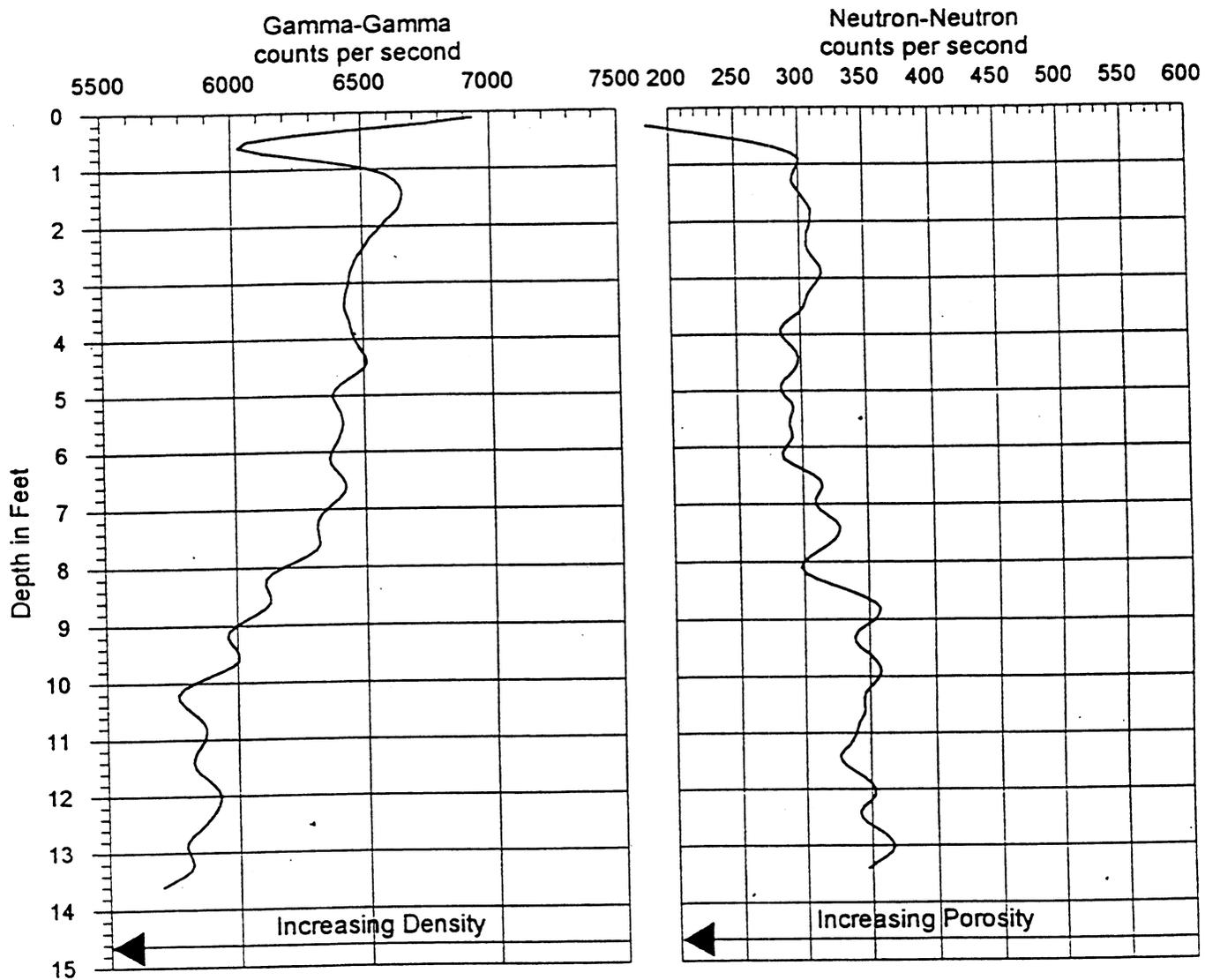
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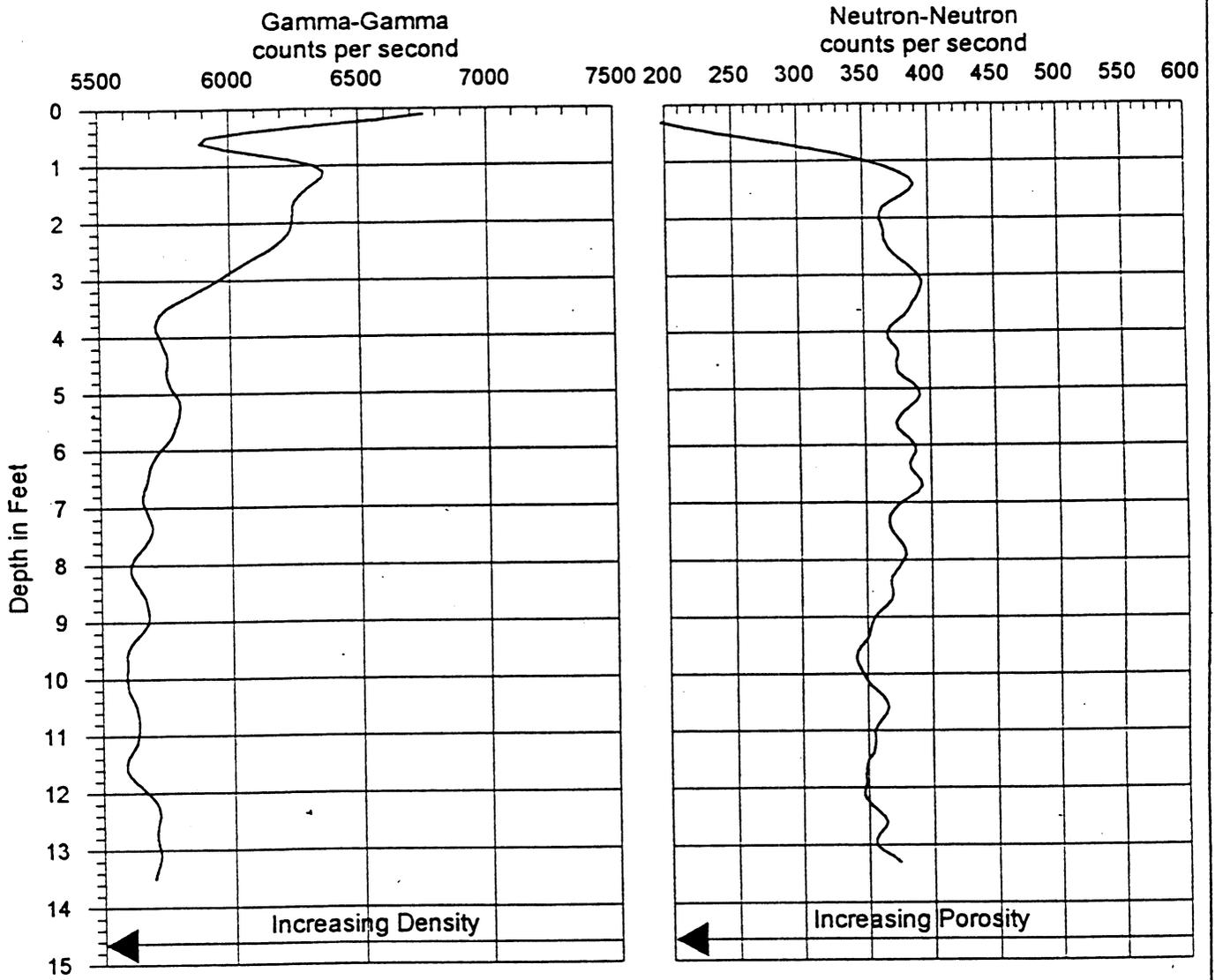
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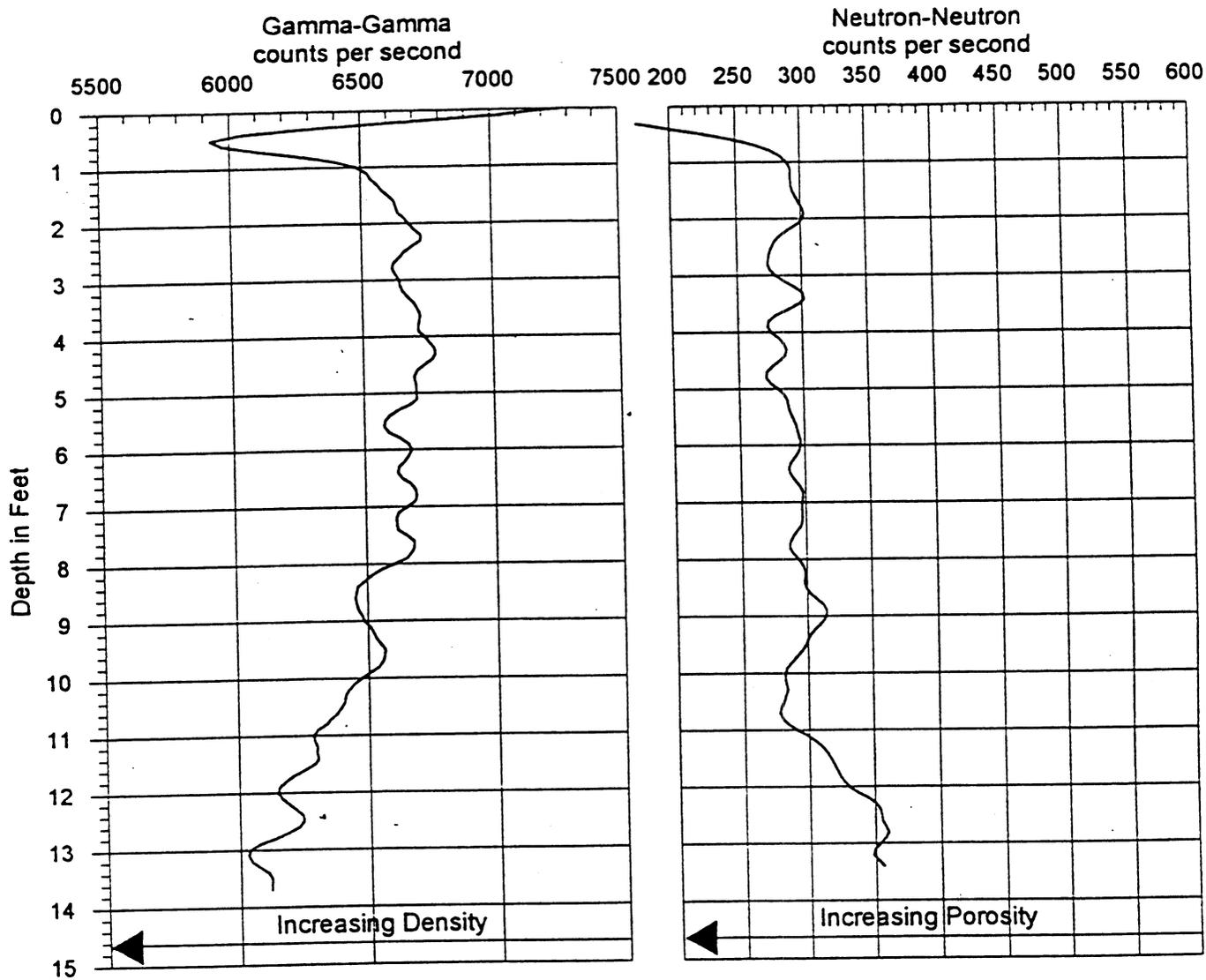
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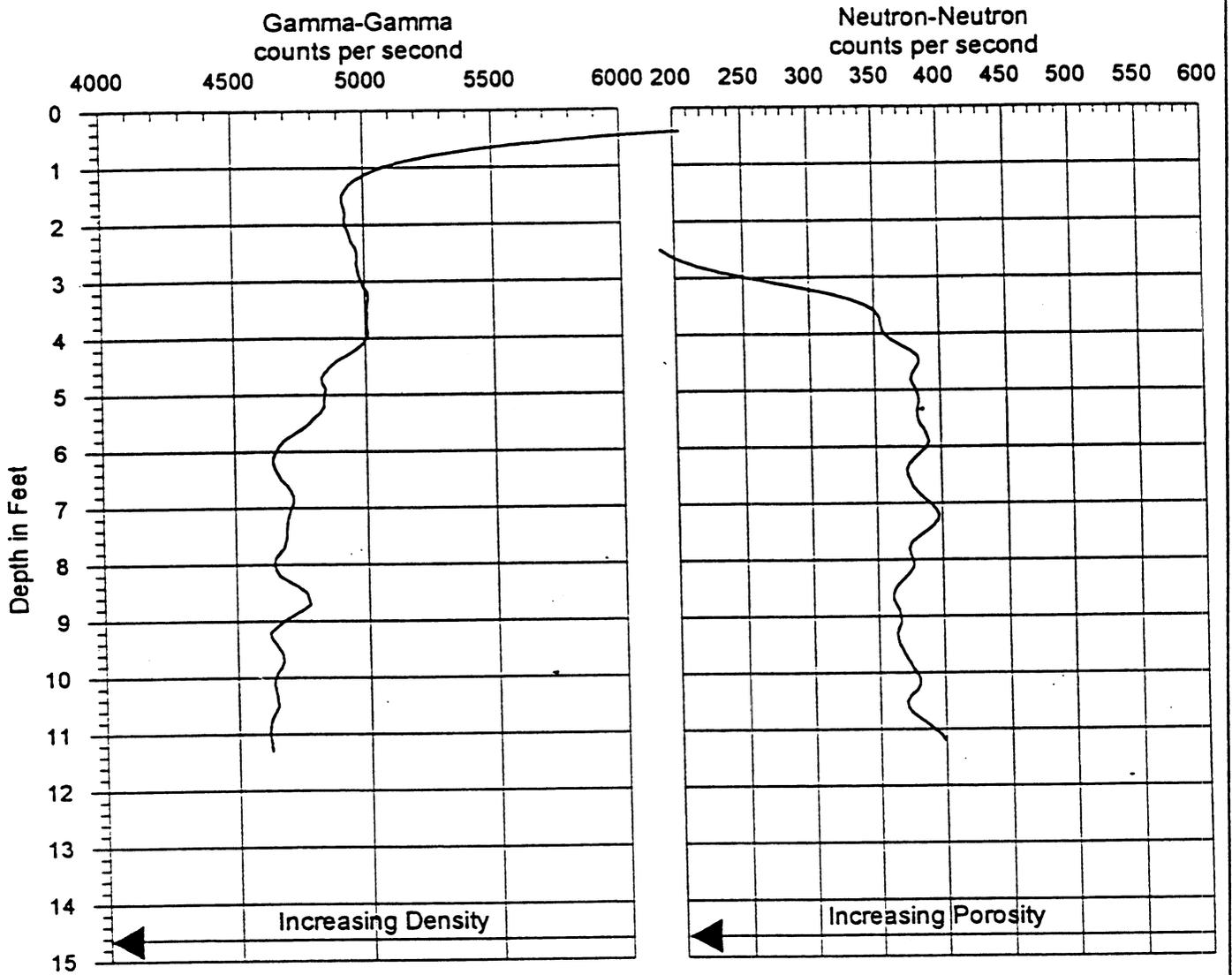
4.2



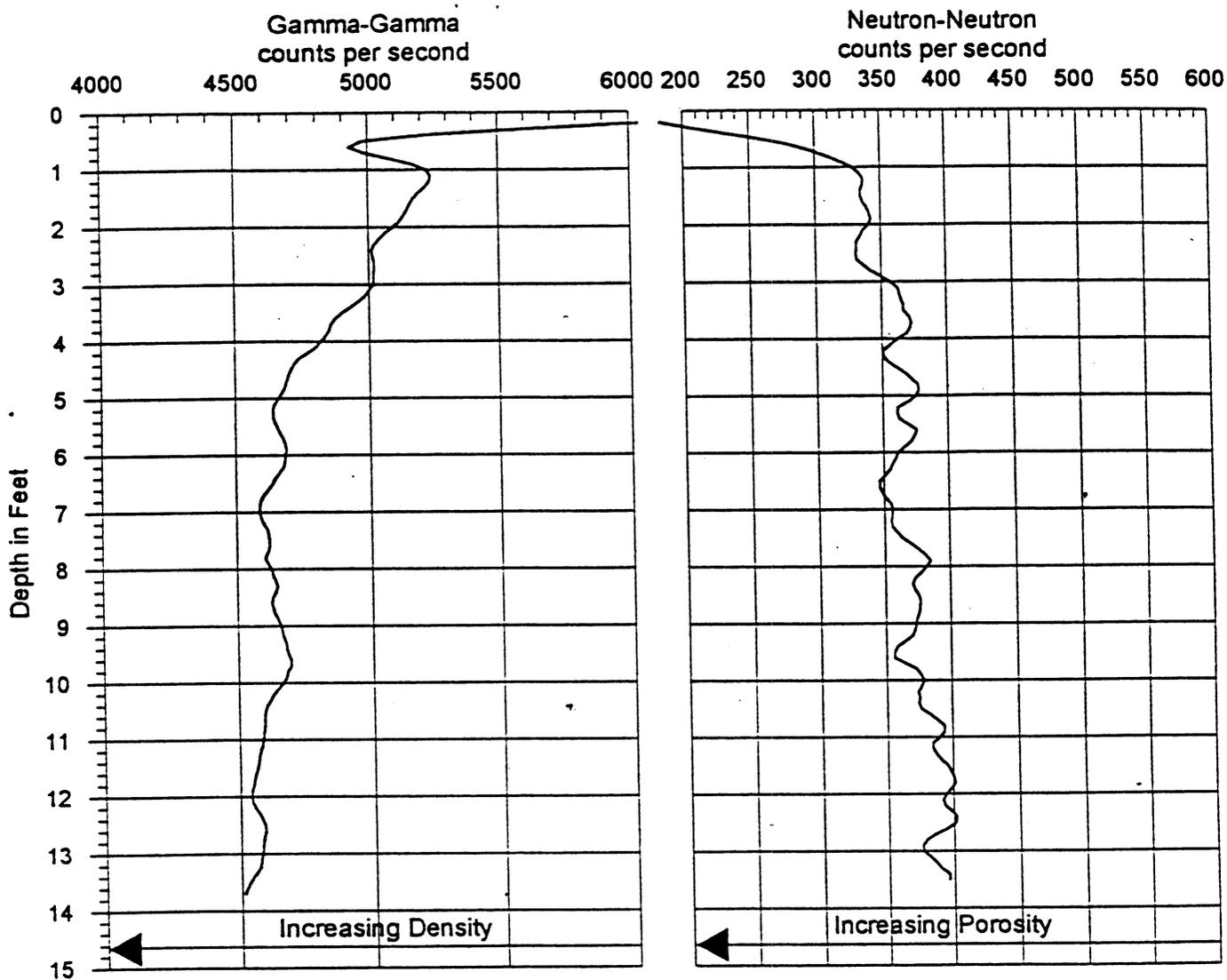
4.3



5.2



5.3



Appendix J

LAW Engineering and Environmental Services, Inc.

July 15, 1999

Mr. Tom Pierro
Florida Atlantic University
Department of Ocean Engineering
Building 36
7777 Glades Road
Boca Raton, Florida 33431

**RE: Report of Pile Driver Noise Impact and Insertion Loss Analysis
Florida Atlantic University
Boca Raton, Florida
LAW Project No. 40750-9-1988**

Dear Mr. Pierro:

Law Engineering and Environmental Services, Inc. (LAWGIBB) is pleased to present the Florida Atlantic University with the results of the Pile Driver Noise Impact and Insertion Loss Analysis conducted on July 8-9, 1999 at the Florida Atlantic University Campus in Boca Raton. Our services were conducted in general accordance with our proposal number 7248-sf dated June 1, 1999. The purpose of this analysis was to establish the noise impact associated with diesel hammer pile driving activities and to determine the insertion loss (noise reduction) associated with the installation of an acoustical blanket at distances of fifty feet, one hundred feet, and three hundred feet.

Noise Analysis Methodology

For the purposes of this analysis, an array of Quest 2900 Type II Sound Level Meters were installed at an angle from the diesel hammer pile driver as depicted on the attached figure. The sound level meters were placed at the following distances from the diesel hammer pile driver: 50 feet, 100 feet, and 300 feet. The sound level meters on the west leg of the array were installed on PVC piping supported by a concrete base at a height of approximately 15 feet above ground level. The sound level meters on the east leg of the array were held by field personnel at a height of 5 feet above ground level. A CEL 593 Real Time One Third Octave (OTO) Band Analyzer was placed at a distance of 100 feet from the diesel hammer pile driver at a location between both of the legs as depicted on the attached figure. This sound level meter was held by field personnel at a height of five feet above ground level. The sound level meters were calibrated prior to the commencement of the pile driving activities. Upon completion of the set up and installation of the sound level meters, field noise monitoring was conducted to establish the noise impact associated with the diesel hammer pile driver. The purpose of this phase of the analysis was to evaluate the magnitude of the noise impact prior to the installation of an acoustical blanket. The next phase of the analysis was to conduct field noise monitoring with the acoustical blanket in place. The purpose of this phase of the analysis was to evaluate the magnitude of the insertion loss (noise reduction) associated with the acoustical.

Noise Analysis Results

The following table depicts the noise impact associated with the pile driver and the insertion loss (noise reduction) associated with the acoustical blanket. The field noise monitoring data obtained from the Quest 2900 Sound Level Meters and the CEL 593 Real Time Octave Band Analyzer is attached.

Date	Location	SLM	Distance From Source	Leq (dBA)* w/o Blanket	Leq (dBA)* w/ Blanket	Insertion Los. (dBA)*
7/8/99	West Leg	Quest 2900	50 Ft./ 15 Ft.	84.9	74.5	10.4
7/9/99	West Leg	Quest 2900	50 Ft./ 15 Ft.	85.4	73.5	11.9
7/9/99	West Leg	Quest 2900	100 Ft./15Ft.	78.5	70.1	8.4
7/9/99	West Leg	Quest 2900	100 Ft./15Ft.	79.4	68.8	10.6
7/9/99	West Leg	Quest 2900	300 Ft./15Ft.	73.3	64.5	8.8
7/9/99	West Leg	Quest 2900	300 Ft./15Ft.	74.0	62.8	11.2
7/9/99	East Leg	Quest 2900	50 Ft. / 5 Ft.	91.4	82.4	9.0
7/9/99	East Leg	Quest 2900	50 Ft. / 5 Ft.	89.7	79.7	10.0
7/9/99	East Leg	Quest 2900	100 Ft./ 5 Ft.	81.6	74.8	6.8
7/9/99	East Leg	Quest 2900	100 Ft./ 5 Ft.	81.4	71.7	9.7
7/9/99	East Leg	Quest 2900	300 Ft./ 5 Ft.	76.3	69.9	6.4
7/9/99	East Leg	Quest 2900	300 Ft./ 5 Ft.	75.7	70.1	5.6
7/8/99	Center	CEL 593	100 Ft. / 5Ft.	90.8	88.7	2.1
7/9/99	Center	CEL 593	100 Ft. / 5Ft.	88.9	85.5	3.4

* Leq is defined as the hourly average sound level over the entire monitoring period measured in decibels on the A weighted scale (dBA).

Noise Analysis Conclusions

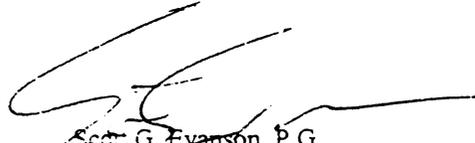
The results of this analysis indicate that the noise impacts associated with the diesel hammer pile driver will require the implementation of noise abatement measures for construction activities adjacent to noise sensitive locations such as residential dwellings, hotels, and any other locations where noise impacts may disrupt normal human activity. Based upon the results of this analysis, noise abatement measures should be considered for noise sensitive locations within three hundred feet of the construction site where diesel hammer pile driving activities are to take place. One feasible on-site noise abatement measure is installing an acoustical blanket on the piledriver assembly. The results of the acoustical blanket utilized in this study indicated that the insertion loss at fifty feet was approximately 10 – 12 dBA at a height of 15 feet while the insertion loss at the same distance at a height of 5 feet was approximately 9 – 10 dBA. The insertion loss at one hundred feet was approximately 8 – 11 dBA at a height of 15 feet while the insertion loss at the same distance at a height of 5 feet was approximately 7 – 10 dBA. Finally the insertion loss at three hundred feet was approximately 9 – 11 dBA at a height of 15 feet while the insertion loss at the same distance at a height of 5 feet was approximately 6 dBA. The results of the real time octave band analysis at a distance of one hundred feet at a height of 5 feet indicated that the insertion loss was approximately 2 – 3 dBA. At this location, the noise monitoring point was directly within the line of sight of the pile driver. While the acoustical blanket provided a respectable level of noise reduction, modifications would be required to provide the optimum noise reduction. The acoustical blanket under study was assembled in ten foot panels and then attached to a metal frame. The assembly time (approximately 3 hours) was lengthy enough to potentially cause delays in the contractor's schedule. Also, the ten foot panels left gaps within the acoustical blanket assembly that likely reduced the insertion loss achieved. Therefore, the following modifications should be considered: single blanket construction with no gaps, cylindrical frame construction to fit around the pile driver, and an enclosure over the top of the pile driver. These modifications should also reduce the on-site installation time, which will be of benefit to the construction contractor.

It has been a pleasure serving you on this project. Should you have any questions, please feel free to contact us at (561) 742-0871.

Very Truly Yours,



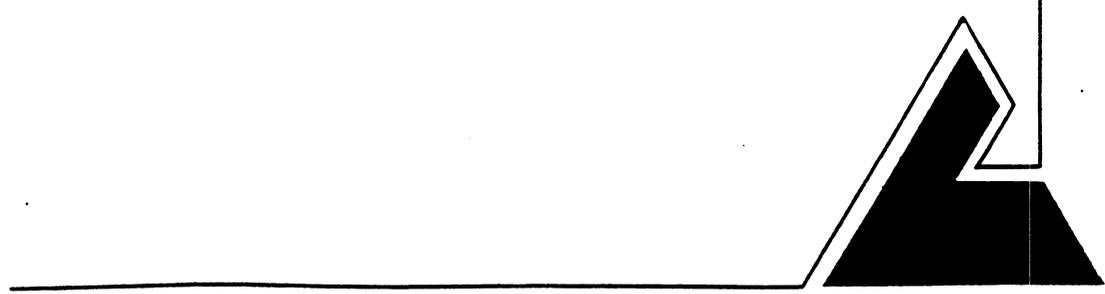
Bernard Kinney Jr.
Senior Acoustical Engineer
Environmental Noise Control

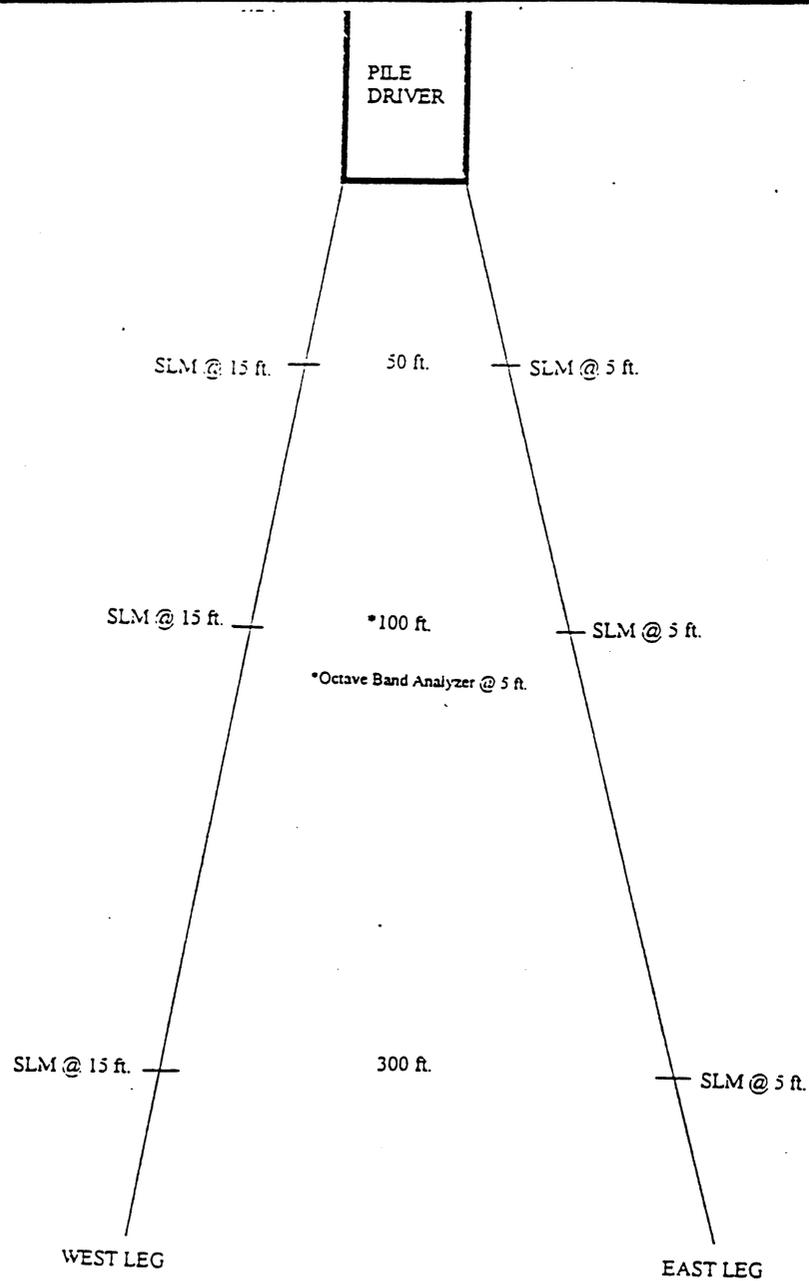


Scott G. Evanson, P.G.
Principal Geologist

Attachments: Figure 1 – Noise Monitoring Locations
 Eastern Leq Quest 2900 Field Data
 Western Leq Quest 2900 Field data
 Octave Band analyzer Data Sheets

Figure 1
Noise Monitoring Locations





SLM = Sound Level Meter

Source: Site Reconnaissance

Scale: Not to Scale

Figure 1

NOISE MONITORING LOCATIONS

Florida Atlantic University
Boca Raton, Florida

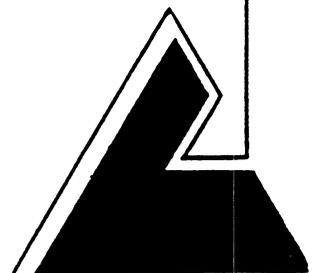
LAW Project No. 40750-9-1988

LAW
WGIBB Group Member



Law Engineering and Environmental Services, Inc.
2104 Corporate Drive
Boynton Beach, Florida 33426
(561) 742-0871
Fax 742-0875

Eastern Leq Quest 2900 Field Data



Quest Technologies
2900 Integrating/Logging Sound Level Meter

Unit Version Number: 02.2

Serial Number: CD7120036

Name: FAU ACOUSTIC PILE DRIVE ANALYSIS

Work Area: FIVE FEET HIGH - FIFTY FEET AWAY

Comments:

Meter Calibration: 114.0dB, 7/7/99 @ 11:39:30

Calibrator Serial Number:

Calibration Date:

Study 1

Measuring Parameters

Range: 60 - 120 dB
 Threshold: Off

Weighting: A
 Exchange Rate: 3dB

Time Constant: Fast
 Peak Weighting: A

Study Started: 7/8/99 @ 12:26:23

Study Stopped: 7/8/99 @ 12:30:07

Run Time: 0:03:43

Peak Level: 129.0dB, 7/8/99 @ 12:29:30

Max Level: 105.7dB, 7/8/99 @ 12:29:29

Min Level: 51.6dB, 7/8/99 @ 12:26:55

Overload: 0.31%

LEQ: 91.4dB

SEL(3): 114.9dB

TWA: 70.3dB

TAKM3: 100.2dB

L_{DN}: 91.4dB

CNEL: 91.4dB

Pa2Sec: 121.7

L5: 99.3dB

L10: 97.0dB

L50: 74.1dB

L90: 58.0dB

Study 1

Logging (15 Seconds)

	LEQ	LMAX	LPeak	LN10	LN90	Ovl
12:26:38	61.7dB	68.9dB	83.2dB	65.2dB	56.1dB	
2:26:53	72.1dB	84.5dB	97.7dB	75.8dB	57.5dB	
12:27:08	70.2dB	85.7dB	99.0dB	74.8dB	54.9dB	
12:27:23	71.0dB	80.1dB	97.2dB	74.6dB	60.8dB	
2:27:38	67.2dB	75.0dB	89.1dB	71.5dB	58.5dB	
12:27:53	62.2dB	71.1dB	86.8dB	65.5dB	56.2dB	
12:28:08	62.8dB	68.9dB	82.1dB	66.5dB	55.5dB	
2:28:23	88.2dB	103.9dB	125.5dB	91.0dB	65.5dB	*
12:28:38	94.5dB	103.4dB	125.4dB	99.3dB	78.6dB	*
12:28:53	94.6dB	104.0dB	125.7dB	99.7dB	78.5dB	*
2:29:08	95.5dB	105.3dB	127.2dB	100.5dB	78.2dB	*
12:29:23	94.2dB	104.3dB	125.1dB	99.1dB	76.5dB	*
12:29:38	94.3dB	105.7dB	129.0dB	99.2dB	75.7dB	*
2:29:53	94.6dB	104.0dB	126.1dB	99.4dB	76.8dB	*

Study 2

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 06:34:09
Study Stopped: 7/9/99 @ 06:41:30
Run Time: 0:07:20
Peak Level: 117.8dB, 7/9/99 @ 06:40:39
Max Level: 98.5dB, 7/9/99 @ 06:40:42
Min Level: 51.6dB, 7/9/99 @ 06:37:26
Overload: 0.00%

LEQ: 81.4dB	SEL(3):107.8dB	TWA: 63.2dB	TAKM3: 89.3dB
LDN: 91.4dB	CNEL: 91.4dB	Pa2Sec: 24.0	
L5: 89.5dB	L10: 82.6dB	L50: 66.3dB	L90: 57.7dB

Study 2

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
06:34:24	76.1dB	88.8dB	103.0dB	80.1dB	64.3dB	
06:34:39	71.3dB	80.7dB	94.0dB	76.0dB	60.6dB	
06:34:54	67.3dB	74.4dB	88.1dB	70.7dB	62.8dB	
06:35:09	65.6dB	74.9dB	85.6dB	69.0dB	59.3dB	
06:35:24	67.5dB	76.6dB	88.0dB	71.7dB	59.6dB	
06:35:39	67.6dB	75.2dB	87.0dB	71.2dB	61.7dB	
06:35:54	73.0dB	84.4dB	94.0dB	77.0dB	59.2dB	
06:36:09	72.7dB	86.1dB	97.3dB	76.3dB	58.0dB	
06:36:24	67.6dB	76.1dB	89.2dB	71.9dB	58.0dB	
06:36:39	72.3dB	85.8dB	96.3dB	76.8dB	58.6dB	
06:36:54	64.5dB	74.8dB	85.6dB	68.3dB	58.2dB	
06:37:09	62.9dB	74.8dB	83.0dB	65.0dB	57.1dB	
06:37:24	73.6dB	85.7dB	96.5dB	78.1dB	56.0dB	
06:37:39	58.3dB	73.3dB	83.2dB	60.5dB	51.8dB	
06:37:54	53.8dB	58.0dB	75.2dB	55.3dB	51.8dB	
06:38:09	60.6dB	67.3dB	81.3dB	63.0dB	53.7dB	
06:38:24	63.0dB	67.7dB	83.8dB	66.4dB	57.6dB	
06:38:39	67.7dB	69.6dB	83.1dB	68.8dB	66.6dB	
06:38:54	67.5dB	70.8dB	84.9dB	69.1dB	66.0dB	
06:39:09	67.0dB	69.5dB	82.5dB	68.3dB	63.1dB	
06:39:24	64.9dB	73.1dB	90.1dB	65.5dB	63.3dB	
06:39:39	78.0dB	90.9dB	106.7dB	81.3dB	63.4dB	
06:39:54	65.6dB	72.6dB	85.8dB	67.3dB	63.8dB	
06:40:09	86.9dB	95.5dB	114.7dB	91.8dB	72.0dB	
06:40:24	87.7dB	96.3dB	115.9dB	92.3dB	73.2dB	
06:40:39	88.9dB	97.7dB	117.8dB	93.8dB	72.9dB	
06:40:54	88.8dB	98.5dB	117.2dB	93.6dB	72.2dB	
06:41:09	89.1dB	98.1dB	117.3dB	94.6dB	71.1dB	
06:41:24	85.1dB	96.6dB	114.7dB	90.6dB	63.9dB	

Study 3

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 09:10:15
Study Stopped: 7/9/99 @ 09:14:18
Run Time: 0:04:03
Peak Level: 109.3dB, 7/9/99 @ 09:12:49
Max Level: 90.6dB, 7/9/99 @ 09:12:48
Min Level: 56.1dB, 7/9/99 @ 09:14:13
Overload: 0.00%

LEQ: 74.8dB	SEL(3):98.7dB	TWA: 54.1dB	TAKM3: 82.6dB
LDN: 74.8dB	CNEL: 74.8dB	Pa2Sec: 3.0	
L5: 82.1dB	L10: 80.4dB	L50: 65.9dB	L90: 61.3dB

Study 3

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
09:10:30	71.9dB	84.2dB	98.4dB	75.6dB	63.3dB	
09:10:45	71.2dB	83.5dB	95.8dB	74.3dB	63.6dB	
09:11:00	71.3dB	83.6dB	100.0dB	76.8dB	60.9dB	
09:11:15	62.9dB	67.3dB	82.2dB	64.8dB	61.2dB	
09:11:30	74.7dB	84.7dB	100.9dB	80.3dB	61.5dB	
09:11:45	77.3dB	84.1dB	99.5dB	82.0dB	66.5dB	
09:12:00	77.8dB	86.7dB	103.2dB	82.3dB	66.5dB	
09:12:15	77.4dB	86.6dB	103.6dB	81.9dB	66.1dB	
09:12:30	76.6dB	84.9dB	103.8dB	81.4dB	64.9dB	
09:12:45	77.3dB	84.8dB	103.4dB	82.2dB	64.5dB	
09:13:00	76.0dB	90.6dB	109.3dB	80.6dB	60.8dB	
09:13:15	61.6dB	65.0dB	81.4dB	62.3dB	60.9dB	
09:13:30	61.9dB	63.1dB	77.4dB	62.7dB	61.2dB	
09:13:45	73.0dB	86.8dB	104.7dB	77.2dB	61.2dB	
09:14:00	77.3dB	88.2dB	108.4dB	82.2dB	63.6dB	
09:14:15	75.2dB	88.6dB	109.1dB	80.6dB	61.5dB	

Study 4

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 09:30:17
Study Stopped: 7/9/99 @ 09:38:19
Run Time: 0:08:01
Peak Level: 111.2dB, 7/9/99 @ 09:38:05
Max Level: 90.7dB, 7/9/99 @ 09:38:04
Min Level: 57.7dB, 7/9/99 @ 09:30:54
Overload: 0.00%

LEQ: 71.7dB	SEL(3):98.5dB	TWA: 53.9dB	TAKM3: 79.3dB
LDN: 71.7dB	CNEL: 71.7dB	Pa2Sec: 2.8	
L5: 79.5dB	L10: 75.0dB	L50: 61.7dB	L90: 59.9dB

Study 4

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
09:30:32	70.5dB	83.6dB	98.8dB	73.4dB	63.7dB	
09:30:47	71.2dB	82.4dB	93.0dB	74.6dB	63.2dB	
09:31:02	67.2dB	77.5dB	89.7dB	71.1dB	59.2dB	
09:31:17	71.5dB	80.3dB	92.9dB	76.1dB	63.4dB	
09:31:32	71.6dB	84.9dB	97.3dB	75.8dB	61.5dB	
09:31:47	71.0dB	83.9dB	97.4dB	73.8dB	61.2dB	
09:32:02	64.5dB	77.7dB	93.9dB	64.0dB	61.5dB	
09:32:17	61.0dB	63.7dB	79.2dB	62.0dB	60.1dB	
09:32:32	60.3dB	61.3dB	77.8dB	60.8dB	59.6dB	
09:32:47	61.3dB	66.0dB	81.3dB	62.5dB	60.1dB	
09:33:02	62.5dB	70.1dB	85.3dB	64.2dB	60.3dB	
09:33:17	63.4dB	70.6dB	86.2dB	65.7dB	61.5dB	
09:33:32	60.7dB	62.2dB	78.8dB	61.6dB	59.7dB	
09:33:47	60.5dB	61.6dB	76.4dB	61.0dB	60.0dB	
09:34:02	63.4dB	75.6dB	94.3dB	64.8dB	60.4dB	
09:34:17	61.1dB	62.6dB	75.7dB	61.7dB	60.7dB	
09:34:32	60.6dB	63.6dB	79.0dB	61.3dB	59.9dB	
09:34:47	60.9dB	63.9dB	77.4dB	62.0dB	59.9dB	
09:35:02	69.7dB	82.3dB	97.4dB	72.3dB	60.7dB	
09:35:17	75.6dB	87.3dB	106.1dB	80.4dB	60.3dB	
09:35:32	60.7dB	66.1dB	80.0dB	61.3dB	60.0dB	
09:35:47	60.3dB	63.5dB	77.8dB	61.0dB	59.7dB	
09:36:02	60.2dB	61.1dB	77.6dB	60.6dB	59.7dB	
09:36:17	60.1dB	63.7dB	80.0dB	60.6dB	59.3dB	
09:36:32	60.0dB	61.6dB	76.3dB	60.4dB	59.5dB	
09:36:47	61.8dB	71.8dB	86.2dB	63.9dB	59.3dB	
09:37:02	76.7dB	85.3dB	103.9dB	81.3dB	66.1dB	
09:37:17	77.1dB	85.3dB	104.8dB	81.7dB	67.7dB	
09:37:32	77.3dB	86.1dB	105.4dB	81.8dB	65.7dB	
09:37:47	77.9dB	86.5dB	105.9dB	82.8dB	65.2dB	
09:38:02	79.1dB	88.1dB	106.9dB	84.0dB	65.3dB	

Study 4

Logging (15 Seconds)

09:38:17

LEQ
75.3dB

LMAX
90.7dB

LPeak
111.2dB

LN10
79.4dB

LN90
61.2dB

Ovl

Study 1

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovi
12:19:07	55.8dB	62.7dB	82.5dB	58.4dB	52.8dB	
12:19:22	65.0dB	73.9dB	90.6dB	69.0dB	56.5dB	
12:19:37	68.5dB	76.3dB	93.5dB	72.2dB	60.5dB	
12:19:52	61.6dB	70.9dB	85.5dB	65.9dB	54.9dB	
12:20:07	57.9dB	65.0dB	82.0dB	61.3dB	53.3dB	
12:20:22	60.5dB	71.5dB	89.6dB	63.6dB	54.5dB	
12:20:37	59.0dB	71.0dB	88.1dB	62.8dB	51.6dB	
12:20:52	59.5dB	66.1dB	83.9dB	62.4dB	55.8dB	
12:21:07	61.7dB	69.6dB	84.5dB	65.3dB	52.9dB	
12:21:22	57.5dB	68.0dB	82.1dB	61.0dB	51.6dB	
12:21:37	61.0dB	71.4dB	87.4dB	64.6dB	53.7dB	
12:21:52	61.6dB	72.1dB	86.8dB	65.0dB	52.7dB	
12:22:07	65.2dB	77.1dB	94.2dB	69.4dB	56.4dB	
12:22:22	60.0dB	68.9dB	88.9dB	64.0dB	54.7dB	
12:22:37	60.1dB	71.1dB	85.7dB	63.2dB	53.8dB	
12:22:52	62.2dB	72.8dB	88.5dB	67.0dB	54.2dB	
12:23:07	65.1dB	73.4dB	89.9dB	69.1dB	57.3dB	
12:23:22	81.1dB	90.3dB	111.7dB	86.6dB	60.7dB	
12:23:37	84.2dB	95.1dB	115.3dB	88.8dB	68.5dB	
12:23:52	85.2dB	99.1dB	122.2dB	89.6dB	68.4dB	
12:24:07	84.4dB	95.5dB	115.9dB	89.2dB	66.0dB	
12:24:22	81.0dB	92.6dB	112.9dB	85.7dB	65.0dB	
12:24:37	80.1dB	90.2dB	110.2dB	85.1dB	62.9dB	
12:24:52	81.1dB	92.8dB	114.5dB	85.7dB	65.2dB	
12:25:07	73.2dB	88.6dB	107.8dB	77.0dB	55.3dB	

Study 2

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 06:28:08
Study Stopped: 7/9/99 @ 06:35:53
Run Time: 0:07:44
Peak Level: 118.2dB, 7/9/99 @ 06:34:19
Max Level: 96.8dB, 7/9/99 @ 06:34:18
Min Level: 51.6dB, 7/9/99 @ 06:31:40
Overload: 0.00%

LEQ: 75.7dB	SEL(3):102.4dB	TWA: 57.8dB	TAKM3: 84.1dB
LDN: 85.7dB	CNEL: 85.7dB	Pa2Sec: 6.9	
L5: 83.2dB	L10: 77.0dB	L50: 62.6dB	L90: 56.4dB

Study 2

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
06:28:23	71.5dB	82.3dB	95.6dB	76.7dB	61.7dB	
06:28:38	73.2dB	82.5dB	96.8dB	76.0dB	67.4dB	
06:28:53	65.8dB	71.9dB	86.0dB	68.6dB	61.9dB	
06:29:08	68.1dB	79.0dB	93.0dB	71.8dB	58.9dB	
06:29:23	65.2dB	75.0dB	89.7dB	68.8dB	57.4dB	
06:29:38	67.0dB	75.6dB	89.3dB	70.6dB	60.1dB	
06:29:53	70.1dB	81.1dB	91.3dB	74.2dB	59.1dB	
06:30:08	72.4dB	82.2dB	96.0dB	76.3dB	62.9dB	
06:30:23	68.8dB	84.4dB	96.9dB	70.1dB	56.6dB	
06:30:38	67.4dB	75.9dB	94.9dB	70.6dB	62.0dB	
06:30:53	64.0dB	72.5dB	88.4dB	67.4dB	56.5dB	
06:31:08	66.6dB	76.9dB	94.8dB	69.7dB	59.7dB	
06:31:23	61.1dB	69.3dB	92.5dB	64.3dB	56.6dB	
06:31:38	55.5dB	61.7dB	79.9dB	57.7dB	53.1dB	
06:31:53	55.8dB	66.9dB	82.4dB	58.2dB	52.3dB	
06:32:08	60.2dB	70.6dB	84.9dB	63.2dB	54.6dB	
06:32:23	61.5dB	69.9dB	85.6dB	64.9dB	57.3dB	
06:32:38	59.7dB	62.9dB	78.8dB	61.7dB	56.6dB	
06:32:53	58.8dB	66.5dB	82.0dB	60.1dB	56.6dB	
06:33:08	59.6dB	62.8dB	80.0dB	61.3dB	58.3dB	
06:33:23	58.7dB	61.9dB	81.5dB	60.0dB	57.0dB	
06:33:38	71.9dB	85.3dB	102.0dB	75.1dB	56.1dB	
06:33:53	60.3dB	65.1dB	79.5dB	62.1dB	56.5dB	
06:34:08	79.1dB	90.1dB	110.3dB	84.6dB	59.8dB	
06:34:23	83.1dB	96.8dB	118.2dB	86.9dB	67.4dB	
06:34:38	84.0dB	94.6dB	114.7dB	88.6dB	68.0dB	
06:34:53	82.0dB	92.5dB	113.0dB	87.0dB	66.5dB	
06:35:08	82.1dB	92.5dB	111.5dB	87.1dB	65.1dB	
06:35:23	81.8dB	91.1dB	108.2dB	87.6dB	63.6dB	
06:35:38	60.0dB	64.1dB	80.1dB	62.4dB	55.3dB	

Study 3

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 09:04:12
Study Stopped: 7/9/99 @ 09:08:17
Run Time: 0:04:05
Peak Level: 109.1dB, 7/9/99 @ 09:06:54
Max Level: 88.7dB, 7/9/99 @ 09:06:53
Min Level: 51.6dB, 7/9/99 @ 09:05:06
Overload: 0.00%

LEQ: 69.9dB	SEL(3):93.8dB	TWA: 49.2dB	TAKM3: 78.1dB
LDN: 69.9dB	CNEL: 69.9dB	Pa2Sec: 0.9	
L5: 76.3dB	L10: 74.4dB	L50: 63.4dB	L90: 54.0dB

Study 3

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
09:04:27	69.4dB	80.0dB	94.8dB	73.6dB	61.3dB	
09:04:42	72.6dB	81.9dB	102.3dB	76.1dB	62.4dB	
09:04:57	63.6dB	71.3dB	96.9dB	67.9dB	54.5dB	
09:05:12	61.6dB	73.9dB	90.9dB	65.5dB	51.6dB	
09:05:27	59.8dB	71.2dB	88.6dB	62.8dB	53.1dB	
09:05:42	73.1dB	81.3dB	98.5dB	77.6dB	62.1dB	
09:05:57	71.2dB	80.9dB	99.8dB	75.8dB	60.2dB	
09:06:12	71.8dB	82.6dB	101.5dB	76.0dB	60.4dB	
09:06:27	71.7dB	81.8dB	99.3dB	76.0dB	60.7dB	
09:06:42	69.4dB	78.9dB	95.1dB	74.1dB	57.7dB	
09:06:57	73.9dB	88.7dB	109.1dB	78.1dB	57.2dB	
09:07:12	54.3dB	65.0dB	79.7dB	55.3dB	52.5dB	
09:07:27	62.7dB	74.7dB	87.7dB	67.1dB	52.6dB	
09:07:42	65.1dB	74.9dB	90.9dB	69.6dB	54.1dB	
09:07:57	70.6dB	79.4dB	98.9dB	74.4dB	58.5dB	
09:08:12	71.1dB	84.1dB	103.7dB	75.0dB	60.3dB	

Study 4

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 09:24:16
Study Stopped: 7/9/99 @ 09:32:18
Run Time: 0:08:01
Peak Level: 110.3dB, 7/9/99 @ 09:24:57
Max Level: 88.8dB, 7/9/99 @ 09:24:31
Min Level: 51.6dB, 7/9/99 @ 09:26:22
Overload: 0.00%

LEQ: 70.1dB	SEL(3):96.9dB	TWA: 52.4dB	TAKM3: 77.9dB
LDN: 70.1dB	CNEL: 70.1dB	Pa2Sec: 2.0	
L5: 77.3dB	L10: 74.2dB	L50: 57.0dB	L90: 51.6dB

Study 4

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
09:24:31	75.4dB	88.8dB	102.3dB	77.6dB	62.1dB	
09:24:46	78.5dB	88.0dB	104.5dB	82.1dB	70.8dB	
09:25:01	78.6dB	87.5dB	110.3dB	82.7dB	69.4dB	
09:25:16	67.5dB	76.4dB	91.0dB	71.4dB	62.0dB	
09:25:31	68.2dB	79.0dB	93.7dB	71.7dB	62.3dB	
09:25:46	67.9dB	81.9dB	97.7dB	69.8dB	59.5dB	
09:26:01	61.1dB	73.2dB	89.1dB	62.7dB	56.6dB	
09:26:16	59.4dB	68.1dB	90.8dB	61.6dB	55.5dB	
09:26:31	54.2dB	58.6dB	76.5dB	56.3dB	52.5dB	
09:26:46	57.0dB	67.2dB	83.9dB	59.8dB	51.7dB	
09:27:01	56.6dB	64.0dB	83.1dB	60.1dB	51.6dB	
09:27:16	53.7dB	59.7dB	76.1dB	56.4dB	51.6dB	
09:27:31	51.6dB	52.0dB	76.3dB	51.6dB	51.6dB	
09:27:46	51.7dB	55.9dB	76.0dB	51.6dB	51.6dB	
09:28:01	51.6dB	51.6dB	76.3dB	51.6dB	51.6dB	
09:28:16	51.6dB	52.6dB	75.8dB	51.6dB	51.6dB	
09:28:31	57.8dB	70.1dB	85.7dB	56.6dB	51.6dB	
09:28:46	58.9dB	67.4dB	83.8dB	63.5dB	51.6dB	
09:29:01	62.4dB	76.2dB	92.1dB	64.3dB	54.1dB	
09:29:16	69.4dB	79.9dB	97.1dB	74.3dB	57.4dB	
09:29:31	67.8dB	84.5dB	101.0dB	61.3dB	52.2dB	
09:29:46	53.5dB	57.4dB	82.6dB	55.1dB	51.6dB	
09:30:01	52.3dB	54.0dB	76.5dB	52.9dB	51.6dB	
09:30:16	54.5dB	58.9dB	77.0dB	56.2dB	51.6dB	
09:30:31	53.6dB	57.1dB	81.6dB	54.5dB	52.7dB	
09:30:46	53.0dB	65.3dB	81.3dB	54.0dB	52.1dB	
09:31:01	69.7dB	81.9dB	97.7dB	75.0dB	54.5dB	
09:31:16	72.1dB	80.9dB	98.6dB	76.6dB	61.5dB	
09:31:31	72.0dB	81.1dB	99.0dB	76.7dB	60.6dB	
09:31:46	73.2dB	83.0dB	102.2dB	78.3dB	58.9dB	
09:32:01	73.5dB	82.8dB	103.5dB	78.5dB	60.1dB	

Study 4

Logging (15 Seconds)
09:32:16

LEQ
73.8dB

LMAX
87.8dB

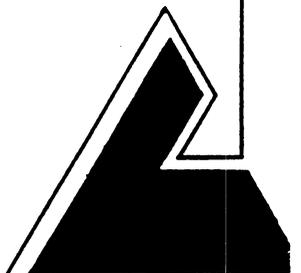
LPeak
107.6dB

LN10
78.3dB

LN90
57.3dB

Ovl

Western Leq Quest 2900 Field data



Study 1

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
13:17:22	62.9dB	75.9dB	86.3dB	63.3dB	59.9dB	
13:17:37	64.9dB	70.6dB	83.2dB	66.4dB	62.9dB	
13:17:52	62.6dB	65.3dB	80.7dB	64.2dB	61.5dB	
13:18:07	60.0dB	64.4dB	79.2dB	63.0dB	56.3dB	
13:18:22	61.1dB	69.2dB	83.0dB	64.0dB	54.7dB	
13:18:37	59.1dB	64.6dB	80.1dB	63.1dB	53.6dB	
13:18:52	69.7dB	86.8dB	97.9dB	61.7dB	53.9dB	
13:19:07	57.4dB	62.4dB	77.6dB	59.9dB	52.3dB	
13:19:22	54.7dB	60.3dB	76.8dB	57.0dB	51.8dB	
13:19:37	56.4dB	60.9dB	77.4dB	58.3dB	54.1dB	
13:19:52	55.3dB	60.8dB	81.2dB	57.4dB	52.9dB	
13:20:07	54.6dB	59.5dB	75.8dB	56.1dB	52.6dB	
13:20:22	56.8dB	61.0dB	77.2dB	58.6dB	54.8dB	
13:20:37	57.2dB	62.2dB	77.0dB	59.1dB	55.0dB	
13:20:52	59.1dB	63.0dB	76.9dB	61.4dB	56.4dB	
13:21:07	61.2dB	66.6dB	81.1dB	65.2dB	56.7dB	
13:21:22	85.8dB	99.6dB	119.3dB	90.0dB	62.0dB	
13:21:37	92.4dB	101.0dB	119.1dB	97.3dB	75.9dB	
13:21:52	91.4dB	99.0dB	117.9dB	96.7dB	74.4dB	
13:22:07	92.0dB	100.5dB	120.3dB	97.0dB	74.0dB	
13:22:22	91.7dB	100.3dB	120.2dB	97.1dB	72.7dB	
13:22:37	92.3dB	100.6dB	119.5dB	97.6dB	73.0dB	
13:22:52	92.0dB	99.8dB	119.0dB	97.3dB	73.5dB	
13:23:07	90.5dB	99.9dB	117.1dB	96.6dB	63.4dB	
13:23:22	60.1dB	65.1dB	79.2dB	63.1dB	55.1dB	
13:23:37	60.7dB	69.0dB	85.8dB	63.9dB	54.8dB	
13:23:52	67.5dB	77.8dB	93.5dB	72.1dB	56.3dB	

Study 2

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 07:27:25
Study Stopped: 7/9/99 @ 07:34:56
Run Time: 0:07:31
Peak Level: 119.9dB, 7/9/99 @ 07:32:10
Max Level: 101.4dB, 7/9/99 @ 07:32:56
Min Level: 52.6dB, 7/9/99 @ 07:29:48
Overload: 0.00%

LEQ: 85.4dB	SEL(3):112.0dB	TWA: 67.4dB	TAKM3: 92.9dB
LDN: 85.4dB	CNEL: 85.4dB	Pa2Sec: 62.7	
L5: 93.8dB	L10: 85.8dB	L50: 64.6dB	L90: 56.3dB

Study 2

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
07:27:40	58.4dB	62.9dB	78.3dB	59.9dB	56.0dB	
07:27:55	64.6dB	68.3dB	83.1dB	67.0dB	58.9dB	
07:28:10	64.0dB	69.3dB	82.2dB	66.1dB	56.8dB	
07:28:25	61.3dB	68.2dB	81.4dB	64.2dB	56.7dB	
07:28:40	59.5dB	64.4dB	78.7dB	63.6dB	55.3dB	
07:28:55	65.4dB	71.2dB	84.8dB	69.9dB	56.5dB	
07:29:10	62.2dB	66.4dB	80.4dB	64.9dB	58.7dB	
07:29:25	58.9dB	63.3dB	78.6dB	61.3dB	55.8dB	
07:29:40	56.6dB	62.6dB	76.7dB	57.9dB	55.1dB	
07:29:55	55.6dB	63.1dB	77.9dB	57.9dB	53.4dB	
07:30:10	56.7dB	60.1dB	82.6dB	58.2dB	55.3dB	
07:30:25	66.5dB	71.2dB	86.4dB	69.6dB	58.4dB	
07:30:40	65.5dB	69.4dB	84.2dB	68.5dB	59.5dB	
07:30:55	68.2dB	69.2dB	84.8dB	68.8dB	67.3dB	
07:31:10	67.4dB	68.6dB	82.0dB	68.1dB	66.8dB	
07:31:25	67.0dB	68.7dB	83.2dB	68.3dB	63.7dB	
07:31:40	68.2dB	77.2dB	92.1dB	69.5dB	65.0dB	
07:31:55	81.4dB	94.4dB	108.5dB	85.6dB	62.6dB	
07:32:10	81.6dB	99.0dB	119.9dB	77.2dB	65.7dB	
07:32:25	91.6dB	99.4dB	119.0dB	96.3dB	76.0dB	
07:32:40	91.9dB	99.9dB	119.2dB	96.5dB	76.6dB	
07:32:55	92.9dB	100.4dB	119.1dB	98.1dB	75.4dB	
07:33:10	93.5dB	101.4dB	118.7dB	99.0dB	74.9dB	
07:33:25	93.3dB	101.4dB	118.9dB	99.2dB	73.8dB	
07:33:40	89.1dB	101.1dB	117.7dB	94.1dB	63.7dB	
07:33:55	59.0dB	67.7dB	79.2dB	61.0dB	55.5dB	
07:34:10	65.6dB	72.9dB	82.8dB	68.7dB	60.9dB	
07:34:25	64.8dB	68.4dB	81.8dB	67.0dB	61.6dB	
07:34:40	62.1dB	66.9dB	81.4dB	65.3dB	57.1dB	
07:34:55	65.0dB	77.5dB	102.7dB	65.7dB	61.9dB	

Study 3

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 09:59:55
Study Stopped: 7/9/99 @ 10:07:51
Run Time: 0:07:56
Peak Level: 110.2dB, 7/9/99 @ 10:04:03
Max Level: 90.5dB, 7/9/99 @ 10:03:36
Min Level: 51.6dB, 7/9/99 @ 10:06:27
Overload: 0.00%

LEQ: 74.5dB	SEL(3):101.3dB	TWA: 56.7dB	TAKM3: 81.2dB
LDN: 74.5dB	CNEL: 74.5dB	Pa2Sec: 5.3	
L5: 83.4dB	L10: 78.0dB	L50: 59.0dB	L90: 51.8dB

Study 3

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:00:10	58.9dB	64.8dB	85.8dB	59.9dB	57.0dB	
10:00:25	59.1dB	63.0dB	77.3dB	59.9dB	58.4dB	
10:00:40	57.4dB	59.0dB	77.2dB	58.0dB	56.9dB	
10:00:55	58.3dB	61.1dB	76.6dB	59.2dB	57.4dB	
10:01:10	60.1dB	64.8dB	77.8dB	61.8dB	58.6dB	
10:01:25	58.6dB	60.2dB	76.7dB	59.1dB	58.2dB	
10:01:40	58.5dB	59.7dB	77.4dB	59.0dB	58.0dB	
10:01:55	58.8dB	60.3dB	77.4dB	59.2dB	58.3dB	
10:02:10	58.9dB	60.3dB	76.6dB	59.4dB	58.4dB	
10:02:25	58.7dB	59.9dB	76.8dB	59.2dB	58.3dB	
10:02:40	59.0dB	62.8dB	78.0dB	59.7dB	58.1dB	
10:02:55	59.7dB	64.0dB	77.6dB	60.6dB	58.9dB	
10:03:10	59.6dB	62.3dB	76.8dB	60.5dB	58.8dB	
10:03:25	61.8dB	70.7dB	81.1dB	64.4dB	59.3dB	
10:03:40	77.3dB	90.5dB	106.7dB	82.3dB	58.9dB	
10:03:55	82.7dB	89.9dB	106.2dB	87.4dB	69.0dB	
10:04:10	81.5dB	89.9dB	110.2dB	86.1dB	68.7dB	
10:04:25	80.5dB	87.5dB	104.6dB	85.3dB	68.2dB	
10:04:40	80.0dB	86.9dB	103.0dB	84.9dB	66.0dB	
10:04:55	79.7dB	86.5dB	101.9dB	84.8dB	65.2dB	
10:05:10	77.9dB	88.2dB	103.0dB	84.2dB	60.2dB	
10:05:25	59.8dB	66.5dB	81.1dB	60.9dB	58.6dB	
10:05:40	58.6dB	60.5dB	77.2dB	59.2dB	57.9dB	
10:05:55	67.7dB	84.2dB	98.5dB	62.7dB	58.6dB	
10:06:10	78.6dB	85.3dB	101.5dB	83.9dB	63.0dB	
10:06:25	76.6dB	86.0dB	101.1dB	83.0dB	58.2dB	
10:06:40	55.0dB	60.2dB	76.9dB	57.9dB	51.6dB	
10:06:55	55.8dB	59.6dB	76.2dB	57.8dB	53.1dB	
10:07:10	52.7dB	57.0dB	74.8dB	54.9dB	51.6dB	
10:07:25	51.7dB	57.2dB	84.7dB	51.6dB	51.6dB	
10:07:40	51.6dB	53.2dB	74.9dB	51.6dB	51.6dB	

Study 4

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 10:21:21
Study Stopped: 7/9/99 @ 10:31:44
Run Time: 0:10:23
Peak Level: 111.4dB, 7/9/99 @ 10:29:14
Max Level: 93.2dB, 7/9/99 @ 10:29:17
Min Level: 51.6dB, 7/9/99 @ 10:31:24
Overload: 0.00%

LEQ: 73.5dB	SEL(3):101.4dB	TWA: 56.8dB	TAKM3: 81.0dB
LDN: 73.5dB	CNEL: 73.5dB	Pa2Sec: 5.5	
L5: 81.3dB	L10: 72.6dB	L50: 55.6dB	L90: 53.7dB

Study 4

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:21:36	56.4dB	66.5dB	94.9dB	57.4dB	54.8dB	
10:21:51	56.9dB	66.9dB	80.2dB	57.5dB	54.7dB	
10:22:06	58.1dB	62.8dB	77.0dB	60.5dB	54.8dB	
10:22:21	57.9dB	62.8dB	76.9dB	61.8dB	54.3dB	
10:22:36	54.5dB	56.8dB	77.2dB	55.2dB	54.0dB	
10:22:51	54.5dB	58.0dB	76.8dB	56.1dB	53.5dB	
10:23:06	54.2dB	57.0dB	75.9dB	55.3dB	53.3dB	
10:23:21	53.6dB	55.5dB	76.2dB	54.3dB	52.8dB	
10:23:36	54.2dB	56.1dB	77.0dB	55.1dB	53.3dB	
10:23:51	55.2dB	60.1dB	76.6dB	58.0dB	52.7dB	
10:24:06	55.2dB	56.7dB	76.5dB	55.8dB	54.5dB	
10:24:21	58.9dB	62.4dB	77.2dB	61.3dB	56.5dB	
10:24:36	55.1dB	56.9dB	76.8dB	55.8dB	54.5dB	
10:24:51	54.3dB	56.1dB	76.7dB	54.9dB	53.7dB	
10:25:06	57.6dB	63.2dB	76.7dB	59.7dB	54.9dB	
10:25:21	57.5dB	63.8dB	78.8dB	60.7dB	54.2dB	
10:25:36	55.2dB	61.9dB	77.8dB	56.8dB	53.7dB	
10:25:51	55.0dB	59.5dB	76.1dB	56.1dB	53.7dB	
10:26:06	55.5dB	61.1dB	77.0dB	56.9dB	54.1dB	
10:26:21	56.5dB	59.8dB	76.3dB	58.2dB	54.8dB	
10:26:36	56.8dB	64.4dB	80.8dB	58.7dB	54.6dB	
10:26:51	56.8dB	68.9dB	83.3dB	58.6dB	53.3dB	
10:27:06	71.0dB	86.8dB	101.7dB	72.7dB	53.8dB	
10:27:21	77.1dB	88.2dB	106.7dB	83.0dB	55.0dB	
10:27:36	76.9dB	89.1dB	106.3dB	82.7dB	53.4dB	
10:27:51	54.8dB	58.2dB	77.0dB	56.4dB	53.6dB	
10:28:06	64.8dB	81.4dB	93.5dB	60.1dB	54.1dB	
10:28:21	57.2dB	65.0dB	80.8dB	58.7dB	55.2dB	
10:28:36	56.8dB	65.7dB	77.9dB	58.5dB	54.5dB	
10:28:51	55.3dB	57.3dB	76.8dB	56.8dB	54.1dB	
10:29:06	78.2dB	89.6dB	106.9dB	83.9dB	54.4dB	

Study 4

Logging (15 Seconds)

	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:29:21	83.7dB	93.2dB	111.4dB	88.4dB	69.0dB	
10:29:36	82.1dB	90.0dB	109.5dB	86.9dB	68.1dB	
10:29:51	80.5dB	88.7dB	104.5dB	85.6dB	65.0dB	
10:30:06	80.1dB	88.4dB	105.5dB	85.2dB	63.9dB	
10:30:21	80.4dB	88.4dB	104.4dB	85.6dB	63.9dB	
10:30:36	63.8dB	78.5dB	89.1dB	63.6dB	56.2dB	
10:30:51	62.1dB	70.2dB	84.7dB	65.0dB	58.0dB	
10:31:06	64.4dB	71.2dB	83.4dB	66.9dB	60.1dB	
10:31:21	55.9dB	63.1dB	76.7dB	57.6dB	54.0dB	
10:31:36	52.7dB	58.9dB	76.3dB	54.5dB	51.6dB	

**Quest Technologies
2900 Integrating/Logging Sound Level Meter**

Unit Version Number: 02.3
Name: FAU ACOUSTIC PILE DRIVING
ANALYSIS

Serial Number: CD7120033

Work Area: FIFTEEN FEET HIGH - 300 FEET
AWAY

Comments:

Meter Calibration: 114.0dB, 7/7/99 @ 12:48:45

Calibrator Serial Number:

Calibration Date:

Study 1

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/8/99 @ 13:15:35

Study Stopped: 7/8/99 @ 13:24:42

Run Time: 0:09:06

Peak Level: 109.8dB, 7/8/99 @ 13:16:56

Max Level: 89.8dB, 7/8/99 @ 13:16:55

Min Level: 51.6dB, 7/8/99 @ 13:18:44

Overload: 0.00%

LEQ: 73.3dB

SEL(3):100.6dB

TWA: 56.1dB

TAKM3: 81.2dB

LDN: 73.3dB

CNEL: 73.3dB

Pa2Sec: 4.6

L5: 81.9dB

L10: 75.2dB

L50: 56.9dB

L90: 51.8dB

Study 1

Logging (15 Seconds)

	LEQ	LMAX	LPeak	LN10	LN90	Ovl
13:15:50	60.1dB	72.8dB	87.5dB	60.6dB	53.7dB	
13:16:05	66.5dB	78.3dB	96.9dB	71.2dB	55.4dB	
13:16:20	65.1dB	79.4dB	97.5dB	65.7dB	56.4dB	
13:16:35	66.0dB	81.4dB	97.8dB	64.0dB	58.5dB	
13:16:50	71.1dB	82.4dB	100.7dB	76.4dB	55.3dB	
13:17:05	77.0dB	89.8dB	109.8dB	82.7dB	56.7dB	
13:17:20	57.0dB	60.3dB	75.8dB	58.8dB	54.3dB	
13:17:35	61.7dB	65.6dB	81.6dB	64.4dB	58.1dB	
13:17:50	56.9dB	63.4dB	81.6dB	60.1dB	53.7dB	
13:18:05	55.5dB	61.7dB	74.9dB	58.0dB	53.3dB	
13:18:20	59.1dB	64.3dB	81.1dB	62.6dB	55.6dB	
13:18:35	56.7dB	59.5dB	75.1dB	58.5dB	55.3dB	
13:18:50	55.9dB	63.7dB	80.6dB	58.9dB	51.8dB	

Logging (15 Seconds)	Study 1					Ovl
	LEQ	LMAX	LPeak	LN10	LN90	
13:19:05	59.2dB	67.1dB	81.2dB	63.6dB	51.7dB	
13:19:20	57.0dB	62.8dB	80.3dB	60.1dB	51.6dB	
13:19:35	61.1dB	77.7dB	88.1dB	57.8dB	51.6dB	
13:19:50	55.2dB	59.5dB	74.6dB	58.3dB	51.6dB	
13:20:05	53.0dB	56.5dB	74.5dB	54.6dB	51.6dB	
13:20:20	53.0dB	56.4dB	74.3dB	54.7dB	51.6dB	
13:20:35	52.0dB	56.6dB	75.3dB	52.9dB	51.6dB	
13:20:50	52.2dB	54.9dB	74.3dB	53.2dB	51.6dB	
13:21:05	53.1dB	57.3dB	75.0dB	54.3dB	51.6dB	
13:21:20	54.1dB	57.1dB	75.7dB	55.9dB	52.5dB	
13:21:35	56.7dB	59.1dB	76.0dB	58.0dB	55.1dB	
13:21:50	57.1dB	61.6dB	75.4dB	58.9dB	55.6dB	
13:22:05	72.6dB	86.5dB	104.9dB	74.0dB	55.5dB	
13:22:20	79.9dB	88.5dB	106.5dB	84.8dB	65.4dB	
13:22:35	80.0dB	89.3dB	107.6dB	85.0dB	64.7dB	
13:22:50	79.5dB	89.2dB	108.5dB	84.7dB	63.3dB	
13:23:05	79.5dB	88.4dB	107.1dB	84.9dB	62.8dB	
13:23:20	79.7dB	88.8dB	106.2dB	85.2dB	62.6dB	
13:23:35	80.3dB	88.6dB	107.1dB	85.5dB	63.4dB	
13:23:50	79.5dB	89.3dB	106.9dB	85.3dB	59.3dB	
13:24:05	56.0dB	61.0dB	74.3dB	58.9dB	52.1dB	
13:24:20	56.7dB	66.1dB	80.2dB	58.8dB	52.5dB	
13:24:35	53.3dB	63.7dB	82.3dB	54.5dB	51.6dB	

Study 2

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 07:29:04
Study Stopped: 7/9/99 @ 07:36:44
Run Time: 0:07:40
Peak Level: 108.7dB, 7/9/99 @ 07:34:09
Max Level: 90.6dB, 7/9/99 @ 07:33:43
Min Level: 51.7dB, 7/9/99 @ 07:29:39
Overload: 0.00%

LEQ: 74.0dB	SEL(3):100.6dB	TWA: 56.0dB	TAKM3: 81.5dB
LDN: 74.0dB	CNEL: 74.0dB	Pa2Sec: 4.6	
L5: 82.0dB	L10: 74.3dB	L50: 61.1dB	L90: 56.9dB

Study 2

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
07:29:19	62.2dB	78.7dB	99.4dB	63.4dB	53.8dB	
07:29:34	63.5dB	69.6dB	83.0dB	68.1dB	54.3dB	
07:29:49	57.6dB	63.5dB	80.7dB	60.9dB	53.0dB	
07:30:04	56.6dB	61.5dB	82.3dB	58.3dB	54.5dB	
07:30:19	58.1dB	61.1dB	75.5dB	59.4dB	56.5dB	
07:30:34	57.7dB	60.4dB	75.5dB	58.9dB	56.3dB	
07:30:49	58.9dB	61.5dB	76.8dB	60.0dB	57.2dB	
07:31:04	62.9dB	68.3dB	80.7dB	66.5dB	58.4dB	
07:31:19	61.8dB	65.7dB	80.9dB	64.1dB	59.7dB	
07:31:34	63.1dB	65.1dB	79.5dB	64.3dB	61.7dB	
07:31:49	61.2dB	62.7dB	77.0dB	62.0dB	60.1dB	
07:32:04	61.3dB	63.2dB	77.4dB	62.0dB	60.4dB	
07:32:19	63.2dB	68.3dB	85.0dB	66.6dB	57.9dB	
07:32:34	70.1dB	82.9dB	98.2dB	74.3dB	56.9dB	
07:32:49	61.2dB	65.7dB	82.1dB	63.3dB	58.9dB	
07:33:04	78.5dB	88.0dB	108.2dB	83.5dB	62.1dB	
07:33:19	80.1dB	88.8dB	106.6dB	84.7dB	65.9dB	
07:33:34	81.4dB	90.3dB	108.5dB	86.3dB	65.8dB	
07:33:49	81.3dB	90.6dB	108.5dB	86.6dB	65.4dB	
07:34:04	82.4dB	90.4dB	107.2dB	88.2dB	64.5dB	
07:34:19	80.2dB	90.1dB	108.7dB	86.6dB	59.3dB	
07:34:34	58.4dB	62.0dB	78.1dB	61.1dB	55.1dB	
07:34:49	61.1dB	65.8dB	77.2dB	63.9dB	56.9dB	
07:35:04	62.4dB	66.3dB	81.3dB	65.0dB	59.3dB	
07:35:19	61.5dB	64.8dB	80.8dB	63.9dB	58.0dB	
07:35:34	61.7dB	66.1dB	81.3dB	63.3dB	59.1dB	
07:35:49	60.5dB	62.6dB	76.1dB	61.5dB	59.1dB	
07:36:04	60.9dB	64.6dB	79.9dB	62.6dB	58.7dB	
07:36:19	60.2dB	62.4dB	78.1dB	61.5dB	58.9dB	
07:36:34	65.8dB	70.7dB	82.7dB	68.4dB	62.6dB	

Study 3

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 10:02:05
Study Stopped: 7/9/99 @ 10:09:55
Run Time: 0:07:50
Peak Level: 98.9dB, 7/9/99 @ 10:04:47
Max Level: 79.8dB, 7/9/99 @ 10:05:08
Min Level: 51.6dB, 7/9/99 @ 10:07:11
Overload: 0.00%

LEQ: 64.5dB	SEL(3):91.2dB	TWA: 46.6dB	TAKM3: 71.4dB
LDN: 64.5dB	CNEL: 64.5dB	Pa2Sec: 0.5	
L5: 72.9dB	L10: 68.0dB	L50: 55.5dB	L90: 51.6dB

Study 3

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:02:20	63.3dB	78.7dB	96.9dB	64.8dB	55.7dB	
10:02:35	55.0dB	56.4dB	74.8dB	55.8dB	54.4dB	
10:02:50	55.3dB	56.9dB	74.3dB	56.1dB	54.5dB	
10:03:05	55.3dB	57.1dB	75.9dB	56.2dB	54.4dB	
10:03:20	55.7dB	57.2dB	74.6dB	56.4dB	54.6dB	
10:03:35	55.6dB	57.6dB	75.9dB	56.7dB	54.7dB	
10:03:50	55.6dB	57.3dB	74.8dB	56.6dB	54.7dB	
10:04:05	56.0dB	58.8dB	76.6dB	57.1dB	54.9dB	
10:04:20	55.9dB	59.3dB	74.5dB	57.6dB	54.7dB	
10:04:35	72.1dB	79.4dB	95.8dB	76.8dB	59.5dB	
10:04:50	71.6dB	79.6dB	98.9dB	76.0dB	61.5dB	
10:05:05	70.0dB	78.3dB	96.3dB	74.3dB	60.0dB	
10:05:20	70.3dB	79.8dB	96.5dB	74.8dB	58.8dB	
10:05:35	69.3dB	76.4dB	95.1dB	74.1dB	59.3dB	
10:05:50	69.7dB	78.5dB	94.3dB	75.0dB	57.0dB	
10:06:05	56.7dB	61.2dB	75.8dB	57.6dB	55.6dB	
10:06:20	56.3dB	57.8dB	75.6dB	57.2dB	55.5dB	
10:06:35	55.8dB	58.9dB	75.1dB	56.9dB	54.4dB	
10:06:50	67.2dB	75.1dB	90.7dB	72.8dB	57.2dB	
10:07:05	67.4dB	74.7dB	91.6dB	72.8dB	56.4dB	
10:07:20	53.5dB	56.5dB	74.7dB	55.8dB	51.6dB	
10:07:35	53.8dB	59.0dB	74.7dB	56.7dB	51.6dB	
10:07:50	51.7dB	53.3dB	75.8dB	52.0dB	51.6dB	
10:08:05	51.7dB	57.1dB	84.6dB	51.6dB	51.6dB	
10:08:20	51.7dB	53.5dB	75.6dB	51.7dB	51.6dB	
10:08:35	51.8dB	56.8dB	74.1dB	51.6dB	51.6dB	
10:08:50	51.6dB	51.6dB	74.0dB	51.6dB	51.6dB	
10:09:05	51.6dB	53.3dB	74.3dB	51.6dB	51.6dB	
10:09:20	51.7dB	54.3dB	75.3dB	51.6dB	51.6dB	
10:09:35	51.6dB	52.5dB	74.4dB	51.6dB	51.6dB	
10:09:50	54.5dB	67.3dB	90.9dB	55.5dB	51.6dB	

Study 4

Measuring Parameters

Range: 60 - 120 dB
Threshold: Off

Weighting: A
Exchange Rate: 3dB

Time Constant: Fast
Peak Weighting: A

Study Started: 7/9/99 @ 10:20:54
Study Stopped: 7/9/99 @ 10:33:23
Run Time: 0:12:28
Peak Level: 99.9dB, 7/9/99 @ 10:30:07
Max Level: 80.4dB, 7/9/99 @ 10:29:58
Min Level: 51.6dB, 7/9/99 @ 10:23:41
Overload: 0.00%

LEQ: 62.8dB	SEL(3):91.6dB	TWA: 47.0dB	TAKM3: 69.8dB
LDN: 62.8dB	CNEL: 62.8dB	Pa2Sec: 0.6	
L5: 70.1dB	L10: 63.5dB	L50: 54.8dB	L90: 52.7dB

Study 4

Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:21:09	62.4dB	78.5dB	95.8dB	64.3dB	54.8dB	
10:21:24	67.6dB	75.2dB	82.5dB	72.9dB	54.7dB	
10:21:39	55.6dB	59.7dB	75.8dB	57.1dB	54.3dB	
10:21:54	55.3dB	57.3dB	74.9dB	56.3dB	54.2dB	
10:22:09	55.6dB	57.8dB	75.5dB	56.8dB	54.8dB	
10:22:24	56.7dB	60.1dB	74.7dB	58.5dB	55.3dB	
10:22:39	55.6dB	57.5dB	76.3dB	56.2dB	55.2dB	
10:22:54	57.0dB	59.5dB	77.5dB	59.0dB	54.5dB	
10:23:09	55.0dB	57.3dB	75.0dB	56.2dB	53.4dB	
10:23:24	54.2dB	55.8dB	75.8dB	54.9dB	53.5dB	
10:23:39	54.2dB	55.6dB	74.5dB	54.9dB	53.5dB	
10:23:54	54.7dB	56.8dB	74.9dB	55.8dB	52.7dB	
10:24:09	54.2dB	56.3dB	76.0dB	55.3dB	53.1dB	
10:24:24	54.8dB	57.0dB	74.5dB	55.8dB	53.8dB	
10:24:39	54.6dB	56.2dB	74.9dB	55.5dB	53.7dB	
10:24:54	56.2dB	61.9dB	76.0dB	57.5dB	54.5dB	
10:25:09	59.2dB	62.9dB	74.7dB	61.5dB	55.4dB	
10:25:24	54.7dB	55.7dB	75.0dB	55.3dB	54.0dB	
10:25:39	54.5dB	56.3dB	75.6dB	55.6dB	53.5dB	
10:25:54	59.8dB	66.5dB	82.3dB	62.2dB	56.6dB	
10:26:09	54.3dB	58.9dB	74.5dB	55.4dB	53.0dB	
10:26:24	54.1dB	55.6dB	74.9dB	54.9dB	53.2dB	
10:26:39	53.9dB	55.7dB	75.1dB	55.0dB	52.9dB	
10:26:54	54.0dB	55.8dB	74.5dB	54.8dB	53.0dB	
10:27:09	54.1dB	56.2dB	75.6dB	55.2dB	53.1dB	
10:27:24	53.9dB	58.5dB	74.6dB	54.7dB	53.1dB	
10:27:39	53.0dB	54.3dB	74.7dB	53.5dB	52.5dB	
10:27:54	61.7dB	76.4dB	92.7dB	63.3dB	52.5dB	
10:28:09	68.5dB	77.6dB	93.3dB	73.7dB	54.9dB	
10:28:24	60.6dB	76.3dB	92.9dB	59.1dB	52.0dB	
10:28:39	53.4dB	54.9dB	75.4dB	54.3dB	52.5dB	

Study 4

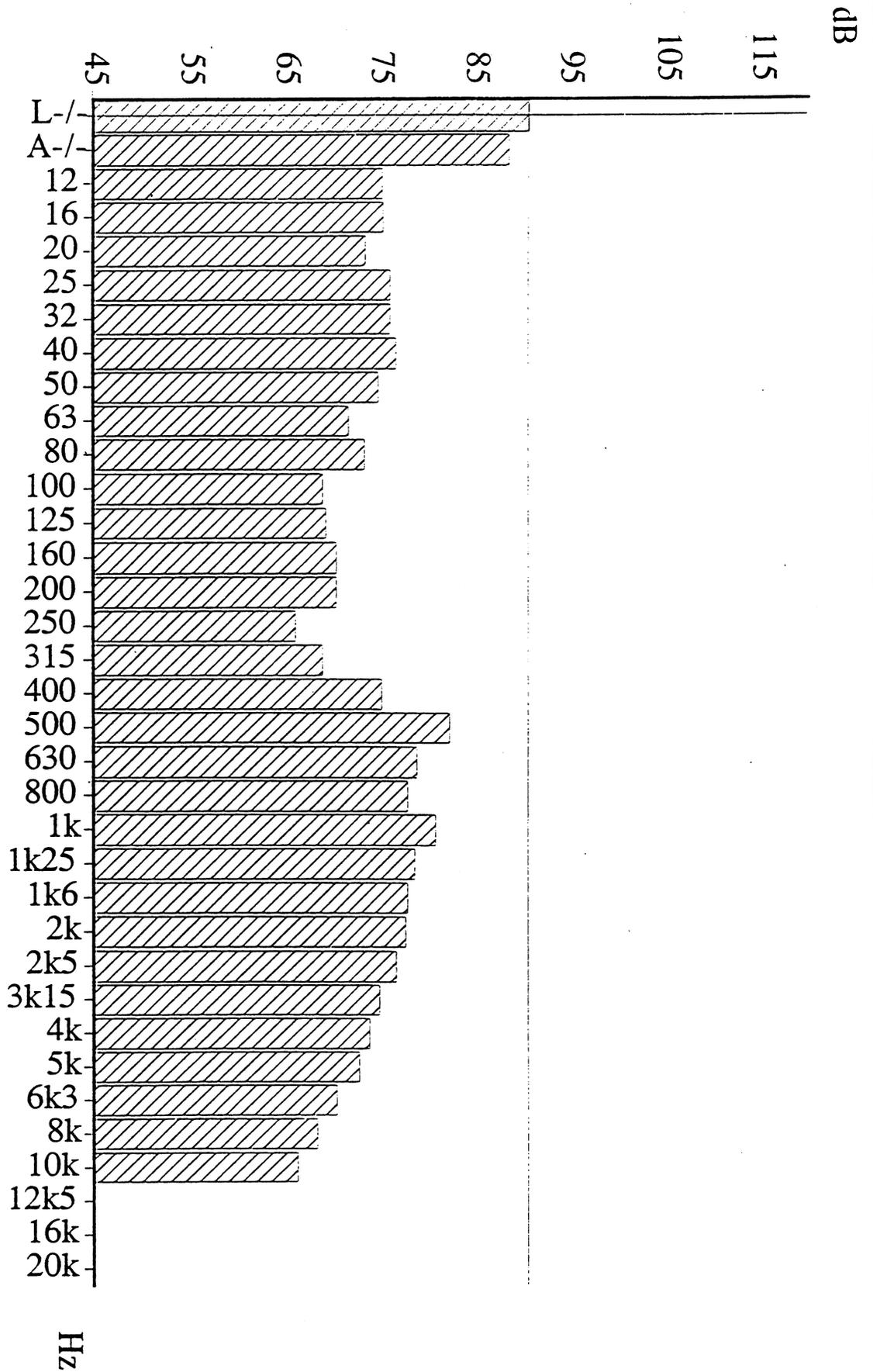
Logging (15 Seconds)	LEQ	LMAX	LPeak	LN10	LN90	Ovl
10:28:54	53.3dB	54.5dB	74.5dB	53.9dB	52.8dB	
10:29:09	54.0dB	56.0dB	76.5dB	54.7dB	53.1dB	
10:29:24	54.3dB	57.0dB	75.1dB	55.0dB	53.6dB	
10:29:39	53.9dB	55.1dB	74.6dB	54.5dB	53.3dB	
10:29:54	69.5dB	79.3dB	96.0dB	75.6dB	54.5dB	
10:30:09	72.2dB	80.4dB	99.9dB	76.8dB	61.7dB	
10:30:24	70.2dB	78.6dB	98.1dB	74.5dB	59.7dB	
10:30:39	68.9dB	76.8dB	92.7dB	73.7dB	56.4dB	
10:30:54	69.5dB	78.0dB	95.5dB	74.6dB	56.4dB	
10:31:09	69.0dB	79.2dB	93.7dB	74.5dB	56.0dB	
10:31:24	56.4dB	60.1dB	76.3dB	57.7dB	54.9dB	
10:31:39	56.5dB	57.9dB	75.0dB	57.4dB	55.3dB	
10:31:54	60.5dB	66.7dB	77.0dB	64.8dB	54.5dB	
10:32:09	52.9dB	55.4dB	75.1dB	53.7dB	51.7dB	
10:32:24	51.6dB	53.2dB	74.6dB	51.6dB	51.6dB	
10:32:39	51.6dB	51.7dB	74.1dB	51.6dB	51.6dB	
10:32:54	52.3dB	58.9dB	75.1dB	52.7dB	51.6dB	
10:33:09	62.4dB	75.3dB	87.6dB	65.7dB	51.6dB	

Octave Band Analyzer Data Sheets

DB2\FAU01.DTA

in summary -

Instrument	CEL-593.C2 Version 6.3 Type 2
Instrument ID (DPB)	107331
Mode	Third-octave band Environmental
Start	07/08/1999 14:33:31
End	07/08/1999 14:35:24
Duration	000 00:01:53.18
Calibration	07/07/1999 14:33:40
Measurement range	65 - 140 dB
Microphone response	Free Field
Normalizing voltage	Off
Frequency weighting	F
Frequency weighting	L, A
Change rate (Q)	3
Hold time	500 ms
Periods too short for LNs	Yes
Files recorded	No
File sample interval	1 s
Number of records	226
Auto enabled	No
Overload occurred	No
Battery occurred	No
File was used	Yes



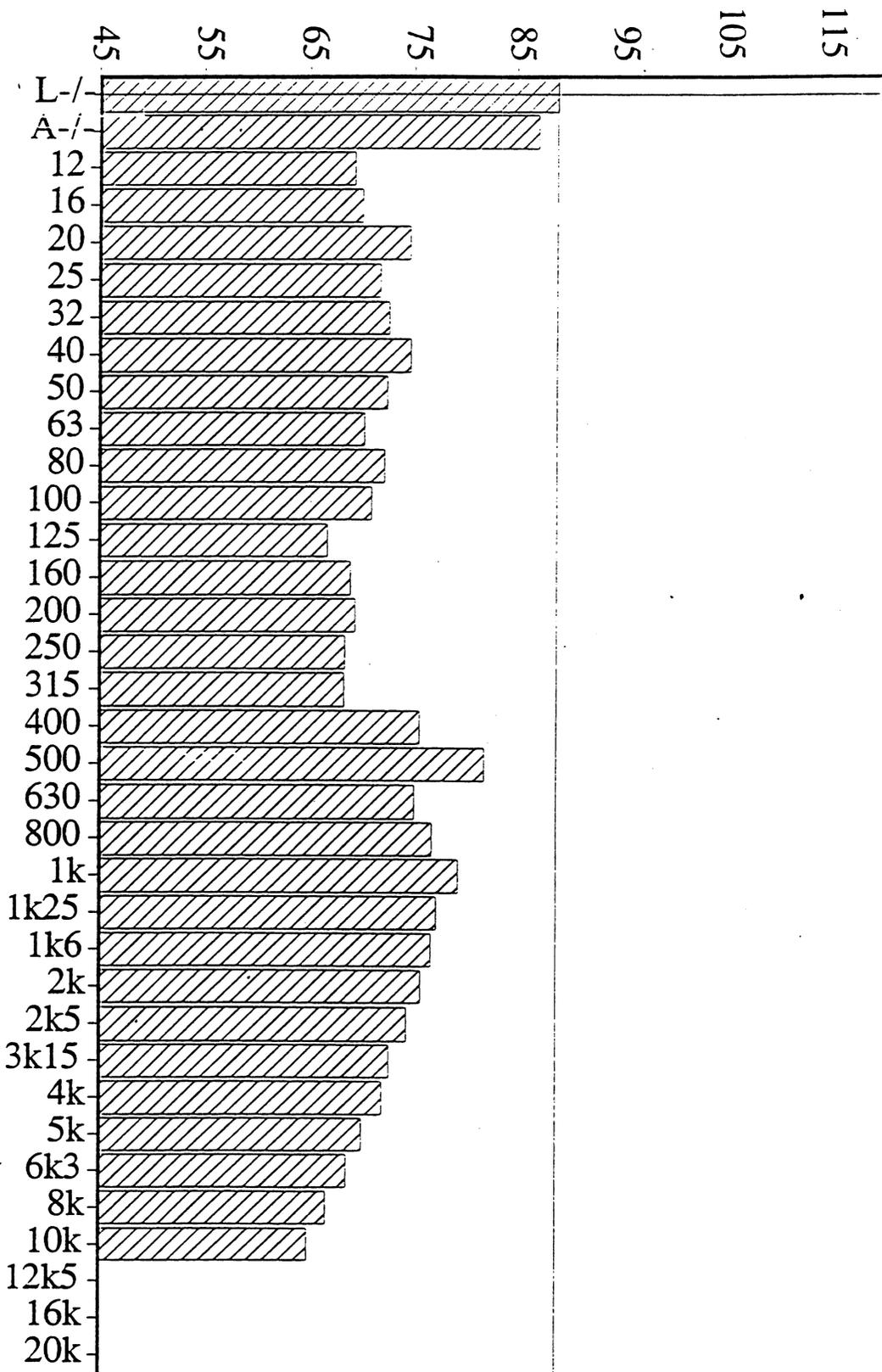
C:\DB2\FAU01.DTA
 Function = Leq
 Record start = 07/08/1999 14:33:31
 Duration = 000 00:01:53.2
 Record = Cumulative
 Flags: --P-
 Cursor: Band = Broadband, Frequency weighting = L, Level = 90.8 dB

.DB2\FAU03.DTA

un summary -

Instrument	CEL-593.C2 Version 6.3 Type 2
Instrument ID (DPB)	107331
Mode	Third-octave band Environmental
Start	07/09/1999 08:43:01
End	07/09/1999 08:45:43
Duration	000 00:02:42.42
Last calibration	07/07/1999 14:33:40
Measurement range	65 - 140 dB
Microphone response	Free Field
Harmonizing voltage	Off
Time weighting	F
Frequency weighting	L, A
Change rate (Q)	3
Load time	500 ms
Periods too short for LNs	Yes
Files recorded	No
File sample interval	1 s
Number of records	324
Auto tests enabled	No
Overload occurred	No
Battery occurred	No
Case was used	No

dB



C:\DB2\FAU03.DTA

Function = Leq

Record start = 07/09/1999 08:43:01

Duration = 000 00:02:42.4

Record = Cumulative

Flags: ----

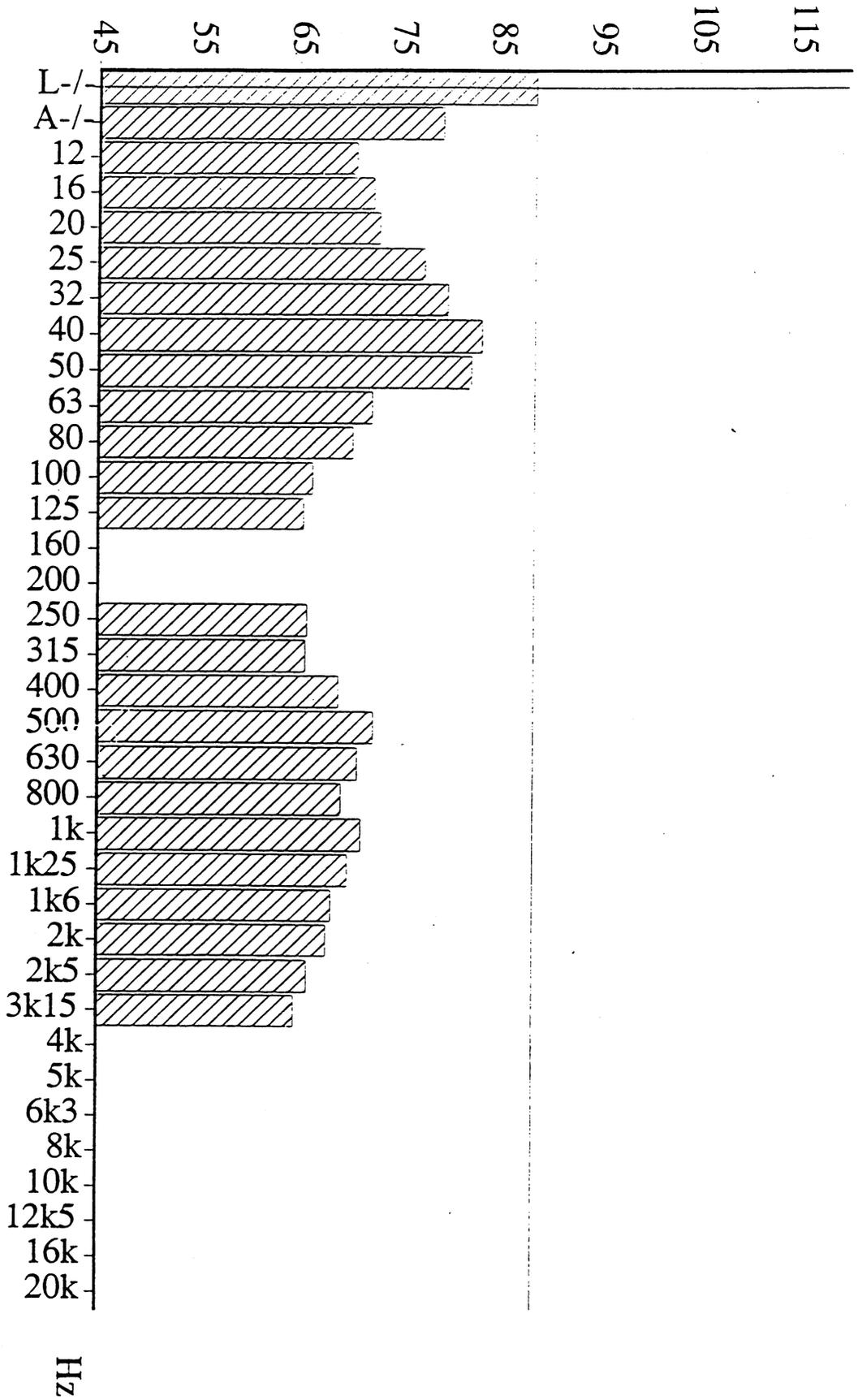
Cursor: Band = Broadband, Frequency weighting = L, Level = 88.9 dB

DB2\FAU04.DTA

Run summary -

Instrument	CEL-593.C2 Version 6.3 Type 2
Instrument ID (DPB)	107331
Run mode	Third-octave band Environmental
Run start	07/09/1999 11:15:31
Run end	07/09/1999 11:17:16
Run duration	000 00:01:45.88
Last calibration	07/07/1999 14:33:40
Measurement range	65 - 140 dB
Microphone response	Free Field
Normalizing voltage	Off
Time weighting	F
Frequency weighting	L, A
Change rate (Q)	3
Period time	500 ms
Periods too short for LNs	Yes
Files recorded	No
File sample interval	1 s
Number of records	211
Events enabled	No
Overload occurred	No
Low battery occurred	No
Power save used	No

dB



C:\DB2\FAU04.DTA

Function = Leq

Record start = 07/09/1999 11:15:31

Duration = 300 00:01:45.9

Record = Cumulative

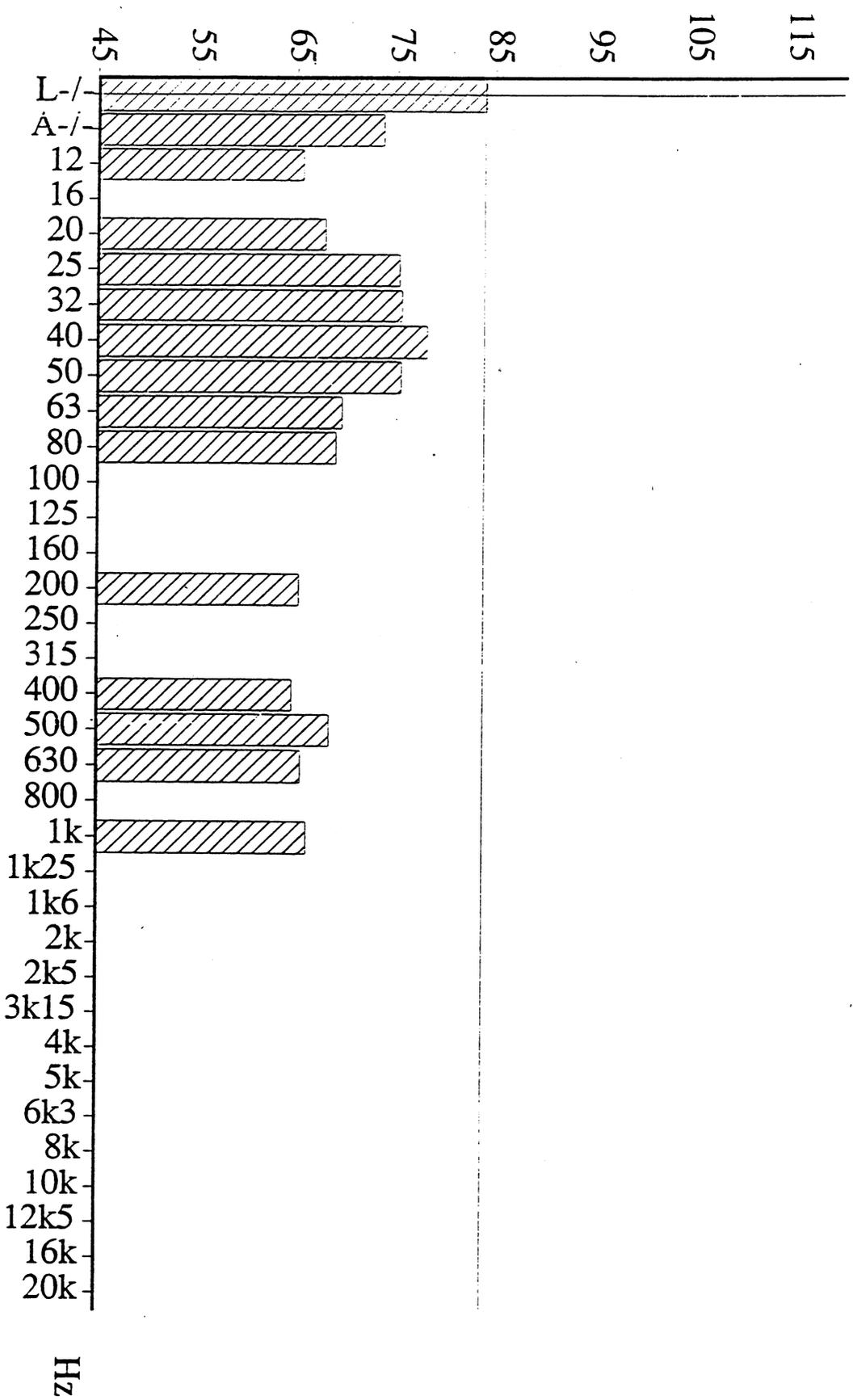
Flags: ----

Cursor: Band = Broadband, Frequency weighting = L, Level = 88.7 dB

Run summary -

Instrument	CEL-593.C2 Version 6.3 Type 2
Instrument ID (DPB)	107331
Run mode	Third-octave band Environmental
Run start	07/09/1999 11:17:29
Run end	07/09/1999 11:18:34
Run duration	000 00:01:05.56
Last calibration	07/07/1999 14:33:40
Measurement range	65 - 140 dB
Microphone response	Free Field
Polarizing voltage	Off
Time weighting	F
Frequency weighting	L, A
Change rate (Q)	3
Period time	500 ms
Periods too short for LNs	Yes
Files recorded	No
File sample interval	1 s
Number of records	131
Events enabled	No
Overload occurred	No
Low battery occurred	No
Fuse was used	No

dB



C:\DB2\FAUC5.DTA

Function = Leq

Record start = 07/09/1999 11:17:29

Duration = 000 00:01:05.6

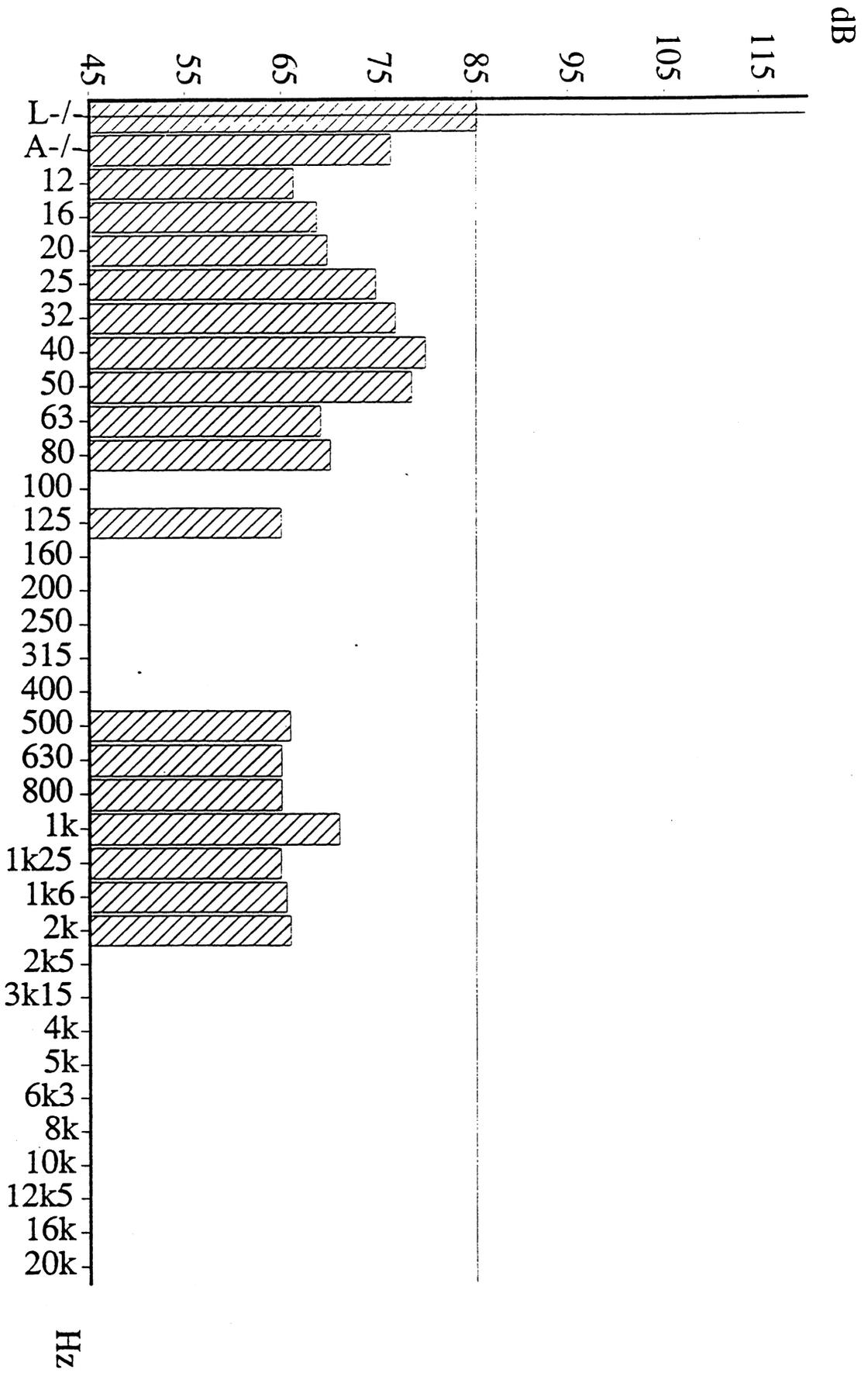
Record = Cumulative

Flags: ----

Cursor: Band = Broadband, Frequency weighting = L, Level = 84.0 dB

Run summary -

Instrument	CEL-593.C2 Version 6.3 Type 2
Instrument ID (DPB)	107331
Run mode	Third-octave band Environmental
Run start	07/09/1999 11:38:59
Run end	07/09/1999 11:42:36
Run duration	000 00:03:37.58
Last calibration	07/07/1999 14:33:40
Measurement range	65 - 140 dB
Microphone response	Free Field
Polarizing voltage	Off
Time weighting	F
Frequency weighting	L, A
Change rate (Q)	3
Period time	500 ms
Periods too short for LNs	Yes
Wavefiles recorded	No
Wavefile sample interval	1 s
Number of records	435
Alarms enabled	No
Overload occurred	No
Low battery occurred	No
Power source used	No



C:\DB2\FAU06.DTA
 Function = Leq
 Record start = 07/09/1999 11:38:59
 Duration = 000 00:03:37.6
 Record = Cumulative
 Flags: ----
 Cursor: Band = Broadband, Frequency weighting = L, Level = 85.5 dB