DYNAMIC CHARACTERIZATION OF DRILLED SHAFTS FOUNDATIONS ON DOREMUS AVENUE BRIDGE

by

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ABSTRACT OF THE THESIS

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Dynamic properties of the drilled shaft foundations supporting Doremus Avenue Bridge were determined by forced vibration testing. The main two objectives of the substructure testing at Doremus Avenue Bridge were: (1) site evaluation with respect to the dynamic soil properties, and, (2) shaft evaluation for the purpose of definition of their dynamic stiffness. The site characterization entailed crosshole testing for the purpose of evaluation of the dynamic properties of soil such as shear wave velocity and shear modulus profile. The drilled shaft impedance evaluation was done through forced excitation using an electromagnetic shaker. The responses of the tested shaft, as well as the response of adjacent shafts, were measured for the purpose of evaluation of the shaft interaction. To gain a better insight into the shaft dynamics, one of the shafts was instrumented with five triaxial geophones distributed along the full length of the shaft. The scope and results of the site characterization, shaft impedance and shaft interaction evaluation are presented and discussed in this research.

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CHAPTER 1

Introduction

American Association of State Highway Transportation Officials (ASHTO) adopted the Load Resistance Factor Design (LRFD) as the standard by which all the future bridges will be designed. The LRFD specifications consider the uncertainty and variability of the materials, loading and the behavior of structural elements through extensive probabilistic analysis and, therefore, continue to be refined and improved. Many of Specifications' design approaches and methodologies have been adopted with limited or virtually no experimental validation.

The Doremus Avenue Bridge located in Newark, NJ, is the New Jersey's first bridge designed according to LRFD specifications. It has been selected for instrumentation, testing and monitoring during the construction and under traffic conditions for the purpose of evaluation of the LRFD specifications. Part of the research project includes the instrumentation, testing and monitoring of the bridge substructure.

The dynamic soil properties are needed for any kind of dynamic foundation analysis and soil-structure interaction analysis. The dynamic characteristics of the bridge foundation are to be used for more realistic modeling of the bridge behavior under dynamic loading

The obtained dynamic stiffness of the drilled shafts will be utilized for two purposes. The first purpose will be to calibrate the existing numerical models. The second purpose will be to implement them in the finite element model of the whole bridge for the purpose of evaluation of the effects of the soil-foundation-structure interaction (SFSI) on this practical bridge. While in most cases taking the SFSI into account can be beneficial for the dynamic response of structures in terms of the reduction of the forces in the structures due to seismic loading, it was also shown that SFSI may have detrimental effects on the imposed seismic demand (Mylonakis and Gazetas, 2000). Certainly, more experimental data is needed to get better insight into the effects of the SFSI by taking into account specific characteristics of soil, foundation, dynamic loading and superstructure.

Even though significant research has been conducted and reported on this topic, there is still a need for further experimental data to attain better insight in deep foundations dynamics, as well as in dynamic soil-foundation-structure interaction.

There are not many guidelines for everyday engineering practice as how to design the foundations under dynamic loading, especially deep foundations, such as piles and drilled shafts. Also, there are little experimental data on a full-scale model, which could confirm and improve existing theoretical models. Scaled models and lab tests alone cannot reveal the realistic behavior of foundations. A significant number of cases of damage to piles and pile-supported structures during earthquakes have been observed, but a few instrumented records of the response of such structures have been obtained.

Proposed research will be done within the Doremus Avenue Bridge Project. The bridge is located in Newark, NJ, and the construction project involves replacement of an existing bridge. Proposed research will concentrate on the characterization of bridge foundations in respect to dynamic loading, such as earthquake and vehicular loading. For that purpose the substructure testing at Doremus Avenue Bridge includes the following:

- Site characterization with respect to dynamic soil properties
- Drilled shafts impedance evaluation
- Drilled shafts and column instrumentation for future bridge monitoring

An extensive study on dynamic pile behavior has been done in the past years. This area is still of big interest in research, because the assumptions taken should be verified and there is a need to verify theoretical and numerical models using the experimental data.

Contribution of the proposed research should be the following:

An extensive research has been done on single pile and pile group foundations. Models for the piles can be used for the drilled shafts, but assumptions, especially on rigid and flexural behavior of the shaft, should be verified. Important difference exist due to installation procedure, large diameter and smaller length to diameter ratio compared to piles (PoLam et al, 1998)

Drilled shafts are usually extended into bridge columns and are not connected with the cap at the top. The pile group effect in previous research has been examined assuming that the piles are connected by the cap. Therefore, the group effect of the drilled shafts will be investigated.

The conducted analyses of pile response have been done using assumptions of specific type of soil, either clay or sand. The proposed research deals with soft soil indicated at the actual sit (fill, peat, silt type of soil).

Experimental data obtained at the location of the Doremus Avenue Bridge should be an addition to the experimental data reported in previous research for the purpose of better understanding the behavior of deep foundations under dynamic loading.

Obtained impedance functions from experiments and numerical analysis and their implementation in the bridge model should provide better insight into soil-structure interaction and advantages and disadvantages of using it in dynamic analysis of the bridge.

1.1 Research Objectives

The main objective of the proposed research is to evaluate the drilled shafts dynamic characteristics (impedance functions) and the effect of foundation flexibility on the response of bridges to dynamic loading like earthquake and vehicular loading. The research will concentrate on field evaluation of the dynamic response (impedances) of drilled shafts.

The specific objectives of the research are:

1. Site characterization with respect to the dynamic soil properties.
2. Evaluation of the dynamic stiffness (impedance functions) of drilled shafts.
3. Calibration of existing theoretical models using the collected data. Pile-soil interaction, as well as interaction between adjacent piles will be considered.
4. Integration of the developed impedance functions in the model of the bridge structure and the study of the effects of soil-foundation-structure interaction for dynamic loading.

1.2 Thesis Organization

Following the introduction in chapter two the background on dynamic behavior of deep foundations is reviewed. The dynamic behavior of single pile and pile groups is described and different approaches and solution presented. Also, previous experimental field testing, as well as laboratory testing, of the deep foundations under dynamic loading is presented.

In chapter three the site and construction of the Doremus Avenue Bridge is described. Soil profile and characteristics of the soil layers are given. For the purpose of obtaining dynamic characteristics of the site crosshole testing was utilized. The crosshole test is described, and obtained data presented. Results of the crosshole testing is shear wave velocity profile and shear modulus profile.

Chapter four presents field testing of the drilled shafts for the purpose of obtaining dynamic stiffness characteristics of the foundations. The test procedure and the equipment used are described as well as the instrumentation of the shafts. Obtained results are presented as dynamic response of the single shaft and for the group of shafts.

Chapter five provides the summary of the research, conclusions and gives suggestion for the future research.

Chapter six provides list of references.

CHAPTER 2

Background

2.1 Introduction

Deep foundations are used to transfer load through week near-surface soil to bedrock or to deeper strong soil deposit. The types of deep foundations can be classified according to several factors, such as: material, methods of transferring load (end bearing or floating), method of installation, and influence of installation on soil or rock. Drilled shafts are large diameter deep foundation constructed by directly pouring concrete in a drilled hole. Nowadays they are widely used to support bridge structures. Usually they are extended in bridge column above the ground. Drilled shafts foundations are used to support heavy load and minimize settlement, support uplift and lateral loads. They can be constructed properly in wide variety of soils. Large diameter drilled shaft can be installed to replace group of smaller diameter driven piles. In that case there is no need for pile cap, which makes drilled shafts more economic solution. To insure quality of poured concrete depth to diameter ratio should not be more than 30 (O'Neil and Reese, 1999).

The analysis and design of drilled shafts are similar for those of driven piles. Important differences exist due to instillation procedure, large diameter, and smaller length to diameter ratio, and drilled shaft analysis requires some additional consideration (PoLam et al, 1998).

In this chapter dynamic analysis and procedures for piles and drilled shafts are reviewed. Much more research has been done for pile foundations, making it start point for the analysis of drilled shafts under dynamic loading. Pile foundations are usually placed in a group and connected with a pile cap. The dynamic response of a pile group is even more complicated than the response of a single pile. That is because of the influence of one pile on other piles in the group. Each pile can be analyzed as a single pile if spacing between them is large. The spacing is considered to be large if it is greater than 6 - 8 pile diameters. For closely spaced piles the effect of the interaction should be considered. Foundations impedance functions are needed for soil-structure interaction analysis. Brief overview of soil-structure interaction methodologies is also given.

Parallel with the developing numerical model for dynamic behavior of deep foundations experimental work was also a big part of the research. Data from the experimental work are voluble information that are helping in developing more realistic models, and give insight in different factors influencing dynamic response of deep foundations. Therefore, experimental work is also a subject of literature review in this chapter.

2.2 Soil-Structure Interaction

Objective of the problem of soil-structure interaction is to determine dynamic response of the structure interacting with surrounding soil. Dynamic behavior of the foundation is influenced by the soil as well as the superstructure. Therefore, to describe the behavior of a pile group under dynamic loading, the soil-pile-structure interaction (SPSI) should be used. There are two main approaches to the solving problem of SPSI: direct method and substructure approach. Direct method takes into account the whole structure and surrounding soil. Using this method nonlinear behavior can be analyzed. Usually this method is computationally complicated. An alternative and computationally efficient method is substructure approach. According to this approach the SPSI problem is decomposed into three components: soil, foundation and superstructure (Gazetas and Mylonakis, 1998). Following is the description of the substructure approach.

The total dynamic stress in a pile can be obtained by the superposition of two independent analyses: kinematic and inertial (Wolf, 1985). Shear waves propagating in soil interact with piles and distort them, thus producing kinematic bending moment and stresses. On the other hand, the acceleration in the superstructure produces base shear and an overturning moment that must be resisted by the foundation.

To analyze the soil-structure interaction, first the free field response of the site should be determined, and then the modification of seismic motion as a result of an interaction between the structure and soil. If the linearity is assumed, then the principle of superposition can be applied and the displacement due to an earthquake can be divided into a kinematic and an inertial displacement. Decomposition of seismic soil-pile-structure response is shown in Figure 2.1.

The kinematic interaction can be computed assuming that the mass of the superstructure is set to be zero. The calculated displacement is than used as the foundation input motion. For the inertial interaction the mass of soil, foundation and structure is taken into account.

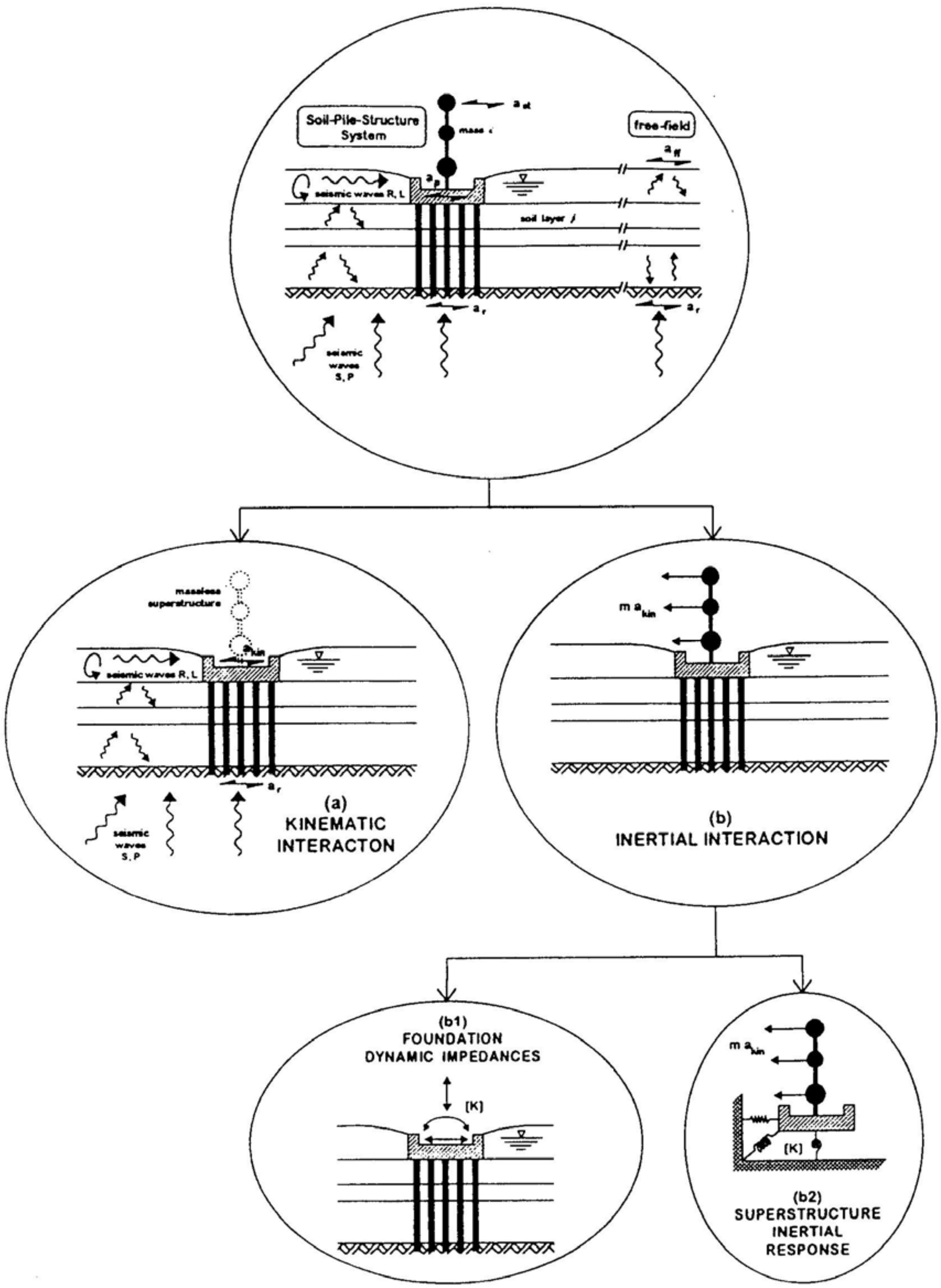


Figure 2.1 Schematic Illustration of Soil-Pile-Structure Interaction and its Decomposition into Kinematic and Inertial Interaction (after Gazetas and Mylonakis, 1998)

The displacement obtained from the kinematic interaction is used as an input in the inertial analysis. The analysis of inertial response is further divided in two independent analyses:

- Computation of the impedance functions at the pile head (springs and dashpots),
- Analysis of the dynamic response of the superstructure supported by springs and dashpots.

It should be noted that the superposition principle exist only for linear behavior of soil, foundation and structure. Soil nonlinearity can be taken into account by iterative viscoelastic wave propagation analysis and superposition can be employed for moderately nonlinear system.

Taking SPSI into account can be beneficial for the dynamic response of structures in terms of the reduction of the forces in the structures due to dynamic loading. It is shown that, in certain seismic and soil environments, SPSI may have a detrimental effect on imposed seismic demand (Mylonakis and Gazetas, 2000). More research is needed on SPSI to examine its effects on the response of structures taking into account characteristics of soil, foundations, dynamic loading and structures.

2.3 Single Pile

Stiffness and damping of a pile are affected by its interaction with surrounding soil. This interaction is considered in terms of continuum mechanics and taking into account elastic wave propagation. Many authors have been investigating this problem (Novak (1974), Dobry(1982), Gazetas(1984), Kaynia(1982), Wolf (1978), etc.) and from conducted studies it can be seen that soil-pile interaction modifies pile stiffness making it frequency dependent and generates geometric damping through energy radiation.

2.3.1 Dynamic Stiffness and Damping of a Pile

Dynamic stiffness and damping of the pile is described using a complex stiffness usually called the impedance function. The concept of dynamic stiffness can be described on an example of a single degree of freedom system (SDOF). For harmonic excitation with frequency ω, the applied load with amplitude $P(\omega)$ will cause displacement with amplitude $u(\omega)$. The loading and the displacement are related by the dynamic stiffness coefficient in the frequency domain $S(\omega)$, as follows:

$$P(\omega) = S(\omega)u(\omega) \tag{2-1}$$

Dynamic stiffness coefficient has complex value:

$$S(\omega) = K - \omega^2 M + i\omega C \tag{2-2}$$

where

K is stiffness (spring) coefficient,

C is viscous damping coefficient, and

M mass of the SDOF.

To make $S(\omega)$ nondimensional, it can be normalized using the static stiffness coefficient K. That leads to the expression of dynamic stiffness as:

$$S(\omega) = K\left(k(\omega) + i\omega\, c(\omega)\right) \tag{2-3}$$

Where

$k(\omega)$ is the dimensionless spring coefficient $k(\omega) = 1 - \omega^2 \frac{M}{K}$, and

$c(\omega)$ is the dimensionless damping coefficient $c(\omega) = \frac{C}{K}$.

Then the dynamic stiffness can be written in simpler form as:

$$S = K_1 + iK_2 \tag{2-4}$$

The real part (K_1) represents the stiffness and the inertia of the pile-soil system, and the imaginary part (K_2) describes the damping due to energy dissipation in the soil and the pile and can be defined in terms of the constant viscous damping (C).

The complex stiffness K, or its parts K_1 and K_2, can be obtained experimentally or theoretically. In the theoretical approach, the dynamic stiffness is obtained by calculating a force needed to produce a unit amplitude displacement at the pile head in the prescribed direction. In the experimental approach, an introduced force and the produced displacement in the same direction are recorded. Since the force and the displacement are related by the dynamic stiffness, then it can be obtained.

The mass used in this description of the complex stiffness corresponds to the mass of the SDOF system. In the problem of the dynamic pile response it is more complex, because it includes the mass of the pile, as well as the inertia of the surrounding soil.

Many studies have been conducted with the purpose of obtaining the dynamic stiffness of the pile. The main goal of almost all of them was to find simple models and methods to define the dynamic stiffness of the piles, so that they can be used in engineering practice. The analytical approaches treat the interaction between soil and the pile in terms of continuum mechanics, schematically shown in Figure 2.2.

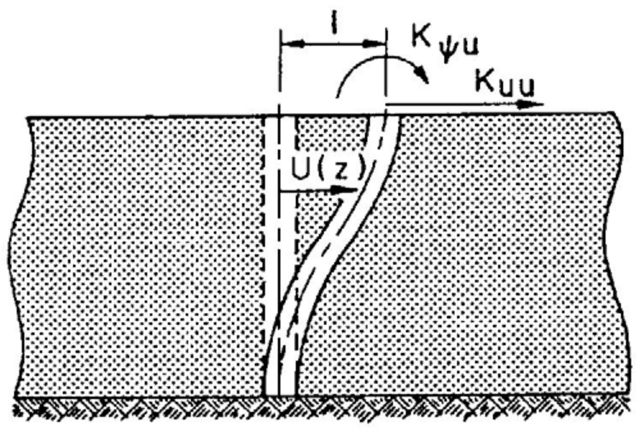


Figure 2.2 Illustration of Soil-Pile Interaction (after Novak, 1991)

It is difficult to solve the problem of a dynamically loaded pile even with the idealistic assumption of linearly elastic, or viscoelasticity homogenous soil and the elastic pile perfectly bounded to the soil. Most of the studies use the following two approaches:

- beam on elastic foundations (BEF), using Winkler's model,
- finite element method (FEM)

Using the BEF method, the surrounding soil is replaced by Winkler's springs along the pile length. In this case the stiffness of springs is derived by relating a harmonic force and displacements at a number of contact points between the pile and the surrounding soil for an assumed deformation pattern. The shape of the displacement pattern depends on the type of excitation and the excitation frequency. This implies that even for an uniform soil profile the equivalent soil springs should vary with depth and their variant should be a function of loading and frequency. This method has been used extensively in theoretical studies and engineering practice due to its analytical simplicity. In most cases each soil layer is represented by its stiffness and equivalent dashpots. Dashpots represent the combined effect of hysteretic (material) and radiation (geometrical) damping in soil. Figure 2.3 shows the model for soil-pile interaction using Winkler's model for seismic excitation (Gazetas and Mylonakis, 1998).

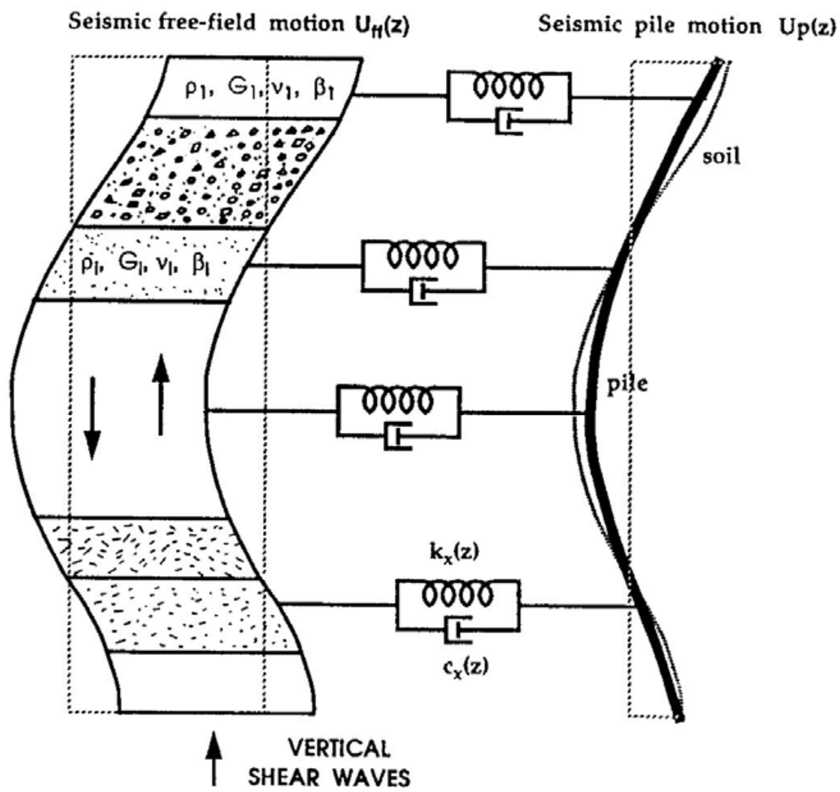


Figure 2.3 Model for Soil-Pile Interaction Using Winkler's Model in Multi-Layered Soil (after Gazetas and Mylonakis, 1998)

Using FEM soil and the pile can be described much more realistically and numerous factors can be taken into account. The advantage of FEM is the ability to perform dynamical analysis in fully coupled manner. It is also possible to model any arbitrary soil profile and study 3-D effects. Since the soil is not a finite structure, the question is how to model the far field so that the energy is not trapped in a 'box'. To represent far field Kuhlemeyer (1979) used energy absorbing boundaries. Chan and Poulos (1993) presented the finite model of the pile and the far field was represented using infinite elements.

The most common assumptions in the soil-pile analysis are as follows:

- soil is assumed to be homogeneous, isotropic and linearly viscoelastic medium with hysteretic damping
- if the soil is assumed to be a layered medium, the soil properties are assumed constant within each layer, but may vary in individual layers
- the pile is vertical, linearly elastic and of a circular cross-section
- perfect bound between soil and pile was assumed

Novak (1974) formulated a simple approach based on plane strain soil reactions, which can be interpreted as dynamic Winkler springs and dashpots attached directly to the pile. The Winkler springs were frequency dependent and complex. A perfect bound between pile and soil was assumed. This solution identified dimensionless parameters and a number of charts and tables for the dynamic stiffness and damping are presented. The study has shown that the pile stiffness and the damping are almost the same for fixed and pinned pile heads if the slenderness ratio of the pile is larger than about 25. The application of the same approach to a vertically vibrating pile (Novak, 1977) indicated the great sensitivity of the pile behavior to tip condition. The analysis of pile stiffness and damping by Novak et al. (1983) was performed for two different types of soil - a homogenous and a parabolic variation of shear modulus. It was shown that the soil with parabolic variation of shear modulus provides much less stiffness and damping than the homogenous one. Similar analysis and result was obtained by Gazetas (1984) for a homogeneous soil, linear and parabolic variation of shear modulus.

Novak et al. (1983) have shown analytically that the parameters that control the stiffness and the damping of the piles are as follows:

- dimensionless frequency, $a_0 = \dfrac{\omega R}{V_s}$,
- relative stiffness of the soil and the pile which can be described as a modulus ratio of the pile material and soil, E_p / G_s ,
- slenderness ratio of the pile, L/R ,
- material damping of soil and the pile,
- tip condition and
- variation of soil and the pile properties with depth.

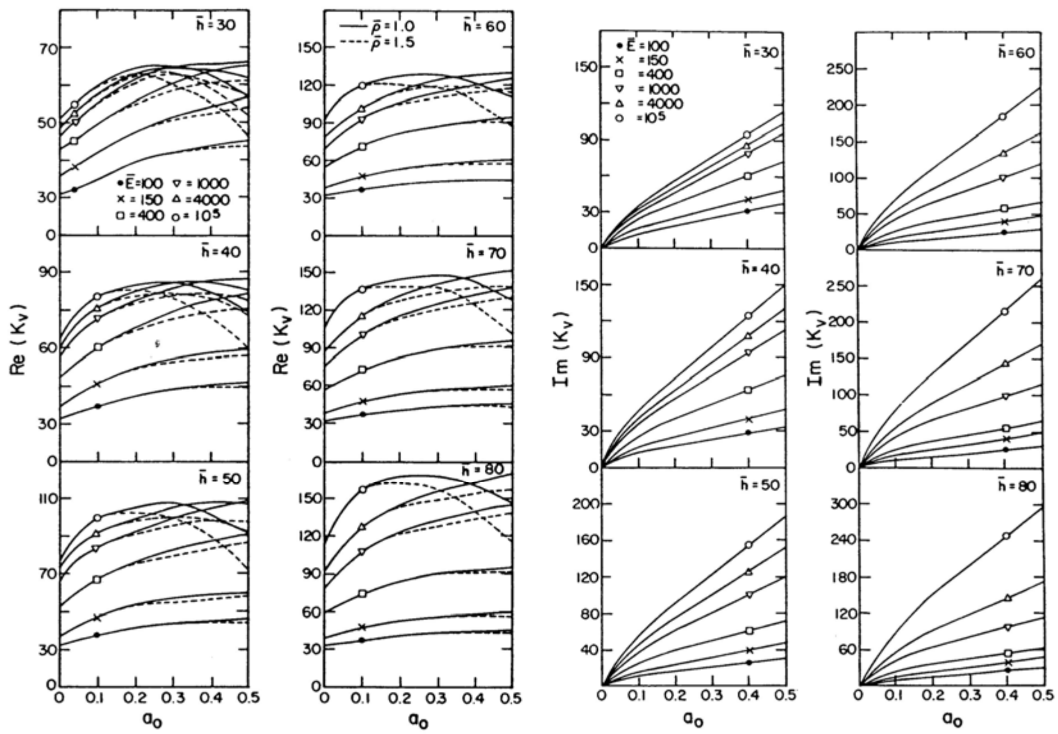
Taking into account those parameters, Novak et al. (1983) provided charts for the dimensionless parameters for pile stiffness and damping as functions of frequency, mass ratio and pile slenderness.

The next step in analyzing the dynamic response of the pile was introducing the layered soil profile into analysis (Gazetas and Dobry 1984). The starting point for this analysis was the determination of the file deflection profile for static force at the top of the pile. From there on, the method was based on simple physical approximations. These approximations refer to the nature of radiation and material damping at different depths, the way these approximations combine and the influence of natural frequency of the whole deposit on the response of the pile. The computation method was quite simple and noncircular pile cross-sections could be analyzed using the provided table and simple radiation damping model.

Rajapakse and Shah (1989) conducted an extensive parametric analysis to investigate the impedance characteristics of a single elastic pile embedded in an elastic soil medium. The main assumption was that the pile was long enough so that it can be consider flexible. If the pile can be considered flexible, than the condition at the pile end has no or just negligible influence on the pile response. Numerical solutions for pile impedances were presented as a set of non-dimensional curves. The impedances are shown as a function of:

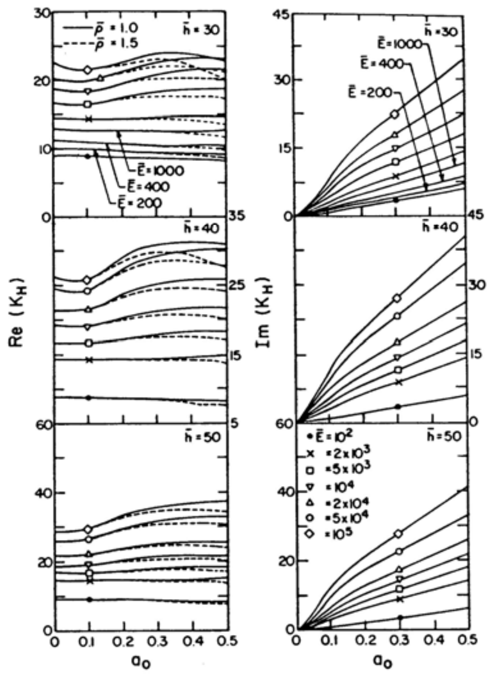
- dimensionless frequency, a_0 ,
- pile flexibility, $\overline{E} = E_p / E_s$,
- slenderness ratio of the pile, $\overline{h} = L/R$,
- mass-density ratio, $\overline{\rho} = \rho_p / \rho_s$, and
- Poisson’s ratio of the soil was taken as $\nu_s = 0.25$.

Real and imaginary part of pile impedance after Rajapakse and Shah (1989) are shown in Figures. 2.4 to 2.6.



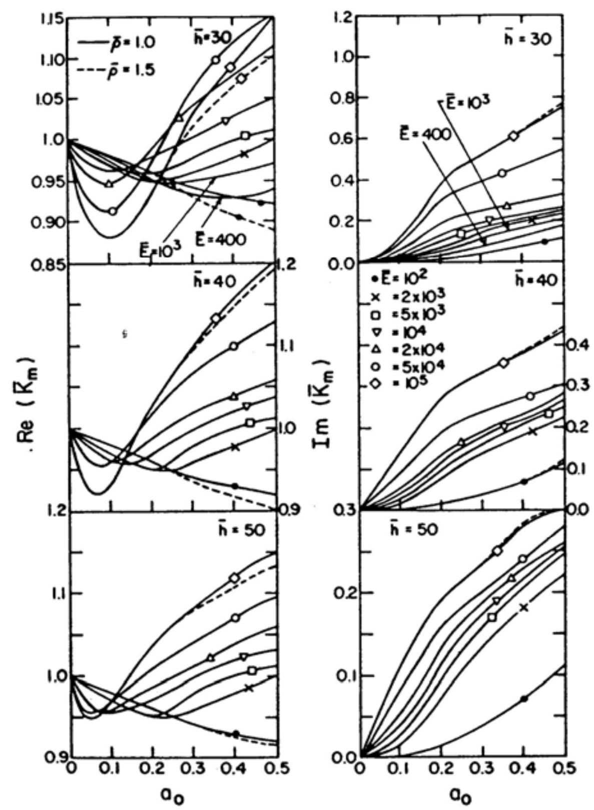
(a) (b)

Figure 2.4 Non-Dimensional Axial Impedance (a) Real Part, (b) Imaginary Part



(a) (b)

Figure 2.5 Non-Dimensional Horizontal Impedance (a) Real Part, (b) Imaginary Part



(a) (b)

Figure 2.6 Non-Dimensional Moment Impedance (a) Real Part, (b) Imaginary Part

In almost all of the above analyses it was observed that if the pile is long enough its behavior under a dynamic load (especially for lateral vibration) is not influenced by the tip condition, or if it is floating or end-bearing pile. Observed were differences in rigid and flexible behavior of a pile. Rigid versus flexible pile behavior is shown in Figure 2.7.

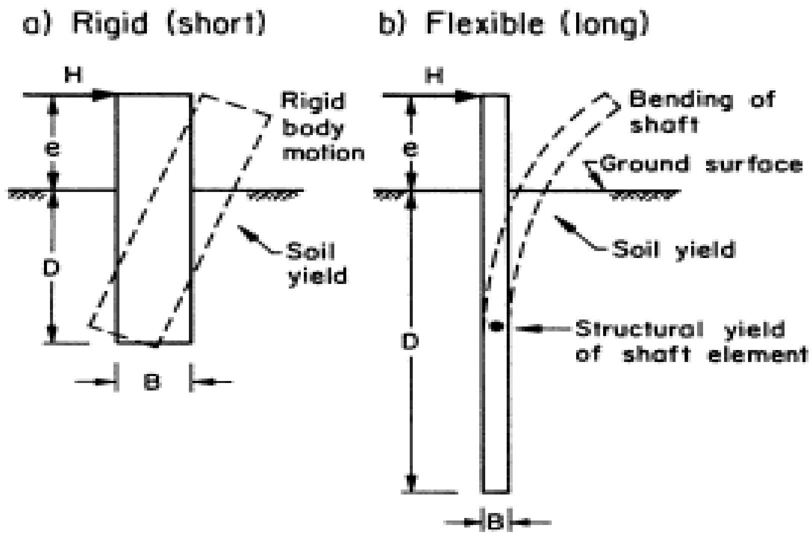


Figure 2.7 Rigid Versus Flexible Pile Behavior

To distinguish between short and long piles various researchers have proposed different criteria. Pulos and Davis (1980) proposed a flexibility factor K_R as follows:

$$K_R = \frac{E_p}{E_s}\frac{I_p}{L^4} \qquad (2\text{-}6)$$

Pile is considered as rigid if $K_R > 10^{-5}$, and flexible if $K_R < 10^{-5}$

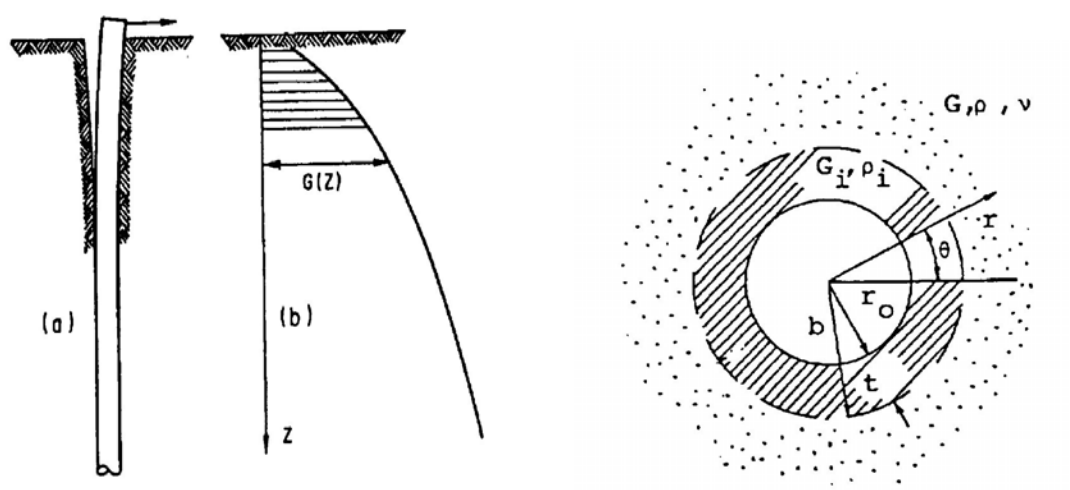
Dobry et al. (1982) introduced a flexibility factor S_H as follows:

$$S_H = \frac{L/R}{\left(E_p/E_s\right)^{1/4}} \tag{2-7}$$

Pile is considered short if $S_H < 5$, and long if $S_H > 5$

The flexibility factor K_R is defined for a pile of an arbitrary cross-section, where I_p is moment of inertia of the pile cross-section. The flexibility factor S_H is a simplified version of the factor K_R used for the pile with circular cross-section.

Assuming that the soil is homogeneous, the stiffness and the damping can be overestimated, which has been shown by comparing the results from experiments with theoretical predictions (Novak et al. 1976). One of the reasons is the separation of the pile from soil schematically shown in Figure 2.8.



(a) (b)

Figure 2.8 (a) Separation of he Pile from soil and modulus reduction towards ground surface, (b) Cylindrical Boundary Zone Around the Pile (after Novak 1991)

Novak et al. (1980) presented an approach to take into account the disturbed zone around the pile (Fig. 1.7 b) to consider the effects of gapping, slippage and lack of bound between the pile and soil. A rigorous approach to these effects is difficult and therefore approximation theories have to be used. A disturbed cylindrical zone around the pile was assumed with a reduced shear modulus and increased material damping. Novak et al. (1980) presented the reactions of such a composite media for all vibration modes. The result of that analysis is shown in Figure 2.9, and comparison between response with and without the weak zone around the pile is shown.

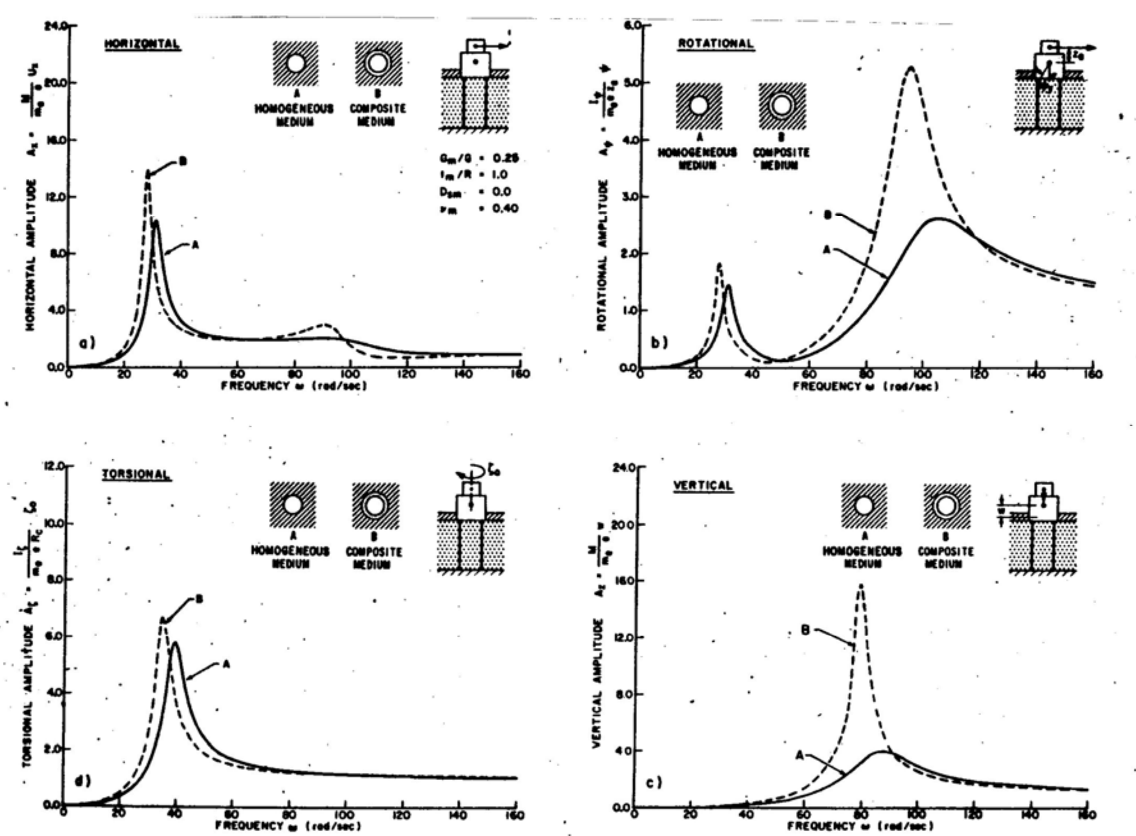


Figure 2.9 Response of the Pile Supported Foundation Calculated with and without Weaker Zone around Pile (after Novak et al. 1980)

The length of pile separation is difficult to define, because it depends on many factors involved in pile response, and at this time only empirical suggestions can be made. Gucunski (1983) analyzed the influence of the weakened zone on the pile response using program PILAY2 and comparison with experimental data was done. It has been shown that the greatest influence on the lateral dynamic pile response has the shear modulus ratio of the weakened zone and surrounding soil. This effect was observed mostly in the upper part of soil, by the depth of 5 – 10 pile diameters. The soil around the pile by the depth of 2 pile diameters could be neglected, i.e. shear modulus can be used as zero.

The response of the piles to the dynamic loading is still of big interest in research because there are many uncertainties in the pile and soil behavior under dynamic loading. Soil is not a linear material and new studies have been implementing nonlinearity and plasticity of the soil in the pile-soil model (Yang et al. 2002). El Naggar and Bentley (2000) developed two dimensional analyses to model the pile response to dynamic loads. The proposed model incorporates static p-y curve approach and the plane strain assumption to present soil reaction. The inclusion of damping resulted in dynamic p-y curves which is function of static p-y curve, velocity of soil particles at a given dept and frequency of loading. Because of the complexity of dynamic behavior of piles there are also attempts to estimate maximum internal forces of piles subjected to lateral seismic excitation using pseudostatic approach (Tabesh and Poulos, 2001).

Ongoing researches are still trying to provide reasonable model of the dynamic response of the pile (Poulos, 1999) with approximations that should be accurate enough, but still easy to use in engineering practice.

2.4 Pile Groups

Pile foundations usually consist of a number of piles placed in a group to support the superstructure. The dynamic stiffness of a pile group is not equal to the sum of all the stiffnesses of each pile, because each pile is affected by its own load and by the load and deflection of its neighboring piles. The interaction effect is caused by waves generated on the periphery of each pile and propagated towards neighboring piles. This pile-to-pile interaction is frequency dependent. If the distance between the piles is large than it is reasonable to say that the total stiffness and damping can be obtained as the sum of individual stiffness and damping coefficients. In most cases that is not the case if the piles are closely placed. Than the displacement is increased due to the displacement of all the others piles, and stiffness and damping of the group are reduced.

Novak (1977) suggested a rough approximation for vertical vibration of a pile group in which the stiffness and the damping of the group can be written as:

$$k_{(g)} = \frac{\sum_{i=1}^{n} k_i}{\sum_{r=1}^{n} \alpha_r} \quad , \quad c_{(g)} = \frac{\sum_{i=1}^{n} c_i}{\sum_{r=1}^{n} \alpha_r} \tag{2-8}$$

Where α_r is the interaction factor describing the contribution of the r^{th} pile to the displacement of the reference pile. Exept for α_1, which equals one, all the other factors are less than 1 and decreasing with distance between piles. An estimated value for α_r can be obtained from a static solution.

One of the first analytical analyses of the pile-soil-pile interaction was conducted by Wolf and von Arx (1978). They used an axisymetric finite element formulation to establish the dynamic displacement field due to ring loads. Impedance and transfer functions of a group of vertical piles placed in horizontally stratified, viscoelastic soil were derived. The method separates soil and the piles, introducing unknown interaction forces. The results showed strong dependence on frequency, number of piles and pile spacing. The pile-soil-pile interaction was shown to be important for all modes of vibration of the pile group.

Kaynia and Kaussel (1982) performed an analytical investigation of the pile group behavior. The formulation was based on the introduction of a soil flexibility matrix, as well as on the dynamic stiffness and the flexibility matrix of the piles. The discretized uniform forces were then related to the corresponding displacements at the pile soil interface. The pile group behavior is shown to be highly frequency dependent. It was also shown that the interaction effects are stronger for softer soil media. Figure 2.10 shows the normalized dynamic stiffness of a 4x4 pile group with different pile spacing *s/d* (*s* is spacing between piles and *d* is the pile diameter). Comparison with the single pile behavior is also shown.

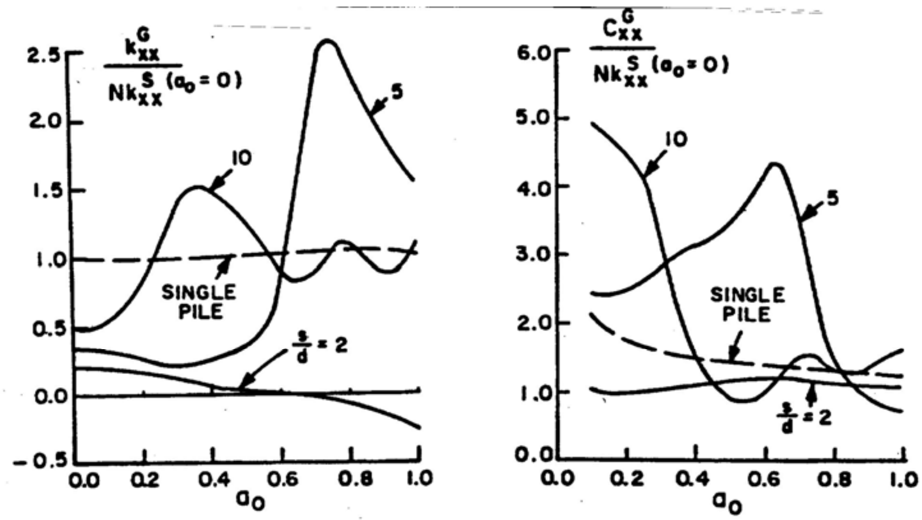


Figure 2.10 Normalized Dynamic Stiffness and Damping of 4x4 Pile Group for Different Spacing Ratios (s/d), after Kaynia and Kaussel (1982)

A group of floating cylindrical piles embedded in a uniform stratum or half-space, subjected to an arbitrary harmonic force, was studied by Dobry and Gazetas (1988). The response of the pile group was obtained from the interaction factors derived from the study of only two piles at a time. In this simplified approach other piles, beside the two studied, and were considered transparent. A dynamic interaction factor was introduced as a function of frequency. Those factors were derived for all modes of vibration. The inputs required in the method are the dynamic impedance of a single pile and the soil parameters (shear wave velocity, Poisson's ratio and damping ratio).

One of proposed procedure to obtain the pile-soil-pile interaction is by Makris and Gazetas (1992) for a harmonic excitation at the pile head and a seismic-type excitation.

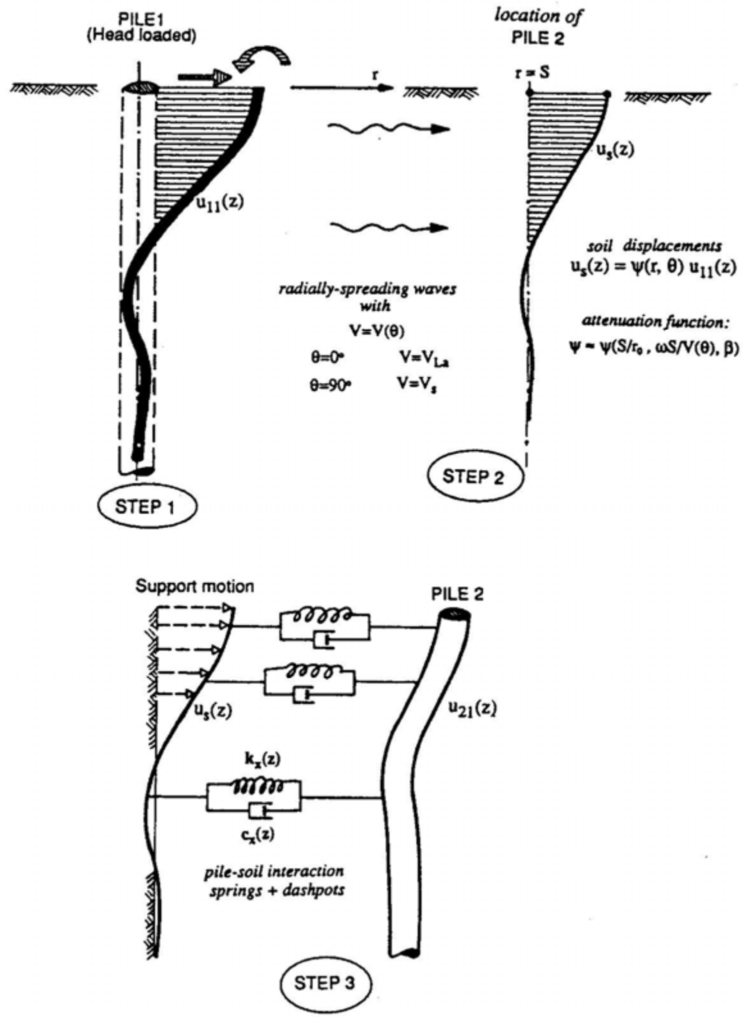


Figure 2.11 Schematic illustration of the procedure for computing the influence of PILE 1 upon the adjacent PILE 2 – deforming under harmonic lateral head load (after Makris and Gazetas, 1992)

Step 1: Obtain the lateral deflection of the pile head of a single pile $u_{11}(z)$ using some of the developed methods such as finite element method, semi-analytical formulation, beam on Winkler foundation method etc.

Step 2: The pile with the head displacement $u_{11}(z)$ generates waves at all points along the pile. It is assumed that these waves spread out horizontally. At the location of pile 2, the arriving attenuated waves will produce soil displacement u_s (if the pile 2 is not present).

Step 3: The presence of pile 2 will modify the arriving wave field $u_s(z)$ by reflecting and diffracting the incoming waves. Results depend on the relative flexural rigidity of the pile. Two extreme cases are possible. Pile 2 may just follow the ground (flexible pile and smooth u_s) or it may remain nearly still (rigid pile and rapidly fluctuating u_s).

One of the main simplifications here is the decoupling of the mechanism that transmits the motion in each of the three steps outlined above. A similar procedure is proposed by the same authors for the seismic type of excitation. The difference is that in Step 2 the difference $\Delta u_{11} = u_{11} - u_{ff}$ between the single pile deflection and free-field soil displacement should be calculated. The schematic procedure is shown in Figure 2.12.

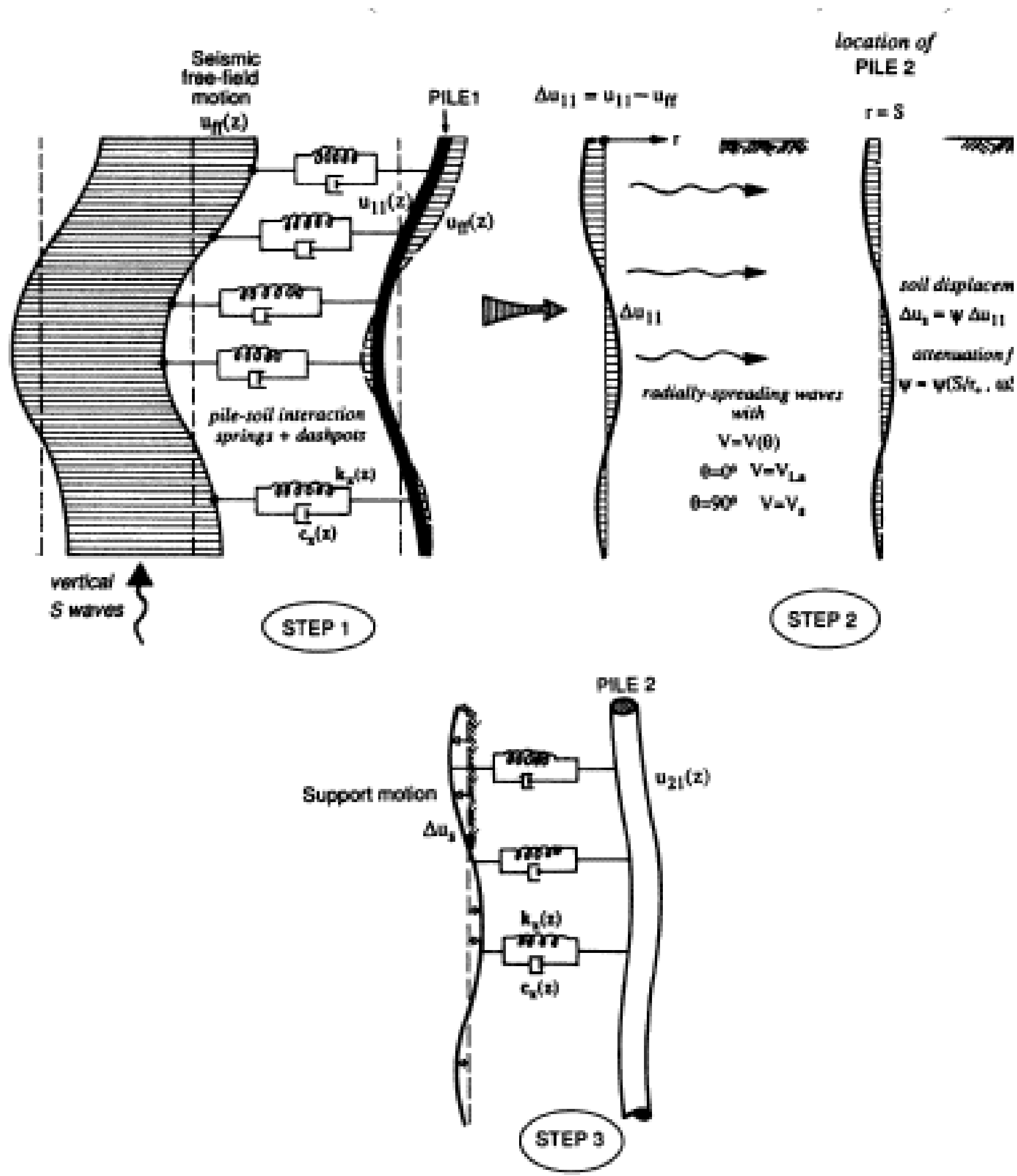


Figure 2.12 Schematic illustration of the procedure for computing the influence of PILE 1 upon the adjacent PILE 2 - deforming under a seismic-type excitation (after Makris and Gazetas, 1992)

Sheta and Novak (1982) included a weakened zone around the piles in the pile group model. The influence of the group effect and the weakened zone effect is shown in Figure 2.13.

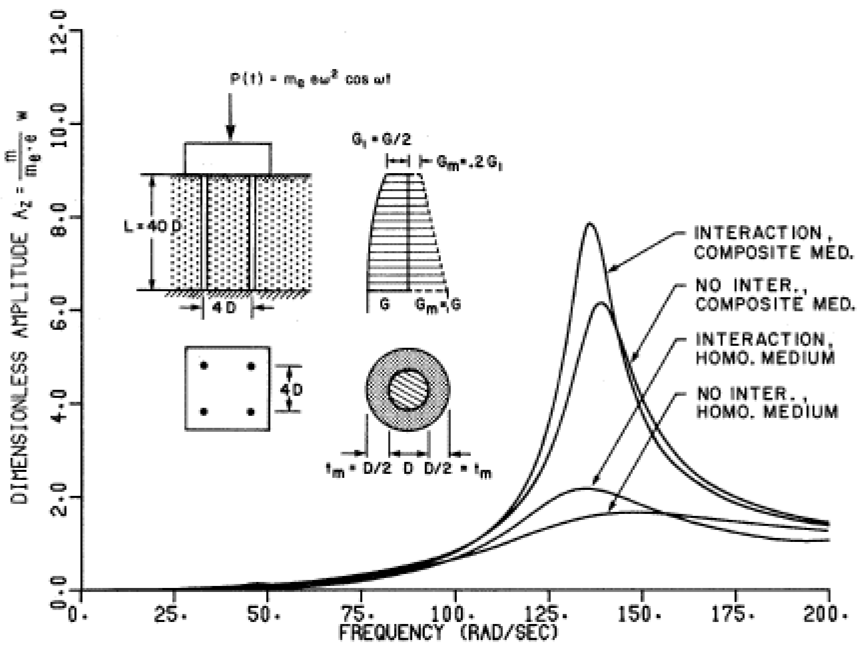


Figure 2.13 Dynamic Response of Pile Group with the Influence of Group Effect and the Weak Zone Effect (after Sheta and Novak, 1982)

El Naggar and Novak (1994) described the nonlinear model for dynamic axial pile response that consisted of a slip zone, inner field and outer field. The same authors (1995) described a dynamic nonlinear time-domain Winkler soil-pile interaction model that could take into consideration both axial and lateral pile group response.

As for single pile, there is a need to develop simple models and procedures for pile group foundations to predict and assess their response to dynamic lateral loading. Blaney and O’Neill (1991) develop a procedure to predict dynamic lateral pile group response in clay from single pile test. Mostafa and El Naggar (2002) introduced “dynamic” p-multipliers to relate dynamic load transfer curves of a pile in a group to the dynamic load transfer curves for a single pile. The dynamic p-multiplier are fund to vary with the spacing between piles, soil type, peak amplitude of loading and the angle between the line connecting any two piles and the direction of loading. The study indicated that the p-multipliers are affected by pile material and geometry, pile installation method and pile head connections. many studies concentrated to studying dynamic response of pile groups in different types of soils like poroelastic medium (Wang et al, 2003), in sands (Yang and Jeremic, 2003).

2.5 Previous Experimental Work

A considerable body of theoretical research has been done in the area of dynamic response of piles, especially under linear elastic assumptions. There has been a need to verify theoretical and numerical results using experimental data. Experiments reported in the literature can be grouped into two categories: tests on pile models and full-scale tests. Full-scale test have the advantage of providing correct soil and pile stresses; however, it is difficult to define site conditions and soil properties due to spatial variability that may not be accurately quantified due to small number of discrete subsurface investigation location and complex geology. Laboratory model tests may be useful for conducting parametric studies in a controlled environment if the models are appropriately scaled using physical scaling laws.

Dynamic lateral pile load tests are typically performed using some type of excitation on the top of a pile. For example, Crouse & Cheang (1987) used a quick-release free vibration test on two concrete pile groups. The diameter of the piles tested was 0.32 m. The piles were embedded in 12.2 m of loose sandy soil overlaying glacial till. The measured natural frequencies of the pile groups in the horizontal direction were between 3.8 and 6.3 Hz. Blaney et al (1987) dynamically tested 3x3 group of steel pipes driven into overconsolidate clay using vertical vibration test. Han and Novak (1988) conducted experiments on large-scale model piles. The piles were 0.133 m diameter steel pipes, and 3.38 m long. Natural frequencies for horizontal response measured in the test were between 4.7 and 12.2 Hz.

El-Marshafi et al. (1992) reported experiments on two pile groups. The first set of tests was performed on a group of six steel model piles. The second set of tests was performed on 0.32 m diameter full-scale reinforced concrete group of piles 7.5 m long. A single pile, identical to those in the group, was tested at the same site using a Lazan type exciter with two rotating eccentric masses. The soil at the site was relatively homogenous sandy clay. The natural frequency for the concrete pile group was about 15 Hz. This test has shown that using the concept of a weak zone around the pile (Novak et al., 1980) improves theoretical model predictions of the pile response.

El Sharnouby and Novak (1992) conducted a test on 102 pile group. The whole group of piles was placed in a hole excavated in the field, and a fly ash/sand mixture, designed to have similar dynamic properties to the free-field, was backfilled around the pile group. Forced vibration, impact and static lateral load test were conducted. Seismic cross-hole test was conducted to verify shear wave velocity of the free-field and backfill soil. Conclusions drown from this work were the following:

- static interaction may only provide an estimate of dynamic group stiffness for small groups at low frequency, but otherwise may underestimate stiffness
- available theories overpredict damping as they do not account for soil-pile gap or soil nonlinearities
- for this particular test setup, the total mass of piles and intervening soil appeared to vibrate as rigid body.

Experimental work has showed that the dynamic response of pile foundations is site dependent. The site dependence is likely due to different soil properties, pile properties and construction method for pile or drilled shaft installation. Vibration test also show nonlinear response for both single isolated piles and pile groups. A reasonable agreement was observed between calculated and measured pile response at small displacement for test performed by Novak (1980). The damping may be overestimated unless some corrections are made for the pile separation and the influence of weakened zone (Novak, 1980). As a result of soil separation, radiation damping decreases, but hysteretic damping increases. Also there is a problem of soil stiffness degradation due to larger displacement which can cause problems in calculations if not adequately considered.

2.6 Summary

In this chapter background on response of deep foundations is presented. Brief overview of soil-pile-structure interaction is given and two direct and substructure approaches are described. Background on response to dynamic loading of a single pile as well as pile groups is presented. Some experimental work on dynamic pile testing and their results are given. Field and laboratory pile load test programs have made a vital contribution to understanding dynamic response of deep foundation and factors affecting them. Dynamic testing of single piles and pile groups has shown that the response is site, frequency, load level and pile arrangement dependent.

CHAPTER 3

Site Characterization

3.1 Introduction

Characterization of the Doremus Avenue Bridge site has been done for the purpose of the study of foundation behavior under dynamic loading. It includes site description with the soil characteristics obtained from the boring logs and dynamic soil properties obtained using seismic crosshole testing Description of the Doremus Avenu Bridge is also presented.

3.2 Doremus Avenu Bridge

The Doremus Avenue bridge construction project involves replacement of the old bridge, originally built in 1918. The Bridge is the main access route to New Jersey's seaports. Over time, a significant increase in the traffic volume over the bridge and high differential settlements of the bridge supports led to very high deterioration, affecting both the safety and serviceability of the bridge. The construction proceeded in two phases. In the first phase, half of the new bridge was constructed while the old bridge remained open to traffic. In the second phase, the old bridge was demolished and the traffic was diverted to the finished half of the bridge. The layout of the Doremus Avenue

Bridge is shown in Figure 3-1, and the photo of the bridge under construction is show in Figyre 3-2.

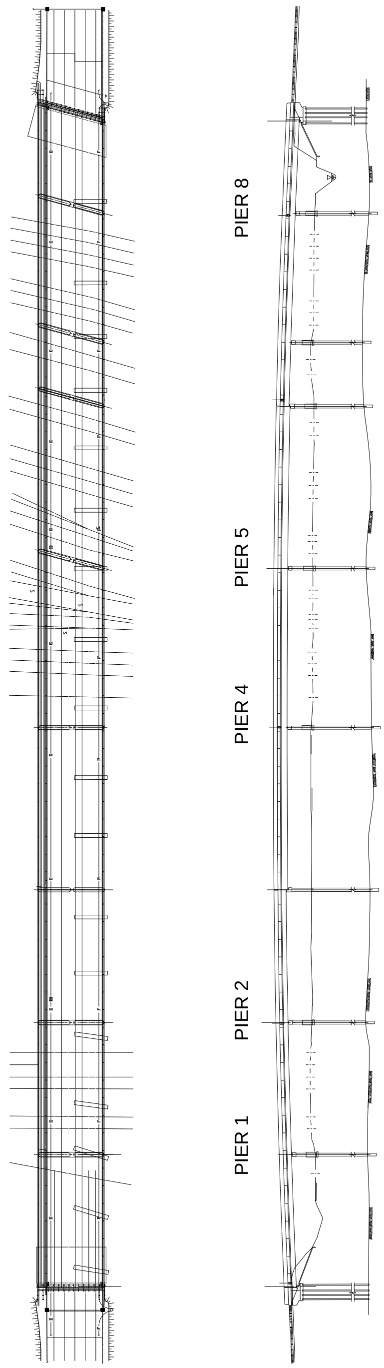


Figure 3-1. Layout of the Doremus Avenue Bridge

Figure 3-2. Doremus Avenu Bridge under construction

3.3 Soil Profile of the Doremus Avenu Bridge Construction Site

Boring logs from the site investigation performed prior to constriction of the bridge indicate that the soil profile can be generally described as a five layer system overlaying bedrock. Following is a description of each stratum and the bedrock.

- The top 4.5 to 7.6 m is granular fill with interlayers of silty clay or organic soils of depth up to 1.5 m. A variety of materials such as glass, plastic, wooden fragments were found within the fill. The average SPT N-values vary within a relatively wide range. Values as low as 1 and as high as 25 were measured.
- Below the fill is a 0.9 to 3 m thick layer of organic silt, clay, and peat. The soil is in relatively soft and compressible state. The SPT N-values were relatively low ranging between 1 and 10.

- A naturally deposited sand stratum with a thickness from 1.5 to 4.6 m underlies the organic soil layer. The sand is generally of a fine to medium to fine gradation with varying amounts of silt. Penetration resistance test indicated that this layer is a medium dense to very dense state. The SPT N-values were between 15 and 50.
- Beneath the sand layer a cohesive soil stratum consisting generally of clayley silt to clay and silt was encountered. Its thickness varies from 6 to 13.7 m. This layer is in stiff to medium stiff state. The SPT N-value of this stratum ranged from 10 to 50
- The layer of medium to fine gravel with varying amount of clay and silt underlines the layer of clay and silt. This layer has thickness of up to 9 m. The SPT –N-values for this stratum were between 15 and 50.
- The bedrock is moderately fractured shale at depths 18 to 24 m below the ground surface.

Soil parameters of the indicated soil layers are given in the Table3-1.

Table 3-1 Soil Parameters

Soli Type	Unit weight γ_s (kN/m^3)	Angle of internal friction ϕ (degrees)	Undrained shear strength S_u (kPa)
Fill	19.0	30-35	
Organic Soils (silt, clay, peat)	11.8		19.2
Sand	19.6	35	
Silt and Clay	18.9		62
Gravel	20.4	38	

3.4 Crosshole Testing

Crosshole is a seismic borehole method used to obtain low strain shear modulus profiles of soils. Seismic methods are based on mechanical disturbances that generate elastic waves in soil. Once when the elastic waves are generated, using appropriate equipment, their velocities are measured. Seismic methods used in geotechnical engineering are useful for determining soil properties such as velocity of wave propagation, Young's modulus, shear modulus and Poisson's ratio. These soil properties are necessary in many situations such as analysis of foundations, evaluation of the response of the site to earthquakes, evaluation of the results of soil improvement like dynamic compaction and grouting (Woods, 1994).

Once the velocities profiles are known they can be related to the shear modulus and elastic modulus of soil using the following relationships.

$$V_s = \sqrt{\frac{G}{\rho}} \quad \text{and} \quad V_c = \sqrt{\frac{E}{\rho}} \tag{3-1}$$

Where

V_s is shear wave velocity,

V_c is compression wave velocity,

G is shear modulus,

E is elastic modulus, and

ρ is mass density

Variation of compression and shear wave velocities using crosshole method can be obtained as a function of depth.

The fundamental assumptions of the crosshole test are the following:

- The system tested is horizontally layered, and
- The Snell's law of refraction applies

Even though different types of equipment can be used for crosshole testing, the test itself is standardized and should be conducted according to ASTM Standard Designation: D 4428 / D 4428M - 91

According to the ASTM Standard for the crosshole test, a preferred test method includes three boreholes, and should be used whenever high quality data are needed to be obtained. An optional method includes two boreholes and should be used in project where a high precision is not required.

3.4.1 Fundamentals of the Crosshole Method

Three boreholes are required to conduct the crosshole test. Coupling between the boreholes and surrounding soil material is critical for good testing. Therefore, the spacing between PVC or metal casings and soil should be well grouted in-place using cement-bentonite non-shrinking grout. The grout should have a unit weight approximately the same as the surrounding soil (Woods, 1994). The basic elements of the crosshole test setup include:

- an energy source,
- receivers, and
- a recording system

The energy source should produce body waves of a required particle motion and energy level. Different types of in-hole hammers can be used as energy sources. Receivers shall be transducers that have appropriate frequency and sensitivity characteristics to determine the seismic wave train arrival. Typical receivers used in crosshole testing include geophones and accelerometers. Receivers should be placed in the boreholes so that a firm contact with the sidewall of the boreholes is insured. A recording system is an instrument that records the wave time histories for all receivers.

The test itself is done so that the energy source (hammer) and receivers are placed in the boreholes at the same elevation. Both the source and receivers should be placed so that a firm contact to the sidewall of the borehole is established. Seismic waves are generated by a hammer impact and detected by receivers. The test is repeated by lowering the source and receivers to a depth determined based on known stratification, but not more than 1.5 m (5.0 ft) from the previous test elevation. The described procedure should be repeated until the bottom of the boreholes is reached.

Of particular interest to this study is the evaluation of shear modulus profiles using shear wave velocities. If the wave trains for two receivers are displayed, the shear wave arrivals will be identified by the following characteristics:

- a sudden increase in the amplitude of at least two times that of the compression wave, and
- an abrupt change in frequency coinciding with the amplitude change.

To determine the velocity of propagation of seismic waves, the travel time is obtained from the difference in wave arrivals at receivers 1 and 2. Since the distance between receivers is known, the velocity of a seismic wave can be calculated. To establish the correct horizontal distance between boreholes deviation survey should be conducted. Using the deviation survey the verticality of each borehole is checked.

3.5 Description of the Crosshole Test at the Doremus Avenue Bridge

3.5.1 Borehole Installation

The crosshole test at Doremus Avenue Bridge was performed at five locations: Pier 1, Pier 2, Pier 4, Pier 5 and Pier 8 (Figure 3-1). Three boreholes were prepared for each crosshole test. Boreholes were aligned nominally in a straight line. The spacing between the first and the second borehole was 3.0 m (10 ft), while the distance between the second and the third borehole was 1.5 m (5 ft). All the boreholes were extended into the bedrock. The depths of the boreholes are summarized in the Table 3-2.

Samples from all distinctive layers were recovered during the borehole installation using Shelby tubes. Besides Shelby tubes the samples were taken using a split spoon sampler. The 2" split spoon samples were recovered during the SPT test.

The borehole casing was driven using a 300 lb hammer falling from the height of 24 in. The grouting was done with Portland cement and bentonite grout. The PVC casing was of a 100 mm (4 in) diameter. The bottom end of each casing was closed with a watertight cap. Pplane view of the borehole arrangement is shown in Figure 3-3 and schematics of the crosshole test in Figure 3-4..

Table 3-2. Borehole Depths and Their Locations

Location	Borehole No.		
	1	2	3
	m	m	m
Pier 1	21.13	23.13	23.10
Pier 2	25.42	25.17	25.63
Pier 4	25.50	25.81	25.35
Pier 5	24.36	24.38	24.41
Pier 8	22.92	23.16	22.98

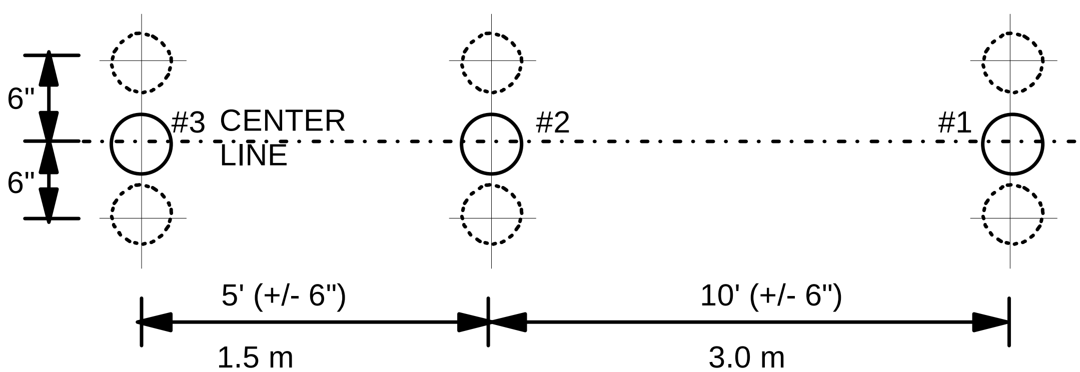


Figure 3-3. Plan View of Cased Boreholes for Crosshole Seismic Testing

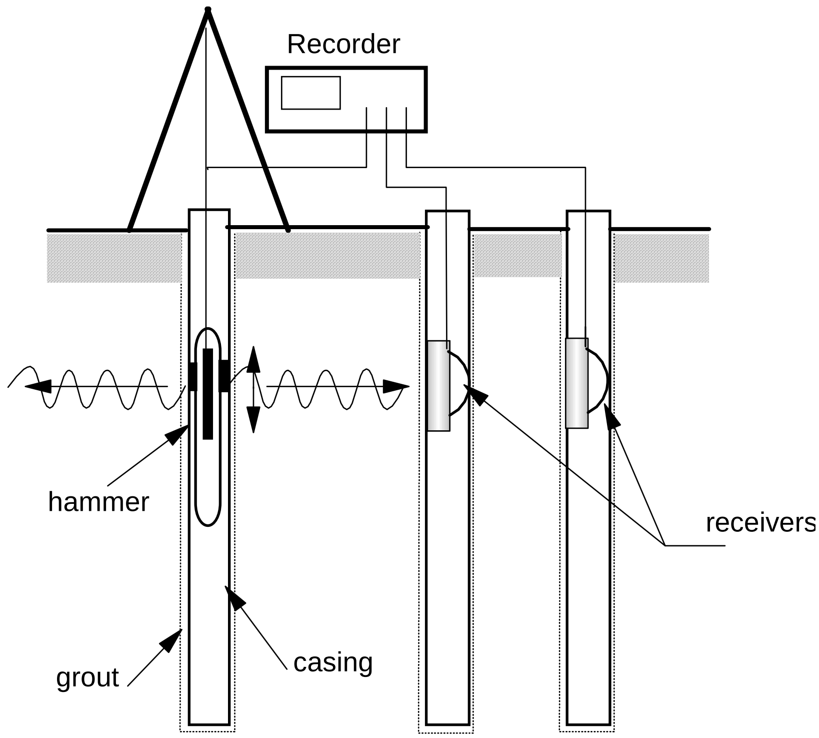


Figure 3-4. Schematics of the Crosshole Test

3.5.2 Equipment Used for Crosshole Testing

A shear type in-hole hammer was used as a source to generate seismic waves. The hammer has hydraulically expanding borehole gripers so that it can be fixed in place. A vertically sliding mass is used to produce dominantly shear waves. The two geophones were placed in the second and the third borehole. The geophones have rubber membranes that can be expanded by a compressed air to fix the geophones in place in the borehole. The distance between the hammer and the first geophone was about 3.0 m (10 ft) and the distance between the geophones was about 1.5 m (5 ft).

The signal from both the hammer and the geophones was recorded by the recording instrument. Records were taken every 3 ft (0.91 m) until the bottom of the borehole was reached. Equipment used for the crosshole test at the Doremus Avenue Bridge is shown in the Figures 3-5 through 3-7 and performance of the actual testing in Figure 3-8.

Figure 3-5. Recording System Used for Crosshole Testing

Figure 3-6. Hammer Used for the Generation of the Seismic Waves

Figure 3-7. Geophones Used as Receivers in Crosshole Testing

Figure 3-8. Crosshole Test at the Doremus avenue Bridge

3.5.3 Borehole Verticality Check

A verticality check was conducted using an inclinometer probe to get an accurate distance between the receivers. An inclinometer system consists of a casing, a probe with a cable and a read out unit (Figure 3-9). The inclinometer probe measures the tilt of the casing. The tilt is used to obtain a lateral distance. In the first step an incremental deviation is obtained for an increment of the casing from the tilt angle. In the second step the sum of incremental deviations is used to get the cumulative deviation. Readings that are displayed by inclinometer reading unit is proportional to the angle of tilt.

Figure 3-9. Inclinometer Probe

The readings were taken in 2 ft (0.61 m) increments for all boreholes and in two perpendicular directions to obtain the spatial position of boreholes. From the spatial positions of the receiver's boreholes the distance between them was obtained.

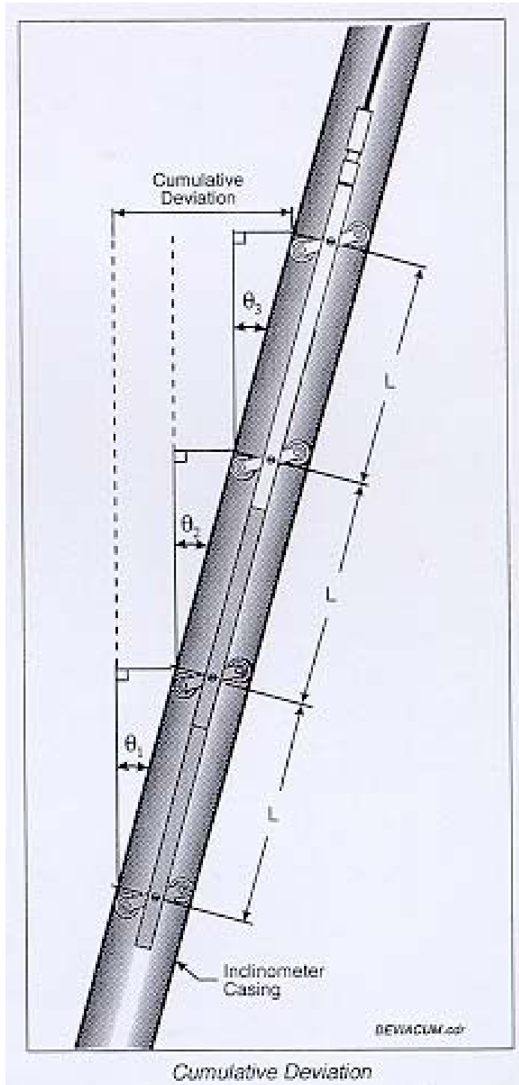


Figure 3-10. Inclinometer Probe (Digital DataMate & DMM Software)

3.5.4 Results from the Crosshole Test

The signal time histories were recorded in 3 ft (0.91 m) increments. Typical wave time-histories recorded are shown in Figure 3-11. The time difference between the shear wave arrivals was determined. Since the distance between boreholes was known, the shear wave velocity V_s is calculated as:

V_s = (distance between borehole) / (time difference between the wave arrivals)

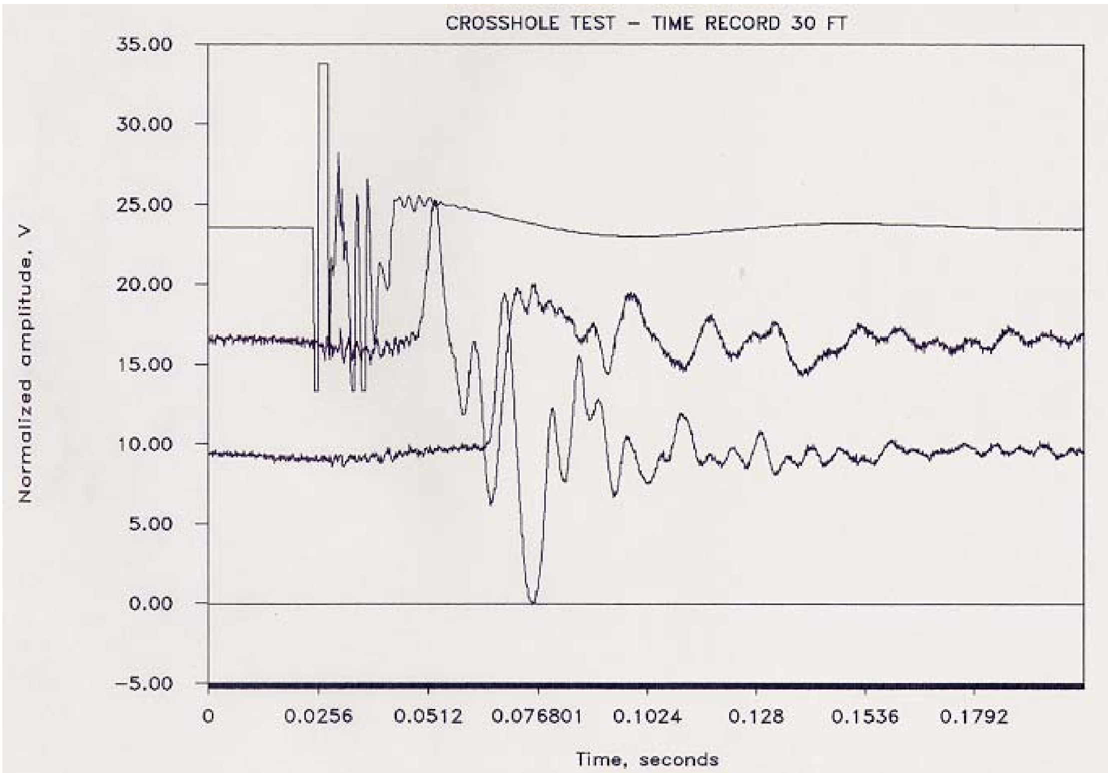


Figure 3-11. Signals recorded (hammer top, receiver 1 middle, receiver 2 bottom)

In the vicinity of the layer interfaces, a wave that first arrives at the receiver does not necessarily have a travel path, which is a straight line. That is because a wave that is traveling along the interface will travel with the velocity of the faster layer. To correct for a curved travel path Snell's law of refraction is used.

The result of data reduction was a shear wave velocity profile for each testing location. Once the shear wave velocity profile is known, the shear modulus profile can be obtained using the relationship between the shear wave velocity and shear modulus given by equation 3-1.

The shear wave velocity profiles for all test locations are given in Figures 3-12 to 3-16.

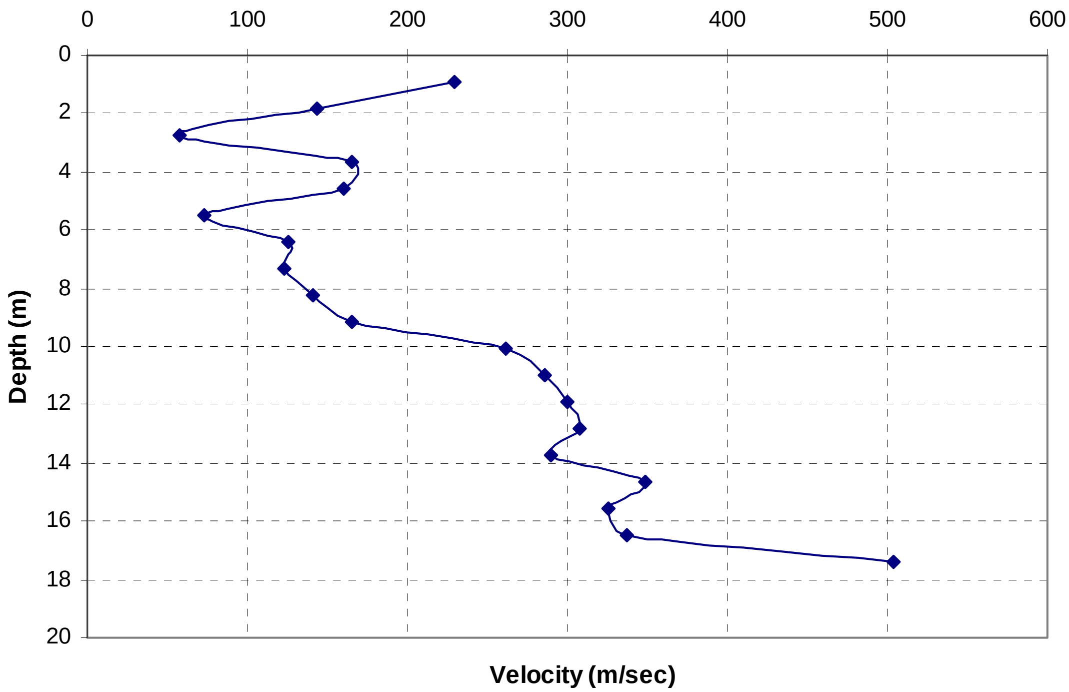


Figure 3-12. Shear Wave Profile for the Location at Pier 1

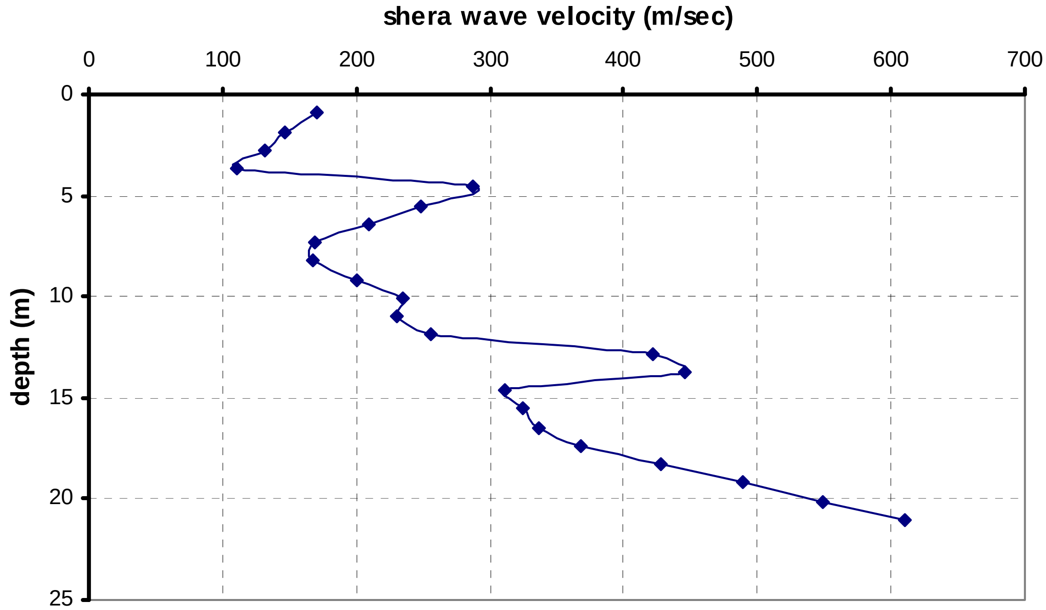


Figure 3-13. Shear Wave Profile for the Location at Pier 2

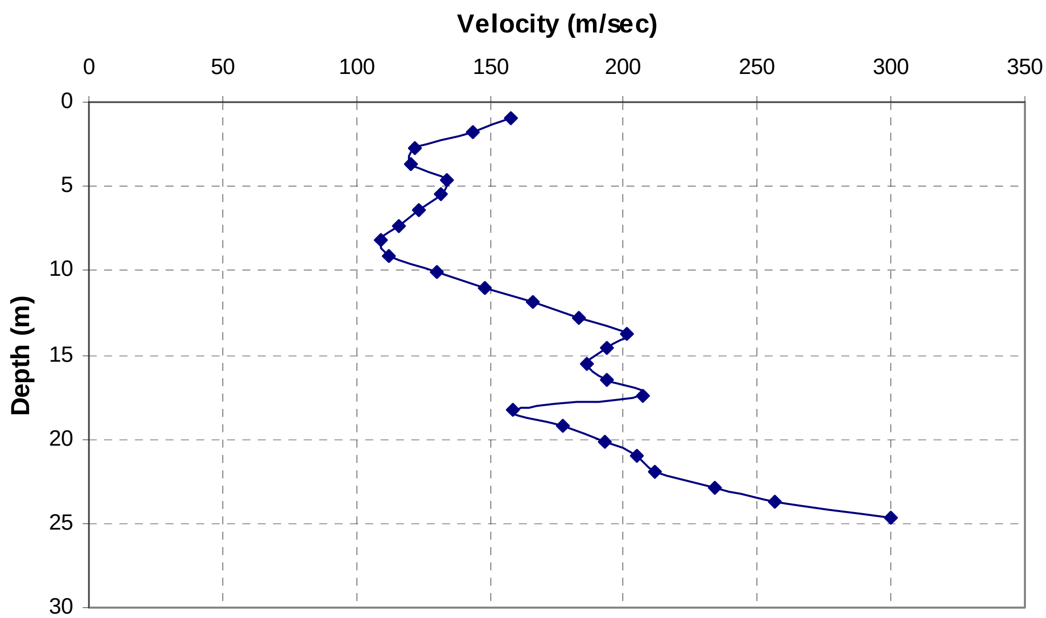


Figure 3-14. Shear Wave Profile for the Location at Pier 4

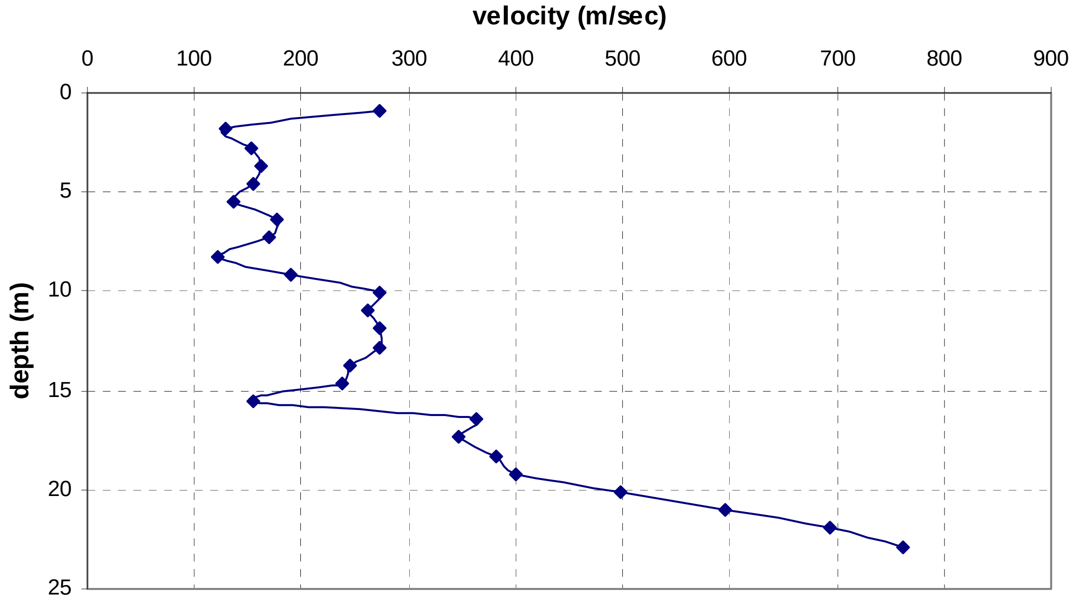


Figure 3-15. Shear Wave Profile for the Location at Pier 5

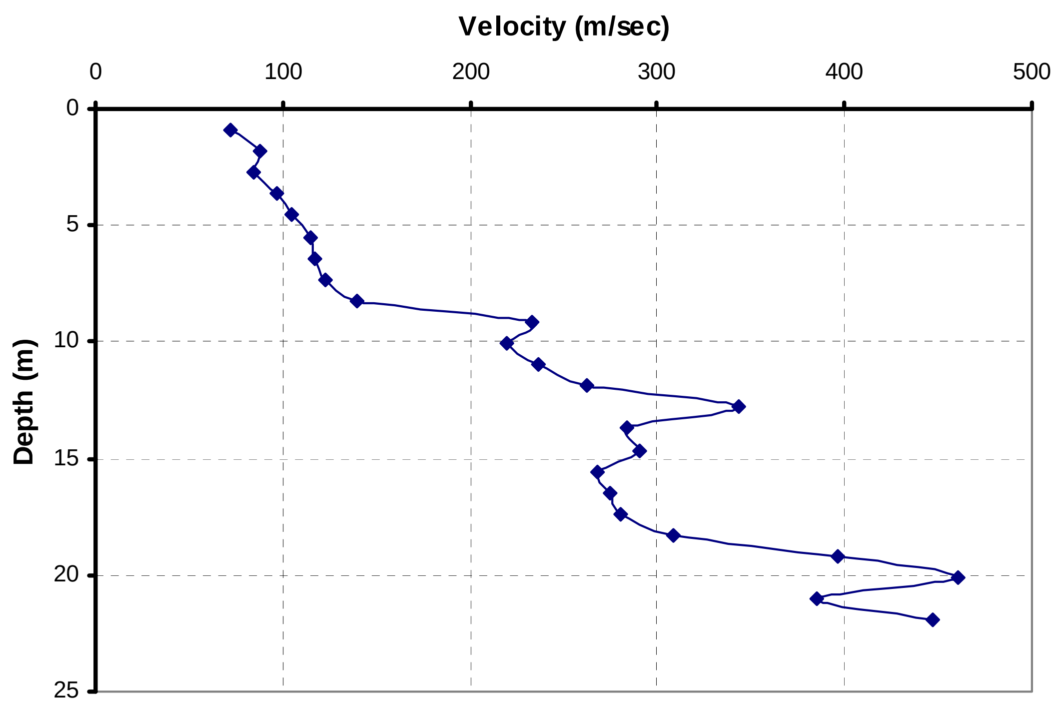


Figure 3-16. Shear Wave Profile for the Location at Pier 8

Obtained shear wave velocities for the tested locations were used to get the shear wave profile of the site in longitudinal direction. From the shear wave profile the shear modulus profile of the site between Piers 1 and 8 is calculated using linear interpolation between obtained profiles at each tested location. Shear Modulus is in MPa. Shear modulus profile is shown in Figure 3-17.

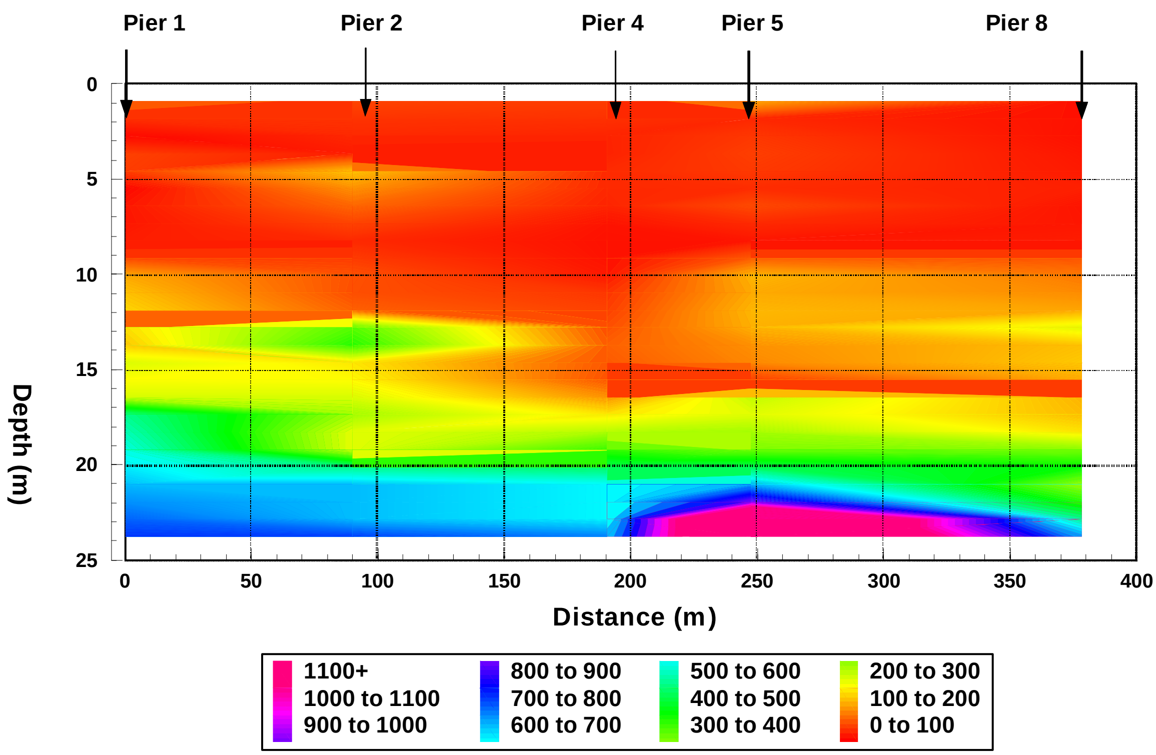


Figure 3-17. Shear Modulus(in MPa) Profile Between Piers 1 and 8 of the Doremus Avenue Bridge

3.5.5 Fundamental Frequencies of the Site

Finally using program SHAKEHIS and data for soil properties at locations matching those for crosshole testing, fundamental frequencies of the site were determined. The fundamental frequencies are given in Table 3-3.

Table 3-3. Fundamental Frequencies of the Site

Location	Fundamental frequency (Hz)
Pier 1	2.05
Pier 2	1.71
Pier 4	1.50
Pier 5	2.14
Pier 8	1.40

3.6 Summary

In this chapter characteristics of the construction site are presented. Described is soil profile obtaining by boring logs and dynamic soil properties obtained using crosshole testing. It can be seen that the sol profile is fairly layered, and crosshole test was suitable for this kind of soil profile. Fundamental frequencies of the site are in the range of 1.4 to 2 Hz. Dynamic soil properties obtained will be used for dynamic analysis of the drilled shat foundations and soil-structure interaction.

CHAPTER 4

Drilled Shaft Testing

4.1 Introduction

For the purpose of dynamic characterization of the drilled shafts foundation at the Doremus Avenue Bridge were instrumented and tested. The goal was to obtain dynamic stiffness (impedance functions) of the foundations. Dynamic characteristics of the foundations are needed for the any kind of soil-structure interaction analysis, as well as analysis of the foundations subjected to any kind of dynamic loading.

Objectives of the dynamic characterization of the drilled shaft foundations were the following:

- evaluation of the shaft dynamic stiffness (impedance functions)
- evaluation of the shaft interaction
- evaluation of the soil-foundation-structure interaction

The instrumentation and the testing of the shafts is presented, equipment used is described and obtained results are discussed.

4.2 Drilled Shaft Foundation

Drilled shaft foundations are deep foundations that are usually extended into bridge piers and diameter ranges from 0.6 to 3.6 m (2 to 12 ft) O'Neal and Reese, 1999. Because of their large diameter and high load carrying capacity they are often advantageous over conventional small diameter pile. Their advantage is also that pile cap is not needed which makes them more economical choice as well. Increased use of these types of foundation can be seen in bridge structures.

Analysis and design of drilled shafts for most parts are similar to those of driven piles. Important difference exist due to installation procedure, large diameter and smaller length to diameter ratio (PoLam et al, 1998).

Doremus Avenue Bridge foundations are concrete drilled shafts with a steel casing left in place. The shafts are socketed 3.0 m into the bedrock and extended into the bridge columns above ground as shown in Figure 4-1

The shaft was constructed so that first the shaft was drilled until the bedrock was reached. The slurry was used to maintain the hole. After that the steel casing 1.22 m in diameter, with the wall thickness of 12 mm, was placed. The shaft is extended into bedrock. Since the steel casing was left in place reinforcement was not required. But rebar cage, consisting of four steel pipes of 100 mm in diameter, was placed for the future check of the concrete integrity in the shaft. Transverse reinforcements (bars #13) were used to hold pipes in place.

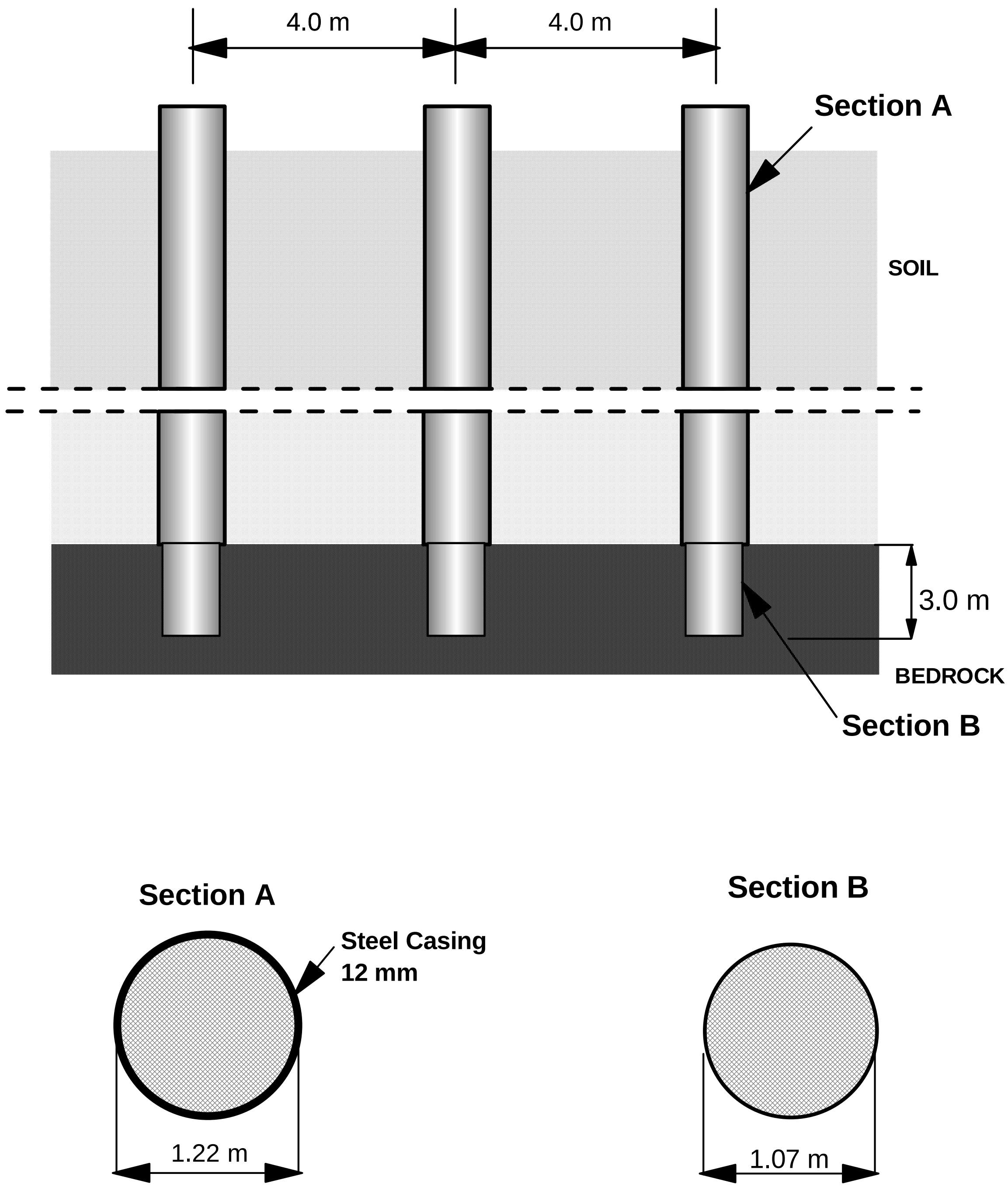


Figure 4-1. Drilled Shaft Foundations at the Doremus Avenue Bridge

There are three drilled shafts in a group at each pier location. The shaft diameter is 1.22 m in the soil and 1.07 m in the bedrock. The properties of the drilled shaft foundation at the piers were the testing was done are given in Table 4-1.

Table 4.1. Drilled shafts properties

Location	Diameter in soil	Diameter in bedrock	Length in soil	Length in bedrock
	m	m	m	m
Pier 2	1.22	1.07	24.78	3.36
Pier 4	1.22	1.07	25.22	3.20
Pier 5	1.22	1.07	23.10	3.10
Pier 8	1.22	1.07	23.26	3.31

4.3 Substructure Instrumentation

Substructure instrumentation involved instrumentation of a drilled shaft, a pier and a pier cap. There were two main objectives of the substructure instrumentation. The first objective was to obtain better insight into the drilled shaft dynamics. The second goal was, in conjunction with the superstructure instrumentation, to achieve a better understanding of the dynamic soil-foundation-structure interaction. Pier 2 of the new bridge was selected for instrumentation. The instrumentation plan is shown in Figure 4-2.

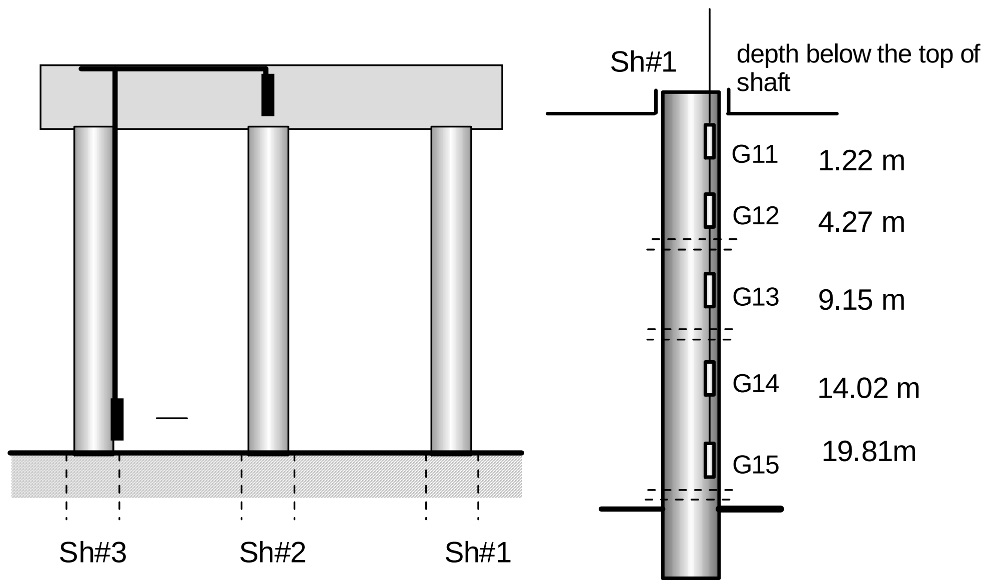


Figure 4-2. Schematics of the Instrumented Drilled Shaft, Pier and Pier Cap

Five triaxial geophones (Mark Products L-22D) were placed in the drilled shaft at Pier 2. The geophones were placed at depths matching the characteristic soil layers. Each of the geophones was placed in a protective casing and fixed to the rebar cage. Installation and placement of the geophones in the shaft are shown in Figure 4-3. All of the geophones will be used for future monitoring of the bridge. Triaxial geophones have been used for this purpose because of the sensibility of the instruments and ability to measure response in three directions using one instrument. Geophones are built so that the instrument is placed in pretty big casing which allowed easier and safer placement in the shaft.

(a)

(b)

Figure 4-3. (a) Installation and (b) Placement of the Geophones in the Shaft

4.4 Drilled Shaft Vibration Testing

The locations of the tested drilled shafts (Pier 2,4,5 and 8)are shown in the Figure 3-1 in chapter 3.. Dynamic properties of the drilled shaft foundation supporting the Doremus Avenue Bridge were determined by forced vibration testing. The objective was to excite the shafts harmonically over the certain frequency range and than record their response.

The testing provided vide range of data so that the following could be observed:

- natural frequency of the shaft,
- the insight in shafts dynamics from the fully instrumented shaft,
- the influence of one shaft on the others in the group.

4.5 Description of the Shaft Testing

The shafts were excited harmonically using an APS Model 400 electromagnetic shaker of the maximum horizontal force of 445 N. The vibration force was introduced as a frequency sweep between 1 and 100 Hz, with a frequency step of 1 Hz. For each frequency step, 5 loading cycles were applied. The shaker was suspended on a frame and attached to the drilled shaft through a steel section anchored into the shaft. A schematic of the test setup is shown in Figure 4-4.

A dynamic signal analyzer was used to generate the harmonic excitation and feed the shaker. The shaker force was controlled by an amplifier, and measured using a load cell placed between the arm of the shaker and the steel section. The Load cell is Dytran Instruments model series 1051V LIVM force sensor.

The response of the loaded and adjacent shafts was measured using triaxial Mark Products L-4C-3D geophones, placed on the top of the shafts. All the time histories (loading and responses) were recorded using a data acquisition system. The data acquisition system used for recording the data consisted of the following: signal conditioning box, connection panel, data acquisition card and computer. As software LabVIEW was used.

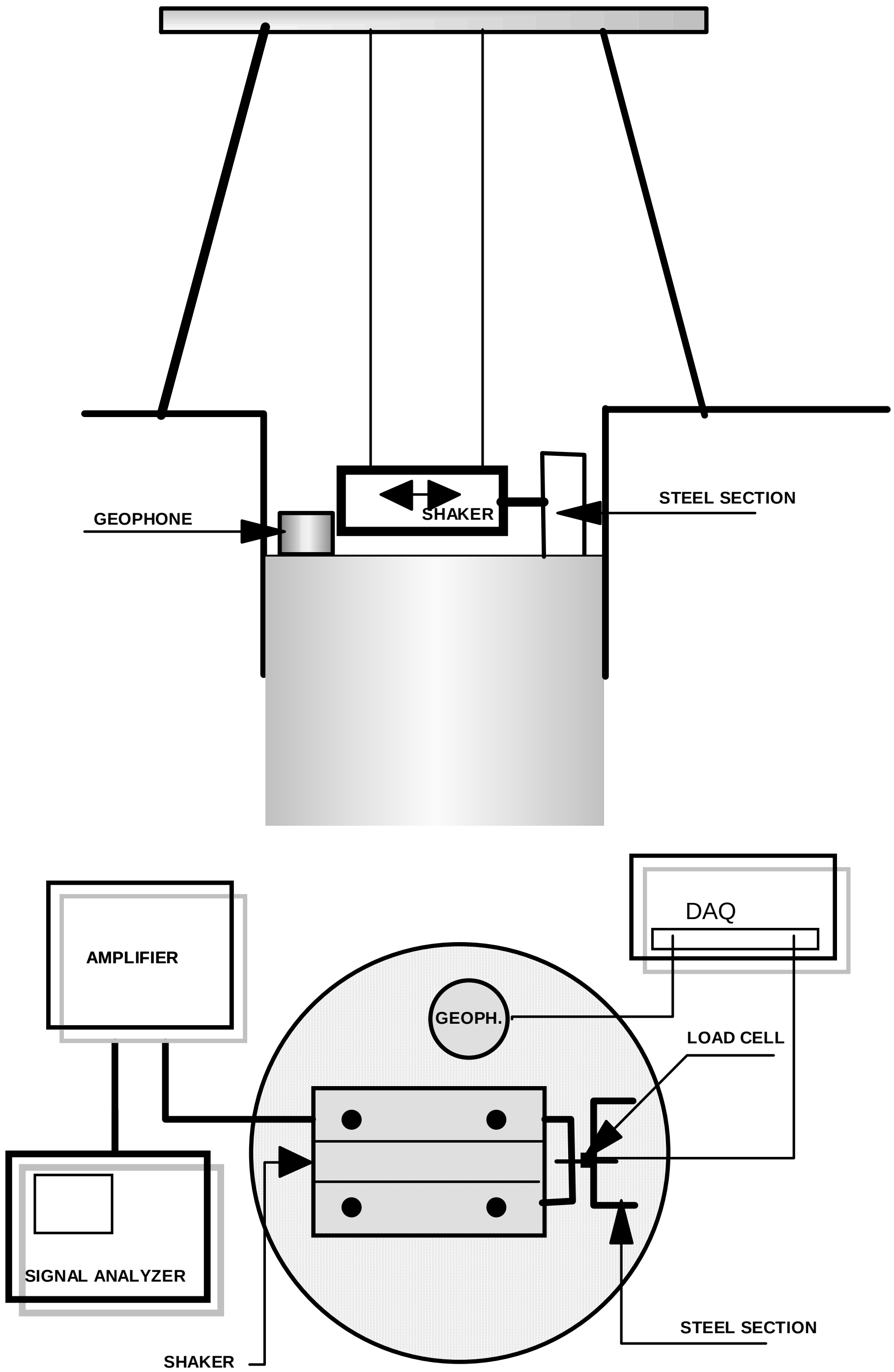


Figure 4-4. Schematics of the Shaft Testing

Actual testing at the Doremus Avenue Bridge is show in Figure4-5 trough Figure 4-7.

Figure 4-5. Shaft Arrangement and Equipment Used

Figure 4-6. Arrangement of Shaker and Geophone on Top of Tested Shaft

Figure 4-7. Triaxial Mark Products L-4C-3D Geophone on Top of the Adjacent Shaft

4.6 Results of the Drilled Shaft Testing

The main objective of the drilled shaft testing was to obtain impedance functions. During the test- introduced force, response of the tested shaft and the responses of adjacent shafts were recorded as time histories. The total length of the time history records is about 120 sec. Only 30 sec of the records is shown in the figures. After looking at the data and data reduction it was observed that the significant frequency range was till about 30 Hz. After that very little movement of the shaft is observed, because the shaft was not able to follow such a vibration due to its mass and stiffness.

The results of the testing are shown for each tested location.

4.6.1 Pier 2

Location of the Pier 2 can be seen on the bridge layout in Figure 3-1. At the time of testing all the shafts at Pier 2 were leveled with ground. Properties of the drilled shafts are as follow:

- diameter in soil 1.22 m
- diameter in bedrock 1.07 m
- length in soil 24.78 m
- length in bedrock 3.36 m

Typical time history and linear spectrum for the forcing function is shown in Figure 4-8. The loading was introduced as a frequency sweep with a frequency step of 1 Hz.

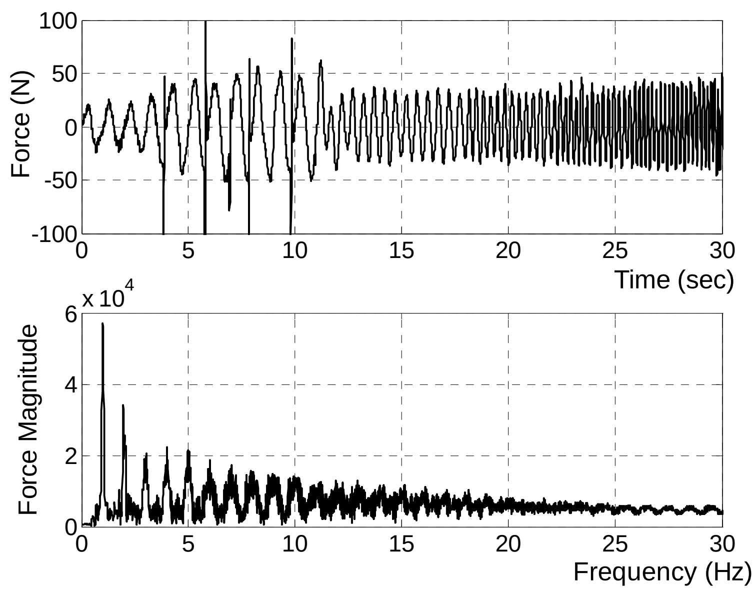


Figure 4-8. Loading Time History and Spectrum

Response is recorded in terms of velocity and the response time history and the linear spectrum are shown in Figure 4-9.

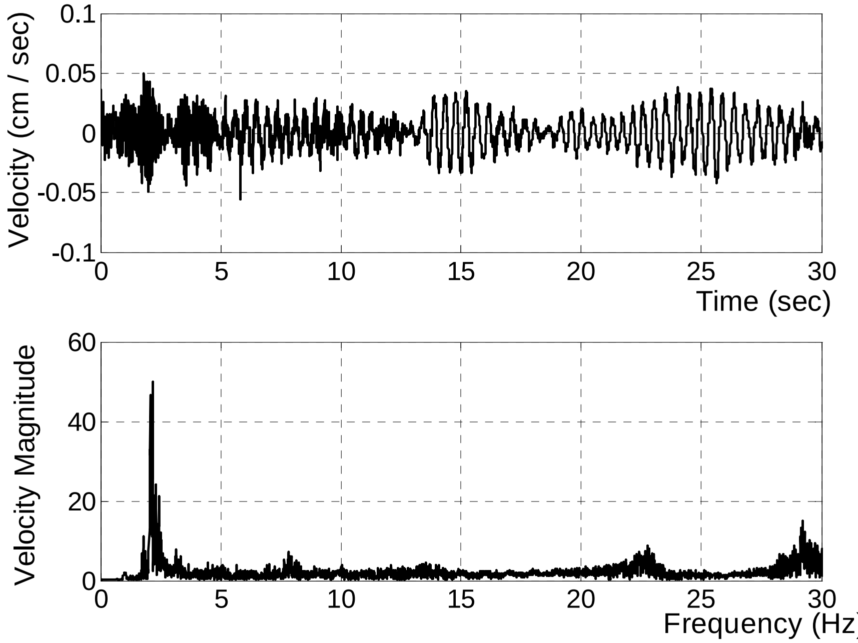


Figure 4-9. Response Time History and Spectrum

The displacement spectrum can be obtained from the velocity spectrum, by dividing it by $i\omega$, where $i = \sqrt{-1}$, and $\omega = 2\pi f$ is the angular frequency. The displacement time history is obtained by applying the inverse Fourier transformation. The obtained displacement spectrum is shown in Figure 4-10.

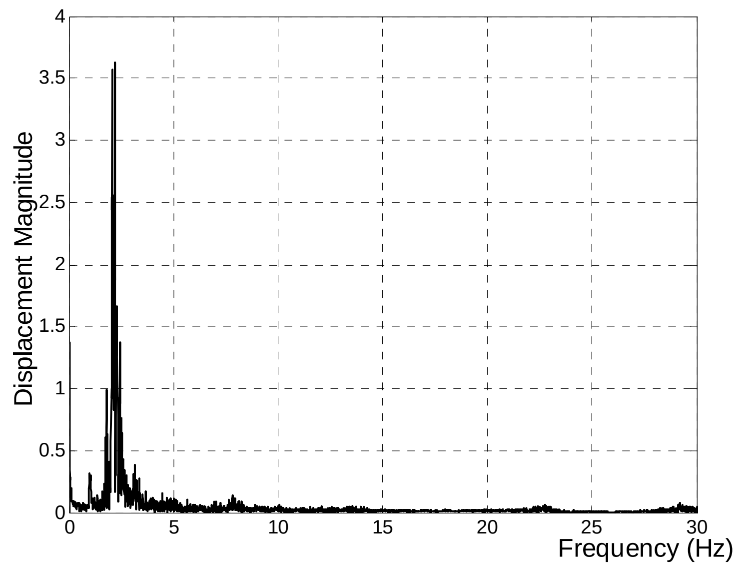


Figure 4-10. Displacement Spectrum

The impedance function is defined as a complex ratio of the forcing function and displacement spectrum. The flexibility spectrum, or the compliance function, is the inverse function of the impedance function. The compliance functions for the shaft at the Pier 2 are shown in Figure 4-11. The fundamental frequency for the shaft was found to be 2.10 Hz.

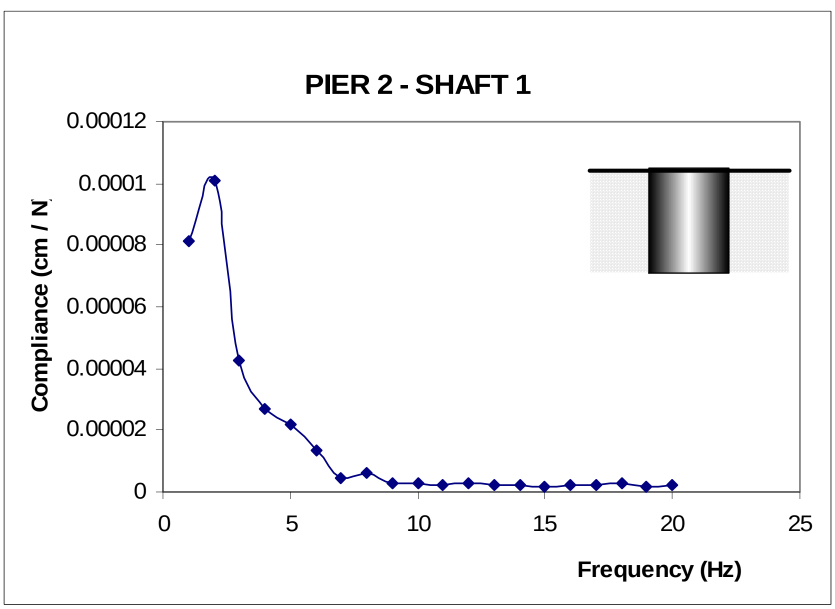


Figure 4-11. Magnitude of Compliance Function of the Shaft at Pier 2

For the instrumented shaft at pier 2, the response was measured at the top of the shaft and at five geophone elevations (geophones G11 to G15, Figure 4-2). The response spectra for the geophone at the pile head, and the five geophones placed inside the shaft are shown in Figure 4-12. A very rapid decrease in the response with depth can be observed.

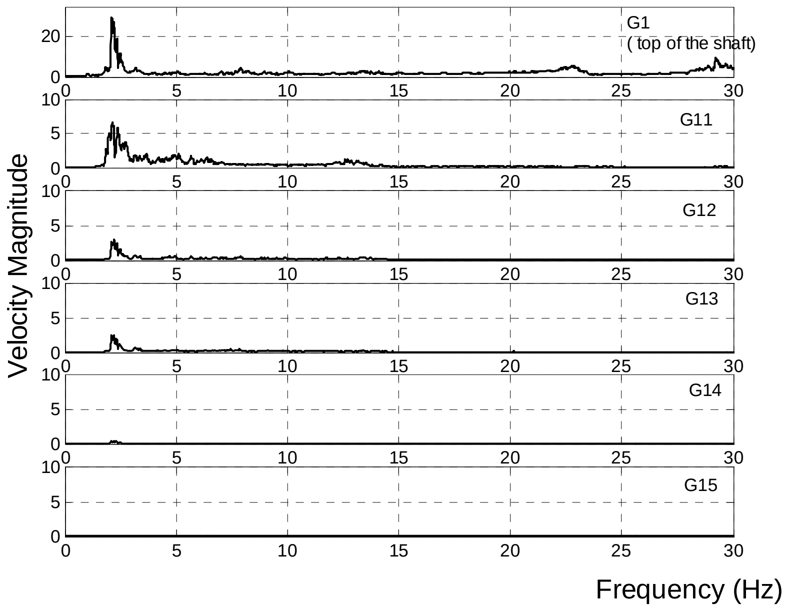


Figure 4-12. Response Spectra of the Top of the Shaft and the Built-in Geophones

Since the response of the shaft is frequency dependent, displacement curves with depth are compared for four frequencies (2, 5, 8 and 10 Hz). It can be observed that the maximum displacement below the depth of two shaft diameters is less than 10% of the top response for all frequencies. Displacements as a function of depth for four analyzed frequencies are shown in Figure 4-13.

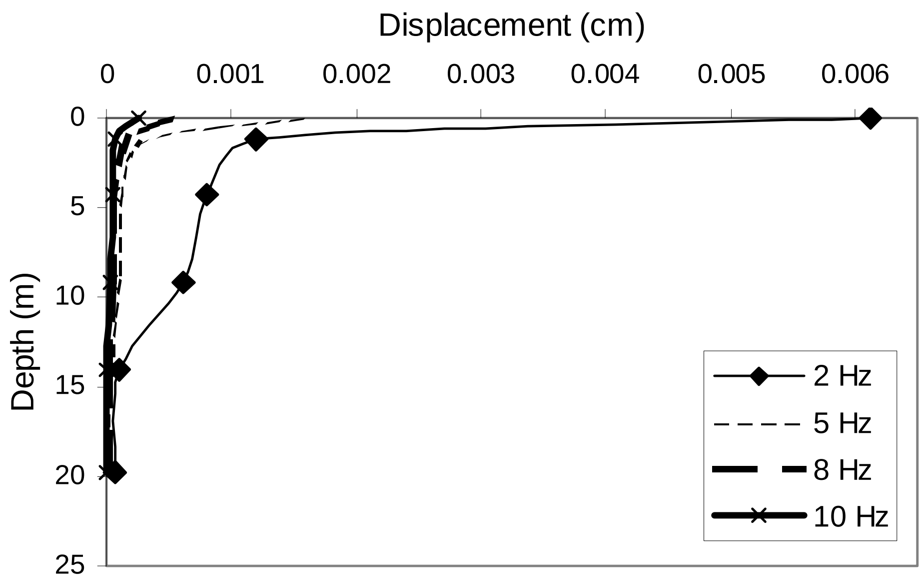


Figure 4-13. Displacements with Depth of the Shaft for Frequencies of 2, 5, 8 and 10 Hz

The maximum displacement is reduced rapidly at increasing depths rate, so that below a depth of two shaft diameters the response is less than 10% of the top response for all frequencies. While the transmissibility in general decreases with frequency, it is highly frequency dependent. To gain a better insight into the dynamics of a shaft, a phase lag between the response of the top of the shaft and the five embedded geophones is plotted in Figure 4-14. Almost linear increases in the phase lag with depth and frequency points to a wave propagation nature of the surface disturbance.

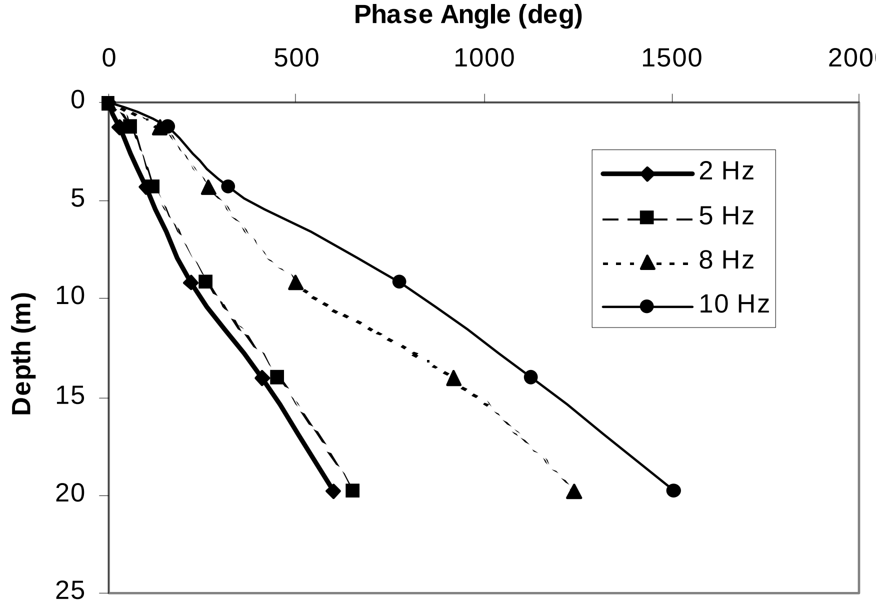


Figure 4-14. Phase Lag with Depth of the Shaft for the Frequencies of 2, 5, 8 and 10 Hz

For the purpose of a future shaft interaction study, the response of adjacent piles was recorded. Kaynia and Kaussel (1982) suggested that the interaction between the piles should not be neglected if the piles are closely spaced. They considered a close spacing to be less than 6 to 8 pile diameters. At the Doremus Bridge the shaft spacing is 3.33 shaft diameters. Velocity spectra for the tested shaft and two adjacent shafts at Pier 2 are shown in Figure 4-15.

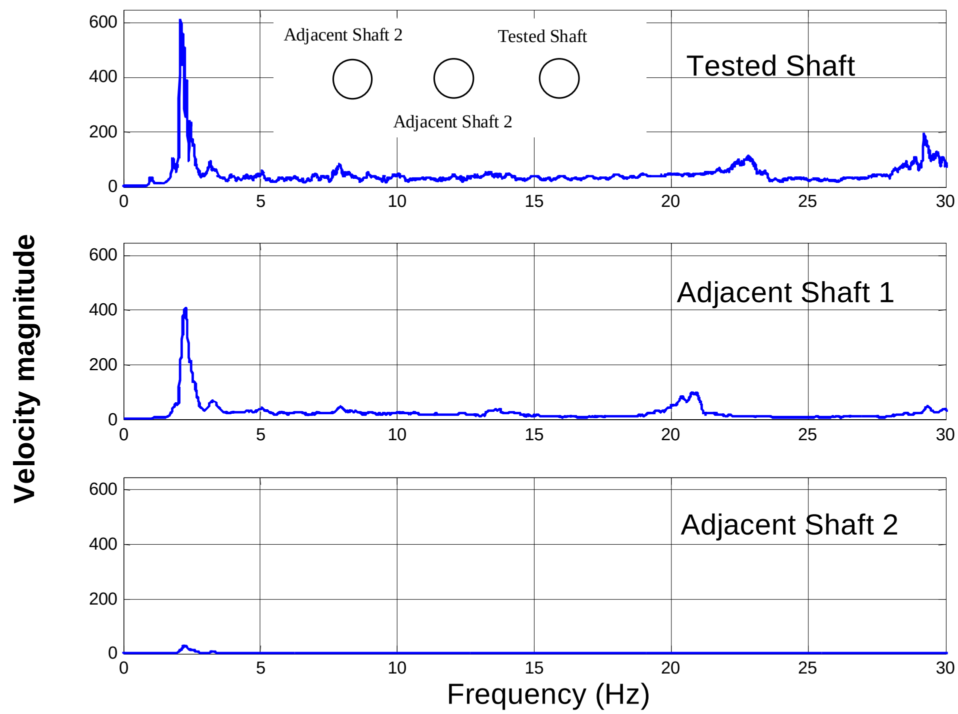


Figure 4-15. Velocity Spectra for the Tested Shaft and Two Adjacent Shafts

From the obtained spectrum it can be seen that the influence of the tested shaft on the first adjacent shaft was quite significant, but the furthest shaft barely showed response due to the excitation of the tested shaft. The same effect can be seen in the Figure 4-16, which shows the displacements with frequencies for the tested shaft and two adjacent shafts.

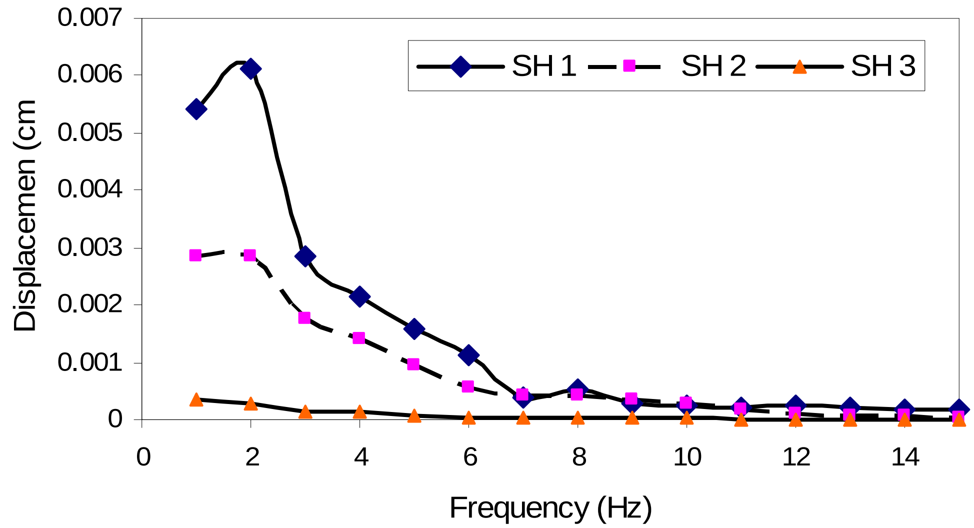


Figure 4-16. Displacements vs. Frequency for the Tested Shaft and Two Adjacent Shafts

4.6.2 Pier 4

Location of the Pier 4 can be seen on the bridge layout in Figure 3-1. At the time of testing the top of the shaft at Pier 4 was 0.6 m below the ground level. Properties of the drilled shafts are as follow:

- diameter in soil 1.22 m
- diameter in bedrock 1.07 m
- length in soil 25.22m
- length in bedrock 3.20 m

Typical time history and linear spectrum for the forcing function can be seen in Figure 4-8, since the same type of the vibration was used for all the tests. Only difference might be in the amplitude because the amplitude of the force was controlled manually trough the amplifier attached to the shaker.

Response is recorded in terms of velocity and the response time history and the linear spectrum are shown in Figure 4-17.

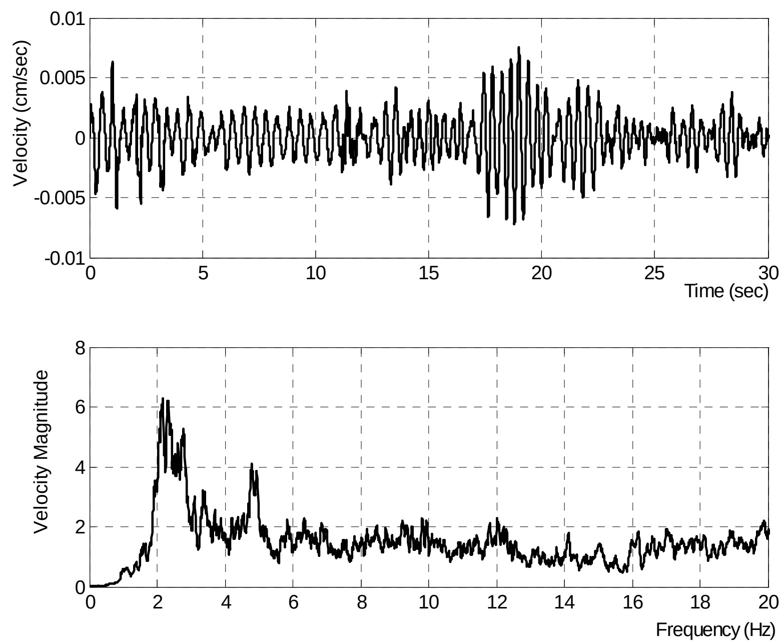


Figure 4-17. Response Time History and the Spectrum (Pier 4)

The displacement spectrum is obtained from the velocity spectrum and is shown in Figure 4-18.

The compliance functions for the shaft at the Pier 2 are shown in Figure 4-19. The fundamental frequency for the shaft was found to be 2.27 Hz.

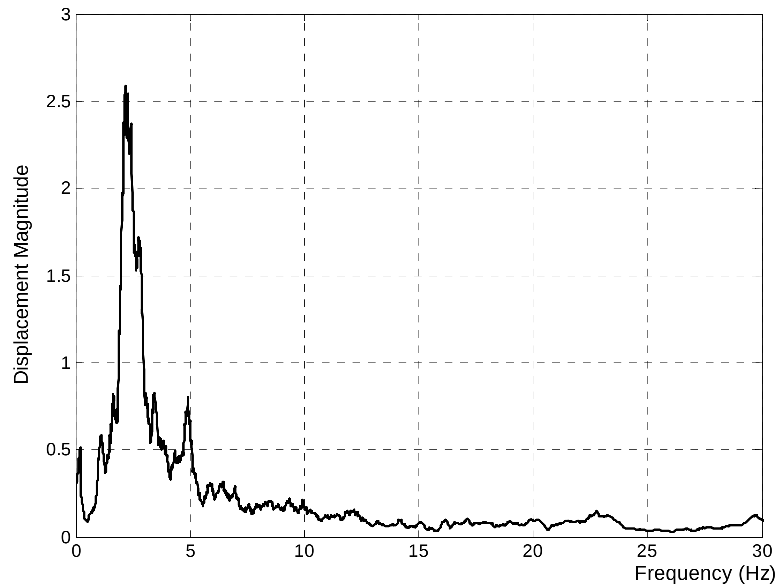


Figure 4-18. Displacement Spectrum (Pier 4)

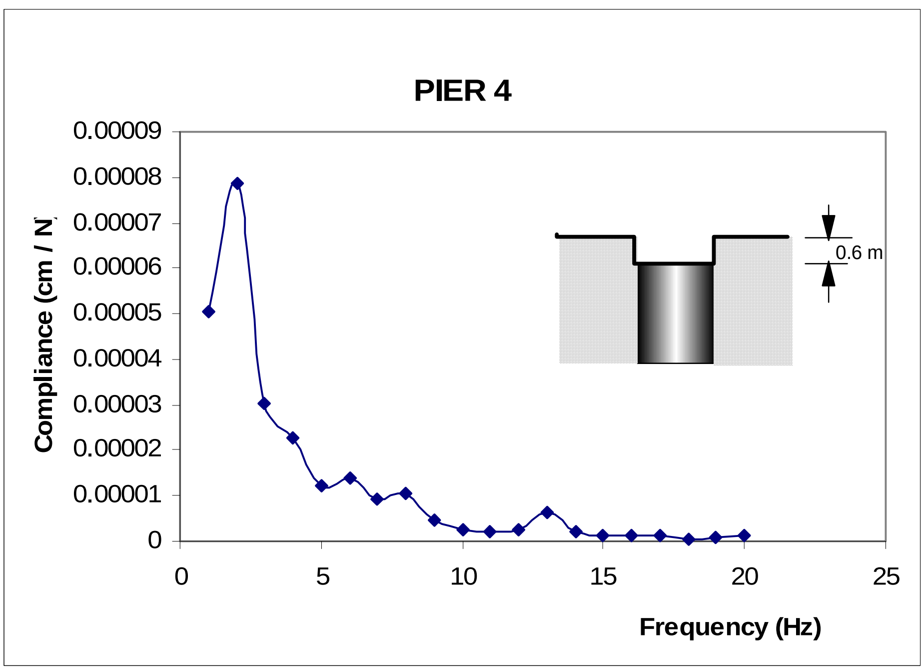


Figure 4-19. Magnitude of Compliance Function of the Shaft at Pier 4

4.6.3 Pier 5

Location of the Pier 4 can be seen on the bridge layout in Figure 3-1. At the time of testing the top of the shaft at Pier 4 was 1.0 m below the ground level. Properties of the drilled shafts are as follow:

- diameter in soil 1.22 m
- diameter in bedrock 1.07 m
- length in soil 23.10 m
- length in bedrock 3.10 m

Typical time history and linear spectrum for the forcing function can be seen in Figure 4-8, since the same type of the vibration was used for all the tests. Only difference might be in the amplitude because the amplitude of the force was controlled manually trough the amplifier attached to the shaker. Response is recorded in terms of velocity and the response time history and the linear spectrum are shown in Figure 4-20.

The displacement spectrum is obtained from the velocity spectrum and is shown in Figure 4-21.

The compliance functions for the shaft at the Pier 2 are shown in Figure 4-22. The fundamental frequency for the shaft was found to be 2.01 Hz.

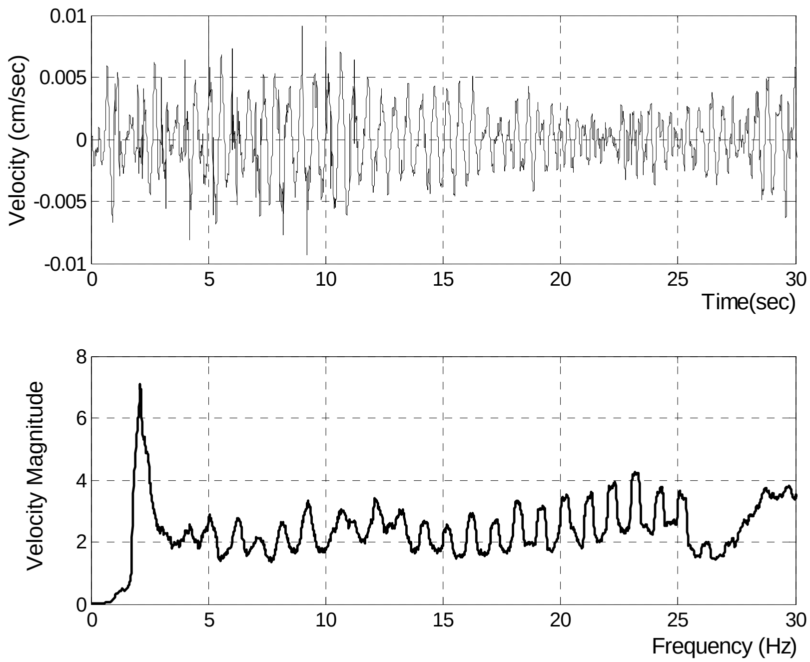


Figure 4-20. Response Time History and the Spectrum (Pier 5)

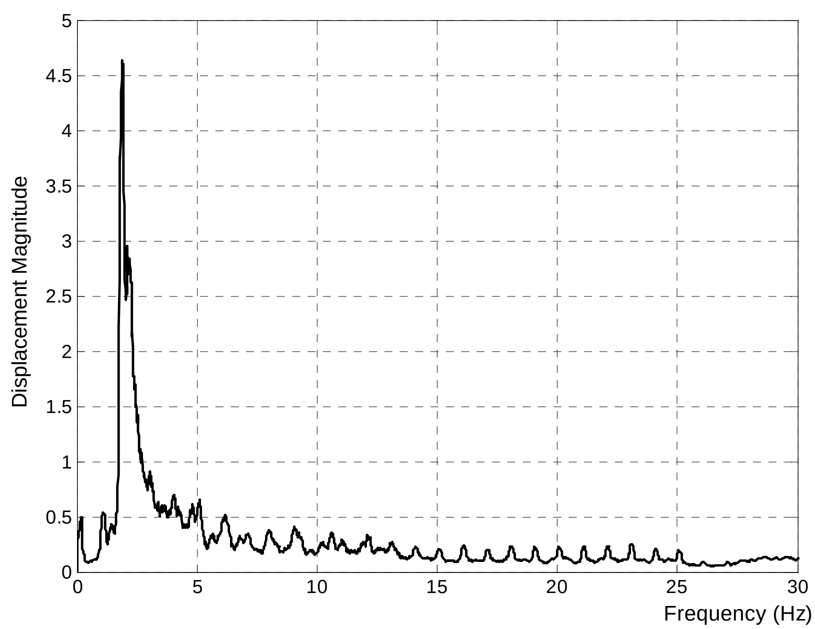


Figure 4-21. Displacement Spectrum (Pier 5)

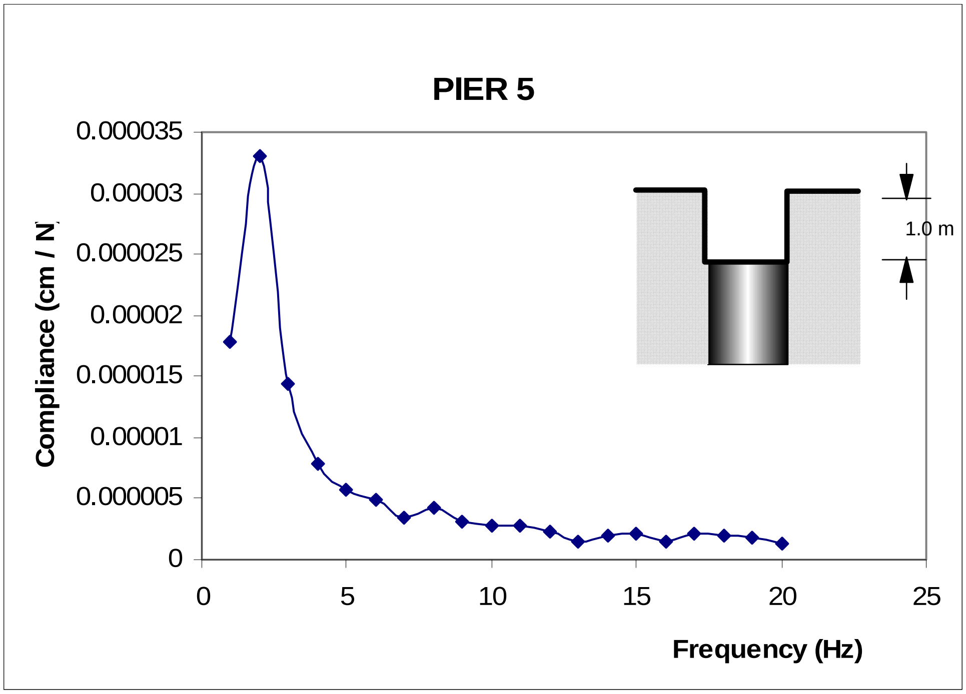


Figure 4-22. Magnitude of Compliance Function of the Shaft at Pier 5

4.6.4 Pier 8

Location of the Pier 4 can be seen on the bridge layout in Figure 3-1. At the time of testing the top of the shaft at Pier 4 was 1.0 m below the ground level. Properties of the drilled shafts are as follow:

- diameter in soil 1.22 m
- diameter in bedrock 1.07 m
- length in soil 23.26 m
- length in bedrock 3.31 m

Typical time history and linear spectrum for the forcing function can be seen in Figure 4-8, since the same type of the vibration was used for all the tests. Only difference might be in the amplitude because the amplitude of the force was controlled manually trough the amplifier attached to the shaker.

Response is recorded in terms of velocity and the response time history and the linear spectrum are shown in Figure 4-23.

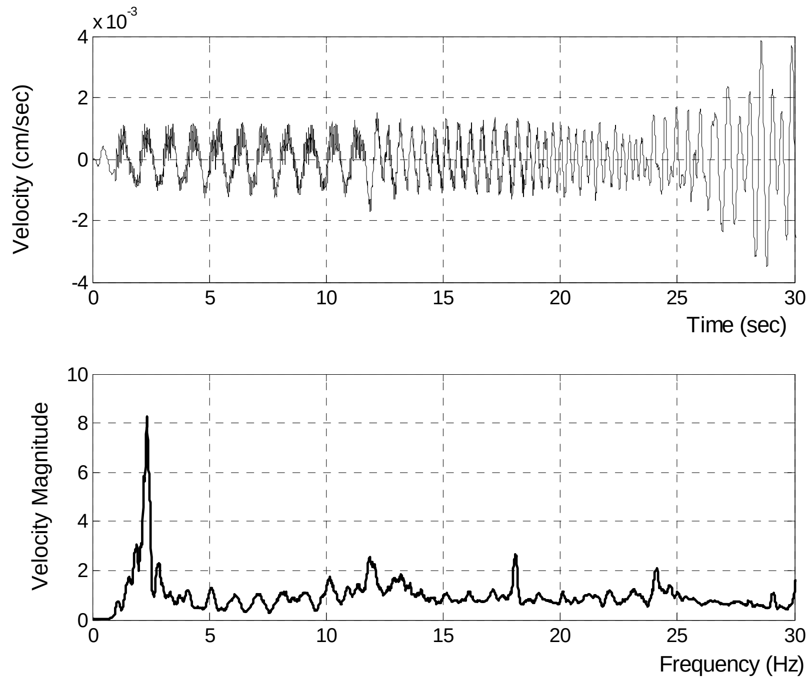


Figure 4-23. Response Time History and the Spectrum (Pier 5)

The displacement spectrum is obtained from the velocity spectrum and is shown in Figure 4-24.

The compliance functions for the shaft at the Pier 2 are shown in Figure 4-25. The fundamental frequency for the shaft was found to be 2.20 Hz.

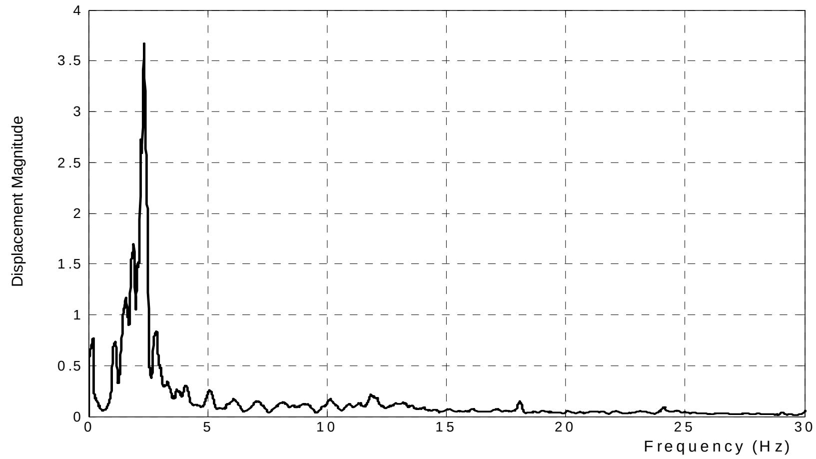


Figure 4-24. Displacement Spectrum (Pier 5)

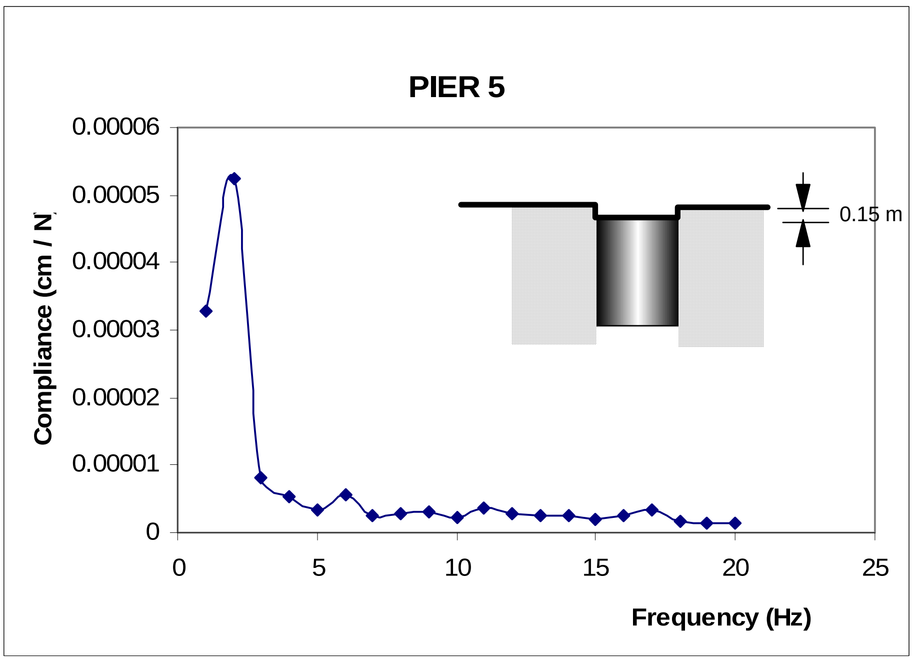


Figure 4-25. Magnitude of Compliance Function of the Shaft at Pier 5

4.7 Summary

Dynamic properties of the drilled shaft foundations supporting the Doremus Avenue Bridge were determined by forced vibration testing. The conditions during the testing cannot be described as an ideal steady state condition, due to the limited number of loading cycles applied for each frequency. This is best illustrated through a comparison of time histories and loading for frequencies close to the shaft's resonant frequency, estimated to be about 2 Hz. In this low frequency range, roughly 2 to 10 Hz, the transient response at the resonant frequency dominates the steady state response at the driving frequency. Outside this range, the response frequency follows very well the frequency of the driving force.

The impedance functions were obtained for the shafts at piers 2, 4, 5 and 8. The impedance function is defined as a complex ratio of the forcing function and displacement spectra. The flexibility spectrum, or the compliance function, is the inverse function of the impedance function. The compliance functions for the shafts at four pier locations are shown in Figure 4-28. The differences in the compliance functions are attributed to differences in soil profiles and the depth of embedment of the top of the shaft. The depths of embedment of the tops of the shafts at Piers 2, 4, 5, and 8 were 0, 0.6 m, 1 m, and 0.15 m, respectively. The fundamental frequency of the shafts stayed in narrow range from 2.0 to 2.27 Hz.

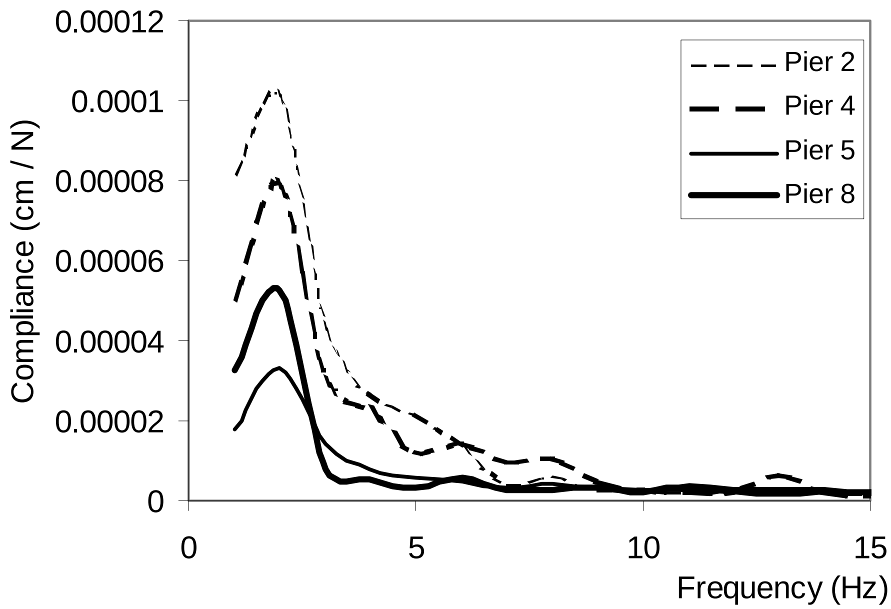


Figure 4-26. Magnitude of Compliance Function of the Shafts at Different Locations

The data and results are in agreement with some of the earlier experimental studies of pile foundations in terms of foundation behavior (Novak 1980, Crouse and Cheang 1987). It is hard to compare obtained natural frequencies because of the big variation in obtained values. Thos differences are attributed to the different foundation dimensions, properties and arrangements, as well as different soil properties. Most of the research has been done for pile and pile groups foundations. Since drilled shafts are getting more and more used for dynamically loaded structures, there is need to relate numerical models for pile foundation to the drilled shaft foundation.

Results from conducted testing, as well as from the data from the previous research, show that the responses of the dynamically loaded deep foundations are site and frequency dependent. The upper part of the foundation is the most affected one approximately by the depth of 2 to 4 shaft diameter. This part of the foundation is also the most affected by separation of the shaft from soil.

Test results presented are useful for low strain dynamic response. It is reasonable to expect large strain to occur in case of strong ground motion or large cyclic loading that bridge may be subjected to. For the case of large strain dynamic response additional analysis should be done.

CHAPTER 5

Summary and Conclusions

5.1 Scope and Findings of the Research

This research is a part of the research project conducted on Doremus Avenue Bridge in Newark, NJ. Since this is the first bridge in New Jersey designed according to the LRFD Specifications, it has been chosen to be tested and monitored during and after the constructions. The main objectives of substructure evaluations were (1) site characterization with respect to the dynamic soil properties, and (2) evaluation of the dynamic stiffness (impedance functions) of drilled shafts.

The objective of the site characterization was to obtain dynamic soil properties of the site. Dynamic soil properties are needed to conduct a site response analysis and any kind of dynamic soil-structure interaction analysis. Dynamic soil properties such as shear wave velocity and shear modulus were determined using crosshole test.

Dynamic properties of the drilled shaft foundations supporting the Doremus Avenue Bridge were determined by forced vibration testing. Response of the tested shaft as well as the responses of adjacent shafts were recorded. One fully instrumented shaft was used to get better insight into shaft behavior under dynamic load. Measured displacement of the instrumented shaft show that the most affected is top part of the shaft. The maximum displacement is reduced rapidly at increasing depths rate, so that below a depth of two shaft diameters the response is less than 10% of the top response for all frequencies. Differences in compliance functions for drilled shafts at different locations are attributed to different soil profiles and different depth of embedment of the top of the shat at the time of testing. Even though the natural frequency for all shafts stayed in the narrow range from 2.0 to 2.27 Hz, significant difference in the compliance function has been observed. Results from the testing show thaht the dynamic response of the drilled shaft is strongly site dependent.

Drilled shafts at the Doremus Avenu Bridge are placed in groups of three shafts at the each pier location, and they are extended into bridge columns. To get an insight into behavior of the group of shafts, response of all shafts was measured while one shaft was tested. The influence of the tested shaft on the first adjacent shaft was quite significant, but the furthest shaft barely showed response due to the excitation of the tested shaft. Shaft interaction should not be neglected for closely spaced shafts. In this case the shafts were spaced at about 3 shaft diameters.

Results from the drilled shafts testing at the Doremus Avenue Bridge should be an addition to the previously done testings to give better insight into dynamic shaft behavior of deep foundation.

5.2 Recommendation for Future Research

An extensive study of the site and the drilled shaft foundations has provided a large volume of data that can be used for future research.

The obtained dynamic stiffness of the drilled shafts can be utilized to calibrate existing numerical models. Dynamic behavior of deep foundations is still of big interest in research because of the strong dependence on site conditions, there is a need to verify numerical models using experimental data.

Obtained impedance functions should be used in the model of the whole structure, in this case in the model of the bridge, to evaluate the effects of the soil-foundation-structure interaction.

The results presented here are obtained under assumption of low strain dynamic response. In case of strong ground motion or large cycling loading large strain dynamic response is reasonable to occur. For that case additional analysis should be done.

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PUBLICATIONS

2004	Balic, M., Dynamic Characterization of Drilled Shafts on Doremus Avenue Bridge, 29th Annual Conference on Deep Foundations, Vancouver, BC, Canada, Sept. 29 - Oct. 1, 2004 (awarded student paper)
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