Dynamic Characterization of the Doremus Avenue Bridge Substructure

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ABSTRACT

Dynamic properties of the drilled shaft foundations supporting Doremus Avenue Bridge in Newark, New Jersey were determined by forced vibration testing. Doremus Avenue Bridge has been selected for instrumentation, testing and monitoring because it is first bridge in New Jersey designed according to LRFD specification. The main objectives of the substructure testing at Doremus Avenue Bridge were: (a) site evaluation with respect to the dynamic soil properties, and, (b) shaft evaluation for the purpose of definition of their dynamic stiffness. The site characterization entailed crosshole testing for the purpose of evaluation of the shear modulus profile. The drilled shaft impedance evaluation was done through forced excitation using an electromagnetic shaker. The response of the tested shaft, as well as the response of adjacent shafts, was 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.

INTRODUCTION

American Association of State Highway Transportation Officials (AASHTO) adopted the Load and Resistance Factor Design (LRFD) as the standard by which all the future bridges will be designed. The LRFD specifications consider the uncertainty and variability in the materials, loading and the behavior of structural elements through extensive probabilistic analysis and, therefore, continue to be refined and improved. Many of the 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 LRFD designed bridge. 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. At the same time the data obtained from the instrumentation and testing will be utilized to calibrate the numerical model of the bridge.

Instrumentation, testing and monitoring of the bridge substructure is a part of the research project. The main objectives of the substructure evaluation were:

1. site characterization with respect to the dynamic soil properties, and
2. drilled shaft dynamic stiffness (impedance) evaluation.

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.

The purpose of this paper is to describe evaluation of dynamic properties of the Doremus Avenue Bridge substructure. The first part of the paper deals with the site characterization using the crosshole test. The second part of the paper deals with evaluation of the dynamic response of piles through forced vibrations using an electromagnetic shaker. Conduct and results of the crosshole and impedance evaluation testing, and drilled shaft instrumentation and data usage, are presented and discussed.

DOREMUS AVENUE BRIDGE

The old Doremus Avenue Bridge was built in 1918. The bridge was the main access route to New Jersey's seaports. Over time a significant increase in the traffic over the bridge and high differential settlements of the bridge supports led to a very high deterioration, affecting both the safety and serviceability of the bridge. The condition of the old bridge is illustrated in Figure 1.

Fig. 1. The Old Doremus Avenue Bridge

The construction of the new bridge falls into the first phase of the Portway Program, a series of projects that will significantly improve links between the seaport, railroad and motor carriers. The construction of the new bridge began in 2000. The project involves the replacement of the old bridge while maintaining the traffic. The construction proceeded in two phases, where in the first phase a half of the new bridge was constructed while the old bridge remained open for traffic. In the second phase the old bridge was demolished and the traffic diverted to the finished half of the bridge. The other half of the new bridge was constructed in the place of the old bridge. The construction is expected to be finished in 2004. The new bridge under construction is shown in Fig. 2.

Fig. 2. Construction of the New Doremus Avenue Bridge

DOREMUS AVENUE BRIDGE SITE CHARACTERIZATION USING CROSSHOLE TEST

The site at the Doremus Avenue Bridge is a soil deposit consisting of five main strata. The top 15 to 25 ft is a fill of granular materials with interlayers of silty clay or organic soil. Below the fill is a 3 to 10 ft thick layer of organic silt, clay and peat, underlain by 5 to 15 ft thick sand deposit. The sand is fine to medium gradation with varying amounts of silt. Below the sand layer, there is a 20 to 45 ft thick layer of medium stiff to stiff clay and silt. Finally, there is a layer of up to a 30 ft thickness of medium to fine gravel, sand and silt. A moderately fractured shale bedrock is underlying the soil at depths of 60 to 80 ft below the ground surface. Soil profiles can be seen in Figs. 6 through 8 as the borings in the velocity profiles.

The crosshole test (Woods 1994) was used for the purpose of evaluation of the low strain shear modulus profile of the Doremus Avenue Bridge site. The shear modulus strain dependence is being evaluated through laboratory testing of recovered soil samples in the torsional shear device. The crosshole testing was conducted according to ASTM Standard D 4428 / D 4428M – 91, with three boreholes. A schematic and the plan view of the crosshole test, according to the ASTM Standard, are shown in Fig. 3.

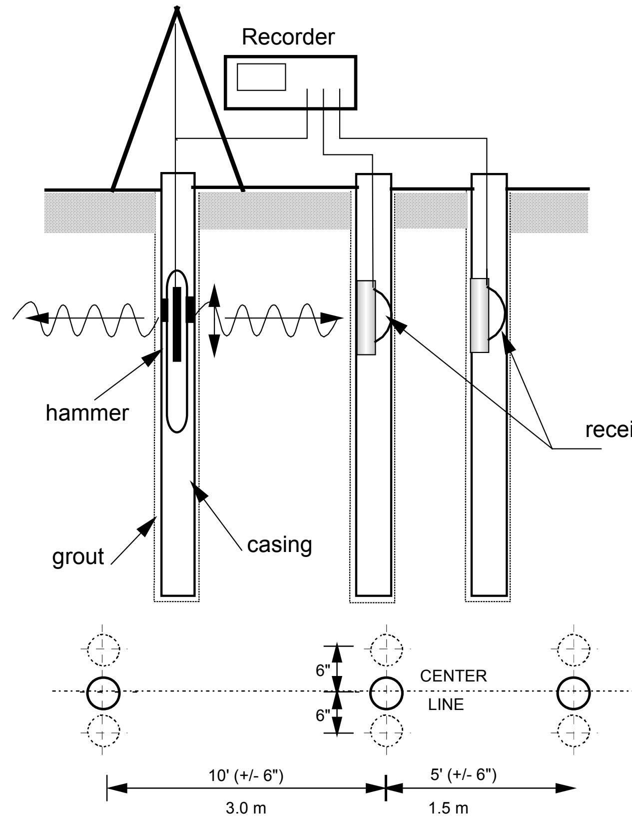


Fig. 3. Schematic and the Plan of the Crosshole Test

A shear type in-hole hammer, with hydraulically expanding borehole gripers was used as the source. The receivers were geophones with a pneumatically expandable rubber membrane to fix the receivers in the borehole. The verticality of each of the boreholes was measured using an inclinometer probe to obtain an accurate distance between the boreholes.

The crosshole test at Doremus Bridge was performed at five locations, at Piers 1, 2, 4, 5 and 8 of the new bridge. The crosshole locations are shown in Fig. 4, while the borehole depth information is provided in Table 1. Three boreholes were prepared for each crosshole test. All the boreholes were extended into the bedrock. The borehole casing was driven using a 300 lb hammer falling from a height of 24 in. The grouting was done with Portland cement and bentonite grout. The PVC casing was of a 4 in diameter, with the bottom end closed with a watertight cap. Crosshole testing at the Doremus Avenue Bridge is shown in Fig. 5.

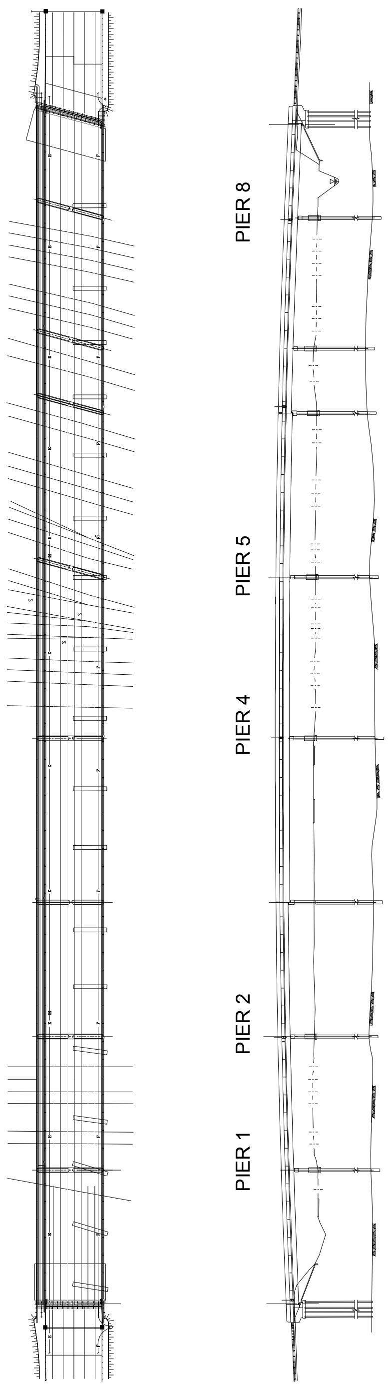


Fig. 4. Doremus Avenue Bridge Layout and Location of Piers and Crosshole Test

Table 1. Borehole Depths

Location	Borehole No.		
	1	2	3
	ft	ft	ft
Pier 1	69.32	75.88	75.78
Pier 2	83.40	82.58	84.09
Pier 4	83.66	84.68	83.17
Pier 5	79.92	79.98	80.09
Pier 8	75.20	75.98	75.39

Fig. 5. Crosshole Test at Doremus Avenue Bridge

Records were taken every 3 ft until the bottom of the borehole was reached. The velocities were calculated from the time difference between the shear wave arrivals at the two receivers and the distance obtained from the inclinometer measurements. The velocities were corrected near interfaces for a curved travel path. Shear wave velocity profiles for pier locations 1, 5 and 8 are shown in Figs. 6 through 8. A shear modulus profile for the site was obtained from the five velocity profiles using the relationship between the shear wave velocity and shear modulus:

$$V_S = \sqrt{\frac{G}{\rho}} \tag{1}$$

where V_S is shear wave velocity, G is shear modulus and ρ mass density of soil. The shear modulus profile is shown in Fig. 7.

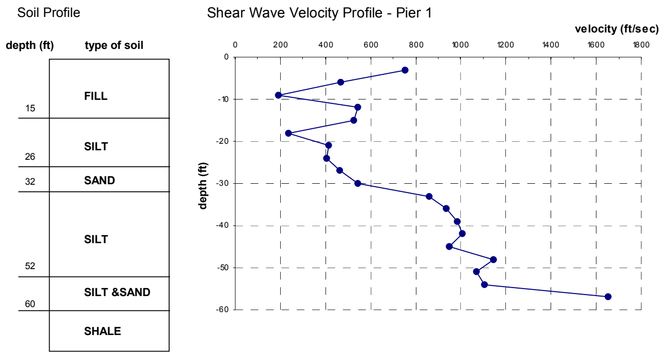


Fig. 6. Shear Wave Velocity Profile at Pier 1

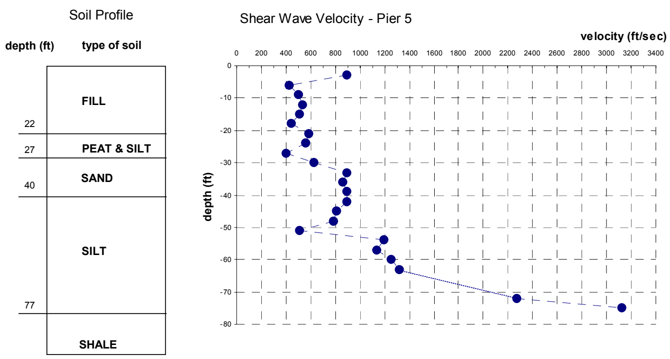


Fig. 7. Shear Wave Velocity Profile at Pier 5

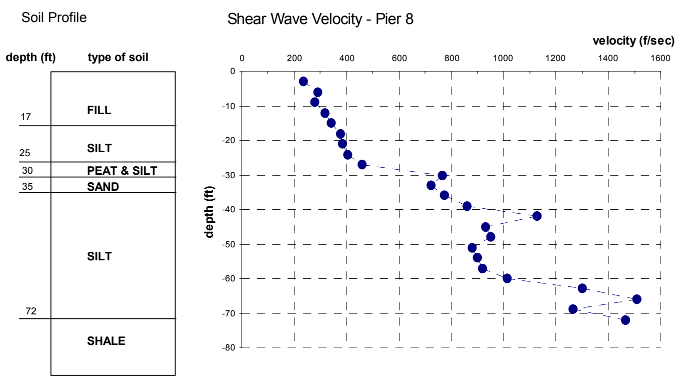


Fig. 8. Shear Wave Velocity Profile at Pier 8

SUBSTRUCTURE INSTRUMENTATION

There were two main objectives of instrumenting the substructure. The first one was to gain a better insight into the dynamics of a drilled shaft. The second one was to, jointly with a superstructure instrumentation, gain a better insight into soil-foundation-structure interaction (SFSI). The instrumentation plan included instrumentation of one of the drilled shafts, piers and a pier cap. The instrumented shaft was selected to be close to the superstructure instrumentation, in this case at Pier 2. The shaft instrumented encompassed five Mark Products L-22D triaxial geophones, placed at depths matching about the middle of each characteristic soil layer. Each of the geophones was placed in a 4-inch diameter protective PVC casing and fixed to a rebar cage.

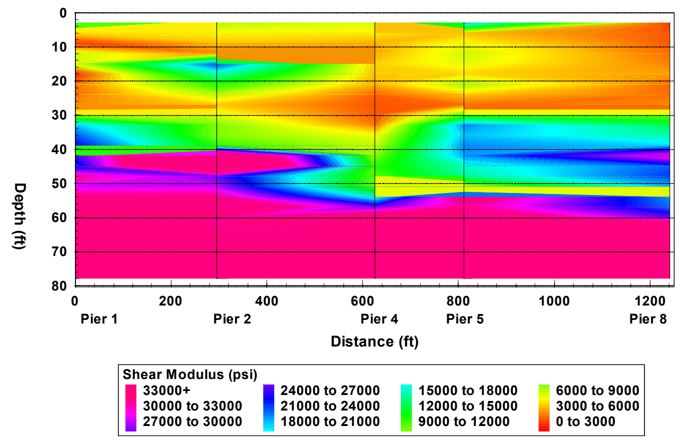


Fig. 10. Shear Modulus Profile Between Piers 1 and 8.

The cables were protected by 2-inch diameter PVC pipes. A schematic of the geophones arrangement in the instrumented shaft is shown in Fig. 11, and their installation and placement in the shaft in Fig. 12.

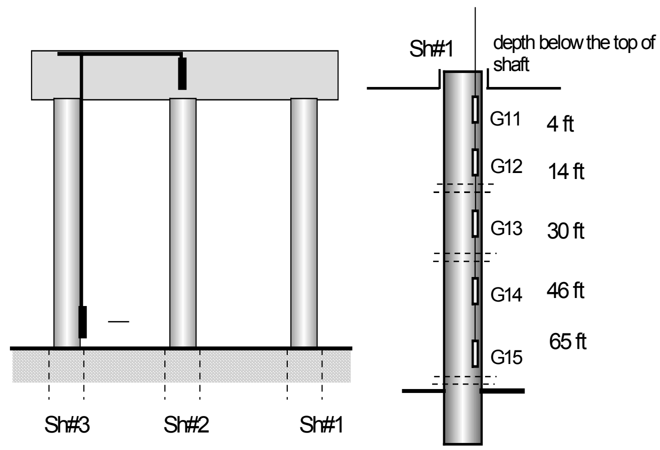


Fig. 11. Schematics of the Instrumented Shaft at Pier 2

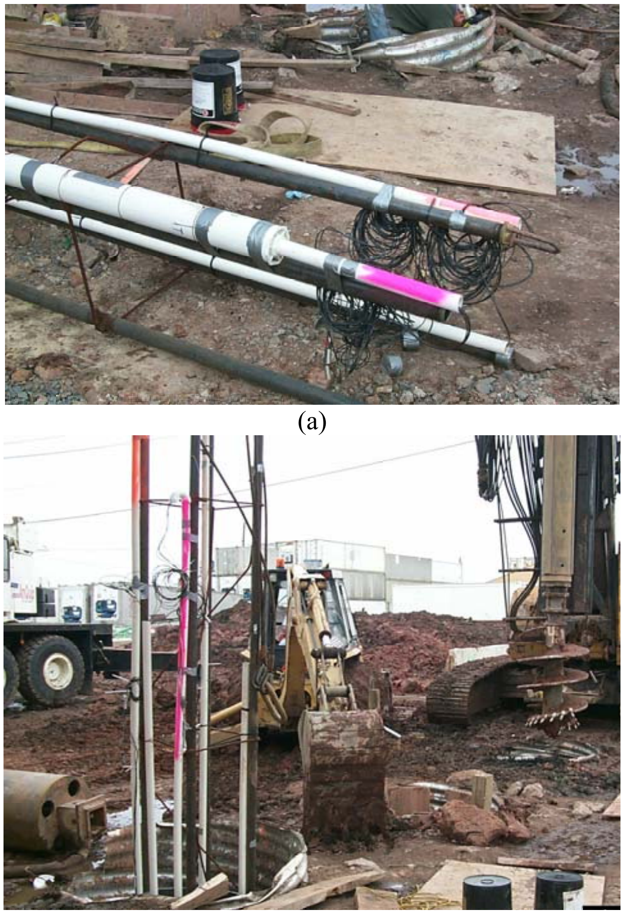
DRILLED SHAFT DYNAMICS EVALUATION

The main objective of the overall substructure characterization on Doremus Avenue Bridge project was to get a better insight into the drilled shaft dynamics. Particular objectives of the shaft testing included evaluation of shaft impedance functions, evaluation of effects of shaft interaction, and evaluation of effects of soil-foundation-structure interaction (SFSI). A schematic and geometrical properties of reinforced concrete drilled shafts are shown in Fig. 11 and Table 2, respectively. The shafts are socketed into the bedrock and are extending straight into the bridge columns above the ground.

Three more geophones at Pier 2 are attached to the piers and the pier cap. The geophones will be used for the future monitoring of the bridge response to vehicular and other dynamic loading, and for calibration of numerical models.

Table 2. Drilled Shaft Properties

Diameter	
in soil	4.0 ft
in bedrock	3.5 ft
Overall Length	91.82 ft
Length in Soil	81.59 ft
Length in Bedrock	10.23 ft
Material	Concrete



(a)

(b)

Fig. 12. (a) Installation and (b)Placement of Geophones in the Shaft

Background

There were many attempts to solve the problem of dynamically loaded piles using theoretical and numerical solutions [Novak (1974), Wolf *et al.* (1978), Kaynia *et al.* (1982), Gazetas (1984), etc.]. Even under idealistic assumptions of a linearly elastic, or viscoelastic homogenous soil, perfect bonding between soil and pile, etc., the solutions only approximate the actual response of piles.

A considerable body of experimental research has been done in the area of dynamic response of piles, 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 tests on actual piles (in-situ). In-situ tests have the advantage of providing actual soil and pile stress conditions. The tests are mostly done using some kind of an excitation placed on the pile head. For example, a quick-release vibration test was performed by Crouse *et al.* (1987) on two full size pile groups. One tested pile group consisted of eight vertical and eight batter concrete piles, and the other group consisted of eight vertical concrete piles. The diameter of the piles was 12.75 inches. The piles were embeded in 40 ft of loose, saturate sandy soil overlaying stiff glacial till. The measured natural frequency for pile groups in the horizontal direction was between 3.8 and 6.3 Hz. The damping ratio in the fundamental mode was between 0.05 and 0.15.

Han and Novak (1988) conducted experiments on large-scale model piles subjected to a strong horizontal and vertical motion, and compared the results to the theoretically predicted ones. The pile was a steel pipe with a diameter of 133 mm (5.24 in) and a length of 3.38 m (133.07 in). The pile cap was placed on top of the pile as a concrete block. The pile was placed in sand. Frequency response curves from those tests are shown in Fig. 13.

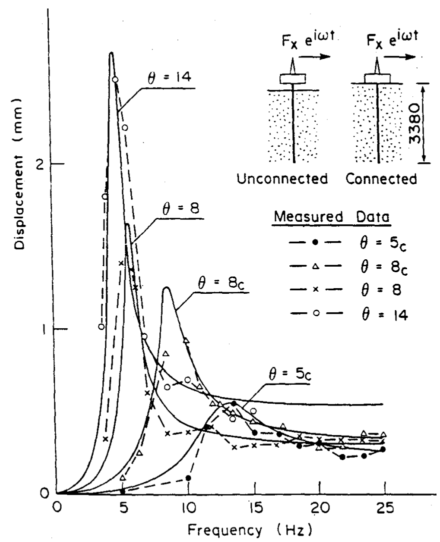


Fig. 13 Theoretical and Experimental Horizontal Rresponse Curve (Han and Novak, 1988)

El-Marsafawi *et al.* (1992) reported experiments on two pile groups. The first set of experiments was conducted on a group of six steel model piles. The piles were steel pipes 101.6 mm (4 in) in diameter. The piles were driven to a depth of 2.75 m (108.27 in) below the ground surface. Soil consisted of a layered stratum of silty fine sand with a gravel seam, resting on dense silty till. The system was harmonically excited using a mechanical oscillator. The second group of tested piles was test on full-scale reinforced concrete piles. The group of pile consisted of six cast in place reinforced concrete piles with a diameter of 0.32 m (12.6 in) and 7.5 m (295.28 in) in length, connected with a rigid reinforced cap. A single pile identical to those in the group was tested at the same site. The soil at the site was relatively homogenous sandy clay. A Lazan type exciter with two rotating eccentric masses was used. El-Marsafawi *et al.* have shown that using the concept of a week zone around the pile (Novak et al., 1980) improves theoretical model predictions of the pile response.

All the experimental work points to high nonlinearity and site dependence of the dynamic response of piles and pile groups. A reasonable agreement was observed in the comparison of the calculated and experimentally obtained stiffness for small displacement amplitude. The pile damping from the theoretical models may be overestimated unless some corrections are made for the pile separation and the influence of weak zone around the pile (Novak et al., 1980).

Test Description

The locations of the tested drilled shafts match the pier group locations of the crosshole test. At each test location two of the three drilled shafts were tested. The shafts were excited harmonically using an APS Model 400 electromagnetic shaker of the maximum horizontal force of 100 lb. The vibration force was introduced as a frequency sweep between 1 and 100 Hz. The shaker was suspended on a frame and attached to the drilled shaft through a steel section anchored into the shaft, as shown schematically in Fig. 14 and in the photo in Fig. 15. The shaker force was controlled by a signal generator and amplifier, and measured using a load cell placed between the arm of the shaker and the steel section. The response of the loaded and adjacent shafts was measured using triaxial Mark Products L-4C-3D geophones, placed on the shaft surface.

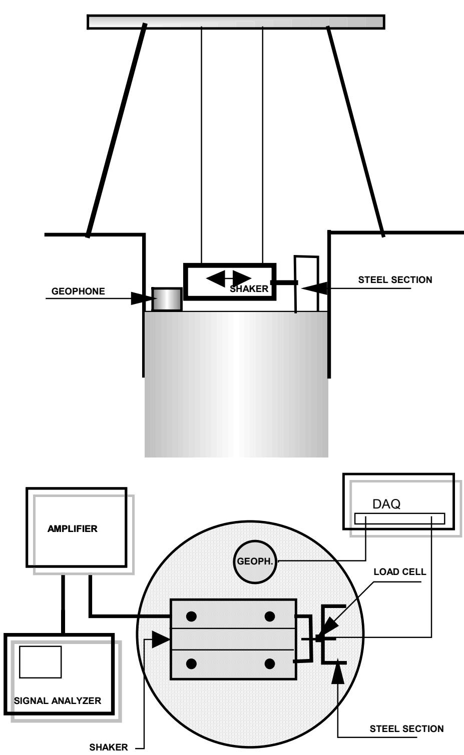


Fig. 14. Schematic of the Test Setup

Fig. 15. Drilled Shaft Test at Doremus Avenu Bridge

Test Results

The ultimate objective of the drilled shaft testing was to obtain impedance functions. The data reduction procedure to obtain the impedances is given in the flow chart in Fig. 16. Typical time histories and linear spectra for the forcing function and the velocity response are given in Figs. 17 and 18, respectively. While the total length of the record is about 120 s, only 30 s of each of the records are shown in the figures. The loading history is a frequency sweep from 1 to 100 Hz, where the frequency step is 1 Hz and for each frequency 5 loading cycles are applied.

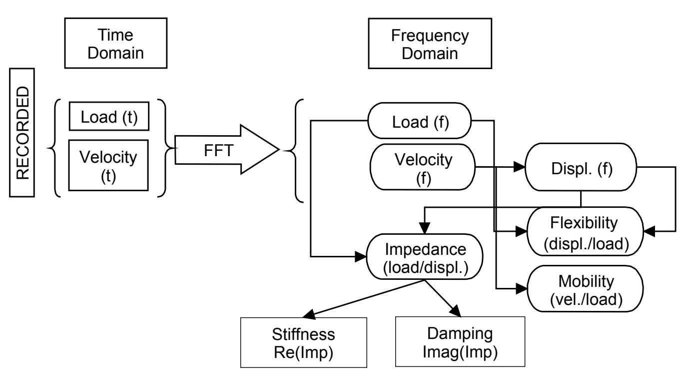


Fig. 16. Data Reduction Procedure

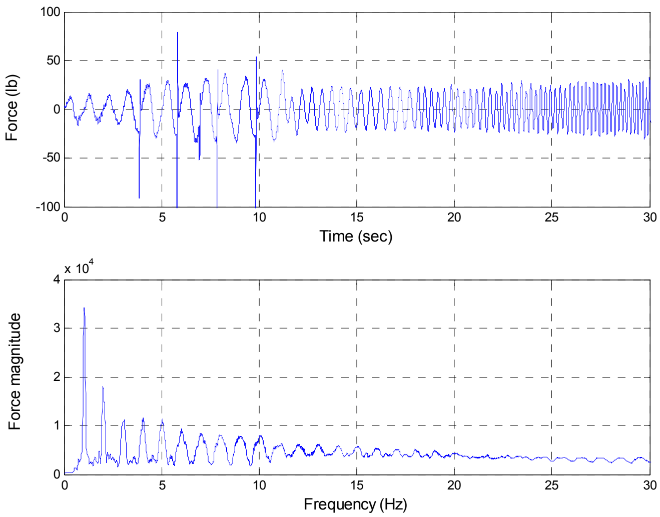


Fig. 17. Time History and Spectrum of Loading

Because of the limited number of cycles for each of the frequencies, the conditions during testing cannot be described as of an ideal steady state condition. 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 2.15 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 follows very well the shape of the driving force function.

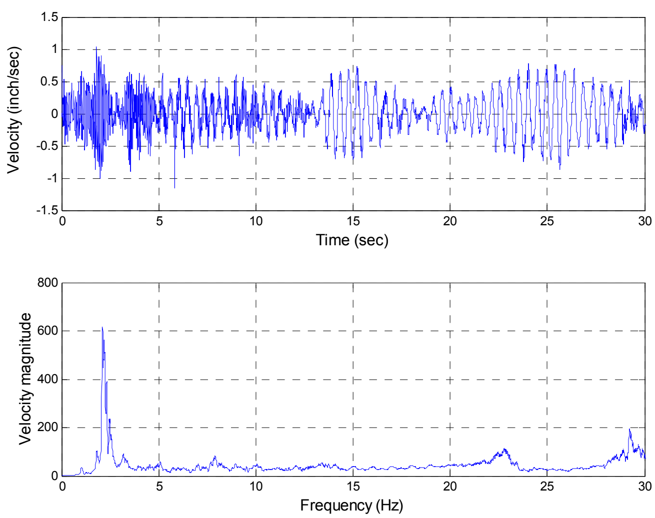


Fig. 18. Shaft Response Time History and Spectrum

For the purpose of evaluation of shaft impedances, the displacement spectra were obtained by dividing the velocity spectra by iω, where i is the imaginary constant and ω=2πf the angular frequency. A displacement spectrum obtained from the velocity spectrum in Fig. 18 is shown in Fig. 19.

In addition to the measurement of the response of the top of the pile, the response was measured at five elevations of the instrumented shaft (Fig. 11). Typical time histories and the corresponding spectra for the geophone at the top of the shaft, and the five embedded geophones are shown in Figs. 20 and 21, respectively. A very fast decrease in the response with depth can be observed. This is best illustrated by the maximum displacement functions with depth for four frequencies in Fig. 22.

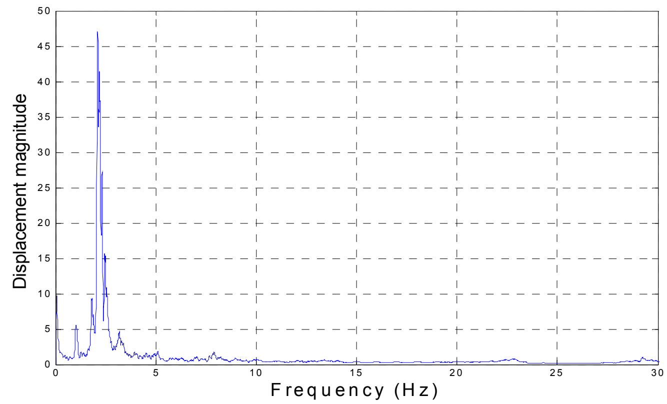


Fig. 19. Shaft Displacement Spectrum

The maximum displacement is diminishing at high rates, so that below the 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 Fig. 23. Almost a linear increase in the phase lag with depth and frequency points to a wave propagation nature of the surface disturbance.

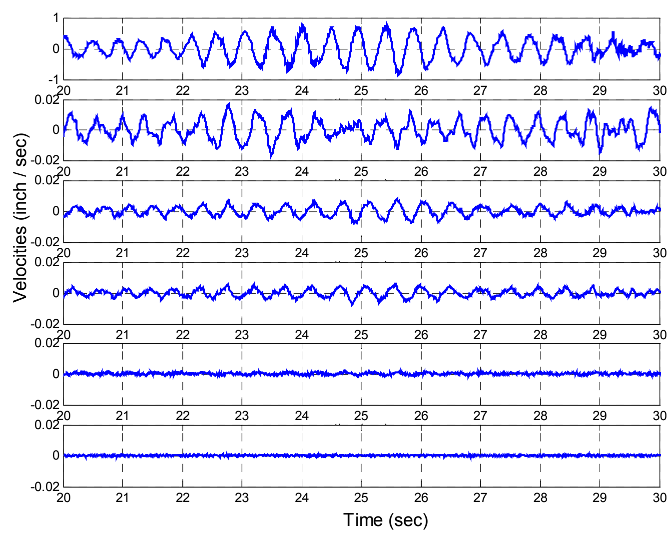


Fig. 20. Time Histories of the Built-in Geophones

Impedance functions were obtained for a number of tested shafts. The impedance function is defined as a complex ratio of the forcing function and displacement response spectra. The inverse of the impedance function is the compliance function, or the flexibility spectrum. The magnitude of the compliance function for four shafts at four pier locations is shown in Fig. 24. Differences in the compliance functions can be observed. Those are attributed to the differences in soil properties and layer thicknesses, and the depth of embedment of the top of the shaft that varied from 0 to 3.5 ft. The fundamental frequency of the shaft was little affected by those differences, and it stayed in a narrow range between 1.9 and 2.2 Hz.

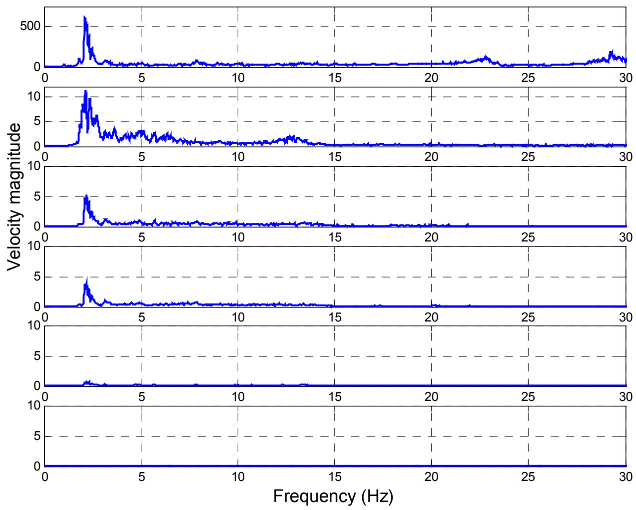


Fig. 21. Response Spectra of the Built-in Geophones

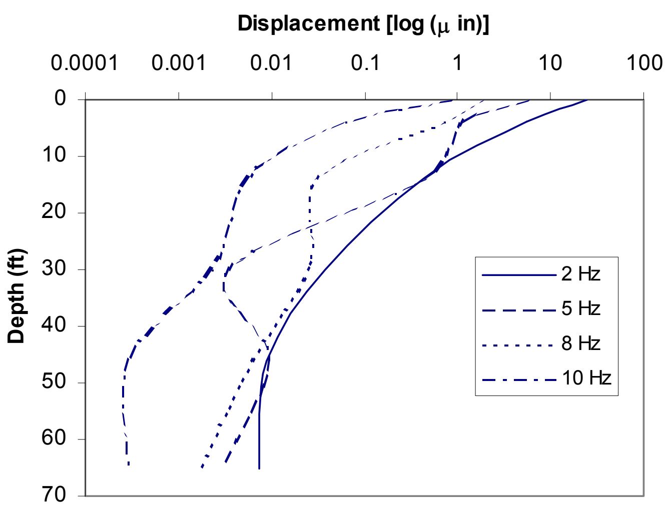


Fig. 22. Displacements with Depth for Four Frequencies

Finally, for the purpose of a future shaft interaction study, the response of adjacent piles was recorded. Kaynia et al. (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 Fig. 25.

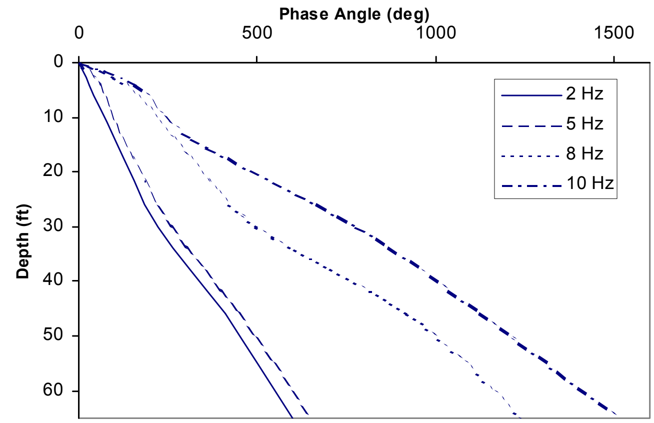


Fig. 23 Phase Lag with Depth of the Shaft for Four Frequencies

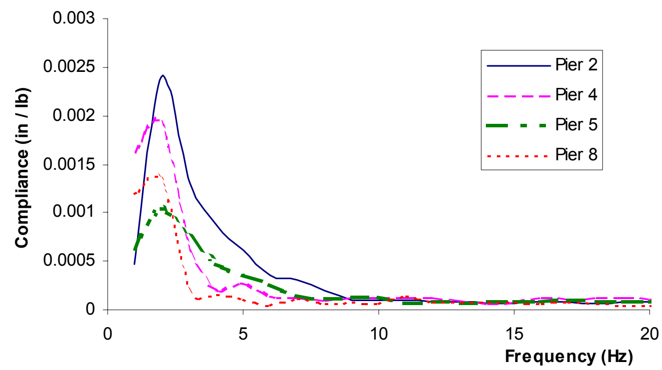


Fig. 24. Magnitude of Compliance Functions for Four Shafts

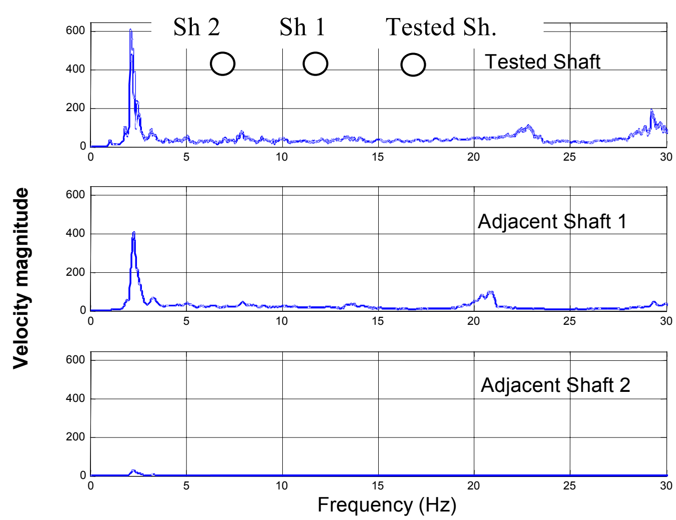


Fig. 25. Velocity Spectra for the Tested Shaft and Two Adjacent Shafts

CONCLUSIONS

An extensive study of the site and the substructure (drilled shafts) at the Doremus Avenue Bridge has generated a large volume of data that will be utilized for several purposes. The first purpose will be evaluation of the shaft dynamics, primarily with respect to their impedances and evaluation of the pile group interaction effects. The second purpose of the study will be to calibrate existing theoretical and numerical models for more realistic practical applications. Finally, the already collected data, and the data that will be collected in the future using the instrumentation installed in the substructure and superstructure, will be utilized to evaluate effects of the soil-foundation-structure interaction. The data and preliminary results of the conducted analyses are in agreement with observations of some of the earlier experimental studies of the shaft dynamics

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