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Analytical Modeling and Instrumentation Planning of The Doremus Avenue Bridge

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Submitted by

Hani Nassif, Ph.D., P.E., Assistant Professor
Nenad Gucunski, Ph.D., Associate Professor

Talat Abu-Amra, Graduate Research Assistant
Mayrai Gindy, Graduate Research Assistant
Martina Balic, Graduate Research Assistant

Dept. of Civil & Environmental Engineering
Center for Advanced Infrastructure & Transportation (CAIT)
Rutgers, The State University
Piscataway, NJ 08854-8014

NJDOT Research Project Manager
Mr. Nick Vittilo

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16. Abstract

In 2007, the American Association of State Highway Transportation Officials (AASHTO) will adopt the Load and Resistance Factored Design (LRFD) Bridge Design Specifications as the mandatory standard by which all future bridge structures will be designed. New Jersey has committed itself to the adoption of the LRFD Specifications on January 2000. The LRFD Specifications consider and ascertain the variability in the behavior of structural elements through extensive statistical analyses and, therefore, continue to be refined and improved. However, many of the Specifications' design approaches and methodologies have been adopted with limited or virtually no experimental validation. Therefore, there is a need to validate these new design procedures and models as well as the integrity of LRFD designed bridge structures.

The Doremus Avenue Bridge, located in Newark, NJ, is New Jersey's initial LRFD design. The construction project will involve replacement of an existing bridge structure that primarily carries truck traffic into the State's seaport. The main objective of this study is to evaluate the analytical behavior of the Doremus Avenue Bridge and to identify the instrumentation procedure(s) and equipment to be used in the field testing and monitoring program. The identification process is implemented in two phases: 1) development of a detailed Finite Element Model (FEM) and 2) the planning and optimization of instrumentation schemes and the sensor location. The aim is to enable the New Jersey Department of Transportation (NJDOT) to successfully select the appropriate instrumentation modifications.

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ABSTRACT

In 2007, the American Association of State Highway Transportation Officials[1] (AASHTO) will adopt the Load and Resistance Factored Design[2] (LRFD) Bridge Design Specifications as the mandatory standard by which all future bridge structures will be designed. New Jersey has committed itself to the adoption of the LRFD Specifications since January 2000. The LRFD Specifications consider and ascertain the variability in the behavior of structural elements through extensive statistical analyses and, therefore, continue to be refined and improved. However, many of the Specifications' design approaches and methodologies have been adopted with limited or virtually no experimental validation. Therefore, it is believed that there is a need to validate these new design procedures and models as well as the integrity of LRFD designed bridge structures.

The Doremus Avenue bridge structure, located in Newark, NJ, is New Jersey's initial LRFD design. The construction project involves the replacement of an existing bridge structure that primarily carries truck traffic into the state's seaport. The main objective of this first-year proposal is to evaluate the analytical behavior of the Doremus Avenue Bridge. The study identifies the procedure(s) and parameters used in bridge instrumentation and develops alternatives for analyzing, testing, and monitoring the new structure. The identification process will be implemented in two phases: 1) development of a detailed Finite Element Model (FEM) that incorporates the nonlinear behavior of concrete material and 2) the planning and optimization of instrumentation schemes and the sensor location. The aim is to provide a methodology that will enable the New Jersey Department of Transportation (NJDOT) to successfully select the appropriate instrumentation procedures and testing equipment.

INTRODUCTION

Scale models and laboratory-based testing that existed prior to the development of the LRFD Specifications alone can not reveal the actual as well as realistic behavior of bridge structures. Moreover, the effort to prioritize as well as schedule repair and rehabilitation of bridge structures requires an accurate as well as systematic assessment and non-destructive monitoring of bridge conditions. This study, with the instrumentation and monitoring of the Doremus Avenue Bridge, provides the engineering community at large with vital feedback on the short and long-term performance of bridges that are designed according to the new AASHTO-LRFD Bridge Design Specifications[2]. The instrumentation plan will achieve the short term goal of identifying the causes of deck cracking during construction and the long term goal of reducing maintenance and life-cycle rehabilitation costs.

It is anticipated that the bridge will be instrumented to monitor its performance over a period of several years (e.g. 5 years). It is also envisioned that the Doremus Avenue Bridge will act as a national "test bed" for verifying certain parameters of the AASHTO-LRFD Bridge Design Specifications.

The following sections describe the objective, scope, and tasks involved in developing analytical models as well as planning instrumentation schemes and sensor locations prior to the actual construction of the Doremus Avenue Bridge. It is expected that the study will continue to allow instrumentation, field-testing, and long term monitoring over a 5-year period that will consist of three Phases:

1. Phase I: Bridge Modeling, Instrumentation Planning, and Coordination of Tasks.
2. Phase II: Bridge Instrumentation, Testing, and Verification prior to Traffic Opening.
3. Phase III: Bridge Testing and Long-Term Monitoring after Traffic Opening.

OBJECTIVE

The main objective of the overall five-year study is to instrument, monitor, and evaluate the structure during and after its construction. The evaluation process aims at assessing the new AASHTO LRFD design procedures and identifying what the New Jersey Department of Transportation (NJDOT) wishes to establish as future bridge design guidelines. The instrumentation schemes will be implemented during the construction phase. This will permit measuring the “undisturbed” behavior of the bridge and establishing the structure’s “finger prints” prior to traffic opening. Both the superstructure and substructure will be instrumented and monitored simultaneously.

The main objective of this one-year study is to evaluate the analytical behavior of the Doremus Avenue Bridge and develop a technical specification for testing equipment and instrumentation procedures. The study identifies the procedure(s) and parameters, used in bridge instrumentation and analysis. The identification process is implemented in two phases: 1) development of a detailed Finite Element Model (FEM) that incorporates various parameters and the nonlinear behavior of concrete material and 2) the planning and optimization of the instrumentation schemes and the sensors location. A general-purpose finite element code, ABAQUS(3), is utilized to derive the model. ABAQUS includes a variety of routines that allow for defining specific material models and provisions, such as concrete cracking and tension stiffening models, reinforcing steel rebars, boundary conditions, bond behavior and interaction between the reinforcing steel bars and concrete, soil layers and its mechanical properties.

SCOPE

This report covers Phase I of the project only and identifies the method(s), procedure(s), and parameter(s) considered in the analysis and instrumentation of the Doremus Avenue Bridge. The following tasks are employed to successfully complete the project:

1. Conduct a literature review and a statistical evaluation of different parameters influencing bridge design and analysis.

2. Develop a technical specification that will describe all equipment used in the instrumentation of the Doremus Avenue Bridge and the procedures to be

followed by the contractor for the implementation of this instrumentation plan. The technical specifications cover both the instrumentation of the bridge superstructure as well as substructure.

3. Develop detailed finite element models that will incorporate the nonlinear and cracking behavior of reinforced concrete and any future field-tested material properties. The model should simulate actual behavior under various types of truck loading as well as environmental loads such as temperature, differential expansion between steel and concrete, etc.

4. Compare results from the FE model and initial calculations. The FE model need's to be validated experimentally in Phase II of the project and utilized henceforth more accurately. Information from bridge sensors will be used to update the FE model. The information obtained from the model will be processed to update the instrumentation plans accordingly. However, for the purpose of developing the instrumentation schemes and plans, a 2-D model is considered sufficient.

5. Perform preliminary analysis and testing of substructure elements, soil characterization, and seismic cross holes.

6. Recommend modifications or additional field verifications during each phase of the instrumentation plan.

LITERATURE SURVEY

Bridge testing is an art and science that has been utilized for several centuries. Earlier tests were basically proof tests to determine if a bridge could satisfactorily carry load at some assumed level. Such tests were initiated whenever a new method of construction had been used or new materials were introduced. However, in recent decades, testing of bridges has been utilized for a number of different purposes. The main purposes are obtaining load distribution, ultimate capacity, load history, dynamic and static responses, truck weight and configuration, and other general information. For brevity, only references related to dynamic analysis and testing of bridges as well code provisions are included in this report. A more comprehensive search pertaining to various types of bridge testing will be included in Phase II of this five-year project.

Theoretical Models

Most theoretical models are based on a bridge beam model subjected to a moving load. The moving load can be constant or variable. The bridge beam can be modeled as either a continuous or a discrete system. Honda et al.(24) treated the bridge as a continuous system of several simple beams. The discrete system may be in the form of a simple beam, simple beam with torsion(21), or orthotropic plate(30). Recently, the three-dimensional (3-D) grillage model[26] and 3-D finite element model of a bridge[45] have been used together with 3-D vehicle models due to the improvements in computer technology. The road roughness profiles are usually represented by artificial bumps on the bridge entrance[21], simple sinusoidal model[23], and random processes together with Fourier series[24,26]. The vehicle models consist of a constant force, one degree-of-freedom[12] (DOF), two DOFs, or multiple DOFs[49]. The types of bridges studied include single span bridges[21], multispan continuous bridges[13,20,23]; Chatterjee et al. 1994), cantilever bridges[49], and suspension bridges[13].

Veletsos and Huang[49] presented dynamic studies of bridge-vehicle interactions performed at the University of Illinois from 1950 to 1970. A simple beam bridge model was adopted in dynamic studies of single span bridges, three-span continuous bridges, and three-span cantilever bridges. An orthotropic plate model of a bridge was also used in the dynamic analysis of single-span bridges. A 2-D vehicle model for three-axle semi tractor-trailer was developed taking into account the effect of interleaf friction in its suspension system. Considerations in modeling bridges, vehicles, and integration procedures of motion equations were presented in detail.

Gupta and Trail-Nash[21] in Australia idealized a single span composite slab-on-girder bridge as a simple beam as well as an orthotropic plate. The road roughness profile was a 45° ramp at the bridge entrance. A standard HS20-44 truck was modeled as a planar two-axle sprung mass system with a frictional device. The effects of combinations of the ramp, braking, and eccentric loading on the Dynamic Load Factor (DLF) were identified as significant. It is found that DLFs from the bridge beam model are higher than those from the orthotropic plate model.

Hawk and Ghali[23] in Canada developed a modal superposition, analytical procedure called the Iterative Dynamic Substructuring Method (IDSM) to determine dynamic behavior of a three-span continuous bridge under multiple moving trucks. The slab-on-girder bridge was modeled as a grid and the road roughness profile was sinusoidal. The trucks were considered to be sprung loads with several contact points (multiple wheel positions). In the tire-suspension systems, the tires were assumed to be undeformable and the stiffness of the suspension springs, as chosen to fit the first natural frequency of the truck. It is found that the relative truck positioning has very strong effects on the DLF, and the multiple truck loading appears to result in a higher DLF. This is contrary to the reduction factor for the DLF in the OHBDC[40].

Mulcahy[30] in Australia used an orthotropic plate model with higher order finite strips instead of a simple beam model of a bridge, that also included the road roughness profiles. The vehicle model was a planar, two-axle, sprung mass system. The wheel loads were applied to the bridge as two equal line loads. The Newmark-β[34] method was used to integrate equations of motion. It was found that vehicle models do not affect the dynamic behavior of a bridge unless the braking effects are considered.

Hwang and Nowak[26] developed a procedure to calculate the DLF for a simple slab-on-girder bridge. The procedure included bridge dynamics, road roughness, and vehicle dynamics. The calculations were carried out for randomly selected road roughness profiles. The Monte Carlo simulation was used to find the statistical parameters of dynamic loads for various bridge spans. The analytical results indicated that the DLF varies from 1.10 to 1.20. The coefficient of variation of dynamic loads varies from 0.08 to 0.10. Savard[45] in Canada used a three-nodded beam and an eight-nodded quadratic plate shell element to model a bridge. The road roughness profiles were randomly generated by PSD. The vehicle was represented as a three-axle sprung mass system with a nonlinear interleaf friction suspension-tire system, having 11 DOFs. The Newmark-β[34] integration was adopted. The computer results were compared with experimental data, but good agreements had not been reached because of the lack of vehicle models.

Humar and Kashif[25] in Canada presented control parameters and design recommendations for bridge-vehicle interactions these were based on an analytical investigation of a simple beam model traversed by a moving mass system.

Green and Cebon[20] in Canada developed a convolution formulation for bridge-vehicle interactions in the frequency domain using Fast Fourier Transformation (FFT). The bridge was modeled as a simply supported beam as well as an orthotropic plate with flexible supports. The vehicle was represented by a “1/4-car” with two DOFs and a “half-car” with four DOFs. The bridge modal analysis was based on experimental data from the impulse testing. Afterwards, the dynamic behavior of the bridge under testing vehicle movements was estimated using the bridge mode shapes and measured wheel loads. The computer results were validated by the acceleration recorders collected on the bridge site. It was also found that the computer simulation program developed in the frequency domain was very efficient in comparison with those in the time domain. Furthermore, Green[20] studied the effects of leaf-spring and air-spring suspensions on the dynamic behavior of short span bridges. A bump at the bridge entrance and a random road roughness profile were used. The air-spring suspension caused a significantly lower bridge dynamic response than leaf-spring suspension.

Chatterjee et al.[13] in India studied the dynamic behavior of a three-span continuous bridge under a moving vehicle load, using closed form iteration. The bridge was modeled as a continuous Bernoulli-Euler beam. The road roughness profile was specified by PSD. The vehicle was idealized as a single unsprung mass system as well as a single sprung mass system with bilinear force-deformation suspensions. It is found that the effects of bridge torsion stiffness on the DLF are not significant. However, random roughness profiles cause a different dynamic behavior of bridges. The difference depends on the vehicle speed and the bridge to vehicle frequency ratio. Moreover, a multispan suspension bridge was analyzed, using 1-D, 2-D, and 3-D vehicle models[13]. The effects of bridge torsion stiffness and vehicle models on the DLF are only significant when the bridge to vehicle frequency ratio is much higher.

Experimental Studies

Experimental studies involve data capture, modal analysis, and the identification of dynamic characteristics of bridges. The measured parameters are usually dynamic strain, acceleration, and deflection of bridges under testing vehicles and/or normal traffic loads. At the present time, few field-testings of bridges have been conducted to validate theoretical models. However, it is believed that dynamic testing of highway bridges will become a routine task to evaluate bridge service conditions. After all, modern electrical and computer technology has provided affordable instruments and high quality techniques to make data processing and decision-making much easier.
Billing[9,10] in Canada tested 27 bridges with various configurations. These tested bridges were steel, concrete and timber bridges with the span lengths ranging from 15 to 366 ft. Four testing vehicles from 54 to 130 kips were used. The dynamic loads were measured in terms of a fraction of static deflections. The experimental results of 22 bridges and 30 spans were made available. The structural types of these bridges included steel and prestressed concrete slab-on-girder, steel box girder, steel truss, and rigid frame. The field testing data were recorded from testing vehicles and normal traffic. It is shown that the means of the DLF are rather low (less than 0.3). On the other hand, the corresponding coefficients of variation of DLF are very high, varying between 0.56 and 1.11. Considerable differences in the experimental data for very similar bridge structures indicate the importance of other affecting factors such as road conditions.

Cantieni[11] in Switzerland tested 226 bridges that were mostly prestressed concrete bridges. With the exception of 11, all of the bridges were loaded with the same vehicles under the same loads and tire pressures; thus, the variability due to vehicle dynamics was minimized. The effects of local unevenness on the bridge surfaces were also investigated. The study showed that the DLFs were as high as 0.7 for the bridges fundamental natural frequencies ranging from 2 to 4 Hz. Recently, Cantieni[12] summarized dynamic testing of highway bridges all over the world in his report titled "Dynamic Behavior of Highway Bridges Under the Passage of Heavy Vehicles."

O'Connor and Pritchard[39] in Australia tested a small span highway bridge. The composite bridge has a concrete slab with I-shaped steel girders. Large impact factors from strain measurements varied from (-0.08) to (+1.32) for both light and heavy vehicles. This indicates that the impact factors may be vehicle dependent and vary with suspension geometry. In order to identify the high-impact vehicles, O'Connor and Chan[38] developed a computer program to predict the theoretical response of bridges under multiple equivalent static axle weights (predictive analysis). Moreover, a computer program to obtain dynamic wheel loads from the measured displacements, bending moments, or accelerations (interpretive analysis) was developed. The bridge was modeled as a simple beam as well as a grid. The moving loads were restricted along the main girders. The finite-integral method (FIM) was chosen. The interpretive computer results were compared with experimental measurements. It was found that the predictions based on measured deflections were more sensitive to error than those based on measured bending moments. Larger errors may occur when acceleration records are used. This is due to high frequency noises. Nassif and Nowak[32] studied the dynamic testing data obtained by Billing[9]. For steel bridges, a range of 1.08 and 1.20, with a standard deviation of 0.05-0.20 was given for the mean values of DLF.

Bakht and Pinjarkar[5] in Canada summarized eight definitions of the impact factor in the history of dynamic studies of bridges. After careful study, they stated that the impact factor is not a tangible entity susceptible to deterministic evaluation. The impact factor may be taken into account for bridge design and evaluation only by a probability approach. The various parameters that might mislead conclusions were discussed. Furthermore, the procedure for obtaining design values of the impact factor through field testing was recommended.

Paultre et al.[41] in Canada reviewed both theoretical and experimental studies of dynamic interactions of the bridge-vehicle system before 1990. The following findings were presented: (1) The DLF is related to the fundamental natural frequency of the bridge; (2) Theoretical models cannot reliably evaluate the DLF. However, they are found to estimate bridge natural frequencies and mode shapes with good accuracy; (3) Full-scale field testing of a bridge under normal traffic is the only economical and practical way to get the DLF, but a general procedure of field testing is required to compare it with different experimental data. As a result, Paultre et al.[42] described a general procedure and equipment for dynamic testing of highway bridges in the Quebec Ministry of Transportation, Canada. The measured bridge vibration frequencies and mode shapes were compared to the FEM model developed by Savard[45].

Code Provisions

Most of the bridge design codes specify dynamic loads as additional static live loads. The AASHTO (1931) formulated the simple empirical equation as 50 / (L+160) to relate DLF to the bridge span length, where L is the bridge span length in feet. The maximum value of the DLF was 1.25. Later, the AASHTO (1944) revised the empirical equation as 50 / (L+125), where the maximum value of the DLF equals 1.30. This simple empirical equation is still in effect in a recent AASHTO specification[1].

The OHBDC of 1979, which was more conservative than the AASHTO, specified the DLF as a function of the fundamental natural frequency of a bridge. Based on testing data from Billing[9], the OHBDC of 1983 reduced the DLF to a range between 1.2 and 1.4, of which the higher value corresponded to the fundamental natural frequencies between 2.5 and 4.5 Hz. However, the actual dynamic behavior of a bridge seems to be characterized by a considerable degree of variation, which points to the importance of other affecting factors such as road conditions and vehicle dynamics.

Similar to the OHBDC, the Swiss bridge design code specifies the DLF as a function of the fundamental natural frequency of a bridge, which reflects the experimental data obtained by Cantieni[11]. Higher values of the DLF are also found around the bridge natural frequency of 3 Hz, which is the approximate vehicle bouncing frequency. The same results from the Swiss and Canadian bridge design codes indicate that the DLF is affected not only by the bridge natural frequency but also by other factors such as road conditions and vehicle dynamics.

FIELD INSPECTION OF THE EXISTING DOREMUS AVENUE BRIDGE

The newly proposed Doremus Avenue Bridge is a vital link in the Portway intermodal corridor providing access to Newark Air and Sea Ports. Figure 1 shows an aerial view and location of the bridge in relation to nearby transportation systems. It is part of an integrated roadway infrastructure system that will carry heavy weight truck traffic. Furthermore, it will eventually become the first bridge designed according to the new Load and Resistance Factor Design (LRFD) – American Association of Transportation Official (AASHTO) Bridge Specifications in the State of New Jersey. It is expected that 15,000 trucks will utilize the bridge every day, resulting in 2 million truck trips for a hauling capacity of 1.4 million containers every day. Trucks require access to railroad yards as well as an integrated bridge-roadway system.

Figure 1. Location of the new Doremus Avenue Bridge, Newark, New Jersey.

(a) (b)

Figure 2. View of the existing Doremus Avenue Bridge a) looking northbound and b) underneath.

(a) (b)

Figure 3. a) View of underside of bridge – girders b) View of the underside of the bridge near pier location.

(a) (b)

Figure 4. a) Condition of plate girders for the old Doremus Avenue Bridge and b) a rendering of the proposed Doremus Avenue Bridge.

Current access to the port is badly deteriorated and congested. The Doremus Avenue Bridge acts as a "bottleneck" for the truck traffic and would be an ideal study point. Figures 2 through 4(a) illustrate the state of the existing Doremus Avenue Bridge and the level of deteroitation that has taken place over the years. Figure 4(b) shows a rendering of the proposed new Doremus Avenue Bridge design. The new Doremus Avenue Bridge is a nine span bridge divided into three units, each consisting of three continuous spans. The focus for this study is placed on unit 1, which consists of spans 1 through 3.

TECHNICAL SPECIFICATIONS

The Equipment Technical Specification is developed in coordination with the NJDOT Bureau of Structural Engineering and is included in the final tendering documents for the Doremus Avenue Bridge. A detailed technical description of the equipment to be used in the instrumentation of the Doremus Avenue Bridge is presented in Appendix A. The technical specifications are developed by the first author based on past experience in field testing of bridges, reviewed by Engineers from Parson Brinckerhoff (PB), and approved by NJDOT Project Manager and technical staff. Moreover, the instrumentation procedure(s) as well as the Contractor's responsibilities towards the Rutgers Research Team are also outlined.

BRIDGE SUPERSTRUCTURE

Finite Element Model

The Project Team has performed extensive work on modeling the bridge deck using the Finite Element Analysis (FEA) method with great success. The model shown in Figure 5 was used in a project for the Michigan DOT and exhibited a good correlation with stresses from field test data. The FE model(s) developed using ABAQUS are validated from experimental observation in the field and developed further to include other extreme events such as thermal changes. Data from field tests will be used to calibrate this type of models to New Jersey bridges and site-specific information. Moreover, a 3-D FE Model is used for checking the deflection, stresses, and cracking behavior of the bridge under various types of loading conditions.

Finite Element Model Verification

The FE model(s) developed using ABAQUS will be validated and calibrated from experimental data collected in the field. The ABAQUS model is extensive and accurate, since it has been verified using results from prior field tests. This 3-D model is capable of incorporating the nonlinear cracking behavior of concrete under heavy loads and environmental conditions. Figure 5 shows a 3-D FE model for a steel girder bridge using two-node, linear and quadratic interpolation beam elements for girders and diaphragms and a triangular shell element to model the concrete deck slab. Various models have been used with shell, beam, as well as solid brick elements. The Project Team will continue to develop the FE model and validate its results by comparing them with results from future field tests. Once the model is validated and calibrated using field observations, the parametric study can be finalized. Figure 6(a) illustrates the 3-D model for unit 1 of the Doremus Avenue Bridge loaded with an HS-20 truck loading. The model combines shell and beam elements representing the deck slab and griders, respectively. The shell and beam elements are connected together using a rigid link that simulates composite action between the deck slab and plate girder. Figure 6(b) shows the deflected shape of the bridge under the effect of the HS-20 truck loading.

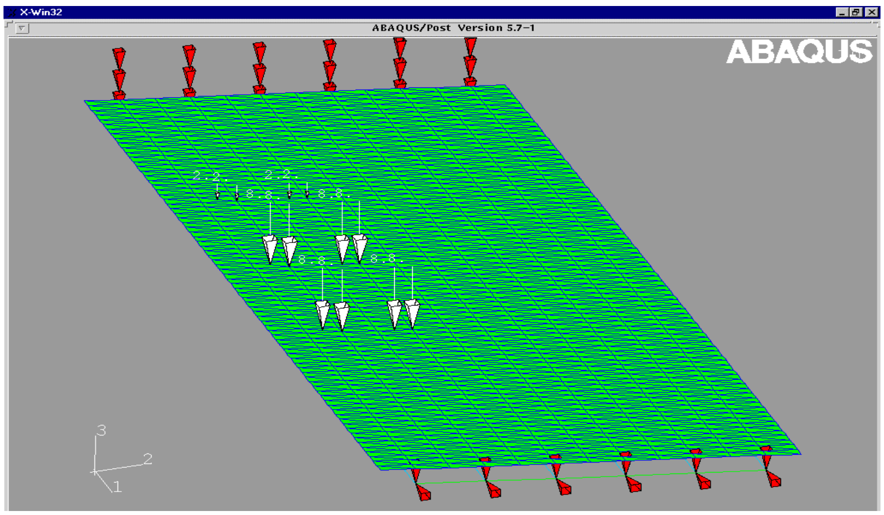


Figure 5. A typical three-dimensional finite element model for a steel girder bridge.

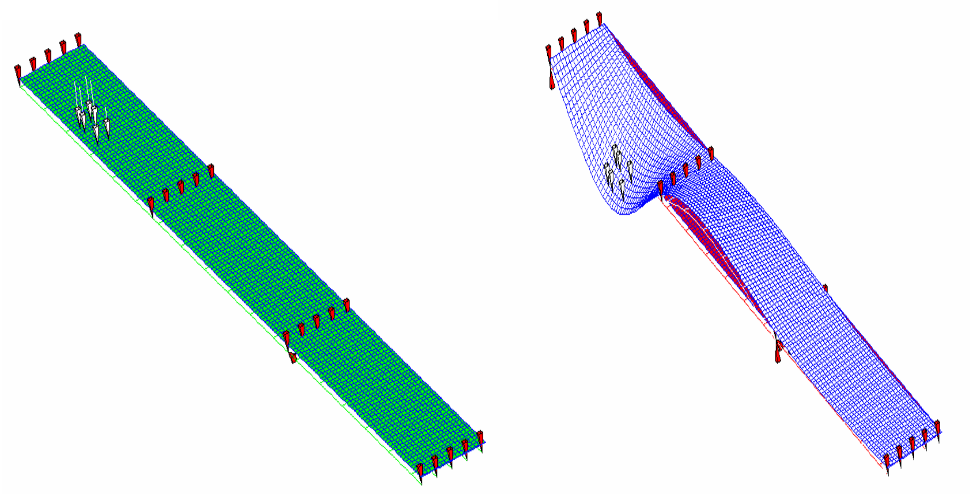

(a) (b)

Figure 6. Three-dimensional finite element model for unit 1 in Doremus Avenue Bridge: (a) span 1 loaded (b) deflected shape under the effect of an HS-20 truck loading.

Instrumentation Schemes and Optimization of Sensors Location

Currently, the project team, in coordination with Parson's Brinckerhoff, the NJDOT Bureau of Structural Design, as well as the Project Manager for the Doremus Avenue Bridge Project at the NJDOT Division of Project Management, has submitted an instrumentation plan and has been established as a part of the Bridge tender drawings. The final sheets of instrumentation are presented in Appendix B. The project team will adjust the instrumentation plan according to the field conditions and requirements of the contractor. Various scenarios have been established using the FE model depicted in figure 6. Figure 7 diagrams the main location considered for the deck instrumentation. Also, Figure 8 shows a typical layout of sensors in the deck slab and girder for each span location.

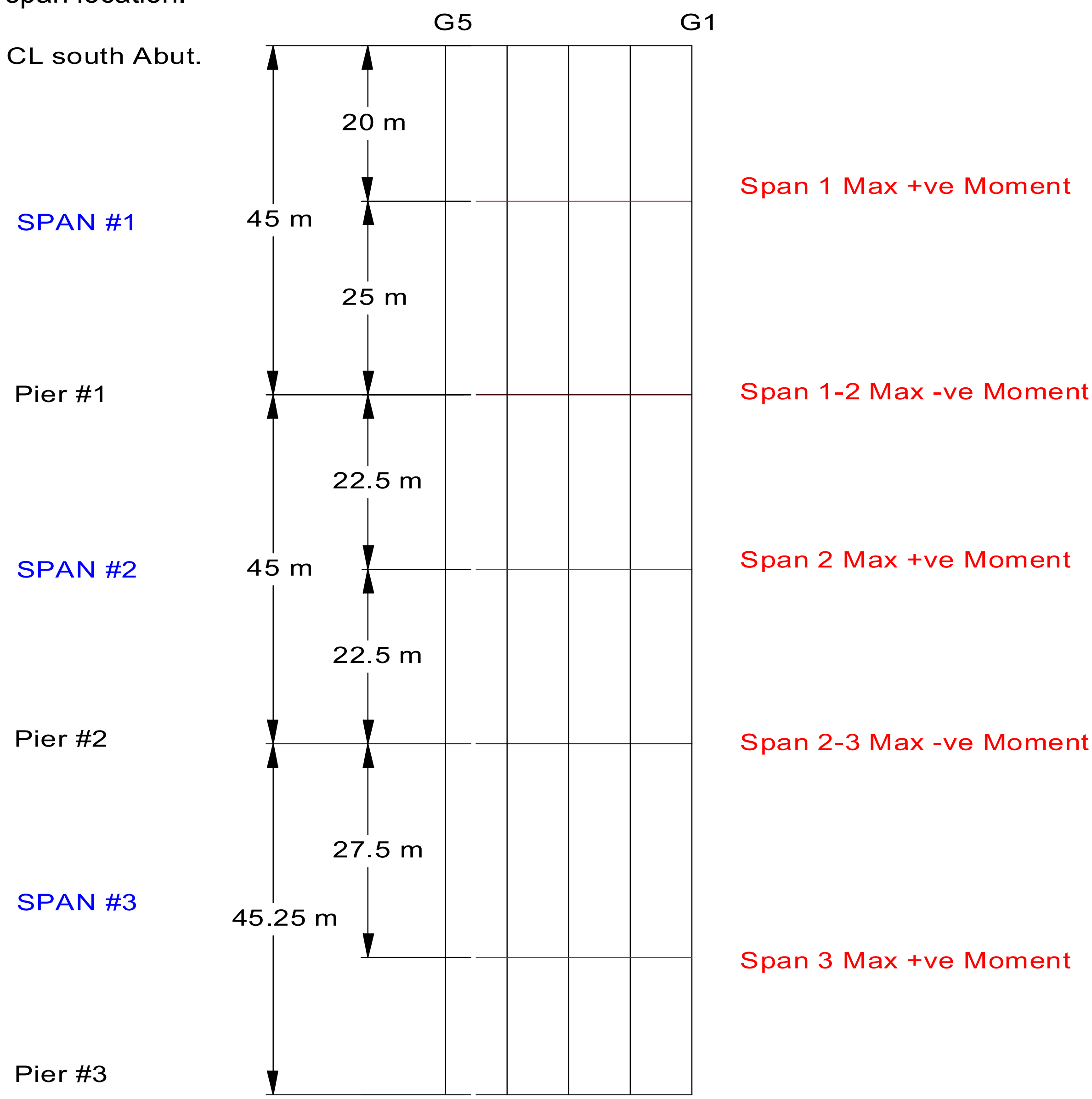
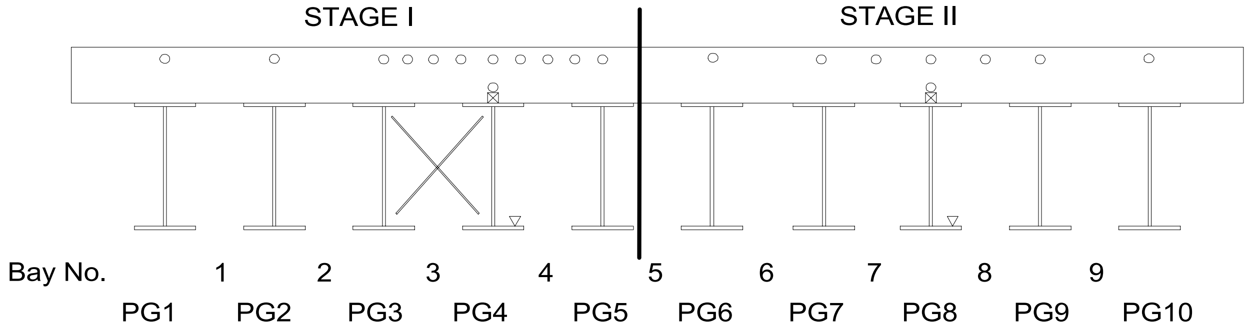
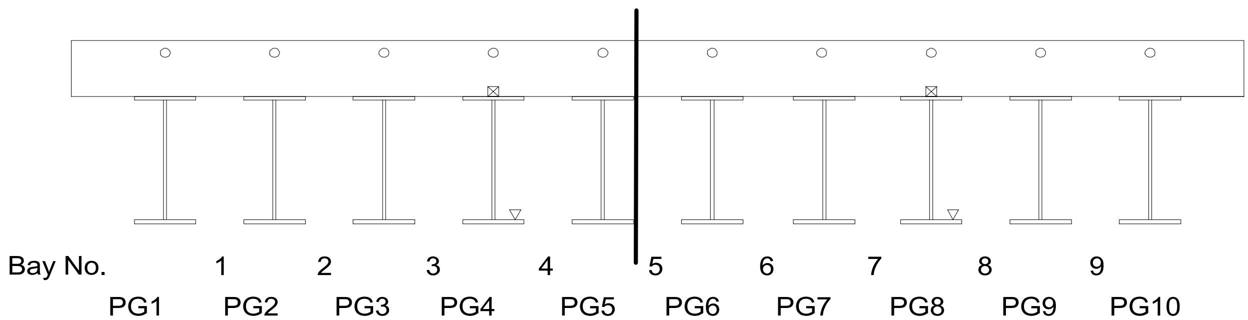


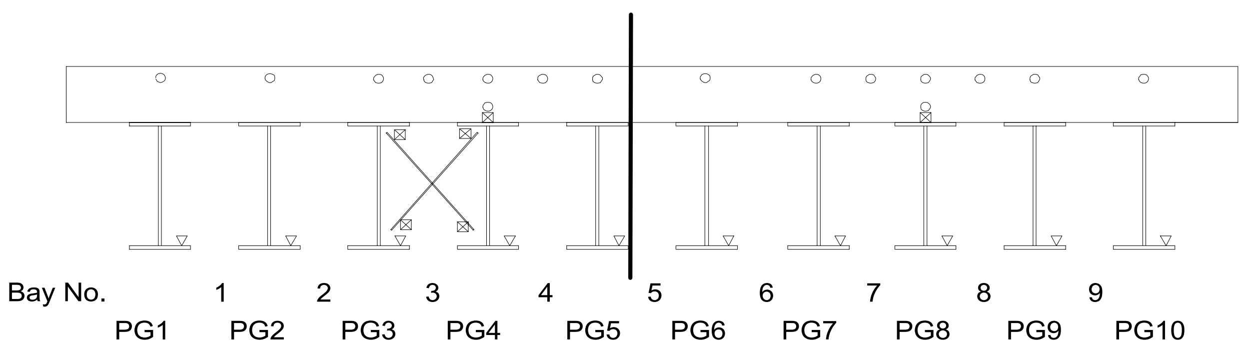
Figure 7. Layout of sensors for each span at positions of maximum moment.



Unit 1, Span 1, Max. +ve Moment



Unit 1, Span 1, Max. -ve Moment



Unit 1, Span 2, Max. +ve Moment

Figure 8. Layout of sensor positions in cross section at maximum moment.

Using the FE model described earlier and a simple line analysis of the three-span continuous Doremus Avenue Bridge, the location of maximum moments as well as deflections in each span due to HS-20 truck loading was determined. Tables 1 and 2 show the maximum moment values and their location with respect to the south abutment for each girder included in Stage I construction (i.e. girders 1 through 5), respectively.

Table 1. Location and values of max positive moments (FEM results).

Span No.	Distance From Abutment (m)	G5	G4	G3	G2	G1	Total Moment (kN.m)
1	20.0	816.3	626.5	438	236.5	47.55	2164.85
2	22.5	695.3	532.1	370.2	195.4	33.14	1826.14
3	27.5	793	609.3	426.4	230.2	47.46	2106.36

Table 2. Location and values of max positive moments (line analysis results).

Span No.	Dist from left (m)	M_{DC} (kN.m)	M_{DW} (kN.m)	+ve $M_{(LL+I)}$ (kN.m)
1	18.00	3418.1	678.9	3313.5
2	22.50	620.0	218.4	2214.8
3	27.00	3418.1	678.9	3313.5

Both analyses yielded close results, therefore the sensors will be placed in locations according to the more accurate FE results. Similarly, Table 3 shows the points of maximum deflection and their location. Table 4 shows a comparison between FE, line analysis, and the original Doremus instrumentation plans. For further illustration, Figure 9 and 10 show the position of the HS-20 truck for the line analysis case and FE, respectively. The results are very comparable, however, it is deemed more accurate to use the FE results in the final recommended instrumentation plans and details to be submitted to the Contractor.

Table 3. LL deflections due to HL-93 loading (line analysis results).

Span #	Max. Deflection (mm)	Location[1] (m)	Truck Loc.[2] (m)
1	21	21	27
2	14	68	73
3	21	115	120

[1] Distance from south abutment to point of maximum deflection
[2] Distance from south abutment to front wheel of truck

Table 4. Comparison of maximum moment location in meters.

Span #	FEM	Line Analysis	Doremus Plans
Span 1	20.0	21.0	15.00*
Span 2	22.5	23.0	22.5
Span 3	27.5	25.0	27.55

* Editorial error; should be 18 m

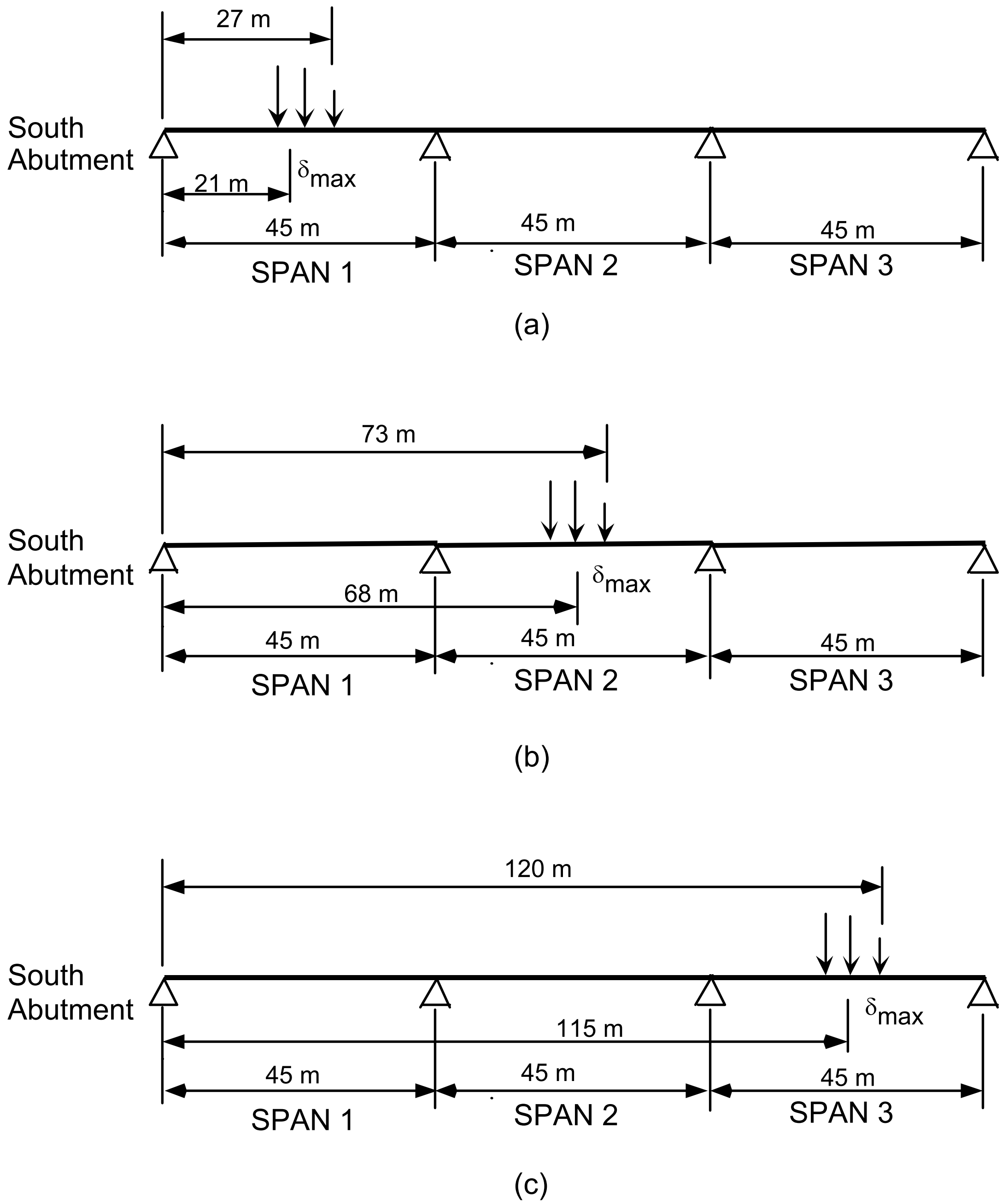


Figure 9. Position of maximum positive moment for (a) span 1, (b) span 2, and (c) span 3.

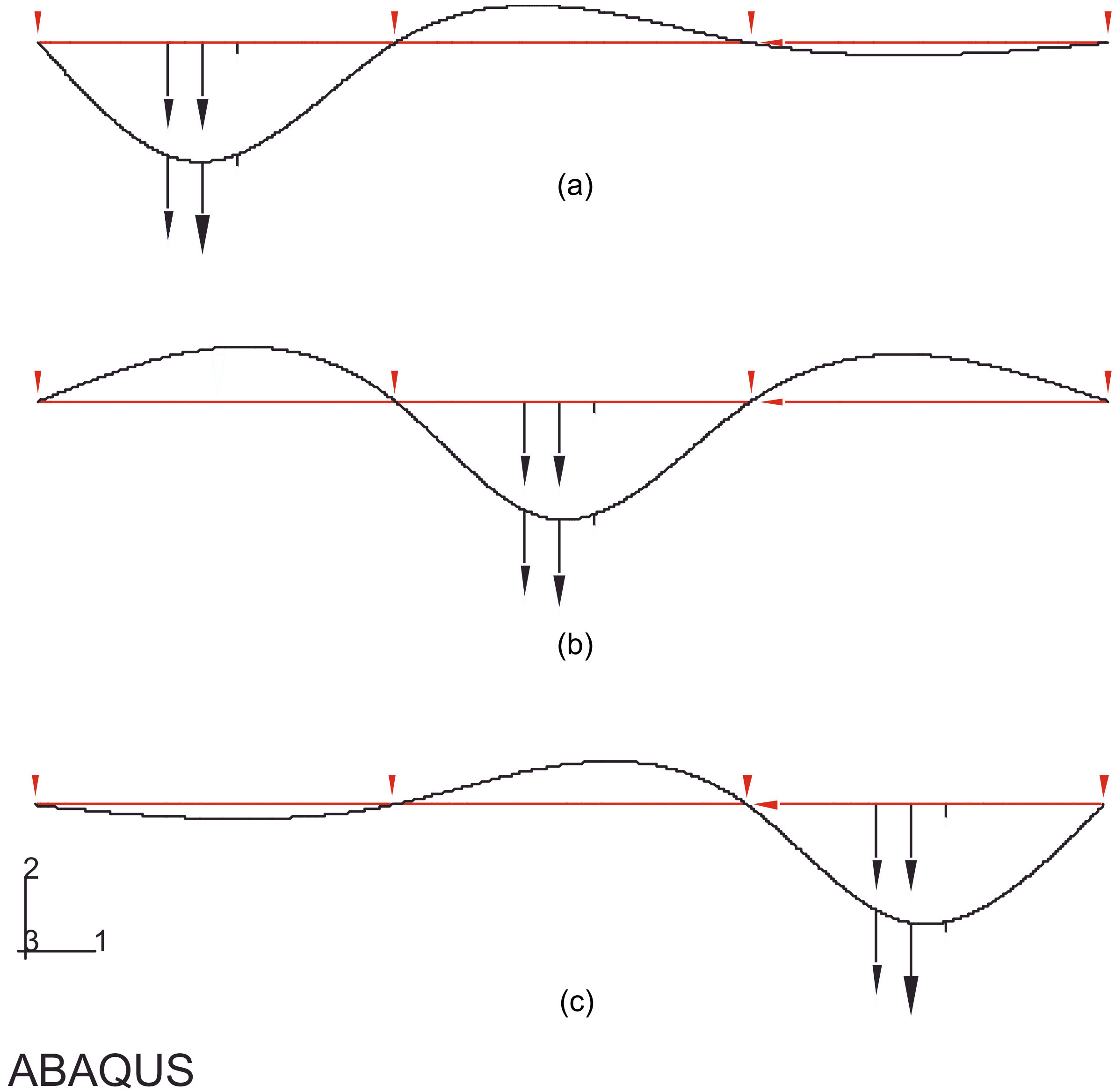


Figure 10. Deflection of bridge under HS-20 truck loading at the three spans.

BRIDGE SUBSTRUCTURE

The following tasks are related to the substructure evaluation and monitoring:

1. Soil Dynamic Properties

2. Site Response Analysis: The primary objective of this task will be to prepare all the necessary elements to conduct the site response study. For this purpose a model of the site, based on the stratigraphy given in the available boring logs, will be prepared. Appropriate accelerograms describing hypothetical distant, medium distant, and near earthquakes will be collected. The accelerograms will be used in the preliminary evaluation of the site response based on the hypothetical dynamic soil properties. In addition to the evaluation of predominant response frequencies, natural frequencies of the site will be evaluated, using the program ProShake and other similar programs. The same analysis will be repeated in the second year of the project, once the dynamic soil properties are evaluated from the field and laboratory measurements.

3. Caisson Impedance Study: Prior to the field evaluation of the drilled shaft dynamic stiffness (impedances), drilled shafts will be modeled and for evaluated hypothetical soil properties impedances using the PILAY2 program. Similar to the site response analysis, the analysis will be repeated in the second year using actual soil properties. Also, procedures for the calibration of the numerical model will be prepared. The results of the analysis will also be used as input in the complete finite element model of the soil-substructure-superstructure model.

4. Field and Laboratory Evaluation of Dynamic Soil Properties: Detailed plans for the field and laboratory evaluation of dynamic soil properties will be prepared. In the case of tests where equipment is already available, preliminary testing will be conducted according to schemes to be implemented at the Doremus bridge site. Rutgers will use some of its equipment for surface soil testing and obtaining soil dynamic properties (e.g. SASW tests, etc.)

In the first phase of the project the main objectives were:

- Site characterization with respect to dynamic soil properties
- Preparation for the substructure instrumentation
- Preparation for the drilled shaft testing

Site Characterization with Respect to Dynamic Soil Properties

Dynamic soil properties are needed to conduct a site response analysis and any kind of soil-structure interaction analysis. To obtain dynamic soil properties, such as shear module and damping, seismic tests like the crosshole can be utilized. The crosshole test was conducted at five locations next to the future bridge foundations. Results of the crosshole test include the shear wave velocity (shear modulus) profiles for each location. Since the shear wave velocity is proportional to the shear modulus of the soil, simultaneously, the shear modulus at each location becomes known.

Preparation for the Substructure Instrumentation

The instrumentation plan included instrumentation of the drilled shaft. The location of the instrumented shaft was selected once the soil profile was obtained and the location of the other superstructure instrumentation was defined. It was decided that one drilled shaft at Pier 2 would be fully instrumented with five triaxial geophones. The geophones will be arranged at various depths so that each is placed in a characteristic soil layer. Three more geophones at Pier 2 will be placed on the piers and the pier cap. The geophones will be used for the future monitoring of the bridge response to dynamic loads and for a comparison with numerical models.

Preparations for the Drilled Shaft Impedance Testing

The drilled shaft impedance will be evaluated by introducing harmonic loading at the top of the shaft. Harmonic vibrations will be introduced using an electromagnetic shaker. The work will concentrate on the measurement of the dynamic response of the drilled shaft. The main objective of the shaft testing is to obtain dynamic stiffnesses (impedances) of the shaft. Results will be used to calibrate numerical models. The response of the tested as well as the response of the adjacent shafts will be measured for the purpose of evaluating the shaft interaction. All the equipment used to conduct the testing will be prepared and tested for operation in the lab.

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 the elastic waves are generated, using the appropriate equipment, their velocities are measured. Seismic methods applied in geotechnical engineering are useful for determining soil properties such as the velocity of wave propagation, Young's modulus, shear modulus, and Poisson's ratio. These soil properties are necessary in many situations such as the analysis of foundations, evaluation of the response of the site to earthquakes, and evaluation of the results of such soil improvement on dynamic compaction and grouting.

Once the velocity profiles are known, they can be related to the shear modulus and elastic modulus of the soil using the following relationships:

$$V_s = \sqrt{\frac{G}{\rho}} \quad \text{and} \quad V_c = \sqrt{\frac{E}{\rho}}$$

Where
V_s is shear wave velocity,
V_s is compression wave velocity,
G is shear modulus,
E is elastic modulus, and
ρ is mass density

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

The fundamental assumptions of the crosshole test are as follows:

- 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 therefore be conducted according to ASTM Standard Designation: D 4428 / D 4428M – 91[4].

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

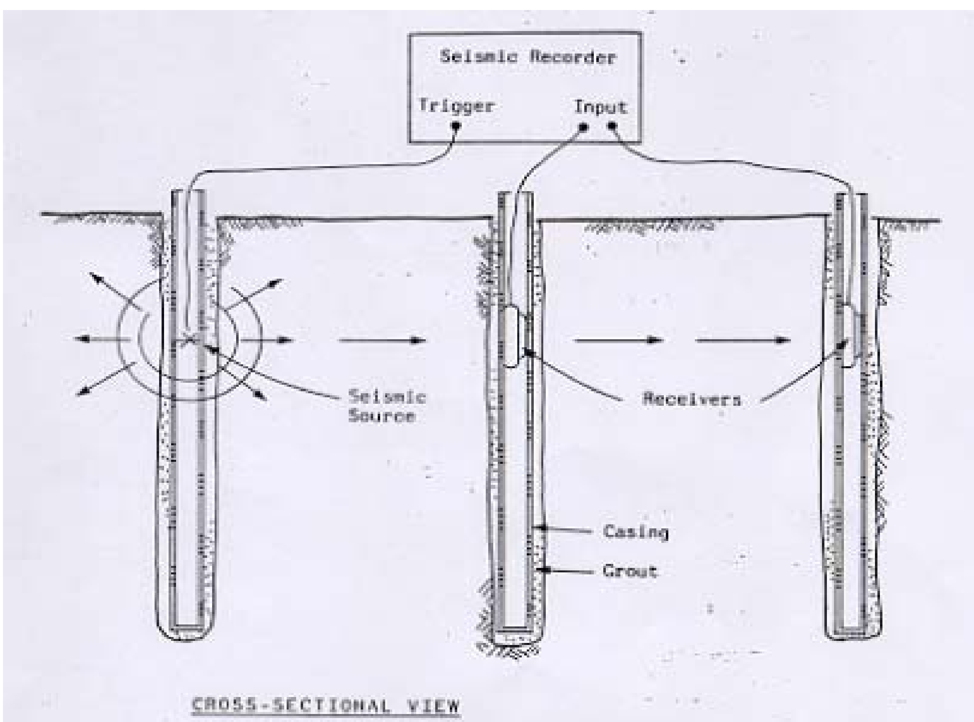


Figure 11. Schematic of the test [ASTM standard[4].

Fundamentals of the Crosshole Method

As already specified, 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 the PVC or metal casings and soil should be well grouted in-place using cement-bentonite non-shrinking grout. The grout should have approximately the same unit weight as the surrounding soil.

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 the appropriate frequency and sensitivity characteristics to determine the seismic wave train arrival. Typical receivers used in crosshole testing include geophones and accelerometers. Receivers shall be placed in the boreholes so that 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 conducted 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 with 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.

Data Reduction and Interpretation

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

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

Description of the Crosshole Test at the Doremus Avenue Bridge

Borehole Installation

The crosshole test at the Doremus Avenue Bridge was performed in five locations: pier 1, pier 2, pier 4, pier 5, and pier 8. 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 Table 5.

Table 5. 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

Samples from all distinctive layers were recovered during the borehole installation using Shelby tubes. Besides Shelby tubes the samples were taken with 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 a height of 24 in. The grouting was executed 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.

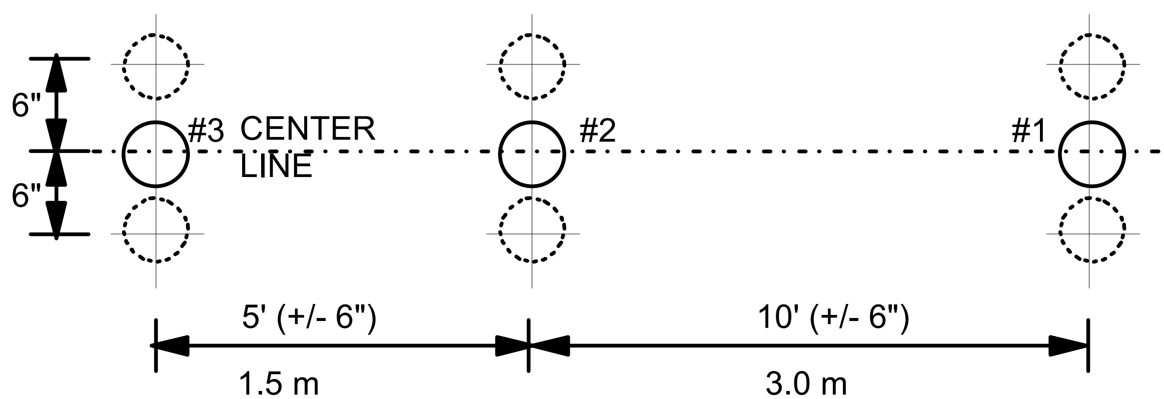


Figure 12. Plan view of cased boreholes for crosshole seismic testing.

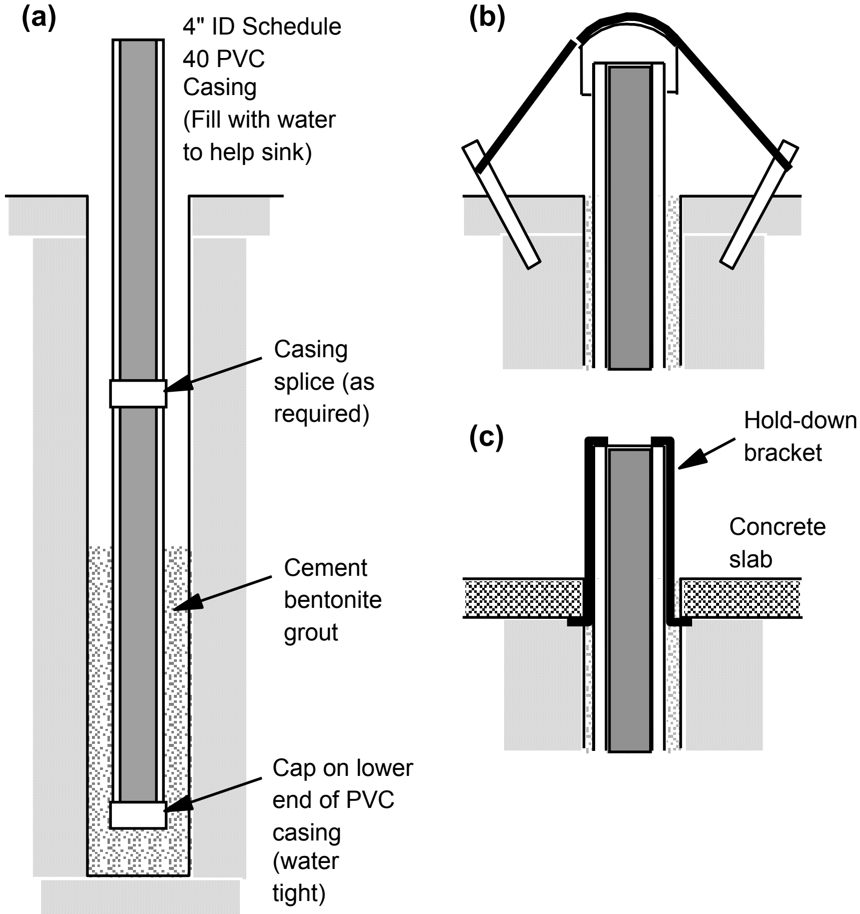


Figure 13. Installation of casing for crosshole seismic testing: (a) installation of the casing; (b) and (c) holding of the casing while grouting.

Equipment Used for Crosshole Testing

A shear type in-hole hammer was the source used 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. Geophones have rubber membranes that can be expanded by compressed air in order to keep them fixed 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) as shown in Figures 12 and 13.

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. Figures 14 to 16 show the equipment set-up used in the crosshole testing.

Figure 14. Recording system used for crosshole testing at Doremus Avenue Bridge.

Figure 15. Crosshole test at the Doremus Avenue Bridge.

Figure 16. Placing of the hammer in the borehole.

Borehole Verticality Check

A verticality check was conducted using an inclinometer probe to obtain an accurate distance between the receivers. An inclinometer system consists of a casing, a probe with a cable, and a read out unit. The inclinometer probe measures the tilt of the casing. The tilt is used to calculate a lateral distance. In the first step an incremental deviation is taken for an increment of the casing from the tilt angle. In the second step the sum of incremental deviations is used to arrive at the cumulative deviation. Readings displayed by the inclinometer reading unit are proportional to the angle of tilt. Figure 17 illustrates a similar inclinometer probe used in the testing set-up.

The readings were taken in 2 ft (0.61 m) increments for all boreholes and in two perpendicular directions to obtain the spatial positions of boreholes. From these spatial positions, the distance between the receiver boreholes was calculated.

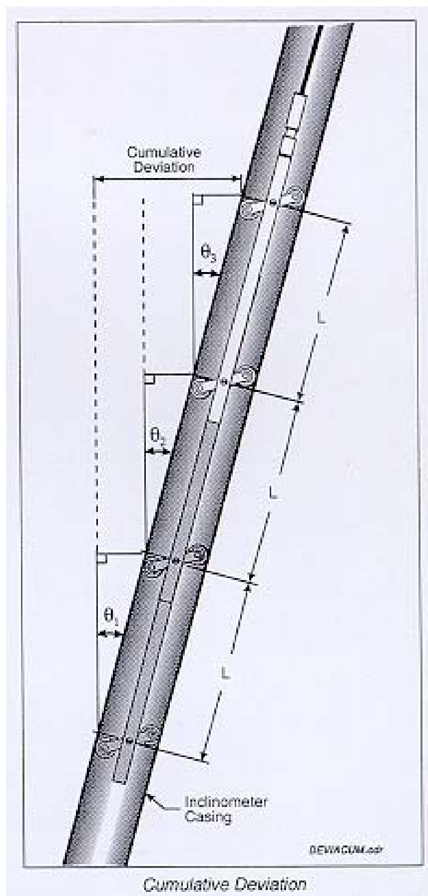


Figure 17. Inclinometer probe (digital data mate & DMM Software).

Results from the Crosshole Test

The signal time histories were recorded in 3 ft (0.91 m) increments. Typical wave time-histories are shown in Figure 18.

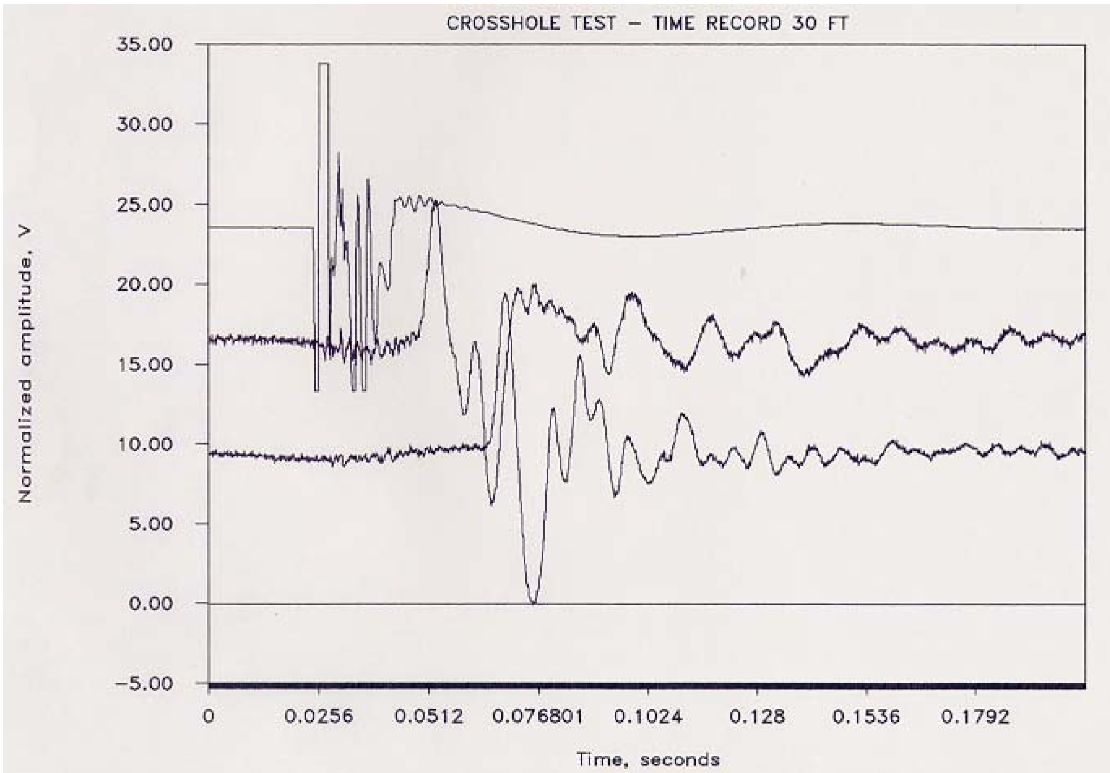


Figure 18. Signals recorded (hammer top, receiver 1 middle, receiver 2 bottom).

The time difference between the shear wave arrivals was then determined. Since the distance between boreholes was known, the shear wave velocity V_s was calculated as:

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

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

A shear wave velocity profile for each testing location resulted from data reduction. 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 mentioned previously in the text.

The shear wave velocity profiles for all test locations are given in Figures 20 to 24. They were used to obtain the shear wave velocity profile in the longitudinal direction of the Doremus Avenue Bridge. The velocities in the longitudinal direction were obtained by interpolating the shear wave velocities between the test locations as shown in Figure 19. Properties of the soil layers at the location of the Doremus Avenue Bridge are displayed in Table 6.

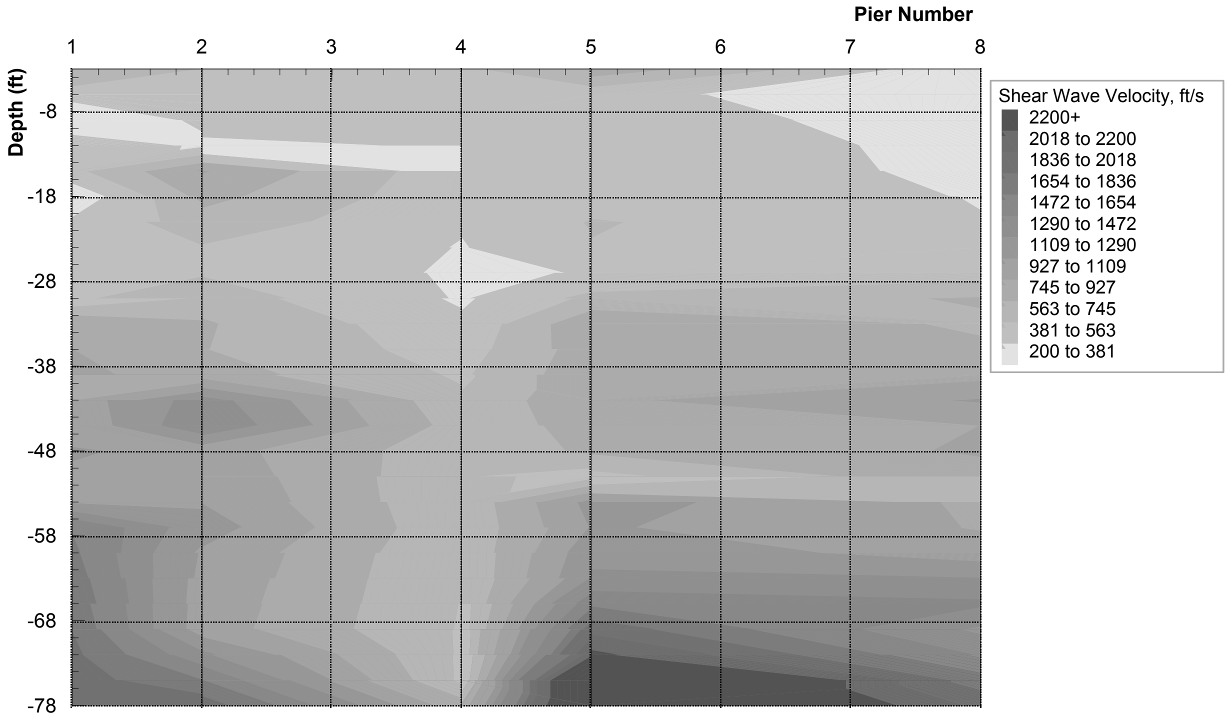


Figure 19. Longitudinal shear wave velocity profile.

Table 6. Soil properties at the location of the Doremus Avenue Bridge.

Type of Soil	Unit weight γ (kN/m^3)	Angle of Internal Friction ϕ (deg)
Fill	19.0	30.35
Silt	11.8 (saturated)	-
Sand	19.6 (saturated)	35
Silt and Sand	18.9 (saturated)	-

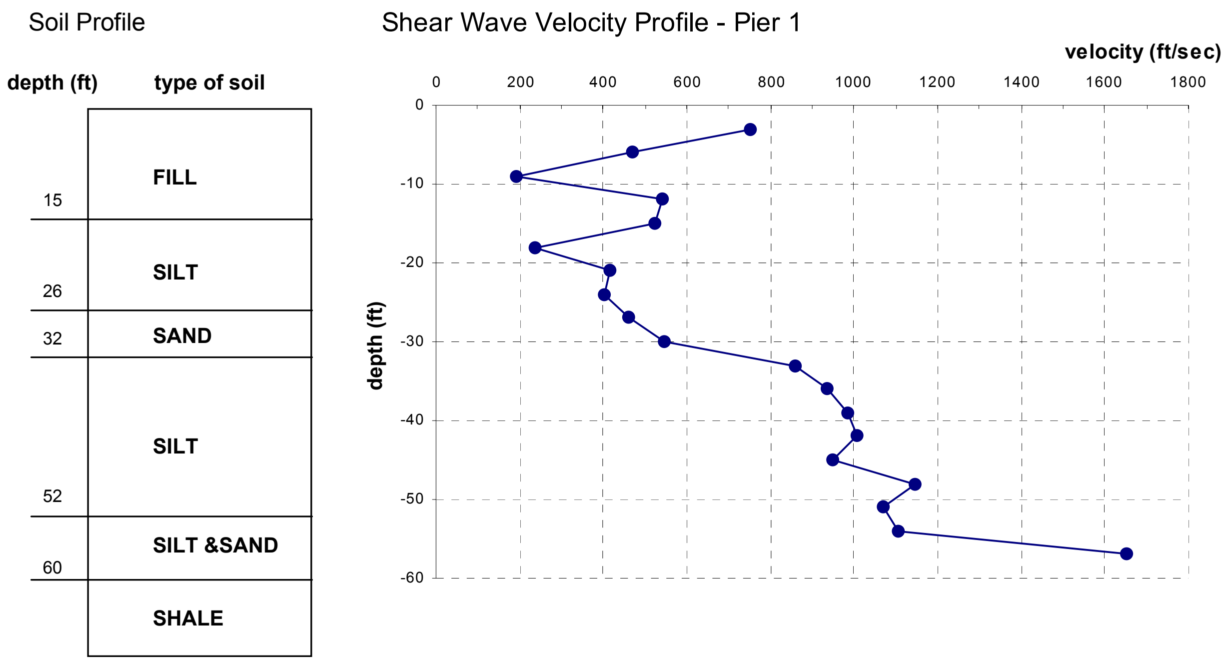


Figure 20. Shear wave velocity profile at pier 1.

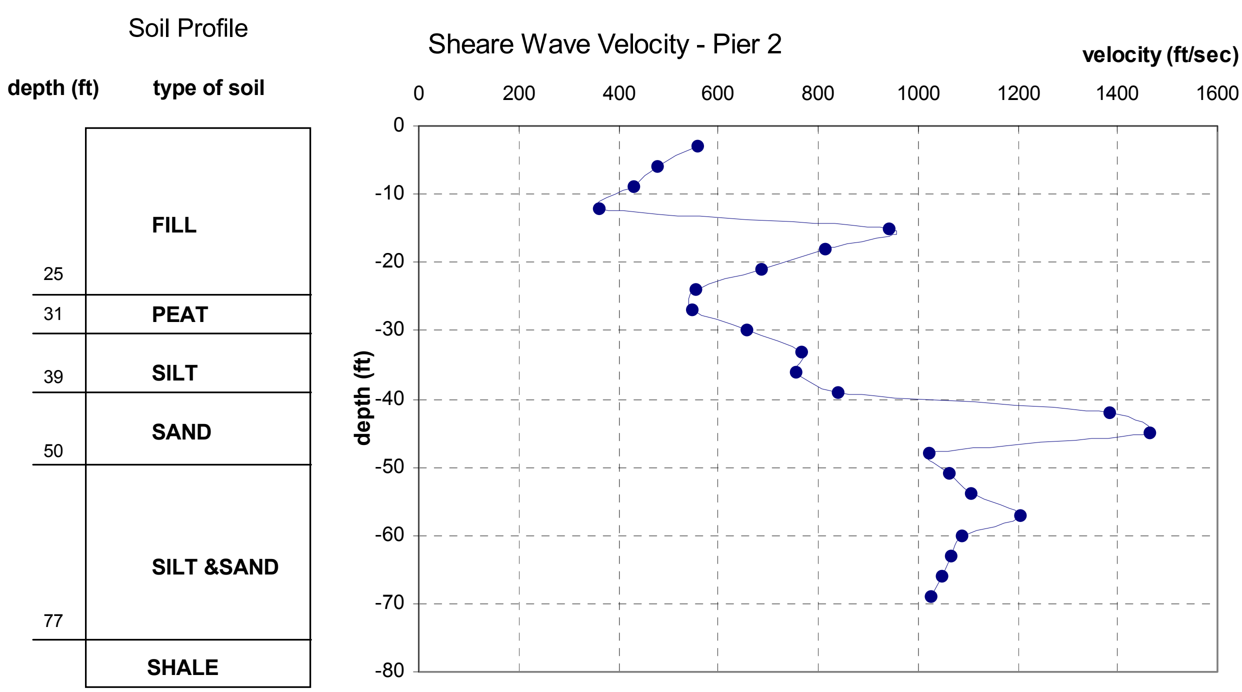


Figure 21. Shear wave velocity profile at pier 2.

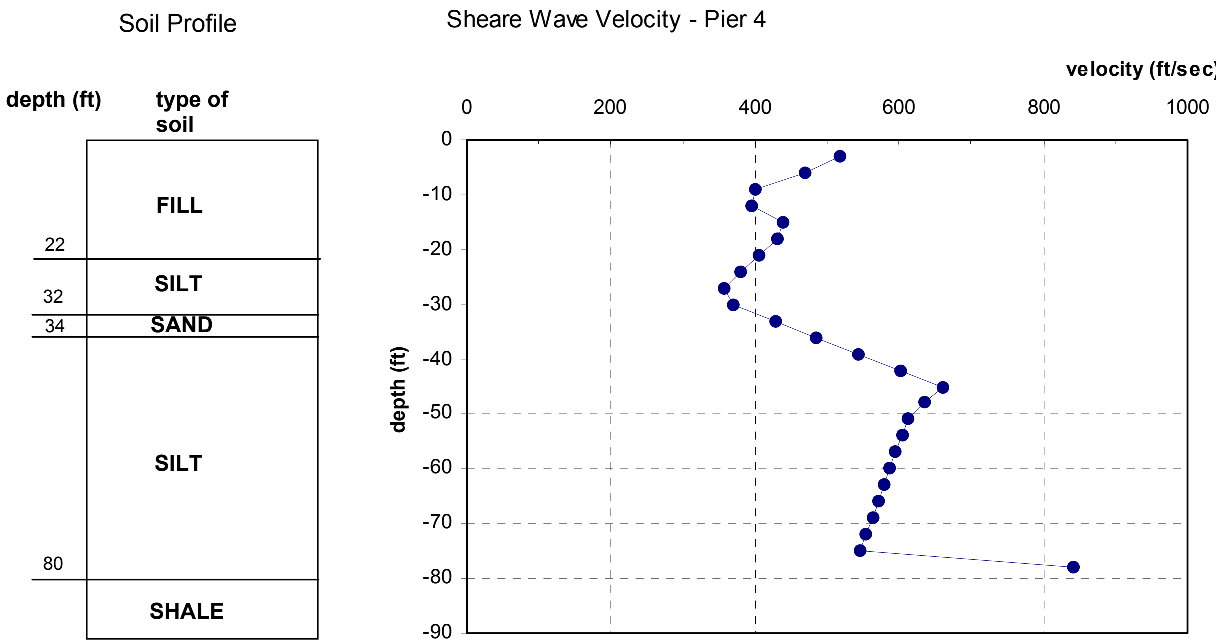


Figure 22. Shear wave velocity profile at pier 4.

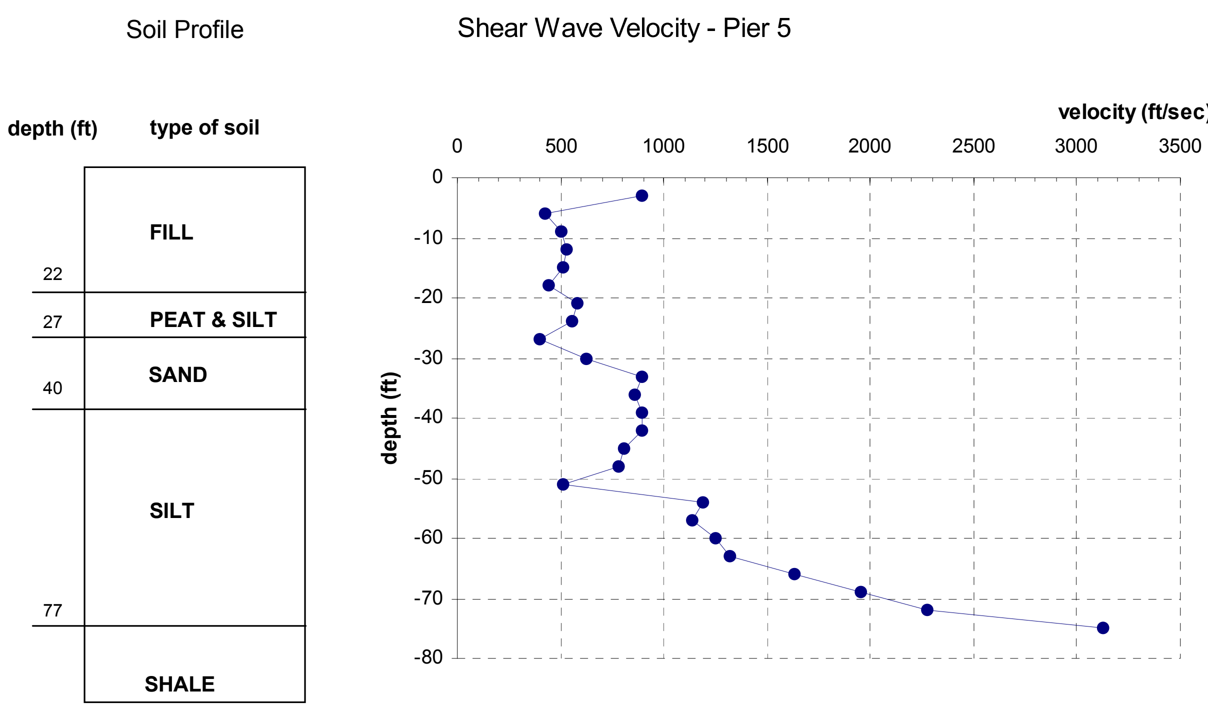


Figure 23. Shear wave velocity profile at pier 5.

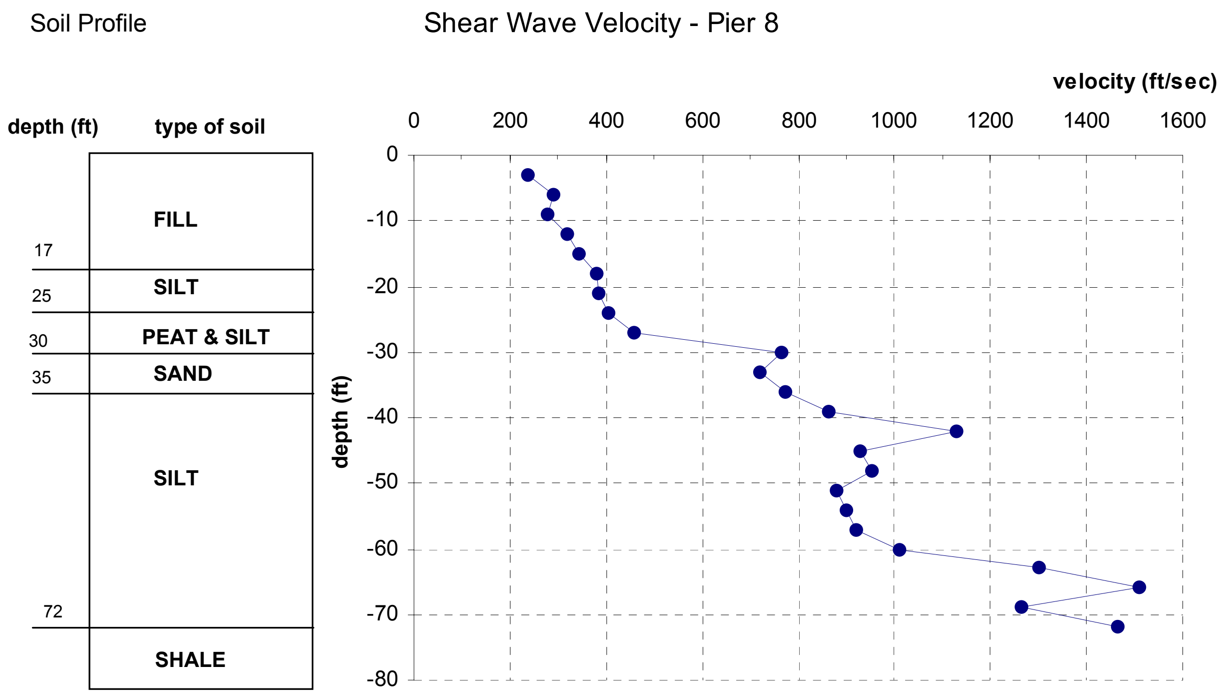


Figure 24. Shear wave velocity profile at pier 8.

Drilled Shaft Impedance Testing

Pile foundations are deep foundations that are used when the upper soil layer is not able to provide good support for the superstructure. The pile foundations are typically placed in soft soil of low shear strength and stiffness, which tends to be troublesome under dynamic loading, such as that of an earthquake.

The pile foundations are usually placed in a group. The dynamic response of the pile group is even more complicated than the response of the single pile. That is because of the influence of one pile on other piles in the group. If the spacing between them is large, then each pile can be analyzed as a single pile. But if they are closely spaced then the effect of the interaction should not be neglected.

In the past two decades this problem has been investigated by numerous authors(14,19,36,37). This area is still of great interest in research, because there are not many guidelines on how to design pile foundations subjected to a dynamic load.

Single Pile

The stiffness and damping of a pile are affected by its interaction with surrounding soil. This interaction is considered in terms of continuum mechanics and takes into account elastic wave propagation. For a single pile the solution is conducted using a continuum approach or the finite element method. From such studies it can be concluded that a soil-pile interaction modifies the pile stiffness and damping, making these frequency dependent.

Pile Stiffness and Damping

Dynamic stiffness and damping of pile can be described in terms of a complex stiffness usually called the impedance function:

$$K = K_1 + iK_2$$

Where K_1 and K_2 are real and imaginary parts, respectively. The real part represents the stiffness and defines the stiffness constant of the pile.

$$k = K_1 = Re(K)$$

The imaginary part of the complex stiffness represents damping due to energy dissipation in the soil and pile. It can be defined in terms of a constant of the equivalent viscous damping.

$$c = \frac{K_2}{\omega} = \frac{Im(K)}{\omega}$$

Similarly, complex stiffness can be written as:

$$K = k + i\,\omega c$$

The complex stiffness K, or the constants k and c, can be obtained experimentally or theoretically. In the theoretical approach, dynamic stiffness is obtained by calculating a force needed to produce unit harmonic oscillation at the pile head in a prescribed direction.

In general, the impedance depends on the following factors:

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

where
E_p is Young's modulus of elasticity of the pile material,
R is pile radius,
G_s is shear modulus of soil,
V_s shear wave velocity,
ω circular frequency, and
L pile length

Pile Groups

The dynamic stiffness of a pile group in any mode of vibration can be computed by adding the stiffnesses of the individual piles. This can be accomplished only if the pile spacing is so large that the pile interaction has negligible effects. Each pile is affected by its own load and by the load and deflection of its neighboring piles. This pile-to-pile interaction is frequency dependent. Results from waves are emitted from the periphery of each pile and propagate towards neighboring piles.

In many cases where piles are closely placed, the displacement of one pile increases due to that of the surrounding piles. Therefore, the stiffness and damping of a pile group are reduced.

Substructure - Superstructure Analysis

To describe the behavior of a pile group under dynamic loading, soil-pile-structure interaction should be used. The total dynamic stress in the piles can be obtained by the superposition of two independent analyses: kinematic and inertial. Shear waves propagating in the soil interact with the piles and distort them, producing a kinematic bending moment and stresses. On the other hand, the acceleration in the superstructure produces a base shear and an overturning moment, which must be resisted by the foundation.

When the pile vibrates, its stiffness is modified, and damping is generated through the interaction of the pile with the surrounding soil. So far, extensive research has been conducted to obtain stiffness and damping coefficients for pile-soil interaction. Still, there is a need to do experimental work and match its results with the numerical models.

Description of the Proposed Test of Drilled Shafts at Doremus Avenue Bridge

The first year of the project was devoted to preparing the equipment needed for shaft testing. The locations of the tested drilled shafts match those of the crosshole test. At each test location two drilled shafts match the locations of the piers at which the crosshole test was conducted. The two shafts will be tested at each location.

A harmonic excitation will be introduced to the shaft using an electromagnetic shaker. A vibration force will also be adopted as a frequency sweep between 0 and 100 Hz. The shaker signal and amplitude will be controlled by a signal generator and an amplifier. The shaker will be suspended on a frame and connected to the drilled shaft via a steel section. The steel section will be embedded into the shaft and should provide a rigid transfer of the shaker force into the shaft.

A signal analyzer will be used to generate a harmonic signal in order to feed the shaker. The force will be measured using a load cell that will be placed between the arm of the shaker and the steel section embedded in the shaft.

The response of the shaft will be measured using a triaxial geophone, which will be placed on the top surface of the tested shaft. The response of adjacent shafts will also be measured by placing geophones on top of each. That way the interaction between the shafts will be measured.

All the time histories will be recorded using a data acquisition system. The data acquisition system consists of the data acquisition board, signal conditioning, connection panel, and computer with the appropriate data acquisition software. The test setup is diagrammed in Figure 25.

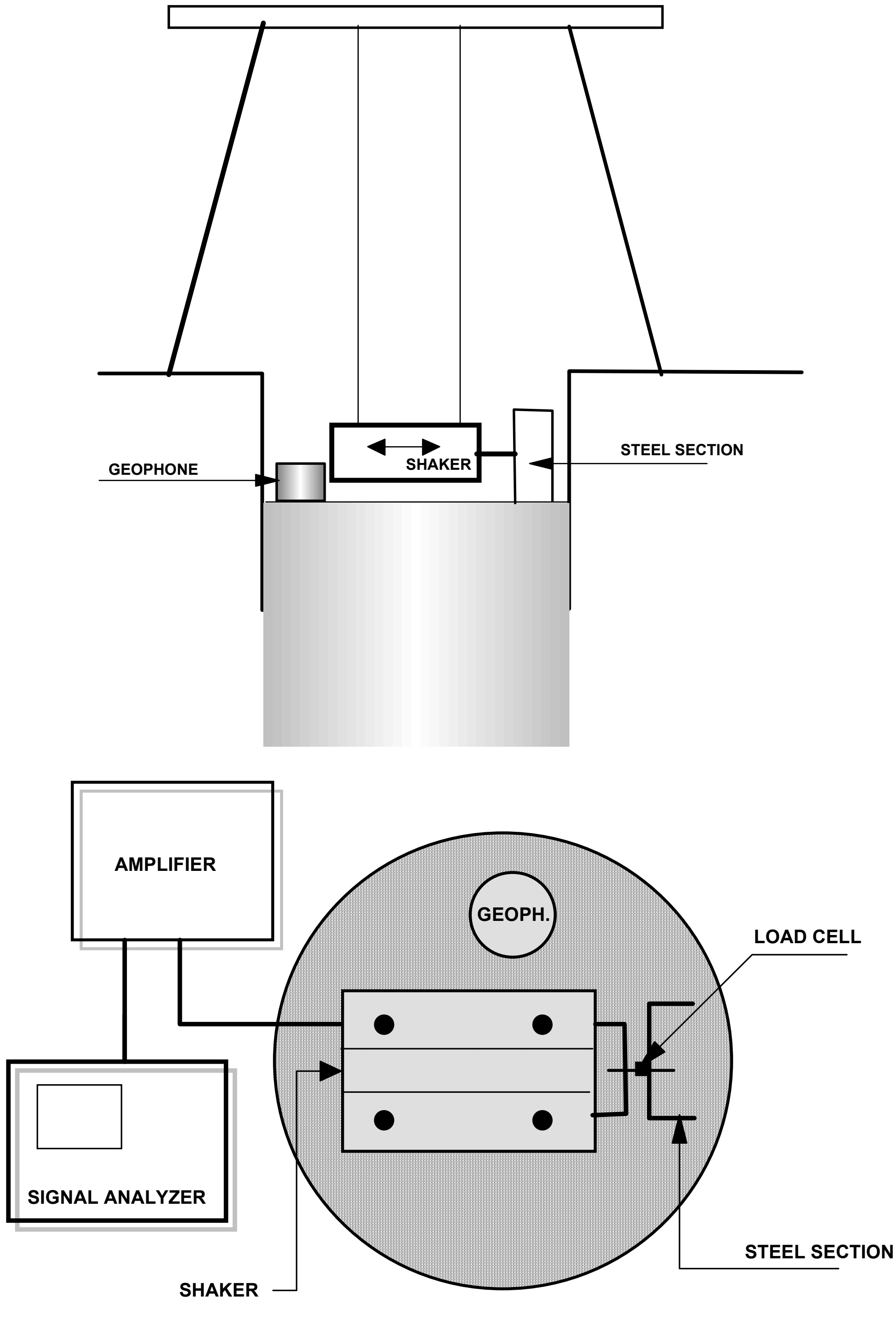


Figure 25. Drilled shaft test setup.

Instrumentation

To get more information about the drilled shaft response, one shaft will be instrumented. The instrumented drilled shaft will be located at pier 2. Five triaxial geophones will be installed for that purpose. The geophones will be arranged according to the depths so that they are placed in a characteristic soil layer. Each of the geophones will be placed in a 4-inch diameter protective PVC casing and fixed to a rebar cage. The cables will be protected by 2-inch diameter PVC pipes.

Three triaxial geophones will be placed at the pier 2 column and cap. The geophone will be strapped to each to ensure an intimate connection between them. The geophones will be weather protected by steel boxes as shown in Figure 26. All the installed geophones will be used later in the vibration monitoring of the bridge.

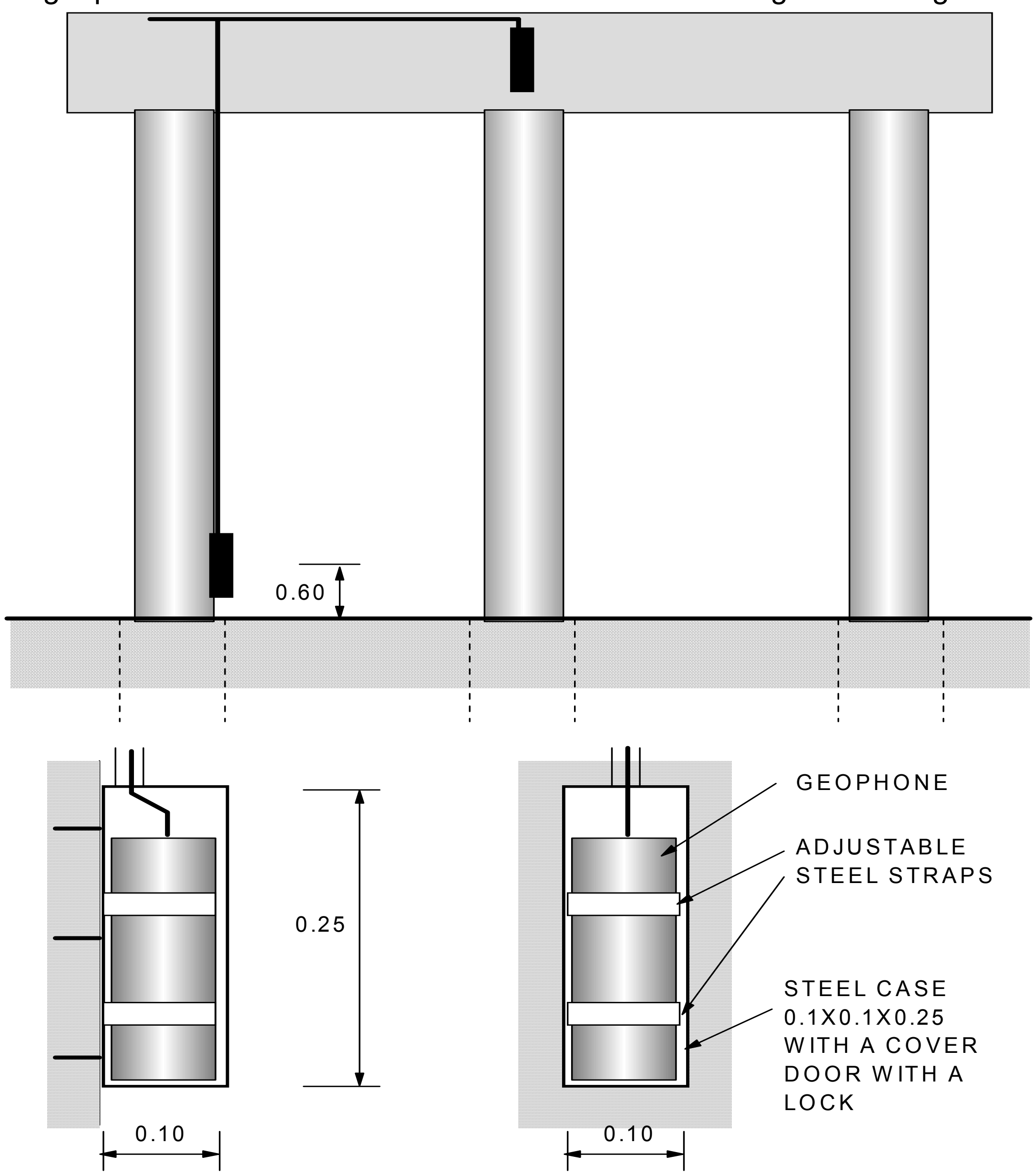


Figure 26. Installation of protective cases for geophones on piers and pier-caps.

RECOMMENDATIONS AND FUTURE RESEARCH

The succesful implementation of the instrumentation plan will depend on the coordination of tasks with the Contractor. A pre-construction meeting was held to coordinate the various tasks. The Rutgers University research team has provided all the instrumentation shscemes to the contractor who will be responsible for the installation of cables, sensors, and testing equipment.

Superstructure Future Tasks

The following tasks will be implemented in coordination with the NJDOT resident Engineer, Consulting Engineer on Site, and Contractor:

1. Install vibrating wire strain transducers (VWST) in the concrete deck for spans 1, 2, and 3, respectively. There will be three types of VWST: 1) sister rebar for hard to reach places, 2) weldable gages for the top of the steel girder, and 3) regular VWSG placed at selected locations between rebars.
2. Install girder transducers to be used with the portable live load testing as well as dynamic strain measurements.
3. Install accelerometers and goephones for dynamic vibration measurements and to predict deflections of the girder as well as mode shapes and vibrations.
4. Develop and install a LVDT-Cable system to measure deflections of at least one girder in each stage. The LVDT-Cable system shall be accurate in detecting deflection to within $\pm$ 1/100 of an inch. Verify the deflection measurement from the LVDT-Cable system with a non-contact Laser system.
5. Install and run all cables needed to contact sensors to data acquisition systems placed in steel cabinets and located at Pier 2.
6. Install and operate a permanent bending plate weigh-in-motion system at the south abutment in Stage I and Stage II. The WIM system will be used to weigh trucks at highway speeds with an accuracy of $\pm$ 5% of gross vehicle weight.
7. Install a fatigue data logger system capable of recording peak stress ranges and rain flow analysis.
8. Perform calibration static testing using truck of known axle weights and configuration. Also perform dynamic tests using the same calibration trucks at speeds close to normal highway speeds.
9. Install a telephone line and establish the remote data collection routine and software needed to download data from various data acquisition systems.
10. Install electric power and/or solar panels, if needed.
11. Collect concrete samples from the deck slab to determine its mechanical properties, which will be used in the FE model.
12. Perform a comparison between analytical and experimental results.
13. Perform long-term monitoring for deflections, fatigue, and durability of the bridge structural elements. Compare results with the AASHTO-LRFD code provisions(2).

Substructure Future Tasks

The next task in the substructure testing at the Doremus Avenue Bridge is to conduct the described shaft test. The results of the test will consist of the time histories for loading and response. From those data, the shaft impedance will be obtained and matched by theoretical models of the dynamic response of the drilled shaft. A numerical model of the drilled shafts will be developed using the FEM and other software like Pilay 2. Shaft impedances will be used for the evaluation of soil-structure interaction effects on the dynamic response of the bridge due to vehicular and earthquake loading.

APPENDIX A

EQUIPMENT SPECIFICATION FOR INSTRUMENTATION, MONITORING, AND LRFD EVALUATION OF THE PROPOSED DOREMUS AVENUE BRIDGE

EQUIPMENT SPECIFICATIONS

SECTION 524 - LOAD AND RESISTANCE FACTOR DESIGN (LRFD) MONITORING

524.01 Description.
This section shall consist of work that is required of the Contractor to facilitate instrumentation, monitoring and evaluation of the new Doremus Avenue bridge superstructure and substructure by personnel from Rutgers University for the purpose of reviewing the AASHTO Load and Resistance Factored Design (LRFD) procedure. The work involved is for research purposes only and not to be used to monitor construction of any item. The Contractor shall coordinate his operations to allow access of Rutgers personnel to the site.

If requested by the Contractor, Rutgers University shall provide a certificate or certificates of insurance showing that insurance coverage for its personnel is provided. At a minimum, coverage, as stated in Subsection 107.23, Subparts 1, 2, 3 and 4 shall be provided.

The work provided by the Contractor shall include but not be limited to furnishing labor, transportation, equipment, materials, and incidentals necessary for installing and maintaining instrumentation required to instrument and monitor short as well as long term effects of truck on the Doremus Avenue Bridge prior and during construction operations. No instrumentation installation shall take place before review and acceptance of instrumentation working drawings by the Engineer and Rutgers University Coordinator.

Responsibilities of Contractor shall include, but not be limited to, the following:

1. Contractor is responsible for purchasing the instrumentation and testing equipment required for the monitoring of the new structure as described in the technical specifications and as shown on the plans.
2. The contractor shall procure and acquire all testing equipment and material needed for all phases of the LRFD Monitoring with sufficient lead-time to allow Rutgers to perform early testing procedures. The Contractor is alerted to the fact that certain equipment/material are proprietary, sole source, and that long lead times may be required for site delivery and shall be reflected in the construction schedule. No claims for delay or increased cost for any items contained herein will be granted due to the Contractor’s failure to properly maintain the schedule.
3. The Contractor has the full responsibility for developing and progressing the construction schedule. The Contractor shall take into consideration all the required activity to enable the NJDOT and Rutgers University to perform the LRFD Monitoring. All construction’s labor required in accordance with specifications shall be included in the various pay items contained herein the supplementary specifications.
4. Furnish components of instrumentation equipment that are to be installed during construction.

5. Prepare and submit working drawing for all testing equipment to be purchased in accordance with the supplemental specifications. The working drawings shall include the layout and installation of the testing support brackets, junction boxes, control cabinets, cross hole locations and material specifications of the gages, computers, cables, piping, clamps, etc. The Contractor shall work closely with the Rutgers Coordinator to ensure that the specific testing equipment and materials are obtained early in the project.
6. During construction protect instrumentation from damage and maintain instruments installed. Repair or replace all damaged or inoperative instruments, within a reasonable time as determined by the Engineer.
7. Provide access to the construction site for Rutgers staff for installation of testing equipment and data collection.
8. Furnish on-site separate office space for Rutgers personnel and for housing instrumentation equipment such computers, printers, fax machine, phone, electric, air conditioning units and heating units. This size of the office space shall be sufficient for two Rutgers personnel with desks, tables, and chairs. The desks, tables, chairs, phone lines, heating and air condition units shall be supplied by the Contractor.
9. Purchase an instrumentation van to house the portable equipment, data acquisition, electric generator, ladders, toolboxes, etc. Specifications of Van size have been defined under sub section 524.03 Testing Systems, Support Equipment.
10. Furnish a minimum of two phone lines and appropriate phone jacks to a predetermined location of the steel enclosure as shown on the plans that houses the data acquisition system. The phone lines are needed to allow for remote monitoring of the bridge instrumentation.
11. Furnish a constant electric power source of 110VAC to two locations as shown on the plans. The Contractor shall also provide a reliable additional source of power such as solar power, a battery, and a recharge system to ensure continuous power supply. The backup power supplies to be purchased by the Contractor have been detailed with sub section 524.03 Testing Systems, Support Equipment.
12. The Contractor shall be responsible for any defective or stolen equipment and the Contractor shall replace in a timely manner that does not affect the schedule of instrumentation and to be determined by the Engineer. All equipment replacement shall be coordinated with Rutgers and NJDOT.
13. The Contractor shall notify Rutgers in writing as to when Rutgers can start their instrumentation schedule providing a reasonable time period.
14. The Contractor shall provide Rutgers with testing and calibration Trucks and written records of their axle loads and spacing, gross vehicle weight, suspension type (e.g., multi-leaf or air), number of axles, truck type (e.g., flat bed or semi-trailer), etc.
15. The Contractor will be responsible for the purchasing of all equipment including sensors, transducers, gages, wiring, and data acquisitions and processing system and other incidentals needed to collect the data. Prior to purchasing the equipment, The Contractor shall submit working drawings, material certification and specifications for the equipment to be purchased to the Engineer and the Rutgers Coordinator for review and approval. The above information, working drawings, shall be submitted within one month from the date of the Notice to Proceed is

received by the Contractor. It is essential for Rutgers staff to receive the testing equipment within three months from the Notice to Proceed by the Contractor. To provide sufficient time for them to prepare equipment for installation.

16. The Contractor shall be responsible for contacting the manufacturer of various equipment and to ensure the manufacturer participation and supervision of equipment installation and to provide warranties for the equipment used during the duration of the project and for two years beyond the conclusion of all works.
17. The Contractor shall be responsible for providing a separate trailer for storage of testing equipment and provide access to this trailer by Rutgers personnel and supervising Engineer only. Moreover, the Contractor is to provide security for all facilities used by Rutgers such as office, equipment storage trailer, and all sensors and equipment installed on bridge during the project duration. The storage trailer shall be no smaller than 2.5 m x 3 m x 4.5 m.
18. The Contractor shall coordinate with the railroad the need for flagman during the installation and removal of testing equipment and during the actual testing of the bridge.

Description of Testing Systems:

1. Bridge Structural Testing System (Portable)
2. Long Term Monitoring System (Permanent)
3. Live load and Dynamic Testing system
4. Deflection Measurements
5. Weigh-in-Motion (WIM) Bending Plate System
6. Substructure Testing System

Definitions:

1. Instrumentation monitoring is the reading of installed instruments at defined time intervals and calculating stresses, changes from initial stresses, lateral displacements, vertical deflections; recording and plotting all instrument readings.
2. Survey control consists of precise field measurements as specified herein, taken by qualified personnel using approved methods and equipment for accurately determining elevation, coordinates, and distances essential for the prosecution of this Section's work.

For purposes of permitting coordination between the Contractor and Rutgers, the contact for Rutgers University will be as follows:

Dr. Hani Nassif, P.E.
Department of Civil and Environmental Engineering
Rutgers, The State University of New Jersey
623 Bowser Road
Piscataway, NJ 08854-8014
Phone No. 732-445-4414 Fax No. 732-445-0577
E-mail Address: nassif@rci.rutgers.edu

524.02 Equipment.

The Contractor shall procure all monitoring/testing equipment for the instrumentation of the bridge superstructure and substructure. The specific equipment required for each instrumentation scheme is described under each testing system. In addition to the procurement of the monitoring/testing equipment, the Contractor will be obligated to provide labor and equipment such as ladders, man-lifts or bucket trucks and to coordinate with Conrail to obtain flagman when necessary in order to assist in the installation of this testing equipment to the structure. The contractor is required to provide calibration trucks that will be used in the testing and calibration of the equipment. The type of trucks is described in each section. The superstructure will have four different systems described below**:**

Material

- Contractor shall provide products, materials, and equipment in conformance with the Plans and Special Provisions so as to fulfill the requirements of the instrumentation work.
- Whenever any product is specified by brand name and model number, such specifications shall be deemed to be used for the purpose of establishing a standard of quality and facilitating the description of the product desired. The term “or approved equal” shall be understood to indicate that the “approved equal” product is the same or better than the product named in the Specifications in function, performance, reliability, quality, and general configuration. This procedure is not to be construed as eliminating from competition other suitable products of equal quality by other manufacturers. In such cases Contractor may submit complete comparative data to the Engineer for consideration of another product. Substitute products shall not be ordered, delivered to the site, or used in the work unless accepted in writing by Rutgers University and NJDOT. Rutgers University will be the sole judge of the suitability and equivalency of the substituted product.

Any request from Contractor for consideration of a substitution shall clearly state the nature of the deviation from the product specified.

Value Engineering for the LRFD Monitoring will not be accepted.

For each instrument type, working drawings / equipment specifications shall provide an instruction manual that shall include the following:

- A description of the purpose of the instrument.
- Theory of operation.
- Step-by-step procedures.
- Pre-installation acceptance test by manufacturer when instruments are received on site, to ensure the instruments are functioning correctly before installation.
- Calibration of readout units.
- A list of calibration equipment required, and recommended frequency of calibration.

- Step-by-step instrument installation procedure including materials, tools, spare parts and any other requirements, and post-installation acceptance tests.
- Maintenance procedure.
- Step-by-step data collection procedure.
- Data reduction, processing, and plotting procedures.

524.03 Testing Systems

1. Bridge Structural Testing System (STS) (Portable)

This system shall be used in measurements of strains in girders and in calibrating the computer models prior to traffic opening and during construction. The goal of these tests shall be to use the field data to “calibrate” a finite element model of the whole structure. The calibrated model will be used for predicting stresses induced by various rating vehicles and permit loads. The method of testing is to measure strains at many points on the structure as a truck of known load crosses the deck at crawling speed. One important aspect of this testing is that the position of the vehicle is accurately tracked as it crosses the structure via remote control. Furthermore, this crawling speed test can be supplemented with high-speed passes of the truck in order to determine the in-site impact factor. Once the model has been calibrated with the strain data, it is quite accurate at predicting deflections.
The BDI Structural Testing System was developed specifically for the above type of testing. It is supplied complete with BDI Strain Transducers and the Remote Control Load Position System. In addition to being able to read strains, it shall read deflections from Linear Variable Differential Transducers (LVDT) and rotations from tilt sensors. It shall be rugged enough to be taken to the field and instrumented the bridge in a few hours.

The data acquisition system shall be as manufactured by the Bridge Diagnostics, Inc., 5398 Manhattan Circle, Suite 100, Boulder, CO, 80303. No substitutions are allowed.

The system shall consist of the following components:

1.1 A Microprocessor Data Acquisition:

- A Microprocessor Data Acquisition and Processing System expandable 64 Channels
  The system shall be supplied with Remote Control Position Indicator that allows the user to track position of the loading vehicle as it crosses the structure at crawling speed as to cause no noticeable vibration.
- The system shall utilize BDI-STS 4-channel units connected in series, meaning only one cable needs to run from the PC up to the bridge.
- The system connections shall be heavy-duty military-grade bayonet connectors that “snap” together.
- The system shall be powered with 110AC or with 12VDC (car battery).

- The system shall be supplied with testing software designed by BDI, Inc. and shall be installed on user's PC.
- The STS shall also be configured to accept other types of sensors such as LVDTs, foil strain gages, and other full-bridge type transducers. This option is important for future long term fatigue monitoring of the bridge components.

The following parameters/specification shall be incorporated in the Data Acquisition system:

- Channels: 64 Expandable in multiple of 4 channels
- Accuracy: ± 2.5% (2% for Strain Transducers)
- Sample Rates: 0.01 to 100 Hz
- Gain Levels: 1, 250, 500, 1000
- Digital Filter: 2,10, 20, 30, and 40 Hz
- Analog Filter: 35 Hz, -3db, 6-pole Bessel
- Power: 10-18VDC or 100 – 230 VAC
- A/D Resolution: ± 4.77mV @ 1V/V (12-Bit ADC)
- Self-Balancing [PARA] Range ± 25 mV @ input with 350Ω (Wheatstone bridge)
- Cable Connections All aluminum military grade, circular bayonet snap lock.

<u>1.2 STS Demountable Strain Gage Transducers (DSGT) (64 units)</u>:
The system shall be supplied with Re-Usable Demountable Strain Transducers. The Strain Transducers (also called "Intelliducers") shall be equipped to identify themselves to the system so that channel numbers do not need to be tracked and calibration factors are automatically applied. The Strain Transducers shall be used on steel, pre-stressed concrete, reinforced concrete, timber, and composite fiber structural members. A <u>total of 64 DST</u> is needed to be used with the portable system to measure strains in other locations in spans 1, 2 and 3, Unit 1. The Transducers will also be used in testing of other critical locations in other spans, if needed. The technical specs of the DST are as follows:

Effective gage length:	76.2 mm
Overall Size:	117mm x 32mm x 12mm
Cable length:	3 m standard, any length available.
Material:	Steel.
Circuit:	Full Wheatstone bridge with 4 active 350Ω foil gages, 4-wire hookup.
Accuracy:	+- 2%, individually calibrated to NIST standards.
Strain Range:	+- 1000 microstrain.
Sensitivity:	Approximately 575 microstrain/mV/V.
Weight:	6 oz. (170 g).
Temperature Range:	-50 ^{0}C to 120 ^{0}C operation range.
Circuit:	4-wire, plus shield.

Cable:	Belden 8723: 22 gage, 2 individually-shielded pairs with drain.
Environmental:	Built in protective covers and fully waterproofed.
Attachment methods:	Steel C-clamps.
Transducer Clamps:	A total of 240 units of steel C-clamps: 120 units of 50 mm and 120 units of 75 mm.

1.3 STS Portable Computer:
The STS system will be used at various stages during the project. It will complement the other permanent systems for measuring live loads and static stresses. Therefore, a dedicated portable computer shall be used for every data acquisition system so as not to have a conflict in software and hardware communication cards.

The portable computer shall be as the following requirements:

Portable Computer Broadax Systems, Inc., Model LCD-VM
550 MHz Pentium III, 128 MB SDRAM, 16.8 GB HD
8 slot ISA-PCI with 4 ISA, 3PCI and 1 CPU slot
1.44 MB floppy, 40XCRROM, 250 MB Zip Drive
15.1" TFT LCD with 4MB PCI controller
U.S. Robotics Sportser 128K fax/modem
Intel Ether Express Pro 10/100 100 BaseT4 PCI Ethernet Adapter
MS Windows 2000

1.4 Contractors Responsibilities:
The Contractors responsibilities for the Bridge Structural Testing System is to provide labor and equipment support for the installation of the Demountable Strain Gage Transducers that will be mounted onto the steel superstructure bottom flange. This will require the use of an aerial lift/ladder to provide access to the structure steel and coordination with Conrail to provide flagman as needed. The installation of these gages shall be performed prior to the deck slab being poured. The installation of these gages will be performed during both construction stages.

Long Term Monitoring System (Permanent Installation)

The permanent Bridge Structural Testing System as shown on the plans, shall consist of mounting Vibrating Wire Strain Gages (VWSG) as well as Vibrating Wire Thermo-Couple (VWTC) within the deck slab and demountable strain gage transducers (DSGT) on the steel superstructure. With the assistance of the Contractor, Rutgers personnel will install these sensors and transducers. The Contractor shall be responsible for the purchasing of all sensors, transducers, gages, wiring, and data acquisition and processing system to collect the data. This system is as manufactured by the Bridge Diagnostic Systems. The system can be purchased from Bridge Diagnostics, Inc., 5398 Manhattan Circle, Suite 100, Boulder, CO, 80303. . No substitutions are allowed.

The system shall consist of the following components:

2.1 Microprocessor Data Acquisition: and data processing System with 96 –channel.

2.2 Printer: shall consist of a HP DeskJet 710C type.

2.3 Cables: The length of cables required for this system is shown schematically in the design plans.

2.4 Vibrating Wire Strain Gages (VWST) 96 units:

The general characteristics of the VWSG needed for the deck instrumentation equipment is as follows:

- Grade 60 Rebar
- Vibrating wire strain gage (VWST)
- Rugged, waterproof construction
- High stability and sensitivity
- Can be used as a sister reinforcing bar that can be incorporated into the deck rebar reinforcement system
- It shall provide long-term stability
- Can be welded or tied to a rebar system
- Must integrate a thermistor for temperature monitoring
- A minimum of 60 VWSG to Instrument Bridge Unit 1, Spans 1, 2 and 3 is required.

2.4.1 High Strength Vibrating Wire (rebar) Instrumentation

The required load sensing element shall be a hollow rebar made of high strength steel that withstands rough handling and loading. Local compressive or tension strains are to be induced directly into the rebar and monitored with a coaxially mounted vibrating-wire sensor. It shall be able to permit direct strain readings to be transformed into axial load or concrete mass strain and stress.

2.4.2 Installation.

Each Instrumented Rebar shall be supplied with an individual calibration factor for optimal accuracy output. The Instrumented Rebar can be incorporated into a rebar reinforcement system by welding or by assembling with optional rolled threaded ends. It shall also be used as a sister bar with plain or rolled special adapters supplied for specific installation such as for support Rebars for ground control applications. These gages will be installed and embedded prior to deck pouring after the reinforcement cage is prepared. The installation period required for installing sensors should not exceed three days.

2.5 Vibrating Wire Thermo-Couple 30 units.

These are temperature sensors used to calibrate the reading in other sensors such DSGT and VWSG. These units are also purchased from BSDI, Inc.

2.6 Demountable Strain Gage Transducers: 6 units.
See Bridge Structural Testing System 1.2 for specifications.

2.7 Contractors Responsibilities:
The Contractors responsibilities for the Long Term Monitoring System is to provide labor and equipment to install bracket support system for testing cables, to install testing cables as directed by a Rutgers representative, and to assist Rutgers staff in providing access to the superstructure so that they can make the final connections to the testing gages. This will require the use of an aerial lift/ladder to provide access to the structure steel and coordination with Conrail to provide flagman as needed. The above installation and connection of the gages shall be performed prior to the deck slab being poured. The installation of these gages will be performed during both construction stages.

3. Live Load and Dynamic Testing System

This system shall be used to monitor the superstructure and substructure. The dynamic response of bridge girders, deck vibration during and after construction, substructure movement and vibration, will be measured using this system. This System for dynamic testing requires the use of accelerometers. These accelerometers shall be magnetically attached to the lower flanges of the bridge girders, at the same location of the demountable strain transducers. The accelerometers shall be connected using coaxial cable to the data acquisition system. The data acquisition system is connected to a portable computer that hosts the system's program.

The dynamic measurement shall be performed using a data acquisition system, accelerometers and geophones. The location of the accelerometers shall be determined based on the maximum static response in each girder. Deflections shall be derived from the measured acceleration response at each girder.

3.1 Data Acquisition System
The system shall be manufactured by OPTIM Electronics, 12401 Middlebrook Road, Germntown, MD 20874, as follows:

Item	OPTIM Part Number
MEGADAC 6510DC	ML 1071
Two modules AC3883VW-1k	PL2049
Two modules AD-1 808FB-1	PL2246
Two modules AD 808QB/350	PL1921
Two modules AD 816TC	PL2179
SCSI Reader Kit	PL2538
PCMCIA-BUS SCSI card and Cables	PL2562

The MEGADAC 6510DC system, no substitutions are allowed consist of the following:

- 16-channel data acquisition unit
- 12 VDC Data Acquisition System, Ruggedized and Portable

- A minimum of 2 Analog Digital Conversion (ADC) modules, each with 8 channels.
- 250,000 Samples per Second Throughput
- 16 bit Analog-to-Digital Converter
- SCSI Laser Drive for internal Mass Storage: 1 Gigabyte DS Disks: Removable & re-writeable, and Laser Drive is shock Mounted for Durability.
- Up to 256 Mbytes of Acquisition & Storage memory
- Memory is Battery-Backed for 5 Hours.
- Power consumption is 180 watts
- DC Input Voltage Range: 10.5-18 VDC
- Supports up to 512 Input Channels
- Supports up to 128 Output Channels, IEEE-488, RS232, RS422, and RS-485

The unit shall be connected to a dedicated portable computer that shall serve as the means of communication. Structural response shall be measured by recording the accelerations (accelerometers and geophones are placed on lower flanges of bridge girders). Electric power shall be provided by an electric power source for 24 hours/day, @ 7 days/week, with battery back up for power outage.

3.2 Portable Computer:

The dynamic system will be used at various stages during he project. It will be used in conjunction with the other static systems for measuring live loads and static stresses. Therefore, a dedicated portable computer should be used for every data acquisition system so as not to have a conflict in software and hardware communication cards.
The portable computer should be as the following requirements:

Portable Computer Broadax Systems, Inc., Model LCD-VM
550 MHz Pentium III, 128 MB SDRAM, 16.8 GB HD
8 slot ISA-PCI with 4 ISA, 3PCI and 1 CPU slot
1.44 MB floppy, 40XCRROM, 250 MB Zip Drive
15.1" TFT LCD with 4MB PCI controller
U.S. Robotics Sportser 128K fax/modem
Intel Ether Express Pro 10/100 100 BaseT4 PCI Ethernet Adapter
MS Windows 2000

<u>3.3 Accelerometers:</u>

The accelerometers shall be Voltage Mode instruments that utilize self-generating quartz crystals and a seismic mass to convert acceleration (vibration or shock) to analogous, low impedance, electrical signal. The accelerometers are to be located at the same location where the Strain Transducers in the main girders are placed. Amplification from strains and deflections can be correlated. Various ranges of frequencies are to be covered by using two types of accelerometers: Type A and Type B. Each type will be used at its proper location to be determined accordingly. 8 units of each type are needed for the installation.

3.3.1 Accelerometer Type A shall be a Dytran 3116A model or have the following equivalent specification (8 Units are needed):

- a reference sensitivity at 100 Hz, of 1000 mv/g $\pm$ 10;
- a maximum transverse sensitivity of 5%;
- a frequency range of .05 – 500 Hz,
- A resolution of .0002g and a range of $\pm$ 5g.
- It shall be able to operate in a temperature range of –60 to +250 ^{0}F.
- Its voltage supply shall be in the range of 18 to 30 VDC.
- It shall have a maximum vibration of 20g,
- maximum shock of 50g, and a thermal coefficient of sensitivity of 0.03% / ^{0}F

3.3.2 Accelerometer Type B shall be a Dytran 3100A model or have the following equivalent specification (8 Units are needed):

- a reference sensitivity at 100 Hz of 100 mv/g + 2%;
- a maximum transverse sensitivity of 5%;
- a frequency range of 1-3500 Hz,
- A resolution of .007g and a range of + 5g.
- It shall operate in a temperature range of –60 to +250 ^{0}F
- a voltage supply range of 18 to 30 VDC
- a maximum vibration of $\pm$ 600g,
- maximum shock of 3000g, and a thermal coefficient of sensitivity of 0.03%/^{o}F
- A natural frequency of at least 26 kHz.

Accelerometers are to be connected through microdot cables to a power unit that is in turn connected to the data acquisition using coaxial cables. It shall be mounted on the upper flat surface of the lower flange of the steel beam.

Live Load Testing

Live load testing will be performed upon the completion of construction stages I & II. For the live load testing of construction stage I, testing will be performed before it will be open to traffic. Therefore, testing can be performed at anytime after completion of the bridge and the approach roadway. Multiple trucks with known weights will be placed in each lane and span at specific locations and readings of the gages will be obtained. This operation will be performed repetitively in each span. For the live load testing of construction stage II, temporary closure of the bridge will be required for short periods of time to obtain live load readings. Under this condition, the testing vehicle will be placed in specified lanes that will be closed to traffic, maximum of two lanes. The other two lanes will be open to traffic when testing is not being performed. The maximum duration of complete closure of the structure will be limited to a period of 15 minutes every half hour or until traffic has resumed to normal flow. Testing will not be performed during peak hours of traffic and may have to be performed at night. Specific hours of testing will be set by the NJDOT.

Dynamic Tests:
The Dynamic Testing shall be performed at different times during the construction and after the completion of the project. The dynamic testing will be performed at the following times:

1 Deck pouring operation of Unit 1 – during Construction Stage I
2 During live load testing of Unit 1– completion of Construction Stage I
3. Deck pouring operation of Unit 1 – during Construction Stage II
4. During live load testing of Unit 1 – completion of Construction Stage II
5. Prior to traffic opening to obtain the bridge “finger prints” which in turn can be used in future health monitoring of the bridge infrastructure status.

For the dynamic testing of construction stage I, testing will be performed during and after construction but prior to opening to traffic. Therefore, testing can be performed at the same time when the live load testing is also performed. Under this condition, the testing vehicle will be traveling at highway speed in specified lanes that will be closed to traffic. As for the testing of construction stage II, completion of the structure, temporary closure of the bridge will be required for short periods of time to obtain dynamic load readings. Under this condition, the tests will proceed as described in section 1, however, more passes of trucks will be required for the dynamic testing. The number of tests shall not exceed more than three truck passes in each lane.

Duration of tests:
The dynamic testing will be performed within 2 weeks of completion of the Stage Construction. However, the Dynamic testing System will be used more frequently during construction and deck pouring of concrete slab. Therefore, the Contractor will coordinate the testing with the Rutgers Coordinator for completion of this test. The test during construction should not take more than one day. However, the Dynamic Testing requires two days for setting up and connecting sensors to data acquisition system. The accelerometers shall be placed on steel girders and deck slab and will be attached by magnetic stands. Therefore, access and equipment support such snooper truck, traffic closure, and traffic control is needed. The Contractor shall coordinate these plans with NJDOT and Rutgers personnel two weeks prior to actual testing dates.

The live load testing will be performed within 2 weeks of completion of the Stage Construction. The Contractor will coordinate the testing with the Rutgers Coordinator for completion of this test. Each live load test will be performed over a one-week period.

3.4 Contractors Responsibilities:
The Contractors responsibilities for the Dynamic Testing are to provide labor and equipment supports for the installation of the Accelerometers that will be mounted onto the steel superstructure bottom flange. This will require the use of an aerial lift/ladder to provide access to the structure steel and coordination with Conrail to provide flagman as needed. The installation of these gages shall be performed prior to the deck slab being poured, simultaneously with the installation of the Demountable Strain Gages. The installation of these gages will be performed during both construction stages.

The Contractor responsibilities for the live load testing shall assist in performing the live load testing of the structure. Live load testing will be performed after the completion of construction of both construction Stage I & II. It shall be the Contractor's responsibility to provide at least four vehicles (truck) with operators with a minimum gross vehicle weight of 36 tons each. The specification of these trucks, axles spacing, width of truck, weight of vehicle, etc shall be provided to the Rutgers Coordinator at the start of the construction. The Contractor will be required to load the vehicles to a specific weight and obtain the exact weight of the vehicle and its axles weights before testing can be performed. Truck operators will be required to move and place the vehicles as directed by the Rutgers personnel.

In addition, the Contractor will install lane closures and coordinate temporary closures of the entire structure during live load and dynamic testing. The Contractor shall provide all the appropriate signing, cones, arrowboard, truck mounted impact attenuator, labor and all other equipment necessary for multiple lane closures and to stop traffic during testing.

<u>Deflection Measurements:</u>

The Contractor will supply a survey crew at specific times during of the project that will assist in obtaining specific measures for the testing program as defined below.

Surveying Equipment**:**

4.1 The Deflection measurements shall be performed at different times during the construction and after the completion of the project. The measurements and surveys will be performed at the following times:

- 4.1.a During Construction at different Stages
- 4.1.b During live load testing of Unit 1– completion of Construction Stage I
- 4.1.c During live load testing of Unit 1 – completion of Construction Stage II
- 4.1.d Prior to traffic opening to obtain the bridge "finger prints" which in turn can be used in future health monitoring of the bridge infrastructure status.
- 4.1.e After traffic opening for future periodic measurements.

The Contractor shall perform continuous field surveys during the construction of Unit 1 to record the deflection of the beams at specific locations up to accuracy of 2/100 millimeter. This survey will be performed during different stages of construction to obtain deflections for beam only, beam and slab dead load, composite beam and superimposed dead loads, and live load deflection after completion.

For this operation, the installation and placement of survey equipment shall be coordinated with the Contractor, so that his operation will not be interrupted. The Rutgers Coordinator shall work with the Contractor's schedule and coordinate this survey work appropriately. The Contractor shall perform all survey using his survey equipment and provide necessary support for loading systems such as trucks with known axle weights, measured axle spacing, truck type and number of axles, heavy concrete blocks to simulate lane loading, and other type of loading systems.

The Contractor shall measure the deflection at the same target points that are specified by Rutgers Coordinator. The target points shall be installed permanently and will remain in place after construction has been completed for continued monitoring of the structure. The Contractor shall measure deflections in Unit 1, Spans 1,2, and 3, at L/4 points and the maximum positive moment in each girder of each span. The Rutgers Coordinator will provide the maximum point of positive moment to the Contractor.

The Contractor shall purchase a Total Station with the required accessories to allow the Rutgers Team to perform similar measurements performed by the Contractor beyond the project duration. The comparison of deflections over a long period of time is vital for processing long-term deflections and the effect of creep and shrinkage.
Surveys of these points will be taken during each stage of construction at the following times:

4.1.1 After the beams have been erected
4.1.2 After the Deck slab has been poured
4.1.3 After the construction stage has been completed (sidewalk and parapets poured, utilities installed)
4.1.4 During Live Load Testing, different load configurations:

- Loads on Span1 in Lane 1
- Loads on Span 1 in Lane 2
- Loads on Span 2 in Lane 1
- Loads on Span 2 in Lane 2
- Loads on Span 3 in Lane 1
- Loads on Span 3 in Lane 2

Spans 1, 2, and 3 are to be loaded with HL-93 or equivalent load (which is comprised of lane loading of 11.6KN/m and HS-20 Truck Load).

4.2 Test Procedure

Stage I Complete:
4.2.1 Load Lane 1 in Span 1 with an equivalent lane load of 11.6KN/m using concrete block, NJ barriers, trucks etc.) Measure deflection at L/4 the span length and maximum positive moment in each Span, for each girder.

4.2.2 Load lane 1 in Span 1 with a truck having a gross vehicle weight of minimum of 36 tons and having axle spacing matching the HS-20 AASHTO Design Truck. The HS-20 Truck has 3 axles and 4.26 meters front and rear axle spacings.

4.2.3 Load Lane 2 in Span1 as in 4.2.1

4.2.4 Load Lane 2 in Span1 as in 4.2.2

Similar cases of loading and measurements shall be performed for Spans 2 and 3 as described in section 4.2.1, 4.2.2, 4.2.3, and 4.2.4 above. A total of 12 loading cases should be performed in Unit I. In each loading case described above, the deflection profile at quarter point and maximum positive moment in each girder (PG1 through PG10) shall be measured.

For loading cases 1 through 6 described in section 4.2, Rutgers shall also collect data simultaneously from the VWSG, Demountable Strain Transducers, and Thermocouples installed in Unit I, spans 1, 2, and 3. The results shall be correlated with the deflected profiles of each girder.

The Contractor shall be responsible for the validity of the data collected from the surveyed profiles of each girder. In case of erroneous data from the survey, the Contractor shall be responsible for performing all of the tests performed and described in section 4.2.

5. Weigh in Motion Bending Plate System

The Bending Plate system shall be used to collect truck load data from each lane. The system weighs trucks traveling at highway speeds and triggers the data acquisition systems installed on the bridge structure. The various bridge response (strains) will be correlated with the corresponding truck load. Data on truck speed, axle configuration, axle loads and position will be recorded.

The system will monitor four lanes of traffic. Each lane shall consist of two (2) loops and two (2) weigh pads with a configuration of loop-weighpad-loop-weighpad as shown in plans. The system requires 120 Volts AC service. The manufacturer of equipment shall provide installation guidance, on site supervision, and materials listed below for the weigh-in-Motion System.

5.1 Bending Plate System (8 Units):

The Bending Plate system will be as manufactured by PAT America, Inc., 1665 Orchard Drive, Chambersburg, PA 17201, (717) 263-7655, Fax 9717) 263-7845. No substitutions are allowed:

1. DAW 190 WIM System Electronics (power consumption less than 5 watts @12 volts) 1 Unit
2. 12 Volt UPS Power Supply 1 Unit
3. Industrial Grade Modem, 28.8 KBPS 1 Unit
4. Hennessy Type M or 3B Cabinet with power distribution panel and surge protection 1 Unit
5. Bending Plates 1.75 meter (260 pounds each) 8 Units

6. Bending Plate Frames (245 pounds each) 8 Units
7. E-Bond Epoxy (5 gallon bucket) 24 Units
8. Weigh pad Lead-in cable 4000’
8. Remote data collection Software and Documentation (Windows Based) 1 Lot
9. Installation Supervision 1 Lot
10. Calibration and Installation Acceptance 1 Lot
11. One Year Warranty 1 Lot
12. Acceptance testing 1 Lot
13. Training (2 Days) 1 Lot

The manufacturer shall also provide installation guidance, on site installation supervision and the following support work:

1. Installation supervision of weigh pads frames weigh pads and loop detectors.
2. Wire and terminate weighpads, loop lead-in cables, power service installation guidance to a WIM electronics cabinet.
3. Commissioning acceptance and calibration of the WIM system.
4. Installation acceptance testing of the WIM system
5. Final acceptance of the system.

Electrical power source shall be provided as needed, and at locations, as advised by Rutgers personnel. The Contractor shall be responsible for the installation of the Weigh in Motion Bending Plate System and all incidental item such electrical cabinets, electric hook up, and all other associate roadway work, etc, The installation will include but not be limited to the weigh pads, weigh pad frames, loop detector, housing cabinet, axle sensors, cables, protective pvc piping for cable, trenches for cables, etc.

5.2 System Calibrations and Operation

The WIM system shall be calibrated using actual truck loads with a minimum weight range between 15-36 tons. The Contractor will supply these trucks. The Contractor will be responsible for loading vehicles and obtain actual axle weights. It is anticipated that 5 separate truck weights will be provided for each bending plate calibration. Each bending plate will be calibrated. A calibration vehicle 5-axle semi loaded to 75,000 to 80,000 pounds is needed for the calibration process.

5.3 Contractors Responsibilities:

The Contractor shall provide all labor, equipment and other incidental items required to install WIM system as shown on the plan. A manufacturer representative shall at the construction site to oversee the Contractors installation of the WIM system. The Contractor shall also provide the required trucks with operators and obtain specific truck weights with loads to calibrate the WIM system after installation. The WIM system needs to be installed, calibrated and operational prior to the live load and dynamic testing of each stage of construction. The Contractor shall provide traffic control, placement of conduits, pull boxes, and concrete for cabinet pedestal, loop wire, loop lead-in, loop sealant (quantity of eight (8) loops, 1.8 m x 1.8 m, 4 turns) The Contractor is also responsible for:

1. Transporting equipment to site.
2. Installation conduits and pull boxes to include conduit under roadway.
3. Installation of in road sensors to include frames weigh pads, and inductive loops.
4. Installation of concrete pedestal for cabinet, and mounting cabinet.
5. Establishing electrical and phone service to control cabinet.
6. Cable pulling and waste disposal.

Portable WIM system: The system is to be manufactured by PAT America, Inc., 1665 Orchard Drive, Chambersburg, PA 17201, (717) 263-7655, Fax 9717) 263-7845. The system consists of a portable WIM system to be used in spot-checking and calibrating the Bending Plate System throughout the project duration. The Rutgers Team will use this portable system to monitor truck loads instantly at various locations on or before the Bridge Bending Plate system. The data will confirm truck load data collected in automatic mode.

6. Support Equipment:

The following equipment is required to support the testing program and shall be purchase at the beginning of the project. Working drawings and equipment specification needs to be submitted to the Engineer and to the Rutgers Coordinator for approval prior to purchasing.

6.1. Power generator (1 Unit):
A source of power supply is required for portable equipment operation and field-testing. A minimum of 2.6-kW Yamaha EF2600 electric generator shall be used to power the system. It shall be capable of continuous use for the test period that usually lasts for more than 10 hours. The refueling step shall be done during operation without causing any interruption of the test process.

Rated Voltage:	120V
Frequency:	60 Hz
Maximum AC Power:	2600 Watts
Rated AC Output:	2300 watts
Rated/Maximum:	19.2/21.7 amps @ 120 V
Overall Dimensions:	20”x16.3”x18.3”
Weight:	85.8 lbs.
Fuel tank Capacity:	1.2 gallons
Continuous Operation Hours at 1/2 rated load:	4.5 hrs.

6.2 Testing Van (1 Unit):
The Van shall transport and house the portable instruments and other support equipment such as electric generator, ladders, drills, tool boxes cables, connectors, etc. The Contractor shall purchase the following van model:

Model: 2000 Dodge Ram Van 1500 Cargo Van 127.2

Brief Description:

3.55 Axle Ratio
GVWR: 6,600 lbs.
Tires: P235/75R15 AS BSW
HD Vinyl Bucket Seats
Engine: 5.2L SMPI V8 Magnum
Transmission: 4-Speed Automatic
Anti-Spin Differential Axle
Air Conditioning
Security Alarm
4-Wheel Antilock Brakes
Radio: AM/FM w/CD/Cassette/EQ
Exterior Color- Bright White
Interior Color- Mist Gray
2nd Choice Exterior Color- Colorado Red
2nd Choice Interior Color- Camel/Tan

6.3 Oscilloscope (1 Unit):

Oscilloscope shall consist of a HP 54645A Model or equivalent. It shall have a minimum of 2 channels and a sensitivity range of 1mV/div to 5 V/div, maximum input of 400 V, and Power supply of 100-240 Vac, 48-440 Hz, and 300 VA maximum.

6.4 Steel Control Cabinets (3 Units)

The Contractor shall purchase three Steel Control Cabinets and assist Rutgers in installing the Control Cabinets at Pier 2. The three steel control cabinets shall be water tight and equipped to house wireless units, modem, battery, and an on-board computer system. The steel control cabinets shall be a minimum size of 32x32x24.

6.5 Wireless Data Links (4 Units):

In many locations on the bridge the distance between the sensors make data links a more economical and attractive alternative to connecting the sensors via hard wire through conduit. Each station is configured with a RF spread-spectrum transceiver and modem that communicates with a similar transceiver at the master station. The master configuration is to install a master station at pier No. 2 and then place remote stations as needed at other locations. The locations will depend on electric noise interference, length of cables, etc. The contractor shall purchase these wireless units from ETI, Instrument Systems, Inc., 1317 Webster Avenue, Fort Collins, Colorado 80524.

6.6 Tiltmeters (4 Units):

Tilt sensors will be placed at few locations in the bridge. The usual orientation of the sensors is to place several of them in pairs at the locations where the Vibrating Wire

Thermo-Couples (VWTC) are placed in the bridge Plate Girder no. 4 and 8, respectively. One sensor in each pair measures tilt along the roadway and the other measure tilt across the roadway. Temperature sensors are normally installed in the tilt sensor housing to monitor the bridge's diurnal temperature changes. Tilt sensors have a sensitivity of 0.01 degrees of tilt. A typical message response issued by the system would be when the tilt in either direction exceeds 0.25 degrees. For each span 4-tilt sensors will used for a total of 12 sensors.

6.7 Video Camera (1 Unit):
A video camera is needed to document test procedures, traffic patterns, and inspection of various bridge locations and sensor array.

6.8 Pre-Wired Strain Gages (50 Units):
These gages are to be purchased from Micro Measurement group, Inc. They are gages that will be applied to the surface of concrete at random locations to be determined later. The location will depend on cracking formation and other concrete mechanical properties observed on site. A total of 50 gages are needed.

II. SUBSTRUCTURE TESTING SYTEMS:

1. Crosshole Seismic Testing:

Equipment:
Cross-Hole, SASW and Attenuation Measurement Devices as outlined below:

1. Hewlett Packard HIP 35670A Dynamic Signal Analyzer (1 Unit), with:

- Opt AY6 2 additional input channels
- Opt 1C2 HP Instrument Basic
- Opt 100 Software Bundle
- Opt UFF 1 MB nonvolatile RAM
- 3251A DC power cable
- hard disk for HIP35670

An alternative dynamic signal analyzer maybe used and shall meet the following basic specifications:

- minimum four channels
- 16-bit ADC/90 dB dynamic range
- 1600 line frequency resolution
- frequency range DC – 100 kHz
- sensitivity max –50 to +50 mV full range
- spectral analysis software (order tracking real-time octave, swept-sine, curve fit)

2. APS Dynamics, Inc. Long Stroke Shaker (1 Unit), that includes:

- Electro-Seis Long Strong Shaker Model 400
- Dual-mode power amplifier Model 144
- Power and control cables

An alternative shaker and power source maybe used and shall meet the following basic specifications:

- 0-1000 Hz range
- Maximum force min 70 to 100 lbs.
- Noise –90 dB
- Sine and swept sine capabilities

3. Geophones Mark Products (18 Units)

- 16 geophones model L-22E 3-DS with 510 Ohm coil resistance, 2 Hz
- 2000 meter of special cables made by manufacturer for geophones
- 2 geophones model L-4-3D with 5500 Ohm coil resistance, 1 Hz
- 36 phono round 5-pin connectors (24 male and 12 female)
- 24 BNC male type connectors

An alternative to L-22 3-DS geophone maybe used and shall have the following characteristics:

- not more than 2.0 Hz natural frequency
- transducer constant of at least 1.0 V/ips
- size not more than 3”x3”x10”

An alternative to L-4-3D geophone maybe used and shall have the following characteristics:

- not more than 1.0 Hz natural frequency
- transduction of at least 7.0 V/ips
- size not more than 8”x8”x10”

4. Slope Indicator 2.75” tubing for vertical inclinometers – 100 meter

5. Rugged Portable Computer Broadax Systems, Inc., Model LCD-VM (1 Unit):

- 550 MHz Pentium III, 128 MB SDRAM, 16.8 GB HD
- 8 slot ISA-PCI with 4 ISA, 3PCI and 1 CPU slot

- 1.44 MB floppy, 40XCRROM, 250 MB Zip Drive
- 15.1” TFT LCD with 4MB PCI controller
- U.S. Robotics Sportser 56K fax/modem
- Intel Ether Express Pro 10/100 100 BaseT4 PCI Ethernet Adapter
- MS Windows 98

An alternative is a system meeting the above specifications and the same level of ruggedness and portability.

6. National Instruments 6071E Family Data acquisition board (MIO-64E-1) and accessories (1 Unit):

- data acquisition board PCI-6071E and NI-DAQ for Win98
- 32 channel SCXI-1100 signal conditioning module
- BNC-2095 Terminal block for SCXI
- SH96-96 2m shielded cable
- SCXI-1353 connector
- SCWI 4 slot chassis

An alternative system maybe used and shall have the following specifications:

- 32 different channels
- minimum 0.5 MS/sec
- minimum 12 bit resolution
- minimum sensitivity full range –50 to +50 mV
- 2 analog outputs, analog and digital triggers

Method of Operation.

Responsibility of Contractor:

<u>1) Crosshole Seismic Testing</u>

Preparation of 200-mm diameter bore holes for crosshole testing.

Tubing for crosshole testing shall be installed as early as possible so that testing can be conducted prior to major construction operations.

a) Location of boreholes:
The crosshole test shall be conducted at five locations, in every other span. While the proposed locations are indicated in the plans, the exact locations shall be determined based on the site accessibility. The Contractor shall consult with Rutgers Coordinator regarding alternative placements of crosshole tubing.

b) Number, spacing, alignment and depth of bore holes and Sample recovery:

Three bore holes shall be prepared for each crosshole test location. The bore holes shall be aligned nominally on a straight line. The spaced distance between the first and second hole shall be 1.5 meters and the distance between the second and the third hole shall be about 3.0 meters. The bore holes should go to the bedrock elevation, approximately 24.5 meters deep.

Samples from all distinctive layers shall be recovered using Shelby tubes during boring operations and provided to Rutgers University. The total number of Shelby tubes is 25. A representative from Rutgers University shall be invited to be present during boring operations and consulted regarding the depths at which the samples are recovered. Rutgers staff will pick up shelby tubes at the site.

c) Excavation and Verticality of Bore Holes

Bore holes shall be excavated by any method which provides a bore hole of constant diameter of 200 mm diameter and which prevents cave-ins during excavation and during installation of the casing. Wash borings and continuous flight augers shall be used where the soil conditions permit. The bore holes shall be drilled as vertical as possible by careful leveling of the rig and boring tools over each hole.

d) Bore Hole Casing

Bore holes shall be cased with Schedule 40 PVC flush joint casing with 100 mm inside diameter. The casing shall be grouted into the bore hole with a Portland cement and bentonite grout proportioned so that the hardened unit weight is about the same as the total unit weight of the soil surrounding the casing. This can usually be accomplished with a mixture consisting of 1 lb. Portland cement, 1 lb. Bentonite and 6.25-lb. water. However, the bentonite may be different depending on the supplier and the mix may have to be modified.

A sufficient volume of grout shall be prepared to fill the annular space between the soil and casing plus 50% of that volume to fill potential cavities in the bore hole wall, and the grout shall be placed in the bore hole. The PVC casing shall be assembled in the lengths convenient for handling and sealed for water tightness. The bottom end shall be closed with a watertight cap. The casing shall be lowered into the bore hole displacing the grout upward around the casing to fill the annular space between the bore hole and the casing. If drilling mud has been used in excavating the bore hole, the grout shall be placed from the bottom of the hole with a tremie pipe, and care taken to ensure that all drilling mud is displaced upward from the bore hole ahead of the grout

Additional sections of casing shall be spliced with watertight joints until the casing pushed down into the bore hole reaches the bottom and extends at least 600 mm above ground level. The casing may be filled with water to aid sinking it to the bottom of the bore hole through the grout. It may be necessary to hold the casing down during set of the grout. This may be done with dead weights, stakes in the ground with wires over the top of the casing, etc.

A concrete cylinder mold shall be filled with grout and kept at the site for inspection. After the grout has set (usually 24 hours) the water used to sink the casing must be removed from the casing, and the casing capped.

Electrical power source shall be provided as needed, and at locations, as advised by Rutgers personnel.

2. Drilled Shaft Monitoring

The dynamic stiffness (impedance) of the drilled shaft will be utilized in numerical simulations (dynamic soil-structure interaction) of the bridge under static and dynamic loads. The impedance evaluation test involves harmonic excitation of drilled shaft by an electromagnetic shaker and measurement of their response as a function of frequency. To ensure complete transfer of the shaker energy into a drilled shaft and accurate measurement of the drilled shaft response, both the shaker and the sensor (geophone) have to be in an intimate contact with the drilled shaft.

Altogether ten drilled shafts will be tested. Locations of the tested drilled shafts match locations of crosshole tests, i.e. for each crosshole test location two nearest cross holes will be tested. The first tested cross hole will be the first installed cross hole in the line. The objective of this will be to evaluate the impedance before potential drilled shaft interaction effects due to other installed drilled shaft. The same drilled shaft will be tested after the others in the line are installed. The second tested Cross hole will be the one in the center of the drilled shaft line, after all the drilled shaft in the line have been installed.

For each of the piles/caissons tested, the Contractor shall on the top firmly anchor/attach a steel plate. The plate shall be of dimensions 70 cm x 70 cm and be made of at least 13 millimeters thick steel. The plate should be attached in a way to ensure that there is no differential movement/rotation between the pile/caisson during the shaking. Rutgers will be responsible for the attachment of the shaker and geophones on the plate and the conduct of the impedance evaluation. Upon the completion of impedance evaluation, the Contractor shall remove the plates. The caissons that are going to be tested are marked in the drawings.

Electrical power source shall be provided as needed, and at locations, as advised by Rutgers personnel.

3. Pile/Caisson Vibration Monitoring

The Contractor shall install an array of tri-axial geophones placed in a tube of approximately 100-mm diameter in the west-most caisson/column of Pier #1. The tube, assembled by Rutgers, shall be placed by the Contractor inside the caisson casing prior to the placement of concrete. To ensure the verticality of the tube, the Contractor should attach spacers to the tube or provide a supporting reinforcement steel cage. The tube should be placed so that it does not interfere with caisson integrity testing. (Most likely close to the caisson steel casing, between integrity testing crosshole pipes.) A 50-mm inner diameter protective PVC tube attached to the geophone tube, a guide for all cables to the instrumentation location, should be protected from accidental damage. The schematic of the protective geophone tubing and pipe installation is shown on the plans.

4. Pier Column and Cap Vibration Monitoring

Three piers and three pile caps, Piers 1, 2 & 3, will be instrumented with tri-axial geophones. As part of the Stage I Construction geophones will be installed at the base of column #1 and at mid-height of the cap over column #2 (see design plan locations). The geophones will be placed in protective cases attached to piers and pile caps. The Contractor shall prepare the 100 mm x 250 mm protective casing and firmly attach it to piers and pile caps, so that any differential motion between the casing and structural elements are prevented. A 25 mm inner diameter protective flexible PVC pipes, at least 30 meters long, shall be attached to the 25 mm diameter opening on the protective case.

General Requirements:

Before the purchase of all testing equipment required above, the Contractor shall consult and confirm with the Rutgers Coordinator the material being purchase, the number of units, the type of equipment etc. so that the correct instrumentation is purchase. In addition, the Rutgers Coordinator may be able to help in the locating of specific instrumentation for the project.

Where Rutgers personnel will be installing instrumentation, the Contractor will assist in providing the necessary equipment such as ladders, man-lift or bucket trucks. In addition, the Contractor will coordinate with Conrail to supply flagman if necessary.

Other responsibilities of the Contractor:
The Contractor shall install and provide electric and phone service for the monitoring operation for the duration of the construction. The electric and phone service shall be provided to the designated location at shown on the plans. The electric and phone access shall be place in a utility type box. This box shall be locked and keys shall be provided to the Construction Superintendent and Rutgers Coordinator.

524.04 Method of Measurement.

For each of the lump sum pay items, the Contractor is to procure the testing equipment listed below and furnish all required labor and equipment but not limited to: installing the support brackets, installing testing cables, providing a survey crew, office space, storage trailer, electric and telephone service, extended warranty to cover equipment after construction, manufacturer representative as required, lift equipment, testing equipment, computers, vans etc. will not be measured.

Testing Equipment to be purchased for each testing system:

1.0 Bridge Structural Testing System (Temporary)

64 Demountable Strain Transducers
64 Channel Data Acquisition and Processing System
1 STS Portable Computer – 550 MHz
6,000 meters of Cable Belden 8723, 22 Gage, 2 pair shield
120 units of 50 mm Steel C-clamps
120 units of 75 mm Steel C-clamps

2.0 Long Term Monitoring System

- 96 Channel Data Acquisition and Processing System with Printer
- 96 Vibrating Wire Strain Gages
- 6 Demountable Strain Transducers
- 30 Vibrating Wire Thermo-Couple

10,000 meters Cable Belden 8723, 22 Gage, 2 pair shield

- 350 meters of 152 mm PVC Pipe
- 125 meters of 200 mm PVC Pipe

3.0 Live Load and Dynamic Testing System

- 16 Accelerometers (8 units Type "A" and 8 units Type "B")
- 16 Channel Dynamic Data Acquisition System (MEGADAC 6510DC)
- 1– 550 MHz Portable Computer
- 4 Deflection/Tiltmeters

4.0 Deflection System

- 1 Total Station
- 150 Target Prisms
- 2 Surveying rod

5.0 Substructure Testing Systems

- Electromagnetic Shaker & Amplifier
- 4 Channel Dynamic Signal Analyzer
- 18 Geophones (16 - L-22E-3-DS & 2 - L-4-3D)
- Vertical Inclinometer
- 1 – 550 MHz Portable Computer
- 16 Channel Data Acquisition System
- 150 meters of 50 mm PVC Pipe
- 2000 meters of Cable from Geophone Manufacturer
- 350 meters of 100 mm PVC Piping

6.0 Weigh in Motion System

- 8 Bending Plate Systems
- 2 Electrical Cabinets
- 1 Portable WIM System

7.0 Support Equipment

- 1 Power Generator
- 1 Testing Van
- 1 Oscilloscope
- 1 Video Camera
- 3 Control Cabinets (SIZE NEEDED)
- 3 Solar panels
- 3 Back-up Rechargeable Batteries with chargers
- 4 Wireless Data Links
- 50 Pre-Wired Embedment Strain Gages.

524.05 Basis of Payment.

Payment will be made under:

Bridge Structural Testing System	Lump Sum
Long Term Monitoring System	Lump Sum
Live Load and Dynamic Testing System	Lump Sum
Survey/Deflection Monitoring	Lump Sum
Substructure Testing System – Crossholes	Lump Sum
Substructure Testing System – Drilled Shaft Monitoring	Lump Sum
Pier Column & Cap Monitoring	Lump Sum
Weigh in Motion System	Lump Sum
Support Equipment	Lump Sum
Miscellaneous Equipment/Materials	Lump Sum

Payment for **Bridge Structural Testing System** will be made in four payments: Each construction stage will account for 50% of the Lump Sum payment. Each construction stage will be separated into two equal payments of 25% of the Lump Sum Payment each. The first payment will be made upon the purchase and receipt of the instrumentation and equipment for each construction Stage. The second payment will be made upon completion of the installation of the monitoring equipment for construction Stage and tested. Payment shall include but not limited to the purchase of testing equipment, labor and equipment to assist Rutgers staff install Demountable Strain Gages, coordinate railroad flagman, etc.

Payment for **Long Term Monitoring System** will be made in four payments: Each construction stage will account for 50% of the Lump Sum payment. Each construction stage will be separated into two equal payments of 25% of the Lump Sum Payment each. The first payment will be made upon the purchase and receipt of the instrumentation and equipment for each construction Stage. The second payment will be made upon completion of the installation of the monitoring equipment for construction Stage and performing the testing. Payment shall include but not limited to the purchase of testing equipment, labor and equipment to assist Rutgers staff install testing equipment, install support brackets, install testing cable, coordinate railroad flagman, install junction box at Pier 2, provide electric and telephone connection, etc.

Payment for **Live Load and Dynamic Testing System** will be made in four payments: Each construction stage will account for 50% of the Lump Sum payment. Each construction stage will be separated into two equal payments of 25% of the Lump Sum Payment each. The first payment will be made upon the purchase and receipt of the instrumentation and equipment for each construction Stage. The second payment will be made upon completion of the installation of the monitoring equipment for construction Stage and performing the testing. Payment shall include but not limited to the purchase of testing equipment, labor and equipment to perform lane closures, provide testing vehicles (trucks) with operators, coordinate railroad flagman, etc.

Payment for **Survey/Deflection Monitoring** will be lump sum. Payment for survey service by the contractor shall be paid for in accordance with the following schedule: 25% after all dead load deflection have been obtained for Stage I, 50% after live load testing has been completed for Stage I, 75% after all dead load deflection have been obtained for Stage II, 100% after live load testing has been completed for Stage II. Payment shall include but not limited to the purchase of a total station and targets for Rutgers, the labor and equipment necessary to mount targets on beams and perform survey as specified above, coordinate field survey with Rutgers coordinator, etc.

Payment for **Substructure Testing System – Crosshole** will be lump sum. Payment shall include the purchase of all testing equipment, labor, equipment, material, grouting of crossholes, boring of crosshole, taken of soil samples and all necessary work to properly install seismic testing system. Payment of item shall be performed on a percentage basis, based upon the number of testing locations completed. Each testing location consists of three testing crossholes.

Payment for **Substructure Testing System – Caisson** will be lump sum. Payment shall include the purchase of all testing equipment, labor, equipment, material, installation of PVC pipe, associated reinforcing steel and all necessary work to properly install geophone array tube in the caisson of Pier 1. Payment of item shall be made upon the completion and testing of the system. Boring of hole, construction of caisson and pier column is not included within this item and will be paid for under separate items.

Payment for **Substructure Testing System – Piers and Caps** will be made in two payments: 50% of the Lump Sum payment will be made upon the purchase and receipt of the instrumentation and equipment. The final 50% of the Lump Sum payment will be made upon completion of the installation of the monitoring equipment. This payment shall include all labor, equipment, material, the fabrication and mounting of protective boxes on piers and other incidental item necessary to properly install and test the system.

Payment for **Weigh in Motion System** shall be Lump Sum. Payment shall include the purchase of the Weigh in Motion System and all associated accessories, all the labor, equipment, and material for installation and other incidental item necessary to properly install and test the system. It shall also include the cost of having a manufacturer representative present at all times during installation. The cost shall include the connections of electric and phone service to the system.

Payment for **Support Equipment** shall be Lump Sum. Payment shall include the purchase of items defined in the specification. This item shall include the cost of the labor to properly install control cabinets on Pier 2.

Payment for **Miscellaneous Equipment / Materials** shall be made on an as approved basis and payment will be made upon completion of the work or receipt of the material. This item will be used on an as need basis and a predetermined amount of $12,000 has been established for this item. No charges or expenditure will be allowed by the Contractor for this item until written approval has been obtained.

The payment for electric and phone services as part of the monitoring program shall be paid by the Contractor for the duration of the construction, until States acceptance of the project. Upon States acceptance, the electric and phone services for the on going monitoring of the bridge will be paid for by the State for a minimum of 2 years after the construction has been completed.

Partial payments will be made, in accordance with the Lump Sum provisions of Subsection 109.05, on a monthly basis.

APPENDIX B

Instrumentation Sheet Plans for the Doremus Avenue Bridge

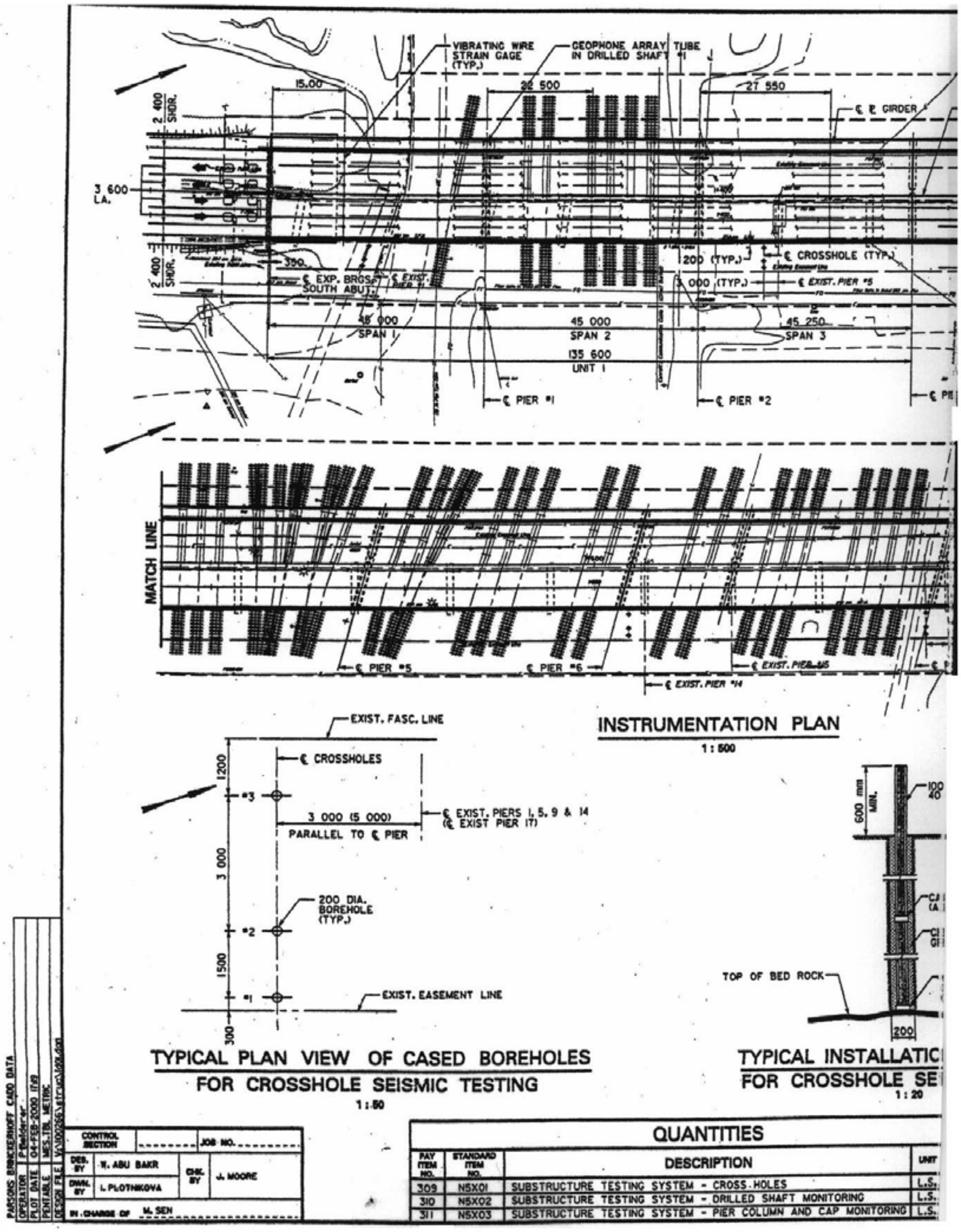


Figure 27. Instrumentation details of VWSG of the Doremus Avenue Bridge.

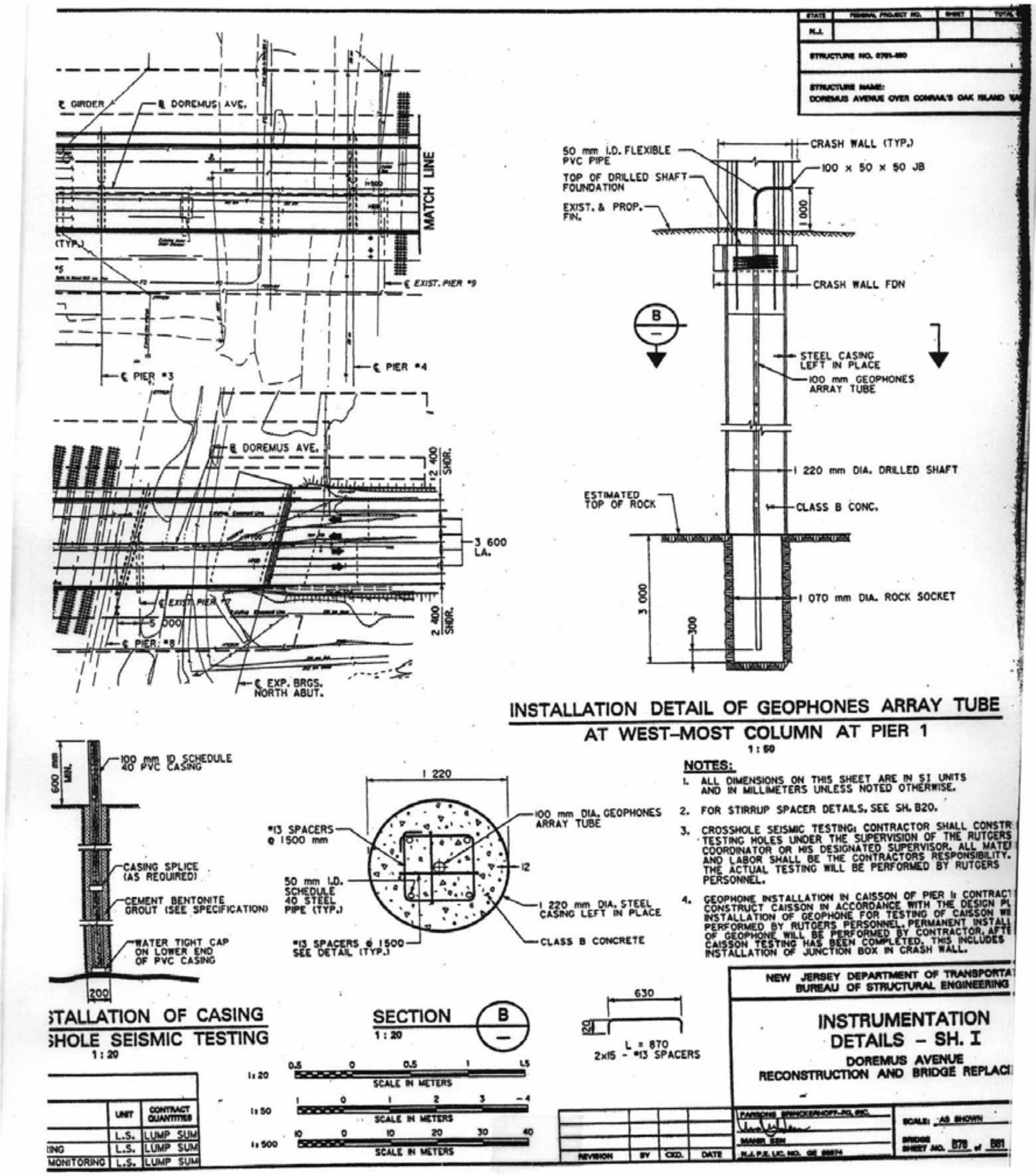


Figure 28. Installation detail of geophones array tube.

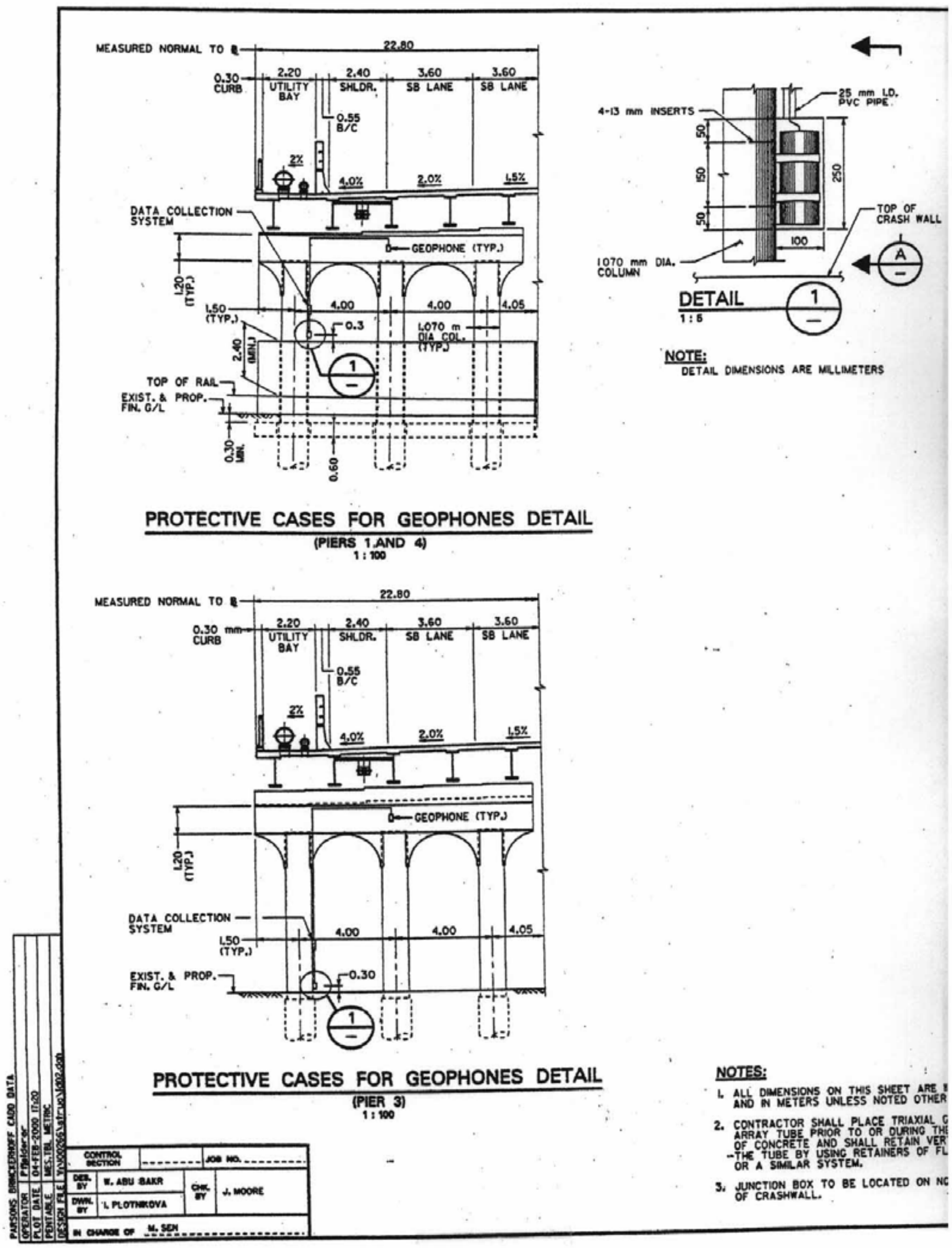


Figure 29. Detail of protective cases for geophones.

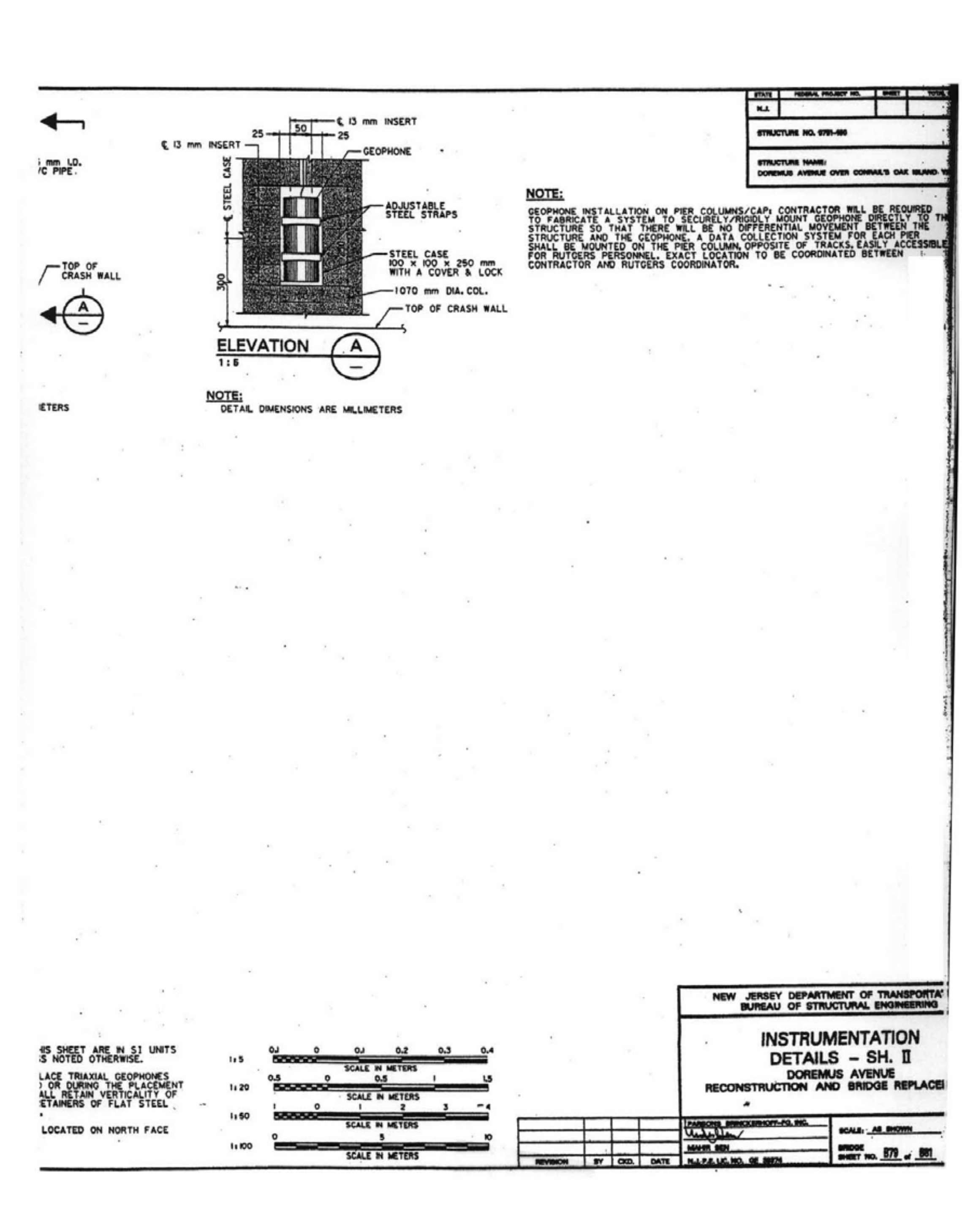


Figure 30. Detail of geophone installation in pier.

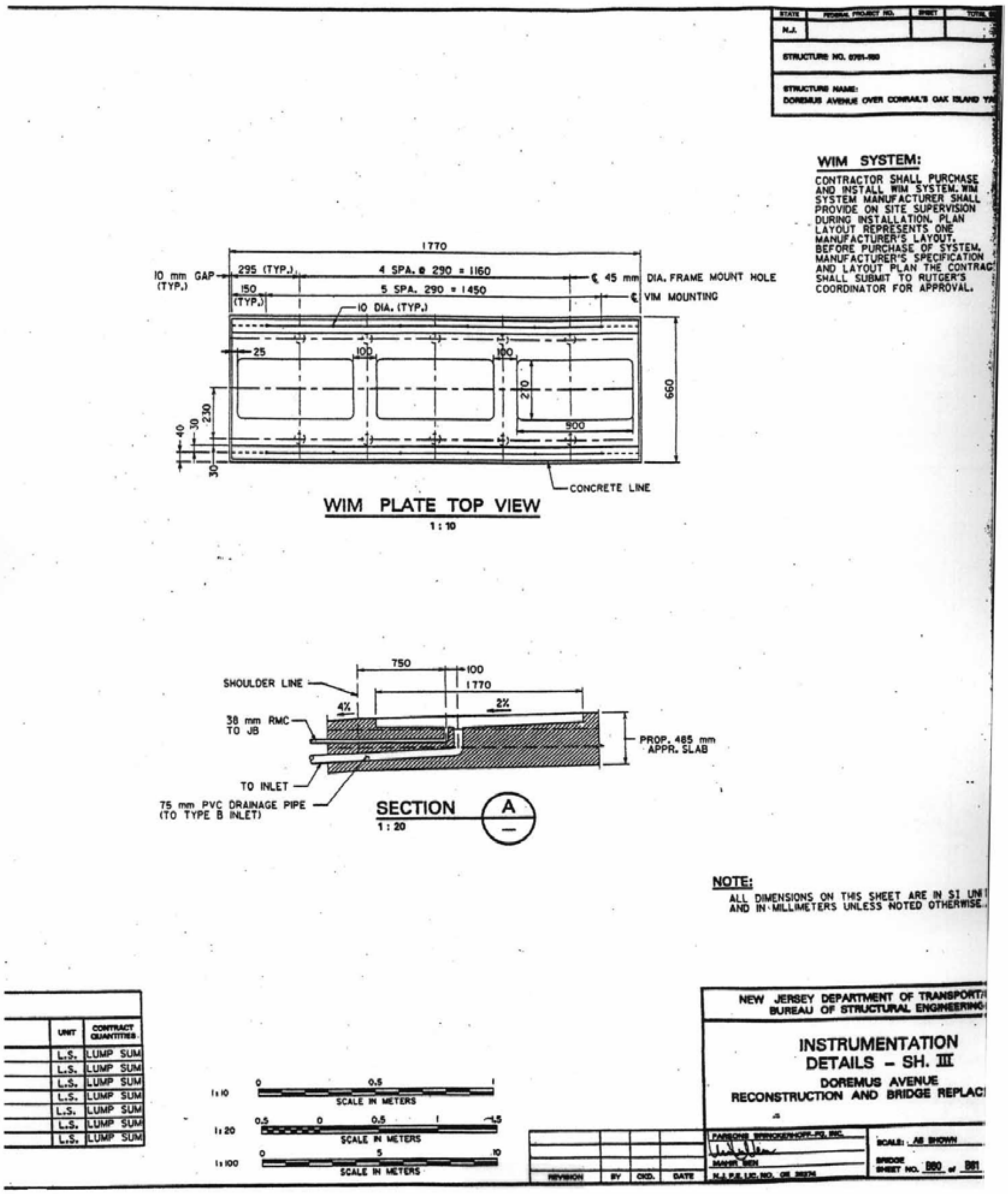


Figure 31. Instrumentation details of WIM plate.

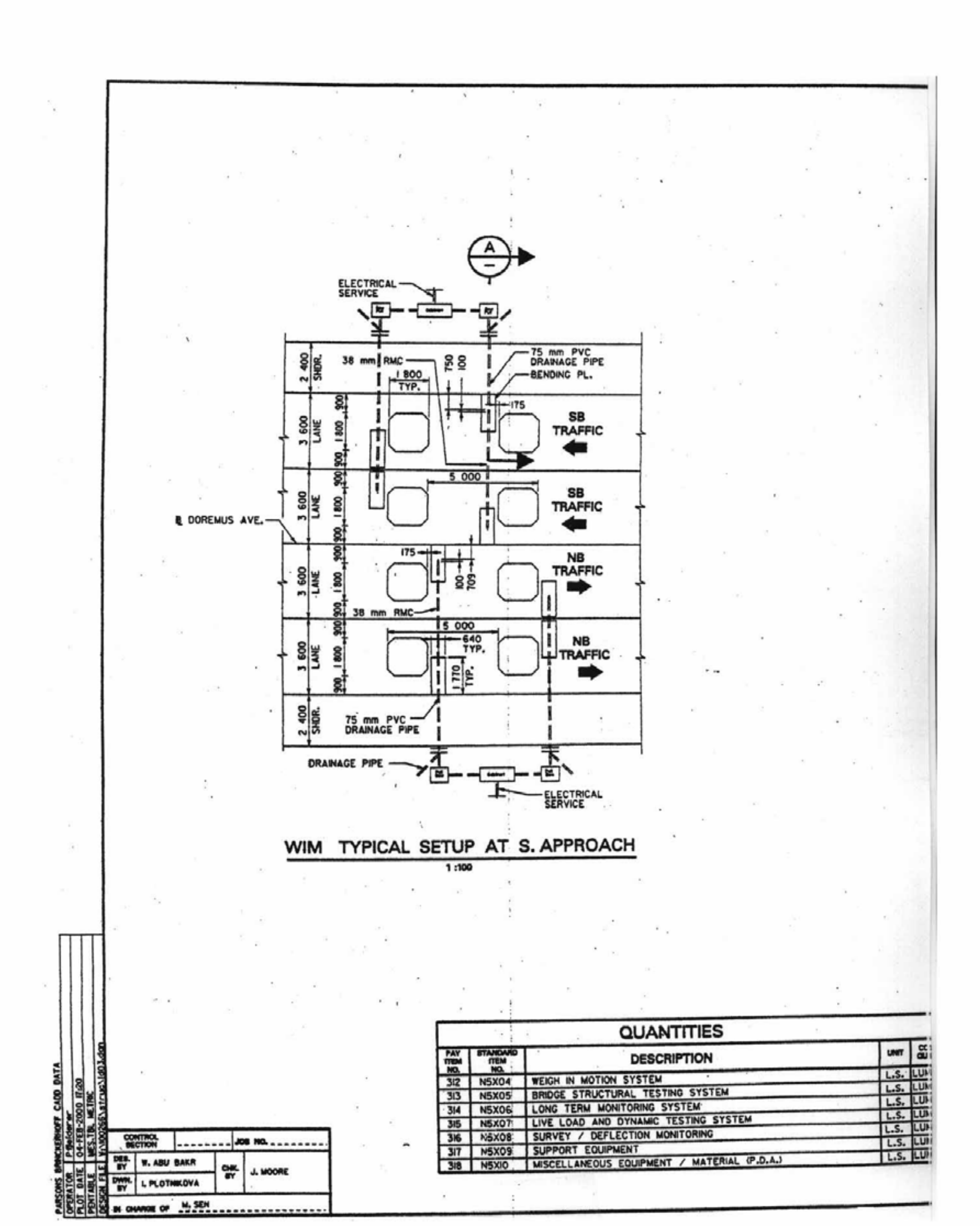


Figure 32. WIM typical setup at south abutment.

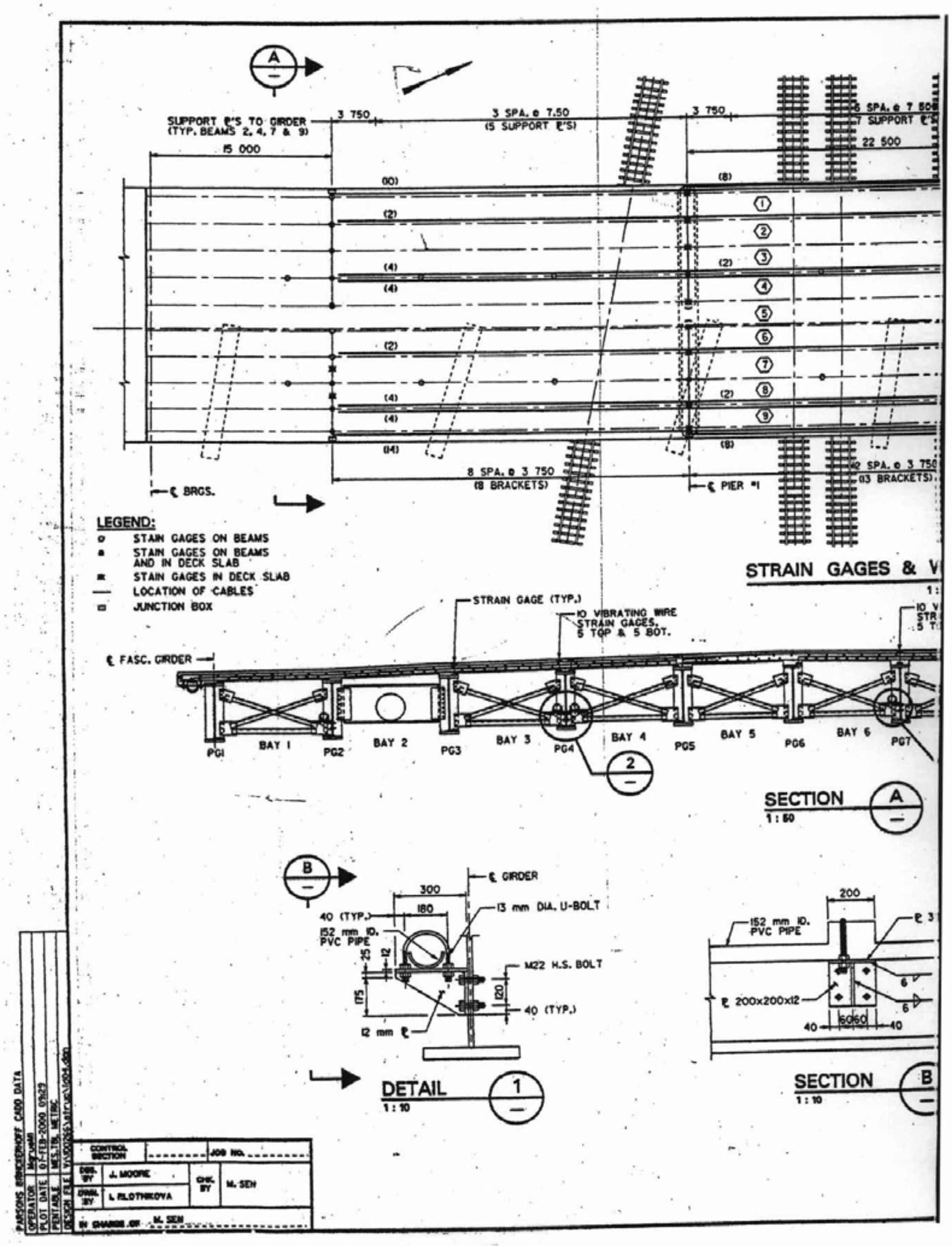


Figure 33. Strain gages and wiring layout plan (part a).

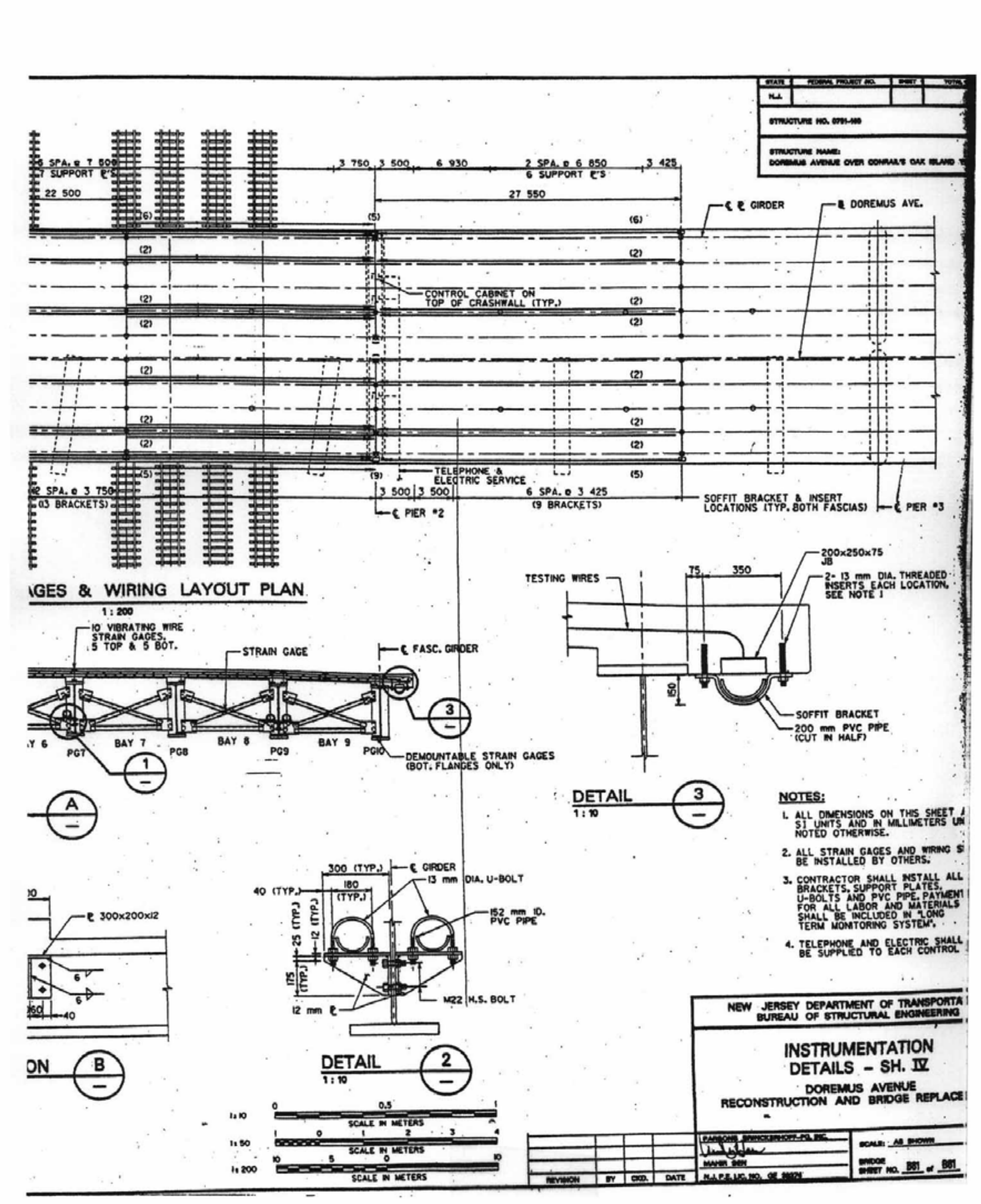


Figure 34. Strain gages and wiring layout plan (part b).

BIBLIOGRAPHY

1. AASHTO, *Standard Specifications for Highway Bridges*. American Association State Highway and Transportation Officials, Washington, D.C., 1994
2. AASHTO-LRFD, *LRFD Specifications for Highway Bridges*. American Association State Highway and Transportation Officials, Washington, D.C.,1988
3. ABAQUS/Standard User's Manual. *Version 6.1, Hibbitt, Karlsson & Sorensen, Inc*. 2000
4. ASTM Standard Designation: D 4428 / D 4428M - 91, *Standard Test Methods for Crosshole Seismic Testing* (Reap proved 1995), pp 604-613
5. Bakht, B., and Pinjarkar, G. "Dynamic Testing of Highway Bridges." In *Transportation Research Record* 1223, 1989, pp 93-100
6. Ballard, R.F. Jr. "Method of Crosshole Seismic Testing" *Journal of Geotechnical Engineering Division, ASCE*, Vol. 102, No. GT 12, (1976), pp 1261-1273
7. Bathe, K.J. and Wilson, E.L. "Stability and Accuracy Analysis of Direct Integration Methods", *J. Earthquake Engrg Struct. Dyna* 1, (1973), pp 283-291
8. Biggs, J.M., Suer, H.S. and Louw, J.M. "Vibration of single Span Highway Bridges" *Transactions, ASCE*, Vol. 124, (1956), pp. 291-318
9. Billing, J.R. *Dynamic Tests of Bridges in Ontario, 1980 Data Capture, Test Procedure, and Data Processing*. Ministry of Transportation and Communication, Research and Development Reports, Vol. 26, 1982
10. Billing, J.R. "Dynamic Loading and Testing of Bridges in Ontario" *Canadian Journal of Civil Engineering*, Vol. 11, No. 4, December, (1984), pp 883-843.
11. Cantieni, R. *Dynamic load tests on highway bridges in Switzerland: 60 years experience of EMPA*. *Tech. Rep. 211, Swiss Federal Lab. for Mat. Testing and Res*., Dubendorf, Switzerland, 1983
12. Cantieni, R. *Dynamic Behavior of Highway Bridges Under the Passage of Heavy Vehicles.* EMPA Report No. 220. EMPA, Dubendorf, Switzerland, 1992
13. Chatterjee, P.K., Datta, T.K., Surana, C.S. "Vibration of Continuous Bridges under Moving Vehicles" *Journal of Sound and Vibration*, Vol. 169, (1994), pp 619-632
14. Curras, C.J., Boulanger, R.W., Kutter, B.L., Wilson, D.W. "Dynamic Experiments and Analysis of a Pile-Group-Supported structure" *Journal of Geotechnical and Geoenvironmental Engineering,* (July 2001), pp 585-596
15. Digital DataMate & DMM Software. Slope Indicator, *Geotechnical & Structural Instrumentation*, May 1994
16. Dobry, R. "Seismic Response of Pile Foundation" *Foundation and Soil Mechanics Group, Metropolitan section of ASCE,* (November 1990), pp 13-14
17. Dobry, R., Vicente, E., O'Rourke, M., and Roesset, J.M. "Horizontal Stiffness and Damping of Single Pile" *Journal of Geotechnical Engineering Division, Vol*. 109, No. 7, (1982), pp 961-974

18. Fancher, H., Ervin, R., MacAdam, C., and Winkler. " Measurement and Representation of the Mechanical properties of truck leaf Springs" *Technical Paper Series 800905, Society of Automotive Engineers*, Warrendale, Pa., 1980
19. Gazetas G. "Seismic Response of End-Bearing Single Pile" *Soil Dynamics and Earthquake Engineering,* Vol. 3, No. 2, (1984), pp 82-93
20. Green, M. F., Cebon, D., and Cole, D.J. "Effects of Vehicle Suspension Design on Dynamics of Highway Bridges", *ASCE, Journal of Structural Engineering*, Vol.121, (1995), pp 272-282
21. Gupta, R.K. and Traill-Nash, R.W. "Bridge Dynamic Loading Due to Road Surface Irregularities and Braking of Vehicle" *Earthquake Eng. Struct. Dynamics*, vol. 8, (1980), pp. 83-96
22. Hall J.F. "Forced Vibration and Earthquake Behavior of an Actual Pile Foundation", *Soil Dynamics and Earthquake Engineering,* Vol. 3, No. 2, (1994), pp 94-101
23. Hawk, H. and Ghali, A. "Dynamic Response of Bridges to Multiple Truck Loading" *Canadian Journal of Civil Engineering,* 8, (1981), pp 392-401
24. Honda, H., Kobori, T., and Yamada, Y. "Dynamic Factor of Highway Steel Girder Bridges" *Proceedings, International Association of bridge and Structural Engineering*, P-98/86, Zurich, Switzerland, (May 1986), pp 57-75.
25. Humar, J.L., and Kashif, A.M. "Dynamic Response of Bridges under Traveling Loads" *Canadian Journal of Civil Engineering*, Vol. 20, 1994, pp 287-298.
26. Hwang, E.S. and Nowak, A.S. "Simulation of Dynamic Load for Bridges", *J. Struct. Engrg. ASCE*, 117(5), 1991, pp 1413-1434.
27. Jaeger, L. G. and B. Bakht. "Multiple Presence Reduction Factors for Bridges" *Proc. Structures Congress, Structural Division, ASCE*, 1987, pp 47-59.
28. Liu, M. *A 3-D Dynamic Model For Bridge-Road-Vehicle System*. Unpublished Mater's Thesis, Bradley University, Peoria, Illinois, 1996, 180p.
29. Makris N., Gazetas G. "Dynamic Pile-Soil-Pile Interaction. Part II: Lateral and Seismic Response" *Earthquake Engineering and Structural Dynamic,* Vol. 21, 1992, pp 145-162
30. Mulcahy, N.L., Pulmano, V.A. and Traill-Nash, R.W. *Dynamic response of bridge decks to vehicle loads by the finite strip approach*. Proc. of 3rd Int. Conf. in Australia on FEM, The University of New South Wales, 1979
31. Nassif, H. H. Liu, M. and Ertekin, A. "Model Validation for Bridge-Road-Vehicle Dynamic Interaction System" *ASCE Journal of Bridge Engineering*, 2002
32. Nassif, H. H. and Nowak, A.S. "Dynamic Load Spectra For Girder Bridges." *Journal of the Transportation Research Board*, No. 1476, TRB, National Research Council, Washington, D.C., 1995, pp 69-83.
33. Nassif, H.H. *Live Load Spectra For Girder Bridges*. Ph.D. Dissertation, The University of Michigan, Department of Civil and Environmental Engineering, 1993, p. 250

34. Newmark, N.M. “A Method of Computation for Structural Dynamics”, *J. Engrg. Mech., ASCE*, 85, 1959, pp 67-94

35. Novak M. “Dynamic Stiffness and Damping of Piles”, *Canadian Geotechnical Journal,* Vol. 11, 1974, pp 574-598

36. Novak M. “Vertical Vibration of Floating Piles”, *Journal of the Engineering Mechanics Division,* ASCE, Vol. 103, No. EM1, 1977

37. Novak M. and El Sharnouby B. ”Stiffness Constants of Single Pile”, *Journal of Geotechnical Engineering,* Vol. 109, No. 7, 1983, pp 961-974

38. O’Connor, C., and Chan, T.H.T. “Dynamic Wheel Loads from Bridges Strains”, *Journal Structural Engineering, ASCE*, Vol. 114, 1988, pp 1703-1723

39. O’Connor, C., and Pritchard, R.W. “Impact Studies on Small Composite Girder Bridge” *Journal Structural Engineering, ASCE*, Vol. 111, 1985, pp 641-653

40. OHBDC. *Ontario Highway Bridge Design Code*. Ontario Ministry of Transportation, Ontario, Canada, 3rd Edition, 1993

41. Paultre, P., Chaalial, O., and Prouix, J. "Bridge Dynamics and Dynamic Amplification Factors - A Review of Analytical and Experimental Findings" *Canadian Journal of Civil Engineering*, Vol. 19, pp 260-278; and discussion Vol. 20, pp 876-878,1992

42. Paultre, P., Prouix, J., and Talbot, M. "Dynamic Testing Procedures for Highway Bridges Using Traffic Loads", *ASCE, Journal of Structural Engineering*, Vol. 121, 1995, pp 362-376

43. Rajapakse R.K.N.D., Shah A.H. “Impedance curves for an elastic pile” *Soil Dynamics and Earthquake Engineering,* Vol. 8, No. 3, 1989, pp 145-152

44. Robertson P.K., Fear C.E. "Borehole Geophysics: Basic Concepts and Geotechnical Engineering Applications", *Geophysical Characterization of Sites,* XIII ICSMFE, 1994, New Delhi, India, 1994, pp 63-67

45. Savard M., Fafard M., Mallikarjuna, and Halchini C. *A Refined 3D Model for Bridge-Vehicle Systems*, 1994

46. Swiss Association for Standardization. SIA 160, *Actions on Structures*. Swiss Society of Engineers and Architects, Zurich, 1989

47. Tabesh A., Poulus H.G. “Pseudostatic Approach for Seismic Analysis of Single Pile” *Journal of Geotechnical and Geoenvironmental Engineering,* Sept. 2001, 2001, pp757-765

48. Ugur, H.E. *Girder Distribution Factors for Skew Steel Girder bridges*. unpublished Master’s Thesis, Bradley University, Peoria, Illinois, 1995, p.157

49. Veletsos, A. S., and Huang, T. “Analysis of Dynamic Response of Highway Bridges” *Journal of the engineering mechanics division, proceedings of ASCE*, Vol. 96, No. EM5, Ref. 35, (October 1970), pp 593-620,

50. Woods R.D. "Borehole Methods in Shallow Seismic Exploration", *Geophysical Characterization of Sites,* XIII ICSMFE, New Delhi, India 1994, pp 91-100

51. Yang, Y.B., and Lin, B.-H. “Vehicle-bridge interaction analysis by dynamic condensation method” *J. Struct. Eng*., 121(11), 1636-1643,” J. Struct. Eng., 1995