# Geotechnical Earthquake Engineering Applications to Bridges

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## ABSTRACT

Seismic geotechnical investigations for a bridge involve several types of analyses, including: establishment of design level rock motions; site characterization and response analyses; ground and foundation motion computations; assessment of liquefaction and its impact on bridge foundations; soil-foundation interaction and impedance calculations; assessment of foundation performance under the design seismic loads; and in the case of existing bridges, if deemed necessary, design of seismic retrofit measures. The outcomes of each of these various investigations have important implications on the assessment of the overall vulnerability of a bridge. Frequently in engineering practice, "conservative" assumptions are employed at various stages of the investigations with the intent of ensuring that uncertainties are accounted for, and to allow the use of simplifying assumptions and analyses. Whereas a certain level of conservatism should be adopted in geotechnical engineering analyses under static loading conditions, a different approach is warranted for seismic loading conditions. In seismic geotechnical analyses, it is often difficult to discern what the impact of a particular assumption will be on the overall assessment of the vulnerability of a bridge. It is likely that in certain situations perceived conservatisms adopted in the seismic investigations may actually lead to an unconservative outcome. Thus, there are inherent *pitfalls of perceived conservatism* in seismic geotechnical investigations. In this paper, through examples of seismic geotechnical analyses of bridges, a case is made in favor of the application of a *rational* approach. The rational approach is to obtain accurate and site-specific geotechnical information, apply the analysis procedures that most accurately model the specific bridge site and foundations, and employ good professional judgment that is based on a thorough understanding of the fundamentals of soil, foundation, and structural dynamics.

## INTRODUCTION

Over the past decade, seismic evaluations of bridges in the northeastern United States have received significant attention. Although the seismic hazard in the eastern United States is lower than that of the west coast, the large inventory of older and historic bridges in the east are nevertheless vulnerable to earthquake damage. AASHTO (1996) prescribed seismic vulnerability studies for all bridges, including those in the northeastern U.S., using a 500-year event, which is associated with a 10% probability of exceedence in 50 years. In 1998, the New York City Department of Transportation adopted seismic guidelines for bridges that use two levels of seismic design for critical bridges, an approach more consistent with the practice on the west coast. Today, a major rehabilitation of an existing bridge or the design of a new bridge in the northeastern U.S. will undergo a comprehensive seismic evaluation.

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The seismic vulnerability assessment of a critical bridge is a major undertaking. Such an investigation may lead to seismic retrofitting of an existing bridge or enhancements in the design of a new bridge, often at considerable cost. Whether a bridge is deemed to be safe against a seismic event depends on the outcomes of a number of critical investigations. Typically, the scope of a seismic evaluation of a critical bridge will involve the following tasks:

1. Investigation of the seismicity of the region and definition of the potential seismic hazard.
2. Selection of rock motions.
3. Analysis of ground motions and soil response, including liquefaction, slope stability, and earth pressures.
4. Soil-foundation interaction analysis and evaluation of foundation springs and dashpots.
5. Foundation-bridge structure interaction analysis.
6. Evaluation of bridge structural vulnerability, including capacity/demand calculations.
7. Evaluation of bridge foundation vulnerability, including stability, capacity/demand, and deformation calculations.
8. Design of retrofit measures for an existing bridge, or enhancement in the design of a new bridge.

The first task described above falls within the field of seismology. Tasks 2 through 4, involve geotechnical earthquake engineering. Finally, tasks 5 through 8 involve collaborative efforts of geotechnical and structural engineers. The successful application of a seismic evaluation of a major bridge will depend upon rational applications of these various interacting tasks. Often, however, the evaluation follows a fragmented approach, with seismological and geotechnical investigations completed separately and the results provided as input into the final bridge analysis. In such cases, each professional conducting his/her own portion of the investigation sometimes attempts to make conservative assessments and computations of parameters to ensure that uncertainties are accounted for, and to permit the use of simplifying assumptions and engineering analyses. However, it is not always true that such conservatism injected at different stages of the overall bridge analysis will ultimately yield an overall conservative assessment of the seismic vulnerability of a bridge. Ironically, it is likely that in certain situations perceived conservatisms adopted in the seismicity investigations or in the geotechnical and structural analyses may lead to an unconservative outcome.

The seismic response of a major facility such as a critical bridge will depend on the dynamic characteristics of the soil-foundation-bridge system, as well as on the nature of the dynamic excitation. The dynamic characteristics of a soil-foundation-bridge system cannot be defined realistically by evaluating the soil (ground), the foundation, and the bridge structure in isolation from each other. In fact, an approach that incorporates apparently conservative decisions in each of the isolated investigations, at times yields unconservative outcomes with regard to the vulnerability of the bridge.

Professor Ralph Peck in 1977 warned against the "Pitfalls of Overconservatism in Geotechnical Engineering", where compounding of conservative assumptions and decisions may lead to prohibitively expensive designs. Extending this notion of Professor Peck, it can be said that in the seismic design analysis of a major structure, the compounding of what are perceived to be conservative assumptions at various stages of the analysis may actually lead to unconservative outcomes. Thus, in earthquake engineering practice there are *pitfalls of perceived conservatism*, and a case needs to be made in favor of the application of *realistic* assumptions and analytical procedures at each step of the seismic analysis of a major structure such as a bridge.

The author has served as a seismic consultant and has been involved in all aspects of the evaluations of many bridges. Based on substantial experience gained from these bridge projects, it is clear that a safe as well as cost-effective new design or retrofit of a bridge require the application of realistic, not conservative, evaluations at every step of the seismic analysis.

This paper presents several case histories of bridge analyses which demonstrate the pitfalls of perceived conservatism in seismic analysis.

Figure 1 presents a general elevation of a hypothetical bridge project. In the figure, three general areas of seismic investigation are identified; namely; 1-seismicity and rock motions; 2-site characterization and ground motions, and 3-foundation performance. The case history examples presented herein fall within these three general areas of investigation.

## SEISMICITY AND ROCK MOTION DETERMINATION

In 1998, the NYCDOT adopted a set of seismic guidelines that provide two levels of rock motions associated with 2500- and 500-year events. Figure 2 shows the acceleration response spectra of the hard rock motions of the two events. The ordinate of the plot in the figure is a measure of the seismic force that a single-degree-of-freedom structure would experience.

The response spectra shown, in effect, define the seismic design level input at outcropping of hard rock. These spectra were established using a probabilistic seismic hazard analysis in which the likelihood of seismic events occurring in the region around New York City, as well as the resulting rock accelerations, were statistically combined. A critical bridge is investigated under both the 2500- and the 500-year events, despite the fact that it may appear at first glance that the 2500-year event, which can induce spectral accelerations about four times larger than the 500-year event, should be the one that controls the design. Associated with these two levels of design motions are two different levels of expected performance for the bridge, and therefore it is not actually obvious which event should govern the design, hence the seismic analysis is done for both levels.

Figure 3a shows a theoretical seismic load versus bridge performance level. Performance level can be defined in terms of bridge movements, or capacity and demand ratios. The stipulation in the NYCDOT seismic guidelines for bridges states that under the 2500-year event the bridge should be safe (no collapse) and repairable within a few days; and under the 500-year event, the bridge should suffer no damage and remain fully operational. The performance criteria under the 500-year event typically also include no cracking of unreinforced masonry, linear structural response, and tolerable movements and permanent displacements in the bridge and its mechanical components, in the case of movable bridges. In an optimal bridge design, the performance criteria for both the 2500-year and the 500-year events are met, as shown in Figure 3a.

Typically, it is not obvious at the start of a bridge investigation which seismic event will be the controlling one. The determination depends on the dynamic response characteristics of the soil-foundation-bridge system, which will define the seismic load levels as well as the bridge performance curve. Figures 3b and 3c show typical situations where either the 2500-year or the 500-year event will control. These figures demonstrate that performing a seismic analysis of a bridge for the 2500-year event alone, which has four times the spectral acceleration of the 500-year event, may not yield a conservative assessment of vulnerability when functionality as well as life safety are required. For this reason, seismic vulnerability is evaluated for both events in the case of critical bridges.

It is noted that the rock spectra provided in the NYCDOT guidelines are applicable to a hard rock site. Hard rock, which is prevalent in the northeastern United States, has a shear wave velocity $V_s$, which is typically larger than 5000 fps. NEHRP (2000) classifies hard rock as Soil Profile A. It is well recognized that seismic waves propagating from hard rock to softer weathered rock can be amplified. For this reason, whenever the rock encountered at a bridge site is considered to be more a soft rock than hard rock (according to NEHRP, where Vs ranges between 2500 and 5000 fps), the NYCDOT seismic guidelines recommend magnification of the hard rock motions by a factor of 1.25. This is similar to the factor 0.8 that is prescribed in NEHRP to convert a soft rock motion intensity to that of a hard rock. Aki and Richards (1980) proposed a simple formulation that describes the amplification of a seismic wave propagating from one medium to another. This amplification ratio is equal to the square-root of the impedance ratios of the two mediums, where impedance is defined as the product of shear wave velocity and mass density of a medium. Based on this formulation, amplification factors of rock motions propagating from a hard rock medium (Vs = 5000 fps, $\gamma$ = 140 pcf) to a softer rock medium ($\gamma$ = 130 pcf) can be computed as a function of rock shear wave velocity. Figure 4 shows the amplification factor that can be used to multiply hard rock motions to account for the softer rock conditions.

From Figure 4 it can be deduced that the amplification factor of NEHRP and NYCDOT is for rock with Vs = 3500 fps. For harder rock with Vs larger than 3500 fps, an amplification factor smaller than 1.25 can be used. Conversely, for softer rock with Vs smaller than 3500 fps, an amplification factor larger than 1.25 would be more appropriate.

In the northeastern United States it is recognized that the quality of the rock, thus the shear wave velocity of the rock, plays an important role in the determination of rock motion intensity and is an important consideration in seismic analysis of bridges.

Accurate determination of the shear wave velocity of rock at a bridge site can be best made using geophysical tests. In the author's experience, the crosshole test conducted at various bridge sites has provided reliable estimates of Vs values of bedrock. The bedrocks that have been encountered ranged in consistency from extremely weathered to very hard rock. In Figure 5, the average measured Vs values are compared with the average RQDs of the rock. The lines in Figure 5 represent the mean and lower and upper bounds of the data.

Shear wave velocity of rock will depend not only on the rock condition (RQD) but also on rock type and other local anomalies that may be present in the bedrock at a particular site. Notwithstanding these factors, the data presented in Figure 5 show a trend in which the Vs of rock decreases with decreasing RQD to a minimum value at about RQD of 60% to 70%. This observation is consistent with variation of rock modulus with RQD that is described in AASHTO. The AASHTO formulation stipulates that rock modulus decreases with RQD to a minimum value of 15% of the intact rock modulus. This translates to a reduction in shear wave velocity by a factor of the square-root of 0.15 equal to 0.39.

The trend and the minimum shear wave velocity range shown in Figure 5 are consistent with this AASHTO formulation. However, the variability in the data is significant indicating that RQD alone is not enough to reliably estimate the shear wave velocity of a rock at a particular bridge site. For example, for an RQD of about 40% to 50% the estimated Vs of rock ranges between 2000 fps and 3000 fps. For this range of Vs, the rock amplification ratio varies from 1.65 to 1.3. Such variability in the amplification ratio can make a critical difference in the extent of the seismic retrofit need of an existing bridge.

In summary, the bedrock encountered at bridge sites in the New York City region varied in consistency from hard to very soft. The shear wave velocity of the bedrock has a very important influence on the intensity of the rock motion that is needed in seismic investigations of a bridge. For critical and essential bridges, in-situ geophysical tests can provide accurate measurements of the shear wave velocity of bedrock.

## SITE CHARACTERIZATION AND GROUND MOTIONS

Once the design level rock motions are established for a bridge, the seismic motions within the soil profile and those that the bridge foundations would experience are computed. In addition, the potential for soil liquefaction, slope instability, and dynamic earth pressures may need to be evaluated, depending on the site conditions. In all of these geotechnical investigations, an accurate assessment of the site conditions and the dynamic soil properties are of paramount importance. In this section, case histories are presented which demonstrate the importance of accurate characterization of a bridge site and the use of realistic models for the computation of ground motions.

To demonstrate the importance of the bedrock profile and of choosing the most appropriate type of ground motion analysis, the Madison Avenue Bridge site in NYC was selected. The Madison Avenue Bridge is a swing bridge with a center pier and two rest piers, one each on the Manhattan and Bronx sides. Figure 6 presents the soil profile at the bridge site, and clearly shows the significant spatial variability in the site conditions. The bedrock elevation changes from about -90 ft. on the Manhattan side to about –10 ft on the Bronx side, within a distance of about 500 ft.

It is well recognized that local site conditions can significantly affect the propagating earthquake motions. In geotechnical earthquake engineering practice, one-dimensional (1-D) wave propagation analysis is typically performed in which a shear wave propagating vertically upward from the base rock to the ground surface is analyzed. To approximately account for spatial variability in site conditions, multiple 1-D analyses are commonly performed for each location of interest, using a soil column that describes the site conditions at that location.

For the Madison Avenue Bridge, this method of accounting for spatial variability in the site conditions was deemed inadequate, considering the sharply dipping bedrock. Ground motions calculated from 1-D analyses of the various bridge pier locations would not have the phase differences associated with the different arrival times of the waves due to the spatially variable geotechnical conditions. For this reason, the finite element procedure was used to determine the influence of the site conditions on the rock motions, and to generate ground motions that were later used in the soil-structure interaction analysis of the bridge.

Figure 7 shows the 2-D finite element mesh used. Selected results are presented in which 1-D and 2-D analyses are compared to demonstrate the importance of the 2-D analysis in estimating the magnitude and spatial variation of the ground motions at the Madison Avenue Bridge location.

Figure 8 presents graphs of peak accelerations and maximum shear strains with depth of soil profile at the location of the Manhattan Rest Pier.

In this figure, comparisons are made between the results of the 1-D and 2-D wave propagation analyses. Clearly, the 1-D analysis underestimates the peak accelerations and the shear strains, particularly within the shallow depth of the soil profile. Such underestimation can have important implications on pile lateral stiffness calculations and liquefaction potential.

Figure 9 illustrates the effect of 1-D versus 2-D analysis on the frequency content of the computed ground motions at the Manhattan Rest Pier and the Center Pier locations. In this figure, the response spectra of the computed motions from the 1-D and 2-D analyses are compared. The results show that the 1-D analysis significantly underestimates the spectral responses, especially in the period range of interest in the bridge analysis (0.6 sec).

To illustrate the importance of 2-D analysis in determining spatially variable ground motions, Figure 10 compares the response spectra of the ground surface motions at the Manhattan Rest Pier with those at the Center Pier. In Figure 10a, a comparison is made between the response spectra of the motions from the 1-D analysis at the two pier locations. As expected, since the soil columns at the locations of the Manhattan Rest Pier and the Center Pier are similar, the 1-D analysis yielded similar results for the two piers. Thus, if multiple 1-D analyses were selected to determine the ground motions at these two pier locations, the two piers would be assigned identical motions, i.e. there would be no spatial variability. However, the 2-D analysis results shown in Figure 10b clearly capture the significant differences in the response spectra at the two pier locations.

Hence, spatial variability in ground motions due to geotechnical site conditions can be significant, even for relatively short span bridges. In such cases, two-dimensional wave propagation analysis can yield more realistic ground motions than the multiple 1-D analyses commonly performed. In the case of the Madison Avenue Bridge, 1-D analyses would have underestimated the earthquake effects on the bridge.

To demonstrate the importance of accurately determining the shear wave velocities of the soils for use in the ground motion and bridge analyses, the case of the Third Avenue Bridge over the Harlem River in NYC is presented. One of the most important soil properties used in a dynamic site response analysis is the shear wave velocity, Vs, of the various soil layers and of the bedrock encountered in a subsurface profile. In geotechnical engineering practice, empirical procedures are often employed that can provide estimates of shear wave velocities for different soils. However, the results of such procedures can be highly uncertain or erroneous. More reliable estimates of shear wave velocities are obtained using field geophysical tests.

One commonly used procedure is the crosshole test, which provides accurate measurements of shear and compressive wave velocities with depth of soil profile. In addition, the crosshole test can be used to measure both the shear and compressive wave velocities of bedrock, parameters that are essential in determining the characteristics of the base rock motion, as was described in the previous section. For these reasons, crosshole tests were conducted at the Third Avenue Bridge site.

Figure 11 shows the subsurface soil profile at the location of the crosshole test and the SPT N-values recorded. Included in the figure are the shear wave velocity measurements obtained from the crosshole test. For purposes of comparison, the Vs values for the soils at the site were also computed using the SPT-N values and the empirical procedures of Sykora and Koester (1988) and Seed et al. (1986). Clearly, the empirical procedures for this site overestimate the shear wave velocities of the soils by a factor of 1.5 to 2. The overestimation is most likely due to the presence of some gravel in the sand layer. In other bridge sites where the silt content is high in the sands, the resulting smaller N-values have led to underestimation of the Vs values. The question that is raised is whether the use of the empirically-estimated higher values of Vs instead of the crosshole values would have led to conservative or unconservative seismic loads.

Figure 12 shows the response spectrum of the free-field motion that was computed using the crosshole-measured Vs values. This motion was subsequently used as input in the seismic analysis of the bridge. Included in Figure 12 is the response spectrum of the motion computed using the empirically-estimated Vs values. There are significant differences between the two spectra. In the period range of a single-degree-of-freedom structure having a period smaller than 0.65 sec, the spectral ordinates (thus, the seismic loads) based on empirically-estimated Vs values are much larger than those obtained using the crosshole-measured Vs values. The reverse trend is observed for periods greater than 0.65. The period range of importance for the bridge, including the higher modes of vibrations, was between approximately 0.5 and 1 seconds. Within this period range the empirically-based Vs values both underestimate and overestimate the spectral accelerations. Hence, using empirically-based Vs values may lead to either conservative or unconservative seismic loads, depending on the site conditions, the bridge dynamic characteristics, and the seismic input motion at the bedrock level. These factors cannot be evaluated in a cursory manner at the start of a project to determine whether in-situ measurement of Vs is essential or not.

This example clearly demonstrates that for an important bridge project, accurate and realistic measurements of dynamic soil and rock properties are required in order to arrive at a realistic assessment of seismic vulnerability.

To demonstrate the importance of establishing realistic seismic input motions to a bridge analysis, the case of the New Woodrow Wilson Bridge in Washington, D.C. is presented. Figure 13 presents the soil profile along the bridge axis.

Although the seismicity of the region is modest, seismic issues were thoroughly addressed in the design of the bridge. The subsurface soil profile consists of about 50 ft of soft or organic clay underlain with a deep deposit of hard sandy clay. The soft clay is vulnerable to significant scour, as shown in Figure 13. The piers of the bridge over the water are founded on frictional and partially-bearing piles. Figure 14 shows the foundation details of the bascule span pier (M1), where the scour potential is high.

Figure 15 shows the pile foundation of a typical pier located on land (V4). In this region, the pile cap will be about 10 ft below the ground surface and scour is not a concern.

Very often in engineering practice when a pile cap of a bridge pier is embedded a few feet in the ground, the seismic motion at the cap level is approximated by computing the seismic motion in the free field away from the influence of the bridge. Typically, the motion that the cap would experience is slightly smaller than the free-field motion due to the "kinematic effect." This practice is considered to be a conservative assumption in the estimation of the seismic loads on the bridge. In the case of the Woodrow Wilson Bridge, the M1 pile cap is above the mudline, and V4 pile cap is in the ground. In the bridge analysis, the motions at the base of the pile cap were computed using three-dimensional soil-pile interaction analyses (employing the computer program ACS-SASSI), and then specified as input in the bridge analysis. Below, comparisons are made between the spectra of the motions computed at the pile cap levels and those in the free field that the "conservative" approach described above would yield.

Figure 14 shows the three-dimensional soil-pile model used in the analysis of the pile foundation of Pier M1. The computed spectrum for the M1 pile cap is shown in Figure 16. A comparison of the free-field and cap-base spectra shown in Figure 16 reveals that there is little difference between the spectra for periods greater than about 0.6 sec. In the period range of 0.1 to 0.4 sec, there is a significant increase in the spectral accelerations at the base of the pile cap compared to that of the free field.

Figure 17 shows the results from the three-dimensional analysis of the pier foundation V4, where the pile cap is on land. It is noted that the response of this pier is quite different from that of pier M1, where the pile cap is in the water. The motion at the cap base is significantly smaller than the free-field motion. Hence, the use of the free-field motions for Pier V4, as is typically done in practice, would have grossly overestimated the spectral accelerations in the period range smaller than 0.4 sec.

In summary, the results of the soil-pile interaction analyses show that the cap-base motions, particularly in the period range of the soil-pile system, can be significantly different. For foundations with long, unsupported piles penetrating soft clays to a hard stratum, the cap-base motion can be significantly larger than the free-field motion. For embedded pile caps, the cap-base motion can be appreciably smaller than the free-field motion. Clearly, the process of determining the impact of a perceived conservatism (in this case use of free-field motions) on the overall seismic vulnerability of a bridge is not straightforward.

The effect of scour was also investigated in the Woodrow Wilson Bridge analysis. In a static design situation one could convincingly argue that a bridge under static loads is more vulnerable with scour being realized during the life of the structure than without scour. However, under seismic excitations the effect of scour may not be so straightforward. To demonstrate this, the cap-base motions of Pier M1 were computed for both scour and no-scour conditions. The design response spectra established using the computed cap-base motions are compared in Figure 18.

As demonstrated in the figure, the effect of scour is to increase the design level accelerations for the period range smaller than 0.3 sec, and to decrease the spectral acceleration for periods larger than 0.3 sec. This effect is only on the cap-base motions. An additional effect of scour is on the stiffness of the soil-pile system. The longitudinal stiffness of the Pier M1 foundation with no scour was estimated to be about 1.33E+04 k/ft. With scour, this stiffness was reduced to 8.86E+3 k/ft. Such reduction in stiffness led to a slightly longer fundamental period of the bridge, and therefore smaller spectral accelerations.

It is noted that the period range of importance in the bridge analysis was greater than 1.0 sec. For the superstructure of the New Woodrow Wilson Bridge, the no-scour conditions where the seismic loads were larger generally controlled. For the foundation design where the pile loads, pile-cap displacements and depth of concrete fill in the pipe piles were of concern, the scour conditions controlled.

Thus, the Woodrow Wilson Bridge project examples demonstrate that what appear to be conservative assumptions, i.e. the use of free-field motions and consideration of scour conditions, may in fact not be conservative. Good engineering requires a more rational approach where the perceived conservative assumptions are avoided, and state-of-the-art engineering practice is followed.

## FOUNDATION RESPONSE

To illustrate the importance of performing realistic seismic geotechnical analysis of bridge foundations, the case of the Roosevelt Island Bridge is presented. Figure 19 shows one of the important piers of the bridge that is founded on a large cap (mostly consisting of tremie concrete) resting on steel H piles.

### Pile Cap Motion

As was stated earlier, when a pile cap of a bridge pier is embedded a few feet in the ground, the seismic motion at the pile cap level is approximated by computing the seismic motion in the free field away from the influence of the bridge. In the case of Pier E1, the pile cap is very large and is deeply embedded. In such a situation, the pile cap motion can be significantly different from the free-field motion.

To evaluate the effect of the soil-pile system on the motion of the pile cap and to compare it with the free-field motion, three-dimensional seismic analysis of the foundation of Pier E1 was performed using the computer program ACS-SASSI. Figure 20 shows the finite element model of the pile cap system. The soil layers are not shown in the figure because program considers the soil layers to extend horizontally starting at the nodal points that are common to the structure, the piles, and the soils. The shear wave velocities of the soils corresponded to the strain-compatible values that were computed from the ground motion analysis of the soil profile at Pier E1.

Figure 21 shows a comparison of the acceleration spectrum of the motion at the bottom of the cap computed from the 3-D analysis with the spectrum of the motion in the free field at the elevation of the bottom of the pile cap. In this case, the difference in the spectra was small and hence the use of free-field motions in the seismic analysis of the bridge was justified. Also, since the motions at different pier foundation levels were demonstrated to be very similar, the seismic analysis of the bridge was performed using a uniform motion at all its supports.

Kinematically Induced Pile Loads

During seismic shaking, the pile group of Pier E1 will undergo deformations induced by the bridge inertial forces as well as by the soil strains associated with the propagation of the ground motion. The pile shear forces and bending moments induced by the ground motion are often referred to as kinematically induced pile loads. These loads can be significant when piles are in a layered soil profile where large differences exist between the layer stiffnesses. In the case of Pier E1, not only there is significant contrast in the impedance of the soil and bedrock in which the piles are socketed, but also the piles are anchored in the deeply embedded pile cap that has a large lateral stiffness due to the surrounding soil.

The shear forces and bending moments induced in the piles by the soil motion (kinematic effect) were computed using the 3-D SASSI analysis that was described earlier. The results are presented in Figure 22. Included in the figure are the shear forces and bending moments in the piles that are induced by the seismic inertial loads from the bridge. It is noted that the kinematically-induced maximum shear force in a pile is 4.4 k compared to the 20 k that is induced by the bridge inertia. Similarly, the maximum bending moment in a pile induced by the soil motion is about 18 k-ft compared to 39 k-ft that is induced by the bridge inertia. Thus, when assessing the adequacy of the piles of Pier E1 under the 2500-year event, the kinematically induced pile loads were included with those induced by the inertia of the bridge and its pile cap.

Foundation Stiffness

In the seismic analysis of a bridge, the soil-foundation system is typically represented through the use of stiffness and damping coefficients (foundation impedances). The pile cap forces and moments computed through the seismic analysis are then used to assess the adequacy of the foundations with respect to load capacities and tolerable deformations. The seismic loads that a bridge foundation may experience from the bridge sub- and super-structures will depend, among many other input parameters, on the foundation impedances.

In engineering practice, in the calculations of the stiffness of a pile group the contribution to this stiffness by the sides of the pile cap is frequently ignored. This practice likely stems from the reluctance in static design to rely on passive resistance (in case in the future it may not exist, or because mobilizing full passive resistance can require deformations that may not be achieved under the design loads). However, under dynamic loads, when a pile cap is rather large and deeply embedded, the contribution of the pile cap sides to the overall foundation stiffness and damping can be significant. A stiffer pile cap may also attract much larger seismic loads. Hence, sides of a pile cap can have an important effect on the overall seismic performance of the foundations of a bridge.

To demonstrate the importance of realistic computation of foundation stiffness, again the pile cap of Pier E1 of the Roosevelt Island Bridge is selected for evaluation.

Figure 23 presents the results of the stiffness calculations showing the contributions to the overall stiffness by the piles as well as by the sides of the pile cap. The results in Figure 23 clearly demonstrate the effect of soil nonlinear behavior on the foundation stiffness. Also, it is evident that when the seismic lateral force on the pile cap is relatively small, (typically associated with the 500-year event), the pile cap contribution to the overall stiffness is also small. When the seismic force on the pile cap is large, the pile and the pile cap contributions to the overall foundation stiffness are comparable.

Under the 2500-year event, the soil-foundation-bridge system had a fundamental period of about 0.5 sec and the pier experienced an average spectral acceleration of about 0.3g, as illustrated in Figure 24. If the contribution of the sides of the pile cap to the overall stiffness of the foundation were ignored, the foundation of the pier would have been more flexible, thus experiencing a smaller spectral acceleration of about 0.2 g. It appears, therefore, that underestimation of the foundation stiffness by ignoring cap-side stiffness, leads to a corresponding underestimation of the pile cap load.

Figure 25 shows the results of the analyses of the pile responses using the seismic loads for both conditions: *with* cap-side stiffness, and *without* cap-side stiffness. The results show that while ignoring the cap-side contribution underestimates the stiffness and hence the seismic loads, the result of the smaller stiffness around the pile cap is that the pile cap deflections and pile bending moments are larger by about 50%.

Hence, in a bridge project very often it would be very difficult, to predict the result of an underestimation or overestimation of foundation stiffness with respect to foundation performance. The rational approach would be to use good soil and foundation information, reliable analytical procedures and good judgment that is not impaired with perceived conservative assumptions and short cuts.

## SUMMARY

Geotechnical earthquake engineering plays an important role in the overall seismic safety analysis of a bridge. The scope of a seismic geotechnical investigation for a bridge may include: establishment of design level rock motion; site characterization and response analyses; ground and foundation motion computations; assessment of liquefaction and its impact on bridge foundations; soil-foundation interaction and impedance calculations; assessment of foundation performance under the design seismic loads; and in the case of existing bridges, if deemed necessary, design of seismic retrofit measures. The outcomes of each of these various investigations can have important implications on the assessment of the overall vulnerability of a bridge. Whereas a certain level of conservatism can be adopted to account for uncertainties and simplified analytical procedures used under static loads, a different approach is required for seismic loads. Under seismic loads, it is often unclear what the implications may be of a particular assumption upon the overall assessment of the vulnerability of a bridge.

To illustrate the potential pitfalls of perceived conservatism in seismic analysis of bridges, case history examples of bridges were presented. It was demonstrated that although a 2500-year seismic event may induce much larger seismic forces on a bridge than a 500-year event, the 500-year event is sometimes more critical. Associated with the two levels of seismic design are different acceptable performance levels for a bridge. The requirement that a bridge be functional during a 500-year event can make this smaller event the one of critical importance, depending on the soil-foundation-bridge system that is being analyzed. Therefore, seismic analyses of important bridges in the northeastern U.S. are performed for both the 2500-year and the 500-year events.

In the case history example of the Madison Avenue Bridge, the 1-D ground motion analysis results compared with those of 2-D analyses *under*-predicted the overall intensity of the motion at the site, and failed to capture the differences in the motions at the different bridge pier locations. In situations where there is significant spatial variation in the site conditions, including sharply dipping bedrock, 2-D ground motion analysis yields more realistic results than the 1-D analysis that is commonly used in practice.

The importance of using reliable estimates of shear wave velocities, Vs of the soils and bedrock at a bridge site was demonstrated in the case of the Third Avenue Bridge. The acceleration response spectrum of the ground motion computed using the Vs values that were accurately measured with a crosshole test was compared with that obtained using Vs values estimated using empirically-based formulas. The empirically-based Vs values yielded both overestimated and underestimated spectral values, depending on the period range selected. Reliable determination of ground motions can be made through the use of in-situ measured shear wave velocities.

The example of the New Woodrow Wilson Bridge illustrated the importance of realistically computing earthquake motions for input in the seismic analysis of a bridge. The seismic motions of a pile cap in water (above the mudline) can be significantly larger than the free-field ground motions. Conversely, pile caps in the ground can experience motions much smaller than those in the free field, which are often used as conservative estimates of the seismic motion to the bridge. The effect of scour upon the overall performance of a bridge and its foundation cannot be easily determined at the outset of a project. In the case of the Woodrow Wilson Bridge, both conditions were investigated.

The importance of realistically estimating foundation stiffness was demonstrated through the example of the Roosevelt Island Bridge. Underestimation of stiffness may lead to underestimation of seismic loads on a pile cap. However, even under these smaller loads, the smaller stiffness can result in the overestimation of the pile deflections and bending moments. Thus, the effect of underestimation or overestimation of stiffness on the pile responses can have unpredictable results. The rational approach is to use accurate soil and foundation information, and to use realistic models to calculate reliable values of the foundation stiffness and damping coefficients.

These examples demonstrate that what may appear to be conservative approaches in the seismic geotechnical investigation of a bridge may ultimately yield unconservative results. In many instances it may not be readily obvious what the implications of a given assumption might be on the final outcome of the seismic vulnerability of the bridge. The rational approach is to obtain accurate and site-specific geotechnical information, apply the analysis procedures that most accurately model the specific bridge site and foundations, and employ good professional judgment that is based on a thorough understanding of the fundamentals of soil, foundation, and structural dynamics.

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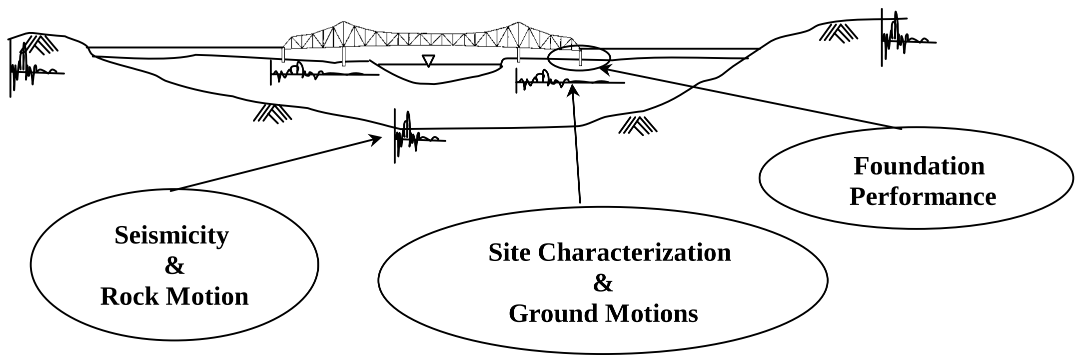



**Figure 1.** **Three general areas of seismic geotechnical investigations discussed in the paper.**

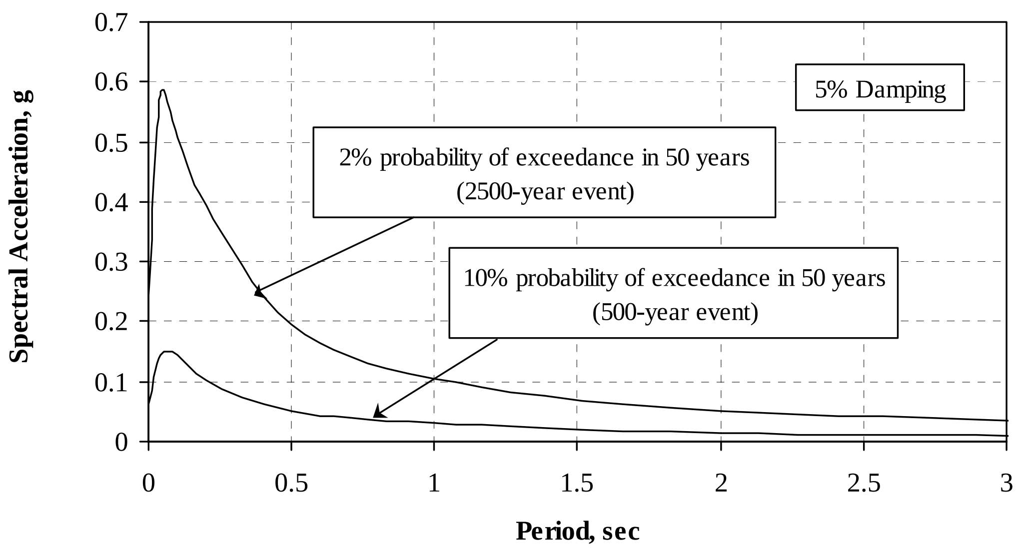



**Figure 2.** **Hard rock spectra provided in the 1998 NYCDOT seismic guidelines.**

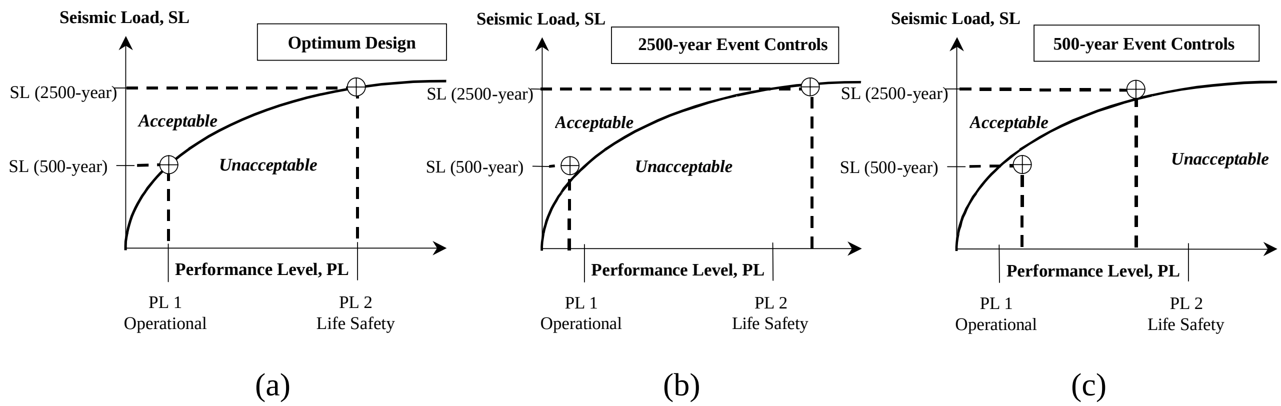



**Figure 3.** **Seismic event that controls the design based on the expected performance levels, (a) optimum design, (b) the 2500-year event controls, (c) the 500-year event controls.**

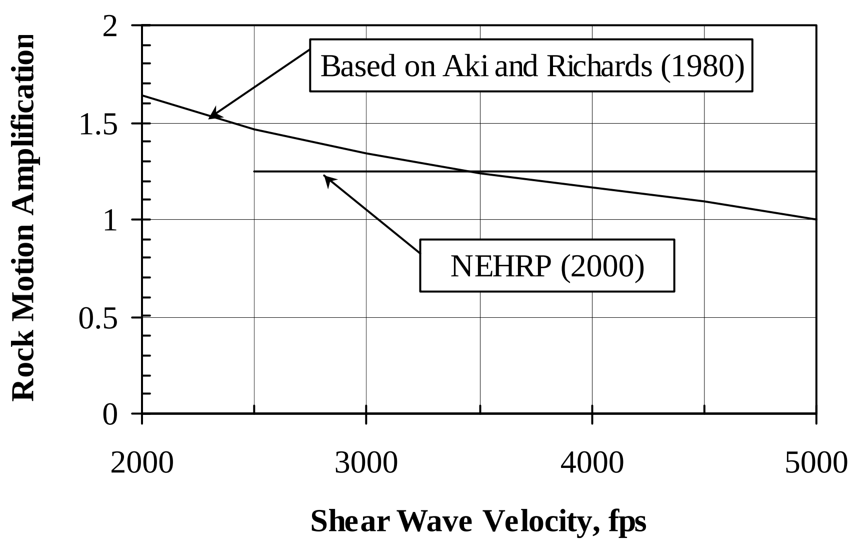



**Figure 4.** **Rock motion amplification from hard rock (Vs = 5000 fps) to softer rock.**

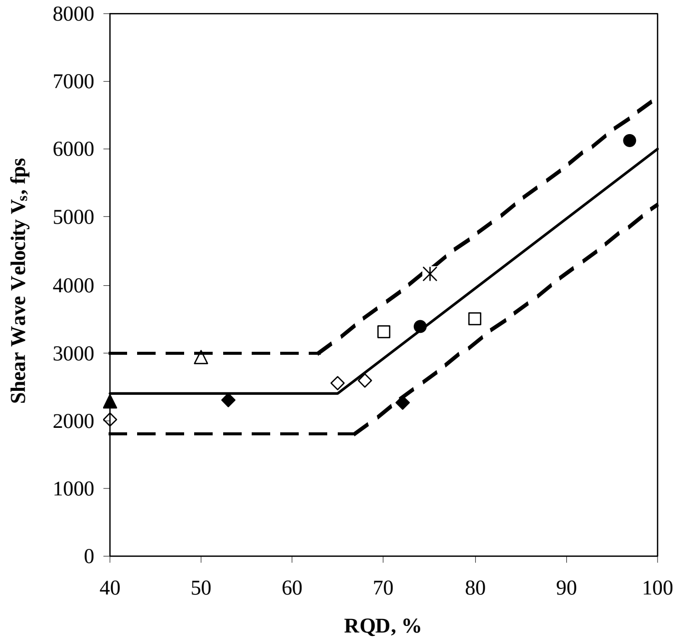



**Figure 5.** **Measured shear wave velocities of bedrock related to the average RQD of the rock.**

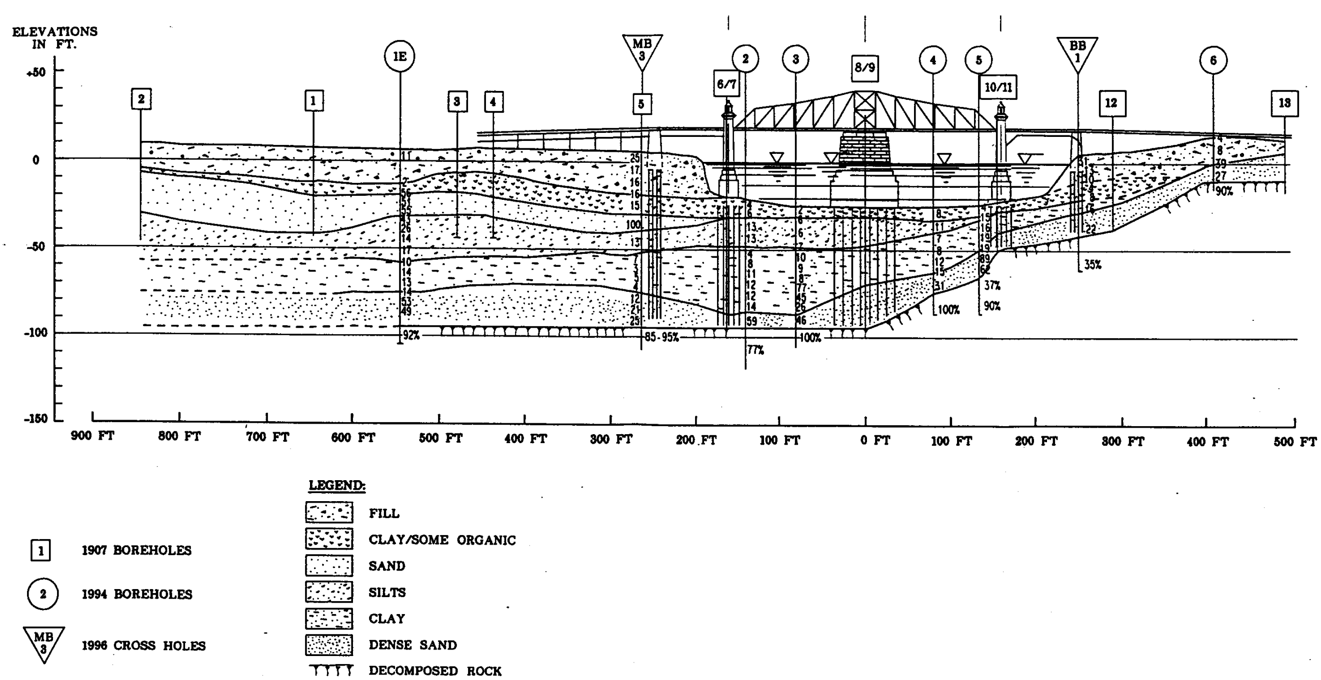
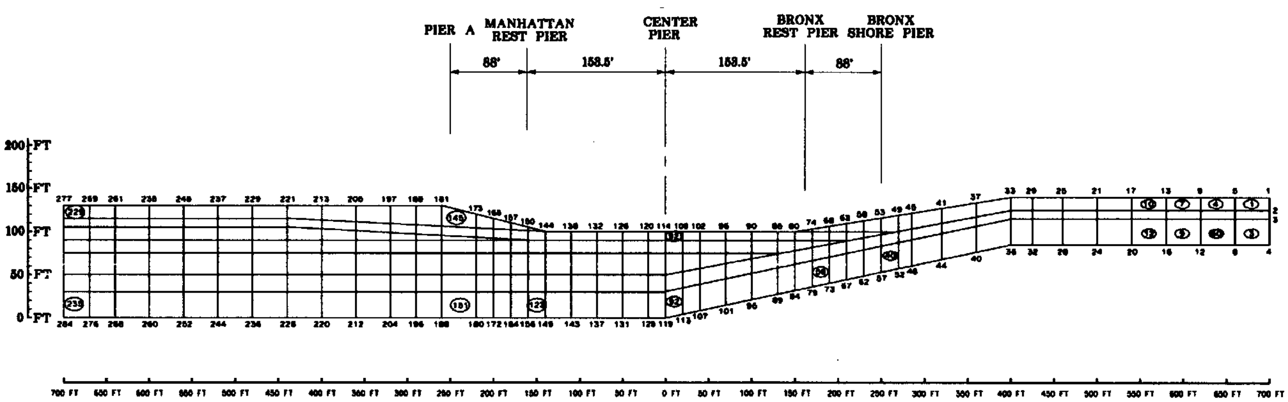



**Figure 6.** **The soil profile and elevation of the Madison Avenue Bridge.**




**LEGEND:**

- INDICATES FINITE ELEMENT NUMBER

- OTHER NUMBERS OF FINITE ELEMENT MESH INDICATE NODES

**Figure 7.** **The finite element mesh used in the ground motion analysis of the Madison Avenue Bridge.**

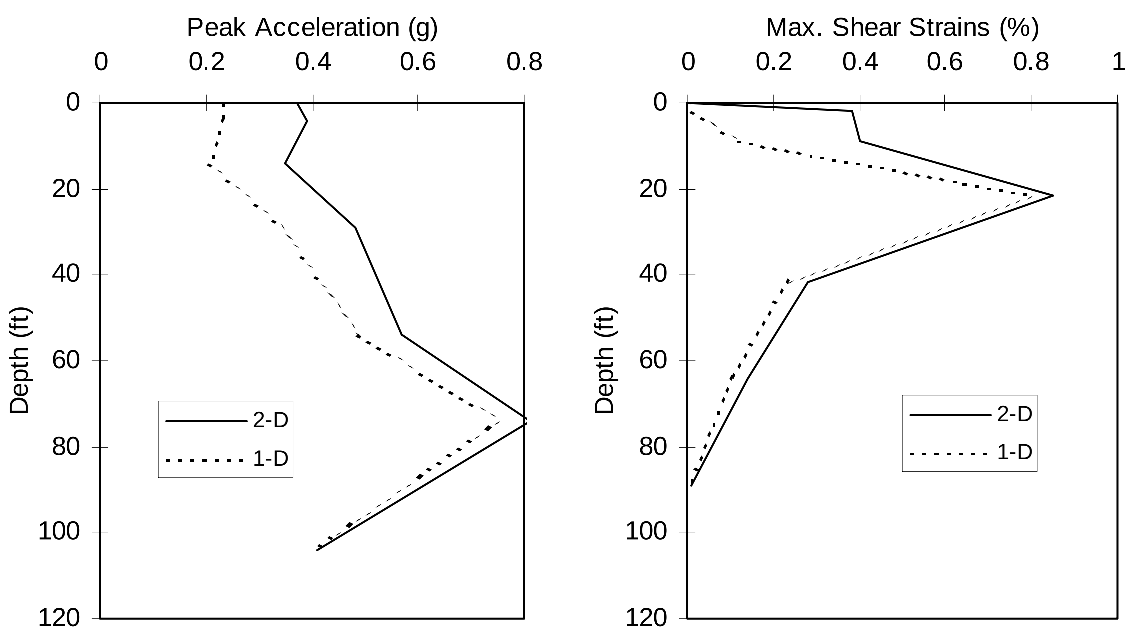
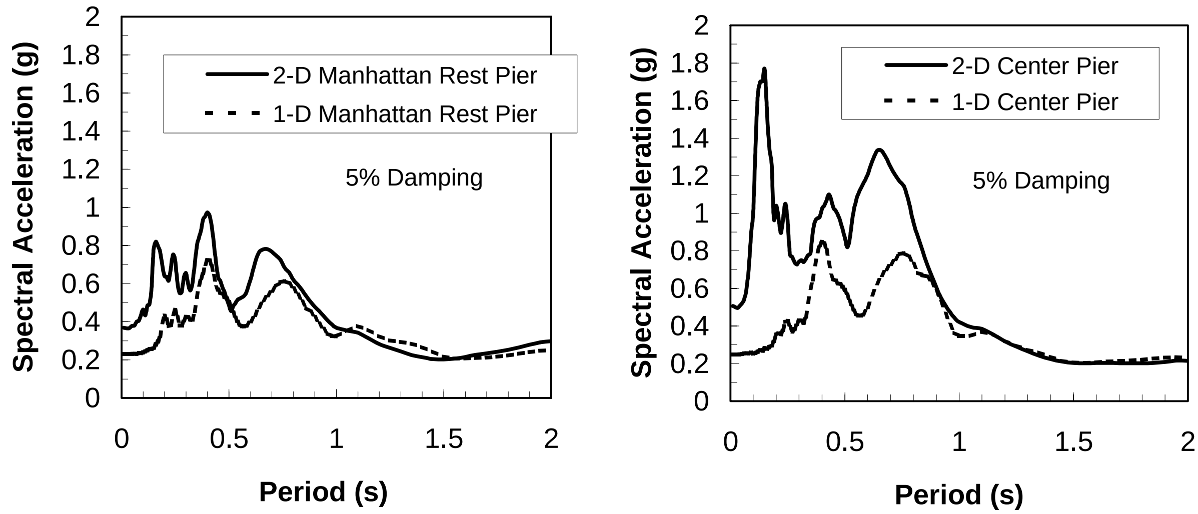
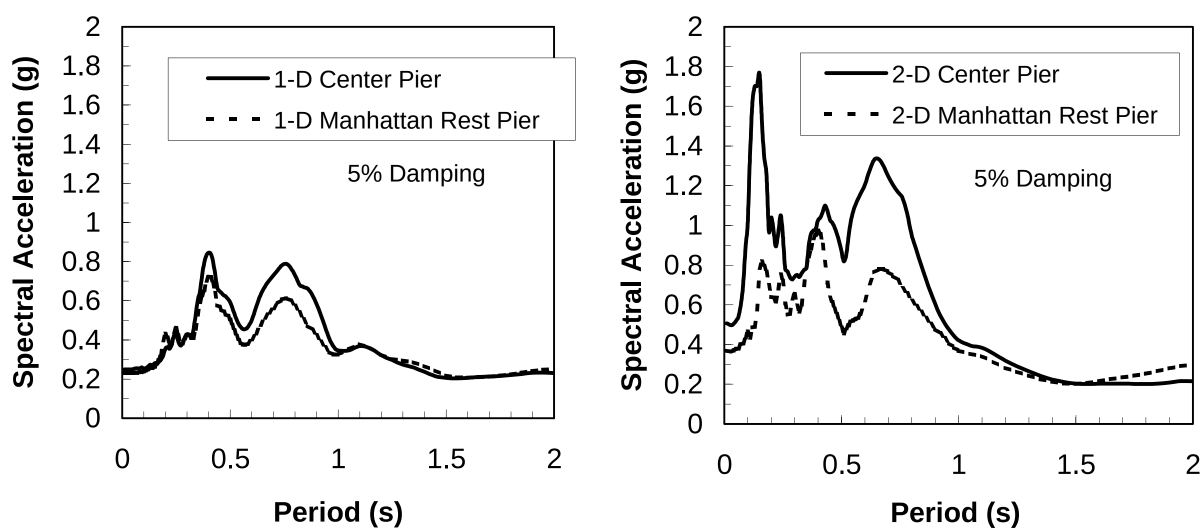



**Figure 8.** **Comparisons of the peak accelerations and shear strains from 1-D and 2-D ground motion analyses.**




(a) (b)

**Figure 9.** **Comparisons of the spectra from 1-D and 2-D ground motion analyses, (a) Manhattan Rest Pier, and (b) Center Pier.**




(a) (b)

**Figure 10.** **Comparisons of the spectra for the Center and Manhattan Rest Piers from (a) 1-D, and (b) 2-D ground motion analyses.**

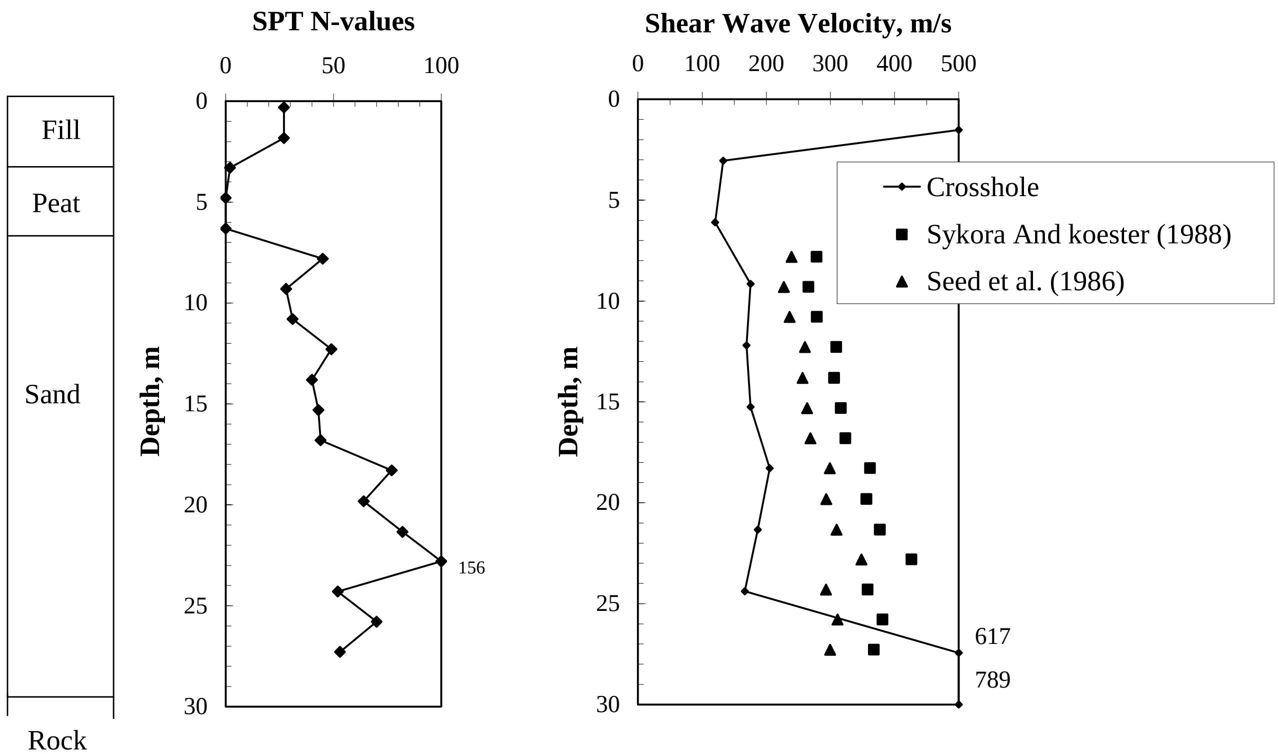



**Figure 11. Comparisons of measured and estimated shear wave velocities.**

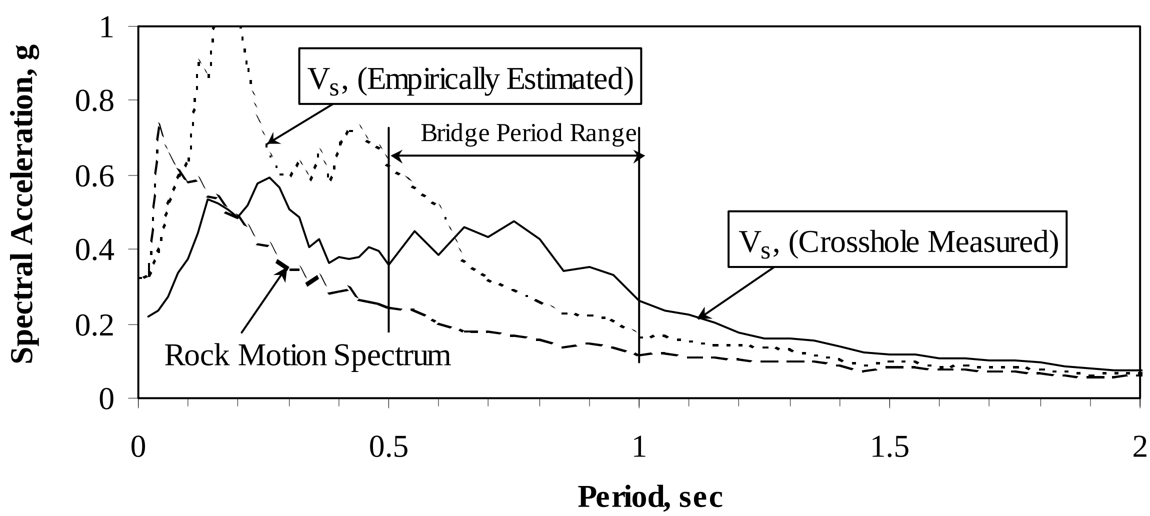



**Figure 12. Comparison of the response spectra from ground motion analyses, using the crosshole-measured, and empirically-estimated shear wave velocities.**

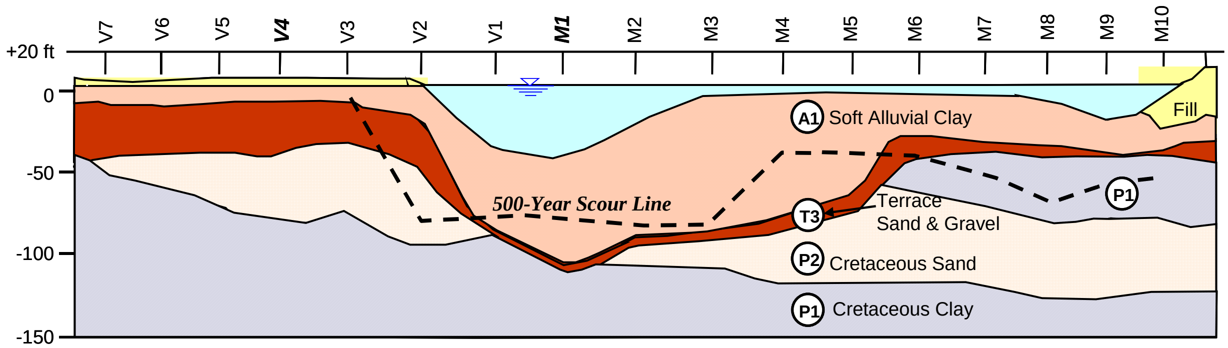



**Figure 13. The soil profile at the site of the New Woodrow Wilson Bridge.**

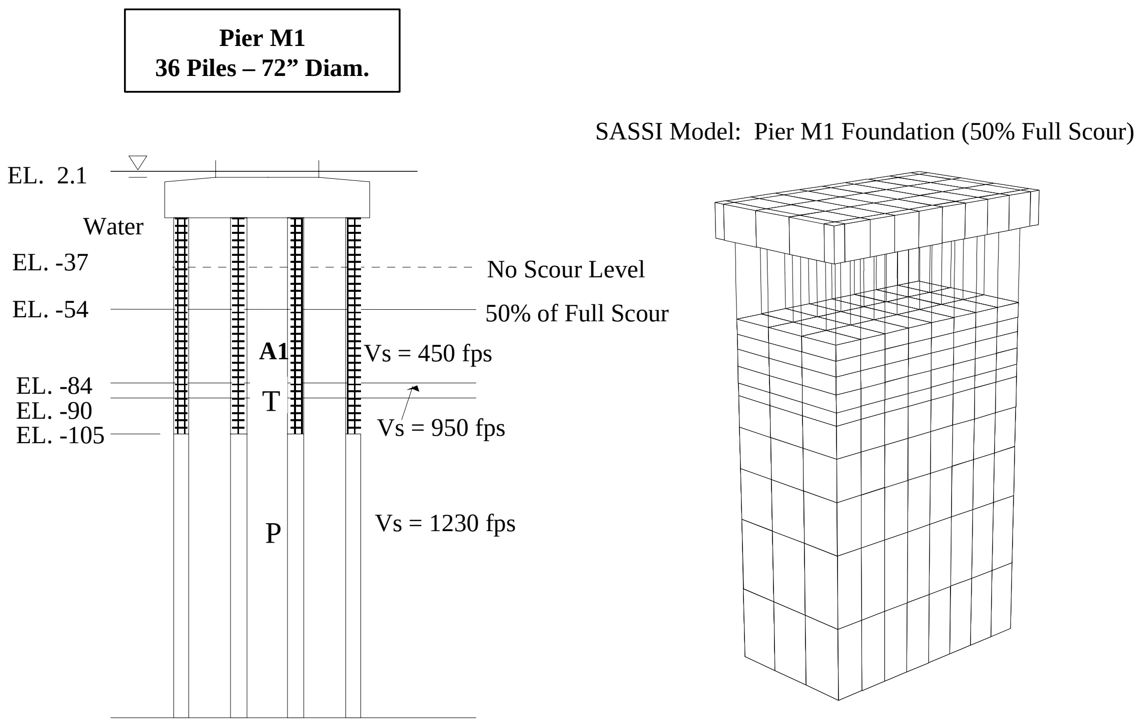



**Figure 14. Pile foundation of Pier M1 and the finite element model used in the 3-D seismic analysis.**

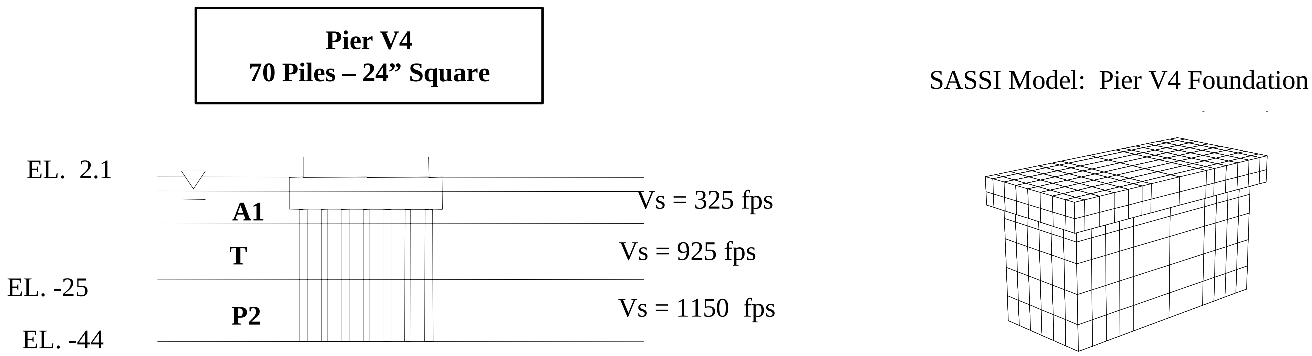



**Figure 15.** **Pile foundation of Pier V4 and the finite element model used in the 3-D seismic analysis.**

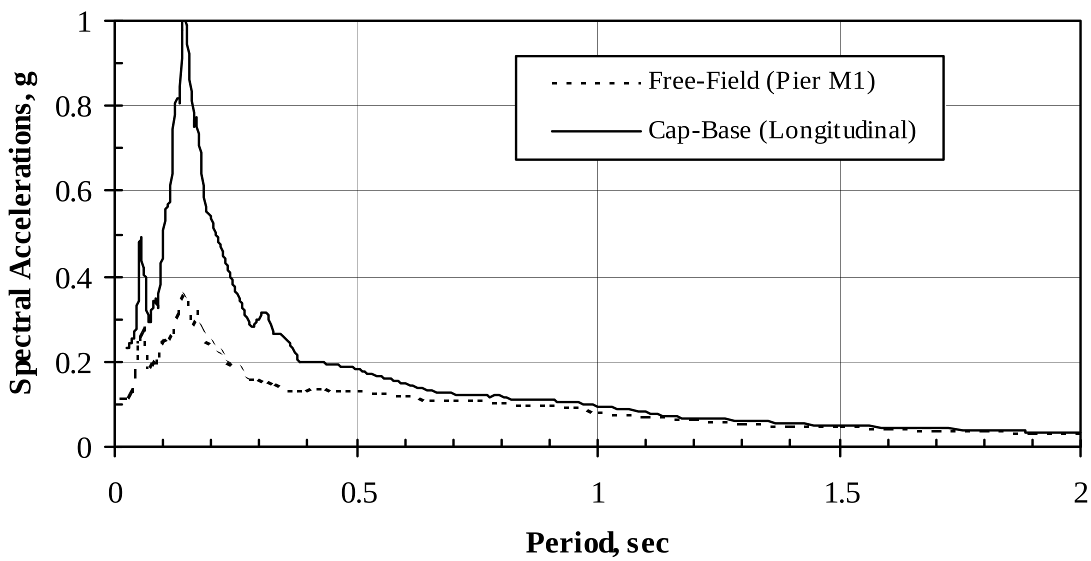



**Figure 16.** **Comparison of spectra of the motions at the Pier M1 pile cap level and in the free field.**

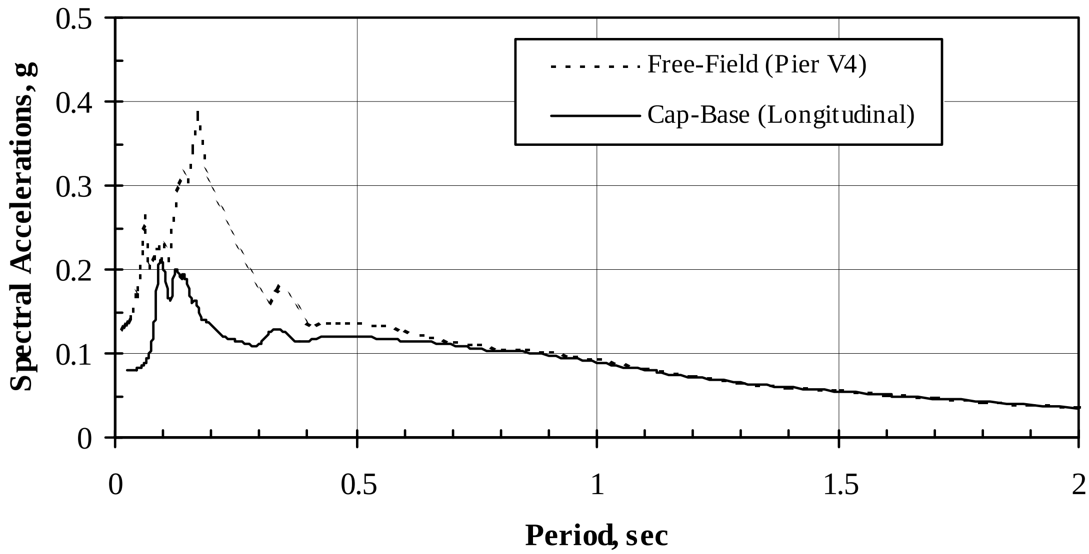



**Figure 17. Comparison of spectra of the motions at the Pier V4 pile cap level and in the free field.**

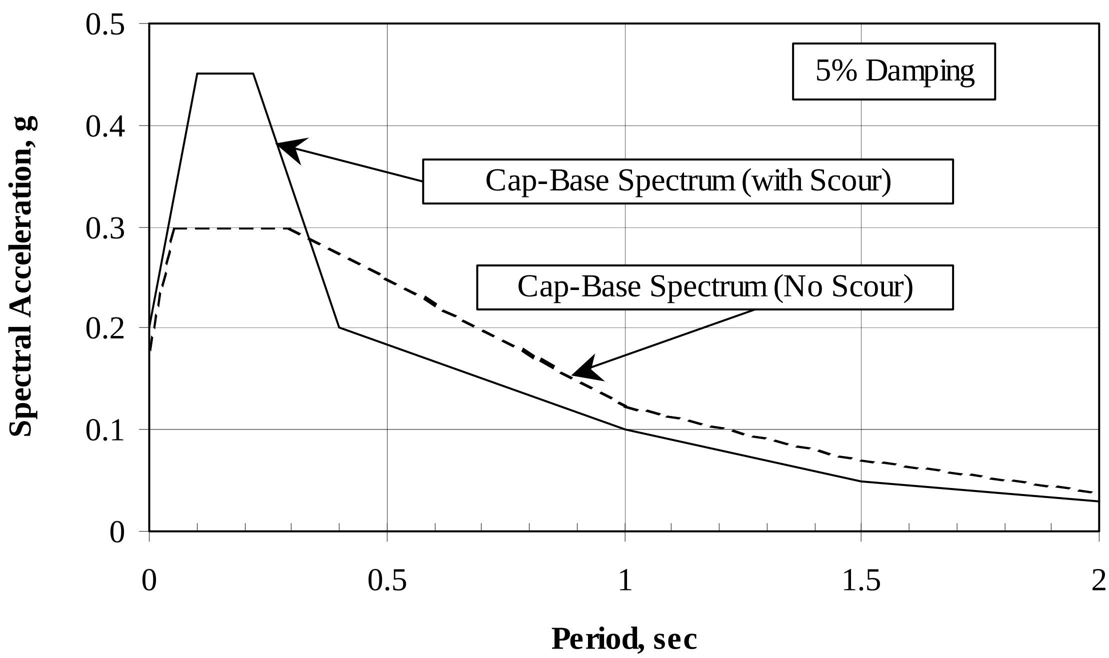



**Figure 18. Comparison of design spectra with and without scour conditions.**

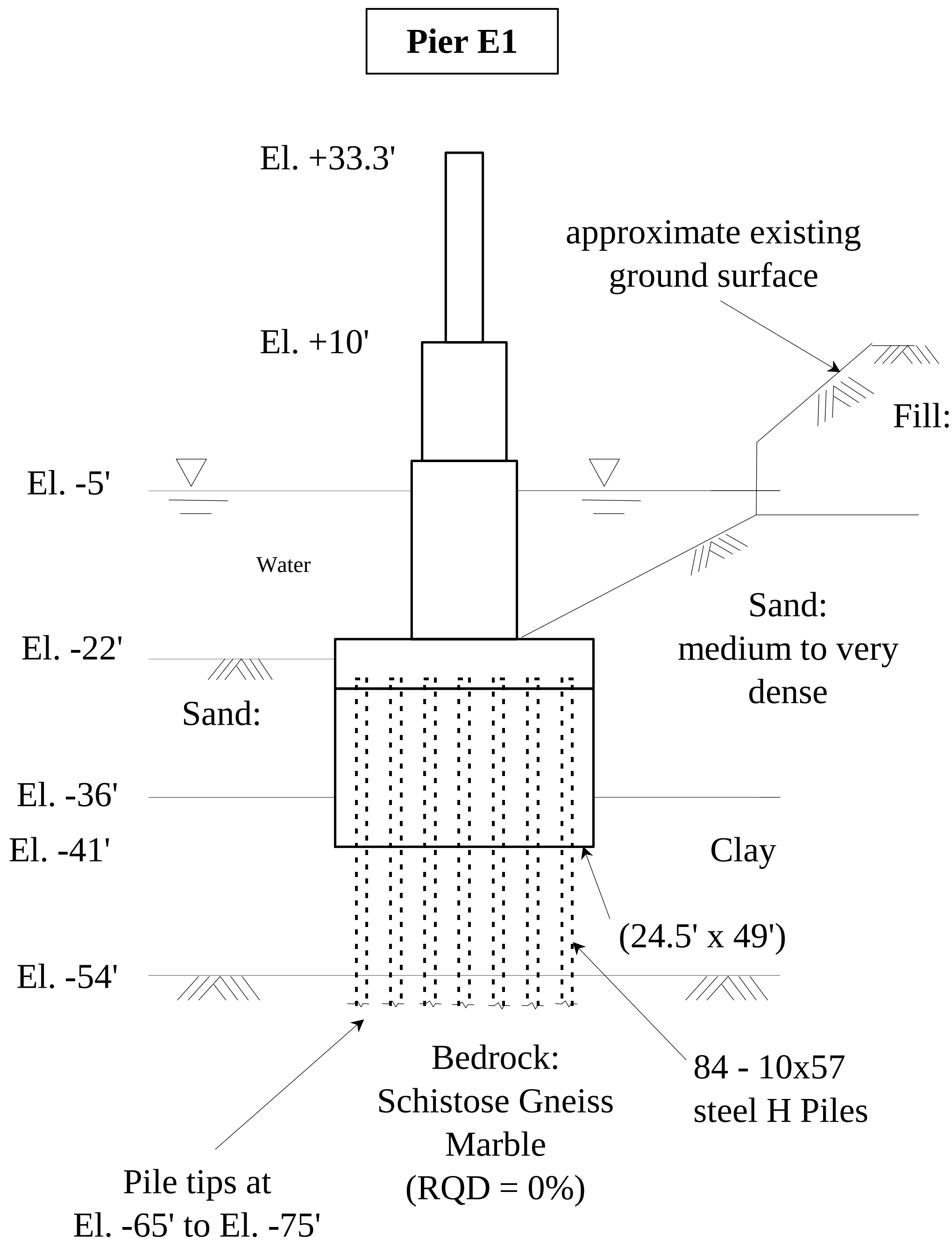



**Figure 19. Elevation of Pier E1 foundation and soil profile of the Roosevelt Island Bridge.**

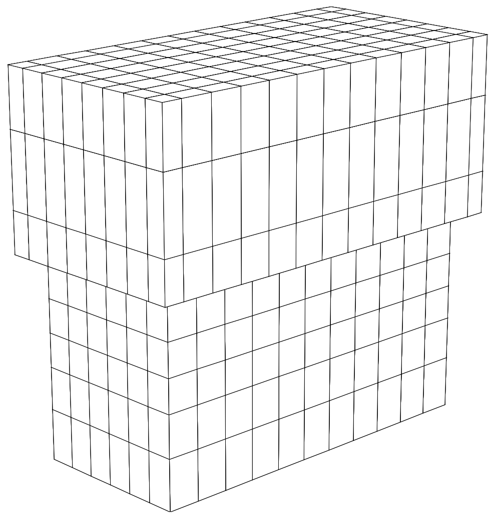


**Figure 20.** **Three-dimensional model of soil-pile system of Pier E1 used in SASSI analysis.**

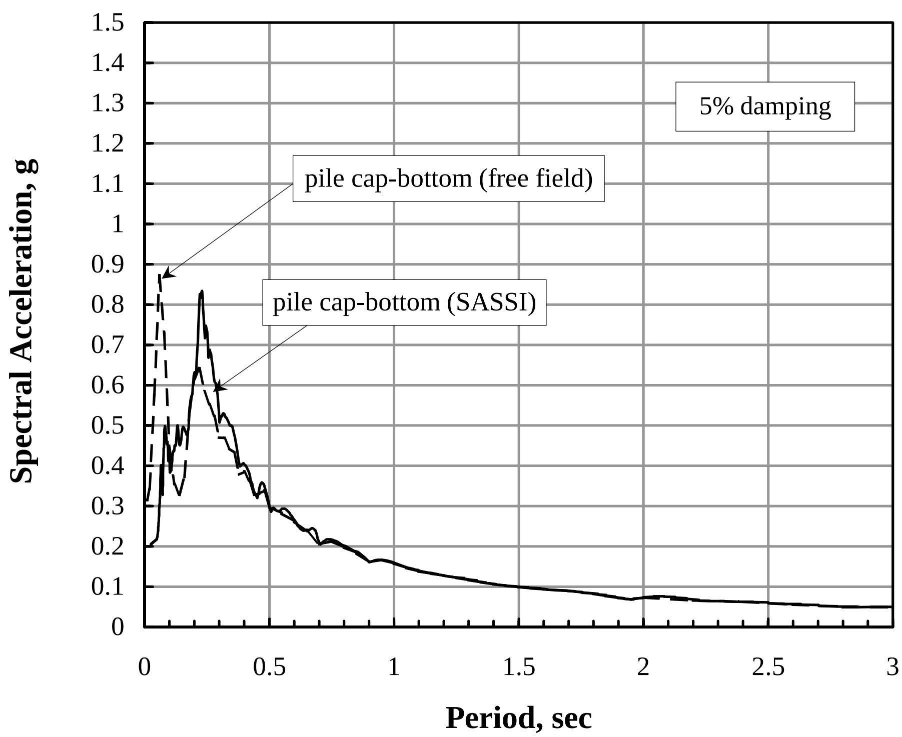



**Figure 21.** **Comparison of the spectra at the pile cap level obtained from 3-D soil-pile analysis and free-field ground motion analysis.**

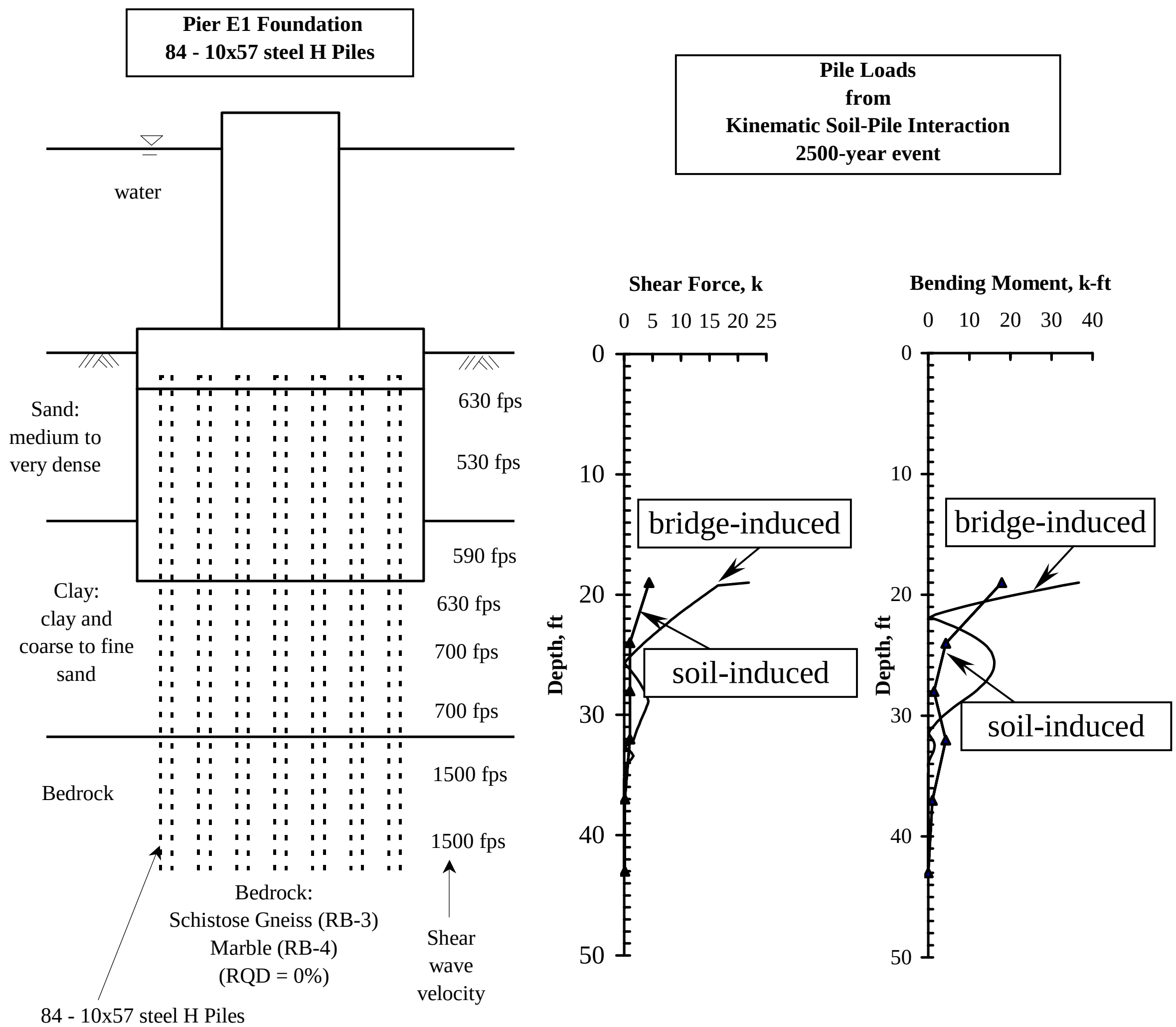



**Figure 22. Pile shear forces and bending moments induced by the soil motion and bridge inertia.**

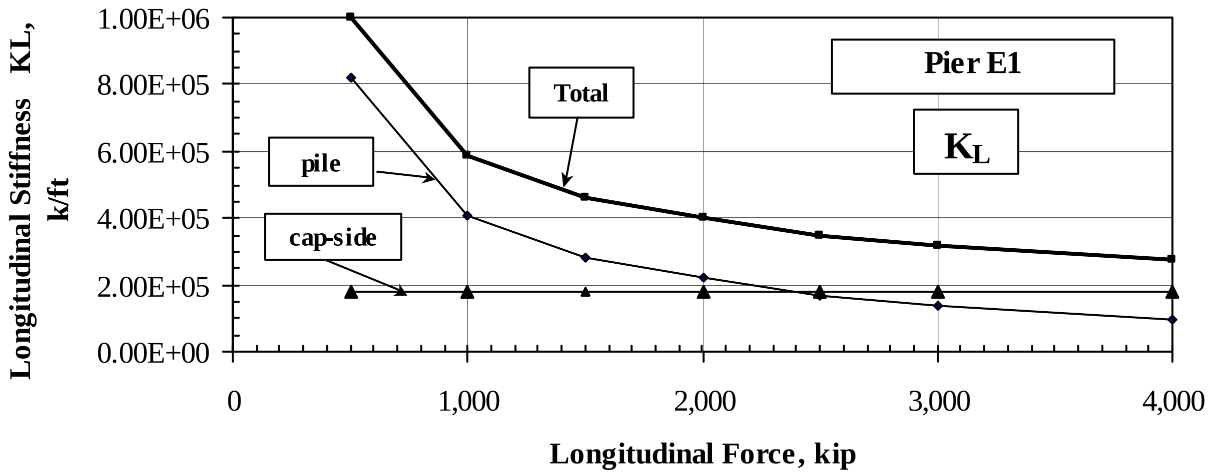



**Figure 23** **Contributions of the piles and the pile cap side to the lateral stiffness of the foundation.**

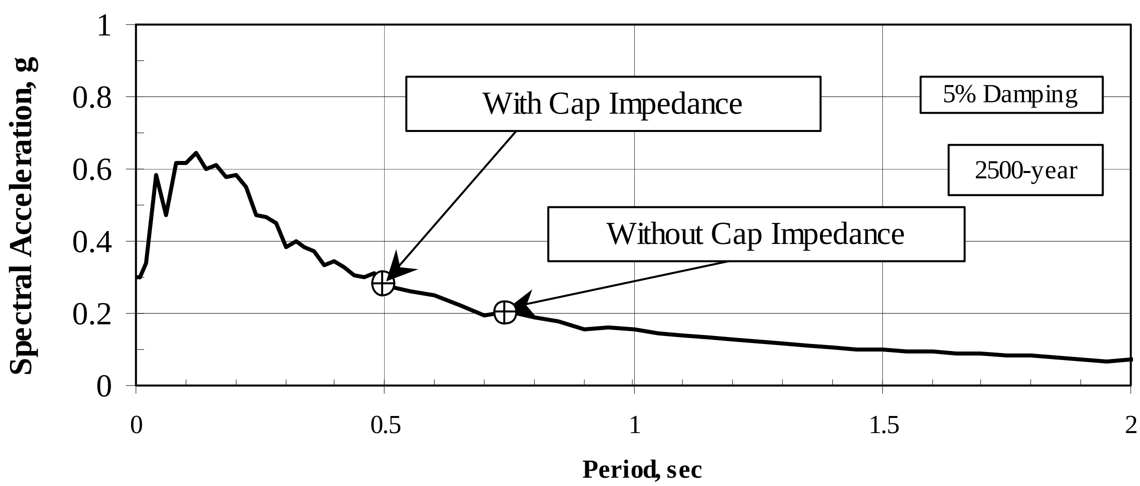



**Figure 24** **The effect of pile cap side stiffness on the average spectral acceleration.**

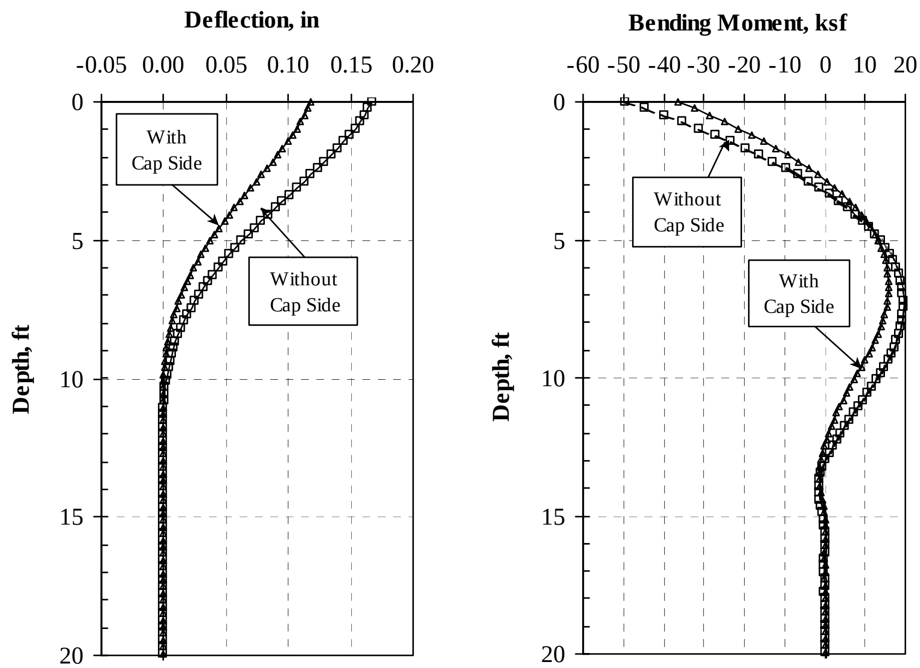



**Figure 25. The effect of including pile cap side resistance on the pile deflections and bending moments.**