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SEISMIC GEOTECHNICAL INVESTIGATIONS OF BRIDGES IN NEW YORK CITY

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ABSTRACT

Seismic vulnerability assessment of a critical bridge is a major undertaking. Such an investigation may lead to requirements with respect to seismic retrofitting of an existing bridge or enhancement of the design of a new bridge, often at considerable cost. A safe and cost-effective new design or retrofit of a bridge requires the application of realistic evaluations at every step of the seismic analysis, in which geotechnical earthquake engineering should play an important role. Soil and site conditions can have important effects on not only the earthquake motions but also on the dynamic response of the soil-foundation-bridge system. This paper presents case history analyses that demonstrate the importance of geotechnical earthquake engineering in seismic safety evaluation of bridges in New York City.

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 significantly lower than that in the west coast, the large inventory of older and sometimes historic bridges in the east are nevertheless vulnerable to earthquake damage. AASHTO prescribes seismic vulnerability studies for all bridges, including those in the northeastern U.S., using a 500-year event. This event has a 10% probability of exceedance in 50 years. In 1998, the New York City Department of Transportation adopted seismic guidelines for bridges that use, for critical bridges, two levels of seismic design. 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.

Whether or not a bridge is deemed to be safe against a seismic event depends on the outcomes of various critical investigations in the fields of seismology, and geotechnical and structural engineering. The successful application of a seismic evaluation of a major bridge will depend upon rational applications of various overarching tasks. Geotechnical earthquake engineering plays a crucial role not only in establishing the earthquake motions but also in contributing to the modeling and analysis of the soil-foundation-bridge system. Furthermore, the survival or the acceptable performance of a bridge during an earthquake is also hinged on the adequate performance of its foundations under the seismic loads. Hence, geotechnical earthquake engineering must play an important role in the seismic analysis of a critical bridge.

This paper describes selected case history analyses that demonstrate the role of geotechnical engineering in seismic safety evaluation of bridges in New York City.

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 1 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.

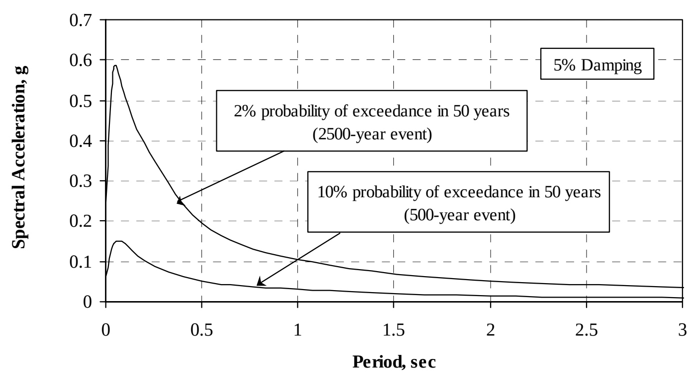

Figure 1 Hard rock spectra provided in the 1998 NYCDOT seismic guidelines.

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.

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, γ = 140 pcf) to a softer rock medium (γ = 130 pcf) can be computed as a function of rock shear wave velocity. Figure 2 shows the amplification factor that can be used to multiply hard rock motions to account for the softer rock conditions.

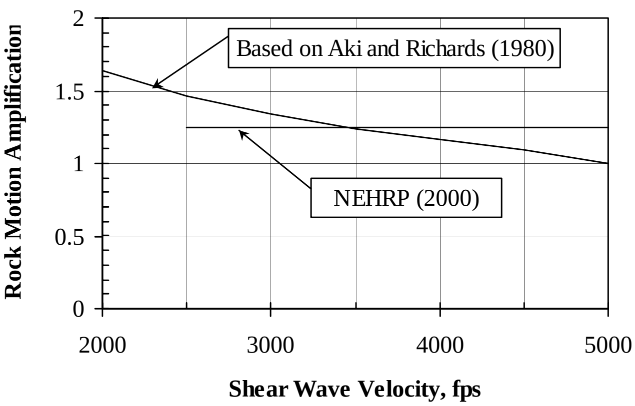

Figure 2 Rock motion amplification from hard rock (Vs = 5000 fps) to softer rock.

From Figure 2 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 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 3, the average measured Vs values are compared with the average RQDs of the rock. The lines in Figure 3 represent the mean and lower and upper bound of the data. Table 1 shows the data in more detail.

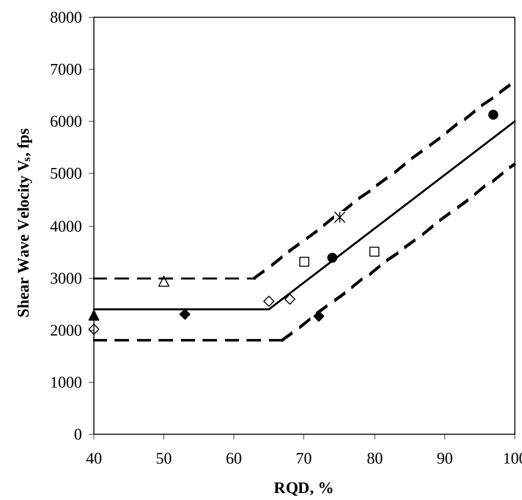

Figure 3 Measured shear wave velocities of bedrock related to the average RQD of the rock.

Table 1 Details of the Vs versus RQD data plotted in Figure 3.

Bridge Location	RQD, %	V_s, fps	Test Type	Rock Type
Madison Avenue, NY	35	2275	Crosshole	Gneiss
3rd Avenue, NY	40	2022	Crosshole	Schist
	68	2587	Crosshole	Schist
	65	2544	Crosshole	Schist
Roosevelt Avenue, NY	72	2273	Crosshole	Gneiss
	53	2300	Downhole	Gneiss
Queensboro, NY	50	2941	Crosshole	Gneiss
Williamsburg, NY	70	3300	Crosshole	Gneiss
	80	3500	Crosshole	Gneiss
Manhattan, NY	74	3373	Crosshole	Gneiss
	94	6128	Crosshole	Gneiss
Secaucus, NJ	75	4166	Downhole	Sandstone

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 3 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 3 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.

GROUND MOTION ANALYSIS

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 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 4 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.

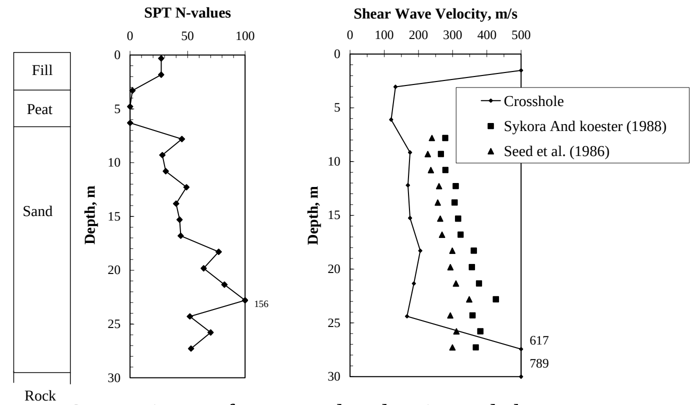

Figure 4 Comparisons of measured and estimated shear wave velocities.

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 5 shows the response spectrum of the free-field motion that was computed using the crosshole measured Vs values.

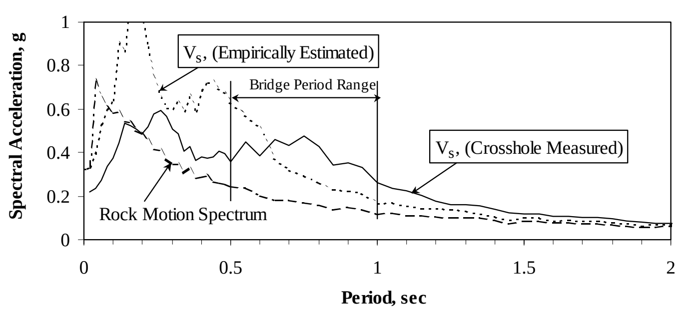

Figure 5 Comparison of the response spectra from ground motion analyses, using the crosshole measured, and empirically-estimated shear wave velocities.

This motion was subsequently used as input in the seismic analysis of the bridge.

Included in Figure 5 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 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.

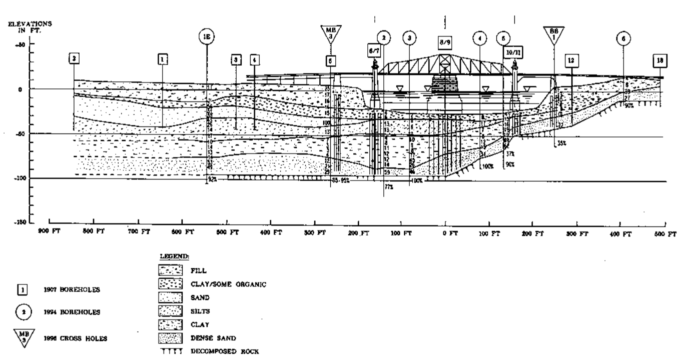

Figure 6 The soil profile and elevation of the Madison Avenue Bridge.

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.

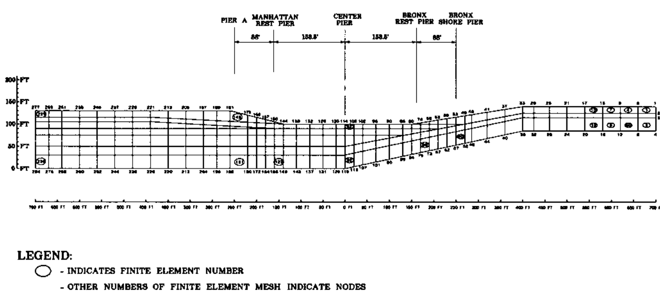

Figure 7 The finite element mesh used in the ground motion analysis of the Madison Avenue Bridge.

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

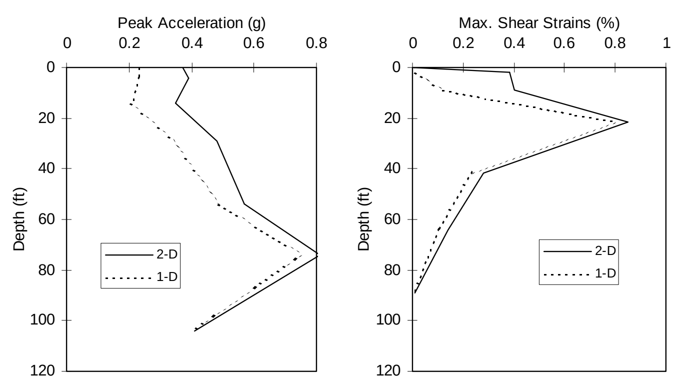

Figure 8 Comparisons of the peak accelerations and shear strains from 1-D and 2-D ground motion analyses.

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).

Figure 9 Comparisons of the spectra from 1-D and 2-D ground motions.

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.

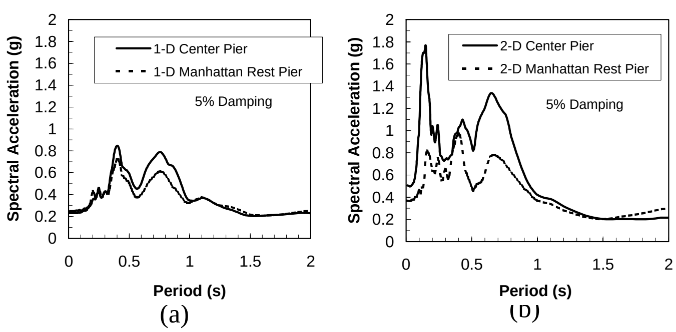

Figure 10 Comparisons of the spectra for the Center and Manhattan Rest Piers from (a) 1-D and (b) 2-D ground motion analyses.

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.

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 11 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.

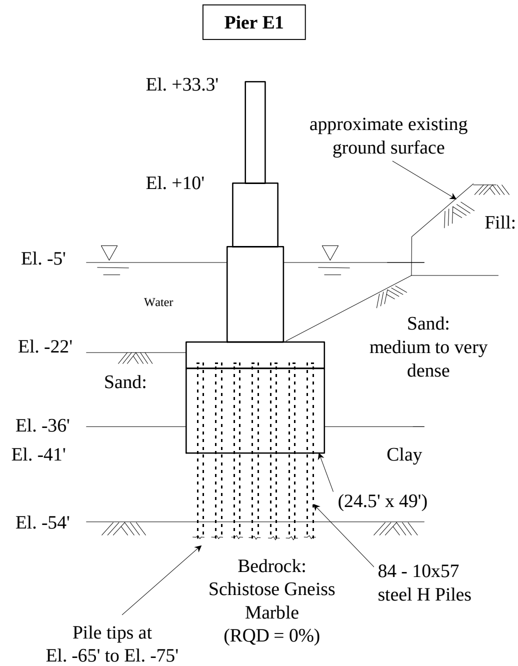

Figure 11 Elevation of Pier E1 foundation and soil profile of the Roosevelt Island Bridge.

Pile Cap Motion

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 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 SASSI (ACS-SASSI). Figure 12 shows the SASSI model of the pile cap system. The soil layers are not shown in the figure because SASSI 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.

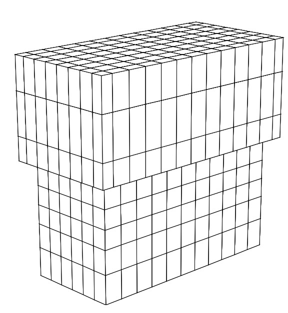

Figure 12 Three-dimensional model of soil-pile system of Pier E1 used in SASSI analysis.

Figure 13 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.

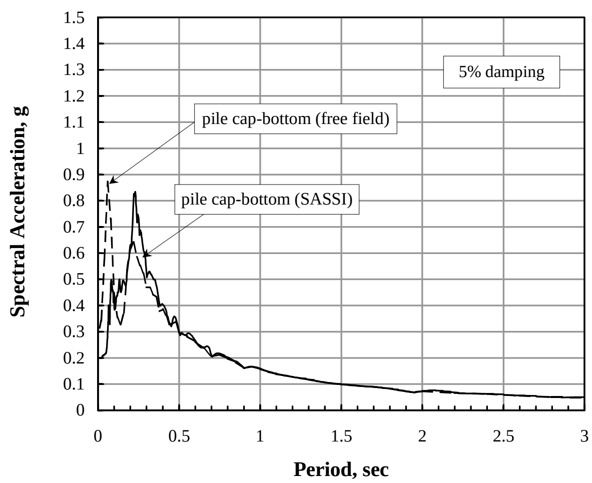

Figure 13 Comparison of the spectra at the pile cap level obtained from 3-D soil-pile analysis and free-field ground motion analysis.

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 14. 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.

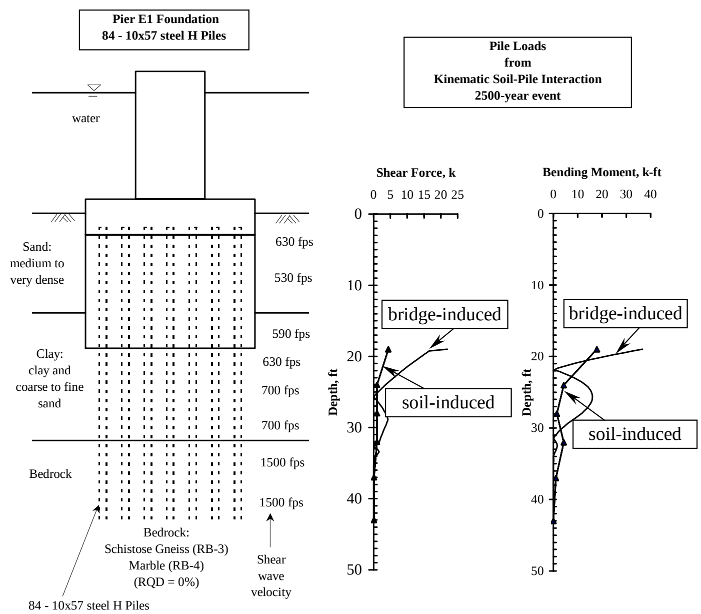

Figure 14 Pile shear forces and bending moments induced by the soil motion and bridge inertia.

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 15 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 15 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.

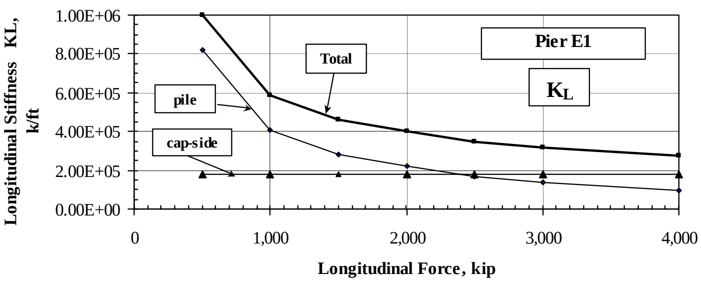

Figure 15 Contributions of the piles and the pile cap side to the lateral stiffness of the foundation.

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. 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, as illustrated in Figure 16. 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 16 The effect of pile cap side stiffness on the average spectral acceleration.

Figure 17 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 it would be very difficult, if not impossible, 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.

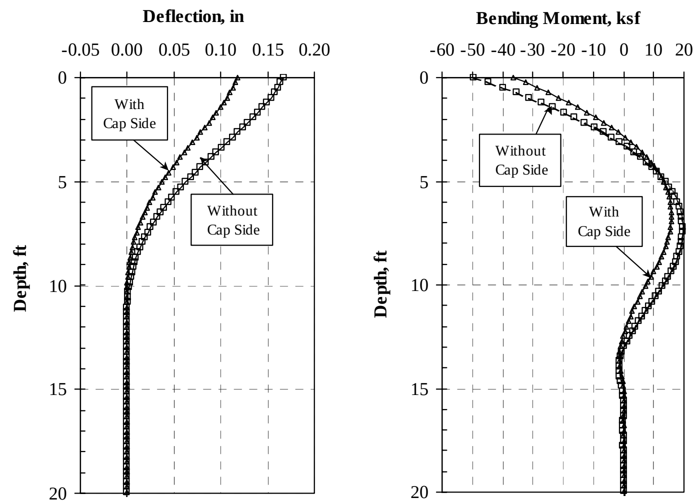

Figure 17 The effect of including pile cap side resistance on the pile deflections and bending moments.

SUMMARY

Example case studies were presented to demonstrate the importance of geotechnical earthquake engineering in the seismic safety evaluation of existing bridges in New York City. The outcomes of the various seismic geotechnical investigations performed for a bridge can have important implications on the need and scope of seismic retrofit measures for an existing bridge. In many instances, it may not be readily obvious what the effect of certain assumptions made in the seismic investigation might be on the final outcome of the seismic safety assessment of a 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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