SEISMIC SOIL-FOUNDATION INTERACTION ANALYSES OF THE NEW WOODROW WILSON BRIDGE

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ABSTRACT

A new bridge that will replace the existing Woodrow Wilson Bridge is under construction over the Potomac River in Washington, DC. The river piers are supported on cylindrical steel piles of 72, 54 and 48 inches in diameter. The longest piles (at the bascule piers) are approximately 200 ft long. Outside the river, nearly all piers are supported on 24-inch square prestressed concrete piles. Although the seismicity of the region is modest, seismic issues were thoroughly addressed in the design to ensure acceptable performance during a 500 and a 2,500-year earthquake. The results from 3-D dynamic soil-foundation interaction analyses presented demonstrate the effect of site conditions and pile foundations on the design seismic motions. In the presence of scour, pile foundations in the river were found to move significantly more than the free field, especially for motions with periods close to the fundamental period of the soil-pile system. Conversely, for foundations outside the river, pile cap motions were found to be smaller than free-field motions. Furthermore, strong kinematic pile bending due to passage of seismic waves was observed at deep elevations below the surface, close to interfaces with large stiffness difference between adjacent soil layers.

Introduction

The Woodrow Wilson Memorial Bridge is the only Potomac River crossing in the southern half of the Washington Metropolitan area. Consisting of fixed spans and a movable (bascule) span, it carries the Capital Beltway (I-495), which is part of I-95, the main north-south interstate route on the East Coast. Approximately 6,000-ft long, the new bridge (Figure 1) has been designed for HS25 loads consisting of six lanes for local traffic, four lanes for express traffic, and two HOV lanes. Provisions are made for future replacement of the HOV lanes with rail. The bascule span is 370 ft long with a navigation channel of 175 ft wide. The water depth at the channel is 35 ft. In 1998, Maryland State Highway Administration awarded Parsons Transportation Group (PTG) the design and construction support services of the new bridge. Mueser Rutledge Consulting Engineers performed the geotechnical engineering services.

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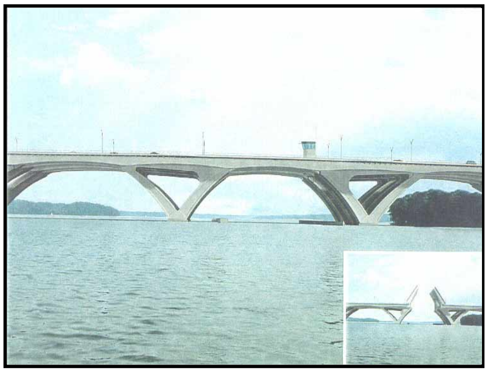

An arch-like appearance has been achieved by introducing V-shaped piers with curved legs, which support haunched girders. This structural system consists of independent structural units (V-piers), and produces zero horizontal thrust forces under dead and live loads. Proper arrangement of the girder spans balances the dead loads and produces minimal bending moments at the piers. This system eliminates the need for using batter piles and results in significant savings in the foundations, especially at the bascule piers located in the deepest part of the river.

Figure 1. Computer generated photograph of the new Woodrow Wilson Bridge.

Figure 2 shows the soil profile along the longitudinal axis of the bridge and the locations of the various piers. Bedrock is 500 to 700 feet below sea level. The subsurface soil profile consists of 50 to 80 ft of soft or silty clay underlain with deep deposit of hard sandy clay. The soft clay is vulnerable to significant scour in the 500-year flood, especially at the main navigation channel where, the entire soft layer was estimated to undergo scour.

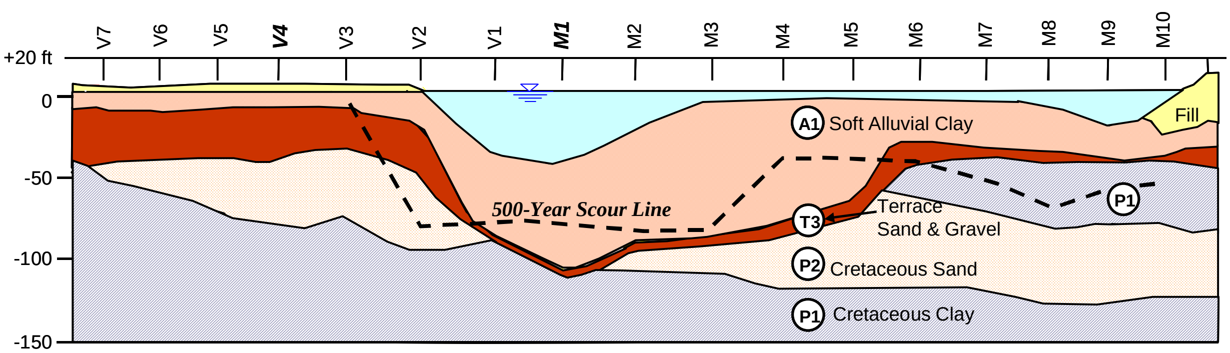

Figure 2. Soil profile and bridge pier locations.

Although the seismicity of the region is modest, because of the importance of the bridge, it was decided that seismic issues would be addressed in detail throughout the design. This paper describes geotechnical earthquake engineering aspects of the design and analysis of the bridge. In particular, the paper presents the design rock spectra and the soil-pile interaction analyses used to obtain the foundation motions, the pile loads induced kinematically by seismic waves, and the foundation spring coefficients representing the soil-structure interaction (SSI) in the global analysis of the bridge.

Free-Field Ground Motions

There is significant spatial variability in the soil profile and in the scour potential along the bridge axis. Thus, the foundations of the bridge are quite different. The piers over the river are on 72, 54 and 48 inch diameter cylindrical steel piles with caps that are as much as 30 feet above the mudline. Six of the piers on the Virginia side are on land. Except for pier V2, which is subject to potential scour, all piers are founded on 24" square prestressed concrete piles with their caps embedded in the ground. Site-specific ground motion analysis was performed using the seismicity of the region and the soil conditions encountered at the bridge site. URS Corporation developed the site-specific free-field motions. Uniform hazard rock spectra for 500 and 2500-year return periods were established by Geomatrix Consultants following a probabilistic seismic hazard analysis approach. Figure 3 shows the 2500-year return period horizontal and vertical design rock spectra that were developed using the regional seismicity data and attenuation relationships applicable to the eastern United States. A confidence interval of 85% was selected to account for parametric uncertainties and subjective judgment.

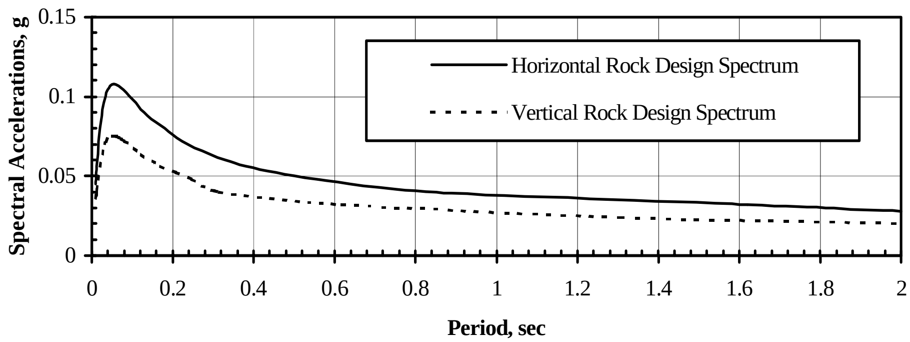

Figure 3. Horizontal and vertical uniform hazard rock spectra for 2500-year return period.

Site specific rock input motions (time histories) were developed. This was achieved using typical rock records that were considered appropriate for seismic analysis in the eastern United States and then adjusting them such that their spectra matched reasonably well with the uniform hazard rock spectrum for the horizontal direction, as shown in Figure 3. These records were then used in the computer program SHAKE to establish the soil amplification characteristics of various soil columns that represented the site conditions along the bridge axis. The resulting amplification ratios were used to adjust the rock spectra thus establishing free-field soil spectra for the various regions investigated. Finally, using these design ground motion spectra as targets, free-field acceleration time histories were generated having response spectra very closely matching the target spectra of the different representative piers studied.

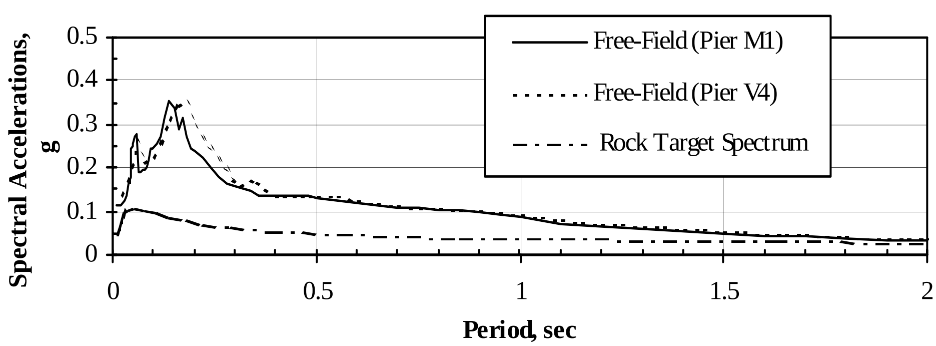

Figure 4. Computed free-field ground surface spectra for pier locations M1 and V4.

Figure 4 shows the spectra of the generated free-field ground motions for two typical locations: the bascule span region (Pier M1) where 50% (25 ft) of scour is included, and the location of Pier V4 where the pile cap is in the ground, and scour is not an issue. These free-field ground motions were subsequently used in the soil-foundation interaction analysis to generate the foundation (cap-base) motions needed in the soil-structure interaction analysis of the bridge.

Soil-Foundation Interaction Analyses

The piers of the bridge over the water are founded on frictional and partially bearing piles penetrating through a soft clay deposit (A) to a compact sand layer (T) and finally a hard sandy clay (P) layer. Figure 5 shows the foundation details of the bascule span pier M1.

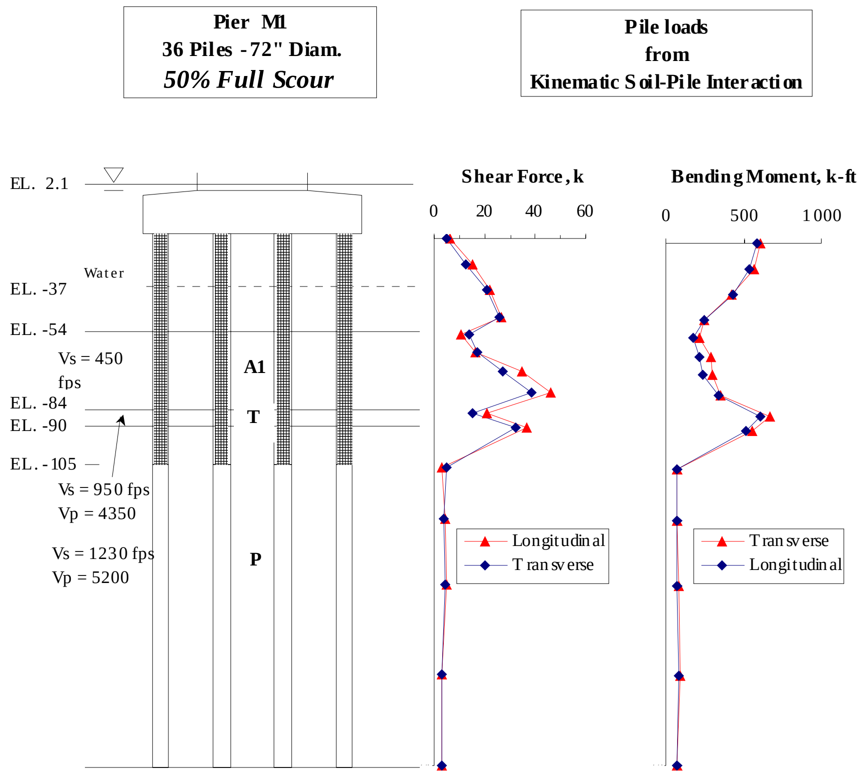

Figure 5. Soil and foundation details for bascule Pier M1, and results of 3-D SSI analysis.

The potential scour in the river channel is high (50 ft). It was decided that seismic loads would be combined with either 50% of full scour or no scour at all. Full scour and zero scour conditions were assumed for service load conditions such as dead and live loads. For the superstructure, which is supported on lead core rubber bearings, the no scour conditions generally controlled.

Figure 6 shows the pile foundation of a typical pier (V4) located on land on the Virginia side. In this region, the pile cap will be about 10 ft below the ground surface, and scour is not a concern.

During a seismic event, ground motions propagating through the foundation soils transmit energy through the piles to the bridge superstructure. In turn, the bridge responds dynamically to these vibrations, and the resulting inertial forces are transmitted back to the soil through the pile foundations. This soil-structure interaction response of the bridge was modeled using foundation springs at the base of the pile caps. Figure 7 shows a schematic of a representative model (bascule pier) for analysis of the soil-foundation interaction and the model representing SSI in the global analysis of the bridge.

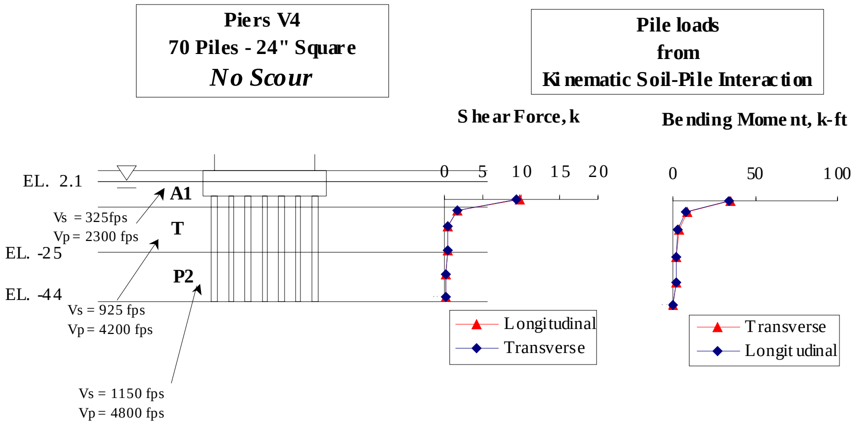

Figure 6. Soil and foundation details for Pier V4, an results of 3-D SSI analysis.

During a seismic event, ground motions propagating through the foundation soils transmit energy through the piles to the bridge superstructure. In turn, the bridge responds dynamically to these vibrations, and the resulting inertial forces are transmitted back to the soil through the pile foundations. This soil-structure interaction response of the bridge was modeled using foundation springs at the base of the pile caps. Figure 7 shows a schematic of a representative model (bascule pier) for analysis of the soil-foundation interaction and the model representing SSI in the global analysis of the bridge.

It is noted that the piles in the river, especially where the scour potential is high (main navigation channel), have long laterally unsupported lengths above the mudline. In such a situation, because of interaction of the piles with their surrounding soils, the motion at the level of the pile cap can be significantly different from the free-field ground surface motion, resulting in large spectral accelerations, especially in the fundamental period of the pile-soil system. The shear forces and bending moments induced by these ground motions and the pile inertia, in addition to the inertial loads coming from the superstructure, have to be accounted for in the design (Gazetas and Mylonakis, 1998). For these reasons, soil-foundation interaction analysis of the pile foundations were performed. The calculated pile cap-base motions (E-cap-base in Figure 7) were then specified as input motions in the global analysis of the bridge, as is depicted in Figure 7.

The first part of the seismic soil-foundation interaction analysis involved calculation of the cap-base motions using the free-field motions. The second part was to compute pile shear forces and bending moments induced by both seismic waves (kinematic effect) and pile inertia. Finally, foundation stiffness coefficients were calculated for use in the global analysis of the superstructure. The following sections describe these three analyses and present the results.

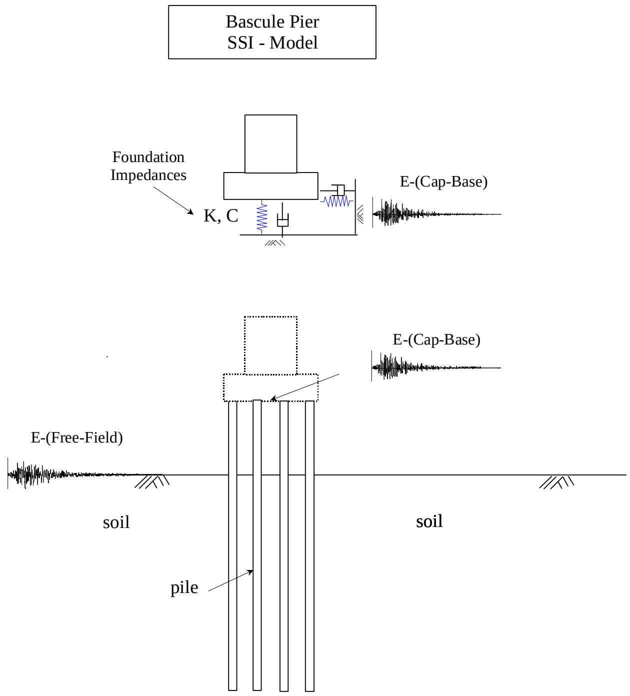

Figure 7. The local model for analysis of the soil-foundation interaction and the model representing SSI in the global analysis of the bridge.

Cap-Base Motions

To compute the earthquake motions at the top of the piles (without considering the cap and the bridge superstructure) three-dimensional seismic soil-foundation interaction analyses were performed using the PC version of the computer program SASSI (ACS SASSI-C, 1998). The program utilizes the finite element method and uses transmitting boundaries that model the deformations and dissipation of energy away from the boundaries of the finite element mesh. Figure 8 shows the SASSI models of the bascule pier foundation M1 (details in Figure 5) and Virginia Pier foundation V4 (details in Figure 6), respectively.

As shown in the figures, solid elements were used to model soils and the concrete cap, and beam elements were used to model the piles. The site-specific free-field motions described earlier were used in SASSI as input at the ground surface. In SASSI analysis, the soil behavior is assumed to be linear and is governed by the soil shear wave velocity. However, it is well recognized that soils exhibit nonlinear stress-strain behavior. Soil modulus degrades and internal damping increases with increasing shear strain. Seismic waves propagating though a soil profile in the absence of the structure and its foundation generate shear strains that lead to reduction in the soil modulus. An approximate way of including soil nonlinear response in a SASSI analysis is through the use of reduced shear wave velocities (or moduli) that include the effect of wave propagation. These strain-dependent velocities, shown in Figures 5 and 6, were obtained from the free-field ground motion analyses performed using SHAKE.

Figure 9 shows the free-field spectrum of the ground surface at the bascule pier site together with the spectrum of the motion computed at the base of the cap in the longitudinal direction. Since the horizontal free-field ground motions were the same in longitudinal and transverse directions, and the pile caps were symmetric about these two axes, the differences in the cap-base spectra for the longitudinal and transverse directions were insignificant.

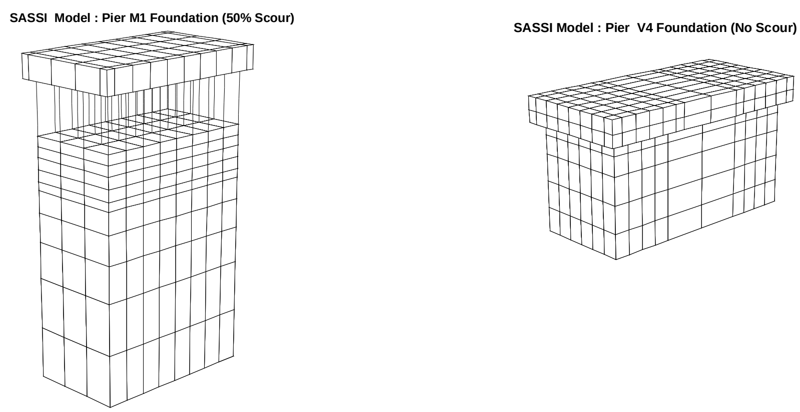

Figure 8. The SASSI models used for analysis of pier foundations M1 and V4.

A comparison of the free-field and cap-base spectra shown in Figure 9 reveals that there is little difference between the spectra for periods greater than about 0.6 s. In the period range of 0.1 to 0.4 s, there is significant increase in the spectral acceleration at the cap-base level compared to that of the free field. It is of interest to note that this period range coincides with the fundamental period range of the soil-pile system and that of the input motion in the free field. The piles have rather small mass and large lateral stiffness. This high frequency amplification of the piles is of little interest once the superstructure mass is included in the global analysis of the bridge. It is noted that in the period range of importance in the bridge analysis (greater than 1.0 s) the cap-base and free-field spectra are almost identical.

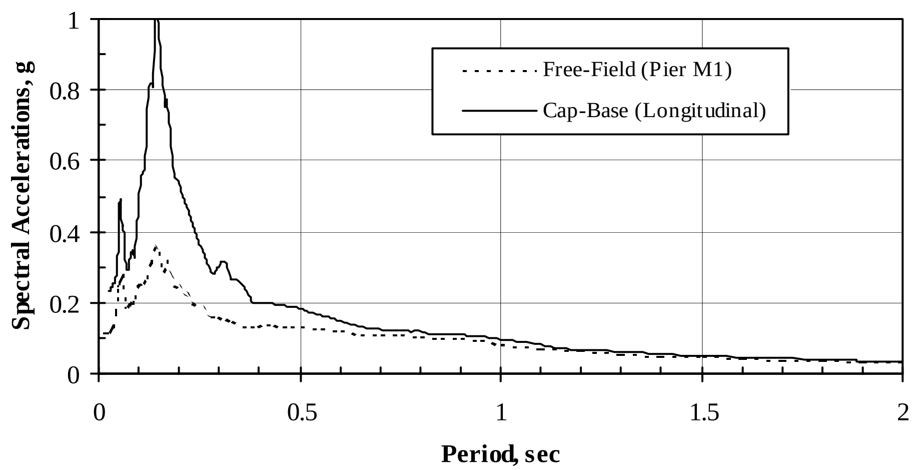

Figure 9. Comparison of cap-base and surface free-field motion spectra at Pier M1.

Figure 10 shows the results of SASSI analysis for pier foundation V4. The pile cap of this pier is on land and scour is not an issue. It is noted that the response of this pier is quite different from that of the bascule span pier M1. The SASSI results indicate significant reduction in spectral accelerations in the fundamental period range of the soil-pile system. This is attributed to kinematic interaction between the piles/pile cap system with the surrounding soil. Again, in the period range of interest, the cap-base spectrum is very similar to the free-field spectrum.

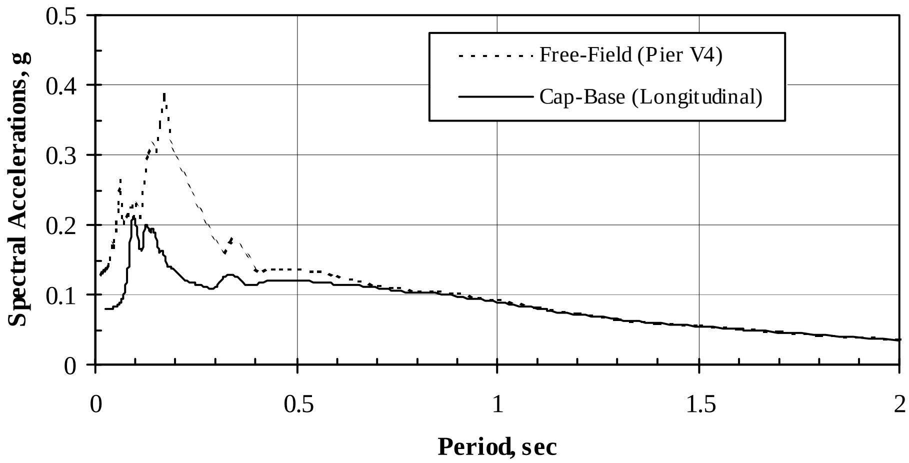

Figure 10. Comparison of cap-base and surface free-field motion spectra at Pier V4.

In summary, the results of the soil-pile interaction analyses show that the cap-base motions, especially 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.

Kinematically Induced Pile loads

During seismic shaking, a pile group even without the presence of a cap and superstructure undergo deformations. These deformations are induced by soil strains associated with the propagation of seismic waves, and by the mass of the piles. The resulting shear forces and bending moments in the piles are often referred to as kinematically induced pile loads. In the case of the piles at the bascule piers, these loads include inertial effects of the pile mass above the mudline. Kinematically induced pile loads can be large when piles are in a layered soil profile with significant stiffness contrast between soil layers. Other factors affecting such loads are the excitation frequencies, the number of effective cycles in the excitation, and the stiffness contrast between pile and soil (Nikolaou et al, 2001).

Using SASSI analysis, the kinematically induced pile shear forces and bending moments were computed for each pile group. Figure 5 shows shear forces and bending moments along the length of a typical pile of the bascule pier M1. It is evident that the kinematic pile loads of the bascule pier piles that penetrate soft clay and then a hard sandy clay layer are significant. It is of interest to note that the maximum shear force and bending moment occur at the transition boundary from soft to hard layers. In the assessment of the adequacy of the piles under seismic excitations, the kinematic loads were added to those induced due to the inertia of the superstructure. Figure 6 shows the kinematically induced pile loads in a typical pile of the Pier V4. In this case, the piles are primarily in stiff soil layers, and the kinematically induced pile shear forces and bending moments are inconsequentially small.

In summary, pile foundations that penetrated from soft to hard soils can experience significant shear forces and bending moments induced by the ground motions. Such kinematically induced loads were included in the seismic design of the bridge foundations.

Foundation Stiffness

The dynamic response analysis of the bridge was performed following the response spectrum approach using foundations springs to represent the soil-structure interaction (Figure 7). The stiffness coefficients of these springs are frequency dependent. The SASSI analysis can provide these frequency-dependent foundation stiffness coefficients. However, as was stated earlier, nonlinear soil-pile interaction can only approximately be accounted for in SASSI. A reliable estimate of the static stiffness coefficients of a non-linear soil-pile system can be made following the p-y curves approach. In this investigation, SASSI was used to calculate the frequency effect on the stiffness coefficients of the soil-pile systems and then this effect was introduced in the static stiffness calculations using the finite element program FLPIER (Bridge Software Institute, 2001). The results of SASSI stiffness analyses showed that there was no appreciable change in the rocking stiffness of the caps within the frequency range of 0 to 2 Hz. The horizontal stiffness decreased only slightly (by a factor of 0.9) as the frequency of the applied unit load increased from zero (static application) to 2 Hz.

To account for non-linear SSI effects, sets of equivalent static stiffness coefficients were calculated at the base of each pile cap. These stiffnesses are linear coefficients representing secant values of the non-linear response of the foundations. FLPIER was used to calculate a 6x6 stiffness matrix corresponding to all six degrees of freedom (longitudinal, transverse, vertical, rotation around the longitudinal and transverse directions, and torsion) of each foundation. The stiffness coefficients were calculated assuming a rigid pile cap with loads applied at the center of the cap. Stiffness coefficients were provided for the no scour and the 50% of full scour condition.

A typical model and results of FLPIER is shown in Figure 11 for Pier M1. The pile cap was modeled as a shell and the piles as beam elements. The soil was represented by lateral (p-y) and vertical (t-z) curves, connected to each soil node. Soil properties were based on results of geotechnical laboratory testing, and the soil moduli were adjusted to be strain-compatible with the excitation level, according to the free-field soil amplification study performed with SHAKE. To account for group effects on the efficiency of the pile group stiffness, a reduction in the p-y curves through multipliers was applied, following the procedure by Reese and Wang (1996). To account for frequency effects on the pile group stiffness, a reduction of 0.9 was applied on the p-y multipliers, as was indicated by the SASSI analyses.

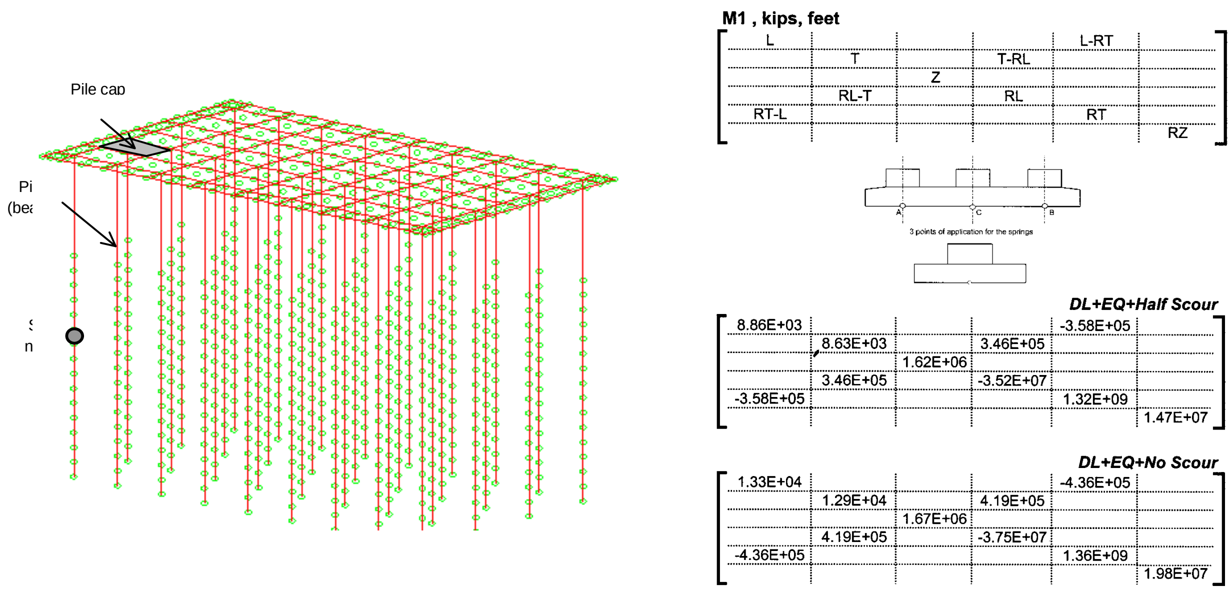

Figure 11. FLPIER modeling and stiffness results for Pier M1.

Summary and Conclusions

Seismic soil-foundation interaction analyses were performed for the new Woodrow Wilson Bridge. Site-specific ground surface motions were generated and then used to compute the motions at the pile cap level for the bridge piers. In cases where the pile caps were above the mudline (in water), and the piles penetrated through a soft clay layer underlying by a stiff underlying layer, within the period range of the soil-pile system, the cap-base motions were larger than the free-field motions. Also, in these cases, the passage of seismic waves and the pile inertia induced significant shear forces and bending moments in the piles. Conversely, in the cases where the pile caps were below the ground surface, the cap-base motions were smaller than the free-field motions, and the kinematically induced pile loads were insignificant.

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