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Technical Report MCEER-00-0011

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NOTICE

MCEER

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Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

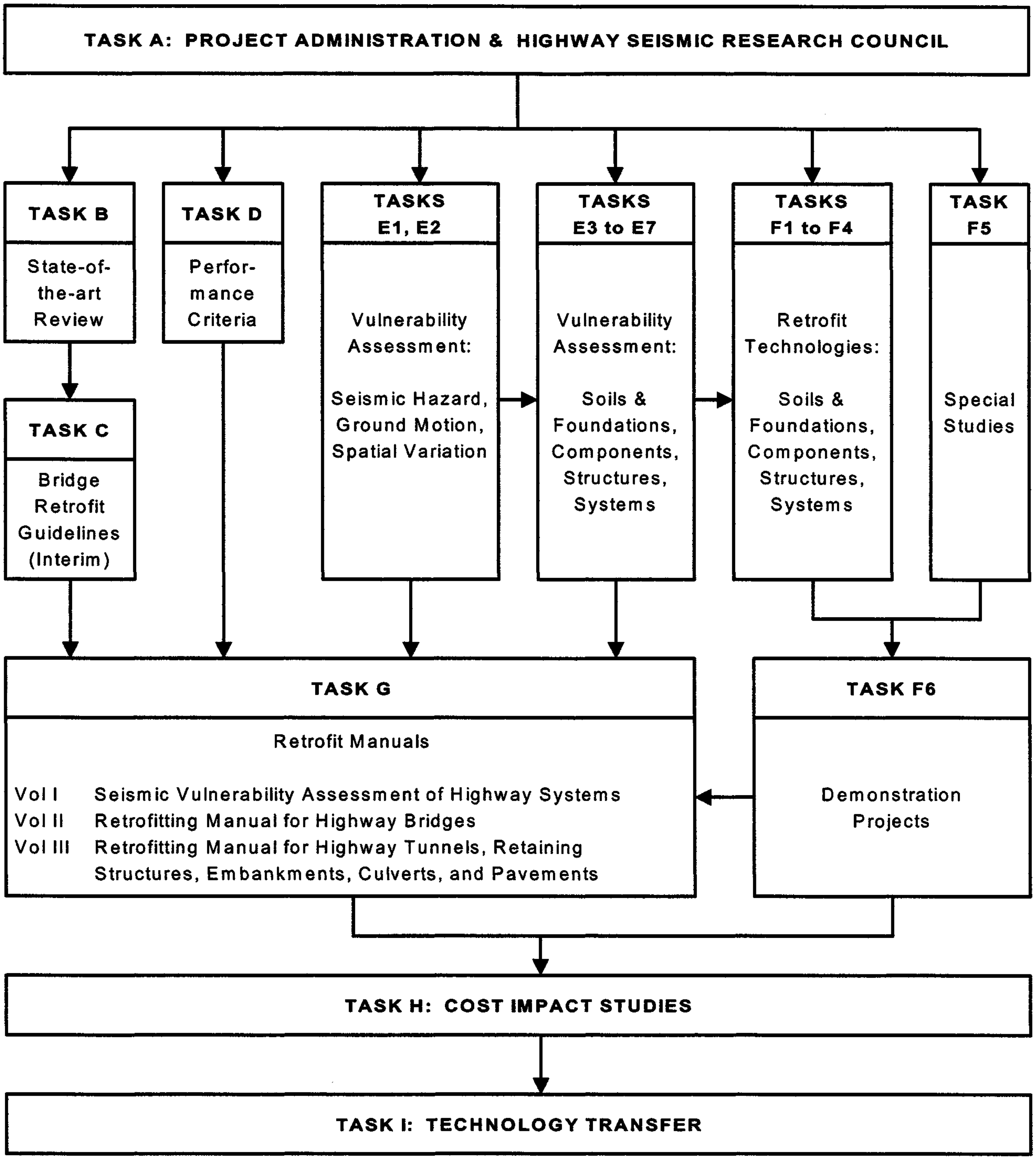
- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the existing highway structures project, and was performed within Task 106-E-4.6, "Seismic Analysis of Large Caisson Foundations" of that project as shown in the flowchart on the following page.

The overall objective of this task was to provide guidelines for seismic soil-structure interaction modeling requirements and analysis procedures for large caisson foundations. Large cellular reinforced concrete caissons exist as foundations of major long-span bridges across waterways in many parts of the country. The purpose of this study is to qualitatively assess the effects of

various factors affecting the caisson response. Several equivalent linear and nonlinear analyses were performed for a typical caisson, idealized from the large caisson at Pier W3 of the west spans of the San Francisco-Oakland Bay Bridge. The results indicate that the effects of the superstructure on the response of the caisson were insignificant. The lateral earth pressure, base bearing pressure, and soil stresses computed by the equivalent linear analysis indicate the possibility of soil-foundation separation (gapping and uplift). The results of nonlinear analyses indicate that motions and stresses developed in the caisson are sensitive to the soil-caisson and rock-caisson interface properties. More sensitivity analyses of caissons founded in different materials are needed to address the effects of soil embedment on the caisson response.

SEISMIC VULNERABILITY OF EXISTING HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00106



ABSTRACT

Large cellular reinforced concrete caissons exist as foundations of major long-span bridges across waterways in many parts of the country. This study was conducted to evaluate the important factors affecting the seismic response of large caissons. The report presents the results of several equivalent linear and non-linear analyses performed for a typical caisson idealized based on the cellular caisson at Pier W3 of the West San Francisco Bay Bridge subject to longitudinal excitation with a peak rock acceleration of 0.6 g. This caisson is 38.7 m (127 ft) long by 22.9 m (75 ft) wide submerged in about 32.6 m (107 ft) of water. It is embedded in 33.5 m (110 ft) of soil deposits and is founded on rock. Equivalent linear 3-D analyses were conducted for the cases with and without the tower superstructure and suspension cables. The results indicate that superstructures have small effects on the seismic caisson response. The computed dynamic earth pressure and base bearing stresses indicate that there will be soil-caisson gapping, rock-caisson base lifting, interface sliding, and soil yielding. The results from equivalent linear 2-D analyses in the direction of the short axis (longitudinal) are similar to those from the 3-D analyses. When the soil embedment is removed from the model, the dynamic stiffness and scattered motions at the caisson top only change slightly. However, the imaginary part of the foundation impedance functions is significantly smaller. Non-linear analyses were performed for 2-D models using computer programs FLAC and SAP2000. The results indicate that side gapping, base lifting, interface sliding, and soil yielding reduce the earth pressure, base bearing stress, caisson shear and bending moment, and caisson motions. However, the frequency characteristics of the responses appear to be relatively unaffected.

ACKNOWLEDGMENT

The research was supported by the Multidisciplinary Center for Earthquake Engineering Research (MCEER), Highway Project No. 106, Seismic Vulnerability of Existing Highway Construction, under the sponsorship of the Federal Highway Administration (FHWA Contract No. DTFH61-92-C-00106). Guidance provided by Ian M. Friedland, contract manager of MCEER during the course of the study is very much appreciated. Two reviewers of this report provided valuable comments.

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SECTION 1
INTRODUCTION

Large cellular reinforced concrete caissons exist as foundations of major long-span bridges across waterways in many parts of the country. Generally, these caissons are deeply embedded in soft soil deposits overlying rock or rock-like materials. In relation to the seismic response and vulnerability evaluation of the bridges supported by large caisson foundations, an important concern is the effects of soil-foundation structure interaction (SFSI) on the superstructure response and the imposed load demands. Approaches used to model the SFSI for large caisson foundations differ substantially in methodology and degree of sophistication. There is little guidance for practitioners to follow in regard to choosing the appropriate approach to incorporate important factors under various situations in their analyses. It is noted that the caisson foundation referred to in this study is a reinforced concrete structure having a plan dimension of several tens of meters in each direction and is different from those commonly referred to as drilled shafts. Thus, the use of discrete spring elements (e.g., p-y and t-z) to approximate SFSI may not be appropriate.

Completed studies of seismic vulnerability of several of these bridges have concluded that the caissons can experience large seismic demands that correlate to significant damage levels. These studies, however, were based on simplified analytical models of foundation behavior, ranging from fully linear elastic dynamic soil-structure interaction models to pseudo-dynamic models that incorporate some inelastic performance of the structural and geotechnical components but neglect some dynamic factors. Some fully dynamic inelastic analyses of foundations have been undertaken, but using analytical tools that are not generally compatible with the time domain implicit models that are used for structural modeling of the superstructures.

This study was part of a research project sponsored by the Federal Highway Administration and conducted by the Multidisciplinary Center for Earthquake Engineering Research in Buffalo, New York to investigate the seismic vulnerability of existing highway construction. The study was performed jointly by Geomatrix Consultants (Geomatrix) and OPAC Consulting Engineers

(OPAC). In this study, parametric sensitivity analyses were performed based on rigorous solution techniques to evaluate the important factors that affect the seismic response of caisson foundations. It includes an evaluation of effects of soil yielding, gapping, slippage, sliding, and uplift on seismic response of a caisson foundation. The results of this study can be used to develop guidelines on appropriate SFSI modeling requirements and analysis procedures for seismic analysis of caisson foundations.

This report presents the results of dynamic equivalent linear and non-linear analyses performed to evaluate the SFSI effects on the seismic response of a typical caisson foundation. The analyzed example is based on the cellular caisson at Pier W3 of the west spans of the San Francisco-Oakland Bay Bridge subject to ground motions with a peak horizontal acceleration of 0.6 g at rock outcrop. The SFSI analyses were performed by both Geomatrix and OPAC using different analysis tools. Geomatrix used the computer programs SASSI (Lysmer et al., 1988) and FLAC (Itasca, 1993) for the SFSI analyses to conduct sensitivity analyses and to examine various issues related to SFSI. OPAC used the computer program SAP2000 to perform both the linear and nonlinear SFSI analyses.

Equivalent linear finite element analyses were performed using the computer program SASSI (Lysmer et al., 1988). Three-dimensional analyses were performed for the cases with and without the superstructure to evaluate the effect of superstructure on the dynamic caisson response and to identify the potential for soil yielding, gapping, sliding, and foundation uplift. Two-dimensional equivalent linear analyses were performed to evaluate the appropriateness of using a 2-D model to approximate the dynamic caisson response along the short axis (longitudinal direction). It should be noted that all equivalent linear analysis cases assumed perfect bond and no slip between soil/caisson and rock/caisson interfaces.

Two-dimensional non-linear finite difference analyses were performed using the computer program FLAC (Itasca, 1993) to assess the effects of soil gapping, sliding, and uplift on the response of the caisson. The computer program SAP2000 was also used for both linear and nonlinear analyses of a 2-D model of the caisson.

The results of the SFSI analyses performed by Geomatrix and OPAC are described in subsequent sections. Section 2 describes the subsurface conditions at Pier W3 analyzed. Section 3 describes the rock ground motion used in this study. Section 4 describes the dimensions and characteristics of the caisson foundation at Pier W3. Section 5 presents the results of the 3-D equivalent linear dynamic analyses using the computer program SASSI. Section 6 presents the 2-D equivalent linear analysis using the computer program SASSI and the 2-D nonlinear dynamic analyses using the computer program FLAC. Section 7 presents the results of the 2-D analyses using the computer program SAP2000. Summary and conclusions of the study are presented in Section 8. References cited in this report are presented in Section 9.

SECTION 2
SUBSURFACE CONDITIONS

Figure 2-1 summaries the subsurface conditions at the site. It is interpreted based on the geotechnical data provided by the California Department of Transportation. The site is covered by about 33.5 m (110 ft) of soil deposits overlying interbeds of weathered sandstone and mudstone. The mudline is located at a depth of 32.6 m (107 ft) below the mean sea level. The top soil consists of about 6 m (20 ft) of very soft Bay Mud underlain by about 9 m (30 ft) of loose to medium dense sandy silt. Below these shallow soft layers is about 9 m (30 ft) of medium dense to dense silty sand overlying about 3 m (10 ft) of dense silty sand and gravel. In between these granular soil layers and the weathered bedrock is about 6 m (20 ft) of hard sandy gravelly clay. The weathered bedrock is located at about 33.5 m (110 ft) below the mudline.

The measured shear- and compression-wave velocity profiles are also shown on Figure 2-1. The shear-wave velocity increases approximately from 180 m/sec (600 ft/sec) at about 9 m (30 ft) below mudline to about 300 m/sec (1000 ft/sec) at about 30 m (100 ft) below mudline. The compression-wave velocity in this depth range is almost constant at 1500 m/sec (5000 ft/sec). Below this depth range, the shear-wave velocity increases almost linearly to about 1370 m/sec (4500 ft/sec) at about 46 m (150 ft) below mudline, while the compression-wave velocity increases to about 3400 m/sec (11000 ft/sec) at 43 m (140 ft) below mudline. Below this depth to about 61 m (200 ft) below mudline, the shear- and compression-wave velocities of the rock are about 1370 m/sec (4500 ft/sec) and 3400 m/sec (11000 ft/sec), respectively. There is no measurement in the top 9 m (30 ft) of soil. The shear-wave velocity in the very soft Bay Mud (about 6 m thick) is assumed to increase from about 76 to 91 m/sec (250 to 300 ft/sec). The shear-wave velocity in the underlying loose sandy silt is assumed based on extrapolation from geophysical measurements. The compression-wave velocity in the top 9 m (30 ft) of soil is assumed to be 1500 m/sec (5000 ft/sec). As described in Section 5.1, the impedance function especially the real part at the top of the caisson is primarily affected by the dynamic properties of the underlying rock. The soft overburden soils have insignificant effects on the real part of the impedance function in both the horizontal and vertical directions of excitation. The P-wave velocity profile primarily affects the vertical response of the caisson. However, the vertical response of the caisson is controlled primarily by the response of the relatively rigid caisson and the underlying rock.

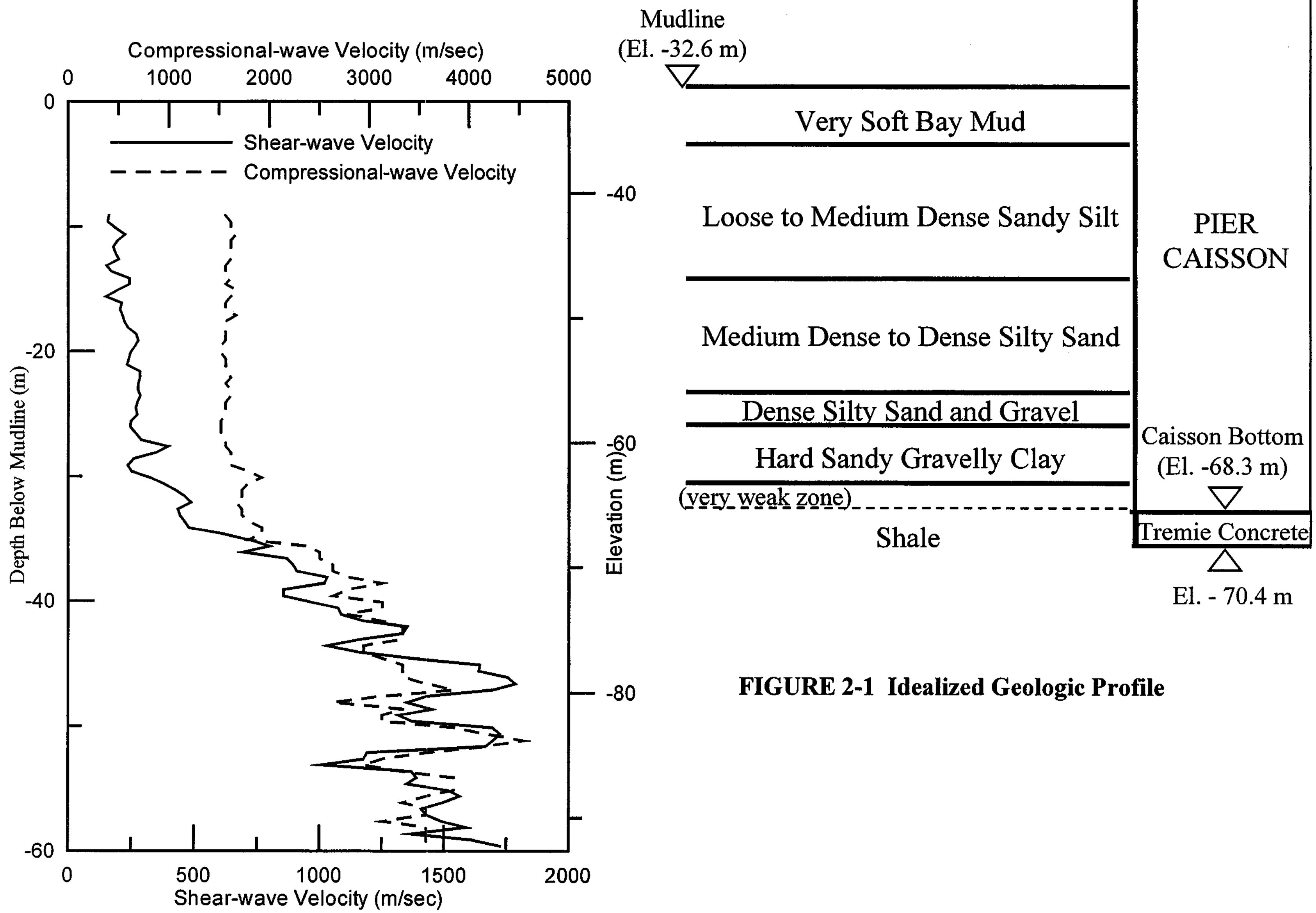


FIGURE 2-1 Idealized Geologic Profile

SECTION 3
DESIGN GROUND MOTIONS

The design rock motions were developed based on the ground motion study performed by Geomatrix (1992) and later modified by Abrahamson (1996). The acceleration, velocity, and displacement time histories of the rock motion (longitudinal, transverse and vertical components) are shown on Figure 3-1. The corresponding 5% damped response spectra are also shown on Figure 3-1. The predominant frequency of this motion is about 3 Hz. The time history used was derived by modifications of an actual time history to approximate the design response spectrum. The actual time history was selected from recordings that were obtained from the earthquake with magnitude and source-to-site distance similar to the design earthquake.

We understand that three sets of motion have been used for seismic retrofit design of the west spans of the San Francisco-Oakland Bay Bridge. For this study, only one set of time history was used. The purpose of the study is to qualitatively assess various factors affecting the caisson response. The effects of using other sets of motion may be secondary and are not addressed in this study.

The time history used was derived by modifications to an actual time history to approximate the design response spectrum. The actual time history was selected from recordings that were obtained from the earthquake with a similar magnitude and source-to-site distance.

3.1 Free-field Site Response

Free-field site response analyses were performed using the computer program SHAKE based on an equivalent linear approach. The shear- and compression-wave velocity profiles used in the analyses were idealized based on the geophysical measurement shown on Figure 2-1. The shear-modulus reduction and damping ratio curves for clays were selected based on Vucetic and Dobry (1991). Those curves for sands were assumed to be the average curves of Seed and Idriss (1970). No degradation in shear modulus of the underlying rock due to shaking is considered.

Based on our judgment, reduction of shear modulus of the underlying rock due to shaking will be small. Thus, the effect of degradation in shear modulus of the underlying rock will be small.

The acceleration, velocity, and displacement time histories of the computed mudline motions are shown on Figure 3-2. The corresponding 5% damped response spectra are shown on Figure 3-3. The results indicate that the site frequency corresponding to the design ground motions is about 0.9 Hz. The strain-compatible shear-wave velocity and damping ratio obtained in the site response analyses were used to obtain the dynamic soil properties for use in the SFSI analyses.

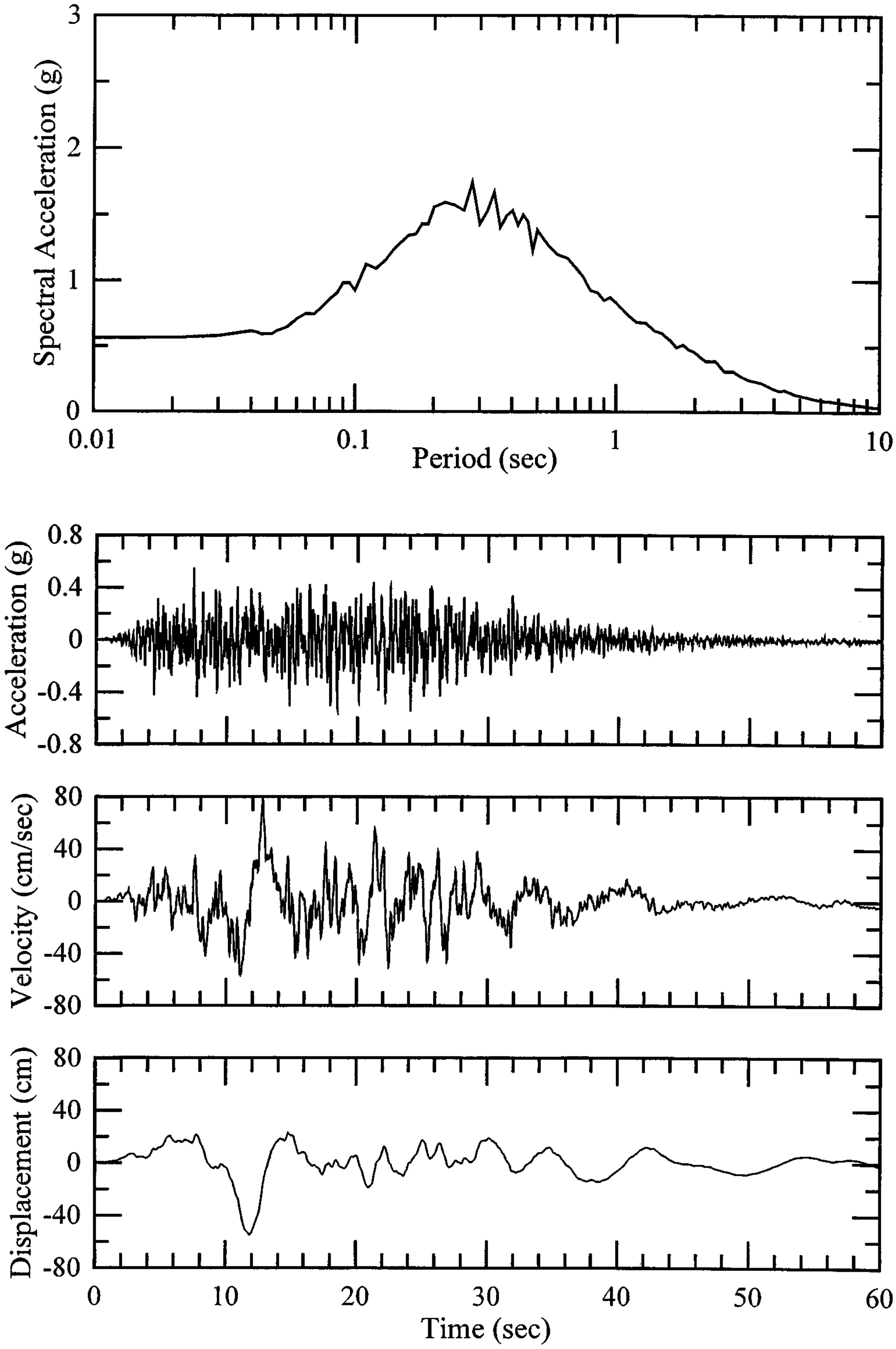


FIGURE 3-1a Longitudinal Input Rock Outcrop Motion

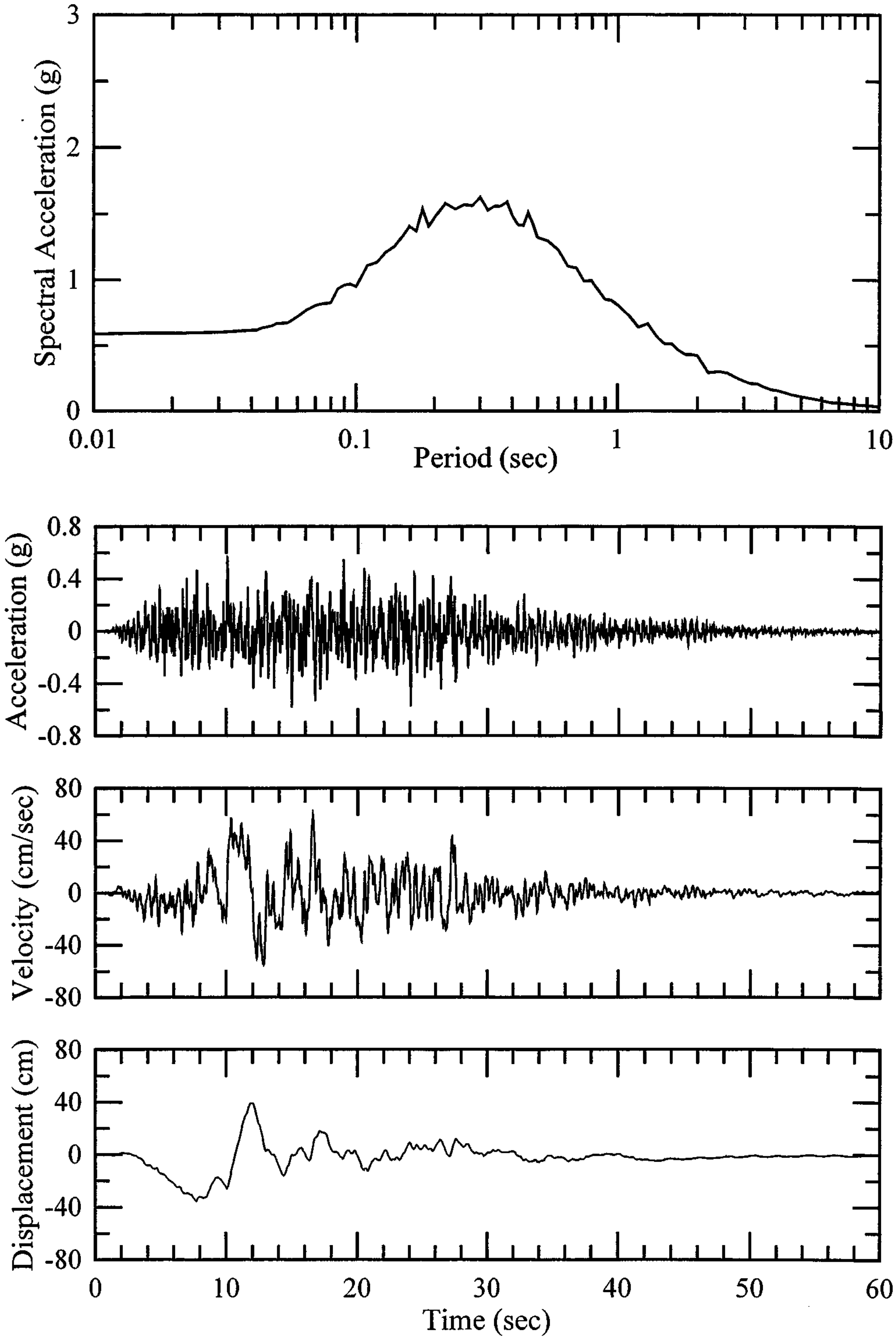


FIGURE 3-1b Transverse Input Rock Outcrop Motion

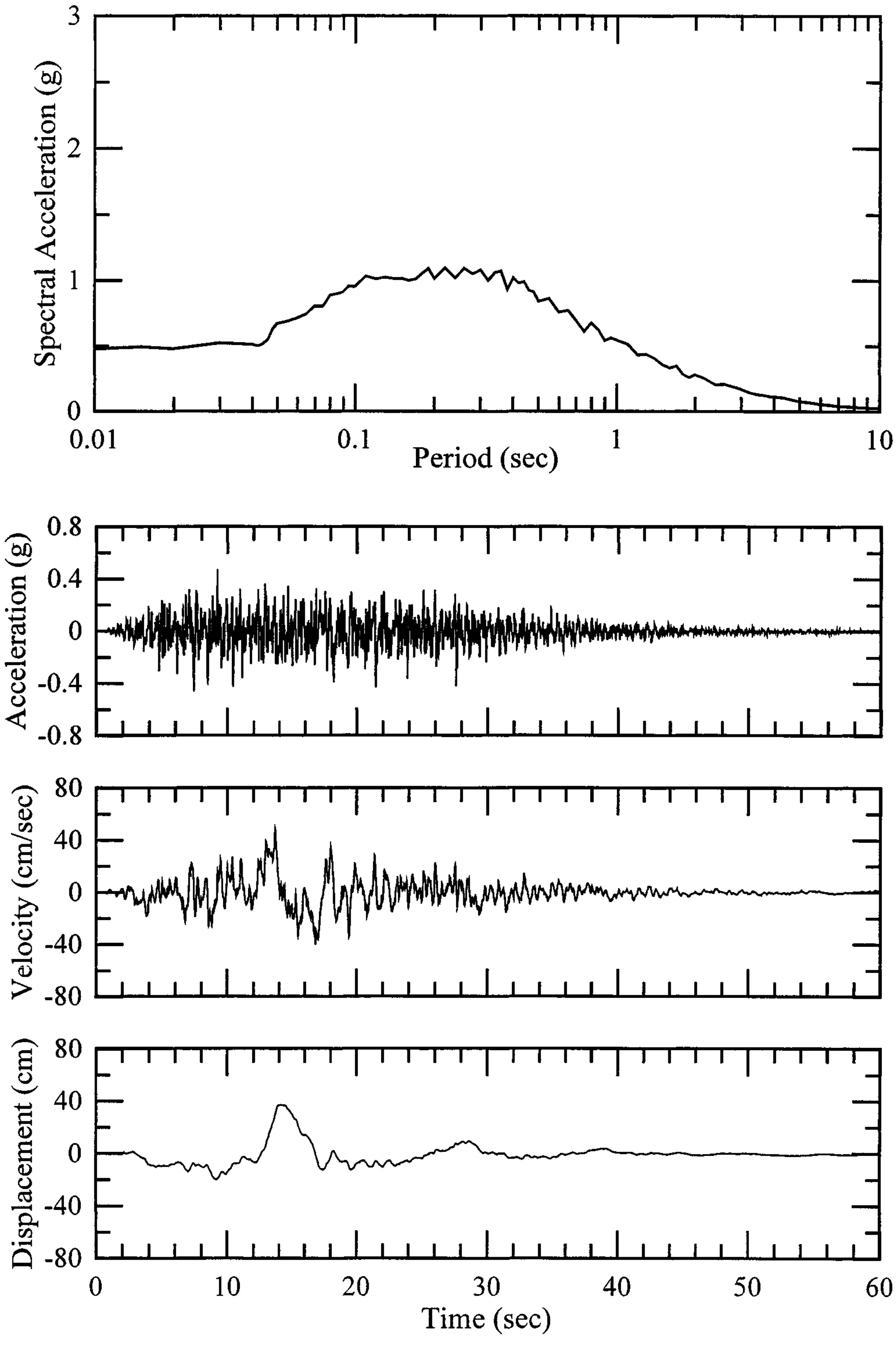


FIGURE 3-1c Vertical Input Rock Outcrop Motion

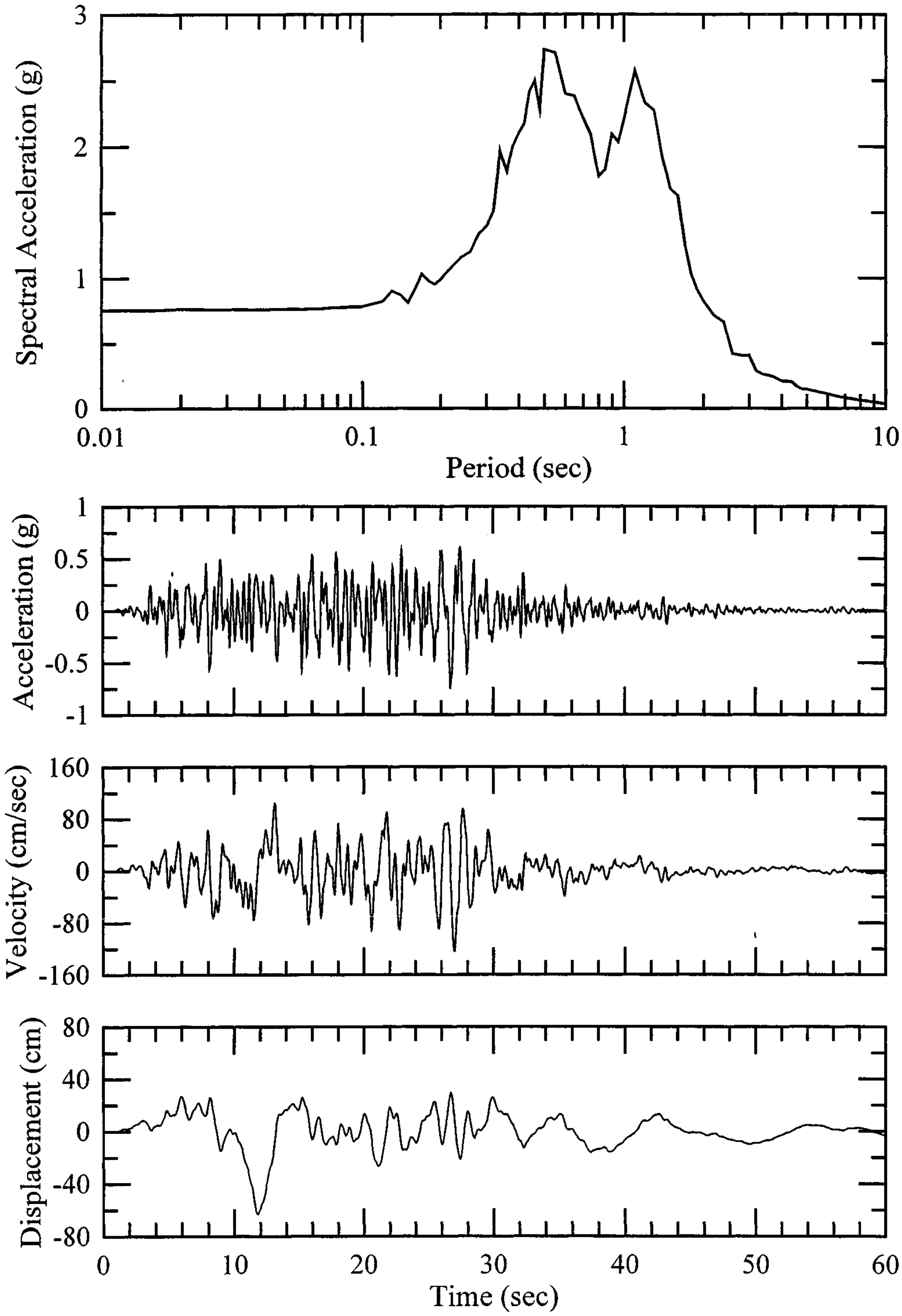


FIGURE 3-2a Computed Longitudinal Free-Field Mudline Motion

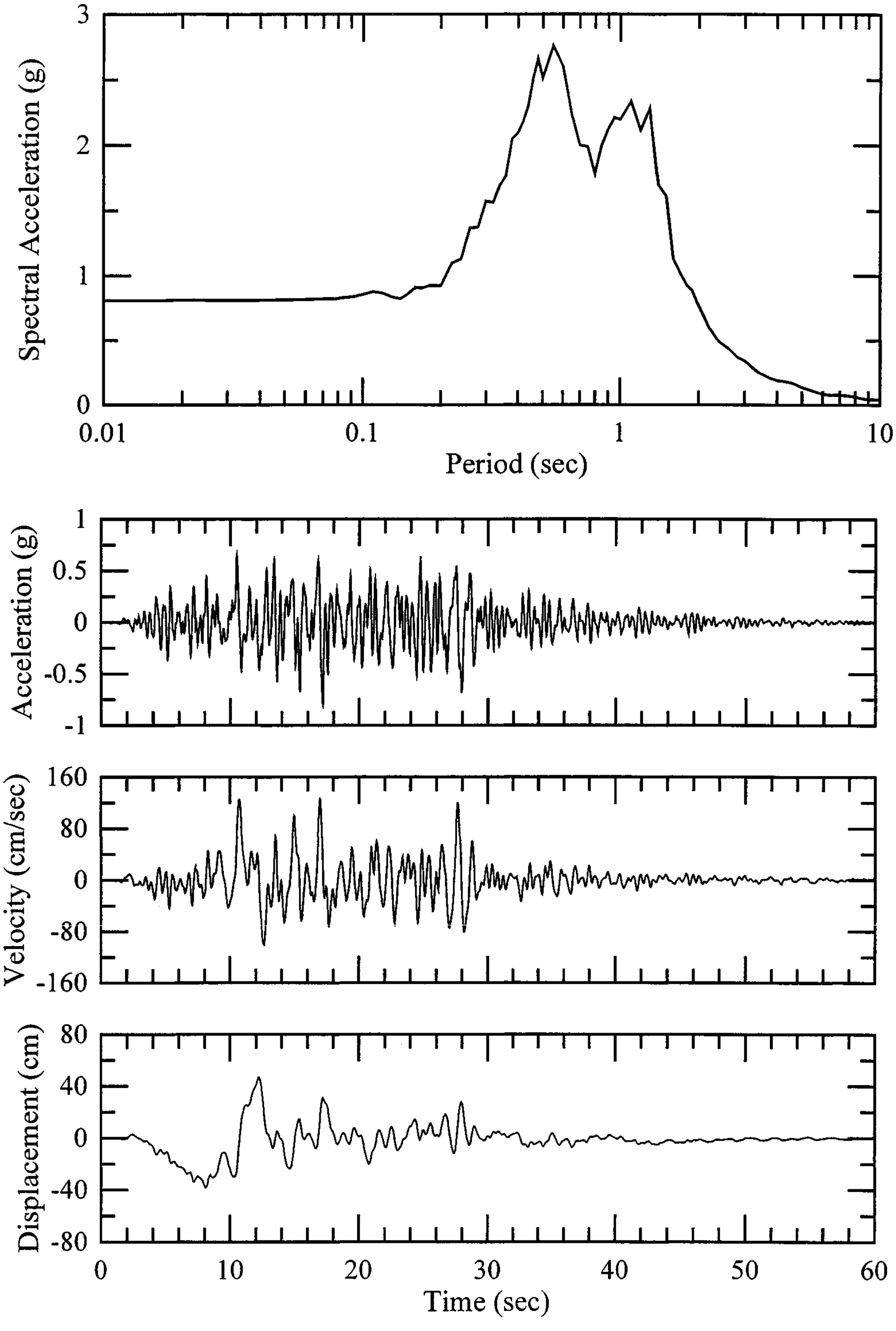


FIGURE 3-2b Computed Transverse Free-Field Mudline Motion

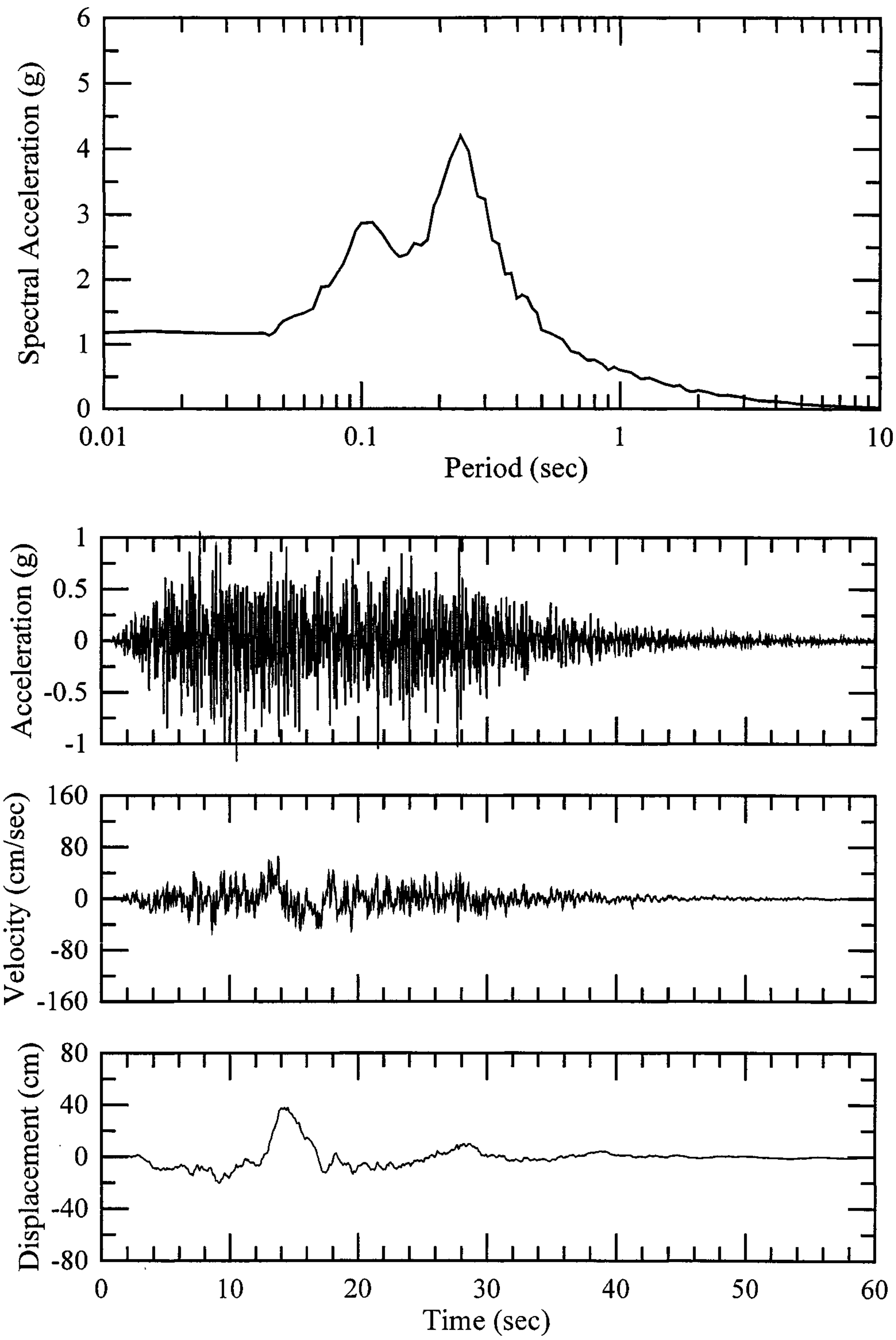


FIGURE 3-2c Computed Vertical Free-Field Mudline Motion

SECTION 4
CAISSON FOUNDATION

The west spans of the San Francisco-Oakland Bay Bridge consist of dual suspension bridges arranged back-to-back around a center anchorage. The general plan of Pier W3 is shown on Figure 4-1. The cellular concrete caisson is submerged in 33 m (107 ft) of water and is embedded in about 34 m (110 ft) of soil deposits. The caisson and the underlying tremie concrete seal penetrate about 4 m (14 ft) into rock. The caisson is 38.7 m (127 ft) long in the transverse direction and 22.9 m (75 ft) wide in the longitudinal direction with twenty-eight (4 by 7) 4.6 m (15 ft) diameter circular openings. The openings are filled with water and extend to 9 m (30 ft) above the caisson bottom. The top of the caisson is located at 7.6 m (25 ft) above the water level.

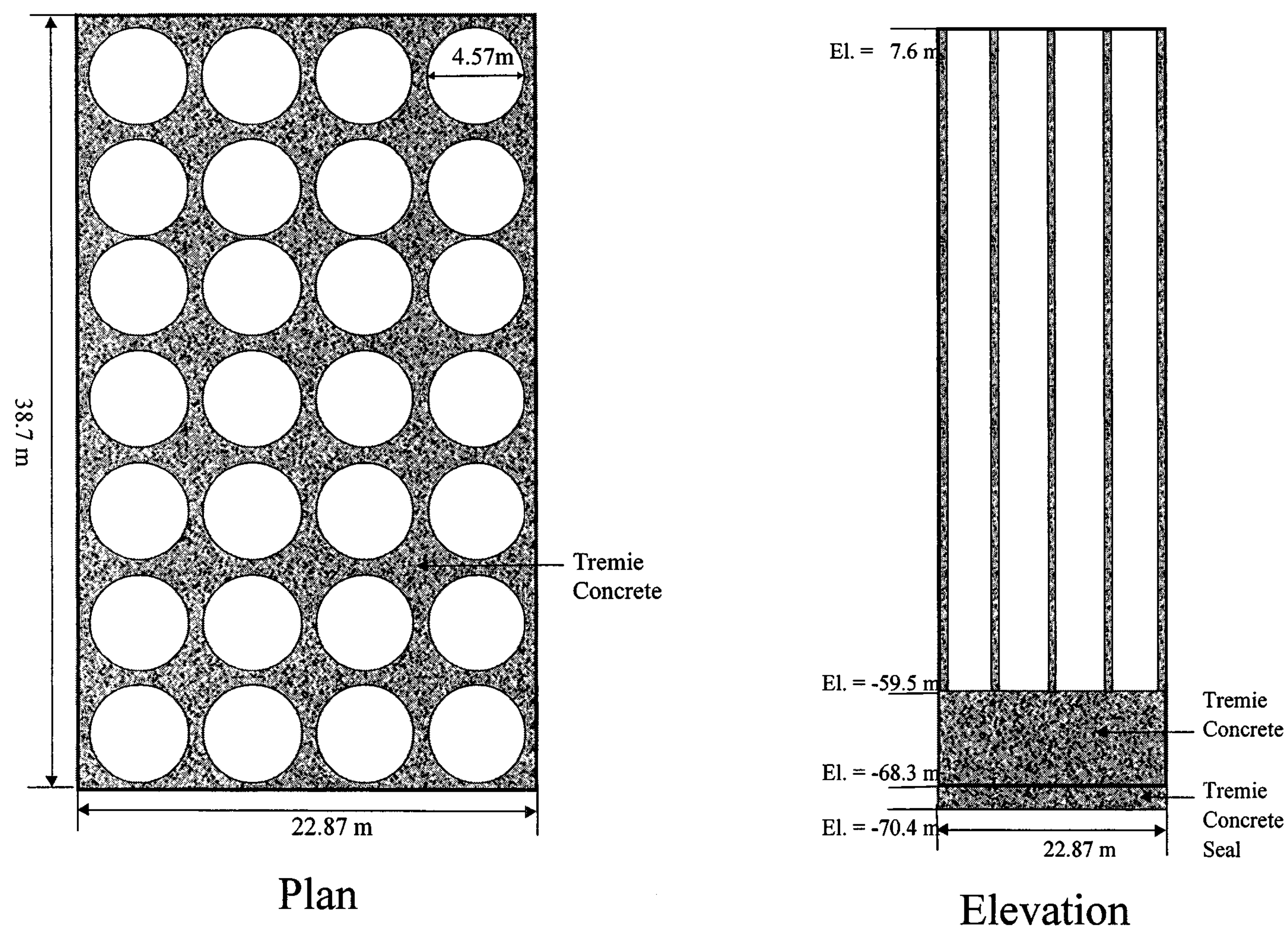


FIGURE 4-1 General Plan and Elevation of Caisson at Pier W3

SECTION 5
THREE-DIMENSIONAL EQUIVALENT LINEAR ANALYSES

Three-dimensional dynamic analyses of the caisson were performed using the SASSI computer program (Lysmer et al., 1988). A quarter model of the caisson was analyzed to take advantage of the symmetrical/anti-symmetrical conditions. The SASSI 'structure' finite element mesh is shown on Figure 5-1. The mesh includes the caisson, superstructure tower, suspension cables, and two layers of soil/rock finite elements surrounding the caisson. Rigid links were added at the top of the caisson to distribute the forces from the superstructure. The caisson was modeled by solid brick elements whose dynamic properties were selected based on smearing the composite flexural and shear rigidities of the caisson. The hydrodynamic masses accounting for the dynamic effects of water surrounding the caisson and inside the circular openings were included in the model (Goyal and Chopra, 1988). The hydrodynamic masses simulating the water in the internal openings were smeared in the model and the hydrodynamic masses simulating the external water surround the caisson were treated as lumped masses. The program SASSI was modified to include frequency-dependent springs for modeling the suspension cables. The springs were connected to the superstructure on one end and free-field rock outcrop excitation motions were prescribed at the other end of the springs.

To account for SFSI in the seismic analysis of a global superstructure model of a long span bridge using a substructuring approach, foundation impedance functions at the base of bridge piers or the top of caissons generally are required as input to the analysis. Also required are the input scattered motions incorporating the SFSI of the caissons at the same locations. To compute the foundation impedance functions and scattered motions at the top of the caisson, a foundation substructure model was created by removing the superstructure and cables from the mesh shown on Figure 5-1.

To study the effects of soil embedment on the seismic response of the impedance functions and scattered motions, another model was created by removing the soil elements in the foundation substructure model.

All cases analyzed by the equivalent linear techniques described in this section and Section 6 assume perfect bond and no slip along the soil-caisson and rock-caisson interface.

5.1 Characteristics of Impedance Functions at Top of Caisson Foundation

The model shown on Figure 5-1 was analyzed using the dynamic finite element computer program SASSI (Lysmer et al., 1988). Figures 5-2 and 5-3 show the real and imaginary parts of the impedance functions (6 x 6), respectively, at the top of the caisson at Pier W3 in the longitudinal direction. The impedance functions are frequency-dependent. To account for the frequency-dependent characteristics of the impedance functions (at least for the real parts) in the dynamic structural analysis using conventional codes, the following idealizations were made.

The impedance functions are defined as follows:

$$K_{ij}(\omega) = k_{ij}(\omega) + i\omega c_{ij}(\omega) \quad (i, j = 1,6) \tag{5-1}$$

where k_{ij} is the real part of the impedance, ωc_{ij} is the imaginary part, and ω is circular frequency. The impedance functions expressed by Equation (5-1) are a 6 x 6 symmetric matrix. The diagonal terms k_{ii}, $i = 1,3$ are associated with translations, and k_{ii}, $i = 4,6$ are associated with rotations. The off-diagonal terms represent coupling between translations and rotations. For the pier analyzed, only the off-diagonal terms associated with the horizontal translation and rocking ($i=1, j=5$; and $i=2, j=4$) are significant; the remaining off-diagonal terms are negligible.

An attempt was made to fit a polynomial function through the real and imaginary parts of the computed impedance functions. The real parts of the impedance functions, k_{ij}, were fitted by:

$$k_{ij}(\omega) = (k_o)_{ij} - \omega^2 m_{ij} \tag{5-2}$$

where $(k_o)_{ij}$ is the static stiffness and $(m)_{ij}$ is the equivalent mass or mass moment of inertia. For the imaginary parts, a third-order polynomial function was used to fit to the data.

$$\omega c_{ij} = A_{ij} + B_{ij}\,\omega + C_{ij}\,\omega^2 + D_{ij}\,\omega^3 \tag{5-3}$$

The real part of the computed impedance was fitted reasonably well by Equation (5-2) (shown as solid lines on Figure 5-2), indicating that the frequency-dependent stiffness can be reasonably approximated by the use of a static stiffness and a mass (or mass moment of inertia) at frequencies up to about 2.5 to 4 Hz for the horizontal translations and rotations. The imaginary parts of the impedance functions shown on Figure 5-3 are strongly frequency-dependent, resulting in dashpot coefficients that also are frequency-dependent.

As described in Section 3, no degradation in shear modulus of the underlying rock is considered. Regarding effects of variations of shear modulus or shear-wave velocity of the underlying rock on the response, reduction in the shear-wave velocity of the rock will proportionally decrease the predominant frequency of the caisson response. However, based on the relatively flat slope of the impedance function (Figures 5-2, 5-3 and 5-6, 5-7) at the predominant frequency of the caisson response (1.4 to 1.7 Hz shown in Section 5.2), we anticipate that the effects on the caisson response are small.

5.2 Scattered Motions at Top of Caisson Foundation

The response spectra (5% damped) of the acceleration time histories (i.e., foundation scattered motion) computed at the top of the caisson without the tower are shown on Figure 5-4. Also shown are the response spectra of the rock motion and the free-field mudline motion. Generally, the response spectra of the motions at the top of the caisson are amplified from the rock motion and are lower than the mudline motion at periods longer than 1 second for the longitudinal component, at periods longer than 0.8 second for the transverse component, and in the entire period range for the vertical component. Comparisons of the response spectra (5% damped) for the motions at the top of the caisson with and without the superstructure (tower) are shown on Figure 5-5. The two sets of motions are similar in frequency content and spectral values, indicating insignificant effects of the tower on the response of this caisson.

5.3 Effects of Soil Embedment on Impedance Functions and Foundations Scattered Motions

Because caisson foundations generally are embedded in soft soil deposits, it is desirable to examine effects of the upper soil deposits on the response of the caisson or specifically on the impedance functions and input foundation motions at the top of the caisson. Impedance functions and input motions were computed and compared for the two caisson models at Pier W3, one with and the other without soil embedment.

The real and imaginary parts of the impedance functions computed at the top of the caisson in the longitudinal direction for the two cases (with and without soil embedment) are compared on Figures 5-6 and 5-7 (solid lines are for the case with soil embedment; dashed lines are for the case without soil embedment), respectively. Generally, the real part of the impedance functions for the case with soil embedment is slightly higher than that for the case without soil embedment; the differences are generally less than a few percent. However, the imaginary part is significantly higher for the case with soil embedment than that for the case without soil embedment, reflecting greater radiation damping associated with soil embedment. The effect increases with frequency. Comparisons of the response spectra of the scattered motions for the two cases (with and without soil embedment) indicate that soil embedment has little effects on the frequency content of the scattered motion.

5.4 Seismically Induced Soil Stresses Surrounding the Caisson

Dynamic stresses in the soils surrounding the caisson (along the base and side of the caisson) were calculated and compared with static hydrostatic stresses. The results indicated that dynamic stresses calculated from the SASSI analyses (based on equivalent linear techniques) are significantly higher than the static hydrostatic stresses, indicating a likelihood of separation (i.e., uplift along the base and gapping along the side of the caisson). Thus there is a need to perform nonlinear response analyses to examine the effects of potential uplift and gapping on the response of the caisson.

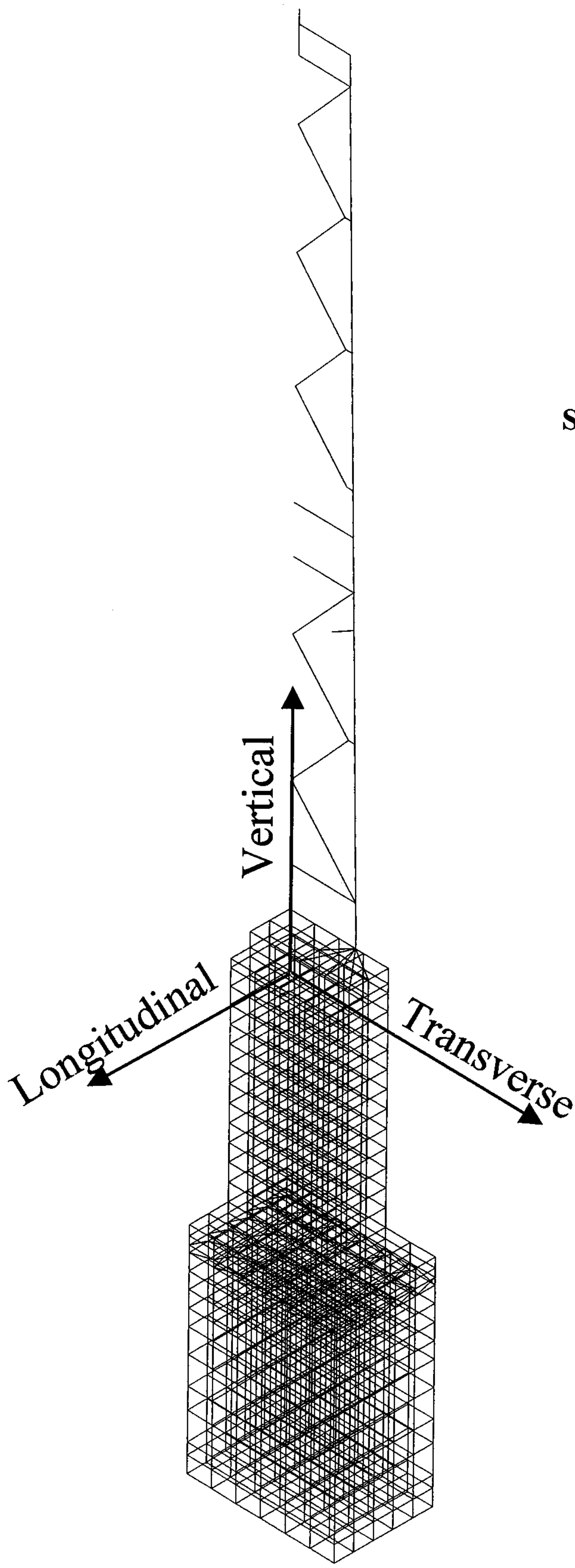


FIGURE 5-1
SASSI Finite Element Quarter Model of Caisson at Pier W3

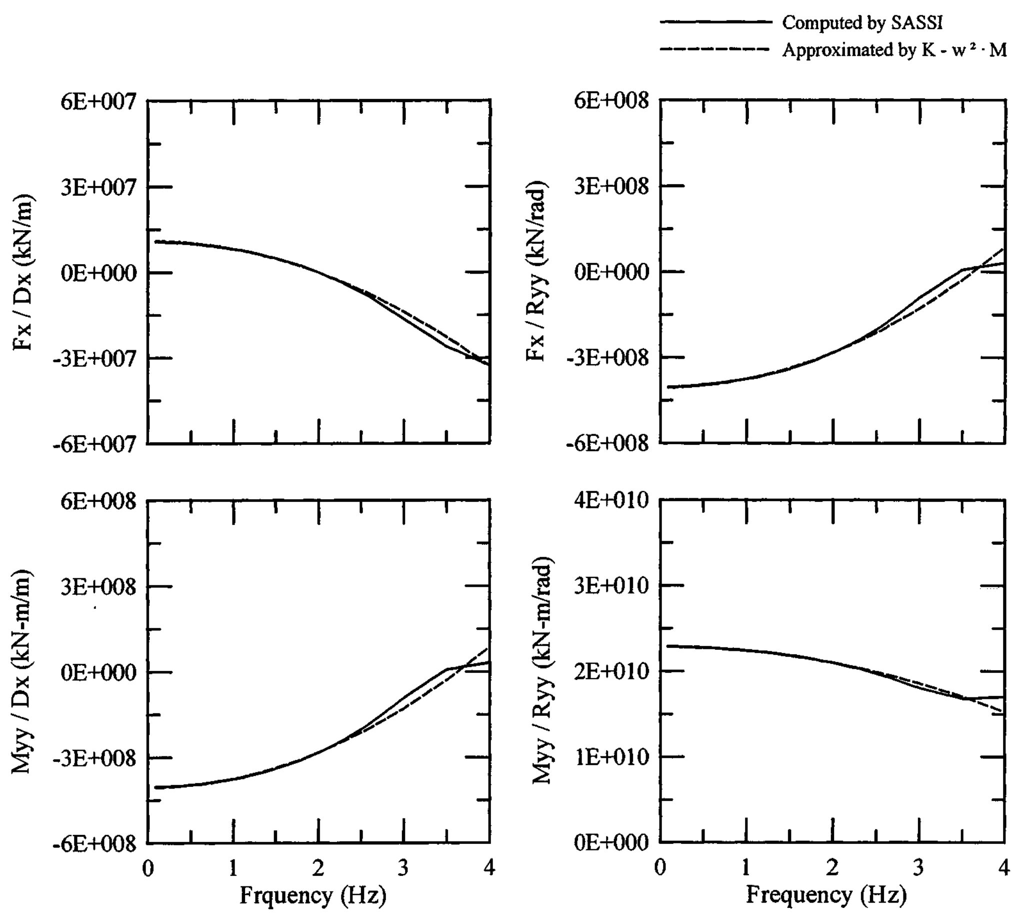


FIGURE 5-2 Real Part of Foundation Impedance Functions at Caisson Top

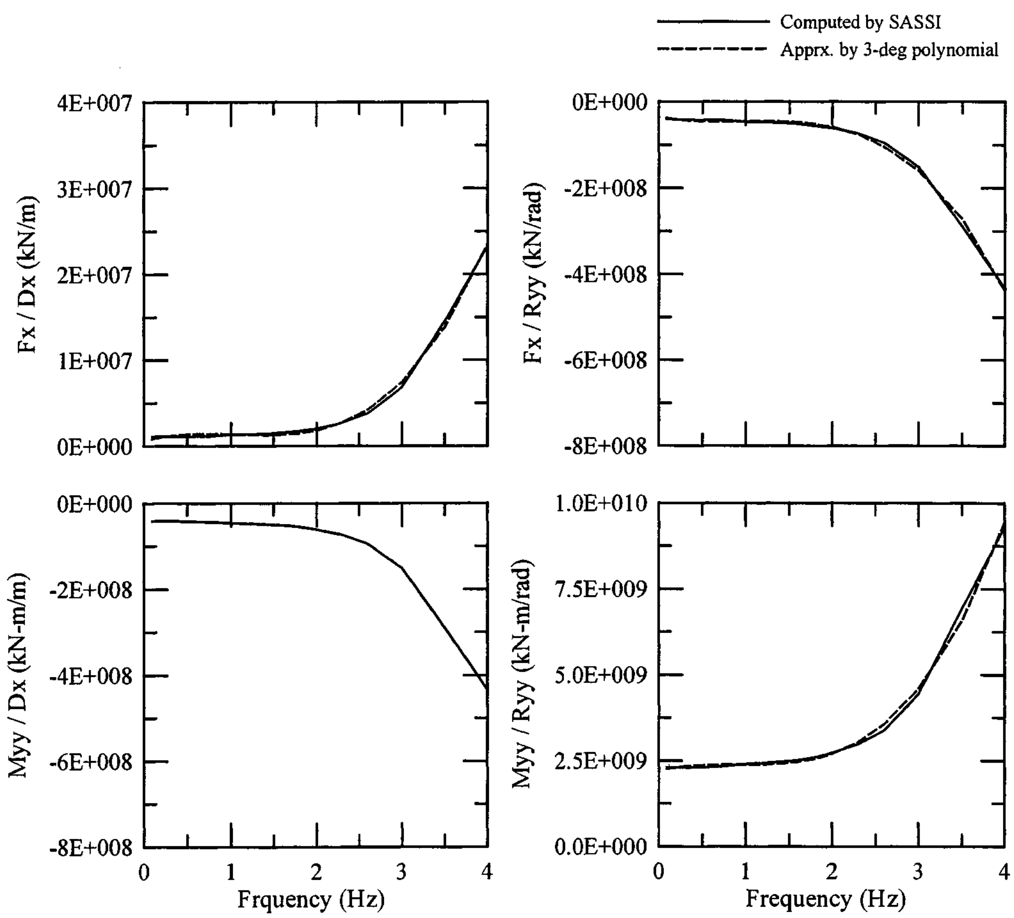


FIGURE 5-3 Imaginary Part of Longitudinal Foundation Impedance Functions at Caisson Top

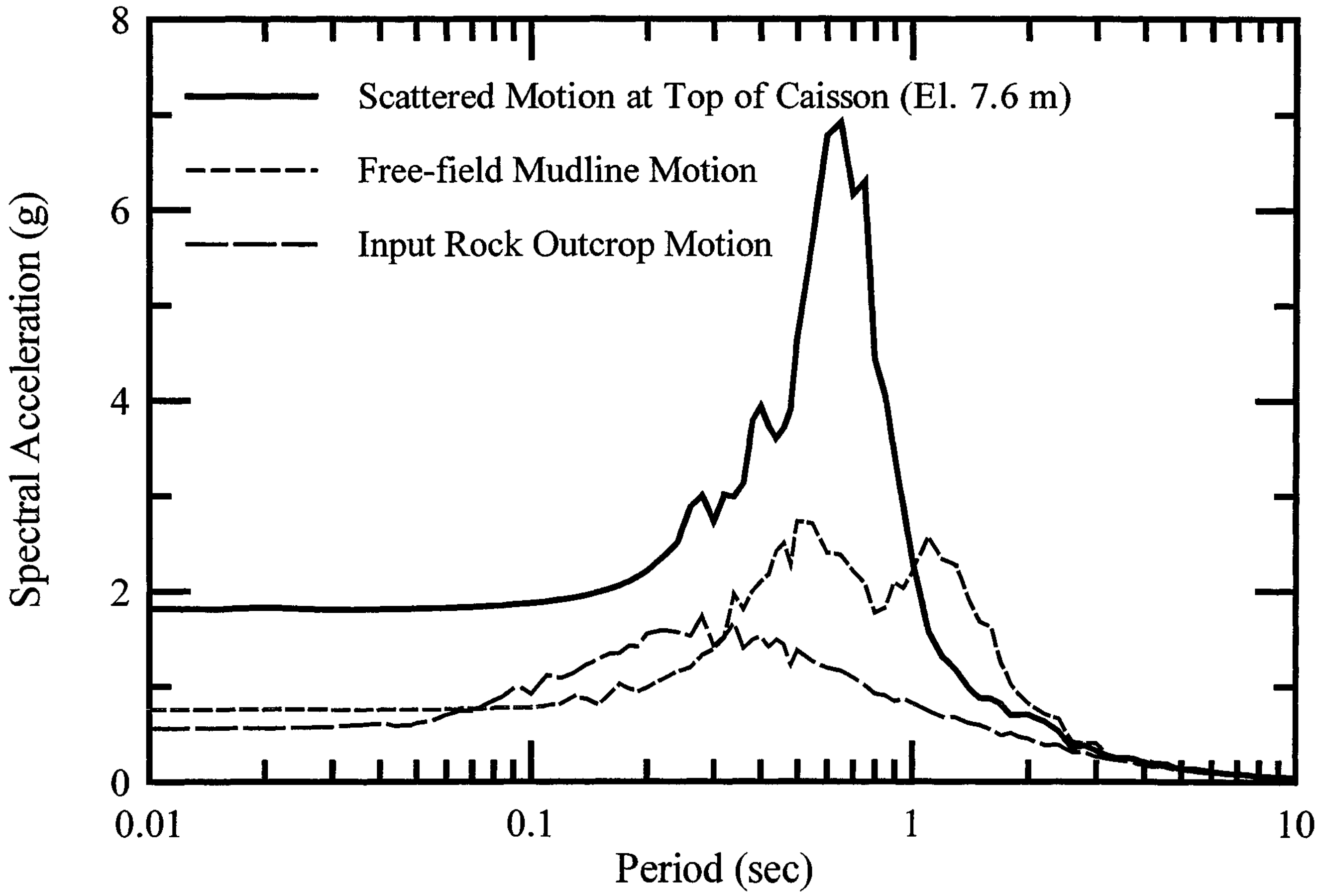


FIGURE 5-4a Comparison of 5% Damped Longitudinal Response Spectra of Substructure Scattered Motions with Computed Free-Field Mudline and Input Rock Outcrop Motions

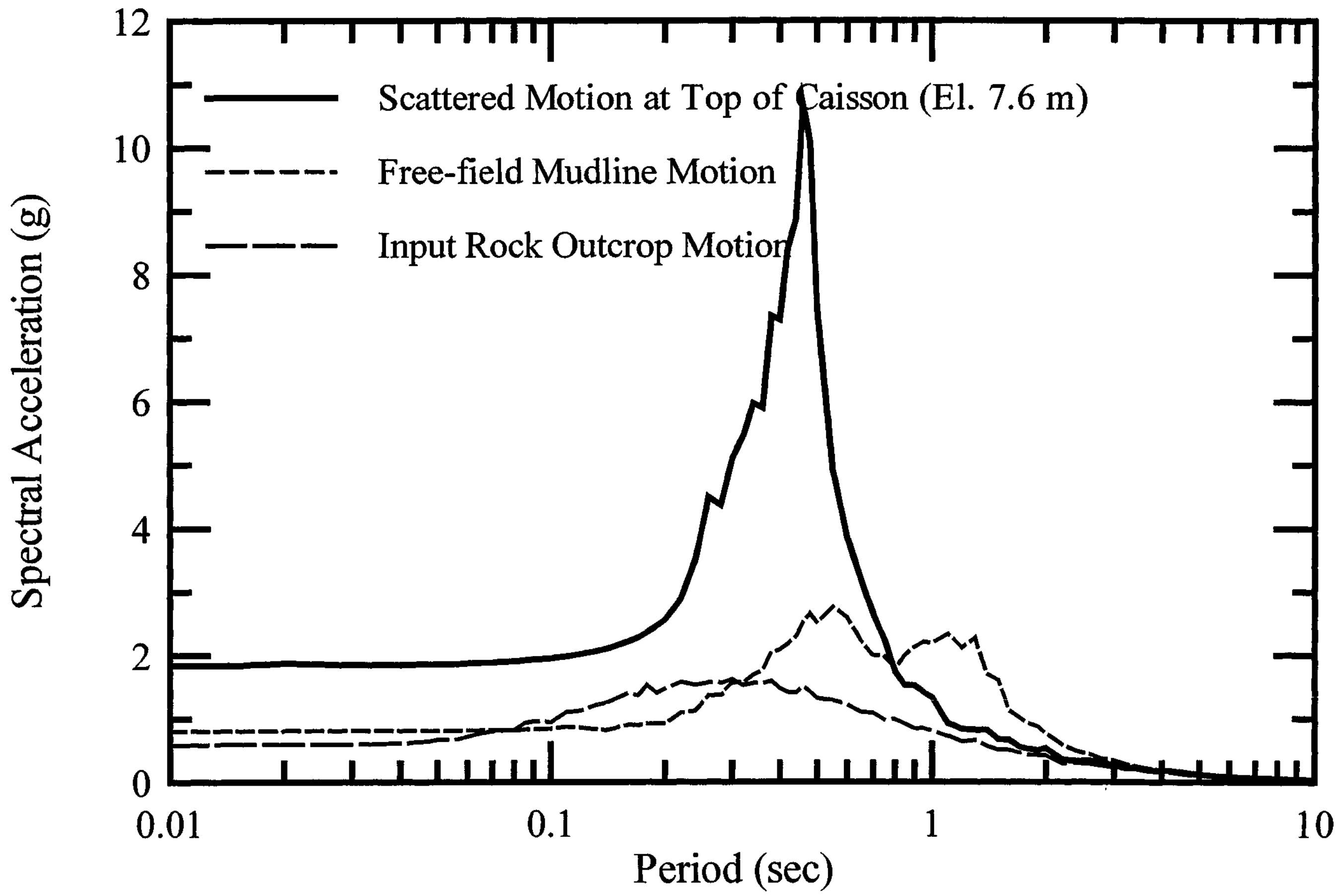


FIGURE 5-4b Comparison of 5% Damped Transverse Response Spectra of Substructure Scattered Motions with Computed Free-Field Mudline and Input Rock Outcrop Motions

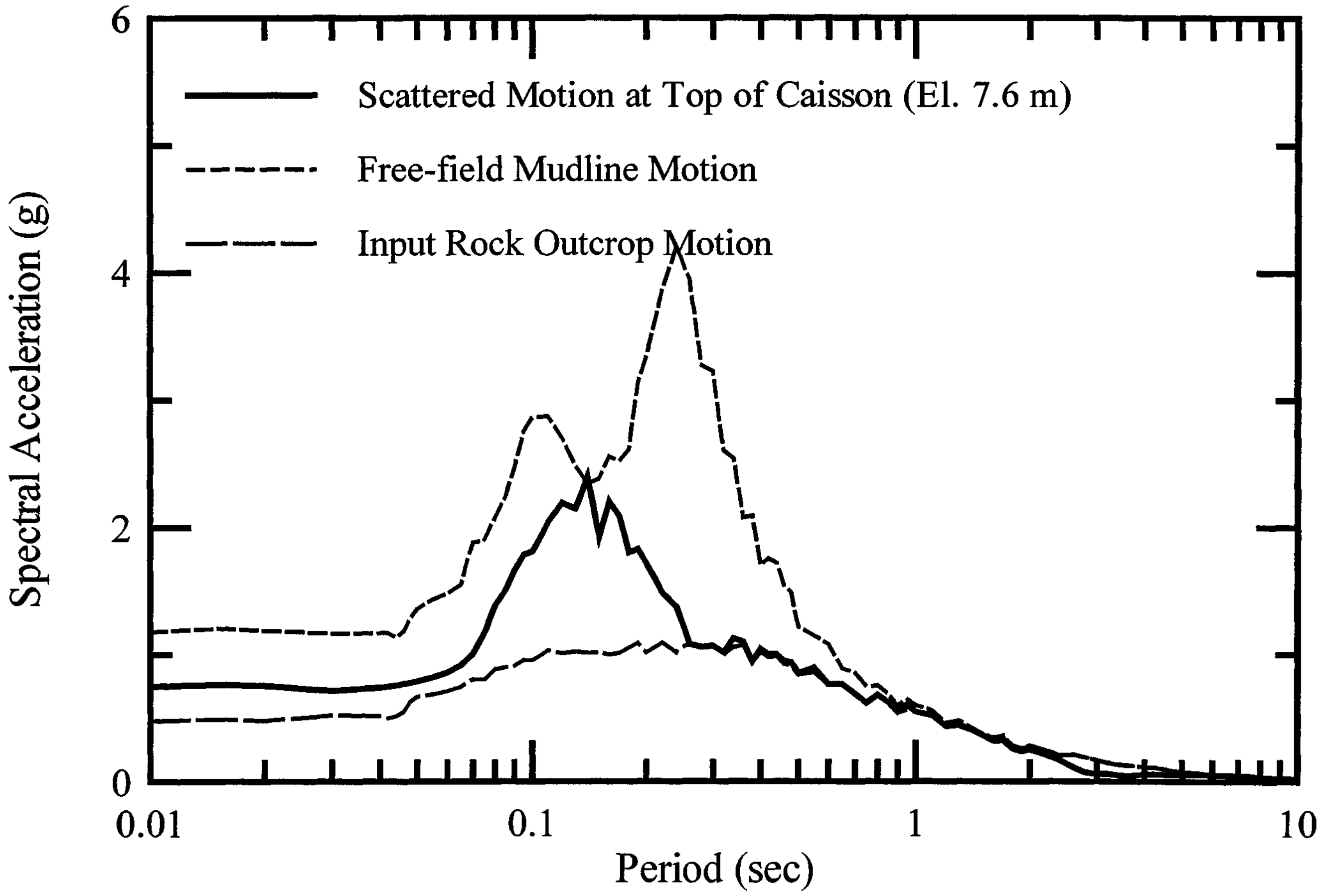


FIGURE 5-4c Comparison of 5% Damped Vertical Response Spectra of Substructure Scattered Motions with Computed Free-Field Mudline and Input Rock Outcrop Motions

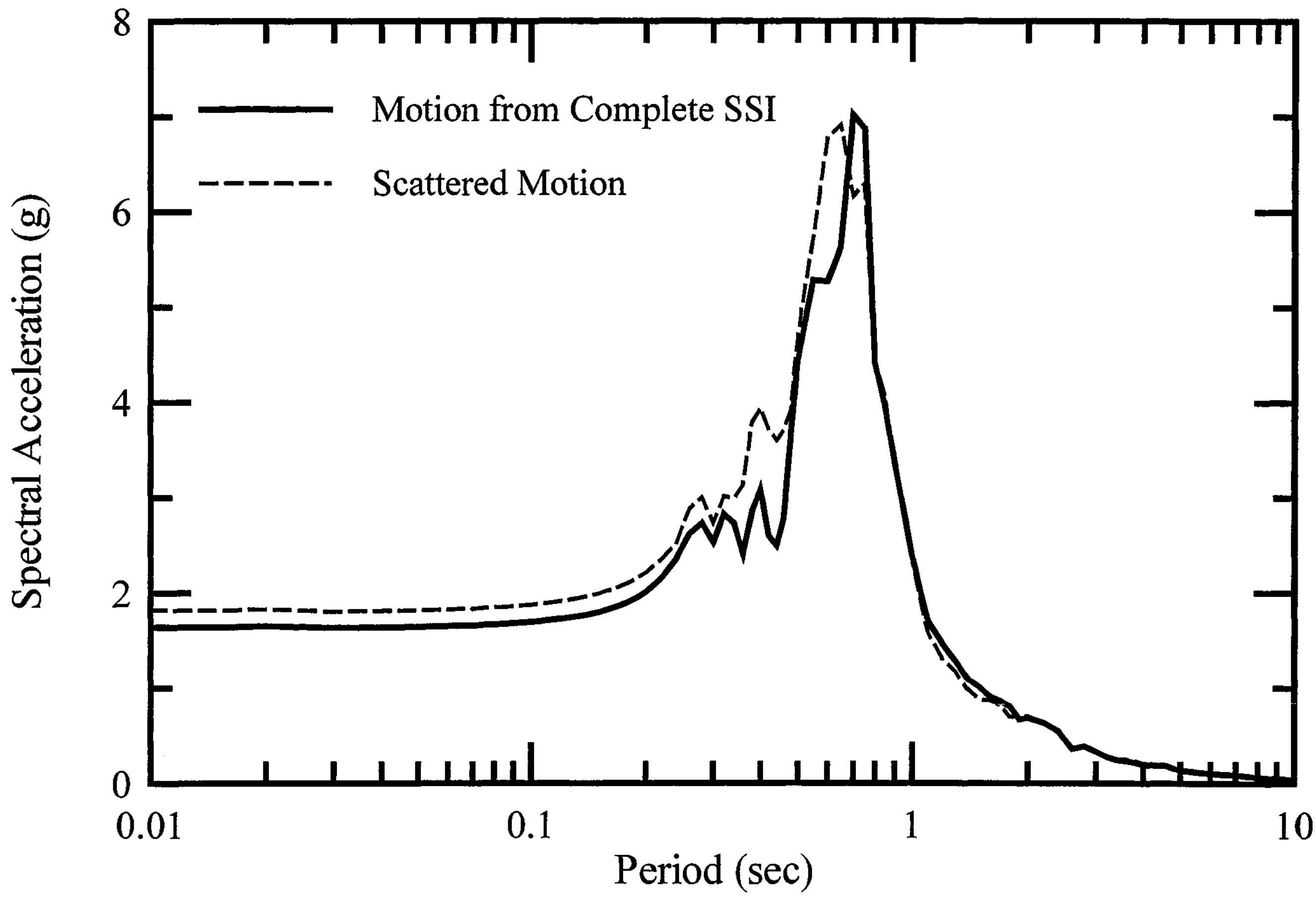


FIGURE 5-5a Comparison of 5% Damped Longitudinal Response Spectra of Substructure Scattered Motions with Motions at Caisson Top from Complete SSI Analyses

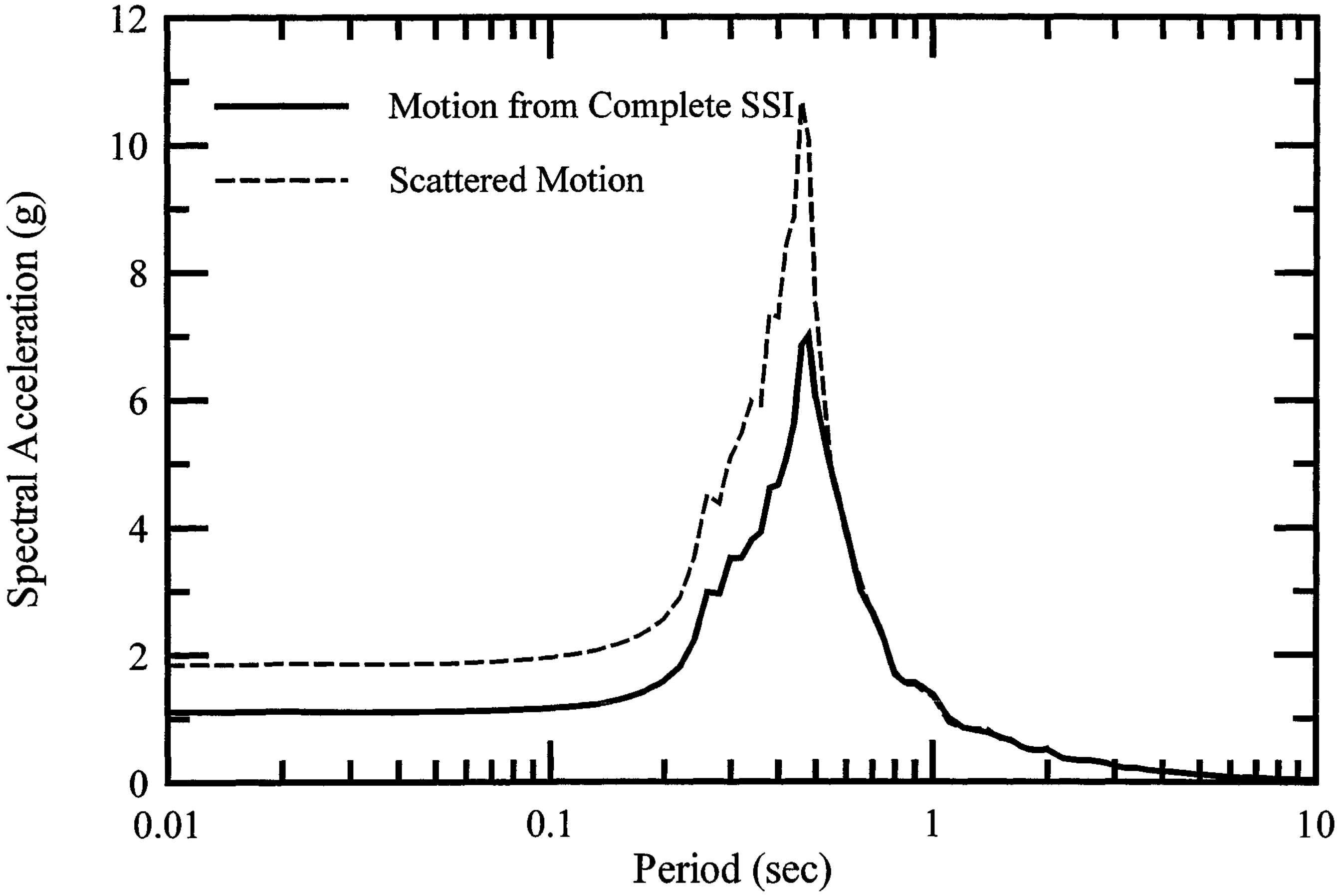


FIGURE 5-5b Comparison of 5% Damped Transverse Response Spectra of Substructure Scattered Motions with Motions at Caisson Top from Complete SSI Analyses

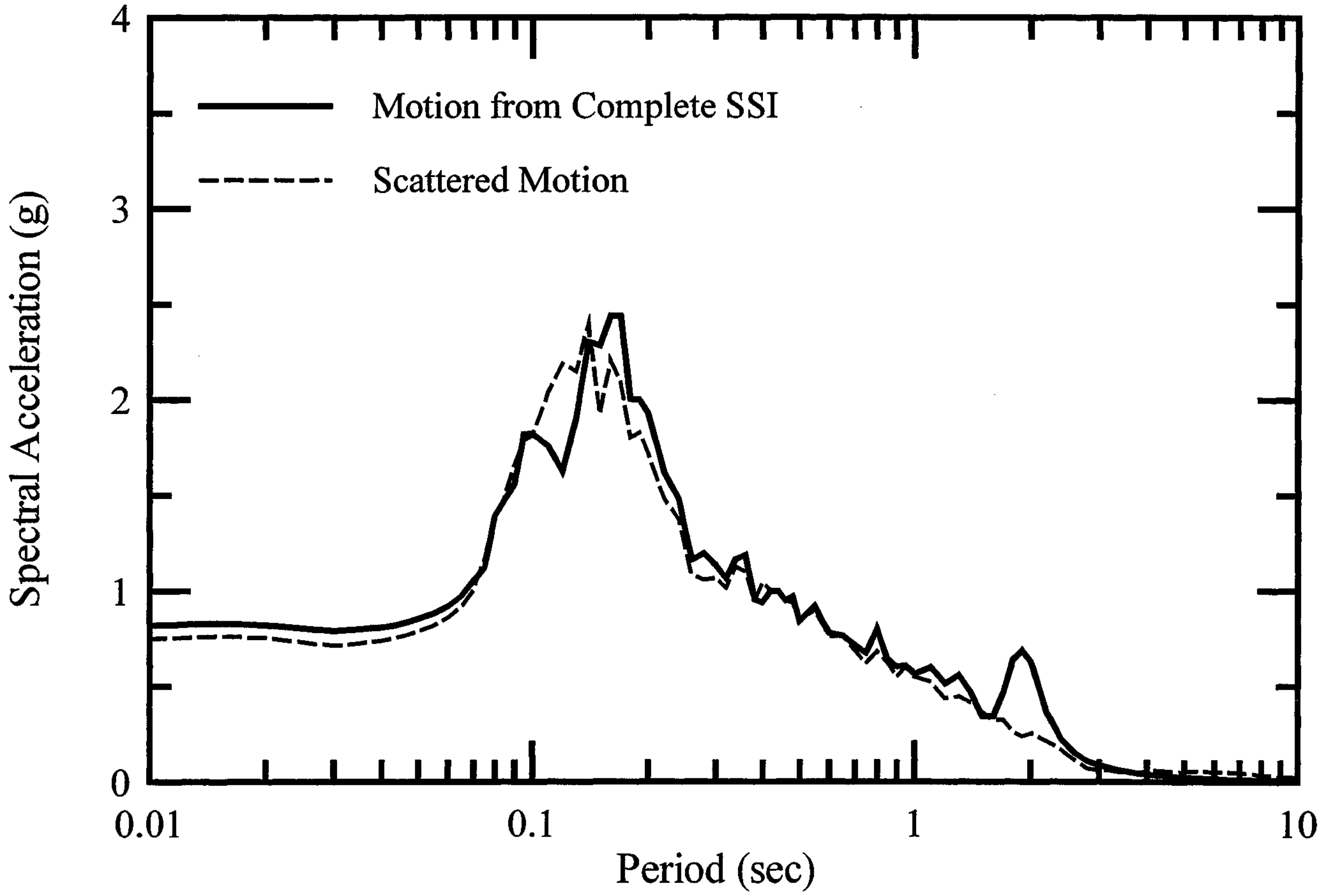


FIGURE 5-5c Comparison of 5% Damped Vertical Response Spectra of Substructure Scattered Motions with Motions at Caisson Top from Complete SSI Analyses

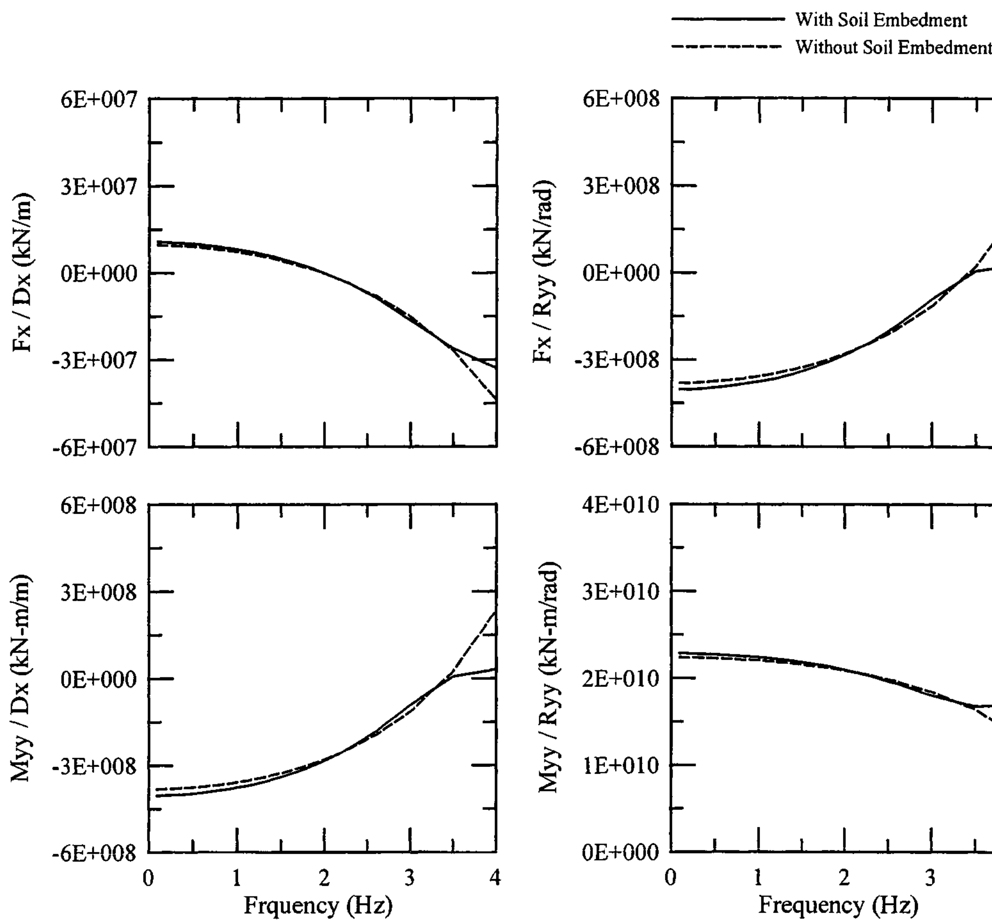


FIGURE 5-6 Comparison of Real Part of Foundation Impedance Functions for Cases With and Without Soil Embedment

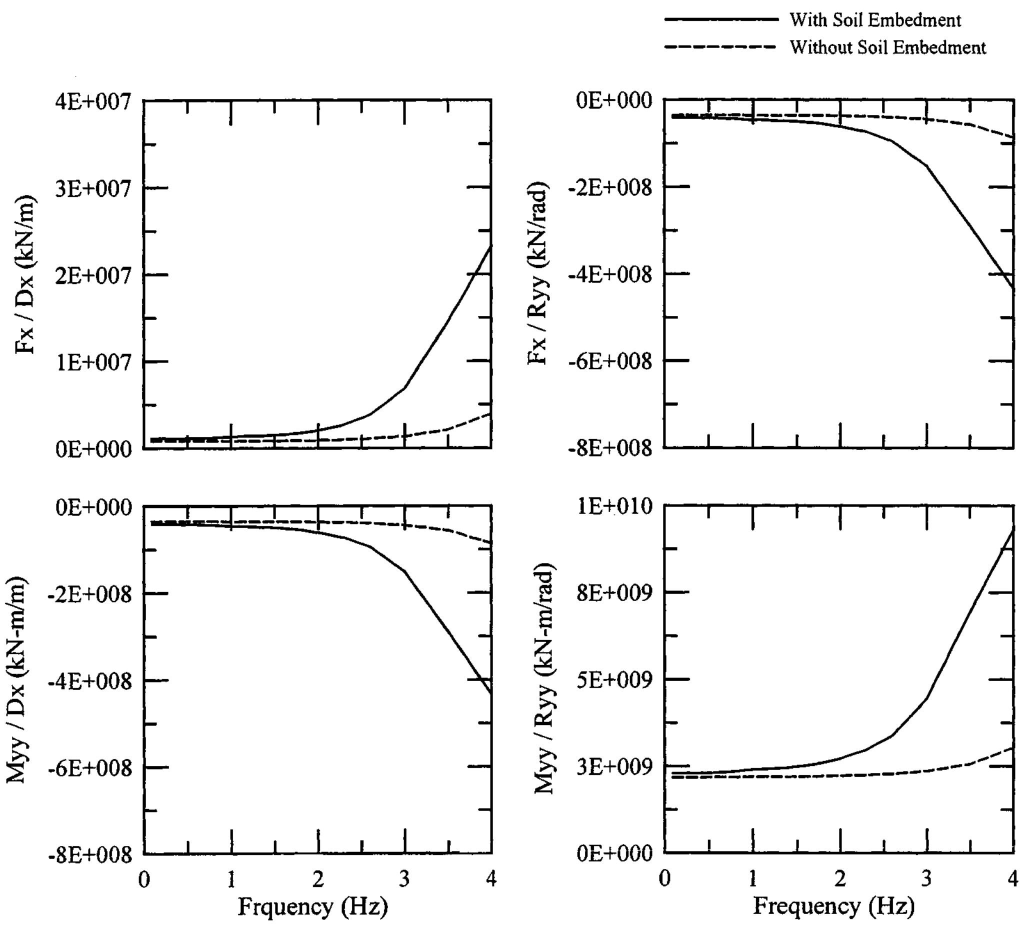


FIGURE 5-7 Comparison of Imaginary Part of Foundation Impedance Functions for Cases With and Without Soil Embedment

SECTION 6

TWO-DIMENSIONAL EQUIVALENT LINEAR AND NONLINEAR ANALYSES USING SASSI AND FLAC

6.1 Introduction

Both the equivalent linear and nonlinear analyses of 2-dimensional models of the caisson of Pier W3 in the longitudinal direction (short axis) were performed. The equivalent linear analysis was performed using the computer program SASSI. The purpose of the equivalent linear analysis of a 2-D model is to examine accuracy of the response of a 2-D model of the caisson as compared with that of a 3-D model analyzed in Section 5. The nonlinear analyses were performed using the computer program FLAC. The purpose of the non-linear analyses is to evaluate the significance of soil-caisson gapping, rock-caisson uplifting separation, and near-field soil softening on the scattered motions and stresses developed in the caisson.

6.2 Two-Dimensional Equivalent Linear Analysis Using SASSI

For the 2-D equivalent linear analysis, the model was developed by considering a unit-width strip of the 3-D model described in Section 5 without the superstructure and cables. The results of the 2-D analysis indicate that the impedance functions and scattered motions obtained from the 2-D analysis are similar to those from the 3-D analyses, suggesting that a 2-D model can reasonably approximate the seismic response of the caisson in the longitudinal direction. Figure 6-1 shows the comparison of the response spectra of the scattered motions computed from the 2-D and 3-D models.

6.3 Two-Dimensional Linear and Nonlinear Analyses Using FLAC

As described previously, both linear and nonlinear analyses were performed using the finite difference program FLAC. In FLAC, a visco-elastic constitutive model was used to represent the dynamic behavior of the soil and rock. Damping was treated as Rayleigh damping. A damping ratio of 5 percent at a frequency of 4 Hz was specified in the analyses. The dynamic soil parameters of this model were calibrated to those used in the equivalent linear analyses. Analyses were also performed using a newer version of FLAC in which variable damping was also used. In the analysis using the option of variable damping, a damping of 15% was used for soil based on the results of the equivalent linear analysis, and a damping of 5% was used for concrete. The analyses were performed in time domain. A Lagrangian approach is used to account for large-strain finite difference grid deformation. The finite difference grid used in the analyses is shown on Figure 6-2. Interfaces were added to model potential gapping, lifting, and sliding at the soil-caisson and rock-caisson contacts. It was developed based on the finite element mesh used in the equivalent linear analyses. The grid boundaries were extended sufficiently far away from the caisson to reduce the boundary effects on the caisson response. Viscous dashpots were attached to the boundaries to simulate the wave propagation through a semi-infinite medium. The input control motion was defined at the base and was obtained as an interface motion at the appropriate depth from the free-field site response analyses.

Figure 6-3 shows the comparison of 5% damped response spectra of the motions computed by the 2-D SASSI and FLAC analyses at the top of the caisson assuming no interface gapping, lifting, or sliding (i.e., linear analyses). The comparison of the acceleration time histories at the center of the caisson at the top, mudline, and base levels is shown on Figure 6-4. The shear and bending moment time histories induced in the caisson at the mudline, above-tremie seal, and tremie seal levels are compared on Figures 6-5 and 6-6. These comparisons show that the results of the equivalent linear models analyzed by SASSI and FLAC programs are similar.

Two sets of the nonlinear analyses were performed. The first set of the analyses was performed to vary the interface strength with the surrounding soil modeled by equivalent linear elastic properties. For these analyses, three cases of the interface strength were analyzed: smooth interface (i.e., zero interface strength), moderate interface strength, and glued interfaces (i.e., perfect contact). The second set of the analyses was performed by softening the moduli of two soil columns adjacent to the caisson. The moduli of the first soil column immediately adjacent to the caisson was reduced by 50 percent and those of the second soil column was reduced by 25 percent.

Figures 6-7 through 6-10 show comparisons of response spectra, acceleration time histories, and caisson shear and bending moment obtained for the first set of analyses for smooth interfaces, moderate interface strength, and glued interfaces. The results indicate that the seismic motions and stresses developed in the caisson are sensitive to the interface properties. A softer interface tends to reduce the peak response, but it does not significantly affect the frequency characteristics of the response. For the extreme case (i.e., smooth interface), the peak spectral value of the scattered motion at the top of the caisson was reduced by 50 percent (Figure 6-7). The peak shear demand (based on a smeared model) was reduced by about 40 percent (Figure 6-9). The predominant frequency appears to be relatively insensitive. This may result from a visco-elastic model used to represent the dynamic rock behavior.

Similar comparisons of response spectra, acceleration time histories, and caisson shear and bending moment obtained for the second set of the analyses with different near-field soil softening were made. The results indicate that the responses are not sensitive to the properties of the soil because the resistance provided by soft soil is small. This behavior is similar to the results obtained by equivalent linear analyses without soil embedment. A comparison of the 5% damped response spectra of the motions computed at various caisson levels is shown on Figure 6-11.

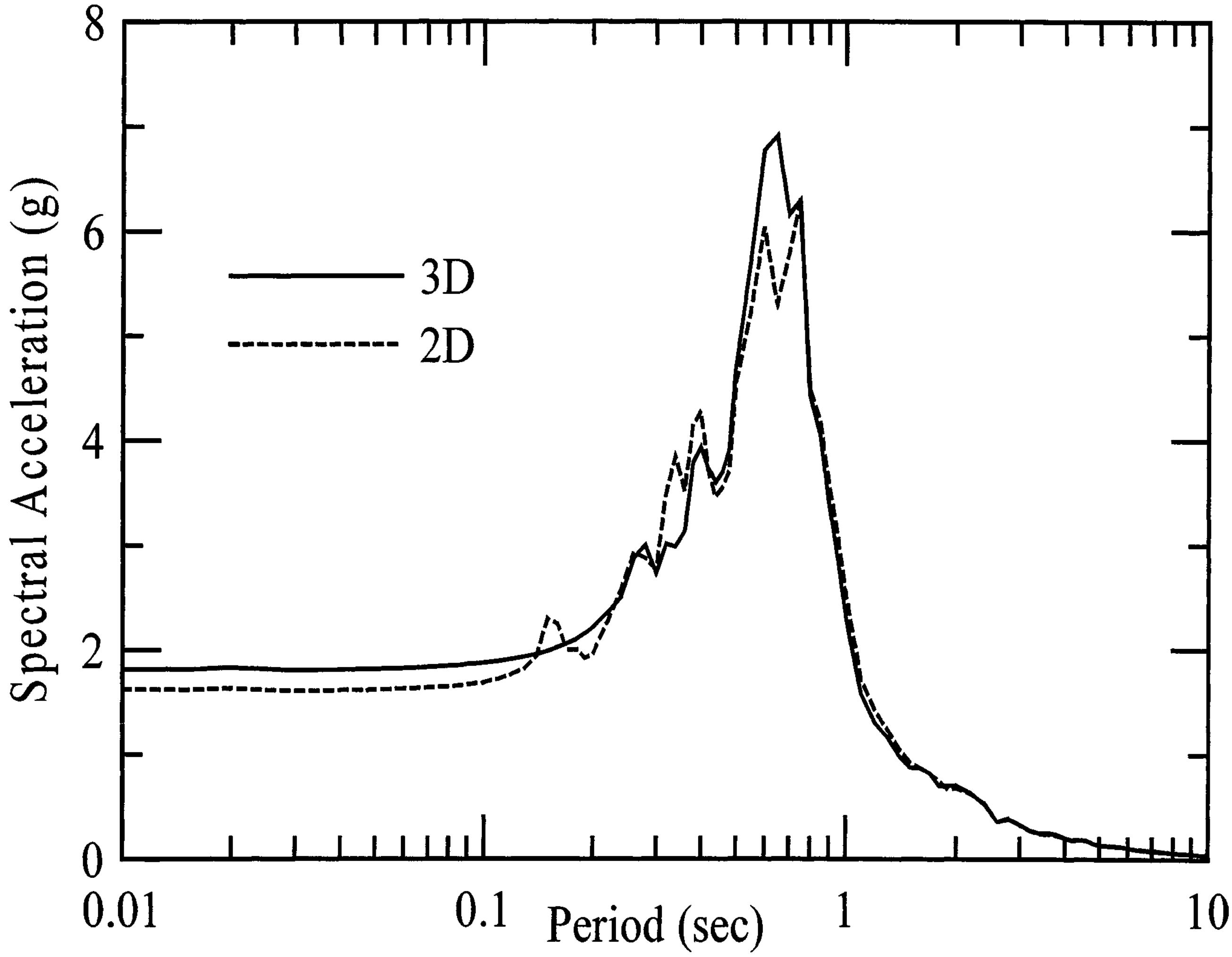


FIGURE 6-1 Comparison of Spectral Accelerations at Top of Caisson for 3D and 2D SASSI models

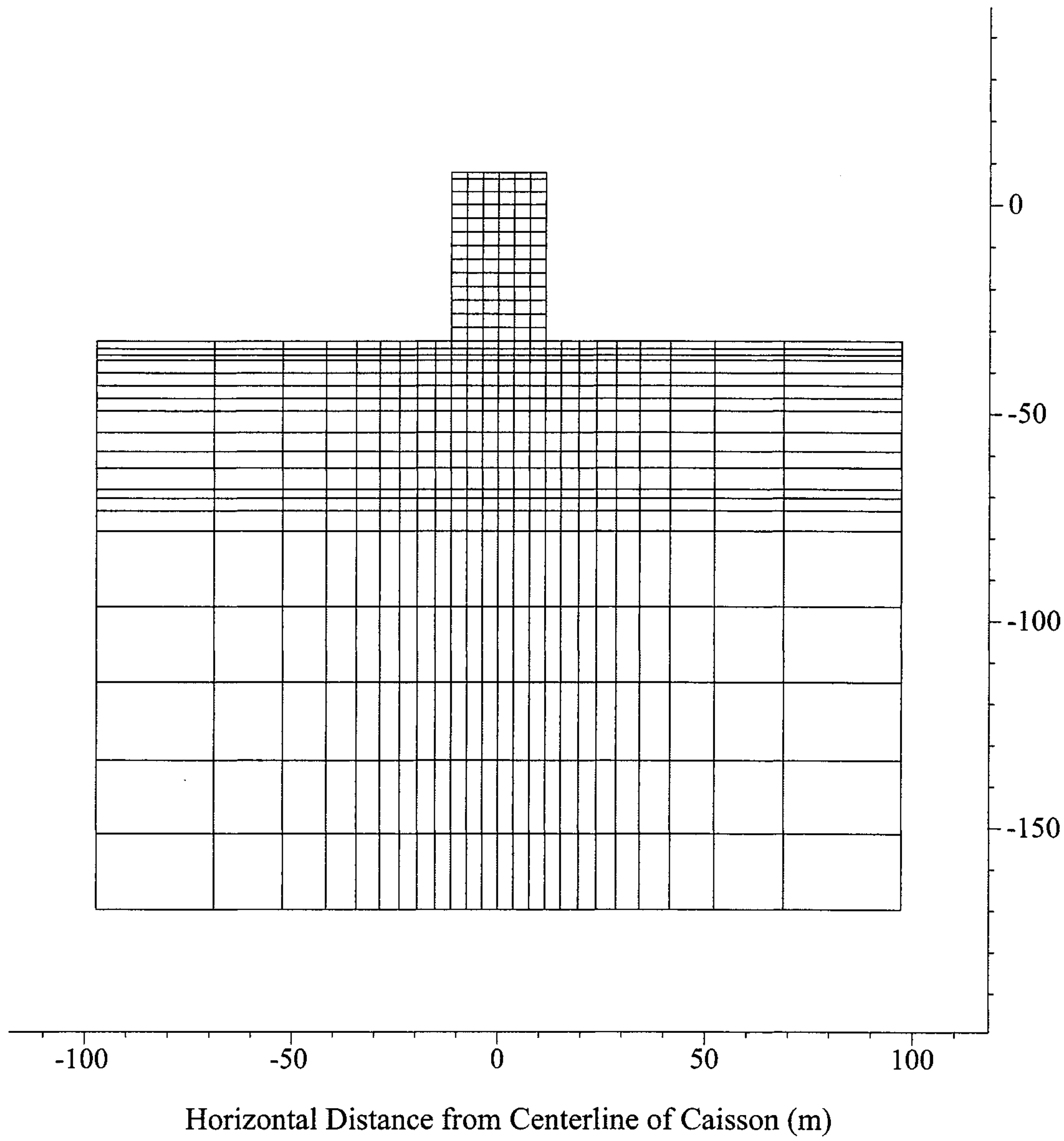


FIGURE 6-2 Finite Difference Grid of a 2-D Model

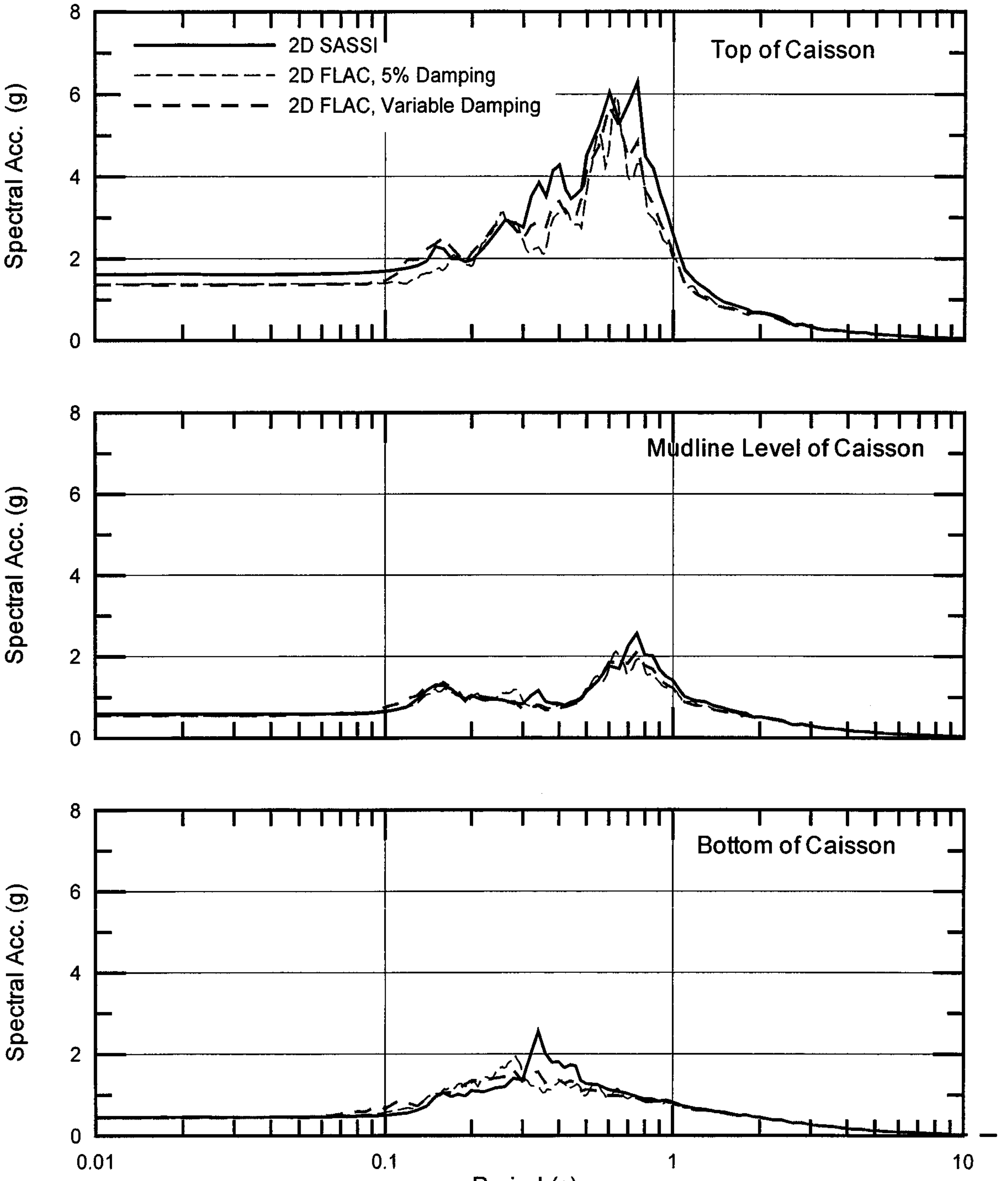


FIGURE 6-3 Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by 2-D SASSI and FLAC Models

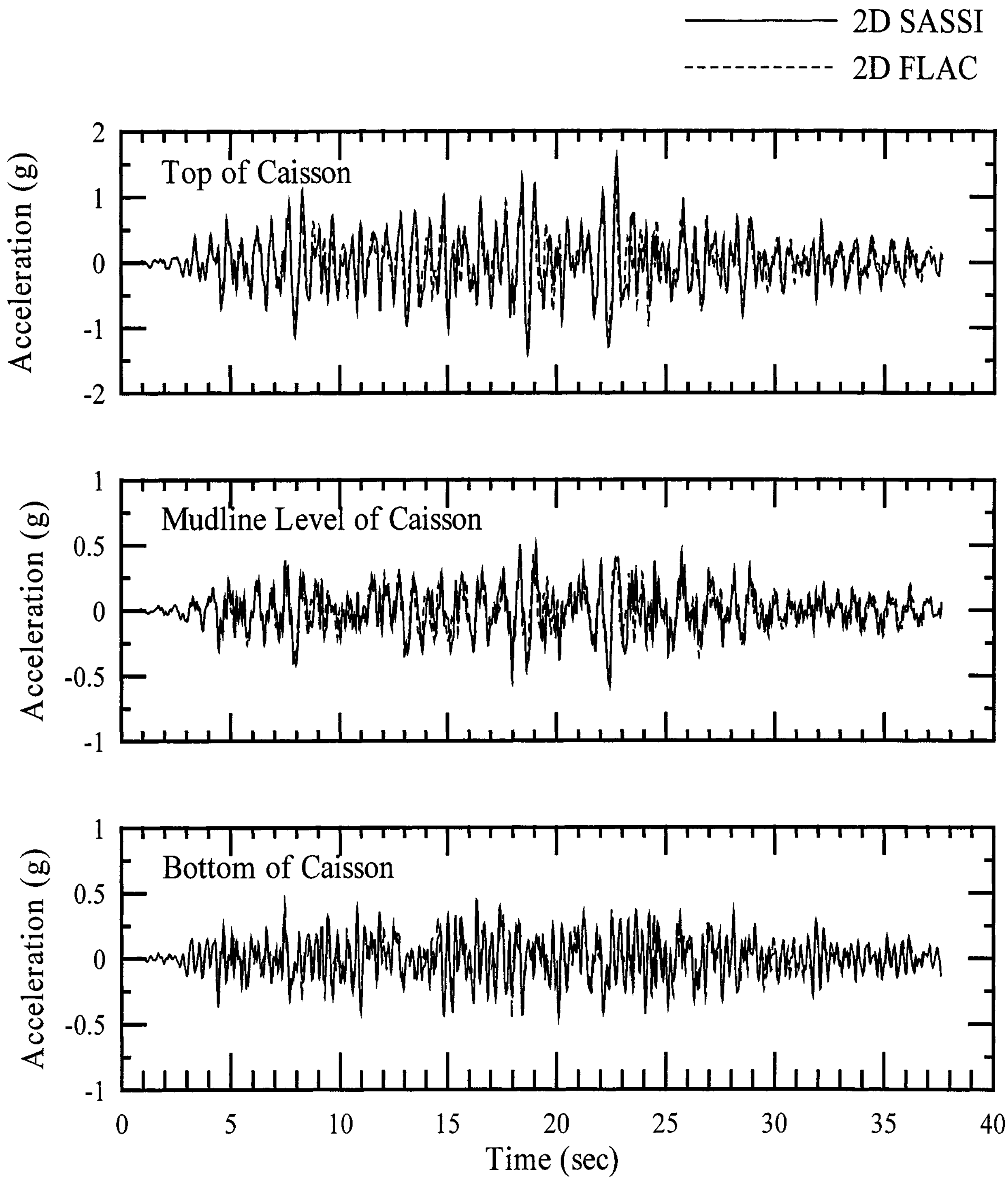


FIGURE 6-4 Comparison of Longitudinal Acceleration Time Histories from 2-D SASSI and FLAC Models

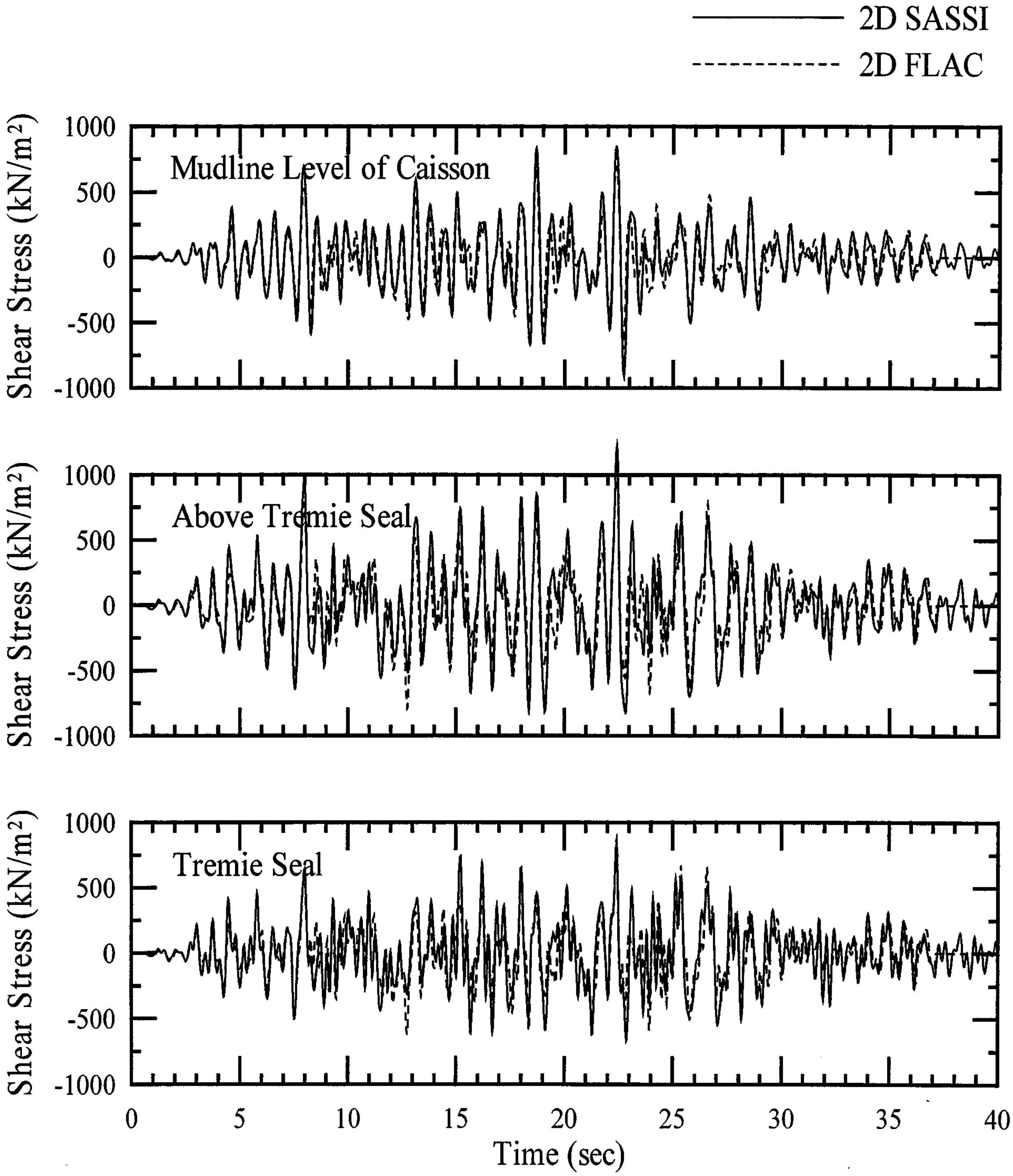


FIGURE 6-5 Comparison of Longitudinal Shear Stress Time Histories from 2-D SASSI and FLAC Models

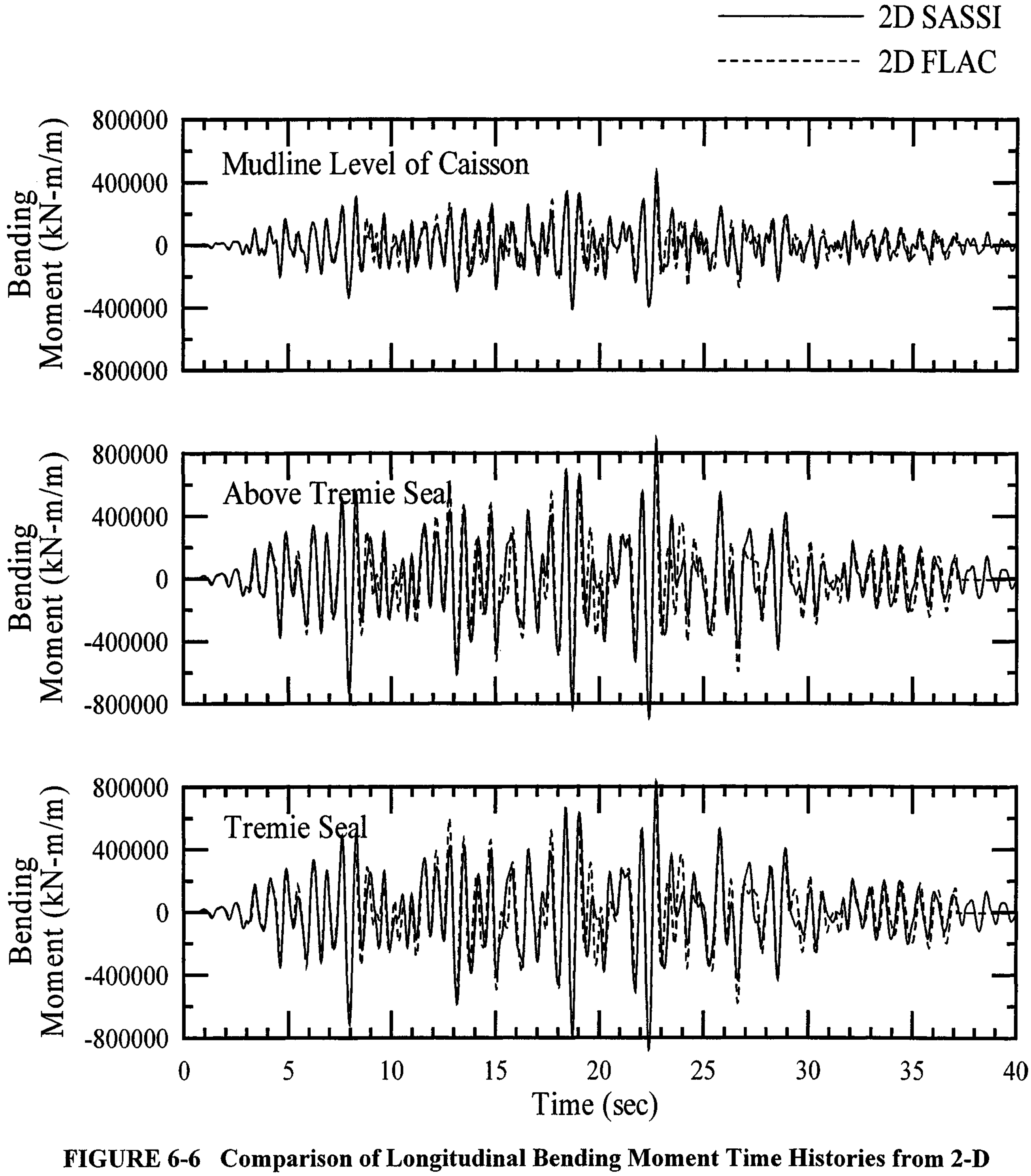


FIGURE 6-6 Comparison of Longitudinal Bending Moment Time Histories from 2-D SASSI and FLAC Models

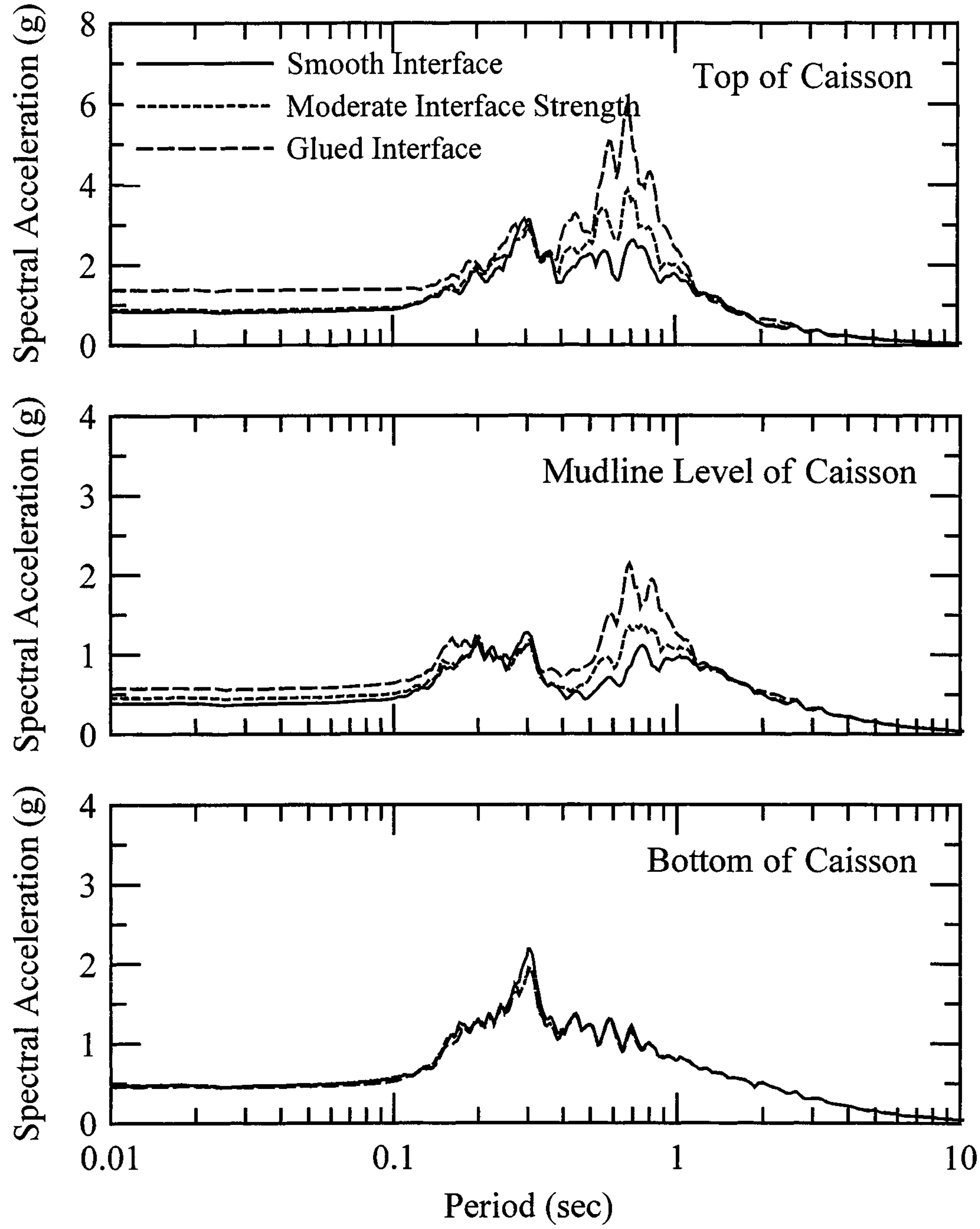


FIGURE 6-7 Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by FLAC Models with Various Interface Properties

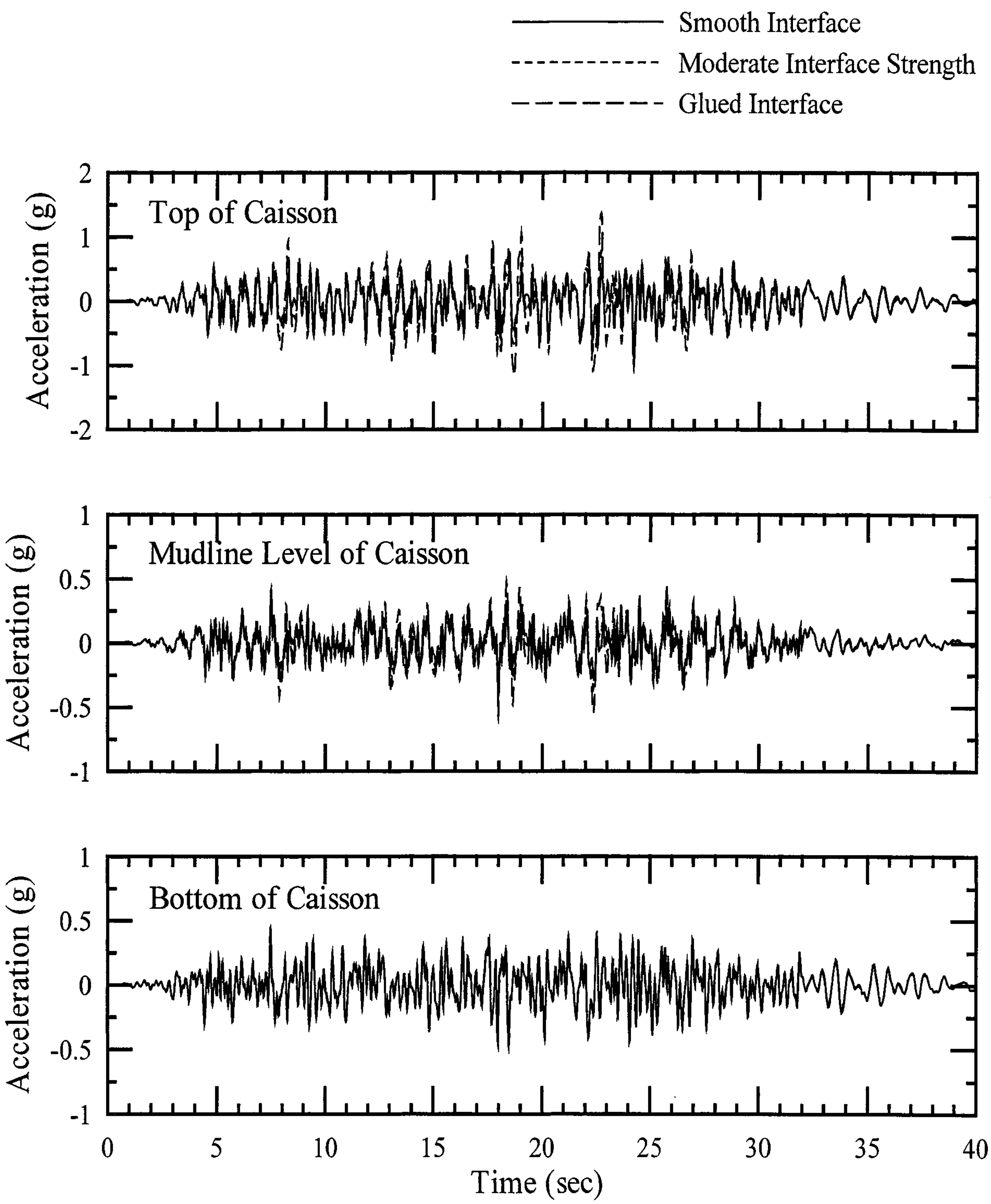


FIGURE 6-8 Comparison of Longitudinal Acceleration Time Histories from FLAC Models with Various Interface Properties

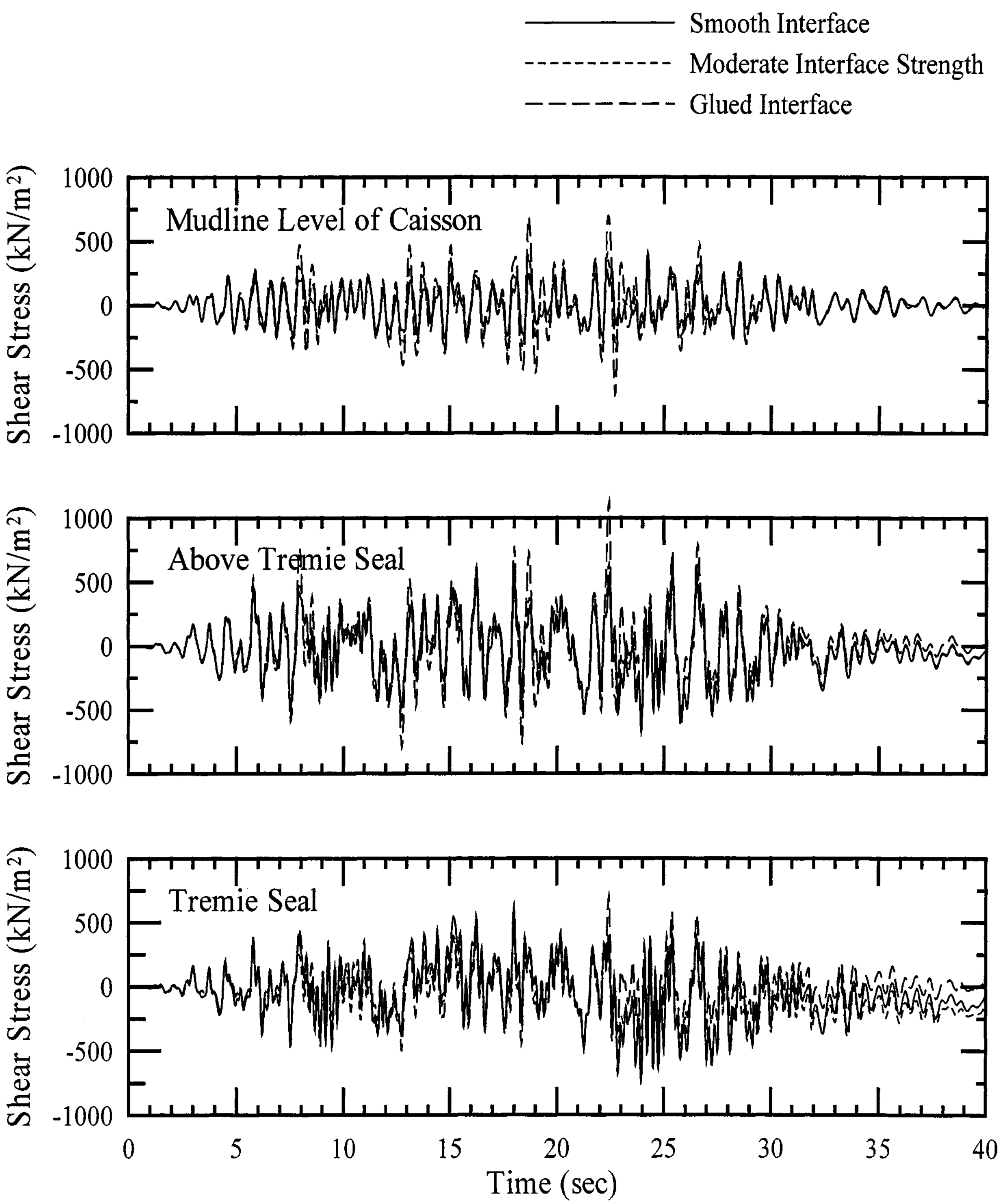


FIGURE 6-9 Comparison of Longitudinal Shear Stress Time Histories from FLAC Models with Various Interface Properties

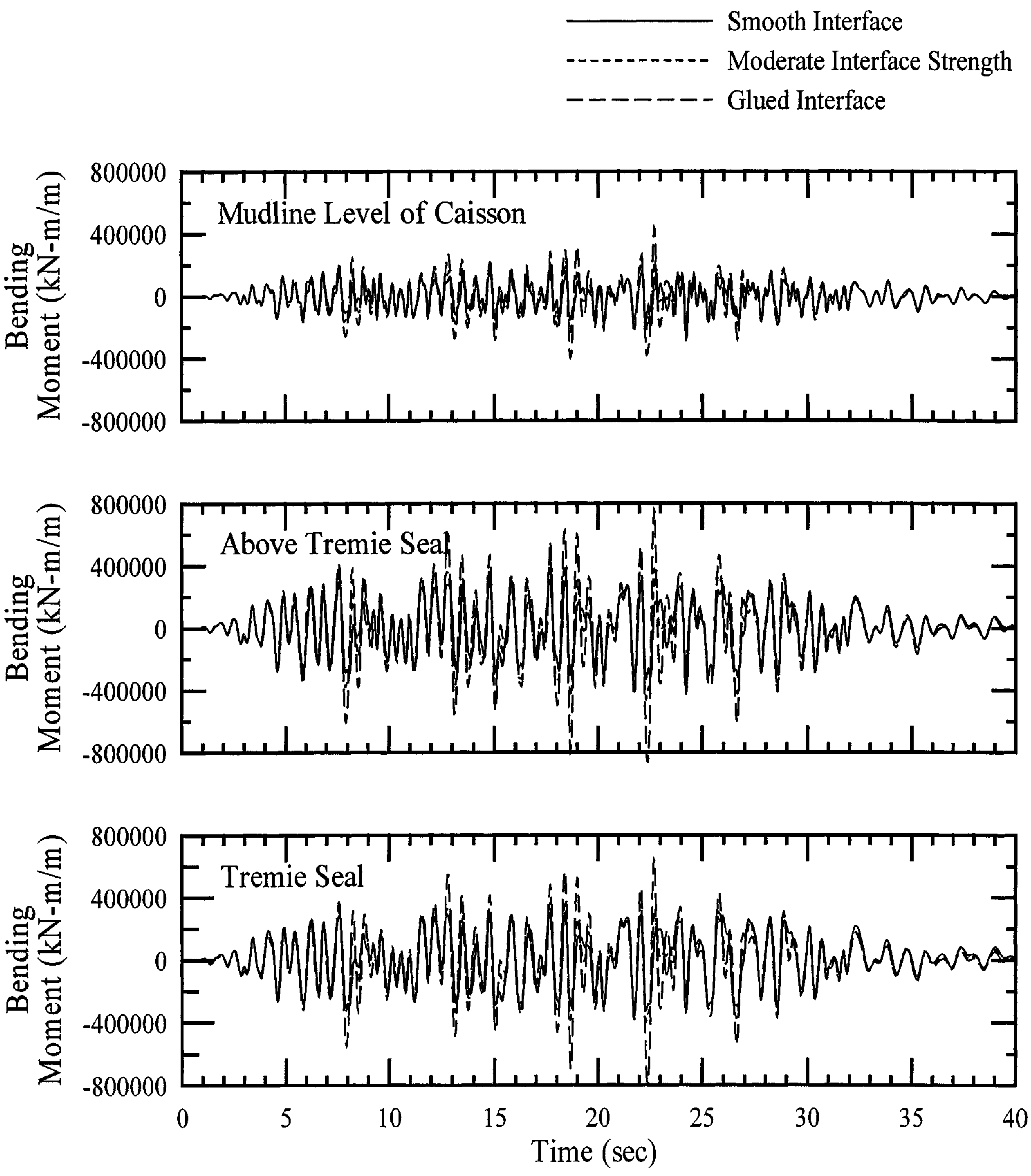


FIGURE 6-10 Comparison of Longitudinal Bending Moment Time Histories from FLAC Models with Various Interface Properties

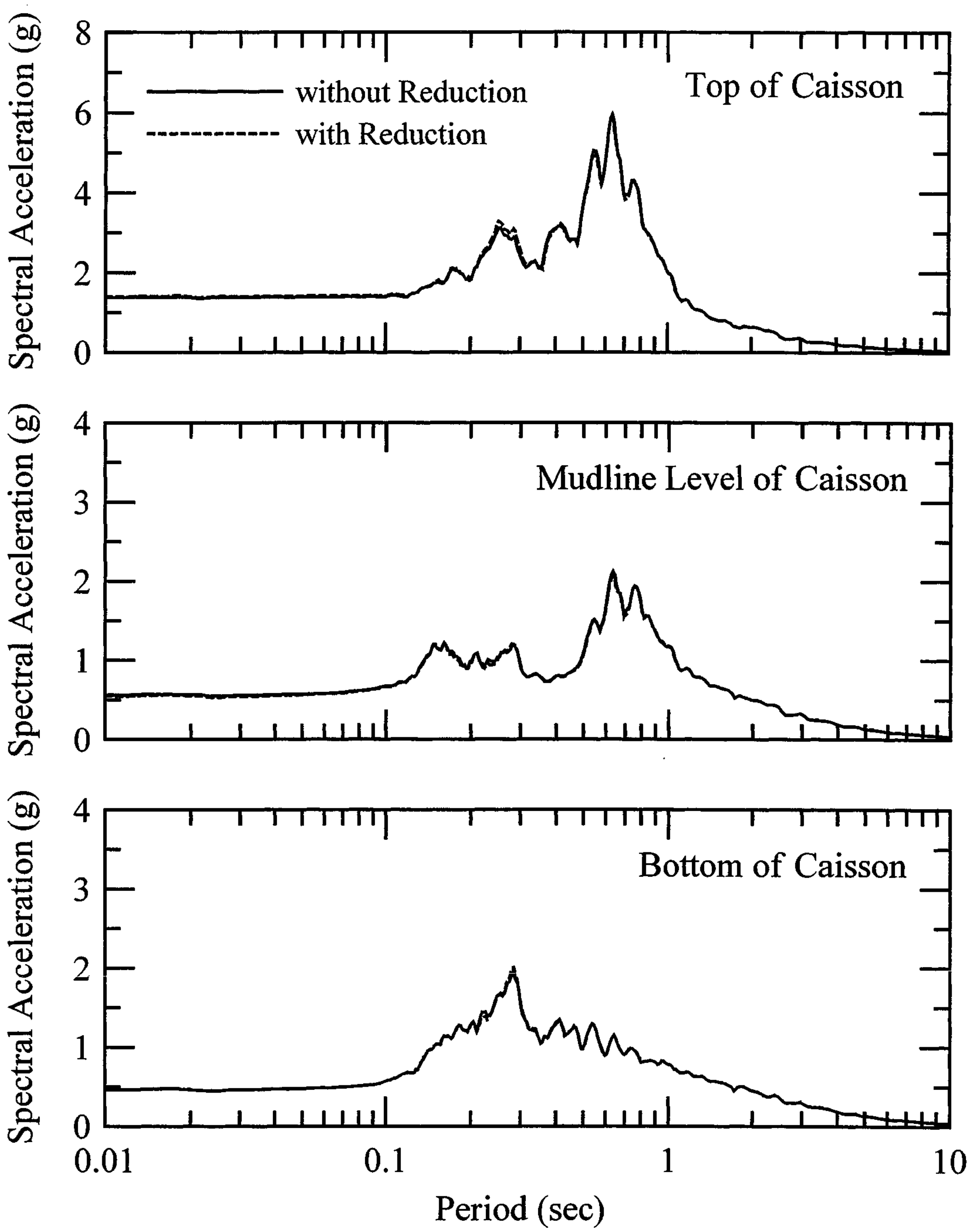


FIGURE 6-11 Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by FLAC Models with and without Soil Stiffness Reduction

SECTION 7
TWO-DIMENSIONAL ANALYSES USING SAP2000

7.1 Introduction

As described previously, linear and nonlinear two-dimensional time-domain dynamic analyses of the caisson and surrounding rock/soil were performed using the computer program SAP2000. These models were analyzed for forces and deformations in structural elements assuming various limiting boundary conditions. The linear model enforces full deformation compatibility between the caisson, rock, and soil. Thus it assumes that the rock/soil at the structure interface acts in both compression and tension under normal stresses, is non-yielding under shear stresses, and does not gap. This model provides upper-bound force demands on the caisson.

The nonlinear model provides for more realistic interaction between the caisson and the surrounding soil/underlying rock. The rock/soil acts in compression only, and shearing stresses are yielding and display hysteretic behavior during the excitation. This model provides "best estimate" force demands on the caisson.

The structural behavior and demands in the various caisson models can be compared with those found using other analysis tools based on alternative behavioral assumptions presented in Section 6, to gain insight into modeling and performance issues.

7.2 Modeling Approach

The modeling approach implemented in this part of the study is incremental in nature, and focuses on systematic refinement of force level demands by solving the linear system first, then adding various nonlinearities until reaching basic compatibility between assumptions and results. Phenomena to consider include inertial and kinematic interaction, nonlinear soil effects, normal gapping and shear yielding at the interface, plus rocking and sliding of the caisson base. This was done through detailed two-dimensional finite element modeling of the caisson and foundation system.

7.2.1 Computer Program

The structural analysis program SAP2000, by Computers and Structures, Inc. (1996) was used as the analysis tool for both linear and nonlinear time history demand calculations. It was selected because it is widely used, and because of its model generation and interpretation capabilities, and its capability of handling large complex problems. This program handles the nonlinearities, including soil gapping and shear yielding at the interfaces, as well as rocking/sliding at the caisson base. The caisson was modeled two dimensionally, focusing on the dynamic caisson response along the short axis (longitudinal direction).

Plane Element

The SAP2000 plane element was used to model the plane-strain behavior of the rock/soil surrounding the caisson. The plane element is a nine-node element of uniform thickness, based upon an isoparametric formulation (Hollings and Wilson, 1978). Stresses and strains were assumed not to vary in the thickness (out of plane) direction. An eight-point integration scheme was used. Stresses and strains in the element local coordinate system were evaluated at the integration points and extrapolated to the joints of the element. The plane element activate the two in-plane translation degrees of freedom at each of its connected joint, the rotational degree of freedom is not activated. The plane-strain formulation is appropriate in this application as demonstrated in Section 6, since the caisson structure is relatively thick compared to its planar dimension. In dynamic analysis, the mass of the plane element is lumped at the element joints.

Beam Element

The SAP2000 beam element used for this study is a general beam-column formulation which includes the effects of in-plane bending, shear, and axial deformation. This element was used to represent the concrete caisson from the bottom of tremie seal (elev -70.4 m [-231 ft]) to the top of caisson (elev +7.6 m [25 ft]). This modeling is different from those used in Section 6 in which 2-D plane elements were used to model the smeared section of the caisson. Since structural nonlinearity was not the focus of this study, a linear formulation was used. As shown on Figure 7-1, rigid, massless outrigger beam elements were used to stub out to the sides of the caisson for connection with the surrounding soil elements.

Radiation Dampers at Boundaries

Linear radiation dampers were derived (Wilson, 1998) for the left and right boundaries of the model. These energy absorbing dampers allow both the plane and shear wave to pass without reflection, and "radiate" away from the model. The SAP2000 damper element used is based upon the Maxwell model of viscoelasticity, having a linear damper in series with a spring. Since pure damping behavior is desired, the spring's effect was made negligible by making it sufficiently stiff, (i.e.: the characteristic time of the spring-dashpot system is two orders of magnitude smaller than the time step).

Gap Elements (Normal Stresses) at Caisson-Foundation Interface

SAP2000 has a simple compression only gap element, which allows only compressive stresses to transfer between the rock/soil and structure (i.e.: the rock/soil can "push", not "pull"). For the linear analyses, this element is modified to allow both tension and compressive stresses to be transmitted. For the nonlinear model, it is important to "pre-load" the gap elements at the caisson base to account for dead load - buoyancy compressive stresses at the interface prior to seismic excitation.

Plastic Elements (Shear Stresses) at Caisson-Foundation Interface

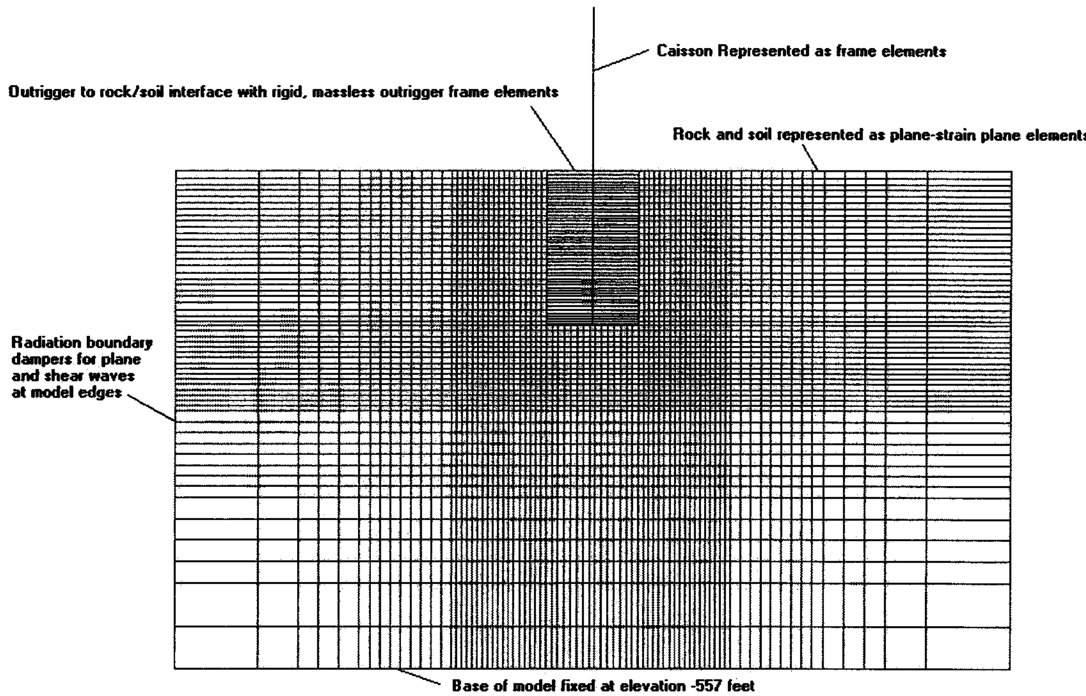
SAP2000 has a simple plasticity element, which was used to model shear yield at the caisson-foundation interface. Hysteretic damping is included during seismic response. Note that for the linear analyses, this element is modified to be linear, elastic (no yielding or hysteretic behavior).

Finite Element Models

Three detailed finite element models were developed and analyzed:

1. Case 1: With tensile/compressive normal stresses transmitted at the foundation/soil interface, non-yielding shearing stresses at interface, full soil mass (upper-bound demands).
2. Case 1nm: With tensile/compressive normal stresses transmitted at the foundation/soil interface, non-yielding shearing stresses at interface, <u>no</u> soil mass. This model was used to determine what effect soil mass has on the caisson.

3. Case 2: With compressive only normal stresses transmitted at the foundation/soil interface (the rock/soil can only "push", not "pull"), yielding shearing stresses at interface, and full soil mass. This model gives the "best estimate" demands.



Global axes are:

- x: longitudinal
- z: vertical

FIGURE 7-1 Detailed Finite Element Model Used in SAP2000 Analyses

Modeling Issues

The model (Figure 7-1) consists of plane-strain finite elements of 38.7 m (127 ft) thickness with the associated soil and rock properties at the site. The cellular reinforced concrete caisson is modeled with standard frame elements of appropriate area and inertia. The hydrodynamic mass, both external and internal to the caisson is included. Radiation boundary conditions at the edges of the soil/rock media are approximated, which allows both plane and shear waves to pass without reflection, and "radiate" away from the model.

Seismic Excitation

The seismic input is the longitudinal component of the rock motions with a peak acceleration of 0.6 g at rock outcrop as described in Section 3. The input motion used is the interface motion obtained from the free-field site response analysis in Section 3. The motion was input with a time step of 0.02 seconds.

Time History Analyses

The method of nonlinear time history analysis used by SAP2000 is extremely efficient and is designed to be used for structural systems which are primarily linear elastic, but which have a limited number of pre-defined nonlinear elements. In SAP2000, Ritz vectors (Wilson et al., 1982) modal superposition is used during time history analysis. The nonlinear modal equations are solved iteratively, and convergence is satisfied for both force and energy tolerance at each time step.

For the detailed finite element models, it was found that 80 structural Ritz vectors would mobilize over 90% of the system mass. Also, one Ritz vector was defined for each nonlinear element.

Material Properties

Material properties used in SAP2000 analyses are described in Appendix A.

7.3 Analysis Results

The fundamental vibrational characteristics, and findings from both the detailed and simplified computer models are included below.

7.3.1 Fundamental Vibrational Characteristics

From a linearized dead load state, the fundamental vibrational characteristics shown on Figures 7-2 to 7-5 were found. Note that in Case 1, these vibrational characteristics remain constant throughout the time histories. However, in Case 2 the nonlinear sliding/rocking at the caisson base, as well as the gapping along the sides tend to lengthening their fundamental period during strong ground shaking.

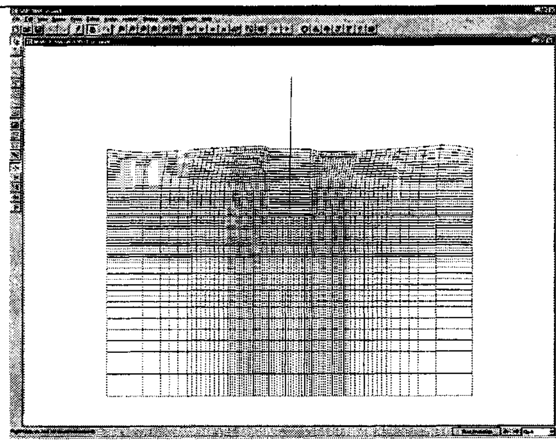


FIGURE 7-2 1st Anti-symmetric (T=0.95 secs)

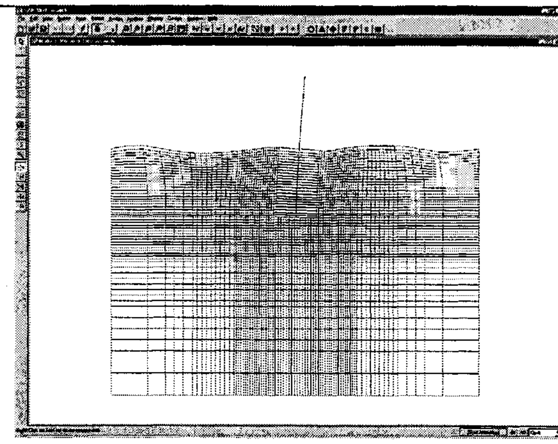


FIGURE 7-4 2nd Anti-symmetric (T=0.7 secs)

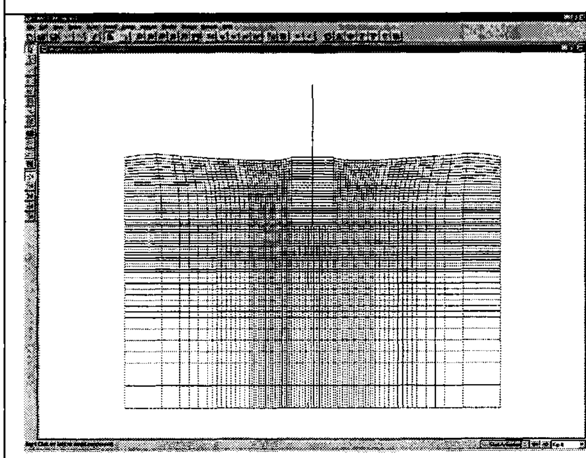


FIGURE 7-3 1st Symmetric (T=0.9 secs)

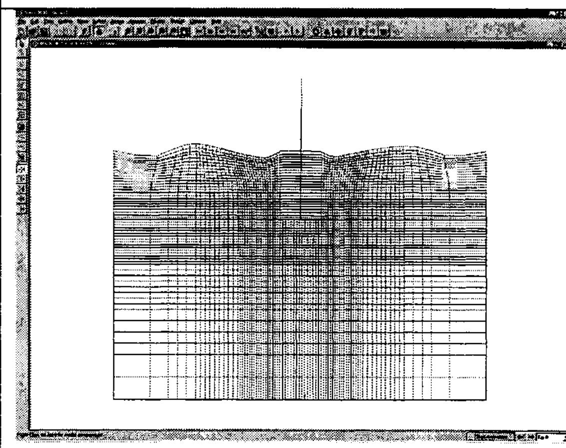


FIGURE 7-5 2nd Symmetric (T=0.6 secs)

7.3.2 Case 1: Linear, Elastic, Full Soil Mass

Figures 7-6 and 7-7 show time histories of shear and moment computed at the mudline, the top of tremie and the bottom of the caisson.

7.3.3 Case 1nm: Linear, Elastic, No Soil Mass

Figures 7-8 and 7-9 show time histories of shear and moment computed at the mudline, the top of tremie and the bottom of the caisson.

7.3.4 Case 2: Nonlinear at Interface and Base, Full Soil Mass

Figures 7-10 and 7-11 show time histories of shear and moment computed at the mudline, the top of tremie and the bottom of the caisson.

It should be noted that the shear stresses computed for the caisson represent the net shear not the gross shear based on a smeared model presented in Section 6. Thus, the shear presented in this section will be higher than those in Section 6.

7.3.5 Comparison of Cases

A comparison is made among the three different cases by looking at the envelope of force demands over the height of the caisson. Comparisons of the peak shear stress and moment distributions with depth in the caisson for the three cases analyzed are shown on Figures 7-12 and 7-13, respectively. The results indicate that the inclusion of nonlinear soil-structure effects significantly lowers caisson force demands. The rocking response of the caisson effectively lengthens the fundamental period of the soil-structural system, producing lower force demands than that of a glued base assumption. In the linearized analyses (Cases 1 and 1nm) the soil mass contributes little to the overall caisson response. That is, the caisson's own inertia tends to dominate the system dynamic response.

Envelopes of peak shear forces and moments computed from the SAP2000 and FLAC analyses are compared on Figure 7-14 and 7-15, respectively. As shown on Figures 7-14 and 7-15, values computed from the SAP2000 analyses are somewhat lower than those computed from the FLAC analyses. It is believed that the differences are due to the different ways of modeling the caisson. The SAP2000 model used a beam with rigid outrigger beam elements to model the caisson whereas the FLAC model used solid elements to model the caisson. Use of the solid elements to model the caisson is believed to be more appropriate because of a large depth-to-height ratio of the caisson.

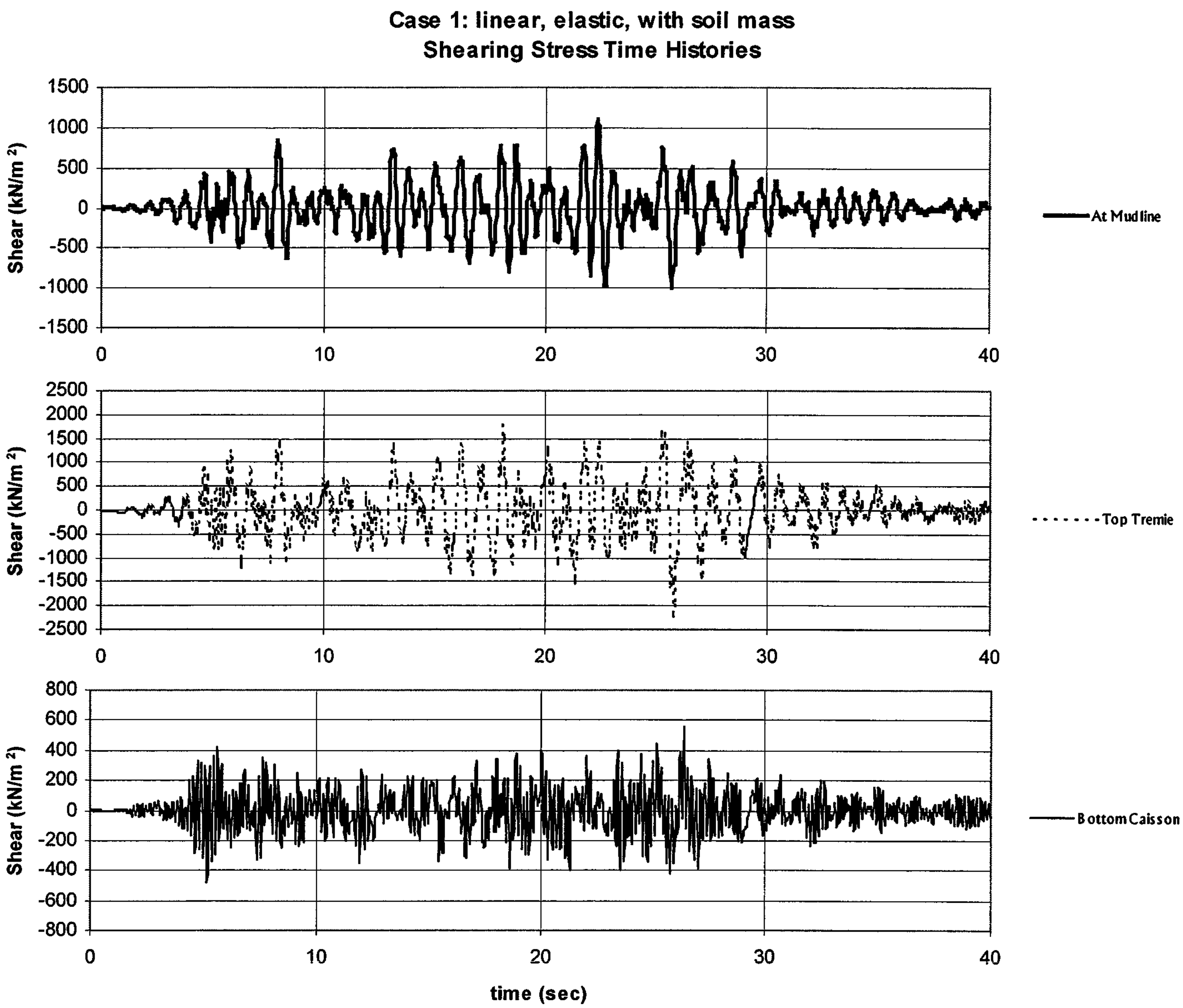


FIGURE 7-6 Longitudinal Shear Time Histories - Case 1

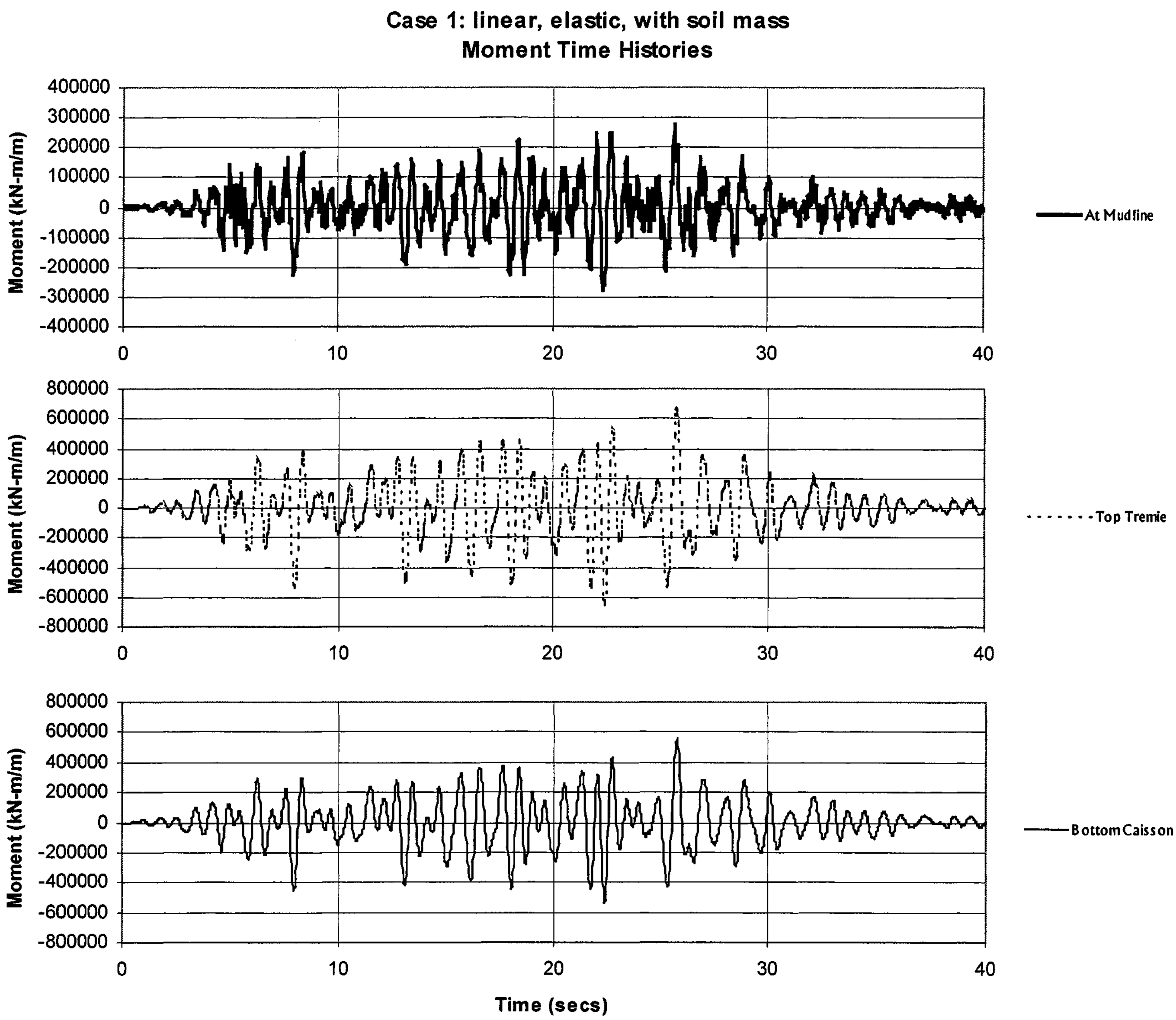


FIGURE 7-7 Transverse Axis Moment Time Histories - Case 1

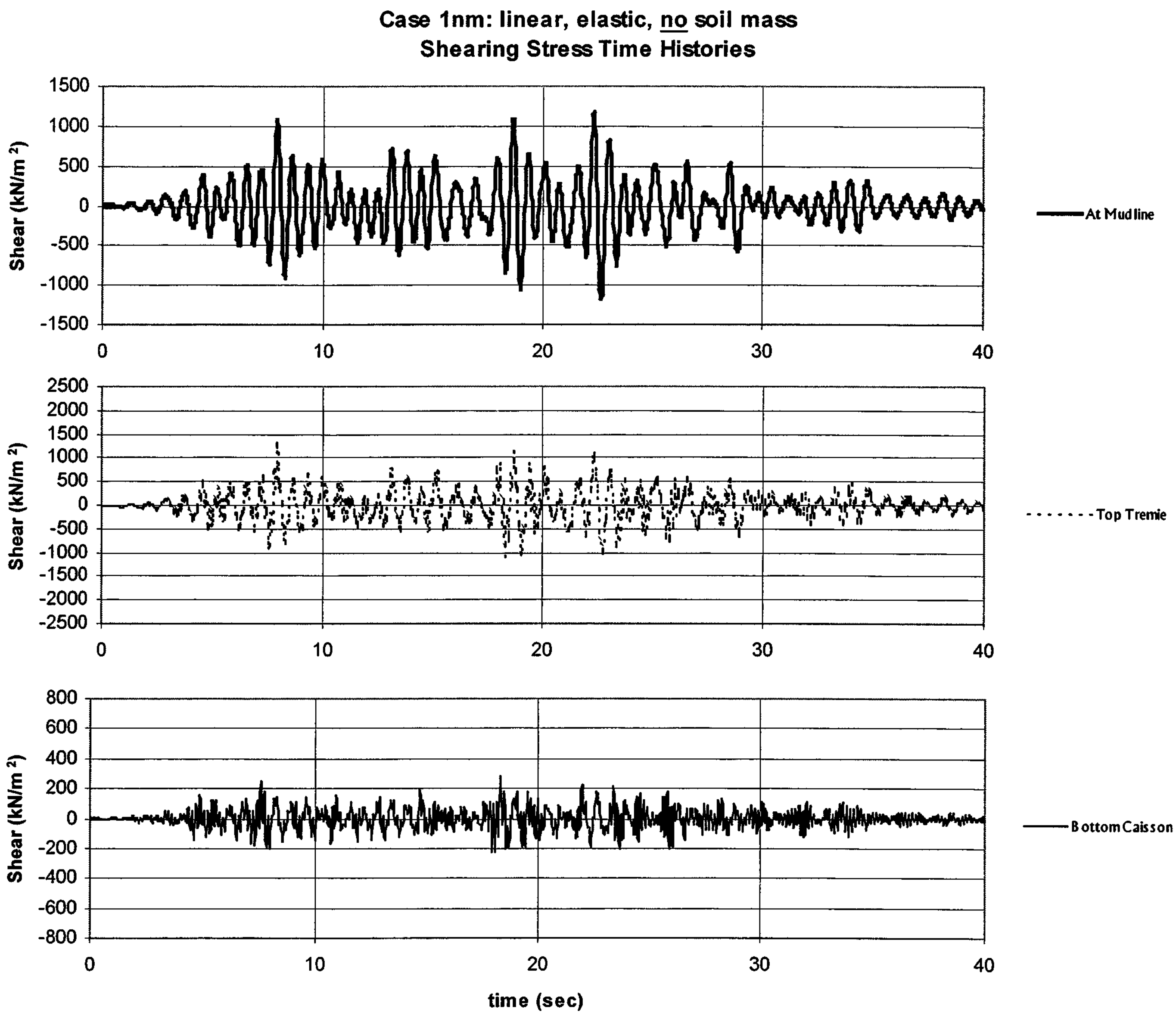


FIGURE 7-8 Longitudinal Shear Time Histories - Case 1nm

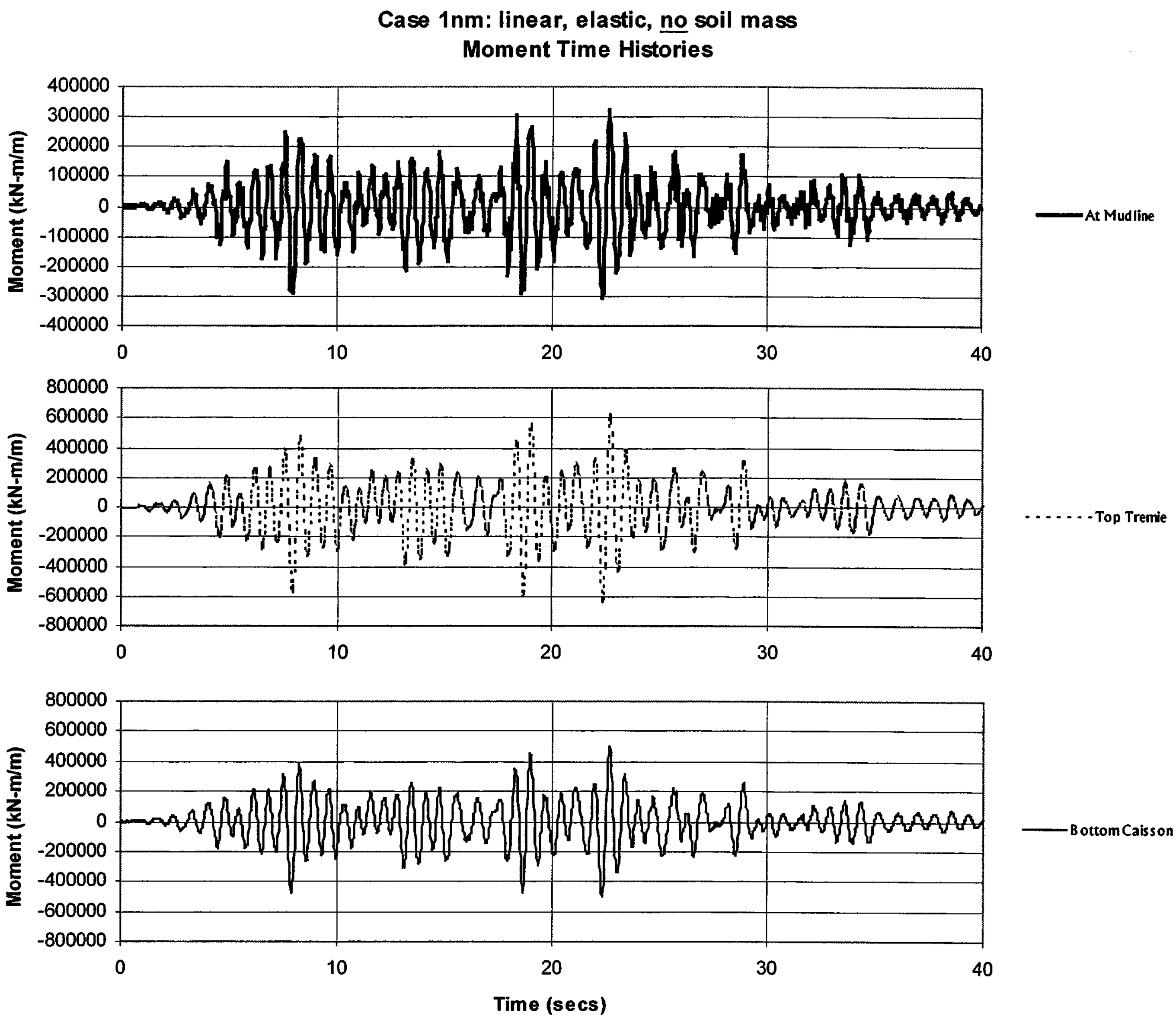


FIGURE 7-9 Transverse Axis Moment Time Histories - Case 1nm

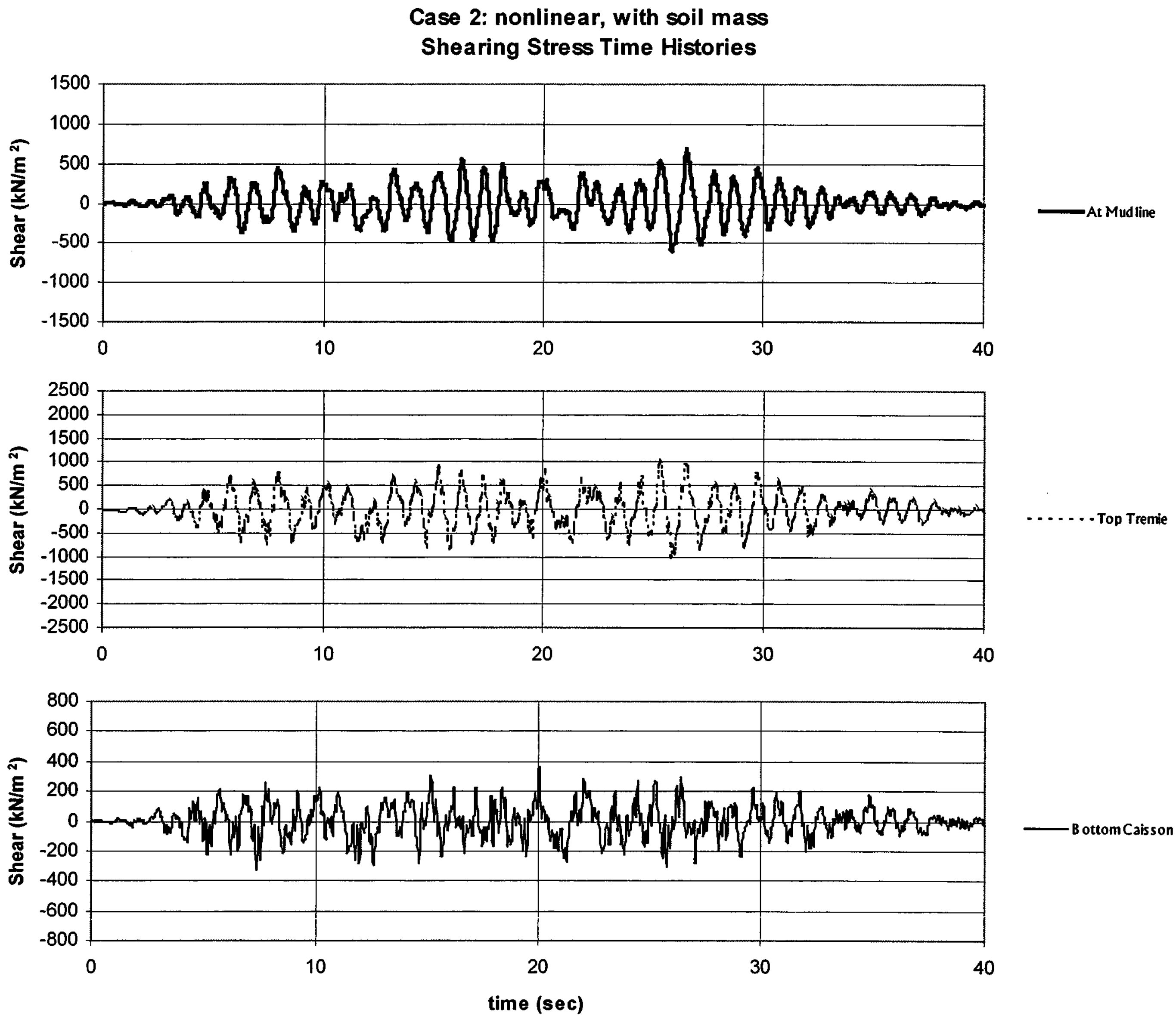


FIGURE 7-10 Longitudinal Shear Time Histories -Case 2

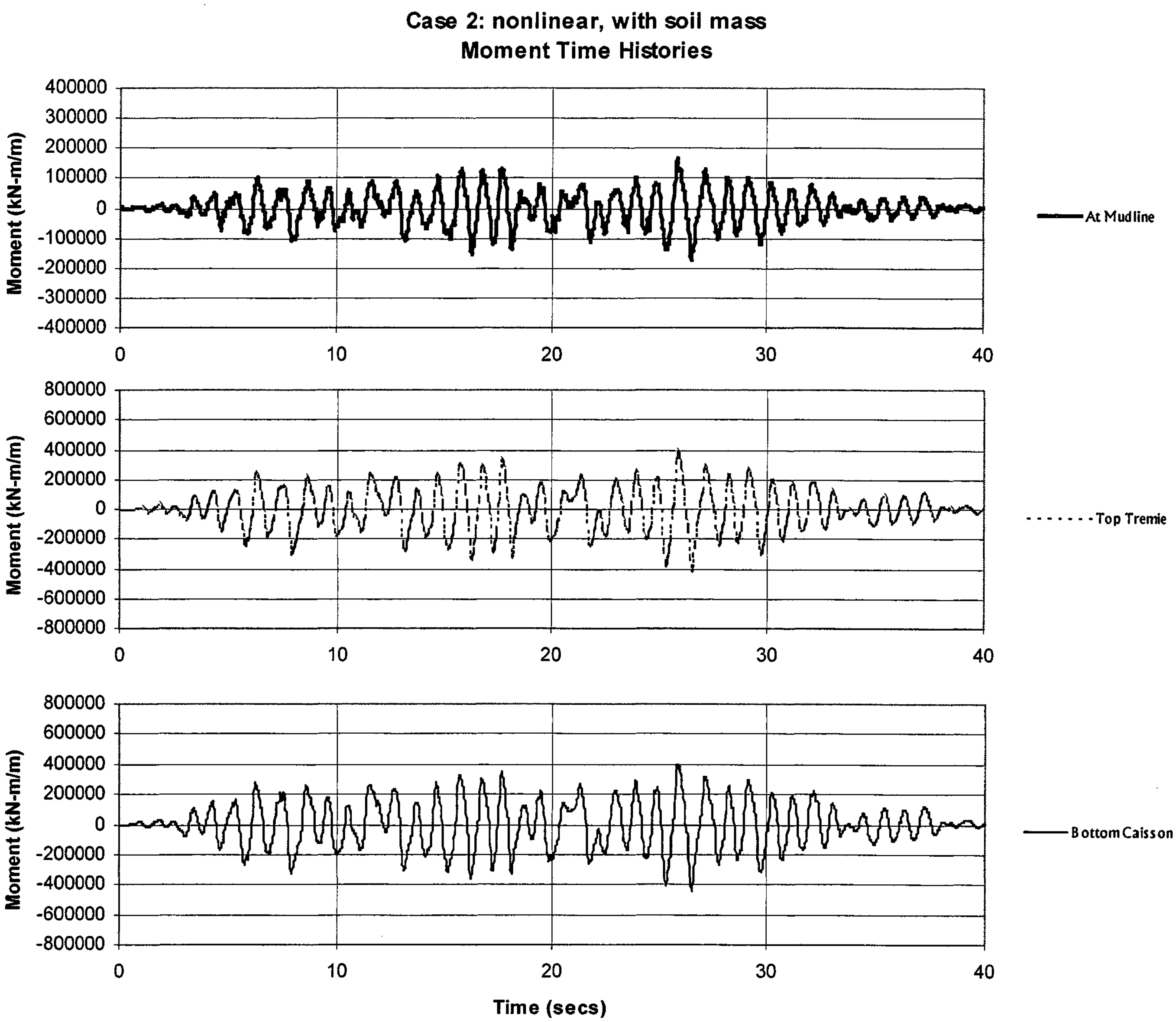


FIGURE 7-11 Transverse Axis Moment Time Histories - Case 2

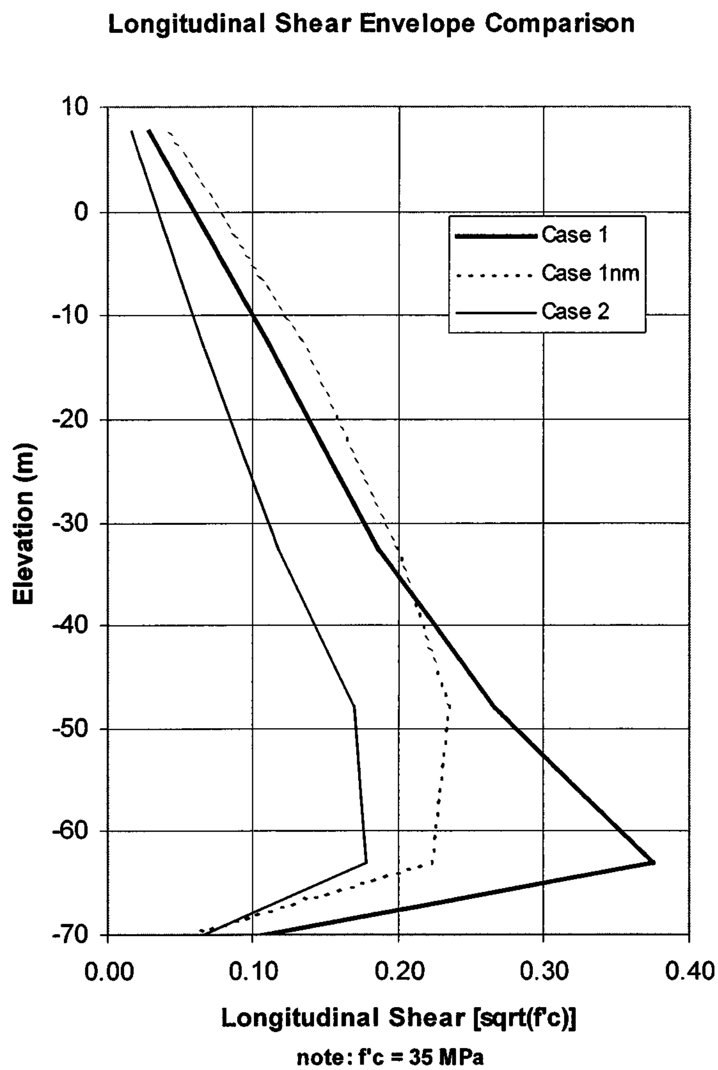


FIGURE 7-12 Longitudinal Shear Envelopes

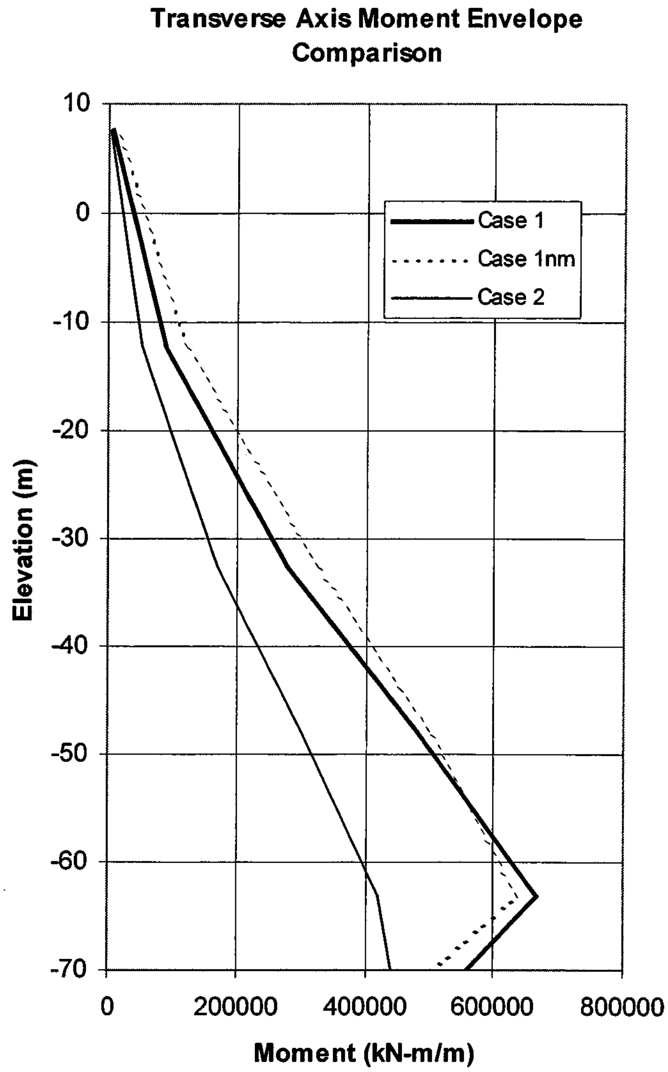


FIGURE 7-13 Transverse Axis Moment Envelopes

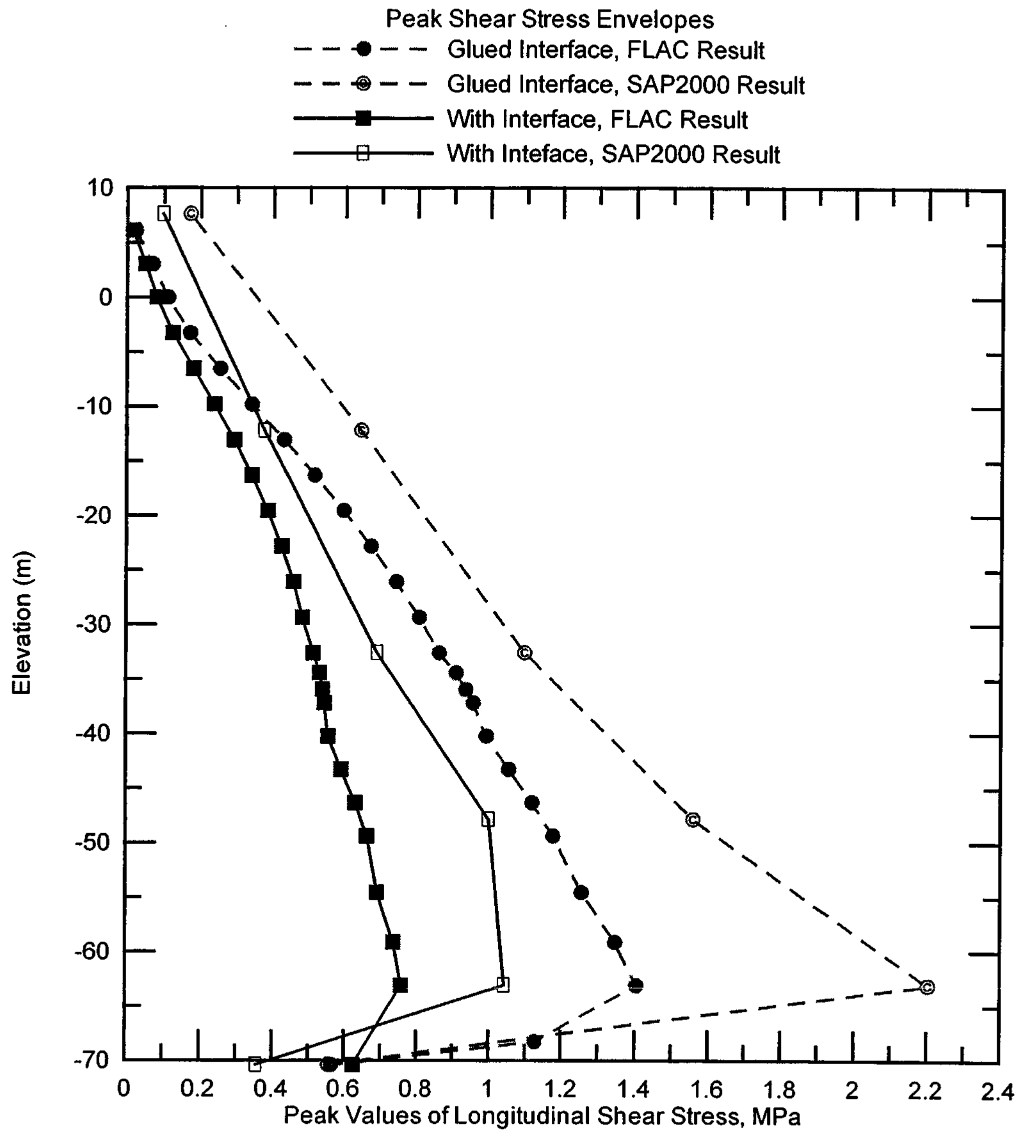


FIGURE 7-14 Comparison of Longitudinal Shear Stress Envelopes from SAP2000 and FLAC Analyses

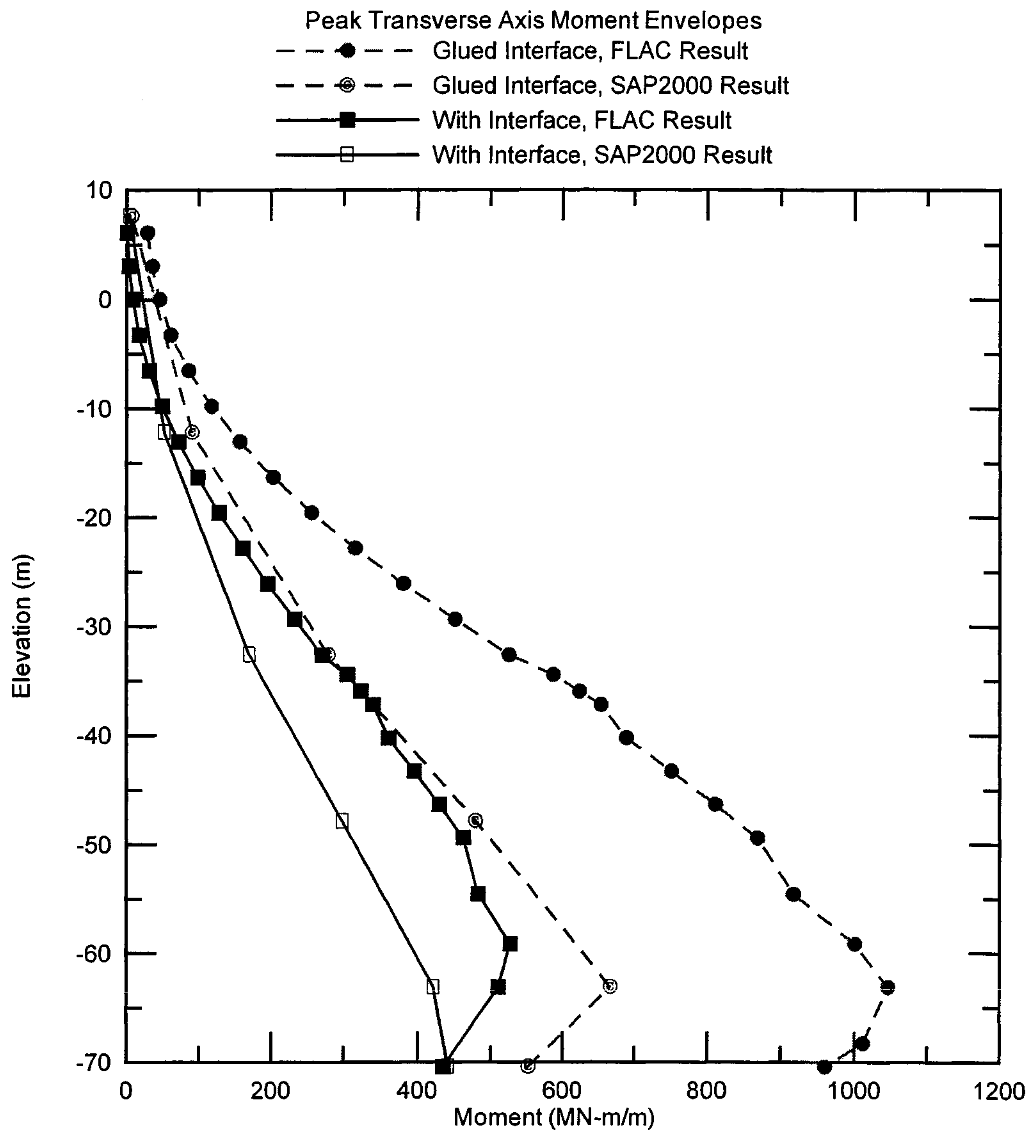


FIGURE 7-15 Comparison of Transverse Axis Moment Envelopes from SAP2000 and FLAC Analyses

SECTION 8
CONCLUSIONS

Seismic response of the large caisson at Pier W3 of the west spans of the San Francisco-Oakland Bay Bridge was analyzed in this study. The results of this study indicate that the effects of the superstructure (tower) on the response of the caisson were insignificant. Soil embedment has significant effects on the imaginary part of the impedance functions computed at the top of the caisson. However, the real part of the impedance functions only decreases slightly when the soil embedment is ignored. The scattered motions at the top of the caisson are very similar for the cases with and without soil embedment.

The lateral earth pressure, base bearing pressure, and soil stresses computed by the equivalent linear analyses indicate the possibility of soil-foundation separation (gapping and uplift). The results of non-linear analyses indicate that motions and stresses developed in the caisson are sensitive to the soil-caisson and rock-caisson interface properties. The peak responses are lower for softer interface strength. The peak shear demand in the caisson may be decreased by as much as 40 percent if no shear resistance is present between the caisson and soil, and the uplifting is allowed at the base of the caisson. However, the frequency characteristics of the caisson response are less affected by the gapping, sliding, and uplifting. Both the linear and nonlinear analyses performed using the computer programs FLAC and SAP2000 provide similar conclusions. However, the results are somewhat different. Envelopes of peak shear forces and moments computed using SAP2000 are somewhat lower than those computed using FLAC. This may be due to the modeling of the caisson by a beam with rigid outrigger beam elements in SAP2000 as compared with the solid elements used in the FLAC analyses. It is believed that modeling the caisson by solid elements is more appropriate than the beam with rigid outrigger beam elements.

Effects of soil embedment on the caisson response may depend on the relative stiffness between the overburden and the rock or rock-like material underlying the caisson. More sensitivity analyses of caissons founded in different materials are needed to address the effects of soil embedment on the caisson response. It appears that the equivalent linear analyses neglecting the gapping, sliding and uplifting will provide conservative estimates of the caisson response and demand. To reduce degree of conservatism, nonlinear analyses incorporating interface elements are recommended if gapping, sliding, and uplifting are likely to occur.

SECTION 9

REFERENCES

Abrahamson, N. (1996), Personal Communication.

Computers and Structures, Inc., 1996, SAP2000 Analysis Reference, Volumes I and II, version 6.0.

Geomatrix Consultants, Inc. (1992), "Seismic Ground Motion Study for West San Francisco Bay Bridge", report submitted to California Department of Transportation.

Goyal, A. and Chopra, A.K. (1988), "Simplified Evaluation of Added Hydrodynamic Mass for Intake Towers", Journal of Engineering Mechanics, Vol. 115, No. 7.

Hollings, J.P. and Wilson, E.L., 1978, 3-9 node isoparametric planar or axisymmetric finite element, Report No. UC SESM 78-3, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley.

Itasca Consulting Group, Inc. (1993). "FLAC, fast Lagrangian analysis of continua, Version 3.2 and Version 4.0."

Lysmer, J., Ostadan, F., Tabatabaie, M., Vahdani, S., and Tajirian, F. (1988). "SASSI - A system for analysis of soil-structure interaction, user's manual", Geotechnical Engineering Division, Civil Engineering Department, University of California, Berkeley, and Bechtel Power Corporation.

OPAC Consulting Engineers, 1994, Seismic analysis of the existing West Bay Spans of the San Francisco-Oakland Bay Bridge, Report to Caltrans Division of Structures, Contract No. 59U064.

Seed, H.B. and Idriss, I.M., (1970). "Soil Moduli and Damping Factors for Dynamic Response Analyses", Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley, California.

Vucetic, M and Dobry, R. (1991). "Effect of Soil Plasticity on Cyclic Response", Journal of Geotechnical Engineering, ASCE, Vol. 117, No.1, pp. 89-107.

Wilson, E.L., 1998, Three Dimensional Static and Dynamic Analysis of Structures, printed by Computers and Structures, Inc., 2nd edition.

Wilson, E.L., Yuan, W., and Dickens, J.M., 1982, Dynamic analysis by direct superposition of Ritz vectors, Earthquake Engineering and Structural Dynamics, Volume 10, pp. 813-823.

APPENDIX A

MATERIAL PROPERTIES USED IN SAP2000 ANALYSES

A.1 Rock/Soil Properties

The estimated in-situ rock/soil properties are shown in Table A-1.

Table A-1 In-Situ Rock/Soil Properties

	Elevation (m)				
Layer	From	To	Weight Density (N/m^3)	E (MP_a)	Poisson's Ratio - μ
Half-space	-169.82	-77.74	0.02358	269390	.427
14	-77.74	-73.48	0.02358	203319	.434
13	-73.48	-70.43	0.02358	120215	.433
12	-70.43	-68.29	0.02279	80509	.429
11	-68.29	-63.11	0.01886	16581	.479
10	-63.11	-59.15	0.01886	2419	.481
9	-59.15	-53.96	0.01886	1838	.481
8	-53.96	49.39	0.01886	1345	.481
7	49.39	-46.34	0.01761	649	.481
6	-46.34	-43.29	0.01651	418	.481
5	-43.29	-40.24	0.01651	422	.481
4	-40.24	-37.2	0.01651	455	.481
3	-37.2	-35.98	0.01493	87	.481
2	-35.98	-34.45	0.01493	155	.481
1	-34.45	-32.62	0.01493	290	.481

Shear strength of the rock/soil is $s = c + \sigma_n \tan(\phi)$, where s = the shear strength, c = cohesion, σ_n = normal stress, and ϕ = friction angle. Unfortunately, SAP2000 doesn't have the capability for shearing yield to be a function of normal stress, thus we used the "average" compressive normal stress from the linear, elastic (full soil mass) as σ_n

throughout the nonlinear analysis. For the nonlinear analysis, $c = 0.4788$ kN/m^2 (0.01 ksf), and $\phi = 25°$.

The normal stress rock/soil stiffness was assumed linear (compression only) at 14,100 MN/m^2/m (90000 ksf/ft) of caisson. When in tension the normal stress rock/soil stiffness is zero.

A.2 Concrete Properties

A concrete breaking strength f'_c = 34,500 kN/m^2 (5000 psi) was assumed, and elastic modulus = 27,800 MN/m^2 (580,400 ksf) was used. The weight density of concrete was assumed to be 23.6 kN/m^3 (150 pcf).

No nonlinear structural analysis was performed on the caisson components (i.e.: the caisson is assumed to remain linear, elastic).

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