International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

30 Mar 2001, 4:30 pm - 6:30 pm

Dynamic Soil-Foundation-Structure Interaction Analyses of Large Caissons

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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 paper presents the results of 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 ground motion 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 and 2-D analyses conducted in the direction of the short axis (longitudinal) were performed using a modified version of computer program SASSI. The results of these 3-D and 2-D analyses are similar. Non-linear analyses were performed for 2-D models using computer program FLAC. 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.

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.

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 paper 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 using different analysis tools. 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 (Chang et al., 1998). 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). 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. Comparisons of the results from non-linear and equivalent linear SFSI analyses are presented in this paper.

SUBSURFACE CONDITIONS

Figure 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 Fig. 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).

DESIGN GROUND MOTION

The design rock motions were developed based on the ground motion study performed by Geomatrix (1992) and later modified by Abrahamson (1996). The design earthquake corresponds to the maximum credible earthquake on the San Andrea fault (M_w 8) located at about 15 km from the site. The time history (Fig. 2) 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.

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 Fig. 3. 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.

THREE-DIMENSIONAL EQUIVALENT LINEAR ANALYSES

A quarter-scale SASSI model of the caisson was analyzed to take advantage of the symmetrical/anti-symmetrical conditions (Chang et al, 1998). The SASSI 'structure' 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 brick elements with dynamic properties based on smearing of the composite flexural and shear rigidities of the caisson. The hydrodynamic masses simulating the dynamic effects of water in the internal circular openings were smeared in the model and the hydrodynamic masses simulating the external water surround the caisson were treated as lumped masses (Goyal and Chopra, 1988). 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.

TWO-DIMENSIONAL EQUIVALENT LINEAR AND NONLINEAR ANALYSES

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. 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). 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.

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 above 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 (Mok, et al., 1998)

Two-Dimensional Linear and Nonlinear Analyses Using FLAC

Both linear and nonlinear analyses were performed using the finite difference program FLAC. The finite difference grid representing the mode domain is shown on Fig. 4. 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. The analyses were performed in time domain. A Lagrangian approach is used to account for large-strain finite difference grid deformation. 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 5 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 Fig. 6. The shear and bending moment time histories induced in the caisson at the mudline, above-tremie seal, and tremie seal levels are compared on Figs 7 and 8. 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 9 through 12 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 (Fig. 9). The peak shear demand (based on a smeared model) was reduced by about 40 percent (Fig. 11). 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 Fig. 13.

CONCLUSION

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 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.

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.

REFERENCES

Abrahamson, N. [1996]. Personal Communication.

Chang, C.-Y., C.M. Mok, Z.-L. Wang, H.M. Gonnermann, M.A. Ketchum, and C.-C. Chin [1998]. "Seismic Analysis of Large Caisson Foundations for Long-Span Bridges", 6th U.S. National Conference on Earthquake Engineering, Seattle.

Geomatrix Consultants, Inc. [1992]. "Seismic Ground Motion Sudy for West San Francisco Bay Bridge", report submitted to California Department of Transportation.

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

Itasca Consulting Group, Inc. [1993]. "FLAC, fast Lagrangian analysis of continua, Version 3.2".

Lysmer, J., F. Ostadan, M. Tabatabaie, S. Vahdani, and F. Tajirian [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.

Mok, C.M., C.-Y. Chang, R. Settgast, Z.-L. Wang, H.M. Gonnermann, C.-C. Chin, and M. Ketchum [1998]. "Dynamic Soil-Foundation Structure Interaction Analysis of Large Caissons", U.S.-Japan Soil-Structure Interaction Workshop, Menlo Park, U.S.G.S.

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 I.M. Idriss, [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 R. Dobry [1991]. "Effect of Soil Plasticity on Cyclic Response", Journal of Geotechnical Engineering, ASCE, Vol. 117, No.1, pp. 89-107.

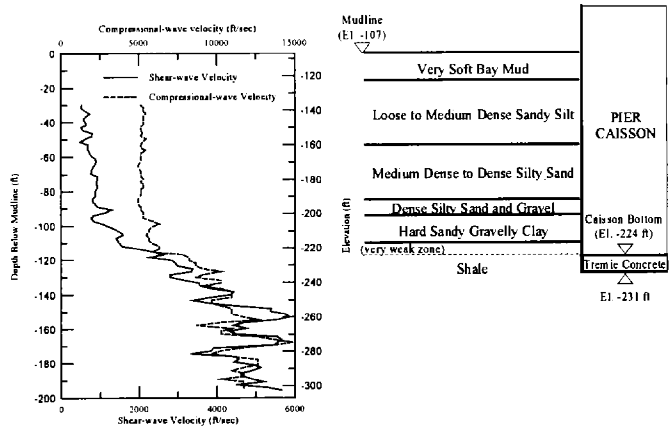

Fig. 1. Idealized Geologic Profile

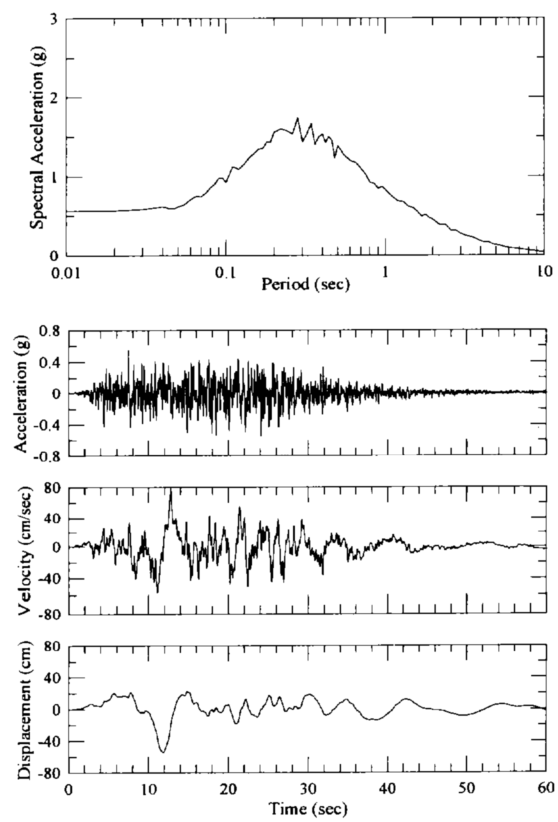

Fig. 2. Longitudinal Input Rock Motions

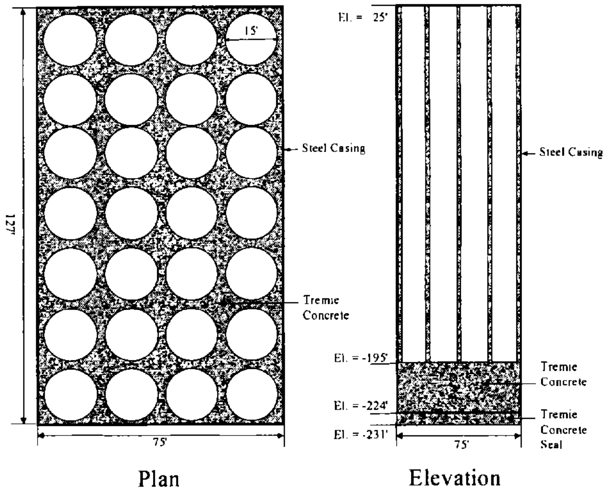

Fig. 3. General Plan and Elevation

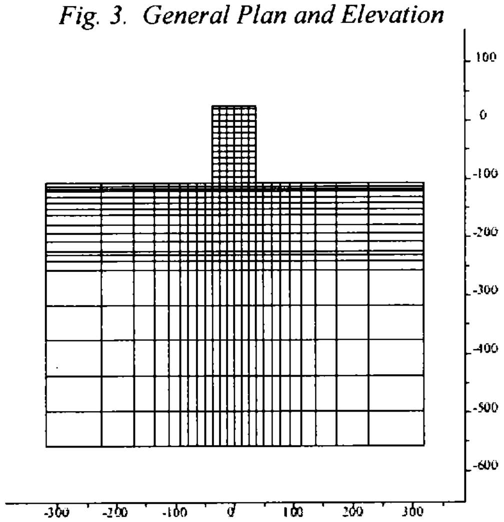

Fig. 4. Finite Difference Grid of 2-D Model

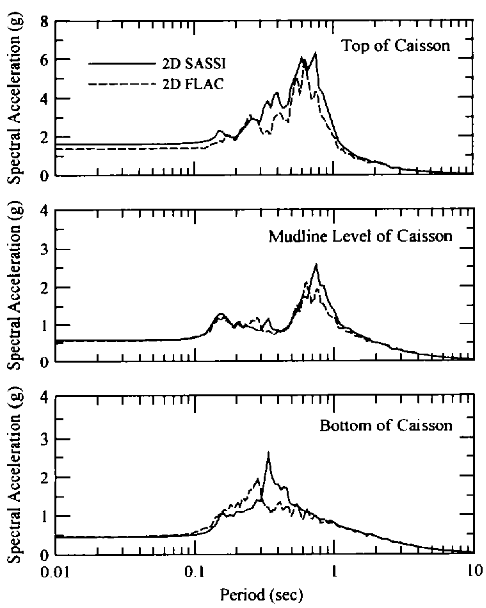

Fig. 5. Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by 2-D SASSI and FLAC Models

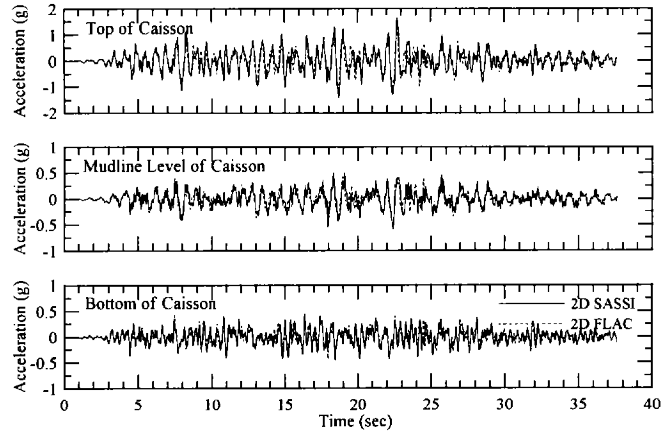

Fig. 6. Comparison of Longitudinal Acceleration Time Histories from 2-D SASSI and FLAC Models

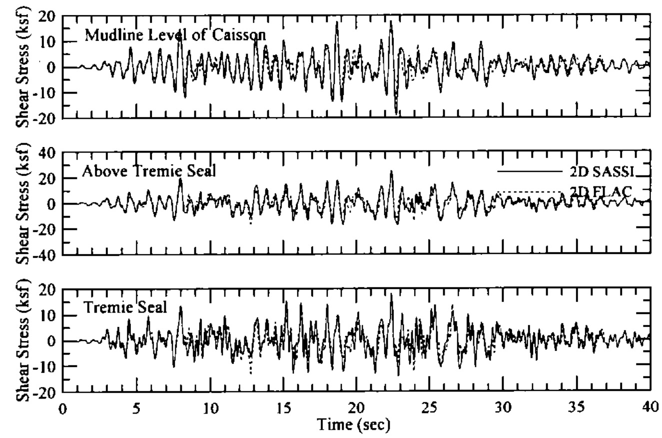

Fig. 7. Comparison of Longitudinal Shear Stress Time Histories from 2-D SASSI and FLAC Models

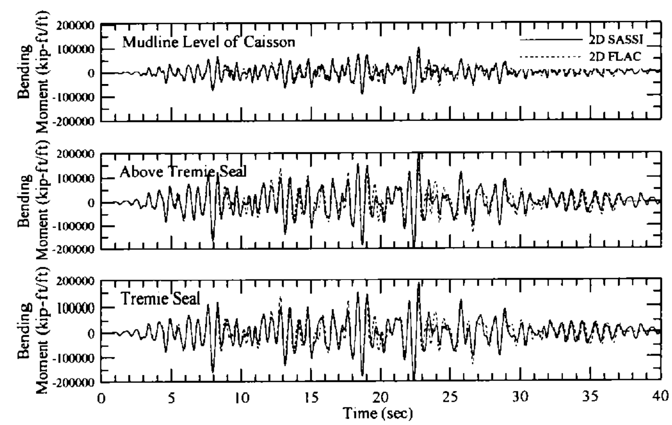

Fig. 8. Comparison of Longitudinal Bending Moment Time Histories from 2-D SASSI and FLAC Models

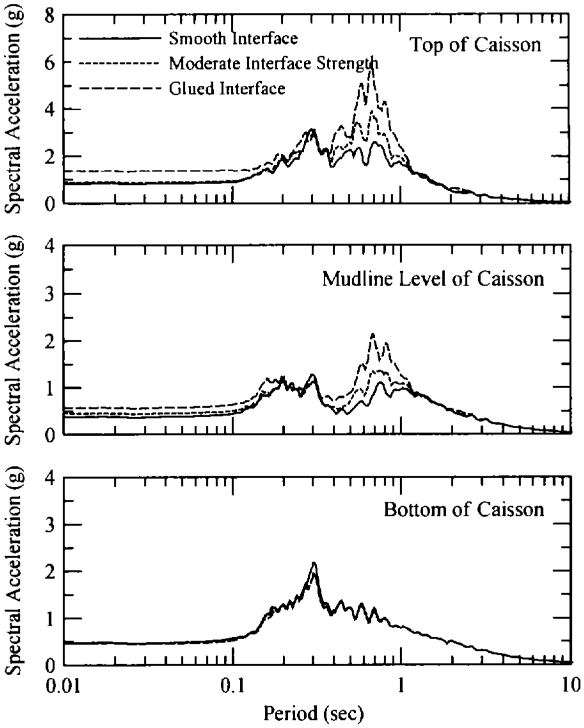

Fig. 9. Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by FLAC Models with Various Interface Properties

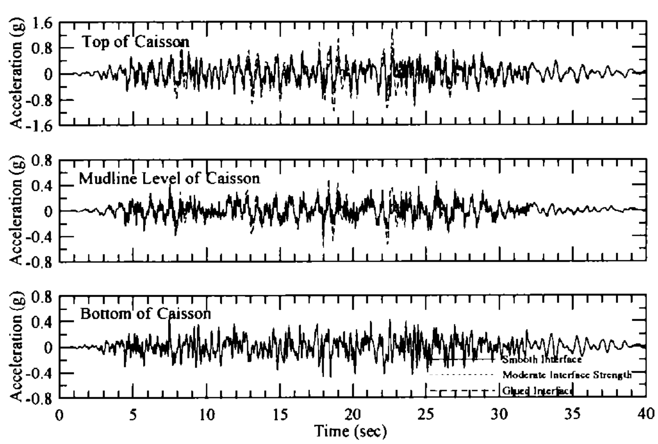

Fig. 10. Comparison of Longitudinal Acceleration Time Histories from FLAC Models with Various Interface Properties

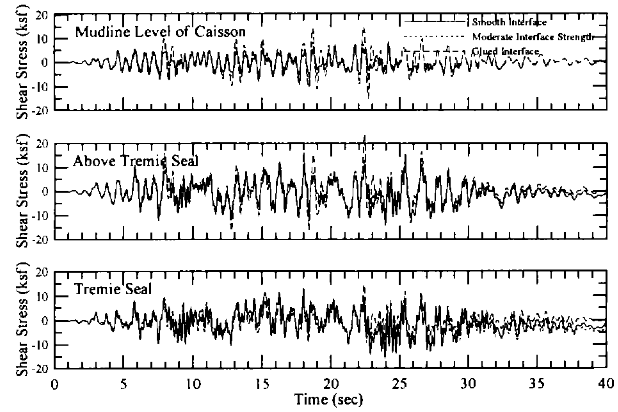

Fig. 11. Comparison of Longitudinal Shear Stress Time Histories from FLAC Models with Various Interface Properties

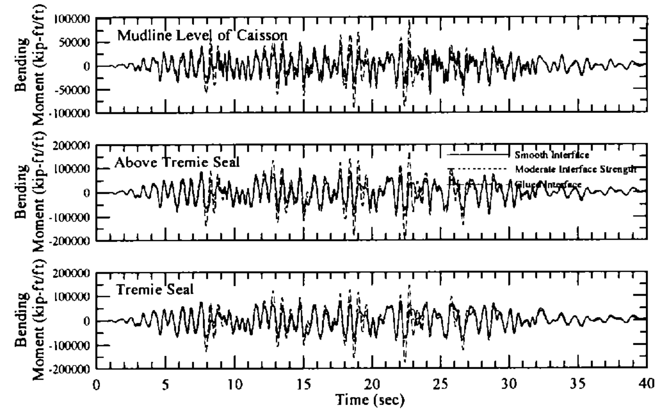

Fig. 12. Comparison of Longitudinal Bending Moment Time Histories from FLAC Models with Various Interface Properties

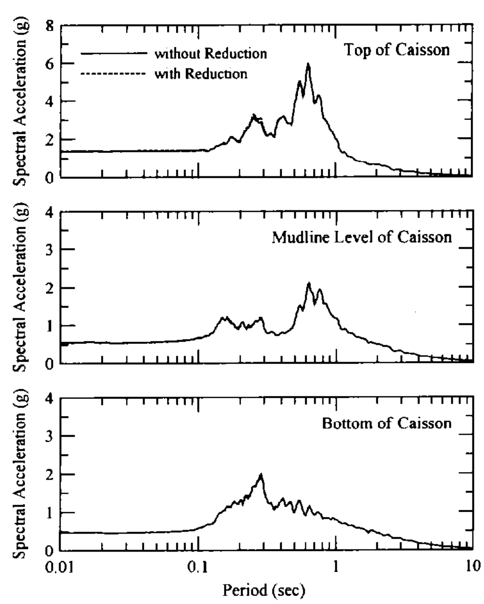

Fig. 13. Comparison of 5% Damped Response Spectra of Longitudinal Motions Computed by FLAC Models with and without Soil Stiffness Reduction