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NRC Seismic Seminar: Soil-Structure Interaction (SSI) including Coherent and Incoherent Ground Motion

August 29, 2007

Washington DC

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Agenda

- Session 1: 9:00 – 10:30 (JJohnson)
  - Elements of SSI
  - Free-Field Ground Motion (Segway from Day 1)
  - SSI Modeling
  - SSI Analysis Methods – Overview
  - CLASSI Substructure Approach
- Break 10:30 – 10:45
- Session 2: 10:45 – 11:45 (FOstadan)
  - SASSI Approach
- Session 3: 11:45 – 12:15 (SShort)
  - Demonstration of SSI Effects
- Lunch Break 12:15 – 1:15

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Agenda (cont)

- Session 4: 1:15 – 2:15 (NAbrahamson)
  - Ground Motion Incoherency
- Session 5: 2:15 – 2:30 (JJohnson)
  - General Treatment of Incoherency
- Session 6: 2:30 – 3:00 (SShort)
  - CLASSI Approach to Incoherency
- Break 3:00 – 3:15
- Session 7: 3:15 – 4:15 (FOstadan)
  - SASSI Approach to Incoherency (SRSS)
- Session 8: 4:15 – 4:45 (SShort)
  - SASSI Approach to Incoherency (Simulation and Algebraic Sum Methods – ACS SASSI)
- Session 9: 4:45 – 5:00 (JJohnson)
  - Summary and Conclusions

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10 Soil–Structure Interaction

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10.1 Soil–Structure Interaction: Statement of the Problem

The response of a structure during an earthquake depends on the characteristics of the ground motion, the surrounding soil, and the structure itself. For structures founded on rock or very stiff soils, the foundation motion is essentially that which would exist in the soil at the level of the foundation in the absence of the structure and any excavation; this motion is denoted the **free-field ground motion.** For soft soils, the foundation motion differs from that in the free field due to the coupling of the soil and structure during the earthquake. This interaction results from the scattering of waves from the foundation and the radiation of energy from the structure due to structural vibrations. Because of these effects, the state of deformation (particle displacements, velocities, and accelerations) in the supporting soil is different from that in the **free field.** As a result, the dynamic response of a structure supported on soft soil may differ substantially in amplitude and frequency content from the response of an identical structure supported on a very stiff soil or rock. The coupled soil–structure system exhibits a peak structural response at a lower frequency than would an identical rigidly supported structure. Also, the amplitude of structural response is affected by the additional energy dissipation introduced into the system through radiation damping and material damping in the soil.

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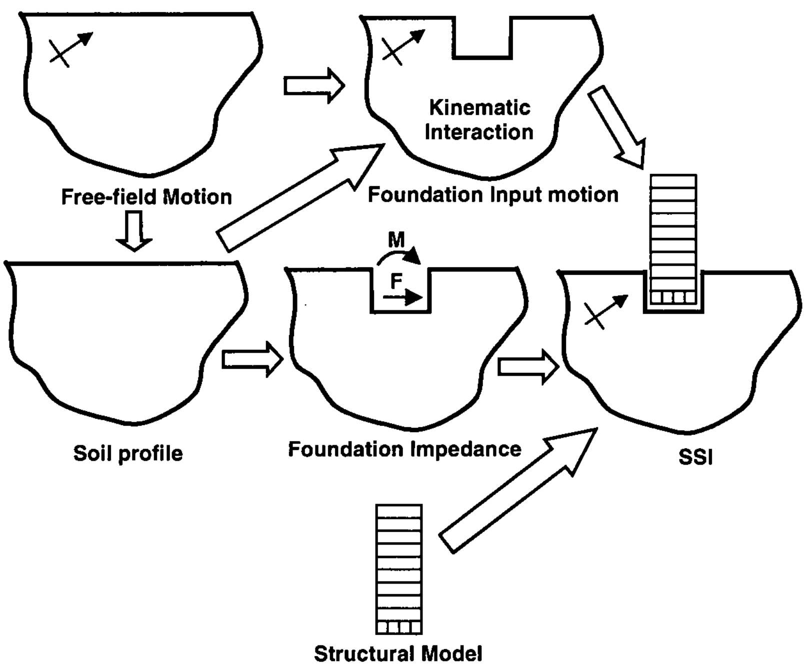


FIGURE 10.1 Schematic representation of the elements of soil–structure interaction.

Much of this chapter focuses on structures for which soil–structure interaction (SSI) is an important phenomenon in the design process of the structure and for systems and components housed therein. The type of structure and its foundation determines the importance of SSI. SSI effects can usually be ignored for conventional structures without significant embedment. Even for conventional structures with embedded foundations, ignoring SSI is usually conservative. However, even for conventional structures, it is prudent to consider and evaluate the potential effects of SSI on structure, system, and component design to assure oneself that excessive conservatism is not being introduced. SSI is most important for massive, stiff structures with mat foundations or with foundation systems significantly stiffened by the structure's load-resisting system. Typical nuclear power plant structures, founded on soil, are particularly affected by SSI. Hence, the references and examples herein are drawn from the nuclear power industry.

10.1.1 Elements of SSI Analysis

The analysis of SSI depends on the specification of the free-field ground motion and the idealization of the soil and structure. Modeling the soil involves determining its configuration and material properties. Modeling the structure includes the structure itself and its foundation. The calculated responses must be interpreted in light of differences between the idealized system and the real physical situation, and the uncertainties known to exist in the phenomenon. Figure 10.1 shows the elements of seismic analysis necessary to calculate seismic response, including SSI effects. Table 10.1 lists the various aspects, including interpretation and recognition of uncertainties in the process.

The state of knowledge of SSI was well documented in 1980 in a compendium [Johnson, 1981] of contributions from key researchers (Luco, Roesset, Seed, and Lysmer) and drew upon other researchers and practitioners as well (Veletsos, Chopra). This reference provided a framework for SSI over the 1980s and 1990s, which were characterized by the accumulation of substantial data supporting and clarifying the roles of the various elements of the SSI phenomenon. Also, significant progress was made in the development and implementation of SSI analysis techniques. In the international nuclear power industry,

TABLE 10.1 Elements of a Seismic Response Analysis Including Soil–Structure Interaction

Specification of the Free Field Ground Motion
Control point
Control motion (peak ground acceleration and response spectra)
Spatial variation of motion (wave propagation mechanism)
Magnitude, duration
Models of soils and structures
Soil Properties
Nominal properties (low and high strain)
Variability
SSI Parameters
Kinematic interaction
Foundation impedances
Structure models
Important features (torsion, floor flexibility, frequency reduction with strain, etc.)
Variability (frequency and mode shapes, damping)
Nonlinear behavior
SSI Analysis
Interpretation of Responses

the 1980s and 1990s saw a revision of important regulatory practices to conform to the current state of knowledge. Documentation of the state of knowledge of SSI was updated in 1991 and 1993 [Johnson and Chang, 1991; Johnson and Asfura, 1993]. The specific purpose of this update was to identify realistic approaches to the treatment of SSI and its uncertainties. This new understanding of the SSI phenomenon permitted the implementation and use of less conservative, more realistic procedures for the design of structures, systems, and components, and for the evaluation of structures, systems, and components when subjected to beyond-design basis earthquake events.

The reader is directed to two other recent references that highlight the current state of the art and practice of SSI analysis. Tseng and Penzien [2000] discuss the SSI problem and its treatment in the context of multi-supported structures, such as bridges. Wolf and Song [2002] summarize numerous elements of the SSI phenomenon and their treatment analytically.

10.1.2 A Significant SSI Experiment

Before proceeding further, it is useful to discuss an important SSI experiment. Very few opportunities exist to record free-field motion and structure response for an earthquake. In the mid-1980s, the Electric Power Research Institute (EPRI), in cooperation with Taiwan Power Company (TPC), constructed two scale-model reinforced concrete nuclear reactor containment buildings (one quarter and one twelfth scale). The scale models were located within an array of strong motion instruments (SMART-l, Strong Motion Array Taiwan, Number 1) in Lotung, Taiwan, sponsored by the U.S. National Science Foundation and maintained by the Institute of Earth Sciences of Academia Sciences of Taiwan. The experiment was extensively instrumented in the free field and in the structures. The objectives of the experiment were to measure the responses at instrumented locations due to vibration tests, and due to actual earthquakes, sponsor a numerical experiment designed to validate analysis procedures and to measure free-field and structure response for further validation of the SSI phenomenon and SSI analysis techniques. This Lotung site was chosen based on its known high seismicity.

Using this data base, a cooperative program to validate SSI analysis methodologies was sponsored by EPRI, TPC, and the U.S. Nuclear Regulatory Commission (NRC). Numerous publications document the results of the SSI analysis studies. A two-volume EPRI report [EPRI, 1989] contains the proceedings of a workshop held in Palo Alto, California on December 11–13, 1987 to discuss the experiment, data

collected, and analyses performed to investigate SSI analysis methodologies and their application. Johnson et al. [1989] performed one set of analyses from which results are presented here to demonstrate various aspects of the SSI phenomenon. A summary of lessons learned was published [Tseng and Hadjian, 1991]. Finally, a summary of sensitivity studies performed was documented in 1997 as the field experiment was completed. When appropriate throughout this chapter, data from the Lotung experiment are presented and discussed.

10.2 Specification of the Free-Field Ground Motion

The term *free-field ground motion* denotes the motion that would occur in soil or rock in the absence of the structure or any excavation. Describing the free-field ground motion at a site for SSI analysis purposes entails specifying the point at which the motion is applied (the **control point**), the amplitude and frequency characteristics of the motion (referred to as the **control motion** and typically defined in terms of ground response spectra, power spectral density functions, and/or time histories), the spatial variation of the motion, and, in some cases, duration, magnitude, and other earthquake characteristics. In terms of SSI, the variation of motion over the depth and width of the foundation is relevant. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion over both the embedment depth and the foundation width should be known.

10.2.1 Control Motion and Earthquake Characteristics

The control motion is defined by specifying the amplitude and frequency content of the earthquake to be considered. Two purposes exist for SSI analysis of a structure: design or evaluation of a facility for a specified earthquake level, or the evaluation of a structure for a specific event. In the former case, statistical combinations of recorded or predicted earthquake motions are typically the bases. In the latter case, recorded earthquake acceleration time histories typically comprise the free-field ground motions.

The frequency content of the motion is one of the most important aspects of the free-field motion as it affects structure response. For linear structural behavior and equivalent linear SSI, the frequency content of the free-field motion compared to important frequencies of the soil–structure system determines response. For structures expected to behave inelastically during the earthquake (and, in particular, structures for which SSI is not important), structure response is determined by the frequency content of the free-field motion, i.e., in the frequency range from the elastic frequency to a lower frequency corresponding to a certain amount of inelastic deformation.

10.2.1.1 Aggregated Ground Motions

A wide variety of ground response spectra has been specified for design of major facilities, such as nuclear power plants, major bridges, critical facilities (e.g., LNG storage and processing plants), and major infrastructure projects. For nuclear power plants, depending on the vintage of the plant and the site soil conditions, the majority of the design ground response spectra has been relatively broad-banded spectra representing a combination of earthquakes of different magnitudes and distances from the site. Construction of such design spectra is usually based on a statistical analysis of recorded motions and frequently targeted to a 50% or 84% nonexceedance probability (NEP). Three points are important relative to these broad-banded spectra. First, earthquakes of different magnitudes and distances control different frequency ranges of the spectra. Small magnitude earthquakes contribute more to the high frequency range than to the low frequency range and so forth. Second, it is highly unlikely that a single earthquake will have frequency content matching the design ground response spectra. Hence, a degree of conservatism is added when broad-banded response spectra define the control motion. Third, a single earthquake can have frequency content that exceeds the design ground response spectra in selected frequency ranges. The likelihood of the exceedance depends on the NEP of the design spectra. Figure 10.2 compares several ground response spectra used in the design or evaluation process. U.S. NRC Regulatory Guide 1.60 [U.S. Atomic Energy Commission, 1973] response spectra defined design criteria

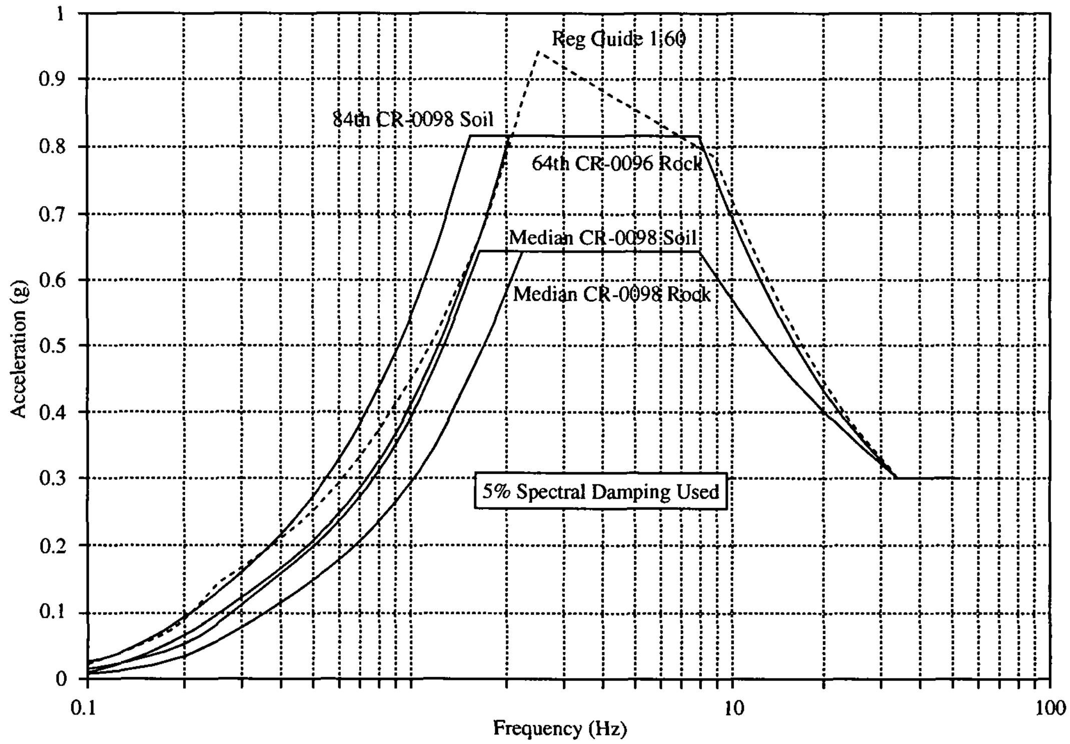


FIGURE 10.2 Examples of aggregated ground motion response spectra.

for U.S. nuclear power plants designed after about 1973. These spectra were targeted to an 84% NEP of the data considered, but exceed this target in selected frequency ranges. Figure 10.2 shows an additional set of broad-banded spectra for rock and alluvial sites and for 50% and 84% NEP. U.S. NRC *NUREG-0098* is the source. These spectra have been used extensively for defining seismic margin earthquakes, for which beyond design basis assessments have been performed for nuclear power plants in the United States and other countries. This broad-banded spectral shape is also used to define design criteria for new design.

In addition to the site-independent ground response spectra discussed above, two additional forms of ground response spectra are being generated and used for site-specific design or evaluations. First, site-specific spectra are generated by accumulating recorded data that meet the design earthquake characteristics and local site conditions, analyzing the data statistically, calculating ground response spectra of various statistical attributes, and selecting response spectra for design or reevaluation. Figure 10.3 shows an example for a specific site; the 84% NEP was selected. Second, seismic hazard studies are performed to generate families of seismic hazard curves which yield estimates of the probability of exceedance of earthquakes with specified peak ground acceleration (PGA) values or greater. Confidence limits for these seismic hazard estimates are derived from the family of curves. Companion to the seismic hazard curves are uniform hazard spectra (UHS), which are ground response spectra generated for a specified return period or probability of exceedance, with various confidence limits. Figure 10.4 shows example UHS for a specific site, 10^{-4} probability of occurrence per annum (sometimes termed a "10,000-year return period"), and 15, 50, and 85% confidence limits. Such spectra are generated by the same probabilistic seismic hazard methodology as is used in generating the seismic hazard curves for PGAs.

In almost all cases, design ground response spectra are accompanied by ground motion acceleration time histories — artificial acceleration time histories generated with response spectra that match or exceed the design ground response spectra. Artificial acceleration time histories are generated by numerical methods and not recorded motions. Due to the enveloping process, additional conservatism is

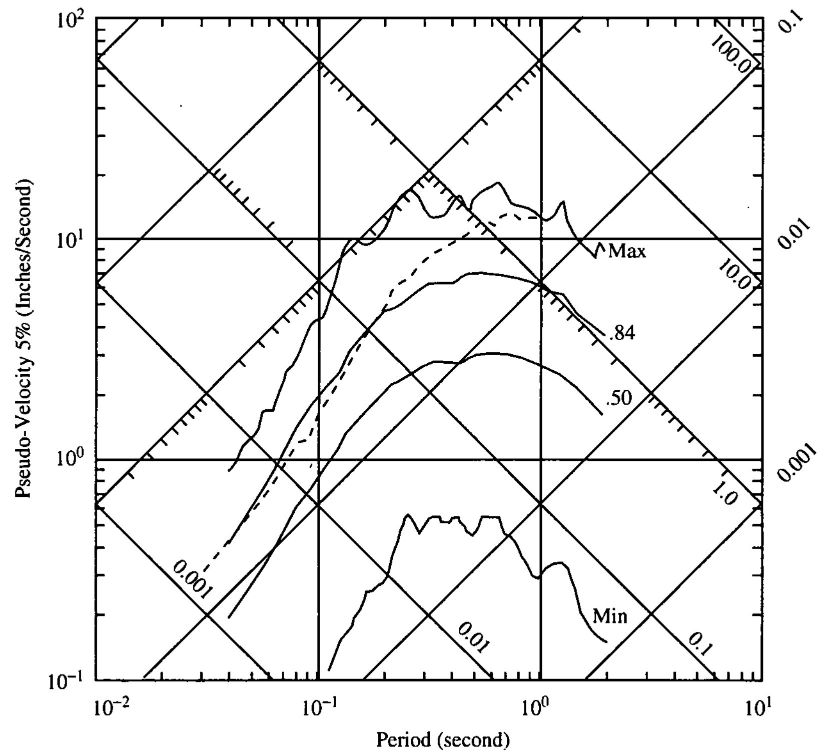


FIGURE 10.3 Site-specific response spectrum.

introduced. Ground motion time histories are used to generate in-structure response — loads, response spectra, displacements, etc.

10.2.1.2 Individual Recorded Events

The previous sections discussed aggregated motions as derived from recorded data or empirical models based on recorded data. These aggregated motions do not represent a single event. They have been developed for various design and evaluation purposes. It is informative to present recorded motions from actual earthquakes and visualize differences between a single event and the aggregated motions. Two earthquakes of note from a SSI standpoint were the May 20, 1986 and November 14, 1986 events which affected the Lotung scale model structures. Figure 10.5 contains response spectra generated from the acceleration time histories recorded on the soil-free surface for the May 20, 1986 event. Each earthquake produced PGAs greater than about 0.2 *g* in the horizontal direction with principally low frequency motion, i.e., less than about 5 Hz.

10.2.1.3 Magnitude Effects

Ground motion frequency content is strongly dependent on specific factors of the earthquake and site. Two particularly important characteristics of the earthquake are its magnitude and epicentral distance from the site. Small magnitude events are characterized by narrow-banded response spectra and high frequency in comparison to moderate magnitude events. Figure 10.6 [Chang et al., 1985] illustrates the effect of magnitude on response spectral shape. In the figure, the response spectral shape obtained from a series of small magnitude ($M_L < 4$) earthquakes is compared with the response spectral shape from a moderate magnitude ($M_L = 6.5$) event. As shown, the small magnitude earthquakes are characterized by a narrow-banded response spectral shape and greater high frequency content than the moderate magnitude event.

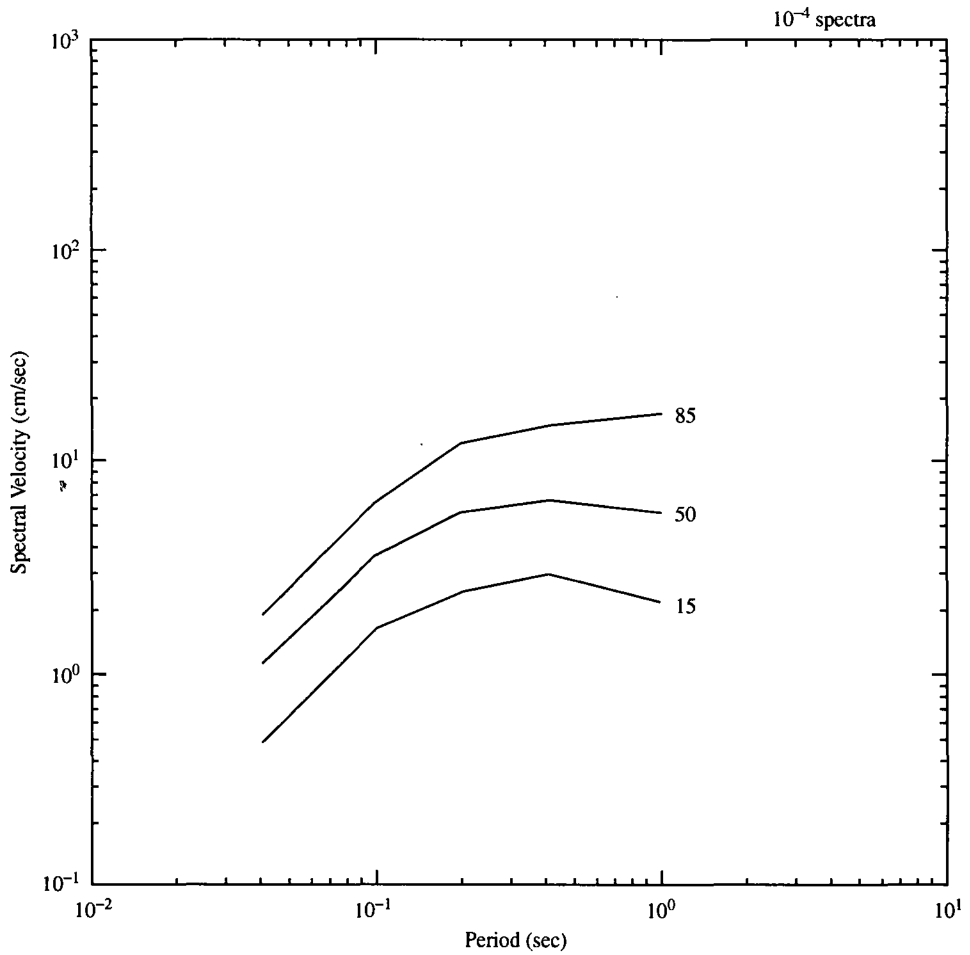


FIGURE 10.4 Uniform hazard spectra.

10.2.1.4 Soil vs. Rock Sites

One of the most important parameters governing amplitude and frequency content of free-field ground motions is the local site conditions, i.e., soil vs. rock and shallow, soft soil overlying a stiff soil or rock. Two sets of data dramatically demonstrate the difference in motions recorded on rock vs. soil for the same earthquake and sites in close proximity. The Ashigara Valley in Japan is located about 80 km southwest of Tokyo. A digital strong-motion accelerograph array network [Kudo et al., 1988] was installed in this seismically very active area. The geological profile of the Ashigara Valley is shown in Figure 10.7. The valley is an alluvial basin with rock outcrops at the mountain side and soft sedimentary soil layers at the basin. Accelerometers were installed in the rock outcrop (KR1), at the surface of the soft layers (KS1 and KS2) and inside the soil (KD2). Figure 10.8 shows response spectra of motions recorded at the rock outcrop (KR1) and at the surface of soft layers (K2S) for an earthquake occurring on August 5, 1990. The figure clearly shows differences in motions due to different site conditions. High frequencies of about 10 Hz and greater are dominant at the rock site and frequencies below about 5 Hz are dominant at the soil surface. Figure 10.8 clearly shows the large amplification of the low frequency waves due to the soil response and the deamplification of the high frequency waves due to the filtering effect of the soft soil.

The Loma Prieta earthquake, October 17, 1989, with epicenter near San Francisco, provided the opportunity to collect and evaluate recorded motions on rock and soft soil sites, in some cases immediately

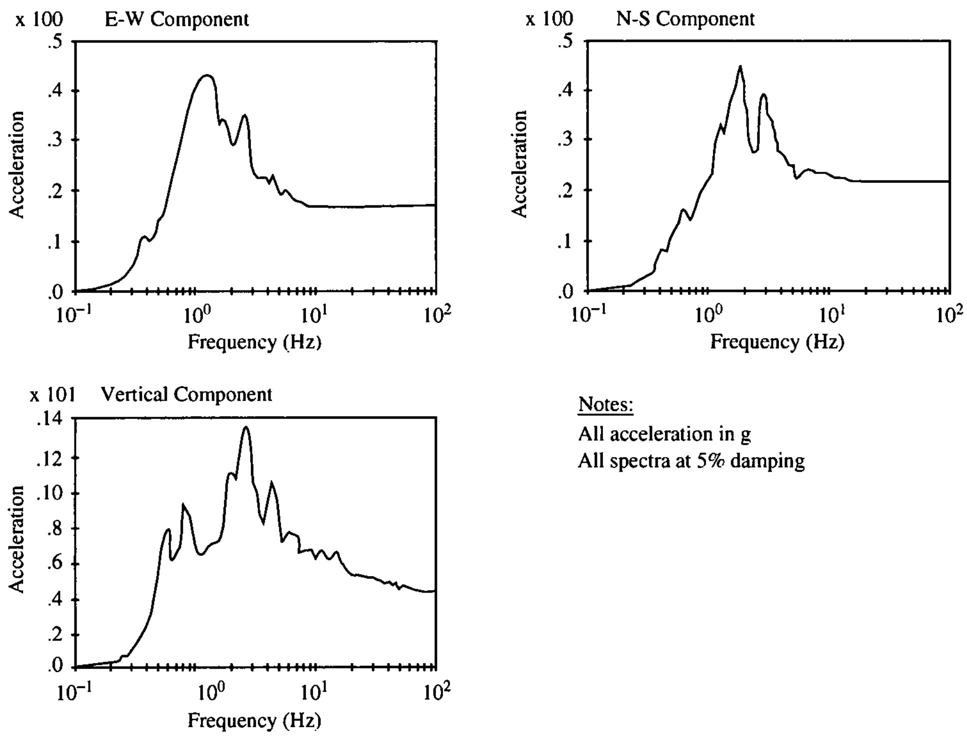


FIGURE 10.5 Response spectra of free-field ground surface motions of the May 20, 1986 earthquake, recorded at Lotung, Taiwan.

adjacent to each other. Figure 10.9 compares response spectra for recorded motions on rock (Yerba Buena Island) and on soil (downtown Oakland). Significant differences in amplification are obvious, with the rock motions being less. Similar observations were made when comparing response spectral amplification factors for soil (Treasure Island) vs. rock (Yerba Buena Island). For horizontal motions, significant amplification of soil over rock is apparent. For vertical motions, the relationship between the rock and soil values was not as clear. As observed in other locations, it is surmised that the presence of a high water table and its effect on vertical ground motion are uncertain. Note that Treasure Island and Yerba Buena Island are the north and south ends of a single land mass (Yerba Buena Island is a natural island with highest elevation several hundred feet above the San Francisco Bay, while the contiguous Treasure Island is of similar areal extent, only a few feet above the bay, and was created by hydraulic land fill in the 1930s).

The evidence supporting the differences between rock and soft soil motions continues to mount, emphasizing the effect of local site conditions on the amplitude and frequency content of the motion.

10.2.2 Control Point

The term **control point** designates the location at which the control motion is defined. The control point should always be defined on the free surface of soil or rock at the site of interest. Specifying the control point at locations other than a free surface ignores the physics of the problem and the source of data used in defining the control motion. Past nuclear regulatory practice specified the control point to be at foundation level in the free field, which is technically untenable. It not only ignores the physics of the problem and the source of recorded data, but also results in motions on the free surface whose response spectra display peaks and valleys associated with frequencies of the embedment layer that are unrealistic. The frequencies of these peaks correspond to the frequencies of the soil layer between the foundation and the free surface. These free surface motions are dependent on the foundation depth rather than

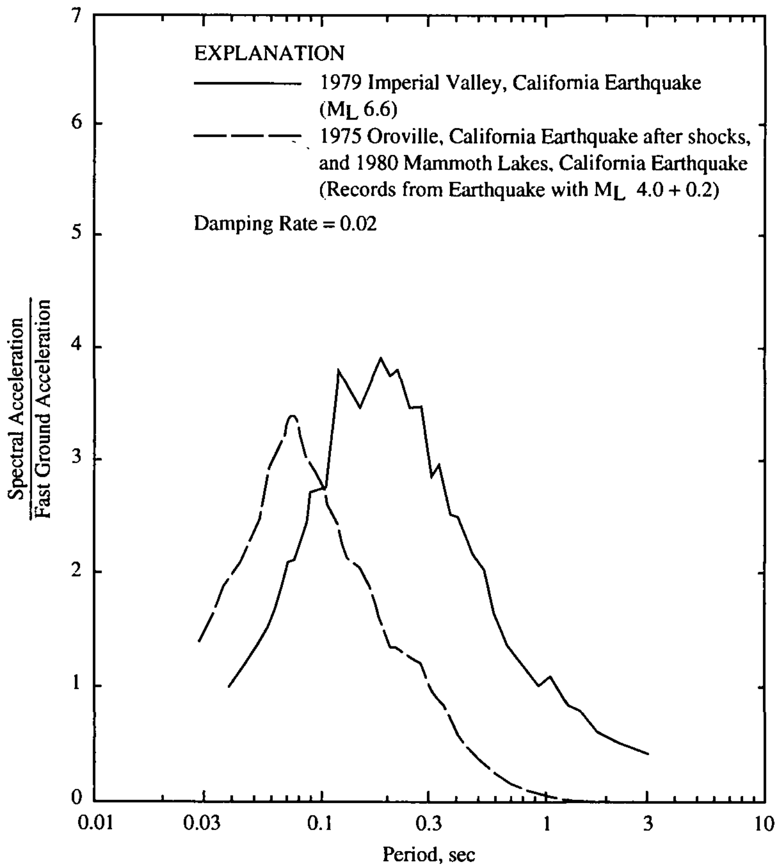


FIGURE 10.6 Illustration of effect of earthquake magnitude on response spectral shape obtained from statistical analysis.

free-field site characteristics. In addition, the peak acceleration of the resulting free surface motion is typically calculated to be significantly greater than the control motion. Hence, by all seismological definitions, a design or evaluation earthquake is increased. A 0.25-*g* earthquake may become a 0.35-*g* earthquake, or greater, depending on the embedment depth, soil properties, and control motion characteristics. Finally, specification of the control point at foundation level rather than the soil free surface effectively penalizes partially embedded structures compared to surface-founded structures, which contradicts common sense and observations.

Simplistic SSI analyses frequently ignore wave scattering effects (kinematic interaction) for embedded foundations. In so doing, the foundation input motion is assumed to be identical to the control motion. Implicit in this assumption is the definition of the control point as any point on the foundation and no spatial variation of ground motion. This assumption is almost always conservative and frequently extremely conservative. Recognition of this conservatism is important in interpreting the results of the SSI analysis.

10.2.3 Spatial Variation of Motion and Kinematic Interaction

Spatial variations of ground motion refer to differences in amplitude and/or phase of ground motions with horizontal distance or depth in the free field. Spatial variations of ground motion are associated with different types of seismic waves and various wave propagation phenomena, including reflection at the free surface, reflection and refraction at interfaces and boundaries between geological strata having

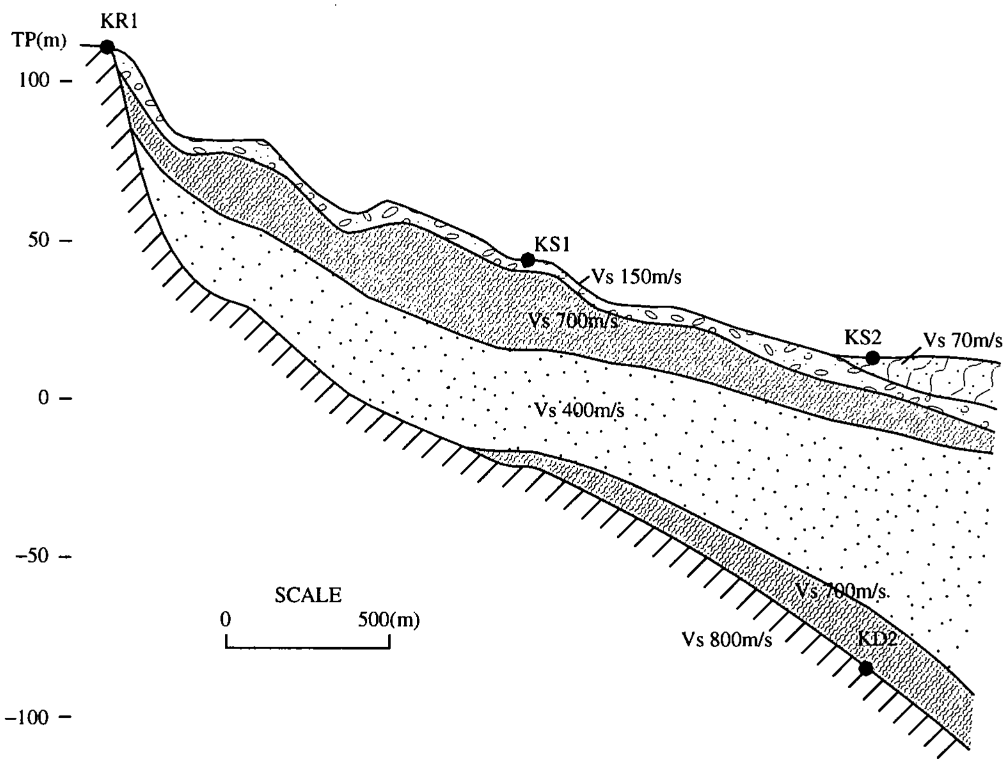


FIGURE 10.7 Ashigara Valley, Japan.

different properties, and diffraction and scattering induced by nonuniform subsurface geological strata and topographic effects along the propagation path of the seismic waves. A vertically incident body wave propagating in such a medium will include ground motions having identical amplitudes and phase at different points on a horizontal plane (neglecting source-to-site attenuation effects over short horizontal distances). A plane wave propagating horizontally at some apparent phase velocity will induce ground motion having identical amplitudes but with a shift in phase in the horizontal direction associated with the apparent horizontal propagation velocity of the wave. In either of these ideal cases, the ground motions are considered to be coherent, in that amplitudes (acceleration time histories and their response spectra) do not vary with location in a horizontal plane. Incoherence of ground motion, on the other hand, may result from wave scattering due to inhomogeneities of soil/rock media and topographic effects along the propagation path of the seismic waves. Both of these phenomena are discussed below.

In terms of the SSI phenomenon, variations of the ground motion over the depth and width of the foundation (or foundations for multifoundation systems) are the important aspect. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion on both the embedded depth and foundation width is important. Spatial variations of ground motions are discussed in terms of variations with depth and horizontal distances.

10.2.4 Validation of Ground Motion Variation with Depth of Soil

10.2.4.1 Variation of Ground Motion with Depth of Soil

For either vertically or nonvertically incident waves, ground motions vary with depth. These variations can generally be expressed in terms of peak amplitudes, frequency content, and phase. Variations of ground motion with depth due to vertically and nonvertically incident body waves and surface waves have been extensively studied analytically by many investigators [Chang et al., 1985]. These studies, based

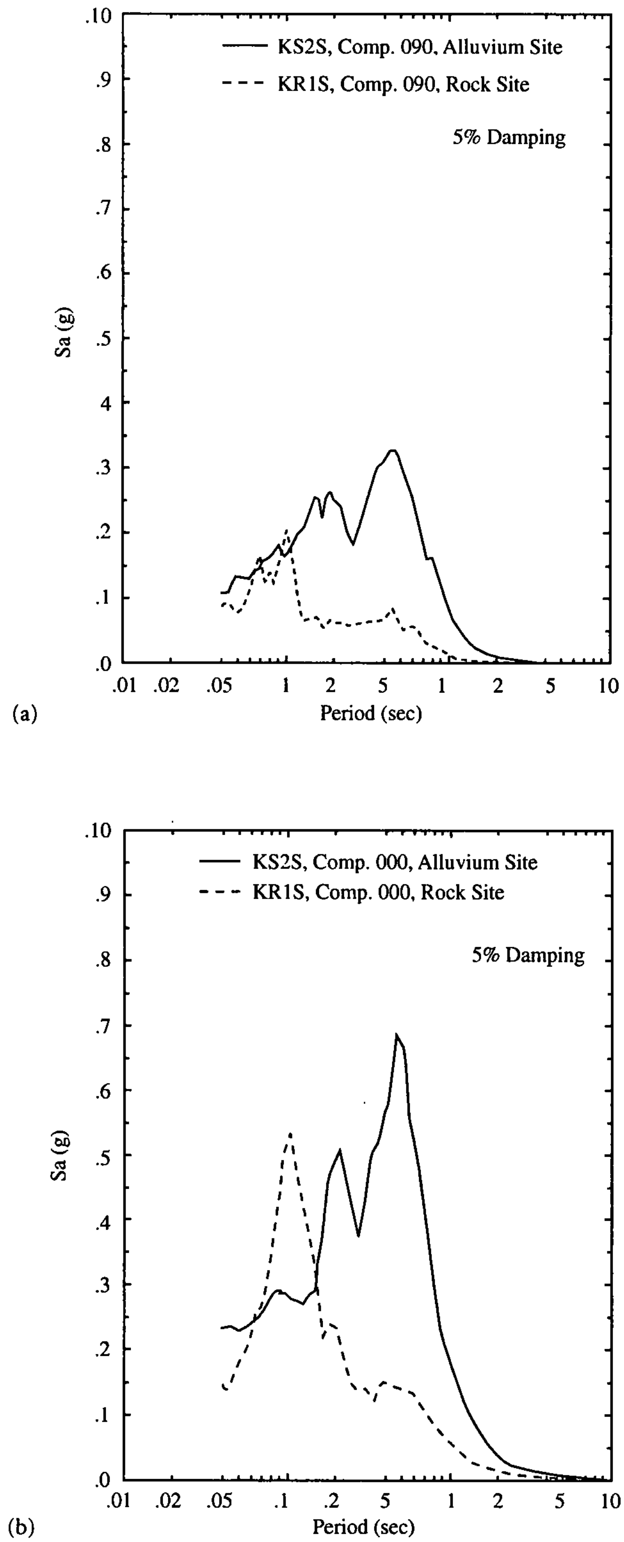


FIGURE 10.8 Response spectra at rock and alluvium sites, Ashigara Valley, Japan.

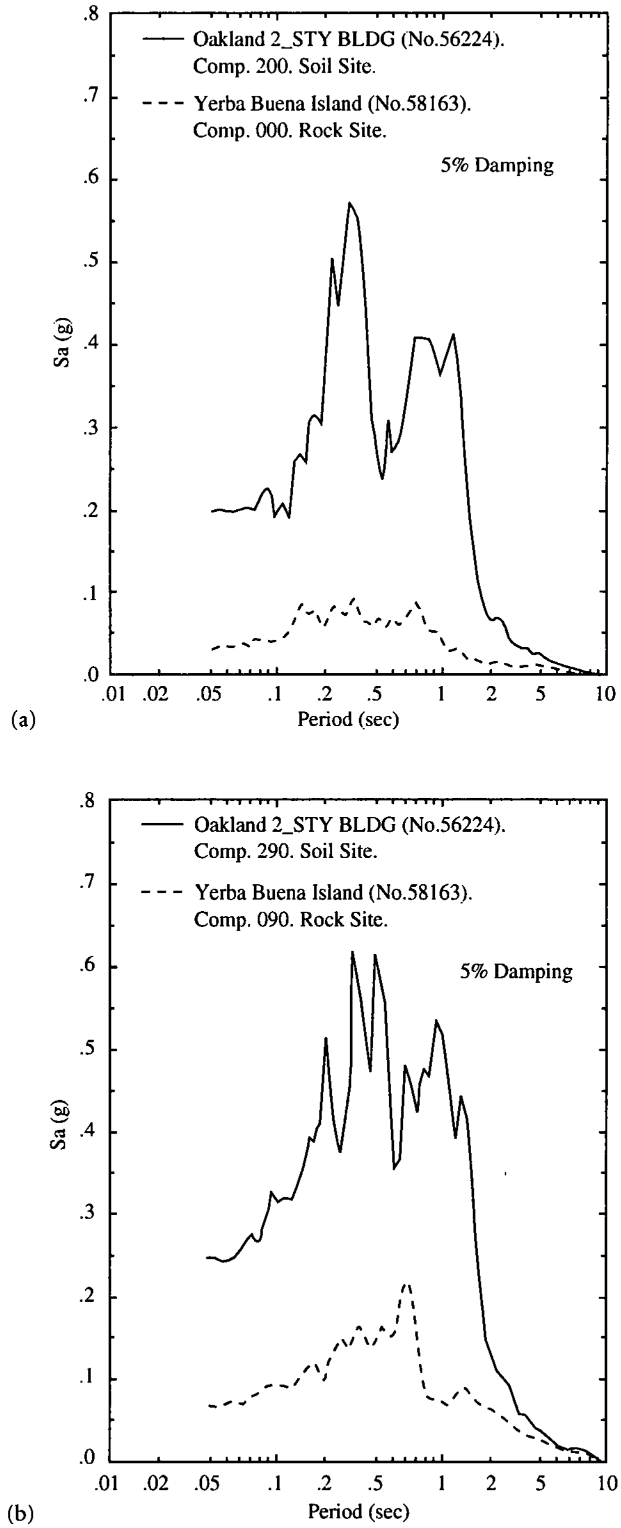


FIGURE 10.9 Response spectra at rock and soil sites, Loma Prieta earthquake.

TABLE 10.2 Free-Field Downhole Arrays

Location	Instrument Depth (m)	Reference
Natimasu, Tokyo	−1, −5, −8, −22, −55	Chang et al. (1985)
Waseda, Tokyo	−1, −17, −67, −123	Chang et al. (1985)
Menlo Park, CA	GL, −12, −40, −186	Chang et al. (1985)
Richmond Field Station, CA	GL, −14.5, −40	Chang et al. (1985)
Ukishima Park, Tokyo	GL, −27, −67, −127	Chang et al. (1985)
Futtsu Cape, Chiba	GL, −70, −110	Chang et al. (1985)
Kannonzaki, Yokosuka City	GL, −80, −120	Chang et al. (1985)
Tokyo International Airport	GL, −50; GL, −65	Chang et al. (1985)
Ohgishima Station, Kawasaki City	GL, −15, −38, −150	Gazetas, G. and Bianchini, G. (1979)
Earthquake Research Institute	GL, −82	Chang et al. (1985)
Miyako, Tokyo	−0.5, −18, −26.5	Chang et al. (1985)
Tomakomai, Hokkaido	GL, −30, −50	Chang et al. (1985)
Tateyama, Tokyo	−26; −38, −100	Chang et al. (1985)
Higashi-Matsuyama City, Saitama	−1, −58, −121	Chang et al. (1985)
Shuzenji-cho, Shizuoka	−36, − 100, −49, −74	Chang et al. (1985)
Choshi City, Chiba	GL, −18	Chang et al. (1985)
Beatty, Nevada	GL, −41	Chang et al. (1985)
Fukushima Nuclear Power Plant	GL, −50; −0.5, −3, −8.5, −23.5	Yanev et al. (1979); Tanaka et al. (1973)
Lotung, Taiwan, ROC	GL, −6, −11, −17, −47	Chang et al. (1990); Chang et al. (1991); Hadjian et al. (1991)
Turkey Flat, CA	GL, −10, −20	Cramer (1991); Proceedings (1992)
Ashigara Valley, Japan	GL, −30, −95	Proceedings (1992); Midorikawa (1992)
Games Valley, CA	GL, −6, − 15, −22, −220	Seale and Archuleta (1991)

on the physics of plane wave propagation through layered media, all indicate that, due primarily to the free surface effect, ground motions generally decrease with depth. The nature of the variation is a function of frequency content and wave type of the incident waves, soil layering, and dynamic soil properties of each soil layer, including shear and compressional wave velocities, damping ratio, and mass density.

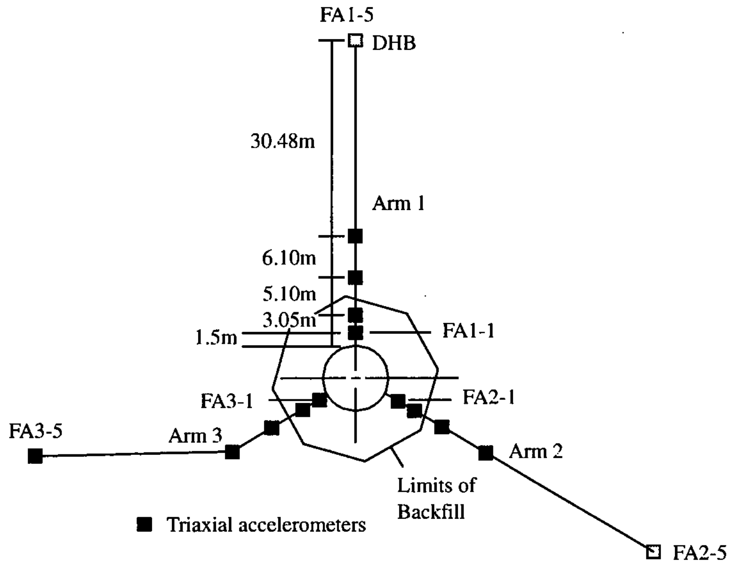
10.2.4.2 Free-Field Motion

A review and summary of observational data on the variations of earthquake ground motion with depth was conducted by Seed and Lysmer [Johnson, 1981] and Chang et al. [1985] reflecting the state of knowledge as of 1980 and 1985, respectively. Recordings from two downhole arrays in Japan were analyzed [Chang et al., 1985] and a review of published data from a number of downhole arrays in Japan and the United States was conducted. Based on the review of these data on the variations of ground motion with depth, it was concluded that there is a good body of data to show that, in general, both peak acceleration and response spectra decrease significantly with depth in the range of typical embedment depths of structures, i.e., less than approximately 25 m; and it appears that deconvolution procedures assuming vertically propagating shear waves provide reasonable estimates of the variations of ground motion with depth.

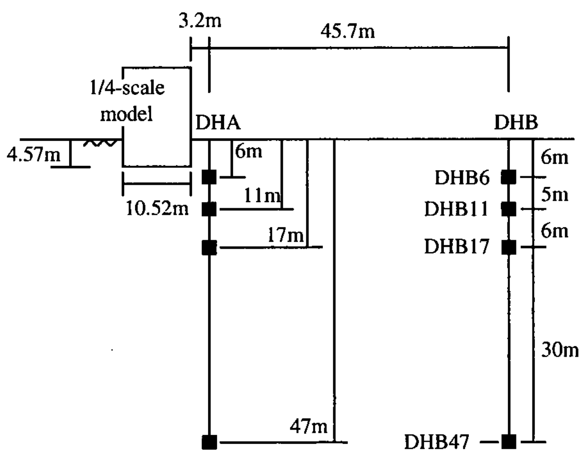
Table 10. 2 summarizes the sources of data evaluated by Seed and Lysmer [Johnson, 1981] and Chang et al. [1985]. Since 1985, substantial additional data have been recorded and evaluated. These sources are included in Table 10.2 and one of the most relevant is discussed next.

10.2.4.3 Lotung, Taiwan

As part of the Lotung experiment, downhole free-field data were recorded at depths of 6, 11, 17, and 47 m. Figure 10.10 [EPRI, 1989] shows the configuration of the arrays. Equivalent linear deconvolution analyses and nonlinear convolution analyses were performed. Deconvolution analysis has as the starting point the free-field surface motion with the objective being to calculate the motion at various depths in the soil. For the Lotung experiment, the depths at which the earthquake responses were calculated corresponded to the



(a) Surface Instrument Arrays



(b) Downhole Instrument Arrays

FIGURE 10.10 Location of (a) surface and (b) downhole accelerographs at the Lotung, Taiwan site.

locations of the downhole accelerometers for comparison purposes. Convolution analysis is the inverse process, i.e., starting with the recorded motion at depth in the soil, calculate the earthquake motion at the free surface or at other depths. These extensive studies investigating analytical modeling of the phenomenon clearly support the observed and analytically determined variation of motion with depth in the soil and, in fact, a reduction of motion with depth as expected. In addition, the assumption of vertically propagating shear waves, and an equivalent linear representation of soil material behavior, well models the wave propagation phenomenon, especially at depths in the soil important to SSI [EPRI, 1989]. Figure 10.11 compares recorded and analytically predicted responses for one of the models.

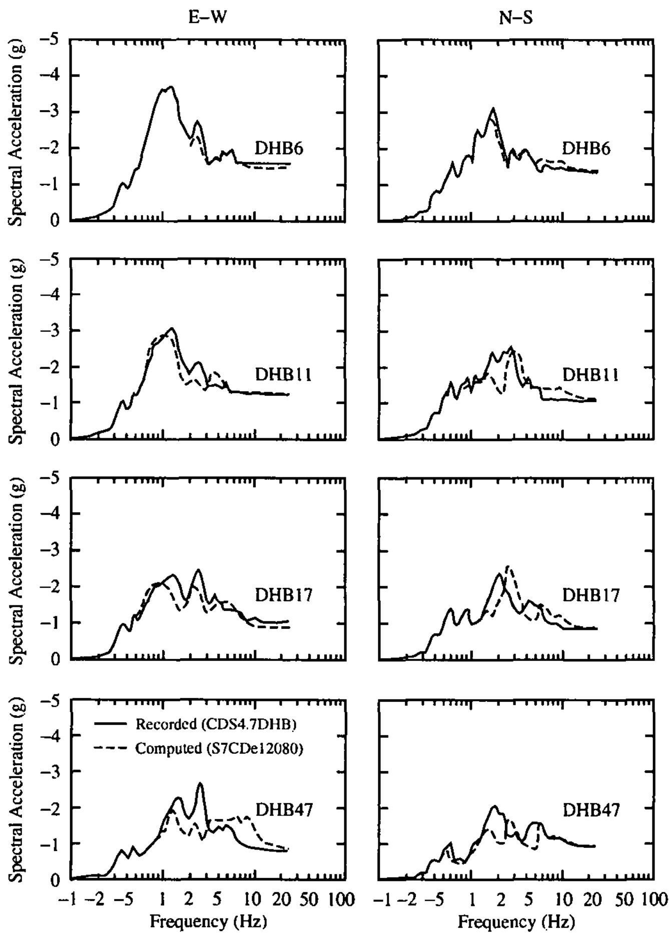


FIGURE 10.11 Comparison of recorded and computed response spectra (5% damping), deconvolved with iterated strain-compatible properties, Event LSST07.

10.2.5 Validation of Kinematic Interaction

10.2.5.1 Foundation Response

The SSI phenomenon can be thought of as two elements: kinematic and inertial interaction. Kinematic interaction is the integrating effect that occurs as portions of the structure and foundation that interface with the soil or rock are subjected to differing free-field ground motion. Variations in translational free-field ground motion result in net translations and accompanying rotations due to this integration or averaging process. Kinematic interaction is typically treated separately from a conceptual standpoint and frequently from a calculational standpoint. The result of accounting for kinematic interaction is to generate an effective input motion, which is denoted *foundation* input motion. The mathematical transformation from the free-field surface motion to the foundation input motion is through the scattering matrix. Inertial interaction denotes the phenomenon of dynamic behavior of the coupled soil–structure system. The base excitation is defined by the foundation input motion accounting for kinematic interaction. The behavior of the foundation on the soil is modeled by foundation impedances (generalized soil springs) that describe the force-displacement and radiation damping characteristics of the soil. The structure is modeled by lumped mass and distributed stiffness models representing its dynamic response characteristics. These elements are combined to calculate the dynamic response of the structure, including the effects of SSI.

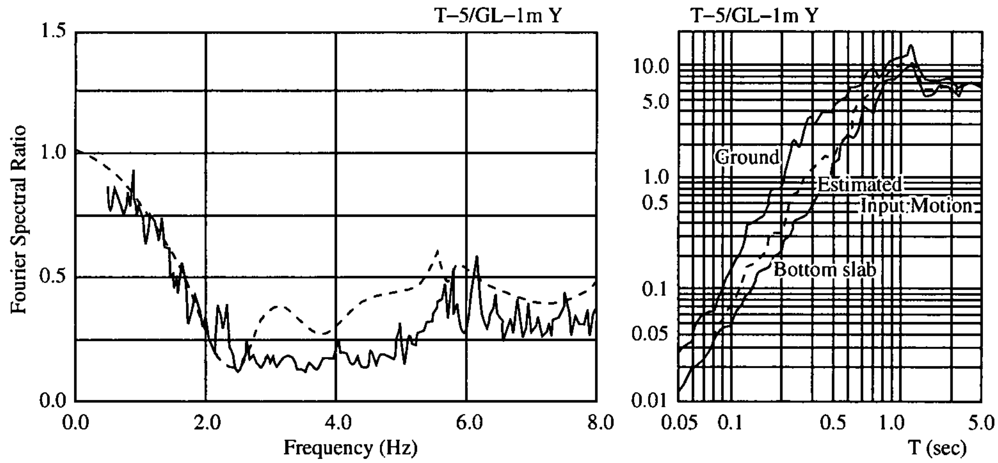


FIGURE 10.12 Observed transfer function between foundation level motion (–26.2 m) of a large-scale below-ground LNG tank and free-field (–1 m) horizontal motion. (——), observed average value over 18 earthquakes; (– – – –), theoretical.

Validation of the component pieces of the process proceeds by considering kinematic interaction. A comparison of motions recorded in the free field and on the base of partially embedded structures provides excellent data validating the effects of kinematic interaction and the spatial variation of ground motion. All recorded data on structures include the effects of SSI to some extent. For purposes of validating the spatial variation of motion with depth in the soil, observations of kinematic interaction are sought. The ideal situation is one where free-field surface motions and foundation motions are recorded for a structure whose embedded portion is stiff, approaching rigid behavior, and the dynamic characteristics of the structure are such that inertial interaction is a minimal effect.

10.2.5.2 Free-Field Surface Motions vs. Foundation Response

Typically, differences in peak values, time histories, and response spectra were observed and transfer functions relating foundation response to free-field surface motion were generated. These frequency-dependent transfer functions are, in essence, one element of the scattering matrix when inertial interaction effects are minimal, i.e., the component relating foundation input motion to horizontal free-field surface motion. In no case was enough information available to generate the rocking component of the scattering matrix from recorded motions. To do so requires recorded rotational acceleration time histories or the ability to generate them from other measurements.

10.2.5.3 LNG Tanks, Japan

Eighteen deeply embedded LNG tanks of varying dimensions were instrumented and motions recorded on their foundations and in the free field for a large number of earthquakes. Many of the earthquakes were microtremors, but detailed response for at least one larger event was obtained. Transfer functions between free-field surface motions and foundation response were generated and compared with a calculated scattering element. Results compared well. A significant reduction in foundation response from the free surface values was observed. The mass and stiffness of the tanks were such that kinematic interaction dominated the SSI effects. Figure 10.12 presents the data [Ishii et al., 1984].

10.2.5.4 Humboldt Bay Nuclear Power Plant

The Humboldt Bay Nuclear Power Plant in northern California has experienced numerous earthquakes over the years. Four events of note are the Ferndale earthquake of June 7, 1975 and the Lost Coast earthquake sequence of April 25, 1992. Accelerometers in place in the free field and on the base of a deeply embedded caisson structure (80 ft) recorded acceleration time histories. Figure 10.13 compares

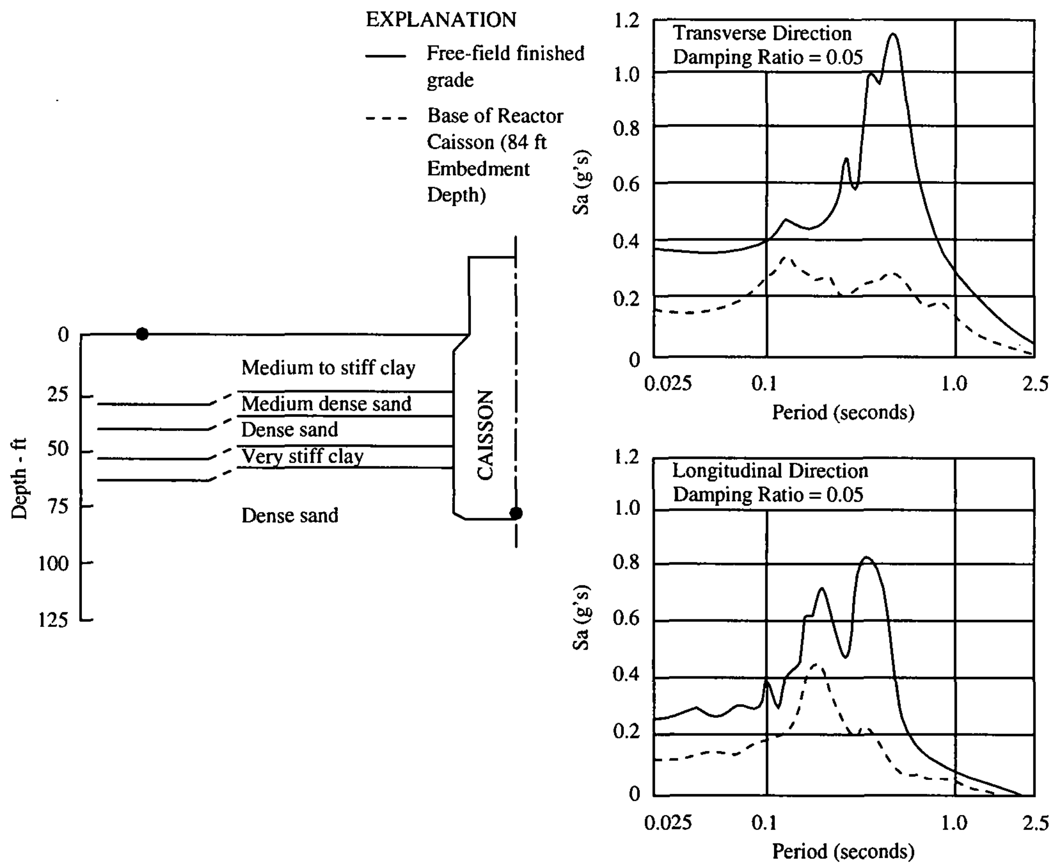


FIGURE 10.13 Comparison of response spectra of accelerograms recorded at finished grade in the free field and at the base of the reactor caisson at the Humboldt Bay plant during the June 6, 1975, Femdale, California earthquake.

TABLE 10.3 Peak Accelerations (Surface/Caisson)

Earthquake	E-W	N-S	Vertical
6/1/75	0.35/0.16g	0.26/0.12g	0.06/0.10g
4/25/92 (11:06 PDT)	0.22/0.14g	0.22/0.11g	0.05/0.08g
4/26/92 (00:14 PDT)	0.25/0.12g	0.23/0.12g	0.05/0.12g
4/26/92 (04: 18 PDT)	0.13/0.07g	0.098/0.057g	0.031/0.037g

free-field surface response spectra with those recorded on the caisson's base for the 1975 event. Table 10.3 shows a comparison of peak accelerations.

For horizontal motions, significant reductions are apparent, i.e., reductions up to 55%. For vertical motions, peak accelerations remained the same or slightly increased. Additional data for vertical motions will undoubtedly illuminate this phenomenon. The major interest for the design and evaluation of structures is horizontal motions, which clearly exhibit reduction with depth. The dominant SSI phenomenon here, as for the LNG tanks, is likely to be kinematic interaction due to the deep embedment of the caisson. Numerous other data exist and are highlighted in other sources, e.g., Johnson and Asfura [1993].

10.2.5.5 Buildings With and Without Basements

A series of buildings in close proximity to each other, with and without basements, subjected to the San Fernando earthquake of 1971 were investigated. Comparing recorded basement motions for buildings

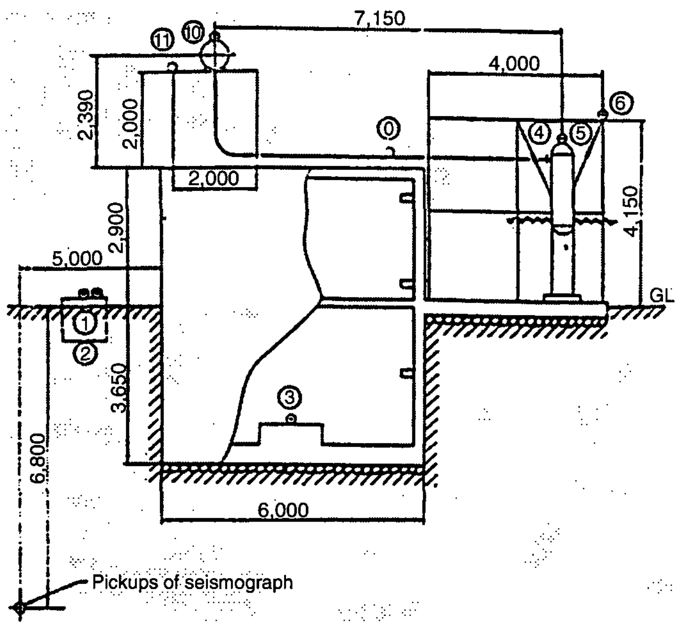


FIGURE 10.14 Cross section through chemical engineering plant model, Chiba Field Station.

with and without embedment documented the effects of kinematic interaction. The results show a definite reduction in motion of embedded foundations compared to surface foundations or free-field surface values. Numerous comparisons are presented by Chang et al. [1985].

10.2.5.6 Model Structures

Two model structures in Japan and the Lotung scale model structures in Taiwan have been instrumented and have recorded ground motions from a number of earthquakes. One model structure is located at the Fukushima Nuclear Power Plant site in northern Japan. Tanaka et al. [1973] report on data recorded in the free field and structure. A reduction in peak acceleration of about 20% is observed from free-field surface motion to foundation. The second model structure is located at Chiba Field Station, Institute of Industrial Science, University of Tokyo. The purpose of the model was to instrument it for earthquakes, record earthquake motions, and investigate variability in ground motion and response. Figure 10.14 [Shibata, 1978; Shibata, 1991] shows a cross section through the scale model structure including components (piping, hanged tank, and vessel) and instrument locations. As of 1978, Shibata shows graphically that the mean response factor for peak acceleration on the foundation compared to the free field is about 0.68, with a coefficient of variation of 0.125 in the horizontal direction, and 0.83, with a coefficient of variation of 0.136 in the vertical direction. A clear reduction in response of the foundation is observed. The third model structure is the Lotung, Taiwan case. Comparing the May 20, 1986 measured free-field surface response with foundation response demonstrates the reduction in free-field motion with depth.

10.2.6 Variation of Ground Motion in a Horizontal Plane

Variations observed in the motions of two points located on a horizontal plane are mainly due to the difference in arrival times of the seismic waves at the two points and to the amplitude and frequency

modification that those seismic waves undergo due to the geotechnical characteristics of the media between the two points. The former is sometimes referred to as a first-order effect and is characterized by an apparent horizontal wave propagation velocity (speed and, in some cases, direction). The latter is often denoted a second-order effect and is characterized by a set of horizontal and vertical ground motion "coherency functions." Understanding of the variation of ground motion over a horizontal extent was significantly advanced with the installation of several dense accelerograph arrays, e.g., SMART 1 and LSST arrays in Taiwan, El Centro and Parkfield in California, and the Chusal differential array in the former U.S.S.R., and the analysis of the recorded data collected from them. Thus, the spatial variation of ground motions over a horizontal plane is characterized by an apparent horizontal wave propagation velocity, which in the absence of detailed site data is generally assumed to be on the order of 2 to 3 km/sec. Complex-valued, frequency-dependent coherency functions have been derived from the recorded data of the differential arrays discussed above. These functions strongly depend on the distance between recording or site locations and on the frequency of the seismic waves. This spatial variation of ground motions is a critical element in the seismic analysis of structures on large foundations and in the analysis of long structures on multiple foundations, such as bridges [Tseng and Penzien, 2000]. For long structures on multiple foundations, accounting for both aspects of horizontal spatial variation of ground motion is essential. For these cases, apparent horizontal wave propagation velocities and coherency functions are used directly in the SSI analysis. For structures supported on large, stiff foundations, recorded data support a "base averaging" effect on the free-field ground motions. That is, certain frequency ranges may be filtered out of the foundation input motion due to the base averaging effect. In the absence of performing detailed SSI analyses incorporating the coherency functions, a simpler approach may be taken, i.e., a filtering of motions. For a 50-m plan dimension foundation, reductions in spectral accelerations of 20% in the > 20-Hz frequency range, of 10% to 15% in the 10- to 20-Hz range, and of 5% in the 5- to 10-Hz range are supported by the data.

10.3 Modeling of the Soil

For soil sites, describing the soil configuration (layering or stratigraphy) and the dynamic material properties of soil is necessary to perform SSI analysis and predict soil–structure response. Determining soil properties to be used in the SSI analysis is the second most uncertain element of the process — the first being specifying the ground motion. Modeling the soil can be visualized in two stages: determining the low strain *in situ* soil profile and associated material properties; and defining the dynamic material behavior of the soil as a function of the induced strains from the earthquake and soil–structure response. In general, dynamic stress–strain behavior of soils is nonlinear, anistropic, elastoplastic, and loading path dependent. It is also dependent on previous loading states and the degree of disturbance to be expected during construction. Practically speaking, all of these effects are not quantifiable in the current state of the art and, hence, contribute to uncertainty in describing soil stress–strain behavior. In a majority of cases, soil is modeled as a linear or equivalent linear viscoelastic medium in SSI analyses for earthquake motion. In some instances, in particular where foundations or footings are founded underwater, it is important to characterize nonlinear behavior of the supporting medium, albeit with substantial uncertainty taken into account. In addition, there have been numerous studies where nonlinear behavior of soil has been analyzed for research objectives. The assumption in this chapter is that the majority of SSI analysis cases of interest to the reader are those where the nonlinear behavior of soil may be treated appropriately as equivalent linear viscoelastic material. In other cases, the reader is referred to sources such as Tseng and Penzien [2000].

A linear viscoelastic material model is defined by three parameters — two elastic constants (frequently shear modulus and Poisson's ratio, although shear modulus and bulk or constrained modulus may be more appropriate) and material damping. The equivalent linear method approximates the nonlinear stress–strain relation with a secant modulus and material damping values selected to be compatible with the average shear strain induced during the motion using an iterative procedure. Requirements for this model are low strain shear modulus and Poisson's ratio (or bulk or constrained modulus), material

damping values, and their variations with strain. The following discussion assumes an equivalent linear viscoelastic material model for soil. Three aspects of developing soil models are field exploration, laboratory tests, and correlation of laboratory and field data. Much of the discussion refers to Woods [1978], which is the most comprehensive documentation of the state of the art at the time of publication. In the ensuing years, a number of authors have published updates to selected aspects of the subject, in particular, measurement of parameters for nonlinear models of soil material behavior, liquefaction, and lateral spreading. Two such publications are CH2M Hill [1991] and Ishihara [1996].

10.3.1 Field Exploration

Field exploration, typically, relies heavily on boring programs, which provide information on the spatial distribution of soil (horizontally and with depth) and produce samples for laboratory analysis. In addition, some dynamic properties are measured *in situ*, for example, shear wave velocity which leads to a value of shear modulus at low strains. Woods [1978] provides a summary of field exploration, in general, and of boring, sampling, and *in-situ* testing, in particular. Low strain shear and compressional wave velocities are typically measured in the field. Various field techniques for measuring *in situ* shear and compressional wave velocities exist, including the seismic refraction survey, seismic cross-hole survey, seismic downhole or uphole survey, and surface wave techniques. The advantages and disadvantages of these techniques are discussed by Woods [1978]. For sites having a relatively uniform soil profile, all of these techniques are appropriate. However, for sites having layers with large velocity contrasts, the most appropriate technique is the cross-hole technique. The cross-hole data are expected to be the most reliable because of better control over and knowledge of the wave path. The downhole technique does not permit as precise resolution because travel times in any layer are averaged over the layer. Most seismic field survey techniques are capable of producing ground motions in the small shearing strain amplitude range (less than 10^{-3}%). Hence, field exploration methods yield the soil profile and estimates of important low strain material properties (shear modulus, Poisson's ratio, water table location).

10.3.1.1 Laboratory Tests

Laboratory tests are used principally to measure dynamic soil properties and their variation with strain: soil shear modulus and material damping. Currently available laboratory testing techniques have been discussed and summarized by Woods [1978]. These techniques include resonant column tests, cyclic triaxial tests, cyclic simple shear tests, and cyclic torsional shear tests. Each test is applicable to different strain ranges. The resonant and torsional shear column tests are capable of measuring dynamic soil properties over a wide range of shear strain (from 10^{-4} to 10^{-2}% or higher). The cyclic triaxial test allows measurement of Young's modulus and damping at large strain (larger than 10^{-2}%).

Typical variations of shear modulus with shear strain for clays, compiled from laboratory test data, are depicted in Figure 10.15. Generally, the modulus reduction curves for gravelly soils and sands are similar. Figure 10.16 shows material damping as a function of shear strain, also for clays. Shear modulus decreases and material damping increases with increasing shear strain levels.

10.3.1.2 Correlation of Laboratory and Field Data

Once shear modulus degradation curves and material damping vs. strain curves have been obtained, it is necessary to correlate laboratory-determined low strain shear modulus values with those *in situ*. Laboratory-measured values of shear moduli at low levels of strain are typically smaller than those measured in the field. Several factors have been found to contribute to lower moduli measured in the laboratory. These factors include effects of sampling disturbance, stress history, and time (aging or period under sustained load). Thus, when the laboratory data are used to estimate *in situ* shear moduli in the field, considerations should be given to these effects. When laboratory data do not extrapolate back to the field data at small strains, one of the approaches used in practice is to scale the laboratory data up to the field data at small strains. This can be done by proportionally increasing all values of laboratory moduli to match the field data at low strain values or by other scaling procedures.

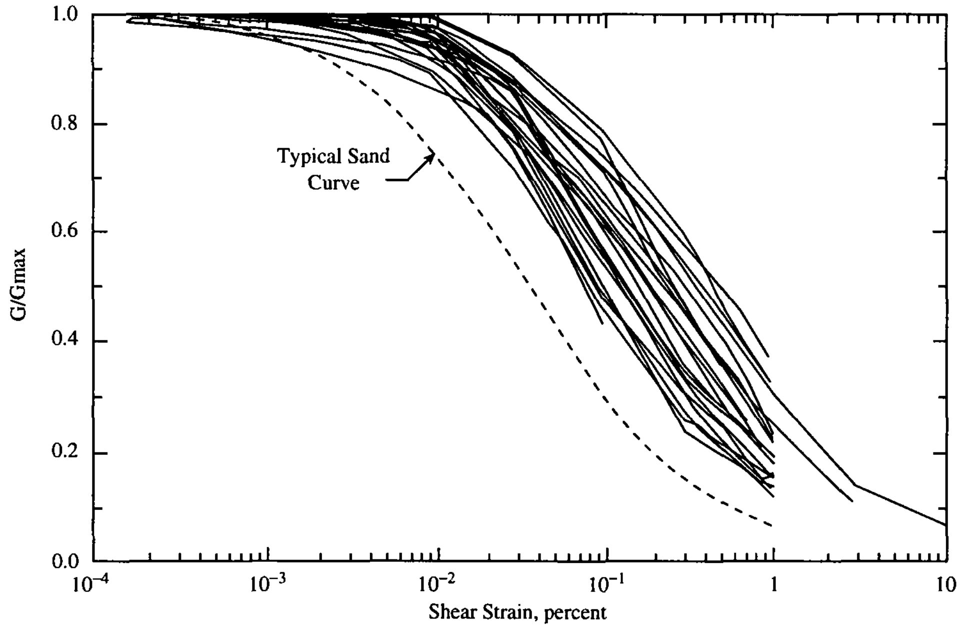


FIGURE 10.15 Normalized modulus reduction relationship for clays with plasticity index between 40 and 80.

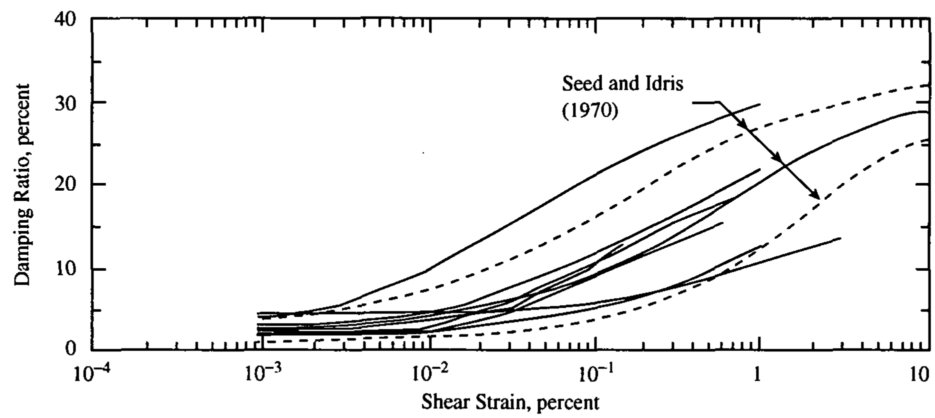


FIGURE 10.16 Strain-dependent damping ratios for clays.

10.3.1.3 Equipment Linear Soil Properties

Given the low-strain soil profile determined from the combination of field and laboratory tests, and the variation of material parameters with strain level, a site response analysis is performed to estimate equivalent linear soil properties for the SSI analysis. The computer program SHAKE [Schnabel et al., 1972] has become an international standard for such analyses.

10.3.1.4 Uncertainty in Modeling Soil/Rock at the Site

There is uncertainty in each aspect of defining and modeling the site soil conditions for SSI analysis purposes. The soil configuration (layer or stratigraphy) is established from the boring program. Even after such a program, some uncertainty exists in the definition of the soil profile. Soils are seldom homogeneous, and they seldom lie in clearly defined horizontal layers — the common assumptions in

SSI analysis. In general, complicated soil systems introduce strong frequency dependence in the site behavior.

Modeling the dynamic stress–strain behavior of the soil is uncertain in two respects. First, modeling soil as a viscoelastic material with parameters selected as a function of average strains is an approximation to its complex behavior. Second, there is uncertainty in defining low strain shear moduli and in defining the variations in shear modulus and observed damping with strain levels, Variability can be observed in reporting test values for a given site and for reported generic test results. The range of coefficients of variation on "stress–strain behavior" estimated by the American Society of Civil Engineers (ASCE) Committee on Reliability of Offshore Structures [ASCE, 1979] is 0.5 to 1.0. Similarly, the ASCE Standard, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary* [1998] recommends implementation of a variation in low-strain shear moduli of 0.5 to 1.0 depending on specific site data acquired and its quality. In the former case, one is dealing with offshore structures with foundations founded underwater, which certainly adds to the uncertainty in the dynamic behavior of soil foundation. In the latter case, variation in shear moduli is intended to account for sources of uncertainty in the SSI analysis process in addition to the uncertainty in soil material behavior. Considering only uncertainty in soil shear modulus itself and the equivalent linear estimates to be used in the SSI analysis, a coefficient of variation of 0.35 to 0.5 on soil shear modulus is estimated. Uncertainty in low strain values is less than uncertainty in values at higher strains.

The Lotung SSI experiment provides a unique opportunity to quantify the uncertainty in estimates of equivalent linear soil properties. The bases for determining the equivalent linear soil properties included field and laboratory tests and the results of forced vibration tests on the structure. The latter responses permitted system identification techniques to be employed to refine estimates of the low strain profile.

Soil property data were extracted from Chang et al. [1985] and are plotted in Figures 10.17 and 10.18. Ten independent sets of data were available and their variability is apparent from the figures. To quantify this variability, the data were evaluated statistically and a weighted coefficient of variation (COV) calculated for soil shear modulus and damping; the weighting factors were soil thickness to a depth of about 100 ft. The resulting COVs for shear modulus and damping were 0.48 and 0.39, respectively. This emphasizes that uncertainty in soil behavior should be taken into account in any SSI analysis.

10.4 Soil–Structure Interaction Analysis

10.4.1 SSI Parameters and Analysis

Several approaches for categorizing SSI analysis methods have been used [Johnson, 1981; Tseng and Penzien, 2000]. Two approaches are **direct methods**, which analyze the soil–structure system in a single step, and the **substructure approach**, which treats the problem in a series of steps, e.g., determination of the foundation input motion and the foundation impedances, modeling of the structure, and the analysis of the coupled system.

For truly nonlinear analysis, the direct method must be employed, since the equations of motion are solved time step by time step, accounting for geometric and material nonlinearities, as appropriate. In the context of the present discussion, it is informative to continue to view the SSI phenomenon in the steps of the substructure approach. Figure 10.1 shows schematically the substructure approach. The key elements not previously discussed follow below.

10.4.1.1 Foundation Input Motion

Section 10.2.3 introduced the concept of foundation input motion which differs from the free-field ground motion in all cases, except for surface foundations subjected to vertically incident waves, provided the spatial variation of the free-field ground motion is taken into account. This is due, first, to the variation of free-field motion with depth in the soil, and second, due to the scattering of waves from the

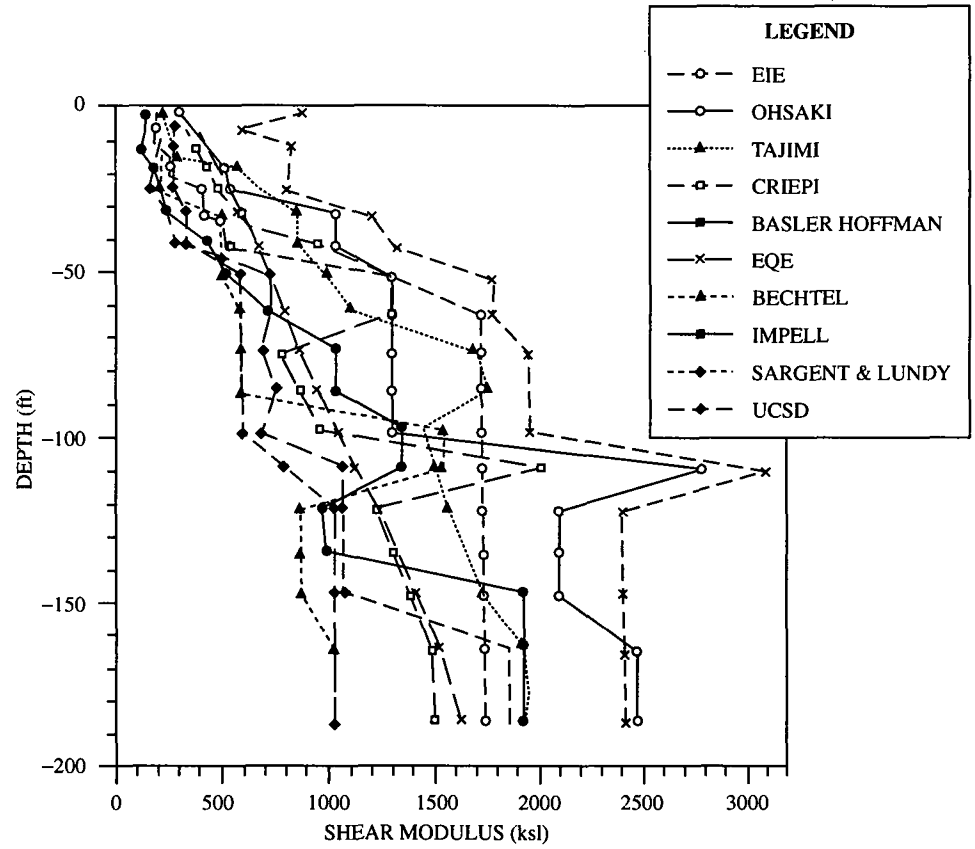


FIGURE 10.17 Variability of equivalent linear shear modulus due to different SSI analyses, earthquake, May 20, 1986, Lotung, Taiwan.

soil-foundation interface, because points on the foundation are constrained to move according to its geometry and stiffness.

10.4.1.2 Foundation Impedances

Foundation impedances describe the force-displacement characteristics of the soil. They depend on the soil configuration and material behavior, the frequency of the excitation, and the geometry of the foundation. In general, for a linear elastic or viscoelastic material and a uniform or horizontally layered soil deposit, each element of the impedance matrix is complex-valued and frequency-dependent. For a rigid foundation, the impedance matrix is 6×6, which relates a resultant set of forces and moments to the six rigid-body degrees of freedom.

10.4.1.3 SSI Analysis

The final step in the substructure approach is the actual analysis. The result of the previous steps — foundation input motion, foundation impedances, and structure models — are combined to solve the equations of motion for the coupled soil–structure system. The entire process is sometimes referred to as a complete interaction analysis, which is separated into two parts: kinematic interaction described by the scattering matrix and inertial interaction comprised of the effects of vibration of the soil and structure.

In terms of SSI parameters, the scattering functions model kinematic interaction and the foundation impedances model inertial interaction. It is these two parameters which are highlighted in the ensuing sections.

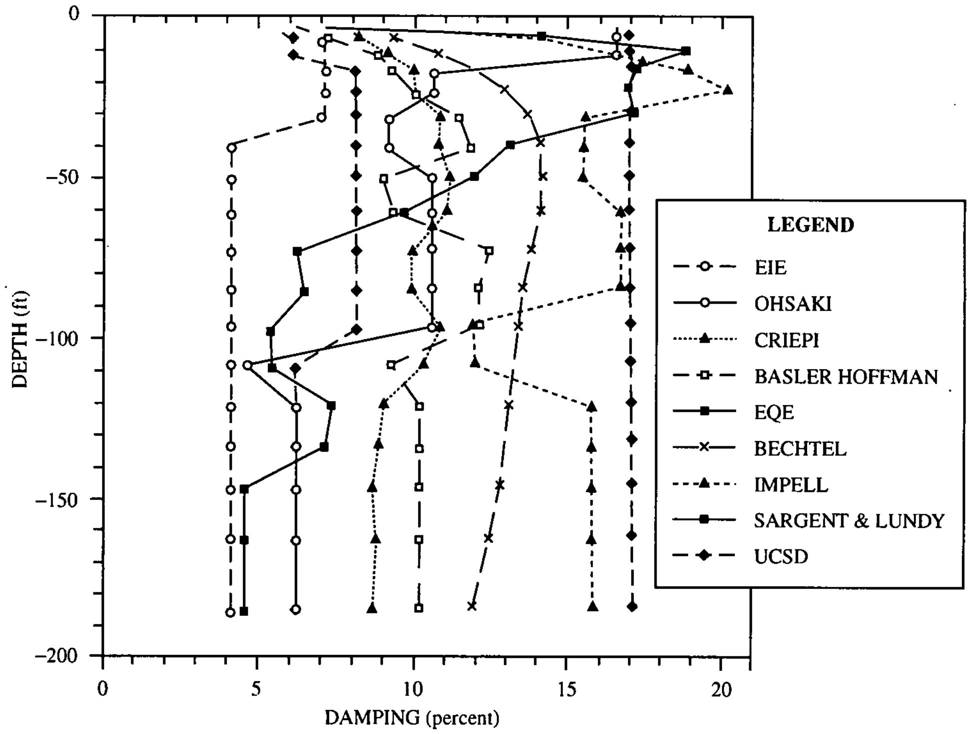


FIGURE 10.18 Variability of equivalent linear damping due to different SSI analyses, earthquake, May 20, 1986, Lotung, Taiwan.

One other point is that the SSI analysis, typically, calculates overall dynamic response of the soil–structure system. The overall structural response is then applied to a detailed structure model as appropriate load cases for structure element and foundation design. In-structure response spectra are delivered to equipment and commodity designers for their seismic qualification. Hence, the SSI analysis is often an intermediate step, albeit an essential one.

10.4.2 Modeling of the Foundation

Three aspects of modeling structure foundations are important: stiffness, embedment, and geometry.

10.4.2.1 Stiffness

The stiffness of a structural foundation is of importance because in almost all SSI analyses, by either direct or substructure methods, foundation stiffness is approximated. Most substructure analysis approaches assume the effective foundation stiffness to be rigid. For direct methods, various representations of foundation stiffness have been used depending on the geometry and other aspects of the structure–foundation system.

Most foundations of the type common to major building structures cannot be considered rigid by themselves. However, structural load-resisting systems, such as shear wall systems, significantly stiffen their foundations. Hence, in many instances, the effective stiffness of the foundation is very high and it may be assumed rigid. One study which has investigated the effects of foundation flexibility on structure response for a complicated nuclear power plant structure of large plan dimension was performed [Johnson and Asfura, 1993]. In this study, the stiffening effect of the structure on the foundation was treated exactly. Even though the structure had nominal plan dimensions of 350 ft by 450 ft, the effect of foundation flexibility on structure response was minimal. The largest effect was on rotational accelerations of the foundation segment as one would expect. Translational accelerations and response spectra,

quantities of more interest, on the foundation and at points in the structure were minimally affected (5%). This conclusion emphasizes the importance of considering the stiffening effect of walls and floors on the foundation when evaluating its effective stiffness.

A counter example is encountered when considering a relatively thin slab supporting massive structurally independent components, such as dry nuclear spent fuel storage casks [Bjorkman et al., 2001]. In the referenced study, sensitivity studies were performed on the dynamic response of dry fuel storage casks and the reinforced concrete pad supporting them. Numerous combinations of number of casks, thickness of pad, and stiffness of soil were considered. In order of significance, the most important parameters affecting cask dynamic response were thickness of the pad, arrangement and number of casks on the pad, and stiffness of the soil.

Hence, practically speaking, for design and evaluation of major structures with stiff load-resisting systems, the assumption of rigid foundation behavior is justified. For predicting the response of a structure to a single recorded event, more refined modeling of the foundation may be required. For structure–foundation conditions of differing characteristics, appropriate sensitivity studies need to be performed to justify all assumptions made, including those related to foundation stiffness modeling.

10.4.2.2 Embedment

Foundations are typically fully embedded and structures are partially embedded. Foundation embedment has a significant effect on SSI. Both the foundation input motion and the foundation impedances differ for an embedded foundation compared to a surface foundation.

Foundation input motion for embedded foundations was discussed extensively in Section 10.2. Foundation impedances comprise a second aspect of foundation modeling. As discussed above, a common and appropriate assumption for many cases is rigid foundation behavior. For a rigid foundation, the impedance matrix describing force-displacement characteristics is, at most, a 6×6 complex-valued, frequency-dependent matrix. The literature, as summarized by Johnson [1981], contains numerous exact and approximate analytical representations of foundation impedances. Also, many analytical/experimental correlations exist with relatively good results reported. Figure 10.19 shows one such example. Figure 10.20 demonstrates the effect of embedment on foundation response for embedded and nonembedded configurations of an assumed rigid foundation. Forced vibration tests were performed. The results clearly demonstrate the effect on inertial interaction of embedment.

10.4.2.3 Geometry

The geometry of major structure foundations can be extremely complicated. Fortunately, many aspects of modeling the foundation geometry are believed to have second-order effects on structure response, in particular, modeling the precise shape of the foundation in detail. However, other overall aspects, such as nonsymmetry, that lead to coupling of horizontal translation and torsion and vertical translation and rocking, can be very important and need to be considered. Modeling the foundation is important with respect to structure response. This is one area where knowledge and experience of the practitioner are invaluable. Complicated foundations must be modeled properly to calculate best estimates of response, i.e., including the important aspects as necessary. Foundation modeling plays a key role in assessing whether two-dimensional models are adequate or three-dimensional models are required.

10.4.3 Modeling of the Structure

10.4.3.1 Linear Dynamic Behavior

Structures for which SSI analysis is to be performed require mathematical models to represent their dynamic characteristics. The required detail of the model is dependent on the complexity of the structure or component being modeled and the end result of the analysis. Structure models are typically of two types: lumped-mass stick models and finite element models. Lumped-mass stick models are characterized by lumped masses defining dynamic degrees of freedom.

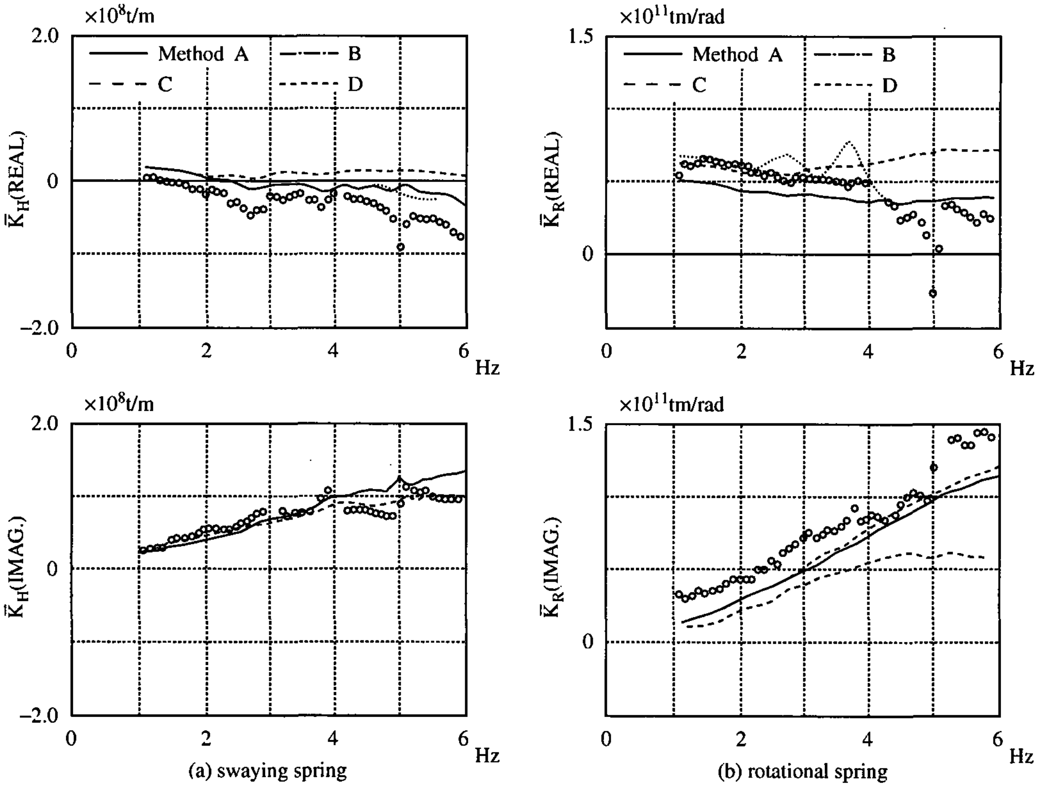


FIGURE 10.19 Analytical/experimental foundation impedances.

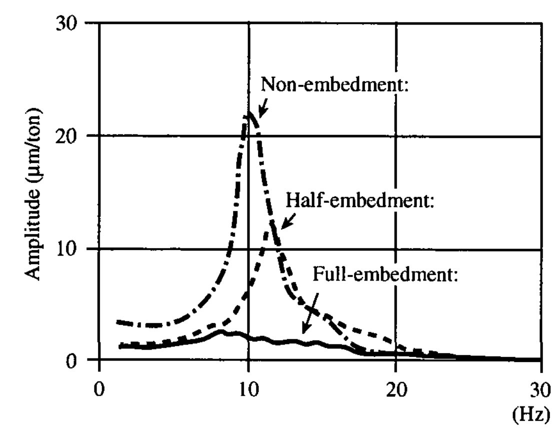


FIGURE 10.20 Comparison of horizontal displacement resonance curves at foundation bottom, forced vibration test.

For buildings, masses are usually lumped at floor slab elevations and simplified assumptions as to diaphragm or floor behavior are made; in particular, floors are frequently assumed to behave rigidly in-plane and, often, for out-of-plane behavior, also. Diaphragm flexibility can be modeled when necessary. Connections between lumped masses are usually stiffness elements whose stiffness values represent columns and/or groups of walls running between the floor slabs. In some instances, an offset is modeled between the center of mass and center of rigidity at each floor elevation to account for coupling between horizontal translations and torsion. Also, lumped-mass stick models are frequently used to model other

regular structures, an example being cylindrical structures (tanks, containment shells, caissons, etc.). Finite element models are used to represent complex structures. They permit a more accurate representation of complicated situations without requiring significant simplifying assumptions. Either model type may be adequate depending on the structural configuration, the detail included in the model, and the simplifying assumptions. Lumped-mass stick models are mathematically simpler than finite element models and usually have a smaller number of degrees of freedom. Lumped-mass stick models take advantage of the judgment of the analyst as to the expected behavior of the structure. Modeling methods and techniques will not be discussed in detail here. The ASCE Standard, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary* [1998] presents many aspects of modeling and a bibliography from which additional information may be obtained.

Modal equivalent models can be used for computational efficiency and to maintain equivalency between the dynamic characteristics of a detailed model in the SSI analysis. A modal equivalent model is comprised of a family of single-degree-of-freedom (SDOF) models, each having the dynamic characteristics of an individual mode of the detailed model (finite element or otherwise). Each SDOF has a mass equal to the modal mass of the detailed model mode, the mass is located at the point above the base to produce the equal moment on the base, and the stiffness elements are selected to reproduce the frequency of the detailed model mode. Each of these SDOFs is exactly equivalent to including the detailed model mode.

One point of note for shear wall structures is that over the last half of the 1980s, testing of shear walls as individual elements and as a portion of a structure assemblage has been performed. One result of these tests is the apparent reduction in stiffness from the linearly calculated values due to small cracks and other phenomena. The ASCE Dynamic Analysis Committee's Working Group on Stiffness of Concrete [1992] evaluated the relevant data and recommended approaches to account for increases and decreases in stiffness previously not treated explicitly. Increases result from items such as increased concrete strength due to aging and achieving minimum specified strengths in a conservative manner.

10.4.3.2 Nonlinear Structure Models

The nonlinear behavior of structures is important in two regards: (1) evaluating the capacity of structural members and the structure itself and (2) estimating the environment (in-structure response spectra and structural displacements) to which equipment and commodities are subjected. Nonlinear structural behavior is characterized by a shift in natural frequencies to lower values, increased energy dissipation, and increased relative displacement between points in the structure. Four factors determine the significance of nonlinear structural behavior to dynamic response.

10.4.3.3 Frequency Content of the Control Motion vs. the Frequencies of the Structure

Consider a rock-founded structure. Depending on the elastic structure frequencies and characteristics of the control motion, the shift in structure frequency due to nonlinear structural response may result in a relatively large reduction in structure response with a substantial reduction in input to equipment when compared to elastic analysis results. If the structure elastic frequency is located on the peak or close to the peak of the control motion's response spectra, the frequency shift will result in a decrease in response. If the elastic frequency is higher than the peak of the control motion's response spectra, the shift in frequency tends to result in increased response. The same type of reduction occurs for structures excited by earthquakes characterized by narrow-band response spectra where the structure elastic frequency tends to coincide closely with the peak of the ground response spectra.

10.4.3.4 Soil–Structure Interaction Effects

The effect of nonlinear structure behavior on in-structure response (forces, accelerations, and response spectra) appears to be significantly less when SSI effects are important at the site. This is principally due to the potential dominating effect of SSI on the response of the soil–structure system. The soil can have a controlling effect on the frequencies of the soil–structure system. Also, if SSI is treated properly, the input motion to the system is filtered such that higher frequency motion is removed, i.e., frequency

content which may not be suppressed by nonlinear structural behavior if the structure were founded on rock. SSI can have a significant effect on the energy dissipation characteristics of the system due to radiation damping and material damping in the soil. Accounting for the effect of the inelastic structural behavior on structure response must be done carefully for soil-founded structures to avoid double-counting of the energy dissipation effects.

10.4.3.5 Degree of Structural Nonlinearity

The degree of structural nonlinearity to be expected and permitted determines the adequacy assessment for the structure and can have a significant impact on in-structure responses. Past reviews of testing conducted on shear walls have indicated that element ductilities of up to about four or five can be accommodated before significant strength degradation begins to occur. However, the allowable achieved ductility for many evaluations will be substantially less. The effect of nonlinear structure behavior on in-structure response spectra has been considered to only a limited extent. In general, increased levels of nonlinearity lead to increasingly reduced in-structure response spectra for a normalized input motion. However, in some instances, higher frequency, i.e., higher than the fundamental frequency, peaks can be amplified. This is an area in which research is currently being performed, which will provide guidance in the future.

10.4.3.6 Magnitude Effects

Earthquake magnitude as it affects the control motion has been discussed earlier. Recall, however, smaller magnitude, close-in earthquakes may be narrow-banded, which are significantly affected by nonlinear behavior as described above.

10.5 Soil–Structure Interaction Response

Numerous guidelines exist defining the required steps to perform SSI analysis for design or evaluation purposes. All methods of analysis are treated. Selected guideline documents are:

ASCE Standard, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary* [1998]
EPRI *Guidelines for Soil–Structure Interaction Analysis* [Tseng and Hadjian, 1991]
Earthquakes and Associated Topics in Relation to Nuclear Power Plant Siting [IAEA, 1991]
Seismic Design and Qualification for Nuclear Power Plants [IAEA, 1992]

The validation of SSI analysis through field data has been difficult due to the lack of well-documented and instrumented structures subjected to earthquakes. Very few cases exist where the data necessary to effectively analyze a soil–structure system have been developed or measured. In addition, for those with data, typically, not all aspects of the SSI phenomenon are important. One case in point is the Lotung one quarter scale model. All appropriate data have been developed and measured but the high stiffness of the structure and the very soft soil conditions eliminate structure vibration as a significant phenomenon. Comparisons of measured and calculated response produced an excellent match. Hence, elements of the SSI analysis process were validated. In addition to the Lotung experiment, there is a recognition in the technical community that additional data appropriate for SSI analysis methods benchmarking and development are necessary. One example was an additional experiment, denoted the Hualien large-scale seismic test for SSI research, constructed in Taiwan in the early 1990s. This experiment intended to eliminate some of the deficiencies of the Lotung experiment by siting a model structure on a stiffer site than Lotung, thereby bringing dynamic structure response into importance. Unfortunately, this experiment has not received the funding necessary to progress. Instrumenting large bridge structures in seismically active areas would be an additional source of data to be pursued.

Many aspects of SSI are well understood and any valid method of analysis is able to reproduce them. Sections 10.2, 10.3, and 10.4 detailed the various aspects of the problem and the current capability to model them. Clearly, uncertainties exist in the process: randomness associated with the earthquake ground motion itself, and the dynamic behavior induced in soil and structures. Even assuming perfect

modeling, randomness in the response of structures and components is unavoidable. Perhaps the best evidence of such randomness is the Chiba Field Station. Shibata [1991] reports the results of 20 years of recorded motion on the model structure; 271 events when taken in total. Analyzing the measured responses of the hung tank yields a coefficient of variation of response of about 0.45 conditional on the horizontal peak ground accelerations of the earthquake. Of course, arguments can be made that many of the events were low amplitude, that grouping events by epicentral area reduced variability, etc. However, the indisputable fact is that significant variability in response of structures and components due to earthquakes is to be expected.

No deterministically exact solution of the SSI problem can be obtained by existing techniques. However, given the free-field ground motion and data concerning the dynamic behavior of soil and structure, reasonable response predictions can be made. These responses can confidently be used in design and evaluation procedures. It is on this premise that analysis guidelines cited above are based. In addition, uncertainties in each of the elements do not necessarily combine in such a fashion as to always increase the uncertainty in the end item of interest (structure response).

Finally, when evaluating the SSI model and the resulting analytical results, one should evaluate intermediate and end results including:

- Fixed-base vs. soil-structure system frequencies
- Amount of soil stiffness softening due to earthquake excitation compared to low strain values
- Effect of soil property variations on dynamic model parameters, such as system frequency, important response parameters, and structural damping values used

For partially embedded structures, response at grade level floors should, as a general rule, be less than the free-field ground surface motion.

Defining Terms

Control motion — Amplitude and frequency characteristics of the input motion.

Control point — Point at which input motion is applied, in an SSI analysis.

Free field — The ground surface in the absence of any structures.

Free-field ground motion — Motion that would essentially exist in the soil at the level of the foundation in the absence of the structure and any excavation.

Uniform hazard spectra (UHS) — Ground response spectra generated so as to have the same probability of exceedance for all structural frequencies of interest.

References

ASCE. (1998). "ASCE Standard, Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE 4–98, American Society of Civil Engineers, Reston, VA.

ASCE Committee on Reliability of Offshore Structures, Subcommittee on Foundation Materials. (1979). *Probability Theory and Reliability Analysis Applied to Geotechnical Engineering of Offshore Structure Foundations*, American Society of Civil Engineers, Reston, VA.

ASCE. (1992). Dynamic Analysis Committee Working Group Report on Stiffness of Concrete, American Society of Civil Engineers, Reston, VA.

Bjorkman, G.S., Moore, D.P., Nolin, J.J., and Thompson, V.J. (2001). "Influence of ISFSI Design Parameters on the Seismic Response of Dry Storage Casks," W01/3, *Proc. Structural Mechanics in Reactor Technology*, August 12–17, Washington, D.C., SMIRT Secretariat, North Carolina State University, Raleigh.

CH2M Hill. (1991). *Proc. NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization*, EPRI NP-7337, vol. 1, Electric Power Research Institute, Palo Alto, CA.

Chang, C.-Y., Power, M.S., Idriss, I.M., Somerville, P.G., Silva, W., and Chen, P.C. (1985). "Engineering Characterization of Ground Motion, Task II: Observational Data on Spatial Variations of Earthquake Ground Motion," NUREG/CR-3805, vol. 3, prepared for the U.S. Nuclear Regulatory Commission, Washington, D.C.

Chang, C.-Y. et al. (1990). "Equivalent Linear Versus Nonlinear Ground Response Analyses at Lotung Seismic Experiment Site," *Proc. Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, CA.

Chang, C.-Y. et al. (1991). "Development of Shear Modulus Reduction Curves Based on Lotung Downhole Ground Motion Data," *Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, MO.

Cramer, C.H. (1991). "Turkey Flat, USA Site Effects Test Area, Report 6 Weak-Motion Test: Observations and Modeling," Technical Report No. 91–1, California Department of Conservation, Division of Mines and Geology, Earthquake Shaking Assessment Project, Sacramento, CA.

EPRI. (1989). *Proc. EPRI/NRC/TPC Workshop on Seismic Soil Structure Interaction Analysis Techniques Using Data from Lotung, Taiwan*, EPRI NP-6154, vols. 1 and 2, Electric Power Research Institute, Palo Alto, CA.

Gazetas, G. and Bianchini, G. (1979). "Field Evaluation of Body and Surface-Wave Soil-Amplification Theories," *Proc. Second U.S. National Conference on Earthquake Engineering*, Electric Power Research Institute, Stanford University, Palo Alto, CA.

Hadjian, A.H. et al. (1991). "A Synthesis of Predictions and Correlation Studies of the Lotung Soil Structure Interaction Experiment," Report No. NP-7307-M, Electric Power Research Institute, Palo Alto, CA.

IAEA. (1991). "Earthquakes and Associated Topics in Relation to Nuclear Power Plant Siting: A Safety Guide," Safety Series No. 50-SG-S1 (Rev. 1), International Atomic Energy Agency, Vienna, Austria.

IAEA. (1992). "Seismic Design and Qualification for Nuclear Power Plants: A Safety Guide," Safety Series No. 50-SG-D15, International Atomic Energy Agency, Vienna, Austria.

Ishihara, K. (1996). *Soil Behavior in Earthquake Geotechnics*, Oxford Engineering Science Series, no. 46, Oxford University Press, London.

Ishii, K., Itoh, T., and Suhara, J. (1984). "Kinematic Interaction of Soil-Structure System Based on Observed Data," *Proc. 8th World Conference on Earthquake Engineering, San Francisco*, vol. 3, Earthquake Engineering Research Center, Berkeley, CA, pp. 1017–1024.

Johnson, J.J. (1981). *Soil Structure Interaction: The Status of Current Analysis Methods and Research*," Lawrence Livermore National Laboratory (LLNL), UCRL-53011, NUREGICR-1780, prepared for the U.S. Nuclear Regulatory Commission, Washington, D.C.

Johnson, J.J. and Asfura, A.P. (1993). "Soil Structure Interaction (SSI): Observations, Data, and Correlative Analysis," in *Developments in Dynamic Soil-Structure Interaction*, P. Gulkan and R.W. Clough, eds., Kluwer Academic Publishers, Dordrecht, pp. 219–258.

Johnson, J.J. and Chang, C.-Y. (1991). "State of the Art Review of Seismic Input and Soil-Structure Interaction," Appendix E in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Rev. l)*, EPRI NP-6041-SL, Electric Power Research Institute, Palo Alto, CA.

Johnson, J.J., Maslenikov, O.R., Mraz, M.J., and Udaka, T. (1989). "Analysis of Large-Scale Containment Model in Lotung; Taiwan: Forced Vibration and Earthquake Response Analysis and Comparison," *Proc. EPRI/NRC/TPC Workshop on Seismic Soil Structure Interaction Analysis Techniques Using Data From Lotung, Taiwan*, EPRI NP-6154, vols. 1 and 2, Electric Power Research Institute, Palo Alto, CA, pp. 13-1–13-44.

Kudo, K., Shima, E., and Sakane, M. (1988). "Digital Strong Motion Accelerograph Array in Ashigara Valley — Seismological and Engineering Prospects of Strong Motion Observations," *Proc. 9th World Conference on Earthquake Engineering*, vol. 8, Earthquake Engineering Research Center, Berkeley, CA, pp. 119–124.

Midorikawa, S. (1992). "A Statistical Analysis of Submitted Predictions for the Ashigara Valley Blind Prediction Test" (Subcommittee for the Prediction Criteria of the Ashigara Valley Blind Prediction Test), *Proc. International Symposium on the Effects of Surface Geology on Seismic Motion*, Odawara, Japan.

Newmark, N.M. and Hall, W.J. (1978). *Development of Criteria for Seismic Review of Selected Nuclear Power Plants*, NUREG/CR-0098, prepared for the U.S. Nuclear Regulatory Commission, Washington, D.C.

Proceedings of the International Symposium on the Effects of Surface Geology on Seismic Motions. (1992). Odawara, Japan.

Schnabel, P.B., Lysmer, J., and Seed, H.B. (1972). *SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites*, Report No. EERC 72–12, Earthquake Engineering Research Center, University of California, Berkeley.

Seale, S.H. and Archuleta, R.J. (1991). "Analysis of Site Effects at the Games Valley Downhole Array Near the San Jacinto Fault," *Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, Paper No. 8.13, St. Louis, MO.

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

Shibata, H. (1978). "On the Reliability Analysis for Structural Design Including Pipings and Equipment," presented at the Seminar on Probabilistic Seismic Analysis of Nuclear Power Plants, January 16–19, Berlin.

Shibata, H. (1991). "Uncertainty in Earthquake Engineering in Relation to Critical Facilities," *Bulletin of Earthquake Resistant Structure Research Center*, Institute of Industrial Science, University of Tokyo, No. 24, 93–104.

Tanaka, T. et al. (1973). "Observations and Analysis of Underground Earthquake Motions," *Proc. 5th World Conference on Earthquake Engineering*, Rome, vol. 1, pp. 658–667.

Tseng, W.S. and Hadjian, A.H. (1991). "Guidelines for Soil Structure Interaction Analysis," EPRI NP-7395, Electric Power Research Institute, Palo Alto, CA.

Tseng, W.S. and Penzien, J. (2000). "Soil-Foundation-Structure Interaction," in *Bridge Engineering Handbook*, W.F. Chen and L. Duan, eds., CRC Press, Boca Raton, FL, pp. 42-1–42-52.

U.S. Atomic Energy Commission. (1973). "Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants," Rev. 1, U.S. Atomic Energy Commission, Washington, D.C.

Wolf, J.P. and Song, C. (2002). "Some Cornerstones of Dynamic Soil-Structure Interaction," *Eng. Struct.*, 24, 13–28.

Woods, R.D. (1978). "Measurement of Dynamic Soil Properties," *Proc. ASCE Specialty Conference on Earthquake Engineering and Soil Dynamics*, vol. 1, Pasadena, CA, pp. 91–178.

Yanev, P.I., Moore, T.A., and Blume, J.A. (1979). "Fukushima Nuclear Power Station, Effect and Implications of the June 12, 1978, Miyagi-ken-Oki, Japan, Earthquake," prepared by URS/John A. Blume & Associates, Engineers, San Francisco.