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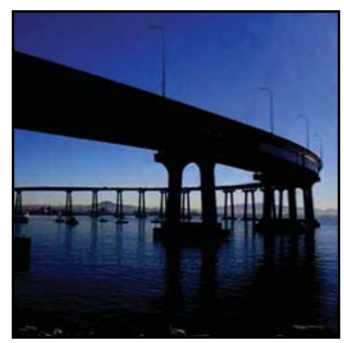
Bridge Foundations: Modeling Large Pile Groups and Caissons for Seismic Design

by
Ignatius Po Lam, Hubert Law and Geoffrey R. Martin
(Coordinating Author)



Technical Report MCEER-07-0018

December 1, 2007

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Preface

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

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

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

The Center's Highway Project develops improved seismic design, evaluation, and retrofit methodologies and strategies for new and existing bridges and other highway structures, and for assessing the seismic performance of highway systems. The FHWA has sponsored three major contracts with MCEER under the Highway Project, two of which were initiated in 1992 and the third in 1998.

Of the two 1992 studies, one performed a series of tasks intended to improve seismic design practices for new highway bridges, tunnels, and retaining structures (MCEER Project 112). The other study focused on methodologies and approaches for assessing and improving the seismic performance of existing "typical" highway bridges and other highway system components including tunnels, retaining structures, slopes, culverts, and pavements (MCEER Project 106). These studies were conducted to:

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

The 1998 study, “Seismic Vulnerability of the Highway System” (FHWA Contract DTFH61-98-C-00094; known as MCEER Project 094), was initiated with the objective of performing studies to improve the seismic performance of bridge types not covered under Projects 106 or 112, and to provide extensions to system performance assessments for highway systems. Specific subjects covered under Project 094 include:

- development of formal loss estimation technologies and methodologies for highway systems;
- analysis, design, detailing, and retrofitting technologies for special bridges, including those with flexible superstructures (e.g., trusses), those supported by steel tower substructures, and cable-supported bridges (e.g., suspension and cable-stayed bridges);
- seismic response modification device technologies (e.g., hysteretic dampers, isolation bearings); and
- soil behavior, foundation behavior, and ground motion studies for large bridges.

In addition, Project 094 includes a series of special studies, addressing topics that range from non-destructive assessment of retrofitted bridge components to supporting studies intended to assist in educating the bridge engineering profession on the implementation of new seismic design and retrofitting strategies.

Current FHWA-sponsored research on large pile groups and caissons is synthesized and summarized into this design-oriented report on the seismic design of bridge foundations for long span bridges. The information is suitable for both retrofit and design applications. Characterization of ground motions for analyses, including a discussion on methods of generating spectrum compatible time histories, spatial variations of ground motion, and effects of local soil conditions on site response, is presented. Modeling approaches for large pile groups focusing on the use of the sub-structuring approach are discussed. The use of p-y curves and associated pile groups, development of linearized Winkler springs, and the assembly of pile group stiffness matrices is discussed in detail and illustrated with examples. Modeling approaches such as the linear stiffness matrix, nonlinear lumped springs and nonlinear distributed springs are discussed for large caissons, which can be employed as foundation piers in many over-water bridges. Case histories of the San Francisco-Oakland Bay Bridge East Span Project and the Tacoma Narrows Bridge are presented to illustrate the modeling approaches.

ABSTRACT

The report synthesizes and summarizes current FHWA-sponsored bridge research and related technical reports on the seismic design of large pile groups and caissons into one design-oriented guideline report on seismic design, suitable for applications to the retrofit of existing long span bridges, or for new design.

The characterization of ground motions for seismic soil-foundation-structure interaction analyses is an important component of the foundation modeling process, and is discussed in detail. Discussion includes methods for generating spectrum compatible time histories, spatial variations of ground motion and effects of local soil conditions on site response.

Modeling approaches for large pile groups focus on the use of the substructuring approach, the use of p-y curves and associated pile group effects to develop linearized Winkler springs and the assembly of pile group stiffness matrices. The analysis approach is illustrated with several case history examples, including the San Francisco-Oakland Bay Bridge East Span project. The complexity of kinematic soil-pile interaction as related to modification of input ground motions is also described.

Modeling approaches for large caisson foundations often employed as piers for large over-water bridges are discussed in detail, with an emphasis on the use of linear stiffness matrices, and nonlinear lumped spring and nonlinear distributed spring approaches. The influence of kinematic interaction on input ground motions is also described. Two case history examples are used to illustrate the modeling approaches, including one used for the new Tacoma Narrows Bridge.

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SECTION 1:
INTRODUCTION

1.1 Background

The modeling guidelines presented in this report synthesize and summarize MCEER research tasks related to the seismic design of foundations for long-span bridges, carried out between 2001 and 2005. The research was sponsored by the Federal Highway Administration (Contract No. DTFH61-98-C-00094) under Task 094-C-2.5. The objective of this Task was to synthesize and summarize research conducted on large pile groups and caissons into one design-oriented guideline report on seismic design, suitable for applications to the retrofit of existing long span bridges or for new design. The report encompasses MCEER Task Reports C-2.2 (Effect of Pile Spacing on the Lateral Stiffness of Pile Groups, Dodds and Martin (2007)) and C-2.3 (Dynamic Analysis and Modeling of Wave Scattering Effects for Pile Groups and Caissons, Lam et al. (2007)) The report also summarizes studies performed during earlier FHWA sponsored MCEER research related to conventional or "ordinary" bridges, specifically Task E-4.10 (Soil-Structure Interaction of Bridges for Seismic Analyses, Lam and Law (2000)). Many of the case histories presented in the report to illustrate modeling approaches, draw on the experience of Earth Mechanics, Inc. in addressing project modeling needs for the California retrofit program for toll bridges.

The report complements Chapter 6 (Geotechnical Modeling and Capacity Assessment) of the FHWA Seismic Retrofit Manual (Part 1 – Bridges) published in 2006, and is also designed to address foundation modeling aspects of the Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges published by the MCEER under Task 094-C-1.1 (Ho et al., 2006).

1.2 Scope of Report

For long span bridges involving large pile groups or caisson foundations, time history seismic response analyses are often conducted. Such analyses allow the incorporation of soil-foundation-structure interaction effects and the potential influence of ground motion incoherence. The characterization of ground motions for analyses is described in Section 2 and is an important component of the modeling process. This section is largely based on an MCEER research report by Lam and Law (2000). The discussion includes methods of generating spectrum compatible time histories, spatial variations of ground motion, and effects of local soil conditions on site response.

Section 3 addresses modeling approaches for large pile groups focusing on the use of the substructuring approach. The use of p-y curves and associated pile group effects to develop linearized Winkler springs, and the assembly of pile group stiffness matrices is discussed in detail, and illustrated with examples. The complexity of kinematic soil-pile interaction as related to the potential modification of input ground motions for structural analyses is also described. Finally, a comprehensive example based on studies for the new San Francisco – Oakland Bay Bridge East Span Project, is presented.

Large caissons are employed as foundation piers for many large over-water bridges. Caisson modeling approaches are described in Section 4 including linear stiffness matrix, nonlinear lumped spring and nonlinear distributed springs approaches. The issue of kinematic interaction on input ground motions is also addressed. Two case history examples including a comprehensive description of the approach used for the new Tacoma Narrows Bridge, are also presented.

SECTION 2:
CHARACTERIZATION OF GROUND MOTIONS FOR ANALYSES

2.1 Overview

The AASHTO Standard Specifications for Highway Bridges (AASHTO, 2006) and the Bridge Seismic Retrofitting Manual Part I (FHWA 2006), apply primarily to the design of so called "ordinary" Highway Bridge with spans less than 500 ft. For such bridges, seismic design methods described in the above publications utilize acceleration response spectra defined by uniform risk probabilistic hazard maps and the site soil conditions at the site. Design methods primarily use modal superposition analysis methods or displacement based pushover analyses. However, for long span bridges of greater lengths involving large pile groups or caisson foundations, linear or nonlinear time history seismic response analyses are often conducted to determine bridge response. The advantage of such analyses is that they allow (a) the incorporation of soil-foundation-structure interaction effects (b) the use of three component time histories (c) the potential effects of incoherence of input motions of foundation supports to be considered and (d) the evaluation of load combinations in structural members on a time dependent basis.

The determination of time histories for analyses entail a number of steps:

(a) the determination of a design firm ground or rock outcrop response spectrum (representative of site conditions at the depth of the bridge foundations). If the importance of the bridge justifies it, a site specific probabilistic seismic hazard analysis may be performed for this purpose,
(b) the determination of representative recorded acceleration time histories based on the tectonic/conditions and risk levels at the site adopted for design,
(c) adjustments to the time histories using spectral matching techniques,
(d) consideration of potential spatial variations or incoherence in firm ground or rock motions at each foundation location, due to wave passage and wave scattering effects (Abrahamson, 1992), and
(e) Site specific one dimensional site response analyses to determine depth varying ground motions at each foundation location

It is generally recognized that the time history analysis is a form of stochastic process, and that using a single earthquake time history does not yield a statistical mean. One needs to examine a suite of earthquakes to obtain statistically stable mean response to form the basis for a design decision. The consensus on this subject is that peak structural response should be used when three sets of time histories are used in the analyses. To use an average of structural responses, a minimum of seven sets of time histories that are matched to the same spectrum must be considered. The peak response from three sets of time histories has been the design basis for the new toll bridges in California, except the East Span San Francisco – Oakland Bay Bridge, which adopted the peak response from six sets of time histories.

Comments on the various aspects of the above steps are given below, together with case history examples.

2.2 Spectrum Matching

The current state of practice relies on matching of a target response spectrum to ensure a broad range of frequency is included in the ground motion time history generated for design. In this case, a reference ground motion time history (usually actual earthquake record) is chosen as a seed motion and is gradually modified through an interactive process so that the response spectrum and the modified time history is compatible with the target spectrum. The reason for this is to minimize the number of sets of time histories for analysis.

Various methods have been developed to perform the spectrum matching. A commonly used method adjusts the Fourier amplitude spectrum based on the ratio of the target response spectrum to the time history response spectrum while keeping the Fourier phase of the reference history fixed. An alternative approach for spectral matching adjusts the time history in the time domain by adding wavelets to the reference time history. A formal optimization procedure for this type of time domain spectral matching was first proposed by Kaul (1978) and was extended to simultaneous matching at multiple damping values by Lilhanand and Tseng (1987, 1988). While this procedure is more complicated than the frequency domain, it has good convergence properties.

Figures 2.1 through 2.3 show an example of spectral matching for the fault normal component of motion obtained from the 1940 Imperial Valley earthquake recorded at El Centro. Figure 2.1 is the initial time history of the original ground motion and Figure 2.2 is the modified time history after spectral matching. The comparison of response spectra from the original and modified motions is shown in Figure 2.3.

Other than matching of response spectrum for each of the ground motion components developed for structural designs, checking of cross-correlation of the two horizontal orthogonal directions is also important to ensure no deficiency in shaking level at any rotated axis. The concept of cross-correlation is derived from digital signal processing to examine dissimilarity of two signals. When the ground motions of two orthogonal directions are dissimilar, the chance of adding up or canceling each other is small after rotating the axes, and therefore this set of ground motions is likely to excite the structure at any oblique angle with reasonable expected energy. For example, if time history analysis of a structure is conducted with two orthogonal components where the X component motion is identical to the Y component motion, shaking level along the direction +45° from the X axis would be $\sqrt{2}$ times higher than that along the principal axis. On the other hand, the direction -45° from the X axis would not be excited at all. Clearly, this set of input ground motions is not robust for design as the time history analysis would fail to examine the structural elements whose behavior is important along or about the direction -45° from the X axis.

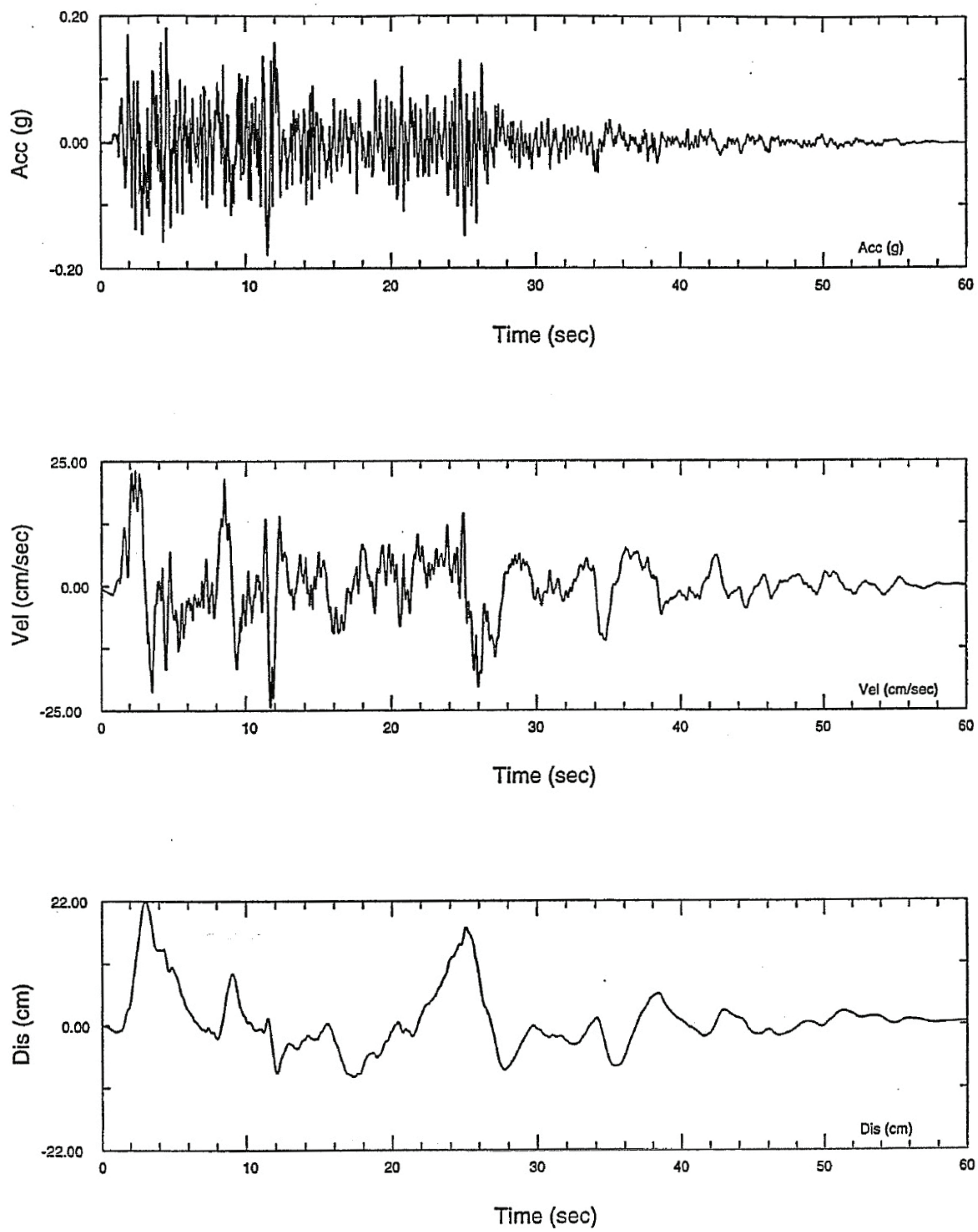


Figure 2-1 Initial Time History for Fault Normal Component of El Centro Record from 1940 Imperial Valley Earthquake

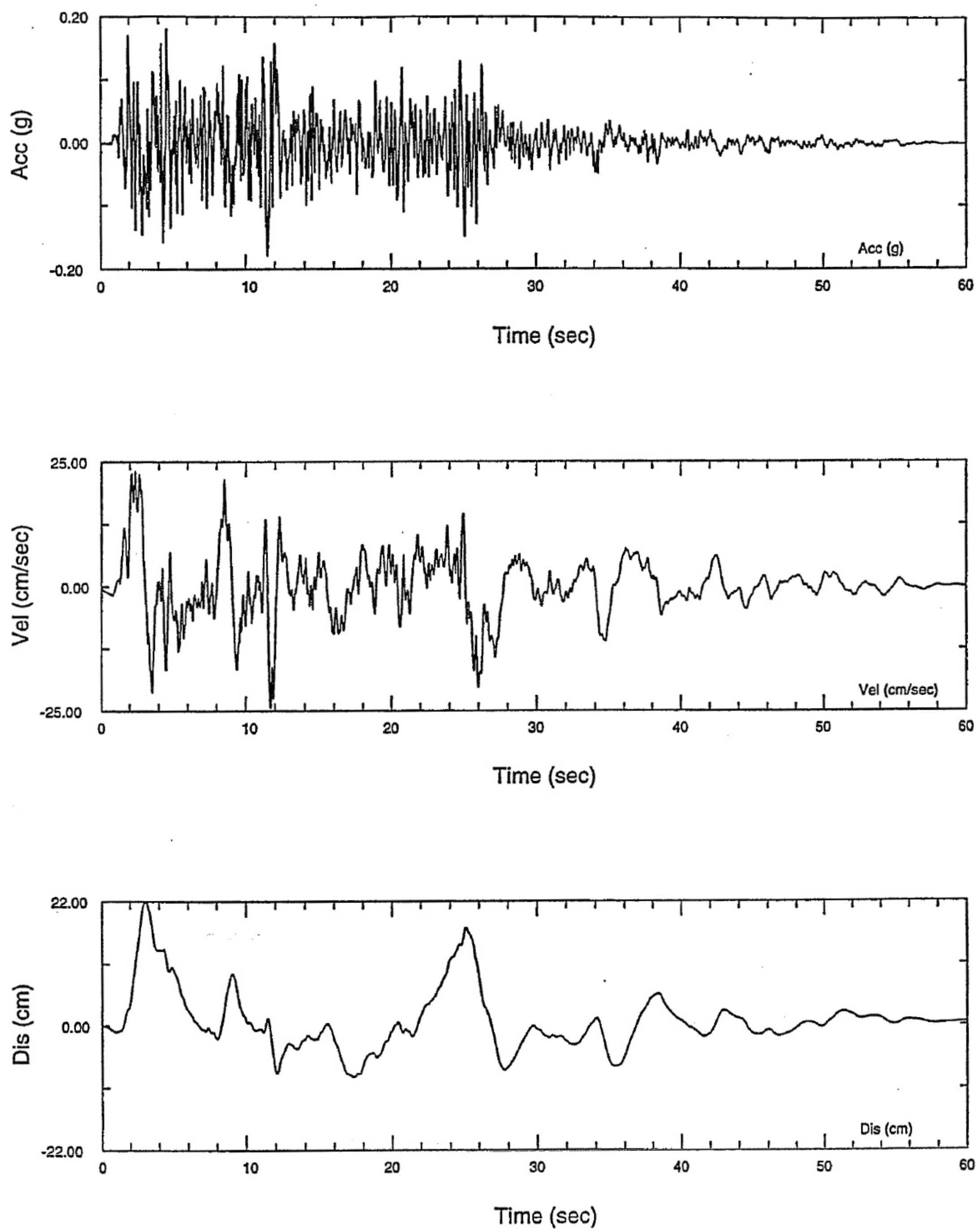


Figure 2-2 Modified Motion after Spectrum Matching

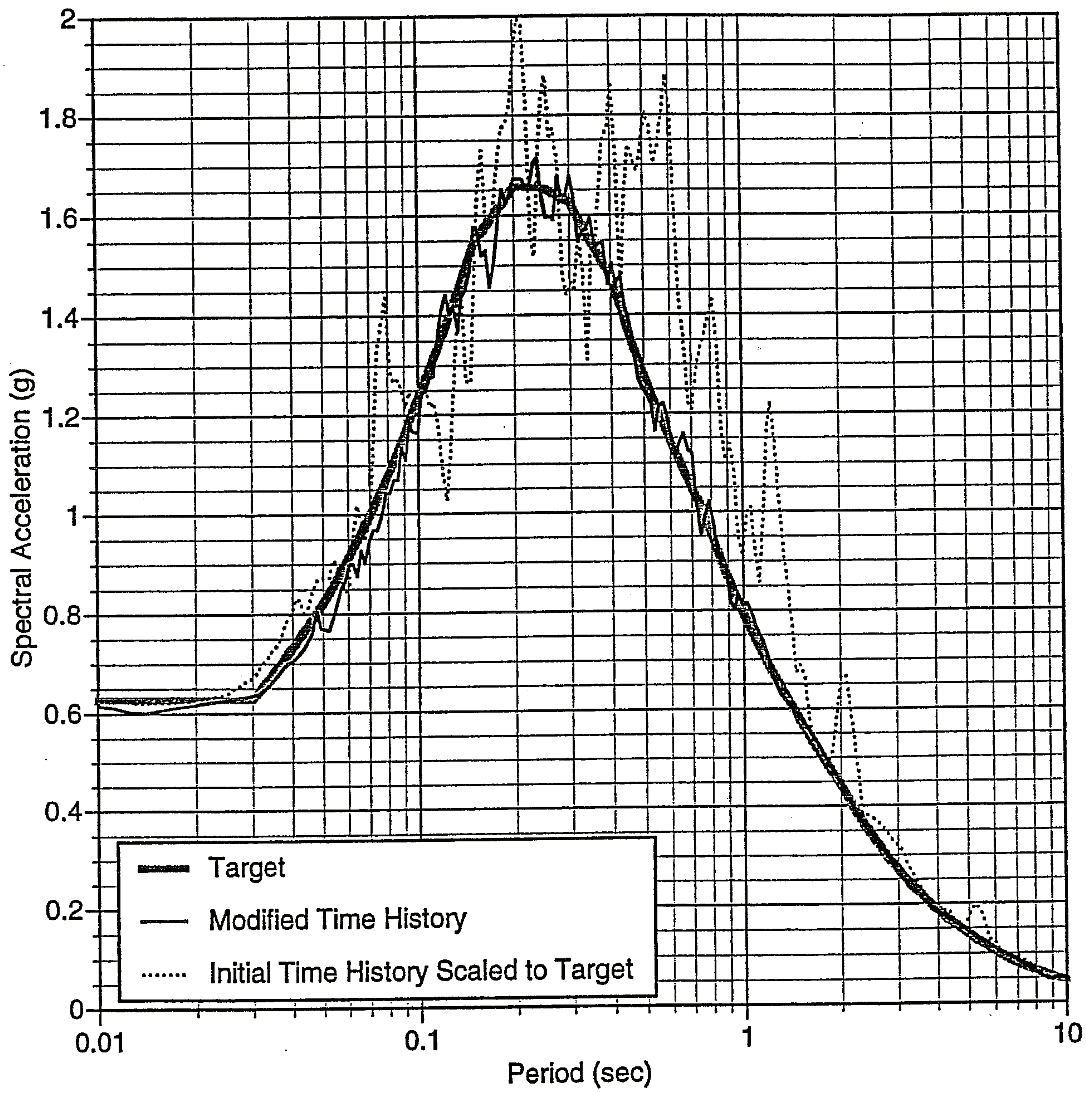


Figure 2-3 Comparison of 5% Damped Response Spectra for the Modified Fault Normal Component and Initial Time History Scaled to the Target PGA

A better representation of shaking level at any direction may be examined by computing spectral accelerations for the periods of interest along the rotated axis. This can be done by first rotating the motion in 360º angles at say every 10º and then computing spectral acceleration for selected periods. Figure 2.4 shows polar plots of spectral acceleration at each rotated azimuth from two-component spectrum compatible motions developed for bridge design. A circular pattern from this polar plot indicates equal shaking intensity for all directions; deficiency of shaking intensity in any direction can be seen on the plot as substantial deviation from a circular pattern.

2.3 Spatial Variation of Rock Motions

Ground motions can vary spatially on a local scale due to scattering and complex wave propagation, and this local variation can be important for long structures such as bridges and pipelines that extend over considerable distance. The spatial variability or incoherence is caused by a number of factors, such as:

1. Wave Passage Effect: nonvertical waves reach different positions on the ground surface at different times causing a time shift between the motions at those locations
2. Extended Source Effect: mixing of wave types and source directions due to differences in the relative geometry of the source and the site
3. Ray Path Effect: scattering of seismic wave by heterogeneity of earth along the travel path causing different waves to arrive at different locations at different times
4. Attenuation Effect: variable distance from the different locations to the seismic source
5. Local Site Effect: variable soil conditions produce different motions at the ground surface

The spatial variation of ground motions at different locations can be characterized by cross covariance in time domain representation or coherency in frequency domain representation.

Considering two points j and k with their acceleration time histories $a_j(t)$ and $a_k(t)$, respectively, the cross covariance is defined as

$$C_{jk}(\tau) = \sum_{i=1}^{N} a_j(t_i) a_k(t_i + \tau) \tag{2.1}$$

where τ is a time increment and N is the number of time samples. Coherency of the same two motions can be described in the frequency domain as

$$\gamma_{jk}(\omega) = \frac{S_{jk}(\omega)}{\sqrt{S_{jj}(\omega) S_{kk}(\omega)}} \tag{2.2}$$

where $S_{jk}(\omega)$ is the complex-valued cross power spectral density function to the two accelerations, $S_{jj}(\omega)$ and $S_{kk}(\omega)$ are real-valued power spectral density functions. The coherency function describes the degree of positive or negative correlation between the amplitudes and phase angles of two time histories at each of their component frequencies. A value of 1 indicates full coherence while a value of zero indicates complete incoherence.

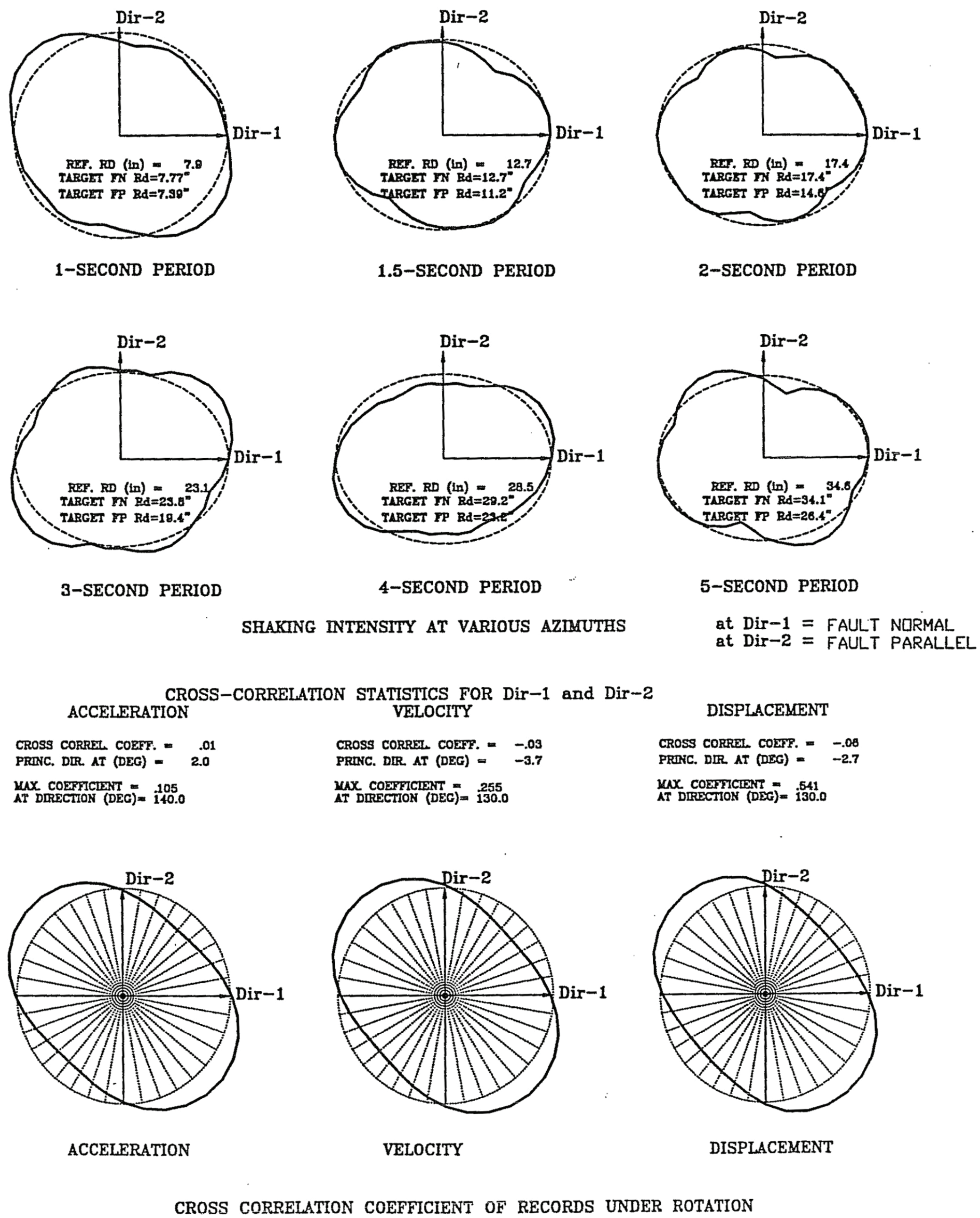


Figure 2-4 Checking of Shaking Intensity for Rotated Axes

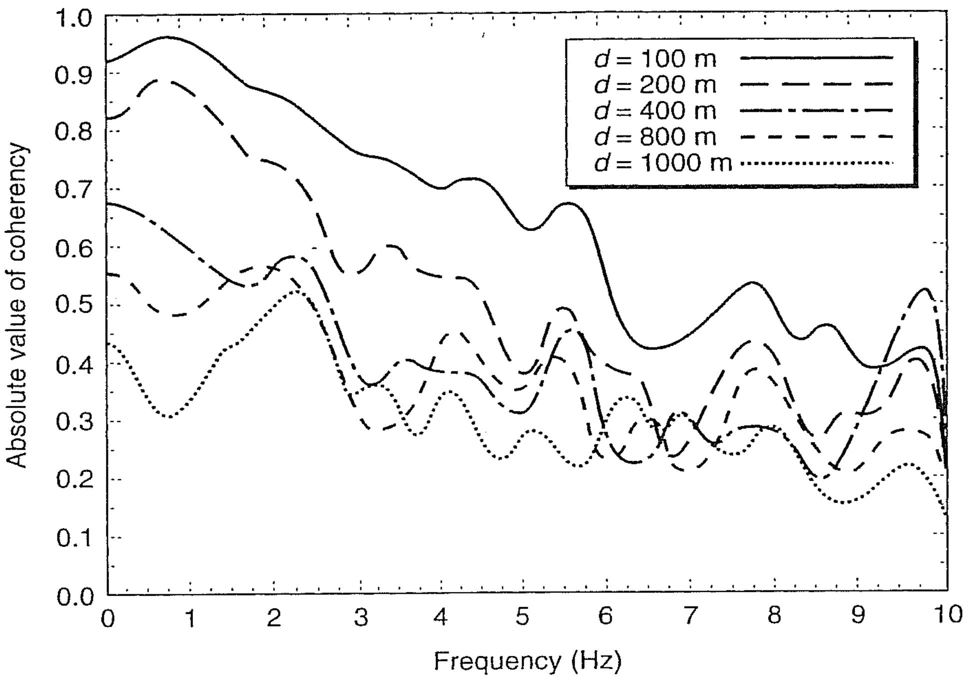
SMART-1 recording array at Lotung, Taiwan has provided numerous ground motion records which show that coherency decreases with increasing distance between measuring points and with increasing frequency. This is illustrated in Figure 2.5. Other measurements from elsewhere in the U.S and Japan show a similar trend, suggesting that the coherence functions may be applicable to other parts of the world as well. Detailed discussion on generation of spatial incoherent motion is described by Abrahamson et al., 1992. The spatial variation of ground motions has been implemented in the recent California seismic retrofit program as well as in the new design of California toll bridges. In these projects, the empirical coherency function as obtained from the dense arrays provides a basis for developing incoherent motions at different bridge supports.

2.4 Effects of Local Site Conditions

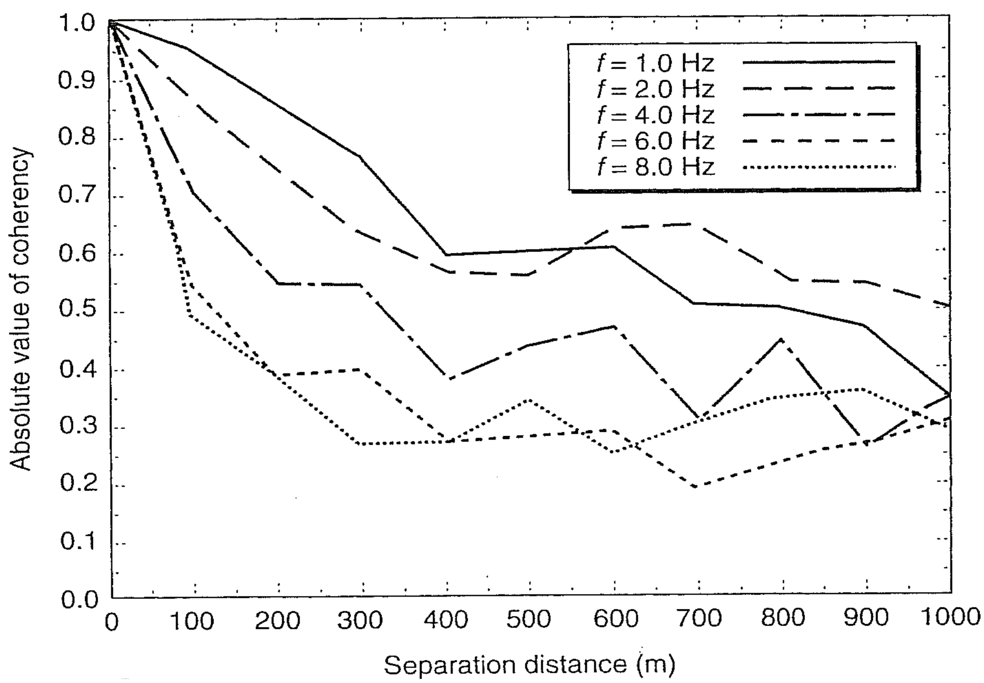
Theoretical evidence as well as actual measurements show that local site conditions profoundly affect the ground motion characteristics in terms of amplitude, frequency content, and duration. As these factors play an important role in earthquake resistant design, it is important to account for the effects of local site conditions.

The influence of local site conditions can be demonstrated analytically using a simple site response analysis. Two soil deposits with an identical depth are considered with a soil deposit at one site being stiffer than the other. Assuming linearly elastic behavior for soil, the amplifications as a function of frequency are shown in Figure 2-6. The softer site (site A) amplifies bedrock motion at a low frequency range while the stiffer site (site B) amplifies high frequency rock motion. Since earthquakes produce bedrock motion over a range of frequencies, some components of the bedrock motion will be amplified more than others.

The effects of local soil conditions are also evident in interpretation of strong motion data from many instrumented sites. For example, recordings of ground motion at several locations in San Francisco during the 1957 earthquake show remarkable variations in ground motion characteristics in terms of peak horizontal acceleration and response spectra along a 4-mile stretch through the city, as shown in Figure 2-7. While the rock outcrop motions are very similar, the amplitude and frequency content of the surface motions are quite different at the sites underlain by thick soil deposits. Similar effects have been observed in other earthquakes.



(a)



(b)

Figure 2-5 Measured Decay of Coherency with Increasing Frequency and Separation Distance for M = 6.9 Event at Hypocentral Depth of 30.6 Km and Epicentral Distance of 116.6 Km from SMART-1 Dense Array at Lotung, Taiwan (Source: Kramer, 1996)

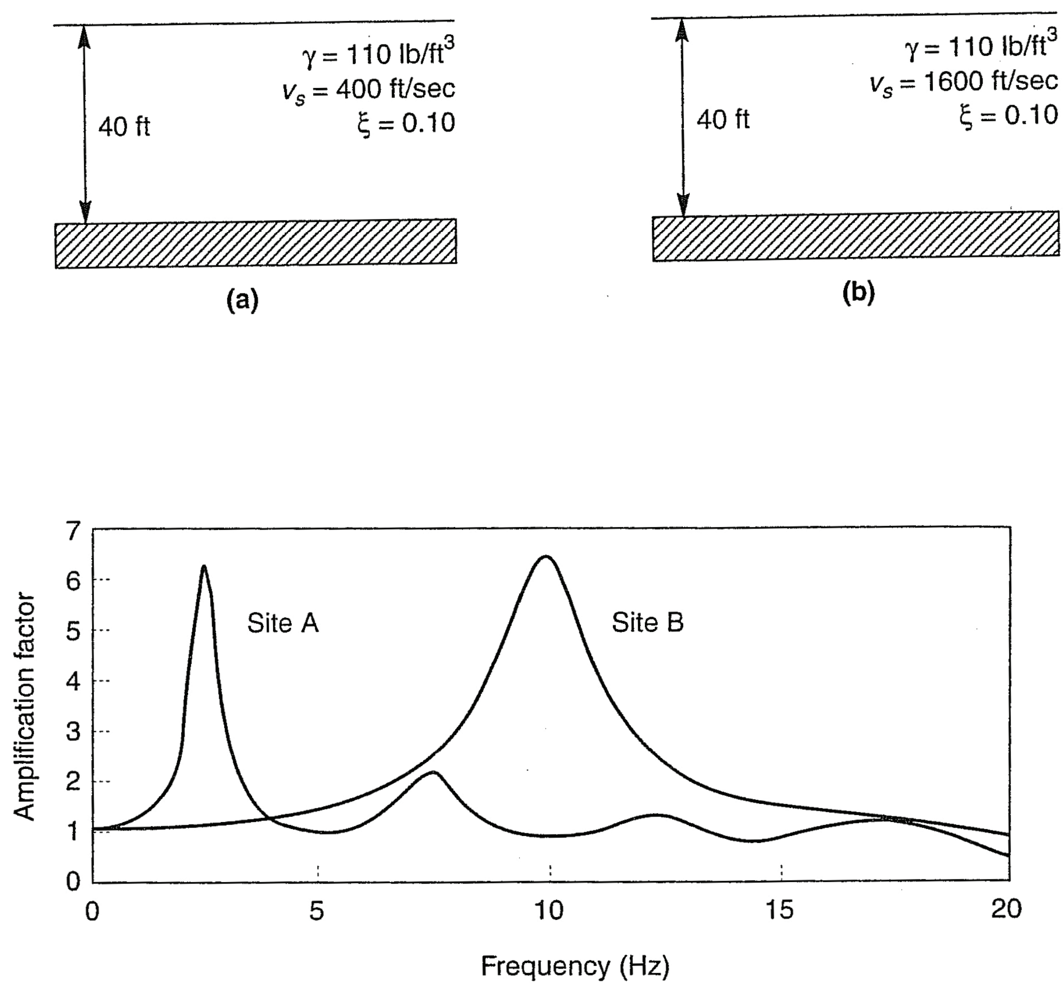


Figure 2-6 Comparison of Amplification for Two Soil Deposits (Source: Kramer, 1996)

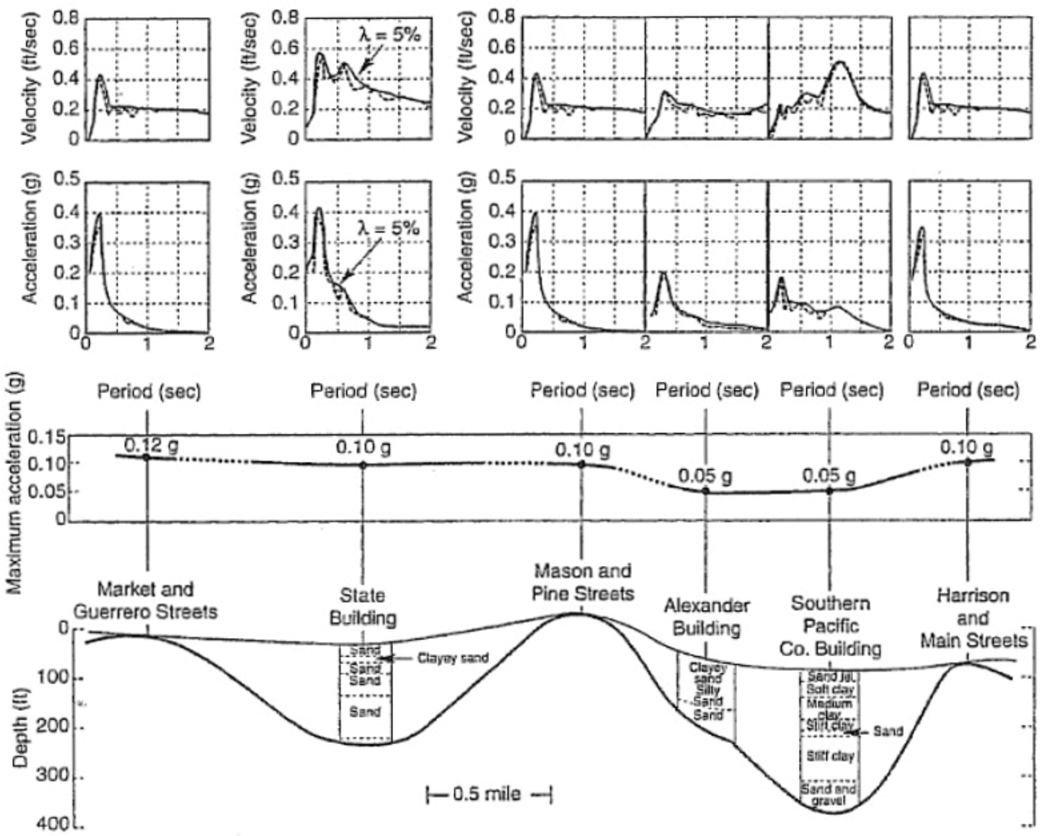


Figure 2-7 Variation of Spectral Velocity, Spectral Acceleration, and Peak Horizontal Acceleration Along a 4-Mile Section Through San Francisco in the 1957 San Francisco Earthquake (After Seed and Idriss, 1976)

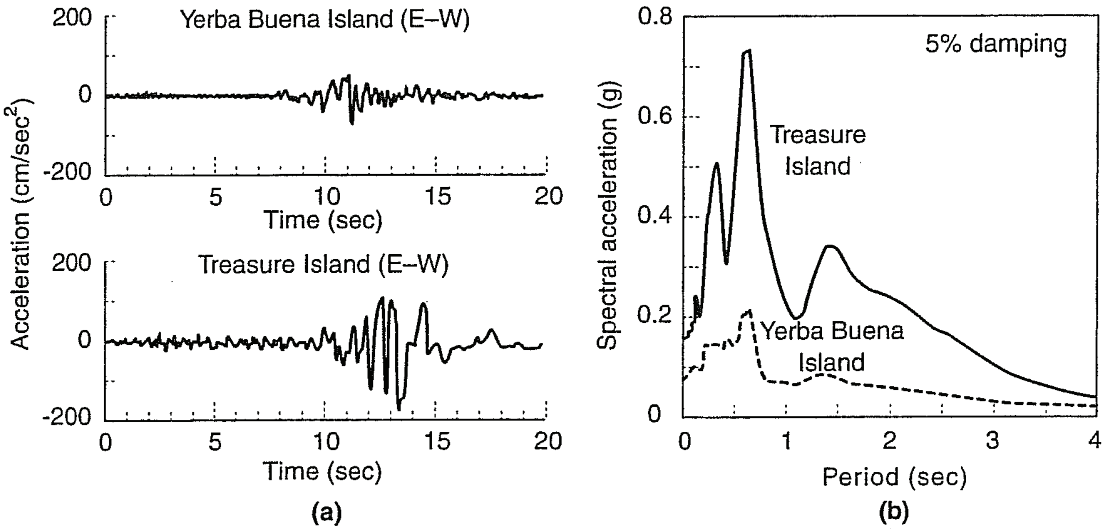


Figure 2-8 Ground Surface Motions at Yerba Buena Island and Treasure Island in the 1989 Loma Prieta Earthquake: (a) Time Histories; (b) Response Spectra (After Seed et al., 1990)

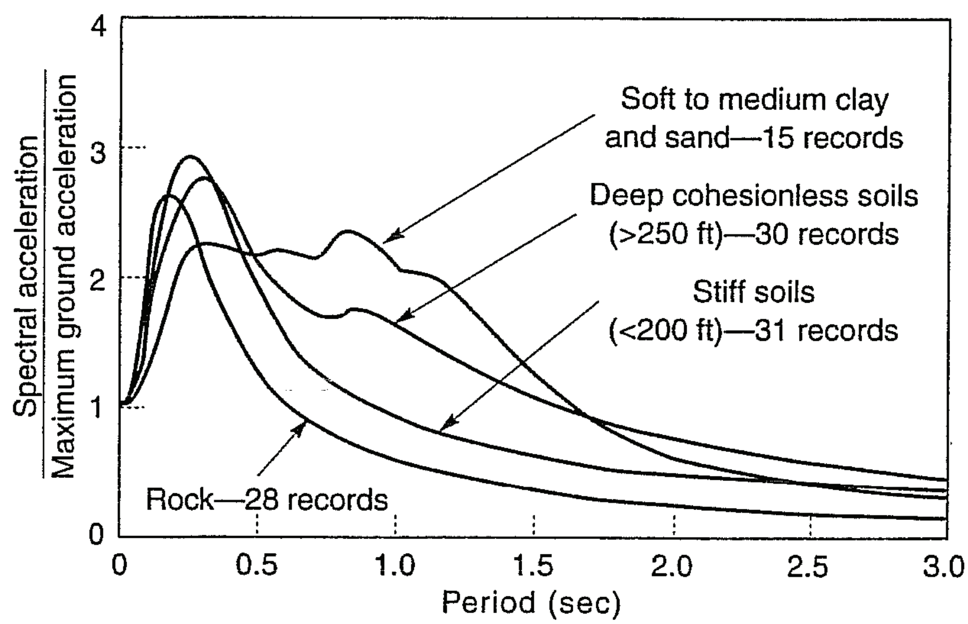


Figure 2-9 Average Normalized Response Spectra (5% Damping) for Different Local Site Conditions (After Seed and Idriss, 1976)

It is of particular interest to compare the response of two seismic records from Yerba Buena Island and Treasure Island in the San Francisco Bay. Yerba Buena Island is a rock outcrop and Treasure Island is a man-made hydraulic fill placed partially on the Yerba Buena shoals. Although Yerba Buena Island and Treasure Island are located at the same distance from the seismic source, the ground motion recorded at these two sites during the 1989 Loma Prieta Earthquake are very different. Figure 2-8 shows the comparison of acceleration time histories and response spectra in the east-west component. Clearly, the presence of the soft soils at the Treasure Island site caused significant amplification of the underlying bedrock motion.

Seed et al. (1976) computed average response spectra from ground motion records at sites underlain by four categories of site conditions: rock sites, stiff soil sites of less than 200 feet, deep cohesionless soil sites greater than 250 feet, and sites underlain by soft to medium-stiff clay deposits. Normalizing the computed spectra by dividing spectral accelerations by the peak ground acceleration illustrates the effects of local soil conditions of the shape of spectra, as shown in Figure 2-9. At periods above 0.5 sec, spectral amplifications are much higher for soil sites than rock sites; this phenomena is significant for long-period structures, such as tall bridges, that are founded on soft soils.

2.5 Multiple Support Motion

Long bridges pose a special challenge to designers because these structures are supported on multiple piers with varying foundation types, subsurface conditions and loading conditions. For this type of structure, multiple-support excitation is a valid form of loading during seismic events. Multiple support time history analysis is the best suited approach for studying effects of spatial variation in ground motion. To perform multiple support time history analyses, time histories of the ground motion arriving at each individual foundation are evaluated. Depending on the level of effort, modeling of the bridge can be a complete system or a simplified substructured system, as shown in Figure 2-10. In the complete system, every essential element of the foundation such as all the piles and all the soil support springs are explicitly included in the bridge model. On the other hand, the substructured model simplifies the foundation as a reduced degree-of-freedom system by using substructuring techniques based on the principle of structural mechanics. As illustrated in Figure 2-10, substructuring can be conducted for individual piles or for the entire pile group. Substructuring methods for pile foundations are discussed in Section 3.

Long bridges typically cross a bay or strait with bridge lengths ranging from 1 to 3 kilometers. As previously discussed, spatial variation of the ground motion due to incoherency and local site conditions leads to multiple support excitations for a long bridge. As a result, the conventional response spectrum approach of designing the global bridge as a primary design tool becomes inadequate unless some measures are implemented to consider the variation in the multiple support motions.

Recognizing the deficiencies in the response spectrum procedure, multiple-support time history analyses have been implemented in the designs (both seismic retrofits and new

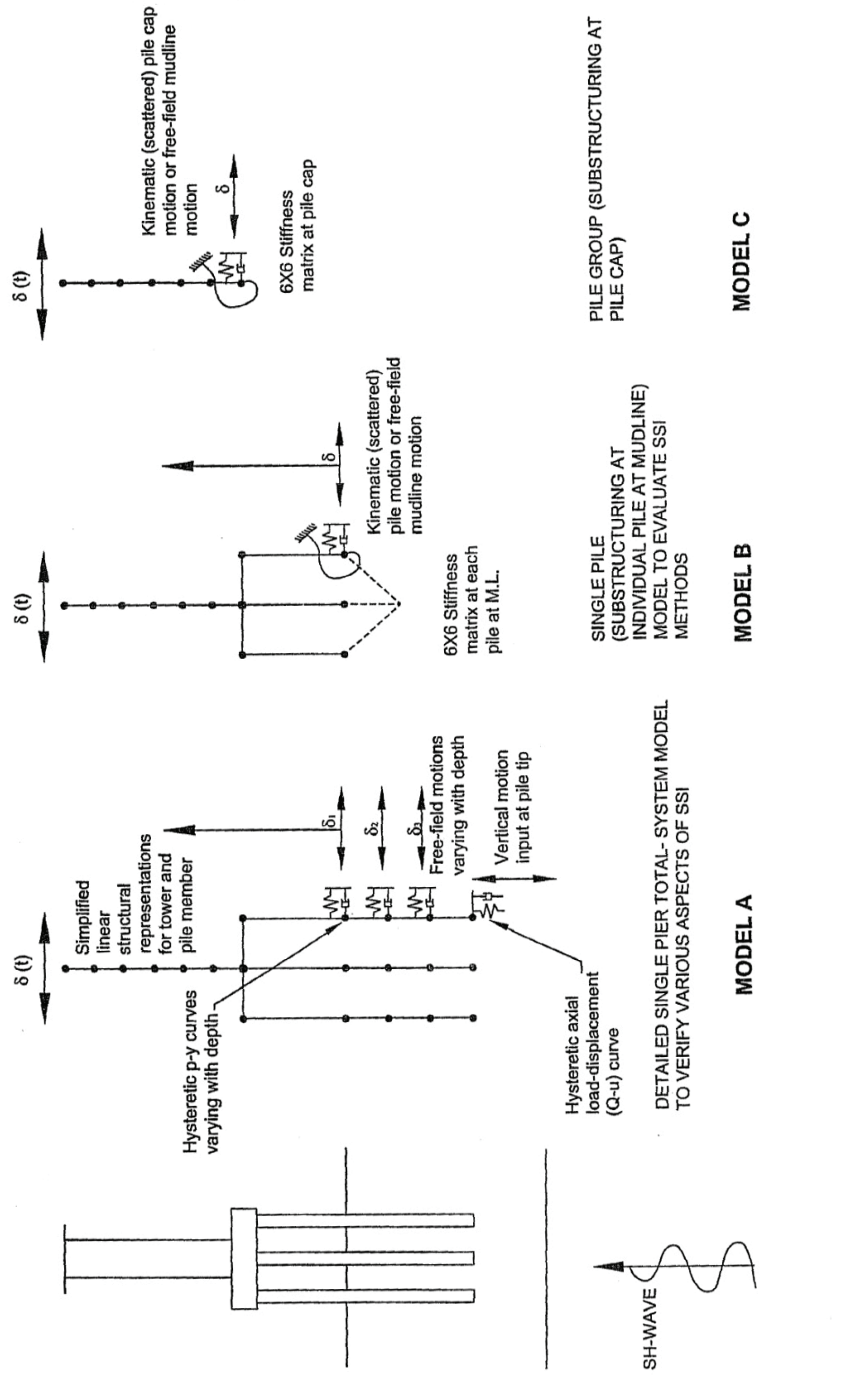


Figure 2-10 Various SSI Models to Evaluate a Bridge Structure

designs) of all California toll bridges. This procedure also allows the designers to address inelastic behavior of the structure and to incorporate such ground motion issues as multiple support excitation and near-field directivity effects. Table 2.1 lists the California toll bridges that have been designed with multiple support time history analyses.

As mentioned earlier, the effects of multiple-support excitation and directivity cannot be fully addressed by the response spectrum procedure. All the bridges listed in Table 2.1 were initially designed using response spectra and were later checked against multiple-support time history analyses. This was necessary during the initial phase of design to reduce the analysis time because execution of multiple-support time analysis is often very time consuming. The success of seismic response analysis for a bridge depends largely upon how well the design response spectra represent the actual loading conditions, including the potential for variations in multiple support excitation and incorporating all essential features of soil-structure interaction elements. It is a challenging task for geotechnical engineers to develop the design response spectra, requiring significant insight into all aspects of earthquake engineering and soil-structure-interaction issues.

Table 2-1 Seismic Design of Toll Bridges in California and Washington

Bridge	Bridge Type	No. of Multiple Support Motions	Directivity Effects
Seismic Retrofit of Existing Structure			
Golden Gate Bridge	Suspension	Three	Not Considered
Carquinez Bridge	Steel Truss	Three	Considered
Benicia-Martinez Bridge	Box Girder	Three	Considered
San Mateo Bridge	Box Girder	Three	Considered
Richmond-San Rafael Bridge	Steel Truss	Three	Considered
San Diego-Coronado Bridge	Box Girder	Three	Considered
Vincent Thomas Bridge	Suspension	Three	Considered
Design of New Structure			
New Carquinez Bridge	Suspension	Three	Considered
New Benicia-Martinez Bridge	Box Girder	Three	Considered
New East Span San Francisco-Oakland Bay Bridge	Suspension/Box Girder	Six	Considered
New Tacoma-Narrows Bridge	Suspension	Three	Not Considered

Irrespective of whether a complete total system or a substructured system for a bridge is modeled, multiple support motions are needed at different support piers. This involves conducting site response analyses to evaluate wave propagation at the pier location using local soil conditions and performing a soil-pile interaction study to derive an effective forcing function at the foundation level. These functions are usually input to the global bridge model as multiple support displacement time histories. The outcome of the study, in fact, includes spatial variation of ground motion, effects of local site conditions, and foundation compliance considerations. The forcing function estimated at each pier is then used in the global bridge model to determine the seismic response of the bridge.

2.6 California Toll Bridge Case Histories

The following discussion summarizes experience gained in addressing ground motion characterization issues documented above, in the case of the California Toll Bridge Seismic Safety Program (Law and Lam, 2006).

2.6.1 Rock Motion Criteria

Site-specific seismic hazard assessments were performed for the California Toll Bridge Seismic Safety Program utilizing both deterministic and probabilistic approaches. A dual seismic design criterion was usually adopted consisting of a Safety Evaluation Earthquake (SEE) and a Functional Evaluation Earthquake (FEE). The SEE is defined as the most severe event that can reasonably be expected to occur at the site. For all the toll bridges, return periods between 1,000 and 2,000 years were selected as the basis for target spectra for the SEE scenario. The FEE is defined as one that has a relatively high probability of occurrence during the lifetime of the structure. Typical return periods of the FEE event ranged from 100 to 300 years. Figure 2-11 shows the rock motion criteria for the East Span San Francisco – Oakland Bay Bridge Replacement project showing three components; fault normal, fault parallel, and vertical.

The seismic performance expectations of the toll bridges varied for different structures. In general, they were designed to meet the repairable damage or the ‘no-collapse’ criteria under the Safety Evaluation Earthquake, and to ensure prescribed levels of services for the Functional Evaluation Earthquake. In addition, the Vincent Thomas and the San Diego-Coronado Bridges were required a third design level, namely for Fault Rupture offset, in conjunction with the ‘no-collapse’ performance requirement for the SEE event. This requirement was due to crossing potentially active faults with a chance of ground surface fault offset within the bridge alignment.

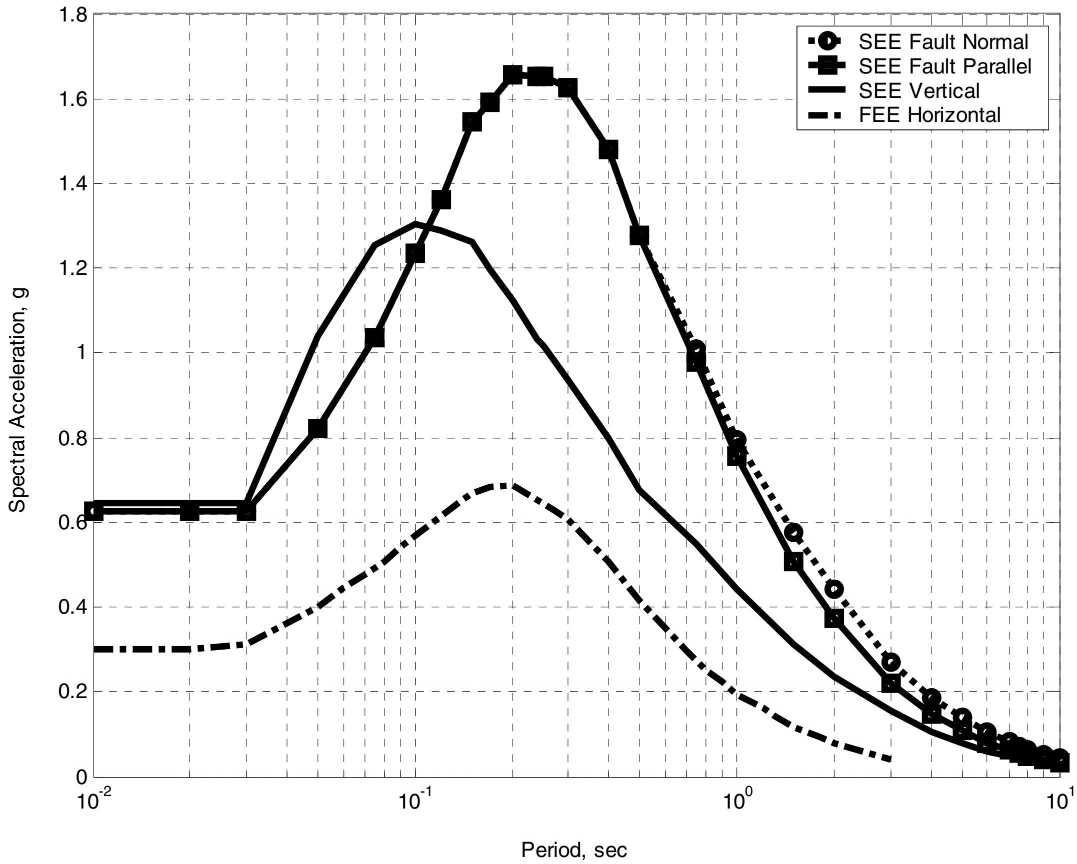


Figure 2-11 Rock Motion Criteria for the East Span
San Francisco – Oakland Bay Bridge

Near fault (within 15km distance to the fault) ground motions are essential elements of the seismic hazard evaluation. Rupture directivity is strongest in the fault normal direction and affects long period response (0.6 sec and longer). The near-fault effects are known to cause severe damage to long-span bridge structures, and were implemented in the toll bridge program. Spectral acceleration values for the fault normal direction are higher than those of the fault parallel direction.

2.6.2 Time History Development

Nonlinear time history analyses of the bridge structures were required due to the long span nature of the bridges, and thus the development of strong motion time histories was an essential part of the definition of seismic hazard. The earthquake time histories were constructed by modifying actual earthquake records in terms of the amplitude of their frequency content to match the entire design spectrum adopted for the design. The resultant time histories, denoted as spectrum-compatible motions, were used for all the toll bridges instead of scaling actual past earthquake records.

Other than matching of the response spectrum for each of the ground motion components developed for structural designs, correlation between the two horizontal orthogonal directions must be checked. In order to ensure that all structural components are adequately excited, the two horizontal components must be sufficiently uncorrelated. The relationship between two orthogonal components was examined by a cross-correlation coefficient. The idea behind this requirement is to guarantee that the input motions to the structural model have a minimum acceptable shaking intensity even after the ground motion time histories are rotated to any structural orientation of interest.

The earlier seismic retrofit work was based on one set of earthquake time histories consisting of three orthogonal components, while three sets of time histories were adopted for the seismic design of the new toll bridges except the San Francisco – Oakland Bay Bridge, which utilized six sets of earthquake time histories. Figure 2-12 depicts six sets of reference fault normal rock motion displacement time histories. From the writers' project experience, the structural designers reported that earthquake demand quantities, derived from three different sets of time histories, sometimes varied as much as 200% or more. When this occurred, the engineers were faced with a dilemma of how to design for such a wide range of results, i.e., to average or envelop the demand quantities from the different time history analyses. This became a serious issue and raised the question to seismologists: "Can a single spectrum-compatible time history ever be developed that will lead to the largest structural demand?" Clearly, the seismologists could not predict which set of earthquake time histories would yield the greatest structural response and the subject was discussed among the design teams and Peer Review Panels on numerous occasions during the course of the Seismic Safety Program.

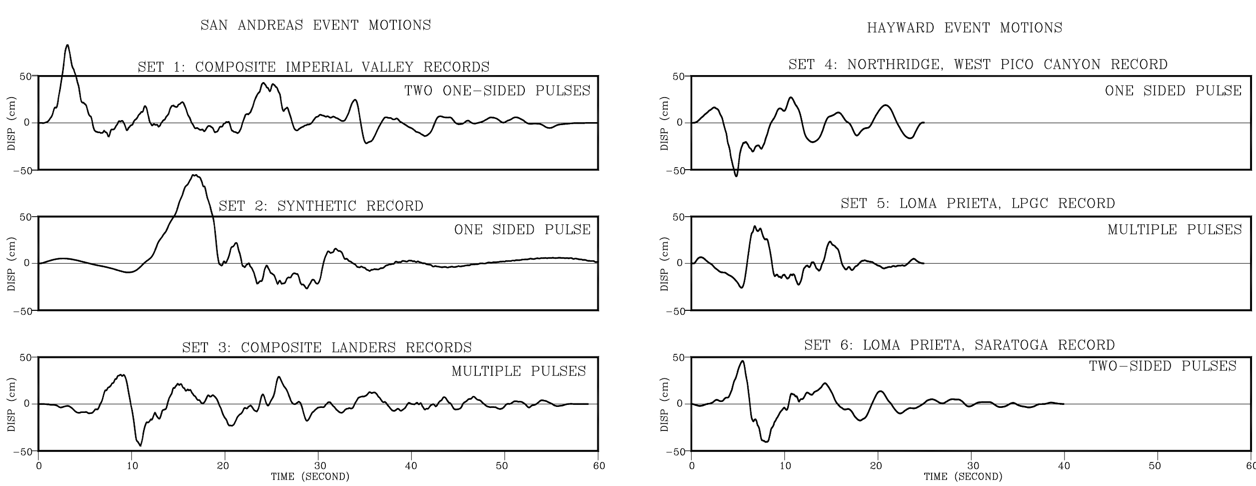


Figure 2-12 Rock Motion Time Histories for the East Span San Francisco – Oakland Bay Bridge

2.6.3 Incoherency

Ground motions can vary spatially along the bridge alignment due to scattering and complex wave propagation. In all the toll bridge projects, incoherent ground motions were considered as part of the multiple support time history analyses (Figure 2-13). This required exciting each bridge support with pier-specific motion, and three orthogonal components were used simultaneously. The spatial variability or incoherence is caused by a number of factors, such as:

- Wave Passage Effect: nonvertical waves reach different positions on the ground surface at different times causing a tine shift between the motions at those locations
- Extended Source Effect: mixing of wave types and source directions due to differences in the relative geometry of the source and the site
- Ray Path Effect: scattering of seismic wave by heterogeneity of earth along the travel path causing different waves to arrive at different locations at different times
- Attenuation Effect: variable distance from the different locations to the seismic source
- Site Response Effect: variable soils conditions produce different motions at the ground surface

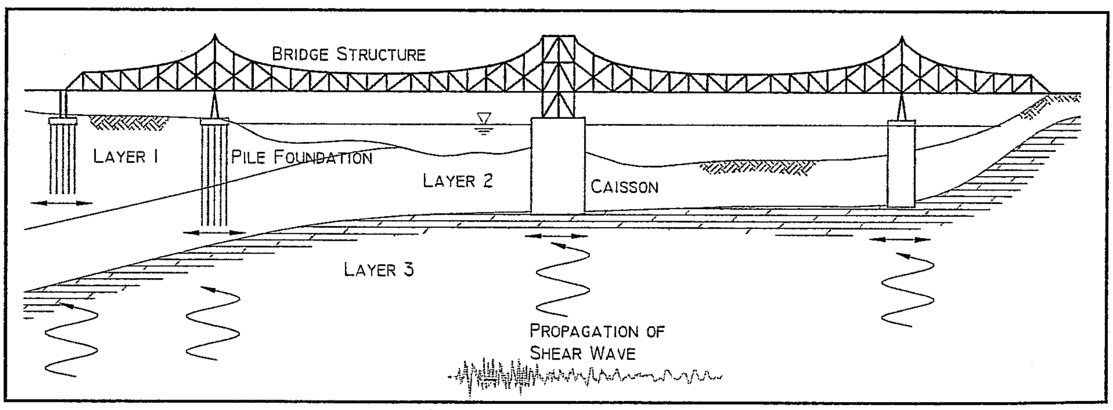


Figure 2-13 Propagation of Seismic Waves

From the toll bridge experience, incoherency arising from the wave passage, extended source, and ray path effects tends to alter high frequency components which do not influence the seismic response of long span bridges, characterized as long period structures. However, the site response effect profoundly influences the ground motion characteristics. This is because local site conditions can vary significantly over the length of the bridge, and the effect often overshadows the other sources of incoherency. For practical purposes, it is often adequate to just consider site response effect for incoherency; other incoherency effects, such as wave passage, extended source, and ray path effects, are not as significant for long span bridges.

2.6.4 Site Response

As seismic waves propagate through a soil deposit, the ground motion characteristics change when they arrive at the bridge supports. The ground motions impact loading to the structure in a form of depth varying motions along the depth of the foundation. For a long span bridge, the effects of local site conditions contribute to all the spatial variation.

For the most part, site response analyses were conducted using one-dimensional equivalent linear programs. Also, nonlinear site response analyses were conducted to appreciate effects of permanent ground displacement in some instances. The shear wave velocity of soils is the one of the most important parameters for conducting the site response analyses. In the past, shear wave velocity measurements were made with a crosshole geophysical sounding, requiring two boreholes for each measurement. Today, suspension P-S logging, a relatively new method, is used for measuring the seismic velocity profile in the borehole, eliminating the need to drill two boreholes, which adds to logistical problems for overwater boreholes. For example, 6 boreholes out of 20 drilled at the Vincent Thomas Bridge were logged with the downhole P-S logging to measure shear wave velocities of the soil (Figure 2-14).

The seismic retrofit analyses of the Vincent Thomas Bridge involved many engineers from different firms. Unintended mistakes were made by others due to miscommunication during early stages of the project when ground motions for multiple support time history analyses of the global bridge model which has a total of 30 supports, were provided. Since only 6 shear wave velocity measurements were made, each of the shear wave velocity profiles was assigned to a group of bridge piers; for example, the velocity profile (V1) was assigned to piers 1 through 5, and the velocity profile (V2) was assigned to pier 6 through 10. At a first glance, this approach seemed reasonable. When the site response analysis was conducted for each pier, no different movement was observed among the piers that utilized the same shear wave velocity profile. However, at the boundary of the two groups of piers (for example between Pier 5 and Pier 6), significant differential movement as much as one foot was observed. When this set of multiple support time histories were first applied in the structural analyses, shear failure was reported at every boundary between adjacent groups of piers.

This was quickly recognized to be artificial and was later corrected by careful interpretation of the soil properties along the bridge length prior to conducting the pier-specific site response analyses. The interpretation included gradual transition of shear wave velocities consistent with the soil stratigraphy developed from soil borings, lab testing, and SPT and CPT results.

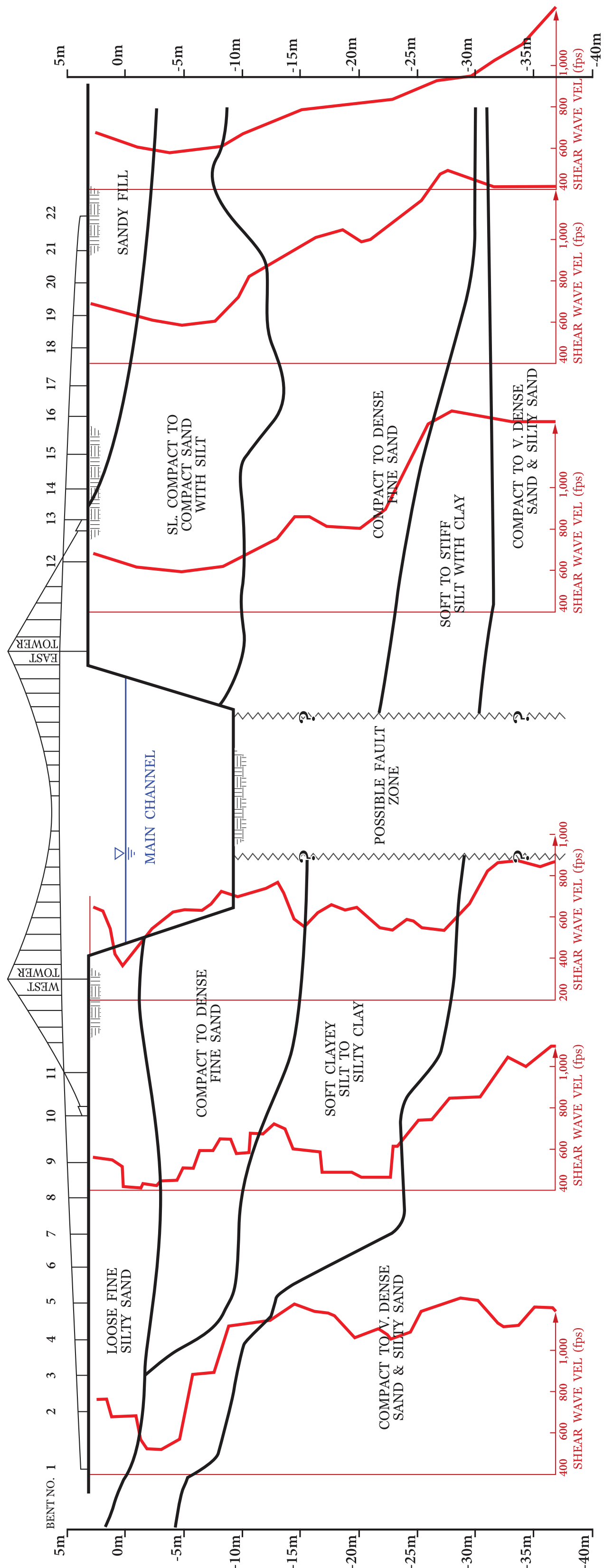


Figure 2-14 Soil Profile Along the Vincent Thomas Bridge

Another problem was related to where the rock outcrop motion was assigned in the site response analyses. In any site investigation program, soil boring depths sometimes differ by as much as 100 feet between two adjacent boreholes. When one-dimensional soil columns were constructed strictly following the boring specific soil data for the purpose of the site response study, the different heights of the two soil columns resulted in different elevations at the base of soil columns where rock outcrop motions were prescribed. Seismic waves in the long soil column had to travel longer than in the short soil column. If the column height is different by 100 feet, seismic waves would have to travel 0.1 second longer in soil media with an average shear wave velocity of 1000 feet per second. Such a delay time could readily lead to differential displacement as much as eight inches between two piers for an earthquake time history having a PGV of 80 inches per second. This is significant enough to tear the bridge apart if such differential movement occurs between two adjacent piers. Therefore it is essential to fix a reference rock motion elevation for any site response analyses to avoid artificial differential movement.

SECTION 3:
MODELING LARGE PILE GROUPS

3.1 Overview

In many long span bridge projects, substructuring is used to reduce the size of the problem. The substructuring technique involves modeling the pile foundation to a convenient interface with the superstructure, e.g., at the base of the pile cap (as shown in Figure 3.1). Static condensation is then used to derive the appropriate foundation substructure stiffness and the effective ground motion transmitted to the superstructure; the resultant effective ground displacement is termed the kinematic ground motion (Lam and Law, 2000). The foundation stiffness matrix is used in conjunction with the kinematic ground displacement to represent the entire foundation system in the superstructure analysis.

Pile foundations are modeled as structural beams on Winkler springs, represented by depth-varying soil springs. These soil springs may be taken as linear or nonlinear p-y curves for a complete system model where the bridge superstructure and individual piles with soil springs are included (see upper sketch of Figure 3.1). Because large degrees of freedom are needed to formulate the complete system, practitioners use an alternative method employing a substructure system in which the foundation element is modeled by a condensed foundation stiffness matrix and mass matrix along with equivalent forcing function represented by the kinematic motion (see lower sketch of Figure 3.1). This approach allows evaluations of foundations and the bridge structure separately as a means of reducing the problem into a manageable size. Note that for global models with spatially varying input motions at different bridge foundation support points, displacement input time histories are required by structural designers as opposed to acceleration input time histories in uniform input response analysis, such as for buildings.

Further discussion on the development of p-y curves and the influence of pile group effects is provided in Section 3.2. Section 3.3 discusses the role of damping arising from soil-pile interaction. Section 3.4 provides a commentary on the development of pile group stiffness matrices and Section 3.5 provides a similar commentary the influence of kinematic soil-pile interaction on input ground motions to the substructure system. A summary of a representative case history is provided in Section 3.6.

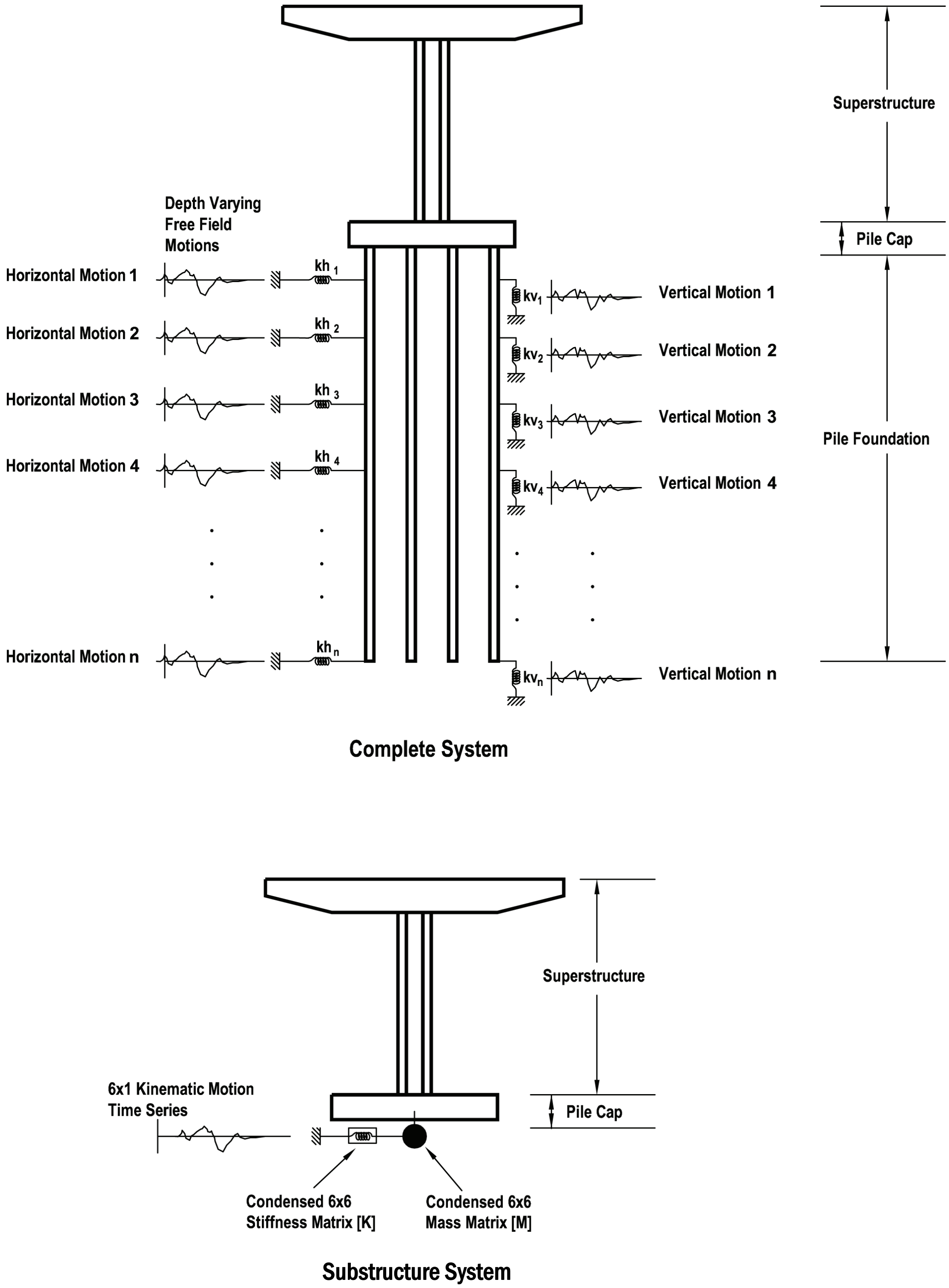


Figure 3.1 Modeling of Pile Foundation for Seismic Design of Bridges

3.2 Soil-Pile Springs (p-y curves) and Pile Group Effects

Empirical p-y curve approaches to represent soil resistance characterizing lateral soil-pile springs have been extensively used by practicing engineers in pile design analyses (Lam and Law, 1996). The term p-y curve was first used by McClelland and Focht (1958), where p is defined as the soil resistance per unit pile length for a given pile deflection, y. Whereas, p-y curves are largely based on empirical pile-load test data, in its purest form, it can be regarded simply as a convenient device to represent the soil reaction-deformation characteristics in an analysis. Uncertainties in the p-y curve values arising from the empiricism, may not be significant in terms of overall lateral pile stiffness. The influence of uncertainties in p-y curves on the lateral stiffness or load-deflection characteristics of piles and the associated bending moment and shear forces, has been discussed by Lam et al. (1998). Based on parametric studies, it was shown that uncertainties in p-y curve values could be overshadowed by assumptions of pile-head fixity in pile caps, choice of bending stiffness parameters for the piles, gapping effects pile embedment effects, and loading conditions.

In addition to the empirical database, other means, including finite-element analyses could be used to develop the appropriate load-deflection characteristics as p-y curves in foundation analyses, as discussed in Section 3.2.2. For pile group spacings of greater than 5 to 6 pile diameters, single pile p-y curves are normally used as discussed in Section 3.2.1. For closer spacings, adjustments for group effects are normally made, as outlined in Section 3.2.2.

3.2.1 Conventional p-y Criteria for Earthquake Loading – Single Piles

The most widely used p-y criteria for single piles are Matlock's soft clay (Matlock, 1970) and the Reese's sand criteria (Reese et at., 1974), both of which were based on full-scale pile load tests. These criteria can be regarded as extensions of the widely used linear subgrade modulus theory advanced by Terzaghi (1955), as discussed by Lam et. al. (1998). The Matlock and Reese procedures have been recommended for design by the American Petroleum Institute (API), and can serve as the benchmark for design to represent the initial monotonic loading path for typical small diameter (less than 2 ft.) driven isolated piles. Matlock's soft clay and Reese's sand p-y criteria have been verified by numerous researchers and practitioners using slow static and cyclic pile-load tests. For soft clays, Lam and Law (1996) show diameter effects are small even for large pile diameters. For large diameter drilled shafts in competent soils, Lam et al. (1998) suggest a scaling factor to increase p-y curves equal to the ratio of the diameter to 2 ft., be applied.

Limited full-scale vibratory and centrifugal model pile load tests have also been performed to verify the validity of the p-y criteria. Some of these dynamic pile load tests have been reported by Lam and Cheang (1995). The Lam and Cheang test data included parallel tests for both slow-cyclic and fast vibratory loading conditions for direct comparison. A common trend in all full-scale dynamic test data discussed above indicates that the pile stiffness would be significantly lower than what is observed during slow (referred as static) pile-load tests (Pender, 1993). The reduction in the pile stiffness is highly dependent on the load level and suggests that it occurs at a wide range of frequencies. The reduction in the apparent pile stiffness has been attributed to various physical phenomena, including:

- Dynamic gapping effects,
- Pore pressure build-up effects in sands,
- Shake down effects, and
- Nonlinear stress-strain effects.

Most of the above reported literature found that except for sensitive soils and liquefiable soils, simple p-y or subgrade modulus approaches give reasonable predictions for pile response at common earthquake loading ranges. The lack of dynamic considerations inherent in p-y and subgrade modulus theories do not appear to be a serious deficiency. Levine and Scott (1983) in backfitting strong motion records at the Melloland bridge found that using simple blowcount data to develop a linear subgrade modulus value led to reasonable solutions of foundation stiffness to predict the observed dynamic response characteristics of the bridge. Several other case histories with similar conclusions have been drawn from test data at other bridge sites.

3.2.2 Effect of Pile Spacing (Group Effects) on p-y Curves

Group effects on the lateral stiffness and capacity of pile groups has been a popular research topic within the geotechnical community for 50 years. Full-size and model tests by a number of authors show that in general, the lateral stiffness and capacity of a pile in a pile group versus that of a single pile (termed “efficiency”) is reduced as the pile spacing is reduced. Other important factors that affect the efficiency and lateral stiffness of the piles are the type and strength of soil, the number of piles, and type and level of loading.

In earlier years (the 1970’s), the analysis of group effects were based mostly on elastic halfspace theory due to the absence of costly full-scale pile experiments. These solutions have many shortcomings, including: (1) they do not account for changes in the soils from pile installation; (2) they do not account for many aspects of soil-pile interaction phenomena (e.g., gapping effects); and (3) they are based on an elastic halfspace model which does not account for reinforcing effects of the piles within the soil mass, or soil compaction from pile driving. These solutions for group effects were used to soften the elastic stiffness (by introducing y-multipliers on p-y curves). However, as more full-scale or model pile load test data became available, it was found that group effects were best accounted for by introducing p-multipliers on the p-y curves (Brown et al., 1987, 1988, McVay et al., 1998, Rollins et al., 1998, Ruesta and Townsend, 1997, Zhang and McVay, 1999, Mokwa and Duncan, 2001, and Rollins et al, 2006).

<u>Small Pile Groups</u>
The major experimental studies noted above have yielded high-quality experimental data from either full-scale or centrifuge model pile load tests. These experiments (on small

pile groups with varying spacing generally 3x3) yielded information that largely corroborated each other on the following aspects:

(1) Most of these experiments first used the single pile data to verify the validity of the widely used Reese's and Matlock's benchmark p-y criteria and all concluded that the Reese and Matlock p-y criteria provide reasonable solutions.

(2) The observed group effects appeared to be associated with shadowing effects and the various researchers found relatively consistent pile behavior in that the leading piles would be loaded more heavily than the trailing piles when all piles are loaded to the same deflection. It was concluded from all reported full-scale or centrifuge model test experiments that the observed group effects cannot be accounted for by merely softening the elastic stiffness of the p-y curves (i.e., using only y-multipliers on p-y curves as those developed from elastic halfspace theory). All referenced researchers recommended modifying the single pile p-y curves by adjusting the resistance value on the single pile p-y curves (i.e. p-multiplier). The p-multipliers were found to be dependent largely on the position of the pile in relation to the pile loading direction. Most of the experiments were conducted on 3x3 pile groups with the center-to-center pile spacing equal to about 3 pile diameters. Table 3-1 summarizes some of the back-calculated p-multiplier values from several individual researchers for 3x3 pile groups with 3D spacing.

The experiments reported by McVay also included data for pile center-to-center spacing of 5D which showed p-multipliers of 1.0, 0.85 and 0.7 for the front, middle and back row piles, respectively. For such multipliers, the group stiffness efficiency would be about 95% and group effects would be practically negligible, whereas for 3D spacing, the group efficiency would be about 50%.

Table 3-1 p-Multipliers (Cycle-1 Loading) from Various Experiments at 3D Center-to-Center Spacing for 3x3 Groups

Pile Test, Soil Description, Reference	**p-multiplier on single-pile p-y curves**		
	Front Row	**Middle Row**	**Back Row**
Free-Head, Medium Dense Sand, D_r = 50% Brown et al. (1988)	0.8	0.4	0.3
Fixed-Head, Medium Dense Sand, D_r = 55% McVay Centrifuge (1995)	0.8	0.45	0.3
Fixed-Head, Medium Dense Sand, D_r = 33% McVay Centrifuge (1995)	0.65	0.45	0.35
Free-Head, Soft to Medium Clays and Silts Rollins et al. (1997)	0.6	0.38	0.43

Summaries and reviews of many of the load tests have been documented by Mokwa and Duncan (2001) and Dodds (2005). Implications of pile load-deflection analyses are discussed by Lam et al. (1998), as illustrated in Figure 3-2.

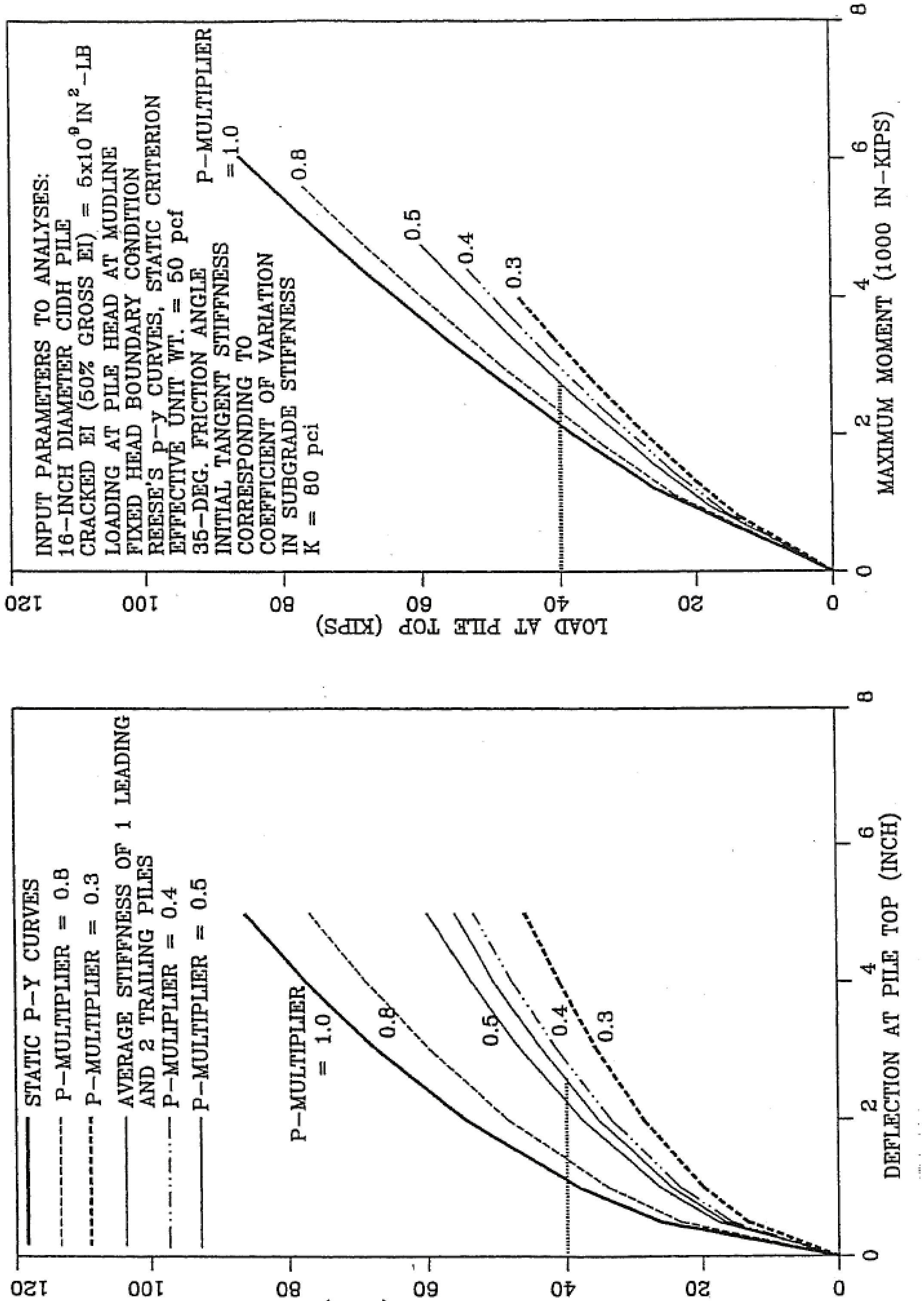


Figure 3-2 Pile Group Effects Reflected by p-y Curve on Pile Response (after Lam, 1998)

Figure 3-2 presents load-deflection analyses using a range of p-multipliers from the pile group effect experiments. Solutions are presented for a 16-inch fixed-head pile embedded in medium sand with a 35° friction angle. The figure presents results for several load-deflection analyses:

(1) A case for the standard static loading p-y curve to illustrate the behavior if one ignores pile group effects.

(2) Cases for p-multipliers of 0.8, 0.4 and 0.3 to represent analyses including group effects for the front row, the center row and the back row piles, respectively, for the range of p-multipliers from the discussed group effect experiments.

(3) In the load-deflection plot, a load-deflection curve from averaging the stiffnesses of three piles (the 0.8 p-multiplier case, representing the front row pile, plus two of the 0.3 p-multipliers from the discussed group effect experiments).

(4) An additional case for a p-multiplier of 0.5 which might represent the average adjustment factor needed to develop an average condition to fit the overall group effect. Such simplification would be needed in practical design for earthquake loading considering the cyclic earthquake loading conditions. The front row pile would become the rear row pile when loading is reversed. Therefore, for cyclic lading conditions, there is relatively little justification to maintain a different p-multiplier specific to the position of the pile. A uniform average multiplier would be adequate to incorporate pile group effects in design practice for typical pile group configurations. This case of a 0.5 p-multiplier would have a resultant pile stiffness quite similar to the average pile stiffness discussed in item (3) above.

The solutions show that group effects (as depicted in the average load-deflection curve) represent a maximum of 50% reduction in the resultant pile-head stiffness (depending on the deflection or the load values of interest) when the static single-pile stiffness is used as the reference point. The significance of the portrayed group effect is further diminished in terms of the assessed pile stress as evident by the moment versus shear load characteristics. As shown in Figure 3-2, for a specific pile load (say 40 kip per pile), there would only be about a 20 to 30% difference in maximum pile moment between the solution for a p-multiplier of 1 (no group effect) and 0.5 (with group effect). This difference would be even further diminished in actual design because the kinematic constraint of the pile cap would enforce all the piles in the group to deflect to the same deflection amplitude. Hence, the stiffer front row pile would carry a higher proportion of pile load as compared to the trailing piles. However, the stiffer front row pile also mobilizes more soil resistance which would lower the pile moment. In effect, this kinematic constraint will lead to stiffer piles to compensate for the weaker piles and will result in a more uniform pile moment distribution among each of the piles. Most of the pile group tests also show that upon cyclic loading, the resultant load and moment distribution among individual piles tends to become more uniform as compared to the initial loading condition. In fact, many researchers have speculated or reported that the cyclic loading condition tends to result in a reduction in the so-called group effects.

While the available data suggests that the group effects would result in a systematic softer pile stiffness, other effects (e.g., soil resistance of the pile cap, pile fixity gapping effects and consideration of cyclic loading effects) also need to be properly taken into consideration. Some of these effects maybe as significant as the pile group effects, as discussed by Lam et al. (1998) and reviewed in Section 3.4.

Extremely Large Pile Groups

Investigations on pile group effects described above provided insight and understanding into behavior of relatively small pile groups, typically 3 x 3. The studies are mostly based on field and centrifuge test information. However, many long-span bridges are supported on large pile groups that involve hundreds of piles; there is no specific design procedure for such large pile groups. Table 3.2 presents some of the large pile foundations supporting major bridges in California and highlights the need for investigating group effects for a large number of piles.

Table 3-2 Large Pile Group Foundations – California Bridges

Location	Bent	Pile Type	Number of Piles	Soil Type
Vincent Thomas Suspension Bridge, Long Beach, CA	East anchorage	14BP117	188	Silty sand
	East tower	14BP117	165	Silty sand
	West tower	14BP117	167	Fine sand/clay
	West anchorage	14BP117	188	Fine sand/clay
San Francisco-Oakland Bay Suspension Bridge, Oakland, CA	Pier E6	Timber	544	Bay mud
	Pier E7	Timber	544	Bay mud
	Pier E8	Timber	544	Bay mud
	Pier E9	Timber	625	Bay mud
Old Carquinez Truss Bridge, CA	Pier 4	18” concrete	225	Bay mud
	Pier 5	Timber	151	Bay mud
	South Anchorage	30” CISS	389	Bay mud

Lam et al. (1998) and Law and Lam (2001) describe the use of finite element analyses coupled with a periodic boundary condition (To representing an infinite repeating pile pattern) to numerically study the problem. Figure 3-3 illustrates the concept of periodic boundary conditions for an infinite number of piles to study the group behavior. The periodic boundaries of the pile group are shown, and the stress field within the region enclosed by one period of boundary would be the same as that in any other region. Deformations along the periodic boundaries at the corresponding locations would be identical; e.g., deformations at points A and B are identical, as are points C and D. This satisfies displacement compatibility between the tributary regions. In fact, one would only need to model this tributary region enclosed by one period of the boundary. By satisfying certain boundary conditions, the single pile model bounded by one period of boundary would effectively simulate an infinite number of piles. A similar concept has been applied to site response analyses based on a shear beam model to simulate a semi-finite medium.

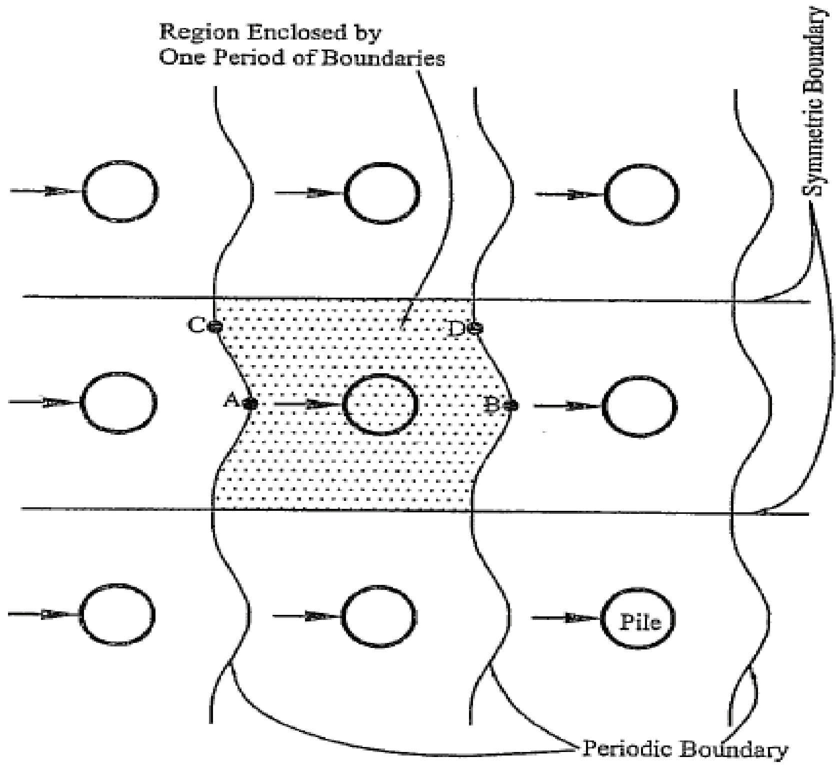


Figure 3-3 Schematic Illustration of Periodic Boundary for Infinitely Large Pile Group

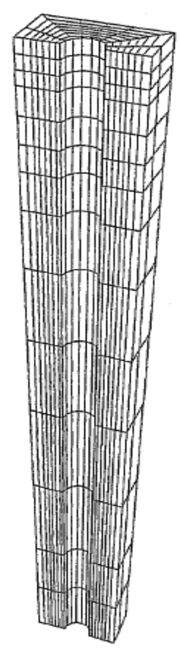


Figure 3-4 View of Finite-Element Mesh with Boundary Nodes Slaved; 3 Pile-Diameter Spacing is Shown

Based on this concept, 3D finite-element models enclosed by one period of boundary were set up, as shown in Figure 3.4. The studies were carried out with the DYNAFLOW program (Prevost 1981). The finite-element model consisted of a pile, 23 m in length, embedded in clay and shale. The clay was 18 m thick and fully submerged. The boundary nodes at the corresponding locations were slaved together to enforce compatibility of their displacements. Eight-node brick elements with simple generalized elasto-plastic constitutive relations were used for the clay to characterize nonlinear soil behavior, which is assumed to be elastic-plastic.

Elastic beam elements were used for the pile with section properties corresponding to the 0.75-m diameter CISS pile since the beam elements do not physically have a diameter, proper steps were taken to include the diameter effects of the pile, e.g., connecting the soil nodes with the pile nodes using radial "spokes." Beam elements with a large rigidity were used as the "spokes" and did not affect the pile's bending stiffness, because they merely served as rigid links between soil and pile nodes. Potential gapping between pile and soil was also considered by implementing interface elements which allowed separation between pile and soil when the normal stress approached zero, i.e., no tension developed. The interface elements were not allowed to slip on the pile surface; however, the shear traction on the pile was limited by the shear strength of the clay. The pile top was loaded laterally while keeping zero rotation at the top to maintain a fixed head condition.

The studies were part of foundation design and seismic analyses performed for a new suspension bridge (Carquinez Bridge) on Interstate 80 in Northern California. The anchorage foundations for the bridge would resist static and seismic cable loads from the bridge deck, as well as the inertia load of the anchorage block itself. The south anchorage is supported on 380 cast-in-steel-shell (CISS) piles 0.75 m in diameter and center-to-center spacing about 3 pile diameters. The site is underlain by clay with a shear strength ratio (s_u/p') about 0.25.

Finite-element analyses were conducted for the periodic boundaries corresponding to the center-to-center pile spacings of 3-pile diameter, 6-pile diameter, or 24-pile diameter. The relations of pile-head shear versus pile-head deflection and pile-head moment versus pile-head deflection are given in Figure 3-5. A p and y multiplier concept was used as shown in Figure 3.6.

The model with the 24-pile diameter spacing represents no pile group effect, i.e., pile spacing is too far to cause interaction among adjacent piles. Therefore, the finite-element results from the 24-pile diameter spacing are compared with beam-column solutions based on an empirical p-y approach using soft clay criteria (Matlock 1970). The beam column solutions with these p-y curves compare favorably with the finite-element results of the 24-pile diameter spacing, as shown in Figure 3-5. In addition, p-y based beam-column analyses were conducted for 6-pile diameter and 3-pile diameter spacings using a scaled version of the p-y curves. An attempt was made with a trial-and-error process to backfit the finite-element solutions for determining suitable scaling factors (multipliers) for the p-y curves.

In this study, both the p-multiplier and y-multiplier were used, because the two parameters offer greater flexibility for backfitting than using only a p-multiplier or a y-multiplier. It appears that the p-y curves using the soft clay criteria would require a p-multiplier value of 0.75 and a y-multiplier value of 2 to match the finite-element solutions for the 6-pile diameter spacing. For the 3-pile diameter pile spacing, the values of the p-multiplier and y-multiplier were found to be 0.5 and 4, respectively.

Based on the studies, the p-multiplier of 0.5 and the y-multiplier of 4 were adopted for the design of the anchorage piles at the Carquinez Suspension Bridge. The same multipliers were applied to all the piles regardless of position within the group. This set of p-y curves is regarded as somewhat conservative, especially for those piles at the edge of the pile group, but prudent for design. In reality, there are more complex phenomena, such as compaction and densification of soil around the pile after installation, that have not been fully accounted for in the analyses. Furthermore, the analyses do not explicitly account for soil degradation effects for cyclic loading expected for an earthquake. Both mechanisms are highly complex and may compensate for each other.

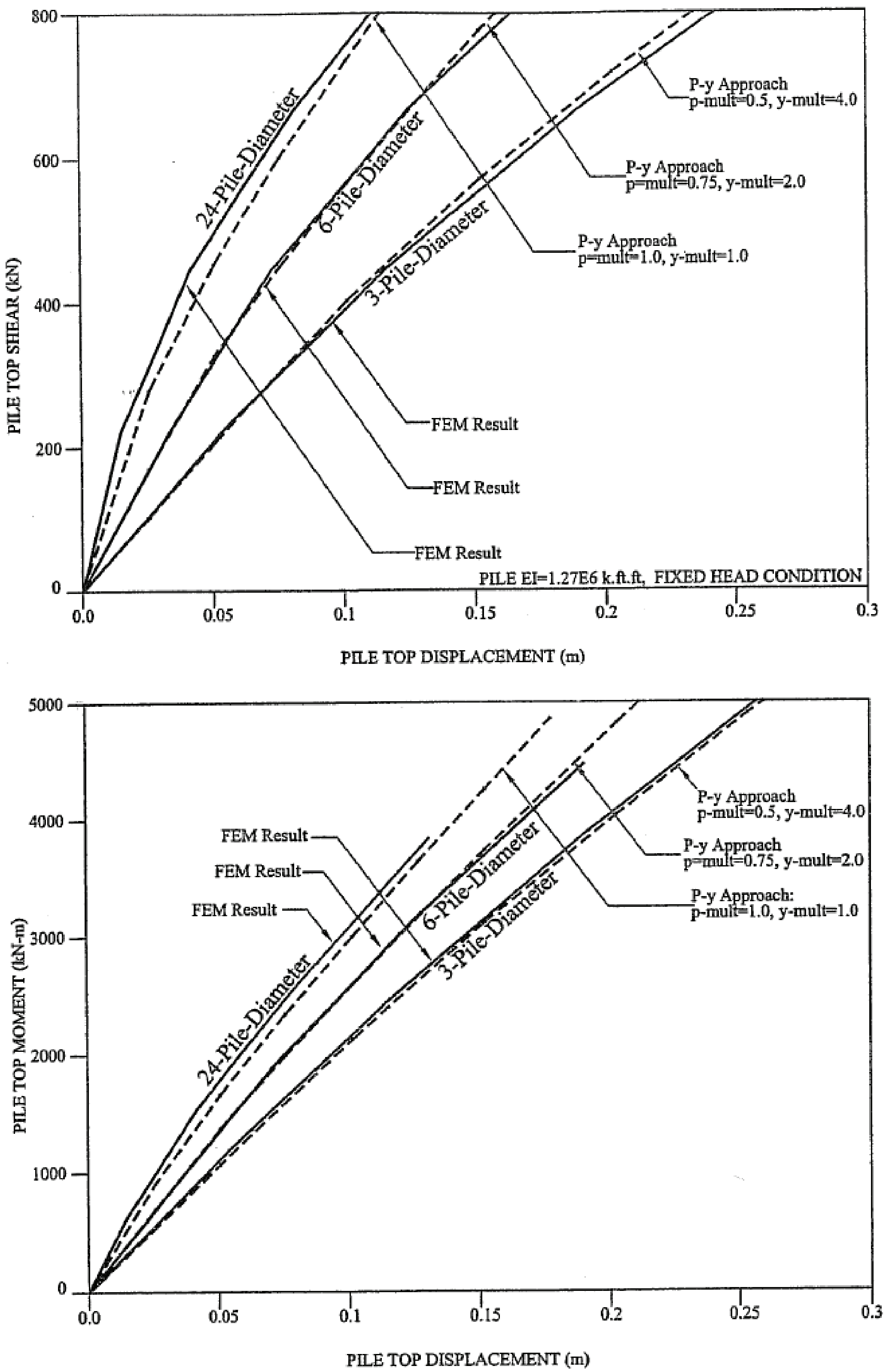


Figure 3-5 Pile-Head Shear Versus Pile-Head Displacement, and Pile-Head Versus Pile-Head Deflection

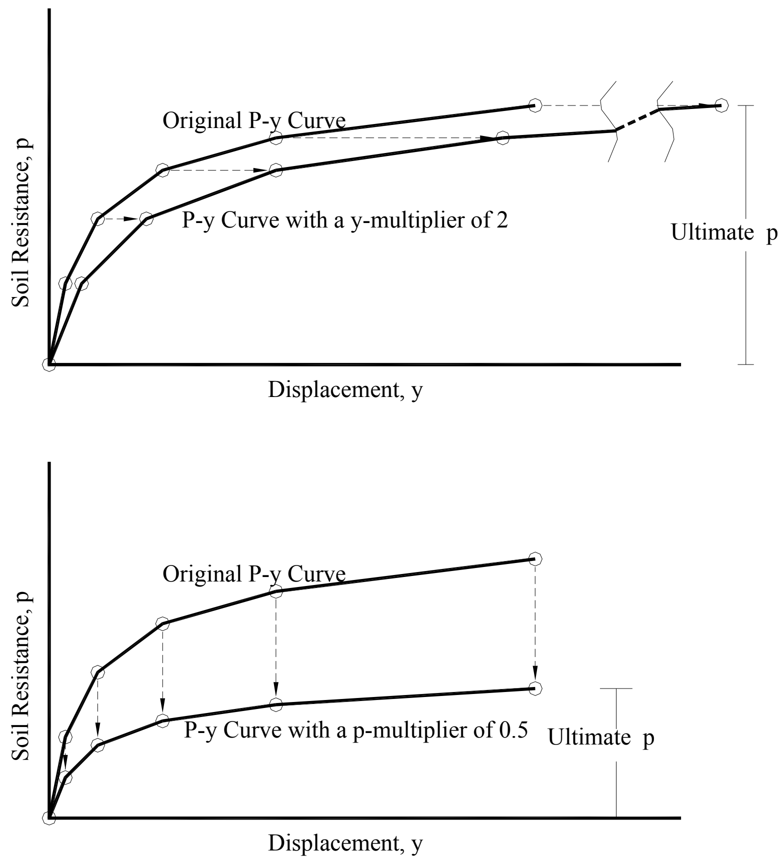


Figure 3-6 Concepts of y-Multiplier and p-Multiplier for p-y Curve

A comprehensive numerical study similar to that described above but using the finite difference computer program FLAC (3D), is described by Dodds (2005) and Dodds and Martin (2007). A p-multiplier only was used in interpreting results of the research which are summarized below, together with recommendations for p-multipliers for lateral resistance of large pile groups.

In this study, the capability of FLAC (3D) to assess group effects was confirmed, both in qualitative and quantitative terms. Utilizing solid element elastic piles, elastic-plastic soil models, and simplified representations of group piles, large pile group effects were studied and from the standpoint of pile head response. Results were generally in good agreement with field observations on small pile groups, although they are suggestive of lower row multipliers, particularly for soft clay. Soil resistance versus pile deflection (i.e. p-y) behavior for the group piles compliments the pile head response behavior in a general sense. However variation of p-multipliers with depth was apparent as a result of different deflected pile shapes and soil resistance patterns compared with isolated piles. Overall, the large pile group study indicated that initial stress state, pile type and pile-head restraint resulted in some differences, but these were relatively weak compared with the influence of soil behavior.

Marked differences in soil response against the piles were primarily associated with the different failure criteria employed for the sand and clay models. These factors demonstrate a need to appreciate a relative pile-soil stiffness influence brought about by group effects. The periodic piles in clay clearly illustrated this influence by exhibiting behavior akin to rigid rather than flexible pile behavior. This is particularly the case for interior piles of large pile groups, where much lower p-multipliers are considered appropriate. Thus significantly softer translational stiffnesses for large pile groups, compared with current recommendations for small groups, should be applied.

In terms of design, the research enabled greater insight into the mechanics of large pile group lateral stiffness due to horizontal translation, providing appropriate magnitudes of p-multipliers to be applied to p-y curves for clay and sand conditions. A trend clear from the research results was the significantly reduced translational resistance of periodic piles compared with isolated and leading piles. Accordingly, design recommendations are provided for p-multiplier values that characterize the translational resistance of large pile groups. These are based on a practically-oriented assessment of the pile head ratio and p-multiplier research results for periodic piles, and are appropriate to typical interior piles of large pile groups. The design recommendations are indicated on Figure 3-7 where they are shown relative to the current design recommendations for smaller pile groups that were previously discussed.

The design recommendations are also tabulated in Table 3-3, together with the large pile group recommendations made by Law and Lam (2001) described above. Given that the Law and Lam recommendations also included respective y-multipliers, it is not possible to make a direct comparison. However, the y-multiplier values recommended by Law and Lam suggest overall reductions in lateral resistance of the same order as found in the recommendations.

Table 3-3 Large Pile Group Recommendations for p-multipliers

Large Pile Group Study	**Soil Condition**	**Center to Center Spacing (s)**	
		$s = 3d$	$s = 6d$
Law and Lam (2001)	Soft clay	0.5[(1)]	0.75[(2)]
Dodds (2005)	Soft clay	0.1	0.5
	Medium dense sand	0.2	0.7

Notes:

(1) *y*-multiplier of 4 also applies.
(2) *y*-multiplier of 2 also applies

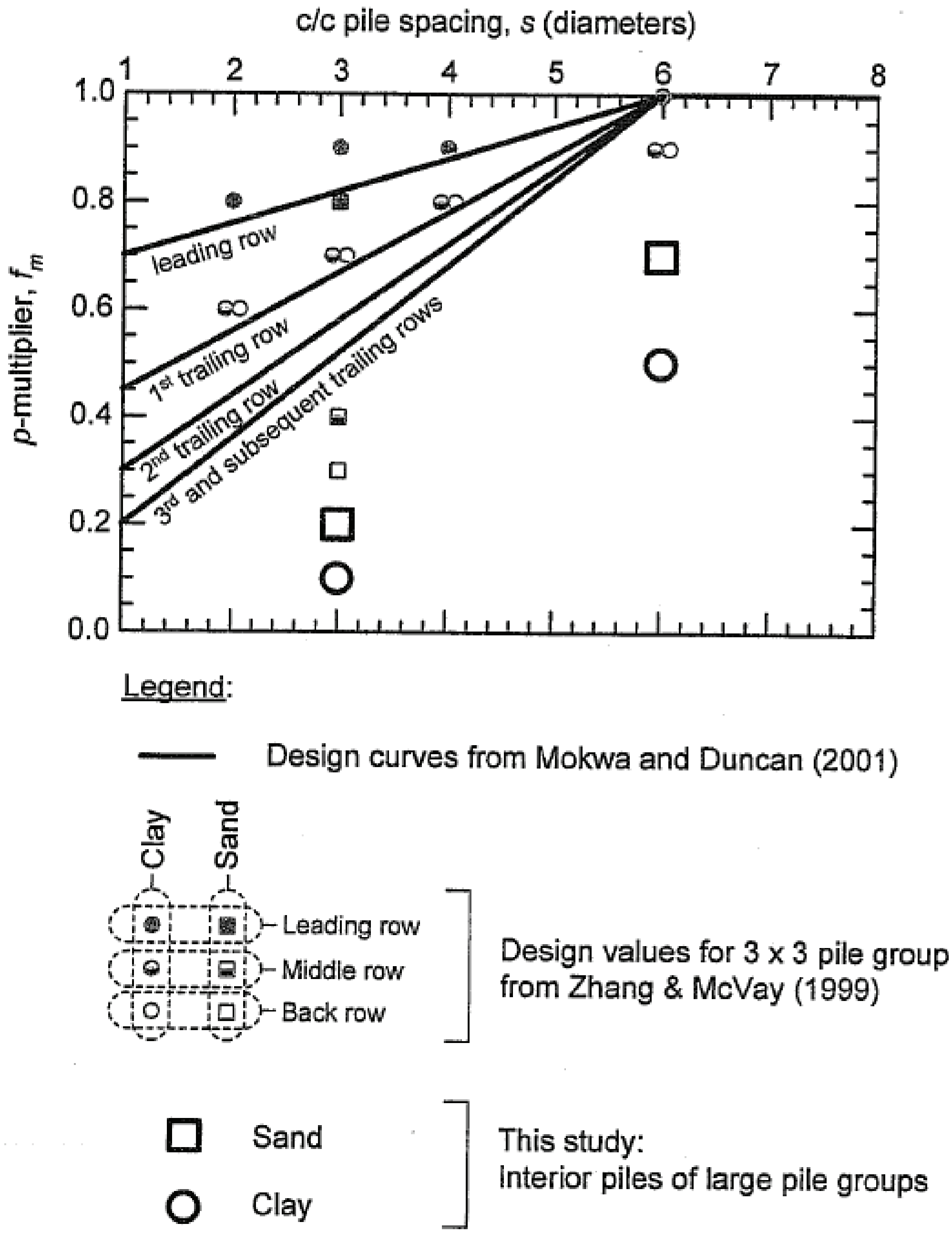


Figure 3-7 Recommended p-Multiplier Design Values for the Translational Mode of Lateral Resistance of Large Pile Groups

3.3 Foundation Damping

The conventional p-y curves developed for piles would have non-linear and inelastic behavior. When these non-linear p-y curves are implemented for cyclic loading allowing for hysteretic behavior in a time history based dynamic analyses, the strain-dependent energy dissipation mechanism is automatically simulated, accounting for material damping in the soil-pile foundation system.

For additional damping associated with radiation of energy through an elastic half space, elasto-dynamic solution has been recommended by Gazetas et al. (1992). The classical method of solving soil-pile interaction problems involving radiation damping has been based on a elasto-dynamic approach in frequency domain (Mylonakis, et al., 1997 and Gazetas et al., 1992). Because of the linear superposition principle in the frequency domain, soil stiffness is usually expressed as Winkler type of linear spring which would have a unit of FL^{-2} (soil resistance per unit pile length per unit pile deflection). The radiation viscous dashpot parameter per linear length of the pile can be derived from shear wave velocity using the following equation

$$c_x = 1.6\rho_s v_s(\omega d/v_s)^{-1/4}$$

Where c_x is the damping coefficient, ω is the angular frequency, d is the pile diameter, ρ_s is mass density and v_s is shear wave velocity. This radiation damping parameter would be frequency dependent.

Care is needed in mixing the conventional nonlinear p-y curves with the viscous coefficient to account for radiation damping that is based on elasto-dynamic approach. Recent centrifuge pile load tests conducted at the University of California at Davis to simulate transient earthquake loading indicated that the use of conventional API type of p-y curves yielded reasonable soil-pile interaction stiffness especially for clayey soils (e.g., Boulanger et al., 1999). However, viscous dashpots based on elasto-dynamic approach using small strain shear wave velocity arranged in parallel with the p-y springs resulted in exaggerated radiation damping solutions (e.g., Wang et al., 1998). Although the most common way to implement radiations dashpots is in a parallel manner for soil-pile interaction analysis, researchers at the University of California, Davis recommend that the dashpot be arranged in series with the p-y springs (see Figure 3-8 for different arrangement of dashpots). This kind of arrangement ensures that forces acting at the viscous dashpot would be transmitted through the p-y springs; thereby reducing the effect of the viscous damper. It has been found in most cases encountered, when the radiation dampers are arranged in series with near-field p-y springs, the p-y springs dominate the pile behavior, and the radiation dampers can be neglected.

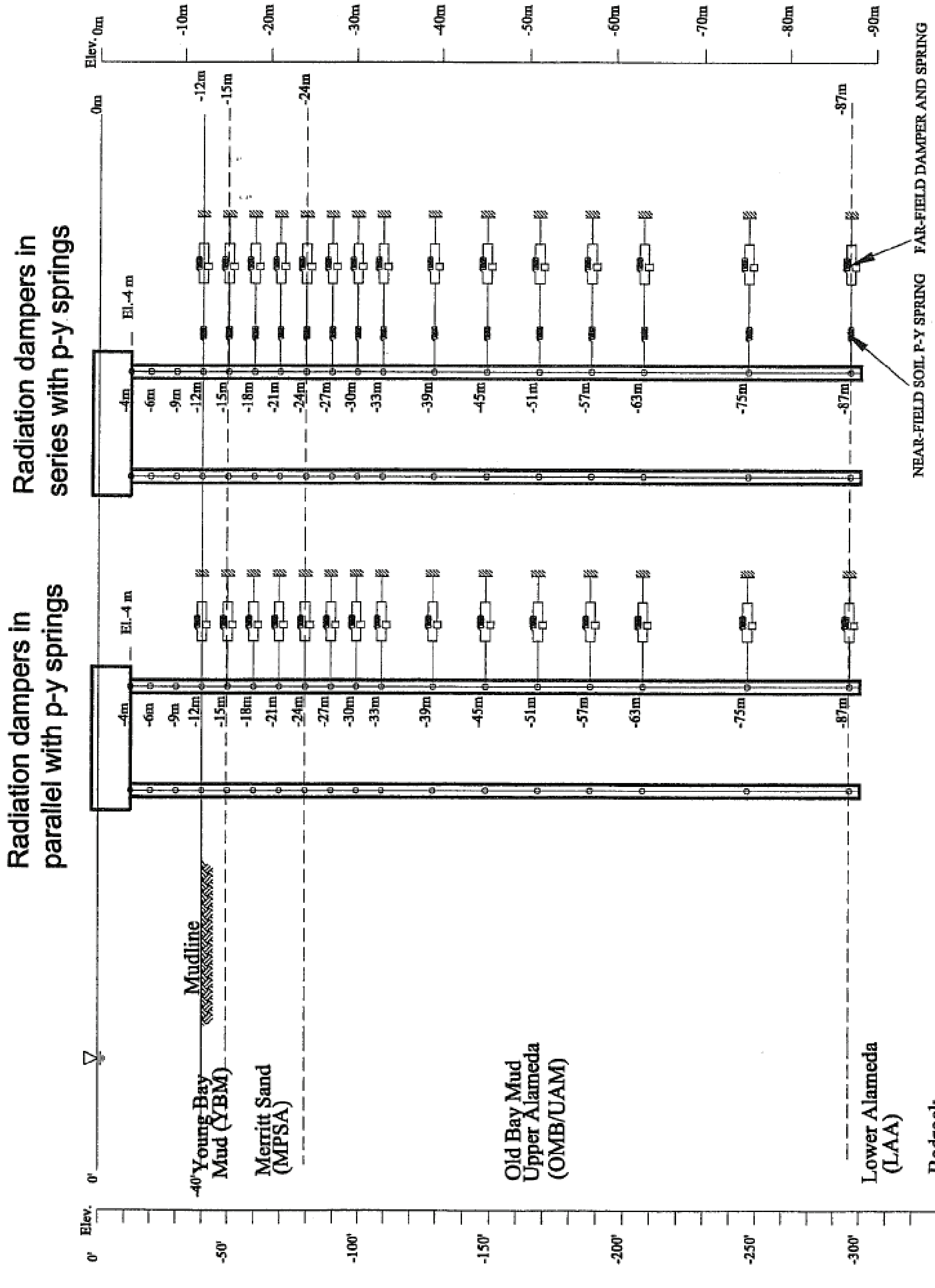


Figure 3-8 Dashpots Representing Radiation Damping Arranged in Parallel and in Series with the Near-field Nonlinear p-y Springs

3.4 Pile Group Stiffness Matrices

In many long span bridge projects, substructuring is used to reduce the size of the problem. The substructuring technique involves modeling the pile foundation to a convenient interface with the superstructure, e.g., at the base of the pile cap as shown in Figure 3-1. Static condensation was then used to derive the appropriate foundation substructure stiffness and the effective ground motion transmitted to the superstructure; the resultant effective ground displacement is termed the kinematic ground motion (Lam and Law, 2000) and is discussed in Section 3.5. The foundation stiffness matrix is used in conjunction with the kinematic ground displacement is represent the entire foundation system in the superstructure analysis.

When the substructuring technique is used to simplify the foundation in the global bridge model, the pile group can be represented by a linear 6x6 stiffness matrix, and the approach has been described by Lam and Martin (1986). Since the p-y curves are nonlinear, the first step towards developing the foundation stiffness matrix involves linearization of the p-y curves by performing lateral pushover analyses of the single pile to a representative displacement level expected during the earthquake. Alternatively, other simplifying assumptions may be used, such as that used by Terzaghi where the secant stiffness of the p-y curve was recommended at half the ultimate capacity. Once the p-y curves are linearized on the basis of lateral pushover analysis, the problem becomes beam on elastic springs, and the method of substructuring was applied to obtain the condensed stiffness matrix. It typically requires one or two iterations between the substructuring process for the foundation and the global bridge analysis, so that the displacement level assumed for linearization of soil p-y curves is consistent with the displacement demand computed by the global bridge model in the seismic response analysis.

The pile-head stiffness matrix of a single pile, computed using static condensation, has the following form.

$$[K_{single}] = \begin{bmatrix} k_x & 0 & 0 & 0 & 0 & 0 \\ 0 & k_y & 0 & 0 & 0 & k_{y\theta z} \\ 0 & 0 & k_z & 0 & -k_{z\theta y} & 0 \\ 0 & 0 & 0 & k_{\theta x} & 0 & 0 \\ 0 & 0 & -k_{y\theta z} & 0 & k_{\theta y} & 0 \\ 0 & k_{z\theta y} & 0 & 0 & 0 & k_{\theta z} \end{bmatrix}$$

The coordinate system for an individual pile is shown in Figure 3-9 in relationship to the global pile group coordinate system, and k_x, k_y, k_z, $k_{\theta x}$, $k_{\theta z}$ are stiffness coefficients of translational and rotational degrees of freedom in the local pile axis. The global stiffness of the pile group is computed by the summation of individual pile-head stiffness, invoking static equilibrium. The procedure, which can be applied to any number of piles and geometric pile group configuration, assumes that the pile cap is infinitely rigid.

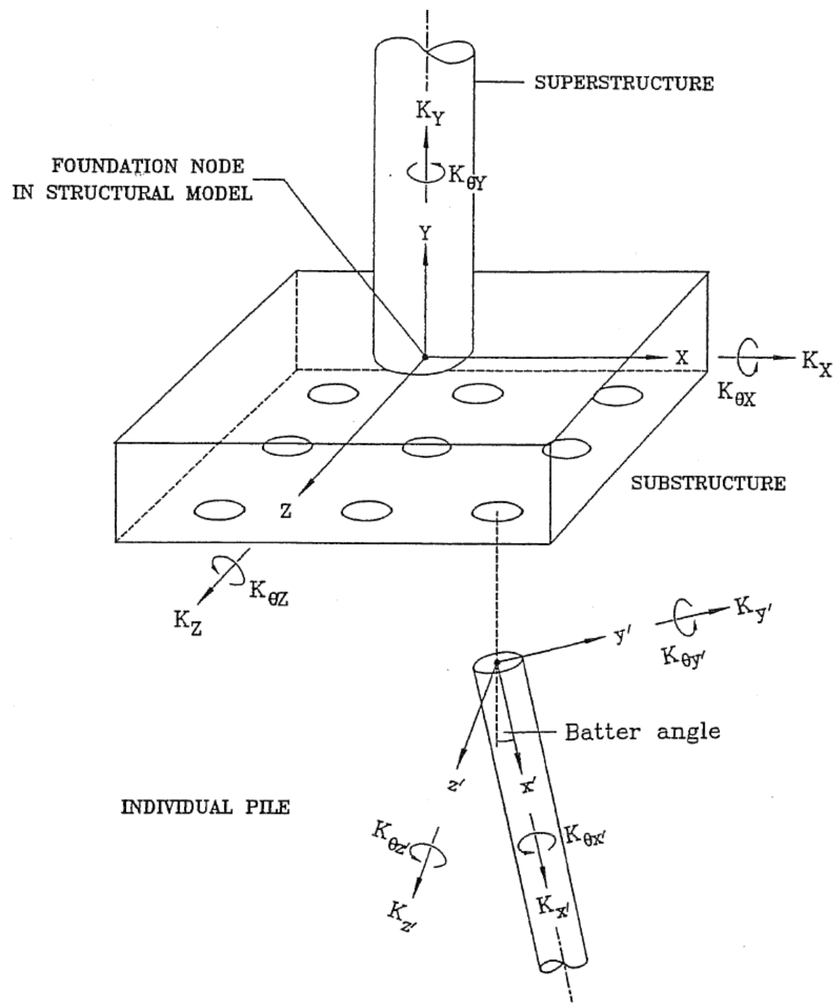


Figure 3-9 Coordinate Systems for Individual Pile vs. Global Pile Group

For a vertical pile group, the form of pile group stiffness matrix will be identical to the individual single pile. The pile group stiffness for the translational displacement terms (two horizontal terms and one vertical term) and the cross-coupling terms can be obtained by merely multiplying the corresponding stiffness components of the individual pile by the number of piles. However, the rotational stiffness terms (two rocking terms and one torsion term) require consideration of an additional component. In addition to individual pile-head bending moments at each pile head, a unit rotation at the pile cap will introduce translational displacements and corresponding forces at each pile head. These pile-head forces will work together among the piles and will result in an additional moment reaction on the overall pile group. The following equation can be used to develop the rotational stiffness terms of a vertical pile group (Lam, et al., 1991):

$$K_\theta = \sum_{piles} k_\theta + \sum_{piles} k_\delta R^2$$

where K_θ and k_θ are rotational stiffness of the pile group and an individual pile, respectively, k_δ is the appropriate translational axial stiffness coefficient of an individual pile, and R is the distance between the individual pile and the center of the pile group. For a general case, the stiffness matrix is a full matrix representing coupling among various displacement terms.

The following examples illustrate the use of a substructuring approach. A pile group at the Richmond – San Rafael Bridge is an example of full stiffness matrix due to a large number of battered HP piles arranged in circular patterns (EMI, 1999). The configuration of the pile group is shown in Figure 3-10 showing a total of 308 piles. The directions of strong axis for these HP piles are oriented tangential to the circumference of circular pattern. The stiffness matrix of this pile group is presented in Table 3-2.

Table 3.4 Pile Group Stiffness Matrix of at Richmond – San Rafael Bridge

Translation			Rotation		
Dis-X	Dis-Y	Dis-Z	Dis-X	Dis-Y	Dis-Z
1.97E+06	-1.89E+00	-3.82E+00	2.30E+03	1.29E+08	-2.83E+04
-1.89E+00	1.99E+06	-7.61E-01	-1.87E+08	7.09E+01	-2.75E+03
-3.82E+00	-7.61E-01	2.19E+07	-7.79E+01	3.92E+02	-2.37E+03
2.30E+03	-1.87E+08	-7.79E-01	4.80E+12	-2.43E+05	1.28E+06
1.29E+08	7.09E+01	3.92E+02	-2.43E+05	1.31E+12	2.49E+06
-2.93E+04	-2.75E+03	-2.37E+03	1.28E+06	2.49E+06	5.91E+11

Note: the coordinate system used in this table is shown in Figure 3-10 where x- and y- axes are horizontal directions and z- axis is the vertical direction. Units: lb, in, rad

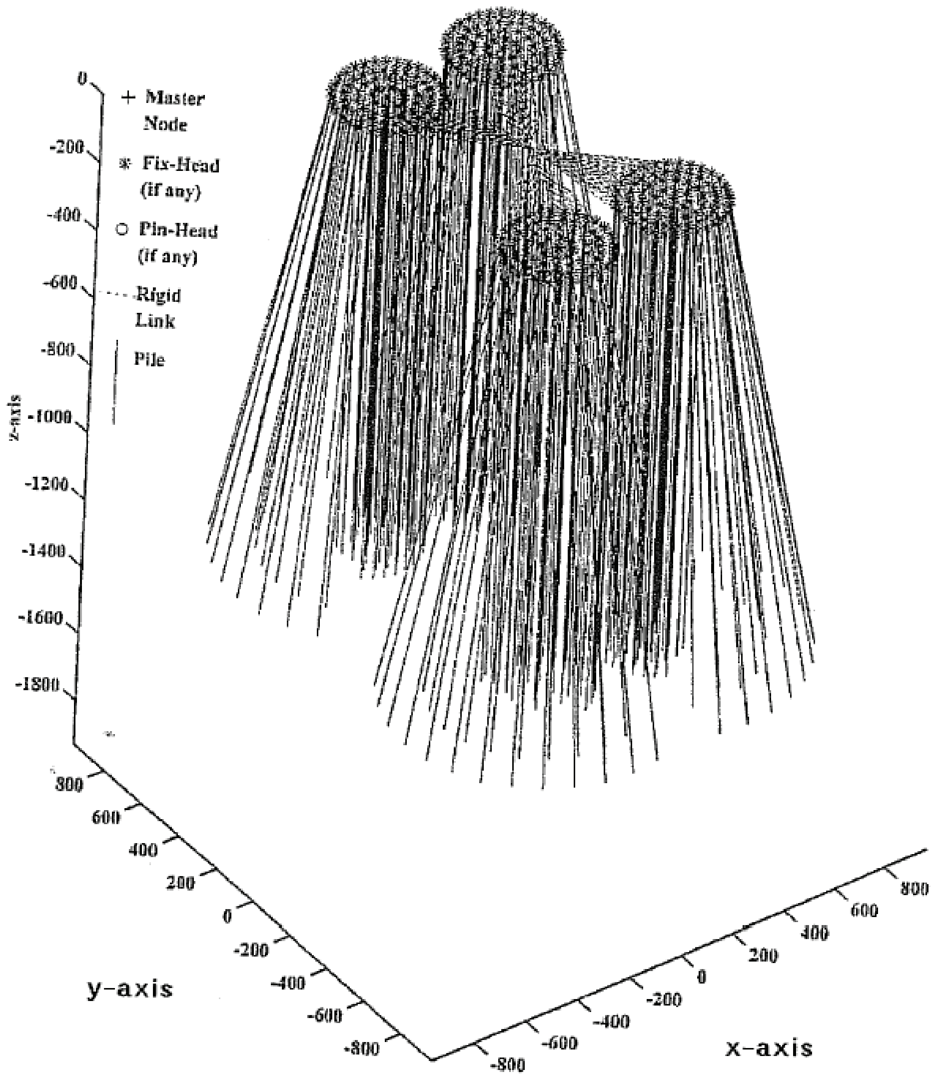


Figure 3-10 Isoparametric View of the Pile Group Model at Richmond-San Rafael Bridge

Since the bridge has over 60 piers, it was not feasible to employ the complete system to include every individual pile and soil support in the global bridge model. For this situation, the substructuring approach is highly suitable to formulate the problem into a manageable size.

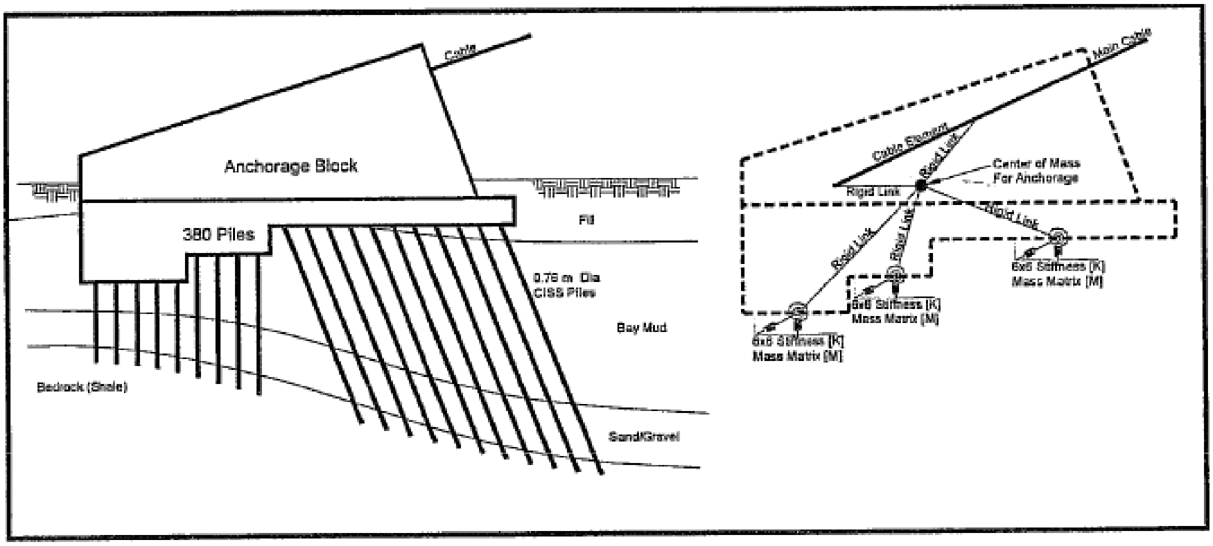


Figure 3-11 South Anchorage Foundation for New Carquinez Bridge

Figure 3-11 shows the pile group layout at the south anchorage of the new Carquinez Suspension Bridge. The anchorage block, which provides an anchoring point for the suspension cables, has a three-step bottom founded on 380 piles, each a 0.75 m diameter Cast-in-Steel-Shell (CISS) pile. These piles are driven to bedrock through soft bay mud. The bedrock is located at variable elevations; the difference in bedrock elevations is as much as 10 meters within the footprint of the foundation. Due to the large size of the anchorage footprint and the variability of the subsurface conditions, the piles were divided into six groups. Each group was represented with a condensed mass matrix and a condensed stiffness matrix. In the global bridge analyses, these six sets of stiffness and mass matrices were rigidly linked together to form the foundation model. After displacement demand son the anchorage were obtained from the global analyses (six degrees of freedom displacement), they were then back-substituted into the foundation substructure to recover individual pile loads by performing pushover analyses. This approach substantially reduced the total number of degrees of freedom needed to model the suspension bridge structure.

As part of the seismic retrofit program for the San Diego – Coronado Bay Bridge, substructuring of individual piles has also been used for the time history analyses of the global bridge. The foundations for this bridge consist of prestressed concrete piles of a 1.4-meter diameter and a 122-mm wall thickness driven into dense sands. Each foundation is supported on 12 to 44 piles in a group, and the piles have substantial cantilever lengths above the mudline. Potential plastic hinging of the piles at the pile-cap connection point was of concern, and needed to be addressed in the global time history analyses. Therefore, the pile segment above the mudline was included in the global bridge model; the portion of the piles below the mudline was represented by a 6x6 stiffness matrix and mass matrix, as shown in Figure 3-12. This modeling technique was found to be very effective in addressing plastic hinging in the piles, and is a compromise between the full substructure model and the complete model approaches.

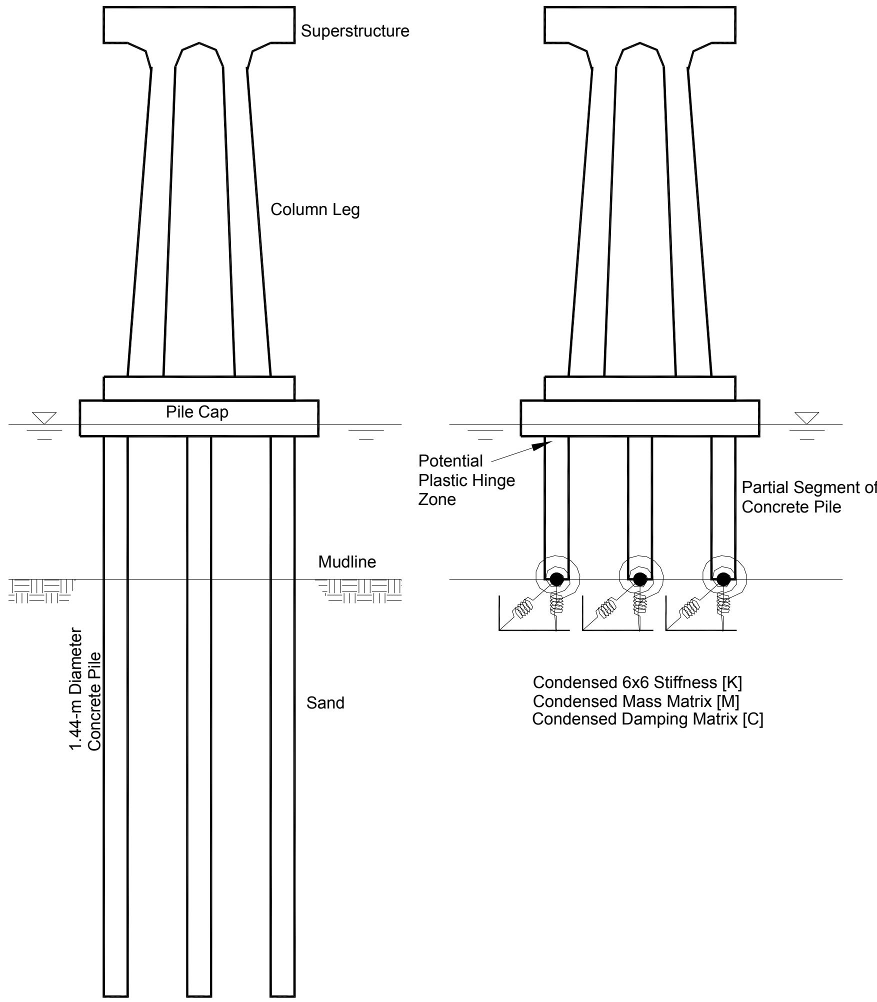


Figure 3-12 Modeling of Piles for San Diego Coronado Bridge

It is noteworthy to discuss errors that can arise when engineers use commercial computer programs without understanding the basis mechanics of formulating pile group foundation stiffness matrices. For example during the early stage of seismic retrofit of the San Diego – Coronado Bay Bridge, stiffness matrices of pile foundations were computed using the program GROUP, which allows nonlinear p-y curves and computes stiffness coefficients for each mode of loading individually. The stiffness coefficient for the lateral translational was computed by applying lateral loading on the piles, and then separate analyses were conducted to compute the stiffness coefficient for rotation by applying moment on the pile top. The group stiffness matrix was later constructed by assigning the individual components which were derived separately from an operation on non-linear soil springs. When this set of stiffness matrices was used in the global bridge model, the structural designers encountered difficulties in convergence of the solutions. It was determined that some of these stiffness matrices were non-positive-definite that is analogous to soil springs having a negative stiffness value. This problem was introduced because of non unique p-y springs implied by separate GROUP analyses. The consequence of non-positive-definite stiffness matrix is that the system would generate energy (instead of dissipating energy) that is physically impossible. The problem can be eliminated if p-y springs are linearized first before stiffness coefficients are computed.

3.5 Kinematic Soil-Pile Interaction

Discontinuity in the material properties would have a significant influence on earthquake wave propagation through the medium. Any incident wave arriving at the discontinuous boundary separating two media would result in reflected and refracted waves (wave scattering), which may be both P-wave and S-wave depending on the incident angle. For a given incident wave, the amplitude, the unit propagation vectors and wave number of the reflected and refracted waves can be computed from the conditions of the displacement and stress at the interface between the two media. While closed form solutions exist for simple plane harmonic waves in an elastic half-space, for general solutions in the case of earthquake loading, finite element methods are used.

Inclusion of a large number of piles in the ground would increase the effective modulus within the footprint resulting in a stiffness contrast between the free-field soil and the foundation region. From a classical wave propagation theory, there would be wave refraction and reflection of P- and S-waves where the stiffness contrast between two media is present. This phenomenon is usually referred to as wave scattering. Although wave scattering has been considered, for example in the design of nuclear containment systems, the current state of practice for seismic design of bridges normally does not consider this effect. The overall effect of wave scattering is to alter the free-field earthquake ground motions used as input to study the response of the bridge-foundation system.

The study summarized below describes wave scattering related to large pile groups, often employed in long span bridges. A more detailed description of these studies is given by Lam et. al. (2004). It is known that wave scattering effects can be important for very short period structures. The significance of wave scattering depends on the wavelength of the media and physical dimensions of the structures. For a given foundation size, long period waves (long wavelength) tend to excite the structure in a more synchronized mode, while short period waves (short wavelength) give rise to out-of-phase motions.

For large pile groups, especially those foundations supporting long span bridges, footing sizes up to 150 ft x 150 ft are not uncommon. Pile group data from some of the long span bridges in California are summarized in Table 3-2 highlighting ranges of foundation sizes. Figures 3-13 to 3-16 illustrate the shapes and dimensions of pile foundations supporting representative existing bridges. Since these large foundations are often used in poor ground conditions where wave propagation speed is low, wavelengths implied by the soil would be relatively short.

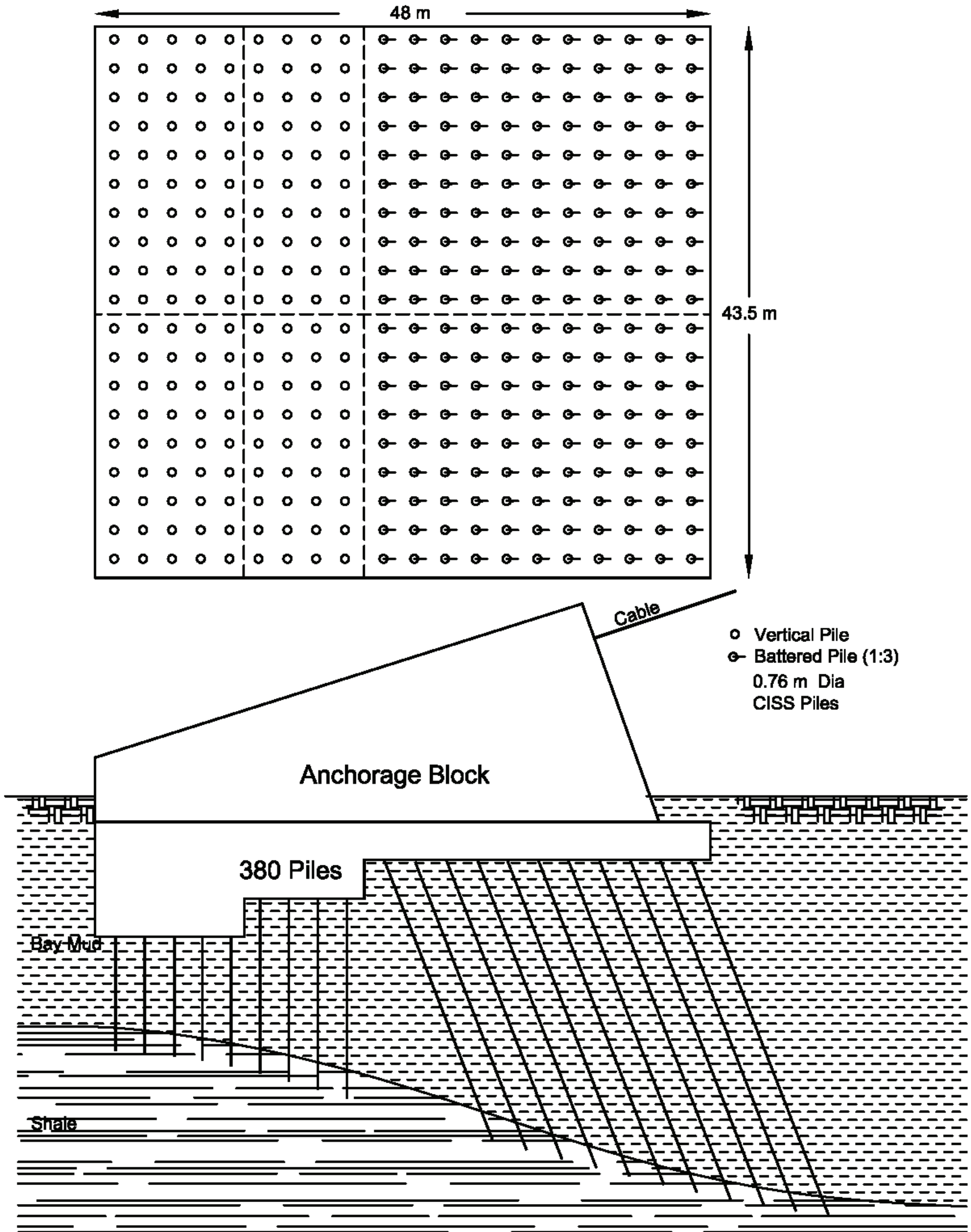


Figure 3-13 South Anchorage Foundation of New Carquinez Bridge, CA

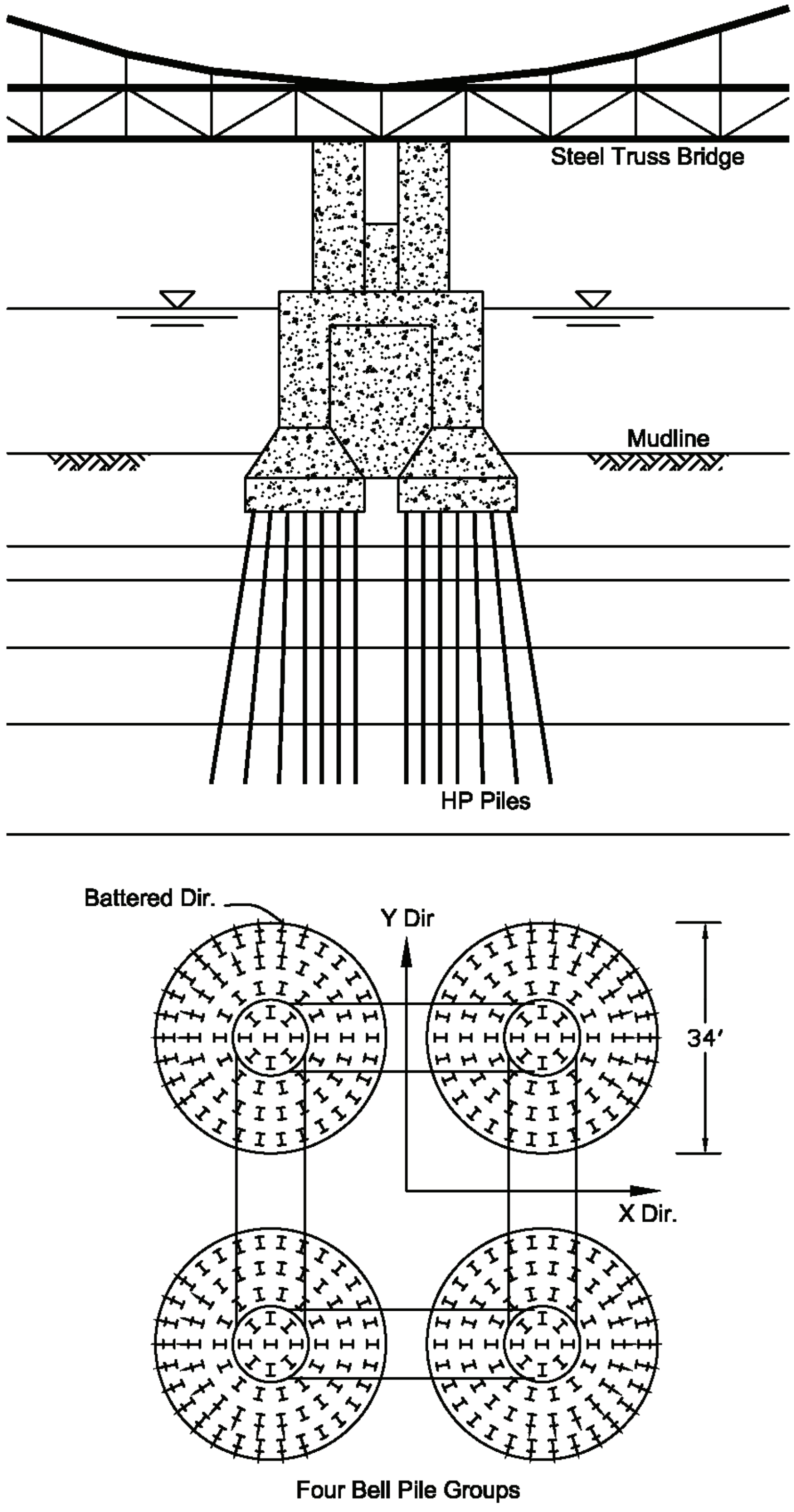
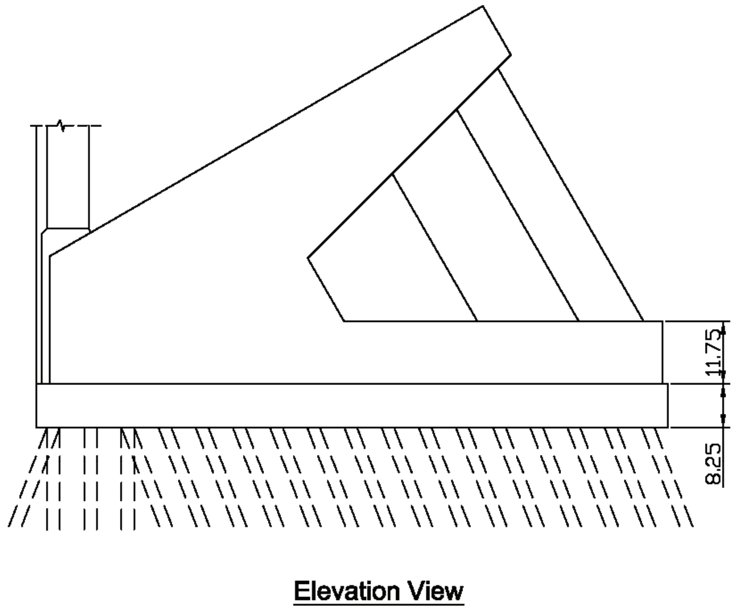


Figure 3-14 Typical Foundation of Main Span at Richmond-San Rafael Bridge, CA



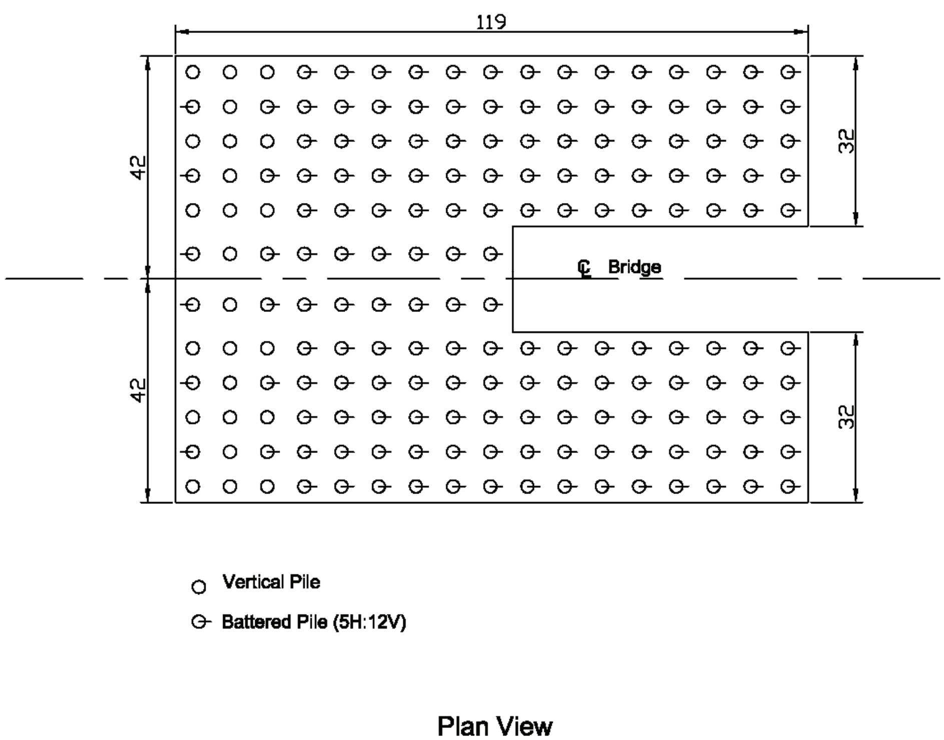


Figure 3-15 Foundation Plan at Anchorage of Vincent Thomas Bridge, CA

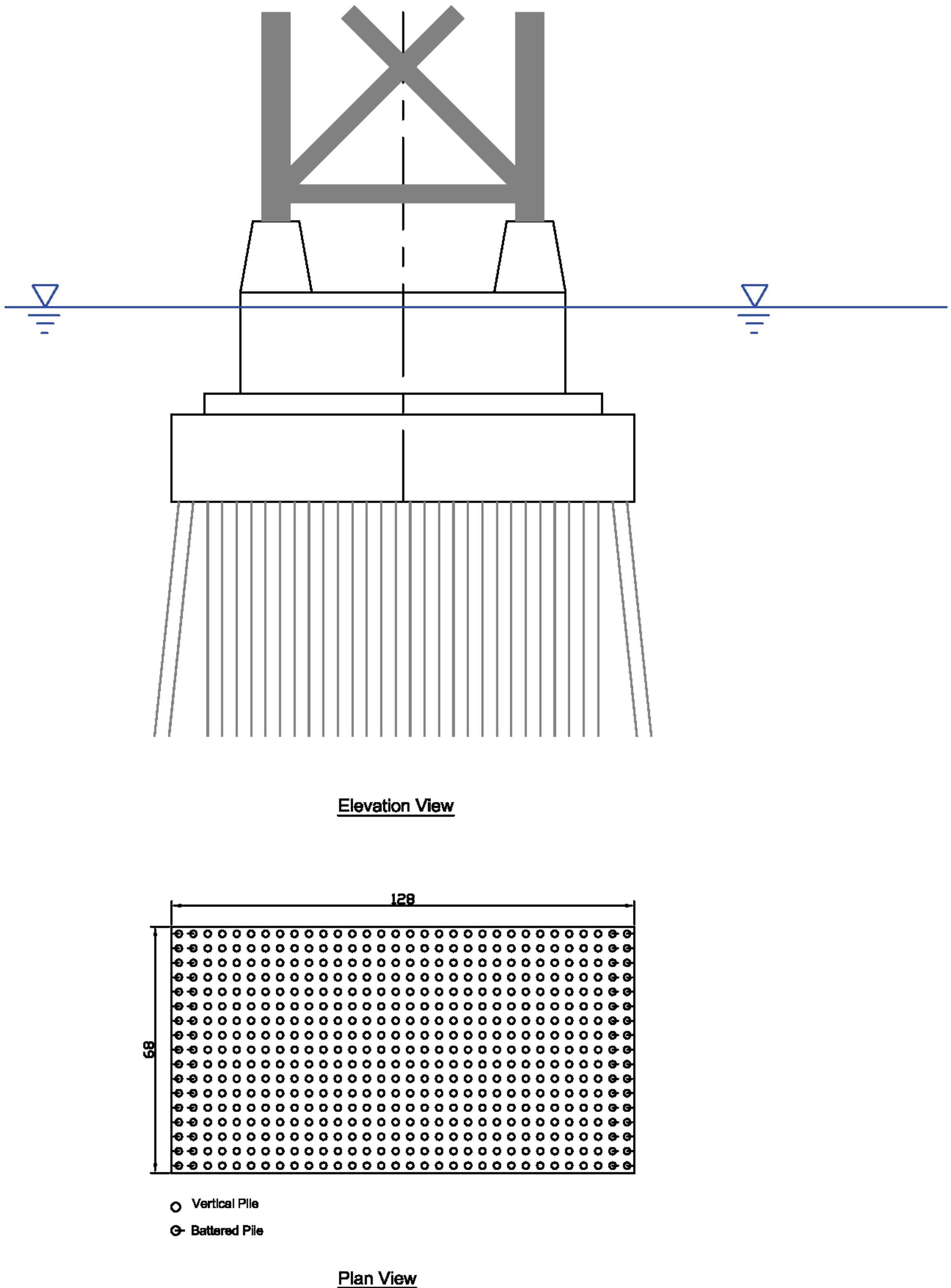


Figure 3-16 Bent Footing Supporting East Span (Existing)
San Francisco – Oakland Bay Bridge, CA

3.5.1 Implementation in a Time Domain Approach

To implement the wave scattering model in a time domain approach, a hypothetical foundation consisting of 3 ft diameter concrete piles arranged in a 7 x 7 grid pattern was established. Figure 3-17 shows a schematic of the hypothetical pile foundation used in this study. The piles extend 65 feet below the ground surface. The piles occupy about 8 to 12% of the soil volume under the footprint of the pile cap, and the overall modulus of the soil-pile composite medium is taken as

$$E_c = a\, E_p + (1 - a)\, E_s$$

where E_c = modulus of soil-pile composite, E_p = modulus of pile, E_s = modulus of soil, and a = percentage of volume occupied by the piles within the foundation footprint.

The wave scattering for this pile foundation problem was first analyzed with the frequency domain based computer program, SASSI. The results from SASSI served to provide benchmark solutions, which are used to compare with the time-domain solutions using the finite element program ADINA. Although SASSI is not regularly used for bridge designs, it has been used in the nuclear industry to study soil-structure interaction for nuclear containment systems. The program SASSI was developed at the University of California, Berkeley under the direction of Late Professor J. Lysmer to solve soil-structure interaction problems. It is formulated in the frequency domain, using the complex frequency response method based on the assumption of linear elastic properties. In this approach, the linear soil-structure interaction problem is subdivided into a series of simpler sub-problems. The program solves each sub-problem separately and combines the results in the final step to provide the complete solution.

Figure 3-18 shows a conceptual SASSI model to study the scattering problem of the pile group. The control motion for this model is provided at the surface as a free field reference motion (away from the pile group); only the horizontal component is considered here. A viscous boundary at the base of the model is used to simulate a halfspace. The side boundary conditions are handled by using an exact analytical solution. Figure 3-19 shows the computed time histories and their response spectra at three selected locations within the foundation footprint along the centerline.

Analyses were made to simulate this pile foundation problem in the time domain using the program ADINA. Figure 3-20 depicts the finite element model of the pile foundation problem with a rigid base. The side boundaries were treated by slaving the two side boundaries. To supply input base excitation to the model, the 'within' motion computed from SHAKE in the deconvolution analysis was used as the rigid base motion. Only a mass proportional damping coefficient (α) was used to represent the damping. The solutions obtained from ADINA are provided in Figure 3-21 showing acceleration time histories and response spectra at the three locations.

Comparison between SASSI and ADINA can be seen in Figure 3-22 in terms of near-field response spectra for this pile foundation problem. This comparison is satisfactory, considering rather different methodologies taken by the two computer programs. The side boundary in the current ADINA model is located about 1.2 times the thickness of the soil deposit measured from the centerline of the model. Based on the comparison of the results, it appears that distance to the side is sufficiently far from the foundation.

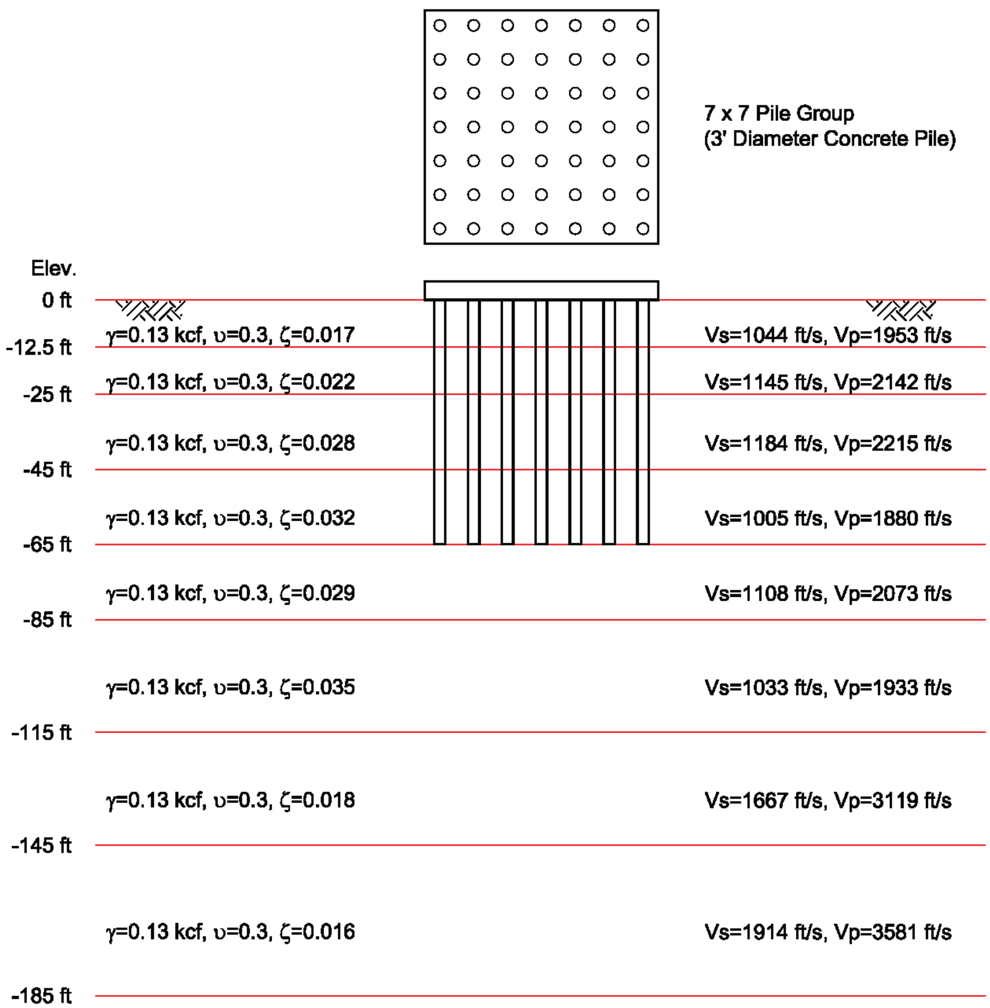


Figure 3-17 Hypothetical Pile Foundation and Soil Strata

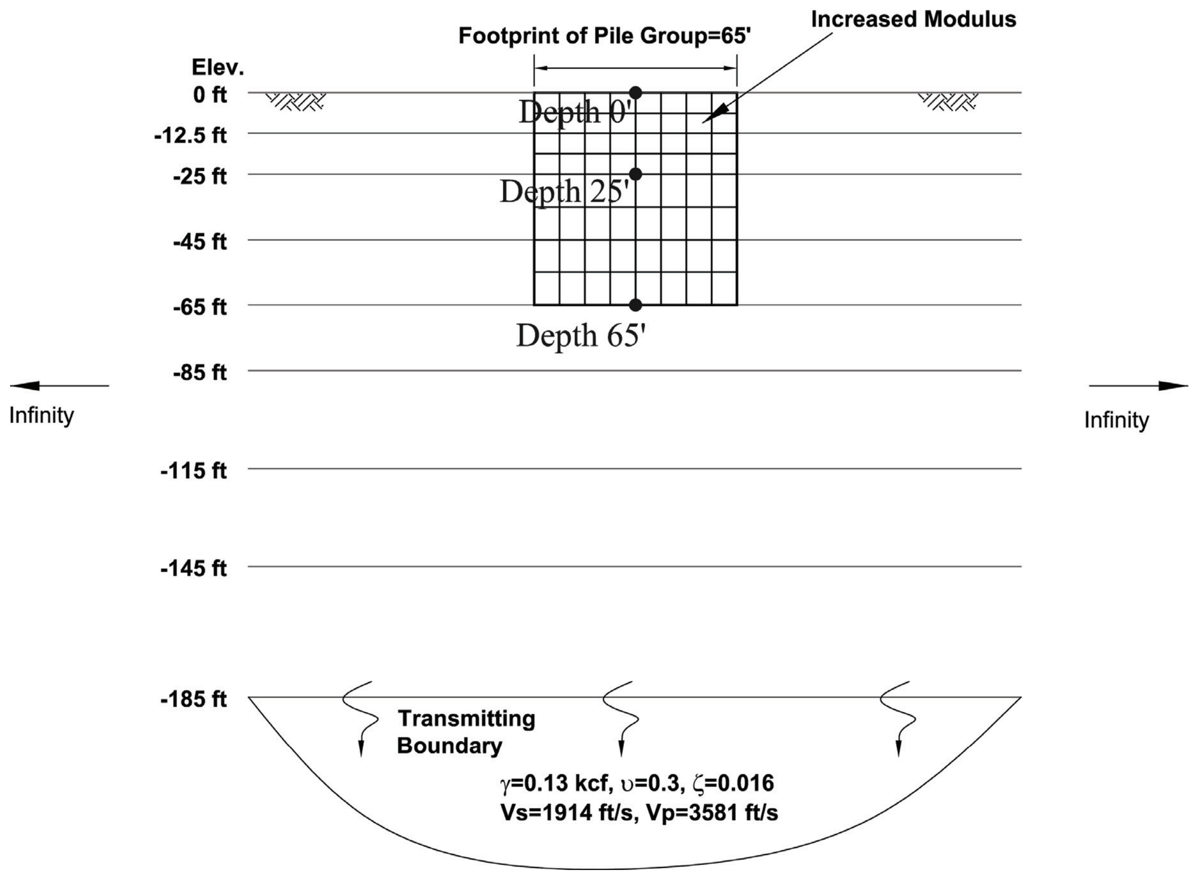


Figure 3-18 Conceptual SASSI Model of Pile Group

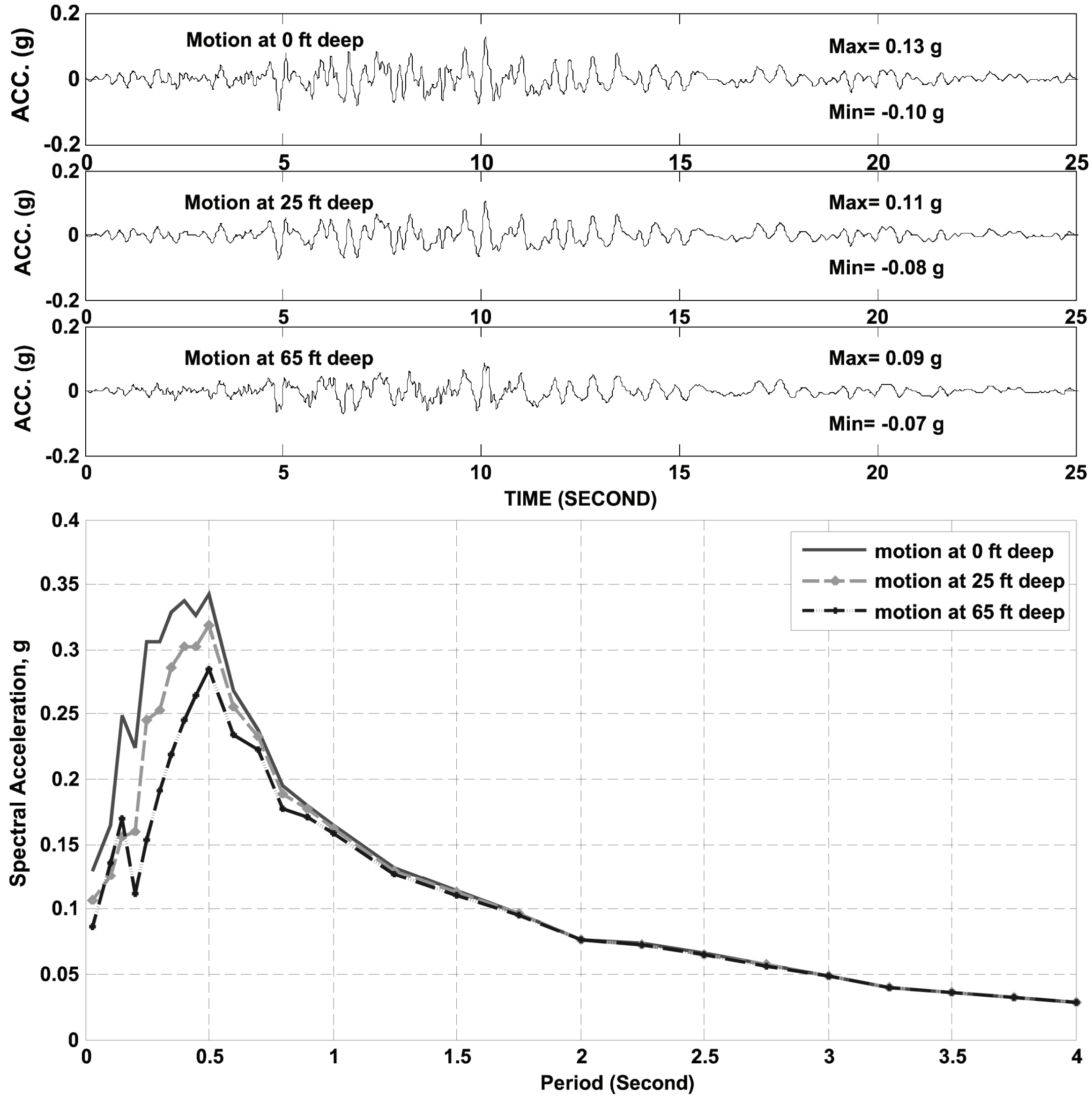


Figure 3-19 SASSI Results of Computed Time Histories and Response Spectra at Three Selected Locations

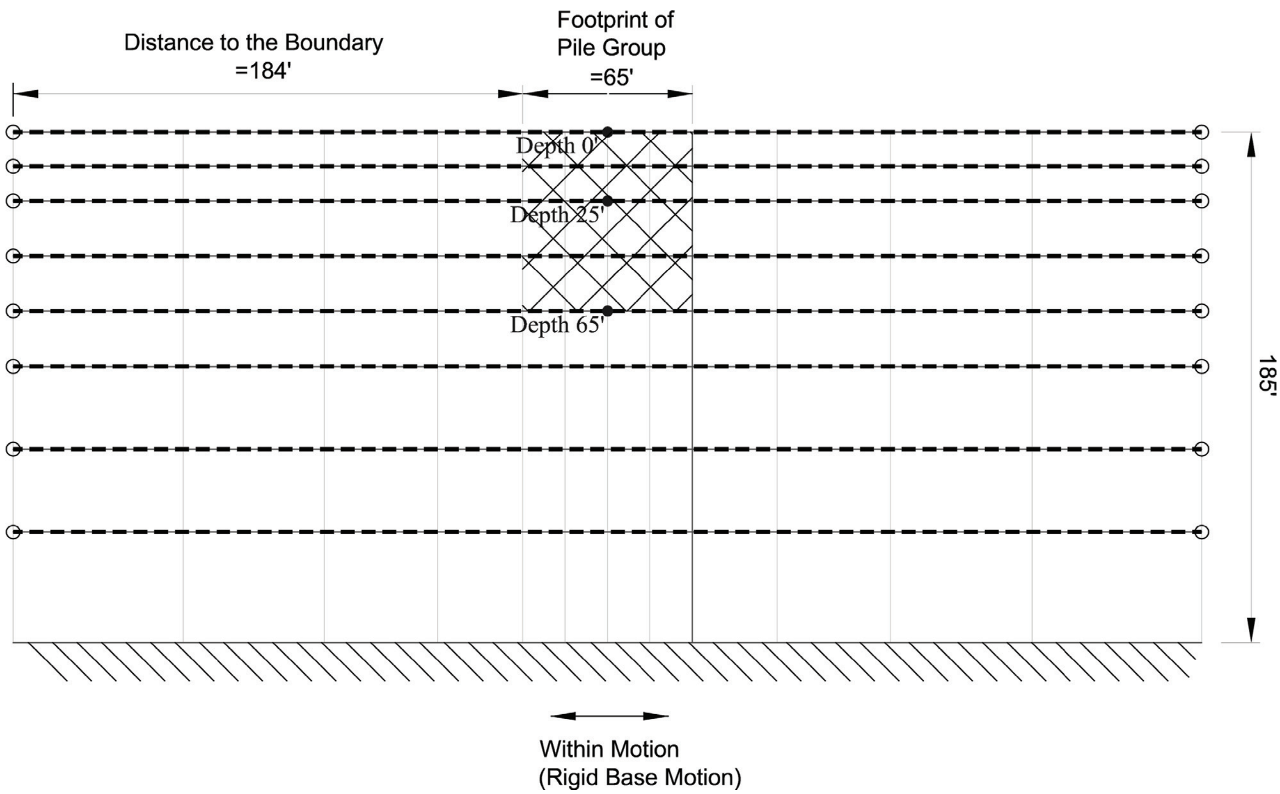


Figure 3-20 ADINA Pile of Model Group with Slaving of the Two Side Boundaries

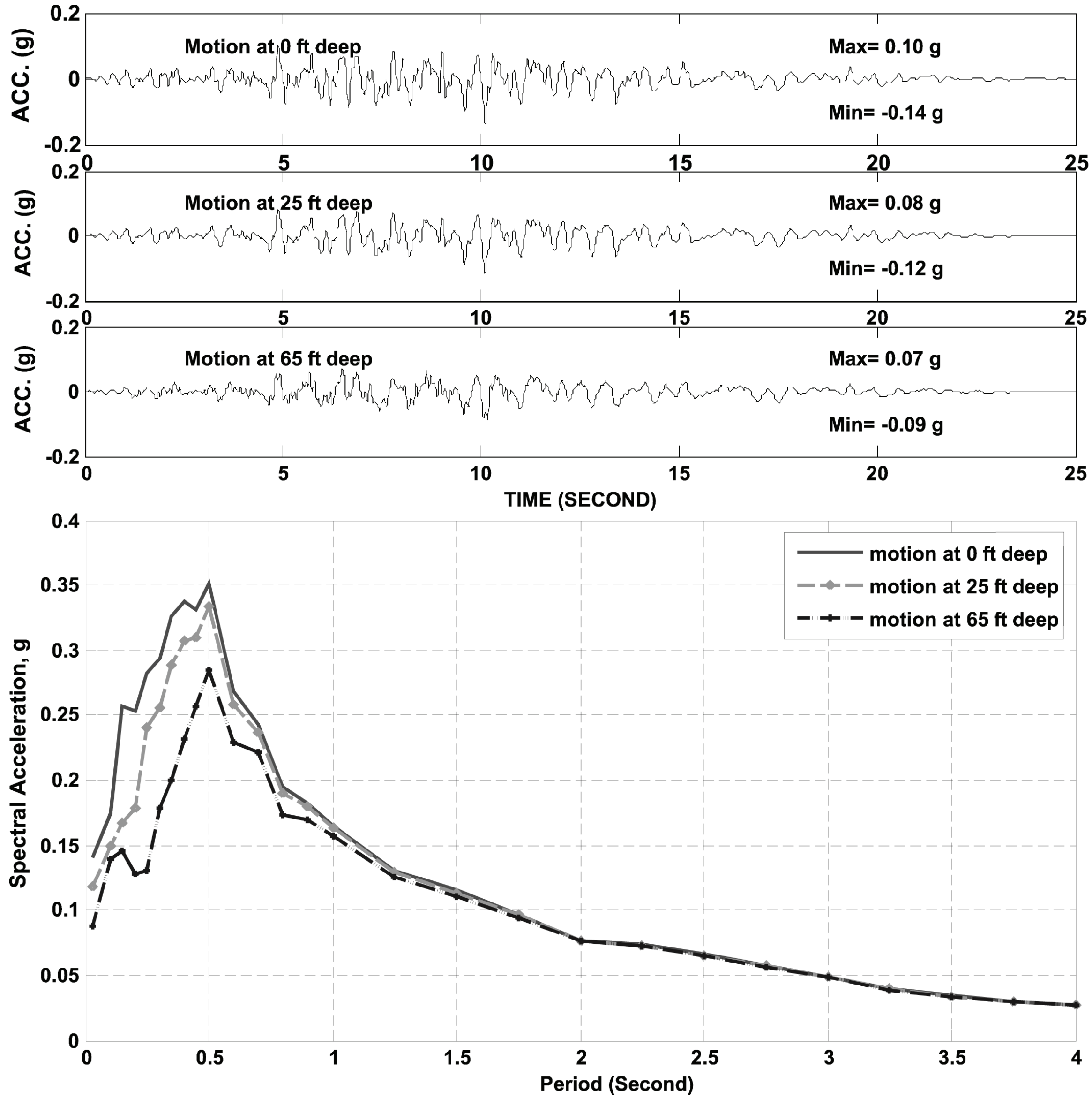


Figure 3-21 ADINA Results of Computed Time History and Response Spectra at Three Selected Locations

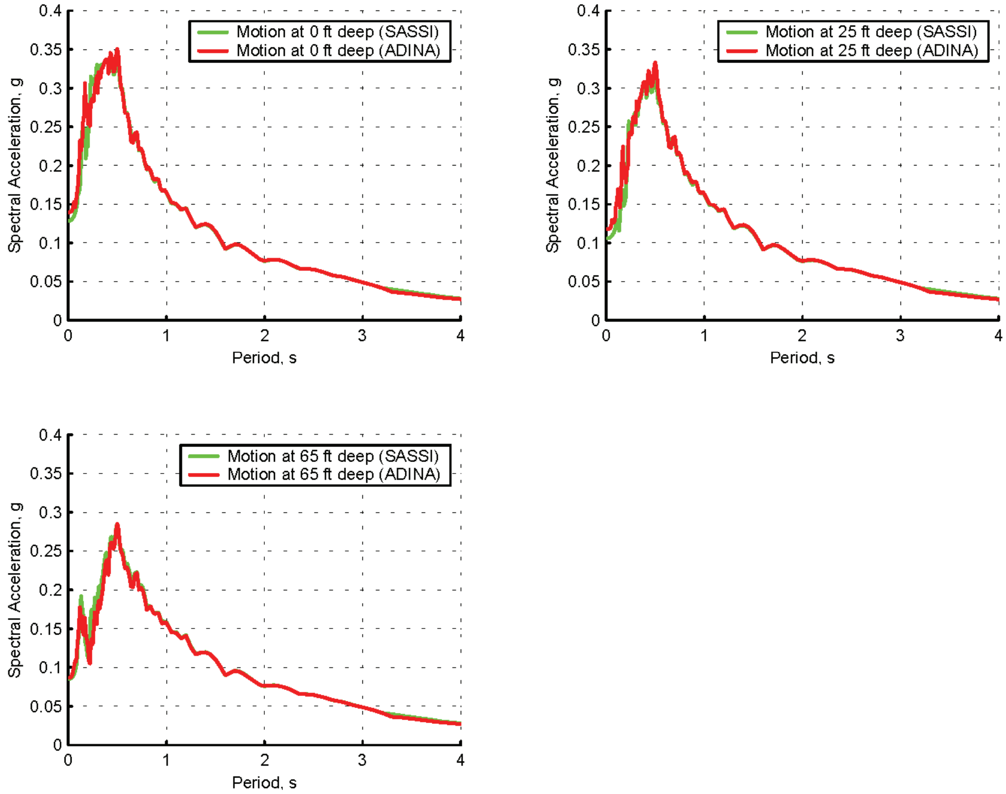


Figure 3-22 Comparisons Between SASSI and ADINA Response Spectra

3.5.2 Wave Scattering of Near-field Soil

The above study demonstrated that the time domain approach can be used to study the wave scattering mechanism affecting the near-field soils of the pile group. However the physical dimensions of the pile group and wave propagation speeds of the soil influence the wave scattering mechanism. To study the effects of foundation size, the model shown in Figure 3-23 was used where the width of the foundation was increased from 65 to 195 feet and the side boundaries are sufficiently far away from the foundation, even for the largest foundation size.

For a bridge superstructure, seismic shaking is transmitted from the soil to the structure throughout the embedded portion of the pile length. A site response analysis considering wave scattering is needed to compute near-field ground motions along the embedded pile length, which could vary dramatically at shallow depths. Multiple soil layers (soil springs) support the piles and the characteristics of near-field ground motions arriving at these soil layers could be significantly different from each other depending on the local soil conditions.

The program ADINA was used to compute depth-varying near-field motions along the pile length. Once the near-field motions were obtained, soil-structure interaction analyses were carried out to evaluate the resultant shaking acting at the foundation level. This is undertaken by applying depth-varying motions at the end of soil springs that are attached to the pile foundation, as shown in Figure 3-24. In this soil-structure interaction analysis, the mass of the structures is not included and only stiffness is considered. The resultant motion computed at the foundation level is termed the "kinematic motion" that effectively drives the bridge superstructure. The method of computing the kinematic motion is based on a linear theory using the substructuring procedure. The approach utilizing kinematic soil pile interaction has been fully implemented in the seismic retrofit as well as the new design for major toll bridges in California. A detailed formulation of the kinematic motion and the discussions can be found in a MCEER technical report (Lam and Law, 2000).

Since the kinematic motion has been derived from the depth-varying near-field motions imparting seismic loads to the pile, it may be considered as a 'weighted average' of the near-field motions within the significant soil-pile interaction zone, normally defined by the characteristic length of the soil-pile system. The kinematic motion defines the effective shaking intensity to the bridge superstructure and is used herein it to evaluate the degree of influence caused by the wave scattering. Instead of examining all the depth-varying near-field ground motions, observation of the resultant kinematic motion serves as a convenient means to compare different scattering scenarios.

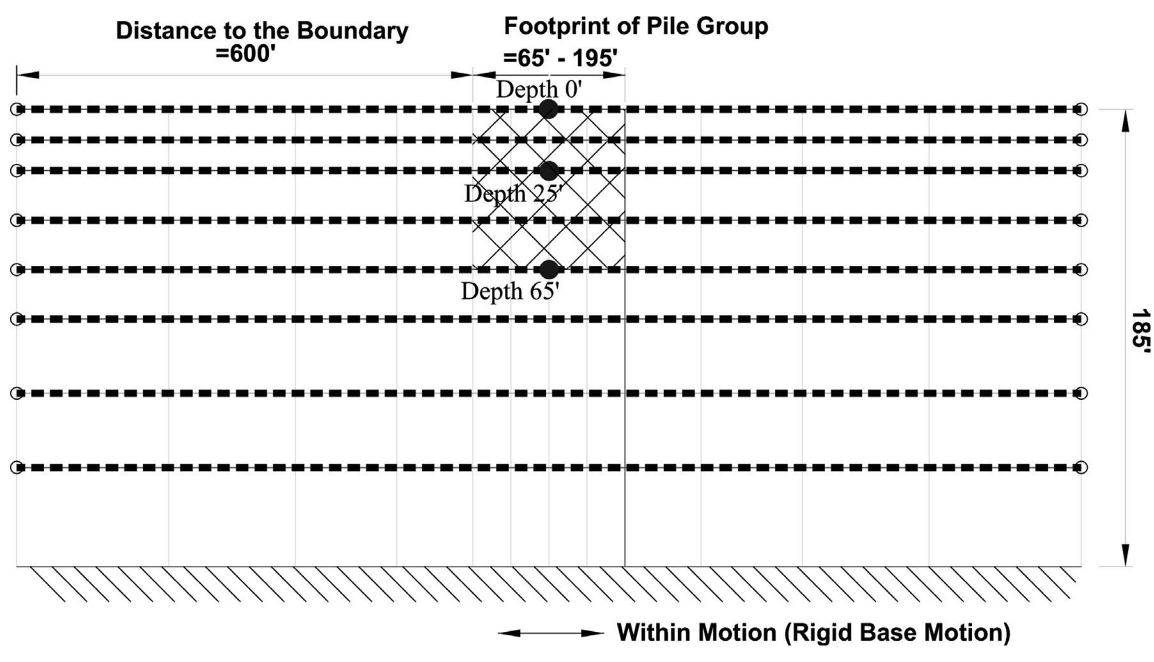


Figure 3-23 ADINA Model to Study Effects of Larger Pile Foundation Size

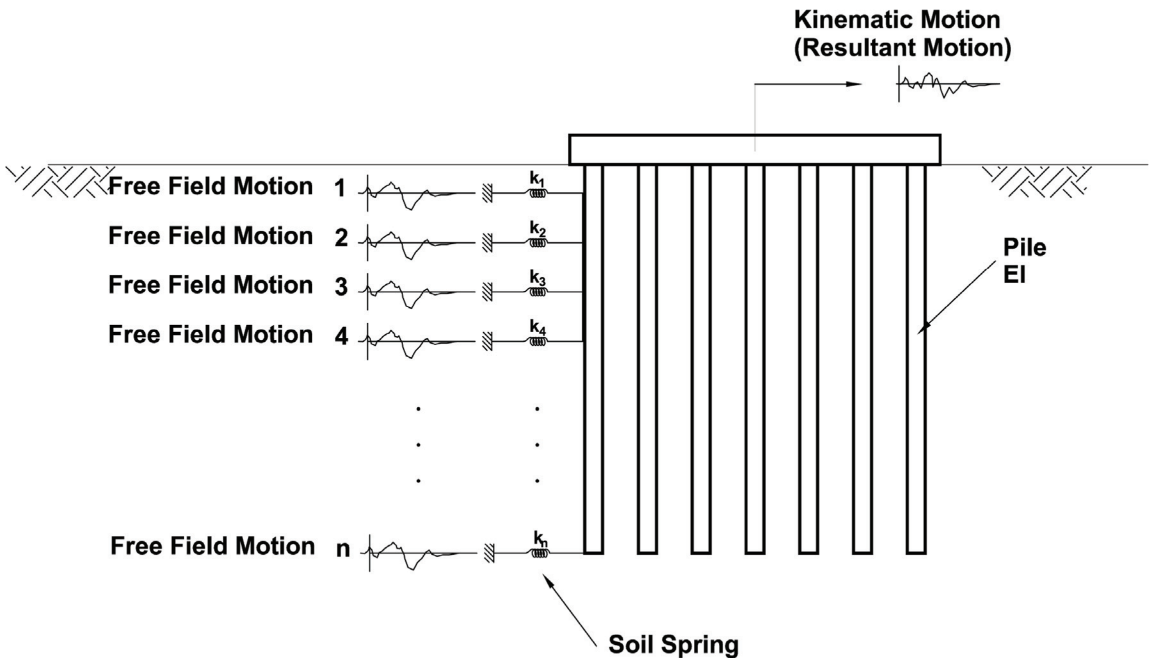


Figure 3-24 Schematic of Kinematic Motion Model

Figures 3-25 and 3-26 show depth-varying near-field motions in terms of time history and response spectrum plots computed from the program ADINA considering the wave scattering of the foundation. The foundation employed in this analysis is 65 ft wide representing a group of 7x7 piles. These depth-varying near-field motions, along with soil-structure interaction analysis results of the kinematic motion of the pile group are also compared to near-field soil motions in the figures.

To evaluate the influence of wave scattering, a comparison is made between the kinematic motions of the pile group with and without wave scattering. The solution that considers wave scattering is based on near-field ground motions computed from ADINA, while without wave scattering is derived from the free-field motions computed from the one-dimensional site response program SHAKE. A comparison is presented in Figure 3-27. Reviews of the two results suggest that wave scattering reduces the shaking level in a low period range (from zero to 0.5 sec) but it does not change the spectral acceleration beyond 0.5 sec. The prime reason for reduction of shaking intensity in the low period range (high frequency) is related to non-synchronized motions of short wavelengths resulting from the seismic scattering. These non-synchronized particle motions within the foundation footprint tend to cancel each other contributing to lower resultant shaking as compared to more synchronized particle motions from the free-field response.

Additional wave scattering studies were also performed using larger foundation sizes as the wave scattering tends to alter free-field ground motions for large foundations. The results from 130 feet and 195 feet wide foundations are also presented in Figure 3-27; minor additional reduction of shaking is observed for larger foundations.

Most long span bridges are characterized by long period modes of vibration. The reduction in shaking intensity due to wave scattering is primarily in short periods below 0.7 sec, and therefore the benefit of implementation of wave scattering in seismic design is very minimal. The current design practice of using one dimensional site response analysis to define near-field motions is adequate for practical purposes.

3.6 Case History Example

This Section presents a brief summary of two aspects of soil-structure interaction studies conducted by Earth mechanics, Inc. (EMI) conducted for the San Francisco-Oakland Bay Bridge East Span project.

The following topics are summarized:

- Site geology
- Seismic ground motion criteria
- Evaluation of site response and soil-structure effects

3.6.1 Site Geology

Figure 3-28 presents a plan view of the bridge alignment and a profile view of the bridge structure along with the geologic cross section. The new bridge can be divided into four segments (from west to east): (1) Yerba Buena Island (YBI) transition structure, (2) Self-Anchored Suspension (SAS) bridge, (3) Skyway Structure Viaduct, and (4) Oakland Mole approach structures. Foundations supporting most of the structures are pile groups. On YBI, spread footings were used at locations of rock outcrops. The plan view shows some of the complexity at the site and recent paleochannels that resulted in significant variability of the surficial soil conditions. Dense sand deposits of the Merritt-Posey-San Antonio formation were found to the south of the paleochannel but not within the paleochannel. It may be seen that the southern boundary of the paleochannel meanders in and out of the bridge alignment.

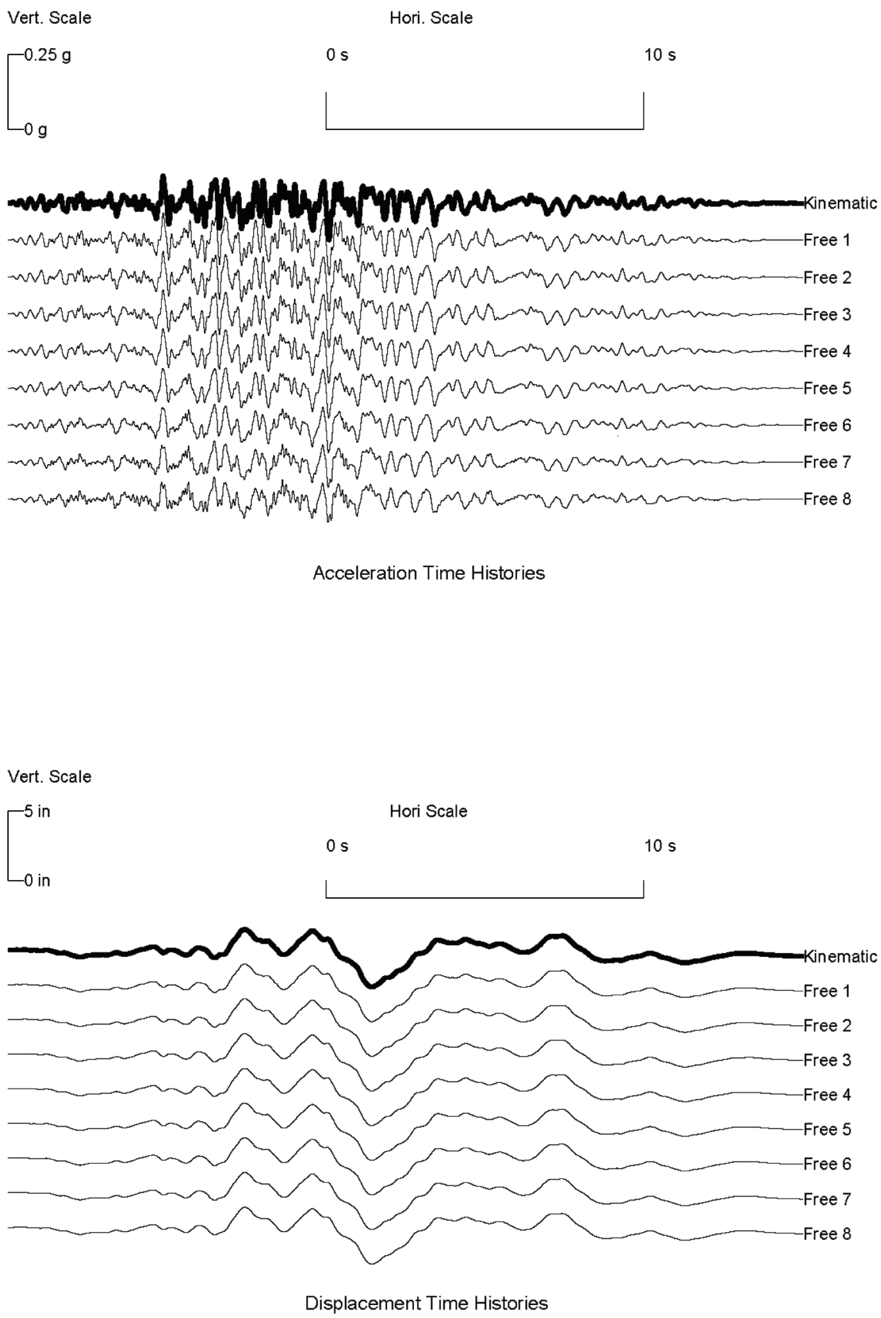


Figure 3-25 Depth-Varying Near-Field Motions vs. Kinematic Motions

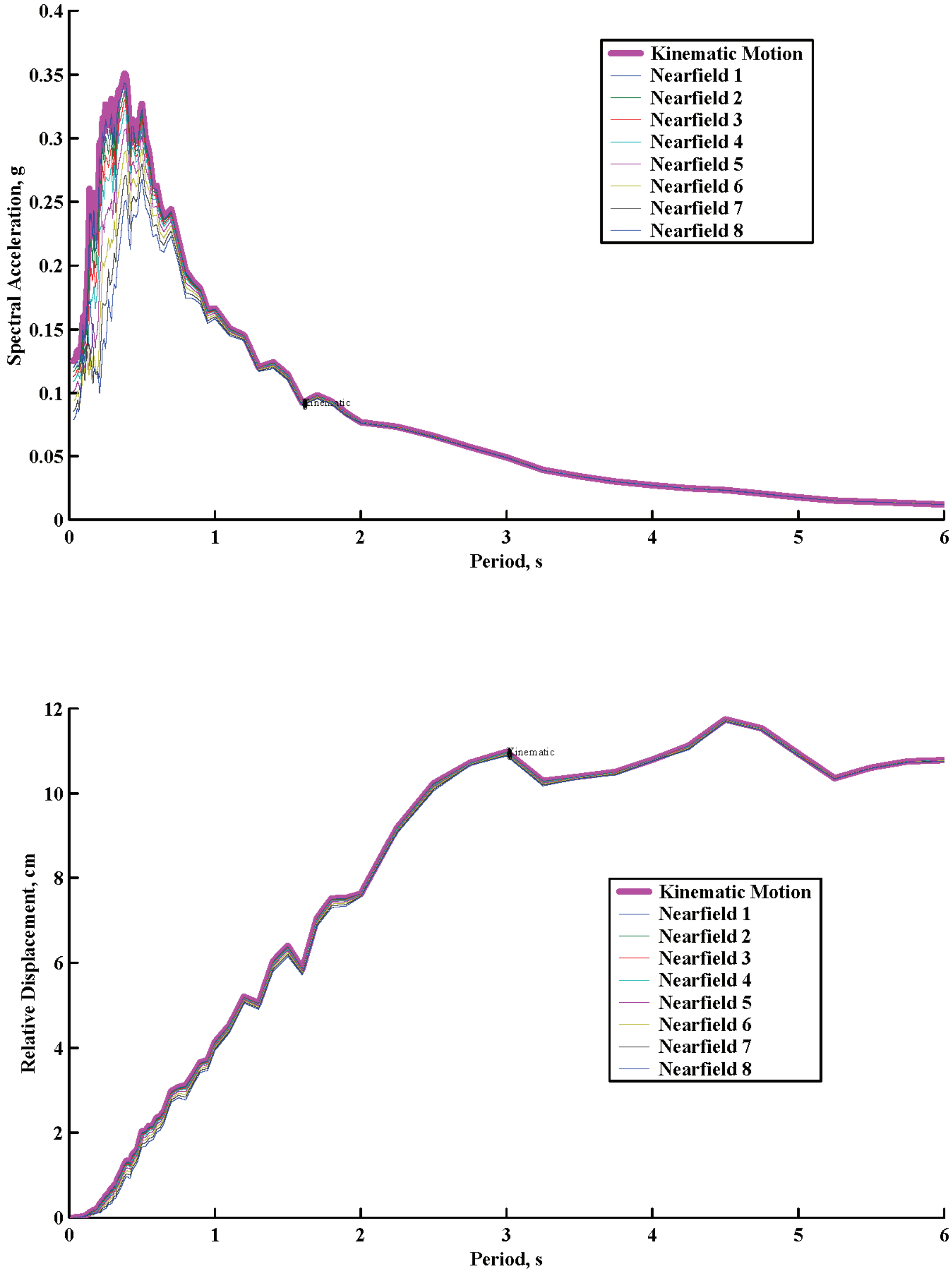


Figure 3-26   Response Spectra of Kinematic Motions and Near-Field Motions

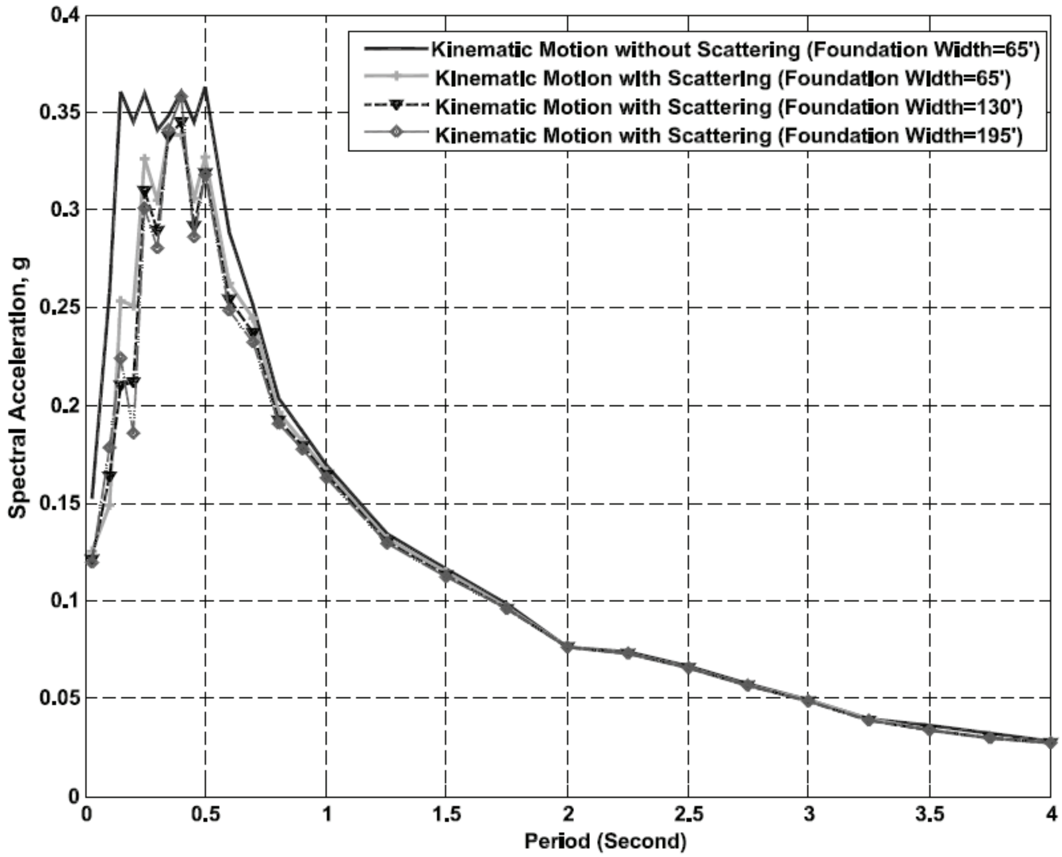


Figure 3-27 Effects of Wave Scattering and Foundation Size in Terms of Response Spectra of Kinematic Motions

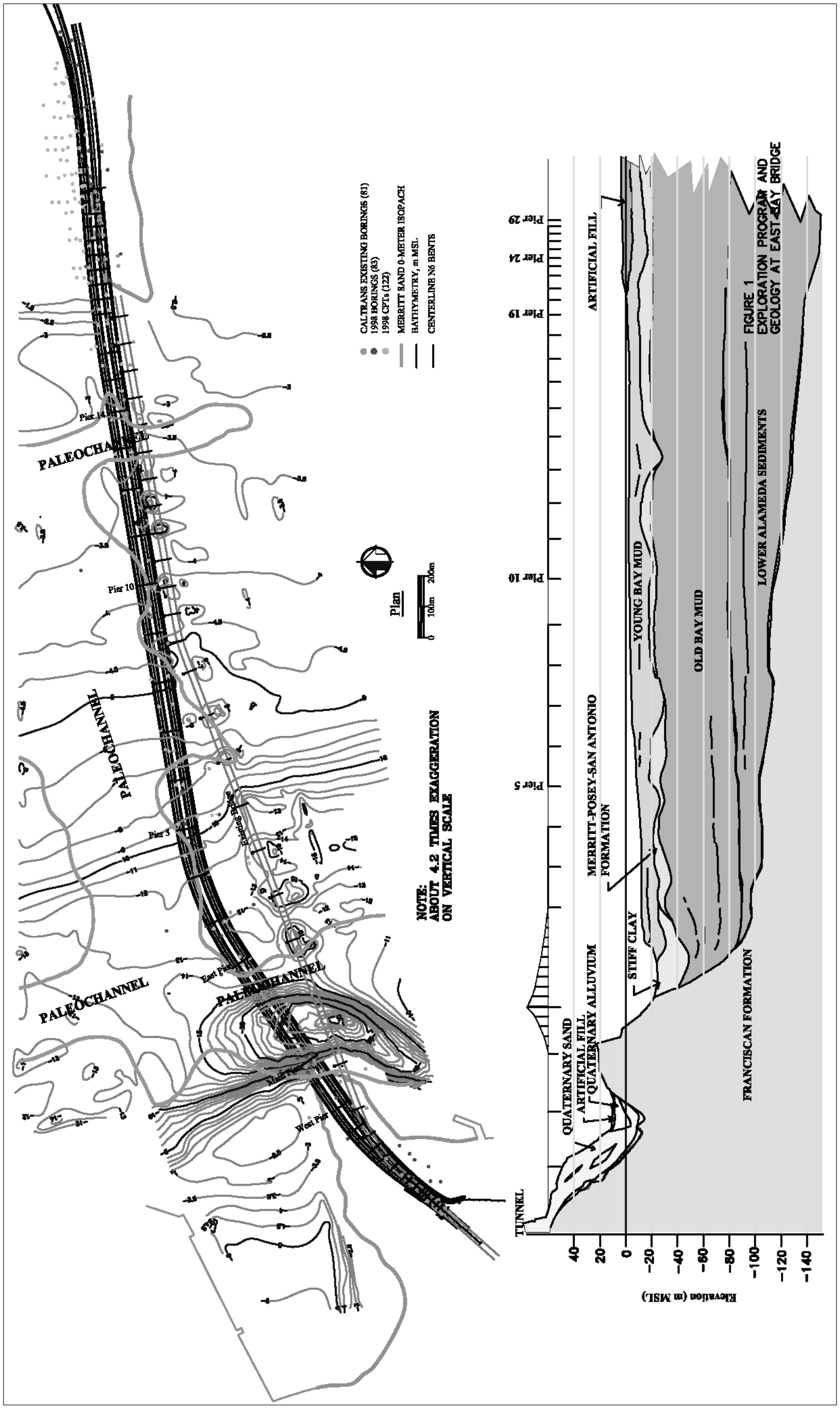


Figure 3-28 Soil Conditions and Paleochannel Features along the SFOBB Bridge Alignment

The geotechnical conditions at the site were characterized by 40 new marine and 78 on-land borings and a comprehensive geophysical survey program. Shear and compression wave velocities were measured in each boring in addition to comprehensive soil sample and insitu testing. The geotechnical conditions are at the same time simple and complex. They are simple in that there are four or five broadly defined strata that traverse the length of the bridge and overlie weathered Franciscan bedrock: the young Bay Muds; the Merritt-Posey-San Antonio deposits (which contain both sands and clays); the Old Bay Clays; the Upper Alameda Marine deposits (which contain very stiff to hard clays); and the Lower Alameda Alluvial deposits (which contain sands, gravels and hard clays). The Old Bay Clays and the Upper Alameda deposits are generally similar and in the broadest view, can be treated as a single stratum.

The geotechnical conditions are complex locally in that all of the strata are variable, especially for the surficial soil strata. The young Bay Muds have a different character inside and outside the paleochannel; the Merritt-Posey-San Antonio deposits are intermittent and contain fluctuating proportions of sand and clays. The absence or presence of the very dense Merritt Sands affects the lateral pile stiffness significantly and, as to be discussed later, plays a role in affecting the decision on whether the piles should be battered or not in the design strategy. The Old Bay Clays and the Upper Alameda deposits show numerous sequences of erosion and deposition with widely varying over-consolidation ratios. The Lower Alameda deposits are variable in composition and the top of the Franciscan bedrock is variably weathered. However, the top surface of the Lower Alameda deposits provides a dominant geophysical reflector at more or less constant elevation providing a robust point of application for input motions for one-dimensional site response analysis for the Skyway portion of the bridge.

3.6.2 Kinematic SSI Design Motion

Whereas the time history response analysis method involving multiple-support depth-varying input motions is the best design approach, it is difficult to document the ground motion design criteria in a transparent manner. Also, there was a desire by the designer to document a design response spectrum (commonly referred as an ARS) design criteria for preliminary design of the skyway structure.

At soft-soil sites, ground motion shaking intensity varies significantly over small depth variations below the mud line and the choice of a rational response spectrum ground motion criteria requires careful evaluation. Free-field mud line motion from site response analysis is commonly used to develop design response spectrum criteria in practice. This was found to be inappropriate for the large SFOBB skyway pile groups embedded in soft soil. Shaking inferred by the free-field mud line motion would be very misleading due to the rather low forces induced to the structure from the very soft soil condition at the mud line. Earthquake excitation forces would be transmitted into the structure along the entire embedded pile length and hence, ground shaking along the entire pile length would play a role in exciting the structure. This needed to be accounted for in formulating rational ARS design spectrum criteria.

A response spectrum criterion was developed for the SFOBB skyway using a detailed soil-structure interaction modeling procedure, referred to as kinematic SSI approach to reconcile the effect of depth-varying ground shaking. The kinematic SSI approach involved modeling the full pile group by beam-column elements representing each individual piles supported by linearized Winkler springs (lateral p-y and axial t-z springs) where depth-varying free-field motions developed from free-field site response analyses are prescribed (see Figure 3-29). Static condensation of the resultant force vector toward the pile cap was conducted at each time step to develop a 6 degree-of-freedom (D-O-F) force vector at the pile cap, which could be operated on by 6 by 6 condensed pile cap stiffness matrix to develop what is referred to as the resultant 6 D-O-F kinematic motion. The response spectrum computed from this kinematic motion was used in formulating the response spectrum criteria for the skyway structure.

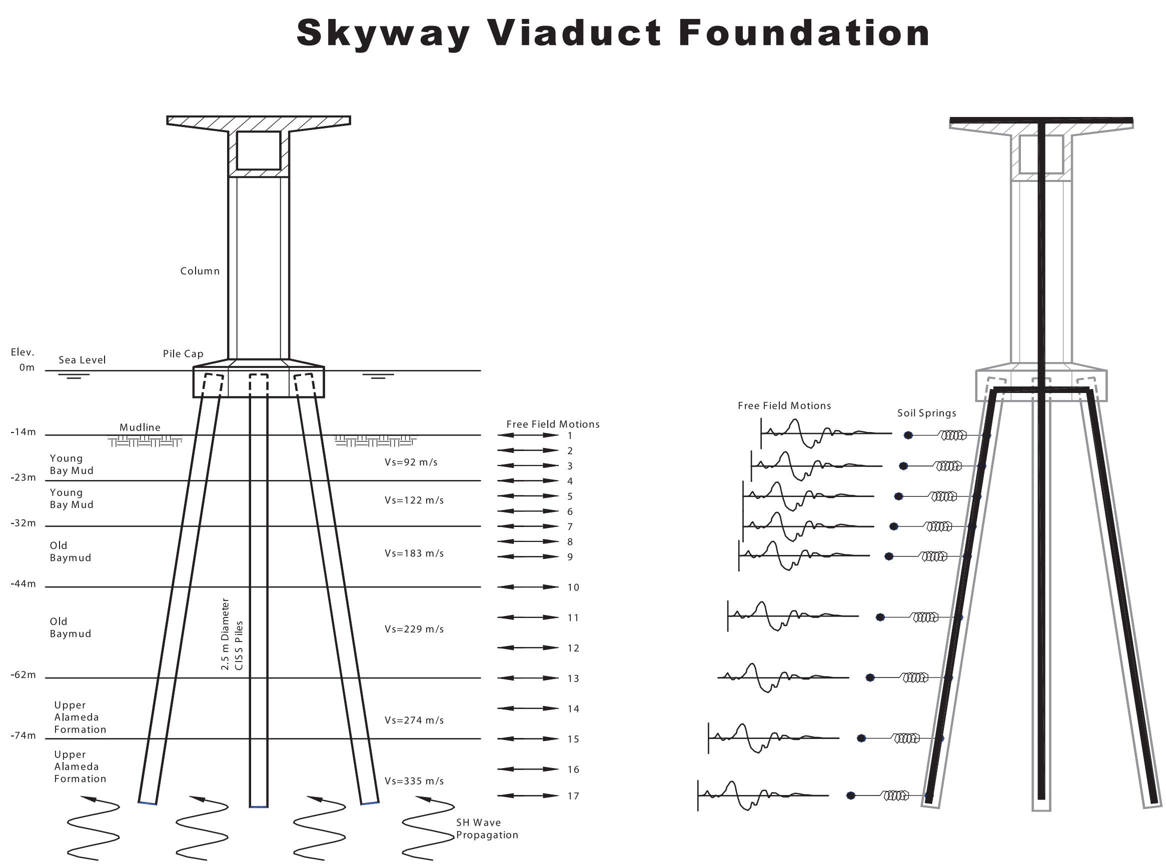


Figure 3-29 Representative Skyway Foundation Model

Verifications of the above discussed kinematic motion substructure technique discussed above were conducted by comparing their solutions to total pile bridge-foundation models conducted by the structural designers (Ingham et at., 1999). The total model explicitly simulates the response of the entire pile group with the fully embedded pile length and representation of the depth-varying nonlinear p-y and t-z springs as well as using depth-varying free field input motions from site response analyses. Results from the kinematic SSI model had to be very close to those from the total structure-foundation model. Figure 3-30 has been extracted from a solution presented by Ingham et al (1999) to illustrate the verification. Similar rigorous verification of the EMI kinematic motion approach has been demonstrated for other major toll bridge retrofit contract conducted for Caltrans. Figure 3-31 presents another example (extracted from Ingham et al, 1999) from the Coronado Bridge retrofit project that compared the global bridge response solutions between the EMI kinematic substructuring approach of using only 3 D-O-F kinematic translational motions (the 3 D-O-F rotational kinematic motions are neglected as approximation) to the detailed full pile system model.

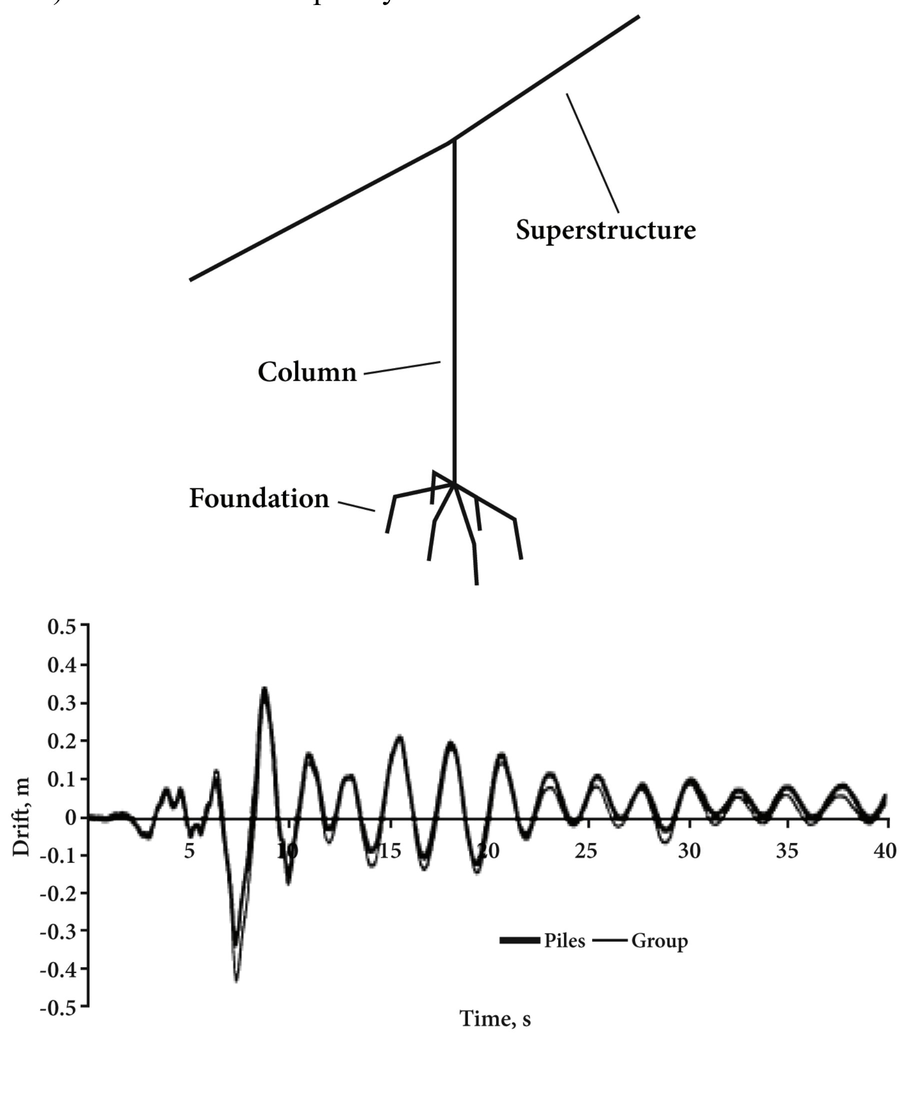


Figure 3-30 Comparison of Input Motions Using EMI Condensed Pile Kinematic Motion ("Group") With Total Pile Group Model ("Piles") from Bay Bridge Skyway

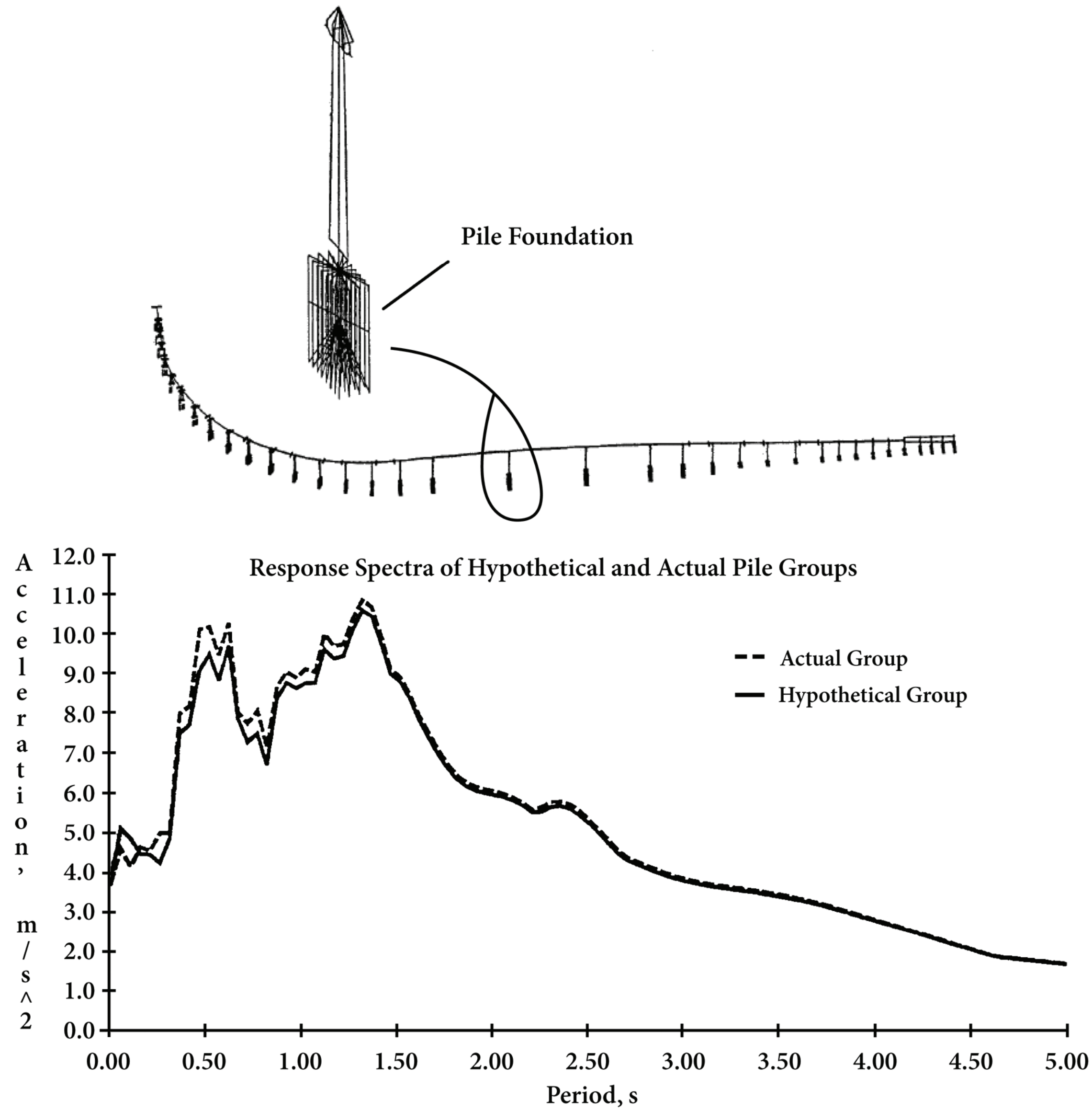


Figure 3-31 Comparison of Kinematic Motion Model ("Hypothetical Group") With Total Pile Model ("Actual Group") from Coronado Global Bridge Solutions

This procedure resulted in more rational structural specific response spectrum criteria in contrast to the conventional mud line surface spectrum from site described above response solutions. Ground shaking based on the described kinematic motion solution can yield solutions very close to time history solutions involve modeling the total foundation using depth-varying motion input at the individual pile nodes. Therefore, a preliminary design based on the response spectrum can be reconciled against subsequent time history analyses using more detailed total foundation model. Figure 3-32 presents some representative solutions of the kinematic pile cap motion as compared to results of depth-varying motions from one-dimensional free-field site response analysis from SHAKE. It can be observed from the response spectra plot that the kinematic motion has shaking close to the free-field motion at about 14-m depth, especially at relatively long period range (over 1 second). This corresponds to a depth of about five pile diameters for the 2.5-m pile used for the skyway structure.

Because the kinematic motion is developed using the pile group model (including pile configuration and pile stiffnesses), the resultant motion would be structure dependent; the resultant spectrum developed for the battered pile option was found to be lower than the vertical pile option as shown in Figure 3-33 presenting a comparison of kinematic motions between the eventual bay bridge skyway battered pile group in contrast to the solution where the piles were artificially converted to a vertical pile configuration using an identical pile top layout at the pile cap. In addition to kinematic motions, Figure - also presents spectra of the free-field motion, clearly showing the systematic reduction in shaking of the free-field motion with depth. Evidently, a battered pile group will be affected by ground motion at a deeper depth due to the contribution of the stiffer axial pile stiffness as opposed to a vertical pile group, which depends merely on the flexural pile stiffness. Effectively, the point of dominant force input (the "point of fixity") would be deeper for a battered versus a vertical pile group. As reported by Ingham et al. (1999), the effect of the lower shaking associated with the battered pile group has also been verified by more detailed total pile model solution which showed a significant lower pile cap drift, pier drift and a pier plastic hinge rotation for the battered pile group.

In addition to theoretically lower shaking, the battered pile option for the SFOBB application has the following benefits:

- Smaller pile cap mass relative to the effective size of the pile group.
- A lower residual permanent drift on the structure because of lower shaking.
- A more uniform lateral stiffness among various piers from a more constant axial pile stiffness influencing the lateral foundation stiffness in a battered pile group as opposed to a vertical pile group, which would depend exclusively on the lateral pile stiffness and is more influenced by the variation in the Merritt Sand stratum due to erosion features of the paleochannel at the bridge alignment.
- Sensitivity studies conducted by the designers showed that the mode of vibration of the pile cap with its pile mass would have a rather long period for a vertical pile group, with periods approaching the period of vibration of the structure itself. Battering the pile would reduce the period of the foundation system and therefore forestall any dynamic interaction between the structure and its foundation.

Because battered piles have a poor performance track record, (e.g. wharf pile failures during the Loma Prieta earthquake), the project team conducted a comprehensive evaluation of the cause of their poor performance. The team concluded that the poor performance in past case histories is largely due to the lack of understanding of battered piles in past design practice, rather than due to fundamental problem in the nature of battered piles themselves.

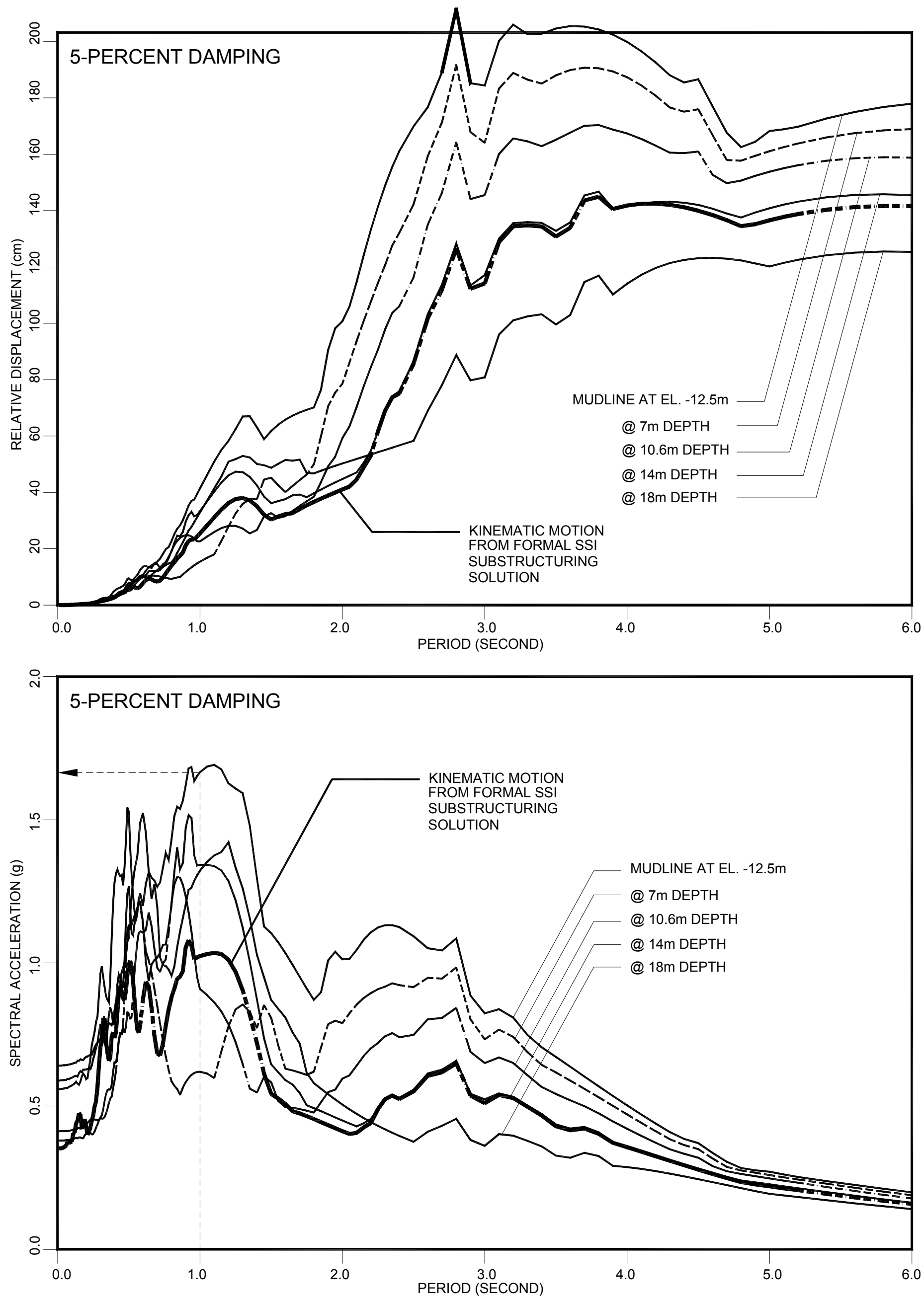


Figure 3-32 Kinematic Motion vs. Depth-Varying Free-Field Motion

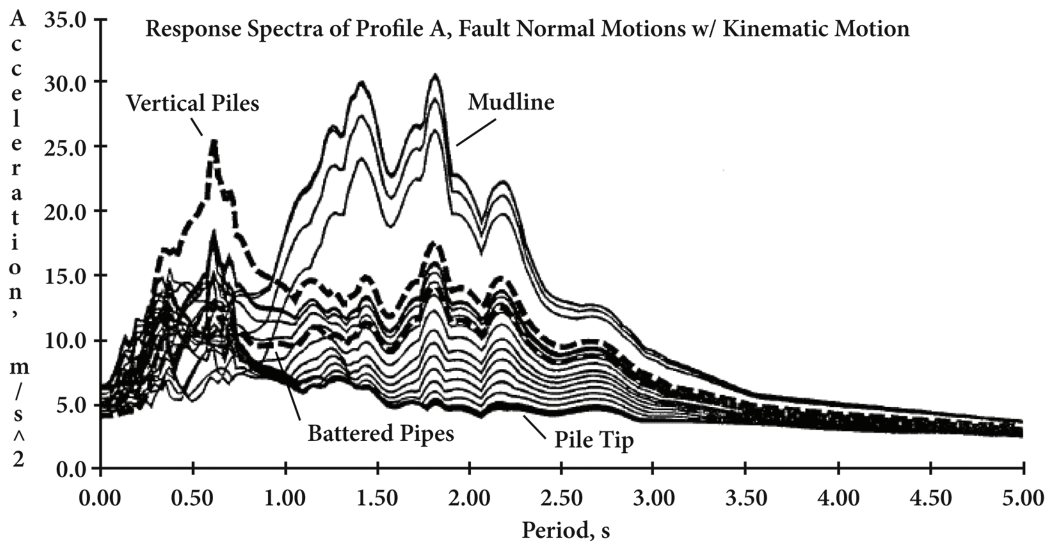


Figure 3-33 Kinematic Motions for Vertical vs. Battered Pile Group, SFOBB Skyway

SECTION 4:
MODELING CAISSONS

4.1 Overview

Large caissons have been employed as piers for many large over-water bridges. They may be prefabricated in an on-land construction site near the water, floated to the pier site, and gradually sunk into the soils. Alternatively, the fabrication of the caisson may be undertaken at its final site, supported on a temporary sand island, and then sunk by dredging out within the open cells of the caisson.

Box caissons are tabular concrete structures with a closed bottom and designed to be buoyant for towing to the bridge site. Allowing water to enter through flooding valves gradually sinks the caisson. Box caissons can be founded on compact granular soils that are not susceptible to erosion by scour or on rock surfaces which are dredged to remove loose material, trimmed to a level surface and covered with a blanket of crushed rock. Skirts are provided to allow the caisson to bed into the blanket and a grout is injected to fill the space between the bottom of the box and the blanket.

Open-well caissons have vertical shafts formed by the interior and outside walls that are open at the top and bottom. The shallow draft bottom section is floated to the pier site and sunk by excavating the soil from the open wells as the walls are raised progressively. This type of foundation is suitable for soils consisting of loose sand or gravels since these materials can be readily excavated within the open well and they do not offer high skin friction resistance to caisson sinking. When the caisson bottom reaches a target elevation, the open caissons are sealed by a layer of concrete under water in the bottom of the wells by tremie pipes. The wells are then pumped dry and additional concrete is placed.

Alternatively, compressed air can be pumped into the well (sealed by an airdeck) to expel water so that men may work at the bottom and excavate the material in the dry. Air pressure in the working chamber is approximately maintained near a pressure that balances the hydrostatic pressure in the ground at the cutting edge of the caisson. The bottom on the caisson is connected to an air lock by a shaft. The air locks permit men and materials to pass back and forth from the atmosphere outside the caisson into the high-pressure working chamber. This type of construction is called a pneumatic caisson. Because of the high costs of pneumatic construction, the construction often begins with sinking of the caisson in the open, and then converting to the pneumatic process by adding of an air deck and docks only for the final sinking.

Suitable site conditions for caissons are bedrock overlain by shallow sediments, in which the cutting shoe can readily penetrate allowing the unit to rest on the competent rock. Caissons are also constructed in deep granular soil sediments which can be dredged out within the open wells.

When designing caissons to support long span bridges, specific design items that must be established include:

- External loading such as seismic and ship impact
- Erosion due to scouring
- Allowable settlement and tilting
- Structure life

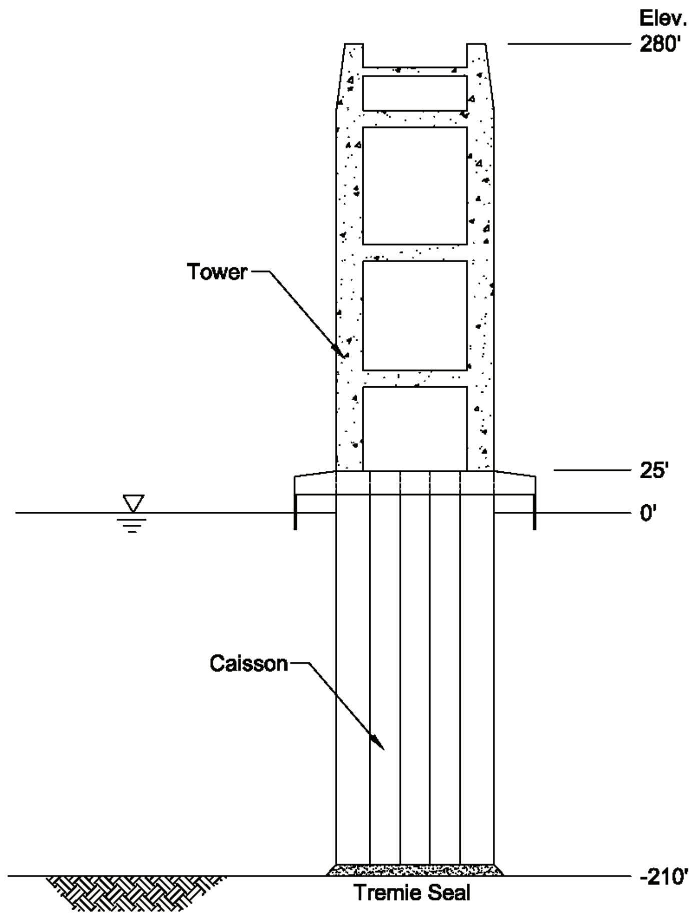
When sizing the caisson, various failure modes must be considered:

- Excessive tilting or base rotation under lateral load acting above the base of the caisson, e.g., during earthquake excitation
- Vertical settlement, resulting from the ratcheting action caused by cyclic rocking
- Stability of base sliding when lateral load exceeds the passive soil resistance on the vertical wall and the friction capacity of the soil at the base of the caisson
- Overturning of the caisson, which in most cases is not likely to be of concern due to a wide foundation base

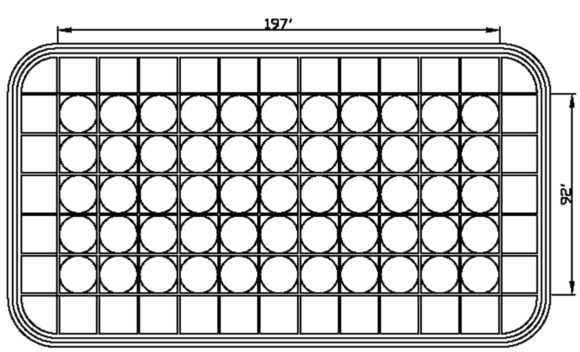
The west span of the San Francisco – Oakland Bay Bridge (SFOBB) in California, which is a multi-span suspension structure, is supported entirely on caissons resting directly on Franciscan rock formation of sedimentary rock. The portion of the east span of the SFOBB near the shallow bedrock is also founded on deep caissons. The Carquinez Bridge is another long span toll bridge in California utilizes caissons at several support locations to cross a strait. Two towers of the Tacoma-Narrows suspension bridge in the State of Washington are supported on large caissons sunk in a sand layer. A second parallel suspension bridge will be built next to the existing bridge using the identical foundation system. The dimensions of the caissons for the two bridges are very similar conforming to a rectangular shape cross section with several open cells. Circular caissons were used as foundation for the Coleman Memorial Bridge in Delaware. Figures 4-1 to 4-4 show the dimensions of these caissons.

Because of the massive volume of concrete, the weight of the caisson constitutes the bulk of the entire bridge structure. Consequently, the seismic behavior of the bridges supported by caissons is highly influenced by the response of the foundation. To capture the accurate dynamic response of the bridge structure, the caisson foundation must be properly modeled.

Proper modeling of the caissons plays a major role in successful seismic analyses and designs of the bridge system. Since the caisson unit is very massive and is a stiff system, the fundamental mode of vibration for most gravity caissons can be characterized as having a very short period motion. It is likely that the response mode of the caisson would be very different from the long period response of the bridge. Because of the two distinct modes of vibration between the caisson and the bridge, it is possible to estimate the seismic behavior of the caisson in an uncoupled manner without consideration of the seismic response of the superstructure. However when evaluating the bridge structure's response, the foundation elements must be included in the total bridge model.



Elevation View of Pier 4



Plan View of Pier and Fender

Figure 4-1 Caisson of Pier 4 of West Span SFOBB, CA

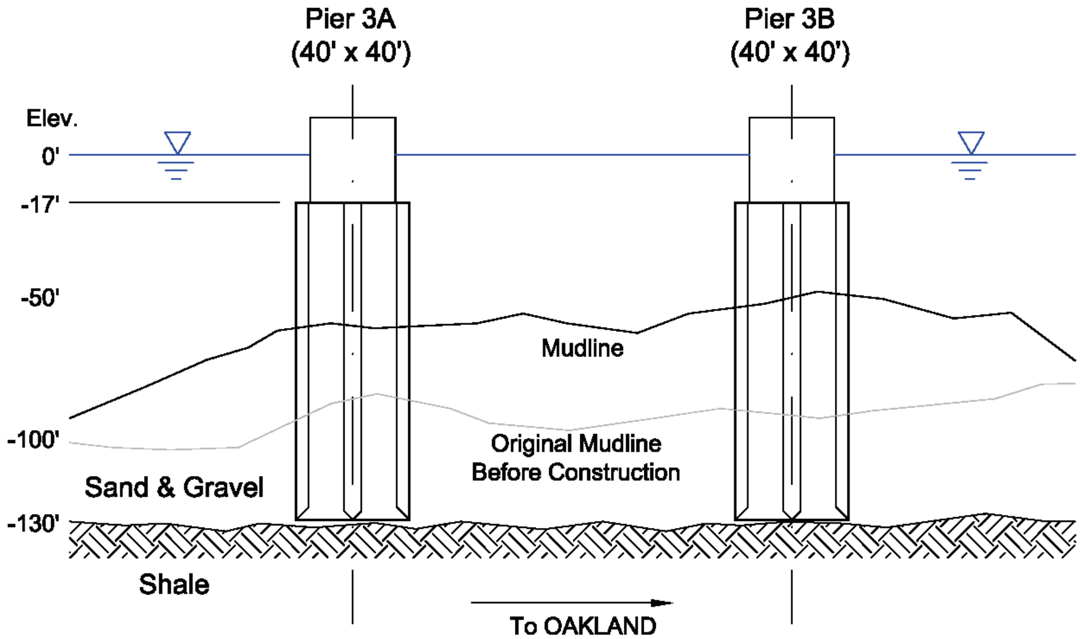


Figure 4-2 Multiple Caissons at Pier 3 of Carquinez Bridge, CA

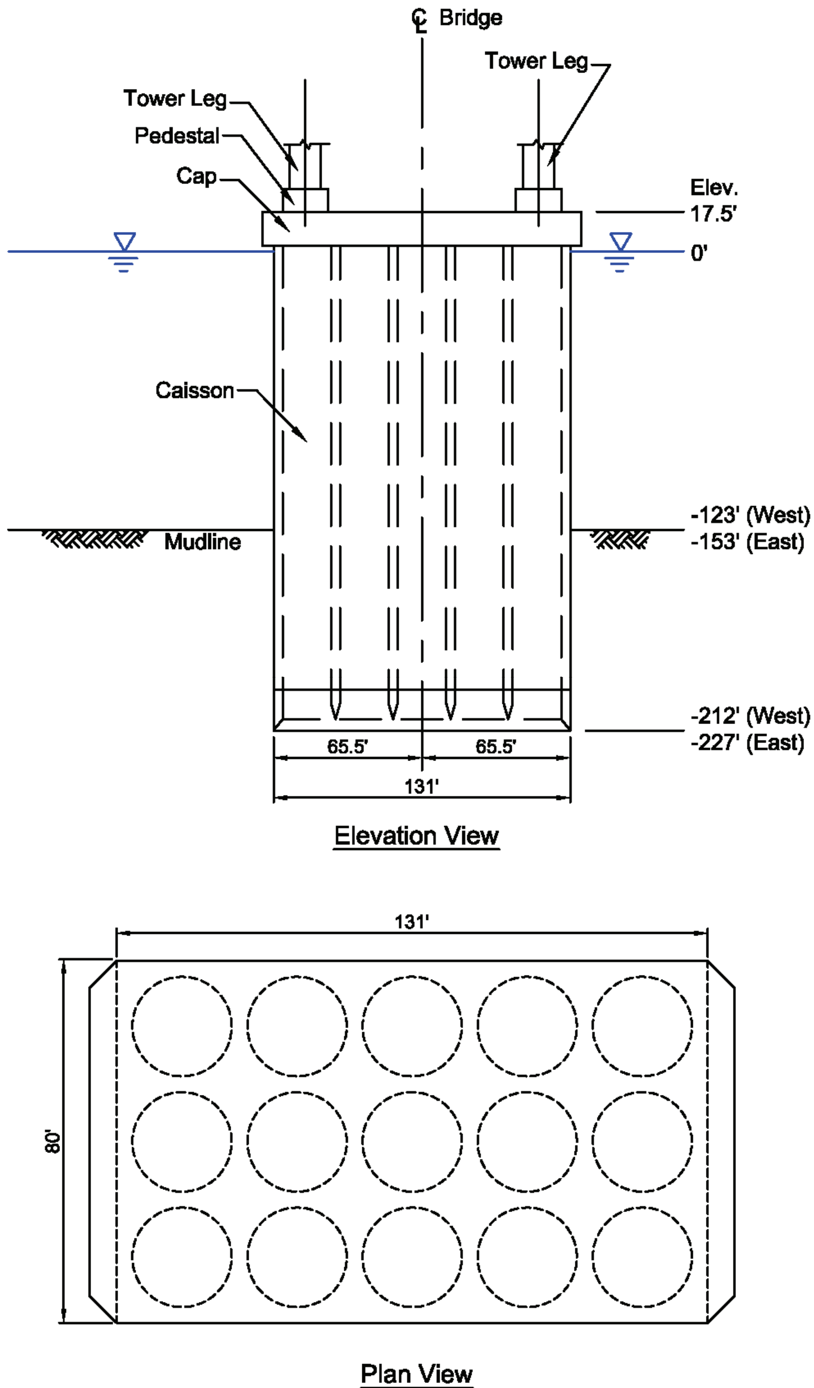
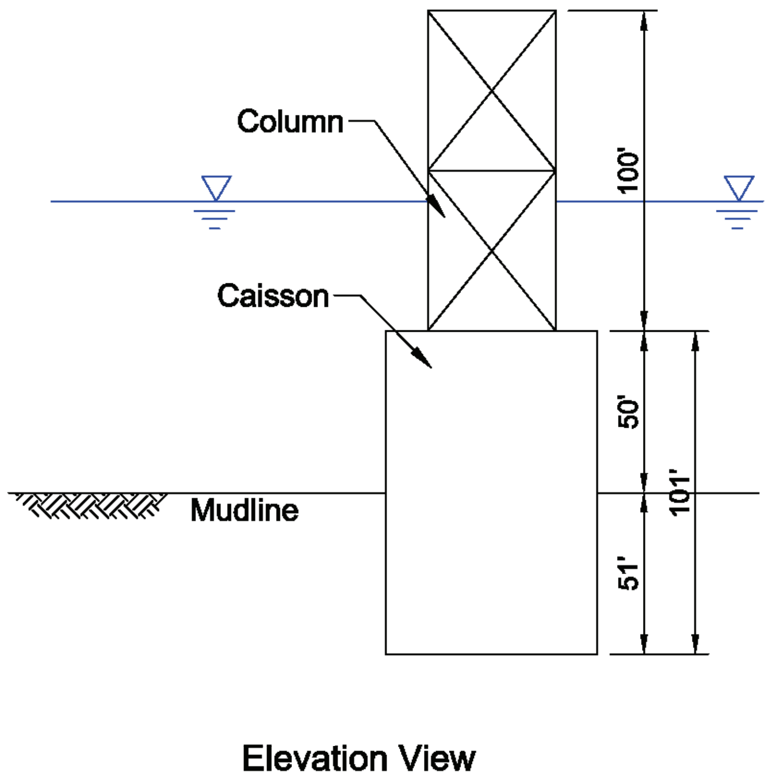


Figure 4-3 Multiple Caissons at Pier 3 of Carquinez Bridge, CA



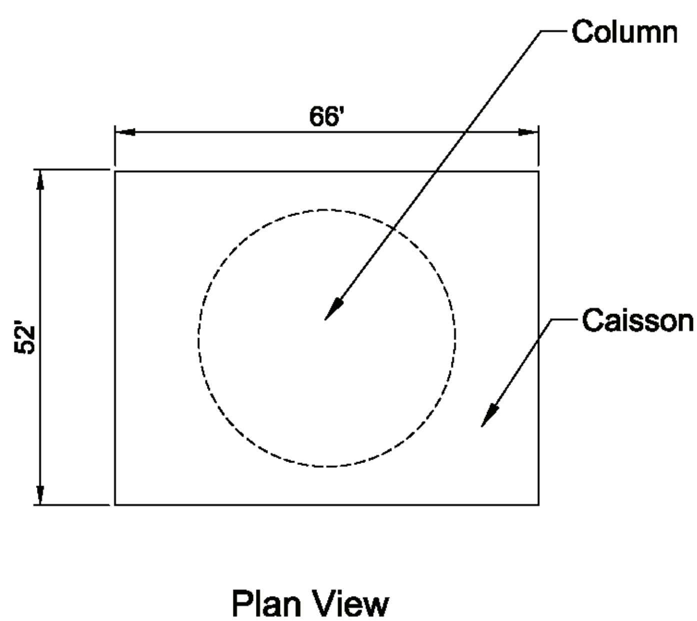


Figure 4-4 Caisson Foundation at George Coleman Bridge, VA

Several modeling approaches for caissons have been used for seismic design. Each approach has advantages and disadvantages, and the engineers should recognize the shortcomings and use them according to the situations encountered. Discussion on modeling approaches is provided in Section 4.2. Section 4.3 provides a commentary on the influence of kinematic soil-caisson interaction on input ground motions, and Section 4.4 provides an overview of two case histories.

4.2 Caisson Modeling Approaches

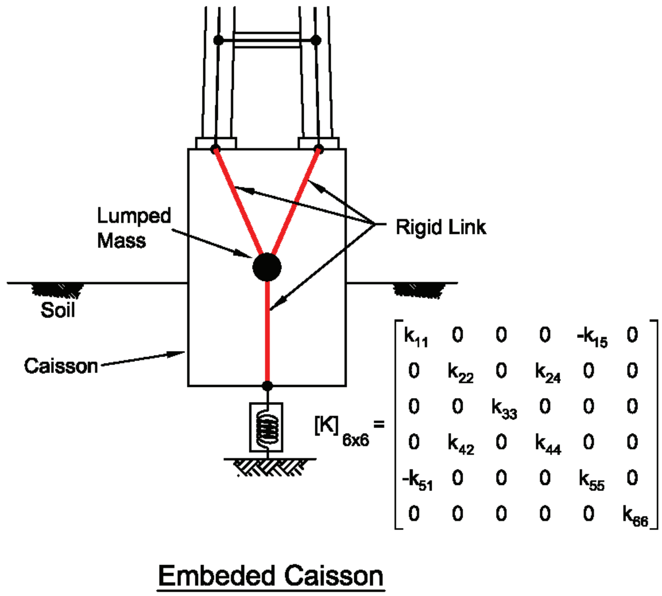
As it is not practical to model the entire caisson and surrounding soil in a continuum finite element model, the caisson is often represented by a lumped mass with a 6x6 stiffness matrix and an appropriate load vector corresponding to input ground motions acting on the foundation. As this approach does not explicitly consider potential uplift of the caisson, an alternative is to support the caisson by nonlinear interface springs representing load versus deflection and moment versus rotation in all three underground directions. For deeply embedded foundations, coupling between shear and overturning moment must also be considered.

4.2.1 Linear Stiffness Matrix Approach

For modeling of a complete system comprising of bridge structure and foundations, the caisson foundation may be represented by a lumped mass model as a first approximation. Since the center of gravity for the caisson is relatively high, the controlling mode of vibration is usually rocking. The end bearing of the caisson base and the passive soil resistance on the vertical wall of the caisson provide resisting mechanisms against seismic loading. Figure 4.5 depicts a schematic of the caisson model used in modeling of the bridge structure in which a set of foundation springs is attached to the node at the bottom of the caisson. A rigid link is established between the center of the gravity node where the caisson mass is lumped and the node where the soil spring is attached. The following form of coupled linear stiffness matrix as derived from an elasto-dynamic theory may be used for the spring:

$$[K]=\begin{bmatrix} k_{11} & 0 & 0 & 0 & -k_{15} & 0 \\ 0 & k_{22} & 0 & k_{24} & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & k_{42} & 0 & k_{44} & 0 & 0 \\ -k_{51} & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix} \tag{4.1}$$

Degrees of freedom 1 through 3 are translation and degrees of freedom 4 through 5 are rotation. The degree of freedom 3 is the translation in the vertical direction. The vertical translational degree of freedom (k_{33}) and torsional degree of freedom (k_{66}) are uncoupled with the other degrees of freedom in the stiffness matrix. However, the two components of horizontal translation are coupled with the two degrees of freedom in rocking rotation



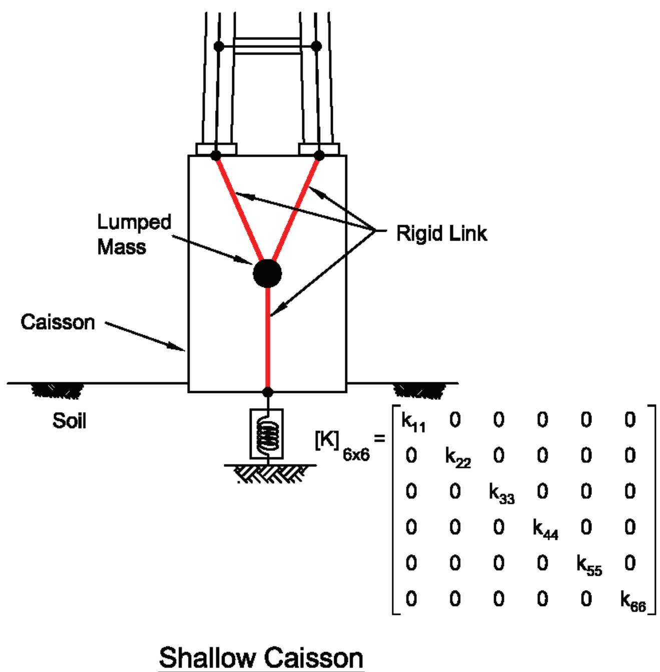


Figure 4-5 Lumped Mass with Linear Stiffness Matrix

in the stiffness matrix. These coupling terms in the stiffness matrix are due to the embeddment of the caisson in soils. When embeddment of the footing is shallow, the off-diagonal (cross-coupling) terms are generally neglected. Therefore, the following form of uncoupled stiffness matrix would be adequate:

$$[K] = \begin{bmatrix} k_{11} & 0 & 0 & 0 & 0 & 0 \\ 0 & k_{22} & 0 & 0 & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & k_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix} \tag{4.2}$$

The solution for a circular footing bonded to the surface of an elastic half space provides the coefficients of the stiffness matrix for the various components of displacement.

4.2.2 Nonlinear Lumped Springs Approach

Under seismic excitation, excessive overturning moment could lead to gapping at the base of the caisson. Therefore, the classical elasto-dynamic approach, which assumes full contact with the underlying soil, tends to overestimate the stiffness. Although the stiffness matrix presented above provides reasonable foundation behavior at a small displacement range, it can lead to erroneous results for large deformations, especially when a force-based design approach is taken. The errors are attributed to the lack of a load-fuse mechanism when the limiting force or moment is reached.

Improvements can be made to the lumped mass system with the allowance for base separation and soil yielding to consider the effects of geometric and material nonlinearity as well as to establish proper load-fuse mechanisms. It represents the foundation stiffness using nonlinear moment-versus-rotation and load-versus-displacement relationships, as illustrated in Figure 4.6. This nonlinear foundation behavior can be established by performing pushover analyses to capture essential elements of the soil-structure interaction phenomena and to consider the limiting force and moment.

When the nonlinear foundation behavior is prescribed, uncoupled springs are used; i.e., the load-versus-displacement relationship and moment-versus-rotation relationship are operated independently. In this case, special attention must be given to the relationship between shear and overturning moment on the caisson.

4.2.3 Nonlinear Distributed Soil Springs Approach

This model entails Winkler springs distributed over the bottom surface of caisson to represent the soil continuum underlying the foundation. Another set of soil springs are attached to the vertical sides of caisson walls to model passive soil pressure acting on the concrete. The soil springs may be nonlinear so that yielding of localized soil can be

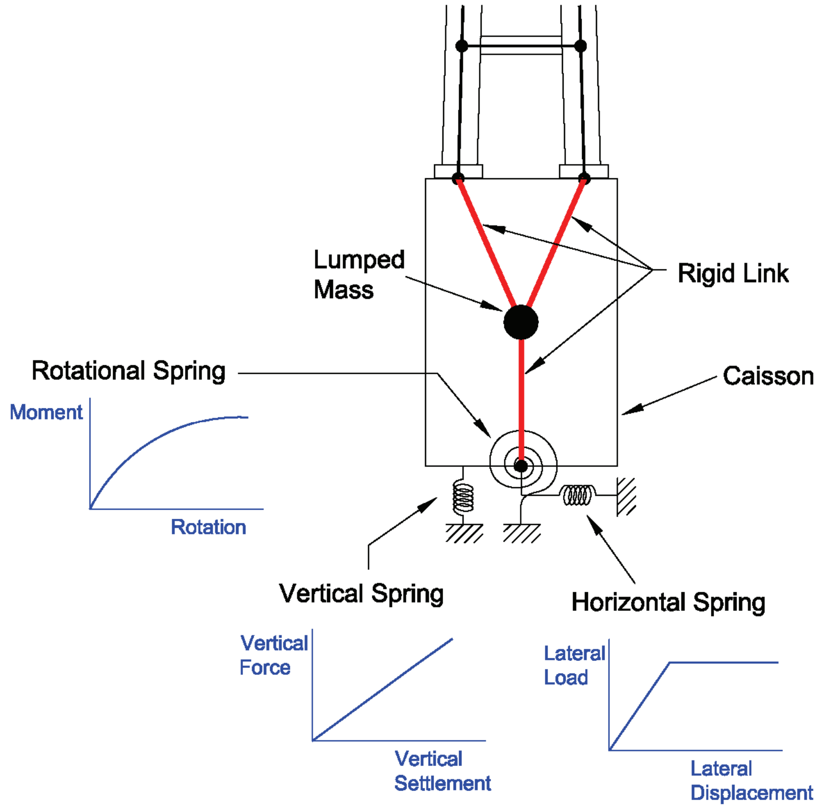


Figure 4-6 Lumped Mass with Nonlinear Uncoupled Springs

considered. In addition, gapping elements can also be implemented in series with the soil springs to engage a full contact between the soil and the caisson during compression and to allow separation under tension.

Figure 4-7 illustrates a distributed soil spring model used in the seismic analyses of a global bridge. This modeling approach would address the two significant features; nonlinear behavior and coupling between lateral loading and overturning moment. Therefore it is a significant improvement over the lumped spring models.

Establishing proper distributed soil springs is a key to successfully modeling the caisson that is used in the global bridge system. It is desirable to perform pushover analyses of the caisson using a finite element method to extract the appropriate soil springs. The solutions obtained from the finite element analysis would represent the overall deformation behavior of the caisson, and also address the stress-strain behavior of the local soil elements. That is to say, the soil springs should capture the geometric non-linearity due to foundation uplift and the ultimate limit state of the foundation.

If the correct soil springs are developed, this model which is characterized by distributed masses of the caisson that are supported by distributed soil springs at its exterior perimeter, effectively reflects the overall caisson behavior in the global bridge model.

Input seismic excitation to this model is given by multiple ground motions applied at each layer of the soil strata to provide a mechanism to implement incoherent motions that result from wave scattering. In the following section, the issue of wave scattering is discussed in relationship to the seismic loading of the caisson supported bridge structure.

4.3 Wave Scattering from Caissons

Deep caissons embedded in soil create an inclusion of a rigid mass in the wave propagation media. Because of the strong stiffness contrast between the caisson and soil, the boundary conditions are highly influenced by the reflected wave. It is anticipated that inclusion of a rigid caisson into the soil could exhibit a amplification/de-amplification phenomena that is frequency (wavelength) dependent. These wave scattering effects are not normally considered in current design practice for caissons.

The framework used to study the wave scattering for caisson structures is based on numerical analyses using the ADINA program. Figure 4-8 shows the design data for a caisson and the soil conditions under consideration. Earthquakes propagate both P- and S-waves, and upon incident at the caisson boundary, reflected and refracted waves of the both kinds would produce a complex scattering process that can only be studied with a finite element method. The wave scattering effects lead to incoherent motions at the perimeter of the caisson, resulting in spatially varying ground motions at the vertical sidewalls and the bottom surface of the caisson.

As the dimensions of the caisson become large, the ground motions arriving at various points on the caisson could vary significantly, depending on the frequency content of the motion. However in current design practice, the ground motions along the vertical height of the caisson are computed using one-dimensional site response analysis, and all the motions at the same elevation are assumed to synchronized; that is to say wave scattering is not considered.

In this study, two-dimensional site response analysis is carried out to extract the spatially varying ground motions at the perimeter of the caisson that is in contact with the surrounding soil. The results from the two-dimensional site response analysis would therefore include wave scattering effects; these spatially incoherent motions would eventually drive the global bridge model through the caisson element represented by distributed massed and soil springs. Figure 4-9 depicts the 1D site response study without considering scattering and the 2D finite element model to capture wave scattering effects.

As the reference motions (one horizontal and one vertical) are given at the surface, deconvolution analyses were conducted to compute the base excitation for each direction, which was then used to excite the finite element model. Figure 4.10 shows a comparison between two-dimensional site response results with one-dimensional motions. It is noted that the caisson region is simulated by an increased modulus corresponding to the concrete material; however, mass is zero. This is to consider the kinematic interaction

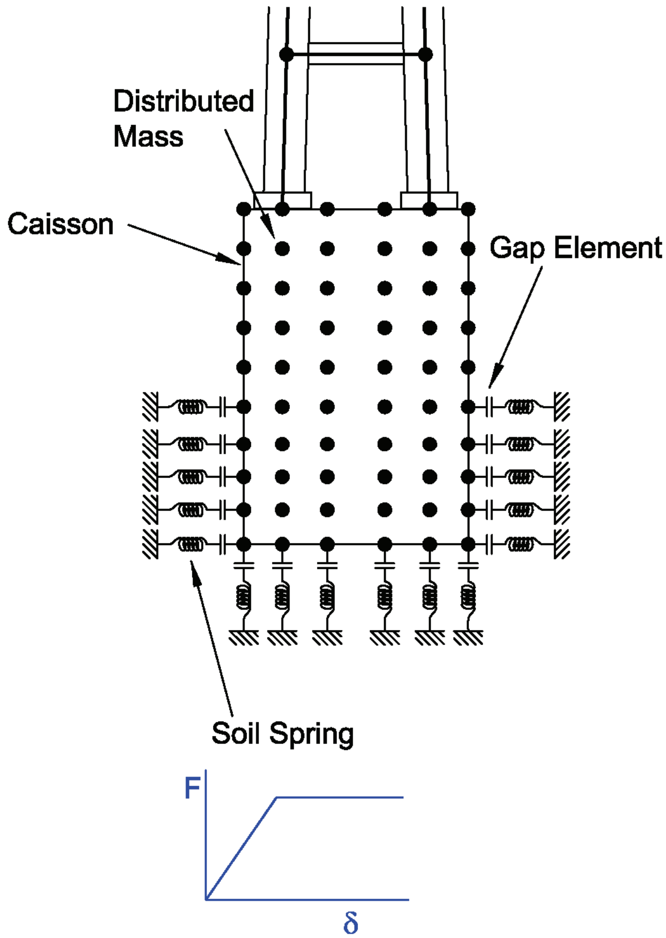


Figure 4-7 Distributed Mass and Soil Springs

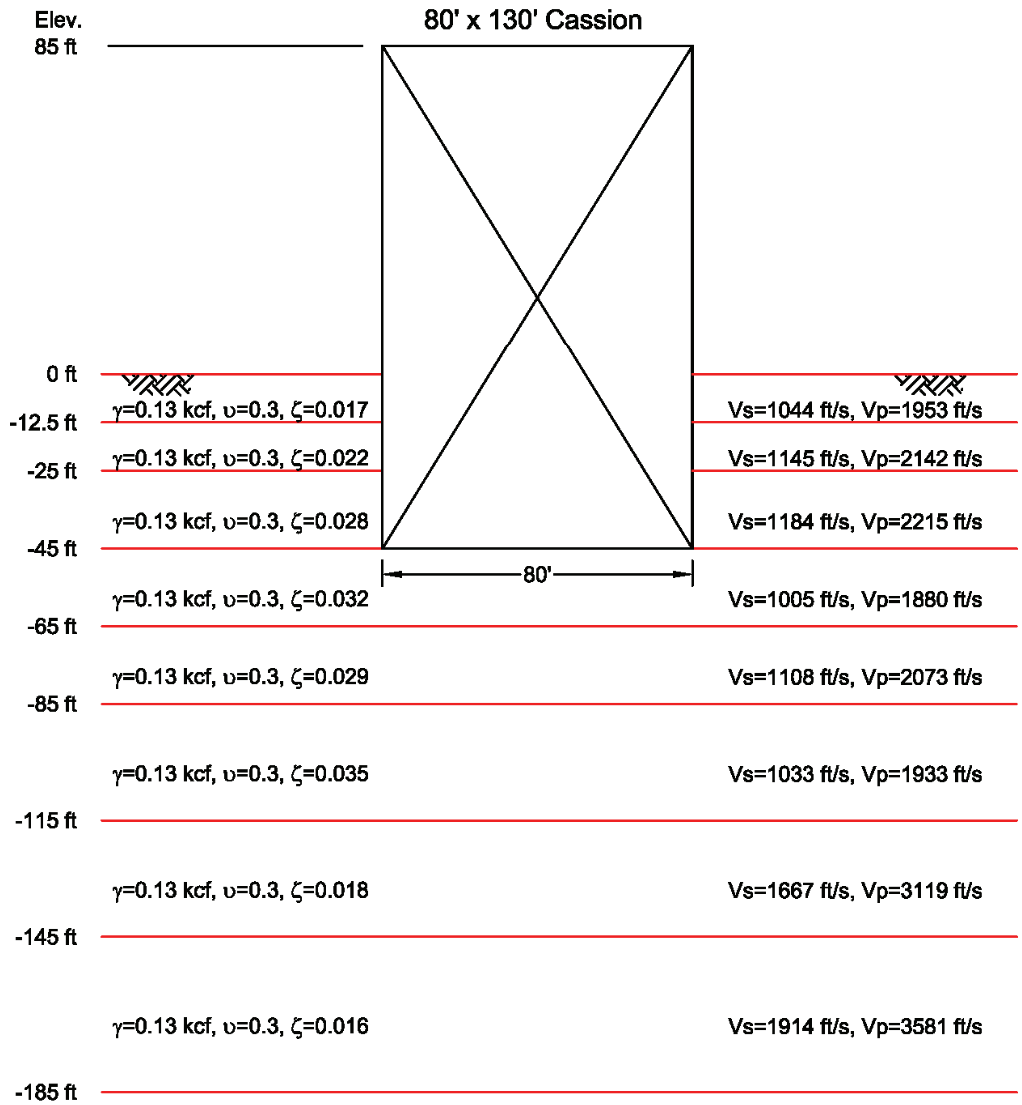


Figure 4-8 Caisson Dimensions and Soil Data

only without inertia. The effect of the caisson's inertia is included in the second step analysis of the global bridge model, where the caisson mass is treated as a lumped or distributed mass.

The comparison results were derived from the caisson with the base width of 80 feet. Additional sensitivity studies were performed for larger caissons sizes (the base width of 160 feet and 240 feet) keeping the caisson depth unchanged. The resultant near-field motions of the three caisson sizes are presented in Figure 4-11; all include wave scattering effects. It appears that the larger caissons, the greater the influence of the scattering motions; however, the differences are mostly in short periods (less than 0.7 sec).

In order to evaluate the influence of wave scattering, the seismic response of the caisson was examined in the second step analysis without the presence of the bridge. The caisson and the surrounding soil are modeled as distributed masses and distributed springs, respectively, and the analyses were conducted with and without wave scattering. The caisson response using the free-field motions computed from the one-dimensional site response analysis is regarded as no wave scattering effect, which is the conventional design procedure. The caisson behavior subjected to near-field motions from the two-dimensional wave propagation includes the wave scattering effect. Figure 4-12 presents the results from the largest caisson (240 feet base width) showing translational displacement at the center of gravity (C.G.) and the rotational displacement (rocking mode of vibration). The two solutions are very close, suggesting that the wave scattering effects are not significant. The reason is that the fundamental period of this caisson is approximately 0.9 second, and at this period the difference in shaking levels with and without scattering is very small.

For the smaller caissons such as 160-foot and 80-foot base widths, the fundamental periods of vibration are longer than the 240-foot caisson (1.1 sec for 160 feet, 2 sec for 80 feet). The influence of wave scattering is expected to be even smaller. From the presented studies, we judge that conventional seismic design of caissons without explicit considerations of wave scattering is reasonable.

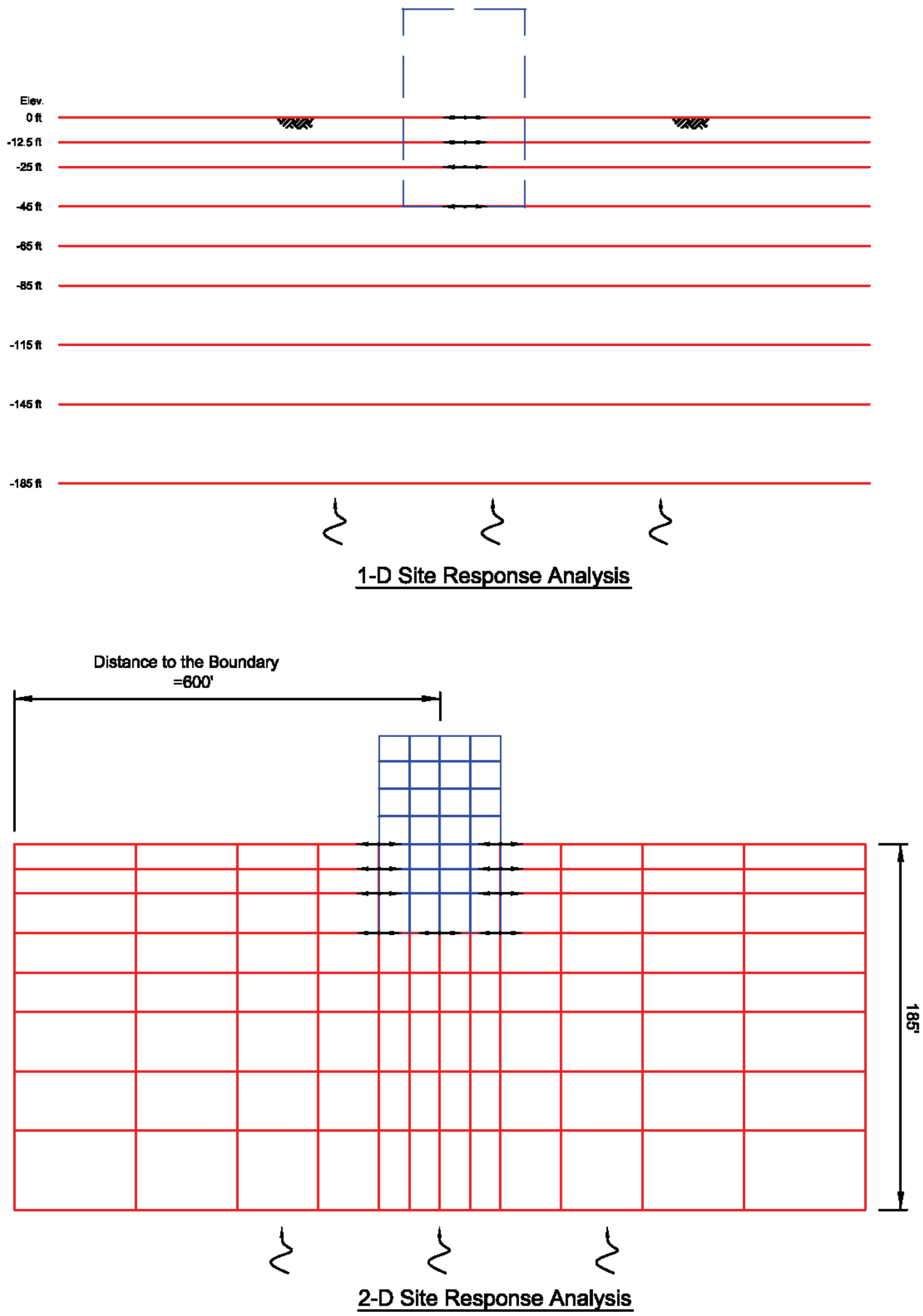


Figure 4-9 1D versus 2D Site Response Analysis

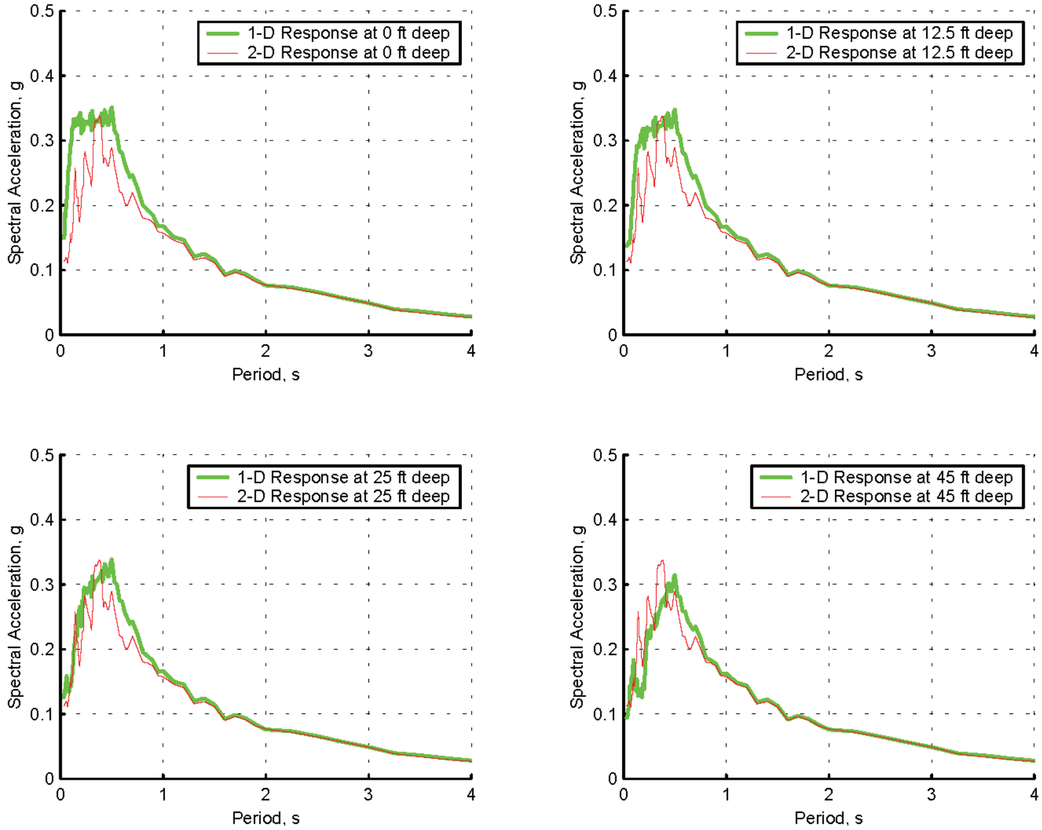


Figure 4-10 Comparison of 1D and 2D Site Response Solution

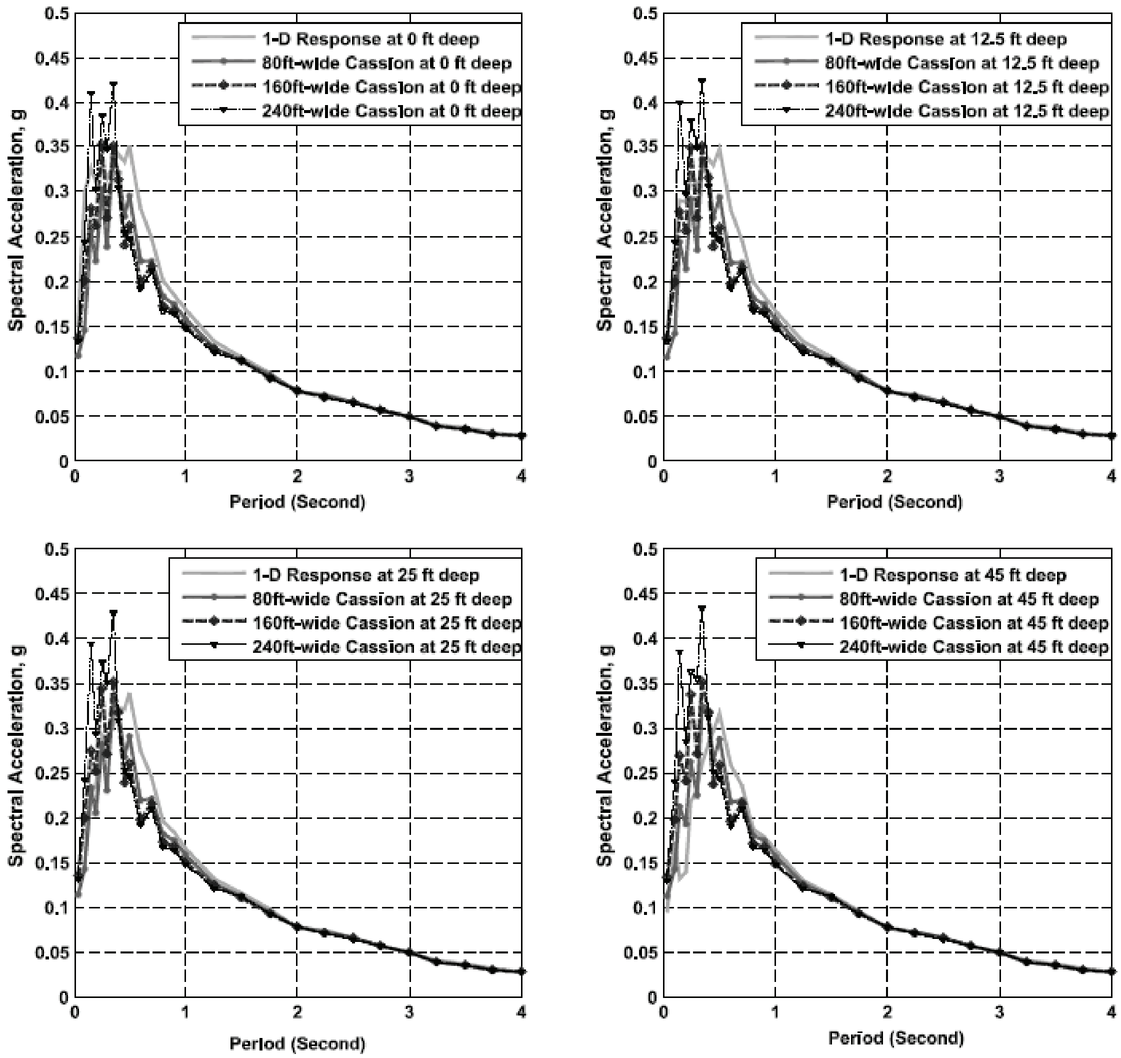


Figure 4-11 Near-Field Motions of Three Caisson Sizes

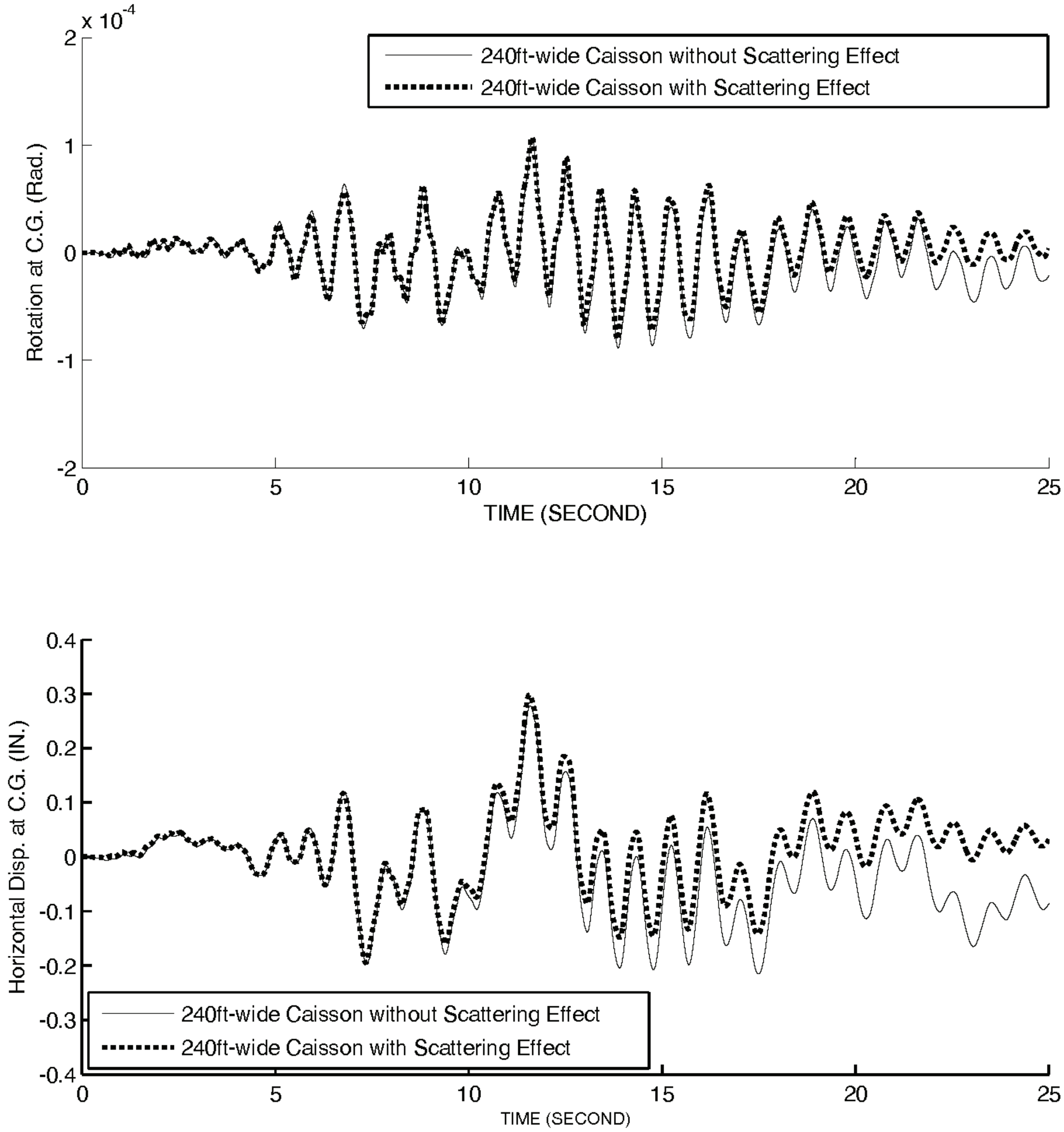


Figure 4-12   Seismic Response of Caisson with the Base Width of 240 feet

4.4 Case History Examples

4.4.1 Carquinez Bridge, California

The following discussion is extracted from a paper published by Law and Lam (2006) that describes experience gained from the Caltrans Seismic Retrofit Program for Toll Bridges in California.

In early 1990 Caltrans was commissioned to conduct a seismic vulnerability assessment of the existing Carquinez bridge. An analytical technique based on an elastic-dynamic approach was used to model the caissons, which requires everything to be linearly elastic, i.e., the caisson is perfectly 'glued' to the ground without allowance for separation and the surrounding soils have unlimited shear strength. This resulted in enormous shear forces within the caissons when used in the global bridge model for seismic loading such that the shear forces would have crushed the caissons. The conclusions from this vulnerability study led to the development of a retrofit strategy for the caisson foundations of the bridge. The estimated cost of retrofitting the caissons exceeded the budget allocated for the bridge retrofit program.

Caltrans then engaged Earth Mechanics, Inc. for a second opinion of the recommended retrofit strategy. After evaluating the previous study, it was determined that the magnitude of the shear forces was not sustainable as the caisson would have toppled at the computed shear forces. To develop a more correct deformation mechanism, nonlinear modeling was adopted to evaluate the performance of the gravity caissons. The nonlinear approach allowed for geometric nonlinearity due to uplifting at the base of the caisson and material nonlinearity due to yielding of the soil. The analyses indicated that the maximum shear force that could develop in the caisson was limited to the overturning moment associated with the deadweight and the half width of the caisson, divided by the height of the center of gravity. This more correct modeling of the caissons also contributed to realistic and successful modeling of the global bridge model for the seismic analysis. The conclusion that the existing caissons of the bridge were not vulnerable to earthquake damage resulted in significant savings in the retrofit cost for the bridge. The retrofit was then successfully completed marking the first completed seismic retrofit of California toll bridges.

The caisson models of the Carquinez bridge that were used in the final Plans, Specifications and Estimates stage were represented using lumped nonlinear moment-versus-rotation and base shear load-versus-lateral displacement relationships, as illustrated in Figure 4-13. This nonlinear lumped foundation behavior was established by performing finite element pushover analyses to capture essential elements of soil-structure interaction phenomena and to consider the limiting force and moment. Because of the nonlinear nature of foundation behavior, uncoupled springs must be used; i.e., the load-versus-displacement relationship and moment-versus-rotation relationship were operated independently. To evaluate whether the uncoupled springs performed appropriately, the shear load and overturning moment of the caisson computed from the finite element global bridge model were checked to ensure that the assumptions made during the pushover analyses were valid.

A refined caisson model can be made to reconcile the importance of coupling between shear and moment loads. This model entails Winkler springs distributed over the bottom surface of the caisson to represent the soil continuum underlying the foundation, and another sets of soil springs attached to the vertical sides of the caisson walls to model passive soil pressure acting on the concrete. The soil springs may be nonlinear to consider yielding of localized soil. In addition, gapping elements can also be implemented in series with the soil springs to engage a full contact between the soil and the caisson during compression and to allow separation under tension. Figure 4-14 illustrates a distributed soil spring model used in the seismic analyses of the Second Tacoma Narrows Bridge Project as discussed in the second case history. This modeling approach addresses the two significant features; nonlinear behavior and coupling between lateral loading and overturning moment. Therefore it is a significant improvement over the lumped spring models. Establishing proper distributed soil springs is a key to successfully modeling the caisson.

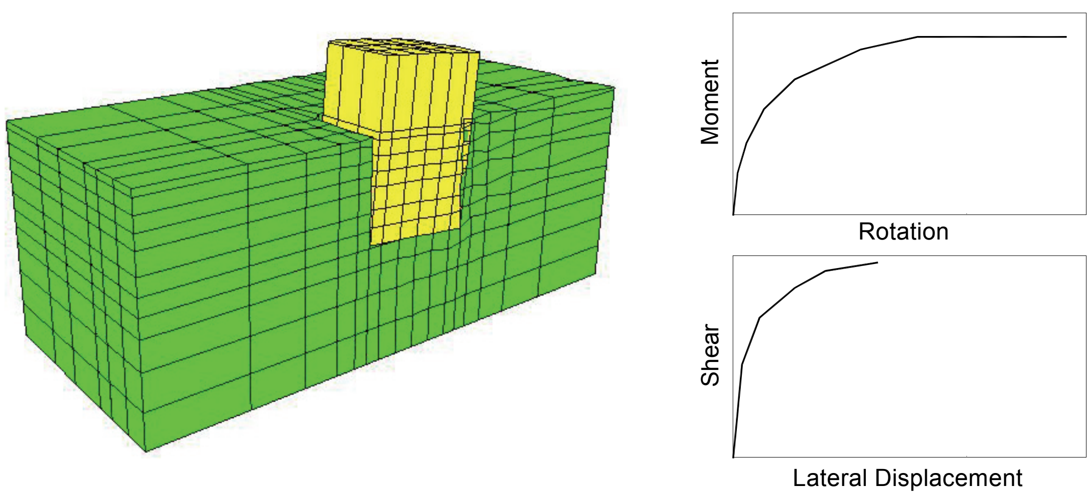


Figure 4-13 Pushover Analysis of Gravity Caisson

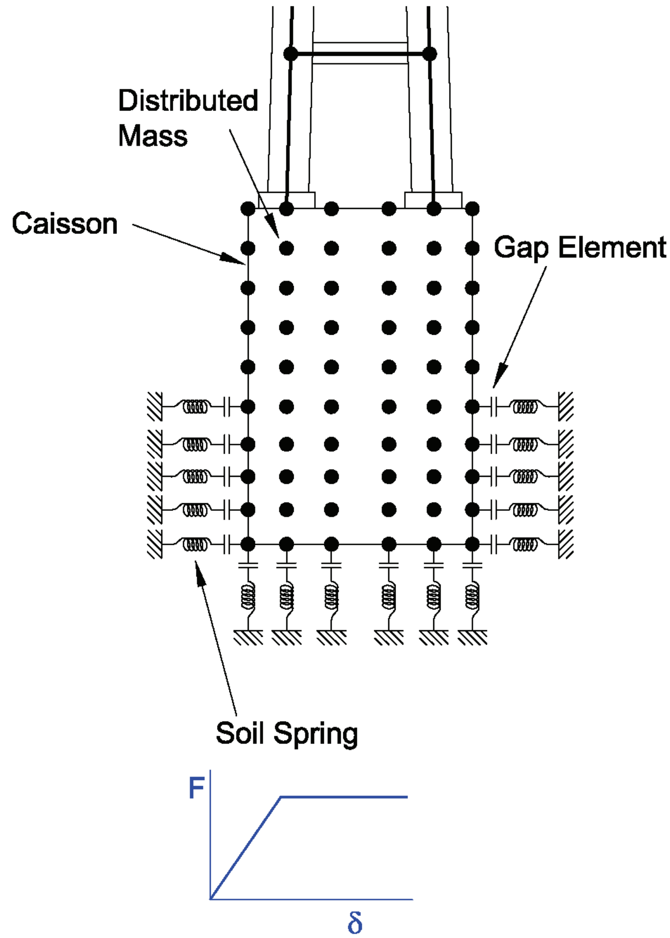


Figure 4-14 Distributed Spring Model

4.4.2 Tacoma Narrows Bridge, Washington

The following discussion is extracted from a paper by Martin and Lam (2003), describing geotechnical issues related to seismic displacement demands on bridge structures.

The design of the Tacoma Narrows bridge second crossing Treyger et al.[2004], provides a very good case history of current trends in implementation of a displacement-based design principal for foundations. The Tacoma Narrows bridge was designed and constructed under a design-build agreement between the State of Washington and Tacoma Narrows Constructors. The bridge is scheduled for completion in 2007. When completed, the new bridge will be the first major suspension bridge in the world to be constructed under a design-build contracting arrangement. It is also the longest span built in the US since the Verrazano Straights Bridge was completed in 1964. The new suspension bridge will consist of a 2,800 ft main span supported by reinforced concrete towers. The towers will be founded on massive gravity caissons of open-dredge construction. The East and West towers of the Tacoma Narrows Bridge are each supported on large cellular caissons with plan dimensions of 80-ft by 130-ft. The approximate heights of the East and West caissons are 230 ft and 210 ft, respectively, of which approximately 60 ft will be embedded below the mud-line. The East caisson has a total dead load of approximately 190,000 kips which increases to over 260,000 kips, when accounting for the water in the dredge cells.

The site is a very deep soil site consisting of glacial deposits that provide a competent bearing stratum for the caissons. The project area is in a region of high seismic potential, capable of producing subduction earthquakes as well as shallow crustal earthquakes. Probabilistic seismic hazard analyses were performed and two levels of earthquakes, 100-year (functional level) and 2,500-year (safety level) were established. For the safety level earthquake, three sets of spectrum-compatible motions were defined by the state, with two sets of motions representative of a subduction zone event from a farther distance, but much longer duration in shaking. The third set of safety level design motion was developed for a closer distance crustal earthquake event, including representation of near-fault directivity forward-rupturing ground motion features of a high velocity pulse in a specified direction. This third set of near-fault forward rupturing input motion was found to govern most of the foundation design aspects, especially the amplitude of permanent deflection of the caisson due to the very high velocity pulse loading the caisson in one direction.

A clause in the state's seismic design criteria specifies a maximum permanent deflection of 1-ft at the top of the caisson that played a major role in determining the caisson dimensions of the caisson. A major issue concerning the soil-structure interaction (SSI) task relates to sensitivity analyses for predicting the permanent deflection of the caisson for various depths of embedment of the caissons. The forgoing presentation discusses the SSI model adopted for the project.

<u>Winkler Spring SSI Model</u>

The caisson foundation model used for the global bridge dynamic response model is schematically shown in Figure 4.15, which consists of a set of nonlinear Winkler springs to capture the local nonlinear aspect of SSI behavior. As shown in Figure 4.15, the Winkler Spring SSI model included the following components:

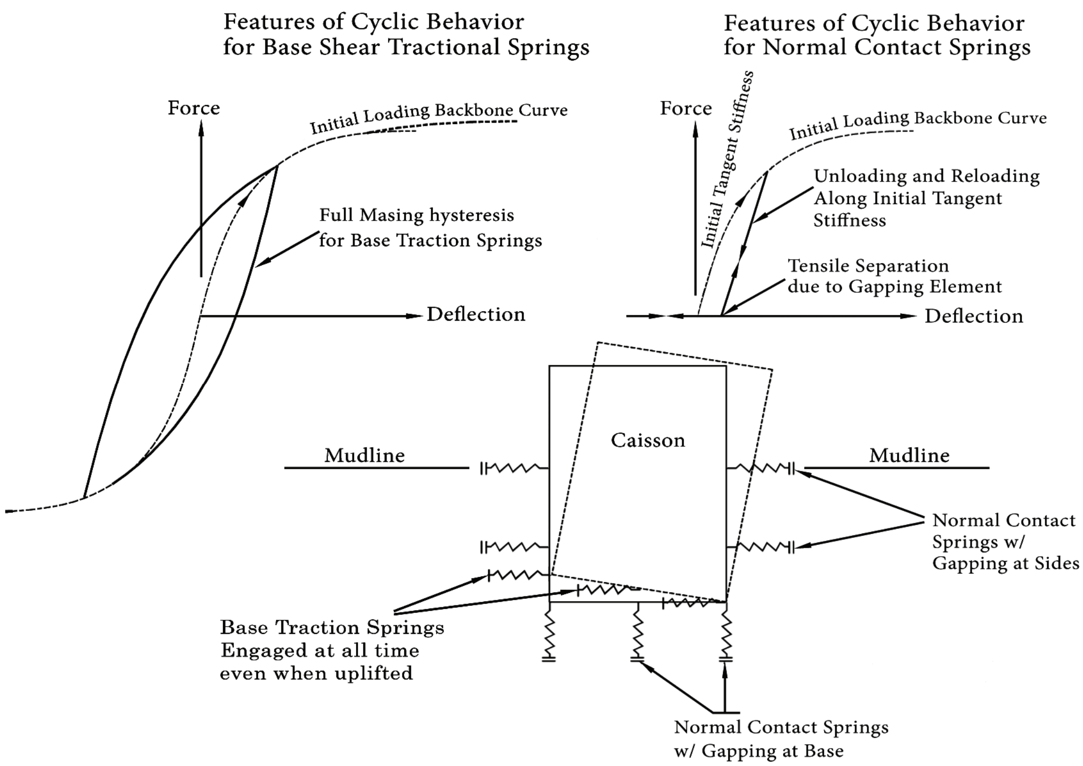


Figure 4-15 Concept of Nearfield Local SSI Winkler Spring Model for Global Response Analysis

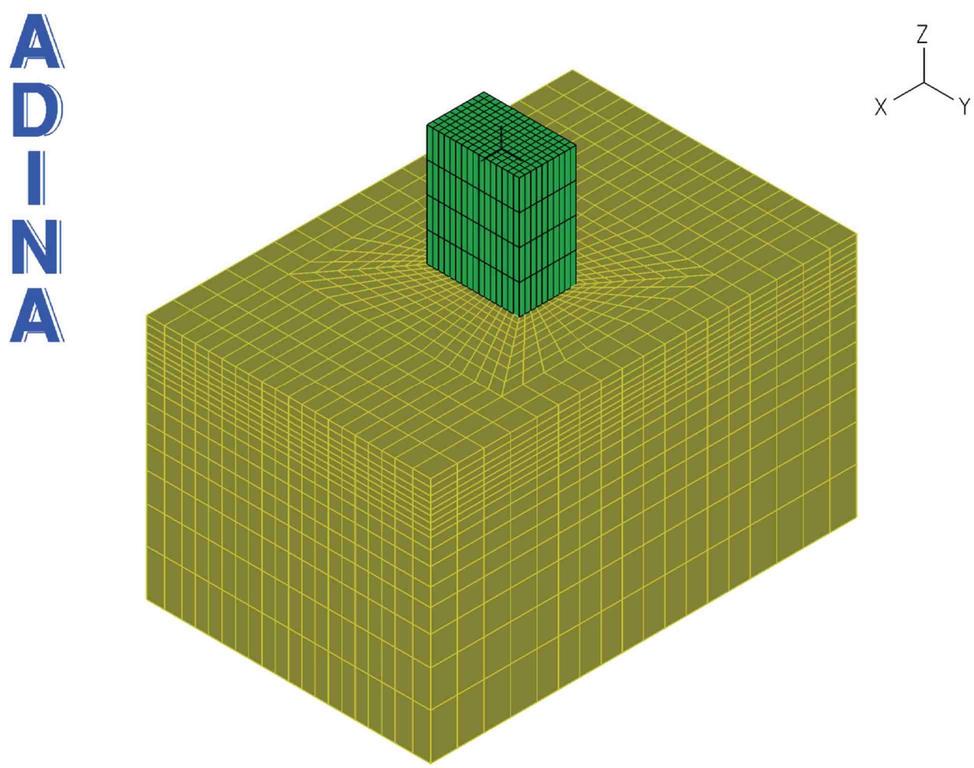


Figure 4-16 ADINA Soil Continuum Model used for Deriving the Winkler Spring Model shown in Figure 4-15

- Normal contact spring elements at the base of the caisson and the four vertical side surfaces of the embedded caisson-soil interfaces. A gapping element is attached in series to each of the nonlinear Winkler spring elements to capture the gapping effect under tensile separation loading conditions.
- Tractional shear spring elements at the base of the caisson-soil interface for simulating the lateral base shear soil reaction.

In addition to the use of the nonlinear Winkler spring SSI model to account for the local nonlinear SSI behavior, each of the nonlinear Winkler spring elements can be connected in series to a far field impedance element. This consists of a relatively stiff linear Winkler spring in parallel with a viscous dashpot element, which can be developed from elasto-dynamic theories to capture radiation damping effects in accordance with the concept proposed by Wang et al (1998). The SSI foundation model was selected to capture the rocking behavior of the caisson foundation (concluded to be the most dominant SSI response feature), including the ability to account for permanent displacements of the caisson.

The SSI model was implemented by the structural analysts into the global bridge dynamic response model, with each discrete SSI element was modeled by a zero-length element with one node attached to the caisson node and the other node connected to the free-field ground node where three-component near field input motions, (derived by free field site response analysis) compatible to the ground motion shaking criteria defined by WSDOT, was prescribed.

Figure 4-16 depicts the soil continuum finite-element model used to derive the local near field SSI Winkler spring element in the global response model discussed above. The ADINA program, used by the structural engineers in global dynamic response analysis, was also used by the geotechnical engineers in performing the soil continuum finite-element SSI solutions for developing the Winkler spring SSI model parameters. This has many advantages including the potential for implementing the soil continuum soil mass system (in place of the Winkler spring model) into the global bridge response model to capture improved SSI solutions, if improvement was necessary in the future. Another advantage of using a common numerical platform is to allow an independent check of the SSI model by the structural engineers.

Close coordination between the structural designers and geotechnical engineers was necessary in defining the discretization scheme of the Winkler spring model shown in Figure 4-15 (the number of springs shown in the figure are schematic only) based on results from the finite element model shown in Figure 4.16. In the course of the project, sensitivity studies were conducted to establish the degree of refinement in the discretization scheme at the caisson base SSI springs to achieve stable rocking behavior of the caisson (see Treyger et al. 2004). A 5x5 (25-node) base spring scheme was ultimately used to capture the caisson rocking behavior. Pushover analysis was first conducted using the finite-element model by a point load applied at a selected elevation of the caisson. The following are some details of the finite-element pushover analysis:

- 3-D brick elements for the soils and caisson, where the caisson was assumed to be rigid as represented by linear elastic concrete properties.
- Nonlinear elasto-plastic soil properties are modeled based on insitu site soil properties.
- Gapping elements are modeled at all caisson-soil interfaces to allow for tensile separation.
- Horizontal interface traction springs are modeled at the base of the caisson-soil surface to account for potential interface friction failure between soil and concrete.

The derived Winkler Spring model is dependent on the loading condition used in the pushover analysis in addition to the soil properties. The elevation of the point of loading in Figure 4.16, was chosen to introduce the proper base moment to the base shear loading condition. Because the dead weight of the caisson constitutes the bulk of the static dead load (more than 60% of the overall bridge), the point of loading shown in Figure 4-16 was selected at an elevation corresponding to the center of gravity of the caisson for a first order approximation of the first mode of response of the caisson. A load-deflection curve was extracted, using the load-deflection characteristics extracted at each caisson-soil interface node (shown in Figure 4-16) which was then used for formulating the backbone curve in the Winkler spring model in the dynamic global response model (shown in Figure 4-15). In effect, the Winkler spring SSI model becomes a substitute model that replaces the soil continuum.

Prior to the use of the derived Winkler spring SSI model (shown in Figure 4.15) for dynamic response analysis, it was compared to the soil continuum SSI model (shown in Figure 4.16) in a pseudo-static load and unload cycle. The exercise included exploring various options available from the ADINA program for extending the initial loading backbone curves to cyclic unloading and reloading conditions. Some of these options include a nonlinear but elastic option where unloading follows the initial loading path, implying zero permanent deflection, and other options capable of generating hysteretic cyclic behavior which have damping and permanent deflection implicitly implied in the model. Some of the results of the load and unloading solutions between the two models are shown in Figure 4-17.

From the comparison, it can be concluded that the Winkler spring model reproduces the behavior observed from the soil continuum model, not only for the initial loading path, but also the unloading cycle, and hence has the potential to capture permanent deflection and damping characteristics in addition to the initial loading stiffness and the failure capacity of the soil-mass system.

Pseudo-Static Back Substitution of Peak Global Response to Continuum SSI Model

The Winkler spring model was adopted for dynamic response solutions in order to reduce the computational effort. It is an approximation of the more rigorously correct continuum model necessary to facilitate sensitivity parametric variation studies necessary in a normal design process. However, the simplifications involved in the Winkler spring model need to be checked for better appreciation of the soil response (e.g. evaluation of soil stresses, etc.) by pseudo-static back substitution analyses using the continuum model. A point load applied at a specified height above the caisson base was used for the pushover load and unload back-substitution analysis. The peak load in the back-substitution analyses was based on solutions of the peak base moment and shear load from the global response solution. The back-substitution pushover solution included examining the simultaneous superposition of the transverse load at the time of the peak longitudinal loading direction. In the course of the design, various parametric studies were evaluated, including scoured and unscoured caisson design scenarios along with best estimated, upper and lower bound soil property runs.

Figures 4-18 and 4-19 present the back substitution solutions for a partially scoured case using the finite-element model shown in Figure 4-16 for a static pushover load and unloading path. Solutions from the global response model for the worst input motion record (the near fault high velocity pulse motion set) were evaluated to formulate the combination of peak caisson base moment and its corresponding base shear in simultaneous orthogonal loading directions (longitudinal and transverse) in the pseudo-static load and unload finite element model solution. Figure 4-18 presents the load-unload force-displacement solution while Figure 4-19 presents information regarding base uplift and peak soil force reaction at the time of peak load. From the presented load-deflection plots, we note that the residual deflection was below, or close to the permanent deflection limit of 1-ft for the pseudo-static load and unload sequence of loading. This is compatible with the most conservative scenario for the large velocity pulse occurring at the end of the earthquake time history. Most of the global model (with Winkler spring SSI model) time history response solutions (shown in Figure 4.20) showed lower amplitudes of permanent caisson deflection. Hence, the design team concluded that the selected caisson dimensions (130-ft by 80-ft plan dimension) and about 60-ft embedment depth met the State's global stability criterion of permanent deflection for the caisson (less than 1 foot).

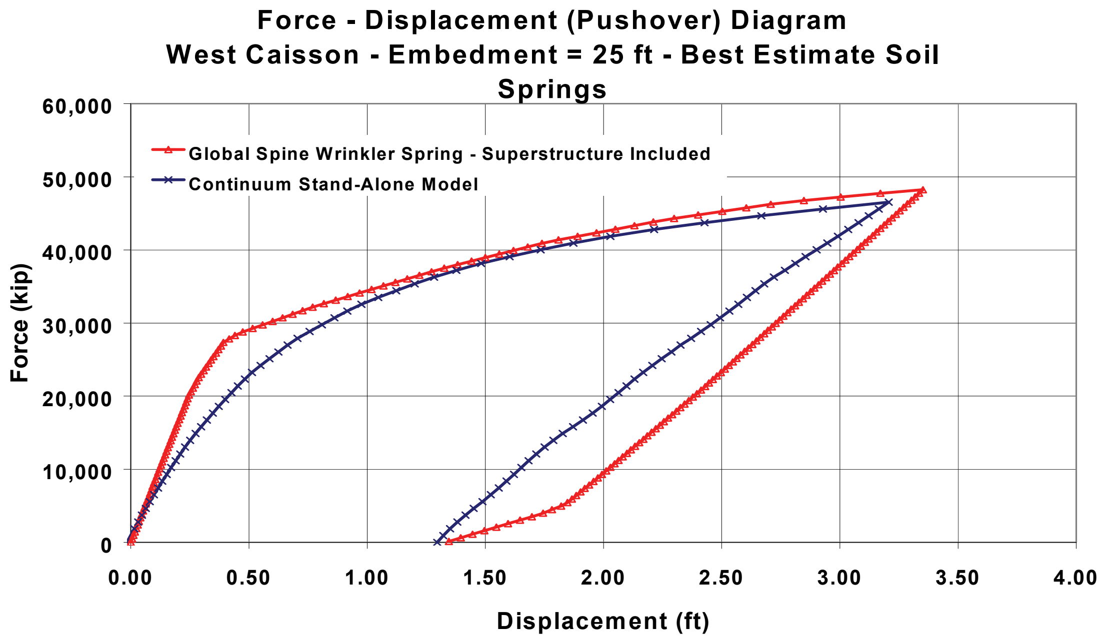


Figure 4-17 Verification of Winkler Spring SSI Model by Comparison to Continuum Model Solution

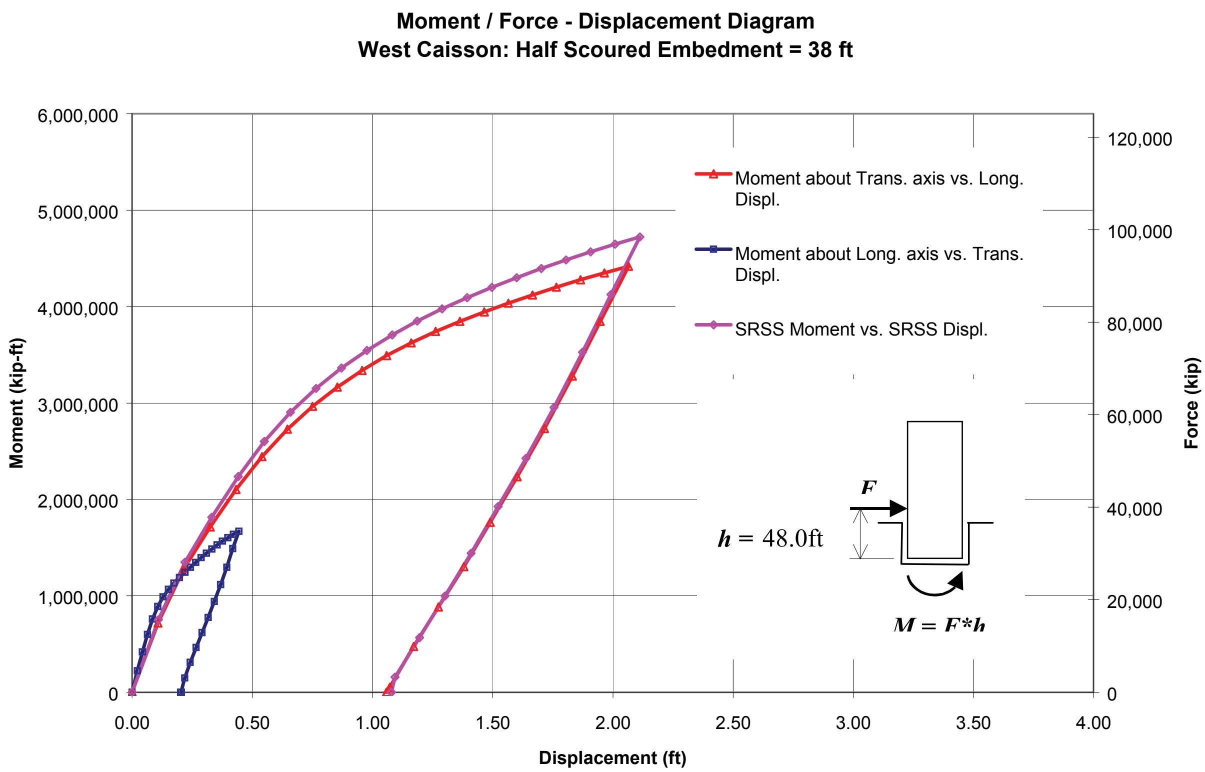


Figure 4-18 Continuum Model Back Substitution Solution for Half Scoured Case

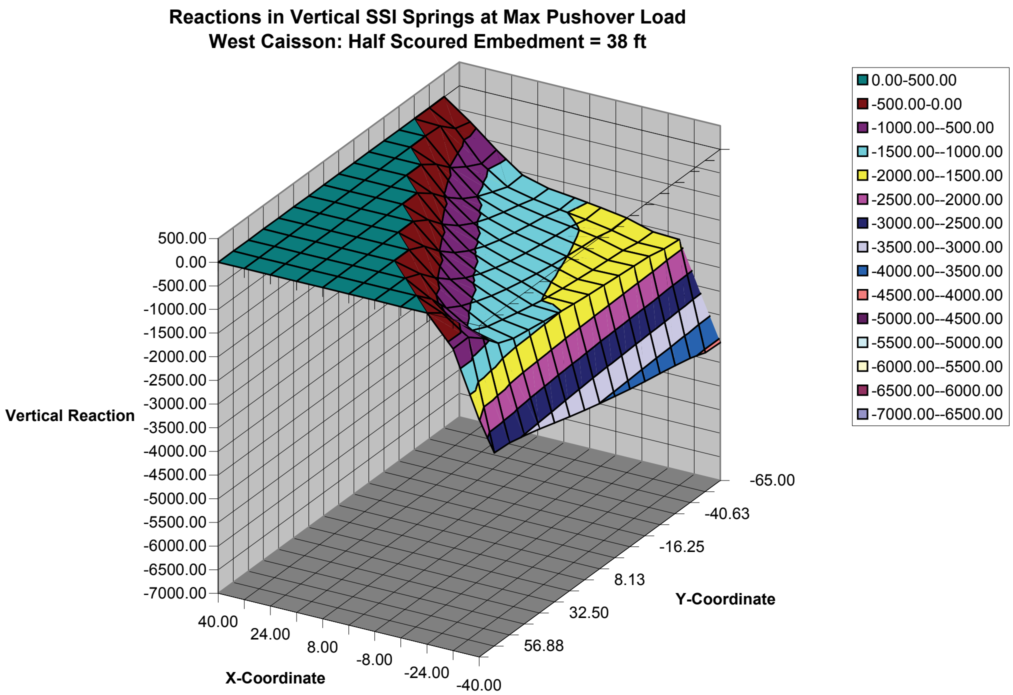


Figure 4-19 Continuum Back Substitution for Soil Reaction at Base. (Divide spring forces by 65 sq. ft for base pressure)

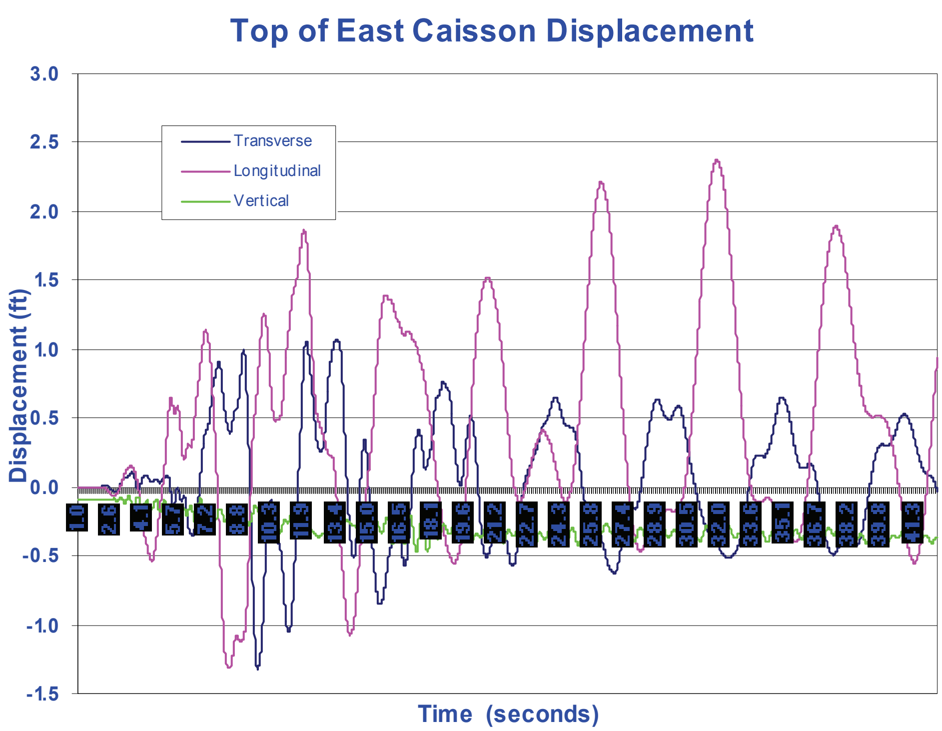


Figure 4-20 Representative Caisson Response Solutions

Displacement Time History Response

Representative displacement time history response at the top of the caisson is shown in Figure 4-20, from the global response model for the near field event time history input. It can be observed that the period of response of the caisson was about 2-seconds initially (as anticipated), but lengthens progressively to over three second period. Further examination of the solution led to the realization that the period lengthening behavior is due to limitations in the Winkler spring model under cyclic loading conditions. The cyclic response option had been formulated based solely on observed soil specimen shear deformation stress-strain behavior, which does not account for the volumetric behavior (e.g. conservation of soil volume) such as Poisson ratio effects. Along with the gapping model shown in Figure 4-15, the Winkler springs are progressively depressed in the course of the earthquake time history. Settlement of the Winkler springs increases toward the edges of the caisson as if rocks back and forth. At the end of the earthquake, the displaced Winkler springs result in the ground deflecting into a bowl shaped soil surface underneath the caisson. Hence the caisson is separated from the soil surface at the outer parts of base. Considering that the caisson base is embedded in about 60-ft of soils, and the soils at the caisson base are highly confined, this permanent deflected ground condition appears artificial. It is speculated that the Winkler spring model overly exaggerated the permanent deflection of the caisson. In the course of the design process for the Tacoma Narrows bridge, sensitivity studies, including a solution using the nonlinear elastic option for the Winkler spring model, and also modifying the Winkler spring backbone curve for an upper bound and a lower bound condition, were necessary to accommodate this limitation when using implement the Winkler spring model for earthquake design problems. Designing for an upper bound stiffness case, which generally leads to a higher force response in the super-structure, is an important design case.

Lessons Learned and Design Issues

Conceptually, seismic design criteria for caisson foundations needs to be related to the anticipated displacement response amplitude, especially when considering the permanent displacement of the foundation after the earthquake time history. However, there are significant difficulties in predicting the permanent deflection. These include:

- Difficulty in identifying and quantifying parameters in the input time history that induces permanent response in structures. In the three sets of spectrum-compatible motions, there is minimal permanent caisson deflection from the two sets of subduction zone input time histories. The observed permanent deflection was highly sensitive to the unsymmetric velocity pulse inherent in the near-fault input time history, especially in the time phasing of the velocity pulse. At the present time, there is very little work in the literature to quantify such complex features (phasing and duration effects) inherent in some of the earthquake input time histories. This aspect will pose difficulties in the implementation of a design criteria keyed to permanent foundation deflection.

- As shown in the above discussions, there is significant difficulty or questions on the ability of the Winkler spring model to predicting permanent deflections. Such models are needed to reduce the size of most global bridge models to be manageable. Also, the simplicity in the implementation of Winkler spring models facilitates sensitivity studies, which are very important for a robust design of any structure. However, it should be recognized that nonlinear Winkler spring models are primarily a means for modeling the foundation secant stiffness in earthquake response. Solutions for permanent deflections need to be used with caution. In general, such models tend to over-exaggerate the computed permanent deflection of the foundations.
- For caisson type foundations, direct implementation of the soil mass using finite-element methods in global response solutions are becoming manageable due to the advances in computer technology. The soil continuum model can more rationally account for Poisson's ratio and soil volume conservation and other kinematic compatibility issues and should yield more realistic permanent deflection solutions. However, there are other difficulties associated with the use of continuum finite-element models to represent the soil mass and more research is needed in: (1) constitutive modeling and (2) improving our understanding in how to treat boundary conditions and in how to introduce the appropriate input motion to achieve the defined reference ground excitation into the structure model.
- At the onset of the project, an attempt was made to apply force-based capacity design principles for the caisson foundation design. Because of the rigid caisson, there is no rational load fuse to key the force level for foundation design, hence implying the need for designing to the full elastic force. The resultant design would become cost prohibitive, not only for the caisson, but also for the superstructure due to much higher elastic forces transmitted to the superstructure. The displacement-based design principles, discussed above provide the only workable rational basis for the design of caisson-type foundation systems.

SECTION 5:
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