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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.

This report documents practical modeling procedures adopted in the bridge engineering community that involve seismic design and retrofit of long span bridges relative to the treatment of wave propagation problems. It also discusses wave scattering issues that arise from irregular foundation boundaries and affect the seismic loading of bridges, which is not explicitly considered in current design practice. The research reported herein developed a systematic procedure that allows geotechnical engineers to perform wave scattering problems in the time domain considering all elements of boundary issues to minimize wave reflection and refraction. The solutions obtained from the time domain approach were rigorously compared against a traditional frequency domain approach. The established time domain procedure was shown to address not only free field conditions but also foundation systems subjected to earthquake loadings. Once the time domain procedure was established, the work was extended to address pile group foundations and gravity caissons under dynamic loading conditions, to gain an understanding of how the scattering of seismic waves from the foundation system alters the near field ground motion transmitted to the superstructure. The effects of wave scattering can now be more easily included in future design procedures.

ABSTRACT

This report documents practical modeling procedures adopted in the bridge engineering community involving seismic designs and retrofits of long span bridges relative to treatment of wave propagation problems. It also discusses wave scattering issues arising from irregular foundation boundaries affecting seismic loading of the bridges, which is not explicitly considered in the current design practice. Wave scattering is generally implemented in the nuclear power plant industry for seismic designs of various containment systems often using frequency domain computer programs. To examine the subject of wave scattering for application to long span bridge foundations, systematic modeling is exercised using a time domain based computer program and verification is made against a frequency domain computer program.

For present day seismic designs of major bridges, nonlinear time history analysis is a common procedure to examine seismic loading of the structure permitting plastic hinging and ductility to be implemented. Thus, the current trend is to adopt time domain based computer programs for bridge structural analyses. The authors also promote time domain computer programs for performing wave scattering analyses which can also serve as a common platform to be used by both geotechnical and structural engineers for the global bridge model. A major benefit is to minimize the amount of work for data transfer and potential error arising from two different groups (geotechnical and structural engineers) working on different computer codes requiring different input/output. By using the same computer code by both geotechnical and structural engineers, many problems are eliminated.

Typically, wave scattering analyses are conducted in the frequency domain. This report presents studies of wave scattering using a time domain computer program. The same computer program can be used by structural engineers to proceed with coding the superstructure model, directly using the results from the wave scattering analysis. The report presents various sensitivity analyses in order to minimize wave reflection and refraction at the model's side boundaries. Numerical integration schemes and implementation of Rayleigh damping parameters are discussed. Careful examination of waves traveling through the bottom boundary allows proper modeling of the half-space below the region of interest.

The studies explore the effects of wave scattering on large pile groups and soft ground conditions, and findings on the frequency ranges where significant scattering is observed are reported. Large caissons are known to affect seismic wave scattering due to the large wave length implied by the dimensions of the foundation embedded in soil. Parametric studies are performed to examine the shaking level that is altered by the wave scattering mechanism. From the current findings, it appears that the wave scattering tends to reduce the shaking level, especially in the high frequency range, and hence is beneficial to the bridge design.

TABLE OF CONTENTS

LIST OF FIGURES

FIGURE **TITLE** **PAGE**

LIST OF FIGURES (CONT'D)

FIGURE TITLE **PAGE**

LIST OF TABLES

TABLE **TITLE** **PAGE**

SECTION 1
INTRODUCTION

1.1 Objectives

This report presents studies on soil-structure interaction of bridge foundations under dynamic loading conditions relating to wave propagation and wave scattering considerations. The framework of this report is to achieve objectives for advancing the state-of-practice for seismic analysis of long span bridges where different foundation types are used including large pile groups and gravity caissons. The objectives of this research are as follows:

(1) Development of a systematic procedure that allows geotechnical engineers to perform wave scattering problems in the time domain considering all elements of boundary issues to minimize wave reflection and refraction. The solutions obtained from the time domain approach must be rigorously compared against a traditional frequency domain approach. The established time domain procedure should be able to address not only free field conditions but also foundation systems subjected to earthquake loadings.

(2) Once the time domain procedure is established, the work is extended to address pile group foundations and gravity caissons under dynamic loading condition. The objective here is to understand how the scattering of seismic waves from the foundation system alters the near field ground motion transmitted to the superstructure. Rational judgment can then be made to include wave scattering effects in future design procedures.

1.2 Overview

The subject of site response and wave scattering has traditionally been treated by frequency domain computer codes such as SHAKE (Schnabel, et al, 1972), FLUSH (Lysmer, et al, 1975) or SASSI (Lysmer, et al, 1999). For the past 20 years, these programs have been used in the nuclear power plant industry for seismic designs of containment systems, which are typically founded on competent soil conditions. Because of strict requirements on seismic performance of these structures, no large deformation beyond an elastic range is allowed. As a result, linear solutions based on frequency domain programs such as SASSI and FLUSH are adequate in the design process.

For bridge engineering, the performance goal is very different from the nuclear industry. The current California Department of Transportation (Caltrans) Bridge Design Specification (BDS) follows the ATC-32 guidelines (Applied Technology Council, 1996), which defines the performance depending on whether the bridge is classified as ‘Important’ or ‘Ordinary’. The performance goal for Ordinary Bridges allows fully ductile behavior by taking maximum advantage of plastic hinging while ensuring the structure safety; this implies considerable damage but with no collapse. For Important

Bridges, limited ductility is allowed to limit the inelastic response to ensure reduced structural damage and yielding of the critical foundation components. Due to allowance for large deformation beyond the elastic range, the frequency domain based computer codes without consideration of nonlinear behavior have limited applications in bridge engineering. In current design practice, time history based computer codes have been used to study global response of bridge structures, especially major long span bridges. However, the site response aspect is often treated in the frequency domain (e.g., SHAKE), which is undertaken by geotechnical engineers.

Because of a lack of common platform for the analyses conducted by the geotechnical and structural engineers, substructuring techniques are needed to implement the results of site response and foundation stiffness (impedances) from the frequency domain approach into the subsequent time domain global bridge analysis model. This two-step approach by separate group of engineers (i.e. structural analysts and geotechnical analysts), conducted on completely different numerical platforms, could lead to confusion and delay in projects. In many cases, the need for transferring large volumes of ground motion data between different computer platforms results in unintended mistakes.

It is highly desirable to advance the state-of-practice such that wave propagation and scattering analyses can be conducted in time domain, or in a numerical platform that is commonly used for global bridge response analyses. This will allow both groups of specialists (geotechnical professionals who is familiar with wave propagation analyses, and structural engineers performing the global structural model) to work simultaneously on the same computer platform to contribute both their expertise to the total problem. The current research program steers into this direction by advocating a numerical procedure, which consists of performing site response and wave scattering problems entirely in the time domain.

1.3 Organization of Report

Section 1 of the report describes the objectives of studies and provides an overview of the problems encountered in current design practice. Organization of the report is described.

Section 2 provides discussion on current design practice and acting on the subject of wave scattering that could potentially affect near-field ground motions of the foundation. The general framework of the research is discussed here.

Section 3 contains a time domain procedure for evaluating wave propagation of level ground with consideration of boundary conditions and treatment of damping. Sensitivity analyses are conducted to examine various parameters that could potentially affect the time domain solutions in comparison with the frequency domain approach.

Section 4 is dedicated to wave scattering of large pile group foundations. This section illustrates that reasonable time domain solutions can be obtained by careful implementation of boundary conditions. Comparison between time domain and frequency domain solutions is documented.

Section 5 discusses seismic wave scattering and the modeling approach for large gravity caissons. Caisson seismic responses with and without wave scattering are compared.

Section 6 provides references cited in the report.

SECTION 2
WAVE SCATTERING AND EFFECTS ON NEAR FIELD MOTIONS

2.1 Current Design Practice

Seismic design of a bridge is often carried out by two groups of professionals: geotechnical engineers and structural engineers. These two professionals are normally employed by separate organizations. The tasks of the geotechnical engineer consist of performing site investigations, defining ground motion parameters and providing foundation recommendations. The structural engineers then build a structural model, perform seismic analyses and provide plan drawings. During this design process, the geotechnical and structural engineers must work together in order to develop the global bridge model, which requires input from both professionals.

It is very common to adopt a substructuring approach where the foundation components are separately developed by the geotechnical engineer. The outcome of the foundation modeling consists of simple representations of the foundation system by a stiffness matrix (foundation impedance) and a loading function in the form of kinematic motion. The structural engineer then integrates the foundation stiffness matrix and the kinematic motion into the global bridge model to perform seismic response analyses. For most major long span bridge projects, the global bridge analyses are conducted using a time history approach, taking advantage of ductility of critical structural components. Modeling of the foundation components, however, has been undertaken in either the frequency domain or the time domain. However, frequency dependency of foundation impedance is not considered and may be shown to be insignificant for typical earthquake loading rates.

2.2 Foundation Modeling

2.2.1 Pile Foundations

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 for a complete system model where the bridge superstructure and individual piles with soil springs are included (see upper sketch of Figure 2.1). Because large degrees of freedom are needed to formulate the complete system, the 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 2.1). This approach allows evaluations of foundation and bridge structure separately as a means of reducing the problem into a manageable size. The substructuring approach is based on a linear superposition principle, and therefore linear soil springs are more applicable in representing soil behavior. When linear soil springs are to be used, the spring stiffnesses are generally evaluated by the pushover analysis of the pile for linearization by displacing the pile to a level that is anticipated during actual loading conditions.

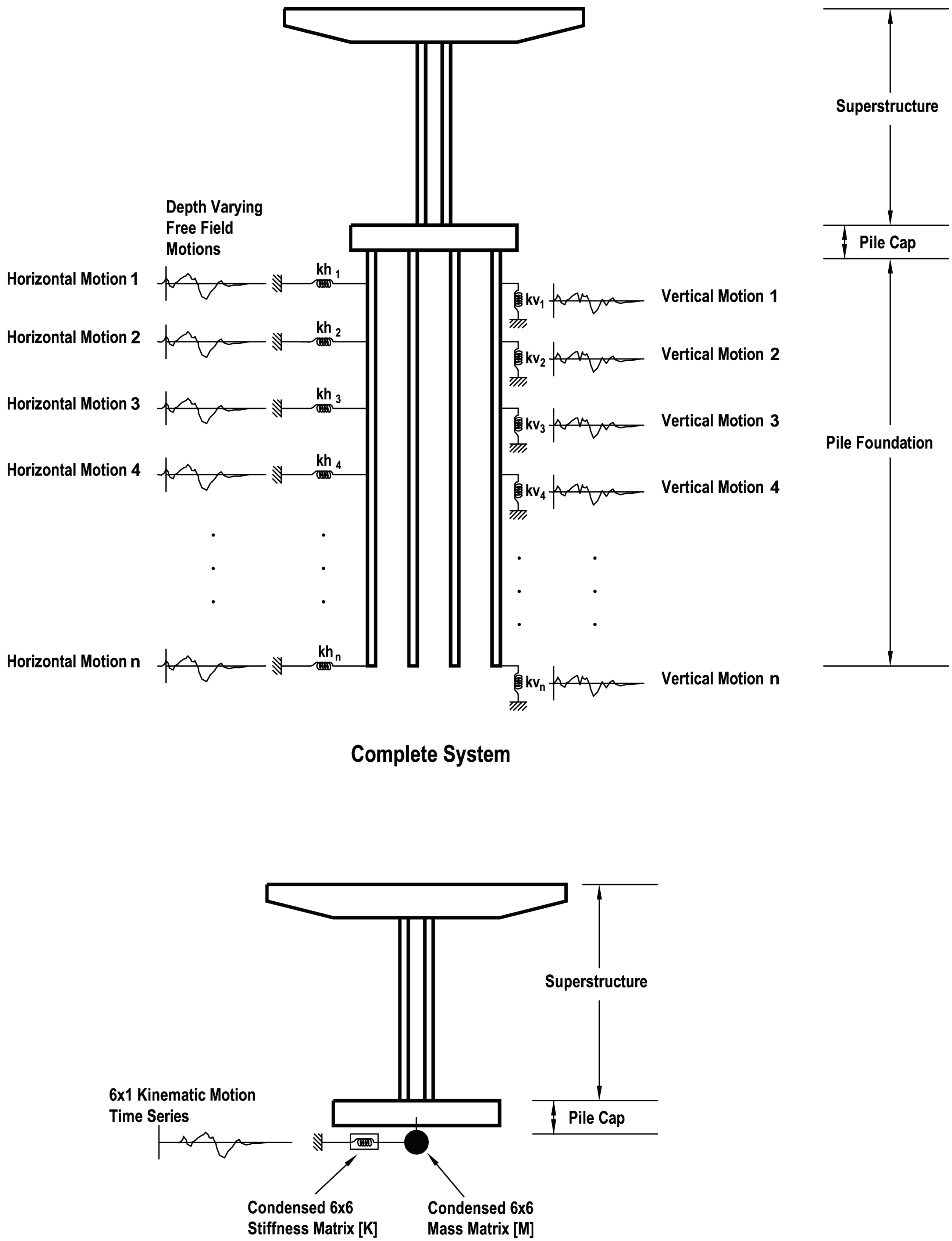


Figure 2.1 Modeling of Pile Foundation for Seismic Design of Bridges

In the case of earthquake excitation, ground motion would impart loading at each soil spring, and particle motions arriving at different soil layers would have different characteristics influenced by local soil conditions. Prescribing depth-varying ground motions at the ends of soil springs is often used to simulate the earthquake loading condition. For the current state of practice, the depth varying input motions prescribed along the pile are extracted from results of one-dimensional wave propagation analyses such as those using SHAKE program (Schnabel et al., 1972). All one-dimensional site response programs assume that soils are horizontally layered and that shear waves propagate vertically.

It is uncertain whether a vertically propagating shear wave assumption is reasonable for pile group problems since inclusion of large number of piles in the soil would have deviated the horizontally layered soil condition at least within the footprint of the foundation. The immediate region where piles are installed would have an increase in effective modulus resulting in a stiffness contrast between the free-field soil and the pile-reinforced soil mass. Furthermore, a slight increase in overall mass density is expected within the foundation footprint.

2.2.2 Caissons

Large caissons have been employed for some of the largest bridge piers. 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 is undertaken at its final site, supported on a temporary sand island, and then sunk by dredging out within the open cells of the caisson. Because of the massive volume of concrete, the weight of the caisson constitutes the bulk of the entire bridge structure. Consequently, seismic behavior of bridges supported by caissons is highly influenced by the response of the foundation. To capture accurate dynamic response of the bridge structure, the caisson foundation must be modeled properly.

Figure 2.2 illustrates some of the modeling practices for gravity caissons in connection with a seismic response study of the bridge. It is not practical to model the entire caisson with a soil continuum in the global bridge 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 (see Figure 2.2.c). However, this does not explicitly consider potential uplift between the caisson and the underlying soil, which sometimes attracts unrealistically high loads due to the assumption of the fact that the caisson is 'glued' to the surrounding soil. An alternative approach is to support the lumped mass of the caisson by non-linear foundation springs representing load versus deflection relations and moment versus rotation relations in all three orthogonal directions (see Figure 2.2.b). This would consider potential uplift when excessive rotation is encountered during the seismic loading. If the caisson is embedded relatively deep into the soils, coupling between the shear and the overturning moment becomes important, and hence care must be taken when developing the non-linear foundation springs so that the relationship between shear and moment is correct while allowing a mechanism for potential uplift.

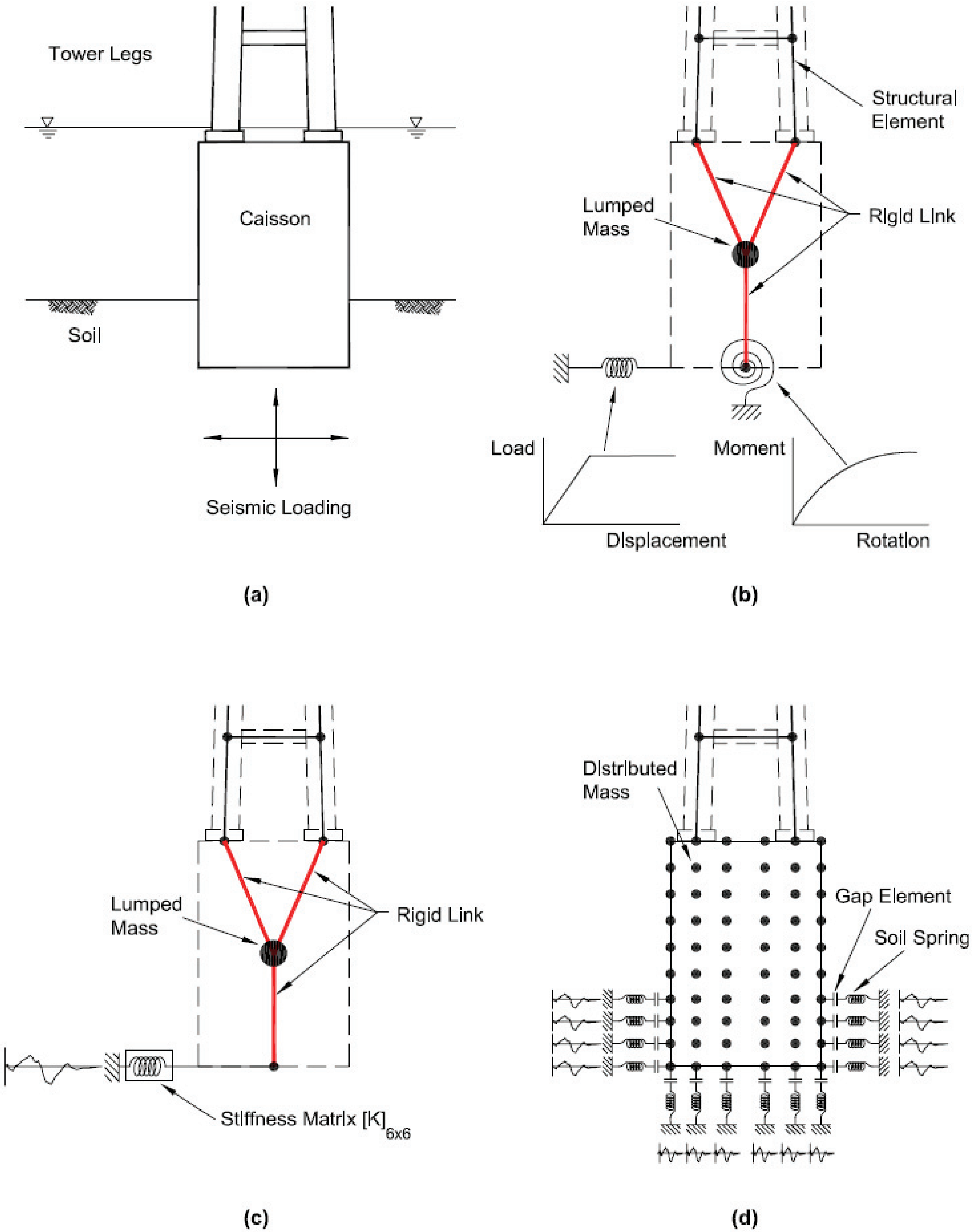


Figure 2.2 Various Modeling Techniques for Gravity Caisson

A more refined model is used in which the gravity caisson is represented by distributed masses; these masses are rigidly linked together (Figure 2.2.d). The exterior nodes of the caisson model are then supported by distributed soil springs with a gapping element to allow separation. The challenge to the geotechnical engineer is the development of appropriate soil springs that would realistically represent the overall behavior of the caisson under the design earthquake.

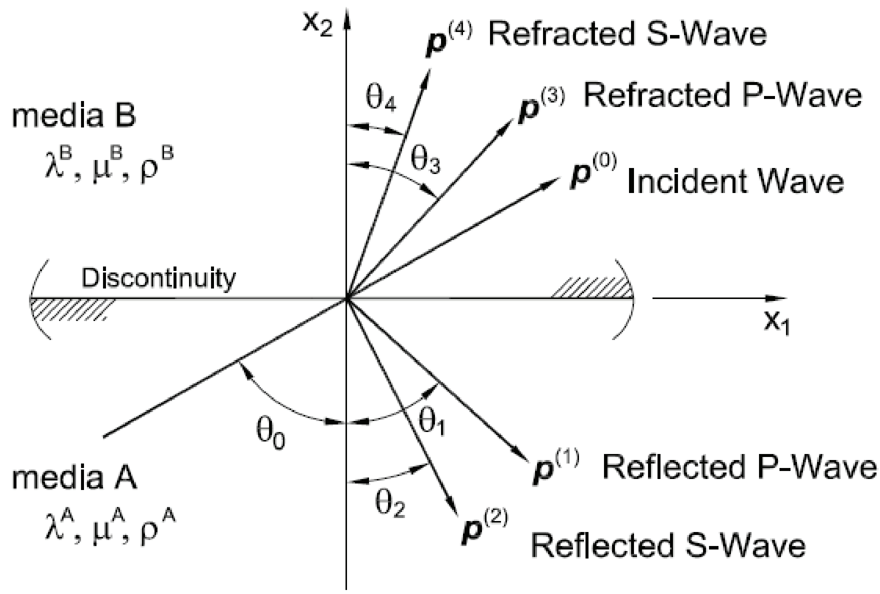
2.3 Wave Scattering

Discontinuity in the material properties has a significant influence on wave propagation through the medium. Any incident wave arriving at the discontinuous boundary separating two media would result in reflected and refracted waves, which may be both P- and S-waves depending on the incident angle, as illustrated in Figure 2.3. 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 elastic half-spaces, for general solutions in the case of earthquake loading, finite element methods are used.

To examine the potential influence of wave scattering resulting from generation of P- and S-waves in the near-field soil, the pile group problem may be simplified by treating the piles and the soil inside the pile group as a uniform continuum having an increased stiffness and mass density as compared to the surrounding free-field soil. The composite stiffness and mass density of this continuum can be estimated based on the percentage of the volume occupied by each of the two components within the footprint of the pile group system. This approach is used as part of the current study in attempting to examine wave scattering issues on the pile group.

The wave scattering mechanism tends to alter ground motions in a high frequency range while the affected frequency range is governed by the physical dimensions of pile group and the wave propagation characteristics of the medium (i.e., wavelength at each frequency). For very large pile groups such as those employed in long span bridges, the plan dimensions of the foundation could be as large as 150 ft x 150 ft, and this type of foundation in very soft materials may alter near-field ground motion in longer periods as compared to regular foundations typically found in highway crossing bridges.

Similar to a large pile group, the presence of large caissons in soil would also result a stiffness contrast between the foundation and the surrounding ground. In fact, the stiffness contrast in the caisson problem is much higher compared to the pile group foundation. Near-field soil motions at the vicinity of the caisson would be highly influenced by wave scattering due to inclusion of very stiff concrete caisson whose dimensions could be as large as 120 feet. Depending on the type of superstructure, the wave scattering may or may not be important. For example, long suspension bridges are typically characterized as very long period structures with a fundamental period of over 3 seconds. For this type of bridge structure, wave scattering is not expected to significant



Incident Wave	$\boldsymbol{p}^{(0)} = \sin\theta_0\, \boldsymbol{i}_1 + \cos\theta_0\, \boldsymbol{i}_2$
Reflected P-Wave	$\boldsymbol{p}^{(1)} = \sin\theta_1\, \boldsymbol{i}_1 - \cos\theta_1\, \boldsymbol{i}_2$
Reflected S-Wave	$\boldsymbol{p}^{(2)} = \sin\theta_2\, \boldsymbol{i}_1 - \cos\theta_2\, \boldsymbol{i}_2$
Refracted P-Wave	$\boldsymbol{p}^{(3)} = \sin\theta_3\, \boldsymbol{i}_1 + \cos\theta_3\, \boldsymbol{i}_2$
Refracted S-Wave	$\boldsymbol{p}^{(4)} = \sin\theta_4\, \boldsymbol{i}_1 + \cos\theta_4\, \boldsymbol{i}_2$

Figure 2.3 Reflection and Refraction of P-wave and S-wave

influence on the seismic response. Short span bridges with shallow column heights might be affected by the wave scattering as it typically alters the response in the short periods.

In addition to the two-dimensional wave propagation phenomena brought about by inclusion of the foundation, natural geologic features could also lead to potential 2D effects arising from those subsurface conditions with steeply inclined bedrock, a sloping ground surface, or inclined layered soil, etc. This is generally known as a basin effect. Such conditions exist throughout many urban regions where alluvium deposits overlie more competent rock with a much higher seismic velocity. Some of the classical basin profiles are illustrated in Figure 2.4. The response of the flat layer case is usually modeled using one-dimensional wave propagation for vertically propagating waves. For waves entering the layer at an angle, the response may resonate in the layer and 2D site response models are necessary to study. It is particularly true when basin edge effects are present near a steeply sloping basin edge. The wave can be trapped in the layer when if the incident angle (i) meets a certain requirement of the critical incident angle (i_c) as illustrated on the figure.

Although basin effects are discussed in the context of wave scattering in two-dimensional site response analyses, this subject is beyond the scope of the current research. The wave scattering mechanism associated with the changes in subsurface conditions due to the presence of a foundation is the subject of the current study. Understanding the impacts of 2-D wave scattering on a pile group and a gravity caisson is critical, and requires a systematic evaluation of the numerical procedures.

2.4 General Methodology

Past experience on the wave scattering analyses on foundation design has been based on frequency domain computer codes, notably FLUSH, SASSI and SHAKE. The program SHAKE has been used for computing free-field ground motions of horizontally layered soils, while the program FLUSH or SASSI has capabilities of modeling up to 3-dimensional wave propagation problems with structure elements. These codes are dedicated programs for primarily dealing with geotechnical problems, and do not serve as a common platform to be used for global bridge response analyses. Because of the frequency domain approach, their applications have been limited to linear elastic analyses, prohibiting their use as a tool for non-linear time history analyses needed for major bridge design projects.

Many of the long span bridges have been studied and designed using time history based finite element computer programs, such as ABAQUS, ADINA, SAP2000, GTSTRUDL, NASTRAN, and ANSYS. Among these commercially available finite element codes, many bridge engineers who have experience working on major bridge projects favor the program ADINA (ADINA R&D Inc., 2003) over other codes. Many of California toll bridges were designed using ADINA; for example, time history analyses of New Carquinez Suspension Bridge, East Span San Francisco – Oakland Bay (Suspension) Bridge, New Benicia-Martinez Bridge, and Gerald-Desmond Cable Stayed Bridge were conducted with the program ADINA.

In this research program, ADINA is used to study wave propagation and wave scattering of horizontally layered soils, pile groups, and gravity caissons. The aim is to establish a time domain procedure to solve the wave propagation problems using the same computer code that is employed by structural engineers to build the complete system bridge model. In the process of developing the time domain procedure, we consider the results obtained

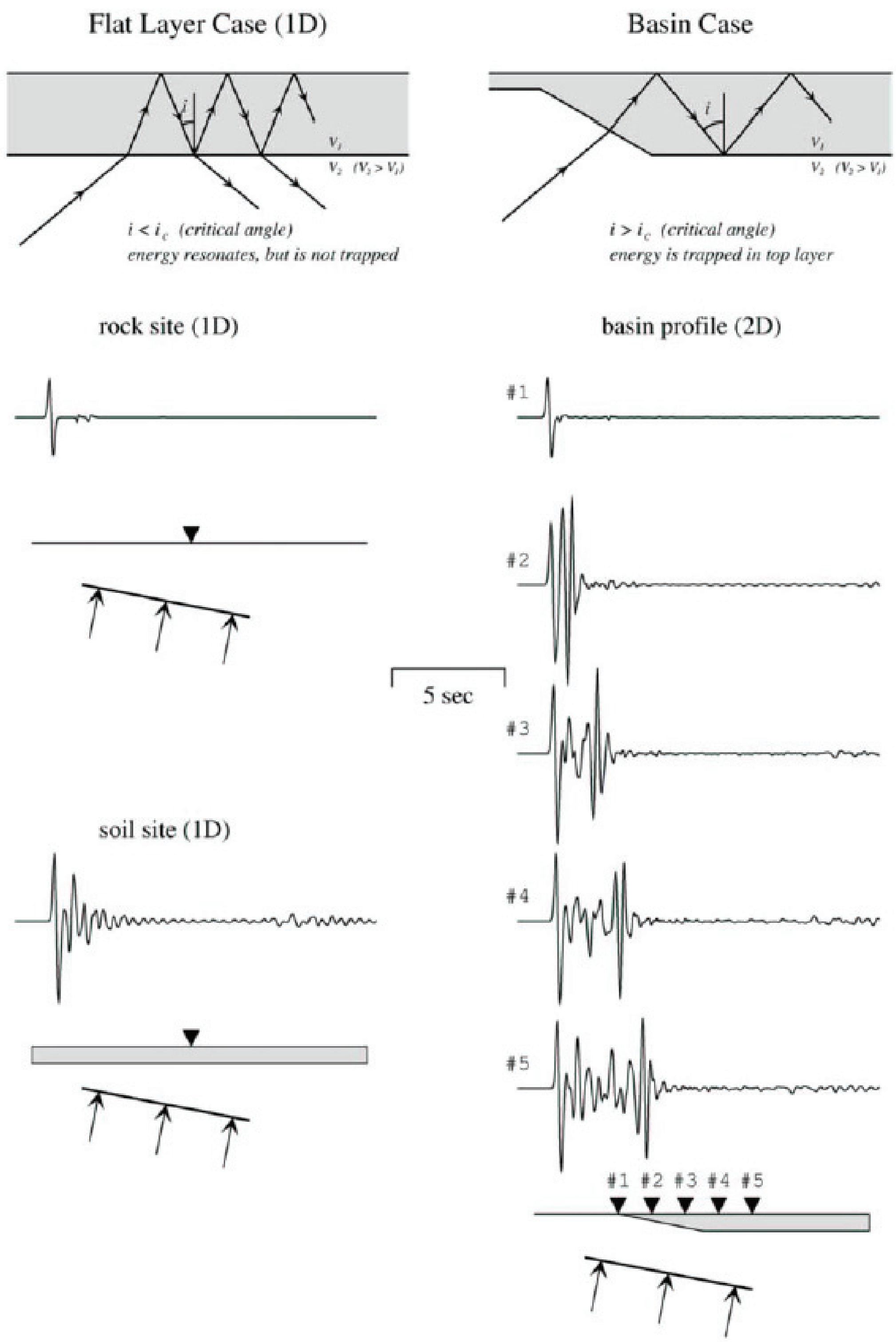


Figure 2.4 Wave Scattering in Typical Soil Profiles

from SHAKE and SASSI to be accurate, and they serve as benchmark solutions for comparison with the time domain solutions.

There are several areas distinguishing the frequency domain method from the time domain method. It is important to understand the differences in order to compromise the comparison between the two solutions. We consider the following to be most significant:

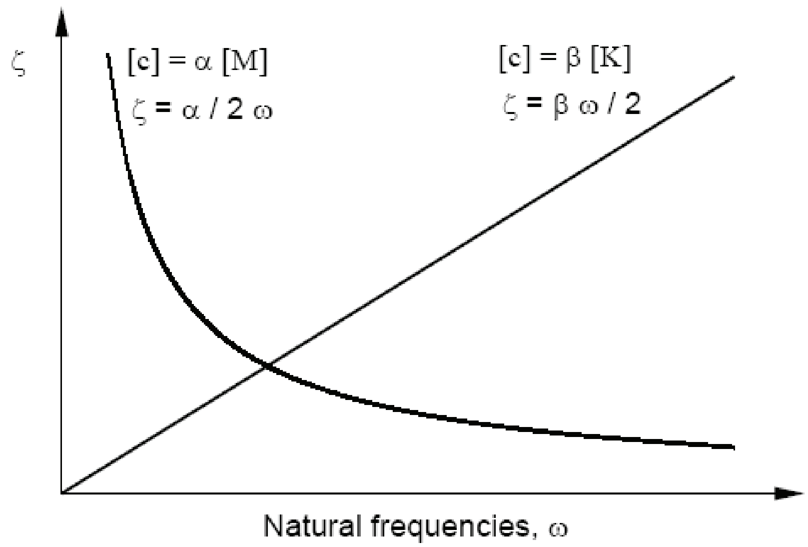
- Both SHAKE and SASSI use complex numbers to solve an impedance function for each frequency. The damping coefficient used in computation of the impedance function is independent of frequency resulting in uniform damping behavior regardless of the frequency. Most time domain based finite element programs, including ADINA, utilize a Raleigh damping concept consisting of mass and stiffness related damping parameters, α and β, respectively. The overall damping matrix is then expressed as,

  $$[C]= \alpha\, [M] + \beta\, [K]$$

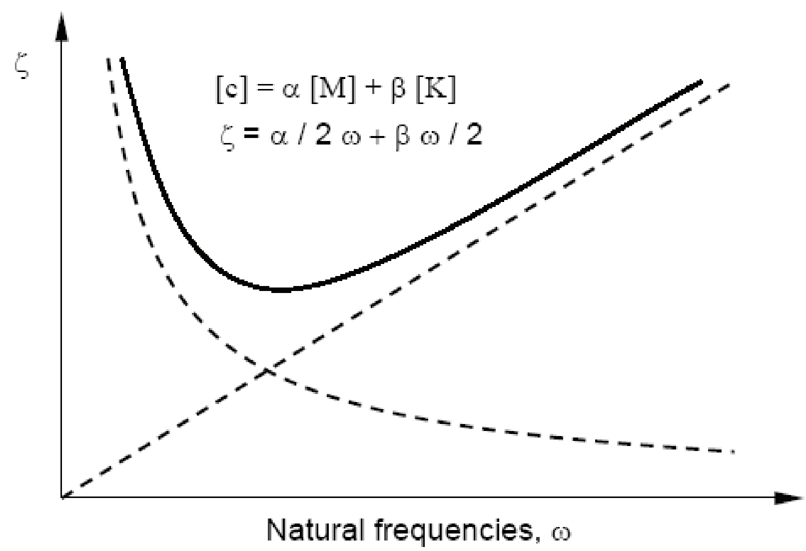
  where [C], [M] and [K] are damping, mass and stiffness matrices, respectively. This relationship leads to frequency dependent damping; the general characteristics of the damping versus frequency are depicted in Figure 2.5.

- In regard to treating boundary conditions, SHAKE and SASSI take the advantage of using an analytical solution integrated in the program. For example, SHAKE adopts 'within motion' and 'outcrop motion' to distinguish a rigid base from a half space, using transfer functions computed among different layers. SASSI, on the other hand, handles the half space by adopting a transmitting boundary at the base of model using a procedure described in the paper by Lysmer and Kulemeyer, 1969, and implementing an analytical solution at the side boundaries. The time domain finite element programs used by most structural engineers do not have these capabilities to address wave reflection/refraction at the boundaries. Consideration must be given to address the boundary conditions when using the time domain based finite element procedures. Without proper treatment of the boundary conditions could result in erroneous solutions arising from the reflection of seismic waves at the boundaries.

In the forgoing sections, sensitivity analyses are presented for the level ground, pile group, and caisson separately. These sensitivity studies were carried out to address the issues affecting the time domain solutions so that the procedure developed here can be used by others.



(a) mass-proportional damping and stiffness-proportional damping



(b) Rayleigh damping

Figure 2.5 Element of Rayleigh Damping

SECTION 3
HORIZONTALLY LAYERED SOIL

3.1 One-Dimensional Site Response

When a seismologist develops a reference rock motion, it usually represents an outcrop motion. The rock outcrop motion is employed in a one-dimensional site response program, e.g., SHAKE, to compute free-field motions for subsequent studies required for foundation designs. Consistent with the definition of the outcrop motion, the input acceleration to SHAKE must be treated as 'outcrop' motion, not as 'within' motion. By treating as 'outcrop', the layer below the boundary where the outcrop motion is specified becomes infinite space eliminating the potential for wave reflection/refraction at the boundary.

When the input motion to SHAKE is used as 'within' motion, the boundary is treated as a rigid base resulting in 'prescribed motion' at the base of the soil column. Consequently, if the input rock motion intended for 'outcrop' is used as 'within' motion in SHAKE, overly conservative and sometimes erroneous solutions are obtained. This is a result of wave trapping within the soil deposit. Unfortunately, some practicing geotechnical engineers do not recognize these mechanics and continue to make these mistakes not only in SHAKE but also in other site response analysis programs.

There are cases where seismologists define a reference motion at the ground surface, especially where rock like material cannot be located within a reasonable depth (e.g., within 200 feet). In this situation, they rely on soil attenuation relationships based on recorded surface motions to establish the reference motions at the ground surface. Depth-varying free-field motions can be computed from the reference surface motion by conducting a deconvolution analysis with the program SHAKE. The deconvoluted motion at any depth may be requested as 'outcrop' or 'within' motion.

For the purpose of this research, we consider hypothetical soil strata, as illustrated in Figure 3.1 showing the material properties assigned to each soil stratum. The reference motions chosen for this study are at the ground surface; Figures 3.2 and 3.3 present the characteristics of the reference motions in two directions, horizontal and vertical, defined at the ground surface. These motions have been spectrum-matched to certain ARS design curves. Given the reference motions defined at the surface, deconvolution analyses were conducted to compute the free-field motions at every soil layer as 'within' motion, while the bottommost layer is requested as both 'within' and 'outcrop' motions. The free-field motions as computed from SHAKE are illustrated in Figures 3.4 and 3.5 for the horizontal and vertical directions, respectively.

Here attempts are made to reproduce the SHAKE solutions using the ADINA program. The base input acceleration to ADINA model is the 'within' motion at the bottommost layer computed from the deconvolution analysis using SHAKE. The prime reason for

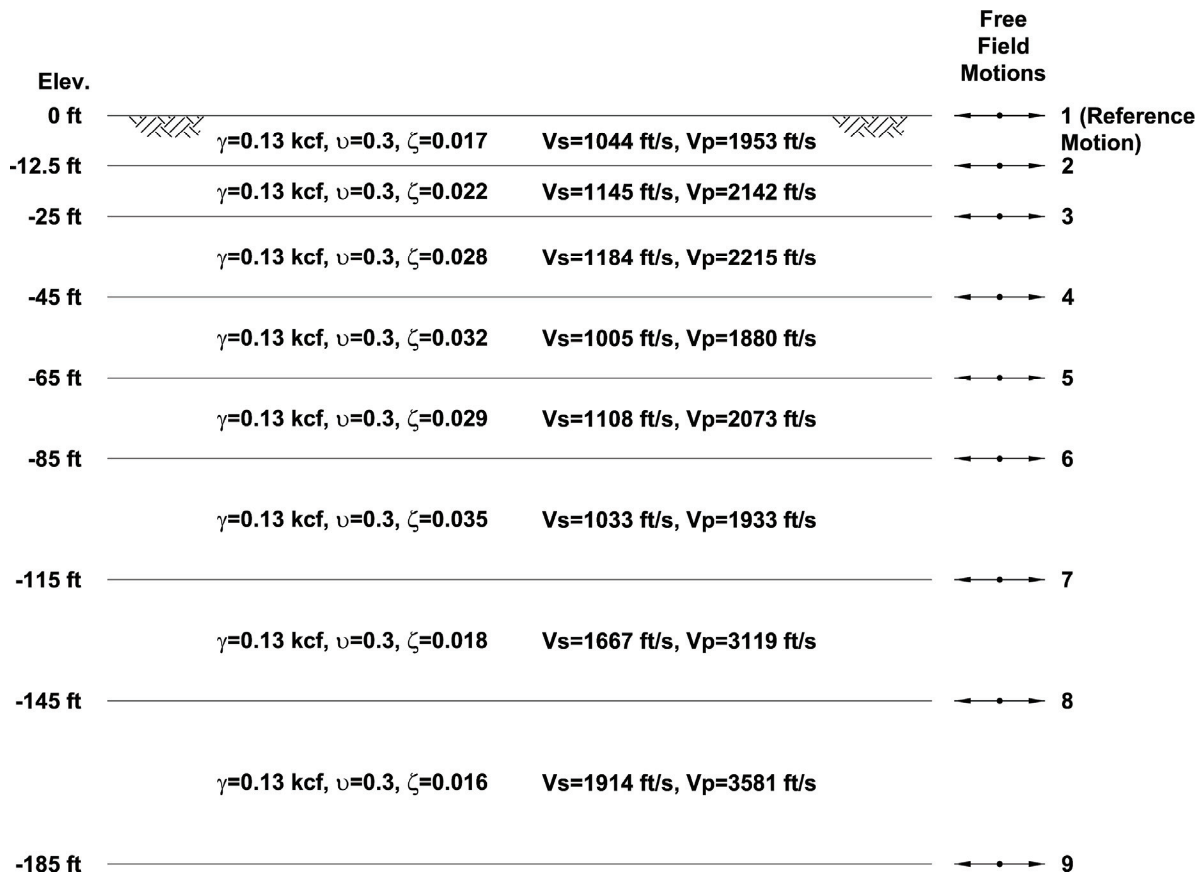


Figure 3.1 The Hypothetical Soil Strata

using the SHAKE's 'within' motion to excite the ADINA's finite element model is that rigid base excitation is implemented without employing transmitting boundary conditions. If the solutions are correct, the surface motion computed from ADINA should duplicate the reference surface motion.

Several parameters affect the ADINA solutions. To understand how they influence the numerical results, comparison between the ADINA solutions and the SHAKE solutions is made by performing a parametric study that addresses the following issues:

- Effects of vertical side boundaries
- Effects of time integration schemes
- Implementation of damping

3.2 Effects of Vertical Side Boundaries

A two-dimensional finite element method is used to simulate the 1-D wave propagation in order to investigate effectiveness of various side boundary conditions. The finite element domain representing the same layered soils used in SHAKE is shown in Figure 3.6. Seismic response analyses of the soil strata were performed using the following boundary conditions:

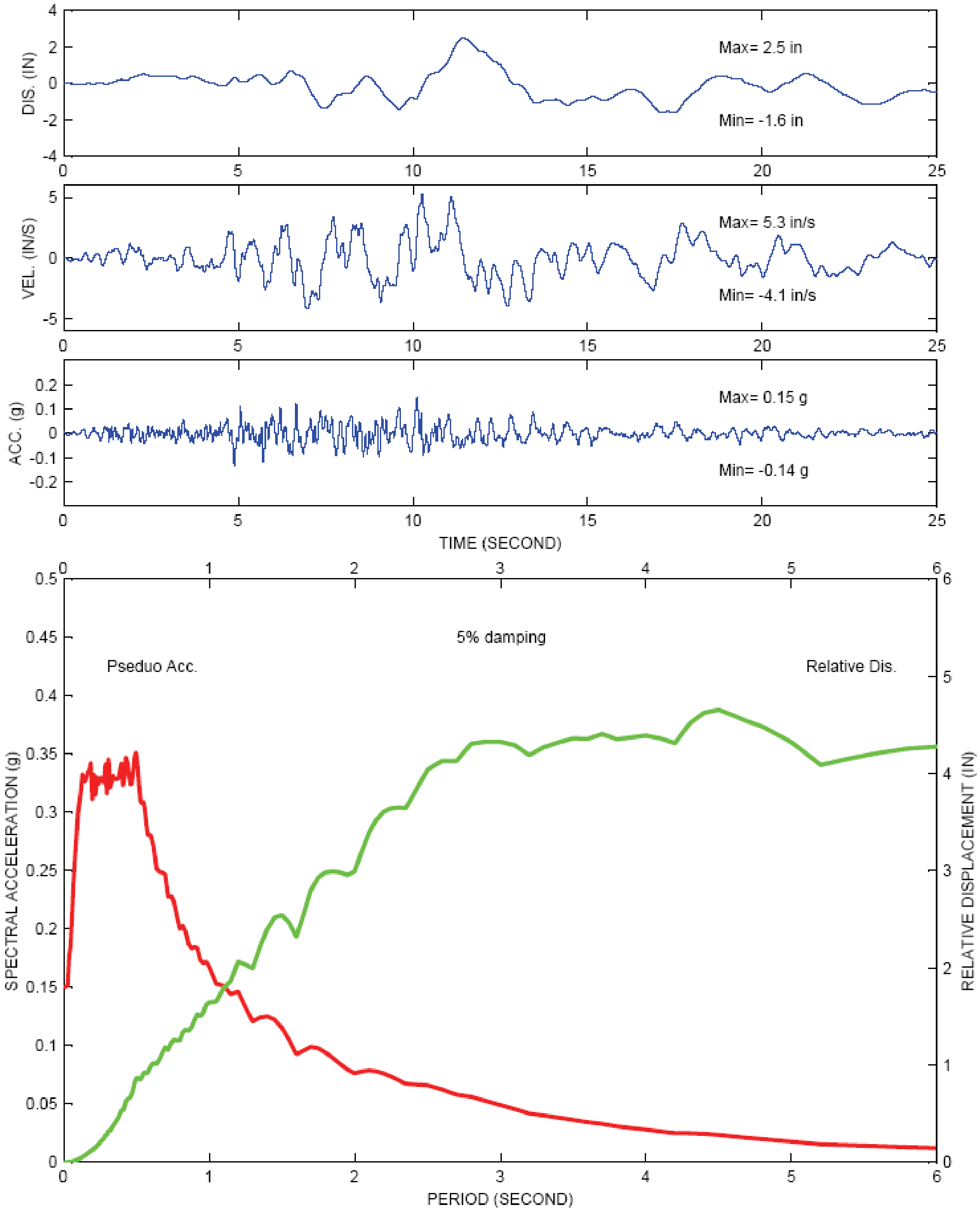


Figure 3.2 Reference Horizontal Ground Motion at Surface

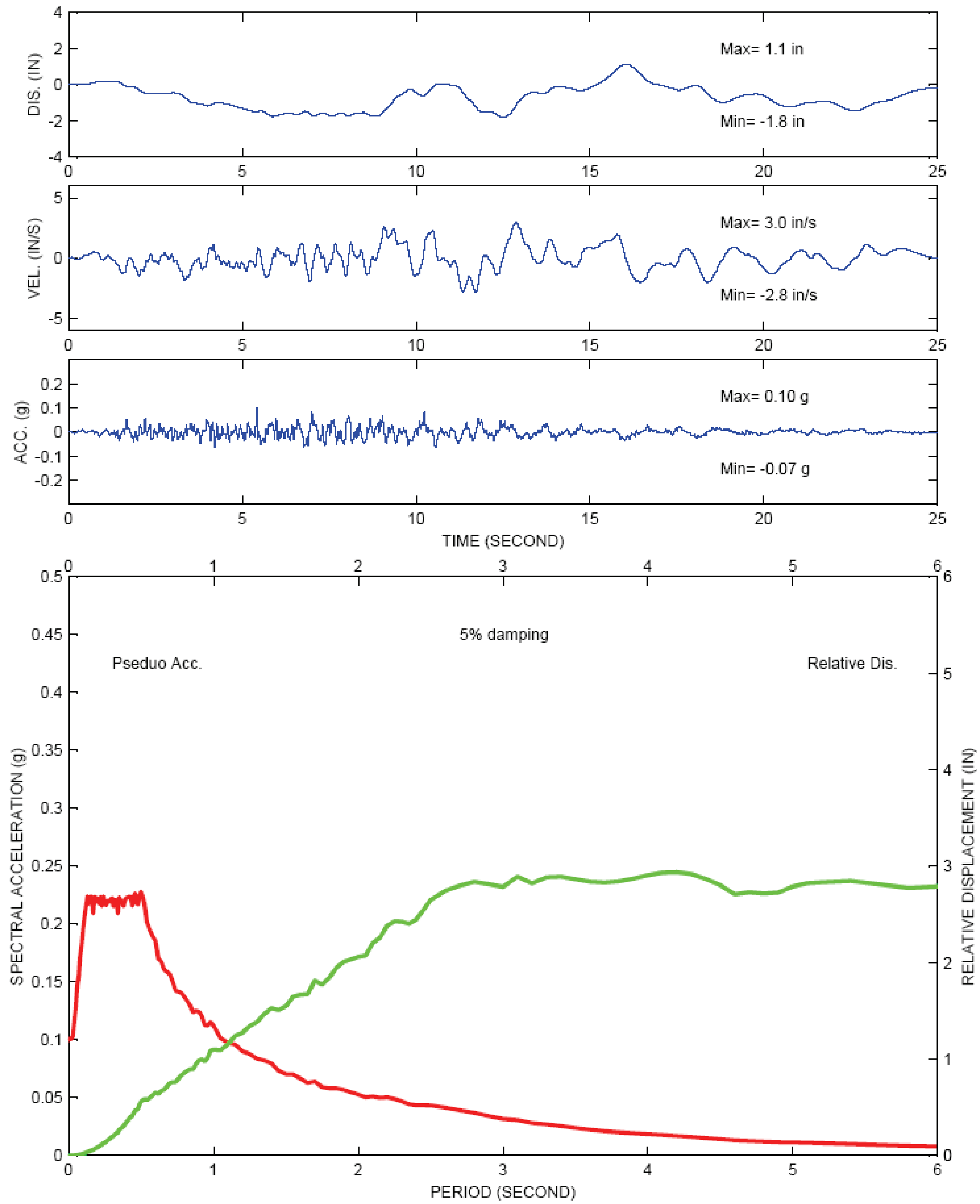


Figure 3.3 Reference Vertical Ground Motion at Surface

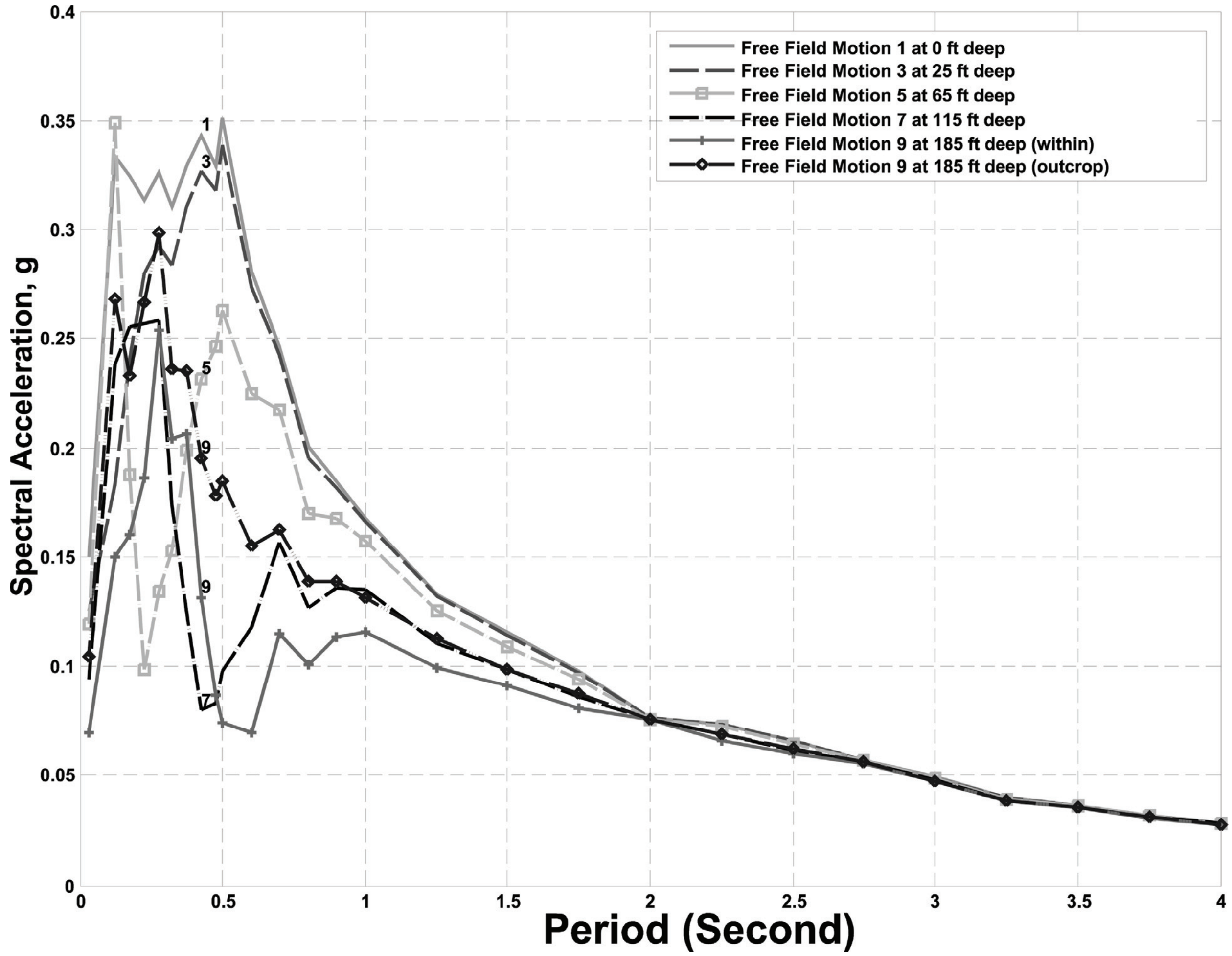


Figure 3.4 Depth Varying Free Field Horizontal Motions

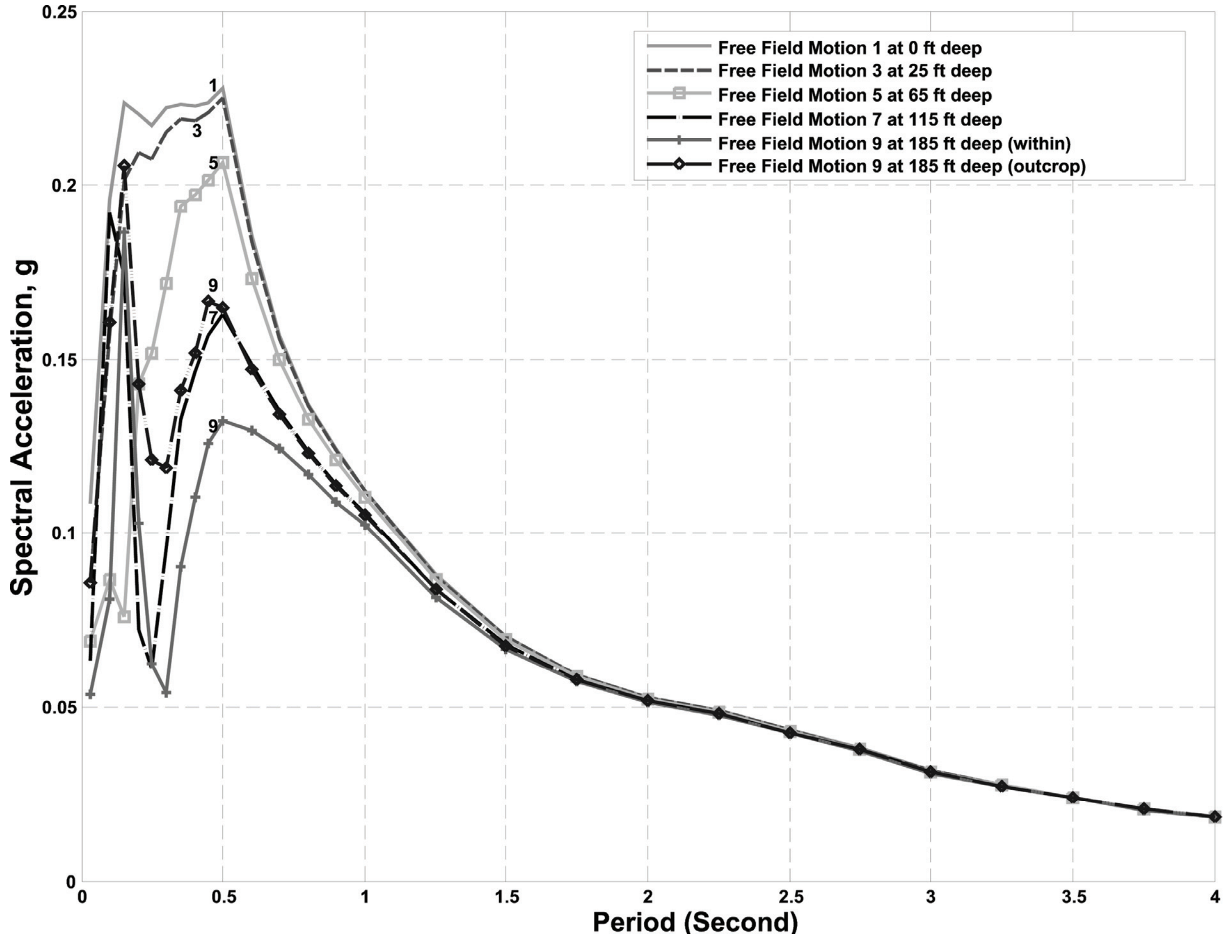


Figure 3.5 Depth Varying Free Field Vertical Motions

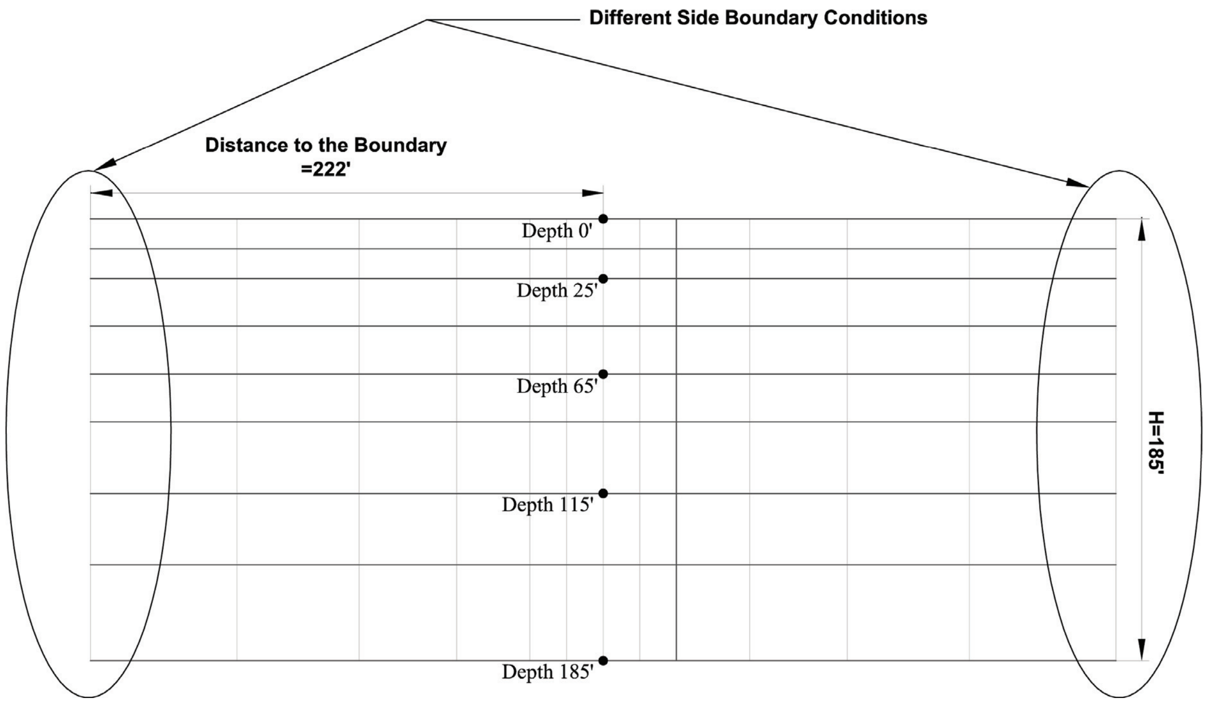


Figure 3.6 Finite Element Model

- Free side boundary
- Fixed side boundary
- Edge column boundary
- Slaving of left and right boundaries
- Transmitting side boundary

Schematics of these side boundary conditions are shown in Figure 3.7.

Figure 3.7(a) is a finite element model with free side boundaries indicating no constraint at the side boundary nodes. These side boundary nodes are free to move in any direction.

Figure 3.7(b) illustrates a finite element model with fixed side boundaries implying that no vertical and horizontal movement relative to the base is allowed at the two side boundaries.

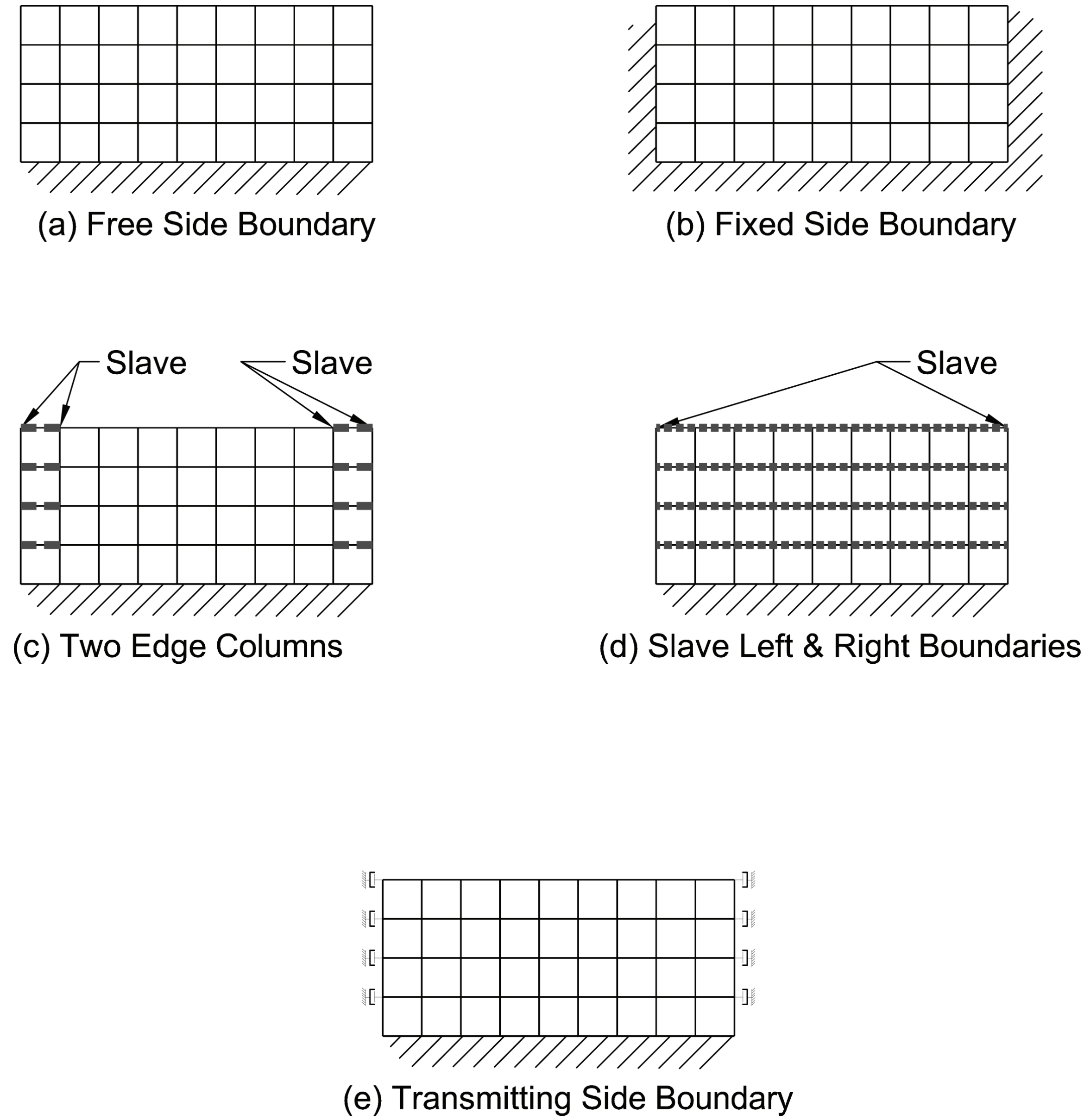


Figure 3.7 Different Side Boundary Conditions

Figure 3.7(c) represents a model with two edge columns at the side boundaries where two corresponding nodes of the soil column at same elevations are slaved together. The intent is to create shear beam columns near the boundaries.

Figure 3.7(d) is an illustration of how one-dimensional wave propagation is modeled by means of slaving the leftmost nodes to the rightmost nodes at the side boundaries. Slaving is done in the both directions, horizontal and vertical, such that the slaved nodes move together.

Figure 3.7(e) depicts treatment of the side boundaries using dashpots similar to those used at the base of some numerical models to represent a halfspace suggested by Lysmer and Kulemeyer (1969).

In all the cases discussed above, rigid base excitation is the primary form of seismic loading to the ADINA finite element model. This is accomplished by assigning the 'within' motion computed from the SHAKE's deconvolution analysis at the base of the soil column. Only horizontal site response behavior is assessed in this exercise (i.e., vertical motion is not included). The horizontal ground response is computed at the surface (depth 0 ft) from each of the finite element model, and the result is compared with the reference ground motion in Figure 3.8 in terms of acceleration response spectra.

It can be seen that the ground response of the fixed-side-boundary model is largely magnified by reflected waves as overly restrained conditions were imposed in the analysis. The free side boundaries also yield superfluous response that is not satisfactory as compared to the reference surface motion. Although a free or fixed boundary condition is typically available in general finite element codes, neither could provide accurate ground response simulating one-dimensional wave propagation.

The finite element with dashpot side boundary also yields unsatisfactory ground response. However Lysmer and Kulemeyer (1969) report effectiveness of the dashpot concept when used at the base representing the halfspace to absorb seismic energy, and the efficiency of absorbing energy reduces largely with the increase of the incident angle. The efficiency is about 95% for a zero incident angle but and reduces to around 10% for a 30-degree incident angle. However, no discussion is found if the dashpot concept can be used at the side boundary. Based on the result obtained from this study, the dashpot concept applied at the side boundaries is not very effective in modeling the one-dimensional shear beam problem. Perhaps the side boundary where the dashpots are attached is too close to the center of the finite element. Of course, if the side boundaries are moved 'far' away from the centerline of the model, the solutions would converge to the one-dimensional situation regardless of the type of boundary.

The solution of the model with a two-edge column concept shows a reasonable degree of accuracy as compared to the benchmark surface motion. In this model, the width of edge column is 61 feet. It is anticipated that width of the edge columns might influence the solutions, and its effects will be discussed later.

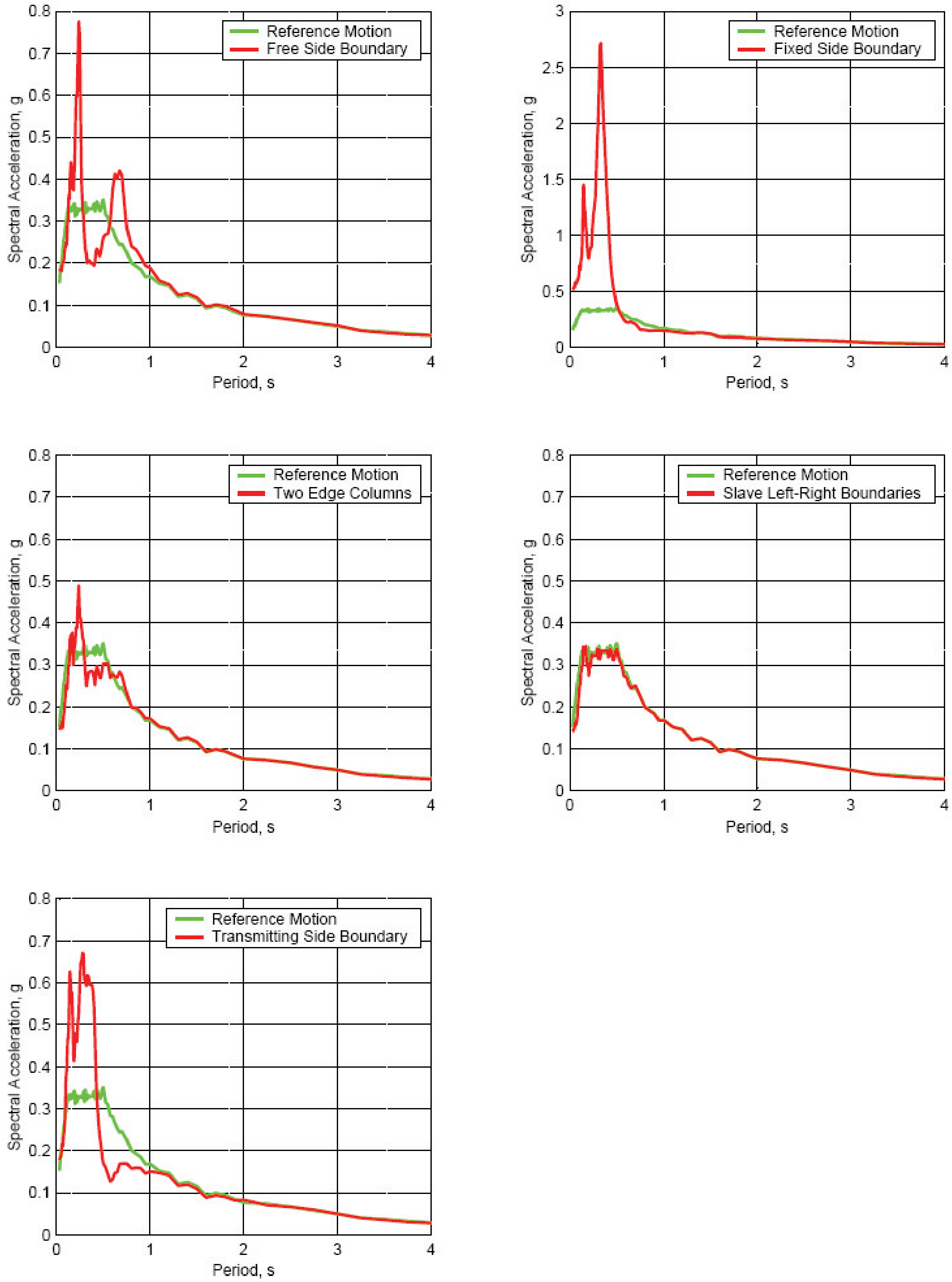


Figure 3.8 Effects of Side Boundary Conditions

The response for the finite element model, which slaves the leftmost nodes to the rightmost nodes converges very close to the benchmark surface motion, and the model appears to be the most suitable for simulating a simple shear beam theory. While this boundary condition can be used for simple models where the left side and right side of the finite element mesh have similar geometric configurations and properties, it would not be suitable if the two sides have different ground elevations, such as for retaining walls and sloping ground conditions.

For these parametric studies, it appears that the edge column concept and the slaving left-and-right boundary concept show promising results that can be used for general application depending on situations. To further examine the results, the depth varying motions from these two models are compared with SHAKE's depth-varying motions, as shown in Figures 3.9 and 3.10.

3.2.1 Effects of Edge Column Widths

As discussed earlier, the technique of slaving the leftmost nodes to the rightmost nodes is not always suitable for all geotechnical problems; such situations exist in the case of a retaining wall, wharf structure and sloping ground. In these cases, the edge column concept may be used. To use two edge columns for general application, it is necessary to understand effects of edge column widths. We considered different widths of edge columns while keeping the distance to the boundary from the centerline of the model unchanged.

In this sensitivity analysis, we elect to use the same hypothetical soil strata as shown in Figure 3.1 which has a total thickness of 185 ft. The side boundaries are 246 ft away from the centerline of the model. Each of the side boundary is attached to an edge column with varying widths, as shown in Figure 3.11. The following edge column widths were considered:

- Edge Column Width = 1/6 H
- Edge Column Width = 1/3 H
- Edge Column Width = 2/3 H
- Edge Column Width = H

where H is the thickness of the ground. The solutions with different column widths are provided in Figure 3.12. When compared to the benchmark surface motion, closer agreements are obtained with the increase of edge column widths. One can imagine that a narrower edge column tends to behave more like a free side boundary, and that a wider edge column approaches to a shear beam model when only horizontal excitation is considered.

In theory, the solutions of the model with a sufficiently wide edge column should approach to the response of the free field motions derived from a shear beam model under horizontal excitation. However, it becomes a tradeoff between the finite element

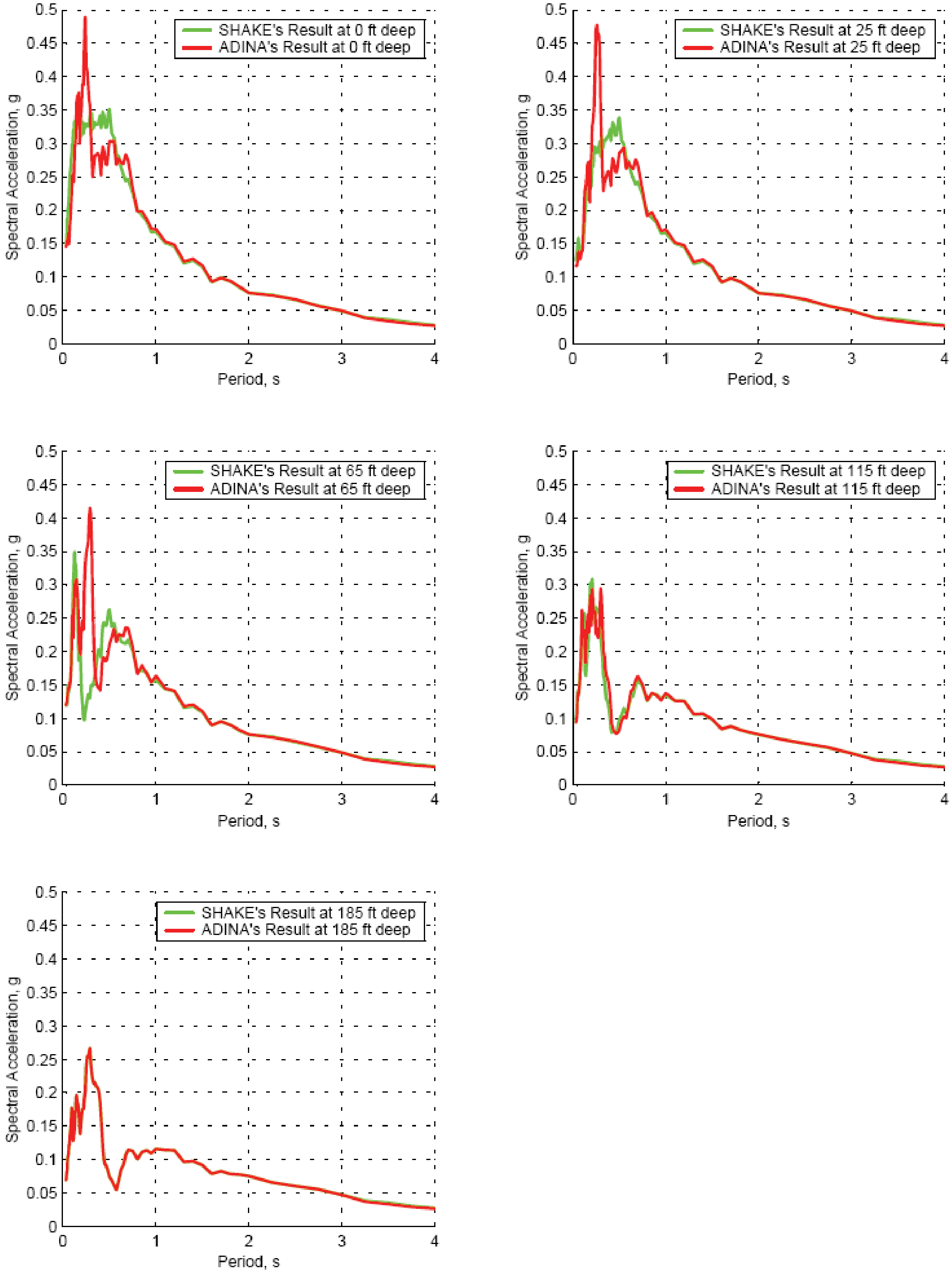


Figure 3.9 Comparison of SHAKE Results and ADINA Results of the Model with Two Edge Columns

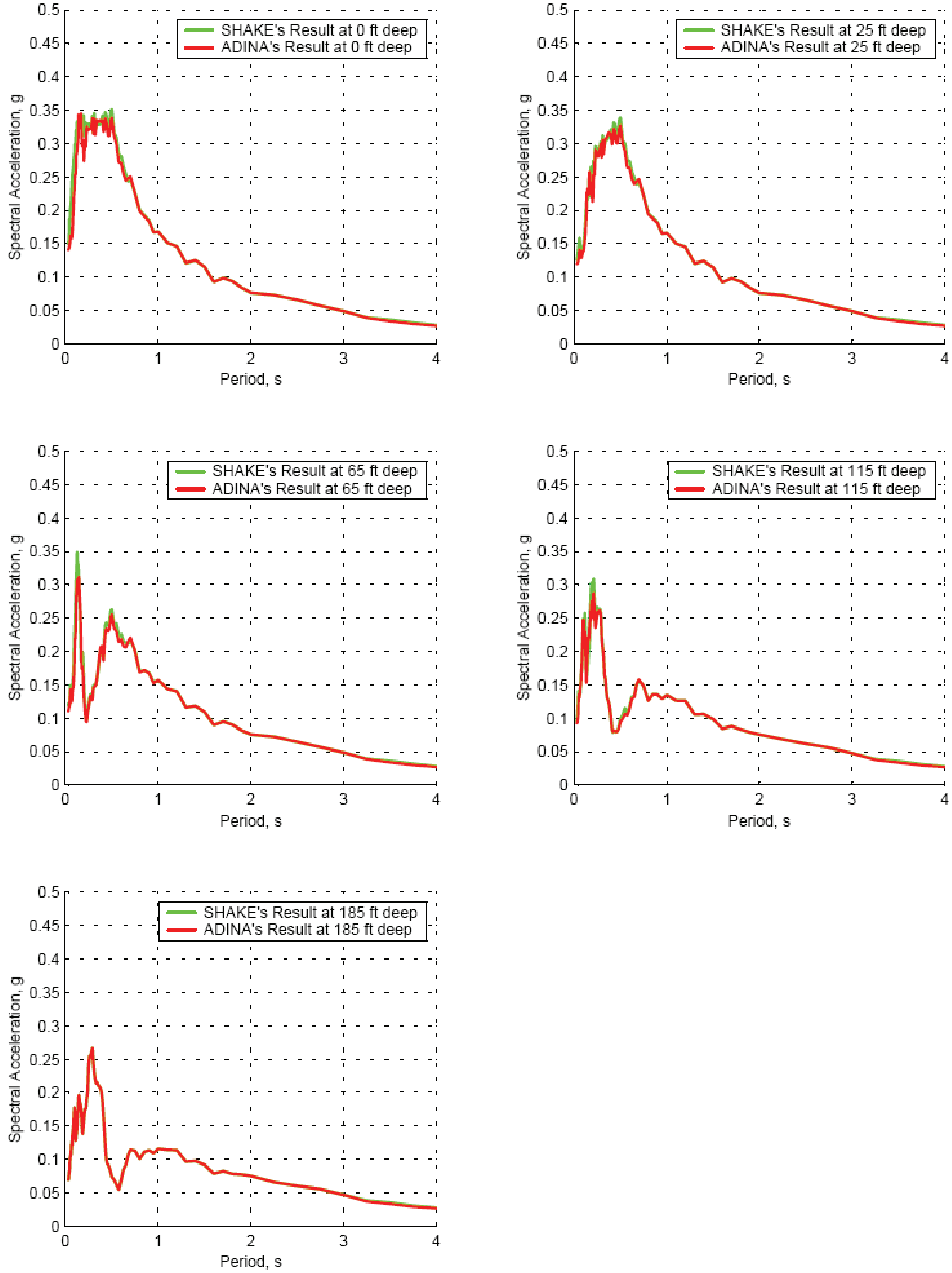


Figure 3.10 Comparison of SHAKE Results and ADINA Results of the Model with Slaving Left and Right Boundaries

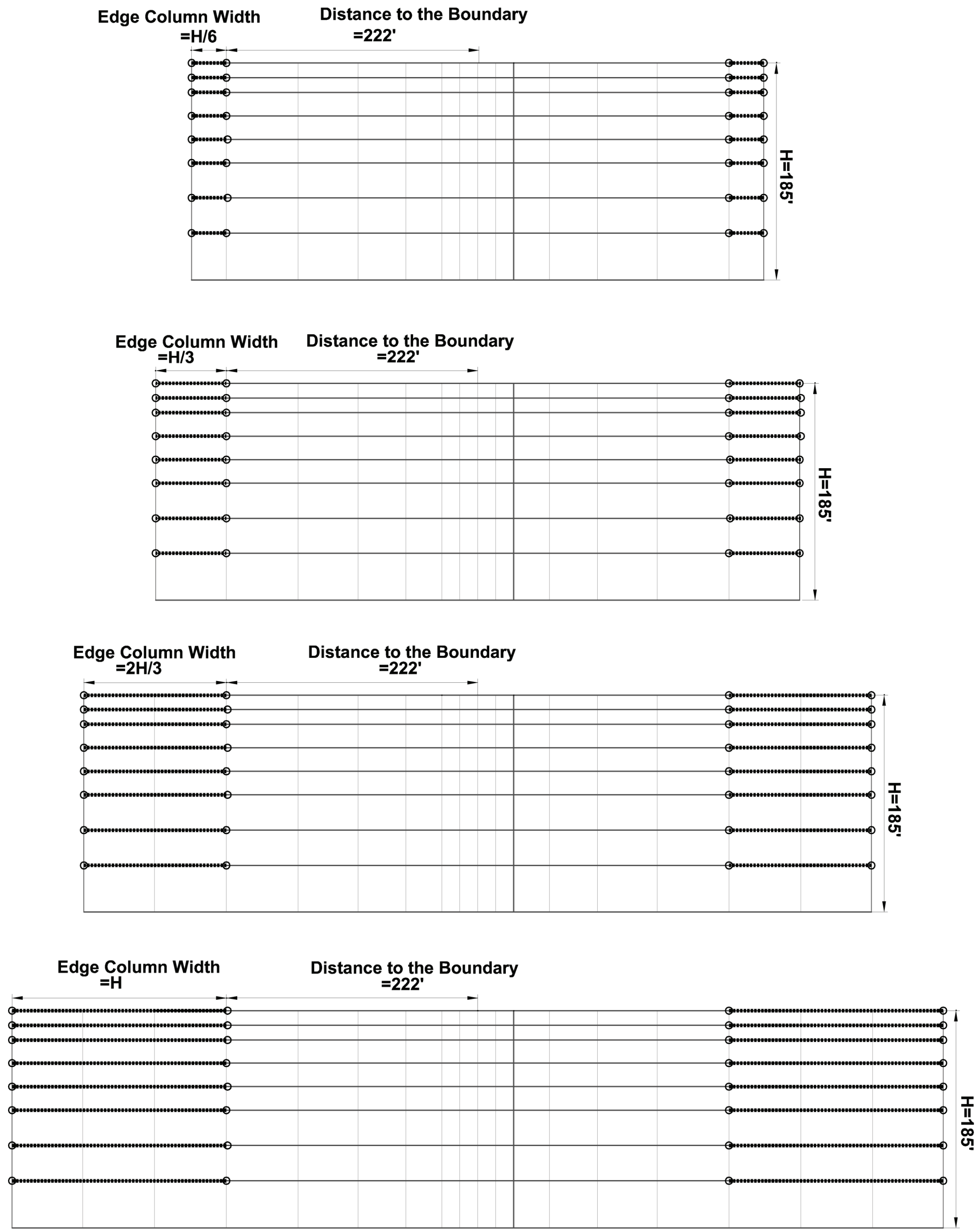


Figure 3.11 FEM Model with Different Edge Column Widths

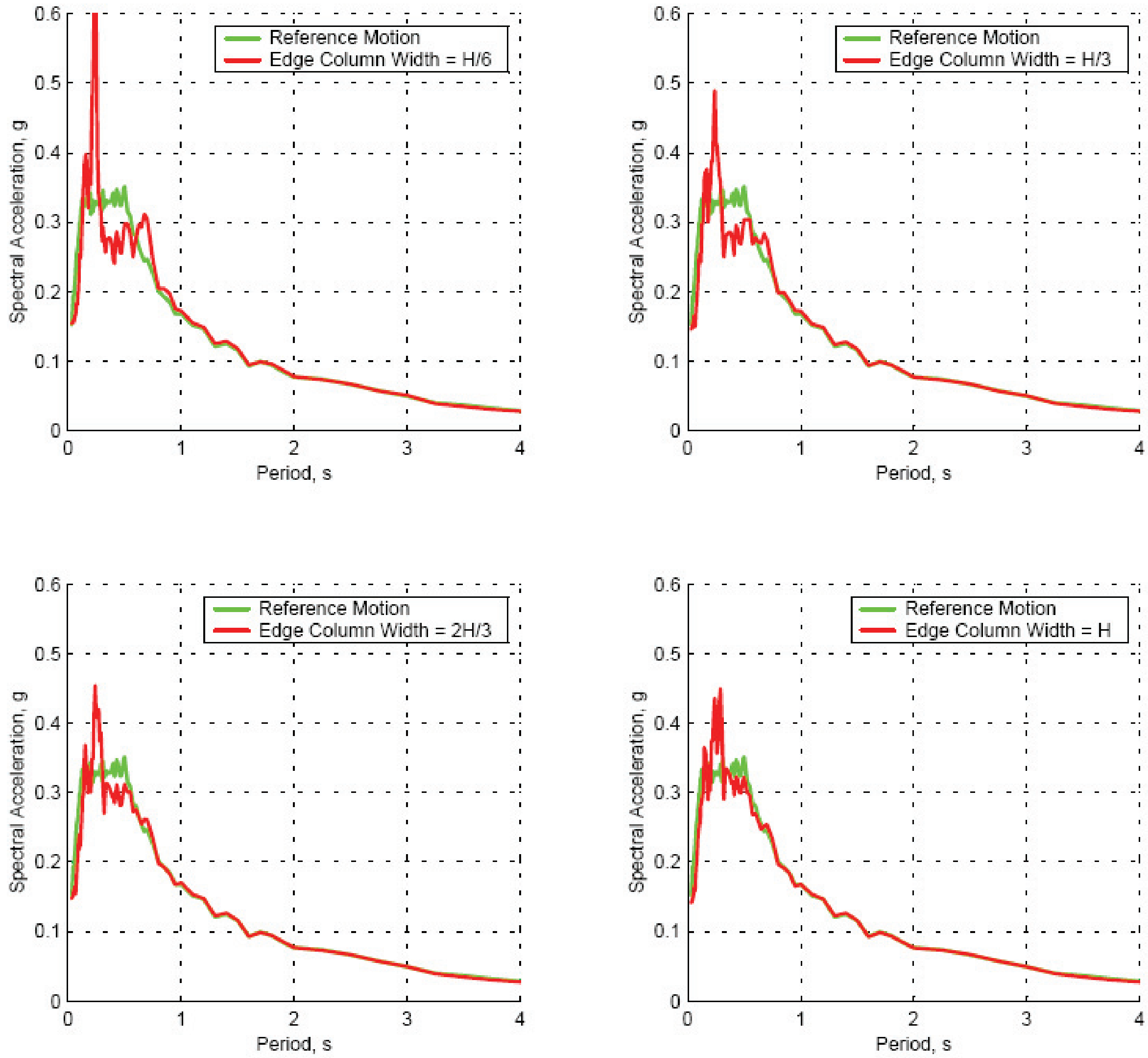


Figure 3.12 Effect of Edge Column Widths

size (or the computing time) and the accuracy. For practical purposes in most bridge engineering applications, a minimum edge column width should be 1/3 H or larger in order to obtain reasonable structural responses beyond a 0.5 sec period. It would be desirable for perform sensitivity analyses like the one presented here, using site specific soil conditions prior to performing actual designs.

3.2.2 Effects of Distances to the Boundary

In the preceding section, the effect of edge column width is evaluated while keeping the distance to the side boundary from the centerline of the model unchanged. In addition to the edge column width, the solutions would also depend on proximity to the side boundary. We have performed sensitivity analyses by varying the distance to the side boundary, but maintaining the edge column width to 1/3 H. The following side boundary distances were evaluated:

- Distance to Side Boundary =1.2 H
- Distance to Side Boundary = 1.5 H
- Distance to Side Boundary = 1.8 H

where H is the thickness of the ground. We consider these distances to be within a practical range for most design applications. Figure 3.13 shows comparison of surface motions from the three models with increasing distances to the side boundary. It appears that the solutions are not very sensitive to the distance to the boundary in this study. However it is not advisable to have the side boundaries too close to the centerline.

3.2.3 Effects of Time Integration Schemes

In the time domain schemes as employed by ADINA and other computer codes, the differential equations are solved using a numerical step-by-step integration procedure. Several integration schemes are available based on different interpolations between displacements, velocity and acceleration, as well as the time step of the equilibrium established (Bath, 1982). The following integration schemes were investigated:

- Wilson's θ Method
- Newmark Method (δ= 0.5 and α = 0.25)
- Newmark Method (δ= 0.65 and α = 0.331)

Figure 3.14 shows results of three different integration schemes for the model with the edge column concept. The responses vary with the different time integration schemes; however the difference is obvious only in a high frequency range. For practicality, any of the time integration method is considered acceptable.

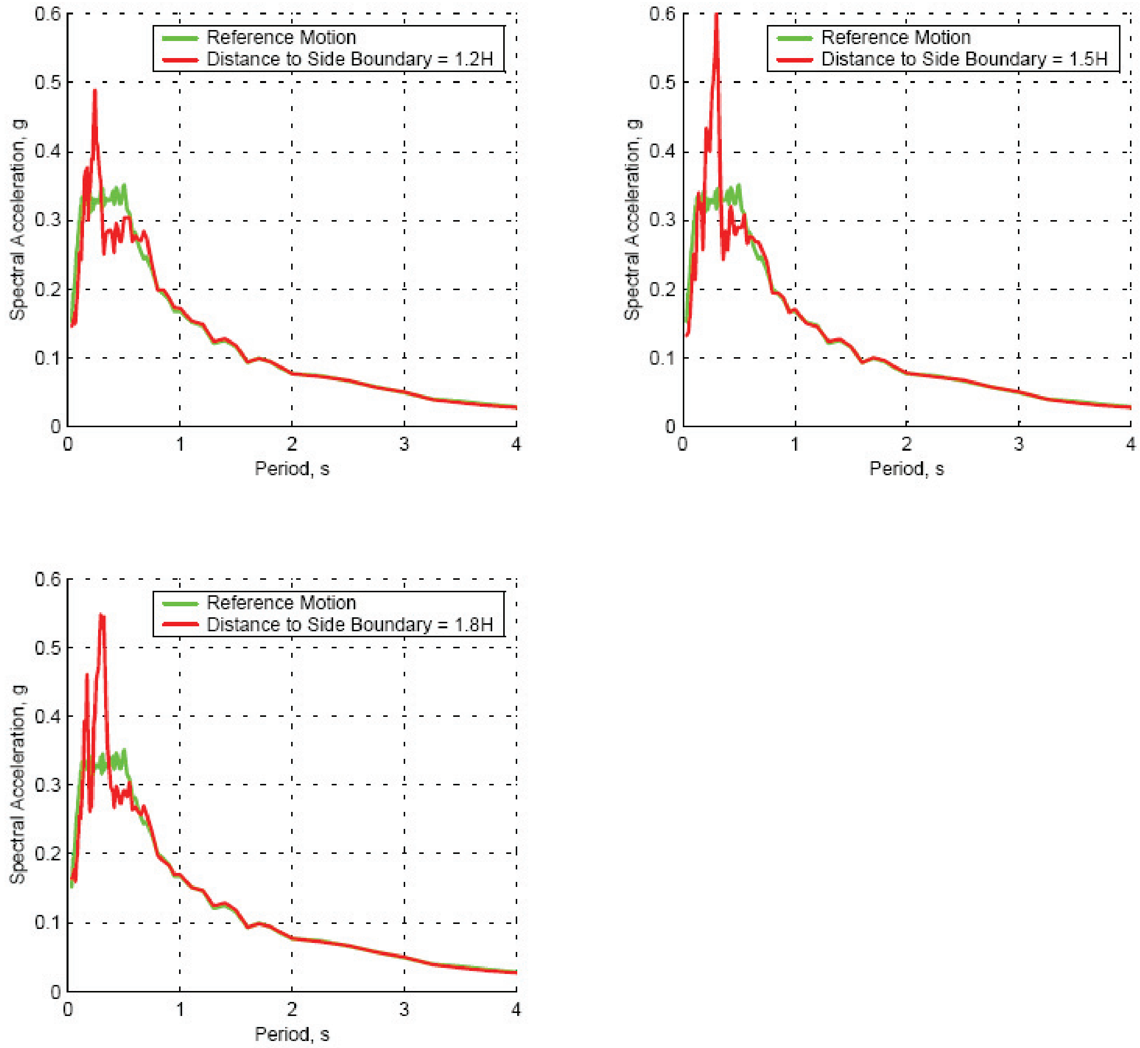


Figure 3.13 Effect of Different Distances to Boundary

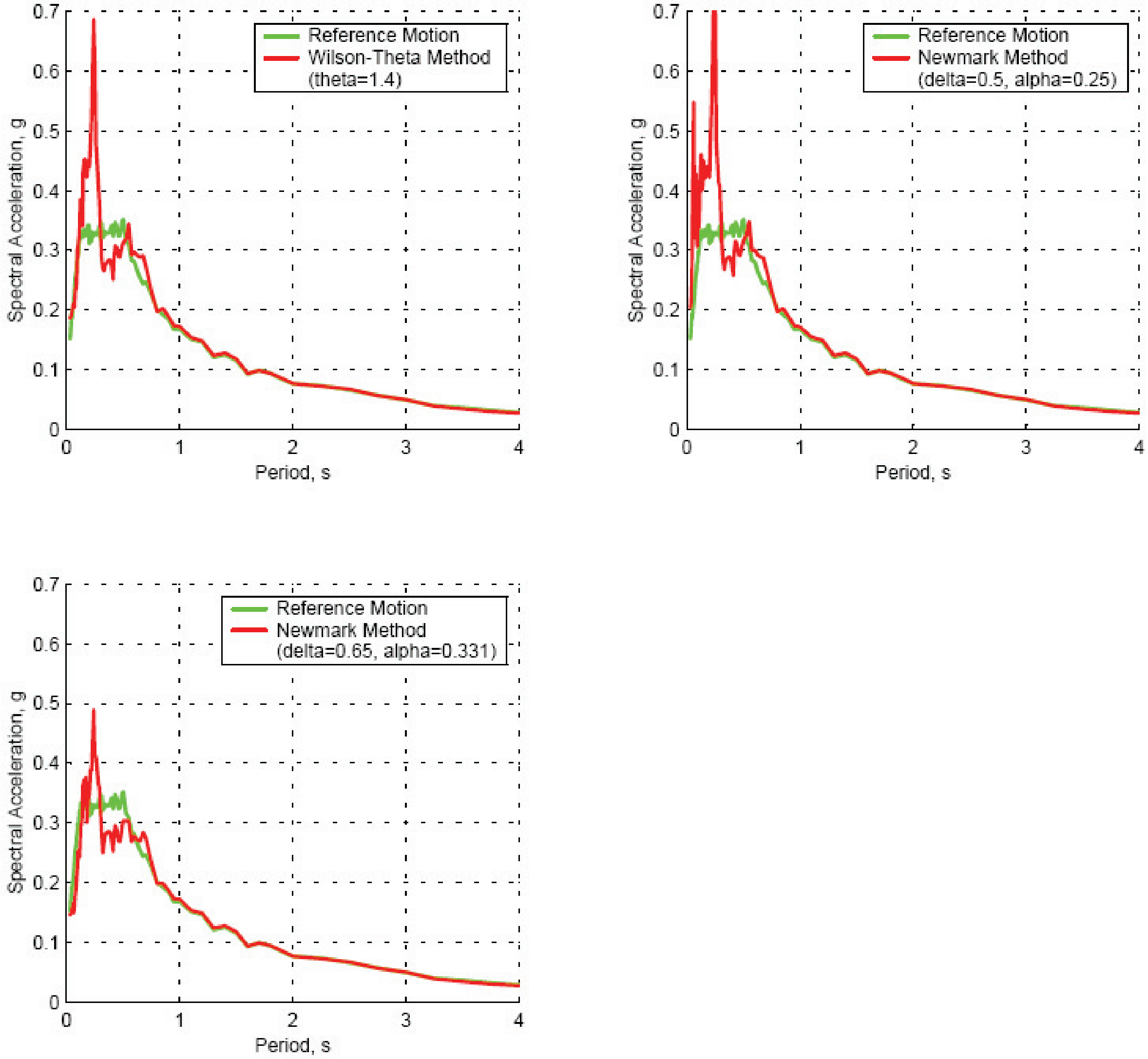


Figure 3.14 Effect of Different Integration Scheme

3.3 Treatment of Rayleigh Damping

Perhaps the greatest difference between SHAKE and ADINIA is treatment of damping in the respective numerical procedures. The frequency response function computed within SHAKE uses a damping ratio, which is frequency independent. This will result in constant energy dissipation across the spectrum. However, most finite element programs, including ADINA employ a Raleigh damping concept, which would lead to frequency dependent damping characteristics. When using Raleigh damping for dynamic analyses, it is possible to adopt one of the following procedures:

- Mass proportional damping only (α)
- Stiffness proportional damping only (β)
- Both mass and stiffness proportional damping (α and β)

Because the damping values vary with frequency in the Raleigh damping concept, it is necessary to anchor the desired damping ratio at a specific frequency, e.g., the vibration period of the system. For the mass proportional damping or stiffness proportional damping, only one anchoring point can be selected. When mass and stiffness proportional damping is used, two anchoring points must be chosen.

For a general case where both mass and stiffness related damping is used, the coefficients α and β can be determined from

$$\alpha + \beta\, \omega_i^2 = 2\, \omega_i\, \xi_i$$

where ξ_i is damping ratio at angular frequency ω_i. The relationship requires two frequencies to solve both coefficients (α, β). If mass proportional damping is used, then the coefficient α is determined as

$$\alpha = 2\, \omega_i\, \xi_i$$

When stiffness proportional damping is used, the coefficient becomes

$$\beta = 2\, \xi_i/\omega_i$$

We have conducted sensitivity analyses using the three different Raleigh damping methods; all calibrated to the same damping ratio used in SHAKE. Using mass and stiffness proportional damping could offer more flexible means of calibrating the damping ratio. Table 3.1 tabulates Raleigh damping parameters for each case, and Figure 3.15 shows the results of the sensitivity analyses. It is note that we employed the finite element mesh with the left and right boundaries slaved together. From comparison among the solutions resulted from different implementation of the damping parameters, it appears that all the methods offer reasonable solutions.

Table 3.1 Rayleigh Damping Parameters

Damping Ratio (ζ)	Mass-Proportional Damping Only	Stiffness-Proportional Damping Only	Rayleigh Damping	
	α	β	α	β
0.017	0.403	0.00287	0.202	0.00143
0.022	0.522	0.00371	0.261	0.00186
0.028	0.664	0.00472	0.332	0.00236
0.032	0.759	0.00540	0.379	0.00270
0.029	0.688	0.00489	0.344	0.00245
0.035	0.830	0.00590	0.415	0.00295
0.018	0.427	0.00304	0.213	0.00152
0.016	0.379	0.00270	0.190	0.00135

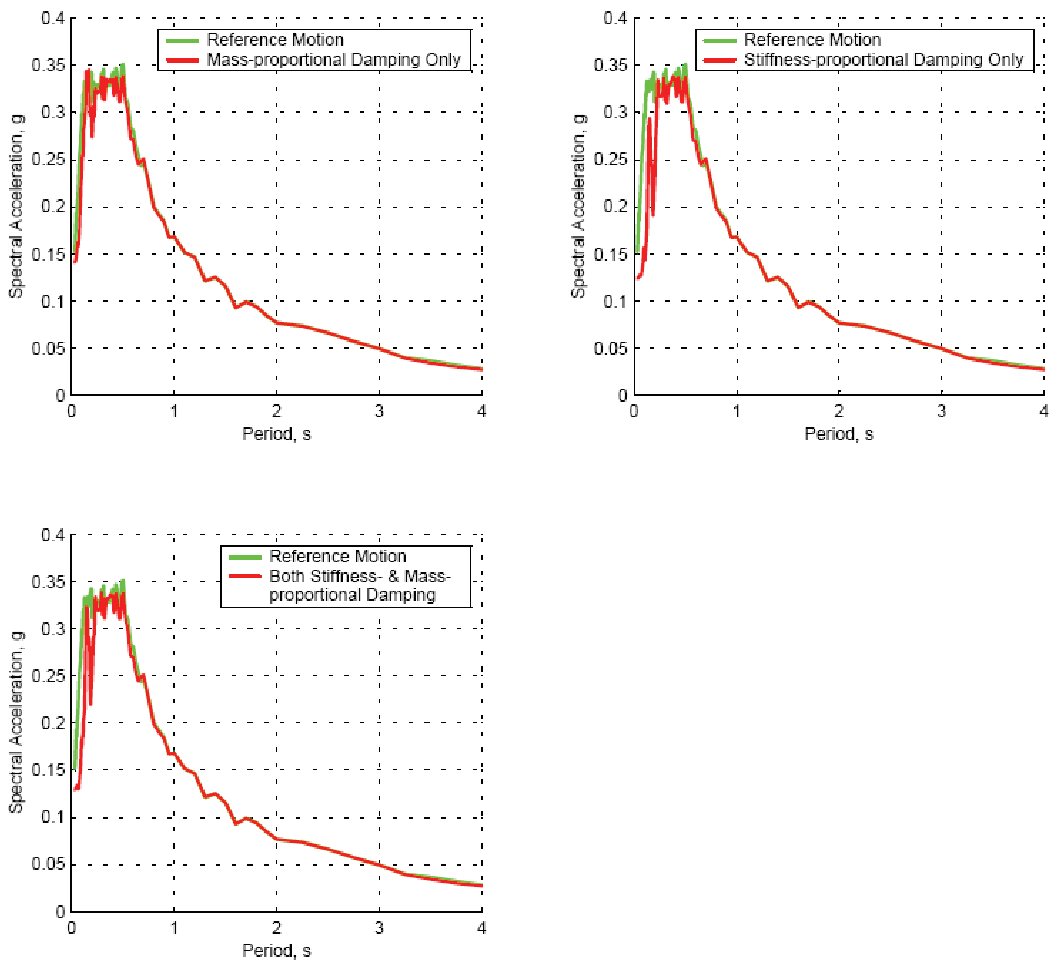


Figure 3.15 Different Damping Assignments

3.4 Two Component Motions

So far, horizontal motion is the only base excitation accounted in the finite element analyses, and thus the response can be checked against the behavior of vertically propagating shear waves which can be treated with a classical shear beam theory. However, most seismic designs consider earthquake loading in two or three directions. To implement two component motions in the finite element model, the reference vertical motion defined at the ground surface requires deconvolution in order to obtain the input base motion; this was accomplished with SHAKE. The ADINA finite element model is then excited with the vertical and horizontal motions simultaneously. The material properties used in ADINA are based on linearly elastic continua adhering to general Hooke's law where Young's modulus (E) and shear modulus (G) are related through Poission's ratio (ν) such that

$$E = 2\ (1+\nu)\ G$$

In practice however when computing depth varying horizontal and vertical motions with SHAKE, separate computer runs are made; one with S-wave velocity (Shear Modulus) profile, and one with P-wave (Constrained Modulus) profile. Often time, a few iterations are performed within SHAKE to achieve strain compatible moduii and it is carried out separately for the horizontal direction and vertical direction. The iteration process could result in incompatibility between S-wave and P-wave velocities. If the iterative solutions are adopted in SHAKE, it is difficult to compromise the comparison between SHAKE and ADINA.

In order to reconcile results of two-component shaking from ADINA with those of SHAKE, one must verify that the P-wave velocity and S-wave velocity employed by SHAKE are uniquely related, similar to the general Hooke's law.

The comparison between SHAKE and ADINA on the two-component motion is shown in Figure 3.16 using the finite element model that slaves the left boundary to the right boundary. The comparison is very favorable although there are some discrepancies in the treatment of damping. For example, SHAKE uses the different damping ratios in the horizontal deconvolution from the vertical deconvolutions, while ADINA allows only one set of Raleigh damping parameters that are applied to the entire system, excited by horizontal and vertical motions simultaneously. The Raleigh damping used in the finite element model has been calibrated to the damping ratios of the horizontal SHAKE's profile. Based on this observation, it seems that minor discrepancy in soil damping does not contribute to significant differences in the seismic response of the ground.

A similar comparison is made between SHAKE and ADINA using the two-edge column concept; the results are shown in Figure 3.17. The following summarizes the ADINA model:

Distance to the side boundary from the centerline = 246 ft
Width of the edge column = 65 ft

The ADINA solutions using the two-edge columns are not as good as those results presented in Figure 3.16. Attempts were made to improve the ADINA solutions by increasing the width of the edge column; however the degree of improvement is poor. The reason is attributed to the fact that the finite element model with two-edge columns is unable to maintain the constraint conditions required in one-dimensional wave propagation especially when vertical motion is introduced. Let us imagine P-wave propagation in the finite element model with two edge columns. Due to the Poisson's ratio effects, the vertical strain in soil due to passage of P-wave would lead to horizontal displacement pushing the edge columns outward. This would have resulted in violation of the constrained modulus assumption made in SHAKE for the vertical wave propagation problem.

To make this point, site response analysis was performed using the vertical input motion only, and snapshots of relative deformation profiles of the two edge boundaries are plotted in Figure 3.18. Without horizontal excitation, the edge boundaries deform laterally as a result of the Poission's effect. If a true one-dimensional condition is maintained, the two side boundaries would have moved together, and the profiles of the two edge columns would have been identical. From inspection of the edge boundary profiles, it is obvious that the two-edge column model tends to deviate from one-dimensional behavior when the vertical motion is introduced. If the vertical motion is absent, the response of the edge column model seems to yield reasonable results.

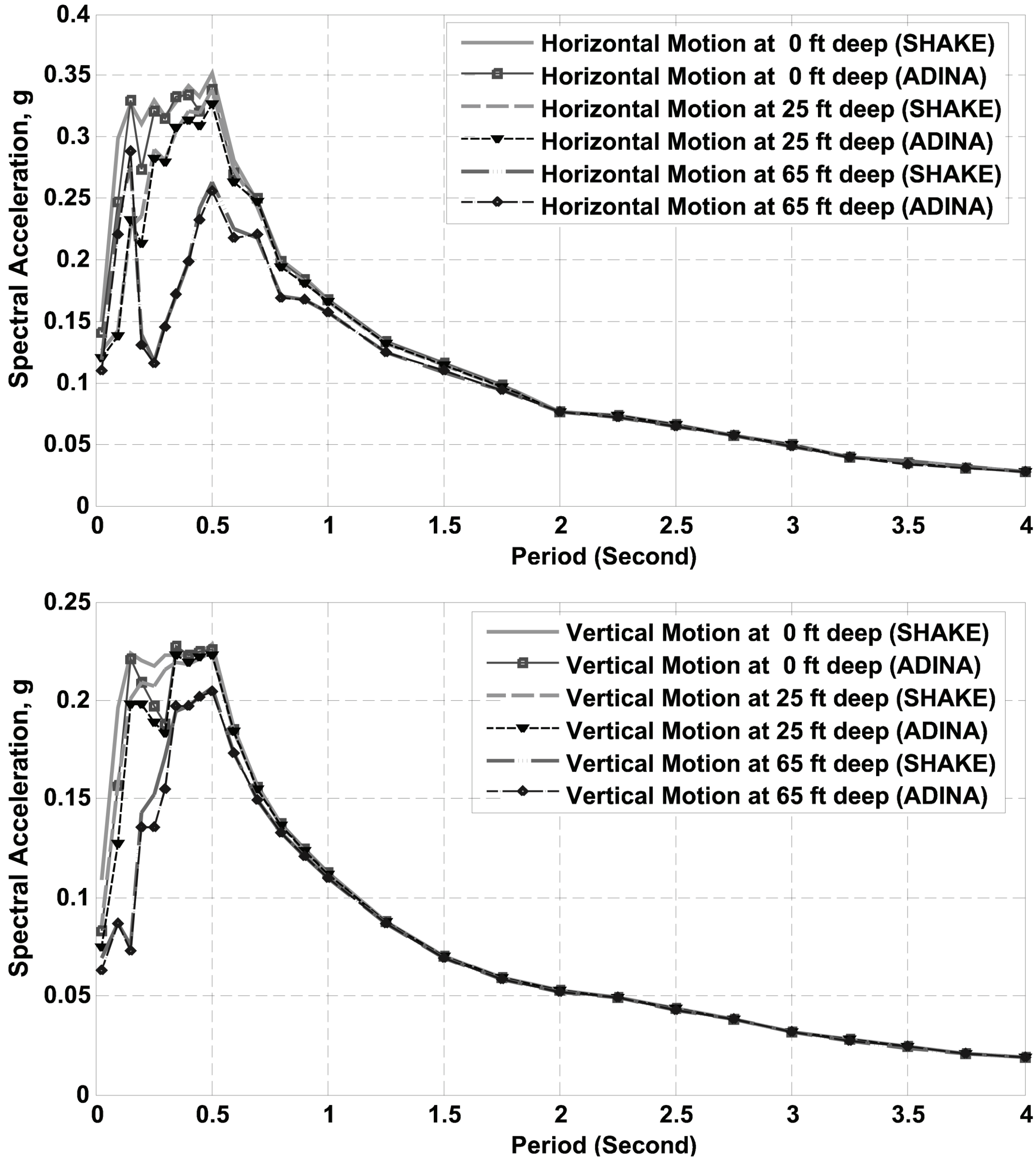
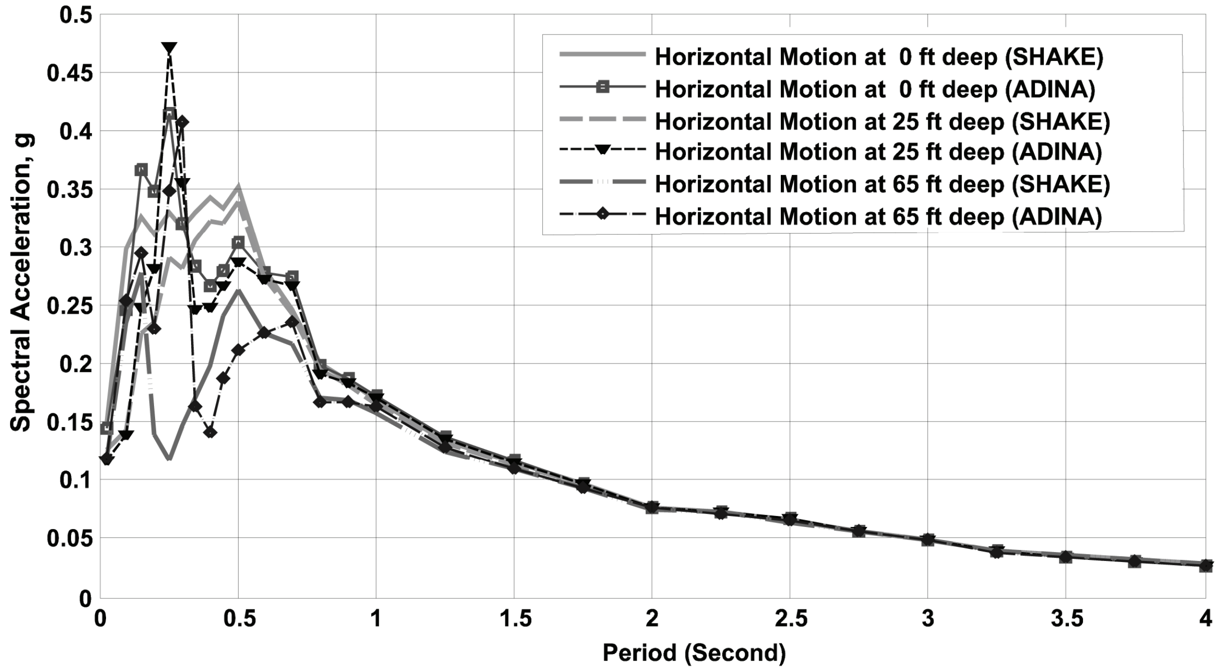


Figure 3.16 Two Component Motions for ADINA Model with Slaving Left and Right Boundaries



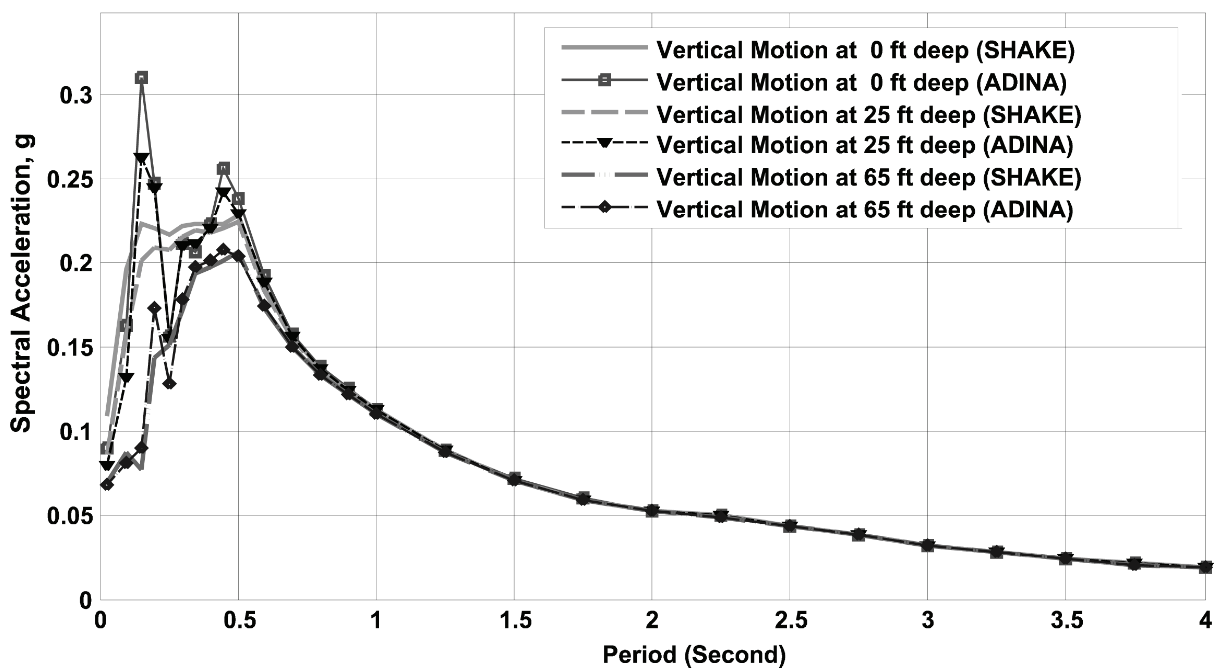


Figure 3.17 Two Component Motions for ADINA Model with Two Edge Columns

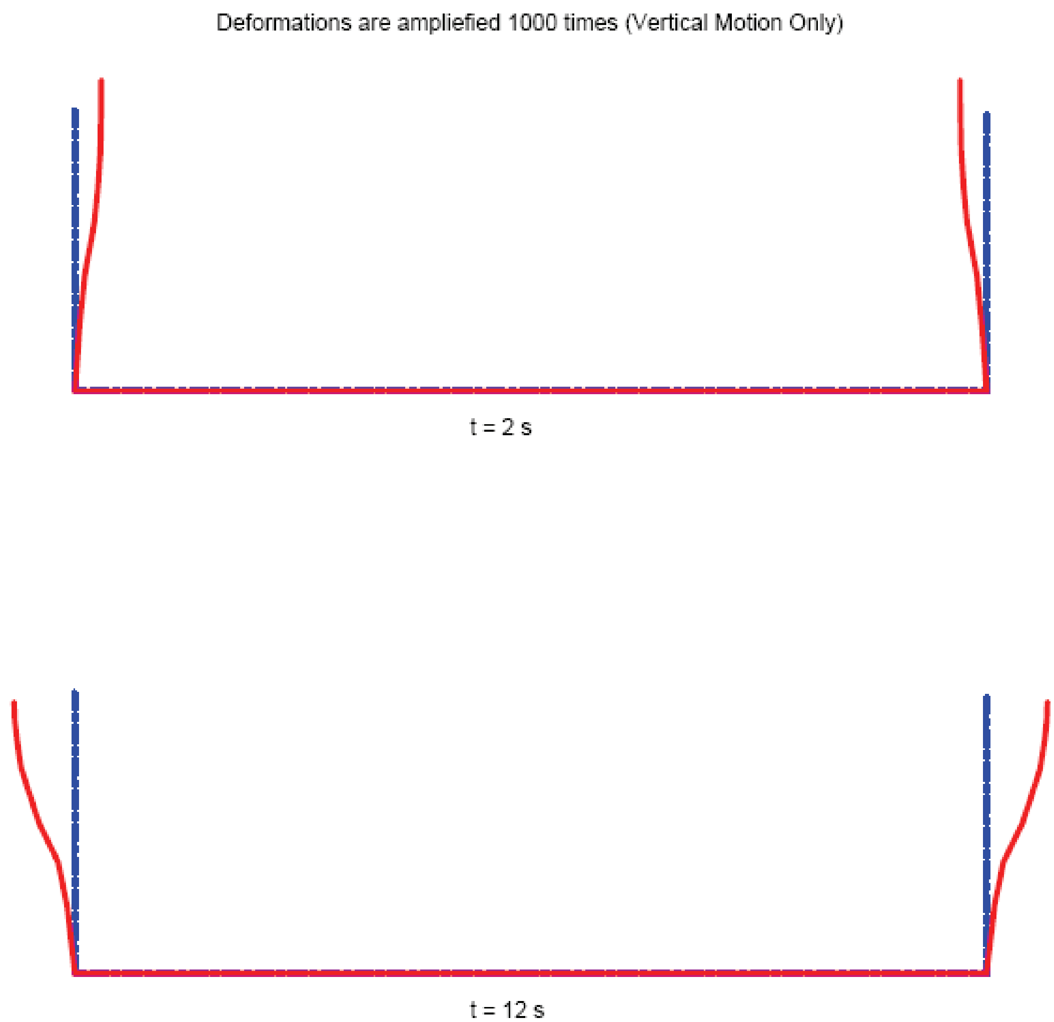


Figure 3.18 Snapshots of Profile of Side Boundary

SECTION 4
PILE GROUP PROBLEM

4.1 Considerations of Wave Scattering

In this section, the wave propagation analysis is extended to include a pile group foundation in order to appreciate how wave scattering of the foundation system alters the nearfield ground motion transmitted to the superstructure. It is critical to understand how pile group problems can be implemented in a time domain finite element program, which is inherently a two-dimensional wave propagation process. Similar to the level ground case, the boundary conditions (both base and side boundaries) are the prime factors affecting the site response behavior.

Inclusion of 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 two types of wave (P- and S-waves) where the stiffness contrast between two media is present. This phenomenon is usually termed "wave scattering." Although wave scattering has been considered in the designs of nuclear containment systems, the current state of practice for seismic design of bridges does not take this effect into account.

The current study evaluates on the wave scattering subject related to large pile groups, often employed in long span bridges. It has been known to some practicing engineers that wave scattering effects would 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 4.1 highlighting ranges of foundation sizes. Figures 4.1 to 4.4 illustrate the shapes and dimensions of some pile foundations supporting the 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, as the wavelength is defined as

$$\lambda = T\,c$$

where λ = wavelength, T = period and c = propagation speed of the medium.

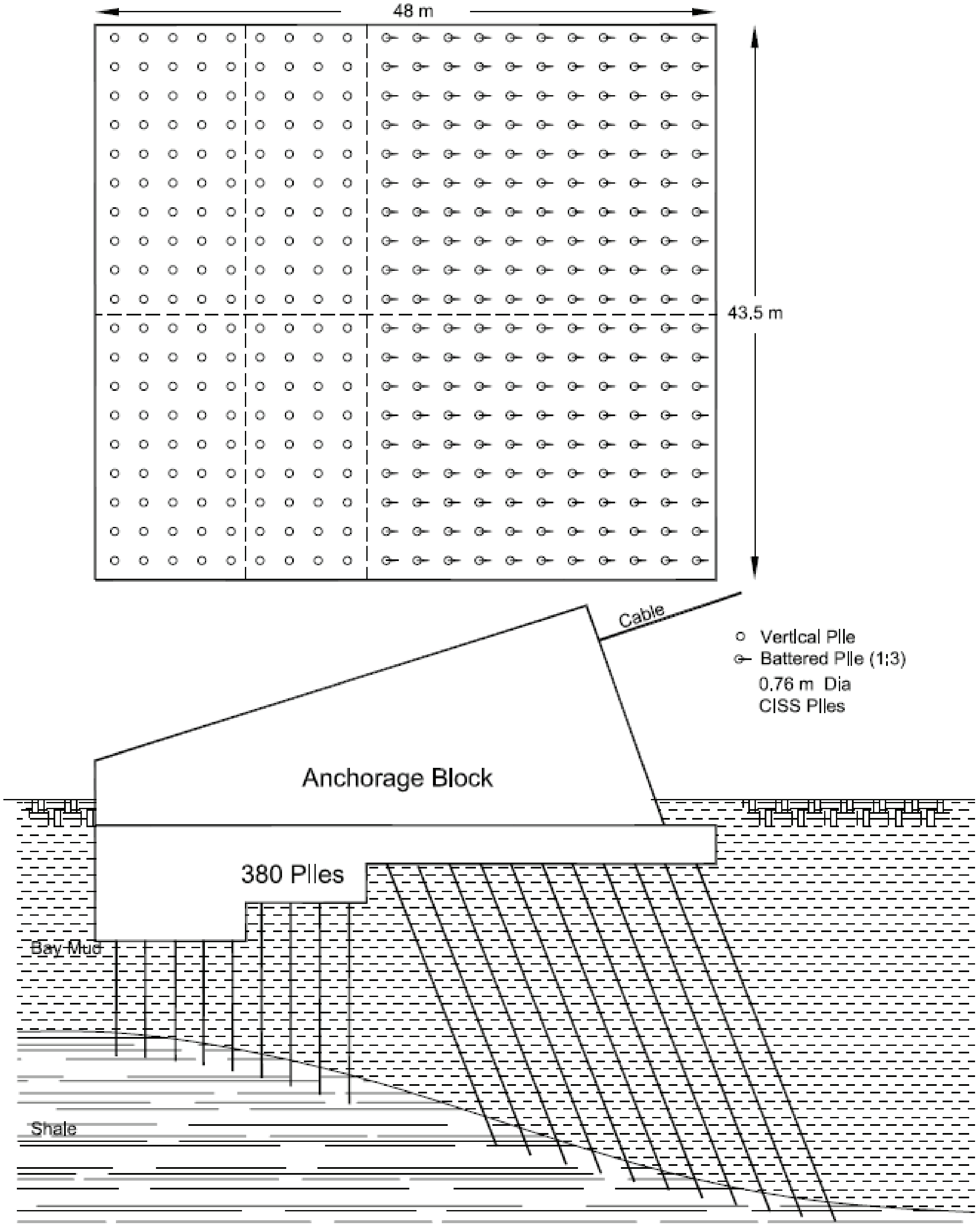


Figure 4.1 South Anchorage Foundation of New Carquinez Bridge, CA

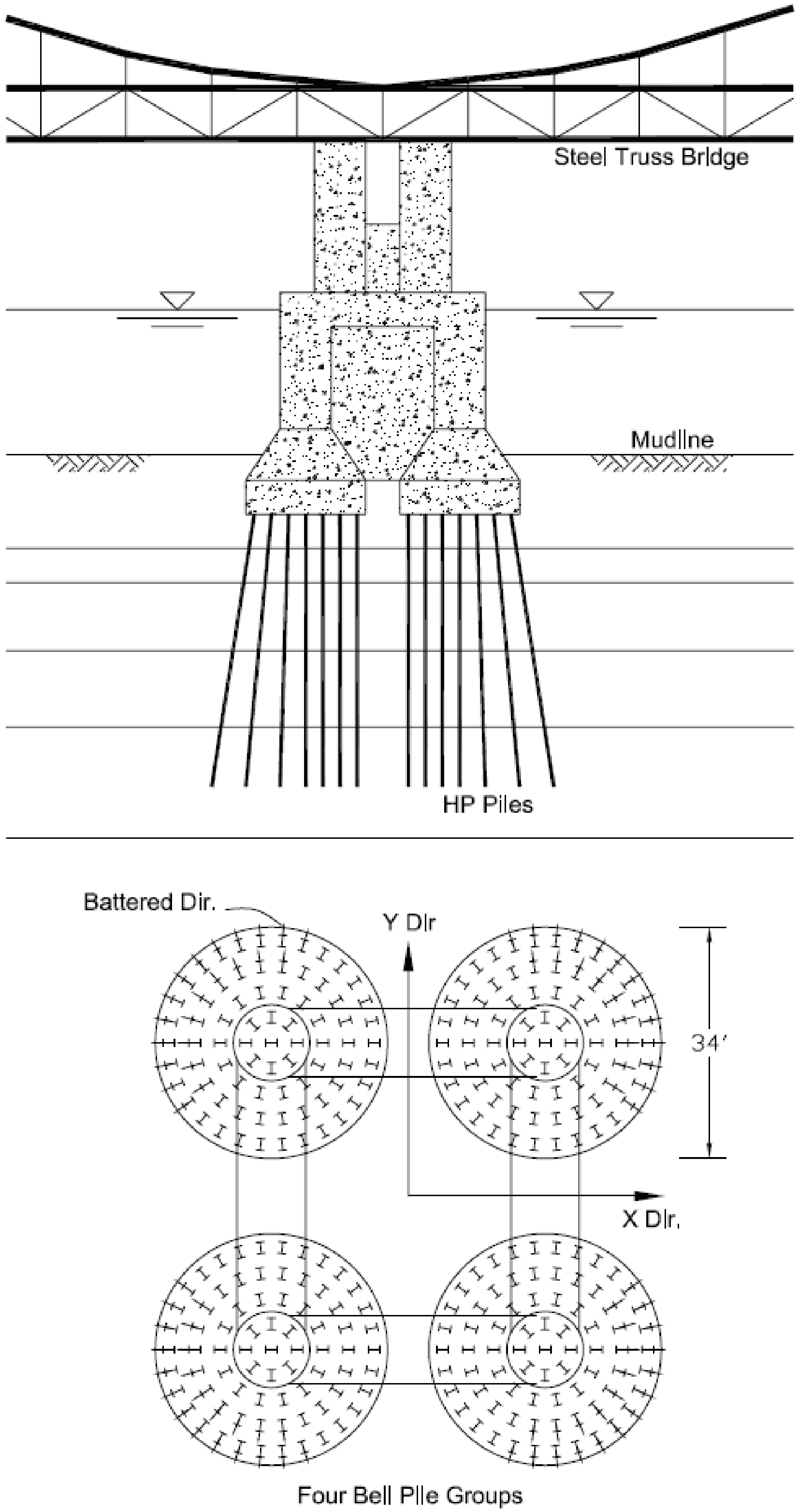
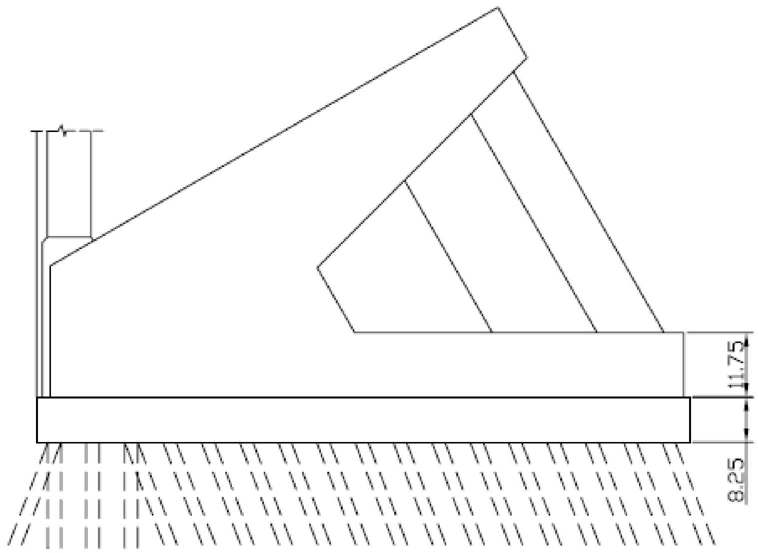
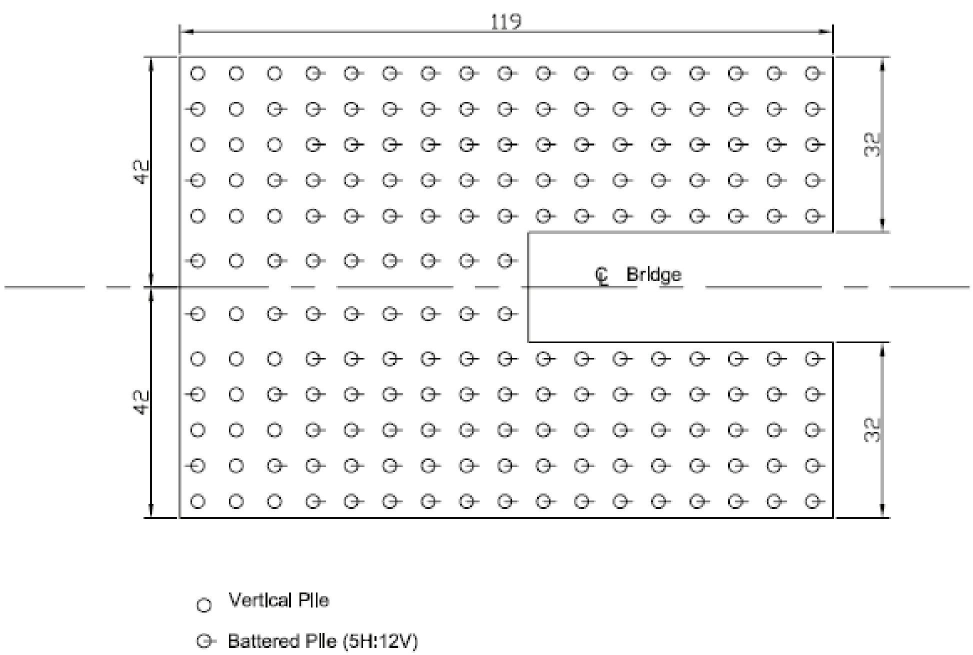


Figure 4.2 Typical Foundation of Main Span at Richmond-San Rafael Bridge, CA



Elevation View



Plan View

Figure 4.3 Foundation Plan at Anchorage of Vincent Thomas Bridge, CA

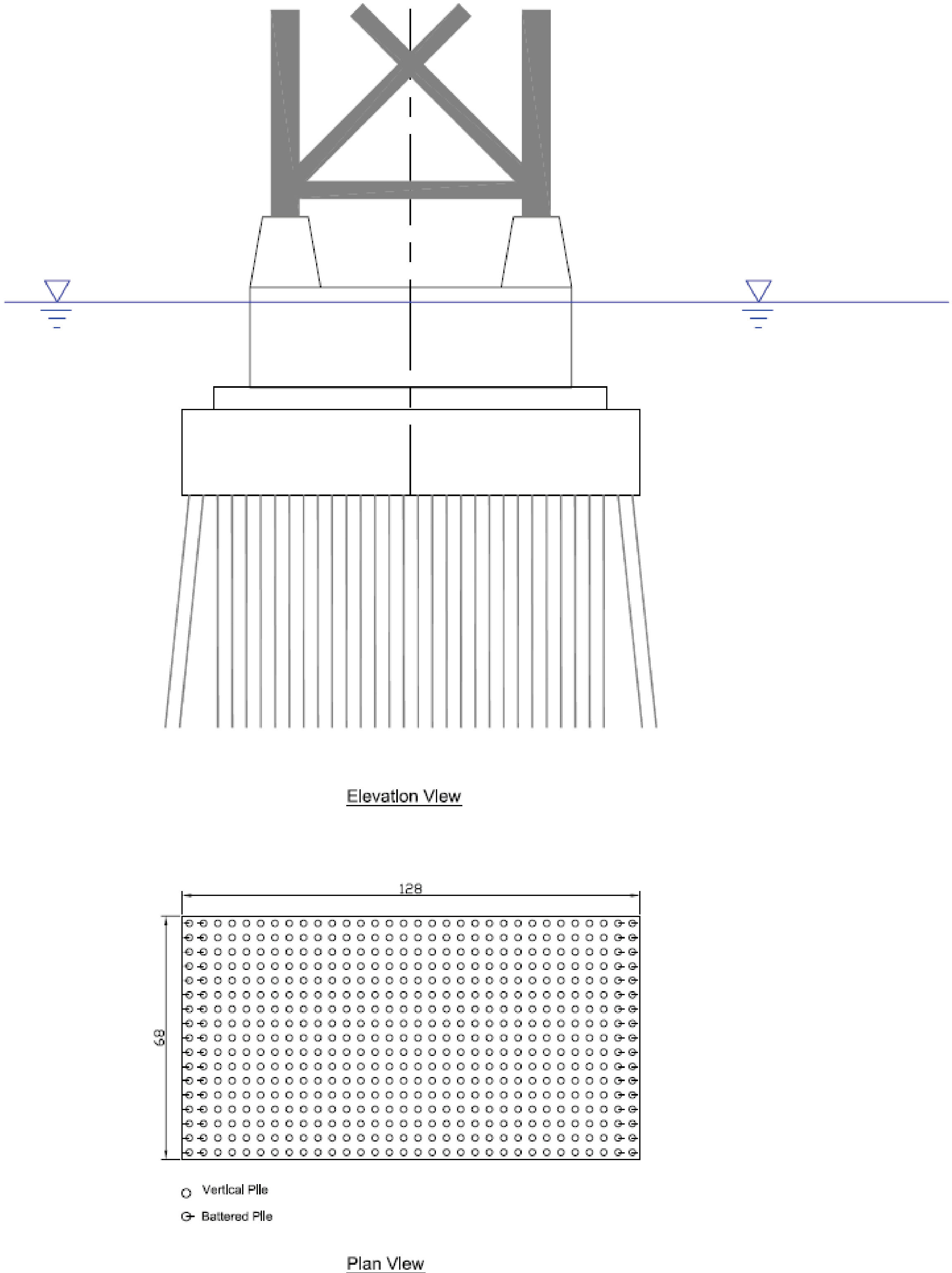


Figure 4.4 Bent Footing Supporting East Span (Existing) San Francisco-Oakland Bay Bridge, CA

Table 4.1 Some Large Pile Groups for California Bridges

Location	Bent	Pile Type	Number of Piles	Soil Type
Vincent Thomas Suspension Bridge, Long Beach, CA	East Anchorage East Tower West Tower West Anchorage	Steel 14BP117 Steel 14BP117 Steel 14BP117 Steel 14BP117	188 165 167 188	Silty Sand Silty Sand Fine Sand/Clay Fine Sand/Clay
Richmond-San Rafael Bridge, Richmond, CA	Pier 34 Pier 48	Steel 14BP117 Steel 14BP117	308 308	Bay Mud Bay Mud
San Francisco – Oakland Bay Bridge, Oakland, CA (Existing Bridge)	Pier E6 Pier E7 Pier E8 Pier E9	Timber Timber Timber Timber	544 544 544 625	Bay Mud Bay Mud Bay Mud Bay Mud
Old Carquinez Truss Bridge, Vallejo, CA	Pier 4 Pier 5	18" Concrete Timber	225 151	Bay Mud Bay Mud
New Carquinez Suspension Bridge, Vallejo, CA	South Anchorage	30" CISS	380	Bay Mud

4.2 Implementation in Time Domain Approach

To implement the wave scattering model in the time domain approach, we establish a hypothetical foundation consisting of 3 ft diameter concrete piles arranged in a 7 x 7 grid pattern. 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. Figure 4.5 shows a schematic of the hypothetical pile foundation used in this study.

The wave scattering of this pile foundation problem was first analyzed with the frequency domain based computer program, SASSI. The results from SASSI serve to provide benchmark solutions, which are used to compare with the time-domain solutions from ADINA. Though 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 in 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

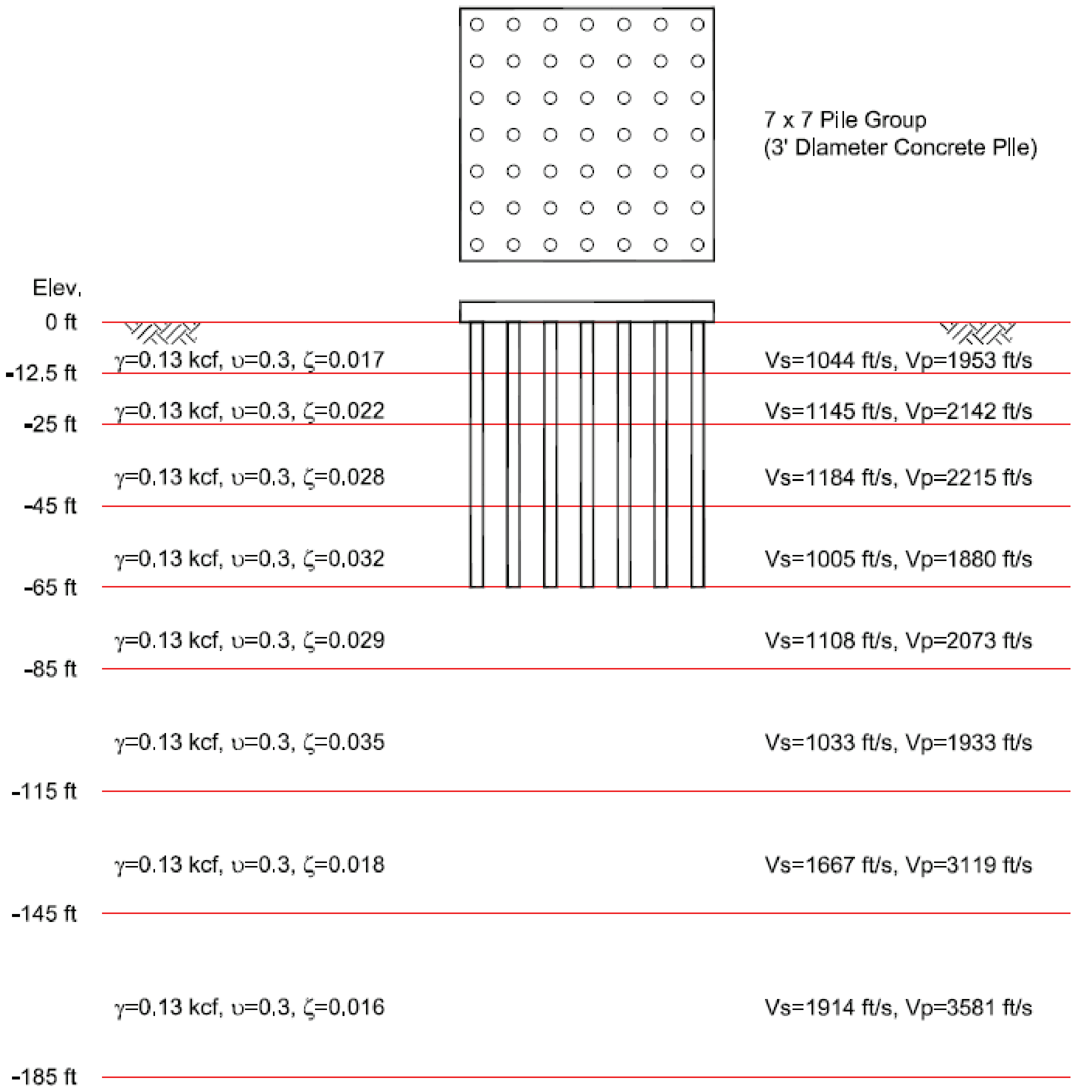


Figure 4.5 Hypothetical Pile Foundation and Soil Strata

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 4.6 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 a 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 4.7 shows the computed time histories and their response spectra at three selected locations within the foundation footprint along the centerline.

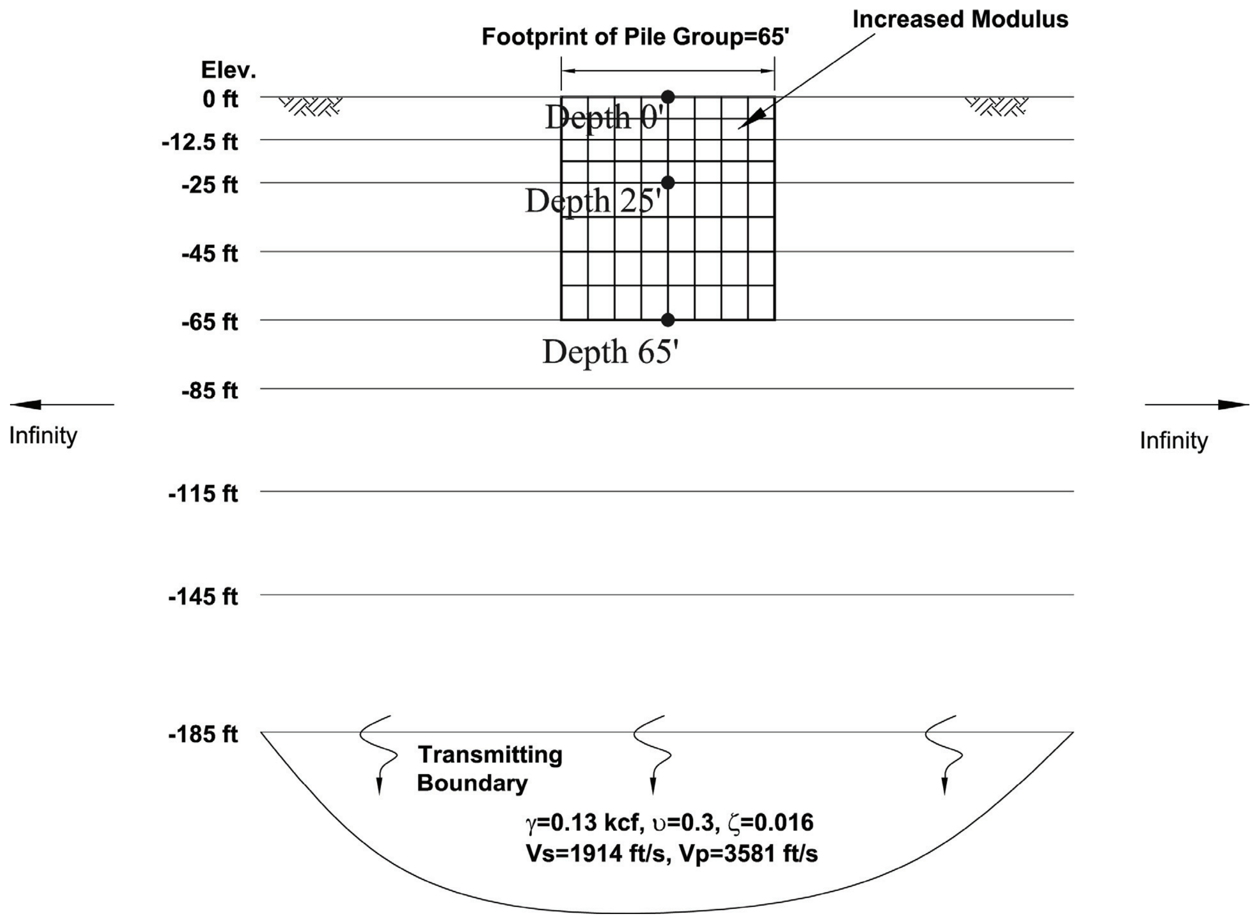


Figure 4.6 Conceptual SASSI Model of Pile Group

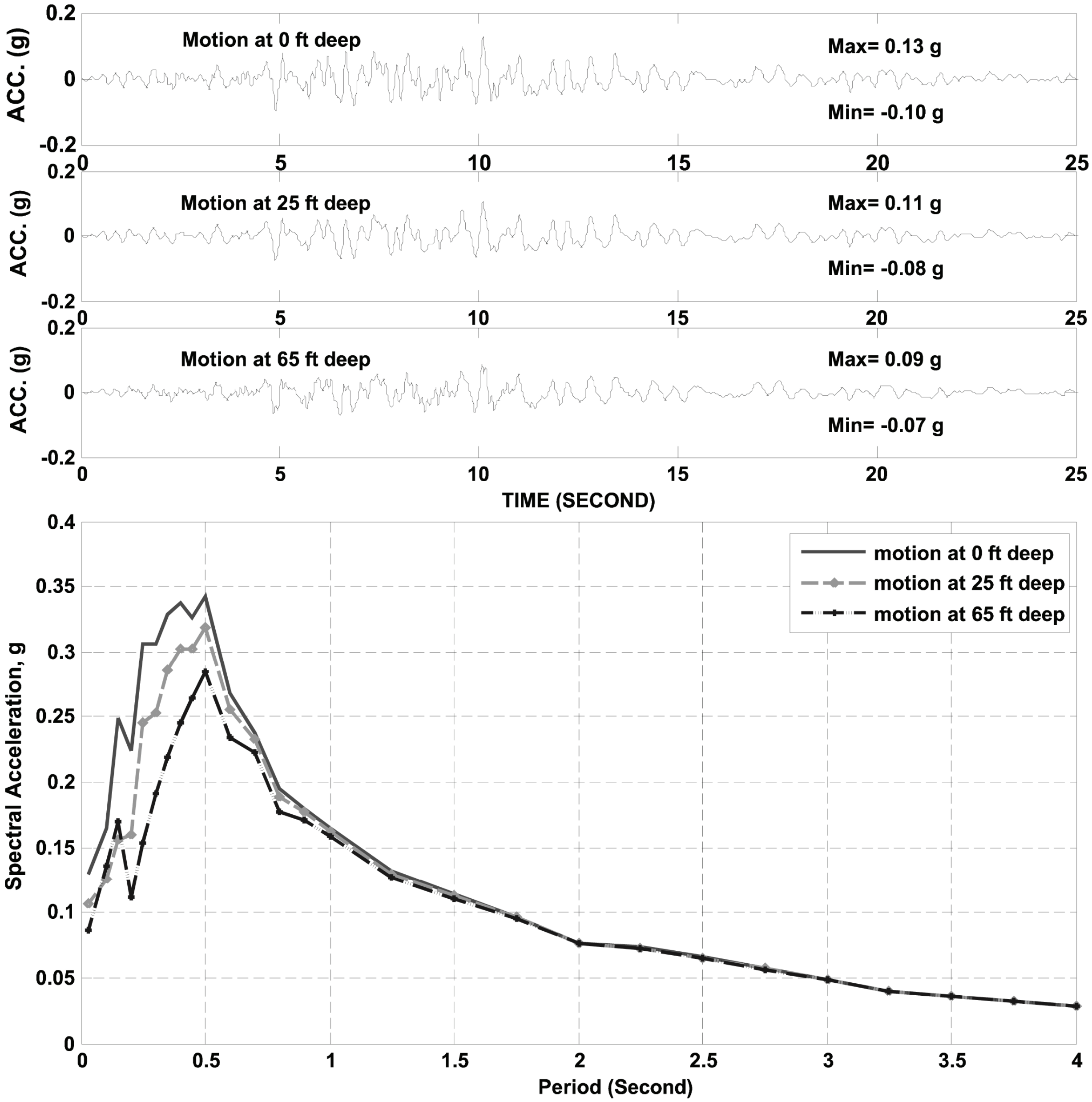


Figure 4.7 SASSI Results of Computed Time Histories and Response Spectra at Three Selected Locations

Attempts were made to simulate this pile foundation problem in time domain using the program ADINA. Figure 4.8 depicts the finite element model of the pile foundation problem with a rigid base. The side boundaries are treated by slaving of 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) is used to include the damping. The solutions obtained from ADINA are provided in Figure 4.9 showing acceleration time histories and response spectra at the three locations.

Comparison between SASSI and ADINA can be seen in Figure 4.10 in terms of near-field response spectra for this pile foundation problem. We consider this comparison to be 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.

4.3 Wave Scattering of Near-field Soil

Wave scattering modifies the near-field soil response when the subsurface conditions are altered by presence of the foundation. It has been demonstrated that the time domain approach can be used to study the wave scattering mechanism affecting the near-field soils of the pile group. Physical dimensions of the pile group and wave propagation speeds of the soil would influence the wave scattering mechanism. To study the effects of foundation size, we consider the model shown in Figure 4.11 where the width of the foundation is increased from 65 to 195 feet and the side boundaries are sufficient 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 others depending on the local soil conditions.

The program ADINA is used to compute depth-varying near-field motions along the pile length. Once the near-field motions are obtained, soil-structure interaction analyses are 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 4.12. In this soil-structure interaction analysis, the mass of structures is not included and only stiffness is considered. The resultant motion computed at the foundation level is termed "kinematic motion" that effectively drives the bridge superstructure. The method of computing the kinematic motion is based on a linear theory making use of 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 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 'weighed average' of the near-field motions which consider soil and pile stiffnesses 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 we will use 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.

Figures 4.13 and 4.14 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 result in the kinematic motion of the pile group that is also presented in the figures comparing against the near-field soil motions.

To evaluate the influence of wave scattering, 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. Such a comparison is presented in Figure 4.15. 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 (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 performed using larger foundation sizes as the wave scattering tends to alter free-field ground motions for large foundations. The results of 130 feet and 195 feet wide foundations are also presented in Figure 4.15; 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 would be adequate for the practical purposes.

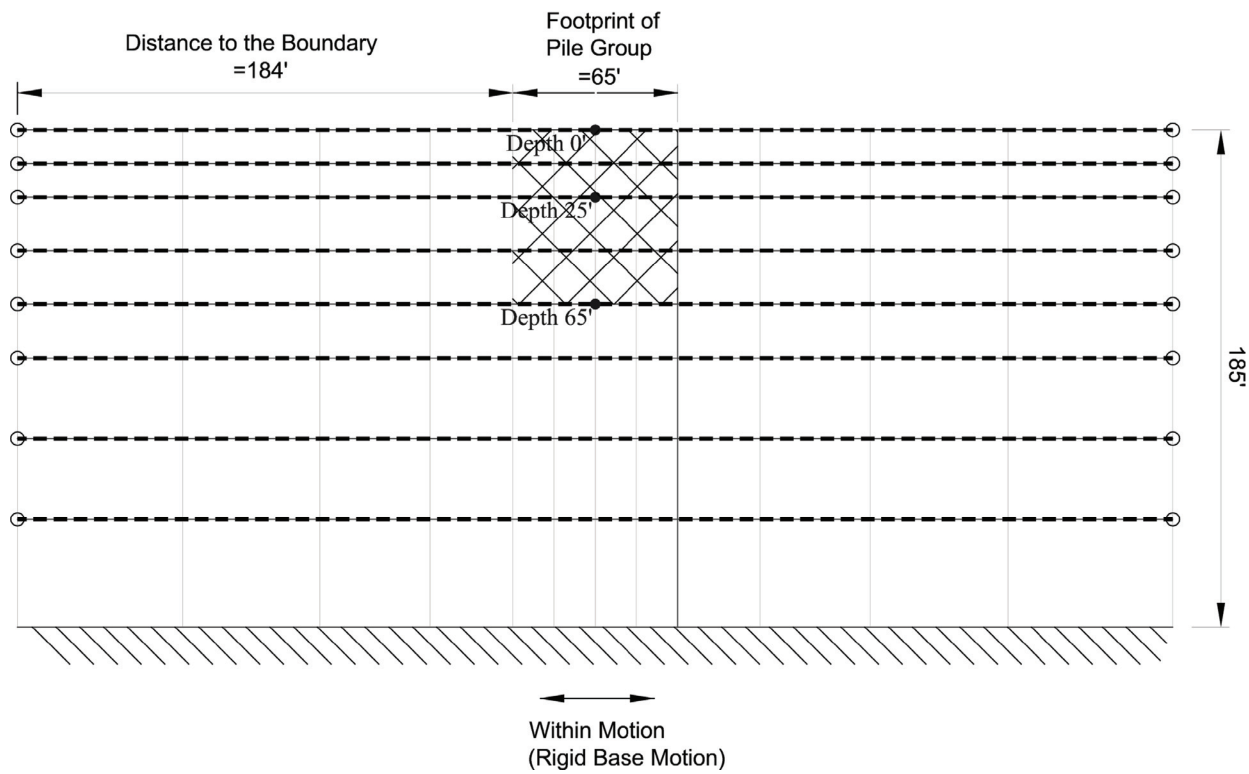


Figure 4.8 ADINA Model of Pile Group with Slaving of the Two Side Boundaries

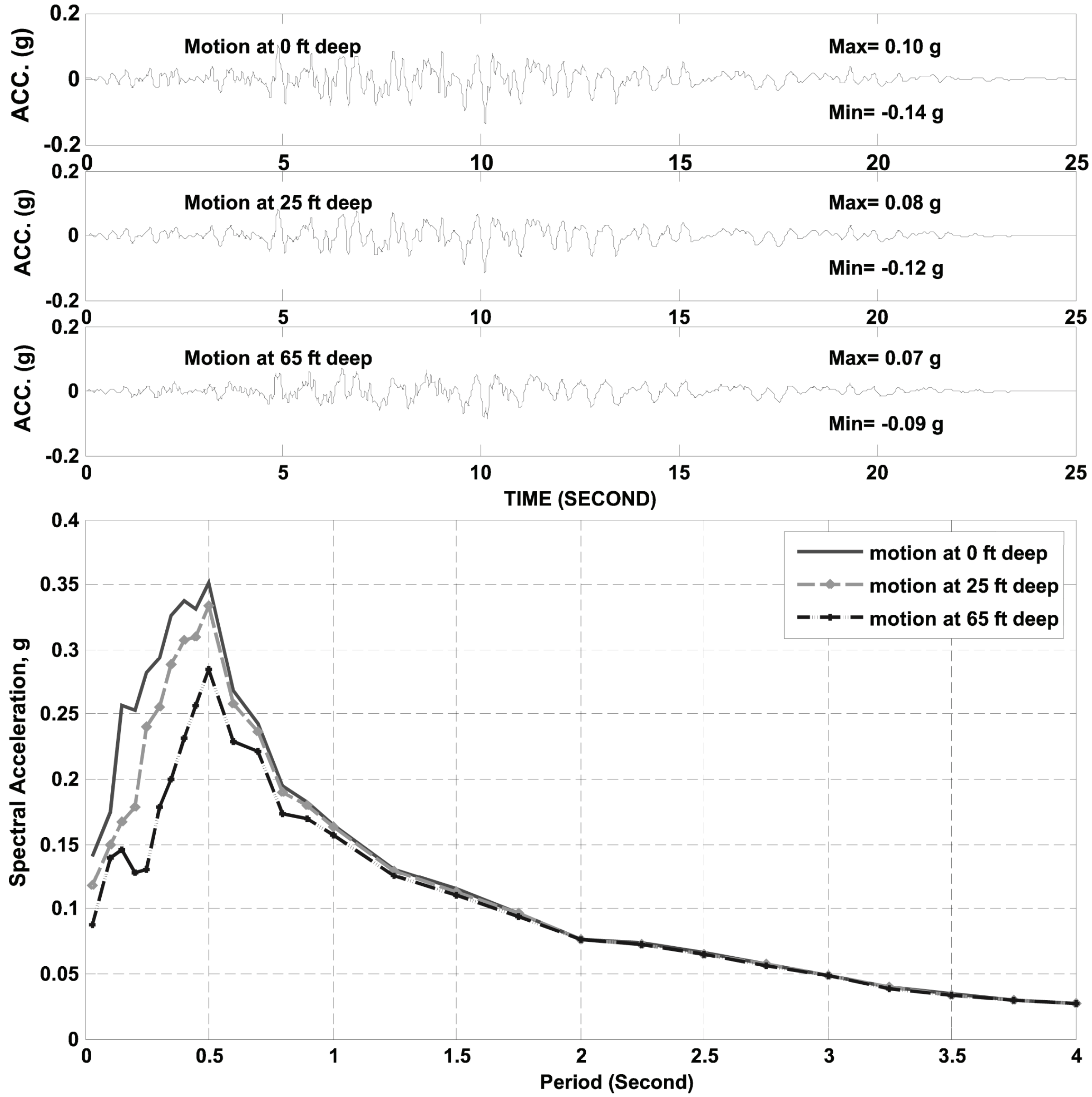


Figure 4.9 ADINA Results of Computed Time Histories and Response Spectra at Three Selected Locations

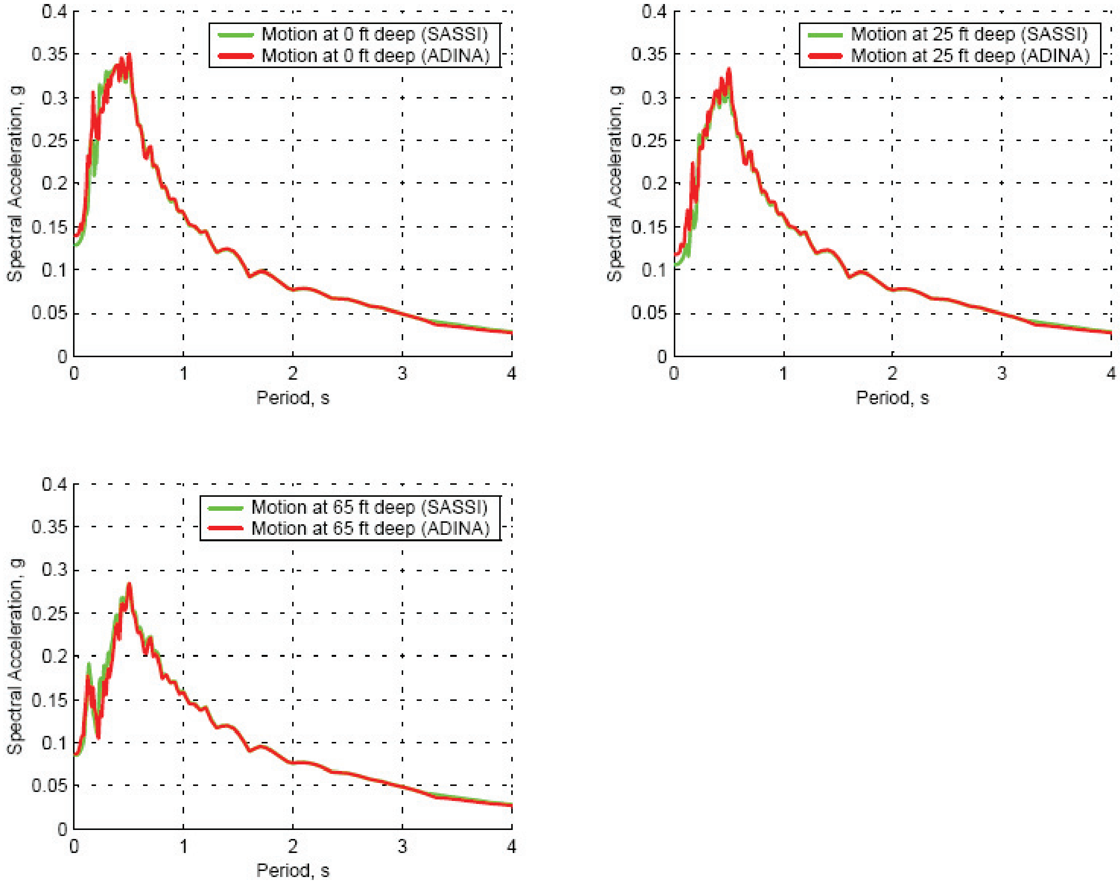


Figure 4.10 Comparisons Between SASSI and ADINA Response Spectra

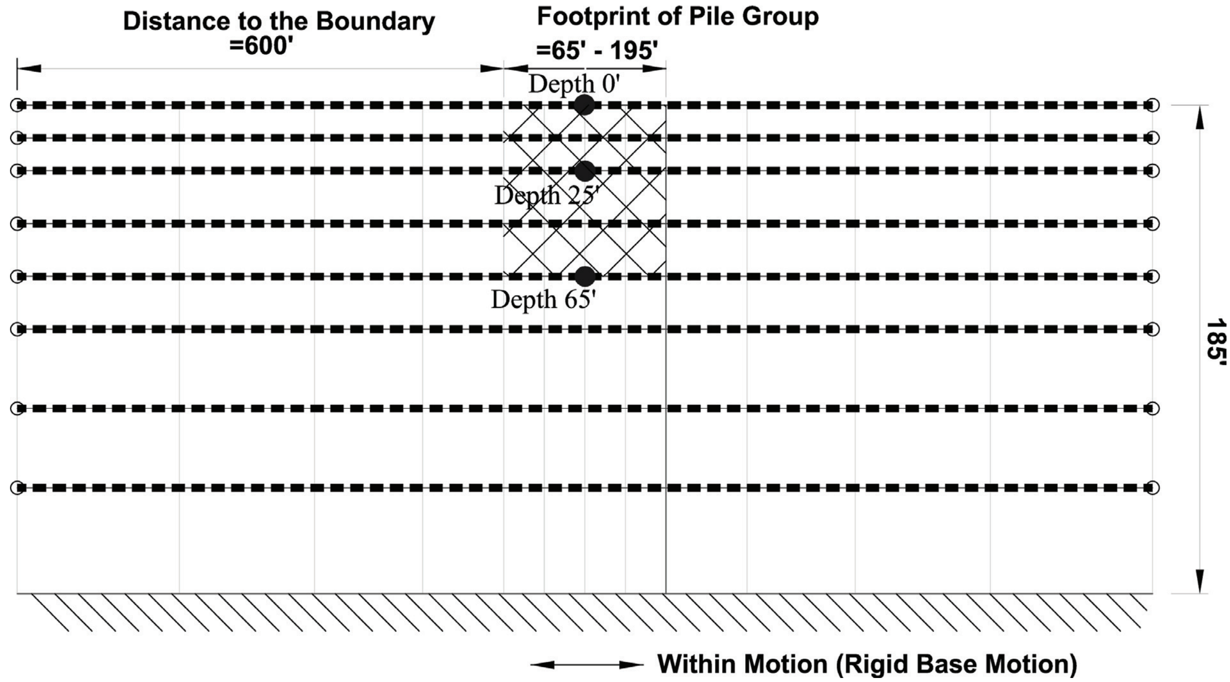


Figure 4.11 ADINA Model to Study Effects of Larger Pile Foundation Size

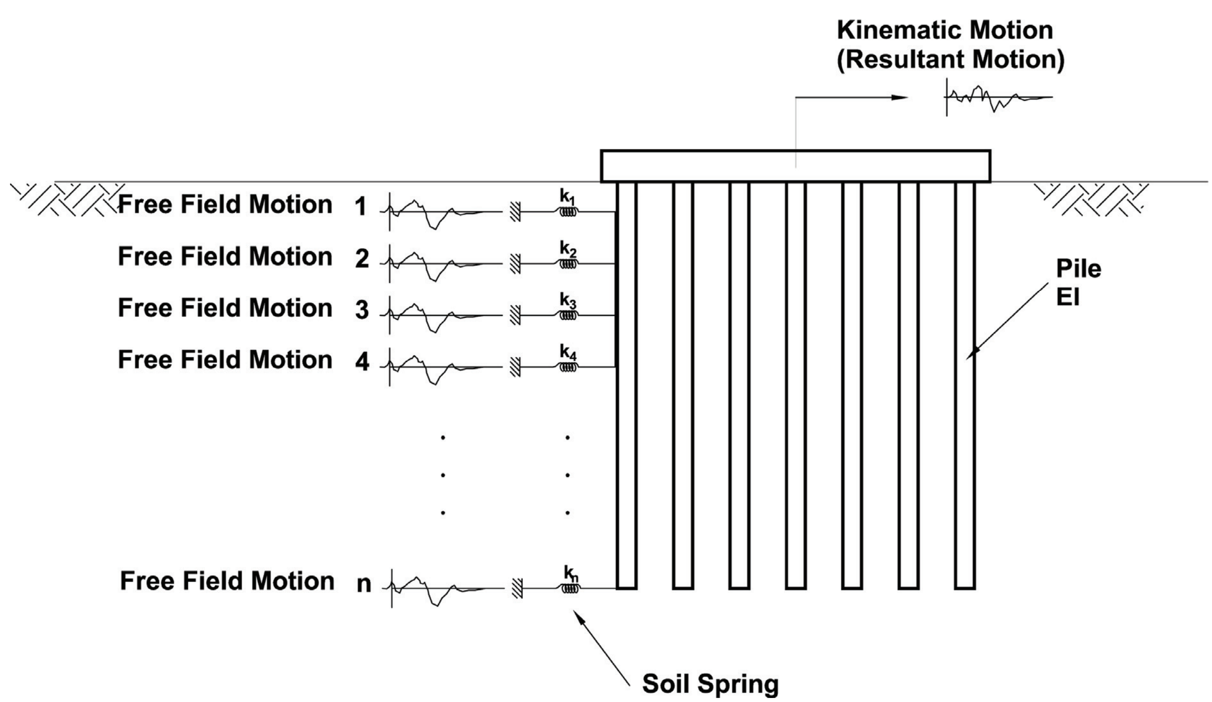


Figure 4.12 Schematic of Kinematic Motion

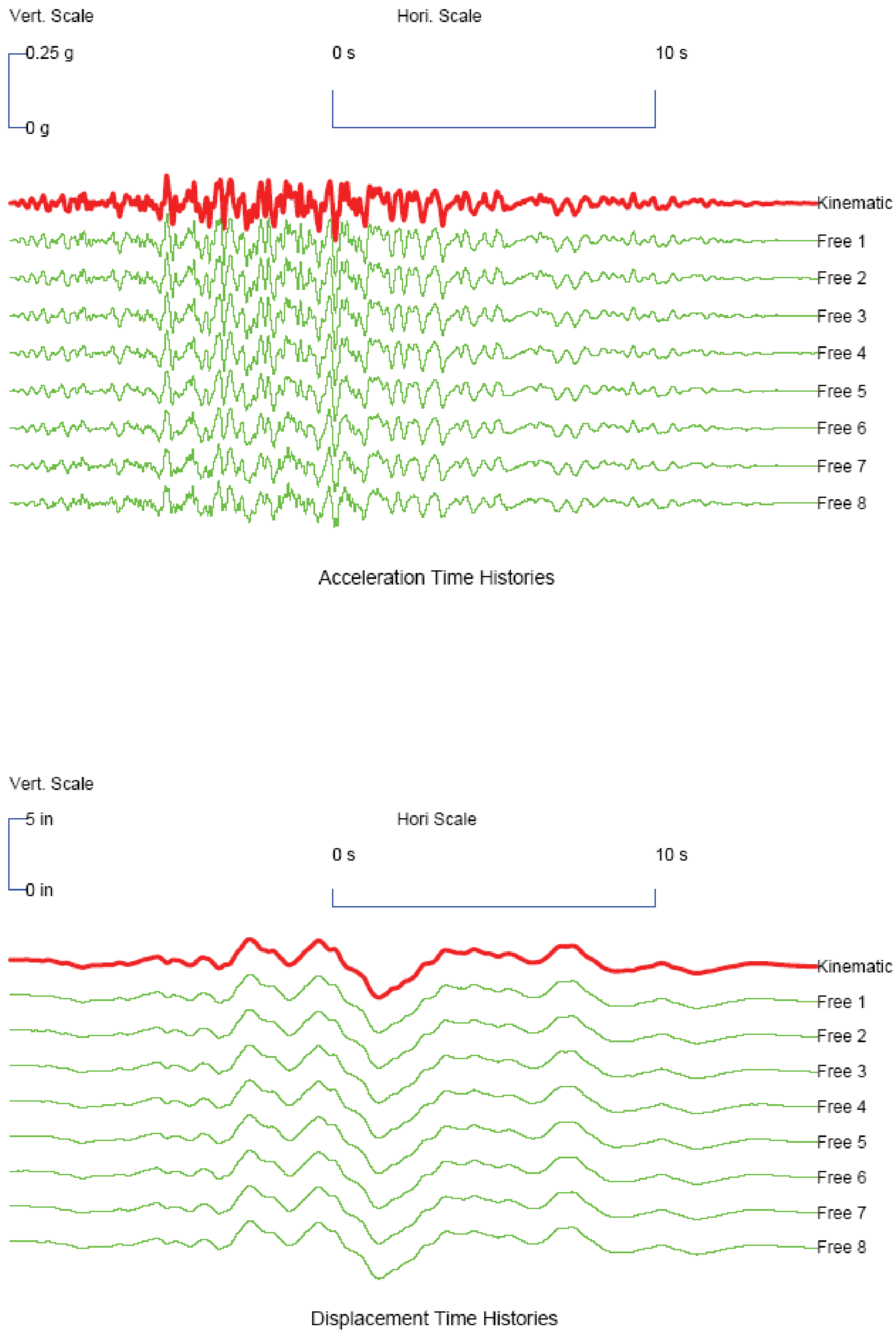


Figure 4.13 Depth-Varying Near-Field Motions vs. Kinematic Motion

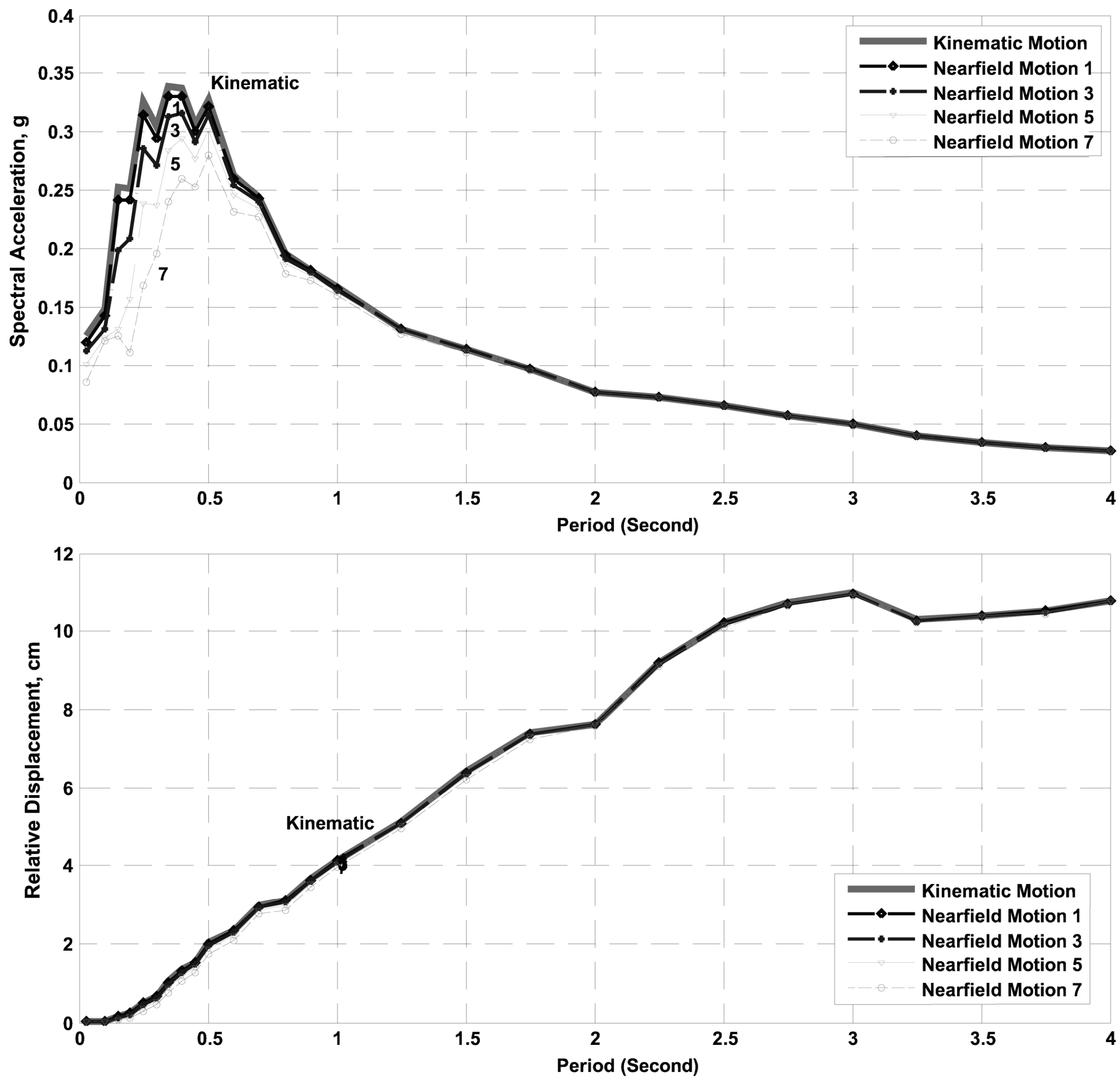


Figure 4.14 Response Spectra of Kinematic Motions and Near-Field Motions

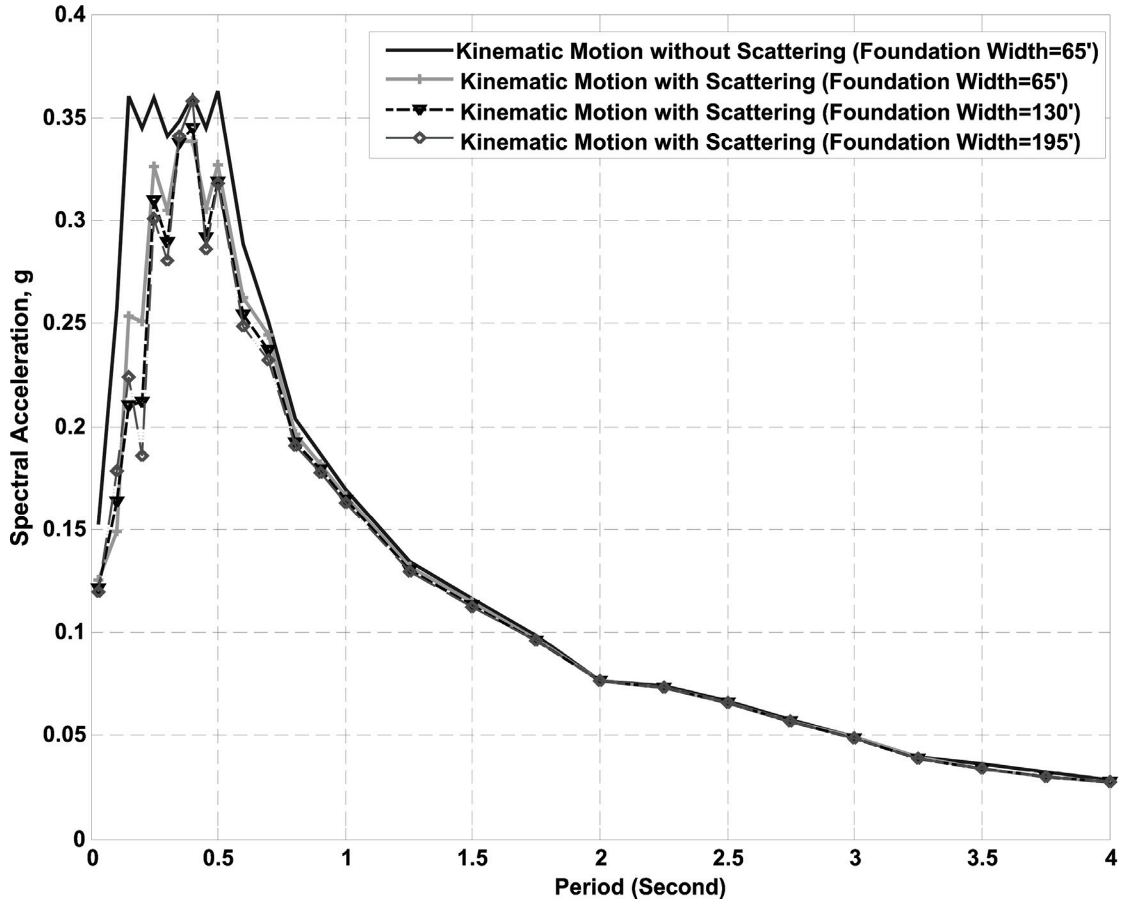


Figure 4.15 Effects of Wave Scattering and Foundation Size in Terms of Response Spectra of Kinematic Motions

SECTION 5
CAISSON FOUNDATION

5.1 Caisson Construction

Caissons are used as the foundation of over-water bridges. Box caissons are tabular concrete structures with a closed bottom and designed to be buoyant for towing to the bridge site. The water depth must be sufficient for floating the unit. Then allowing water through flooding valves gradually sinks the caisson. Box caissons can be used for founding on compact granular soils that are not susceptible to erosion by scour or on a rock surface which is 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 grabbing out the soil from the open wells as the walls raised progressively. This type of foundation is suitable for soils consisting of loose sand or gravels since these materials can be readily excavated by grabbing within the open well and they do not offer high resistance in skin friction to the sinking of the caisson. 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 further concrete is placed.

During the sinking operation, water in the open wells seeks the level of the ground water and thus all excavation must be conducted blind through the water. Alternatively, compressed air can be pumped into the well 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 maintained approximately 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 addition of an air deck and docks only for the final sinking. It is beneficial for excavation and penetrating the cutting shoe into sloping rock and then completing the final sinking through materials that are not readily dredged open.

5.2 Geotechnical Considerations

The site conditions and structure requirement dictate the design criteria of the caisson. It is not common to design and construct caissons in very soft soil although the sinking operation is easier in this kind of material. Suitable site conditions are bedrocks overlain by shallow sediment, which the cutting shoe can readily penetrate, and the unit can rest on the competent rock. Caissons are also constructed in deep sediments with granular soil which can be dredged out within the open wells. When designing caissons to support long span bridges, the specific design items that must be established include:

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

After obtaining the applicable general information, the engineer sets dimensions of the caisson to meet the design requirement. The design methods are similar to those of MSE (Mechanically Stabilized Earth) generally based on conventional bearing capacity and earth pressure theories that examine stability. When sizing the caisson, various failure modes must be considered, as illustrated in Figure 5.1. These stability considerations consist of the following in the general order of importance:

- 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 from 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

Many large deep caissons were built and sunk by the pneumatic method up until World War II. However, with inflation of wages and with limitation in working hours, pneumatic caissons have been priced out of ordinary construction. To overcome the problems associated with working under air pressure, robotic cutters have been developed to excavate and remove soil within the chamber.

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 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. Both bridges were built prior to World War II.

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 can be found as foundation for the Coleman Memorial Bridge in Delaware. Figures 5.2 to 5.5 show dimensions of some of the caissons.

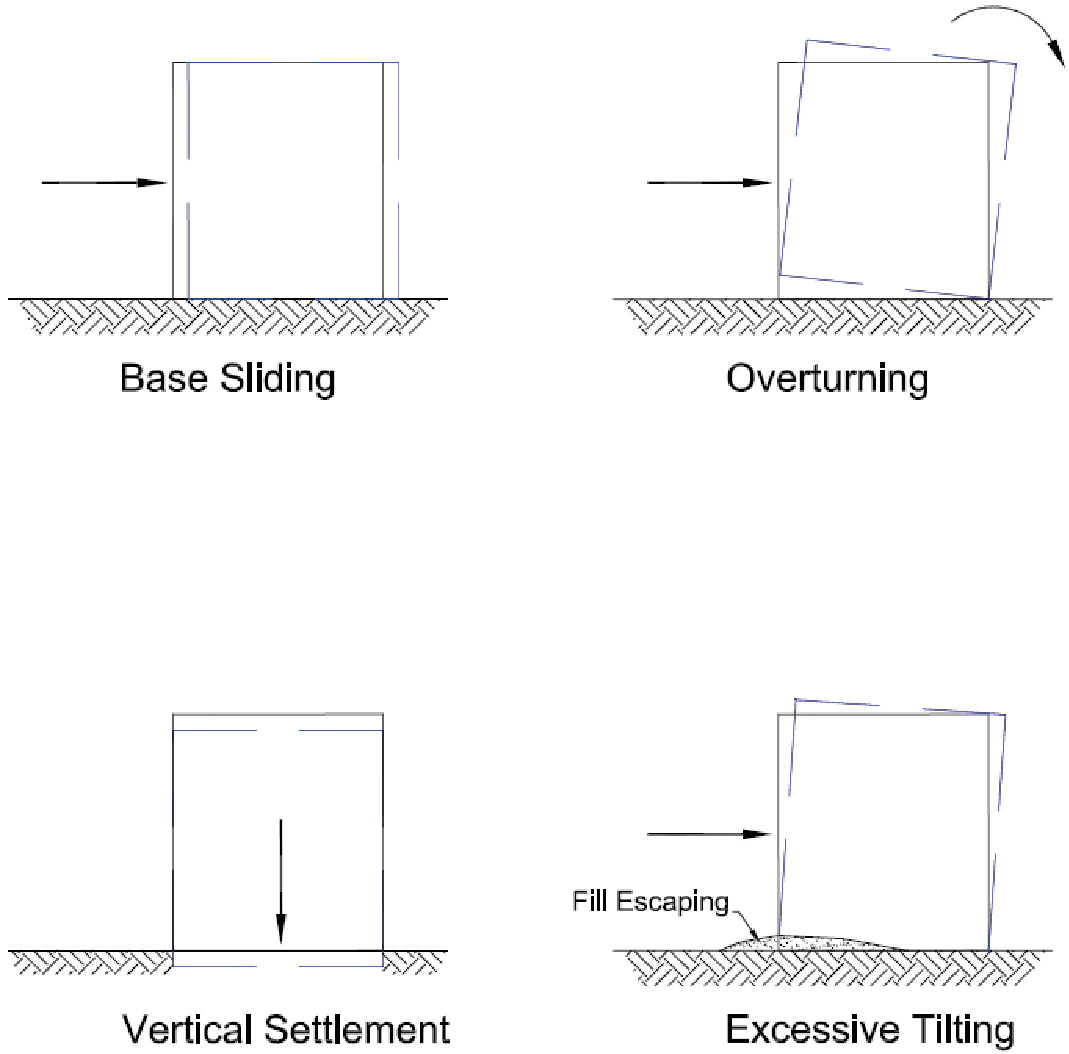


Figure 5.1 Possible Failure Modes of Caissons

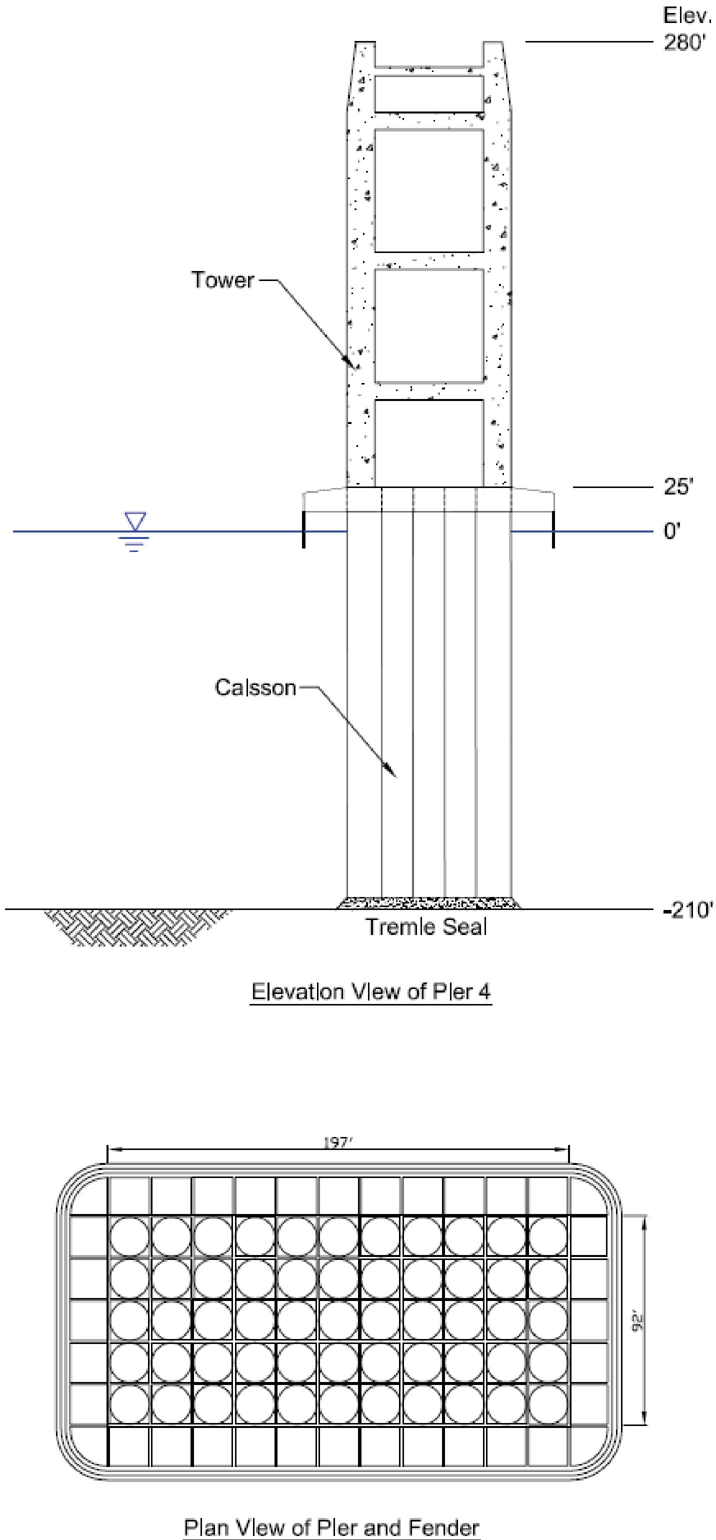


Figure 5.2 Caisson of Pier 4 of West Span SFOBB, CA

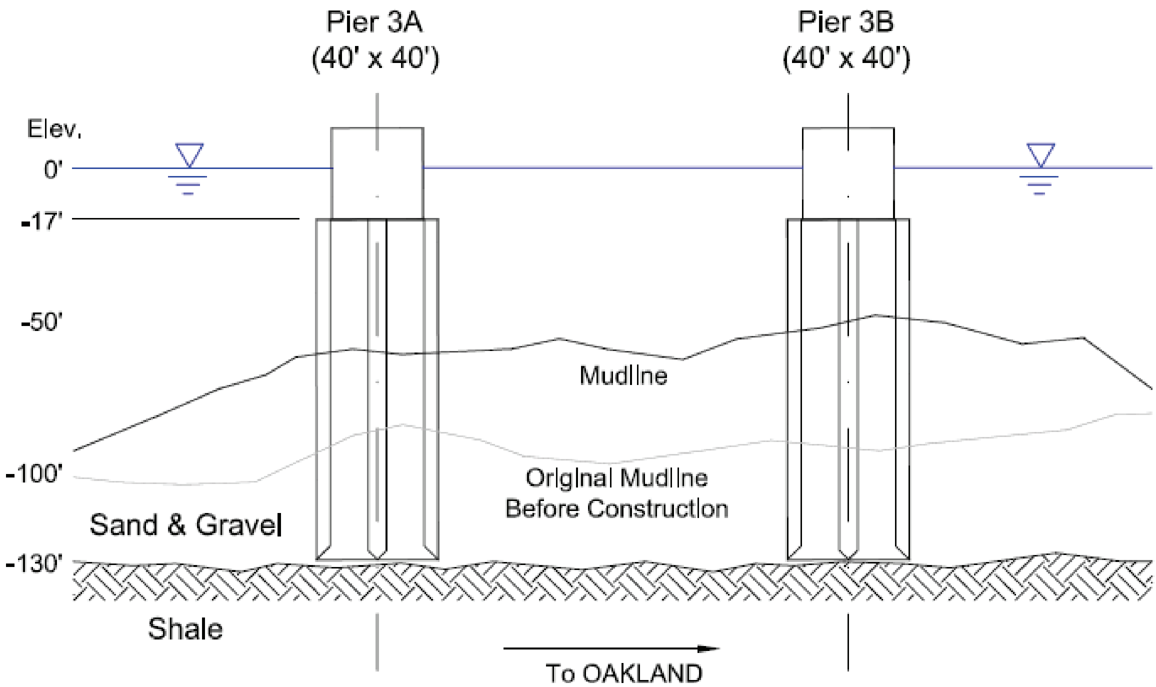


Figure 5.3 Multiple Caissons at Pier 3 of Carquinez Bridge, CA

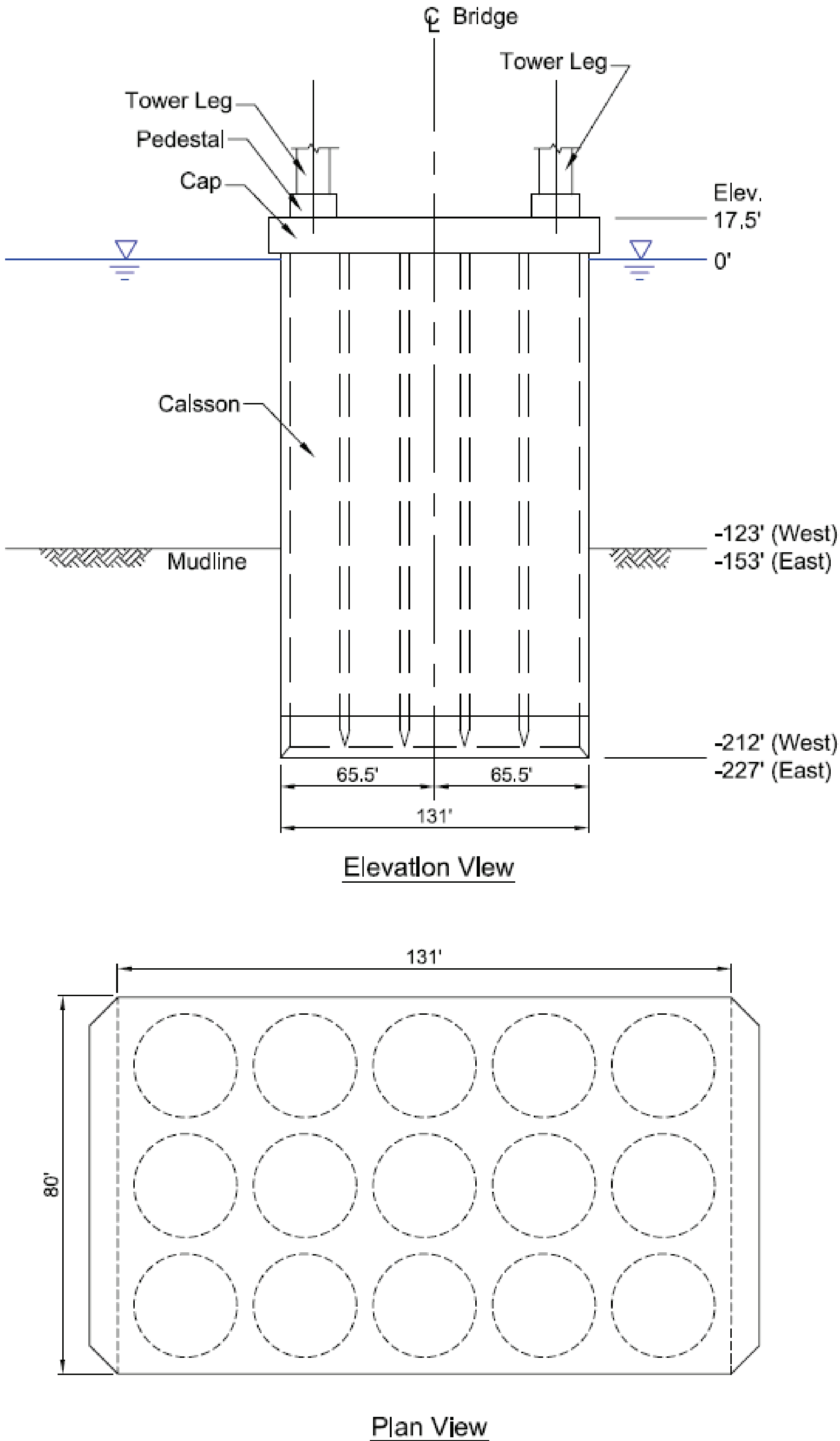


Figure 5.4 Caisson of Tower Foundation at Tacoma Narrow Bridge, WA

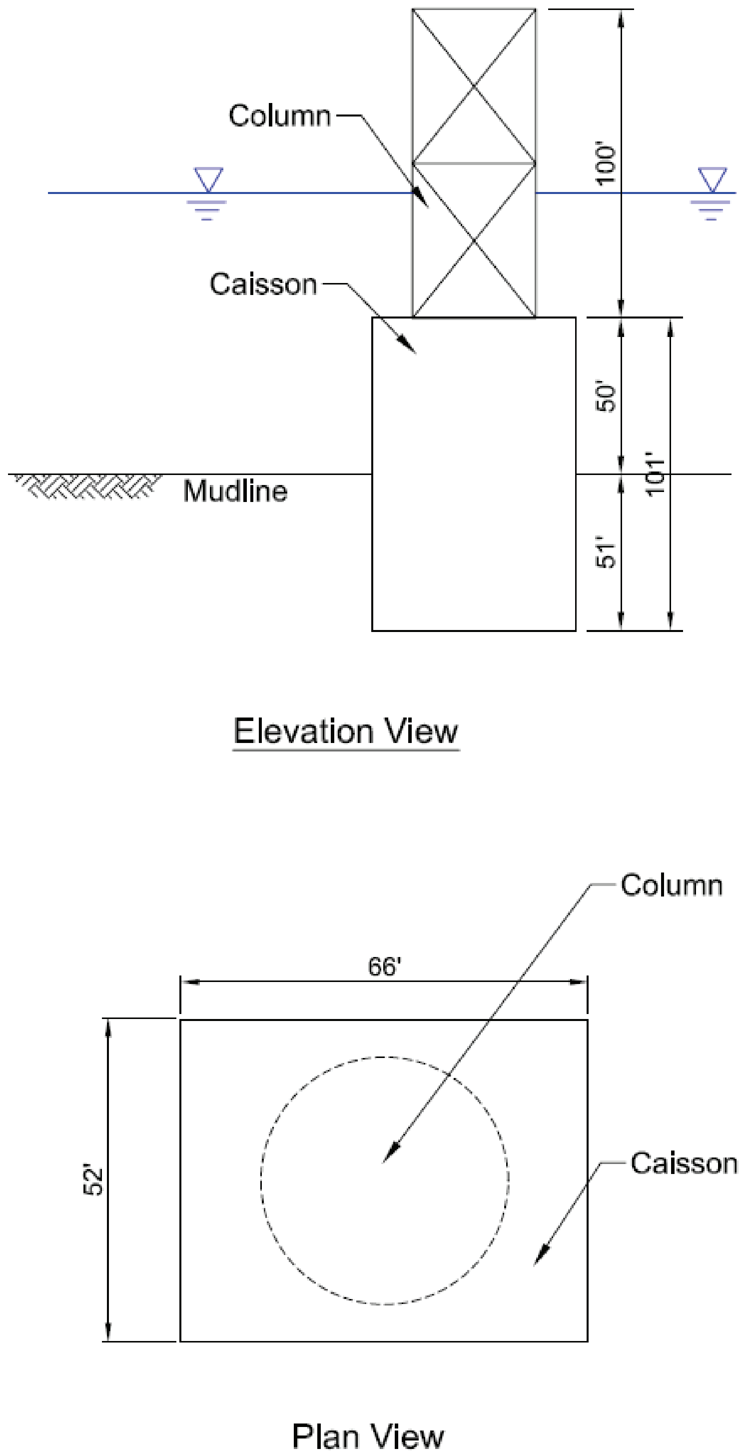


Figure 5.5 Caisson Foundation at George Coleman Bridge, VA

5.3 Caisson Modeling

Gravity caissons are massive and sometimes constitute a major portion of the weight of the entire structure. 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 exhibits a stiff system, the fundamental mode of vibration for most gravity caissons can be characterized as 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 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.

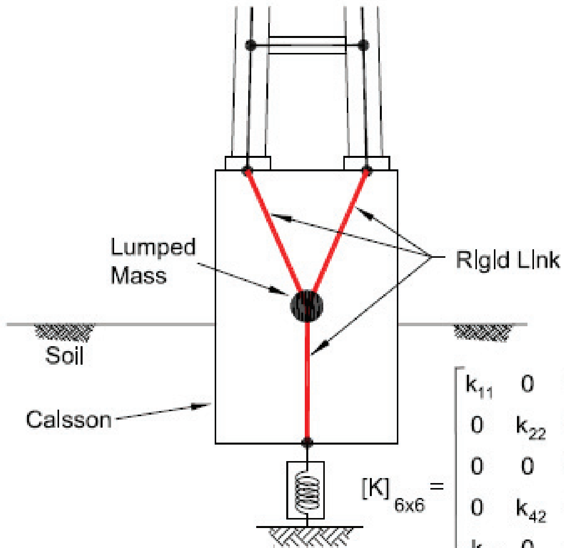
Several modeling approaches for the caisson have been in use for seismic design. Each approach has advantages and disadvantages, and engineers should recognize the shortcomings and use them according to the situations encountered. Some of the modeling approaches are briefly described below.

5.3.1 Linear Stiffness Matrix

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. 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 5.6 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 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:

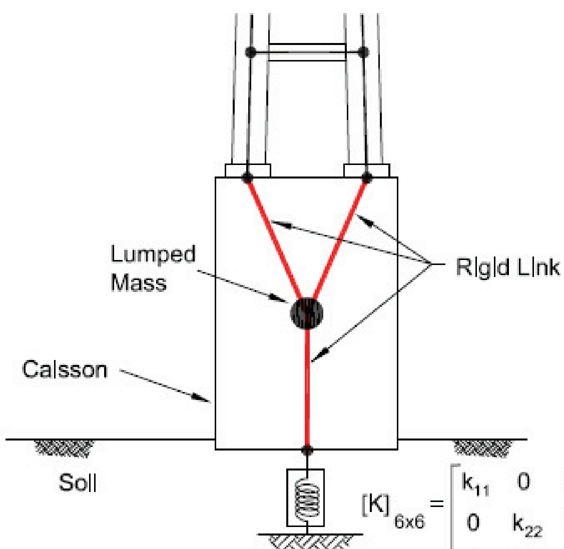
$$[K]=\begin{bmatrix} k_{11} & 0 & 0 & 0 & -k_{15} & 0 \\ 0 & k_{22} & 0 & k_{24} & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & k_{42} & 0 & k_{44} & 0 & 0 \\ -k_{51} & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix} \tag{5.1}$$

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



$$[K]_{6x6} = \begin{bmatrix} k_{11} & 0 & 0 & 0 & -k_{15} & 0 \\ 0 & k_{22} & 0 & k_{24} & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & k_{42} & 0 & k_{44} & 0 & 0 \\ -k_{51} & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix}$$

Embeded Caisson



$$[K]_{6x6} = \begin{bmatrix} k_{11} & 0 & 0 & 0 & 0 & 0 \\ 0 & k_{22} & 0 & 0 & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & k_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix}$$

Shallow Caisson

Figure 5.6 Lumped Mass with Linear Stiffness Matrix

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 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:

$$[K] = \begin{bmatrix} k_{11} & 0 & 0 & 0 & 0 & 0 \\ 0 & k_{22} & 0 & 0 & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & k_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix} \tag{5.2}$$

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.

5.3.2 Nonlinear Lumped Springs

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 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 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, by representing the foundation stiffness using nonlinear moment-versus-rotation and load-versus-displacement relationships, as illustrated in Figure 5.7. 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 deal with the relationship between shear and overturning moment on the caisson.

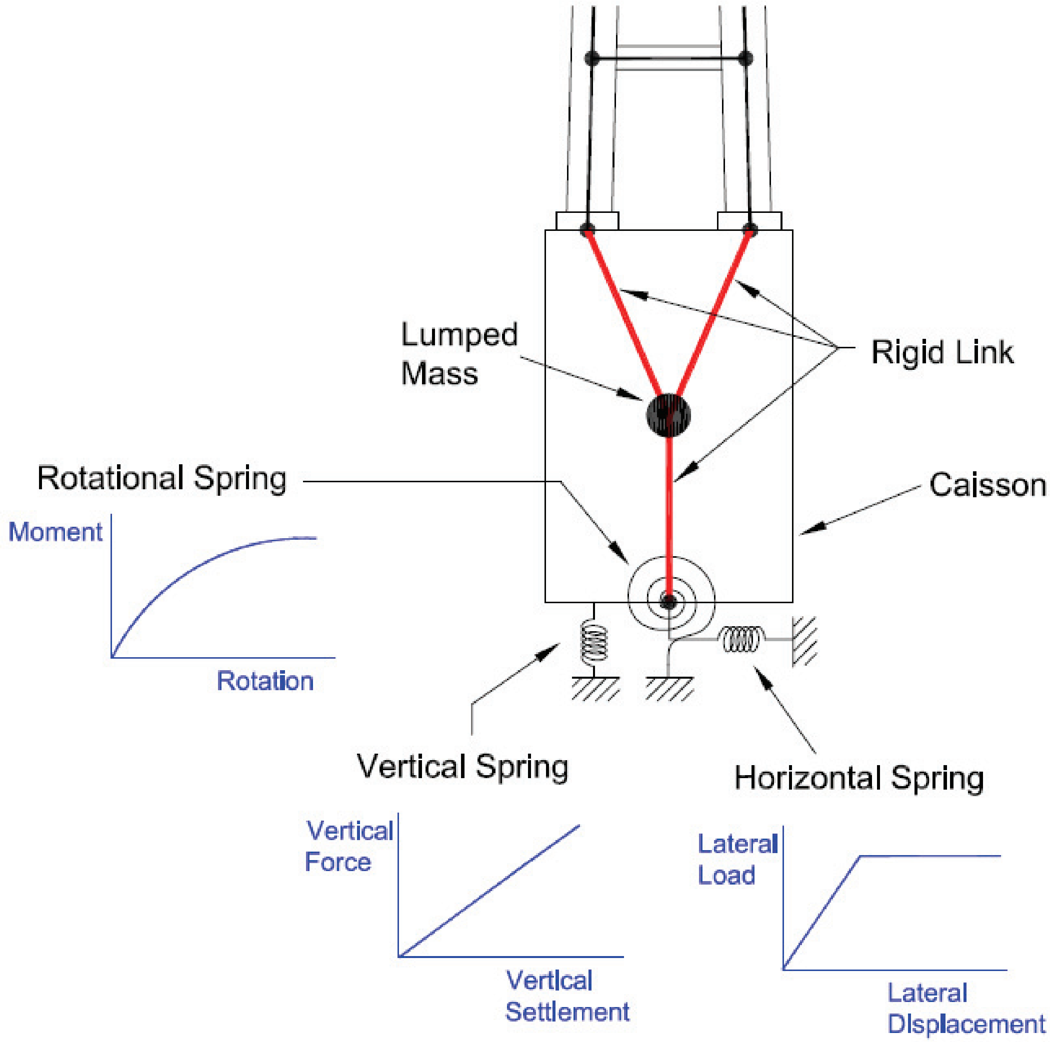


Figure 5.7 Lumped Mass with Nonlinear Uncoupled Springs

5.3.3 Nonlinear Distributed Soil Springs

This model entails Winkler springs distributed over the bottom surface of the caisson to represent the soil continuum underlying the foundation. Another set of soil springs is attached to the vertical sides of caisson walls to model passive soil pressure acting on the concrete. The soil springs may be nonlinear for consideration of 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 5.8 illustrates a distributed soil spring model used in the seismic analyses for a global bridge. This modeling approach would address the two significant features; nonlinear behavior and coupling between lateral loading and overturning moment, and hence exhibits significant improvements over the lumped spring models.

Establishment of the proper distributed soil springs is a key to successful modeling of 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 in extracting 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 correct soil springs are developed, this model characterized by distributed masses of the caisson which are supported by distributed soil springs at the caisson's exterior perimeters would effectively reflect 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 providing a mechanism to implement incoherent motions as a result of wave scattering. In the section below, the issue of wave scattering is discussed in relationship to the seismic loading of the caisson supported bridge structure.

5.4 Wave Scattering of Caisson

On the subject of wave scattering arising from surface features, several researchers have conducted studies on the effect of surface topography (such as ridges and canyons) subjected to vertically incident SH wave. Based on their findings, amplification as well as de-amplification of ground motion magnitude has been reported near these features due to wave scattering phenomena, and the amplification factor is frequency dependent. For example in the case of ridges, the effects are prominent when a wavelength is equal to the ridge width. Similarly for canyons, amplification or de-amplification is significant when wavelengths are similar to or smaller than the canyon dimensions.

Deep caissons embedded in soil would create inclusion of a rigid mass in the wave propagation media. The boundary conditions at which the caisson contacts with soil are different from those of canyons and ridges, which are stress free boundaries. Because of 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 would exhibit a different kind of amplification/de-amplification phenomena that is also frequency (wavelength) dependent. These wave scattering effects are not considered in the current design practice for caissons.

The framework to study the wave scattering for the caisson is based on numerical analyses using the ADINA program. Figure 5.9 shows the design data of a caisson and the soil conditions under consideration. Earthquake propagates both P- and S-waves, and upon incident at the caisson boundary, reflected and refracted wave of the both kinds

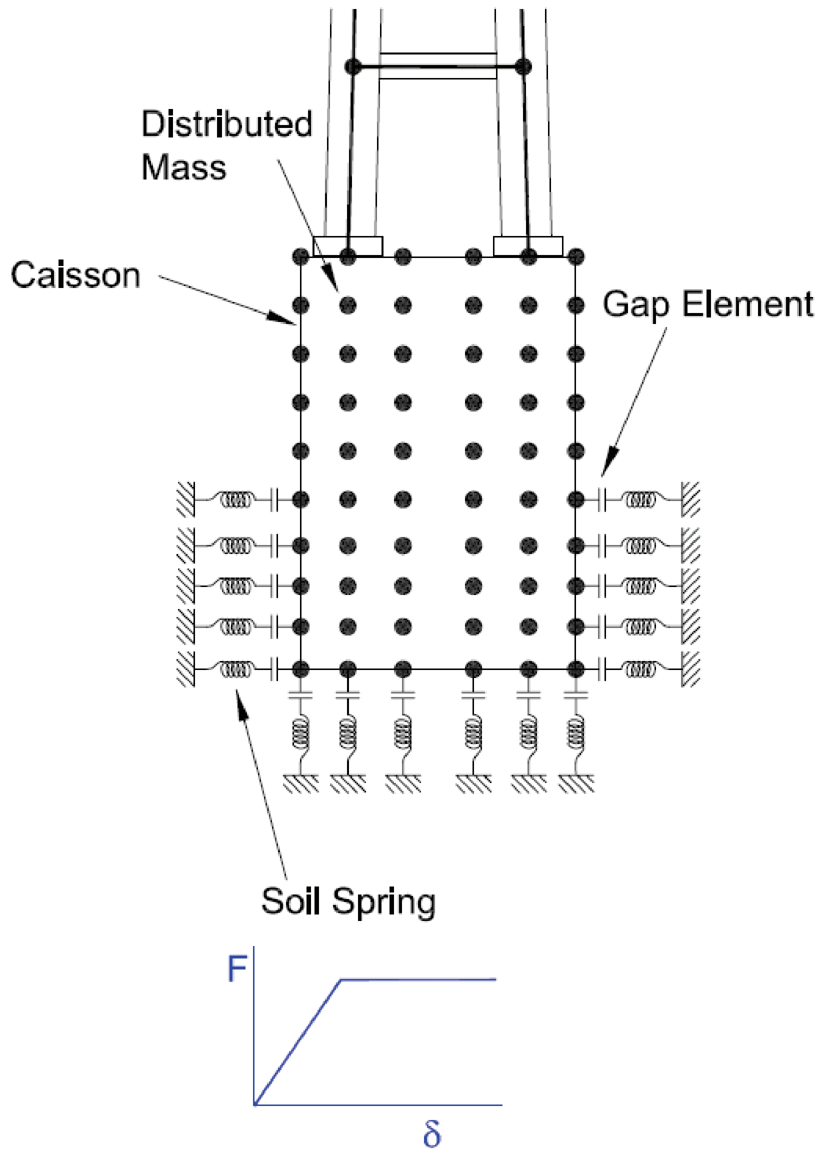


Figure 5.8 Distributed Mass and Soil Springs

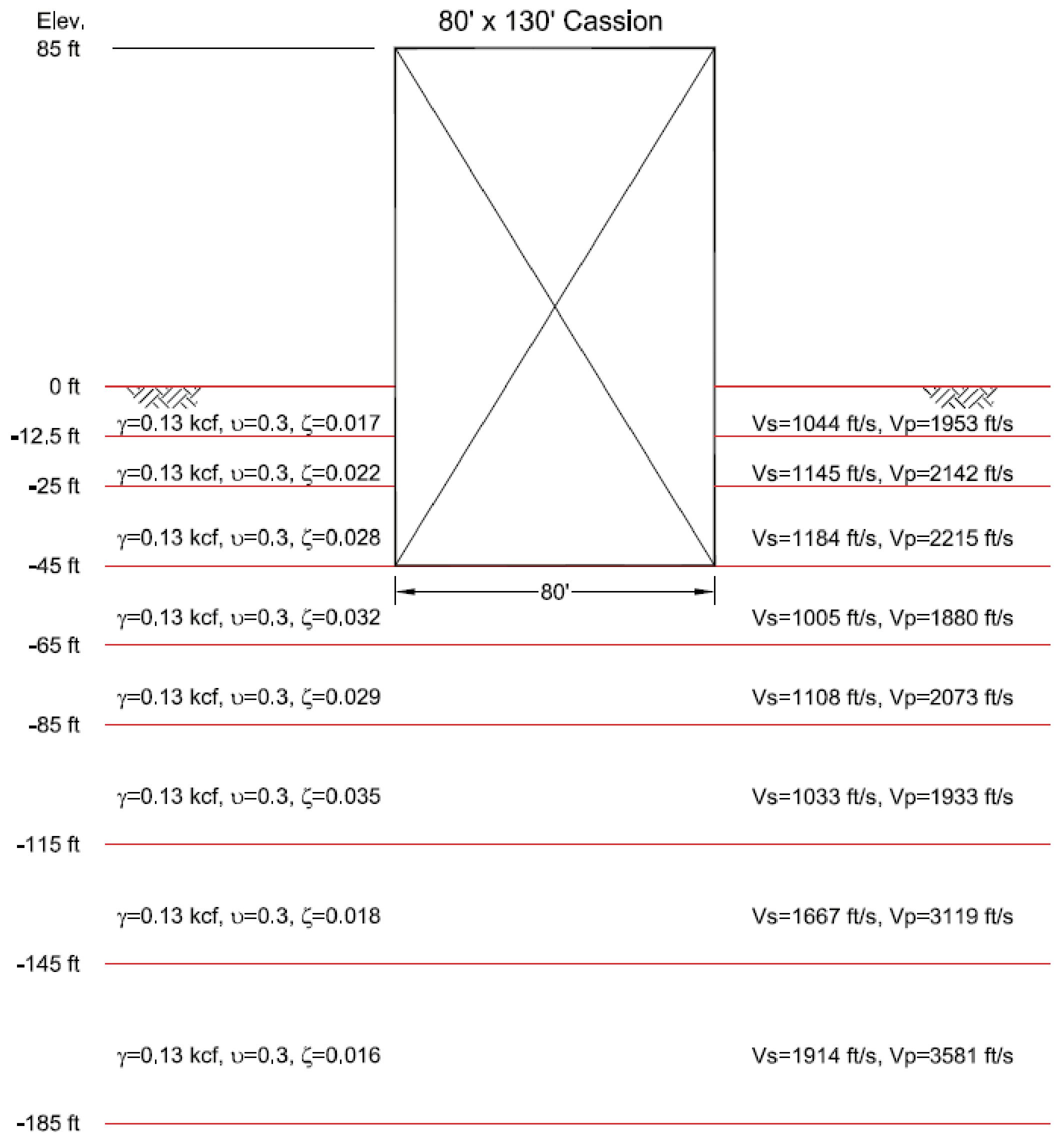


Figure 5.9 Caisson Dimensions and Soil Data

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 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 be 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 5.10 depicts the 1D site response study without consideration of 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 are conducted to compute the base excitation for each direction, which is then used to excite the finite element model. Figure 5.11 shows 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 only without inertia. The effect of caisson's inertia is included in the second step analysis of the global bridge model where the caisson mass will be treated as lumped mass or distributed mass.

The presented comparison results are 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 5.12; all include wave scattering effects. It appears that the larger the caisson, the more 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 is examined in a 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 5.13 presents the results of the largest caisson size (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 suggestion 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 between with scattering 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 the conventional seismic design of caissons without explicit considerations of wave scattering is reasonable.

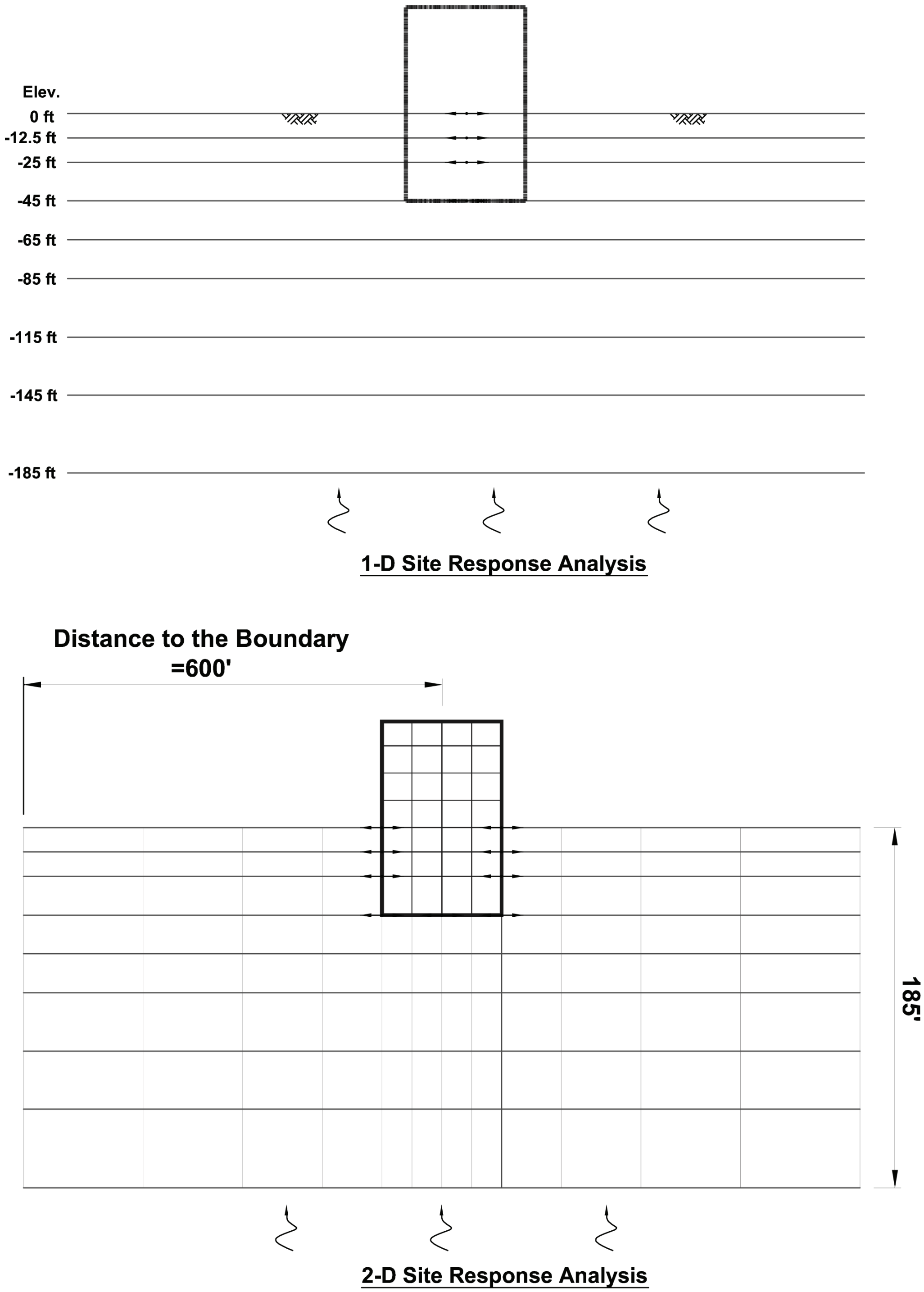


Figure 5.10 1D versus 2D Site Response Analysis

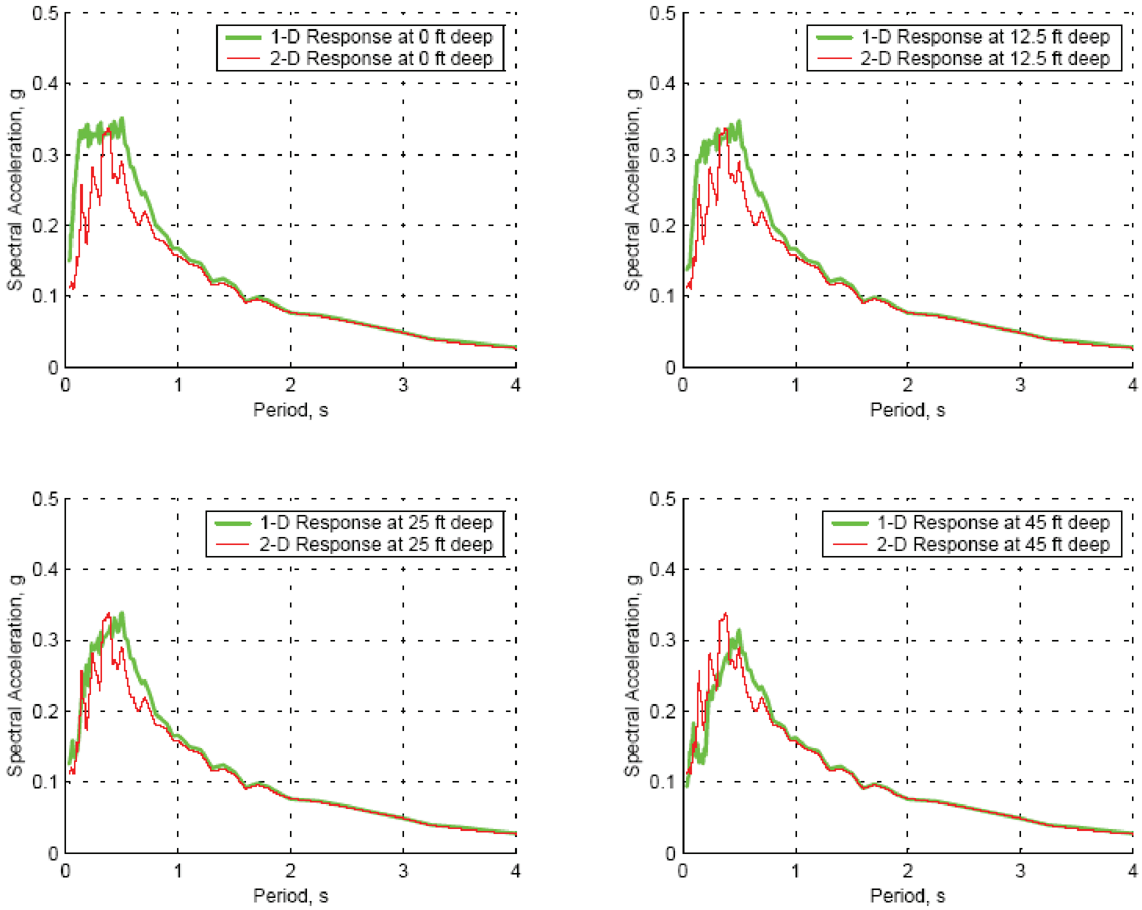


Figure 5.11 Comparison of 1D and 2D Site Response Solution

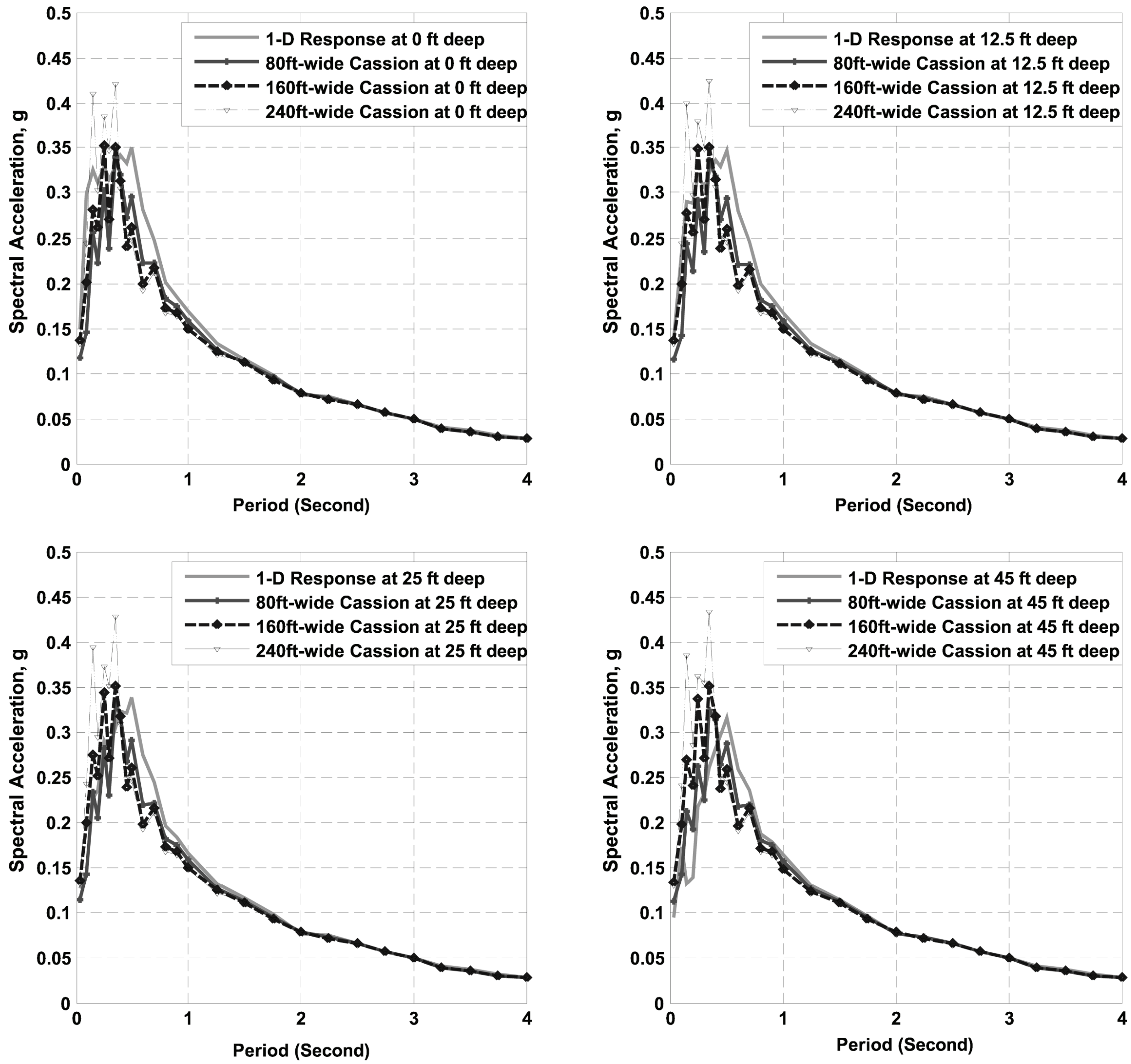


Figure 5.12 Near-Field Motions of Three Caisson Sizes

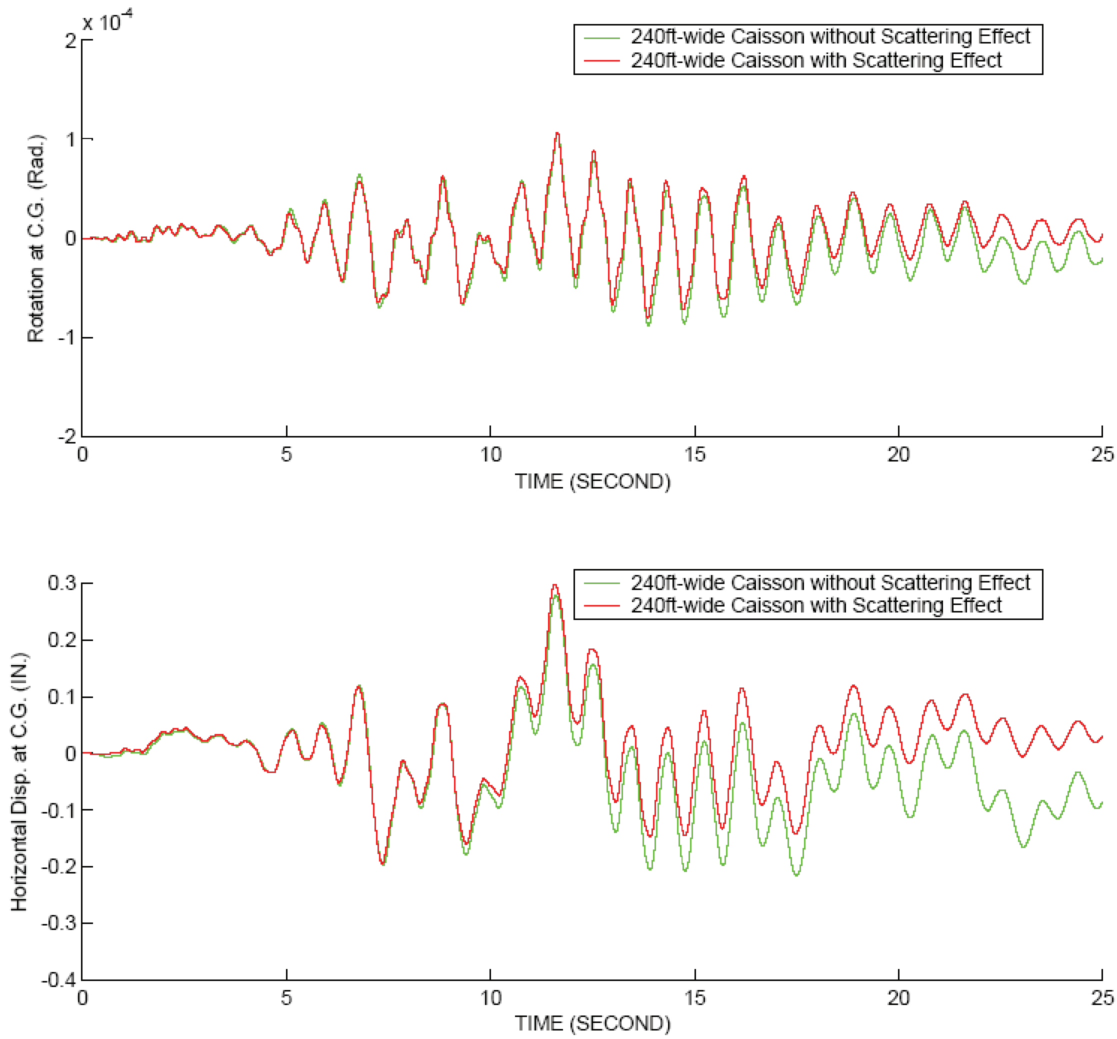


Figure 5.13 Seismic Response of Caisson with the Base Width of 240 feet

SECTION 6
CONCLUSIONS

This study demonstrates that a time domain approach can be used to study many wave scattering issues encountered in bridge engineering. However, treatment of boundary conditions must be exercised to minimize wave reflection and refraction. For dealing with the base boundary to include the elastic halfspace, the 'within motion' from SHAKE can be used that effectively consider potential wave passage across the model boundary. The side boundaries of finite element models may be simulated by slaving left and right edge boundaries in all directions. When there are situations where the heights and material properties at two edge boundaries are different, creating two edge columns by slaving the adjacent nodes offers an alternative method to mitigate superfluous motions due to wave reflection at the finite element side boundary.

In spite of different techniques used to implement damping in the frequency domain and time domain computer programs, the outcome of site response solutions are not affected by the discrepancy in modeling procedure. In other words, Raleigh damping does a good job in matching frequency independent damping used in frequency domain computer programs.

Wave scattering is not considered in the current state of practice for most bridge designs. It typically affects high frequency and the affected frequency range depends on propagation speeds of the foundation soil and physical dimensions of the pile group or the caisson. Wave scattering tends to produce incoherent motions when pile foundations or caissons are embedded in the ground; these non-synchronized motions of soil would sometimes result in canceling of the out-of-phase particle velocity. From the current research, the overall shaking to the bridge, when wave scattering is considered, tends to be lower than the seismic shaking without consideration of wave scattering. However, the effected shaking level is typically in a relatively high frequency range (low period). The period of interest for most long span bridges are generally well above 2 seconds. Therefore we consider the current design practice without consideration of wave scattering is still a valid procedure.

SECTION 7
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