EVALUATION OF SOIL-STRUCTURE INTERACTION IN BUILDINGS DURING EARTHQUAKES

by

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PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Directed Research Project. The objective of the this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong-motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Directed Research Project are to:

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.

2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.

3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.

4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong-motion data.

5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is the first in a series of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Directed Research Project.

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ABSTRACT

A large number of analytical studies of dynamic interaction between buildings and the supporting soil have been performed using a variety of sophisticated techniques. There has, however, been limited evaluation of soil–structure interaction from building response data recorded during moderate and strong earthquakes. This report presents an evaluation of soil–structure interaction effects in a multistory building from the response obtained by the California Strong Motion Instrumentation Program (CSMIP) in the 1 October 1987 Whittier earthquake.

The building, a fourteen story reinforced concrete warehouse, has been the subject of previous investigations of soil–structure interaction using data from several earthquakes. The building response in the 1987 Whittier earthquake is the strongest to date and it provides an excellent opportunity for investigation of soil–structure interaction effects.

A mathematical model of the complete building–foundation–soil system is developed to determine response quantities not directly available from the records and to ascertain the effects of interaction. The model is calibrated using the dynamic properties of the building as determined from the processed records. The model is then used to evaluate the effects of soil–structure interaction on the maximum base shear force, overturning moment and displacement for the building in the 1987 Whittier earthquake. The analysis demonstrates that soil–structure interaction has a significant effect on the base forces and roof displacement in the longitudinal direction of the building compared to the typical assumption in which interaction would be neglected. Soil–structure has less effect on the transverse response of the building. The effects of soil–structure interaction for this building and earthquake is approximately represented by proposed building code provisions.

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Chapter 1

INTRODUCTION

For a structure founded on rock or very stiff soil, the motion at the base is similar to the free–field ground motion, the motion that would occur at the site if the structure was not present. For structures on softer soils, the dynamic interaction between the structure and soil can have a significant effect on the earthquake response. Soil–structure interaction modifies the motion at the base compared to the free–field ground motion, and hence changes the response of the structure. Depending on the vibration properties of the structure, properties of the soil, and characteristics of the ground motion, the neglect of soil–structure interaction can provide misleading displacements and forces in the structure. The realistic earthquake response analysis and design of many structures requires consideration of the soil and dynamic interaction effects.

1.1 Objectives of the Study

In the past fifteen years there have been a large number of analytical investigations of soil–structure interaction in building response to earthquakes. There has not been a similar amount of study of building–soil systems using response data recorded during earthquakes. The lack of investigation is partly caused by the limited availability of strong motion data from buildings in which soil–structure interaction is important.

The 1 October 1987 Whittier earthquake was located in the Los Angeles metropolitan area, one of the most instrumented regions in the world. The earthquake generated the largest set of strong ground motion records ever obtained from one event. The California Strong Motion Instrumentation Program (CSMIP) collected and processed data from several building stations that included the motion at a nearby ground location. The ground instruments indicate the free–field ground motion at the building site during the earthquake.

The objective of this investigation is to analyze the building and free–field records obtained in the 1987 Whittier earthquake to elucidate the effects of soil–structure interaction in buildings. Particular emphasis is placed on determining the effect of interaction on the maximum base shear force, overturning moment, and displacement of buildings. The effects determined from the earthquake records are compared with proposed building code provisions for including soil–structure interaction in earthquake resistant design.

1.2 Summary of Soil–Structure Interaction Effects

Ground motion is caused by seismic waves propagating from the earthquake source through the soil layers and reaching the surface near the site. For many sites the motion at the ground surface is substantially different than the motion at bedrock or nearby rock outcrops because of the dynamic characteristics of the soil. The modification of the ground motion by the soil is called the site response effect and it occurs whether or not structures are present at the ground (or excavated) surface. This study is aimed at the evaluation of soil–structure interaction given the ground motion at the site. The site response effects are not considered.

When considering coupling between a building and the soil, the effects of interaction can be conveniently divided into two types (Wolf, 1985). Kinematic interaction affects the motion of the foundation and structure with the mass set to zero. For a system with a rigid foundation, kinematic interaction gives the motion of the massless foundation only. Kinematic interaction effects are produced by the reflection and scattering of incident seismic waves from the foundation. The scattering of waves modifies the foundation motion compared to the motion that would occur at the surface if the foundation was not present. The importance of kinematic interaction depends on the type, frequency, and angle of incidence of the seismic waves, and the embedment and flexibility of the foundation. Kinematic interaction is generally significant for high frequency components because the wavelengths are comparable to the dimensions of the foundation.

The second category of soil–structure interaction involves the motion caused by the foundation and structure mass. This type of interaction is called inertial interaction because it depends on the mass of the structure, foundation, and soil. These loads develop interaction forces at the foundation which deform the soil, further modifying the motion at the base, and hence altering the response of the structure.

A rigorous analysis of a soil–structure system should include site response effects and kinematic and inertial interaction. In practice, the site response effects are considered separately from the dynamic analysis of a building–soil system by the use of site dependent ground motion or spectra for the free–field ground motion. It is common to simplify the analysis of building response to free–field ground motion by neglecting either kinematic or inertial interaction depending on the properties of the building, foundation, and soil.

1.3 Organization of Report

A summary of the response of three CSMIP instrumented buildings with nearby ground stations to the 1 October 1987 Whittier earthquake is presented in Chapter 2. One building, a fourteen story warehouse, was 25 km from the epicenter and it experienced moderate amplitude response. The Warehouse Building is selected for further, detailed study of soil–structure interaction effects.

Previous investigations of the Warehouse Building using the response from earlier earthquakes are described in Chapter 3. The response to previous earthquakes is compared to the response in the 1987 Whittier earthquake to illustrate characteristics of soil–structure interaction in buildings.

To determine response quantities not directly available from the processed records, particularly the base shear force, a mathematical model of the building–foundation–soil system is developed in Chapter 4. The vibration properties of the superstructure are determined from a three–dimensional mathematical model. The model of the soil–structure system is based on the substructure approach.

In Chapter 5 the transfer functions for the model of the building–foundation–soil system are compared to the transfer functions from the response recorded in the Whittier earthquake. The parameters of the model are selected to calibrate the response of the model with the recorded response. After selecting damping properties for the building and soil, a response history analysis is performed to determine the effects of soil–structure interaction on the maximum base forces and roof displacement for the building.

In Chapter 6, the results of the analysis are compared to proposed building code provisions for soil–structure interaction. Conclusions of the study and recommendations for instrumenting and studying soil–structure interaction in buildings are presented in Chapter 7.

Chapter 2

SOIL–STRUCTURE EFFECTS ON BUILDINGS IN THE WHITTIER EARTHQUAKE

2.1 Introduction

The 1 October 1987 Whittier earthquake occurred in the east Los Angeles, California, area. The main event had a magnitude of $M_L = 5.9$ and was caused by a rupture along a previously unmapped thrust fault located just north of the Whittier Narrows at a focal depth of 11 to 16 km (Hauksson, *et al*, 1988). Several aftershocks followed the main event. The largest magnitude aftershock ($M_L = 5.3$) occurred on 4 October 1987.

The main event was the strongest earthquake in southern California since the 1971 San Fernando earthquake. It generated the largest set of strong ground motion records ever obtained from one event. The Strong Motion Instrumentation Program (CSMIP) of the California Division of Mines and Geology collected acceleration records from 101 stations, including 63 ground stations and 27 building stations (Shakal, *et al*, 1987). Each building station had an average of twelve instruments distributed in plan and elevation through the building.

Eight building stations had instruments at the ground surface near the building in addition to the instrumentation for the structure. The records obtained from these stations provide an opportunity to assess the effect of the foundation and soil on the earthquake response of the buildings. The motion recorded at a ground station close to a building can be representative of the free–field ground motion, the motion that would occur at the site if the building was not present. The difference between the recorded free–field motion and the recorded base motion is a measure of soil–structure interaction effects.

This chapter describes the response of three buildings with nearby ground motion records from the 1987 Whittier earthquake. One building is identified for detailed examination of soil–structure interaction effects.

2.2 Response of Buildings with Free–Field Records

Three buildings with nearby ground motion instruments experienced small to moderate amplitude motion in the region closest to the epicenter of the 1 October 1987 Whittier earthquake, Area 1 as defined by CSMIP (Shakal, *et al*, 1987). Although CSMIP obtained free–field acceleration records for five other instrumented buildings, the response amplitudes were not large and thus they were not selected for this study.

The three buildings, identified by the CSMIP designations, are:

- Los Angeles Warehouse Building (SN 236) is a fourteen story reinforced concrete warehouse structure. In addition to the main event, records were obtained for the aftershock of 4 October 1987.

- Long Beach Office Building (SN 323) is a seven story steel structure with concrete floor slabs. The building consists of two sections connected by a seismic joint and it has a pile foundation on hydraulic fill.

- Sylmar Hospital (SN 514) is a six story combined reinforced concrete and structural steel building. The two lower stories of the building have a rectangular plan and the upper four stories are cruciform in plan.

The epicentral distance and maximum acceleration at the free–field ground station, base, and roof level for each building are given in Table 2.1.

TABLE 2.1
Maximum Acceleration of Buildings with Free–Field Instruments in the 1 October 1987 Whittier Earthquake[1]

Building	Epicentral Distance (km)	Maximum Acceleration (g) Free–Field	Base	Roof
Los Angeles Warehouse Building	25	0.20	0.11	0.20
Long Beach Office Building	36	0.070	0.073	0.11
Sylmar Hospital	45	0.051	0.057	0.15

[1]From instrument corrected and bandpass filtered records (CSMIP, 1989)

Although maximum acceleration is not an ideal parameter for characterizing earthquake ground motion and building response, the maximum acceleration at the base of the Los Angeles Warehouse Building is about one–half of the maximum free–field acceleration. This is a significant difference in response over a distance of approximately 250 ft. In comparison, the maximum free–field and base acceleration are very close for both the Long Beach Office Building and Sylmar Hospital. From the maximum acceleration values in Table 2.1 it can be expected that soil–structure interaction effects are more important in the Warehouse Building than the other two buildings.

Additional reasons for concentrating the study on the Los Angeles Warehouse Building compared to the other two buildings are:

- The low amplitude responses of the Long Beach Office Building and Sylmar Hospital are not indicative of the response of buildings close to the source of moderate and strong earthquakes.

- The dynamic response of soil is dependent on the strain level and hence amplitude of motion. The effects of soil–structure interaction generally increase with response because of the reduced modulus and increased material damping for soil.

- The seismic joint and pile foundation in the Long Beach Office Building and the irregular configuration of the Sylmar Hospital increases the complexity of the modeling and analysis for soil–structure interaction.

- Little or no data on soil properties at the sites of the Long Beach Office Building and Sylmar Hospital are readily available making it difficult to estimate the effects of soil–structure interaction.

- Examination of the response spectra for the free–field and base motion of the two buildings indicates that soil–structure interaction effects were not particularly important in the earthquake.

2.3 Preliminary Evaluation of the Los Angeles Warehouse Building

The Los Angeles Warehouse Building provides an opportunity to study the earthquake response of a building–foundation–soil system because of the moderate amplitude of the response, regular configuration of the building, and availability of data for the soil at the site. In addition, the response of the Warehouse Building to previous earthquakes has been studied extensively.

The location of the accelerometers installed and maintained by CSMIP in the Warehouse Building is shown in Figure 2.1. There are twelve accelerometers in the building and three accelerometers located in a small shelter 139 ft west of the building in the parking lot. The instrumentation provides the horizontal translational response of the building at four levels. The torsional response can be determined at the basement, 8th floor, and roof assuming the diaphragms have a large in–plane stiffness. There is only one vertical instrument in the building, in the basement, so it is not possible to determine the rocking of the building about a horizontal axis.

The parking lot instrument was located to give an indication of the free–field ground motion, the motion that would occur at the site if the building was not present. It should be noted, however, that the parking lot instrument is less than one foundation length away from the building in the longitudinal direction. It is very likely that the ground motion recorded by the parking lot instrument is affected by the motion of the building. The interpretation of the response in consideration of coupling between the parking lot and building instruments will be discussed further.

Table 2.2 summarizes the maximum horizontal acceleration of the Warehouse Building for the main event and aftershock. There is a significant reduction in the acceleration at the base compared to the parking lot along both axes of the building. The average reduction in acceleration at the base is 46% for the main event and 30% for the aftershock. The reduction is primarily due to soil–structure interaction and it is related to the amplitude of response because of the nonlinear properties of the soil. The larger the amplitude of response, the more soil–structure interaction is important and the larger the difference between the free–field motion, represented by the parking lot record, and motion at the base of the building.

The acceleration history at the parking lot, base, and roof in the transverse and longitudinal directions for the main event are shown in Figures 2.2 and 2.3 (CSMIP, 1989). There are substantial differences between the acceleration history at the parking lot and base. The high frequency components of the response in the parking lot records are absent from the base records, and the maximum acceleration is substantially less. In the longitudinal direction, the base motion has a frequency content with a component similar to the predominant frequency of the roof response that is not apparent in the parking lot record, particularly during the cycles with large acceleration peaks (Figure 2.3). The base response is characteristic of soil–structure interaction because the building tends to drive the base near the fundamental frequency of the complete system.

The difference in acceleration at the parking lot and base of the Warehouse Building is due to interaction effects between the building and soil. This response data provides a unique opportunity to study soil–structure interaction effects in a building in a moderate earthquake. The remainder of this report presents a study of the interaction effects in the Warehouse building in the 1 October 1987 Whittier earthquake.

TABLE 2.2
Maximum Acceleration of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake and 4 October 1987 Aftershock[1]

		Maximum Acceleration (g)		
Earthquake Event	Building Direction	Parking Lot	Base	Roof
Main Event	transverse	0.20	0.11	0.20
Main Event	longitudinal	0.11	0.058	0.19
Aftershock	transverse	0.086	0.063	0.11
Aftershock	longitudinal	0.052	0.034	0.081

[1]From instrument corrected and bandpass filtered records (CSMIP, 1989)

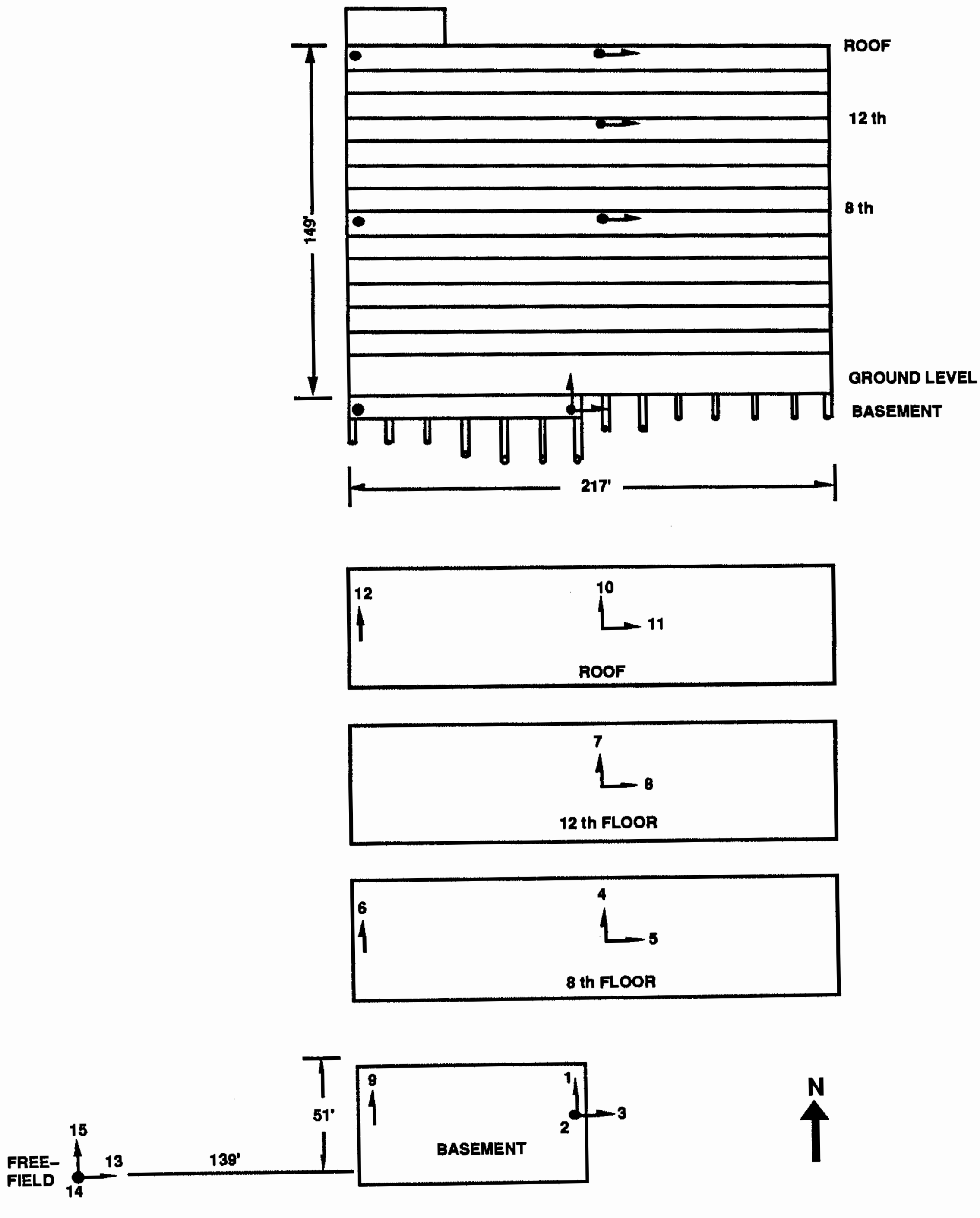

Figure 2.1 Location of Accelerometers in the Los Angeles Warehouse Building (Shakal, *et al*, 1987)

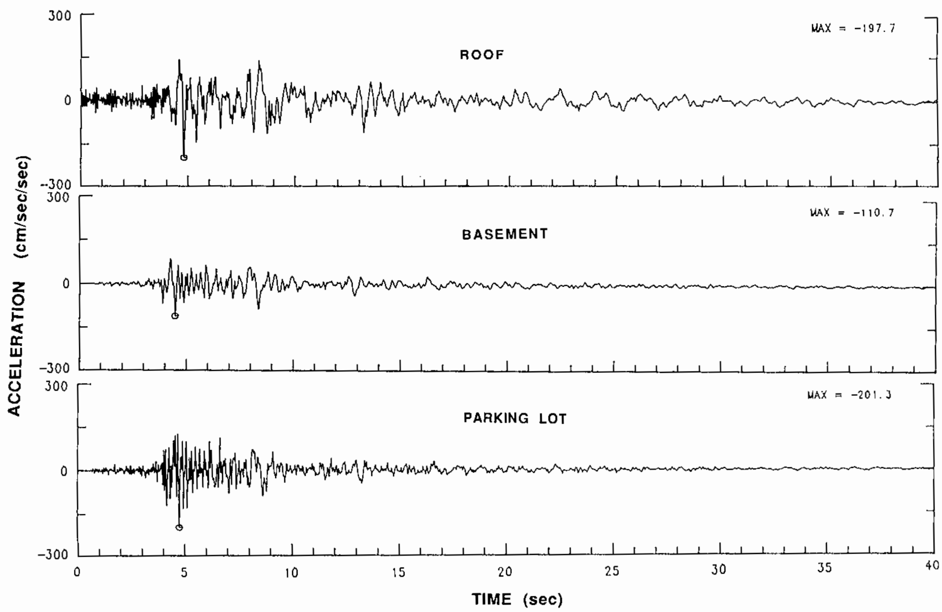

Figure 2.2 Free–Field, Base, and Roof Acceleration Response in the Transverse Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake (Channels 15, 1, and 10)

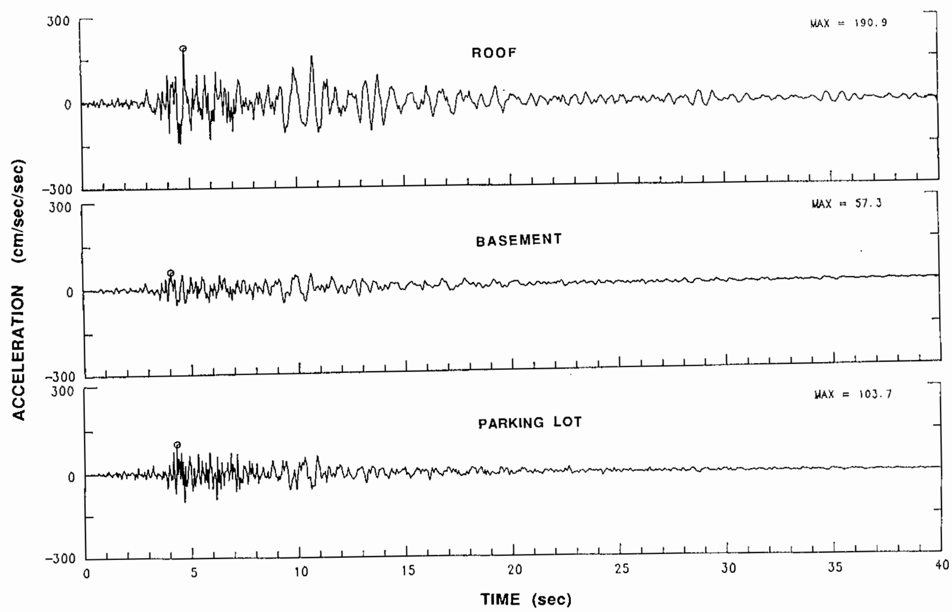

Figure 2.3 Free–Field, Base, and Roof Acceleration Response in the Longitudinal Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake (Channels 13, 3, and 11)

Chapter 3

EARTHQUAKE RESPONSE OF THE LOS ANGELES WAREHOUSE BUILDING

3.1 Introduction

The Los Angeles Warehouse Building is located in the Hollywood area of Los Angeles, California. The building has been the subject of investigations using response data recorded from earthquakes in 1933, 1952 (Kern County), and 1971 (San Fernando). The location of the building in relation to major earthquakes in the past fifty years is shown in Figure 3.1. The building is suited for the study of soil–structure interaction because of the available building and ground motion records for several earthquakes, the building's regular configuration, and its isolation from other tall buildings.

The Warehouse Building is described in this chapter, and the conclusions from previous investigations of soil–structure interaction are cited. The response of the building to earlier earthquakes and the 1987 Whittier earthquake are presented to illustrate important characteristics of soil–structure interaction.

3.2 Description of the Warehouse Building and Site

The geometry and structural configuration of the Warehouse Building, which was designed and constructed in 1925, is shown in Figure 3.2. It is a fourteen–story, 149 ft tall structure with a rectangular cross section 217 ft in the longitudinal (EW) direction and 51 ft in the transverse (NS) direction. The vertical load resisting system consists of 8 inch thick concrete slabs supported by columns with capitals. In the three longitudinal bays on the west side of the building, one–way slabs on joists are supported by the transverse frames. The lateral force resisting system consists of reinforced concrete frames in both directions. The two exterior longitudinal frames and the transverse westward frame are infilled with 8 inch thick panels. Two penthouses are located at the roof level and radio antennas installed above the penthouses during the construction were removed in 1954.

A basement under the west half of the building is embedded 9 ft below the ground level. The foundation consists of concrete piles that vary in length from 12 ft at the edge of the building to 30 ft near the building center.

Data about the soil at the site of the Warehouse Building have been obtained from a report by Duke and Leeds (1962). As shown in Figure 3.3, a boring revealed that the building is founded on an approximately 200 ft deep layer of sandy clay with the unit weight varying from 110 lb/ft^3 at the surface to 130 lb/ft^3 at the bottom of the layer. The measured P–wave velocity is 2400 ft/sec within the layer, except in the superficial shallow stratum of clay loam where it is 1090 ft/sec. The sandy clay layer is underlaid by approximately 2000 ft of Pleistocene (Quaternary) and Tertiary sedimentary formations, which in turn rest on Santa Monica slate. The sedimentary formations consist mainly of sand and gravels probably deposited as fan material by streams originating in the south slope of the Santa Monica mountains.

3.3 Summary of Previous Studies

From 1933 to 1986 four earthquakes in the Los Angeles area triggered the strong motion accelerometers installed in the Warehouse Building. Figure 3.1 shows the location of the building with respect to the epicenters of the earthquakes. Table 3.1 summarizes the earthquakes and the maximum acceleration of the building and the ground at the parking lot instrument.

TABLE 3.1
Earthquakes Recorded at the Los Angeles Warehouse Building[1]

Earthquake	M_L	Epicentral Distance (km)	Maximum Acceleration (g)					
			Parking Lot		Base		Roof	
			tran.	long.	tran.	long.	tran.	long.
Southern Calif. 2 Oct 1933	5.4	39	—[2]	—[2]	0.03	0.03	0.04	0.09
Kern County 21 July 1952	7.2	122	0.06	0.04	0.06	0.04	0.12	0.15
San Fernando 9 Feb 1971	6.4	35	0.17	0.21	0.11	0.15	—[3]	—[3]
Whittier Narrows 1 Oct 1987	5.9	25	0.20	0.11	0.11	0.06	0.20	0.19
Whittier Aftershock 4 Oct 1987	5.3	25	0.09	0.05	0.06	0.03	0.11	0.08

[1]Borrego Mountain earthquake of 8 April 1968 is not included.
[2]Parking lot instrument not installed.
[3]Roof instrument did not trigger.

The southern California earthquake of 2 October 1933 triggered two U.S. Coast and Geodetic Survey triaxial accelerometers at the basement and roof. The maximum acceleration for the 1933 event was 0.03 g at the basement and 0.09 g at the roof. At that time no information about the free–field ground motion was obtained.

In subsequent years a third triaxial accelerometer was installed in the parking lot, 139 ft west of the Warehouse Building (Figure 2.1), protected by a small shelter. The first set of ground and building acceleration records at the Warehouse Building was obtained during the Kern County earthquake of 21 July 1952. Because of the large epicentral distance, the maximum ground acceleration at the Warehouse Building was only 0.06 g in the transverse direction. An early comparison of the pseudo–velocity response spectrum of the parking lot record and the spectrum for the basement record in the longitudinal direction of the building showed differences that were attributed to soil–structure interaction (Housner, 1957). An attenuation of high frequency components was observed in the basement records when compared to the parking lot for periods shorter than 0.20 sec in the transverse direction and 0.60 sec in the longitudinal direction. This low–pass filtering of ground motion by the building basement is a consequence of kinematic interaction first identified by Housner (1957).

The response of the building in the 1952 Kern County earthquake was analyzed in the frequency domain using the Fourier spectra of the records (Duke, *et al*, 1970). The transfer functions between two instrument locations were obtained by dividing the spectral values of the records. The theoretical transfer function between the basement and parking lot in the longitudinal direction was based on an infinitely long elastic shear wall on a foundation excited by a plane horizontal shear wave travelling in the vertical direction (Luco, 1969). The model foundation was a rigid semicircular section embedded in an elastic soil. There was agreement between the theoretical transfer function and the one computed from the response records. The agreement between the two transfer functions was particularly good for excitation periods less than 0.20 sec, and both indicated a reduction of the high frequency response because of kinematic interaction. An analogous comparison with a shear wall model on a surface foundation gave less satisfactory agreement between the theoretical and observed transfer functions (Hradilek and Luco, 1970). The two studies indicate that the partial basement and concrete piles respond as an embedded foundation particularly for excitation periods less than 0.20 sec. The conclusion was that foundation embedment affects kinematic interaction in the building.

The epicenter of the 9 February 1971 San Fernando earthquake was 35 km from the Warehouse Building. The maximum ground acceleration at the parking lot and basement was 0.21 g and 0.15 g, respectively, in the longitudinal direction. No record was obtained for the roof because the instrument malfunctioned. Crouse and Jennings (1975) analyzed the response of the Warehouse Building in the 1952 Kern County earthquake and the 1971 San Fernando earthquake. The transfer functions from the recorded response were compared with theoretical transfer functions based on a fourteen story linear spring–mass model supported on a rigid, circular foundation on an elastic halfspace. Only partial agreement was obtained between the theoretical and observed transfer functions. The authors attributed the differences to the approximations involved in modeling the foundation and errors in considering the parking lot record as the free–field ground motion.

Newmark, Hall and Morgan (1977) proposed a simple method to model the low–pass filter effect of a building foundation due to kinematic interaction. The free–field acceleration record is averaged over a time interval, τ, corresponding to the time required for incident seismic waves to traverse the foundation. A modified response spectrum is calculated using the average acceleration history. A time interval of τ=0.08 sec gives reasonable agreement between the modified response spectra and the observed response spectra at the basement of the Warehouse Building for the 1952 Kern County and 1971 San Fernando earthquakes. The time interval corresponds to a horizontal seismic wave velocity of 1310 ft/sec using 105 ft as the geometric mean dimension of the foundation. The τ–averaging procedure was also used to compute the horizontal basement motion from the parking lot record accounting for translation and torsion (Whitley, *et al*, 1977). While there was fair agreement between the average motion and recorded basement motion for the 1952 Kern County earthquake, the difference was significant for the 1971 San Fernando earthquake. The authors attributed the difference to foundation flexibility and soil–structure interaction effects not accounted for by the averaging procedure. They also concluded that no significant torsional motion of basement occurred in the earthquakes.

A numerical low–pass filter can model the attenuation of high excitation frequency components of the free–field motion by kinematic interaction (Shioya and Yamahara, 1980). The filter predicts with fair accuracy the basement motion of the Warehouse Building from the parking lot in the 1971 San Fernando earthquake. A more refined numerical filter gives a better prediction of the basement motion for high frequency components (Ishii, Itoh, and Suhara, 1984).

In addition to the analytical study of earthquake response of the Warehouse Building, the vibration modes and periods were first measured in August 1934 from ambient response (Carder and Jacobsen, 1936). The fundamental period in the transverse direction was 1.2 sec, and the period in the longitudinal direction was 0.49 sec. Forced vibration tests were performed in 1938 (Carder, 1964). The periods from the forced vibration tests, listed in Table 3.2, coincide with the ambient vibration observations.

TABLE 3.2
Vibration Periods of the Los Angeles Warehouse Building (Carder, 1964)

Vibration Mode	Translational Transverse (sec)	Translational Longitudinal (sec)	Torsional (sec)
First	1.20	0.50	0.60–0.64
Second	0.37	—	0.17
Third	0.22	—	0.11

3.4 Response in the Kern County and San Fernando Earthquakes

A comparison of the motion at the parking lot with the motion at the basement of the Warehouse Building in the 1952 Kern County and 1971 San Fernando earthquakes provides information about the effects of soil–structure interaction.

Figures 3.4 and 3.5 show the absolute acceleration response spectra for the parking lot and the basement records using data from the California Institute of Technology (Hudson, Trifunac, and Brady, 1970 to 1975). The spectra have also been presented in Chang, *et al.* (1986). The spectra give the maximum absolute (or total) acceleration of a single degree–of–freedom oscillator with a specified period and 5% viscous damping ratio to the recorded motion. The spectrum ordinate for the period of the oscillator multiplied by the oscillator mass is approximately equal to the maximum restoring force. For a lightly damped building, the spectral ordinate multiplied by the generalized mass in the fundamental vibration mode gives an estimate of base shear force for the building.

In the earthquake analysis of buildings neglecting soil–structure interaction, the seismic input is assumed to be the free–field ground motion. The motion at the base of the building, however, is the input motion that a building actually experiences. If interaction between the building and soil are significant, there may be large differences between the free–field and base motions. For this reason, differences between the response spectra for the parking lot and basement of the Warehouse Building suggest modification of the input motion due to soil–structure interaction, and hence modification of the maximum base shear force developed in the earthquake.

The response spectra for the parking lot and the basement records for the 1952 Kern County earthquake (Figures 3.4) are similar indicating that soil–structure interaction did not significantly modify the base motion compared to the free–field motion.

Consequently, the parking lot record gives a reasonable estimate of the seismic input to the Warehouse Building. This conclusion contradicts an earlier evaluation of soil–structure interaction for this earthquake (Housner, 1957). The use of different processed data may explain the different interpretations.

For the 1971 San Fernando earthquake, the response spectra show much larger differences in certain ranges of periods (Figures 3.5). The basement spectrum is considerably less than the parking lot spectrum for periods less than 0.30 sec in the transverse direction and 0.50 sec in the longitudinal direction. The reduction of base response in the short period range is caused by kinematic interaction, and the τ–averaging procedure discussed in the previous section approximately accounts for the change (Newmark, Hall, and Morgan, 1977). The difference between the spectra in the short period range can also be caused by the lack of coherence between the motion at the two stations spaced 139 ft apart.

The response spectra for parking lot and basement records in the transverse direction (Figure 3.5) are similar for periods greater than 0.5 sec. The fundamental period of the building in transverse direction is 1.7 to 1.9 sec, and the response spectra are very close in this period range. The similar response spectral ordinates indicate that soil–structure interaction does not affect the fundamental mode response of the building in the transverse direction.

The building is relatively stiff in the longitudinal direction because of the long dimension and infilled frames. The large stiffness develops large shear and overturning forces at the base which deform the soil. The deformation modifies the motion at the base compared to the motion in the free–field. The vibration of the base and soil dissipates energy through material damping and wave propagation, adding damping to the system. Generally, the added damping reduces the response at the base and the response of the building. An important reduction in the longitudinal response at the base of the Warehouse Building with respect to the parking lot occurs in the period range of 0.50 to 0.60 sec (Figure 3.5), which is near the fundamental period of the building in the longitudinal direction. The spectra indicates a 20–25% reduction in the absolute acceleration at the base compared to the parking lot because of inertial interaction. Consequently, the difference in input motion would be expected to reduce the maximum base shear by a similar amount.

3.5 Response in the 1987 Whittier Earthquake

The absolute acceleration response spectra for the parking lot and base records of the Warehouse Building in the 1 October 1987 Whittier earthquake are shown in Figures 3.6. They resemble the spectra for the 1971 San Fernando earthquake in Figure 3.5. Kinematic interaction attenuates the response at the base in both directions for periods approximately less than 0.30 sec. In the transverse direction, there is no reduction of the base response in the range of the fundamental period, 1.7 to 1.9 sec. In the longitudinal direction, there is a significant reduction in the spectral ordinate in the fundamental period range of 0.50 to 0.60 sec. Figure 3.6 shows a 20% to 30% reduction in the absolute acceleration of the base compared to the parking lot at the fundamental period of the building in the longitudinal direction.

Another view of the motion at the parking lot and base of the Warehouse Building is given by the transfer function obtained from the recorded data. The transfer function gives the amplitude of the acceleration at the base for a unit harmonic acceleration at the parking lot for each excitation period. Transfer function values less than unity indicate a reduction of the base motion compared to the parking lot. The absolute value (or modulus) of the acceleration transfer function between the parking lot and base, shown in Figure 3.7, is obtained by dividing the ordinate of the Fourier spectrum for the base acceleration record by the ordinate of the Fourier spectrum for the parking lot acceleration record for each excitation period.

In the transverse direction, the transfer function fluctuates about unity for a broad range of excitation periods indicating little change in the base motion compared to the parking lot (Figure 3.7). The exception is for excitation periods less than 0.50 sec were there is a reduction of the base motion because of kinematic interaction and lack of coherence. The transfer function in the short period range explains the reduction in maximum acceleration shown in Table 2.2 and the lack of high–frequency components in the acceleration history of the base (Figure 2.2). The reduction of short period response primarily affects the higher vibration modes of the structure. However, the response of the fundamental vibration mode, in which soil–structure interaction is most important, is not greatly affected.

The transfer function for motion in the longitudinal direction is more complicated than for the transverse direction (Figure 3.7). The large value of the transfer function near 3 sec is fictitious because it is beyond the period range used in filtering the processed response data. Kinematic interaction reduces the transfer function for excitation periods less than about 0.50 sec because of the foundation–averaging effect. The numerous, narrow band peaks in the short period range are a consequence of low amplitude components in the parking lot record and are not indicative of soil–structure interaction effects. At an excitation period of about 0.55 sec there is a sharp reduction in the transfer function. This reduction was evident in the response spectra for motion in the longitudinal direction shown in Figure 3.6. The anomalous peak in the longitudinal transfer function at about 1 sec is caused by the torsional response of the building which affects the translational acceleration at the base. This torsional peak complicates efforts to model soil–structure interaction as described in Chapter 5. In other period ranges, particularly 1 to 2 sec, the transfer function fluctuates about unity indicating small modification of the base motion compared to the parking lot for these excitation periods.

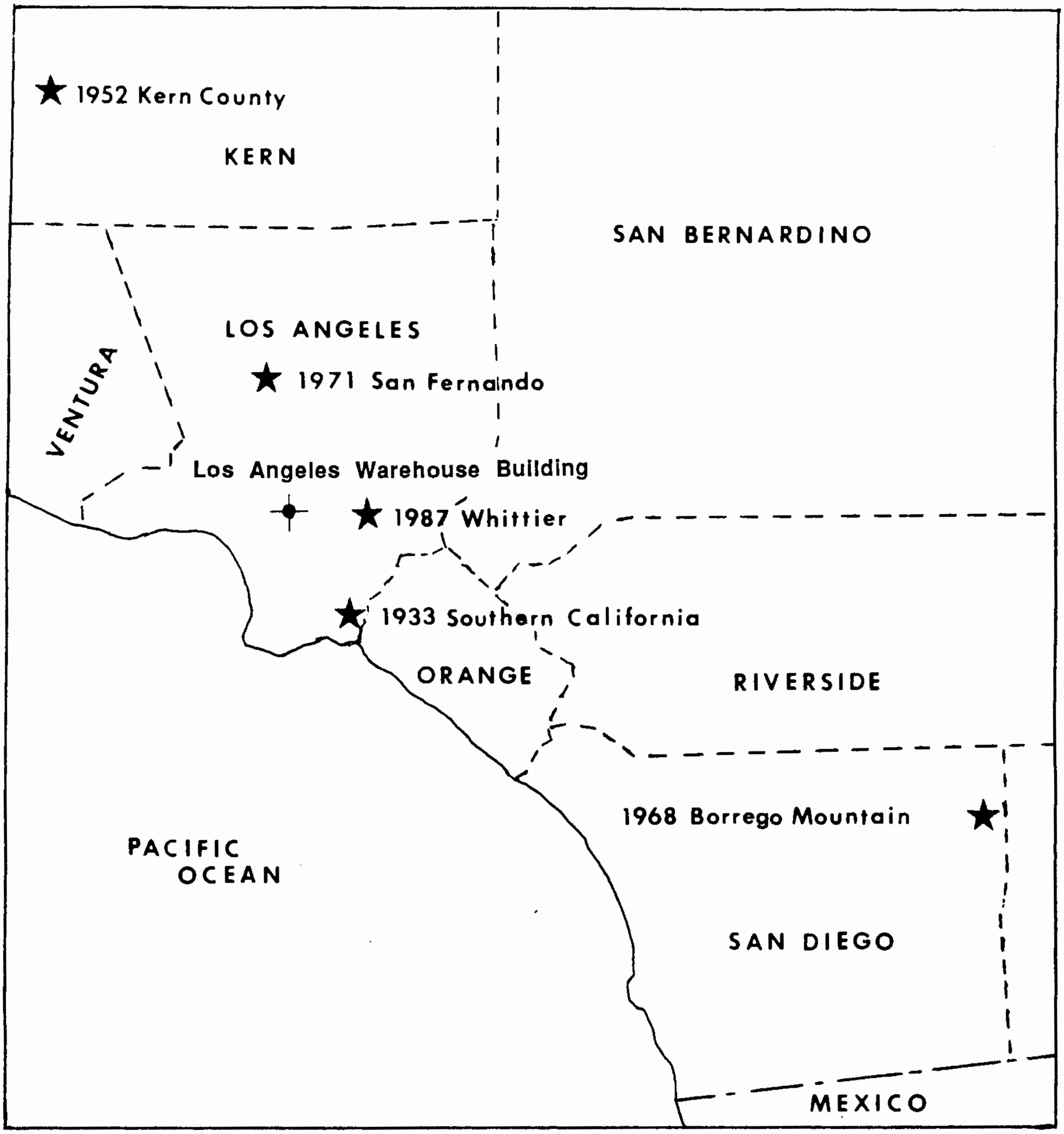

FIGURE 3.1 Location of the Los Angeles Warehouse Building and Epicenters of Earthquakes Recorded at the Building

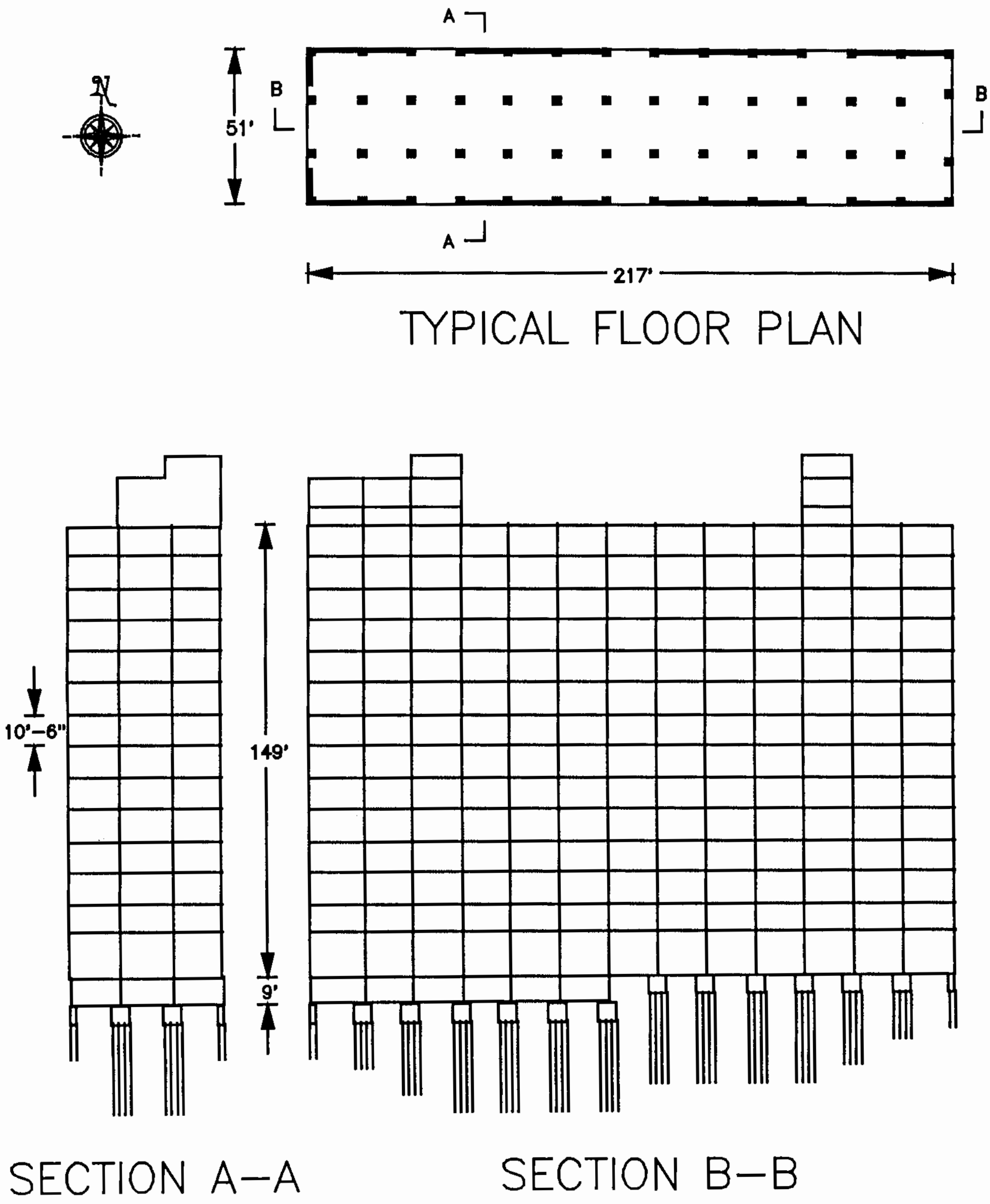

FIGURE 3.2 Structural Configuration of the Los Angeles Warehouse Building

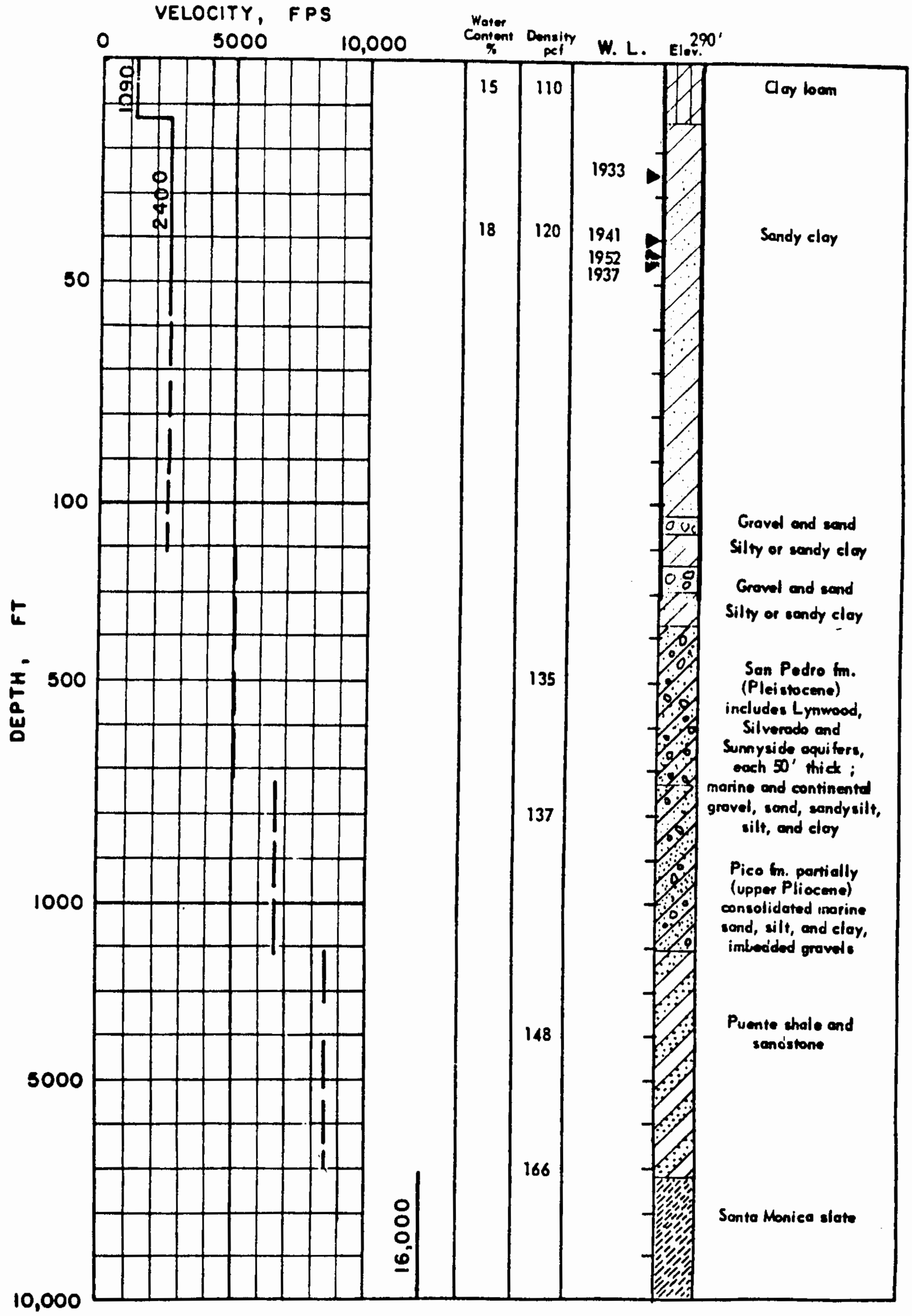

FIGURE 3.3 Characteristics of Soil at the Site of the Los Angeles Warehouse Building, Including Measured P–Wave Velocity (Duke and Leeds, 1962)

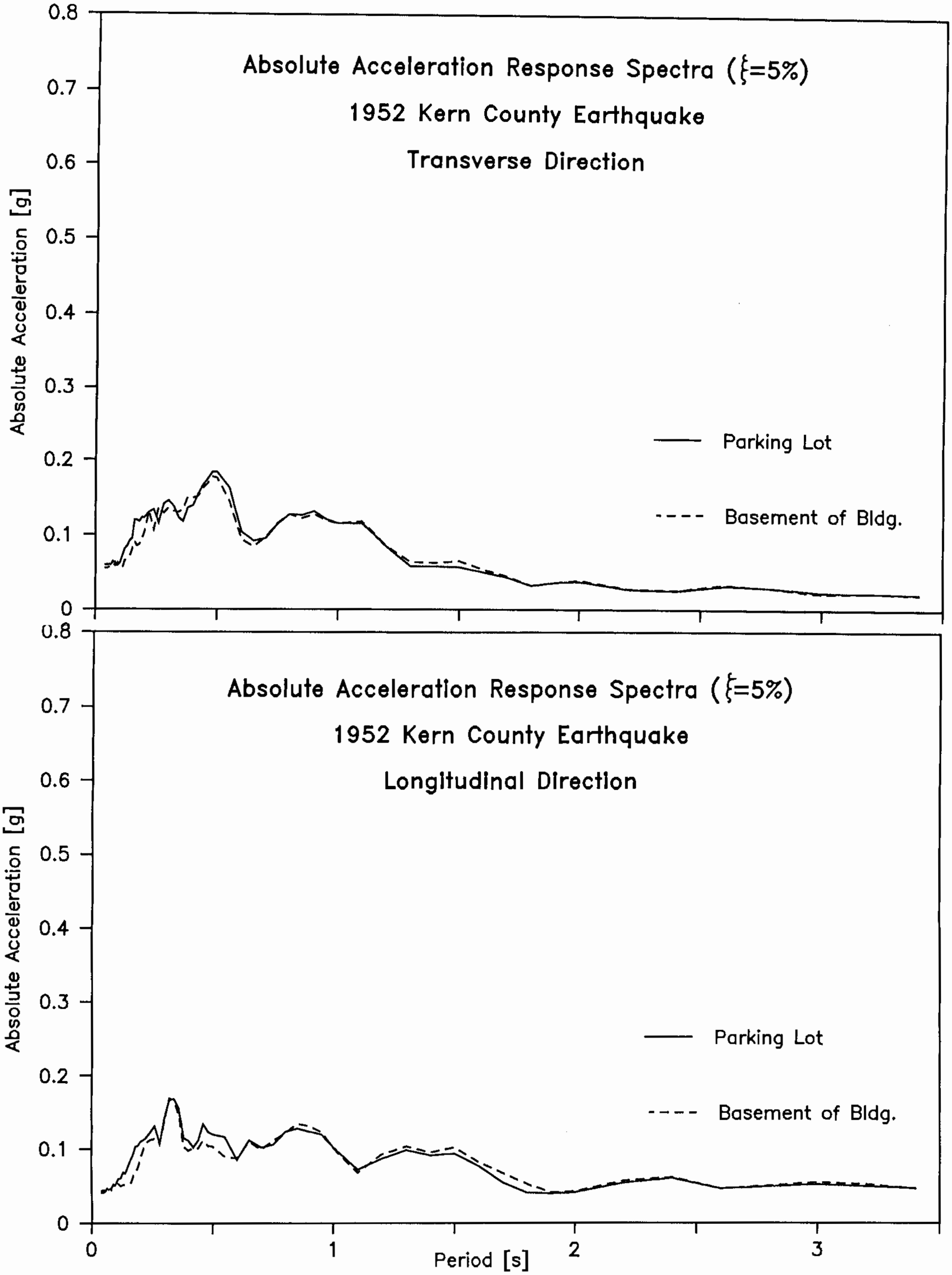

FIGURE 3.4 Absolute Acceleration Response Spectra for the Parking Lot and Basement of the Los Angeles Warehouse Building in the 21 July 1952 Kern County Earthquake

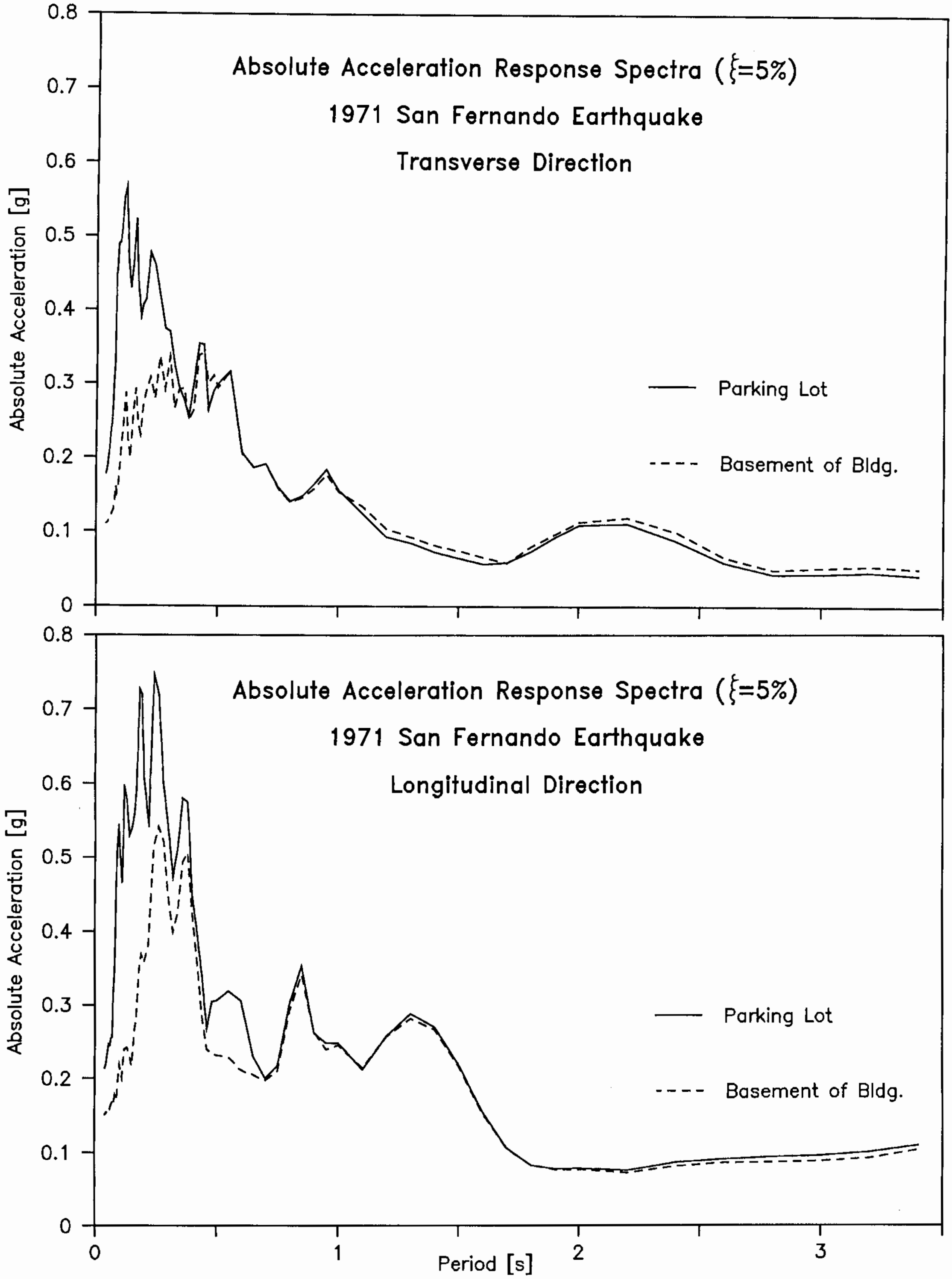

FIGURE 3.5 Absolute Acceleration Response Spectra for the Parking Lot and Basement of the Los Angeles Warehouse Building in the 9 February 1971 San Fernando Earthquake

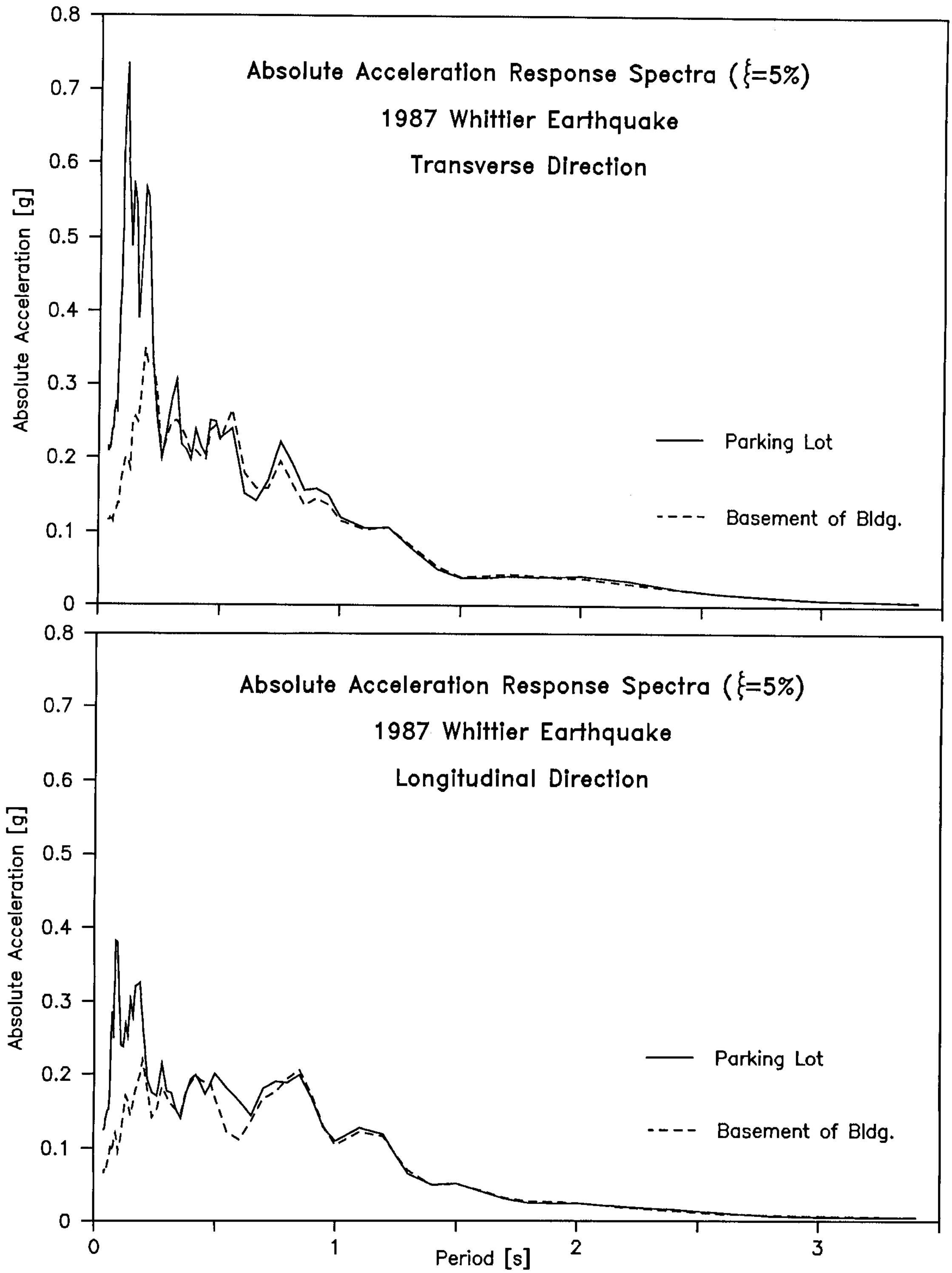

FIGURE 3.6 Absolute Acceleration Response Spectra for the Parking Lot and Basement of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

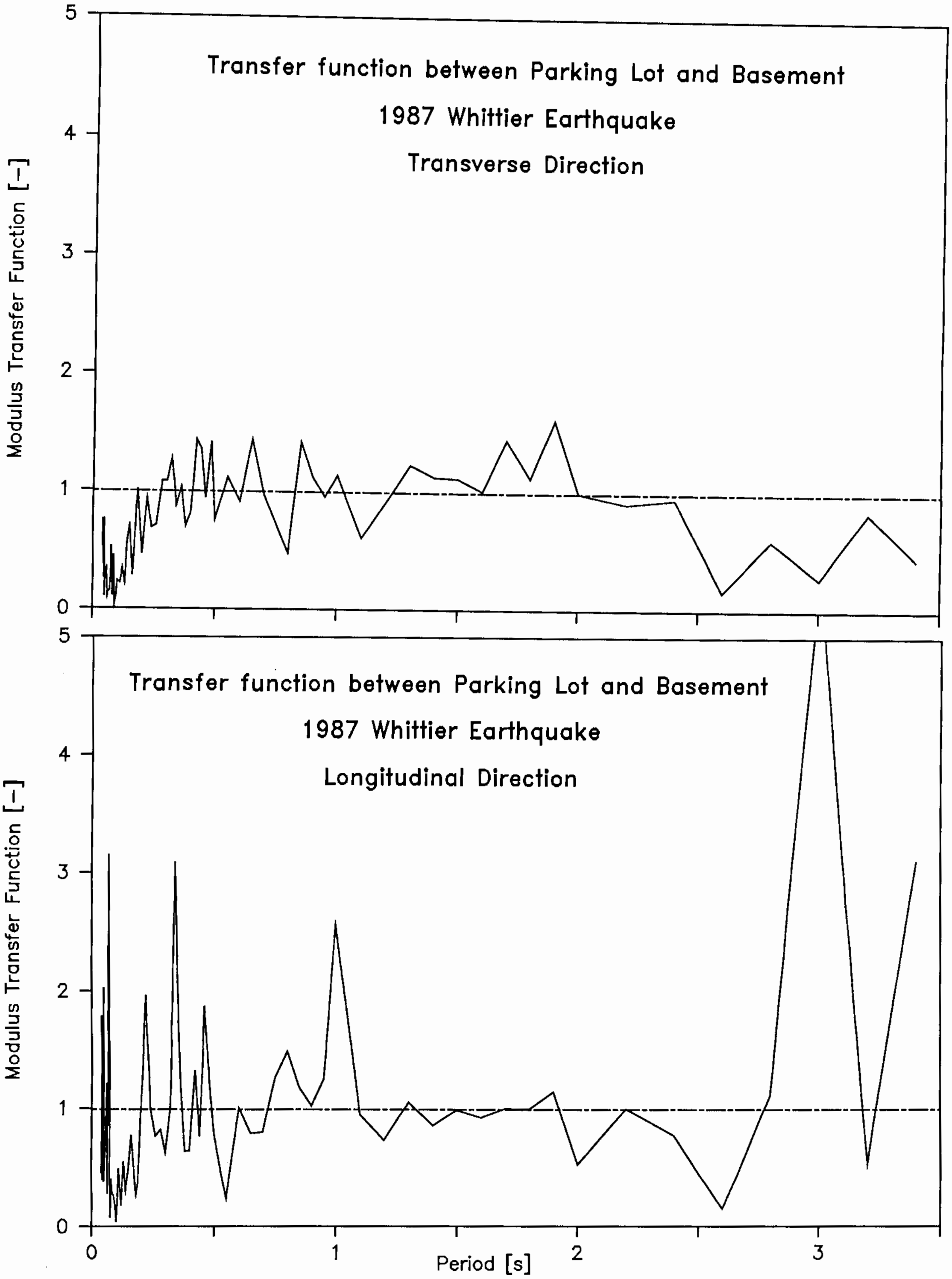

FIGURE 3.7 Modulus of Acceleration Transfer Function Between the Parking Lot and Basement of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

Chapter 4

MODEL FOR EVALUATING SOIL–STRUCTURE INTERACTION EFFECTS

4.1 Introduction

The instrumentation for the Los Angeles Warehouse Building recorded the overall translational and torsional response in the 1987 Whittier earthquake. The absolute acceleration response spectra and transfer functions show a reduction in response to the base motion compared to the parking lot motion at the fundamental period in the longitudinal direction. In the transverse direction there is little change in the response to the base motion at the fundamental period.

However, the strong motion records cannot be used to evaluate directly the effects of soil–structure interaction for the following reasons:

- The one vertical instrument at the base is not sufficient for evaluating the rocking of the building about a horizontal axis. Foundation rocking occurs when soil flexibility is important and it affects the response of the building.

- Although the base shear force can be estimated by interpolating the translational acceleration at each floor level and using the mass of the floors, the estimate would include the damping forces in the structure. If soil–structure interaction is significant, the damping in the system is increased by the added damping of the soil and it would adversely affect the base shear estimate.

- It is impossible to determine the response of the building if soil–structure interaction was neglected and the parking lot motion used as the input motion to the building.

A rational approach for evaluating soil–structure interaction for the Warehouse Building is to analyze a mathematical model of the building, foundation and soil system. The properties of the model are selected to provide a close correspondence with the response data from the 1987 Whittier earthquake. The model is then analyzed to determine the effects of soil–structure interaction in that earthquake. Various approximations in the modeling are accepted as long as they do not detract from the important features of soil–structure interaction in the overall response of the building.

The mathematical model of the Warehouse Building is developed in this chapter. The evaluation of soil–structure interaction in the 1987 Whittier earthquake using the model is presented in the next chapter.

4.2 Methodology for Evaluating Interaction Effects

The following methodology was adopted to study soil–structure interaction for the Los Angeles Warehouse Building in the 1987 Whittier earthquake.

i) Develop a mathematical model of the building, foundation, and soil. Calibrate the vibration periods and mode shapes of the model with forced vibration data and earthquake response.

ii) Compare the transfer functions for the model with the transfer functions computed from the response in the Whittier earthquake. Select damping parameters to obtain a good match between the recorded and model transfer functions.

iii) Compute the earthquake response of the calibrated mathematical model including and neglecting the effects of soil–structure interaction.

In this study no attempt is made to determine forces in the structural members in the earthquake. The model is used to only compute overall response quantities dominated by the fundamental vibration mode, such as roof displacement, base shear, and overturning moment.

Three major assumptions are made in the analysis for soil–structure interaction effects. The first assumption is that the response of the building is linear elastic. This is reasonable because there was no observed damage to the Warehouse Building in the 1987 Whittier earthquake. Also, the assumption of linear behavior implies that the foundation is able to transfer base forces to the soil without slip or uplift. The second assumption is that the response in the transverse and longitudinal directions of the building is uncoupled and can be analyzed independently. Even though the structural frame is almost symmetrical, there is some stiffness eccentricity from the infilled panels that resulted in torsional response in the Whittier earthquake. The period of torsional vibration, however, is not close to the translational vibration periods and the judgment was made that using the translation response alone would not significantly alter the conclusions. A practical consideration is that the mathematical modeling of a three–dimensional building–foundation–soil system is considerably more complex than for a two–dimensional system. Finally, kinematic interaction is neglected in the evaluation. Kinematic interaction alters the higher vibration mode response of the building, so the effect on the fundamental mode response and related quantities is assumed to be small.

4.3 Model of Building Superstructure

The linear, elastic mathematical model consists of the discretization of the mass, stiffness and damping properties of the building. The model of the superstructure, above the foundation level, was developed using the computer program SUPER–ETABS. The vibration properties of the model are matched with the vibration properties determined from forced vibration tests and the recorded earthquake response.

4.3.1 Description of the Analysis Program

The SUPER–ETABS program performs linear structural analysis of frame and shear wall buildings subjected to static and earthquake loadings (Maison and Neuss, 1983). A building is idealized as a system of independent frames and shear walls interconnected by floor diaphragms which are assumed rigid in their own plane. Rectilinear frames located arbitrarily in plan may be specified, and kinematic compatibility is enforced between the degrees–of–freedom common to the rigid floor diaphragms. Bending, axial and shear deformations of the columns are considered. Beams may be nonprismatic, and bending and shear deformations are included. Very stiff joints can be modeled by rigid zones at ends of beam and column elements. Panel elements model discontinuous shear walls and infilled frames.

The frames are treated as independent substructures. Each joint in a frame has six degrees–of–freedom, and the DOF that are not common to the rigid floor diaphragms are eliminated by static condensation. The structural stiffness matrix is assembled from the frame substructures. A consequence of this procedure is that compatibility of DOF common to more than one frame is not satisfied. However, for structural systems with frames orthogonal in plan, such as the Warehouse Building, the only incompatibility is different axial displacements of columns common to more than one frame. The effect of this incompatibility is small except for very tall structures. A general finite element analysis program would have been necessary if the building under study had a structural system with a more complicated geometry than the Warehouse Building.

4.3.2 Model of the Warehouse Building Superstructure

The data on the geometry and the dimensions of the structural members in the Los Angeles Warehouse Building were obtained from the 1925 design drawings made available by CSMIP. The partially embedded basement was not included and the penthouse masses were concentrated at the roof level.

As shown in Figure 4.1, the model of the superstructure consists of eighteen frames, four in the longitudinal direction and fourteen in the transverse direction. The member sizes were based on the gross dimensions from the design drawings. Shear panels were included in the exterior north, south, and west frames. The bending stiffness of the floor slabs was approximated by T–beams in the individual frames. The flange width of the T–beams was determined from the effective width of the slabs using the ACI 318 (1983) procedure. Shear deformation in the beams and columns was included, and rigid links were used to represent stiffness of the joints. The P–Δ effects were not included.

The two exterior longitudinal frames and the westward transverse frame are infilled with 8 inch thick concrete panels. Although the infill walls were not designed as a structurally continuous system, particular attention was given to evaluating their contribution to the stiffness of the building in the longitudinal direction. Openings in the first story panels were considered in an approximate manner. The good agreement between the vibration periods computed for the mathematical model and those obtained from the 1987 Whittier earthquake records clearly indicates that the infill panels must be included in the model to obtain a satisfactory estimate of the building stiffness.

The mass of the building frame, walls, and slabs was computed with a unit weight of 150 lb/ft^3 for concrete. Additional mass from nonstructural members and live loads present at the time of the earthquake are important in modeling dynamic response. Data about the live load rating or the live loads present in the warehouse at the time of the 1987 Whittier earthquake were not available. From a credible estimate based on the use of the building (goods storage), a uniformly distributed load of 110 lb/ft^2 was assumed to act on all floor surfaces. This load also accounts for the nonstructural components.

After modeling the building geometry, member properties, and mass, the only remaining parameter is the modulus of elasticity for the concrete. The modulus was selected to match the vibration periods of the model with the periods obtained from the response to the 1987 Whittier earthquake. The order of the vibration modes was checked against the forced vibration test data summarized in Table 3.2 (Carder, 1964). The fundamental vibration periods were modified for the soil flexibility, as described in the remainder of Section 4.3. This process gives a modulus of elasticity for concrete of E_c=2800 kip/in^2. This a reasonable value that accounts for limited cracking and creep over the life of the structure and it reflects the amplitude of building response in the Whittier earthquake. A Poisson's ratio of 0.16 was assumed for concrete, which gives a shear modulus of G_c=1200 kip/in^2.

4.3.3 Evaluation of Vibration Periods

The response data from forced vibration tests and earthquakes provides information about the dynamic properties of the Warehouse Building which was used to develop the mathematical model of the superstructure. In selecting the properties of the superstructure model, it was necessary to modify the vibration periods observed in the earthquake for the effects of soil flexibility.

The periods of the Warehouse Building in the lower translational modes and the torsional mode from earthquake response records are shown in Table 4.1. The periods were determined from the peaks of the transfer function between the base and roof computed from the processed records. The period in each mode of vibration increases with the maximum acceleration at the roof, which is a measure of the amplitude of motion experienced by the building. The period lengthening is due to nonlinear response of the building, particularly micro–cracking of concrete and bond slip of reinforcing steel leading to stiffness degradation, with increasing amplitude of motion. The periods obtained from the 1964 forced vibration tests (Table 3.2) are considerably less than the periods observed in earthquake response, particularly in the transverse direction, because of the low level excitation in the tests.

Because the goal of the analysis is to evaluate the response of the Warehouse Building to the 1987 Whittier earthquake, the lower mode vibration periods of the building obtained from the Whittier earthquake were used to calibrate the mathematical model.

TABLE 4.1
Lower Mode Vibration Periods of the Los Angeles Warehouse Building
From Earthquake Records and Forced Vibration Tests

Excitation	Max. Accel. at Roof (g)	Period of Vibration (sec) Transverse	Longitudinal	Torsional
Southern Calif. 2 Oct 1933	0.09	1.5	0.55	0.80
Kern County 21 July 1952	0.15	1.7	0.60	0.80
Whittier Narrows 1 Oct 1987	0.20	1.9	0.60	0.95
Forced Vibration Tests	—	1.2	0.50	0.60–0.64

4.3.4 Modification of Periods for Soil–Structure Interaction

An important effect of soil–structure interaction is to lengthen the vibration periods of a building–foundation–soil system compared to the periods of the building on a fixed base because of the soil flexibility. Soil–structure interaction has the largest effect on the fundamental period, with very little change of the higher vibration periods. The difference in the periods must be accounted for when calibrating the mathematical model of the superstructure with the periods observed in earthquake response. A procedure for estimating lengthening of the fundamental period is based on the relative stiffness of the superstructure compared to the stiffness of the foundation acting on the soil. This approximate procedure is based on principles of soil–structure interaction which are described further in Chapter 5 (Veletsos, 1976; Jennings and Bielak, 1973). In this study there is no modification of the higher vibration periods.

From the P–wave velocity data for the soil presented in Figure 3.3 (Duke and Leeds, 1962) and approximating the strain level dependency, an S–wave velocity of 1190 ft/sec was selected for the soil at the site. The basis for this determination is described in Section 5.2. With this S–wave velocity, the approximate procedure gives lengthening ratios for the fundamental period of 1.04 in the transverse direction and 1.08 in the longitudinal direction of the building. Using the periods of the soil–structure system obtained from the Whittier earthquake (Table 4.1), the fixed base fundamental period of the superstructure is 1.80 sec and 0.56 sec in the transverse and longitudinal directions, respectively.

The value of the elastic modulus for concrete, E_c=2800 kip/in^2, was selected as mentioned in Section 4.3.2, to provide these fundamental vibration periods of the superstructure in the transverse and longitudinal directions. The vibration periods of the three–dimensional mathematical model of the Warehouse Building superstructure are given in Table 4.2. The torsional mode period, which was not calibrated, is fairly close to 0.95 sec observed in the Whittier earthquake given the approximations in the modeling. The projections of the mode shapes in each translational direction are shown in Figures 4.2 and 4.3. The fundamental modes from the mathematical model of the superstructure compare favorably with a much simpler model used in an earlier study (Duke, *et al*, 1970).

4.4 Model of Soil–Structure System

The model of the complete building, foundation, and soil system was developed using the substructure approach. The building is one substructure and the foundation and soil are a second substructure. The substructures are coupled by equilibrium and compatibility requirements at the interface between the building and foundation. The advantage of the substructure approach is that different models appropriate for the building and soil can be selected. The use of the vibration modes of the superstructure as generalized coordinates significantly reduces the computational effort while realistically representing earthquake response of buildings (Chopra and Gutierrez, 1973).

The vibration properties of the three–dimensional superstructure show that the translational response of the lower modes in the longitudinal direction is nearly uncoupled from the transverse translation and torsional response. There is small coupling between translation in the transverse direction and torsional response because the east exterior frame lacks infill panels. Given the near symmetry of the structure, however, it is a reasonable approximation to use two different two–dimensional models, in the longitudinal and transverse directions, for the purpose of assessing soil–structure interaction effects in the Warehouse Building.

In each translational direction, the building is idealized as a fourteen story system with the vibration modes and periods determined from the three–dimensional superstructure model presented in Section 4.3 (Table 4.2, and Figures 4.2 and 4.3). Three vibration modes are used in the transverse direction and two modes in the longitudinal direction. Because response quantities, such as roof displacement, base shear, and overturning moment are of interest, the use of lower vibration properties obtained from the three–dimensional model is justified. Proportional viscous damping is assumed for the building.

The foundation is idealized as a rigid, circular disk attached to the surface of the soil. This is an important approximation of the elongated plan of the foundation and the irregular basement in the Warehouse Building. The radius of the disk is selected to provide a stiffness equivalent to the rectangular foundation in the Warehouse Building. For aspect ratios up to four, as in the foundation of the Warehouse Building, the stiffness of a rigid rectangular plate and that of an equivalent circular disk are nearly equal (Gazetas, 1983). The foundation model does not include the embedment of the partial basement nor the short piles. The depth of the foundation embedment, 9 ft, and the maximum pile length, 30 ft, are relatively small compared to the plan dimensions of the building.

TABLE 4.2
Vibration Periods of Mathematical Model
of the Los Angeles Warehouse Building

Mode No.	Mode Type[1]	Period (sec)
1	transverse	1.80
2	torsional	0.88
3	longitudinal	0.58
4	transverse	0.55
5	transverse	0.29
6	torsional	0.24
7	transverse	0.19
8	longitudinal	0.18

[1]Primary characterization of vibration mode

The soil supporting the Warehouse Building is idealized as a homogeneous, viscoelastic halfspace. The homogeneous halfspace is appropriate for obtaining structural response at the Warehouse Building site because there are no significant changes in the properties of the sandy clay from the surface to a depth of approximately 200 ft and the depth of the rock layers is about 2000 ft (see Figure 3.3). The nonlinear behavior of the soil is represented by selecting elastic and damping properties consistent with the level of strain (Seed and Idriss, 1970).

With these assumptions for the superstructure, foundation, and soil, the equations of motion can be formulated using the substructure approach. The seismic input to the model is the free–field ground motion. The model accounts for all effects of inertial soil–structure interaction within the assumptions described previously. Kinematic interaction effects are not included because the free–field ground motion is assumed to result from vertically propagating shear waves and the foundation is at the ground surface.

The formulation the substructure analysis and numerical procedure are presented in Appendix A. The results of the procedure are transfer functions for the building–foundation–soil system which can be compared with the transfer functions from the recorded response. The transfer functions are used in a response analysis for a specified free–field ground motion.

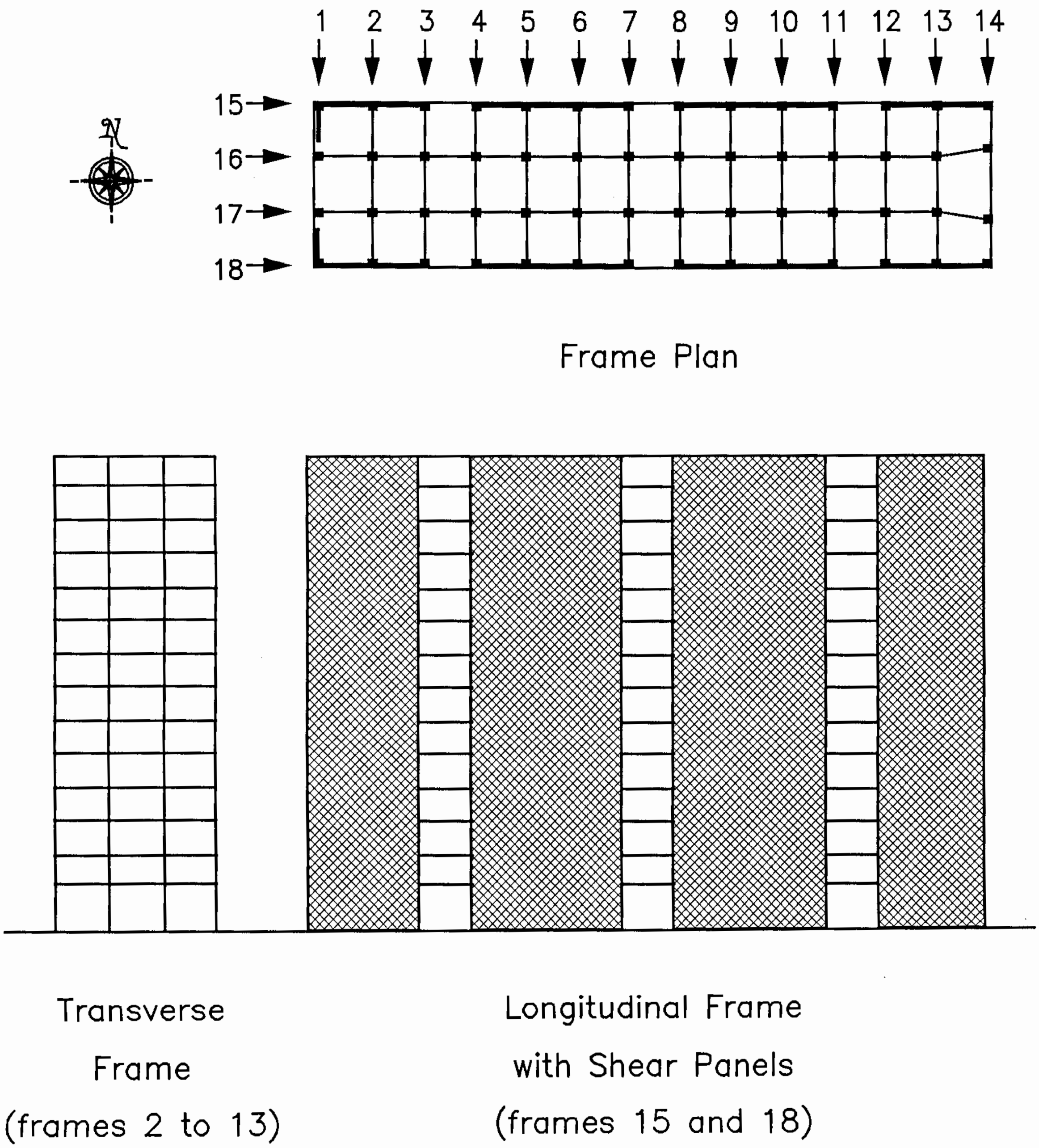

FIGURE 4.1 Frames and Shear Panels in the Mathematical Model of the Los Angeles Warehouse Building Superstructure

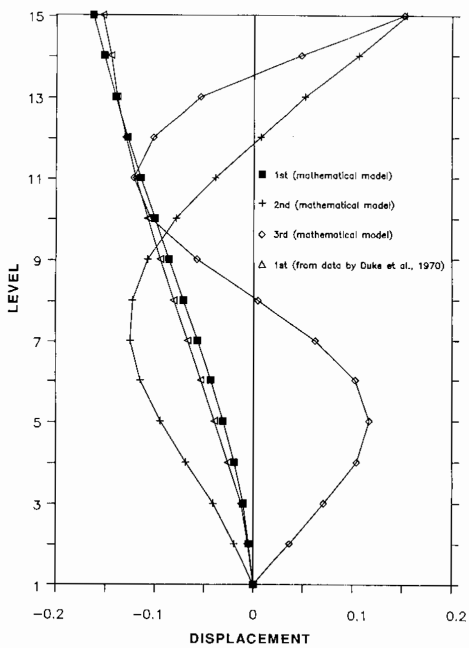

FIGURE 4.2 Vibration Mode Shapes of the Los Angeles Warehouse Building in the Transverse Direction from the Mathematical Model of the Superstructure

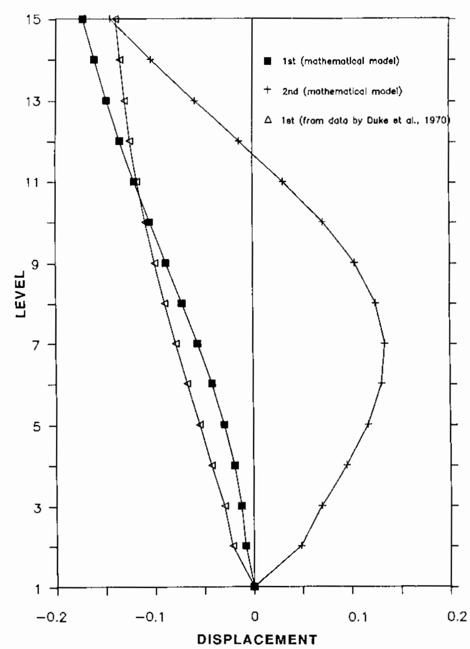

FIGURE 4.3 Vibration Mode Shapes of the Los Angeles Warehouse Building in the Longitudinal Direction from the Mathematical Model of the Superstructure

Chapter 5

EVALUATION OF SOIL–STRUCTURE INTERACTION EFFECTS

5.1 Introduction

The model of the Los Angeles Warehouse Building, including the foundation, and soil, was described in the previous chapter. In this chapter, the parameters for the soil and damping ratios for the building are selected to match the transfer functions of the model to the transfer functions obtained from the response to the 1 October 1987 Whittier earthquake. With the calibrated model, the response of the building including and neglecting the effects of soil–structure interaction is computed using a time history procedure.

5.2 Selection of Parameters for Model

The material properties for the homogeneous, viscoelastic model of the soil are the unit weight, Poisson's ratio, S–wave velocity, and hysteretic damping coefficient. The soil region of interest is the one significantly affected by the forces acting on the foundation. The depth of the region depends on the dimensions of the foundation and mode of foundation motion, horizontal translation or rocking. In this study the depth of the soil affecting the dynamic response of the Warehouse Building is assumed to be 200 ft, approximately the largest foundation dimension. The soil data (Duke and Leeds, 1962) indicates a unit weight varying from 110–130 lb/ft^3 in the upper 300 ft, so a unit weight of 120 lb/ft^3 was assumed. A Poisson's ratio of 0.33 was selected for the sandy clay.

As shown in Figure 3.3, the upper 200 ft of soil has an almost constant P–wave velocity of 2400 ft/sec, which corresponds to an S–wave velocity of 1190 ft/sec for the assumed Poisson's ratio. In comparison, the model of the building used to study the response to the 1952 Kern County earthquake assumed an S–wave velocity varying from 606 ft/sec at the surface to 1820 ft/sec at a depth of 200 ft, with a weighted average of 1360 ft/sec (Duke, *et al*, 1970). Considering that the level of soil strain in the 1987 Whittier earthquake was greater than in the 1952 earthquake, 1190 ft/sec is a reasonable estimate of the S–wave velocity. For a unit weight of 120 lb/ft^3, the corresponding shear modulus for the soil is 36.6 kip/in^2.

This shear modulus for the soil was used to calibrate the vibration periods of the building–foundation–soil model as described in Section 4.3. The periods of vibration for the model neglecting soil–structure interaction (fixed base) and including interaction effects are given in Table 5.1. The vibration periods observed in the 1987 Whittier earthquake are also listed in Table 5.1. Recognizing the assumptions in modeling the building, foundation, and soil, the comparison between the periods of the model and the observed periods is good.

TABLE 5.1
Comparison of Vibration Periods of the Mathematical Model
with Periods Observed in the 1 October 1987 Whittier Earthquake

Mode	Transverse Periods (sec)			Longitudinal Periods (sec)		
No.	w/o SSI[1]	SSI	Observed	w/o SSI[1]	SSI	Observed
1	1.8	1.9	1.9[2]	0.58	0.63	0.60[2]
2	0.55	0.56	0.48	0.18	0.19	0.19
3	0.29	0.30	0.30	—	—	—

[1]From Table 4.2
[2]From Table 4.1

The estimation of damping in the building and soil is difficult because there are no data available for the structural damping or for the soil damping. In the model, the damping in the structure is specified by a viscous damping ratio in each mode of vibration. The damping for the viscoelastic model of the soil is given by the hysteretic damping coefficient, η_s, defined as:

$$\eta_s = \frac{1}{4\pi}\frac{\Delta W}{W}$$

where ΔW is the area of the hysteresis loop when the soil is undergoing harmonic shearing deformation, and W is the maximum strain energy in a cycle of deformation. The damping ratio is an increasing function of the maximum strain in the soil (Seed and Idriss, 1970).

The damping parameters for the structure and soil are selected by comparing the peaks in the transfer function from the mathematical model with the transfer functions obtained from the response in the 1987 Whittier earthquake. The comparison of the transfer functions, which is presented in the next section, gives a hysteretic damping factor of η_s=0.20 for the soil. Although this is a large damping factor, the amplitude of ground motion at the parking lot instrument was fairly large (0.20 g). By matching the peaks of the transfer functions, damping ratios of 3.5%, 7.0%, and 9.0% for the first, second, and third modes, respectively, in the transverse direction were selected. The damping ratios are 8.0% and 9.0% for the first and second modes in the longitudinal direction.

5.3 Comparison of Model and Recorded Response

Figure 5.1 shows the modulus of the transfer functions between the parking lot and the base, 8th floor, 12th floor and roof center in the transverse direction obtained from the response of the Los Angeles Warehouse Building in the 1987 Whittier earthquake. The peaks in the transfer functions correspond to the first and second vibration periods at 1.9 sec and 0.48 sec, respectively. A wider, smaller third mode peak occurs at approximately 0.30 sec. The corresponding transfer functions between the parking lot and four levels of the building obtained from the mathematical model of the building–foundation–soil system are shown in Figure 5.2. The agreement between the computed and the recorded transfer functions is very good, although the peak corresponding to the second mode occurs at a slightly longer period (0.56 sec in Figure 5.2). The transfer function between the free-field and the building base is almost a constant value of unity for both the model and the recorded response, indicating negligible soil–structure interaction effects in the transverse direction. The transfer functions show a nodal point in the second mode at the 12th floor for the model and recorded data.

The transfer functions between the base of the building and the upper three levels in the longitudinal direction are shown in Figures 5.3 (recorded) and 5.4 (model). These transfer functions, which show the change in dynamic response from the base of the building to the various floor levels, compare well in the two modes of vibration.

Figures 5.5 and 5.6 show the recorded and model transfer functions between the free–field and four levels of the building in the longitudinal direction. These transfer functions account for the modification of input motion from the parking lot (assumed free–field) to the base of the building due to soil–structure interaction. The recorded transfer functions do not show the simple response observed in the transverse direction and the correlation with the model transfer functions is not as good. The peaks in the model and recorded transfer function corresponding to the two modes of vibration match in period and amplitude, but the recorded transfer functions have several additional peaks that are not represented in the model. The recorded response peak at 1 sec (Figure 5.5) is probably due to torsional motion of the building, which was not included in the two–dimensional substructure model. The two very narrow band peaks in the recorded transfer function between 0.30 sec and 0.50 sec are caused by small components in the free–field motion that are not necessarily representative of the dynamic response of the system.

It must also be recognized that in the longitudinal direction, the ground motion recorded at the parking lot station may not be the free–field ground motion for the building. The parking lot station is only 139 ft west of the building, about 0.64 times the foundation dimension in the longitudinal direction. The close proximity of the parking lot instrument to the basement may result in contamination of the parking lot motion by seismic waves scattered by the building foundation (Trifunac, 1972). This effect is more pronounced for the high frequency components, which explains the complicated nature of the recorded transfer functions in the short period range, as shown in Figure 5.5.

As an additional verification of the model, the displacement history recorded in the 1 October 1987 Whittier earthquake is compared to the displacements from the model. The horizontal displacement at the center of the roof with respect to the center of the basement is plotted in Figure 5.7 for the transverse direction and Figure 5.8 for the longitudinal direction. The model represents the displacement history recorded in both directions very well. However, the model overestimates the peak roof displacement relative to the base by 15%, even with the large damping parameters for the structure and soil. The comparsion of the recorded and model displacements at the 12th and 8th floors is also fairly good.

5.4 Effect of Soil–Structure Interaction on Base Forces and Displacements

The transfer functions for the substructure model of the Los Angeles Warehouse Building presented in the previous section (Figures 5.2 and 5.6) were used to perform a response history analysis of the building to the 1 October 1987 Whittier earthquake. The parking lot records were taken as the free–field ground motion. The response analysis procedure is described in Appendix A.

The analysis was performed for two cases: i) including the effects of soil–structure interaction using the soil properties given in Section 5.2; and ii) neglecting soil–structure interaction by assuming a rigid soil. Of particular interest is the effect of interaction on the maximum base shear force, overturning moment, and roof displacement.

The maximum base shear and seismic coefficient are summarized in Table 5.2. As expected from the recorded response and development of the model, soil–structure interaction has a small effect on the base shear in the flexible, transverse direction of the building. This is expected because of the similar acceleration response spectral ordinates for the parking lot and base motion at the fundamental period of the the building in the transverse direction (1.8–1.9 sec) as presented in Figure 3.6. The difference in base shear is mainly due to the response contribution of the second vibration mode.

In contrast, there are substantial soil–structure interaction effects in the stiff, longitudinal direction of the building. Interaction reduces the maximum base shear by 17% compared to the case where interaction is neglected. Examination of the acceleration response spectra in the longitudinal direction (Figure 3.6) shows this magnitude difference in the ordinates for the parking lot and base records, confirming that a decrease in base shear force is expected from soil–structure interaction.

TABLE 5.2
Maximum Base Shear Force for the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

	Base Shear Force (Seismic Coefficient)		
Direction	Neglecting SSI	Including SSI	Change due to SSI
Transverse	2300 kip (0.053)	2220 kip (0.0052)	–3%
Longitudinal	4310 kip (0.10)	3590 kip (0.083)	–17%

The effect of soil–structure interaction on the overturning moment is given in Table 5.3. The change in overturning moment due to soil–structure interaction is similar to the change in base shear.

TABLE 5.3
Maximum Overturning Moment for the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

	Overturning Moment (1000 kip–ft)		
Direction	Neglecting SSI	Including SSI	Change due to SSI
Transverse	146	136	–7%
Longitudinal	432	367	–15%

The effect of soil structure interaction on the maximum horizontal displacement at the roof level is given in Table 5.4. Two components of displacement are listed in the table. Structural displacements are relative to the rigid body motion of the base, both horizontal translation and rocking. Displacements relative to the base include the rocking component but not the translation of the base. The maximum roof displacement due to rocking, however, is very small: 0.14 inches in the transverse direction, and 0.051 inches in the longitudinal direction. In the longitudinal direction, soil–structure interaction reduces the maximum roof displacement. The reduced structural displacement is consistent with the reduction in base shear and overturning moment. In the transverse direction, where soil–structure interaction is less important, the structural displacement is not affected by interaction, whereas the displacement relative to the base increases. The increase in the latter is mostly from the rocking component.

TABLE 5.4
Maximum Roof Displacement for the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

(a) Structural displacement

Direction	Neglecting SSI	Including SSI	Change due to SSI
Transverse	2.03 in	2.03 in	0 %
Longitudinal	0.71 in	0.61 in	– 14%

(b) Displacement relative to the base

Direction	Neglecting SSI	Including SSI	Change due to SSI
Transverse	2.03 in	2.15 in	+ 6%
Longitudinal	0.71 in	0.65 in	– 8%

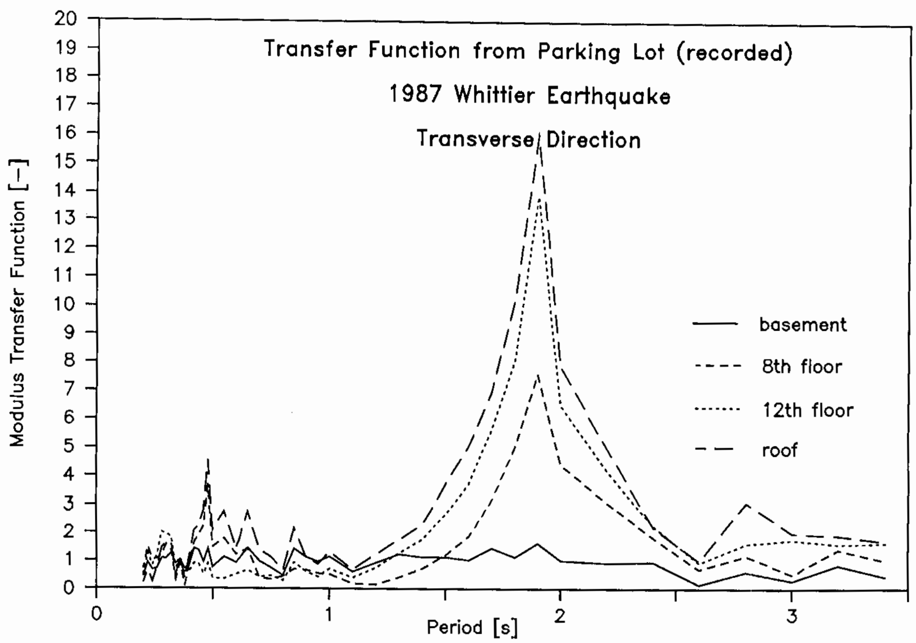

FIGURE 5.1 Modulus of Acceleration Transfer Functions Between the Parking Lot and Four Levels in the Transverse Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

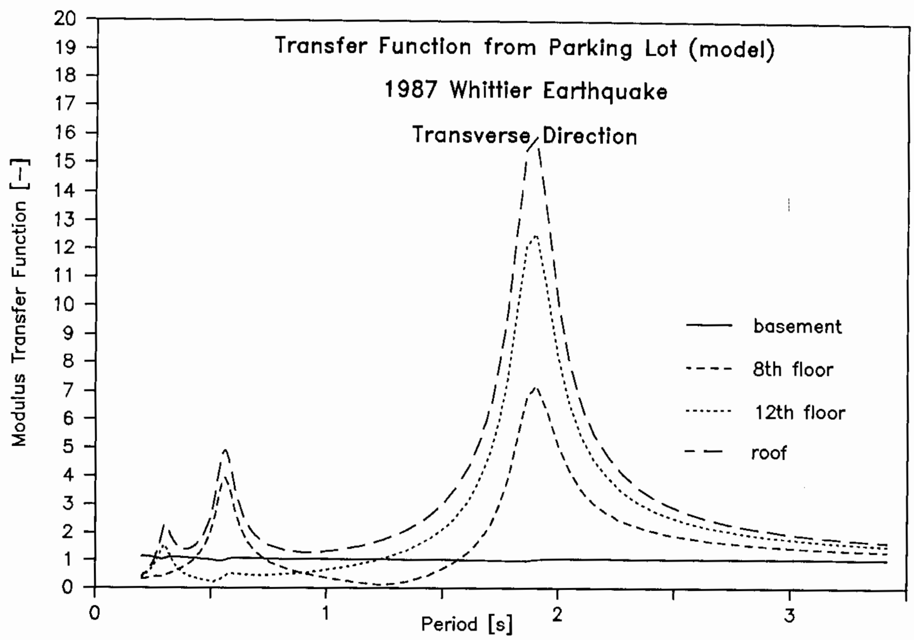

FIGURE 5.2 Modulus of Acceleration Transfer Functions Between the Free–Field and Four Levels in the Transverse Direction for the Mathematical Model of the Los Angeles Warehouse Building

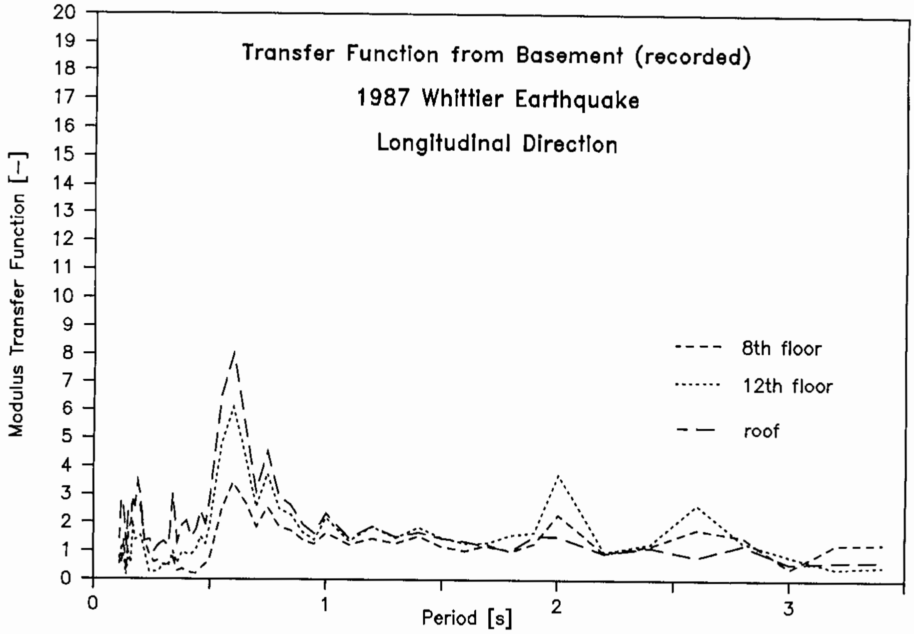

FIGURE 5.3 Modulus of Acceleration Transfer Functions Between the Base and Three Levels in the Longitudinal Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

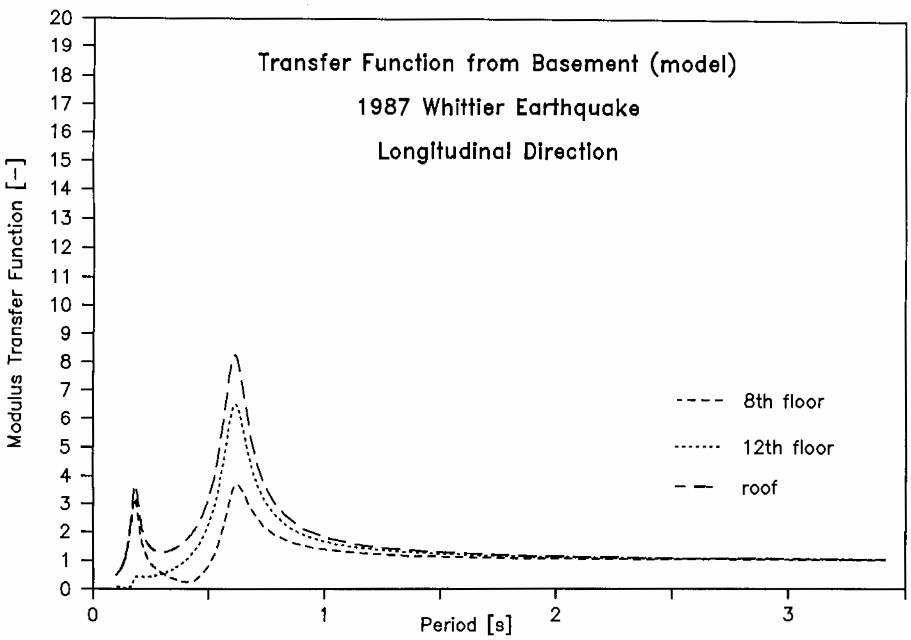

FIGURE 5.4 Modulus of Acceleration Transfer Functions Between the Base and Three Levels in the Longitudinal Direction for the Mathematical Model of the Los Angeles Warehouse Building

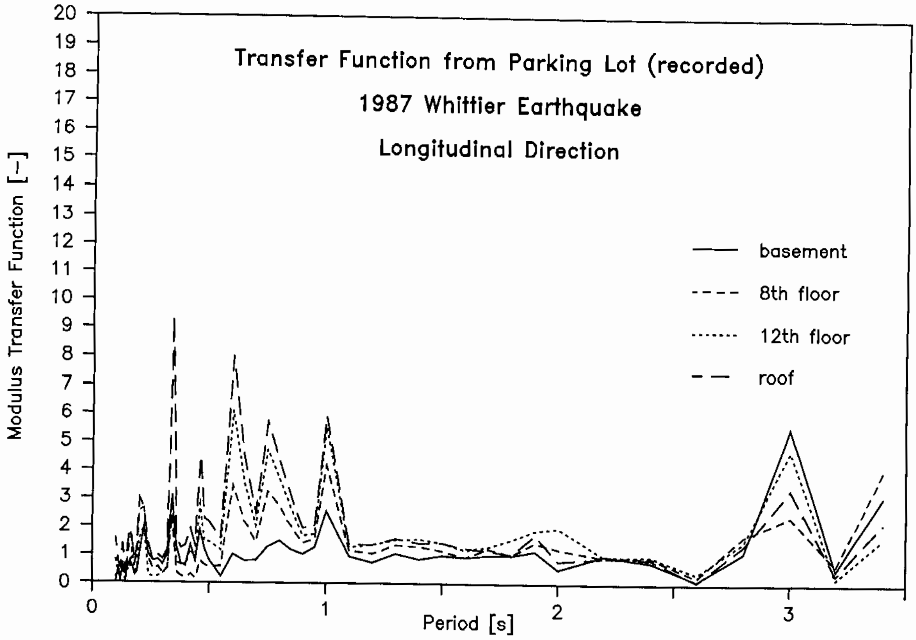

FIGURE 5.5 Modulus of Acceleration Transfer Functions Between the Parking Lot and Four Levels in the Longitudinal Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

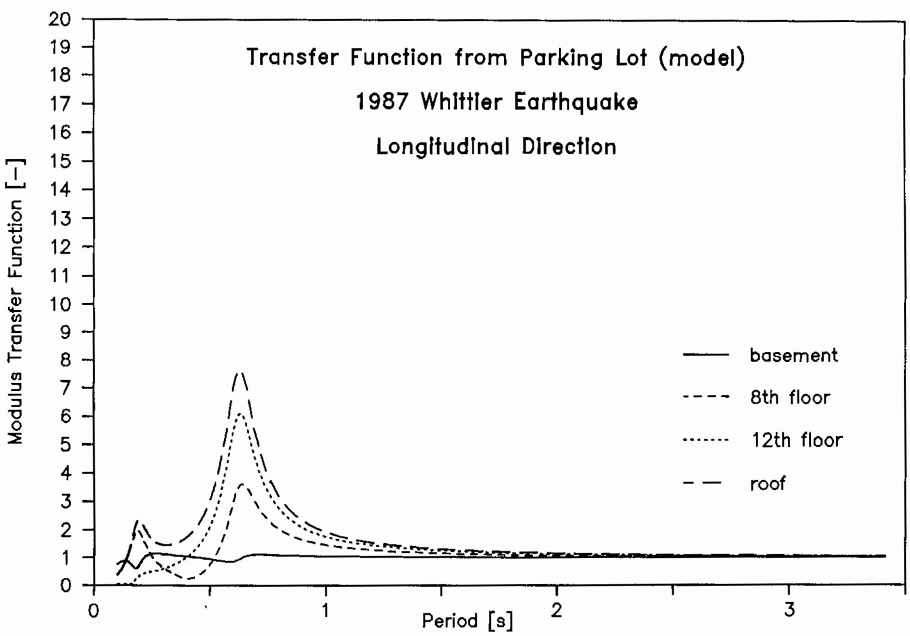

FIGURE 5.6 Modulus of Acceleration Transfer Functions Between the Free–Field and Four Levels in the Longitudinal Direction for the Mathematical Model of the Los Angeles Warehouse Building

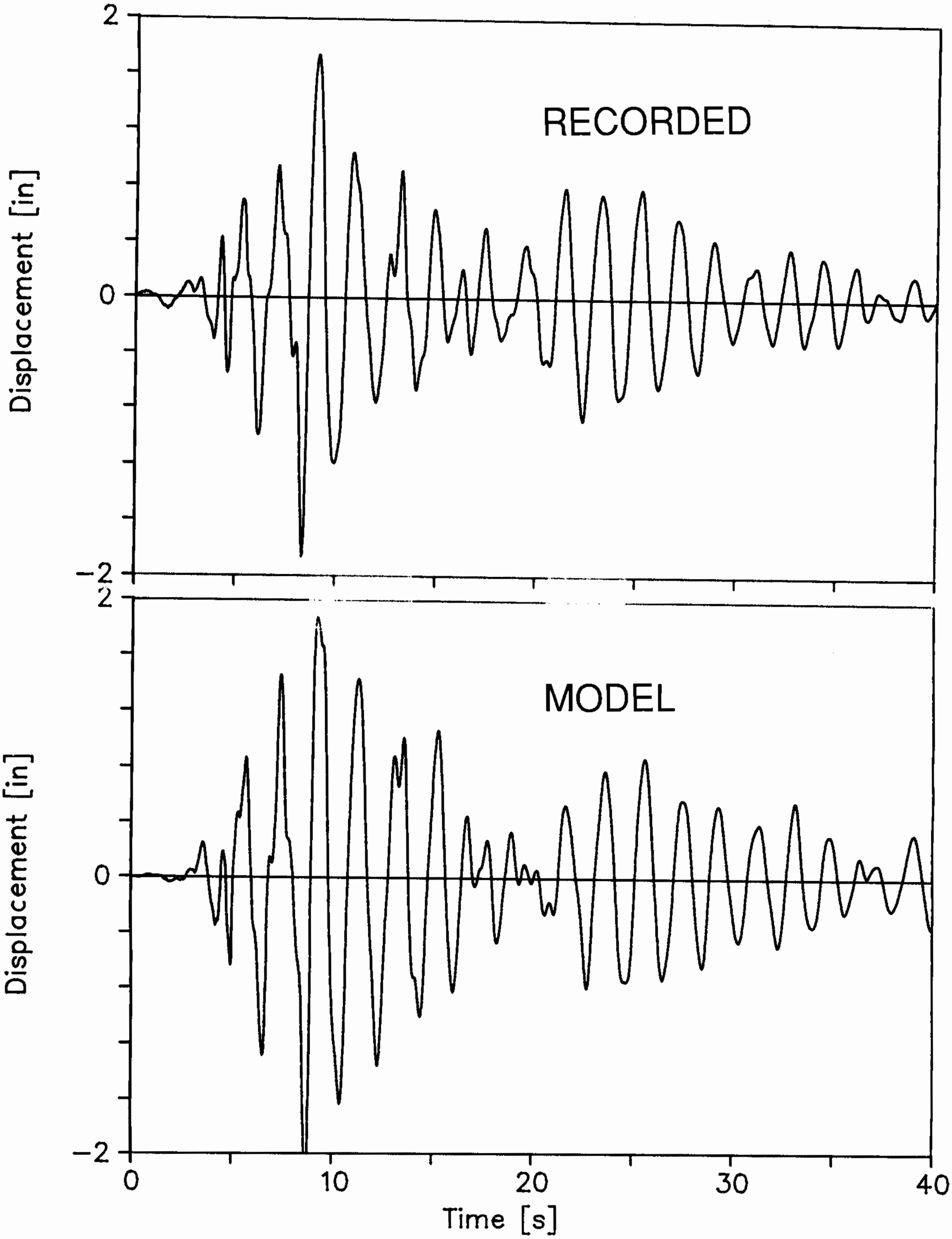

FIGURE 5.7 Horizontal Displacement of Roof with Respect to the Base in the Transverse Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

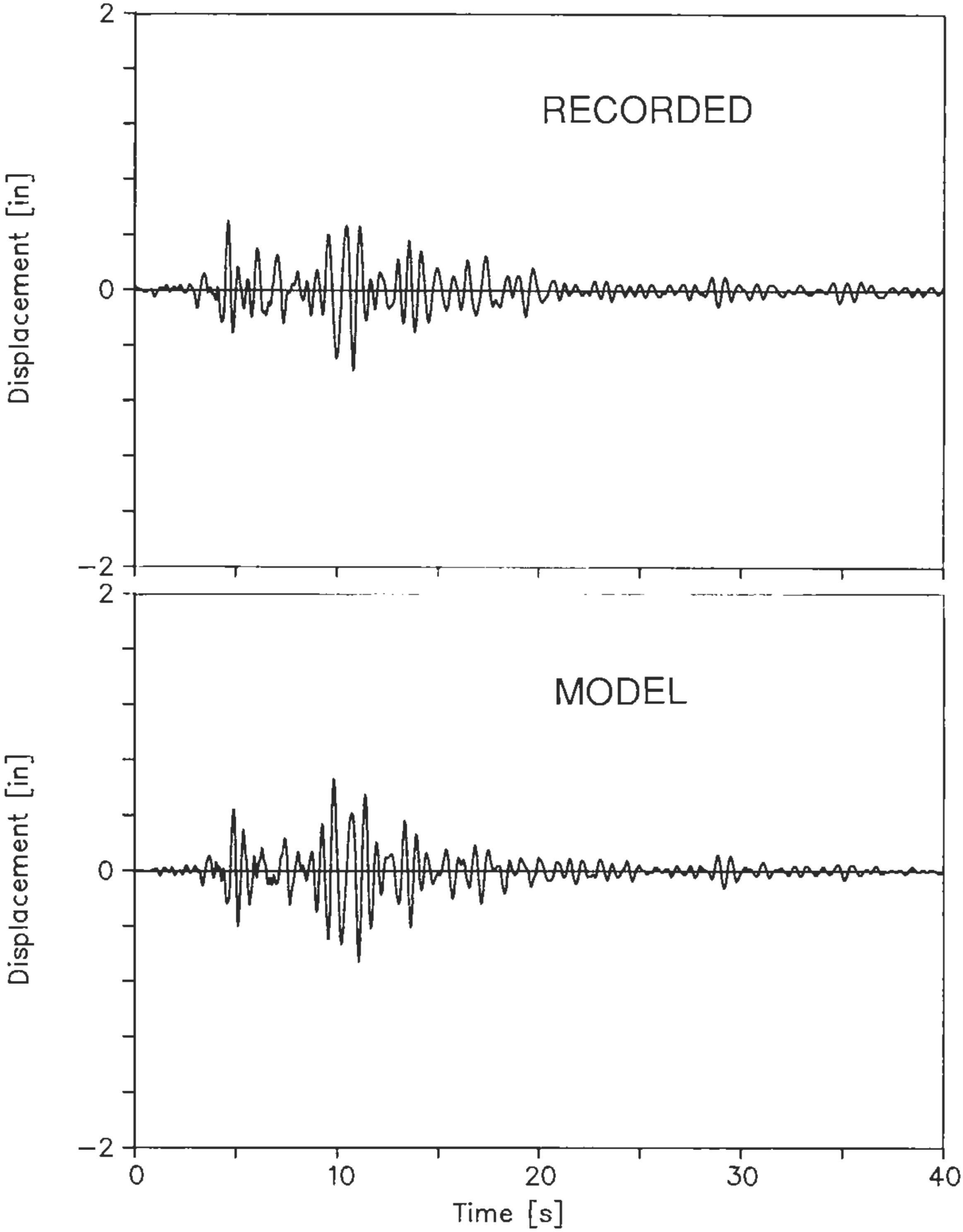

FIGURE 5.8 Horizontal Displacement of Roof with Respect to the Base in the Longitudinal Direction of the Los Angeles Warehouse Building in the 1 October 1987 Whittier Earthquake

Chapter 6

BUILDING CODE PROVISIONS FOR SOIL–STRUCTURE INTERACTION

6.1 Introduction

In the earthquake resistant design of buildings, it is not typical to model and analyze soil–structure systems using the techniques presented in Chapters 4 and 5. However, it is possible to extract the important effects of soil–structure interaction and use them in a simplified analysis procedure. This chapter briefly describes the use of the simplified analysis procedure for soil–structure interaction which is the basis for proposed building code provisions. The ability of the proposed provisions to recognize the effects of soil–structure interaction in the Los Angeles Warehouse Building are evaluated.

6.2 Building Code Provisions for Soil–Structure Interaction

Soil–structure interaction primarily affects the fundamental mode response of buildings because that is the mode in which the largest base forces are developed and hence the largest amount of foundation motion occurs (Jennings and Bielak, 1973). Compared to a building on a fixed base, the effects of interaction on the fundamental mode response are (Veletsos, 1976):

- The vibration period increases because of soil flexibility.
- The damping ratio changes (usually increases) because of the added damping due to material damping and wave propagation in the soil.

Based on these effects, a procedure for estimating the maximum base shear in a building including soil–structure interaction has been proposed in the framework of the equivalent lateral force procedure commonly used in design (Veletsos, 1976; ATC, 1978; NEHRP, 1988).

The fundamental vibration period of a building including soil flexibility, $\tilde{T}$, is given by:

$$\tilde{T} = RT \tag{6.1a}$$

where T is the vibration period of the building on fixed base (neglecting interaction) and the period lengthening ratio, R, is defined as:

$$R = \sqrt{1 + \frac{k}{K_x} + \frac{kh^2}{K_\theta}} \tag{6.1b}$$

in which k is the lateral stiffness of the building, h is the height, and K_x and K_θ are the horizontal translational and rotational stiffness, respectively, of the foundation and soil. The period lengthening ratio, R, is never less than unity because soil flexibility always lengthens the period of a fixed base structure. Equation 6.1 can be derived from the equations of motion for the building–foundation–soil system (Veletsos, 1976; Fenves and Chopra, 1985). The static stiffness of the soil is sufficient for an accurate estimate of the fundamental vibration period in Eq. 6.1 (ATC, 1978). For multistory buildings, the lateral stiffness and height of the building are the generalized stiffness and height of the building in its fundamental mode of vibration.

If the foundation is idealized as a rigid, circular disk on an elastic halfspace model of the soil, the static stiffness coefficients are:

$$K_x = \frac{8G_s r_0}{2-\nu} \tag{6.2a}$$

$$K_\theta = \frac{8G_s r_0^3}{3(1-\nu)} \tag{6.2b}$$

where r_0 is the radius of the foundation, and G_s and ν are the shear modulus and Poisson's ratio, respectively, for the soil. Guidelines for selecting the properties and stiffness coefficients for other foundation geometries and soil models are described in ATC (1978) and NEHRP (1988).

The modification of damping for a building due to soil–structure interaction is given by (Veletsos, 1976; Fenves and Chopra, 1985; Wolf, 1985):

$$\tilde{\xi} = \frac{1}{R^3}\xi + \xi_0 \tag{6.3}$$

where $\tilde{\xi}$ is the effective viscous damping ratio for the building including interaction, R is given in Eq. 6.1(b), ξ is the viscous damping ratio for the building on fixed base, and ξ_0 is the added damping for the soil. The added damping includes the effects of wave propagation and material damping. It is a function of the period lengthening ratio R, the ratio h/r_0, and the hysteretic damping coefficient for the soil (Veletsos, 1976). As can be seen in Eq. 6.3, the effective damping ratio can increase or decrease compared to ξ due to soil–structure interaction. For typical buildings, however, the effective damping increases.

The effects of soil–structure interaction, lengthened period and increased damping compared to the fixed base case, on the maximum base shear force are illustrated in Figure 6.1. The base shear coefficient is shown schematically for a smoothed response spectrum and a typical response spectrum used in building codes. In the response to an actual earthquake, soil–structure interaction can increase or decrease the maximum base shear depending on the period of the soil–structure system and the spectrum for the earthquake. For a code spectrum, however, soil–structure interaction never increases the base shear force compared to the base shear neglecting interaction effects.

6.3 Application of Provisions to the Los Angeles Warehouse Building

The proposed procedure for soil–structure interaction (ATC, 1978; NEHRP, 1988) can be applied to the Los Angeles Warehouse Building. Using the soil properties given in Section 5.2 and the fundamental mode properties from the mathematical model of the superstructure (generalized stiffness and height), Eq. 6.1 gives a period lengthening ratio, R, of 1.04 and 1.08 in the transverse and longitudinal direction of the building, respectively. These factors were used in Section 4.3.4 to calibrate the vibration periods of the mathematical model with the periods observed in the 1 October 1987 Whittier earthquake.

The important parameters for calculating the design base shear according to ATC (1978) or NEHRP (1988) are:

- Effective peak velocity–related acceleration, A_V=0.40
- Soil profile coefficient, S=1.2
- Response modification factor, 5.5
- Weight of building, W=43,000 kip

The fundamental vibration periods of the building used in the code evaluation of base shear are given in Table 5.1.

Assuming 5% damping for the building on fixed base, the effective damping including soil–structure interaction is 5.2% and 6.0% in the transverse and longitudinal directions, respectively. The resulting base shear and seismic coefficients according to the proposed code provisions are given in Table 6.1.

TABLE 6.1
Maximum Base Shear Force for the Los Angeles Warehouse Building Using Proposed Code Provisions

	Base Shear Force (Seismic Coefficient)		
Direction	Neglecting SSI	Including SSI	Change
Transverse	3010 kip (0.070)	2920 (0.068)	– 3%
Longitudinal	6450 kip (0.15)	5850 kip (0.136)	– 9%

The base shear values in Table 6.1 using the code provisions are, of course, not directly comparable with the base shear for the 1 October 1987 Whittier earthquake given in Table 5.2 because of the different basis used for code forces compared to the response to a specific, moderate earthquake. What is important, however, is how well the proposed code provisions represent the change in base shear force because of soil–structure interaction. In the transverse direction, where soil–structure interaction effects are small, the code allows a 3% reduction in base shear. In the longitudinal direction, where soil–structure interaction is more significant, the code allows a reduction in base shear of 9%.

The reduction of base shear given by the code can be compared to the response of the building to the 1987 Whittier earthquake. In the transverse direction there is a small change of base shear due to soil structure interaction. The code reduction of 3% is equal to the reduction determined from analysis of the Warehouse Building to the 1987 Whittier earthquake. In the longitudinal direction the code reduction of 9% is less than the 17% reduction in base shear in the Whittier earthquake. The difference in the code predicted reduction and actual reduction for response in the longitudinal direction of the Warehouse building is primarily caused by the large amount of material soil damping apparent in the response to the 1987 Whittier earthquake. The large damping value is not represented in the code procedures, hence the reduction in base shear is underestimated.

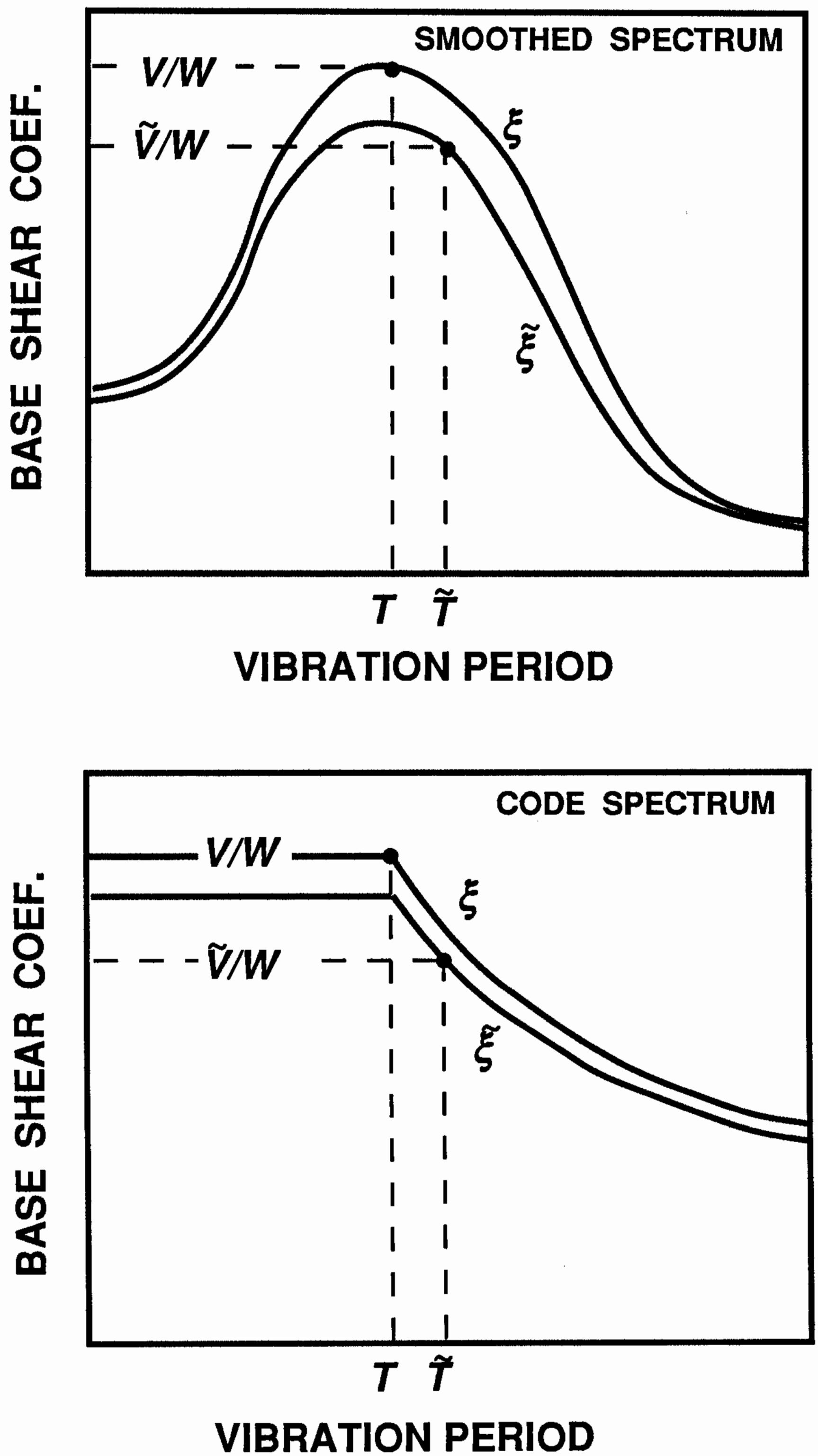

FIGURE 6.1 Effect of Soil–Structure Interaction on Seismic Coefficient for Base Shear

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

A mathematical model of the Los Angeles Warehouse Building, including the foundation and supporting soil, was developed to determine the effects of soil–structure interaction in the response to the 1 October 1987 Whittier earthquake. The parameters of the model were selected so that the transfer functions for the model matched the transfer functions computed from the processed response data.

The evaluation of soil–structure interaction effects in the Warehouse Building was based on three assumptions. First, the foundation was modeled as a rigid, circular disk, ignoring the complexity of the foundation and partial basement. The second assumption was that the ground motion recorded at the parking lot was the free–field ground motion, the motion that would occur at the site if the building was not present. Finally, the effects of kinematic interaction were neglected.

Using these assumptions, analysis of the model demonstrated that soil–structure interaction had a small effect on the maximum base shear force in the flexible, transverse direction of the building. In contrast, soil–structure interaction had a substantial influence on the maximum base shear in the stiffer, longitudinal direction. The analysis shows that the base shear for the Warehouse Building in the Whittier earthquake was 17% less than the base shear neglecting the effects of interaction.

Proposed building code provisions that account for soil–structure interaction in an equivalent lateral force procedure were examined. In the context of the code spectrum and assumptions on damping, soil–structure interaction can never increase the base shear force in a building. The provisions conservatively estimate the reduction in base shear force in the longitudinal direction of the Warehouse Building. In the transverse direction, the provisions are fairly accurate.

The current study, although limited in scope, demonstrates that soil–structure interaction can significantly modify the response of typical buildings. However, there are many assumptions and limitations involved in the analysis. Future study should address the following issues:

- A more detailed modeling of the foundation and soil, accounting for embedment, piles, and soil layers, may improve the correlation between the model and recorded transfer functions, particularly in the short period range.

- The effects of kinematic interaction on building response, particularly on the higher vibration modes should be determined.

- The coupling between the parking lot instrument and basement should be explicitly considered to determine if the parking lot instrument accurately reflects the free–field ground motion.

- A three–dimensional model of the system is necessary to include the torsional response of the building.

Based on the evaluation of soil–structure interaction effects, several recommendations for instrumentation of buildings for soil–structure interaction can be made:

- The current procedures for collecting and processing building and ground response data are very good and no major changes appear necessary.

- This evaluation examined one building in a moderate earthquake. To improve the understanding of soil–structure interaction, additional response data are essential. The increased use of free–field instruments in current or proposed building stations would provide important response data on soil–structure interaction effects.

- It is possible to screen buildings for importance of soil–structure interaction based on the vibration periods and properties of the soil. This is recommended for current and proposed building stations to identify sites where interaction is important.

- A moderate increase in vertical instrumentation at the base of a building can provide the rocking response, particularly for buildings with stiff foundations. The availability of rocking directly from the response would aid in identifying soil–structure interaction effects and evaluating drifts in a building. A minimum of three vertical instruments at the base are necessary to determine the rocking.

- A further increase in instrumentation for additional ground motion stations near a building may be warranted. A better resolution of the ground motion in the vicinity of a building would improve understanding of how interaction with the building modifies the motion at the base. Building instrumentation could be combined with an established or planned array to accomplish this goal.

Chapter 8

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Appendix A

NUMERICAL PROCEDURE FOR SUBSTRUCTURE ANALYSIS OF SOIL–STRUCTURE SYSTEMS

A.1 Introduction

The substructure procedure for earthquake analysis of building–foundation–soil systems in presented in this appendix. The equations of motion are formulated in the frequency domain to include the frequency dependent response of the soil. The numerical procedure gives the transfer functions for the model. The building response to a specified free–field ground acceleration record can be obtained using discrete Fourier transform techniques.

A.2 Formulation of Equations of Motion

The substructure model for two–dimensional response is shown in Fig. A.1. The model has N translational DOF at the floor levels and two DOF at the base, horizontal translation, $u_0(t)$, and rotation, $\theta_0(t)$.

The equations of motion for the masses at the floor levels are:

$$\mathbf{m}\ddot{\mathbf{u}}_t + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} = \mathbf{0} \tag{A.1}$$

where $\mathbf{m}$, $\mathbf{c}$, and $\mathbf{k}$ are the mass, damping, and stiffness matrices of the building, respectively; $\mathbf{u}_t$ and $\mathbf{u}$ are vectors of floor displacements relative to a fixed frame of reference and the foundation, respectively.

Translational and rotational equilibrium of the building–foundation system are given by two additional equations of motion,

$$\mathbf{1}^T\mathbf{m}\ddot{\mathbf{u}}_t + m_0(\ddot{u}_0 + \ddot{u}_g) + v(t) = 0 \tag{A.2a}$$

$$\mathbf{h}^T\mathbf{m}\ddot{\mathbf{u}}_t + I_0\ddot{\theta}_0 + m(t) = 0 \tag{A.2b}$$

in which $\mathbf{1}$ and $\mathbf{h}$ are vectors of 1's and story heights, respectively; m_0 and I_0 are the mass and mass moment of inertia of the foundation, respectively; $v(t)$ and $m(t)$ are the base shear force and overturning moment, respectively, that the foundation exerts on the soil; and $\ddot{u}_g$ is the horizontal free–field ground acceleration with respect to the reference frame.

The total motion at the floor levels is:

$$\mathbf{u}_t = \mathbf{u} + (u_0 + u_g)\mathbf{1} + \theta_0\mathbf{h} \tag{A.3}$$

which upon substitution into Eqs. A.1 and A.2 gives:

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} + \mathbf{m}\mathbf{1}\ddot{u}_0 + \mathbf{m}\mathbf{h}\ddot{\theta}_0 = -\mathbf{m}\mathbf{1}\ddot{u}_g \tag{A.4a}$$

$$\mathbf{1}^T\mathbf{m}\ddot{\mathbf{u}} + m_t\ddot{u}_0 + L_t\ddot{\theta}_0 + v(t) = -m_t\ddot{u}_g \tag{A.4b}$$

$$\mathbf{h}^T\mathbf{m}\ddot{\mathbf{u}} + L_t\ddot{u}_0 + I_t\ddot{\theta}_0 + m(t) = -L_t\ddot{u}_g \tag{A.4c}$$

where,

$$m_t = m_0 + \mathbf{1}^T\mathbf{m}\mathbf{1} \tag{A.5a}$$

$$L_t = \mathbf{h}^T\mathbf{m}\mathbf{1} \tag{A.5b}$$

$$I_t = I_0 + \mathbf{h}^T\mathbf{m}\mathbf{h} \tag{A.5c}$$

The base shear, $v(t)$, and moment, $m(t)$, deform the soil with respect to the frame of reference. The relationship between forces and deformation of the soil depends on the excitation frequency. Because of the frequency dependence, the equations of motion, Eq. A.4, are solved in the frequency domain. For a unit harmonic free–field ground acceleration, $\ddot{u}_g(t) = e^{i\omega t}$, a steady–state response function (such as displacement, force, etc.) is given by $r(t) = R(\omega)e^{i\omega t}$. The frequency dependent relationship between forces and deformation of the soil is given by:

$$\begin{Bmatrix} V \\ M \end{Bmatrix} = \begin{bmatrix} K_{VV} & K_{VM} \\ K_{MV} & K_{MM} \end{bmatrix} \begin{Bmatrix} U_0 \\ \Theta_0 \end{Bmatrix} \tag{A.6}$$

where the upper case variables are response functions that depend on the excitation frequency ω. The impedance functions for the soil, K_{VV}, K_{MM}, and K_{VM}=K_{MV}, are complex valued, frequency dependent functions that depend on the the foundation and properties of the soil.

After transformation to the frequency domain, the equations of motion, Eq. A.4, in conjunction with Eq. A.6, are:

$$(-\omega^2\mathbf{m} + i\omega\mathbf{c} + \mathbf{k})\mathbf{U} - \omega^2\mathbf{m}\mathbf{1}U_0 - \omega^2\mathbf{m}\mathbf{h}\Theta_0 = -\mathbf{m}\mathbf{1} \tag{A.7a}$$

$$-\omega^2\mathbf{1}^T\mathbf{m}\mathbf{U} + (-\omega^2 m_t + K_{vv})U_0 + (-\omega^2 L_t + K_{VM})\Theta_0 = -m_t \tag{A.7b}$$

$$-\omega^2\mathbf{h}^T\mathbf{m}\mathbf{U} + (-\omega^2 L_t + K_{MV})U_0 + (-\omega^2 I_t + K_{MM})\Theta_0 = -L_t \tag{A.7c}$$

For each excitation frequency the N+2 equations in Eq. A.7 must be solved for the N+2 response quantities. An effective approach for reducing the number of unkowns is to express the relative floor displacements as a summation of the response of the J modes of vibration of the structure on fixed base (Chopra and Gutierrez, 1973). The transformation of coordinates in the frequency domain is given by the following expression:

$$\mathbf{U}(\omega) \cong \sum_{j=1}^{J} Y_j(\omega)\boldsymbol{\phi}_j \tag{A.8}$$

where Y_j is the frequency response function of the jth generalized coordinate, and the mode shape is given by the eigenvalue problem for the structure on fixed base:

$$\mathbf{k}\boldsymbol{\phi}_j = \omega_j^2\mathbf{m}\boldsymbol{\phi}_j \tag{A.9}$$

In this study, the vibration properties of the superstructure in each translational direction were used in Equation A.8.

Substituting Eq. A.8 into Eq. A.7, premultiplying by $\boldsymbol{\phi}_j$, using the orthogonality property of $\boldsymbol{\phi}_j$ with respect to $\mathbf{m}$ and $\mathbf{k}$, and assuming proportional damping in the structure, gives:

$$M_i(-\omega^2 + 2i\xi\omega_i\omega + \omega_i^2)Y_i - \omega^2 L_i^h U_0 - \omega^2 L_i^r \Theta_0 = -L_i^h \tag{A.10a}$$

$$-\omega^2 \sum_{j=1}^{J} L_j^h Y_j + (-\omega^2 m_t + K_{VV})U_0 + (-\omega^2 L_t + K_{VM})\Theta_0 = -m_t \tag{A.10b}$$

$$-\omega^2 \sum_{j=1}^{J} L_j^r Y_j + (-\omega^2 L_t + K_{MV})U_0 + (-\omega^2 I_t + K_{MM})\Theta_0 = -L_t \tag{A.10c}$$

in which,

$$M_j = \boldsymbol{\phi}_j^T \mathbf{m} \boldsymbol{\phi}_j \tag{A.11a}$$

$$2M_j \xi_j \omega_j = \boldsymbol{\phi}_j^T \mathbf{c} \boldsymbol{\phi}_j \tag{A.11b}$$

$$\omega_j^2 M_j = \boldsymbol{\phi}_j^T \mathbf{k} \boldsymbol{\phi}_j \tag{A.11c}$$

$$L_j^h = \boldsymbol{\phi}_j^T \mathbf{m1} \tag{A.11d}$$

$$L_j^r = \boldsymbol{\phi}_j^T \mathbf{mh} \tag{A.11e}$$

The solution of Eq. A.10 can be obtained in a very efficient manner by first solving for the J generalized coordinates from Eq. A.10(a) in terms of the base displacements,

$$Y_i = H_i(\omega)\left[\omega^2 L_i^h U_0 + \omega^2 L_i^r \Theta_0 - L_i^h\right], \qquad i = 1,2,\ldots J \tag{A.12a}$$

where,

$$H_i(\omega) = \frac{1}{M_i(-\omega^2 + 2i\xi_i\omega_i\omega + \omega_i^2)} \tag{A.12b}$$

Substitution of Eq. A.12 into Eqs. A.10(b) and (c) gives two equations in terms of the two unknown base displacements.

$$\begin{bmatrix} S_{11} & S_{12} \\ S_{12} & S_{22} \end{bmatrix} \begin{Bmatrix} U_0 \\ \Theta_0 \end{Bmatrix} = \begin{Bmatrix} R_1 \\ R_2 \end{Bmatrix} \tag{A.13a}$$

where,

$$S_{11} = -\omega^4 \sum_{j=1}^{J} \left(L_j^h\right)^2 H_j - \omega^2 m_t + K_{VV} \tag{A.13b}$$

$$S_{22} = -\omega^4 \sum_{j=1}^{J} \left(L_j^r\right)^2 H_j - \omega^2 I_t + K_{MM} \tag{A.13c}$$

$$S_{12} = S_{21} = -\omega^4 \sum_{j=1}^{J} L_j^r L_j^h H_j - \omega^2 L_t + K_{VM} \tag{A.13d}$$

$$R_1 = -\omega^2 \sum_{j=1}^{J} \left(L_j^h\right)^2 H_j - m_t \tag{A.13e}$$

$$R_2 = -\omega^2 \sum_{j=1}^{J} \left(L_j^r\right)^2 H_j - L_t \tag{A.13f}$$

The solution of Eq. A.13 gives the frequency response functions U_0 and Θ_0 which can be substituted into equation A.12(a) to give the frequency response functions of the generalized coordinates. Finally Eq. A.8 gives the frequency response functions for displacements, from which transfer functions between two locations can be computed.

A.3 Impedance Functions for the Soil

The impedance functions for the soil region, K_{VV}, K_{VM}, and K_{MM}, are the harmonic forces acting on the foundation required to produce harmonic displacement U_0 and Θ_0.

In this analysis, the foundation is assumed to be a rigid, circular disk on a halfspace model of the soil. Impedance functions have been obtained for an elastic half–space (Veletsos and Wei, 1971). The coupling impedance, K_{VM}, is small compared to the K_{VV} and K_{MM}, and is neglected. The translational and rotational impedance functions can be expressed as:

$$K_{VV} = K_x\left[k_x + ia_0 c_x\right] \tag{A.14a}$$

$$K_{MM} = K_\theta\left[k_\theta + ia_0 c_\theta\right] \tag{A.14b}$$

where k_x, c_x, k_θ, and c_θ, are dimensionless coefficients that depend on the Poisson's ratio, ν, for the soil and the dimensionless frequency parameter,

$$a_0 = \frac{\omega r_0}{c_s} \tag{A.14c}$$

where ω is the excitation frequency, r_0 is the radius of the foundation, and c_s is the elastic S–wave velocity of the soil. K_x, and K_θ are the static translational and rotational stiffness of the foundation on the soil,

$$K_x = \frac{8G_s r_0}{2-\nu} \tag{A.15a}$$

$$K_\theta = \frac{8G_s r_0^3}{3(1-\nu)} \tag{A.15b}$$

Material damping in the soil has an important effect on the dynamic response of soil–structure systems. Although the impedance functions for a rigid, circular foundation on a viscoelastic halfspace have been determined (Veletsos and Verbic, 1973), in this investigation the approximate impedance functions are obtained from the elastic case (Wolf, 1985). The effect of hysteretic damping in the soil material on the impedance functions for the elastic halfspace is included using the correspondence principle (Bland, 1960). The viscoelastic impedance functions are obtained from the elastic impedance functions by replacing the shear modulus of the soil, G_s, by a complex–valued modulus:

$$G_s^* = G_s(1+2i\eta_s) \tag{A.16}$$

where η_s is the hysteretic damping coefficient for the soil, defined as:

$$\eta_s = \frac{1}{4\pi}\frac{\Delta W}{W} \tag{A.17}$$

in which ΔW is the area of the hysteresis loop when the soil is undergoing harmonic shearing deformation, and W is the maximum strain energy in a cycle of deformation. The damping coefficient is an increasing function of maximum strain in the soil (Seed and Idriss, 1970).

Using the correspondence principle, the approximate impedance functions for the viscoelastic halfspace model of the soil are:

$$K_{VV}^* = K_x^*\left[k_x^* + ia_0^* c_x^*\right] \tag{A.18a}$$

$$K_{MM}^* = K_\theta^*\left[k_\theta^* + ia_0^* c_\theta^*\right] \tag{A.18b}$$

and,

$$K_x^* = \frac{8G_s^* r_0}{2-\nu} = K_x(1+2i\eta_s) \tag{A.19a}$$

$$K_\theta^* = \frac{8G_s^* r_0^3}{3(1-\nu)} = K_\theta(1+2i\eta_s) \tag{A.19b}$$

$$a_0^* = \frac{\omega r_0}{c_s^*} = \frac{a_0}{\sqrt{1+2i\eta_s}} \cong a_0(1-i\eta_s) \tag{A.19c}$$

The complex shear modulus for the soil affects the impedance functions in three ways. The static stiffness coefficients are complex–valued as given in Eqs. A.19(a–b). Second, the dimensionless frequency parameter is complex–valued, a_0^*, as approximated in Eq. A.19(c). Finally, the dimensionless impedance coefficients are functions of a_0^* instead of a_0. Neglecting only the last effect (Wolf, 1985), the impedance coefficients for use in Eq. A.18 are:

$$k_i^* \cong k_i - \eta_s a_0 c_i, \quad i = x,\theta \tag{A.20a}$$

$$c_i^* \cong c_i + 2\frac{\eta_s}{a_0}k_i, \quad i = x,\theta \tag{A.20b}$$

A.4 Response Analysis Procedure

The response of a building–foundation–soil system can be computed once the frequency response functions for the generalized coordinate have been obtained from Eqs. A.12 and A.13 for excitation frequencies in the range of interest.

The generalized coordinates are given by the Fourier integral as a superposition of responses to individual harmonics of free–field ground acceleration,

$$Y_i(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} Y_i(\omega)\, \ddot{u}_g(\omega) e^{i\omega t}\, d\omega \tag{A.21a}$$

where $\ddot{u}_g(\omega)$ is the Fourier transform of the specified free–field acceleration $\ddot{u}_g(t)$:

$$\ddot{u}_g(\omega) = \int_{0}^{d} \ddot{u}_g(t) e^{-i\omega t}\, dt \tag{A.21b}$$

in which d is the duration of the ground motion. The Fourier integrals in Eq. A.21 are computed in their discrete form using the Fast Fourier Transform (FFT) algorithm.

The displacements in the building are computed from an expression similar to Eq. A.8, and the base shear and overturning moment can be computed from the modal contributions given by the generalized coordinates.

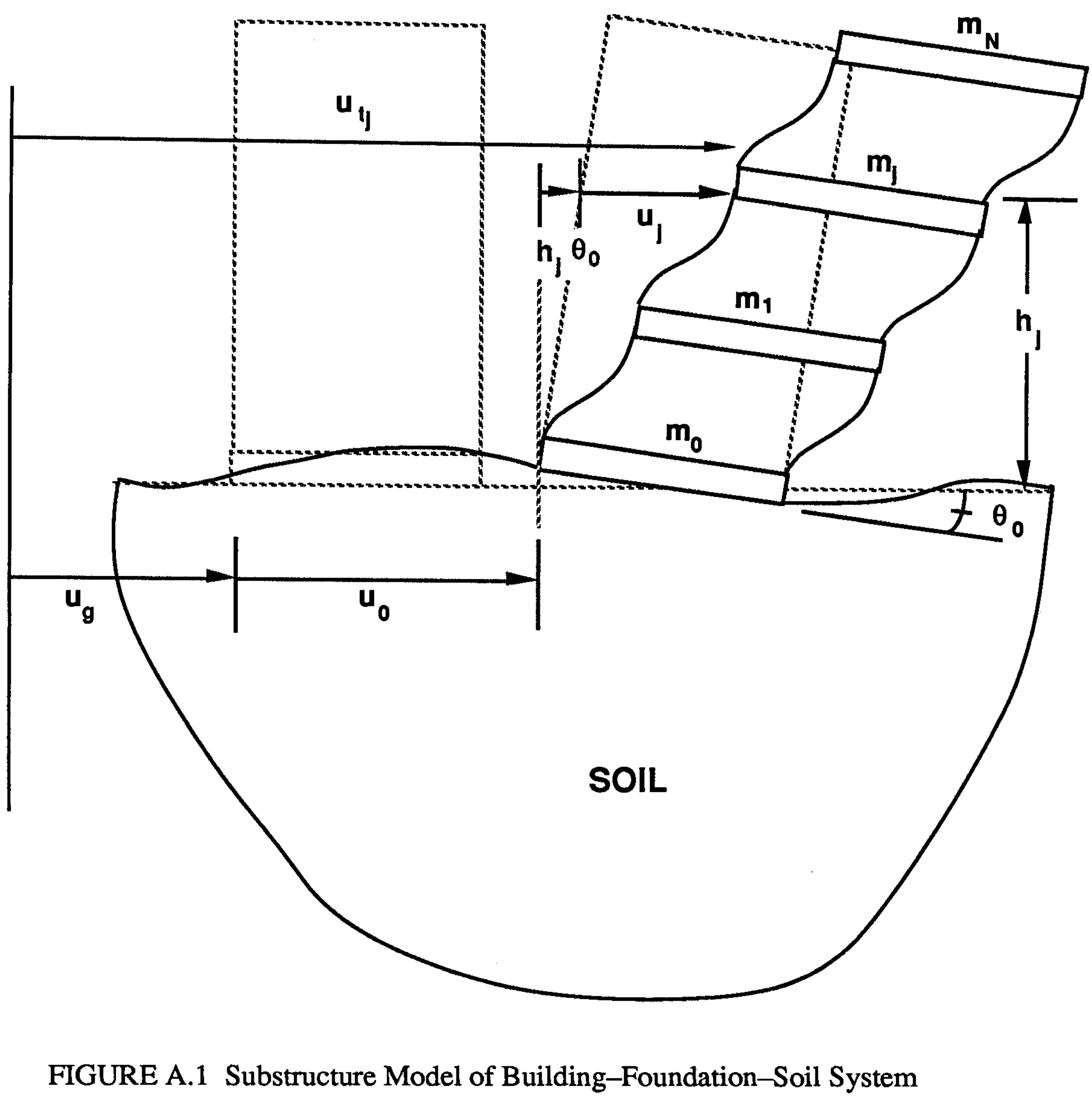

FIGURE A.1 Substructure Model of Building–Foundation–Soil System