Division of Structural Engineering, Mechanics and Materials

DYNAMIC SOIL-STRUCTURE INTERACTION IN BUILDING RESPONSE FROM EARTHQUAKE RECORDS

by

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ABSTRACT

Although a large number of theoretical studies have been carried out on the subject of the dynamic interaction between a structure and the soil region on which it is founded, and a variety of sophisticated models and analytical techniques have been proposed, there are limited studies on the evaluation of the interaction effects during actual earthquakes. The purpose of this report is to analyze the response records obtained from an instrumented building in recent earthquakes, to assess if and to what extent soil-structure interaction modified the response of the building, and to evaluate the validity of the concept of base shear reduction to represent the effects of soil- structure interaction in design.

The building under study is the Hollywood Storage Building, a 14-story reinforced concrete structure located in Los Angeles, California. It has been the subject of several investigation in the past aimed at evaluating the effects of dynamic soil-structure interaction. The accelerograms recorded at the building site during the Whittier Narrows earthquake of October 1, 1987 were the first useful records obtained since a recent significant upgrade of the building earthquake instrumentation. On these records the attention is focused in the present study.

An analytical model of the superstructure-foundation-soil system has been developed to help understand the dynamic behavior and to obtain some response quantities not directly available from the building records, particularly the base shear. The input quantities for the model have been obtained from the many available sources of data (original design drawings, results of vibration tests and soil mechanics boring and tests, previous studies found in the literature) and from the analysis of the response recorded at the building site during the different earthquakes. A three-dimensional mathematical model of the building superstructure has also been derived in order to obtain the vibration periods and mode shapes of the building on a fixed base.

It has been found that the effects of soil-structure interaction are significant in only one of the two main building directions. In that direction, the analytical model of the complete system has indicated a reduction of 16.6% of the base shear during the 1987 Whittier earthquake when compared to the case of the structure resting on a fixed base.

ACKNOWLEDGMENTS

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CHAPTER 1

INTRODUCTION

Dynamic soil-structure interaction is the overall effects that the presence of a structure at a certain site has on the local ground motion compared to the theoretical case in which the structure was not present. These effects are in same cases of significant importance in computing the response of a structure during an earthquake and can have noticeable implications in design. Intuitively, the influence of soil-structure interaction on the dynamic response of a structure depends on the properties of the structure as well as the local properties of the soil.

The ground motion at a site is also affected by the underlying soil layers and usually the motion developed at the ground surface has quite different characteristics from those developed on the bedrock or on nearby rock outcrops. These modifications constitute the so-called site response effects, and occur whether or not there are structures on the ground surface. It is important to clearly conceptually separate the site response effects from the soil-structure interaction effects. The present study is aimed to the evaluation of soil-structure interaction only, and the influence of the site response effects has not been considered.

When a structure is founded on rock, no effect of soil-structure interaction is present. Also if the structure is embedded, the motion will be practically constant in the rock over the height of the embedment. As the rock is very stiff, the transverse shear and overturning moment developed at the base of the structure due to its dynamic response induce no additional deformation in the supporting medium, and consequently no alteration of the input motion. When the soil is relatively soft with respect to the structure, the presence of the latter can significantly affect the incoming ground motion. It is important at this stage to distinguish between two different parts of soil-structure interaction, called kinematic and inertial interaction. The first one is due only to the structure embedment in the soil and to the flexibility of the foundation, while the second is a consequence of the deformation induced in the soil by the loads generated by the structural response.

If the structure is embedded, the embedment causes reflection and scattering of the seismic waves, which modifies the input motion. Furthermore, if the foundation is sufficiently rigid with respect to the soil, it will tend to move with a single motion, thus averaging the variations from point to point of the input motion on the underside of the foundation. This effect is particularly significant at high frequencies, as it affects the seismic waves whose length is comparable or smaller than the base dimensions. The effect of the embedment together with the base averaging effect constitute the kinematic part of soil-structure interaction.

On the other hand, the inertial loads applied to the structure lead to a transverse shear and overturning moment at the base which cause further deformation in the soil and, in turn, modify the input motion. This effect depends on the dynamic properties (mass, damping and stiffness) of both the structure and the soil and forms the inertial part of soil-structure interaction.

In general, to compute the response of a structure to an incoming ground motion taking into account the local soil conditions, all the above mentioned effects need to be included in the analysis, i.e. the site response effects, and both kinematic and inertial soil-structure interaction. If, however, the first two effects are neglected when the analysis is performed and only the inertial part of the interaction is taken into account in the analysis, the response of the structure in terms of base shear and overturning moment is usually smaller than the one obtained from a fixed-base analysis (Veletsos, 1977), which obviously neglects the influence of the soil. The fixed base analysis leads therefore to a conservative design. It must be noticed, however, that due to the foundation rocking, the displacement at the top of the structure with respect to its base may be larger when taking the inertial soil-structure interaction effects into account. The recommended seismic regulation for new buildings NEHRP (1986), derived from ATC-3 (1978), indicate a procedure which considers only the inertial part of interaction and allows a reduction of up to 30% in the design base shear. It is important to point out that this procedure neglects all the other effects discussed above.

The epicenter of the Whittier Narrows earthquake of October 1, 1987 was located in the Los Angeles metropolitan area, one of the most instrumented region in the world. The earthquake generated the largest set of strong ground motion records ever obtained from a single event. Some of these records were obtained in building structures and included the motion in a nearby station which can approximately indicate the "free-field" motion. The free-field motion is the ground motion that would occur at the site if the building was not present. Of the several structures with a free-field instrument, the Hollywood Storage Building, a 14-story reinforced concrete structure, was closest to the epicenter and the one which experienced the strongest shaking. This offers an important opportunity for studying soil-structure interaction effects in building response during earthquakes.

The response of the same building during four previous earthquakes has been recorded in the past fifty years. For two of these events (1952 Kern County earthquake and 1971 San Fernando earthquake) the accelerograms recorded at the site have been analyzed in several studies. Chapter 2 gives a general description and basic information on the Hollywood Storage Building, and a summary of these previous studies.

In Chapter 3 general information about the 1987 Whittier earthquake is given, and the upgrade of the building strong motion instrumentation, which took place in 1976, is described. Selected response functions obtained from the 1987 event are presented. They are compared with the similar functions obtained during the 1952 Kern County and 1971 San Fernando earthquakes in order to detect common tendencies about the modification of the parking lot motion due to soil-structure interaction. Data are also presented about the torsional response of the building, which were for the first time obtained during the 1987 event thanks to the improved number and more rational location of the strong-motion sensors.

In order to describe some response quantities not directly available from the earthquake records, particularly the base shear, an analytical model of the complete superstructure-foundation-soil system is developed. The next three Chapters describe the model, which is based on the substructure method, its rational basis and assumptions, and the methodology used to select the model parameters. In the substructure method the complete system is broken into two parts, the soil subsystem and the structure-foundation subsystem. These two parts can be modeled separately, also using completely different methods. The soil is usually modeled as an unbounded domain, and the force-displacement relationships of the degrees of freedom in common with the structure are determined. These so-called dynamic stiffness coefficients, or impedance functions, of the soil can be interpreted as generalized spring and dashpot. The structure is modeled indipendently, and it may be analyzed with much more detail than the soil subsystem. The final step of the analysis is to compute the response of the structure supported on the generalized spring-dashpot subsystem to the dynamic loading represented by the free-field motion.

In Chapter 4 the vibration properties of the building superstructure are obtained from a three-dimensional mathematical model. The assumptions and capabilities of the analysis program are described, together with the methodology used to model the building. The geometry and properties of the structural elements have been obtained from the original design drawings. The material properties have been selected such that the superstructure vibration periods correspond with the vibration periods derived from the response of the building to the 1987 Whittier earthquake. In Chapter 5 the analytical model of the complete system is presented and the equations of motion are developed. The model of the soil subsystem is also described, together with the procedure used to include hysteretic material damping in the soil.

Chapter 6 describes how the damping parameters for the building and soil were chosen. The frequency response functions at the basement and at three different floor levels are presented for the model and compared with the corresponding response functions for the recorded motion. The time history response of the model is then computed, and from this the base shear response and its maximum value is derived. This quantity is then compared with the maximum base shear obtained assuming that the structure is founded on a fixed base.

CHAPTER 2

PREVIOUS INVESTIGATIONS OF THE HOLLYWOOD STORAGE BUILDING

2.1 Description of the Building

The Hollywood Storage Company Building is located at 1025 North Highland Avenue, at Santa Monica Boulevard, in Los Angeles, California (Figure 2.1). It has been the subject of several earthquake investigations in the past. The availability of both free-field and building records obtained during earthquakes which occurred in the last fifty years, together with the structure's simplicity in design and isolation from other tall buildings, made it particularly suited to study soil-structure interaction effects.

The geometry of the building, designed and constructed in 1925, is shown in Figure 2.2. It is a fourteen-story, 149 ft (45.4 m) tall structure with a rectangular cross section, and it measures 217 ft (66.1 m) in the longitudinal EW direction and 51 ft (16 m) in the transverse NS direction. The lateral force resisting system consists of reinforced concrete frames in both directions. The two external longitudinal frames and the transverse westward frame are infilled with 8 in (0.20 m) thick shear walls. The vertical load carrying system is mainly formed by 8 in (0.20 m) thick concrete slabs supported by the frame columns through concrete capitals. In the three longitudinal bays on the west side of the building, one-dimensional slabs with joists supported by the transverse frames are present. A partial basement is located 9.0 ft (2.7 m) below the ground level. The foundation consists of reinforced concrete footings on Raymond concrete piles which vary in depth from 12 ft (3.7 m) beneath the footings at the end of the building to about 30 ft (9.1 m) near the center. Two large radio antennas were installed on the roof of the building after its construction, but were removed in 1954.

Data about the soil characteristics at the site of the Hollywood Storage Building have been obtained from a report by Duke and Leeds (1962). As shown in Figure 2.3, a soil boring to a depth of 300 ft (91.4 m) revealed that the building is founded on an approximately 100 ft (30.5 m) deep layer of soft, sandy clay with the unit weight varying from 110 lb/ft^3 (17.3 kN/m^3) at the surface to roughly 130 lb/ft^3 (20.4 kN/m^3) at the bottom of the layer. The measured P-wave velocity has a nearly constant value of 2400 ft/s (732 m/s) within the layer, except in the superficial shallow stratum of clay loam where it is 1090 ft/s (332 m/s). The sandy clay layer is underlied by approximately 7000 ft (2134 m) of Pleistocene (Quaternary) and Tertiary sedimentary formations, which in turn rest on the Santa Monica slate. The sedimentary formations consist mainly of sand and gravels probably deposited as fan material by streams originating from the south slope of the Santa Monica mountain.

2.2 Summary of Previous Studies

The fundamental periods of the building were first measured in August 1934 from ambient vibration observation under the California Seismological Program of the U. S. Coast and Geodetic Survey. As reported by Carder and Jacobsen (1936), it was found that the first period in the transverse direction was 1.2 s, and the one in the longitudinal direction 0.49 s. A complete set of forced vibration tests was carried out in December 1938 (Carder, 1964). The natural periods obtained from these tests are listed in Table 2.1, where it can be noticed that the fundamental translational periods coincide with those measured during the 1934 ambient vibration observations.

Los Angeles is a highly seismic area. From 1933 to 1987 several earthquakes occurred in this zone, and five of them triggered the strong motion accelerographs installed in the Hollywood Storage Building. Figure 2.4 is a map which shows the location of the building and the epicenters of these earthquakes, and Table 2.2 summarizes the characteristics of the earthquakes and the maximum recorded acceleration response at the building site.

The Southern California earthquake of October 2, 1933 ($M_L = 6.3$) triggered the two US Coast and Geodetic Survey triaxial accelerographs housed in the building, one in the basement and the other on the roof. The building was 24 miles (39 km) distant from the epicenter. The maximum peak acceleration recorded during the 1933 event was 0.03 g at the basement and 0.09 g at the top of the building.

In the subsequent years an additional triaxial accelerograph was installed in a small shelter structure located in the parking lot 139 ft (42.4 m) west of the main building and it was connected to the other two with common starting and timing devices. The location of this instrument is shown in Figure 2.1. This instrument was installed to obtain a ground motion record that, due to the distance from the main building, should not be affected by the dynamic response of the structure and its interaction with the soil. It should represent the so called "free-field" motion that would occur at the site if the building is not present. It has been shown, however (Trifunac, 1972), that the waves scattered from a rigid foundation can contribute significantly to the surface ground motion near the building within a distance proportional to a characteristic length of the foundation, especially at high frequencies. The parking lot instrument is less than one foundation length away from the building in the longitudinal direction, and it is very likely that the ground motion recorded by it includes reflections from the building. To obtain a more accurate free-field ground motion, the earthquake ground acceleration should be recorded at a great distance from the structure. But this would also increase the importance of local geological effects. For the purpose of this and most previous studies the tacit assumption is made that the parking lot instrument is representative of the free-field ground motion.

The first set of ground and building acceleration records was obtained during the Kern County earthquake of July 21, 1952 (M_L = 7.7). The epicenter was 75.8 miles (122 km) apart from the building, and because of this relatively large epicentral distance, the ground motion was not severe, the maximum acceleration being 0.06 g in the transverse direction. The pseudo-velocity response spectra of the basement and the parking lot record in the two directions, computed for zero damping, are shown in Figures 2.5 and 2.6. The spectra have been plotted from the data contained in the processed accelerogram records distributed by the California Institute of Technology (CIT) (Hudson, Trifunac and Brady, 1970 to 1975). These spectra differ from those reported by Housner (1957), who first analyzed the accelerograms recorded at the building site during the 1952 Kern County earthquake, although a general agreement in the shape and maximum values can be observed. The remarkable difference shown by Housner between the basement and the parking lot spectra in the longitudinal direction of the building was indicated as a consequence of the soil-structure interaction effects. This difference is not present in Figure 2.6, or at least is not evident as in Housner's paper. It has been clearly demonstrated by different studies (Hardin and Drnevich, 1972a and 1972b; Seed et al., 1986) that the equivalent dynamic stiffness of a soil sample decreases with an increase in the level of the excitation. Because of the low intensity experienced at the site of the building during the 1952 Kern County earthquake, the equivalent stiffness of the soil was not small enough, compared to the building stiffness, to produce significant interaction effects. As it will be seen later from the examination of the acceleration transfer function between the parking lot and the building basement, a general, though small, attenuation of the high frequency components was observed during the earthquake in the basement records when compared to the parking lot records, the attenuation occurring for periods shorter than 0.20 s in the transverse direction and 0.60 s in the longitudinal direction.

This low-pass filtering effect of the building basement is a consequence of the base-slab averaging effect and was also discussed by Housner (1957) from the examination of the pseudo-velovity response spectra, but is not so evident in Figures 2.5 and 2.6.

The same accelerograms recorded during the 1952 Kern County earthquake were later analyzed by Duke et al. (1970). Using the Fast Fourier Transform technique, Fourier spectra were computed for the two horizontal components of the records obtained at the roof, in the basement and at the parking lot. A smoothing process was applied to eliminate large fluctuations and make the spectra easier to interpret. By dividing the spectral values, the transfer functions between the different recording locations were obtained in both horizontal directions. Dynamic models of the building, the foundation and the subsurface conditions were developed. The theoretical transfer function between the motion at the base of the structure and in the parking lot in the longitudinal direction was computed using a solution by Luco (1969). The solution is rigorous in the case of an infinitely long elastic shear wall founded on a rigid embedded foundation of semicircular section excited by a plane horizontal shear wave travelling in the vertical direction. The motion is parallel to the direction of the shear wall and the soil is assumed elastic, isotropic and homogeneous. A good agreement was found between this theoretical transfer function and the one derived from the earthquake records. For period shorter than 0.20 s the agreement between the two transfer functions was particularly good, and both indicated a reduction of the high frequency component, which was again attributed to kinematic soil-structure interaction.

It is interesting to note that an analogous comparison carried out by Hardilek and Luco (1970) for the same infinitely long elastic shear wall, but founded on a surface flat foundation, gave a less satisfactory agreement between the two functions. This indicates that, particularly for periods shorter than 0.20 s, the presence of the partial basement and of the concrete piles causes the foundation to behave as embedded. It can therefore be important to take into account the embedment when modeling the building foundation in studying the kinematic soil-structure interaction effects.

The Borrego Mountain earthquake of April 8, 1968 (M_L = 6.5) triggered the instruments located in the parking lot and in the roof of the Hollywood Storage Building. Because of the large epicentral distance of 142 miles (229 km), the maximum acceleration was only 0.01 g in the free field and 0.04 g in the roof. Therefore insufficient data were obtained to study the effects of soil-structure interaction.

The epicenter of the San Fernando earthquake of February 9, 1971 (M_L = 6.4) was only 22 miles (35 km) from the building. In both the basement and the parking lot the maximum acceleration was recorded in the EW direction, and was 0.21 g in the former and 0.15 g in the latter. No record was obtained in the roof of the building, probably due to instrument malfunction.

Crouse and Jennings (1975) analyzed the data recorded at the Hollywood Storage Building during both the 1952 Kern County earthquake and the 1971 San Fernando earthquake. Using techniques similar to those previously used by Duke et al. (1970), they computed the modulus of the transfer functions between the parking lot and the basement in the two horizontal directions for the two events. These experimental transfer functions were then compared with those derived using a mathematical model consisting of a linear, viscously damped, fourteen-story spring-mass structure supported on a rigid circular foundation bound to a linearly elastic, homogeneous isotropic half-space. Although different choices were made of the base mass, of the structure's fixed base mode shapes and its modal damping ratios, only partial agreement was obtained between the theoretical and the actual transfer functions. The authors attributed these differences to the approximations involved in modeling the building foundation as a rigid circular surface disk and in considering the parking lot record as the free-field motion.

The modulus of the same experimental transfer functions analyzed by Crouse and Jennings (1975) have been computed from the CIT data and are shown in Figures 2.7 through 2.10. It can be observed that they fluctuate around the value one in the long period range, down to approximately 0.20 to 0.30 s in the transverse direction and 0.60 s in the longitudinal direction. For shorter periods, the modulus of the transfer functions is generally less than unity, particularly for the 1971 San Fernando earthquake. As already noted earlier, this effect is primarily due to kinematic soil-structure interaction.

Newmark, Hall and Morgan (1977) proposed a simple method to model the low-pass filter effect of the building foundation. Their approach was based on averaging the free-field acceleration record over a transit time interval, τ, corresponding to the time required by the incoming waves to travel along an effective foundation dimension. A window of width τ is moved along the acceleration time history, and an average acceleration assigned to the mid-point of the interval. The modified response spectrum is then calculated using the average acceleration history. The comparison of these modified response spectra with the actual response spectra relative to both the horizontal components recorded in the basement of the Hollywood Storage Building during the 1952 Kern County earthquake and the 1971 San Fernando earthquake showed that a transit time of τ=0.08 s gives a good agreement between the theoretical and the experimental response spectra. This corresponds to an apparent horizontal seismic wave velocity of 1310 ft/s (400 m/s), assuming a geometric mean dimension of the building of 105 ft (32.0 m).

The same τ-averaging technique has been used by Whitley et al. (1977) to compute the horizontal base motion from the free-field record, taking into account both horizontal rotation and translation. The goal was to compare the results of the spectral response calculated from the record derived with the proposed method with those obtained from the actually recorded accelerations. The motion was determined by assuming that the ground wave transits a rigid base of given dimension. While fair agreement was obtained for the 1952 Kern County earthquake, the difference between the derived and the actual spectra were significant for the 1971 San Fernando earthquake. This fact was attributed by the authors to factors such as foundation distortion and soil-structure interaction effects not accounted for by the averaging procedure. It was also concluded that probably no significant torsional effects arise in the basement of the building during the two earthquakes studied.

To model the attenuation effect of the foundation on the high frequency components of the free-field motion, Shioya and Yamahara (1980) proposed a simple numerical low-pass filter. As shown by the comparison between the estimated input ground motion of the foundation and the actual motion at the basement, the proposed filter is able to predict with fairly good accuracy the foundation input motion of the building in both the horizontal directions from the free-field motion recorded in the parking lot during the 1971 San Fernando earthquake. According to the authors, the numerical filter is to be applied to the free-field acceleration time history. The effective motion thus obtained can then be used as input motion for a suitable model of the structure-foundation-soil system that is able to account for the inertial interaction effects. A more refined numerical filter which gives a better prediction of the building basement motion has been later proposed by Ishii, Itoh and Suhara (1984).

Using the information described above, it is possible to interpret the Figures 2.11 through 2.14, which show the absolute acceleration response spectra of the parking lot and the basement records obtained during the 1952 Kern County and the 1971 San Fernando earthquake, and to make a summary of the soil-structure interaction effects observed at the Hollywood Storage Building during the two earthquakes. The spectra are derived from the CIT data, and have also been previously presented by Chan et al. (1986). From the observation of these figures it is immediately noticed that the spectra relative to the parking lot and the basement closely follow each other for the 1952 Kern County earthquake, while a large difference exists in some range of periods for the 1971

San Fernando earthquake, indicating significant soil-structure interaction. Due to the low level of excitation experienced by the building during the first earthquake, the soil was relatively stiff with respect to both the foundation and the building, and therefore the effects of both kinematic and inertial interaction were not significant. In the second earthquake, for periods shorter than 0.30 s in the transverse direction and 0.60 s in the longitudinal direction, a considerable attenuation of the basement spectra when compared to the parking lot spectra is observed. This is a consequence of the base-slab averaging effect of the building foundation. In the longitudinal direction the building is relatively stiff compared to the soil and is therefore able, when vibrating at its fundamental period, to cause significant deformations in the soil which in turn modify the motion at the base. The radiation of energy of the waves propagating away from the structure results in an increase in the damping in the complete dynamic system and therefore a reduction of its response. The reduction of the basement spectra in the range of periods close to the fundamental period of the building in the longitudinal direction (0.50 to 0.60 s), shown in Figure 2.14, is for the most part due to this inertial interaction effect, although, as seen above, kinematic interaction is also present in this period range.

Mode of vibration	Translational (transverse direction) [s]	Translational (longitudinal direction) [s]	Torsional [s]
Fundamental	1.20	0.50	0.60-0.64
Second	0.37		0.17
Third	0.22		0.11
Others			1.0, 0.20

Table 2.1 Natural periods of vibration of the Hollywood Storage Building as indicated by the 1938 forced vibration tests (Carder, 1964).

No.	Earthquake	Focal depth [miles (km)]	Local Magnitude [M_L]	Epicentral Distance [miles (km)]	Maximum recorded acceleration [g]					
					Parking Lot		Basement		Roof	
					tran.	long.	tran.	long.	tran.	long.
1	Southern California October 2, 1933		6.3	24 (39)	---	---	0.03	0.03	0.04	0.09
2	Kern County July 21, 1952		7.7	75.8 (122)	0.06	0.04	0.06	0.04	0.12	0.15
3	Borrego Mountain April 8, 1968		6.5	142 (229)	0.01	0.01	---	---	0.03	0.04
4	San Fernando February 9, 1971	5 (8)	6.4	22 (35)	0.17	0.21	0.11	0.15	---	---
5	Whittier Narrows October 1, 1987	8.7 (14)	5.9	16 (25)	0.21	0.12	0.12	0.06	0.21	0.20

--- indicates record not available

Table 2.2 Characteristics of the earthquakes recorded at the Hollywood Storage Building and maximum accelerations at the building site.

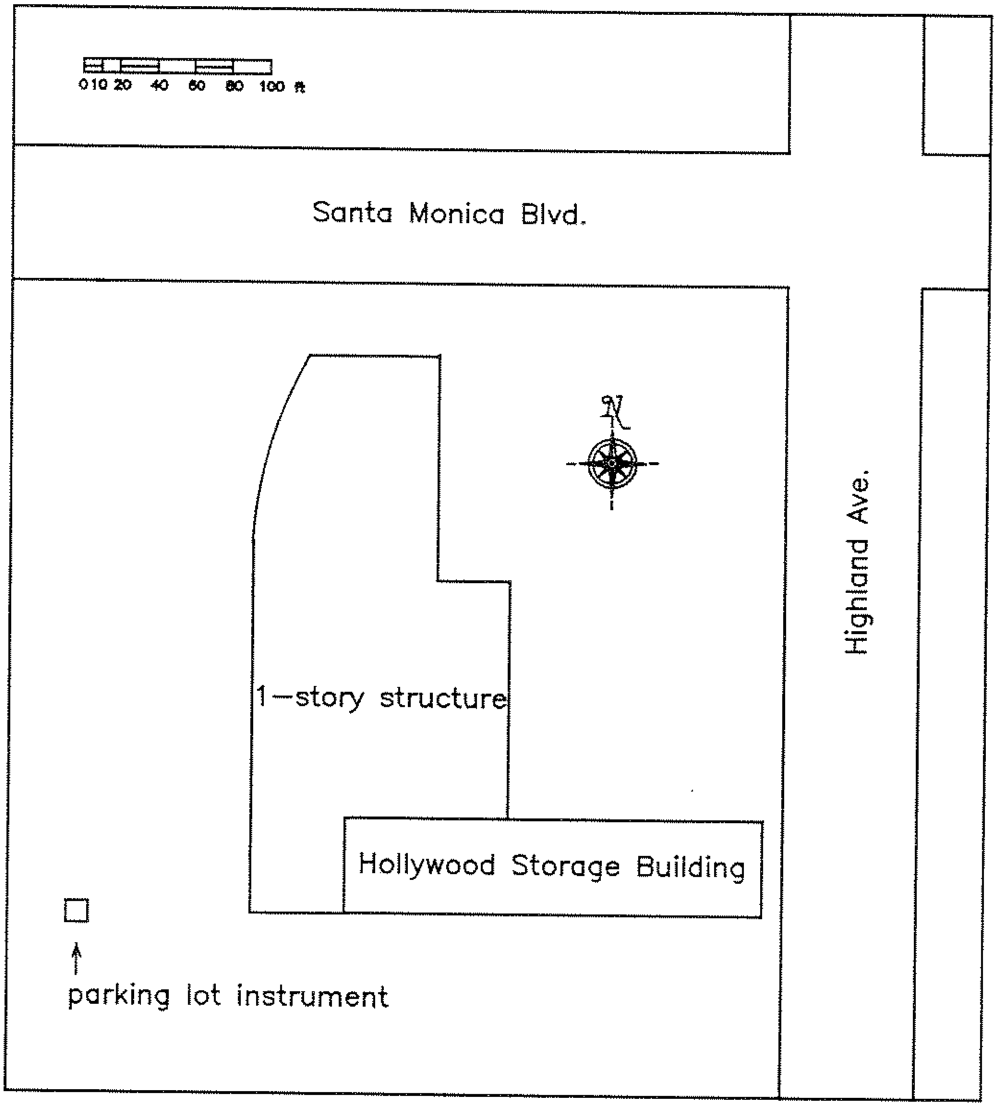

Figure 2.1 Location of the Hollywood Storage Building.

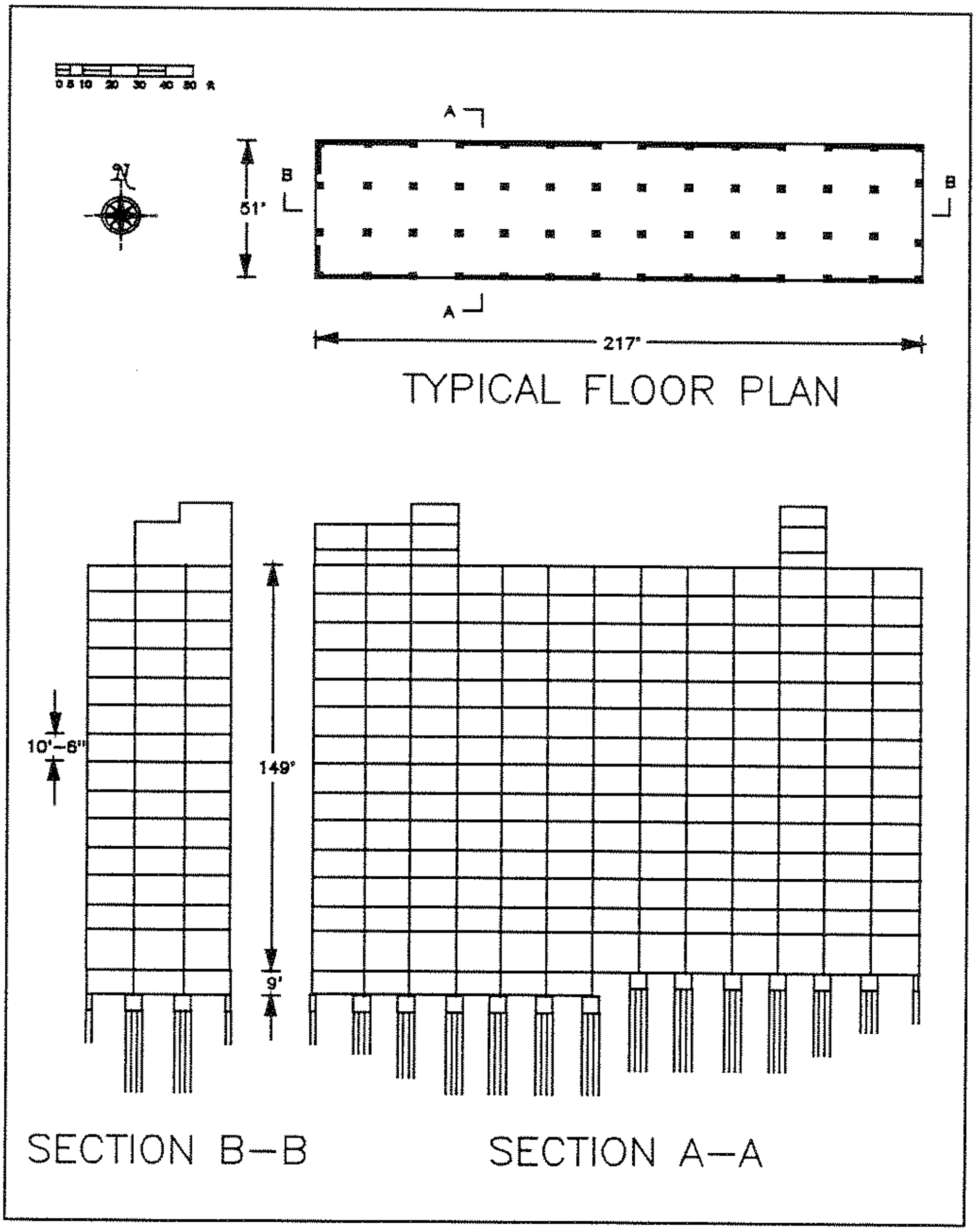

Figure 2.2 Sketch of the Hollywood Storage Building with its main features and dimensions.

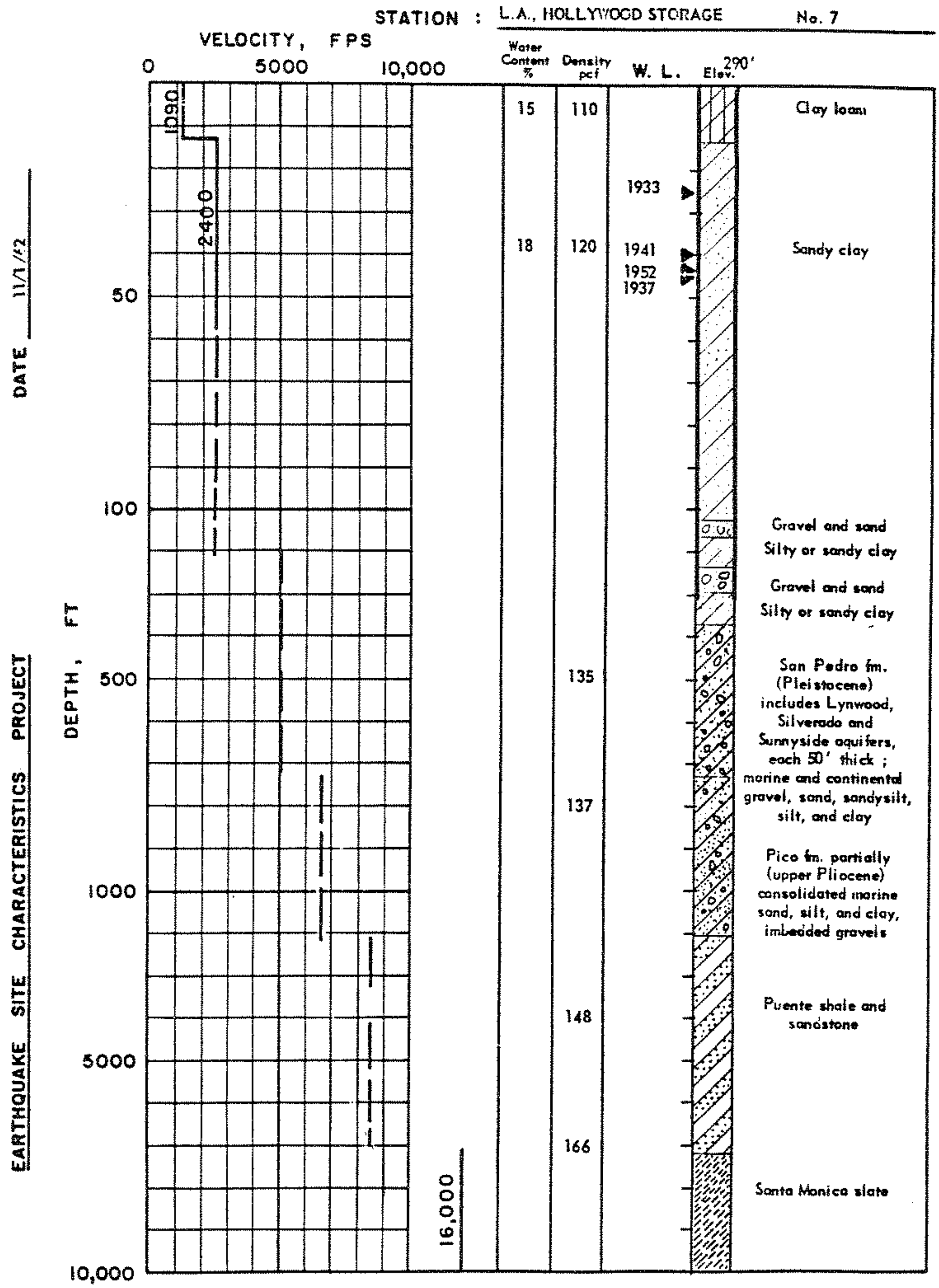

Figure 2.3 Site characteristics at the Hollywood Storage Building site (Duke and Leeds, 1962).

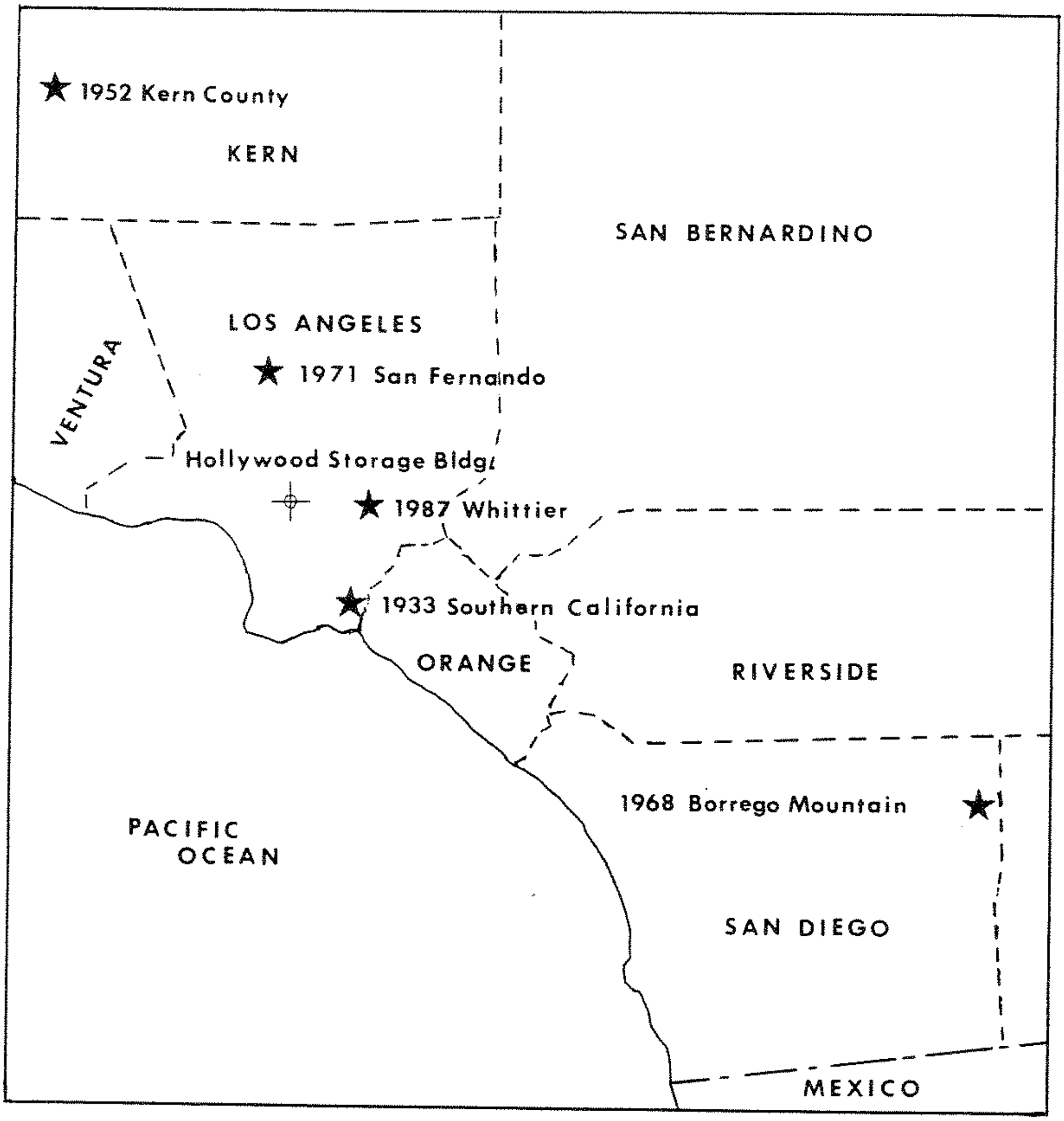

Figure 2.4 Location of Hollywood Storage Building and of epicenters of earthquakes recorded at the building site.

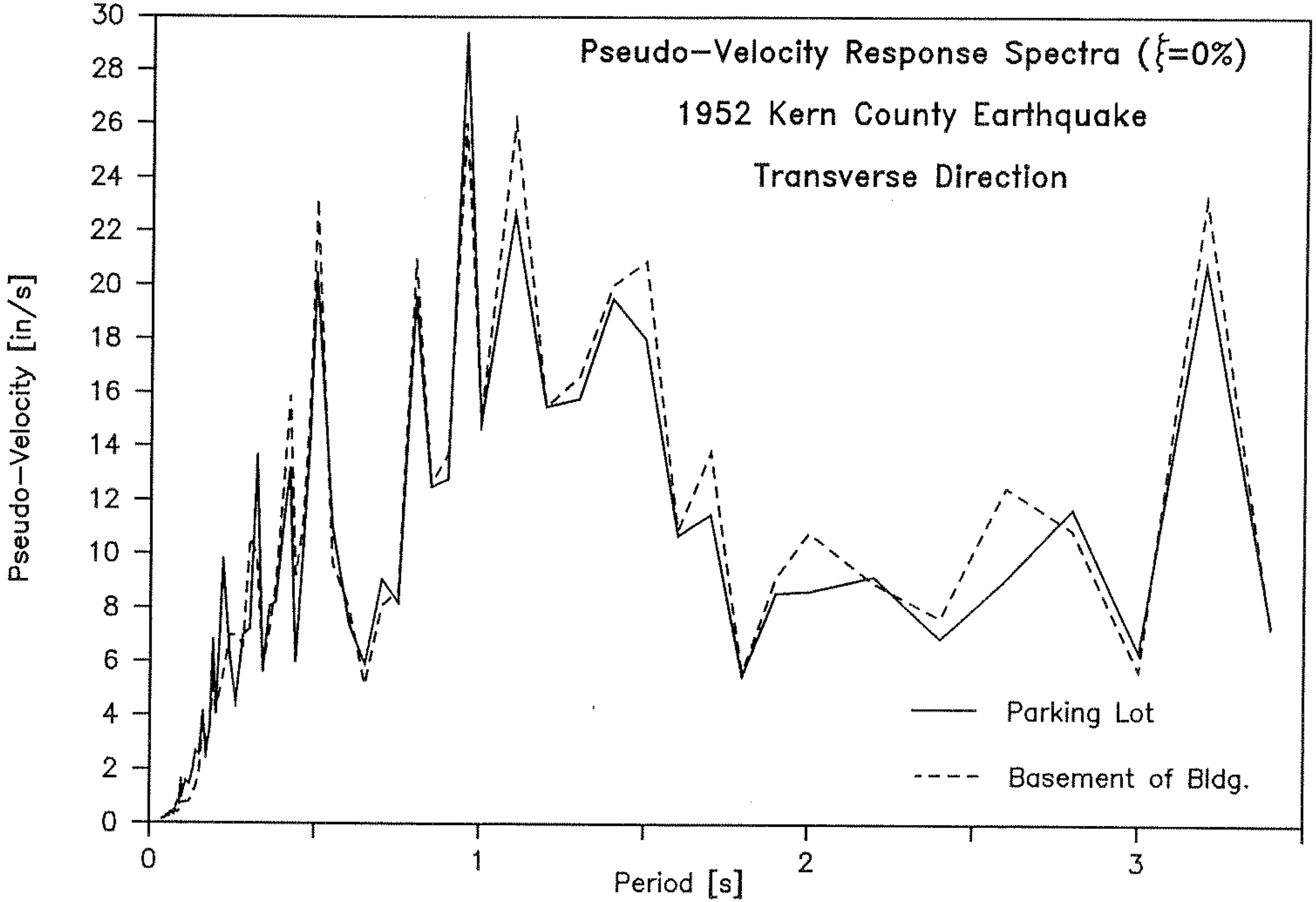

Figure 2.5 Pseudo-velocity response spectra of the parking lot and basement motions recorded in the transverse direction at the Hollywood Storage Building during the 1952 Kern County earthquake (damping ratio = 0%).

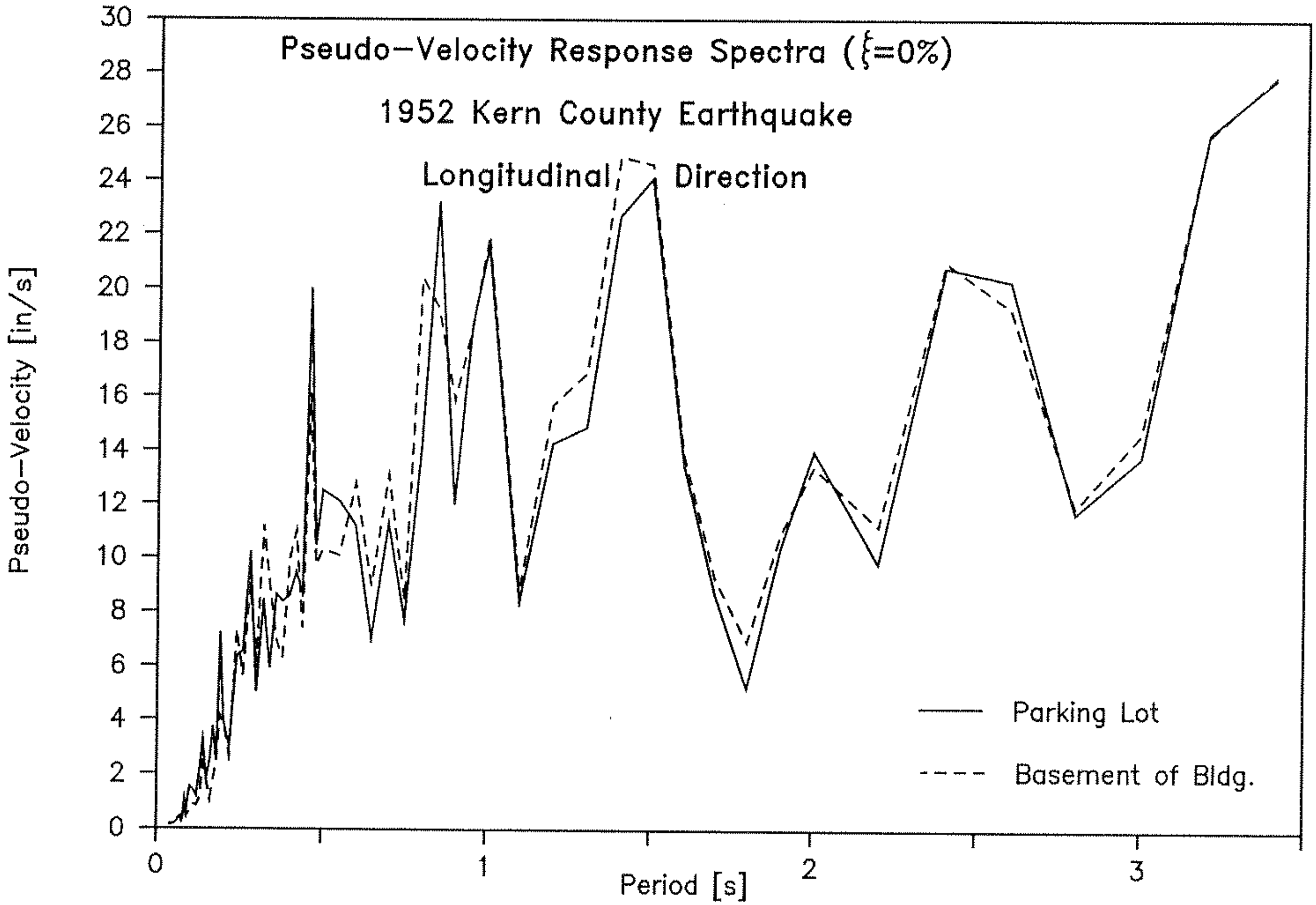

Figure 2.6 Pseudo-velocity response spectra of the parking lot and basement motions recorded in the longitudinal direction at the Hollywood Storage Building during the 1952 Kern County earthquake (damping ratio = 0%).

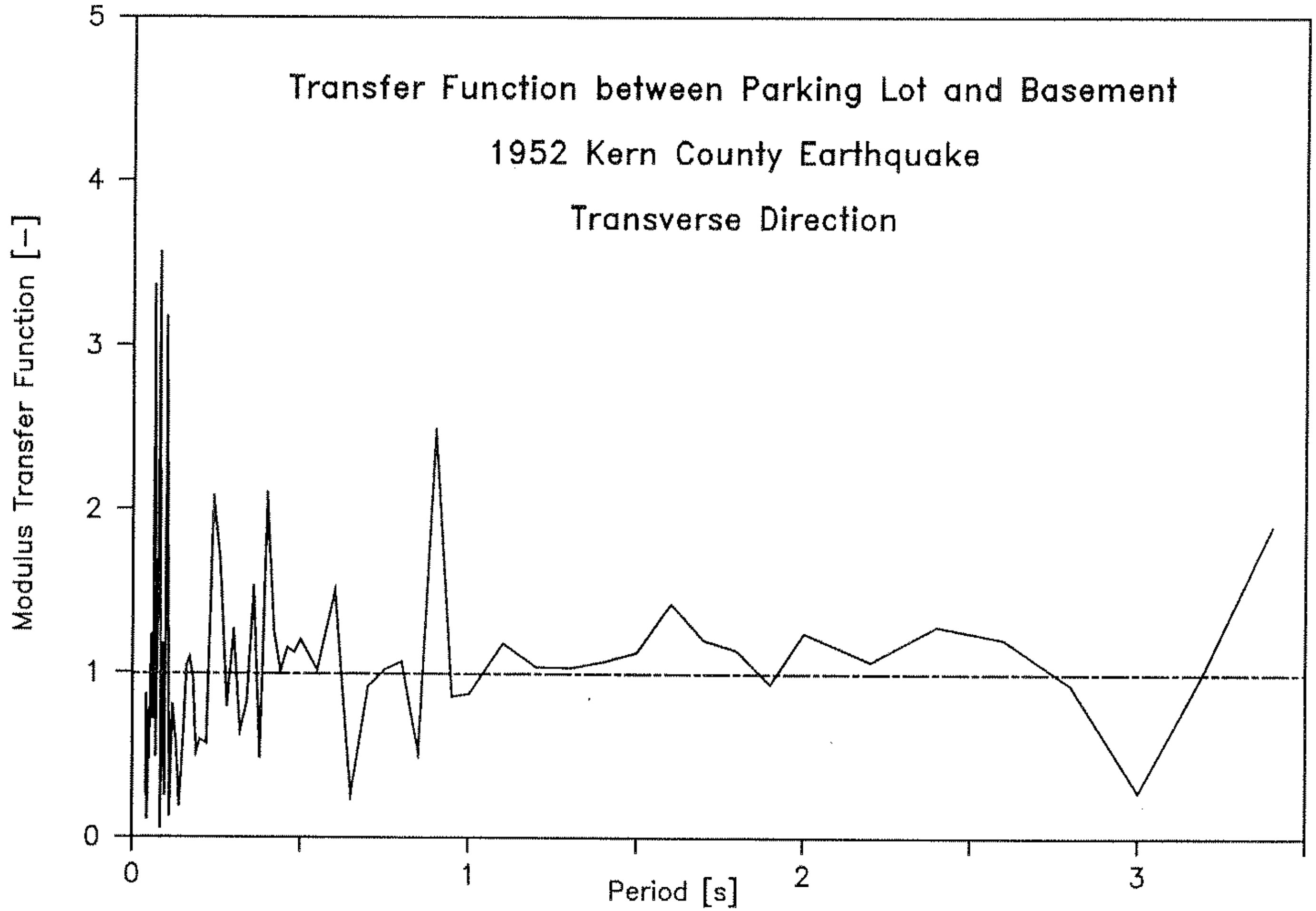

Figure 2.7 Modulus of the transfer function between the parking lot and the basement in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1952 Kern County earthquake.

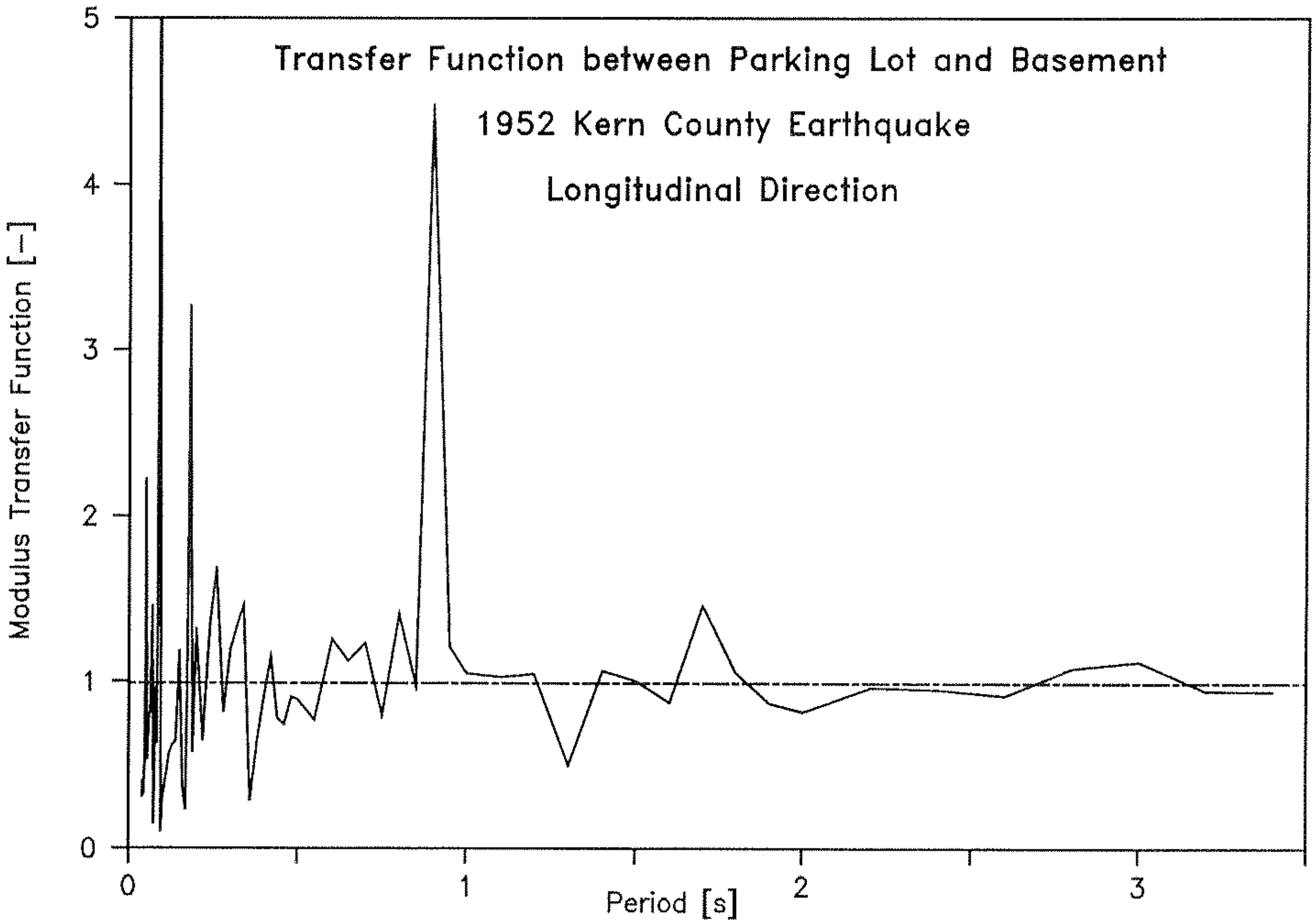

Figure 2.8 Modulus of the transfer function between the parking lot and the basement in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1952 Kern County earthquake.

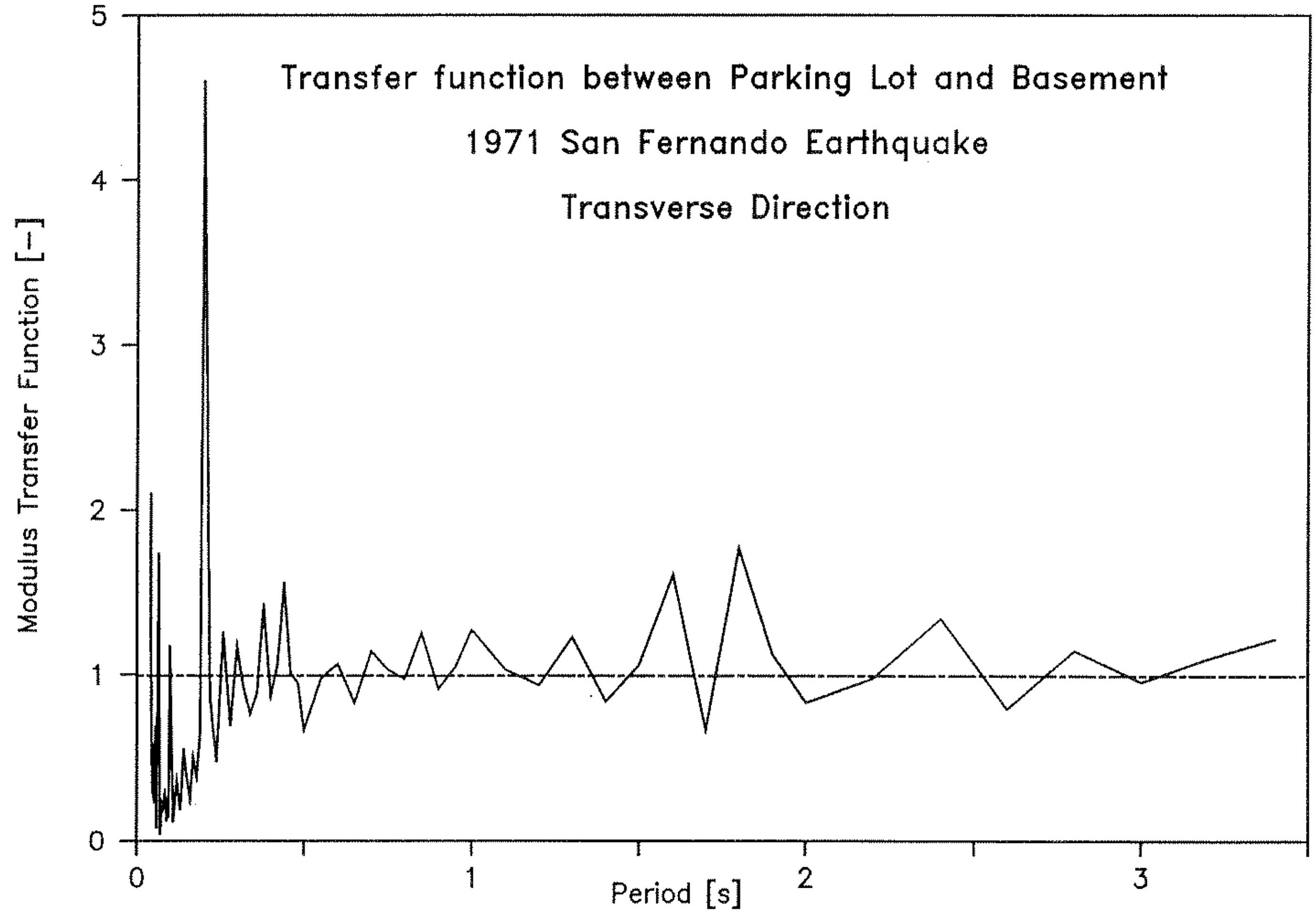

Figure 2.9 Modulus of the transfer function between the parking lot and the basement in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1971 San Fernando earthquake.

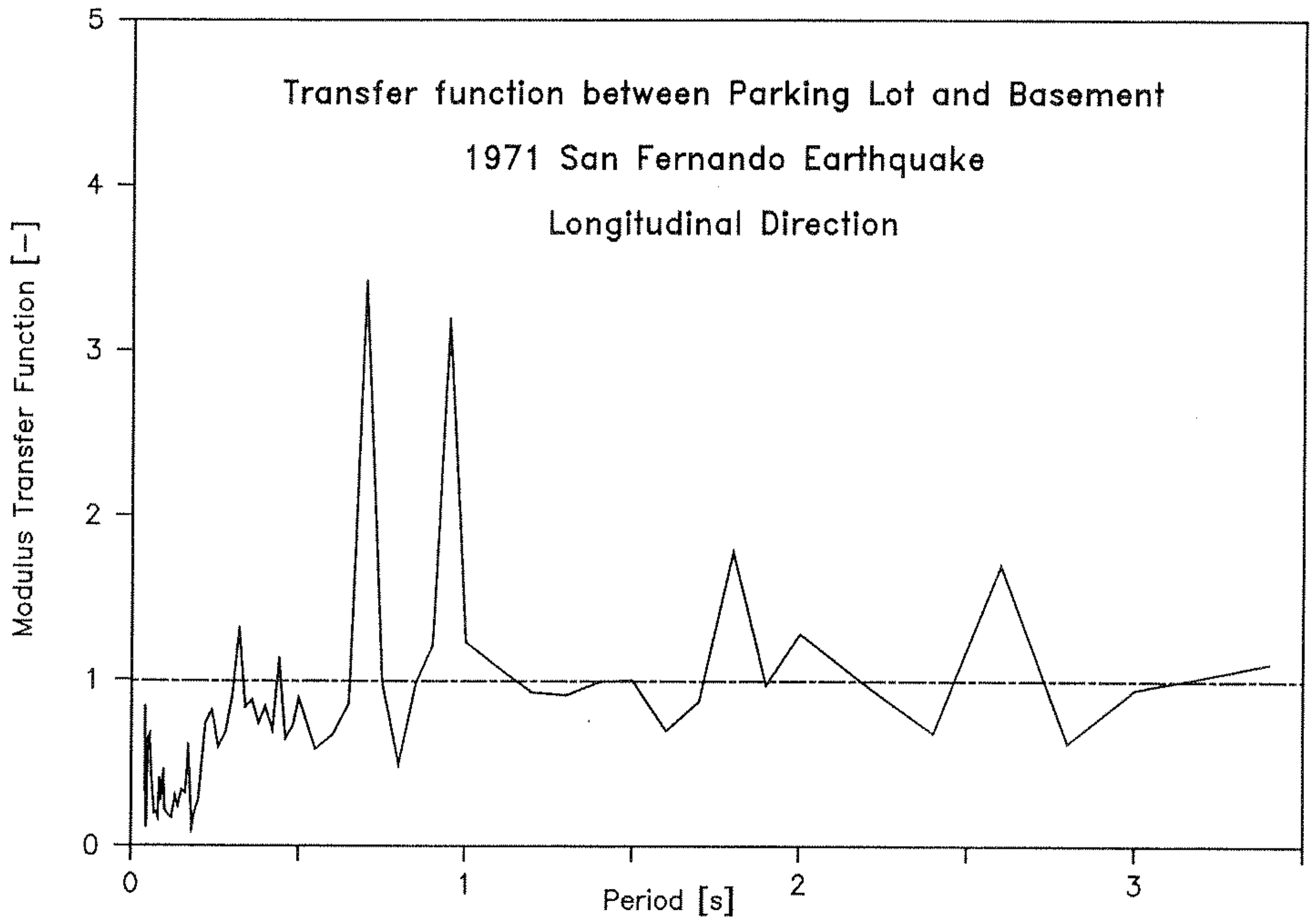

Figure 2.10 Modulus of the transfer function between the parking lot and the basement in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1971 San Fernando earthquake.

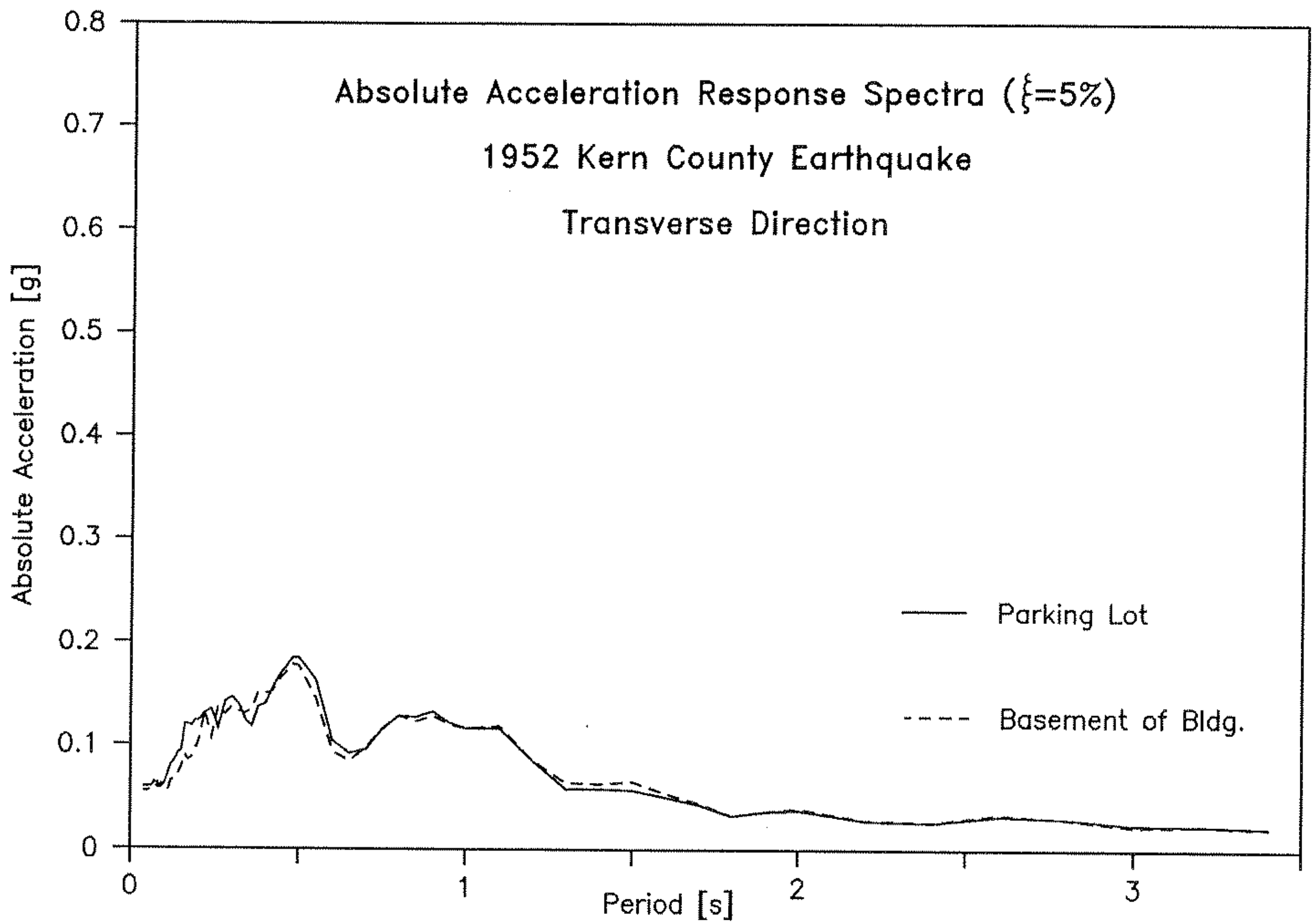

Figure 2.11 Absolute acceleration response spectra of the parking lot and basement motions recorded in the transverse direction at the Hollywood Storage Building during the 1952 Kern County earthquake (damping ratio = 5%).

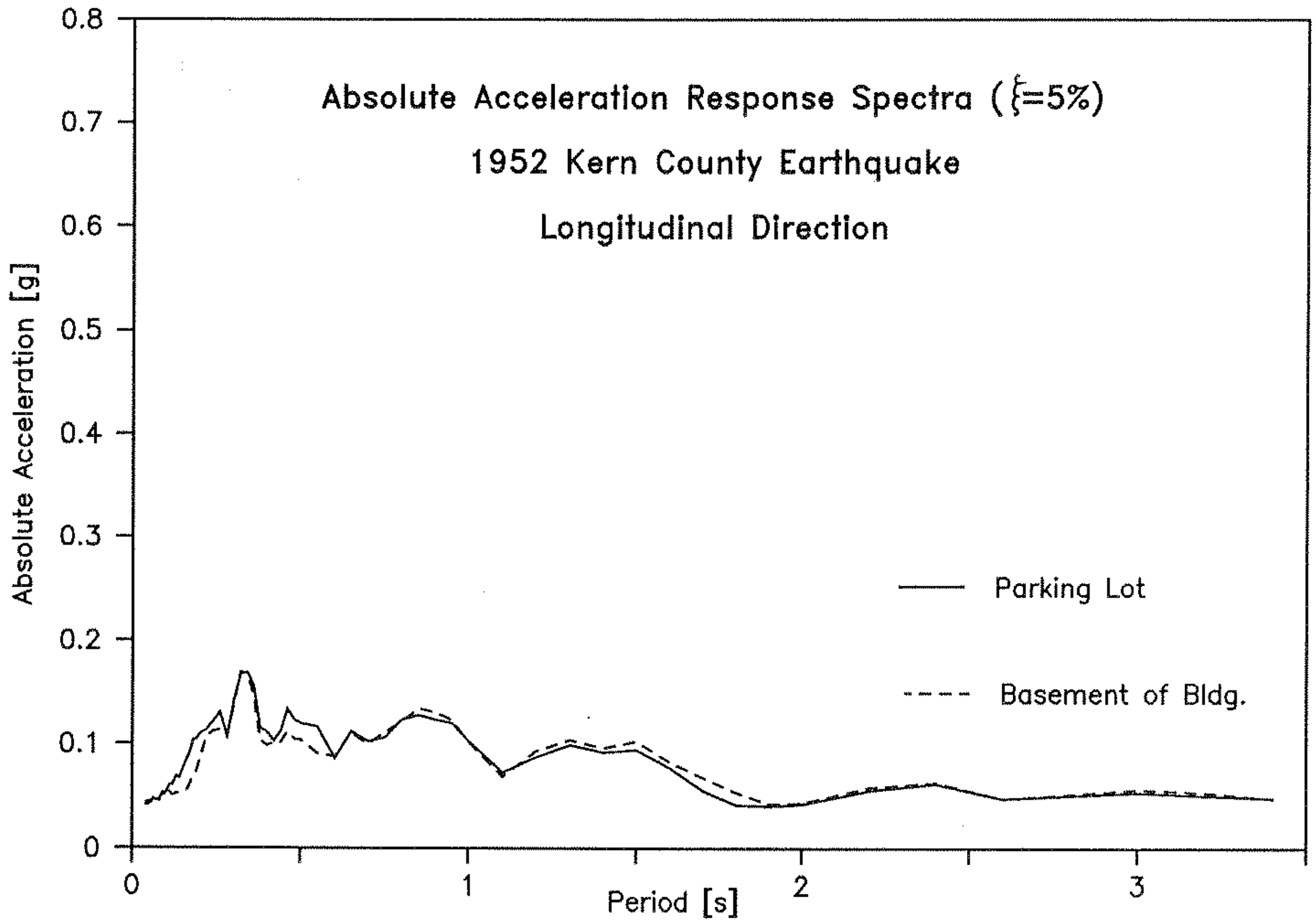

Figure 2.12 Absolute acceleration response spectra of the parking lot and basement motions recorded in the longitudinal direction at the Hollywood Storage Building during the 1952 Kern County earthquake (damping ratio = 5%).

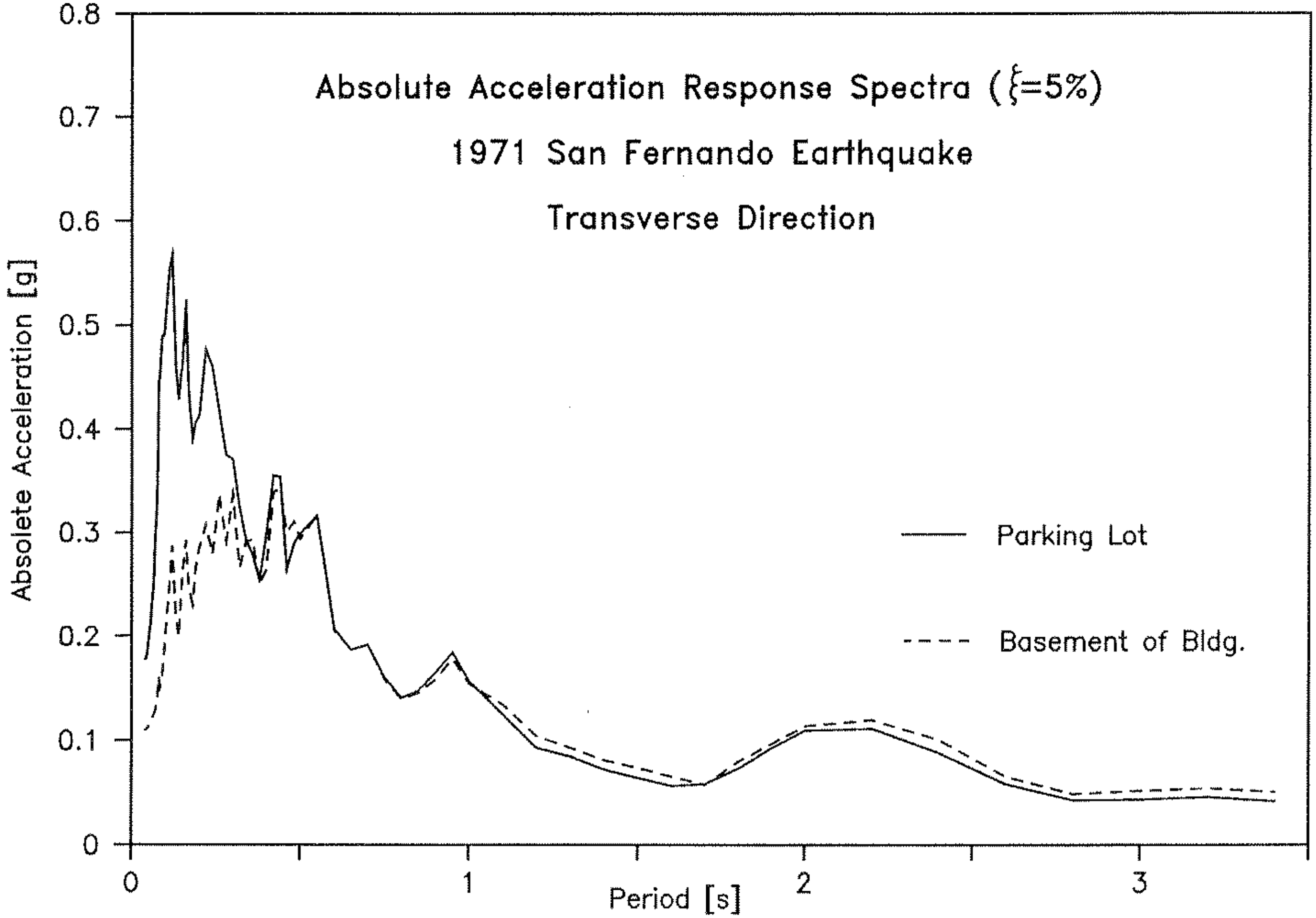

Figure 2.13 Absolute acceleration response spectra of the parking lot and basement motions recorded in the transverse direction at the Hollywood Storage Building during the 1971 San Fernando earthquake (damping ratio = 5%).

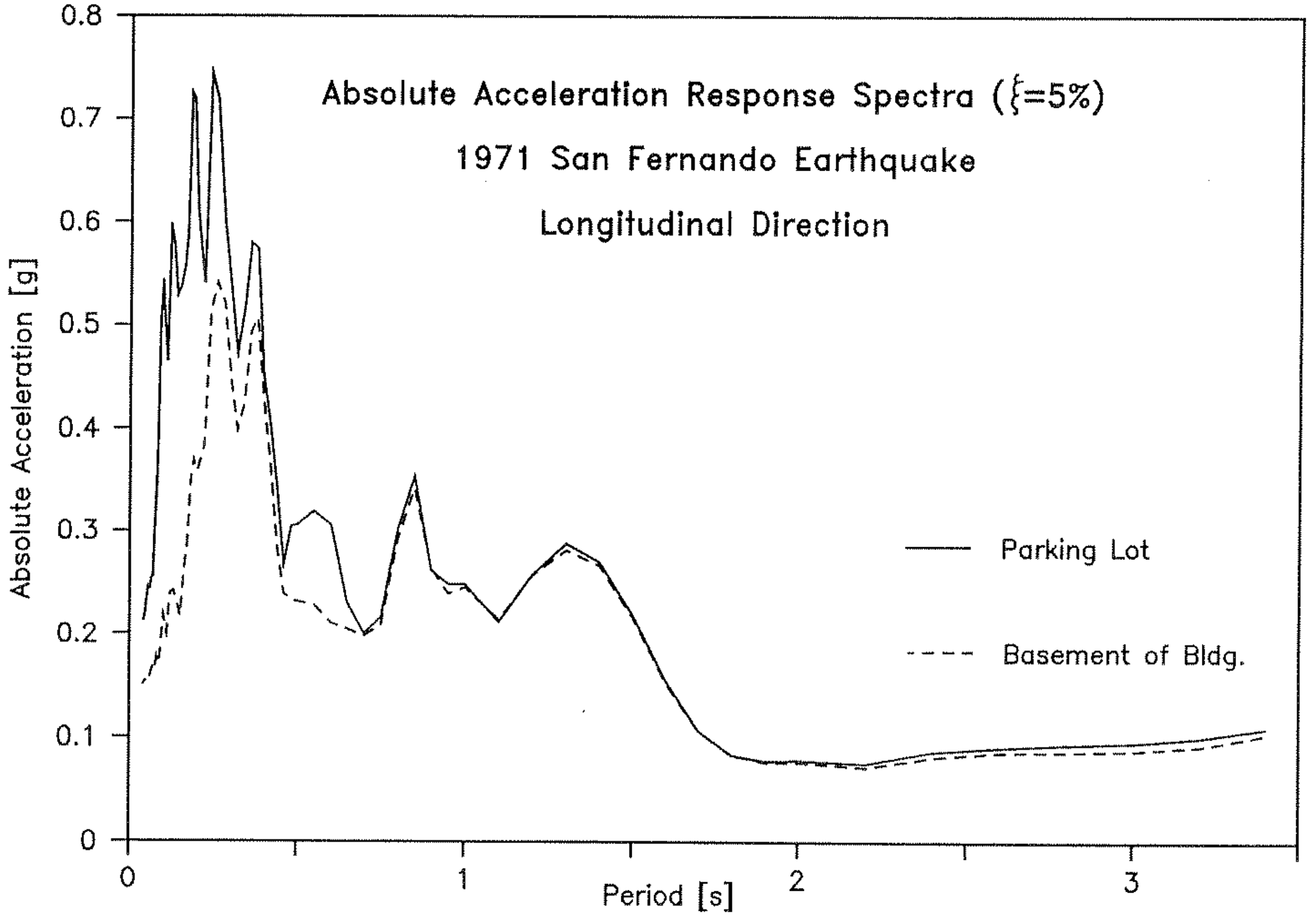

Figure 2.14 Absolute acceleration response spectra of the parking lot and basement motions recorded in the longitudinal direction at the Hollywood Storage Building during the 1971 San Fernando earthquake (damping ratio = 5%).

CHAPTER 3

PRELIMINARY ANALYSIS OF THE BUILDING RECORDS OBTAINED DURING THE 1987 WHITTIER EARTHQUAKE

3.1 The Whittier Earthquake of October 1, 1987

The October 1, 1987 Whittier Narrows earthquake occurred in the east Los Angeles metropolitan 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 to the north of the Whittier Narrows at a depth between 6.8 and 10 mile (11 to 16 km) (Hauksson et al., 1988). Several aftershocks followed this event, the largest of which (M_L = 5.3) occurred on October 4. Though the earthquake was of relatively moderate size, it represents the largest earthquake occurred west of the San Andreas fault in southern California since the 1971 San Fernando earthquake.

As the epicenter was located in one of the most instrumented areas in the world, the earthquake generated the largest set of strong ground motion records ever obtained from a single event. The earthquake triggered more than 250 accelerograph stations. The majority of the records were obtained by three networks in the Los Angeles area: the state-wide California Strong Motion Instrumentation Program (CSMIP) of the Division of Mines and Geology, the Los Angeles Network of the University of Southern California (USC), and the National Strong-Motion Instrumentation Network (NSMIN) of the United States Geological Survey. The CSMIP recovered data from 101 stations (Shakal et al., 1987), the USC from 68 stations (Trifunac, 1988), and the NSMIN from 52 stations (Etheredge and Porcella, 1987). In addition records were obtained from other small, specialized networks, as those maintained by the California Institute of Technology, Southern California Edison and other agencies.

Of the 101 CSMIP stations triggered by the earthquake, 63 are ground-response stations and 38 located in structures, which include 27 buildings, 8 dams, a suspension bridge, an airport control tower and a power plant. In eight of the instrumented structures, ground response stations were installed to provide data for soil-structure interaction studies. Among these, the Hollywood Storage Building was the one closest to the epicenter and the one in which the largest peak ground acceleration was recorded (0.21 g in the parking lot and 0.12 g in the basement). For this reason the building was selected as the subject of the present study to assess the effects of soil-structure interaction in the earthquake response of a typical building.

3.2 Preliminary Analysis of the Building Records

In April 1976 a number of sensors were added under the CSMIP to those already present in the Hollywood Storage Building. The structure and the nearby parking lot station were instrumented with a total of 15 sensors. The location of the instruments are indicated in Figure 3.1. With respect to the earlier sensor arrangement, it can be seen that the basement and the roof accelerographs were moved from the west end to the center of the building, and new instruments were installed at the center of the 8th and 12th floor levels. To obtain data about the torsional response of the building, three sensors were installed near the west wall center at the basement, at the 8th floor and at the roof. Unfortunately, only the instrument located in the basement (at the center) is able to record vertical motion in the building. Consequently it is not possible to obtain information about the rocking of the building directly from the earthquake records.

Using the CSMIP data recorded at the Hollywood Storage Building site during the 1987 Whittier earthquake, plots similar to those presented in Chapter 2 for the Kern County and the 1971 San Fernando earthquakes have been obtained. Figures 3.2 and 3.3 show the modulus of the transfer function between the parking lot and the basement in the transverse and longitudinal directions. As the previous two earthquakes (see Figures

2.7 to 2.10), the transfer functions fluctuate around the value one for periods longer than 0.30 s in the transverse direction and 0.60 s in the longitudinal direction. There is a general reduction of the transfer functions in the short period range, which can be attributed to the effects of kinematic interaction. The large spike present in Figure 3.3 for a period of 3.0 s is probably fictitious, and may be due to some distortion generated during the accelerogram correction process by the band-pass filter used, which has a constant value of one up to 2.5 s, and then reduces linearly to zero between 2.5 s and 5.0 s.

The absolute acceleration response spectra for the parking lot and the basement records are presented in Figures 3.4 and 3.5. A strong resemblance with the corresponding spectra obtained from the 1971 San Fernando earthquake (Figures 2.13 and 2.14) can be observed. The records obtained at the building site during the two earthquakes had a very similar frequency content. Kinematic interaction is responsible for the strong attenuation of the response values in the basement observed in both directions for periods shorter than approximately 0.20 s. The other reduction observed in the basement spectra in the longitudinal direction in the range of periods from 0.48 to 0.65 s is due to inertial interaction effects, as will be described in detail in subsequent chapters of this report.

Fourier amplitude spectra of the torsional angular acceleration at the basement, at the 8th floor and at the roof level have been computed, and are shown in Figure 3.6. Assuming a rigid floor diaphragm, the torsional acceleration Fourier spectrum of the i-th floor diaphragm is given by:

$$\ddot{\Theta}^i(T) = \frac{\ddot{U}^i_w(T) - \ddot{U}^i_c(T)}{d} \tag{3.1}$$

where T is the period, $\ddot{U}^i_w(T)$ and $\ddot{U}^i_c(T)$ are the acceleration Fourier spectra relative to the records in the transverse direction at the west wall and center of the floor respectively, and d is the distance between the two recording instruments. Figure 3.6 clearly indicates that, though the torsional response input at the basement level was very small, a torsional response was present at the 8^{th} floor and roof level. This implies that stiffness and mass eccentricities were present in the building superstructure at the time of the earthquake. A prominent response peak is observed for both elevation levels at 0.95 s, indicating a fundamental torsional period of the system.

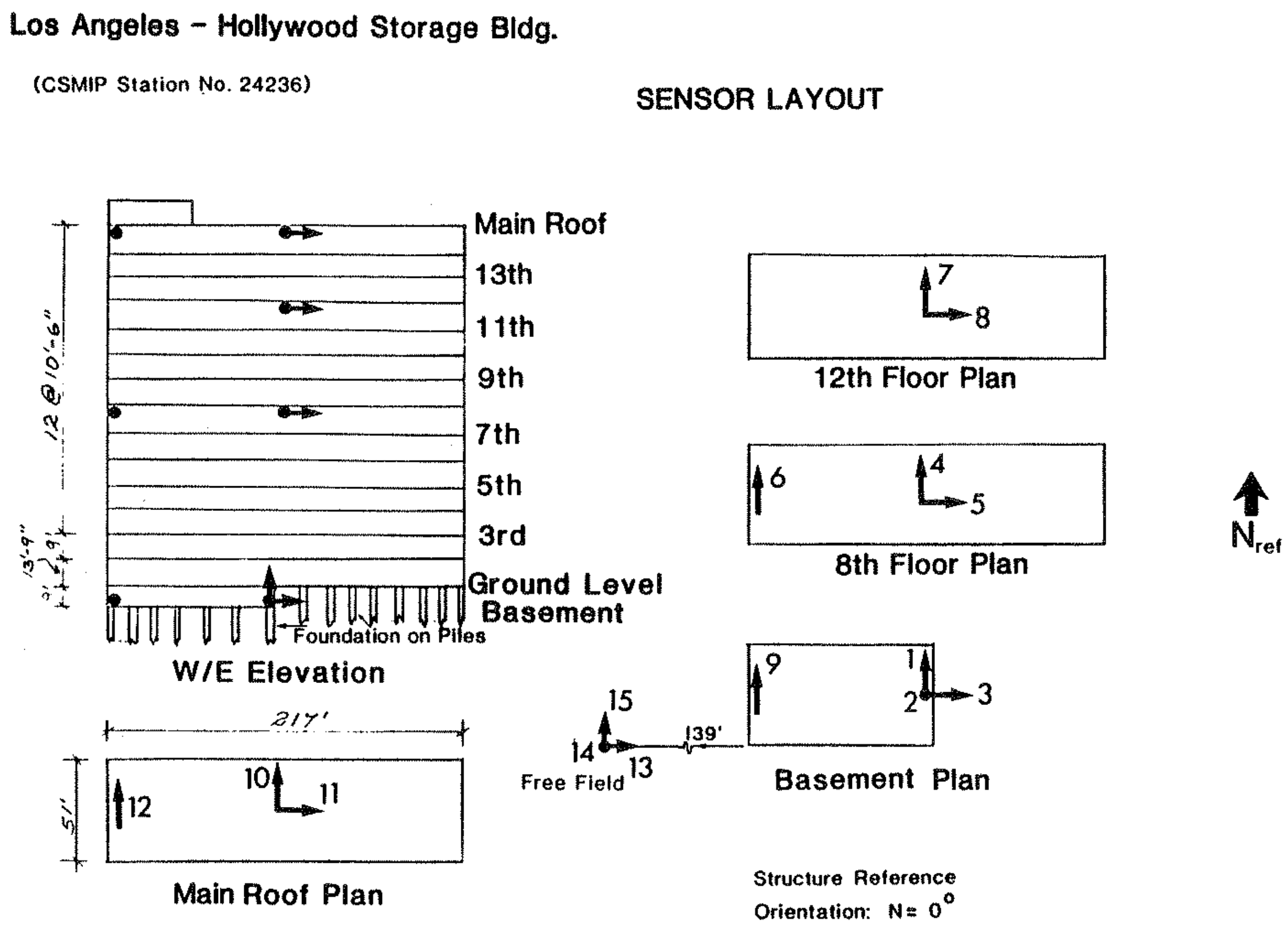

Figure 3.1 Locations of the sensors in the Hollywood Storage Building after April 1976 (Shakal et al., 1987).

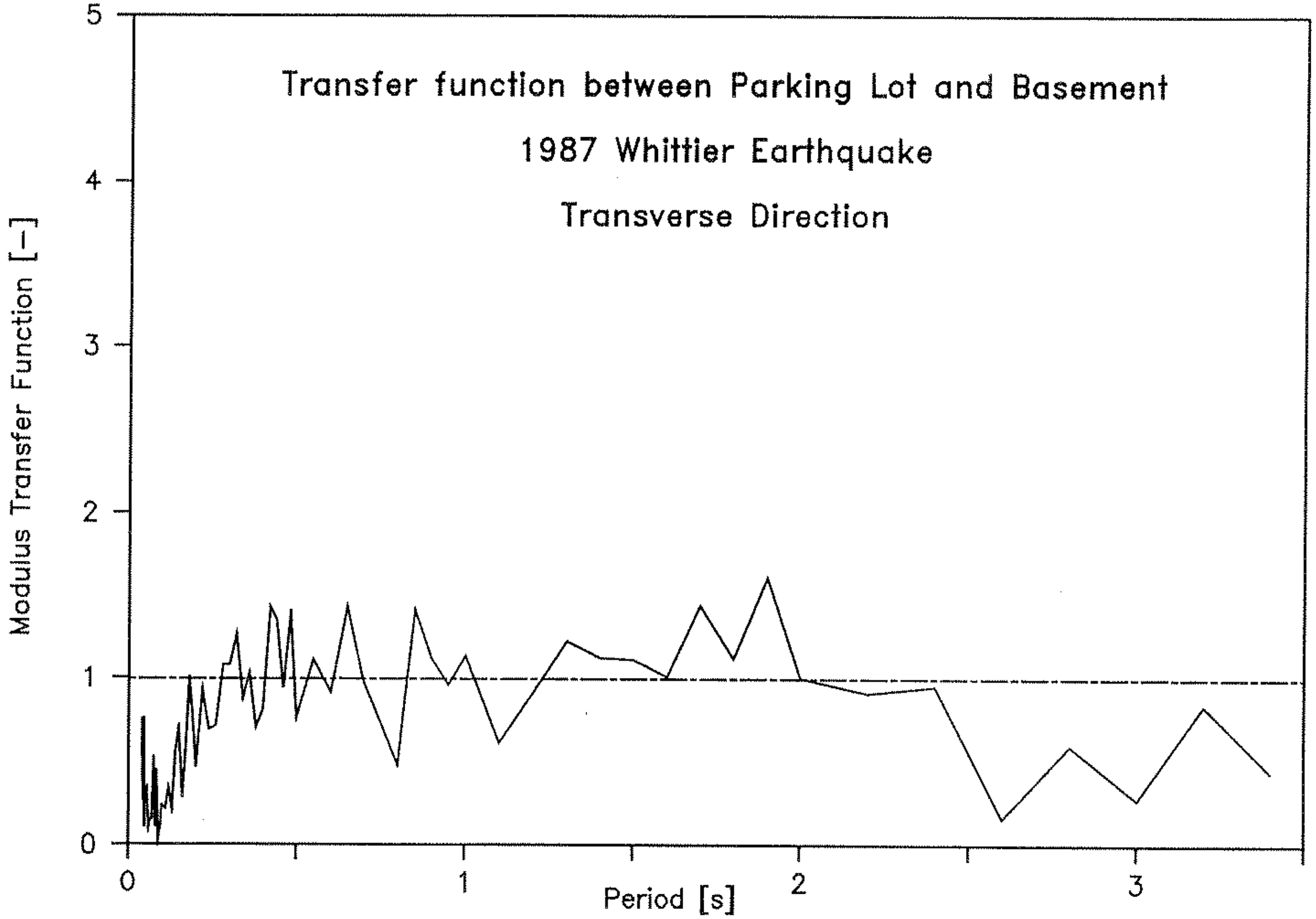

Figure 3.2 Modulus of the transfer function between the parking lot and the basement in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

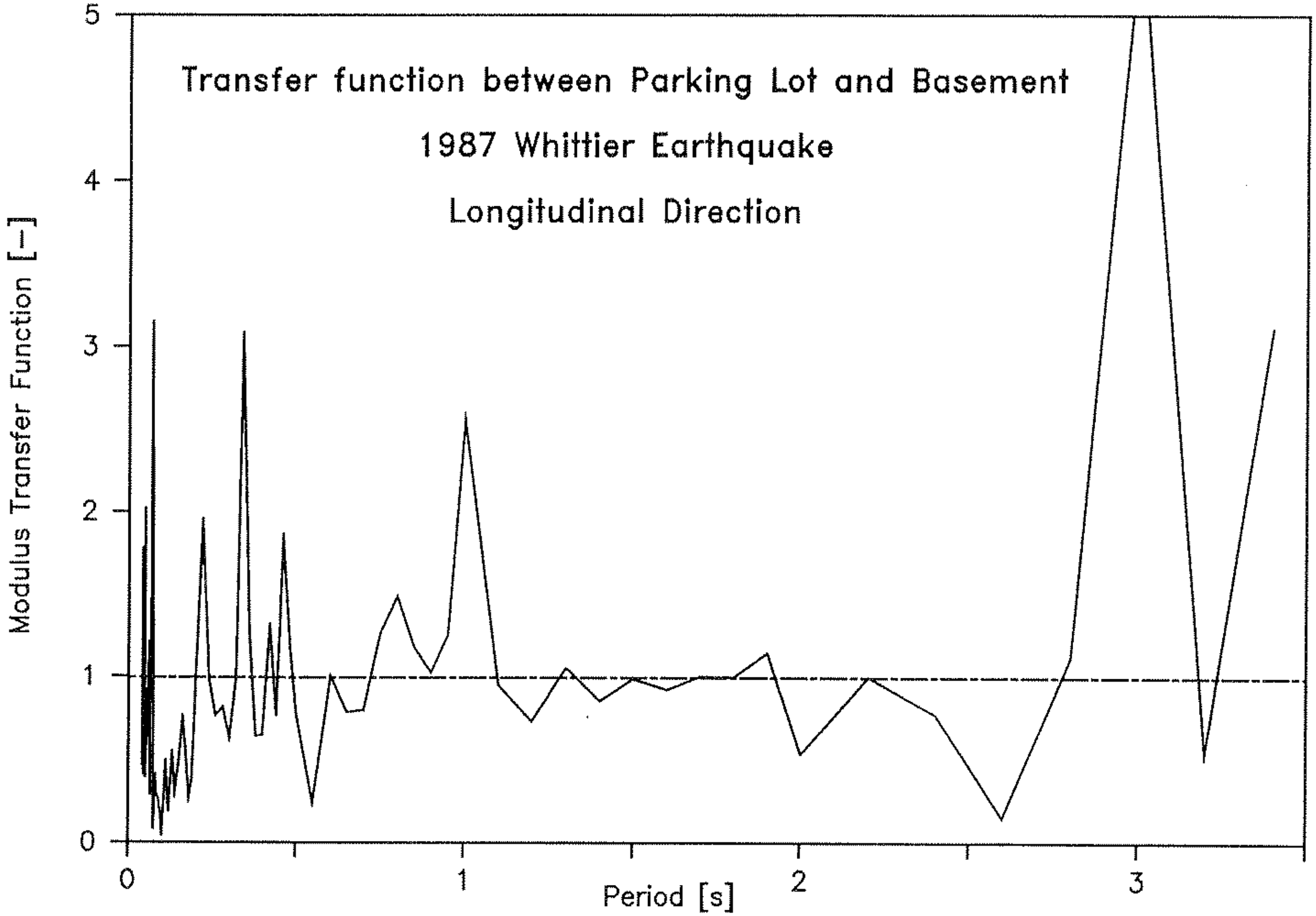

Figure 3.3 Modulus of the transfer function between the parking lot and the basement in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

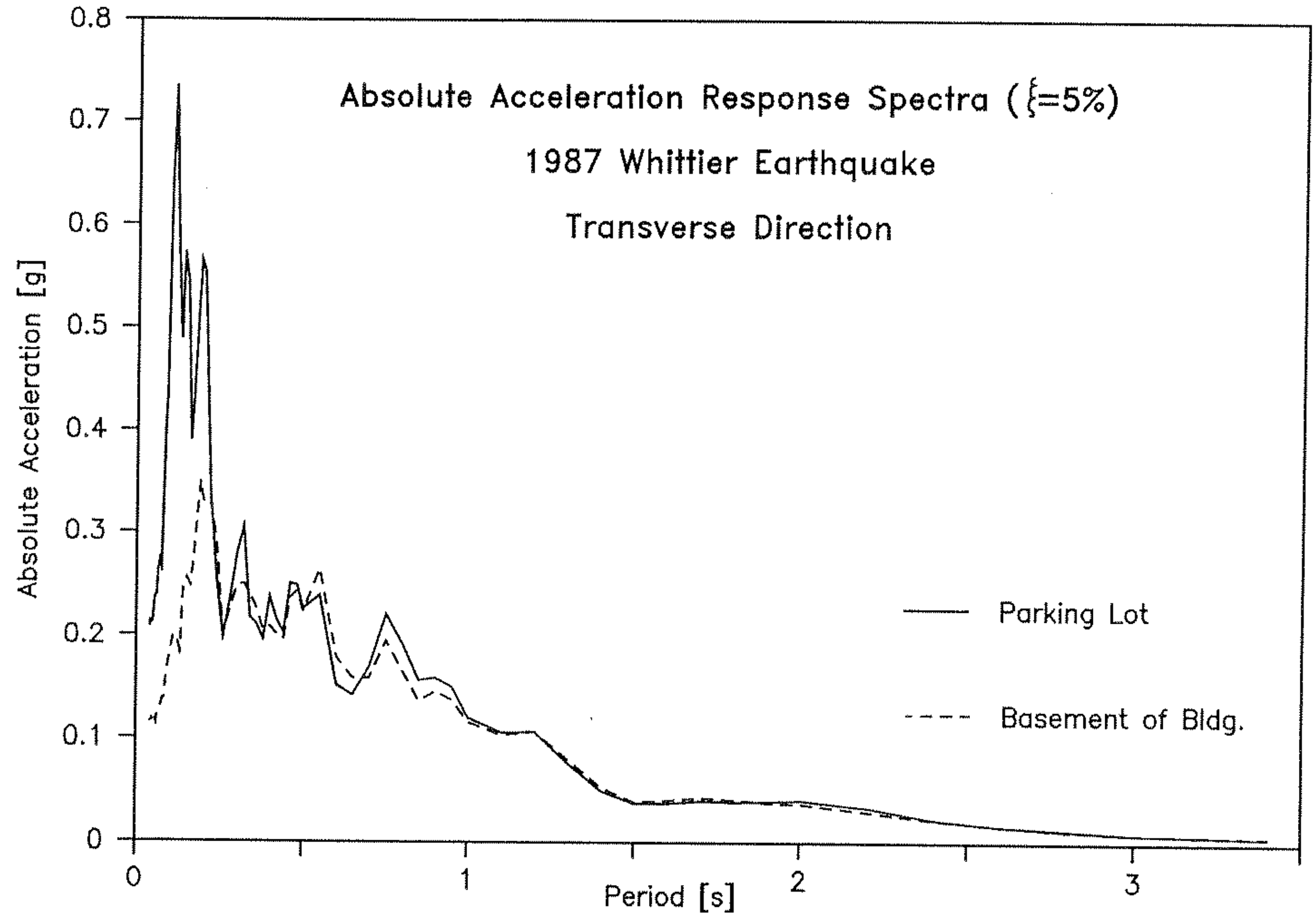

Figure 3.4 Absolute acceleration response spectra of the parking lot and basement motions recorded in the transverse direction at the Hollywood Storage Building during the 1987 Whittier earthquake (damping ratio = 5%).

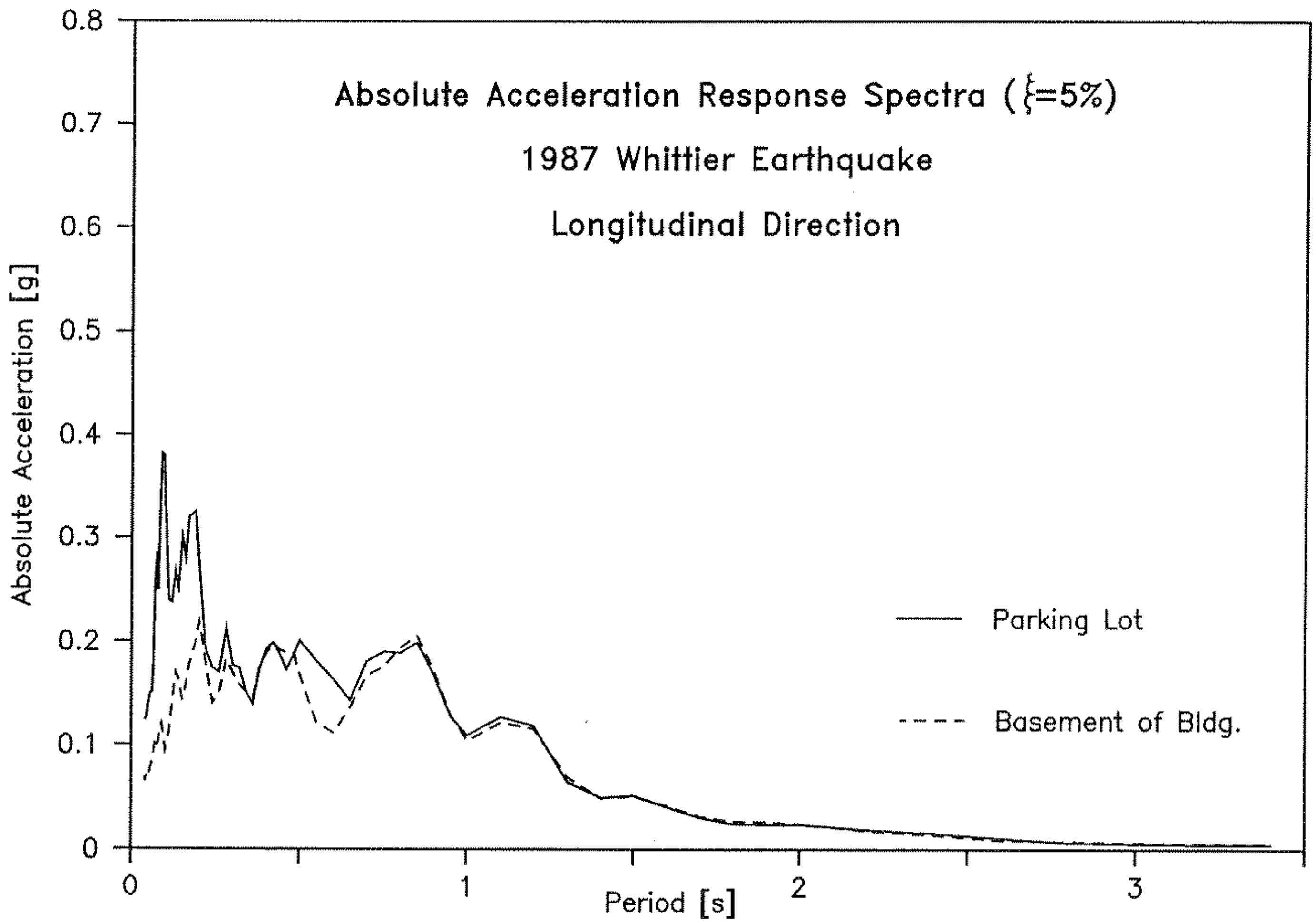

Figure 3.5 Absolute acceleration response spectra of the parking lot and basement motions recorded in the longitudinal direction at the Hollywood Storage Building during the 1987 Whittier earthquake (damping ratio = 5%).

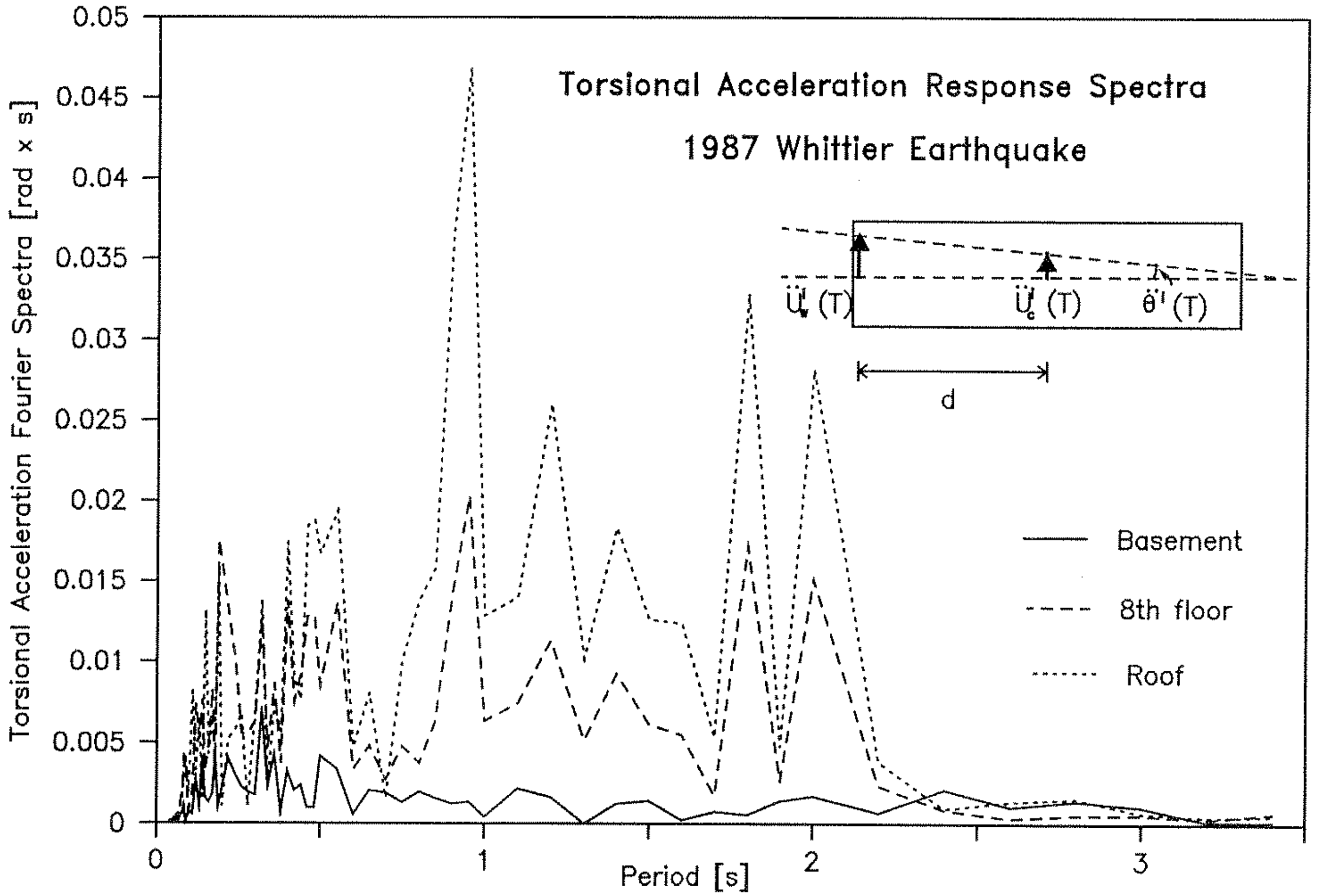

Figure 3.6 Fourier amplitude spectra of the torsional angular acceleration at the basement, 8th floor and roof level computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

CHAPTER 4

MATHEMATICAL MODEL OF THE BUILDING SUPERSTRUCTURE

4.1 Description of the Analysis Program

A mathematical model of the building superstructure is needed to perform a dynamic analysis of the complete superstructure-foundation-soil system. The mathematical model of a building consists of a discretization of the mass, stiffness and damping properties. In this work linear response due to the applied static and dynamic loads is assumed. The building superstructure model has been implemented using the computer program SUPER-ETABS (Wilson, Hollings and Dovey, 1975; Maison and Neuss, 1983; Neuss, Maison and Bouwkamp, 1983).

The SUPER-ETABS program performs linear structural analysis of frame and shear wall buildings subjected to static and earthquake loadings. The 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 the building plan may be specified, and kinematic compatibility is enforced among the frames' degrees of freedom which are common to the rigid floor diaphragms. Bending, axial and shearing deformations of the columns are considered. Beams may be nonprismatic, and bending and shear deformations are included. Deformations within joints can be eliminated by using rigid zones at the ends of beam and column elements. Shear walls continuous over the height of the building can be idealized as column elements in a frame. Panel elements allow discontinuous shear walls to be modeled. Vertical and lateral static loading conditions are possible, and they may be combined with a lateral earthquake input that is specified either as an acceleration response spectrum or as a ground acceleration record. The program is also able to account for the P-Δ effects due to the vertical loads.

Frames are treated as independent substructures. Each frame joint has six degrees of freedom, and the degrees of freedom which are not in common to the rigid floor diaphragms are eliminated by static condensation when the stiffness matrix of the frame is formed. The complete structure stiffness matrix is then assembled. An important consequence of this procedure is that compatibility is not enforced with regard to degrees of freedom common to more than one frame. These degrees of freedom are the vertical displacement and the rotations in the two vertical planes. However, for structural systems in which the frames are orthogonal in plan, as is the case of the Hollywood Storage Building, the joint rotations are uncoupled. The only incompatibility is therefore due to different axial displacements in columns common to more than one frame. The effect of this incompatibility is small except for very tall structures. If the building under study had a structural system more complicated geometrically than the Hollywood Storage Building, the use a more general finite element program would have been more appropriate.

4.2 Model of the Hollywood Storage Building Superstructure

Data on the geometry and the dimensions of the structural members in the Hollywood Storage Building have been obtained from the original 1925 design drawings. In developing the model, the building was assumed unconnected with the adjacent one-story structure. The presence of the partial, embedded basement was not included in the superstructure model, and the mass of the two penthouses at the top of the building were concentrated at the roof level. As shown in Figure 4.1, the building was idealized as a fourteen story structure consisting of eighteen independent frames, four in the longitudinal direction and fourteen in the transverse direction. Shear panels were included in the exterior North, South and West frames, and kinematic compatibility between the panels and the frames was enforced. The contribution of the floor slabs to bending stiffness was approximately included in modeling the beams of the individual frames. The width of the slab effective as a T-beam flange was computed following the ACI 318 (1983) provisions. Shear deformation in the beams and columns was included, as were rigid links to represent the clear span of the members. P-Δ effects were not taken into account.

The mass distribution of the building was computed assuming a unit weight of the concrete of 150 lb/ft^3 (23.6 kN/m^3). Data about the live load rating or the live loads present in the building at the time of the 1987 Whittier earthquake were not available. From a credible estimate based on the particular use of the building (goods storage), a uniformly distributed live load of 110 lb/ft^2 (538 N/m^2) was assumed on all floor surfaces.

The procedure described in the next two paragraphs was used to determine the natural periods of the fixed-base building superstructure from the analysis of the 1987 Whittier earthquake records. An appropriate value of the Young's modulus of the concrete E_c was then selected so to equate these periods with those relative to the three-dimensional mathematical model. A value of E_c = 2780 kips/in^2 (19.2x10^3 MPa) was derived, which is reasonable for concrete. The Poisson's ratio of the concrete was assumed $\nu_c = 0.16$, which gives a shear modulus of G_c = 1200 kips/in^2 (8.27x10^3 MPa).

The 8 in (0.20 m) thick external concrete walls are present almost exclusively in the longitudinal direction and particular attention has been given to evaluating their contribution to the longitudinal stiffness of the building. Some openings are present in the walls at the first story level, and their effect has been considered in an approximate way. From the examination of the original design drawings it is apparent that the walls are actually concrete infill panels for the frames and were not designed as structural continuous systems. Although the walls do not provide the same lateral strength as currently designed shear walls would, the good agreement between the natural periods computed for the mathematical model which includes the wall panels and those obtained from the 1987 Whittier earthquake records clearly indicated that the walls must be included in the model to obtain a satisfactory estimate of the building stiffness in the longitudinal direction.

4.3 Evaluation of Vibration Periods from Analysis of Earthquake Records

The availability of building response data from the 1933 Southern California, the 1952 Kern County and the 1987 Whittier earthquakes gave valuable information about the dynamic properties of the Hollywood Storage Building that was used in developing the mathematical model. There is no data about the roof response during the 1971 San Fernando earthquake because the instrument failed to trigger.

Division of the Fourier spectra values from the accelerograph at the roof by the corresponding values of the basement record gives the transfer functions between the basement and the roof. The plot of the modulus of these transfer functions helps identify which frequencies are amplified by the building, and therefore determine the vibration periods observed during the earthquake motion. The observed periods refer to the complete structure-foundation-soil system and are always longer than the periods of the building superstructure on a fixed base. This process was seen as providing sufficient information for validating the mathematical model, as opposed to a formal system identification approach.

Figures 4.2, 4.3 and 4.4 show the observed transfer functions for the three earthquakes in the transverse direction. From the first two figures it can be seen that the first natural period of the structure-foundation-soil system in this direction was 1.5 s for the 1933 Southern California earthquake and 1.7 s for the 1952 Kern County earthquake. The second large peak observed at 0.75 s and 0.80 s respectively for the two events are due to the torsional response of the system. This happens because the roof instrument was located in the South West corner of the building and was therefore also able to detect the torsional response. During the 1987 Whittier earthquake two instruments were installed on the roof, one at the building center and one at the west wall. The plots relative to the records obtained at these two locations are shown in Figure 4.4. The first translational period in the transverse direction and the first torsional period are at 1.9 s and 0.95 s respectively, and a second translational period at 0.48 s is also detected by the instrument located at the center of the building.

Figures 4.5, 4.6 and 4.7 show the modulus of the transfer function between the basement and the roof for the same three earthquakes in the longitudinal direction. The functions indicate that the first fundamental period in the longitudinal direction was 0.55 s during the 1933 Southern California earthquake and 0.60 s during the 1952 Kern County and the 1987 Whittier earthquakes. The other two peaks at 0.75 s and 0.90 s for the 1952 event are probably due to some torsional response.

The fundamental periods of the Hollywood Storage Building superstructure-foundation-soil system are summarized in Table 4.1. It can be observed that the natural periods in each direction increase with the peak acceleration recorded at the basement, which can be considered a rough estimate of the intensity of the amplitude of motion experienced by the building. The period lengthening is probably due to nonlinear effects and stiffness degradation in the superstructure, and it is interesting to note that the increase is largest in the transverse direction (27%), in which the building is more flexible and weaker. In the longitudinal direction the building is much stiffer and stronger due to the presence of the longitudinal shear walls, and consequently the change of the fundamental period is much less apparent (9.1%). The torsional stiffness and resistance of the building are a combination of the corresponding translational quantities, and this explains the intermediate value of the percentage increment for the torsional fundamental period (19%).

The difference between the natural periods observed during the early ambient and forced vibration tests reported in the previous chapter and those observed from the earthquake records are due to the higher level of excitation induced by the earthquakes in comparison with the vibration tests. In table 4.2 the fundamental periods obtained from the 1938 forced vibration tests are compared with those observed during the recorded seismic event closest in time, the 1933 Southern California earthquake. Again it is observed that the lengthening of the natural period is greater in the transverse direction (25%) than in the longitudinal direction (10%).

4.4 Approximate Modification of Vibration Periods to Account for Soil-Structure Interaction

As mentioned in Chapter 1, an important effect of soil-structure interaction is to lengthen the fundamental vibration period of the structure-foundation-soil system compared to the period of the structure an a fixed base. Because of the flexibility of the foundation and soil, the complete system is more flexible than the fixed-base structure, and has a longer natural period, the period lengthening increasing with the relative flexibility between the soil and the superstructure.

The procedure indicated in the soil-structure interaction chapter of the recommended seismic regulations for new buildings NEHRP (1986), which is based on ATC-3 (1978), has been applied to determine the fundamental vibration period T of the fixed-base building superstructure from the vibration period of the structure-foundation-soil system $\tilde{T}$ observed during the 1987 Whittier earthquake. From the P-wave velocity data of the soil already presented in Figure 2.3 (Duke and Leeds, 1962) and assuming a Poisson's ratio of the soil $\nu_s = 0.33$, an S-wave velocity $V_s = 1190$ ft/s (363 m/s) was derived. Using these values it has been found that $\tilde{T}/T = 1.04$ for the transverse direction and $\tilde{T}/T$ = 1.08 for the longitudinal direction of the building. In the transverse direction $\tilde{T}$ = 1.9 s (see Table 4.1), and this gives T = 1.9/1.04 = 1.8 s. In the longitudinal direction $\tilde{T}$ = 0.60 s, so that T = 0.60/1.08 = 0.56 s.

4.5 Verification of Model and Summary of Vibration Properties

The vibration periods and frequencies obtained from the mathematical model of the Hollywood Storage Building superstructure are listed in Table 4.3. There is small coupling between rotation and translation in the transverse direction, while the translation in the longitudinal direction is uncoupled. As expected, the first and third periods correspond to the translational modes in the two directions and are very close to the fixed-base periods computed for the 1987 Whittier earthquake. The second mode is a torsional mode and it has a period of T=0.88 s, which can be compared with the value $\tilde{T}$ =0.95 s observed during the earthquake. The fourth mode (second translational in the transverse direction) has T=0.53 s, while it has been observed $\tilde{T}$ = 0.48 s (see Figure 4.4). In addition the order of vibration modes is in agreement with that observed during the 1938 forced vibration tests (see Table 2.1). Table 4.4 lists the three-dimensional mode shapes of the model, which are also shown in Figures 4.8, 4.9 and 4.10. For the two translational directions, the first mode shapes computed from the data by Duke et al. (1970) assuming a simple shear-type building are also shown. There is a good agreement between the two models in the transverse direction. Because of the presence of the shear walls along the exterior sides, in the longitudinal direction the building behaves almost like a cantilever, and therefore larger differences are observed between the two models.

Earthquake	Peak acceleration at basement [g]	Translational (trans. dir.) [s]	Translational (long. dir.) [s]	Torsional [s]
Southern California October 2, 1933	0.03	1.5	0.55	0.80
Kern County July 21, 1952	0.06	1.7	0.60	0.80
Whittier Narrows October 1, 1987	0.12	1.9	0.60	0.95

Table 4.1 Fundamental longitudinal and torsional periods of the Hollywood Storage Building superstructure-foundation-soil system observed during different earthquakes.

	Translational (trans. direction) [s]	Translational (long. direction) [s]	Torsional [s]
Forced Vibration Tests December 6-14, 1938	1.2	0.50	0.60-0.64
Southern California October 2, 1933	1.5	0.55	0.80

Table 4.2 Comparision between the fundamental periods of the Hollywood Storage Building superstructure-foundation-soil system observed during the 1938 forced vibration tests (Carder, 1964) and during the 1933 Southern California earthquake.

Mode No.	Mode type	Period [s]	Frequency [Hz]	Cir. Frequency [rad/s]
1	mainly translational (transverse direction)	1.8	0.55	3.5
2	mainly torsional	0.88	1.1	7.1
3	translational (longitidinal direction)	0.58	1.7	11
4	mainly translational (transverse direction)	0.55	1.8	11
5	mainly translational (transverse direction)	0.29	3.4	21
6	mainly torsional	0.24	4.2	26
7	mainly translational (transverse direction)	0.19	5.1	32
8	translational (longitidinal direction)	0.18	5.6	35

Table 4.3 Vibrational periods and frequencies of the mathematical model of the Hollywood Storage Building Superstructure.

Level	Direction	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
	longitudinal dir.	-.000314	.001661	.172875	-.007222	-.000198	-.000210	.000122	.145150
15	transverse dir.	.163010	.061916	-.006993	-.154490	-.151842	-054825	.125718	.000485
	torsional	.000066	-.000216	-.000002	-.000082	-.000092	.000182	.000083	.000001
	longitudinal dir.	-.000271	.001479	.161083	-.006770	-.000107	-.000025	-.000115	.103731
14	transverse dir.	.152075	.053859	-.004707	-.106084	-.048149	-.037546	-.028430	.000017
	torsional	.000062	-.000200	-.000001	-.000058	-.000031	.000127	-.000012	.000001
	longitudinal dir.	-.000230	.001295	.148568	-.006281	-.000041	.000162	-.000242	.058964
13	transverse dir.	.139983	.045992	-.002232	-.052005	.054042	-.019705	-.135103	-.000403
	torsional	.000058	-.000183	.000000	-.000030	.000028	.000067	-.000078	.000000
	longitudinal dir.	-.000187	.001102	.135065	-.005777	-.000107	.000374	-.000124	.013776
12	transverse dir.	.128022	.038219	-.000254	-.006924	.101875	.002230	-.116120	-.000397
	torsional	.000054	-.000166	.000001	-.000009	.000051	.000012	-.000066	.000000
	longitudinal dir.	-.000150	.000919	.120230	-.005190	-.000169	.000526	.000017	-.030961
11	transverse dir.	.114635	.031727	.001709	.039079	.121861	.018275	-.040629	-.000274
	torsional	.000048	-.000147	.000002	.000015	.000062	-.000045	-.000027	.000000
	longitudinal dir.	-.000166	.000746	.104449	-.004548	-.000235	.000626	.000150	-.071203
10	transverse dir.	.100430	.026369	.003340	.078660	.104951	.029072	.051297	-.000109
	torsional	.000043	-.000128	.000003	.000035	.000054	-.000095	.000021	.000000
	longitudinal dir.	-.000086	.000587	.088208	-.003869	-.000285	.000660	.000248	-.103288
9	transverse dir.	.085842	.021961	.004471	.107267	.057513	.036217	.111254	.000039
	torsional	.000037	-.000108	.000003	.000050	.000029	-.000133	.000054	.000000
	longitudinal dir.	-.000061	.000446	.072031	-.003177	-.000321	.000635	.000299	-.124561
8	transverse dir.	.071246	.018192	.005041	.122772	-.004250	.040979	.111944	.000144
	torsional	.000030	-.000088	.000004	.000058	-.000004	-.000157	.000055	.000000
	longitudinal dir.	-.000041	.000323	.056481	-.002504	-.000340	.000573	.000288	-.133625
7	transverse dir.	.056963	.014969	.005062	.124858	-.062536	.043720	.058654	.000216
	torsional	.000024	-.000070	.000003	.000059	-.000034	-.000165	.000029	-.000001
	longitudinal dir.	-.000025	.000219	.042132	-.001884	-.000329	.000496	.000226	-.130514
6	transverse dir.	.043318	.012088	.004603	.114752	-.102904	.044091	-.019503	.000262
	torsional	.000018	-.000052	.000003	.000054	-.000055	-.000157	-.000011	-.000001
	longitudinal dir.	-.000013	.000135	.029566	-.001339	-.000288	.000408	.000148	-.116690
5	transverse dir.	.030675	.009343	.003770	.094838	-.117107	.041352	-.086415	.000267
	torsional	.000013	-.000037	.000002	.000044	-.000062	-.000135	-.000046	-.00001
	longitudinal dir.	-.000006	.000074	.019390	-.000891	-.000299	.000304	.000080	-.094794
4	transverse dir.	.019600	.006513	.002715	.068743	-.104138	.034515	-.114206	.000213
	torsional	.000008	-.000024	.000002	.000032	-.000055	-.000104	-.000059	-.000001
	longitudinal dir.	-.000001	.000037	.012289	-.000568	-.000171	.000192	.000032	-.069684
3	transverse dir.	.100528	.003783	.001608	.040894	-.070947	.024307	-.095630	.000119
	torsional	.000004	-.000014	.000001	.000019	-.000037	-.000070	-.000049	-.000001
	longitudinal dir.	.000001	.000017	.008108	-.000370	-.000127	.000102	.000007	-.048322
2	transverse dir.	.004673	.001956	.000766	.019606	-.036571	.015092	-.054636	.000056
	torsional	.000002	-.000007	.000001	.000009	-.000019	-.000042	-.000028	.000000

Table 4.4 Three-dimensional mode shapes of the mathematical model of the Hollywood Storage Building superstructure.

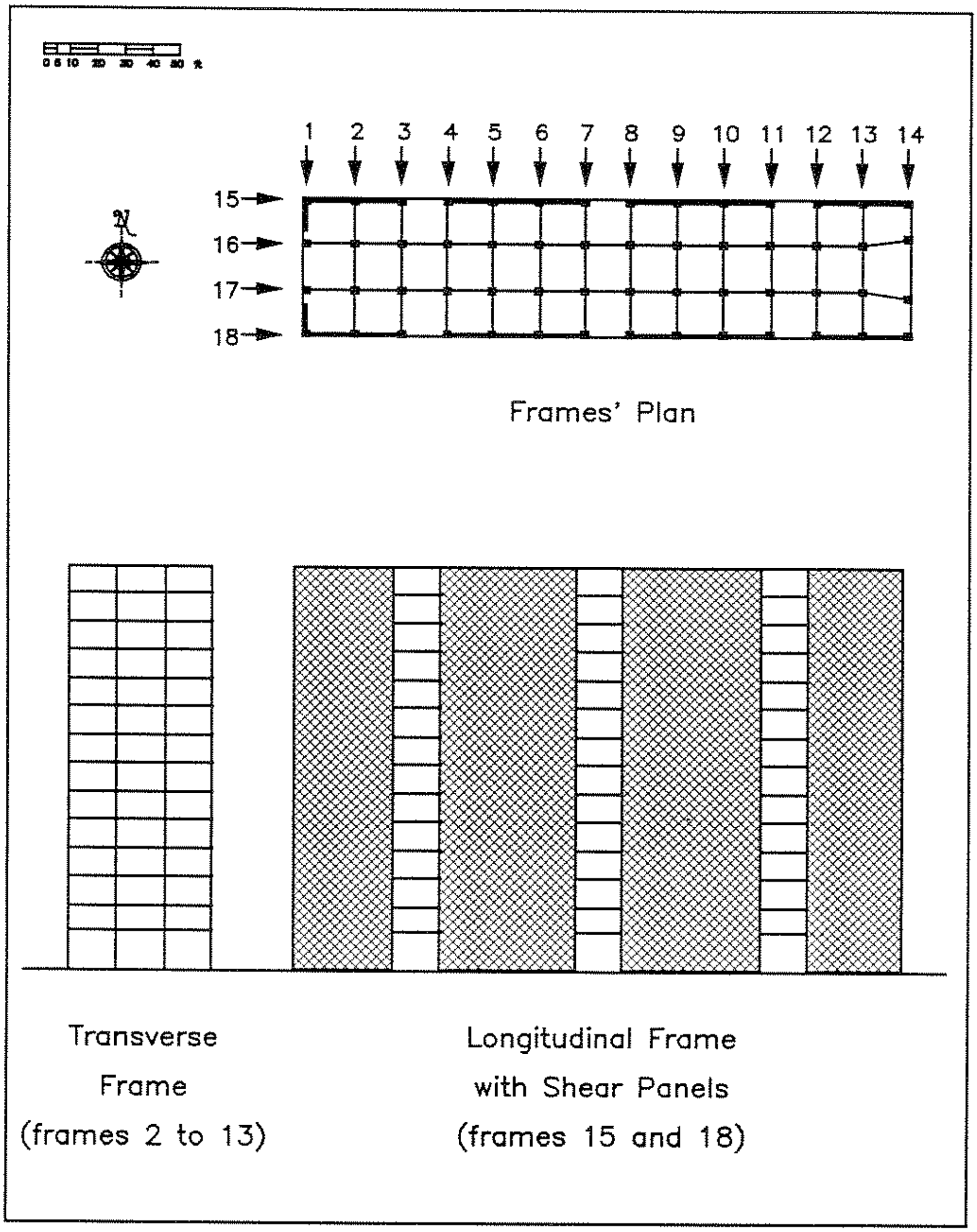

Figure 4.1 Frames and shear panels considered in the mathematical model of the building superstructure.

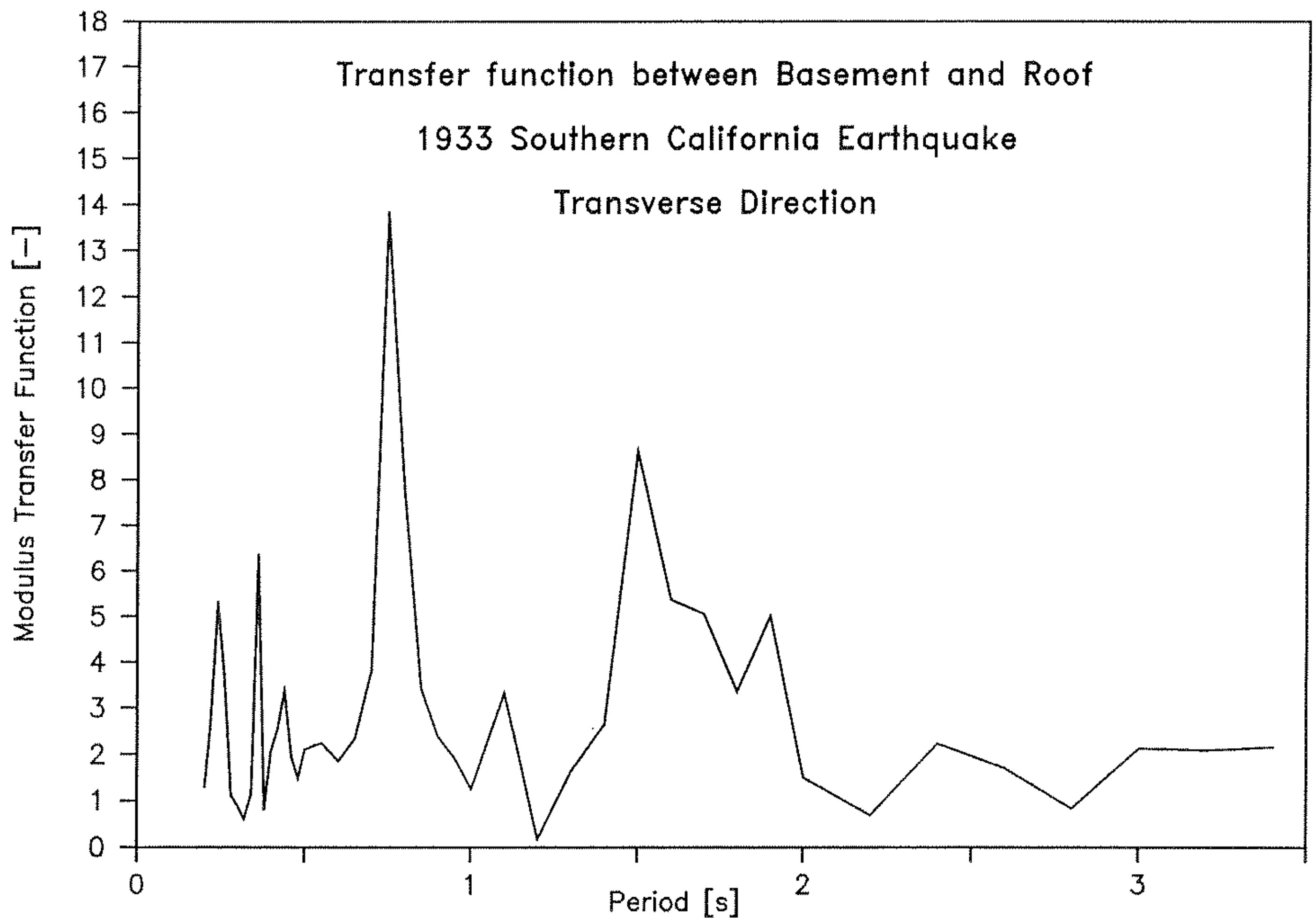

Figure 4.2 Modulus of the transfer function between the basement and the roof in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1933 Southern California earthquake.

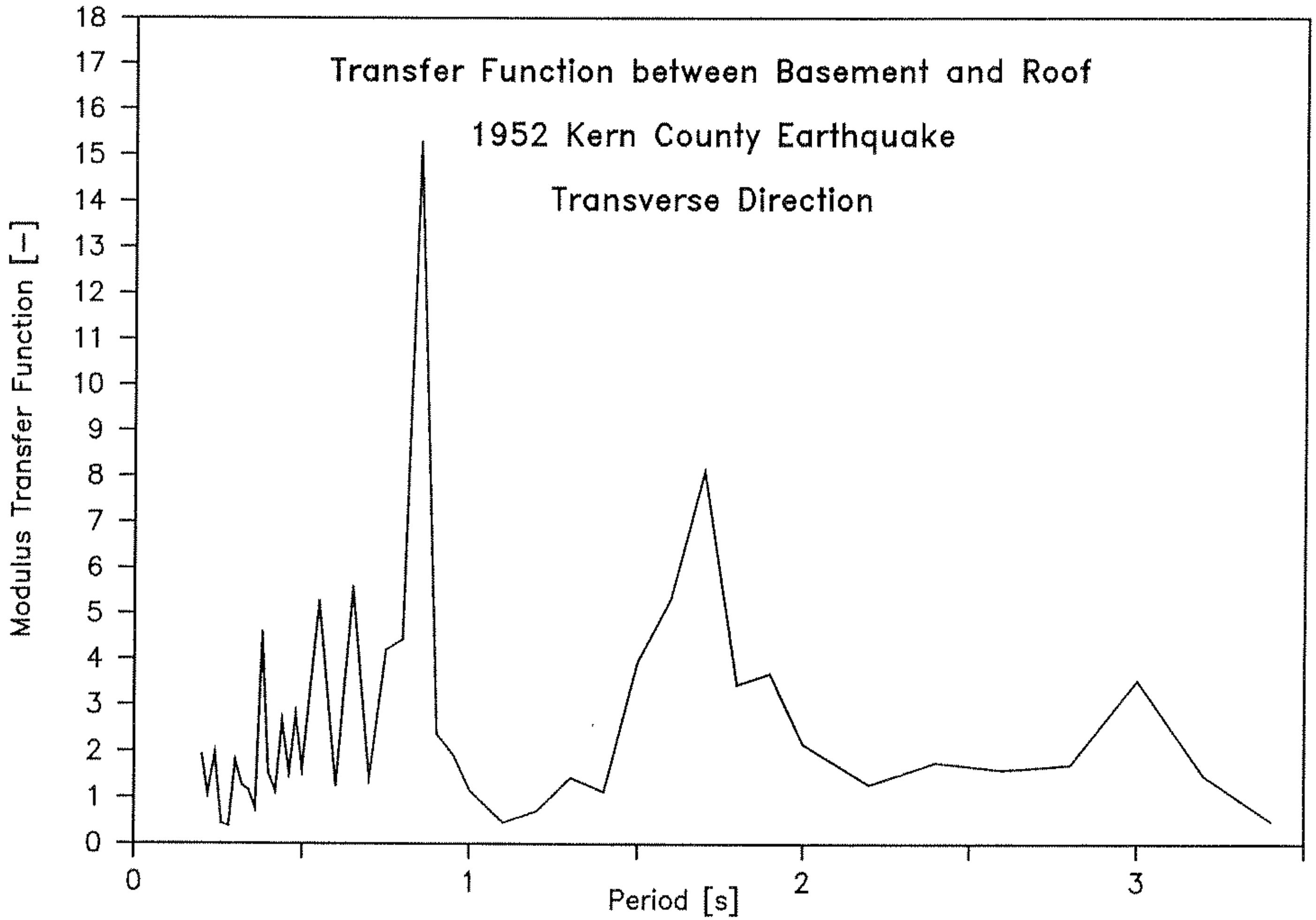

Figure 4.3 Modulus of the transfer function between the basement and the roof in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1952 Kern County earthquake.

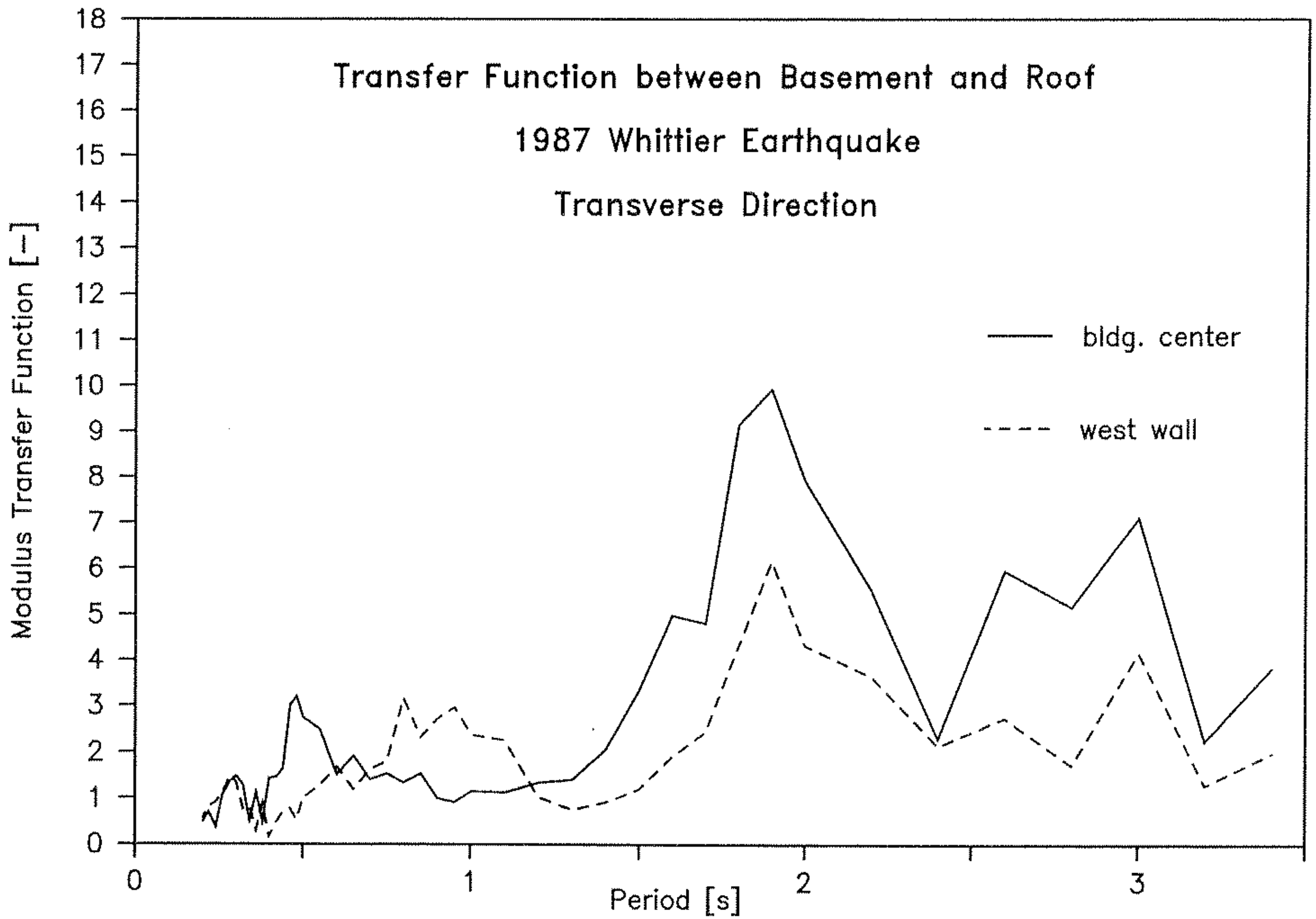

Figure 4.4 Modulus of the transfer function between the basement and the roof in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

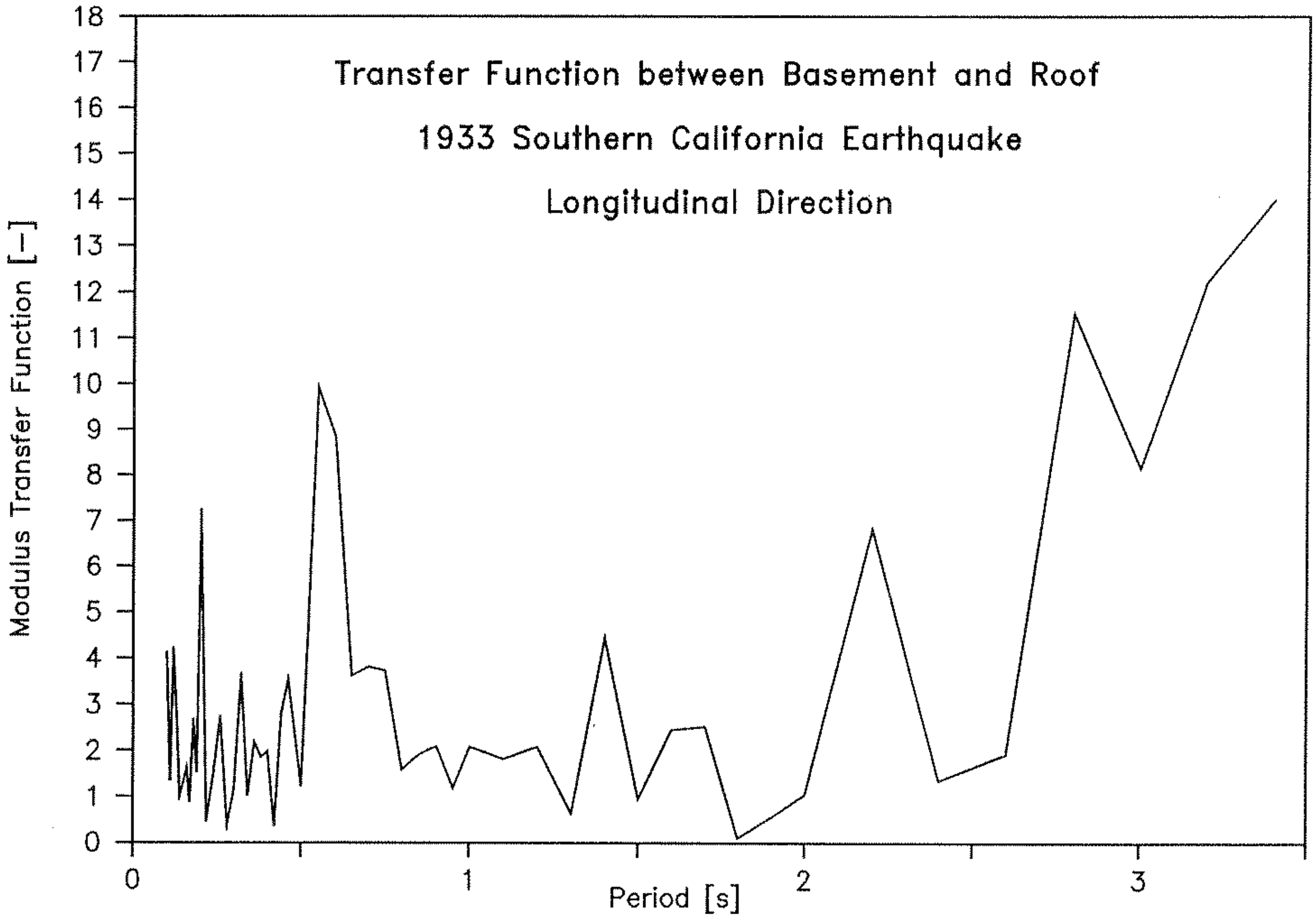

Figure 4.5 Modulus of the transfer function between the basement and the roof in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1933 Southern California earthquake.

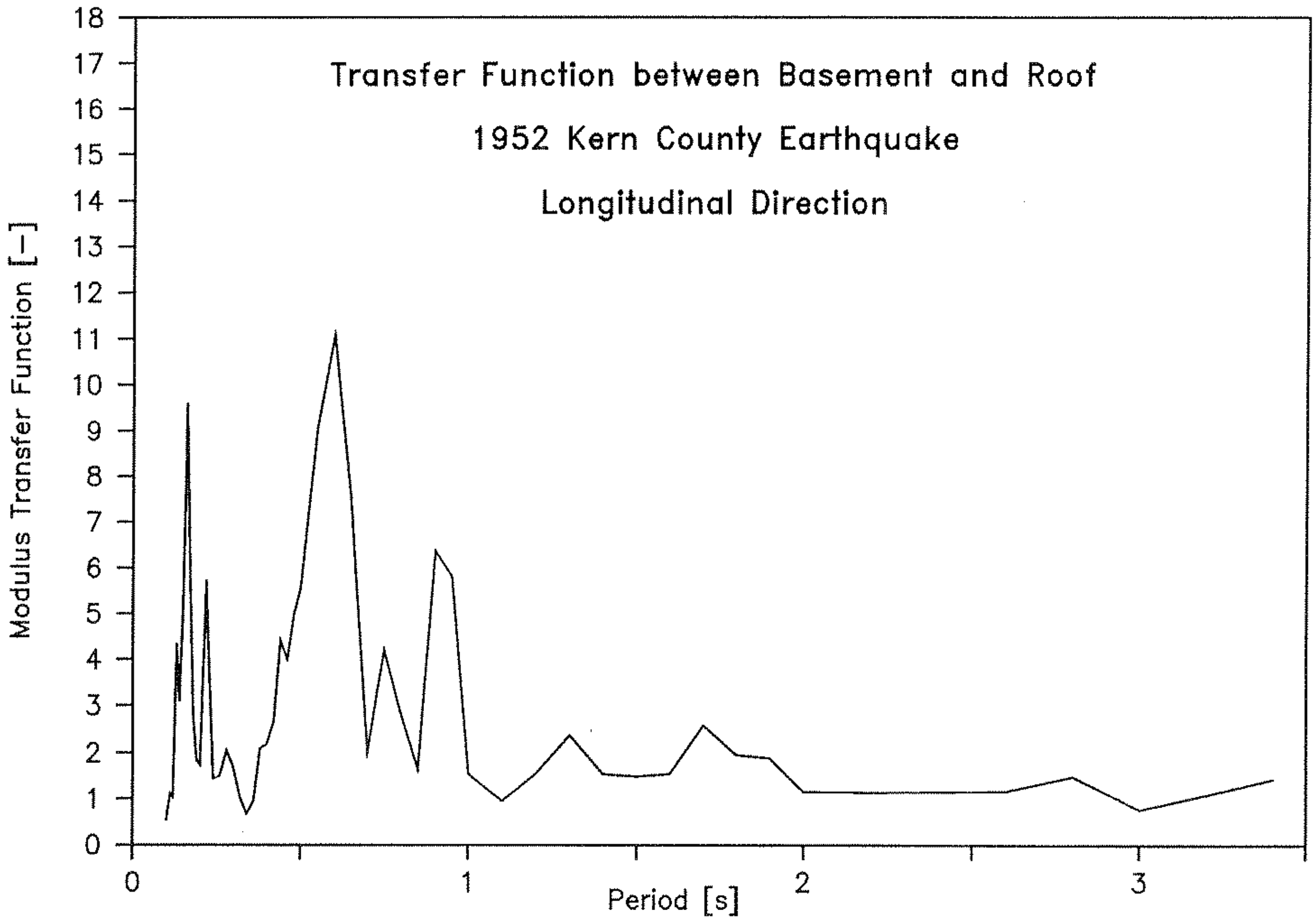

Figure 4.6 Modulus of the transfer function between the basement and the roof in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1952 Kern County earthquake.

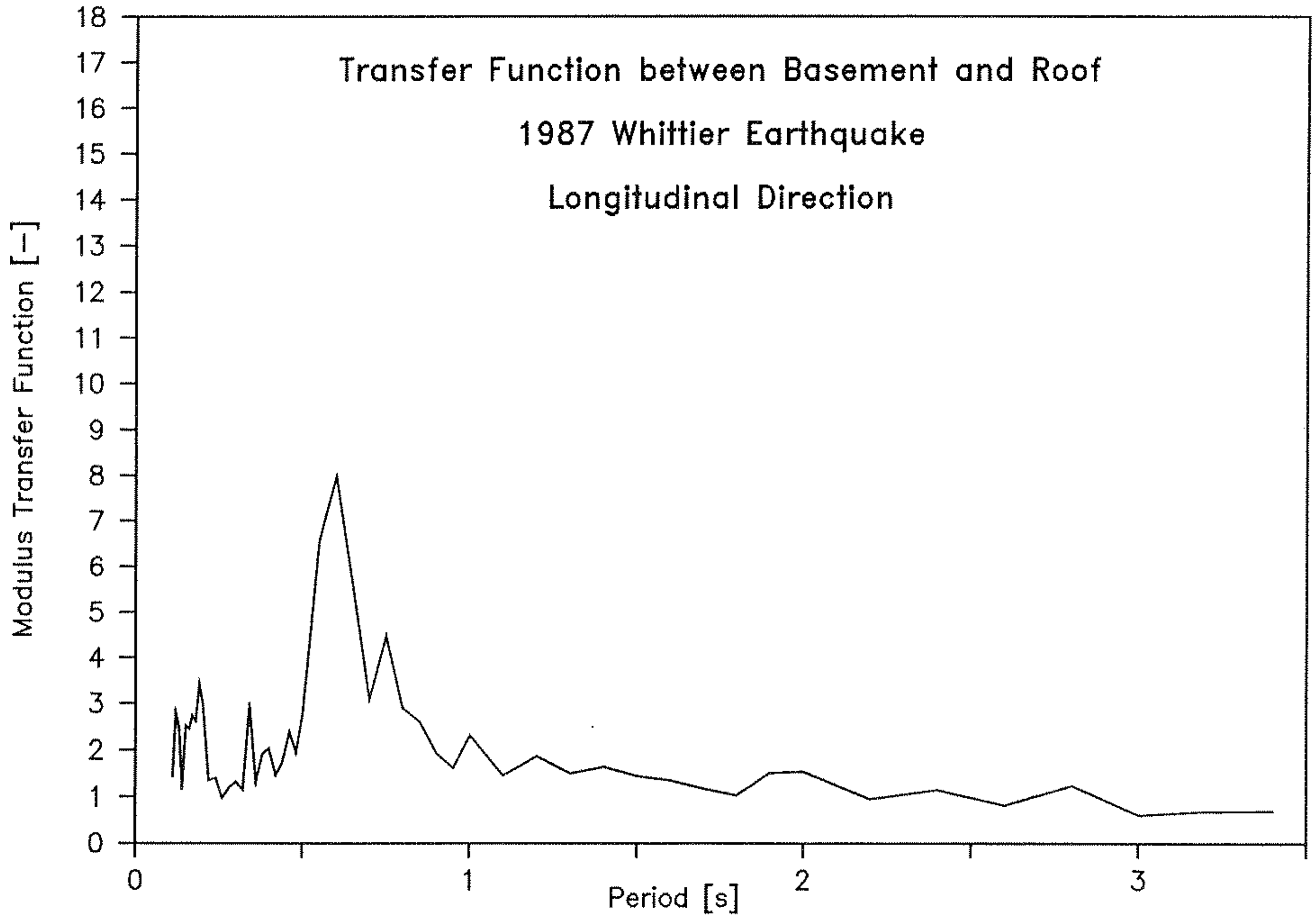

Figure 4.7 Modulus of the transfer function between the basement and the roof in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

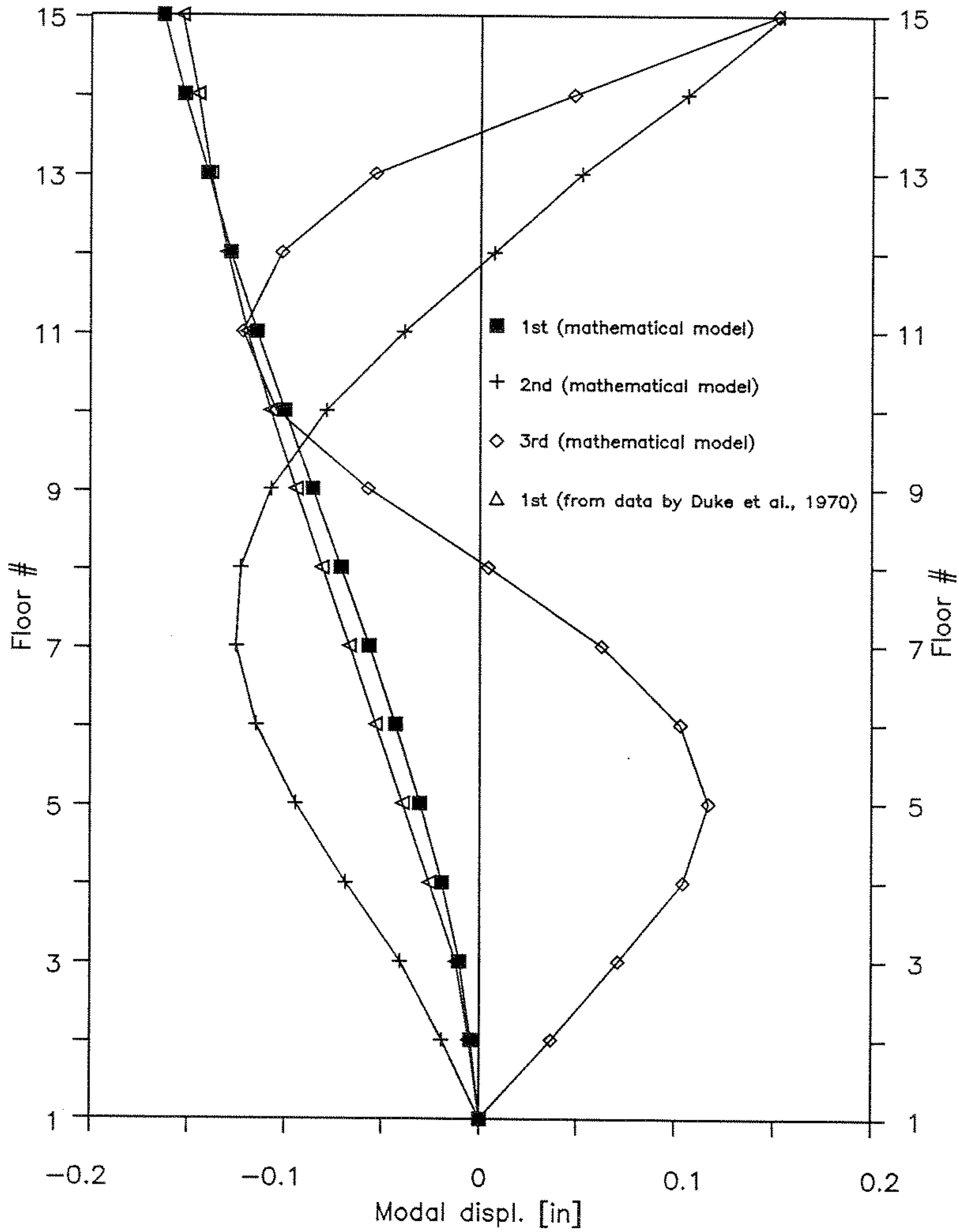

Figure 4.8 Mode shapes of the building superstructure in the transverse direction computed from the mathematical model and from the data reported by Duke et al. (1970).

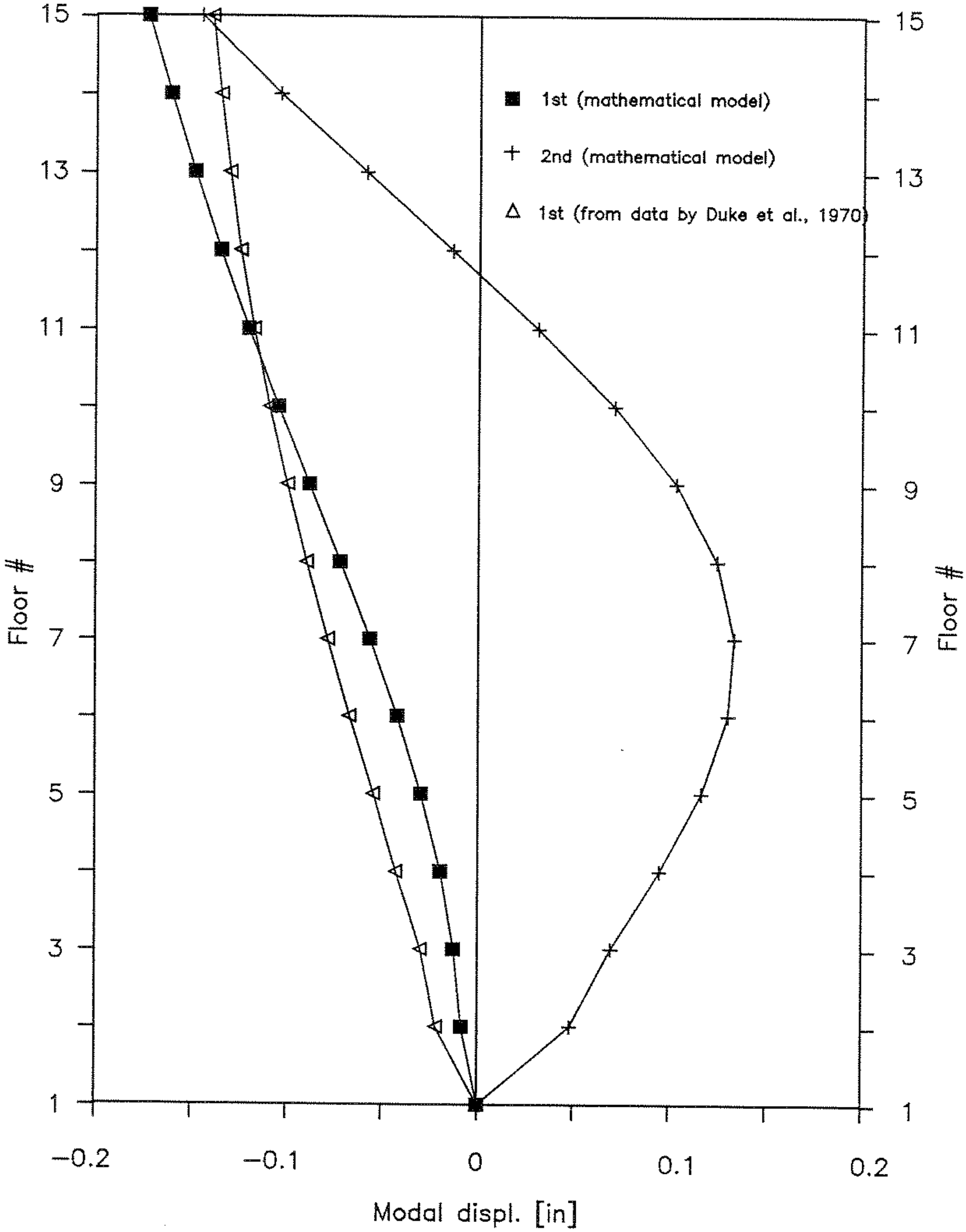

Figure 4.9 Mode shapes of the building superstructure in the longitudinal direction computed from the mathematical model and from the data reported by Duke et al. (1970).

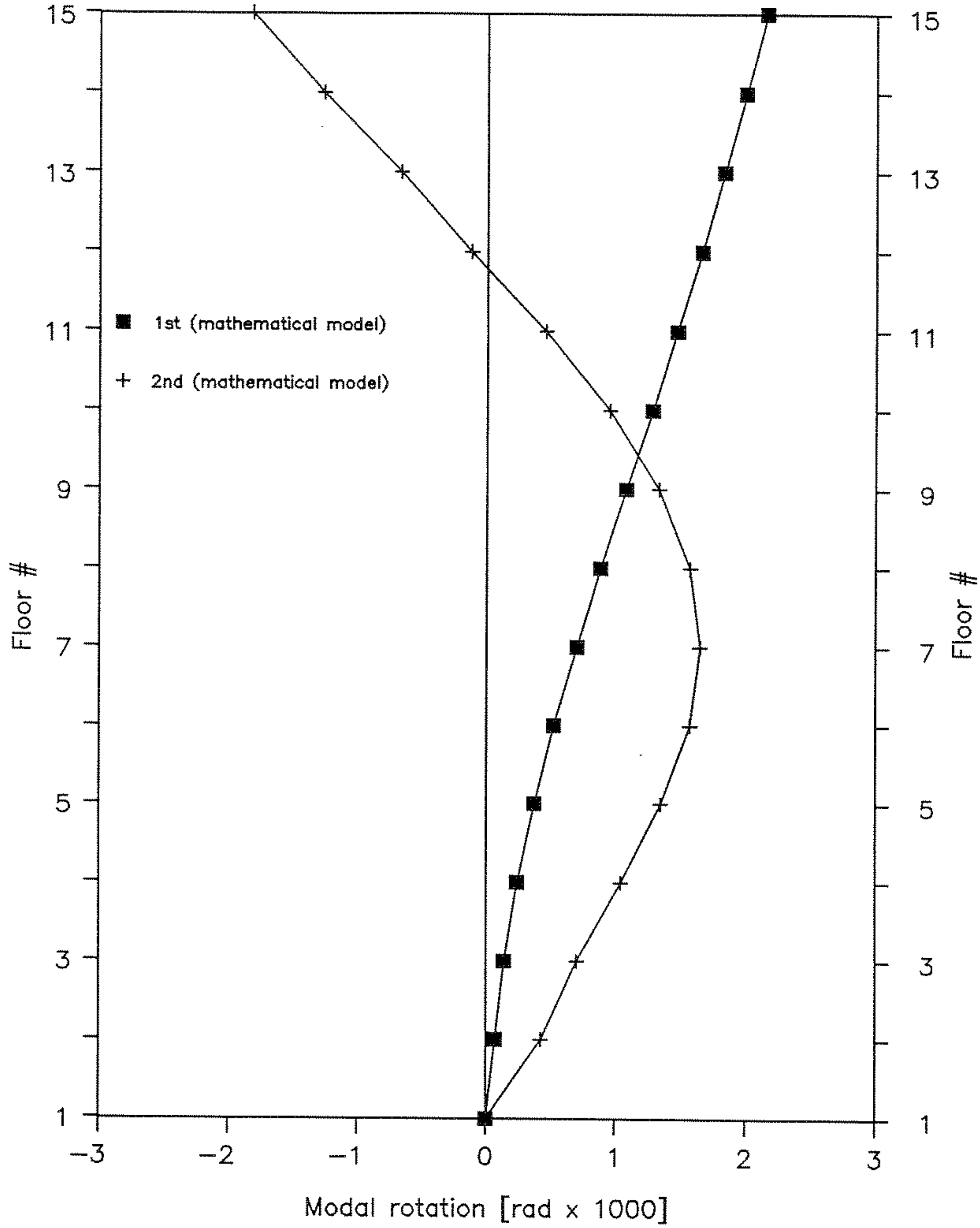

Figure 4.10 Torsional mode shapes of the building superstructure computed from the mathematical model.

CHAPTER 5

SUBSTRUCTURE MODEL OF THE BUILDING-FOUNDATION-SOIL SYSTEM

5.1 Description of the Substructure Model

In order to obtain some response quantities not directly available from the accelerograms recorded at the building site, such as the foundation rocking and the base shear, an analytical model has been developed for the complete building-foundation-soil system. The parameters of the model are determined by comparing the computed response with the recorded response in the Hollywood Storage Building.

The results of the three-dimensional dynamic analysis of the building superstructure presented in Chapter 4 show that the translational response in the longitudinal direction is nearly uncoupled, while there is small coupling between rotation and the translation in the transverse direction. It was therefore a reasonable approximation to consider two different two-dimensional models, one for each axis of the building.

The analysis has been performed using the substructure model of the complete building, foundation and soil system. This model has been extensively used in soil-structure interaction studies. It was first developed for the case of an elastic halfspace by Parmalee (1967) and Parmalee, Perelman and Lee (1969). It has been shown later that the effects of soil-structure interaction occurs predominantly in the fundamental mode (Jennings and Bielak, 1973) and the use of the natural modes of the superstructure on fixed base can significantly reduce the computational efforts (Chopra and Gutierrez, 1973).

The superstructure is idealized as a 14-story shear type building, with the mass concentrated at each floor level, and having the same vibration modes and periods of the three-dimensional superstructure model in the direction considered. Viscous damping has been assumed for the superstructure, of such a form that, when the base is considered fixed, decomposition into classical normal vibration modes is allowed (Caughey, 1960). The building foundation is idealized as an equivalent circular rigid disk attached to the surface of a viscoelastic halfspace with hysteretic damping. As the earthquake excitation was not strong enough to cause yielding in the building, the assumption of a linear behavior of the superstructure is certainly adequate. The effect of nonlinearities in the soil was included by choosing appropriate values of the elastic and damping properties, using the equivalent linear method (Seed and Idriss, 1970).

The homogeneous halfspace representation was chosen for the soil region, because there are no significant changes in the soil properties from the surface to a depth of approximately 100 ft (30.5 m), as shown in Figure 2.3. It must be noted that the assumption of a rigid and circular base is an approximation of the irregular basement and foundation of the building, and the model does not include the embedment due to the partial basement, nor the presence of the short piles. However, it has been shown that for aspect ratios of the foundation up to 4, as it is the case of the Hollywood Storage Building, the dynamic stiffness of a rigid rectangular plate and that of an equivalent cicular disk are almost the same, the error involved in assuming an equivalent circular disk being sufficiently small to be insignificant for all practical purposes (Gazetas, 1983). Furthermore, the depth of the embedment and the maximum pile length are 9.0 ft (2.7 m) and 30 ft (9.1 m), respectively, which are relatively small when compared to the plan dimensions of the building. Lastly, the good agreement between the response computed from the analytical model and the recorded response indicates that the foundation-soil model is sufficiently accurate for determining the overall response of the building.

The dynamic analysis is carried out in the frequency domain. The complex-valued frequency dependent stiffness coefficients of the foundation are computed, and incorporated in the governing equations for the complete system written in terms of complex valued frequency response functions due to a unit harmonic horizontal free-field ground acceleration. The applied free-field ground motion is transformed into its Fourier components, and the response is determined for each frequency component. Finally, an inverse Fourier transform is performed to obtain the time history of the response.

5.2 Formulation of the Equations of Motion

A schematic representation of the substructure model is shown in Figure 5.1. It is a N+2 DOF system. These are the N translational displacements of the building floors, $u_j^t(t)$ (j = 1, 2, ..., N), and the horizontal base displacement and rotation $u_b(t)$ and $\theta(t)$, respectively. In what follows, lower case letters indicate response quantities in the time domain and upper case letters indicate response quantities in the frequency domain. For example, for the horizontal base displacement: $u_b = u_b(t)$ and $U_b = U_b(\omega)$.

The equations of motion for the floor masses are:

$$[M]\{\ddot{u}^t\}+[C]\{\dot{u}\}+[K]\{u\}=0 \tag{5.1}$$

where $\{u^t\}$ and $\{u\}$ denote the column vectors of the absolute displacements of the floors and those relative to the building base, respectively, $[M]$ is the diagonal matrix of the floor masses m_j, $[K]$ is the stiffness matrix, and $[C]$ is the viscous damping matrix of the fixed-base superstructure.

The other two equations of motion for the complete system express the global translational and rotational equilibrium of the building:

$$\{1\}^{\mathrm{T}}[M]\{\ddot{u}^t\}+m_b\ddot{u}_b+v=0 \tag{5.2}$$

$$\{h\}^{\mathrm{T}}[M]\{\ddot{u}^t\}+I_b\ddot{\theta}+m=0 \tag{5.3}$$

where $\{1\}$ is the unity column vector; $\{h\}$ is the column vector of the floor heights h_j relative to the base; m_b and I_b are the base mass and mass moment of inertia about the rocking axis, respectively; and v and m are the base shear and moment that the foundation exerts on the halfspace, respectively. In equation (5.3) the contribution of the mass moment of inertia of the floors is neglected.

The base displacement is given by the free-field motion u_g modified by the interaction displacement u_o:

$$u_b = u_g + u_o \tag{5.4}$$

and the total motion of the floors is:

$$\{u^t\} = \{1\}u_g + \{1\}u_o + \{h\}\theta + \{u\} \tag{5.5}$$

Substituting Equations (5.4) and (5.5) into equations (5.1), (5.2) and (5.3) gives:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} + [M]\{1\}\ddot{u}_o + [M]\{h\}\ddot{\theta} = -[M]\{1\}\ddot{u}_g \tag{5.6}$$

$$\{1\}^{\mathrm{T}}[M]\{\ddot{u}\} + m_t\ddot{u}_o + S_t\ddot{\theta} + v = -m_t\ddot{u}_g \tag{5.7}$$

$$\{h\}^{\mathrm{T}}[M]\{\ddot{u}\} + S_t\ddot{u}_o + I_t\ddot{\theta} + m = -S_t\ddot{u}_g \tag{5.8}$$

where:

$$m_t = m_b + \{1\}^{\mathrm{T}}[M]\{1\} = m_b + \sum_{j=1}^{N} m_j \tag{5.9}$$

$$S_t = \{1\}[M]\{h\}^{\mathrm{T}} = \{h\}^{\mathrm{T}}[M]\{1\} = \sum_{j=1}^{N} m_j h_j \tag{5.10}$$

$$I_t = I_b + \{h\}^{\mathrm{T}}[M]\{h\} = I_b + \sum_{j=1}^{N} m_j h_j^2 \tag{5.11}$$

If a unit harmonic free-field acceleration of frequency ω is applied, $\ddot{u}_g(t) = 1e^{i\omega t}$, each steady state response function is given by $r(t) = R(\omega)e^{i\omega t}$. Substituting this expression for each response quantity in equations (5.6)-(5.8), and canceling the $e^{i\omega t}$ terms on both sides gives:

$$(-\omega^2[M] + i\omega[C] + [K])\{U\} - \omega^2[M]\{1\}U_o - \omega^2[M]\{h\}\Theta = -[M]\{1\} \tag{5.12}$$

$$-\omega^2\{1\}^{\mathrm{T}}[M]\{U\} - \omega^2 m_t U_o - \omega^2 S_t\Theta + K_{VV}U_o + K_{VM}\Theta = -m_t \tag{5.13}$$

$$-\omega^2\{h\}^{\mathrm{T}}[M]\{U\} - \omega^2 S_t U_o - \omega^2 I_t\Theta + K_{MV}U_o + K_{MM}\Theta = -S_t \tag{5.14}$$

in which it has been considered that the base shear and moment are related to the interaction displacements through the expression:

$$\begin{Bmatrix} V \\ M \end{Bmatrix} = \begin{bmatrix} K_{VV} & K_{VM} \\ K_{MV} & K_{MM} \end{bmatrix} \begin{Bmatrix} U_o \\ \Theta \end{Bmatrix} \tag{5.15}$$

The stiffness (or impedance) functions K_{VV}, K_{MM} and $K_{VM} = K_{MV}$ are frequency dependent, and are discussed subsequently.

For each excitation frequency, Equations (5.12)-(5.14) represent a system of N+2 linear complex equations. The unknowns are the N relative floor displacements U_j, the base interaction displacement U_o and the base rotation Θ. As shown by Chopra and Gutierrez (1973), an effective approach for significantly reducing the number of response quantities, and consequently the computational efforts, is to express the relative floor displacements as the superposition of response in the first J normal modes of vibration of the fixed-base superstructure:

$$\{u(t)\} \cong \sum_{j=1}^{J} y_j(t)\{\phi^{(j)}\} \tag{5.16}$$

or, in terms of the corresponding frequency response functions:

$$\{U(\omega)\} \cong \sum_{j=1}^{J} Y_j(\omega)\{\phi^{(j)}\} \tag{5.17}$$

Introducing the above transformation in the equations of motion (5.12)-(5.14), premultiplying the j-th Equation (5.12) by $\{\phi^{(j)}\}^T$, and using the orthogonality properties of the vibration modes leads to:

$$\sum_{j=1}^{J} M_j(-\omega^2 + 2i\xi_j\omega_j\omega + \omega_j^2)Y_j - \omega^2 L_j^h U_o - \omega^2 L_j^r \Theta = -L_j^h \qquad j = 1,2,\ldots,\ J \tag{5.18}$$

$$-\omega^2 \sum_{j=1}^{J} L_j^h Y_j + (-\omega^2 m_t + K_{VV})U_o + (-\omega^2 S_t + K_{VM})\Theta = -m_t \tag{5.19}$$

$$-\omega^2 \sum_{j=1}^{J} L_j^r Y_j + (-\omega^2 S_t + K_{MV})U_o + (-\omega^2 I_t + K_{MM})\Theta = -S_t \tag{5.20}$$

in which:

$$\{\phi^{(j)}\}^T[M]\{\phi^{(j)}\} = M_j \qquad \{\phi^{(j)}\}^T[C]\{\phi^{(j)}\} = 2M_j\xi_j\omega_j \qquad \{\phi^{(j)}\}^T[K]\{\phi^{(j)}\} = \omega_j^2 M_j \tag{5.21}$$

$$\{\phi^{(j)}\}^{\mathrm{T}}[M]\{1\}=L_j^h \tag{5.22}$$

$$\{\phi^{(j)}\}^{\mathrm{T}}[M]\{h\}=L_j^r \tag{5.23}$$

The solution of the system of Equations (5.18)-(5.20) can be obtained in an especially efficient way. In the first J equations, each generalized coordinate can be expressed in terms of U_o and Θ:

$$Y_j=H_j(\omega)\,[\omega^2 L_j^h U_o+\omega^2 L_j^r\Theta-L_j^h] \qquad j=1,2,\ldots,\ J \tag{5.24}$$

where:

$$H_j(\omega)=\frac{1}{M_j(-\omega^2+2i\xi_j\omega_j\omega+\omega_j^2)} \qquad j=1,2,\ldots,\ J \tag{5.25}$$

is the complex frequency response function of the SDOF corresponding to the j-th mode of vibration of the superstructure on fixed-base. Substitution of Equations (5.24) into (5.19) and (5.20) leads to a system in the only two unknowns response quantities U_o and Θ:

$$\left[-\omega^2 m_t+K_{VV}-\omega^4\sum_{j=1}^{J}(L_j^h)^2H_j\right]U_o+\left[-\omega^2 S_t+K_{VM}-\omega^4\sum_{j=1}^{J}L_j^hL_j^rH_j\right]\Theta=-m_t-\omega^2\sum_{j=1}^{J}(L_j^h)^2H_j \tag{5.26}$$

$$\left[-\omega^2 S_t+K_{MV}-\omega^4\sum_{j=1}^{J}L_j^hL_j^rH_j\right]U_o+\left[-\omega^2 I_t+K_{MM}-\omega^4\sum_{j=1}^{J}(L_j^r)^2H_j\right]\Theta=-S_t-\omega^2\sum_{j=1}^{J}(L_j^r)^2H_j \tag{5.27}$$

which can easily be solved. Backsubstitution of U_o and Θ in Equations (5.24) gives the values of the generalized coordinates Y_i.

5.3 Impedance Functions for the Soil

The impedance functions K_{VV}, K_{MM} and $K_{VM}=K_{MV}$ can be obtained from the solution of the steady state response of a rigid plate supported on the soil halfspace and excited by two different loading conditions: an harmonic horizontal force and a harmonic overturning moment. Approximate solutions of these two mixed boundary value problems in the case of a rigid circular disk on an elastic halfspace have been obtained by several authors. Veletsos and Wei (1971) obtained the results for a wider range of the parameters, and without making any a priori assumptions concerning the distribution of the contact pressure. The authors also showed that the coupling impedance function K_{VM} $= K_{MV}$ is generally small in comparison to the diagonal terms K_{VV} and K_{MM}, and for this reason the coupling term has been neglected in this investigation. For the case of a static horizontal force and moment ($\omega = 0$), the diagonal stiffness terms are given by:

$$K_{VV}\big|_{\omega=0} = K_x = \frac{8G_s r_o}{2-\nu_s} \tag{5.28}$$

$$K_{MM}\big|_{\omega=0} = K_\theta = \frac{8G_s r_o^3}{3(1-\nu_s)} \tag{5.29}$$

where r_o is the radius of the disk, and G_s and ν_s are the shear modulus and Poisson's ratio of the soil. G_s can be expressed in terms of the S-wave velocity V_s and the mass density of the soil ρ_s as $G_s = \rho_s V_s^2$.

For an excitation frequency ω, the complex valued impedance functions for translation and rotation of the base are given by the form:

$$K_{VV}(\omega) = [k_{11}(a_o,\nu_s) + i a_o c_{11}(a_o,\nu_s)]K_x \tag{5.30}$$

$$K_{MM}(\omega) = [k_{22}(a_o,\nu_s) + i a_o c_{22}(a_o,\nu_s)]K_\theta \tag{5.31}$$

in which k_{ii} *and* c_{ii} are dimensionless coefficients that depend on ν_s and the dimensionless frequency parameter:

$$a_o = \frac{\omega r_o}{V_s} \tag{5.32}$$

The effect of hysteretic damping in the soil has been included in the impedance coefficients of the elastic system using the correspondence principle (Bland, 1960). The damped solution is obtained from the elastic one by replacing the elastic shear modulus of the soil G_s by a complex valued modulus:

$$G_s^* = G_s(1+2i\zeta_s) \tag{5.33}$$

where ζ_s is the hysteretic damping coefficient for the soil. Implicit in the statement above is the assumption that the Poisson's ratio for the damped soil is a real-valued quantity equal to that for the elastic soil. Using the correspondence principle, Veletsos and Verbic (1973) derived the impedance functions for the rigid circular foundation on the hysteretic halfspace from the analytical expressions obtained for the elastic case. It is also possible to use an approximate procedure in which the impedance functions for the hysteretic case are obtained directly from the dimensionless coefficients of the elastic case. Using this technique, the impedance functions are given by:

$$K_{VV}^{\zeta}(\omega) = [k_{11}^{\zeta}(a_o^*,\nu_s) + i a_o^* c_{11}^{\zeta}(a_o^*,\nu_s)]K_x^* \tag{5.34}$$

$$K_{MM}^{\zeta}(\omega) = [k_{22}^{\zeta}(a_o^*,\nu_s) + i a_o^* c_{22}^{\zeta}(a_o^*,\nu_s)]K_\theta^* \tag{5.35}$$

where a subscript ζ has been added to indicate the damped case and:

$$K_x^* = \frac{8G_s^* r_o}{2-\nu_s} = K_x(1+2i\zeta_s) \tag{5.36}$$

$$K_\theta^* = \frac{8G_s^* r_o^3}{3(1-\nu_s)} = K_\theta(1+2i\zeta_s) \tag{5.37}$$

$$a_o^* = \frac{\omega r_o}{V_s^*} = \frac{a_o}{\sqrt{1+2i\zeta_s}} \cong a_o(1-i\zeta_s) \tag{5.38}$$

It is noted that the complex shear modulus affects the impedance functions in three ways. First, the static stiffness coefficients K_x and K_θ are multiplied by $(1 + 2i\zeta_s)$. Second, a_o^* is substituted for a_o. Third, the coefficients $k_{ii}(a_o,\nu_s)$ and $c_{ii}(a_o,\nu_s)$ are replaced by their complex counterparts $k_{ii}(a_o^*,\nu_s)$ and $c_{ii}(a_o^*,\nu_s)$. Neglecting only this last effect, the following values of the impedance functions including hysteretic damping are:

$$k_{ii}^{\zeta}(a_o^*,\nu_s) \cong k_{ii}^{\zeta}(a_o,\nu_s) = k_{ii}(a_o,\nu_s) - \zeta_s a_o c_{ii}(a_o,\nu_s) \tag{5.39}$$

$$c_{ii}^{\zeta}(a_o^*,\nu_s) \cong c_{ii}^{\zeta}(a_o,\nu_s) = c_{ii}(a_o,\nu_s) + 2\frac{\zeta}{a_o} k_{ii}(a_o,\nu_s) \tag{5.40}$$

This procedure, which has been proved to give a very good approximation in various cases (Wolf, 1985), has been used in this study.

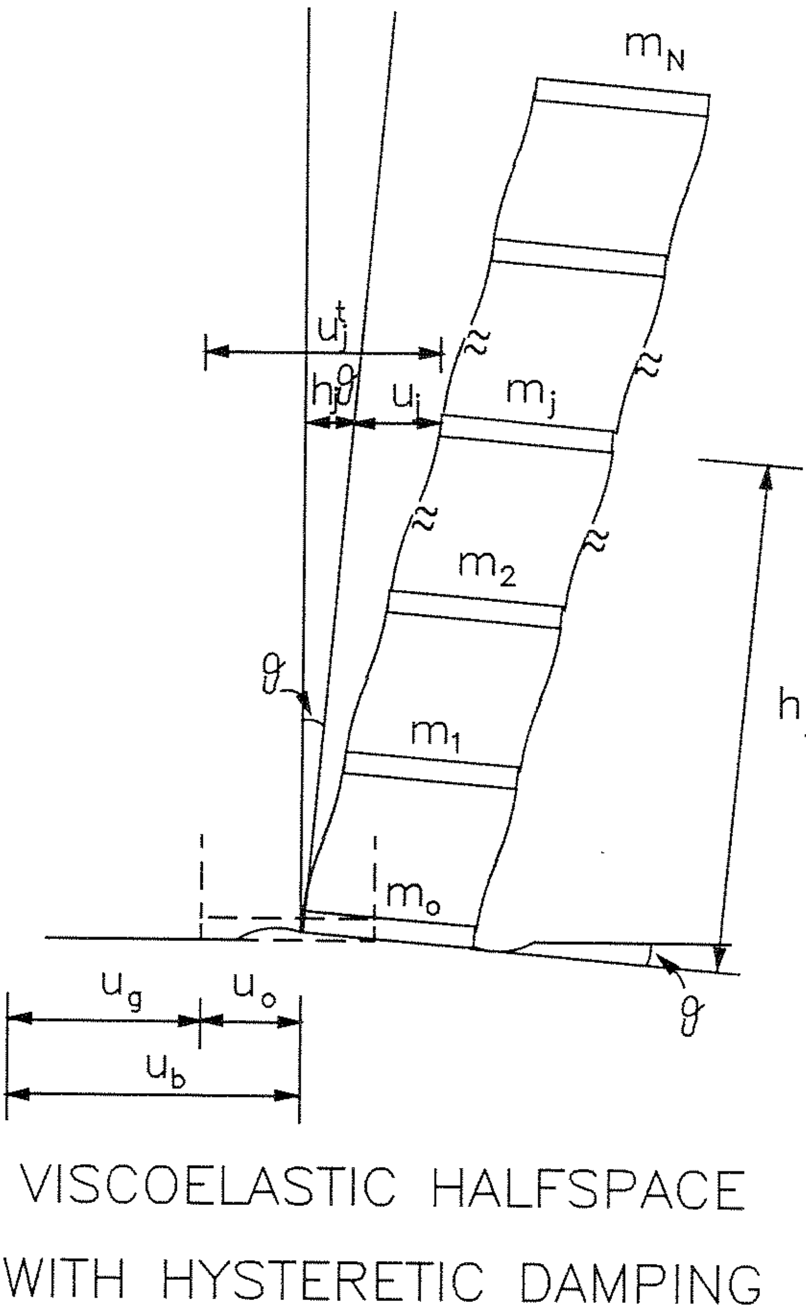

Figure 5.1 Substructure model of the building-foundation-soil system.

CHAPTER 6

CORRELATION OF THE SUBSTRUCTURE MODEL RESPONSE WITH RECORDED RESPONSE

6.1 Selection of Parameters for the Substructure Model

In this section it is described how the input parameters for the substructure model have been selected. Basically, the mass and stiffness parameters for the soil have been estimated from the available data (in-situ soil tests, previous studies), while for the structure have been obtained from the three-dimensional mathematical model. The hysteretic damping in the soil has been estimated, while the viscous damping ratios for the structure have been selected so to obtain the best agreement with the recorded response.

The material properties of interest for the soil are the unit weight γ_s, the Poisson's ratio ν_s and the S-wave velocity V_s (or the associated shear modulus G_s), and the hysteretic damping coefficient ζ_s. As mentioned in Section 2.1, the soils underlying the building are roughly uniform, and it is a reasonable approximation to use constant values for these parameters. The soil region which is of interest in the analysis of the complete system is the one affected by the forces acting on the foundation. The depth of the region depends on the dimensions of the foundation base and on the direction of the response involved, and is generally larger for the horizontal motion than for the rocking motion. For the present study is reasonable to assume for this soil region a depth of 200 ft (61.0 m), which is approximately the largest building dimension in plan. The soil boring data available (Duke and Leeds, 1962) indicated a unit weight varying from 110 lb/ft^3 (17.3 kN/m^3) to 130 lb/ft^3 (20.4 kN/m^3) within the upper 300 ft (91.4 m). A mean value of γ_s = 120 lb/ft^3 (18.8 kN/m^3) was assumed. For the Poisson's ratio a value of ν_s = 0.33 was chosen, which is reasonable for a sandy clay. For the upper 200 ft (61.0 m) soil layer, the in-situ test reported by Duke and Leeds (1962) indicated an almost constant value of the P-wave velocity of 2400 ft/s (732 m/s) and this corresponds, for the assumed Poisson's ratio, to an S-wave velocity of 1190 ft/s (363 m/s). Whereas for the dynamic model of the subsurface conditions, developed when analyzing the 1952 Kern County earthquake records obtained at the building, Duke et al. (1970) used an S-wave velocity varying from 606 ft/s (185 m/s) at the surface level up to 1820 ft/s (555 m/s) at the bottom of the layer, with a weighted average of 1360 ft/s (415 m/s). Considering that the level of strain in the soil was higher during the 1987 Whittier earthquake than during the 1952 earthquake, it is believed that a value of V_s = 1190 ft/s (363 m/s) is a reasonable estimate of the S-wave velocity. Using of γ_s = 120 lb/ft^3 (18.8 kN/m^3) reported above, the shear modulus of the soil is G_s = 36.6 kips/in^2 (253 MPa).

As mentioned in Section 5.1, the building foundation was modeled as rigid and the presence of the shallow embedment and the short piles was neglected. An equivalent circular foundation was assumed, as it has been shown that it can well approximate the stiffness of a rigid rectangular foundation for an aspect ratio as high as 4 (Gazetas, 1983), which is the case of the present study. Therefore the only foundation property is the radius of the equivalent circular disk, and two different values were used. For the translational stiffness, the equivalent radius obtained by equating the area of the contact surfaces of the circular and rectangular foundation, while for the rocking stiffness the moments of inertia around the rocking axis were equated.

The mass and stiffness properties of the building superstructure enter in the substructure model through its vibration mode shapes and frequencies. Coupling between response in the two directions was neglected, as justified in Chapter 4. For each principal direction of the building the vibration properties used were those obtained from the three-dimensional mathematical model, reported in Tables 4.3 and 4.4. Only the first three modes were considered in the transverse direction, and only the first two in the longitudinal direction, according to the effective approach for solving the equations of motion outlined in Section 5.2.

The estimation of the damping in the system is difficult, because there is no data available for the structural damping nor for the soil damping. A procedure has been used in which first a plausible value of the hysteretic damping of the soil was selected. This parameter, already introduced in Section 5.3, is measured by the dimensionless ratio:

$$\zeta_s = \frac{1}{4\pi}\frac{\Delta W_s}{W_s} \tag{6.1}$$

in which ΔW_s is the area of the hysteresis loop in the stress-strain diagram measured during a test in which the soil specimen is undergoing harmonic shearing deformation and W_s is the maximum strain energy stored in a linear elastic material with the same stiffness subjected to the same maximum stress and strain. The damping ratio is an increasing function of the magnitude of the imposed peak strain, and therefore increases with the level of the earthquake excitation. In the present study, taking into account the intensity of the earthquake motion at the building site (the maximum peak acceleration recorded by the parking lot instrument was 0.21 g) a value of $\zeta_s = 0.20$ was chosen. The lateral force resisting system is mainly composed of moment resisting frames in the transverse direction and shear walls in the longitudinal direction. It is quite reasonable to suppose that the modal viscous damping ratios were different in the two building directions. In each direction considered, the viscous damping ratio were supposed equal for all the vibration modes. The damping ratio was selected so that the substructure model shows the same maximum values of the acceleration transfer function between the free-field and the top of the building as obtained in the recorded motion. The viscous damping ratios obtained with this procedure were ξ = 3.5% in the transverse direction and ξ = 8.0% in the longitudinal direction.

6.2 Comparison Between Model and Recorded Response

Figure 6.1 shows the modulus of the transfer functions between the parking lot and the building basement, 8^{th} floor, 12^{th} floor and roof center. As also indicated in Figure 6.1, the peaks corresponding to the first and second vibration periods occur at 1.9 s and 0.48 s respectively. A wider and smaller peak occurs at approximately 0.30 s, thus indicating the period of the third vibrational mode. The corresponding curves obtained from the substructure model are shown in Figure 6.2. The very good agreement between the computed and the recorded transfer functions is evident, although the peak corresponding to the second mode is slightly longer (it occurs at at 0.56 s in Figure 6.2 and at 0.48 s in Figure 6.1), and the magnitude of the peaks of both the second and third mode in the model are larger than the measured peaks, indicating a probable underestimation of the structural damping for these two modes. The transfer function between the free-field and the building basement has almost a constant unity value for both the model and the observed response. This indicates negligible effects of soil-structure interaction in this direction. It is interesting to note that the presence of a nodal point for the second mode of vibration at the 12^{th} floor level (see Table 4.4 and Figure 4.8) gives an almost zero value of the transfer function for the 12^{th} floor at the second period of vibration, and this fact, of course detected by the model, is also clearly present in the observed response.

Figure 6.3 and 6.4 show the same observed and computed transfer functions from the free-field in the longitudinal direction of the building. The observed transfer functions do not show the simple, clear response observed in the transverse direction. The reason of the more complex behavior is probably the fact that in the longitudinal direction the parking lot record cannot be considered a free-field record for the building. The parking lot station is located 139 ft (42.4 m) west of the main building, this distance being 0.64 times the building dimension in the longitudinal direction, and this implies that its record is affected by the waves scattered from the building foundation, this effect being more pronounced at higher frequencies. Furthermore, as noted in Section 3.2, the kinematic interaction effects are noticeable in the longitudinal direction up to a period of 0.60 s, which is in the range of the fundamental vibration period in that direction.

To provide additional insight into the response in the longitudinal direction, the transfer functions from the basement to the three building floor levels have been obtained for both the substructure model and the observed response are shown in Figures 6.5 and 6.6. A much better agreement is now observed, as these transfer functions reflect the dynamic response of the building only. In Figures 6.5 and 6.6 the peaks corresponding to the first and second modes are evident, and both the amplitude of the peaks and the periods for which they occur are nearly the same.

In summary, a very good agreement is found in the transverse direction of the building between the model and the observed transfer functions, and it is found that the soil-structure interaction effects are negligible. Such a good agreement is not found in the longitudinal direction, probably because the parking lot station cannot be considered as a free-field station, and also because significant kinematic interaction is present in the same range of periods of the fundamental period of the system. In this direction, to match the computed response and the recorded one, quite high values of damping in the structure had to be chosen.

6.3 Effect of Soil-Structure Interaction on Base Shear

The transfer functions shown above have been used to compute the time history of the response of the complete system during the 1987 Whittier earthquake in both the building directions. The parking lot ground motion is transformed into its Fourier components, which are then multiplied by the corresponding frequency components of the substructure response functions. Performing an inverse Fourier transform the time history of each response quantity is obtained. Once the relative building displacements are known, the inertia forces acting at each floor level can be computed, and consequently the base shear time history. Of particular interest are the results obtained for the maximum value of the base shear V_{max}. It has been found V_{max} = 3960 kips (17.7 MN) and V_{max} = 924 kips (4.13 MN) in the longitudinal and transverse direction respectively. To compute the variation of the base shear due to the flexibility of the soil and the inertial soil-structure interaction effects, the same calculations have been repeated assuming a very high value of G_s, to simulate a fixed-base condition for the building, and it has been found V_{max} = 4750 kips (21.3 MN) and V_{max} = 927 kips (4.15 MN) for the two directions. This indicates a reduction in the base shear for the flexible supported building with respect to the fixed-base one equal to 16.6% in the longitudinal direction and there is essentially no reduction in the transverse direction.

Mode of vibration	Transverse direction [s]			Longitudinal direction [s]		
	T	$\tilde{T}$ (model)	$\tilde{T}$ (obs.)	T	$\tilde{T}$ (model)	$\tilde{T}$ (obs.)
Fundamental	1.8	1.9	1.9	0.58	0.63	0.60
Second	0.55	0.56	0.48	0.18	0.19	0.19
Third	0.29	0.30	0.30			

Table 6.1 Comparison between the vibration periods of the superstructure three dimensional model (T), and those of the complete system ($\tilde{T}$) from the model and as observed during the 1987 Whittier earthquake.

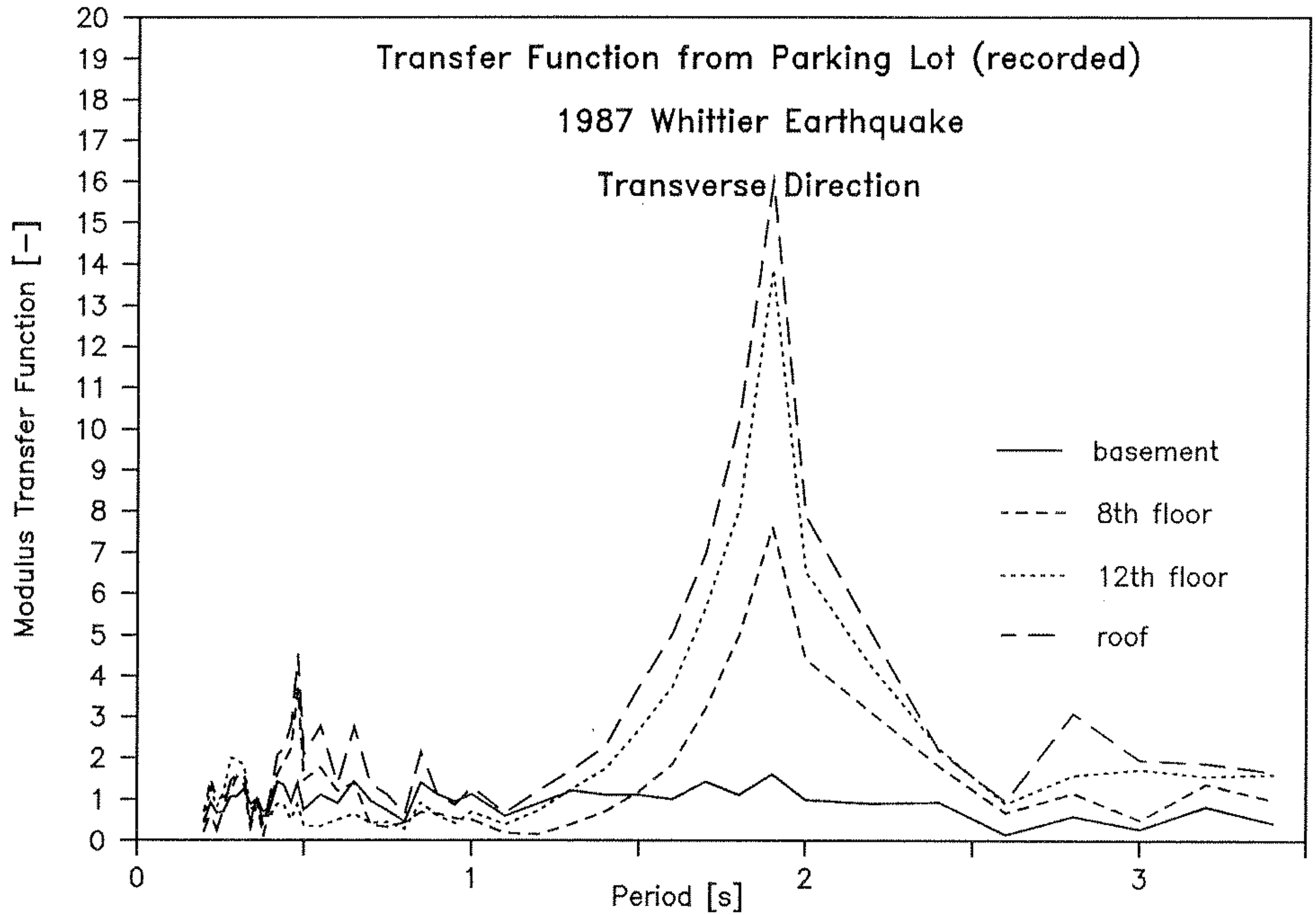

Figure 6.1 Modulus of the transfer function between the parking lot and the basement, 8th floor, 12th floor and roof center in the transverse direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

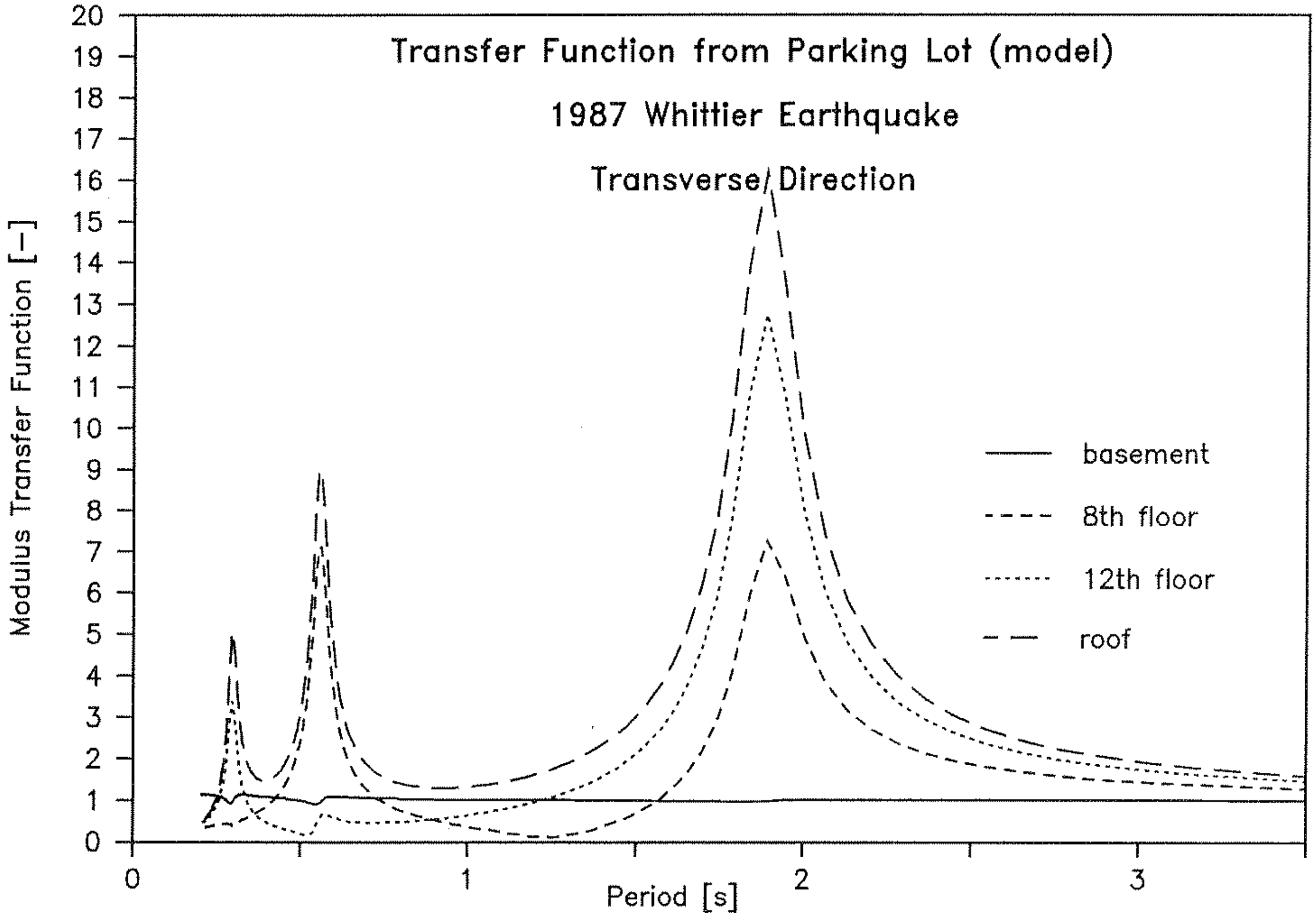

Figure 6.2 Modulus of the transfer function between the parking lot and the basement, 8th floor, 12th floor and roof center in the transverse direction computed from the substructure model.

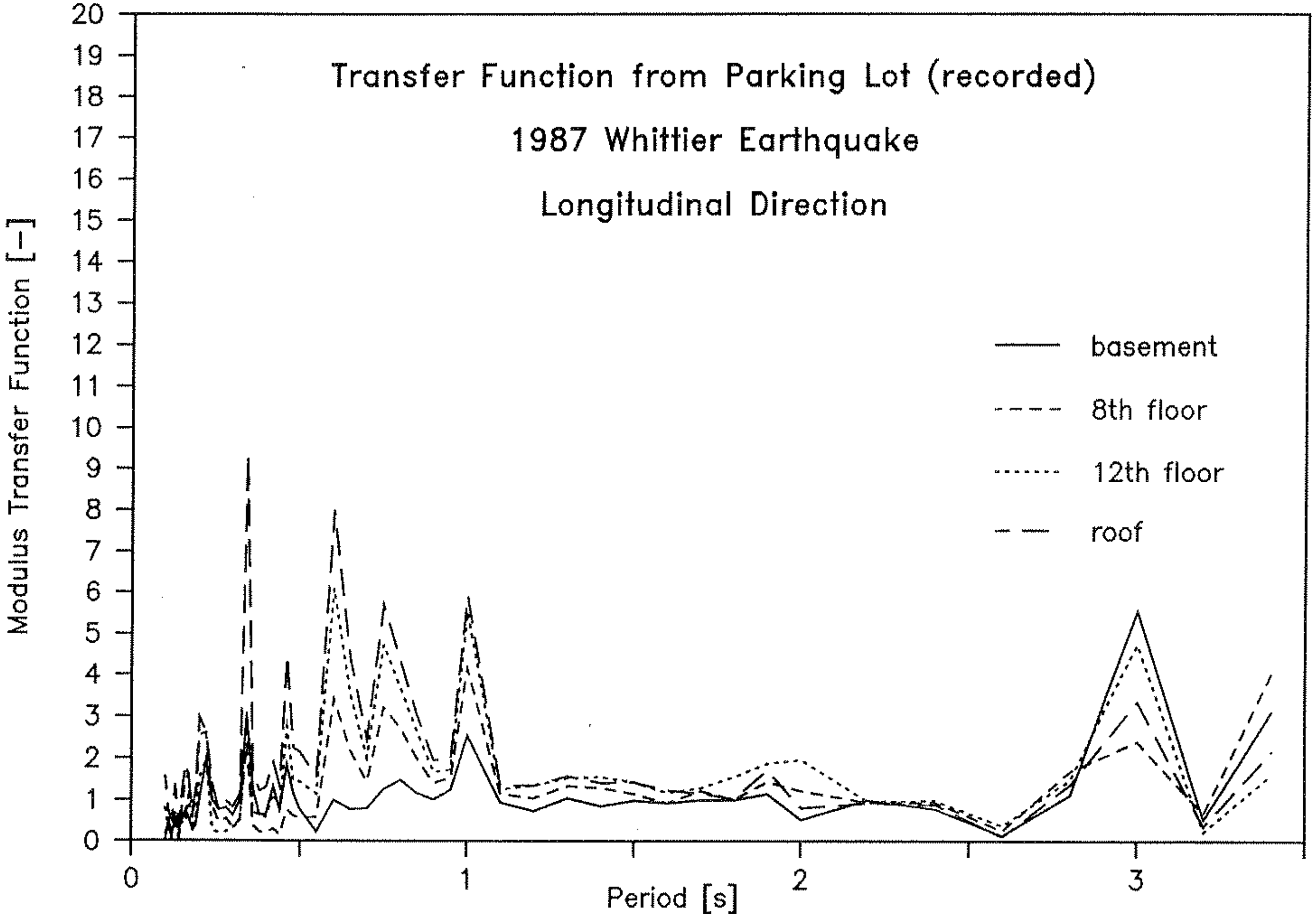

Figure 6.3 Modulus of the transfer function between the parking lot and the basement, 8th floor, 12th floor and roof center in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

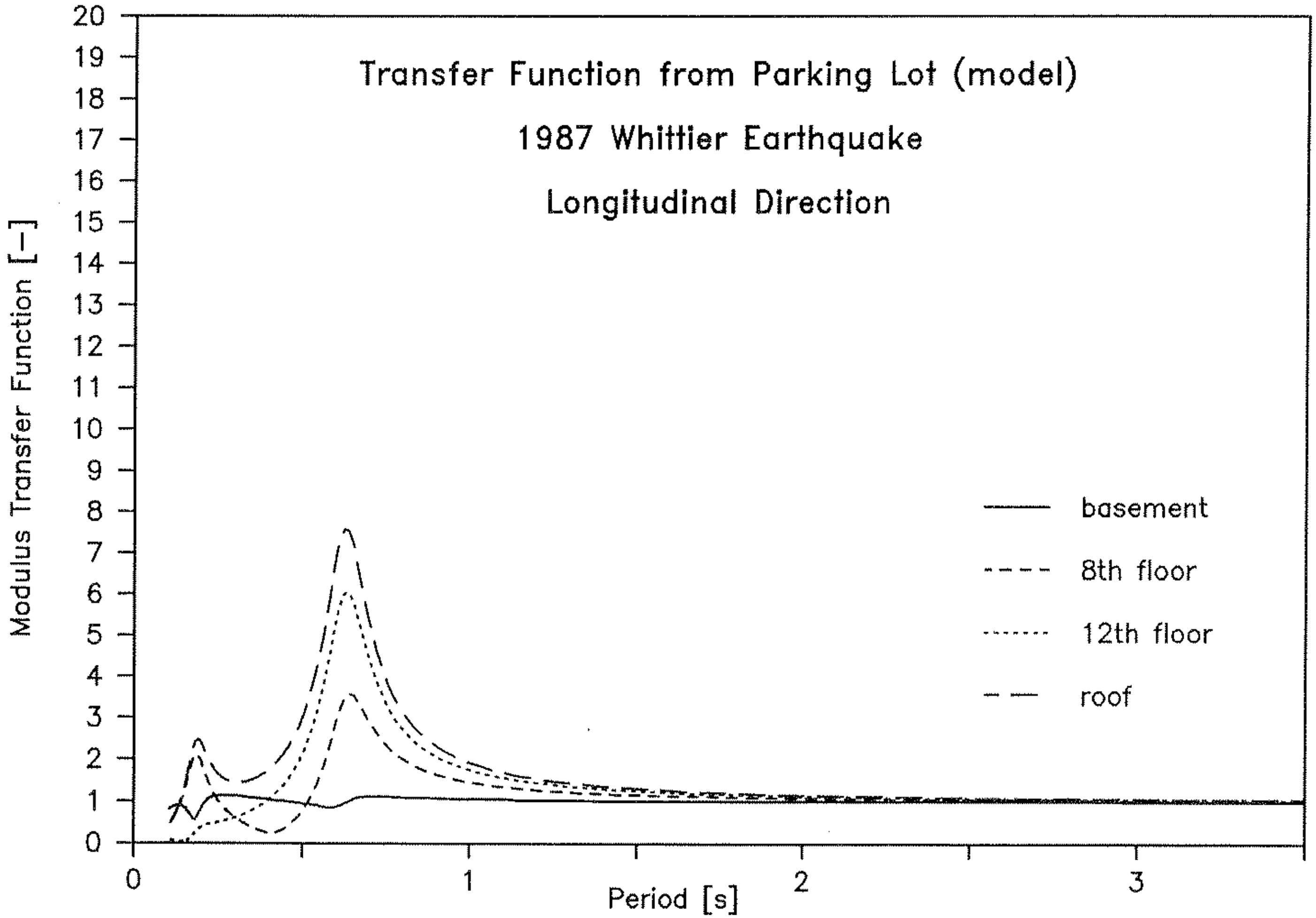

Figure 6.4 Modulus of the transfer function between the parking lot and the basement, 8th floor, 12th floor and roof center in the longitudinal direction computed from the substructure model.

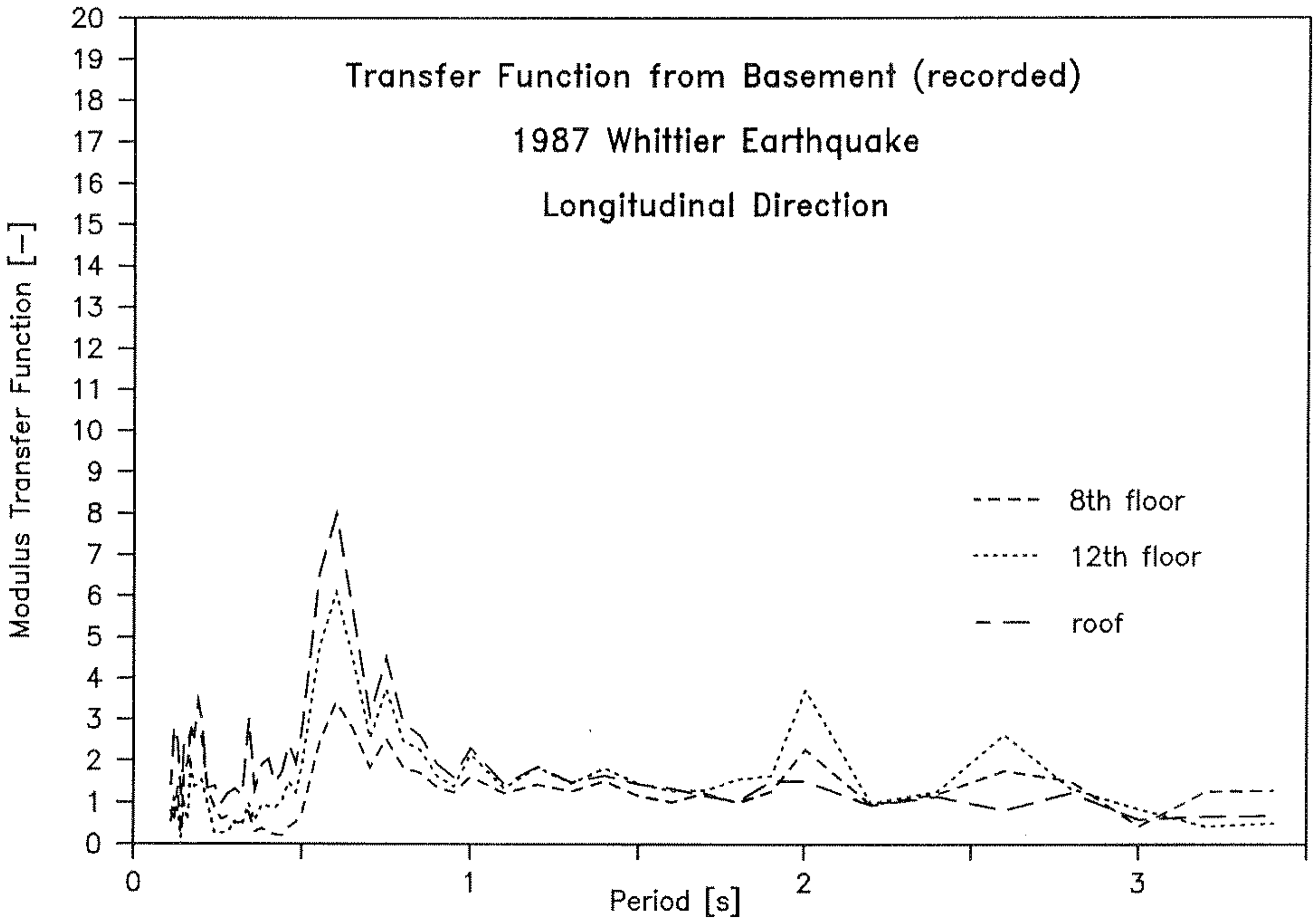

Figure 6.5 Modulus of the transfer function between the basement and the 8th floor, 12th floor and roof center in the longitudinal direction computed from the records obtained at the Hollywood Storage Building during the 1987 Whittier earthquake.

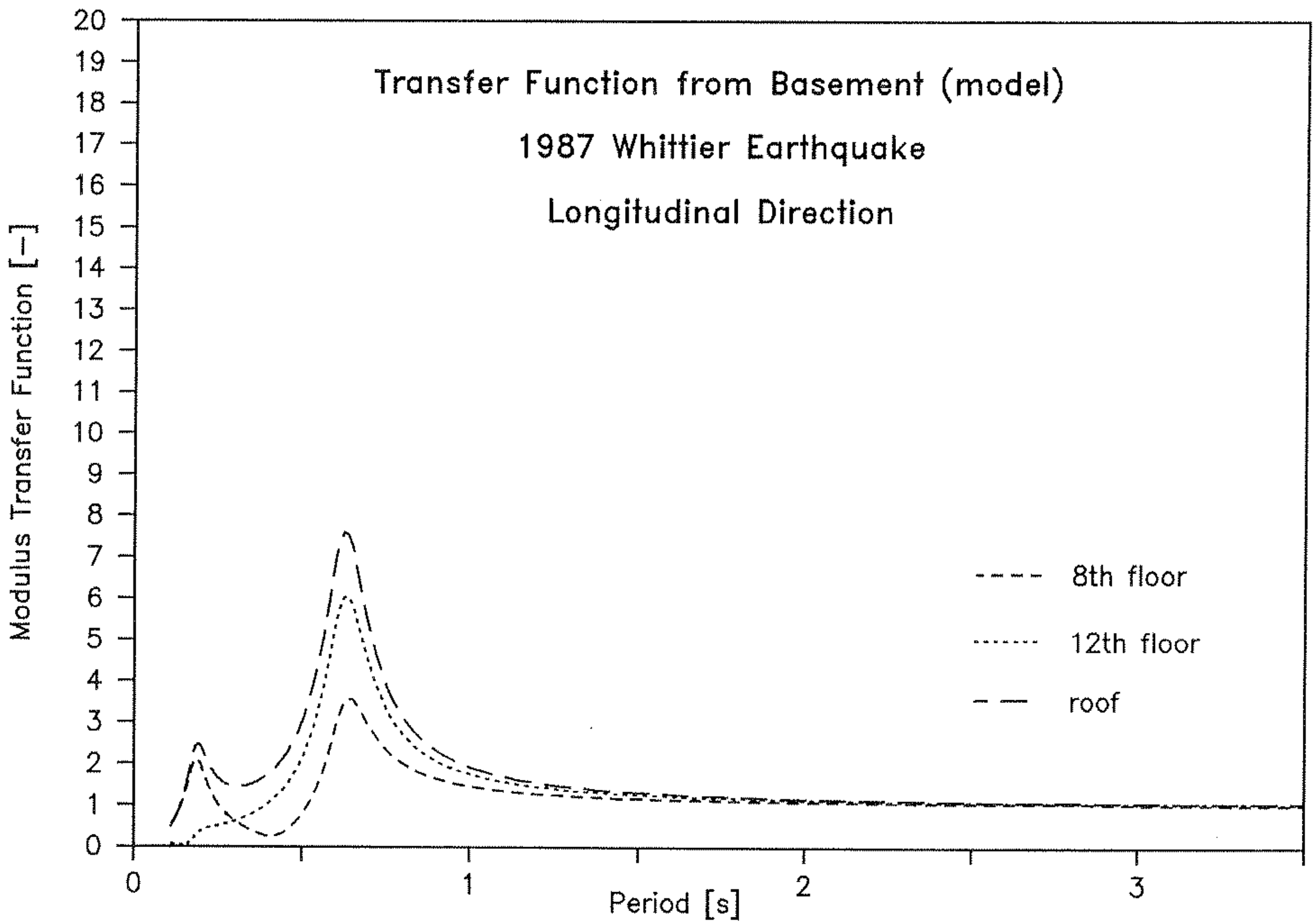

Figure 6.6 Modulus of the transfer function between the basement and the 8th floor, 12th floor and roof center in the longitudinal direction computed from the substructure model.

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