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FULL-SCALE EXPERIMENTAL STUDIES OF SOIL-STRUCTURE INTERACTION – A REVIEW

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> "Strong motion accelerograms properly interpreted are the nearest thing to scientific truth in earthquake engineering" (Duke et al. (1970).

Keywords: soil-structure interaction; full-scale testing, large-scale tests, Hollywood Storage Building.

ABSTRACT

This paper presents a review of full-scale testing of structures related to soil-structure interaction and associated phenomena. It starts with a review of the early research on soil-structure interaction (including the work of Suyehiro, Sesawa and Kanai, and Biot), and proceeds with studies of the Hollywood Storage Building, of Millikan Library in Pasadena, and more recent studies. The usefulness of full-scale testing is illustrated by presenting recent results for the Hollywood Storage Building by the authors. Finally, an analysis of general trends in full-scale testing and soil-structure interaction research is presented based on the number of publications on these topics in the Earthquake Engineering Abstracts database. This analysis shows that the number of papers on the general topic of *soil-structure interaction* peaked at 35/year during the late 1970s, and at present is only about 10/year. Papers dealing with *experimental* aspects of soil-structure-intersection do not exhibit major fluctuations, and since 1970 appear at an average rate of 3.3/year. Only about 1.2 papers/year deal with *full-scale experiments*, about 2/3 of the experimental papers being devoted to *laboratory testing*. The only year with five papers on full-scale tests involving soil-structure-interaction was 1975, and about 45% of the time there were no contributions on this topic recorded in this database. Obviously, the priorities in earthquake engineering research are not properly balanced, and this situation is alarming. While small-scale laboratory tests and computer simulations are useful for understanding selected phenomena in soil-structure interaction, they lack the completeness of the full-scale tests. Laboratory experiments are designed to measure what the researcher has decided to study and may lead to discovering new physics only by accident, while the as-built environment contains all the physical properties of reality. This paper also presents a discussion on various difficulties in interpretation of earthquake response data recorded in structures and the use of full-scale test data. The main difficulty appears to be nonuniqueness of the interpretation of these data due to inadequate strong motion instrumentation and lack of processed data recorded in structures with intermediate amplitudes (between strong motion and microtremor levels). Finally, it is recommended that soil-structure models be refined, and more detailed seismic monitoring instrumentation be installed in buildings, including rotational transducers which will measure point rotation and will help separate the contribution of foundation rocking and structural deformation from the total recorded response.

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1 INTRODUCTION

1.1 The Problem of Soil-Structure Interaction

Dynamic soil structure interaction occurs during the passage of earthquake waves through the soil-structure system. It involves scattering of the incident waves from the foundation system, transfer of incident wave energy into the structure, and radiation of the structural vibration energy back into the soil. During this process, the motion of the soil is altered relative to what it would have been in the absence of the structure. Also, the motion of the building is different from what it would have been if the soil were rigid. Because of soil-structure interaction, the soil experiences additional motion, for example, horizontal and vertical translations Δ and V, and in-plane rotation φ for in-plane excitation (see the two-dimensional example in Fig. 1), which are added to the "free-field" ground motion u_g^H, u_g^V and ψ (Todorovska and Trifunac, 1990a). The interpretation and quantitative identification of soil-structure interaction phenomena occurring during strong earthquake shaking of buildings are difficult, because at present there are no strong motion records of rotation of building foundations (angles φ and ψ in Fig.1). Typical building instrumentation consists only of translational accelerometers.

In a more general sense, soil-structure interaction is a collection of phenomena in the response of structures caused by the flexibility of the foundation soils, as well as in the response of soils caused by the presence of structures. Modeling of its effects requires additional degrees-of-freedom, and for some applications use of wave propagation methods. In general, it lengthens the apparent system period, increases the relative contribution of the rocking component of ground motion to the total response, and usually reduces the maximum base shear (Todorovska and Trifunac, 1992). The reduction of structural response results from the scattering of the incident waves from the foundation, and from radiation of the structural vibration energy into the soil. When the soil surrounding the foundation experiences small to moderate levels of nonlinear response, the soil-structure interaction will lead to significant absorption of the incident wave energy thus reducing the available energy to excite the structure. An important challenge for future seismic design is to quantify this loss and exploit it in design of soil-structure systems.

The simplest soil-structure interaction models are those in which the building is supported by a rigid foundation. These models require only six additional degrees-of-freedom (three translations and three rotations), but may be too simple for practical applications. Models with flexible foundations are rare (Iguchi and Luco, 1982; Liou and Huang, 1994) and difficult to validate against data. As far as we know, there is no strong motion program to document distortions and warping of foundations of structures during the passage of seismic waves (Trifunac et al., 1999).

The extent to which soil-structure interaction alters the system response ranges from negligible to profound, and depends mainly on the (dynamic) stiffness of the soil relative to the structure. Recorded strong motion in structures indicates that destructive shaking is often accompanied by nonlinear response of the foundation soils (Luco et al., 1986; Trifunac and Todorovska, 1998; Trifunac et al., 1999, 2001a,b), and that the time dependent changes of the apparent frequencies of the response are often due to significant contribution of soil-structure interaction (Udwadia, and Trifunac, 1974). Critical for the successful performance of base-isolation, health monitoring and control of structural response during strong ground shaking is how accurately the excitation from ground shaking is represented in the respective models predicting the structural response. Clearly, such models must include nonlinear soil-structure interaction phenomena.

1.2 Studies of Soil-Structure Interaction

Contributions to the subject of soil-structure interaction have been reviewed on numerous occasions, during conferences devoted specifically to this subject (e.g. International Symposium on Soil-Structure Interaction, 1977), symposia following World Conferences on Earthquake Engineering (State-of-the-art in Earthquake Engineering, 1981), and workshops (Celebi and Okawa, 1999). The subject has been reviewed in specialized reports (Luco, 1980) and researched in numerous doctoral dissertations (e.g. Merritt, 1953; Luco, 1969a; Lee, 1979), and books (Wolf, 1985; 1994).

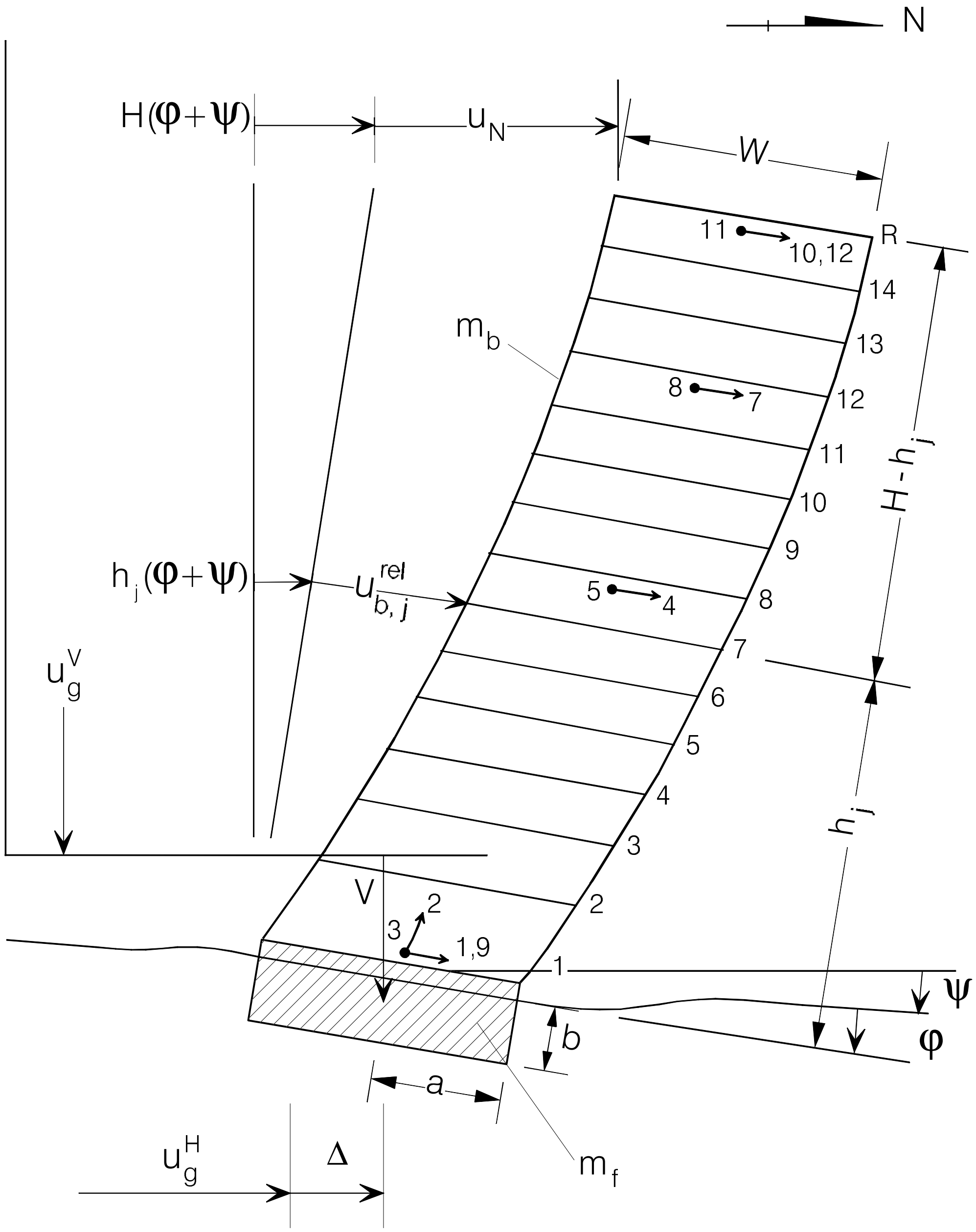

Fig. 1 Two-dimensional model of a soil-foundation-structure system excited into motion by ground translations u_g and u_g , and rotation ψ. The local soil deformation, caused by soil-structure interaction, is described by translations Δ and V and rotation φ. The building and the foundation have masses per unit length m_b and m_f. The arrows correspond to the locations of recording channels 1 to 12 in Hollywood Storage Building (see Fig 3b).

In response analyses, first a mathematical model of the full-scale structures is defined, followed by numerical calculations, which produce a simulated response for the chosen mathematical models and input motion. Mathematical models are ideally validated by full-scale experiments. The type of experiments, and scope of the measurements and data analysis depends on the available sources of excitation and instrumentation, and on the specific purpose of the experiment. Components of full-scale structures may be tested in the laboratory, but evaluation of complete structures should be performed in full-scale. This limits the experiments to completed or to similar existing structures, and cannot be performed before the structure is built. However, systematic testing of existing full-scale structures, and careful interpretation and documentation of the results can go a long way towards creating a body of intuitive understanding, physical insight, and experience on how to extrapolate, and what steps to take to get to the core of complex new problems.

Experimental studies of soil-structure interaction are best conducted in full-scale, in actual buildings during microtremors (Trifunac, 1970a,b, 1972a), forced vibrations (Blume, 1936; Hudson, 1970) and earthquake excitation (Luco et al., 1987). The difficulties of conducting experiments in the laboratory are not only due to the similarity laws that have to be satisfied, but are mainly due to modeling the (semi-infinite) half-space boundary condition. Laboratory tests can be very useful, but can never be as complete as the full-scale experiments. Even the most carefully and completely planned laboratory work will represent only those aspects of the problem, which the experiment designer chose to study and had incorporated into the model. That is, the best and the most complete laboratory tests can be used to verify and quantify mainly those aspects of the problem that the investigator knows. Except when fortunate accidents occur, we do not know how to model what we are not aware of and what we do not understand. The full-scale tests present a completely different set of practical problems, and the as-built environment contains all the physical properties of the reality. We only have to find ingenuous ways to discover, record and interpret the reality (Trifunac and Todorovska, 1999a).

1.3 Objectives and Organization of this Paper

The objective of this paper is to review full-scale testing of structures related to soil-structure interaction and associated phenomena. A review of full-scale testing of structures in general is out of the scope of this paper, as well as review of numerical simulations and forward statistical modeling of data using simplified models of soil-structure interaction. Section 2 briefly reviews the early research on soil-structure interaction, starting from the early 1930s, and including the work of Suyehiro, Sesawa and Kanai, and Biot. Section 3 continues with a review of studies of the Hollywood Storage Building in the U.S. (the first structure in California where earthquake strong motion was recorded, in 1933), starting from the work of Housner in the 1950s and through the 1990s; a separate section is devoted to this material due to its length. Section 4 presents a general review of research on soil-structure interaction and full-scale testing from the 1970s to present. Section 5 reviews specific examples of full-scale tests in actual structures, to illustrate how such tests contribute towards formulation of more realistic models and assumptions. Section 6 reviews experimental work on large model tests. Section 7 analyzes trends in research on full-scale testing and soil-structure interaction via the number of published journal and conference papers obtained by databases searches. Section 8 presents a discussion and conclusions.

2 THE EARLY RESEARCH ON SOIL-STRUCTURE INTERACTION

In this section we review the contributions of several pioneers who contributed to the first studies and to the evolution of the concept of soil-structure interaction.

Figure 2 outlines the milestones of the early work on soil-structure interaction, the years of the “major” earthquakes (from earthquake engineering point of view), and the years of the early World Conferences on Earthquake Engineering (up to the 1970s).

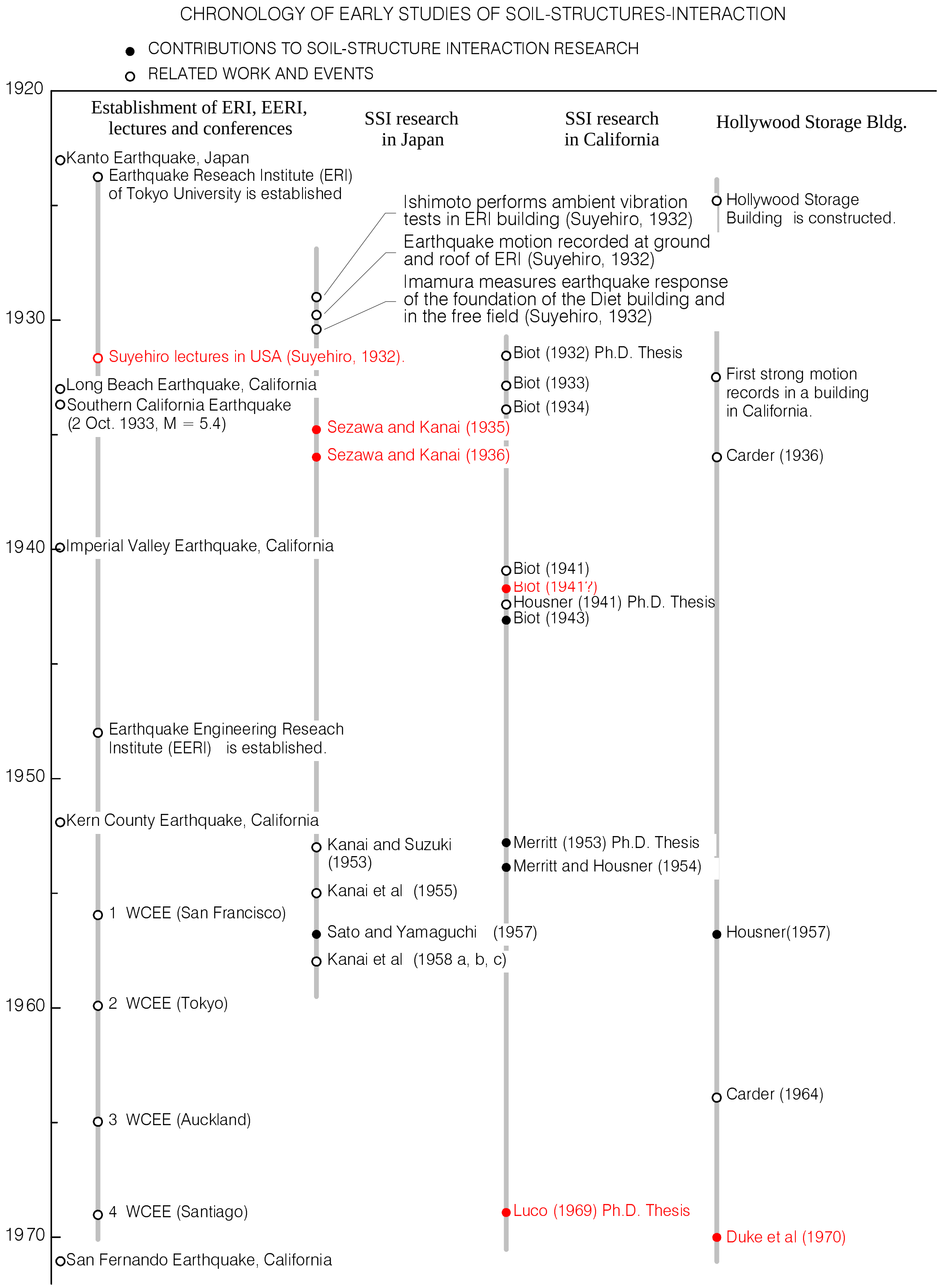

Fig. 2 Historical milestones in Earthquake Engineering, with emphasis on the early contributions to the subject of soil-structure interactrion. The years of selected earthquakes in Japan and California, of the establishment of ERI and EERI, and of full-scale experiments and earthquake measurements in Hollywood Storage Building are also shown.

2.1 The Work of Suyehiro

In the fall of 1931, Professor Kyoji Suyehiro visited the United States and presented a series of three lectures on Engineering Seismology (Suyehiro, 1932). His third lecture (III) entitled "Vibration of Buildings in an Earthquake" is of particular interest for this review. It seems that the term Engineering Seismology - *Jishin Kogaku* - was first used at this time (Kanai, 1983). In this lecture Suyehiro discussed the response and observed damage of "rigid", "medium rigid" and "weak" buildings situated on "soft" (loose clay) and "rock" ground. He explained how the "rigid" building "moved as a rigid body on the ground-bed" and suffered little or no damage. In contrast the "weak" buildings on "rock" ground were either damaged or destroyed. Searching for an explanation Suyehiro states that "very probably the primary cause is the yielding of the ground-bed due to oscillation of the foundation…". He concluded "such cushioning action of the ground at the time of an earthquake may serve more or less to relieve the destructive action of a strong earthquake in the case of masonry [i.e. rigid] buildings". These remarkable observations were confirmed many times by earthquake damage patterns seen since 1932. The most recent observations of damage following the Northridge, California, earthquake of 1994 are no exception. The readers may peruse the papers by Trifunac and Todorovska (1998; 1999b) to see how insiteful and meaningful were Suyehiro's conclusions 62 years prior the 1994 Northridge, California, earthquake. Suyehiro then describes microtremor measurements in the Earthquake Research Institute, in the building and on the adjacent ground. Professor Ishimoto performed these measurements in 1929 (see Fig. 55 on page 91 of Suyehiro's lectures).

In Fig. 57 (page 93) Suyehiro shows the records of an earthquake on November 26, 1930, taken on the roof and ground level of the Earthquake Research Institute Building. After about seven seconds of recording, this record goes off scale. From similarity of the recorded motions at the roof and on the ground, Suyehiro concludes that the relative deformation of the building was small: "*From these facts, it can be inferred that the dynamic stress induced in a strongly constructed rigid building by an earthquake is likely to be equal to the static stress which would be induced, had the building been subjected to the static load of the intensity given by the mass of the building multiplied by the horizontal acceleration of the seismic vibration*". These and two other examples *of vibrations caused by earthquakes* (Marounochi Building, Fig. 59 on page 96 and in Yurakukan Building, Fig. 60 on page 97 of Suyehiro, 1932) appear to be the first, full-scale records of building motions caused by earthquakes.

It is fascinating to read how Suyehiro describes the scattering of short waves from a "rigid" foundation and the resulting averaging (smoothing) action of the foundation. "*According to observations made by Professor Imamura on the vibration of the Diet Building during construction, some very rapid ripples, having a period of about 0.1 sec., disappeared in the motion of the foundation although the foundation moved about as much as the neighboring ground*".

Professor Ishimoto's investigation of the velocity of ripples on the ground is very useful in this connection. According to him, "*on the surface of the ground where our Institute building stands, the P-wave has a velocity of above 120 m. per sec. And the S-wave about 65 m. per sec. Therefore, very probably, the wavelength of ripples having a period of 0.1 sec. is between 6.5 to 12.0 m.; hence, they are less than the linear dimensions of the building. Consequently, a building on soft ground is not sensitive to those quick and short ripples. It may also be mentioned that this fact may be attributed to a certain extent to another behavior of the vibration of soft ground, in which the amplitude of the component of a seismic vibration of very short period decreases quickly with depth. Therefore, foundations at some depth below the surface will be less affected by the rapid components of seismic vibrations.*"

In his published lectures, Suyehiro does not use the modern term "soil-structure-interaction", but it is obvious that one of the main topics of his lecture III is in fact soil-structure-interaction. Of course, from today's viewpoint, his observations were intuitive and for the most part qualitative, but his insight and ability to interpret observations were remarkable.

Kyoji Suyehiro was the member of Imperial Academy, Professor of Applied Mechanics of Tokyo Imperial University and Director of Earthquake Research Institute. He died on April 9, 1932.

2.2 The Work of Sesawa and Kanai

To seismologists, the names Sezawa and Kanai are synonymous with their prolific contributions to the theory of surface wave propagation (Ewing et al., 1957) and to general (theoretical) seismological studies, starting in the 1920'. Later on, Kanai independently pioneered the studies of microtremors, for which he is known and recognized worldwide. Sezawa and Kanai (1935, 1936) also made pioneering studies of soil-structure-interaction. Their method was based on wave propagation, and even though they did not use the term "soil-structure-interaction", that is what they were studying. Their stated aim was to investigate the *"decay of the seismic vibration of a simple or tall structure by dissipation of their energy into the ground."* Summarizing this work and his observations over the span of 50 years, Kanai (1983) writes: *"the excellent agreement between the calculated waveform and the observed seismograms seems to indicate that most of the vibrational damping of the buildings and dams during earthquakes occurred at the contact surface between the structure and the ground."* To better appreciate the mathematical formulation and the physical insight of Sezawa's and Kanai's work, it is helpful to begin by reading Luco (1969b) and Trifunac (1972b).

During the 1950s, Kanai and coworkers carried out numerous tests on full-scale structures. They studied the influence of the ground stiffness on the response of structures (Kanai et al., 1953; 1955a,b), using excitation by microtemors, vibration generators (Kanai et al., 1958a,b) and earthquakes (Kanai et al., 1958c).

2.3 The Work of Biot

"Earthquake Engineering as such could be considered to have been born with Biot's concept of a response of an idealized structure to ground motion" (Krishna, 1981). Here Krishna is referring to the second chapter in Biot's Ph.D. Dissertation, defended at Caltech in 1932, and entitled "Vibration of Buildings During Earthquakes" (Biot, 1932). These ideas were further refined and published during the following two years (Biot, 1933; 1934). Nine years later, Biot briefly returned to the subject of Earthquake Engineering, describing computation of response spectra by means of a mechanical analyzer (Biot, 1941), and formulating the general theory and principles of response analysis and response spectrum superposition in Biot (1942).

It is not known that Biot was also working on the subject of soil-structure interaction. In an unpublished note (Biot, 1941 ?) he states *"the problem is extremely complex because it involves a complete knowledge of the propagation and properties of the seismic waves in the strongly heterogeneous surface layers of the earth, as well as their diffraction and reflection by objects built on the surface…" In the present investigation, we have attempted to answer the following question: What is the influence of the elasticity of the ground on the rocking motion of a building? How resistant is the surrounding soil to the rocking displacement of a foundation; what are the factors influencing this rigidity, and can we expect this effect to have a practical influence in the action of earthquakes on buildings? The problem is simplified by neglecting the radiation of elastic wave due to the rocking"*. The ideas and equations from this unpublished note appear in an abridged form in Section V entitled "Influence of Foundation on Motion of Blocks" of Biot's (1942) paper. One cannot but marvel how well Biot understood the physical nature of the problem, almost 30 years before the "modern" research results started to evolve in 1970s.

M.A. Biot (1905-1985) was an engineer, physicist and applied mathematician. After graduating in electrical and mining engineering and in philosophy, and receiving D.Sci. degree (1931) from the University of Louvain (Belgium), he went to Caltech where he received a Ph.D. (1932) in aeronautical sciences. He was a student and then collaborator of Theodore von Karman. He taught briefly at Louvain, Harvard, Columbia, Caltech and Brown Universities. As an independent scientific consultant, he worked for Shell Development, Cornell Aeronautic Laboratory, and Mobil Research. A man of great and unique talent, he worked without students and essentially alone.

3 STUDIES OF THE HOLLYWOOD STORAGE BUILDING

The Hollywood Storage Building (HSB Fig. 3) is the first structure in California equipped with permanent strong motion accelerographs, in 1933. It is also the first building in California where strong motion was recorded (October, 1933), and also the first building for which it could be shown that both theoretical analysis and observation of soil-structure interaction are consistent (Duke et al., 1970). This building served as a testing ground for intuitive (Housner, 1957) and theoretical and quantitative (Duke et al., 1970) studies of soil-structure interaction. The data recorded in and near this building was also used in several other related studies, for example, scattering of waves by a "rigid" foundation, the associated "filtering" of high frequency motions and the associated torsional excitation of the foundation (Cloud, 1978; Gupta and Trifunac, 1990: Shioya and Yamahara, 1980; Whitley et al., 1977). Since 1933, there were numerous triggers of the strong motion acceleraographs in in this building, but only a few, so far, have been processed and are available for analysis. This building was also studied using ambient and forced vibration tests (Carder, 1936; 1964).

3.1 The Early Work

In 1957, Housner performed visual comparison of the accelerograms recorded in this building and in the parking lot, during the 1952 Kern County earthquake, as well as comparison of the response spectra (spectral velocity, SV) computed by an analog computer (Housner and McCann, 1949). He concluded that there is *"significant difference between the east-west components of motion recorded in the basement and in the parking lot..."* and *"no significant difference"* between the north-south components of motion.

We reexamined his conclusions after recomputing spectra (Fourier and response spectra) for the recorded motions. Figure 4 shows Fourier amplitude spectra of acceleration (NS, EW and vertical components) recorded in the basement (at the south-west corner) and at the "free-field site", in the parking lot, 112 feet to the west (Fig. 3). These spectra were computed by FFT. A comparison of the spectra at the two sites shows no significant differences. No significant difference is seen also between the NS and EW motions. Beyond about 5 Hz, the spectra of horizontal motions in the building are smaller than in the free-filed, as expected, indicating scattering of the incident waves from the building foundation. The amplitudes and the frequency content of this scattered energy obviously depend on the azimuth of wave arrivals. Further, we compared the relative velocity spectra (SV) presented by Housner (Figs 3 through 6 in his 1957 paper) with relative velocity spectra computed by standard data processing programs (Trifunac and Lee, 1973; 1979). The results of this comparison are shown in Figure 5, showing SV spectra versus undamped oscillator period, as follows. Figures 5a compares the spectra at the "free-field site", for the NS (top) and (EW) components. The bold dashed lines correspond to Housner's spectra and the weak solid lines to the spectra we computed; the different curves correspond to five damping values (0.0, 0.02, 0.05, 0.10 and 0.20). Figures 5b compares the spectra at the basement. It is seen that for periods longer than about 1 s, Housner's spectra show major discrepancies with the digitally computed spectral amplitudes. For shorter periods, the agreement is somewhat better, except in Fig. 5b (bottom) where major discrepancies are seen for all frequencies. Apparently, the accuracy of computing response spectra with an analog computer (Housner and McCann, 1949) was poor, or perhaps there is also an error in scaling, close to a factor of two. Finally, in Fig. 5c, we show a comparison only of the zero damped SV spectra, plotted on the same graph for the basement and for the "free-field" motions. It is seen that Housner's conclusion that for EW motions "the *spectra show an appreciable difference between the parking lot and basement*" was incorrect. Much of Housner's (1957) analysis and interpretation, based on these spectra, is also incorrect.

Housner also discussed the coupling of building and soil motions. He stated *"if the horizontal coupling were strong, then the oscillation of the building in this fundamental mode would impart a periodic motion to the base and this would be recorded on the accelerogram. If this were the case the spectrum curve for zero damping would have a peak at the fundamental period of vibration..."*. In the case of Hollywood Storage Building the spectral analysis shows that there are no such peaks in the spectra. The weak link in Housner's reasoning can be understood by perusal of several analytical solutions of the related problem, for anti-plane motions (Luco 1969b; Trifunac 1972b) and in-plane motions (Todorovska and Trifunac, 1990a). If $\widetilde{T}$ is the system period and Δ is the horizontal displacement of the foundation, these analytical solutions

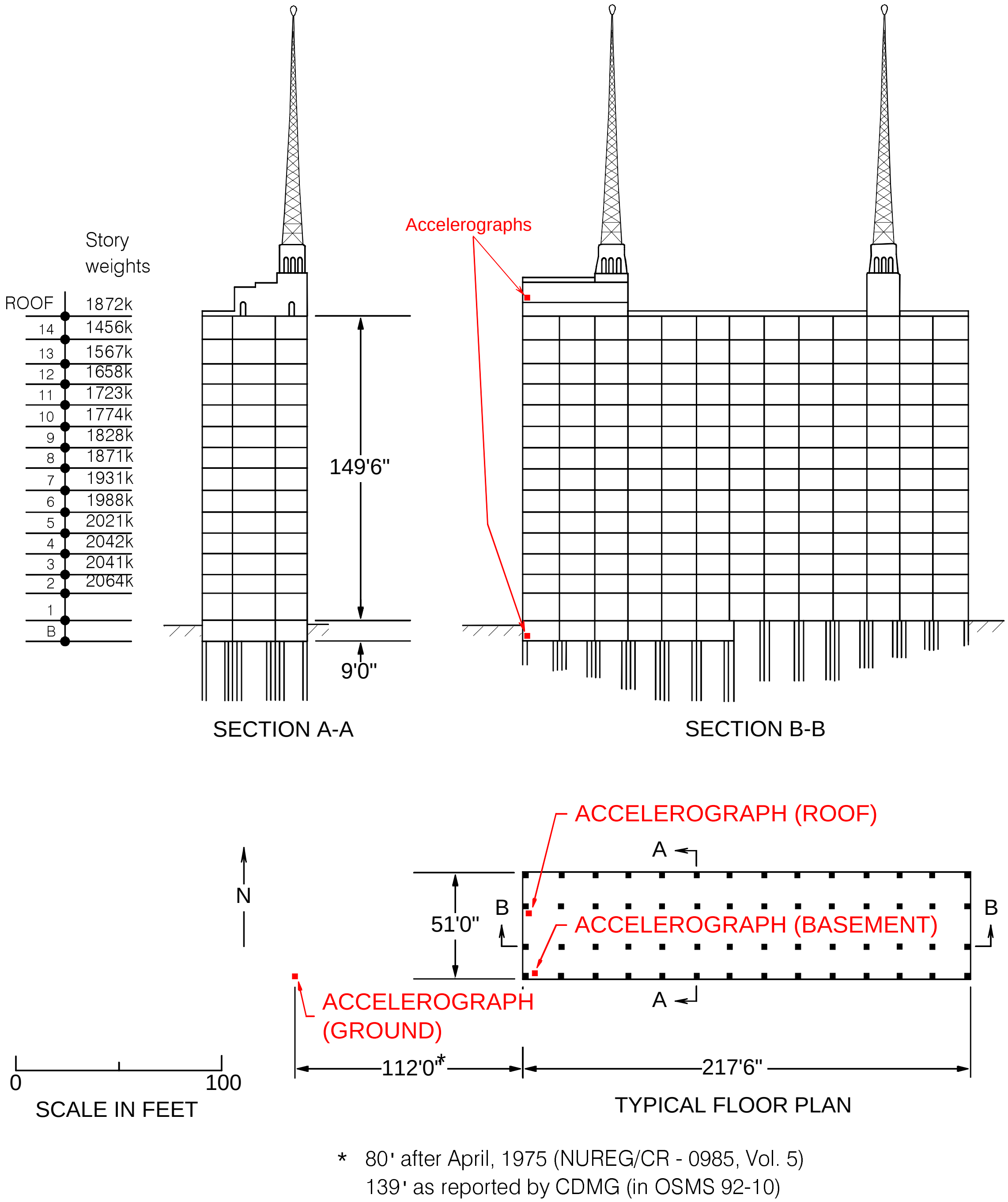

Fig. 3(a) A sketch of the Hollywood Storage Building in the early 1950s. The location of the strong motion accelerographs is indicated, one in the basement, one on the roof and one at a "free-field" site, 112 feet west of the south-west corner of the building (after Duke et al., 1970).

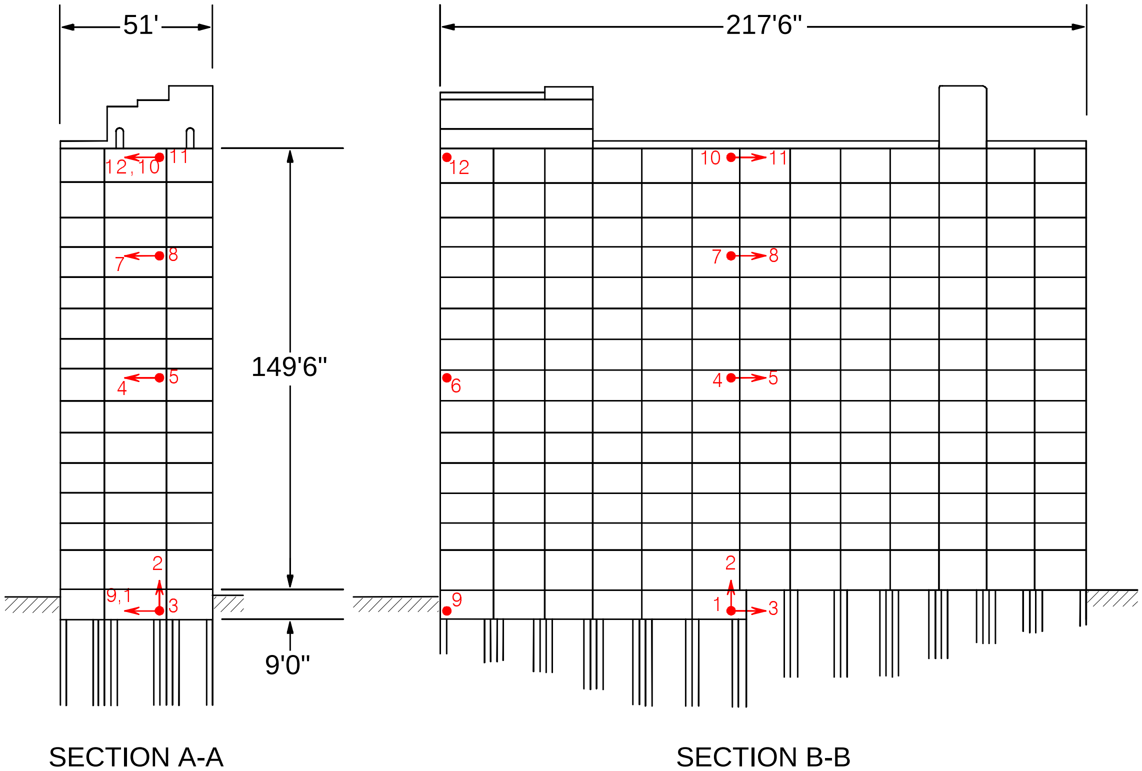

Fig. 3(b) A sketch of the Hollywood Storage Building showing the sensor locations and orientation for its strong motion instrumentation since 1976 (twelve accelerometers), maintained by California Division of Mines and Geology.

how that $|\Delta| \to 0$ near $T=\tilde{T}$. We recall that the apparent system period, $\tilde{T}$, is related to the first fixed-base period T_1 and to the translational and rocking periods T_h and T_r (respectively the periods of a rigid building vibrating horizontally and in rocking) via

$$\tilde{T}^2 = T_1^2 + T_h^2 + T_r^2 \tag{1}$$

Consequently, if the soil-structure system were to emit harmonic energy into the surrounding soil, it would do this at period $\tilde{T}$ instead of T_1. Larger amplitudes of ground motion near period T_1 however can occur between the nodes of the standing waves generated by interference of the incident and the waves reflected and scattered from the foundation (Trifunac, 1972b).

Further on, Housner appears to base his reasoning on the assumption that the foundation of the Hollywood Storage Building is "*sufficiently strong*" (i.e. rigid). He observes that *"the 51 ft width of the building (see Fig. 3) is relatively small in comparison with the wavelengths in the soft foundation soil, however it is reasonable that the 217 ft length would tend to iron out the high frequency components of ground motion"*. At the end, he concluded "*if the Hollywood storage building had been only one or two stories high, it would have undergone stresses approximately 40 percent smaller in the 217 ft direction than in the 51 ft direction. It thus appears that on very soft ground a low, stiff building is benefited by very large dimensions...*". However, if the foundation is flexible and follows the soil deformations during the passage of earthquake waves (Trifunac et al., 1999), the differential motion of the first story columns can lead to much larger stresses than what would be computed by conventional spectral analyses (Trifunac, 1997; Trifunac and Todorovska, 1997).

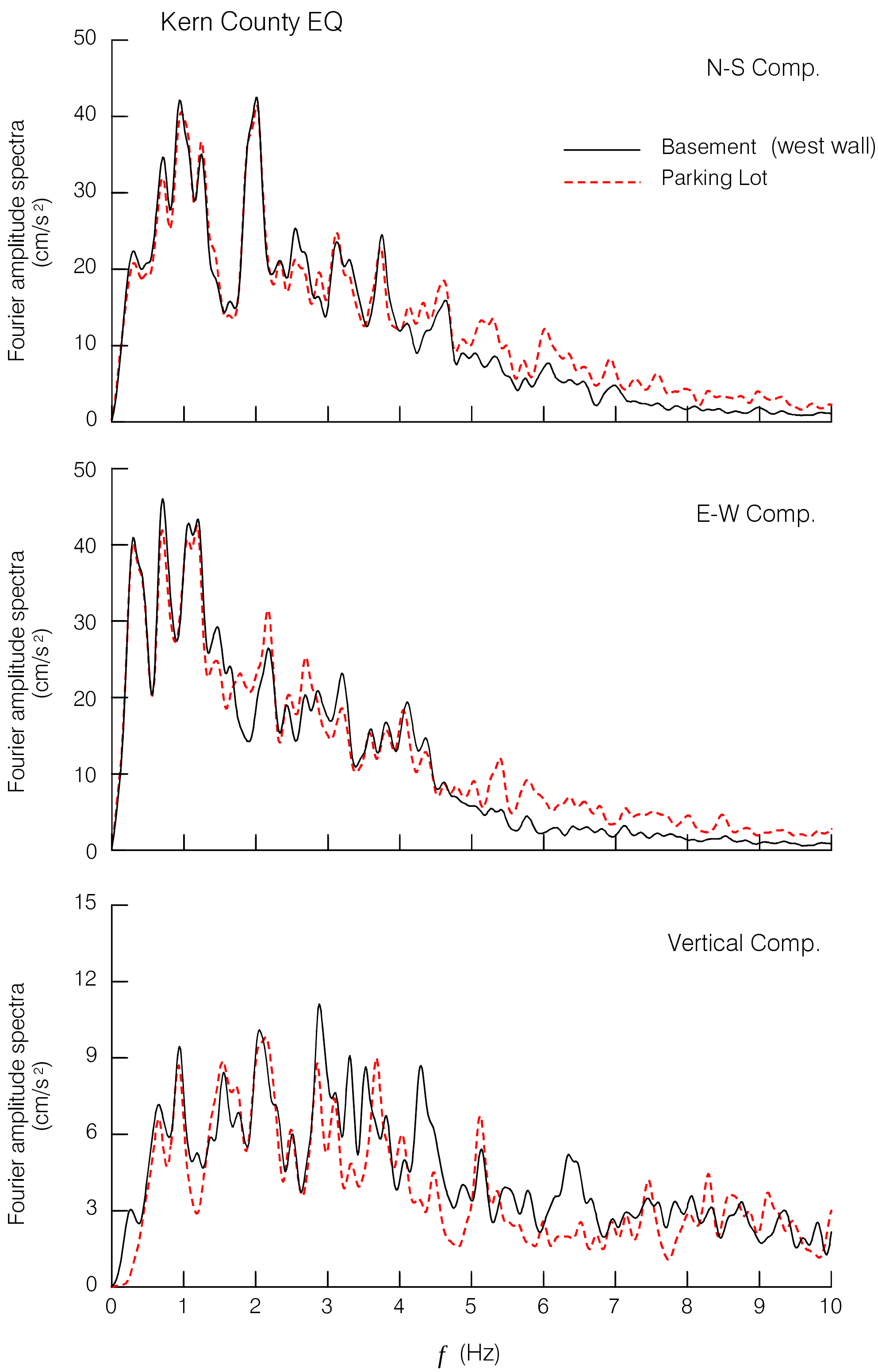

Fig. 4 Comparison of Fourier amplitude spectra of acceleration for the NS, EW and vertical components recorded in the basement of Hollywood Storage Building (at the west wall) with those recorded at the "free-field" site (dashed lines).

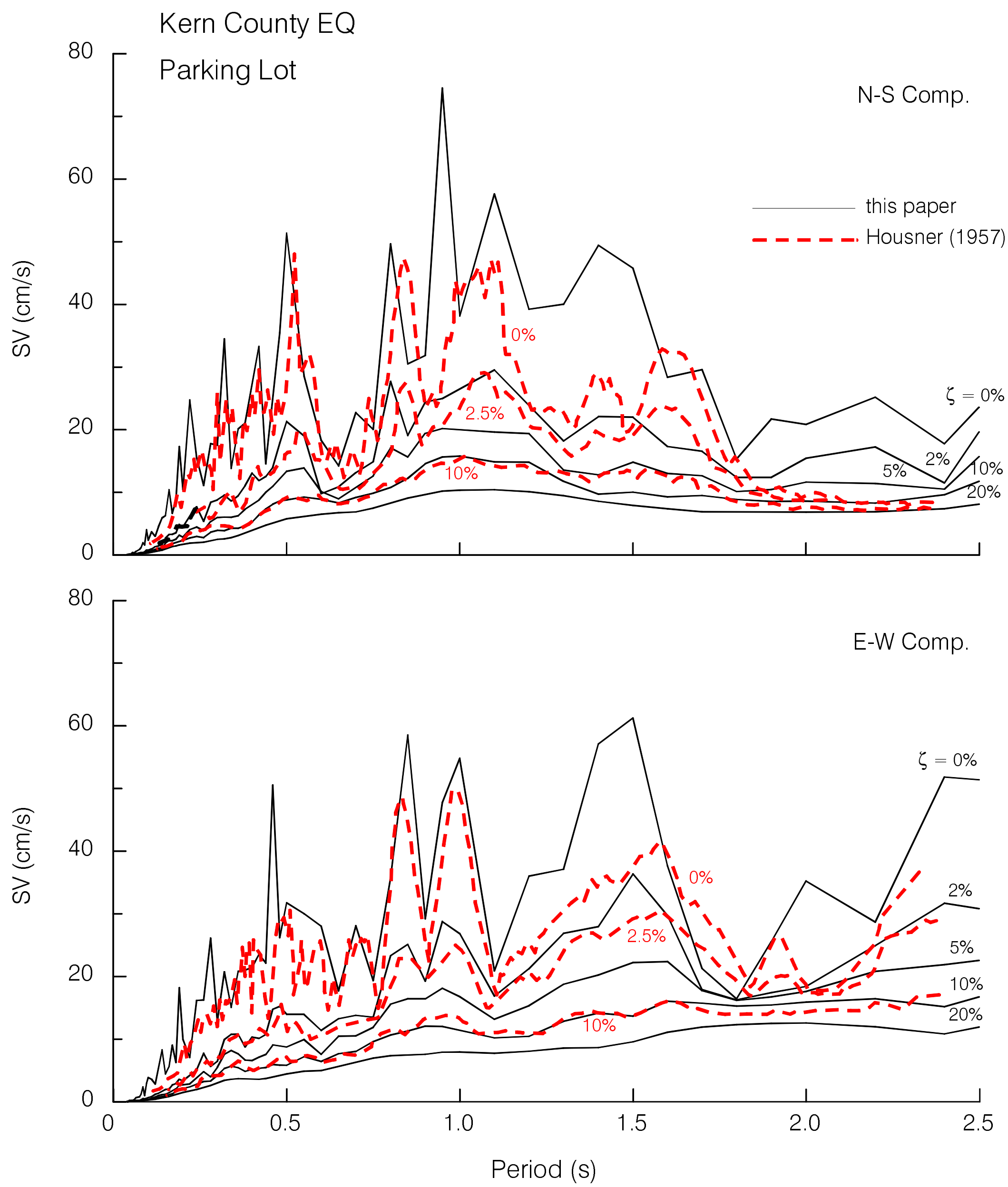

Fig. 5(a) Comparison of relative velocity response spectra (SV) of motions from the Kern County, 1952, earthquake, recorded in the Hollywood Storage Building, computed by a digital computer (Trifunac and Lee, 1973) (light solid lines), with those presented by Housner (1957) (heavy dashed lines), computed with an electrical analog computer (Housner and McCann, 1949). N-S (top) and E-W (bottom) motions recorded at the "free-field site" (parking lot).

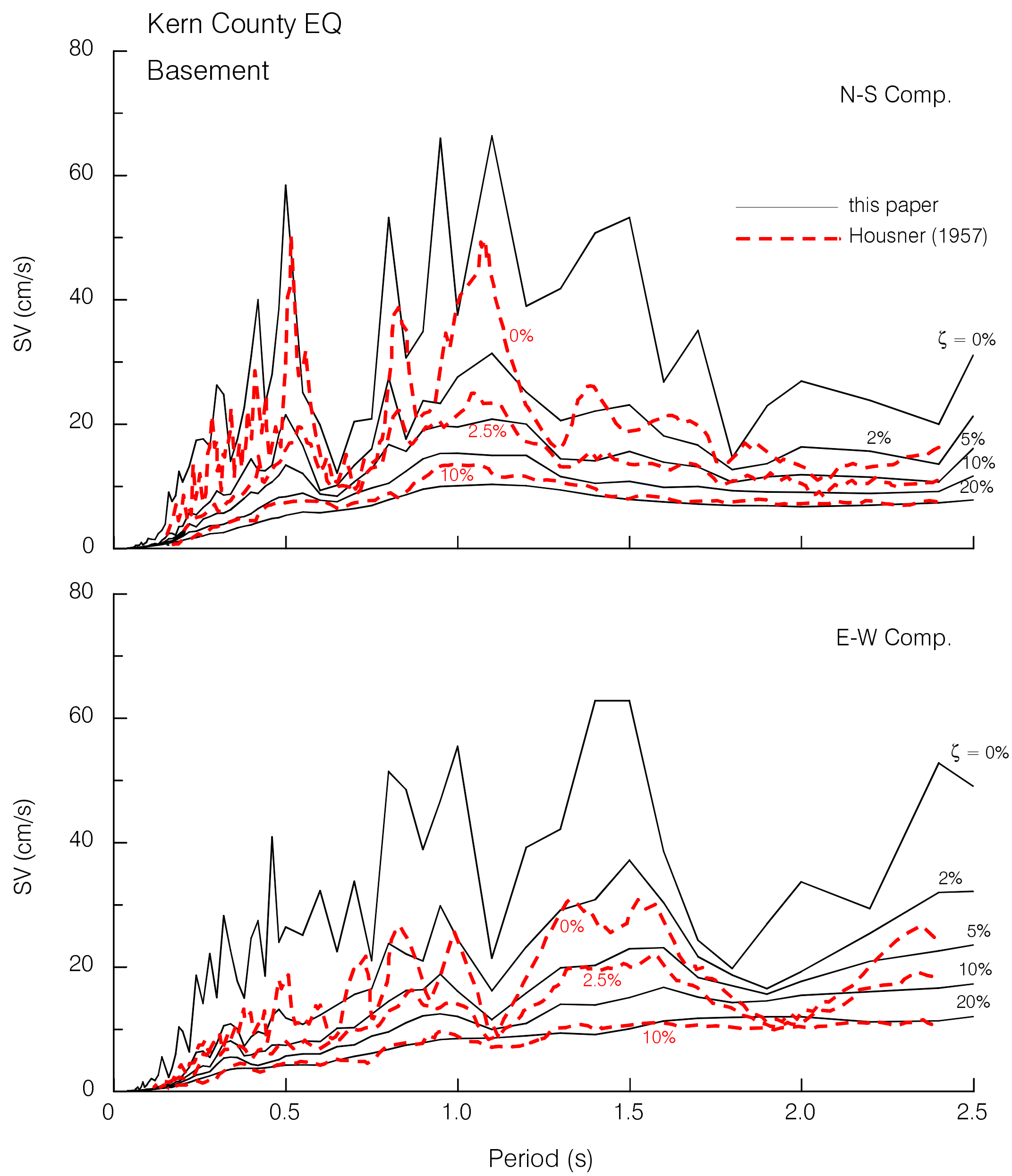

Fig. 5(b) Comparison of relative velocity response spectra (SV) of motions from the Kern County, 1952, earthquake, recorded in the Hollywood Storage Building, computed by a digital computer (Trifunac and Lee, 1973) (light solid lines), with those presented by Housner (1957) (heavy dashed lines), computed with an electrical analog computer (Housner and McCann, 1949). Same as Fig. 5(a) but for motions recorded in the basement.

3.2 The Later Work

As illustrated above, studies of the effects of soil-structure interaction may be based on comparison of the motions recorded in the structure with those recorded at a "free-field site" (typically several hundred feet away from the structure). It is usually assumed that the "free-field" record approximates the motions in the absence of the structure (Trifunac, 1972b). The transfer-functions between the foundation motion and the corresponding motions at the "free field site" are then used in the analysis. The first successful interpretation of observed data, using analysis of this type, was presented by Duke et al. (1970). They interpreted the EW recorded motions (along the longitudinal building direction, see Fig. 3) of the Hollywood Storage Building in terms of an analytic solution of soil-structure interaction, with a rigid hemi-cylindrical foundation, and for vertically incident SH waves (see the dashed line in Fig. 6, Luco, 1969b). Duke et al. (1970) did not interpret the soil-structure interaction for NS (transverse) response, because at that time a theoretical solution did not exist for in-plane motion. Figure 7 shows results for in-plane motion of a model similar to the two-dimensional model of Luco (1969b) and Trifunac (1972b), i.e. a shear wall supported by a cylindrical foundation embedded into an elastic homogeneous half-space, redrawn from Todorovska and Trifunac (1990a). It shows the foundation horizontal displacement, Δ, rocking angle, φ, relative horizontal displacement of the top of the shear wall, u_b^{rel}, foundation vertical motion, V, and relative vertical displacement of the top of the shear wall, v_b^{rel}, for incident unit amplitude P- and SV-waves with incident angle γ=30º and Rayleigh waves with unit horizontal amplitude on the surface, and for different model parameters ($\varepsilon=\beta H/\beta_b a$, H/a, W/H, m_s/m_f, m_s/m_f, where a is the half width of the foundation, H and W are the shear wall height and width, and m_b, m_f and m_s are the mass of the shear-wall, foundation and soil replaced by the foundation). It is seen that, as for the solutions of Luco (1969b) and Trifunac (1972b) for SH wave excitation, Δ has minima at the natural frequencies of the building. However, for all other frequencies, the transfer-functions for Δ are complicated and different for different-incoming waves, and angles of wave incidence. Thus, selecting a simple model and formulating an interpretation in terms of transfer-functions of recorded horizontal motions only, is difficult. In contrast, transfer-functions for vertical motion (V) are simpler and more similar for all incident waves and incident angles. The transfer-functions of rocking motions (φ in Fig. l) are very dependent on the type of incident waves. Since actual strong motion consists of all body and surface waves, the observed transfer-functions for in-plane motion (assuming linear behavior of the foundation soils), would be more complicated and different from any of the Δ transfer-functions illustrated in Fig. 7.

In the analysis of Duke et al. (1970) and Todorovska and Trifunac (1990a; Fig. 7), it was assumed that the building foundation can be represented by a hemi-cylindrical rigid mass. Clearly this is a very rough approximation for the foundation system of the Hollywood Storage Building, which is on Raymond concrete piles 12 ft to 30ft long (Fig. 3). Thus, if this foundation is to be modeled by a rigid equivalent foundation, it would be good to select some more representative embedment ratios, as this affects the nature of the waves scattered from the foundation (Wong and Trifunac, 1974). It is more likely however that this foundation does not behave like a rigid body, especially for intermediate and high frequency waves (Trifunac et al. 1999). How to represent soil-structure interaction with flexible three-dimensional foundation has not been studied so far in sufficient detail to allow any definite interpretation, and so we leave this interesting topic for a future analysis.

An example of an early analysis of the rocking period of a rigid building on flexible soil can be found in Biot (1942). Merritt and Housner (1954) also investigated the rocking motions, from which Housner (1957) concluded *"significant effects could be expected only with exceptionally soft ground"*. It is interesting to note that Housner (1957) and Duke et al. (1970) papers appear to have left an impression on subsequent researchers, who state for example that the *"evidence of soil structure interaction can be quantitatively detected in the frequency domain by the ratio"* $\left|\Delta + u_g^H\right| \ / \ \left|u_g^h\right|$ (e.g. Hradilek et al., 1973). Rocking and torsional contribution to interaction are rarely addressed in papers which aim to interpret earthquake accelerograms recorded in buildings.

Duke et al. (1970) concluded "*soil-structure interaction produced marked change in the horizontal base displacements, in the east-west direction...*" with little or no rocking in this direction. For the north-south

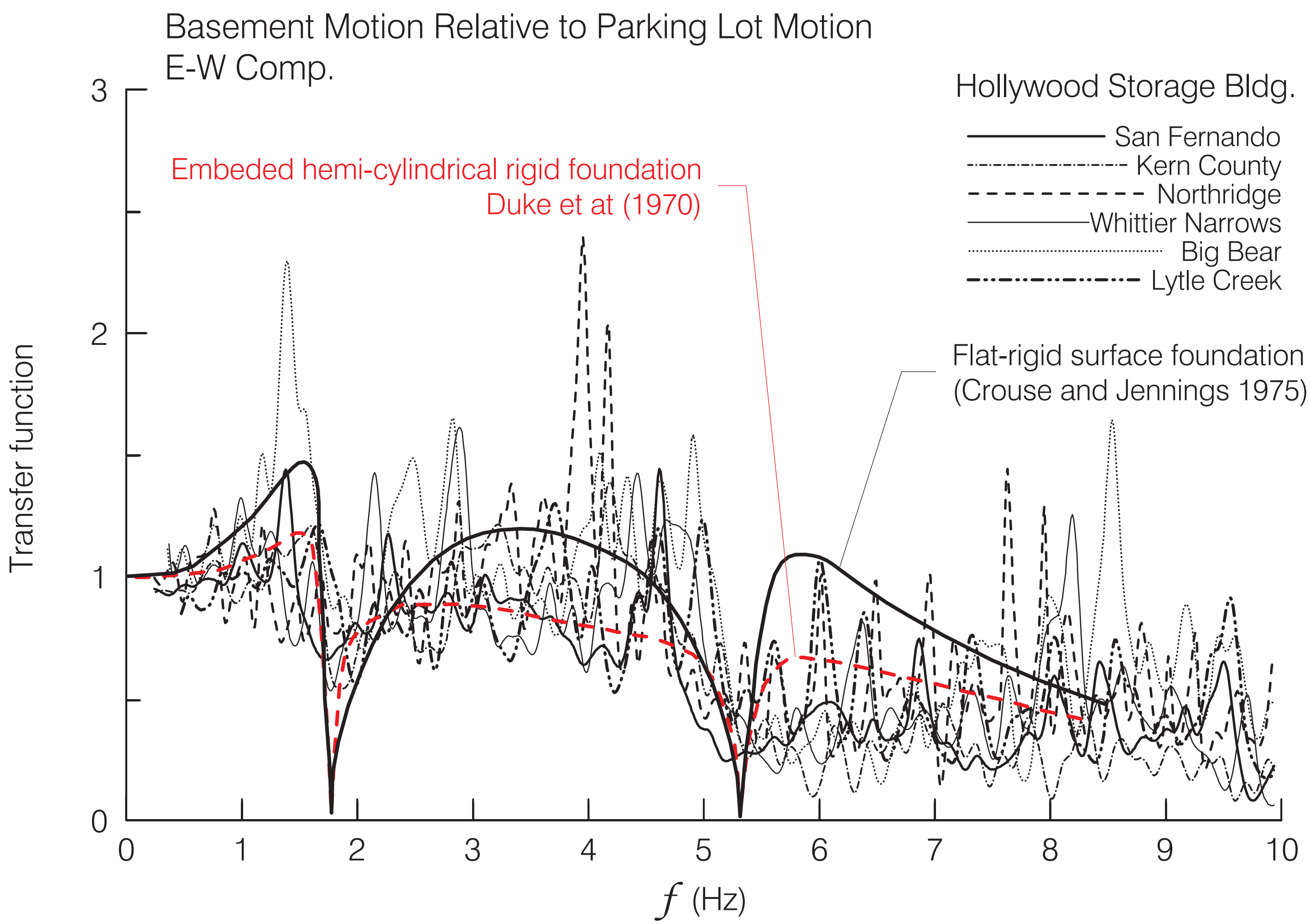

Fig. 6 E-W acceleration transfer-function between motion recorded at the basement of Hollywood Storage Building and at the "free-field" site (in the parking lot, about 100 feet west of the building, see Fig. 3) during six earthquakes. The heavy solid and dashed lines correspond to two examples of theoretical transfer-functions.

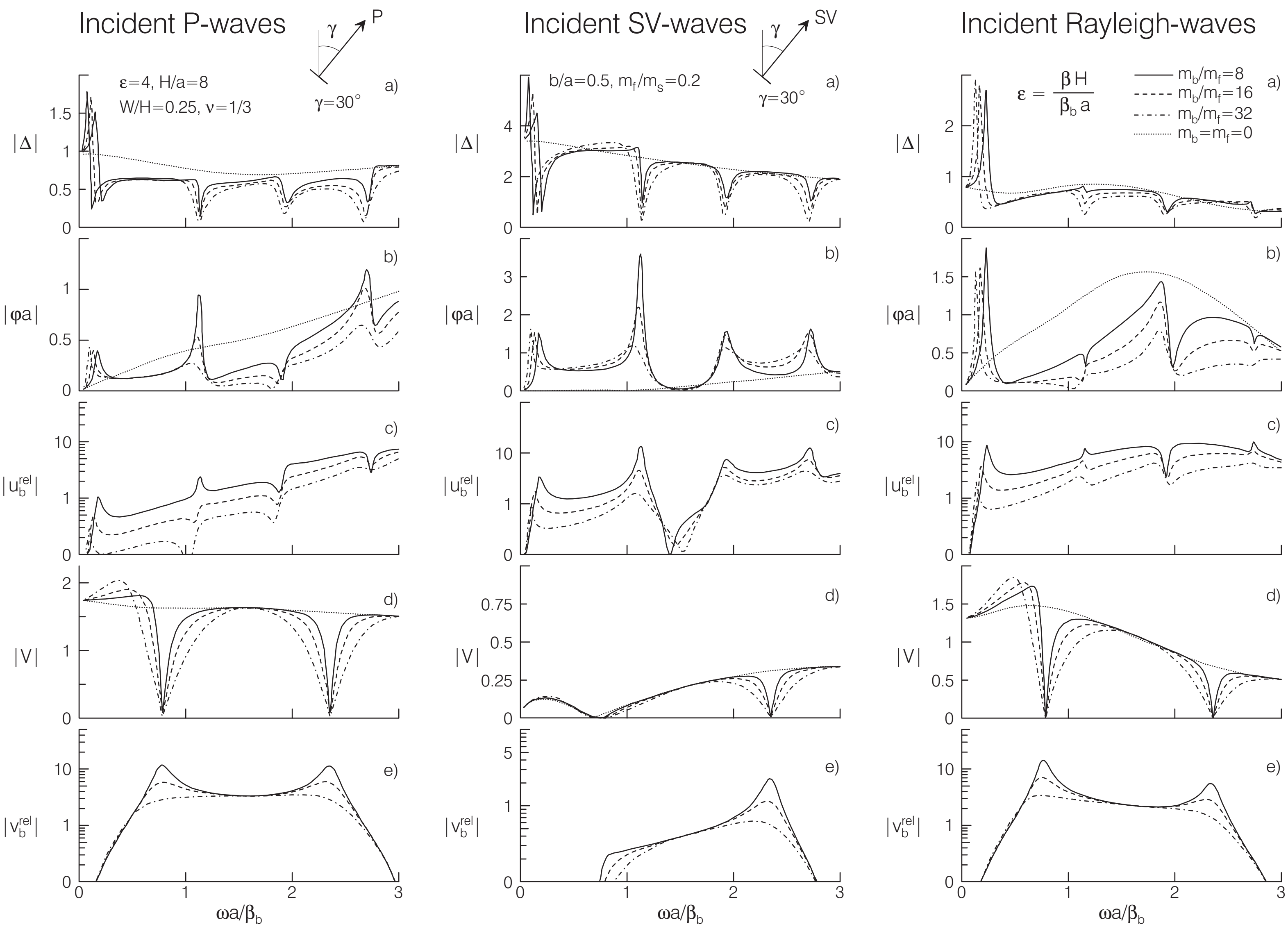

Fig. 7 Transfer-funcation amplitudes for (a) horizontal motion of foundation, Δ, (b) rocking of foundation, φa, (c) relative horizontal motion of building, u_b^{rel}, (d) vertical motion of foundation, V, and (e) relative vertical motion of building, v_b^{rel}, versus dimensionless frequency $\omega a/\beta_b$ for incident P-waves (left), SV-waves (center) and Rayleigh waves (right) (after Todorovska and Trifumac, 1990a).

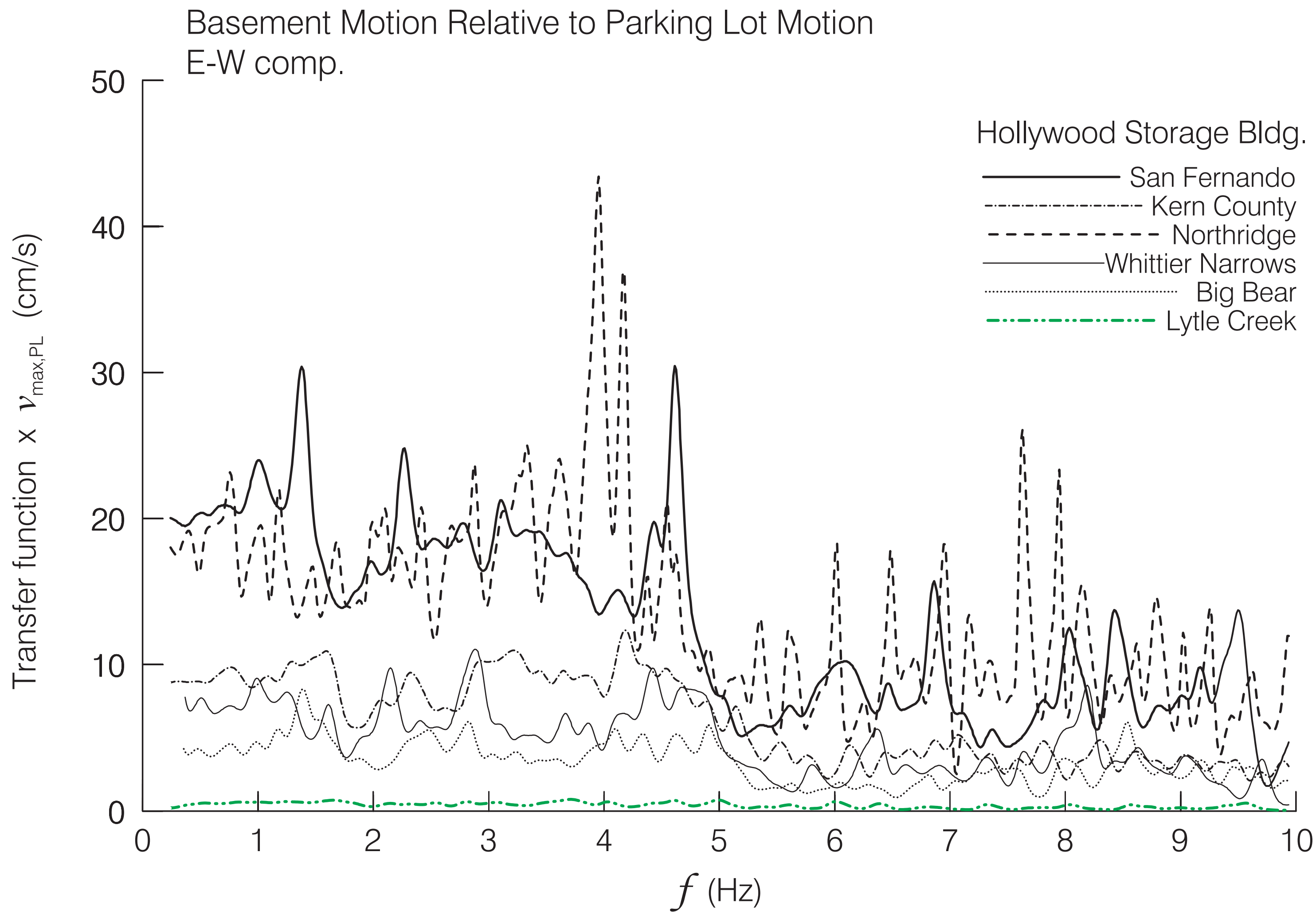

Fig. 8(a) Transfer-functions between EW motions recorded in the basement of Hollywood Storage Building and at the "free-field" site, myltiplied by the peak velocity of the motion at the "free-field" site, $v_{max,PL}$, during six earthquakes.

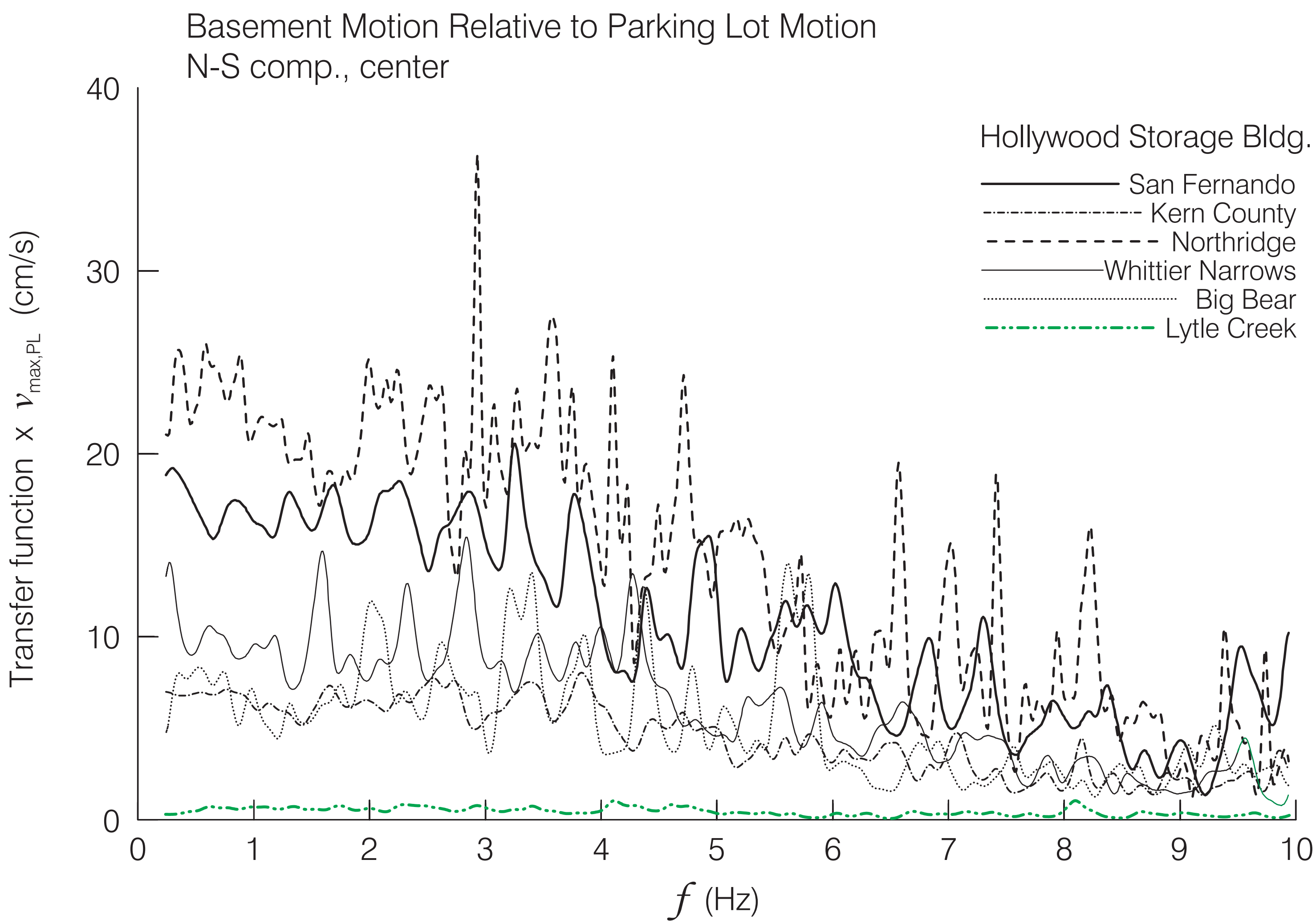

Fig. 8(b) Transfer-functions between NS motions recorded in the basement of Hollywood Storage Building and at the "free-field" site, multiplied by the peak velocity of the motion at the "free-field" site, $v_{max,PL}$, during six earthquakes.

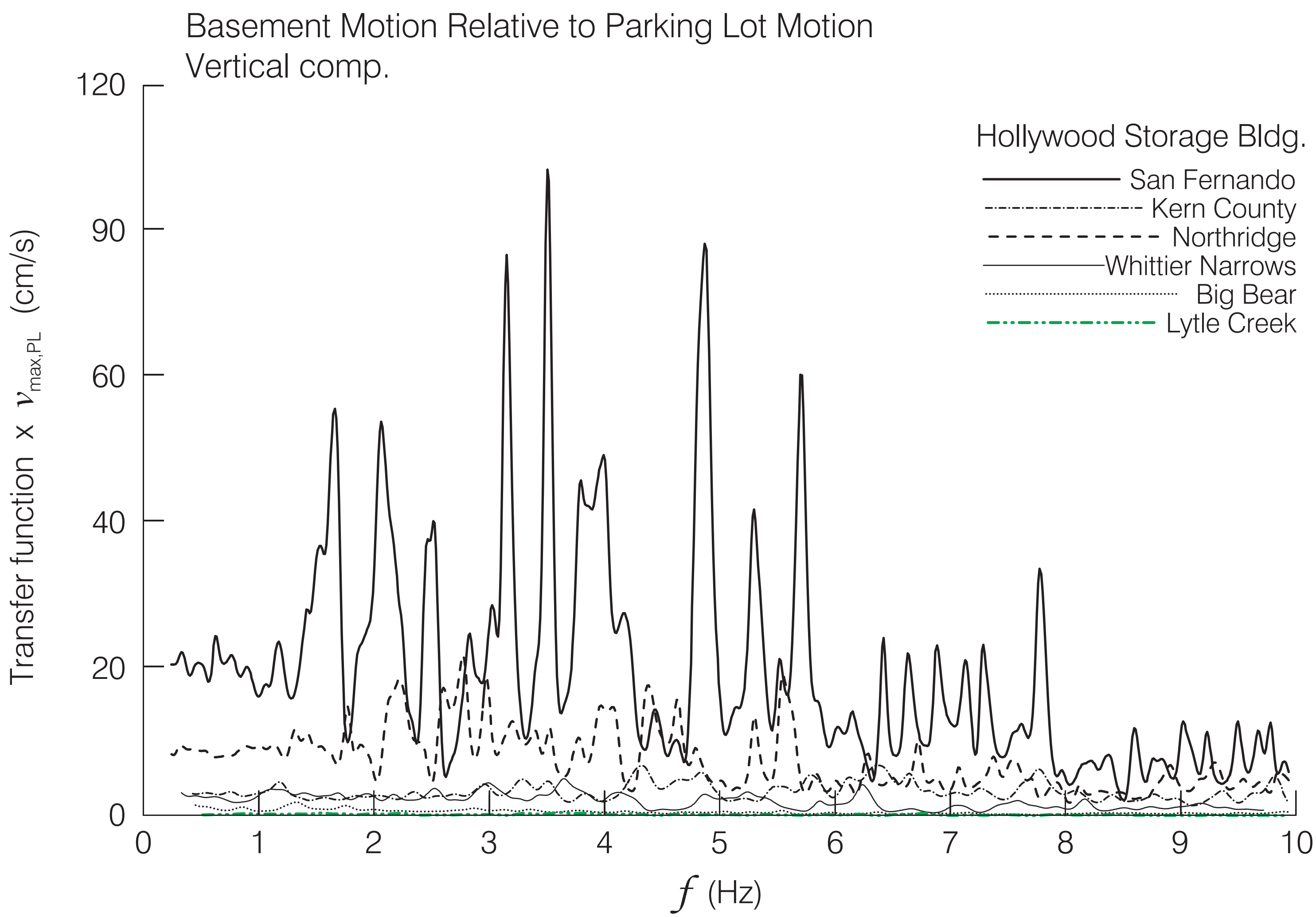

Fig. 8(c) Transfer-functions between vertical motions recorded in the basement of Hollywood Storage Building and at the "free-field" site, multiplied by the peak velocity of the motion at the "free-field" site, $v_{max,PL}$, during six earthquakes.

direction, the soil-structure interaction did not affect drastically the horizontal base displacements, but produced rocking of the foundation, as can be observed by analysis of the roof motion.

Crouse and Jennings (1975) compared strong motion recorded during the 1952 Kern County earthquake with a new set of strong motion accelerograms obtained during the 1971 San Fernando earthquake. They noted *"the differences in the experimentally determined transfer functions for the two earthquakes at the site of the Hollywood Storage Building are somewhat larger than might have been hoped. In studying these differences it was noted that there are some features of the experimental transfer functions that suggest a non-linear effect in the soil. If the soil had a lower modulus during the larger strains experienced in the San Fernando earthquake, the transfer functions for San Fernando event would be lower for high frequencies, and peaks, in general, would be to the left of corresponding peaks in the transfer function for the weaker motion of the Arvin-Tehachapi (Kern County) earthquake. These effects are seen to some degree…, but are not convincing and additional measurements are required....".* To explore these ideas further, we computed transfer-functions of recorded motions of six earthquakes and scaled them by the peak ground velocity at the free-field site, $v_{\max,\ \mathrm{F.F.}}$ to describe qualitatively the levels of motion. The results are shown in Fig. 8. It is seen that all earthquakes produce an oscillatory transfer-function, but the peaks of these transfer-functions are at different frequencies. In few instances two or three peaks do coincide, but the evidence appears weak, so that we conclude that this may not be the best way to identify the presence of nonlinear response in a soil-foundation-structure system.

Papageorgiou and Lin (1991) analyzed the response of Hollywood Storage Building after the Whittier-Narrows earthquake of 1987. They used *modal minimization method* to find the apparent system frequencies. In translation, in NS direction, they identified frequencies at 0.53, 2.00 and 3.33 Hz, and in the EW direction at 1.56 and 5.26 Hz. In torsion, they identified 1.12 and 6.25 Hz. They concluded that there is "*clear evidence of SSI in the longitudinal (EW) direction and weak SSI in the transverse (NS) direction*".

4 RESEARCH ON FULL-SCALE EXPERIMENTS AND SOIL-STRUCTURE INTERACTION AFTER 1970

Journal and conference papers explicitly dealing with analysis of soil-structure interaction, in full-scale, and in terms of recorded earthquake response are rare. Examples include studies of a three story long building founded on soft soil (Muria-Vila and Alcorta, 1992), a study of free-field motions surrounding a building, and of the motions of the building (Kashima and Kitagawa, 1988), creation of a database of earthquake records for response of a concrete tower (Ganev et al., 1993), and earthquake response analysis of a caisson–type foundation of Sasame bridge (Kaino and Kikuchi, 1988). Analysis of earthquake records and identification of soil-structure interaction from recorded accelerograms in buildings is discussed in Safak (1992).

Full-scale tests of soil-structure interaction using periodic force excitation of structures are more common. The examples include tests of bridges (Crouse et al., 1987; Maragakis et al., 1996), buildings with prefabricated panels (Petrovski, 1978; Erdik and Gulkan, 1984), a steel frame building (Shinozaki et al., 1994), tall concrete silo tower (Ellis, 1986), nuclear reactor buildings (Erdik et al, 1985; Mizuno and Tsushima, 1975; Casirati et al., 1988; and Iguchi et al., 1988), and foundations supported by piles (Urao et al., 1988; and Yahata et al., 1992). Wave motion resulting from soil-structure interaction during forced vibration tests of a nine-story reinforced concrete building (Luco et al., 1986) is described in Luco et al. (1975).

Nonlinear response of soil may course significant changes in the apparent frequencies of the building-soil system (Trifunac et al., 2001a,b), and this may lead to different results for small and large amplitudes of response (Luco et al., 1986). These differences can be quantified and interpreted by comparison of experimental results for small and large amplitudes of response (Trifunac, 1972; Fukuoka, 1977; Ueshima, 1988; and Tobita et al., 2000).

5 EXAMPLES OF FULL-SCALE TESTS IN ACTUAL STRUCTURES

In the following we review several examples of full-scale tests in actual structures to illustrate how those tests can contribute towards formulation of realistic models and model assumptions (Millikan Library in Pasadena, Hollywood Storage Building, and a seven story reinforced concrete building in Van Nuys, California, severely damaged by the 1994 Northridge earthquake).

5.1 Rigid Versus Flexible Foundation Models

When soil-structure interaction is considered in the dynamic analysis of soil-structure systems, it is convenient to assume that the foundation is rigid. This assumption simplifies the analysis and reduces the number of additional degrees-of-freedom required to model soil-structure interaction, and thereby the number of simultaneous equations to be solved. Whether such assumption can be made must be carefully investigated, and the outcome does not depend only on the relative rigidity of the foundation and of the soil, but can be influenced also by the overall rigidity and type of the structure, its lateral load resisting system and its orientation. This can be illustrated by comparison of the NS and EW vibrations of Millikan Library in Pasadena, a nine-story reinforced concrete structure, studied by Luco et al. (1986). Even though the foundation system of the building is relatively flexible, for NS vibrations, two symmetric shear walls at each end (east and west) of the building act to stiffen the foundation slab, and this allows one to proceed with a rigid foundation representation (Fig. 9a and c). For EW vibrations, the building carries lateral loads by an elevator core, which deforms the foundation slab in the middle, while the shear walls act as membranes providing axial constraints, but little bending stiffness (Fig. 9b and d). For EW vibrations, the foundation slab cannot be approximated by a rigid foundation model. These three-dimensional deformation shapes, which showed how this structure deforms while vibrating in NS and EW directions, were measured during forced vibration tests (Foutch et al., 1975) and were essential for this interpretation. Fig. 10 shows schematically the relative contribution of horizontal deformation of the soil (4 percent), roof displacement resulting from rigid body rocking (25 percent) and relative deformation of the building (71 percent), during steady state forced-vibrations in the NS direction (as in Fig. 9a).

Recent ambient vibration tests in a seven-story reinforced concrete moment resistant frame building in Van Nuys, California showed that the foundation supported by piles deforms during passage of microtremor waves. It can be inferred that the same happens during passage of strong motion waves that have much larger amplitude. A detailed ambient vibration survey of this symmetric structure on symmetric pile foundations showed that the center of torsion for this structure is outside the building plan, close to its south-east corner (Trifunac et al., 1999). Subsequent reexamination of the strong motion records in this building has shown that this eccentricity may have been present in all post 1971 excitations, and that it is associated with some asymmetry in the soil-pile system since the date of its construction, in 1966, or that it was caused by some partial damage during the 1971 San Fernando earthquake (Trifunac et al., 1999).

Differential motions of building foundations (Trifunac 1997) may reduce the translational response at the upper floors, but leads to large additional shear forces and bending moments in the columns of the first floor. The response spectrum method can be modified to include the consequences of such differential motion (Trifunac and Todorovska, 1997), but it is necessary to study this further via full-scale measurements during future strong earthquakes, and to correlate the theory with observations.

The assumption that foundations can be represented by rigid “slabs” seems to be implicit in most full-scale instrumentation programs for buildings where strong motion has been recorded so far. Technically, it should be easy to supplement the existing instrumentation to provide data on differential motion of building foundations. Ideally this should be done first in instrumented buildings where strong motion has already been recorded during many past earthquakes, so that additional value can be added to the existing data, interpretation and analyses.

5.2 Surface Versus Imbedded Foundation Models

Following many ambient, forced vibration, and earthquake recording experiments in Millikan Library (Fig. 9) and apparent inconsistencies in the data and its interpretation, we decided in mid 1970s to develop a

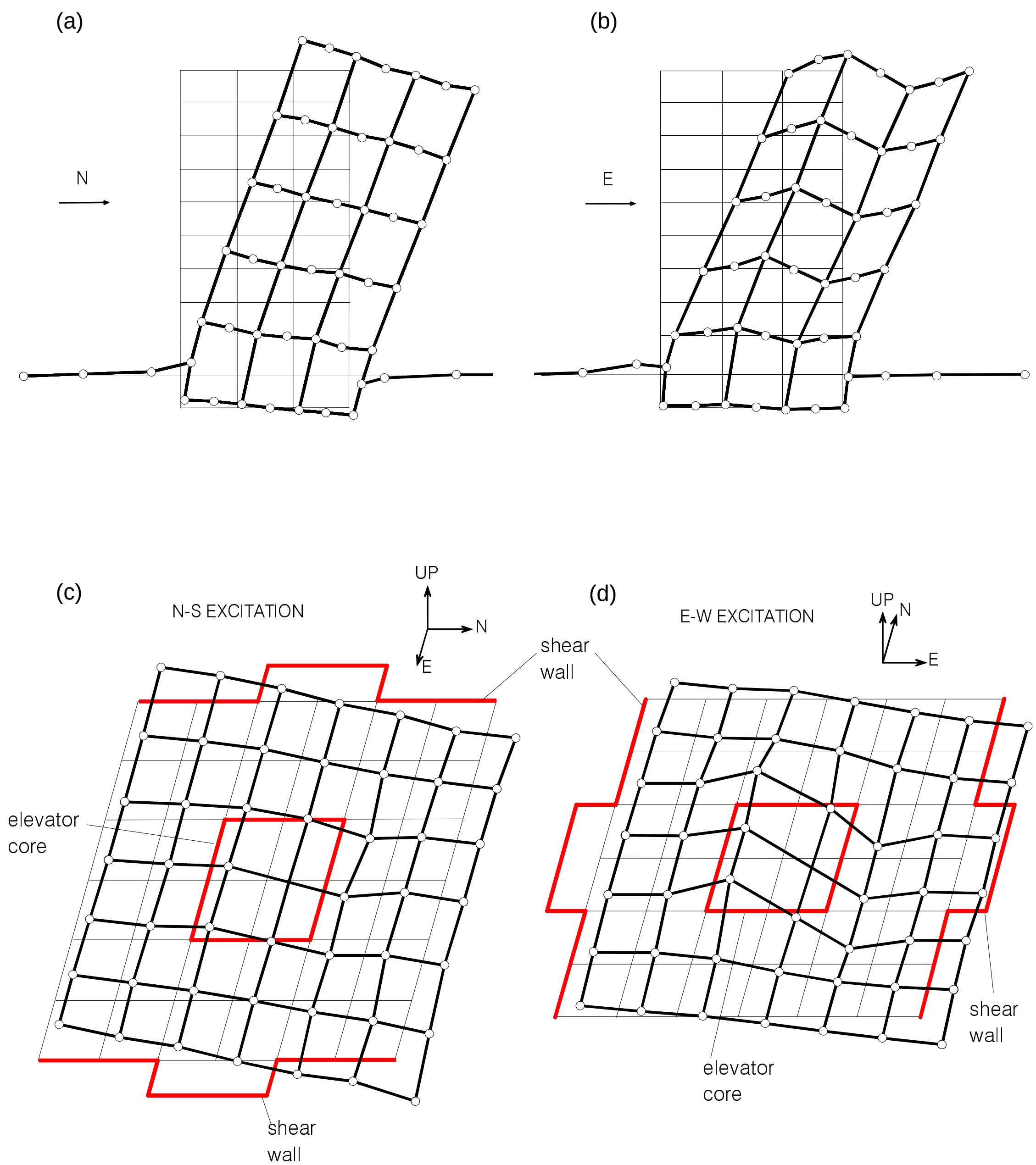

Fig. 9 Deformation of Millikan Library, a nine-story reinforced concrete building, excited at the roof by a shaker with two counter rotating masses, (a) along the west shear wall during NS excitation, (b) along a section through the elevator core during EW excitation, (c) of the basement slab during NS excitation, and (d) of the basement slab during EW excitation.

comprehensive model, which includes soil-structure interaction, so that we could use it in interpretation of all the recorded data. When this model was completed, comparison of the theoretical predictions with the recorded motions showed that the theory for computation of compliances (available at that time, for rigid surface foundations) was not adequate to interpret the results. Our analysis and writing of the report were interrupted, and we started to work on refinement of compliance functions, so that the embedment could be considered explicitly. After new compliance equations were developed and tested, the original full-scale tests of the building could be explained, now resulting in excellent agreement between the theory and the measurements. Finally, it was possible to finish the report, almost ten years after it was started (Luco et al., 1986). Not every iteration of an experimental verification will take ten years to complete. With more focus and effort, our work could have been completed earlier, but it should be understood that complicated subjects take more time to understand and to master.

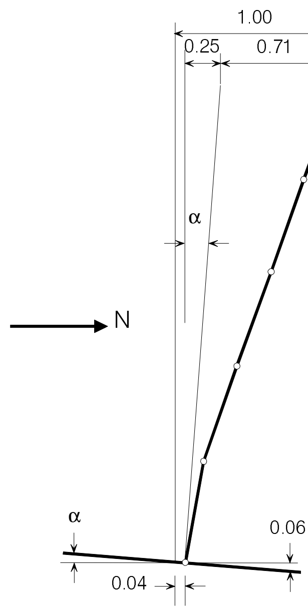

Fig. 10 Contributions of foundation translation and rocking to the roof motion of Millikan Library, for N-S shaking.

5.3 Experimental Estimates of Impedance Functions

The horizontal force H_s and moment M_s that the foundation exerts on the soil can be computed by considering linear and angular moments of the superstructure and of the foundation. To complete the formulation, it is necessary to invoke the relations between H_s and M_s and horizontal displacement Δ and rocking angle φ of the foundation (see Fig. 1), resulting from the flexibility of the soil. Then it is possible to write

$$H_s = GL\,(K_{HH}\,\Delta + K_{HM}\,L\varphi) \tag{2a}$$

$$M_s = GL^2\,(K_{MH}\,\Delta + K_{MM}\,L\varphi) \tag{2b}$$

where K_{HH}, $K_{HM} = K_{MH}$ and K_{MM} represent the normalized, complex, frequency-dependent impedance functions for the foundation assumed rigid, G is the reference shear modulus of the soil, and L is a characteristic length, which depends on the shape of the foundation.

Numerous analytical and numerical procedures have been developed for computation of frequency dependent impendence functions. These procedures require simplified representation of the soil medium, usually in terms of parallel homogeneous layers and of infinite extent horizontally. Also, equivalent dynamic soil moduli must be specified on the basis of standard field and laboratory geotechnical tests. Carefully and well-designed full-scale experiments on structures are therefore invaluable to verify those methods and to evaluate the adequacy of the theoretical approximations and selection of the governing parameters.

Luco et al. (1986) and Wong et al. (1988), for example, have described forced vibration tests of Millikan Library in which the response at the top of the structure and the translational and rocking response at the base can be used to calculate the force and the moment the foundation exerts on the soil. When the coupling impedances K_{MH} and K_{HM} are small, the K_{HH} and K_{MM} can be approximated from experimental measurements. We found excellent agreement between theoretical and experimental estimates of rocking impedance functions, and not so good agreement for the corresponding horizontal impedance functions, particularly for the EW response. We concluded *"These discrepancies in the E-W direction are associated with the failure of the simple foundation model to account for the flexibility of the actual foundation and for the large radiation damping in horizontal vibrations obtained experimentally. It seems then, that if the foundation acts as a rigid body it is possible to predict quite accurately the effects of soil-structure interaction during forced vibration tests by use of simple models. Analytical models more complex than those used in this study may be required for highly flexible foundations"* (Wong et al., 1988).

5.4 Torsion

Torsional response in non-symmetric structures is caused by geometrical separation of the centers of mass and of rigidity. For symmetric structures, torsional response may occur because of nonsymmetric foundation system or may be excited by the wave passage effects (Luco, 1976; Trifunac et al., 1999) or both. Long and narrow symmetric buildings, for example, can experience significant torsional response and whipping (Todorovska and Trifunac1989, 1990b), when excited by earthquake waves propagating along the longitudinal axis of the structure-soil system.

Full-scale measurements of torsional response and of torsional components of soil-structure interaction cannot be performed directly, because no rotational strong motion accelerographs, at present, are installed in buildings in California. It is possible only to estimate average rotations, when multiple recorders in the structures are arranged so that relative motions can be computed from the differences in translational motions. In the following, we illustrate this for Hollywood Storage Building.

The locations and orientations of the strong motion accelerographs since 1976 are shown in Fig. 3b. For this instrument configuration, processed strong motion data are available only from four earthquakes: 1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear, and 1994 Northridge. By suitable combination of displacements y_i, computed from the recorded accelerograms by double integration, it is possible to estimate the average torsion in the west side of the building. The expressions

$$\phi_b(t) = (y_9(t) - y_1(t))/3060 \tag{3}$$

and

$$\phi_r(t) = (y_{12}(t) - y_{10}(t))/3060 \tag{4}$$

where 3060 cm is the separation distance between the recorders at each level, give respectively the average torsion of the foundation and of the western half of the building at the roof. The expression $y_{10}(t) - y_1(t)$ describes the relative NS vibrations at the center of the building, and $y_{12}(t) - y_1(t)$ describes the NS motion of the roof (in the western end of the building) relative to the central station at ground level. Then $y_{12}(t) - y_{10}(t) - y_9(t) + y_1(t)$ gives the contribution to the motion of the western end of the building, at roof level, associated with torsion of the building, relative to its base. Figures 11a and b illustrate these displacements versus time for motions during the Landers and Northridge earthquakes. It is seen that

$$y_{12}(t) - y_{10}(t) - y_9(t) + y_1(t) \sim y_{12}(t) - y_1(t) \tag{5}$$

and

$$y_{12}(t) - y_1(t) \sim (y_{10}(t) - y_1(t)) / 2 \tag{6}$$

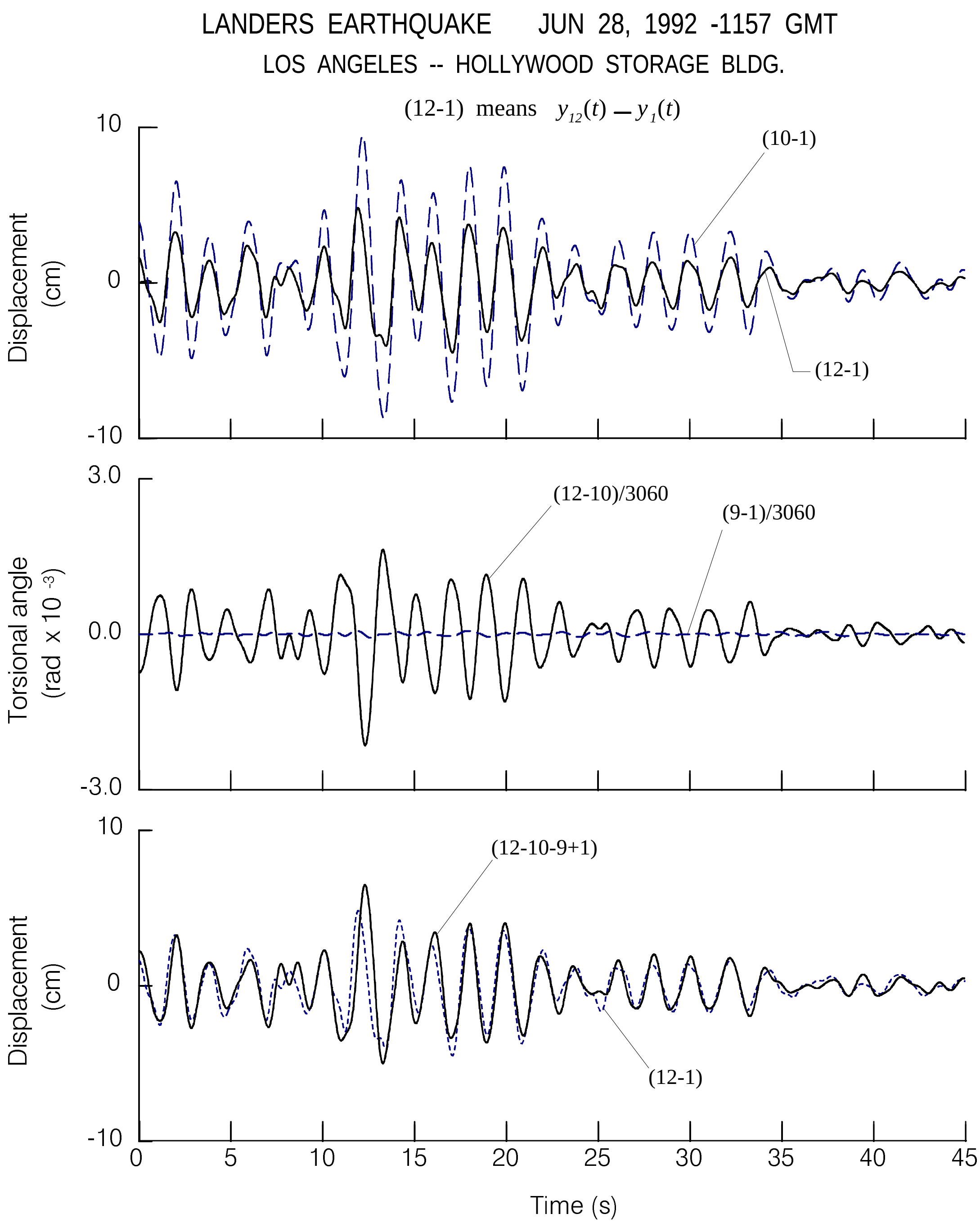

Fig. 11(a) Response of Hollywood Storage Building during 1992 Landers earthquake: Top: Comparison of relative (with respect to basement-center) displacements recorded at the roof west end ("12–1", solid line) and at the roof center ("10–1" light dashed line). Center: Comparison of average torsion of the western half of the building ("(12–10)/3060", solid line) and at ground level ("(9–1)/3060", light dashed line). Bottom: Comparison of relative (with respect to basement-center) displacements at the roof west end due to torsion alone ("12–10–9+1", solid line) and due to torsion and translation ("12–1", dashed line).

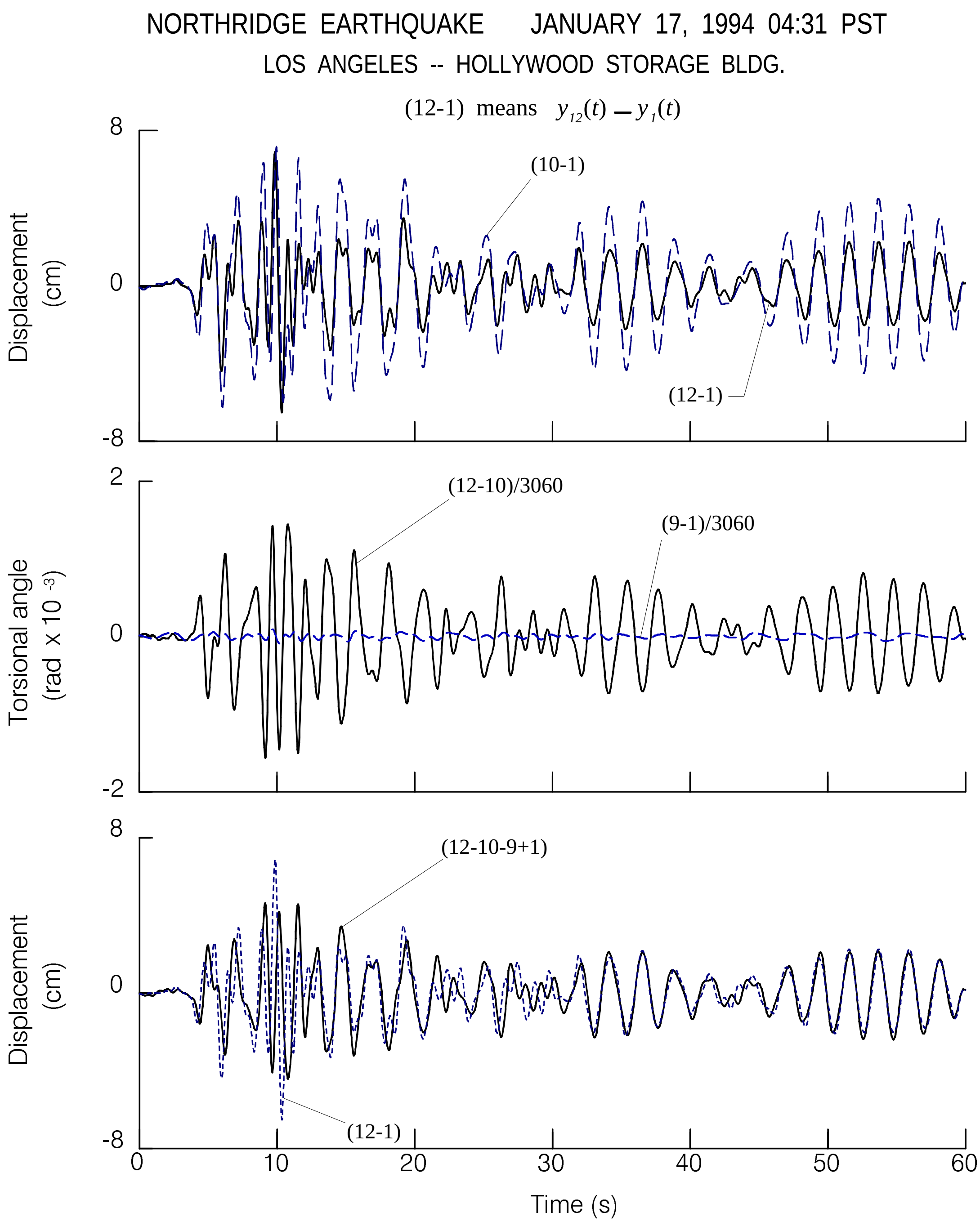

Fig. 11(b) Response of Hollywood Storage Building during 1994 Northridge earthquake: Top: Comparison of relative (with respect to basement-center) displacements recorded at the roof west end ("12–1", solid line) and at the roof center ("10–1" light dashed line). Center: Comparison of average torsion of the western half of the building ("(12–10)/3060", solid line) and at ground level ("(9–1)/3060", light dashed line). Bottom: Comparison of relative (with respect to basement-center) displacements at the roof west end due to torsion alone ("12–10–9+1", solid line) and due to torsion and translation ("12–1", dashed line).

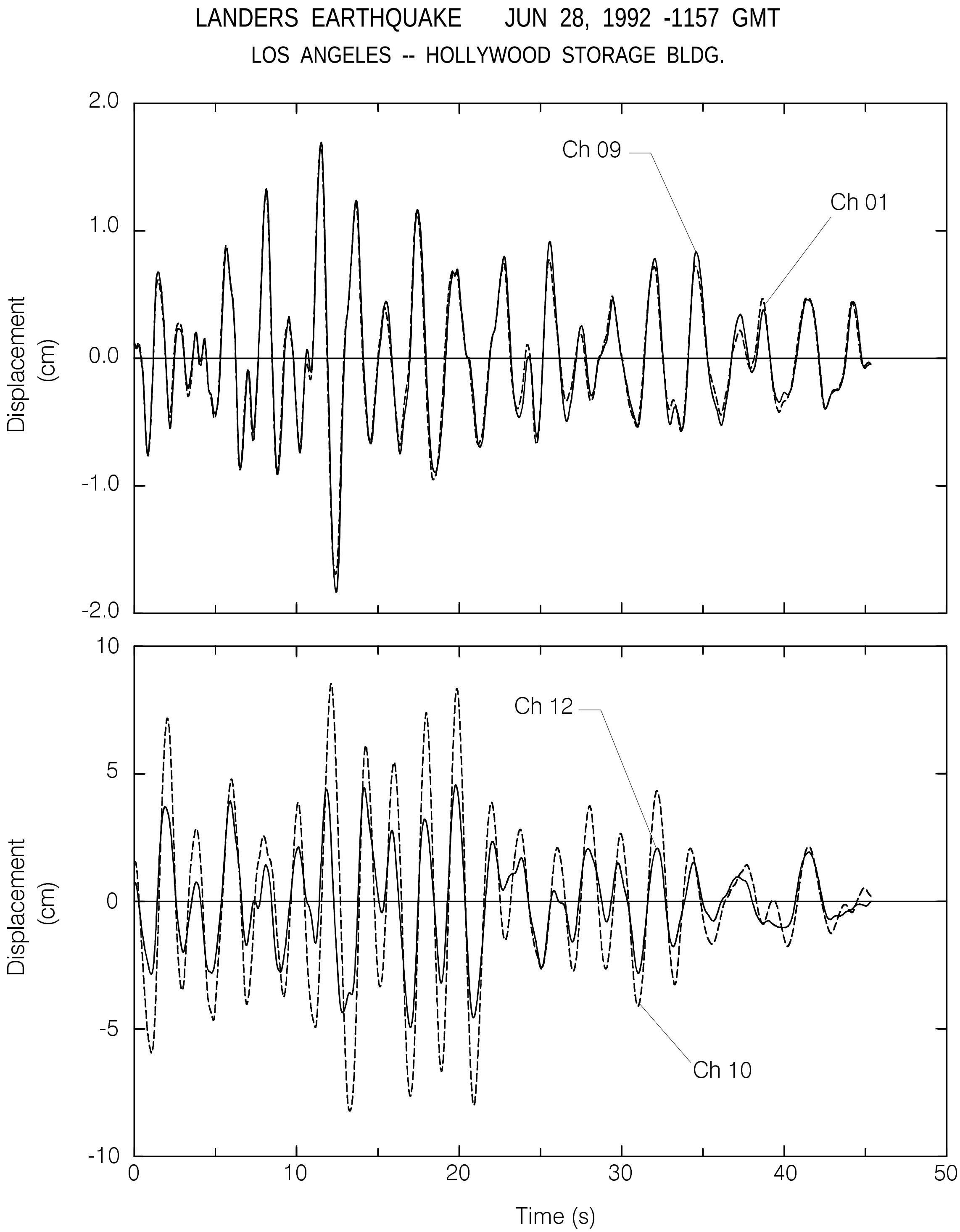

Fig. 12(a) Response of Hollywood Storage Building during 1992 Landers earthquake: Comparison of simultaneous displacements recorded by Channels 9 and 1 (at ground level) and Channels 12 and 10 (at roof).

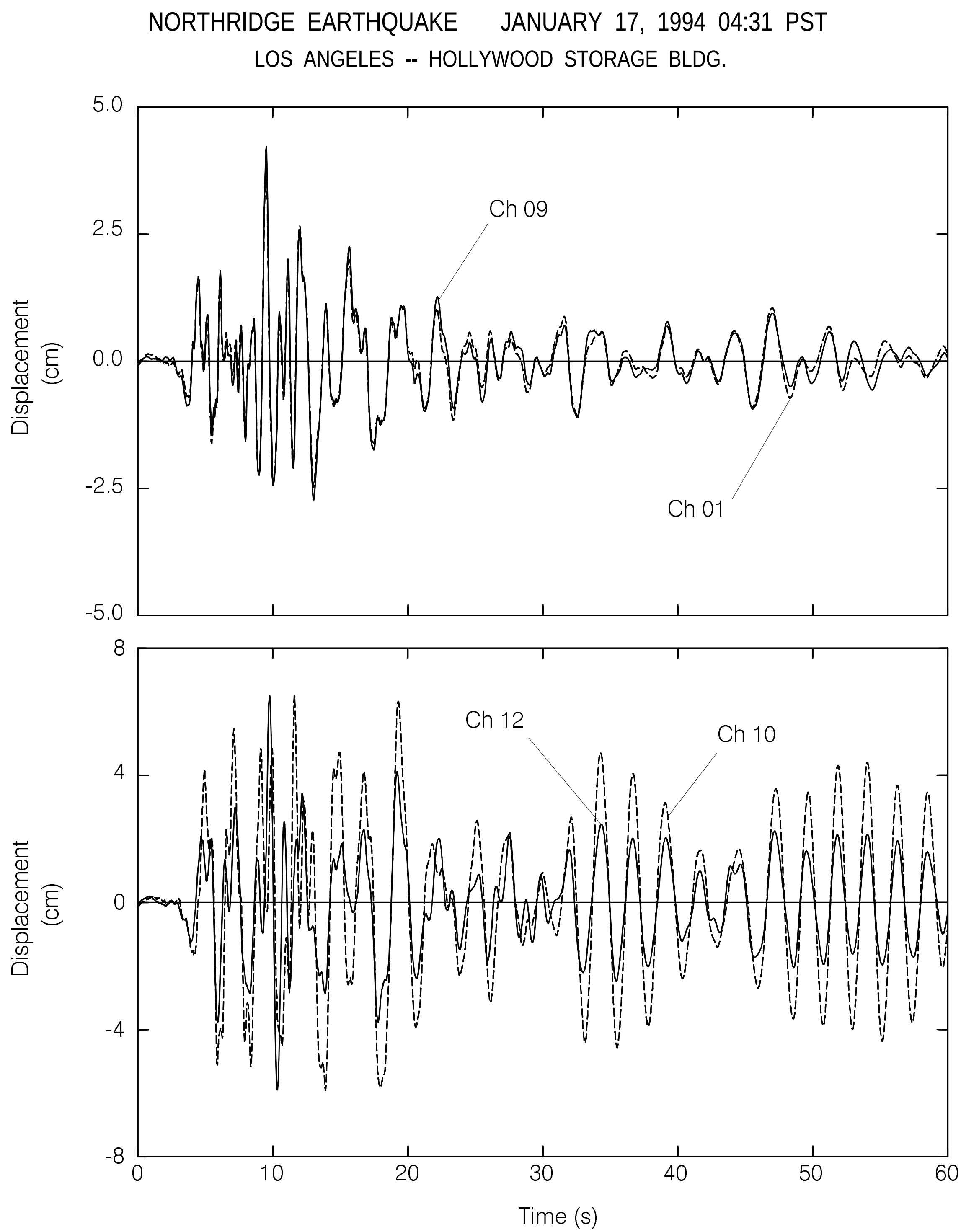

Fig. 12(b) Response of Hollywood Storage Building during 1994 Northridge earthquake: Comparison of simultaneous displacements recorded by Channels 9 and 1 (at ground level) and Channels 12 and 10 (at roof).

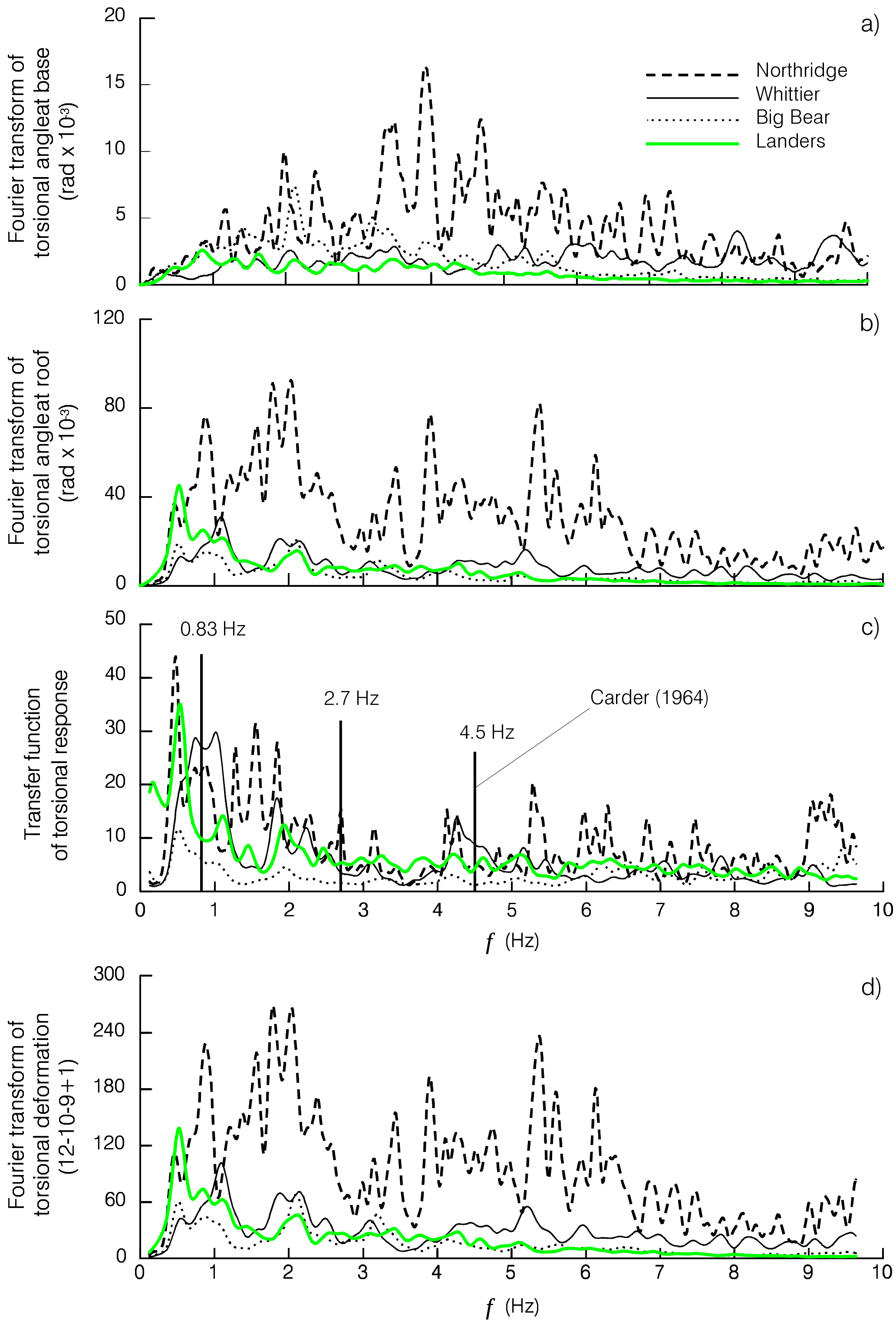

Fig. 13 Hollywood Storage Building: Comparison of (a) Fourier transform of the torsional angle at the base, (b) Fourier transform of the torsional angle at the roof, (c) transfer function between the torsional response at the roof and at the base, and (d) Fourier transform of torsional deformation, during four earthquakes (1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear and 1994 Northridge).

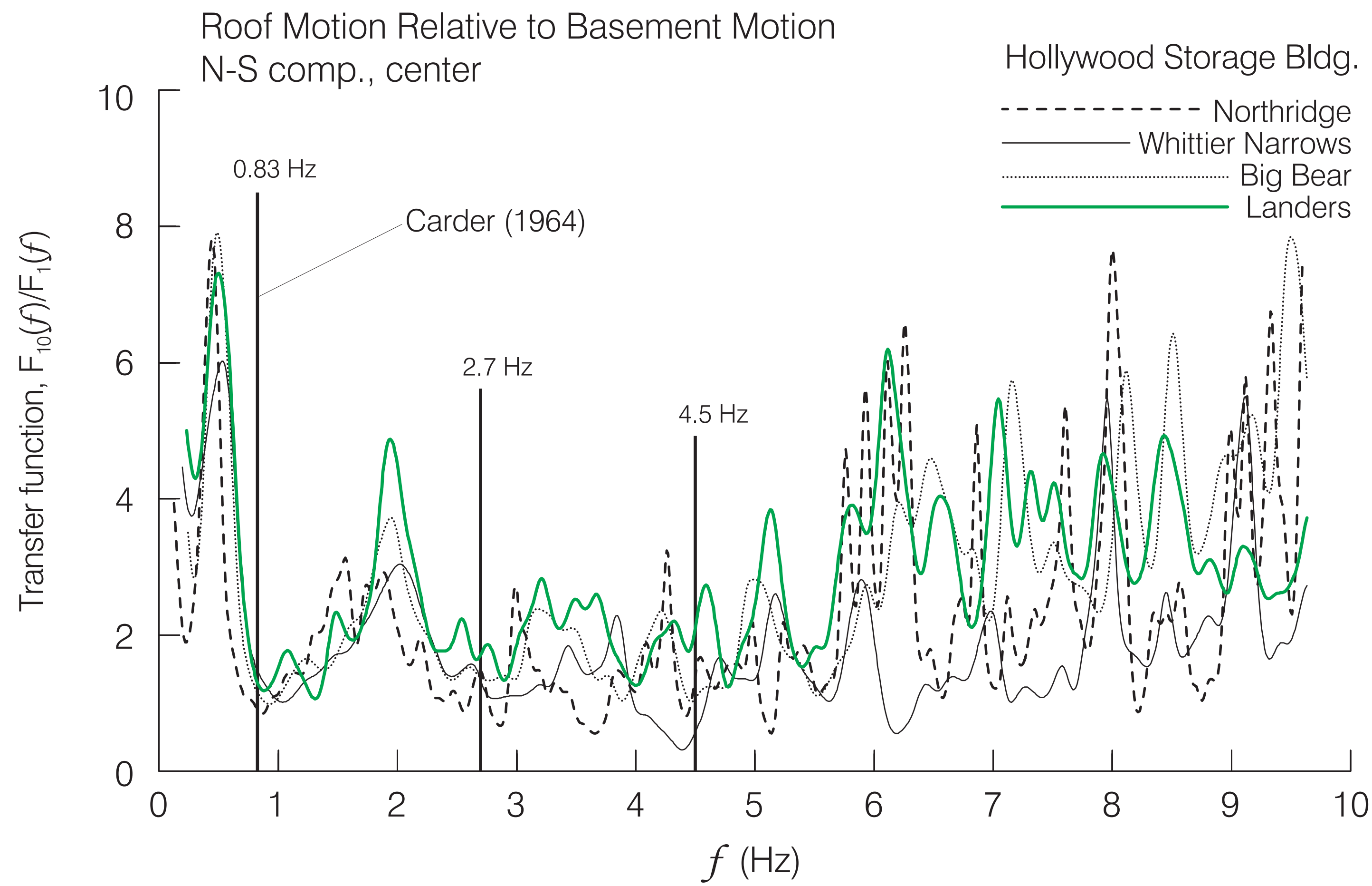

Fig. 14 Hollywood Storage Building: Comparison of amplitudes of transfer function between "translational" response at the roof center and at the base center, during four earthquakes (1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear and 1994 Northridge).

It is also seen that (most of the time) the relative motions at the recording site 12 are about one half of the motions at the site of 10, and that the two motions are in phase. An exception to this (see Fig. 11b) occurred during the Northridge earthquake, 8 to 12 s after trigger time. Thus, most of the time, the building is twisting about a point west of the center of symmetry of the base. We reported on a similar behavior for a seven-story reinforced concrete building also supported by pile foundation (Trifunac et al., 1999). In the present case, such behavior may be caused in part by non-symmetry of the foundation (the building has basement only beneath its western half, see Fig. 3). Such torsional eccentricity thus causes whipping of the eastern and of the building, particularly for EW arrivals of SH and Love waves (e.g. Whittier-Narrows 1987; Landers, 1992; and Big Bear 1992 earthquakes, see also Fig. 17 in Todorovska and Trifunac, 1989). Unfortunately, there are no strong motion instruments along the eastern end of Hollywood Storage Building to verify this interpretation.
Figures 12a and b compare simultaneous motion at stations 9 and 1 (at ground level) and at 12 and 10 (at roof level). Bottom parts of Fig. 12a and b show motions at stations 12 and 10. As it could be seen in Fig. 11, the displacements at station 10 are about two times larger than those at station 12, mainly during the later phases of the excitation.

Figure 13 compares Fourier amplitude spectra of the time functions shown in Fig. 11 (spectra for two other earthquakes, Whittier-Narrows and Big Bear are also included). The transfer-function in Fig. 13 shows three frequencies where all recorded motions contain some periodic content: ~ 0.5, 1.8–1.9, and 4.2 Hz. These frequencies appear to be related to 0.83, 2.7 and 4.5 Hz, the three translation frequencies observed by Carder (1964), but are smaller because of reduced soil stiffness during strong motion.

Fig. 14 shows the transfer-function of displacements between stations 10 and 1. For symmetric soil-structure system, with no contribution of torsional response, this would represent the NS translational transfer-function. As in Fig. 11, the same three frequencies are excited by most earthquakes. We conclude that, for the Hollywood Storage Building, torsional and NS translational motions are coupled and both contribute significantly to NS motions.

5.5 Time and Amplitude Dependent Response

Trifunac et al. (2001a,b) reported on systematic and significant amplitude dependent changes of soil-structure system frequency $\tilde{f} = 1/\tilde{T}$ of a seven-story reinforced concrete frame building supported by piles, in Van Nuys, California, damaged by the 1994 Northridge earthquake. They used the conceptual model in Fig. 15 to explain the observed changes. This model consists of a building with height H and an embedded foundation. In the initial stages of the response when the amplitudes are small and the soil stiffness is "linear" as the building begins to "push" the soil sideways, its effective depth of "fixity" (indicated by d_{eq} in Fig. 15a) changes as a function of the response amplitudes and history. Larger d_{eq} leads to smaller stiffness $K \sim 1/(H + d_{\mathrm{eq}})^2$ and smaller $\tilde{f}$. While the building pushes the soil, d_{eq} decreases and K increases ("hardening" behavior). When the direction of motion reverses and the building moves away from the soil, a gap forms between the two, d_{eq}, increases and K decreases ("softening" behavior). This results in nonlinear system behavior, which can be modeled by a nonlinear spring or by a group of springs with gap elements. As the amplitudes of motion increase further, the soil begins to yield and material nonlinearity is introduced into the system reducing or canceling the "hardening" part of the cycle. This behavior repeats as long as the successive amplitudes of vibration increase and the soil can be pushed sideways. At the time of the largest response amplitudes ($\theta_{\max}$), the gap between the building foundation and the soil sidewalls is the largest. By forcing it to yield, the soil can be "compacted" also below the corners of the "rigid" foundation (see Fig. 15b), reducing the equivalent length, L_{eq} of the contact between the foundation and the soil, resulting in reduction of the system stiffness ($K \sim L_{eq}^3$; Luco et al., 1987) and reduction of $\tilde{f}$. Following the largest amplitudes of response, as the strong motion amplitudes begin to decrease, the depth of "fixity" d_{eq} and the contact length L_{eq} remain constant (many equivalent gap elements

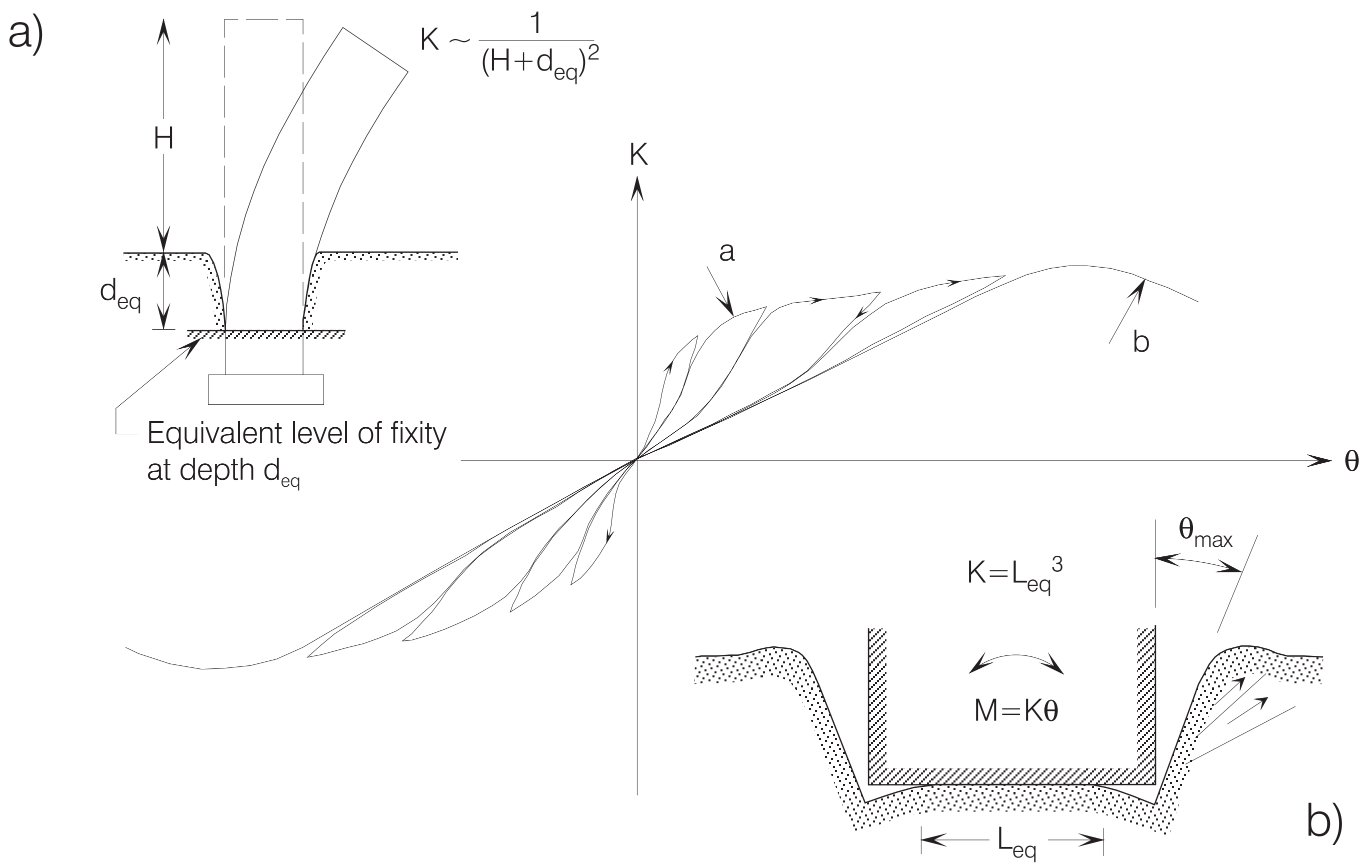

Fig. 15 (a) Nonlinear changes in the rocking stiffness caused by passive soil pressure on the sidewalls of the building and variable equivalent depth of fixity d_{eq}. (b) A schematic representation of "permanent" soil deformation after large rocking response.

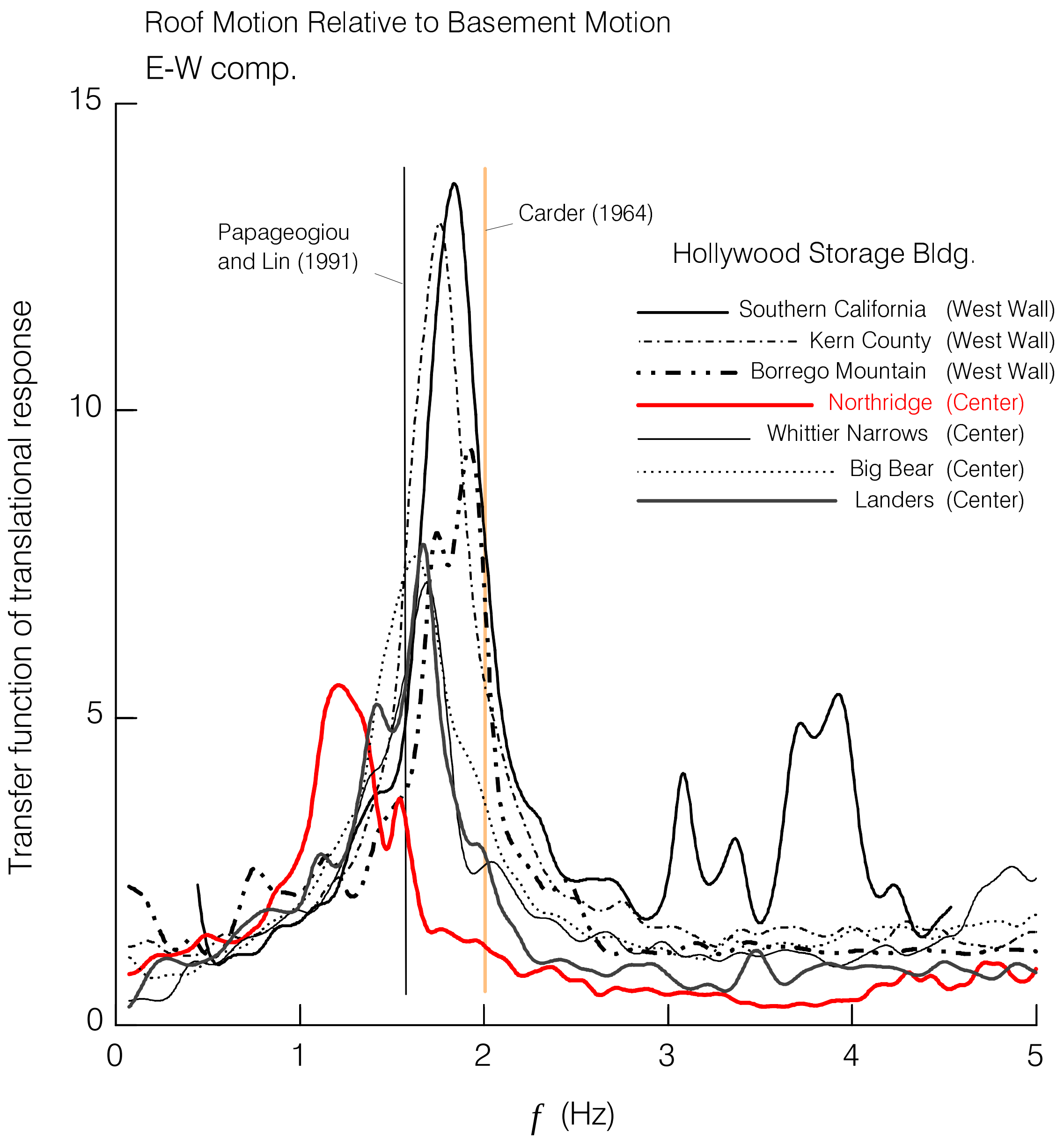

Fig. 16 Hollywood Storage Building: Amplitudes of the transfer functions between EW translation at the roof center and basement center, during seven earthquakes. The vertical lines show the systen frequencies determined from small amplitude full-scale tests in 1938 (Carder, 1964), and from modal minimization method using recorded response to earthquake excitation (Papageorgiou and Lin, 1991).

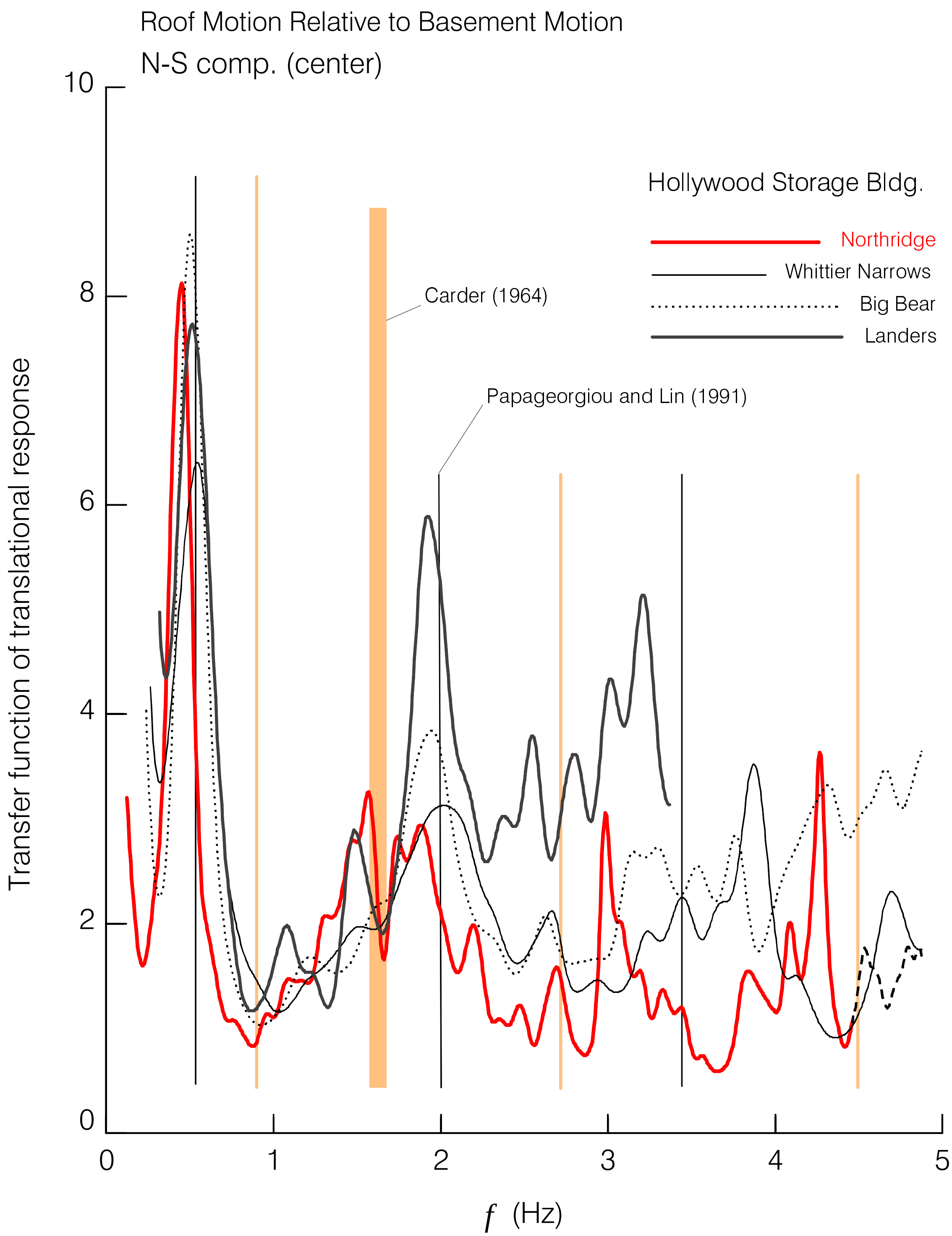

Fig. 17 Same as Fig. 16, but for NS motions recorded at the center of the building.

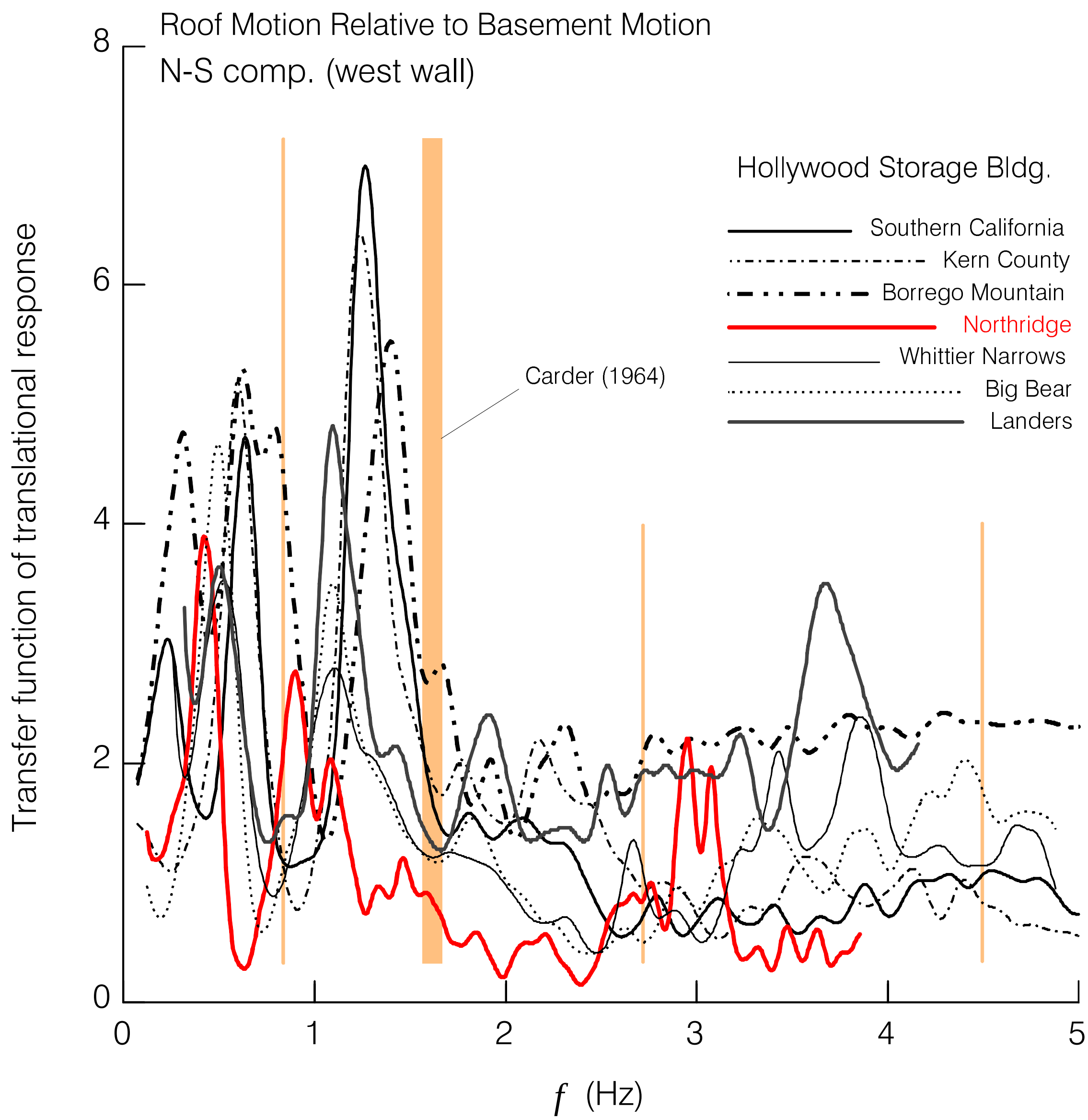

Fig. 18 Same as Fig. 16, but for NS motions recorded along the western end of the building.

remain open). The building responds with smaller amplitudes, and the period of response is longer. Continued shaking, aftershocks and "subsequent" earthquakes may activate a "healing process". Through dynamic compaction and settlement of the soil material, which was loosened and pushed aside by the preceding strong motion, the soil is packed back around the piles, grade beams, and sides of the building, rebuilding or even increasing the previous system stiffness. It seems that this cycle may be repeated many times, depending on the sequence of aftershocks and earthquakes during the "quiet" intervals between strong motion events.

In the following, we analyze the data recorded in Hollywood Storage Building assuming that a similar conceptual model may apply. Figures 16, 17 and 18 show transfer-functions of EW and NS (roof/basement) responses, recorded near the west end or the building (see Fig. 3). The three figures show only frequencies up to 5 Hz. Analysis of the recorded motions shows that with increasing amplitude of the incident waves, the system becomes softer. For EW motions, the system frequency is near 1.9 Hz (close to 2.0 Hz as

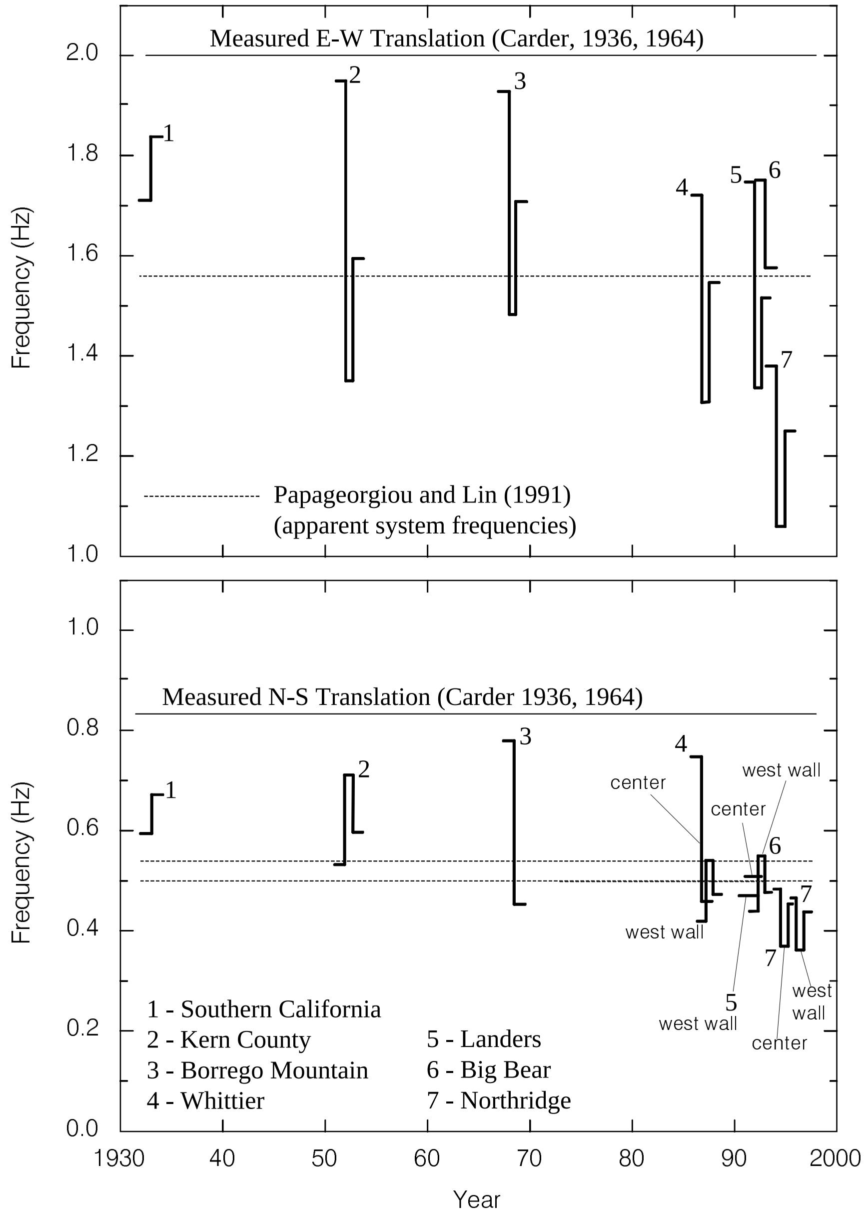

Fig. 19 Hollywood Storage Building: Summary of the time dependent changes of the system frequency during seven earthquakes, between 1933 and 1994. The horizontal lines show the system frequencies determined from ambient vibration and forced vibration tests (EW and NS translations, light solid lines, Carder 1936, 1964), and those identified by Papageorgiou and Lin (1991) (dashed lines). For each earthquake, the horizontal ticks represent pre and post earthquake estimates of the system frequencies.

reported by Carder, 1936; 1964), for excitation during Borrego Mountain Earthquake of 1968. As Fig. 16 shows, the system frequency decreases for larger amplitudes of motion, and for excitation during Northridge earthquake it is near 1.25 Hz. For NS translational response, the system frequency is near 0.6 to 0.7 Hz for small ground motions (Southern California earthquake of 1933) and falls to ~ 0.45 Hz during the largest recorded motions (Northridge earthquake of 1994). During ambient and forced vibration tests this frequency was 0.83 Hz (Carder, 1964). The fundamental frequency in torsion was reported by Carder (1964) to be in the range 1.57–1.67 Hz. Figure 18 shows it was as low as 1.1 Hz during excitation by the 1992 Landers earthquake (waves arriving from the east).

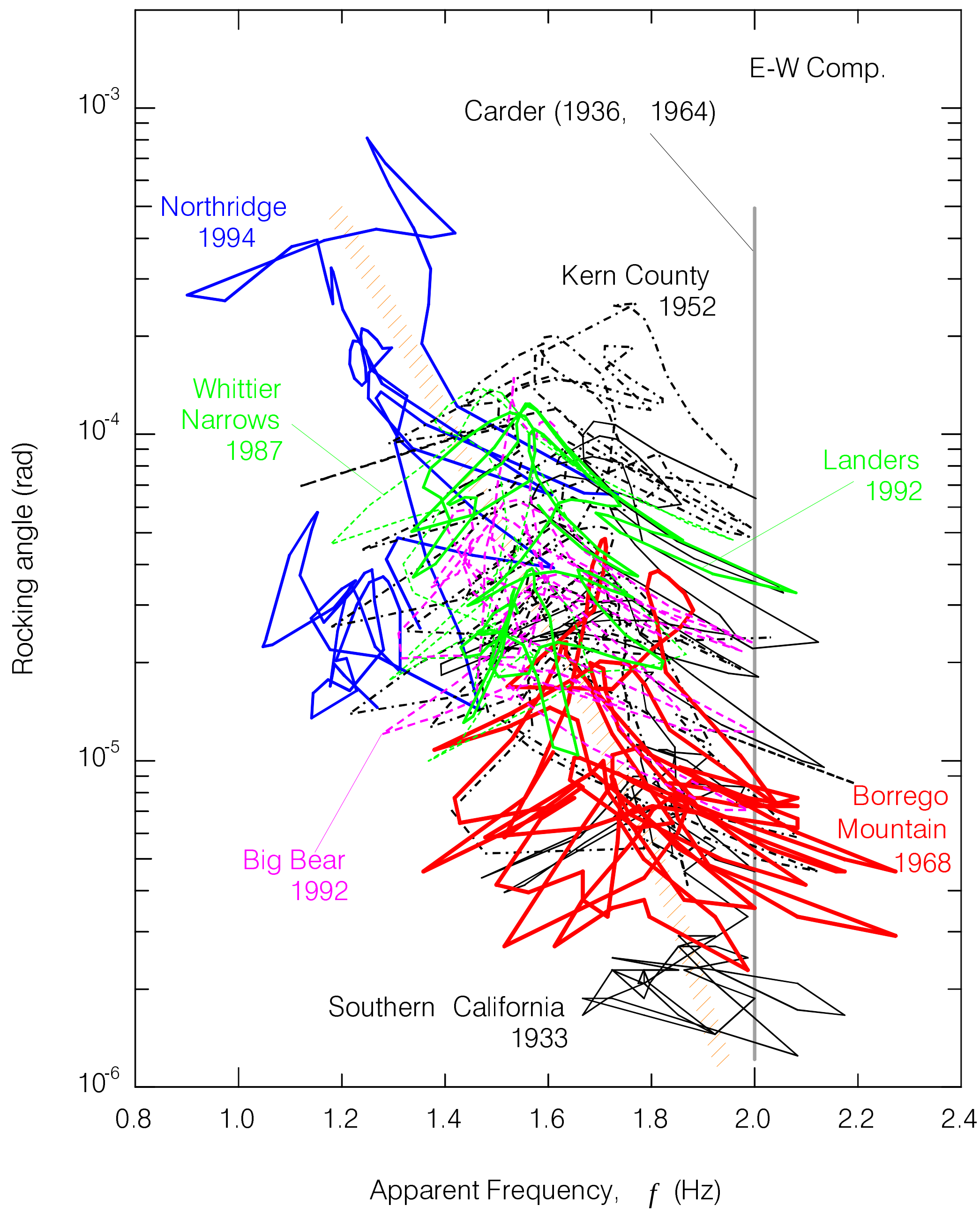

Fig. 20(a) Hollywood Storage Building: Dependence of the apparent system frequency on the amplitude of EW response ("rocking angle"). The solid vertical lines show estimates of the system frequencies determined from small amplitude (ambient vibration and forced vibration) tests by Carder (1936, 1964).

The peaks in transfer functions in Figures 16, 17 and 18 are "broad", because the system changes during the excitation. Figure 19 summarizes the above trends in the changing system frequencies. It also shows the measured EW and NS translational frequencies observed by Carder (1936; 1964). The dashed lines show the apparent system frequencies identified by Papageoriou and Lin (1991).

Figures 20a and b compare the "rocking angles" (displacement at roof minus displacement at ground level divided by the building height) versus the instantaneous apparent frequency computed for most half-period

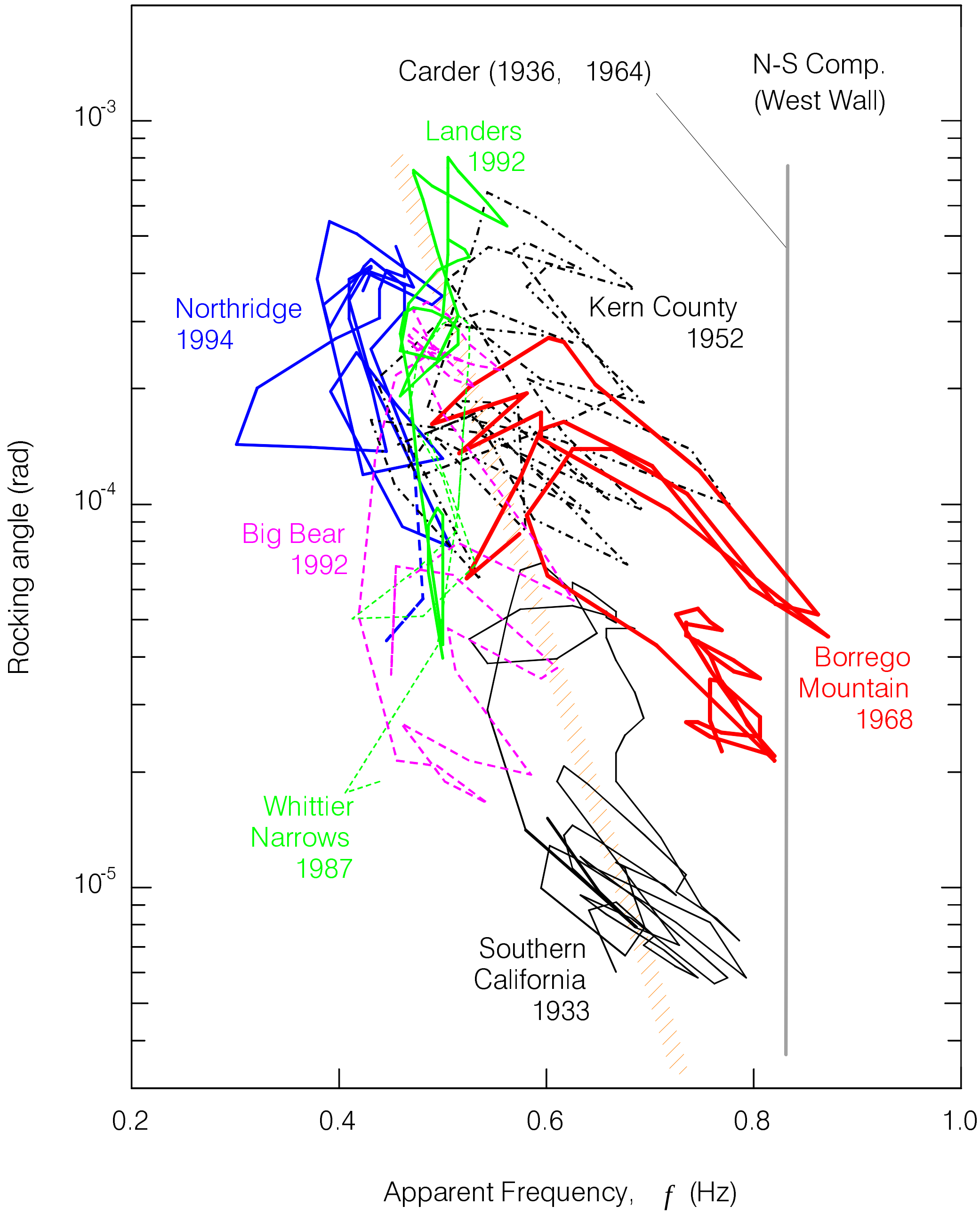

Fig. 20(b) Hollywood Storage Building: Dependence of the apparent system frequency on the amplitude of NS response ("rocking angle"). The solid vertical lines show estimates of the system frequencies determined from small amplitude (ambient vibration and forced vibration) tests by Carder (1936, 1964).

segments of response of the Hollywood Storage Building during all seven earthquakes. It is seen, again, that the apparent system frequency depends on the amplitude of excitation and for small amplitudes approaches the frequencies measured by Carder (1936; 1964) during full-scale ambient and forced vibration tests. These trends are consistent with the non-linear soil structure model shown in Fig. 15.

6 LARGE MODEL TESTS

To understand soil-structure interaction and to validate different modeling and analysis methods, *in situ* experimental investigations are essential. Because it is difficult to simulate half-space conditions in small specimens on shaking tables, a viable alternative is to construct scaled down models in seismically active areas. This approach offers numerous other advantages: choice of embedment depth, control of backfill soil material, possibility to install detailed instrumentation, ability to control the surroundings of the site, etc. (Tang et al., 1989).

As the full-scale structures, large-scale models can be excited by periodic actuators or shakers (e.g. Petrovski, 1975; Mizuno, 1978; Fujimori et al., 1992; Inukai et al., 1992; Ohtsuka et al., 1992; Tuzuki et al., 1992; and Uchiyama, 1992), earthquakes (Iguchi et al., 1988), or by both (Ohtsuka et al., 1996; Tohma et al., 1985; Toki and Kiyono, 1992). Small amplitude measurements during microtremor excitation can also be compared with results from forced vibration tests, and with response to earthquake excitation (Mizuno, 1980). A summary of large model testing in Japan was presented by Kitada et al. (1999).

A special category of full-scale experiments deals with soil-structure interaction characteristics of foundations for strong motion accelerographs. Because of their small dimensions, studies of these full-scale foundations give results analogous to those from *in situ* tests of scaled models mentioned above (Crouse and Hushmand, 1990; Crouse et al., 1984; Ramirez-Centeno and Ruiz-Sandoval, 1996).

7 GENERAL TRENDS IN RESEARCH ON SOIL-STRUCTURE INTERACTION AND FULL-SCALE TESTING

After a brief and productive period, from 1965 to about 1975, when many informative and useful full-scale experiments were conducted (see Luco et al., 1986, for an example of a detailed study of a forced-vibration test, and Ivanović et al., 2000, for a review of ambient vibration tests), the Earthquake Engineering profession seems to have converged toward small-scale laboratory experiments. In 1996, "Earthquake Spectra" (edited by Abrams 1996) published a theme issue entitled "Experimental Methods". Interestingly, *none* of the nine papers mentioned or referenced full-scale tests of structures.

With the recent development of electronic library services and systematic organization of databases in general fields of Science and Engineering (e.g. Web of Science: http://wos.isiglobalnet.com/), and specifically in Earthquake Engineering (e.g. National Information Service in Earthquake Engineering, NISEE: http://www.eerc.berkeley.edu/eea.html), it is now possible to perform systematic literature searches, quickly and efficiently. These databases are not complete, and in general cover systematically only the period from 1970 to present. Also, the quality of attributes and keywords for each abstract are not complete and uniform. Nevertheless, the wealth of overall information and general coverage surpasses even the best-equipped engineering libraries. In the following, we present some results of searches of the Earthquake Engineering Abstracts database of NISEE to show trends in the research on soil-structure interaction and full-scale testing.

Figure 21 shows a histogram of the number of papers (per year) that contain "soil-structure-interaction" in their titles. Shades are used to indicate the relative distribution of papers among the following subtopics: *piles*, *non-linear*, *experimental*, and *full-scale* (obtained by using additional keywords). It is seen that the number of papers dealing with *general aspects* of soil-structure interaction peaked near 35/year during the late 1970s and at present is only about 10/year. The overall number of papers dealing with soil-structure-interaction oscillates starting with the 1980s, but has increased during the recent ten years, mainly due to increased number of contributions dealing with *piles* and with *nonlinear* response.

Papers dealing with *experimental* aspects of soil-structure-intersection do not exhibit major fluctuations, and since 1970 appear at an average rate of 3.3/year. Manual analysis of these papers shows that only about 1.2 papers/year deal with *full-scale experiments*, about 2/3 of the *experimental* papers being devoted to *laboratory testing*. The only year with five papers on full-scale tests involving soil-structure-interaction

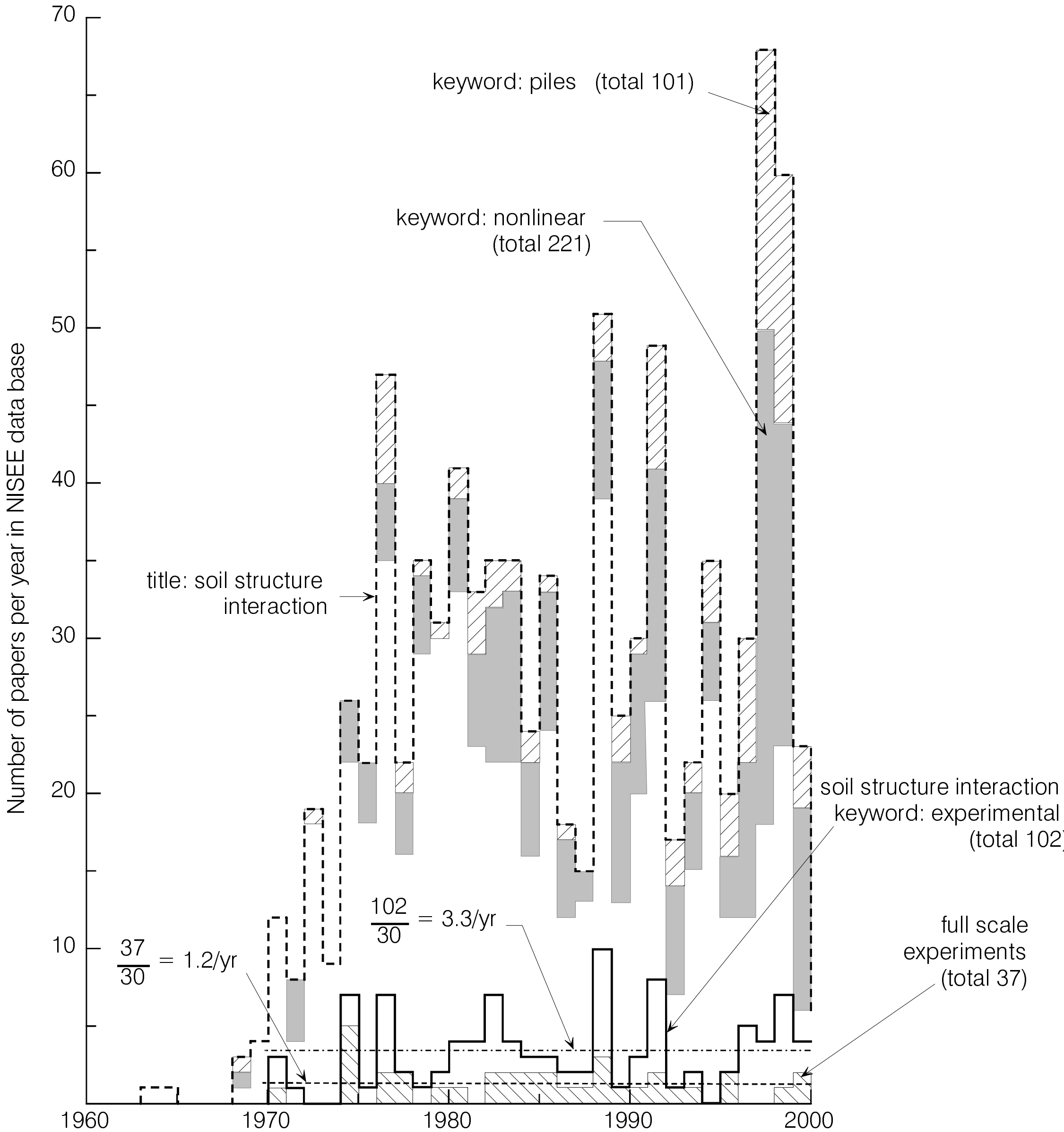

Fig. 21 Histogram of the number of published papers, per year, on the general subject of soil-structure interaction, determined by searches of the NISEE Earthquake Engineering Abstracts database. The light gray and cross hatched zones show the numbers of papers devoted respectively to soil-structure interaction involving "piles" and to "non-linear" response. The solid heavy line shows the number of published papers, per year, with key word "experimental". The average rate of contributions in this category is about 3.3 papers per year. The dark histogram (at bottom) shows the number of papers, per year, dealing with "full-scale" experimental work, involving soil-structure interaction. The average rate of contributions is 1.2 papers per year.

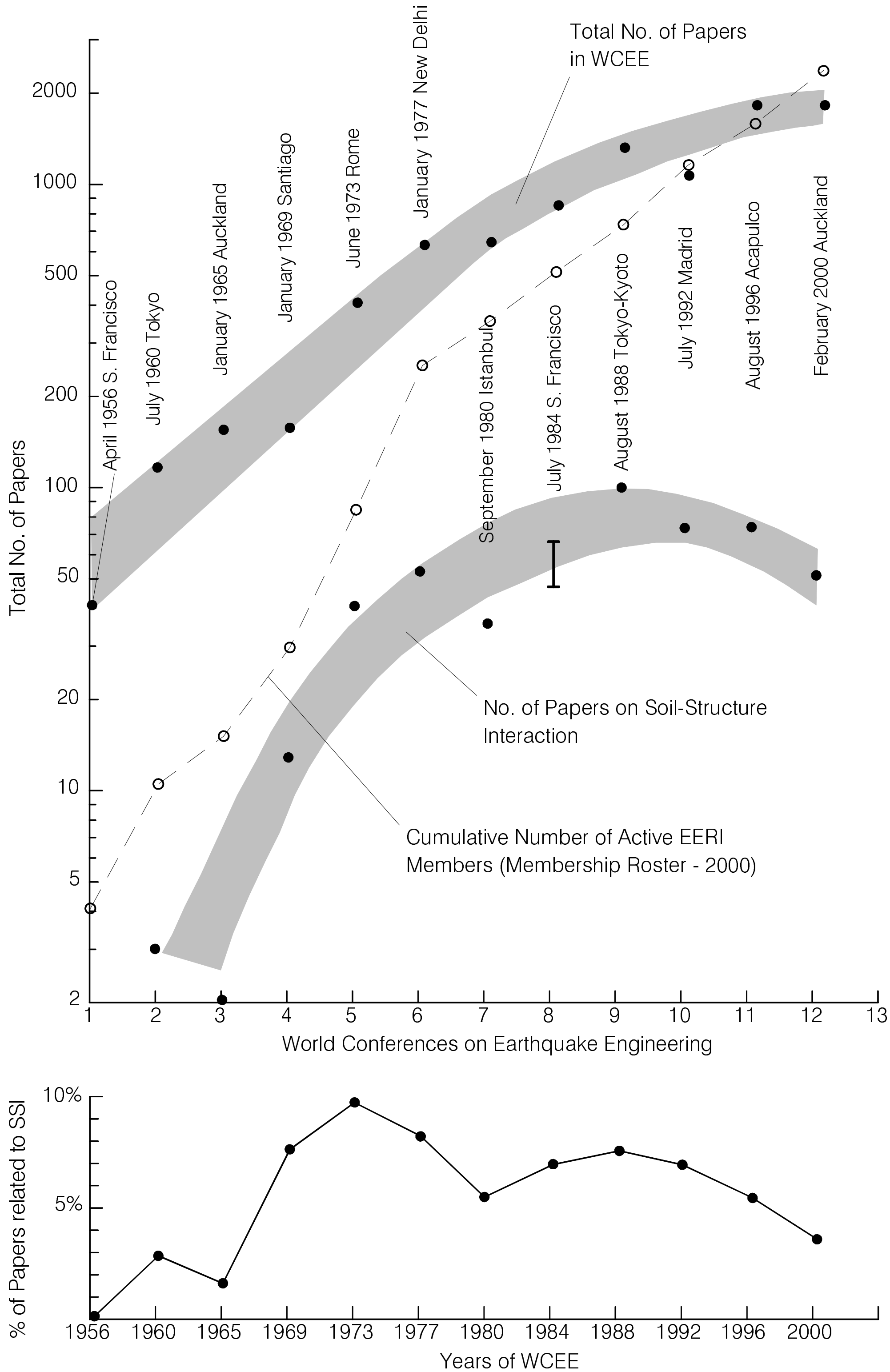

Fig. 22 Top: Total number of papers published in proceedings of World Conferences on Earthquake Engineering, and the number of papers dealing with soil-structure interaction. The cumulative number of active EERI Members (based on membership roster-2000) is shown by open circles. Bottom: Percentage of papers related to the subject of soil-structure interaction.

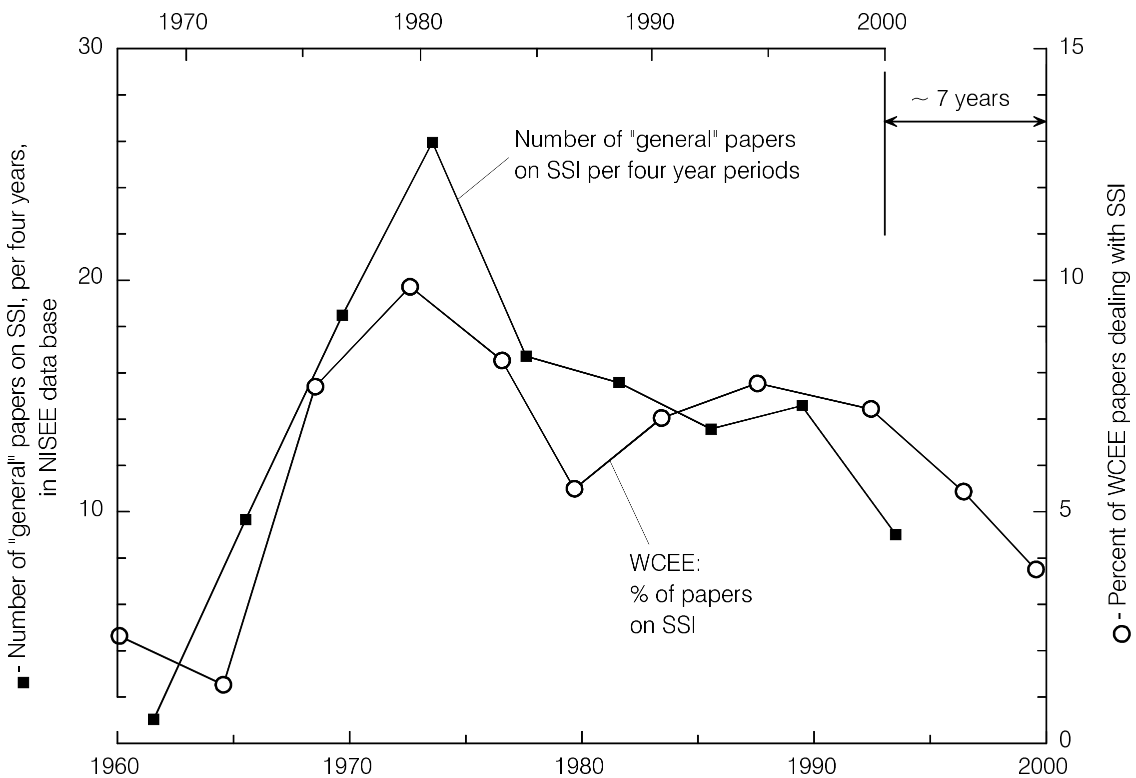

Fig. 23 Percentage of all papers published in proceedings of World Conferences on Earthquake Engineering, which deal with the subject of soil-structure interaction versus time (bottom scale), and the number of papers dealing with the general subject of soil-structure interaction in NISEE data base, per four year periods, versus time (top scale). Note the relative shift of seven years, between the top and bottom time scales.

was 1975. During 1972–1974, 1976, 1979, 1982 1995, 1997 and 1998 (or 45 percent of the time), there were no contributions recorded in this database on this topic. Our search may have missed to identify some papers (misplaced to a wrong category due to incomplete list of keywords), but this finding is nevertheless alarming.

In Fig. 22 (top) we show the trend of the number of papers presented at World Conferences on Earthquake Engineering. During the first conference, in 1956 in San Francisco, 40 papers were presented, and during the last two conferences (Acapulco, 1996 and Auckland, 2000) about 1440 papers (each) were presented. The Proceedings of the 10th World Conference in Madrid in 1992 were the last to be published in printed form, and the proceedings of the most recent two conferences (11th and 12th) are published on a CD-ROM. This figure also shows the number of papers related to the subject of soil-structure interaction. Figure 22 (bottom) shows the percentage of papers dealing with soil-structure interaction, relative to the total number of papers in each conference. It is seen that the largest number of papers were presented in 1988, during the 9th conference, held in Tokyo and Kyoto. The largest percentage of papers devoted to soil-structure interaction was presented in 1973 during the 5th conference. It is seen that the general interest in soil-structure-interaction, at least among researchers and practitioners who publish in World Conference Proceedings, is decreasing.

A more detailed analysis of the above trends for the period 1988 to 1997 in Japan has been presented by Iguchi and Yasui (1999). They find that the number of papers dealing with piles has increased significantly after the 1995 (following Hyogo-Ken Nanbu earthquake), in agreement with the trends shown in Fig. 21.

It is interesting to note that there is a strong correlation between the percentage of papers in WCEE proceedings and the number of papers published in journals, other conferences, and reports. This is illustrated in Fig. 23, where the percentage of papers devoted to *soil-structure interaction* during world conferences is plotted with a *7 year shift* forward relative to the total number of papers on the same subject (per four year periods) in the database. An explanation may be that the world conferences contribute toward influencing the researchers on what is relevant and useful to work on. The process which begins with recognition that a subject or an idea are worth working on, organization (funding) and actual beginning of work, successful completion of work, submission of the finished papers to journals, peer review, revisions, and eventual publication, all appear to take on the average about 7 years. If this is indeed so, it would be interesting to contrast this with the typical 5 to 6 years probation period for tenure track assistant professors, or with duration of funding initiatives of the National Science Foundation, for example. All this is based on an old fashioned assumption, that a recognized and true measure of successful completion of a research task is marked by publication in a respectable professional journal.

8 DISCUSSION AND CONCLUSIONS

8.1 State of Soil-Structure Interaction Research and Full-Scale Testing

The presented review in this paper shows that the research on soil-structure interaction evolved very well since its first (implicit) introduction in the 1930s (not many research topics in earthquake engineering have evolved so well). Also, there is hardly any other topic that is more a *condicio sine qua non* for linear and nonlinear response analyses. Advanced research on structural *system identification*, *health monitoring* and *response control* (via base-isolation, active or/and passive control and use of smart materials) all depend on the ability to write *correct and representative governing equations*, *boundary conditions* and *input excitation*. As the soil-structure interaction alters both the system (additional degrees-of-freedom, modified frequency and damping) and modified input motion, it is essential that it is considered appropriately in such research. Soil-structure interaction also offers powerful possibilities for use as passive energy dissipation mechanism, but at this time it is not sufficiently understood for such application in engineering practice.

Monitoring earthquake response in and around buildings and comprehensive *full-scale tests* of structures are the *best experimental method* for investigating soil-structure interaction. Such full-scale laboratories are the only laboratories where scaling and similarity laws do not pose problems, and where the boundary conditions are satisfied exactly (Trifunac and Todorovska, 1999). Surprisingly, full-scale testing of structures and soil-structure interaction research in the United States (except for piles) has dropped to an *alarmingly low level*, and the reasons appear to lie in our educational programs. Small-scale laboratory tests and computer simulations are useful for understanding selected phenomena in soil-structure interaction. However, laboratory experiments lack the completeness of the full-scale tests, which is particularly true for investigation of soil-structure interaction phenomena (the semi-infinite soil boundary is practically impossible to model in the laboratory). Laboratory experiments are designed to measure what the researcher has decided to study and may help discover new physics only by accident. On the other hand, the as-built environment contains all the physical properties of reality, and the investigators only need to find ingenuous ways to record and interpret them. Obviously, the *priorities* in earthquake engineering research are *not properly balanced* and we are loosing precious time. If we are to improve public safety and reduce financial losses during future earthquake disasters, we must act now to change these trends.

8.2 Difficulties in Interpreting Earthquake Response Data

The physical completeness and the reality of the full-scale structures are necessary but not sufficient conditions to guarantee correct end results. The discovery and understanding of the true nature of response tend to be born by the difficult labor involving reconciliation between our imperfect theories, modeling and analyses, with often *incomplete data* from measurements. Experienced experimentalists know that the first test rarely produces results, as we inevitably forget to measure something, or what we measure does not turn out to be useful. Thus, iterations are almost a rule, in both experiments and in the analyses. Only by following the endless chain

concept→experiment→theory→verification→more detailed experiment→improved theory→ …

we can learn more from the observations of nature and refine our theories and models (Trifunac and Todorovska, 1999).

Often, the difficulty in interpreting earthquake response data recorded in buildings lies in the *non-uniqueness* of the starting models and assumptions. For example, the transfer-functions of horizontal roof displacement of a fixed-base building, and of the same building on flexible soil, have very similar appearance near the first fixed-base frequency, or near the apparent frequency of the soil-structure system. Using a simple identification technique, it is easy to estimate the frequency and the associated fraction of critical damping from full-scale measurements during an earthquake, but it is not easy to identify the factors that control these peaks. *Separation* of the structure's fixed-base frequency from the rocking and translation frequencies associated with soil-structure interaction is less straightforward and can be performed only if additional instrumentation is available. The list of investigators who overlook this non-uniqueness is so long that it seems that this problem is ignored in most published work. Along the same lines, it is common to find papers presenting analyses of nonlinear response of structural components, with discussions of structural ductility and how it relates to the observed changes in the response period, without including in their analyses the fact that, shortly after the earthquake, the apparent period of the soil-structure system was back at or near its pre-earthquake value, indicating that the main source of nonlinearity was not in the structure, but in the soil supporting it (Trifunac et al., 2001a,b).

The non-uniqueness in future data can be eliminated in great part by placing additional instruments to *measure rotation* of the building foundation (Luco et al., 1986; Moslem and Trifunac, 1986). It is interesting to note that in spite of the fact that transducers that record rotational acceleration and velocity have been constructed and tested (Phinney et al., 1962; Shibata et al., 1976; Whitcomb, 1969), essentially no buildings are equipped with such instruments, and so far the earthquake engineers do not seem to request such data. Rocking of the foundation can be calculated from the difference of recorded vertical motions at two points on a line perpendicular to the axis of rotation. The result represents the *average* rotations between the two points. To evaluate the *actual point rotations*, it is necessary to map the pattern of deformations of the building foundation, associated with the apparent frequency of the system prior to and following the earthquake (provided no damage occurred), using forced vibration or ambient vibration tests, for example. This requires detailed full-scale testing and is not available for most buildings (Ivanović et al., 2000).

8.3 Practical Benefits from Full-Scale Testing

Soil-structure interaction can be used to *reduce the structural response*, by utilizing and increasing the effects of scattering of the incident waves from the foundation, radiation of the structural vibration energy into the soil, and (under controlled conditions) via nonlinear response of the soil. Namely, in presence of soil-structure interaction, the system damping depends on the damping in the building and in the soil, and on the scattering of wave energy from and through the foundation (Luco et al., 1986). Design of foundations to scatter efficiently high frequency (short) waves can increase the apparent system damping and can reduce the amplification of the system response near the first mode of vibration (Todorovska and Trifunac, 1992).

Studies of the *dissipation of seismic wave energy by scattering* of incident waves from the foundation, and by radiation of vibrational energy from the structure into the half space, are among the oldest topics studies in the subject area of soil-structure interaction (Sezawa and Kanai, 1935; 1936). Modeling the dissipation of energy of a vibrating structure is constrained by the mathematical methods of analysis, and by the lack of comprehensive measurements, which would show the physical nature of this dissipation (Moslem and Trifunac, 1986; Crouse, 1999). Many linear response analyses use normal mode representation and, to maintain the advantages of working with decoupled equations, approximate the damping matrix by a linear combination $\alpha[m]+\beta[k]$ of the mass and stiffness matrices, $[m]$ and $[k]$, where α and β are constants. For an n degree-of-freedom system, this allows one to choose the damping only for two modal frequencies, ω_i and ω_j, and the remaining $n-2$ modes then have equivalent damping ratios $\zeta_k=0.5(\alpha/\omega_k+\beta\omega_k)$, which are not realistic. A common practice is to use constant damping ratios for all mode-shapes in the response analyses. This, of course, ignores the fact that the solution then violates the original differential equations.

Earthquake response records in combination with detailed full-scale testing of structures, before and after significant earthquake shaking, may be used to *detect* location and extent of *damage* in the structure. However, this would require improved models and theory, which can be done through design of more detailed experiments and more detailed earthquake-monitoring instrumentation in building.

It is often assumed that symmetric buildings, supported by symmetric foundations, on uniformly layered soil, will experience little or no *torsional response*. Also most response analyses ignore torsional excitation caused by the passage of seismic waves across the finite horizontal dimensions of the foundation. Hidden asymmetrics in the foundation system, coupled with the wave passage effects can result in significant torsional response, which cannot be ignored (e.g. see Fig. 11a,b). Such asymmetries can be detected only by full-scale tests of structures (using forced vibration, microtremors and recorded earthquake response). Systematic tests on different structures can be carried out to find how wide spread there asymmetries are and to estimate them empirically for use in the design process, e.g. via accidental torsion eccentricities.

8.4 Recommendations

The bulk of processed earthquake response data recorded in buildings is for *large amplitude response* (e.g. rocking angles greater than 10^{-4} rad.). This results in very different system frequency estimates from strong motion response, and from estimates based on microtremor excitation (e.g. see Fig. 20a, b). These large differences have lead investigators to conclude that the ambient vibration tests are *difficult* to interpret or *not reliable* for inferences on strong motion response. However, as it is seen from Fig. 20, both strong motion data and ambient vibration tests give mutually consistent results, and together help determine the expected changes of the system frequencies for a range of response amplitudes. Consequently, to develop sound mathematical models of soil-structure systems, such changes of system frequencies must be modeled realistically, by incorporating geometric and material nonlinearities into both representations of soil and of structure. To enable this work, numerous *intermediate and small amplitude* earthquake recordings in structures *must be processed and distributed* to researchers. These data will enable researchers to quantify the changes in system frequencies, and to define empirically how those depend on type of structure, foundation, and on the properties of the underlying soil. Correct estimate of the extent of the variability of the system frequency of structures is important in design as the seismic design coefficient *C(T)* depends on the system period.

The significant changes in the system frequencies shown in Fig. 20 for the Hollywood Storage Building are *not unique to this building* and to the soil at this site. We have documented similar changes for other buildings with and without pile foundations (Luco et al., 1986; Trifunac et al., 2001a,b). The range of system frequency variations, shown in Fig. 20 is probably similar for many other buildings. This range may become broader for buildings on piles and on very soft soil, and narrower for buildings on stronger soil and on “rock”. Thus, the dynamic response analyses of soil-structure systems must be based on models that can incorporate the observed nonlinear mechanisms. This, coupled with the fact that most foundation systems of buildings cannot be assumed to be rigid (for purposes of soil-structure interaction analysis), implies that *more complex and realistic models must be developed* both in research and in design applications. Simple models using equivalent linear single-degree-of-freedom system, supported by rigid foundation on elastic soil, neither have adequate number of the degrees-of-freedom, nor have physical properties to allow one to model response of full-scale structures during excitation by moderate and large strong motion amplitudes. Ignoring this, and fitting simple models to the recorded response of full-scale structures, can only create confusion, erroneous interpretation, and misleading generalizations, all caused by non-unique nature of the relationships between the hypotheses and the observed data.

The *next generation of realistic models of soil-structure systems* can be developed and refined only through full-scale studies of buildings and their response to excitations ranging from microtremors, to aftershocks, and all the way to large and destructive motions. To gather *key new data* and to support future work in full-scale studies of soil-structure interaction, *rotational strong motion accelerographs must be added* to the existing instrumented structures, or new accelerographs capable of recording three translations and three rotations simultaneously, must be deployed in place of the old instruments which record translation only. Also, the nonlinear and time-dependent changes of the system behavior will require one to determine *time-dependent transfer-functions* for the system, which can be obtained accurately using the Continuous Wavelet Transform.

9 REFERENCES

1. Abrams, D.P. (Editor) (1996). Experimental Methods, *Earthquake Spectra*, **12**(1), 1-80.

2. Biot, M.A. (1932). Vibrations of Building During Earthquake, Chapter II in *Ph.D. Thesis No. 259* entitled "Transient Oscillations in Elastic Systems", Aeronautics Department, Calif. Inst. Of Tech., Pasadena, California.

3. Biot, M.A. (1933). Theory of Elastic Systems Vibrating Under Transient Impulse with an Application to Earthquake – Proof Buildings, *Proc. National Academy of Sciences*, **19**(2), 262-268.

4. Biot, M.A. (1934). Theory of Vibration of Buildings During Earthquake, *Zeitschrift für Angewandte Matematik and Mechanik*, **14**(4), 213-223.

5. Biot, M.A. (1941). A Mechanical Analyser for the Prediction of Earthquake Stresses, *Bull. Seism. Soc. Amer*., **31**(2), 151-171.

6. Biot, M.A. (19941?). Influence of Foundation on Motion of Blocks (unpublished note).

7. Biot, M.A. (1942). Analytical and Experimental Methods in Engineering Seismology, *ASCE, Transactions*, **108**, 365-4088.

8. Blume, J.A. (1936). The Building and Ground Vibrator, Chapter 7 in "Earthquake Investigations in California 1934-1935", U.S., Dept. of Commerce, Coast and Geoletic Survey, *Special Publication No. 201*, Washington, D.C.

9. Carder, D.S. (1936). Vibration Observation, Chapter 5 in "Earthquake Investigations in California 1934-1935", *Special Publication No. 201*, U.S. Dept. of Commerce, Coast and Geodetic Survey, 49-106.

10. Carder, D.S. (1964). (Editor). Earthquake Investigations in the Western United States 1931-1964, U.S. Dept. of Commerce, Coast and Geodetic Survey, Publication 412.

11. Casirati, M., Castoldi, A., Panzeri, P., Pezzoli, P., Martelli, A., and Masoni, P. (1988). On-Site Experimental Dynamic Analysis to Assess the Seismic Behavior of the Italian PEC Fast Reactor Building Taking into Account the Soil-Structure Interaction Effect, *Proc. Ninth World Conf. on Earthquake Eng*., Vol. VI, 741-746.

12. Celebi, M., and Okawa, I. (editors) (1999). Proc. UJNR Workshop on Soil Structure Interaction, Menlo Park, California, September 22-23, 1998, *Open File Report 99-142*, U.S. Geological Survey.

13. Crouse, C.B., and P.C. Jennings (1975). Soil-Structure Interaction During the San Fernando Earthquake, *Bull Seism. Soc. Amer.*, **65**(1), 13-36.

14. Cloud, W.K. (1978). Modification of Seismic Waves by a Building, *Proc. Sixth European Conf. on Earthquake Eng*., Dubrovnik, Yugoslavia, 289-295.

15. Crouse, C.B. (1999). Energy Dissipation in Soil-Structure Interaction: A Consultant's Perspective, Proc. UJNR Workshop on Soil-Structure Interaction, USGS, *Open File Report 99-142*, 6/1-17.

16. Crouse, C.B., and Hushmand, B. (1990). Soil-Structure Interaction and Nonlinear Site Response at the Differential Array Accelerograph Station, *Proc. 4th U.S. national Conf. On Earthquake Eng.*, Vol. 3, 815-823.

17. Crouse, C.B., Liang, G.C., and Martin, G.R. (1984). Experimental Study of Soil-Structure Interaction at an Accelerograph Station, *Bull. Seism. Soc. Amer*., **74**(5), 1995-2013.

18. Crouse, C.B., Hushmand, B., and Martin, G.R. (1987). Dynamic Soil-Structure Interaction of a Single-Span Bridge, *Earthquake Eng. and Structural Dynamics*, **15**, 711-729.

19. Duke, C.M., Luco, J.E., Carriveau, A.R., Hradilek, P.J., Lastrico, R., and Ostrom, D. (1970). Strong Earthquake Motion and Site Conditions: Hollywood, *Bull. Seism Soc. Amer*., **60**(4), 1271-1289.

20. Ellis, B.R. (1986). The Significance of Dynamic Soil-Structure Interaction in Tall Buildings, *Proc. Instn. Civil Engrs*., Part 2-Research and Theory, Vol. 81, 221-242.

21. Erdik, M., and Gülkan, P. (1984). Assessment of Soil-Structure Interaction Effects on Prefabricated Structures, *Proc. 8th World Conf. On Earthquake Eng*., Vol. III, 913-920.

22. Erdik, M., Yilmaz, C., and Gülkan, P. (1985). Experimental Assessment of the Soil-Structure Interaction Effects for a Research Reactor, *Trans. 8th International Conf. on Structural Mechanics in Reactor Technology*, Vol. K(a), Paper K 7/9, 311-316.

23. Ewing, W.M., Jardetsky, W.S., and Press, F. (1957). Elastic Waves in Layered Media, *McGraw Hill*, New York.

24. Foutch, D.A., Luco, J.E., Trifunac, M.D., and Udwadia, F.E. (1975). Full Scale Three-dimensional Tests of Structural Deformations During Forced Excitation of a Nine-story Reinforced Concrete Building, *Proc. U.S. National Conf. on Earthquake Eng*., 206-215.

25. Fujimori, T., Tsunoda, T., Izumi, M., and Akino, K. (1992). Partial Embedment Effects on Soil-Structure Interaction, *Proc. 10th World Conf. on Earthquake Eng*., Vol. 3, 1713-1718.

26. Fukuoka, M. (1977). Interaction Problems of Well Foundation and Piers, *International Symposium on Soil-Structure Interaction*, Sarita Prakashan, Vol. I, 537-556.

27. Ganev, T., Nagata, S., and Katayama, T. (1993). Creation of Database of Earthquake Records from a Reinforced Concrete Tower and Observation of Soil-Structure Interaction Effects, *Bull. Earthquake Resistant Structure Research Center*, No. 26, 57-73.

28. Gupta, V.K., and Trifunac, M.D. (1990). A Note on Contributions of Ground Torsion to Seismic Response of Symmetric Multistoried Buildings, *Earthquake Eng. and Eng. Vibration*, **10**(3),27-40.

29. Housner, G.W. (1957). Interaction of Building and Ground During an Earthquake, *Bull Seism. Soc. Amer*., **47**, 179-186.

30. Housner, G.W., and McCann, G.D. (1949). The Analysis of Strong-Motion Earthquake Records with the Electric Analog Computer, *Bull. Seism. Soc. Amer*., **39**, 47-56.

31. Hradilek, P.J., Carriveau, A.R., Saragoni, G.R., and Duke, C.M. (1973). Evidence of Soil structure Interaction in Earthquakes, *Fifth World Conf. on Earthquake Eng*., Rome, Italy, Vol. 2, 2076-2079.

32. Hudson, D.E. (1970). Dynamic Tests of Full-Scale Structures. Chapter 7 in “Earthquake Engineering”, edited by R. Wiegel, *Prentice Hall*, New Jersey.

33. lguchi, M., and Luco, J.E. (1982). Vibration of Flexible Plate on Visoclaitic Medium. *J. of Engng. Mech*., ASCE, **108**(6), 1103-1120.

34. Iguchi, M., and Yasui, Y. (1999). Soil-Structure Interaction Researches Relating to Recent Strong Earthquakes in Japan, UJNR Workshop on Soil-Structure Interaction, Sept. 22-23, 1998, *Open File Report 99-142*, Paper 1, 1-17.

35. Iguchi, M., Akino, K., Jido, J., Kawamura, S., Ishikawa, Y., and Nakata, M. (1988). Large Scale Model Tests on Soil-Reactor Building Interaction, Part I: Forced Vibration Tests, *Proc. Ninth World Conf. on Earthquake Eng*., Vol. III, 697-702.

36. Iguchi, M., Niwa, M., Tsunoda, T., Nakai, S., Akino, K., and Noguchi, K. (1988). Large-Scale Model Tests on Soil-Reactor Building Interaction, Part II: Earthquake Observation, *Proc. 9th World Conference on Earthquake Eng*., Vol. VIII, 315-320.

37. International Symposium on Soil Structure Interaction (1977). University of Roorkee, Department of Civil Engineering, Roorkee, Published by Sarita Prakashan, Meerut, India.

38. Inukai, T., Imazawa, T., Kusakabe, K., and Yamaoto, M. (1992). Dynamic Behavior of Embedded Structure on Hard Rock Site, *Proc. 10th World Conf. On Earthquake Eng*., Vol. 3, 1695-1700.

39. Ivanović, S.S., Trifunac, M.D., and Todorovska, M.I. (2000). Ambient Vibration Tests of Structures – A Review, *Indian Society of Earthquake Tecnology Journal*, **37**(4), (in press).

40. Kaino, T., and Kikuchi, T. (1988). Earthquake Response of Pier with Caisson-Type Wall Foundation and its Analysis, *Proc. Ninth World Conf. On Earthquake Eng*., Vol. III, 751-756.

41. Kanai, K. (1983). Engineering Seismology, *Univ. of Tokyo Press*, Tokyo.

42. Kanai, K., and Suzuki, T. (1953). Relation Between the Property of Building Vibration and the Nature of Ground, *Bull. Earthq. Res. Inst*., **31**, 301-317.

43. Kanai, K., Suzuki, T., Hisada, T., and Nakagawa, K. (1955a). Vibration Experiments in the Buildings on Various Kinds of Ground, I, *Report Archit. Inst*., Japan, **33**, p. 185.

44. Kanai, K., Tanaka, T., and Suzuki, T. (1955b). Relation Between the Property of Building Vibration and the Nature of Ground, *Bull. Earthq. Res. Inst*., **34**, p. 61.

45. Kanai, K., Kishinouye, F., and Nasu, N. (1958a). Vibration of Reinforced Concrete Building Moved with Vibration Generators, *Bull. Earthq. Res. Inst*., **36**, 165-182.

46. Kanai, K., Tanaka, T., and Suzuki, T. (1958b). Rocking and Elastic Vibrations of Actual Building I (Experiments with Vibration Generator), *Bull Earthq. Res. Inst*., **36**, 183-199.

47. Kanai, K., Tanaka, T., and Suzuki, T. (1958c). Rocking and Elastic Vibrations of Actual Buildings II (Observation of Earthquake Motion), *Bull Earthq. Res. Inst.*, **36**, 201-226

48. Kashima, T., and Kitagawa, Y. (1988). Observation and Analysis of Earthquake Motions in and Around the SRC Building as Local Earthquake Instrument Array System, *Proc. Ninth world Conf. on Earthquake Eng*., Vol. III, 721-726.

49. Kitada, Y., Kinoshita, M., Iguchi, M., and Fukuwa, N. (1999). Soil-Structure Interaction effect on an NPP Reactor Building-Activities of NPEC; Achievements and the Current Status, Proc. UJNR Workshop on Soil-Structure Interaction, *Open File Report 99-192*, U.S. Department of the Interior, U.S. Geological Survey 18/1-14.

50. Krishna, J. (1981). On Earthquake engineering, in "State of the Art in Earthquake Engineering, 1981", Edited by O. Ergunay and M. Erdik, Ankara, Turkish National Committee in Earthquake Eng., *Kelaynak Publishing House*, Ankara, Turkey.

51. Lee, V.W. (1979). Investigation of Three-Dimensional Soil-Structure Interaction, *Dept. of Civil Eng. Report No. 79-11*, Univ. of Southern California, Los Angeles, California.

52. Liou, G.-S., and Huang, P.H. (1994). Effect, of Flexibility on Impedance Functions for Circular Foundations, *J. of Engng. Mech*., ASCE, **120**(7), 1429-1446.

53. Luco, J.E. (1969a). Application of Singular Integral Equations on the Problem of Forced Vibrations of a Rigid Foundation, *Ph.D. Thesis*, U. California, Los Angeles, Los Angeles, California.

54. Luco, J.E. (1969b). Dynamic Interaction of a Shear Wall with the Soil, *J. Eng. Mechanics Division*, ASCE, **95**, 333-346.

55. Luco, J.E. (1976). Torsional Response of Structures to Obliquely Incident Seismic SH Waves, *Earthquake Eng. and Structural Dynamics*, **4**(3), 207-219.

56. Luco, J.E. (1980). Linear Soil Structure Interaction, in "Seismic Safety Margins Research Program (Phase I)", U.S. Nuclear Regulatory Commission, Washington D.C.

57. Luco, J.E., Trifunac, M.D. and Udwadia, F.E. (1975). An Experimental Study of Ground Deformations Caused by Soil-Structure Interaction, *Proc. U.S. National Conf. On Earthquake Eng.*, 136-145.

58. Luco, J.E., Wong, H.L., and Trifunac, M.D. (1986). Soil-Structure Interaction Effects on Forced Vibration Tests, Dept. of Civil Engrg, *Report No. 86-05*, University of Southern California, Los Angeles, California

59. Luco, J.E., Trifunac, M.D., and Wong, H.L. (1987). On the Apparent Change in Dynamic Behavior of a Nine-Story Reinforced Concrete Building, *Bull. Seism. Soc. Amer*., **77**(6), 1961-1983.

60. Maragakis, E., Douglas, B.M., Hagne, S. and Sharma, V. (1996). Full-Scale Resonance Tests of a Railway Bridge, Build. and International Community of Structural Eng., *Proc. Structures Congress XIV*, Chicago, Vol. 1, 183-190.

61. Merritt, R.G. (1953). Effect of Foundation Compliance on the Earthquake Stresses in Typical Buildings, *Ph.D. Thesis*, Calif. Inst. of Tech., Pasadena, California.

62. Merritt, G.H., and Housner, G.W. (1954). Effects of Foundation Compliance on Earthquake Stresses in Multistory Buildings, *Bull. Seism. Soc. Amer*., **44**(4), 551-569.

63. Mizuno, H. (1978). Effect of Dynamic Inputs on Structure-Soil-Structure Interaction, *Proc. 5th Japan Earthquake Eng. Symposium*, Paper No. 68, 539-544.

64. Mizuno, H. (1980) Effects of Structure-Soil-Structure Interaction During Various Excitations, *Proc. 7th World Conf. on Earthquake Eng*., Vol. 5, 149-156.

65. Mizuno, N., and Tsushima, Y. (1975). Experimental and analytical Studies for a BWR Nuclear Reactor Building, Evaluation of Soil-Structure Interaction Behavior, *Trans. 3rd International Conf. on Structural Mechanics in Reactor Technology*, Vol. 4, Paper k3/2, 1-13.

66. Moslem, K., and Trifunac, M.D. (1986). Effects of Soil Structure Interaction on the Response of Buildings During Strong Earthquake Ground Motions. Dept. of Civil Engrg, *Report No. 86-04*, Univ. of Southern California, Los Angeles, California.

67. Muria-Vila, D., and Alcorta, R.G. (1992). Soil-Structure Interaction Effects in a Building, *Proc. Tenth World Conf. on Earthquake Eng*., Vol. 3, 1905-1910.

68. Ohtsuka, Y., Fukuoka, A., Yanagisawa, E., and Fukudome, H. (1992). Embedment Effect on Dynamic Soil-Structure Interaction, *Proc. 10th world Conf. on Earthquake Eng*., Vol. 3, 1707-1712.

69. Ohtsuka, Y., Fukuoka, A., Akino, K., and Ishida, K. (1996). Experimental Studies on Embedment Effects on Dynamic Soil-Structure Interaction, *Proc. 11th World Conf. on Earthquake Eng*., Disc 1, Paper No. 59.

70. Papageorigiou, A.S., and Lin, B.C. (1991). Analysis of Recorded Earthquake Response and Identification of a Multi-story Structure Accounting for Foundation Interaction Effects, *Soil Dynamics and Earthquake Eng*., **10**(1), 5-64.

71. Petrovski, J. (1975). Evaluation of Soil Structure Interaction parameters from Dynamic Response of Embedded Footing, *Proc. 5th European Conf. on Earthquake Eng*., Vol. 1, Paper 43, 1-8.

72. Petrovski, J. (1978). Influence of Soil-Structure Interaction Effects on Dynamic Response of Large Panel Prefabricated Buildings, *Proc. Second International Conf. on Microzonation for Safer Construction-Research and Application*, Vol. III, 1269-1277.

73. Phinney, R., Oilman, R., and Press, F. (1962). Progress Report on a Short Period Rotational Seismometer, *J. Geophys. Res*., **67**(9).

74. Ramirez-Centeno, M., and Ruiz-Sandoval, M. (1996). Experimental Study of the Soil-Structure Interaction Effects on Three Different Kinds of Accelerometric Bases, *Proc. 11th World Conf. on Earthquake Eng*., Disc 2, Paper No. 719.

75. Safak, E. (1992). On Identification of Soil-Structure Interaction from Recorded Motions of Buildings, *Proc. Tenth World Conf. on Earthquake Eng*., Vol. 3, 1885-1890.

76. Sezawa, K., and K. Kanai (1935). Decay in the Seismic Vibration of a Simple or Tall Structure by Dissipation of Their Energy into the Ground, *Bull. Earth. Res. Inst*., XIII, Part 3, 681-697.

77. Sezawa, K., and K. Kanai (1936). Improved Theory of Energy Dissipation in Seismic Vibrations on a Structure, *Bull. Earth. Res. Inst*., XIV, part 2, 164-168.

78. Shibata, H., Shigeta, T., and Sone, A. (1976). A Note on Some Results of Observation of Torsional Ground Motions and their Response Analysis. *Bull. Earthquake Resistant Struct. Research Center*, 10,43-47.

79. Shinozaki, Y., Kobori, T., Nagashima, I., Nakamura, M. and Ichikawa, M. (1994). Soil-Structure Interaction Tests of Five-Story Steel-Frame Building by Servo Hydranlic-Type Vibrator, Part 2 (in Japanese), *Jour. of Structural and Construction Eng.* (Transactions AIJ), 465, 53-59.

80. Shioya, K., and Yamahara, H. (1980). Study on Filtering Effect of Foundation Slab Based on Observational Records, *Seventh World Conf. On Earthquake Eng*., Istanbul, Turkey, Vol. 5,181-188.

81. State of the Art in Earthquake Engineering (1981). State of the Art panel Reports, Edited by Ergunay, O. and Erdik, M., Turkish National Committee on Earthquake Eng., Ankara, Turkey.

82. Suyehiro, K. (1932). Engineering Seismology Notes on American Lectures, *Proc. ASCE*, **58**(4), 1-110.

83. Tang, H.T., Tang, Y.K., Stepp, J.C., Wall, I.B., Lin, E., Chang, S.C. and Lee, S.K. (1989). A Large-Scale Soil-Structure Interaction Experiment: Design and Construction, *Nuclear Eng. and Design*, Vol. III, 371-379.

84. Tobita, J., Fukuwa, N. and Yagi, S. (2000). Experimental Evaluation of Dynamic Characteristics of Low and Medium-Rise Buildings with Soil-Structure Interaction, *Proc. 12th World Conf. on Earthquake Eng*., Paper No. 2244.

85. Todorovska, M.I., and Trifunac, M.D. (1989). Antiplane Earthquake Waves in Long Structures, *J. Engrg. Mech*., ASCE, **115**(12), 2687-2708.

86. Todorovska, M.I., and M.D. Trifiunac (1990a). Analytical Model for Building Foundation. Soil Interaction : Incident P, SV and Raleigh Waves, Dept. of Civil Eng., *Rep. No. 90-01*, Univ. of Southern California, Los Angeles, California.

87. Todorovska, M.I., and Trifunac, M.D. (1990b). A Note on the Propagation of Earthquake Eaves in Buildings with Soft First Floor, *J. Engrg Mech*., ASCE, 116(4), 892-900.

88. Todorovska, M.I., and Trifunac, M.D. (1992). The System Damping, the System Frequency and the System Response Peak Amplitudes During in-Plane Building-Soil Interaction, *Earthquake Engrg and Struct. Dynam.*, **21**(2), 127-144.

89. Tohma, J., Ohmoto, K., Kurimoto, M., and Arii, K. (1985). Experimental Study on Dynamic Soil-Structure interaction, *Trans. 8th International Conf. On Structural Mechanics in Reactor Technology*, Vol. K(a), Paper K 7/3, 287-292.

90. Toki, K., and Kiyono, J. (1992). Seismic Analysis of Nonlinear Soil-Structure Interaction System using a Hybrid Experiment, *Proc. 10th World Conf. on Earthquake Eng*., Vol. 3, 1839-1844.

91. Trifunac, M.D. (1970a). Wind and Microtremor Induced Vibrations of a 22-story Steel Frame Building. Earthquake Engrg Res. Lab., *Report EERL 70-01*, Calif. Inst. of Tech., Pasadena, California.

92. Trifunac, M.D. (1972a). Interaction of Shear Wall with the Soil for Incident Plane SH Waves, *Bull. Seism. Soc. Amer*., **62**(1), 63-83.

93. Trifunac, M.D. (1970b). Ambient Vibration Test of a 39-story Steel Frame Building, Earthq. Engrg Res. Lab., *Report EERL 70-01*, Calif. Inst. of Tech., Pasadena, California.

94. Trifunac, M.D. (1972b). Comparison Between Ambient and Forced Vibration Experiments, *Earthq. Engrg and Struct. Dynam*., **1**,133-150.

95. Trifunac, M.D. (1997). Differential Earthquake Motion of Building Foundations, *J. of Structural Engrg*, ASCE, **123**(4), 414-422.

96. Trifunac, M.D., and Lee, V.W. (1973). Routine Computer Processing of Strong Motion Accelerograms, Earthquake Eng. Res. Lab., *Report EERL 73-03*, Calif. Inst. of Tech., Pasadena, California.

97. Trifunac, M.D., and V.W. Lee (1979), Automatic Digitization and Processing of Strong- Motion Accelerograms, Parts I and II, Dept. of Civil Eng., *Rep. No. 79-15*, Univ. of Southern California, Los Angeles, California.

98. Trifunac, M.D., and Todorovska, M.I. (1997). Response Spectra and Differential Motion of Columns, *Earthquake Eng. and Structural Dynamics*, **26**(2),251-268.

99. Trifunac, M.D., and Todorovska, M.I. (1998). Nonlinear Soil Response as a Natural Passive Isolation Mechanism - the 1994 Nonhridge, California, Earthquake, *Soil Dynam. and Earthquake Engrg*., **17**(1), 41-51.

100. Trifunac, M.D., and Todorovska, M.I. (1999a). Recording and Interpreting Earthquake Response of Full Scale Structures, *Proc. NATO Advanced Research Workshop on Strong Motion Instrumentation for Civil Eng. Structures*, June 2-5, Istanbul, Kluwer.

101. Trifunac, M.D., and Todorovska, M.I. (1999b). Reduction of Structural Damage by Nonlinear Soil Response, *J. Structural Eng.*, ASCE, **125**(1), 89-97.

102. Trifunac, M.D., Ivanović, S.S., Todorovska, M.I. , Novikova, E.I., and Gladkov, A.P. (1999). Experimental Evidence for Flexibility of a Building Foundation Supported by Concrete Friction Piles, *Soil Dynamics and Earthquake Eng*., **18**(3), 169-187.

103. Trifunac, M.D. Ivanović, S.S., and Todorovska, M.I. (2001a). Apparent Periods of a Building, Part I: Fourier Analysis, *J. of Structural Engrg*, ASCE (in press).

104. Trifunac, M.D. Ivanović, S.S., and Todorovska, M.I. (2001b). Apparent Periods of a Building, Part II: Time-Frequency Analysis, *J. of Structural Engrg*, ASCE (in press).

105. Tuzuki, M., Inada, O., Yamaguchi, M., Yahata, K., Naito, Y., and Kitamura, E. (1992). Field Testing and Analysis of Dynamic Loaded Pile Group, *Proc. 10th World Conf. On Earthquake Eng*., Vol. 3, 1787-1790.

106. Uchiyama, S., Suzuki, Y., and Konno, T. (1992). Dynamic Tests of Concrete Block on Gravel Deposits, *Proc. 10th World Conf. On Earthquake Eng*., Vol. 3, 1859-1864.

107. Udwadia, F.W. and Trifunac, M.D. (1974). Time and Amplitude Dependent Response of Structures, *Earthquake Engrg and Struct. Dynam*., **2**, 359-378.

108. Ucshima, T., Sawada, Y., Yajima, H., Ohtomo, K., and Sawada, M. (1988). Dynamic Characteristics of Large Structure "JOYO" Deeply Embedded in Quaternary Ground and Evaluation of Embedment Effect, *Proc. 9th World conf. on Earthquake Eng*., Vol. III, 739-744.

109. Urao, K., Masuda, K., Kitamura, E., Sasaki, F., Ueno, K., Miyamoto, Y., and Mori, T. (1988). Forced Vibration Test and its Analytical Study for Embedded Foundation Supported by Pile Group, *Proc. Ninth World Conf. on Earthquake Eng.*, Vol. III, 673-678.

110. Watakabe, M., Matsumoto, H., Ariizumi, K., Fukahori, Y., Shikama, Y., Yamanouchi, K., and Kuniyoshi, H. (1992). Earthquake Observation of Deeply Embedded Building Structure, *Proc. Tenth World Conf. on Earthquake Eng*., Vol. 3, 1831-1838.

111. Whitcomb, J. (1969). Detecting of SH-type Seismic Shear Waves by Means of Angular Accelerometers, Geol. Survey, *Professional Paper No. 599-D.*

112. Whitley, J.R., Morgan, J.R., Hall, W.J., and Newmark, N.M. (1977). Base Response Arising from Free-Field Motion, *Trans. 4th Instructional Conf. of Structural Mechanics in Reactor Technology*, K2/15,1-10.

113. Wolf. J.P. (1985). Dynamic Soil-Structure Interaction, *Prentice-Hall*, Englewood Cliffs, N.J.

114. Wolf, J.P. (1994). Foundation Vibration Analysis Using Simple Physical Models, *Prentice-Hall*, Englewood Cliffs, N.J.

115. Wong, H.L. and Trifunac, M.D. (1974). Interaction of a Shear Wall with the Soil for Incident Plane SH Waves: Elliptical Rigid Foundation. *Bull Seism. Soc. Amer*., **64**, 1825-1842.

116. Wong, H.L., Trifunac, M.D. and Luco, J.E. (1988). A Comparison of Soil Structure Interaction Calculations with Results of a Full-Scale Forced Vibration test, *Soil Dynamics and Earthquake Eng*., **7**(1), 22-31.

117. Yahata, K., Naito, Y., Urao, K., Nakayama, T., Inada, O., Mori, F. and Tuzuki, M. (1992). Full Scale Vibration Test on Pile-Structure and Analysis, *Proc. Tenth World Conf. on Earthquake Eng.*, Vol. 3, 1781-1786.