The Loma Prieta, California, Earthquake of October 17, 1989—Building Structures

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PERFORMANCE OF THE BUILT ENVIRONMENT
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BUILDING STRUCTURES

INTRODUCTION

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Several approaches are used to assess the performance of the built environment following an earthquake—preliminary damage surveys conducted by professionals, detailed studies of individual structures, and statistical analyses of groups of structures. Reports of damage that are issued by many organizations immediately following an earthquake play a key role in directing subsequent detailed investigations. Examples of these preliminary studies for the Loma Prieta earthquake are the readily available excellent reports cited below:

Astaneh, A., Bertero, V., Bolt, B., Mahin, S., Moehle, J., and Seed R., 1989, Preliminary report on the seismological and engineering aspects of the October 17, 1989 Santa Cruz (Loma Prieta) earthquake: University of California, Berkeley/Earthquake Engineering Research Center Report 89/14, October 1989, 51 p.

Benuska, L., ed., 1990, Earthquake spectra: Loma Prieta Earthquake Reconnaissance Report, May 1990, v. 6 (supp.), 448 p.

Housner, G.W., Chairman, and Thiel, C.C., ed., 1990, Competing against time: Report to the Governor of California from the Governor's Board of Inquiry on the 1989 Loma Prieta earthquake, 264 p.

Lew, H.S., ed., 1990, Performance of structures during the Loma Prieta earthquake of October 17, 1989: U.S. Dept. of Commerce, National Institute of Standards and Technology, National Institute of Standards and Technology Special Publication 778 (ICSSC TR11), 201 p.

Bay Area Regional Earthquake Preparedness Project (BAREPP) and Federal Emergency Management Agency (FEMA), 1990, Putting the pieces together—the Loma Prieta earthquake one year later, *in* Proceedings of a national conference, Oct. 15-18, 1990: 253 p.

Earthquake Engineering Research Institute, 1989, Loma Prieta earthquake of October 17, 1989—Preliminary reconnaissance report: Earthquake Engineering Research Institute Report 89-03, 51 p .

Plafker, G. ,and Galloway, J., eds., 1989, Lessons learned from the Loma Prieta, California, earthquake of October 17, 1989: U.S. Geological Survey Circular 1045, 48 p.

Detailed studies of individual structures and statistical analyses of groups of structures may be motivated by particularly good or bad performance during an earthquake (see table 1). Beyond this, practicing engineers typically perform stress analyses to assess the performance of a particular structure to vibrational levels experienced during an earthquake. The levels may be determined from recorded or estimated ground motions; actual levels usually differ from design levels. If a structure has seismic instrumentation to record response data, the estimated and recorded response and behavior of the structure can be compared. Following the Loma Prieta earthquake, the two reports listed below played an important role in providing information on recorded ground and structural response:

Maley, R., Acosta, A., Ellis, F., Etheredge, E., Foote, L., Johnson, D., Porcella, R., Salsman, M., and Switzer, J., 1989, U.S. Geological Survey strong-motion records from the Northern California (Loma Prieta) earthquake of October 17, 1989, U.S. Geological Survey Open-File Report 89-568, 85 p.

Shakal, A.F., Huang, M., Reichle, M., Ventura, C., Cao, T., Sherburne, R., Savage, R., Darragh, R., and Petersen, C., 1989, CSMIP strong-motion records from the Santa Cruz Mountains (Loma Prieta), California, earthquake of 17 October 1989: California Office of Strong Motion Studies Report 89-06, 196 p.

These reports are issued by two organizations that have established structural instrumentation programs: the California Strong Motion Instrumentation Program (CSMIP) of the California Division of Mines and Geology (CDMG) of the State of California and the National Strong Motion Program of the United States Geological Survey (USGS).

The paper in this volume by Çelebi provides an extensive summary of studies of recorded responses for instrumented structures in the San Francisco Bay area. Such studies constitute an integral part of earthquake-hazard-reduction programs leading to improved design/analyses procedures. In addition to the aim of studying recorded responses for buildings and other structures to improve

Table 1. Papers presented in this volume categorized as detailed investigations and statistical summaries

Author(s)	Title of paper
	Detailed Investigations
Çelebi	Performance of building structures—A summary
Wood	Measured response of two tilt-up buildings
Anderson and Bertero	Seismic response of a six-story reinforced concrete building
Anderson and Bertero	Seismic response of a 42-story building
	Statistical Summaries
Lizundia and others	A summary of unreinforced masonry building damage patterns—Implications for improvements in loss-estimation methodologies
Comerio	Housing repair and reconstruction after the earthquake
Perkins and Chuaqui	Impact of the earthquake on habitability of housing units

design/analyses procedures, a second motivation for studying the Loma Prieta earthquake response data from instrumented structures is that the probability of magnitude 7 or larger earthquakes occurring in the San Francisco Bay Area from major faults, including the San Andreas and Hayward faults, is considered to be approximately 67 percent or higher within a 30-year period (Working Group, 1990). Furthermore, for the tall buildings in San Francisco and vicinity, epicenters of these expected earthquakes may be closer than the distances to the Loma Prieta epicenter. These buildings may thus be subjected to motions larger and different from those recorded during the Loma Prieta earthquake. Therefore, studies of this type will help to better predict the performance of structures during future earthquakes. Furthermore, a considerable number of these tall buildings are on soft soil sites in San Francisco and vicinity, which provides an opportunity to assess their responses and design parameters under amplified motions. The paper summarizes numerous studies of recorded response data from instrumented structures that have been published to date. Also, the paper includes references to the low-amplitude (ambient) vibration testing of five buildings that also recorded the Loma Prieta earthquake.

The paper by Wood and Hawkins investigate the seismic behavior of two tilt-up buildings (a two-story building in Milpitas and a one-story building in Hollister), both built within the 10 years prior to the earthquake. Both buildings are within 50 km from the epicenter and recorded similar responses despite the fact that they were constructed by different methods. The authors report that the transverse accelerations at the center of the roof of each building were approximately three times that at the base of the buildings. The significance of this study is that design provisions of tilt-up buildings must be improved so that the flexibility of diaphragms is decreased. Similar studies and changes made in the building codes (for example, Uniform Building Code, 1991) are included in the paper by Çelebi.

Anderson and Bertero present studies of 6-story and 42-story buildings that had recorded response data. They developed three-dimensional, linear elastic models for both buildings and studied their responses. In the case of the 6-story building, under recorded Loma Prieta base motions the models confirm that limited inelastic behavior takes place, as was observed in the actual inspection of the building following the earthquake. They report that the 42-story building remained elastic during the earthquake. They attribute this to the fact that the designers of the building opted to use a site-specific design-response spectra range that was more conservative than the code minimum requirements.

Lizundia and others studied the performance of unstrengthened, unreinforced masonry (URM) bearing-wall buildings damaged during the earthquake from a data base of 4,800 such buildings. The results are compared with those from data bases of past earthquakes and correlated with intensity scales. They present a loss estimation methodology using the correlated results.

Perkins and Chuaqui studied the impact of the earthquake on 16,000 housing units in the San Francisco and Monterey areas that were assessed to be uninhabitable—defined as unable to be occupied due to structural problems. The data set was collected by telephone and in-person interviews with additional effort to quantify other characteristics of the units (such as the age and type of construction). Using models designed to provide estimates of a number of uninhabitable units, they produced such estimates for the San Francisco Bay area and Monterey areas for future earthquake scenarios. These estimates are important in directing efforts to retrofit vulnerable structures.

Comerio also studied housing losses that occurred as a result of the earthquake and consequent attempts to provide emergency, temporary, and housing recovery service in the San Francisco, Oakland, Santa Cruz, and Watsonville areas. She makes specific recommendations on how Federal, state and local governments can improve recovery services. These recommendations are (1) postearthquake housing recovery requires planning, (2) existing recovery programs should be streamlined to expedite services, and (3) housing recovery programs will be most effective if they are administered at the local level.

REFERENCES CITED

Uniform Building Code, 1991, International conference of building officials: Whittier, Calif., 1050 p. (and other editions).

Working Group on California Earthquake Probabilities, 1990, Probabilities of large earthquakes in the San Francisco Bay Region, California: U.S. Geological Survey Circular 1053, 51 p.

BUILDING STRUCTURES

PERFORMANCE OF BUILDING STRUCTURES—A SUMMARY

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CONTENTS

ABSTRACT

The purpose of this paper is to summarize studies of the performance of building structures during the earthquake. The majority of studies summarized herein are of those buildings which were instrumented prior to the earthquake and whose responses were recorded during the earthquake. Planning for, acquiring, and studying the recorded responses of building structures is an important part of earthquake-hazard-reduction programs. Such studies help in forecasting performance during future events and therefore are essential for mitigation efforts. Furthermore, such studies facilitate confirmation and improvement of design and analyses methods.

There is as great a variation in the type of buildings studied as there is in their performance. In this summary, the studies reflect, in varying detail, those issues related to the design and/or analyses methods. The dynamic characteristics of the buildings, if identified, have been included. The behavior of the buildings is discussed in terms of translational and torsional modal characteristics and actions such as soil-structure interaction (translational, rotational or rocking, and radiation damping), drift ratios, and resonation (or combination thereof) exhibited and identified from the recorded responses. Specific conclusions that are derived from the studies are also summarized.

Included in the paper are summaries of some specific studies of performance characteristics such as pounding based on observations made following the earthquake.

INTRODUCTION

Studies of recorded responses of instrumented structures constitute an integral part of earthquake-hazard-reduction programs leading to improved design/analyses procedures. The California Strong-Motion Instrumentation Program (CSMIP) of the California Division of Mines and Geology (CDMG) of the State of California and the National Strong Motion Program of the United States Geological Survey (USGS) have established structural instrumentation programs to measure structural responses to earthquakes. While these programs are prominent, other institutions and private owners have also instrumented structures throughout the continental United States, Alaska, Hawaii, and Puerto Rico.

During the earthquake the response of numerous buildings and other structures throughout the San Francisco Bay area, Santa Cruz, and vicinity were recorded. Summaries of strong-motion records retrieved by CDMG and USGS from different types of structures and ground stations are provided by Shakal and others (1989) and Maley and others (1989). A summary of records from instrumented buildings (only) are provided in table 1, which lists 5 buildings instrumented by USGS and 23 by CSMIP. Although there are records from buildings instrumented privately by their owners, they are not included in the table because the data from such structures are not generally available. Buildings instrumented by owners

Table 1.—*Summary of instrumented buildings that recorded the earthquake*

Epicentral distance(km)	Building description	Number of channels	Peak acceleration (g) (horizontal)	Organization
96	Pacific Park Plaza, 633 Christie Ave.,Emeryville ; 30 stories, symmetrical, three-winged reinforced concrete (on bay mud)	24+3 FF FF=Free-Field	FF(0.26 g) Ground (0.22 g) Roof wing (0.39 g)	USGS
74	Hayward City Hall; 11-story, reinforced concrete framed structure (on consolidated alluvium)	12+6 FF	FF(0.10 g) Ground(0.07 g) 12th floor (0.13 g)	USGS
99	Great Western Bldg., 2168 Shattuck Ave., Berkeley; reinforced concrete core, truss structure at roof supports the suspended floors (on stiff soil)	18	Basement (0.11g) 13th floor (0.23 g)	USGS
96	Chevron Bldg., 575 Market St., San Francisco; 41-story, moment-resisting steel framed structure on precast piles	14	Basement (0.11 g) 25th floor (0.23 g)	USGS
97	Transamerica Bldg.; 48-story+204 ft tower steel framed on 9 ft basemat (on stiff soil)	22	Basement (0.11 g) 49th floor (0.31 g)	USGS
18	4-story concrete bldg., Watsonville (CSMIP No. 47459)	13	Ground (0.39 g) Roof (1.24 g)	CSMIP
21	3-story steel bldg., San Jose (57562)	10	Ground (0.28 g) Roof (0.67 g)	CSMIP
27	1-story gymnasium, West Valley College, Saratoga (58235)	11	Ground (0.33 g) Roof (0.87 g)	CSMIP
28	2-story historic commercial building, Gilroy (57476)	6	Ground (0.25 g) Roof (0.99 g)	CSMIP
48	1-story warehouse, Hollister (47391)	13	Ground (0.18 g) Roof (0.82 g)	CSMIP
33	10-story concrete residential bldg., San Jose (57356)	13	Ground (0.13 g) Roof (0.37 g)	CSMIP
33	10-story concrete commercial bldg., San Jose (57355)	13	Ground (0.11 g) Roof (0.38 g)	CSMIP
35	13-story, steel, Santa Clara County Office Bldg., San Jose (57357)	22	Ground (0.11 g) Roof (0.36 g)	CSMIP
50	2-story masonry office bldg, Palo Alto (58264)	7	Ground (0.21 g) Roof (0.55 g)	CSMIP
57	3-story concrete school office bldg., Redwood City (58263)	6	Ground (0.09 g) Roof 0.17 g)	CSMIP
65	2-story concrete office bldg., Belmont (58262)	7	Ground (0.11 g) Roof (0.20 g)	CSMIP
81	9-story concrete government office bldg., San Bruno (58394)	16	Ground (0.16 g) Roof (0.36 g)	CSMIP
81	6-story office bldg., San Bruno (58490)	13	Ground (0.14 g) Roof (0.46 g)	CSMIP
85	4-story steel hospital bldg, So. San Francisco (58261)	11	Ground (0.15 g) Roof (0.68 g)	CSMIP
95	6-story, concrete UCSF bldg., San Francisco (58479)	13	Ground (0.09 g) Roof (0.28 g)	CSMIP

Table 1.—*Continued.*

Epicentral distance(km)	Building description	Number of channels	Peak acceleration (g) (horizontal)	Organization
95	18-story, steel/concrete, commercial bldg., San Francisco (58480)	13	Ground (0.14 g) Roof (0.27 g)	CSMIP
96	47-story steel office bldg., San Francisco (58532)	18	Ground (0.20 g) Roof (0.48 g)	CSMIP
124	3-story steel/concrete office bldg., San Rafael (68341)	16	Ground (0.04 g) Roof (0.13 g)	CSMIP
171	14-story concrete residential bldg., Santa Rosa (68489)	16	Ground (0.06 g) Roof (0.21 g)	CSMIP
172	5-story concrete commercial bldg., Santa Rosa (68387)	16	Ground (0.06 g) Roof (13 g)	CSMIP
43	2-story, tilt-up, industrial bldg., Milpitas (57502)	13	Ground (0.14 g) Roof (0.58 g)	CSMIP
69	6-story concrete office bldg., Hayward (58462)	13	Ground (0.12 g) Roof (0.45 g)	CSMIP
70	13-story steel/concrete CSUH Admin. Bldg., Hayward (58354)	16	Ground (0.09 g) Roof (0.24 g)	CSMIP
70	4-story, concrete, CSUH Science Bldg., Hayward (58488)	16	Ground (0.05 g) Roof (0.18 g)	CSMIP
91	24-story, concrete, residental bldg., Oakland (58483)	16	Ground (0.18 g) Roof (0.38 g)	CSMIP
92	2-story masonry/steel office bldg., Oakland (58224)	10	Ground (0.26 g) Roof (0.69 g)	CSMIP
93	3-story concrete Piedmont Jr. High School., Piedmont (58334)	11	Ground (0.08 g) Roof (0.18 g)	CSMIP
97	2-story steel hospital bldg., Berkeley (58496)	12	Ground (0.12 g) Roof (0.30 g)	CSMIP
98	10-story concrete commercial bldg., Walnut Creek (58364)	16	Ground (0.10 g) Roof (0.25 g)	CSMIP
102	3-story concrete commercial bldg., Pleasant Hill (58348)	12	Ground (0.13 g) Roof (0.24 g)	CSMIP
105	8-story, masonry, residential bldg., Concord (58492)	13	Ground (0.06 g) Roof (0.24 g)	CSMIP
108	3-story, concrete, City Hall, Richmond (58503)	13	Ground (0.12 g) Roof (0.24 g)	CSMIP
112	3-story, steel, office bldg., Richmond (58506)	12	Ground (0.12 g) Roof (0.29 g)	CSMIP

according to the Uniform Building Code (UBC) recommendations also are not included in this table. Figure 1 shows the locations of some of the buildings discussed in this summary relative to the epicenter.

While the primary motivation in studying recorded responses of buildings and other structures is to improve design/analyses procedures, a second motivation for studying the Loma Prieta response data is that the probability of magnitude 7 or larger earthquakes occurring in the San Francisco Bay area from major faults, including the San Andreas and Hayward faults, is considered to be approximately 67 percent or higher within a 30-year period (Working Group, 1990). Furthermore, these earthquakes may originate at distances that are closer to major urban areas than the 97-km distance of the Loma Prieta event from San Francisco and may generate motions larger than those recorded during Loma Prieta. Therefore, studies of this type will help to better predict the performance of structures during future earthquakes.

A third motivation for these studies is that in the San Francisco Bay area there are several tall buildings on soft soil sites having seismic instrumentation. Records obtained from these buildings are particularly important to evaluate the performance of such structures in response to amplified ground motions and possible soil-structure interaction effects. Figure 1 shows the location of some of the buildings that are covered in this paper and that were subjected to amplified motions during the earthquake at

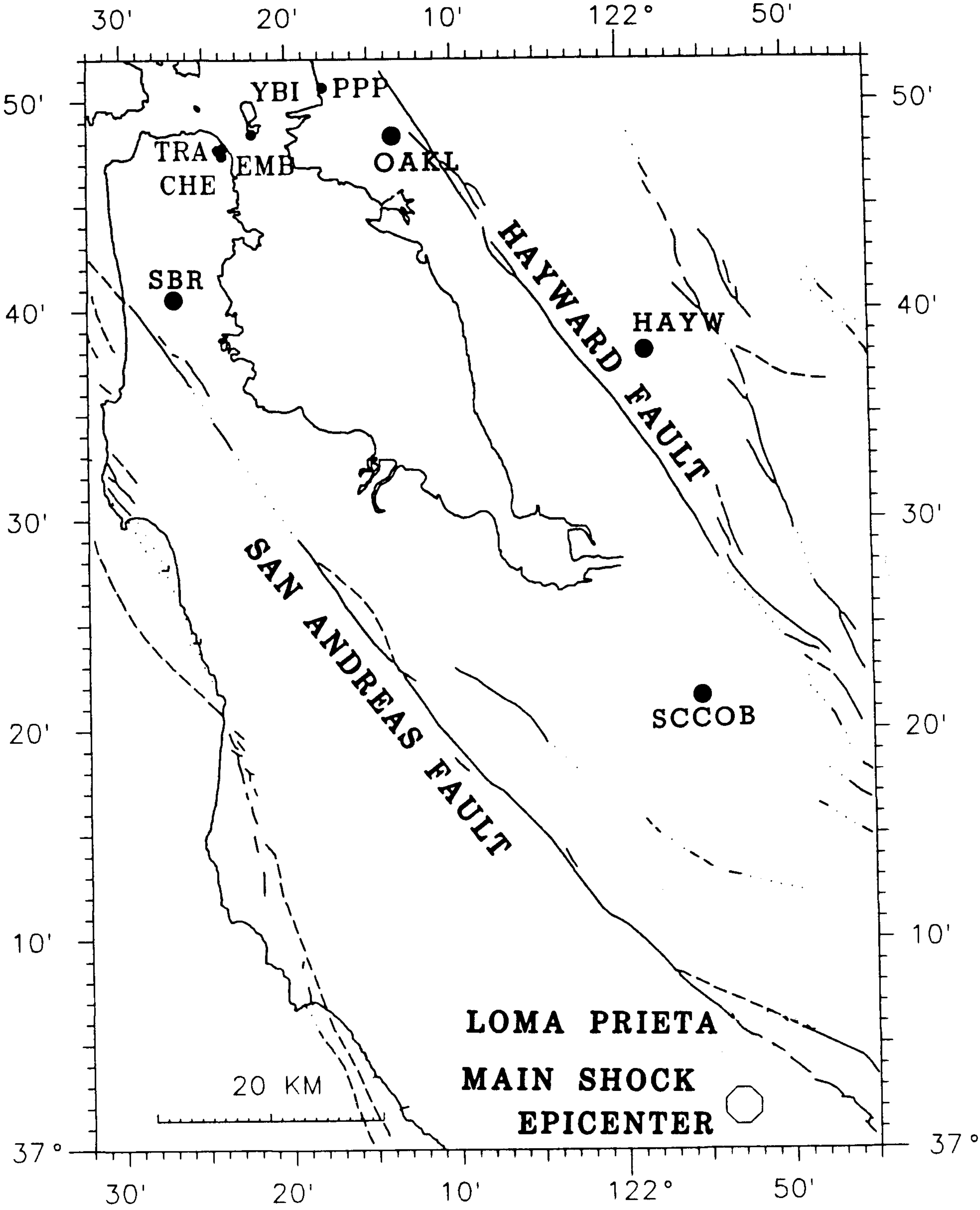


Figure 1.—Location of some instrumented buildings relative to epicenter. Pacific Park Plaza (PPP), Transamerica Building (TRA), Embarcadero Building (EMB), Chevron Building (CHE), two-story building in Oakland (OAKL), California State University (Hayward) (HAYW), Santa Clara County Office Building (SCCOB), Yerba Buena Island (YBI).

approximately 100 km from the epicenter. To demonstrate the degree of amplified motions at these sites during the earthquake, response spectra of ground motions recorded at dedicated free-field stations in the vicinity or at the ground floor or basement of the four tall buildings are compared (fig. 2) to the spectrum from the station on

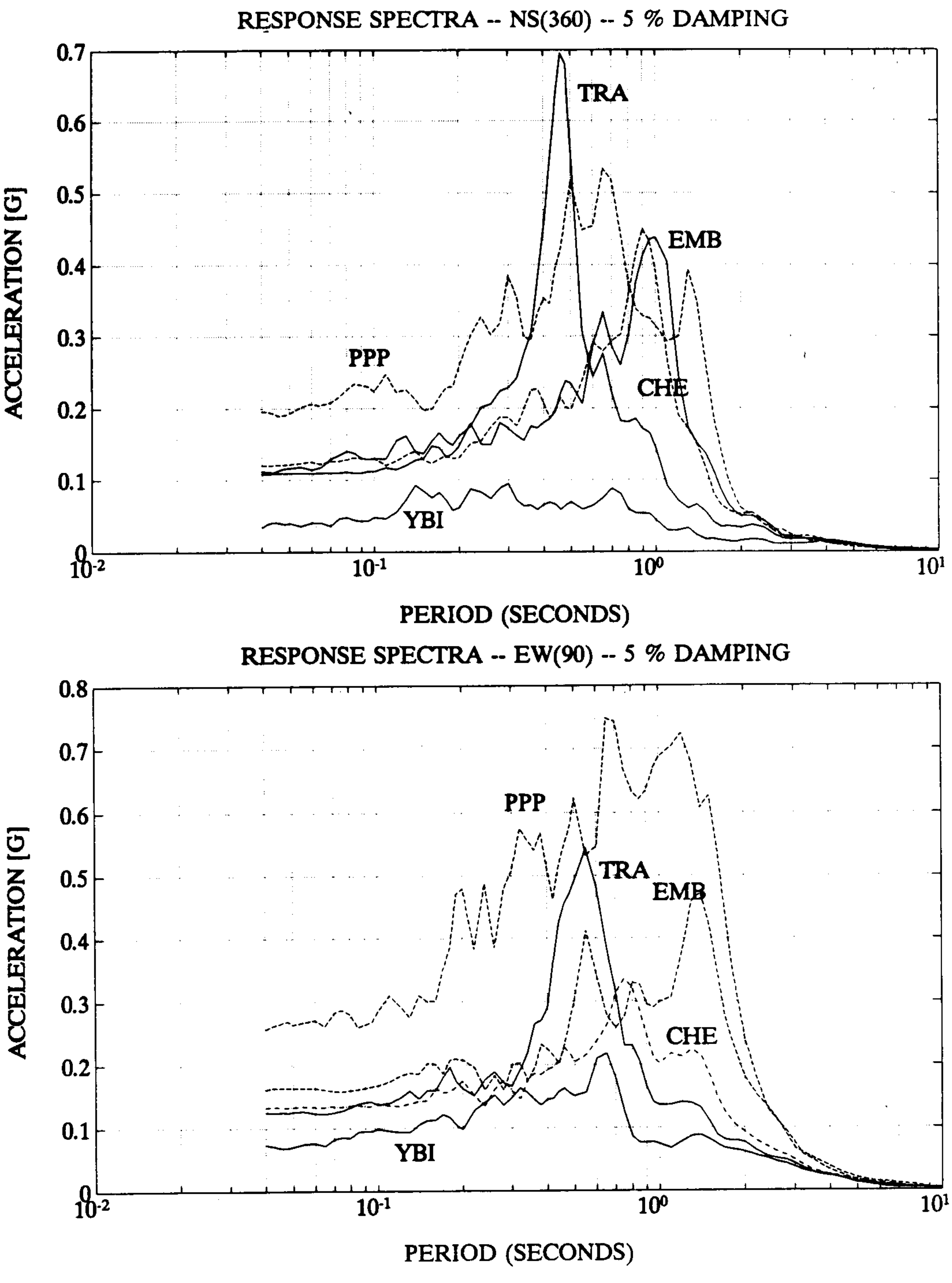


Figure 2.—Response spectra of free-field Emeryville site of Pacific Park Plaza (PPP), Transamerica Building (TRA), Embarcadero Building (EMB), and Chevron Building (CHE) compared to response spectrum of rock site Yerba Buena Island (YBI).

Yerba Buena Island, a rock site, also located at approximately 100 km from the epicenter. While the largest peak acceleration at Yerba Buena Island was 0.06 g, the amplified peak accelerations at the soft soil sites of some of these tall buildings varied between 0.12 and 0.26 g. The response spectra depict the degree of amplification of peak acceleration, represented by zero-period accelerations as well as the frequency (period)-dependent spectral acceleration. Particularly between the periods 0.1 and 2 seconds, which is of engineering interest, the spectral amplification ratio is as high as 5 or 6.

The purpose of this paper is to review and summarize studies of instrumented buildings that recorded the Loma Prieta earthquake. Since the earthquake, almost all of the recorded building response data has been made available by CSMIP and USGS, and a significant number of studies of the data set have been completed. Damage surveys or subsequent related studies of buildings are not within the scope of this paper. For further information of such studies, the readers are referred to the numerous reports published since the earthquake (see Benuska, 1990). Although every effort was made to include summaries of all the studies herein, it is likely that some were not in wide circulation and thus were not available.

Also of interest to the engineering and scientific community is the low-amplitude (ambient) vibration testing of five of the buildings in table 1 (Marshall and others, 1991, 1992; Çelebi and others, 1991, 1993; Çelebi, 1996). The results are summarized later in this paper.

METHODS OF ANALYSES

In studying the recorded responses of buildings and other structures, several methods have been used, including spectral techniques, system identification methods, and finite element modeling and analyses. The spectral analyses are based on Fourier amplitude spectra, autospectra S_x and S_y, cross-spectral amplitudes S_{xy}, and coherence functions (γ) and associated phase angles using the equation from Bendat and Piersol (1980):

$$\gamma^2_{xy}(f) = S^2_{xy}(f) / S_x(f) S_y(f). \tag{1}$$

The procedures used in system identification analyses estimate a model based on observed input-output data (Ljung, 1987). Simply stated, the input is the basement or ground-floor motion and the output is the roof-level motion or one of the levels where the structural response is detectable. In most of the system identification analyses presented in this paper (for example, Çelebi, 1996), the ARX (acronym meaning AR for autoregressive and X for extra input) model based on the least-squares method for single input-single output (Ljung, 1987) coded in commercially available system identification software was used (The MathWorks, 1988).

The damping ratios are extracted by system identification analyses in accordance with the procedures outlined by Ghanem and Shinozuka (1995) and Shinozuka and Ghanem (1995). These procedures are based on the following equations:

$$\xi = \frac{\delta_j}{\sqrt{\lambda_j^2 + \delta_j^2}} \tag{2}$$

$$\omega_j = \frac{\sqrt{\lambda_j^2 + \delta_j^2}}{\Delta t} \tag{3}$$

where

$$\delta_j = -\ln\left|z_j\right|^2 \tag{4}$$

$$\lambda = arg\left|z_j\right| \tag{5}$$

which are readily calculable from results of system identification routines. In this case, z_j is the j-th pole of S(z) which represents the system of linear equations defining the model accepted to represent observations. The poles, z_j, are the positive imaginary part represented by the equation:

$$z_j = e^{\left(-\xi w_j + w_j\sqrt{1-\xi^2}\right)\Delta t} \tag{6}$$

The low-amplitude (ambient) vibration data was analyzed by conventional spectral analysis techniques. System identification techniques were not applied to the ambient vibration data because of the unknown system input characteristics. Furthermore, because of the higher noise-to-signal ratio, system identification techniques do not minimize the errors simply and solutions appear unreliable.

RESPONSE OF REINFORCED CONCRETE BUILDINGS

PACIFIC PARK PLAZA (EMERYVILLE)

The set of records from Pacific Park Plaza is possibly the most studied building response data during this earthquake. The building was constructed in 1983 and instrumented in 1985, is 30 stories, and is the tallest reinforced concrete building in northern California. A general view, a plan view, a three-dimensional schematic, and its instrumentation are shown in figure 3 (Çelebi, 1992, 1996). Twenty-one channels of synchronized uniaxial accelerometers are deployed throughout this structure, with an additional three channels of accelerometers located at the north free-field outside the building. All are connected to central recording systems. In addition, a triaxial strong-motion accelerograph is deployed at a free-field site on the south side of the building (SFF or EMV[1]).

The building is an equally spaced three-winged, cast-in-place, ductile, moment-resistant reinforced concrete framed structure. The foundation is a 5-foot-thick concrete mat supported by 828 (14-inch-square) prestressed concrete friction piles, each 20-25 m in length, in a primarily soft-soil environment, with an average shear-wave velocity between 250 and 300 m/s and a depth of approximately 150 m to harder soil. The building had considerably amplified input motions but was not damaged during the earthquake. The east-west components of acceleration recorded at the roof and the ground floor of the structure and at the associated free-field station (SFF in fig. 3*B*), all at approximately 100 km from the epicenter, are shown in figure 4*A*. The motion at Yerba Buena Island (YBI), the closest rock site, had a peak acceleration of 0.06 g and is shown in figure 4 to indicate the level of amplified shaking at the free-field site and at the ground floor of the building. The corresponding response spectra are shown in figure 4*B*. For all three components of acceleration, the calculated response spectra of the south free-field site (SFF or EMV) of Pacific Park Plaza and Yerba Buena Island are compared in figure 4*C* (Çelebi, 1992). These response spectra show that the motions at EMV were amplified by as much as five times when compared with YBI. This is also inferred by the amplitude of the peak accelerations (0.26 g for EMV and 0.06 g for YBI). Furthermore, the differences in peak acceleration at the free-field station (0.26 g) and that at the ground floor of the building (0.21 g) (fig. 4*A*) suggest the possibility of significant soil-structure interaction.

The building has been studied in detail by Çelebi and Şafak (1992), Şafak and Çelebi (1992), Anderson and Bertero (1994), Anderson and others (1991), Bertero and others (1992), Kagawa and others (1993), Kagawa and Al-Khatib (1993), Aktan and others (1992), and Kambhatla and others (1992). The predominant response modes of the building and the associated frequencies (periods) [0.38 Hz (2.63 s), 0.95 Hz (1.05 s), and 1.95 Hz (0.51 s)] are identified by all these investigators using different methods, including spectral analyses, system identification techniques, and mathematical models. These three modes of the building are torsionally-translationally coupled (Çelebi, 1996). The frequencies are clearly identified in the cross-spectra (Sxy) of the orthogonal records obtained from the roof and ground floor (fig. 5*A*, *B*), the south free-field site (SFF) (fig. 5*C*), and the normalized cross-spectra of the orthogonal records (fig. 5*D*). A site

Figure 3.—*A*, Pacific Park Plaza. *B*, Plan layout and three-dimensional schematic of Pacific Park Plaza showing dimensions and strong-motion instrumentation (Çelebi, 1992, 1996).

[1] In most studies, the site of the south free-field (SFF) is referred to as the Emeryville site (EMV). The data from this site is one of the most-used ground motion records from the Loma Prieta strong-motion data set.

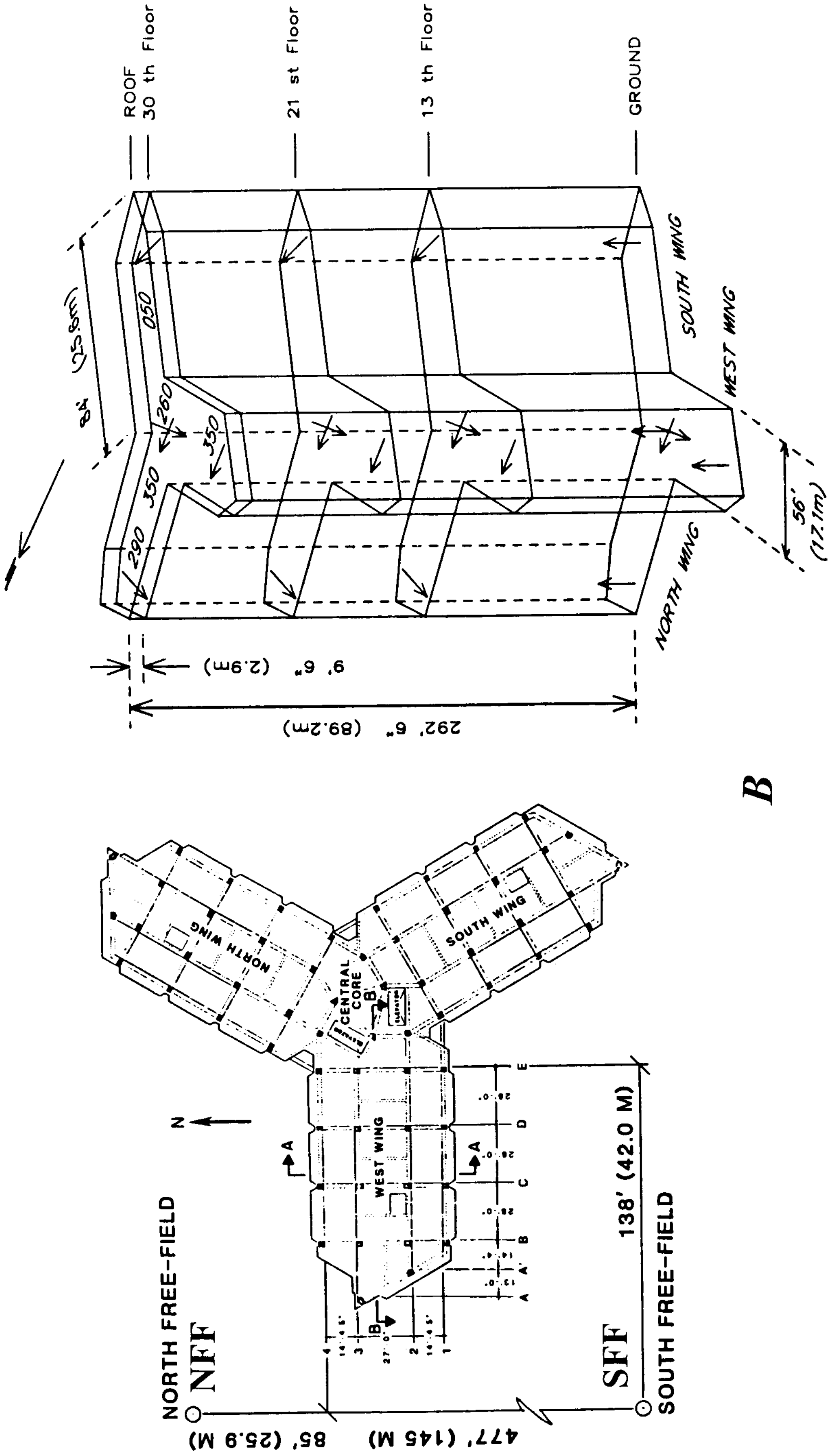


Figure 3.—Continued.

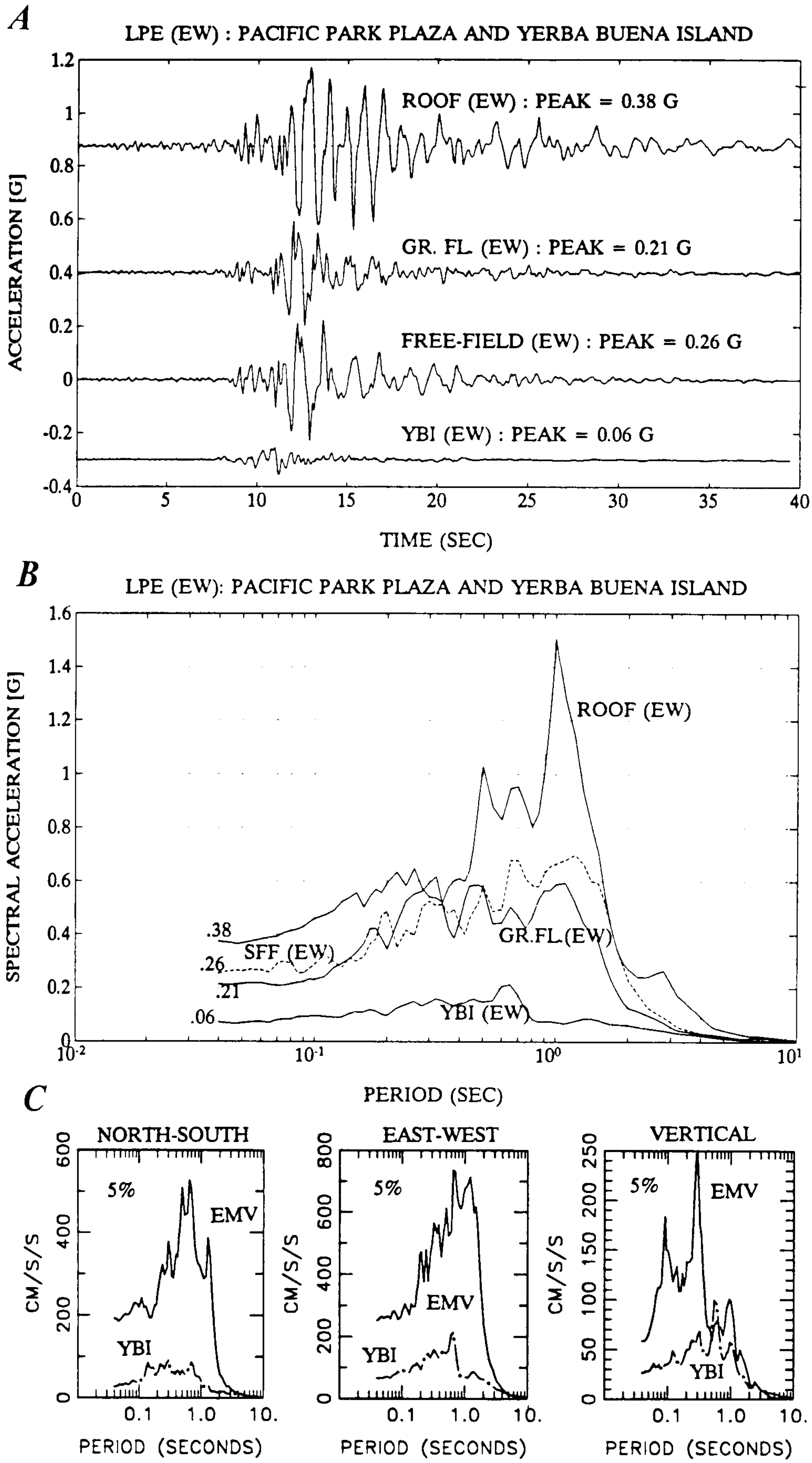


Figure 4.—*A*, Loma Prieta earthquake (LPE) east-west components of acceleration recorded at roof, ground floor, and south free-field (SFF) station of Pacific Park Plaza. Also shown is east-west component of acceleration at Yerba Buena Island (YBI). *B*, Response spectra of motions. *C*, response spectra of all three components of acceleration at south free-field (EMV) compared with YBI.

frequency at 0.7 Hz (1.43 s) is also identified. The peak at 0.7 Hz that appears in the cross-spectrum of the roof (fig. 5*A*) appears as the dominant peak in the cross-spectra of the ground floor and the south free-field (SFF) (figs. 5*B*, *C*). When the normalized cross-spectra are calculated for the ground floor and free-field, the site frequency at 0.7 Hz is distinguishable from the structural frequencies in the normalized cross-spectrum of the roof (fig. 5D). The 0.7-Hz site frequency is further confirmed by the transfer function (fig. 6) calculated by using Haskell's shear-wave propagation method (Haskell, 1953, 1960) and site characterization information by Gibbs and others (1994). The figure shows the transfer function and the variation of shear-wave velocities with depth. The depth to bedrock has been adopted from a map by Hensolt and Brabb (1990) as 150 m (~500 ft).

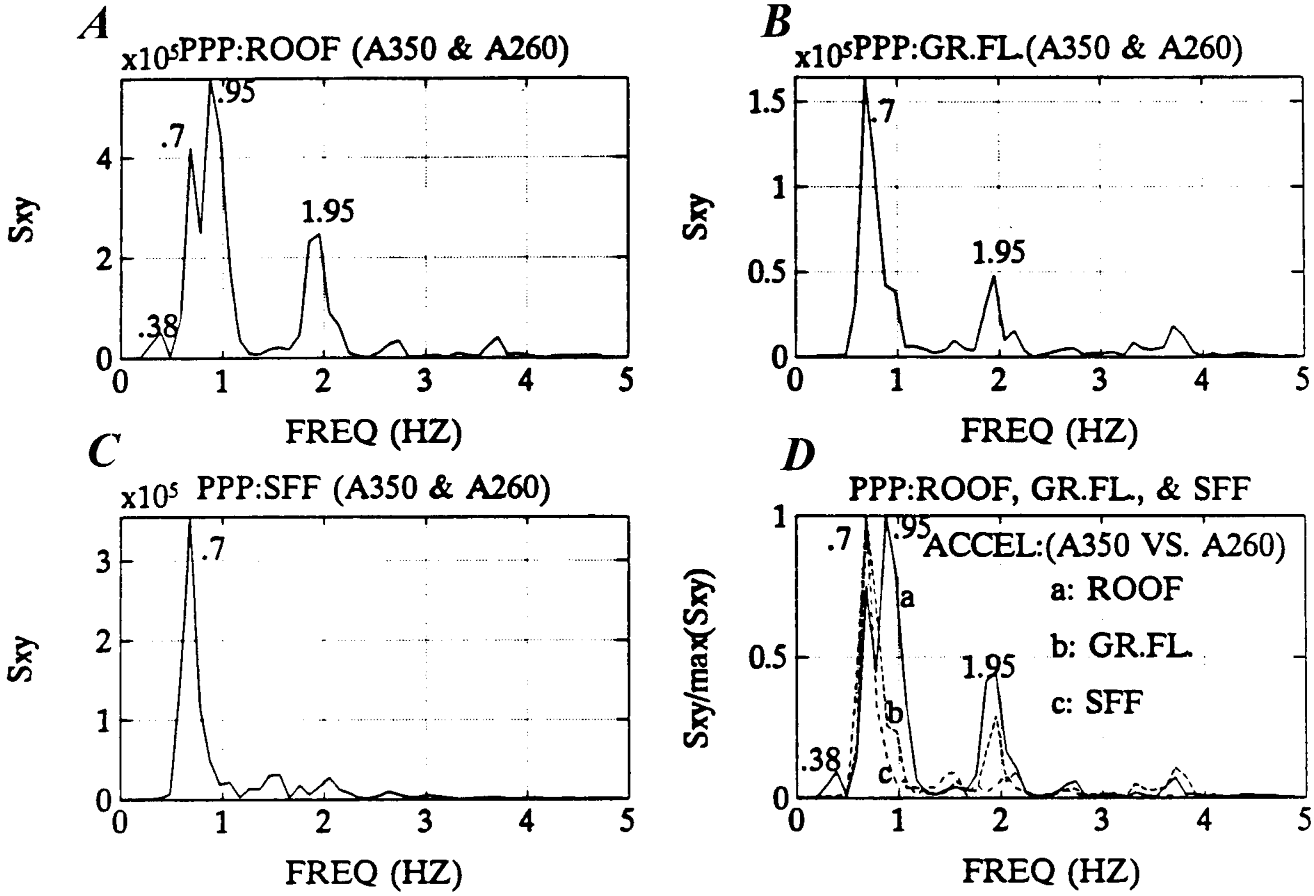


Figure 5.—Cross-spectra of accelerations at (*A*) roof, (*B*) ground floor, and (*C*) south free-field (SFF). Normalized cross spectra (*D*) show site frequency 0.7 Hz at ground floor and SFF but not at roof.

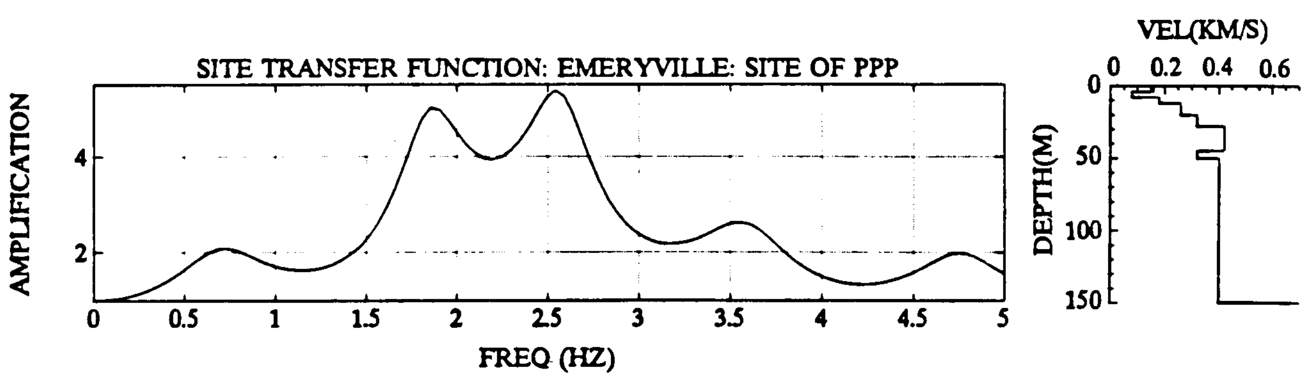


Figure 6.—Site transfer function of Pacific Park Plaza indicates first peak at 0.7 Hz.

Figure 7 shows the results of applying the system identification technique. The match between the observed and calculated response is excellent, as evidenced by comparison of the calculated and observed responses at the top floor and by comparison of the amplitude spectra of these responses. The damping ratios extracted from the system identification analyses corresponding to the 0.38-Hz first-mode frequency are 11.6 percent (north-south)

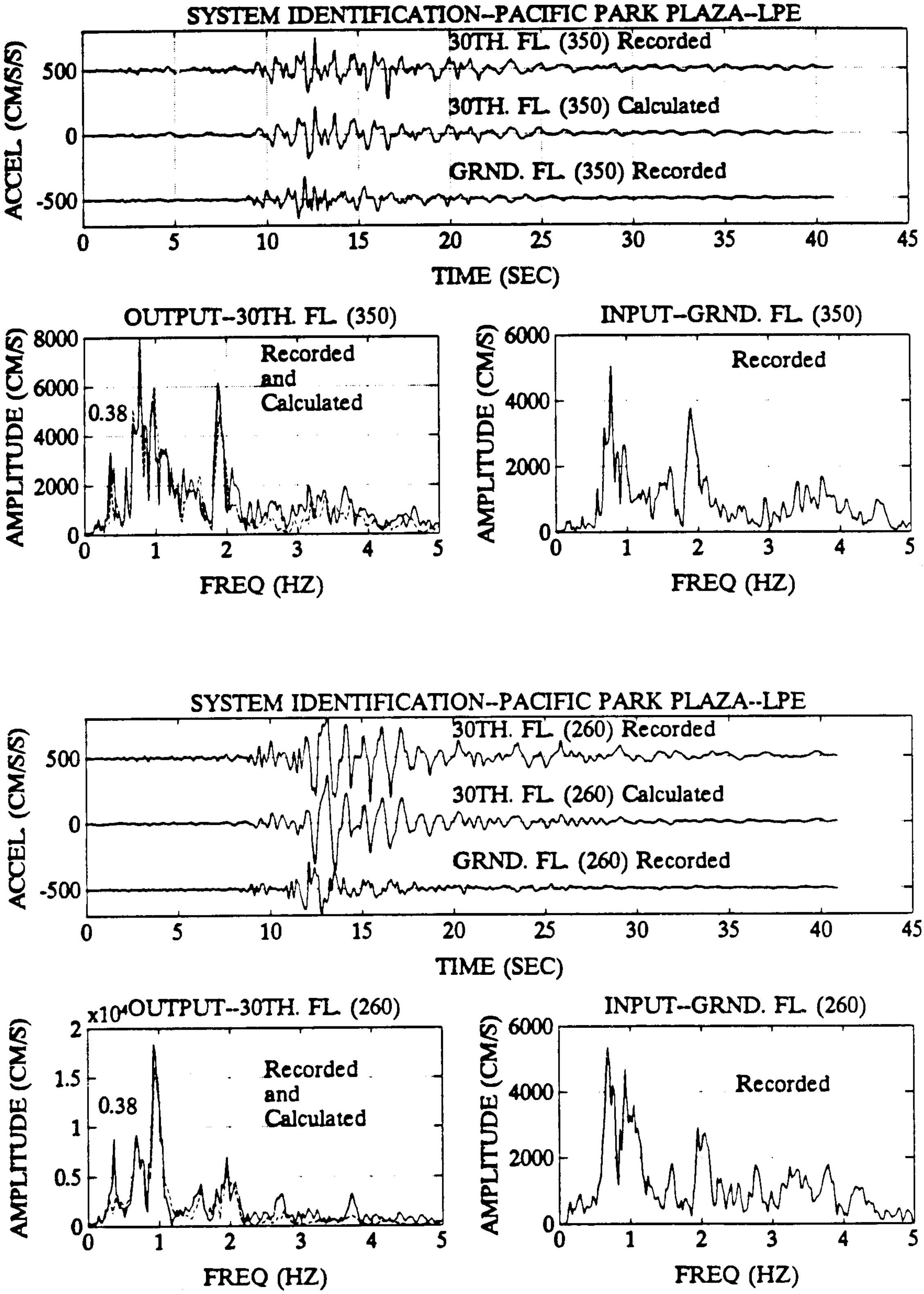


Figure 7.—System identification applied with accelerations recorded at roof and ground floor of Pacific Park Plaza.

Table 2.—*Summary of dynamic characteristics for Pacific Park Plaza*

[- -, not available]

		Frequencies (Hz)			Damping (percent)		
		MODE			MODE		
		1	2	3	1	2	3
1990 AMBIENT TESTS (from Çelebi, Phan, and Marshall, 1993)							
N-S		0.48	---	---	0.6	---	---
E-W		0.48	---	---	3.4	---	---
1989 (LPE) STRONG-MOTION TESTS (from Çelebi, Phan, and Marshall, 1993)							
N-S		0.38	0.95	1.95	11.6	---	---
E-W		0.38	0.95	1.95	15.5	---	---
1985 FORCED VIBRATION TESTS (from Stephen and others, 1985)							
N-S		0.590	1.660	3.09	1.7	1.3	2.9
E-W		0.595	1.675	3.12	1.8	1.9	3.2
Torsion		0.565	1.700	3.16	1.5	1.32	1.7
1985 AMBIENT VIBRATION TESTS (from Stephen and others, 1985)							
N-S		0.586	1.685	3.149	2.6	1.8	0.8
E-W		0.586	1.685	3.125	2.6	1.2	0.4
Torsion		0.586	1.709	3.125	3.8	1.4	1.0
MODAL ANALYSES [rigid (R) and flexible (F) foundation] (from Stephen and others, 1985)							
N-S	R	0.596	1.666	3.115			
	F	0.595	1.650	3.081			
E-W	R	0.596	1.666	3.115			
	F	0.595	1.650	3.081			
Torsion	R	0.565	1.711	3.275			
	F	0.562	1.686	3.220			

and 15.5 percent (east-west) (Çelebi, 1996). Such unusually high damping ratios attributed to a conventionally designed/constructed building require explanation. The building with its large mat foundation in a relatively soft geotechnical environment is capable of energy dissipation in the soil due to radiation (or foundation) or material damping. The subject of radiation damping for this building has been discussed in detail by Çelebi (1996).

The dynamic characteristics determined from Loma Prieta response records of Pacific Park Plaza as well as those determined from low-amplitude tests prior to (Stephen and others, 1985) and after the earthquake (Marshall and others, 1992; Çelebi and others, 1993) are summarized in table 2. These low-amplitude tests will be discussed later in this paper; however, it is important to note that there are significant differences in the dynamic characteristics of the building that were derived from the strong (Loma Prieta) shaking data and from the low-amplitude data. Also, it is noted in table 2 that although flexibility of the foundation was considered in the 1985 analyses, the structural frequency remained the same as the frequency determined with fixed base assumption. The differences in the frequencies for strong- and low-amplitude motions are attributed to soil-structure interaction (SSI), as studied from the records and mathematical modeling (Çelebi and Şafak, 1992; Şafak and Çelebi, 1992; Kagawa and others, 1993; Kagawa and Al-Khatib, 1993; Aktan and others, 1992; Kambhatla and others, 1992). A study of the building for dynamic-pile-group interaction by (Kagawa and Al-Khatib, 1993; Kagawa and others, 1993) indicates that there is significant interaction. The study shows that computed responses of the building using state-of-the-art techniques for dynamic-pile-group interaction compares well with the recorded responses.

Anderson and others (1991) and Anderson and Bertero (1994) compared the design criteria, code requirement, and the elastic and nonlinear dynamic response due to the earthquake. They also compared current U.S. and Japanese design procedures and requirements for this type of building and analyzed probable performance under more severe base motions. In order to achieve these objectives, linear elastic and nonlinear dynamic response analyses were conducted using both simplified and detailed analytical models. The results have been compared with Japanese design procedures. Contrary to others, the authors conclude that soil-structure interaction was insignificant for Pacific Park Plaza during the earthquake.

The response of the building was also found to be sensitive to the dominant orientation of the maximum energy of Loma Prieta ground motions. For this building, the orientation was similar to the rupture direction of the earthquake. A significant effect of the orientation of the ground motion for an unsymmetrical three-winged building such as Pacific Park Plaza was that it exhibited a disproportionate (as much as three times) response in one wing of the building compared to another, as shown in figure 8 (Çelebi, 1992). Therefore, the propagation direction of different waves (in most cases, surface waves) arriving at a building can be significant. As a general conclusion, because the energy of the ground motions can be azimuthally variable, structures with wings or unsymmetrical structures can be significantly affected by it.

SIX-STORY OFFICE BUILDING (SAN BRUNO)

A general view and the instrumentation scheme a six-story, reinforced concrete framed building in San Bruno (SBR) is shown in figure 9 (Çelebi, 1996). The building is rectangular in plan and has four moment-resistant frames in the exterior and one in the interior in the transverse direction (355°). Anderson and Bertero (this chapter) developed three-dimensional, linear elastic models of the building and studied its response under recorded base motions and code-prescribed lateral forces. They reported that under Loma Prieta motions the models confirmed limited inelastic behavior, as was observed in the building following the earthquake.

Phan and others (1994) also studied the building by using a mathematical model with a fixed base and with springs simulating soil-structure interaction effects. Their analyses showed that the frequency of the building determined from Loma Prieta response data (0.98 Hz, north-south, and 1.17, east-west) was approximately 69 percent of the frequency determined from ambient test data (1.41 Hz, north-south, and 1.72 Hz, east-west) (Marshall and others, 1991,1992; Çelebi and others, 1991, 1993, Çelebi, 1996). Furthermore, the frequency determined from ambient vibration data matches the analysis results of the model with a fixed base. On the other hand, the Loma Prieta frequency matched the results of the model with soil-structure interaction springs. They concluded that soil-structure interaction plays a significant role in the response of this building.

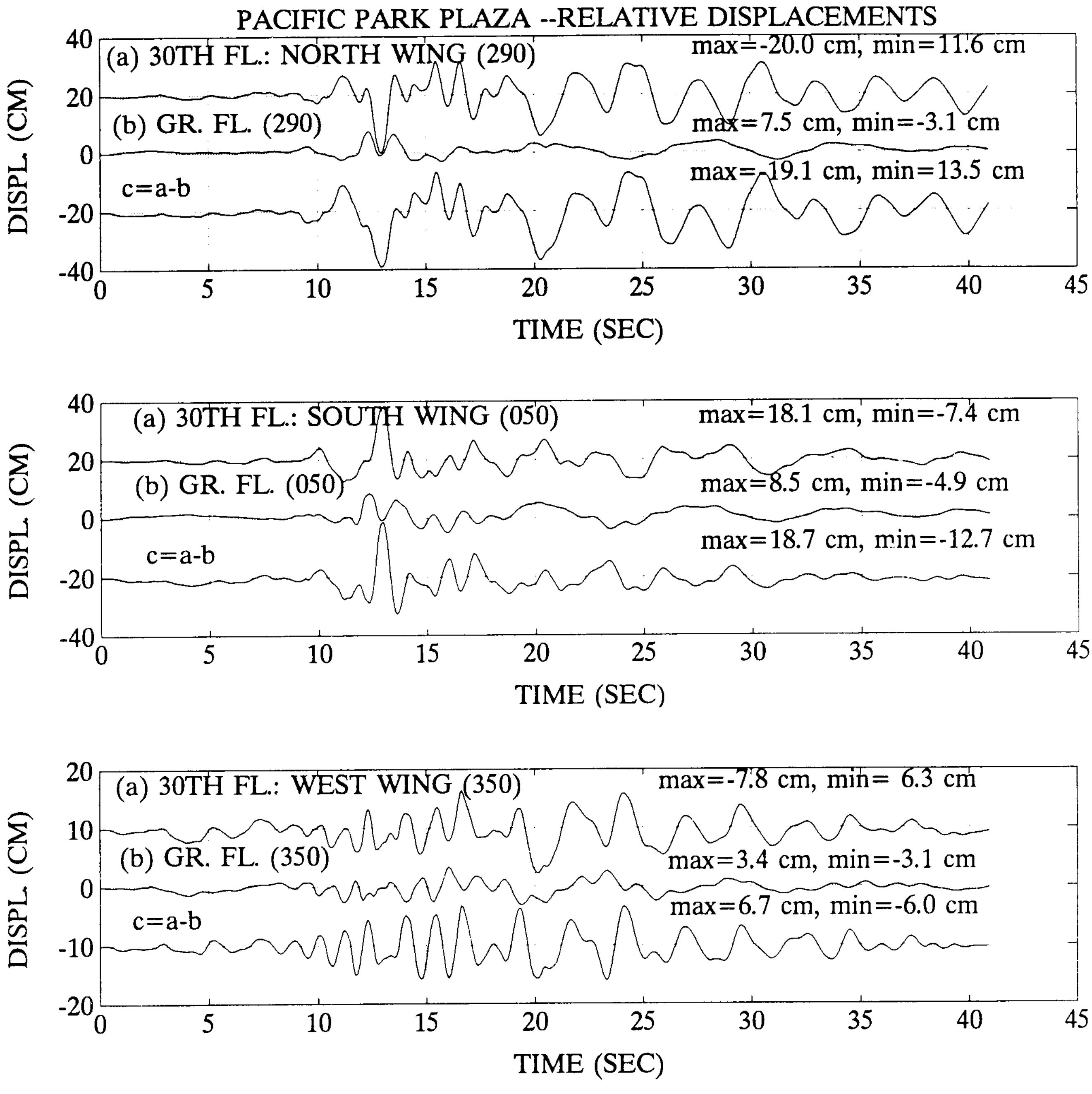
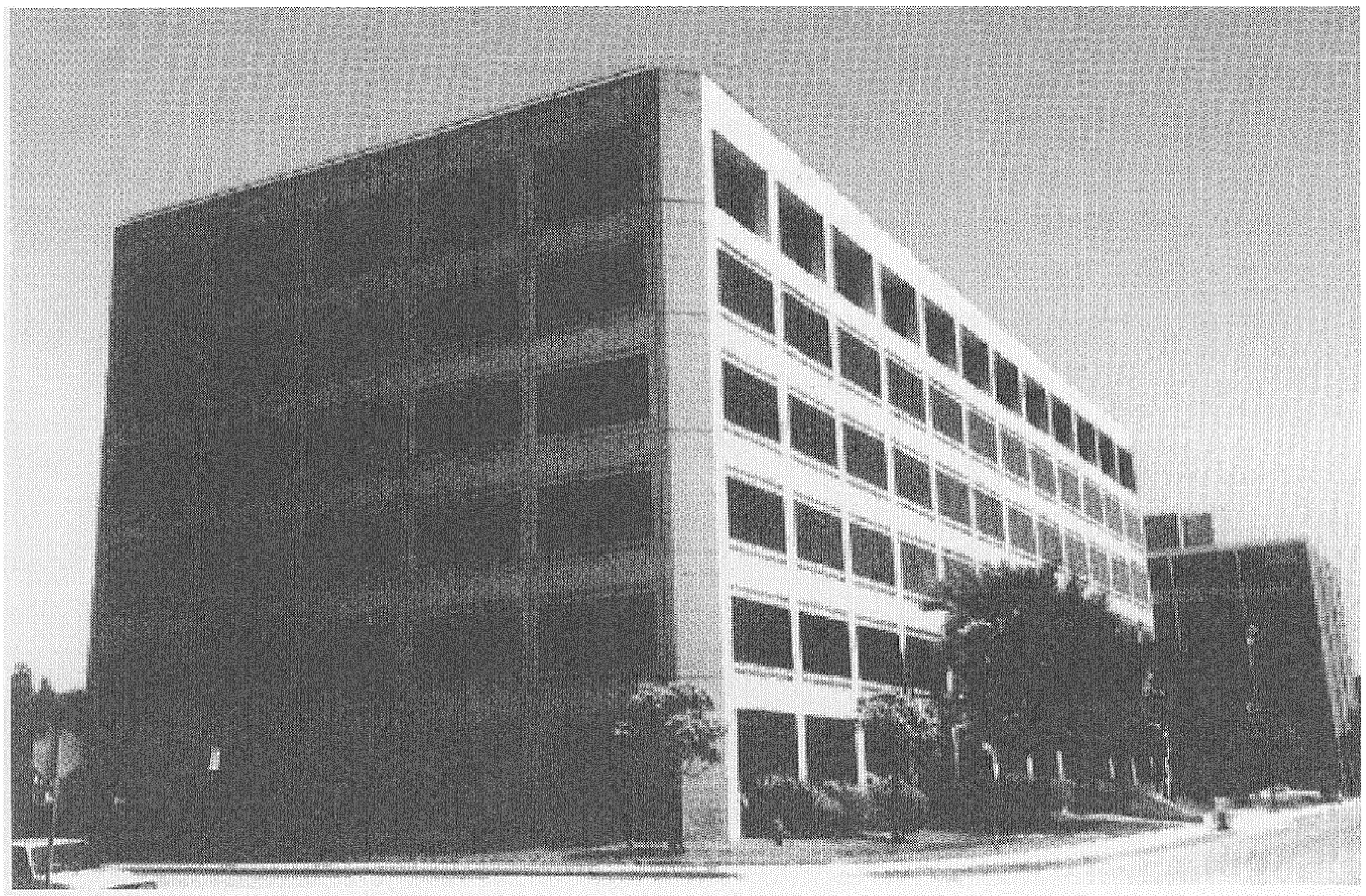
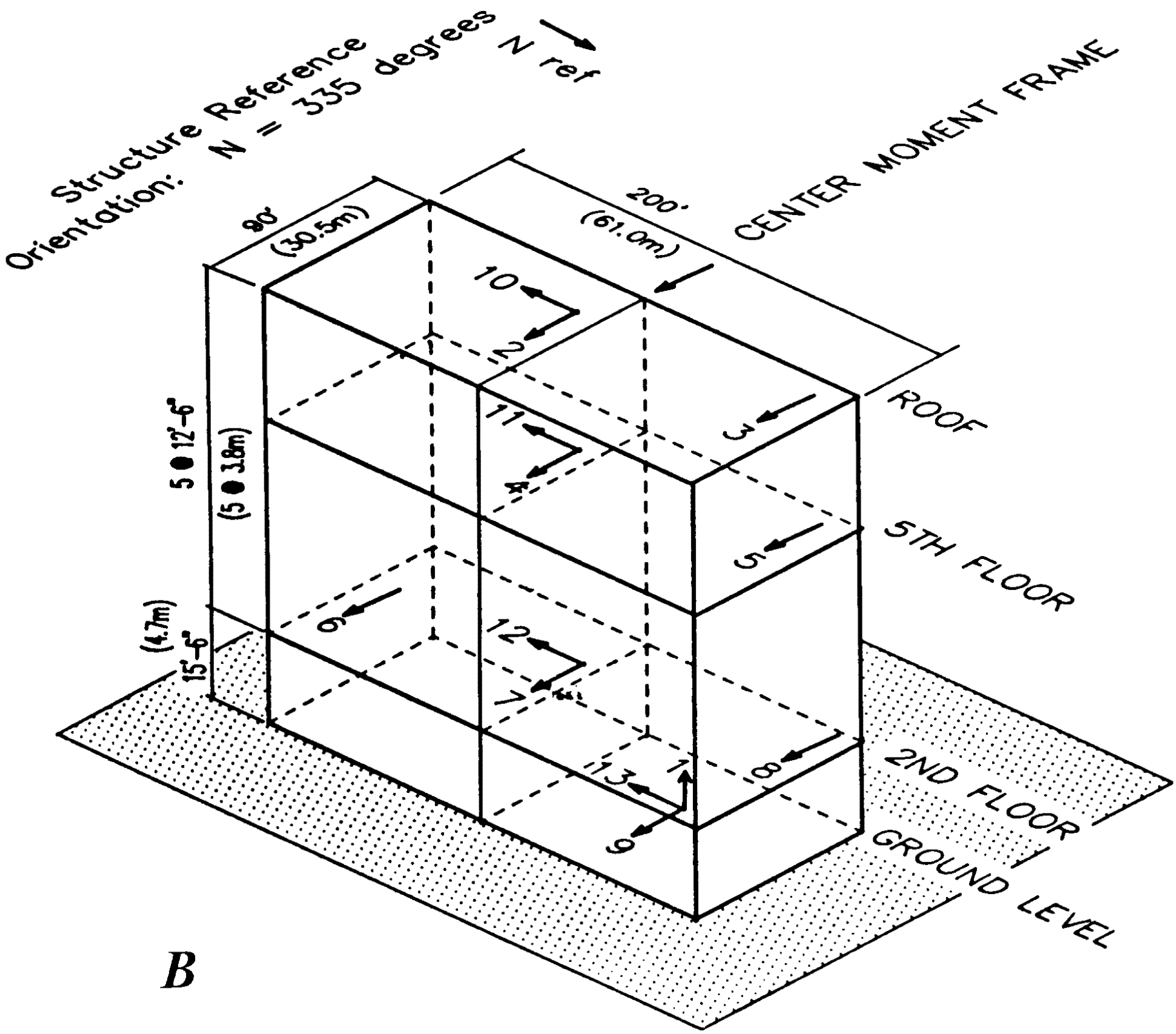


Figure 8.—Relative displacements at the wings of 30th floor of Pacific Park Plaza.

A





B

Figure 9.—*A*, Six-story commercial building in San Bruno. *B*, Three-dimensional schematic of San Bruno building.

RESPONSE OF STEEL STRUCTURES

TRANSAMERICA BUILDING (SAN FRANCISCO)

The response of one of the landmarks of San Francisco, the pyramidal Transamerica Building, 97 km from the epicenter of the earthquake, was recorded through an array of strong-motion instruments deployed by the USGS in 1985. The building was designed according to code requirements of that time; however, design evaluation was made using a site-specific design- response spectrum with seismic forces that were higher than the code requirements (R. Clough, personal commun., 1990). The building is 60 stories, 257.3 m (844 ft) high, and square in plan. At ground level, the plan dimensions are 53×53 m (174×174 ft). This plan starts reducing at the second floor to 44×44 m (145×145 ft) at the fifth floor and then follows an exterior wall slope of 1 to 11 upward. A perimeter truss system decorates and supports the building between the second and fifth floors. In addition to the exterior frame system, interior frames extend to the top of the structure, with some of them ending at the 17th and 45th floors. The exterior pre-cast concrete panels are attached structurally to the exterior frames. The basement (three levels below the ground level) consists of a very rigid shear wall box system. The foundation of the building consists of a 2.7-m (9 ft)-thick basemat without piles. The underlying soil media consists, in general, of clays and dense sands. Below the ground level to a depth of 8 m (25 ft), there is weak and compressible sand and rubble fill and recent bay deposits of sand and clay. Below 20 m (60 ft), the sands are partially cemented. The bedrock is between 48 and 60 m (145-185 ft) below the present street grade.

A general view, a three-dimensional schematic, overall dimensions, the instrumentation scheme, and recorded accelerations and displacements at some locations of the building are shown in figure 10. The instrumentation scheme was designed and implemented to study the response and associated dynamic characteristics of the building, including its translational, rocking, and torsional motions. There are a total of 22 channels.[2] Three triaxial strong-motion accelerographs with a total of nine channels are deployed synchronously with 13 uniaxial force-balance accelerometers, all connected to a central recorder with common-time recording capability. The three triaxial accelerographs are located on the 49th, 29th, and basement levels. At the 21st, 5th, and ground levels, three uniaxial accelerometers are deployed, two parallel to one another at the nominal west and east ends (nominal north-south orientation—actually 351° clockwise from true north) and the third with a nominal east-west orientation (081° clockwise from true north). These orientations are coincident with the orientations of the horizontal channels of the three triaxial accelerographs at the 49th, 29th, and basement levels. The remaining four uniaxial accelerometers are deployed in the basement; one each is positioned vertically at three corners of the building, and one is positioned horizontally and parallel to the nominal north-south horizontal channel of the triaxial accelerograph in the basement. The senses of the orientations of the channels are also shown in figure 10. The perpendicular distance between the two parallel vertical sensors at the basement level is 58.98 m (193.5 ft). In summary, there are parallel pairs of horizontal accelerometers in each of the 21st, 5th, ground, and basement levels and another single accelerometer deployed orthogonally to the pair in the horizontal direction at the same levels.

The response of the Transamerica Building has been studied in detail by Çelebi and Şafak (1991) and Şafak and Çelebi (1991). The peak accelerations and displacements derived from the processed data are summarized in table 3. The fundamental frequency (period) is 0.28 Hz (3.6 s) in both the north-south and east-west directions, as extracted from the spectral analyses and system-identification techniques. Other frequencies are 0.5, 1.2, 1.5, and 1.8 Hz for the east-west direction and 1.0, 1.35, 2.0, and 2.6 Hz for the north-south direction. Figure 11 shows the results of the application of the system-identification technique for the Transamerica Building records at the 49th floor as output and at the basemat as input (Çelebi, 1996). The match between the observed and calculated response is excellent, as evidenced by comparison of the calculated and observed responses at the 49th floor and by comparison of the amplitude spectra of these responses. The critical viscous damping ratios extracted from the system-identification analyses corresponding to the 0.28 Hz first mode frequency are 4.9 percent (north-south) and 2.2 percent (east-west) (Çelebi, 1996).

The analyses of the records showed that there is no significant torsional motion, as evidenced by the differences in the parallel accelerations and displacements on each floor. These relative displacements or the relative accelerations, as nominal torsional motions (and their corresponding Fourier spectra, not shown here), are negligible compared to those of the translational components.

The possibility of rocking was investigated using both the vertical motions recorded at the basemat and the horizontal motions recorded at the ground level and the basemat. Shown in figure 12 are the coherency, phase angle, and cross-spectrum plots for both north-south (351°) and east-west (081°) directions of pairs of horizontal acceleration on the 21st floor and vertical acceleration of

[2]It is noted herein that channels 11 and 12 of the central recording system did not function properly. However, the remaining records are sufficient to perform analyses of the response of this important building.

the basemat. It is observed from these that the rocking motion occurs at 2.0 Hz (or 0.5 s) in the north-south (351°) direction and at 1.8 Hz (or 0.56 s) in the east-west (081°) direction, since at these frequencies the horizontal motion at the 21st floor and the vertical motion in the basement are coherent and in phase. These frequencies are also observed in the Fourier amplitude spectra for the horizontal acceleration components (both directions) at the roof level,

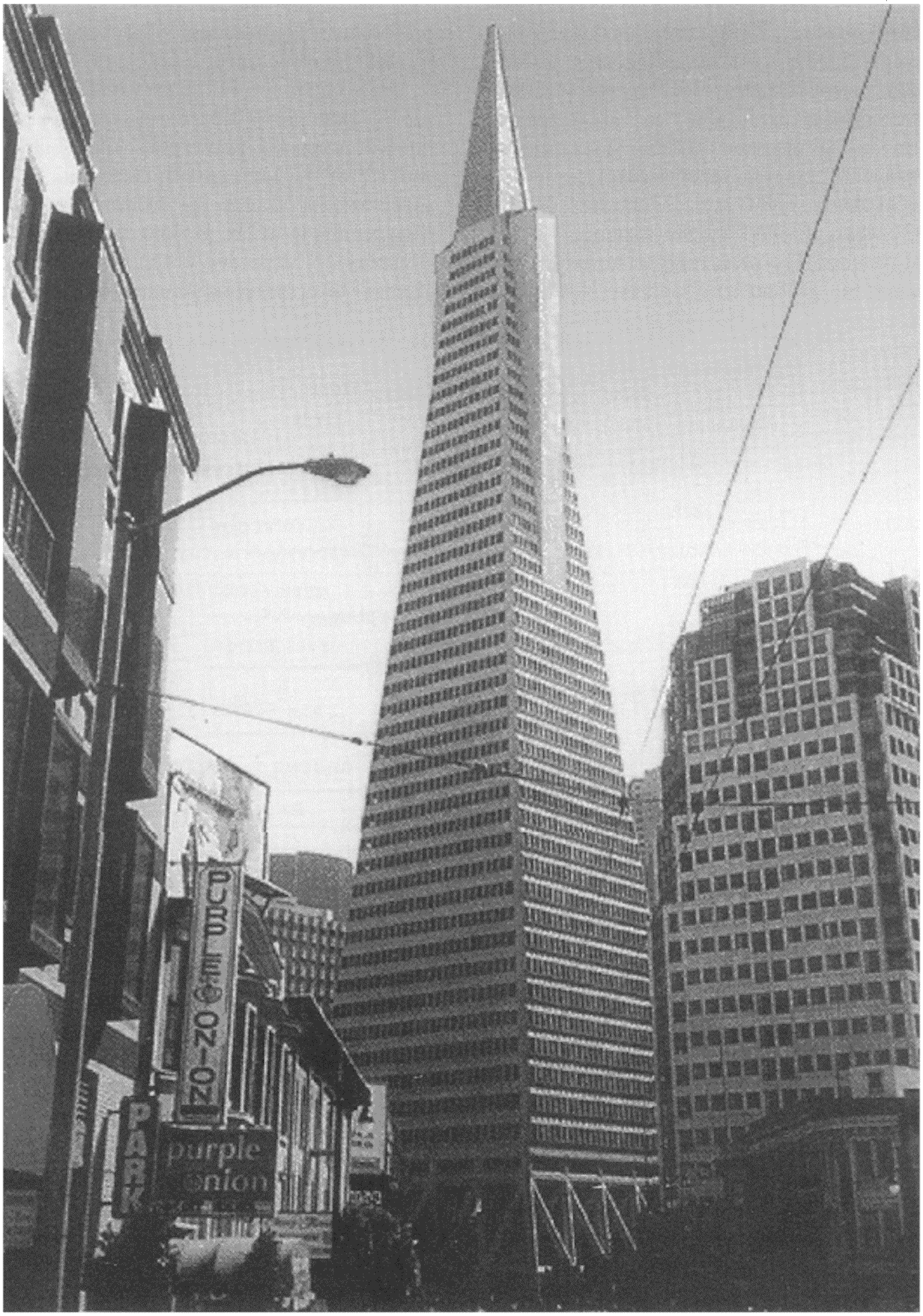
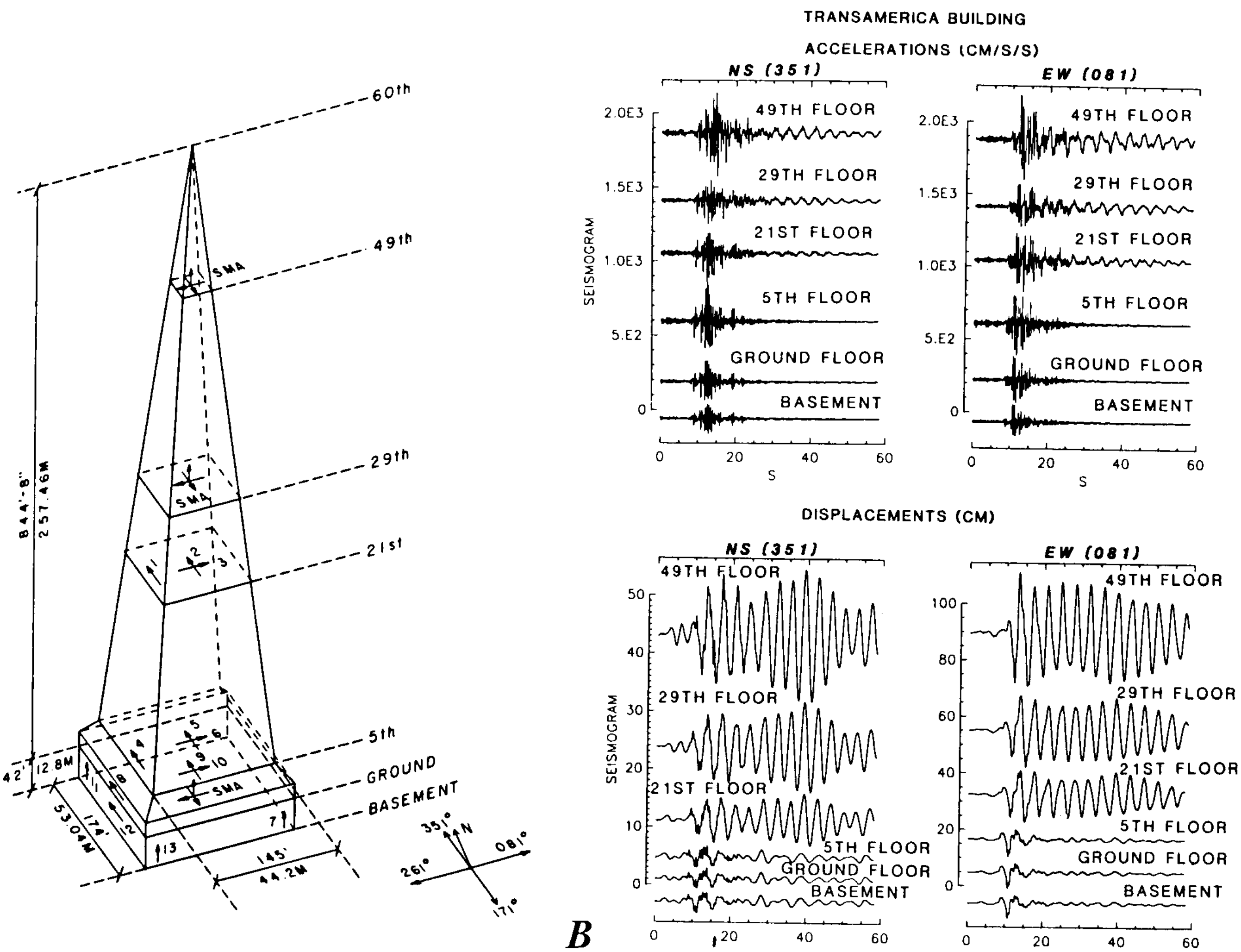


Figure 10.—A, Transamerica Building. B, Three-dimensional schematic of Transamerica Building and recorded accelerations and displacements (Çelebi, 1992, 1996).

as shown in figure 11. Figure 13 shows (*A*) the east-west component of acceleration at the 49th floor, (*B*) its amplitude spectrum, (*C*) the east-west displacement at the 49th floor, (*D*) the rocking contribution of acceleration at the 49th floor, (*E*) its amplitude spectrum clearly displaying the 2 Hz (0.5 s) rocking frequency (period), and (*F*) the rocking contribution of displacement at the 49th floor. It is noted that amplitudes of the rocking contribution (calculated by multiplying the rotation by the total distance between the basemat and the 49th floor) to the 49th floor displacement are very small.

The maximum vertical displacement due to rocking motion (the difference in the two vertical displacements at the two corners of the basemat) is 0.313 cm or 5.31×10^{-5} radians (0.003°) when divided by the distance between the two vertical sensors. The peak relative horizontal displacement between the ground level and the basement is 0.59 cm in the 081° (east-west) direction and 0.77 cm in the 351° (north-south) direction, which translates into 4.6×10^{-4} radians (or 0.026° of rotation around the 351° axis and 6.0×10^{-4} radians (or 0.034°) around the 081° axis. All of these peaks occur at approximately 11 s into the record. These rotations are shown in figure 14. The rotation of the wall around the north-south axis is approximately tenfold that of the rotation of the basemat around the same axis.

The comparison of the rotations around the north-south (351°) axis shows that there is a very significant difference between the peak rotations calculated from the difference of the vertical displacements at the basemat and those calculated from the difference of east-west direction displacements at the ground floor and the basemat. This disparity could be due to (1) the bending of the basemat, (2) the shear deformation and bending of the shear walls



B

Figure 10.—Continued.

Table 3.—*Peak accelerations and displacements for Transamerica Building*

[--, only one channel in this direction; +++, not placed in this direction; SMA, triaxial strong-motion accelerograph; FBA, force-balance accelerometer; CH, channel]

Floor	Acceleration				Displacement			
	081°	351°	351°	Up	081°	351°	351°	Up
	(g)	(g)	(g)	(g)	(cm)	(cm)	(cm)	(cm)
49 (SMA)	0.28	0.29	---	0.13	18.6	11.3	---	1.22
29 (SMA)	0.14	0.16	---	0.11	12.9	7.7	---	0.92
Base (SMA)	0.10	0.10	---	0.05	5.2	1.9	---	1.10
21 (FBA)	0.19	0.13	0.14	+++	8.5	4.4	4.8	+++
5 (FBA)	0.19	0.26	0.26	+++	3.5	2.0	2.3	+++
Ground (FBA)	0.17	0.14	0.16	+++	3.3	2.0	2.0	+++
Base (CH13)(FBA)	+++	+++	+++	0.04	+++	+++	+++	1.0
Base (CH 7)(FBA)	+++	+++	+++	0.07	+++	+++	+++	0.9

and columns in the three levels below the ground level, and (3) perhaps, the effect of the smaller stiffness of the embedment in the horizontal direction as compared to the vertical direction. Another possibility is the presence of integration errors introduced during processing of the digitized data. The large ratio of the wall-to-basemat rotation may be important in assessing forces used in the design of basements. A detailed study of this issue was carried out by Soydemir and Çelebi (1992). In usual practice, the basements are designed for seismic forces similar to that of the ground floor. No difference in the

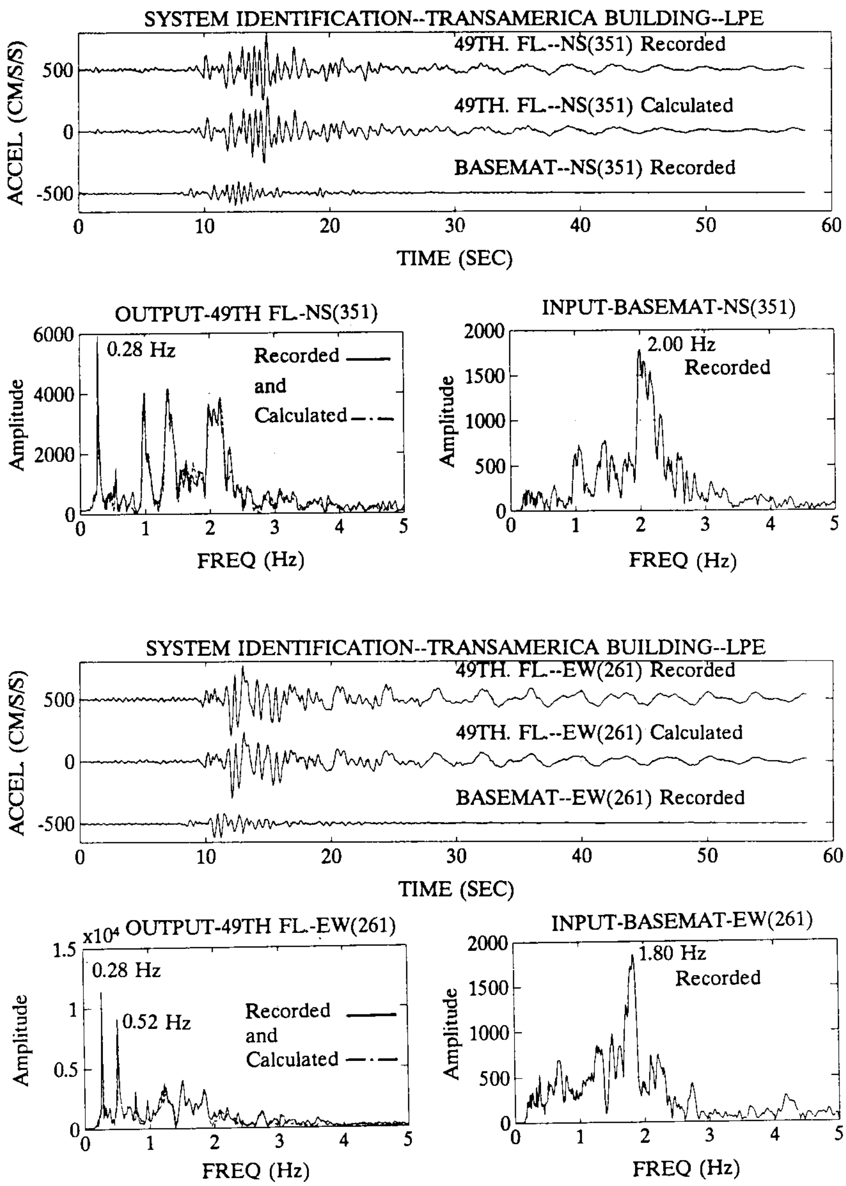


Figure 11.—System identification applied with accelerations recorded at 21st floor and basement of Transamerica Building.

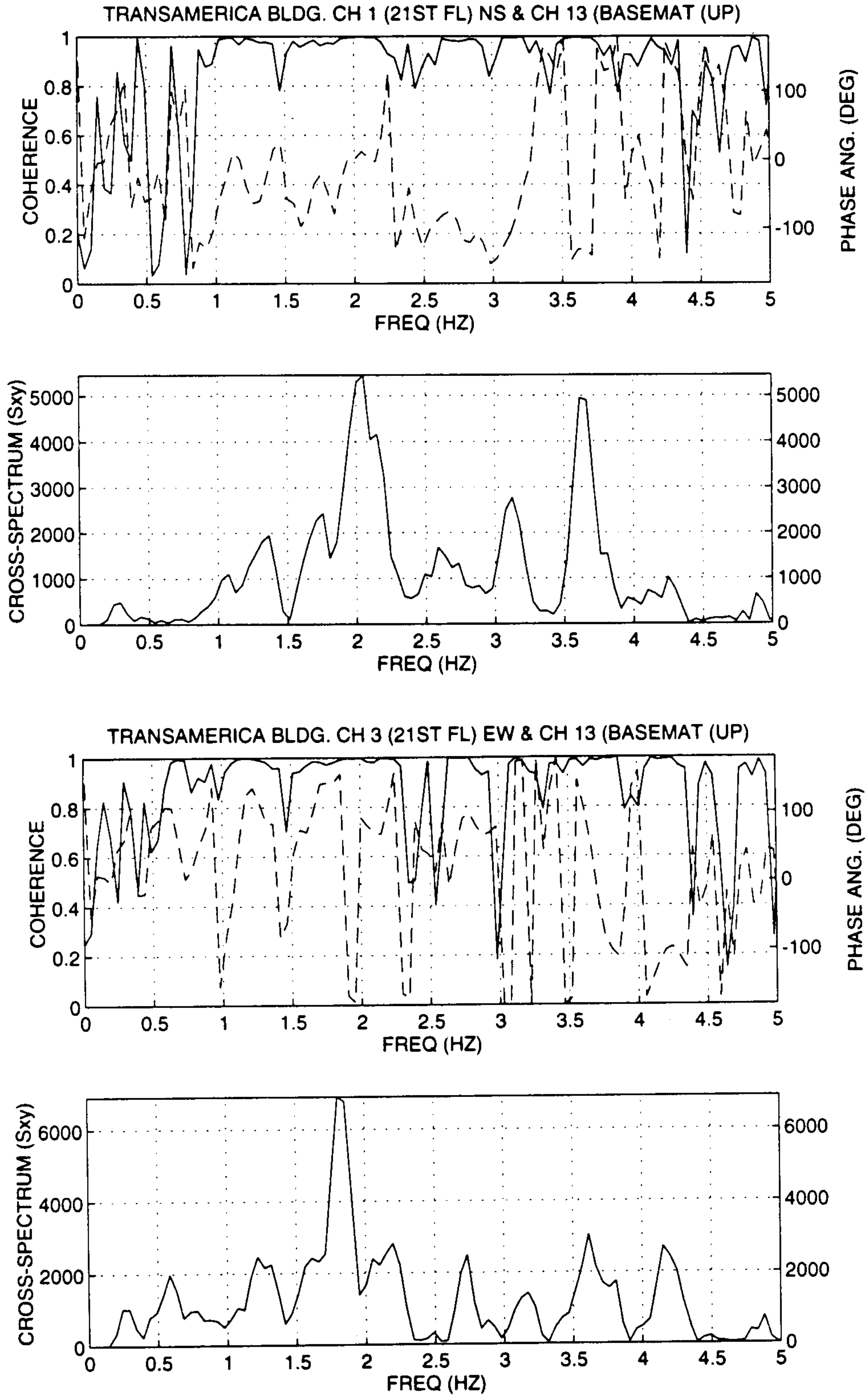


Figure 12.—Rocking investigated with cross-spectrum, coherency (solid line), and phase angle (dashed line) plots of horizontal motions at 21st floor and vertical motion at basemat of Transamerica Building.

seismic forces and/or accelerations are considered between the ground level and the embedded basement levels. However, the data set from the Transamerica Building shows that significant differences may occur between the motions at the ground floor and those at the basemat level of embedded basements and that the deformations of basemat and basement walls can also be significantly different. To compensate for this in design, a simplified approximate procedure has been developed by Soydemir and Çelebi (1992). A similar disparity in the deformation of the basemat and walls of the basements has been observed from the data of Embarcadero Building in San Francisco, discussed later in this paper.

Although very small in amplitude, the rocking motions significantly influenced the motions at the basement and the ground level. This is evidenced by the normalized response spectra (fig. 15) for the records from the three components of the triaxial accelerograph at the basement. Thus, this type of response may be pertinent to the incorporation of response spectra used in the design process of buildings. The design response spectra represents, in general, free-field motions assumed to be applicable at the foundation level of a structure; while it may include site effects, it should not include the effect of the vibration of the structure or soil-structure interaction. In this case, it was shown that the motions at the foundation (basemat) level are influenced by the soil-structure interaction effects.

Forced vibration tests and dynamic analyses were performed on the Transamerica Building in 1972-73 by Stephen and others (1974). Ambient vibration tests were performed by Kinemetrics (1979). The data from these investigations permit (1) the comparison of the actual earthquake response characteristics with those from small amplitude tests and (2) the assessment of the validity of various assumptions made in the dynamic analyses. The results of these small-amplitude tests and related analyses are summarized later in this paper. It is important to note that both the forced-vibration and ambient-vibration tests were performed when the construction of the building was just completed and the building was not yet occupied. Therefore, it did not contain nonstructural partitions and live load.

The dynamic analysis was performed with a mathematical model that considered only one-quarter of the building above the plaza (ground) level with appropriate boundary conditions. Translational and rotational responses were assumed to be uncoupled (Stephen and others, 1974). This assumption is valid for a symmetrical structure such as the Transamerica Building. Although the attachments of exterior panels were detailed with the intention to minimize their effect on the lateral stiffness of the building during small amplitudes of vibration, Stephen and others (1974) concluded that the panels contributed significantly to the lateral stiffness.

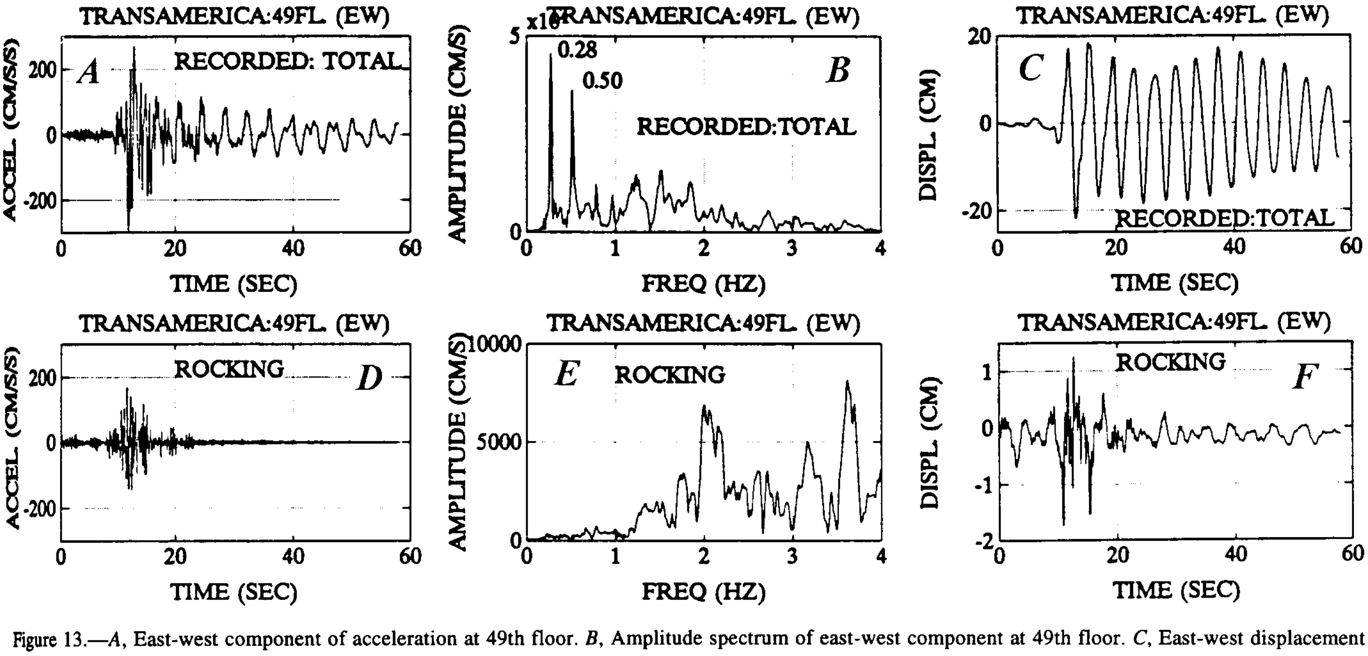


Figure 13.—*A*, East-west component of acceleration at 49th floor. *B*, Amplitude spectrum of east-west component at 49th floor. *C*, East-west displacement at 49th floor. *D*, Rocking contribution of acceleration at 49th floor. *E*, Amplitude spectrum of rocking contribution of acceleration at the 49th floor clearly displaying the 2 Hz (0.5 s) rocking frequency (period). *F*, Rocking contribution of displacement at 49th floor.

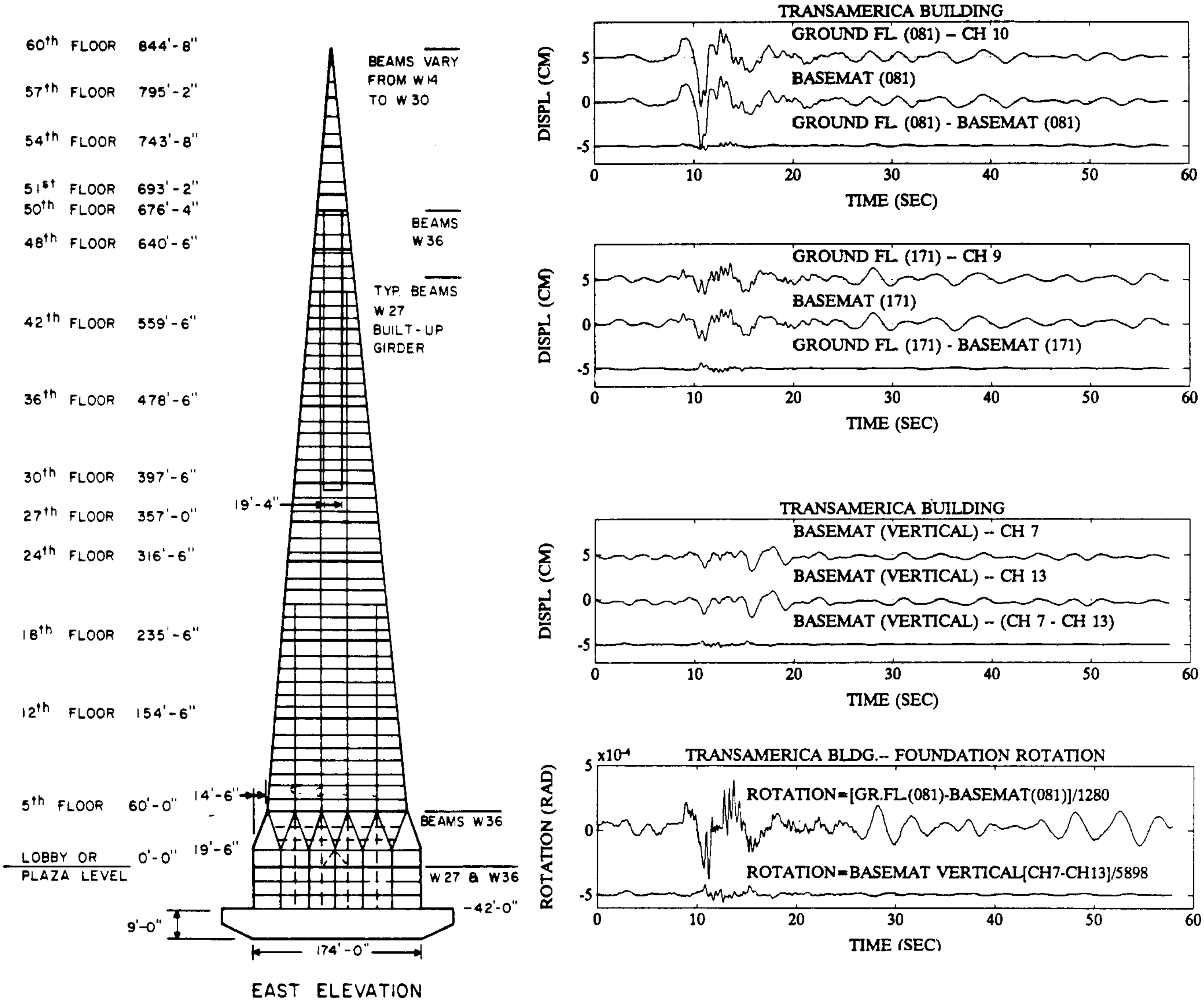


Figure 14.—Rotation of basemat and basement walls at Transamerica Building.

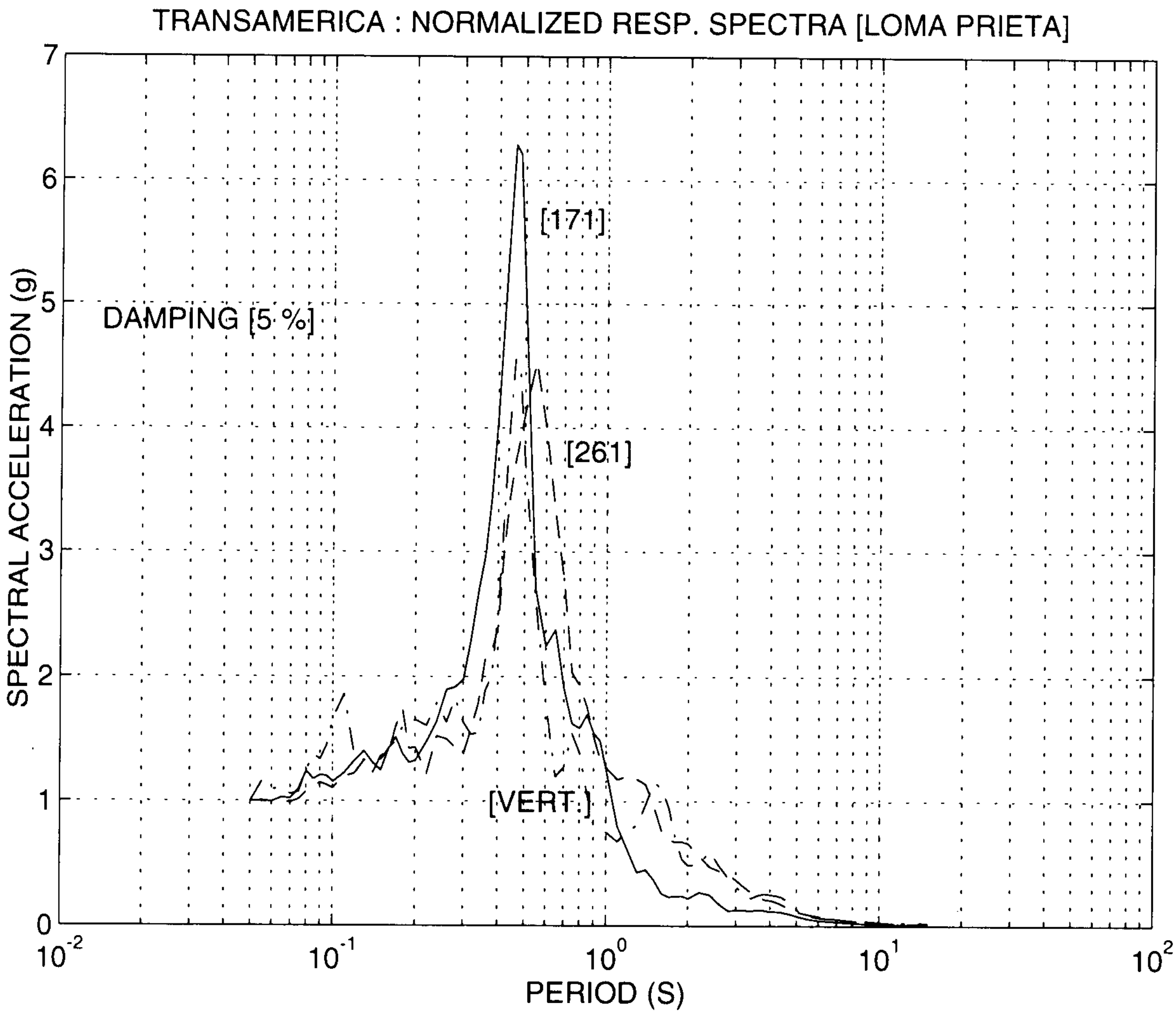


Figure 15.—Normalized response spectra for records from three components of the triaxial accelerograph at basement of Transamerica Building.

EMBARCADERO BUILDING (SAN FRANCISCO)

The 47-story Embarcadero Building (No. 4) is the first building in the United States constructed with eccentric bracing. The building, approximately 97 km from the epicenter of the earthquake, was constructed in 1979 in accordance with the UBC 1976 design provisions but using design response spectra relating to much higher seismic performance requirements.

The Embarcadero Building is 172 m (564 ft) high. The building actually consists of two structures; an above-grade 47-story building and an above-grade 3-story building. The below-grade 2-story shear walls and diaphragms are common to the two structures. The 47-story tower, referred to as the Embarcadero Building herein, is a moment-resisting steel-framed structure. Four center north-south frames are eccentrically braced (two up to the 41st floor and two up to the 29th floor). Plan dimensions decrease above the 39th and again above the 41st floors. Eccentric diagonal braces consist of double angles which are offset by approximately 1.5 m (4.5 ft) from the column-beam intersections. Recent codes (Uniform Building Code, 1991) term this type of structural framing as special moment-resisting frames. A general view of this building, the vertical sections showing the eccentric bracing and a typical plan view with significant dimensions, and a three-dimensional view of the building and instrumentation scheme are provided in figure 16 (Çelebi, 1993).

The Embarcadero Building is located in the Lower Market area of San Francisco, which is reclaimed fill area well known for its soft-soil characteristics that amplify ground motions originating at long distances. Due to these amplified motions, the Embarcadero Freeway, located within 100 m of the Embarcadero Building suffered extensive damage during the earthquake and was razed in 1991 (fig. 16*A*). The building was not damaged during the earthquake.

The building base is a 1.67-m (5 ft)-thick, reinforced concrete mat supported by approximately 50-67-m (150-200 ft)-long composite concrete and steel bearing piles. The underlying soil media consists of approximately 8.5 m of a top layer of silty fine sand fill with rubble (estimated shear wave velocity, V_s , 200 m/s, followed by 25 m of soft very dark greenish-gray Holocene silty clay (Bay mud, V_s 150 m/s), 9 m of sand (V_s , 250 m/s) and 21.5 m of very stiff to hard silty clay (old Bay mud, V_s 230 m/s). The rock at 64 m depth is sandstone with V_s of approximately 1,000 m/s (T. Fumal, personal commun., 1992). The transfer function, from rock to the surface, based on estimated shear-wave velocities of the layered media (provided by Fumal) and calculated by Haskell's formulations (Haskell, 1953, 1960; Silva, 1976) using software by C.S. Mueller (personal commun., 1992), is shown in figure 17. From this, a site frequency (period) of 0.8 Hz (1.25 s) is inferred. Alternatively, using the well-known formula $T_s = 4H/V_s$, a depth (H) of 64 m, and an average shear-wave velocity (V_s) of 200 m/s, an approximate site frequency (period) of 0.77 Hz (1.3 s) is obtained (Çelebi, 1993). The match between the calculated site periods is very good. The significance of the site period is discussed later in this section.

The site characteristics of any area are deemed very important. To further assess the impact of soft-soil characteristics on a densely built-up urban environment, such as the Lower Market area of San Francisco, the USGS has installed downhole accelerograph arrays within 40 m of the Embarcadero Building (between the building and the razed Embarcadero Freeway).

For the design of the building, site-specific design response spectra based on two levels of performance of the building were used. The first level of performance requires elastic response without structural or nonstructural damage under a moderate earthquake (M=7) that is likely to occur during the economic life of the building. The second level of performance demands that the structure will not collapse under the most severe (major) earthquake (M=8.3) that could occur during the economic life of the building. Substantial structural and nonstructural damage (without collapse) is considered acceptable under such an event. Design response spectra based on United States Nuclear Commission Regulatory Guide 1.60 (United States Nuclear Regulatory Commission, 1973) of earthquake level I (anchored at zero period acceleration, 0.3 g and 3 percent damping) and earthquake level II (zero period acceleration, 0.5 g and 7 percent damping) are provided in figure 18, which also shows the 1976 Uniform Building Code spectrum for comparison (Çelebi, 1993).

The strong-motion instrumentation scheme implemented in 1985 by the California Strong Motion Instrumentation Program of the California Division of Mines and Geology at six different levels of the building is shown in figure 16C (Shakal and others, 1989). There are 6 digital seismic accelerographs (DSA-1) with a total of 18 channels of syncronous, uniaxial (FBA-11) and biaxial (FBA-21) force-balance accelerometers within the structure. One of the unidirectional accelerometers is in the adjacent building basement. The reference north-south orientation is 345° clockwise from true north (fig. 16*C*).

The processed 120 seconds of the recorded acceleration and the calculated displacements at different levels are shown in figures 19 and 20. This data has been band-pass filtered with ramps at 0.07-0.14 Hz and 23-25 Hz (Shakal and others, 1989). Peak accelerations and displacements are summarized in table 4. In figures 19 and 20 and in table 4, it is noted that the north-south peak responses (accelerations and displacements) at the 39th floor are

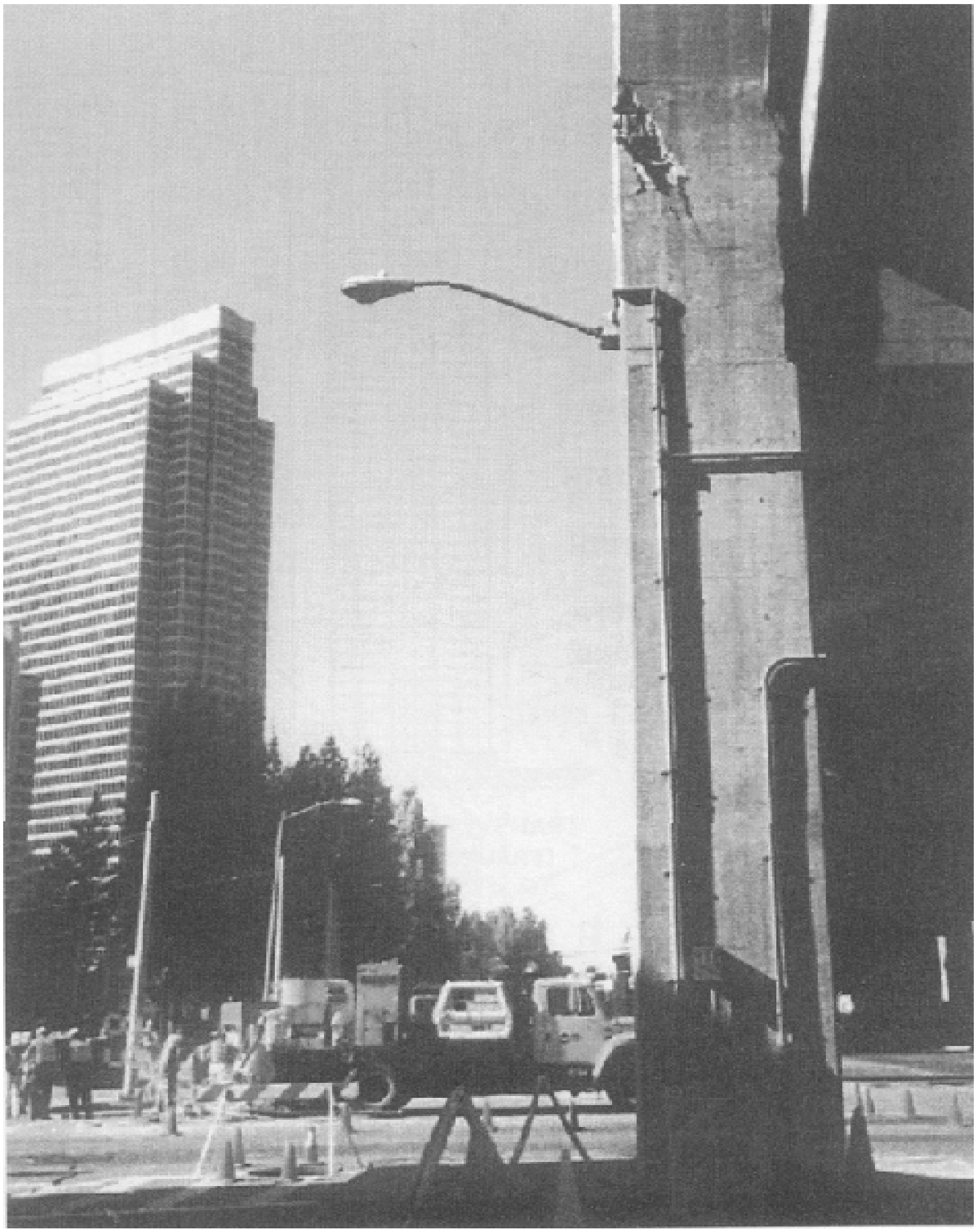


Figure 16.—A, Embarcardero Building. Also shown is one of the damaged columns of Embarcadero Freeway, razed in 1991. B, Typical plan view and vertical sections of Embarcadero Building. C, Three-dimensional view and instrumentation scheme of Embarcadero Building.

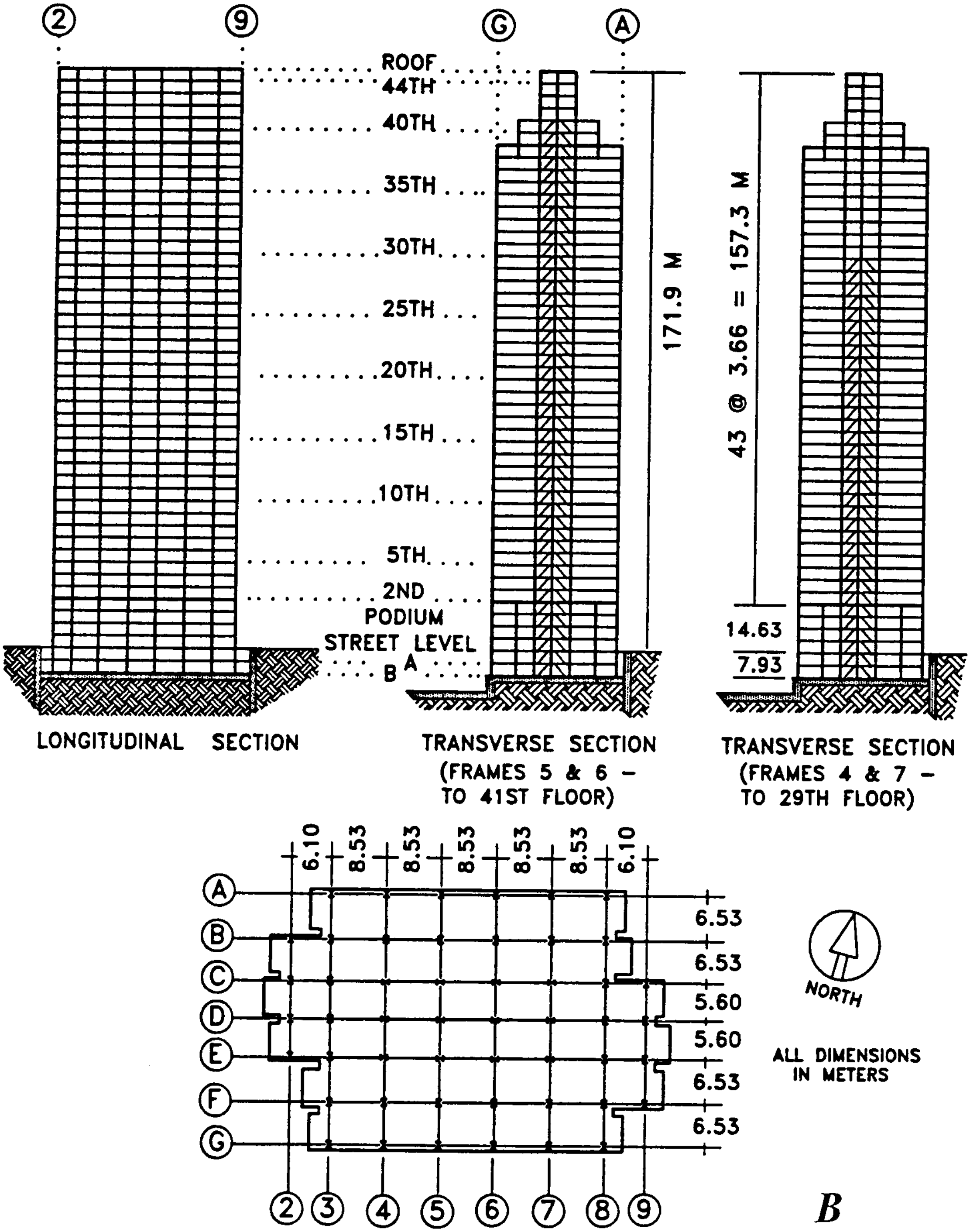
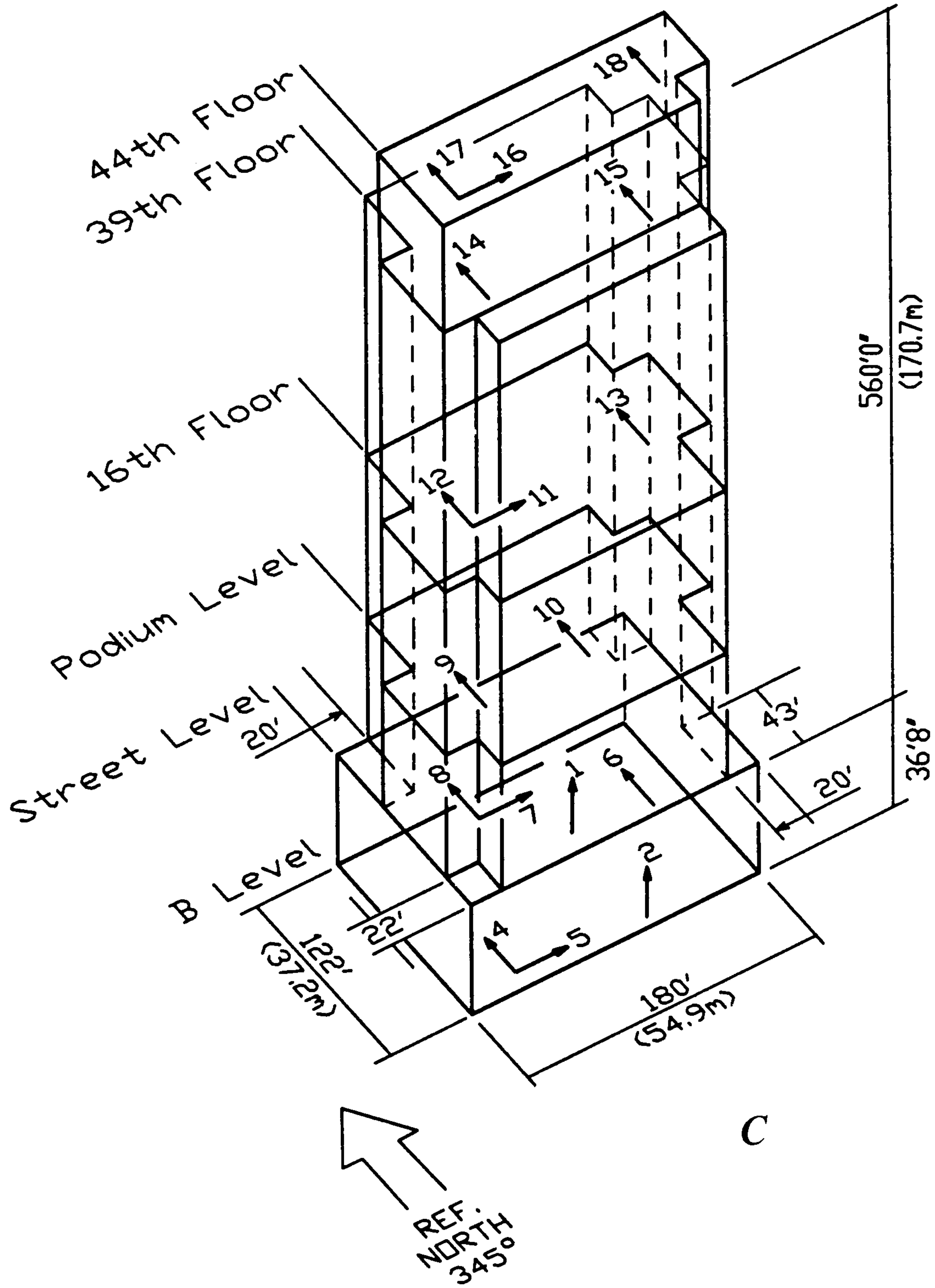


Figure 16.—Continued.



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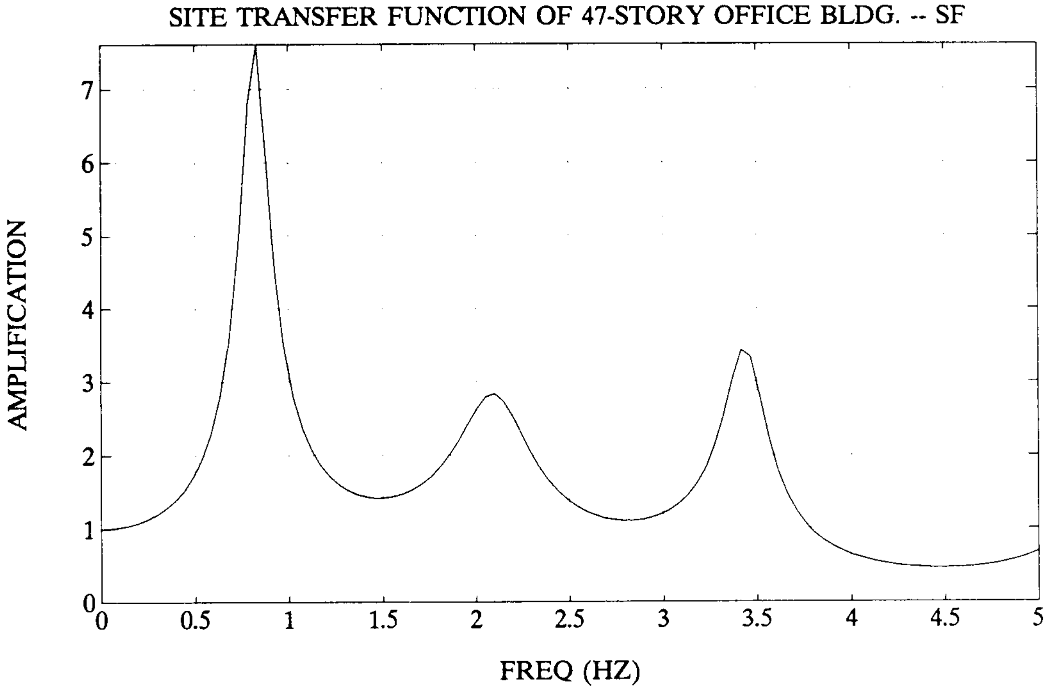


Figure 17.—Site transfer function of Embarcadero Building site.

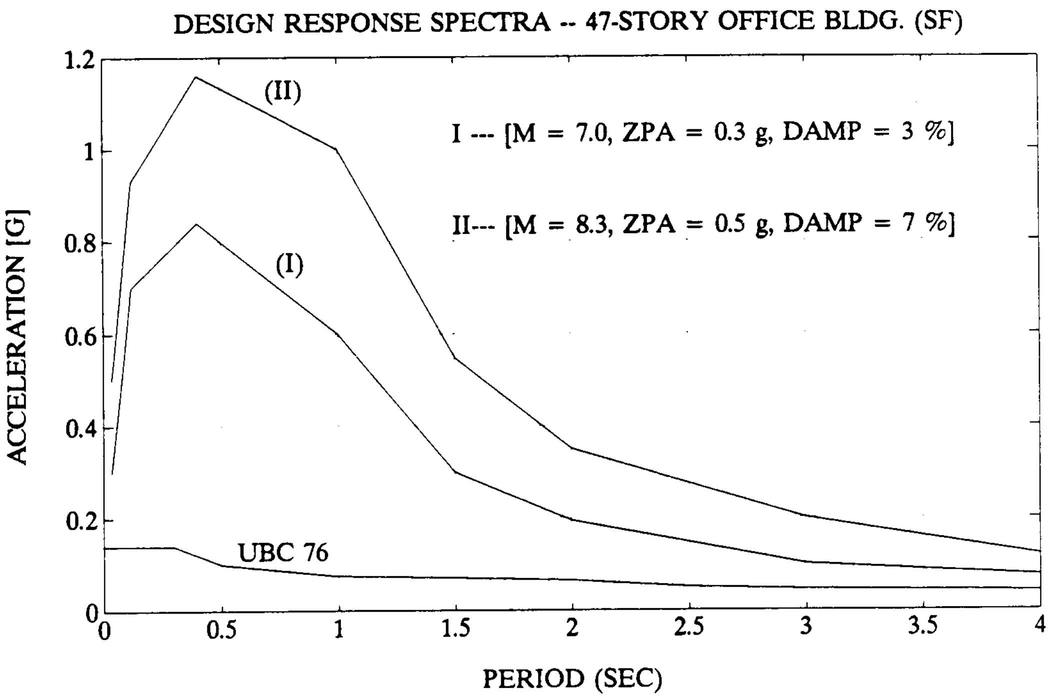


Figure 18.—Design response spectra of Embarcadero Building.

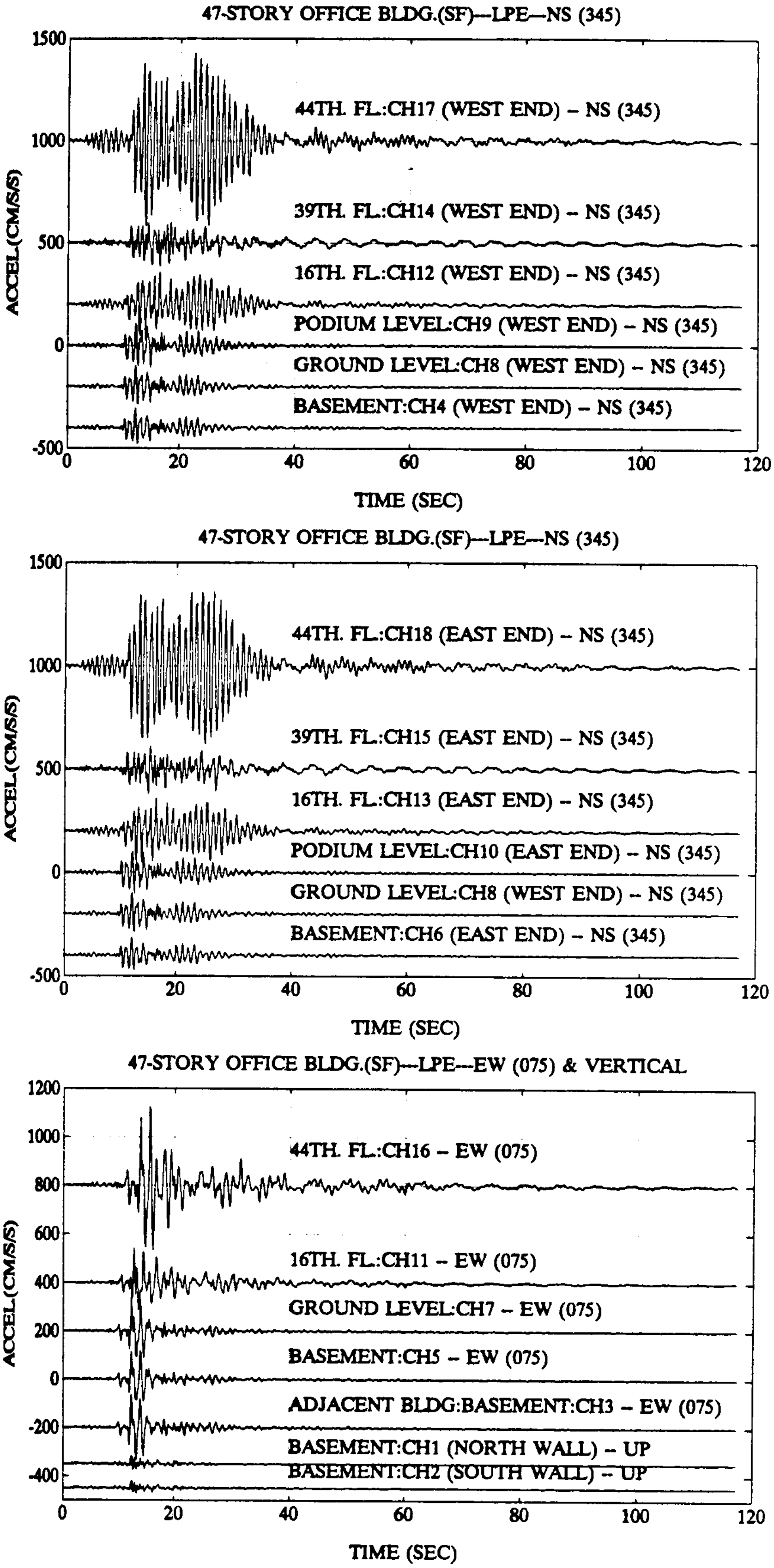


Figure 19.—Recorded accelerations. This data has been band-pass filtered with ramps at 0.07-0.14 Hz and 23-25 Hz (Shakal and others, 1989).

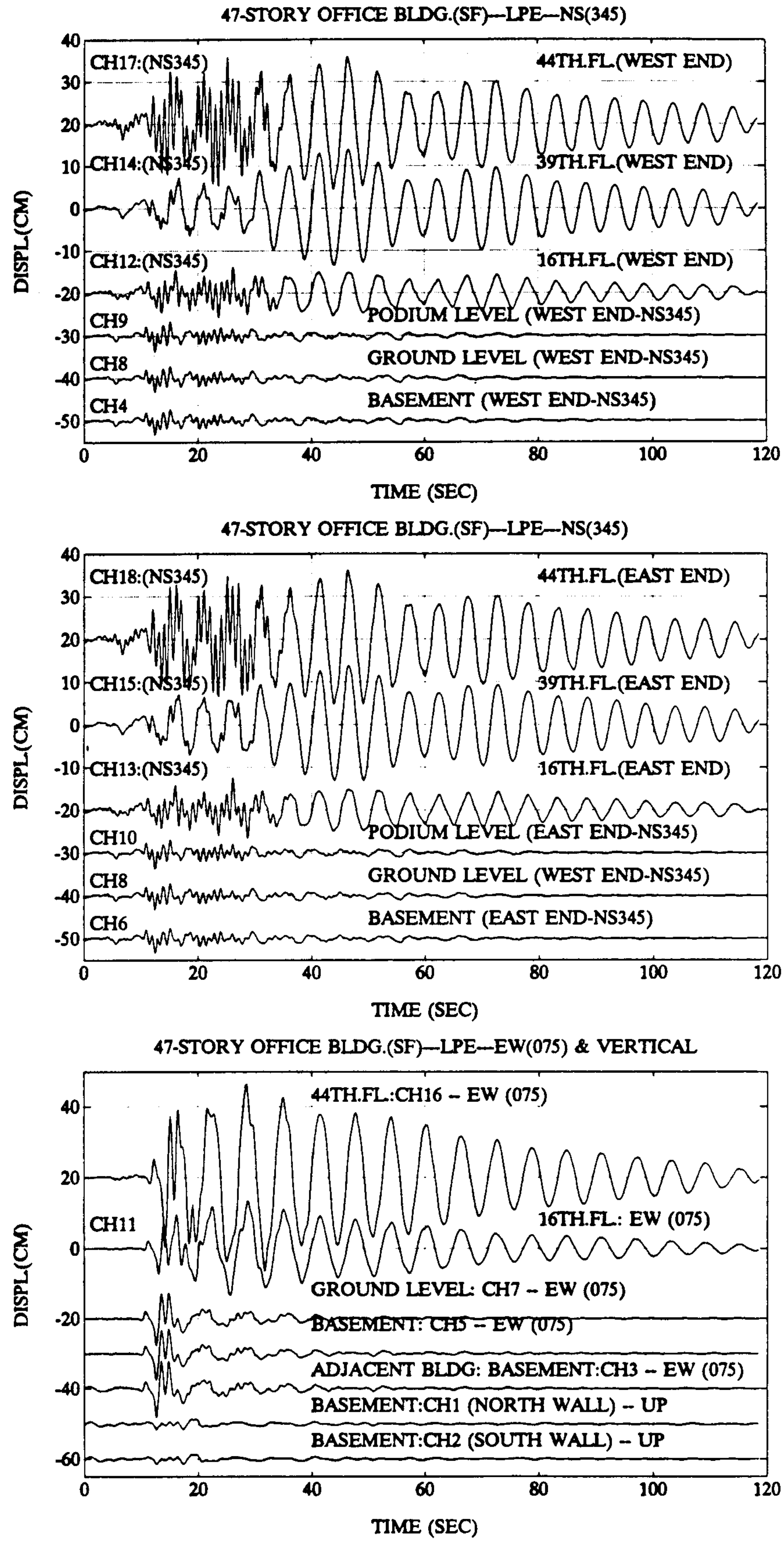


Figure 20.—Recorded displacements. This data has been band-pass filtered with ramps at 0.07-0.14 Hz and 23-25 Hz (Shakal and others, 1989).

Table 4.—*Peak accelerations and displacements for Embarcadero Building*

Level	Sensor	Direction	Acceleration (g)	Displacement (cm)
44	16	EW	0.38	27.2
44	17	NS	0.47	16.2
44	18	NS	0.43	16.4
39	14	NS	0.13	14.3
39	15	NS	0.12	14.1
16	11	EW	0.19	13.7
16	12	NS	0.19	6.3
16	13	NS	0.17	7.6
Podium	9	NS	0.15	3.9
Podium	10	NS	0.14	4.0
GR	7	EW	0.20	7.9
GR	8	NS	0.12	3.4
Basement B	4	NS	0.11	3.4
Basement B	5	EW	0.16	7.7
Basement B	6	NS	0.10	3.3
Basement B	1	UP	0.043	1.6
Basement B	2	UP	0.055	1.6
Basement C(*)	3	EW	0.17	8.4

* At C level below adjacent building.

less than the north-south peak responses at the 16th floor. This may be attributed to several reasons, including the discontinuity of stiffness at the 39th and 41st floors, as discussed later.

Figure 21 shows results of system-identification analysis, using 80 seconds of the recorded basement accelerations as input and the recorded 44th floor accelerations as output. Figure 22 shows the same, using displacements to obtain better identification at the lower frequencies. Both approaches exhibit excellent match between the calculated and observed outputs (Çelebi, 1993).

The modal acceleration contributions are extracted from the system identification analyses and compared to the total response at the 44th floor for the most significant four frequencies that contribute to the overall motions of the building (fig. 23, 0.19, 0.57, 0.98, and 1.33 Hz for the north-south direction; fig. 24, 0.16, 0.46, 0.77, and 1.06 Hz for the east-west direction). The figures also show the Fourier amplitude spectrum of each mode superimposed on that of the total response. The damping values, also determined from system identification procedures, are provided in these figures and summarized in table 5. It is noted that the four significant modal periods in each of the principal axes of the building follow the general rule of thumb approximation of T, T/3, T/5, and T/7. For all four modes in each of the two principal axes, modal damping percentages determined by system identification vary between 1.4 and 3.7 percent (table 5). As expected, the damping percentages of the north-south (braced) direction are lower than the east-west (unbraced) direction. The east-west modal damping percentages vary between 2.5 percent and 3.7 percent. The damping percentage of the east-west fundamental mode, when determined alternatively by the logarithmic decrement approach using the 44th floor displacements, yields 3.5 percent. The fundamental mode damping percentages are 2.5 percent and 3.7 percent for north-south and east-west, respectively (table 5). In summary, such low modal damping percentages explain why the response records are longer than the processed 120 seconds (figs. 19 and 20) (Çelebi, 1993).

The extracted modal displacements at the top three north-south instrumented floors are grouped and plotted in figure 25. A separate system identification analysis is performed for each of the additional input-output pairs (39th and ground floor, and 16th and ground floor). This is done primarily to investigate why the 39th level north-south motions are smaller than those of the 16th floor. It is noted in figure 25 that the third-mode contribution to the 39th floor motions is much smaller than at the 44th or the 16th floor. On the other hand, while the second-mode contribution is comparable in amplitude for the 44th, 39th, and 16th floors, it is in phase between the 44th and the 39th floor, 180° out of phase between the 39th and 16th floors. This anomaly of reduced motions at the 39th floor may be directly attributable to the discontinuity of the stiffness at the 40th floor, causing (1) the 39th floor to behave as the upper nodal point of the third mode, (2) a whipping effect of the top floors, and/or (3) a resonating appendage effect of the floors above the 39th floor, where the bracing is discontinued and the plan of the floors gets smaller (in stiffness and mass). Whatever combination occurs, the main reason is the change in stiffness (Çelebi, 1993; Astaneh and others, 1991; Chen and others, 1992).

The explanations provided above are also related to discussions on drift. For the north-south direction, figure 26 shows superimposed drift ratio time-history between the 44th and 39th floors and between the 44th floor and street level. Figure 26 shows superimposed drift ratios between the 16th floor and street level, between the 39th and 16th floors, and between the 44th floor and street level. When average drift ratios between the 44th floor and street level are considered, the peak is approximately 0.0014 or less (28 percent of 1976 Uniform Building Code allowable). Average drift ratio exceeds the 0.005 allowable only between the 44th and 39th floors (with a peak of 0.006). This may be attributed to either the discontinuity of the eccentric bracing above the 40th floor or the whipping effect at the top floor, which cannot be confirmed since there are no sensors at the two consecutive top floors. Similarly, for the east-west direction, the superimposed drift ratio time-histories between the 16th floor and street level, between the 44th floor and the 16th floor, and between the 44th floor and street level are provided in figure 26. Since, in the east-west direction, there are no sensors on the 39th floor or the two top consecutive floors, it is not possible to determine whether the code-allowable drift ratio of 0.005 was exceeded. However, there appears to be a mostly consistent average drift ratio with a peak of less than 0.002 in this direction. Maximum drift ratio is 0.0023 between the 44th and 16th floors. Therefore, drift ratios on the average were about 40 percent (or less) of

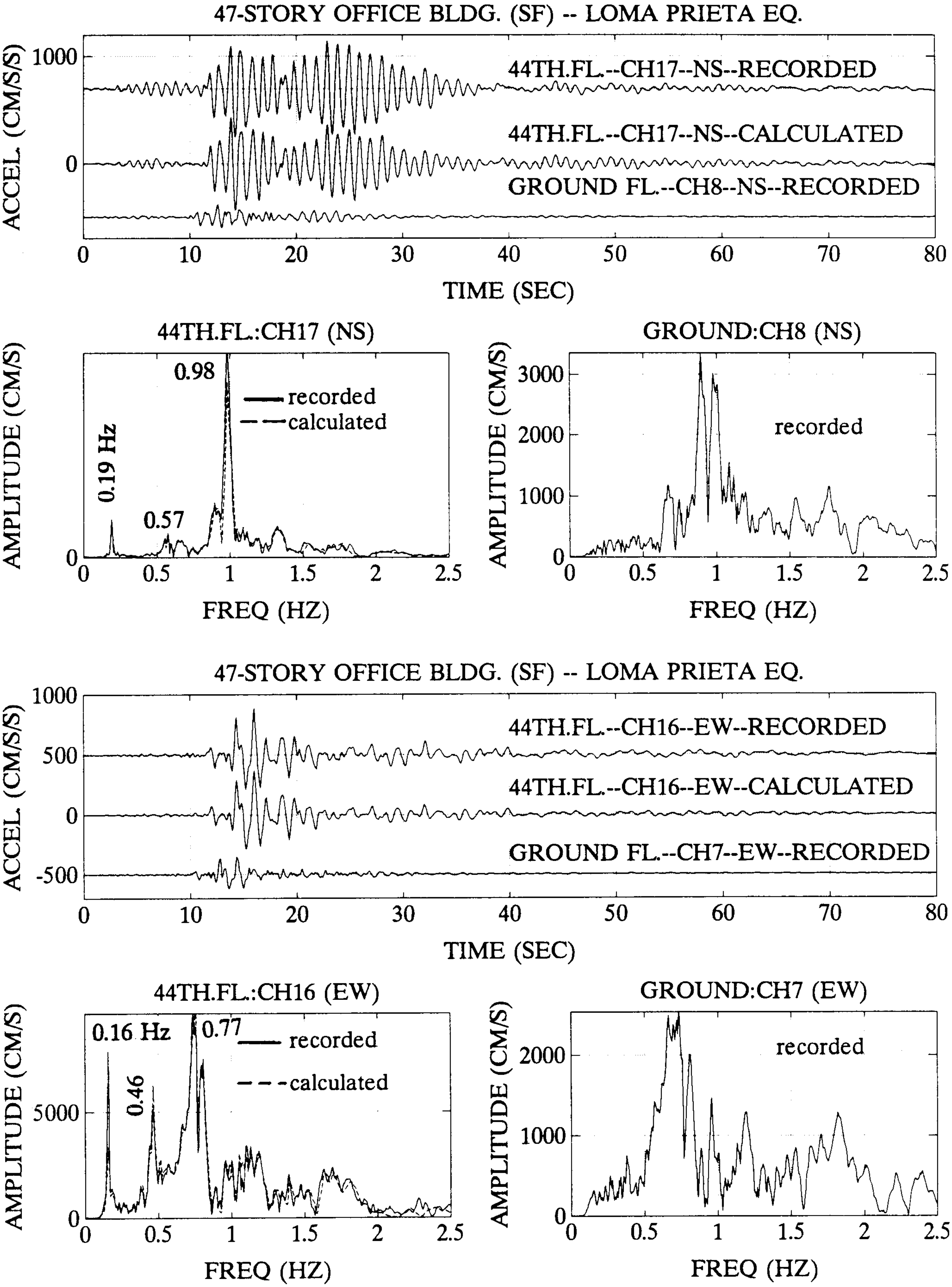


Figure 21.—System identification using ground-floor accelerations as input and 44th-floor accelerations as output for Embarcadero Building.

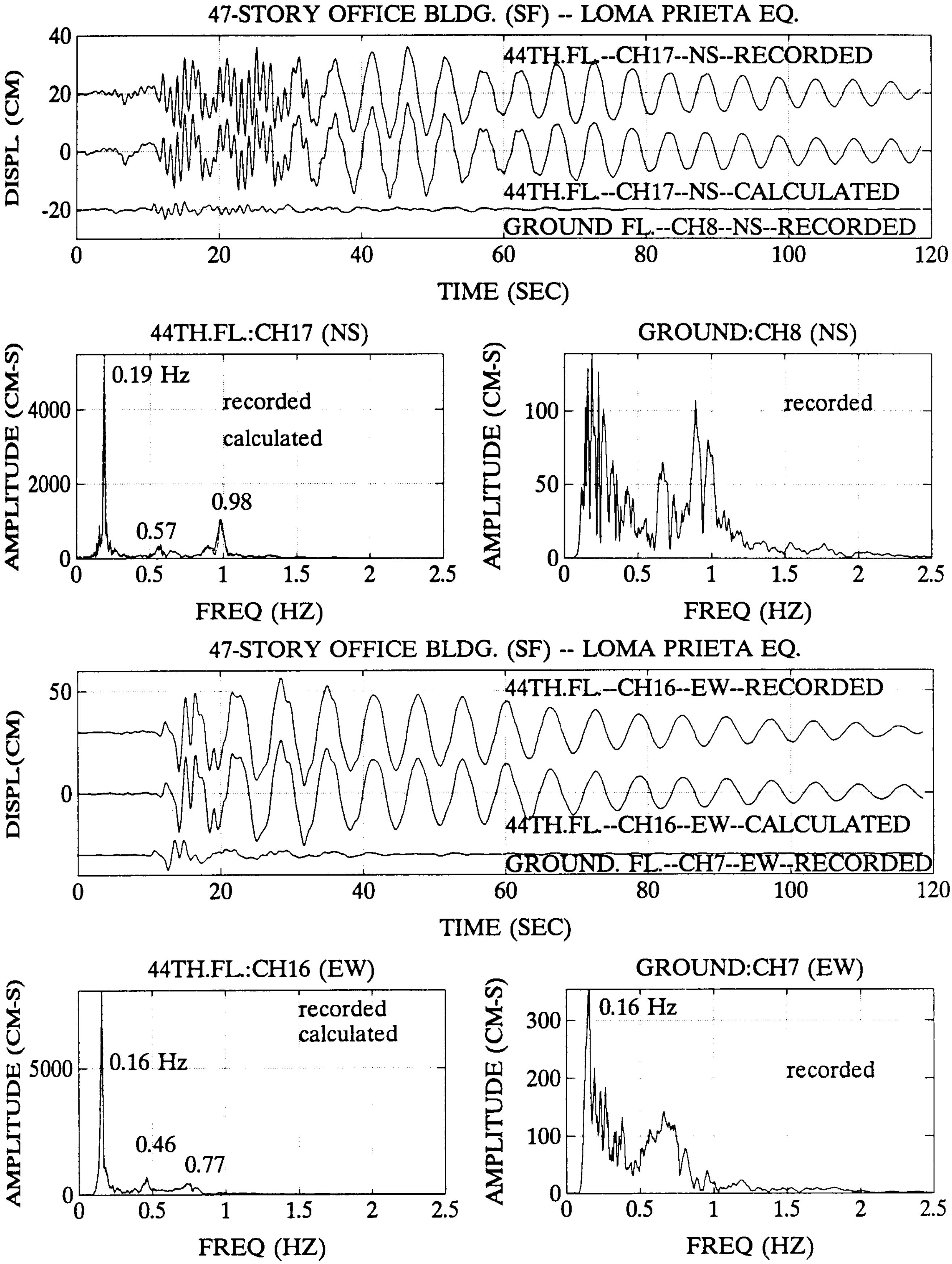


Figure 22.—System identification using ground-floor displacements as input and 44th-floor displacements as output for Embarcadero Building.

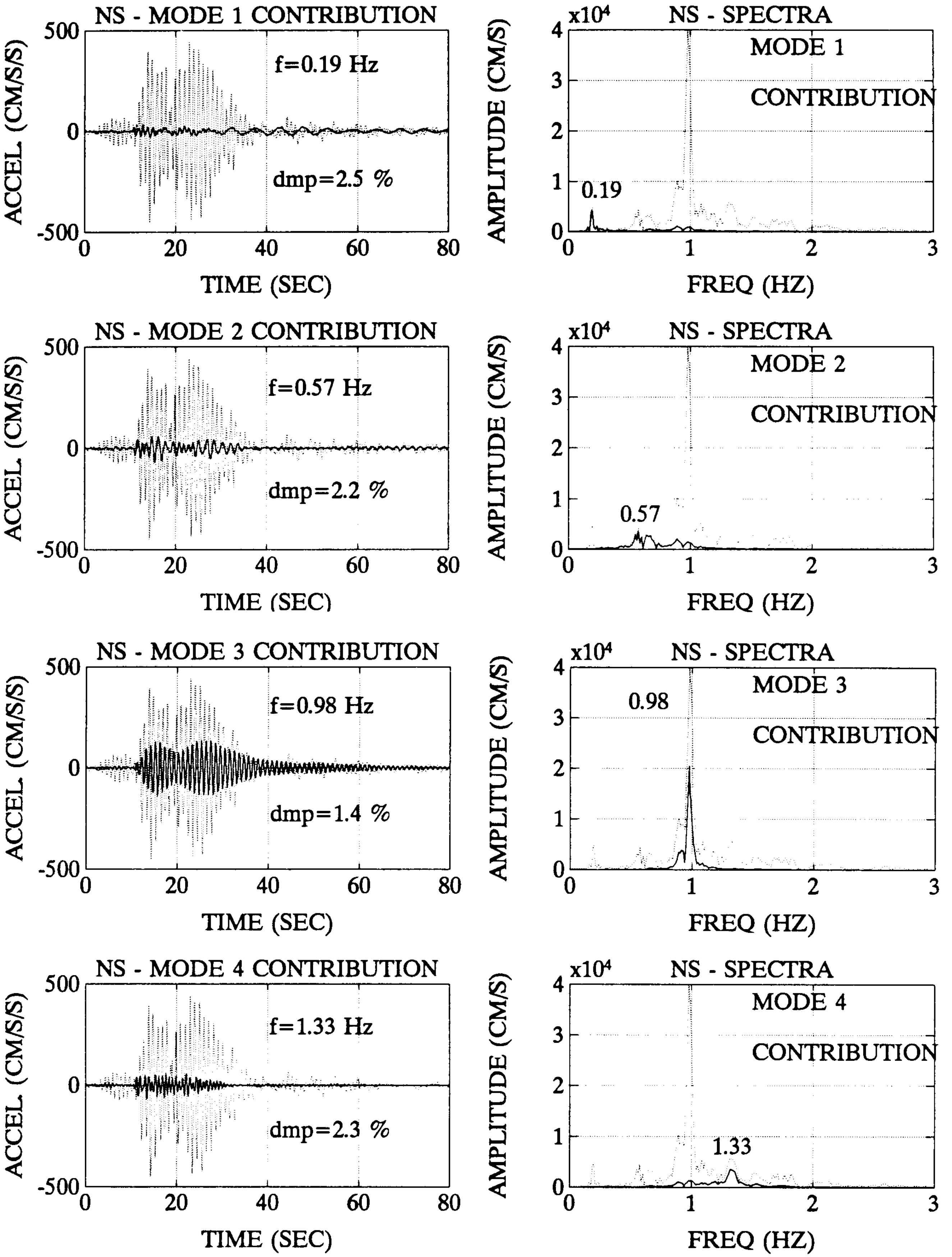


Figure 23.—Significant north-south modal accelerations (and their amplitude spectra) of the 44th-floor for Embarcadero Building extracted from system identification analysis and comparison with the total 44th-floor acceleration.

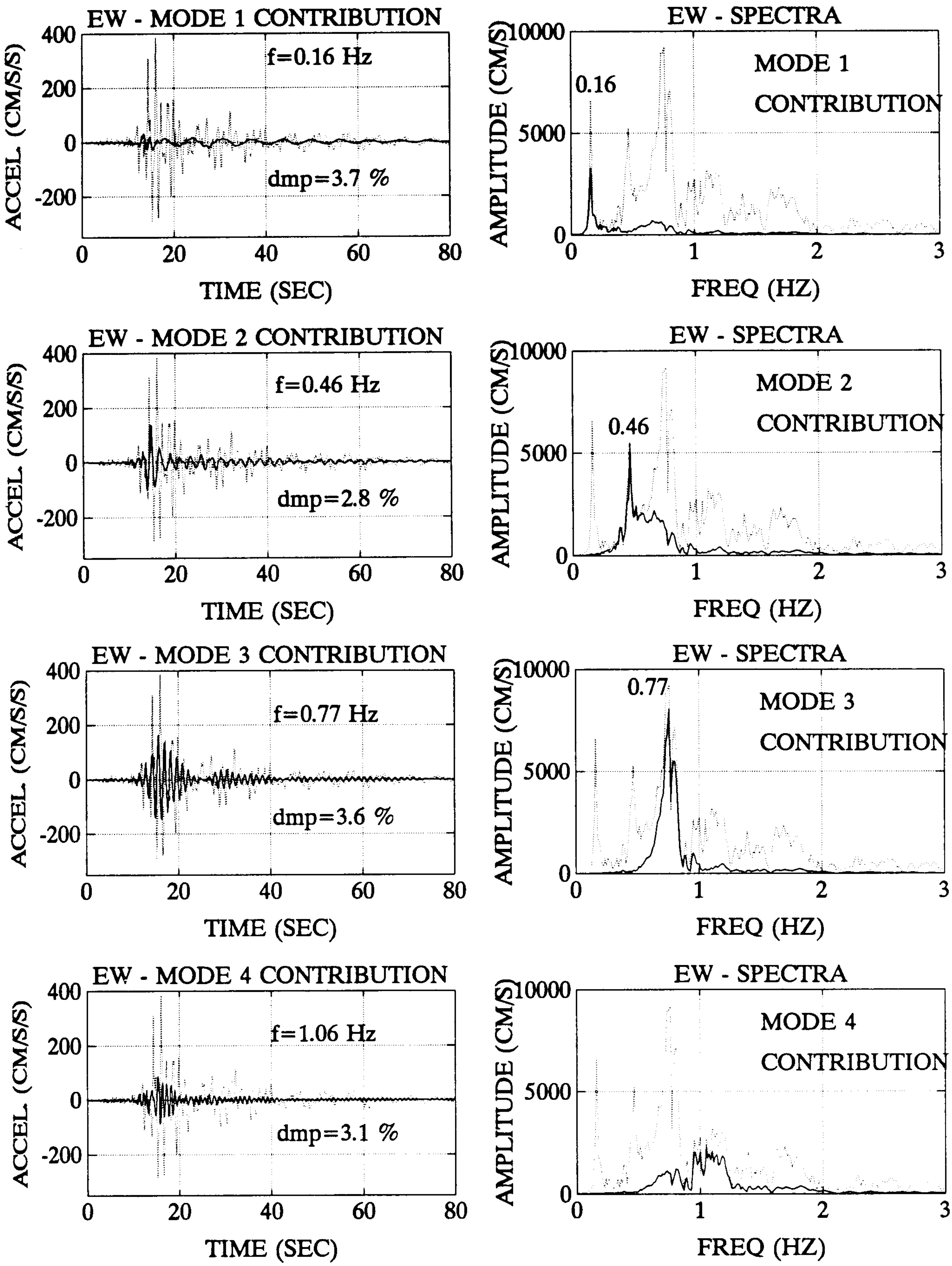


Figure 24.—Significant east-west modal accelerations (and their amplitude spectra) of the 44th-floor for Embarcadero Building extracted from system identification analysis and comparison with the total 44th-floor acceleration.

Table 5.—*Identified dynamic characteristics for Embarcadero Building*

MODE	NS f (Hz)	NS T (s)	NS ξ (pct.	EW f (Hz)	EW T (s)	EW ξ (pct.)
1	0.19	5.26	2.5	0.16	6.25	3.7
2	0.57	1.75	2.2	0.46	2.17	2.8
3	0.98	1.02	1.4	0.77	1.30	3.6
4	1.33	0.75	2.3	1.06	0.94	3.1

code-allowable ratios during the earthquake. It is safe to predict that the allowable drift ratios could be exceeded for the design level II earthquake. This conclusion is from studies of Astaneh and others (1991), Chen and others(1992), and Çelebi (1993).

Torsional response and rocking of the building is insignificant (Astaneh and others, 1991; Chen and others, 1992; Çelebi, 1993).

The amplitude at approximately 0.75-0.8 Hz (1.25-1.33 s), noted in the Fourier amplitude spectra (figs. 21, 22), is attributed to belonging to the site because the amplitude at the roof does not increase significantly when compared to that at the ground floor. Furthermore, the site period, estimated to be around 1.25-1.40 s, falls within the range of modal periods of the building, and therefore the building may have been subjected to a double-resonance effect. It is noted that the site-specific response spectra (fig. 18) do not adequately reflect the resonating site period (fig. 17) (Çelebi, 1993).

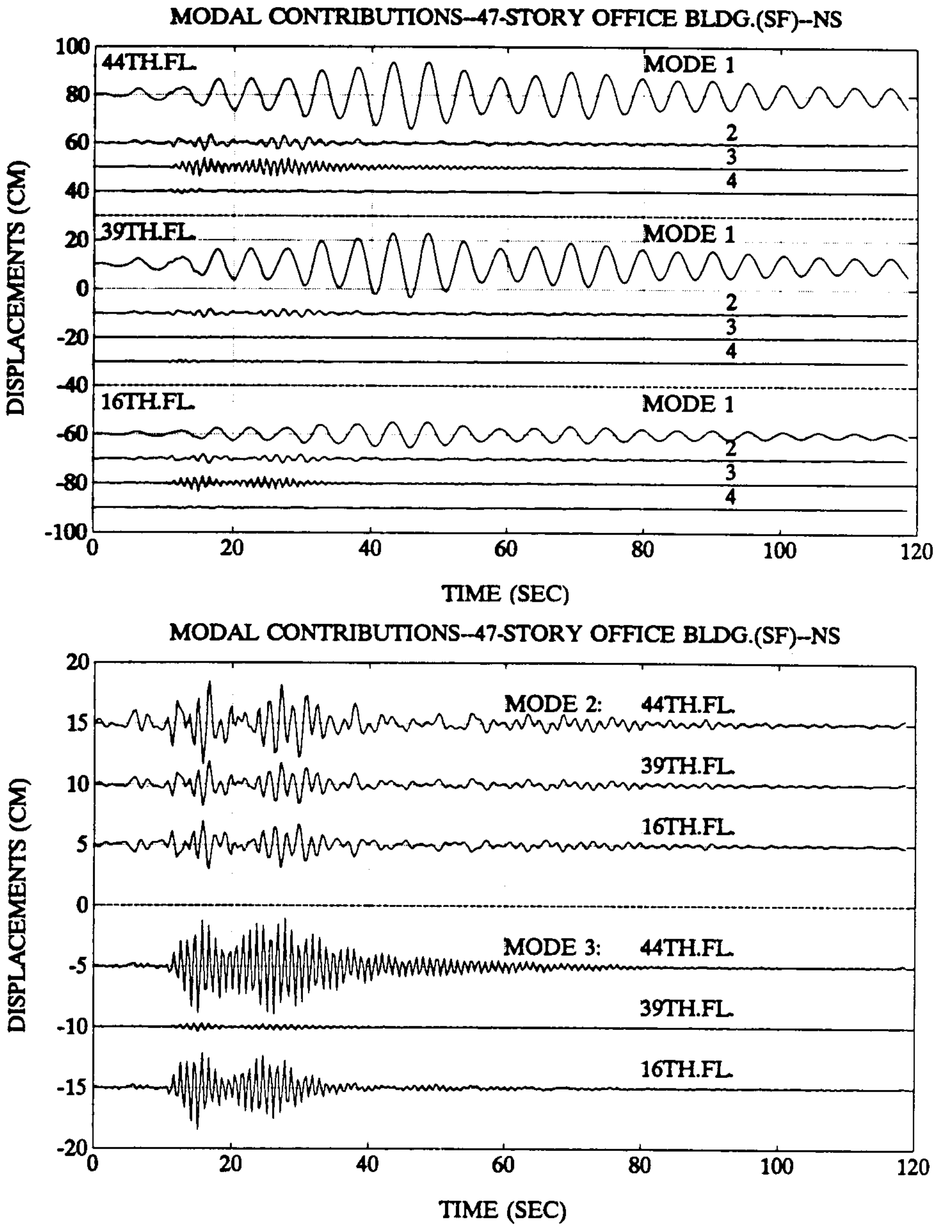


Figure 25.—Comparison of modal displacement contributions at the top three north-south instrumented floors for Embarcadero Building.

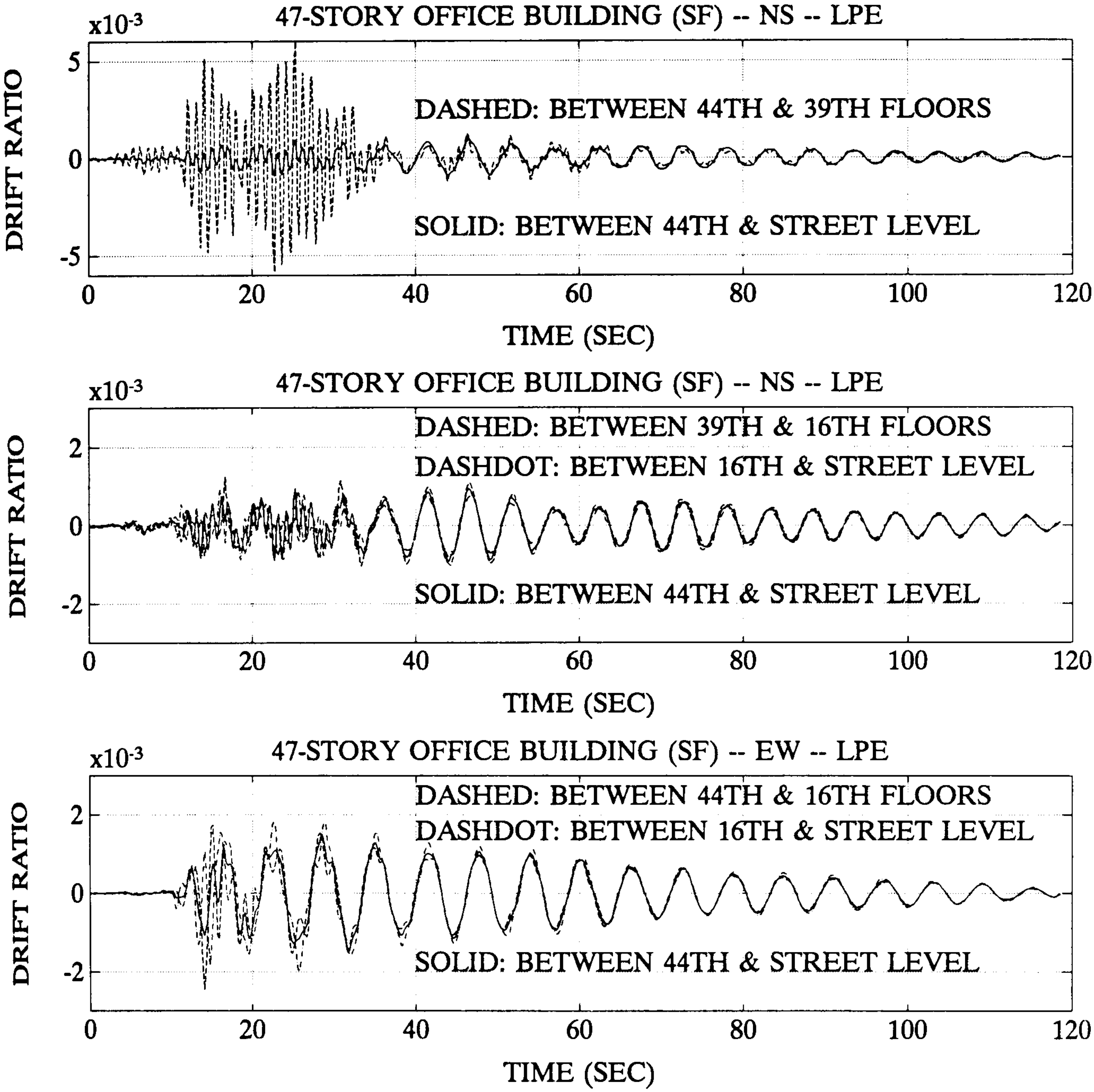


Figure 26.—Drift ratios for Embarcadero Building.

In figure 27, rotation time-histories (calculated from displacement time-histories) of the mat as well as the basement walls are shown. The significance of this plot is that (1) the mat rotation has a peak value of 0.000057 radians (0.0033°) while the peaks of the wall rotations are (north-south) 0.0004 radians (0.023°) and (east-west) 0.00048 radians (0.028°), and (2) the frequencies of these rotations are the same as those of significant translational frequencies in respective directions, thus confirming the presence of structural frequencies in the basement motions. The anomaly here is that the rotations of the walls are 7-8 times larger than the rotations of the mat. Although there may be some numerical errors related to data processing, these ratios are too high to ignore. Similar magnitude peak rotations were observed from the response data of the nearby Transamerica Building (Çelebi and Şafak, 1991; Soydemir and Çelebi, 1992).

In addition to detailed analyses presented above, Chajes and others (1992) used an approximate technique to conduct dynamic analysis of the Embarcadero Building. Their analysis utilizes a continuum methodology to create a reduced-order representation of the building. The accuracy of the approximate dynamic analysis is established by comparing the computed results to the actual response recorded during the earthquake.

Astaneh and others (1991) and Chen and others (1992) performed detailed dynamic elastic time-history and response spectrum analyses of the Embarcadero Building and compared the calculated responses with actual recorded responses. They varied several parameters in the development of their mathematical models. The model buildings were then subjected to ground accelerations recorded at the basement level of the building during the earthquake. The set of analyses using the mathematical models produced floor displacement time-histories which were then compared with the recorded values. The results show that at Loma Prieta level ground motions, the design seems sufficient. They also performed nonlinear dynamic analyses using two-dimensional models of a typical frame, whereby the mathematical models were subjected to base excitations of (1) actual recorded motions during the earthquake, (2) motions represented by scaling-up of the peak accelerations of the earthquake record, and (3) motions of a hypothetical San Andreas M=8+ plus event to simulate a major earthquake. The trends in development of plastic hinges throughout the structures for these cases indicate that significant inelastic behavior occurs at the floors between the 36th and 42d levels and is attributed to the change in stiffness at those levels. The development of plastic hinges for cases 2 and 3 described above are exhibited in figures 28 and 29 (Astaneh and others, 1991; Chen and others, 1992; Bonowitz and Astaneh, 1994). These figures are important in that they indicate the behavior that may be expected from the Embarcadero Building during future earthquakes with large input motions (either due to larger magnitude earthquakes or earthquakes at closer distances to the building).

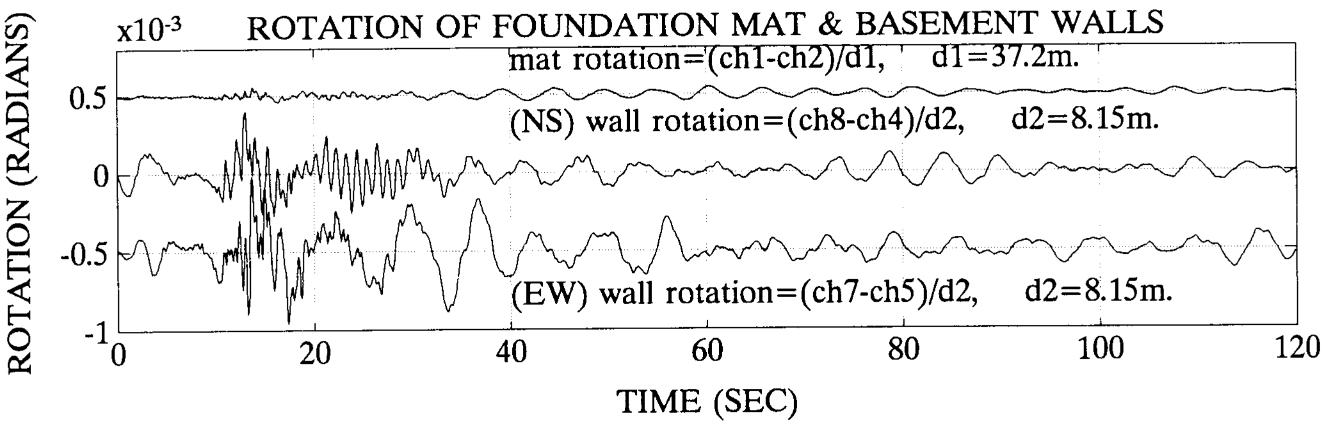


Figure 27.—Rotation of basemat compared to rotation of basement walls for Embarcadero Building.

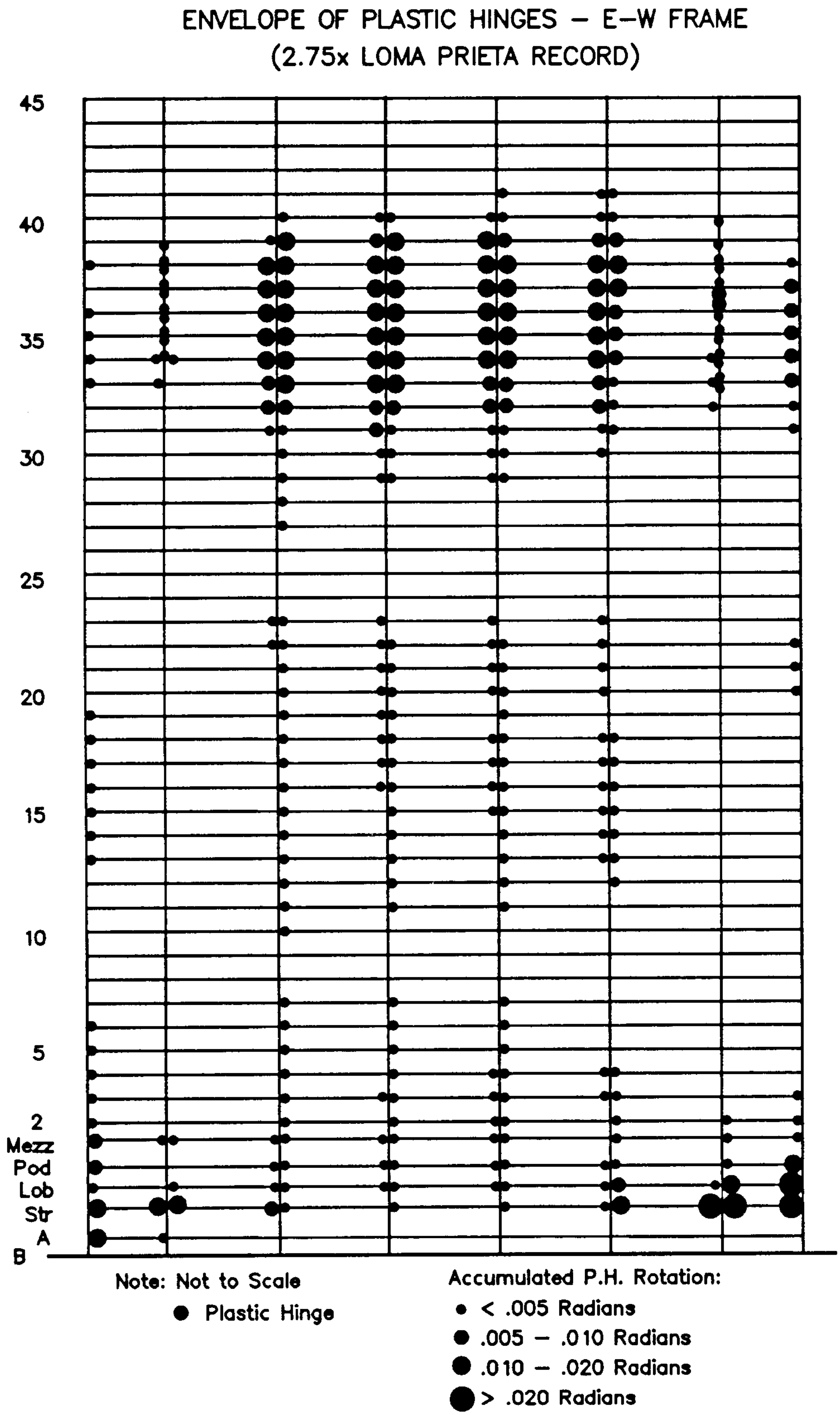


Figure 28.—Development of plastic hinges formed in the east-west frame modeled by Astaneh and others (1991) for 2.75 times the Loma Prieta earthquake motions (with permission of A. Astaneh).

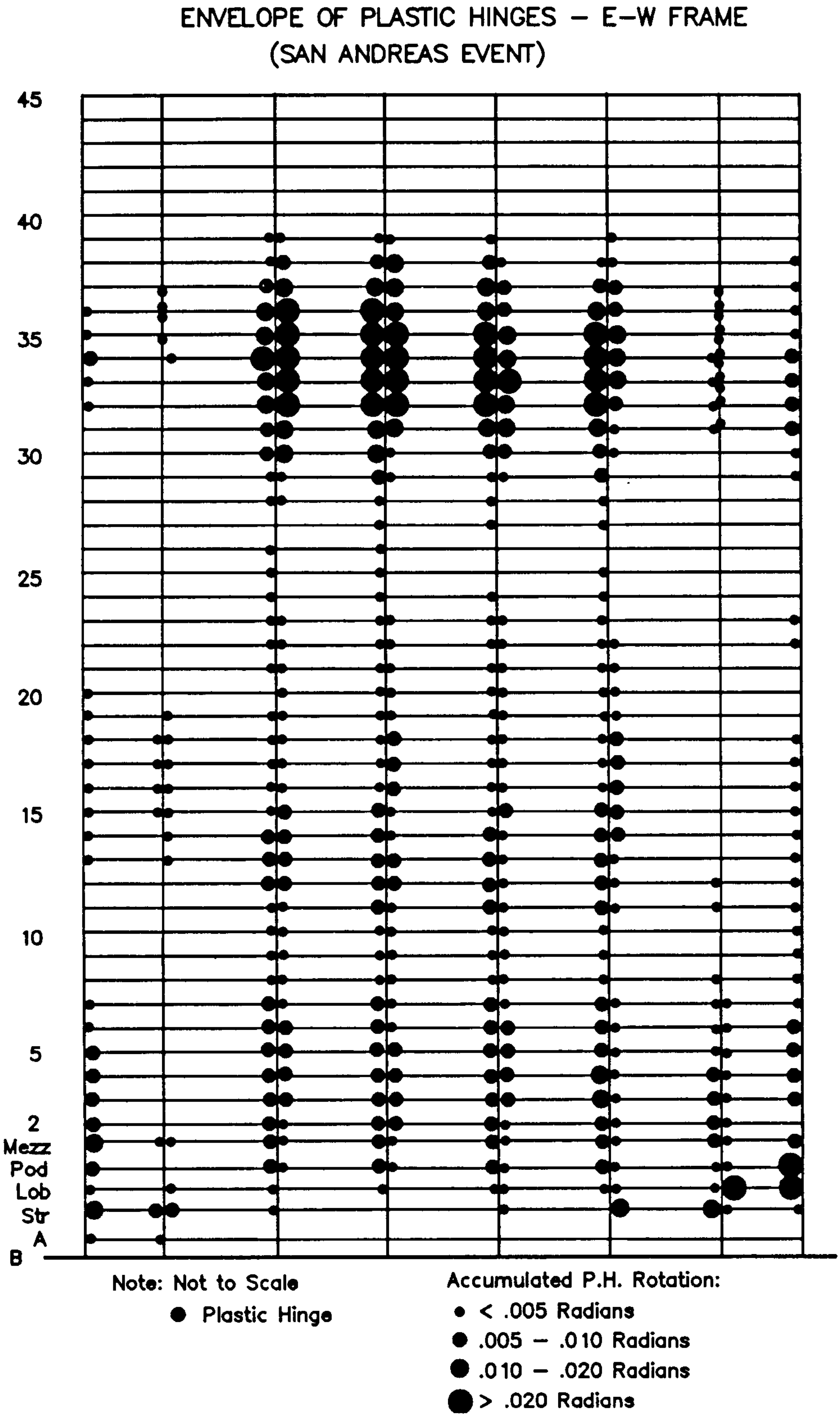


Figure 29.—Development of plastic hinges formed in the east-west frame modeled by Astaneh and others (1991) for a hypothetical M=8+ San Andreas event (with permission of A. Astaneh).

SANTA CLARA COUNTY OFFICE BUILDING (SAN JOSE)

One of the buildings that benefited from studies of its recorded responses during strong-motion events is the Santa Clara County Office Building in San Jose (fig. 30). A decision to retrofit was based on response data of the building recorded during the Loma Prieta earthquake and two previous but smaller earthquakes, the 24 April 1984 Morgan Hill (M_s=6.1) and the 31 March 1986 Mt. Lewis (M_s=5.5) (Huang and others, 1985; R. Darragh, personal commun., 1991). The set of data from these three earthquakes has been studied by Lin and Papageorgiou (1989), Boroschek and others (1990), Boroschek and Mahin (1991), and Çelebi (1994a). Although this building did not suffer structural damage during these earthquakes, it suffered extensive nonstructural and contents damage. Rihal (1994) studied the performance of nonstructural members of the building during these three earthquakes, when the occupants were uncomfortably and severely shaken during the prolonged and resonating vibration of the building. Consequently, in 1994, the building was retrofitted with viscous elastic dampers (Crosby and others, 1994).

A general view of the Santa Clara County Office Building is seen in figure 30A. The location of the building, the epicentral locations of the three earthquakes referred to above, and the instrumentation scheme are shown in figure 30*B* (Çelebi, 1994a). This 13-story, 56-m-tall, moment-resisting steel-framed building was built in 1975 according to the Uniform Building Code (1970). There are six column lines in each direction of the approximately 51×51m plan of the building.

Figure 31 shows acceleration responses at the roof recorded during the three earthquakes. The figure clearly indicates the long-duration and resonating responses of the structure and the beating effect observed in the responses. Figure 32 shows typical system identification analysis results (Çelebi, 1994a) for only Loma Prieta data. Peak accelerations at the roof and basement levels, the fundamental translational and torsional frequencies (and periods), and damping percentages extracted by system identification techniques (for all three earthquakes) are presented in table 6. The second and the third modes (at approximately 1.45 and 2.50 Hz) contribute very little to the overall response; therefore, the remainder of the discussion will be devoted to the fundamental modes only. The small differences between the periods determined from the three earthquake response data are noted. Since the structural characteristics of the building during the three earthquakes are very similar, only Loma Prieta is chosen to further characterize the structural response.

Figure 33*A* shows acceleration responses at the roof level, as well as the difference of parallel accelerations at the roof level (nominal torsional accelerations–actual torsional accelerations can be calculated by dividing the nominal torsional accelerations by the distance between the two parallel sensors). Figure 33*B* shows the amplitude spectra of the unidirectional responses clearly peaking at the translational frequency (period) 0.45 Hz (2.22 s). Figure 33C shows amplitude spectra of the nominal torsional accelerations (calculated from parallel records in the north-south and east-west directions respectively) that peak at 0.57 Hz (1.69 s) and 0.45 Hz (2.22 s). Figures 33*D* and *E* show the coherence, phase angle, and normalized cross-spectra of the unidirectional response and confirm the translational frequency at 0.45 Hz. At this frequency, the motions are coherent and are in phase, clearly indicating that they are related. Figure 33*F*, on the other hand, confirms the torsional frequency at 0.57 Hz that were identified from the nominal torsional accelerations represented by the differential accelerations of two parallel sensors at a floor. There is unity coherence and the phase angle is zero at 0.57 Hz (Çelebi, 1994a). The proximity of the torsional frequency (period) at 0.57 Hz (1.69 s) to the translational frequency (period) at 0.45 Hz (2.2 s) causes the observed coupling and beating effect.

Lin and Papageorgiou (1989) studied the Santa Clara County Office Building response data from the Morgan Hill earthquake and concluded that strong beating-type phenomena occur in buildings with identifiable close-coupled torsional and translational modal characteristics, as in the case of the Santa Clara County Office Building. Boroschek and Mahin (1991) further elaborated on this issue. In their investigation of the behavior of this lightly damped, torsionally coupled structure, they developed three-dimensional linear and nonlinear numerical models and performed several elastic and inelastic computer analyses. As in the recorded data, Boroschek and Mahin found that the calculated responses of the building are characterized by (1) long duration, narrow-banded periodic motions with strong amplitude modulation, (2) large displacements and torsional motions, (3) large amplification of the input ground motions, and (4) slow decay of the building's dynamic responses. They concluded that the unusual response characteristics of the building are due to design parameters that produced a structural system with low equivalent viscous damping, resonance, and beating.

The close-coupling of the torsional and translational frequencies at low damping percentages (lightly damped system) clearly explains that the translational and torsional modes reinforce one another during vibration, with only small dissipation, and that beating occurs with a period of $T_b=2T_1T_2/(T_1-T_2)=2(2.22)(1.69)/(2.22-1.69)=$ 14.2 s (Borosheck and Mahin, 1991; Çelebi, 1994a).

The mat foundation of the building rests on alluvial site conditions. The depth to bedrock at the site is estimated to be between 270 m and 500 m. Figure 34 shows the site transfer function plots for two estimated depths to

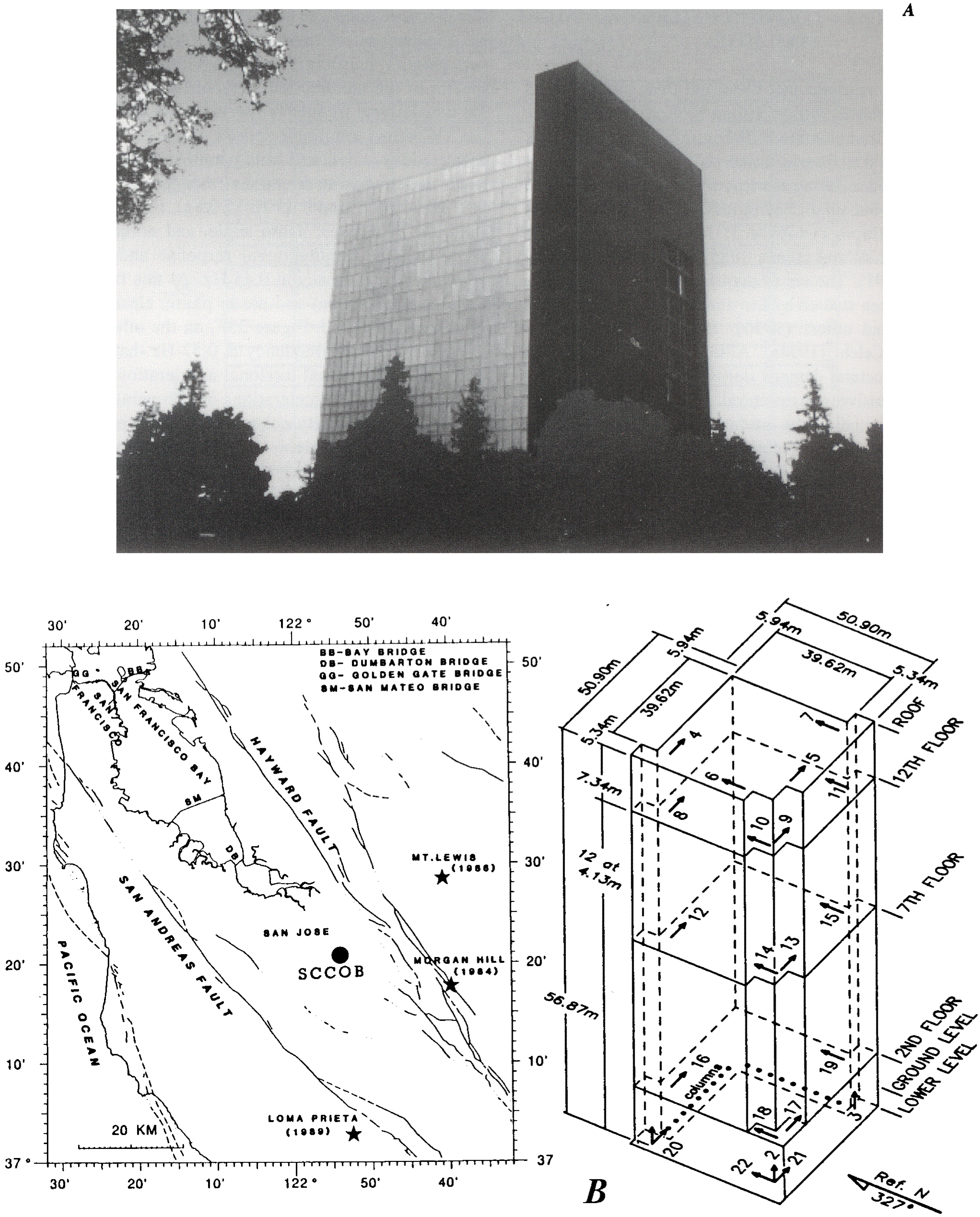


Figure 30.—*A*, Santa Clara County Office Building (SCCOB). *B*, Location of Santa Clara County Office Building, its overall dimensions and instrumentation, and epicenters of three nearby earthquakes.

bedrock and shear-wave velocities (V_s) assigned to each layer based on available geotechnical reports (Earth Sciences Associates, 1971). The figure indicates that the site is capable of generating resonating surface waves at low frequencies that are, as will be shown, close to the frequencies of the building. In a study of the Santa Clara Valley following Loma Prieta, Frankel and Vidale (1992) concluded that 2-5 second long-period motions in the basin can be generated during earthquakes.

Figure 35 shows the coherence, phase angle, and normalized cross-spectra for the roof and basement motions calculated only for the east-west direction, since at the basement the two parallel sensors are in this direction. The proximity of the site frequency at 0.33-0.38 Hz and the fundamental frequency at 0.45 Hz is the cause of the resonating (more or less steady-state surface wave) motions of the building. This is simply explained using the relationship for the amplification (A) of a damped system (in percent), with a frequency ratio (r) of ground to structure):

$$A = 1/\left[(1-r^2)^2 + (2\xi r)^2\right]^{1/2}.$$

For a lightly damped system with r=0.33/0.45=0.73 or 0.38/0.45=0.84, significant amplification in the response can be expected (for example, $A \approx 1/(1-r^2) \approx 2.14$-$3.40$) (Çelebi, 1994a).

The average drift ratios calculated between the roof and the basement and between the 12th floor and 2nd floor reach 0.8 percent and exceed allowable code drift ratios (0.5 percent). The drift ratio between the roof and 12th floor is smaller than the average drift ratio or the code allowable.

In a recent study, Porter (1996) showed that the observed structural response of the building can be explained by the geometrical configuration between earthquake epicentral location and the orientation of the structure.

Rihal (1994) studied nonstructural damage in the building following the earthquake to correlate the recorded California Strong Motion Instrumentation Program response data with observed nonstructural component damage. A methodology is presented to assess the performance and behavior of nonstructural building components during earthquakes. One main objective of this case study was to investigate the relationship between seismic response parameters (for example, peak response acceleration levels, frequency content, and inter-story drift levels) and corresponding nonstructural component damage observed during the earthquake. Significant nonstructural component damage was observed to have occurred, particularly at the 7th and 11th floor levels. Comparison of the observed nonstructural damage and peak recorded accelerations at the 7th floor and at the 12th floor show the thresholds of response accelerations that produce nonstructural component damage. Rihal proposed a nonstructural component damage index expressed as a percentage of components damaged to characterize observed nonstructural component damage data.

In retrofitting the building, Crosby and others (1994) installed viscous elastic dampers in selective bays of the building. Figure 36*A* and *B* show the plan view and vertical sections. The bays of the framed structure where the dampers were installed are indicated in the figures. Figure 36*C* shows a typical viscous elastic damper (Crosby and others, 1994). Çelebi and Liu (1997) performed ambient tests of the building following the retrofit; preliminary results show that for low-amplitude excitation, the dampers produce small changes in the dynamic characteristics but are expected to alter the dynamic behavior significantly during strong shaking. Before the retrofit, Marshall and others (1991) and Çelebi (1996) observed significant differences in the dynamic characteristics of the building for strong- and low-amplitude shaking. This is discussed later in this paper.

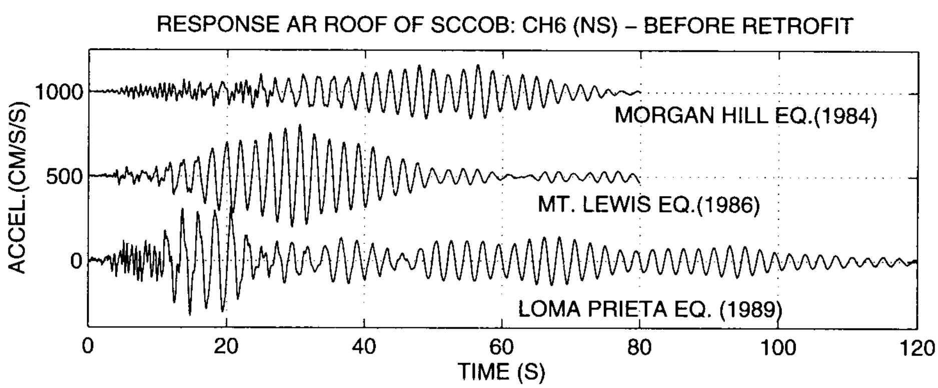


Figure 31.—Acceleration responses at the roof of Santa Clara County Office Building recorded during the Loma Prieta, Morgan Hill, and Mt. Lewis earthquakes.

In summary, there are three causes for the long-duration response of the Santa Clara County Office Building: (1) basin effect and site characteristics that contribute to resonating excitation, (2) the close-coupled translational-torsional mode that causes beating phenomena to occur, and (3) the inherent low-damping of the building. Understanding the cumulative structural and site characteristics that affect the response of the building is important in assessing earthquake hazards to other similar buildings (Çelebi, 1994a). The results emphasize the need to better evaluate structural and site characteristics in developing earthquake resisting designs that avoid resonating effects.

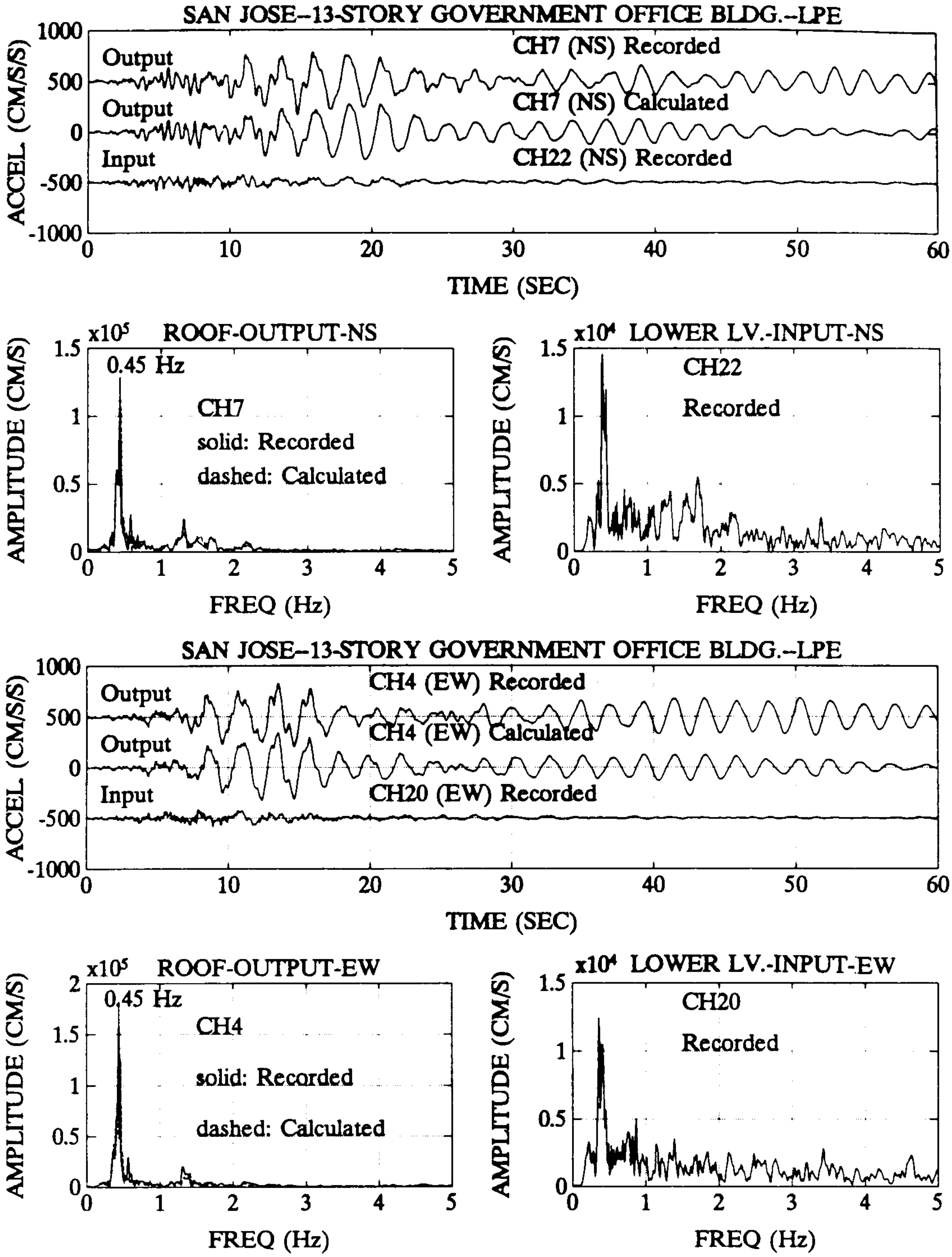


Figure 32.—Typical system identification analysis results of Santa Clara County Office Building for Loma Prieta data (Çelebi, 1994).

Table 6.—*Peak accelerations and dynamic characteristics for Santa Clara County Office Building*

		Earthquake		
		Loma Prieta	Morgan Hill	Mt. Lewis
Peak Acceleration (*g*)	Roof (NS)	0.34	0.17	0.32
	Roof (EW)	0.34	0.17	0.37
	Base (NS)	0.10	0.04	0.04
	Base (EW)	0.09	0.04	0.04
Translational	Period [T (s)]	2.22	2.17	2.08
	Frequency [f (Hz)]	0.45	0.46	0.40
	Damping [ξ (pct.)]	2.70	1.95	2.12
Torsional	Frequency [f (Hz)]	0.57	0.59	0.58
	Damping [ξ (pct.)]	1.69	1.70	1.72

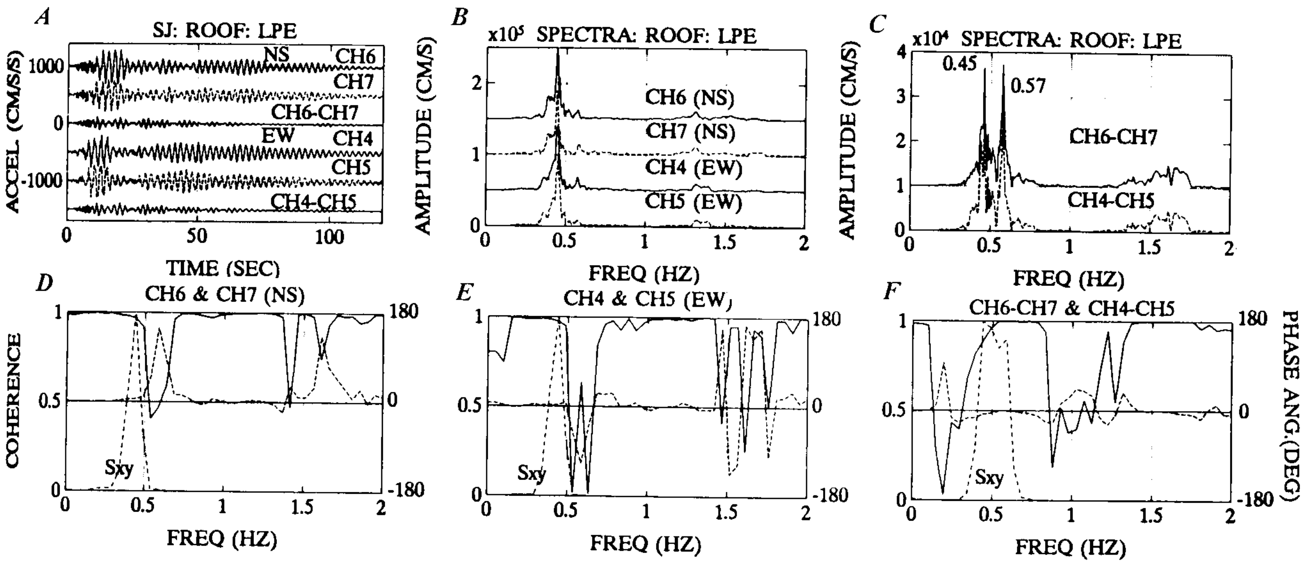


Figure 33.—*A*, Acceleration time histories of roof of Santa Clara County Office Building. *B*, *C*, Amplitude spectra. *D*, *E*, *F*, Coherence (solid line), phase angle (dashed line) plots with cross-spectra, S_{xy}, (dashed-dot line) superimposed to distinguish translational and torsional frequencies.

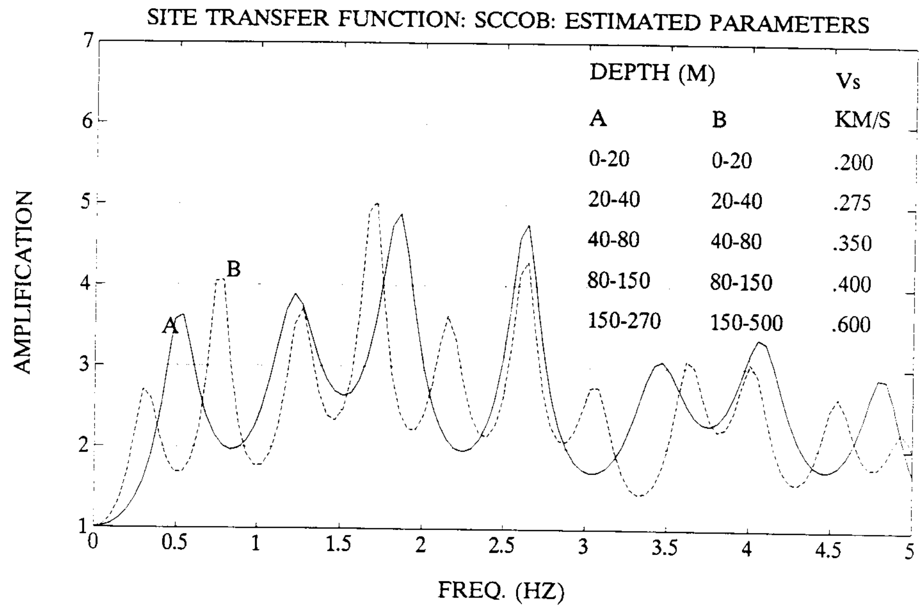


Figure 34.—Site transfer function for Santa Clara County Office Building.

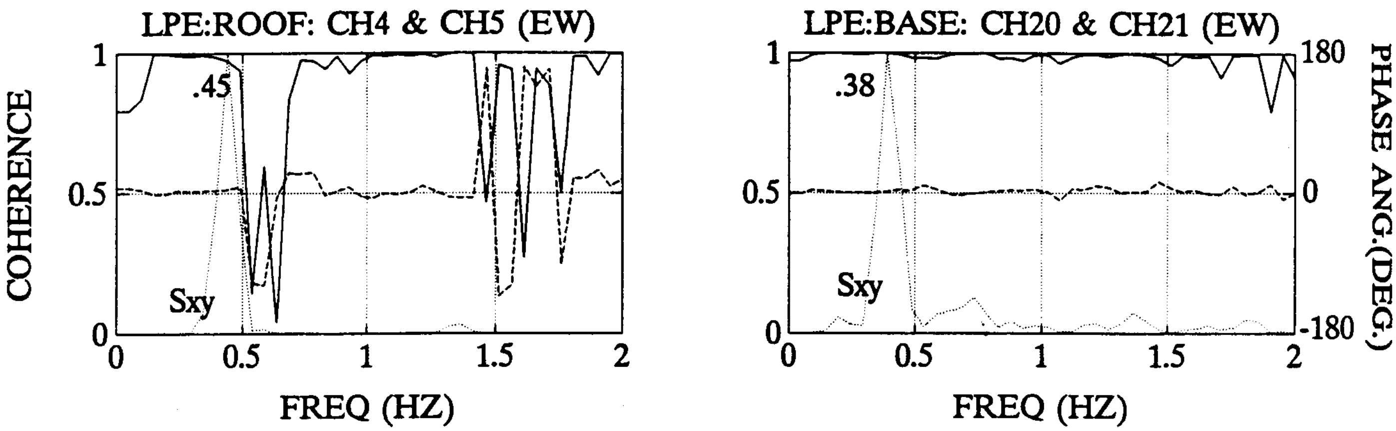
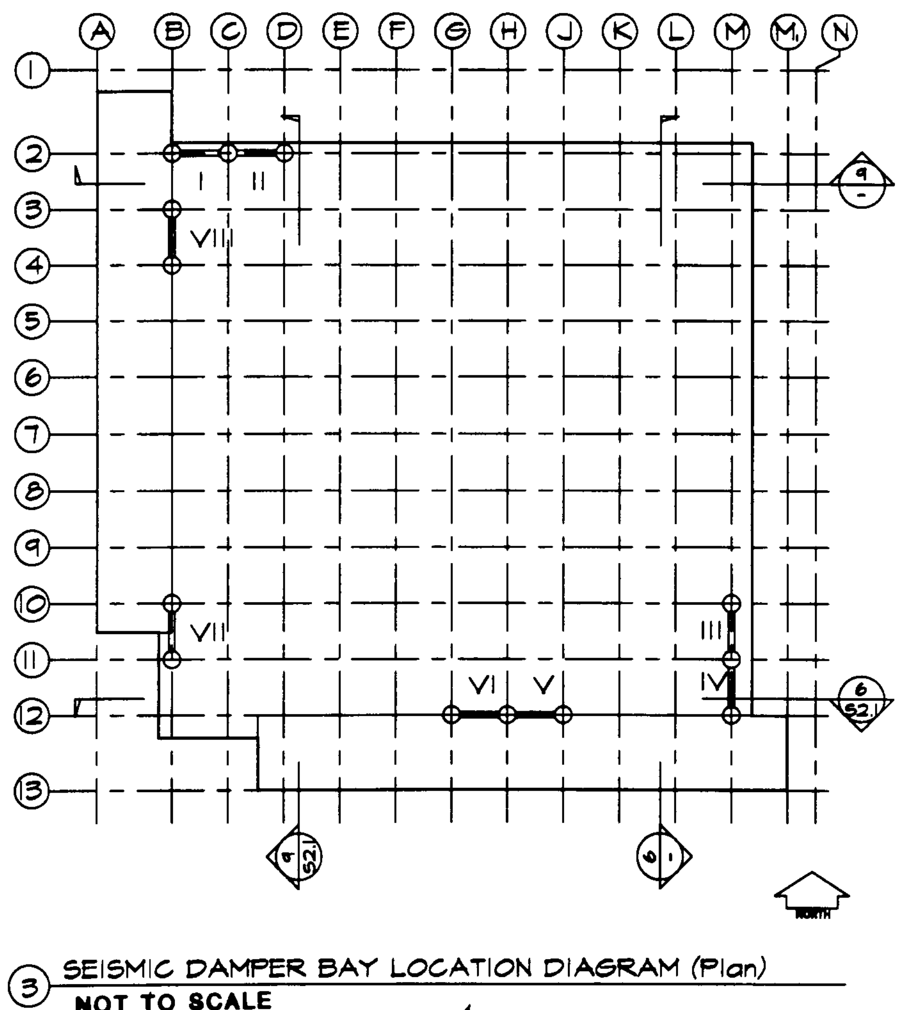
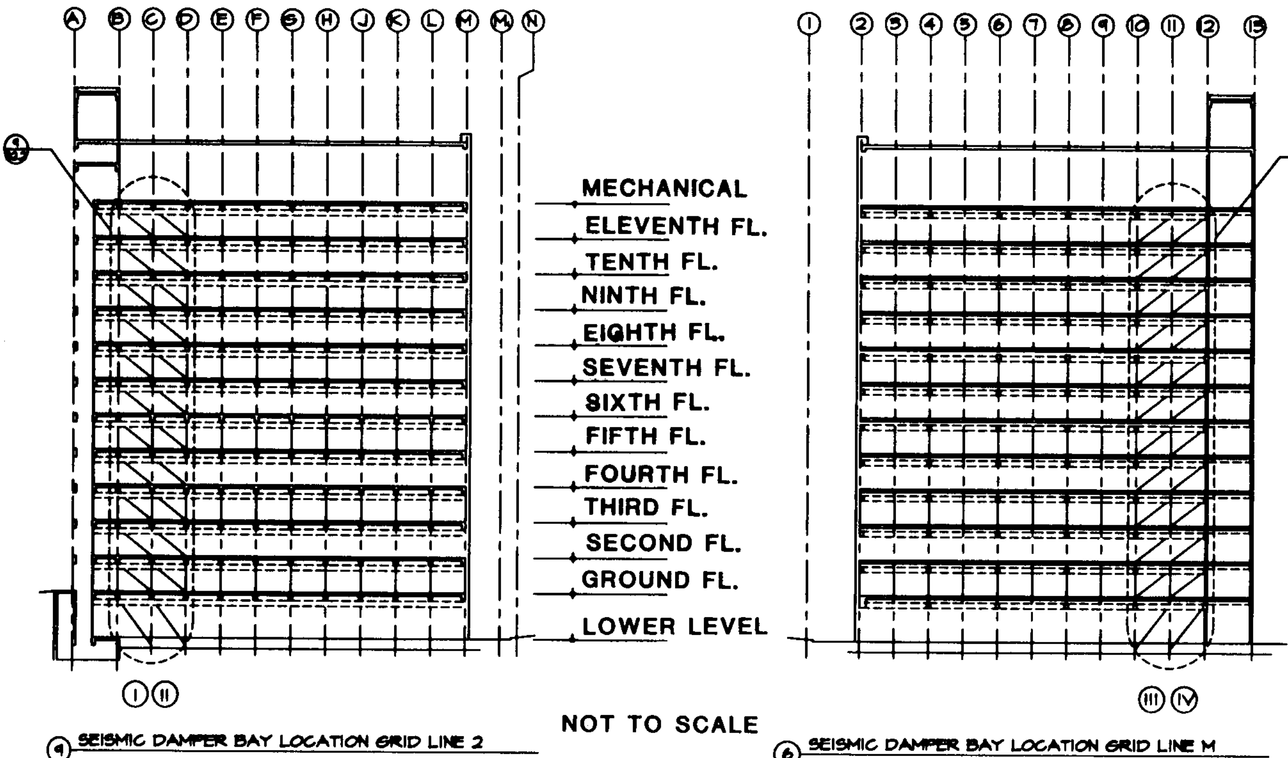


Figure 35.—Cross-spectra (dotted line), coherence (solid line), and phase angle (dashed line) plots for east-west motions at the roof and basement of Santa Clara County Office Building to distinguish structural and site frequencies.



A

Figure 36.—*A*, Plan view of Santa Clara County Office Building. *B*, Vertical sections of building show bays where viscous elastic dampers have been installed for retrofitting the structural system to alter its dynamic behavior. *C*, Typical damper installed. (All figures courtesy of P. Crosby, The Crosby Group, Redwood City, Calif.)



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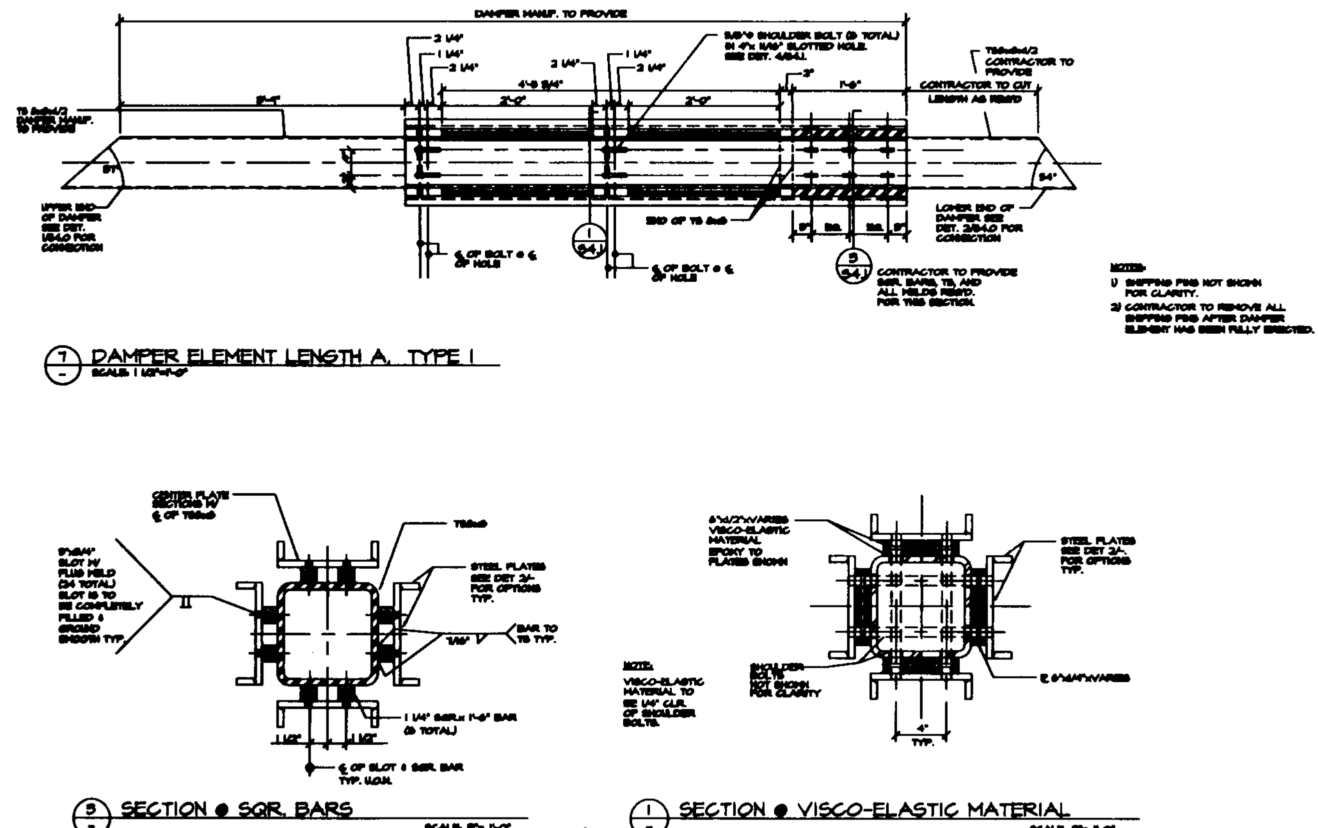


C

Figure 36.—Continued.

CHEVRON BUILDING (SAN FRANCISCO)

The 42-story Chevron Building at 575 Market St. is a steel, moment-resisting framed, slender, rectangular in-plan building. It has two levels of basements and is built on 10-m-long precast pile clusters. A three-dimensional schematic of the building and its instrumentation scheme is shown in figure 37 (Çelebi, 1992). Recorded accelerations at different levels of the building and corresponding displacements are shown in figure 38. System identification analyses results are shown in figure 39. Identified first-mode frequencies (periods) are 0.16 Hz (6.25 s) in the 225° direction and 0.21 Hz (4.76 s) in the 135° direction. Figure 40 shows coherency and phase-angle plots for (1) two parallel 225°-oriented motions (horizontal plane) at the roof, (2) two parallel 225° motions (in the vertical plane), and (3) two parallel 135° motions (also in the vertical plane) of the building. From these plots, we conclude that (1) torsion is insignificant since the motions in the horizontal plane at the roof are in phase and coherent, (2) 0.55 Hz and 1.0 Hz in the 225° direction are the second and third modal frequencies since the phase angles are 180° and -180° out of phase, and (3) similarly for the 135° direction, 0.61 Hz and 1 Hz are the second and third modal frequencies. The recorded peak accelerations and extracted frequencies (periods) and damping percentages are summarized in table 7 (Çelebi, 1992).

Şafak (1993) studied the building in detail using a system identification method based on the discrete-time linear-filtering and the least-squares estimation techniques. He concluded that higher modes contribute significantly to the overall response and that soil-structure interaction occurs at 1.0 Hz. Anderson and Bertero (this chapter) studied the building also and concluded that the building remained elastic during the earthquake. They attributed this to the fact that the designers of the building opted to use site-specific design response spectra that was more conservative than the minimum code requirements.

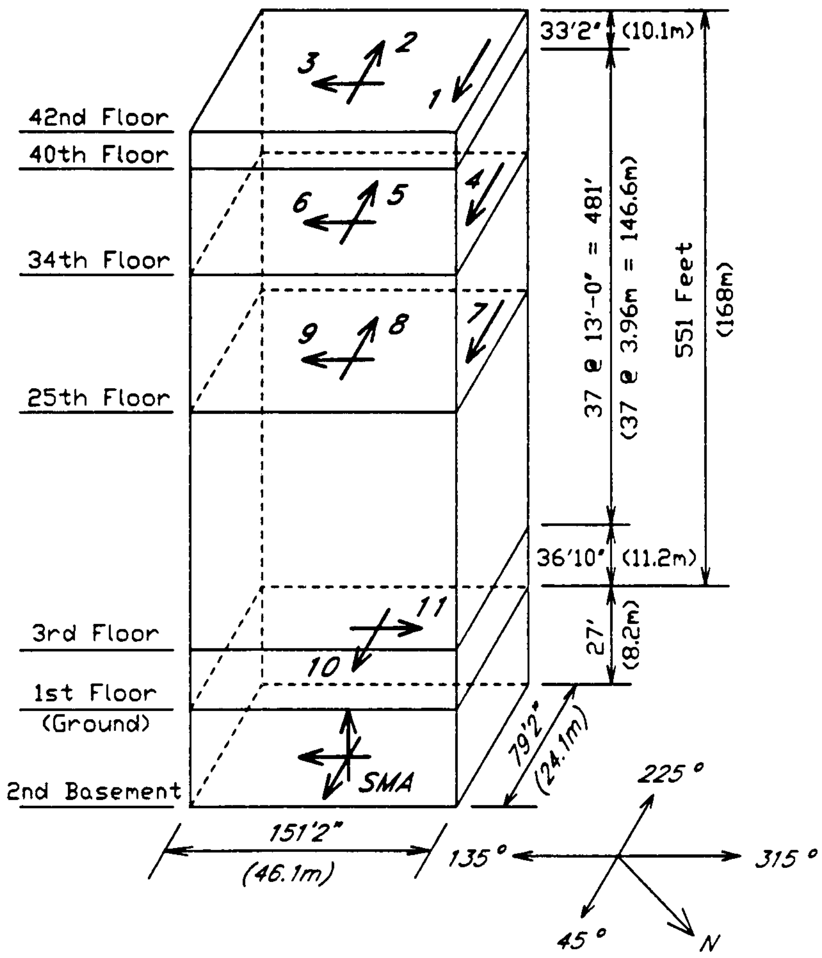


Figure 37.—General three-dimensional view and instrumentation scheme of Chevron Building, San Francisco.

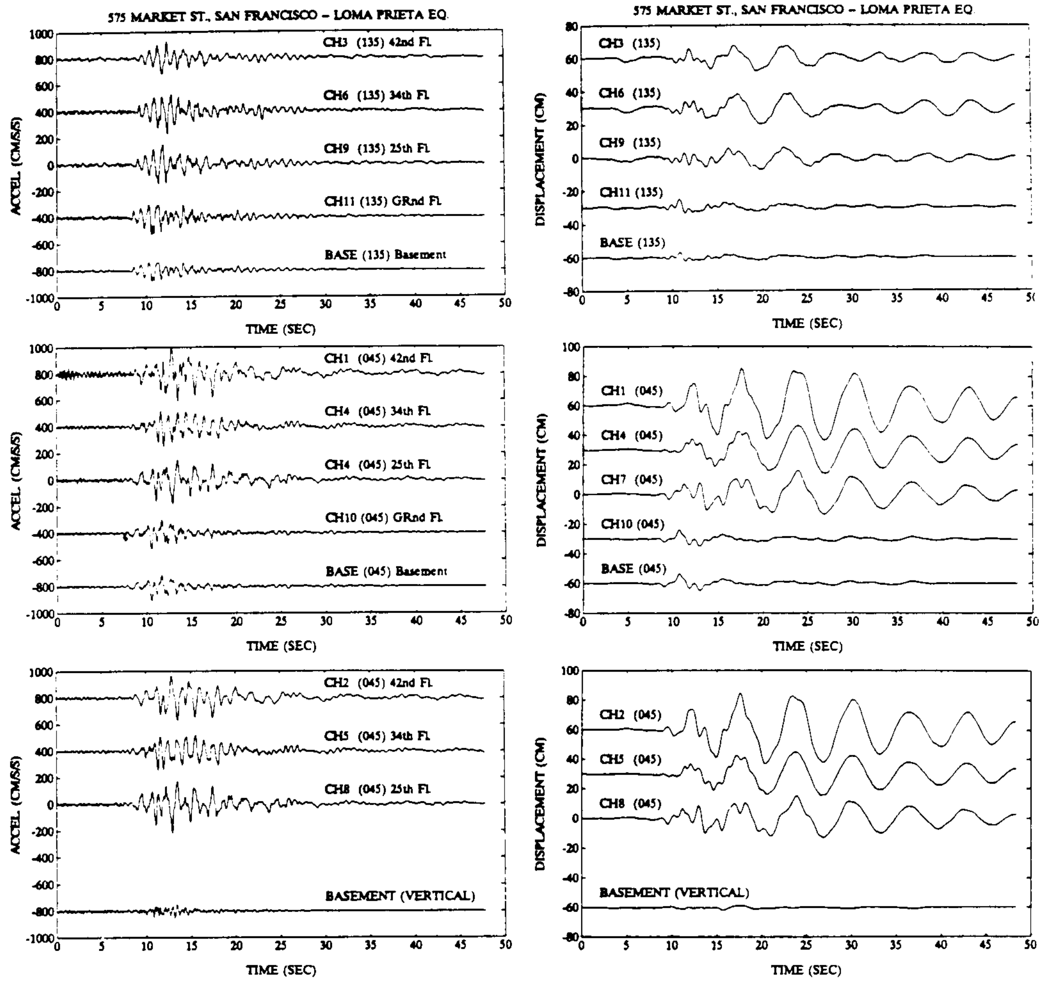


Figure 38.—Acceleration and displacement time-histories of Chevron Building.

Table 7.—*Peak responses and dynamic characteristics of Chevron Building*

	A. Peak responses			
	Accel. (g)		Displ. (cm)	
Roof	0.31		18.6	
Basement	0.12		3.3	
	B. Dynamic characteristics			
Mode	Direction	f (Hz)	T (s)	ξ (pct.)
1	225	0.16	6.25	4.1
	135	0.21	4.76	5.1
2	225	0.55	1.82	4.5
	135	0.61	1.64	3.4
3	225	0.98	1.02	3.6
	135	1.00	1.00	9.2

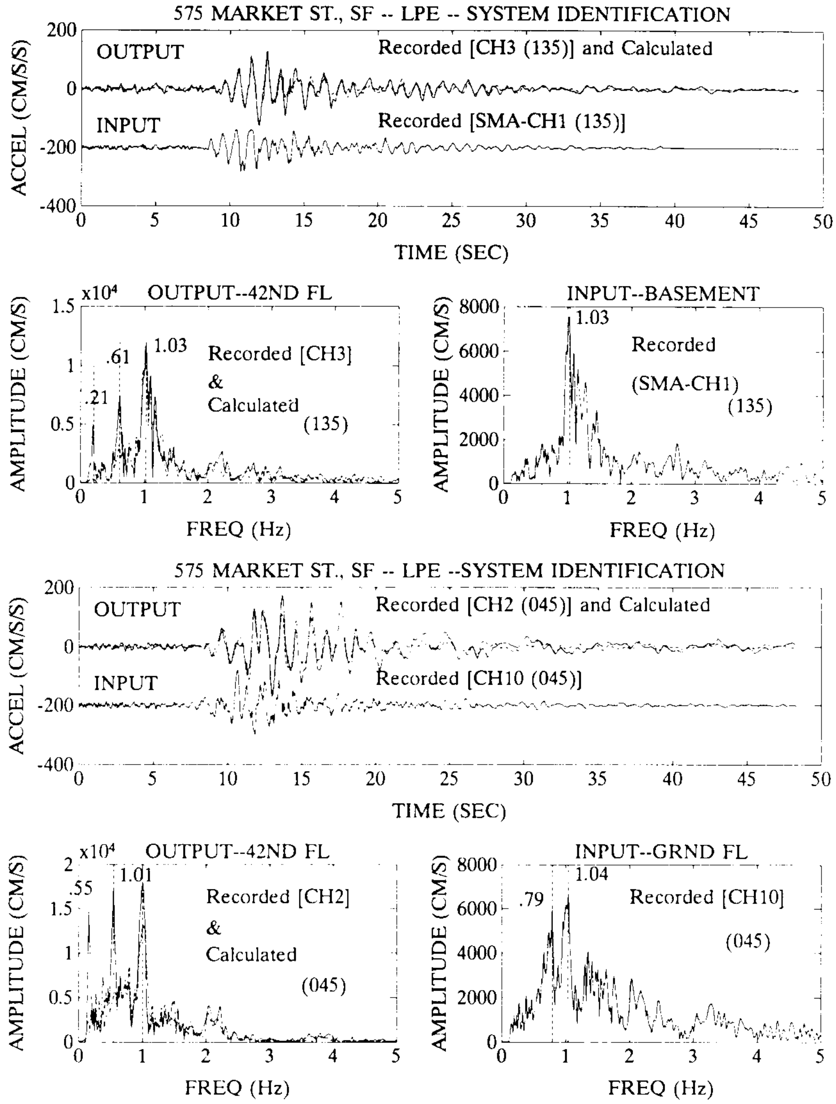


Figure 39.—System identification for Chevron Building.

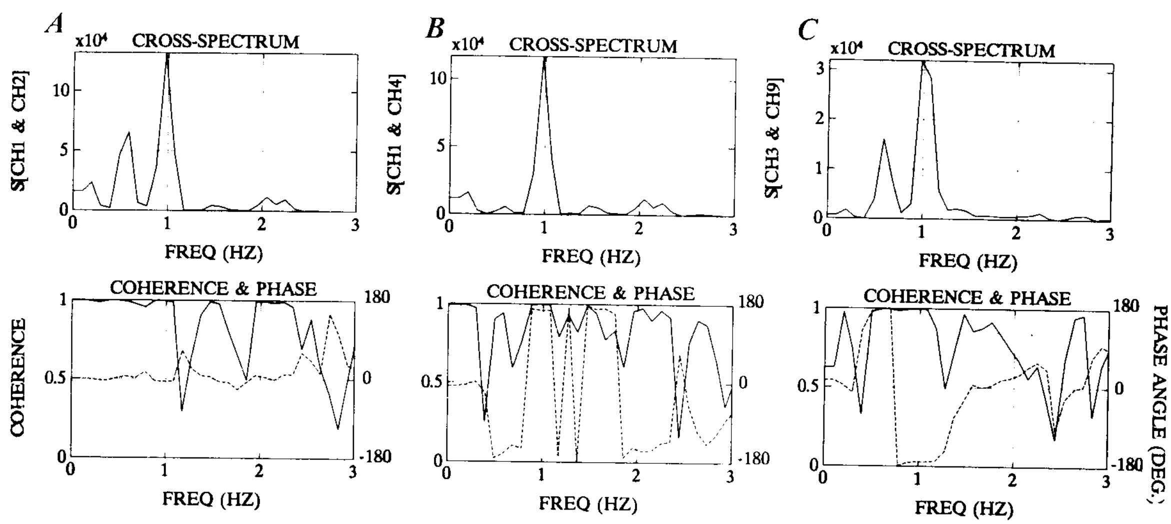


Figure 40.—Cross-spectrum, coherence, and phase angle plots for (*A*) parallel (225°) motions at 42d floor, (*B*) parallel (225°) motions at 42d and 34th floors, and (*C*) parallel (135°) motions at 42d and 34th floors.

MIXED CONSTRUCTION

CSUH ADMINISTRATION BUILDING (HAYWARD)

The 13-story California State University at Hayward (CSUH) Administration Building was constructed in 1971 in accordance with the Uniform Building Code (1967) design provisions. It is approximately 70 km from the Loma Prieta epicenter and is located within less than 5 km of the Hayward fault, also capable of generating large earthquakes. The general location of the building relative to the epicenter is shown in figure 1 (marked as HAYW). A general view of the building is shown in figure 41*A*. A three-dimensional view of the building and instrumentation scheme are shown in figure 41*B*. The availability of a free-field station on the university stadium grounds provides an opportunity to assess site and soil-structure interaction effects, if any. The building is 61.29 m high, with a structural system consisting of interior core moment steel frame and exterior perimeter concrete moment frame. The plan dimensions are 34.29×34.29 m. Up to the second floor, there are concrete shear walls around the elevator shafts. The building has a two-story extension bridge structure (enclosed on the second story) at its east side connecting it to an adjacent building (fig. 41*B*). The bridge structure is free to move on friction bearings at its juncture with the adjacent structure. The building sits on bearing piles with a 45-cm-thick reinforced concrete mat on grade (Çelebi, 1994b).

Channels 2, 7, and 10 (parallel sensors in the north-south direction at the roof, 2d, and 1st floors, respectively) (fig. 41*B*) malfunctioned during the earthquake (Shakal and others, 1989); therefore, identification of torsional motions of the structure cannot readily be made.

The processed 40 seconds of the recorded acceleration and the displacements at different levels are shown in figure 42. The data from the building have been band-pass filtered with ramps at 0.2-0.4 Hz and 23-25 Hz (Shakal and others, 1989). The band-pass filter of the free-field data has a low-frequency ramp at 0.08-0.16 Hz. It is noted that the peak accelerations and displacements summarized in table 8 show significant differences, particularly in the displacement of the free-field versus the basement, due to the 0.08-0.16 Hz band-pass ramped filter of the free-field records. Also, longer periods are distinct in displacement plots of the free-field in figure 43. Such filtering errors and inconsistencies can provide misleading interpretations and can lead to wrong results in calculations using displacement (for example, drift).

In figure 44, calculated output of system identification analysis using 40 seconds of the basement accelerations (recorded input) and roof accelerations (recorded output) of the building are shown. System identification calculations were also performed using the 5th floor accelerations as recorded output. The three significant frequencies (periods) and damping values also determined from system identification procedures are summarized in table 9. Figure 45 shows the structural frequencies in the spectral ratios calculated from amplitude spectra of roof and basement motions. It is noted that the three significant modal periods in each of the principal axes of the building follow the approximation of T, T/3, and T/5.

For all three modes in each of the two principal axes, modal damping percentages determined by system identification vary between 1.3 and 6.4 percent (table 9).

A

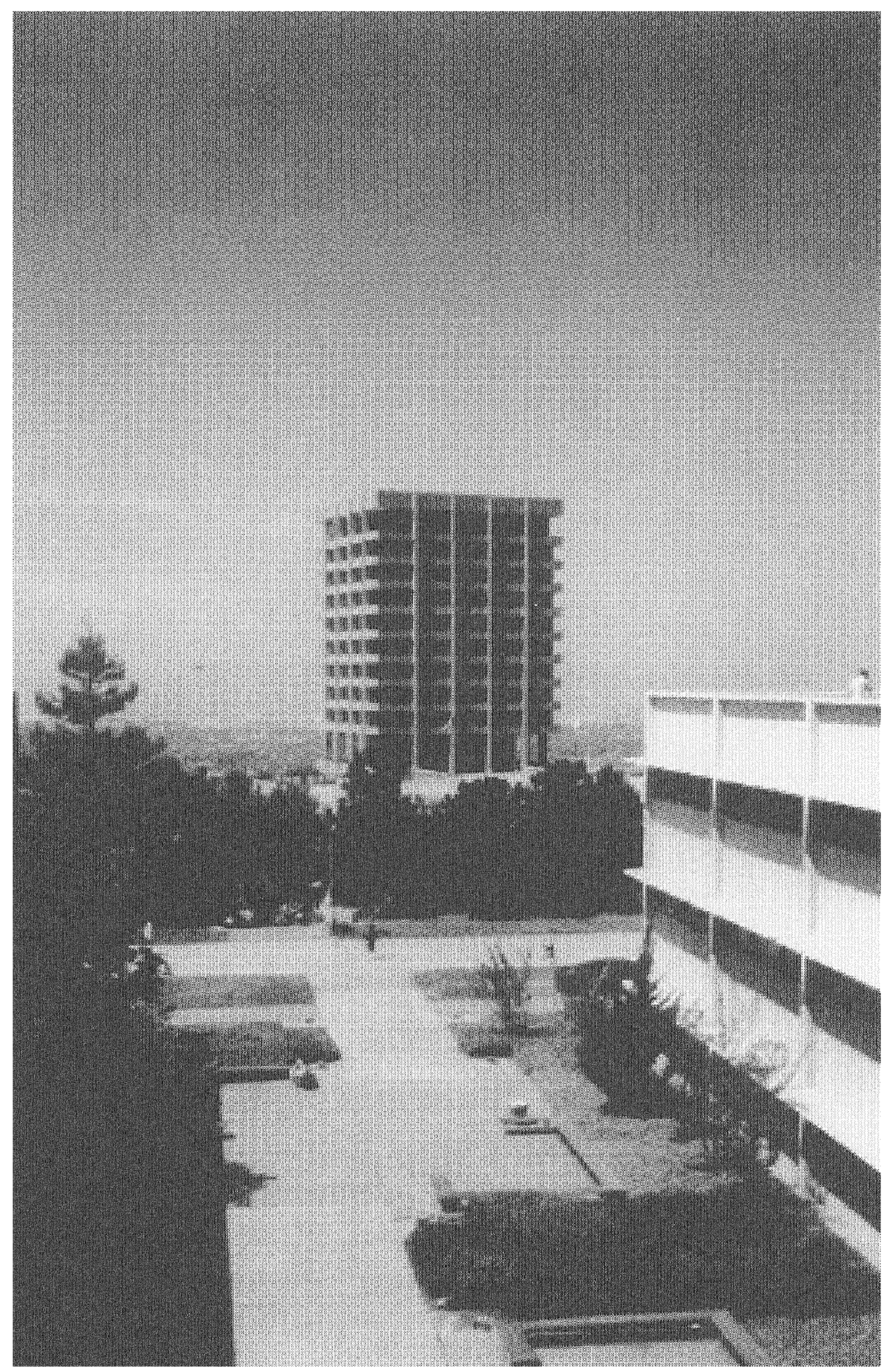


Figure 41.—*A*, California State University (Hayward) Administration Building. *B*, Three-dimensional schematic of building

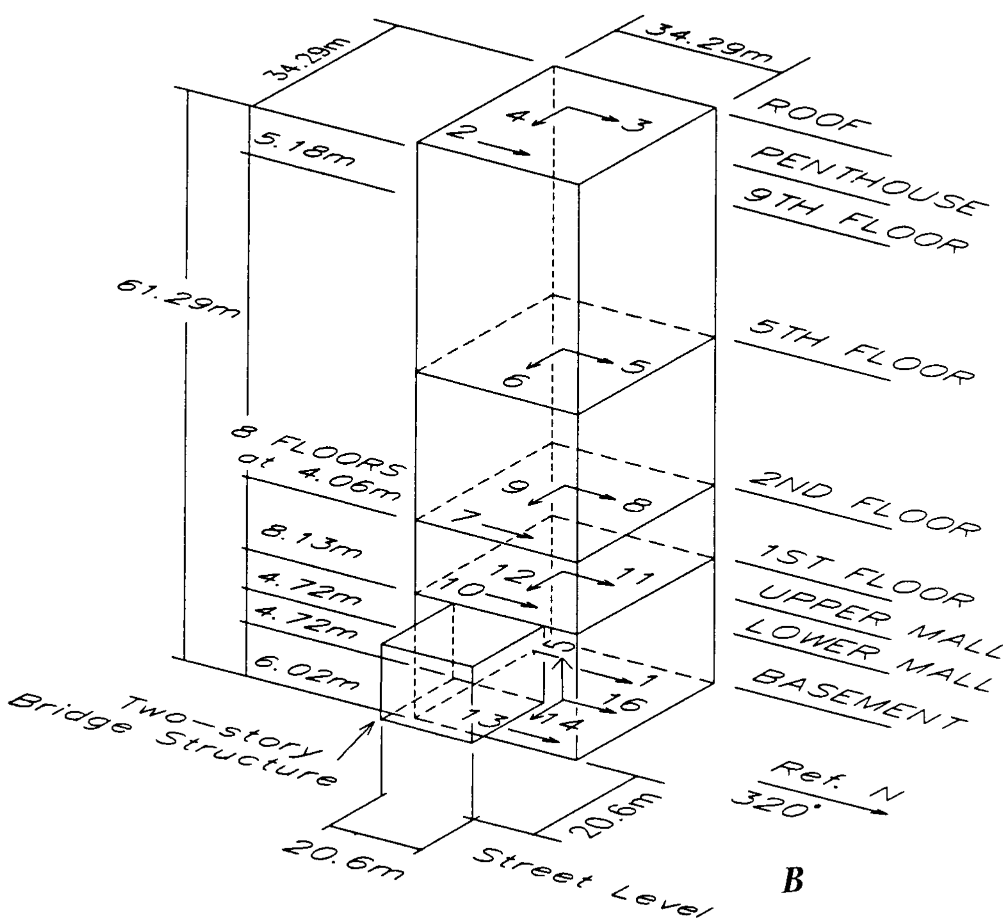
The possibility of torsion was investigated by coherence function and phase angle plots as shown in figure 46. At 0.76 Hz, the two horizontal orthogonal motions at the roof are not coherent and are not in phase, which may imply that torsion is insignificant. However, this may be misleading if the two horizontal sensors are located at or close to the center of rigidity of the floor, in which case the torsional contributions would not show in the unidirectional motions. Thus, the importance of acquiring two parallel motions at a floor is emphasized.

Figures 46*B* and *C* show the peaks of the first three modes in the cross-spectrum of north-south and east-west motions at the roof and 5th floor, respectively, indicating that the coherence is unity for all modes and the phase angles are 0° at the fundamental frequency and 180°at the second and third modal frequencies.

Figures 47*A* and *B* show 5 percent damped response spectra of the north-south and east-west free-field and basement motions of the building. Figure 47*C* shows the basement north-south and east-west response spectra superimposed. It is clear from these spectra that both free-field and basement have peaks in the range 0.25-0.3 s (3.3-4.0 Hz). Figure 47*C* shows roof and basement response spectra. Clearly, the first three modal frequencies (periods) can be identified directly from these spectra.

In the absence of a geotechnical report, available geotechnical and geological data of the strong-motion station site at the university stadium are used. It is assumed that the site conditions at the Administration Building are similar to those at the stadium. The top 3-4 m consists of sandy clay and clay (estimated V_s of 175 m/s) followed by 9-10 m of deeply weathered rhyolite (V_s of 315 m/s), followed by moderately weathered rhyolite (V_s of 825 m/s) (Fumal, 1991). The recommended average (to 30 meter depth) V_s is 525 m/s (Fumal, personal commun., 1991).

Due to absence of field measurements of site period, and considering only the top layer, a site period of approximately 0.1 seconds is estimated using $T_s = 4H/V_s$. This estimated period implies that this particular building would not be significantly influenced by the site effects. Furthermore, no site amplification is expected.



B

Figure 41.—Continued.

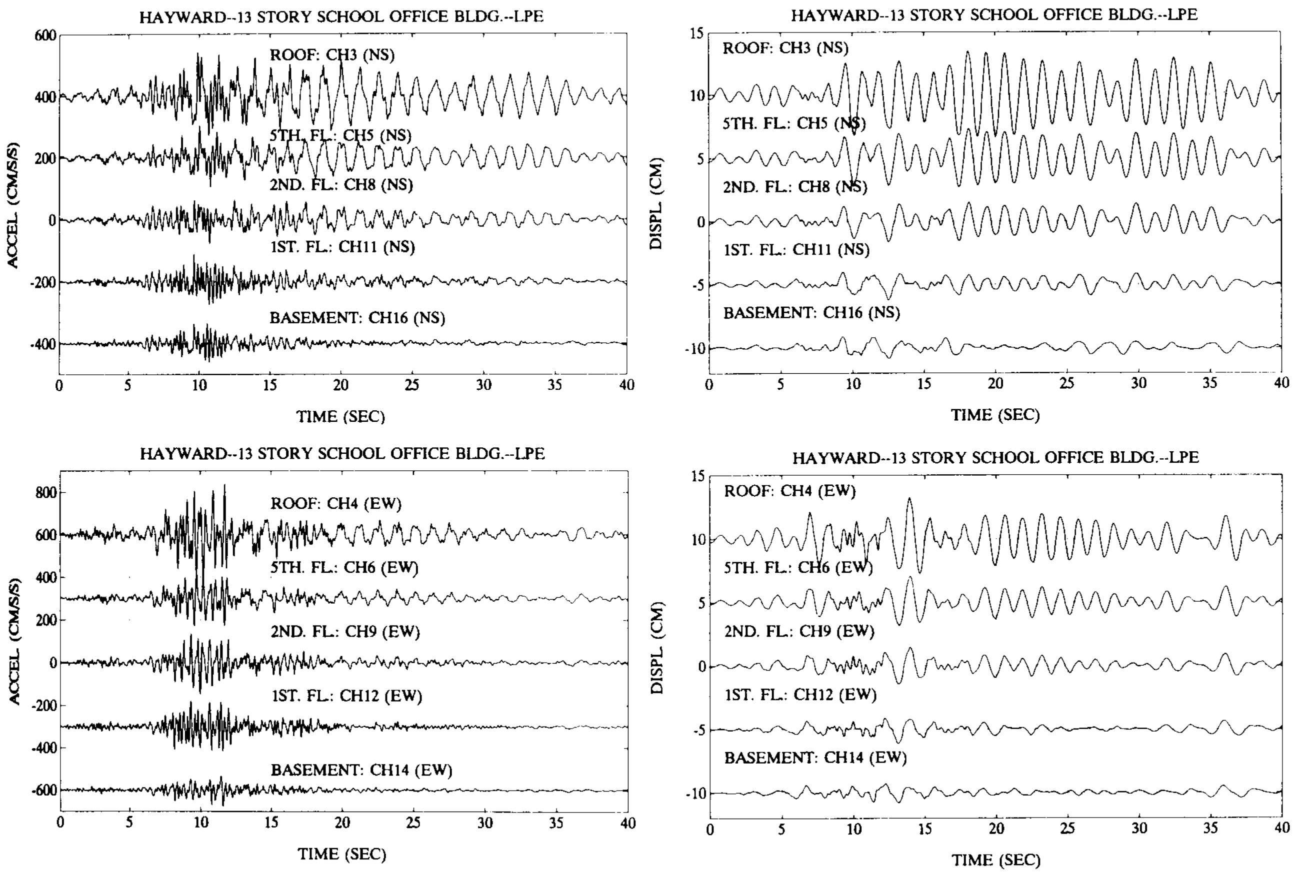


Figure 42.—Acceleration and displacement time-histories of California State University (Hayward) Administration Building.

Table 8.—*Accelerations and displacements at California State University (Hayward) Administration Building*

Location	Sensor	Direction	Accel. (g)	Displ. (cm)
Roof	3	NS(320°)	0.142	3.50
	4	EW(050°)	0.239	3.25
5th. floor.	5	NS	0.105	2.33
	6	EW	0.128	2.07
2d floor.	8	NS	0.078	1.69
	9	EW	0.146	1.50
1st floor.	11	NS	0.090	1.24
	12	EW	0.122	1.13
Basement	1	NS	0.069	0.90
	13	NS	0.065	0.89
	16	Ns	0.067	0.88
	14	EW	0.077	0.80
	15	UP	0.042	0.53
Free Field	FF1	EW (090°)	0.083	2.79
	FF2	UP	0.044	2.29
	FF3	NS(360°)	0.073	2.74

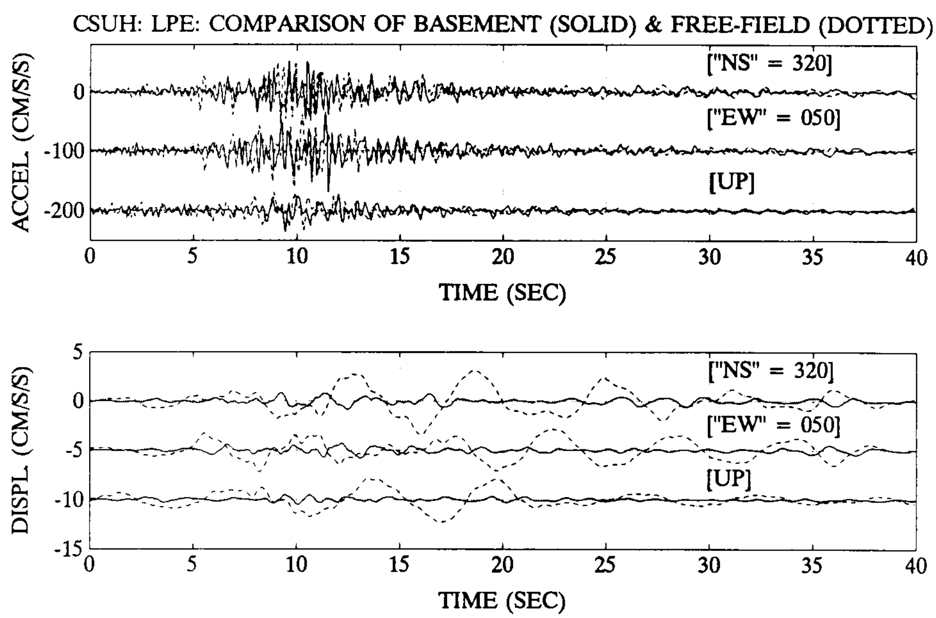


Figure 43.—Comparison of basement (solid line) and free-field (dash-dot lines) motions for California State University (Hayward) Administration Building.

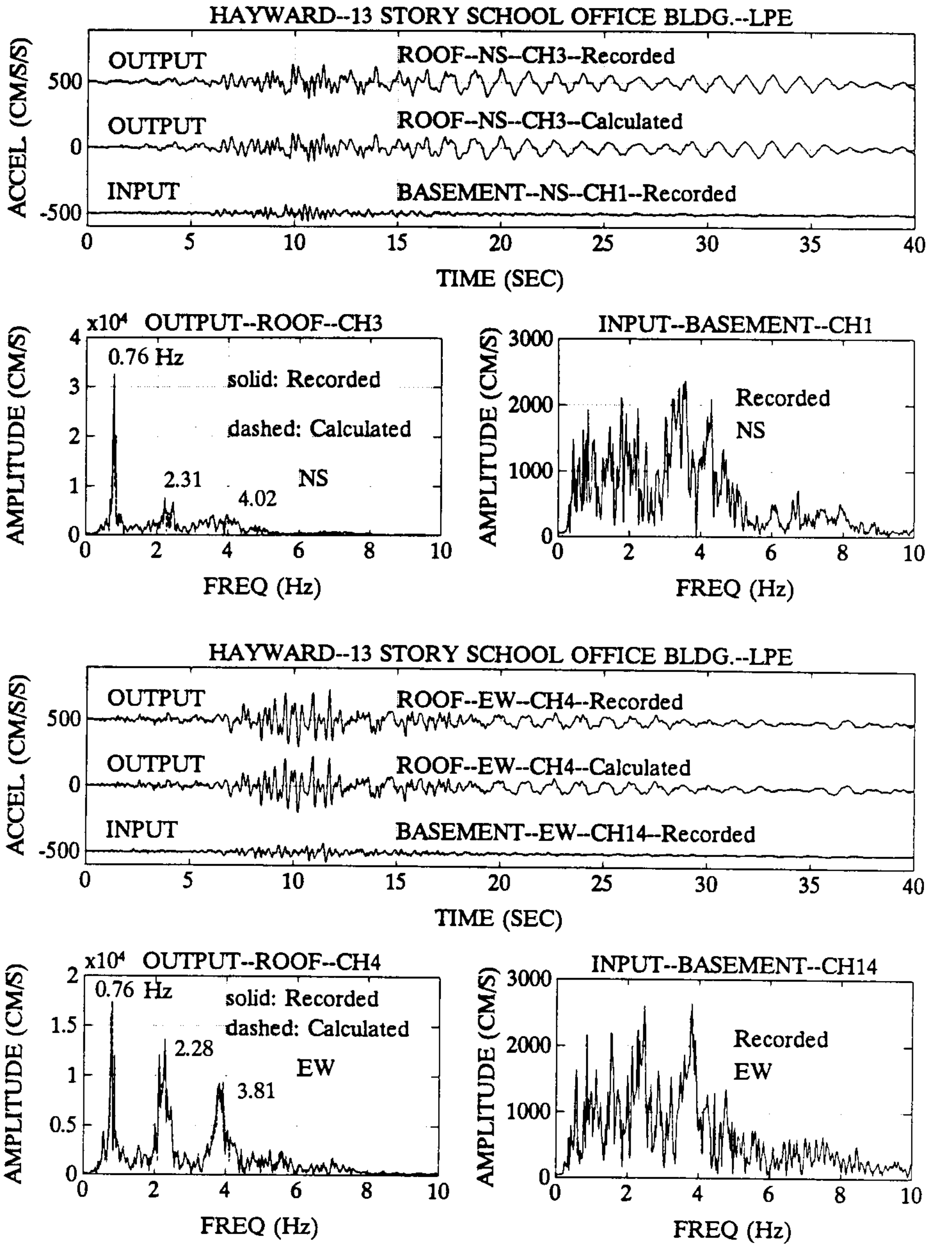


Figure 44.—System identification for California State University (Hayward) Administration Building.

Table 9.—*Identified dynamic characteristics for California State University (Hayward) Administration Building*

MODE	NS f (Hz)	NS T (s)	NS ξ (pct.)	EW f (Hz)	EW T (s)	EW ξ (pct.)
1	0.76	1.32	3.4	0.76	1.32	2.3
2	2.37	0.42	3.4	2.20	0.45	3.9
3	4.02	0.25	6.4	3.95	0.25	4.7

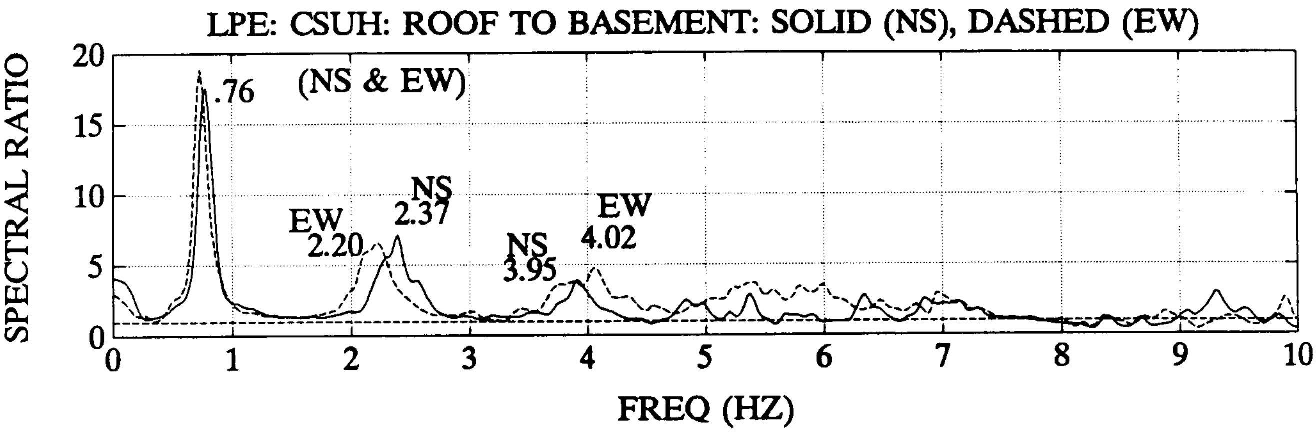


Figure 45.—Spectral ratios for California State University (Hayward) Administration Building.

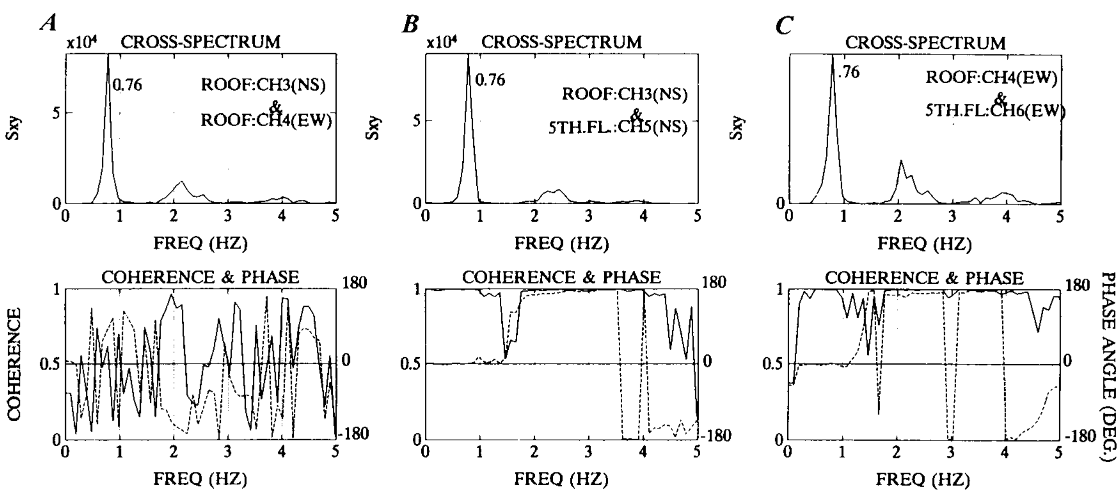


Figure 46.—Cross-spectrum, coherence, and phase-angle plots for California State University (Hayward) Administration Building.

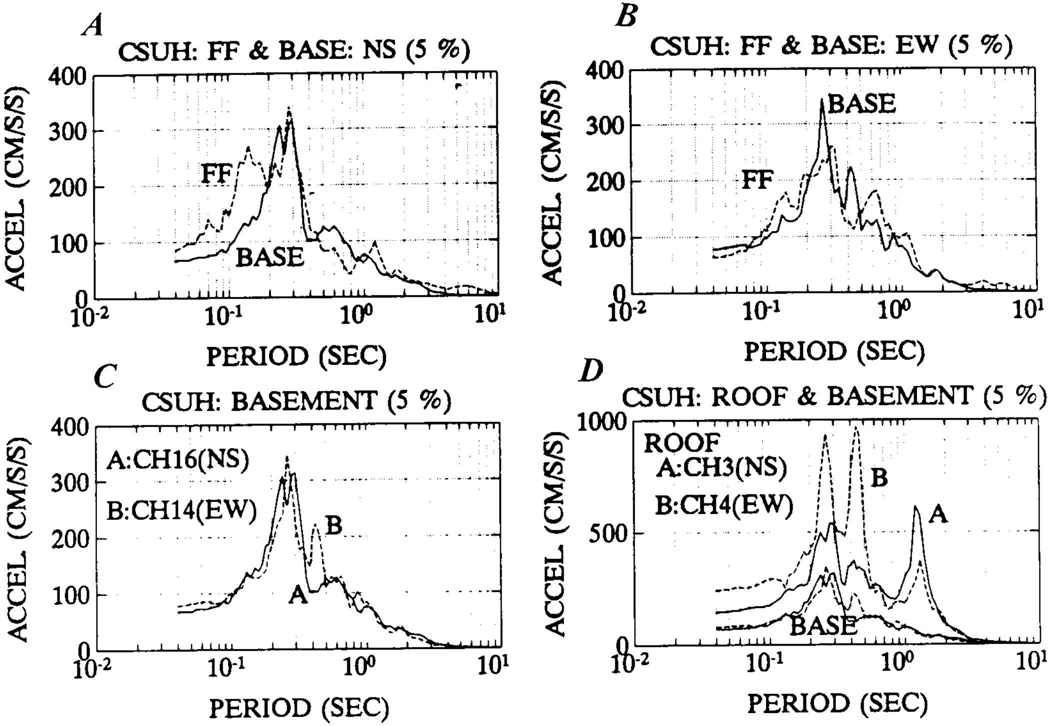


Figure 47.—Comparison of response spectra for California State University (Hayward) Administration Building. *A*, *B*, Free-field vs. basement. *C*, Orthogonal motions of basement. *D*, Roof and basement.

TWO-STORY OFFICE BUILDING (OAKLAND)

McClure (1991) analyzed the data from the two-story Oakland office building designed by him in 1964. The building was designed according to the 1961 Oakland City Building Code (same as 1961 Uniform Building Code). The building is essentially structural steel framed with reinforced concrete block masonry infill walls. A three-dimensional schematic of the building, its dimensions, and the instrumentation scheme is shown in figure 48 (Çelebi, unpub. data, 1997). As showm in figure 48, the building had a severe plan torsional irregularity. The building was subjected to large peak accelerations (ground 0.26 g, second-floor 0.54 g, and roof 0.69 g). Recorded accelerations and their peaks are shown in figure 49. The building suffered no damage. Its inherent stiffness provided by infill walls reduced the drift ratio. The three-dimensional computer model analyses by McClure (1991) showed that the building behaved elastically when subjected to Loma Prieta motions. The time-histories of the translational and torsional roof accelerations and corresponding amplitude spectra are shown in figure 50 (Çelebi, unpub. data, 1997). The amplitude spectra by themselves reveal frequencies at approximately 0.8, 1.7, and 1.95 Hz. To distinguish these frequencies, spectral ratios are shown in figures 51*A* and *B*. It is noted that the spectral ratio in the north-south direction is approximately unity up to 2 Hz because the east wall of the building does not amplify the structural response (figure 51*B*). The building was subjected to severe torsional behavior that is close-coupled with the translational mode at approximately 1.67 and 2 Hz (0.67 and 0.5 s). These frequencies are identified in the spectral ratios of torsional accelerations of the roof (figs. 51C and D). The weak peak at approximately 0.7 Hz observed in the amplitude spectra is attributed to the site frequency (Çelebi, unpub. data, 1997). McClure (1991) stated that the ambient vibration tests of the building performed in 1965 by the USGS (then U.S. Coast and Geodetic Survey) indicated a period of 0.47 s for the building. Later, in 1966, forced vibration tests of the building by Bouwkamp and Blohm (1966) yielded a first-mode period of 0.416 s. The mathematical model McClure used to perform dynamic analyses of the building was based on matching the period of the building to one of those from the low-amplitude tests. However, McClure noted that the building periods were 0.5-0.6 s during Loma Prieta and attributed this to possible disengagement of the nonstructural elements that were not well connected to the structural frame (1991).

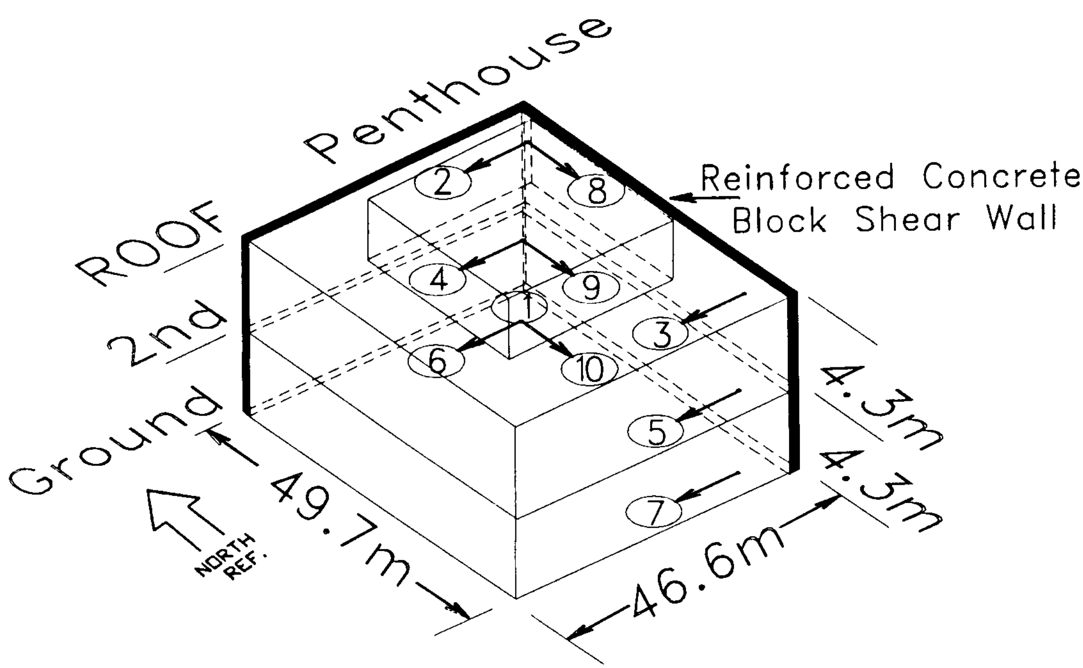


Figure 48.—Three-dimensional schematic of two-story building in Oakland.

TILT-UP BUILDINGS AND BUILDINGS WITH FLEXIBLE DIAPHRAGMS

Particularly used as industrial and storage facilities, tilt-up buildings commonly are designed with large aspect ratios and flexible long span plywood roof diaphragms. Bouwkamp and others (1991) studied some of the buildings with flexible diaphragms, including a warehouse in Hollister (which also has records from the 1984 Morgan Hill earthquake and the 1986 Hollister earthquake), West Valley College gymnasium in Saratoga (which has records also from the 1984 Morgan Hill earthquake), and a two-story building in Milpitas (which also has records from the 1988 Alum Rock earthquake). The availability of data from several earthquakes for these tilt-up buildings allowed comparison of the frequencies and the loss of stiffness and the amplification of motions at the center of the roof diaphragm compared to its edges and the base. Bouwkamp and others (1991) reported that the stiffness of the Hollister building during Loma Prieta was approximately 50 percent of that calculated from the records of the 1984 Morgan Hill earthquake. The amplifications of motions at the walls were negligible, as they should be

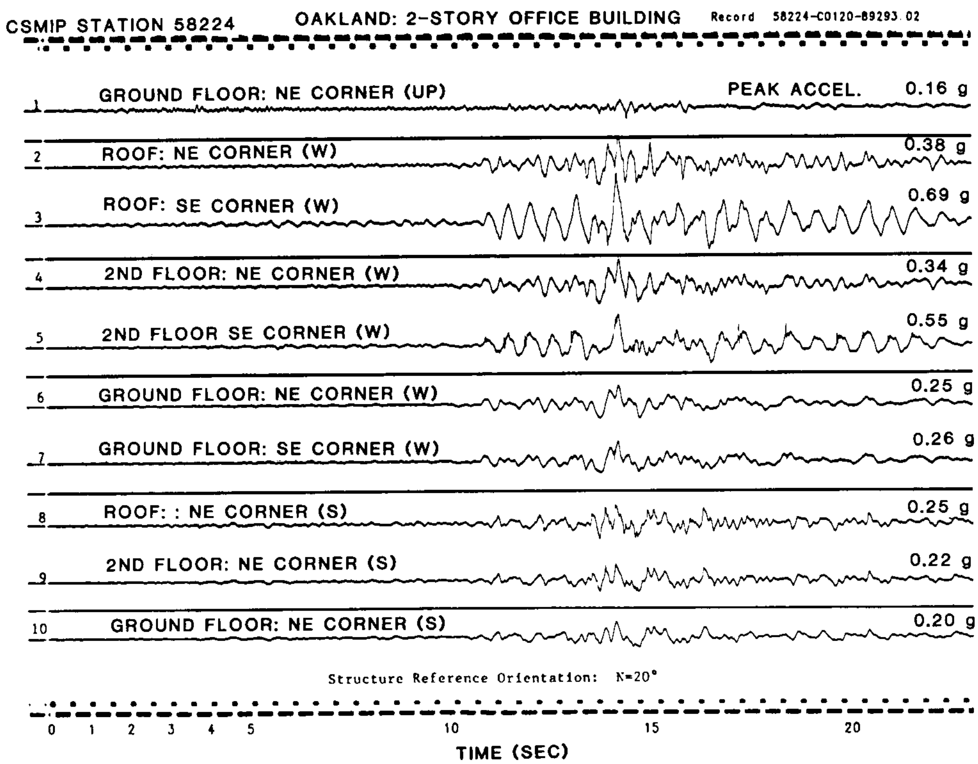


Figure 49.—Recorded responses of two-story building in Oakland.

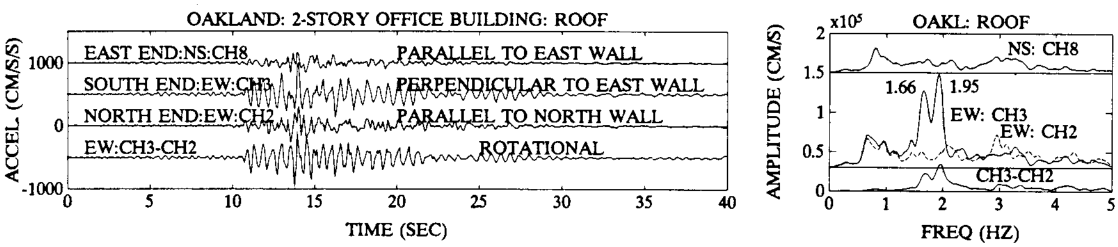


Figure 50.—Roof responses and amplitude spectra of two-story building in Oakland.

since the walls are extremely rigid in their planes. Furthermore, the code equations for estimating the diaphragm displacement did not match well with the recorded responses.

The West Valley College gymnasium was studied in detail by Çelebi and others (1989) using the 1984 Morgan Hill earthquake records. A general view and schematic of the gymnasium and its instrumentation scheme is shown in figure 52 (Çelebi and others, 1989). The Morgan Hill earthquake records are compared to the Loma Prieta records in figure 53. In short, the records of the building are greatly influenced by the diaphragm frequency at approximately 4 Hz. Significant peak accelerations of the gymnasium are summarized in table 10 for the two earthquakes (Çelebi, 1990). As a result of these studies, the design requirements for the restraints at the edges of the roof diaphragm were increased in the 1991 Uniform Building Code by 50 percent.

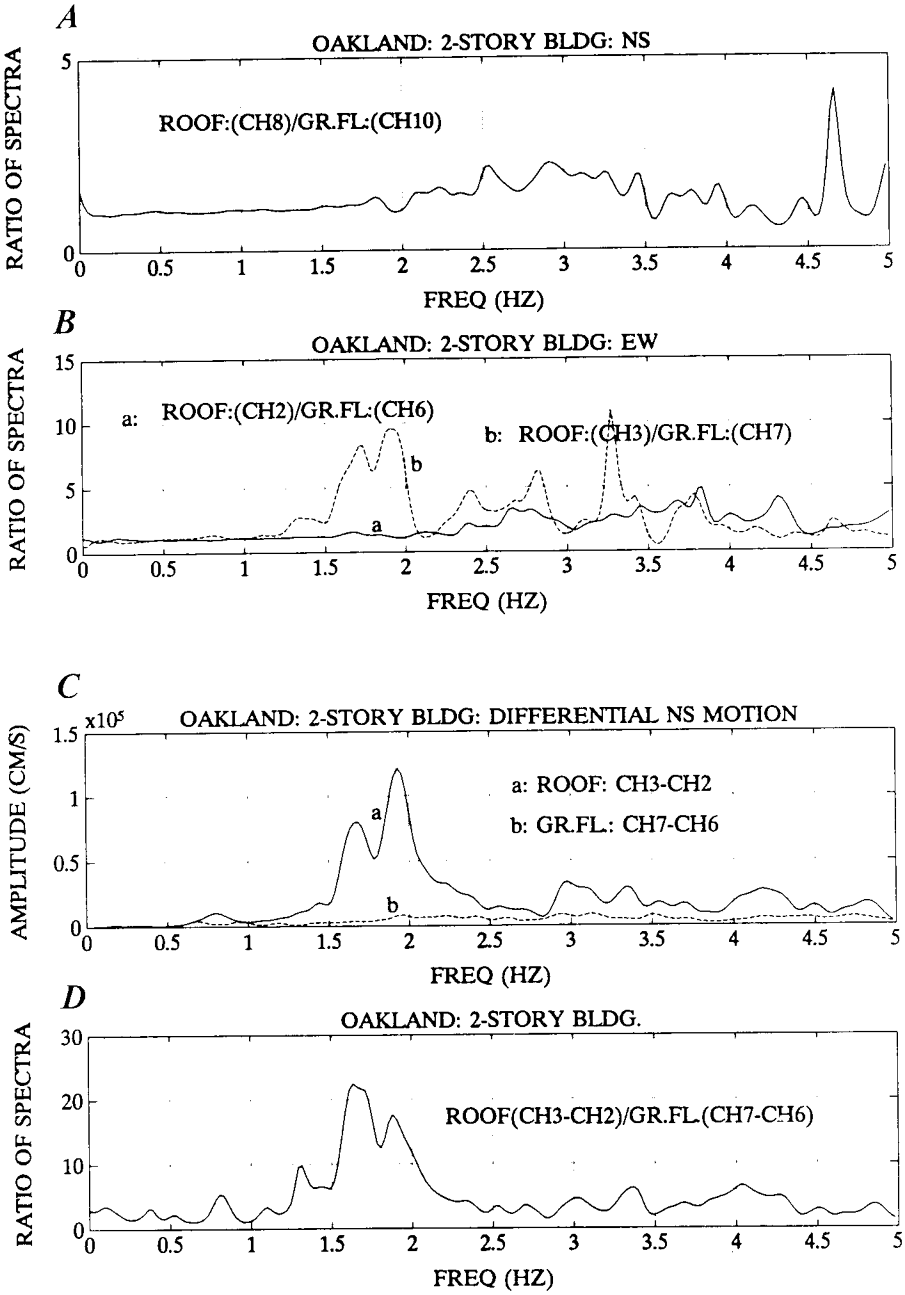
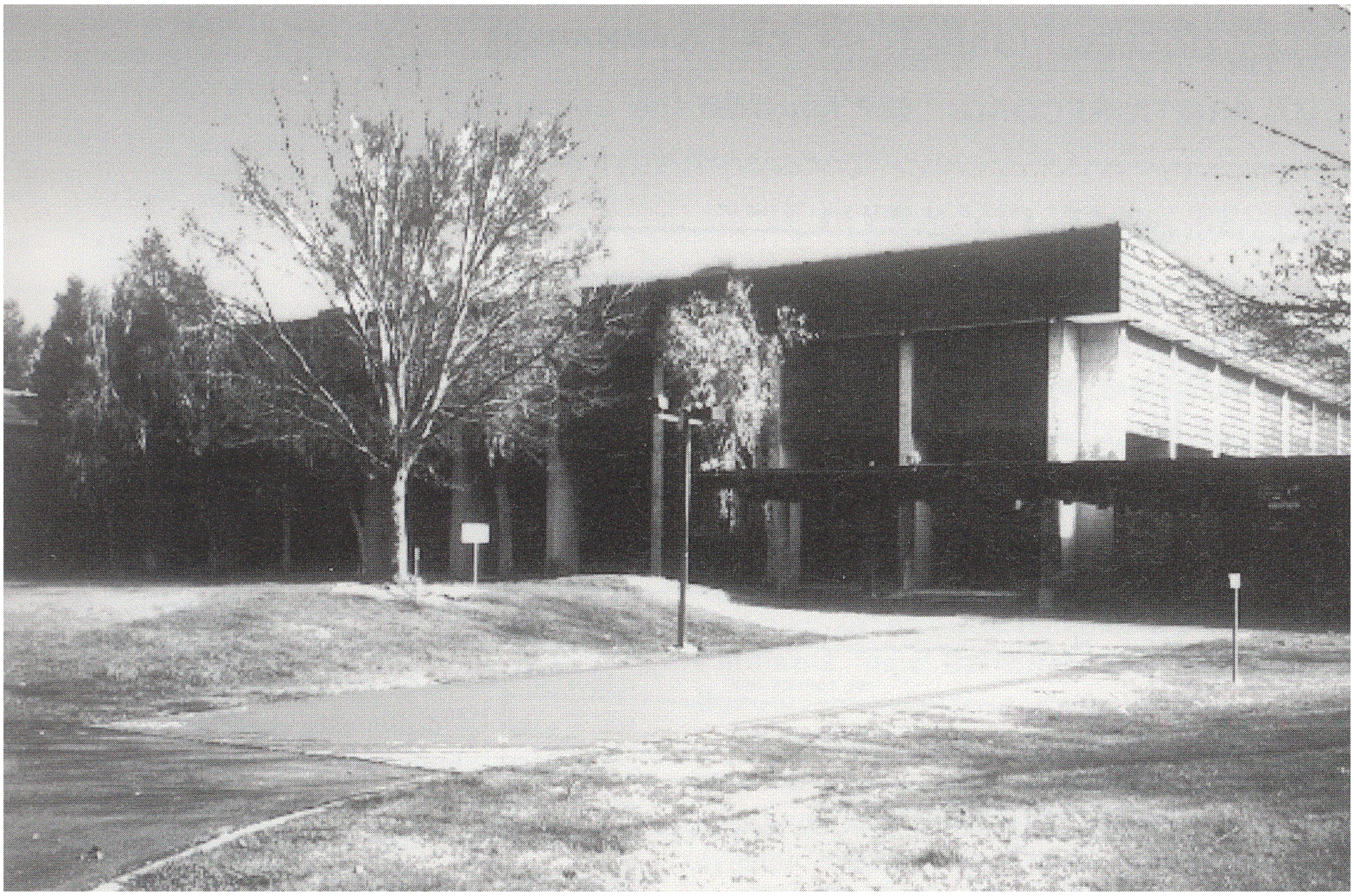


Figure 51.—Spectral ratios of amplitude spectra of two-story building in Oakland.



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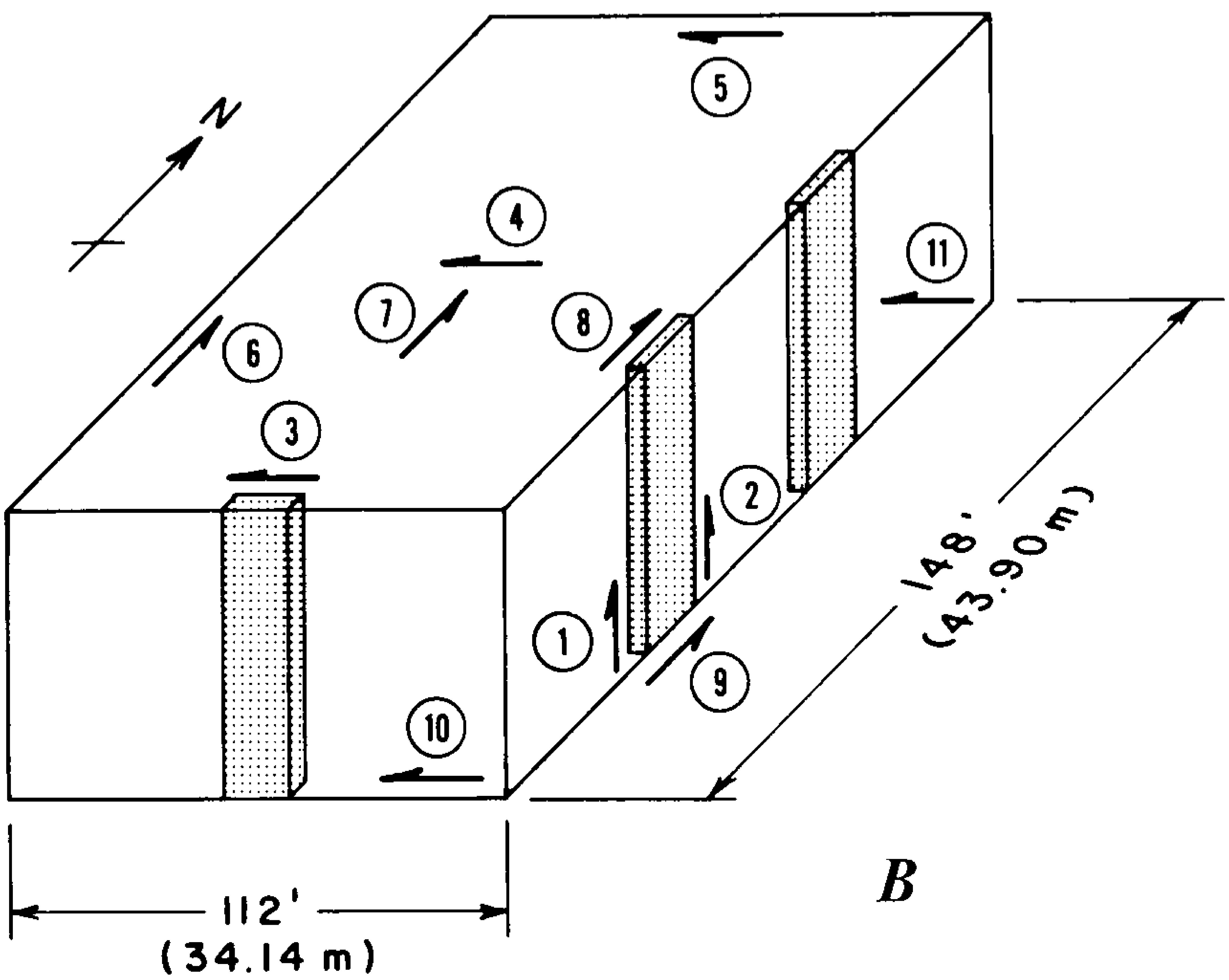

B

Figure 52.—*A*, West Valley College Gymnasium. *B*. Three-dimensional schematic of West Valley College Gymnasium.

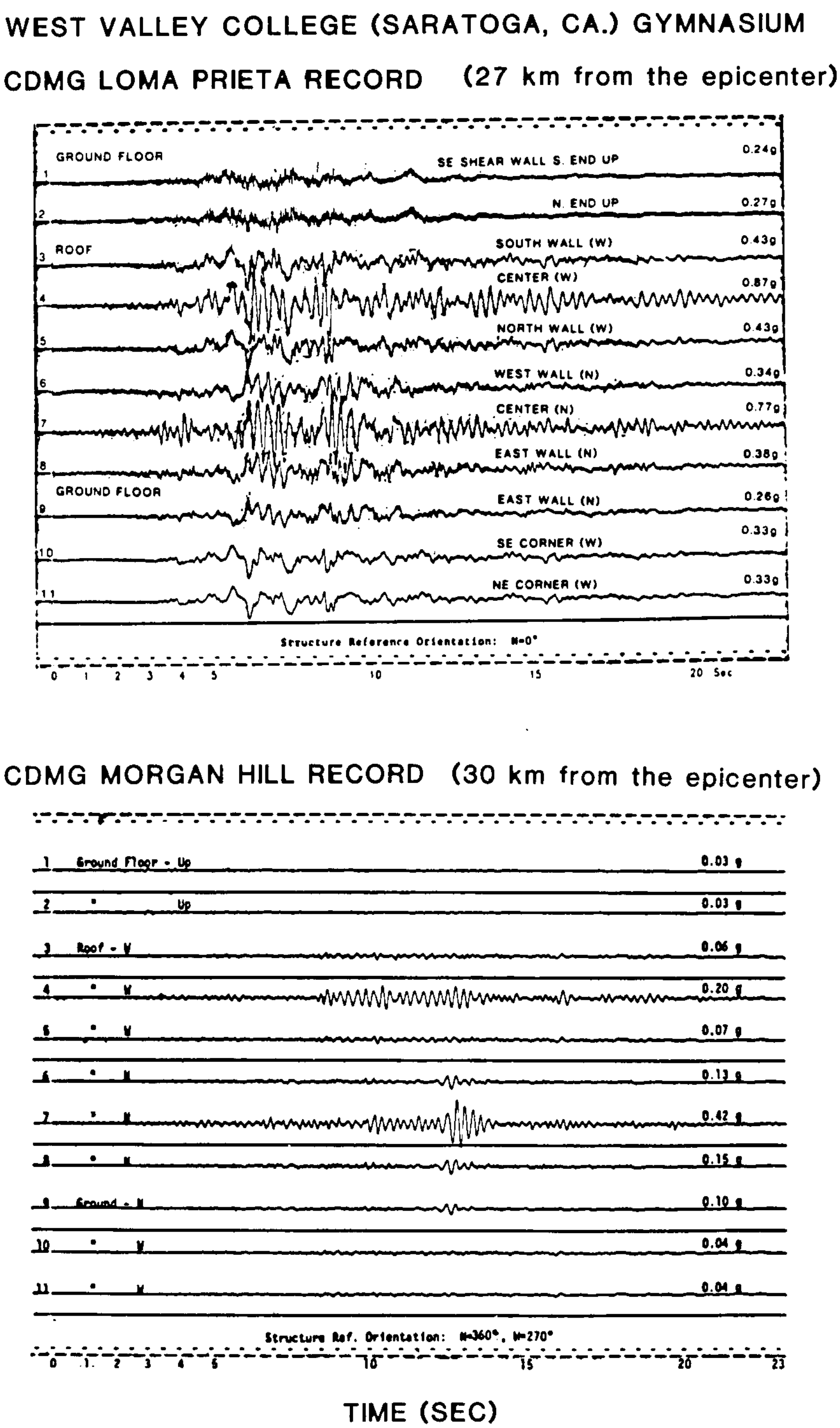

Figure 53.—Recorded responses of West Valley College Gymnasium for two earthquakes.

Table 10.—*Peark acceleration responses for two earthquakes from West Valley College (Saratoga) Gymnasium*

		Morgan Hill M_s =6.1 April 24, 1984	Loma Prieta M_s =7.1 October 17, 1989
Location	Direction	Accel (*g*)	Accel. (*g*)
Ground	NS	0.10	0.26
	EW	0.04	0.33
Roof edge	NS	0.14	0.36
	EW	0.065	0.43
Roof center	NS	0.42	0.77
	EW	0.20	0.87

Wood and Hawkins (this chapter) also investigated the trends in seismic behavior of two tilt-up buildings (the same two-story building in Milpitas and the same one-story building in Hollister). Both buildings were constructed by different methods, and both were relatively new buildings (constructed according to codes between 1979 and 1989). Both buildings are within 50 km of the Loma Prieta epicenter and both recorded similar responses. The authors report that the transverse accelerations at the center of the roof of both buildings were approximately three times that at the base of the buildings. This indicated that such buildings designed according to codes prior to Loma Prieta resulted in very flexible diaphragms. Therefore, the 1991 changes made in the Uniform Building Code are warranted.

The findings of Wood and Hawkins (this chapter) are similar to those of Tena-Colunga and Abrams (1992) and Abrams (1995), who studied the flexible diaphragm problem of a two-story, masonry office building located at Palo Alto, California. The latter investigations provide further insight into the role of floor or roof diaphragms in dynamic response of unreinforced masonry building systems and into deliberations on the impact of these responses to the design of masonry buildings.

BUILDINGS WITH NEW TECHNOLOGIES

At the time of the Loma Prieta earthquake, there were no buildings in the greater San Francisco Bay area that were constructed or retrofitted with base-isolation technology. Only an overpass bridge (Sierra Point in South San Francisco) had been retrofitted with base isolation (Kelly and others, 1991). However, since Loma Prieta, a number of base isolated buildings have been either recently constructed (for example, San Francisco Main Library Building) or retrofitted with this technology (for example, Court of Appeals Building in San Francisco, a four-story apartment building in the Marina District of San Francisco, and Oakland City Hall). Also, the Santa Clara County Office building in San Jose has been retrofitted with viscous elastic dampers (see section "Santa Clara County Office Building (San Jose)") (Crosby and others, 1994).

LOW-AMPLITUDE TESTING

The dynamic characteristics of five of the previously discussed buildings in the San Francisco Bay area that recorded the Loma Prieta earthquake were studied by recording their responses to low-amplitude ambient vibratory motions (Marshall and others, 1991, 1992; Çelebi and others, 1993; Çelebi, 1996). The five buildings studied are: (1) the Pacific Park Plaza Building , (2) the Transamerica Building, (3) the Santa Clara County Office Building , (4) an office building in San Bruno, and (5) the California State University (Hayward) Administration Building .

The primary reason for selecting these five buildings was that they recorded the earthquake and therefore low-amplitude tests provide an opportunity to compare their dynamic characteristics under strong- and low-amplitude motions. The Loma Prieta response of all these buildings has been investigated by several investigators, as summarized earlier in this paper.

The location of these five buildings relative to the Loma Prieta epicenter is shown in figure 1. The general dimensions and instrumentation schemes of each of the five buildings are provided previously in this paper. However, significant characteristics of these buildings (structural type, foundation type, and general dimensions), distance from the epicenter, number of channels of strong-motion sensors within the superstructure, and peak accelerations at ground level (or foundation level) and at roof level are summarized in table 11. The reference north building orientation in degrees clockwise from true north, provided in the table, is different than the adopted nominal north-south and east-west directions shown in the table. Results from dynamic analyses and low-amplitude tests performed on the Transamerica Building (Stephen and others, 1974; Kinemetrics, 1979) and Pacific Park Plaza Building (Stephen and others, 1985) prior to the earthquake and dynamic characteristics for the Santa Clara County Office Building extracted from response data of two earthquakes that occurred prior to the earthquake (R. Darragh, personal commun., 1991) are included in comparative studies by Marshall and others (1991, 1992), Çelebi and others (1993), and Çelebi (1994a).

The testing of each building was conducted from a recording room that contained the junction box of the cables of the force-balance accelerometers permanently deployed throughout the superstructure. These cables were hooked up to a digital (PC-based) data acquisition system (Marshall and others, 1991, 1992; Çelebi and others, 1993). This (1)

Table 11.—*Building characteristics and Loma Prieta peak accelerations*

[Data from Marshall and others (1992) and Çelebi (1996). The reference north building orientation in degrees clockwise from true north is different from adopted nominal north-south and east-west directions. Number of floors (N_A, above ground level; N_B, below ground level). Height of building=H. Distance to epicenter=D. Number of channels in instrumented building=n. Orientation of reference north (clockwise from true north)=N]

Building (instrumentation administrator, in paranthesis)	Comments	H (m)	N_A/N_B	D (m)	n	Loma Prieta Peak Accel. (g)		
						Nom. direc.	Gr. flr.	Roof
Pacific Park Plaza (USGS) N=350°	Reinforced concrete moment resisting frame (1.5-m-thick concrete mat on piles).	94	30/1	97	21	NS EW UP	0.17 0.21 0.06	0.24 0.38
Transamerica Bldg. (USGS) N=351°	Steel frame, 48th floor is the top occupied floor (2.75-m-thick concrete mat - no piles).	257	60/3	97	22	NS EW UP	0.11 0.12 0.07	0.29 0.31
Santa Clara County Office Building (CDMG) N=337°	Moment-resisting steel frame (Concrete mat - no piles]	57	12/1	35	22	NS EW UP	0.10 0.09 0.10	0.34 0.34
San Bruno Office Building (CDMG) N=335°	Reinforced concrete moment resisting frame (individual spread footing)	24	6/0	81	13	NS EW UP	0.14 0.11 0.12	0.25 0.32
California State University (Hayward) Admistration Building (CDMG) N=320°	Steel moment-frame core; exterior reinforced concrete moment frame (0.45-m-thick slab on grade and bearing piles)	61	13/0	70	16	NS EW UP	0.07 0.09 0.05	0.15 0.24

facilitated easy access to various floors of the building without actually going to those floors or without having to provide temporary cables and sensors, (2) allowed ready access to record response data, (3) made it possible to compare directly the ambient and strong-motion response.

The damping ratios are extracted by system identification analyses of Loma Prieta data in accordance with the procedures outlined by Ghanem and Shinozuka (1995) and Shinozuka and Ghanem (1995) (see "Methods of Analyses" section). The low-amplitude (ambient) vibration data was analyzed by conventional spectral analysis techniques. System identification techniques were not applied to the ambient vibration data because of the unknown system input characteristics. Furthermore, because of the lower signal-to-noise ratio, system identification techniques do not minimize the errors simply and solutions may be unreliable. The results of analyses of the strong-motion response data and the ambient vibration data are summarized in table 12. Also included for comparison are results of previous tests and analyses for the Pacific Park Plaza and Transamerica Buildings.

In each of the five buildings tested, the first-mode periods associated with the strong-motion records are longer than those associated with the ambient vibration records. The highest first-mode period ratio (Loma Prieta earthquake/ambient) is 1.47. Also, the percentages of critical damping for the first mode for the ambient data are significantly smaller than those from the strong-motion data.

Table 12.—*Dynamic characteristics of the five buildings discussed in report*

[Data from Marshall and others (1992) and Çelebi (1996). f, frequency (Hz); T, period (s); x, damping (percent)]

Building	Nominal direction	Pre-Loma Prieta test and analyses		Loma Prieta data		Post-Loma Prieta Ambient	
		f/(T)	ξ (pct.)	f/(T)	ξ(pct.)	f/(T)	ξ(pct.)
Pacific Park Plaza [PPP]	NS	0.59 (1.70)	2.6	0.38 (2.63)	11.6	0.48 (2.08)	0.6
	EW	0.59 (1.70)	2.6	0.38 (2.63)	15.5	0.48 (2.08)	3.4
Transamerica Building [TRA]	NS	0.34 (2.94)	0.9	0.28 (3.57)	4.9	0.34 (2.94)	0.8
	EW	0.34 (2.94)	1.4	0.28 (3.57)	2.2	0.32 (3.12)	1.4
Santa Clara County Office Building [SCCOB]	NS			0.45 (2.22)	2.7	0.52 (1.92)	-
	EW			0.45 (2.22)	2.7	0.52 (1.92)	-
San Bruno Office Building [SBR]	NS			1.17 (0.85)	7.2	1.72 (0.58)	2.2
	EW			0.98 (1.02)	4.1	1.41 (0.71)	2.3
California State University (Hayward) Administration Building [CSUH]	NS			0.76 (1.32)	3.4	0.92 (1.09)	0.6
	EW			0.76 (1.32)	2.3	0.86 (1.16)	0.6

These differences in periods and damping percentages may be caused by several factors, including (1) possible soil-structure and/or possible pile-foundation interaction, which is more pronounced during strong-motion events than during ambient excitations, (2) nonlinear behavior of the structure (such as microcracking of the concrete at the foundation or superstructure), (3) slip of steel connections, and (4) interaction of structural and nonstructural elements.

In the case of the Santa Clara County Office Building, the largest critical damping ratio assessed from the records is about 2.7 percent (tables 4, 12). Damping from the low-amplitude test data could not be determined due to its low signal/noise ratio. It is noted again that this building is now retrofitted with visco-elastic dampers to change its dynamic characteristics so that the global damping and the fundamental frequency both will increase. The resulting shift in the fundamental period and increase in damping is expected to eliminate excessive responses and the beating effect (Crosby and others, 1994). Low-amplitude tests on the now retrofitted building recently performed by Çelebi and Liu (1997) preliminarily indicate that there is improvement in the dynamic characteristics.

For the Pacific Park Plaza, the post-Loma Prieta frequency (0.48 Hz) is lower than the pre-Loma Prieta frequency (0.59 Hz), but both are larger than the Loma Prieta frequency (0.38 Hz). For Transamerica Building, the frequencies from the pre- and post-LPE low-amplitude tests are in good agreement (0.34 vs. 0.34 and 0.32 Hz) and are larger than the Loma Prieta frequency (0.28 Hz) (Stephen and others, 1974; Kinemetrics, 1979; Stephen and others, 1985). The immediate conclusion drawn from table 12 is that the damping ratios and periods are consistently and significantly larger for the strong-shaking than for the low-amplitude vibrations.

In addition to these general conclusions, a specific characteristic for Pacific Park Plaza is noted: The damping ratios extracted from the system identification analyses corresponding to the 0.38-Hz first mode frequency are 11.6 percent (north-south) and 15.5 percent (east-west). Such unusually high damping ratios for a conventionally designed/constructed building require explanation. The building with its large mat foundation in a relatively soft geotechnical environment is capable of energy dissipation in the soil due to radiation (or foundation) or material damping. Therefore, the 0.38-Hz frequency should be considered as the fundamental frequency that incorporates soil-structure interaction at the level of amplified shaking summarized in table 1. Analyses showed that rocking was insignificant.

Accepting 0.38 Hz (2.63 s) as the frequency (period) with soil-structure interaction and 0.48 Hz (2.08 s) or 0.57 Hz (1.75 s) as that without soil-structure interaction, then it is possible to make some quantification of the amount of foundation damping using equations developed by Veletsos (1977) and included in the Applied Technology Council's publication ATC 3-06 (Applied Technology Council, 1978). The area (A) of the foundation mat is approximately 1,600 m^2. The equivalent circular radius of the foundation is calculated as $r = (A/r)^{0.5} = 22.6$ m (this number is possibly too low considering that the Pacific Park Plaza is a three-winged building, and therefore the effective radius is possibly larger than the conversion provided above). The height of the building (h) is 89.2 m; the effective height (h_{eff}) as defined by the Applied Technology Council (1978) is $h_{eff} = 0.7h = 62.4$ m. For $h_{eff}/r = 62.4/22.6 = 2.8$, and for $T_{ssi}/T = 2.63/2.08 = 1.26$ or $2.63/1.75 = 1.50$, the foundation (including radiation and material) damping can be approximated at 5-6 percent from figure 54 (adopted from Applied Technology Council, 1978). Inserting these into Veletsos' equation for effective damping (1977):

$$\xi_{eff} = \xi_{str} + \xi_{ssi} = \xi_{str} + \xi_o / (T_{ssi} / T)^3.$$

The value of ξ_o here is the structural damping without soil-structure interaction, normally accepted from empirically prepared tables to be between 3 and 7 percent. Therefore with T_{ssi}/T varying between 1.26 and 1.50, the structural damping is reduced to 29-50 percent of this accepted value. Thus, the estimated foundation damping of 5 to 6 percent is not unusual, given the fact that the ξ_{eff} determined from observed response data is 11-15 percent. More detailed discussions on foundation damping can be found in Veletsos (1977), Luco (1980), Roesset (1980), Dobry and Gazetas (1985), Todorovska (1992), and very recently in Wolf and Song (1996). It can therefore be categorically stated that radiation damping is beneficial in reducing responses of structures.

It should be stated that consideration of the effect of soil-structure interaction in estimation of damping for analysis and design purposes has not yet found its way into design offices, except for those involved with critical structures. In the past, during design/analysis processes of engineered structures, it was assumed that a structure's foundation is fixed to the underlying media. State-of-the-art knowledge and analytical approaches require, when warranted, the structure-foundation system to be represented by mathematical models that include the influence of the sub-foundation media. Identification of beneficial and adverse effects of soil-structure interaction is a necessity. Adverse effects of soil-structure interaction during

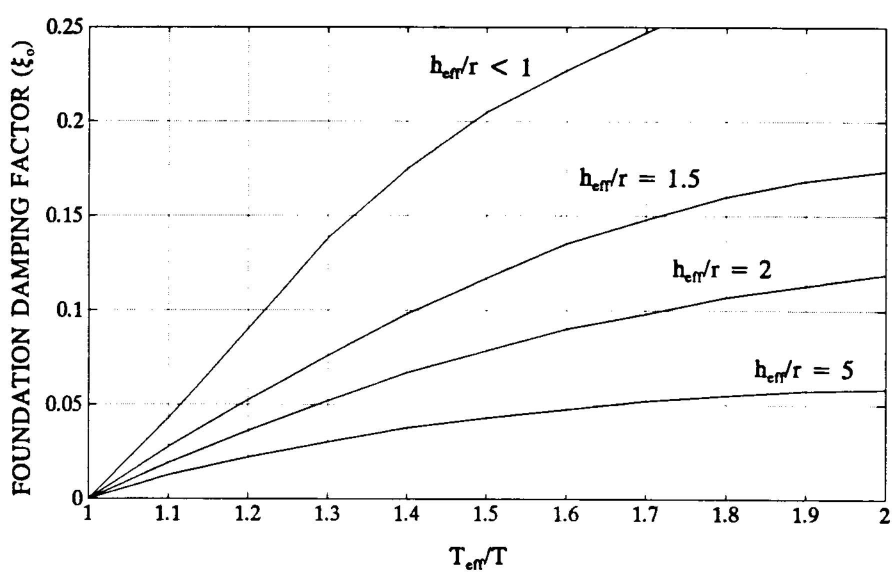

Figure 54.—Foundation damping in terms of ratio of periods and ratio of building height-to-equivalent radius of foundation (from Veletsos (1977) and Applied Technology Council (1978)).

the 1985 Michoacon (Mexico) earthquake was addressed by Tarquis and Roesset (1988), who showed that lengthened fundamental periods due to soil-structure interaction of mid-rise buildings (5-15 stories) in the Mexico City lakebed area placed them close to the resonating 2-s site period. This study points out that by neglecting the effect of soil-structure interaction, the dynamic characteristics, and specifically damping in structures can be underestimated if the values of damping assessed from low-amplitude testing are used in lieu of values supposedly for strong motions.

To repeat, current analyses and design procedures using estimated damping values may not be realistic due to (1) change in construction and design practices and (2) the fact that soil-structure interaction effects in most cases are ignored for buildings that are not considered to be critical facilities. However, results presented herein suggest that the critical damping percentages used for analyses and design of some buildings may not have been properly estimated (specifically, overestimated for the Santa Clara County Office Building and underestimated for the Pacific Park Plaza) and possibly not even considered for some buildings. Furthermore, changes in damping values and fundamental values commensurate with inferred strong-motion values should be considered to improve design and analyses results.

OTHER STUDIES

POUNDING OF STRUCTURES

Kasai and others (1991) surveyed the damage due to pounding in the San Francisco Bay area following the Loma Prieta earthquake. They determined that significant pounding of buildings occurred at sites ranging up to more than 90 km from the epicenter and discussed the implications of this finding in forecasting future possible catastrophic damage that may occur during earthquakes having epicenters closer to dense urban areas. They also present analytical research on pounding that includes development of dynamic analysis programs that incorporate pounding. Parametric studies were performed on building pounding response as well as appurtenance response. These analyses included a spectrum method to obtain peak pounding responses, actual case studies, and a spectrum method to determine required building separations to preclude pounding. Their analytical studies did not relate to any recorded pounding responses of buildings. This may be due to the fact that there is no specific instrumentation scheme implemented, by either California Strong Motion Instrumentation Program or U.S. Geological Survey, in adjacent buildings with possibility of pounding to record their responses during earthquakes.

MASONRY BUILDINGS

Hart and Jaw (1991) investigated the performance of a tall (approximately 87 ft) reinforced masonry building located in Santa Cruz, near the epicentral region. They observed that the building was not damaged, in spite of a probable maximum ground motion of 0.3-0.5 g.

OVERTURNING FORCES

Chen (1992) presented a newly developed time-delay method for earthquake analysis which incorporates the attribute of finite wave speed propagating upward into a structure. He showed verification of the method using data from the California Strong Motion Instrumentation Program applied to calculate the overturning moment of structures. He used one detailed three-dimensional model and nine simplified two-dimensional models for this purpose.

Gates and others (1994) investigated three high-rise shear wall buildings to evaluate overturning forces in the shear walls under three recent northern California earthquakes: 1984 Morgan Hill, 1986 Mt. Lewis, and 1989 Loma Prieta. The buildings are in the 9-10 story range, with three different shear wall configurations of (1) perimeter walls, (2) core walls, and (3) distributed walls. They employed two methods of data reduction and analysis to assess the significance of soil-structure interaction on building overturning forces: (1) simplified data analysis procedures using recorded motion, mode shapes, and building weights to assess dynamic performance and (2) three-dimensional linear elastic dynamic analyses using soil-structure models for the shear walls and foundation systems. They reported that the recorded responses show ample evidence of foundation/shear wall rocking under the moderate and strong shaking provided by the cited earthquakes. They also reported that the lengthening in the periods of the buildings take place due to both rocking and inelastic behavior. The analytical results are compared with code procedures for predicting the periods of the structures as well as the distribution of overturning forces. They showed that careful modeling of buildings refined by system identification techniques using actual recorded responses provides useful steps in the evaluation of buildings.

CODE ASSESSMENT AND SERVICEABILITY REQUIREMENTS

Fenves (1990) evaluated the lateral force procedures for buildings with irregular plans or vertical irregularities. He used recorded strong-motion response data from two instrumented buildings to determine vibration properties and distribution of lateral forces. One of the two buildings is a four-story hospital with records from both Morgan Hill earthquake (1984) and Loma Prieta and the other with only Loma Prieta records. Fenves reported that due to the irregular configuration of the buildings, the distribution of lateral forces changes substantially with changes in the amplitude of earthquake response.

Uang and Maarouf (1991), Werner and others (1992), and Beck and others (1992) investigated the Uniform Building Code and serviceability requirements from building response data. They all investigated several buildings and compared their analyses with observed performance of each building during the Loma Prieta earthquake and other earthquakes for which data are available.

SOIL-PILE-STRUCTURE INTERACTION

Gould and Ahn (1991) studied one of the six-story wings of the Clarion Hotel near San Francisco International Airport. The reinforced concrete shear-wall and framed building rests on pile foundation penetrating into stiff, clayey soils underlying the soft Bay mud. With the premise that, in general, lengthening of the building period results in lower inertial forces, they studied the building with mathematical models that includes soil-pile-structure interaction. They used ground motion recorded at the San Francisco International Airport as surrogate input base motion to the subject building model. To differentiate the effect of the soil-pile-structure interaction, they performed fixed-base analyses of the structure. Their analyses show that the base shear and the base overturning moment are significantly reduced with lengthening of the building period from 0.35 s (fixed-base) to as much as 1.33 s due to soil-pile-structure interaction.

TRENDS IN BEHAVIOR OF REGULAR BUILDINGS

Li and Mau (1997) recently completed a study of 21 regular (typical, symmetric) buildings which have recorded response records. Of these, 11 buildings are from the San Francisco Bay area with records from the Loma Prieta earthquake and 9 are from the Los Angeles area with records from the 1987 Whittier earthquake. With this data base, they performed extensive analyses using system identification techniques as a tool. They arrived at the following conclusions:

1. Estimates of building periods by code formulas is less reliable for shear wall buildings than for framed buildings.
2. Estimates of building frequencies by code formulas for longitudinal and transverse direction is variable and is dependent on the structural framing and presence and distribution of shear walls.
3. The data base indicates that there is great variation of the damping ratios. The variation is larger for concrete buildings (2-14 percent) than for steel buildings (1-6 percent).
4. For reinforced concrete and steel framed buildings, the maximum story drift occurs at a middle or lower story, but for shear wall buildings the maximum story drift occurs at a middle or higher story.
5. Rocking occurs in some buildings as a result of translational vibration. For stiffer buildings, rocking can occur as a distinguishable mode of vibration.
6. Large variations in building frequency can be detected within the time-history of a response record of a building. This may or may not be a symptom of damage but can be used as an indicator of possible damage.
7. There is strong correlation between change of frequency and variation of drift.

CONCLUSIONS

This paper summarizes studies of performances of buildings during the Loma-Prieta earthquake. The studies referred to herein are mostly on the recorded responses of instrumented buildings. The conclusions derived from these studies are as follows:

1. Instrumentation of structures as part of hazard reduction programs is very beneficial, as studies of this type will help to better predict the performance of structures in future earthquakes.

   a. Studies of recorded responses of buildings help researchers and practicing professionals to better understand the cumulative structural and site characteristics that affect the response of buildings and other structures. Such studies impact mitigation efforts.

   b. In turn, the behavior that may be expected from buildings during future earthquakes with large input motions (either due to larger magnitude earthquakes or earthquakes at closer distances to the building) can be forecast. This is particularly true for the San Francisco Bay area where—

      i. The probability of magnitude 7 or larger earthquakes occurring on major faults, including the San Andreas and Hayward faults, is considered to be approximately 67 percent or higher within a 30-year period (Working Group, 1990).

      ii. There is a large inventory of buildings within 0-10 km of the two major faults capable of generating $M>7$ earthquakes. This is particularly important because, very recently, the Structural Engineers Association of California (SEAOC) issued the 1996 edition of the "Recommended Lateral Force Requirements and Commentary," which has provisions for increasing the design base shear by 0-100 percent depending on the 0-10-km distance of a building from a fault. This implies that the forecasting of performance of buildings within 0-10 km of major faults must be done more informatively. This requisite information can be achieved only through acquiring and studying response data from buildings during earthquakes.

iii. Furthermore, some of the building response data are from tall buildings that are on soft soils. The motions at the soft-soil sites of some of the important tall buildings (for example, Pacific Park Plaza, Transamerica Building, Embarcadero Building, and Chevron Building) are amplified by 3-5 times within the periods of engineering interest when compared with the motions of Yerba Buena Island, a rock site approximately the same distance away from the epicenter of the earthquake.

2. There is an acute need to better evaluate structural and site characteristics in developing earthquake resisting designs of building structures. Studies in this paper show, as in the case of Santa Clara County Office Building, that designs of buildings with low structural damping, resonation, and beating effects caused by closely coupled translational and torsional modes must be avoided. Also, as expected in most tall buildings, higher modes are excited. As in the case of Embarcadero Building, higher modes play an important role in the response of building structures and therefore must be carefully evaluated to assess their future performances.

3. Drift ratios calculated from observed data in certain cases exceed code drift limitations for part or all of the structural systems. Assessing the drift exposure of structural systems are ever more important since the design/analyses of buildings are recently being shifted toward a performance-based design procedure.

4. Soil-structure interaction is one of the least understood actions that affects structural behavior. There may be beneficial or detrimental effects of this interaction to the overall behavior of structures. It stands to be prominent in the behavior of several buildings (for example, Pacific Park Plaza, Santa Clara County Office Building, Transamerica Building, and others) as assessed from studies of the recorded responses of buildings during the Loma Prieta earthquake. Therefore, two specific issues are that (1) identification of beneficial and adverse effects of soil-structure interaction is a necessity and (2) design offices must be informed and trained in consideration of the effect of soil-structure interaction in estimation of fundamental period and damping of a building, as this is not yet the case. Specifically, the damping percentages are overestimated for the Santa Clara County Office Building and underestimated for the Pacific Park Plaza, and possibly not even considered for some buildings.

5. Development of design response spectra deserves more intensive consideration by geotechnical engineers since site effects play an important role in the response of building structures. There are significant discrepancies in the comparison of the response spectra derived from recorded motions with the actual design response spectra. Amplified motions due to soft soil conditions, site-specific resonating, and frequency content must be kept in mind in the development of design response spectra (for example, the Embarcadero Building).

6. The propagation direction of surface waves arriving at buildings affect particularly unsymmetrical buildings or buildings with wings (for example, Pacific Park Plaza and Santa Clara County Office Building).

7. The basemat rotation of tall buildings with basements calculated by the displacements in the corners of the basemat is considerably smaller than the rotation of the basement walls calculated by the displacements derived from the horizontal sensors at the street level and basemat. This is observed for both the Transamerica Building and Embarcadero Building. The implication is that the current practice which assumes that the inertial forces at ground level and basemat level to be the same is not correct.

8. Low-amplitude tests have been conducted on five buildings that recorded the Loma Prieta earthquake. Results indicate, as expected, that the first-mode periods extracted from strong-motion response records are longer than those associated with the ambient vibration records. Similarly, the percentages of critical damping for the first mode for the ambient data are significantly smaller than those from the strong-motion data. These differences may be caused by several factors including (1) possible soil-structure interaction which is more pronounced during strong-motion events than during ambient excitations, (and similarly, in buildings with pile foundations, possible pile-foundation interaction which may not occur during ambient excitation), (2) nonlinear behavior of the structure (such as microcracking of the concrete at the foundation or superstructure), (3) slip of steel connections, and (4) interaction of structural and nonstructural elements. Changes in damping values and fundamental period values commensurate with inferred strong-motion values should be considered to improve design and analyses results.

9. In processing of data of recorded responses for buildings, use of different filters for basement versus free-field causes gross differences in displacements (as in the case of California State University, Hayward). A common rational filtering for basement and associated free-field motions is recommended.

10. Specific instrumentation schemes of some of the already instrumented buildings and of those buildings yet to be instrumented must be improved and/or implemented so that the response characteristics expected of that building can be captured (for example, soil-structure interaction, pounding, and variation of drift due to abrupt changes in stiffness). When applicable, specific buildings should be specially instrumented extensively to better capture their behavior in response to actions such as pounding and soil-structure interaction.

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