PB93-113579

NISTIR 4884

MEASUREMENT OF STRUCTURAL RESPONSE CHARACTERISTICS OF FULL-SCALE BUILDINGS: COMPARISON OF RESULTS FROM STRONG-MOTION AND AMBIENT VIBRATION RECORDS

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October 1992

U.S. Department of Commerce
Barbara Hackman Franklin, *Secretary*
Technology Administration
Robert M. White, *Under Secretary for Technology*
National Institute of Standards and Technology
John W. Lyons, *Director*

PB93-113579

NIST-114A (REV. 3-90)	U.S. DEPARTMENT OF COMMERCE NATIONAL INSTITUTE OF STANDARDS AND TECHNOLOGY	1. PUBLICATION OR REPORT NUMBER NISTIR 4884
	BIBLIOGRAPHIC DATA SHEET	2. PERFORMING ORGANIZATION REPORT NUMBER
		3. PUBLICATION DATE OCTOBER 1992

4. TITLE AND SUBTITLE

Measurement of Structural Response Characteristics of Full-Scale Buildings: Comparison of Results from Strong-Motion and Ambient Vibration Records

5. AUTHOR(S)

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6. PERFORMING ORGANIZATION (IF JOINT OR OTHER THAN NIST, SEE INSTRUCTIONS)

U.S. DEPARTMENT OF COMMERCE
NATIONAL INSTITUTE OF STANDARDS AND TECHNOLOGY
GAITHERSBURG, MD 20899

Branch of Engineering and Seismology & Geology
U.S. Geological Survey
Menlo Park, CA 94025

7. CONTRACT/GRANT NUMBER

8. TYPE OF REPORT AND PERIOD COVERED
Final

9. SPONSORING ORGANIZATION NAME AND COMPLETE ADDRESS (STREET, CITY, STATE, ZIP)

NIST CATEGORY
NIST 140

10. SUPPLEMENTARY NOTES

11. ABSTRACT (A 200-WORD OR LESS FACTUAL SUMMARY OF MOST SIGNIFICANT INFORMATION. IF DOCUMENT INCLUDES A SIGNIFICANT BIBLIOGRAPHY OR LITERATURE SURVEY, MENTION IT HERE.)

This report describes the collection and analysis of ambient vibration data from five buildings in the San Francisco Bay area that experienced strong shaking during the Loma Prieta earthquake of October 17, 1989. The buildings represent a range of construction materials, structural systems, foundation systems and building dimensions. Results of the analyses are compared with similar analyses carried out on strong-motion response records obtained from the same buildings during the earthquake. While the lower modes of vibration can be reliably identified from ambient vibration records, the frequencies of these modes are in each case higher than the corresponding frequencies derived from strong-motion response records. When soil-structure interaction is involved, the strong-motion modal frequencies may range from 70 to 80 percent of the corresponding values extracted from ambient vibration records. Estimates of structural damping derived from ambient vibration data are substantially smaller than those derived from strong-motion data and are consistent with predictions of a damping model based on forced vibration tests. The lower bound of damping estimates obtained from strong-motion response records in this study is consistent with published data. Where soil-structure interaction is a significant factor, the overall damping for strong-motion response may be 3 to 4 times the indicated lower bound.

12. KEY WORDS (6 TO 12 ENTRIES; ALPHABETICAL ORDER; CAPITALIZE ONLY PROPER NAMES; AND SEPARATE KEY WORDS BY SEMICOLONS)

buildings; damping; earthquake; instrumentation; dynamic response; field measurements; signal processing; structural dynamics.

13. AVAILABILITY

- [X] UNLIMITED
- [ ] FOR OFFICIAL DISTRIBUTION. DO NOT RELEASE TO NATIONAL TECHNICAL INFORMATION SERVICE (NTIS).
- [ ] ORDER FROM SUPERINTENDENT OF DOCUMENTS, U.S. GOVERNMENT PRINTING OFFICE, WASHINGTON, DC 20402.
- [X] ORDER FROM NATIONAL TECHNICAL INFORMATION SERVICE (NTIS), SPRINGFIELD, VA 22161.

14. NUMBER OF PRINTED PAGES
85

15. PRICE
A05

ELECTRONIC FORM

ABSTRACT

This report describes the collection and analysis of ambient vibration data from five buildings in the San Francisco Bay area that experienced strong shaking during the Loma Prieta earthquake of October 17, 1989. The buildings represent a range of construction materials, structural systems, foundation systems and building dimensions. Results of the analyses are compared with similar analyses carried out on strong-motion response records obtained from the same buildings during the earthquake. While the lower modes of vibration can be reliably identified from ambient vibration records, the frequencies of these modes are in each case higher than the corresponding frequencies derived from strong-motion response records. When soil-structure interaction is involved, the strong-motion modal frequencies may range from 70 to 80 percent of the corresponding values extracted from ambient vibration records. Estimates of structural damping derived from ambient vibration data are substantially smaller than those derived from strong-motion data and are consistent with predictions of a damping model based on forced vibration tests. The lower bound of damping estimates obtained from strong-motion response records in this study is consistent with published data. Where soil-structure interaction is a significant factor, the overall damping for strong-motion response may be 3 to 4 times the indicated lower bound.

Keywords: buildings, damping, earthquake, instrumentation, dynamic response, field measurements, signal processing, structural dynamics

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1. INTRODUCTION

Reliable estimates of modal frequencies, stiffness and damping of structural systems are essential to the prediction of dynamic response under loading conditions associated with serviceability or structural safety. Analytical models for predicting dynamic response usually involve substantial simplification of the real structure and, for various reasons, detailed physical models for controlled testing in the laboratory often prove impractical. Consequently, each of these approaches must depend on reliable full-scale field measurements for validation and/or improvement of the modeling technique. In other cases, it is desired to establish the dynamic characteristics of existing structures because of change of function, planned structural upgrade, or assessment of risk of failure. The question may then arise as to what measurement procedures are required to obtain reliable information and what interpretation is to be given to the measurement results.

Numerous studies of dynamic response have been carried out on tall buildings, long-span bridges, and large dams. In most cases the approach has been to rely on ambient vibrations for structural excitation while other studies have resorted to the use of one or more mechanical shakers to excite specific modes of vibration. More recently, it has been possible to obtain detailed response records from heavily instrumented buildings and other structures that experienced extreme events such as earthquakes or wind storms. Generally, these measurements suggest a strong dependence of dynamic characteristics on displacement amplitude, thus raising questions as to the utility and proper interpretation of response measurements obtained under low levels of excitation. The Loma Prieta earthquake of October 17, 1989, provided a unique opportunity to carry out a program of field measurements and data analysis for certain existing structures to better understand the significance of factors such as displacement amplitude in full-scale response measurements and the proper interpretation of such measurements.

2. SCOPE OF WORK

The study described herein involved the selection of a number of undamaged buildings that were subjected to strong ground shaking during the Loma Prieta earthquake and for which reliable response records were available. Ambient vibration data were then obtained from the selected buildings and the dynamic characteristics derived from the two sets of data were compared. Estimates of structural damping were compared with model predictions for low levels of excitation and for strong-motion response. In addition, a finite element model of one of the buildings was developed to assess the effects of soil-structure interaction on apparent structural stiffness. The extent to which reliance can be placed on low-amplitude response measurements and their utility in predicting strong-motion response characteristics were examined.

3. BUILDINGS INVESTIGATED

The buildings investigated in this study were drawn from a large number of candidate structures affected by the Loma Prieta earthquake. In making a selection, the following criteria were considered:

- o Availability of reliable strong-motion records from key locations within the building.
- o No visible signs of damage from Loma Prieta or other earthquakes.
- o Accessibility to the building for the purpose of conducting ambient vibration studies.
- o Availability of detailed structural drawings and soil conditions at the site.
- o Structural design in accordance with reasonably current code requirements.
- o Buildings that represented a range of construction materials, structural systems, foundation systems and building dimensions.
- o Degree of complexity in developing analytical models of the building structures.

A total of five buildings were selected for the study. Their locations in the San Francisco Bay area and the location of the earthquake epicenter are shown in Figure 1. Epicentral distance, orientation, building height and peak response to the Loma Prieta earthquake are listed in Table 1. Additional information on the buildings and details of the permanently installed instrumentation are provided in Section 7 of this report. Photographs of the buildings, preliminary assessments of natural frequencies, and peak ground accelerations at nearby sites can be found elsewhere (Marshall, Phan and Çelebi, 1991).

4. STRONG MOTION RECORDS

The first three buildings listed in Table 1 were instrumented by the Strong Motion Instrumentation Program, California Division of Mines and Geology (CDMG). The remaining buildings in Table 1 were instrumented by the U.S. Geological Survey (USGS). In each case the instrumentation consists of force balance accelerometers (FBA) having a nominal sensitivity of 2.5 volts/g. Recording was accomplished by means of strip-film recorders triggered by vertically-oriented accelerometers located on the base slab of the building. The strip-film records were subsequently digitized at an effective rate of 200 samples per second. The records were then corrected

TABLE 1. BUILDING LOCATION, ORIENTATION AND PEAK RESPONSE TO LOMA PRIETA EARTHQUAKE

BUILDING	EPICENTRAL DISTANCE (km)	REF. NORTH (Degrees)	FLOORS ABOVE/BELOW GROUND	BUILDING HEIGHT (m)	MAX INSTRUMENT HEIGHT (m)	PEAK ACCELERATION GROUND (g)	PEAK ACCELERATION ROOF (g)	ROOF DISPLACEMENT PEAK (mm)	ROOF DISPLACEMENT MPPD[a] (mm)
(1) CSUH Administration Building Hayward, CA	70	320	13/0	61	61	(N-S) 0.07 (E-W) 0.08 (Vert) 0.05	0.15 0.24	35 33	50 35
(2) Santa Clara County Office Building San Jose, CA	35	337	12/1	57	57	(N-S) 0.10 (E-W) 0.09 (Vert) 0.10	0.34 0.34	443 416	300 360
(3) Commercial Office Building San Bruno, CA	81	335	6/0	24	24	(N-S) 0.14 (E-W) 0.11 (Vert) 0.12	0.25 0.32	64 65	50 90
(4) Transamerica Building San Francisco, CA	97	351	60/3	257	206	(N-S) 0.11 (E-W) 0.12 (Vert) 0.07	0.24 0.31	113 186	170 265
(5) Pacific Park Plaza Emeryville, CA	97	350	30/1	94	91	(N-S) 0.17 (E-W) 0.21 (Vert) 0.06	0.29 0.38	65 219	95 165

[a] Mean peak-to-peak displacement

for instrument errors and bandpass filtered with ramps at 0.12-0.24 Hz and at 23.0-25.0 Hz. The resulting acceleration time histories and the integrated velocity and displacement time histories were made available on floppy disks. The corresponding time steps for these time histories is 0.005 second in the case of the USGS data files and 0.020 second for the CDMG files. Details of the CDMG data format have been described by Shakal and Huang (1985) and the data utilized in this study were made available through California Strong Motion Instrumentation Program reports (CSMIP 1990, 1991a, 1991b). Peak ground and upper floor accelerations, peak structural displacements, and estimated mean peak-to-peak displacements (MPPD) for the five buildings are listed in Table 1. MPPD values are used subsequently to compare results of this study with findings from other studies.

5. AMBIENT VIBRATION MEASUREMENTS

The ambient vibration measurements were carried out using the existing accelerometer array in each building. A preamplifier provided a gain of up to x500 and this was followed by a low-pass 3-pole Butterworth filter set at a cutoff frequency of 10 Hz. A programmable-gain amplifier was then used to adjust the signal level for optimum dynamic range. The digital data acquisition system was equipped with 16 sample-and-hold amplifiers and sampling was carried out at a net rate of 50 samples per second for each data channel. The dynamic range of the 12-bit A/D converter was $\pm$ 10 volts. Battery powered bucking circuits were installed at each signal input to suppress the dc offset of the accelerometer. Typically, the overall gain of the signal conditioning system was x4,000, providing an input dynamic range of $\pm$ 2.5 millivolts and a theoretical resolution of about 1.2 microvolts. However, system noise limited the effective resolution to about 5 micro-g. The recorded data and all subsequent analyses retained the units of microvolts, referred to transducer output. For those buildings having more than 16 accelerometers, it was necessary to repeat recordings using different accelerometer combinations. Typically, record lengths were 200 seconds.

6. METHODS OF ANALYSIS

Both the recorded strong-motion responses and the ambient vibration responses were analyzed to identify relevant dynamic characteristics and to provide a basis for comparison of these characteristics. Throughout this paper, unless otherwise stated, response records have been analyzed by system identification techniques (MathWorks, 1988) or by conventional spectral analysis methods (Brook and Wynne, 1988) using commercially-available software.

6.1 System Identification

The procedure used in system identification analysis is to estimate a model based on observed input-output data (Ljung, 1987). Simply stated, the input is the recorded basement or ground floor motion and the output is the observed structural response at roof or upper floor level. Specifically, the ARX model based on the least-squares method for single-input, single-output (SISO) dynamic linear systems has been used here. For a detailed development of this and other models, see Lujung (1987) and Şafak (1989).

The general model for SISO linear systems with noise can be represented by

$$A(q)y(t) = q^{k}[B(q)/F(q)]x(t) + [C(q)/D(q)]e(t) \qquad (6.1)$$

where x(t) and y(t) are the discrete-time domain system input and output sequences, respectively, A, B, C and D are polynomials in the backward-shift operator, q^{j}, defined as

$$q^{-j}y(t) = y(t-j)$$

k is the time delay between input and output, and e(t) is a white-noise sequence with zero mean.

Given a pair of input and output sequences, system identification involves the determination of the coefficients of the polynomials. Usually, not all the polynomials in Eq. 6.1 are needed. And by eliminating various polynomials, a number of special forms of the model are obtained. For the ARX model, polynomials C and D are unity and the SISO linear system model reduces to the form

$$A(q)y(t) = B(q)x(t - k) + e(t) \qquad (6.2)$$

where A and B are polynomials in q^{-1}

$$A(q) = 1 + a_1q^{-1} + \ldots + a_{n_a}q^{-n_a}$$

$$B(q) = b_0 + b_1q^{-1} + \ldots + b_{n_b}q^{-n_b}$$

By taking the Z-transform of the discrete-time domain equations, the SISO system can be modeled in the frequency domain with the transfer function

$$H(z) = B(z)/A(z) \qquad (6.3)$$

in which the variable z has replaced the operator q. The roots of B(z) and A(z) are the zeros and poles, respectively, of the transfer function. Stability requires that the poles be in complex-conjugate pairs with modulus less than 1. If the pairs of terms corresponding to the pairs of complex roots are combined, it can be shown that

$$H(z) = \sum_{j=1}^{n_a/2} H_j(z) \tag{6.4}$$

where n_a is the order of the polynomial A and each $H_j(z)$ is a second-order filter corresponding to a simple damped oscillator with damping ratio ζ_j and center frequency f_j. ζ_j and f_j are calculated from the poles of the transfer function by the following equations:

$$\zeta_j = \ln(1/r_j)/[\phi^2_j + \ln^2(1/r_j)]^{1/2} \tag{6.5}$$

$$f_j = \ln(1/r_j)/[2\pi\zeta_j T] \tag{6.6}$$

where r_j and ϕ_j are the modulus and arguments of the jth pole and T is the sampling interval. H(z) is thus equivalent to $n_a/2$ second-order filters $H_j(z)$ operating in parallel on x(t), and each $H_j(z)$ can be considered a mode of the system.

The coefficients a_k and b_k of the transfer function are evaluated by adaptive prediction techniques which minimize the mean square of the identification errors (departures from y(t)) summed over the length of the discrete-time sequence. Knowing these coefficients, the modal characteristics of the building and the modal contributions to its response can then be determined. The order of the polynomials used in the identification process must be sufficient to represent the contributing modes.

6.2 Conventional Signal Analysis Techniques

For ambient vibration records, the system input is either poorly defined or unknown. Therefore, conventional Fourier techniques were used in the analysis of these records. A critical assumption with this approach is that the recorded response time histories are stationary, a condition that is seldom realized where wind effects (fluctuations in wind speed and direction) constitute the primary structural forcing function. For tall buildings with low damping, it has been demonstrated that record lengths of several hours may be required to obtain acceptable estimates of structural response spectra (Jeary and Ellis, 1980). This can only be accomplished with ensemble averaging techniques in which the spectra for several response records obtained under similar wind conditions are averaged. In this study, the number of sets of ambient vibration records was limited to three or four per building, with each set consisting of up to 16 accelerometer outputs of approximately 200 seconds duration. Selection of the sets used in the analyses reported here was based on a subjective assessment of stationarity. Individual spectra are based on 2,048 sequential samples representing a time segment of 41 seconds.

Estimates of structural damping were obtained using the auto-correlation technique (Jeary and Winney, 1972). This approach makes use of the fact that for a lightly damped, single degree of freedom system subjected to excitation by white noise, the auto-correlation function of the response has an exponentially decaying envelope given by

$$R_{xx}(\tau) = \text{constant x} \exp(-\zeta\omega_n\tau) \tag{6.7}$$

where τ is the delay time, ζ is the damping ratio and ω_n is the undamped natural frequency. For the nth cycle of the auto-correlation function, $\tau\omega_n = 2\pi n$ and the damping ratio becomes

$$\zeta = [1/(2\pi n)]\ln[R_{xx}(0)/R_{xx}(n)] \tag{6.8}$$

To apply this technique to response signals with contributions from more than one natural mode (the usual case), it is necessary to isolate modes by selective filtering. For this study, estimates of damping ratio were obtained for the first translational mode only and higher modes were removed by digital filtering (Blackman filter with attenuation rate of 74 dB per octave). As with response spectra, several hours of stationary records may be required to obtain reliable damping estimates, particularly for tall buildings with long natural periods and low damping.

7. RESULTS OF ANALYSIS

This section presents a summary of the analysis of strong-motion records obtained during the Loma Prieta earthquake and of subsequent ambient vibration measurements from the five buildings included in this study using permanently installed accelerometers.

7.1 CSUH Administration Building, Hayward

This 13-story building, instrumented by CDMG, has plan dimensions of 34.29 x 34.29 m and a height of 61.29 m. The structural system consists of an exterior perimeter reinforced concrete moment-frame, concrete shear walls around the elevator shafts up to the 2nd floor, and a steel moment-frame core above the 2nd-floor level. The building is connected to an adjacent building on the east side through a two-story enclosed bridge structure spanning the street (see Figure 2). The bridge structure is free to move on friction bearings located at its juncture with the adjacent building. The foundation system consists of a 0.45 m slab on grade and bearing piles. Building dimensions and the instrumentation scheme are shown in Figure 2.

7.1.1 Strong-Motion Data

Of the 16 accelerometers installed at the time of the earthquake, three channels (Numbers 2, 7 and 10) failed to record properly with the result that torsional response of the building could not be evaluated. However, because of the square plan of the building and symmetrical arrangement of the structural system, it is unlikely that significant torsional motions developed during the earthquake. Strong-motion acceleration and integrated displacement time histories are shown in Figure 3.

Roof-level accelerations calculated by system identification techniques with N-S and E-W basement-level accelerations as input are shown in Figure 4, along with the Fourier amplitude spectra. The fundamental N-S translational mode is clearly identified at 0.76 Hz and higher modes are apparent at 2.31 and 4.02 Hz. Analyses of the records from the lower floors showed the spectral peak at 2.31 Hz to be associated with the 2nd translational mode.

For the E-W direction, the 1st and 2nd translational modes are centered at 0.76 and 2.28 Hz, respectively, and a higher mode occurs at 3.81 Hz. Identical frequencies for the N-S and E-W 1st translational modes suggest the possibility of a torsional component as well, although this could not be confirmed from the records available for analysis. The pronounced spectral peak at 3.81 Hz is apparent at each instrumented floor level and is the predominant frequency at the first floor level. This suggests some participation of the connecting bridge structure described earlier.

Modal components of the roof-level accelerations calculated by system identification techniques (solid lines) are compared with recorded accelerations (dotted lines) in Figure 5. Also indicated in Figure 5 are the modal damping ratios (percent of critical) for the first three modes in the N-S and E-W directions, respectively. Locations of the accelerometers from which the most significant time histories were obtained and results of the response analysis, based in large part on these time histories, are given in Table 2.

TABLE 2. RESULTS OF LOMA PRIETA EARTHQUAKE RESPONSE ANALYSIS FOR CSUH ADMINISTRATION BUILDING, HAYWARD

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-Center	N-S	1 (Translation)	0.76	3.4
		2 (Translation)	2.31	3.4
		3	4.02	6.4
Roof-Center	E-W	1 (Translation)	0.76	2.3
		2 (Translation)	2.28	3.9
		3	3.81	4.7

7.1.2 Ambient Vibration Data

Ambient vibration response records and Fourier amplitude spectra at roof level for the N-S and E-W directions are shown in Figure 6. The 1st and 2nd N-S translational modes are centered at 0.92 and 2.71 Hz, respectively. While there appears to be a scaling difference between the accelerometers located at the center and at the east side of the roof slab, the locations of the spectral peaks in the two records are consistent. In the E-W direction spectral peaks occur at 0.86, 2.44 and 2.54 Hz with a lesser peak at 3.95 Hz. The first two peaks are believed to represent the 1st and 2nd translational modes, based on spectra obtained from lower floors.

Estimates of structural damping were obtained from the ambient vibration records using selective filtering and auto-correlation techniques. A typical auto-correlation curve and least-squares fit to the amplitude decay are shown in Figure 7. Results obtained from the ambient vibration measurements are summarized in Table 3.

TABLE 3. RESULTS OF AMBIENT VIBRATION ANALYSIS FOR CSUH ADMINISTRATION BUILDING, HAYWARD

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-Center	N-S	1 (Translation)	0.92	0.6
		2 (Translation)	2.71	
Roof-Center	E-W	1 (Translation)	0.86	0.6
		2 (Translation)	2.44	
		3	2.54	
		4	3.95	

7.2 Santa Clara County Office Building, San Jose

The Santa Clara County Office Building is a 12-story moment-resisting steel frame structure with a concrete mat foundation. The building has a basic square planform with elevator and stair towers on the south and west faces. Plan dimensions are 50.90 x 50.90 m (overall) and the height is 57.15 m. The building arrangement and instrumentation scheme, also installed by CDMG, are shown in Figure 8.

7.2.1 Strong-Motion Data

Instrument-corrected acceleration time histories and the corresponding integrated displacements are shown in Figures 9 and 10, respectively. The roof-level accelerations, calculated by system identification techniques with lower-level accelerations as inputs, are shown in Figure 11. Also shown are the Fourier amplitude spectra. Fourier amplitude spectra for intermediate floors and the vertical component at the lower level are shown in Figure 12.

The fundamental translational mode in the N-S direction is centered at 0.45 Hz. What appears to be the second translational mode is centered at 1.33 Hz. The corresponding frequencies in the E-W direction also are 0.45 and 1.33 Hz. Analysis of the difference signals for accelerometers with parallel orientations indicates that the mode at 0.45 Hz is actually a combined translation/torsion mode. No other significant peaks are apparent in the translational response spectra.

The frequency of 0.45 Hz also appears as a clearly defined peak on the Fourier amplitude spectra of the vertical acceleration records taken on the concrete base mat and designated as CH 1-UP and CH 3-UP (see Figure 12). This suggests a rocking motion in both the N-S and E-W directions. There does not appear to be any rocking motion involved with the 2nd translational mode. Results of the strong-motion analysis and damping estimates obtained with system identification techniques are summarized in Table 4.

TABLE 4. RESULTS OF LOMA PRIETA EARTHQUAKE RESPONSE ANALYSIS FOR SANTA CLARA COUNTY OFFICE BUILDING, SAN JOSE

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-SE	N-S	1 (Trans/Torsion)	0.45	2.7
		2 (Translation)	1.33	
Roof-NW	E-W	1 (Trans/Torsion)	0.45	2.7
		2 (Translation)	1.33	

7.2.2 Ambient Vibration Data

N-S acceleration time histories and Fourier amplitude spectra for two locations at roof-level (channels 6 and 7) are shown in Figures 13 and 14, respectively. The spectrum for channel 6 exhibits clear peaks at 0.52, 0.68 and 1.59 Hz. Channel 7, on the other hand, exhibits these peaks as well as an additional peak at 1.95 Hz. In the E-W direction, channel 4 exhibits spectral peaks at 0.52, 0.68, 1.61 and 1.95 Hz (see Figure 15). Channel 5 exhibits spectral peaks at 0.52, 0.68 and 1.61 Hz (see Figure 16).

The spectral peaks at or near 0.52 Hz in the N-S and E-W directions are evident for all instrumented floors down to the 2nd-floor level. Analysis of difference signals indicates that these spectral peaks correspond to the 1st translational modes. Likewise, the spectral peak at 0.68 Hz is evident on all channels down to the 2nd-floor level, suggesting a torsional mode. This was confirmed by spectral analysis of the difference signal between accelerometers having parallel orientations. The spectral peaks at or near 1.59 Hz N-S and 1.61 Hz E-W are identifiable at these same levels and vary in amplitude consistent with 2nd translational modes. The spectral peak at 1.95 Hz suggests torsional response with the center of rotation located between the geometric center and the SW corner of the building, consistent with the stiffness imparted by the elevator/stair towers along the south and west sides of the building. It was not possible to extract consistent estimates of structural damping for this building using the auto-correlation technique. In view of the results obtained from the analysis of the strong-motion records, this problem possibly is due to the presence of a small torsional component in the spectral peaks observed at 0.52 Hz for both the N-S and E-W directions. Results of the analysis of ambient vibration data are summarized in Table 5.

TABLE 5. RESULTS OF AMBIENT VIBRATION ANALYSIS FOR SANTA CLARA COUNTY OFFICE BUILDING, SAN JOSE

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-SW	N-S	1 (Translation)	0.52	---
		2 (Torsion)	0.68	
		3 (Translation)	1.59	
Roof-SE	N-S	1 (Translation)	0.52	---
		2 (Torsion)	0.68	
		3 (Translation)	1.59	
		4	1.95	
Roof-NW	E-W	1 (Translation)	0.52	---
		2 (Torsion)	0.68	
		3 (Translation)	1.61	
		4	1.95	
Roof-SW	E-W	1 (Translation)	0.52	---
		2 (Torsion)	0.68	
		3 (Translation)	1.61	

7.3 Commercial Office Building, San Bruno

This building contains precast, post-tensioned floor beams acting integrally with the floor slabs, and the perimeter columns are cast in cavities formed by the precast wall panels. A heavily reinforced E-W interior frame is located 4.88 m south of the geometric center of the building. This eccentricity introduces a twisting component to the E-W response. Plan dimensions are 60.96

x 27.43 m (N-S x E-W) and the building height is 23.77 m. The foundation consists of individual spread footings located just below the ground floor which is a slab on grade. This 6-story structure is instrumented by CDMG and the overall dimensions and instrumentation scheme are shown in Figure 17.

7.3.1 Strong-Motion Data

Instrument-corrected acceleration time histories and the corresponding integrated displacements are shown in Figure 18. The N-S and E-W roof-level accelerations calculated by system identification techniques with the ground-level (1st floor) accelerations as input are shown in Figure 19. Also shown in this figure are the Fourier amplitude spectra.

In the N-S direction the spectral peak at 0.76 Hz is present at approximately the same amplitude on all N-S spectra, including the base slab, and is judged to be a soil system resonance frequency. Thus the building exhibits nearly uniform translational motion at this frequency, consistent with the ground motion. The 1st translational mode of the building in the N-S direction is apparent at 1.17 Hz. The peaks at 1.37 and 1.87 Hz have not been identified.

In the E-W direction the 1st translational mode of the building is identified at 0.98 Hz. This mode also involves a twisting effect due to the fact that the elastic center of the building is located slightly south of the geometric/mass center, resulting in higher accelerations along the north wall than are observed at the geometric center of the building. The spectral peak at 1.32 Hz can be shown to be a torsional mode by examining the difference signal between accelerometers having N-S separation.

Damping estimates for the fundamental modes were obtained by system identification techniques. Results of the strong-motion data analysis are summarized in Table 6.

TABLE 6. RESULTS OF LOMA PRIETA EARTHQUAKE RESPONSE ANALYSIS FOR COMMERCIAL OFFICE BUILDING, SAN BRUNO

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-Center	N-S	1 (Soil resonance)	0.76	
		2 (Translation)	1.17	7.2
		3	1.37	
		4	1.87	
Roof-Center	E-W	1 (Translation)	0.98	4.1
		2 (Torsion)	1.32	

7.3.2 Ambient Vibration Data

Ambient vibration response records and the corresponding Fourier amplitude spectra at roof level for the N-S direction and for the E-W direction at the center of the building and at the north wall are shown in Figure 20. For the N-S direction there is only one clear spectral peak (at 1.72 Hz) and this is judged to be the 1st translational mode of the building in that direction. In the E-W direction there are spectral peaks at 1.41 and 1.95 Hz. The peak at 1.41 Hz is judged to be the 1st translational mode while the peak at 1.95 Hz can, on the basis of difference signals, be shown to be a torsional mode. The spectral peaks at frequencies higher than 4 Hz are believed to be due to mechanical equipment vibrations that were present at the time of the ambient vibration survey.

Estimates of structural damping obtained by the auto-correlation technique are 2.2 percent of critical for the N-S direction and 2.3 percent of critical for the E-W direction. Results of the ambient vibration analyses are summarized in Table 7.

TABLE 7. RESULTS OF AMBIENT VIBRATION ANALYSIS FOR COMMERCIAL OFFICE BUILDING, SAN BRUNO

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
Roof-Center	N-S	1 (Translation)	1.72	2.2
Roof-Center	E-W	1 (Translation)	1.41	2.3
		2 (Torsion)	1.95	

7.4 Transamerica Building, San Francisco

Overall dimensions and the instrumentation layout for the Transamerica Building are shown in Figures 21 and 22. For a detailed description of this building, instrumented by USGS, and the analysis of the Loma Prieta earthquake response records, see Çelebi and Şafak (1991) and Şafak and Çelebi (1991). A summary of their work is presented here.

7.4.1 Strong-Motion Data

Processed records of translational motions (accelerations and displacements) during the Loma Prieta earthquake are shown in Figure 23. Peak accelerations of 0.29 g N-S and 0.28 g E-W were measured at the 49th floor. The corresponding peak displacements were 11.3 and 18.6 cm. Recorded and calculated (by system identification) responses in the N-S and E-W directions at the 49th floor with basemat accelerations as input are shown in Figure 24. Also shown in this figure are the corresponding Fourier amplitude spectra.

Analysis of the strong-motion records shows the response is dominated by a translational mode at 0.28 Hz in the NE-SW direction, almost parallel to one of the building's diagonals. The motion in the E-W direction has an additional dominant mode at 0.52 Hz and higher modes of vibration were excited in both directions during the strong-motion portion of the earthquake. There is no evidence of any torsional response. However, a significant amount of rocking motion is observed at a frequency of 2.0 Hz in the N-S direction and 1.8 Hz in the E-W direction. Principal findings of the strong-motion analysis are summarized in Table 8.

TABLE 8. RESULTS OF LOMA PRIETA EARTHQUAKE RESPONSE ANALYSIS FOR TRANSAMERICA BUILDING, SAN FRANCISCO

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
49th Floor	N-S	1 (Translation)	0.28	4.9
		2	1.00	5.4
		3	1.36	2.3
		4 (Rocking)	2.00	4.4
49th Floor	E-W	1 (Translation)	0.28	2.2
		2	0.52	3.6
		3 (Rocking)	1.80	6.1

7.4.2 Ambient Vibration Data

Several ambient vibration records were obtained from this building following the Loma Prieta earthquake on a day when winds (gusts estimated at 10 to 15 ms^{-1}) were from the southwest. It was not possible to obtain recordings from the SMA units on the 29th and 49th floors simultaneous with outputs from accelerometers at lower levels. However, individual recordings were obtained from the 29th and 48th floors using portable accelerometers.

Typical acceleration records and the corresponding Fourier amplitude spectra from the 48th floor are shown in Figure 25. Dominant frequencies for the N-S direction occur at 0.34, 0.61, 0.78 and 1.94 Hz. Based on records from the 29th floor and on simultaneous records from the 21st and lower floors, the frequency at 0.34 Hz is identified as the fundamental translational mode. The next two frequencies are associated with higher translational modes while the value of 1.94 Hz possibly is a rocking mode in view of the strong-motion results previously described. A typical acceleration time history (21st floor, west side, N-S direction) and Fourier amplitude spectrum are shown in Figure 26.

In the E-W direction, the dominant modes occur at 0.32, 0.60, 1.06 and 1.94 Hz. Again, these values represent the fundamental translational mode, two higher modes, and possibly a rocking mode. Because of the poor signal-to-noise ratio at the ground floor and lower levels, it was not possible to confirm any rocking mode in the signals from those accelerometers.

Because of the limited length of record exhibiting an acceptable degree of stationarity, it was not possible to obtain consistent estimates of damping for this building at frequencies below 1 Hz. The damping estimates reported here are associated with frequencies in the range 1.5 to 2 Hz and a typical auto-correlation curve and least squares fit to the amplitude decay are shown in Figure 27.

In a forced vibration study reported by Stephen et al. (1974), five translational modes in the N-S direction and in the E-W direction were identified for this building. The modal frequencies ranged from 0.345 to 1.85 Hz for each direction and the corresponding damping estimates were obtained by the free decay technique. These estimates ranged from 0.9 to 1.5 percent of critical for the N-S direction and from 0.7 to 1.8 percent of critical for the E-W direction. The 1st-mode estimates were 0.9 (N-S) and 1.4 (E-W). The forced vibration study was carried out shortly after completion of construction, but prior to occupancy and installation of office partitions. However, it is unlikely that occupancy effects on damping would be significant for the amplitudes associated with ambient vibrations and, therefore, the estimate of 2.2 percent of critical obtained in the current study probably is high. Results of the ambient vibration analysis are summarized in Table 9 with the damping estimate of 2.2 for the E-W direction replaced by the lower 1st-mode value of 1.4 percent of critical.

TABLE 9. RESULTS OF AMBIENT VIBRATION ANALYSIS FOR TRANSAMERICA BUILDING, SAN FRANCISCO

Transducer Location	Direction	Mode	Frequency (Hz)	Damping (% Critical)
21st Floor	N-S	1 (Translation)	0.34	0.8*
		2	0.61	
		3	0.78	
		4	1.94	
21st Floor	E-W	1 (Translation)	0.32	1.4
		2	0.60	
		3	1.06	
		4	1.94	

* Based on frequency range of 1.5 to 2 Hz.

7.5 Pacific Park Plaza, Emeryville

Pacific Park Plaza is a 30-story reinforced concrete moment-frame/shear-wall structure consisting of a central core and three radial wings with equal angular spacing of 120 degrees. Reference north is 350 degrees and the axis of the west wing is oriented at 260 degrees from true north. Plan dimensions of each wing are 17.1 x 33.92 m overall and the building height is 94.18 m. Shear walls extend to the 2nd-floor level in the central core and in each of the wings. The foundation consists of a 1.52 m thick concrete mat on friction piles driven along the column lines. The soil profile consists of Bay Mud to a depth of 33-50 m, followed by very stiff to hard silty clay (old Bay Mud). Two free-field sites are operated in conjunction with this site, one approximately 50 m to the northwest and another approximately 50 m directly south of the building site. The building, instrumented by USGS, contains 21 permanently-installed accelerometers. The building layout and arrangement of instrumentation are shown in Figures 28 and 29, respectively.

7.5.1 Strong-Motion Data

Acceleration records obtained from this building during the Loma Prieta earthquake have been analyzed and reported by Çelebi and Şafak (1992) and by Şafak and Çelebi (1992). Principal findings resulting from these studies are summarized here. Instrument-corrected time histories of acceleration are shown in Figure 30 and integrated displacements are shown in Figure 31. Recorded and calculated (by system identification) N-S and E-W central core responses at the 30th floor with recorded ground-level accelerations as inputs are shown in Figure 32. Also shown in this figure are the corresponding Fourier amplitude spectra.

The response records reveal significant torsional motion. In fact, all of the predominant modes of vibration appear to involve translational motion of the central core combined with relative motion (flapping) of the building wings. Certain dominant structural frequencies are identifiable on the Fourier amplitude spectra from both of the free-field sites, indicating significant feedback from the structural motions. The dominant motion of the building follows an elliptical path oriented in the NW-SE direction with a frequency centered at 0.34 Hz.

Cross-spectra for horizontal ground-level motions and those in the upper floors of the building indicate that soil-structure interaction for this site is characterized by a mode at 0.7 Hz. This mode involves E-W translation of the central core, combined with flapping of the west wing. Based on available information, 0.7 Hz is believed to represent the fundamental frequency of the surrounding soil medium. Similar cross-spectra for vertical components of ground motion and horizontal structural motions did not reveal any significant rocking motion for this building.

Estimates of structural damping ranged from 11 to 15 percent of critical for the 1st translational modes of the central core in the N-S and E-W directions. These high values of damping appear questionable, even when it is considered that contributions to damping from soil-structure interaction and from certain construction details for this building probably are significant. The principal findings from the analyses of the strong-motion records for Pacific Park Plaza are summarized in Table 10.

TABLE 10. RESULTS OF LOMA PRIETA EARTHQUAKE RESPONSE ANALYSIS FOR PACIFIC PARK PLAZA, EMERYVILLE

Modal Frequency (Hz)	Mode Description	Damping (% Critical)
0.38	Translation in NW-SE direction.	N-S 11.6 E-W 15.5
0.7	Translation in E-W direction with flapping of the west wing.	
1.94	Same as mode at 0.7 Hz, but mode shape changes sign at 21st floor, suggesting 2nd translational mode.	11

7.5.2 Ambient Vibration Data

Ambient vibration time histories for the three wings and central core are shown in Figure 33. Also included in the figure are the Fourier amplitude spectra. An examination of the acceleration spectra for the central core shows a strong peak at 0.48 Hz. This is the case for both the N-S and E-W components on the 30th, 21st and 13th floors. Furthermore, this same peak is clearly identifiable at each of the instrumented floors in each wing. A second spectral peak is located at 1.35 Hz in the N-S direction and at 1.44 Hz in the E-W direction. These peaks pass through zero near the 21st floor, indicating 2nd-mode translational response. For the wings, these peaks occur over the range 1.35 to 1.47 Hz, indicating possible flapping motion combined with the 2nd-mode translation of the central core. It was not possible to extract with any consistency the modal peaks from the ground-floor spectra.

Estimates of structural damping were obtained using the previously described technique of selective filtering and auto-correlation function. For the 30th floor, these estimates ranged from 0.6 (N-S) to 3.4 (E-W) for the central core and from 0.7 to 1.6 for the tower wings.

Forced and ambient vibration tests were carried out on Pacific Park Plaza by Stephen et al. (1985). First-mode frequencies in the N-S direction were found to be 0.590 and 0.586 Hz from the forced vibration and ambient vibration tests, respectively. The corresponding frequencies in the E-W direction were found to be 0.595 and 0.586 Hz. Damping values of 1.7 and 2.6 percent of critical for the N-S direction were obtained from the forced vibration and ambient vibration tests, respectively. The corresponding values for the E-W direction were 1.8 and 2.6 percent of critical. The highest value of damping obtained from these tests (ambient vibration) was 3.8 percent of critical for the 1st torsional mode at 0.586 Hz.

A summary of the response frequencies obtained from the ambient vibration data for Pacific Park Plaza is presented in Table 11.

TABLE 11. RESULTS OF AMBIENT VIBRATION ANALYSIS FOR PACIFIC PARK PLAZA, EMERYVILLE

Modal Frequency (Hz)	Mode Description	Damping (% Critical)
0.48	First N-S translational mode, central core and wings.	0.6
0.48	First E-W translational mode, central core and wings.	3.4
1.35	Second N-S translational mode, central core.	
1.44	Second E-W translational mode, central core.	
1.35 to 1.47	Combined central core second translational mode and flapping motion of tower wings.	

7.6 Summary of 1st-Mode Response

Results of the analyses carried out on strong-motion and ambient vibration records for the five buildings included in this study are summarized in Table 12. Also included in Table 12 are the observed 1st-mode frequency ratios, f_{LPE}/f_{Amb}.

TABLE 12. SUMMARY OF RESULTS OF ANALYSIS FOR 1ST-MODE RESPONSE

Building	Nominal Direction	Loma Prieta Earthquake f(Hz)	Loma Prieta Earthquake ζ(%)	Ambient Vibration f(Hz)	Ambient Vibration ζ(%)*	Frequency Ratio (f_{LPE}/f_{Amb})
(1) CSUH Admin. Building Hayward, CA	N-S	0.76	3.4	0.92	0.6	0.83
	E-W	0.76	2.3	0.86	0.6	0.88
(2) Santa Clara County Office Building San Jose, CA	N-S	0.45	2.7	0.52	**	0.87
	E-W	0.45	2.7	0.52	**	0.87
(3) San Bruno Office Building San Bruno, CA	N-S	1.17	7.2	1.72	2.2	0.68
	E-W	0.98	4.1	1.41	2.3	0.70
(4) Transamerica Building San Francisco,CA	N-S	0.28	4.9	0.34	0.8	0.82
	E-W	0.28	2.2	0.32	1.4	0.88
(5) Pacific Park Plaza Emeryville, CA	N-S	0.38	11.6	0.48	0.6	0.79
	E-W	0.38	15.5	0.48	3.4	0.79

* Determined by auto-correlation technique
** Unidentifiable due to combined modes

8. FREQUENCY RATIO

From Table 12 it is seen that the CSUH Administration Building, Santa Clara County Office Building, and the Transamerica Building all have similar frequency ratios (strong-motion/ambient), averaging about 0.86. The first building has a steel moment-frame core and the others are purely steel frame structures. The average frequency ratios for Pacific Park Plaza and the San Bruno Office Building are 0.79 and 0.69, respectively. If the reductions in resonance frequency with displacement amplitude are attributed entirely to changes in structural stiffness, these stiffness changes are then approximately equal to the square of the frequency ratio. This "stiffness" ratio ranges from a low of 0.48 for the 6-story San Bruno Office Building (N-S) to a high of 0.77 for the CSUH Administration Building and the Transamerica Building.

8.1 Effect of Soil-Structure Interaction

Of course the contributions of soil/structure interaction to the observed frequency ratios cannot be totally discounted for the five buildings in the current study. The mean frequency ratio of 0.69 for the San Bruno Office Building can in large part be attributed to vertical motions of the individual spread footings as is described later in Section 9 of this report.

The frequency ratio of 0.79 for Pacific Park Plaza can also be attributed, at least in part, to soil-structure interaction. This structure is supported by a 1.52 m thick concrete mat and friction piles driven beneath and along the column lines into several layers of silty, fine sand fill. In their analysis of the recorded response during the Loma Prieta earthquake, Çelebi and Şafak (1992) have shown that although the foundation experienced some relative vertical motion between the central core and the wings, these motions are within the error margin of signal processing and integration. This was confirmed by comparing amplitudes of cross-spectra of vertical motions with their auto-spectra and, therefore, rocking motions in this case were negligible. Additional studies by Şafak and Çelebi (1992) indicate that the soil-structure interaction at Pacific Park Plaza is characterized by a spectral peak at 0.7 Hz which is believed to represent the fundamental frequency of the surrounding soil medium. Also, additional evidence of soil/structure interaction at this site is provided by the strong correlation of certain frequency components in the structural response records at the mat foundation with those in the free-field ground acceleration records.

8.2 Other Factors

In the case of the San Bruno Commercial Office Building and Pacific Park Plaza, there are other factors that could have contributed to the relatively large changes in 1st-mode frequencies. These include micro-cracking of the concrete, although no visible damage was experienced in either structure, and possibly joint slip in the precast wall panels. At San Bruno the exterior cast-in-place columns were formed by precast wall panels and it is possible that some relative slip developed between these panels and the monolithic column cores at large displacements. This possibility is discussed in more detail in Section 9.

It is of interest to compare the observations made in the current study with those made by Wood (1972) on a 9-story steel frame building in Pasadena, CA, following the San Fernando earthquake of February 9, 1971. In that case the plan dimensions were 12.2 x 67.1 m (N-S, E-W) and the structural height above the basement walls was 39.6 m. Peak accelerations were 0.22 g (N-S) and 0.39 g (E-W) at roof level, and 0.14 g (N-S) and 0.21 g (E-W) at basement level. Roof-top displacements were not calculated directly from the acceleration records but were estimated to be at least 6.5 cm in each direction. Effects of soil-structure interaction in the case of this building were shown to be negligible.

Resonance frequencies for the fundamental translational modes for this building during the earthquake were 0.69 Hz (N-S) and 0.78 Hz (E-W). Subsequent ambient vibration measurements showed these frequencies to be 0.87-0.90 Hz and 0.95-1.00 Hz, respectively. The corresponding frequency ratios (earthquake/ambient) are 0.78 and 0.80, suggesting a reduction in stiffness of about 38 percent during the earthquake. A detailed analysis showed that most of this observed reduction in stiffness could be attributed to nonlinear behavior of the perimeter columns which were partially encased in concrete. Until more definitive information becomes available, it would not be appropriate to conclude from the results presented herein that large reductions in resonance frequency during strong-motion events are unique to reinforced concrete structures.

Frequency ratios (f_{LPE}/f_{Amb}), peak displacement ratios (δ_{pk}/H), and mean peak displacement ratios (δ_{mpk}/H) for the five buildings included in the current study and the Pasadena building excited during the San Fernando earthquake are listed in Table 13. See Table 1 for values of building height, peak displacements, and mean peak-to-peak displacements.

TABLE 13. SUMMARY OF 1ST-MODE FREQUENCY RATIOS AND DISPLACEMENT RATIOS

Building	Nominal Direction	Frequency Ratio	δ_{pk}/H (x 1000)	δ_{mpk}/H (x 1000)
(1) CSUH Admin. Building Hayward, CA	N-S	0.83	0.57	0.41
	E-W	0.88	0.54	0.29
(2) Santa Clara County Office Building San Jose, CA	N-S	0.87	7.77	2.63
	E-W	0.87	7.30	3.16
(3) San Bruno Office Building San Bruno, CA	N-S	0.68	2.67	1.04
	E-W	0.70	2.71	1.88
(4) Transamerica Building San Francisco,CA	N-S	0.82	0.55	0.41
	E-W	0.88	0.90	0.64
(5) Pacific Park Plaza Emeryville, CA	N-S	0.79	0.73	0.53
	E-W	0.79	2.46	0.93
(*) Nine Story Steel Frame Building Pasadena, CA	N-S	0.78	-	1.64
	E-W	0.80	-	1.64

* San Fernando Earthquake, February 9, 1971

8.3 Effect of Displacement Amplitude on Frequency Ratio

To gain additional understanding of 1st-mode frequency reduction with displacement amplitude, particularly at lower values of the displacement ratio, it is useful to consider the results of forced vibration tests carried out by Ellis and Littler (1988) on a 23.7(N-S) x 17.9(E-W) x 64 m high precast concrete panel building. Their results are summarized in Table 14. Frequencies and peak displacements at resonance for the 1st translational modes (N-S and E-W) are listed in Table 14. Also listed are the 1st-mode frequencies extracted from ambient vibration records (mean wind speed = 6.0 ms^{-1}), the ratios of the mean peak displacement to building height (δ_{mpk}/H), and damping estimates based on the forced vibration amplitude decay curves. Note that for steady-state forced vibrations the peak displacement and the mean peak displacement are equal. These data and the data obtained from the 6 buildings listed in Table 13 are plotted as frequency ratio (f/f_{Amb}) vs. mean peak displacement/height (δ_{mpk}/H) in Figure 34. It can be seen from the plot that the values of δ_{mpk}/H associated with the Loma Prieta earthquake range from 10 to 100 times the highest values obtained by Ellis and Littler in their carefully conducted forced vibration tests.

TABLE 14. FORCED VIBRATION TEST DATA FOR A 64-m TALL PRECAST CONCRETE PANEL BUILDING (Ellis and Littler, 1988)

Mode	Frequency (Hz)	Frequency Ratio (f/f_{Amb})	Peak Displacement (mm)	δ_{mpk}/H (x 1000)	Decay Damping (% Crit.)
N-S(1)	1.100	0.965	2.4	0.038	1.20
	1.100	0.965	2.0	0.031	1.16
	1.105	0.969	1.5	0.023	1.13
	1.105	0.969	1.0	0.016	1.15
	1.110	0.974	0.55	0.009	1.15
	1.120	0.982	0.30	0.005	1.05
	1.14 (ambient vibration test)				
E-W(1)	0.905	0.973	2.7	0.042	0.89
	0.905	0.973	2.2	0.034	0.89
	0.910	0.978	1.6	0.025	0.94
	0.910	0.978	1.2	0.019	0.89
	0.915	0.984	0.62	0.010	0.86
	0.920	0.989	0.34	0.005	0.76
	0.93 (ambient vibration test)				

9. COMPUTER MODEL OF SAN BRUNO COMMERCIAL OFFICE BUILDING

To better understand the large shift in modal frequencies of the San Bruno Commercial Office Building with increasing level of response (frequency ratio = 0.68 N-S and 0.70 E-W), the building was modeled using the finite element technique. A detailed description of the model and results of modal and dynamic analyses are described by Phan et al. (1992). Only the major features of the model and results are described here.

The finite element analysis module of Patran P/FEA (PDA Engineering, 1989) was used in the modeling and analyses. In creating the computer model, all structural components were discretized into finite elements interconnected at corner or end nodes to form the three-dimensional geometry. A total of 484 nodes, 837 beam, column and bar elements, and 336 plate elements were used in the model.

Columns and beams were modeled using a linear two-node bar element with 6 degrees of freedom at each node. Column section properties, including the cross-sectional area and moments of inertia with respect to the major and minor axes, were calculated using transformed and uncracked sections. The material properties include a concrete modulus of 28,100 MPa, based on a design strength of 34.47 MPa. The monolithic perimeter columns are encased by proprietary precast wall panels which were not included in the model because of their unknown properties. Thus, the overall stiffness of the building will be underestimated if these panels act integrally with the cast-in-place column sections. There are 13 different column cross-sections in the model.

Transformed, uncracked section properties also were used for the perimeter cast-in-place beams and the interior post-tensioned beams. A total of 4 different sets of section properties were used for the cast-in-place perimeter beams and the post-tensioned interior beams. The interior beams, which have 10, 11 or 12 tendons, were modeled with a transformed section based on 11 tendons.

The floor and roof slabs were modeled using 4-node quadrilateral and 3-node triangular plate bending elements. There are 6 degrees of freedom per corner node. Actual slab thickness was used for the plate elements to represent the stiffness, and modified mass densities were used to account for the floor and roof dead and live loads.

Both modal and transient dynamic analyses were performed. To accomplish this, it was necessary to link the model through beam elements of infinite stiffness to an artificially large mass located at ground level. The size of the mass was selected to be several times that of the total structure, but limited so that the response accelerations calculated for the column bases did not differ significantly from the acceleration record used to drive the mass and linked building.

9.1 Modal Analysis: Fixed-Base Model

Modal analyses were performed using two different sets of boundary conditions; a fixed-base condition and a spring-supported condition. In the first case, the column bases were assumed to be fixed to the rigid links as this is believed to be the best representation of the ambient vibration condition for which the effects of soil-structure interaction are completely ignored. Results of the modal analysis for the fixed-base model are summarized in Table 15. Also listed in Table 15 for comparison are the frequencies identified in the analysis of the ambient vibration records.

Although the first three predicted and observed modes are in the proper order, it is seen from Table 15 that the natural frequencies identified for ambient vibration conditions are approximately 25 percent higher than the predicted values. Refinement of the model would, no doubt, provide better agreement. For example, as was noted earlier, the column stiffnesses probably are underestimated by ignoring the contribution of the precast wall panels. However, it is the relative change in natural frequencies with modeling assumptions that is of interest here and, consequently, no "fine tuning" of the original model was carried out at this stage.

TABLE 15. NATURAL FREQUENCIES FOR FIXED-BASE CONDITION, SAN BRUNO COMMERCIAL OFFICE BUILDING

Mode Number	Natural Frequency (Model-Hz)	Mode Description	Natural Frequency (Ambient-Hz)
1	1.217	E-W Translation	1.41
2	1.283	N-S Translation	1.72
3	1.599	Torsion	1.95
4	3.74	E-W Bending with floor twisting	
5	3.99	N-S Bending with floor twisting	

9.2 Modal Analysis: Spring-Supported Model

The potential influence of soil-structure interaction on the response of the San Bruno Commercial Office Building was evaluated using a spring-supported model. To accomplish this, springs were inserted between the column bases and the infinitely rigid links to the artificial mass. The modulus of subgrade reaction was modeled by 48 vertical linear springs of finite extensional stiffness, and 96 linear springs (48 N-S and 48 E-W) were used to represent the lateral and rotational stiffness of the building at the foundation level.

The average load per footing was estimated by dividing the total load (dead plus live) for the building (9,278 tons) by the number of individual spread footings (48) for an average load of 193 tons per footing. An average subgrade settlement of 4.2 mm was assumed for the soil conditions at the site and a corresponding extensional stiffness of 448 MN/m was assigned to each vertical spring (Bowles, 1977).

The total lateral stiffness at the 1st story was computed using a concrete elastic modulus of 28,100 MPa and a column height of 4.3 m. The resulting lateral 1st-story stiffnesses are 3.038 x 10^6 kN/m N-S and 3.002 x 10^6 kN/m E-W. Note that the contribution of the precast wall panels to the stiffness of the perimeter columns has not been included in these estimates. The corresponding lateral spring stiffnesses were determined to be 63,300 kN/m N-S and 62,500 kN/m E-W.

Because of the uncertainties involved with the selection of a rotational spring stiffness with which to model the behavior of the individual spread footings, a range of stiffnesses was assumed in the analysis of the spring-supported case. Results of the analysis are summarized in Table 16.

TABLE 16. NATURAL FREQUENCIES FOR SPRING-SUPPORTED CONDITION, SAN BRUNO COMMERCIAL OFFICE BUILDING

Mode Number	Rotational Stiffness (kN-m/rad)				
	1×10^8	1×10^7	1×10^6	0.5×10^6	1×10^3
1	0.895	0.873	0.871	0.871	0.870
2	0.932	0.910	0.908	0.908	0.908
3	1.153	1.131	1.129	1.129	1.129
4	3.064	3.035	3.032	3.032	3.032
5	3.238	3.211	3.210	3.208	3.208

From the results listed in Table 16, it is obvious that the rotational stiffness of the individual spread footings has relatively little effect on the modal frequencies for the San Bruno Commercial Office Building. However, the modulus of subgrade reaction appears to have a pronounced effect on these frequencies as can be seen by comparing the values in Table 16 with those of Table 15 which were obtained for the fully fixed condition, i.e., no mobilization of soil compression under ambient vibration conditions.

Frequency ratios for the first three modes identified from the Loma Prieta and ambient vibration records, f_{LPE}/f_{Amb}, ranged from 0.68 to 0.70. For the modal analysis using the finite element model, the frequency ratio based on spring-mounted and fully-fixed boundary conditions, f_s/f_f, ranges from 0.71 to 0.81 for the first five modes, assuming a rotational spring stiffness of 1×10^3 kN-m/rad. At most, this ratio is only 3 percent greater when the rotational spring stiffness is increased by a factor of 100,000.

While the observed frequency ratio for the San Bruno Commercial Office Building can be explained in terms of soil-structure interaction using realistic values for the modulus of subgrade reaction, it is likely that other factors also contribute to this reduction in frequency with increasing displacement amplitude. As was noted earlier, these factors include the possibility of slip between the core and the precast panels at the exterior columns, joint slip at the precast floor beams, and micro-cracking of the concrete. Similar analyses could be carried out for Pacific Park

Plaza which also exhibits a significant reduction in modal frequencies with displacement amplitude. However, the required model would be considerably more complicated in view of the foundation system which includes a concrete mat and friction piles.

10. STRUCTURAL DAMPING

Estimates of structural damping (percent of critical) for the first translational modes and based on analyses of response records obtained from the Loma Prieta earthquake and from subsequent ambient vibration records were summarized in Table 12. In the following paragraphs these estimates are compared with predictions based on both small and large amplitudes of displacement.

10.1 Comparison of Damping Estimates With a Damping Model for Small Amplitudes of Displacement

Engineering Sciences Data Unit (ESDU) Item 83009 (1983) contains recommendations for the selection of structural damping ratios for tall buildings subjected to wind excitation, i.e., small displacement amplitudes. The bases for these recommendations are the results of carefully conducted forced vibration tests and ambient vibration measurements conducted on both steel and reinforced concrete buildings representing a range of structural systems, building heights and aspect ratios. In the damping model used to represent these full-scale test results, the modal structural damping ratio, ζ_{s0}, for the 1st translational mode of vibration is assumed to vary linearly with modal frequency according the expression

$$\zeta_{s0} = f/K \qquad (10.1)$$

where f is the modal frequency in Hz, K = 100 for the "most probable" values and K = 250 for the expected lower limit of the structural damping ratio. The range defined by K = 60 to K = 250 includes 90 percent of the data used to formulate the ESDU model. The upper limit of structural damping ratio recommended in ESDU 83009 is given by the expression

$$\zeta_s \leq 60/(100 \times H) + 0.013 \qquad (10.2)$$

where H is the height of the structure in meters.

Estimates of structural damping (percent of critical) obtained for the five buildings in this study are compared with the ESDU damping model in Table 17. Also included for comparison are the forced vibration test results for the precast concrete panel building described in Section 8.3 (Table 14). The damping estimates based on ambient vibration data are plotted against the corresponding 1st-mode translational frequencies in Figure 35. Also plotted in Figure 35 are the lower limit, most probable values, and the 90

TABLE 17. COMPARISON OF MODAL DAMPING ESTIMATES WITH ESDU MODEL

BUILDING	DIR	HEIGHT (m)	ESTIMATED DAMPING (LPE) (% of Critical)	(AMBIENT)	ESDU SMALL AMPLITUDE RANGE ζ_∞(% Crit.)	ESDU UPPER LIMIT ζ_s(% Crit.)
CSUH ADMIN. BUILDING	N-S	61	3.4	0.6	0.37 - 1.53	2.28
	E-W		2.3	0.6	0.34 - 1.43	2.28
SANTA CLARA CO. BLDG.	N-S	57	2.7	--	0.21 - 0.87	2.35
	E-W		2.7	--	0.21 - 0.87	2.35
SAN BRUNO OFFICE BLDG.	N-S	24	7.2	2.2	0.69 - 2.87	3.80
	E-W		4.1	2.2	0.56 - 2.35	3.80
TRANSAMERICA BUILDING	N-S	257	4.9	0.8	0.14 - 0.57	1.53
	E-W		2.2	2.2	0.13 - 0.53	1.53
PACIFIC PARK PLAZA	N-S	94	11.6	0.6	0.19 - 0.80	1.94
	E-W		15.5	3.4	0.19 - 0.80	1.94
PANELIZED CONCRETE BUILDING (Ellis and Littler)	N-S	64		1.05	0.46 - 1.90	2.24
	E-W			0.76	0.37 - 1.55	2.24

percent range of modal damping ratio with K = 250, 100 and 60, respectively. It can be seen from Figure 35 that, with the exception of the Transamerica Building and the anomalous estimate for Pacific Park Plaza, the estimates of structural damping obtained from ambient vibration records are reasonably consistent with those values covered by the ESDU small amplitude range. Estimates derived from the Transamerica ambient vibration records are consistently higher than the recommended range and this may be due to the gusty wind conditions prevailing at the time of the ambient vibration survey. As expected, the damping estimates derived from the Loma Prieta earthquake response records for the five buildings in this study exceed the ESDU upper-limit damping ratio in every case.

10.2 Comparison of Damping Estimates With Data for Large Displacement Ratios

A summary of damping estimates and corresponding amplitudes of vibration for 165 buildings has been prepared by Davenport and Hill-Carroll (1986). Included in this data base are results from ambient vibration surveys, forced vibration studies, and strong motion measurements obtained under strong wind conditions and from earthquakes. These data are represented by the envelope shown in Figure 36 with the displacements plotted in terms of the rms amplitude divided by the height of the structure. The rms rather than the peak amplitude is used on the grounds that the energy dissipation should be approximately the same for sinusoidal and random vibrations having the same rms amplitude. The upper region of the envelope represents buildings of up to 10 stories in height while the lower region of the envelope represents data obtained from buildings of 20 stories or more in height. Damping estimates obtained from the strong-motion records for the five buildings included in the current study are plotted in Figure 36 using estimates of the mean peak-to-peak amplitudes (see Table 1), averaged over several cycles and converted to rms amplitude by multiplying by 0.707/2. Also plotted in Figure 36 are the largest damping ratios for the forced vibration data listed in Table 14.

Regression analyses carried out by Davenport and Hill-Carroll on selected groupings of the 165 buildings included in their data set suggest that the mean or expected damping ratio can be expressed as

$$\zeta_s = A(\delta_{rms}/H \times 1000)^n \qquad (10.3)$$

in which δ_{rms}/H is the ratio of the rms displacement amplitude to the building height, and A and n are constants. Values of A and n are listed in Table 18 along with the coefficient of variation for the selected groupings. Two of these regression lines (concrete: 5-20 stories, and steel: > 20 stories) are plotted in Figure 36.

TABLE 18. DAMPING PROPERTIES OF BUILDINGS
(Davenport and Hill-Carroll, 1986)

Height (Stories)	Construction	A	n	C.O.V.
5-20	Steel	0.03	0.075	0.40
	Concrete	0.03	0.11	0.40
> 20	Steel	0.02	0.11	0.40
	Concrete	0.025	0.11	0.40

The following trends in the expected or mean values of damping were observed by Davenport and Hill-Carroll in their analysis:

1. Lower buildings (5-20 stories) tend to have roughly 60 percent more damping than do taller buildings over 20 stories. This is largely due, it is expected, to the stronger influence of the foundation on damping in low-rise buildings.

2. Taller concrete buildings tend to have roughly 30 percent more damping than steel buildings. This is a somewhat smaller margin than has been suspected earlier; however, the data for large-amplitude response of taller concrete buildings are limited.

3. Above 20 stories there is no indication of further influence of building height on damping.

4. The increase in damping with amplitude corresponds approximately to a 1/9 or 1/10 power law.

It is seen from Figure 36 that the regression lines in the above analysis represent a reasonable lower bound to the strong-motion damping estimates obtained for the five buildings included in the current study.

10.3 Summary of Damping Measurements

Based on the damping estimates obtained in the current study and on published results of earlier studies covering the range from ambient vibrations to strong-motion response, the following conclusions are drawn:

1. Structural damping (percent of critical) associated with ambient vibrations can be estimated by the expression

$$\zeta_{so} = f/K$$

in which f is the fundamental translational mode frequency in Hz and K is a factor that ranges from 60 to 250.

2. The lower bound of structural damping for strong-motion response ($\delta_{rms}/H \times 1000 > 0.1$) can be described by the relation

$$\zeta_s = 0.025(\delta_{rms}/H \times 1000)^{0.11}$$

in which δ_{rms}/H is the rms displacement divided by the building height.

3. For buildings in which soil-structure interaction plays a significant role, the overall damping for strong motion response may be 3 to 4 times the indicated lower bound for strong-motion response.

11. THE NEED FOR STANDARD METHODS OF OBTAINING, ANALYZING AND INTERPRETING AMBIENT VIBRATION DATA.

The major objective of the work described in this report was to compare dynamic characteristics of selected buildings derived from ambient vibration tests with those derived from strong-motion response records. However, in the case of ambient vibration data, it also was possible to compare results from record to record for a given building. For two of the buildings selected for this study (Transamerica Building and Pacific Park Plaza), it was possible to compare certain test results with earlier tests employing both forced and ambient vibrations. Based on these comparisons, it is considered likely that the measurement techniques and methods of analysis employed are major contributors to the observed variability in test results. By adopting minimum requirements for testing and data analysis, the value of ambient vibration tests and strong-motion response records could be increased substantially.

11.1 Current Standards

<u>Vibration of Buildings - Guidelines for the Measurement of Vibrations and Evaluation of Their Effects on Buildings</u> is a draft standard being proposed as an accredited American National Standard (ANSI, 1990). This document was developed under the jurisdiction of Accredited Standards Committee S2 - Mechanical Shock and Vibration. Its purpose is the establishment of basic principles for carrying out vibration measurements and data processing in the evaluation of vibration effects on buildings.

The draft standard contains guidelines relating to the following topic areas:

- Source-Related Factors to be Considered
- Building-Related Factors to be Considered
- Quantities to be Measured
- Methods of Measurement
- Data Collection, Reduction and Analysis
- Measuring Instruments
- Evaluation of Data

While the fixing or coupling of sensors to structural elements is addressed in detail, the positioning of sensors in a building or other structure is covered only in general terms, and this same limitation applies to the analysis and evaluation of test data. The document does address the effect of damping and modal frequency on bias and variance errors in spectral analysis. However, the types of measurements and the analysis required for the assessment of structural damping are not addressed. Nevertheless, this draft standard does provide a logical base for the development of comprehensive requirements for the assessment of dynamic characteristics of buildings and other structures by means of ambient and forced vibration tests.

Topic areas of the draft standard that require further development and selected sources of information are described in the following sections.

11.2 Sensor Selection and Positioning

Both the strong-motion and ambient vibration records used in this study were obtained from accelerometers permanently installed in the selected buildings under the strong-motion instrumentation programs supported by CDMG and USGS. The strong-motion records were recorded on multi-channel strip-film recorders and subsequently digitized while the ambient vibration records were obtained using a portable, 16-channel digital data recorder. These systems allow for the simultaneous recording of all transducer outputs, thus making it possible to select various pairs of transducers for subsequent signal analysis. This study has demonstrated that with proper signal conditioning the signal-to-noise ratio of permanently installed strong-motion accelerometers (nominal sensitivity of 2.5 volts/g) is sufficiently high to obtain reliable ambient vibration data.

Depending on the complexity of the structural motion to be analyzed, two or more accelerometers may be required. For exploratory ambient vibration surveys, a single accelerometer may suffice while the determination of mode shapes requires the use of at least two accelerometers with one positioned at the top of the building and the other moved to lower floor levels. For combined translational and torsional response, at least three accelerometers are required on a given floor level. For recommendations and guidance in the positioning of accelerometers to assess structural response, see Çelebi et al. (1985).

11.3 Data Acquisition Systems

Dual-channel signal analyzers are available which make possible on-site studies of structural response without the need for recording and subsequent analysis of data sets. These devices are equipped with signal conditioning components, including amplifiers, high-pass, low-pass and band-pass filters. Their processing capabilities include auto- and cross-spectra, auto-and cross-correlation functions, coherence functions and transfer functions. Their application to structural response studies has been described by Diehl (1986).

With the advent of microcomputer-based data acquisition systems, it has become possible to provide on-site "quick-look" signal analysis and multi-channel recording capabilities in a highly portable package. The main advantage of this system is that detailed signal processing can be carried out at a later date using various combinations of data channels. Such a system with an eight-channel capability has been described by Lin and Verser (1987).

11.4 Sampling Rate

It is well known that the sampling rate should be at least twice the highest frequency component in the signal to avoid aliasing errors. Frequency components higher than one-half the sampling frequency (called the Nyquist frequency) are folded back and superimposed on the lower frequencies of the spectrum. To avoid excessively high sampling rates, the signal is filtered using a low-pass filter with a sharp cutoff set at the highest frequency of interest. While a sampling rate equal to twice this frequency will avoid aliasing or foldover errors, a sampling rate of from 5 to 10 times the cutoff frequency may be required to obtain spectral amplitudes with acceptable error levels.

11.5 Length of Record

For a given sampling rate, the length of record or data segment determines the frequency resolution; i.e., the resolution is equal to half the sampling rate (Nyquist frequency) divided by the number of points or samples in the segment. Thus a data segment consisting of 1024 points and sampled at a rate of 50 times a second provides a frequency resolution of 25/1024 = 0.02 Hz. Spectral amplitudes obtained from a data segment are estimates of the true amplitudes and contain errors due to the finite length of the segment and the non-stationary effects that are always present. In fact, it can be shown that the standard deviation of the estimate is of the order of the estimate itself. To increase the reliability of the estimates, it becomes necessary to average the spectra from several stationary data segments obtained under the same conditions of excitation. Depending on the nature of the signal being analyzed, this may require data segments totaling several hours of record. For a detailed discussion of estimation errors and signal processing techniques, see Brook and Wynne (1988).

11.6 Identification of Modes of Vibration

The techniques used in this study suggest that ambient vibration data can in most cases be used to identify the first two or three modes of vibration. However, because the modal frequencies are amplitude dependent, they are only indicators of their strong-motion counterparts. The data plotted in Fig. 34 suggest that the frequencies of the 1st translational modes obtained from ambient vibration testing should be multiplied by a factor of from 0.8 to 0.9 to estimate the corresponding strong-motion frequencies. For those cases where there is reason to believe soil-structure interaction plays a significant role, this factor may range from 0.7 to 0.8. As an aid to identifying the 1st translational mode, use can be made of the empirical relationship

$$f = 46/H$$

where f is the frequency of the 1st translational mode in Hz and H is the height of the building in meters (Ellis and Littler 1987). This relationship is based on measured frequencies obtained from 163 buildings with heights ranging from 6 to 200 meters and fundamental frequencies ranging up to approximately 6 Hz. However, the user should note that errors of 50 percent or more are not uncommon with this type of empirical prediction.

12. SUMMARY AND CONCLUSIONS

12.1 Summary

Ambient vibration records were obtained from five buildings in the San Francisco Bay area approximately one year after the Loma Prieta earthquake of October 17, 1989. These records were obtained using the permanently installed accelerometers that registered strong-motion responses during the earthquake. The buildings selected for this study represent a cross-section of contemporary structural systems, building aspect ratios, construction materials and foundation systems. None of the buildings included in the study exhibited any damage due to strong shaking during the Loma Prieta earthquake.

Digital signal processing was performed on both sets of response records with the objective of determining to what degree dynamic characteristics derived from ambient vibration data represent those characteristics associated with strong-motion response. Both conventional spectral analysis and system identification techniques were employed in the processing of response records.

12.2 Conclusions

The following conclusions are drawn from this study:

- With proper signal conditioning, the sensitivity and signal-to-noise ratio of conventional force-balance accelerometers (FBA) used in strong-motion response studies are sufficient to obtain reliable ambient vibration measurements.

- The techniques used in this study suggest that ambient vibration data can be used to identify the first two or three modes of vibration without difficulty. However, there is a need for the development of comprehensive requirements for the assessment of dynamic characteristics of buildings and other structures by means of ambient and forced vibration tests.

- First-mode frequency ratios, f_{LPE}/f_{Amb}, for the five buildings included in this study range from a high of 0.88 (CSUH Admin. Building and Transamerica Building) to a low of 0.68 (San Bruno Commercial Office Building).

- Strong-motion records for the two buildings exhibiting the lowest 1st-mode frequency ratios contain identifiable soil resonance frequencies; 0.76 Hz in the case of the San Bruno Commercial Office Building and 0.7 Hz in the case of Pacific Park Plaza, suggesting some effect from soil-structure interaction.

- The unusually low 1st-mode frequency ratios (0.68 and 0.70) observed for the San Bruno Commercial Office Building can be attributed in large part to soil-structure interaction associated with the individual spread footings located at shallow depth.

- Results of this study suggest that the frequencies of the 1st translational modes obtained from ambient vibration studies should be multiplied by a factor of from 0.8 to 0.9 to estimate the corresponding strong-motion frequencies. For those cases where there is reason to believe soil-structure interaction plays a significant role, this factor may range from 0.7 to 0.8.

- For all five buildings, the damping ratios estimated from the LPE response records, either by system identification techniques or by auto-correlation techniques, are always higher than those ratios estimated from ambient vibration records.

- In general, estimates of the structural damping ratio (percent of critical) obtained in this study of ambient vibration records are consistent with published results of forced vibration tests (ESDU 1983) which can be described by the expression

$$\zeta_{so} = f/K$$

  where f is the 1st-mode translational frequency in Hz and K is a factor that ranges from 60 to 250.

- The lower bound for estimates of structural damping ratio obtained from Loma Prieta response records for which $\delta_{rms}/H \times 1000 > 0.1$ can be described by the relation

$$\zeta_s = 0.025(\delta_{rms}/H \times 1000)^{0.11}$$

  in which δ_{rms}/H is the rms displacement at roof level divided by the building height.

- For buildings in which soil-structure interaction is a significant factor, the overall damping for strong-motion response may be 3 to 4 times the indicated lower bound.

13. ACKNOWLEDGMENTS

The authors wish to acknowledge the support provided by CDMG and USGS in obtaining the ambient vibration data used in this study and in making available strong-motion records collected during the Loma Prieta earthquake of October 17, 1989. Dr. M. Huang and Mr. Robert Land of CDMG and Mr. Marion Salsman of USGS provided valuable assistance in carrying out the ambient vibration studies. Dr. Robert Darragh and Dr. Anthony Shakal, both of CDMG, provided additional information on the selected buildings and the strong-motion records. Dr. Erdal Şafak of USGS provided many useful suggestions concerning the analysis and interpretation of the field data. Mr. R. J. Mele of NIST and Mr. Ray Eis and Mr. Emmett Dingel of USGS provided drafting services.

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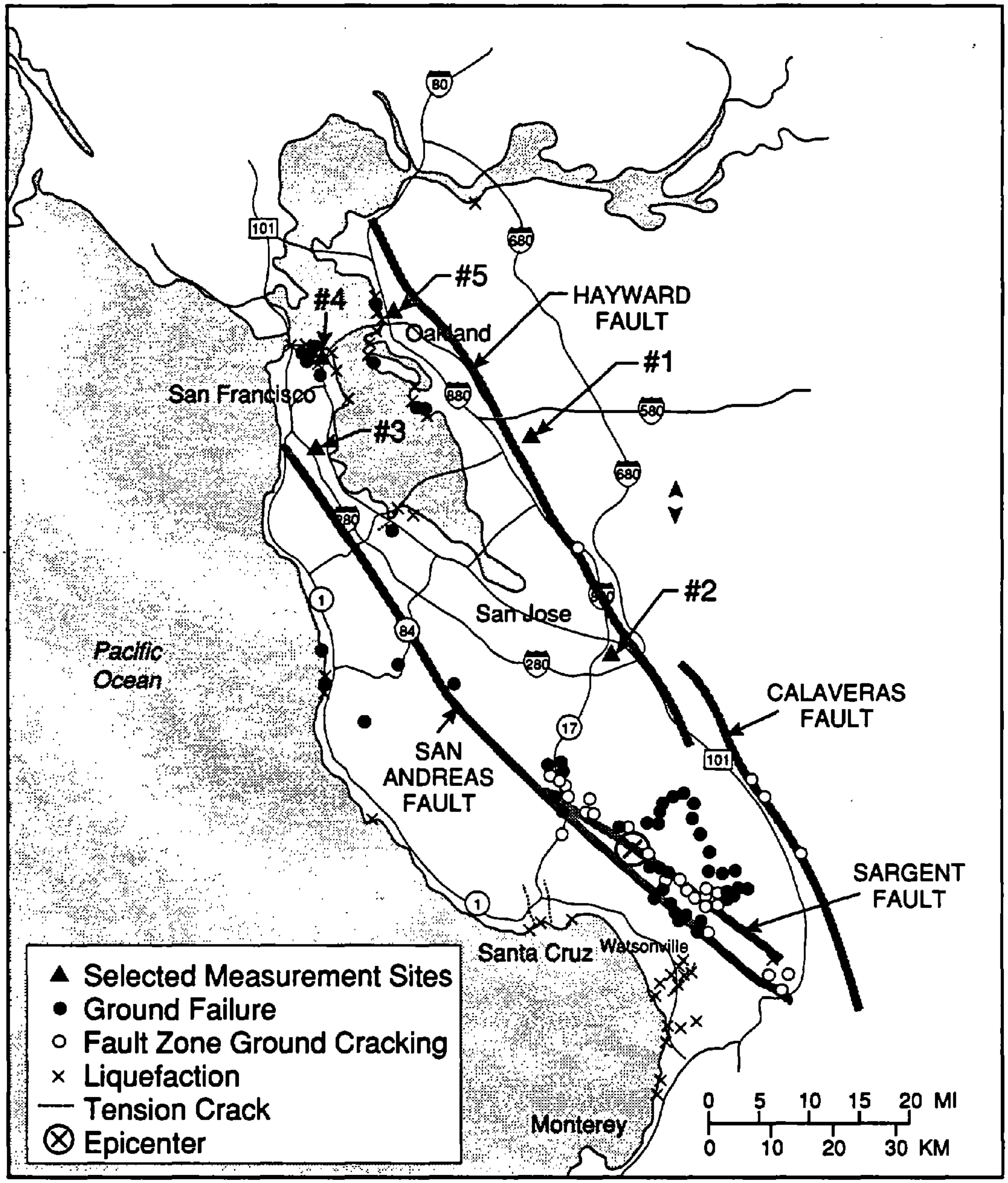

Figure 1. Locations of selected measurement sites and major faults.

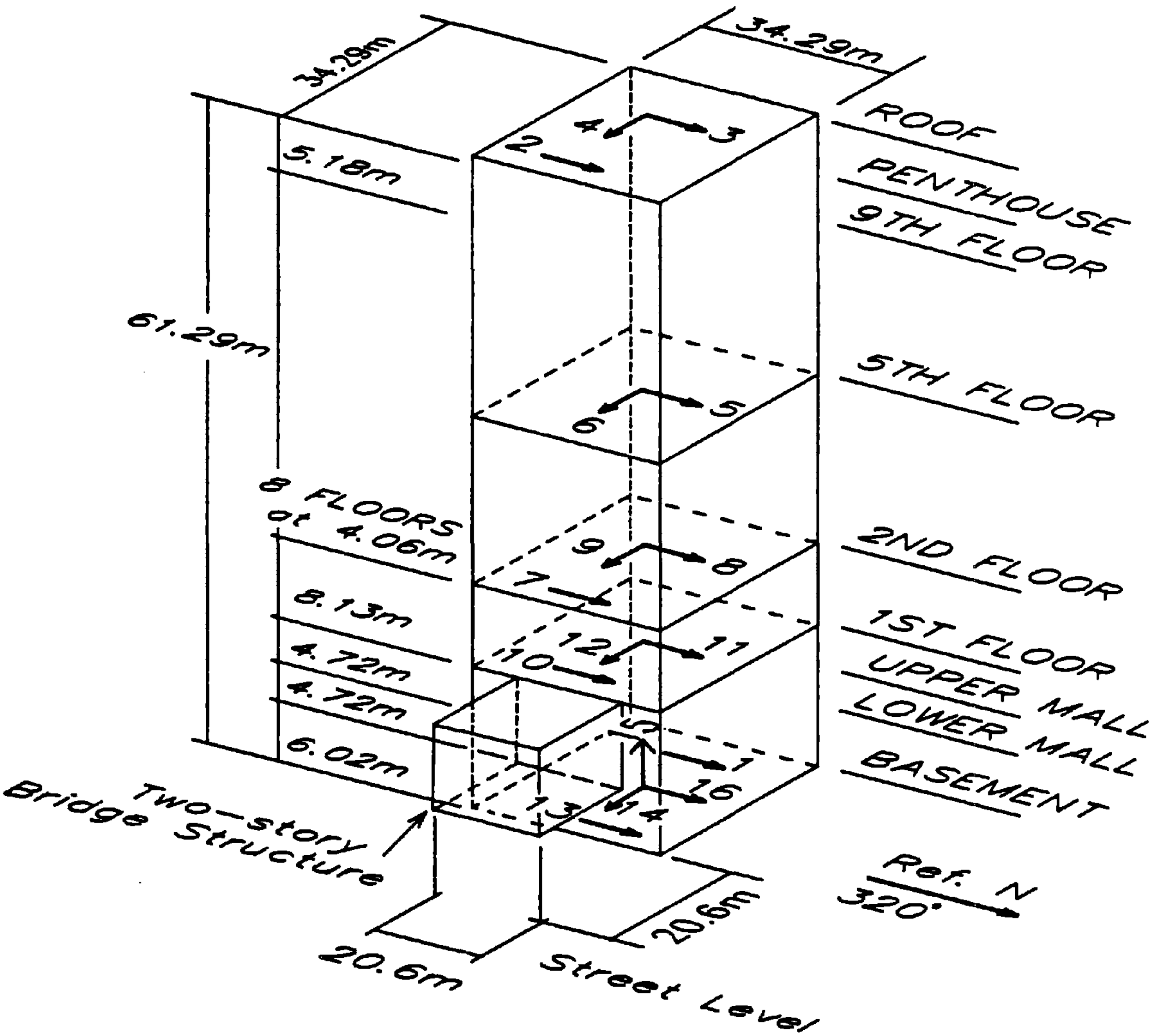

Figure 2. Building details and instrumentation scheme, Administration Building, California State University (CSUH), Hayward.

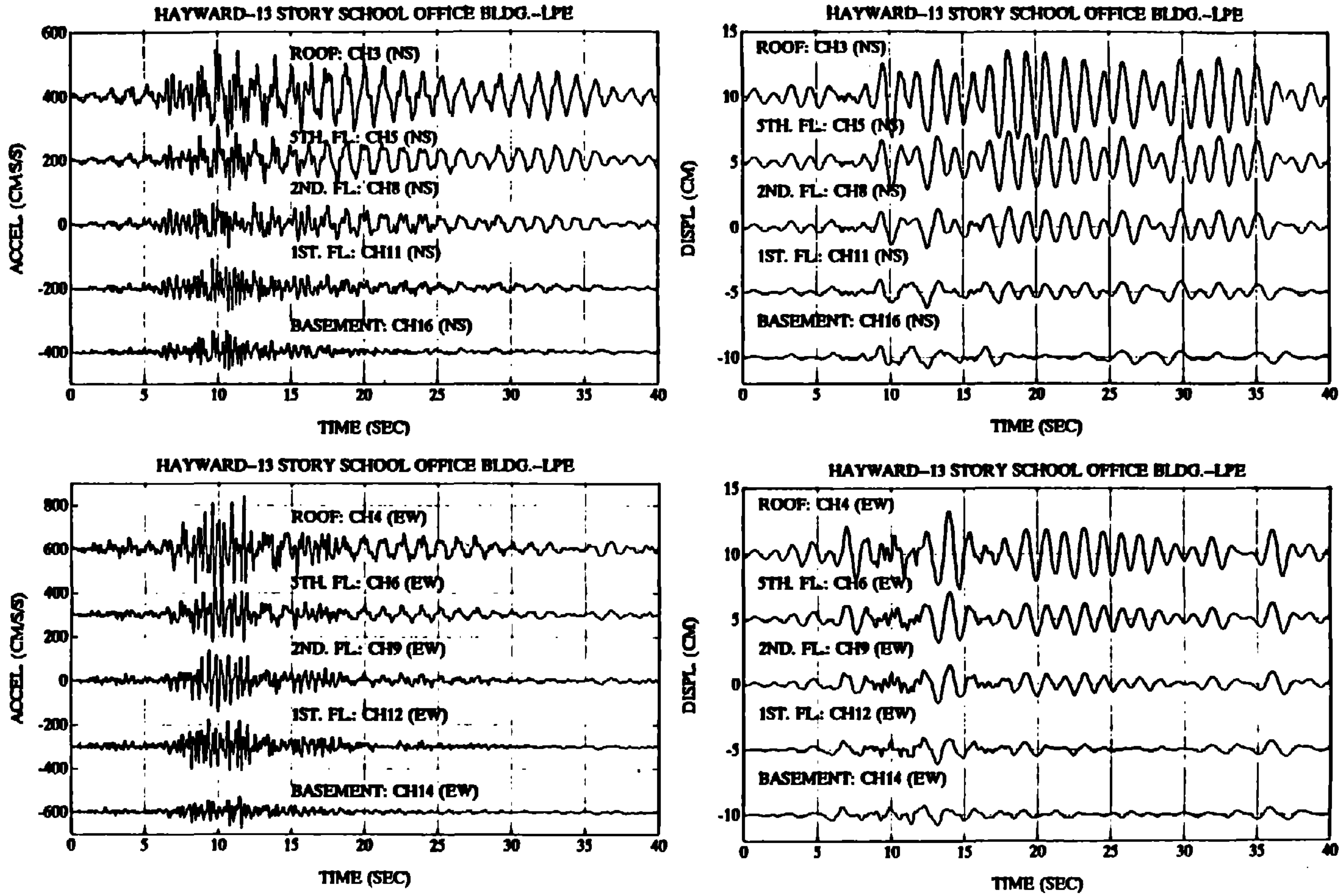

Figure 3. Strong-motion acceleration and integrated displacement time histories, CSUH Administration Building, Hayward.

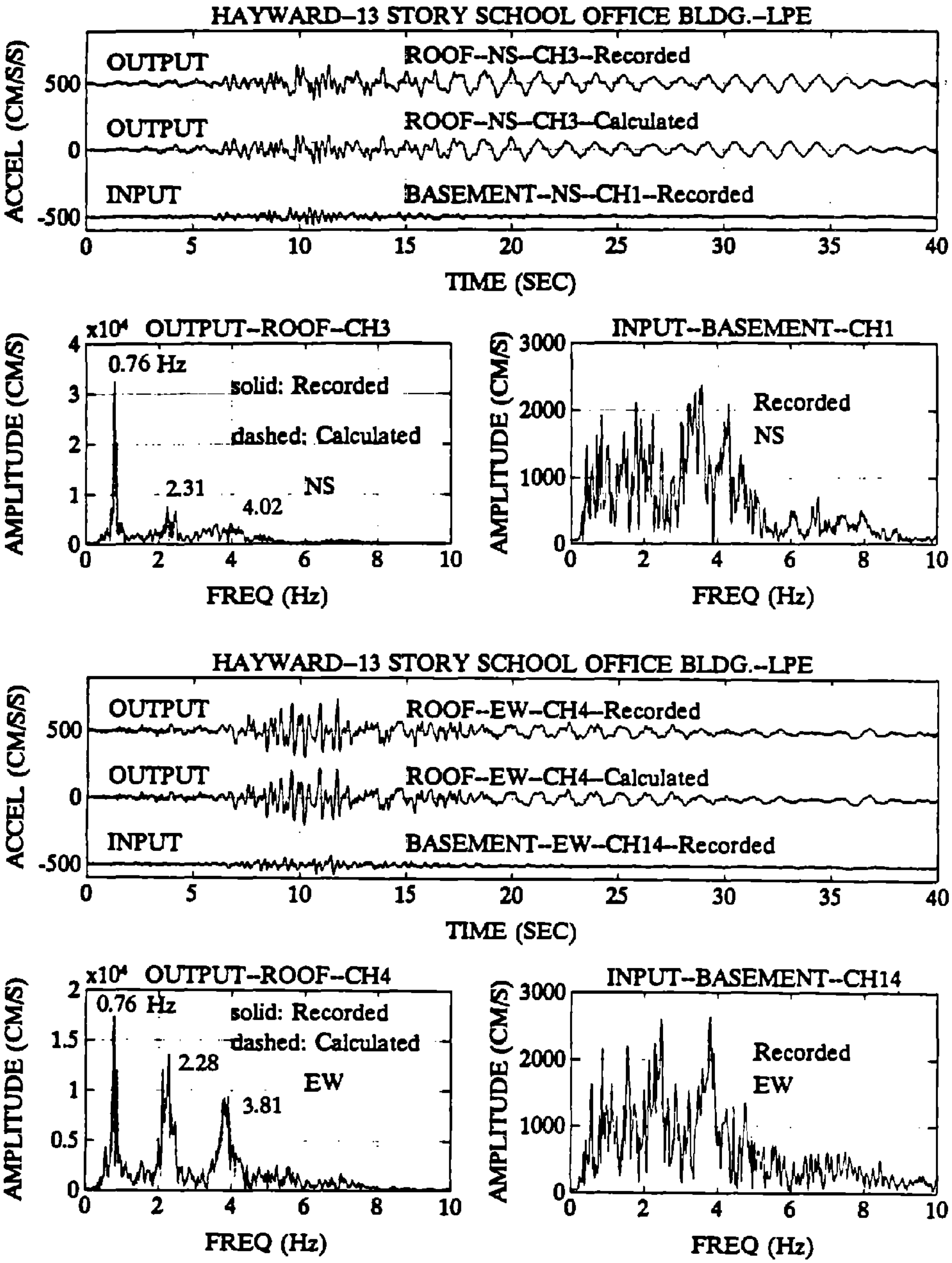

Figure 4. Recorded and calculated (by system identification) roof-level accelerations and Fourier amplitude spectra with basement motion as input, N-S and E-W components, CSUH Administration Building, Hayward.

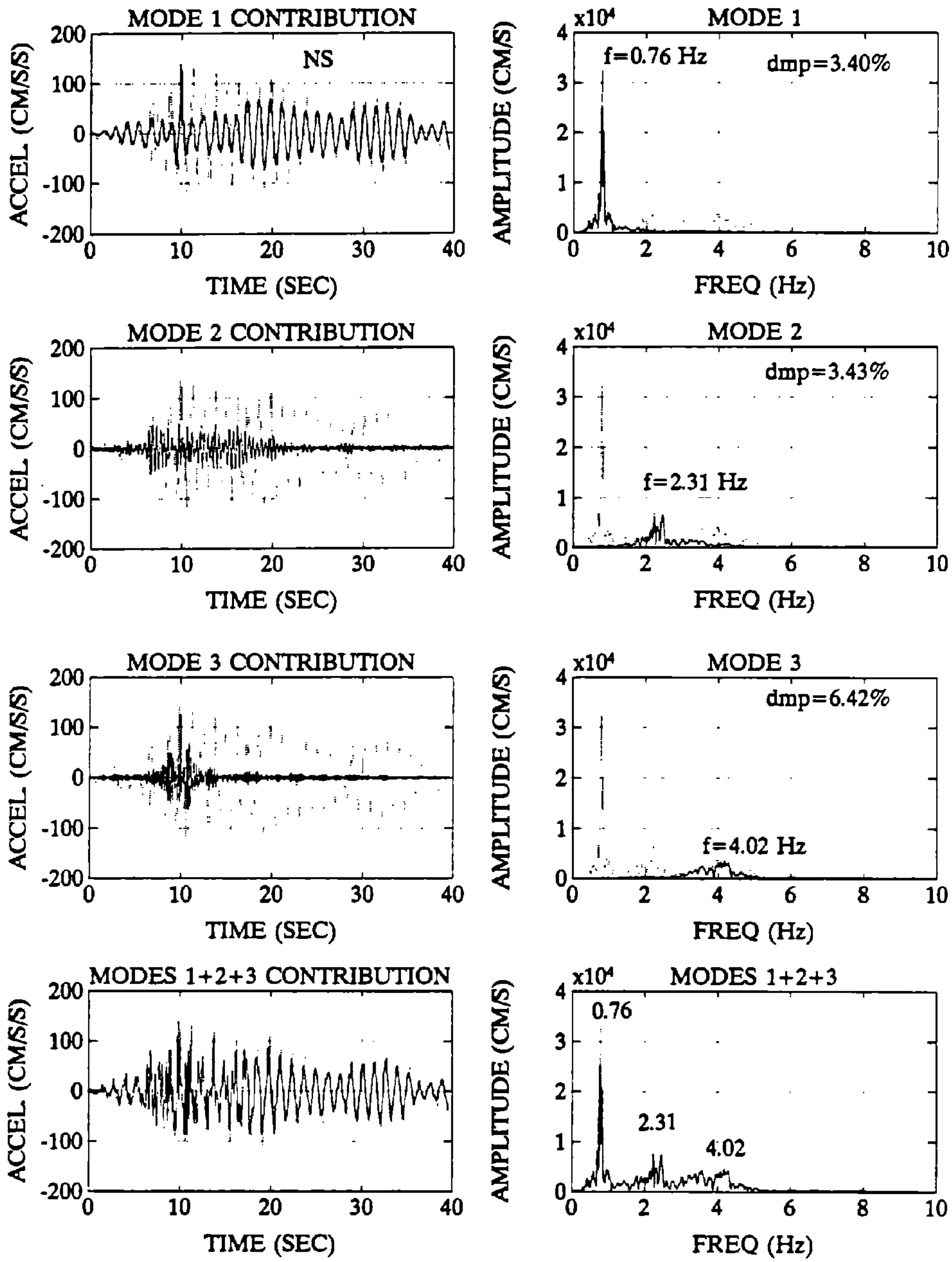

Figure 5. Calculated modal components (solid lines) of roof-level accelerations and comparison with recorded accelerations (dotted lines), CSUH Administration Building, Hayward.

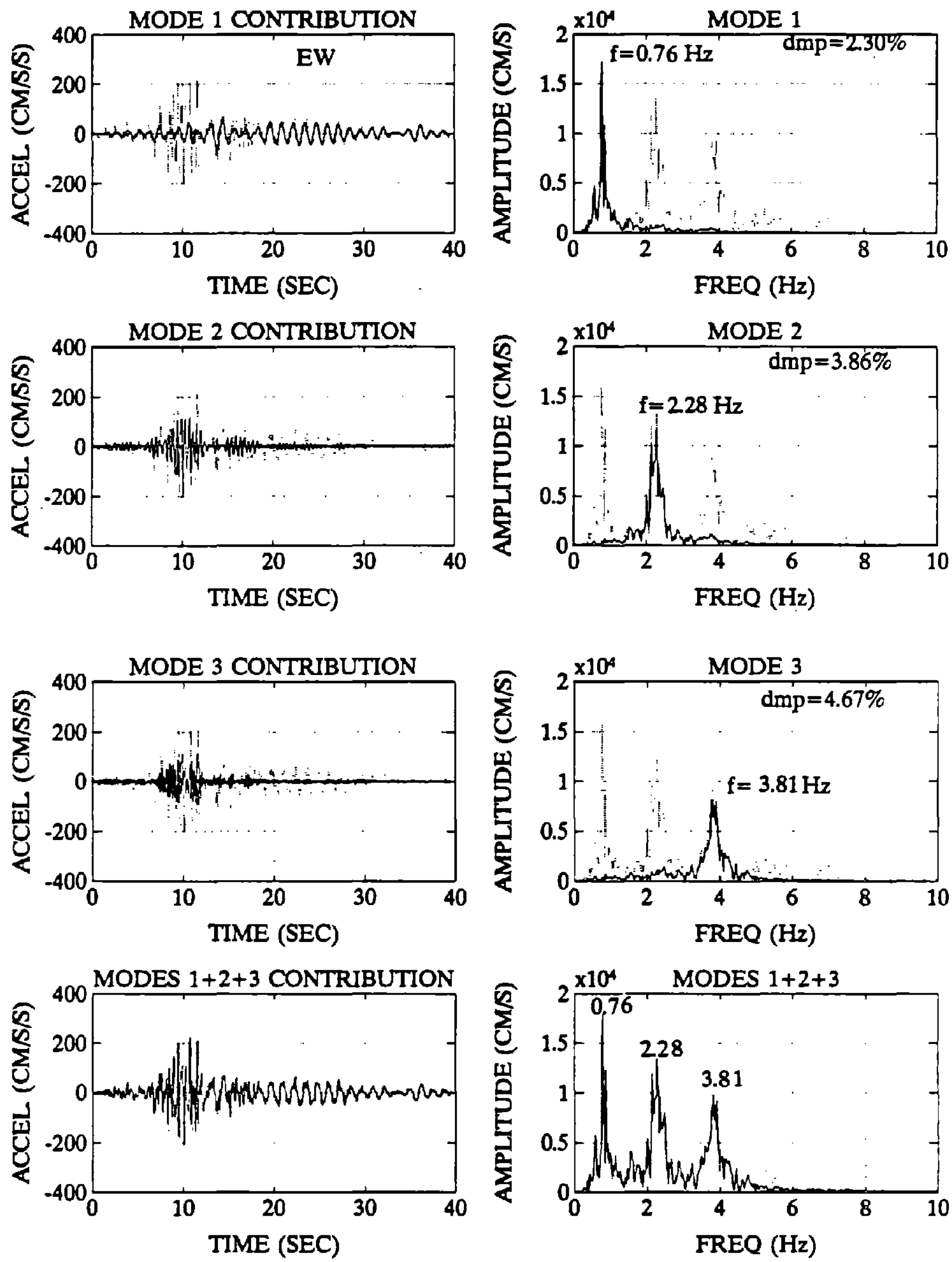

Figure 5. (Continued). Calculated modal components (solid lines) of roof-level accelerations and comparison with recorded accelerations (dotted lines), CSUH Administration Building, Hayward.

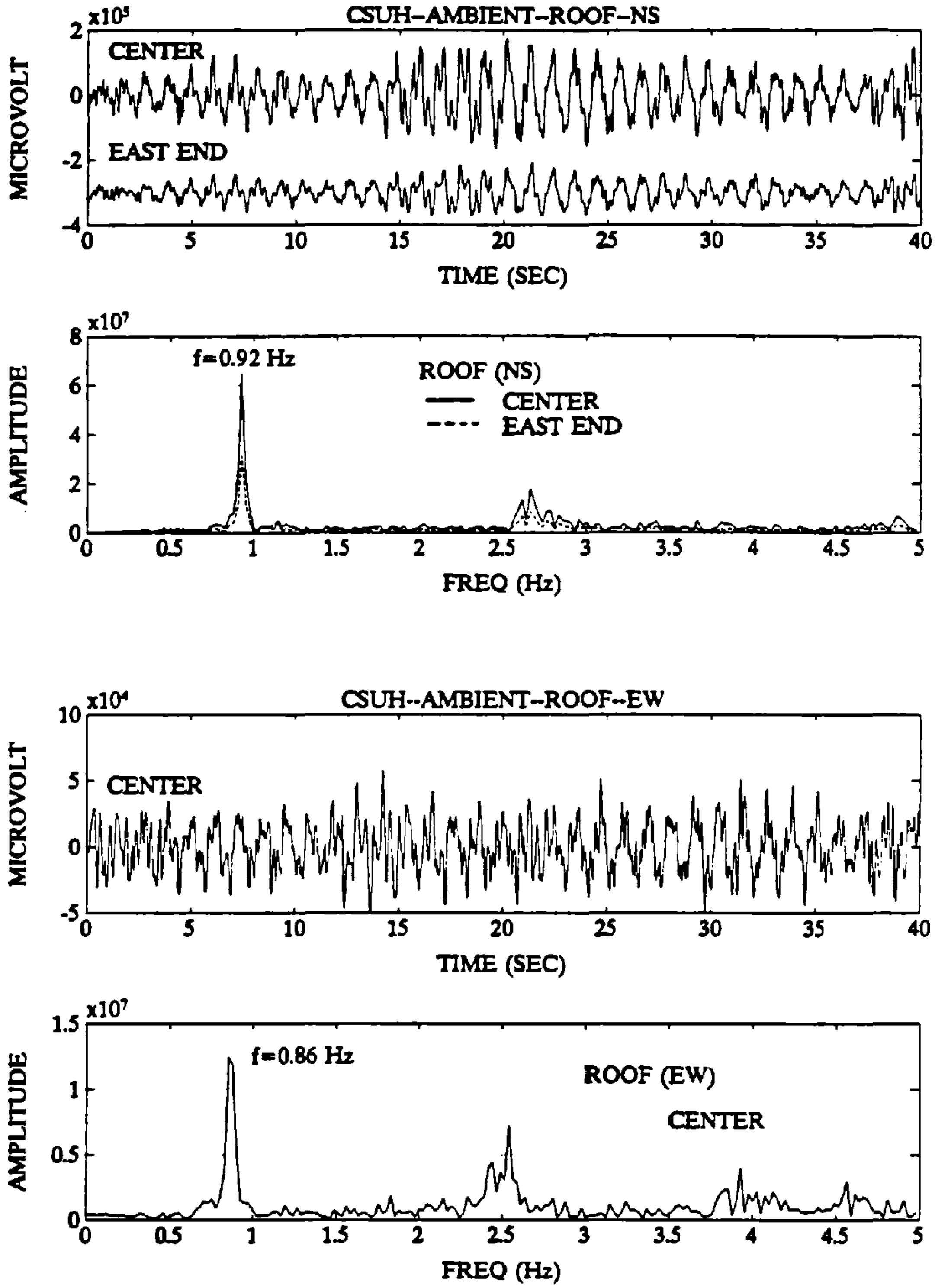

Figure 6. Ambient vibration response at roof level and Fourier amplitude spectra, N-S and E-W components, CSUH Administration Building, Hayward.

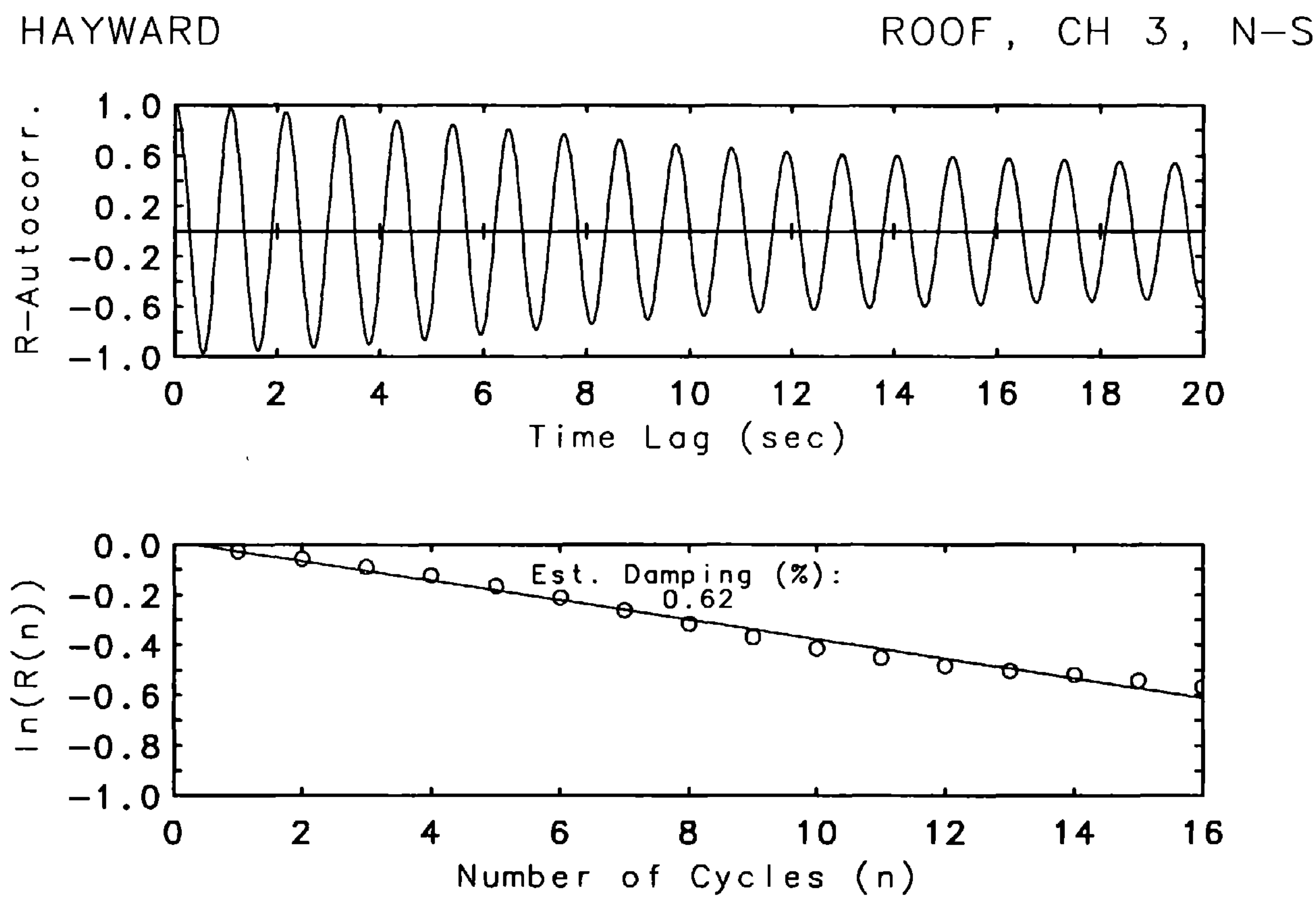

Figure 7. Auto-correlation curve and least-squares fit to amplitude decay, roof level, N-S component, CSUH Administration Building, Hayward.

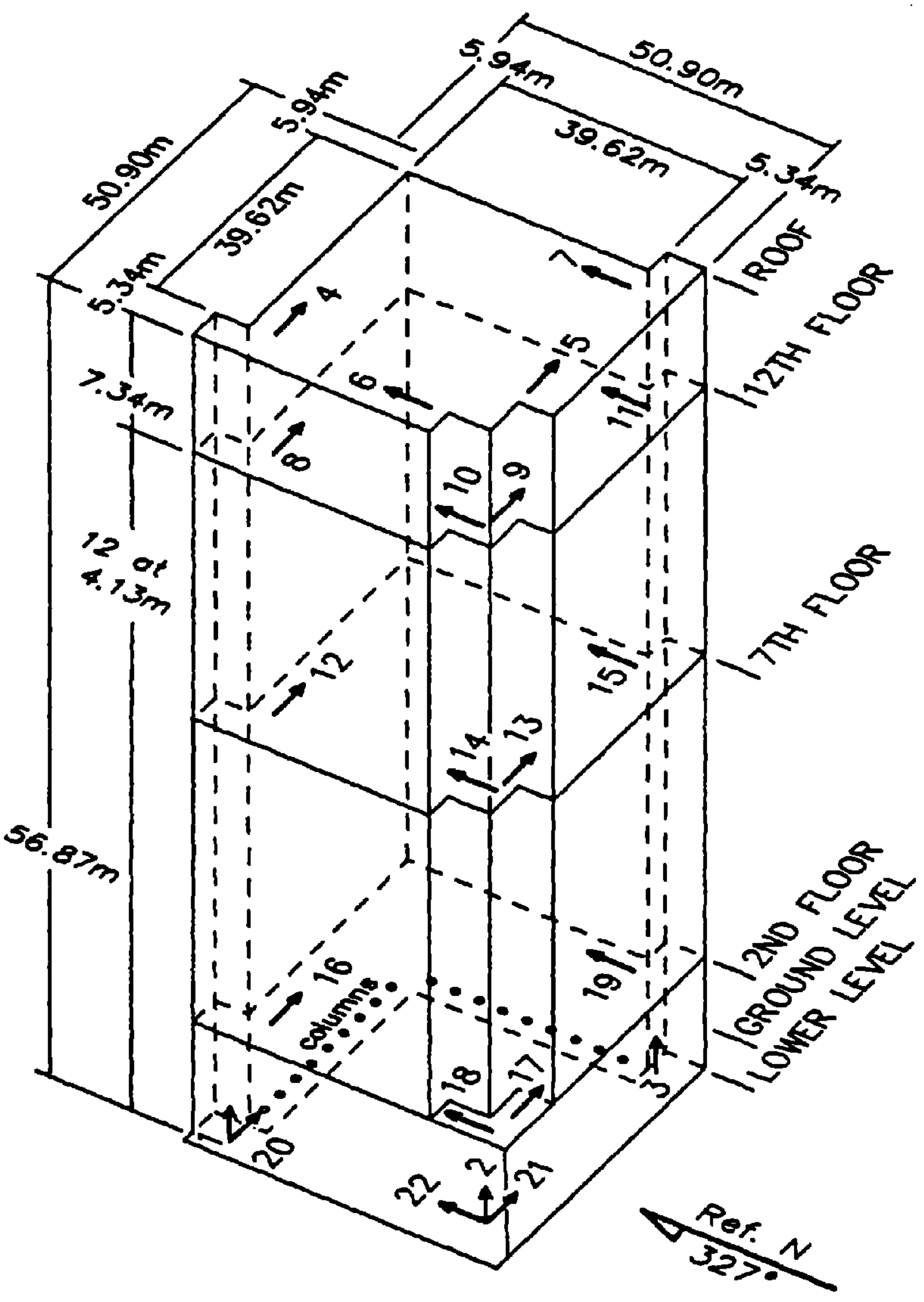

Figure 8. Building details and instrumentation scheme, Santa Clara County Office Building, San Jose.

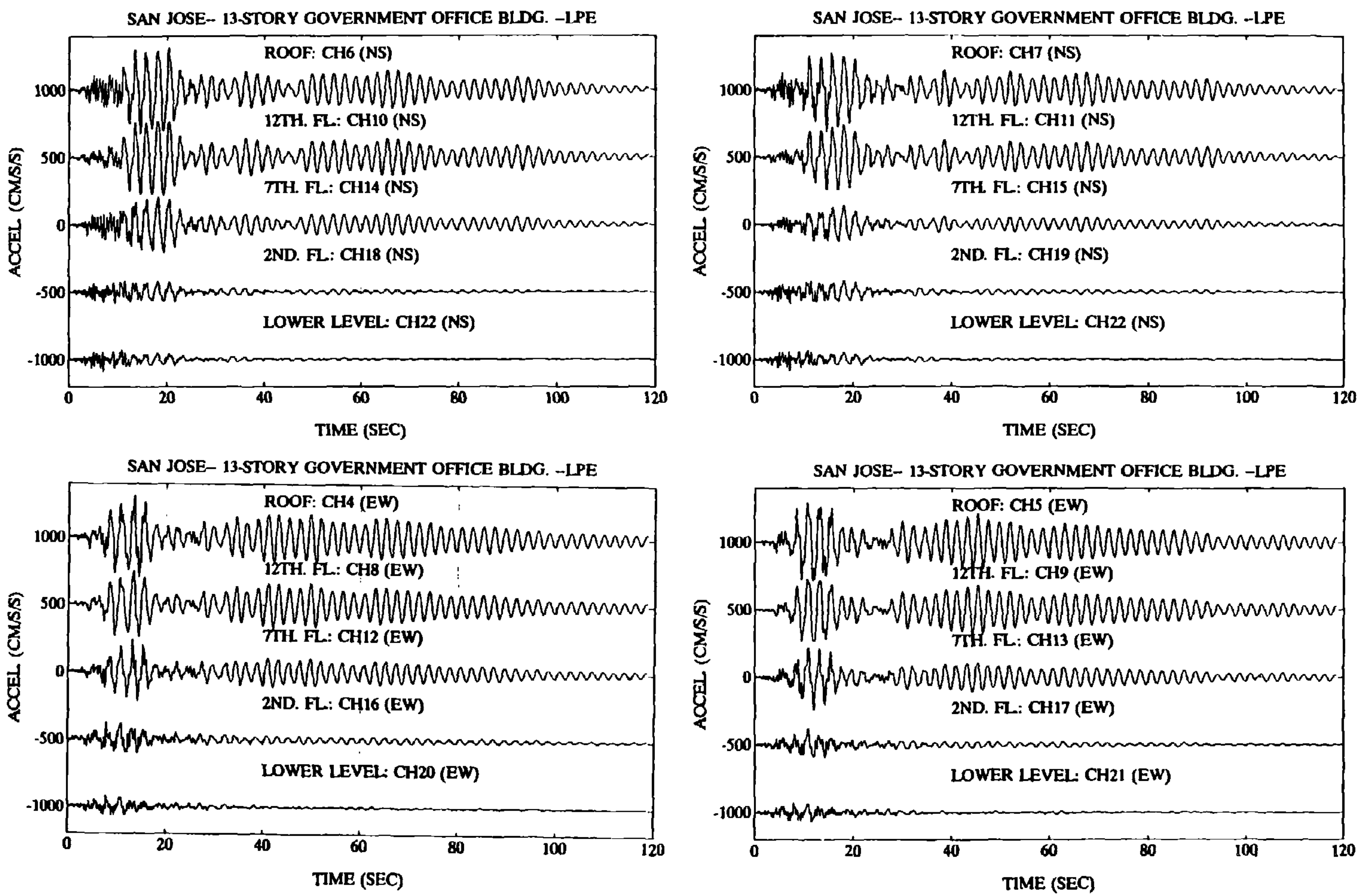

Figure 9. Strong-motion acceleration time histories, Santa Clara County Office Building, San Jose.

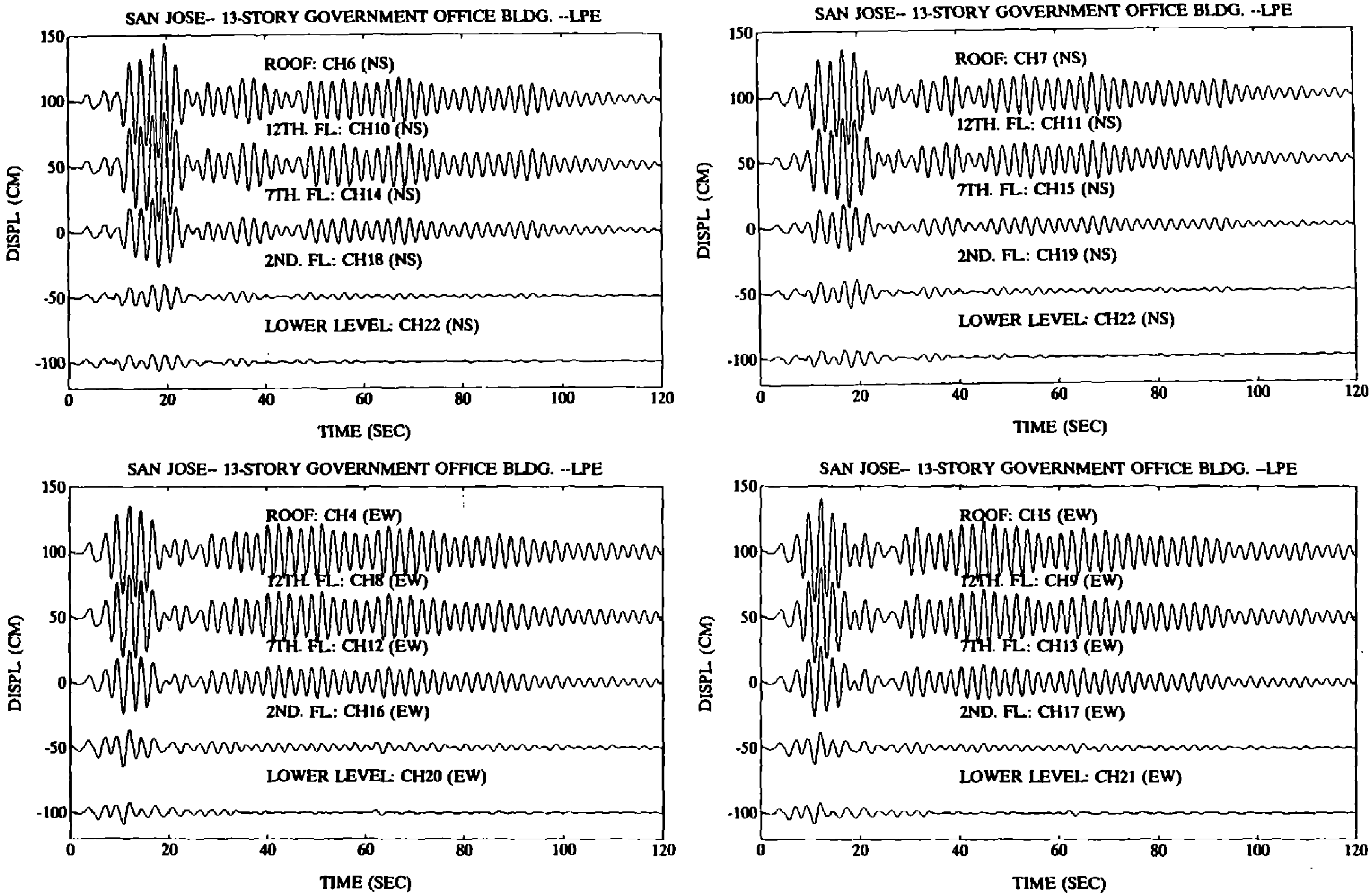

Figure 10. Integrated displacement time histories, Santa Clara County Office Building, San Jose.

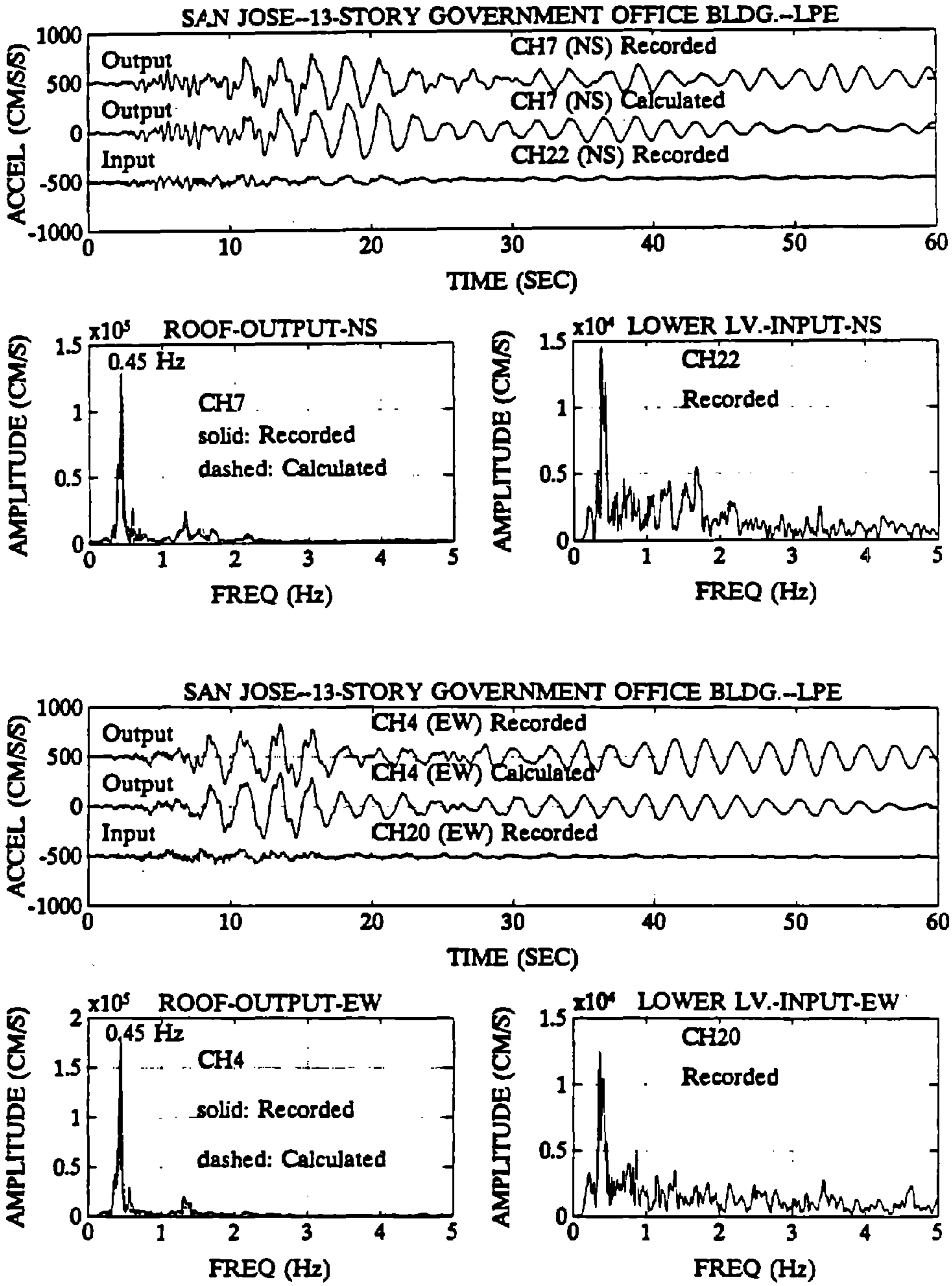

Figure 11. Recorded and calculated (by system identification) roof-level accelerations and Fourier amplitude spectra with basement motion as input, N-S and E-W components, Santa Clara County Office Building, San Jose.

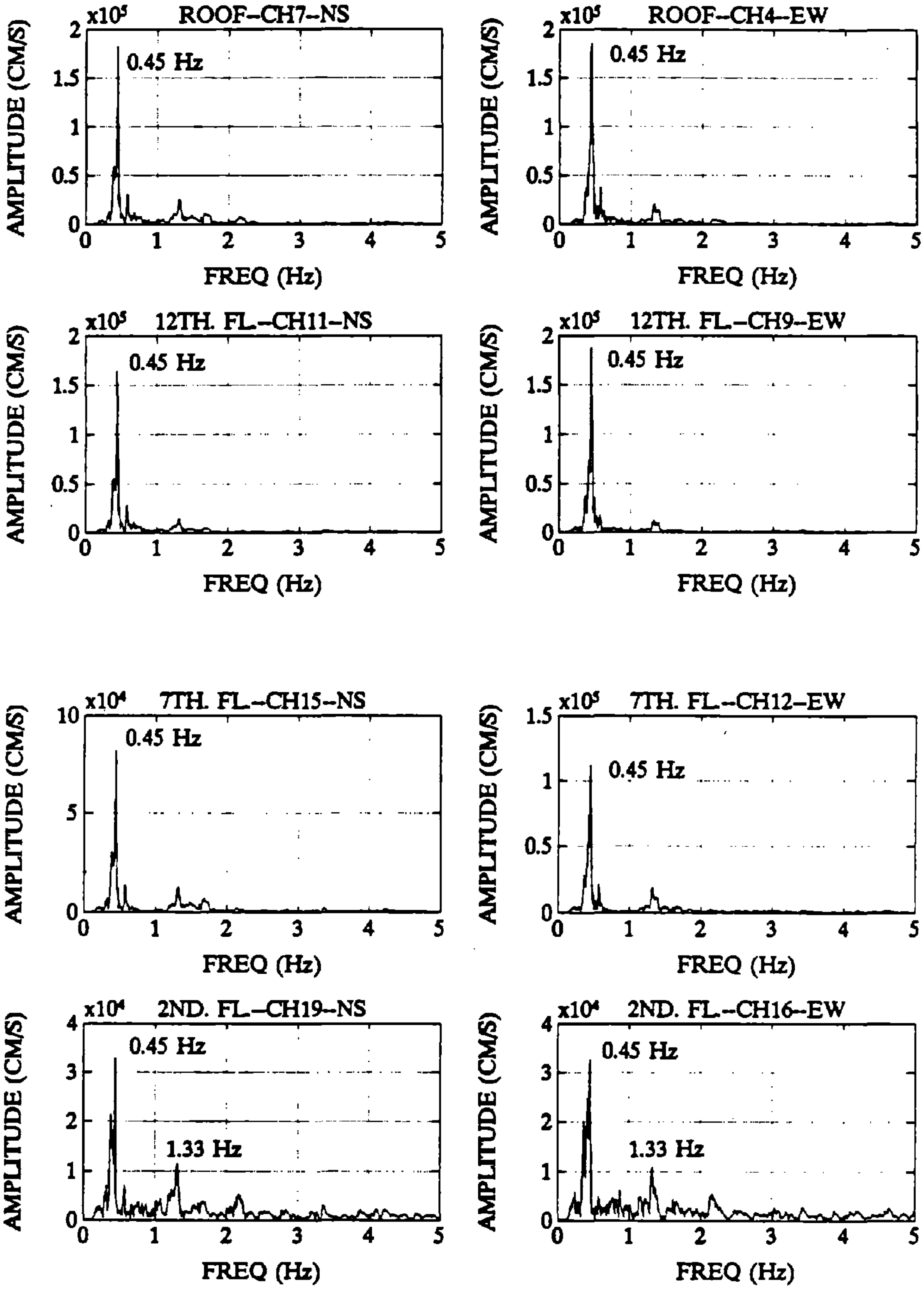

Figure 12. Fourier amplitude spectra for intermediate floors and lower level, Santa Clara County Office Building, San Jose.

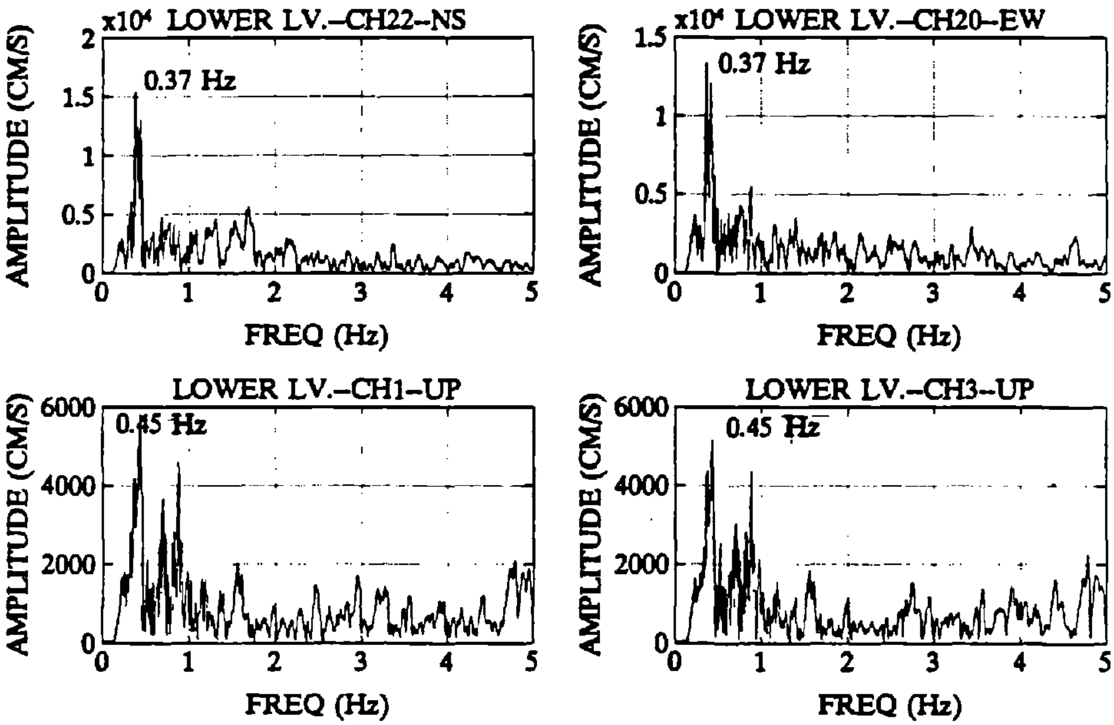

Figure 12. (Continued). Fourier amplitude spectra for intermediate floors and lower level, Santa Clara County Office Building, San Jose.

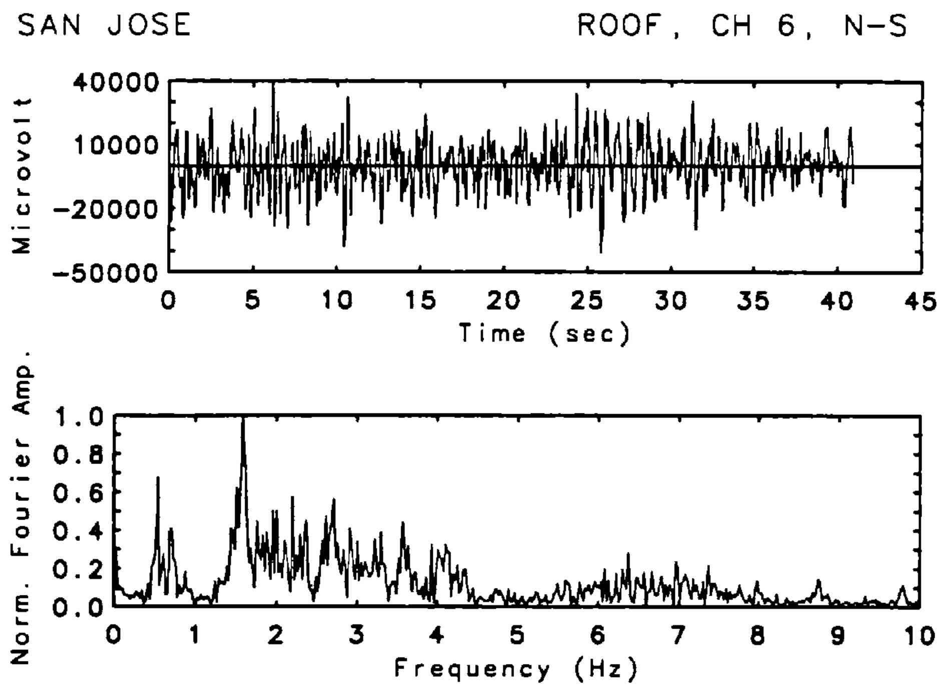

Figure 13. Ambient vibration response and Fourier amplitude spectrum, SW corner roof, N-S component, Santa Clara County Office Building, San Jose.

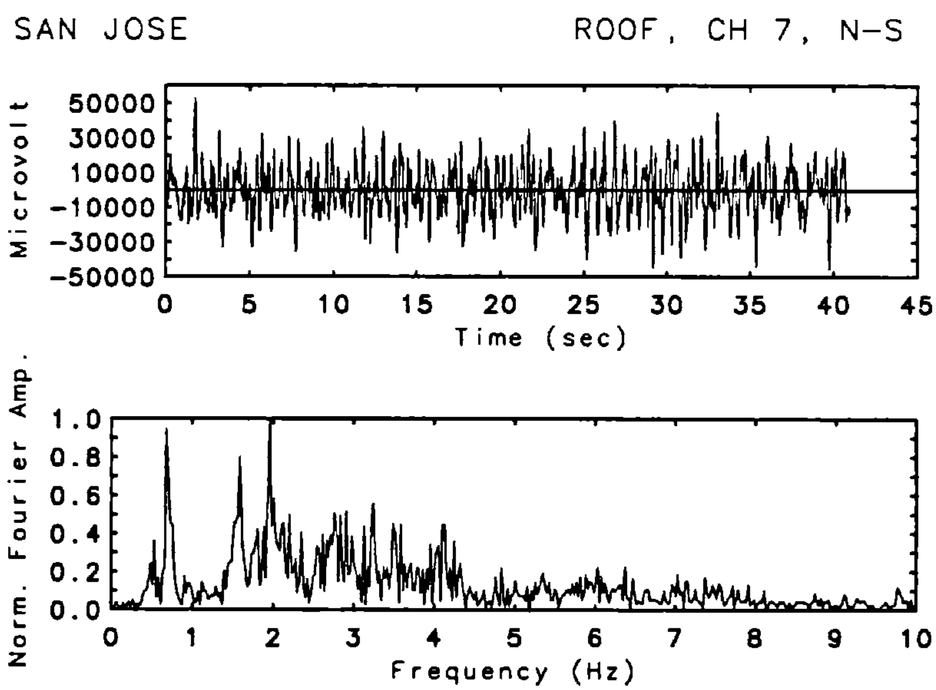

Figure 14. Ambient vibration response and Fourier amplitude spectrum, SE corner roof, N-S component, Santa Clara County Office Building, San Jose.

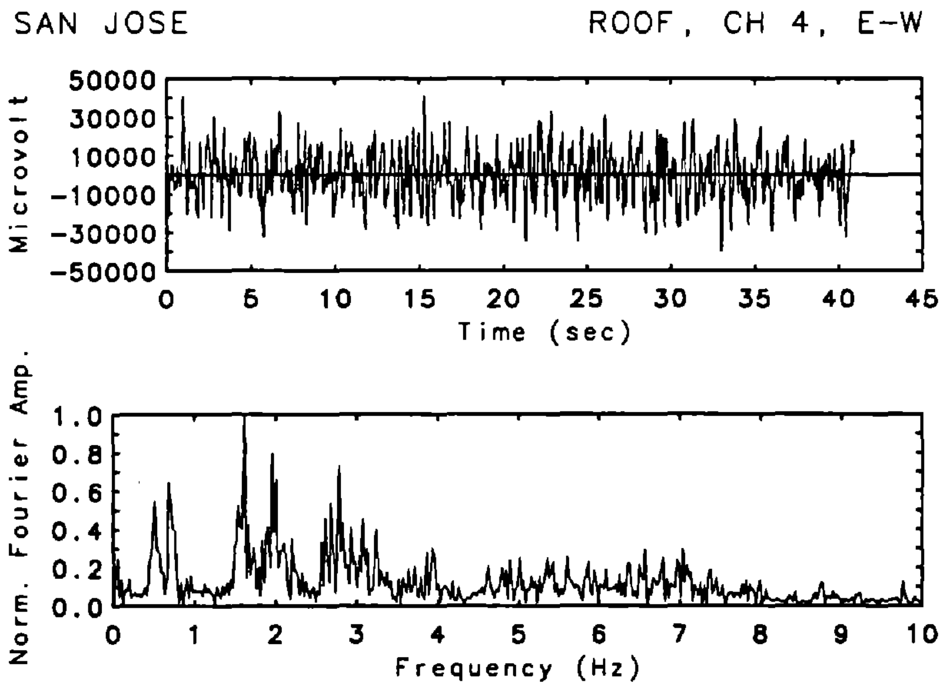

Figure 15. Ambient vibration response and Fourier amplitude spectrum, NW corner roof, E-W component, Santa Clara County Office Building, San Jose.

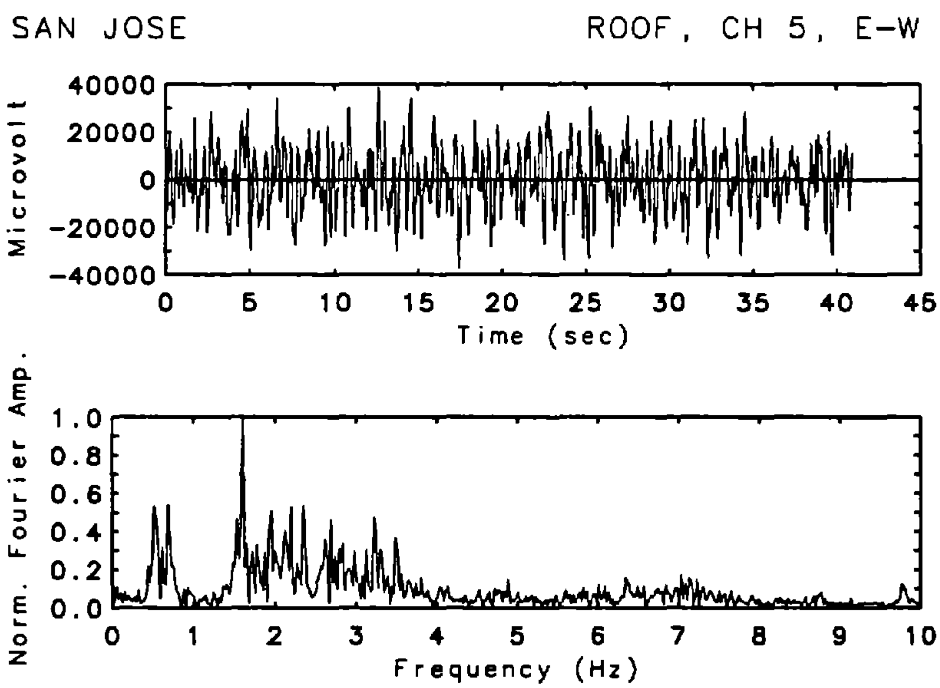

Figure 16. Ambient vibration response and Fourier amplitude spectrum, SW corner roof, E-W component, Santa Clara County Office Building, San Jose.

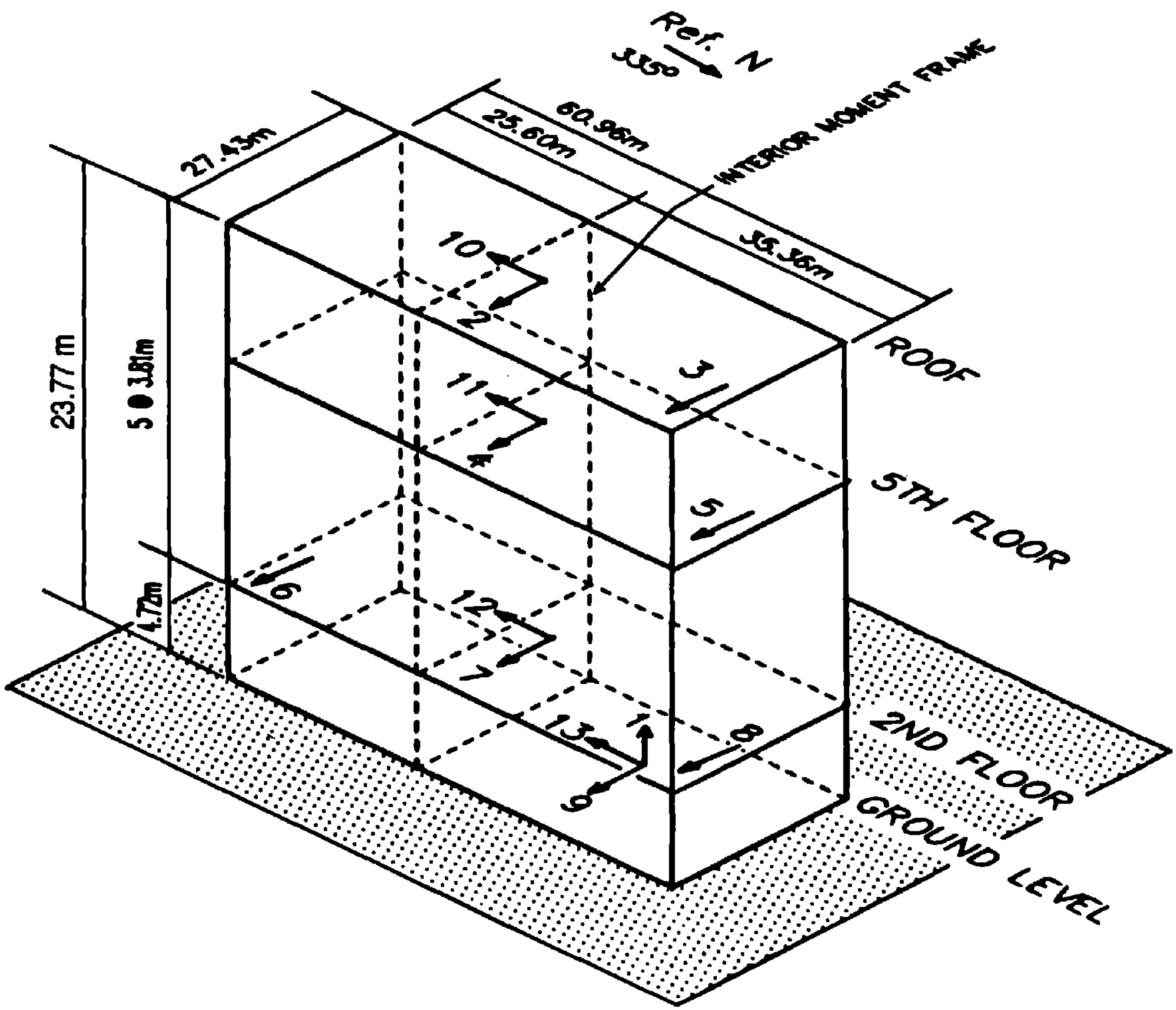

Figure 17. Building details and instrumentation scheme, Commercial Office Building, San Bruno.

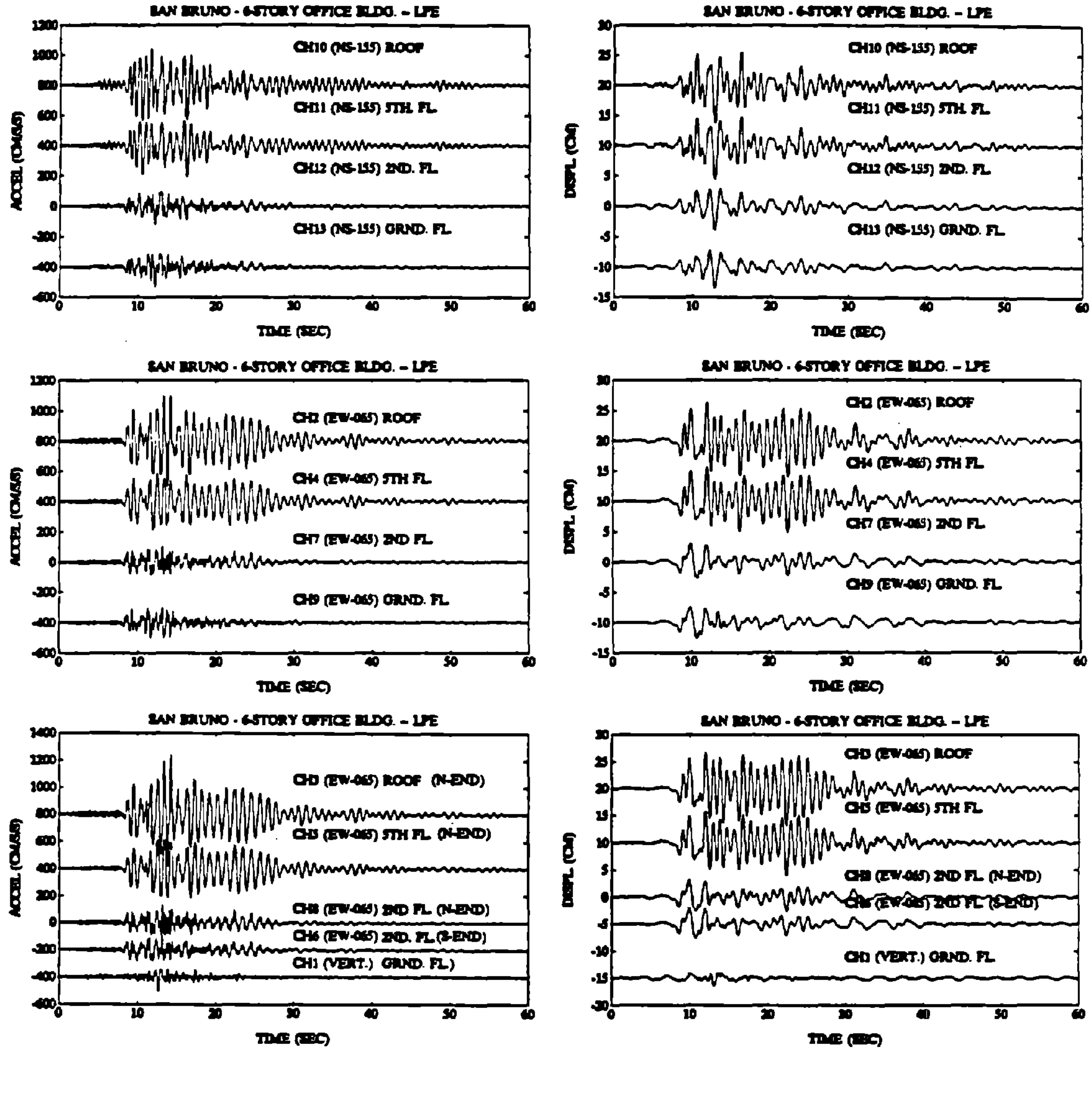

Figure 18. Strong-motion acceleration and integrated displacement time histories, Commercial Office Building, San Bruno.

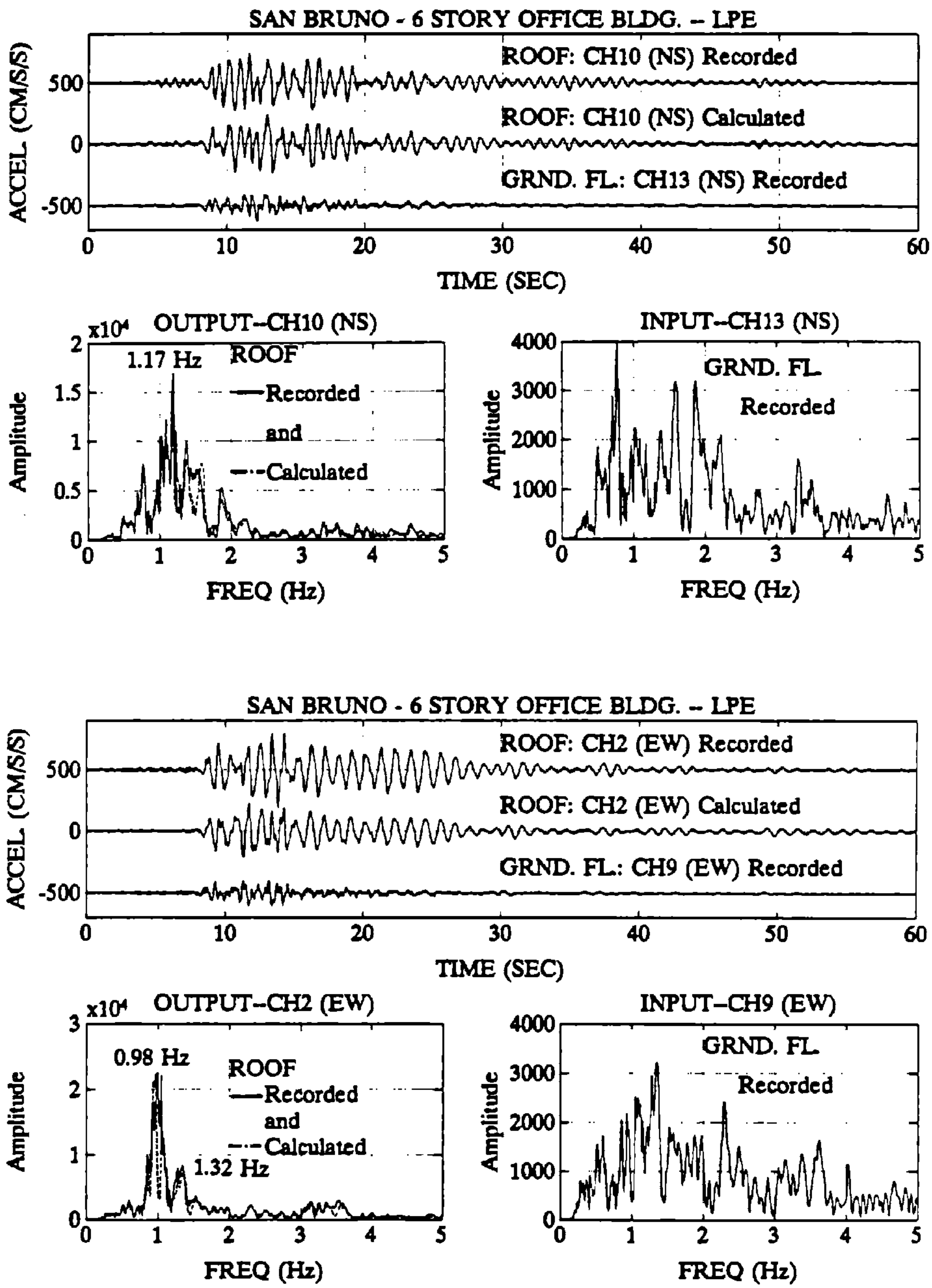

Figure 19. Recorded and calculated (by system identification) roof-level accelerations and Fourier amplitude spectra with ground-level motion as input, N-S and E-W components, Commercial Office Building, San Bruno.

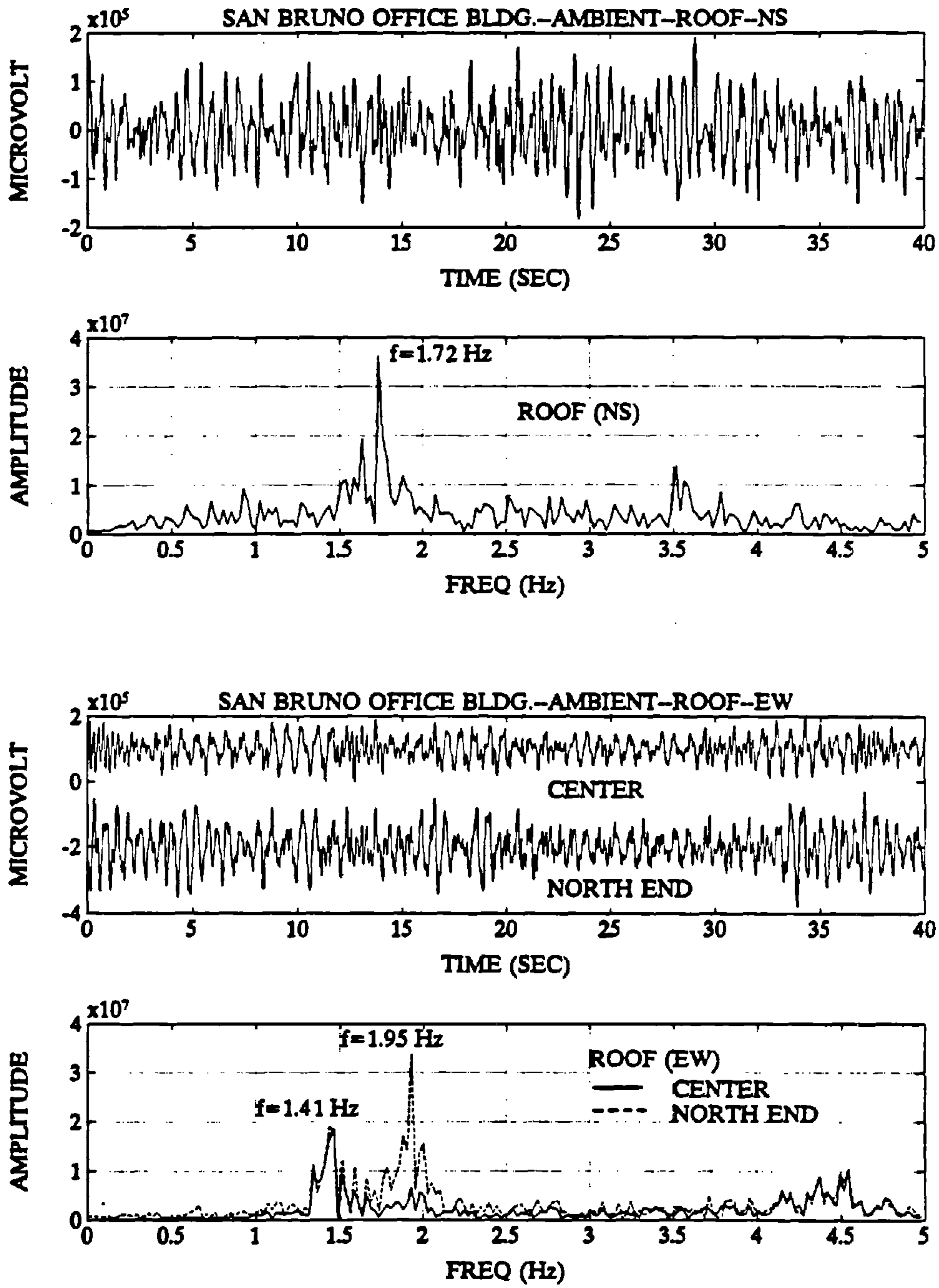

Figure 20. Ambient vibration response at roof level and Fourier amplitude spectra, N-S and E-W components, Commercial Office Building, San Bruno.

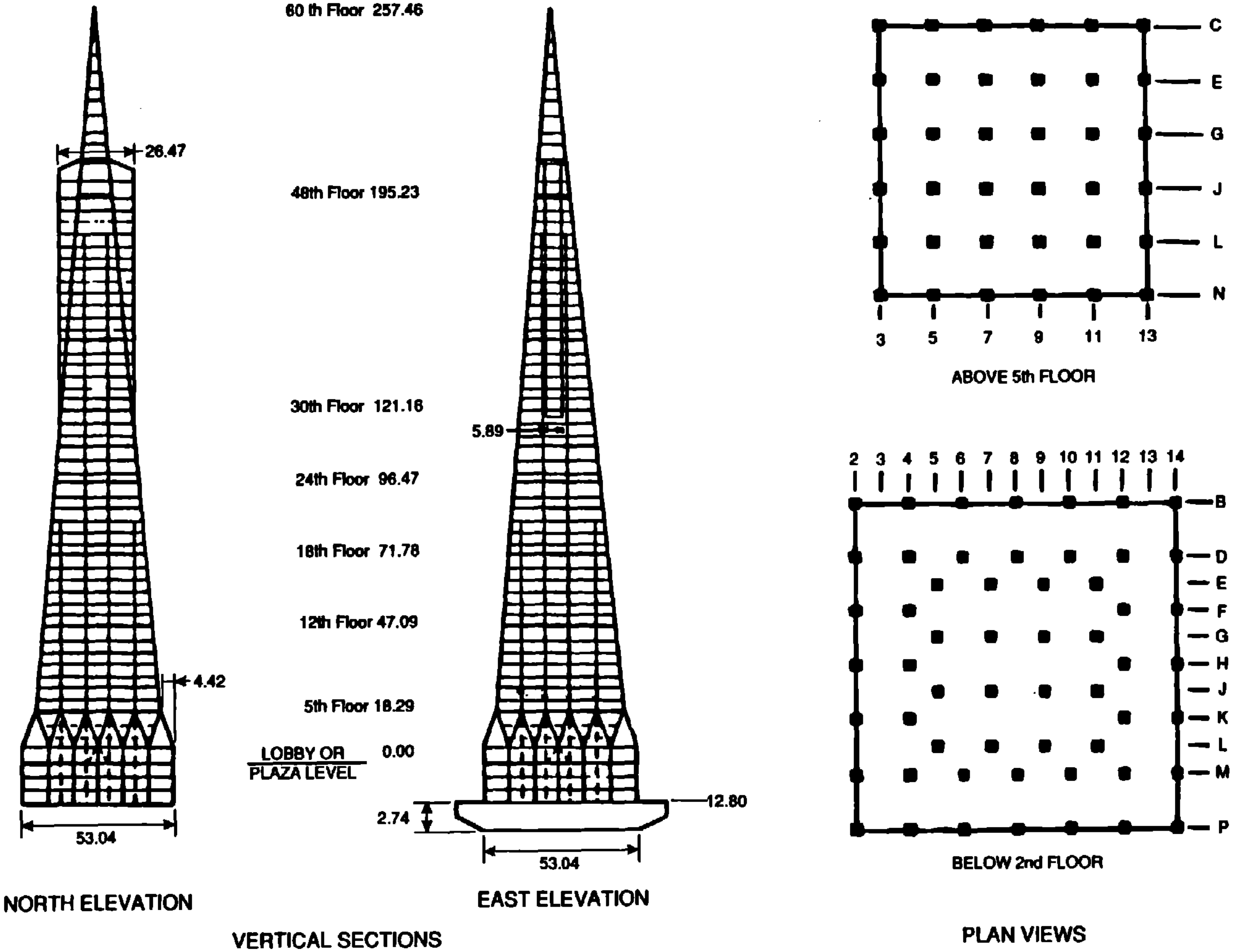

Figure 21. Building details and dimensions, Transamerica Building, San Francisco (adapted from Çelebi and Şafak, 1991). All dimensions in meters.

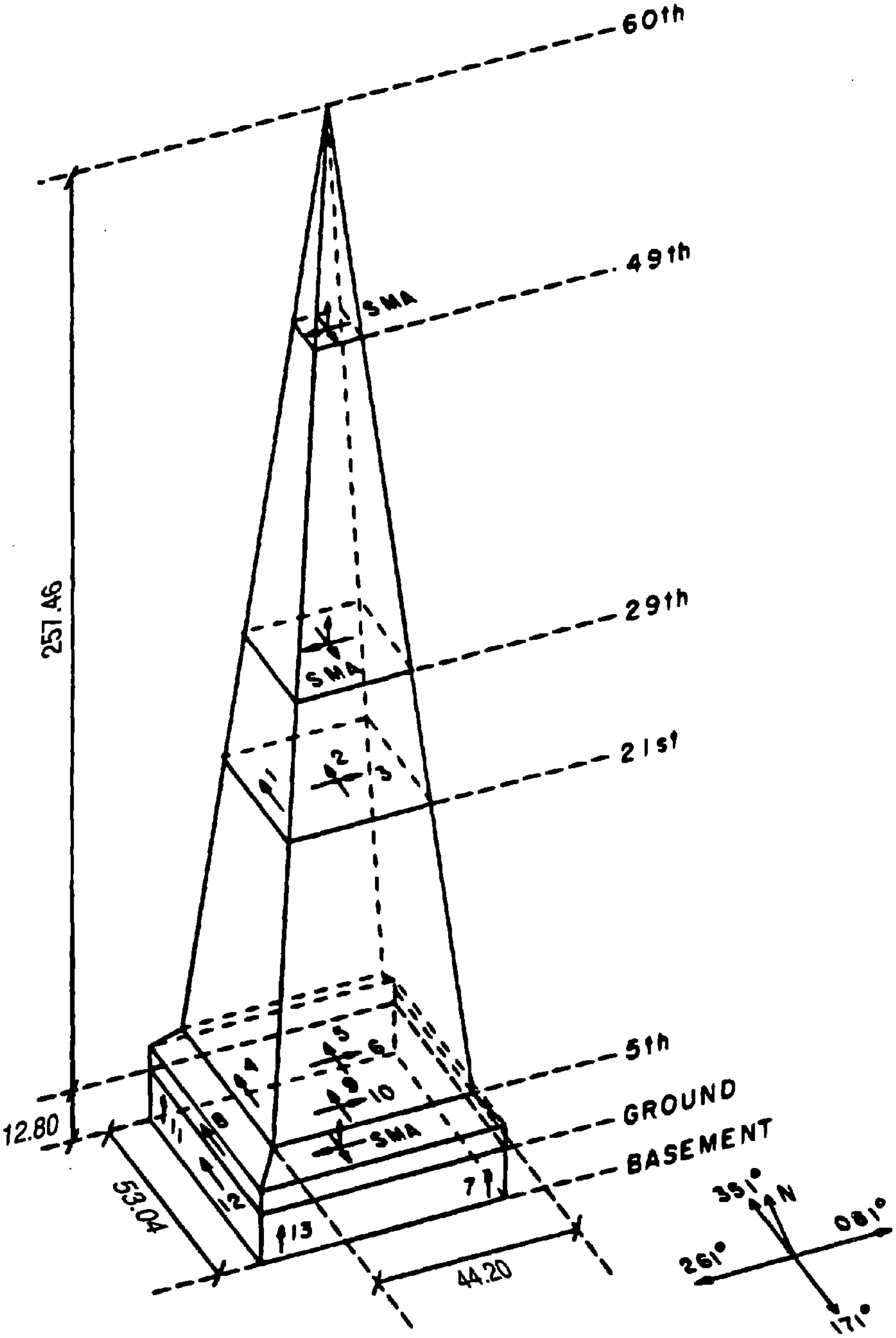

Figure 22. Instrumentation scheme, Transamerica Building, San Francisco (adapted from Çelebi and Şafak, 1991). All dimensions in meters.

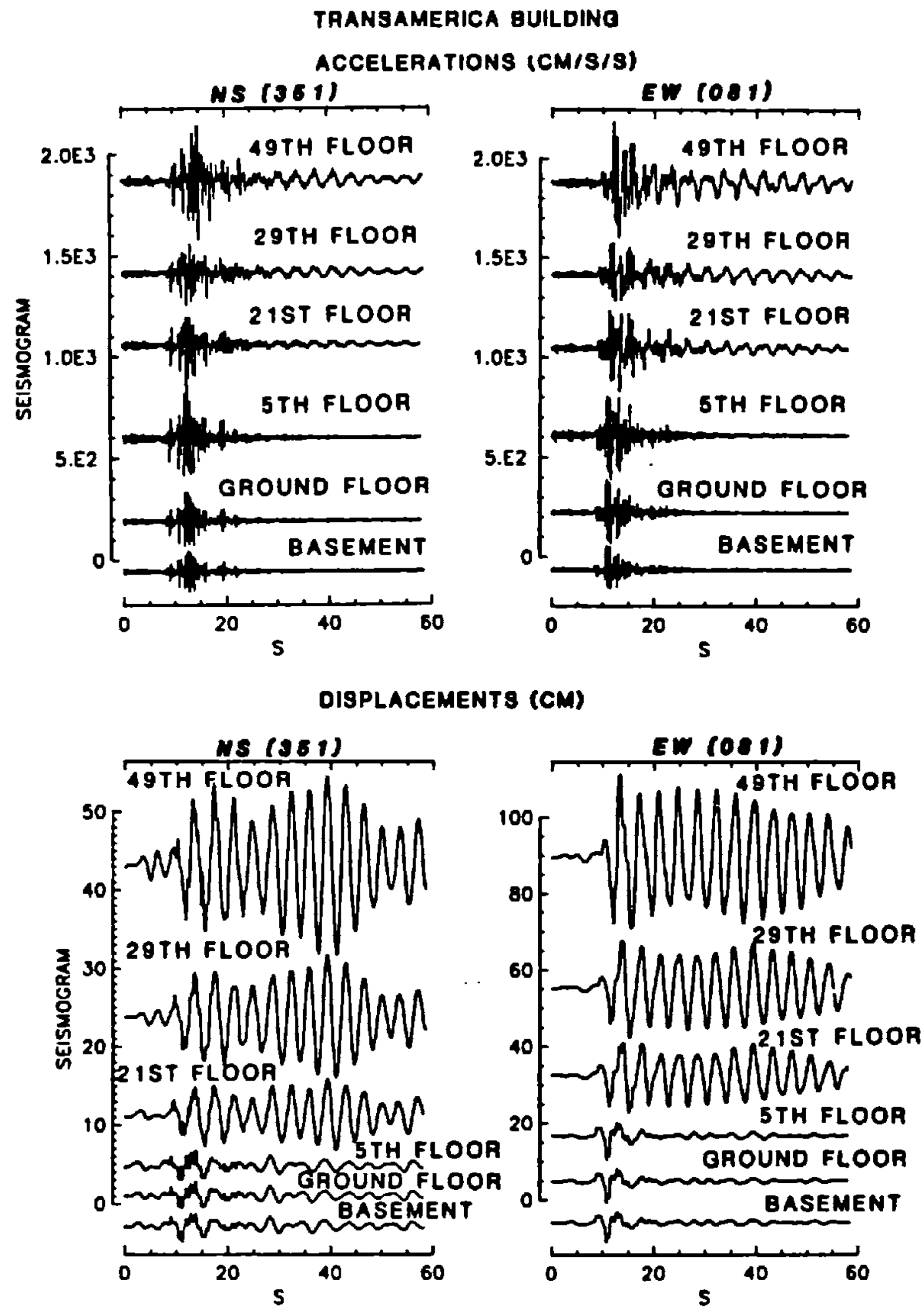

Figure 23. Strong-motion and integrated displacement time histories, Transamerica Building, San Francisco.

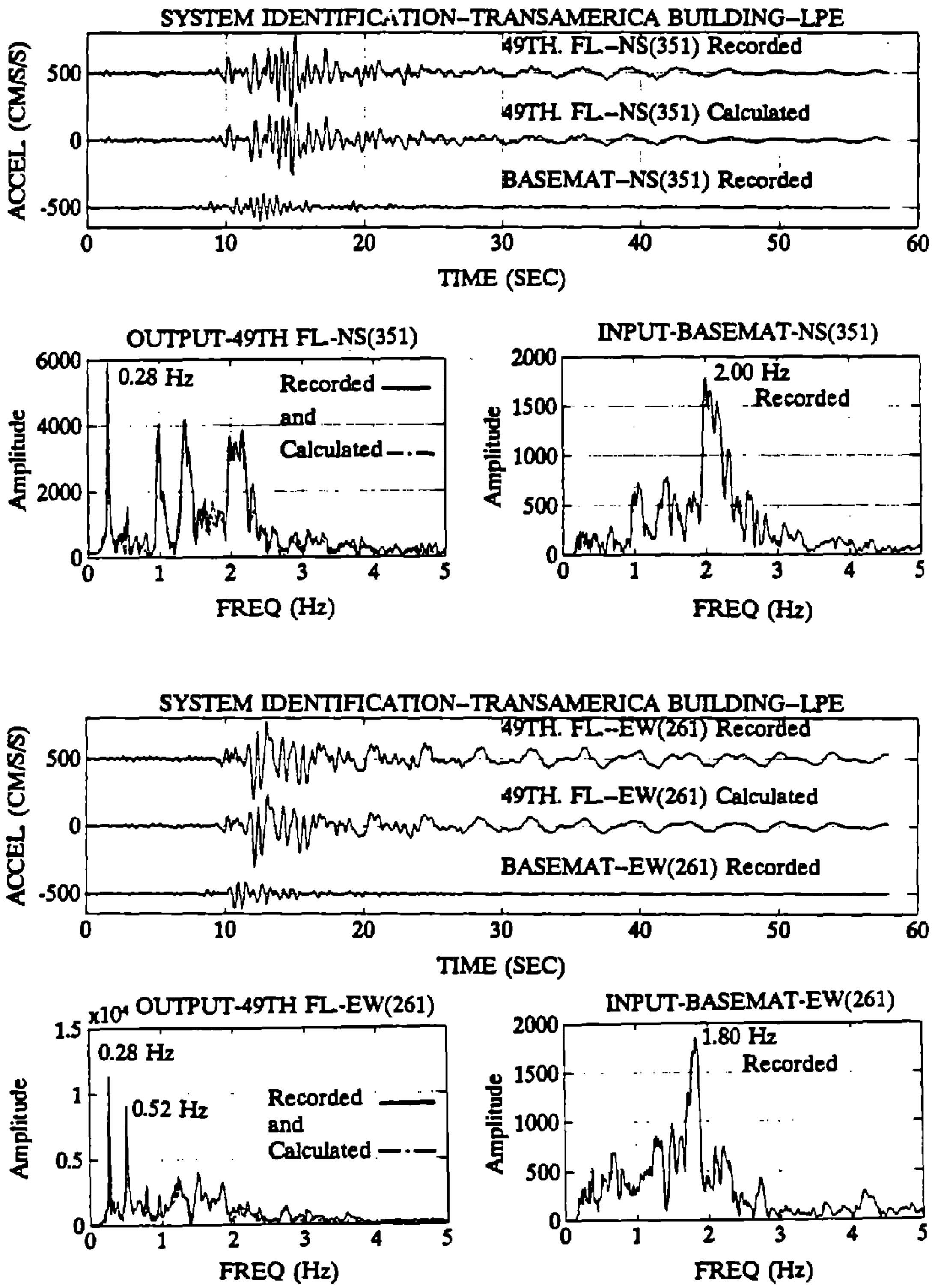

Figure 24. Recorded and calculated (by system identification) 49th floor accelerations and Fourier amplitude spectra with basemat accelerations as input, N-S and E-W components, Transamerica Building. San Francisco.

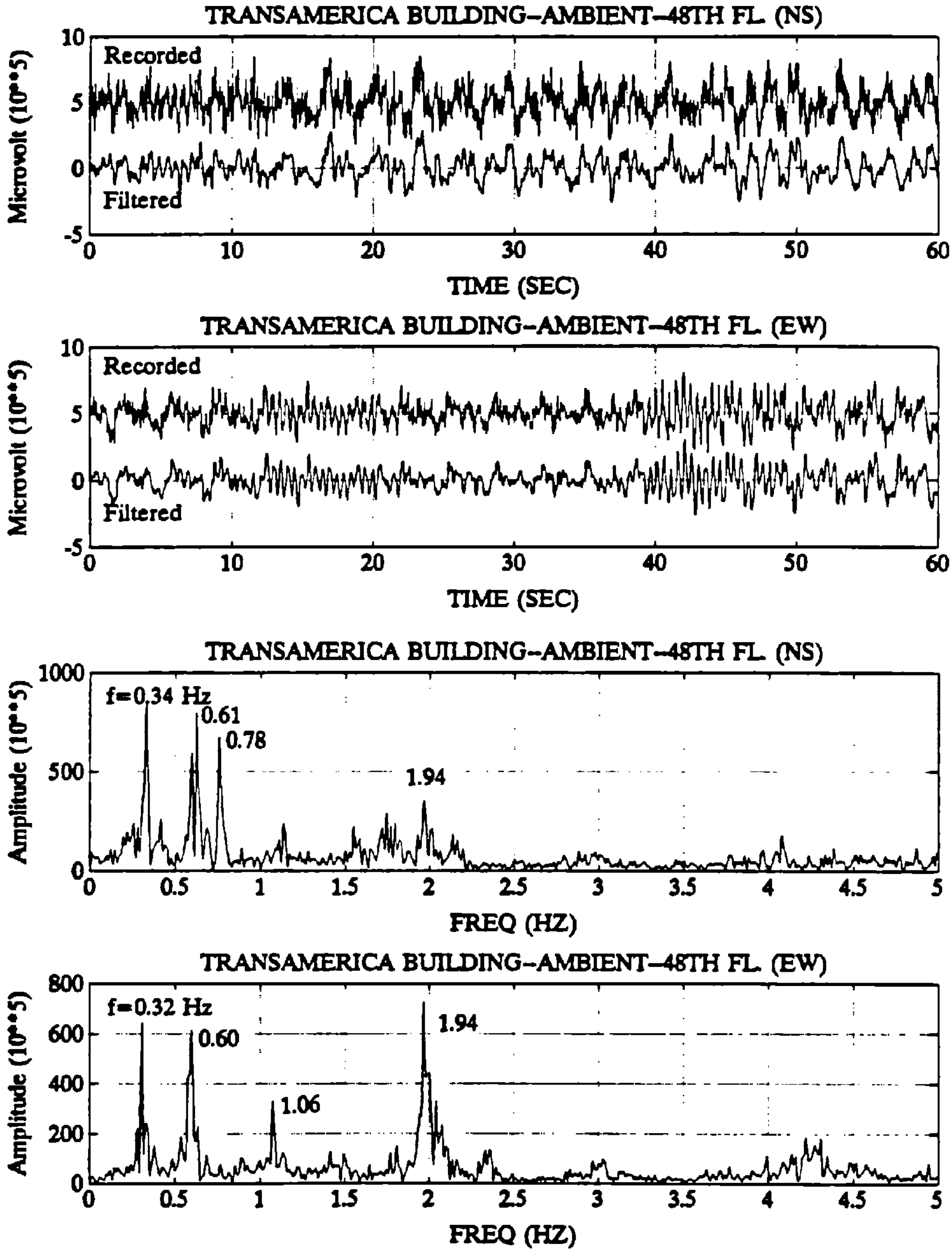

Figure 25. Ambient vibration response and Fourier amplitude spectra, 48th floor, N-S and E-W components, Transamerica Building, San Francisco.

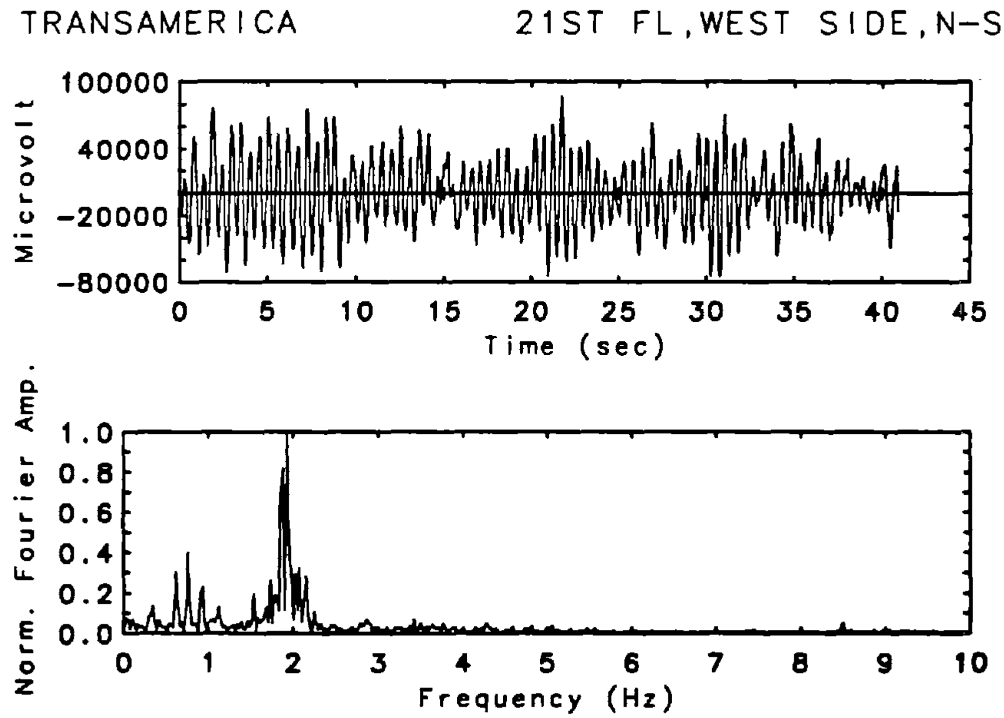

Figure 26. Ambient vibration response and Fourier amplitude spectrum, 21st floor, west side, N-S direction, Transamerica Building, San Francisco.

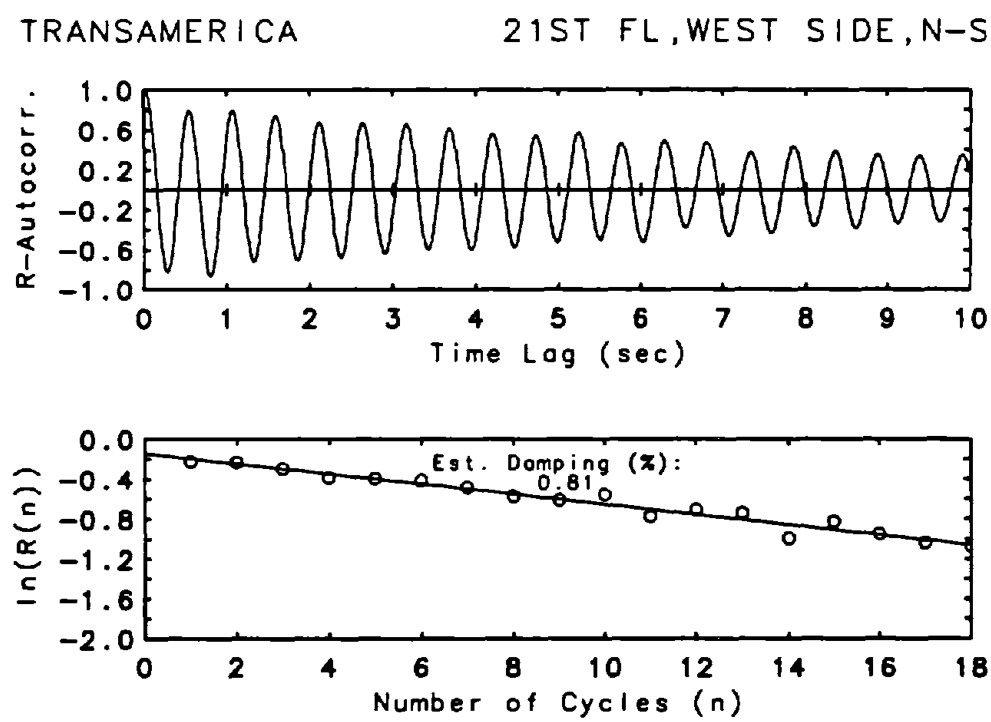

Figure 27. Auto-correlation curve and least-squares fit to amplitude decay, 21st floor, west side, N-S direction, Transamerica Building, San Francisco.

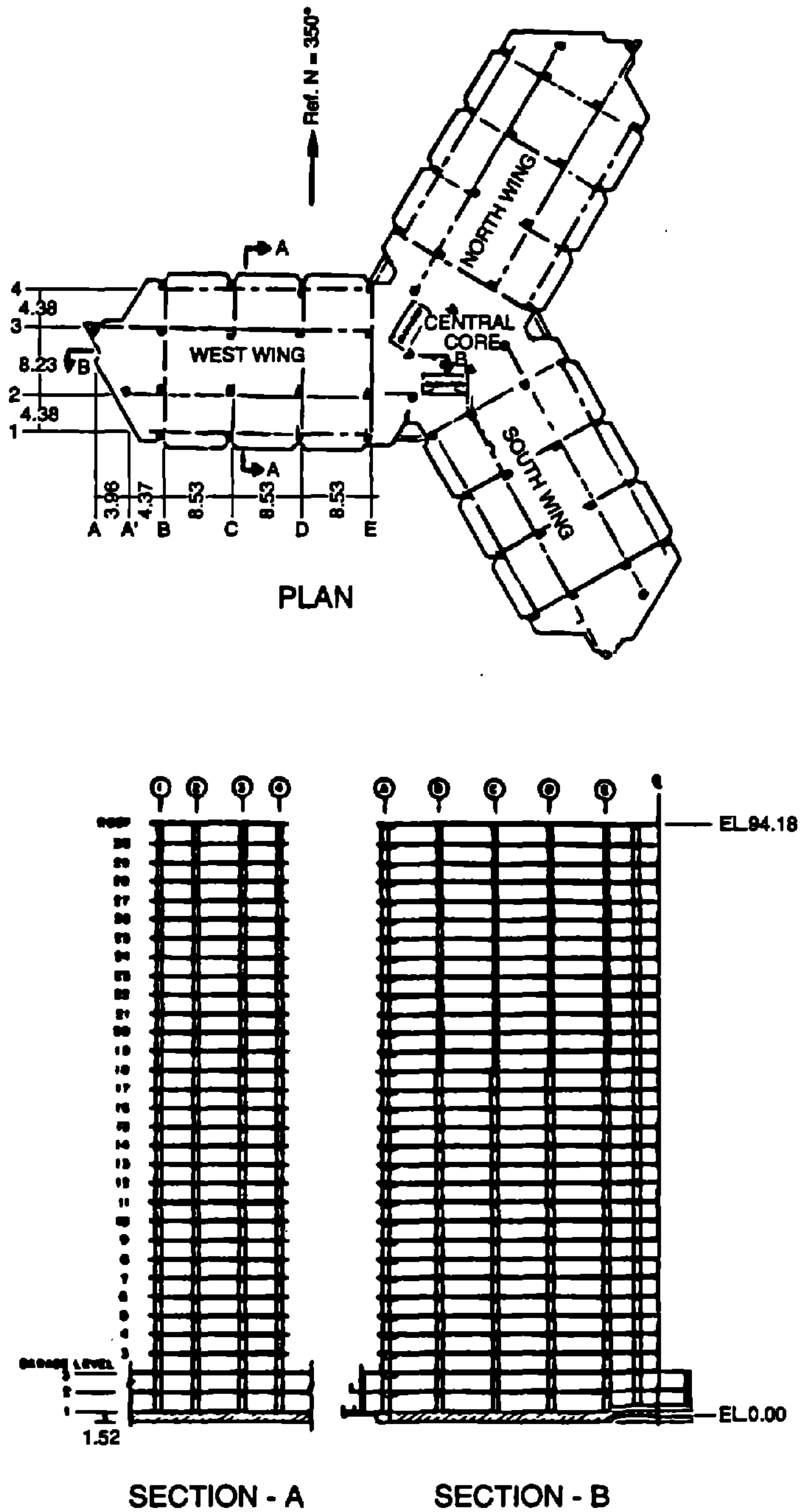

Figure 28. Building plan and sectional elevations, Pacific Park Plaza, Emeryville (adapted from Çelebi and Şafak, 1992). All dimensions in meters.

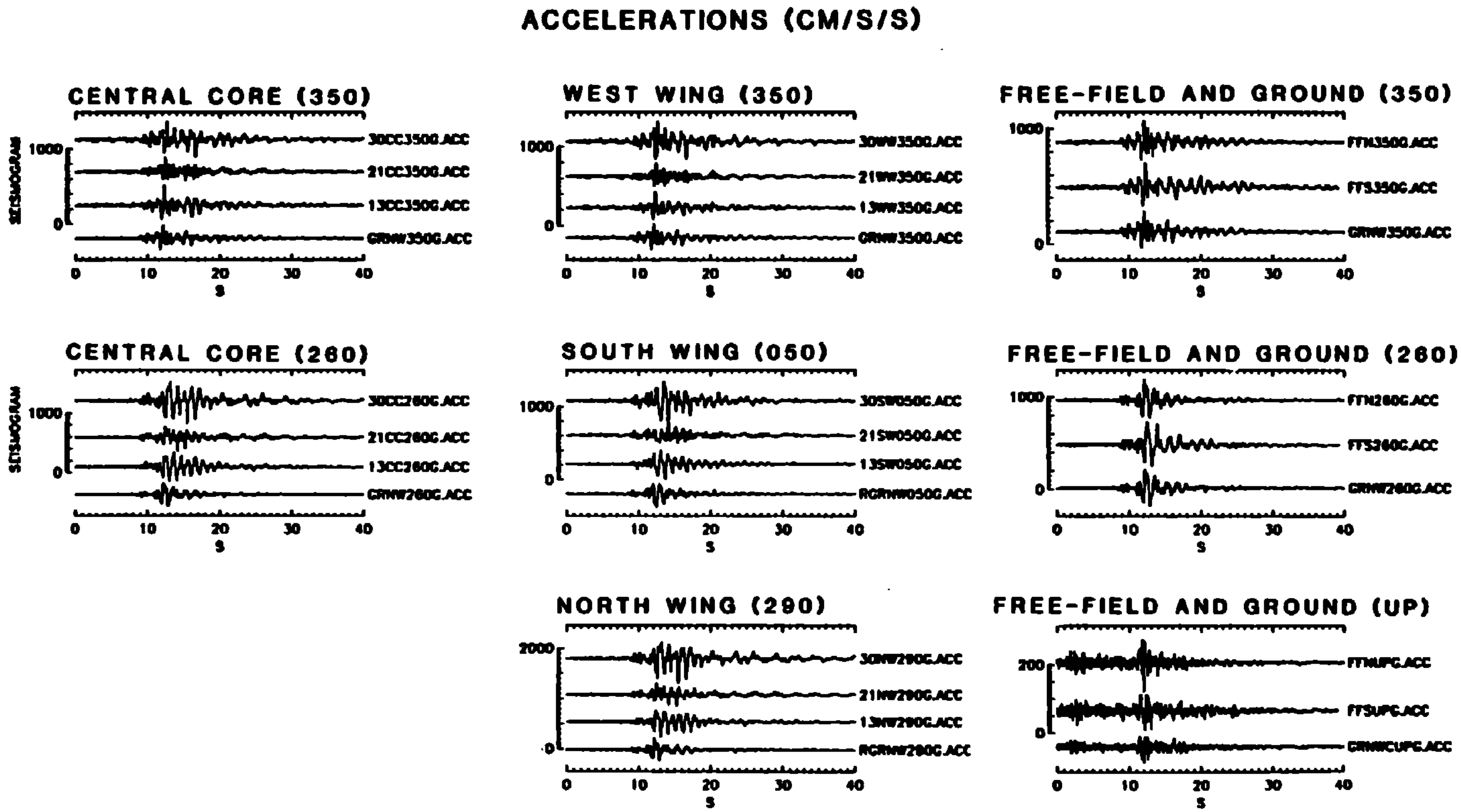

Figure 30. Strong-motion acceleration time histories, Pacific Park Plaza, Emeryville.

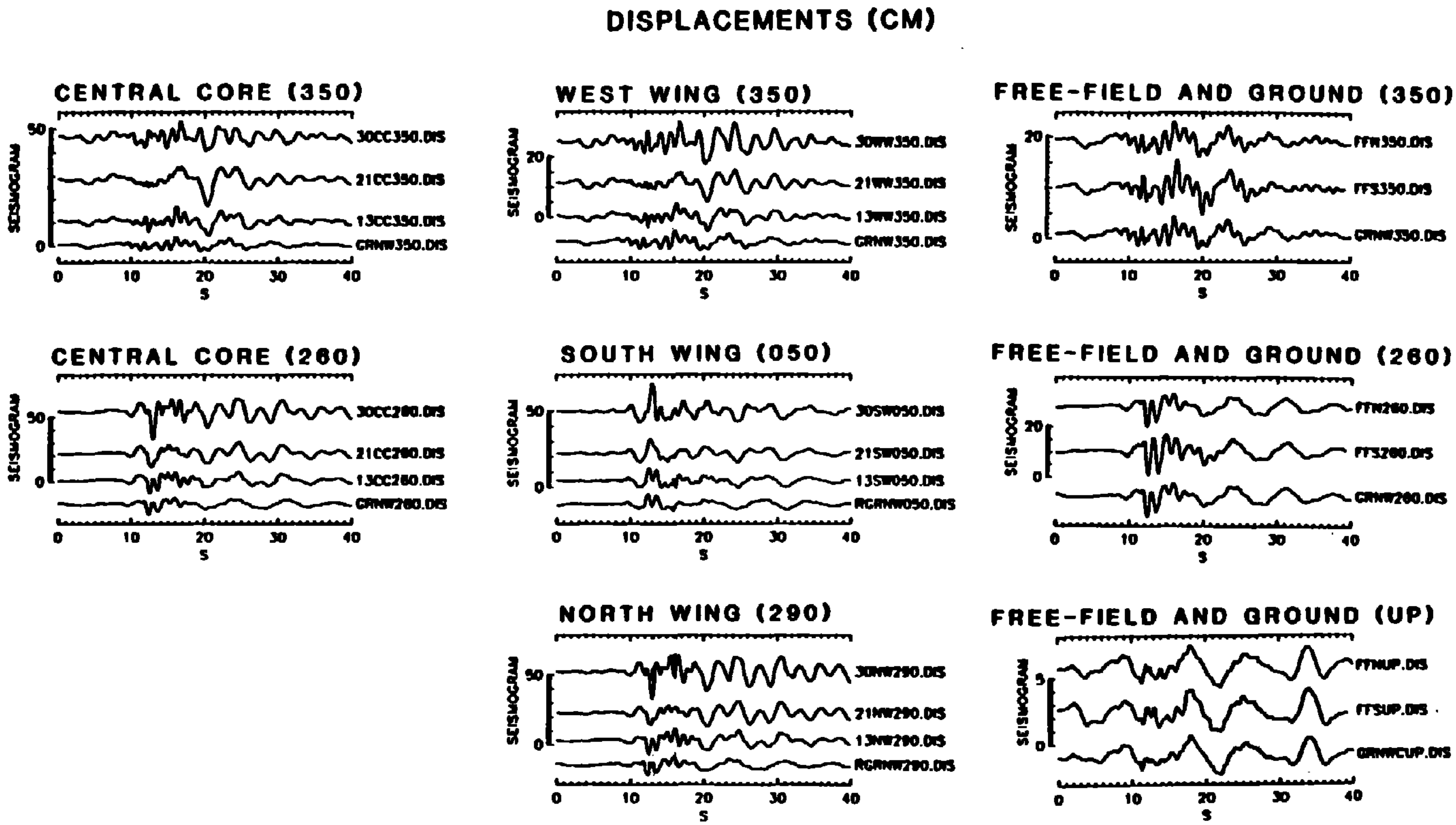

Figure 31. Integrated displacement time histories, Pacific Park Plaza, Emeryville.

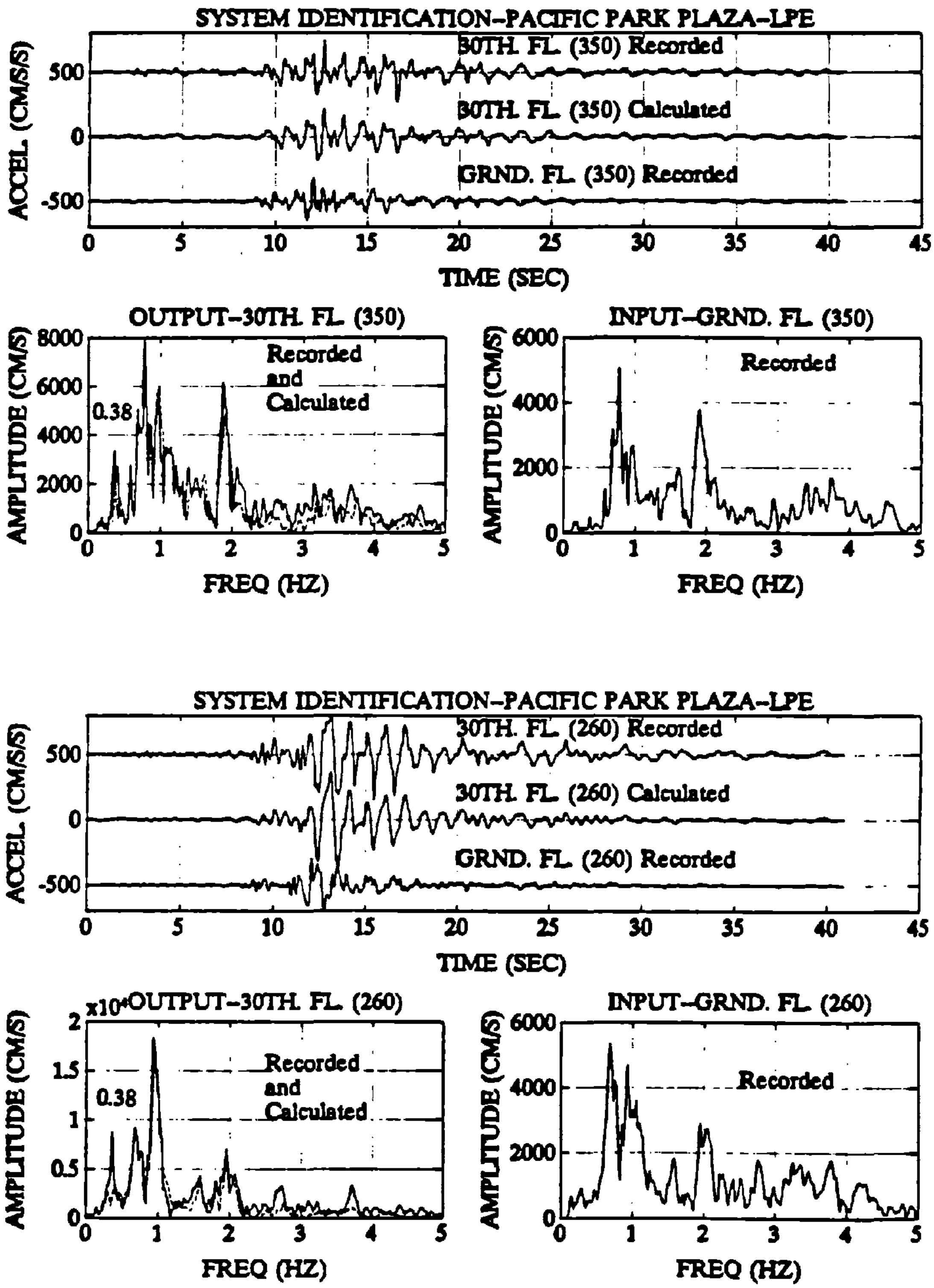

Figure 32. Recorded and calculated (by system identification) 30th floor central core accelerations and Fourier amplitude spectra with ground-floor motion as input, N-S and E-W components, Pacific Park Plaza, Emeryville.

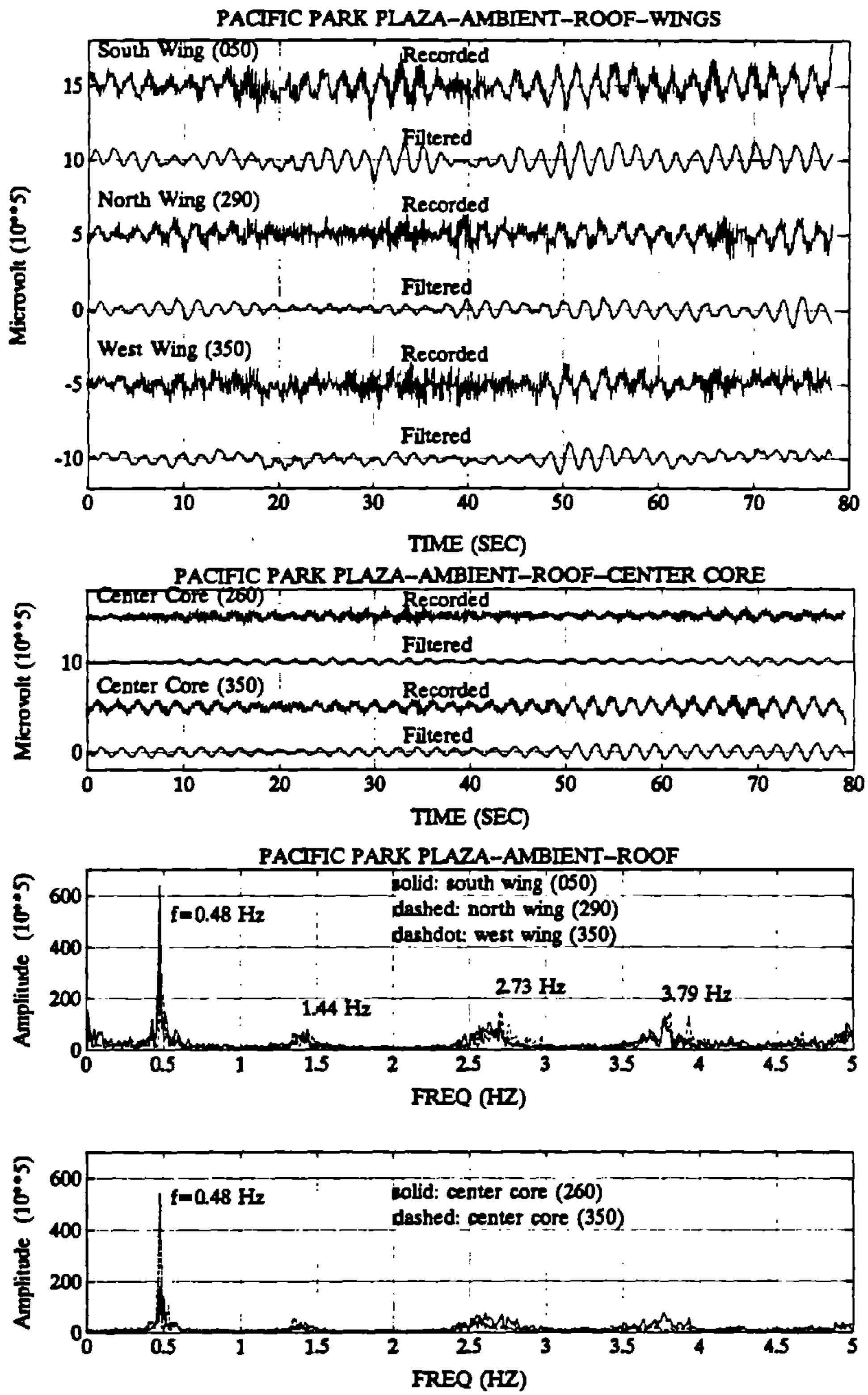

Figure 33. Ambient vibration response and Fourier amplitude spectra for 30th floor, north, south and west wings, Pacific Park Plaza, Emeryville.

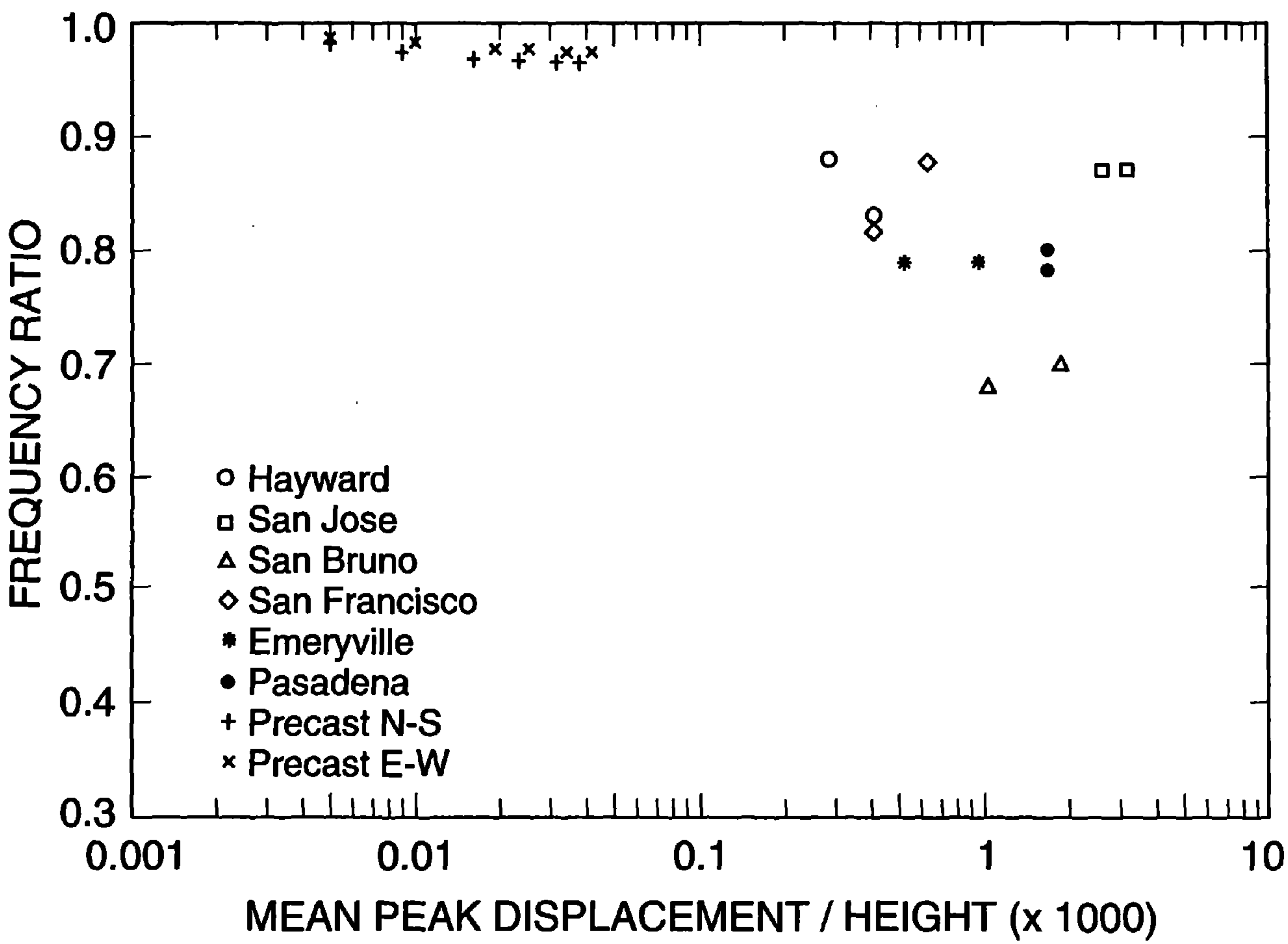

Figure 34. Frequency ratio vs. mean peak displacement ratio.

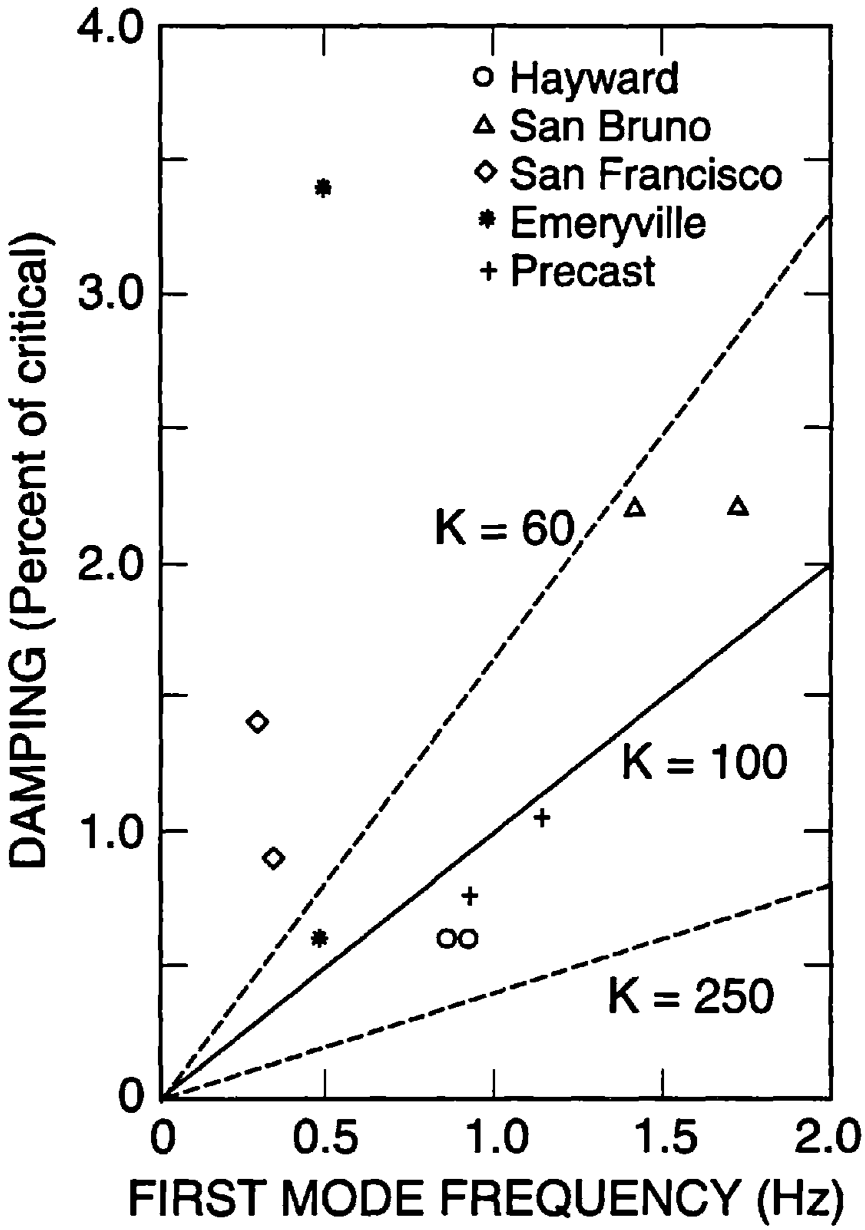

Figure 35. Ambient vibration damping estimates vs. 1st-mode frequency - comparison with ESDU model.

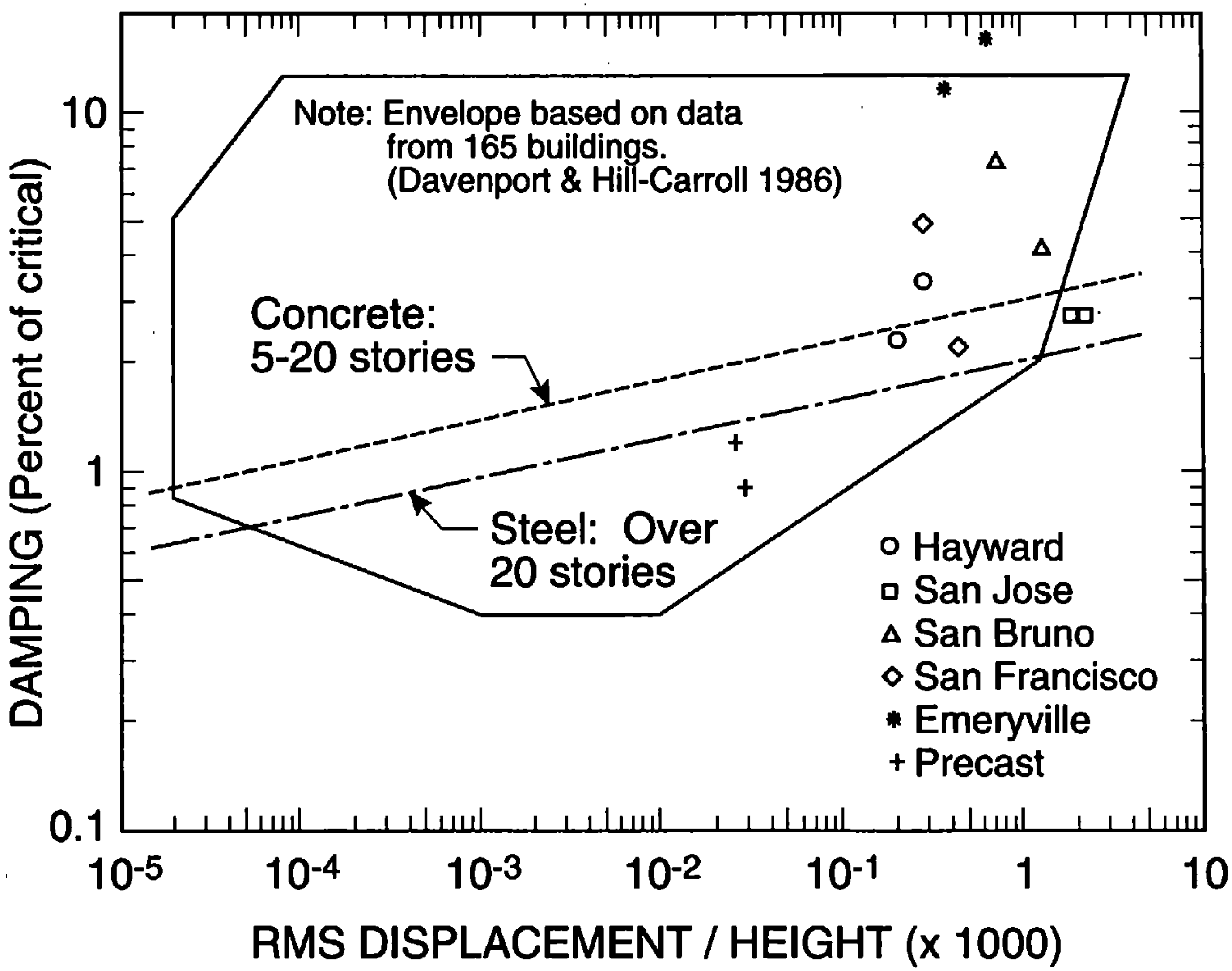

Figure 36. Strong-motion damping estimates vs. rms displacement ratio - comparison with published data and recommended values.