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ANALYSIS OF SOIL-STRUCTURE INTERACTION EFFECTS
ON BUILDING RESPONSE FROM EARTHQUAKE
STRONG MOTION RECORDINGS AT 58 SITES

by

Jonathan P. Stewart and Alisa F. Stewart

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Earthquake Engineering Research Center
College of Engineering
University of California, Berkeley

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1.0 Introduction

This report was prepared as part of a project which investigated the effects of soil-structure interaction (SSI) on seismic structural response using strong motion data from 58 sites. Presented here are site conditions, modal parameters evaluated from system identification analyses, and comparisons of empirically determined SSI effects with predictions from simplified analytical formulations. It is not the purpose of this report to provide detailed descriptions of the analytical procedures nor to interpret the cumulative results from all the sites; these discussions are provided in a companion report (Stewart, 1996).

The 58 sites considered in this study are listed in Table 1, and many of the site and earthquake locations are shown in Figs. 1. The sites are broken down into two broad categories: (1) 'A' sites which have a free-field accelerograph and a structure instrumented with at least foundation- and roof-level lateral sensors, and (2) 'B' sites which have a structure instrumented with foundation- and roof-level lateral sensors as well as foundation-level vertical sensors to detect base rocking, but no free-field instrument. Data and analysis results for the 'A' and 'B' sites are presented in Appendices A and B of this report, respectively.

Sections 2.0 and 3.0 of this report outline the procedures by which relevant geotechnical, structural, and foundation conditions at the sites were quantified for use in SSI analyses. A brief overview of the system identification and predictive analytical formulations are presented in Sections 4.0 and 5.0. Finally, the organization of the individual site studies presented in the appendices is discussed in Section 6.0.

2.0 Characterization of Geotechnical Site Conditions

2.1 General Soil Properties

For each site considered in this study, subsurface data was gathered so that geologic conditions could be characterized in the form of a generalized soil column. These soil/rock columns are presented for each site as Fig. A*.3 and Fig. B*.2 for 'A' and 'B' sites, respectively. The geotechnical data reported varies depending on the available information, but generally consists of the site stratigraphy, measured or estimated shear wave velocities, depth to groundwater, and various measured properties such as void ratio, density, plasticity index, SPT blow count, and unconfined-unconsolidated shear strength.

2.2 Shear Wave Velocity Profiles

(a) *General Considerations*

For 29 of the 58 of the sites considered in this study, shear wave velocities (V_s) were measured in situ using techniques such as the downhole method, the cross-hole method, P-S suspension logging, spectral analysis of surface waves (SASW), or seismic refraction. For sites where measurements by one or more of these techniques have been made, the data are shown on

Table 1: Sites included in this study

Site*	Station	No. Eqs.	Rock/Soil	Avg. Vs (ft./sec)	Piles (Y/N)	Embed. (ft.)	No. Stories	Structural System**
'A' Sites								
1	Eureka Silvercrest Apts.	1	S	800	N	0	5	SW
2	Fortuna 1-St. Supermarket	2	S	915	N	0	1	SW
3	Humboldt Bay Power Plant	2	S	980	N	86	1	SW
4	Emeryville Pacific Pk. Plaza	1	S	620	Y	0	31	CF
5	Hayward City Hall	1	R	2210	Y	0	11	DWF
6	Hayward 13-St. School Bldg.	1	R	1350	Y	0	13	DWF
7	Hollister 1-St. Warehouse	1	S	670	N	0	1	SW
8	Piedmont Jr. High School	1	R	1830	N	0	3	SW
9	Pleasant Valley Pump. Plant	2	S	1010	N	32	1	DWF
10	Richmond City Hall	1	S	830	N	10	3	DWF
11	San Jose 3-St. Offc. Bldg.	1	R	2690	N	0	3	SF
12	El Centro Imp. Co. Ser. Bldg.	1	S	580	Y	0	6	DWF
13	Indio 4-St. Gov't Offc. Bldg.	1	S	760	Y	16	4	DWF
14	Lancaster 3-St. Offc. Bldg.	1	S	920	Y	0	3	SW
15	Lancaster 5-St. Hospital	1	S	1030	Y	0	5	SF
16	Lancaster Fox Airfield	1	S	1000	N	0	5	SF
17	Loma Linda VA Hos.	1	S	1520	Y	0	4	SW
18	Long Beach Har. Ad. Bldg.	1	S	700	Y	0	7	SF
20	Long Beach VA Hospital	1	S	1210	N	23	11	SW
21	LA 2-St. FCCB	3	R	1030	N	0	2	BI
22	LA 3-St. Comm. Bldg.	1	S	1160	N	23	3	DWF
23	LA 6-St. Offc. Bldg.	1	S	500	N	14	5	SF
24	LA 6-St. Pkg. Garage	1	S	1200	Y	0	6	SW
25	LA 7-St. USC Hos.	2	R	1170	N	0	7	BI
26	LA 7-St. UCLA Bldg	1	S	700	N	14	7	DWF
27	LA 15-St. Offc. Bldg.	2	R	1180	N	0	17	SF
28	LA 19-St. Offc. Bldg.	1	S	1160	Y	38	19	SF
29	LA Hollywood Storage Bldg.	4	S	960	Y	9	14	CF
30	LA Wadsworth VA Hospital	1	S	1130	Y	31	6	SF
31	Newport Beach Hoag Hospital	2	S	1050	N	0	11	SW
32	Norwalk 12400 Imp. Hwy.	1	S	910	Y	14	7	SF
33	Norwalk 12440 Imp. Hwy.	2	S	1000	Y	15	7	SF
34	Palmdale 4-St. Hotel	1	S	1620	N	0	4	SW
35	Pomona 2-St. Bldg.	2	S	1280	Y	11	2	CF
36	Pomona 6-St. Bldg.	2	S	1250	N	13	6	CF
37	Rancho Cucamonga LJC	5	S	1190	N	14	4	BI
38	San Bernardino 3-St.	1	S	980	N	0	3	SF
39	San Bernardino 5-St.	1	S	1310	N	13	5	SW
40	San Bernardino Vanir Tower	1	S	920	Y	0	9	SF
41	San Bernardino Co. Govt Cntr	1	S	1060	Y	0	5	SF
42	Santa Susana Bldg 462	1	R	4500	N	0	8	SF
43	Seal Beach Rockwell Bldg 80	2	S	970	Y	16	8	BI
44	Sylmar Olive View Med. Cen	2	S	1530	N	0	6	SW
45	Ventura 12-St. Hotel	1	S	920	Y	0	12	SW

46	Lotung Containment Structure	1	S	460	N	15	1	SW
'B' Sites								
1	Milpitas 2-St. Offc. Bldg.	1	S	<i>690</i>	N	0	2	SW
2	San Bruno 9-St. Offc. Bldg.	1	S	<i>1010</i>	N	0	9	SW
3	San Fran. 47-St. Offc. Bldg.	1	S	570	Y	39	47	SF
4	San Fran. Transamerica Tow.	1	S	960	N	51	60	SF
5	San Jose 10-St. Resid. Bldg.	1	S	<i>840</i>	Y	0	10	SW
6	San Jose 13-St. Gov't Offc.	1	S	770	N	14	12	SF
7	Walnut Creek 10-St. Offc.	1	R	1450	Y	0	10	DWF
9	El Segundo 14-St. Offc. Bldg.	1	S	1010	N	0	14	SF
10	LA 9-St. Offc. Bldg.	1	S	960	N	13	9	DWF
11	LA 17-St. Resid. Bldg.	2	R	1200	Y	0	17	SW
12	LA 32-St. Offc. Bldg.	1	R	1380	N	54	32	DWF
13	LA 54-St. Offc. Bldg.	1	R	1430	N	59	54	SF
14	Whittier 8-St. Hotel	1	S	870	N	0	8	SW

Shear wave velocities in *italics* are estimated.

*Sites A19 and B8 omitted

** Lateral force resisting systems:

SW = masonry or concrete shear walls

DWF = dual wall/frame system

CF = concrete frame

SF = steel frame

BI = base isolated

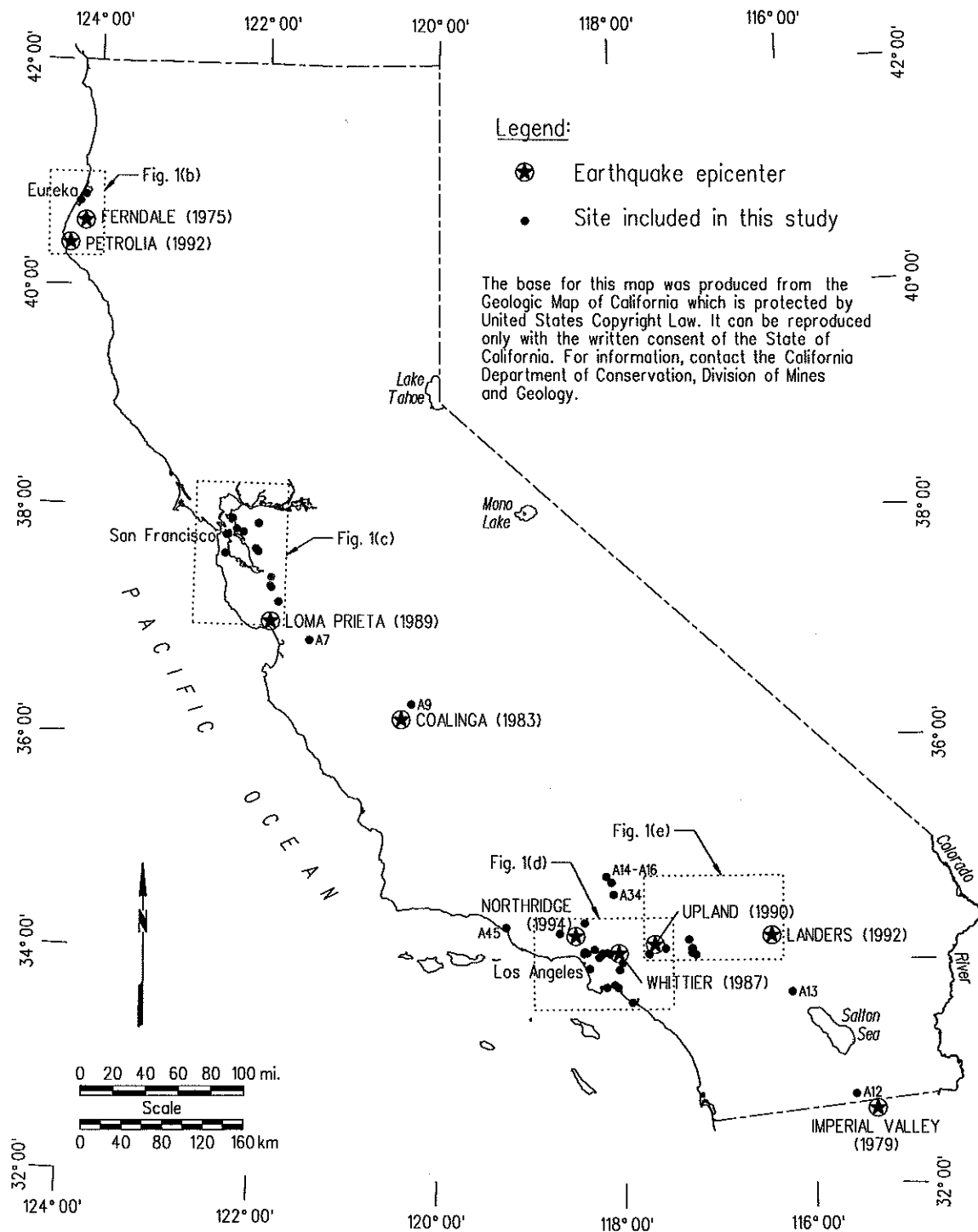


Fig. 1(a): Map of California showing locations of sites and selected earthquakes considered in this study

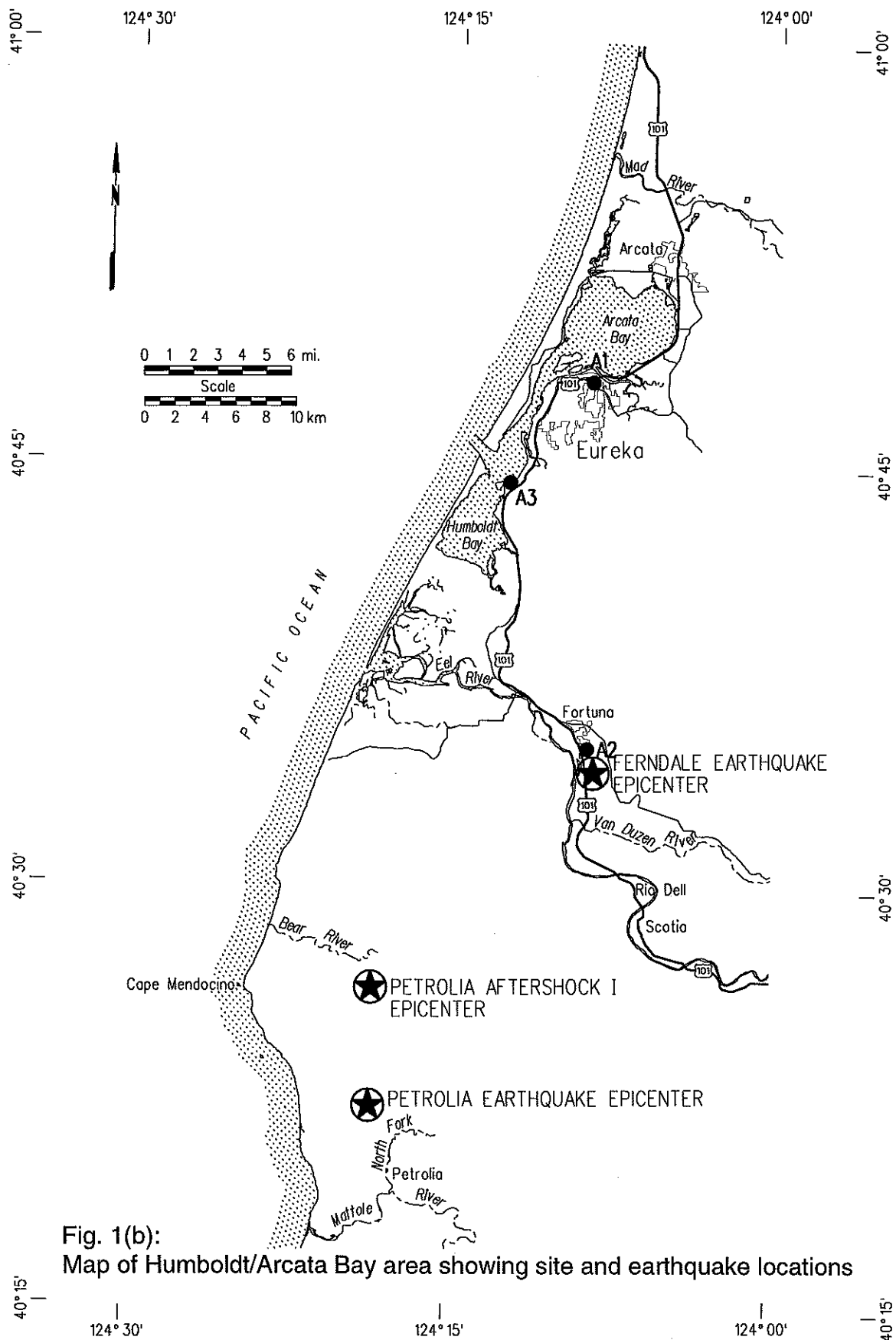


Fig. 1(b):
Map of Humboldt/Arcata Bay area showing site and earthquake locations

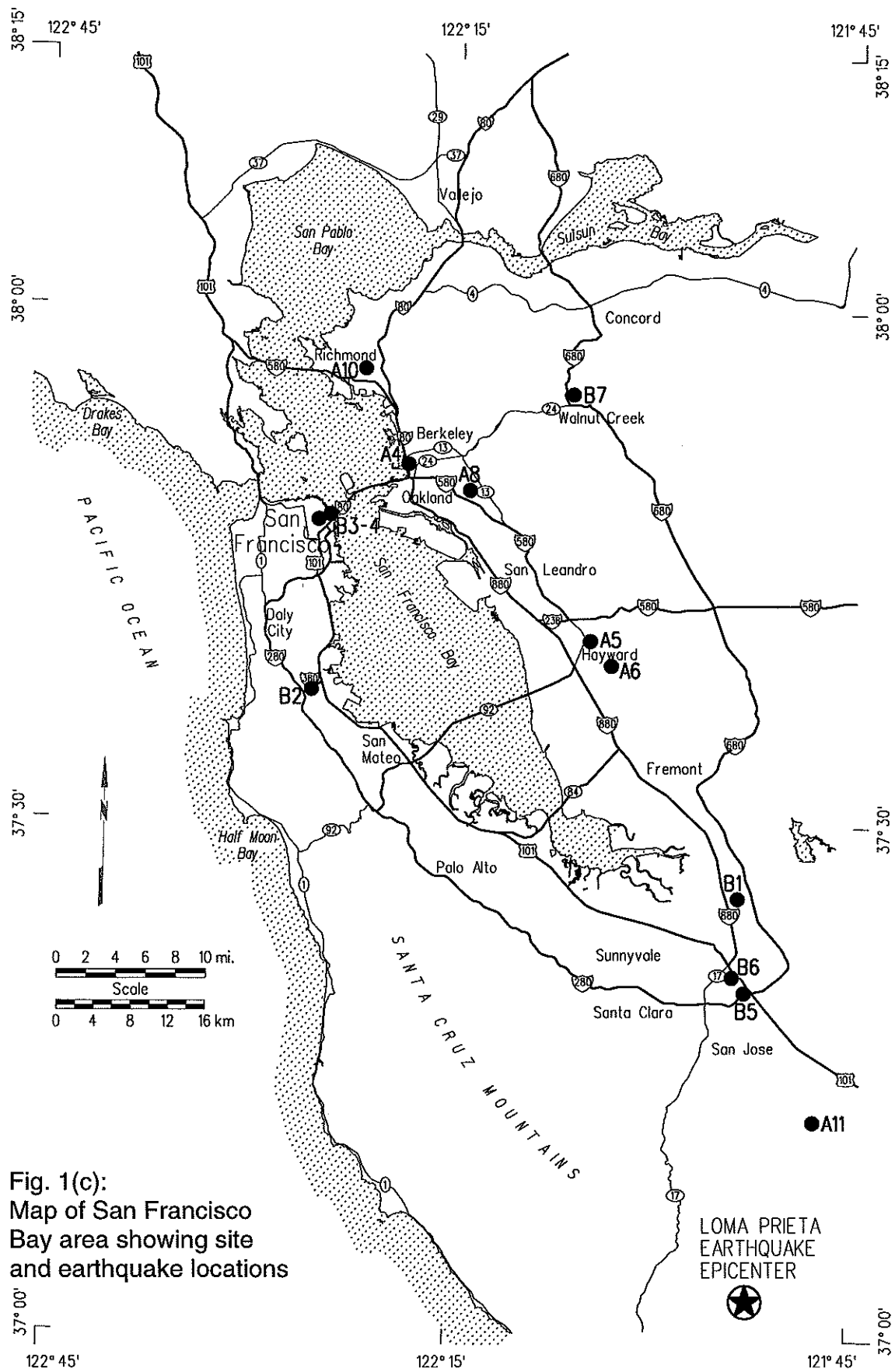


Fig. 1(c):
Map of San Francisco
Bay area showing site
and earthquake locations

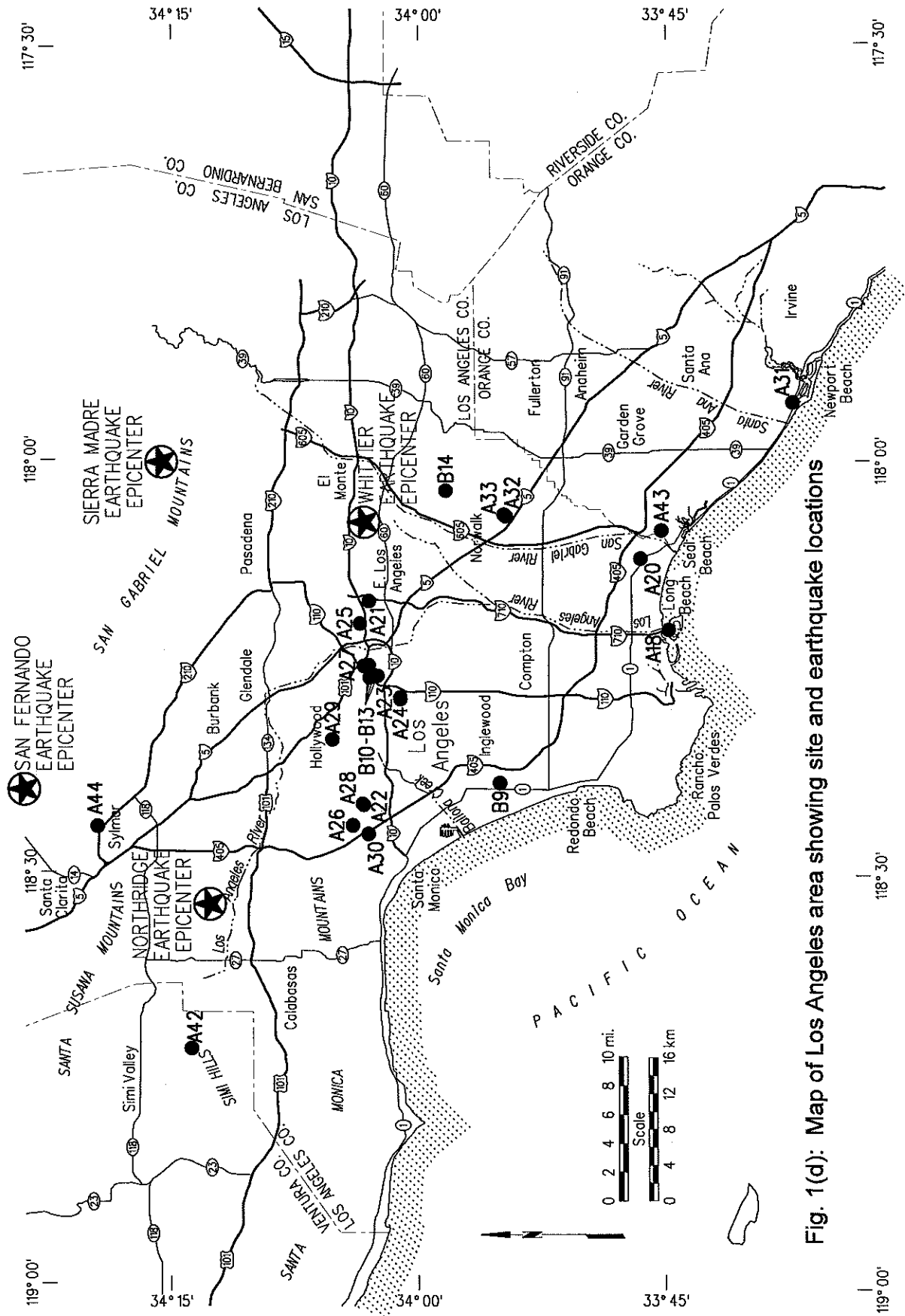


Fig. 1(d): Map of Los Angeles area showing site and earthquake locations

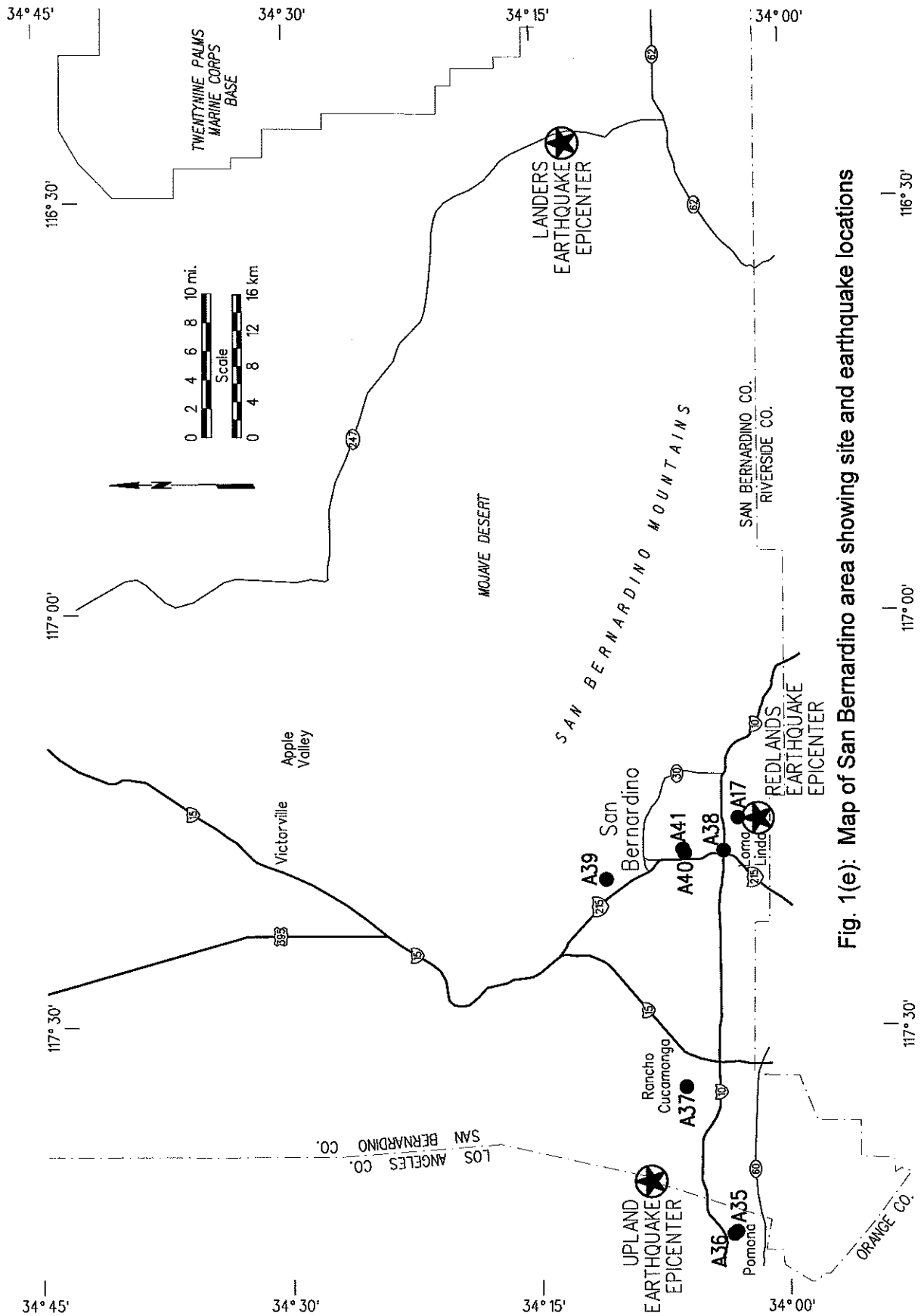


Fig. 1(e): Map of San Bernardino area showing site and earthquake locations

the generalized soil profile as "Measured V_s ," and the specific measurement technique employed is reported in the notes.

Measurements of V_s were not available at the remaining 29 sites, thus requiring estimates to be made from other geotechnical data by one of the following means:

- When V_s measurements are available from a nearby site (i.e. < 1/2 mile away) in geologic materials similar to those at the subject site, these data were used. Such V_s estimates were used at six sites (A7, A18, A26, A27, A30, B1).
- For rock sites, where in situ V_s measurements are available from other sites in a similar geologic formation (but possibly at significant distance, i.e. > 1/2 mile), these data were used. Profiles of V_s were estimated by these means at two sites (A21 and A42).
- For soil sites with laboratory data from soil samples and/or in situ measurements of soil penetration resistance, V_s was estimated from published correlations. Such estimates were made at 21 sites.

The estimation of V_s from correlation relationships is discussed below.

(b) *Estimation of Shear Wave Velocity from Published Correlations*

Correlation studies have been conducted to relate in situ V_s measurements to physical soil properties for the Southern California area (Fumal and Tinsley, 1985), the San Francisco Bay area (Fumal, 1978 and Dickenson, 1994), and nationwide (Sykora, 1987). Other studies have sought to develop generalized relationships between key soil parameters and V_s for clayey soils (e.g. Hardin and Drnevich, 1972a,b) and sandy soils (e.g. Seed et al., 1984). As reported by Dickenson (1994), the correlation coefficients for these relations are often greater for localized studies than for more general studies or studies of large regions.

For the estimation of V_s in this study, region-specific correlations were most heavily relied upon where available. For sites in the Southern California area, the relationship in Fig. 2 by Fumal and Tinsley (1985) was used for cohesive and sandy soils. For sandy soils, V_s estimates from the procedure in Seed et al. (1984) were also considered. This procedure relates small strain shear modulus G_{max} ($=V_s^2 \times \text{mass density}$) to mean effective pressure and relative density as follows,

$$G_{max} = 1000(K_2)_{max} \sqrt{\sigma'_m} \quad (1)$$

where σ'_m = mean effective stress in psf, G_{max} is in psf, and $(K_2)_{max}$ is related to the soil's relative density. Values of $(K_2)_{max}$ range from about 30 for loose sands to 75 for dense sands, and can be related to corrected blow counts [$[(N_1)_{60}]$ values] from Standard Penetration Tests (SPT) as follow (Seed et al., 1984):

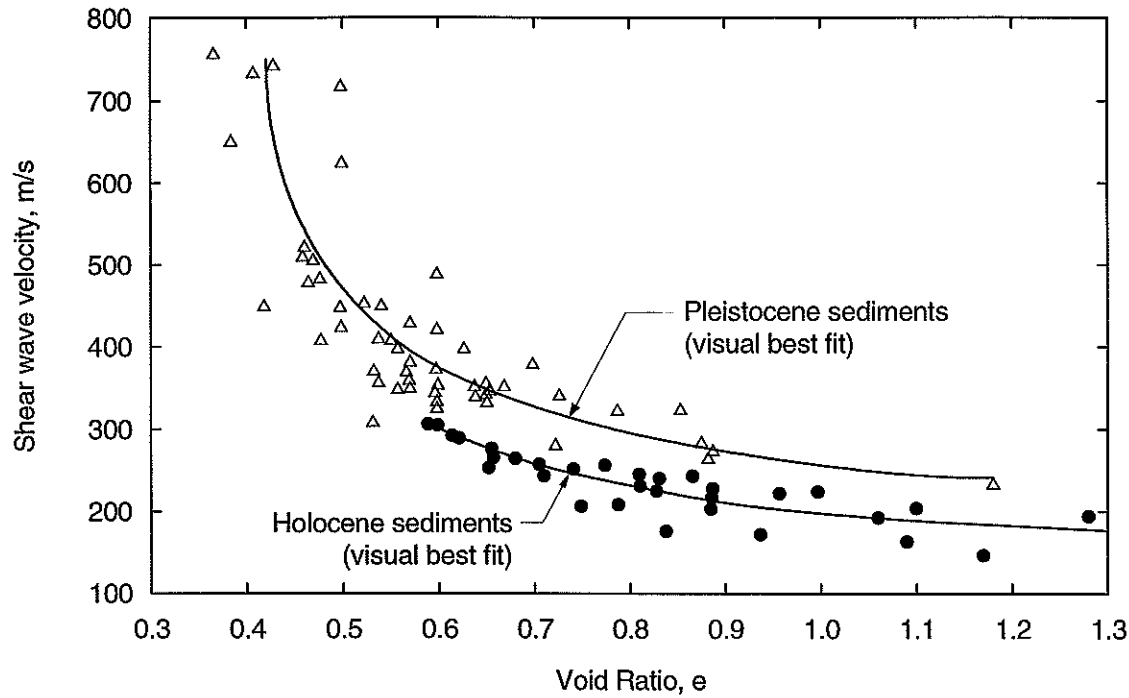


Fig. 2: Variation of measured shear wave velocity with void ratio for Holocene and Pleistocene sediments in the Los Angeles region. Data are for sand and cohesive soils (Fumal and Tinsley, 1985).

Table 2: Measured shear wave velocity in various sedimentary deposits (Fumal, 1978).

Geotechnical Unit	Geologic and Physical Property of Units Included	Depth (m)	V_s (m/s)
Bay Mud	Clay-silty clay	0-10	55-100
	v. soft-soft	10-20	100-115
Holocene fine-grained alluvium	Clay-silty clay	0-7.5	160-175
	med.-stiff		
Late Pleistocene fine-grained alluvium	Clay-silty/sandy clay, silt loam	0-12	165-275
	Interbedded fine and coarse sediment	12-30	165-300
Sand	Dune sand ($D < 7.5m$), artificial fill, Holo. v. fine-fine sand, Merritt sand, Merritt sand	0-15	150-245
Sand dense-v. dense nr. surf.	Merced Fm., Colma, Fm., Holocene coarse, Purisima Fm	0-12	250-340
Dune Sand	Dune sand, Merritt sand	5-20	300-380
Gravel and v. coarse sand	Late Pleistocene med. & crs. Santa Clara Fm.	2.5-20 10-30	340-500 320-740
Merced Fm. at depth	Merced Fm., Purisima Fm.	12-30	310-490
Colma Fm. at depth	Colma Fm.	7.5-30	450-520

$(N_1)_{60}$	$(K_2)_{\max}$
5	34
8	40
10	18
18	52
28	61
44	71

As used here, $(N_1)_{60}$ refers to the blow count from a 140-lb hammer dropping 30-inches with a standard 2-inch outside diameter "split spoon" sampler, with the data corrected to an equivalent one atmosphere mean effective pressure and 60% hammer/rod efficiency (Seed et al., 1985). Blow count data obtained from other samplers such as the commonly used "Modified California" sampler was not used. Where SPT blow counts were not available for sandy soils, field descriptions of soil relative density were used to estimate $(K_2)_{\max}$ for use in Eq. 1.

Estimates of V_s from the Fumal and Tinsley (1985) correlation, with supplemental insight from the Seed et al. (1984) correlation for sandy soils, were developed for 15 sites in the Southern California region (i.e. sites A13-16, A31-33, A35-41, and A45). These procedures were also used for site A9 (near Coalinga, CA), which is located in a region having an arid climate similar to that in Southern California.

For sites in the San Francisco Bay area, correlations developed by Fumal (1978) for different geologic units were used (Table 2). Results obtained from these correlations were compared to results from the void ratio correlation by Fumal and Tinsley (1985) and the procedure by Seed et al. (1984) for sands. V_s profiles were developed in this way for site B2 (Colma Formation), site B4 (Dune Sand, Holocene and Late-Pleistocene alluvium), and sites B5-B6 (Holocene and Late-Pleistocene alluvium).

For a sandy soil site in Northern California (site A2), the procedure in Seed et al. (1984) was used to estimate V_s .

2.3 Estimation of Poisson's Ratio of Geologic Materials

A soil/rock property of interest in this study, but generally not available from laboratory test data, is the Poisson's ratio ν . This parameter was estimated as $\nu = 0.33$ for unsaturated sands, $\nu = 0.4$ for unsaturated low- to moderate-plasticity clays, and $\nu = 0.45$ for saturated deposits and high-plasticity clays. Soil profiles were considered to be saturated if the ground water level was in the upper half of the effective profile depth, where the profile depth was based on the foundation dimensions (Section 2.4).

2.4 Evaluation of Dynamic Soil Properties for Impedance Functions

The evaluation of foundation impedance functions under earthquake loading conditions requires that the effects of ground shaking on geologic media within the zone of significant influence of the foundation be accounted for. Relatively simple impedance function formulations

were employed in this study which require single values of shear modulus G (or shear wave velocity V_s) and soil hysteretic damping β . For a given site, once the small strain V_s profile is defined, the two principal factors involved in evaluating representative G and β values are: (1) the degradation of soil shear stiffness and associated increase in damping with shear strain, and (2) the size of the zone within the ground most significantly affected by the foundation (i.e. the extent of the geologic materials which must be considered in developing the impedance function).

The degrading effects of large ground strains on dynamic soil behavior are often quantified by relationships for modulus degradation ($G/G_{\max}$) and damping (β) vs. shear strain. Material-specific relationships from published laboratory test results were used at three sites (A4, Guha et al., 1993; A46, Bechtel, 1991; B3, Dickenson, 1994). At the other sites, published relationships for different soil types were used. For cohesive soils (Fig. 3), the modulus degradation curves by Sun et al. (1988) were used in conjunction with the damping curves by Vucetic and Dobry (1991). For cohesionless soils (Fig. 4), the modulus degradation and damping relationships presented in Seed et al. (1984) were used. For intact bedrock materials (Fig. 5), the modulus degradation and damping relationships proposed by Schnabel (1973) were used. Heavily weathered bedrock was generally treated as cohesive or cohesionless soil, depending on the composition of the rock.

The geometric extent of geologic materials to include in an impedance function evaluation is dependent on the variation of shear modulus with depth. As discussed in Sec. 2.2.2(b) of Stewart (1996), most profiles can be characterized by an equivalent halfspace or a finite soil layer overlying a rigid base. Geologic profiles without a large, abrupt increase in V_s within a depth of about two foundation radii were modeled as a halfspace, with the shear modulus and damping for the halfspace defined from the materials within one foundation radius (r_1 , see Eq. 5 below) of the base of the structure. Where at least a two-fold, abrupt increase in V_s is present at a depth of about 1/2 to 2 foundation radii, the soil was modeled as a uniform finite layer overlying a rigid base. This condition was only encountered at site A24.

Using the appropriate modulus degradation and damping relationships, strain-dependent V_s and β values were calculated for the layers in the profile using a deconvolution procedure. For profiles modelled as a halfspace, the depth over which the surface motions were deconvolved was about r_1 plus the embedment depth. The calculations were performed using the equivalent-linear dynamic ground response analysis program SHAKE (Schnabel et al., 1972). Using the strain-dependent V_s and β values for the soil/rock layers within the profile, the "average" V_s and β values were calculated as follow:

$$V_s = \frac{r_1}{\sum_i th_i / (v_s)_i} \quad \beta = \sum_i w_i \beta_i \quad (2)$$

where th_i = thickness of layer i ($\sum th_i = r_1$), $(v_s)_i$ and β_i are the shear wave velocity and damping of layer i , and w_i are weighting factors which sum to one and vary linearly from a maximum at the ground surface to zero at the effective depth of the profile (i.e. r_1 plus the embedment depth).

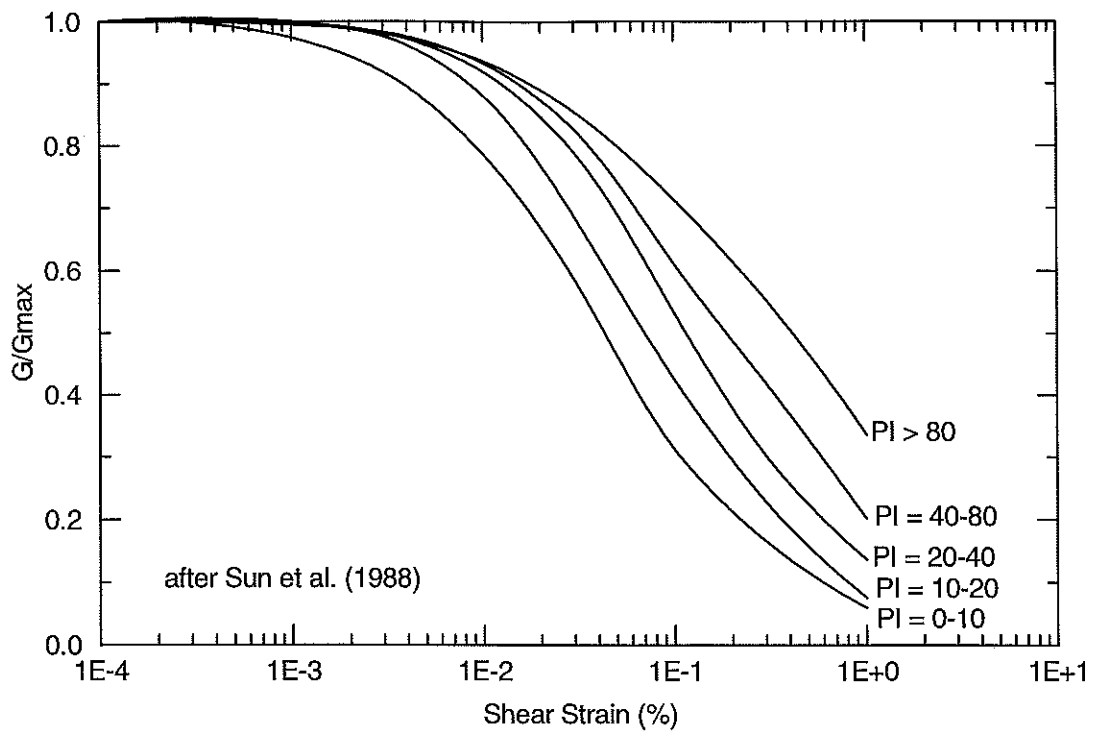
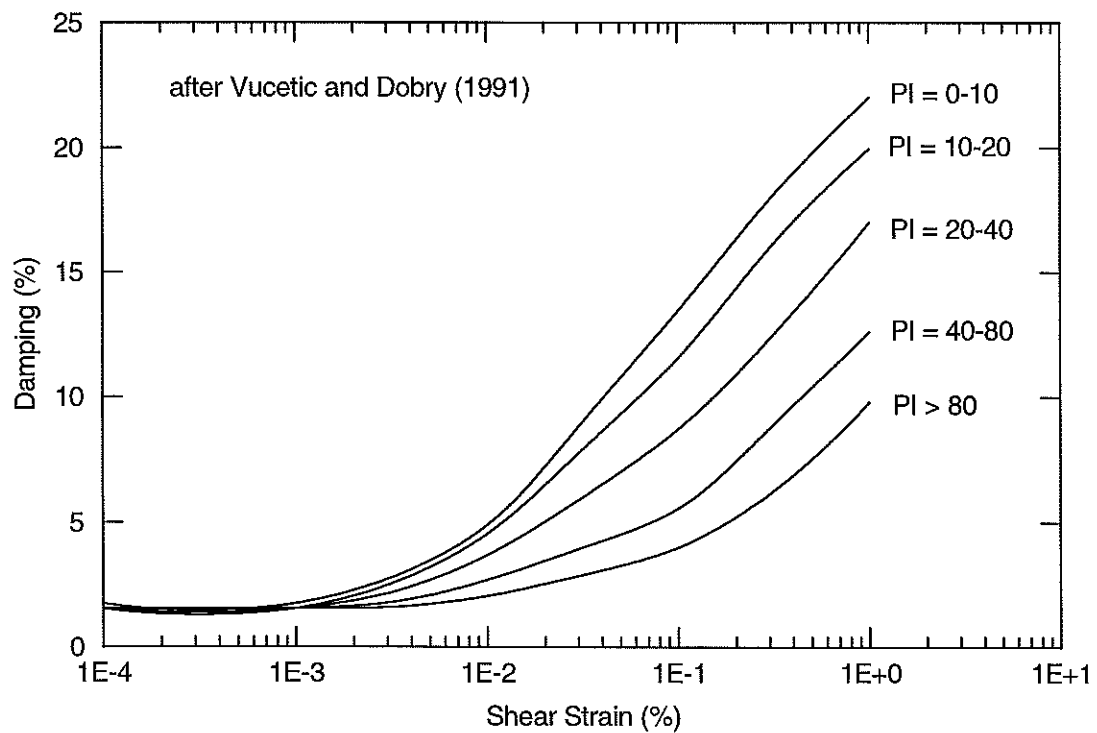


Fig. 3: Modulus degradation and damping curves for clay used in this study. (adapted from Sun et al., 1988 and Vucetic and Dobry, 1991)

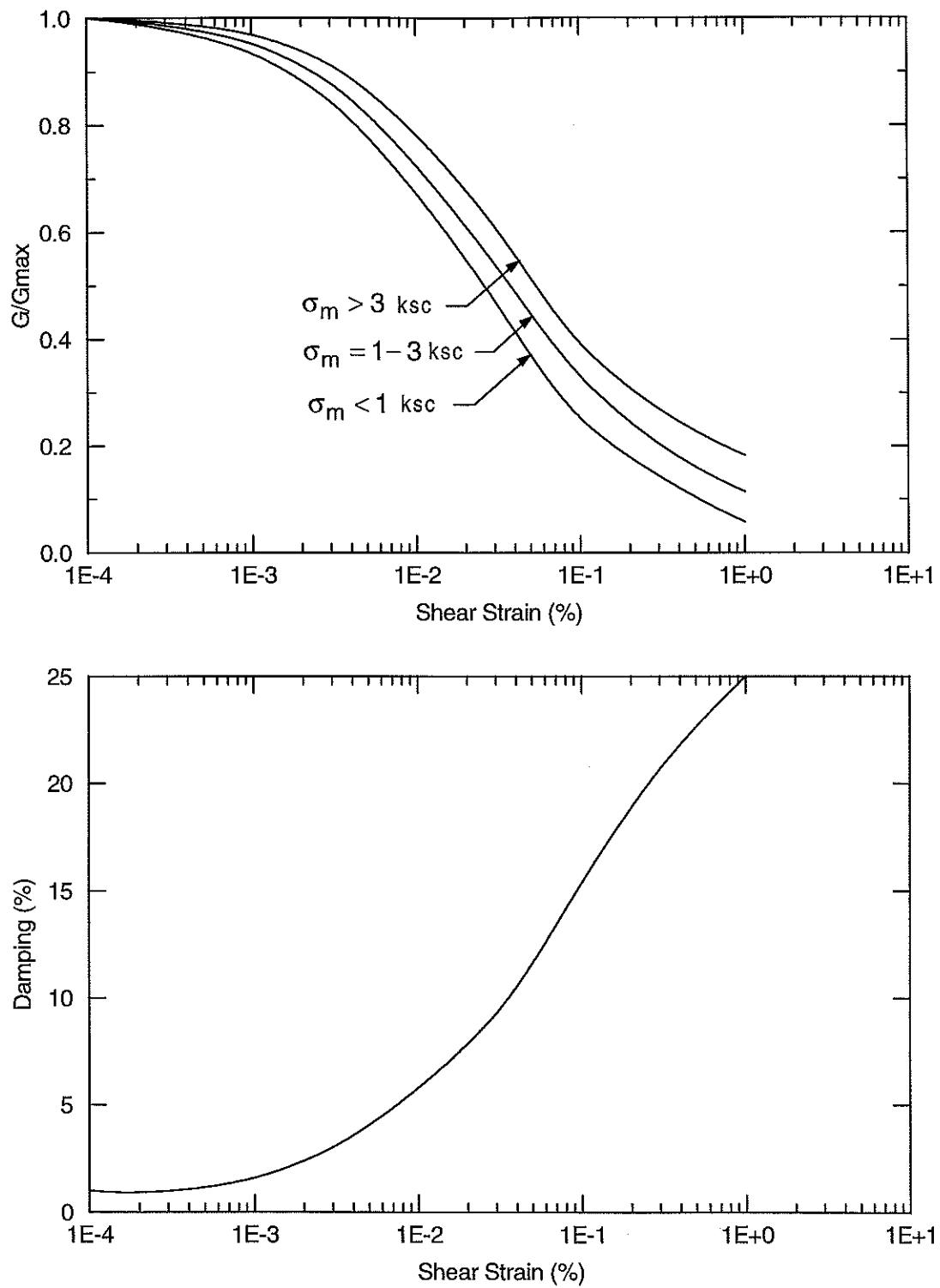


Fig. 4: Modulus degradation and damping curves for sand used in this study.
(adapted from Seed et al., 1984)

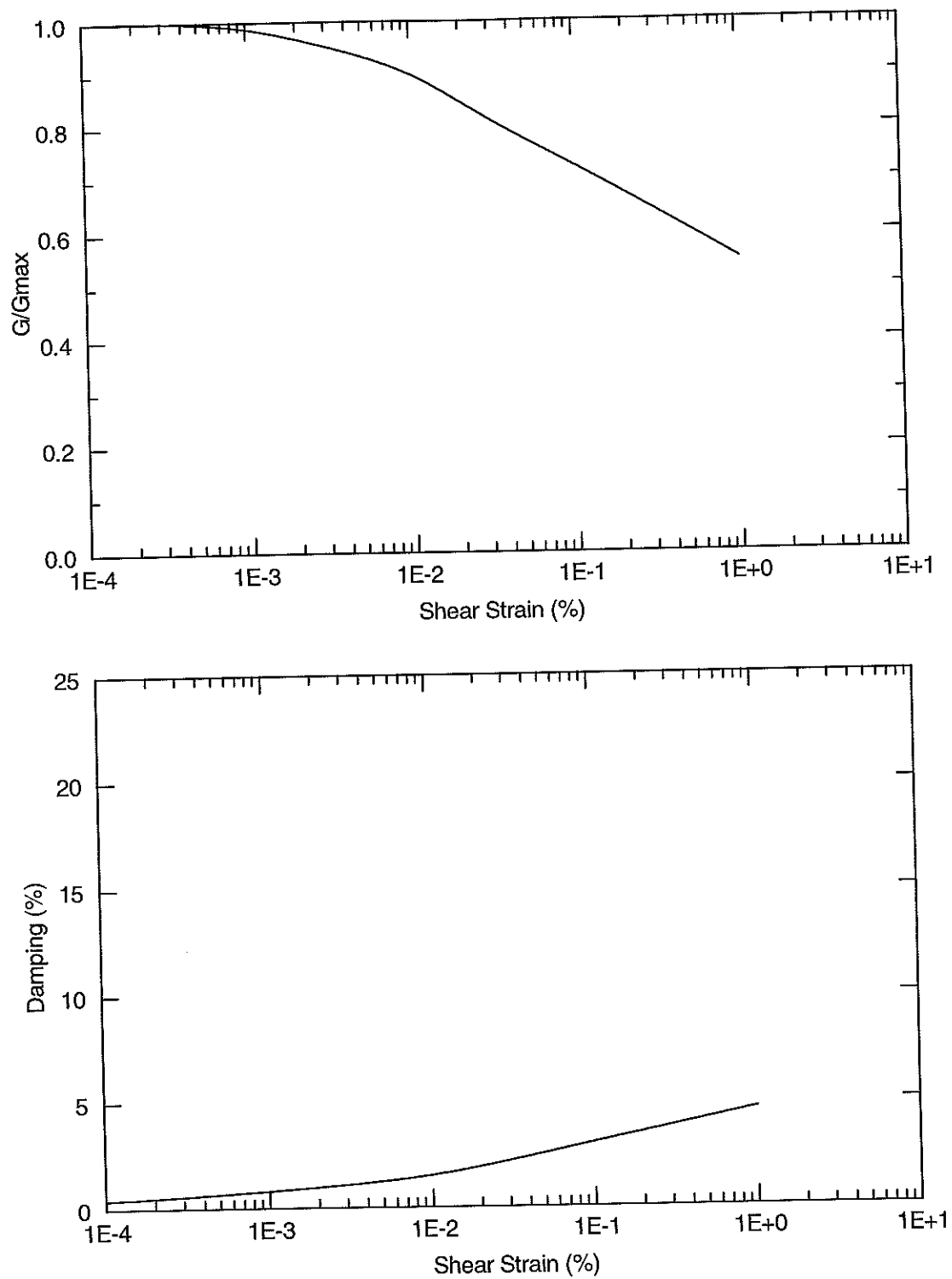


Fig. 5: Modulus degradation and damping curves for rock used in this study.
(adapted from Schnabel, 1973)

The calculation of V_s and β values in this way gives the most weight to the softer materials in the profile and materials nearest the foundation.

3.0 Characterization of Structure and Foundation

For the simplified SSI analyses performed in this study, the structure is simulated by a single degree-of-freedom system with height h , mass m , stiffness k , and viscous damping ratio c (Fig. 6). The foundation is assumed to be a rigid circular disk resting on the ground surface, or embedded into the ground. The purpose of this section is to define how the building height, mass, and foundation radii were evaluated.

The effective height of the single degree-of-freedom structural model was generally taken as the distance above the foundation level at which the building's mass could be concentrated to yield the same base moment that would occur in the actual structure assuming a linear first-mode shape. This effective height can be calculated as,

$$h^2 = \frac{\sum_i m_i h_i^2}{\sum_i m_i} \quad (3)$$

where m_i = mass at level i and h_i = height of level i . For many structures, the effective height was found to be $h \approx 0.7H$ (where H = distance from foundation to roof of structure). However, for tall buildings, the effective height using Eq. 3 was typically less than $0.7H$. In such cases, in order to account for potential higher mode effects, the effective height was simply taken as $0.7H$.

The mass of the structure was generally not precisely known. However, in the formulation of the SSI analytical procedure, the structural mass is normalized by the mass of soil underlying the foundation as follows,

$$\gamma = \frac{m}{\rho \pi r^2 h} \quad (4)$$

where ρ = mass density of soil. For conventional building structures, $\gamma \approx 0.1$ to 0.2 , and the representative value of $\gamma \approx 0.15$ recommended by Veletsos and Meek (1974) was adopted for most structures in this study. In only a few cases (e.g. sites A42 and A46) was the structural mass more precisely computed.

Foundation radii were generally computed to match the area (A_f) and moment of inertia (I_f) of the actual foundation as follow,

$$r_1 = \sqrt{\frac{A_f}{\pi}} \quad r_2 = \sqrt[4]{\frac{4 \cdot I_f}{\pi}} \quad (5)$$

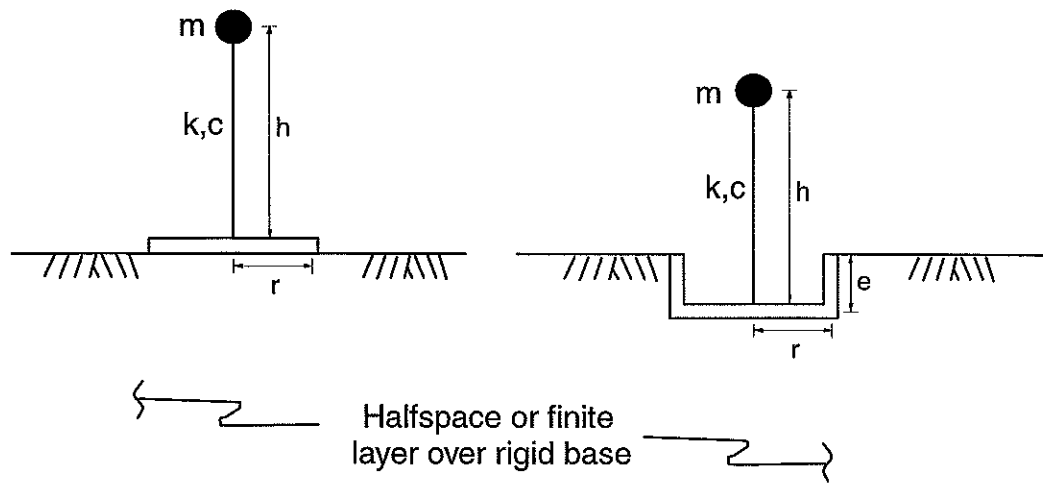


Fig. 6: Simplified surface and embedded structure considered in SSI analyses

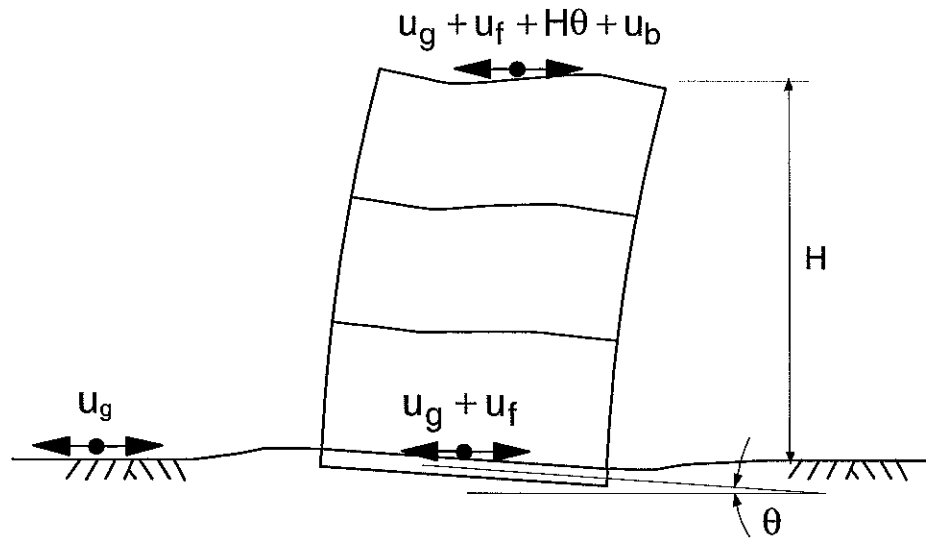


Fig. 7: Motions used as inputs and outputs for system identification of structures

In general, the full footprint area of the foundation was used in the calculation of r_1 and r_2 . Exceptions to this were made when each of the following conditions were met: (1) the lateral force resisting system of the structure is concentrated in a particular area or several distinct areas of the structure, (2) the structural instrumentation used in system identification analyses is located on the lateral force resisting system, and (3) the foundation for the instrumented lateral force resisting system is uncoupled from the rest of the foundation, or the foundation beyond the perimeter of the lateral force resisting system is relatively flexible and does not significantly contribute to the foundation impedance. Having met these criteria, a reduced footprint area was used at sites A23, A24, B1, B7, B11, and B14. In these cases, the area of the foundation beneath the instrumented lateral force resisting system was used.

4.0 Evaluation of Modal Parameters

4.1 Introduction

The quantification of inertial interaction effects requires the evaluation of modal parameters for two cases of base fixity: (1) a “fixed-base” case in which the only flexibility in the system is that of the structure, and (2) a “flexible-base” case in which the system flexibility includes that of the structure as well as lateral and rocking deformations of the foundation. Fixed-base parameters describe the structural behavior in the absence of SSI, while flexible-base parameters describe the performance of the actual structure-foundation-soil system. Since the instrumentation at sites is seldom sufficient to enable both the fixed- and flexible-base parameters to be evaluated directly, a third case is generally considered which can aid in the estimation of whichever set of parameters is not directly evaluated. This third “pseudo flexible-base” case applies to a condition in which the system flexibility includes structural deformations and foundation rocking, but not lateral foundation motions relative to the free-field.

Modal parameters for the structures considered in this study were evaluated using system identification analyses, as described in Chapter 3 of Stewart (1996). Referring to the displacement quantities defined in Fig. 7, the input-output pairs used to evaluate parameters for various base fixity conditions are summarized in Table 3.

Table 3: Required input and output to evaluate system parameters for various conditions of base fixity

	System	Input	Output
a	Flexible-Base	u_g	$u_g + u_f + h\theta + u$
b	Pseudo Flexible-Base	$u_g + u_f$	$u_g + u_f + h\theta + u$
c	Fixed-Base	$u_g + u_f + h\theta$	$u_g + u_f + h\theta + u$

As noted previously, ‘A’ sites have a free-field accelerograph and a structure instrumented with at least foundation- and roof-level lateral sensors, while ‘B’ sites have structures instrumented to record base rocking and foundation- and roof-level lateral motions. Hence, the

flexible- and pseudo flexible-base parameters can be evaluated at 'A' sites, and pseudo flexible-base and fixed-base parameters can be evaluated at 'B' sites. Some 'A' sites also have foundation-level vertical sensors to measure base rocking (A4, A8, A23-25, A31, A33, A37, A43, and A45-46), and hence parameters for all three base fixity conditions were identified in such cases.

Section 4.2 presents an overview of system identification procedures, while Section 4.3 summarizes procedures for estimating fixed-base parameters for some 'A' sites and flexible-base parameters for 'B' sites.

4.2 System Identification Analysis

The general steps carried out in system identification analyses for structures are summarized below. Results of these analyses are presented in the appendices.

1. Acceleration time histories were plotted for free-field motions ('A' sites) as well as structural foundation, mid-height, and roof motions, as available. These data were generally not decimated prior to plotting, but free-field data was rotated as necessary to align it with structural axes. Unprocessed data was filtered before plotting.
2. Power spectral density functions were computed from the horizontal free-field ('A' sites), foundation and roof time histories to illustrate the relative frequency content of these motions. For 'A' sites, response spectra at 5% damping were also computed and plotted for the free-field and foundation motions.
3. Nonparametric transmissibility and coherency functions were computed and plotted for the following output/input pairs: (a) roof/free-field ('A' sites only), (b) roof/foundation, and (c) foundation/free-field ('A' sites only).
4. Vertical motions were analyzed to detect potential base rocking effects. Vertical acceleration time histories and power spectral density functions were plotted. At 'A' sites, transmissibility and coherency functions for foundation/free-field pairs were plotted. These functions can indicate the presence of base rocking effects, as foundation-level vertical motions tend to be amplified relative to free-field motions near the first-mode lateral response frequency if there is significant base rocking. Where base rocking could be directly quantified with pairs of foundation-level vertical instruments, rigid body roof translations from rocking (computed as the product of base rotation and building height) were compared with the difference between roof and base translations to examine the influence of rocking on the roof motions.
5. Parametric system identification analyses were performed by the Cumulative Error Method [CEM; see Stewart (1996), Sec. 3.4.3(a)] to evaluate modal frequencies and damping ratios for appropriate input-output pairs. For each pair, the analysis requires the selection of a delay (d) between the input and output and the number of modes to include in the analysis (J). The selection of d and J is guided by trying to minimize the error between the model output and the

recorded roof time history while not over-parameterizing the model. Results for lower modes whose influence is apparent in the transmissibility function are reported in the appendices.

6. Parametric system identification analyses by the Recursive Predictive Error Method [RPEM; see Stewart (1996), Sec. 3.4.3(b)] were performed to examine the time variability of the first-mode parameters and to confirm the CEM results. The model order and delay (i.e. d and J) from the CEM were used in these analyses.

4.3 Estimation of Fixed- and Flexible-Base First-Mode Parameters

Analytical procedures were developed in Sec. 3.6 of Stewart (1996) for the estimation of fixed-base first-mode parameters from flexible- and pseudo flexible-base parameters, and the estimation of flexible-base parameters from fixed- and pseudo flexible-base parameters. For the few 'A' sites where all three sets of parameters could be directly evaluated from system identification analyses, estimated and empirical parameters generally compare favorably (i.e. Sec. 5.3.2 of Stewart, 1996). However, an exception was that variations between $\tilde{\zeta}^*$ and estimated $\tilde{\zeta}$ values were occasionally overpredicted, and in such cases, true $\tilde{\zeta}$ values are more reliably estimated by $\tilde{\zeta}^*$.

Conditions necessary for application of the parameter estimation procedures are that $\tilde{f}_1 < \tilde{f}_1^*$ for estimating fixed-base parameters and $\tilde{f}_1^* < f_1$ for estimating flexible-base parameters. Though these criteria were seldom violated, there were several sites where $\tilde{f}_1 < \tilde{f}_1^*$. In such cases, fixed-base parameters were crudely estimated as follow:

- For embedded structures of moderate height where the amount by which $\tilde{f}_1$ exceeds $\tilde{f}_1^*$ is less than about 0.01 Hz, $\tilde{f}_1$ was decreased slightly (0.01 to 0.015 Hz) to correct for the fact that the flexible-base identification was effectively applied to a structure of height H_e whereas the pseudo flexible-base identification was over the full height H . The amount of this correction is based on CEM results for two moderate-height structures with roof/basement and roof/ground-level instrumentation (A33 and A43). With this correction, the fixed-base parameters could be estimated by the standard procedures in Sec. 3.6 of Stewart (1996). Fixed-base parameters were estimated in this way at four sites, A13, A33, A36, and A43.
- For sites where the above $\tilde{f}_1$ correction could not be applied (i.e. non-embedded structures or cases where $\tilde{f}_1 < \tilde{f}_1^*$ by more than 0.015 Hz), the vertical transmissibility functions were examined for evidence of base rocking (as described in Step 4 of Sec. 4.2, above). If rocking was evident, results for the site were discarded since the actual f_1 is likely significantly less than $\tilde{f}_1^*$ (e.g. sites A6, A23, A32). Where base rocking was not evident, the pseudo flexible-base parameters were used as crude estimates of fixed-base parameters (e.g. sites A9, A11, A18, A21, A25, A27-A29, A32, A37, A39, A40, and A42).

With regard to flexible-base parameters, failure to meet the $\tilde{\zeta}_1^* < \zeta_1$ criteria only occurred at site B12 (longitudinal direction). It was not possible to correct for this error, so the results were discarded. The more significant issue with estimated flexible-base parameters was an occasionally excessive difference between pseudo flexible- and flexible-base damping ratios [e.g. sites A31 (Landers), B1, B14]. As noted above, for such cases, flexible-base damping is more accurately estimated by $\tilde{\zeta}^*$.

5.0 Predictive SSI Analysis Procedures

The objective of predictive analyses for inertial SSI effects is to estimate the first-mode period lengthening ratio $\tilde{T}/T$ and foundation damping factor $\tilde{\zeta}_0$. The information required for these analyses consists of geotechnical conditions (Section 2.0), structure and foundation characteristics (Section 3.0), and fixed-base first-mode parameters for the structure calculated or estimated from system identification analyses (Section 4.0). The specific input parameters needed can be summarized as follow:

- Soil conditions: characterization of the site as a halfspace or finite soil layer over rigid base; shear wave velocity V_s and hysteretic damping ratio β which are representative of the site stratigraphy and the level of ground shaking; representative Poisson's ratio ν .
- Structure/Foundation Characteristics: effective height of structure above foundation level, h ; embedment, e ; foundation radii which match the area and moment of inertia of the actual foundation, r_1 and r_2 ; appropriate corrections to the foundation impedance for embedment, shape, and non-rigidity effects.
- Fixed-Base First-Mode Parameters: period and damping ratio, T and ζ .

A detailed description of the predictive analytical methodologies is presented in Chapter 2 of Stewart (1996).

The standard predictive analysis procedure applied to all sites is referred to as the "modified Veletsos" methodology, since it is based primarily on the impedance function solution in Veletsos and Verbic (1973) and the solutions for $\tilde{T}/T$ and $\tilde{\zeta}_0$ in Veletsos and Nair (1975). The "modified" term refers to the extension of Veletsos' basic model of a single degree-of-freedom structure with a rigid circular disk foundation resting on a homogeneous visco-elastic halfspace to account for various factors including non-uniform soil profiles and embedded, non-circular, and non-rigid foundations.

The method by which the "modified Veletsos" formulation accounts for embedment effects consists of crude adjustments to static horizontal and rocking foundation stiffnesses. A more robust simulation of the dynamic impedance of embedded foundations is provided in the solution by Bielak (1975). The soil and structure models are similar to those considered by Veletsos, but the disk foundation is embedded an amount e into the soil. In this study, predictive analyses for embedded structures were performed using both "modified Bielak" and "modified Veletsos" approaches.

6.0 Organization of the Appendices

The site data and analysis results are compiled and presented in a consistent manner for all 58 sites included in these analyses. Appendix A covers sites with both an instrumented structure and at least one free-field accelerograph. Appendix B covers sites with only an instrumented structure with the capability of measuring base rocking. The following information is presented:

- Cover sheet, contains basic site information, peak accelerations, and details about the preprocessing of strong motion data.
- Discussion, description of site conditions and interpretation of analysis results.
- Site Plan, structure and free-field instrument locations ('A' sites only).
- Sensor Locations, building instrumentation and basic structural system.
- Generalized Soil Profile, geologic conditions and shear wave velocity profile.
- Analysis Results, 'A' sites, figures organized as (1) horizontal acceleration time histories, (2) power spectral density functions for selected horizontal motions, (3) acceleration response spectra of foundation and free-field accelerations, (4) nonparametric transmissibility and coherency functions, (5) analysis of vertical motions, and (6) time variations of first-mode parameters. 'B' sites, figures organized as (1) time histories and power spectral density functions, (2) nonparametric transmissibility and coherency functions, (3) time variation of first-mode parameters.

For brevity, a number acronyms were used in the site discussions, as listed in Table 4. Ground motion pairs used in system identification analyses are referred to as output/input, input-output, or input→output. Common shorthand for different motions used in these analyses are FF (free-field), B (basement), 1F (first floor), GF (ground floor), and R (roof).

References are given at the end of each site discussion for data pertinent to that site. General references applicable to multiple sites are given below.

Table 4: Commonly-used acronyms and symbols in the appendices

CEM	cumulative Error Method (parametric sys. ident.)
CSMIP	California Strong Motion Instrumentation Program
e	soil void ratio
$f_i, \tilde{f}_i, \tilde{f}_i^*$	fixed-, flexible-, and pseudo flexible-base angular frequencies for mode i .
f_{NWD}	Nyquist frequency
f_{HP}, f_{LP}	high-pass, low-pass frequency used in data filtering
FF	free-field
h	effective height of single degree-of-freedom structural model, Eq.3
H	denotes halfspace model for calculation of impedance functions
L	longitudinal direction of structure
LCA	LeRoy Crandall Assoc. or Law/Crandall Assoc.
MHA	maximum horizontal acceleration
PI	soil plasticity index
r_1, r_2	radii which match the area and moment of inertia, respectively, of circular disk foundation in impedance function formulation to the actual foundation area and moment of inertia, Eq. 5
RxH	rigid body roof motion due to base rocking
RPEM	recursive Prediction Error Method (parametric sys. ident.)
SPT	standard penetration test
SSI	soil-structure interaction
Str.	structure
SW/AA	Shannon and Wilson/Agbabian Assoc.
S_u	soil undrained shear strength
tr	transverse direction of structure
USGS	United States Geological Survey
V_s	soil small strain shear wave velocity
WCC	Woodward-Clyde Consultants
β	soil hysteretic damping ratio
γ	<i>in geologic log:</i> moist unit weight <i>elsewhere:</i> ratio of structure-to-soil mass, Eq. 4
ν	soil Poisson ratio
$\zeta_i, \tilde{\zeta}_i, \tilde{\zeta}_i^*$	fixed-, flexible-, and pseudo flexible-base damping ratios for mode i .

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SITE A1

Site Name: Eureka 5 Story Apartment Building
Sensors: CSMIP 89494/89509

Lat: 40.802°
Long.: 124.163°

Floor Levels: 5
Bsmt. Levels: 0
Structure: concrete block shear walls

Foundation: footings
Site Geology: Alluvium
Design Date: 1979

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Petrolia	0.15	0.18	0.042	0.15	0.17	0.055	0.27	0.33	--
1992 Petrolia AS 1*	0.07	0.06	0.01	0.08	0.04	0.03	0.14	0.14	--
1992 Petrolia AS 2*	0.06	0.06	0.03	0.04	0.05	0.03	0.13	0.13	--

-- = no instrument

*Not digitized

Preprocessing

Petrolia Earthquake (PT)

Preliminary: *Str. Data*: digitized and processed in this study, $f_{HP} = 0.13-0.3$ Hz, $f_{LP} = 20-30$ Hz, $f_{Nyq} = 100$ Hz. *FF Data*: CSMIP; filtered, $f = 0.16-23$ Hz; $f_{Nyq} = 25$ Hz; FF/str. data not synchronized

Final: FF data decimated by 4 to $f_{Nyq} = 25$ Hz; FF/str. data synchronized using cross correlation of vertical accelerations (Fig. A1.PT0).

Figures

A1.1-A1.3
A1.PT0-6

Site Plan, Sensor Locations, Generalized Soil Column
PT Analysis Results



DISCUSSION

Site Description

The 5-level building is 40.7 feet tall with no basement, and consists of two adjacent rectangular structures connected at a seismic joint. As shown in Fig. A1.2, the two structures both have plan dimensions of 54.7 by 183 feet, and the structural systems consist of concrete block walls in both directions. The foundations consist of 6-foot wide by 4-foot deep continuous spread footings. A 4-inch thick reinforced concrete slab-on-grade is present across the ground floor. As shown on Fig. A1.1, a 6-level elevator shaft is located near the south end of the building. Structural joints separate this concrete block structure from the apartment building, and it is not designed to be part of the building's lateral force resisting system. Two 2-story buildings are located about 40 and 100 feet south of the apartment building. As shown in Fig. A1.2, accelerographs are present on the ground, 2nd, and roof levels of the structure. The free-field instrument is located in a parking lot about 340 feet southwest of the apartment building.

The site is located on a level marine terrace, about 3500 feet from Arcata Bay. Soil conditions at the site were investigated to a depth of 50 feet as part of a foundation investigation by Laco Associates (1979) and a geophysical study by Cooksley Geophysics (1979). Additional subsurface data in similar geologic materials for a location about 4200 feet west of the site is available from a 400-foot deep boring by SW/AA (1980). Based on these sources, the soil profile at the site consists of about 90 feet of Holocene estuarine silty sands with occasional interbedded layers of clayey sand and peat, and sandy soils belonging to the nonmarine Hookton Formation which extend to the maximum depth explored of 350 feet. Soil conditions at greater depth, and the depth to bedrock, are poorly characterized in the area. Based on measurements by Laco Associates, the groundwater table is located at a depth of 15 feet. The shear wave velocities in Fig. A1.3 were measured by Cooksley Geophysics (1979) to 50 feet, and SW/AA (1980) at greater depth.

Instrument Selection

The building consists of two separate (east and west) structures, with ground floor instrumentation only available in the west structure. Sensors from the west end of the west structure were generally used in the analyses, as indicated in Table A1.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A1.PT1.

Table A1.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 13	Ch 2	Ch 1	Ch 9	Ch 4

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A1.PT4) and the spectra of the horizontal motions (Fig. A1.PT2-3) generally indicate minor amplification of NS and EW GF motions relative to the free-field for frequencies less than about 3 to 4 Hz, and minor de-amplification at higher frequencies.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A1.PT5. Amplification is evident near the transverse first-mode frequency, indicating that rocking likely contributed to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A1.PT4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A1.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A1.PT6).

Table A1.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=2, j=2		L (EW) d=0, j=2		tr (NS) d=0, j=2		L (EW) d=0, j=2	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
PT	1	6.21	13.0	4.43	5.1	4.87	11.2	4.32	8.5	4.21	16.7	4.08	12.4

Comments on the analytical results follow:

- Power spectra indicate that the roof motions are dominated by first-mode structural response and the propagation of lower frequency energy (prominent in the free-field and ground floor records) through the structure.
- For the transverse direction, the large difference between the fixed- and pseudo flexible-base parameters is consistent with the rocking effect evident in vertical GF/FF transmissibility functions. There appears to be no significant rocking in the longitudinal direction, as the fixed- and pseudo flexible-base frequencies are similar.
- For both directions, the moderate differences between pseudo flexible- and flexible-base parameters indicate a significant SSI effect associated with base translation.

The RPEM results (Fig. A1.PT6) indicate minor reductions in first-mode frequencies of about 0.6 to 1.0 Hz for both directions at about 8 seconds (the approximate shear wave arrival time). The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and 1F-R first-mode frequencies and damping ratios.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A1.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 57 feet using the recorded free-field motions and the profile in Fig. A1.3. The two structures at the site were assumed to be independent, and the footprint area of just the west structure was used to evaluate foundation radii. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse direction were increased by 20% to account for shape effects.

Table A1.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 800$ fps	$h = 31$ ft	emb.=0	$\gamma = 0.15$
PT	$\nu = 0.45$		tr:	$r_1 = 57$ ft	$r_2 = 42$ ft
	$\beta^* = 5.1\%$	$V_s^* = 700$ fps	L:	$r_1 = 57$ ft	$r_2 = 77$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A1.3 with the fixed-base modal parameters in Table A1.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A1.4.

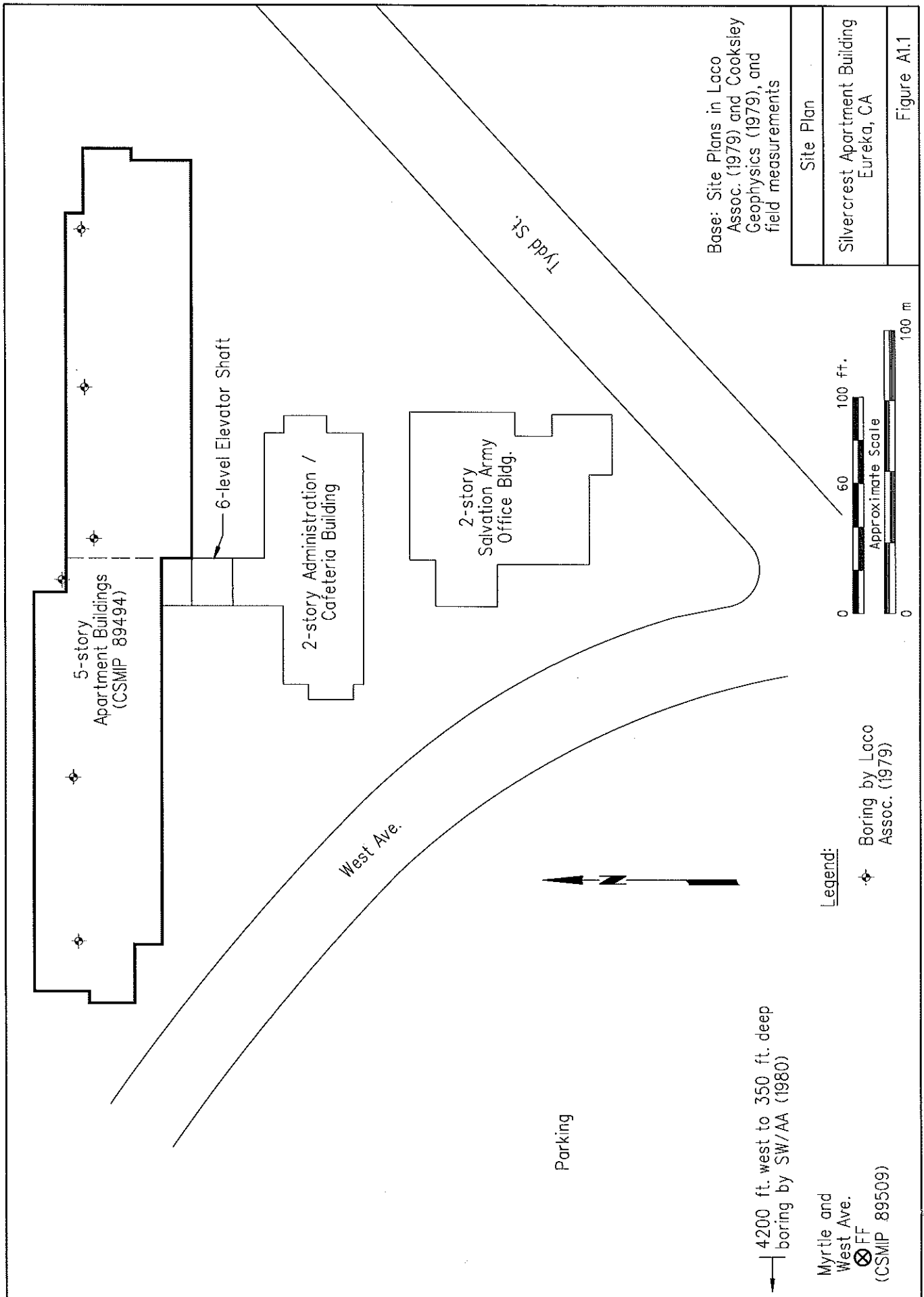
Table A1.4: Empirical and predicted inertial interaction effects

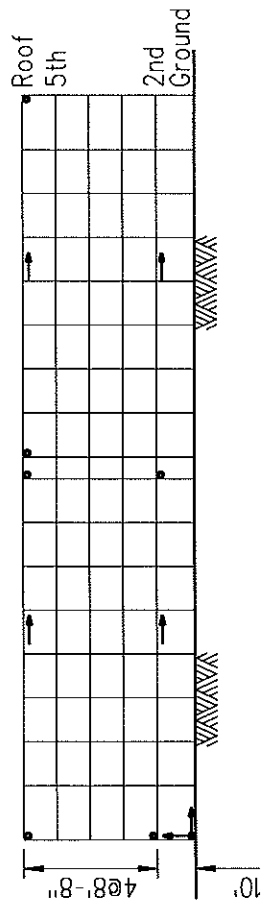
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
PT	Sys. ID	1.47	12.6	16.7	1.08	8.4	12.4
	Prediction	1.45	10.0	14.3	1.16	11.3	14.6

The predicted period lengthening and foundation damping are satisfactory, particularly for the transverse direction. These SSI effects are somewhat overpredicted for the longitudinal direction.

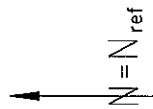
Data Sources

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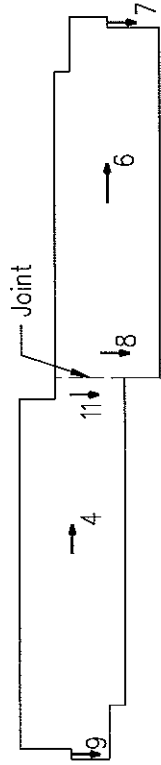




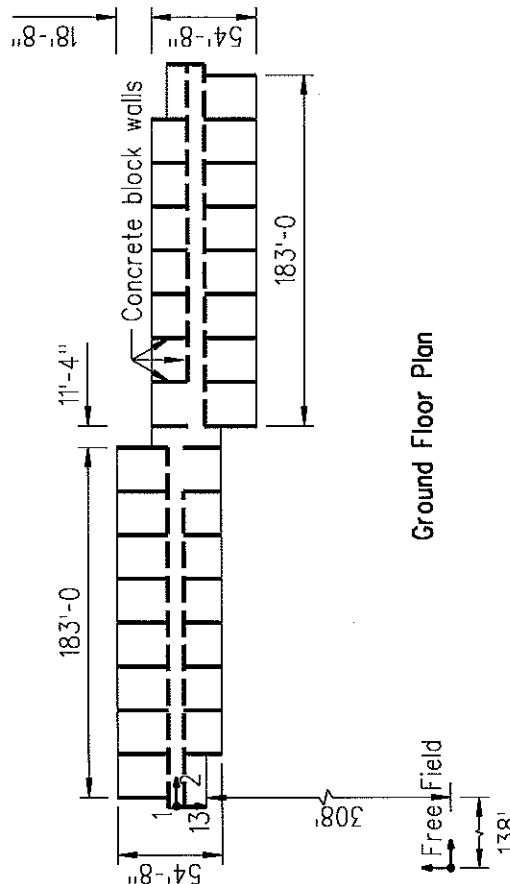
W/E Elevation



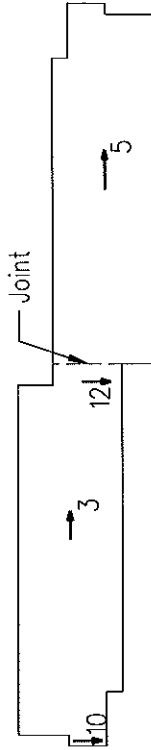
Roof Plan



Ground Floor Plan

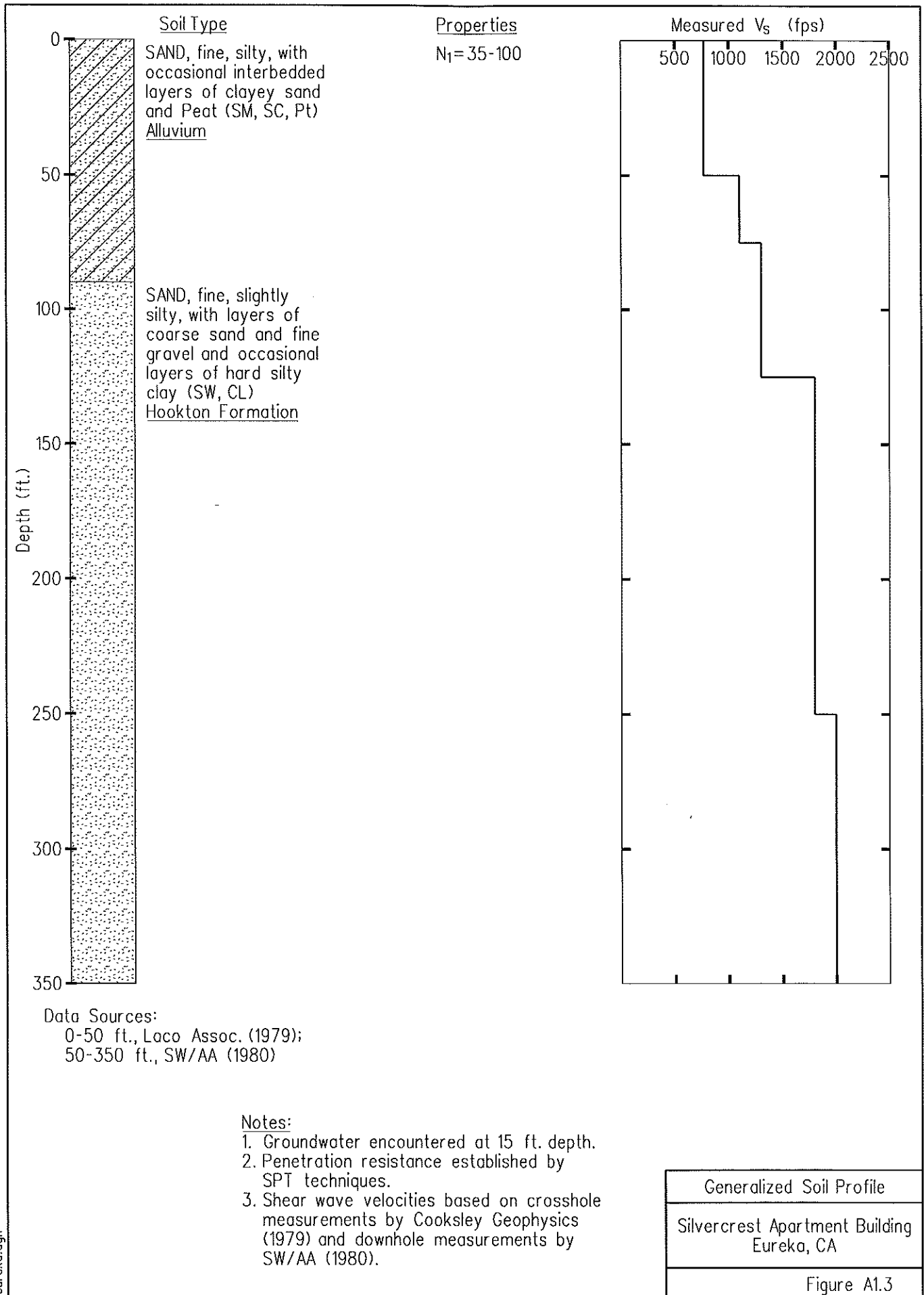


2nd Floor Plan



Sensor Locations
Silvercrest Apartment Building Eureka, CA
Figure A1.2

Note: Data provided by CSMIP
eureka.dgn



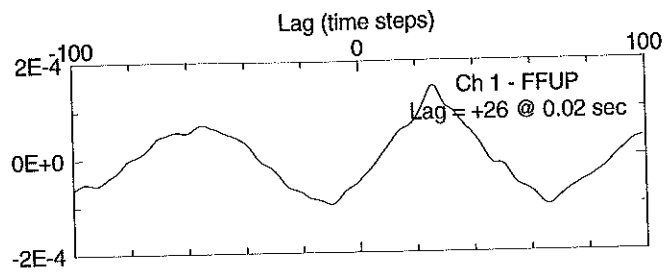


Fig. A1.PT0: Cross-correlation function between vertical ground floor and free field accelerations

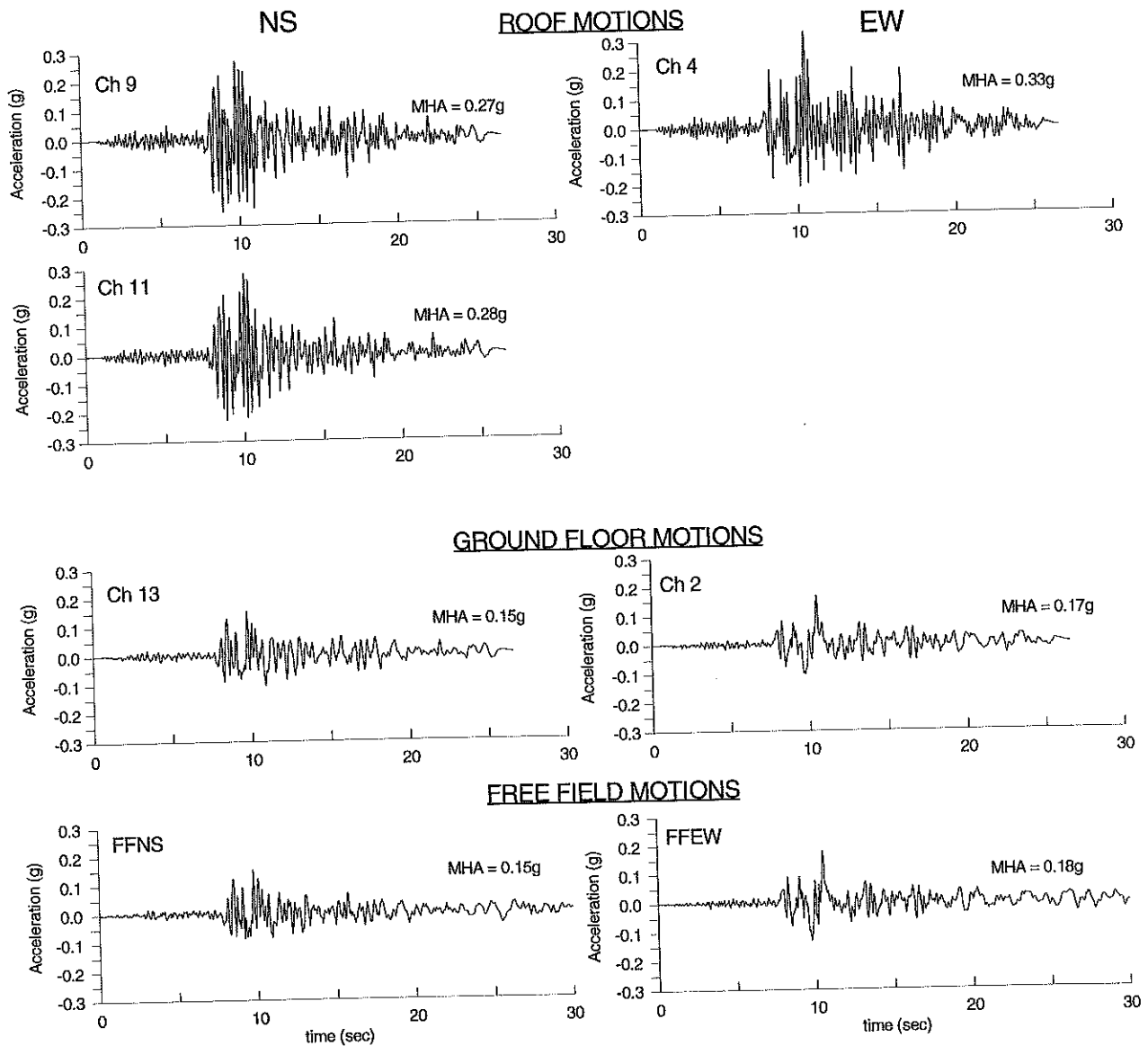


Fig. A1.PT1: EUREKA 5 STORY APARTMENT BUILDING
Horizontal Acceleration Time Histories
1992 Petrolia Earthquake

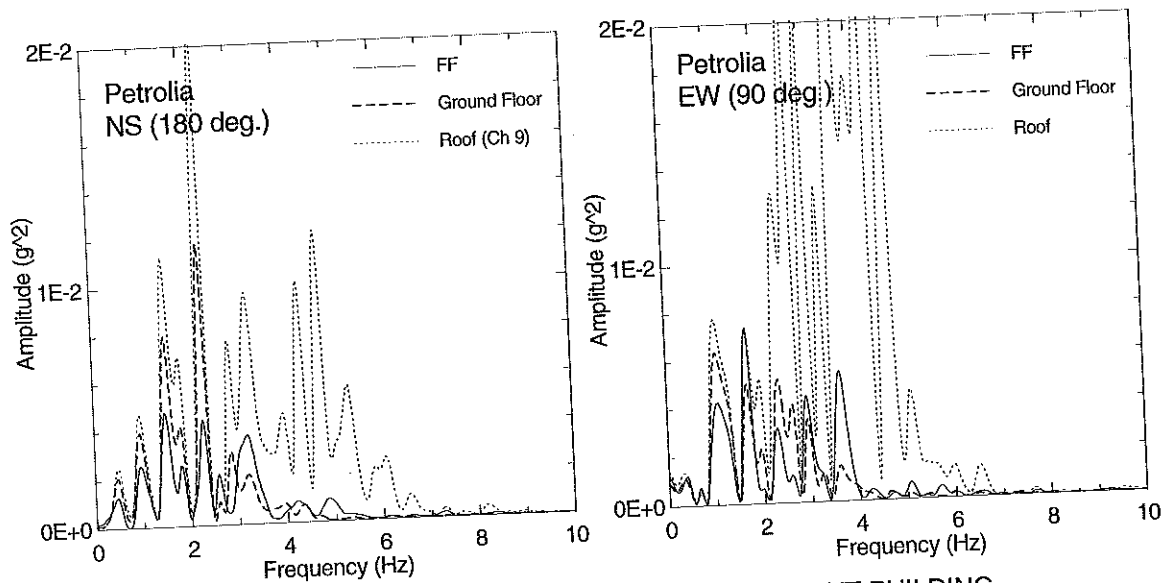


Fig. A1.PT2: EUREKA 5 STORY APARTMENT BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1992 Petrolia Earthquake

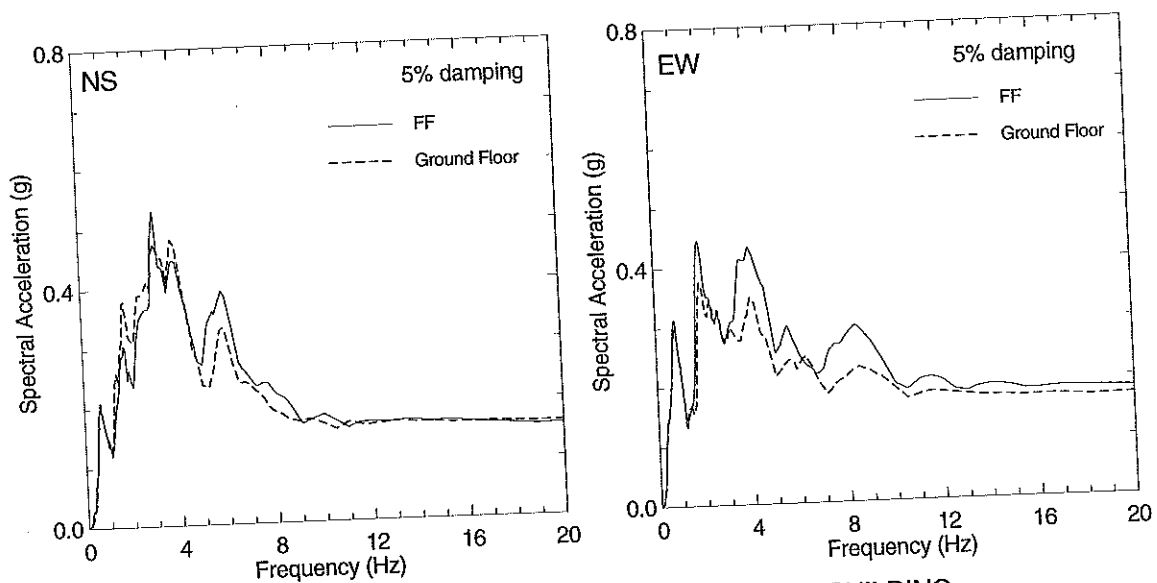


Fig. A1.PT3: EUREKA 5 STORY APARTMENT BUILDING
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1992 Petrolia Earthquake

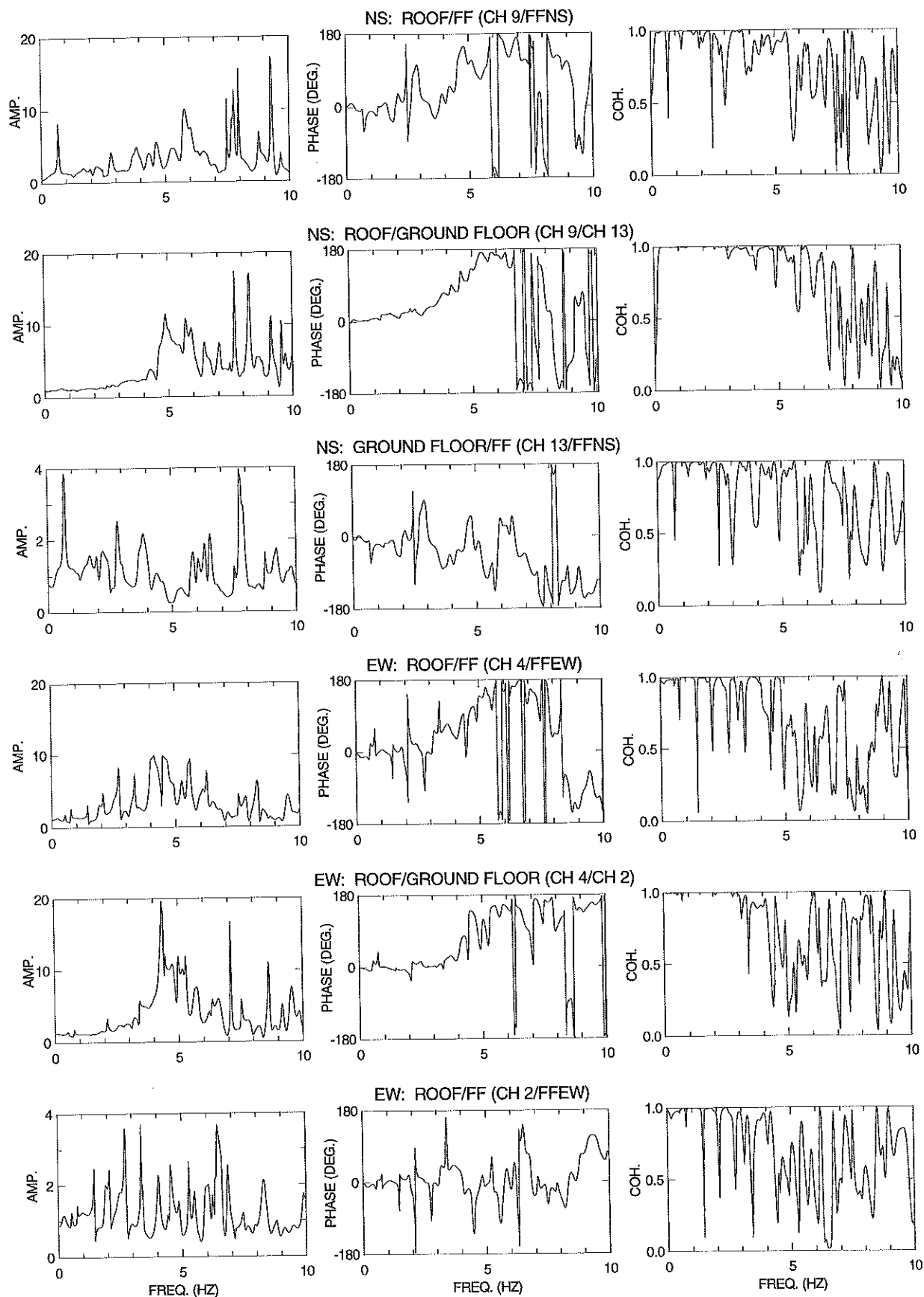


Fig. A1.PT4: EUREKA 5 STORY APARTMENT BUILDING
Transmissibility and Coherency Functions
1992 Petrolia Earthquake
A1-10

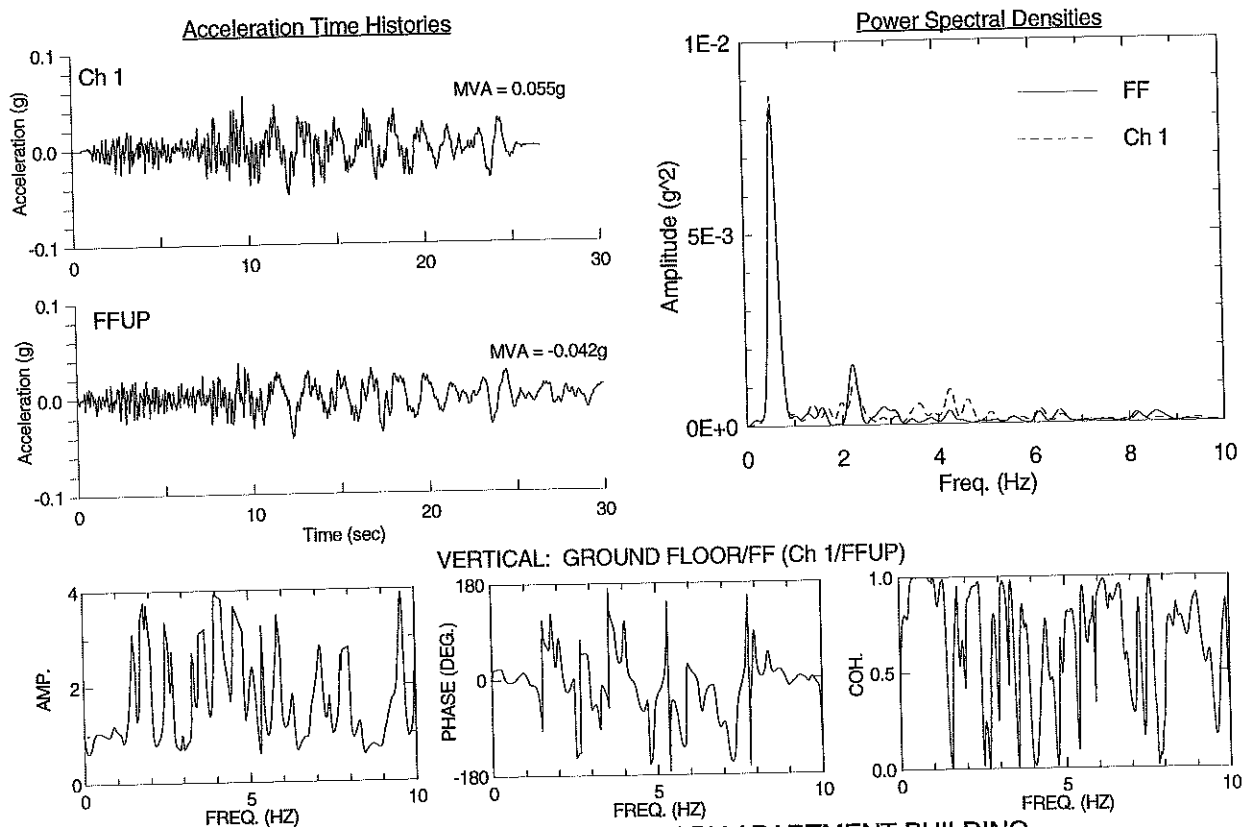


Fig. A1.PT5: EUREKA 5 STORY APARTMENT BUILDING
Analysis of Vertical Motions
1992 Petrolia Earthquake

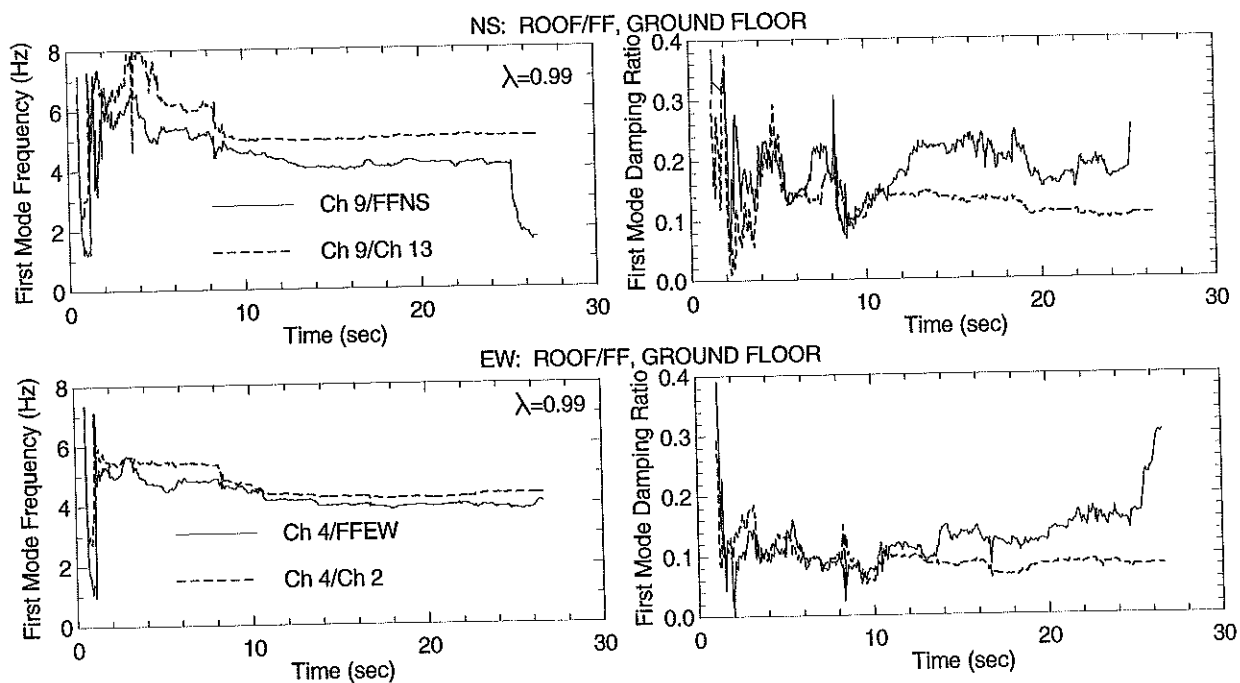


Fig. A1.PT6: EUREKA 5 STORY APARTMENT BUILDING
Time Variation of First Mode Parameters
1992 Petrolia Earthquake

SITE A2

Site Name: Fortuna Supermarket
Sensors: CSMIP 89473/89486

Lat: 40.584°
Long.: 124.145°

Floor Levels: 1
Bsmt. Levels: 0
Structure: plywood roof diaphragm and
concrete block shear walls

Foundation: footings
Site Geology: Alluvium
Design Date: 1979

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Petrolia	0.12	0.11	0.049	0.14	0.12	0.054	0.29	0.14	--
1992 Petrolia AS 1	0.19	0.18	0.042	0.16	0.17	0.047	0.68	0.30	--
1992 Petrolia AS 2*	0.15	0.18	0.07	0.14	0.19	0.07	0.20	0.20	--

*Not digitized

-- = No instrument

Preprocessing

Petrolia Earthquake (PT)

Preliminary: CSMIP; filtered, $f = 0.4\text{--}23\text{ Hz}$ (str.), $0.07\text{--}23\text{ Hz}$ (FF); $f_{\text{Nyq}} = 25\text{ Hz}$; FF/str. data synchronized

Final: none

Petrolia Aftershock 1 (PTA)

Preliminary: CSMIP; filtered, $f = 0.4\text{--}23\text{ Hz}$ (str. + FF); $f_{\text{Nyq}} = 25\text{ Hz}$; synchronized

Final: none

Figures

A2.1-A2.3

Site Plan, Sensor Locations, Generalized Soil Column

A2.PT1-6

PT Analysis Results

A2.PTA1-5

PTA Analysis Results



View of structure with FF in foreground

DISCUSSION

Site Description

The single-level building is 22 feet tall with no basement, and is nearly rectangular in plan view with dimensions 222 by 202 feet. The building has a plywood diaphragm roof supported on perimeter concrete block shear walls which resist lateral loads. The foundation consists of spread footings, and a 4-inch thick slab-on-grade is present across the base of the building. Single level stores are present immediately south of the building. As shown in Fig. A2.2, accelerographs are present on the ground floor and roof of the structure. The free-field instrument is located in a parking lot about 345 feet west of the structure.

The site is located in Fortuna on level terrain about 3500 to 4000 feet east of the Eel River. Relatively little subsurface data is available for the site, so the soil conditions in Fig. A2.3 were estimated using trench logs for a nearby restaurant (Foundation Engineering Company, 1979) and borings for a nearby highway bridge (Caltrans, 1960). Based on these sources, soil conditions at the site appear to consist of about 25 to 35 feet of surficial clayey and sandy alluvium underlain by denser sandy alluvial soils. Based on depth-to-groundwater measurements by Foundation Engineering Company, the soils at the site are assumed to be saturated below a depth of 3 feet. The shear wave velocities in Fig. A2.3 were estimated to a depth of 45 feet using correlations with penetration resistance. Soil conditions at greater depths and the depth to bedrock are poorly characterized in the area.

Instrument Selection

Instrumentation was selected to enable the dynamic behavior of the concrete block walls to be assessed both parallel and transverse to the wall lines. The sensors used to evaluate this behavior are indicated in Table A2.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A2.**1.

Table A2.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (par. to wall)		Roof (perp. to wall)	
NS	EW	V	NS	EW	NS	EW
Ch 12	Ch 5	Ch 1	Ch 7	Ch 2	Ch 9	Ch 3

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A2.**4) and the spectra of the horizontal motions (Fig. A2.**2-3) generally indicate minor amplification of both NS and EW motions for frequencies greater than about 1 Hz.

Transmissibility, coherency and power spectral density functions for FF and GF vertical motions are presented in Fig. A2.**5. The transmissibility function amplitude results are fairly spiky, but there is no evidence of amplification at the modal frequencies for lateral response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A2.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A2.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A2.PT6).

Table A2.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) PT: d=1, j=5 PTA: d=0, j=5		L (NS) PT: d=0, j=5 PTA: d=4, j=5		tr (EW) PT: d=2, j=5 PTA: d=1, j=5		L (NS) PT: d=0, j=5 PTA: d=4, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
PT	1	2.83	22.3	3.35	19.2	2.81	22.8	3.31	22.1	2.60	30.1	3.02	32.7
PTA	1	3.07	21.9	3.59	16.1	3.06	21.4	3.58	20.4	3.04	18.1	3.50	27.3

Comments on the analytical results follow:

- The shear walls have a large in-plane stiffness. This is illustrated in R/GF transmissibility functions (Ch2/Ch5, Ch7/Ch12) by amplitudes and coherencies close to unity and nearly zero phase shift from 0 to 10 Hz. Out-of-plane roof sensors were used for parametric analyses, so the results reflect the flexibility of the plywood roof diaphragm.
- Though a value of $j = 5$ was used to develop parametric models of the structure, the first-mode was dominant, and only these parameters are reported in Table A2.2. As indicated by the broad transmissibility function amplitude peaks, modal frequencies are poorly defined for this structure, and the values reported in Table A2.2 are approximate.
- The small variations between the fixed- and pseudo flexible-base modal parameters are consistent with the lack of evidence for a rocking response in the vertical GF/FF transmissibility functions.
- The structure had a softer and more highly damped response to the main shock of the Petrolia Earthquake than the aftershock, despite higher MHAs in the aftershock. This results from the main shock motions having a longer duration and greater low-frequency energy content.

Time-dependent reductions in the first-mode frequency were apparent in the first 10 seconds of shaking in both the NS and EW directions from the results of RPEM analyses for the PT (Fig. A2.PT6). The results of RPEM analyses were generally consistent with those from CEM analyses with regard to the relative magnitudes of FF-R and GF-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A2.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 115 feet using the recorded free-field motions and the profile in Fig. A2.3. The structure was assumed to be independent of adjoining one-story buildings to the south for the computation of foundation radii. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A2.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 915$ fps	$h = 22$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.45$		tr: $r_1 = 115$ ft		$r_2 = 112$ ft
PT	$\beta^* = 5.6\%$	$V_s^* = 770$ fps	L: $r_1 = 115$ ft		$r_2 = 122$ ft
PTA	$\beta^* = 5.8\%$	$V_s^* = 790$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A2.3 with the fixed-base modal parameters in Table A2.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A2.4.

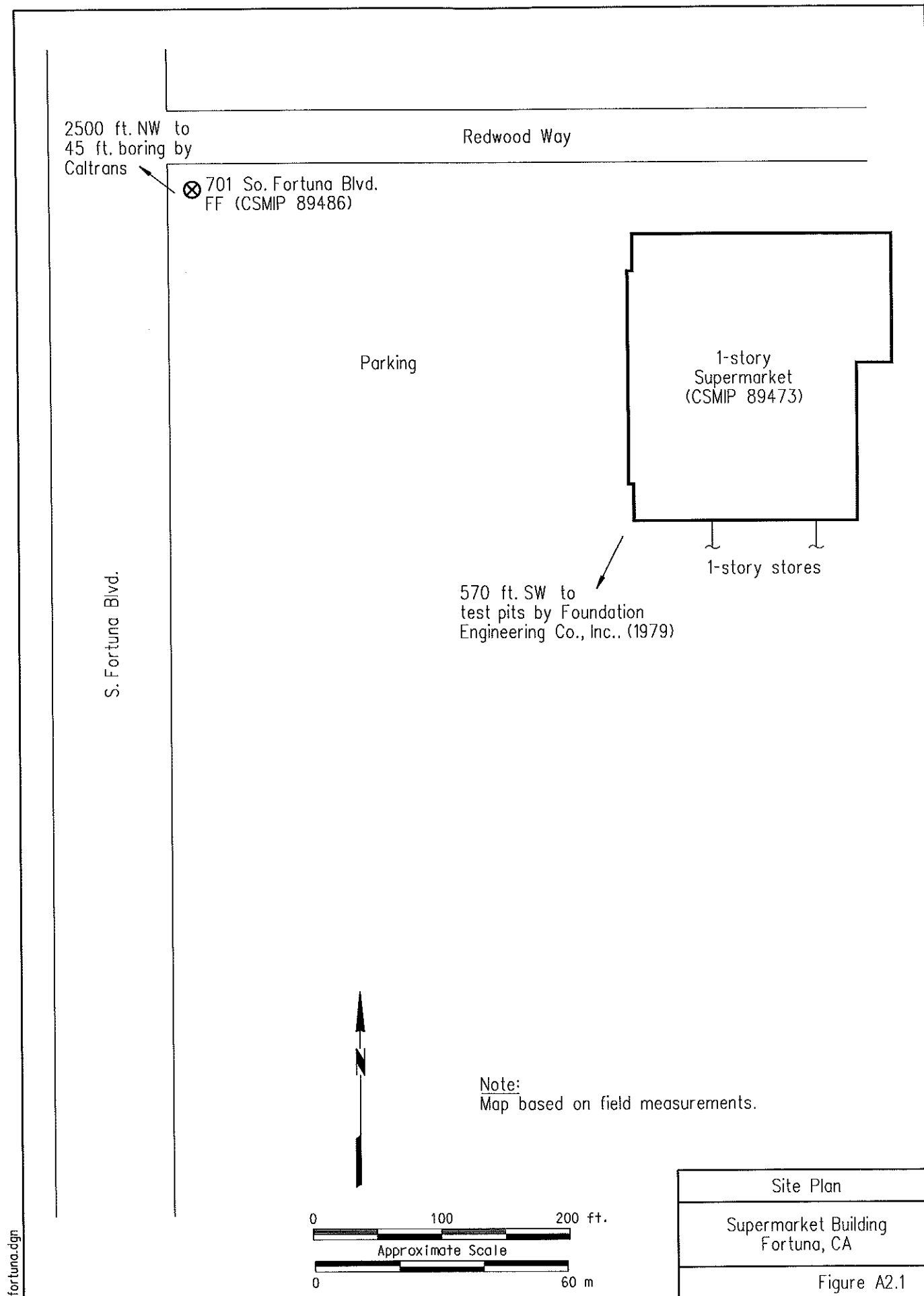
Table A2.4: Empirical and predicted inertial interaction effects

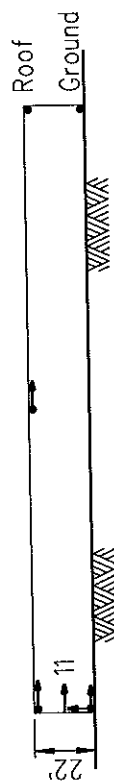
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
PT	Sys. ID	1.09	12.8	30.1	1.11	18.6	32.7
	Prediction	1.07	4.9	23.2	1.10	7.0	21.6
PTA	Sys. ID	1.01	-3.2	18.1	1.03	12.5	27.3
	Prediction	1.08	5.6	23.1	1.11	7.8	19.6

The predicted period lengthening for the Petrolia mainshock is satisfactory, though the foundation damping was underpredicted. The agreement is relatively poor for PTA, which likely results from poor CEM results due to the short-duration of shaking. Negative foundation damping for PTA in the transverse direction indicates a numerical error in the results.

Data Sources

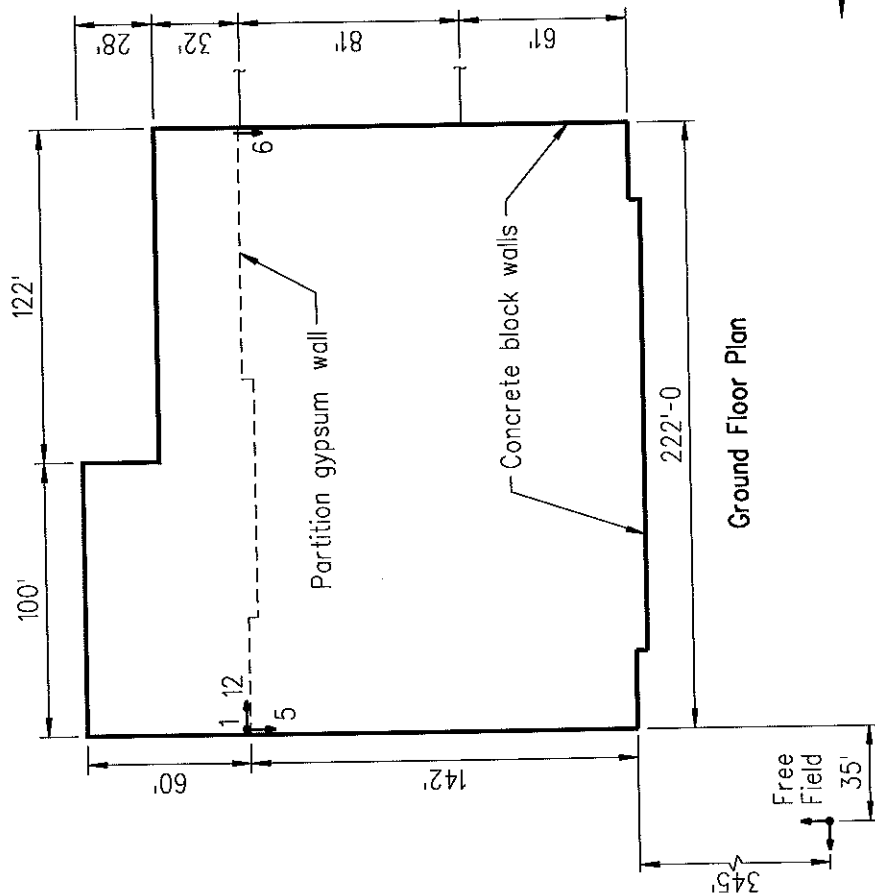
Caltrans (1960). Borings for Highway 101 overcrossing at 12th Street, Fortuna, CA.
Foundation Engineering Co., Inc. (1979). "Foundation Investigation at 901 Fortuna Blvd., Fortuna, CA."



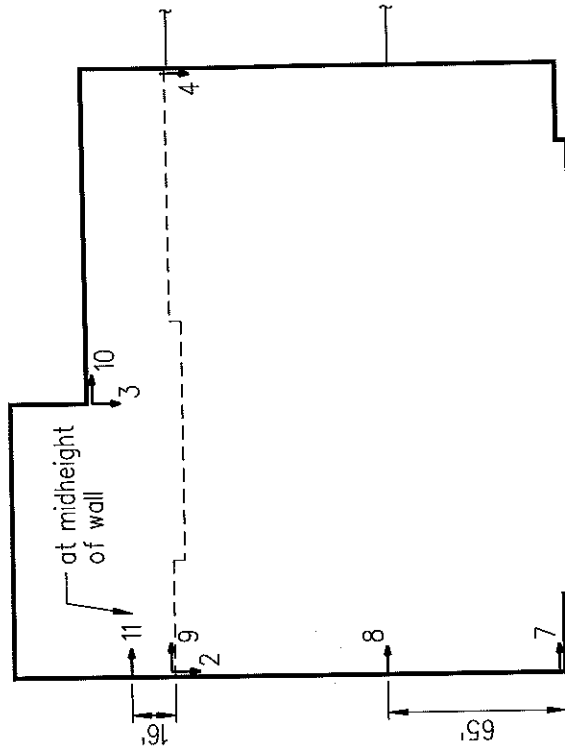


N / S Elevation

Sensor 3 mounted on glu-lam beam.
Other sensors mounted on walls.



Ground Floor Plan

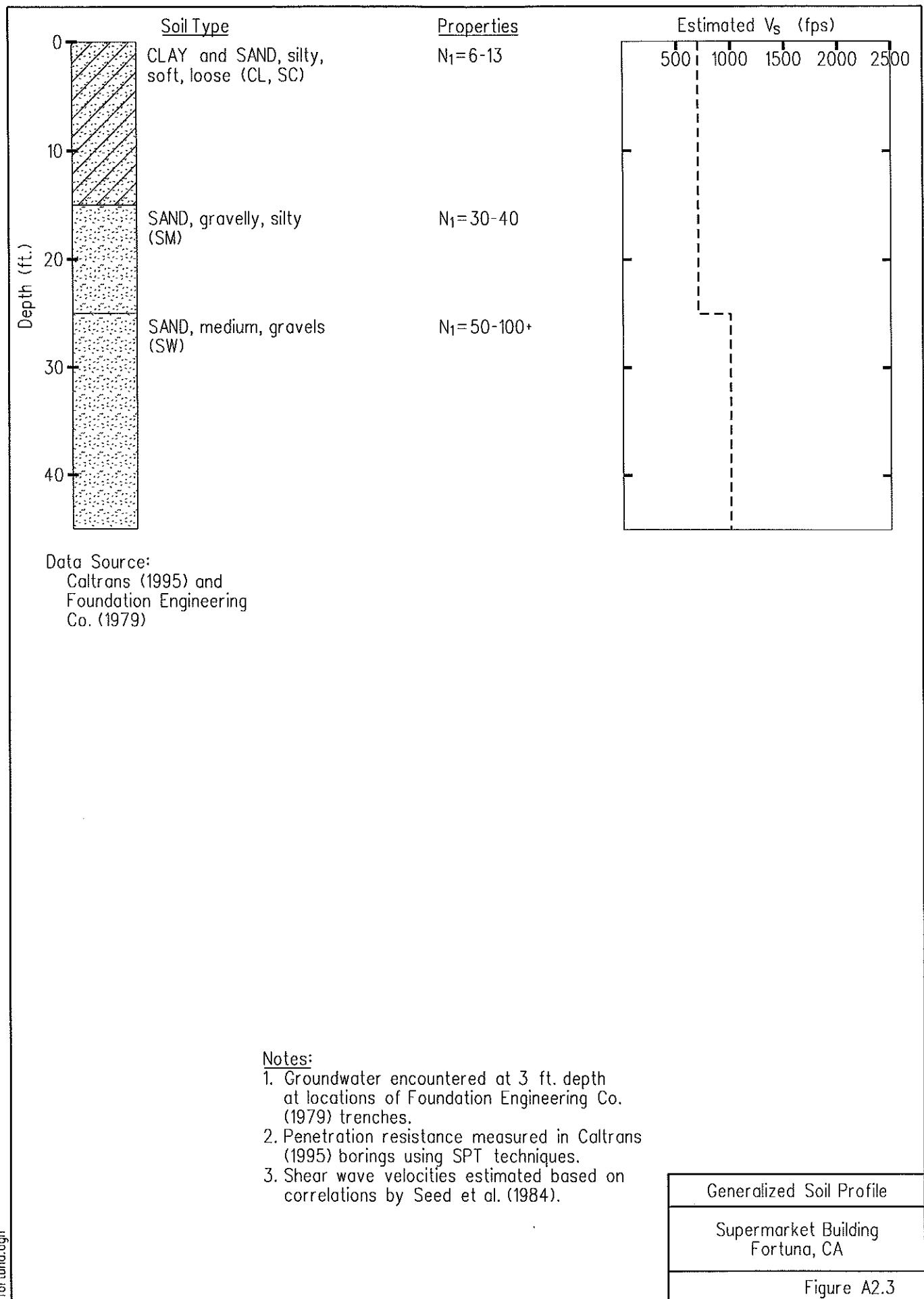


Roof Plan

Sensor Locations
Supermarket Building Fortuna, CA
Figure A2.2

Note: Data provided by CSMIP

fortuna.dgn



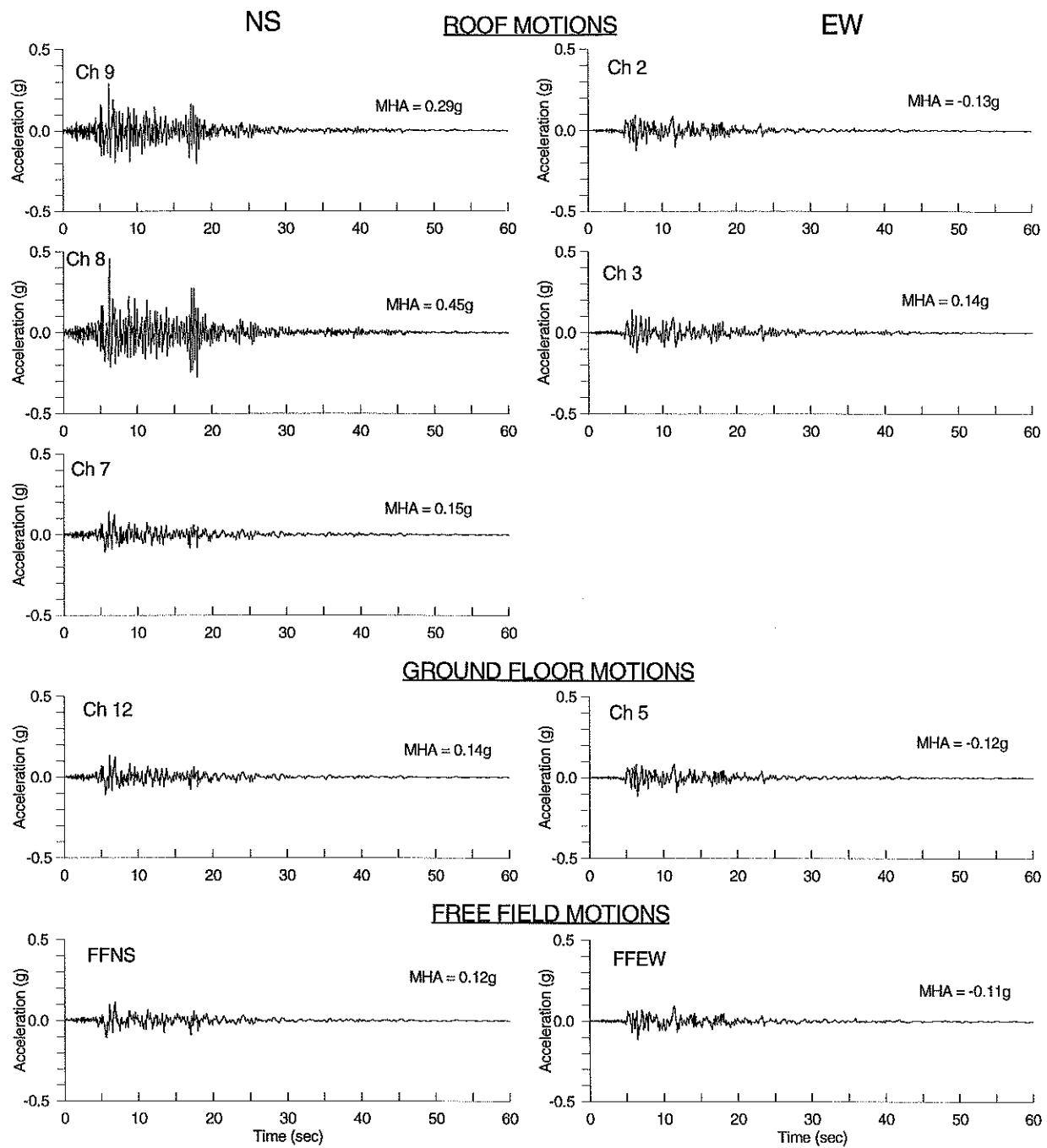


Fig. A2.PT1: FORTUNA SUPERMARKET
Horizontal Acceleration Time Histories
1992 Petrolia Earthquake

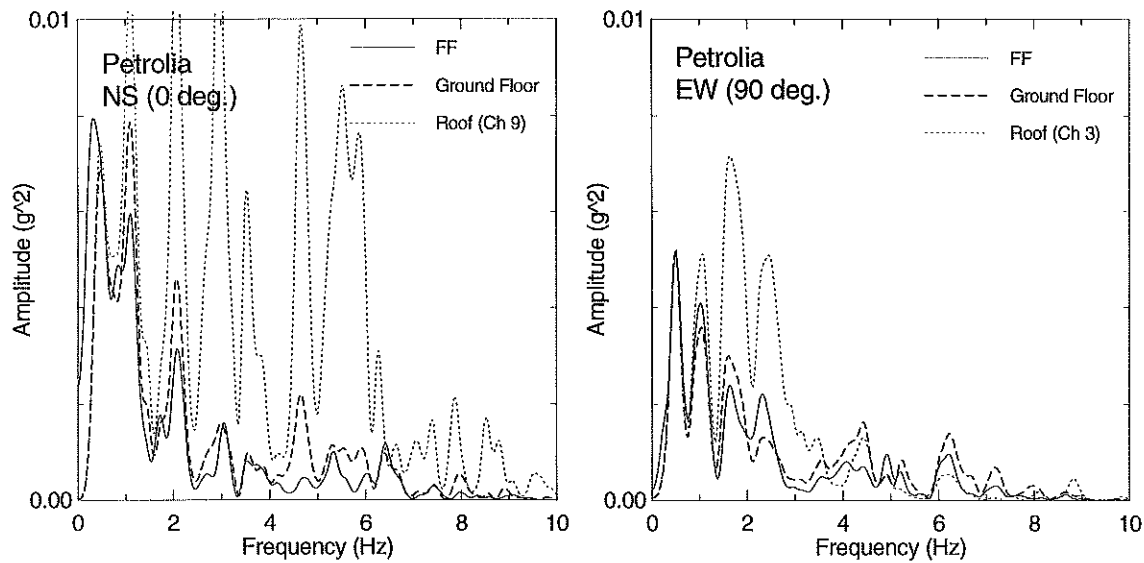


Fig. A2.PT2: FORTUNA SUPERMARKET
Power Spectral Density of Selected Horizontal Accelerations
1992 Petrolia Earthquake

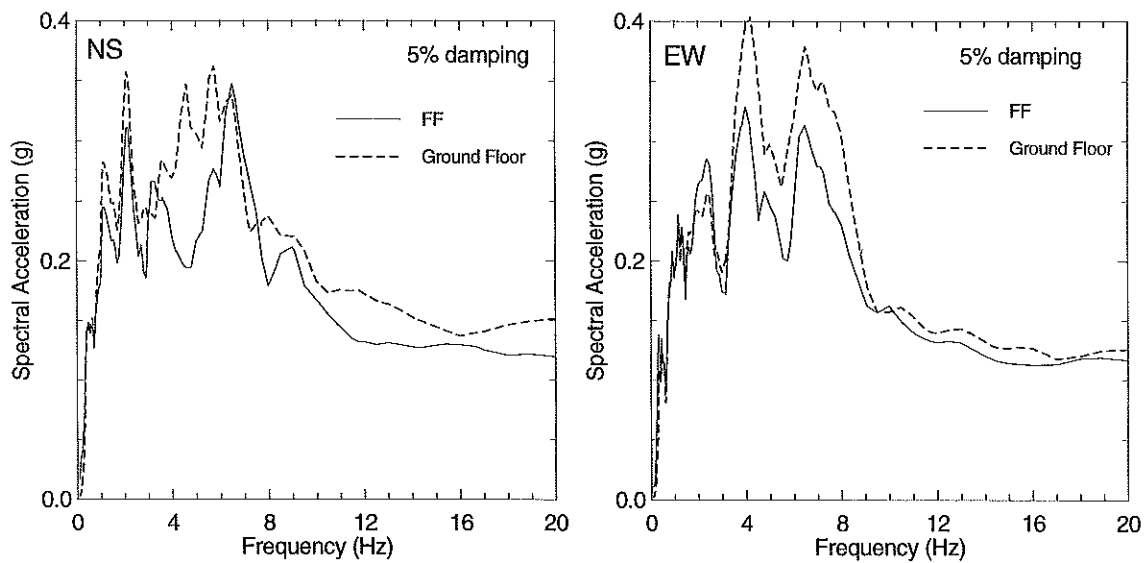


Fig. A2.PT3: FORTUNA SUPERMARKET
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1992 Petrolia Earthquake

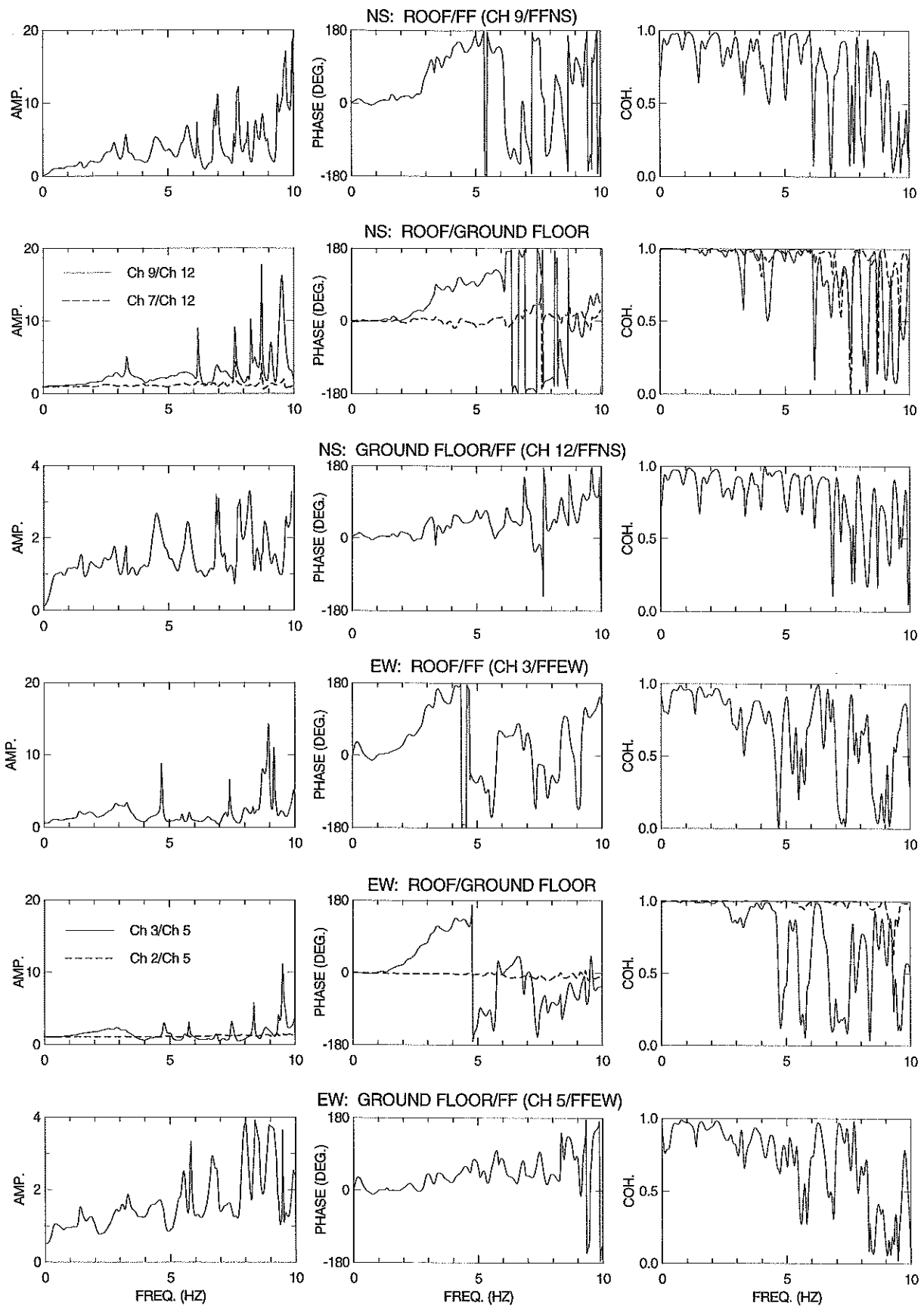


Fig. A2.PT4: FORTUNA SUPERMARKET
Transmissibility and Coherency Functions
1992 Petrolia Earthquake
A2-10

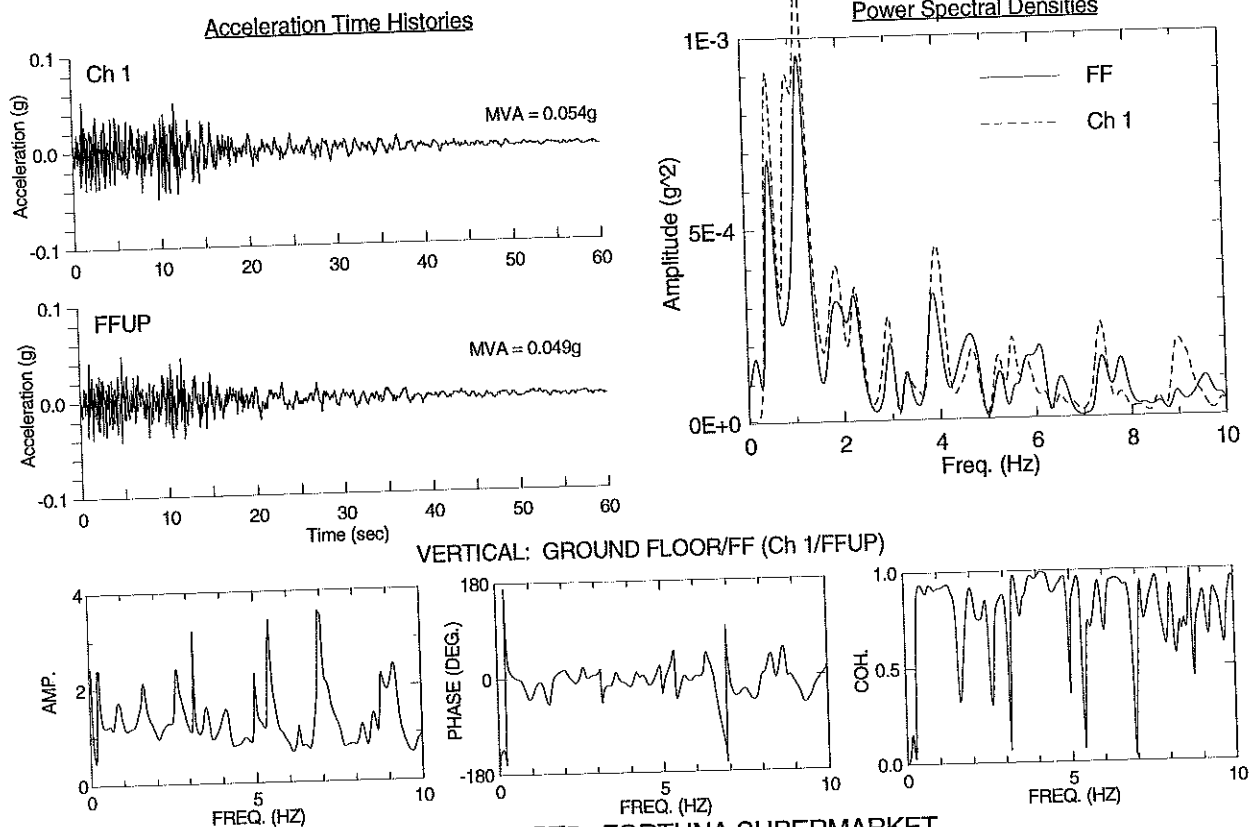


Fig. A2.PT5: FORTUNA SUPERMARKET
Analysis of Vertical Motions
1992 Petrolia Earthquake

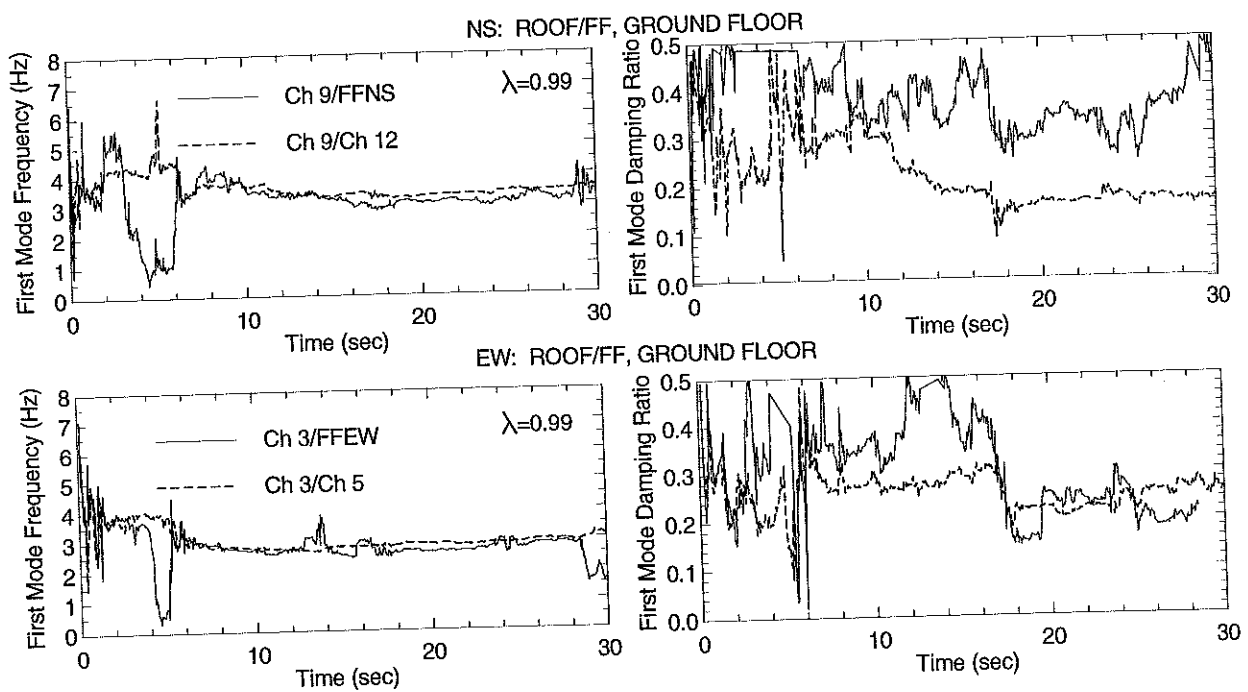


Fig. A2.PT6: FORTUNA SUPERMARKET
Time Variation of First Mode Parameters
1992 Petrolia Earthquake

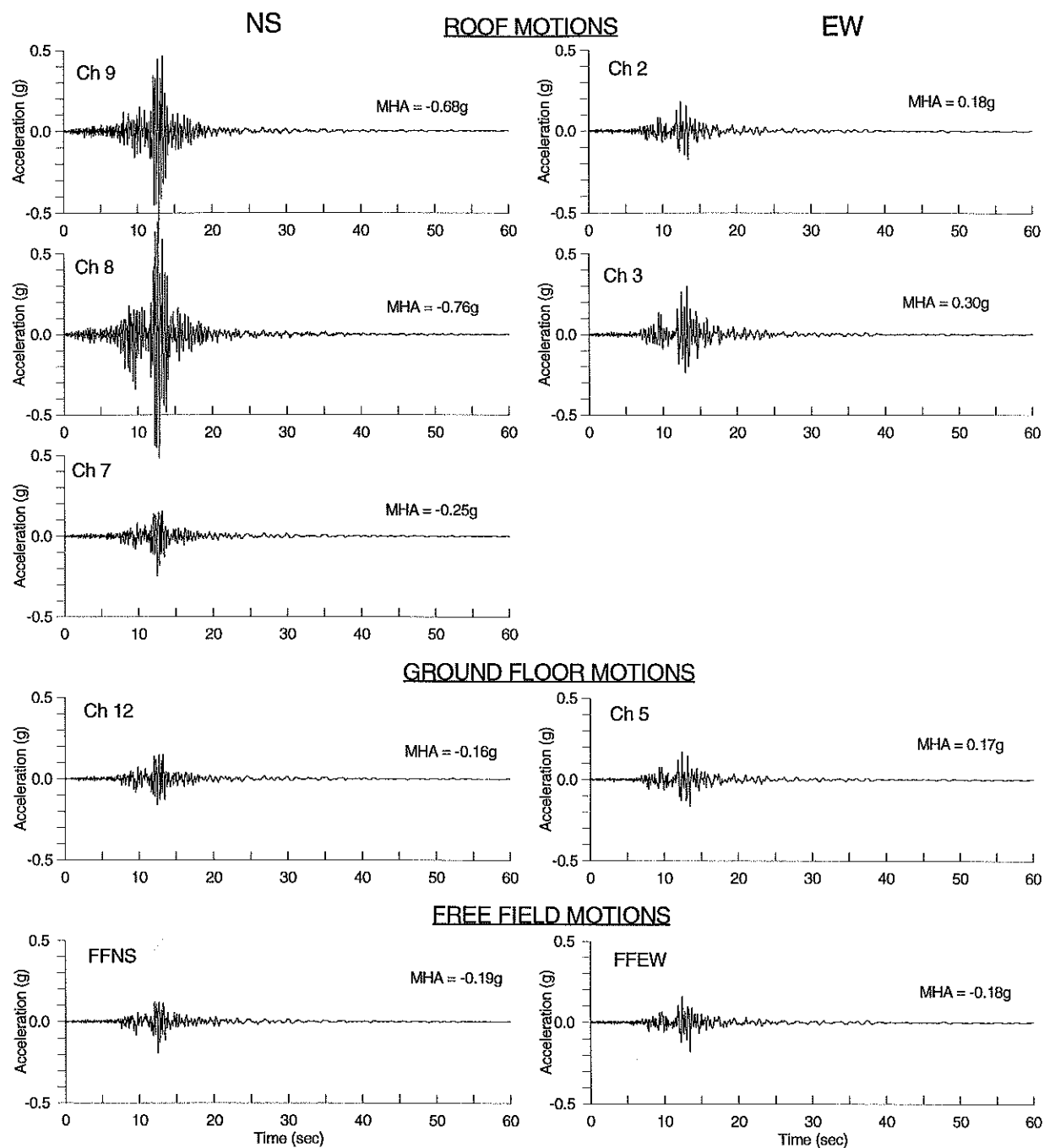


Fig. A2.PTA1: FORTUNA SUPERMARKET
Horizontal Acceleration Time Histories
1992 Petrolia Earthquake Aftershock I

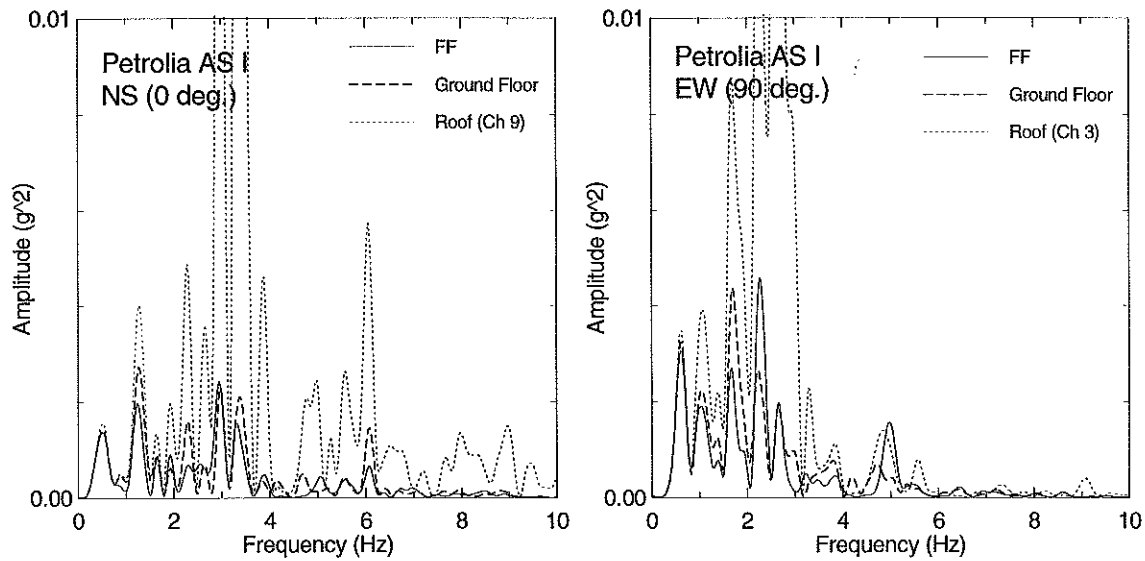


Fig. A2.PTA2: FORTUNA SUPERMARKET
Power Spectral Density of Selected Horizontal Accelerations
1992 Petrolia Earthquake Aftershock I

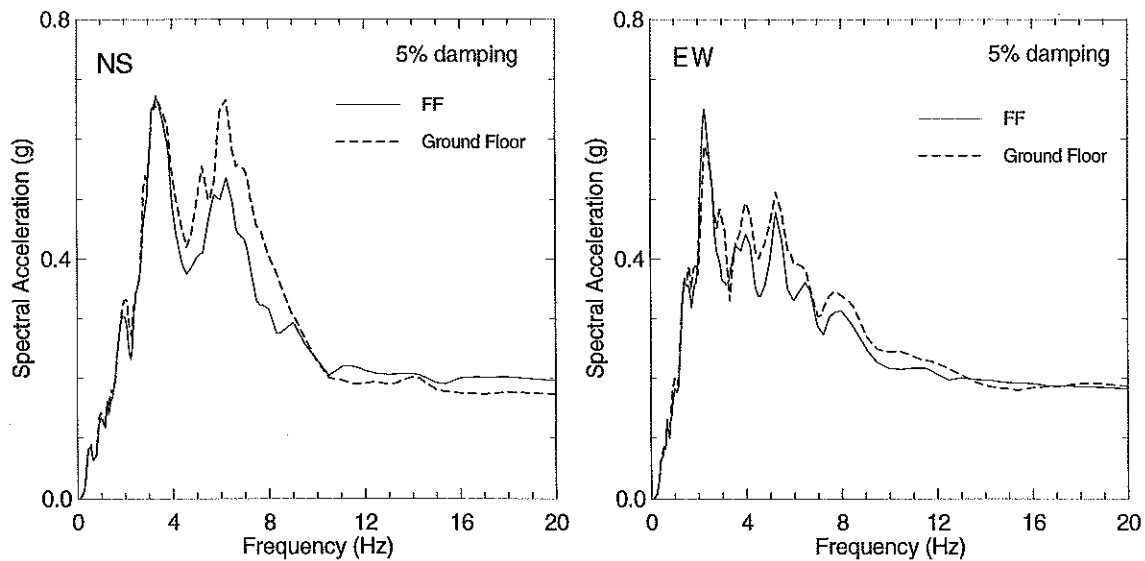


Fig. A2.PTA3: FORTUNA SUPERMARKET
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1992 Petrolia Earthquake Aftershock I

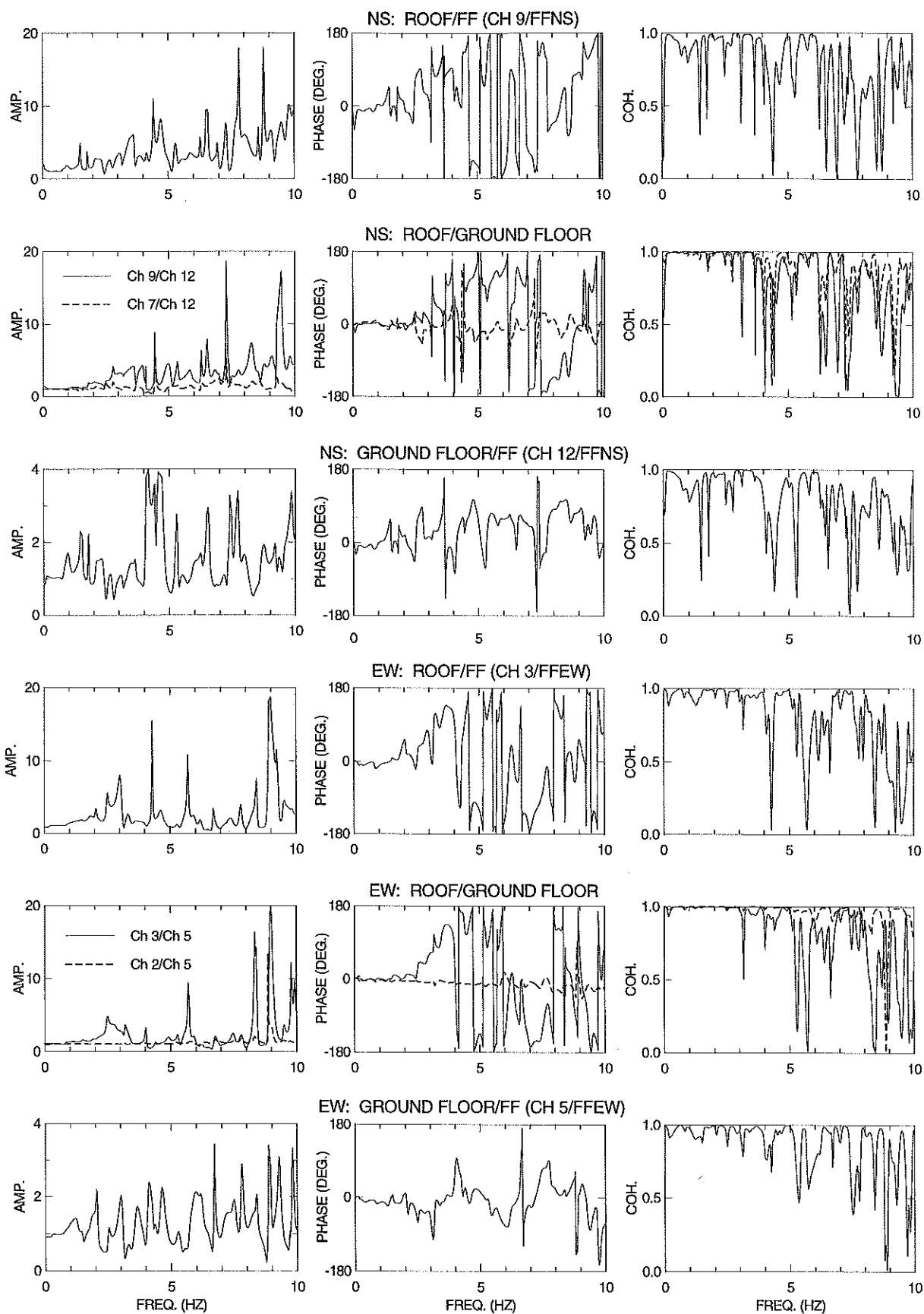


Fig. A2.PTA4: FORTUNA SUPERMARKET
Transmissibility and Coherency Functions
1992 Petrolia Earthquake Aftershock I
A2-14

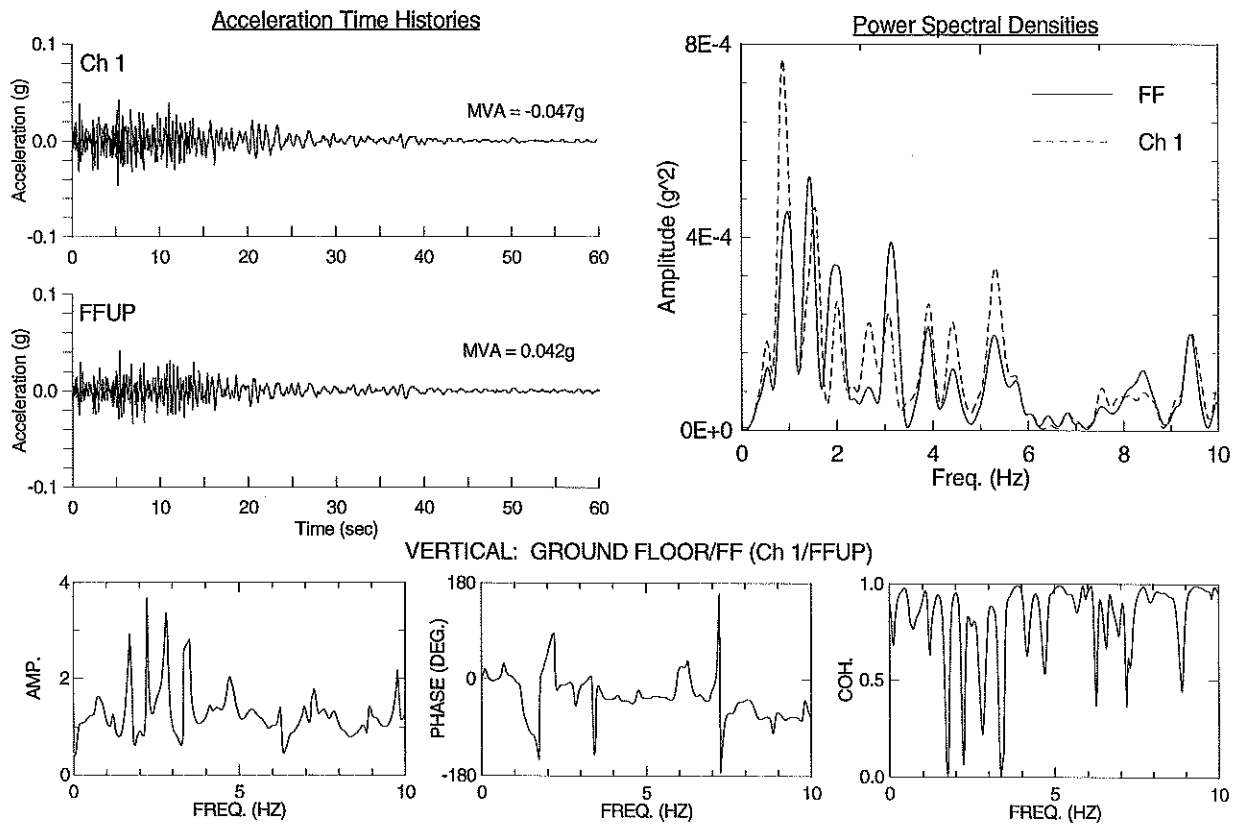


Fig. A2.PTA5: FORTUNA SUPERMARKET
Analysis of Vertical Motions
1992 Petrolia Earthquake Aftershock I

SITE A3

Site Name: Humboldt Bay Power Plant	Lat: 40.741°
Sensors: Pacific Gas and Electric Co. (PG&E)	Long.: 124.209°
Floor Levels: 2 above ground	Foundation: footings
Bsmt. Levels: see text	Site Geology: Alluvium
Structure: 86-ft. deep concrete caisson	Const. Date: 1963

Recorded Peak Accelerations (g)

	Free-Field (+12 ft.)			Caisson (-66 ft.)			Refuel Building (Plant Grade +12 ft.)		
	NS	EW	V	NS	EW	V	NS	EW	V
1975 Ferndale	0.25	0.18	0.028	0.11	0.083	0.067	0.23	0.18	0.10
1992 Petrolia	0.27	0.24	0.076	0.14	0.13	0.11	0.22	0.23	0.067

Preprocessing

Ferndale Earthquake (FN)

Preliminary: digitized (this study) at $f_{Nyq} = 100$ Hz (reliable from 0.4 to 10 Hz only)

Final: data decimated by 2 to $f_{Nyq} = 50$ Hz; data from 3 triaxial accelerographs synchronized

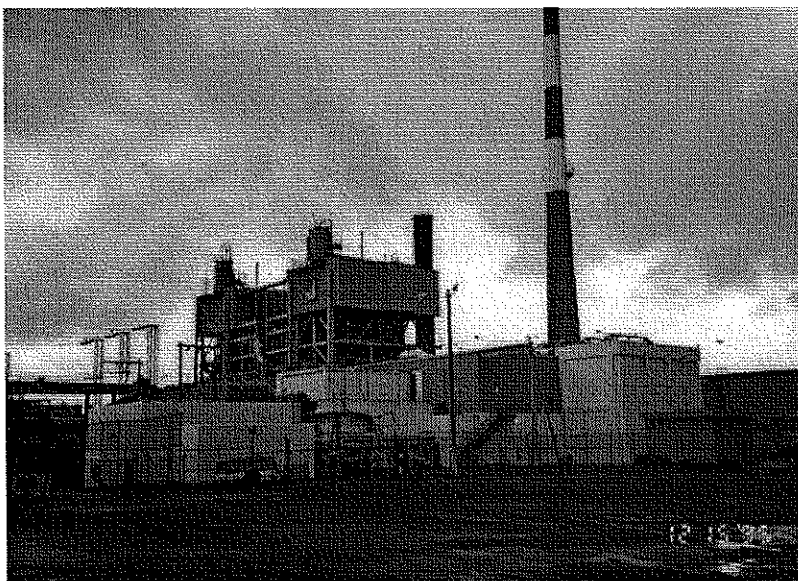
Petrolia Earthquake (PT)

Preliminary: digitized data obtained from PGE, $f_{Nyq} = 100$ Hz; FF/str. data not synchronized

Final: data decimated by 2 to $f_{Nyq} = 50$ Hz and high pass filtered at $f_{HP} = 0.17$ Hz; no synchronization due to small lags.

Figures

A3.1-A3.3	Site Plan, Sensor Locations, Generalized Soil Column
A3.FN1-4	FN Analysis Results
A3.PT1-4	PT Analysis Results



View of power plant, looking west

DISCUSSION

Site Description

Units 1 and 2 at the plant are for processing fossil fuel, while Unit 3 is an inactive nuclear power generation facility. The surface structures at the plant are generally fairly light concrete block or steel/concrete frame buildings. The nuclear power generation unit is located below the refueling building in an 86-foot deep concrete caisson. This caisson structure is instrumented with triaxial accelerographs atop the base slab (-66 feet elevation) and at the plant grade level (+12 feet). The roof of the refueling building over the caisson was not instrumented until 1994 and hence data from the FN and PT is unavailable. Prior to November 1987, the free field instrument was located in a single story, 28 by 65 foot storage building with a concrete slab-on-grade floor located about 290 feet south of the refueling building. Since November 1987, the free-field instrument has been located in an open area about 240 feet north of refueling building.

The site is located on level terrain near the eastern margin of Humboldt Bay on alluvial sediments. Soil conditions at the site were investigated by Dames and Moore (1959) prior to the original construction of the plant and by WCC (1980) during a seismic re-evaluation of the plant site. Based on these sources, the soil profile at the site (Fig. A3.2) consists of: 25 feet of soft silty clay, 27 feet of gravely sands, 18 feet of silty clay, 12 feet of interbedded silty clays and sand, 88 feet of sand and gravel, 160 feet of silty clay, and 20 feet of sand which extends to the maximum depth explored of 350 feet. Based on measurements by PG&E (1985), the depth-to-groundwater is about 6 feet. The shear wave velocities shown in Fig. A3.3 were measured by crosshole and downhole in situ testing by WCC (1980).

Instrument Selection and Free-Field Conditions

The instrumentation of the caisson during the FN and PT was limited to triaxial accelerographs near the base of the caisson and at the plant grade level. Horizontal acceleration time histories from these sensors are shown in Fig. A3.**1. The FN data was recorded on analogue equipment, and was digitized for this study from accelerograms in Dames and Moore (1975). The PT data was recorded by digital accelerometers and is more reliable.

Analysis Results

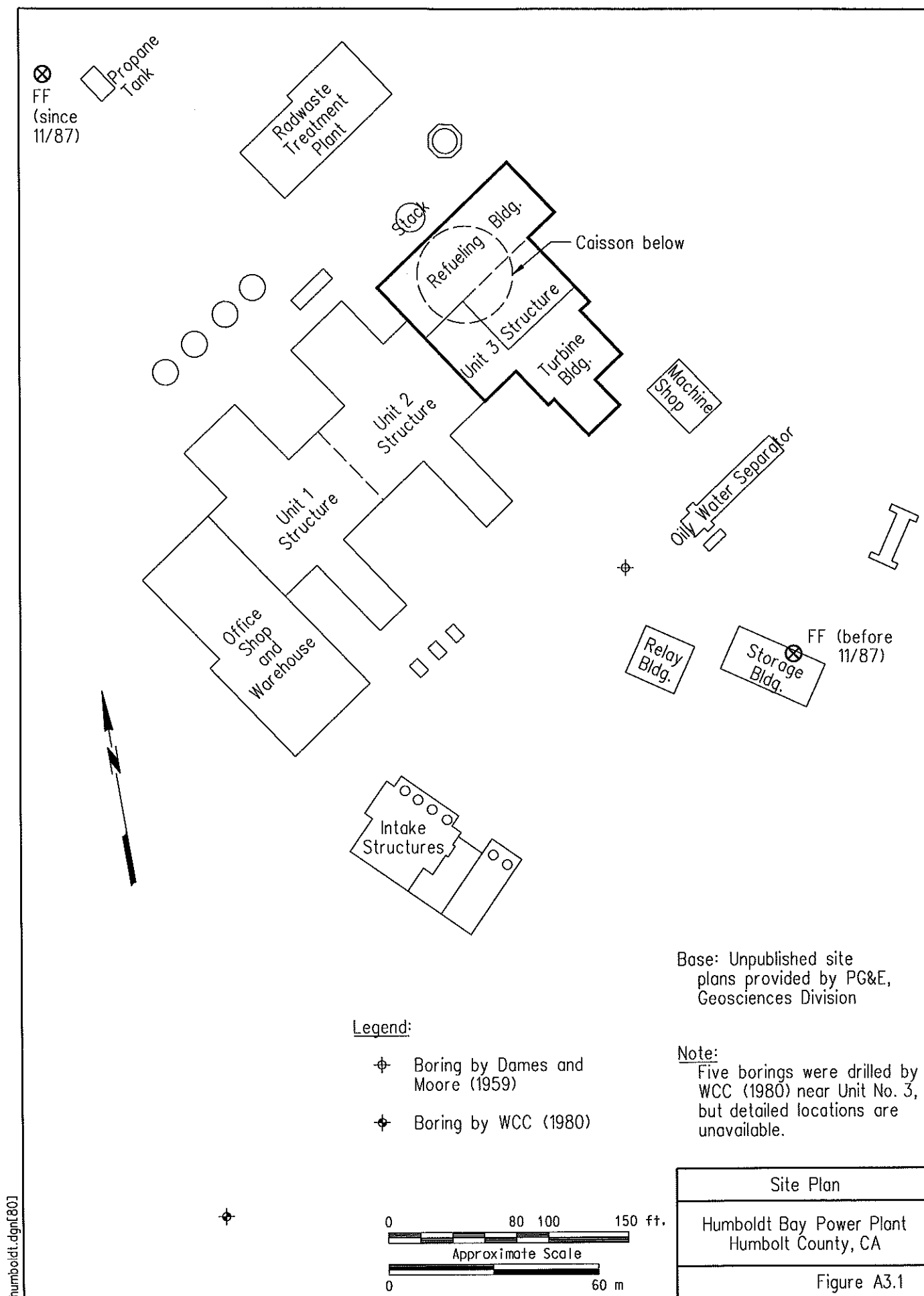
As no roof recordings are available for the refueling building, system identification analyses could not be performed. Instead, the primary insights to be gained from strong motion data at this site pertain to the variation of shaking between the free-field and base-of-caisson. The significant embedment of the caisson makes it useful for evaluating embedment effects on base-of-structure motions. Reported here are transmissibility functions for plant grade/caisson (PG/C) and caisson/free field (C/FF) motion pairs.

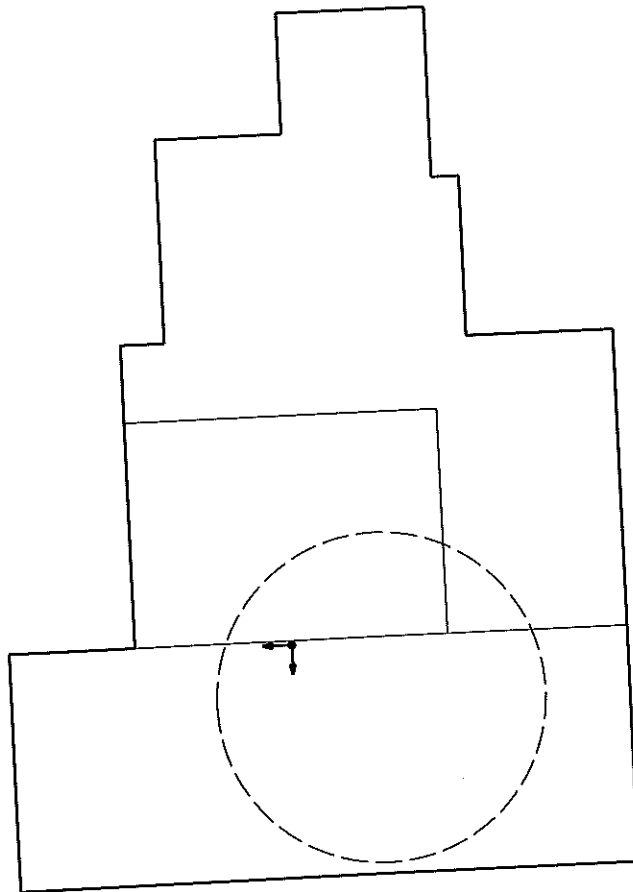
As shown in Fig. A3.**4, PG/C results indicate the fundamental frequency of the caisson-soil system is about 2.8-3.0 Hz in the FN event and 2.0 to 3.0 Hz in the PT event. This is consistent with the site frequency from elevation -66 feet to the surface.

For both the FN and PT, C/FF transmissibility functions (Fig. A3.**4) and the horizontal motion spectra (Fig. A3.**2-3) indicate significant de-amplification of caisson motions at frequencies greater than about 1 Hz. The de-amplification is particularly acute near apparent frequencies of the caisson-soil system (about 2 to 3 Hz), but is significant at higher frequencies as well. A comparison of these transmissibility function amplitudes with predictions from a simple kinematic interaction analysis for embedded structures is presented in Stewart (1996).

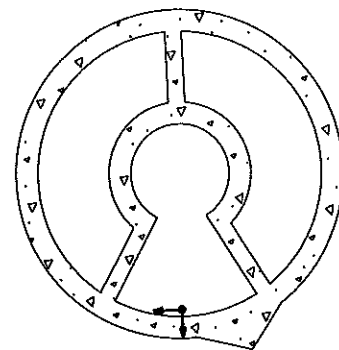
Data Sources

- Dames and Moore (1959). "Borings for Site Foundation Investigation," Appendix VI to Final Hazards Summary Report, Humboldt Bay Power Plant, Unit No. 3.
- Dames and Moore (1975). "P.G & E. Humboldt Bay Power Plant Digitization and Evaluation of Strong Motion Records, Ferndale Earthquake of June 7, 1975," *Job. No. 0160-181*.
- Pacific Gas and Electric, PG&E (1985). "Toxic Pits Cleanup Act, Hydrogeologic Assessment Report for Surface Impoundments at the Humboldt Bay Power Plant," December.
- Stewart, J.P. (1996). "An empirical assessment of soil-structure interaction effects on the seismic response of structures," *Ph.D. Dissertation*, University of California, Berkeley.
- Woodward Clyde Consultants, WCC (1980). "Evaluation of the Potential for Resolving the Geologic and Seismic Issues at the Humboldt Bay Power Plant Unit No. 3," Appendices.

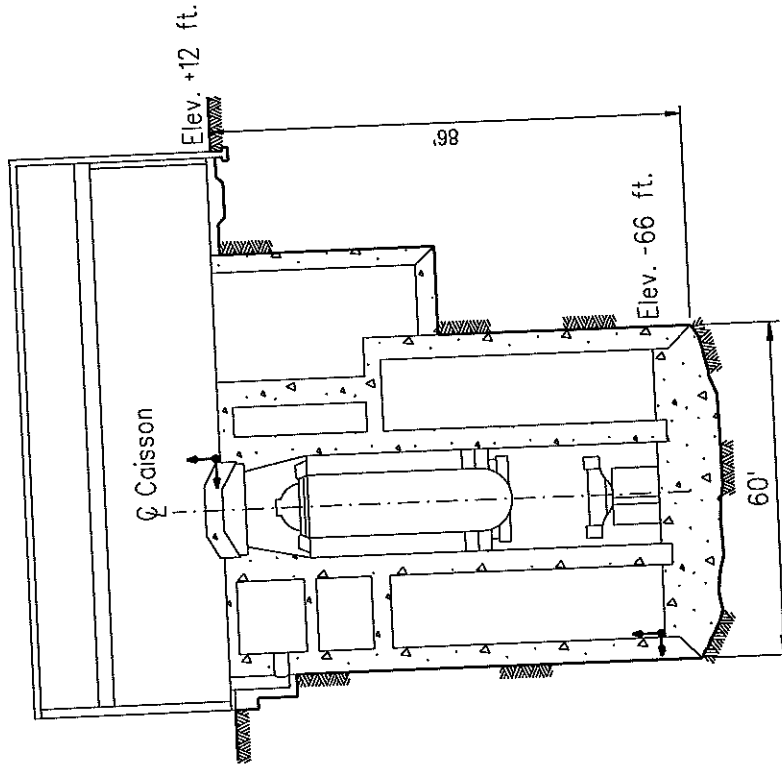




Plan at Elevation +12 ft.



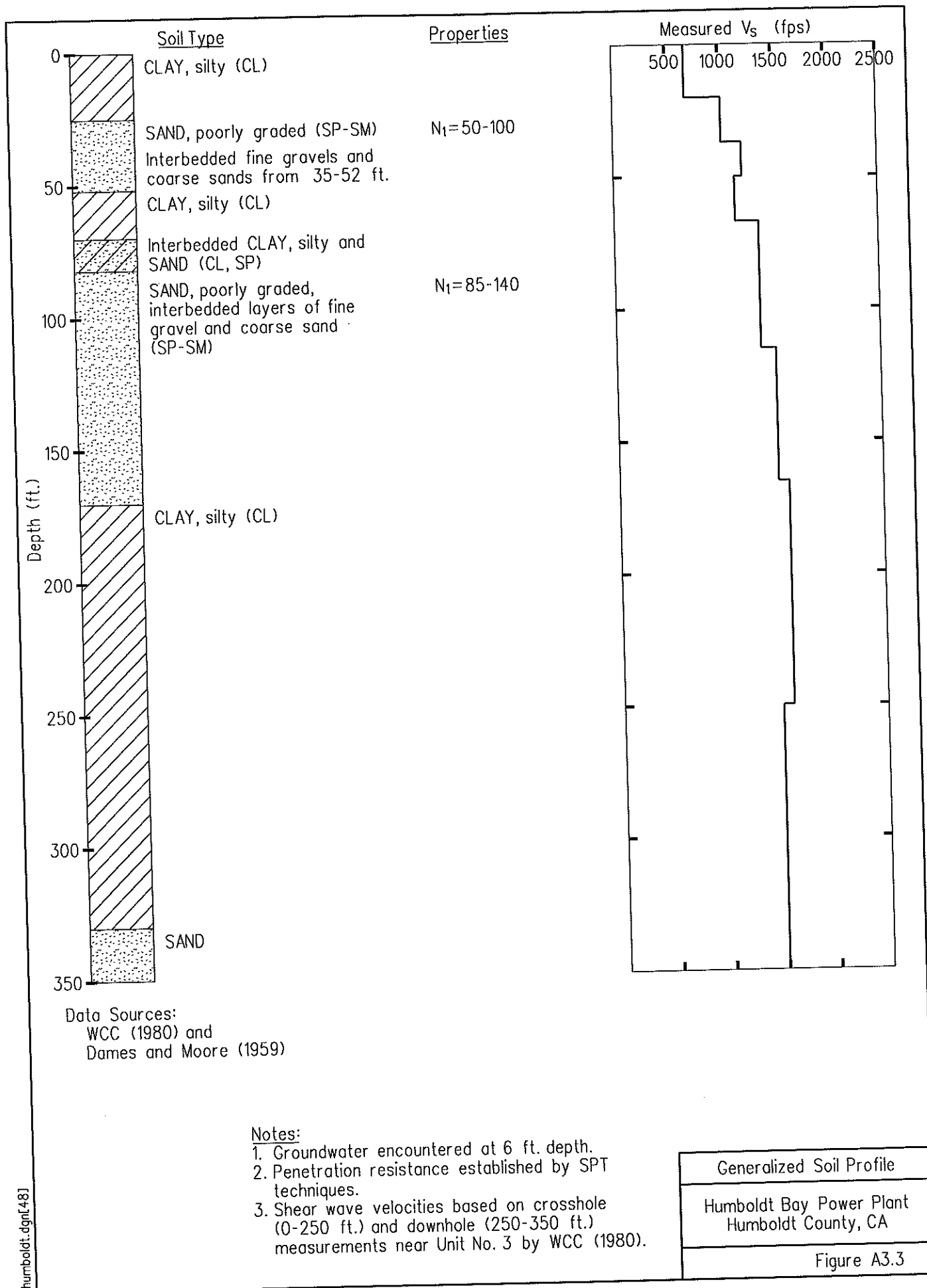
Plan at Elevation -66 ft.



N / S Elevation of Refueling Building

Sensor Locations
Humboldt Bay Power Plant Humboldt County, CA
Figure A3.2

Data provided by PG & E



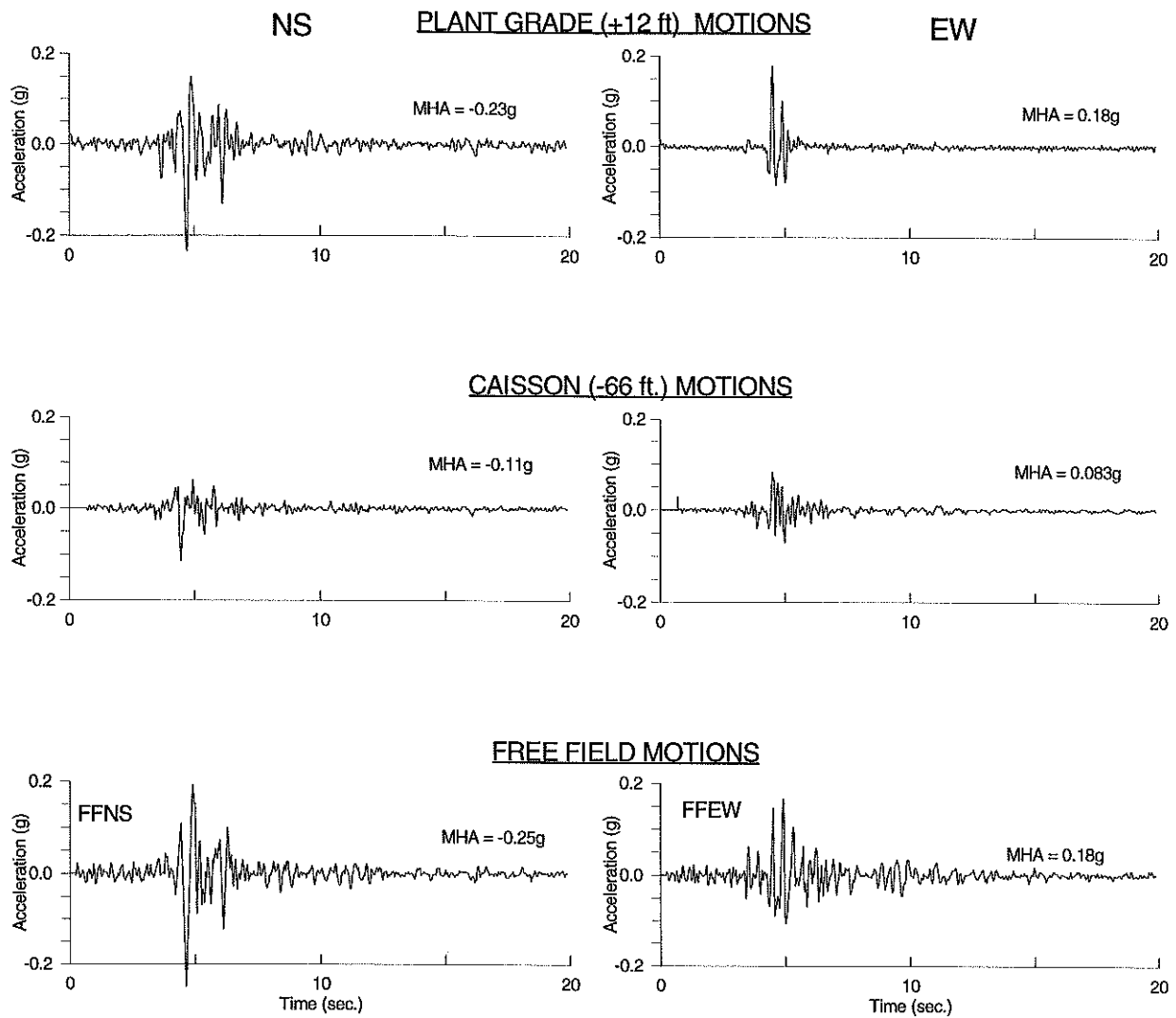


Fig. A3.FN1: HUMBOLDT BAY POWER PLANT
Horizontal Acceleration Time Histories
1975 Ferndale Earthquake

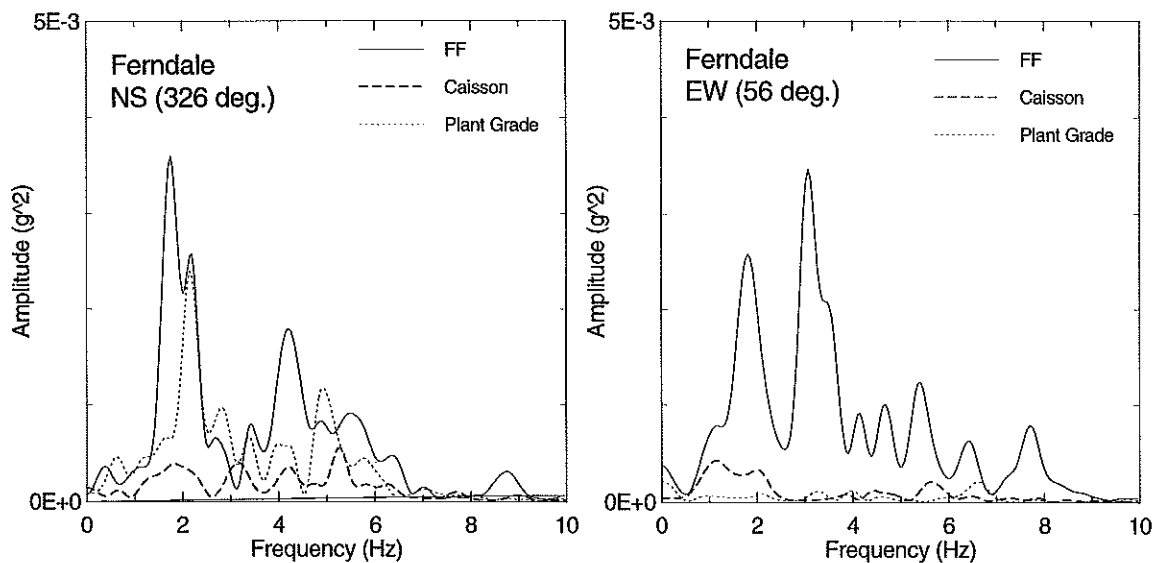


Fig. A3.FN2: HUMBOLDT BAY POWER PLANT
Power Spectral Density of Selected Horizontal Accelerations
1975 Ferndale Earthquake

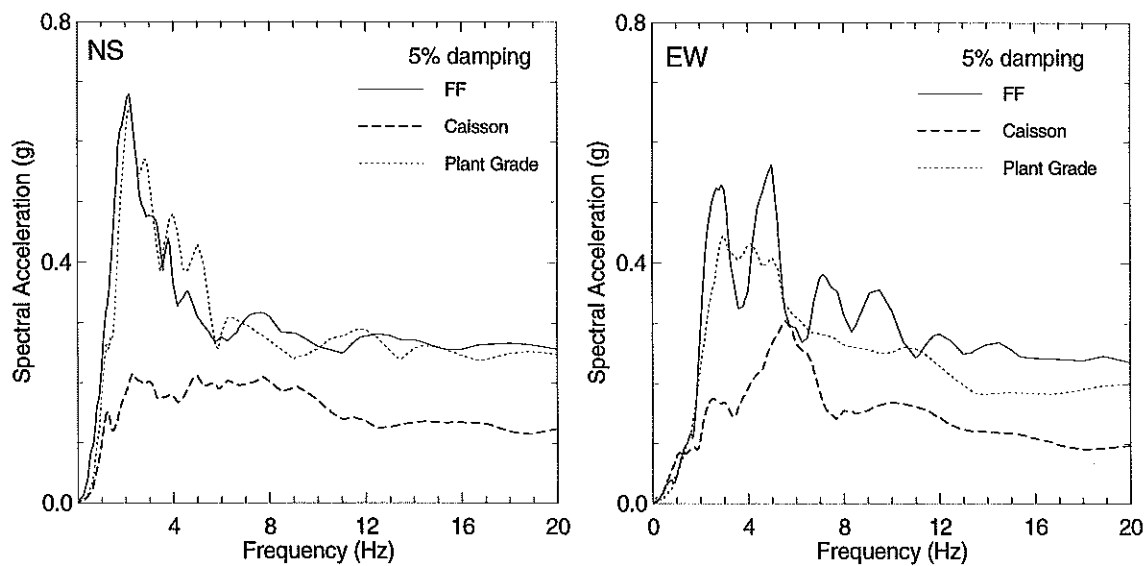


Fig. A3.FN3: HUMBOLDT BAY POWER PLANT
Acceleration Response Spectra of Caisson and Free-Field Motions
1975 Ferndale Earthquake

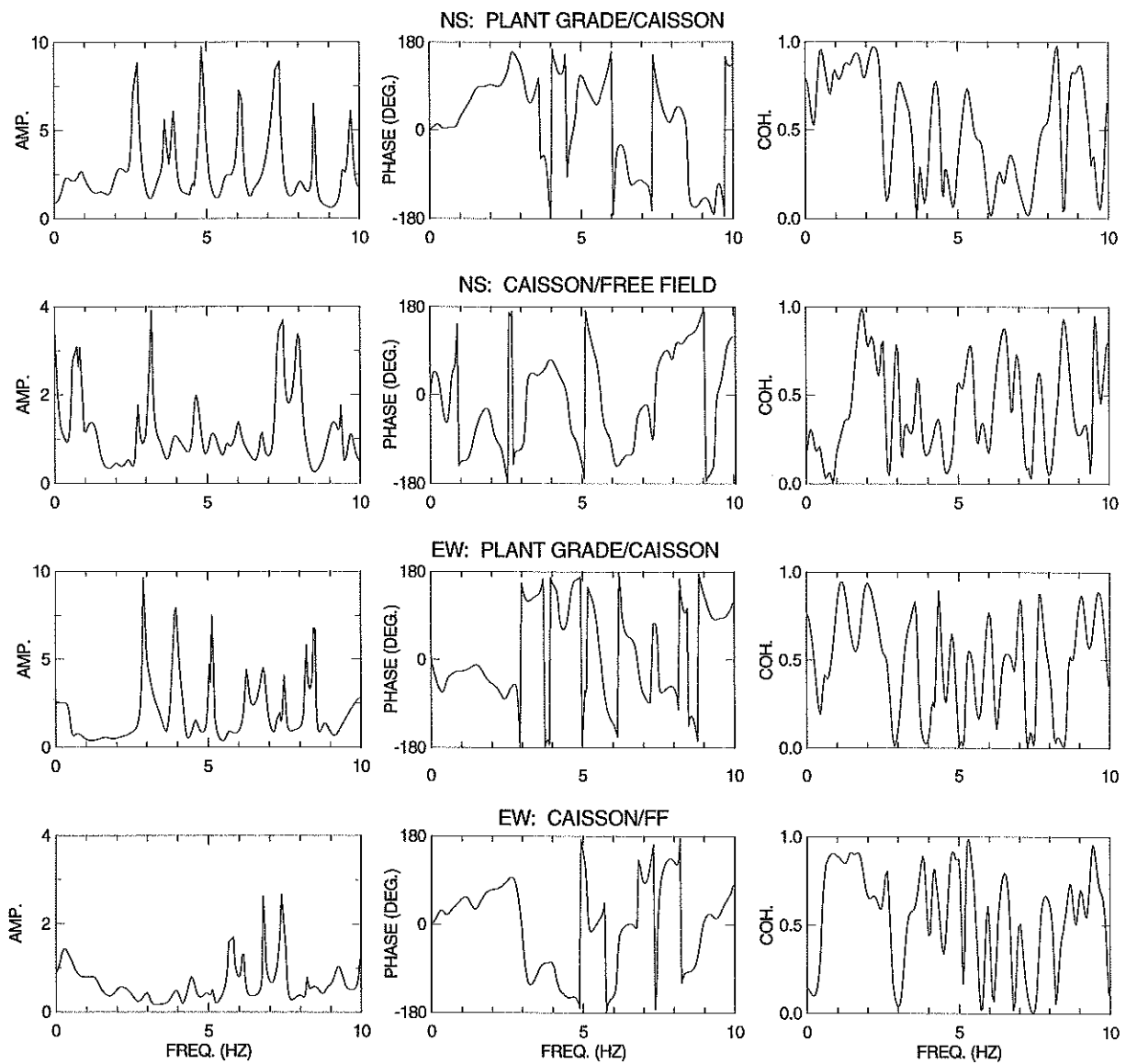


Fig. A3.FN4: HUMBOLDT BAY POWER PLANT
Transmissibility and Coherency Functions
1975 Ferndale Earthquake

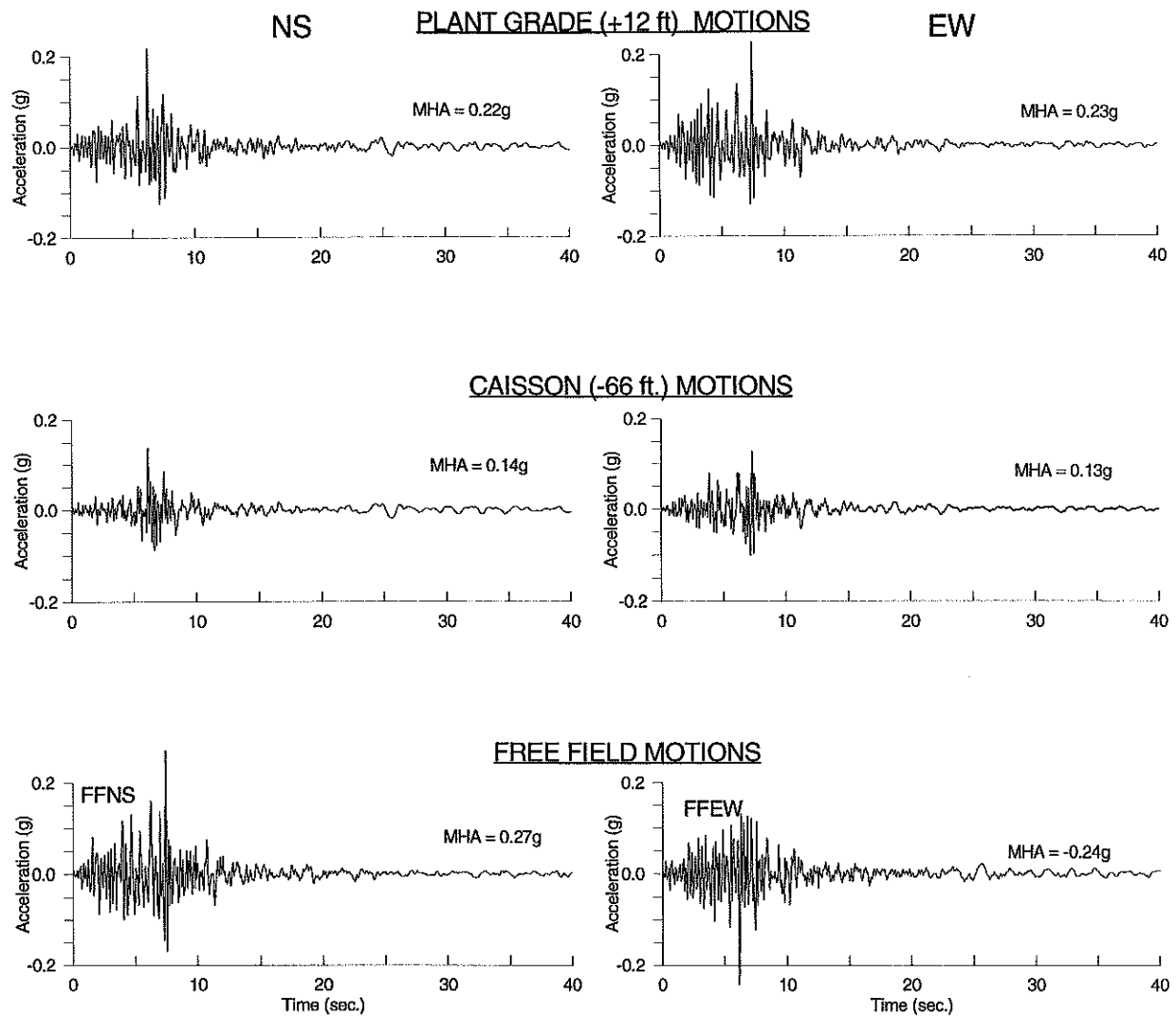


Fig. A3.PT1: HUMBOLDT BAY POWER PLANT
Horizontal Acceleration Time Histories
1992 Petrolia Earthquake

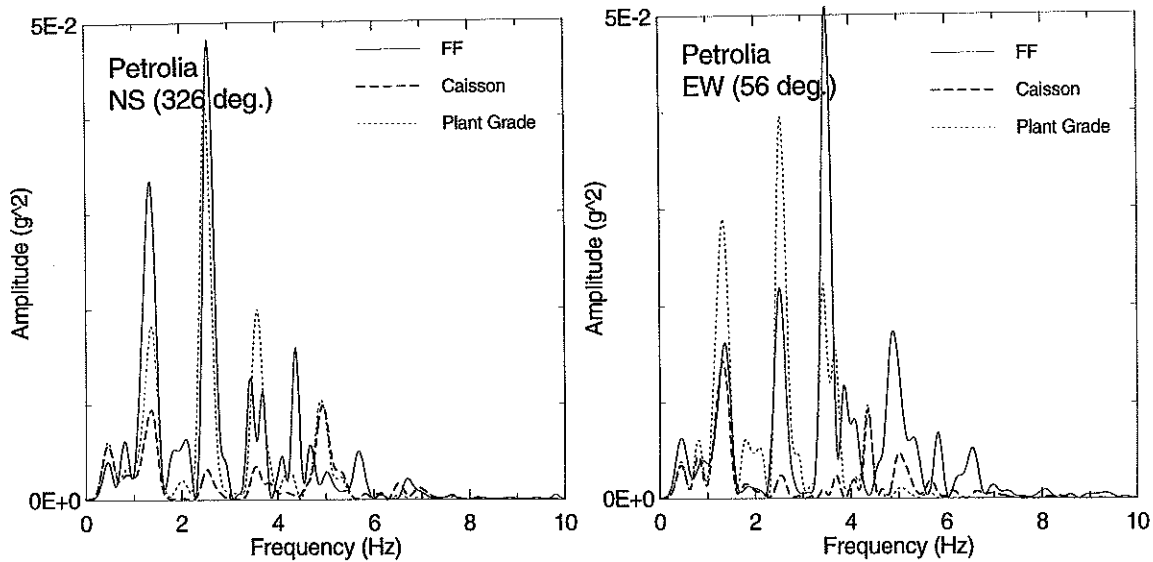


Fig. A3.PT2: HUMBOLDT BAY POWER PLANT
Power Spectral Density of Horizontal Accelerations
1992 Petrolia Earthquake

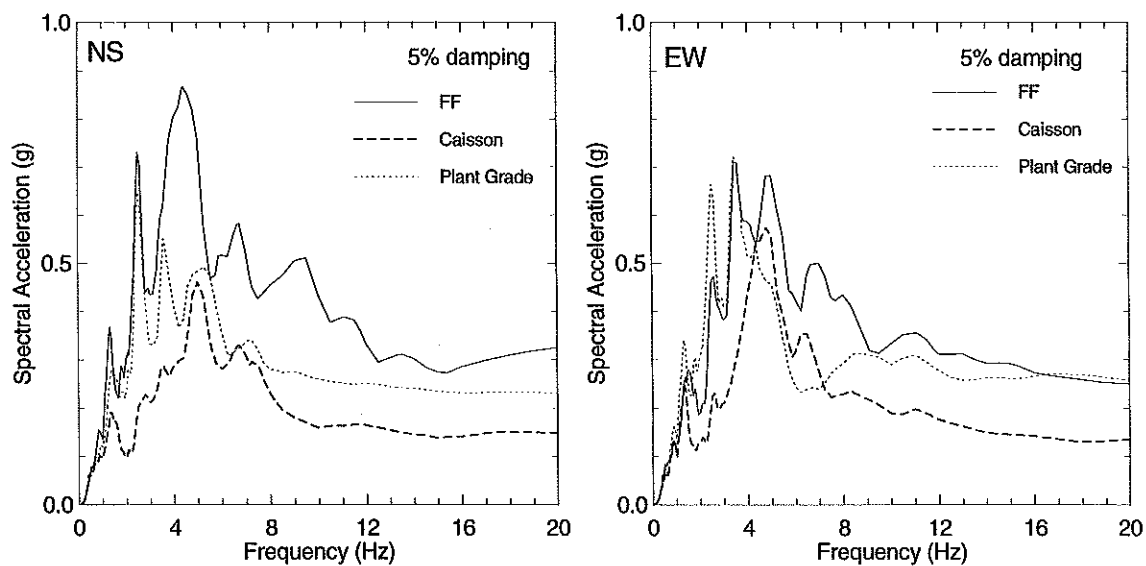


Fig. A3.PT3: HUMBOLDT BAY POWER PLANT
Acceleration Response Spectra of Horizontal Motions
1992 Petrolia Earthquake

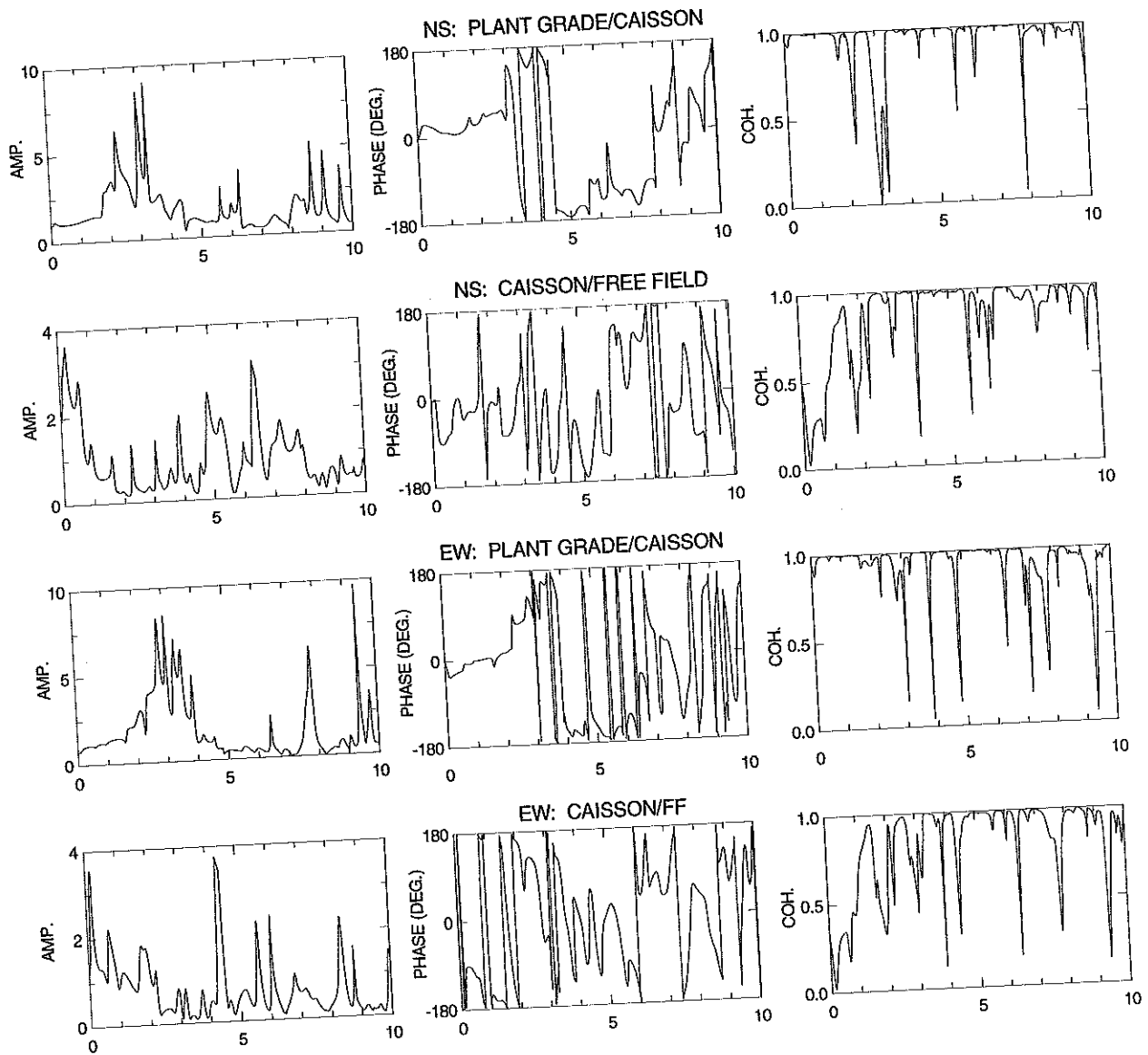


Fig. A3.PT4: HUMBOLDT BAY POWER PLANT
Transmissibility and Coherency Functions
1992 Petrolia Earthquake

SITE A4

Site Name: Emeryville Pacific Park Plaza
Sensors: USGS 1662

Lat: 37.844°
Long.: 122.295°

Floor Levels: 31
Bsmt. Levels: 0
Structure: Concrete moment frame

Foundation: concrete piles
Site Geology: soil
Design Date: 1982

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.21	0.25	0.059	0.18	0.21	0.048	0.25	0.37	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: USGS, filtered at $f_{HP} = 0.1$ Hz; digitized at $f_{Nyq} = 100$ Hz; FF/str. data not synchronized

Final: decimated by 8 to $f_{Nyq} = 12.5$ Hz, FF/str. synchronized using cross-correlation of vertical accelerations (Fig. A4.LP0).

Figures

A4.1-A4.3
A4.LP0-6

Site Plan, Sensor Locations, Generalized Soil Column
LP Analysis Results

View of Pacific Park Plaza
and garage from FF south



DISCUSSION

Site Description

The building is 31 stories tall above ground level with no basement, and is Y-shaped in plan with three wings connected to a central core. The central core contains two elevator shafts and each wing has a stairwell. A two- to five-story parking garage is located adjacent to the west wing of the building (the connection is non-structural). Shear walls are present up to the second floor in the central core and wings; the remainder of the structure is a reinforced concrete ductile moment resisting space frame. The foundation is a 5-foot thick concrete mat supported on 828 14-inch square by 100-foot deep prestressed concrete piles. As shown in Fig. A4.2, accelerographs are present on the ground, 13th, 21st, and roof levels. There are two free-field instruments at the site. The south instrument is located in a parking lot about 530 feet south of the tower and about 200 feet south of the parking garage, the north instrument is about 130 feet northwest of the building and about 40 feet northwest of the parking garage.

The site is a reclaimed tidal mudflat on the east shore of the San Francisco Bay. Based on soil borings by Guha et al. (1993), Gibbs et al. (1994), and WCC (1981), the generalized soil profile (Fig. A4.3) consists of: 12 feet of silty sandy fill, 14 feet of soft silty clay, 9 feet of silty and clayey sands and gravels, 30 feet of a silty clay known as Old Bay Clay, and a series of interbedded layers of clays and sands/gravels. The maximum depth explored at the site was 200 feet; conditions at greater depth were inferred by Rogers and Figures (1991) from an array of deep borings in the area, and consist of clays with occasional sand/gravel layers which extend to sedimentary bedrock at a depth of 500 feet. Depth-to-groundwater measurements indicate the soils at the site are saturated below a depth of about 10 feet. The shear wave velocities shown in Fig. A4.3 are based on in situ downhole measurements by Gibbs et al. (1994). Using the subsurface data and the power spectra of the free-field motions (Figs. A4.LP2), a site frequency of $f \approx 0.7$ Hz is estimated, which is in general agreement with results of one-dimensional site response analyses (0.7 to 0.8 Hz; Dickenson, 1994).

Instrument Selection and Free-Field Conditions

System identification analyses for this site focused on the response of the structure's central core, so the sensors used were from the core except at the ground floor level for which motions are only available in the north wing. The use of wing lateral motions to represent the core is unlikely to introduce significant errors as the 5-foot thick base mat should maintain relatively uniform ground-floor horizontal motions. Above the foundation level, the wings in the structure have been previously shown by Safak and Celebi (1991) to develop displacements relative to the core in several high vibration modes (i.e. > 3 rd mode), but these effects were not investigated in this study. Four vertical instruments located at the ground floor level were used to evaluate rocking effects in both the NS and EW directions. The specific sensors used are indicated in Table A4.1; acceleration time histories recorded by these and other sensors are shown in Fig. A4.LP1.

Table A4.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 2-9	Ch 2-7	Ch 2-4, 2-5, 2-6, 2-8	Ch 1-4	Ch 1-5

The following inputs were used in system identification analyses to evaluate “fixed-base” structural parameters for the NS and EW directions,

$$\begin{aligned} \text{input, ns} &= (\text{Ch2} - 9) + [(\text{Ch2} - 5) - (\text{Ch2} - 8)] \cdot 310/200 \\ \text{input, ew} &= (\text{Ch2} - 7) + [(\text{Ch2} - 6) - (\text{Ch2} - 4)] \cdot 310/120 \end{aligned} \quad (1)$$

Located in an open parking lot about 530 feet south of the structure, the FF south instrument is unlikely to be significantly affected by structural vibrations. Examination of the ground floor and FF south spectra in Fig. A4.LP2 confirms that the FF south data does not contain significant energy at several structural vibration frequencies (e.g. 0.4, 1.1, 2 Hz, etc.). The FF north instrument, however, is located relatively close to the structure (130 feet separation), and appears to be strongly influenced by the structural vibrations as the FF north and ground floor spectra in Fig. A4.LP2 are similar at the structure’s modal frequencies. Based on these observations, only FF south motions were used in parametric system identification analyses.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

GF/FF transmissibility functions (Fig. A4.LP4) and the spectra of horizontal motions (Fig. A4.LP2-3) generally indicate some de-amplification of GF motions for frequencies < 3 Hz, and minor amplification from about 3 to 7 Hz.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A4.LP5. Vertical GF motions were amplified near some lateral response frequencies ($f \approx 1.1$ and 3.6 Hz), suggesting that rocking may have influenced the structural response. The effects of base rocking on roof motion are directly examined in Fig. A4.LP5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). RxH displacements for both directions have low frequency oscillations near the high-pass cutoff frequency which may be unreliable. There is little similarity between RxH displacements and relative roof displacements, indicating that base rocking did not significantly contribute to the structure’s lateral roof displacements. The same conclusion was reached by Safak and Celebi (1992), who found no significant peaks in the cross correlation function between vertical GF and lateral roof motions.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A4.LP4), (2) modal parameters from CEM analyses (Table A4.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A4.LP6). For comparison, modal frequencies identified from pre-earthquake ambient and forced vibration tests are provided in Table A4.3.

Table A4.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF So-R)			
		tr (NS) d=12, j=25		L (EW) d=10, j=20		tr (NS) d=12, j=25		L (EW) d=10, j=20		tr (NS) d=9, j=25		L (EW) d=9, j=20	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	0.40	6.3	0.38	6.4	0.40	6.7	0.37	8.5	0.40	12.9	0.38	13.1
	2	1.00	9.1	0.99	3.7	1.04	9.6	0.98	3.6	1.17	6.0	0.96	5.3
	3	1.84	8.0	1.57	11.0	1.87	8.0	1.62	13.6	1.87	2.5	1.54	11.3
	4	2.63	4.7	2.02	4.2	2.64	4.5	2.01	6.5	2.64	3.7	1.99	5.4
	5	3.15	3.2	2.68	2.8	3.16	2.3	2.67	2.7	3.21	5.0	2.74	6.8
	6	3.60	2.1	3.21	5.3	3.63	2.2	3.23	6.8	3.63	3.2	3.07	10.0

Table A4.3: Modal frequencies (in Hz) identified from ambient and forced vibration tests (Stephen et al., 1985)

Mode	Ambient Vibration			Forced Vibration		
	NS	EW	Torsion	NS	EW	Torsion
1	0.59	0.59	0.59	0.59	0.60	0.57
2	1.69	1.69	1.71	1.66	1.68	1.70
3	3.15	3.13	3.13	3.09	3.12	3.16
4	4.52	4.44	4.44	4.36	4.39	4.53
5				6.24	6.19	
6				7.45	7.45	

Comments on the analytical results follow:

- As shown in Fig. A4.LP2, the roof motions in the NS direction are dominated by the third mode at $f \approx 1.9$ Hz and by excitation at the site frequency of $f \approx 0.7$ Hz. In the EW direction, roof accelerations are dominated by the second and third modes ($f \approx 1$ and 2 Hz, respectively) and by excitation at the site frequency.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response. However, flexible-base damping is seen to significantly exceed fixed-base damping.
- CEM analyses of FF-R response for the NS direction identified a second mode at $f = 0.85$ Hz. Roof motions have significant energy at this frequency because it is dominant in the input (free-field) motions. However, no resonant structural response occurs at this frequency, and it does not appear to be a modal frequency (and hence is unreported in Table A4.2).

- Modal frequencies determined from low-amplitude forced and ambient vibration tests (Table A4.3) were generally at least 50% higher than those identified from the earthquake data.

The RPEM results indicate minor reduction in first-mode frequency (Fig. A4.LP6) from about 10 to 25 seconds. These results are generally consistent with recursive analyses by Safak and Celebi (1992), who found the building response to be nonlinear in the first 17 seconds. It should be noted that no damage was reported to the building from the LP event. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the first-mode FF-R and GF-R parameters.

(c) Analysis of SSI Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A4.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 87 feet using the recorded FF south motions and the profile in Fig. A4.3. The tower was assumed to be independent of the neighboring parking structure for the calculation of foundation radii. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A4.4: Soil/structure parameters used in predictive analyses

LP	Geotechnical		Structural		
	Model = H	$V_s = 620$ fps	$h = 218$ ft	$emb. = 0$	$\gamma = 0.15$
	$\nu = 0.45$		tr: $r_1 = 87$ ft		$r_2 = 94$ ft
	$\beta^* = 6.8\%$	$V_s^* = 450$ fps	L: $r_1 = 87$ ft		$r_2 = 94$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A4.4 with the fixed-base modal parameters in Table A4.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A4.5.

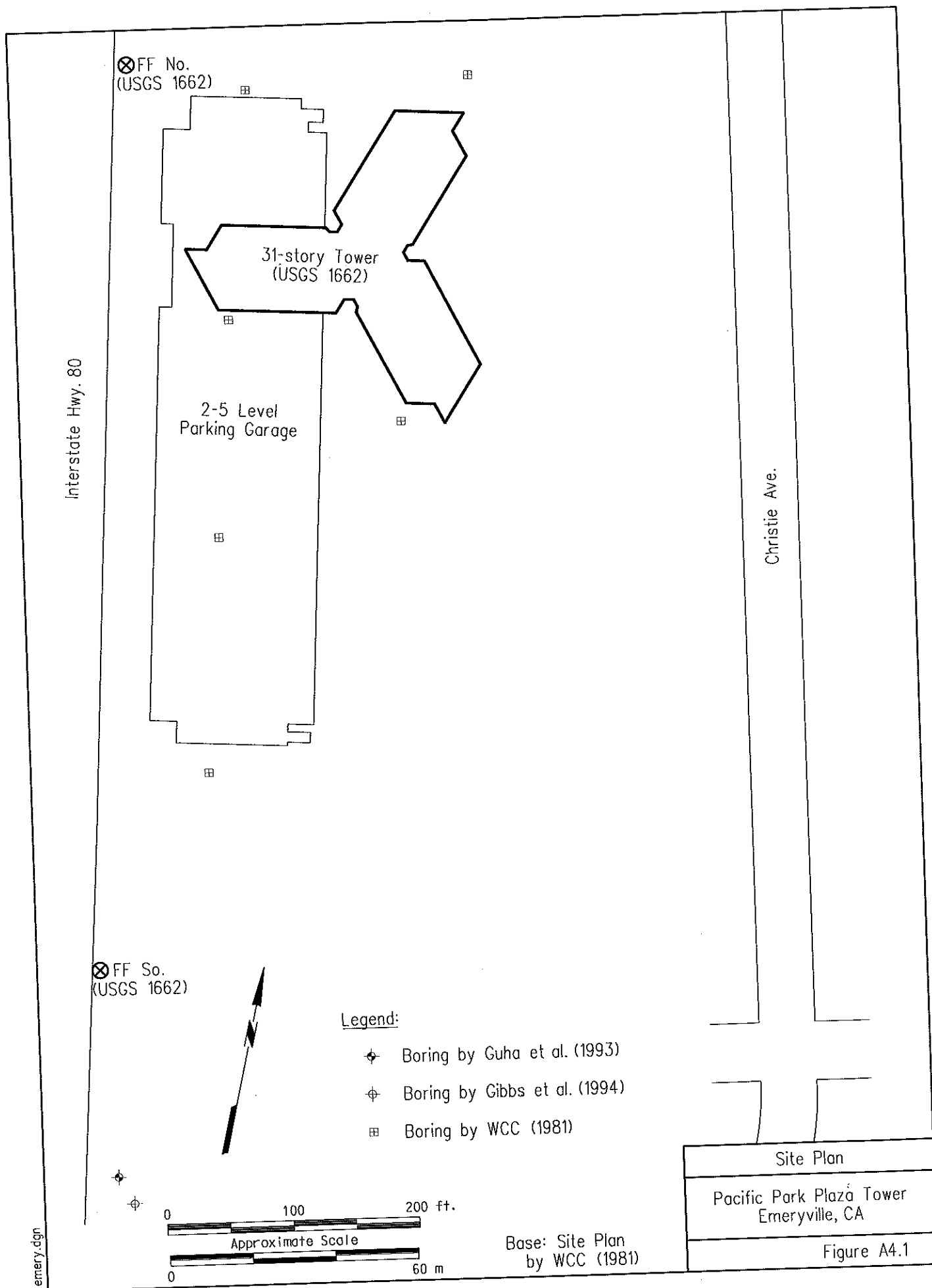
Table A4.5: Empirical and predicted inertial interaction effects

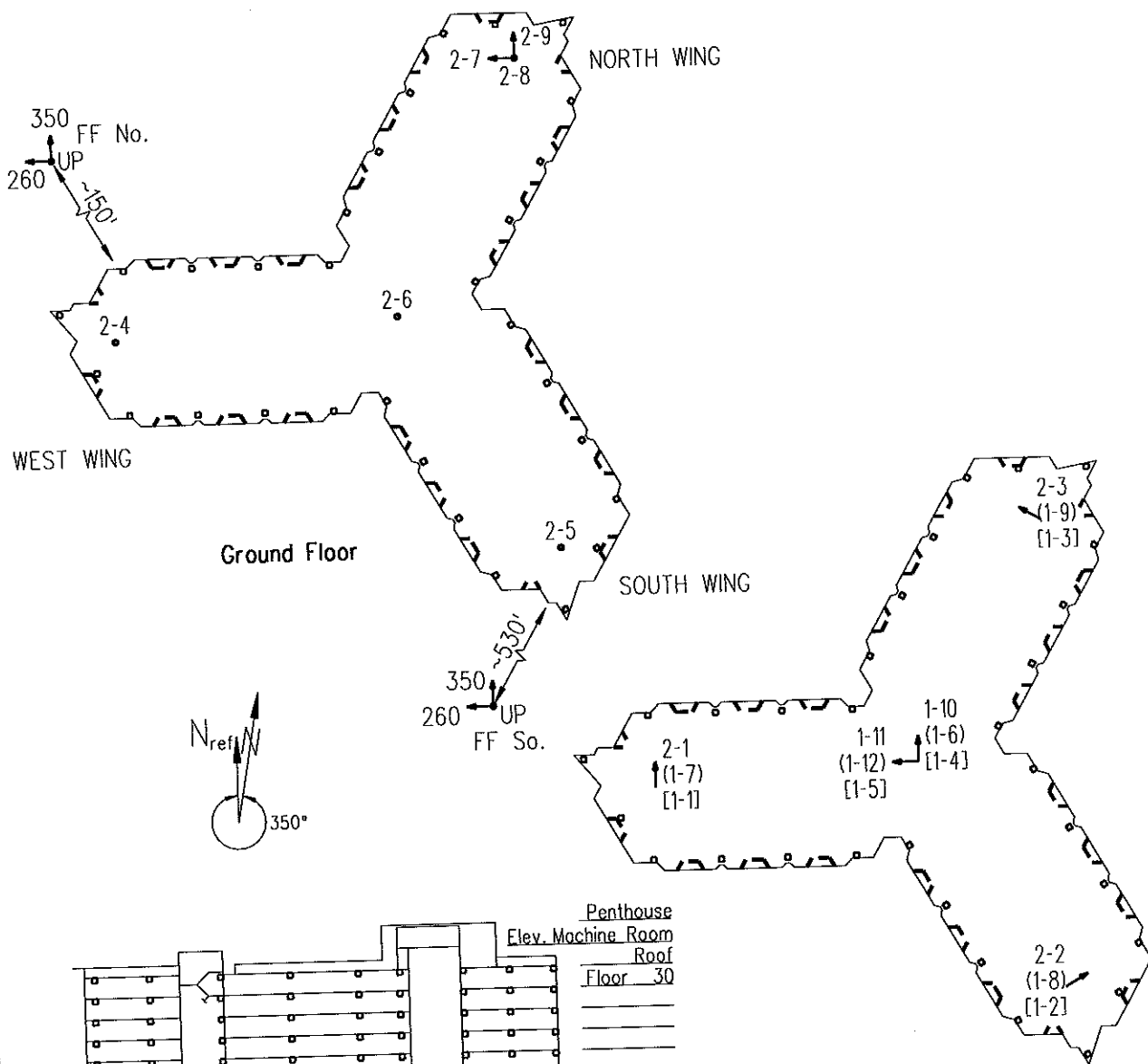
LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.00	6.7	12.9	1.00	6.7	13.1
	Prediction	1.17	1.6	5.6	1.14	1.4	5.7

The period lengthening was significantly overpredicted and the foundation damping underpredicted for this structure. This result is likely associated with the influence of the pile foundations, which are ignored in the development of the predictions, on the foundation impedance. The discrepancy in the results is consistent with the expected effects of piles; namely, to increase the rocking stiffness and the radiation damping relative to the surface foundation case.

Data Sources

- Dickenson, S. A. (1994). "The dynamic response of soft and deep cohesive soils during the Loma Prieta Earthquake of October 17, 1989," *Ph.D. Dissertation*, U.C. Berkeley.
- Guha, S., Bray, J.D., and Drnevich, V.P. (1993). "Characteristics of the deep Old Bay Clay deposits in the East San Francisco Bay Area," *Rpt. No. UCB/GT/93-09*, Geotechnical Engineering Div., Dept. of Civil Engineering, University of California, Berkeley.
- Rogers, J.D. and Figures, S.H. (1991). "Engineering geologic site characterization of the greater Oakland-Alameda area, Alameda and San Francisco Counties, California," *Grant No. BCS-9003785*, Report to National Science Foundation.
- Safak, E. and Celebi, M. (1991). "Analyses of the recorded responses of two high-rise buildings during the Loma Prieta Earthquake of 18 October, 1989," *Bull. Seism. Soc. Am.*, 81(5), 2087-2110.
- Safak, E. and Celebi, M. (1992). "Recorded seismic response of Pacific Park Plaza II: System Identification," *J. Structural Engrg.*, ASCE, 118(6), 1566-1589.
- Stephen, R.M., Wilson, E.L., and Stander, N. (1985). "Dynamic properties of a thirty-story condominium tower building," *Rpt. No. UCB/EERC 85-03*, Department of Civil Engineering, U.C. Berkeley.
- Gibbs, J.F., Fumal, T.E., and Powers, T.J. (1994). "Seismic velocities and geologic logs from borehole measurements at 7 strong motions stations that recorded the 1989 Loma Prieta Earthquake," *Open-File Report 94-222*, U.S. Geological Survey.
- Woodward Clyde Consultants, WCC (1981). "Geotechnical Engineering Study, Final Design Project, East Bay Park Condominium, Emeryville, CA," *Proj. No. 14884B*.





13th and 21st floors and Roof

Legend:

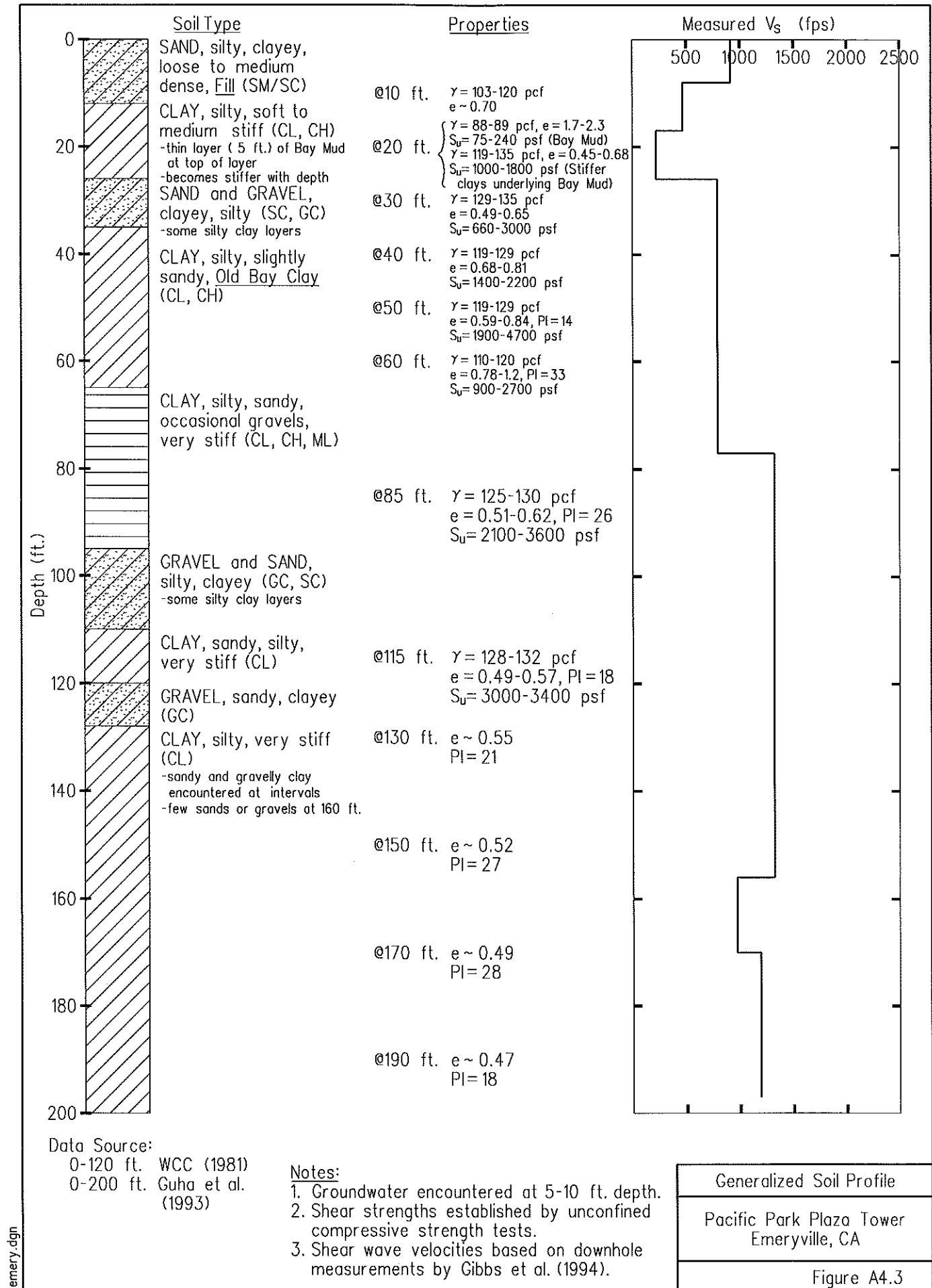
- 2-3 instrument on 13th floor
- (1-6) instrument on 21st floor
- [1-2] instrument on Roof

Note: Data provided by USGS

Sensor Locations

Pacific Park Plaza Tower
Emeryville, CA

Figure A4.2



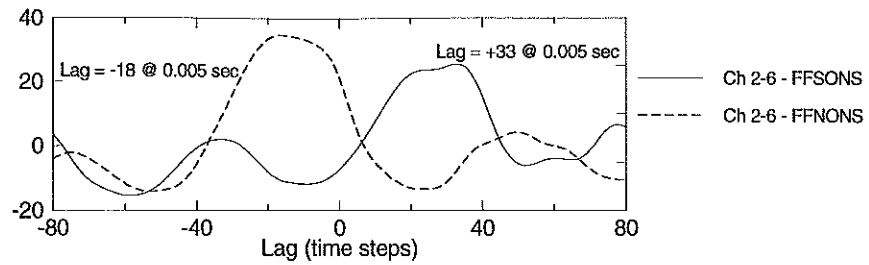


Fig. A4.LP0: Cross-correlation function between vertical ground floor and free field accelerations

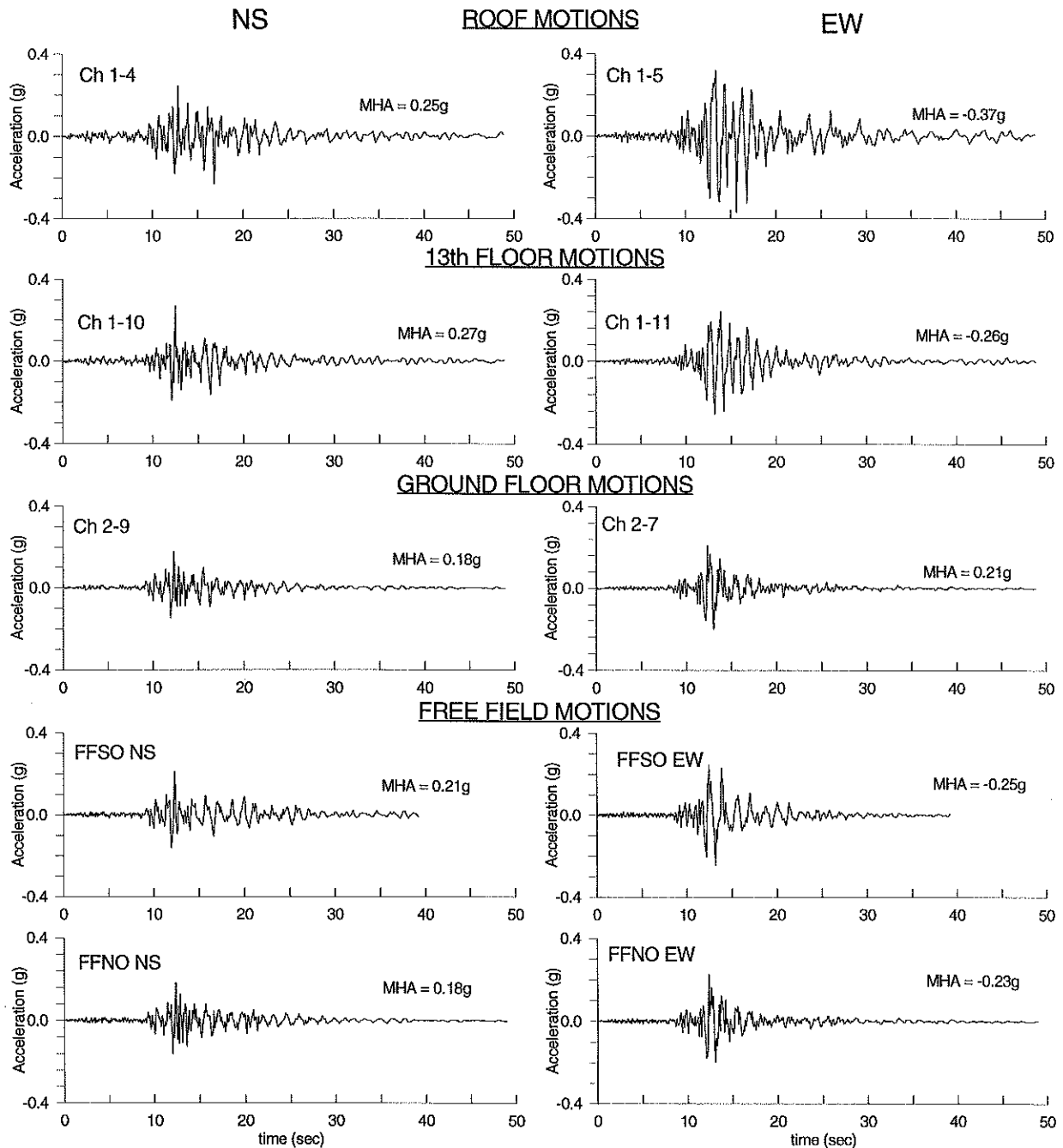


Fig. A4.LP1: EMERYVILLE PACIFIC PARK PLAZA
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

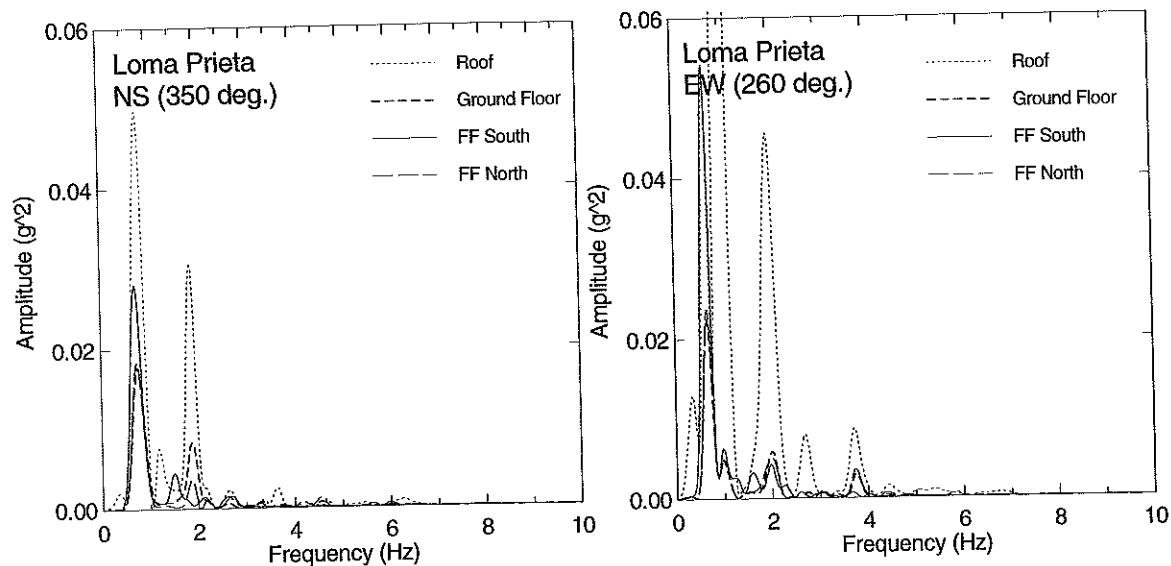


Fig. A4.LP2: EMERYVILLE PACIFIC PARK PLAZA
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

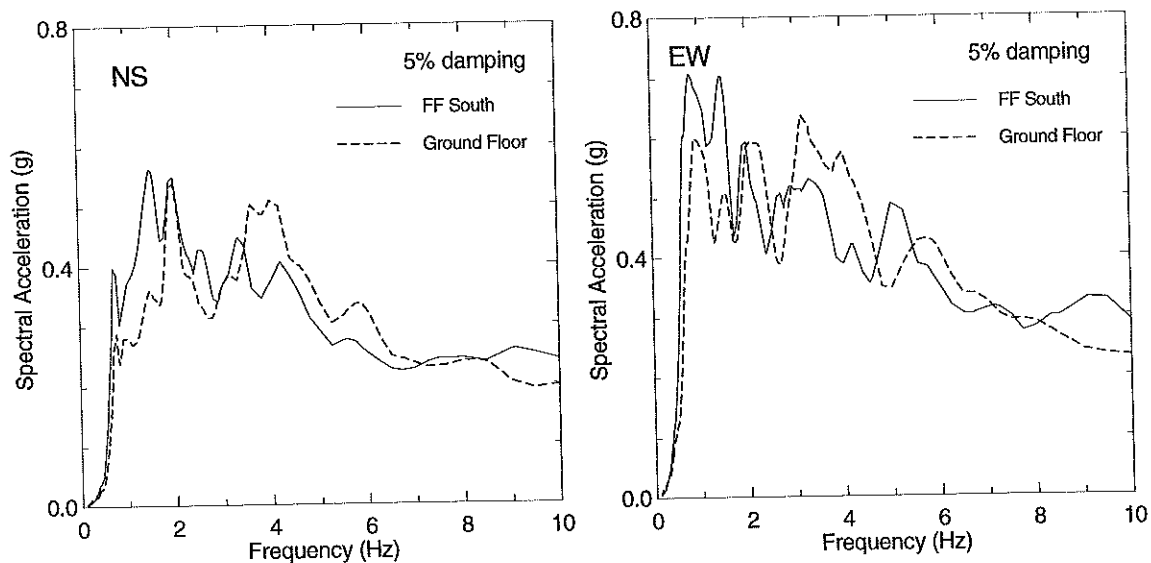


Fig. A4.LP3: EMERYVILLE PACIFIC PARK PLAZA
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1989 Loma Prieta Earthquake

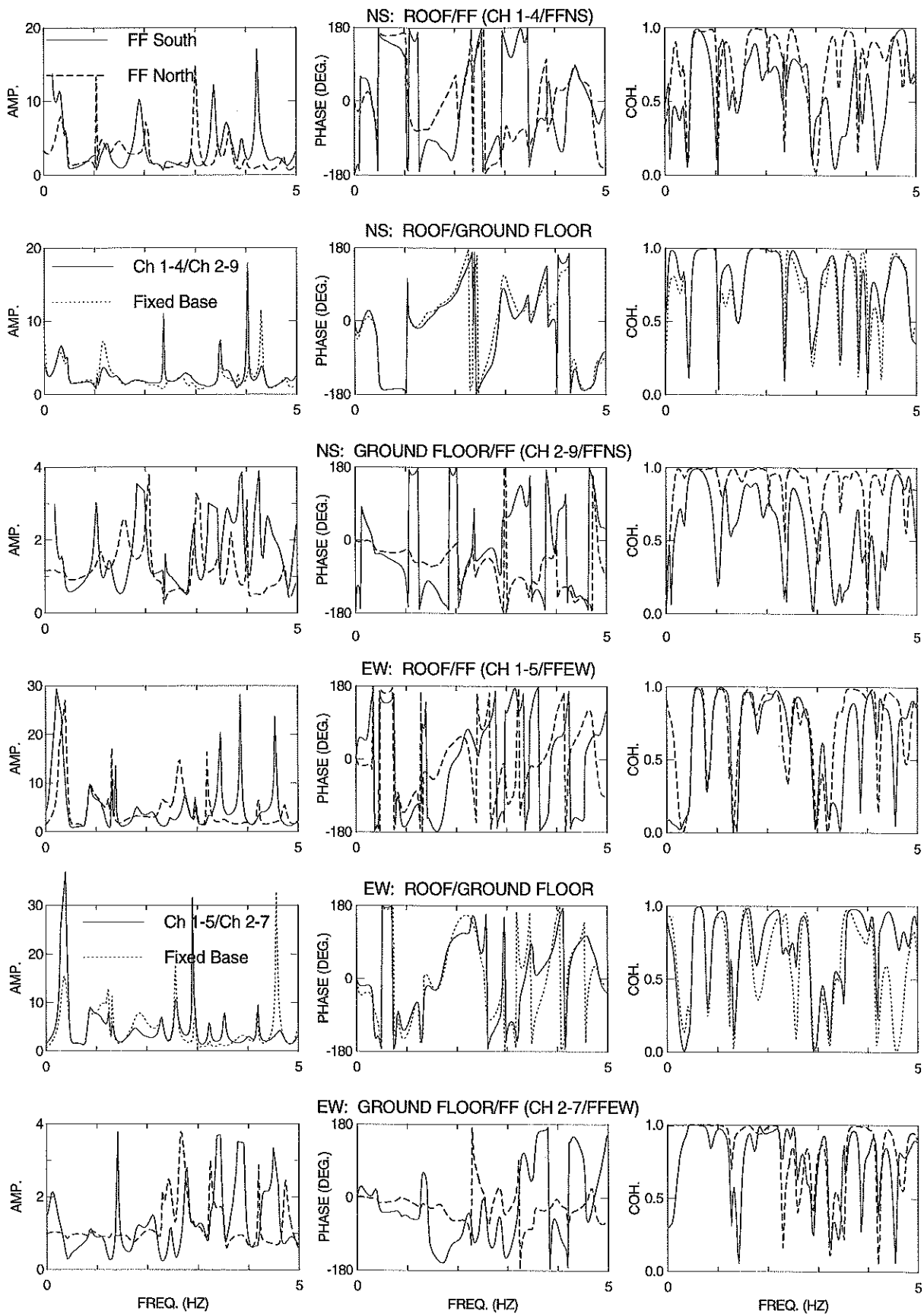


Fig. A4.LP4: EMERYVILLE PACIFIC PARK PLAZA
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

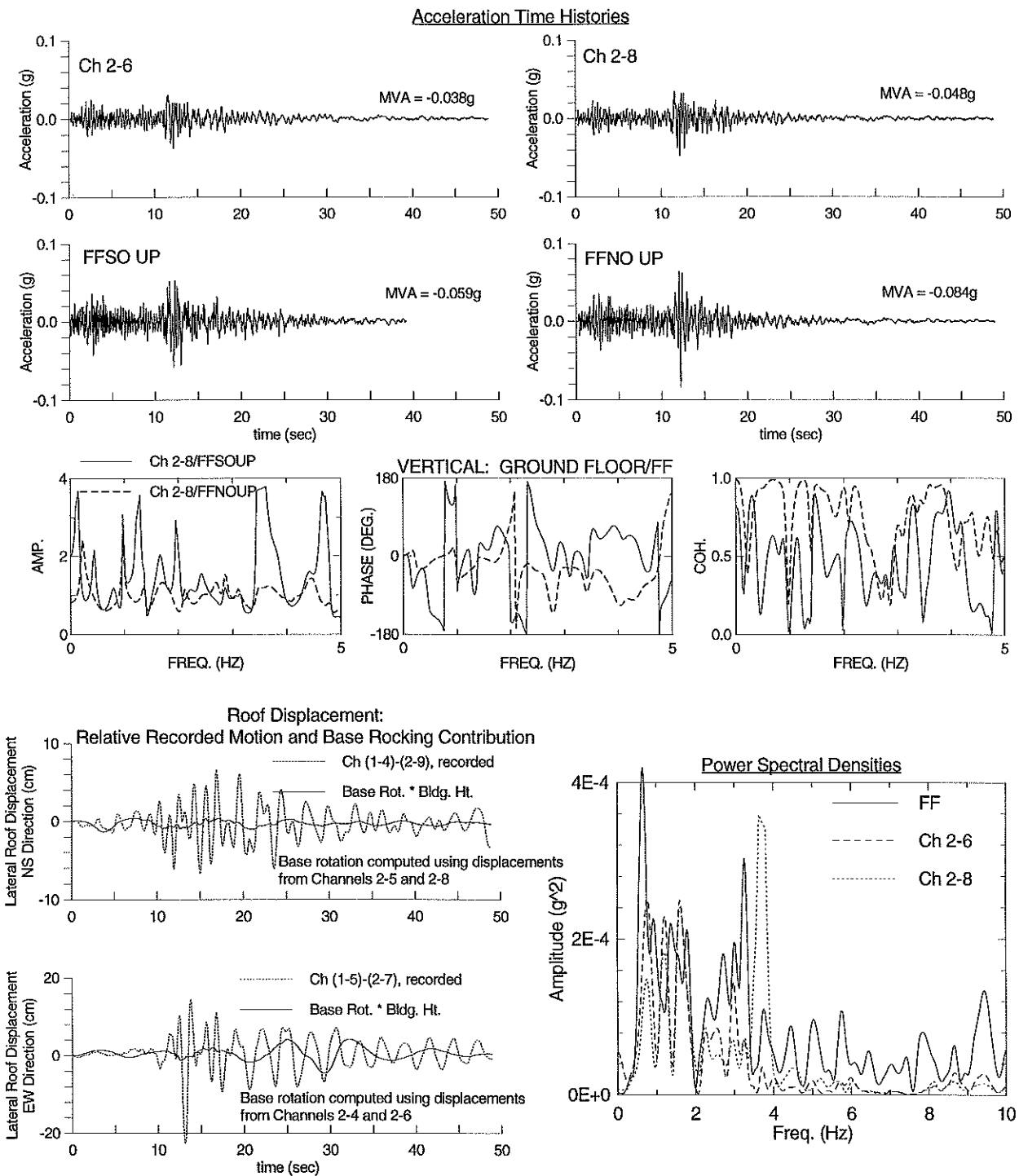


Fig. A4.LP5: EMERYVILLE PACIFIC PARK PLAZA
Analysis of Vertical Motions and Rocking Effects
1989 Loma Prieta Earthquake

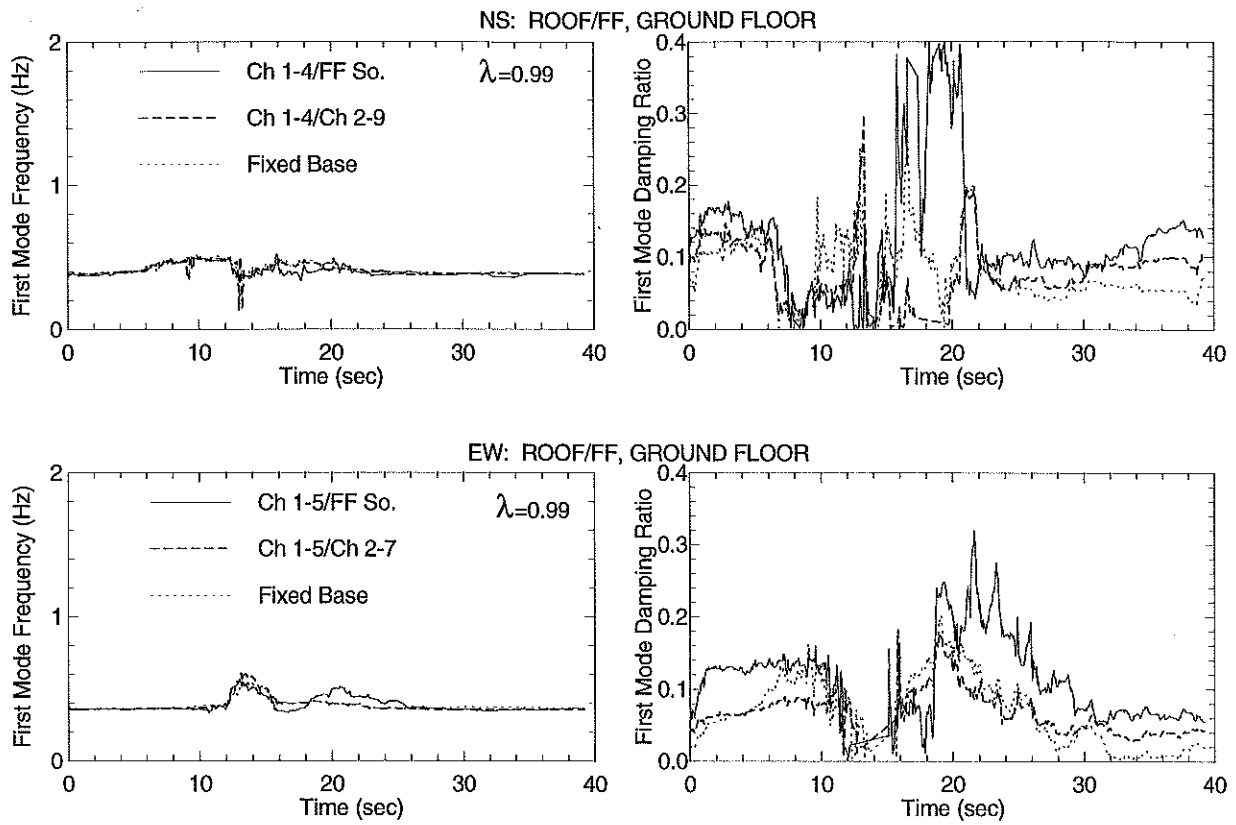


Fig. A4.LP6: EMERYVILLE PACIFIC PARK PLAZA
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE A5

Site Name: Hayward City Hall
Sensors: USGS 1129

Lat: 37.679°
Long.: 122.082°

Floor Levels: 11
Bsmt. Levels: 0
Structure: Concrete moment frame and
concrete/steel stud walls

Foundation: concrete piers
Site Geology: rock
Design Date: 1967

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.048	0.048	0.027	0.054	0.043	0.032	0.095	0.13	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

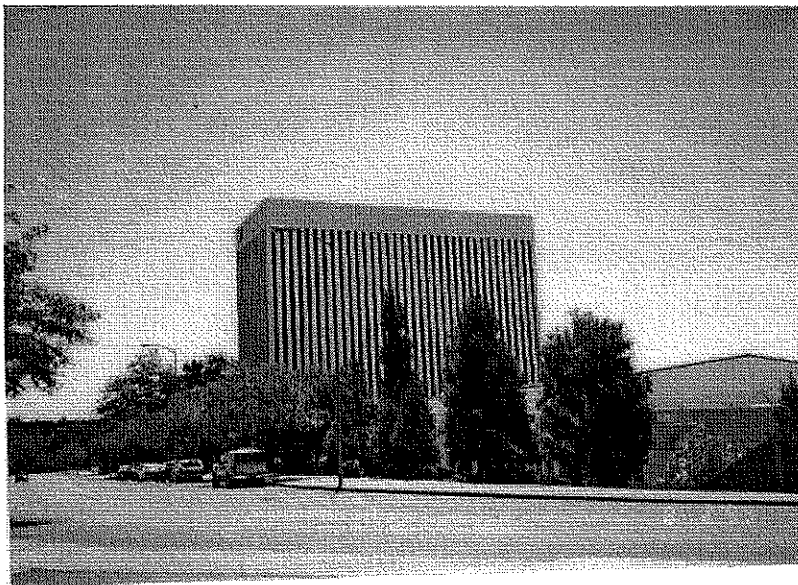
Preliminary: USGS, filtered at $f_{HP} = 0.1$ Hz; digitized at $f_{Nyq} = 100$ Hz; FF/str. data not synchronized

Final: decimated by 8 to $f_{Nyq} = 12.5$ Hz, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A5.LP0).

Figures

A5.1-A5.3
A5.LP0-6

Site Plan, Sensor Locations, Generalized Soil Column
LP Analysis Results



View of city hall from FF north

DISCUSSION

Site Description

The building is 13 stories tall above ground level and has no basement, though portions of the ground floor are partially below grade. The building is rectangular in plan view with dimensions 77.5 by 177.5 feet. Two-story concert hall and parking garage structures are located about 95 feet north and 75 feet south of the building, respectively. The building's lateral load resisting system consists of a reinforced concrete frame, with a concrete block wall in the longitudinal (NS) direction and two steel-stud walls with X-bracing in the transverse (EW) direction (see Fig. A5.2). The NS concrete block wall is only present from the 3rd floor to the roof. The foundation at the west end of the building consists of 25-foot deep reinforced concrete belled piers tied together at their heads with grade beams, while the east side foundations are footings cast directly into rock (Hamburger and Cole, 1992). Due to concerns about the seismic stability of the building, the structure was abandoned in February, 1991. As shown in Fig. A5.2, accelerographs are present on the ground, 3rd, 7th, and roof levels of the structure. Two free-field instruments are present at the site, one about 470 feet north of the building (FF north) and the other about 500 feet southeast of the building (FF south).

The site is located near the base of hills in eastern Hayward, and the terrain near the building slopes gradually down to the west. Geologic conditions have been established by a number of geotechnical investigations (Gibbs et al., 1994; Judd Hall and Assoc., 1976; Woodward Clyde Sherard Assoc., 1967), and as shown in Fig. A5.3 generally consist of about 5 feet of sandy silt overlying weathered and sheared rhyolite and gabbro bedrock which extends to the maximum depth explored of 98 feet. These conditions are representative of the free-field sites and the east side of the building; at the west end of the building, soil thicknesses increase to about 20 to 30 feet. The shear wave velocities in Fig. A5.3 are based on in situ downhole measurements by Gibbs et al. (1994).

Instrument Selection and Free-Field Conditions

As shown in Fig. A5.2, instrumentation is generally located near the centroid and the west end of the building. Data at the centroid locations was used in the analyses (Table A5.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A5.LP1.

Table A5.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 15	Ch 13	Ch 14	Ch 2	Ch 3

No large structures are located close to either free-field instrument, though both are near slopes. Accelerograms are not available for the FF south instrument from the LP. The FF north instrument is near the base of a 30 to 40-foot tall 1.5-2H:1V cut slope into rock. Based on an array of borings in the area as well as observed bedrock outcroppings, the geologic conditions near the free-field instruments appear to be similar to those at the east end of the building.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A5.LP4) and the spectra for horizontal motions (Fig. A5.LP2-3) generally indicate no significant filtering of GF motions relative to the FF.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A5.LP5. No amplification is observed near the first-mode frequencies for lateral response, suggesting that base rocking was unlikely to have significantly contributed to the structural response. No assessment was made in this study of potential rocking at the base of the longitudinal concrete block shear wall present between the 3rd floor and roof, as such rocking would be associated principally with deflections of the 3rd floor diaphragm.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A5.LP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A5.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A5.LP6).

Table A5.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=6, j=8		L (EW) d=3, j=6		tr (NS) d=5, j=8		L (EW) d=3, j=6	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	0.88	3.2	1.19	4.1	0.87	3.2	1.17	3.9	0.87	3.6	1.15	3.9
	2					2.55	5.4	3.50	5.7	2.55	4.5	3.56	4.2
	3					4.09	6.0	6.08	3.7	3.98	7.0	6.01	3.9

Comments on the analytical results follow:

- As seen from the power spectra of the horizontal roof accelerations (Fig. A5.LP2), the structural response in the transverse direction is dominated by the first two modes, while the longitudinal response is dominated by the first mode.
- The small variation between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for a rocking response in the vertical GF/FF transmissibility function.

The RPEM results (Fig. A5.LP6) indicate no significant time-dependent changes in the first-mode parameters of the structure.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A5.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 66 feet using the recorded free-field motions and the profile in Fig. A5.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 1.5 and 2.0, respectively, to account for shape effects.

Table A5.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 2210$ fps	$h = 84$ ft	emb.=0	$\gamma = 0.15$
LP	$\nu = 0.4$		tr: $r_1 = 66$ ft		$r_2 = 54$ ft
	$\beta^* = 0.8\%$	$V_s^* = 2210$ fps	L: $r_1 = 66$ ft		$r_2 = 82$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A5.3 with the fixed-base modal parameters in Table A5.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A5.4.

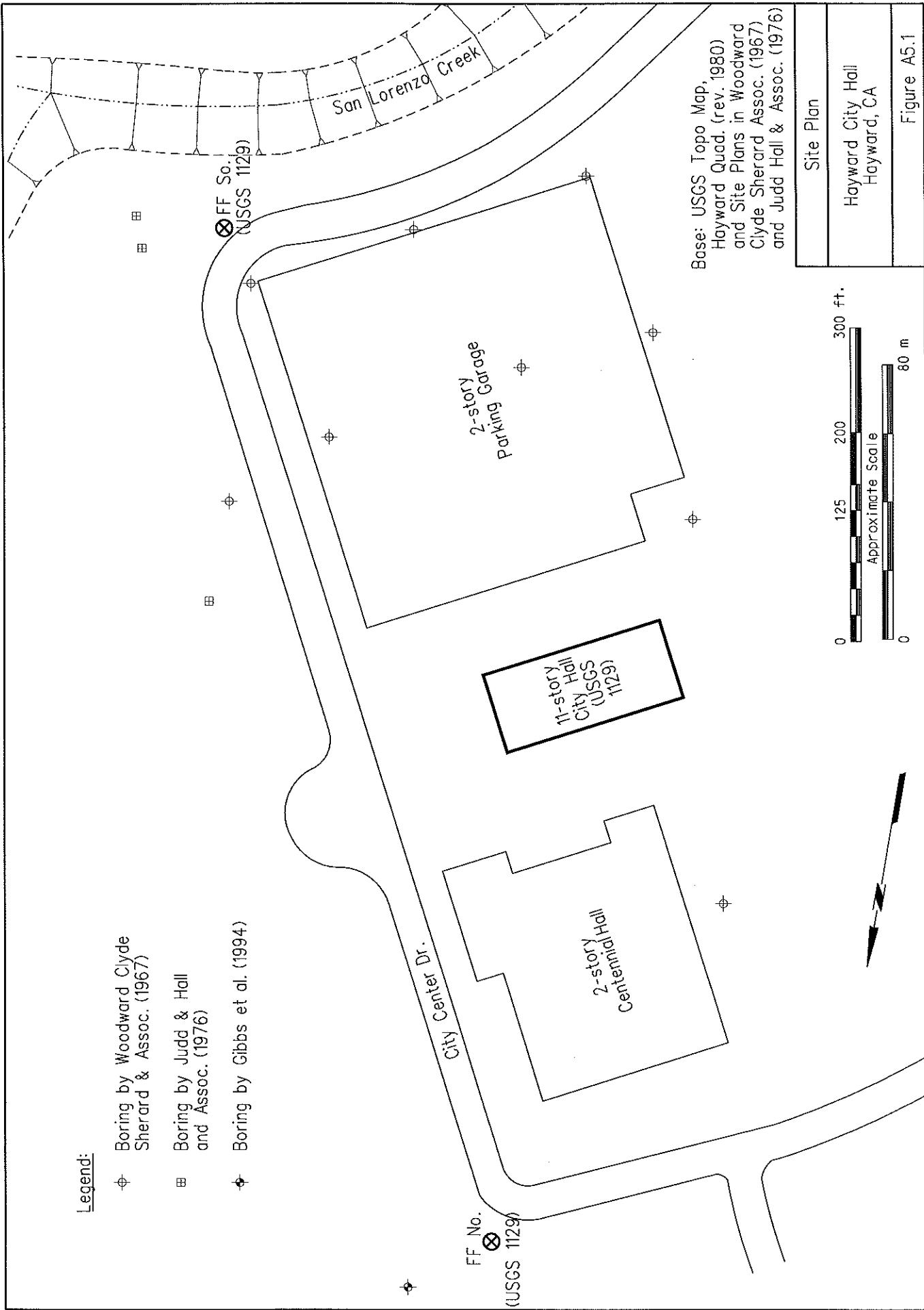
Table A5.4: Empirical and predicted inertial interaction effects

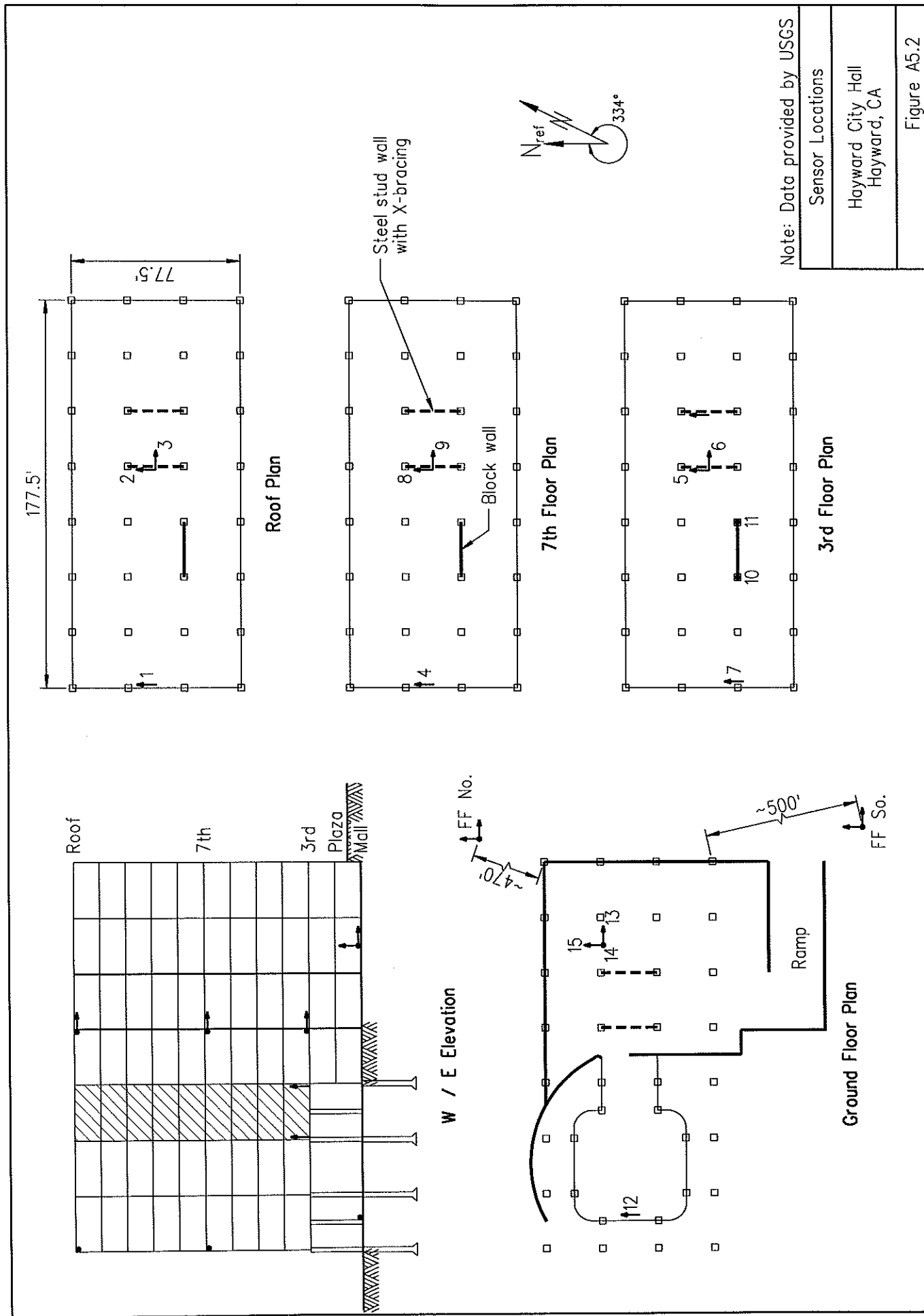
LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.014	0.5	3.6	1.033	0.2	3.9
	Prediction	1.007	0.0	3.2	1.006	0.1	4.1

The period lengthening and foundation damping are slightly underestimated, but the minor influence of SSI on the response of this building was satisfactorily predicted.

Data Sources

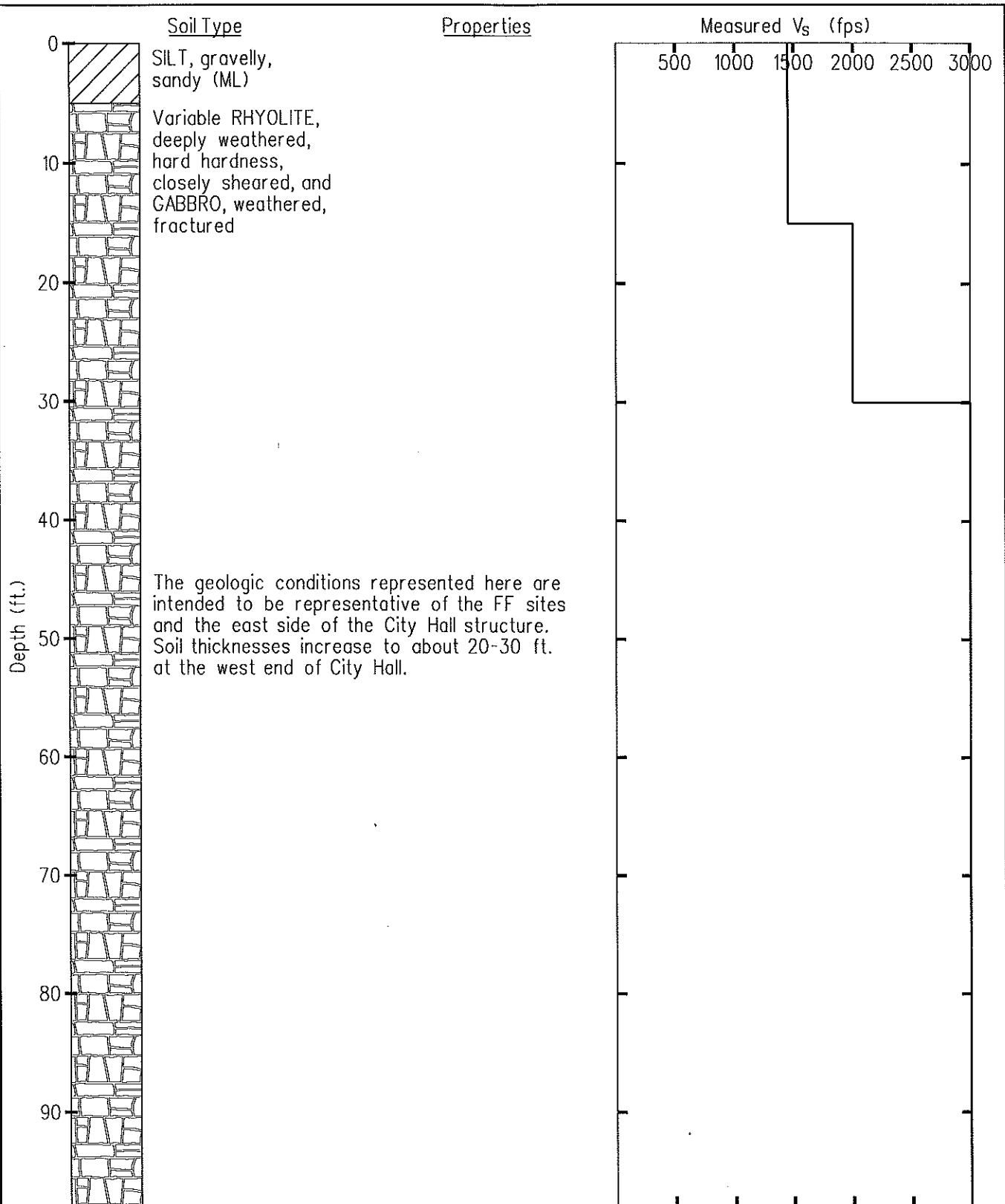
- Gibbs, J.F., Fumal, T.E., and Powers, T.J. (1994). "Seismic velocities and geologic logs from borehole measurements at 7 strong motions stations that recorded the 1989 Loma Prieta Earthquake," *Open-File Report 94-222*, U.S. Geological Survey.
- Hamburger, R.O. and Cole, C.A. (1992). "Seismic retrofit of Hayward City Center Building," *Proc. 2nd Conf. on Earthquake Hazards in the Eastern San Francisco Bay Area*, G. Borchardt et al. (ed.), California Dept. of Conservation, Div. Mines and Geology Sp. Pub. 113, 519-524.
- Judd Hull and Associates (1976), "Soil investigation report for proposed residential village, Hayward City Center, Hayward, CA," *Proj. No. 181-A28-E6*.
- Woodward Clyde Sherard and Associates (1967). "Soil Investigation for the proposed Civic Plaza Parking Structure," *Proj. No. S-10893A*, and "Soil Investigation for the proposed Civic Plaza Office Building, Hayward, CA," *Proj. No. S-10893*.





Note: Data provided by USGS

Sensor Locations
Hayward City Hall
Hayward, CA
Figure A5.2



Data Sources:
Gibbs et al. (1994),
Woodward Clyde
Sherard & Assoc.
(1967), and Judd Hall
Assoc. (1976)

Notes:

1. Groundwater encountered at 30-40 ft. depth.
2. Shear wave velocities based on downhole measurements by Gibbs et al. (1994).

Generalized Soil Profile

Hayward City Hall
Hayward, CA

Figure A5.3

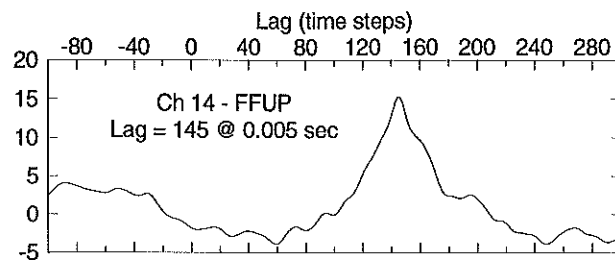


Fig. A5.LP0: Cross-correlation function between vertical ground level and free field accelerations

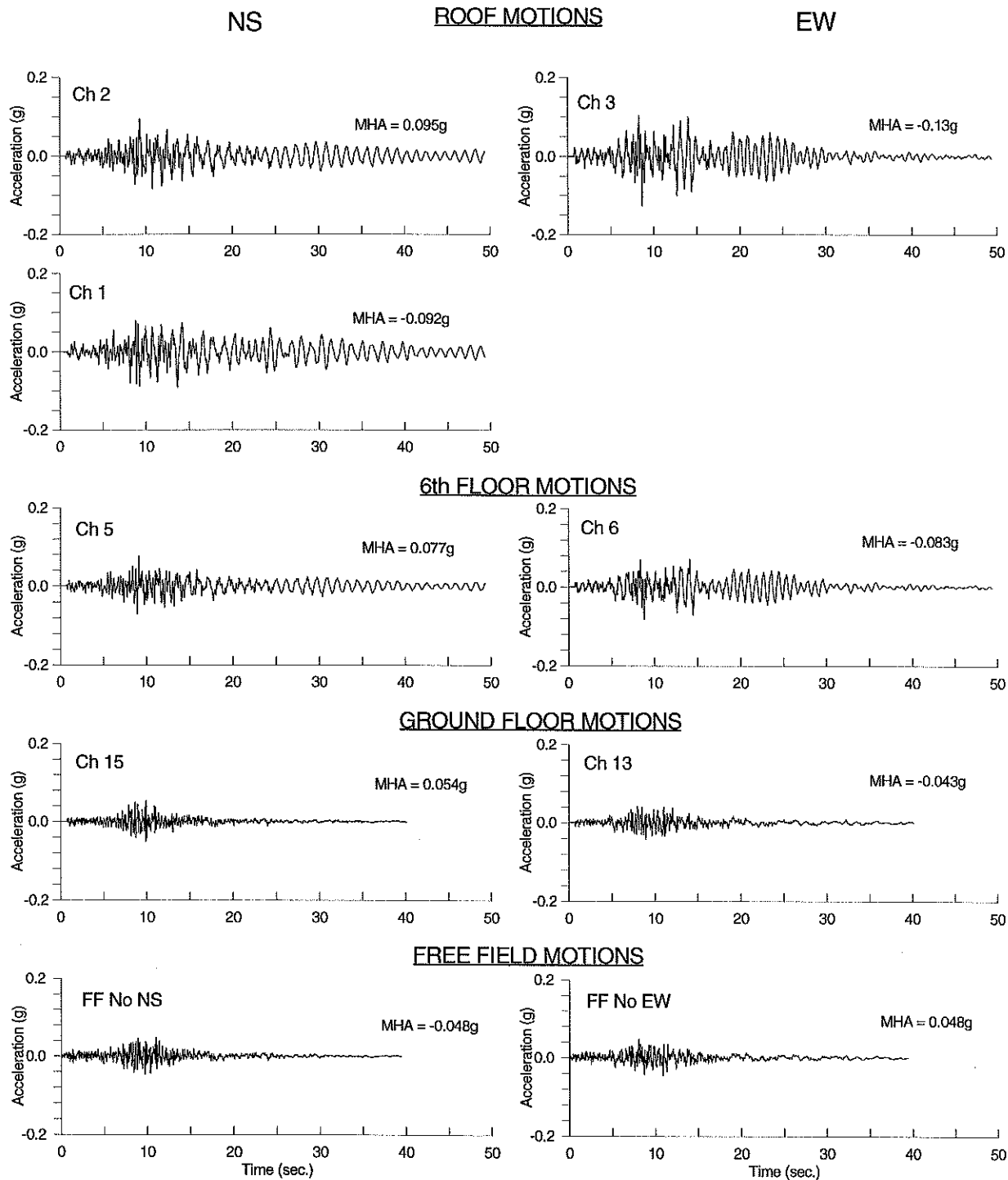


Fig. A5.LP1: HAYWARD CITY HALL
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake
A5-8

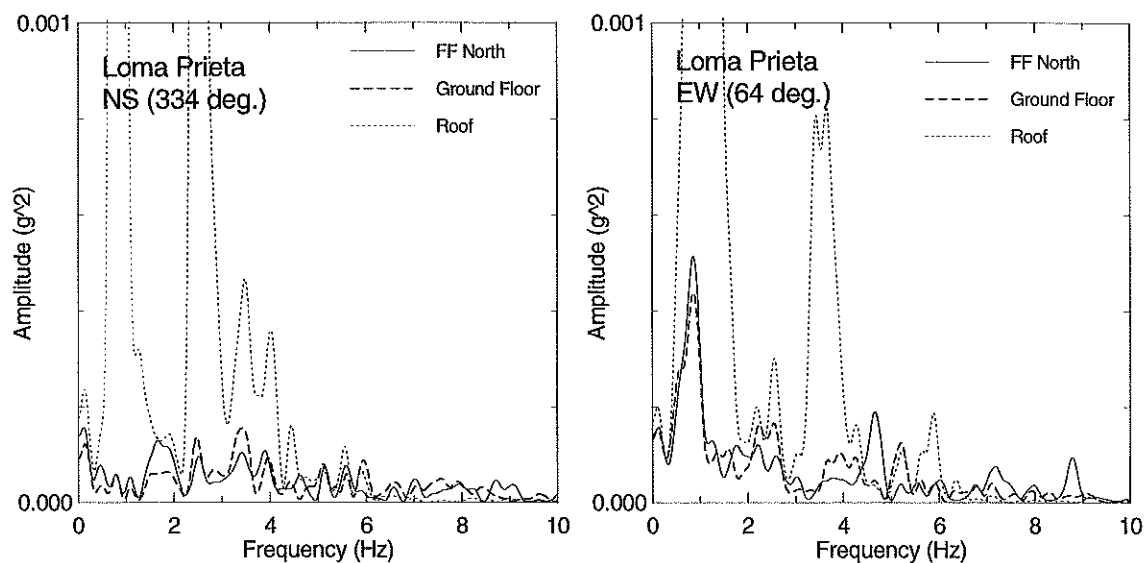


Fig. A5.LP2: HAYWARD CITY HALL
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

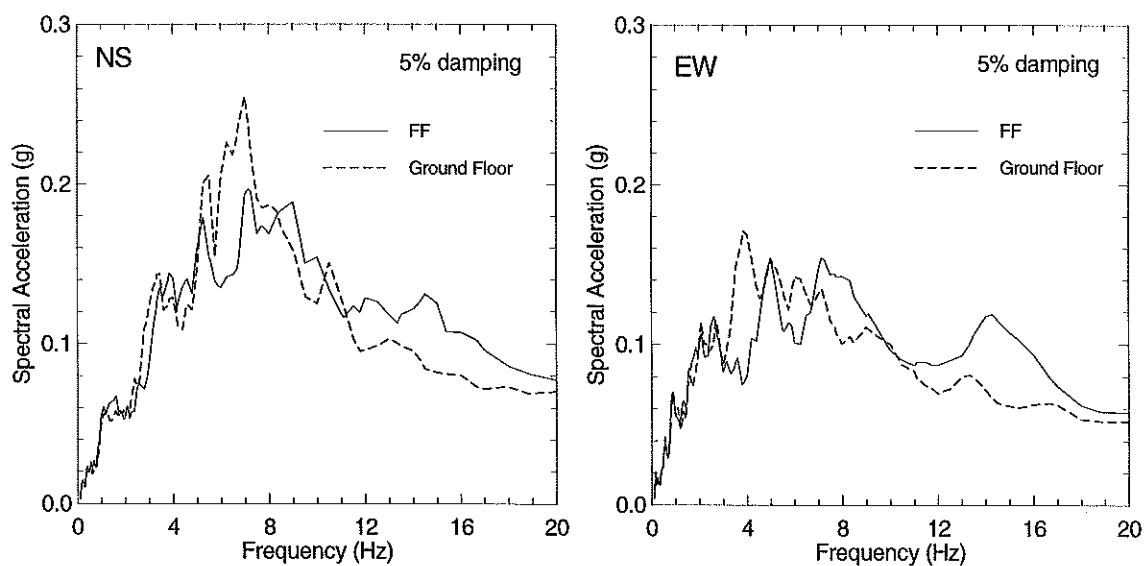


Fig. A5.LP3: HAYWARD CITY HALL
Acceleration Response Spectra of Horizontal Ground Floor and Free-Field Motions
1989 Loma Prieta Earthquake

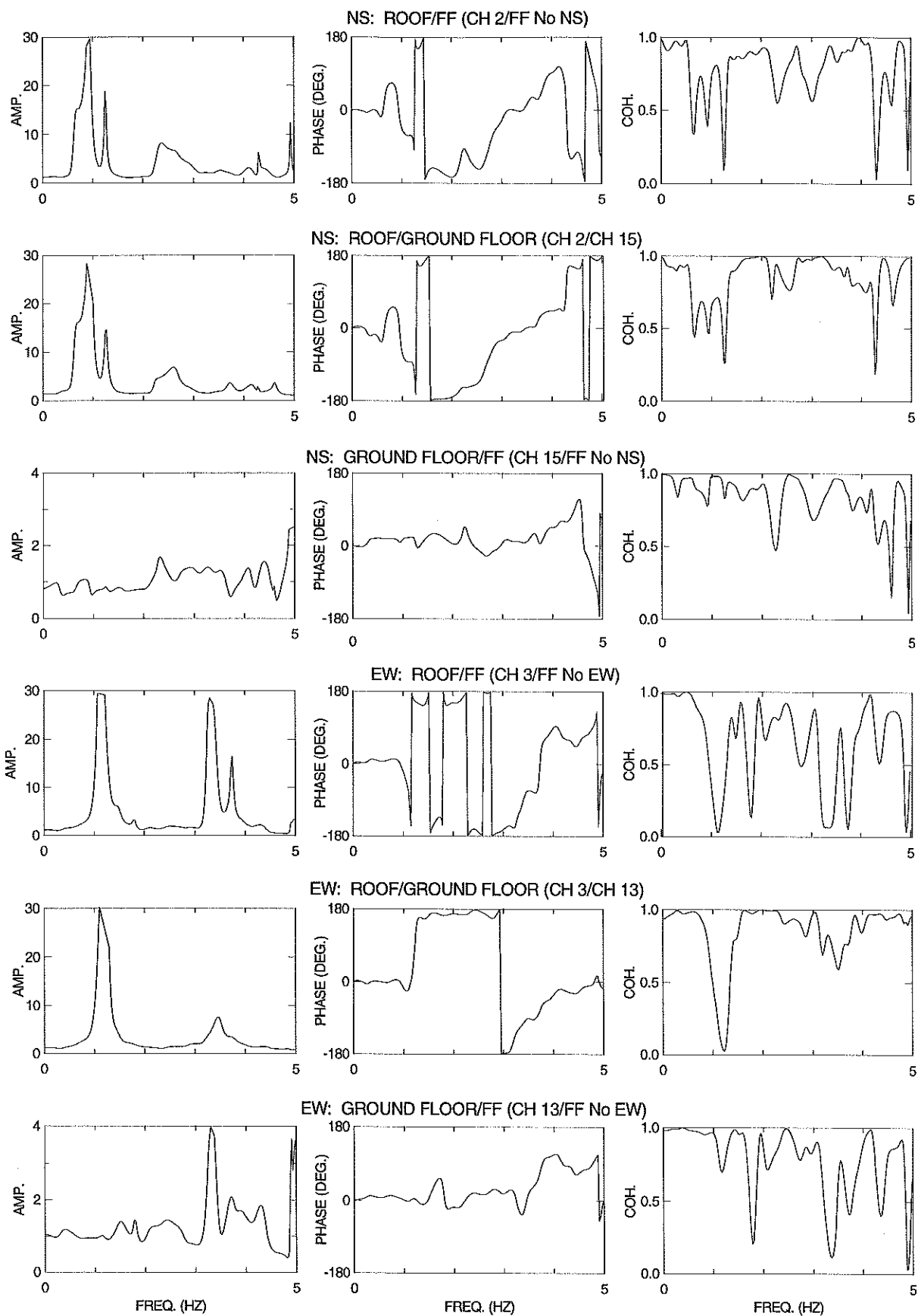


Fig. A5.LP4: HAYWARD CITY HALL
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A5-10

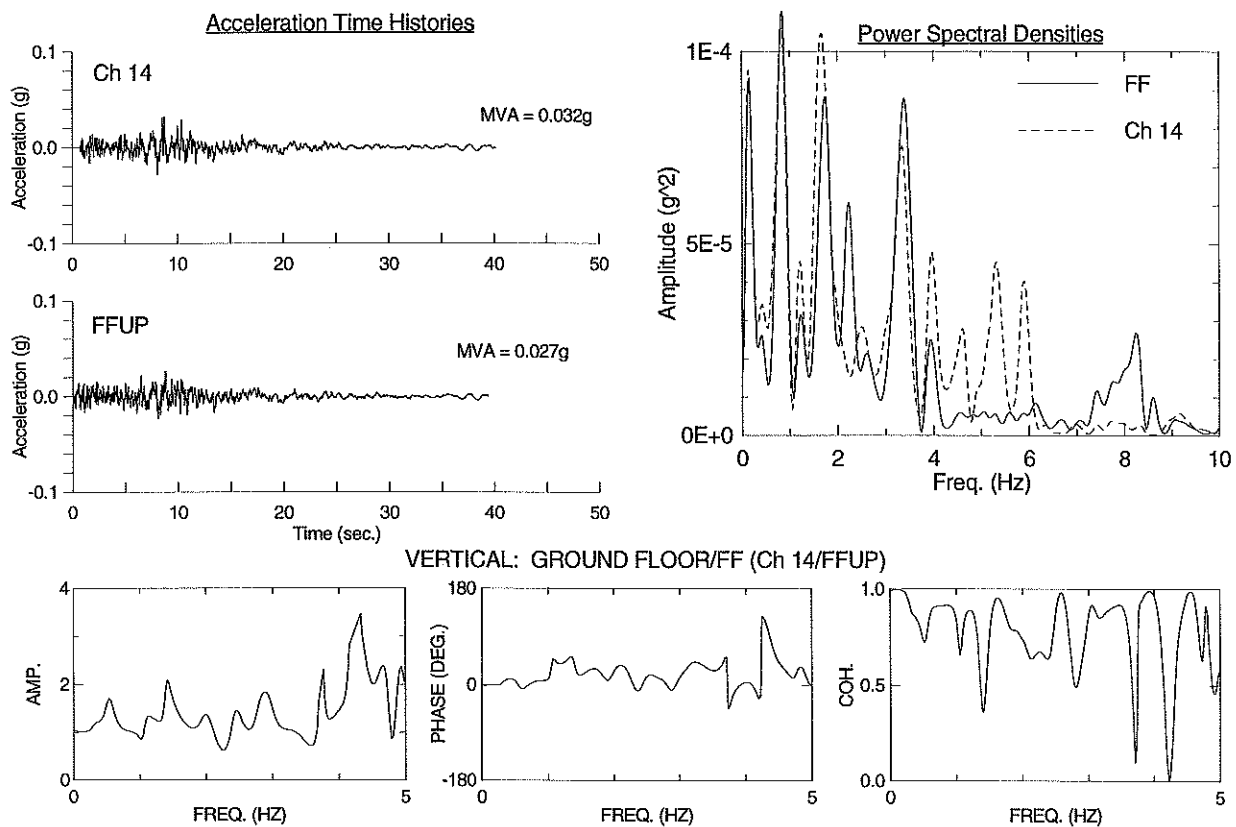


Fig. A5.LP5: HAYWARD CITY HALL
Analysis of Vertical Motions
1989 Loma Prieta Earthquake

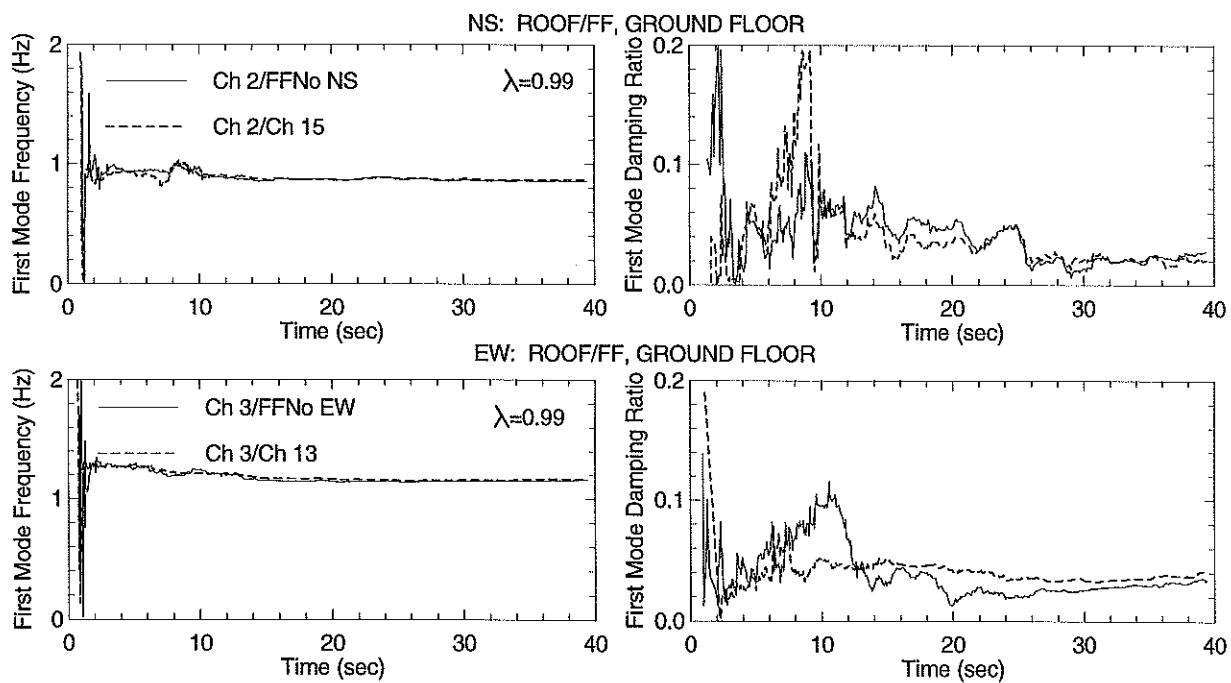


Fig. A5.LP6: HAYWARD CITY HALL
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE A6

Site Name: Hayward CSU Administration Building
Sensors: CSMIP 58354/58219

Lat.: 37.655°
Long.: 122.056°

Floor Levels: 13
Bsmt. Levels: 0
Structure: Concrete and steel moment
frames and concrete shear walls

Foundation: concrete piers
Site Geology: rock
Design Date: 1969

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.085	0.066	0.045	0.068	0.079	0.042	0.15	0.24	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f=0.4-23$ Hz (structure channels), $0.16-23$ Hz (free-field); $f_{Nyq}=25$ Hz; FF/str. data not synchronized; FF data not aligned with structure

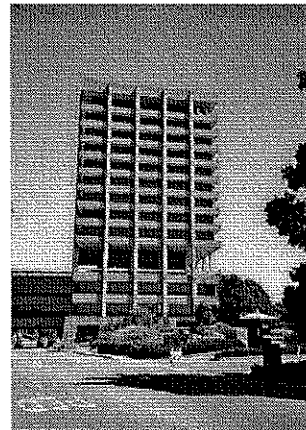
Final: FF data rotated 40° CCW to align with str.; data decimated by 2 to $f_{Nyq}=12.5$ Hz; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A6.LP0).

Figures

A6.1-A6.3
A6.LP0-6

Site Plan, Sensor Locations, Generalized Soil Column
LP Analysis Results

Right: Structure from West Loop Rd.
Below: View of FF (in shack) and structure



DISCUSSION

Site Description

The building is 13 stories tall above ground level and has no basement. The building is 112.5-foot square in plan view, and is attached to a 3-story library structure on its east side through a bi-level pedestrian bridge. The bridge is free to move on friction bearings where it joins the library structure (Celebi, 1994). Other than the library, no other significant structures are present near the building. Lateral loads are resisted by an interior core steel moment frame and an exterior perimeter concrete moment frame, as well as concrete shear walls around the elevator to the 2nd floor. The foundation consists of piles in groups of 10 to 18 beneath each column with roughly 3.5- to 5-foot thick pile caps, and an 18-inch thick concrete slab at the ground floor level. The depth of the piles is not known, but based on the construction specifications, the piles could be as long as 60 feet. As shown in Fig. A6.2, accelerographs are present on the ground, 1st, 2nd, 5th, and roof levels of the structure. The free-field instrument is located about 1260 feet west of the structure near the campus football stadium.

The site is located in the hills of eastern Hayward, with the building situated on a broad level pad established by cut/fill grading. As shown in Fig. A6.3, subsurface conditions at the free-field site consist of: 10 feet of clayey fill, about 30 feet of deeply weathered greenstone bedrock, and moderately weathered and fractured greenstone bedrock which extends to the maximum depth explored of 100 feet. (Gibbs et al., 1976). The shear wave velocities shown in Fig. A6.3 were measured at the free-field site using downhole techniques. No geologic data could be retrieved for the structure site. However, based on the significant pile penetration (60 feet) anticipated by the building's designers, the building may be underlain by several tens of feet of soil before rock is encountered. Reconnaissance of the site suggests the building is located on a cut/fill grading pad, though it is not clear whether the building is in a cut or fill area.

Instrument Selection and Free-Field Conditions

As shown in Fig. A6.2, instrumentation is located at the centroid and at the east end of the building. Data from the centroid locations was used in this study, as indicated in Table A6.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A6.LP1.

Table A6.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 16	Ch 14	Ch 15	Ch 3	Ch 4

No significant structures are located near the free-field instrument, but the instrument is about 15 feet from the top of an approximately 2H:1V slope. Studies by Ashford and Sitar (1994) have shown that less than 20% topographic amplification of MHA occurs at the top of slopes with gradients of 2H:1V or less. The poorly characterized geotechnical conditions at the building site are more problematic. Different ground conditions at the free-field and structure locations would significantly contribute to incoherence in motions recorded at the two locations.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A6.LP4) and the spectra for horizontal motions (Fig. A6.LP2-3) indicate significant amplification of NS and EW GF motions relative to the FF near the second mode structural response frequency ($f \approx 2$ Hz). De-amplification of GF motions was observed near the first-mode frequency ($f \approx 0.8$ Hz) and at higher frequencies ($f > 4$ Hz). The cause of the amplification at 2 Hz is unknown, but may be associated with SSI effects or a site frequency at the building location.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A6.LP5. Minor amplification is observed near the first and second mode frequencies for lateral response, suggesting that rocking may have contributed to the structural response. However, given the location of the vertical instrument near the centroid of the structure, no firm conclusions about rocking can be drawn from these results.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A6.LP4), (2) modal parameters from CEM analyses (Table A6.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A5.LP6).

Table A6.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=4, j=10		L (EW) d=5, j=8		tr (NS) d=4, j=10		L (EW) d=5, j=8	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	na	na	na	na	0.78	3.3	0.75	2.7	0.79	3.2	0.78	4.0
	2					2.37	3.6	2.21	4.1	2.25	7.7	2.27	1.8
	3					4.00	5.1	3.97	4.5	3.82	9.5	3.84	1.6

Comments on the analytical results follow:

- As indicated in Fig. A6.LP2, roof accelerations in both directions are dominated by the first two modes at $\tilde{f} \approx 0.8$ and 2.3 Hz, respectively. The modal parameters in Table A6.2 are similar to those reported by Celebi (1994).
- Fixed-base modal parameters for the NS and EW directions could not be estimated from the CEM results because $\tilde{f}_1 > \tilde{f}_1^*$. The difference between fixed- and flexible-base parameters may be significant as a moderate rocking effect was apparent in the GF/FF transmissibility functions. The condition of $\tilde{f}_1 > \tilde{f}_1^*$ may have resulted from the recorded FF motions not properly representing the true FF shaking at the building site (due to excessive spatial

incoherence possibly associated with different ground conditions at the FF and building sites). Incoherence effects are evident from differences in the FF and GF acceleration time histories and from low coherencies for GF/FF data pairs (Fig. A6.LP4).

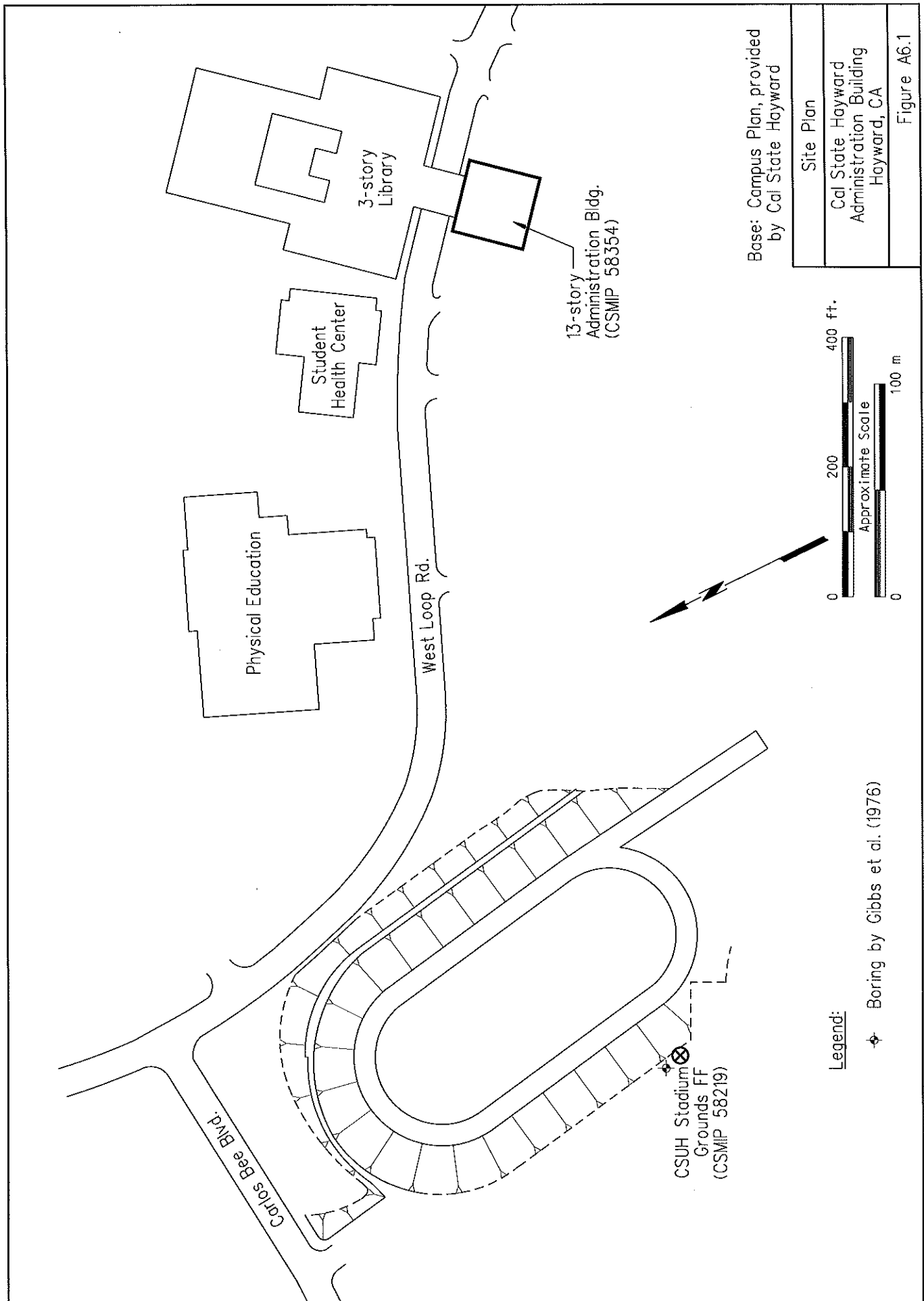
The RPEM results (Fig. A6.LP6) indicate no significant time-dependent changes in the first-mode parameters of the structure.

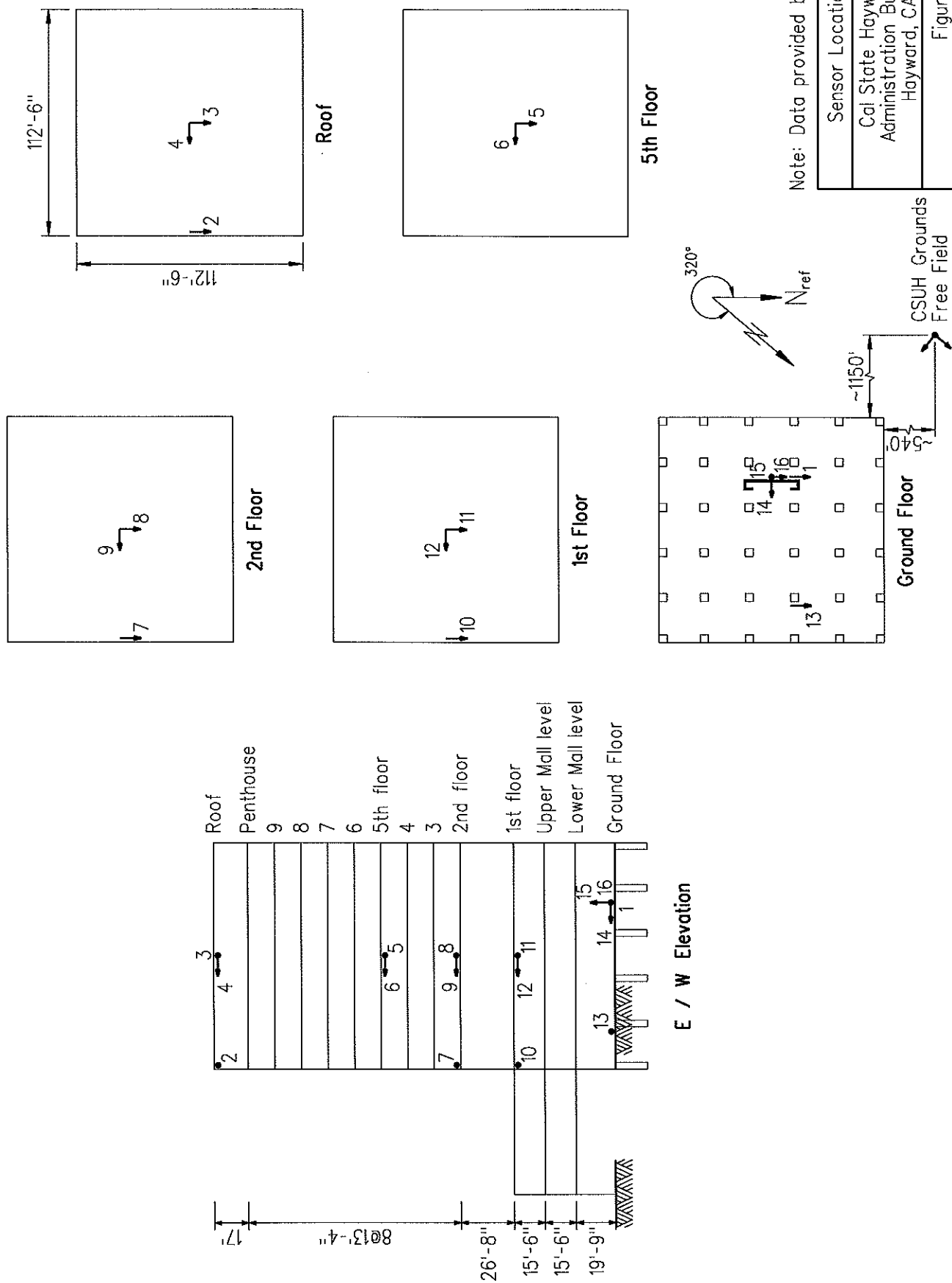
(c) Comparison of Empirical and Predicted Inertial Interaction Effects

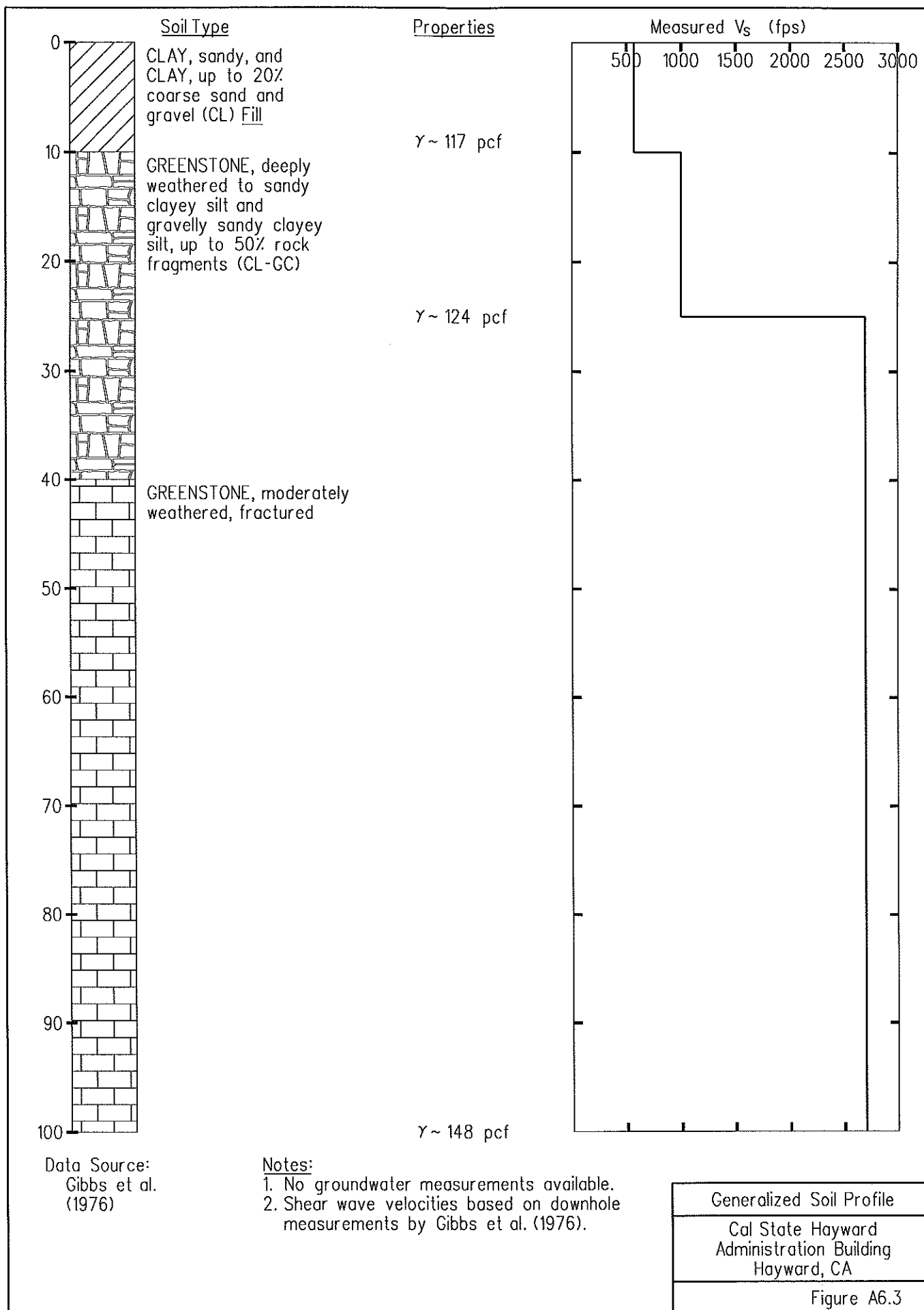
As fixed-base modal parameters could not be estimated, an empirical quantification of inertial interaction effects was not possible.

Data Sources

- Ashford, S.A. and Sitar, N. (1994). "Seismic Response of Steep Natural Slopes," *Rpt. No. UCB/EERC-94/05*, Earthquake Engrg. Res. Cntr., University of California, Berkeley, May.
- Celebi, M. (1994). "Seismic response of a 13-story building," *5th U.S. National Conf. on Earthquake Engrg.*, EERI, Chicago, IL, 87-96.
- Gibbs, J.F., Fumal, T.E., and Borchardt, R.D. (1976). "In-situ measurements of seismic velocities in the San Francisco Bay Region...Part III," *Open File Report 76-731*, U.S. Geological Survey







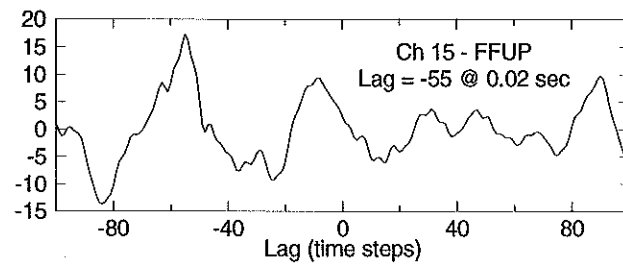


Fig. A6.LP0: Cross-correlation function between vertical ground floor and free-field accelerations

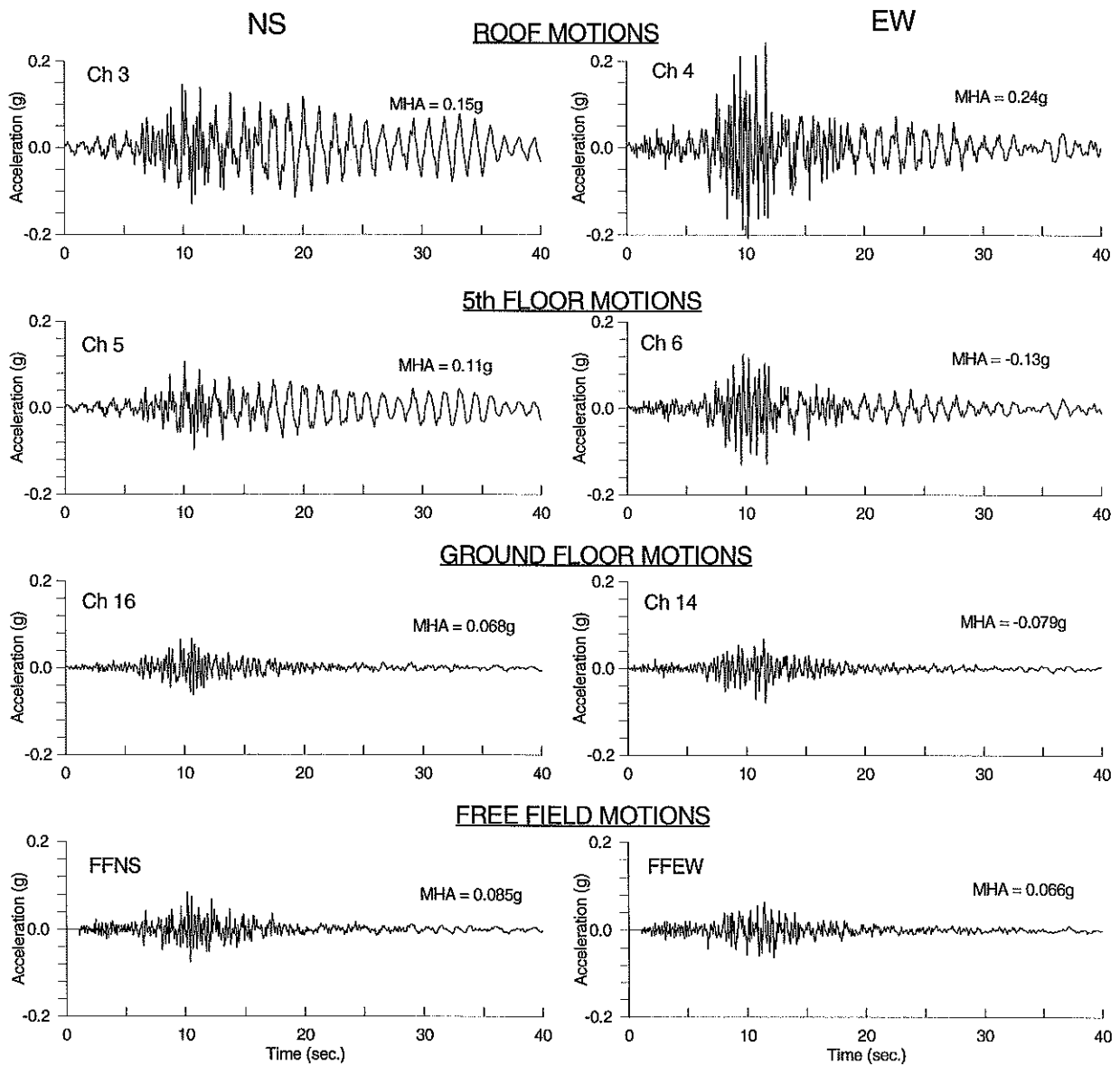


Fig. A6.LP1: HAYWARD CSU ADMINISTRATION BUILDING
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

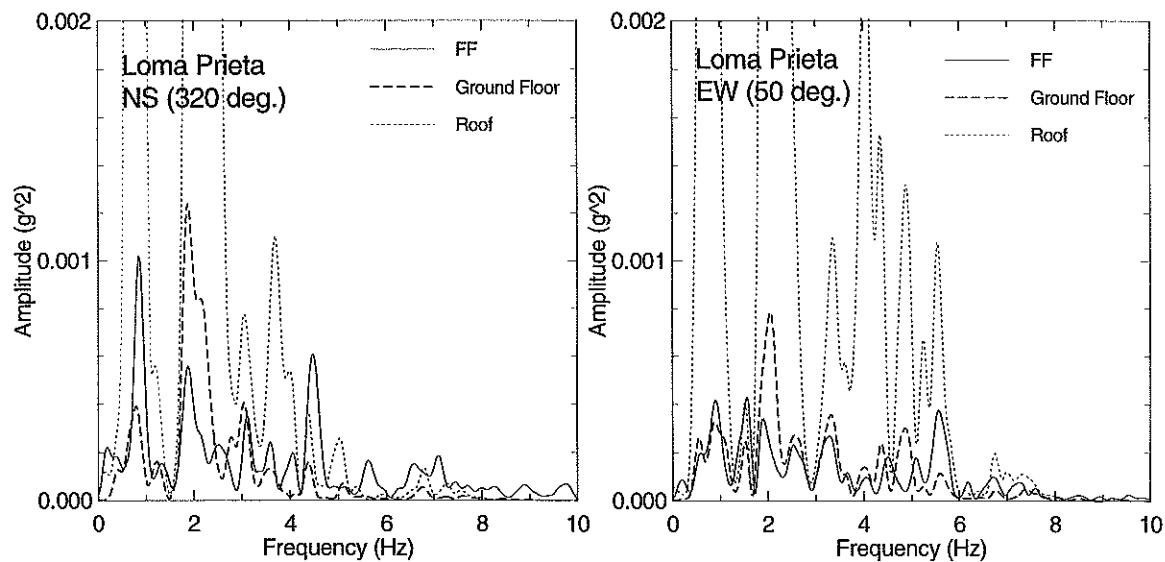


Fig. A6.LP2: HAYWARD CSU ADMINISTRATION BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

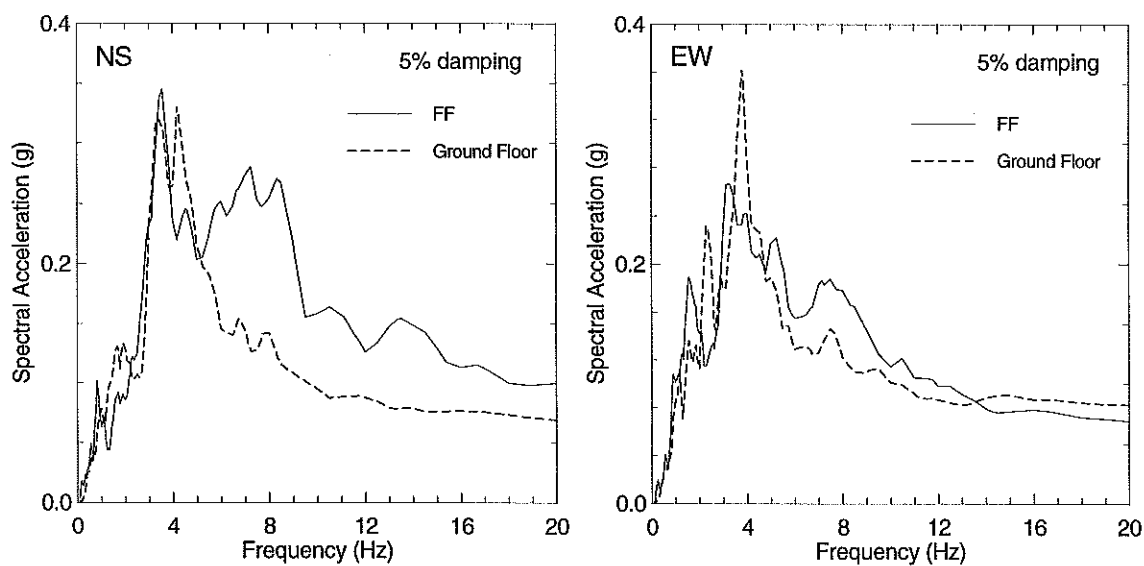


Fig. A6.LP3: HAYWARD CSU ADMINISTRATION BUILDING
Acceleration Response Spectra of Horizontal Ground Floor and Free-Field Motions
1989 Loma Prieta Earthquake

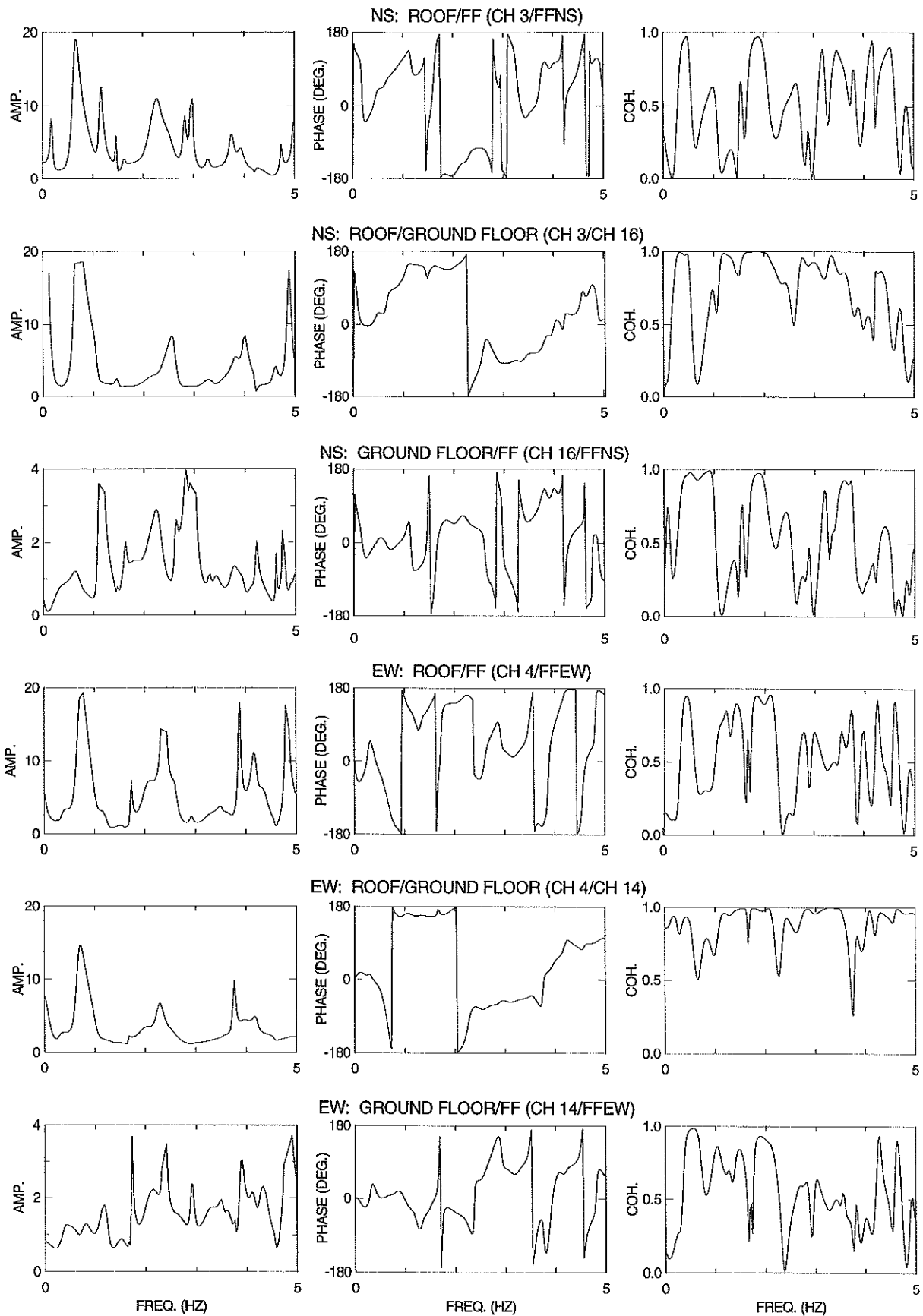


Fig. A6.LP4: HAYWARD CSU ADMINISTRATION BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A6-10

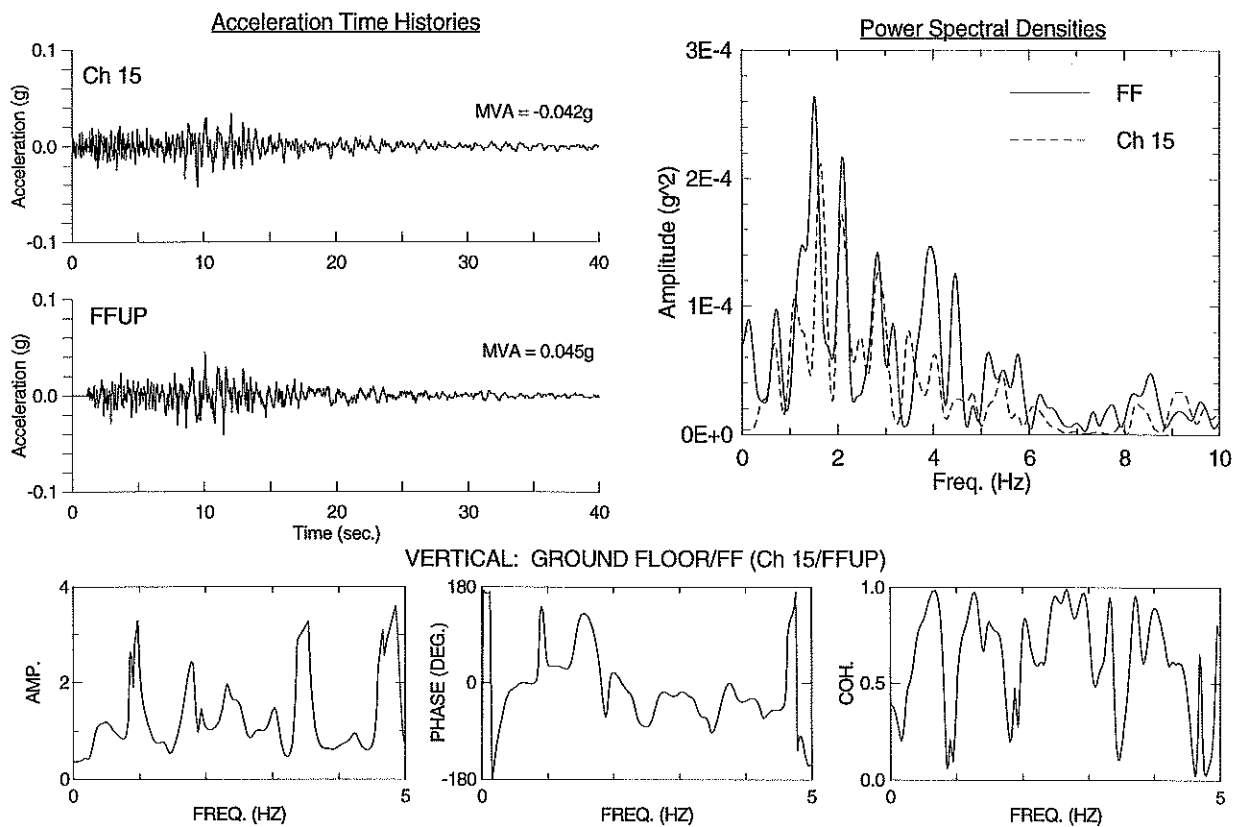


Fig. A6.LP5: HAYWARD CSU ADMINISTRATION BUILDING
Analysis of Vertical Motions
1989 Loma Prieta Earthquake

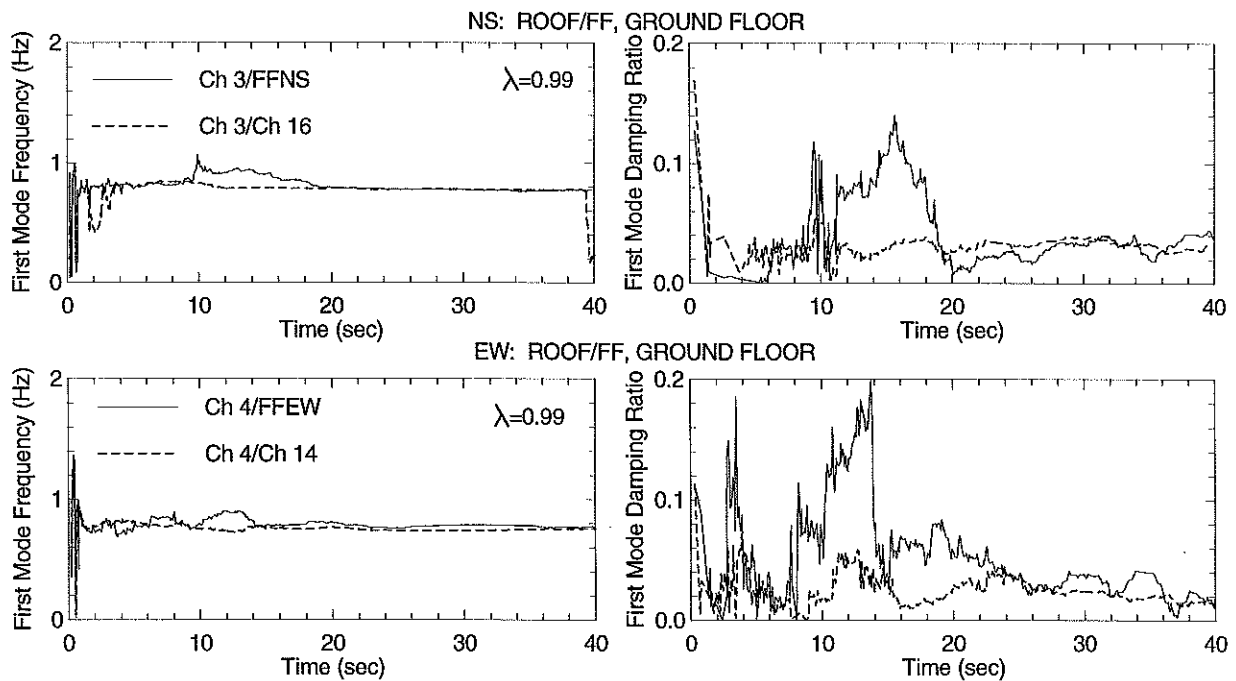


Fig. A6.LP6: HAYWARD CSU ADMINISTRATION BUILDING
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE A7

Site Name: Hollister Warehouse
Sensors: CSMIP 47391/47524

Lat: 36.851°
Long.: 121.398°

Floor Levels: 1
Bsmt. Levels: 0
Structure: plywood roof diaphragm and
concrete tilt-up shear walls

Foundation: footings
Site Geology: Alluvium
Design Date: 1979

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.36	0.22	0.20	0.36	0.25	0.17	0.44	0.79	--

-- = No instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.14\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized; FF data not aligned with str.

Final: FF data rotated 25° CCW to align with str.; FF/str. data synchronized using cross correlation of vertical accelerations (Fig. A7.LP0)

Figures

A7.1-A7.3
A7.LP0-6

Site Plan, Sensor Locations, Generalized Soil Column
LP Analysis Results



View of structure, looking north

DISCUSSION

Site Description

The single-level building is about 30 feet tall with no basement, and is rectangular in plan view with dimensions 100 by 300 feet. Lateral loads are resisted by a plywood diaphragm roof supported on concrete tilt-up shear walls along the building perimeter. The foundation consists of spread footings. No significant structures are located near the building, but large storage containers are often stacked in a yard around the warehouse. As shown in Fig. A7.2, accelerographs are present on the ground floor and roof of the structure. The free-field instrument is located adjacent to the yard with the storage containers, and is about 400 feet south of the structure.

The site is located on level terrain at the southern end of the alluviated Santa Clara Valley. While no subsurface data is available for the warehouse site, a geotechnical investigation was conducted at the nearby Hollister City Hall by SW/AA (1980). The general soil conditions at the City Hall and warehouse sites are assumed to be similar. These soil conditions are characterized by 86 feet of stiff silty clay, 97 feet of stiffer sands and silts, and very dense sands which extend to the maximum depth explored of 349 feet. Soil conditions at greater depths and the depth to bedrock are poorly characterized in the area. The shear wave velocities in Fig. A7.3 were measured using downhole techniques.

Instrument Selection and Free-Field Conditions

Instrumentation was selected to enable the dynamic behavior of the tilt-up wall to be assessed both parallel and transverse to the wall lines. The sensors used to evaluate this behavior are indicated in Table A7.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A7.LP1.

Table A7.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (par. to wall)		Roof (perp. to wall)	
NS	EW	V	NS	EW	NS	EW
Ch 13	Ch 7, 8	Ch 1	Ch 10	Ch 3	Ch 11	Ch 4

It is not known whether storage containers were present near the free-field instrument at the time of the Loma Prieta Earthquake. If present, vibrations of the containers could have affected the free-field recordings at high frequencies. However, the free-field recordings have relatively little energy content at frequencies greater than about 4 Hz, indicating that any such contamination would have likely been minor.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A7.LP4) and the spectra of the horizontal motions (Fig. A7.LP2-3) generally indicate minor de-amplification of GF motions, though some amplification of EW motions near the first-mode frequency of 1.5 Hz was observed.

Transmissibility and power spectral density functions for FF and GF vertical motions are presented in Fig. A7.LD5. Though spiky, the transmissibility function amplitudes do not show amplification at the first-mode response frequency, indicating that base rocking is unlikely.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A7.LP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A7.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A7.LP6).

Table A7.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) d=1, j=5		L (NS) d=--, j=--		tr (EW) d=0, j=5		L (NS) d=--, j=--	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	1.50	17.3	na	na	1.45 1.50	17.5	na	na	1.49	20.6	na	na

Comments on the analytical results follow:

- The shear walls have a large in-plane stiffness. This is illustrated in R/GF transmissibility functions (Ch10/Ch13, Ch3/Ch8) by amplitudes and coherencies close to unity and nearly zero phase shift from 0 to 5 Hz. Out-of-plane roof sensors were used for parametric analyses, so the results reflect the flexibility of the plywood roof diaphragm.
- Parametric models for longitudinal response could not be developed due to the similarity of the input and roof motions in this direction.
- As discussed below, the CEM $\tilde{f}^*$ value of 1.45 Hz was adjusted based on RPEM results.
- The small variation between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for a rocking response in the vertical GF/FF transmissibility function.
- Though a value of $j = 5$ was used to develop parametric models of the structure, the first mode was dominant, and only these parameters are reported in Table A7.2. As indicated by the broad transmissibility function amplitude peaks, modal frequencies are poorly defined for this structure, and the values reported in Table A7.2 are approximate.
- A study of the Hollister Warehouse by Wood and Hawkins (1994) also found high in-plane wall stiffness, but significant out-of-plane response in the transverse direction.

Time-dependent reductions in the first-mode frequency were apparent from the RPEM results (Fig. A7.LP6) in the first 12 seconds of shaking in EW direction, as were subsequent frequency increases when the shaking intensity gradually declined. Unlike the CEM analyses, RPEM analyses indicated GF-R frequencies which are slightly higher than FF-R frequencies (i.e., as indicated in Table A7.2, a better estimate is $\tilde{f}^* \approx 1.5$ Hz). As with the CEM results, FF-R damping ratios consistently exceeded GF-R damping ratios in the RPEM analyses.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A7.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 97 feet using the recorded free-field motions and the profile in Fig. A7.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse direction were increased by 20% to account for shape effects.

Table A7.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 670$ fps	$h = 30$ ft	emb.=0	$\gamma = 0.15$
LP	$\nu = 0.4$		tr: $r_1 = 97$ ft		$r_2 = 75$ ft
	$\beta^* = 7.7\%$	$V_s^* = 500$ fps	L: $r_1 = 97$ ft		$r_2 = 130$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A7.3 with the fixed-base modal parameters in Table A7.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A7.4.

Table A7.4: Empirical and predicted inertial interaction effects

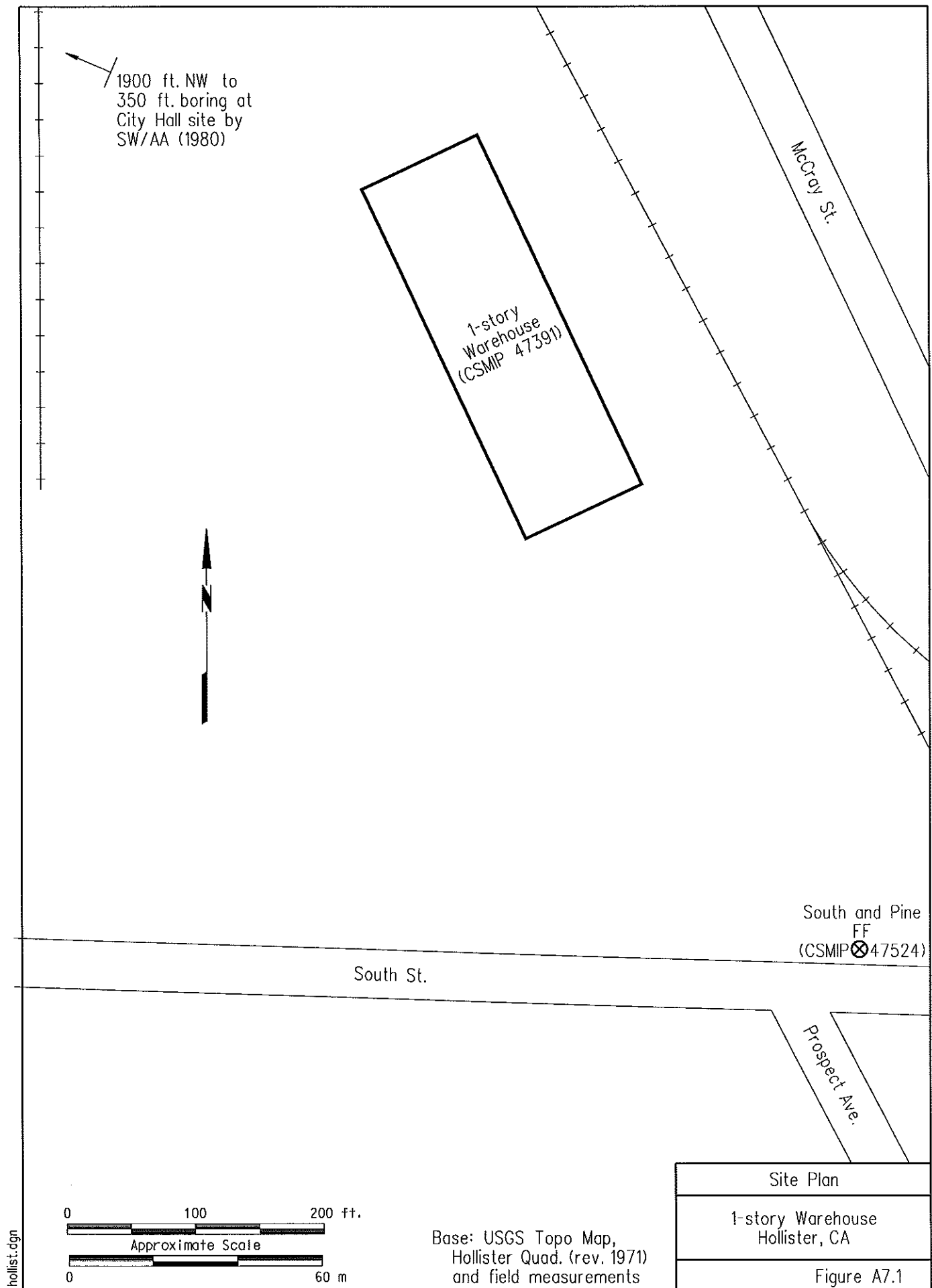
LP	Sys. ID	tr (EW)			L (NS)		
		$\tilde{T} / T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T} / T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Prediction	1.06	3.5	17.9	--	--	--

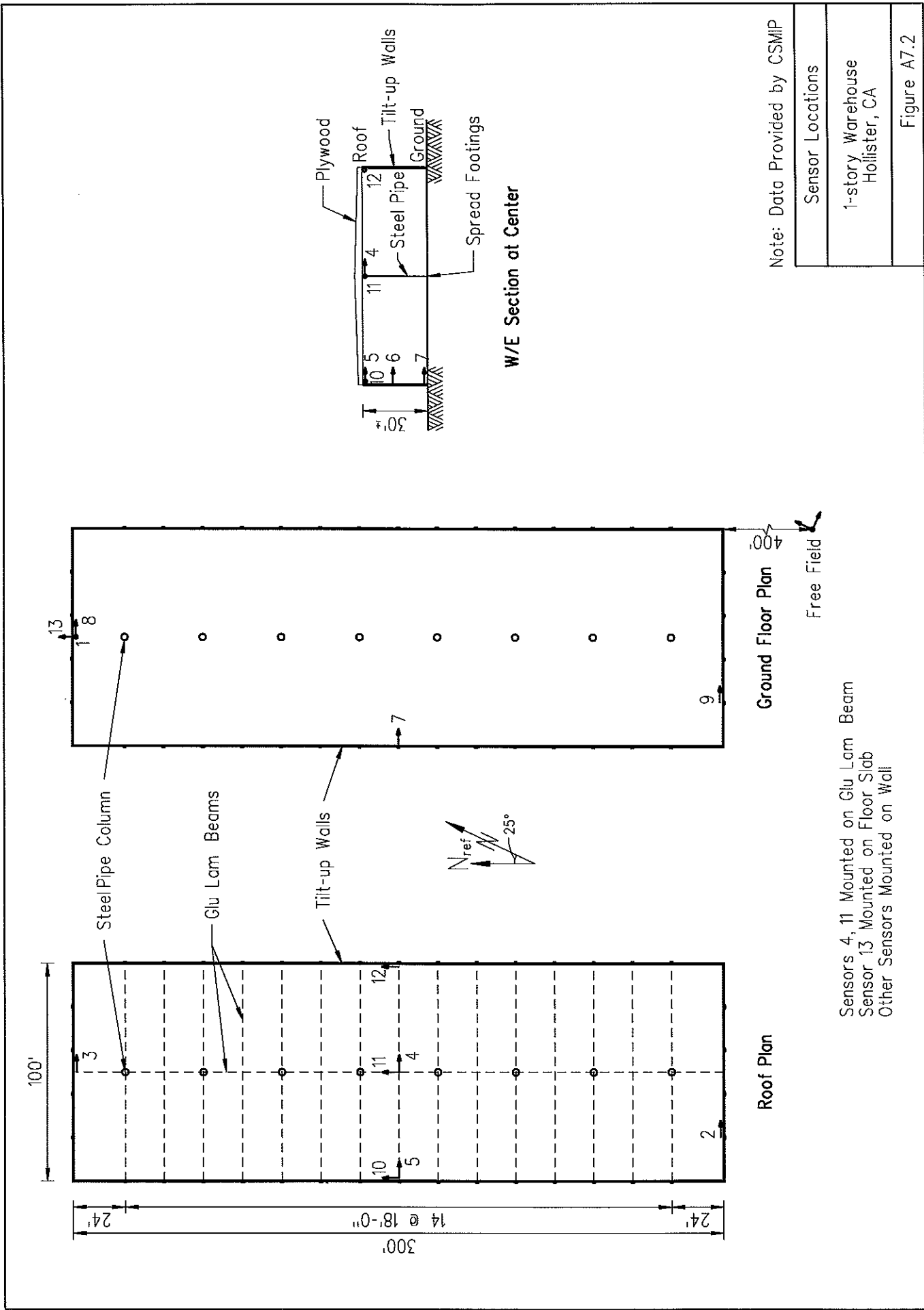
The predicted period lengthening and foundation damping are satisfactory given the uncertainty associated with the first-mode frequencies identified by the CEM.

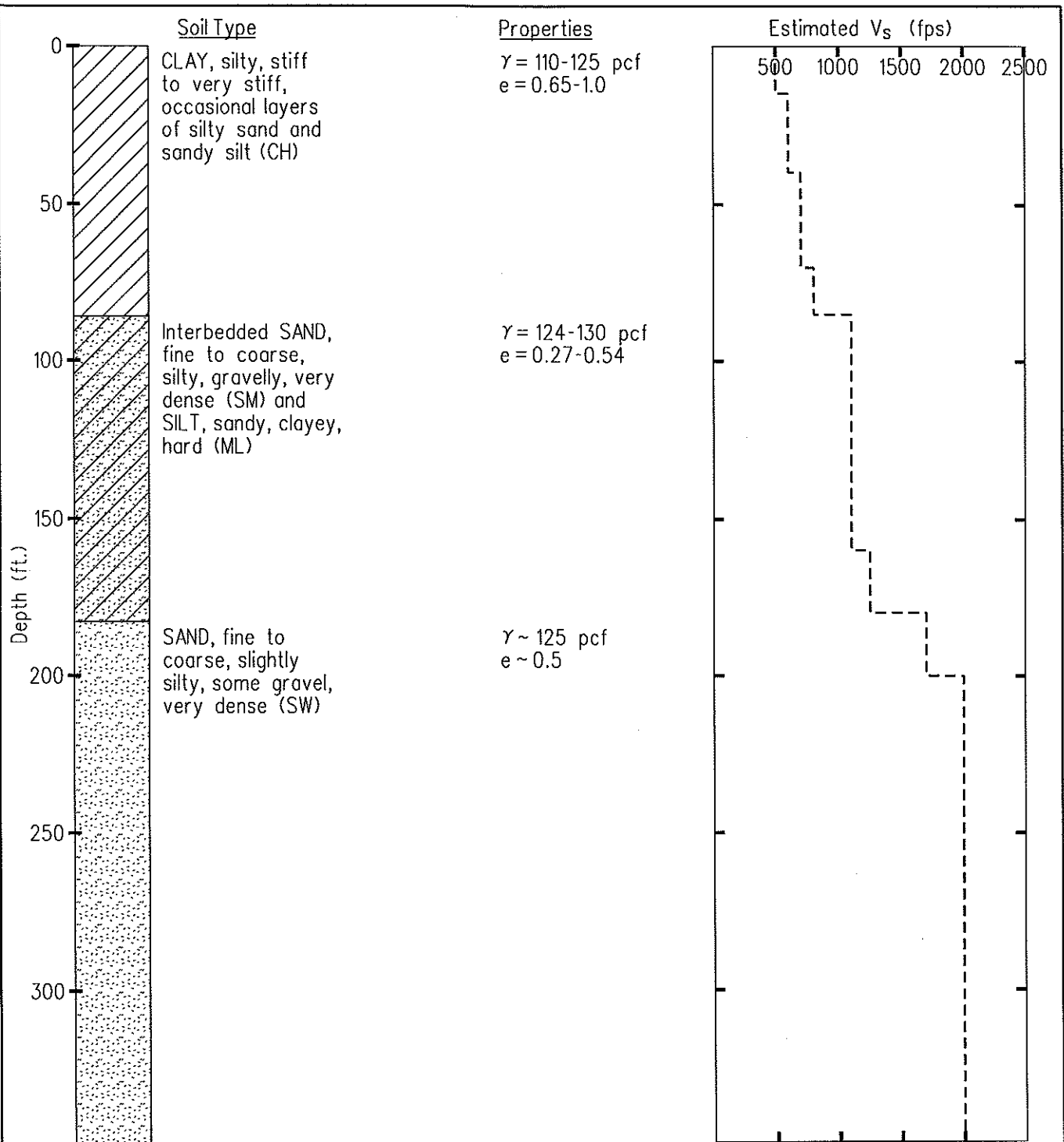
Data Sources

Wood, S.L. and Hawkins, N.M. (1994). "Seismic response of concrete tilt-up systems," *Proc. Structures Congress XII*, ASCE, Vol. 2, 1484-1489.

Shannon and Wilson/Agbabian Associates, SW/AA (1980). "Geotechnical and strong motion data from U.S. accelerograph stations," *NUREG/CR-0985*, Vol. 5, Prepared for the U.S. Nuclear Regulatory Commission.







Data Source:
SW/AA (1980)

Notes:

1. No groundwater measurements available.
2. Shear wave velocities estimated based on downhole measurements by SW/AA (1980) at a nearby site.

Generalized Soil Profile

1-story Warehouse
Hollister, CA

Figure A7.3

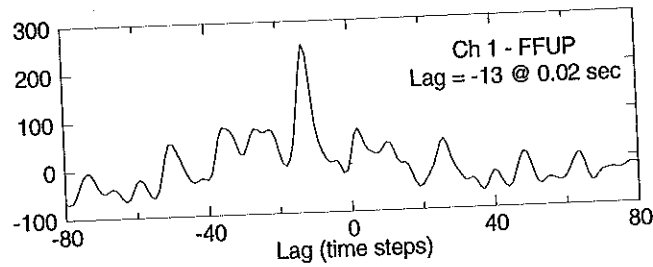


Fig. A7.LP0: Cross-correlation function between vertical ground floor and free-field accelerations

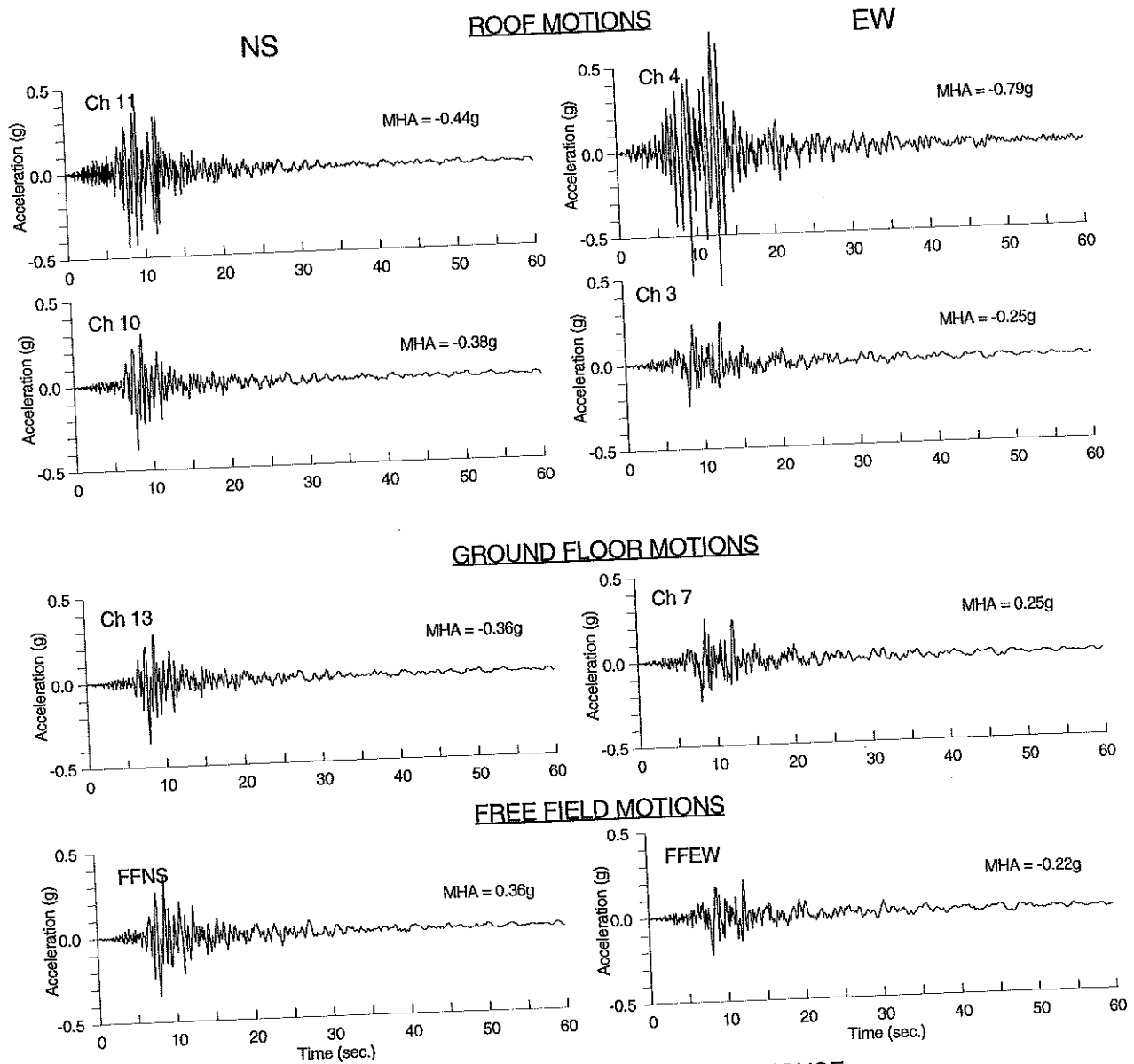


Fig. A7.LP1: HOLLISTER WAREHOUSE
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

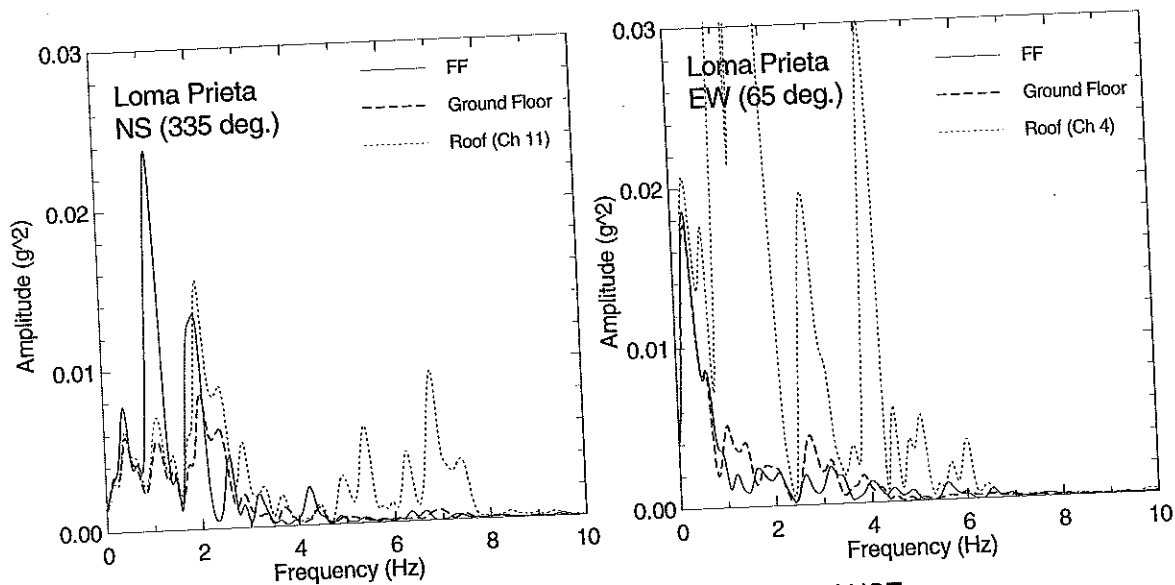


Fig. A7.LP2: HOLLISTER WAREHOUSE
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

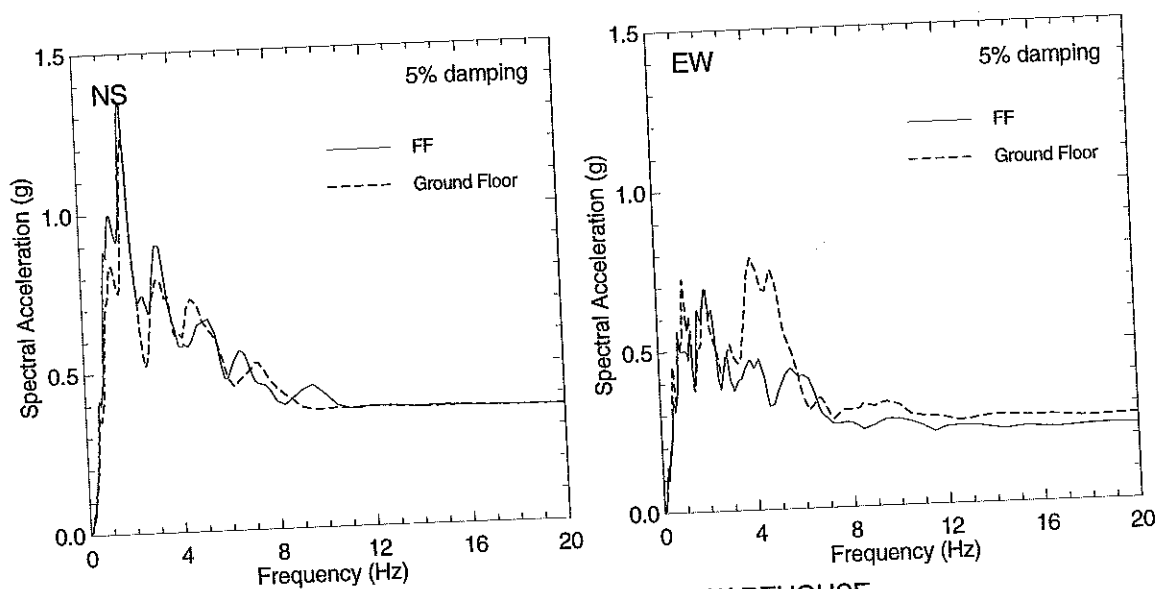


Fig. A7.LP3: HOLLISTER WAREHOUSE
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1989 Loma Prieta Earthquake

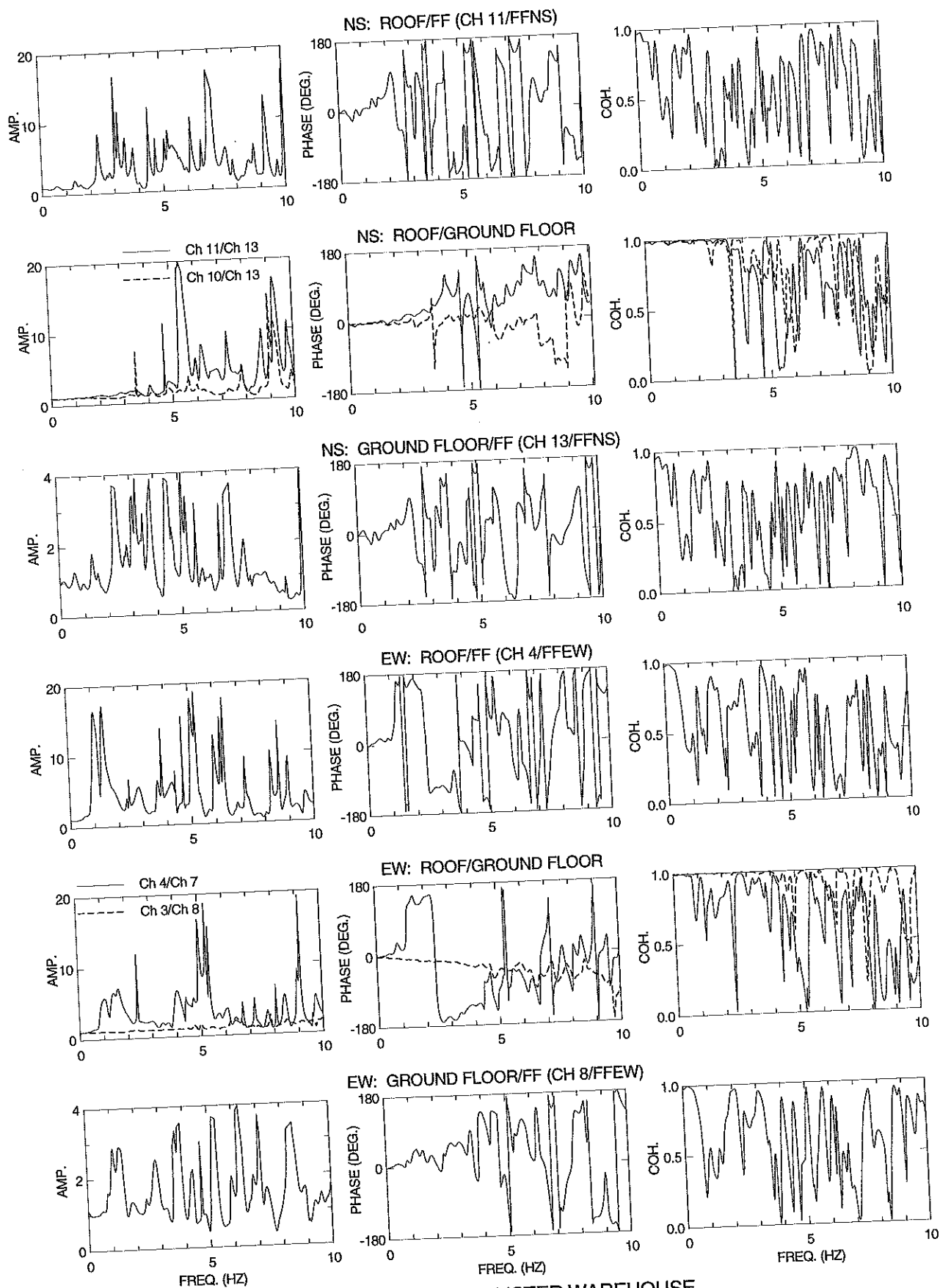


Fig. A7.LP4: HOLLISTER WAREHOUSE
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A7-10

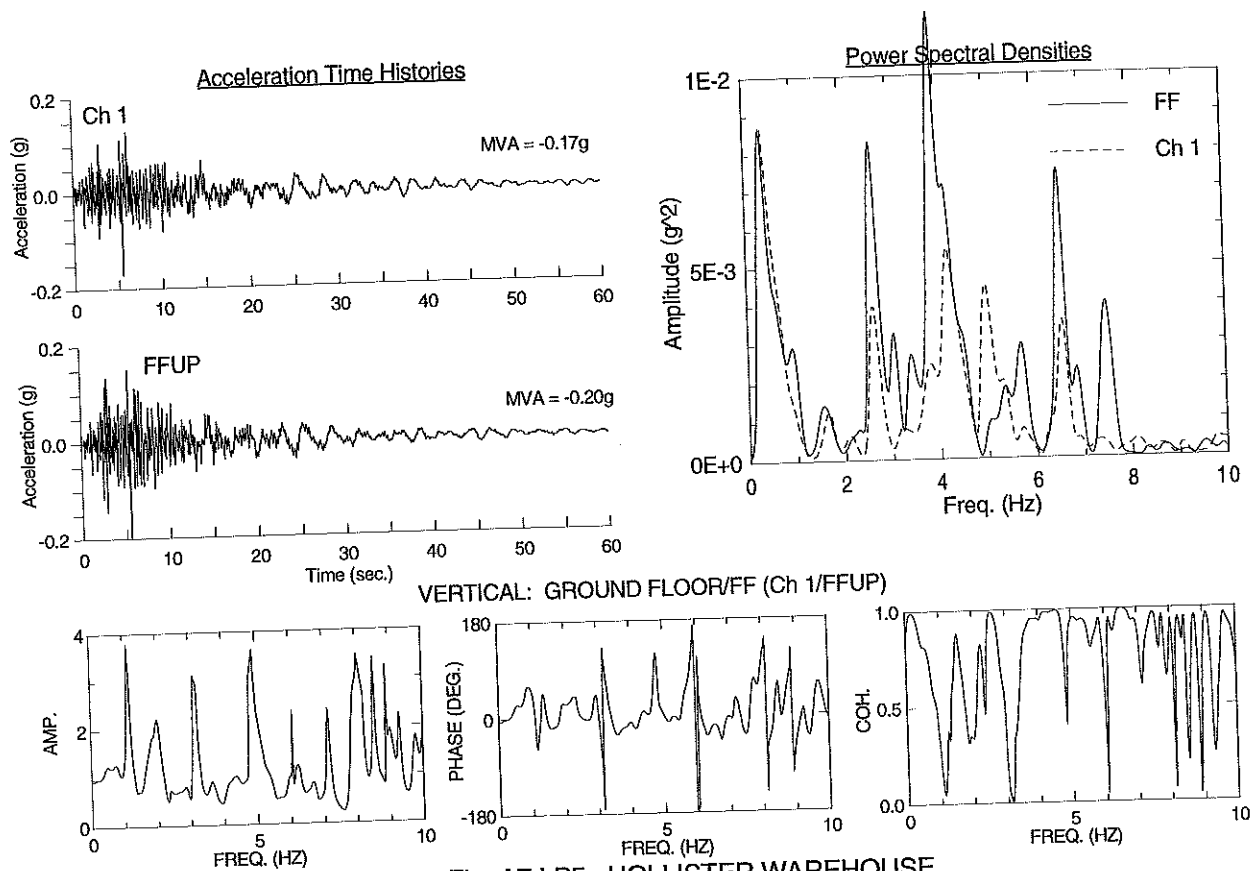


Fig. A7.LP5: HOLLISTER WAREHOUSE
Analysis of Vertical Motions
1989 Loma Prieta Earthquake

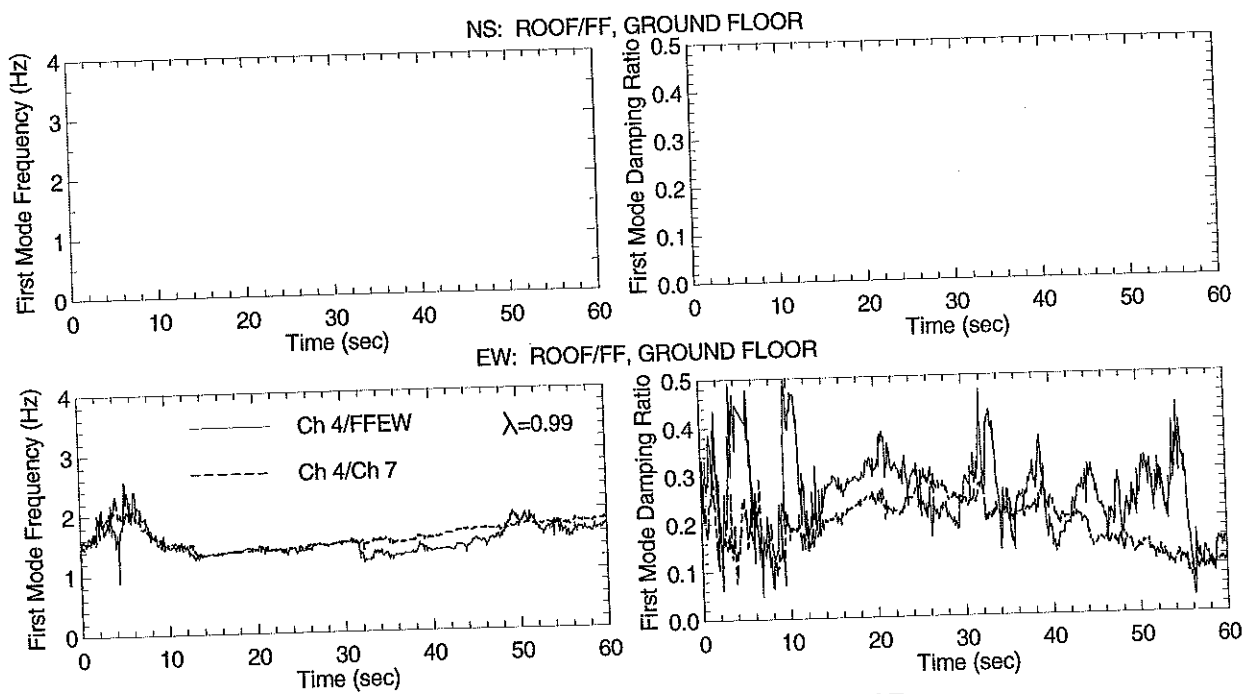


Fig. A7.LP6: HOLLISTER WAREHOUSE
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE A8

Site Name: Piedmont Jr. High School
Sensors: CSMIP 58334/58338

Lat: 37.823°
Long.: 122.233°

Floor Levels: 3
Bsmt. Levels: 0
Structure: Concrete shear walls

Foundation: footings/piers
Site Geology: rock
Design Date: 1973

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.070	0.084	0.026	0.086	0.072	0.033	0.18	0.15	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.3\text{--}23\text{ Hz}$; $f_{\text{Nyq}} = 25\text{ Hz}$; FF/str. data not synchronized

Final: FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A8.LP0).

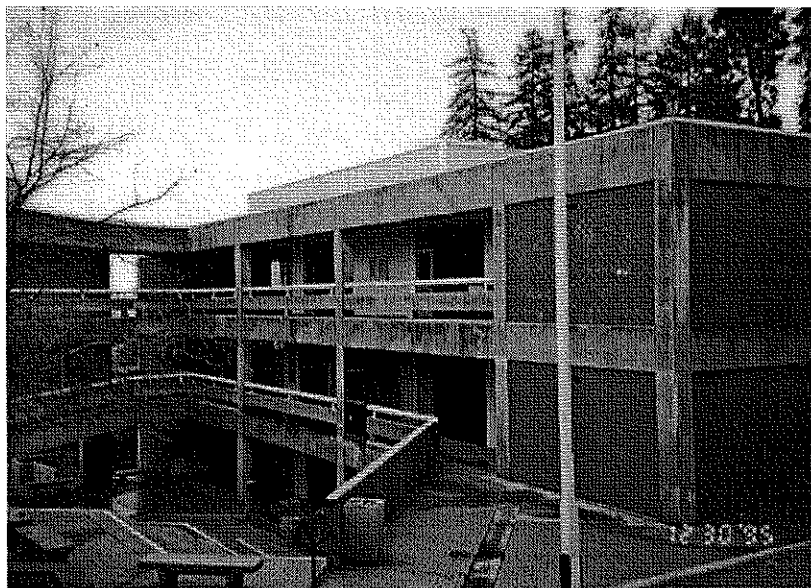
Figures

A8.1-A8.3

Site Plan, Sensor Locations, Generalized Soil Column

A8.LP0-6

LP Analysis Results



View of structure, looking west

DISCUSSION

Site Description

The building is 3 stories tall above ground level with no basement, though the ground floor is partially below grade and has a 10-foot retaining wall at its east end. The structure is rectangular in plan view with dimensions 73 by 115 feet. Two other three-story buildings are located near the structure. Lateral loads are resisted by 12-inch thick, 17-foot long shear walls at all four corners of the building. The foundation consists of belled piers of variable depth interconnected in the longitudinal (EW) direction by tie beams. Pier depths range from 2.5 feet (at the east end of the building which is underlain by rock) to 19 feet (in the building's northwest corner where colluvium overlies bedrock). A 4-inch thick reinforced concrete slab-on-grade is present across the ground floor level. Accelerographs are present on the ground, 2nd, and roof levels of the structure. At the time of the Loma Prieta Earthquake, the free-field instrument was located about 120 feet southwest of the structure near a basketball court.

The site is on moderately sloping terrain in the Oakland hills, with the building located on a level cut/fill pad. Based on geologic exploration by Powers and Fumal (1993) near the free-field instrument, subsurface conditions consist of sandstone and shale bedrock which is weathered in the upper 25 feet. The shear wave velocities shown in Fig. A8.3 were measured using downhole and suspension logging techniques by Agabian Associates (1993).

Instrument Selection and Free-Field Conditions

Motions from the west end of the structure were used in the analyses due to the presence of vertical sensors for recording transverse base rocking at that location (Table A8.1). However, it should be noted that the transverse structural response is not uniform. As illustrated by the acceleration time histories (Fig. A8.LP1), roof motions at Ch 2 (west end of building) exceed those at Ch 1 (east end of building). This irregularity results from a relatively stiff east end of the building due to its first level being partially below grade.

Table A8.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 8	Ch 9	Ch 10,11	Ch 2	Ch 3

The following input was used in system identification analyses to evaluate fixed-base structural parameters for the transverse direction,

$$\text{input} = \text{Ch8} + (\text{Ch11} - \text{Ch10}) \cdot 36/73 \quad (1)$$

No significant structures or slope breaks were located near the FF instrument. Although the FF instrument was only 120 feet from the instrumented structure, power spectra of the horizontal FF motions do not indicate significant energy content near the structure's modal frequencies, indicating that structural vibrations were unlikely to have influenced the FF recordings.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

GF/FF transmissibility functions (Fig. A8.LP4) and the spectra for horizontal motions (Fig. A8.LP2-3) indicate significant amplification of NS GF motions relative to the FF from about 1 to 8 Hz, with the amplification being most pronounced at the first-mode frequency of 5.5 Hz. No significant amplification of GF motions in the EW direction is observed.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A8.LP5. Vertical GF motions were amplified near the first-mode transverse lateral response frequency ($\tilde{f} \approx 5.5$ Hz), suggesting that rocking may have contributed to the structural response. Rocking contributions to roof motion are directly examined in Fig. A8.LP5 by comparing relative lateral transverse roof accelerations with the product of base rotation and building height (RxH). Components of the RxH accelerations are in phase with the relative roof accelerations, and contain up to 100% of the relative roof acceleration amplitude. Though longer-period energy is also present in the RxH accelerations, these results indicate the strong influence of base rocking on lateral roof accelerations. A similar comparison of RxH and relative roof displacements was impractical due to the structure's high fundamental frequency.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A8.LP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A8.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A8.LP6).

Table A8.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) d=0, j=2		L (EW) estimated		tr (NS) d=0, j=2		L (EW) d=0, j=3		tr (NS) d=0, j=2		L (EW) d=0, j=3	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	6.32	2.2	5.90	5.2	5.96	3.8	5.89	5.3	5.44	4.7	5.87	7.3
	2	11.9	20.4			11.8	20.5	--	--	10.7	24.8	--	--

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The modest differences between the first-mode fixed- and flexible-base parameters for the transverse direction is consistent with the rocking effect noted in Part (a). No significant inertial interaction effects were evident in the longitudinal direction.

The RPEM results (Fig. A8.LP6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitudes of FF-R and GF-R modal parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A8.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 52 feet using the recorded free-field motions and the profile in Fig. A8.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by 30% to account for shape effects.

Table A8.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1830$ fps	$h=25$ ft	emb.=0	$\gamma = 0.15$
LP	$\nu=0.4$		tr: $r_1 = 52$ ft		$r_2 = 46$ ft
	$\beta^*=1.3\%$	$V_s^*=1820$ fps	L: $r_1 = 52$ ft		$r_2 = 59$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A8.3 with the fixed-base modal parameters in Table A8.2, the first-mode period lengthening and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A8.4.

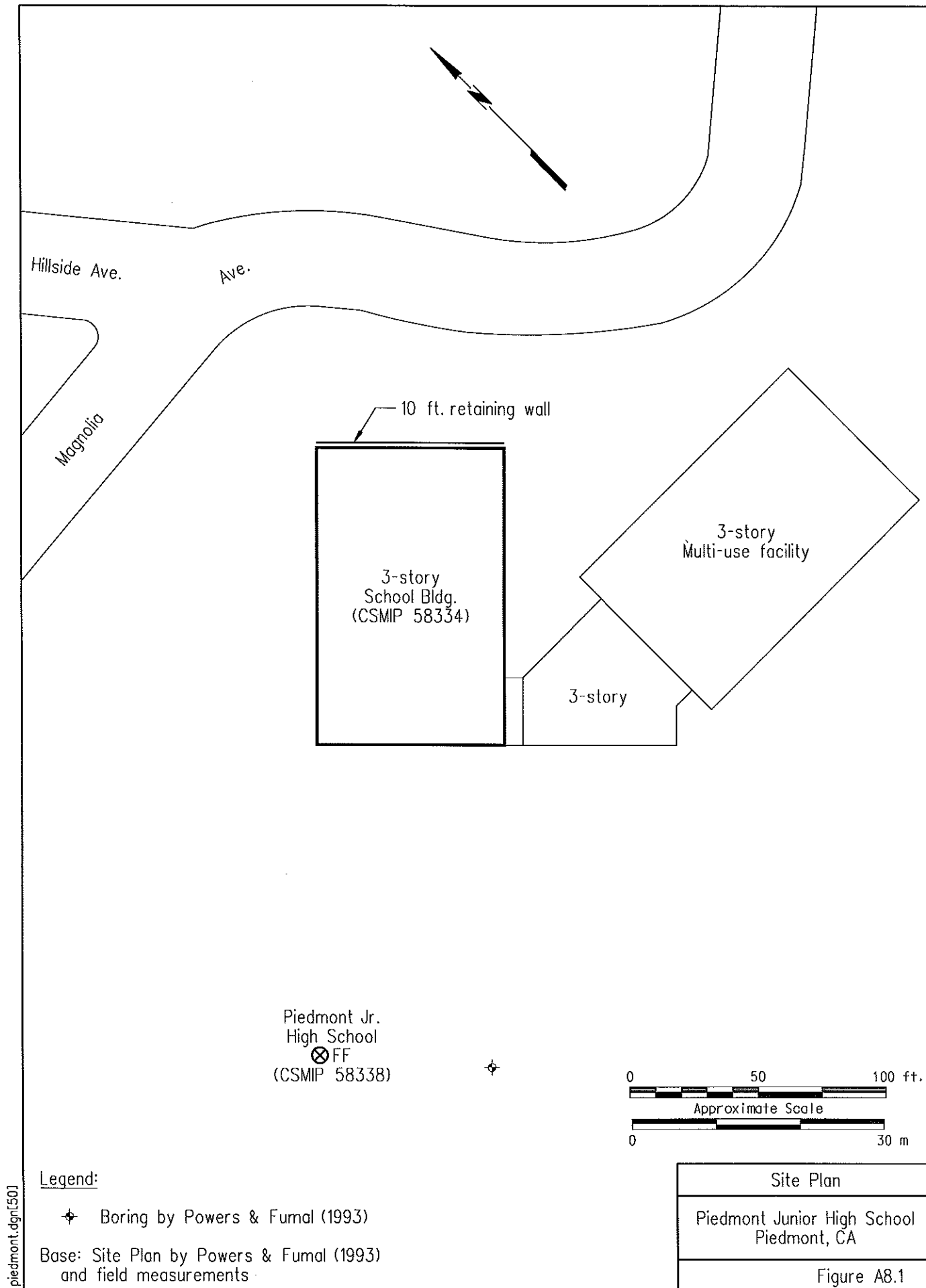
Table A8.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$	$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$
	Prediction	1.04	1.5	3.5	1.03	1.4	6.1

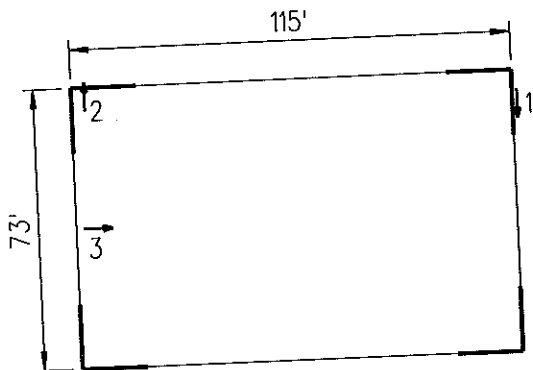
The period lengthening and foundation damping for the transverse direction were underpredicted. The shear wave velocities used in the analyses (Fig. A8.3) are from the free-field location, may not accurately represent the colluvium/soil profile at the structure.

Data Sources

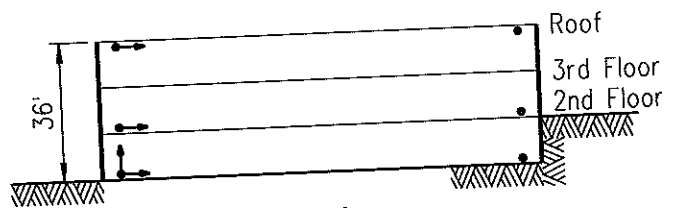
- Powers, T.J. and Fumal, T.E. (1996). "Geologic logs from 25 boreholes near strong motion accelerographs that recorded the 1989 Loma Prieta, California, Earthquake," *Rpt. No. OFR 93-502*, U.S. Geological Survey.
- Agbabian Assoc. (1993). "Borehole geophysical measurements at 24 soil and rock sites," *Rpt. No. 9225-6427*.



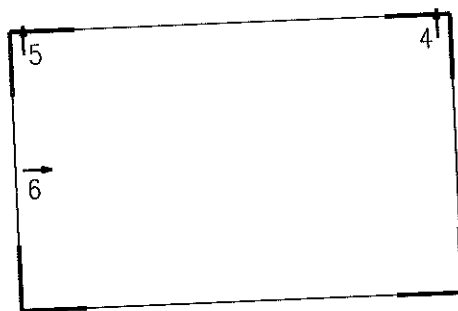
piedmont.dgn[50]



Roof Plan

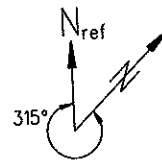
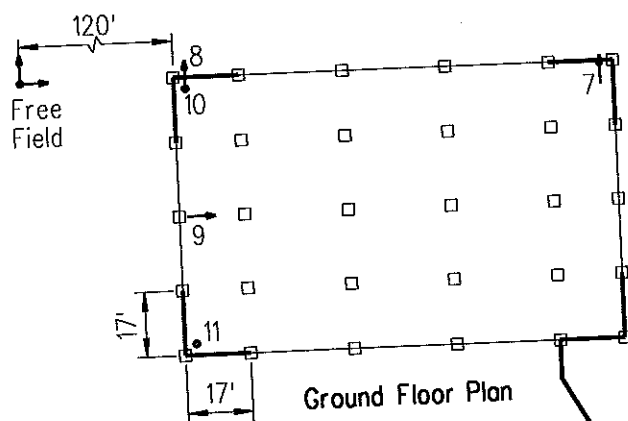


Elevation



2nd Floor Plan

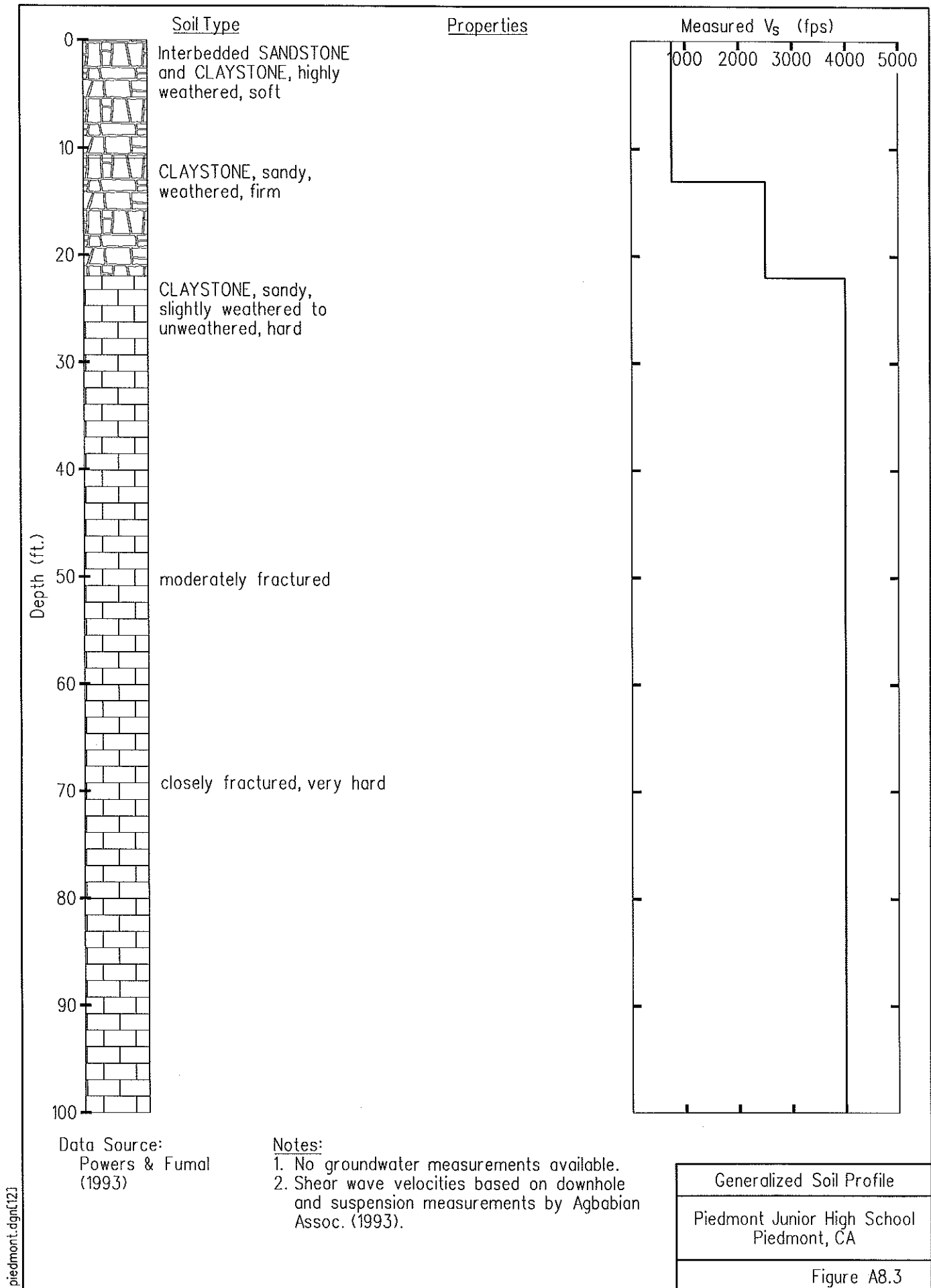
Concrete shear wall



Ground Floor Plan

Retaining wall

Sensor Locations
Piedmont Junior High School Piedmont, CA
Figure A8.2



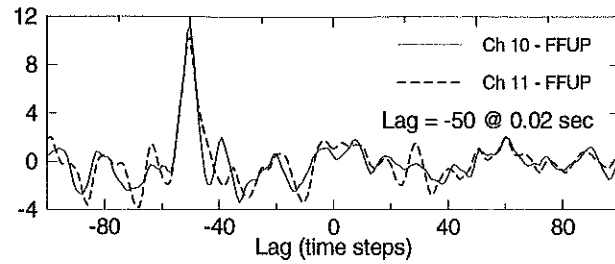


Fig. A8.LP0: Cross-correlation function between vertical ground floor and free-field accelerations

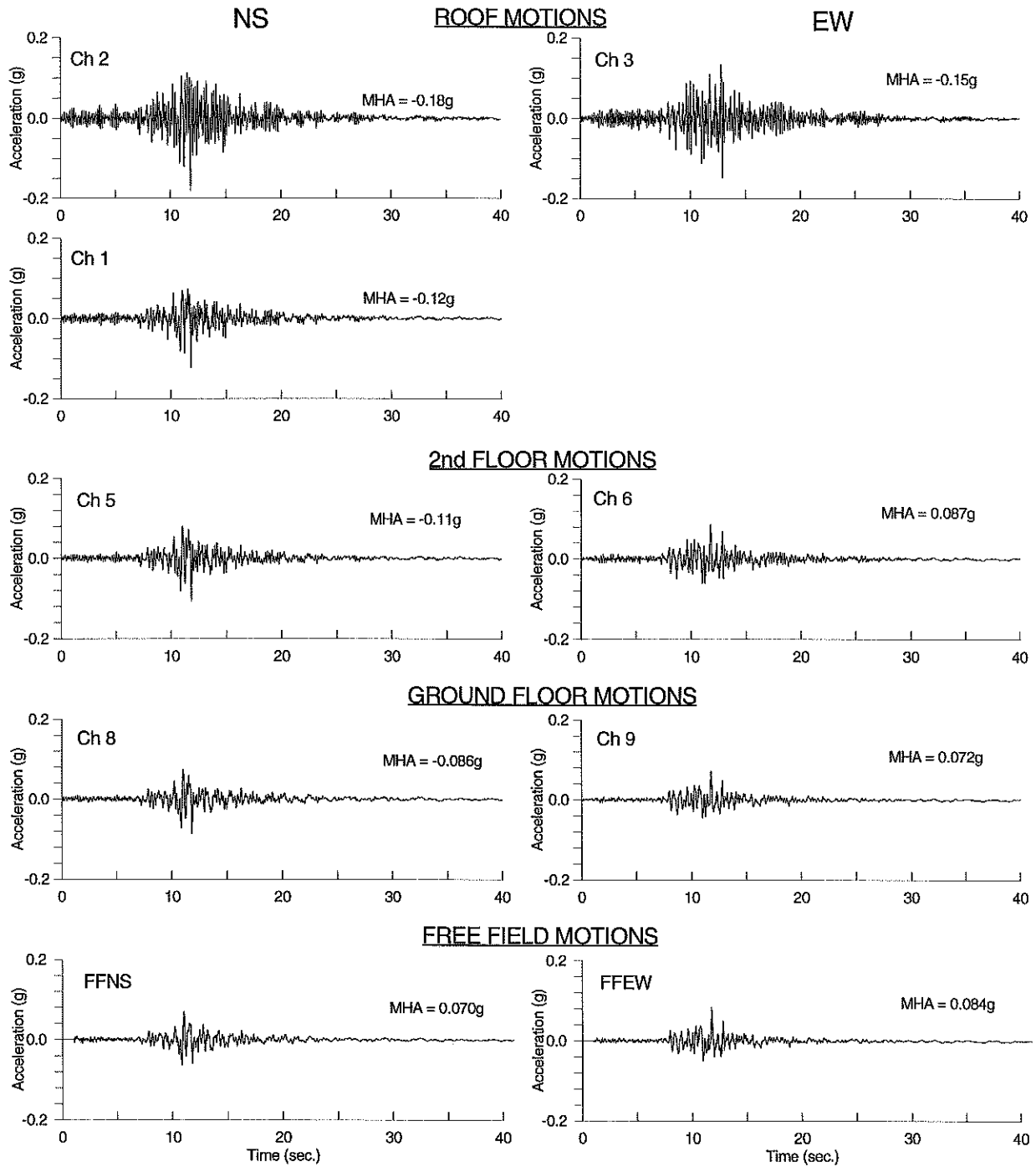


Fig. A8.LP1: PIEDMONT JR. HIGH SCHOOL
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

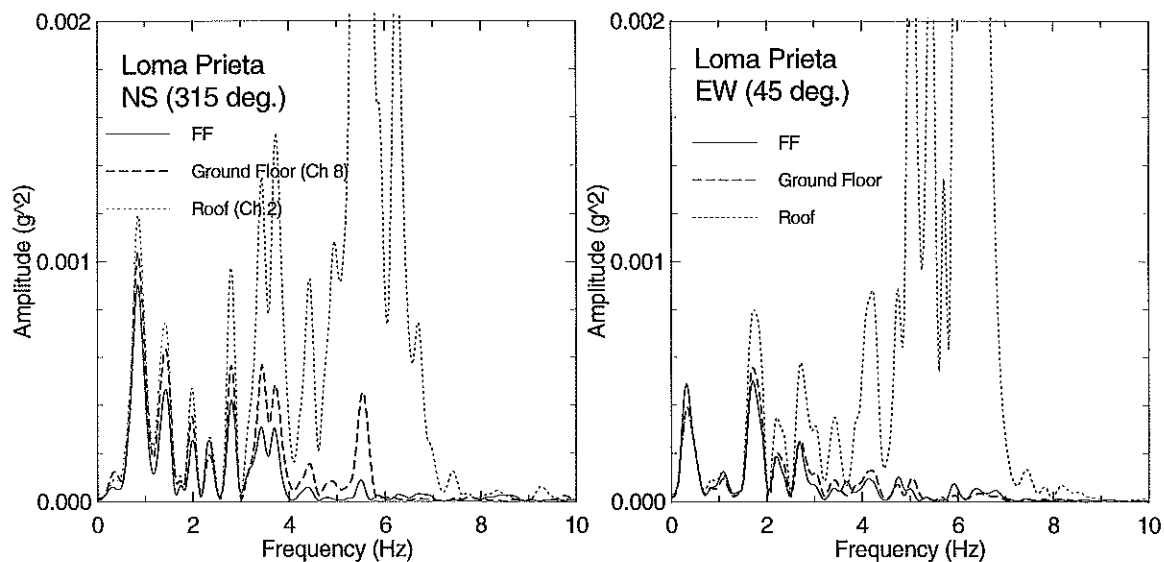


Fig. A8.LP2: PIEDMONT JR. HIGH SCHOOL
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

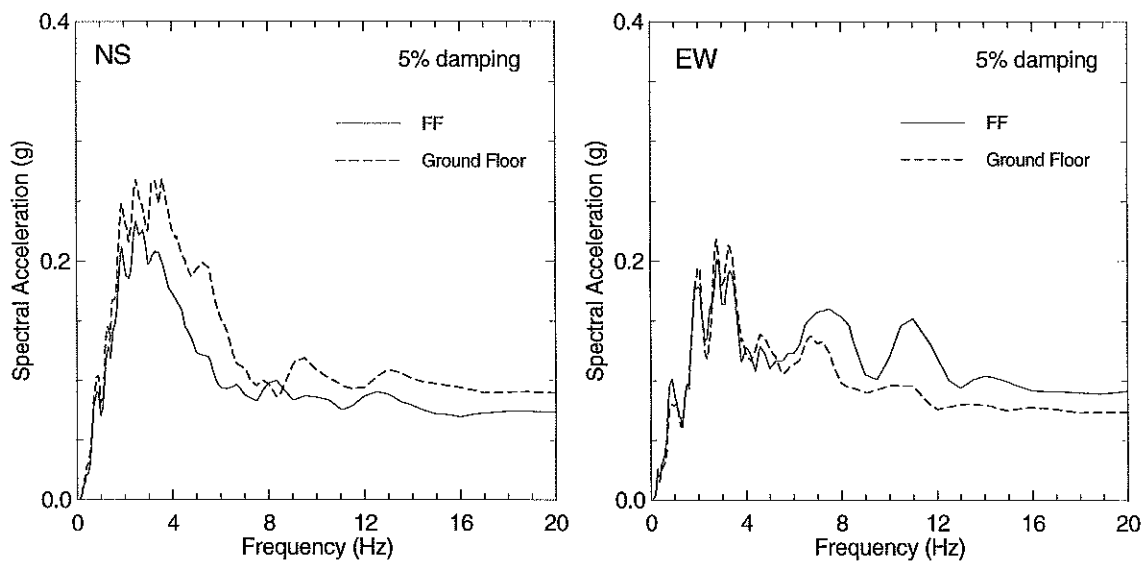


Fig. A8.LP3: PIEDMONT JR. HIGH SCHOOL
Acceleration Response Spectra of Horizontal Ground Floor and Free-Field Motions
1989 Loma Prieta Earthquake

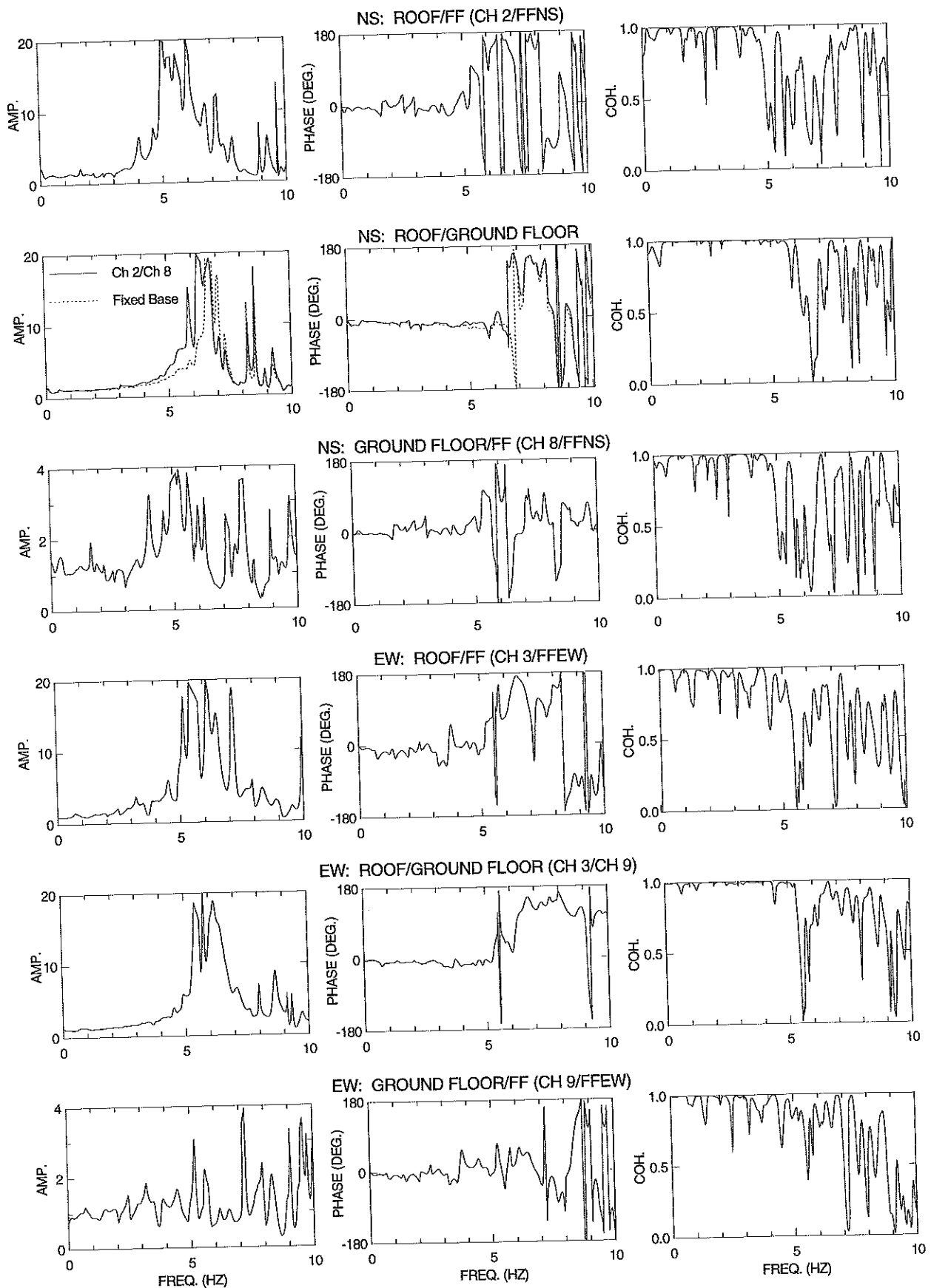


Fig. A8.LP4: PIEDMONT JR. HIGH SCHOOL
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A8-10

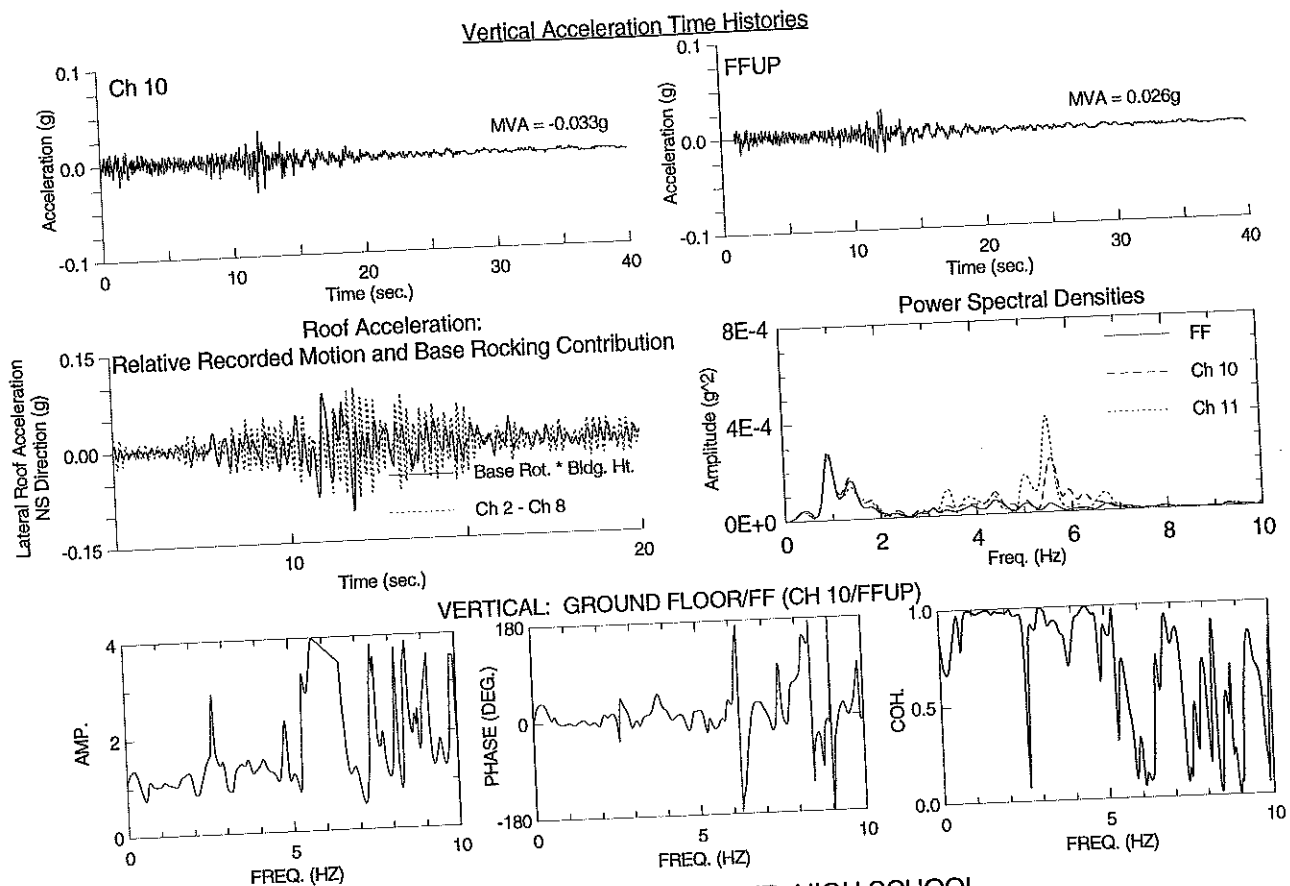


Fig. A8.LP5: PIEDMONT JR. HIGH SCHOOL
Analysis of Vertical Motions and Rocking Effects
1989 Loma Prieta Earthquake

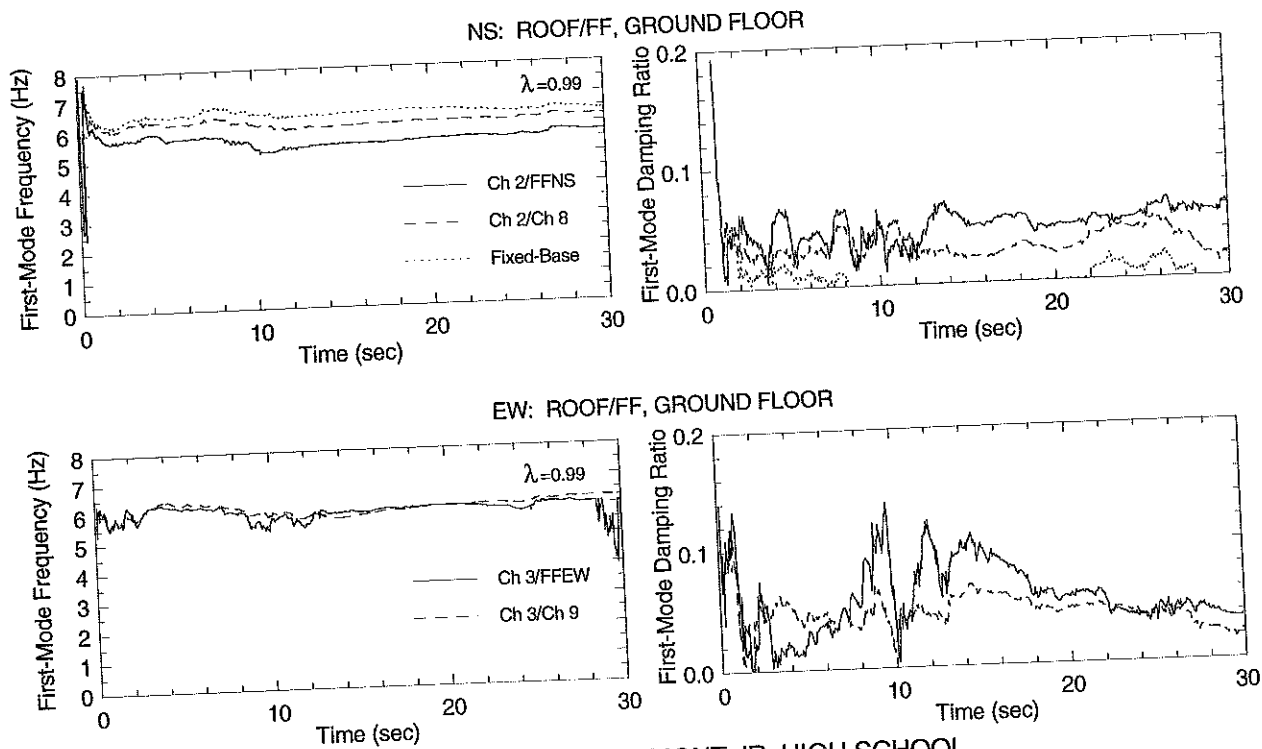


Fig. A8.LP6: PIEDMONT JR. HIGH SCHOOL
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE A9

Site Name: Pleasant Valley Pumping Plant
Sensors: USGS 2342

Lat: 36.308°
Long.: 120.249°

Floor Levels: 2
Bsmt. Levels: 1
Structure: concrete shear walls and steel frame

Foundation: concrete mat
Site Geology: Alluvium
Design Date: 1966

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1983 Coalinga MS	0.59	0.52	0.36	0.30	0.27	0.21	--	--	--
1983 Coalinga AS	0.095	0.21	0.093	0.053	0.13	0.035	0.23	0.22	0.052

-- = No recording

Preprocessing

Coalinga Earthquake Mainshock (CG)

Preliminary: USGS, filtered at $f_{HP} = 0.1$ Hz; digitized at $f_{Nyq} = 100$ Hz; FF/str. data are synchronized

Final: data decimated by 4 to $f_{Nyq} = 25$ Hz

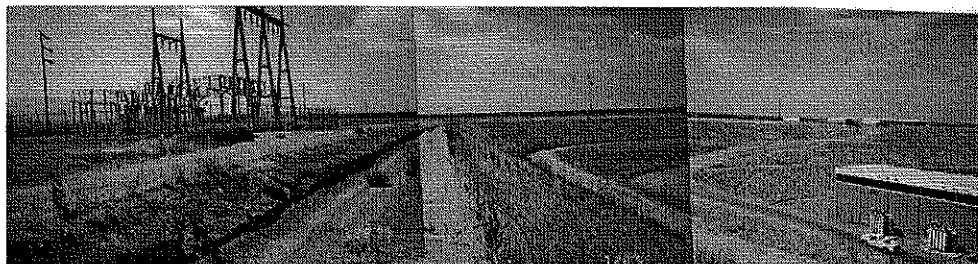
Coalinga Aftershock (CGA)

Preliminary: USGS, filtered, $f = 0.5-50$ Hz; $f_{Nyq} = 100$ Hz; FF/str. data not synchronized

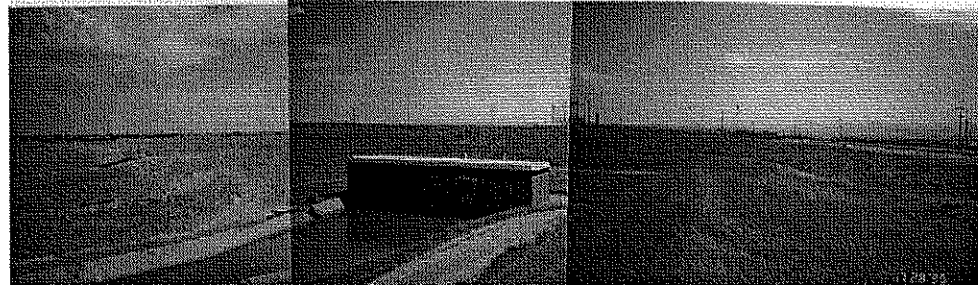
Final: data decimated by 4 to $f_{Nyq} = 25$ Hz; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A9.CGA0)

Figures

A9.1-A9.3 Site Plan, Sensor Locations, Generalized Soil Column
A9.CG1-5 CG Analysis Results
A9.CGA0-5 CGA Analysis Results



View of switchyard and pumping plant, looking north-west



View of structure from entrance road, looking south

DISCUSSION

Site Description

The layout of the site and the subsurface conditions are presented in Figs. A9.1 to A9.3. The bi-level building houses nine water pumps which lift water 199 feet from the San Luis Canal to the Coalinga Canal System. The structure has a broad concrete base (approximately 100 by 200 feet in plan dimension) in which the large pumps and water pipes are located. There are cavities in the base for servicing the pumps, which are referred to as the basement level. Above the base is a 35 foot tall steel frame service bay with dimensions of 50 by 200 feet in plan. As shown in Fig. A9.2, accelerographs are present in the basement, service bay floor (denoted as "1st floor") and roof. The free-field instrument is located in a switchyard 300 feet southwest of the plant and 84 feet above the basement level.

The site is located on flat terrain in the San Joaquin Valley, about 20 miles northeast of Coalinga. Alluvial fan sediments derived from the Diablo Range to the west underlie the site (Bureau of Reclamation, 1974). As shown in Fig. A9.3, soil borings drilled prior to the excavation for the plant (Bureau of Reclamation, 1966) indicated interbedded silty, clayey sand and clay layers (typical layer thickness of about 5 feet) to the maximum depth explored of 200 feet. The shear wave velocities shown in Fig. A9.3 were estimated using correlations with SPT blow count and void ratio. The overall thickness of the valley sediments near the site exceeds 1000 feet near the plant, and 2000 feet immediately to the east (Davis and Poland, 1957).

Instrument Selection and Free-Field Conditions

The building contains only three triaxial accelerographs, and of these, only the basement instrument was triggered by the Coalinga mainshock. All three were triggered by a smaller aftershock, and these data were used for system identification (Table A9.1). Acceleration time histories recorded by these sensors are shown in Fig. A9.**1.

Table A9.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 1	Ch 2	Ch 9	Ch 7

The free-field instrument is located in an electrical switchyard above the plant. No significant structures are located near the instrument, but an approximately 70-foot tall slope is present between the instrument and the plant. The slope inclination is 2H:1V, though with two benches at 1/3 and 2/3 of the slope height, the net slope angle is closer to 3H:1V. As the plant is located at the toe-of-slope and the free-field instrument is located about 55 feet from the top-of-slope, topographic de-amplification of plant motions and amplification of free-field motions is possible. Studies by Ashford and Sitar (1994) have shown that less than 20% topographic amplification of MHA occurs at the top-of-slopes with gradients of 2H:1V or less. Hence, combined with similar de-amplification at the slope toe, these topographic effects could influence the relative magnitudes of the free-field and basement motions. Site amplification

effects in the upper 90 feet of soil between the basement and free-field elevations are unlikely to significantly affect the relative magnitudes of these motions as no significant impedance contrasts are present in the soil column.

System Identification Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A9.**4) and the spectra of the horizontal motions (Fig. A9.**2-3) indicate significant de-amplification of basement motions from $f = 0$ to 8 Hz. As noted previously, some of this de-amplification likely resulted from topographic effects.

Transmissibility, coherency, and power spectral density functions for vertical FF and basement motions are presented in Fig. A9.**5. The vertical motions were de-amplified, and no amplification (or reduced de-amplification) is evident at modal frequencies.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A9.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A9.2). No RPEM analyses were performed due to the short duration of the CGA time histories.

Table A9.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=5, j=5		L (EW) d=--, j=--		tr (NS) d=3, j=5		L (EW) d=--, j=--	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
CGA	1	--	--	na	na	1.85	5.9	na	na	1.91	8.0	na	na
	2					5.98	6.0	na	na	5.93	4.7	na	na

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, transverse structural response is dominated by the first two modes, and there is no significant longitudinal response.
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical B/FF transmissibility function, transverse fixed-base parameters can be approximated by the pseudo flexible-base parameters.
- A value of $j = 5$ was necessary to develop parametric models of the structure with accurate representations of lower mode response, though only the lower two modes are meaningful.
- Parametric models for longitudinal response could not be developed due to the similarity of the input and roof motions in this direction.

- Due to the low duration of the CGA time histories, there was an insufficient number of data points with which to estimate coherency functions while maintaining adequate frequency resolution. Hence, coherency functions are not shown in Fig. A9.CGA4.

(c) Comparison of Empirical and Predicted and Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A9.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 53 feet using the recorded free-field motions and the profile in Fig. A9.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though the rocking radiation damping for the transverse direction was increased by a factor of 2.5 to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A9.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_S = 1010$ fps	$h = 48$ ft	emb. = 32 ft	$\gamma = 0.15$
CGA	$\nu = 0.33$		tr: $r_1 = 53$ ft	$r_2 = 37$ ft	
	$\beta^* = 3.2\%$	$V_S^* = 970$ fps	L: $r_1 = 53$ ft	$r_2 = 77$ ft	

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A9.3 with the fixed-base modal parameters in Table A9.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A9.4.

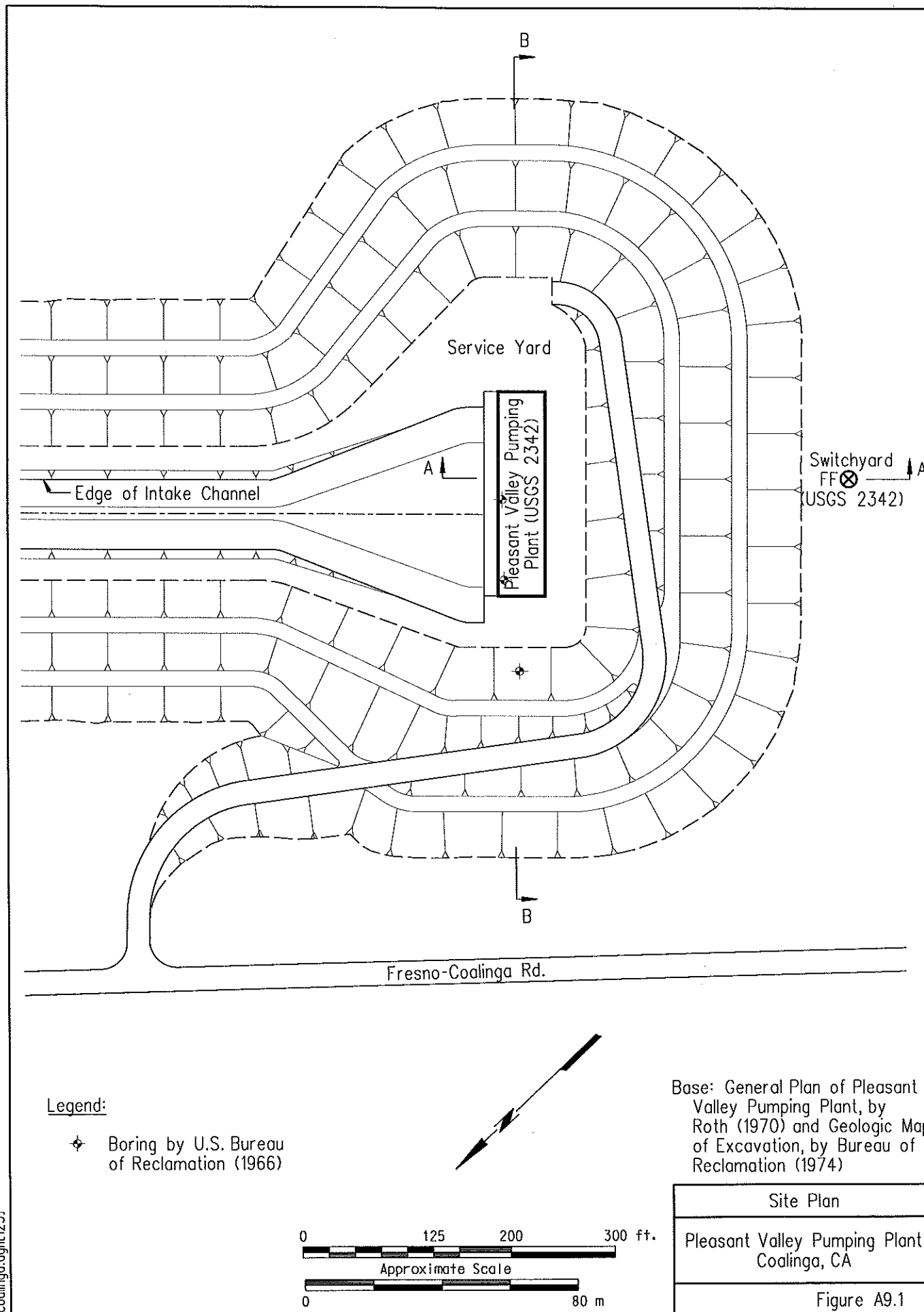
Table A9.4: Empirical and predicted inertial interaction effects

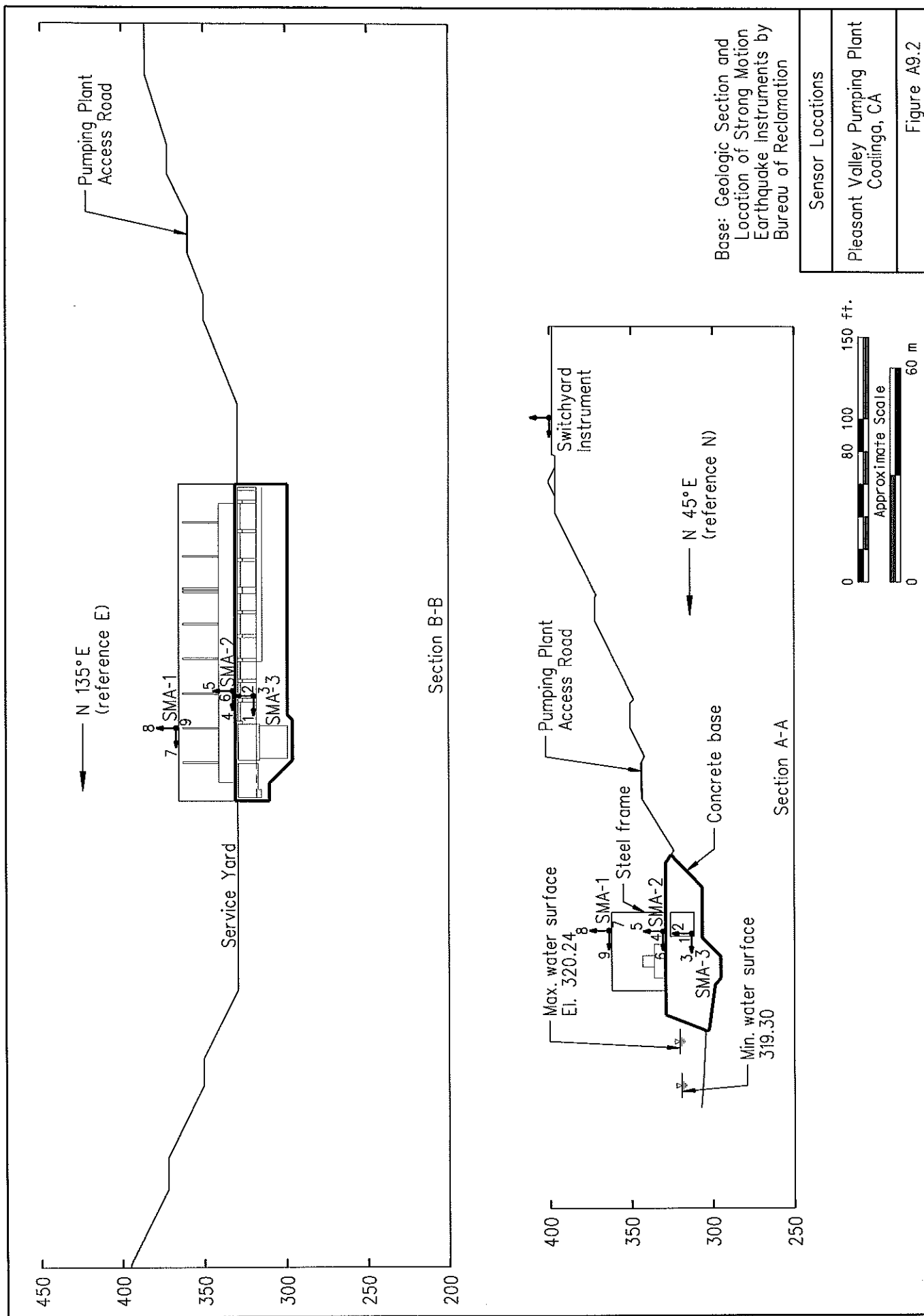
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
CGA	Sys. ID	0.96	2.1	8.0	--	--	--
	Pre.-Vel.	1.03	0.3	5.7	--	--	--
	Pre.-Bie.	1.03	1.7	7.1	--	--	--

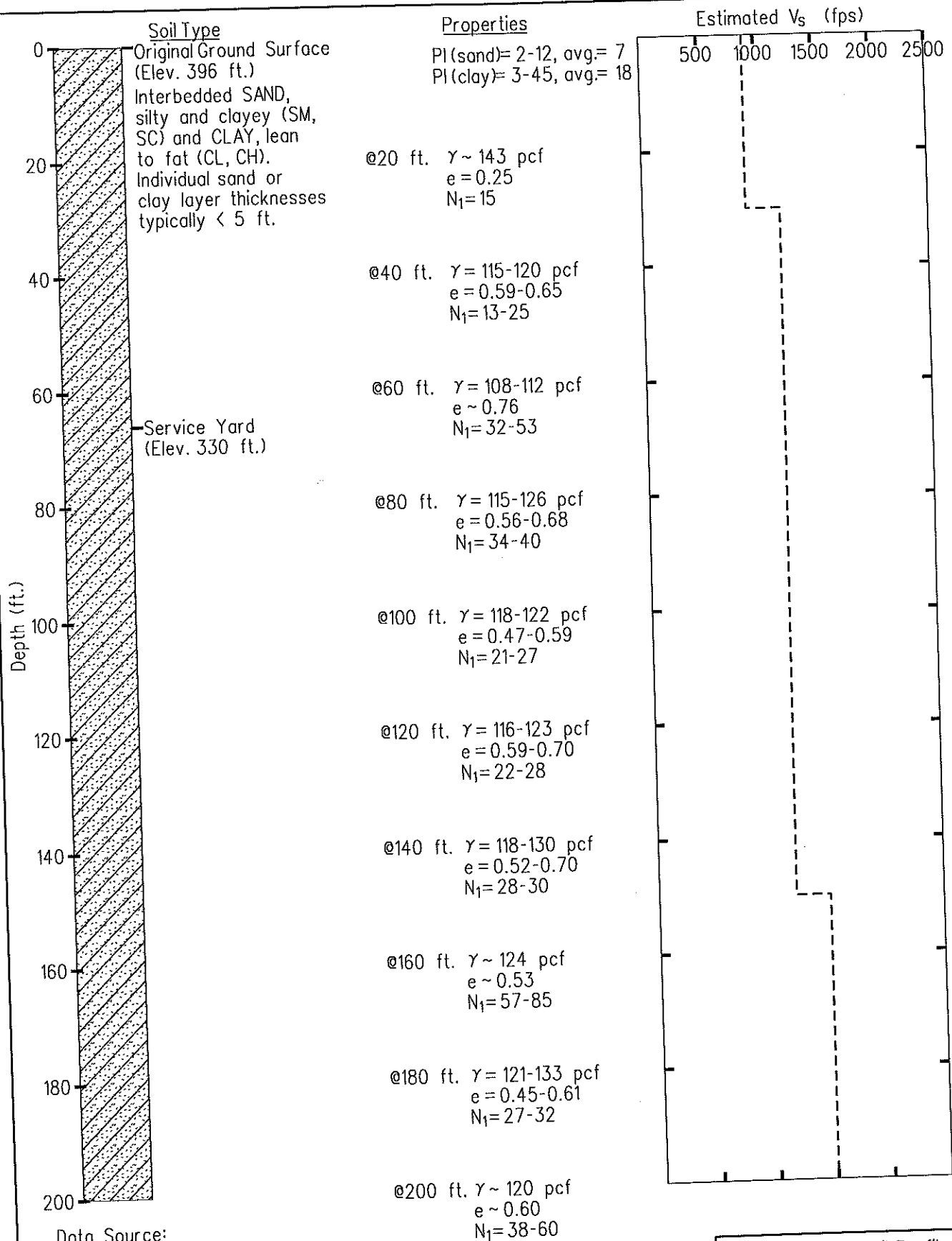
Given the obvious uncertainty in the empirical results, the predicted period lengthening by both formulations is satisfactory. Foundation damping $\tilde{\zeta}_0$ is predicted more accurately by the "modified Bielak" formulation.

Data Sources

- Ashford, S.A. and Sitar, N. (1994). "Seismic Response of Steep Natural Slopes," *Rpt. No. UCB/EERC-94/05*, Earthquake Engrg. Res. Cntr., U.C. Berkeley, May.
- Davis, G.H. and Poland, J.F. (1957). "Ground-water conditions in the Mendota-Huron Area, Fresno and Kings Counties, CA," *Water Supply Paper 1360-G*, U.S. Geological Survey.
- U.S. Bureau of Reclamation (1966). Geologic Logs of Drill Holes PV911-913.
- U.S. Bureau of Reclamation (1974). "Technical record of design and construction, Central Valley Project, West San Joaquin Division, San Luis Unit, CA," Vol. 1, Chapter IV.







Data Source:
Boring logs by
U.S. Bureau of
Reclamation (1966)

- Notes:**
1. No groundwater encountered.
 2. Penetration resistance established by SPT techniques.
 3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile
Pleasant Valley Pumping Plant
Coolinga, CA
Figure A9.3

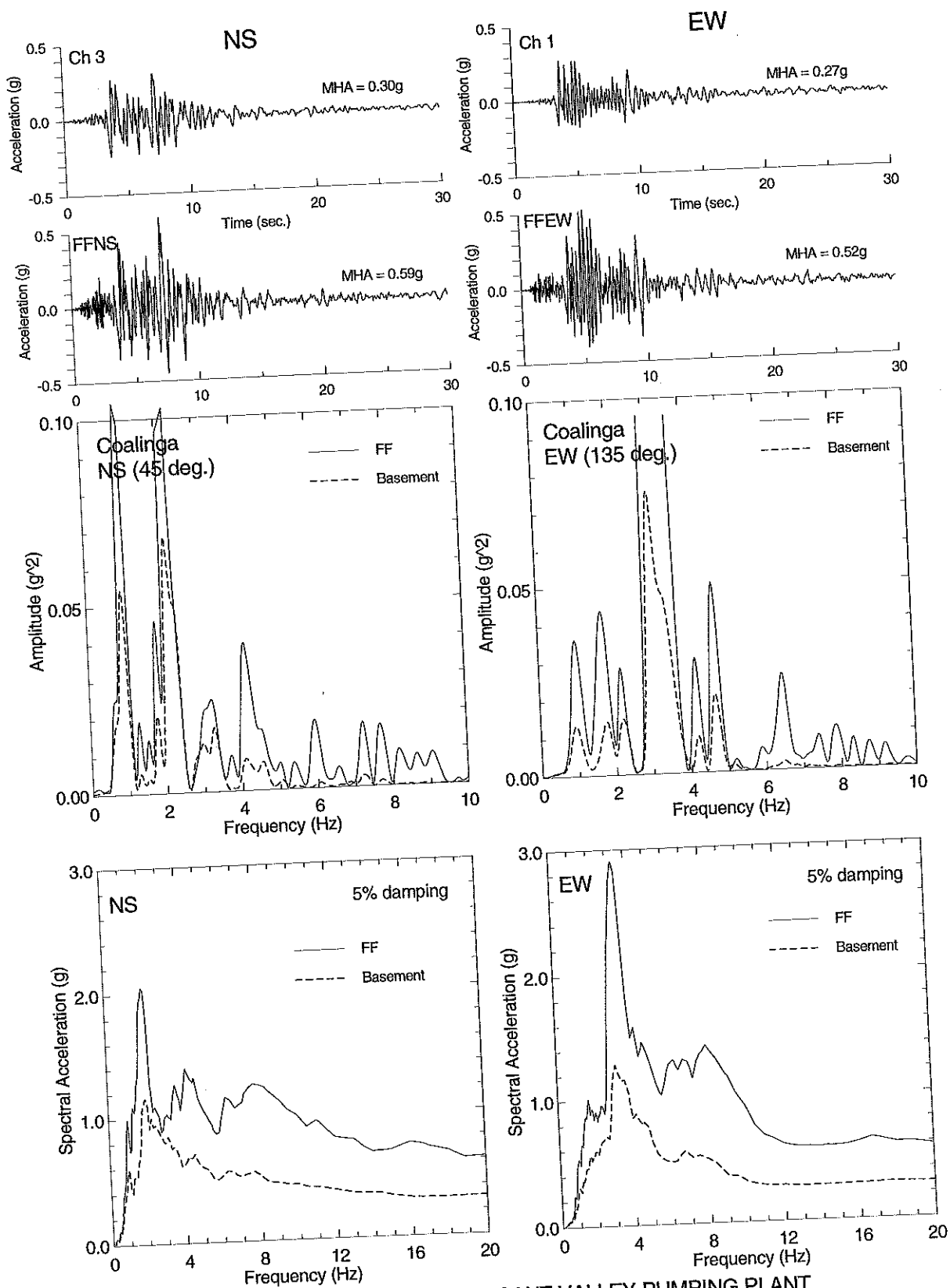


Fig. A9.CG 1 - 3: PLEASANT VALLEY PUMPING PLANT
 (1) Horizontal Acceleration Time Histories,
 (2) Power Spectral Density of Selected Horizontal Accelerations, and
 (3) Acceleration Response Spectra of Horizontal Basement and Free-Field Accelerations
 1983 Coalinga Earthquake Mainshock

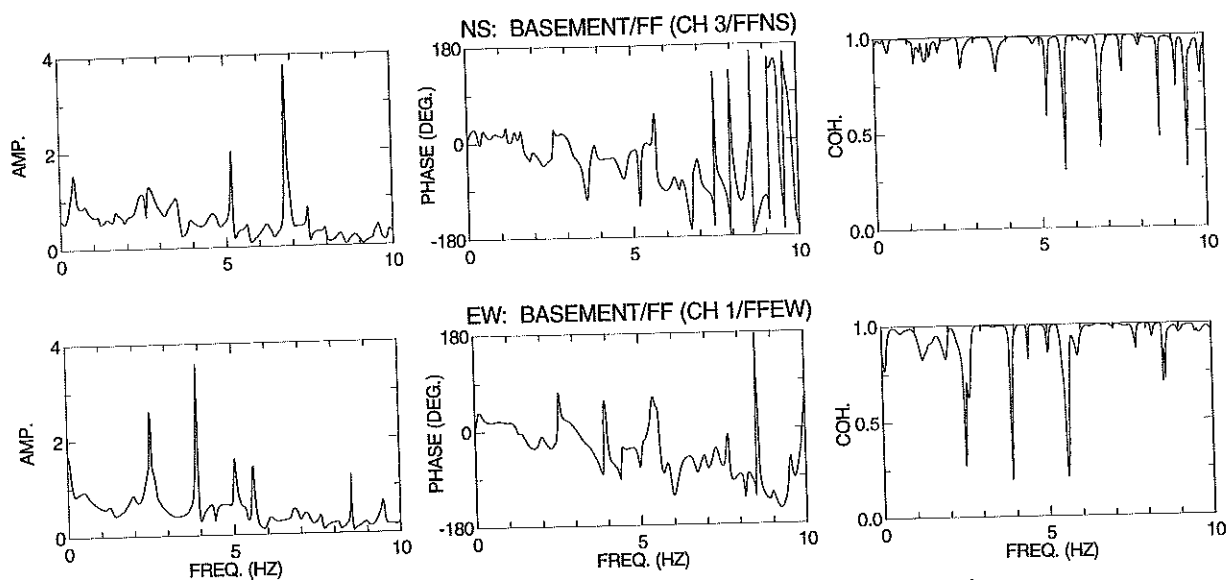


Fig. A9.CG4: PLEASANT VALLEY PUMPING PLANT
Transmissibility and Coherency Functions
1983 Coalinga Earthquake Mainshock

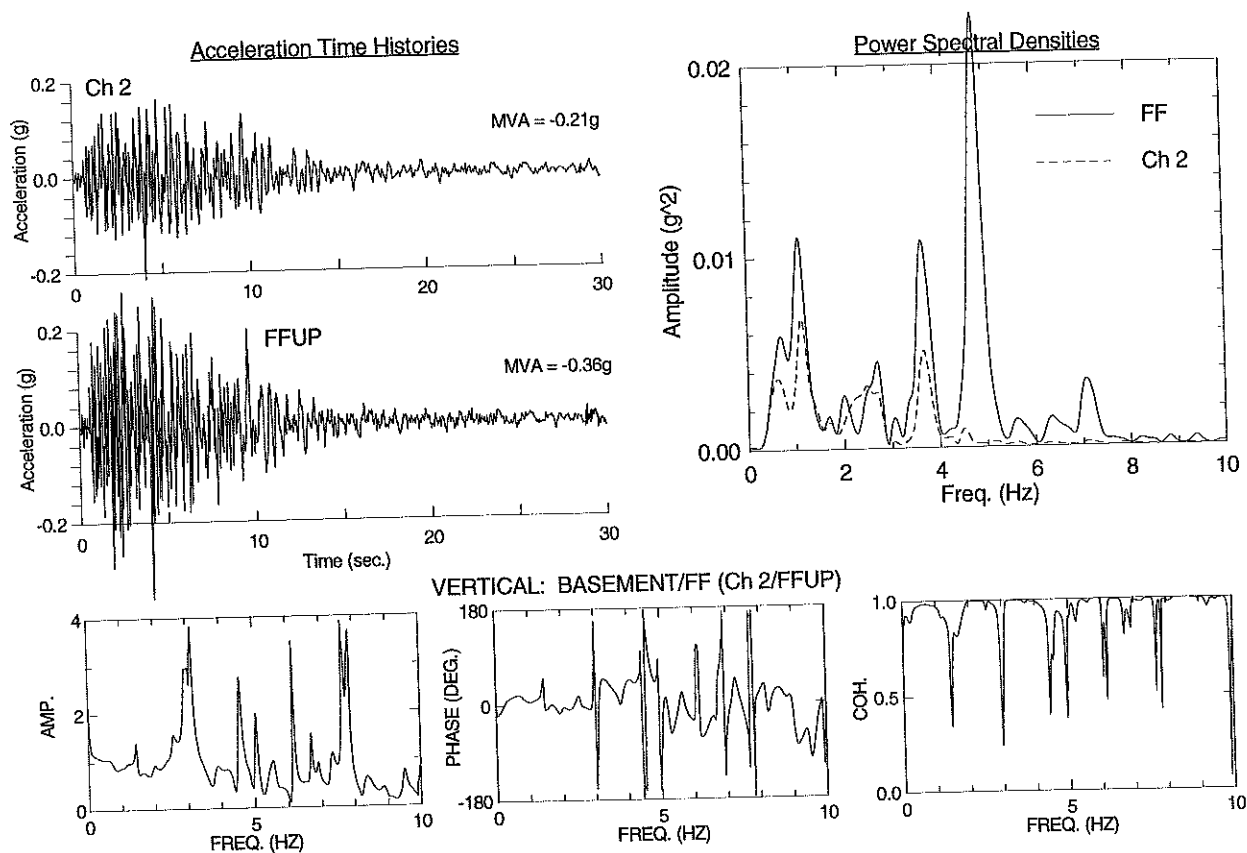


Fig. A9.CG5: PLEASANT VALLEY PUMPING PLANT
Analysis of Vertical Motions
1983 Coalinga Earthquake Mainshock

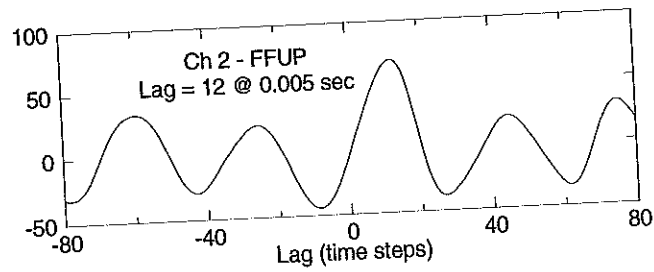


Fig. A9.CGA0: Cross-correlation function between vertical basement and free field accelerations

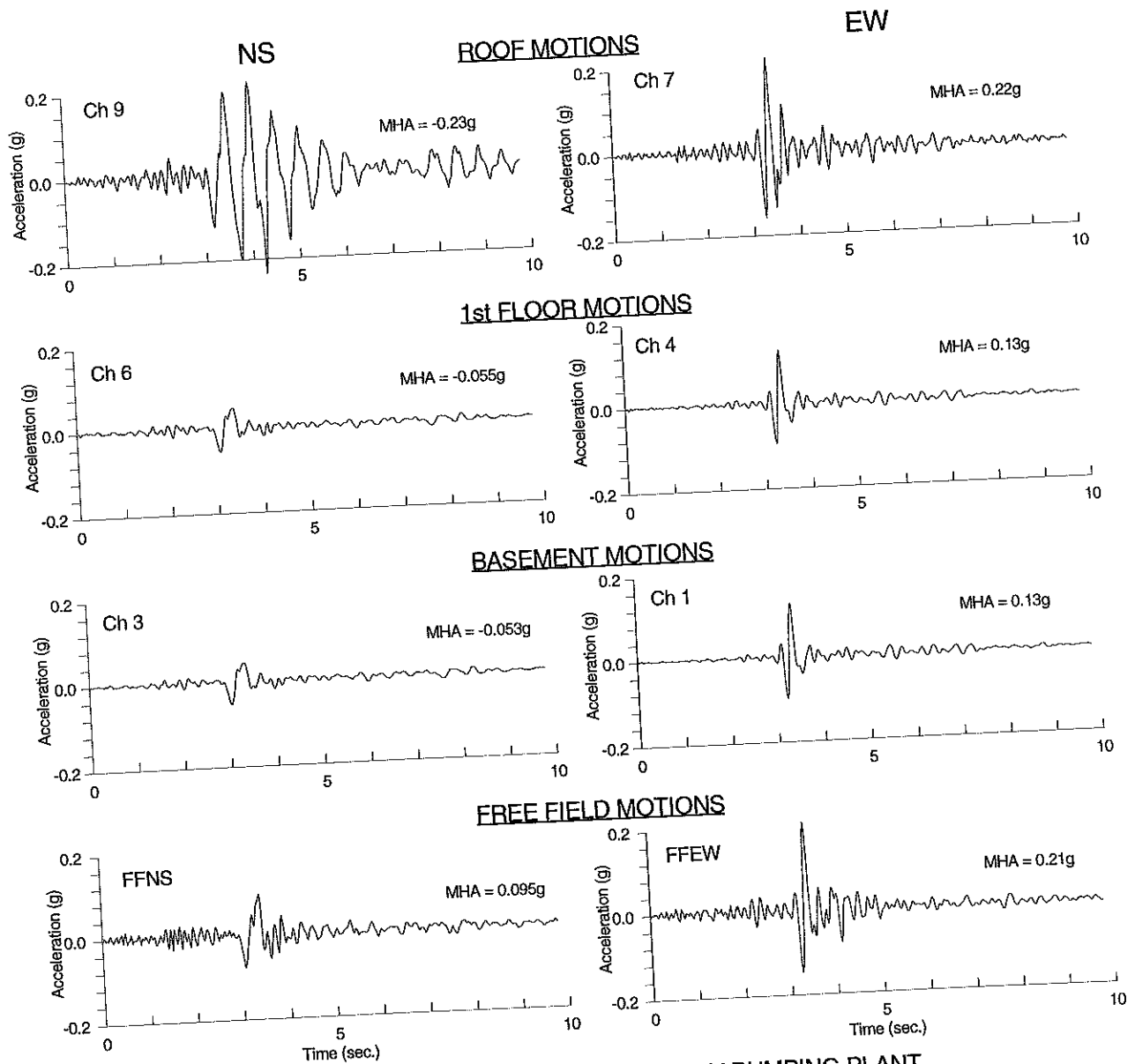


Fig. A9.CGA1: PLEASANT VALLEY PUMPING PLANT
Horizontal Acceleration Time Histories
1983 Coalinga Earthquake Aftershock

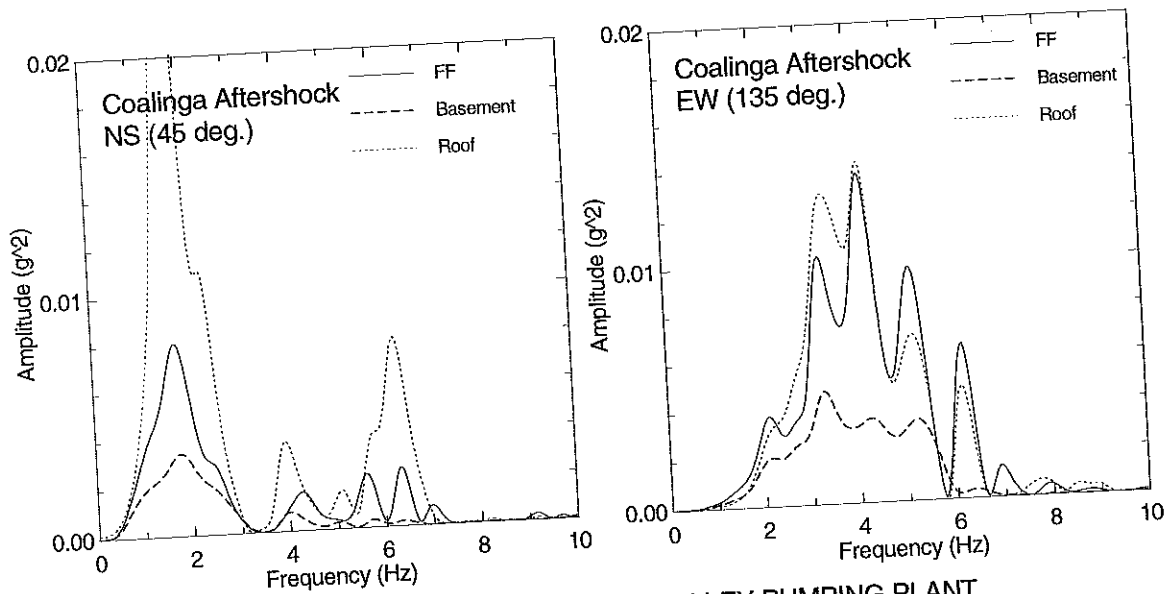


Fig. A9.CGA2: PLEASANT VALLEY PUMPING PLANT
Power Spectral Density of Selected Horizontal Accelerations
1983 Coalinga Earthquake Aftershock

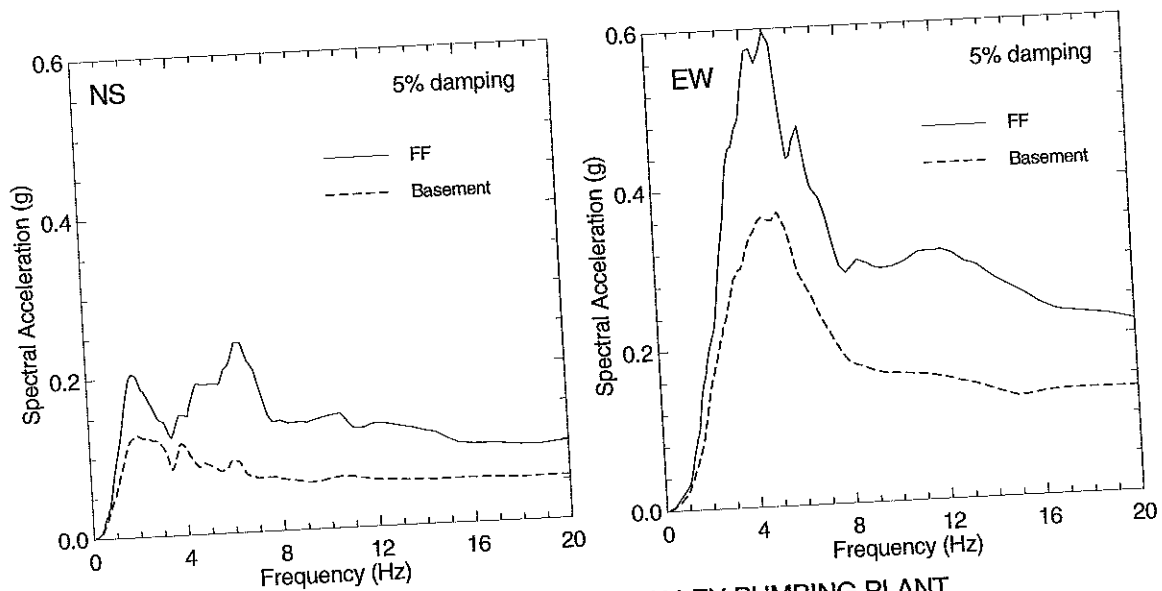


Fig. A9.CGA3: PLEASANT VALLEY PUMPING PLANT
Acceleration Response Spectra of Horizontal Basement and Free-Field Motions
1983 Coalinga Earthquake Aftershock

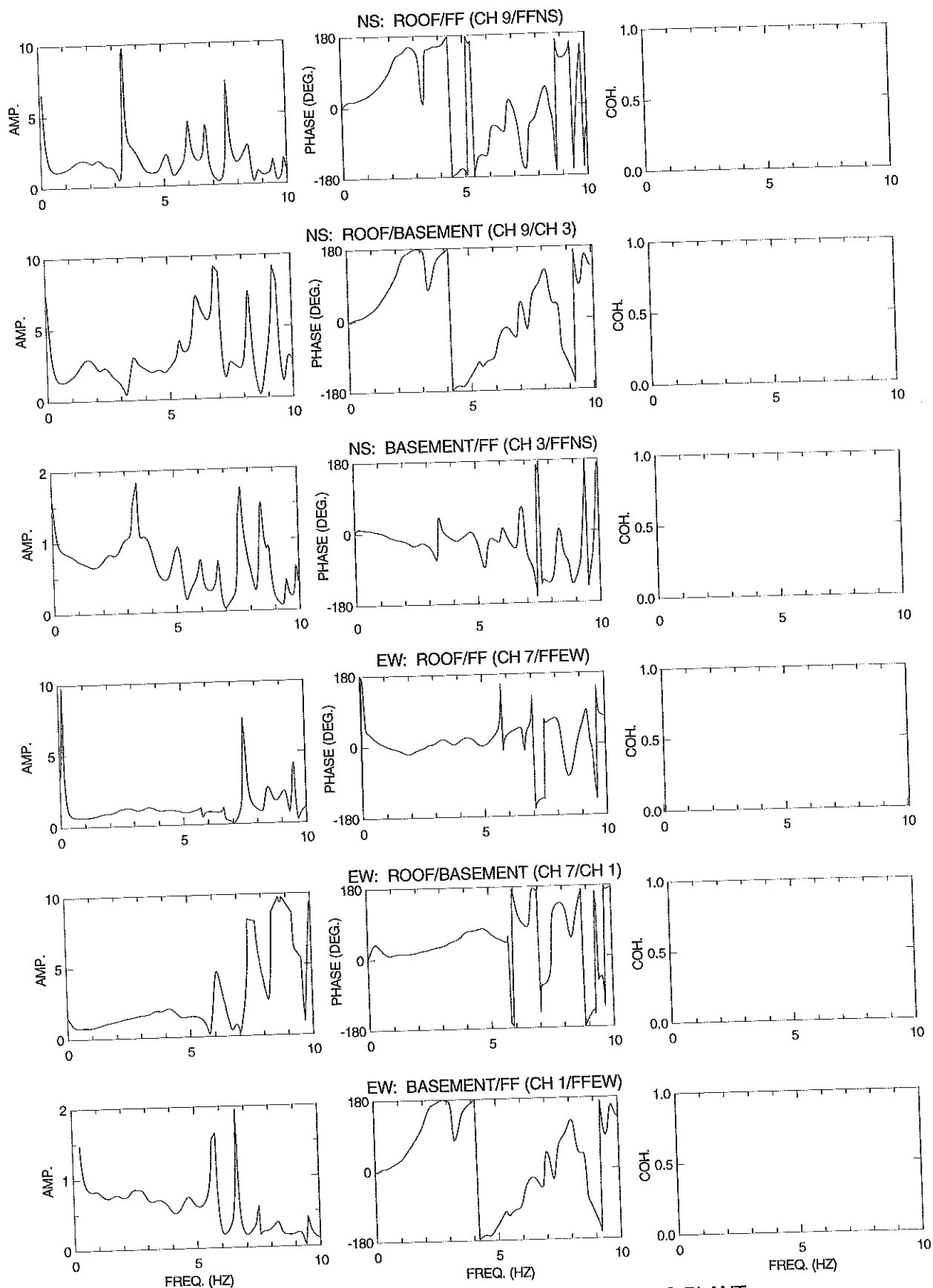


Fig. A9.CGA4: PLEASANT VALLEY PUMPING PLANT
Transmissibility and Coherency Functions
1983 Coalinga Earthquake Aftershock
A9-12

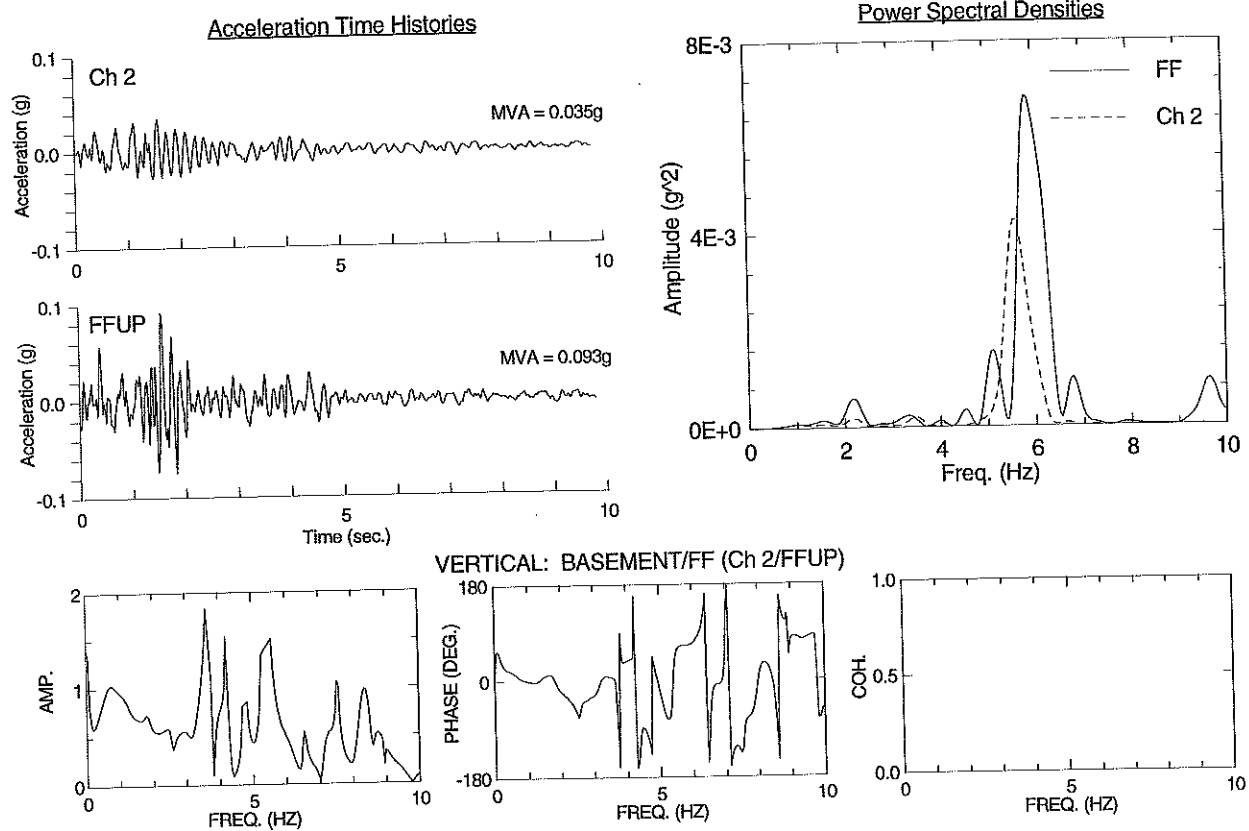


Fig. A9.CGA5: PLEASANT VALLEY PUMPING PLANT
Analysis of Vertical Motions
1983 Coalinga Earthquake Aftershock

SITE A10

Site Name: Richmond City Hall
Sensors: CSMIP 58503/58505

Lat: 37.938°
Long.: 122.342°

Floor Levels: 4
Bsmt. Levels: 1
Structure: Concrete moment frame with shear walls

Foundation: footings
Site Geology: soil
Design Date: 1948

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.12	0.11	0.031	0.12	0.086	0.035	0.24	0.19	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.3\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized

Final: FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A10.LP0).

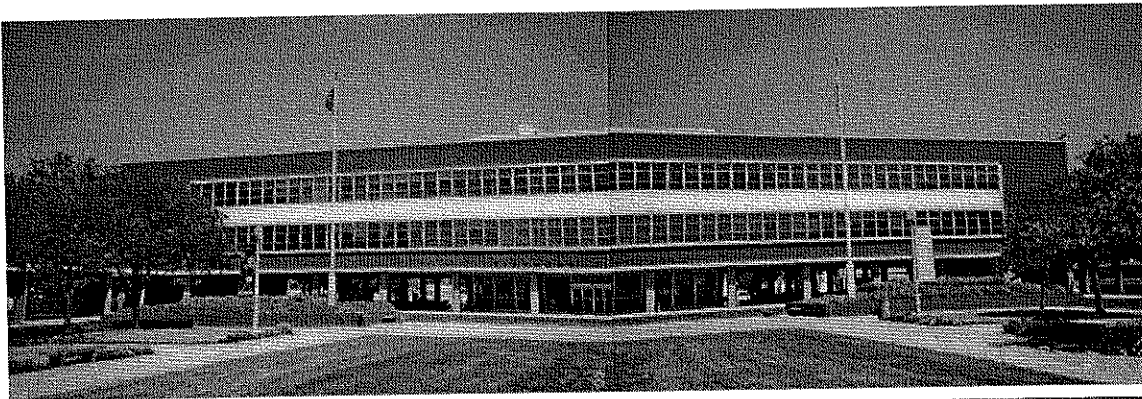
Figures

A10.1-A10.3

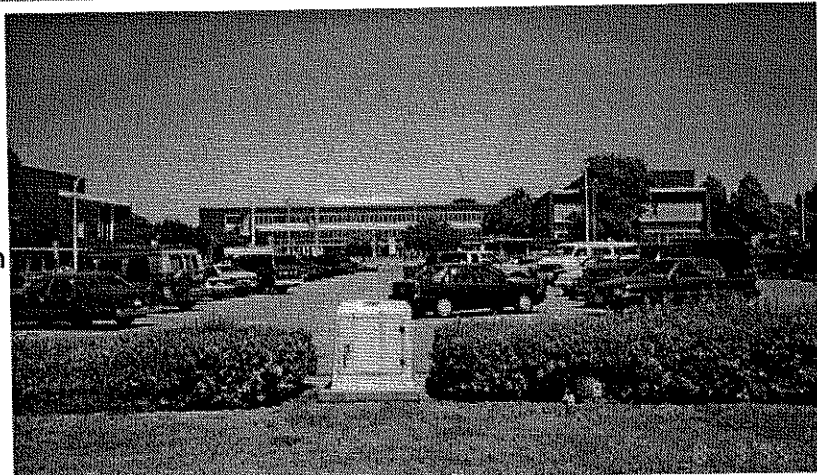
Site Plan, Sensor Locations, Generalized Soil Column

A10.LP0-6

LP Analysis Results



Above: View of structure,
looking north
Right: View of structure with
FF in foreground



DISCUSSION

Site Description

The building is 3 stories tall above ground level with a single-level basement, and is rectangular in plan view with dimensions 68 by 260 feet. Several other 2 to 3-story structures are present in the area, including a 2 story library located about 150 feet west of the free-field instrument. Lateral loads are resisted by a concrete moment frame coupled with transverse shear walls which extend from the basement to the roof. The foundation consists of spread footings. Accelerographs are present on the basement, 2nd, 3rd and roof levels. The free-field instrument is located about 650 feet south of the structure in a parking lot.

The site is located on level terrain. Subsurface conditions at the free-field location were investigated by WCC (1992), and consist of: 2 feet of sandy fill, 6 feet of Bay Mud, about 152 feet of firm to stiff silty clay, and 30 feet of clayey gravels and sandy clays which extend to sandstone bedrock at a depth of 190 feet. The upper 25 feet of the sandstone bedrock is highly weathered, with decreased weathering below 215 feet depth. Shear wave velocities were measured by downhole and suspension logging techniques by Agbabian Associates (1993).

Instrument Selection and Free-Field Conditions

Structural sensors are located along the east and west shear walls. Sensors on the east side of the building were used in the analyses (Table A10.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A10.LP1.

Table A10.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 8	Ch 13	Ch 1	Ch 2	Ch 10

The free field instrument is on level terrain, and the only significant structure near the instrument is a two-story public library about 140 feet to the west. As shown in Fig. A10.LP2, the significant energy in the EW free-field motions at $f \approx 3.2$ Hz may be associated with transverse vibrations of the library structure.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A10.LP4) and the spectra for horizontal motions (Fig. A10.LP2) generally indicate de-amplification of basement motions, although minor amplification of transverse basement motions occurred at the structure's fundamental frequency.

Transmissibility, coherency, and power spectral density functions for the FF and basement vertical motions are presented in Fig. A10.LP5. Minor amplification occurred near the transverse fundamental frequency, indicating that rocking may have contributed to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A10.LP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A10.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A10.LP6).

Table A10.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=1, j=4		L (EW) d=2, j=4		tr (NS) d=1, j=4		L (EW) d=0, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	3.53	2.7	3.88	1.9	3.44	3.5	3.84	7.7	3.31	10.1	3.73	14.4

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The small variation between the transverse fixed- and pseudo flexible-base parameters is consistent with the minor rocking detected in the vertical B/FF transmissibility function.
- Parametric models of FF-R response for both directions are of moderate quality, possibly due to incoherence effects associated with the 650-foot separation between the structure and FF.

The RPEM results for FF-R response were fairly unstable after 20 seconds (Fig. A10.LP6). As with the CEM analyses, this instability is likely associated at least in part with incoherence effects. Nevertheless, no time-dependent changes in the first-mode parameters of the structure were evident from the results. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and B-R first-mode frequencies.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A10.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 75 feet using the recorded free-field motions and the profile in Fig. A10.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the transverse direction were increased by 30% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A10.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 830$ fps	$h = 33$ ft	emb. = 10 ft	$\gamma = 0.15$
LP	$\nu = 0.45$		tr: $r_1 = 75$ ft	$r_2 = 54$ ft	
	$\beta^* = 3.7\%$	$V_s^* = 770$ fps	L: $r_1 = 75$ ft	$r_2 = 106$ ft	

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A10.3 with the fixed-base modal parameters in Table A10.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A10.4.

Table A10.4: Empirical and predicted inertial interaction effects

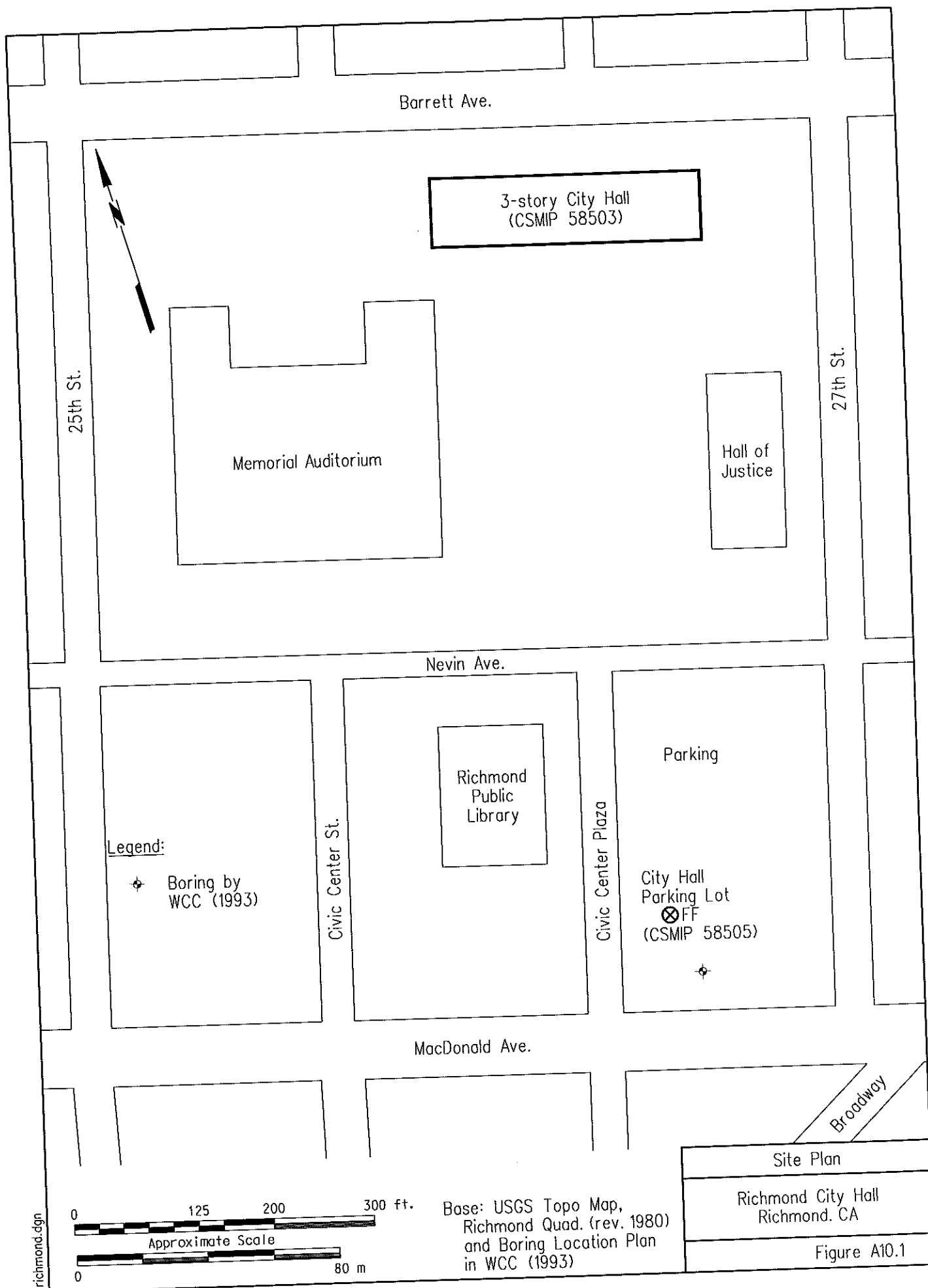
LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.07	7.9	10.1	1.04	12.7	14.4
	Pre.-Vel.	1.13	5.0	6.8	1.13	9.5	10.8
	Pre.-Bie.	1.11	11.4	13.4	1.16	22.1	23.4

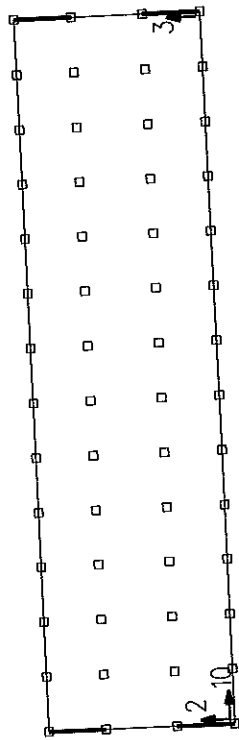
The period lengthening is overpredicted somewhat by both formulations, whereas the empirical foundation damping is intermediate between the two predictions.

Data Sources

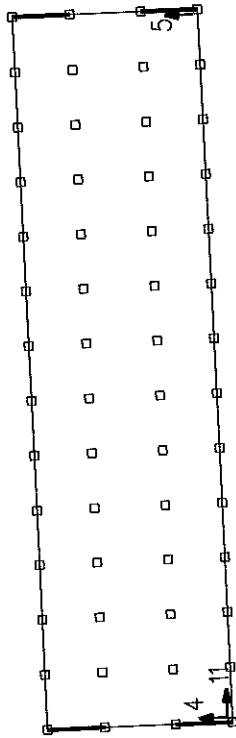
Agbabian Assoc. (1993). "Borehole geophysical measurements at 24 soil and rock sites," *Rpt. No. 9225-6427*.

Woodward-Clyde Consultants, WCC (1993). "Field investigations at selected strong motion accelerograph sites," *Project No. 90C0588*, prepared for Electrical Power Research Institute.

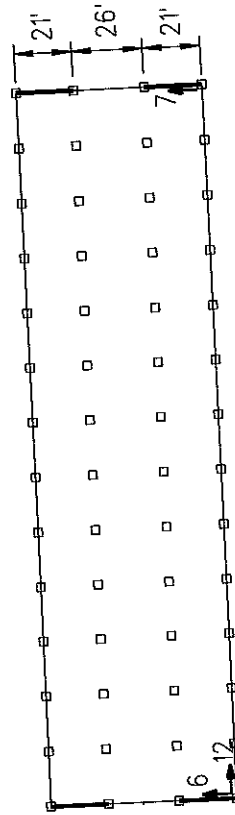




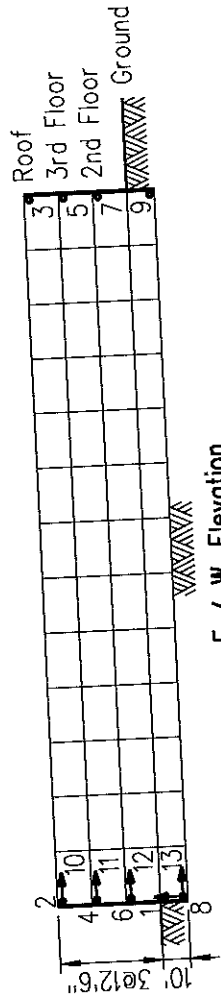
Roof Plan



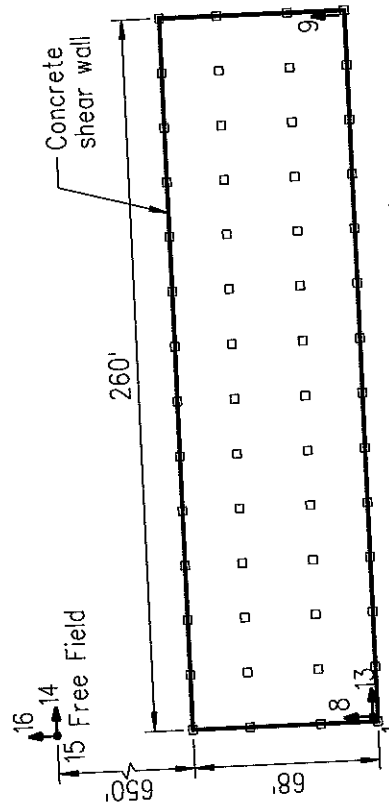
3rd Floor Plan



2nd Floor Plan



E / W Elevation



Basement Plan

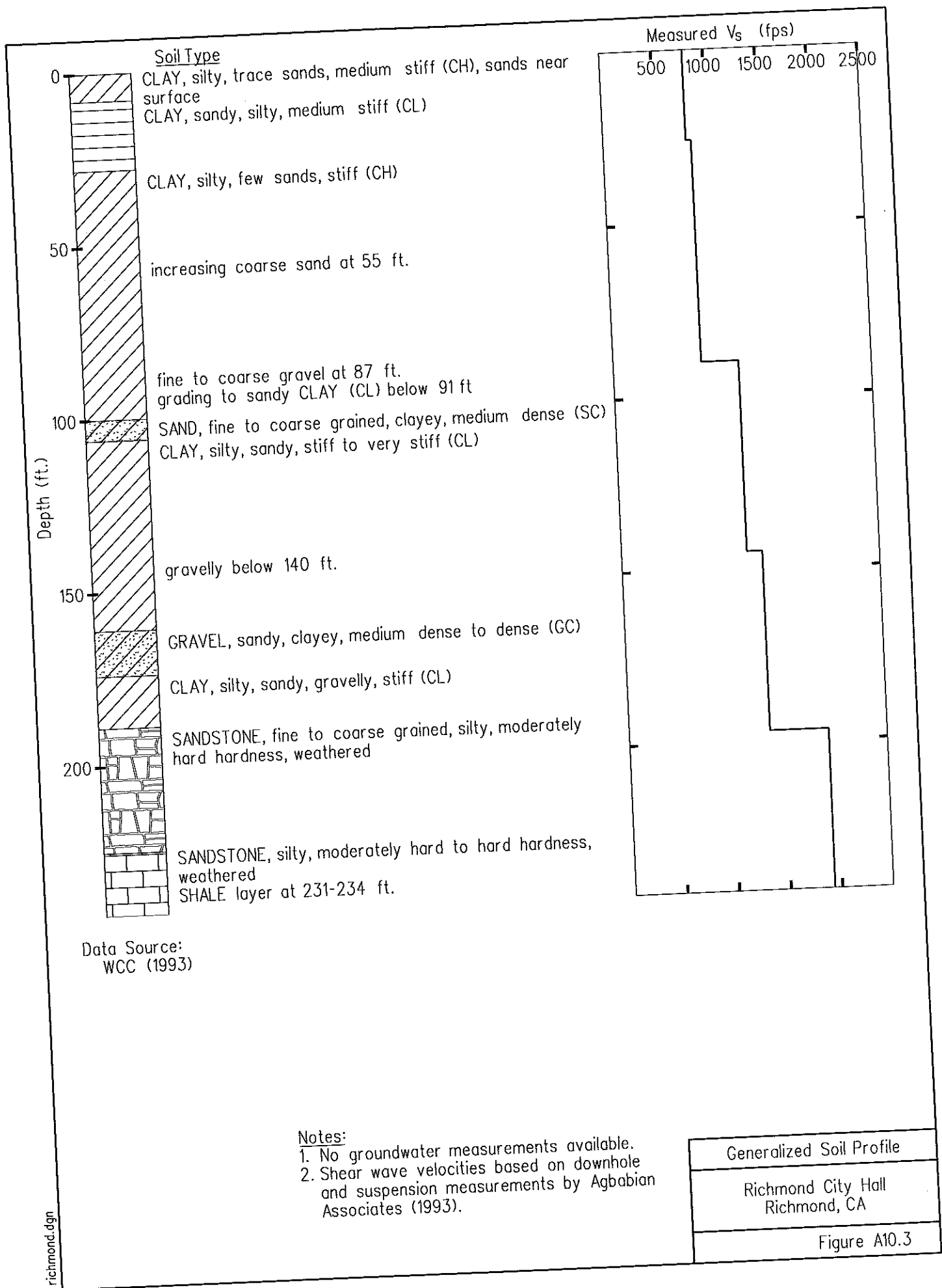


Concrete moment-resisting frame
and concrete shear walls

Sensor Locations

Richmond City Hall
Richmond, CA

Figure A10.2



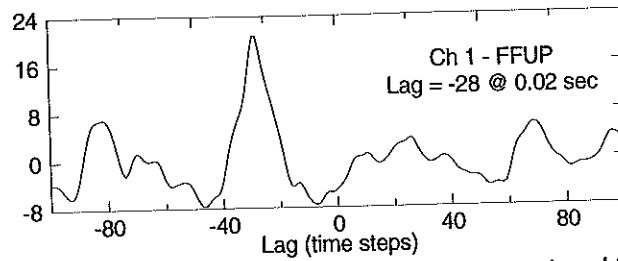


Fig. A10.LP0: Cross-correlation function between vertical basement and free-field accelerations

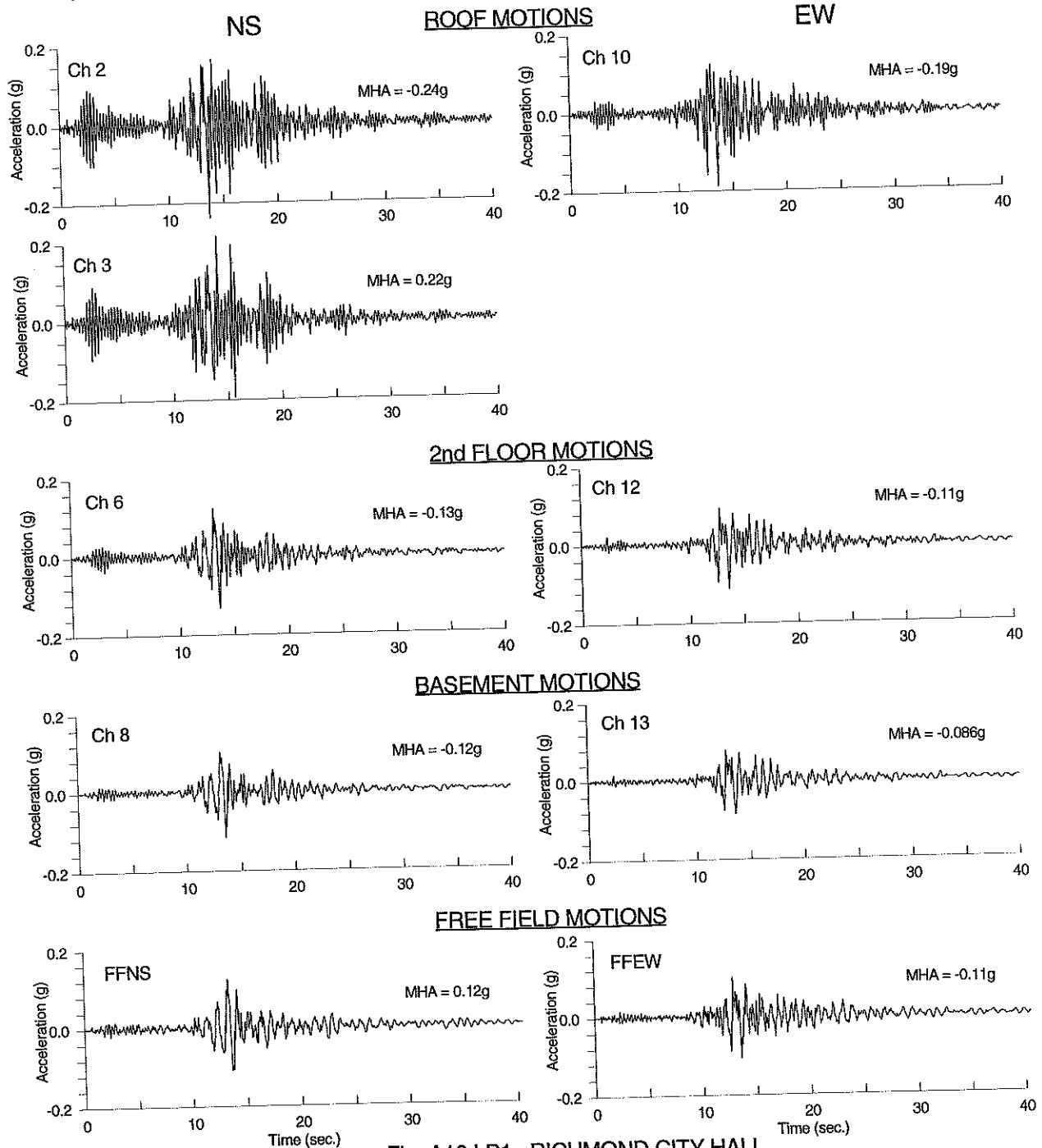


Fig. A10.LP1: RICHMOND CITY HALL
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

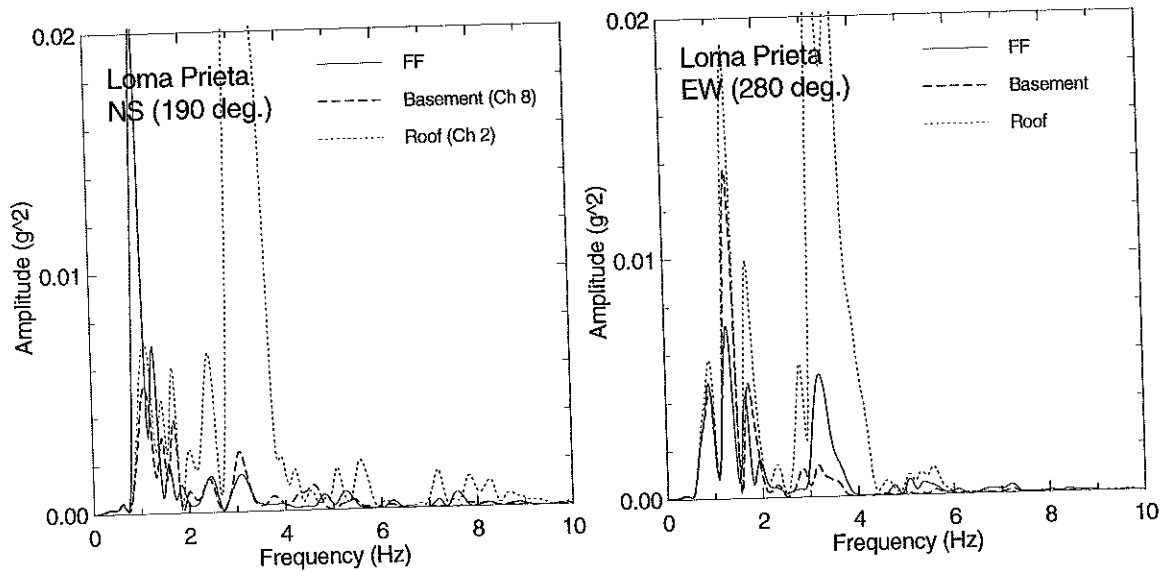


Fig. A10.LP2: RICHMOND CITY HALL
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

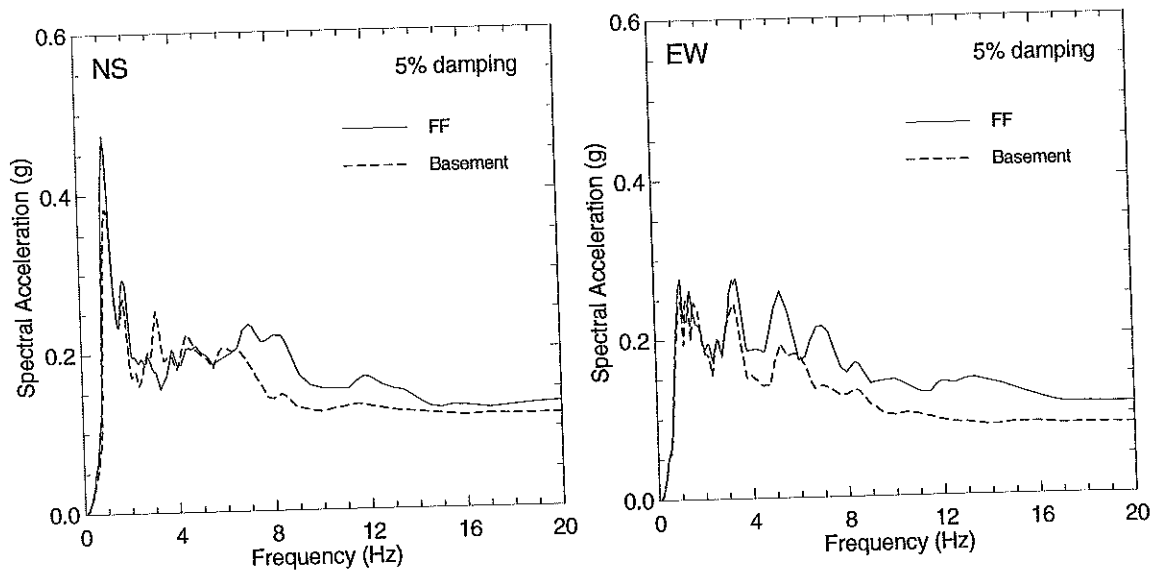


Fig. A10.LP3: RICHMOND CITY HALL
Acceleration Response Spectra of Horizontal Basement and Free-Field Motions
1989 Loma Prieta Earthquake

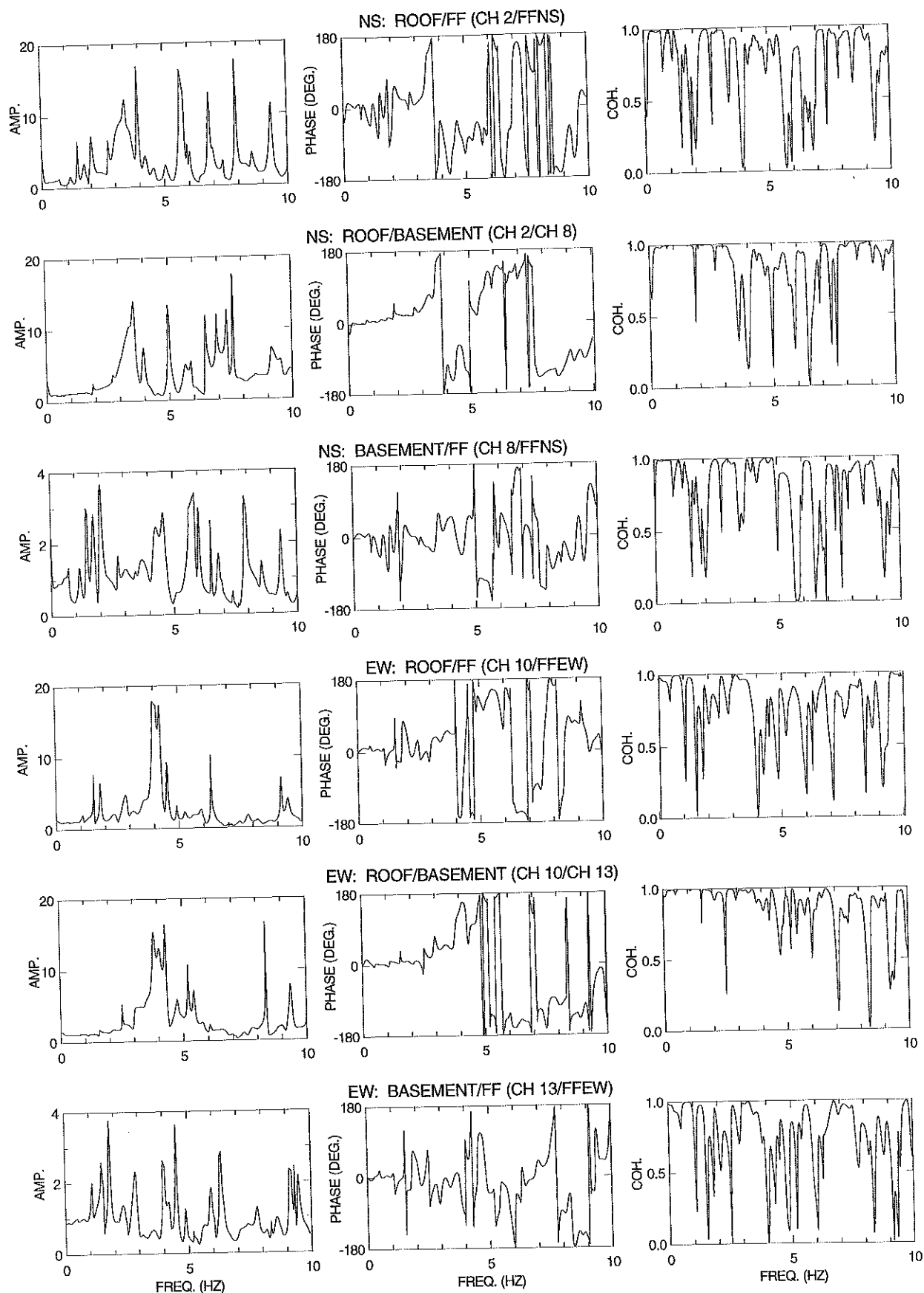


Fig. A10.LP4: RICHMOND CITY HALL
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A10-10

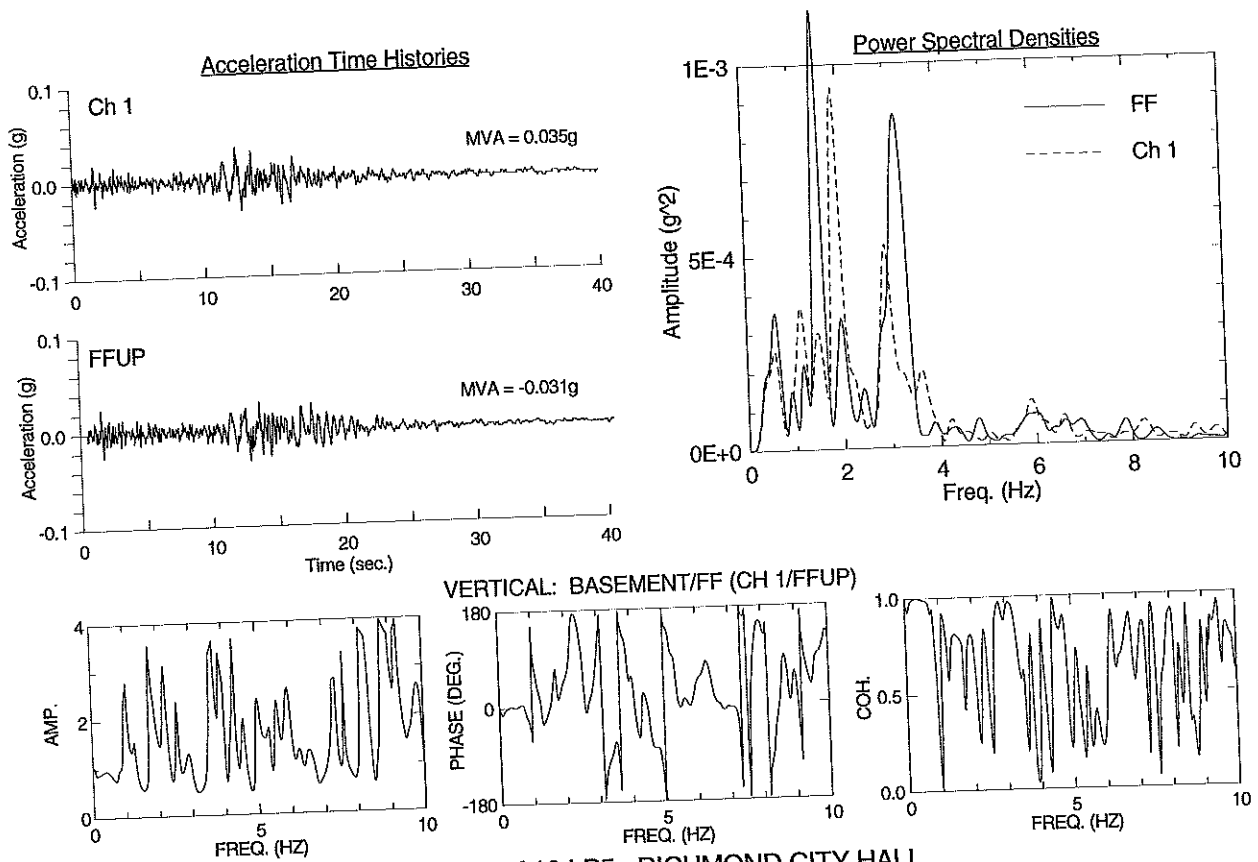


Fig. A10.LP5: RICHMOND CITY HALL
Analysis of Vertical Motions
1989 Loma Prieta Earthquake

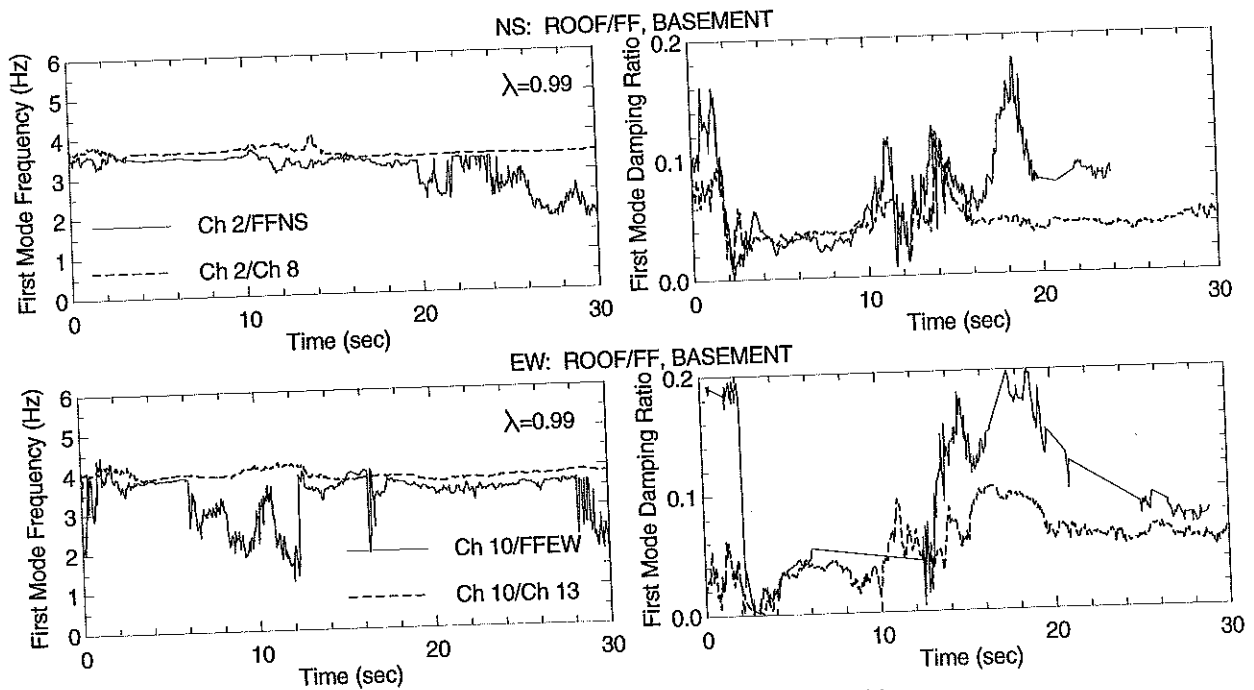


Fig. A10.LP6: RICHMOND CITY HALL
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE A11

Site Name: San Jose IBM Almaden Research Facility	Lat: 37.212°
Sensors: CSMIP 57562/57563	Long.: 121.803°
Floor Levels: 3	Foundation: footings
Bsmt. Levels: 0	Site Geology: rock
Structure: Steel moment frame	Design Date: 1983

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.27	0.23	0.21	0.16	0.20	0.11	0.68	0.58	--
1993 Alum Rock*	0.031	0.032	0.013	0.017	0.027	0.014	0.043	0.066	--

* = Not processed -- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

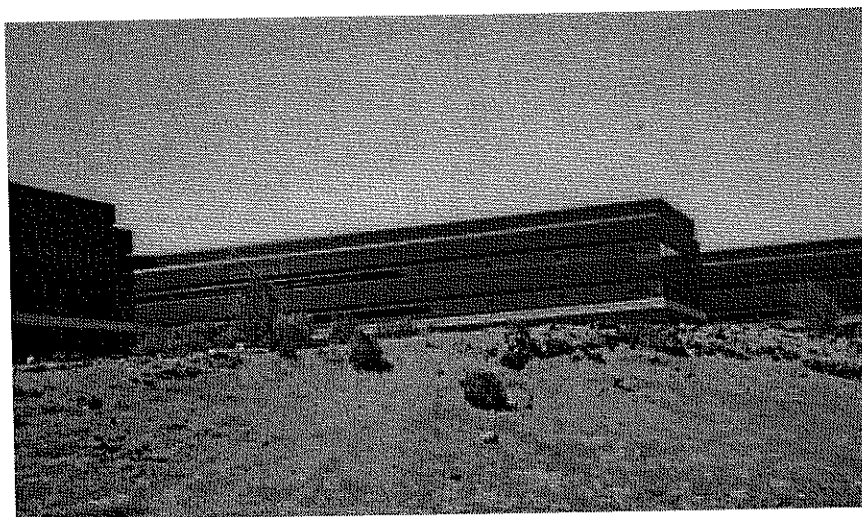
Preliminary: CSMIP; filtered, $f = 0.16\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized

Final: FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A11.LP0).

Figures

A11.1-A11.3
A11.LP0-6

Site Plan, Sensor Locations, Generalized Soil Column
LP Analysis Results



View of structure from Harry Road, looking south-west

DISCUSSION

Site Description

The building is 3 stories tall above ground level with no basement, and is essentially rectangular in plan with dimensions 93 by 250 feet. As shown in Fig. A11.1, the subject building is one wing of a facility comprised of six interconnected structures. The building was designed to be structurally independent of the adjoining building (a 4-inch separation joint is present at the interface). The building is founded on spread footings and has a concrete base slab. Accelerographs are present on the 1st, 3rd, and roof levels of the structure. The free-field instrument is located 508 feet south of the structure in a parking lot.

The site is situated on a level building pad atop a broad, EW trending ridge in the Santa Teresa Hills of San Jose. The building pad was established by cuts of up to 35 feet and fill sections up to 15 feet thick around the perimeter of the site. Both the instrumented building and the free-field instrument are located in cut areas. Geologic exploration of the site was conducted prior to development by WCC (1982). The materials encountered below the present ground surface elevation were a fractured serpentinite mass characterized by hard rock blocks within a softer, sheared matrix. The hard rock blocks were most frequently encountered at depths of about 5 to 30 feet below the present ground surface. The shear wave velocities in Fig. A11.LP3 were estimated from seismic refraction measurements (of P-wave velocity) using a Poisson Ratio of $\nu = 0.33$.

Instrument Selection and Free-Field Conditions

Structural instrumentation is located at the east and west ends of the building, and sensors on the east end were used in the analyses (Table A11.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A11.LP1.

Table A11.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 2	Ch 1	Ch 9	Ch 8

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 1st Floor and Free-Field Motions

1F/FF transmissibility functions (Fig. A11.LP4) and the spectra for horizontal motions (Fig. A11.LP2-3) generally indicate de-amplification of both NS and EW motions for frequencies greater than about 2 Hz and 4 Hz, respectively.

Transmissibility, coherency, and power spectral density functions for the FF and 1F vertical motions are presented in Fig. A11.LP5. No significant amplification is observed at the first-mode response frequencies, suggesting that rocking was unlikely to have significantly affected the structural response. However, definitive conclusions about rocking can not be derived from these data since the vertical sensor is located at the center of the building in the transverse direction.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A11.LP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A11.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A11.LP6).

Table A11.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=4, j=4		L (EW) d=3, j=4		tr (NS) d=3, j=4		L (EW) d=4, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	--	--	--	--	1.61	26.7	1.65	26.0	1.55	23.4	1.70	21.5
	2					5.90	26.5	6.64	20.5	5.91	24.9	6.58	22.0

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- Fixed-base parameter estimation procedures could not be invoked for the longitudinal direction because $\tilde{f}_1 > \tilde{f}_1^*$. As noted below, a similar condition was encountered in RPEM analyses for the transverse direction. However, since no significant rocking effect was evident from the vertical 1F/FF transmissibility function, the fixed-base parameters can be approximated by the pseudo flexible-base parameters for both directions.

From the RPEM results (Fig. A11.LP6), first-mode frequencies are seen to be high (and somewhat unstable) during the build-up in shaking amplitude over the first 10 seconds, to remain at a relatively low and steady value for the next 10 seconds, and then to increase gradually during the low-level shaking after 20 seconds. Damping values were generally observed to decrease after the high intensity shaking. No structural damage was reported by the building owner (Saade, 1990). The RPEM results for the transverse direction contradicted the CEM results by indicating $\tilde{f} > \tilde{f}^*$. This condition was also encountered in the longitudinal direction, which, in this case, was consistent with the CEM result.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A11.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 86 feet using the recorded free-field motions and the profile in Fig. A11.3. The subject building was assumed to be independent of the adjacent building for the calculation of foundation radii. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for both the transverse and longitudinal directions were increased by a factor of 2.0 to account for shape effects.

Table A11.3: Soil/structure parameters used in predictive analyses

LP	Geotechnical		Structural		
	Model = H	$V_s = 2690$ fps	$h = 35$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.33$		tr: $r_1 = 86$ ft		$r_2 = 68$ ft
	$\beta^* = 0.9\%$	$V_s^* = 2640$ fps	L: $r_1 = 86$ ft		$r_2 = 111$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A11.3 with the fixed-base modal parameters in Table A11.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A11.4.

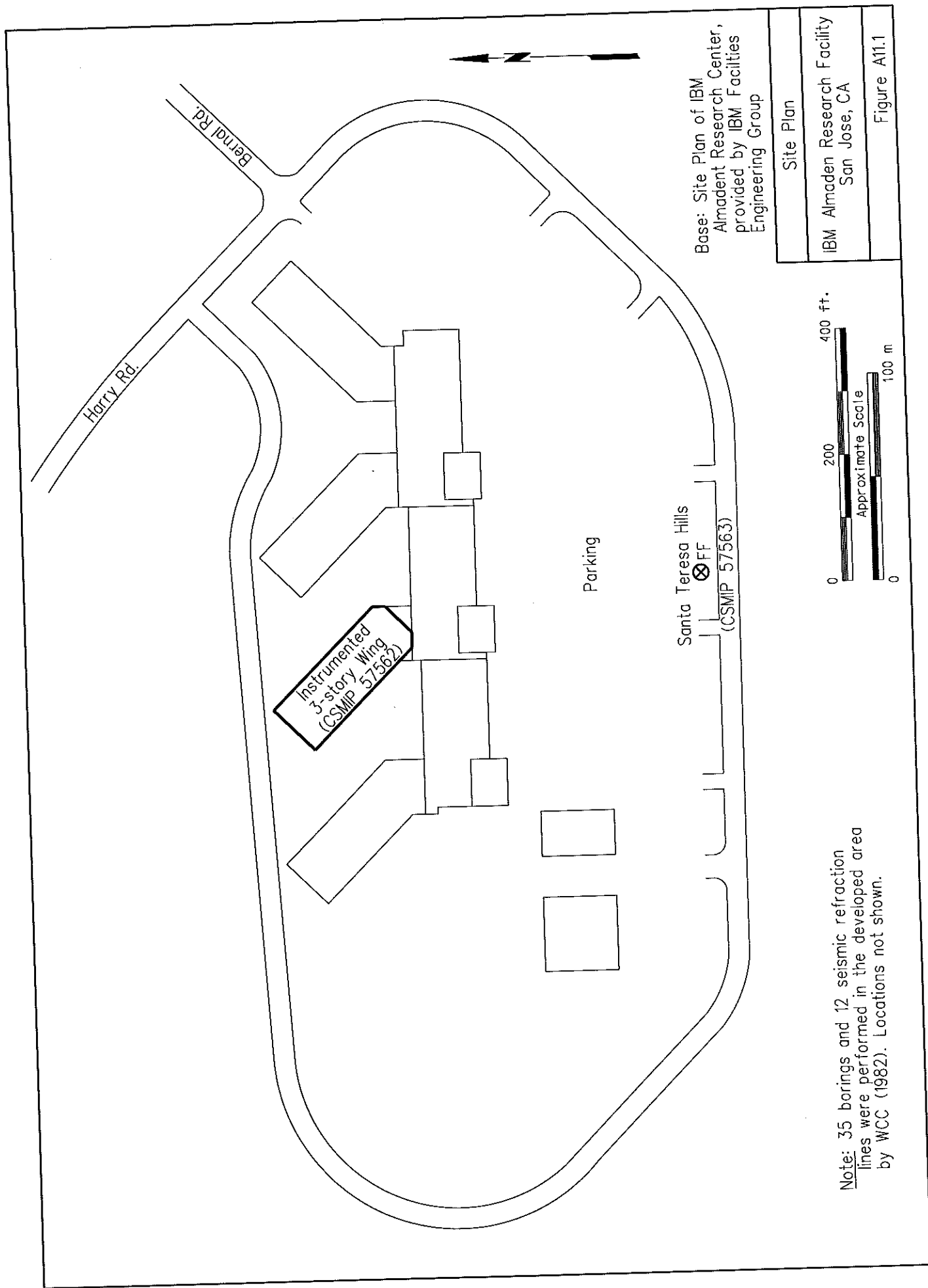
Table A11.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.04	-0.4	23.4	0.97	-4.5	21.5
	Prediction	1.003	0.0	26.5	1.003	0.1	25.8

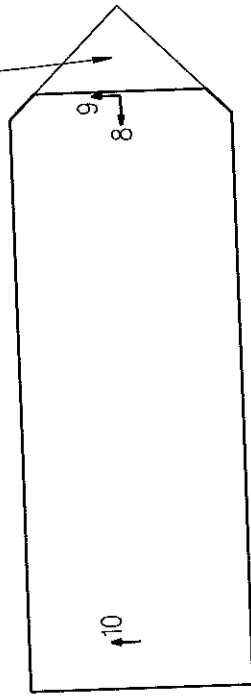
The predicted period lengthening and damping are generally satisfactory. The negative value for soil damping factor $\tilde{\zeta}_0$ and $\tilde{T}/T < 1$ from the system identification analyses indicate small numerical errors in the system identification results. Nonetheless, the minor effects of SSI on the response of this building were satisfactorily predicted.

Data Sources

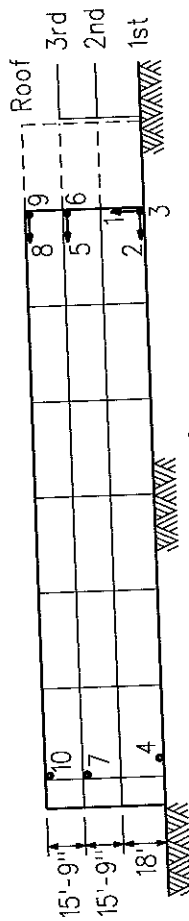
- Saade, N.M. (1990). "Post Loma Prieta Earthquake assessment summary," IBM internal memorandum.
- Woodward-Clyde Consultants, WCC (1982). "Geotechnical Investigation, IBM Almaden Research Laboratory, Stile Ranch, San Jose, CA," *Project No. 14250-W*.



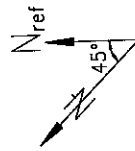
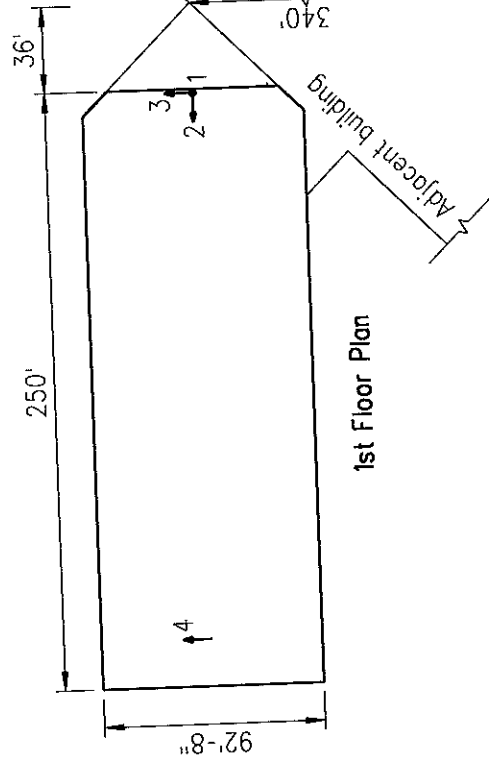
Framing in this area is not part of the lateral force resisting system.



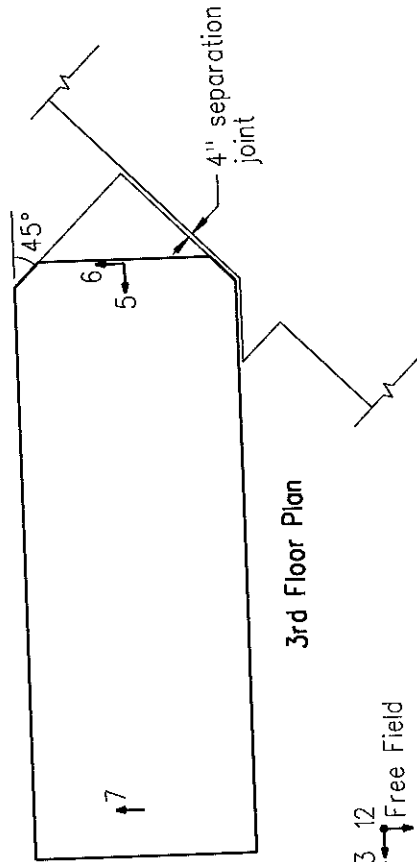
Roof Level Plan



W / E Elevation



1st Floor Plan



3rd Floor Plan

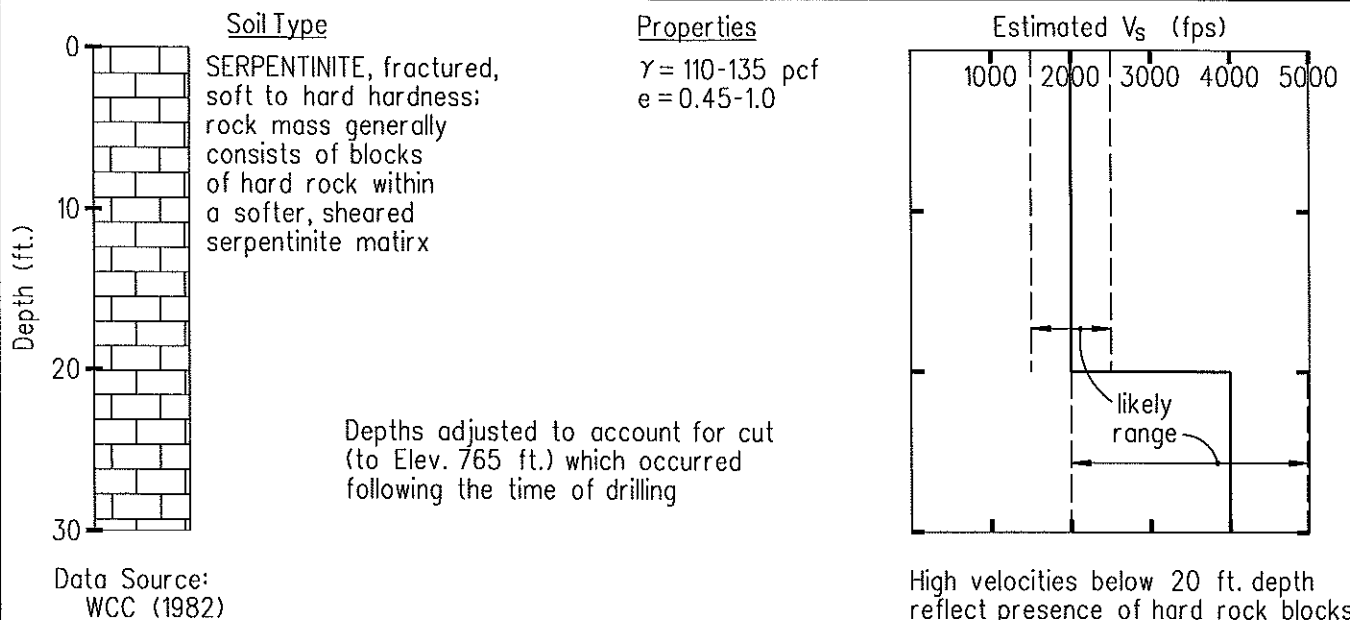
Sensor Locations

IBM Almaden Research Facility
San Jose, CA

Figure A11.2

Note: Data provided by CSMIP

sanjose.dgn



High velocities below 20 ft. depth reflect presence of hard rock blocks commonly encountered at depths varying from about 5-30 ft. in seismic refraction lines.

Notes:

1. Groundwater not encountered in most borings, but was encountered at depths as shallow as 5 ft. below the present ground surface in some borings.
2. Shear wave velocities based on seismic refraction measurements of p-wave velocity by WCC (1982) and an assumed Poisson Ratio of $\nu = 0.33$.

Generalized Soil Profile
IBM Almaden Research Facility San Jose, CA
Figure A11.3

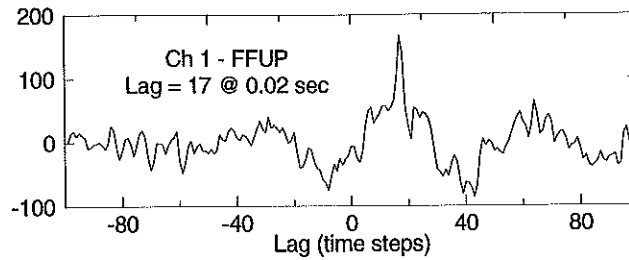


Fig. A11.LP0: Cross-correlation function between vertical 1st floor and free-field accelerations

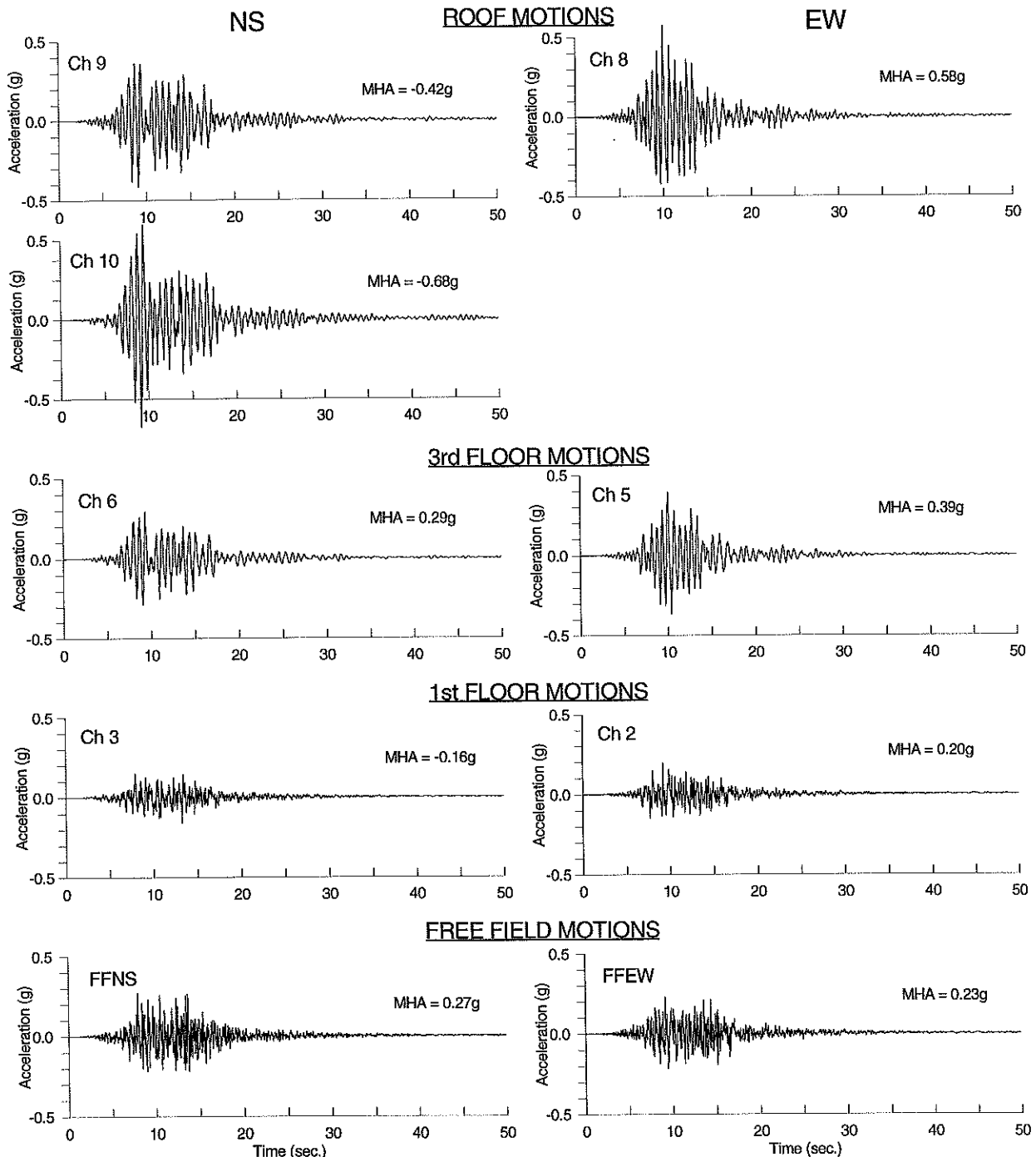


Fig. A11.LP1: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Horizontal Acceleration Time Histories
1989 Loma Prieta Earthquake

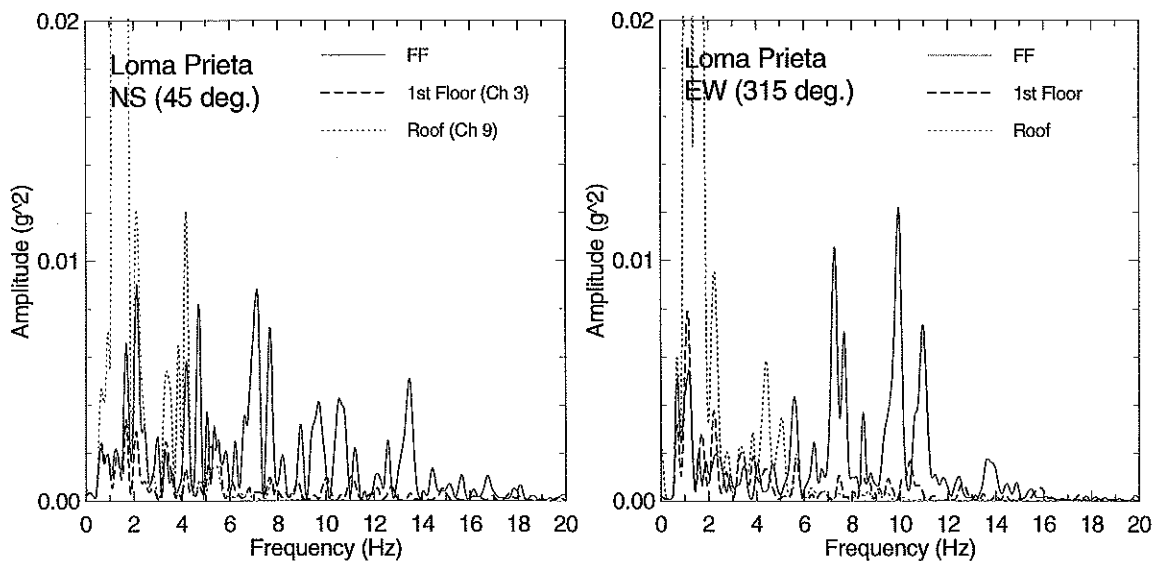


Fig. A11.LP2: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Power Spectral Density of Selected Horizontal Accelerations
1989 Loma Prieta Earthquake

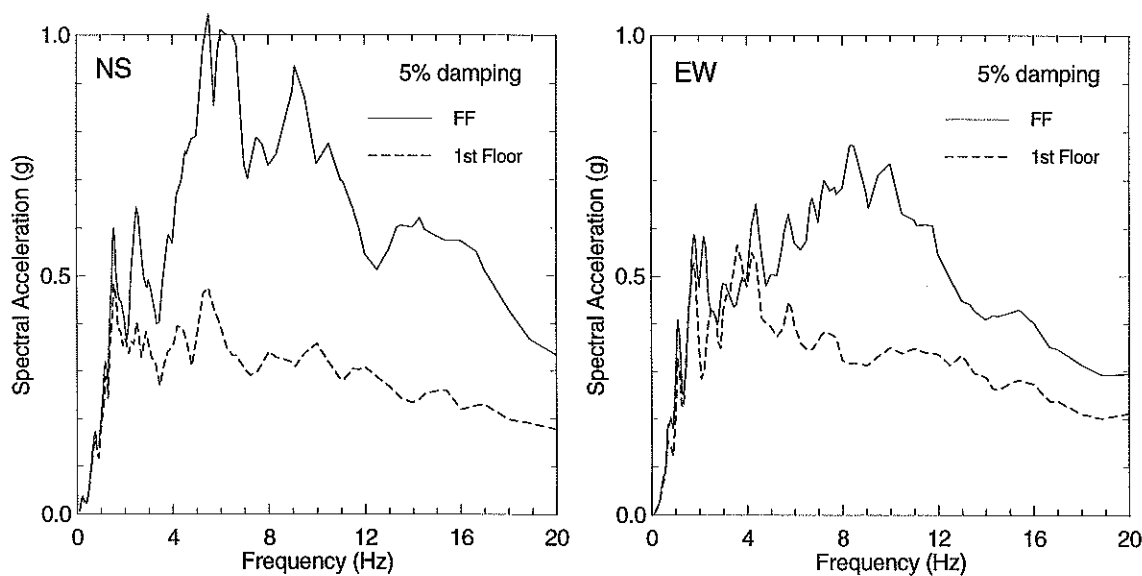


Fig. A11.LP3: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Acceleration Response Spectra of Horizontal 1st Floor and Free-Field Motions
1989 Loma Prieta Earthquake

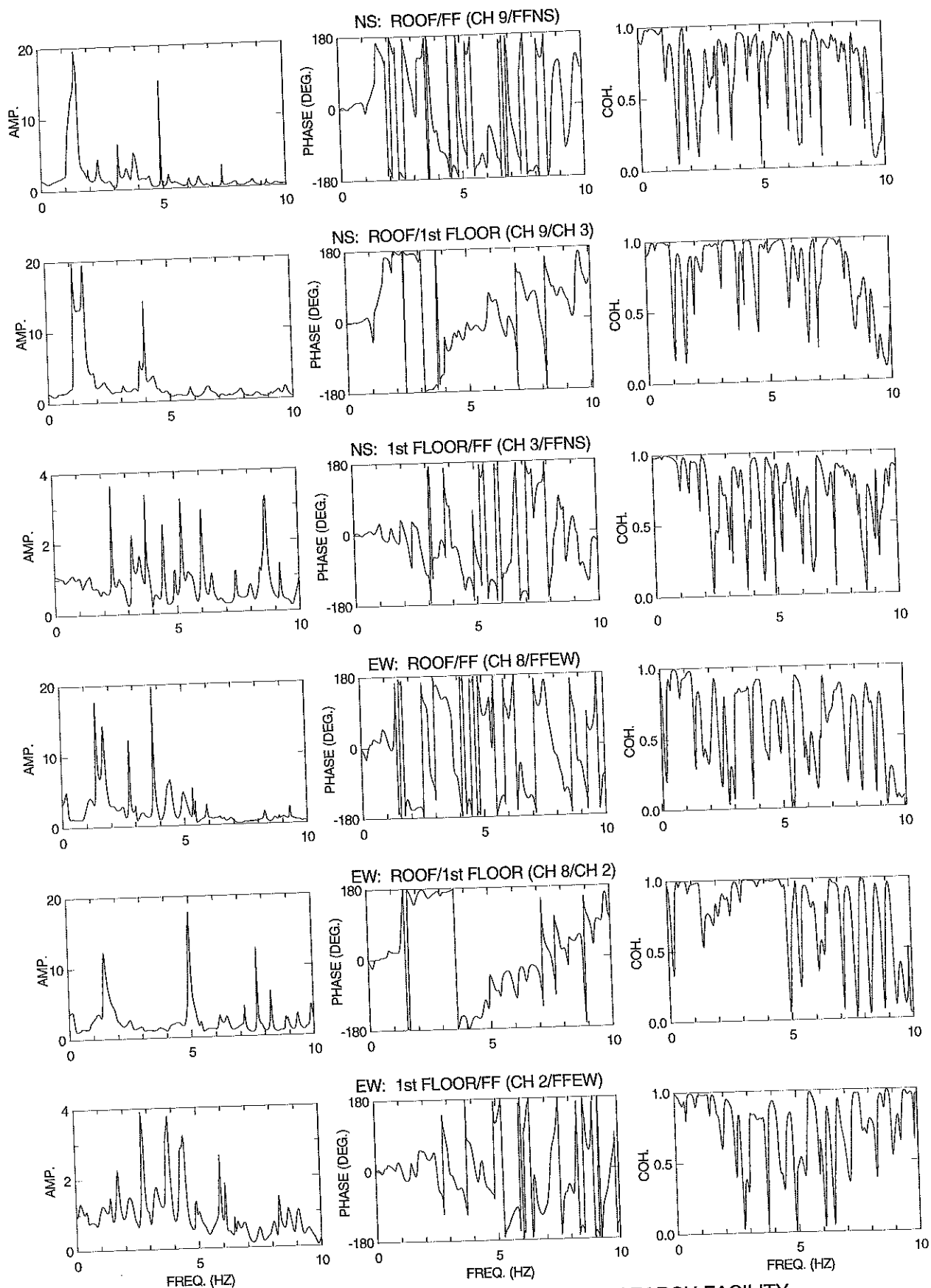


Fig. A11.LP4: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake
A11-10

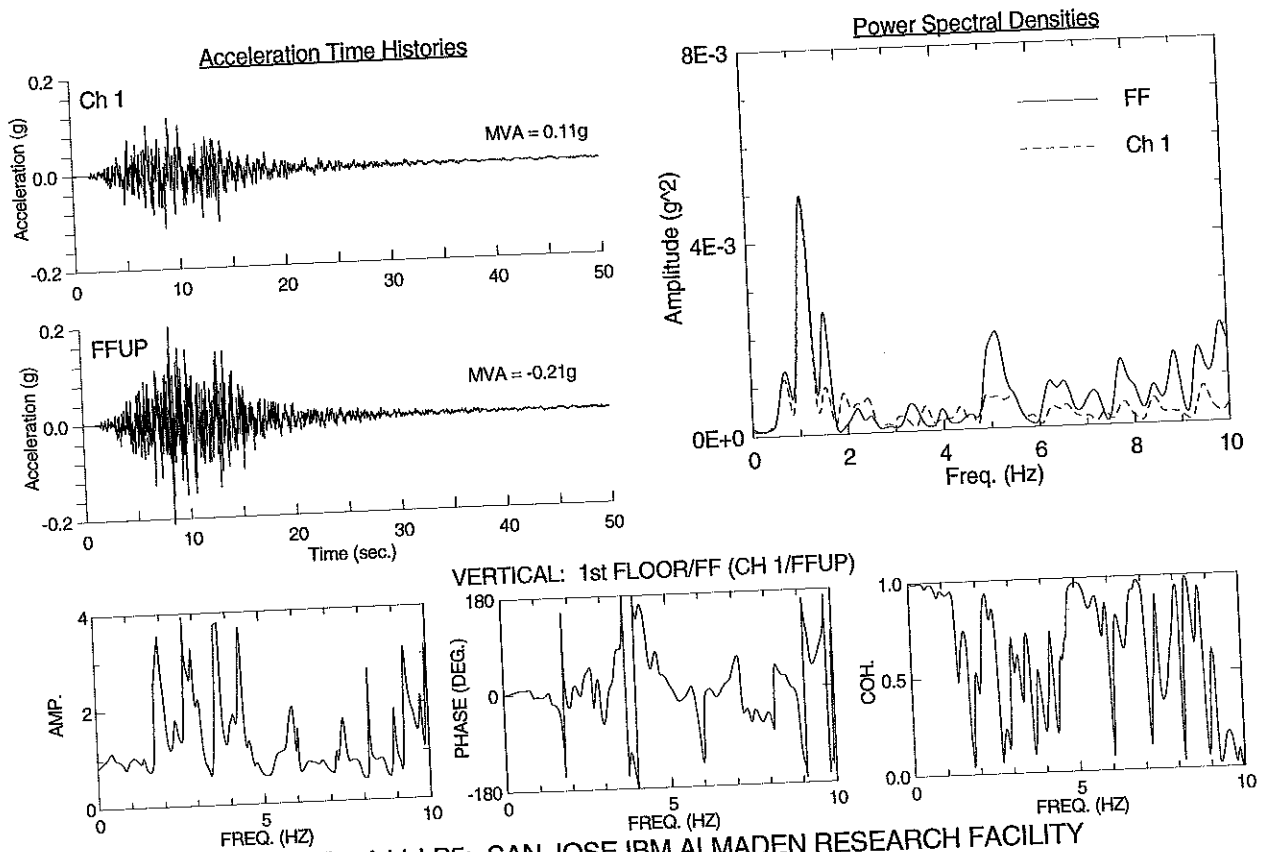


Fig. A11.LP5: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Analysis of Vertical Motions
1989 Loma Prieta Earthquake

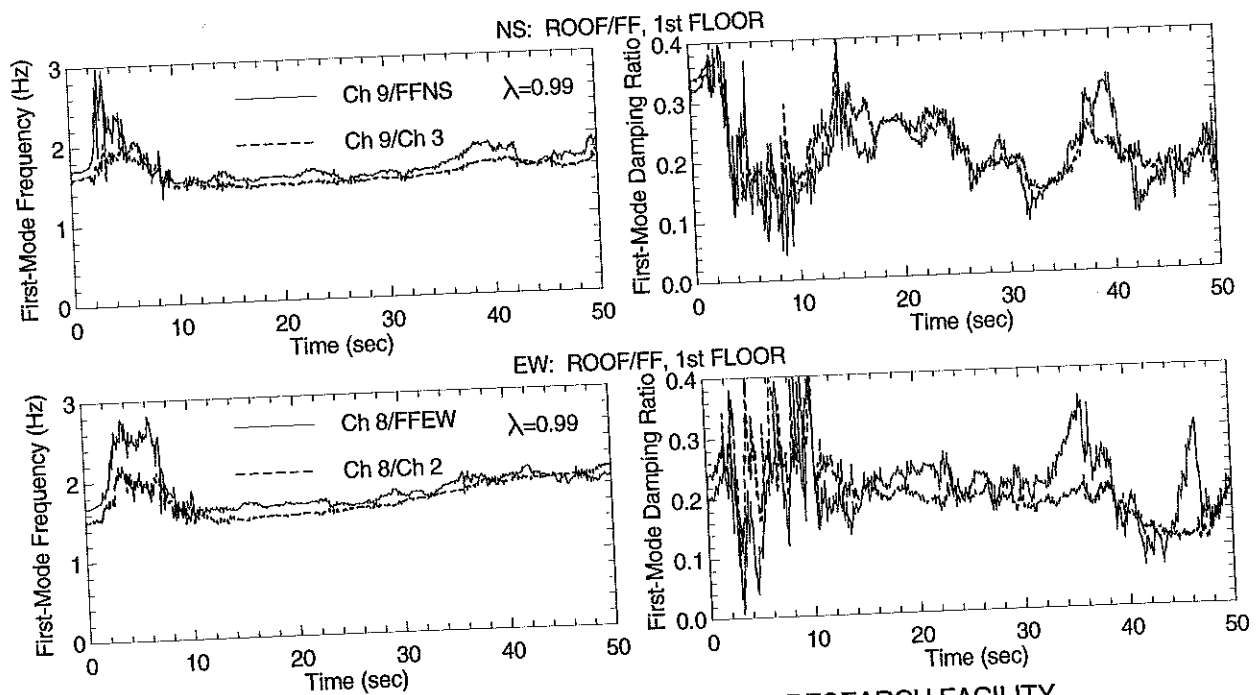


Fig. A11.LP6: SAN JOSE IBM ALMADEN RESEARCH FACILITY
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE A12

Site Name: El Centro, Imperial County Services Bldg.	Lat: 32.792°
Sensors: CSMIP 1260/1335	Long.: 115.564°
Floor Levels: 6	Foundation: concrete piles
Bsmt. Levels: 0	Site Geology: Alluvium
Structure: Concrete frames and shear walls	Design Date: 1968

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1979 Imperial Valley	0.21	0.24	0.24	0.29	0.33	0.18	0.56	0.45	--

-- = No instrument

Preprocessing

Imperial Valley Earthquake (IMP)

Preliminary: by CSMIP; filtered, $f=0.17-23$ Hz; $f_{Nyq}=50$ Hz; FF/str. data not synchronized

Final: decimated by 2 to $f_{Nyq}=25$ Hz; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A12.IMP0).

Figures

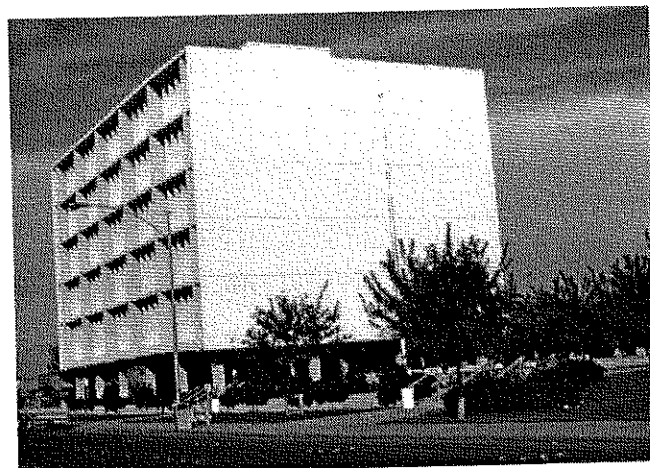
A12.1-A12.3

Site Plan, Sensor Locations, Generalized Soil Column

A12.IMP0-6

IMP Analysis Results

View of structure,
looking north-west
(photo from Karl V.
Steinbrugge collection)



View of damaged columns
at east end of structure
(photo from Karl. V.
Steinbrugge collection)



DISCUSSION

Site Description

The original 6-story Imperial County Services building was severely damaged by the 1979 Imperial Valley Earthquake and subsequently demolished. The building was replaced in 1981 with a 2-story steel frame structure. The analyses discussed herein refer to the original 6-story structure. The building had no basement and was rectangular in plan view with dimensions of about 85 by 137 feet. Lateral loads were resisted by reinforced concrete frames in the longitudinal direction and reinforced concrete shear walls in the transverse direction. The foundation of the former and existing structure consists of 45 to 60-foot deep Raymond step-tapered concrete piles inter-connected at their heads by tie beams. As shown in Fig. A12.2, strong motion sensors were present at the ground, 2nd, 4th, and 7th (roof) levels. The most significant damage from the 1979 earthquake was the partial collapse and buckling of four columns at the building's east end just above ground level. The pattern of column damage is suggestive of frame yielding in the longitudinal direction and axial-force failure due to transverse overturning (Rojahn and Mork, 1981).

The site is located on level terrain in the central portion of the arid Salton Trough, a large alluvial depression extending from the San Bernardino Mountains to the Gulf of California. Surficial soil conditions in El Centro have been fairly well established by a 400-foot boring at the Imperial Irrigation District Terminal Station Property (SW/AA, 1975) located about 4500 feet northeast of the site, as well as borings up to 100-feet deep at the subject site (Porcella, 1984; WCC, 1981). The generalized soil profile is shown in Fig. A12.3, and consists of interbedded silty clays and sands which gradually increase in stiffness with depth to the maximum depth explored of 400 feet. The shear wave velocities shown in Fig. A12.3 are based on downhole measurements by Porcella (1984) and SW/AA (1975). Depth to bedrock is poorly established in the area. Using the subsurface data and power spectra of the free-field motions (Figs. A12.IMP2), a site frequency of $f \approx 0.7$ to 0.8 Hz is estimated for the levels of soil strain associated with this earthquake.

Instrument Selection and Free-Field Conditions

Instruments near the centroid of the building (except at the ground floor) were selected for use in the analysis (Table A12.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A12.IMP1.

Table A12.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 11	Ch 13	Ch 12	Ch 2	Ch 4

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A12.IMP4) and the spectra for horizontal motions (Fig. A12.IMP2-2) indicate significant amplification of GF motions (especially in the NS direction) for $f < 5$ Hz, and de-amplification at higher frequencies. The low-frequency amplification has been investigated as possibly being associated with a resonant foundation response frequency which is distinct from the site and structure frequencies. Forced vibration tests on the foundation in the transverse (NS) direction (after removal of the original building) showed no significant frequency dependence in the foundation stiffness (Hall, 1984), indicating that GF/FF amplification associated with a fundamental foundation vibration mode is unlikely.

Transmissibility, coherency, and power spectral density functions for FF and GF vertical motions are presented in Fig. A12.IMP5. Amplification near the transverse first-mode response frequency ($\tilde{f}_1 \approx 1.5$ Hz) indicates that base rocking likely contributed to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A12.IMP4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A12.2), and (3) time variation of first-mode parameters from the RPDM (Fig. A12.IMP6).

Table A12.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=4, j=8		L (EW) d=7, j=8		tr (NS) d=7, j=8		L (EW) d=9, j=8	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
IMP	1	1.66	12.5	0.82	38.4	1.56	13.5	0.81	37.4	1.50	20.8	0.80	36.2

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode. Damage to the structure during the earthquake resulted in high first-mode damping ratios ($> 20\%$).
- Due to the highly nonlinear structural response, the condition of linear, time-invariant structural behavior assumed in CEM parametric analyses is not met. Since the damage occurred fairly early in the time history (see RPDM results below), the CEM results are principally based on the post-damage vibration properties of the structure.
- The modest difference between the transverse fixed- and pseudo flexible-base frequencies is consistent with the rocking effect evident from the vertical GF/FF transmissibility function. There appears to be no significant inertial interaction effect in the longitudinal direction.

RPEM parametric analyses are better suited than CEM analyses to evaluate the modal parameters associated with the nonlinear response of this structure. Results for the transverse (NS) direction indicate significant reductions in first-mode frequency (about 0.3 to 0.5 Hz) at about 6-8 seconds and 16-20 seconds. In the longitudinal direction, a decrease in first-mode frequency of about 0.5 Hz occurred at 6-10 seconds. The flexible-base frequencies in Fig. A12.IMP6 generally agree with frequency estimates by Hadjian et al. (1990) made from response spectra ratios for different intervals in the time history. Damping ratios greater than 30% occurred for $t < 22$ seconds (when the structural damage occurred), and decreased subsequently to $< 15\%$ during lower amplitude shaking in the later portion of the time history.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A12.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 61 feet using the recorded free-field motions and the profile in Fig. A12.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though the rocking radiation damping for the longitudinal direction was increased by 40% to account for shape effects.

Table A12.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 580$ fps	$h = 54$ ft	emb.=0	$\gamma = 0.15$
IMP	$\nu = 0.4$		tr: $r_1 = 61$ ft		$r_2 = 55$ ft
	$\beta^* = 6.6\%$	$V_s^* = 460$ fps	L: $r_1 = 61$ ft		$r_2 = 69$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A12.3 with the fixed-base modal parameters in Table A12.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A12.4.

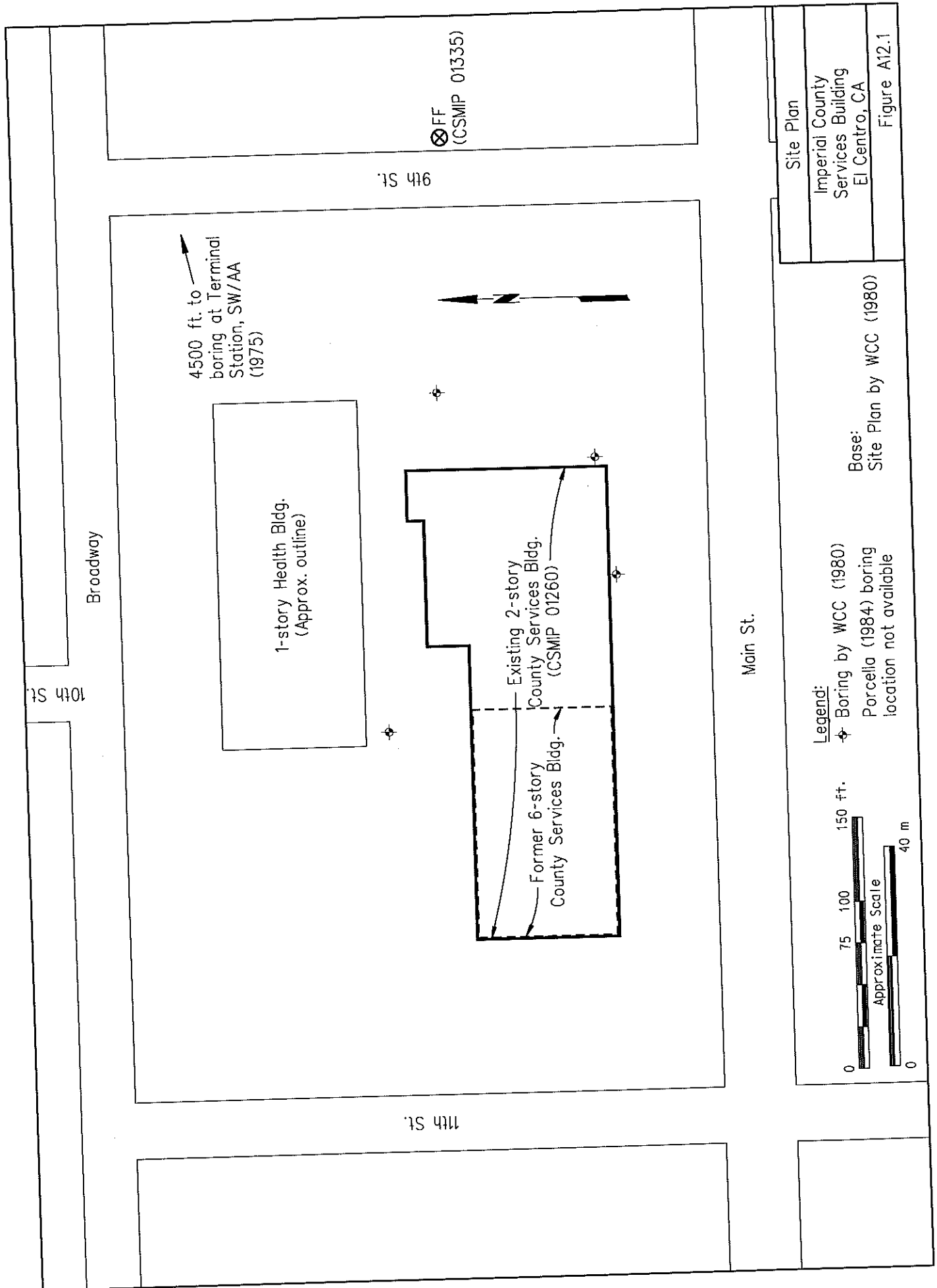
Table A12.4: Empirical and predicted inertial interaction effects

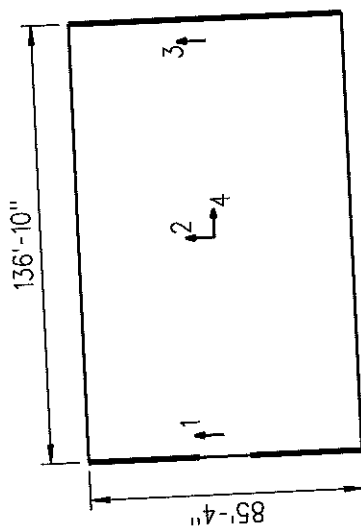
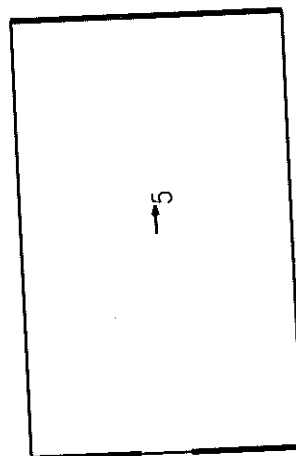
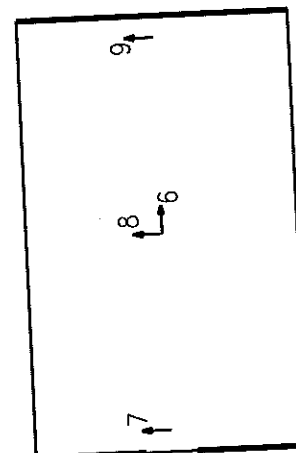
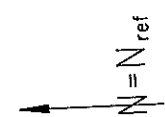
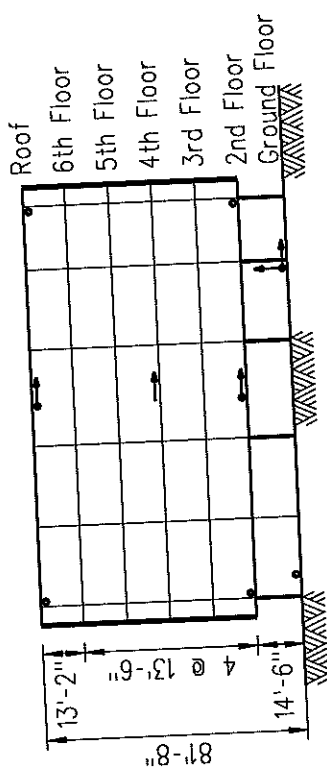
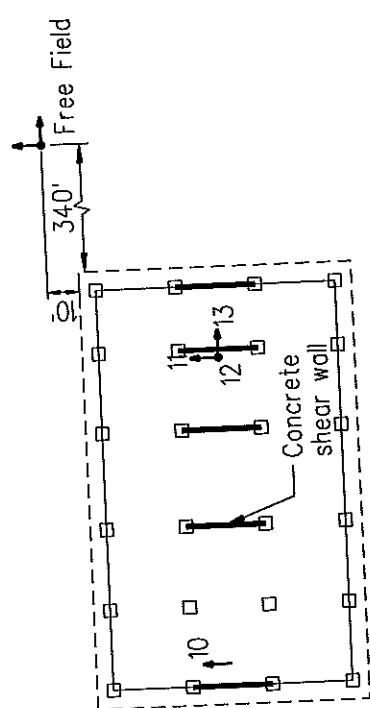
IMP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
		1.11	11.6	20.8	1.02	0.5	36.2
	Prediction	1.18	4.4	12.0	1.03	1.0	35.9

For the transverse direction, the period lengthening is overpredicted and the foundation damping underpredicted, while for the longitudinal direction the minor effects of SSI on the response of this building were satisfactorily predicted. This discrepancies for the transverse direction may be associated with the influence of the pile foundations, which tend to increase the rocking stiffness and the radiation damping relative to the surface foundation case assumed in the predictive analyses.

Data Sources

- Hadjian, A.H., Fallgren, R.B., and Lau, L. (1990). "Imperial County Services Building revisited: A re-evaluation with pile-soil-structure interaction," *Proc. 4th U.S. Nat. Conf. Earthquake Engrg.*, Vol. 3, EERI, 835-844.
- Hall, J.F. (1984). "Forced vibration and earthquake behavior of an actual pile foundation," *Soil Dyn. and Earthquake Engrg.*, 3(2).
- Porcella, R.J. (1984). "Geotechnical investigations at strong motion stations in the Imperial Valley, California," *Open-File Rpt. 84-562*, U.S. Geological Survey.
- Rojahn, C. and Mork, P.N. (1981). "An analysis of strong-motion data from a severely damaged structure, the Imperial County Services Building, El Centro, California," *Open File Rpt. 81-194*, U.S. Geological Survey.
- Shannon and Wilson/Agbabian Associates, SW/AA (1975). "Geotechnical and strong motion earthquake data from U.S. accelerograph stations, Ferndale, Cholame, and El Centro, CA," *Rpt. No. NUREG-0029*, Vol. 1.
- Woodward Clyde Consultants, WCC (1981). "Soil Investigation for the Imperial County Services Building, Eleventh and Main Streets, El Centro, CA," *Proj. No. 50268G-SI01*.

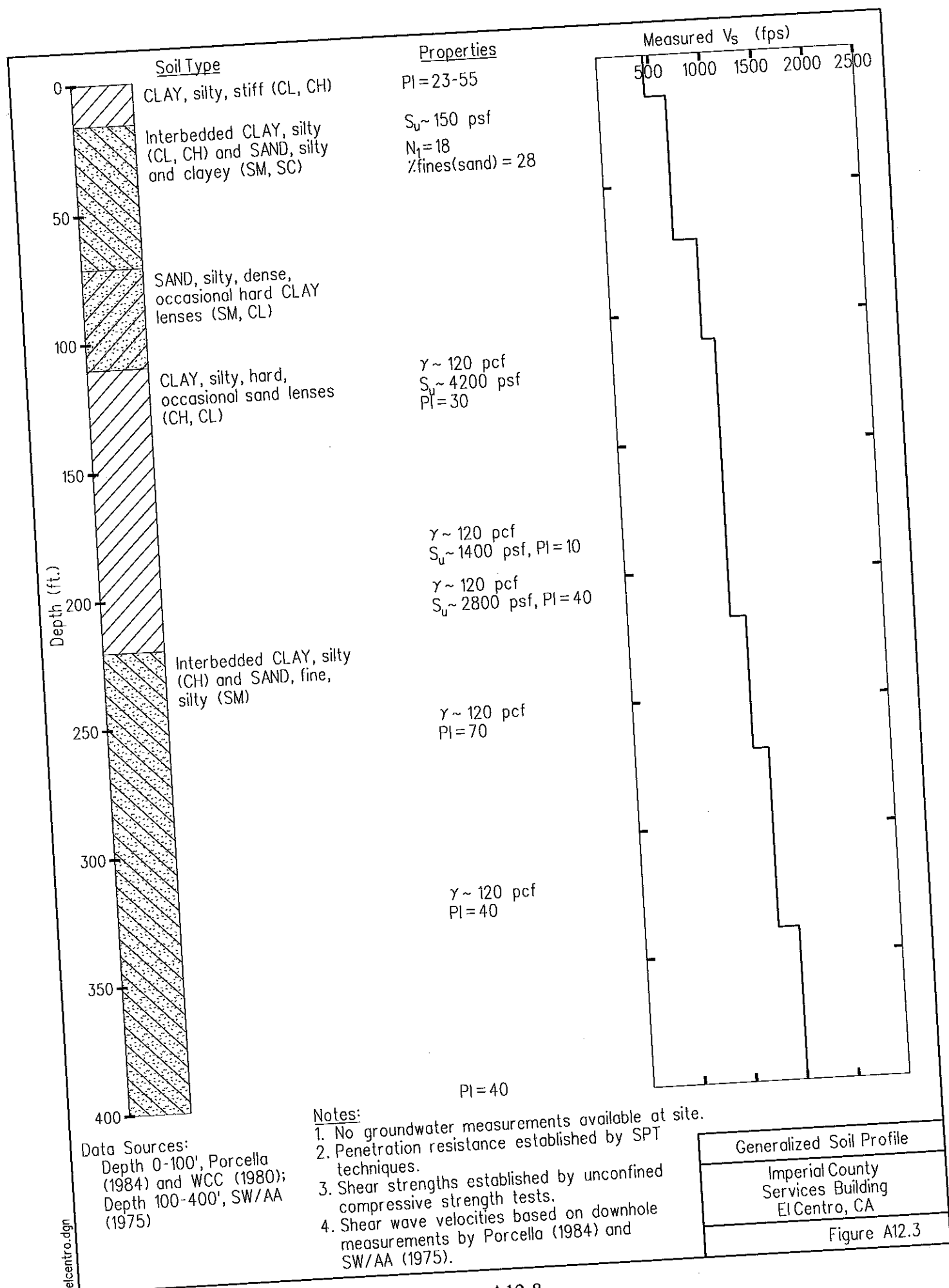




Sensor Locations
Imperial County Services Building El Centro, CA
Figure A12.2

Note: Data provided by CSMIP

elcentro.dgn[60]



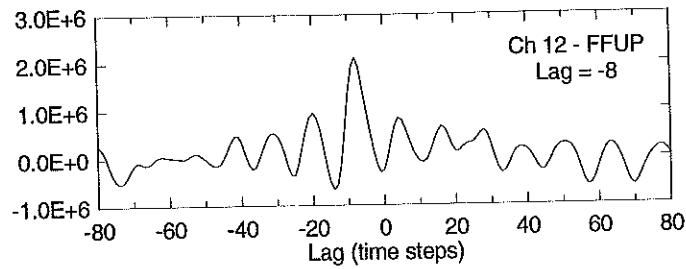


Fig. A12.IMP0: Cross-correlation function between vertical ground floor and free field accelerations, 1979 Imperial Valley Earthquake

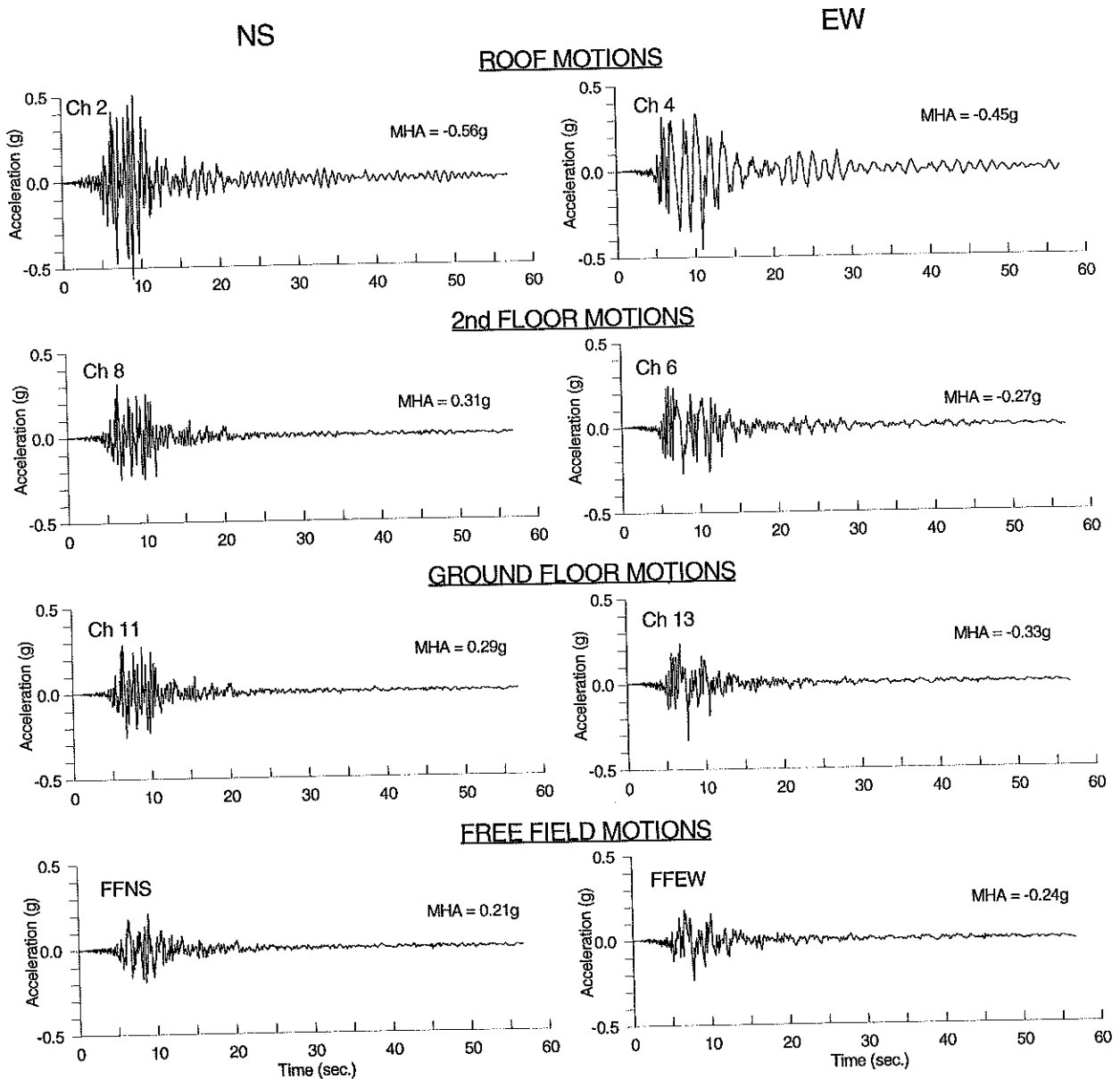


Fig. A12.IMP1: EL CENTRO, IMPERIAL COUNTY SERVICE BUILDING
Horizontal Acceleration Time Histories
1979 Imperial Valley Earthquake

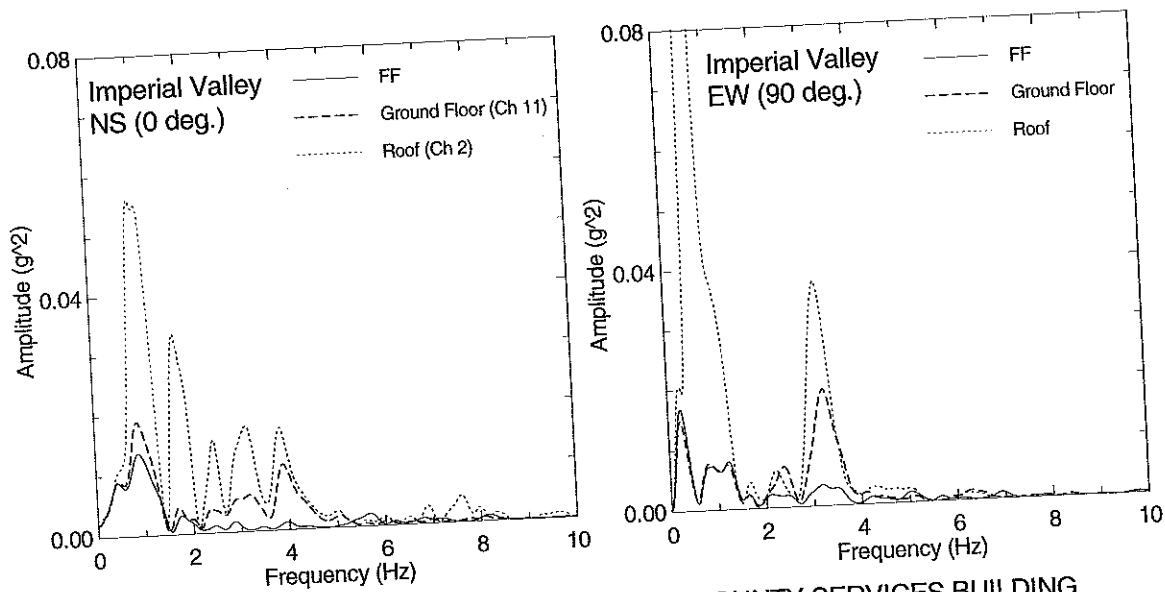


Fig. A12.IMP2: EL CENTRO IMPERIAL COUNTY SERVICES BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1979 Imperial Valley Earthquake

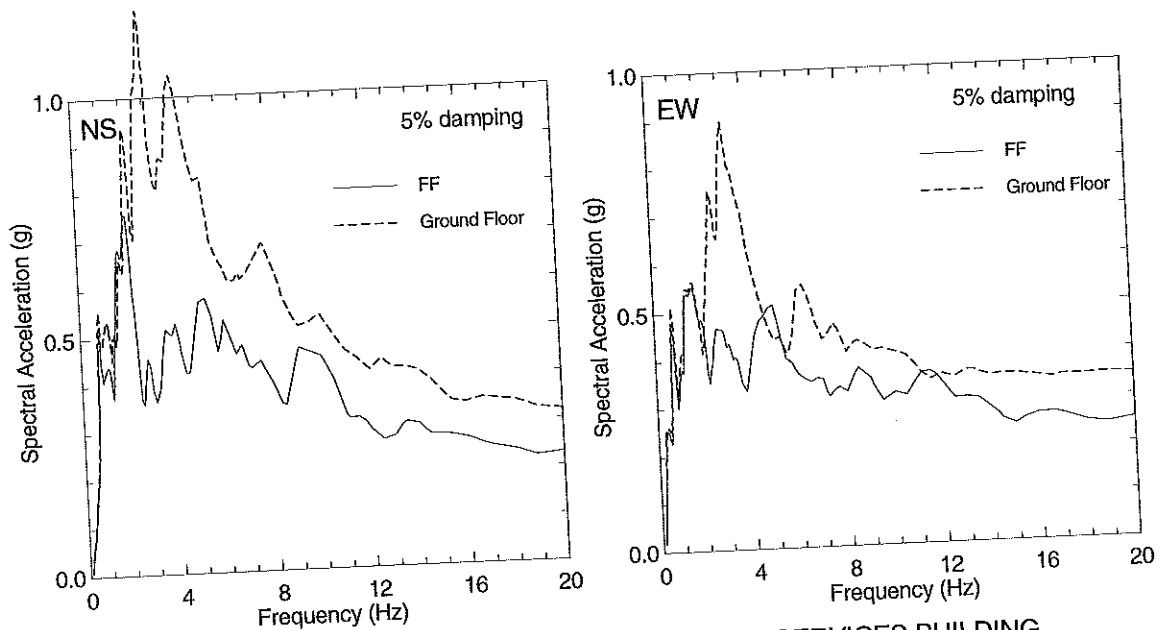


Fig. A12.IMP3: EL CENTRO IMPERIAL COUNTY SERVICES BUILDING
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1979 Imperial Valley Earthquake

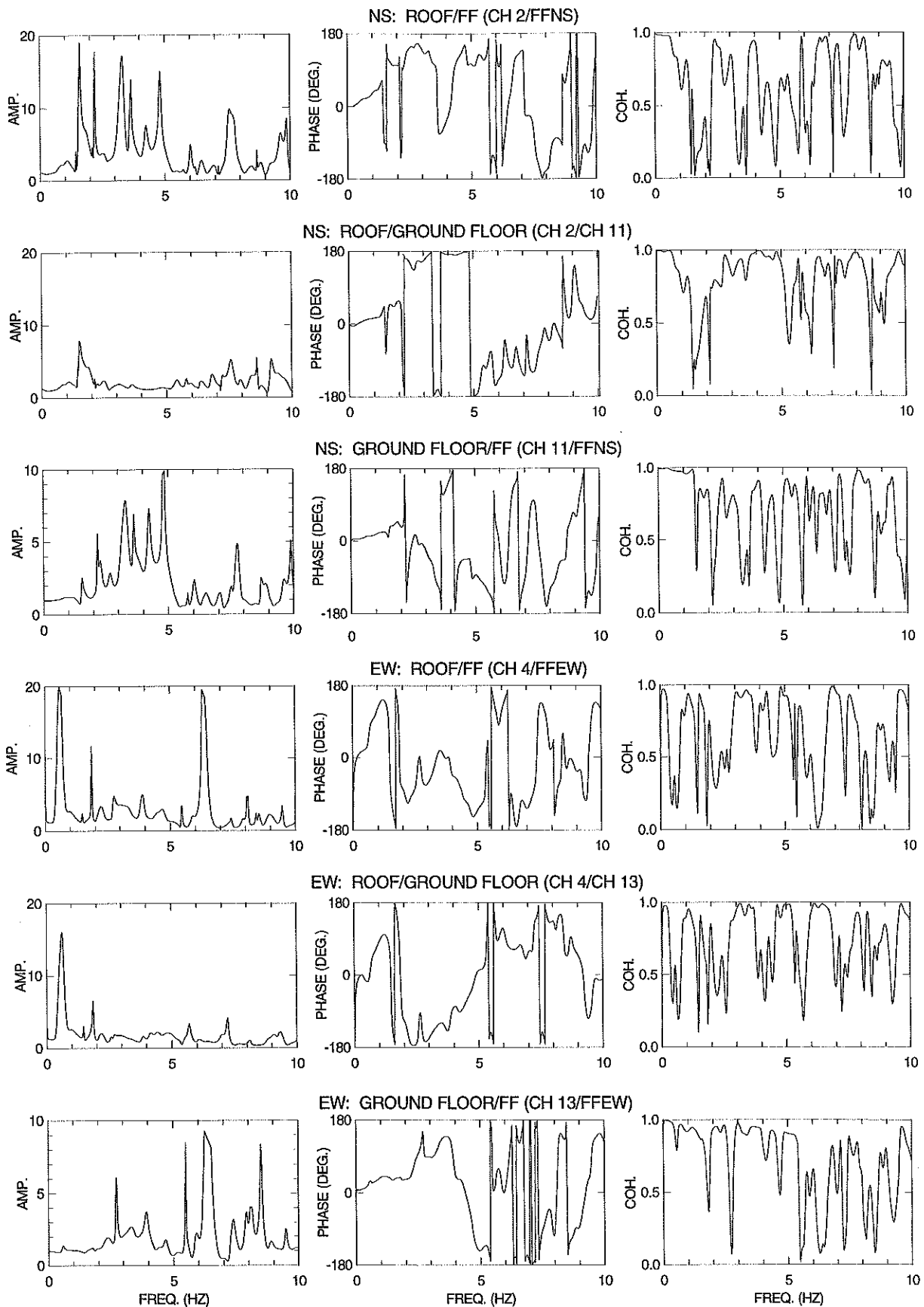


Fig. A12.IMP4: EL CENTRO IMPERIAL COUNTY SERVICES BUILDING
Transmissibility and Coherency Functions
1979 Imperial Valley Earthquake
A12-11

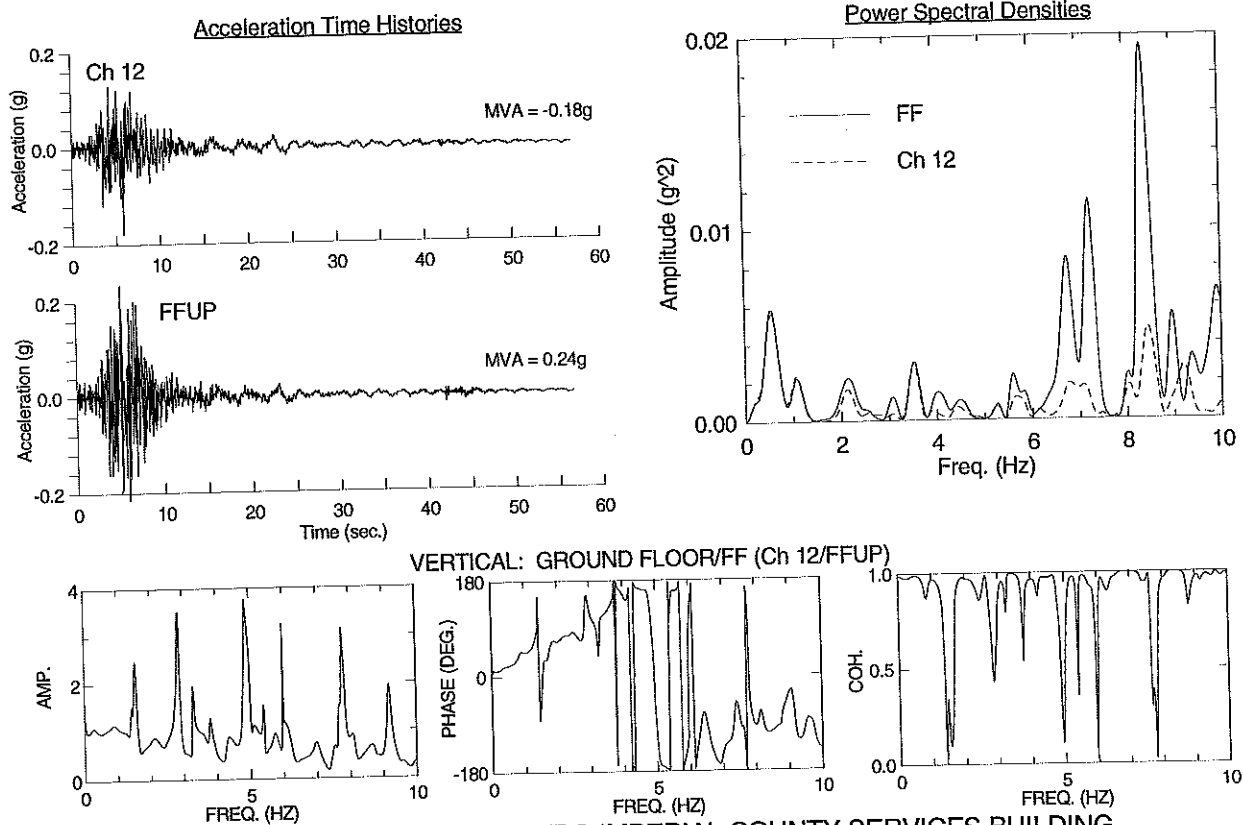


Fig. A12.IMP5: EL CENTRO IMPERIAL COUNTY SERVICES BUILDING
Analysis of Vertical Motions
1979 Imperial Valley Earthquake

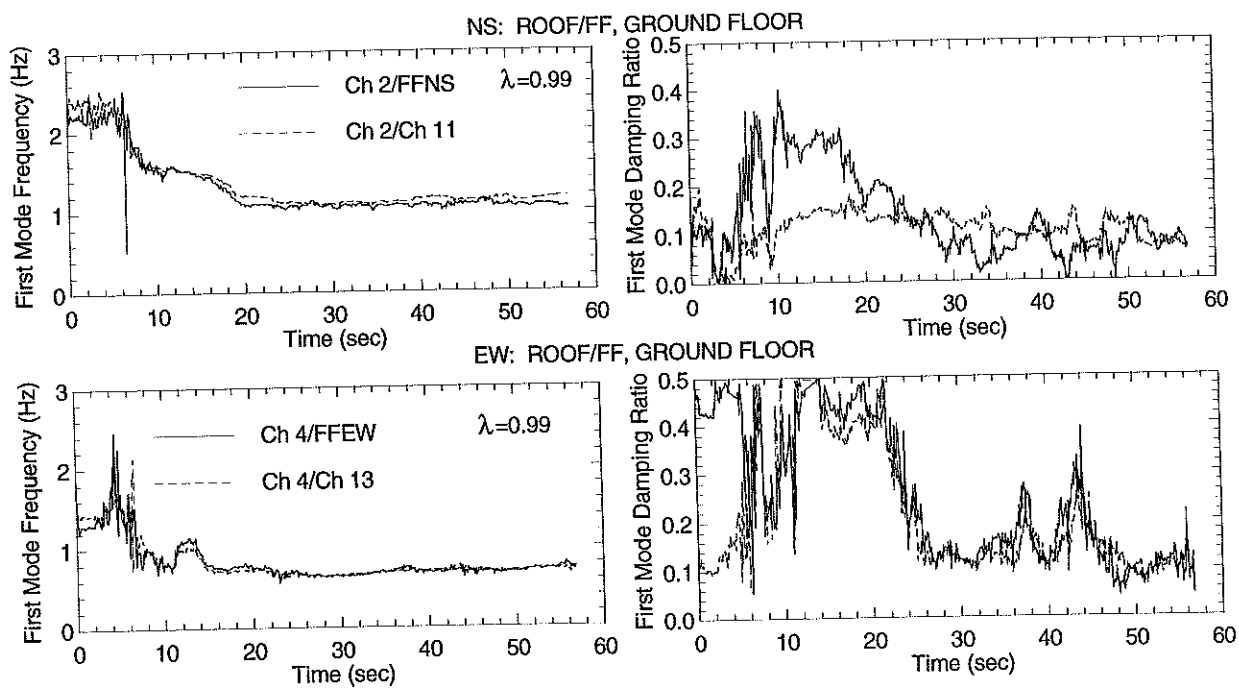


Fig. A12.IMP6: EL CENTRO IMPERIAL COUNTY SERVICES BUILDING
Time Variation of First Mode Parameters
1979 Imperial Valley Earthquake

SITE A13

Site Name: Indio 3 Story County Services Bldg.
Sensors: CSMIP 12493/12543

Lat: 33.714°
Long.: 116.221°

Floor Levels: 5
Bsmt. Levels: 1
Structure: Concrete moment resisting frames
and shear walls

Foundation: piles
Site Geology: soil
Design Date: 1966

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers	0.083	0.086	0.054	0.082	0.076	0.033	0.18	0.23	--

-- = no instrument

Preprocessing

Landers Earthquake (LD)

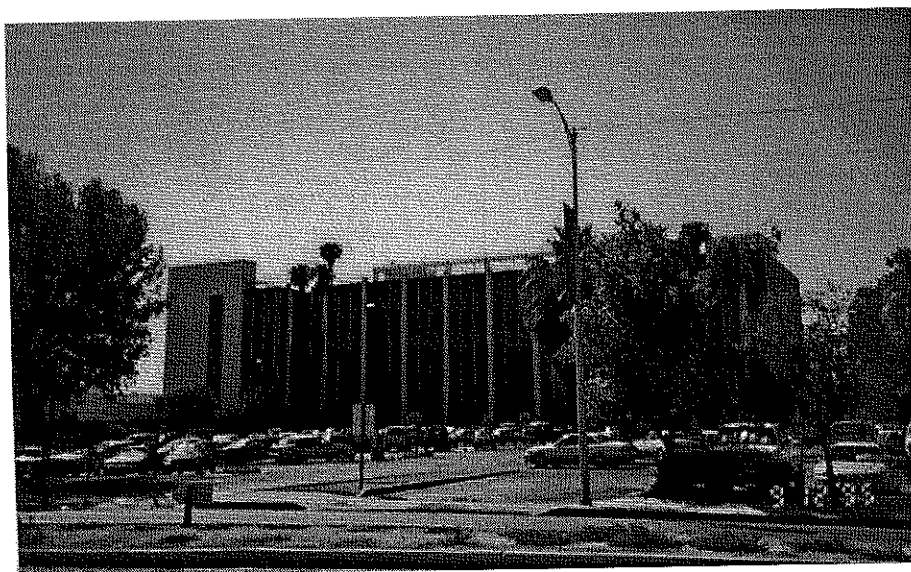
Preliminary: *Str. Data*: digitized and processed in this study, $f_{HP} = 0.1-0.23$ Hz, $f_{LP} = 20-23$ Hz, $f_{Nyq} = 100$ Hz. *FF Data*: CSMIP; filtered, $f = 0.14-46$ Hz; $f_{Nyq} = 50$ Hz; FF/str. data not synchronized or aligned.

Final: FF rotated 5° CCW to align with str.; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A13.LD0); FF data decimated by 4, str. data by 2 to $f_{Nyq} = 25$ Hz.

Figures

A13.1-A13.3
A13.LD0-6

Site Plan, Sensor Locations, Generalized Soil Column
LD Analysis Results



View of structure from Avenue 46, FF near driveway in foreground
(hut was temporarily removed at the time this photograph was taken)

DISCUSSION

Site Description

The building is 4 stories tall above ground level with a single level basement and is rectangular in plan view with dimensions of 84 by 180 feet. The building is connected to a 4-story addition on its east side by a 3-level bridge (between the 2nd story and roof). Lateral loads are resisted by moment resisting concrete frames in both directions and shear walls which extend from the basement to the roof and the 1st floor to the roof. Most of the columns are founded on 2x2 pile groups. The piles are tapered with a 16-inch head diameter, and are typically 50 feet long. The pile caps are 3.5 feet thick, and a 6-inch thick concrete slab-on-grade is present across the basement. Accelerographs are present on the basement, 2nd, and roof levels of the structure. The free-field instrument is located about 240 feet north of the structure in a parking lot.

The site is located on level terrain in an alluvial basin south of the Indio Hills. Several borings were drilled at the site as part the foundation investigation for the original structure (LCA, 1965). Based on these data, the generalized soil profile consists of: 13 feet of sand and silt, 7 feet of clayey, sandy silt, 30 feet of silt and clay with occasional sand layers, and silty sand which extends to the maximum depth explored of 74 feet. The soils encountered in the borings are likely Holocene alluvial and wind blown sediments, and it appears that older Pleistocene soils were not encountered. Groundwater was encountered by LCA at a depth of 23 feet. Soil conditions at greater depth and the depth to bedrock are poorly characterized in the area. The shear wave velocities in Fig. A13.3 were estimated using correlations with void ratio.

Instrument Selection and Free-Field Conditions

Structural instrumentation near the building centroid was used in the analyses (Table A13.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A13.LD1.

Table A13.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 5	Ch 2	Ch 1	Ch 7	Ch 4

There are no large structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A13.LD4) and the spectra for horizontal motions (Fig. A13.LD2-3) indicate minor de-amplification of basement motions relative to the free field for frequencies greater than about 1.5 Hz. No amplification was apparent at the first-mode lateral response frequencies.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Fig. A13.LD5. The transmissibility function amplitude has a peak near the first-mode lateral response frequencies ($\tilde{f}_1 \approx 1.4$ to 1.5 Hz), indicating the possible presence of rocking in the structure. However, given the location of the vertical instrument near the building centroid, no firm conclusions about the influence of rocking in the structural response can be drawn from these data.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A13.LD4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A13.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A13.LD6).

Table A13.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=3, j=5		L (EW) d=3, j=5		tr (NS) d=3, j=5		L (EW) d=2, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	1.44	6.8	1.55	9.4	1.43	7.0	1.54	9.4	1.43	10.1	1.52	10.5
	2					6.83	21.0	6.78	23.2	5.75	15.4	5.53	28.0

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The small variations between the fixed- and pseudo flexible-base parameters for both directions suggest that rocking did not significantly influence the structural response. This finding is not necessarily inconsistent with the apparent rocking detected in the vertical B/FF transmissibility function, as the power spectral densities for vertical motions show that little energy is present in the vertical basement motions near $\tilde{f}_1$.
- Roof accelerations at the west end of the structure have significantly larger amplitudes than accelerations at the centroid (see Fig. A13.LD1), suggesting that the structure may be stiffer at its east end where it is attached to the adjacent 4-story addition.

The RPEM results (Fig. A13.LD6) indicate no significant time variations in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and B-R first mode frequencies and damping ratios.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A13.3. The strain-dependent soil properties were estimated from deconvolution analyses to a

depth of 69 feet using the recorded free-field motions and the profile in Fig. A13.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by 50% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A13.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_S = 760$ fps	$h = 56$ ft	emb. = 15.5 ft	$\gamma = 0.15$
LD	$\nu = 0.45$		tr: $r_1 = 69$ ft		$r_2 = 58$ ft
	$\beta^* = 4.1\%$	$V_S^* = 695$ fps	L: $r_1 = 69$ ft		$r_2 = 85$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A13.3 with the fixed-base modal parameters in Table A13.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A13.4.

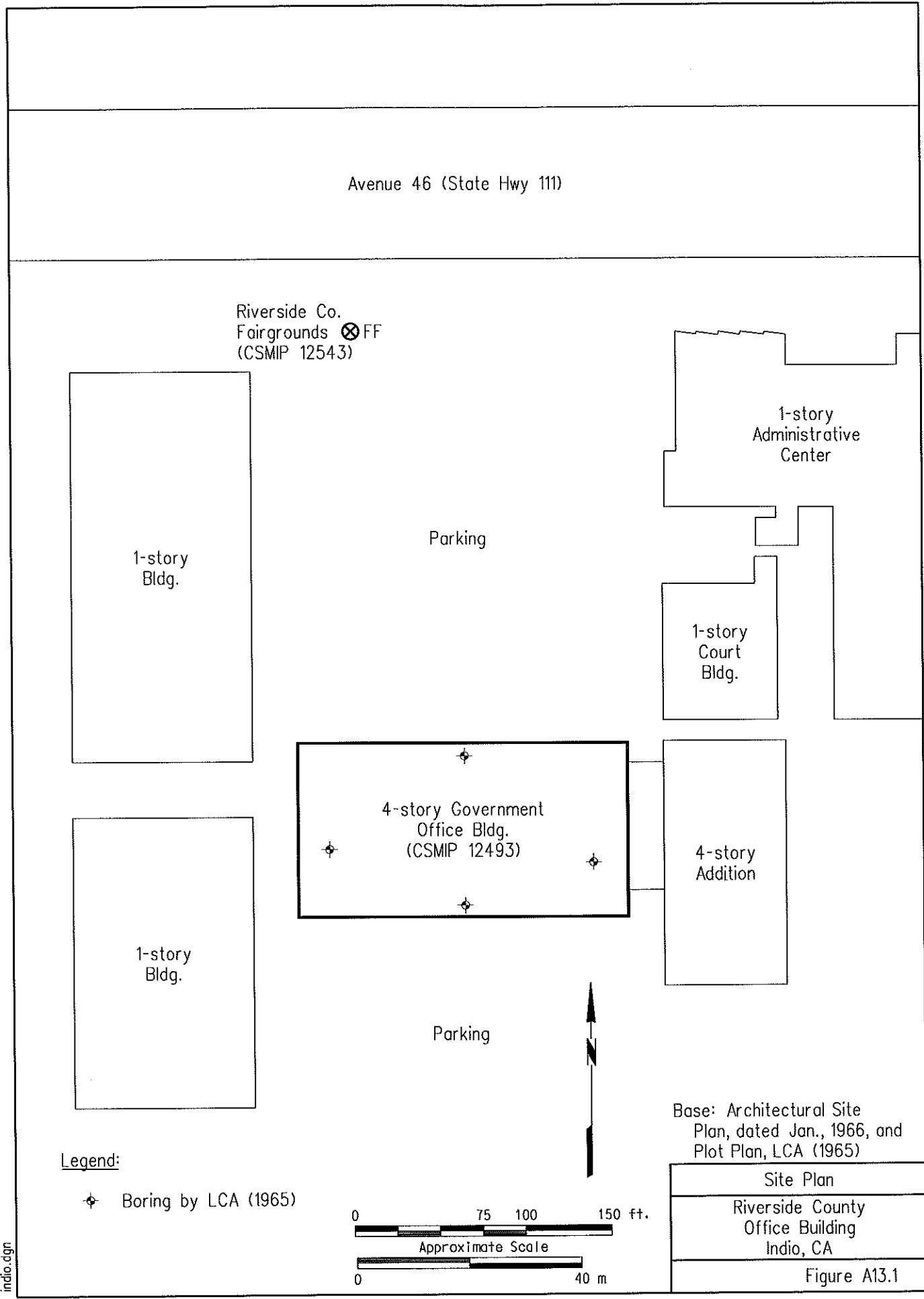
Table A13.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LD	Sys. ID	1.01	3.6	10.1	1.02	1.6	10.5
	Pre.-Vel.	1.05	1.1	6.9	1.04	1.6	9.9
	Pre.-Bie.	1.05	2.5	8.4	1.06	3.3	11.2

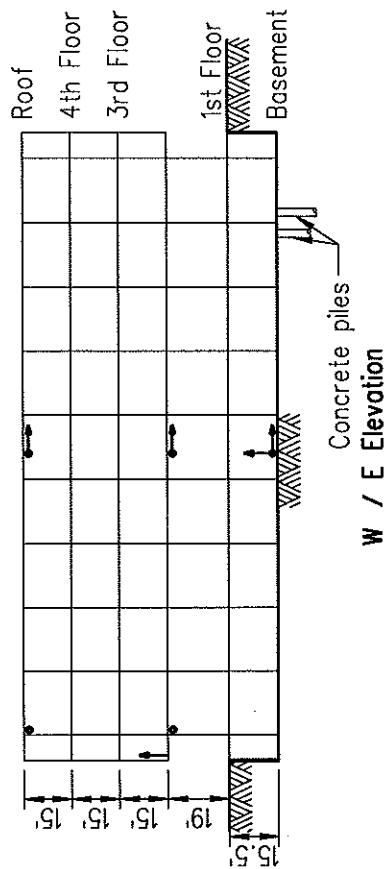
The period lengthening is overpredicted by both formulations. Foundation damping for the transverse and longitudinal directions was more accurately predicted by the "modified Bielak" and "modified Veletsos" formulations, respectively. The overprediction of period lengthening may be associated with the influence of the pile foundations, which tend to increase the rocking stiffness relative to the surface foundation case assumed in the predictive analyses.

Data Sources

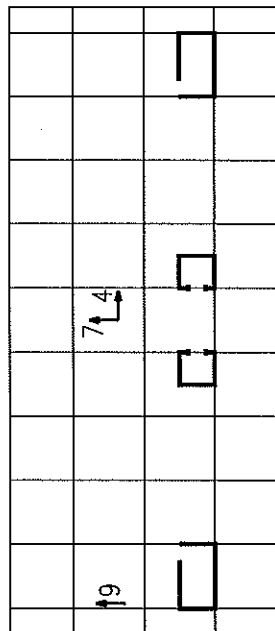
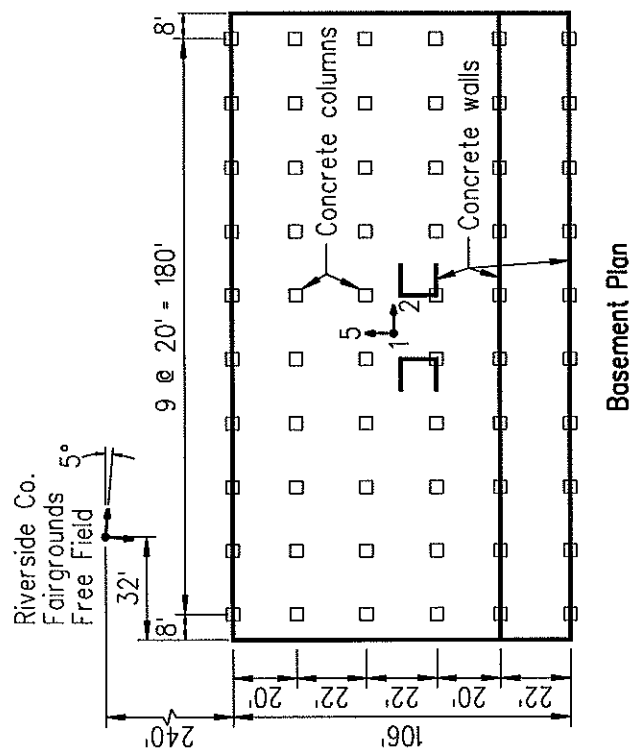
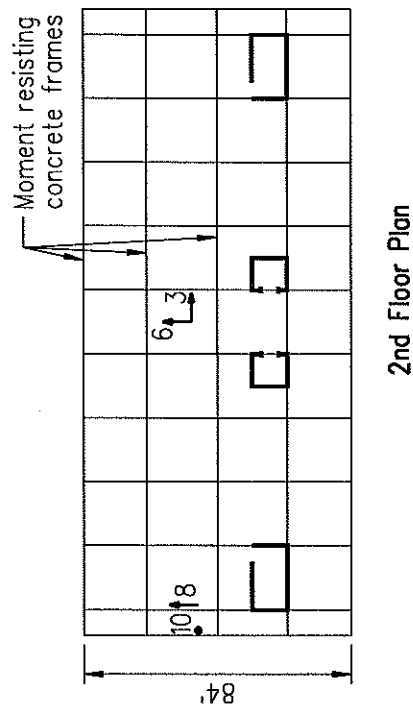
Leroy Crandall Assoc., LCA (1965). "Report of Foundation Investigation, Proposed Building, Highway 111 and Oasis Avenue, Indio, CA," *Proj. No. A65086*.



indio.dgn



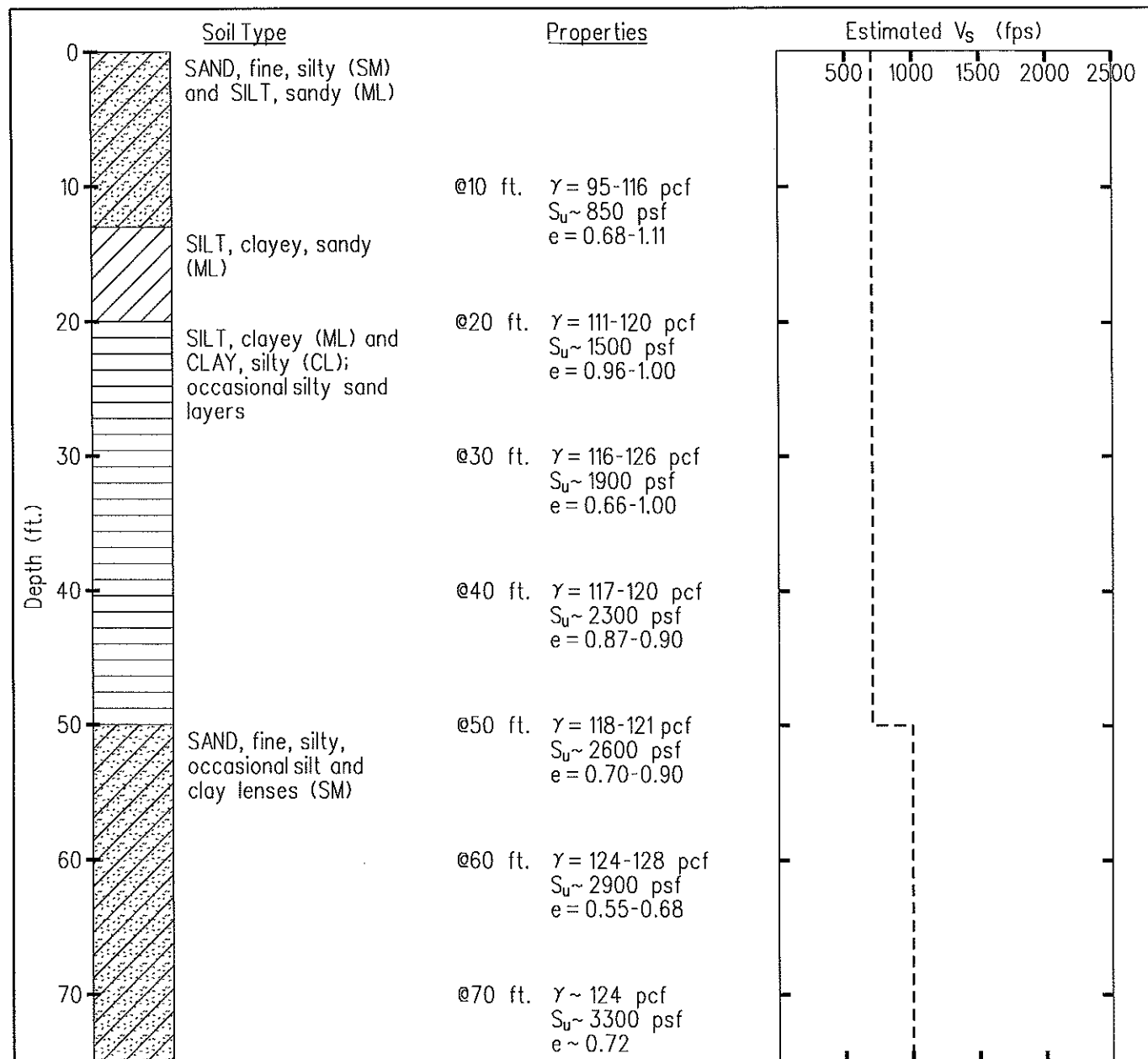
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Sensor Locations
Riverside County Office Building Indio, CA
Figure A13.2

Note: Data provided by CSMIP

indio.dgmt607



Data Source:
LCA (1965)

Notes:

1. Groundwater encountered at 23 ft. depth.
2. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985).

Generalized Soil Profile

Riverside County
Office Building
Indio, CA

Figure A13.3

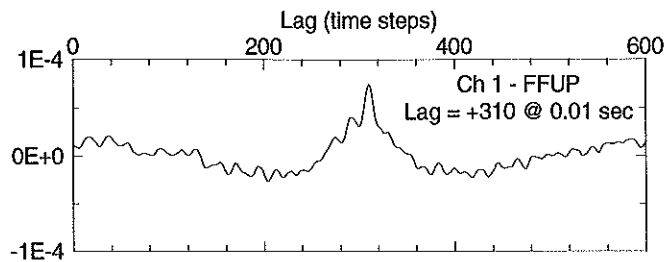


Fig. A13.LD0: Cross-correlation function between vertical basement and free field accelerations

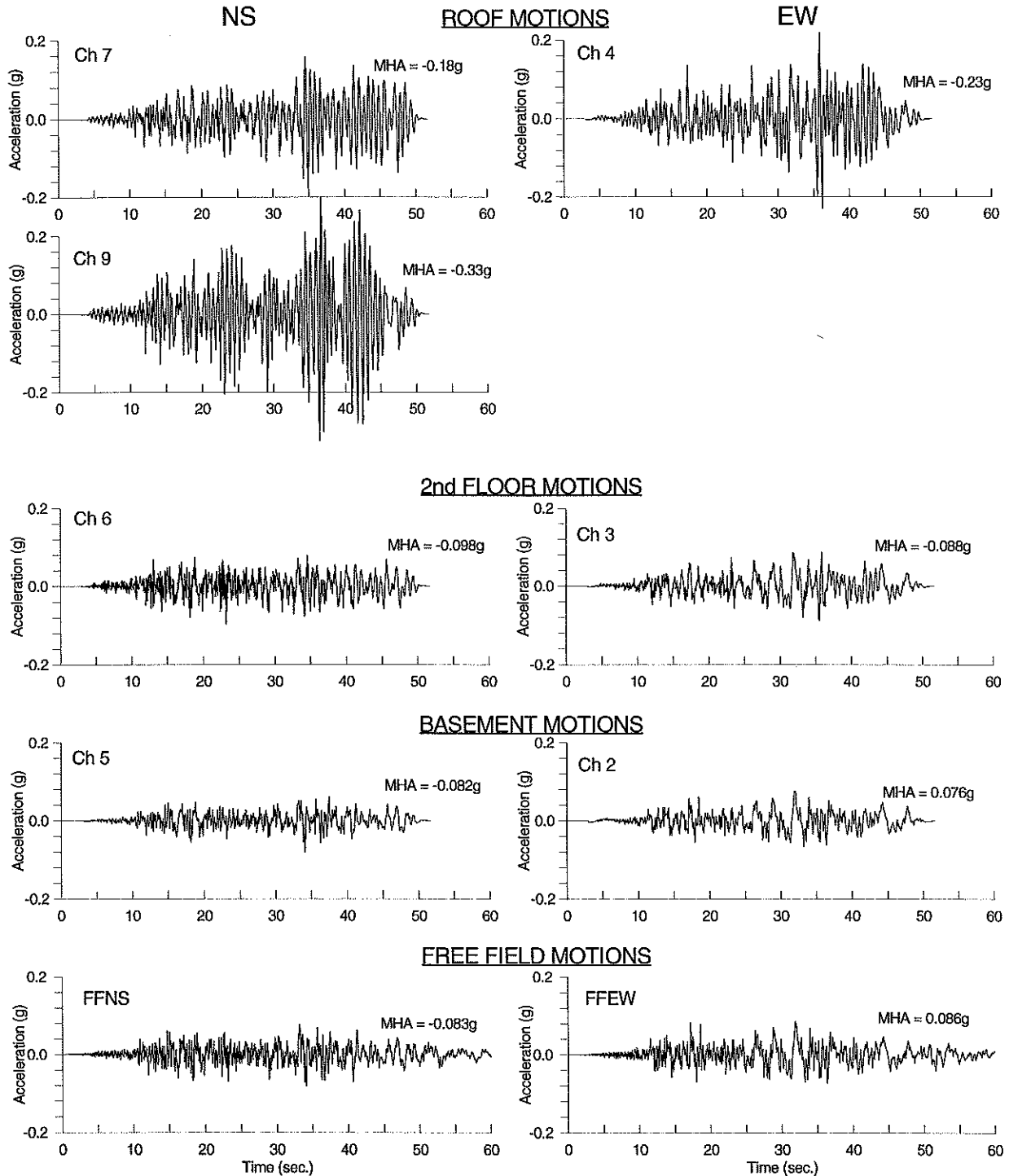


Fig. A13.LD1: INDIO 4 STORY COUNTY SERVICES BUILDING
Horizontal Acceleration Time Histories
1992 Landers Earthquake

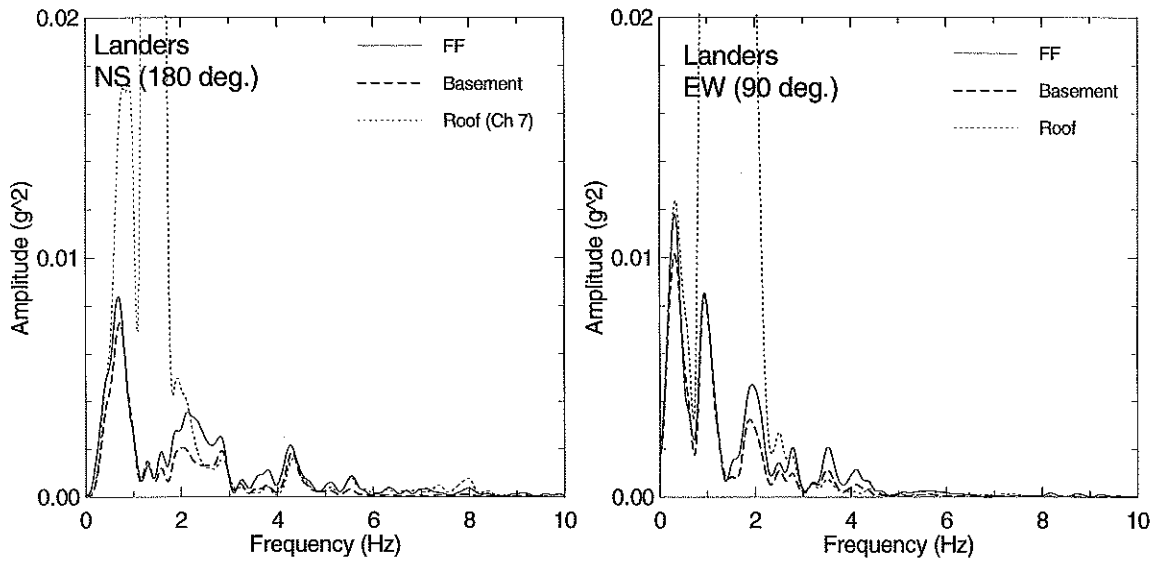


Fig. A13.LD2: INDIO 4 STORY COUNTY SERVICES BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

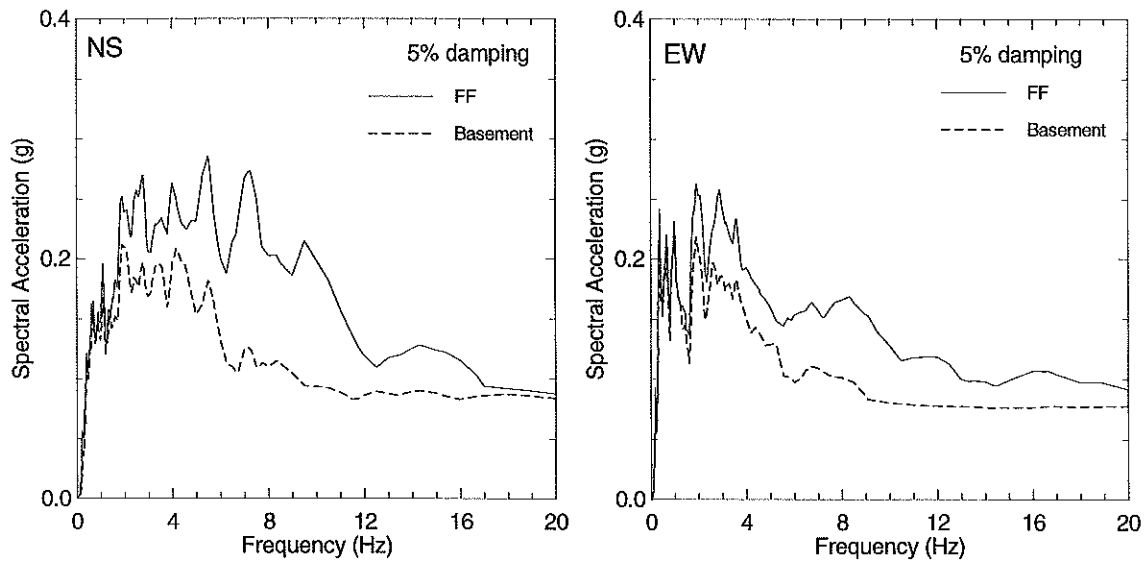


Fig. A13.LD3: INDIO 4 STORY COUNTY SERVICES BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1992 Landers Earthquake

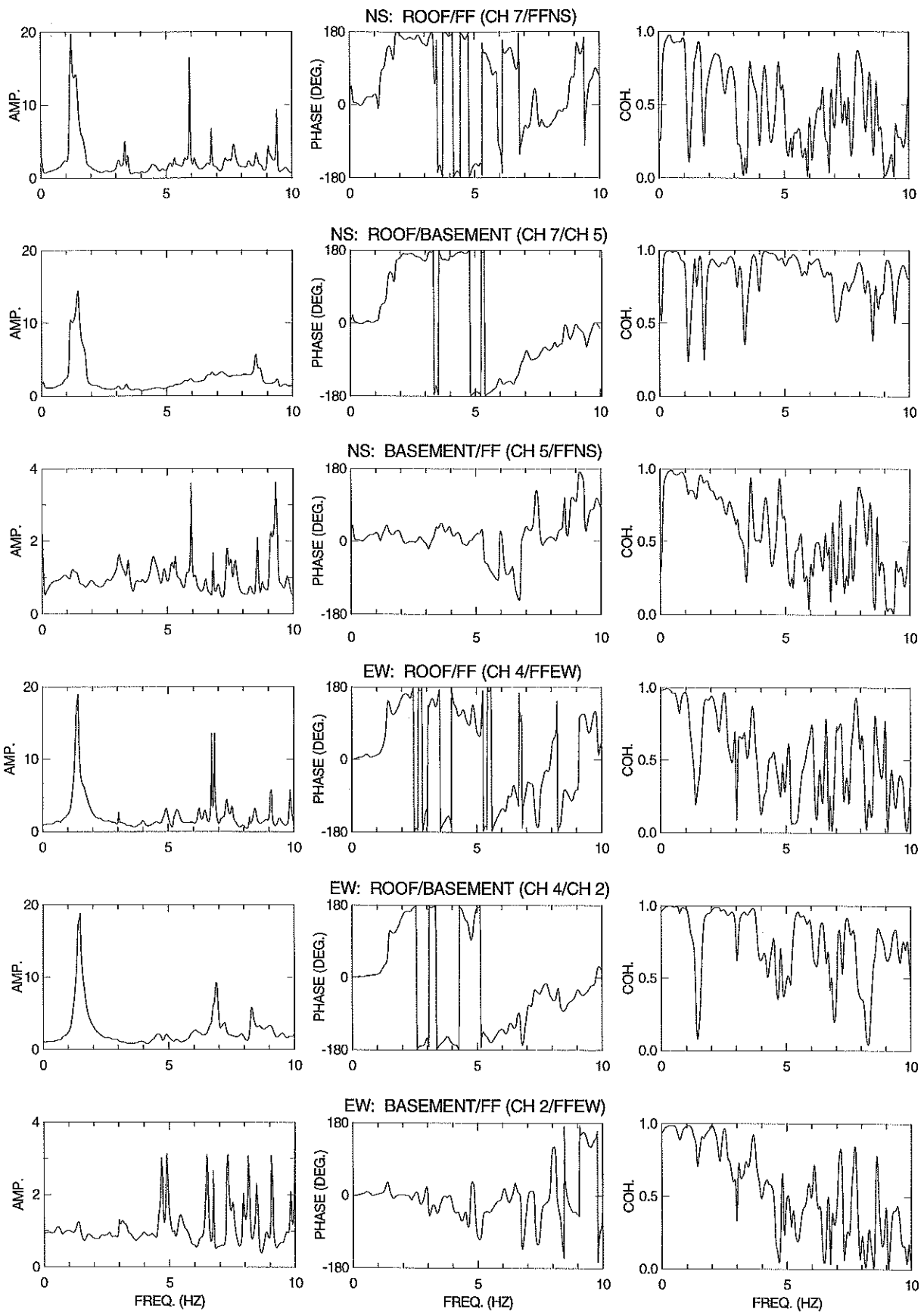


Fig. A13.LD4: INDIO 4 STORY COUNTY SERVICES BUILDING
Transmissibility and Coherency Functions
1992 Landers Earthquake
A13-10

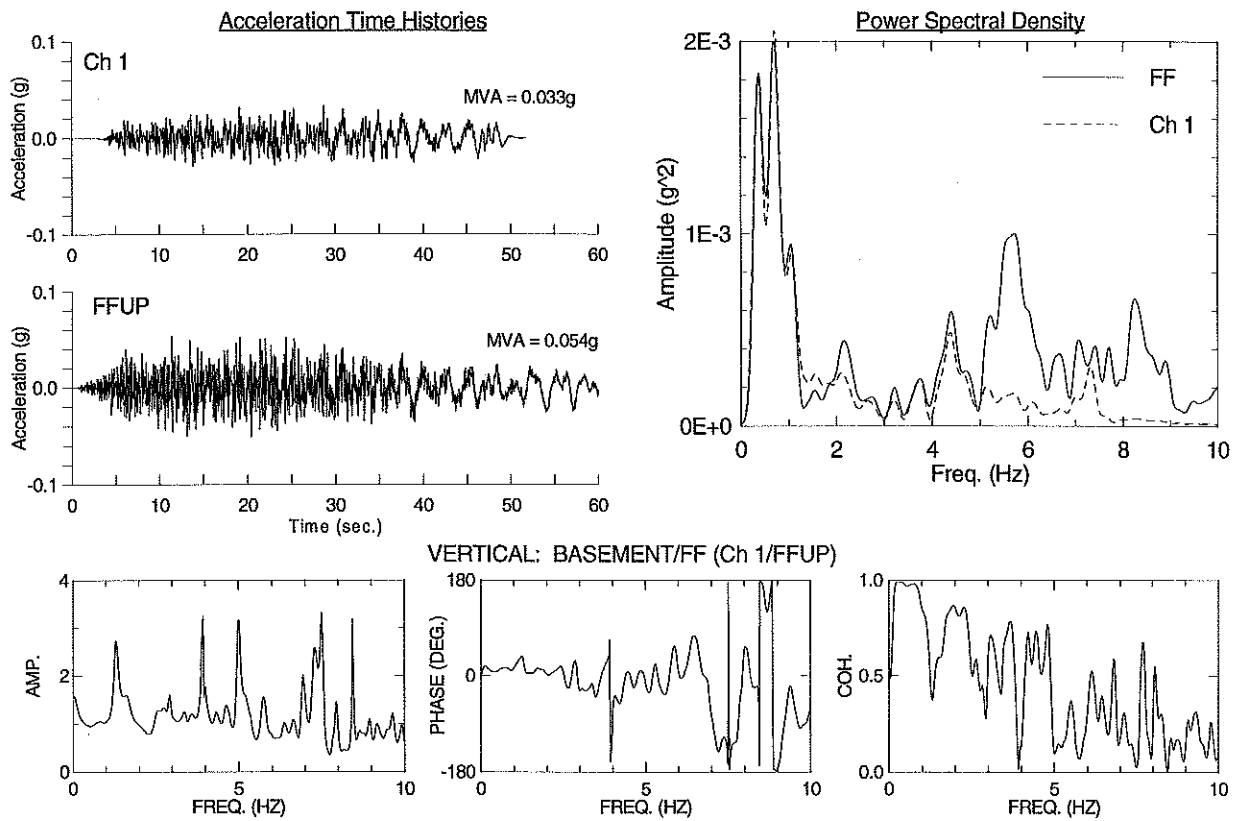


Fig. A13.LD5: INDIO 4 STORY COUNTY SERVICES BUILDING
Analysis of Vertical Motions
1992 Landers Earthquake

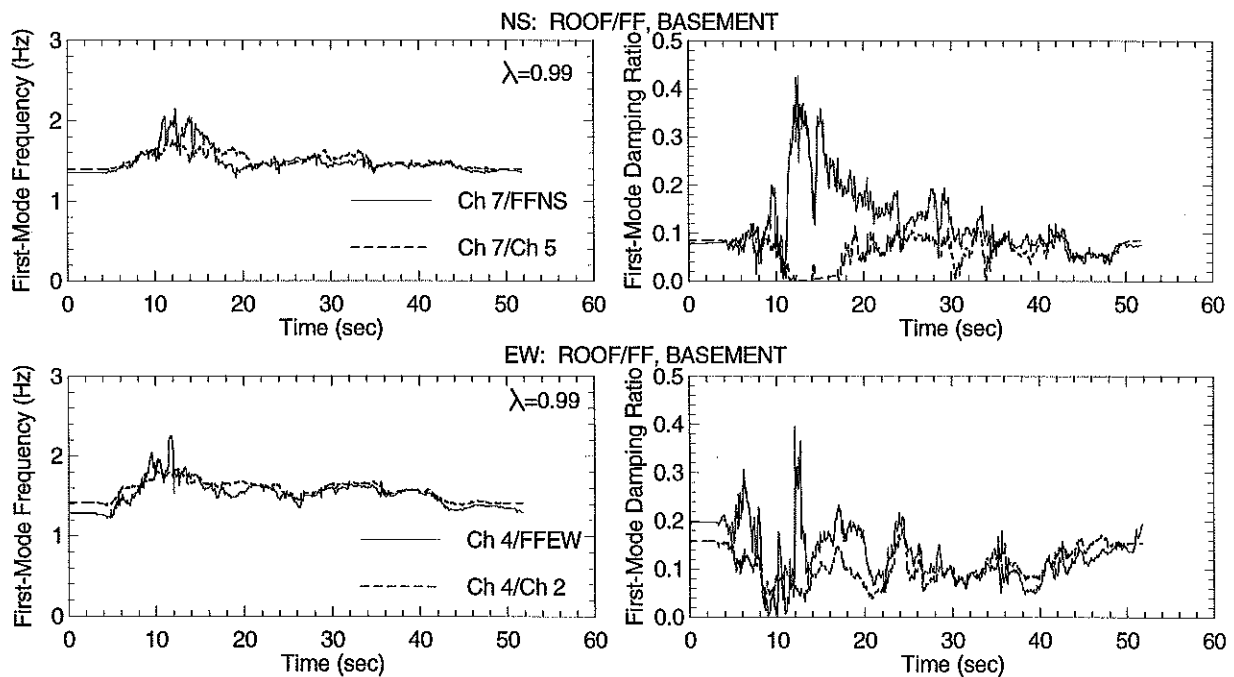


Fig. A13.LD6: INDIO 4 STORY COUNTY SERVICES BUILDING
Time Variation of First-Mode Parameters
1992 Landers Earthquake

SITE A14

Site Name: Lancaster 3 Story Building
Sensors: CSMIP 24517/24526

Lat: 34.688°
Long.: 118.157°

Floor Levels: 3
Bsmt. Levels: 0
Structure: plywood floor and roof diaphragms
concrete block shear walls

Foundation: piers
Site Geology: Alluvium
Design Date: 1975

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	0.056	0.067	0.026	0.045	0.057	0.018	0.065	0.18	--
1992 Landers*	0.05	0.08	0.03	0.04	0.07	0.04	0.07	0.25	--
1994 Northridge*	0.071	0.064	0.057	0.07	0.05	0.04	0.09	0.11	--

*Not digitized

-- = No instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.8\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data synchronized, FF not aligned with str.

Final: FF data rotated 345° CCW to align with str.

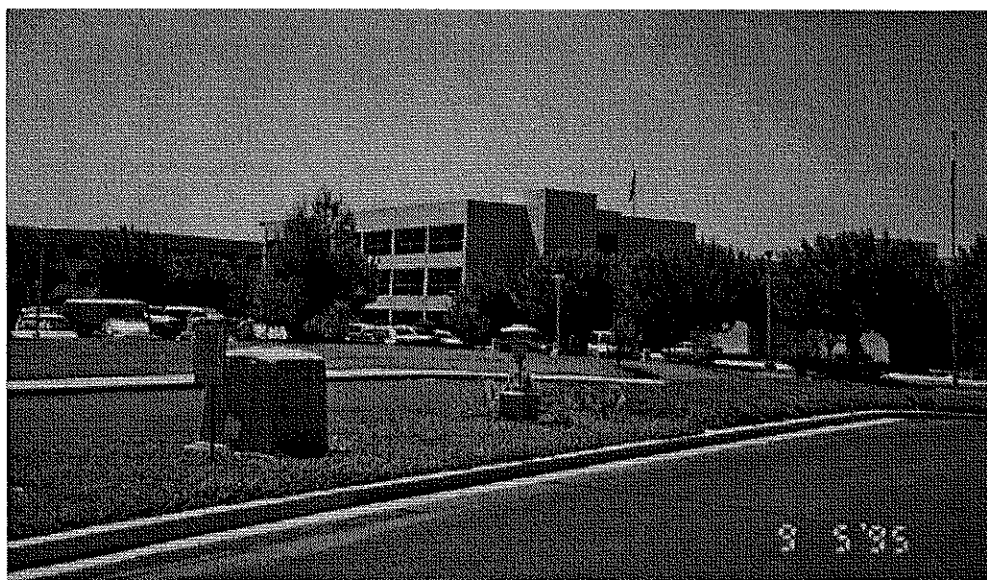
Figures

A14.1-A14.3

Site Plan, Sensor Locations, Generalized Soil Column

A14.WT1-5

WT Analysis Results



View of structure with FF in foreground

DISCUSSION

Site Description

The 3-level building is 37.5 feet tall with no basement, and is nearly rectangular in plan view with dimensions 70 by 109 feet. As shown in Fig. A14.2, the structural system consists of plywood floor and roof diaphragms and concrete block shear walls in both directions. The building is supported on a pier foundation, and a concrete slab-on-grade is present across the base of the building. Two one to two-story hospital buildings are located about 20 feet from the building. As shown in Fig. A14.2, accelerographs are present on all three floors of the structure. The free-field instrument is located in a parking lot about 250 feet northeast of the structure.

The site is located on level terrain within the Antelope Valley. Several borings have been drilled at the site as part of foundation investigations for various buildings on the hospital campus (e.g. Buena Engineers, 1984). Based on these data, surficial soil conditions at the site consist of interbedded silty sands and clays of moderate stiffness which extend to the maximum depth explored of 90 feet. Soil conditions at greater depth and the depth to bedrock are poorly characterized in the area. The shear wave velocities in Fig. A14.3 were estimated using correlations with void ratio.

Instrument Selection

Instrumentation was selected to assess the in-plane response of the concrete block walls in both directions, and the out-of-plane response in the transverse (EW) direction. The accelerographs used for these analyses are indicated in Table A14.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A14.WT1.

Table A14.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 13	Ch 9	Ch 1	Ch 10	Ch 2, 3

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A14.WT4) and the spectra of the horizontal motions (Fig. A14.WT2-3) indicate increasing de-amplification of GF motions with frequency in both directions.

Transmissibility, coherency, and power spectral density functions for FF and GF vertical motions are presented in Fig. A14.WT5. Although the transmissibility function amplitudes are fairly spiky, there is no evidence of amplification at the structure's fundamental frequency.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A14.WT4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A14.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A14.WT6).

Table A14.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW)* d=2, j=3 (d=0, j=3)		L (NS) d=0, j=3		tr (EW)* d=0, j=3 (d=--, j=--)		L (NS) d=--, j=--	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	5.03	9.3	na	na	4.98 (5.35)	15.9 10 (22)	5.27	19.2	4.90 (--)	11.2 15 (--)	na	na
	2					9.41 (11.1)	8.7 (27)	10.6	10.9	8.77 (--)	12.7 (--)	na	na

* Results for in-plane wall response (i.e. using Ch 2) are shown in parentheses (), other results are for out-of-plane wall response.

Comments on the analytical results follow:

- The shear walls have a large in-plane stiffness. This is illustrated in R/GF transmissibility functions (Ch2/Ch9, Ch10/Ch13) by amplitudes and coherencies close to unity and nearly zero phase shift from 0 to 5 Hz. Out-of-plane roof sensors were used for transverse parametric analyses, so the results reflect the flexibility of the plywood roof diaphragm.
- Parametric models for FF-R longitudinal response could not be developed.
- As discussed below, the CEM $\tilde{\zeta}$ and $\tilde{\zeta}^*$ values of 11.2% and 15.9% were adjusted based on RPEM results.
- The structural responses for both directions are dominated by the first mode. As shown in Fig. A14.WT4, the first-mode responses have broad transmissibility function amplitude peaks. For this reason, CEM results are especially sensitive to the j parameter.
- The small difference between the transverse fixed- and pseudo flexible-base frequencies are consistent with the lack of evidence for rocking in the vertical GF/FF transmissibility function.

The RPEM analytical results (Fig. A14.WT6) do not indicate any significant time-dependent variations in the first-mode parameters of the structure. The RPEM results support the CEM results with regard to the relative magnitudes of the FF-R and GF-R first-mode frequencies.

However, contrary to CEM results, RPEM damping ratios were about 15% and 10% for the FF-R and GF-R cases, respectively (as indicated in Table A14.2).

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A14.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 54 feet using the recorded free-field motions and the profile in Fig. A14.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A14.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 920$ fps	$h=26$ ft	emb.=0	$\gamma = 0.15$
WT	$\nu=0.4$		tr: $r_1 = 54$ ft		$r_2 = 47$ ft
	$\beta^*=2.1\%$	$V_s^*=910$ fps	L: $r_1 = 54$ ft		$r_2 = 63$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A14.3 with the fixed-base modal parameters in Table A14.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A14.4.

Table A14.4: Empirical and predicted inertial interaction effects

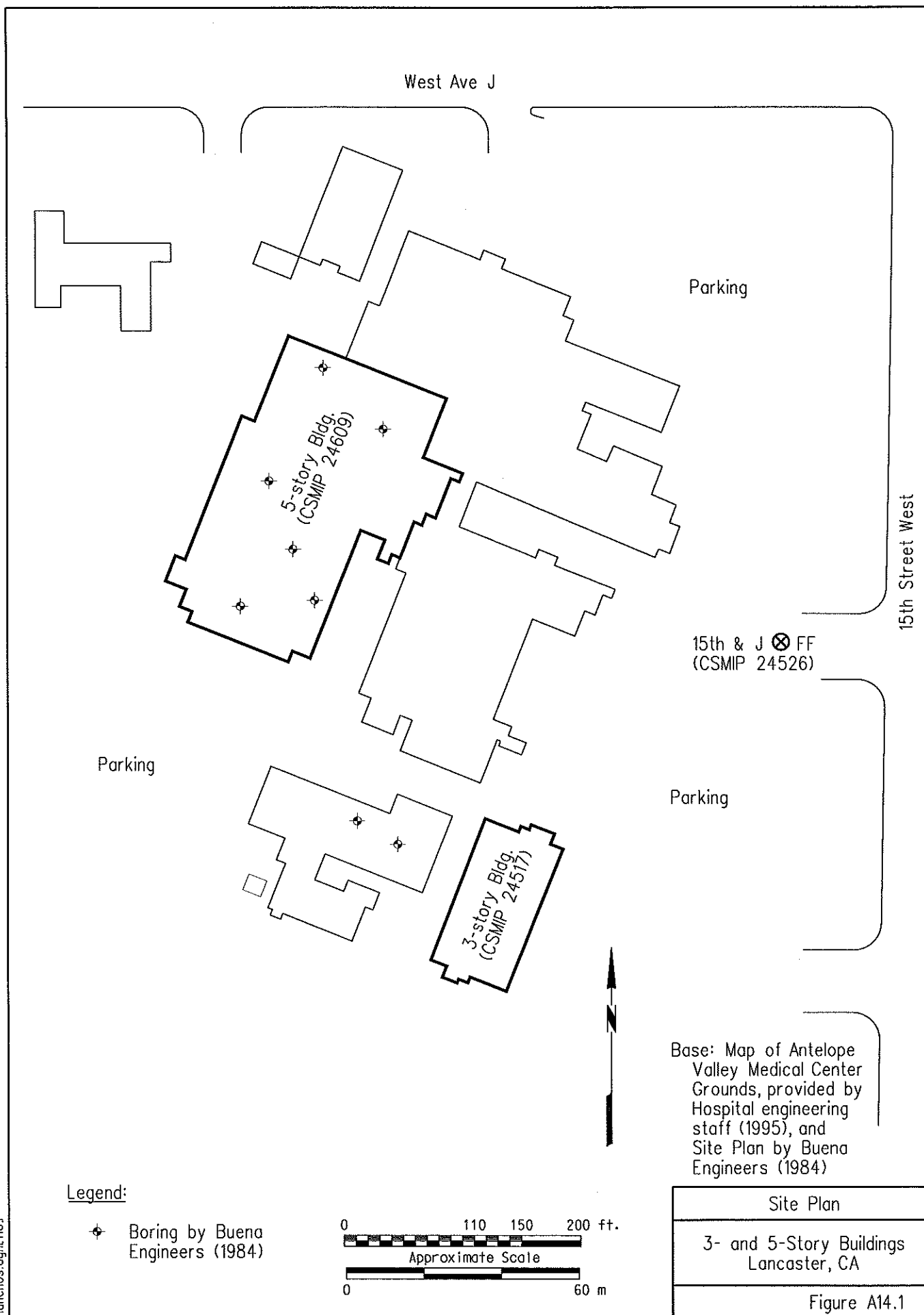
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.03	6.4	15.0	--	--	--
WT	Prediction	1.12	5.1	11.7	--	--	--

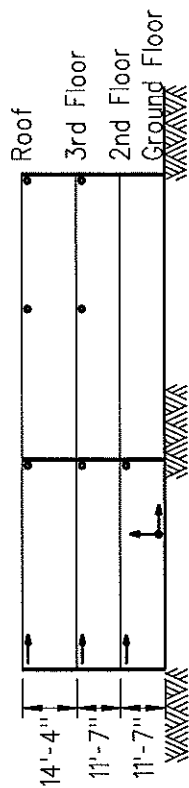
The period lengthening is overpredicted and the foundation damping is well predicted. The two most likely sources of error in the predictive analyses are the V_s of the soil and the estimated fixed-base structural parameters. There is uncertainty in both cases (i.e. the V_s profile was estimated from correlation relationships, and the fixed-base modal parameters were derived from parametric system identification results in which there was considerable sensitivity to the j parameter), and the source of error between the empirical and predicted results is unknown.

Data Sources

Buena Engineers (1984). "Geotechnical Engineering Report, Proposed Antelope Valley Hospital Medical Center Expansion, Lancaster, CA," *Rpt. No. B-2283-L01*.

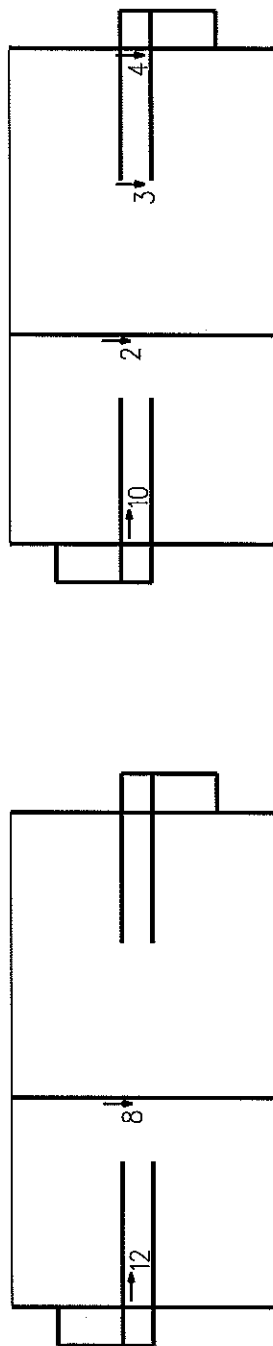
lanchos.dgn[110]





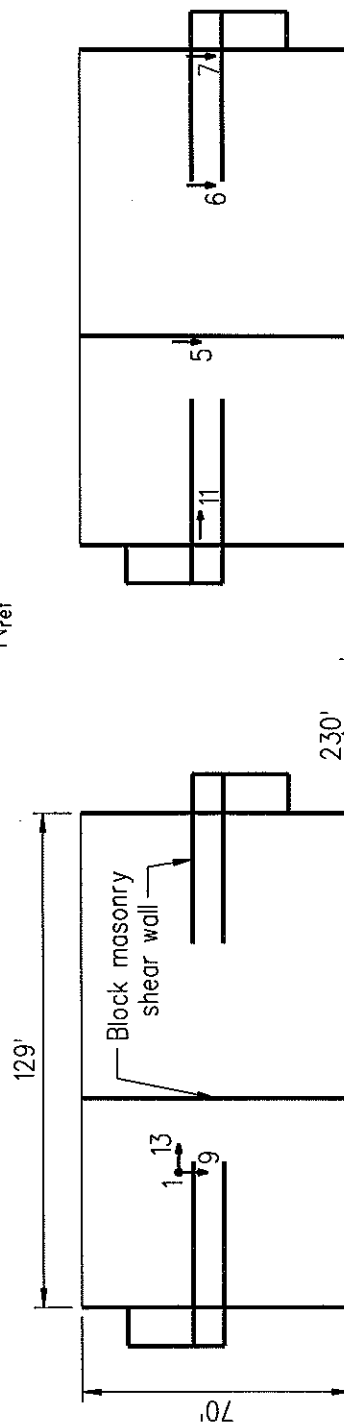
Block masonry shear walls with
plywood floor and roof diaphragms.
Design date: 1975

S / N Elevation



2nd Floor Plan

Roof Plan



Ground Level Plan

3rd Floor Plan

10° (1987
orientation)

Free Field

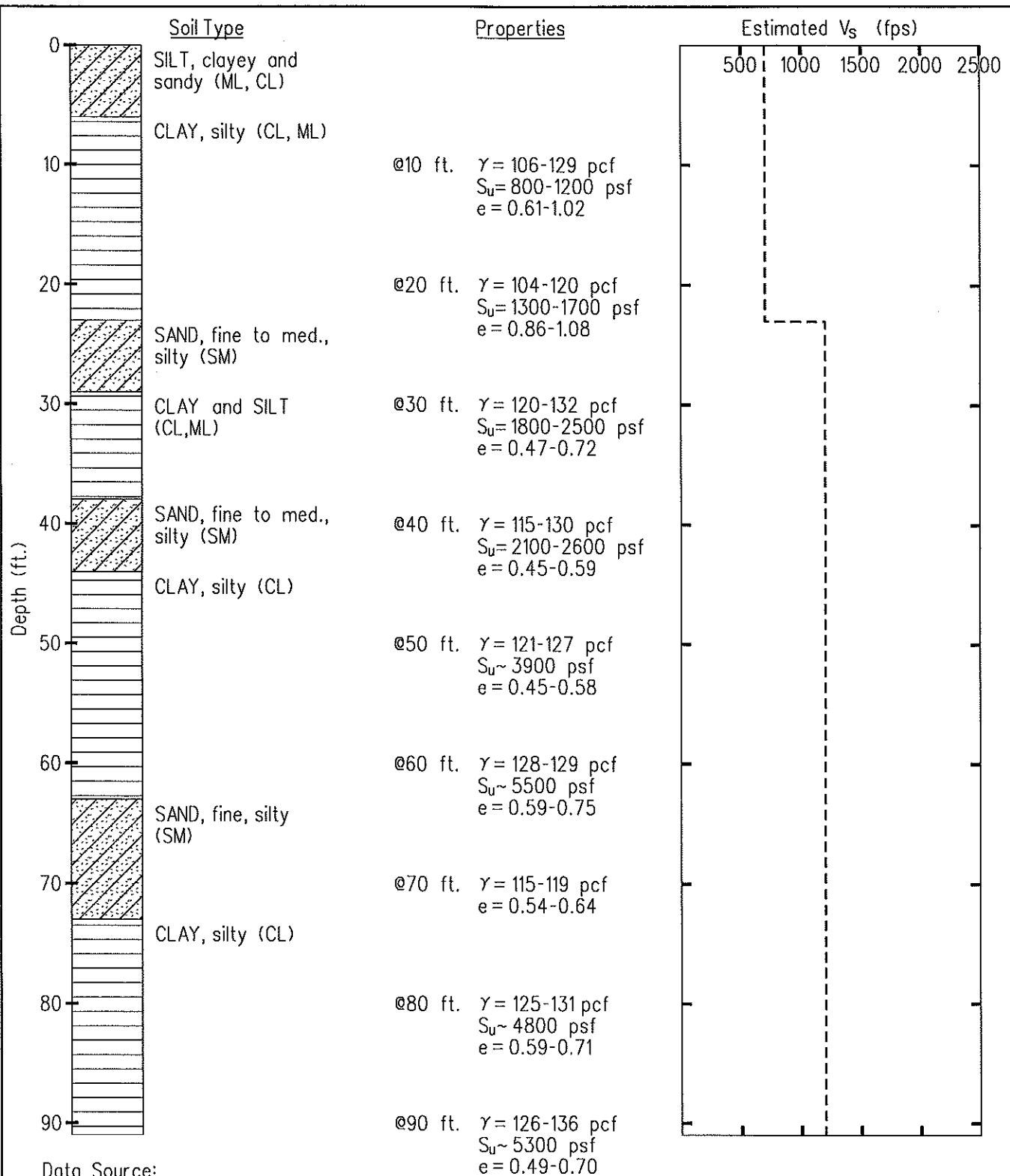
Sensor Locations

3-Story Building
Lancaster, CA

Figure A14.2

Note: Data provided by CSMIP

lanchos.dgn(50)



Data Source:
Buena Engineers (1984)

Notes:

1. Groundwater encountered at 75 ft. depth.
3. Penetration Resistance values established using ASTM D3550 3" O.D. sampler.
3. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
4. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985).

Generalized Soil Profile

3- and 5-Story Buildings
Lancaster, CA

Figure A14.3

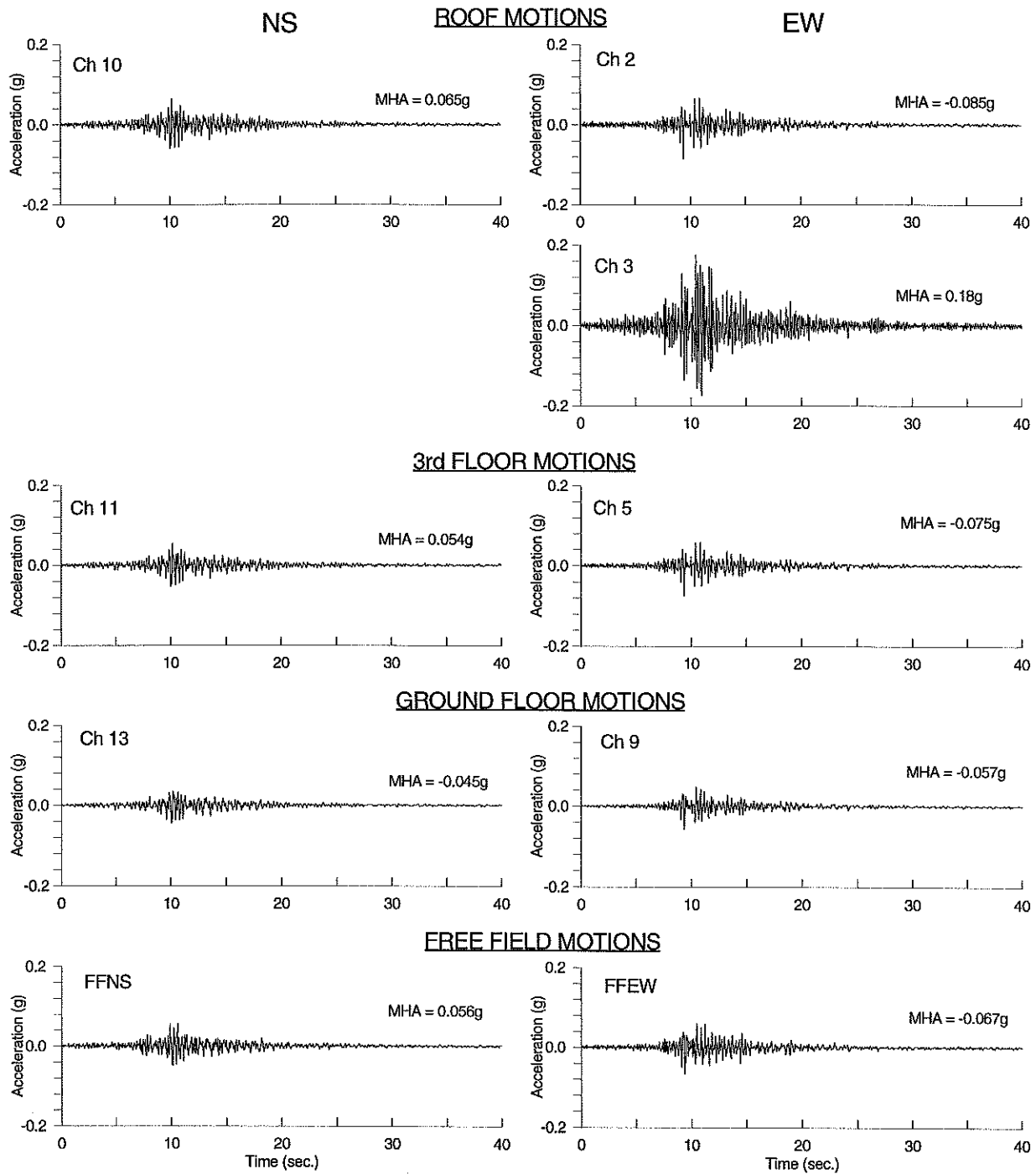


Fig. A14.WT1: LANCASTER 3 STORY BUILDING
Horizontal Acceleration Time Histories
1987 Whittier Earthquake

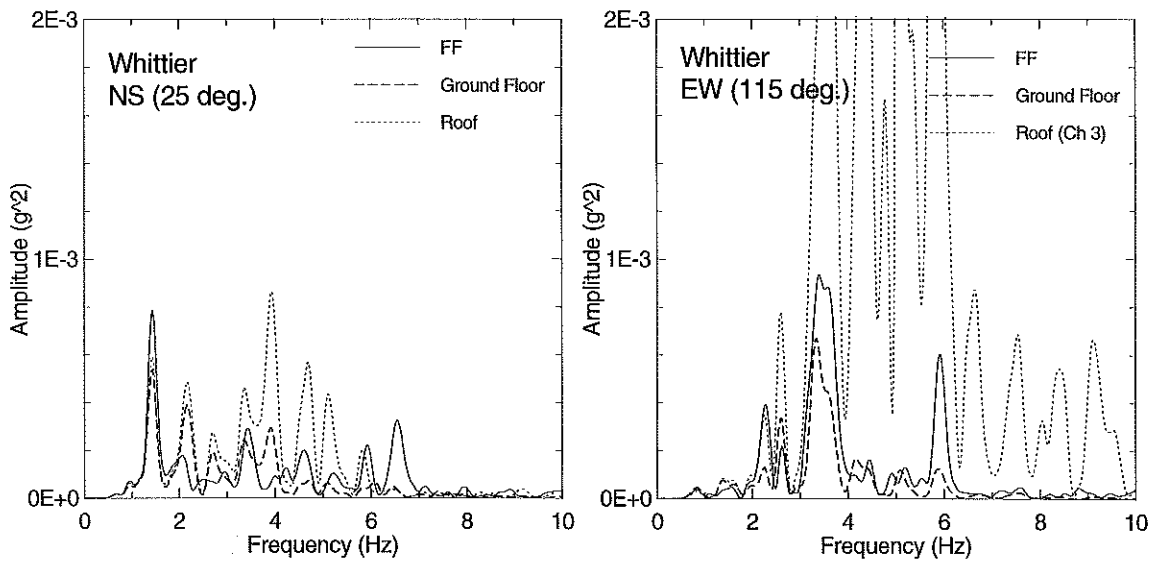


Fig. A14.WT2: LANCASTER 3-STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

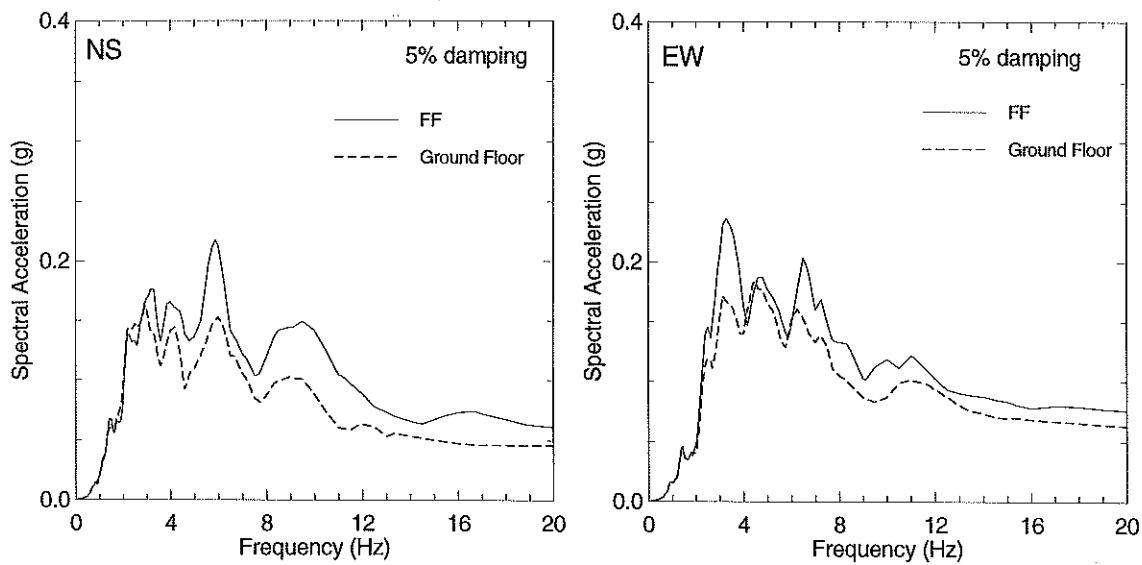


Fig. A14.WT3: LANCASTER 3 STORY BUILDING
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1987 Whittier Earthquake

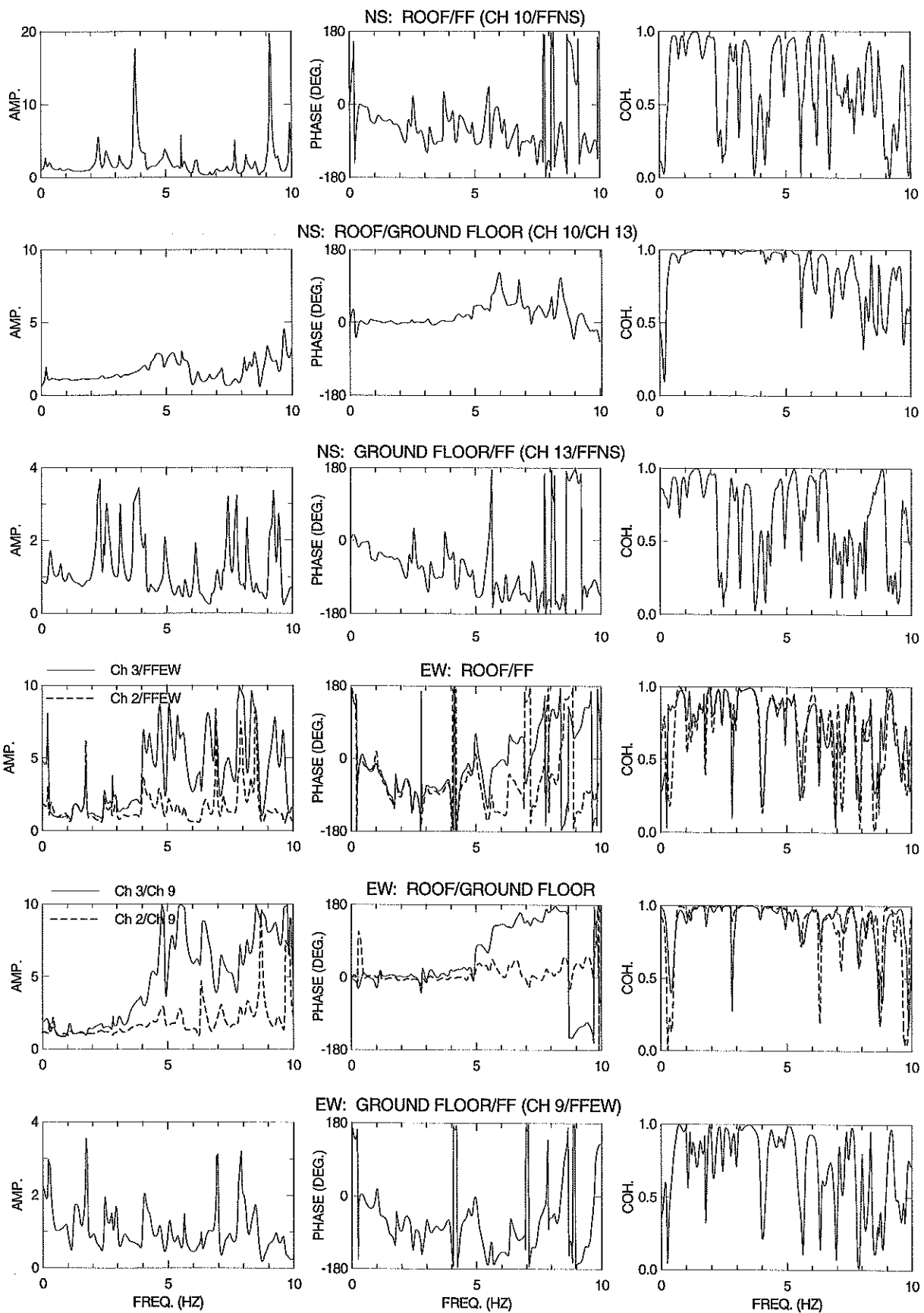


Fig. A14.WT4: LANCASTER 3 STORY BUILDING
Transmissibility and Coherency Functions
1987 Whittier Earthquake

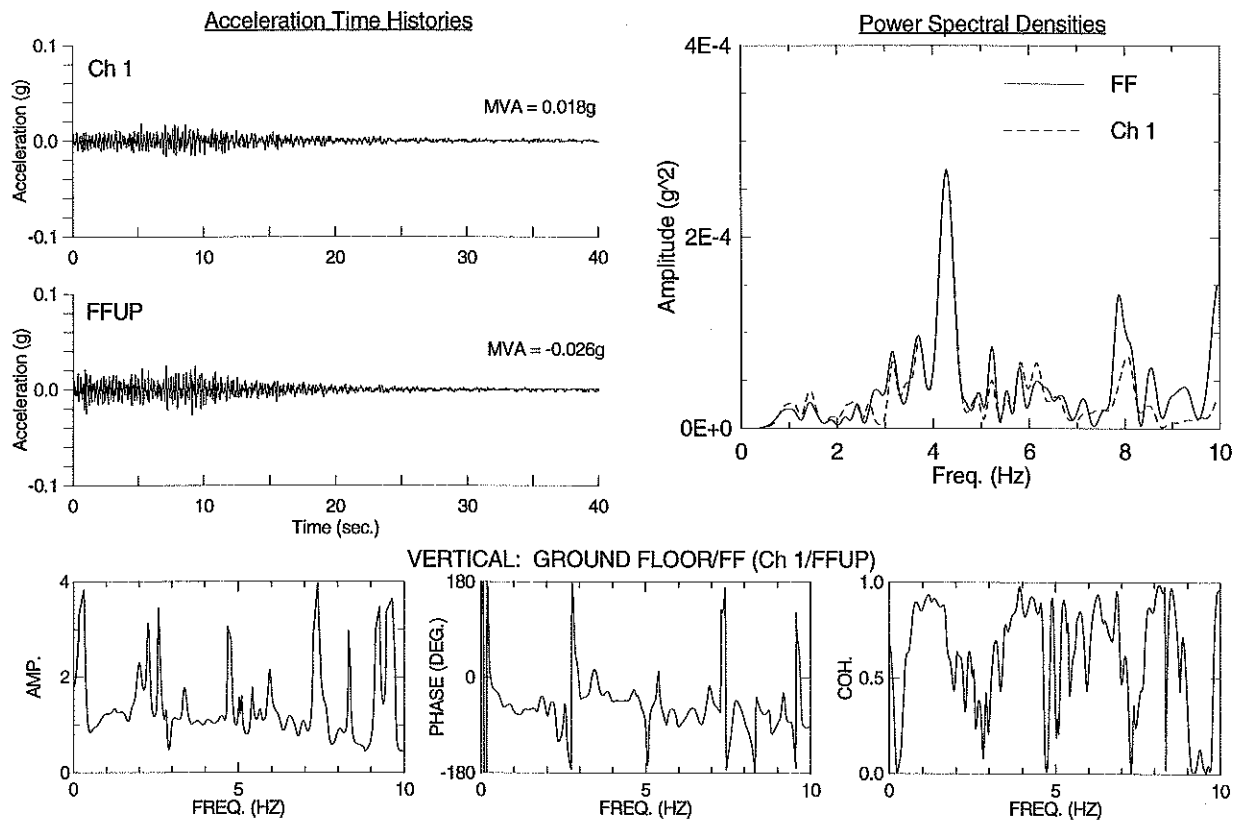


Fig. A14.WT5: LANCASTER 3 STORY BUILDING
Analysis of Vertical Motions
1987 Whittier Earthquake

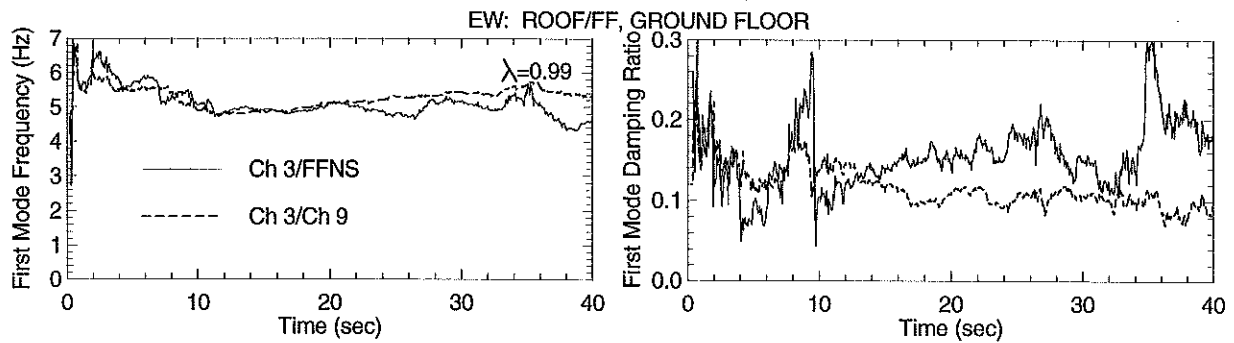


Fig. A14.NR6: LANCASTER 3 STORY BUILDING
Time Variation of First Mode Parameters
1987 Whittier Earthquake

SITE A15

Site Name: Lancaster 5 Story Building
Sensors: CSMIP 24609/24526

Lat: 34.688°
Long.: 118.158°

Floor Levels: 5
Bsmt. Levels: 0
Structure: Steel moment resisting frames

Foundation: piers
Site Geology: soil
Design Date: 1986

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers *	0.05	0.08	0.03	0.05	0.08	0.03	0.15	0.26	--
1994 Northridge	0.071	0.064	0.057	0.068	0.054	0.040	0.28	0.15	--

* Not digitized

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: *Str. Data*: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{Nyq} = 50\text{ Hz}$; FF/str. data not synchronized. *FF Data*: digitized and processed in this study, $f_{HP} = 0.23\text{--}0.25\text{ Hz}$, $f_{LP} = 13\text{--}25\text{ Hz}$, $f_{Nyq} = 100\text{ Hz}$

Final: FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A15.NR0); FF data decimated by 8, str. data decimated by 4 to $f_{Nyq} = 12.5\text{ Hz}$.

Figures

A14.1, A14.3 Site Plan, Generalized Soil Column
A15.1 Sensor Locations
A15.NR0-6 NR Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The building is 5 stories tall above ground level with no basement. The structure is part of a large hospital complex, and the north and east sides of the building are attached to adjacent structures one to three stories tall. As shown in Fig. A15.1, the north side of the structure is 3 stories tall, while the tower at the south end is 5 stories tall. Lateral loads are resisted by moment resisting steel frames in both directions. The foundation consists of 20-inch diameter concrete piers 15 to 50-foot long. Accelerographs are present on the 1st, 4th, and roof levels of the structure. The free-field instrument is located about 360 feet east of the structure in a parking lot. Soil conditions at the site are described with Site A14.

Instrument Selection and Free-Field Conditions

As shown in Fig. A15.1, structural instrumentation is present at the north and south ends of the building at the 1st and 4th levels, and at the north and south ends of the tower at the 4th and roof levels. Transverse (EW) response was evaluated using sensors at the south end of the building, while NS response was evaluated using sensors at the south and north ends of the tower (the only available sensor locations for the NS direction) (Table A15.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A15.NR1.

Table A15.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 2	Ch 3	Ch 1	Ch 10	Ch 11

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 1st Floor and Free-Field Motions

1F/FF transmissibility functions (Fig. A15.NR4) and the spectra for horizontal motions (Fig. A15.NR2-3) indicate increasing de-amplification of 1F motions with frequency in both directions, and no apparent amplification at the first-mode frequency.

Transmissibility, coherency, and power spectral density functions for FF and 1F vertical motions are presented in Fig. A15.NR5. The transmissibility function amplitude has a minor peak near $\tilde{f}_1$, indicating that rocking may have influenced the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A15.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A15.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A15.NR6).

Table A15.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) d=4, j=4		L (NS) d=3, j=4		tr (EW) d=3, j=4		L (NS) d=3, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.43	7.4	1.42	7.4	1.41	7.2	1.42	7.3	1.36	6.7 8.2	1.40	7.2
	2					4.00	6.4	3.62	10.5	4.12	8.9	3.55	12.6

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- As discussed below, the EW CEM $\tilde{\zeta}$ value of 6.7% was adjusted based on RPEM results.
- The small variation between the fixed- and pseudo flexible-base parameters suggests that rocking did not significantly influence the structural response. This is not necessarily inconsistent with the minor peak near $\tilde{f}_1$ in the vertical 1F/FF transmissibility function, as little energy is present in the 1F vertical motion at this frequency.

The RPEM results (Fig. A15.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. Unlike the CEM analyses, RPEM analyses indicated FF-R damping ratios which exceed those for the 1F-R case by an average absolute difference of about 1% (i.e., as indicated in Table A15.2, a better estimate is $\tilde{\zeta} \approx 8.2$ Hz). The CEM and RPEM results were consistent with regard to the relative magnitude of the FF-R and 1F-R first-mode frequencies.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A15.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 99 feet using the recorded free-field motions and the profile in Fig. A14.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by a 30% to account for shape effects.

Table A15.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1030$ fps	$h = 40$ ft	$emb. = 0$	$\gamma = 0.15$
NR	$\nu = 0.4$		tr: $r_1 = 99$ ft		$r_2 = 83$ ft
	$\beta^* = 2.5\%$	$V_s^* = 1000$ fps	L: $r_1 = 99$ ft		$r_2 = 121$ ft

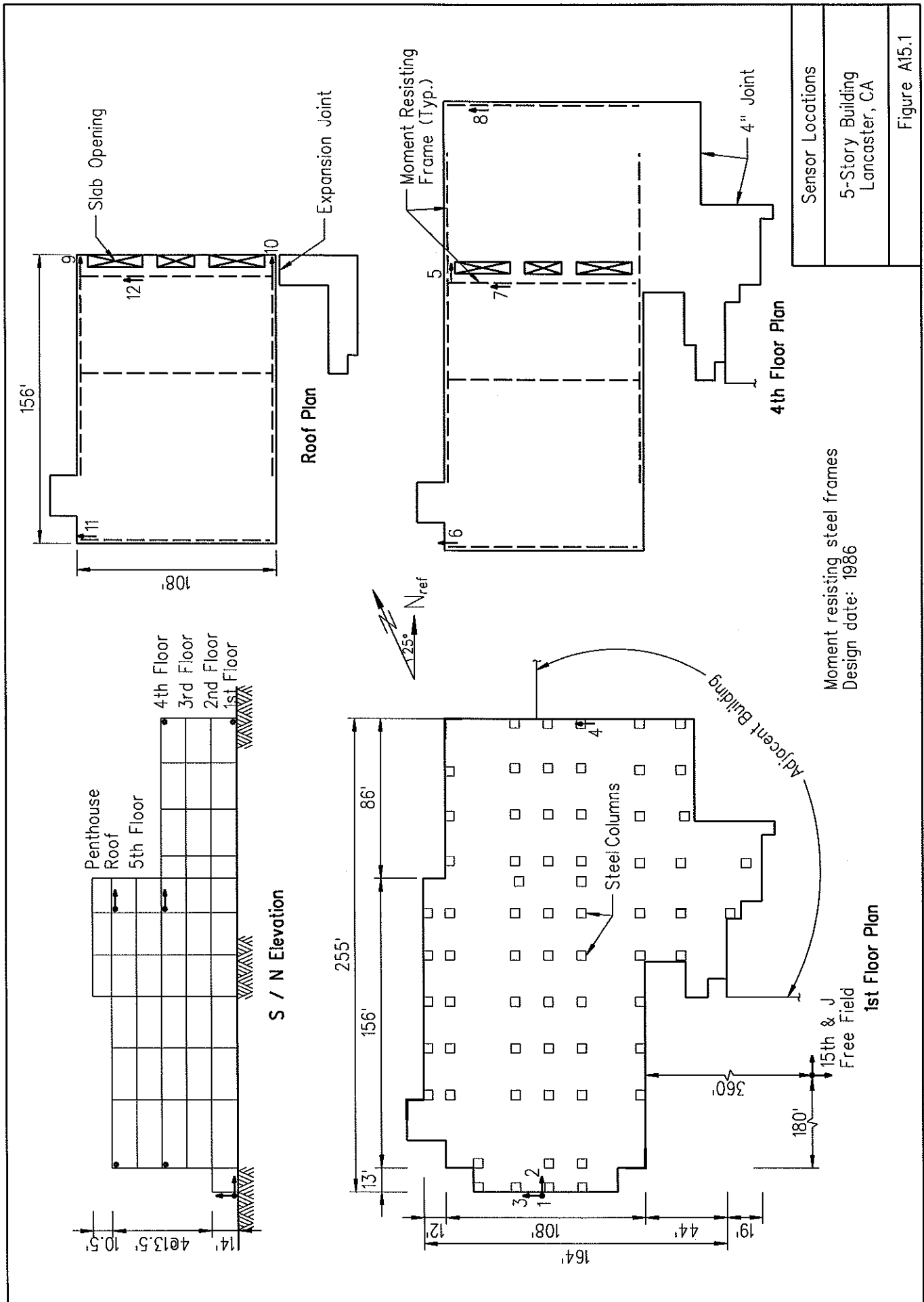
* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A15.3 with the fixed-base modal parameters in Table A15.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” formulation, with the results in Table A15.4.

Table A15.4: Empirical and predicted inertial interaction effects

NR		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.05	1.8	8.2	1.01	0.1	7.2
	Prediction	1.02	0.7	7.7	1.02	0.8	7.8

The predicted period lengthening and foundation damping satisfactorily represent the minor effects of SSI on the seismic response of this building.



A15-5

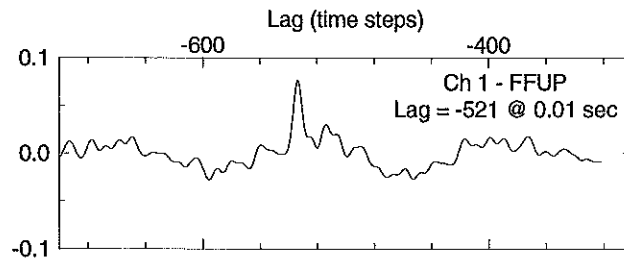


Fig. A15.NR0: Cross-correlation function between vertical 1st floor and free-field accelerations

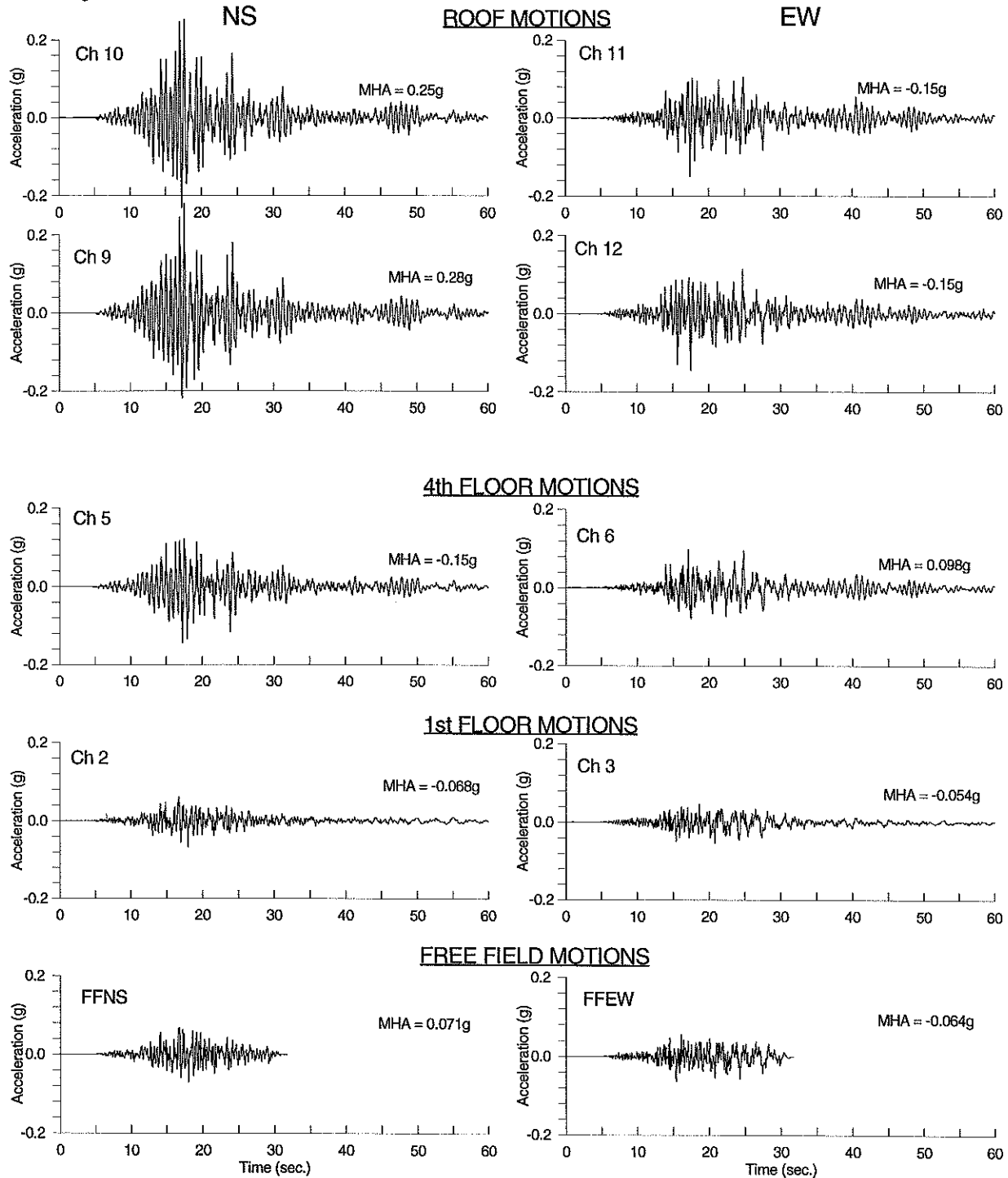


Fig. A15.NR1: LANCASTER 5 STORY BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A15-6

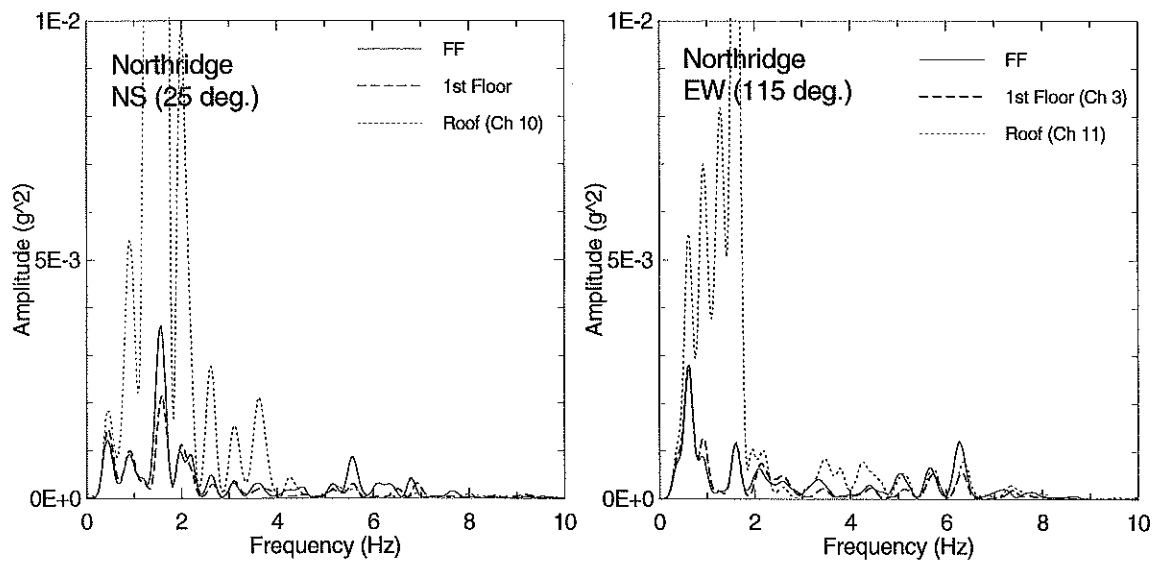


Fig. A15.NR2: LANCASTER 5 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

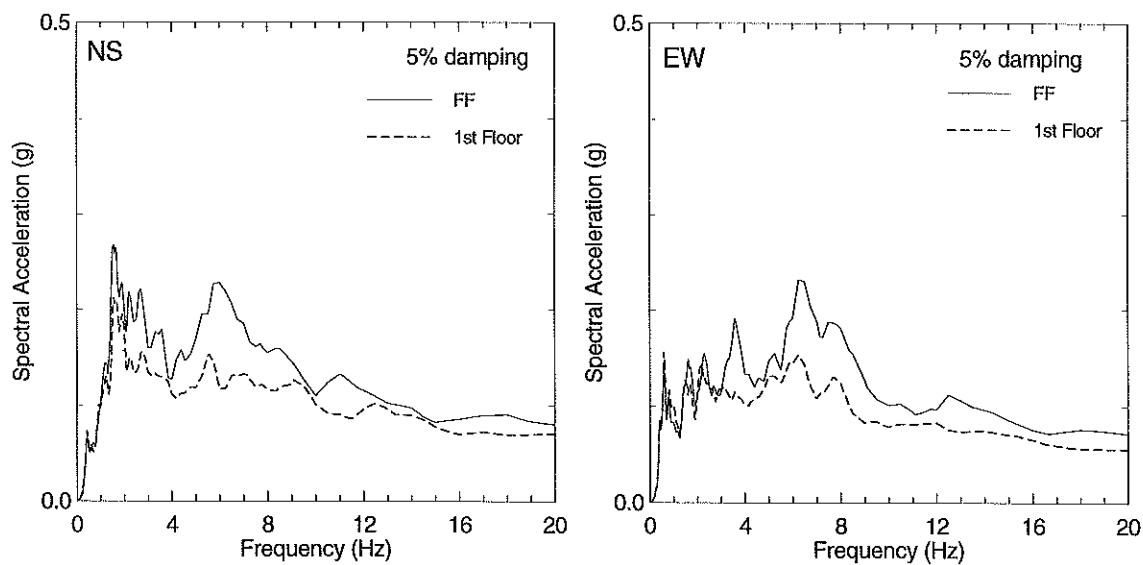


Fig. A15.NR3: LANCASTER 5 STORY BUILDING
Acceleration Response Spectra of 1st Floor and Free-Field Motions
1994 Northridge Earthquake

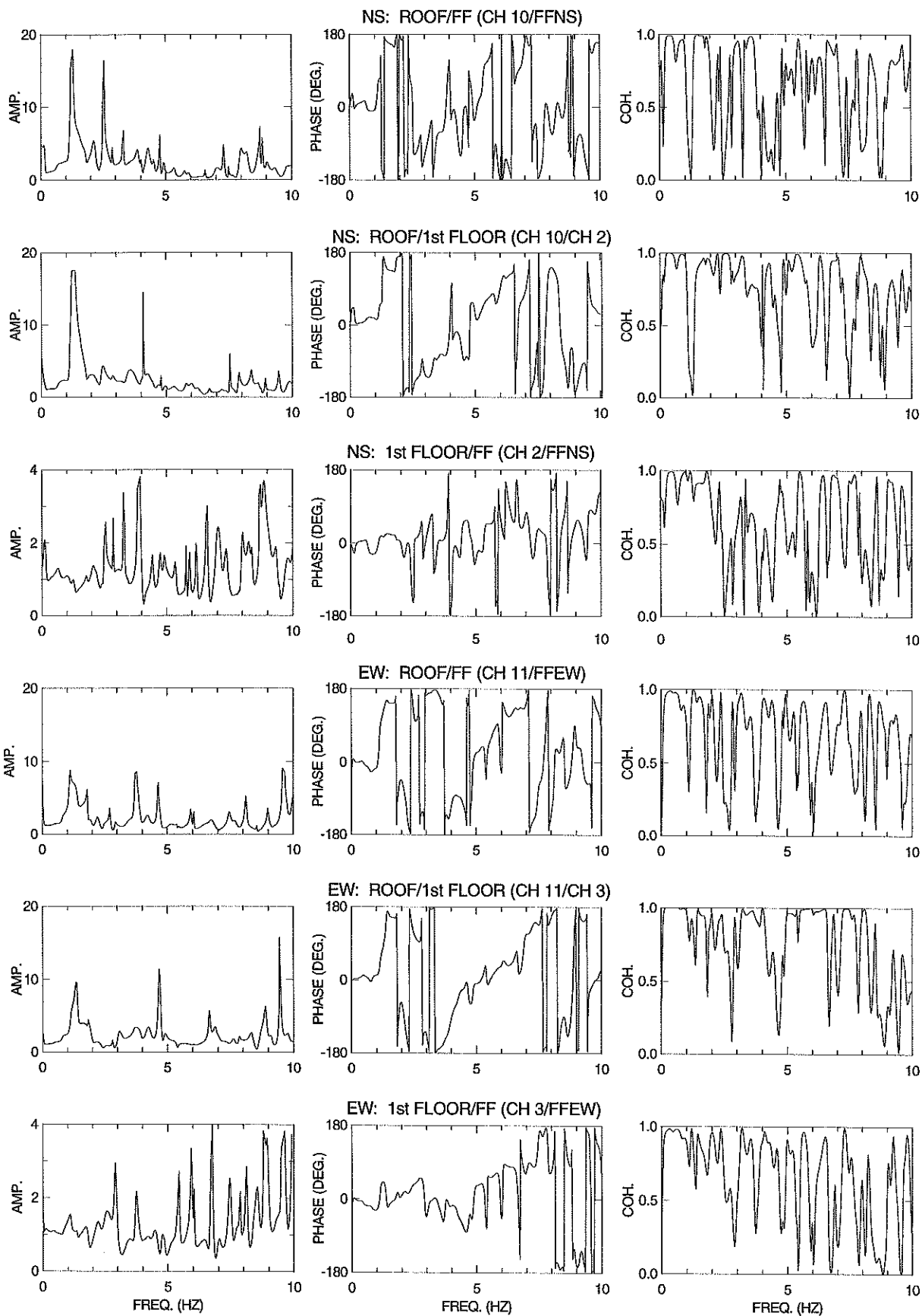


Fig. A15.NR4: LANCASTER 5 STORY BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A15-8

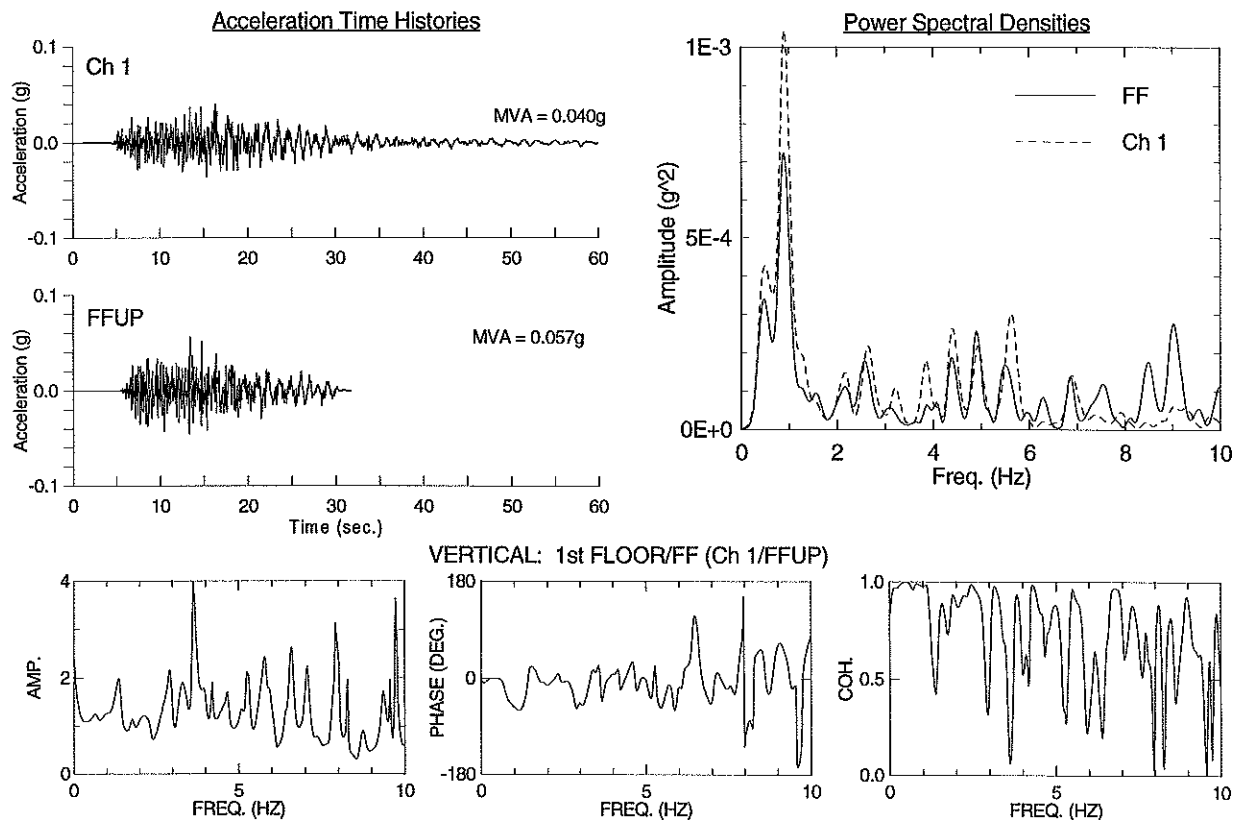


Fig. A15.NR5: LANCASTER 5 STORY BUILDING
Analysis of Vertical Motions
1994 Northridge Earthquake

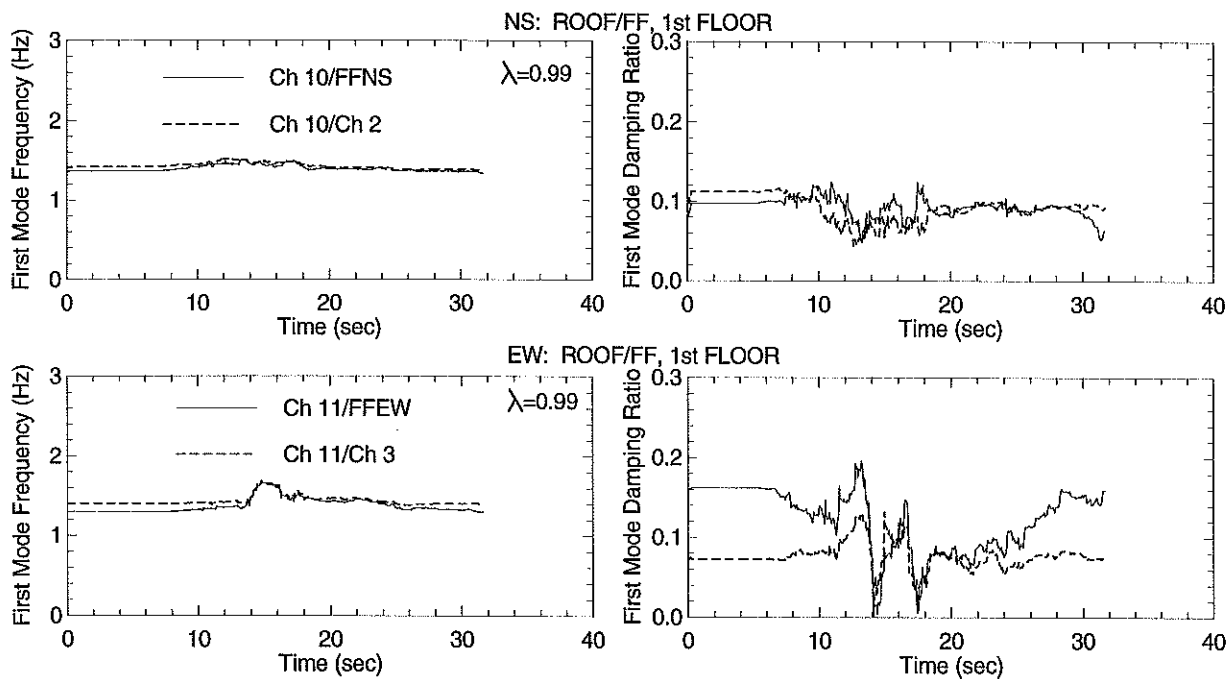


Fig. A15.NR6: LANCASTER 5 STORY BUILDING
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A16

Site Name: Lancaster Airport Control Tower
Sensors: CSMIP 24474/24475

Lat: 34.739°
Long.: 118.214°

Floor Levels: 5
Bsmt. Levels: 0
Structure: concrete columns and steel frames

Foundation: mat
Site Geology: soil
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Control Room Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier *	0.02	0.03	0.02	0.03	0.02	0.02	?	?	--
1992 Landers *	0.04	0.05	0.03	0.04	0.06	0.03	0.12	0.10	--
1994 Northridge	0.079	0.077	0.048	0.079	0.085	0.048	0.16	0.20	--

* Not digitized -- = no instrument

Preprocessing

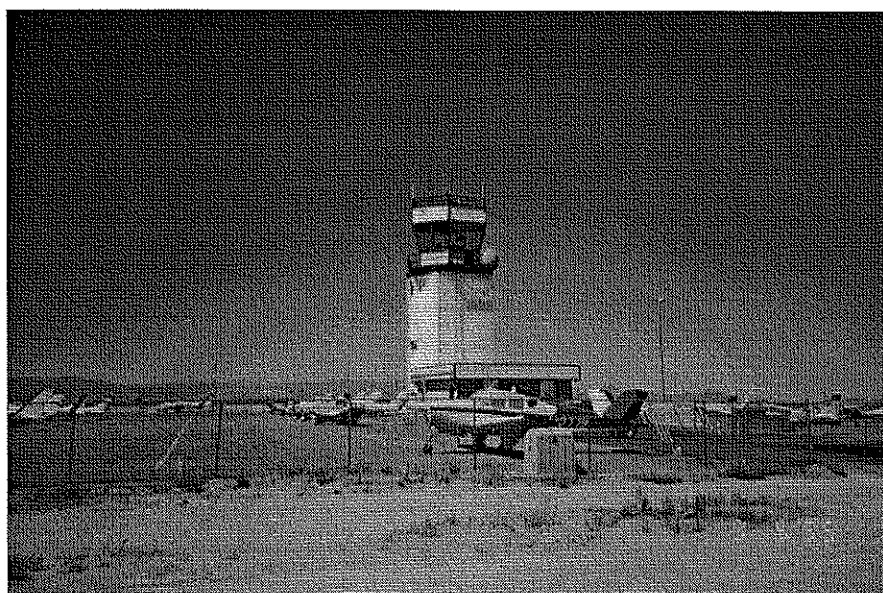
Northridge Earthquake (NR)

Preliminary: *Str. Data*: digitized and processed in this study, $f_{HP} = 0.25-0.30$ Hz, $f_{LP} = 20-30$ Hz, $f_{Nyq} = 100$ Hz. *FF Data*: CSMIP; filtered, $f = 0.3-23$ Hz; $f_{Nyq} = 25$ Hz; FF/str. data synchronized. FF not aligned with str.

Final: FF data decimated by 4 to $f_{Nyq} = 25$ Hz and rotated 30° CCW to align with str.

Figures

A16.1-A16.2 Sensor Locations, Generalized Soil Column
A16.NR1-6 NR Analysis Results



View of tower with FF in foreground

DISCUSSION

Site Description

The building is 5 stories tall above ground level with no basement, and has a total height of 58.5 feet. The lower 4 levels are square in plan view (17.5 x 17.5 feet), and the control room on the 5th level has a hexagon shape which is 9 feet 4 inches on a side at the control room floor and 11 feet 10 inches on a side at the control room roof. The structural system in the lower four levels consists of concrete columns at the four corners with steel beams and exterior sheet metal sheathing. The structure at the control room level is steel frame. No structures of significant size are located near the control tower. The foundation consists of a concrete mat founded 6 feet below grade, with concrete walls transferring loads from the base of the columns at ground level to the mat. Accelerographs are present on the ground, control room floor, and control room roof levels of the structure. The free-field instrument is located about 200 feet south of the building near an airplane parking lot.

The site is located on level terrain in the Antelope Valley, California. Based on borings by Osborne Laboratories (1971), soils at the site consists of Holocene silty sands and fine sands which extend to the maximum depth explored of 90 feet. Soil conditions at greater depth, and the depth to bedrock, are poorly characterized in the area. The shear wave velocities in Fig. A16.2 were measured by the downhole method at the Rosamond Dry Lake location (about 6.6 miles northeast of the site) in geologic materials similar to those at the site (Fumal et al., 1982).

Instrument Selection and Free-Field Conditions

The sensors used to evaluate the structural response are indicated in Table A16.1. Instrument errors contaminated the film traces for sensors 4 and 7, so these data were not digitized. For this reason, a control room floor record (Ch 8) was used to evaluate the EW structural response. Acceleration time histories recorded by the various sensors are shown in Fig. A16.NR1.

Table A16.1: Sensors in structure used for analyses

Ground Floor (GF)			Cntr. Room (CR)	
NS	EW	V	NS	EW
Ch 6	Ch 9	Ch 1	Ch 3	Ch 8

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A16.NR4) and the spectra for horizontal motions (Fig. A16.NR2-3) indicate no significant filtering of GF motions relative to the FF for frequencies up to 10 Hz.

Transmissibility, coherency, and power spectral density functions for FF and GF vertical motions are presented in Fig. A16.NR5. Some amplification of vertical GF motions occurred near the first-mode response frequency. However, power spectral densities of these motions show little energy content near the structure's fundamental frequency, indicating that significant base rocking is unlikely.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A16.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A16.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A16.NR6).

Table A16.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-CR)				Flex.-Base (FF-CR)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=2, j=4		L (EW) d=0, j=6		tr (NS) d=4, j=4		L (EW) d=1, j=6	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	3.86	17.6	3.66	12.5	2.97	8.1	3.13	8.1	2.92	7.7	3.09	8.4
	2					7.35	13.4	8.69	19.2	6.96	14.8	8.30	7.9

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- Differences between the fixed- and pseudo flexible-base frequencies are quite large in both directions, which contradicts the lack of evidence for a significant rocking effect from the vertical GF/FF transmissibility function. The estimated fixed-base parameters are highly sensitive to GF-CR and FF-CR parameters due to the structure's large aspect ratio, and may be somewhat unreliable.

The RPEM results (Fig. A16.NR6) show the first-mode frequency dropped about 0.5 Hz over the first 15 seconds and subsequently rose to near the original value. These changes in frequency are not likely associated with structural damage, but may have resulted from softening of the foundation soils during strong shaking. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-CR and GF-CR first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A16.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 18 feet using the recorded free-field motions and the profile in Fig. A16.2. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A16.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1000$ fps	$h = 45$ ft	emb. = 6 ft	$\gamma = 0.15$
NR	$\nu = 0.33$		tr:	$r_1 = 11.9$ ft	$r_2 = 11.9$ ft
	$\beta^* = 1.7\%$	$V_s^* = 953$ fps	L:	$r_1 = 11.9$ ft	$r_2 = 11.9$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A16.3 with the fixed-base modal parameters in Table A16.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A16.4.

Table A16.4: Empirical and predicted inertial interaction effects

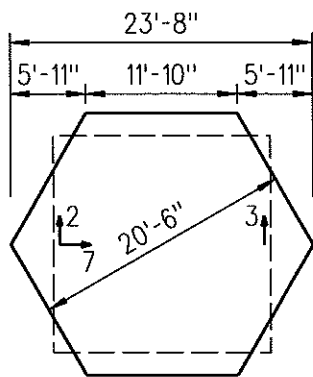
NR	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
		1.32	0.1	7.7	1.18	0.8	8.4
	Pre.-Vel.	1.15	0.2	11.8	1.14	0.2	8.8
	Pre.-Bie.	1.23	1.9	11.4	1.21	1.7	8.8

Given the uncertainties in the system identification results for the fixed-base case noted above, the accuracy of the predictions is satisfactory. Soil damping $\tilde{\zeta}_0$ was overpredicted by the “modified Bielak” formulation, and the “modified Veletsos” results appear to be slightly superior.

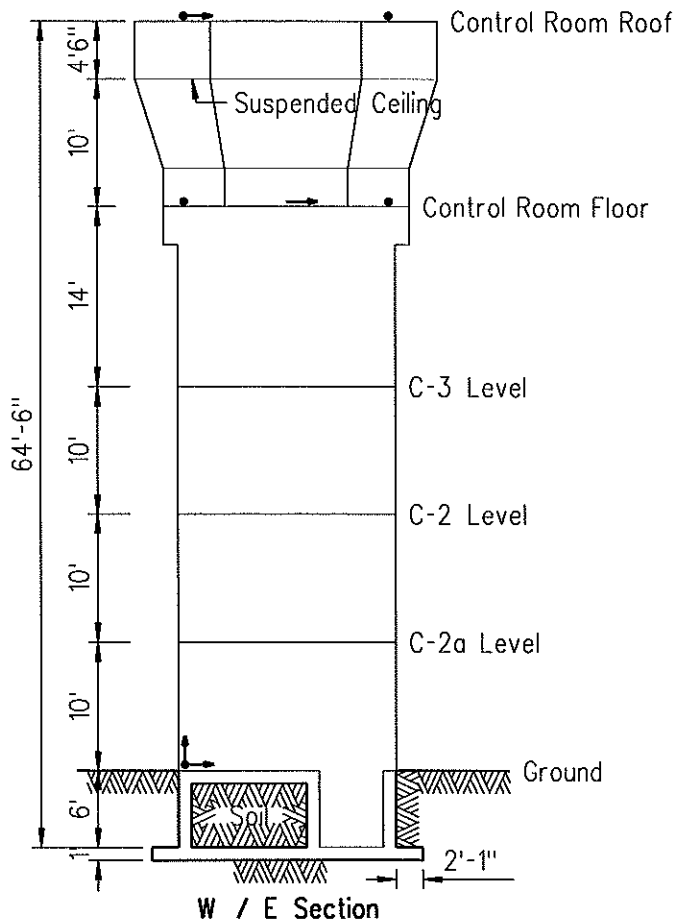
Data Sources

Fumal, T.E., Gibbs, J.F., and Roth, E.F. (1982). “In-situ measurements of seismic velocity at 22 locations in the Los Angeles, California Region,” *Open File Rpt. 82-833*, U.S. Geological Survey.

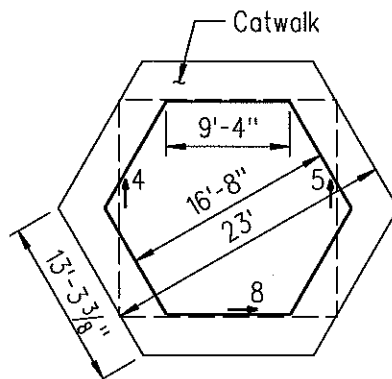
Osborne Laboratories (1971). boring log for Lancaster Fox Airfield Control Tower, obtained from project plans.



Control Room Roof Plan

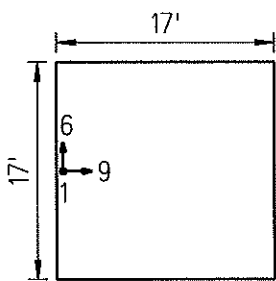


W / E Section



Control Room Floor Plan

Fox Airfield Control Tower
(CSMIP 24474)

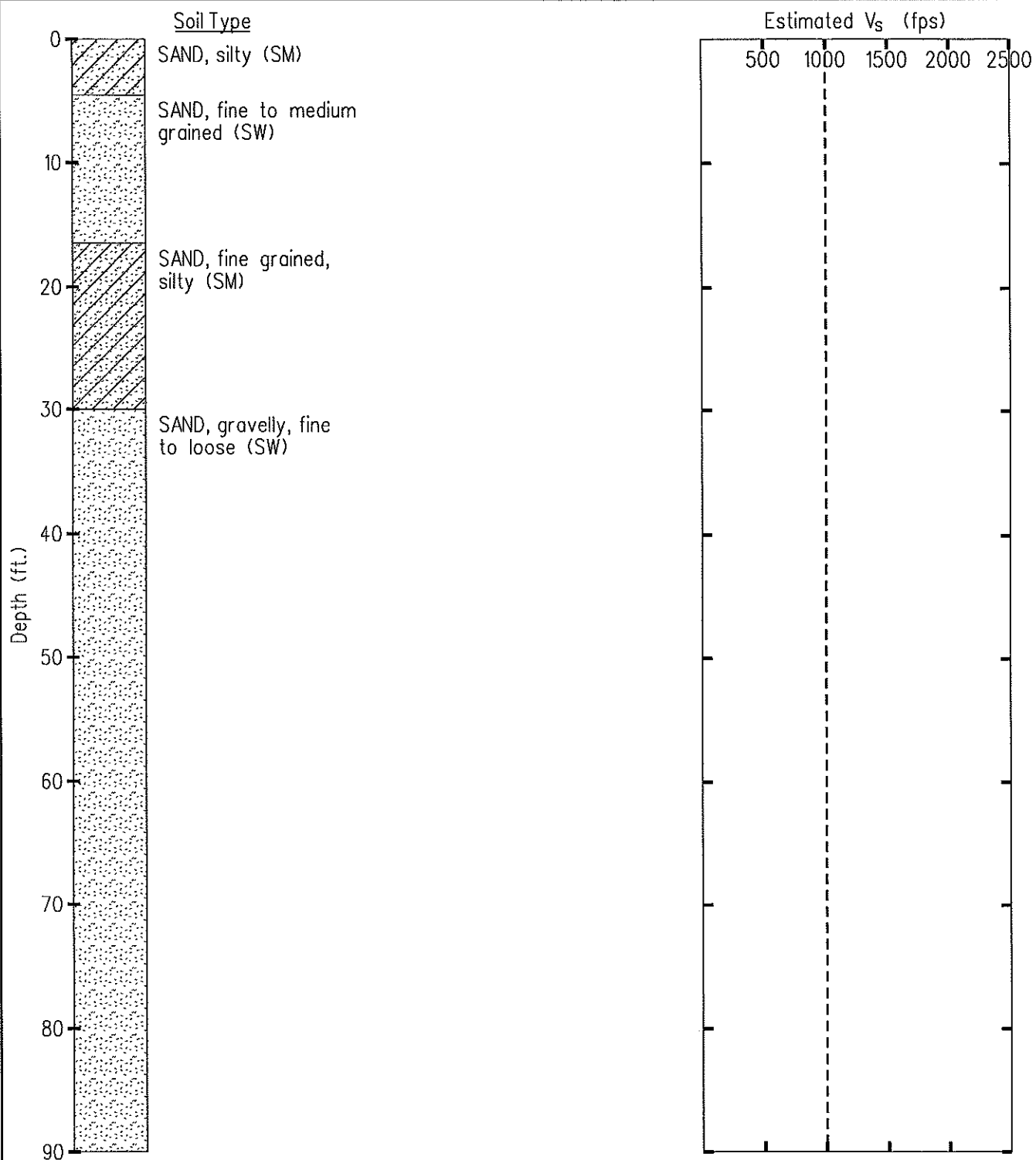


Ground Floor Plan



~200'
Fox Airfield Grounds
Free Field (CSMIP 24475)

Sensor Locations
Airport Control Tower Lancaster, CA
Figure A16.1



Data Source:
Osborne Laboratories (1971)

Notes:

1. No groundwater information available.
2. Shear wave velocities based on in situ measurements in similar geologic materials (Fumal et al., 1982).

Generalized Soil Profile
Airport Control Tower Lancaster, CA
Figure A16.2

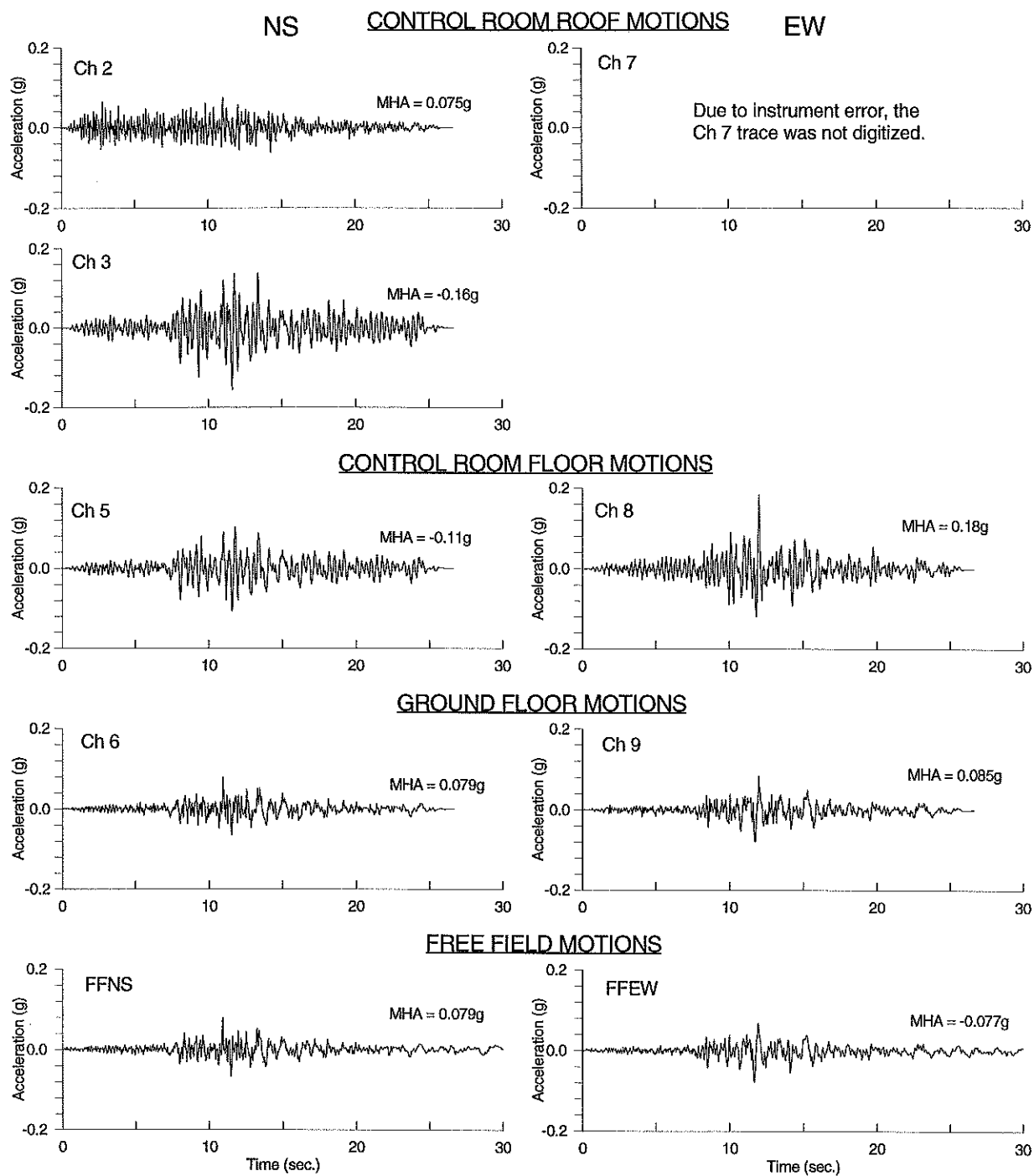


Fig. A16.NR1: LANCASTER AIRPORT CONTROL TOWER
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

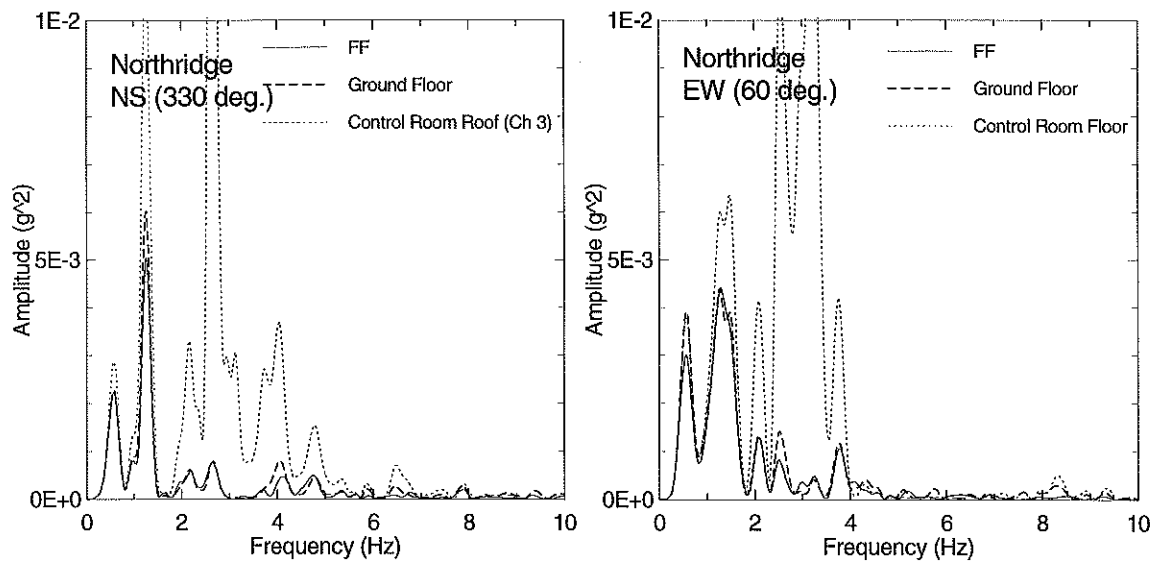


Fig. A16.NR2: LANCASTER AIRFIELD CONTROL TOWER
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

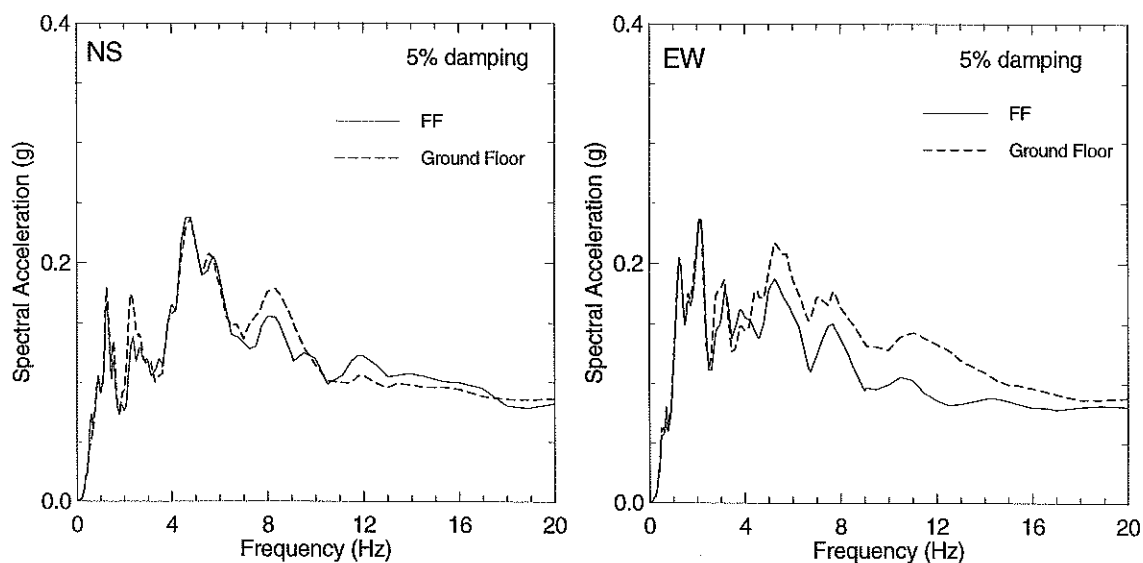


Fig. A16.NR3: LANCASTER AIRFIELD CONTROL TOWER
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1994 Northridge Earthquake

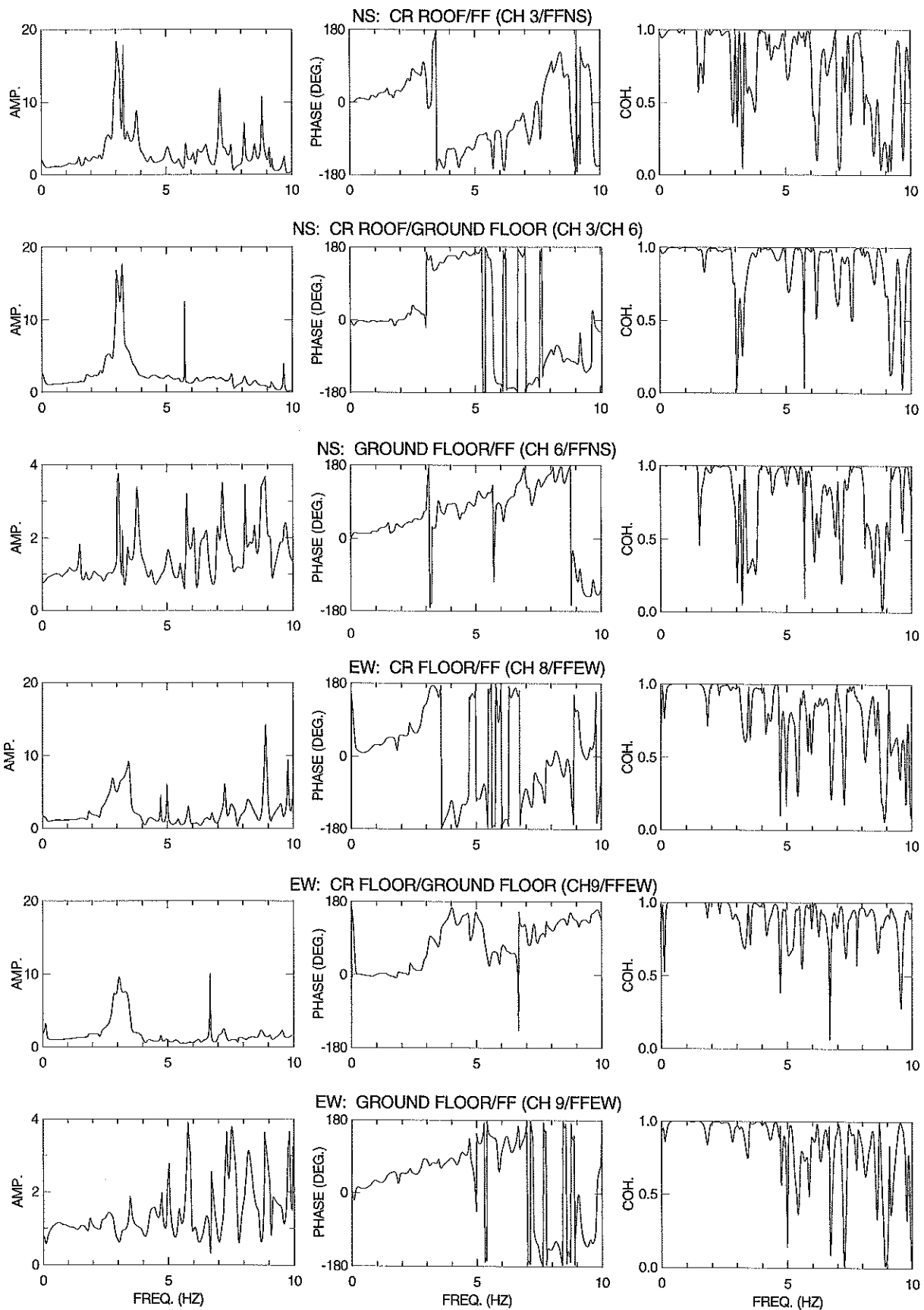


Fig. A16.NR4: LANCASTER AIRFIELD CONTROL TOWER
Transmissibility and Coherency Functions
1994 Northridge Earthquake

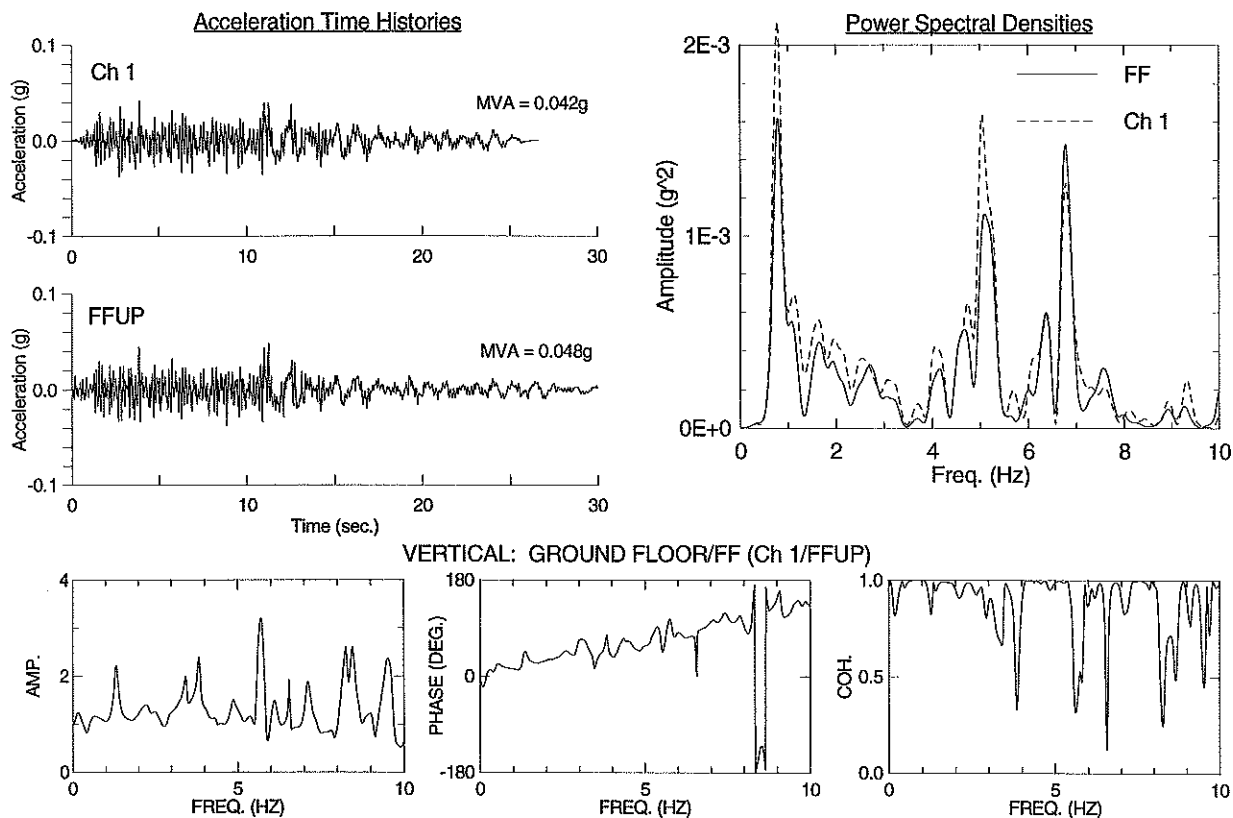


Fig. A16.NR5: LANCASTER AIRPORT CONTROL TOWER
Analysis of Vertical Motions
1994 Northridge Earthquake

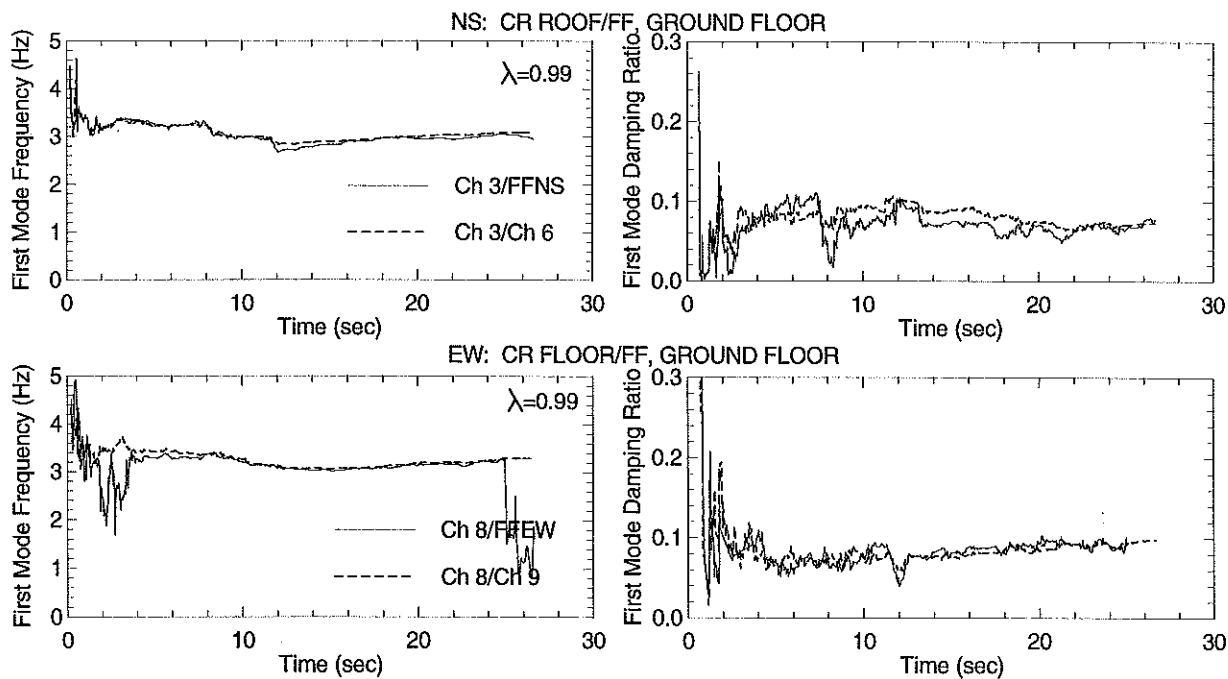


Fig. A16.NR6: LANCASTER AIRPORT CONTROL TOWER
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A17

Site Name: Loma Linda VA Hospital
Sensors: USGS 5229

Lat: 34.050°
Long.: 117.249°

Floor Levels: 4
Bsmt. Levels: 0
Structure: Reinforced concrete shear walls

Foundation: concrete piers
Site Geology: soil
Design Date: 1975

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier*FFNo.	0.04	0.04	0.03	0.03	0.02	0.01	0.04	0.07	--
FFSo.	0.04	0.02	0.02						
1992 Landers*FFNo.	0.08	0.09	0.10	0.08	0.08	0.04	0.15	0.22	--
FFSo.	0.08	0.10	0.08						
1994 Northridge No.	0.055	0.046	0.022	0.053	0.060	0.025	0.095	0.18	--
FFSo.	0.042	0.057	0.019						

* Not processed -- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: USGS, digitized at $f_{Nyq} = 100$ Hz; no processing, FF/str. data not synchronized

Final: filtered (this study), $f_{HP} = 0.2$ Hz; decimated by 4 to $f_{Nyq} = 12.5$ Hz; synchronized using cross correlation of vertical accelerations (Fig. A17.NR0).

Note: Channels 2 and 3 in the original data were switched

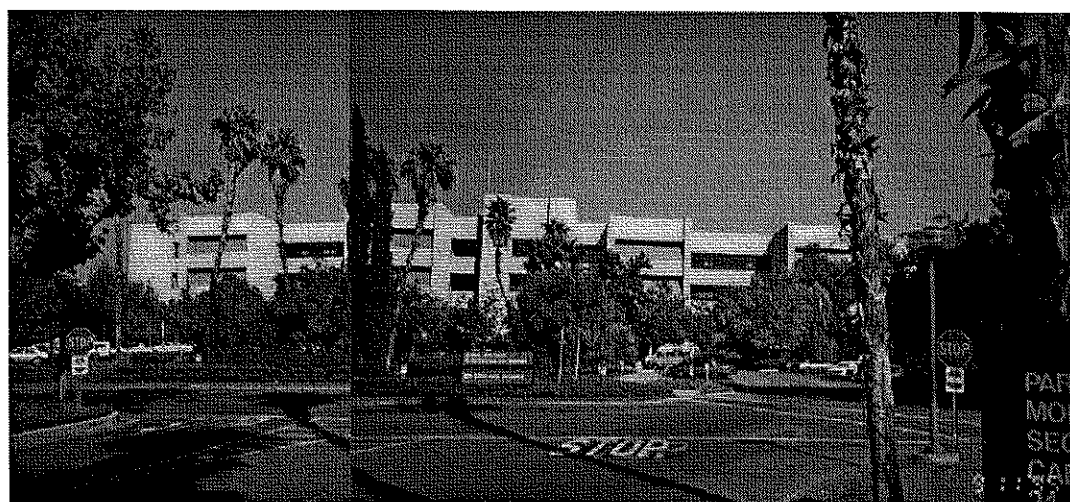
Figures

A17.1-A17.3

Site Plan, Sensor Locations, Generalized Soil Column

A17.NR0-5

NR Analysis Results



View of structure, looking east

DISCUSSION

Site Description

The building is 4 stories tall above ground level with no basement, and is essentially rectangular in plan view with dimensions of 432 by 441 feet. Single story additions are present immediately east and south of the main structure, but no significant structures are located near the building. The lateral load resisting system consists of reinforced concrete shear walls in both directions, while moment-resisting steel frames resist vertical loads (Holmes et al., 1976). The foundation consists of 24 to 42-inch diameter concrete piers which generally extend to 60 to 80-foot depth. Tie beams interconnect the heads of the piers, and a concrete slab is present at the first floor level. As shown in Fig. A17.2, accelerographs are present on the 1st floor and roof levels of the structure. The north and south free-field instruments are located in parking lots, about 400 feet north and 400 feet southwest of the building, respectively.

The site is located at the south end of the San Bernardino Valley on alluvial fan deposits which slope mildly down to the north. Based on borings by Woodward Lungren Assoc. (1973), soil conditions consist of (Fig. A17.3): 115 feet of medium dense to dense silty gravely sands, about 50 feet of denser and stiffer interbedded sands and silts, and markedly stiffer silts and sands which extend to the maximum depth explored of 200 feet. The shear wave velocities shown in Fig. A17.3 are based on in situ crosshole measurements by Woodward Lungren Assoc..

Instrument Selection and Free-Field Conditions

Accelerographs near the centroid of the building were selected for use in analyses, the specific sensors are indicated in Table A17.1. Acceleration time histories recorded by these sensors are shown in Fig. A17.NR1. Note that channels 2 and 3 were switched in the original data provided by USGS.

Table A17.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 2	Ch 1	Ch 6	Ch 4

There are no structures or significant slope breaks near the free-field instruments that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 1st Floor and Free-Field Motions

1F/FF transmissibility functions (Fig. A17.NR4) and the spectra for horizontal motions (Fig. A17.NR2-3) generally indicate increasing de-amplification of 1F motions with frequency, though minor amplification is evident in the NS direction near the first-mode frequency ($\tilde{f}_1 \approx 3.5$ Hz).

Transmissibility, coherency, and power spectral density functions for FF and 1F vertical motions are presented in Fig. A17.NR5. The 1F motions do not appear to be amplified near the first-mode frequency, indicating that base rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A17.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A17.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A17.NR6).

Table A17.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF No/So - R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=0, j=2		L (EW) d=0, j=2		tr (NS) d=0, j=2		L (EW) d=0, j=2	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	4.09	6.3	3.43	6.3	4.01	8.37	3.40	6.8	3.51/3.41	15.3/17.2	3.16/3.32	9.8/12.2

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- Based on a comparison of the model output and recorded roof records, FF south data gave a better EW model, while FF north data gave a slightly better NS model.
- The small variation between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for a rocking response in the vertical 1F/FF transmissibility function.
- Notable differences are apparent in the spectra of the motions from the two free-field instruments (Fig. A17.NR2), indicating that spatial incoherence effects at the site were significant.

The RPEM results (Fig. A17.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the 1F-R and FF-R first-mode parameters. RPEM analyses were performed for FF-R response using the FF north data.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A17.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 246 feet using the recorded free-field motions and the profile in Fig. A17.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A17.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1520$ fps	$h = 50$ ft	emb.=0	$\gamma = 0.15$
NR	$\nu = 0.33$		tr: $r_1 = 246$ ft		$r_2 = 248$ ft
	$\beta^* = 3.4\%$	$V_s^* = 1415$ fps	L: $r_1 = 246$ ft		$r_2 = 250$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A17.3 with the fixed-base modal parameters in Table A17.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A17.4.

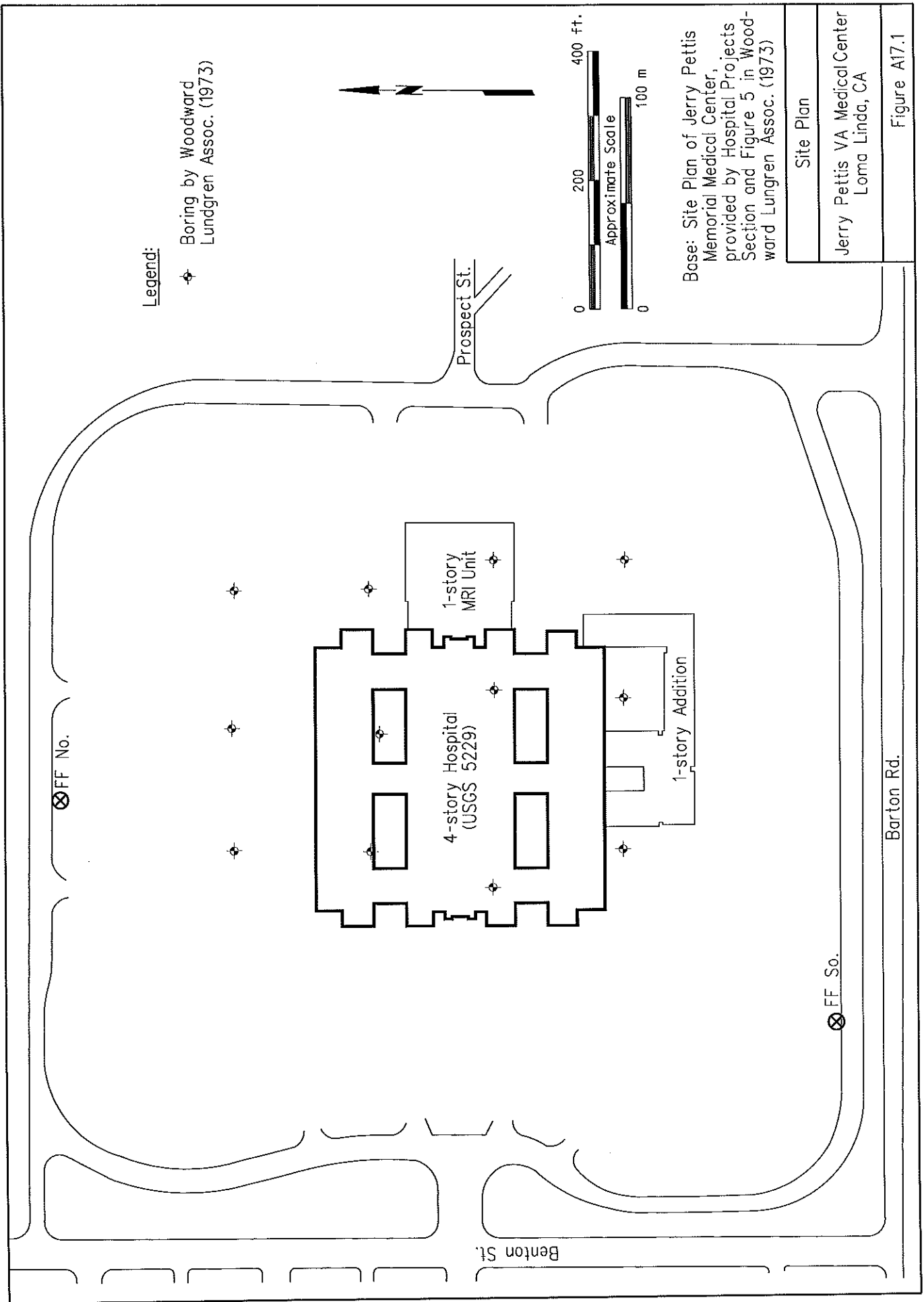
Table A17.4: Empirical and predicted inertial interaction effects

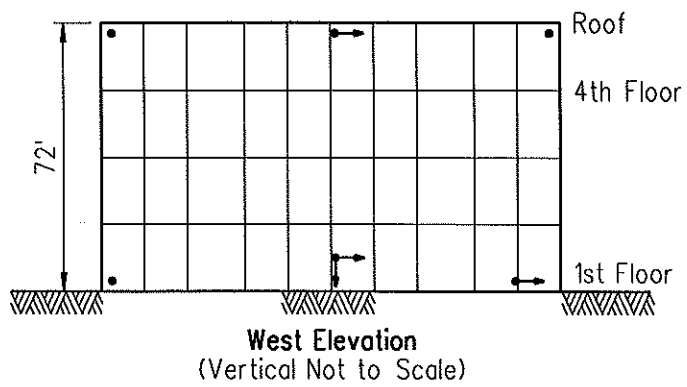
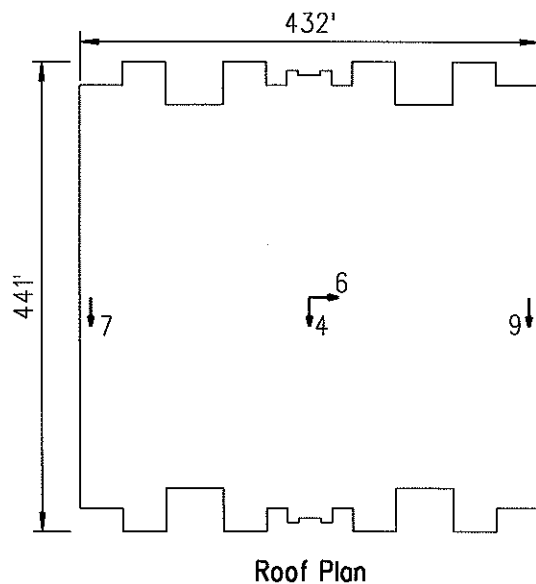
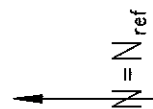
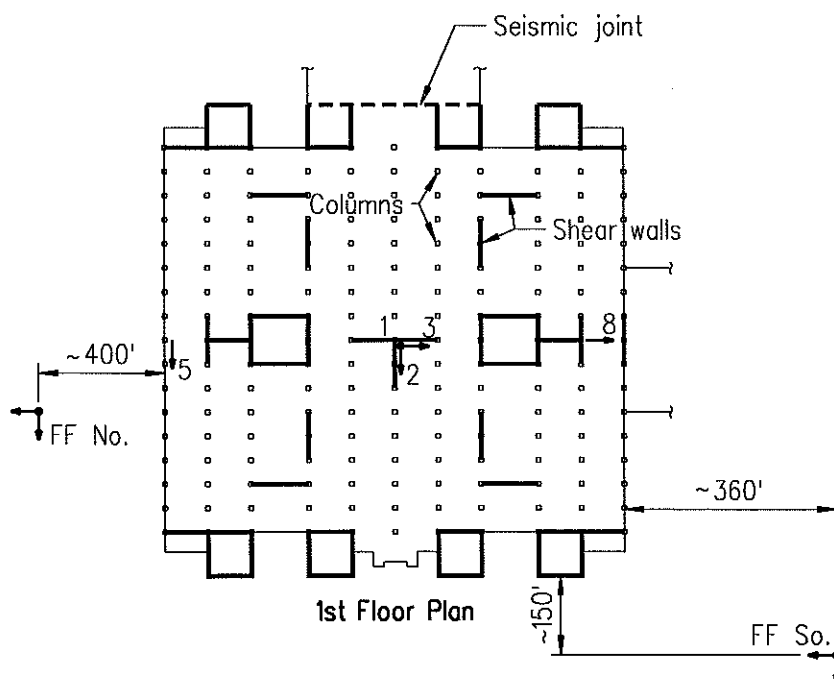
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.16	11.3	15.3	1.08	4.8	9.8
	Prediction	1.21	12.5	16.1	1.15	9.9	14.1

The predicted period lengthening and foundation damping are satisfactory, particularly for the transverse direction. These SSI effects are somewhat overpredicted for the longitudinal direction.

Data Sources

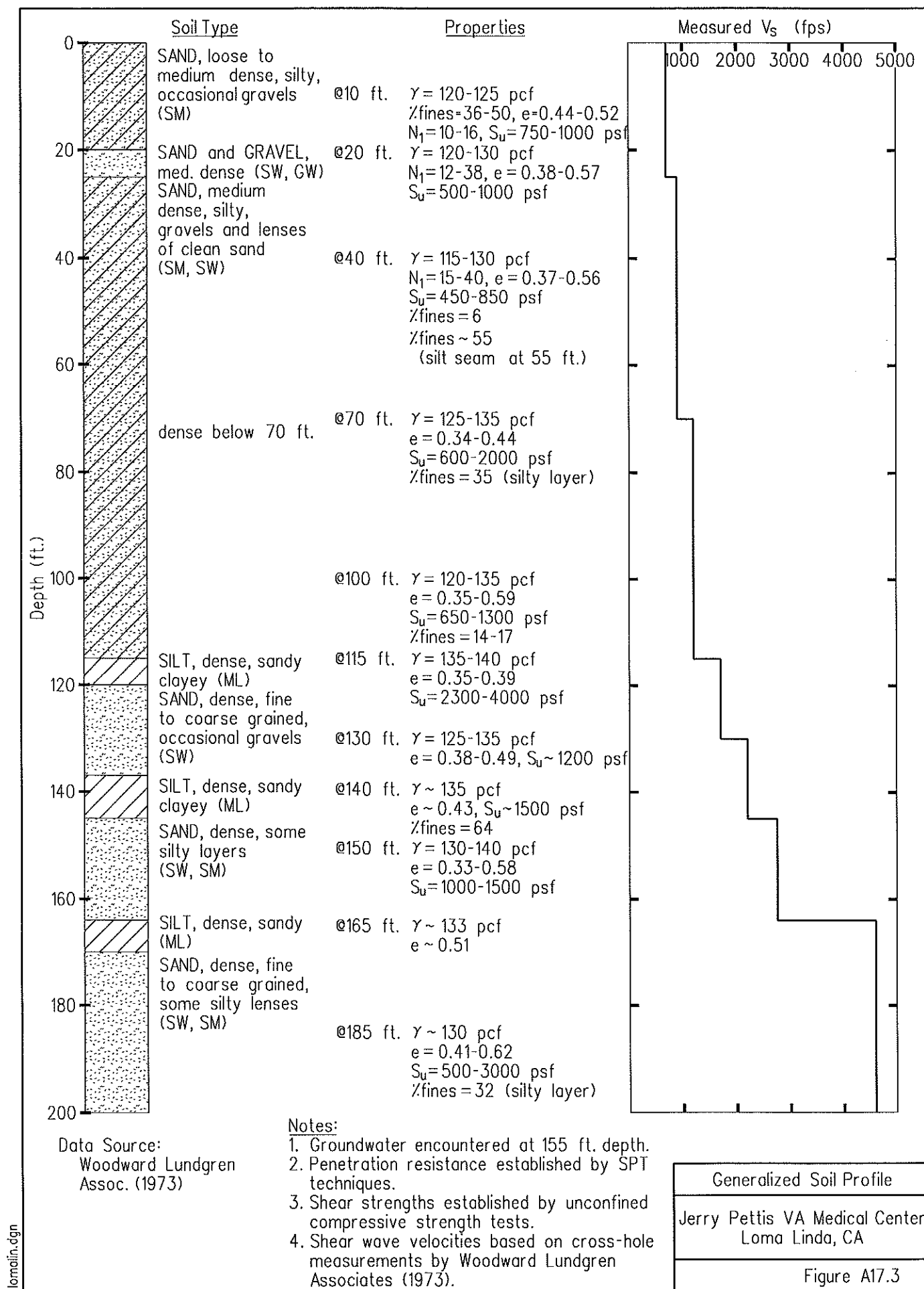
- Holmes, W.T. (1976). "Seismic design of the Veteran's Administration Hospital at Loma Linda, California," *Proc. Int. Symposium on Earthquake Structural Engrg.*, St. Louis, MS, Vol. 2, 823-841..
- Woodward Lundgren Assoc. (1973). "Preliminary soil investigation and site response analyses for the proposed veterans administration hospital, Site No. 4, Loma Linda, CA," *Job No. G-12616*.





Note: Data provided by USGS and Holmes (1976)

Sensor Locations
Jerry Pettis VA Medical Center Loma Linda, CA
Figure A17.2



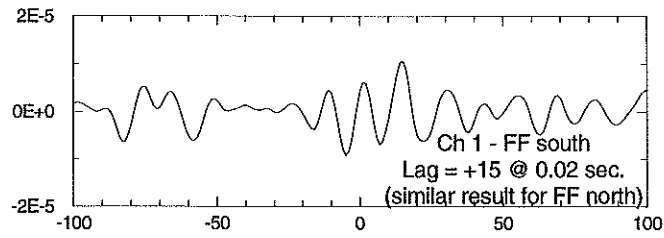


Fig. A17.NR0: Cross correlation function between vertical 1st floor and free-field accelerations

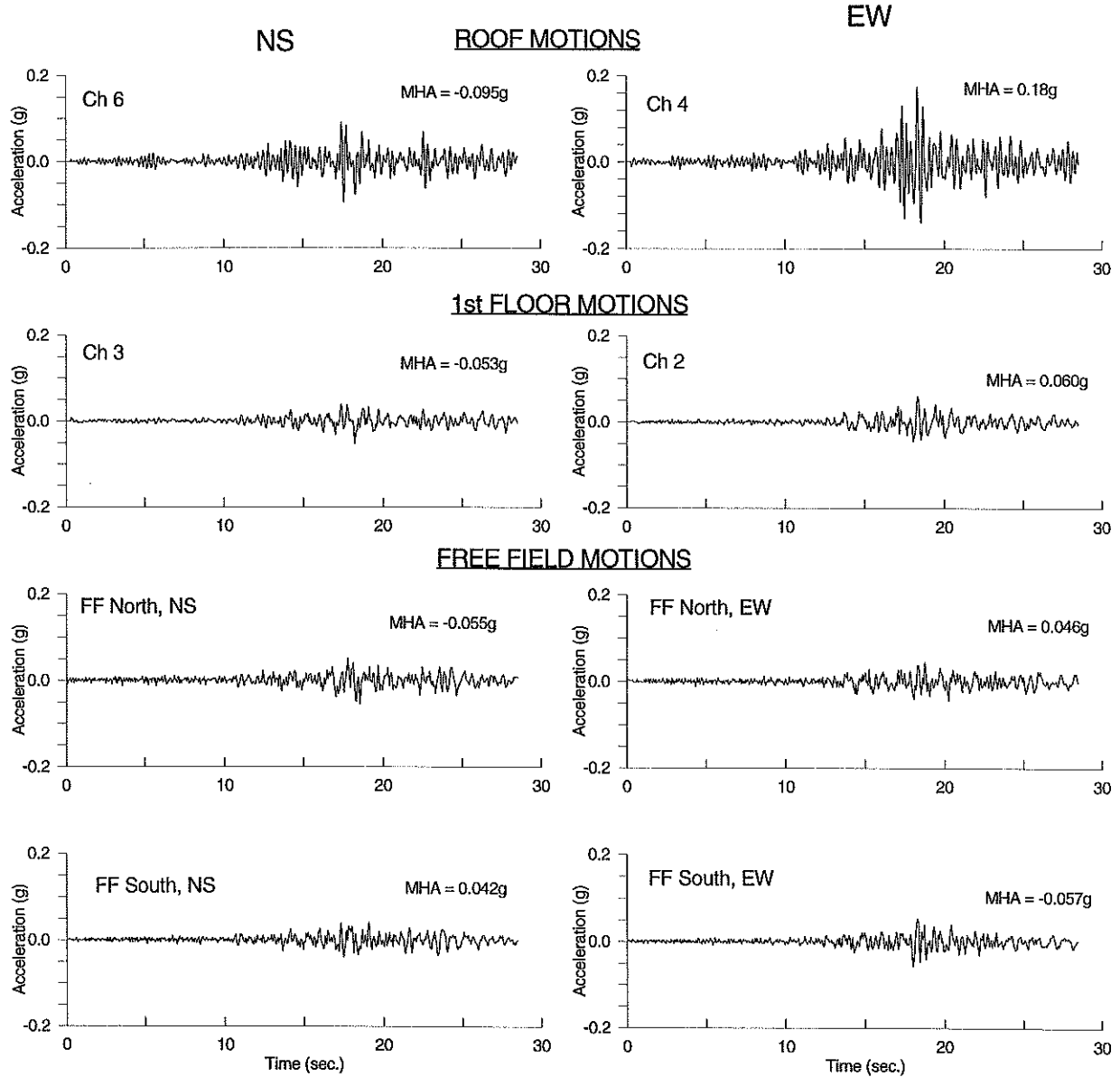


Fig. A17.NR1: LOMA LINDA VA HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

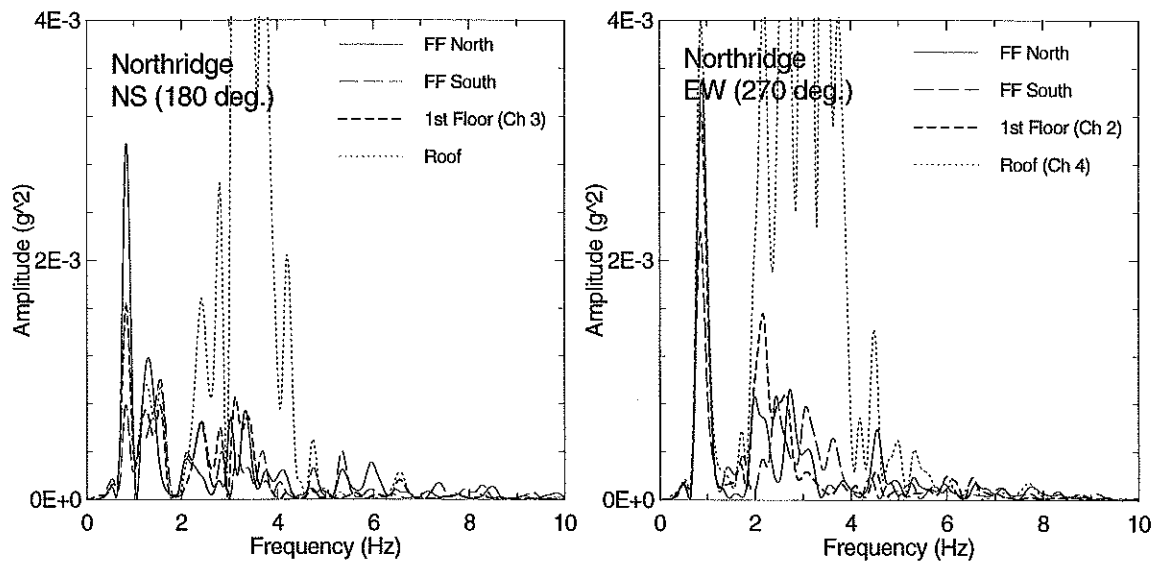


Fig. A17.NR2: LOMA LINDA VA HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

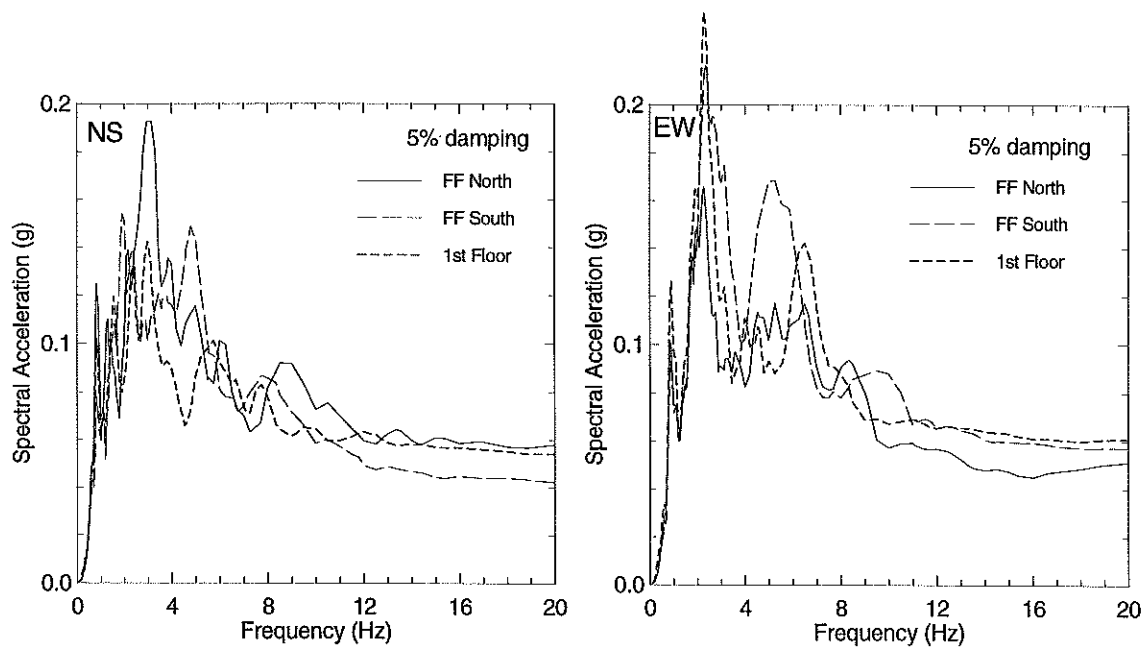


Fig. A17.NR3: LOMA LINDA VA HOSPITAL
Acceleration Response Spectra of 1st Floor and Free-Field Motions
1994 Northridge Earthquake

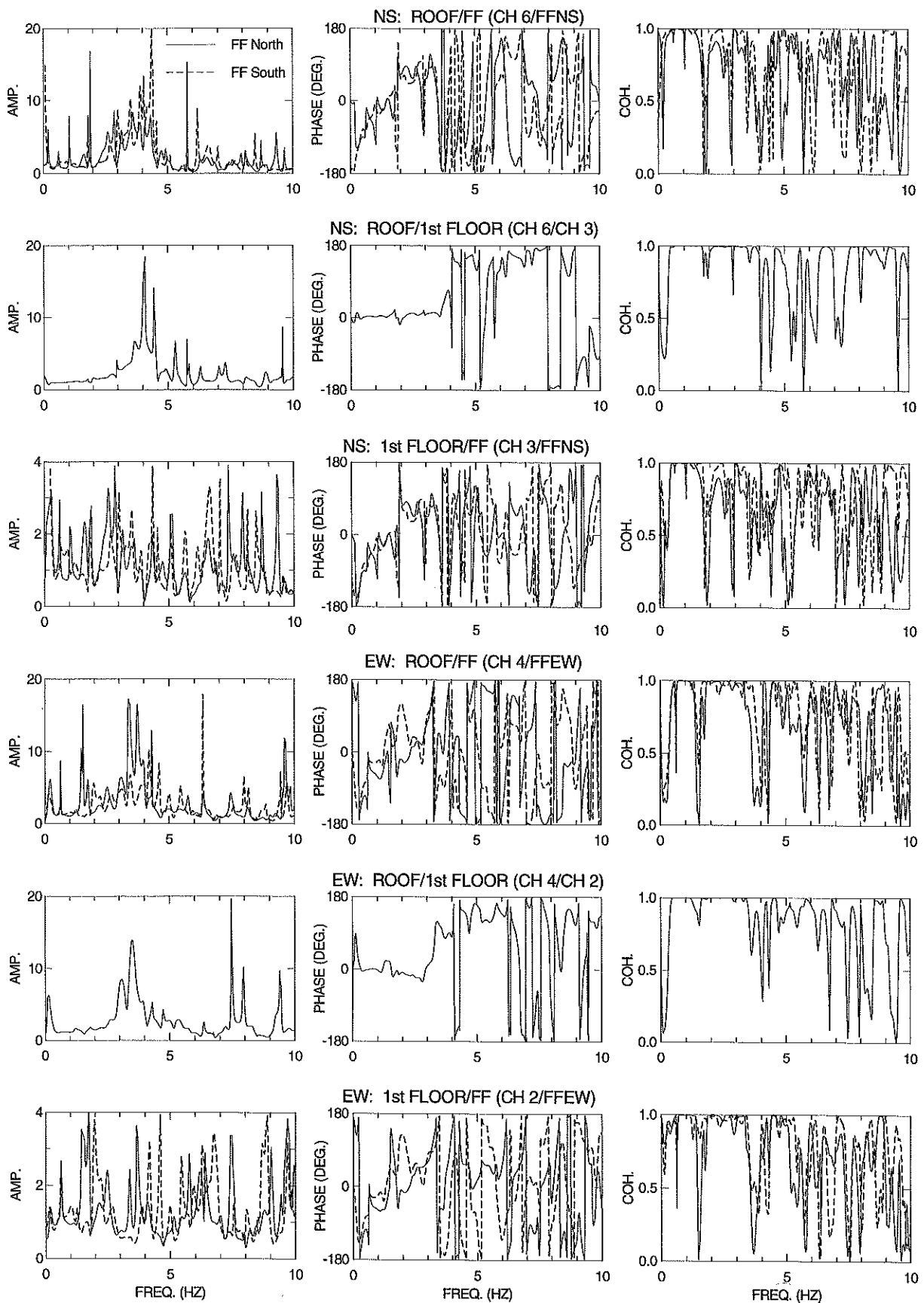


Fig. A17.NR4: LOMA LINDA VA HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A17-10

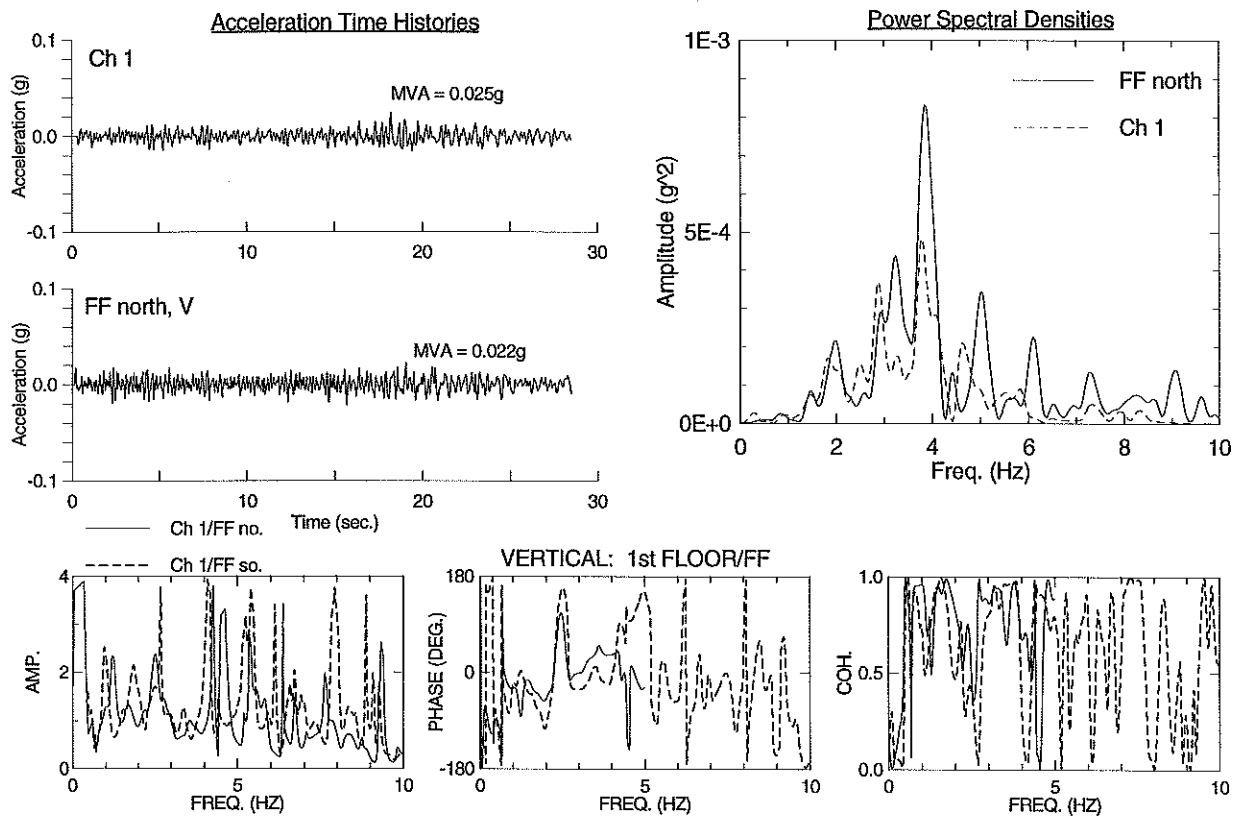


Fig. A17.NR5: LOMA LINDA VA HOSPITAL
Analysis of Vertical Motions
1994 Northridge Earthquake

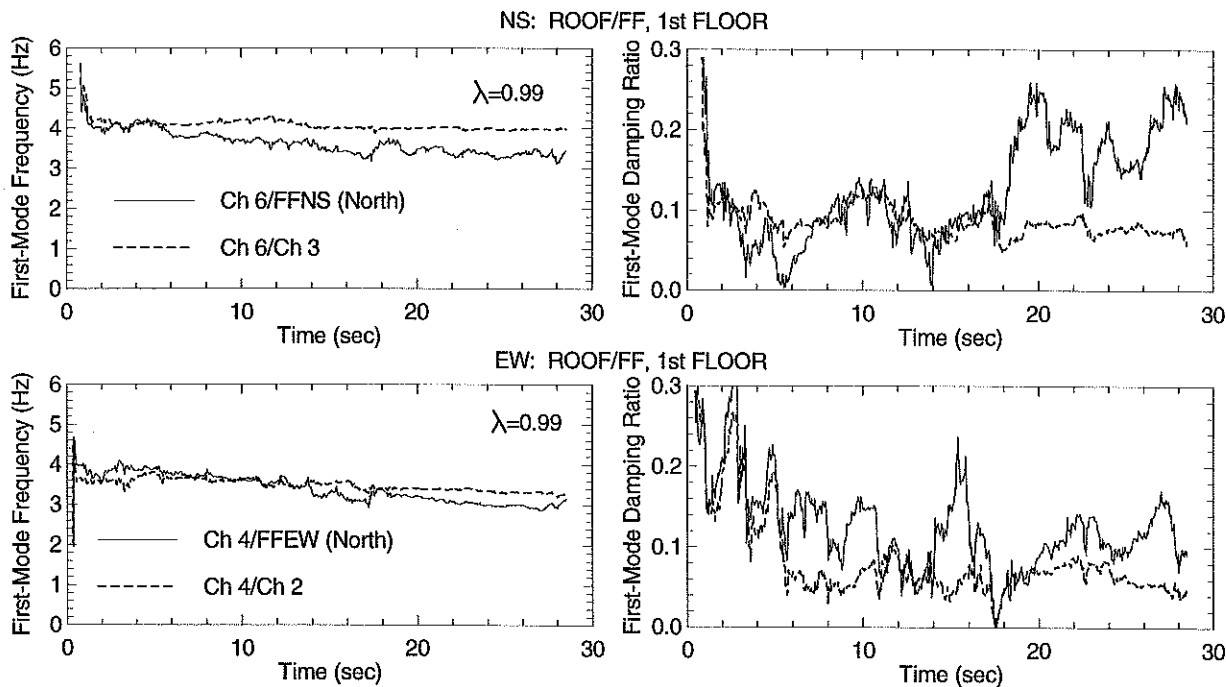


Fig. A17.NR6: LOMA LINDA VA HOSPITAL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A18

Site Name: Long Beach Harbor Administration Bldg.
Sensors: CSMIP 14323/14395

Lat: 33.755°
Long.: 118.200°

Floor Levels: 7+penthouse
Bsmt. Levels: 0
Structure: Moment-resisting steel frames

Foundation: concrete piles
Site Geology: soil
Design Date: 1967

Recorded Peak Accelerations (g)

	Free-Field			Ground Floor			Roof		
Earthquake	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	0.049	0.070	0.024	0.039	0.073	0.024	0.12	0.11	--
1994 Northridge*	0.06	0.06	0.03	0.06	0.07	0.02	0.09	0.13	--

* Not processed -- = no instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.5\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized

Final: FF/str. data synchronized using cross-correlation of vertical accelerations (Fig.

A18.WT0); data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Figures

A18.1-A18.3

Site Plan, Sensor Locations, Generalized Soil Column

A18.WT0-6

WT Analysis Results



View of structure, looking south-west

DISCUSSION

Site Description

The building is 7 stories tall above ground level with a single level penthouse and no basement. The building is rectangular in plan view with dimensions 220 by 75 feet. As shown in Fig. A18.2, the building consists of two adjacent structures separated by a 4.5-inch wide seismic joint which crosses the building in the transverse direction about 99 feet from its south end. No other structures are present near the building. The lateral load resisting system consists of moment-resisting steel frames in both directions. Each column is founded on groups of 4 to 7 reinforced concrete piles which extend to 36 to 42-foot depth. At the pile heads are 3-foot thick pile caps, and an 8-inch thick slab-on-grade is present across the ground floor level. As shown in Fig. A18.2, accelerographs are present in both the north and south structures at the ground, 2nd, and roof levels. At the time of the Whittier and Northridge Earthquakes, the free-field instrument was located about 198 feet southwest of the structure in a parking lot.

The site is located on level reclaimed land within the Port of Long Beach. Geotechnical data for the area comes from a variety of sources including up to 100-foot deep borings by Dames and Moore (1957) at the building site, up to 80-foot deep borings by L.T. Evans (1970) 500 to 1000 feet east of the site, and shear wave velocity measurements to a depth of 100 feet in similar geologic materials located in the general vicinity of the Port of Long Beach (Fugro Inc., 1977; Gibbs et al., 1980). As shown in Fig. A18.3 soil conditions consist of 55 feet of sandy and silty fill soils underlain by natural silt and sand deposits which extend to the maximum depth explored of 100 feet. The depth to bedrock is poorly characterized in the area. The shear wave velocities shown in Fig. A18.3 were estimated using in situ downhole measurements at Pier J in the port (Fugro, Inc. 1977) and at the intersection of Magnolia and Broadway in downtown Long Beach (Gibbs et al., 1980).

Instrument Selection and Free-Field Conditions

Instrumentation in the south structure is located near its centroid in plan view, while the north structure is instrumented at the north end. The sensors in the south structure were selected for most of the analyses, although nonparametric analyses were performed using data from the north structure as well. A complete list of the sensors used in the analyses is given in Table A18.1; acceleration time histories recorded by these sensors are shown in Fig. A18.WT1.

Table A18.1: Sensors in south structure used for analyses
(North structure sensors are shown in parenthesis)

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 15 (Ch 18)	Ch 13 (Ch 16)	Ch 14 (Ch 17)	Ch 1 (Ch 3)	Ch 5 (Ch 8)

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A18.WT4) and the spectra for horizontal motions (Fig. A18.WT2-3) generally indicate increasing de-amplification of GF motions with frequency.

Transmissibility, coherency, and power spectral density functions for FF and GF vertical motions are presented in Fig. A18.WT5. Amplification occurred from about 1 to 3 Hz, but not at the structure's transverse fundamental frequency (≈ 0.7 Hz). Definitive conclusions about rocking can not be derived from these data since the vertical GF sensor is located near the center of the building.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A18.WT4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A18.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A18.WT6).

Table A18.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) d=5, j=7		L (NS) d=3, j=5		tr (EW) d=3, j=7		L (NS) d=2, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	na	na	--	--	0.73	4.9	0.89	6.4	0.75	7.1	0.89	6.8
	2					2.23	7.5	2.93	9.4	2.13	9.0	2.82	10.9
	3					3.76	13.7	5.05	7.4	3.51	13.3	4.79	8.3

Comments on the analytical results follow:

- The transverse structural response has significant modes at $f \approx 0.7$ and 2.1 Hz, though the first mode has more energy. The longitudinal response is dominated by the first mode.
- Based on the nonparametric analyses, the seismic responses of the north and south structures were similar (especially in the longitudinal direction)
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. Given the high aspect ratio of this structure ($h/r_2 \approx 1.3$), rocking in the transverse direction may be significant (despite the fact that no such effect was evident in the vertical GF/FF transmissibility function), so transverse fixed-base parameters were not estimated by pseudo flexible-base parameters. In the longitudinal direction, rocking is not likely to be significant, so fixed-base parameters can be approximated by pseudo flexible-base parameters.

The RPEM results (Fig. A18.WT6) do not indicate any significant time-dependent variations in the first-mode properties of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitudes of FF-R and GF-R modal parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A18.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 49 feet using the recorded free-field motions and the profile in Fig. A18.3. The south structure was considered for this analysis, and was assumed to be independent of the north structure for the calculation of foundation radii. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A18.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 700$ fps	$h = 58$ ft	emb.=0	$\gamma = 0.15$
WT	$\nu = 0.33$		tr: $r_1 = 49$ ft		$r_2 = 46$ ft
	$\beta^* = 3.5\%$	$V_s^* = 615$ fps	L: $r_1 = 49$ ft		$r_2 = 53$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A18.3 with the fixed-base modal parameters in Table A18.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A18.4.

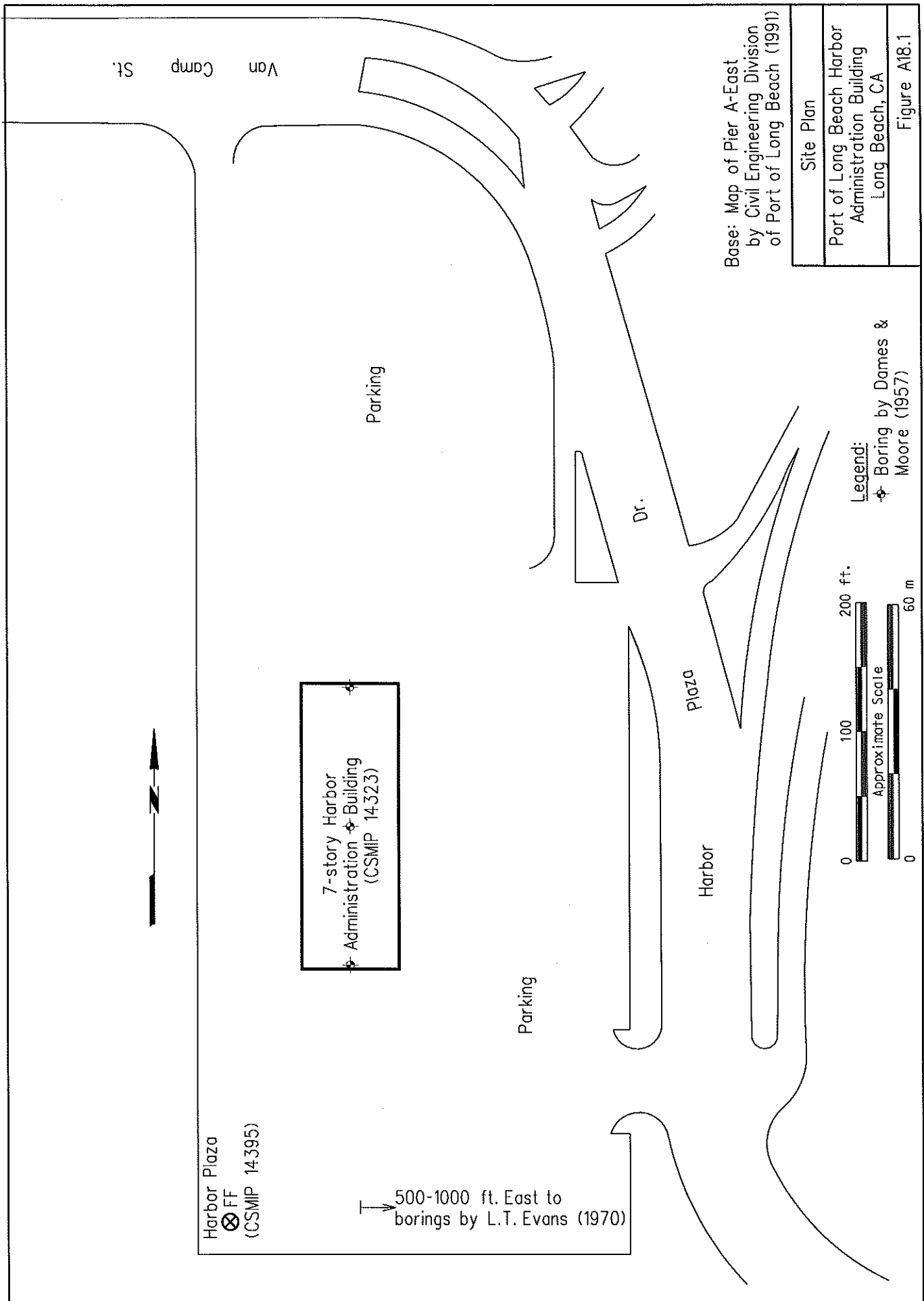
Table A18.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
WT	Sys. ID	--	--	--	1.00	0.4	6.8
	Prediction	--	--	--	1.03	0.4	6.3

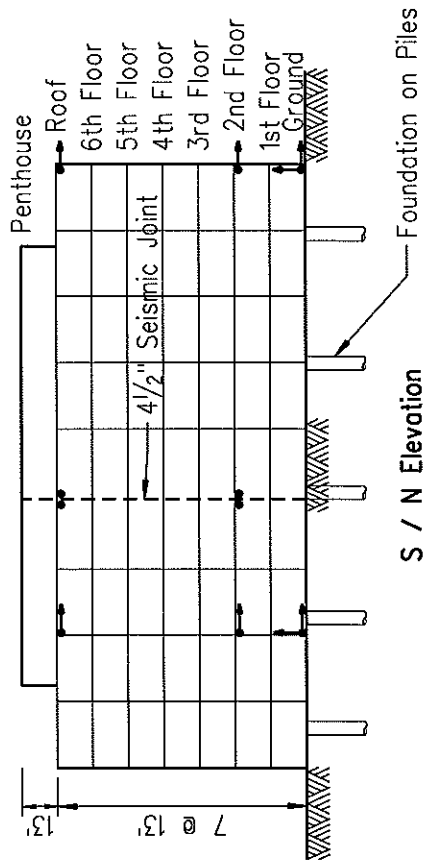
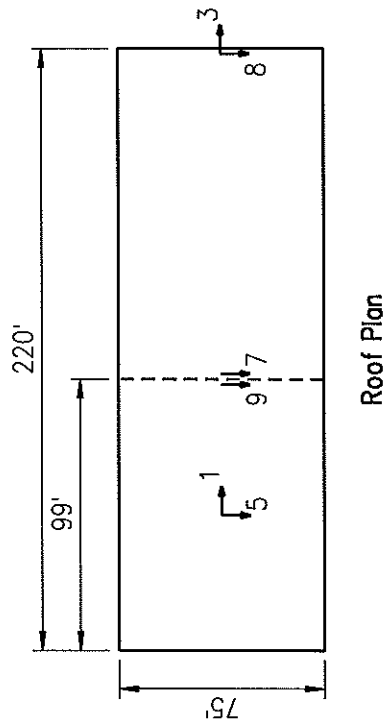
Fixed-base parameters could not be estimated for the transverse direction, so no comparison of SSI effects was possible. For the longitudinal direction, the small inertial interaction effects at this site were adequately predicted.

Data Sources

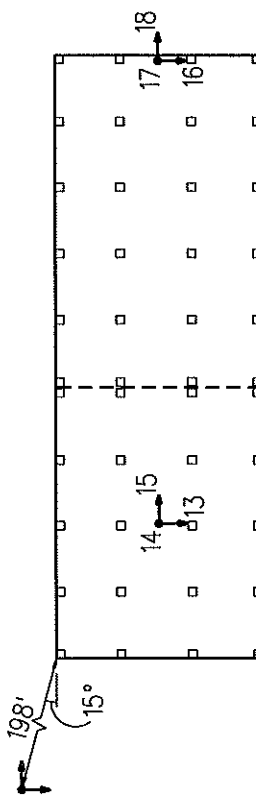
- Dames and Moore (1957). "Report of Foundation Investigation, Proposed Port of Long Beach Administration Building, End of Pier A, Long Beach Harbor, Long Beach, CA."
- L.T. Evans (1970). borings for Sea Land project at the corner of Harbor Plaza and Panorama Drive, Long Beach, CA, data provided by staff of the Port of Long Beach.
- Fugro, Inc. (1977). "Appendix B, Drilling Sampling and Downhole Logging, Sohio Oil Terminal and Pier J Expansion," *Project No. 76-120-E*.
- Gibbs, J.F., Fumal, T.E., and Roth, E.F. (1980). "In-situ measurements of seismic velocity at 27 locations in the Los Angeles, California region," *Open File Rpt. 80-378*, U.S. Geological Survey.



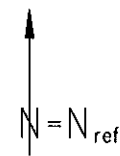
lbport.dgn[100]



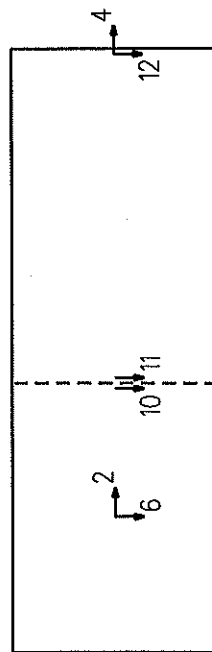
Harbor Plaza
Free Field



Steel moment frames.
Design date: 1967



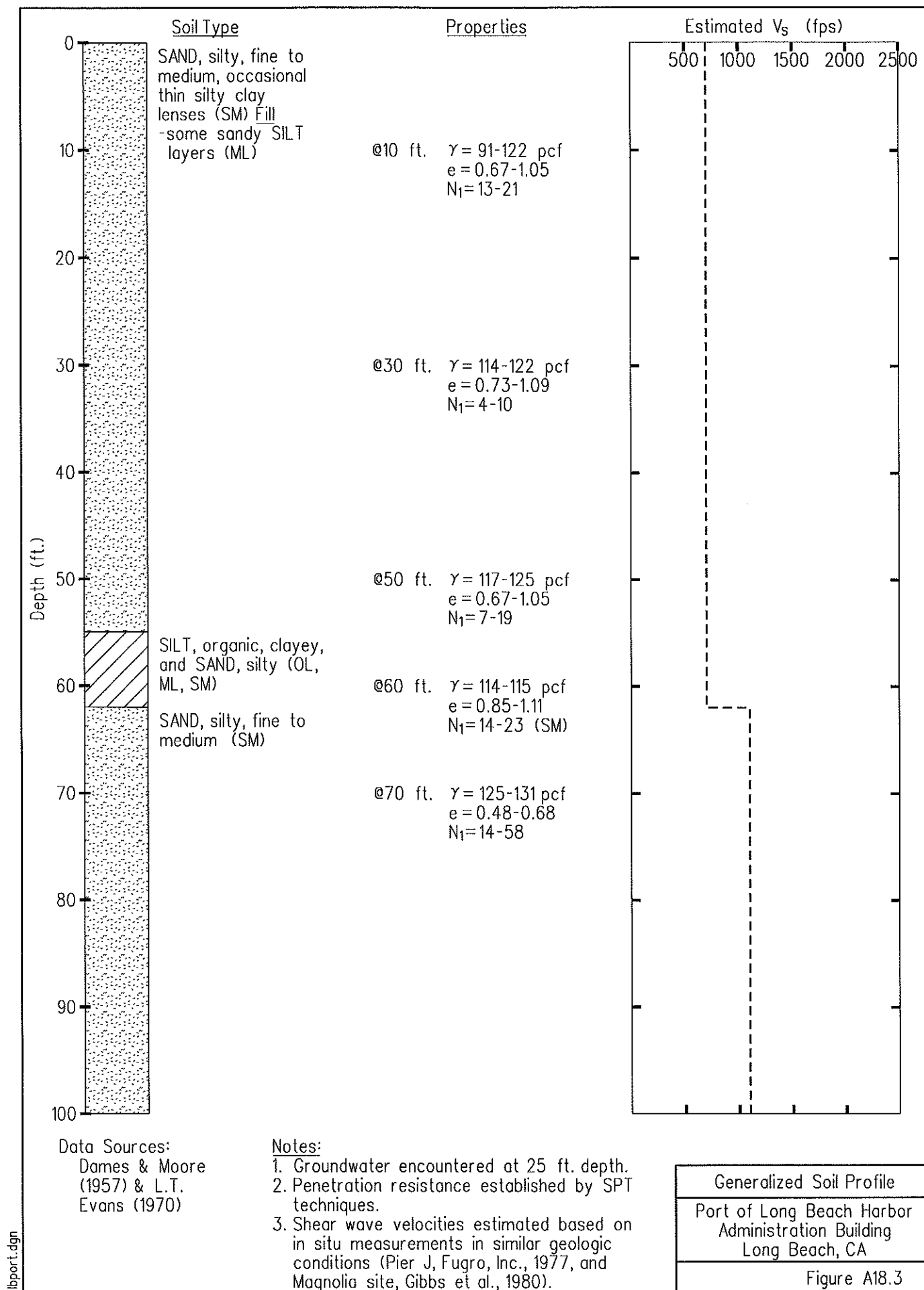
2nd Floor Plan



Sensor Locations
Port of Long Beach Harbor Administration Building Long Beach, CA
Figure A18.2

Note: Data provided by CSMIP

lbport.dgn(70)



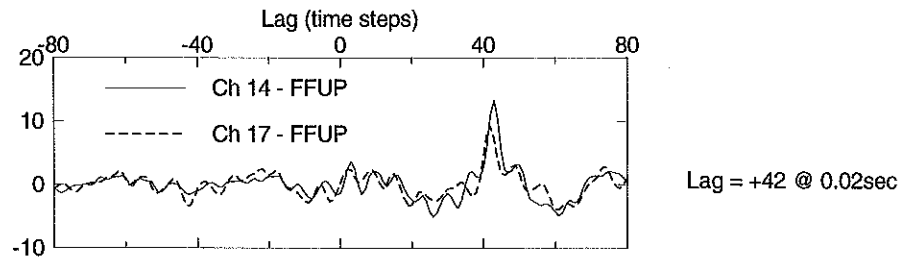


Fig. A18.WT0: Cross-correlation function between vertical ground floor and free-field accelerations

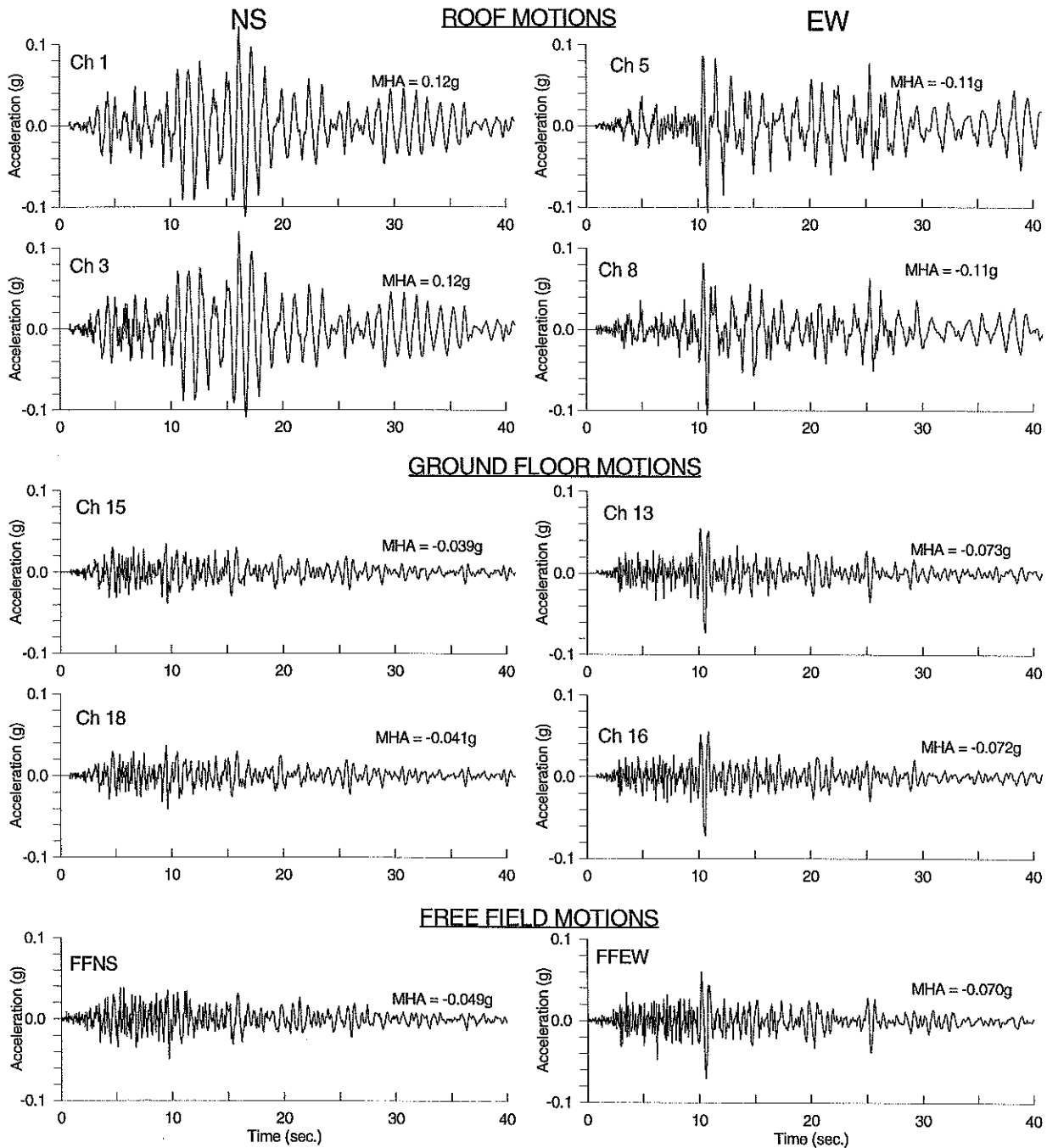


Fig. A18.WT1: LONG BEACH HARBOR ADMINISTRATION BUILDING
Horizontal Acceleration Time Histories
1987 Whittier Earthquake

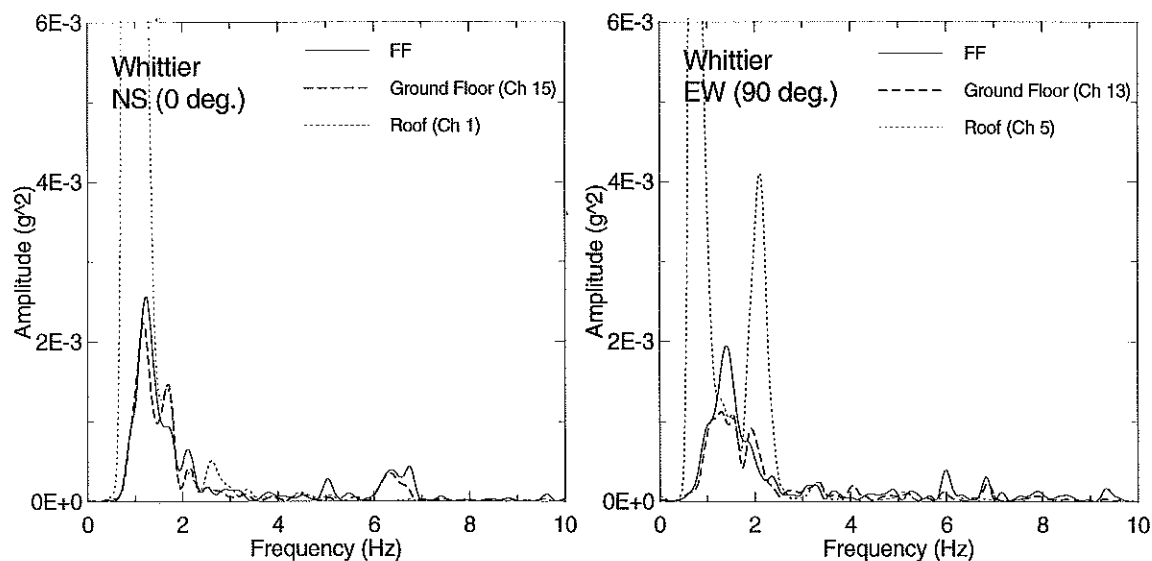


Fig. A18.WT2: LONG BEACH HARBOR ADMINISTRATION BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

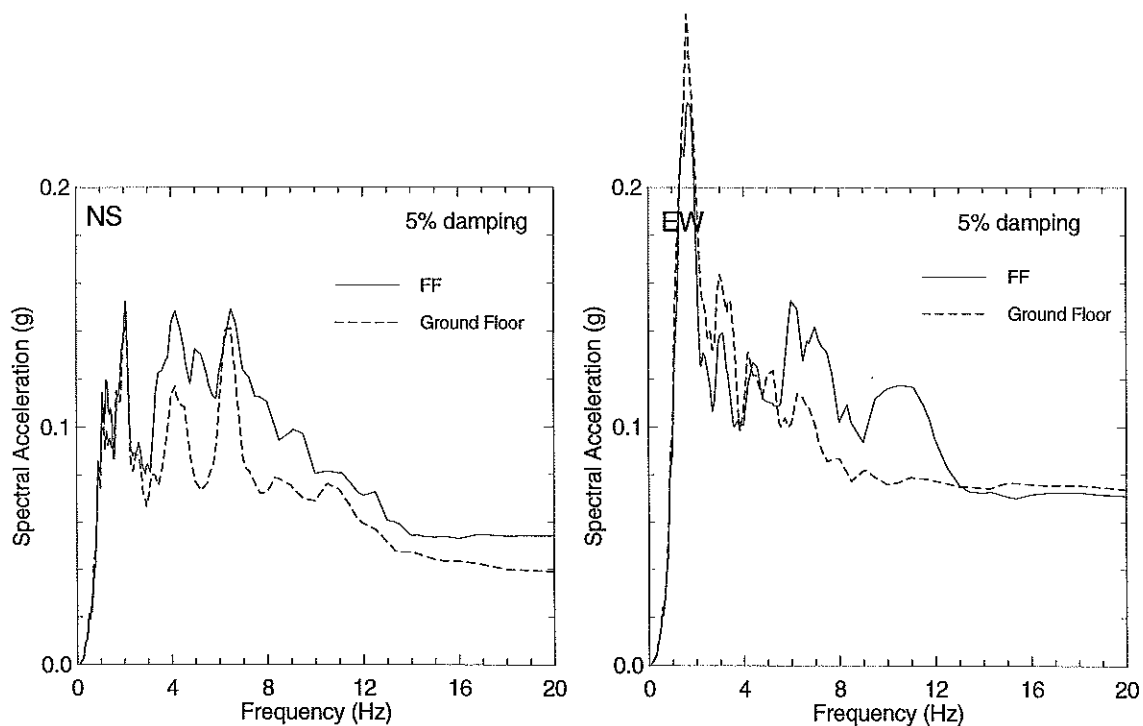


Fig. A18.WT3: LONG BEACH HARBOR ADMINISTRATION BUILDING
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1987 Whittier Earthquake

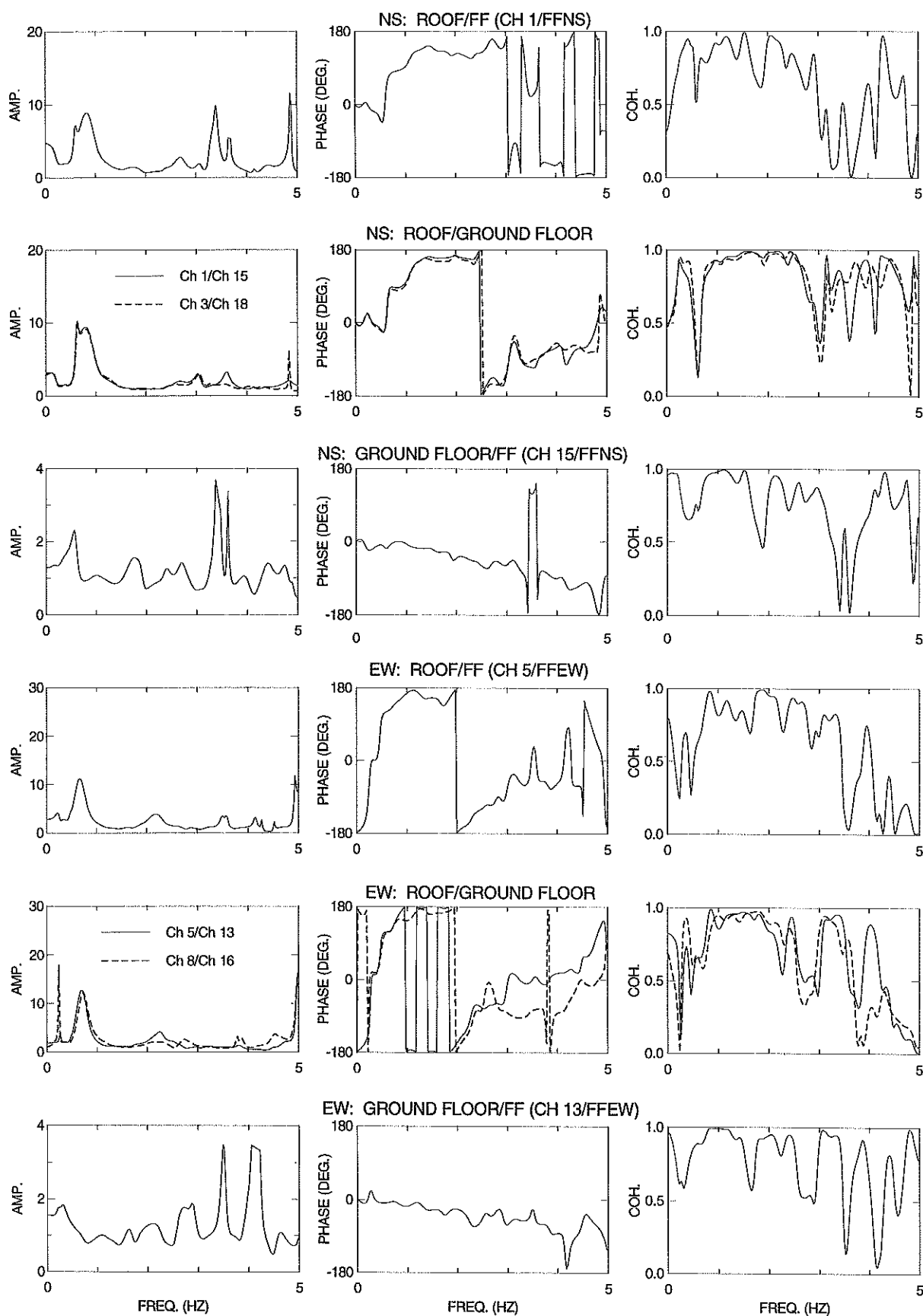


Fig. A18.WT4: LONG BEACH HARBOR ADMINISTRATION BUILDING
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A18-10

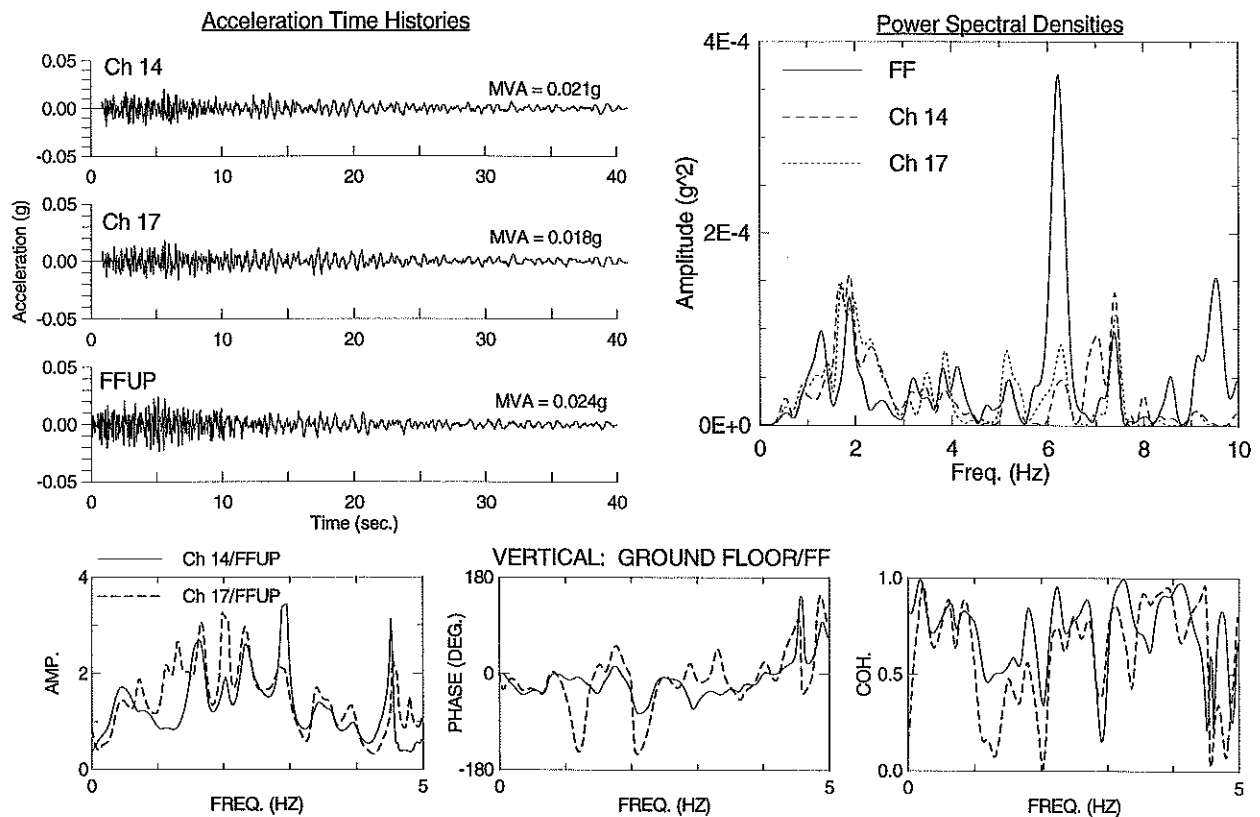


Fig. A18.WT5: LONG BEACH HARBOR ADMINISTRATION BUILDING
Analysis of Vertical Motions
1987 Whittier Earthquake

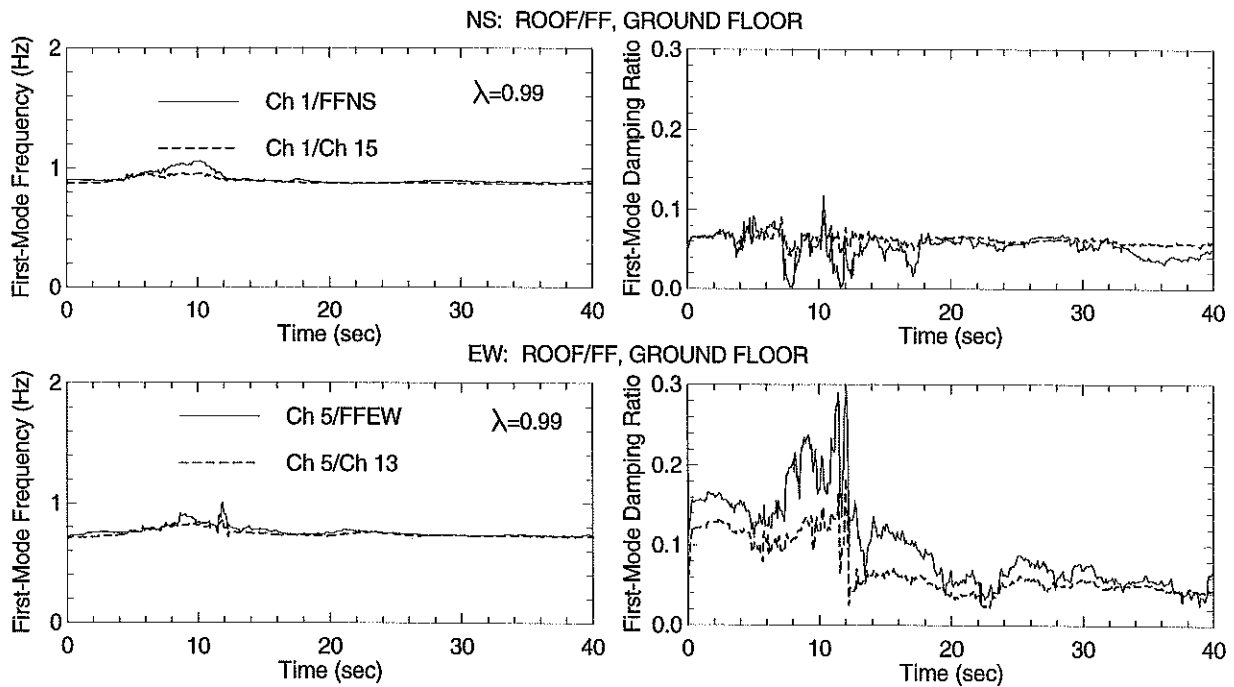


Fig. A18.WT6: LONG BEACH HARBOR ADMINISTRATION BUILDING
Time Variation of First-Mode Parameters
1987 Whittier Earthquake

SITE A20

Site Name: Long Beach VA Hospital
Sensors: USGS 5106

Lat: 33.778°
Long.: 118.118°

Floor Levels: 12
Bsmt. Levels: 1
Structure: Concrete shear walls

Foundation: footings
Site Geology: soil
Design Date: 1964

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers*	0.03	0.03	0.02	0.03	0.03	0.02	0.12	0.12	0.03
1994 Northridge	0.063	0.067	0.033	0.058	0.048	0.036	0.20	0.21	0.082

* Not processed

Preprocessing

Northridge Earthquake (NR)

Preliminary: by USGS; filtered, $f = 0.17\text{--}50\text{ Hz}$; $f_{Nyq}=100\text{ Hz}$; FF/str. data not synchronized

Final: decimated by 8 to $f_{Nyq}=12.5\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A20.NR0).

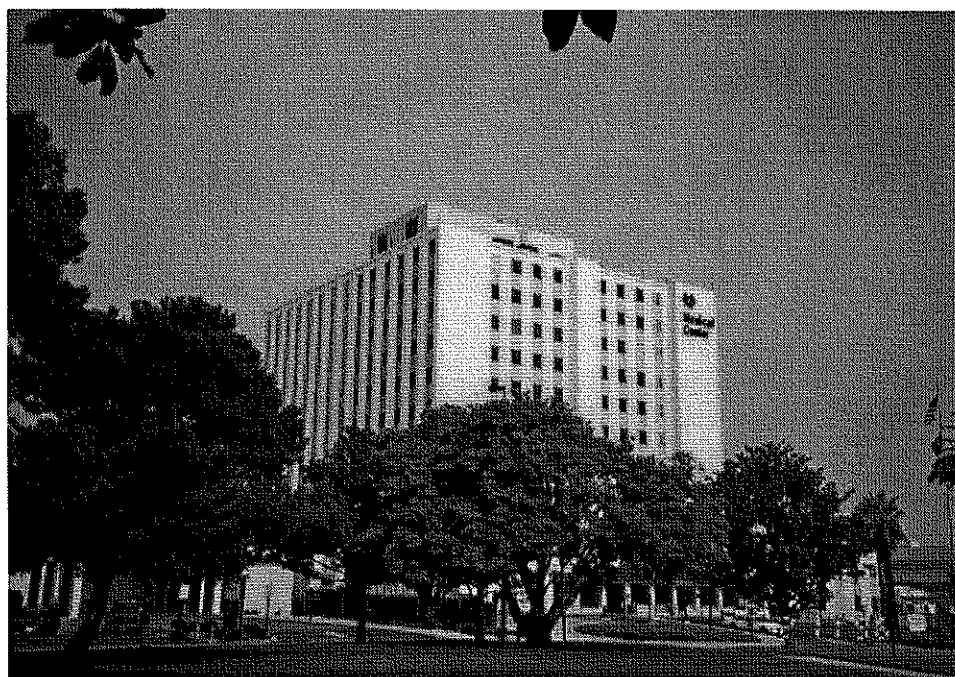
Figures

A20.1-A20.3

Site Plan, Sensor Locations, Generalized Soil Column

A20.NR0-6

NR Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The building is 11 stories tall above ground level and has a full single-level basement and a partial second basement level. The structure is rectangular in plan view, with base dimensions of about 197 by 270 feet and tower dimensions (above the 3rd level) of about 141 by 156 feet. As shown in Fig. A20.1, a 2-story building is located immediately west of the structure, and several other relatively short buildings are present in the area. The lateral load resisting system consists of concrete shear walls around the building perimeter as well as several interior walls, while vertical loads are carried by a concrete frame. Following the Northridge Earthquake, the building was retrofitted with base isolators beneath the columns and shear walls. The foundation consists of shallow footings. Accelerographs are present at the basement, 6th, and roof levels of the structure. The free-field instrument is located near a parking lot, about 980 feet east of the tower.

The site is located on level terrain within the Los Angeles Basin. Soil conditions have been established to a depth of 150 feet by Gibbs et al. (1980) and LCA (1986). As shown in Fig. A20.3, the soil profile consist of interbedded layers of clay/silt and sand which extend to the maximum depth explored at the site of 150 feet. Based on oil well logs near the site (USGS, 1995), shale bedrock appears to occur at a depth of about 1600 feet. The shear wave velocities shown in Fig. A20.3 are based on in situ downhole measurements (Gibbs et al., 1980).

Instrument Selection and Free-Field Conditions

The only instrumentation in the building at the time of the Northridge event were three triaxial accelerographs, one each at the basement, 6th, and roof levels (Table A20.1). Acceleration time histories recorded by the available sensors are shown in Fig. A20.NR1.

Table A20.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 2	Ch 3	Ch 1	Ch 8	Ch 9

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A20.NR4) and the spectra for horizontal motions (Fig. A20.NR2-3) indicate that amplification of basement motions relative to the free-field occurred in both directions near the first-mode response frequencies. Basement motions were generally de-amplified at higher frequencies.

Transmissibility, coherency, and power spectral density functions for the FF and B vertical motions are presented in Fig. A20.NR5. Significant amplification was observed near the first-mode frequencies, indicating that rocking likely contributed to the first-mode structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A20.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A20.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A20.NR6). For comparison, frequencies identified from forced vibration tests are presented in Table A20.3.

Table A20.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) d=3, j=5		L (NS) d=2, j=5		tr (EW) d=3, j=5		L (NS) d=2, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.96	4.6	1.83	3.9	1.82	3.7	1.77	3.6	1.73	3.1	1.72	3.8
	2					3.32	14.2	3.79	27.3	3.25	19.1	3.43	22.2
	3					5.41	6.5	5.42	8.0	5.08	8.0	5.42	7.9

Table A20.3: First-mode frequencies (in Hz) identified from forced vibration tests (Etheredge, 1972)

Mode	Forced Vibration	
	NS	EW
1	1.9-2.2 (avg. 2.1)	1.9-2.4 (avg. 2.2)

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The modest difference between the fixed- and pseudo flexible-base parameters is consistent with the rocking effect evident in the vertical B/FF transmissibility function.
- In the NS direction, transmissibility function amplitudes for R/B and R/FF pairs have a sharp peak at about 1.5 Hz (near the low end of the broader first-mode peak). Based on the power spectral densities (Fig. A20.NR2), these peaks are associated with the absence of energy at this frequency in the B and FF motions, and are not related to a distinct vibration mode.

The RPEM results (Fig. A20.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitudes of FF-R and B-R modal parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A20.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 84 feet using the recorded free-field motions and the profile in Fig. A20.3. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A20.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1210$ fps	$h = 98$ ft	emb. = 23 ft	$\gamma = 0.15$
NR	$\nu = 0.33$		tr: $r_1 = 84$ ft		$r_2 = 83$ ft
	$\beta^* = 2.9\%$	$V_s^* = 1140$ fps	L: $r_1 = 84$ ft		$r_2 = 87$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A20.4 with the fixed-base modal parameters in Table A20.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A20.5.

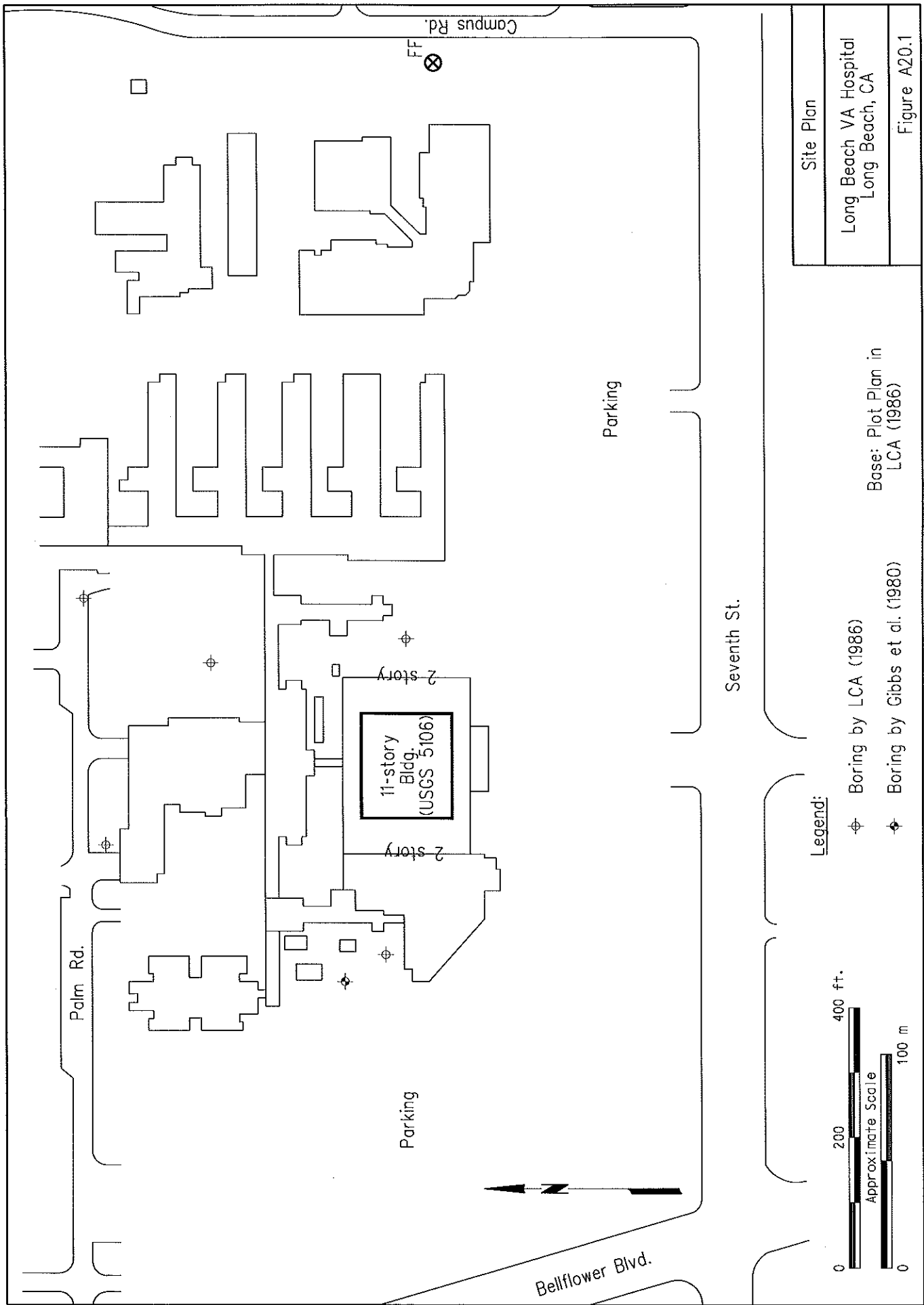
Table A20.5: Empirical and predicted inertial interaction effects

NR	Sys. ID	tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.13	-0.1	3.1	1.06	0.5	3.8
	Pre.-Vel.	1.09	1.6	5.2	1.08	1.5	4.6
	Pre.-Bie.	1.12	4.4	7.8	1.10	3.9	6.8

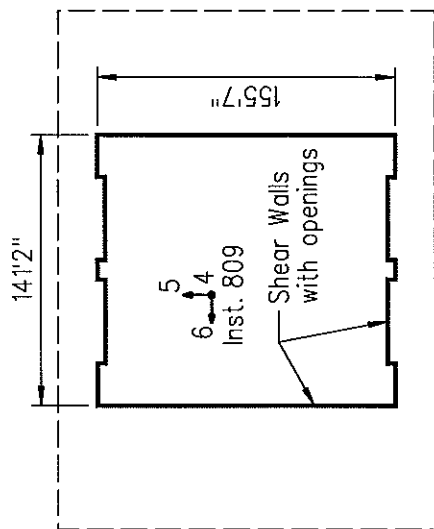
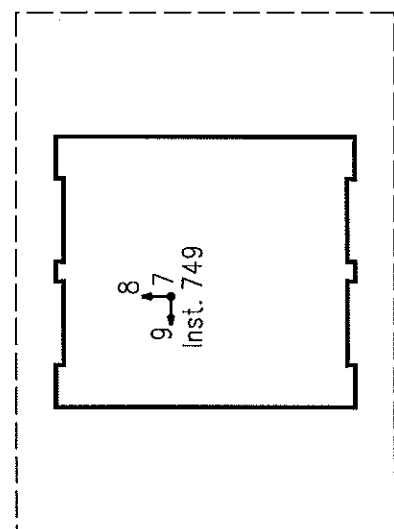
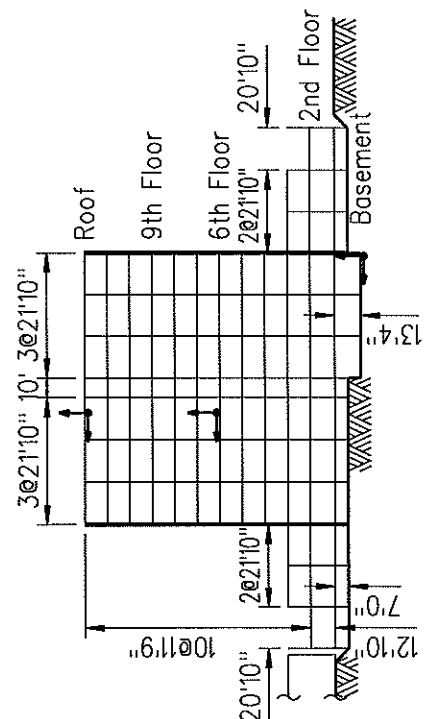
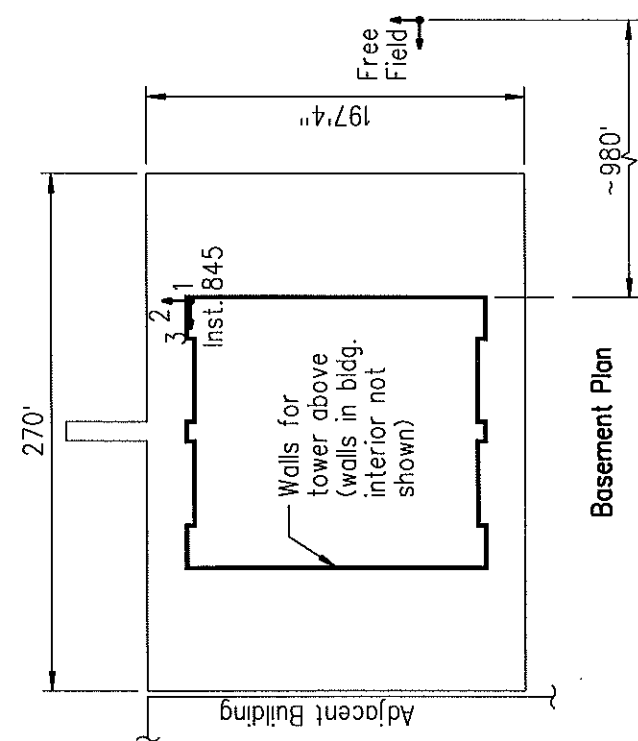
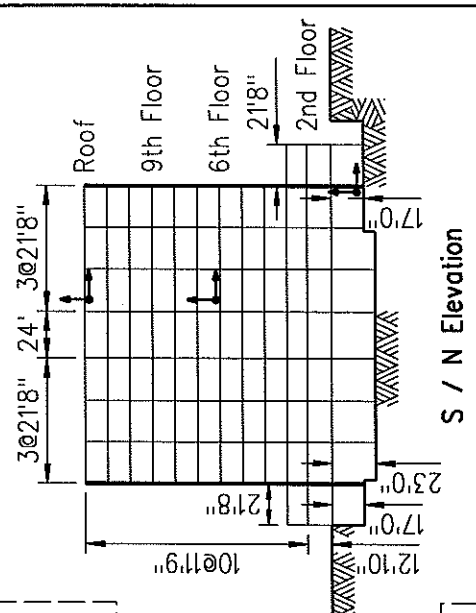
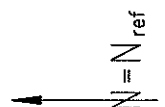
The period lengthening is satisfactorily predicted by both formulations. Foundation damping is significantly overestimated by both formulations, but the “modified Veletsos” results are superior. The negative soil damping $\tilde{\zeta}_0$ from the system identification analyses for the transverse direction indicates a numerical error in these results.

Data Sources

- Etheredge, E. (1972). “Forced vibration test results, Veterans Administration Hospital, Building 126, Long Beach, CA,” unpublished data from USGS files.
- Gibbs, J.F., Fumal, T.E., and Roth, E.F. (1980). “In-situ measurements of seismic velocity at 27 locations in the Los Angeles, California region,” *Open File Rpt. 80-378*, U.S. Geological Survey.
- LeRoy Crandall Assoc., LCA (1986). “Report of Geotechnical Investigation, Phase II Seismic Evaluation, Veterans Administration Medical Center, Long Beach, CA,” *Job. No. AD-86152*.
- U.S. Geological Survey (1995). Personal communication with Walt Jungblut.

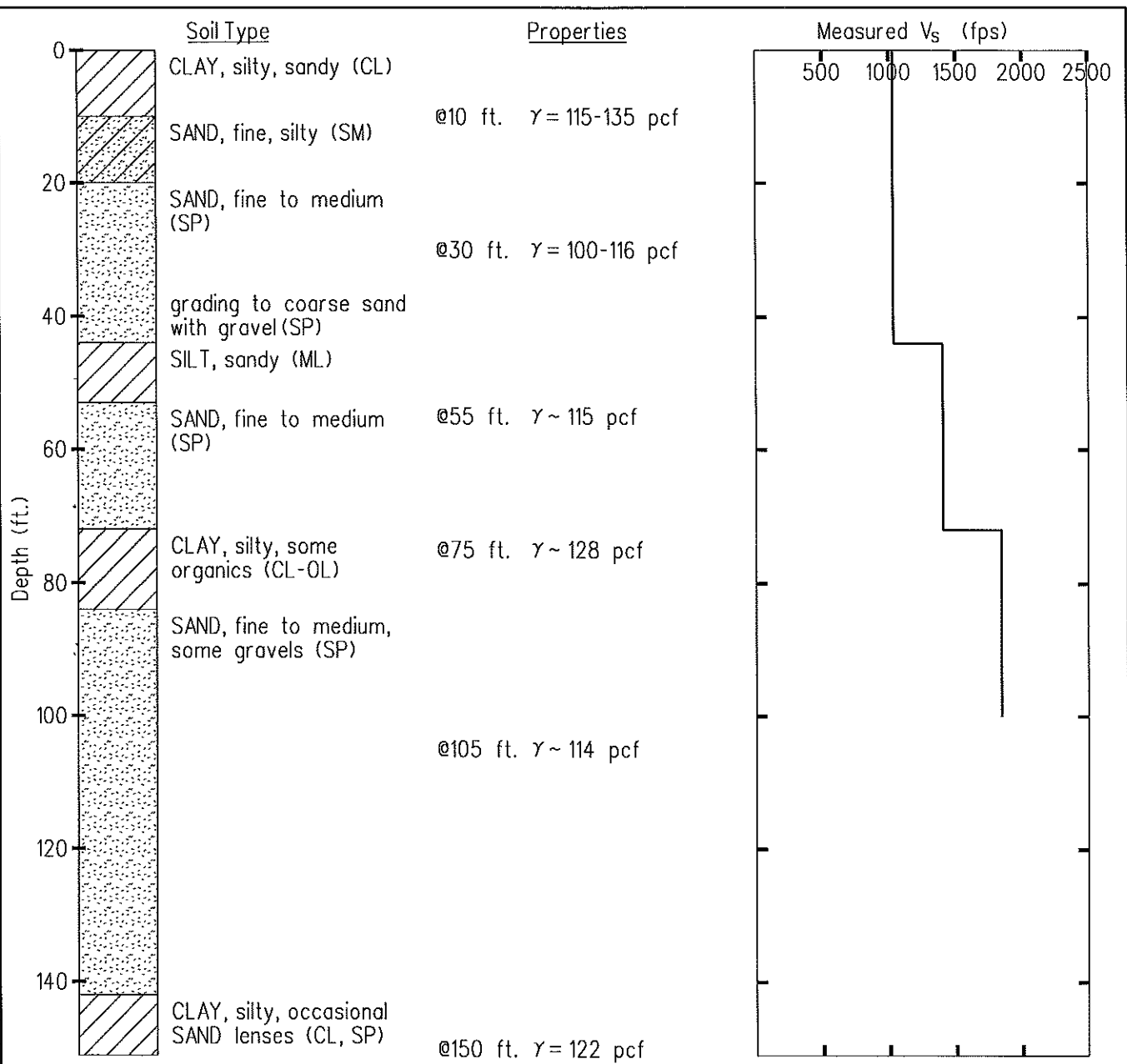


lbva.dgn



Note: Data provided by Long Beach VA Engineering Staff

Sensor Locations
Long Beach VA Hospital Long Beach, CA
Figure A20.2



Data Source:
LCA (1986) and
Gibbs et al. (1980)

Notes:

1. No groundwater encountered.
2. Shear wave velocities based on downhole measurements by Gibbs et al. (1980).

Generalized Soil Profile

Long Beach VA Hospital
Long Beach, CA

Figure A20.3

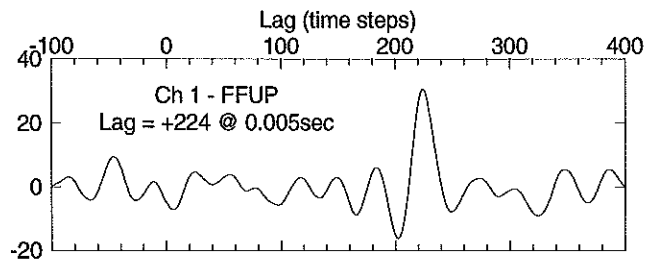


Fig. A20.NR0: Cross-correlation function between vertical basement and free-field accelerations

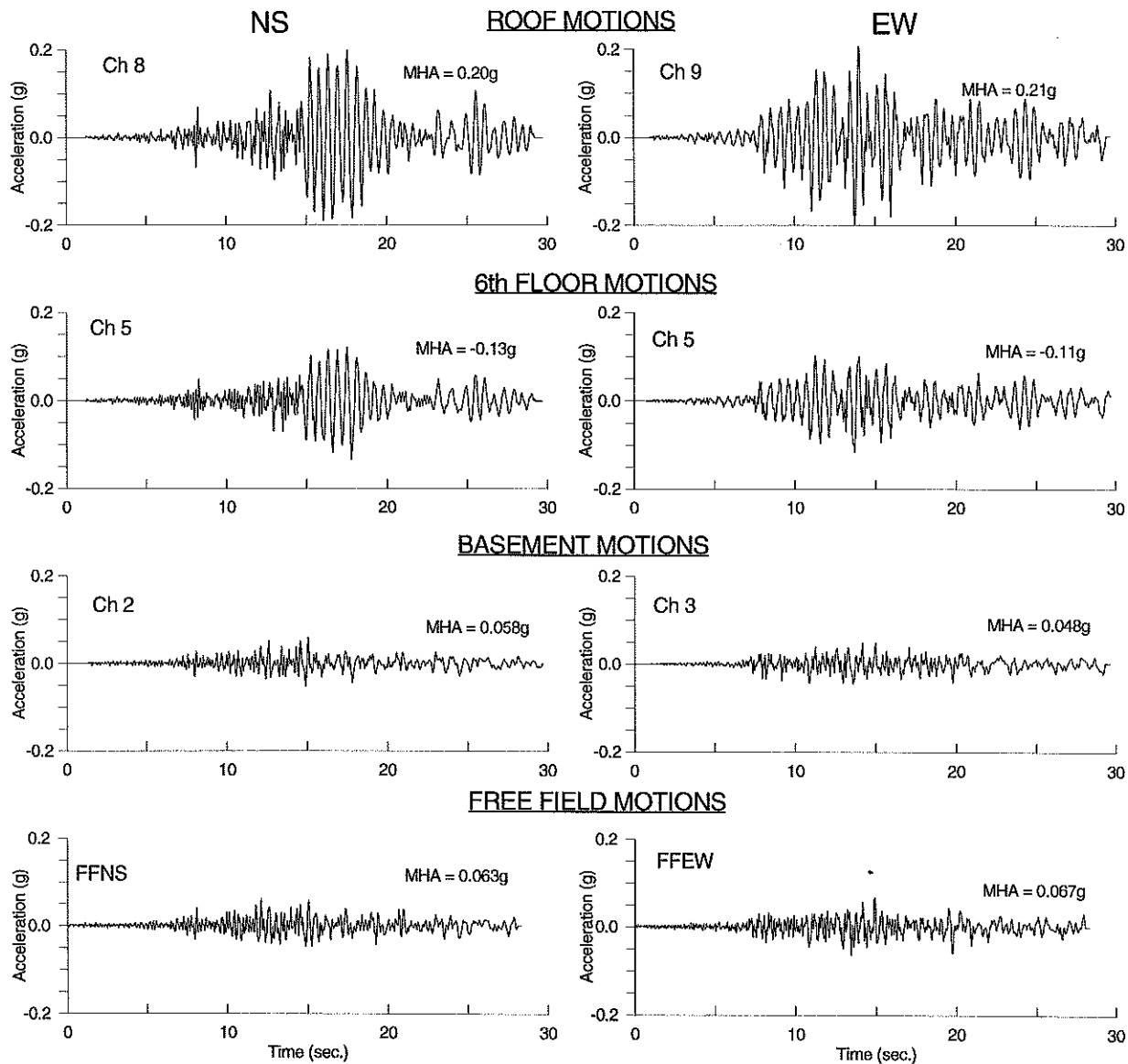


Fig. A20.NR1: LONG BEACH VA HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

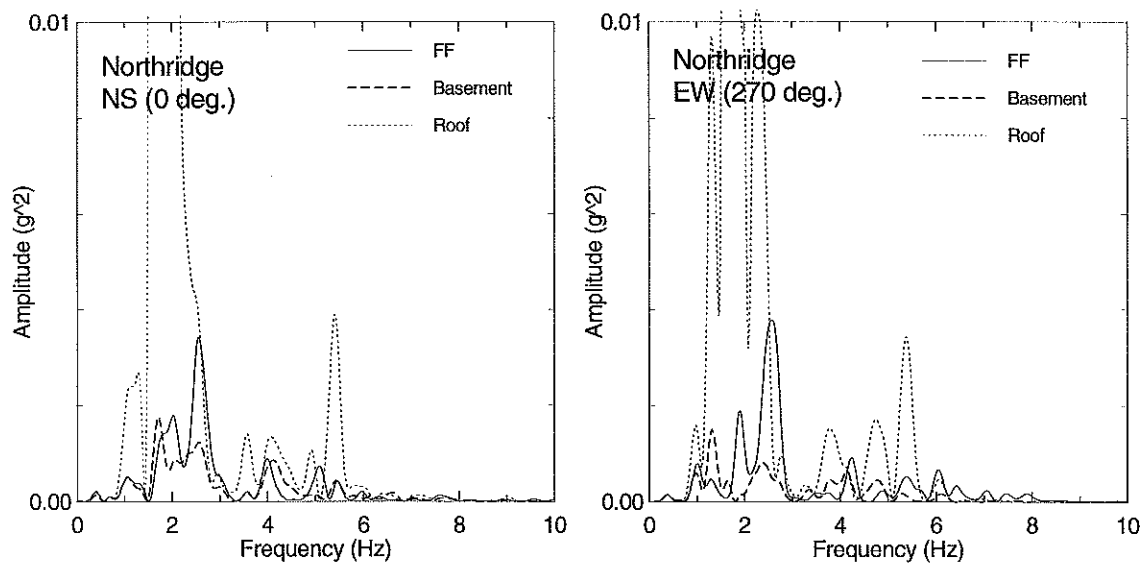


Fig. A20.NR2: LONG BEACH VA HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

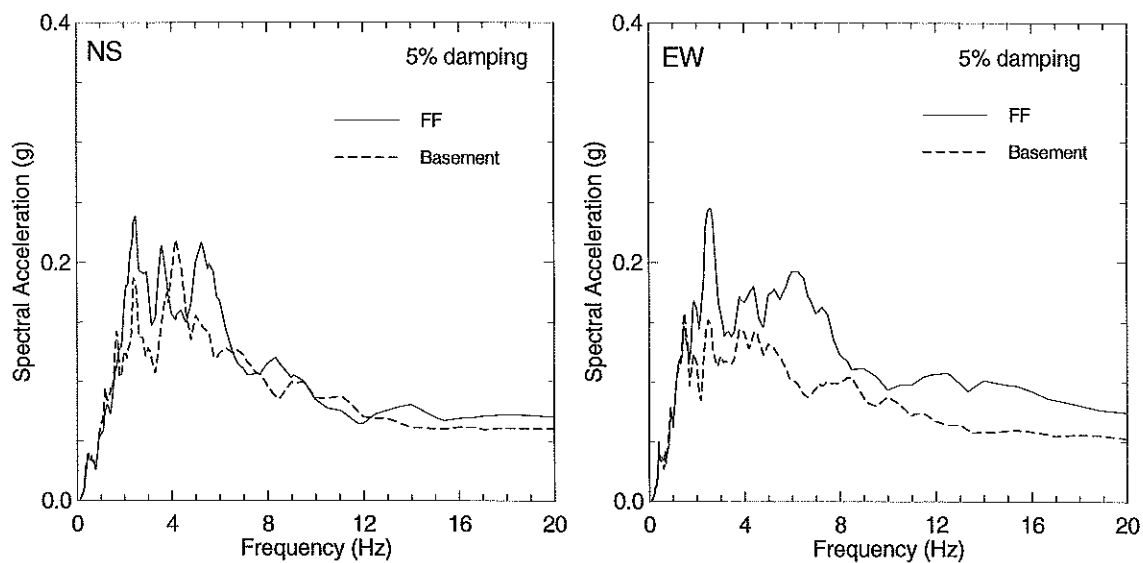


Fig. A20.NR3: LONG BEACH VA HOSPITAL
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

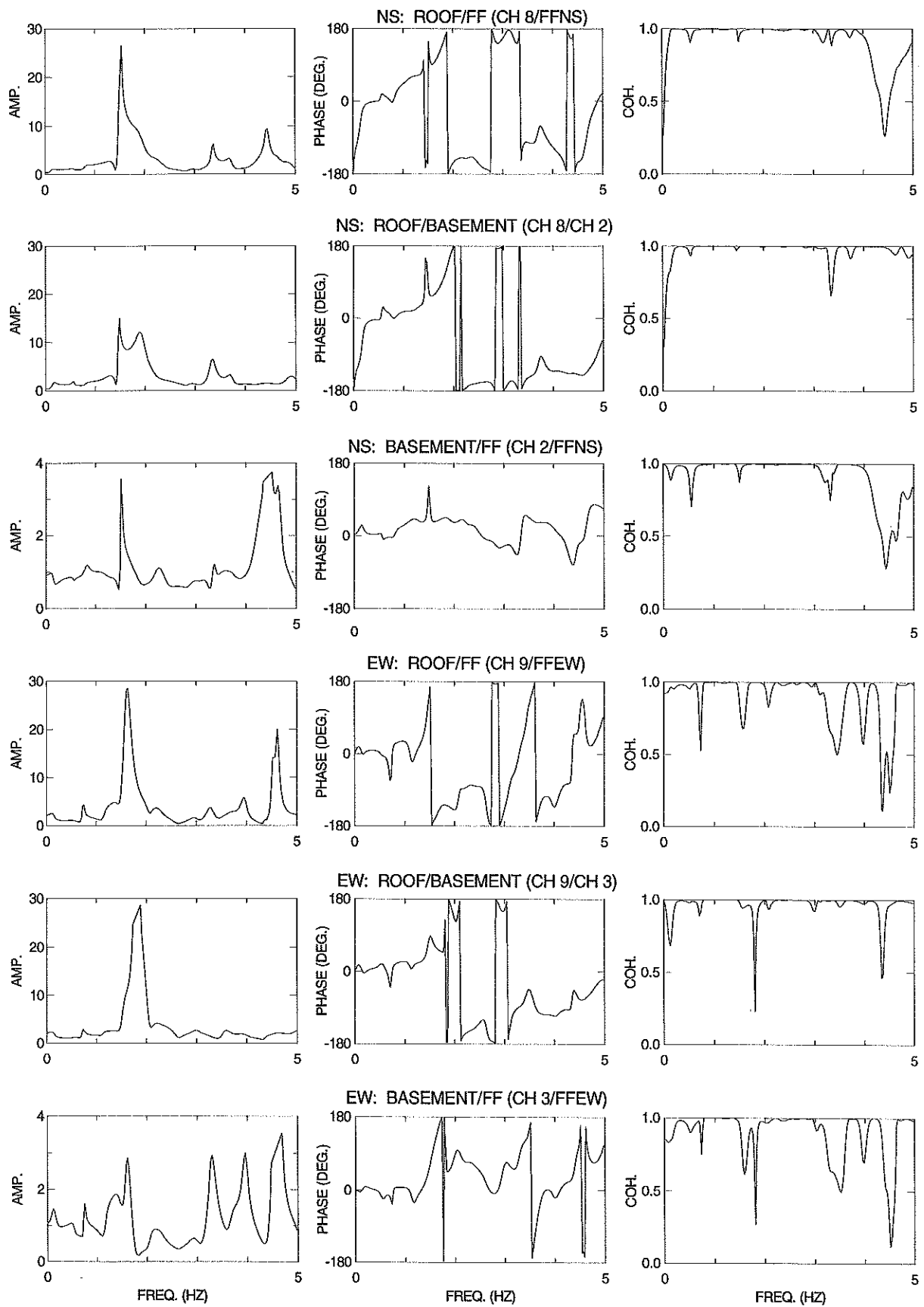


Fig. A20.NR4: LONG BEACH VA HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A20-10

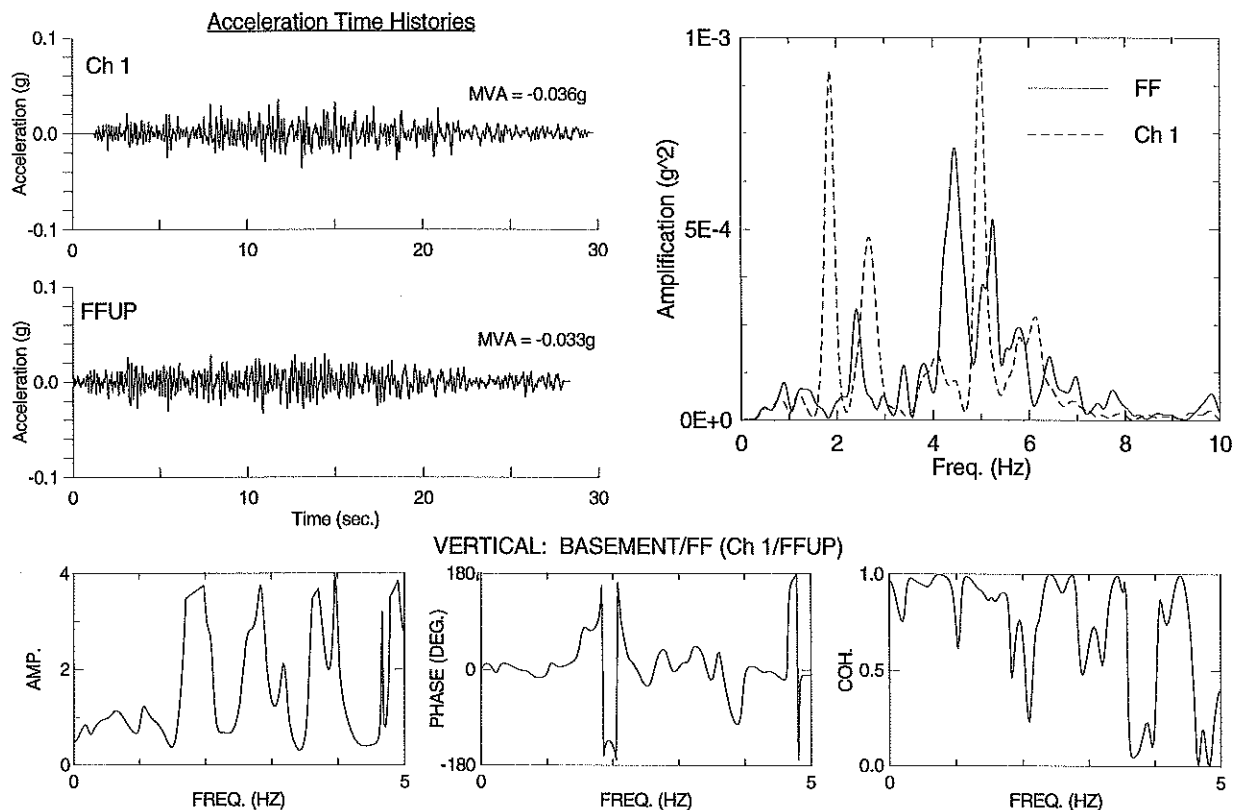


Fig. A20.NR5: LONG BEACH VA HOSPITAL
Analysis of Vertical Motions
1994 Northridge Earthquake

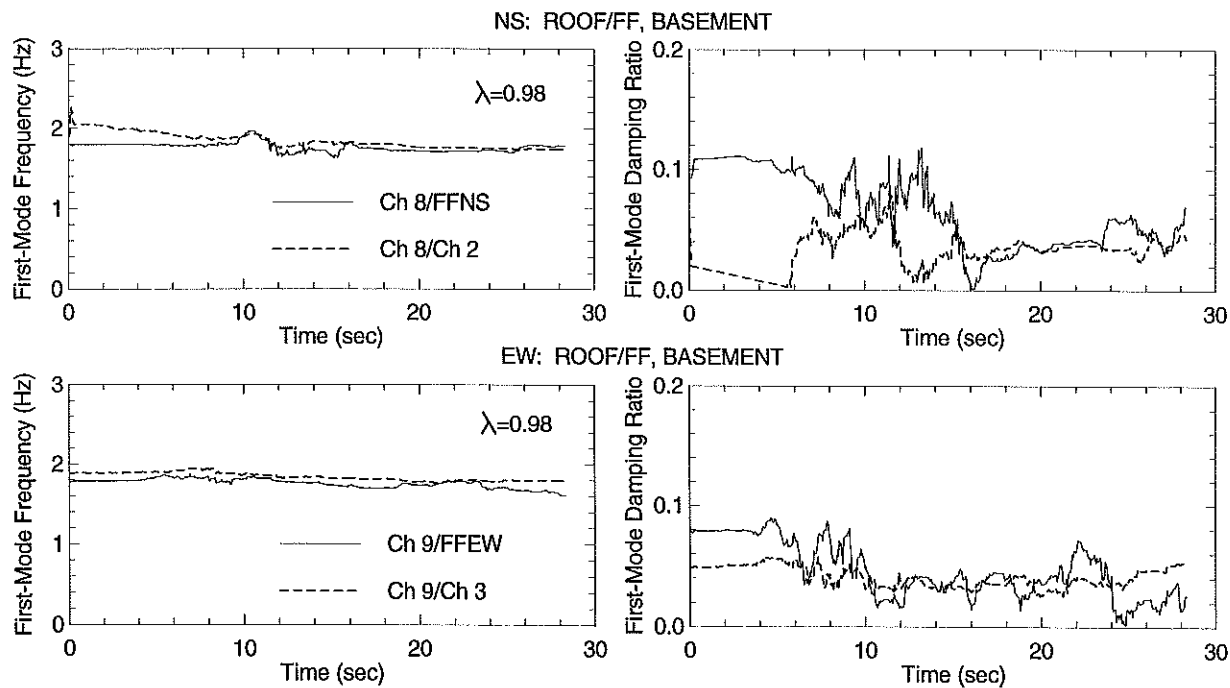


Fig. A20.NR6: LONG BEACH VA HOSPITAL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A21

Site Name: LA 2 Story Fire Command and
Control Facility
Sensors: CSMIP 24580/24592

Lat: 34.053°
Long.: 118.171°

Floor Levels: 2
Bsmt. Levels: 0
Structure: Isolators, braced steel frames

Foundation: footings
Site Geology: rock
Design Date: 1988

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Foundation Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1991 Sierra Madre	0.089	0.11	0.056	0.075	0.076	0.046	0.059	0.079	--
1992 Landers	0.050	0.048	0.032	0.052	0.051	0.024	0.059	0.085	--
1992 Big Bear*	0.06	0.04	0.03	0.05	0.04	0.03	0.05	0.04	--
1994 Northridge	0.22	0.32	0.13	0.17	0.22	0.11	0.091	0.23	--

*Not digitized

-- = No instrument

Preprocessing

EACH EVENT: FF data rotated 140° CCW to align with str.; where necessary, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A21.SM0,LD0,NR0).

Sierra Madre Earthquake (SM)

Prelim.: CSMIP; filtered $f = 0.2\text{--}23\text{ Hz}$ (str.), $0.2\text{--}46\text{ Hz}$ (FF); $f_{Nyq} = 25\text{ Hz}$, 50 Hz (str., FF).

Final: FF data decimated by 4, str. data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Landers Earthquake (LD)

Prelim.: CSMIP; filtered, $f = 0.16\text{--}23\text{ Hz}$ (str.), $0.16\text{--}46\text{ Hz}$ (FF); $f_{Nyq} = 25\text{ Hz}$, 50 Hz (str., FF).

Final: FF data decimated by 4, str. data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Northridge Earthquake (NR)

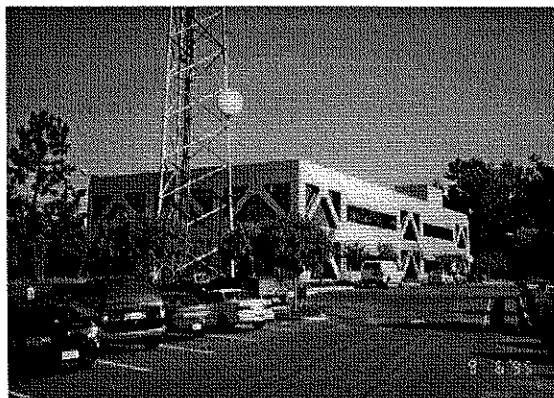
Prelim.: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$ (str. and FF); $f_{Nyq} = 50\text{ Hz}$.

Final: FF/str. data decimated by 2 to $f_{Nyq} = 25\text{ Hz}$.

Figures

A21.1-A21.3	Site Plan, Sensor Locations, Generalized Soil Column
A21.SM0-5	SM Analysis Results
A21.LD0-5	LD Analysis Results
A21.NR0-6	NR Analysis Results

View of structure,
looking north



DISCUSSION

Site Description

The building is 2-stories tall with no basement and has a rectangular shape in plan (253 by 303 feet). No large buildings are located near the structure. The lateral load resisting system consists of perimeter steel inverted chevron braced frames as well as 32 elastomeric bearings to provide base isolation below the 1st floor. Foundations consist of spread footings. As shown in Fig. A21.2, accelerographs are present at the foundation level (below the isolators), 1st floor (above the isolators), 2nd floor and roof. The free-field instrument is located adjacent to a parking lot about 100 feet north of the building.

The site is located on a level building pad established by a cut along the crest of a ridgeline. Approximately 2H:1V slopes are present along the ridge northwest and southeast of the building. The structure is located about 25 feet inward from the top-of-slope, the free-field is within 5 feet of the top-of-slope. Near-surface geologic data is available from a 12-foot boring by Franklin and Associates (1984) and trench logs by WCC (1988). Shear wave velocities were estimated from in situ measurements in similar bedrock materials at Site A25.

Instrument Selection and Free-Field Conditions

Instruments along the south wall of the structure were selected for analysis as this was the only portion of the structure having bi-directional accelerometers at each level of interest (Table A21.1). Acceleration time histories recorded by these sensors are shown in Fig. A21.**1.

Table A21.1: Sensors in structure used for analyses

Foundation Level (FL)			1st Floor (1F)		Roof (R)	
NS	EW	V	NS	EW	NS	EW
Ch 5	Ch 6	Ch 1	Ch 8	Ch 9	Ch 13	Ch 14

The FF instrument is only 100 feet from the building, hence its recordings might be expected to be influenced by structural vibrations. As shown in Figs. A21.**2, significant FF energy content at the first-mode structural response frequency ($\tilde{f}_1 \approx 1.2$ Hz) was not evident from the NR event, but was present in the SM and LD data (NS direction). The lack of FF energy at $\tilde{f}_1$ during the NR event suggests that the FF motions were not significantly contaminated by the structural vibrations. Also of potential concern is the location of the FF instrument and structure near the top of a 2H:1V slope. Ashford and Sitar (1994) have shown that less than 20% amplification generally occurs at the top of slopes with gradients of 2H:1V or less. Hence, such effects are fairly minor, and would have affected free-field and structural strong motion recordings similarly. Hence, it appears unlikely that the validity of the FF data was compromised by structural vibrations or topographic amplification effects.

Analysis Results

(a) Comparison of Foundation Level and Free-Field Motions

FL/FF transmissibility functions (Fig. A21.**4) and the spectra for horizontal motions (Fig. A21.**2-3) for LD and NR data indicate amplification of EW FL motions relative to the FF at about 2 to 2.5 Hz, but de-amplification at other frequencies and in the NS direction.

Transmissibility and coherency functions for FF and FL vertical motions are presented in Fig. A21.**5. No significant amplification occurred near first-mode frequencies during any of the earthquakes, indicating that rocking did not significantly contribute to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A21.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A21.2), and (3) time variation of first-mode parameters from the RPEM, NR only (Fig. A21.NR6). For comparison, first-mode frequencies anticipated in design are shown in Table A21.3.

Table A21.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (FL-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) SM: d=1, j=6 LD: d=0, j=6 NR: d=5, j=6		L (NS) d=1, j=6 d=1, j=6 d=2, j=6		tr (EW) d=1, j=6 d=1, j=6 d=3, j=6		L (NS) d=0, j=6 d=0, j=6 d=0, j=6	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
SM	1	1.27	13.3	--	--	1.27	13.2	1.22	21.3	1.26	9.4	1.25	22.6
LD	1	--	--	1.12	18.8	1.03	14.9	1.12	18.8	1.03	14.2	1.11	18.8
NR	1	--	--	--	--	1.05	40.1	1.17	39.3	1.08	34.4	1.19	33.0

Table A21.3: Estimated fixed-base response frequencies (in Hz) (Bachman et al., 1990)

Mode	With Isolators		Without Isolators	
	EW	NS	EW	NS
1	0.46	0.46	2.5	2.8

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode. The base isolators cause large first-mode damping ratios of 13 to 40% (FL-R case).
- In many cases, fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical FL/FF transmissibility functions, the fixed-base parameters can be approximated by the pseudo

flexible-base parameters. Where fixed-base parameters were estimated, no significant variation relative to pseudo flexible-base parameters was observed.

- The structural response above the isolators was much stiffer and more lightly damped than is reflected by the parameters in Table A21.2 (which include the effects of the isolators). For comparison, first-mode parameters for 1F-R response in the NR event were 4.15 Hz and 9.3% damping (tr) and 4.36 Hz and 10.6% damping (L).
- In the LD and NR events, the transverse response of the structure was affected by impacts between the building and a small access bridge (Bachman, 1994). The impacts caused high-amplitude positive peaks in the acceleration time histories which may have increased high frequency components of structural response. No significant structural damage was reported from these impacts.
- Response frequencies evaluated by the building designer from elastic frame analyses for the "With Isolators" case (Table A21.3) are significantly lower than the frequencies identified from earthquake data (Table A21.2). These analyses were performed using an elastic model of isolator response, with the isolator stiffness being appropriate for higher amplitude shaking (MHA = 0.5g) than occurred in the recorded motions.
- The structure had a stiffer response in SM than LD, despite the higher peak accelerations in SM. This likely resulted from the long duration and larger low-frequency energy content of the LD motions relative to the SM motions.

In the RPEM parametric analyses for NR, significant decreases in first-mode frequency were observed in both directions at about 12 and 16-18 seconds. This softening likely occurred within the base isolation system, which was subject to high amplitude accelerations at these times. As the shaking levels subsided after about 25 seconds, the frequency of the system gradually recovered. Damping was high during the strong shaking (though the results early in the time history are somewhat unstable), and gradually decreased as shaking subsided.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A21.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 71 feet using the recorded free-field motions and the profile in Fig. A21.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by 80% and 70%, respectively, to account for shape effects.

Using the parameters in Table A21.4 with the fixed-base modal parameters in Table A21.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A21.5.

Table A21.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1030$ fps	$h = 22$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 71$ ft		$r_2 = 59$ ft
SM	$\beta^* = 1.3\%$	$V_s^* = 1010$ fps	L: $r_1 = 71$ ft		$r_2 = 88$ ft
LD	$\beta^* = 1.2\%$	$V_s^* = 1010$ fps			
NR	$\beta^* = 1.7\%$	$V_s^* = 980$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

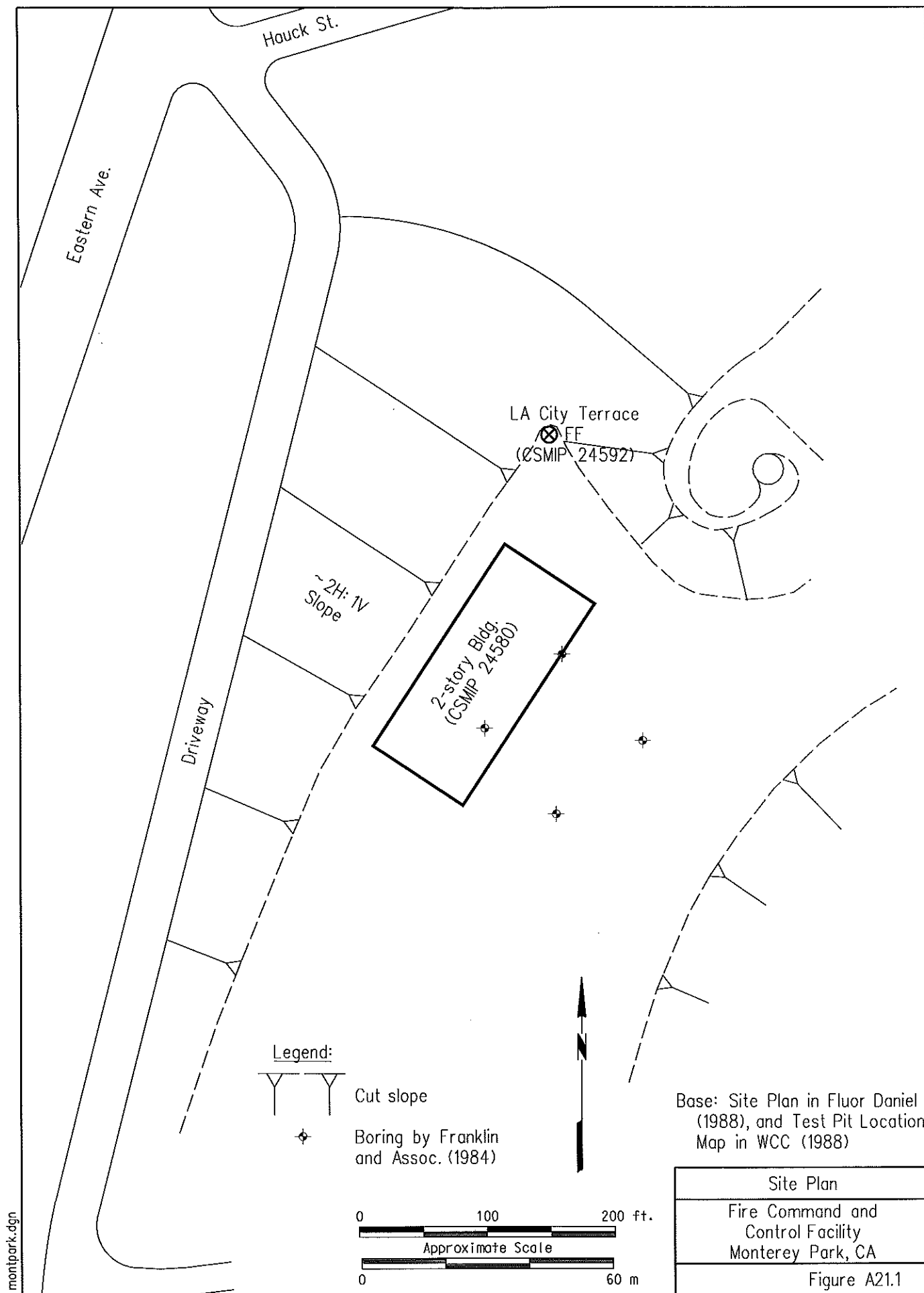
Table A21.5: Empirical and predicted inertial interaction effects

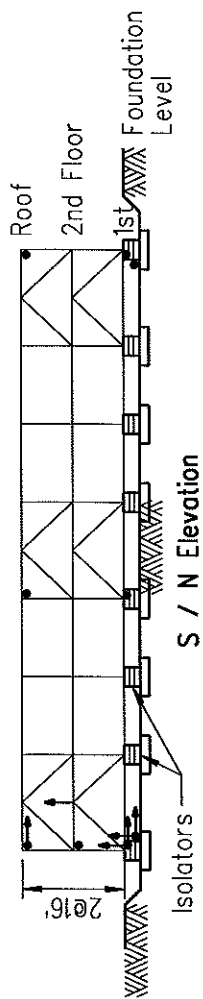
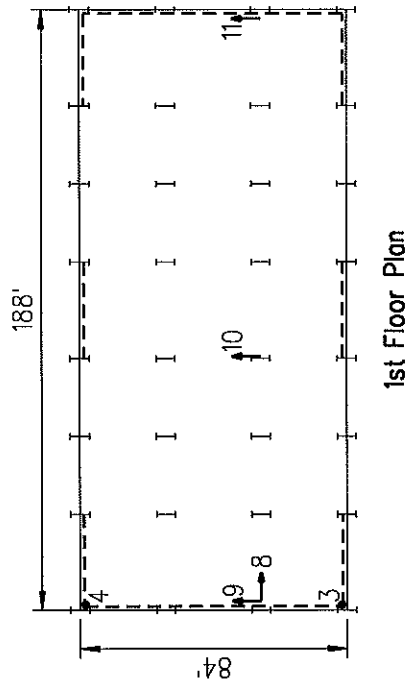
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
SM	Sys. ID	1.01	-3.7	9.4	0.97	1.3	22.6
	Prediction	1.01	0.2	13.2	1.00	0.1	21.2
LD	Sys. ID	1.00	-0.7	14.2	1.00	0.3	18.8
	Prediction	1.00	0.1	14.8	1.00	0.1	18.7
NR	Sys. ID	0.97	-5.7	34.4	0.98	-6.3	33.0
	Prediction	1.00	0.1	39.7	1.00	0.1	38.9

The negative values of foundation damping $\tilde{\zeta}_0$ from the system identification analyses indicate numerical errors in these results. Nonetheless, it is apparent from the small values of $\tilde{\zeta}_0$ and the period lengthenings near unity that the predictive analyses simulated reasonably well the negligible SSI effects at this site.

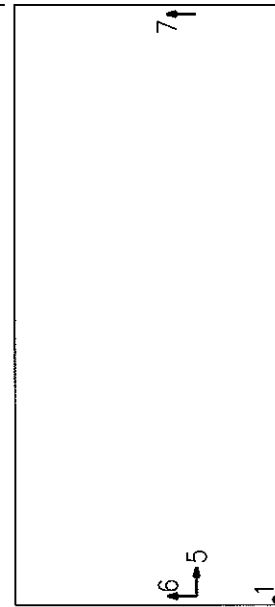
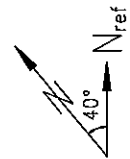
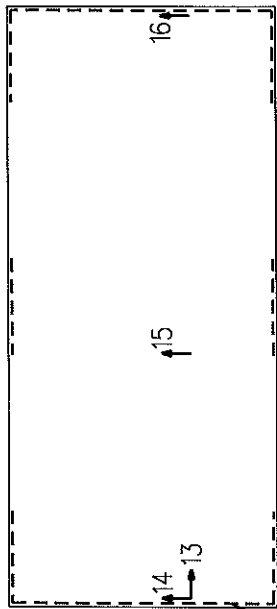
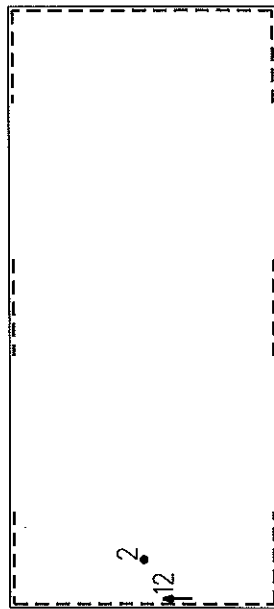
Data Sources

- Ashford, S.A. and Sitar, N. (1994). "Seismic Response of Steep Natural Slopes," *Rpt. No. UCB/EERC-94/05*, Earthquake Engrg. Res. Cntr., University of California, Berkeley, May.
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- Franklin and Assoc. (1984). "Report of Foundation Investigation, Proposed Command and Control Building, Los Angeles County Fire District, 132 North Eastern Avenue, Los Angeles, CA."
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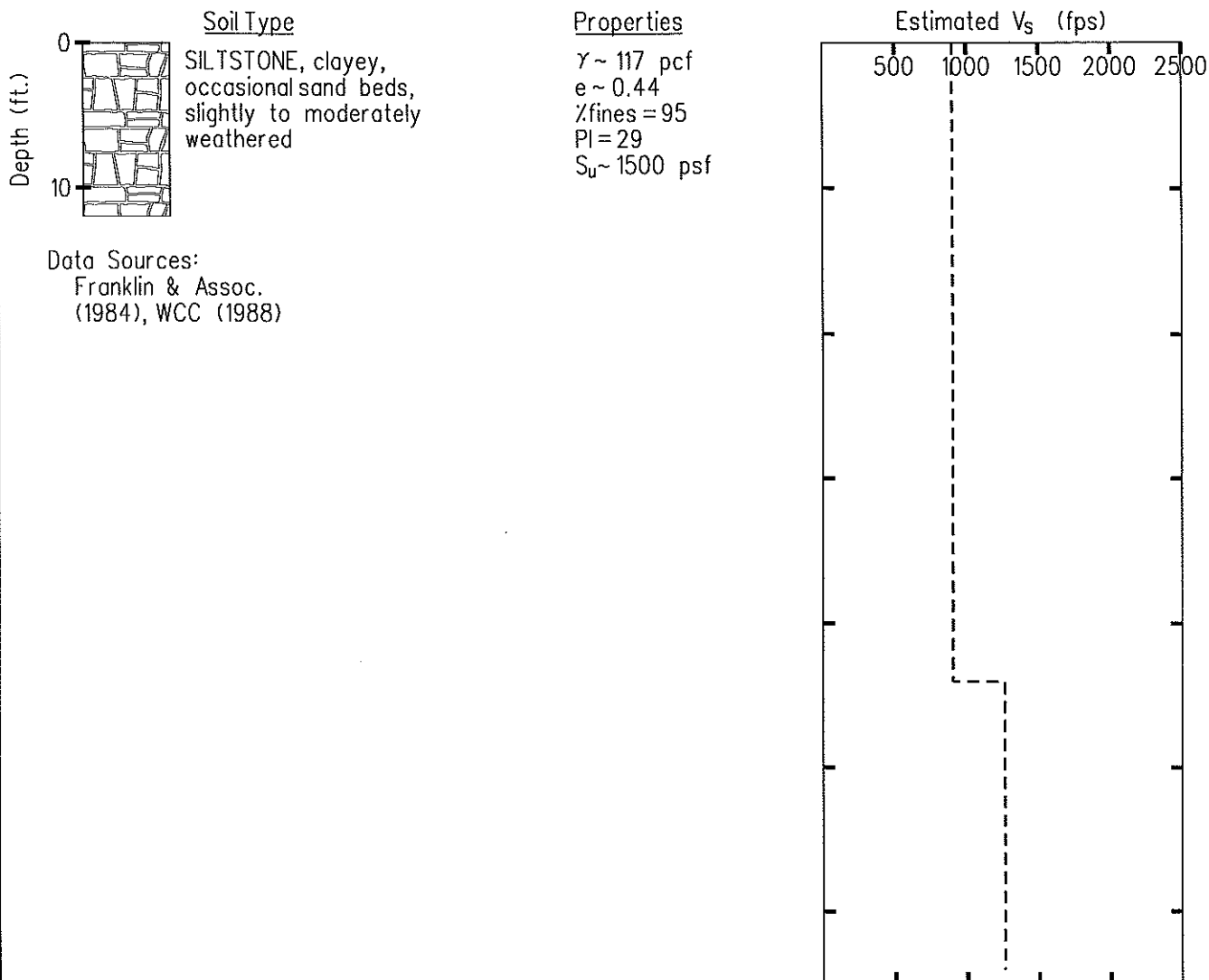


100' City Terrace Free Field



Note: Data provided by CSMIP

Sensor Locations
Fire Command and Control Facility
Monterey Park, CA
Figure A21.2



Notes:

1. No groundwater encountered.
2. Shear strength established by unconfined compressive strength tests by WCC (1988).
3. Shear wave velocities estimated based on in situ measurements in similar geologic materials at nearby Site A25.

Generalized Soil Profile

Fire Command and
Control Facility
Monterey Park, CA

Figure A21.3

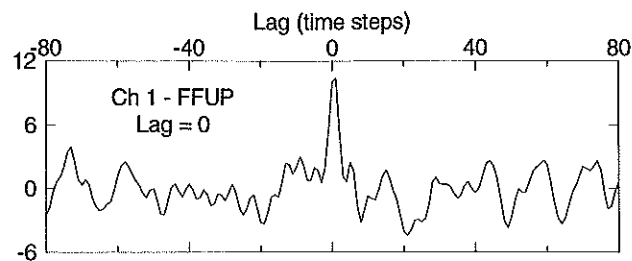


Fig. A21.SM0: Cross-correlation function between vertical foundation and free-field accelerations

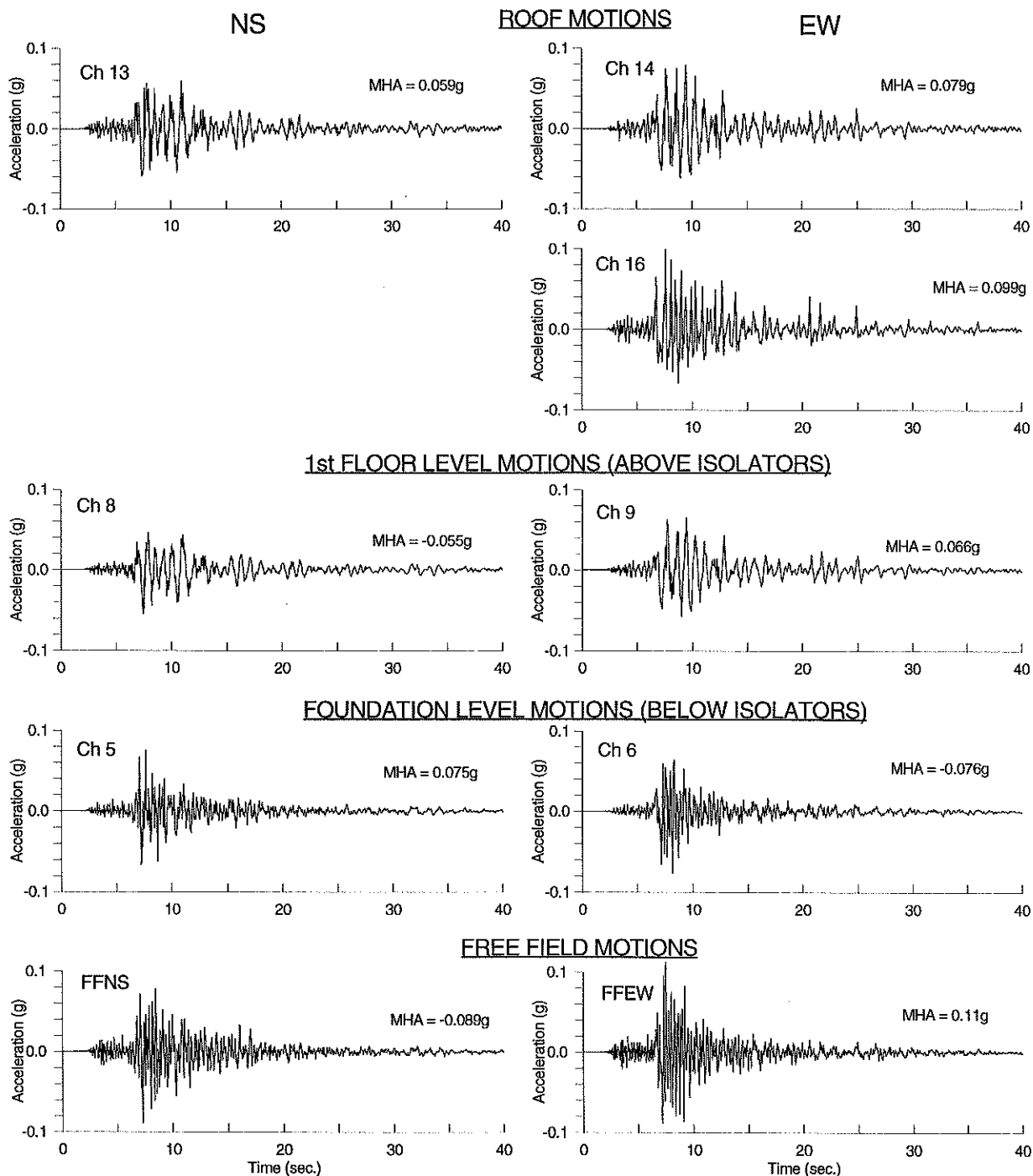


Fig. A21.SM1: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Horizontal Acceleration Time Histories
1991 Sierra Madre Earthquake

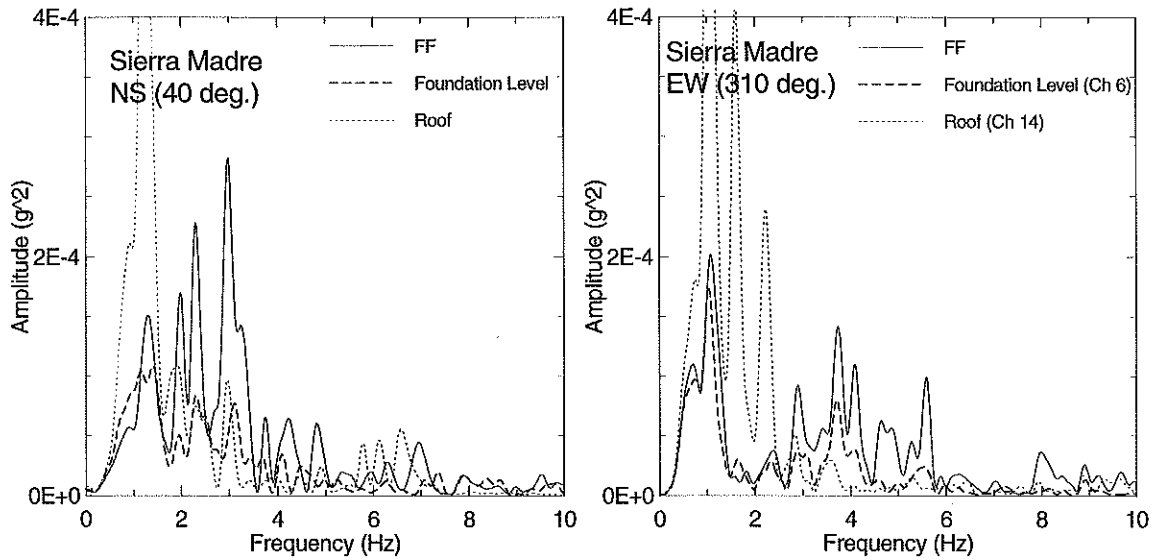


Fig. A21.SM2: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Power Spectral Density of Selected Horizontal Accelerations
1991 Sierra Madre Earthquake

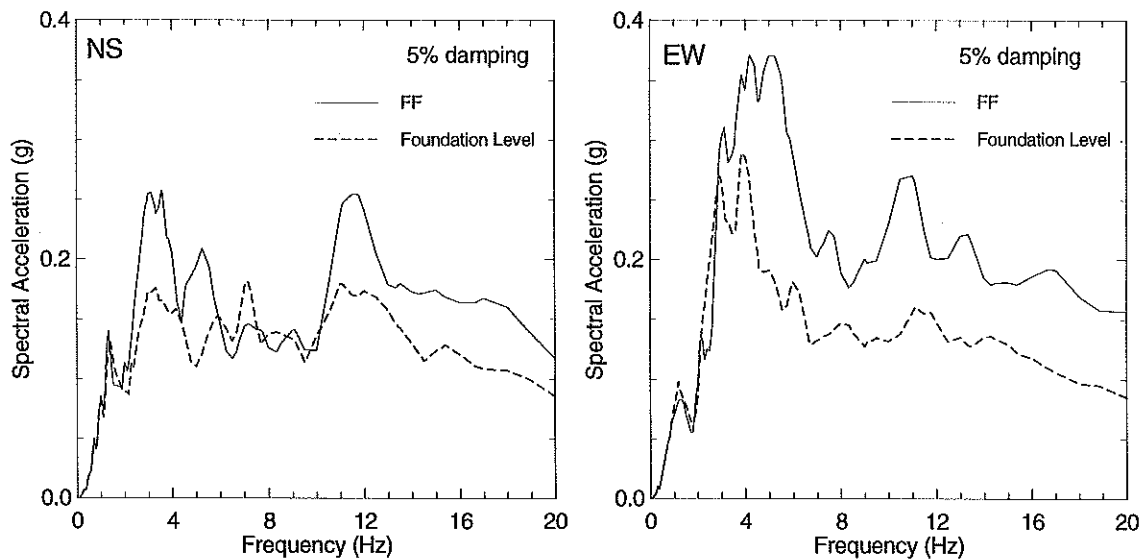


Fig. A21.SM3: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Acceleration Response Spectra of Foundation Level and Free-Field Motions
1991 Sierra Madre Earthquake

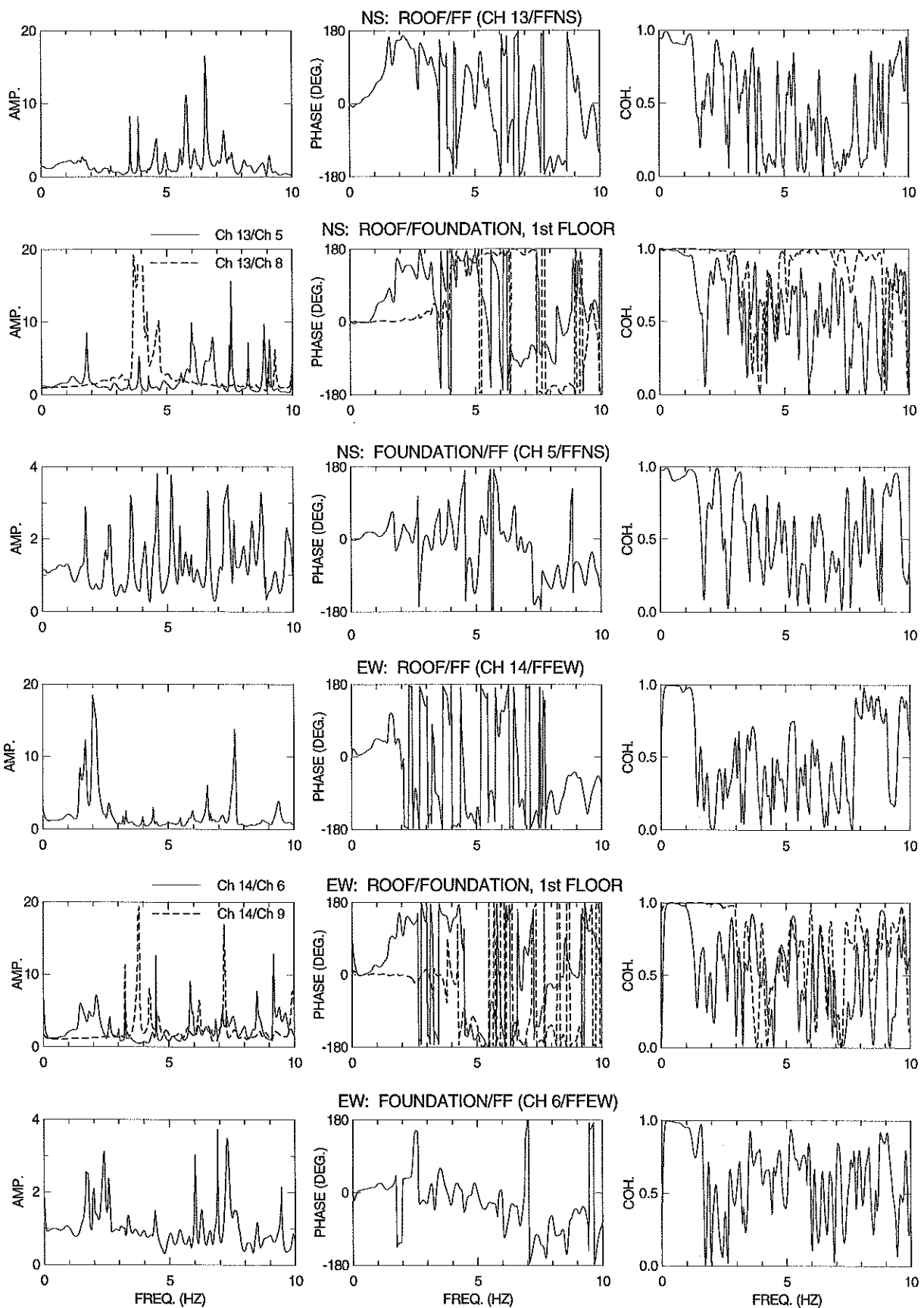


Fig. A21.SM4: LA 2 STORY FIRE COMMAND AND CONTROL FACILITY
Transmissibility and Coherency Functions
1991 Sierra Madre Earthquake
A21-11

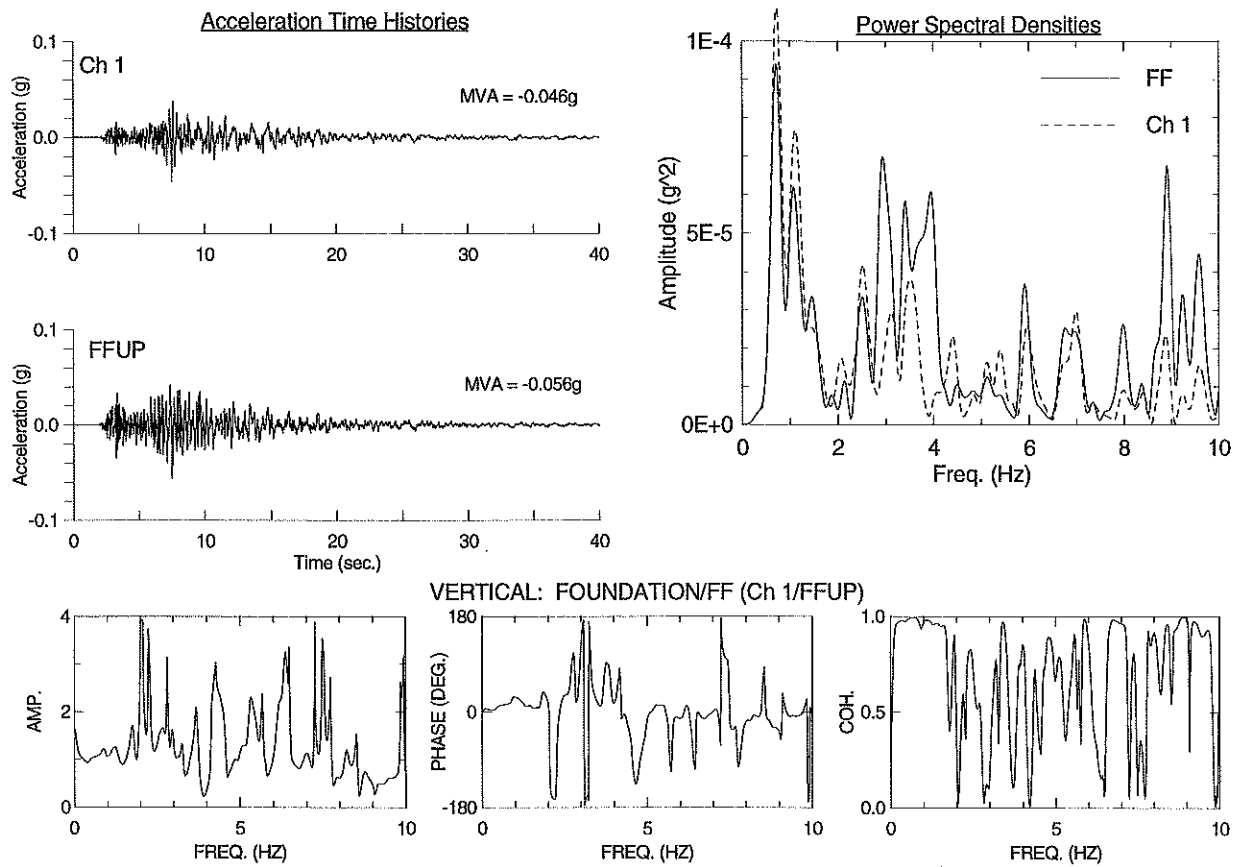


Fig. A21.SM5: LA 2 STORY FIRE COMMAND AND CONTROL FACILITY
Analysis of Vertical Motions
1991 Sierra Madre Earthquake

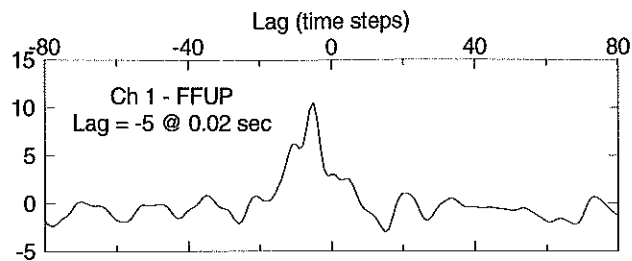


Fig. A21.LD0: Cross-correlation function between vertical foundation and free-field accelerations

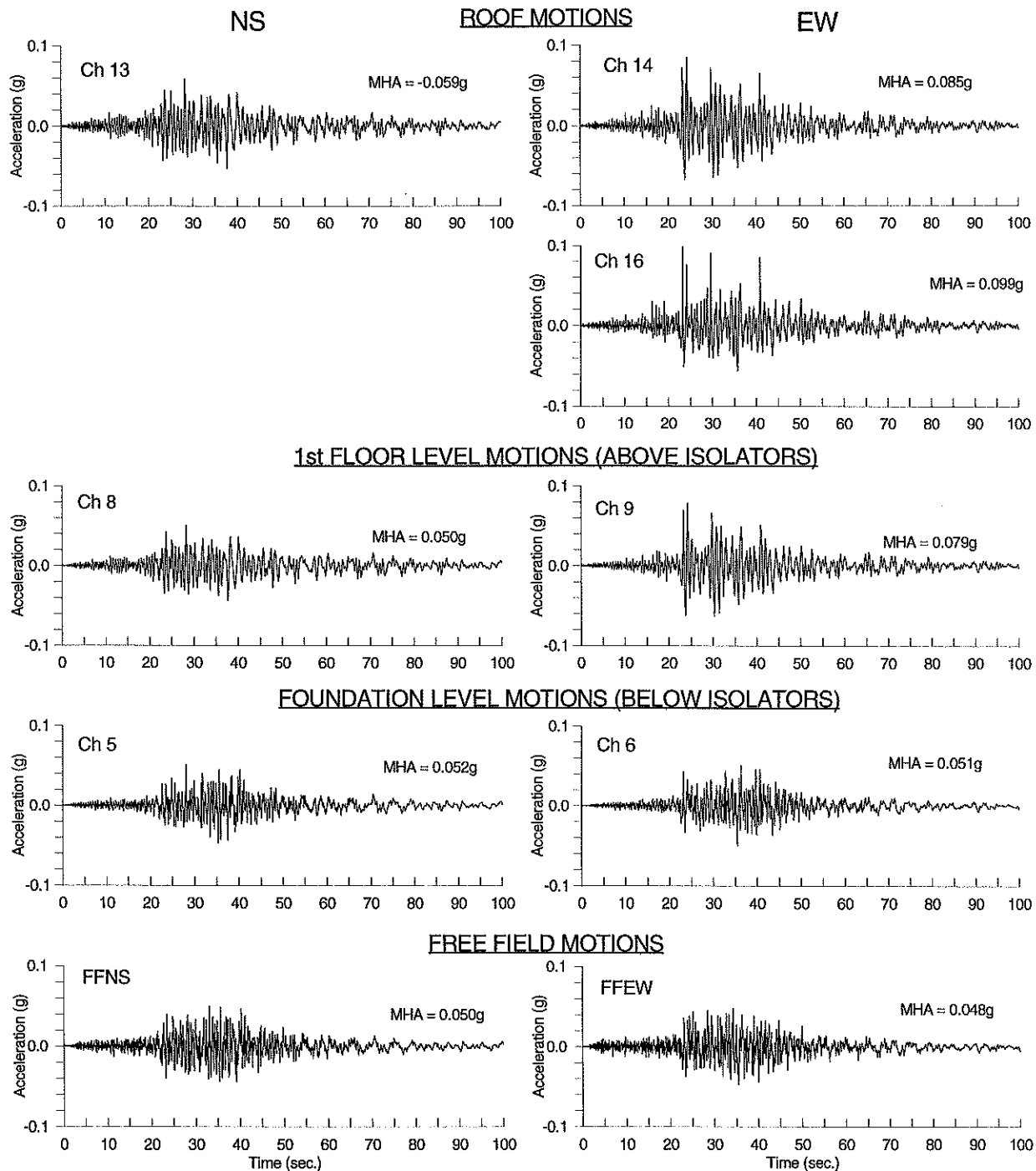


Fig. A21.LD1: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Horizontal Acceleration Time Histories
1992 Landers Earthquake
A21-13

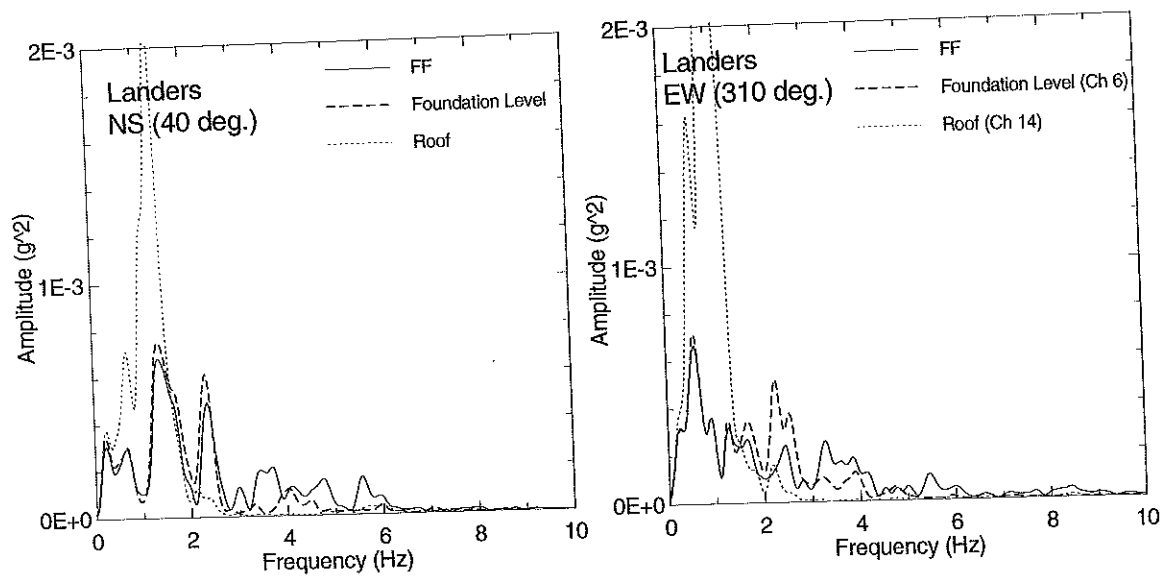


Fig. A21.LD2: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

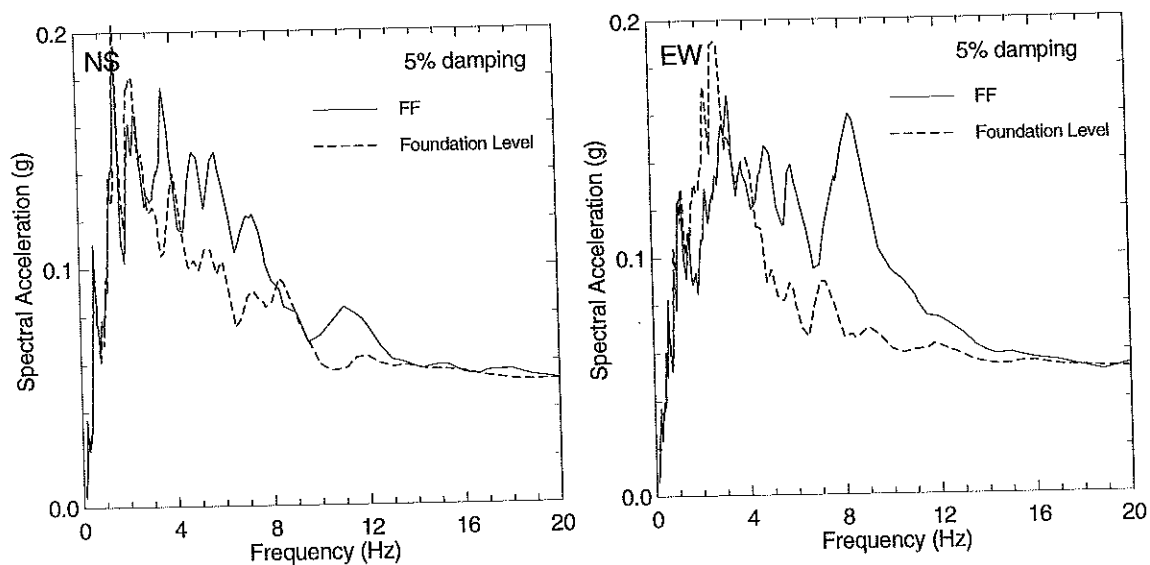


Fig. A21.LD3: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Acceleration Response Spectra of Foundation Level and Free-Field Motions
1992 Landers Earthquake

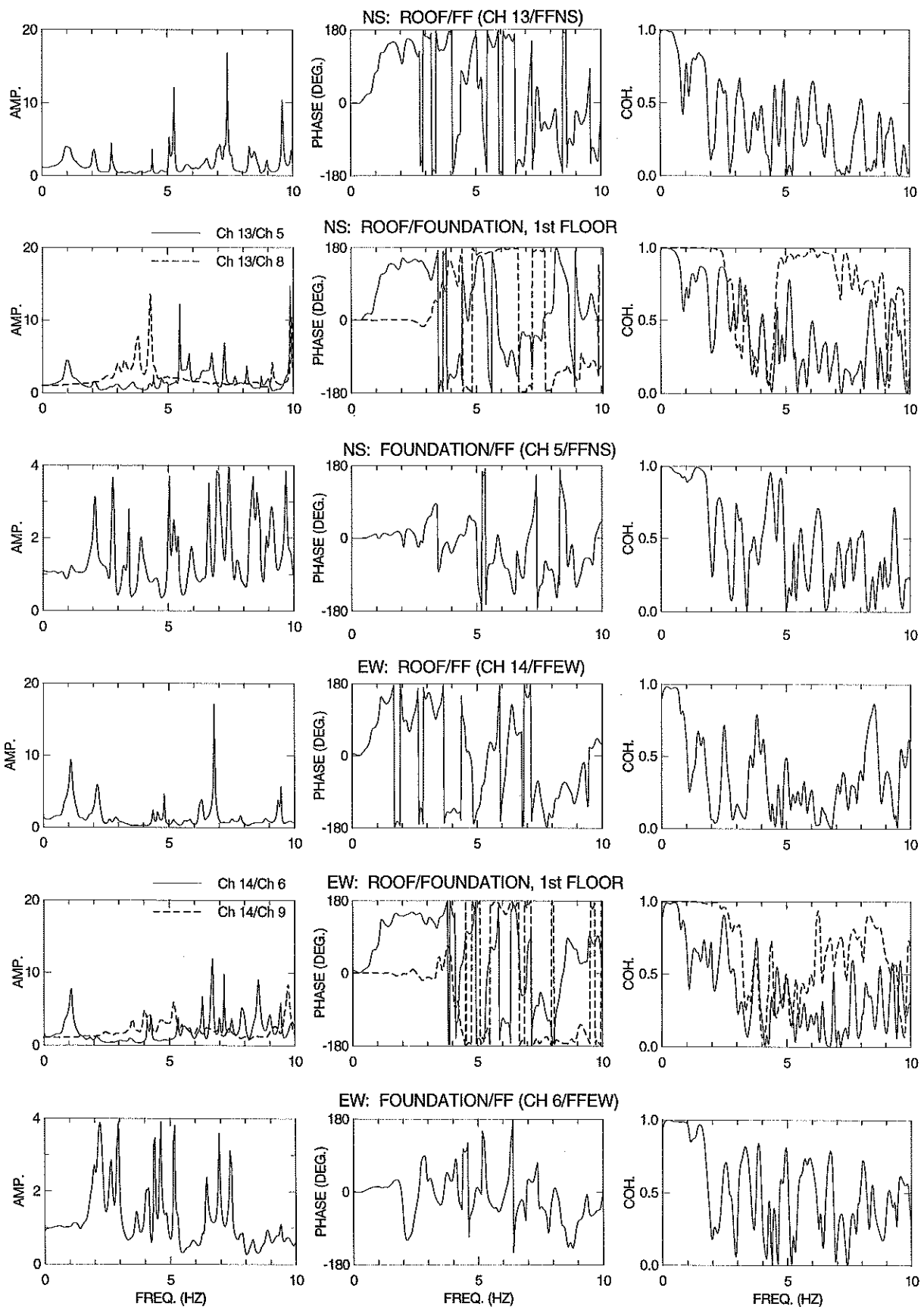


Fig. A21.LD4: LA 2 STORY FIRE COMMAND AND CONTROL FACILITY
Transmissibility and Coherency Functions
1992 Landers Earthquake
A21-15

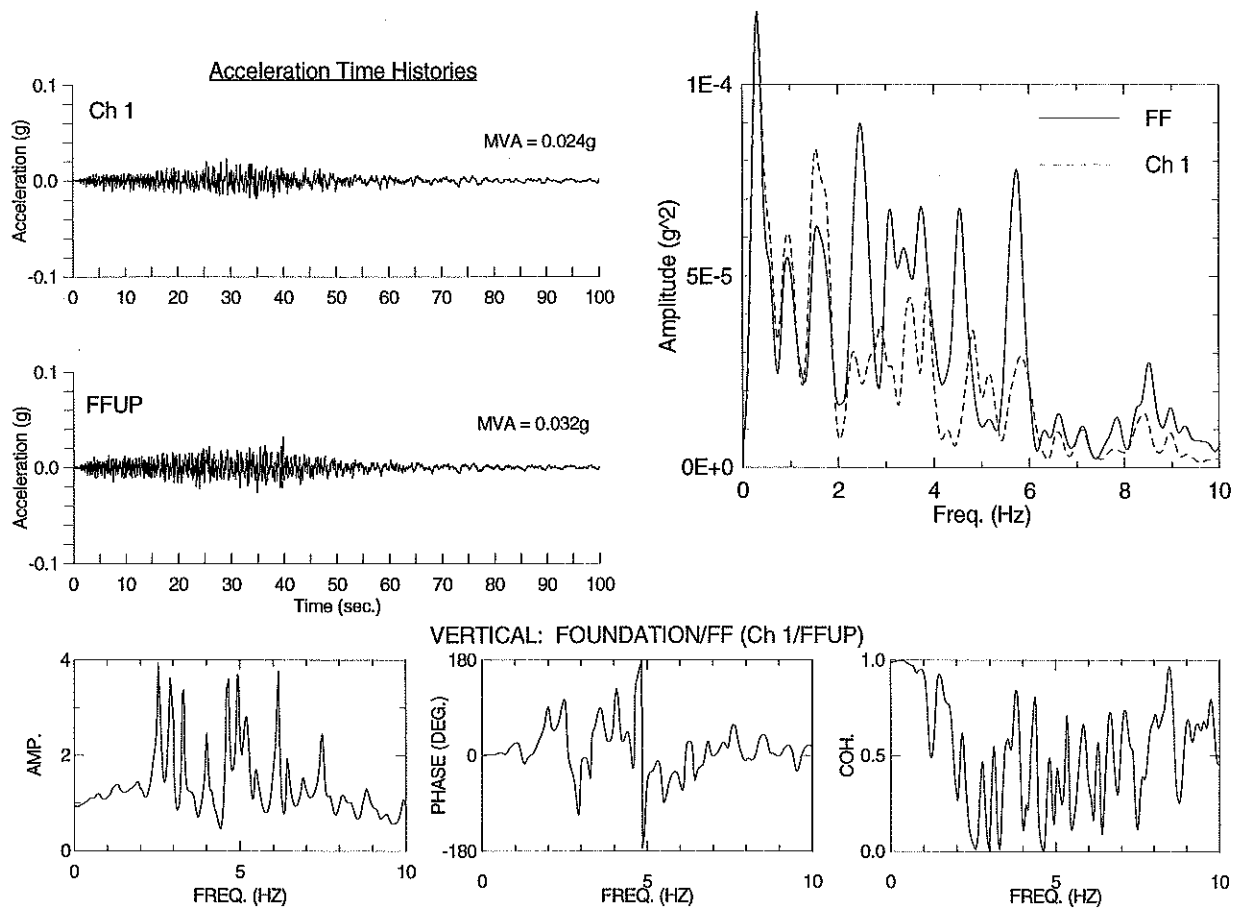


Fig. A21.LD5: LA 2 STORY FIRE COMMAND AND CONTROL FACILITY
Analysis of Vertical Motions
1992 Landers Earthquake

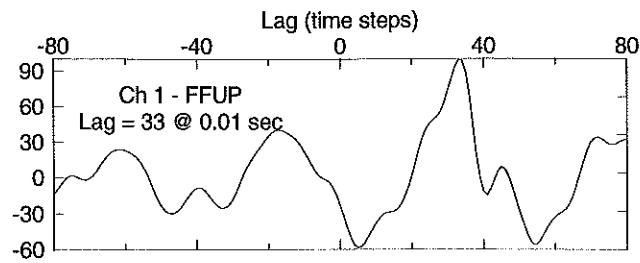


Fig. A21.NR0: Cross-correlation function between vertical foundation and free-field accelerations

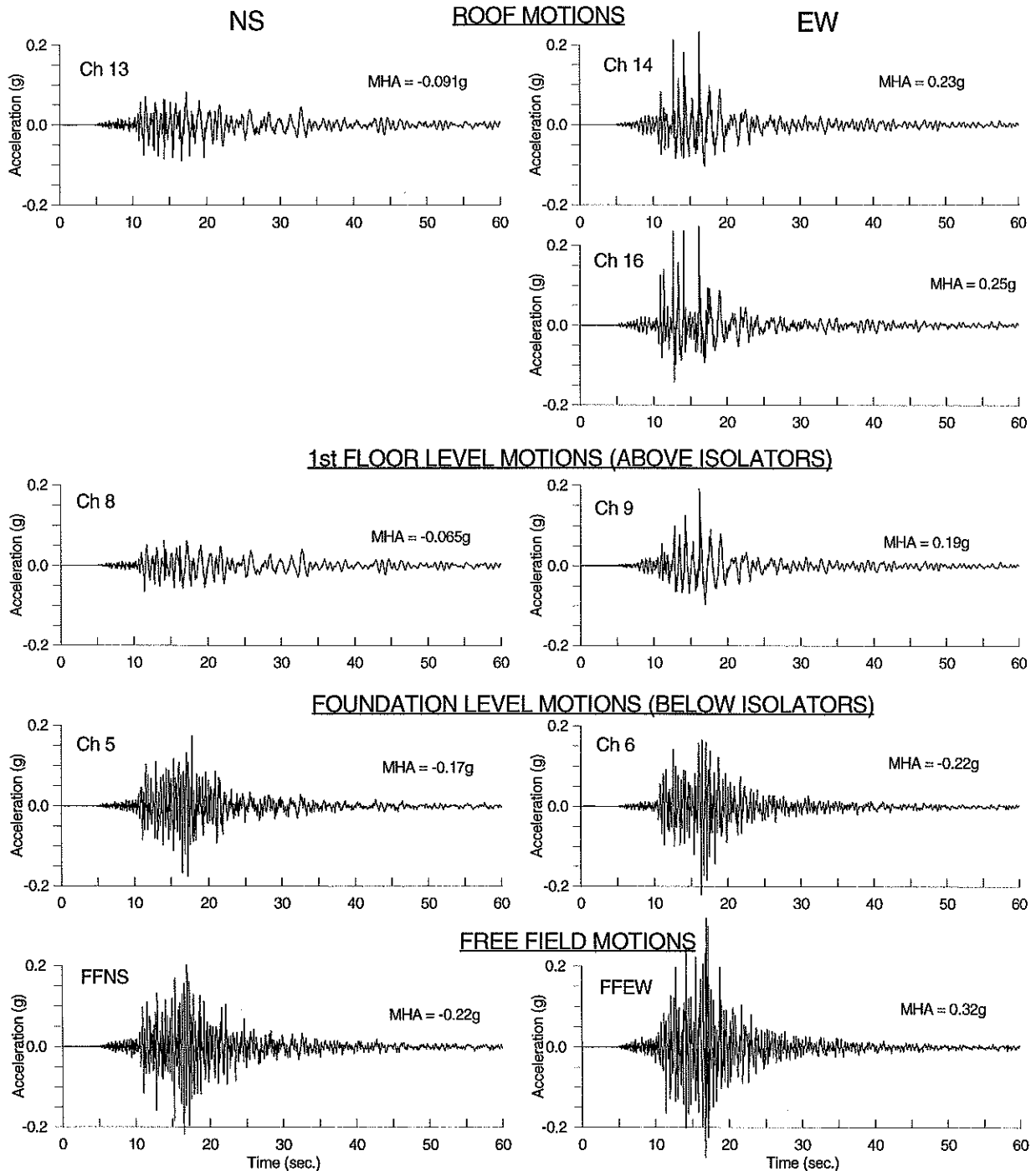


Fig. A21.NR1: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

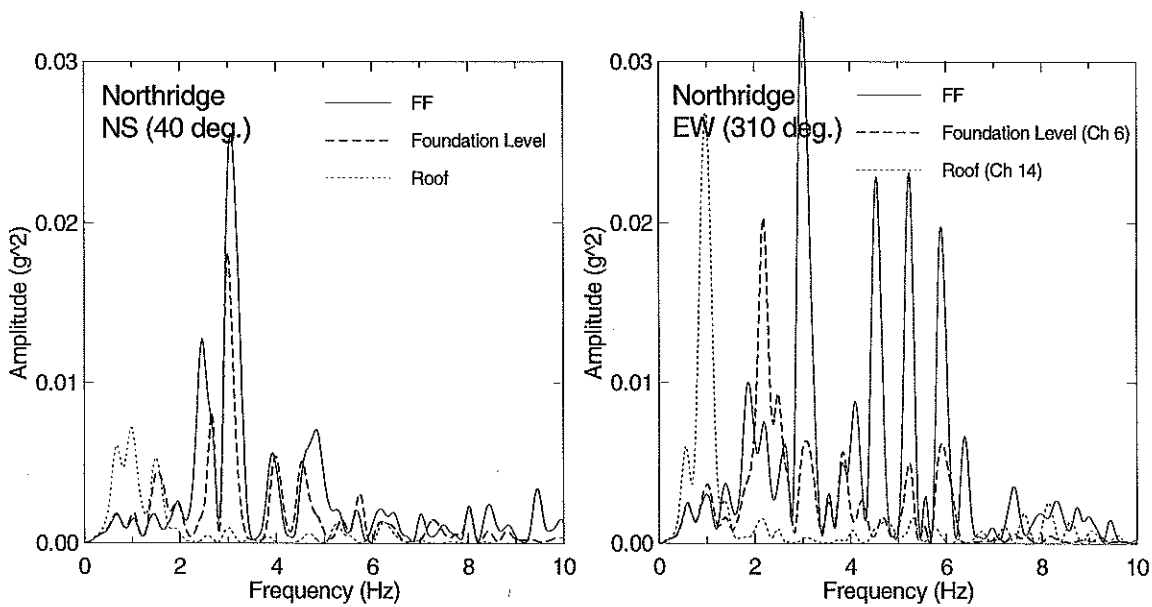


Fig. A21.NR2: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

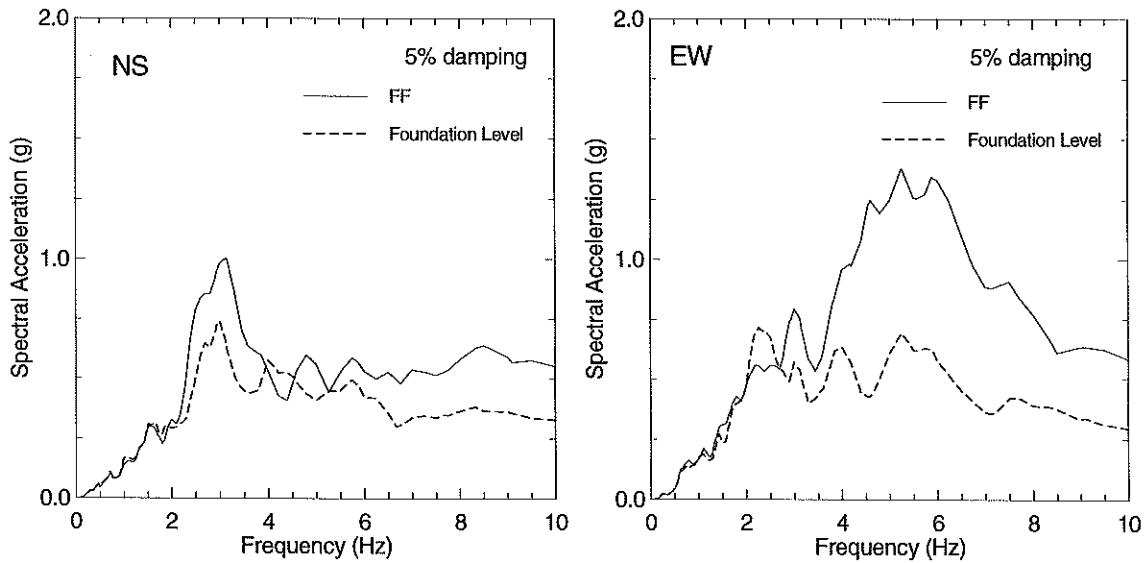


Fig. A21.NR3: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Acceleration Response Spectra of Foundation Level and Free-Field Motions
1994 Northridge Earthquake

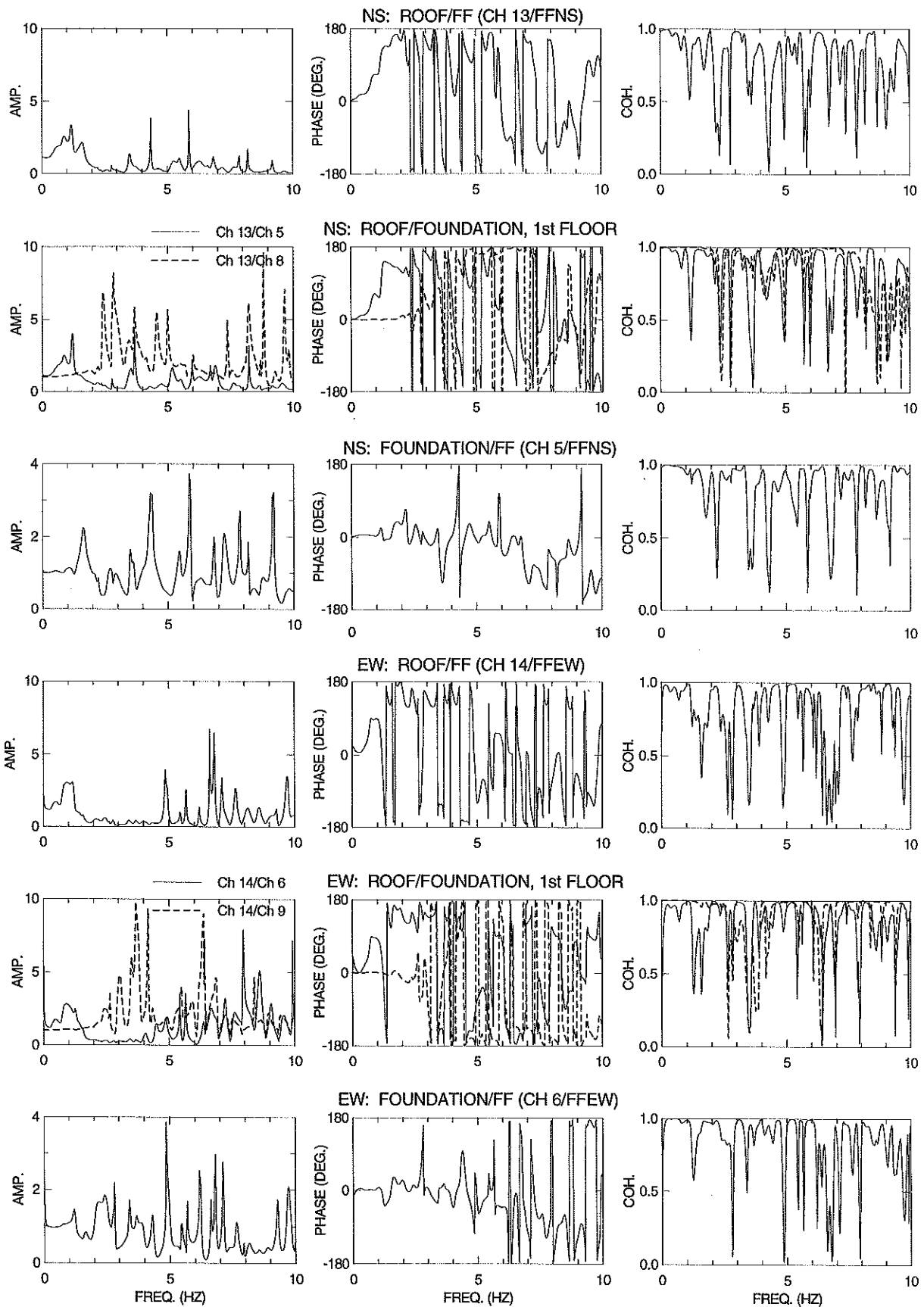


Fig. A21.NR4: LA 2 STORY FIRE COMMAND AND CONTROL FACILITY
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A21-19

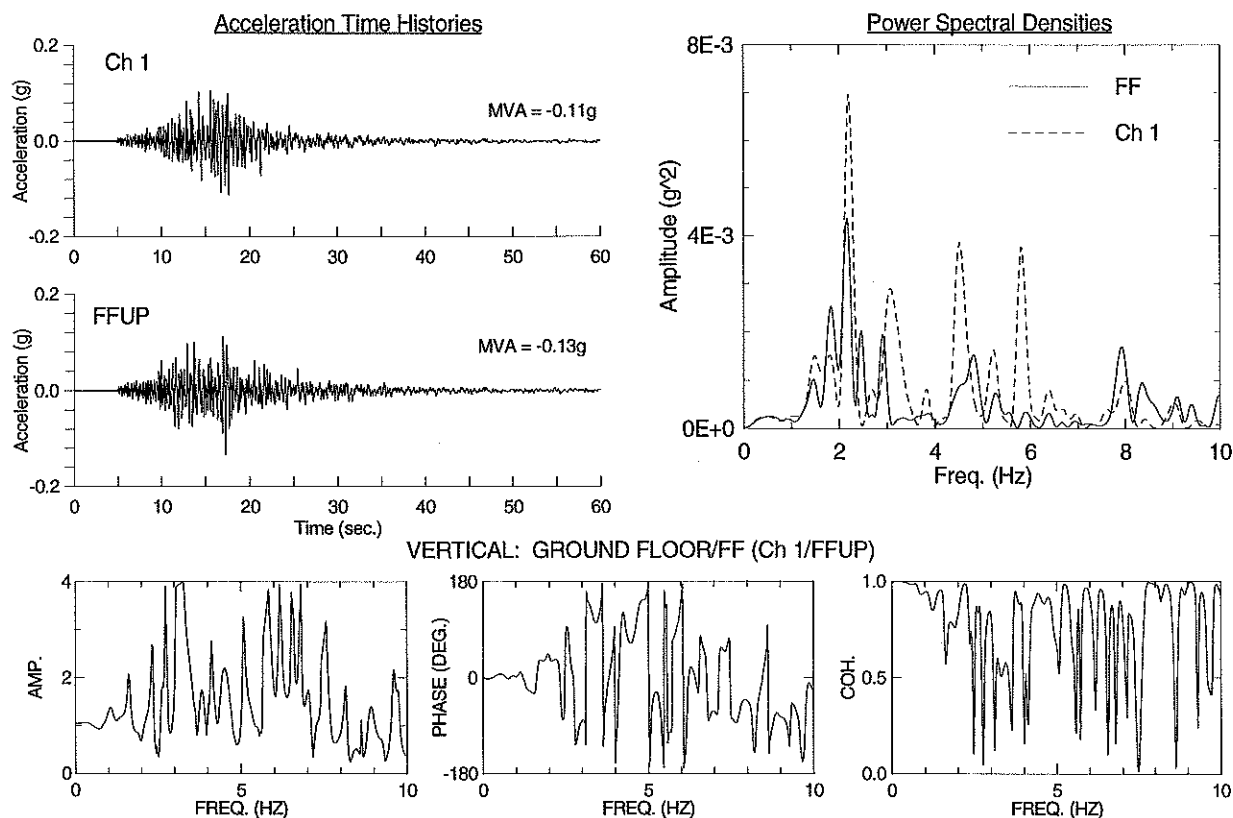


Fig. A21.NR5: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Analysis of Vertical Motions
1994 Northridge Earthquake

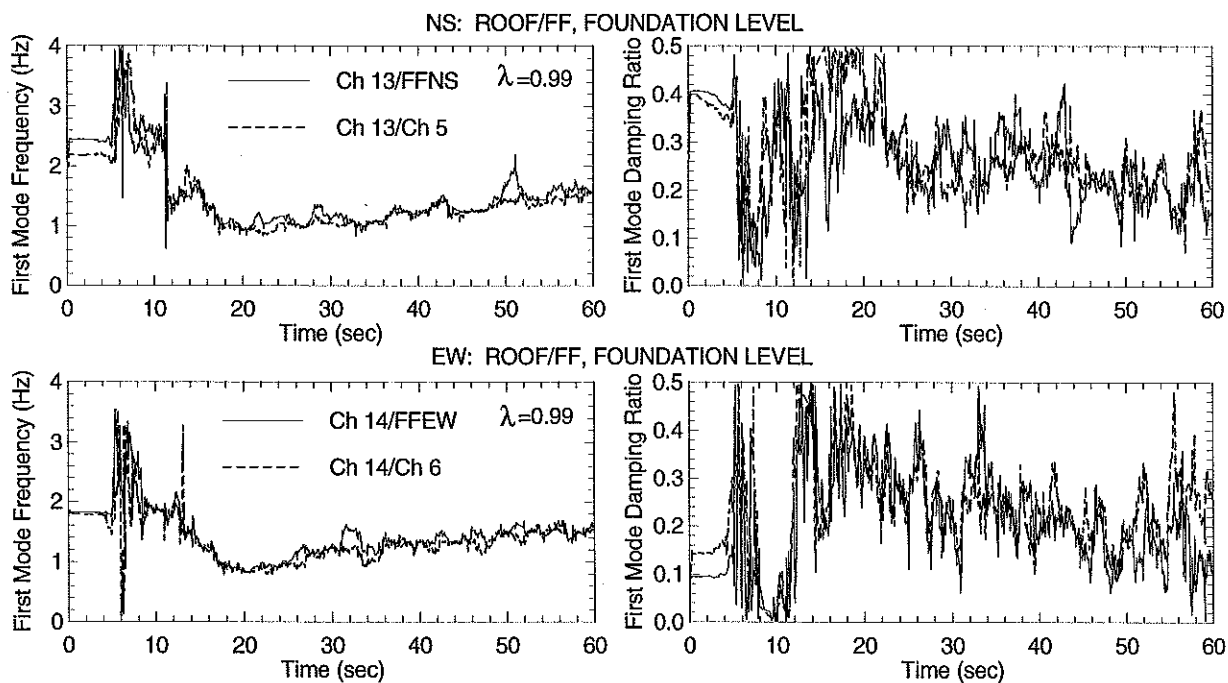


Fig. A21.NR6: LA 2 STORY FIRE COMMAND CONTROL FACILITY
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A22

Site Name: LA 3 Story Building
Sensors: CSMIP 24332/24390

Lat: 34.058°
Long.: 118.417°

Floor Levels: 5
Bsmt. Levels: 2
Structure: Steel braced frames and shear walls

Foundation: footings/piers
Site Geology: soil
Design Date: 1974

Recorded Peak Accelerations (g)

Earthquake	Free-Field			'B' Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	0.059	0.068	0.025	0.061	0.047	0.027	0.18	0.18	--
1994 Northridge	0.28	0.24	0.080	0.33	0.32	0.14	0.64	0.89	---

-- = no instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: CSMIP, filtered 0.6-23 Hz, $f_{Nyq} = 25$ Hz, could not synchronize str. and FF data, no analyses performed with this data.

Northridge Earthquake (NR)

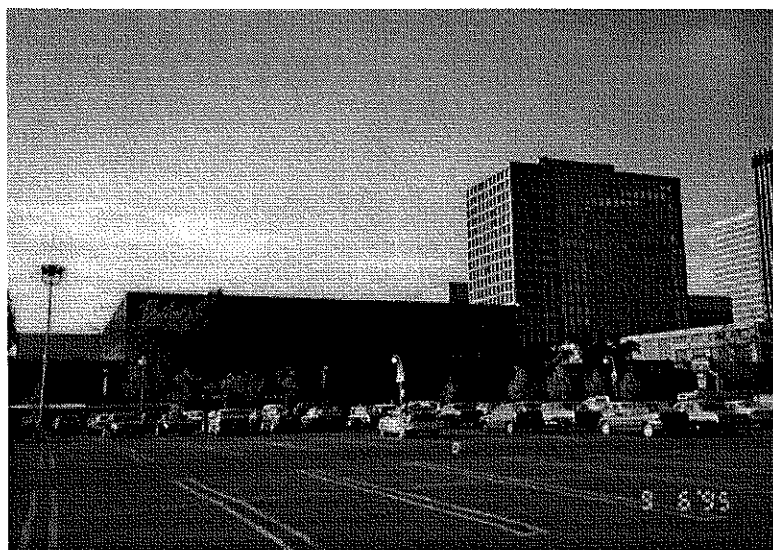
Preliminary: *Str. Data*: CSMIP, filtered $f = 0.24$ to 23 Hz (str.); $f_{Nyq} = 25$ Hz. *FF Data*: digitized and processed in this study, $f_{HP} = 0.13$ -0.20 Hz, $f_{LP} = 12.5$ -25 Hz, $f_{Nyq} = 100$ Hz. FF/str. not synchronized; FF not aligned with str.

Final: FF data rotated 39° CCW to align with str.; FF/str. data synchronized using cross-correlation of vertical displacements (Fig. A22.NR0); FF data decimated by 4 to $f_{Nyq} = 25$ Hz

Figures

A22.1-A22.3
A22.NR0-6

Site Plan, Sensor Locations, Generalized Soil Column
NR Analysis Results



View of structure, looking north (LA 19-story building in background)

DISCUSSION

Site Description

The structure is three stories tall with a bi-level basement, and is rectangular in plan (241 by 219 feet). A number of relatively small, one to two-story buildings are present near the building. Above the basement level, lateral loads are resisted by braced steel frames. Foundations consist of spread footings and shallow belled piers. As shown in Fig. A22.2, accelerographs are present on the 'B', Mall, 2nd, and roof levels. The free-field instrument is located in a golf course, about 1700 feet north of the structure.

The site is located on level terrain near the north end of the Los Angeles basin. Numerous borings have been drilled to depths of up to 200 feet in the vicinity of the building and free-field sites as part of foundation investigations (e.g.: LCA, 1966, 1967, 1973, 1980). A 400-foot boring by SW/AA (1978) drilled about 1000 feet northeast of the building is also available. Based on these sources, the soil conditions at the site consist of: about 60 feet of sandy silt and clay, 60 feet of silty and clayey sand, 90 feet of relatively dense sand, and stiff sandy, clayey silt which extends to the maximum depth explored of 400 feet. Based on measurements by LCA, groundwater appears to occur at a depth of at least 50 feet. The shear wave velocities shown in Fig. A22.3 are based on in situ downhole measurements by SW/AA (1978).

Instrument Selection and Free-Field Conditions

Sensors at the south end of the building were used in system identification analyses (Table A22.1). Acceleration time histories from these and other sensors are shown in Fig. A22.NR1.

Table A22.1: Sensors in structure used for analyses

'B' Level (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 15	Ch 13	Ch 14	Ch 1	Ch 2

There are no structures or significant slope breaks near the FF instrument that would be expected to compromise the validity of the recorded free-field ground motions. However, due to the large FF/structure separation of about 1700 feet, incoherence between these motions is a concern. Excessive incoherence in the WT data prevented synchronization of the structure and FF data, precluding analyses with these data.

Analysis Results

(a) Comparison of 'B' Level and Free-Field Motions

B/FF transmissibility functions (Fig. A22.NR4) and the spectra for horizontal motions (Fig. A22.NR2-3) generally indicate significant amplification of basement motions relative to the FF for frequencies less than 6 Hz. De-amplification occurred at higher frequencies, but these results are less reliable due to spatial incoherence.

Transmissibility, coherency, and power spectral density functions for the FF and basement vertical motions are presented in Fig. A22.NR5. Amplification occurred near the first-mode frequencies for both directions, indicating that rocking could potentially have contributed to the structural response. A direct examination of rocking effects for a single longitudinal frame is made in Fig. A22.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements and have about 50% of the roof displacement magnitude, indicating that Mall-level rocking significantly influenced the response of the frame. However, these data cannot be used to assess the rocking response of the building as a whole.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A22.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A22.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A22.NR6).

Table A22.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=6, j=6		L (EW) d=5, j=6		tr (NS) d=4, j=13		L (EW) d=9, j=13	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	2.00	14.0	2.18	14.2	1.95	12.9	2.11	13.1	1.71	6.1	1.74	11.5
	2					4.72	7.1	4.99	6.9	--	--	--	--

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- As a result of the large FF-structure separation and associated incoherence, first-mode damping ratios for the flexible-base case may not be reliable.
- There are minor differences between the fixed- and pseudo flexible-base parameters, but fairly significant differences between the pseudo flexible- and flexible-base parameters. This suggests that base translation effects were more significant than base rocking.
- A high model order was necessary to evaluate the FF-R response due the significant noise in the transfer functions (possibly due to spatial incoherence).

The RPEM results (Fig. A22.NR6) indicate drops in longitudinal $\tilde{f}_1^*$ at about 5 and 9 seconds (decreasing from about 2.6 to 2.0 Hz). Transverse $\tilde{f}_1^*$ also dropped from about 2.4 to 2.0 Hz at about 5 seconds. The RPEM damping results for FF-R response have large scatter and are not shown.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A22.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 130 feet using the recorded free-field motions and the profile in Fig. A22.3. The entire footprint area of structure was used to calculate the foundation radii. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A22.3: Soil/structure parameters used in predictive analyses

	Geotechnical	Structural
NR	Model = H $V_s = 1160$ fps	$h = 46$ ft emb. = 22.5 ft. $\gamma = 0.15$
	$\nu = 0.45$	tr: $r_1 = 130$ ft $r_2 = 128$ ft
	$\beta^* = 5.9\%$ $V_s^* = 980$ fps	L: $r_1 = 130$ ft $r_2 = 134$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A22.3 with the fixed-base modal parameters in Table A22.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A22.4.

Table A22.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.17	--	--	1.25	--	--
	Pre.-Vel.	1.05	--	--	1.06	--	--
	Pre.-Bie.	1.05	--	--	1.06	--	--

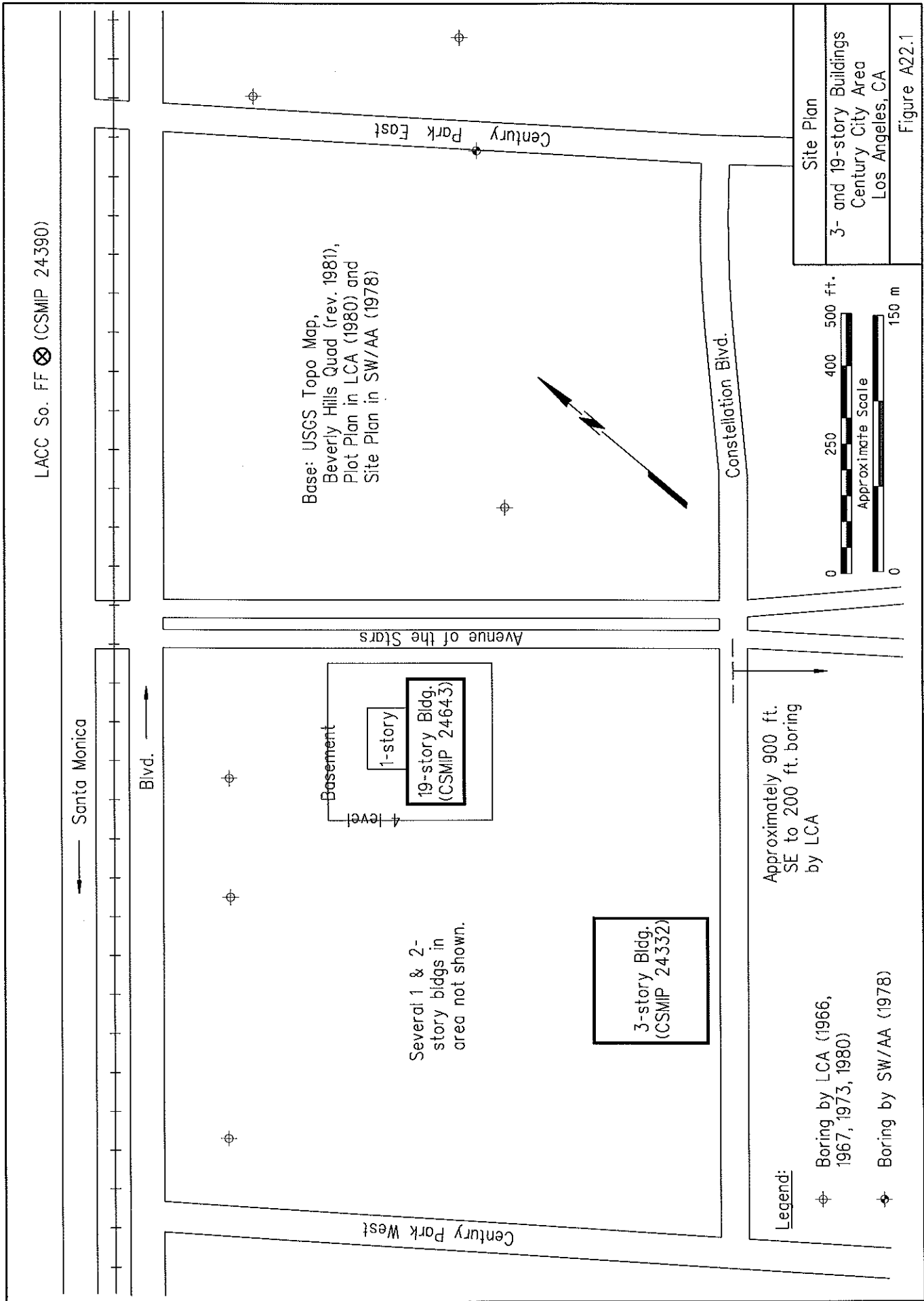
Both formulations underpredict period lengthening. However, the unknown effect of spatial incoherence on flexible-base parameters from system identification analyses diminishes the general confidence level in these results.

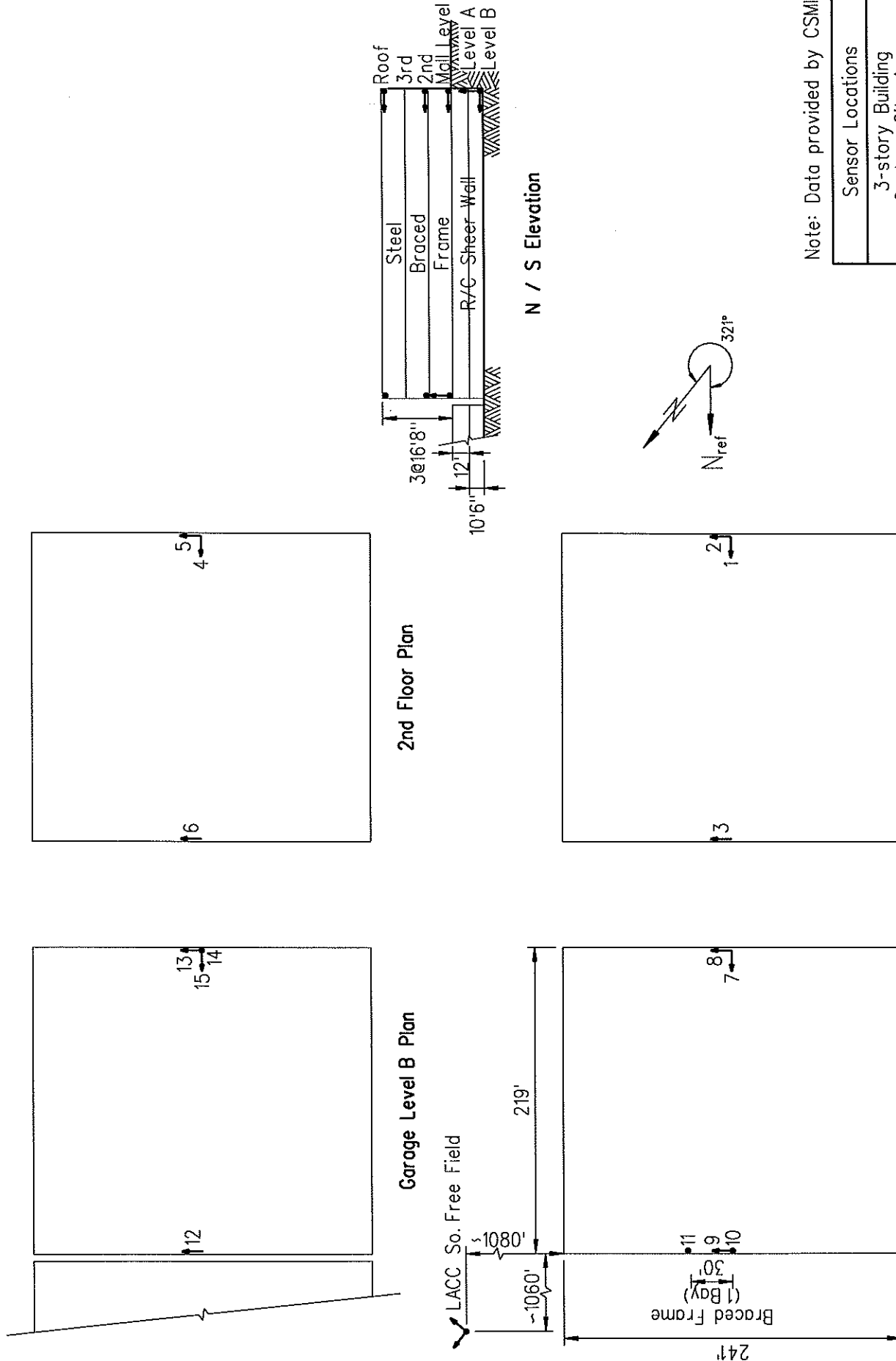
Data Sources

LeRoy Crandall Assoc., LCA (1966, 1967, 1973). borings from foundation investigations, borings provided in SW/AA, 1978 (see below).

LeRoy Crandall Assoc., LCA (1980). “Report of Supplemental Foundation Investigation, Proposed Additions to Shopping Center, Santa Monica Boulevard and Century Park West, Century City, CA,” *Proj. No. A80182*.

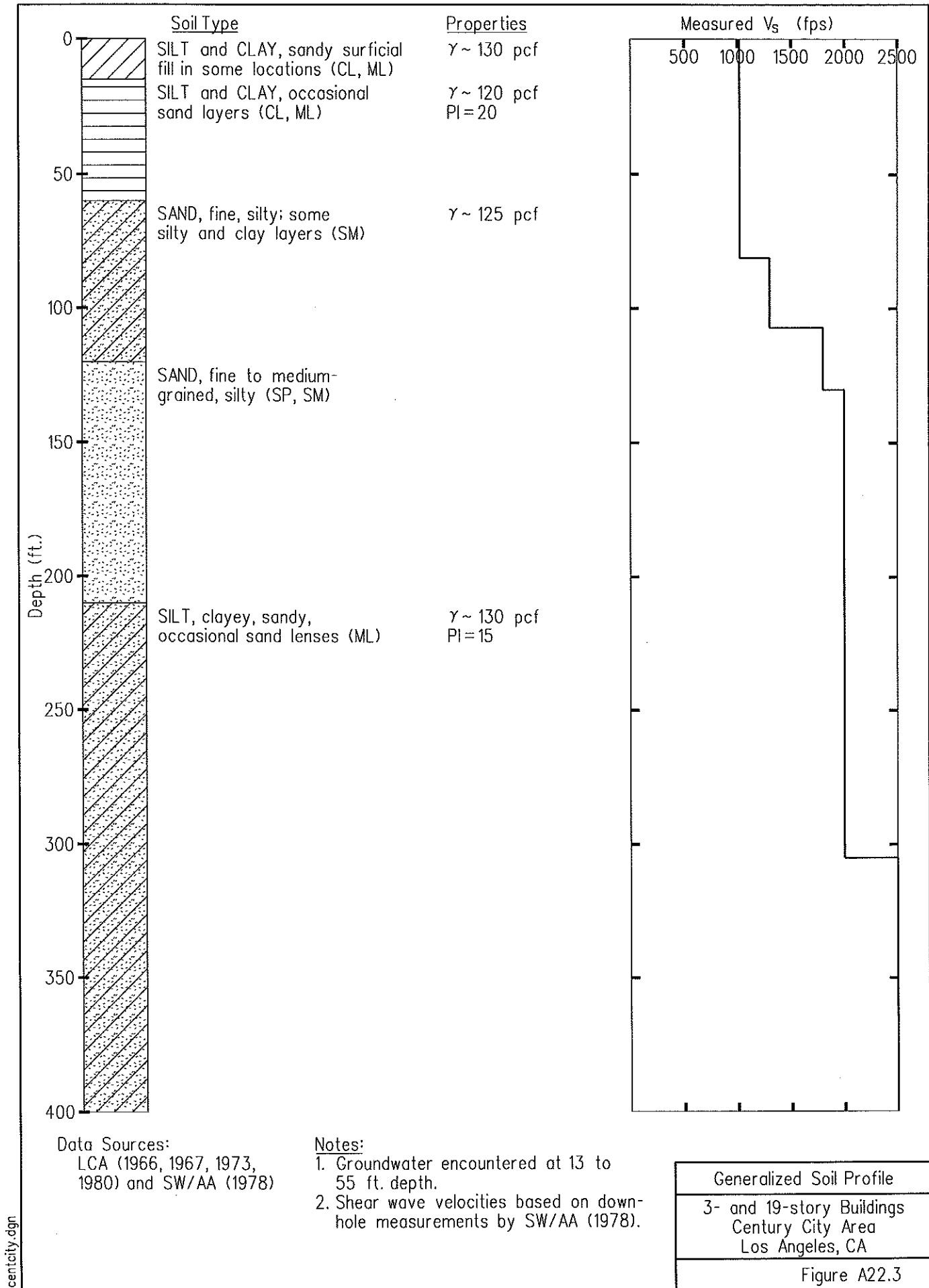
Shannon and Wilson/Agbabian Assoc., SW/AA (1978). “Data from Selected Accelerograph Stations at Wilshire Boulevard, Century City, and Ventura Boulevard, Los Angeles, CA,” *Rpt. NUREG/CR-0074*, for U.S. Nuclear Regulatory Commission.





Note: Data provided by CSMIP

Sensor Locations
3-story Building
Century City Area
Los Angeles, CA
Figure A22.2



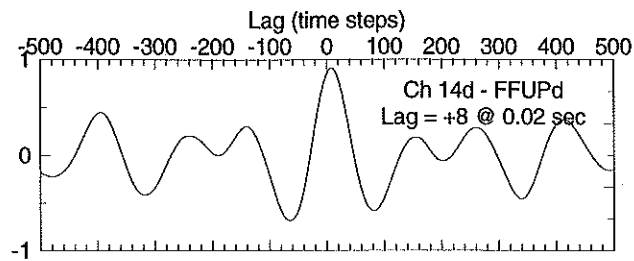


Fig. A22.NR0: Cross-correlation function between vertical 'B' level and free-field displacements

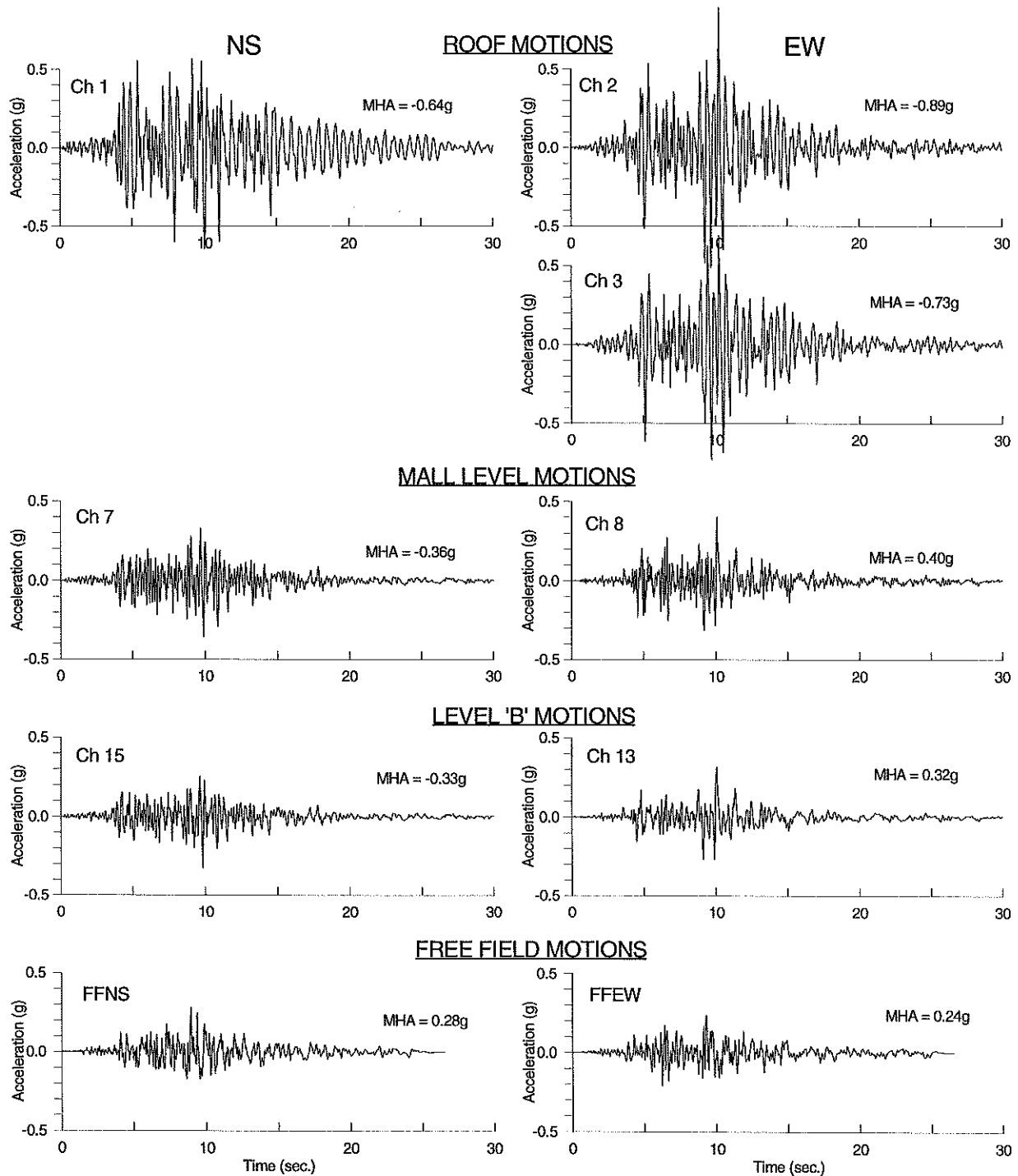


Fig. A22.NR1: LA 3 STORY BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

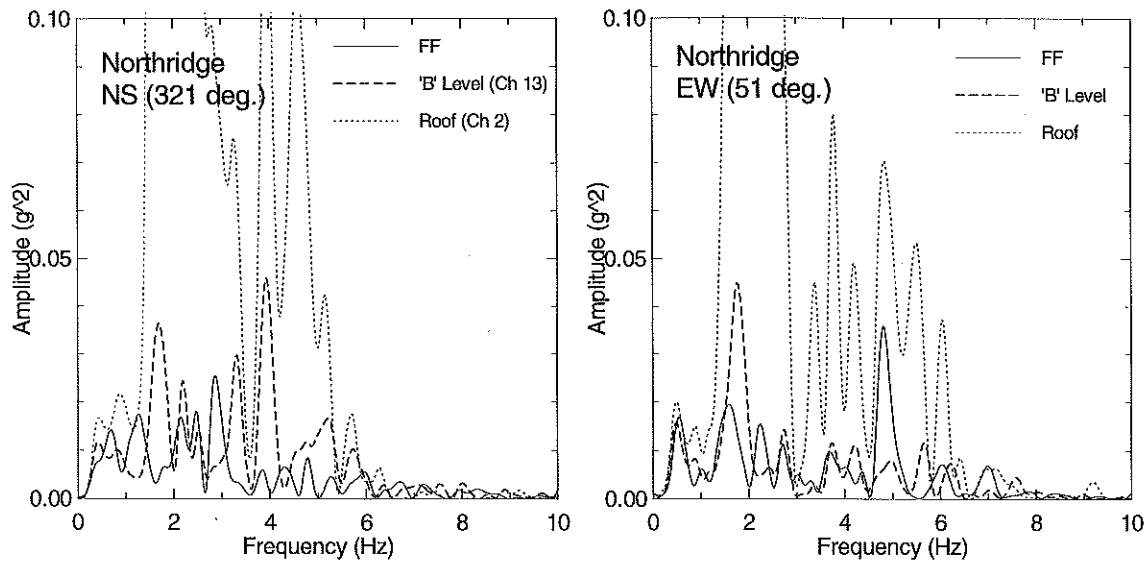


Fig. A22.NR2: LA 3 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

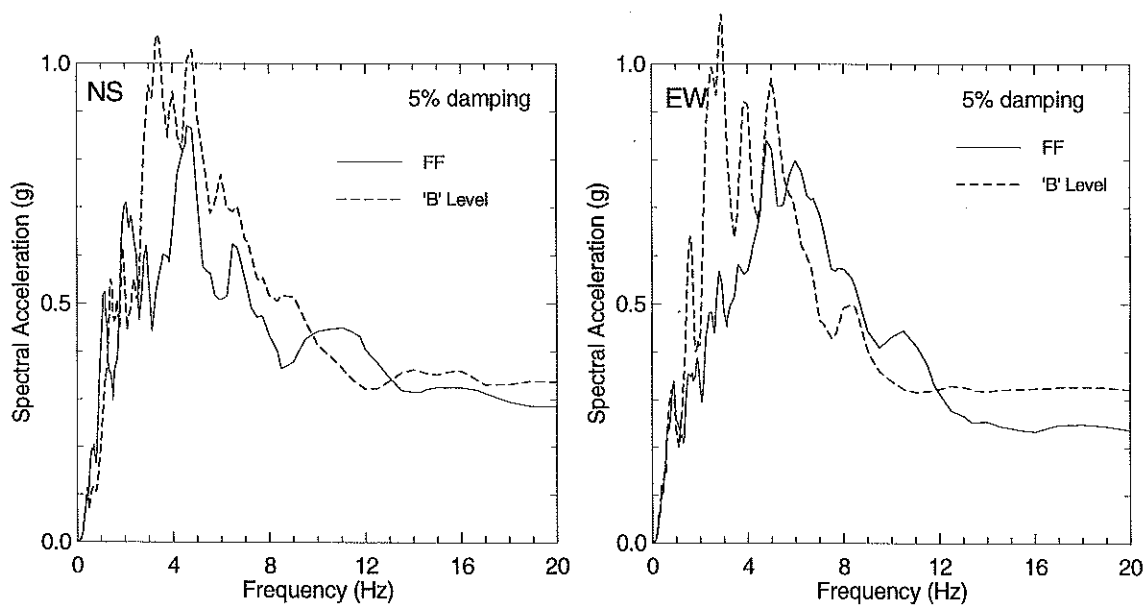


Fig. A21.NR3: LA 3 STORY BUILDING
Acceleration Response Spectra of 'B' Level and Free-Field Motions
1994 Northridge Earthquake

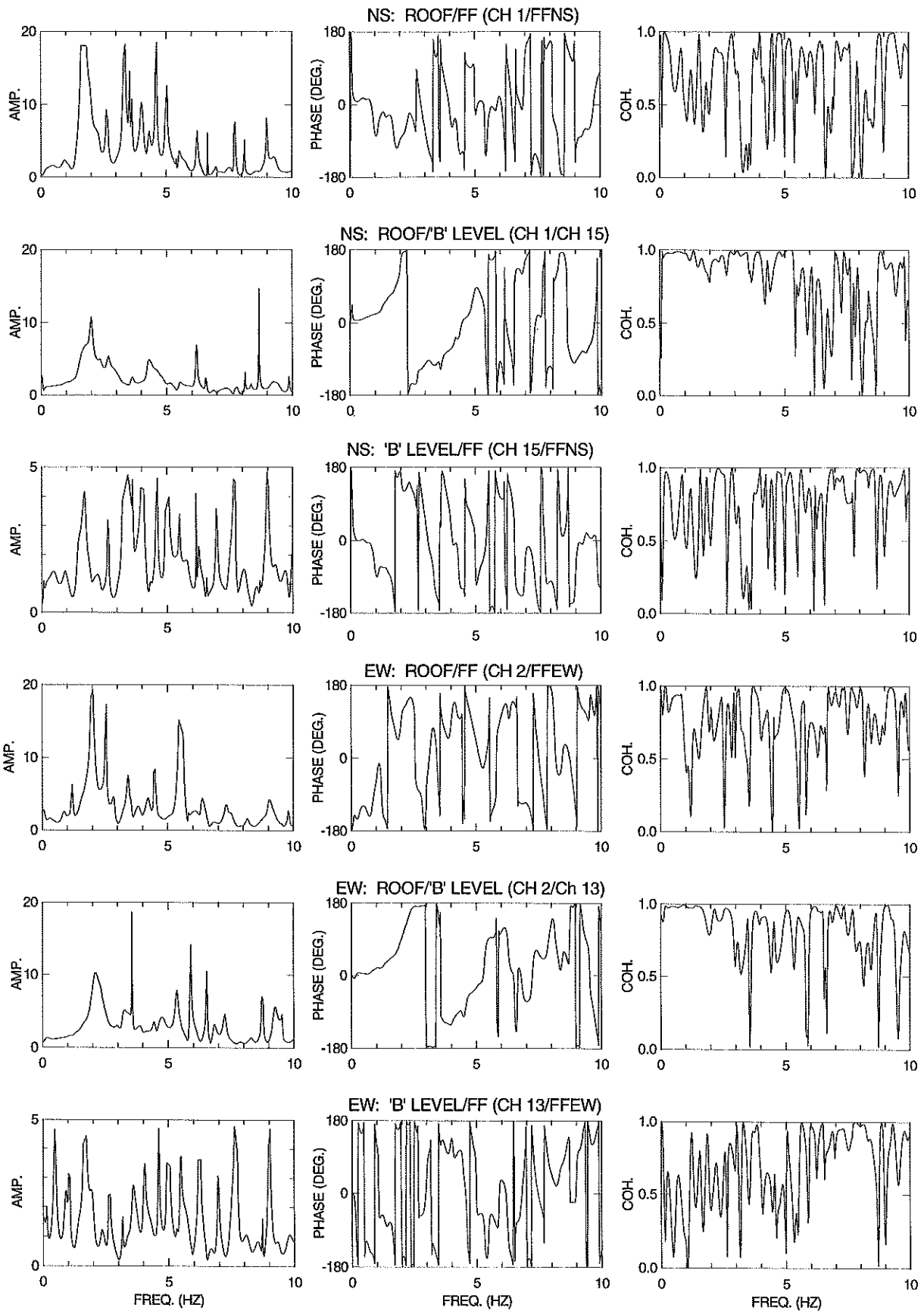


Fig.A22.NR4: LA 3 STORY BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A22-10

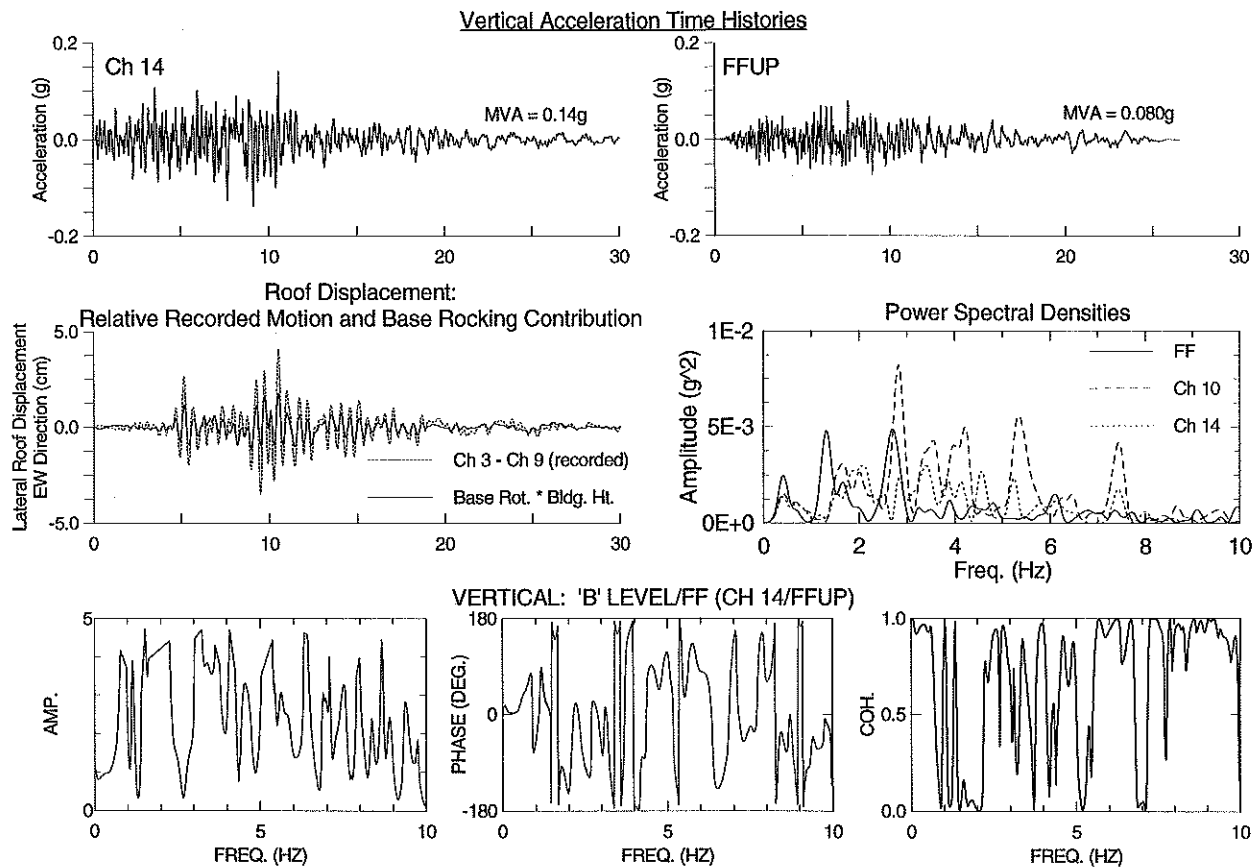


Fig. A22.NR5: LA 3 STORY BUILDING
Analysis of Vertical Motions and Mall-Level Rocking Effects
1994 Northridge Earthquake

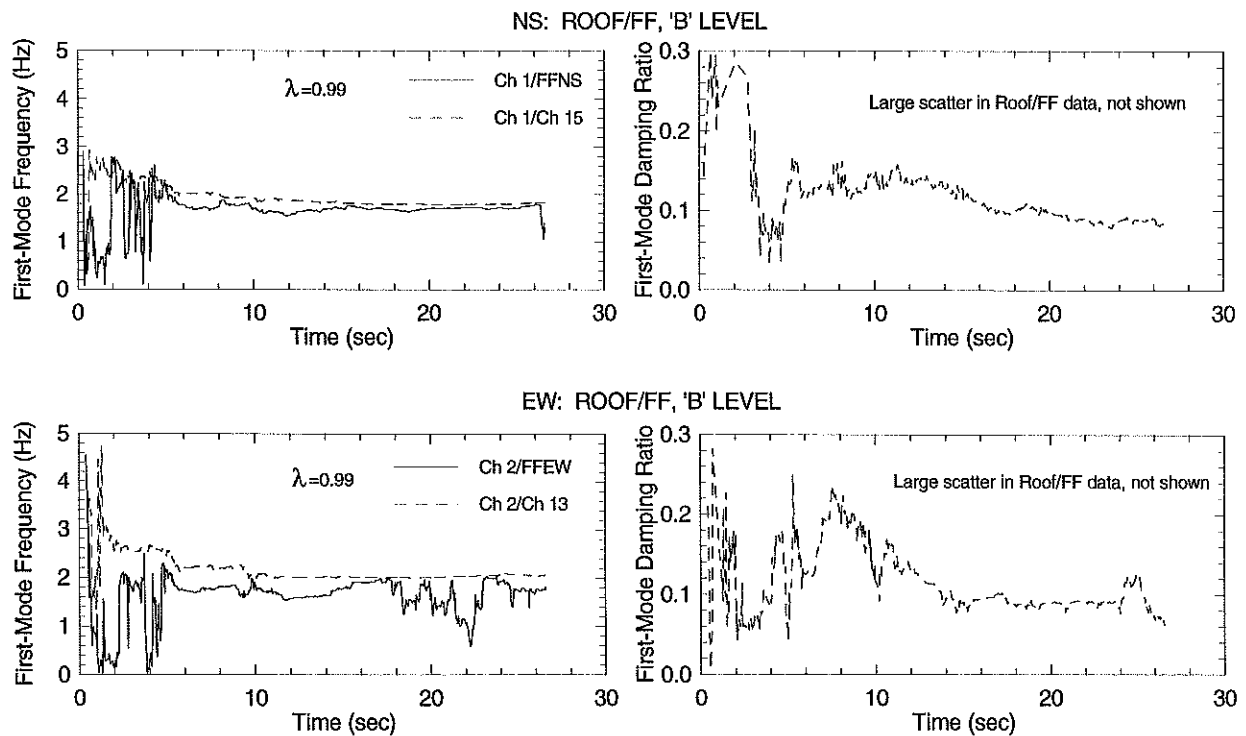


Fig. A22.NR6: LA 3 STORY BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake
A22-11

SITE A23

Site Name: LA 6 Story Office Building
Sensors: CSMIP 24652, USC 096

Lat: 34.021°
Long.: 118.287°

Floor Levels: 6
Bsmt. Levels: 1
Structure: Steel braced and torsional frames

Foundation: mats/footings
Site Geology: soil
Design Date: 1988

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.12	0.25	0.064	0.16	0.24	0.076	0.29	0.48	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: CSMIP(str.), USC (FF); filtered $f = 0.24$ to 46 Hz (str.), 0.14-0.28 to 46 Hz (FF, different high-pass frequency for each channel); $f_{Nyq} = 50$ Hz, 25 Hz (str., FF); FF/str. not synchronized; FF not aligned with str.

Note: The channel directions in the original USC FF data were mislabeled; 'Up' and 'N90E' were switched, 'NOE' is correct.

Final: FF data rotated 15° CCW to align with str.; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A23.NR0); FF data decimated by 2, str. data decimated by 4 to $f_{Nyq} = 12.5$ Hz

Figures

A23.1-A23.3
A23.NR0-6

Site Plan, Sensor Locations, Generalized Soil Column
NR Analysis Results



View of structure, looking south-east

DISCUSSION

Site Description

The structure has 5 above-ground levels and a full single-level basement. The building is square in plan view with 17 by 20-foot towers at each corner. Within these towers are steel braced frames and moment resisting frames which comprise the lateral force resisting system of the structure. The building's four towers are supported on 4 to 5-foot thick mat foundations, while the remainder of the building is supported on spread footings. The mat beneath the instrumented west tower has lateral dimensions of 34.4 x 41.7 feet. Several 1 to 4-story buildings are located within about 100 feet of the structure. As shown in Fig. A23.2, accelerographs are present in the basement, 1st, 3rd, and roof levels. The free-field instrument is located in a parking lot about 830 feet northwest of the structure.

The site is located on level terrain within the greater Los Angeles basin. Soil conditions have been investigated to a depth of 40 feet at the site by LCA (1975 and 1986) and to a depth of 400 feet at the nearby Exposition Park by Duke and Leeds (1962). As shown in Fig. A23.3, the soil profile consist of 50 feet of silty, gravely sand, 300 feet of stiffer clay and gravel, and 50 feet of sand and gravel which extends to the maximum depth explored of 400 feet. The depth to bedrock in the area has been estimated to be 600 to 700 feet by Duke and Leeds. The near-surface shear wave velocities (0 to 50 feet) shown in Fig. A23.3 are based on SASW data obtained near the free-field instrument (Rodriguez-Ordenez, 1994). V_s data at greater depth were estimated using seismic refraction data from Exposition Park (Duke and Leeds, 1962).

Instrument Selection and Free-Field Conditions

Sensors on the west tower were used in system identification analyses. The two vertical instruments at the base of the tower were used to quantify EW rocking. The specific channels used are indicated in Table A23.1; acceleration time histories recorded by these and other sensors are shown in Fig. A23.NR1. No digitized records were available for the 1st Floor sensors.

Table A23.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 4	Ch 3	Ch1,2	Ch 13	Ch 12

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch3} + (\text{Ch2} - \text{Ch1}) \cdot 85.5/11.67 \quad (1)$$

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

B/FF transmissibility functions (Fig. A23.NR4) and the spectra for horizontal motions (Fig. A23.NR2-3) indicate no consistent trend of amplification or de-amplification of basement motions relative to the FF, though significant amplification of EW basement motions occurred from about 4 to 8 Hz.

Transmissibility functions for B/FF vertical pairs (Fig. A23.NR5) indicate modest amplification near the first-mode frequency for both directions, suggesting that base rocking may have affected the tower response. A direct examination of rocking contributions to roof motion in the EW direction is made in Fig. A23.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements and have about 10-20% of the roof displacement amplitude, indicating that base rocking influenced the roof motions.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A23.NR4), (2) modal parameters from CEM analyses (Table A23.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A23.NR6).

Table A23.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (EW) d=3, j=4		L (NS) estimated		tr (EW) d=3, j=4		L (NS) d=3, j=4		tr (EW) d=2, j=4		L (NS) d=3, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.24	6.2	na	na	1.12	6.4	1.16	2.6	1.13	5.6	1.18	3.3
	2	3.93	9.5			3.86	8.3	3.97	6.7	3.83	9.1	3.80	6.1
	3	6.51	9.9			6.35	8.3	6.64	10.1	6.04	11.8	6.04	12.0

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- Fixed-base parameter estimation procedures could not be invoked for the longitudinal direction because $\tilde{f}_1 > \tilde{f}_1^*$.
- For the transverse direction, the modest difference between the fixed- and pseudo flexible-base parameters is consistent with the rocking effect noted in Part (a).

The RPEM results (Fig. A23.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and B-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A23.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 31 feet using the recorded free-field motions and the profile in Fig. A23.3. The foundation of the building's west tower was assumed to be independent of the remainder of the building for the calculation of foundation radii. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A23.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model=H	$V_s=500$ fps	$h=56$ ft	emb.=14 ft	$\gamma = 0.15$
NR	$\nu=0.33$		tr:	$r_1 = 21.4$ ft	$r_2 = 20.6$ ft
	$\beta^*=9.3\%$	$V_s^*=305$ fps	L:	$r_1 = 21.4$ ft	$r_2 = 22.7$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A23.3 with the fixed-base modal parameters in Table A23.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A23.4.

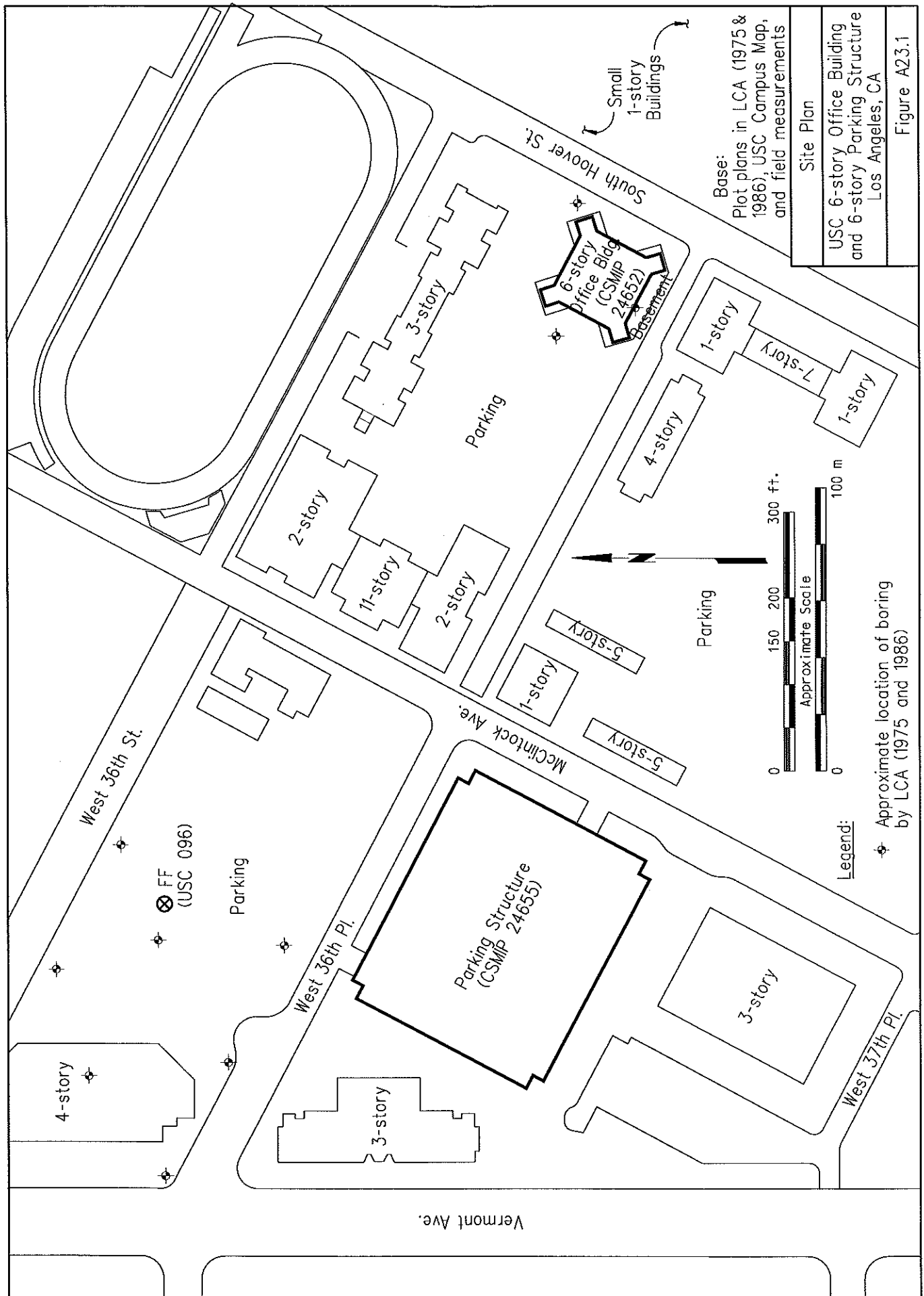
Table A23.4: Empirical and predicted inertial interaction effects

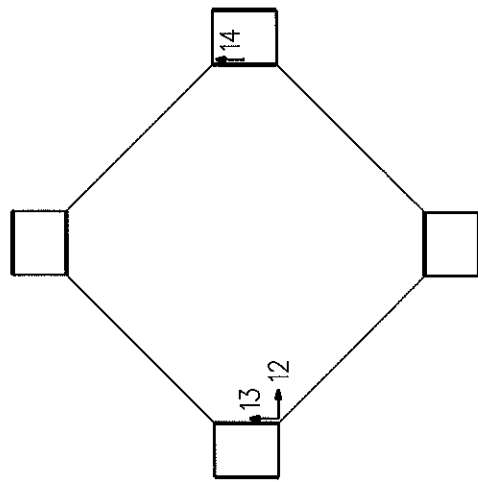
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.10	0.9	5.6	--	--	--
	Pre.-Vel.	1.17	1.0	4.9	--	--	--
	Pre.-Bie.	1.26	7.1	10.3	--	--	--

The period lengthening and foundation damping were satisfactorily predicted using the "modified Veletsos" formulation. The effects of SSI were significantly overpredicted by the "modified Bielak" formulation.

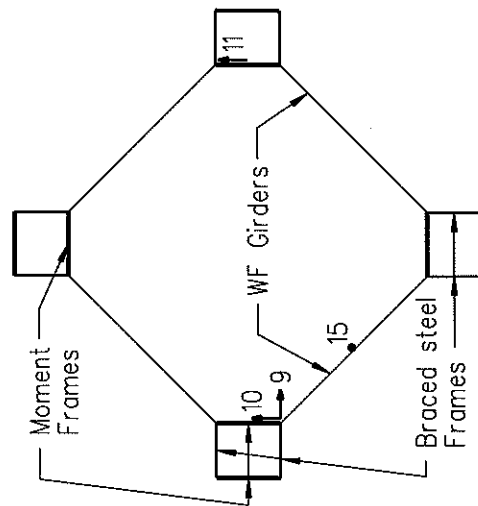
Data Sources

- Duke, C.M. and Leeds, D.J. (1962). "Site characteristics of Southern California strong motion earthquake stations," *Rpt. No. 62-55*, Dept. of Engrg., Univ. of California, Los Angeles.
- LeRoy Crandall Assoc., LCA (1975). "Report of Foundation Investigation, Proposed Parking Structure, Vermont Avenue and 36th Place, Los Angeles, CA." *Job. No. A-74207*.
- LeRoy Crandall Assoc., LCA (1986). "Report of Foundation Investigation, Proposed Hedco Neuroscience Building, Hoover Street and Downey Way (36th Place), Los Angeles, CA," *Job. No. A-86258*.
- Rodriguez-Ordonez, J.A. (1994). "A new method for interpretation of surface wave measurements in soils," *Ph.D. Dissertation*, North Carolina State Univ., Raleigh.

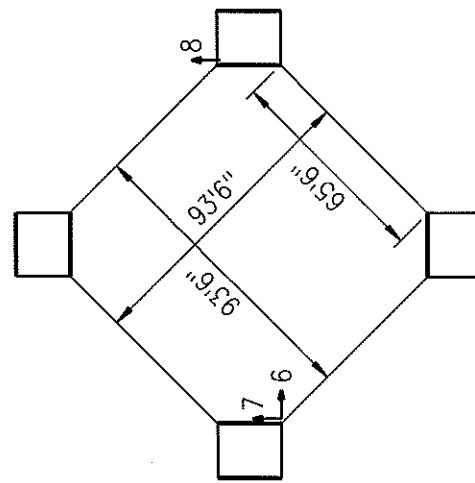




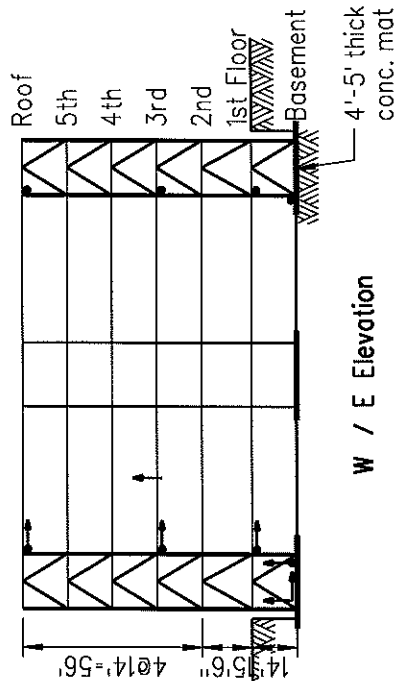
Roof Plan



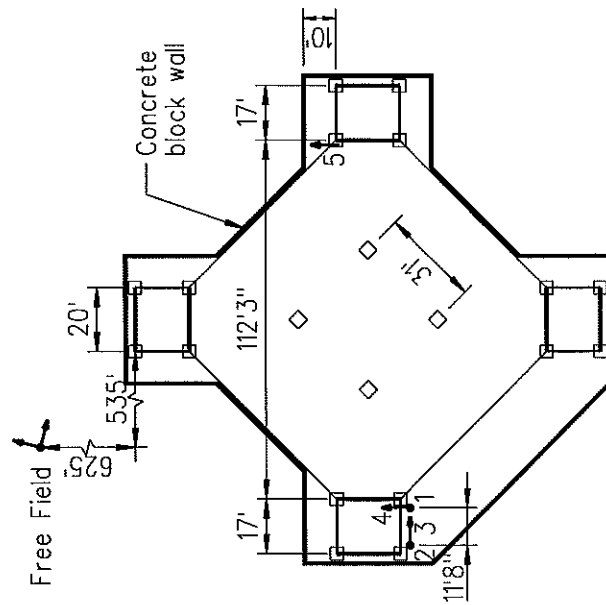
3rd Floor Plan



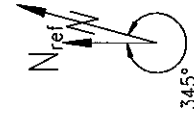
1st Floor Plan



W / E Elevation



Basement Plan

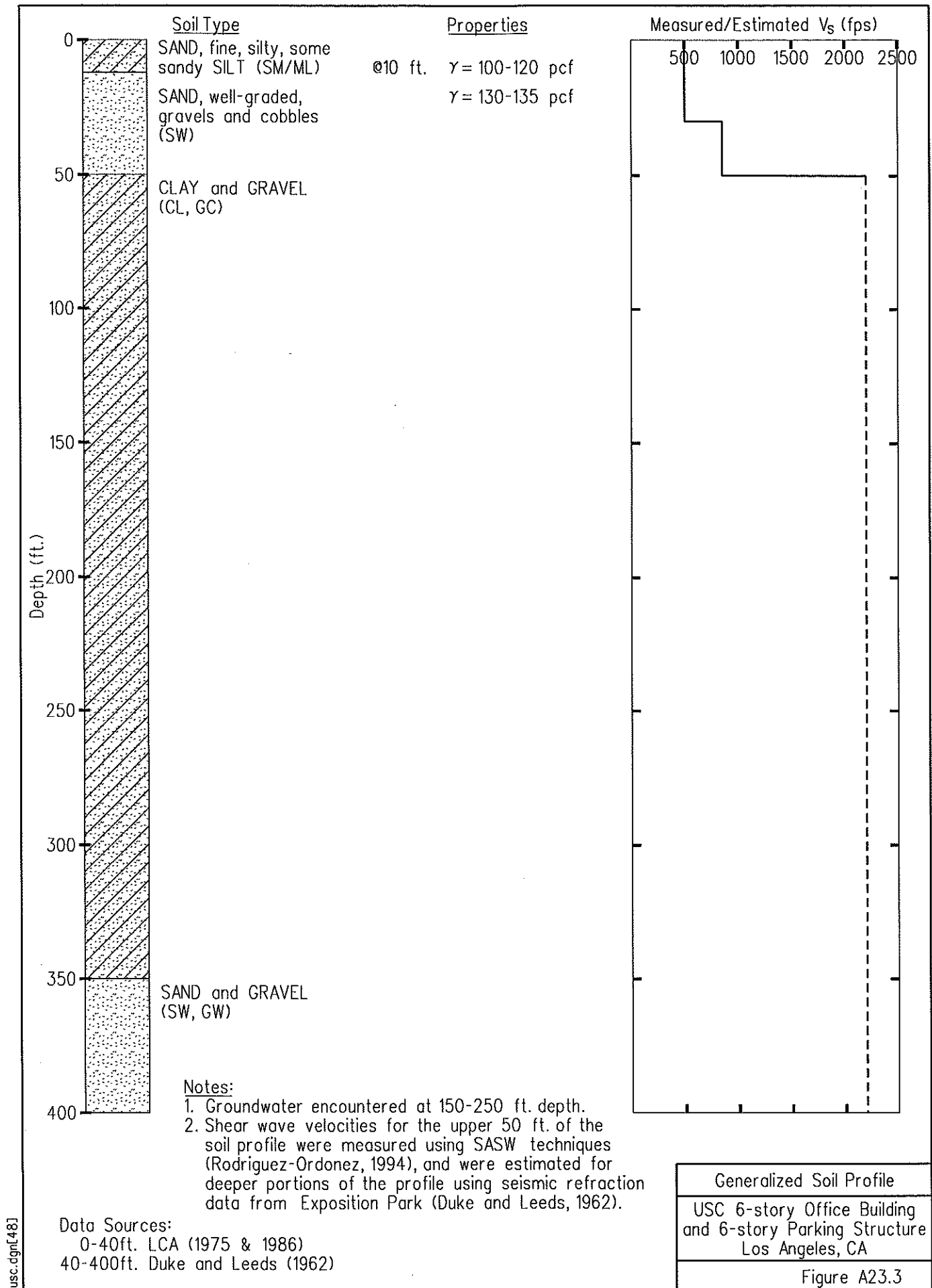


Note: Data provided by CSMIP

Sensor Locations

USC 6-story Office Building
Los Angeles, CA

Figure A23.2



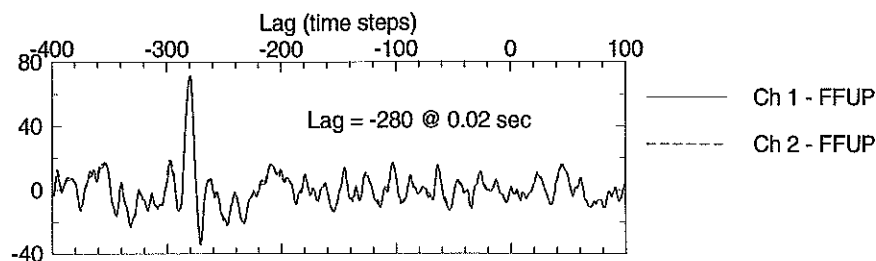
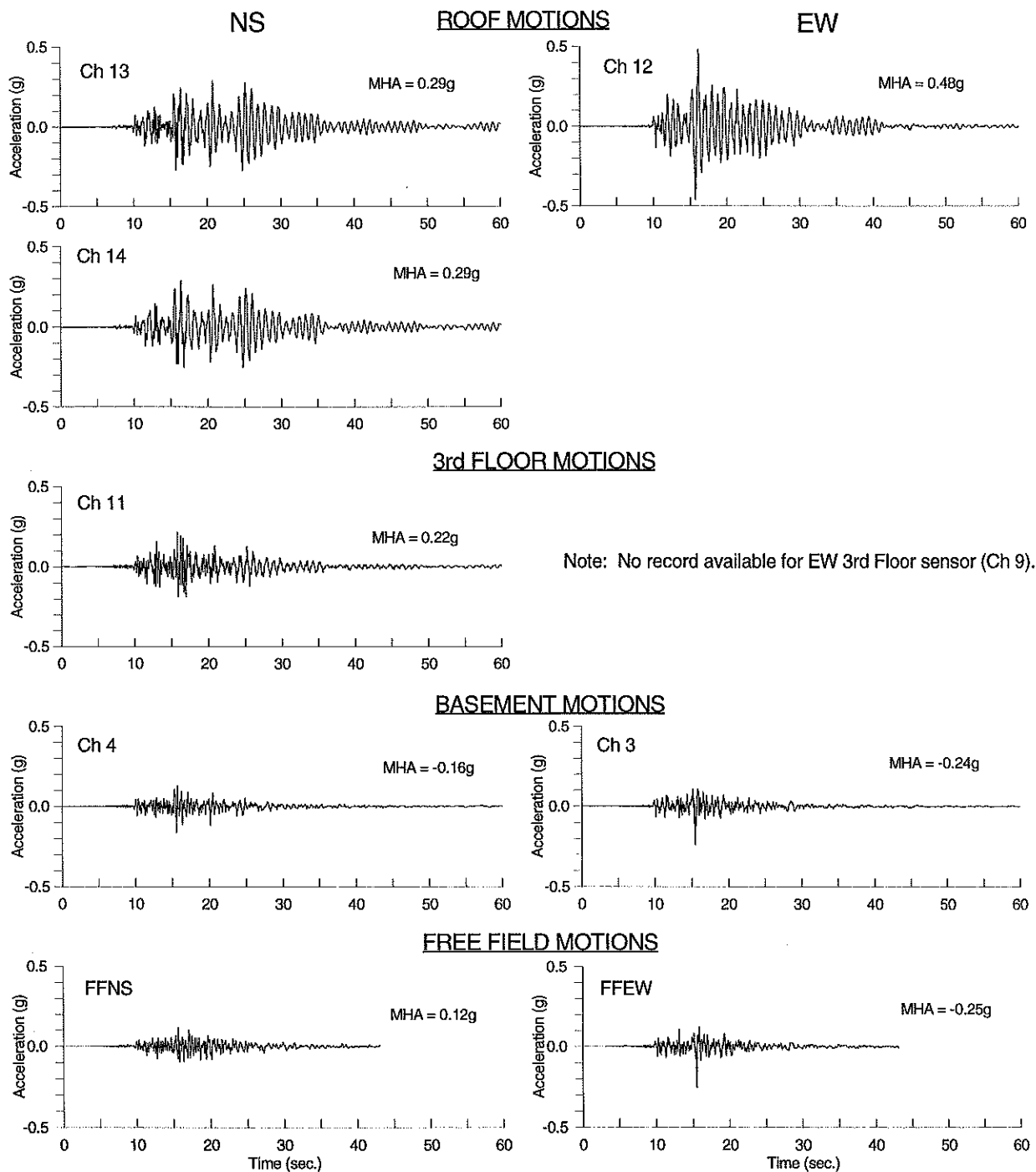


Fig. A23.NR1: LA 6 STORY OFFICE BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A23-8

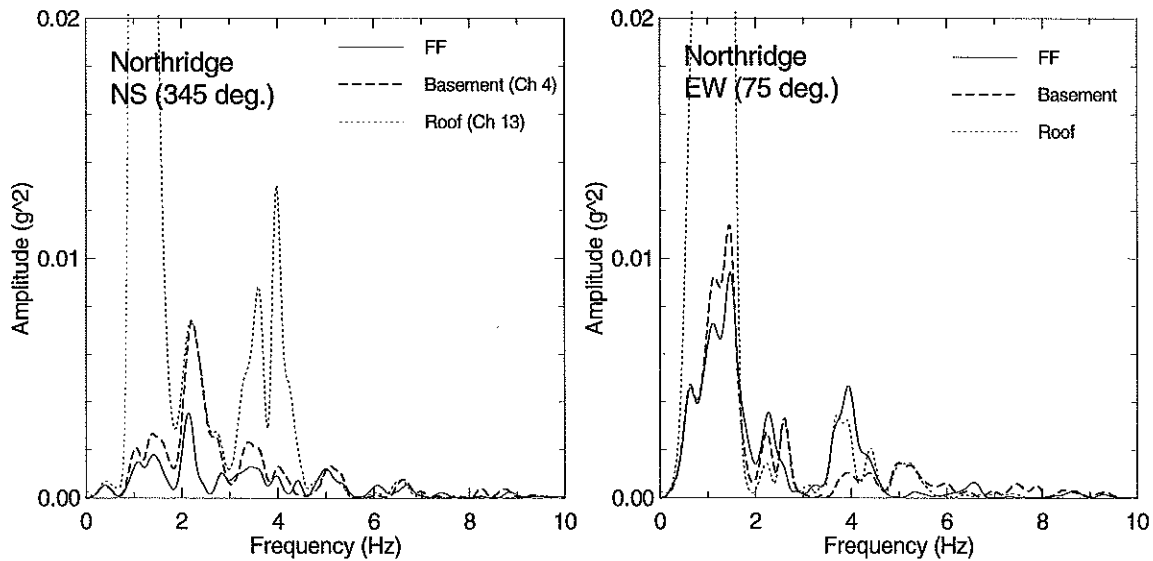


Fig. A23.NR2: LA 6 STORY OFFICE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

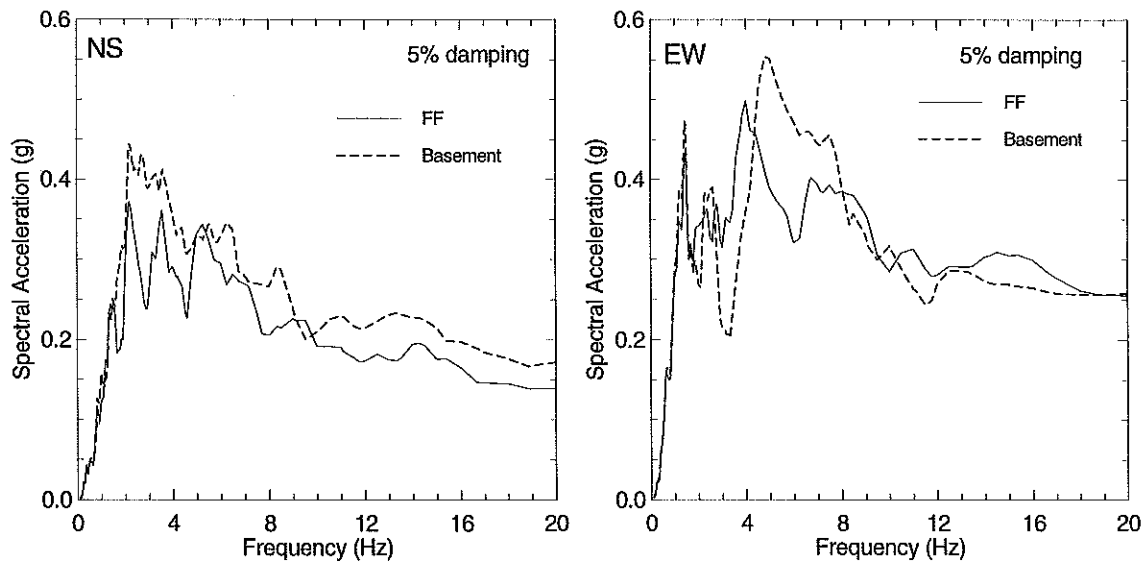


Fig. A23.NR3: LA 6 STORY OFFICE BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

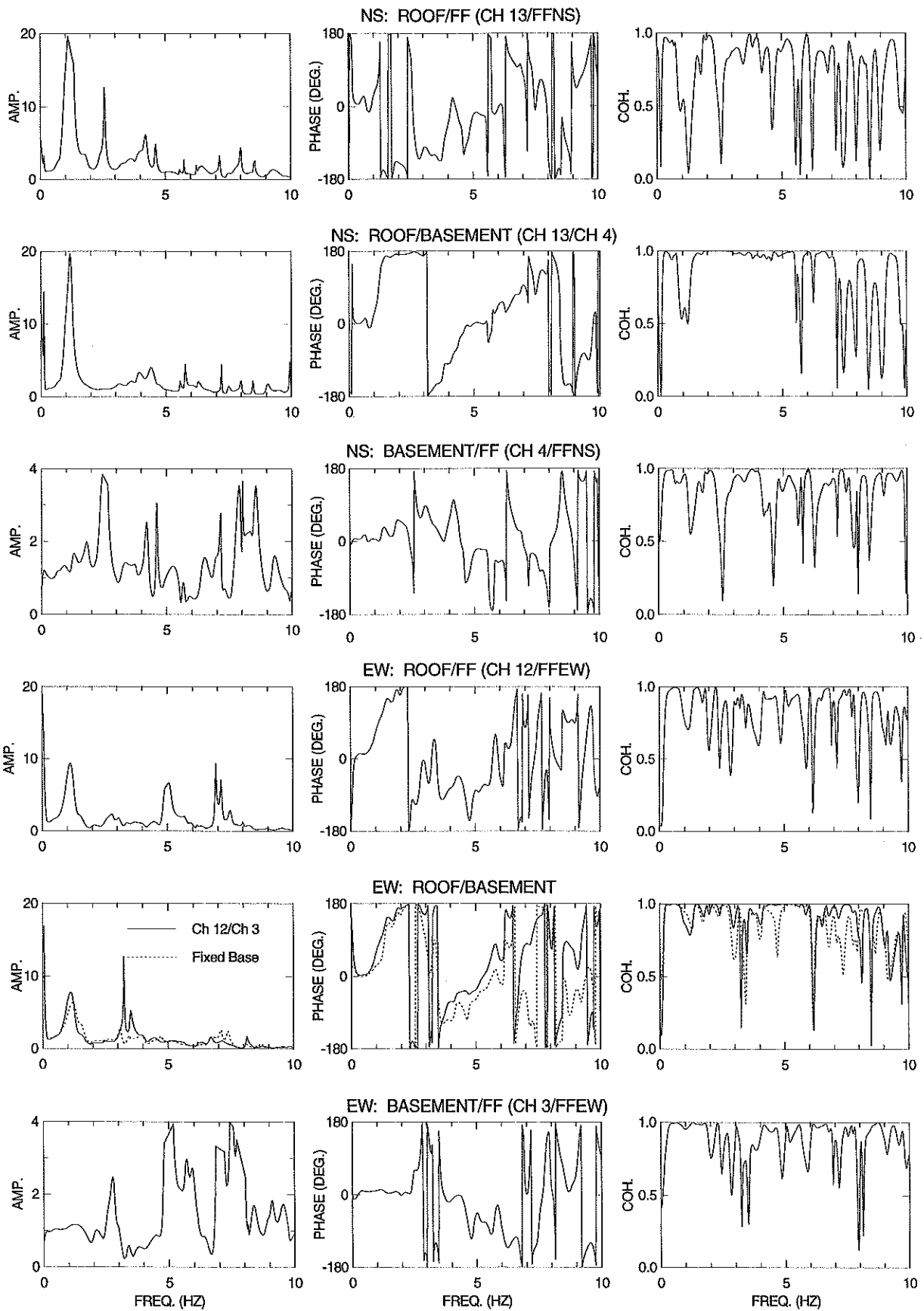


Fig. A23.NR4: LA 6 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A23-10

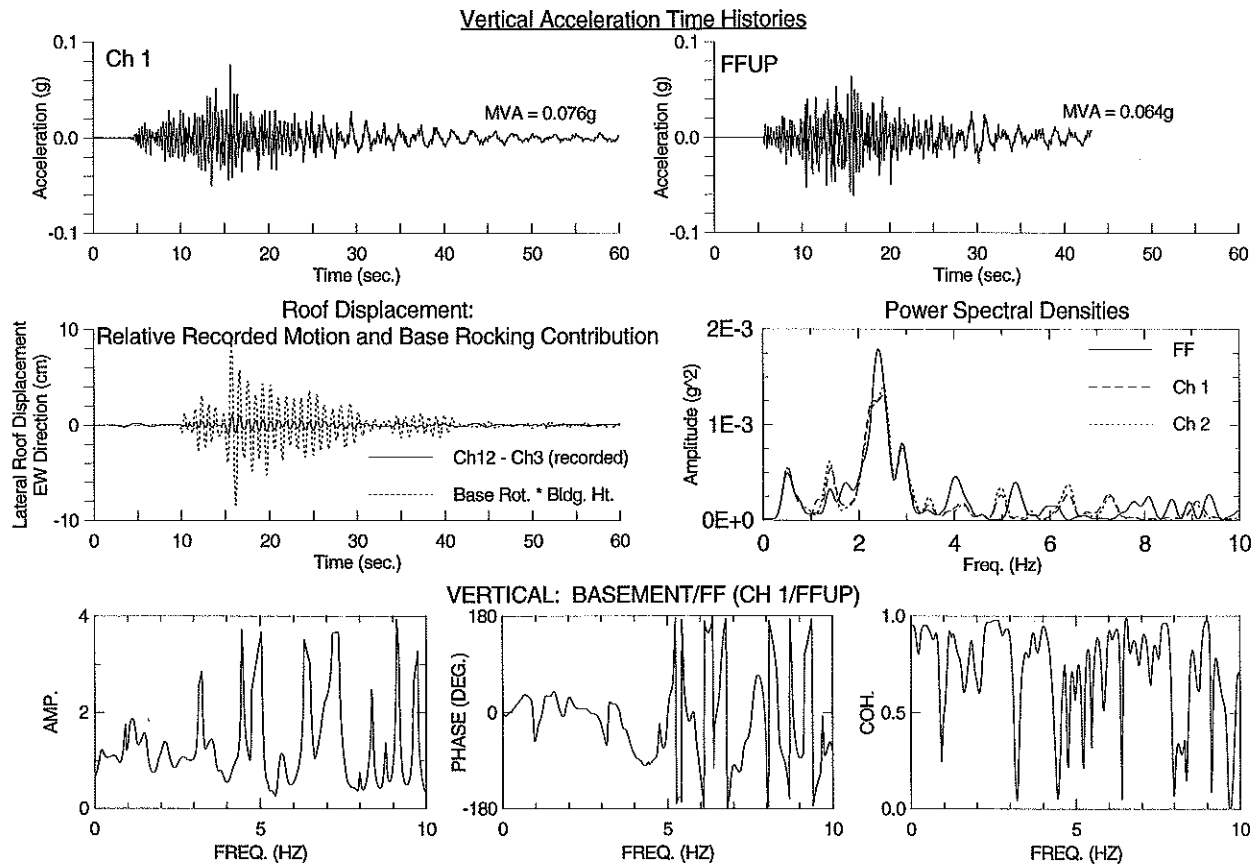


Fig. A23.NR5: LA 6 STORY OFFICE BUILDING
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

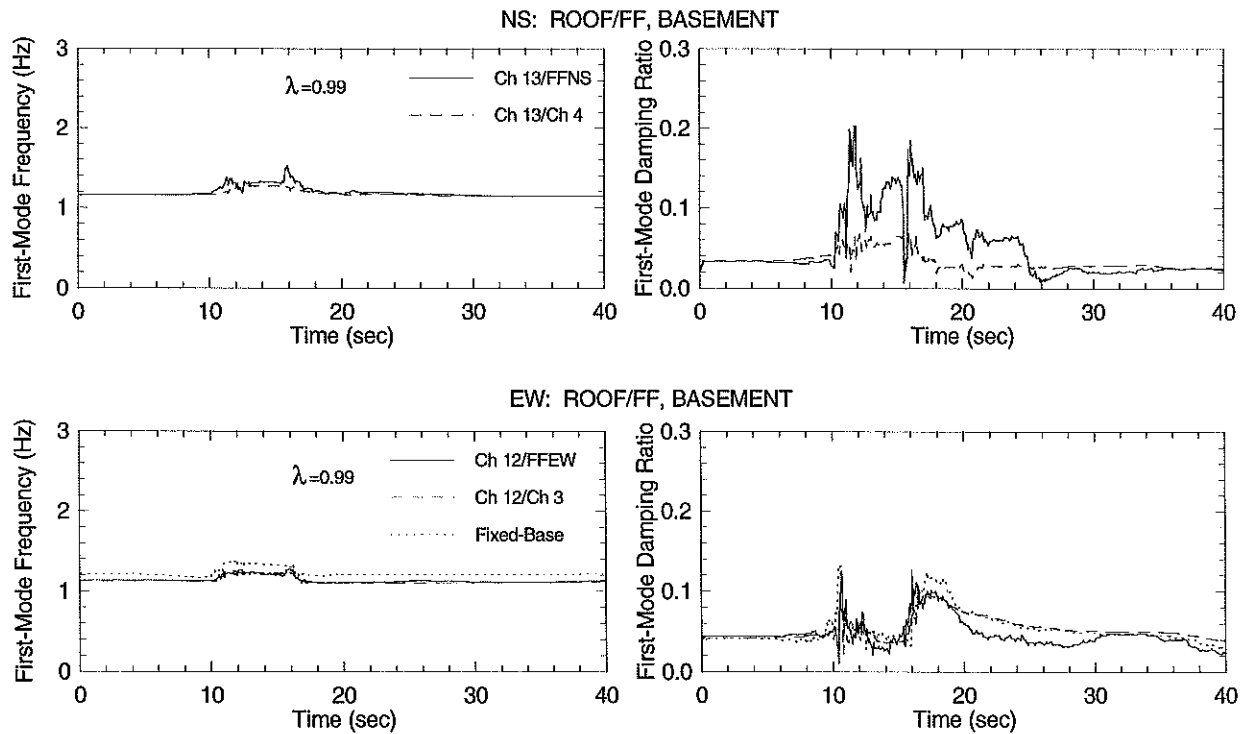


Fig. A23.NR6: LA 6 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake
A23-11

SITE A24

Site Name: LA 6 Story Parking Garage
Sensors: CSMIP 24655, USC 096

Lat: 34.021°
Long.: 118.289°

Floor Levels: 6
Bsmt. Levels: 0
Structure: Concrete slabs and shear walls

Foundation: belled piers
Site Geology: soil
Design Date: 1978

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.22	0.15	0.064	0.28	0.15	0.21	0.57	0.31	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

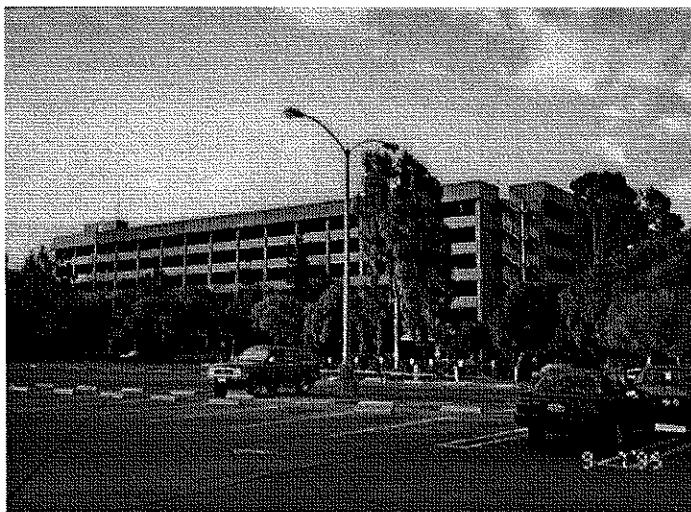
Preliminary: CSMIP (str.), USC (FF); filtered $f = 0.2$ to 46 Hz (str.), 0.14-0.28 to 46 Hz (FF, different high-pass frequency for each channel); $f_{Nyq} = 50$ Hz, 25 Hz (str., FF); FF/str. data not synchronized; FF not aligned with str.

Note: The channel directions in the original USC FF data were mislabeled; 'Up' and 'N90E' were switched, 'NOE' is correct

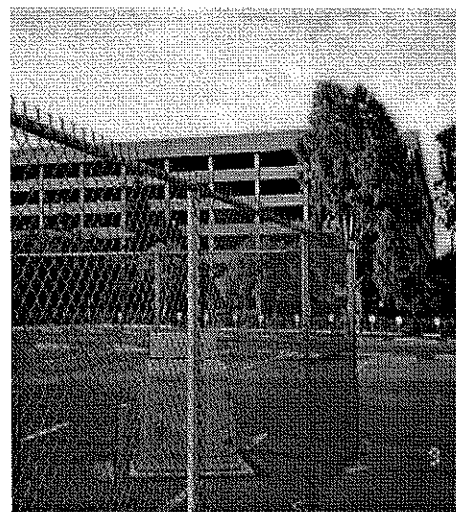
Final: FF data rotated 330° CCW to align with str.; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A24.NR0); Str. data decimated by 2 to $f_{Nyq} = 25$ Hz.

Figures

A23.1, A23.3 Site Plan, Generalized Soil Column
A24.1 Sensor Locations
A24.NR0-6 NR Analysis Results



View of structure, looking south



View of FF, north-west corner of structure in background

DISCUSSION

Site Description

The structure has 6-levels with no basement, and has a rectangular shape in plan (259 by 306 feet). Several one to five-story buildings are located within about 80 feet of the structure. The lateral load resisting system consists of six concrete shear walls in the transverse (NS) direction and two in the longitudinal (EW) direction. The building is supported on a pier and grade beam foundation with concrete belled piers generally expected to be about 15 to 25 feet deep (specific depths are not given on the building plans). A 4-inch thick slab-on-grade is present across the first floor level. As shown in Fig. A24.2, accelerographs are present on the 1st, 4th, and roof levels of the structure. The free-field instrument is located in a parking lot about 240 feet northwest of the structure. Soil conditions at the site are described in the discussion for Site A23.

Instrument Selection and Free-Field Conditions

Accelerographs near shear walls were selected for the system identification analyses, as indicated in Table A24.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A24.NR1. Two vertical instruments at the base of an EW shear wall were used to evaluate base rocking of the wall.

Table A24.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 5	Ch1,2	Ch 9	Ch 12

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch5} + (\text{Ch1} - \text{Ch2}) \cdot 61/72 \quad (1)$$

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

- (a) Comparison of 1st Floor and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

1F/FF transmissibility functions (Fig. A24.NR4) and the spectra for horizontal motions (Fig. A24.NR2-3) indicate modest amplification of 1F motions relative to the FF for frequencies less than about 6 Hz (including the first-mode frequency), and de-amplification at higher frequencies.

Transmissibility functions for 1F/FF vertical pairs (Fig. A24.NR5) indicate significant amplification from $f \approx 2$ to 4 Hz, suggesting that base rocking of the wall likely contributed to the structural response. A direct examination of rocking contributions to roof motion is made in Fig. A24.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements and have about 60-70% of the roof displacement magnitude, indicating that base rocking strongly influenced the lateral response of the wall.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A24.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A24.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A24.NR6).

Table A24.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) d=0, j=4		tr (NS) d=1, j=4		L (EW) d=1, j=4		tr (NS) d=2, j=4		L (EW) d=1, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.98	5.2	3.63	6.5	1.97	5.3	2.50	5.4	1.91	6.5	2.26	6.2
	2			6.92	7.1	7.56	9.7	7.28	7.8	7.52	11.3	6.35	15.6

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- For the longitudinal direction, the large difference between the fixed- and pseudo flexible-base parameters is consistent with the significant rocking effect noted in Part (a).

The RPEM results (Fig. A24.NR6) indicate significant reductions in first-mode frequency for $t < 18$ seconds in both directions. The drops for the NS direction are acute at 10 and 16 seconds. The times of these drops in system frequency correspond closely with large increases in the amplitude of shaking in the structure.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A24.3. Two soil models were considered, a halfspace and a 50-foot layer overlying a rigid base. For the halfspace soil model, the strain-dependent soil properties were estimated from deconvolution analyses to a depth of 159 feet using the recorded free-field motions and the profile in Fig. A23.3. For the 50-foot soil layer model, the deconvolution was only carried down to a depth of 50 feet.

For the transverse (NS) direction, the entire footprint area of the foundation was used to calculate foundation radii because the lateral force resisting elements are located along the building perimeter. Further, as the foundation radius ($r_1 = 159$ feet) significantly exceeds the 50-foot finite soil layer thickness, only the halfspace soil model was used (the full foundation area and halfspace soil model were also used for estimating the fixed-base parameters in Table A24.2). For the longitudinal (EW) direction, only the foundation area enclosed by the central core walls was used because (1) these walls comprise the lateral force resisting system for the EW direction, and (2) the foundation system is not expected to be particularly stiff beyond the walls, and hence is unlikely to transmit significant loading beyond the area enclosed by the walls. Calculations for the EW direction were made with both soil models (halfspace and 50-foot soil layer). No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A24.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model=H (1) 50-ft layer(2) $v=0.33$	$V_{S1}=1200$ ft/s $V_{S2}=600$ ft/s	$h=40$ ft	$emb.=0$	$\gamma = 0.15$
NR	$\beta_1^*=9.3\%$	$V_{S1}^*=860$ ft/s	tr: $r_1 = 159$ ft		$r_2 = 154$ ft
	$\beta_2^*=10.7\%$	$V_{S2}^*=400$ ft/s	L (core): $r_1 = 54$ ft		$r_2 = 47$ ft

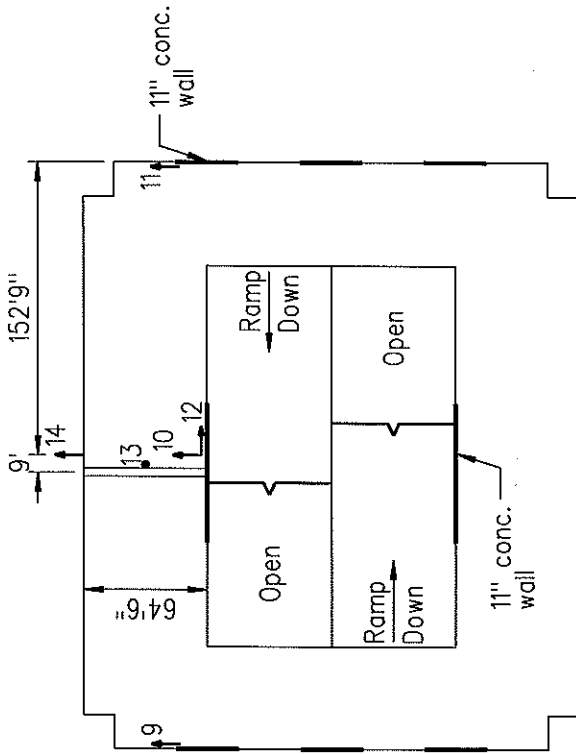
* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A24.3 with the fixed-base modal parameters in Table A24.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A24.4.

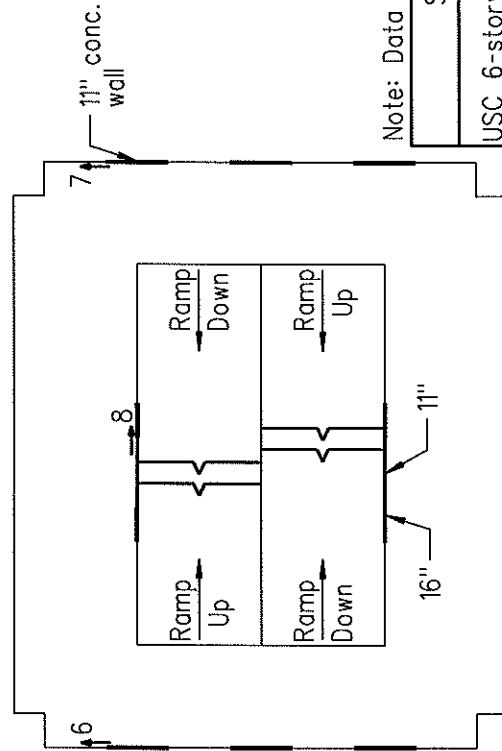
Table A24.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.04	1.9	6.5	1.60	4.6	6.2
	Pre.-H	1.08	5.3	9.4	1.16	4.6	8.8
	Pre.-50' L	--	--	--	1.55	7.2	8.9

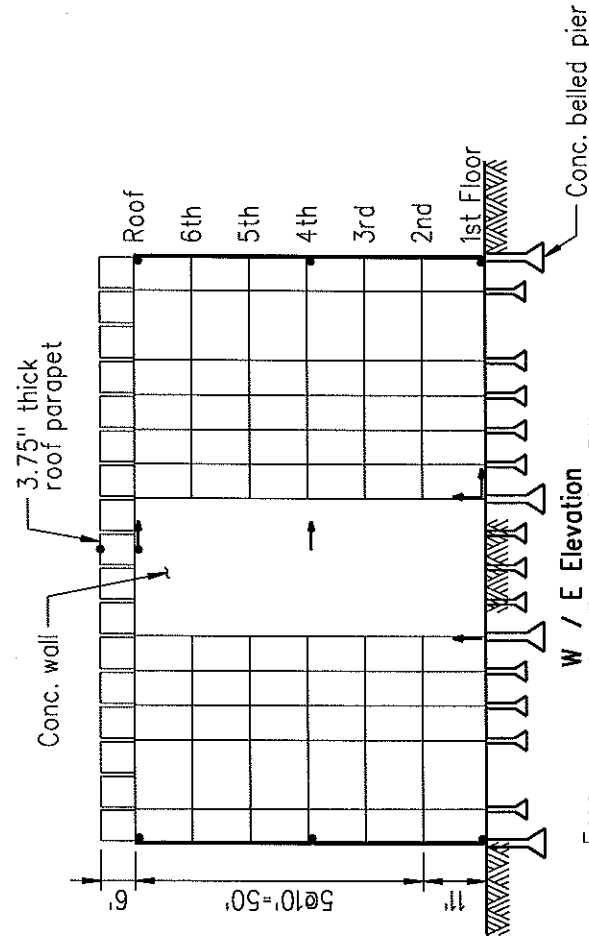
The period lengthening and foundation damping for the transverse direction were somewhat overpredicted. For the longitudinal direction, the 50-foot soil layer model provided a better prediction of period lengthening, while the halfspace model provided a better foundation damping prediction. Based on these results, it appears that the stiff soil at 50-foot depth affects the foundation stiffness, but that the impedance contrast is not so great as to preclude radiation damping for frequencies greater than the layer frequency. Nonetheless, the 50-foot layer appears to provide the best overall model of the foundation impedance.



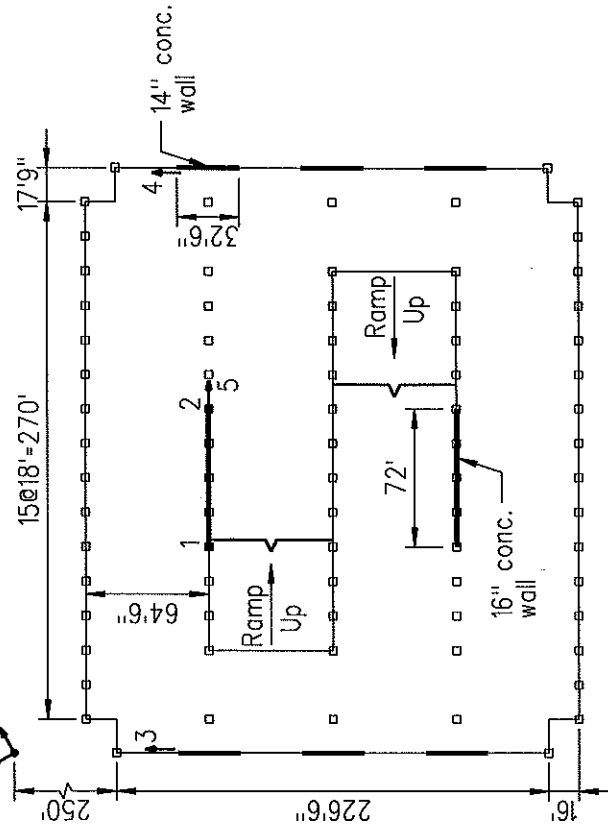
Roof Plan



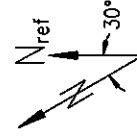
4th Floor Plan



W / E Elevation
(Vertical Exaggeration 3X)



1st Floor Plan



Note: Data provided by CSMIP

Site Plan

USC 6-story Parking Structure
Los Angeles, CA

Figure A24.1

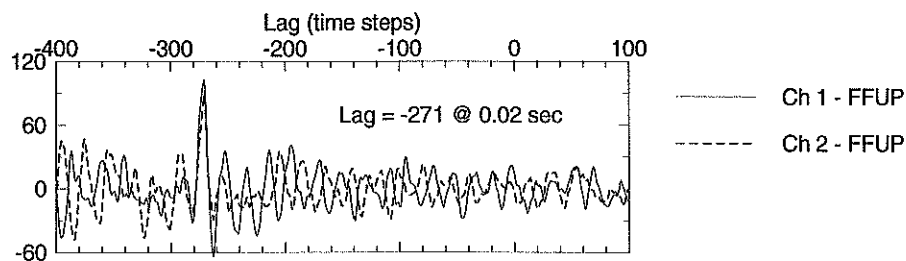


Fig. A24.NR0: Cross-correlation function between vertical 1st floor and free-field accelerations

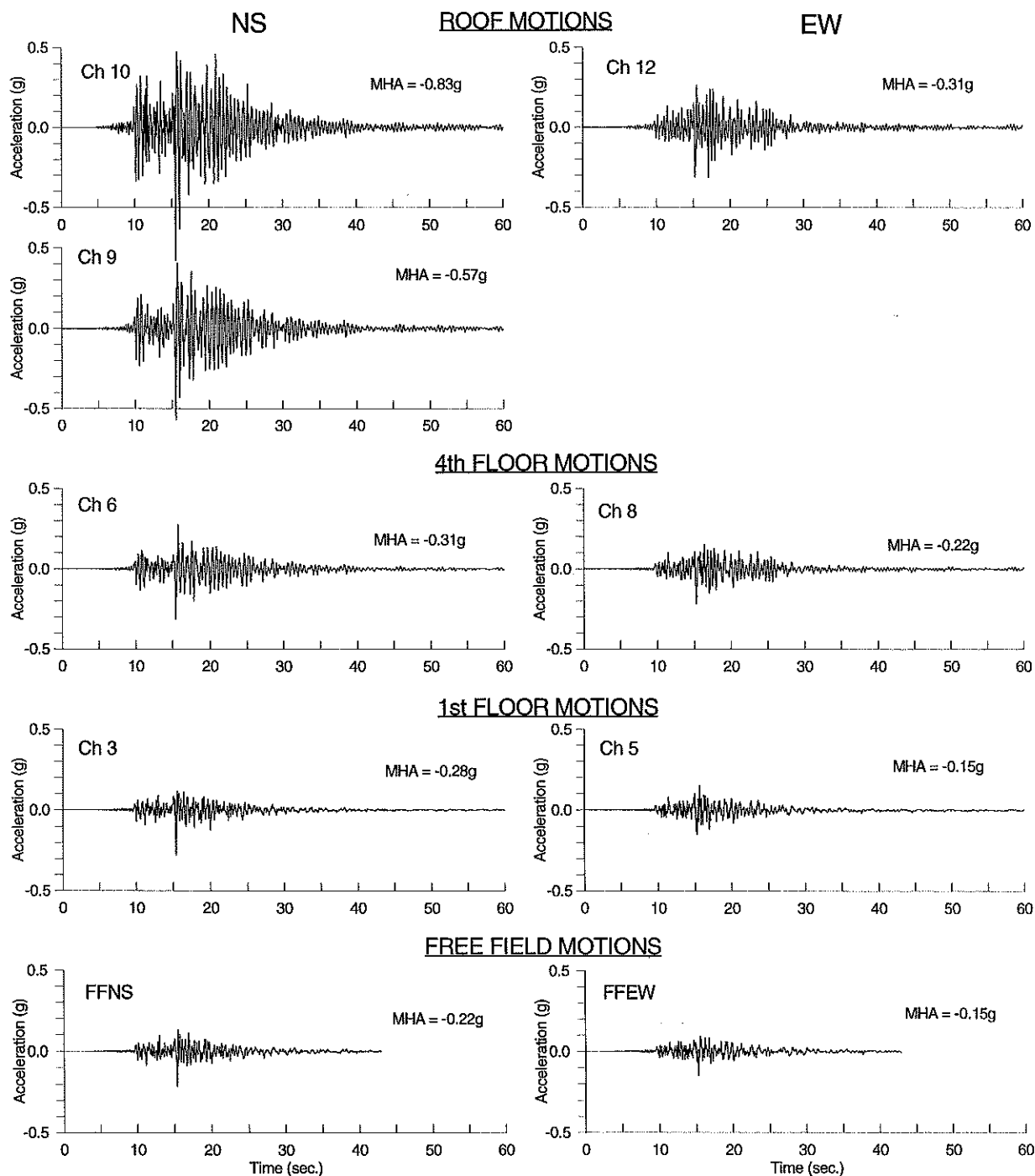


Fig. A24.NR1: LA 6 STORY PARKING STRUCTURE
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A24-6

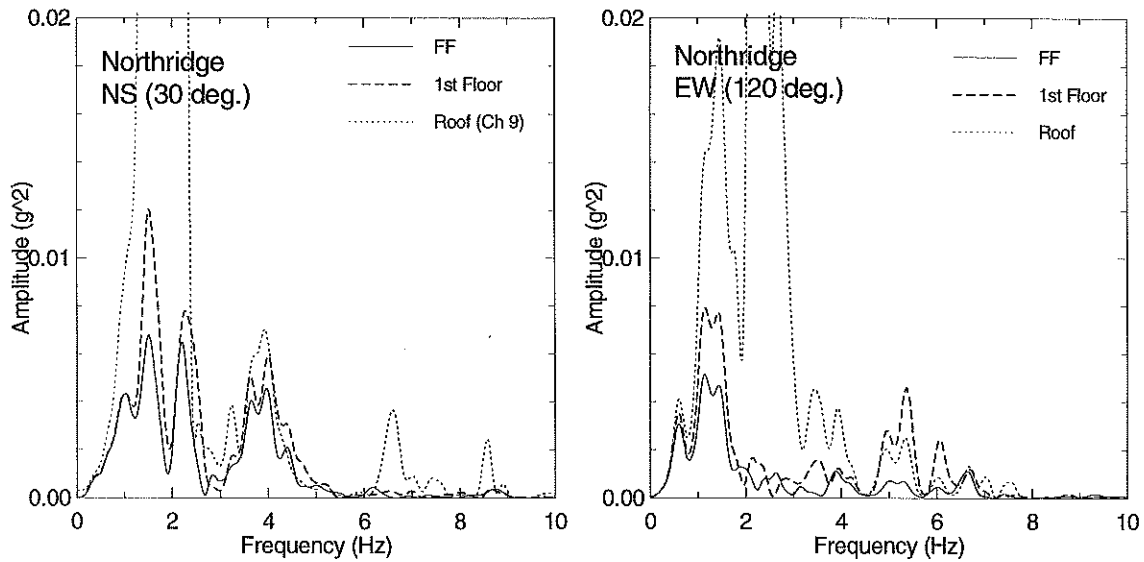


Fig. A24.NR2: LA 6 STORY PARKING STRUCTURE
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

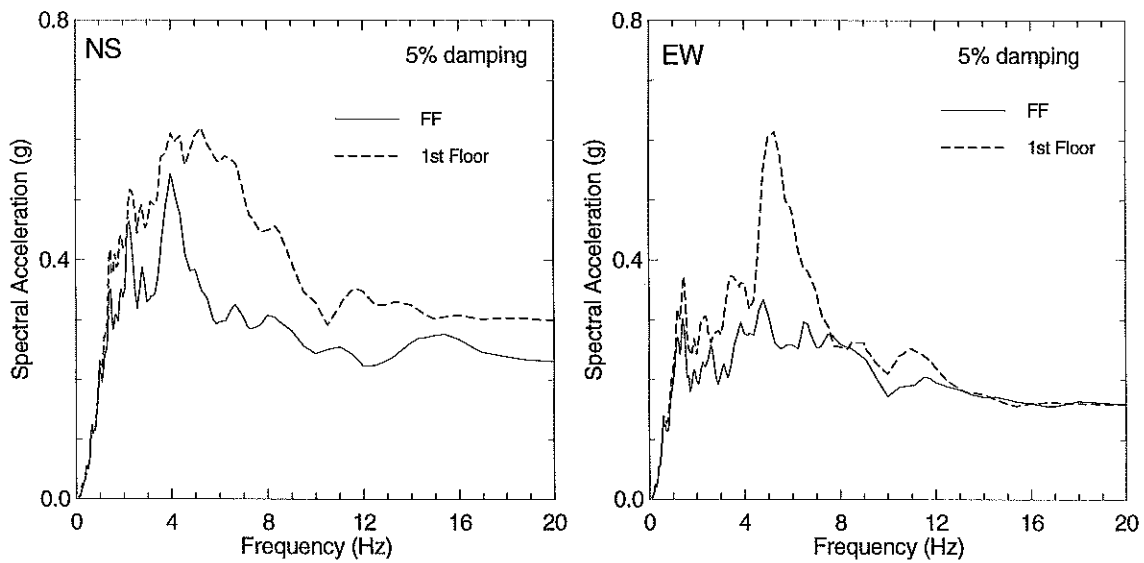


Fig. A24.NR3: LA 6 STORY PARKING STRUCTURE
Acceleration Response Spectra of 1st Floor and Free-Field Motions
1994 Northridge Earthquake

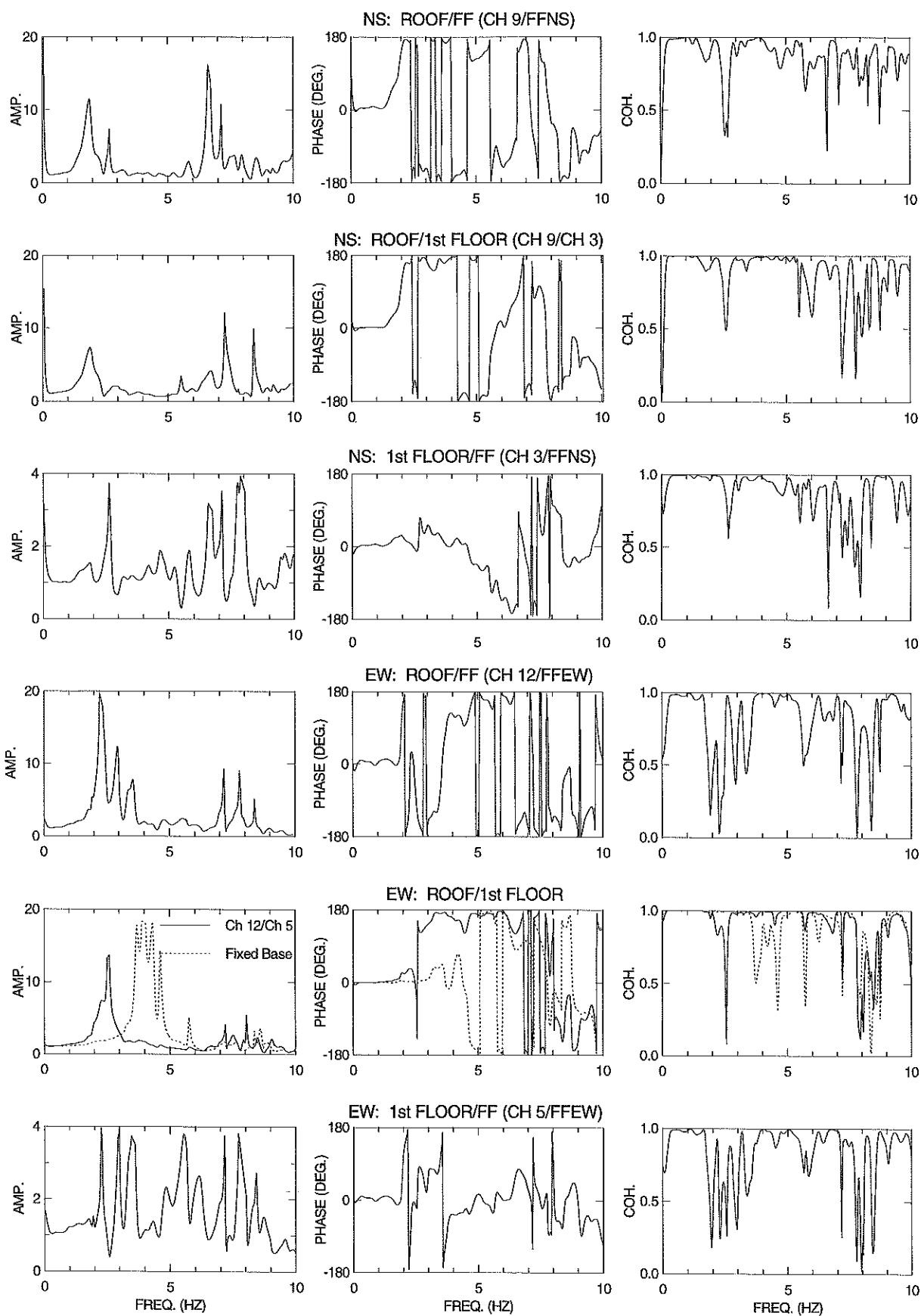


Fig.A24.NR4: LA 6 STORY PARKING STRUCTURE
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A24-8

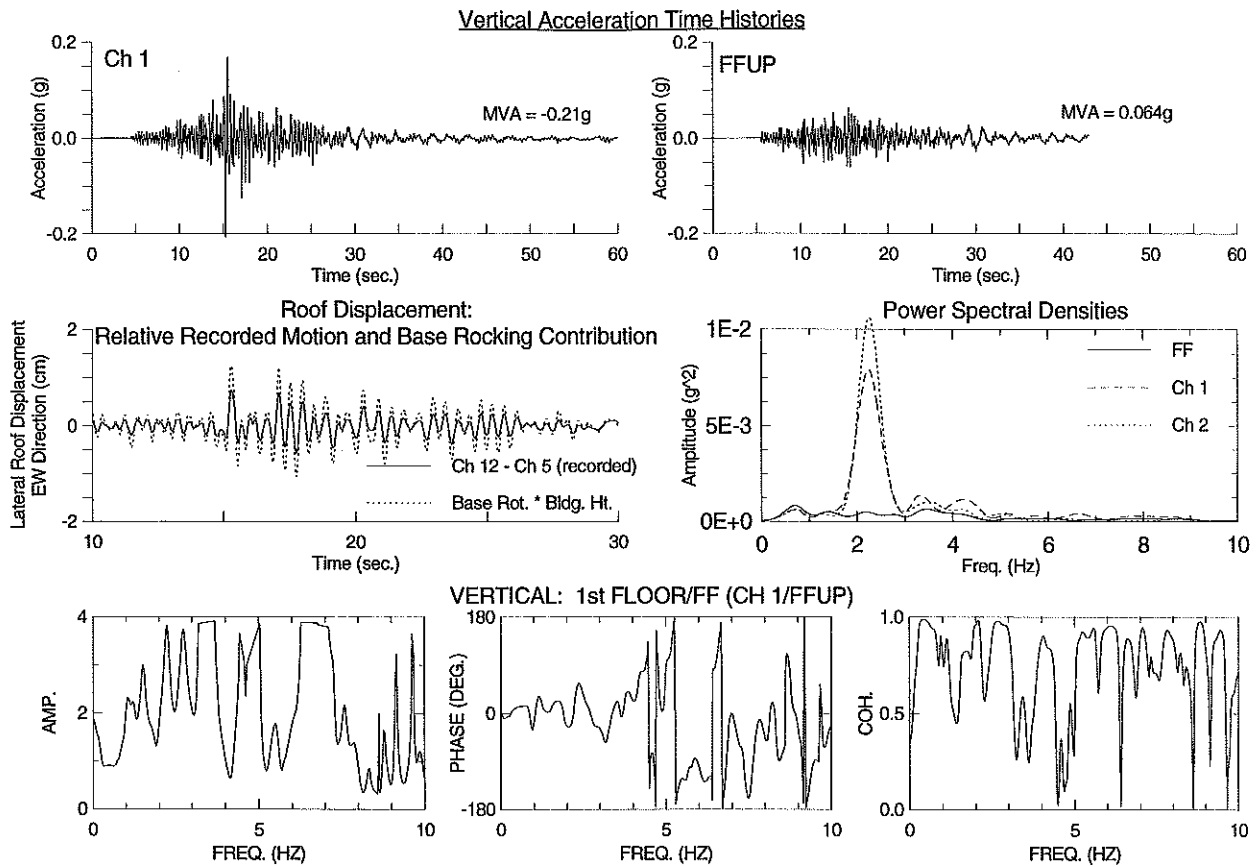


Fig. A24.NR5: LA 6 STORY PARKING STRUCTURE
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

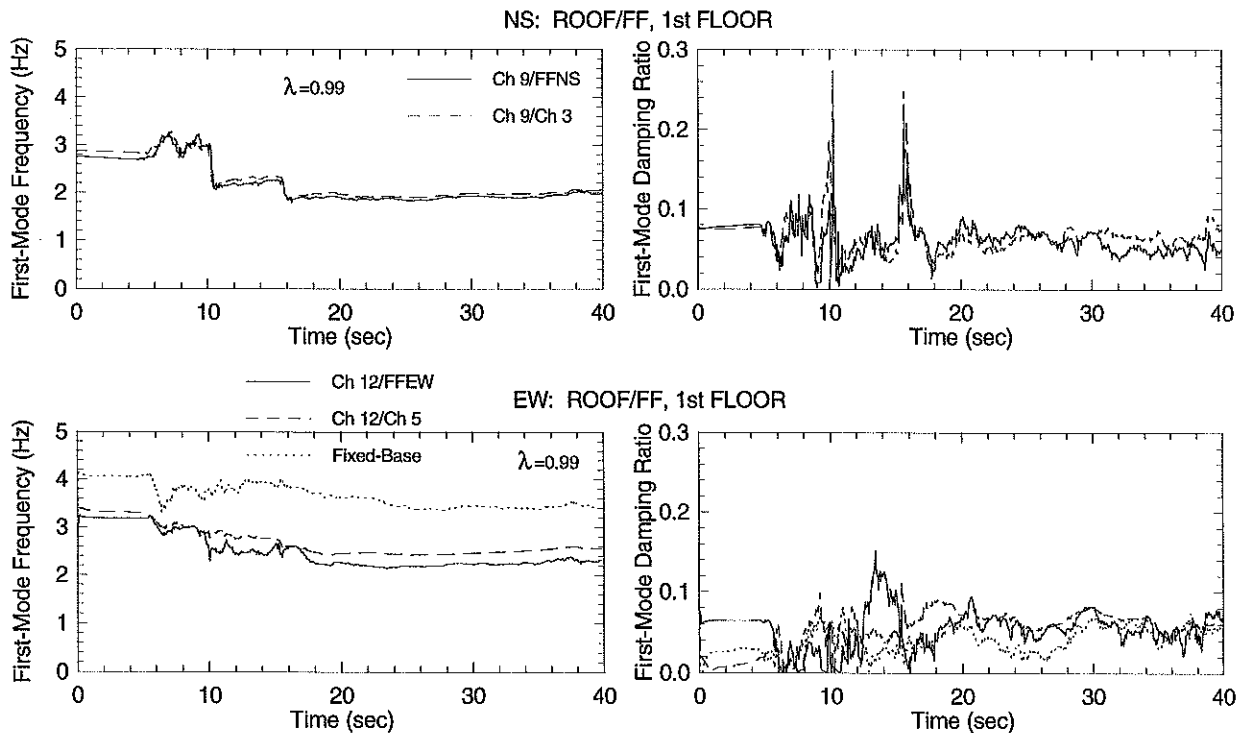


Fig. A24.NR6: LA 6 STORY PARKING STRUCTURE
Time Variation of First-Mode Parameters
1994 Northridge Earthquake
A24-9

SITE A25

Site Name: LA 7 Story USC Hospital
Sensors: CSMIP 24605

Lat: 34.062°
Long.: 118.198°

Floor Levels: 8
Bsmt. Levels: 1
Structure: Isolators, braced steel frames

Foundation: footings
Site Geology: Rock
Design Date: 1988

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Foundation Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1991 Sierra Madre*	0.13	0.16	0.07	--	0.09	0.04	0.07	0.09	--
1992 Landers	0.041	0.043	0.021	0.035	0.027	--	0.089	0.067	--
1992 Big Bear*	0.06	0.03	0.02	0.05	0.02	0.02	0.06	0.05	--
1994 Northridge	0.49	0.21	0.12	0.37	0.16	0.065	0.21	0.16	--

*Not digitized

-- = No instrument, or instrument not installed

Preprocessing

Landers Earthquake (LD)

Preliminary: CSMIP; filtered, $f = 0.16\text{--}23\text{ Hz}$ (str.), $0.16\text{--}46\text{ Hz}$ (FF); $f_{Nyq} = 25\text{ Hz}$, 50 Hz (str., FF); FF/str. data synchronized.

Final: FF data decimated by 4, str. data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$ (str. and FF); $f_{Nyq} = 50\text{ Hz}$; FF/str. data not synchronized.

Final: decimated by 4 to $f_{Nyq} = 12.5\text{ Hz}$; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A25.NR0).

Figures

A25.1-A25.3	Site Plan, Sensor Locations, Generalized Soil Column
A25.LD1-5	LD Analysis Results
A25.NR0-6	NR Analysis Results



View of structure and parking area with FF instrument
(FF on hill near driveway on right side of photograph)

DISCUSSION

Site Description

The building is 7-stories tall with a partial basement, and has an irregular shape with maximum plan dimensions of 253 by 303 feet. No large buildings are located near the structure. The lateral load resisting system consists of diagonally braced perimeter steel frames, and lead-rubber and elastomeric base isolators below the lower level. The foundation consists of footings. As shown in Fig. A25.2, accelerographs are present at the foundation level (below the isolators), lower level (above the isolators), 4th and 6th story, and roof level of the structure. The free-field instrument is located in a parking lot about 350 feet southeast of the building.

The site is located on fairly level terrain, though the free-field instrument is on a small hill. Geologic exploration in the area consists of borings at the site by LCA (1987), seismic refraction measurements in Hazard Park (south of the site) by Duke et al. (1971), and borings about 1000 feet west of the building by LCA (1978). As shown in Fig. A25.3, geologic conditions consist of about 5 feet of silty clay overlying weathered siltstone and shale bedrock which extends to the maximum depth explored of 64 feet. Groundwater was encountered at 25 feet by LCA (1987). The shear wave velocities in Fig. A25.3 are based on in situ downhole measurements by LCA (1978) and the seismic refraction measurements in Hazard Park.

Instrument Selection and Free-Field Conditions

Instruments near the building centroid were selected for analysis (Table A25.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A25.**1. Two foundation level vertical instruments were used to evaluate transverse (EW) rocking from the NR data (vertical data was not recorded in the LD event).

Table A25.1: Sensors in structure used for analyses

Foundation Level (FL)			Lower Level (LL)		Roof (R)	
NS	EW	V	NS	EW	NS	EW
Ch 5	Ch 7	Ch1,3	Ch 9	Ch 11	Ch 21	Ch 23

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch7} + (\text{Ch1} - \text{Ch3}) \cdot 117/160 \quad (1)$$

The free-field instrument is located in a parking lot, and is unlikely to be significantly affected by vibrations from structures in the area. The slope of the minor hill on which the instrument is located is sufficiently gradual that topographic effects are unlikely to be significant.

Analysis Results

(a) Comparison of Foundation Level and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

FL/FF transmissibility functions (Figs. A25.**4) and the spectra for horizontal accelerations (Fig. A25.**2-3) generally indicate de-amplification of FL motions for $f > 1$ to 1.5 Hz.

Transmissibility functions for FL/FF pairs of vertical motions (Fig. A25.NR5) indicate minor amplification of FL motions near the first-mode frequency, although power spectra of vertical FL motions indicate that little energy is present at these low frequencies. A direct examination of rocking effects on roof motions is made in Fig. A25.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are negligibly small compared to the relative roof displacements, indicating that base rocking did not significantly influence the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A25.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A25.2), and (3) time variation of first-mode parameters from the RPEM, NR only (Fig. A25.NR6).

Table A25.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (FL-R)				Flex.-Base (FF-R)			
		tr (EW) -- d=5, j=8		L (NS) estimated		tr (EW) d=4, j=7 d=5, j=8		L (NS) d=3, j=7 d=4, j=7		tr (EW) d=4, j=7 d=2, j=8		L (NS) d=3, j=7 d=1, j=7	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	--	--	--	--	0.85	12.0	0.91	12.9	0.87	11.7	0.92	10.1
	2					2.04	10.0	2.16	10.1	2.02	6.1	2.11	7.2
NR	1	0.86	23.2	0.87	28.9	0.87	24.5	0.85	27.0	0.86	27.0	0.82	16.4
	2	1.93	10.7			1.90	11.8	2.05	10.5	1.89	11.0	1.97	6.9

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, significant structural responses occur in both directions in the first and second modes. The base isolators cause large FL-R first-mode damping ratios of 25 to 27% (NR) and 12 to 13% (LD).
- For the transverse direction, the negligible difference between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for base rocking noted in Part (a).
- Fixed-base parameter estimation procedures could not be invoked for the LD data because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the NR data, the fixed-base parameters can be approximated by the pseudo flexible-base parameters.
- The structural response above the isolators was much stiffer and more lightly damped than is reflected by the parameters in Table A25.2 (which include the effects of the isolators). For

comparison, first-mode parameters for LL-R response in the NR event were 1.14 Hz and 6.3% damping (tr) and 1.31 Hz and 1.7% damping (L).

RPEM analyses with NR data indicated a significant decrease in first-mode frequencies at about 16 to 20 seconds, which corresponds to the time of highest amplitude ground shaking. Damping was also high at these times, but decreased subsequently. These results reflect the dependence of isolator softening and damping on the amplitude of shaking.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A25.3. Strain-dependent soil properties were estimated from deconvolution analyses to a depth of 110 feet using the recorded free-field motions and the profile in Fig. A25.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A25.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1170$ fps	$h = 68$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.45$		tr: $r_1 = 110$ ft		$r_2 = 110$ ft
LD	$\beta^* = 1.2\%$	$V_s^* = 1150$ fps	L: $r_1 = 110$ ft		$r_2 = 110$ ft
NR	$\beta^* = 2.5\%$	$V_s^* = 1065$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A25.3 with the fixed-base modal parameters in Table A25.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A25.4.

Table A25.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LD	Sys. ID	0.97	-0.3	11.7	0.99	-2.8	10.1
	Prediction	1.01	0.2	11.8	1.01	0.3	12.7
NR	Sys. ID	1.01	4.6	27.0	1.06	-7.9	16.4
	Prediction	1.01	0.3	22.7	1.01	0.3	28.1

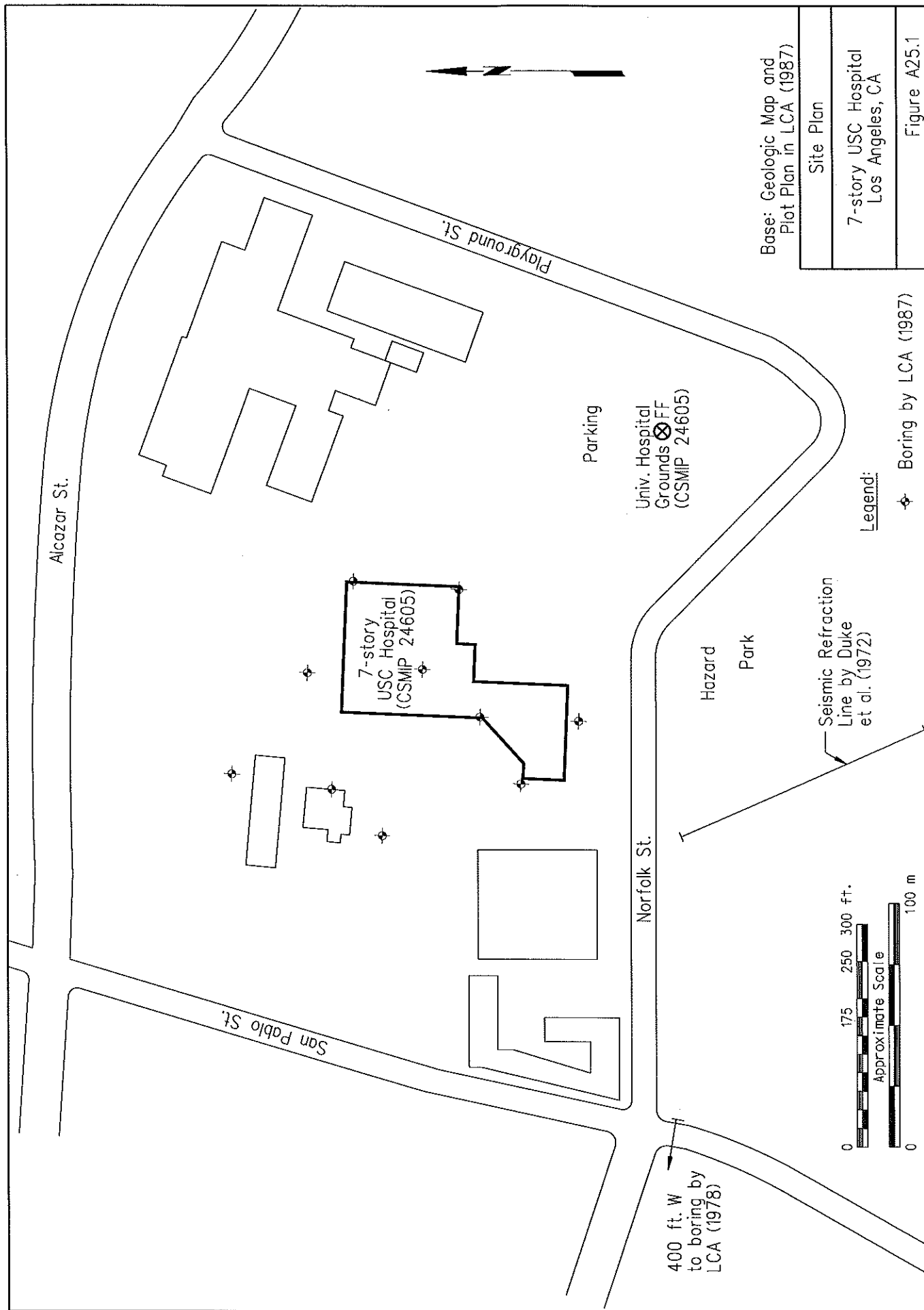
The negative values of foundation damping $\tilde{\zeta}_0$ from the system identification analyses indicate numerical errors in these results. Nonetheless, the small SSI effects at this site were reasonably well-predicted by the "modified Veletsos" analyses.

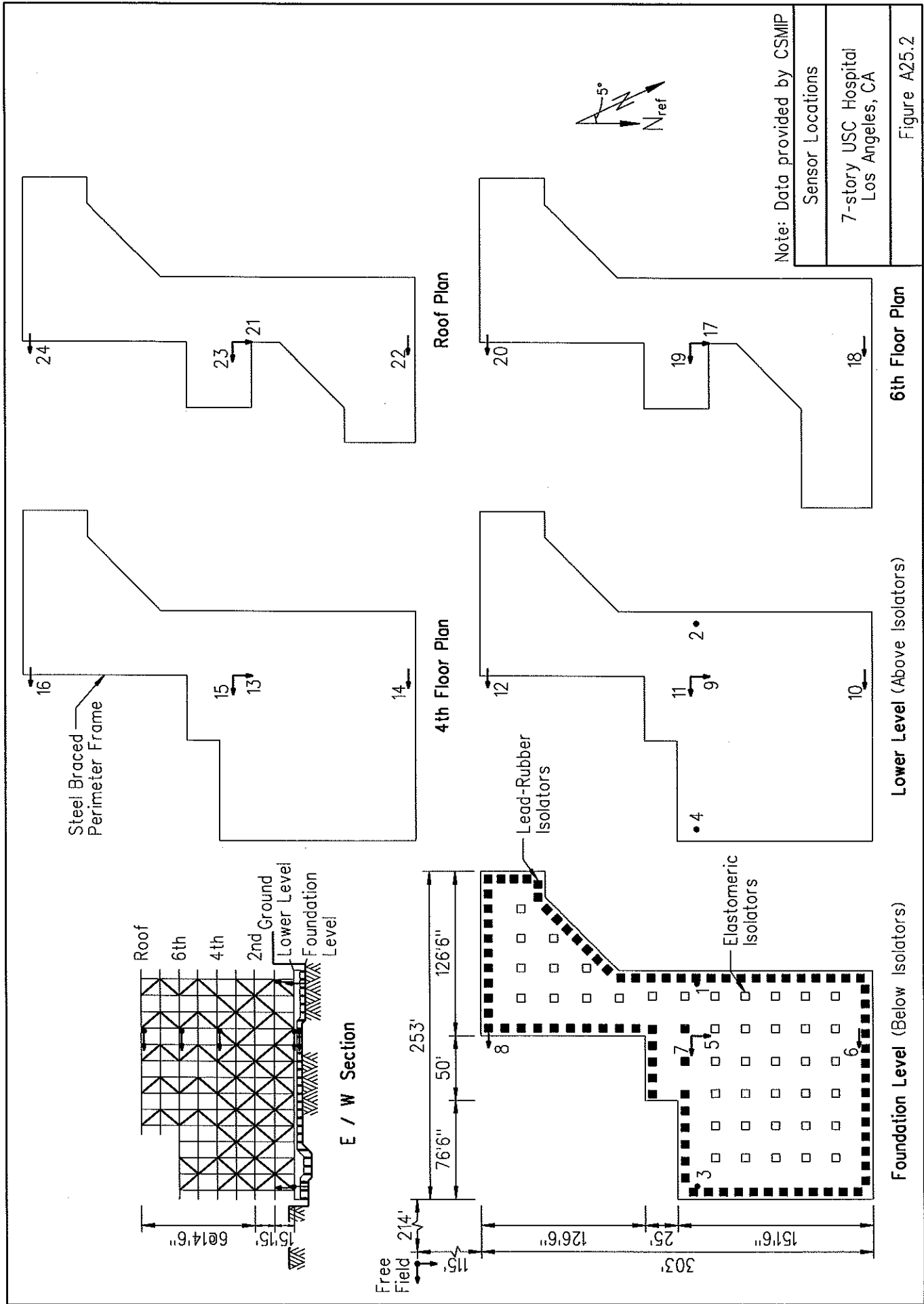
Data Sources

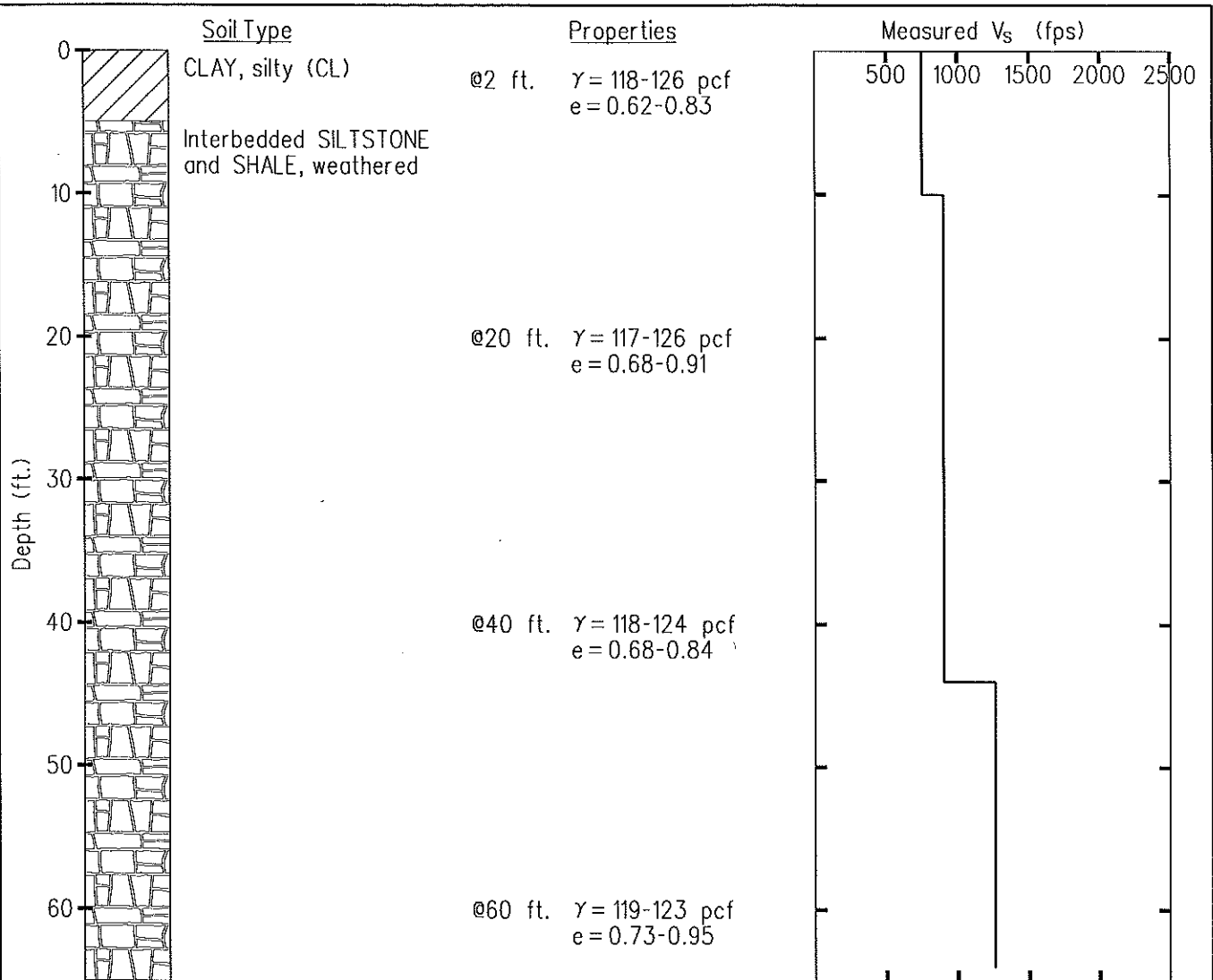
LeRoy Crandall Assoc., LCA (1978 and 1987). unpublished geotechnical investigations at USC hospital, *Job. Nos. ADE-78046 and ADE 87124B*.

Duke, C.M., Johnson, J.A., Kharraz, Y., Campbell, K.W., and Malpiede, N.A. (1971).

"Subsurface site conditions and geology in the San Fernando Earthquake Area," *Rpt. No. UCLA-ENG-7206*, School of Engrg. and Applied Sciences, Univ. of Calif., Los Angeles.







Data Source:
LCA (1987)

Notes:

1. Groundwater encountered at 25 ft. depth.
2. Shear wave velocities based on downhole measurements by LCA (1978) and surface refraction measurements by Duke et al. (1972).

Generalized Soil Profile

7-story USC Hospital
Los Angeles, CA

Figure A25.3

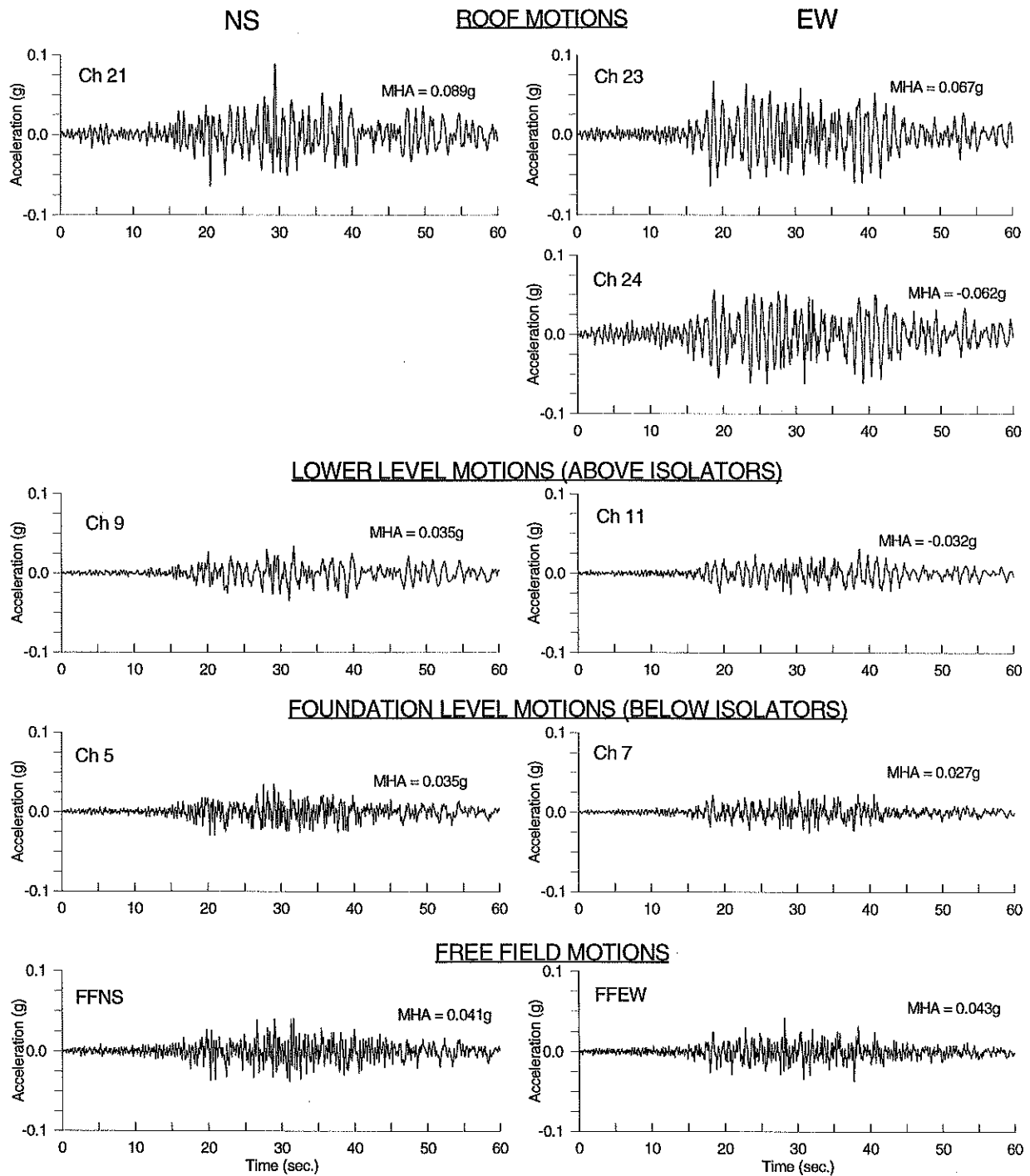


Fig. A25.LD1: LA 7 STORY USC HOSPITAL
Horizontal Acceleration Time Histories
1992 Landers Earthquake

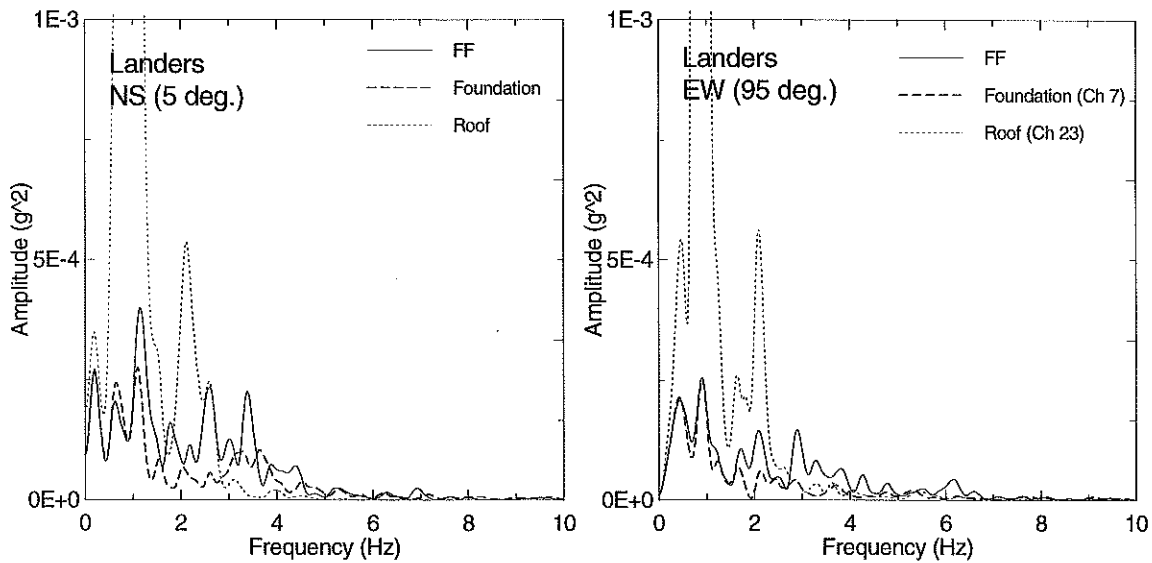


Fig. A25.LD2: LA 7 STORY USC HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

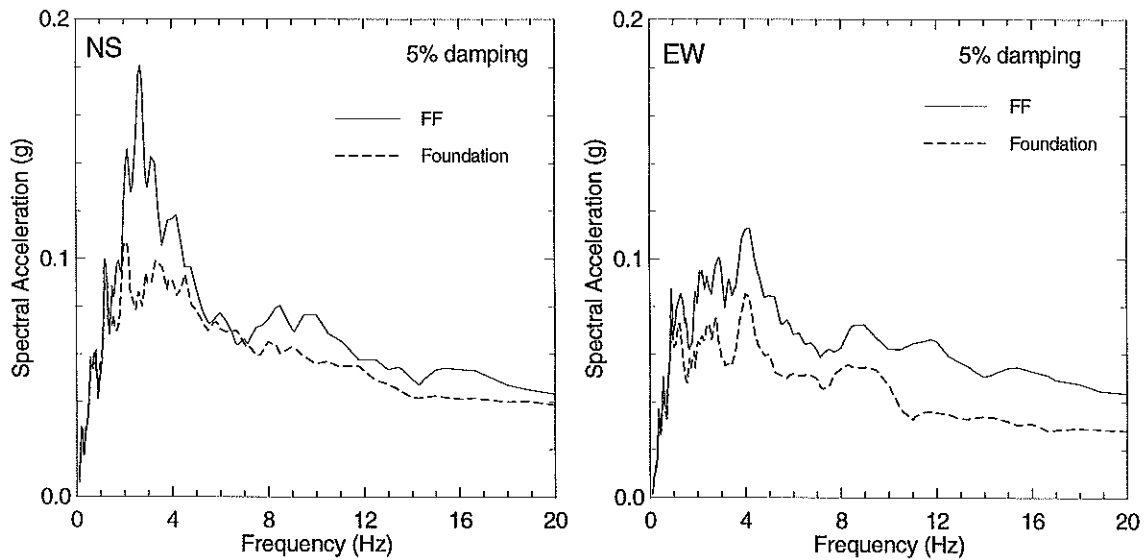


Fig. A25.LD3: LA 7 STORY USC HOSPITAL
Acceleration Response Spectra of Foundation-Level and Free-Field Motions
1992 Landers Earthquake

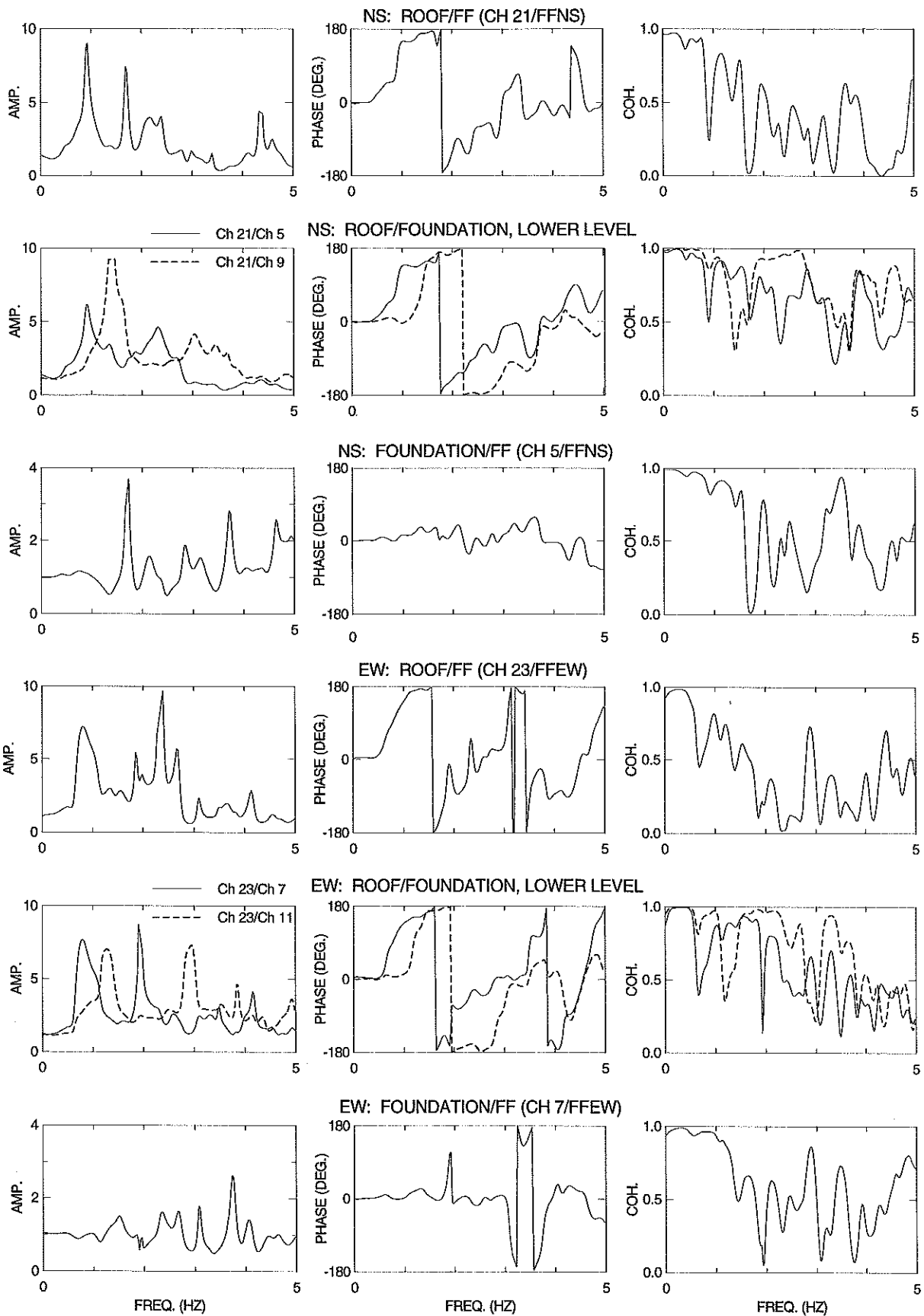


Fig. A25.LD4: LA 7 STORY USC HOSPITAL
Transmissibility and Coherency Functions
1992 Landers Earthquake
A25-10

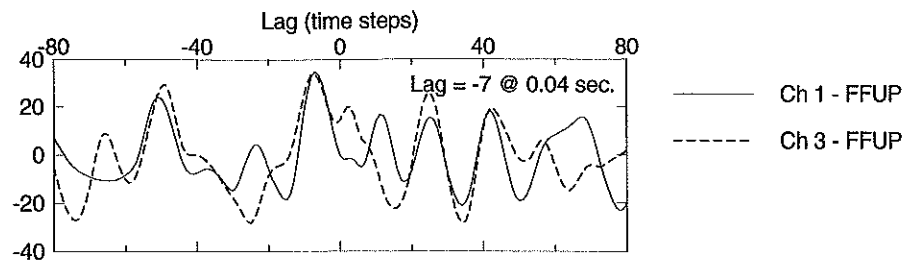


Fig. A25.NR0: Cross-correlation function between vertical foundation level and free-field accelerations

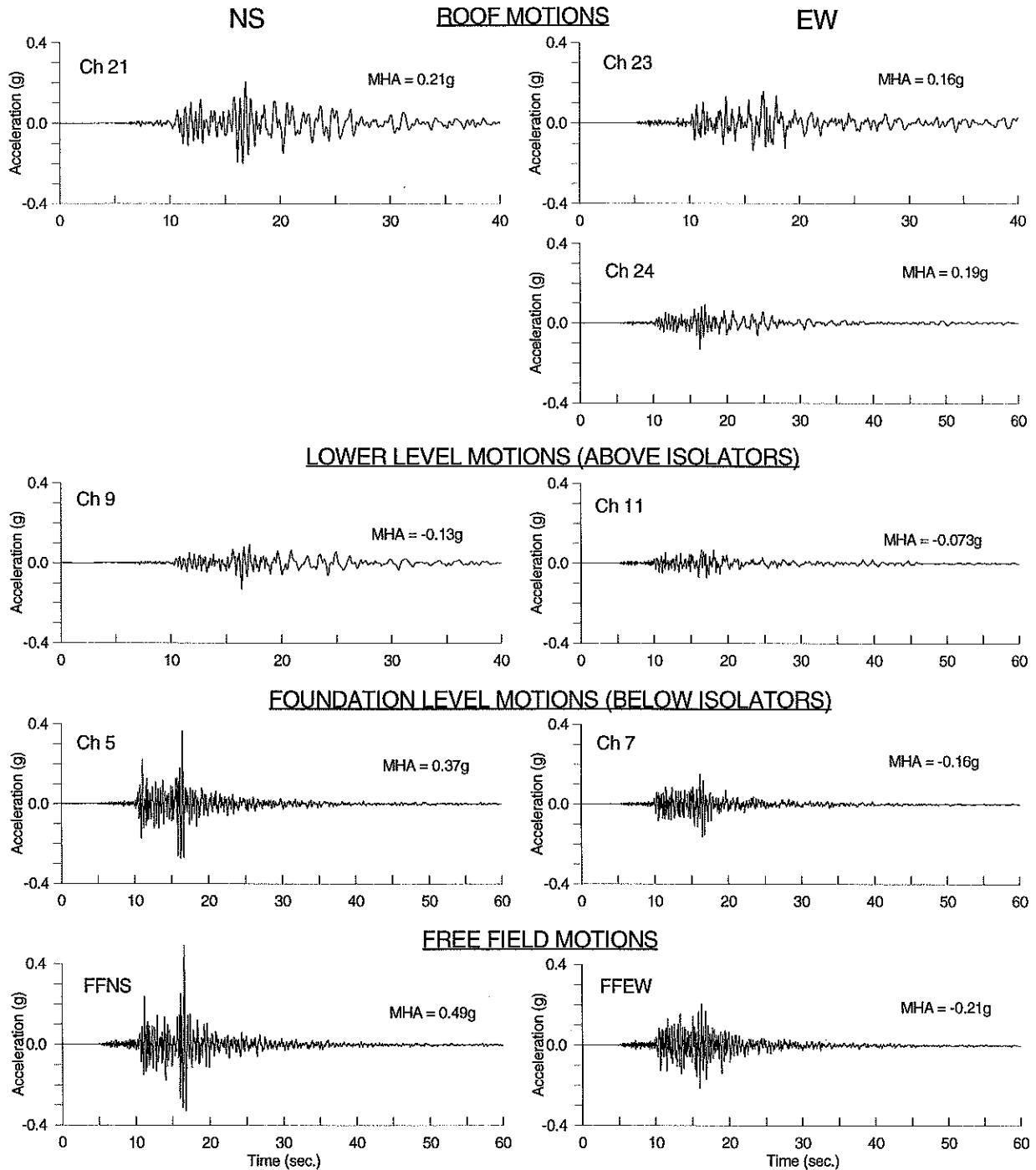


Fig. A25.NR1: LA 7 STORY USC HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

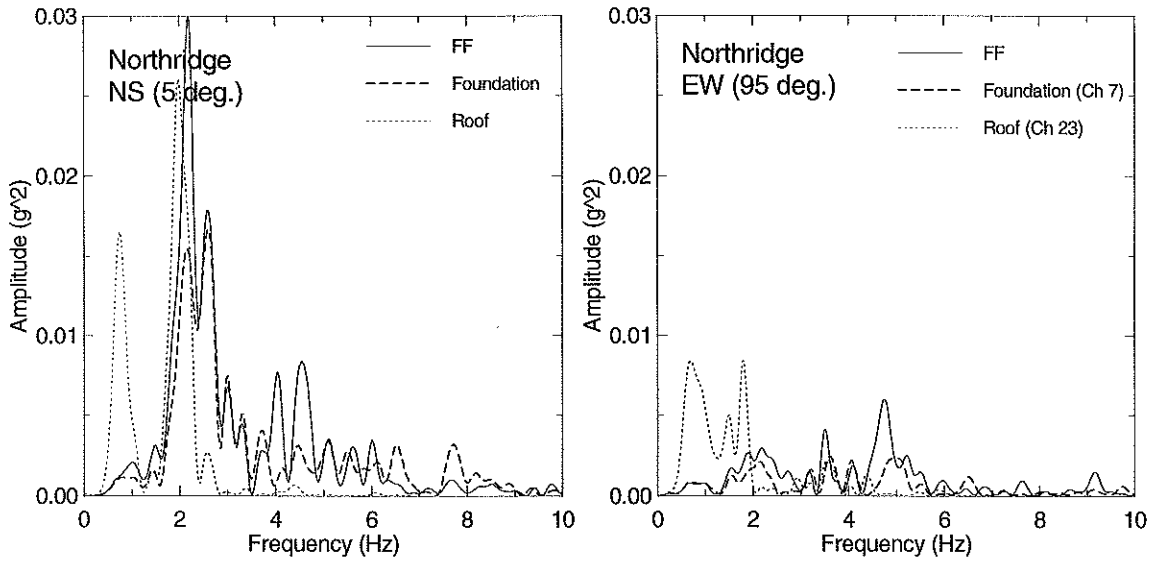


Fig. A25.NR2: LA 7 STORY USC HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

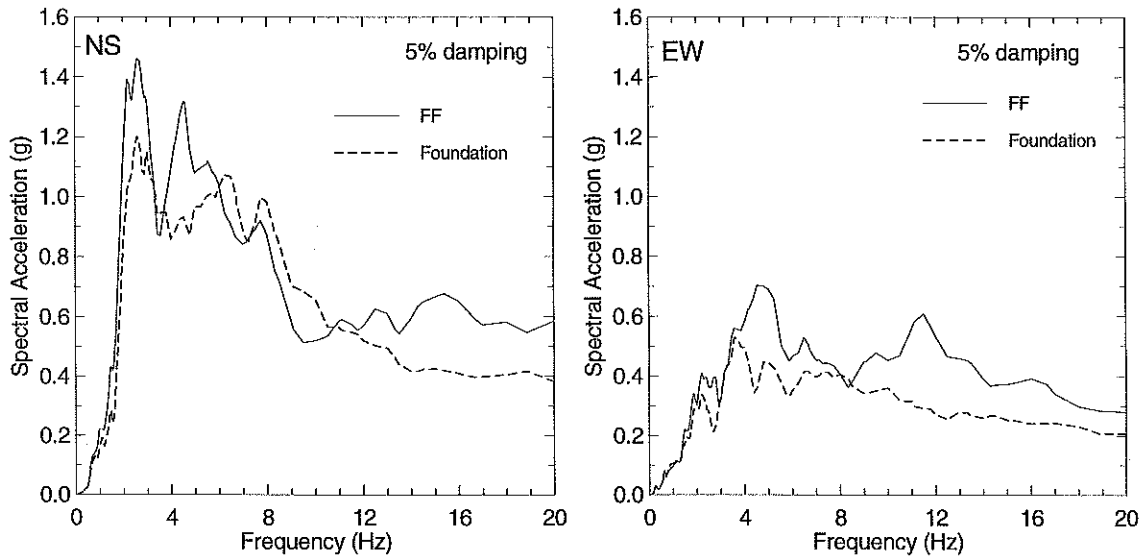


Fig. A25.NR3: LA 7 STORY USC HOSPITAL
Acceleration Response Spectra of Foundation-Level and Free-Field Motions
1994 Northridge Earthquake

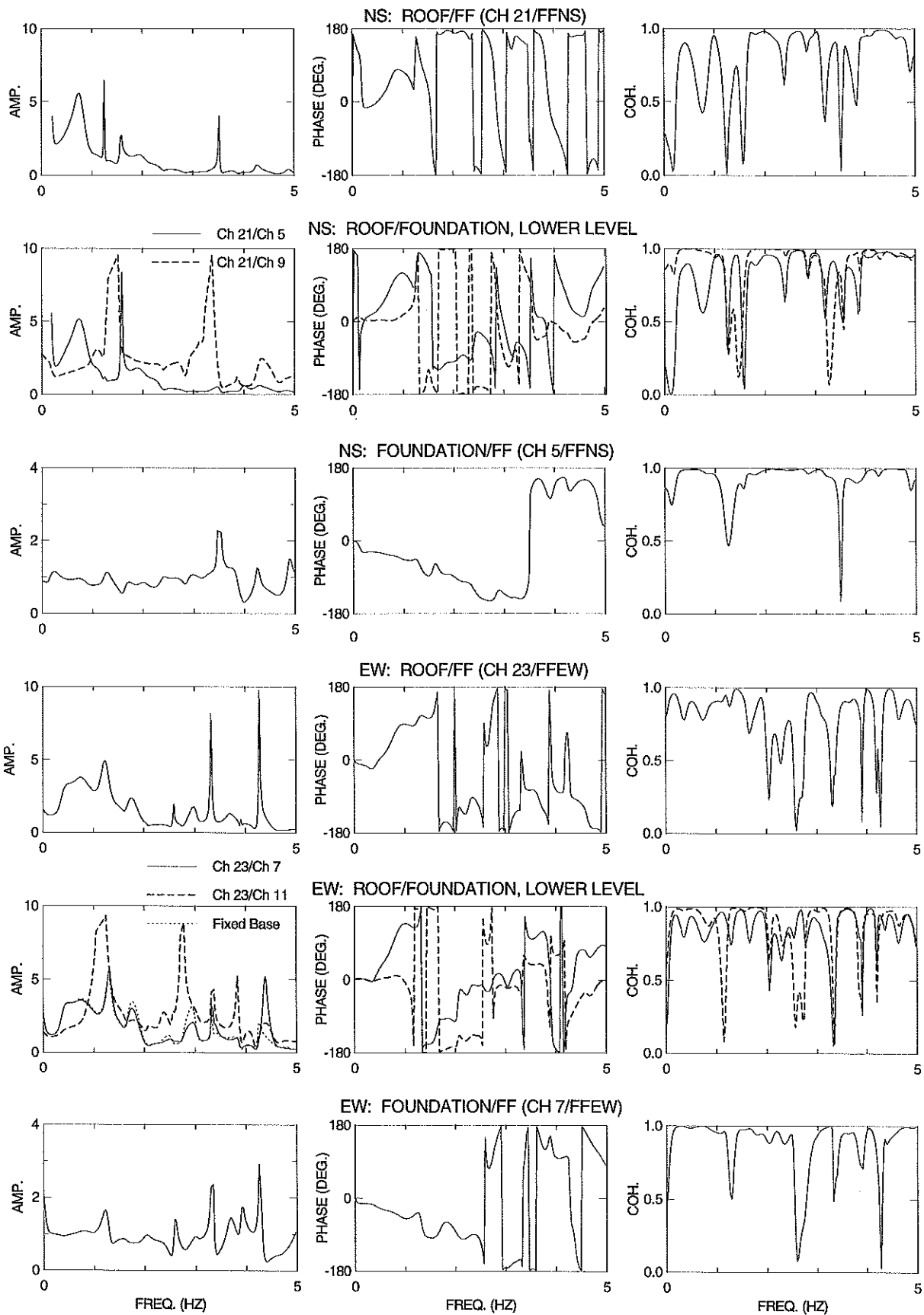


Fig. A25.NR4: LA 7 STORY USC HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A25-13

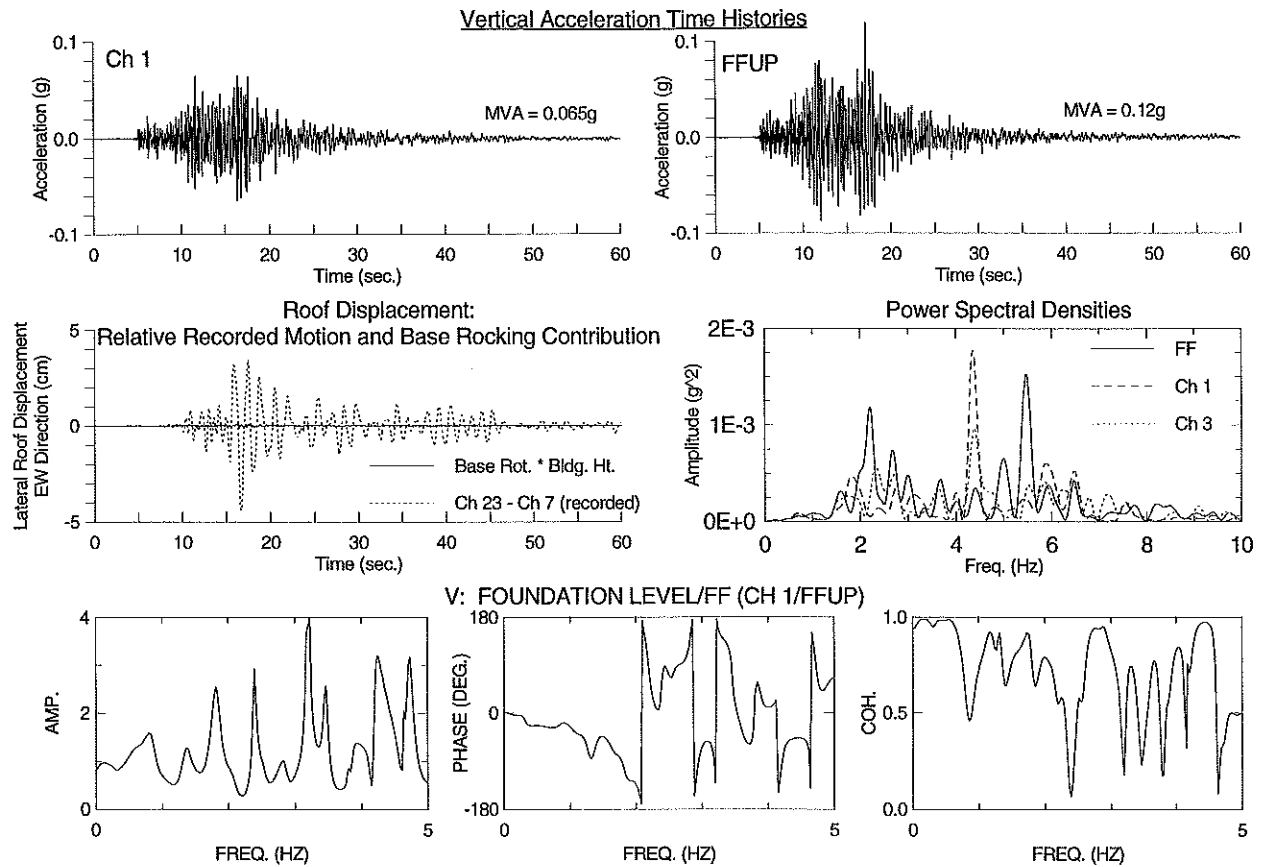


Fig. A25.NR5: LA 7 STORY USC HOSPITAL
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

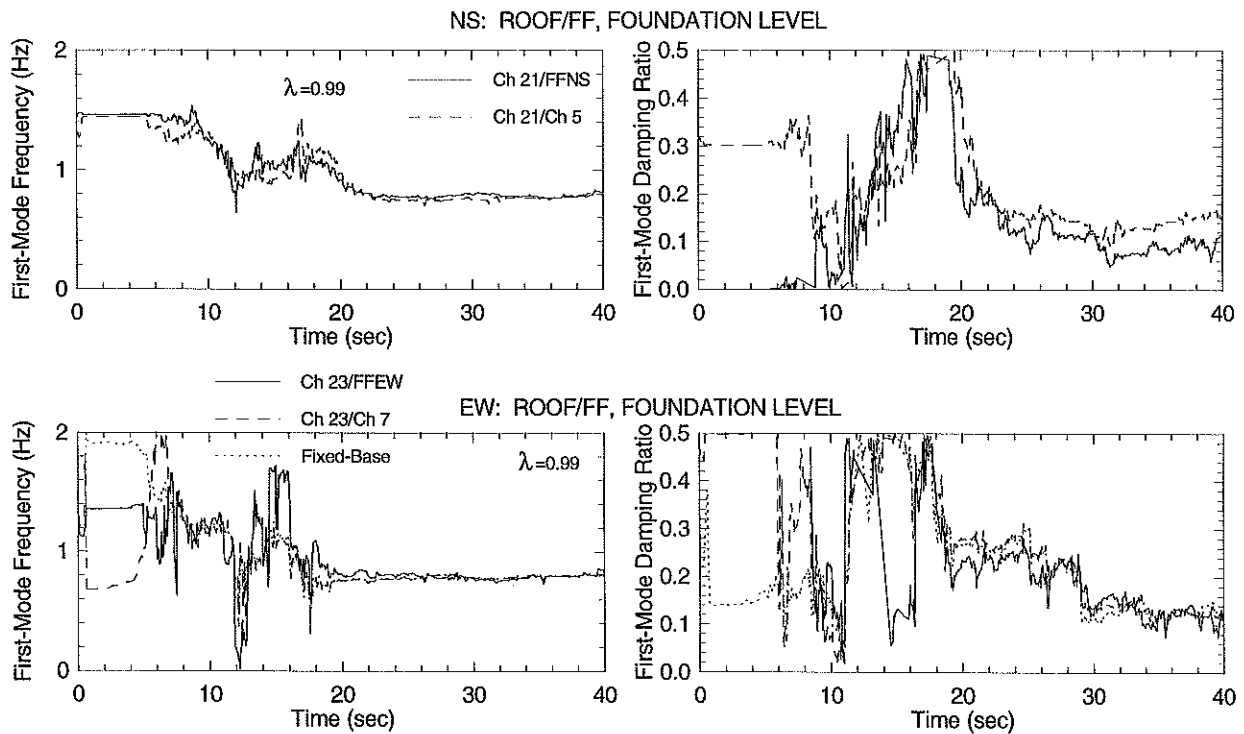


Fig. A25.NR6: LA 7 STORY USC HOSPITAL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A26

Site Name: LA 7 Story UCLA Math/Sci. Building
Sensors: CSMIP 24231/24688

Lat: 34.069°
Long.: 118.442°

Floor Levels: 7
Bsmt. Levels: partial
Structure: Steel frame and concrete shear walls

Foundation: footings
Site Geology: alluvium
Design Date: 1959/1965

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.47	0.28	0.27	0.28	0.22	0.24	0.50	0.45	--

-- = No instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{-}23\text{ Hz}$ (str.), $0.16\text{-}23\text{ Hz}$ (FF); $f_{\text{Nyq}}=25\text{ Hz}$; FF/str. data not synchronized.

Final: decimated by 2 to $f_{\text{Nyq}}=12.5\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A26.NR0).

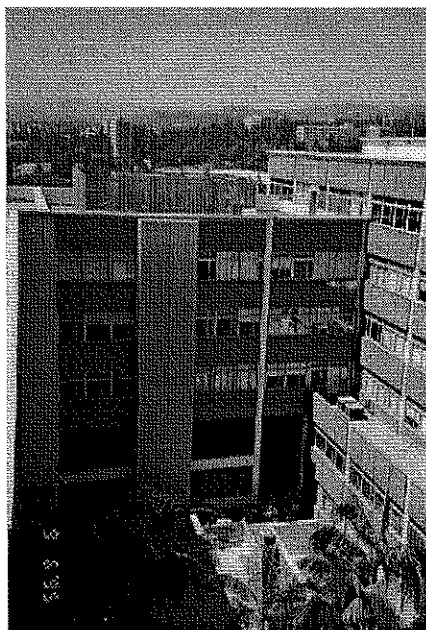
Figures

A26.1-A26.3

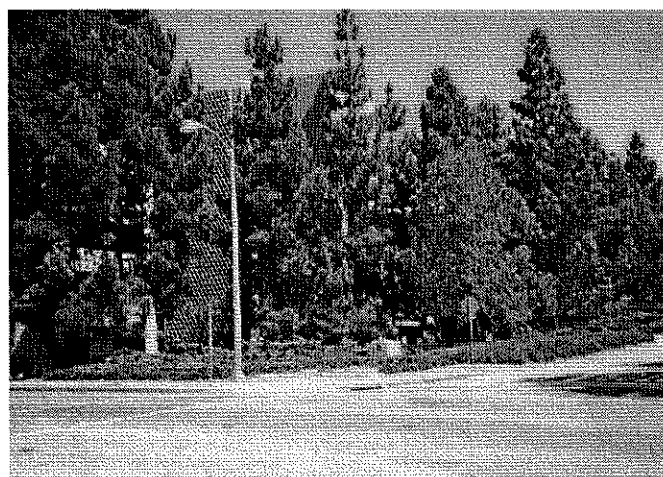
Site Plan, Sensor Locations, Generalized Soil Column

A26.NR0-6

NR Analysis Results



View of structure, looking west



View of FF, looking north-east
(parking garage in background)

DISCUSSION

Site Description

The building is 7-stories tall with a partial basement, and is rectangular in plan (72 by 48 feet at the base). The lower two levels of the north and east sides of the structure serve as retaining walls which accommodate about a 27-foot drop in the ground surface elevation. The building is located immediately adjacent to similar buildings (see Fig. A26.1), with six-inch seismic joints separating the structures. The structure has two distinct sections which were constructed at different times. The original, bi-level structure (1959) housed a nuclear reactor. This structure has reinforced concrete shear walls, is founded on spread footings, and has a 6-inch thick concrete floor slab. In 1965, a 5-level steel frame building was built atop the concrete structure. As shown in Fig. A26.2, accelerographs are present on the 1st, 3rd, 5th, and roof levels of the building. The free-field instrument is located about 930 feet southeast of the building, but is only about 50 feet from a 7-level parking structure.

The site is located on terrain having a mild slope down to the south. Soil conditions in the area have been investigated with borings at the structure and free-field sites by L.T. Evans (1964 and 1967), seismic refraction measurements near Pauley Pavillion by Duke et al. (1971) and Eguchi et al. (1976), and borings with downhole shear wave velocity measurements at two nearby sites (LCA, 1976 and 1977). The soil profile is generally characterized by about 65 feet of surficial clayey silts and sands overlying weathered sandstone bedrock. The shear wave velocity data shown in Fig. A26.3 were estimated from the downhole and seismic refraction measurements in geologic media similar to that encountered at the site.

Instrument Selection and Free-Field Conditions

The accelerographs used in the analyses are indicated in Table A26.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A26.NR1.

Table A26.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	UP	NS	EW
Ch 1	Ch 3	Ch 2	Ch 10	Ch 11

The FF instrument is located approximately 50 feet from a 7 story parking garage. Power spectra of the FF motions (Fig. A26.NR2) indicated possible contamination associated with vibrations of the garage at about $f \approx 3.5\text{--}4$ Hz (NS direction) and $f \approx 5.5$ Hz (EW direction).

Analysis Results

(a) Comparison of 1st Floor and Free-Field Motions

1F/FF transmissibility functions (Fig. A26.NR4) and the spectra for horizontal motions (Fig. A26.NR2-3) generally indicate increasing de-amplification of 1F motions with frequency in both

directions, though some amplification occurred in the EW direction near the first-mode frequency ($\tilde{f}_1 = 0.96$ Hz).

Transmissibility, coherency, and power spectral density functions for the FF and 1F vertical motions are presented in Fig. A26.NR5. Minor amplification occurred near the first-mode frequencies for both directions, indicating that rocking may have influenced the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A26.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A26.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A26.NR6).

Table A26.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=3, j=4		L (EW) d=4, j=4		tr (NS) d=1, j=4		L (EW) d=13, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.64	20.2	0.97	13.7	1.52	13.7	0.96	11.4	1.49	9.7	0.96	7.3
	2					4.04	15.5	3.50	14.4	4.01	17.3	3.69	8.7

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The modest difference between the transverse fixed- and pseudo flexible-base parameters is consistent with the minor rocking effect evident from the vertical 1F/FF transmissibility functions. No significant rocking appears to have occurred in the longitudinal direction.
- The CEM parametric analyses indicate that $\tilde{\zeta} < \tilde{\zeta}^*$ to an extent which is physically impossible because the foundation damping factors ($\tilde{\zeta}_0$) would have to be negative. This represents an error in the analyses, possibly resulting from the large FF/structure separation or the contamination of FF recordings from the nearby parking structure.

The RPEM results (Fig. A26.NR6) indicate reductions of first-mode frequency in both directions of about 0.3 Hz. These reductions occur at about 6 to 10 seconds, which corresponds to a period of high amplitude shaking (Fig. A26.NR1). Damping values were also relatively high at these times.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A26.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 33 feet using the recorded free-field motions and the profile in Fig. A26.3. The

instrumented structure was assumed to be independent of the neighboring buildings for the calculation of foundation radii. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A26.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 695$ fps	$h = 66$ ft	emb. = 13.5 ft.	$\gamma = 0.15$
NR	$\nu = 0.4$		tr: $r_1 = 33$ ft		$r_2 = 30$ ft
	$\beta^* = 7.6\%$	$V_s^* = 550$ fps	L: $r_1 = 33$ ft		$r_2 = 37$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A26.3 with the fixed-base modal parameters in Table A26.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A26.4.

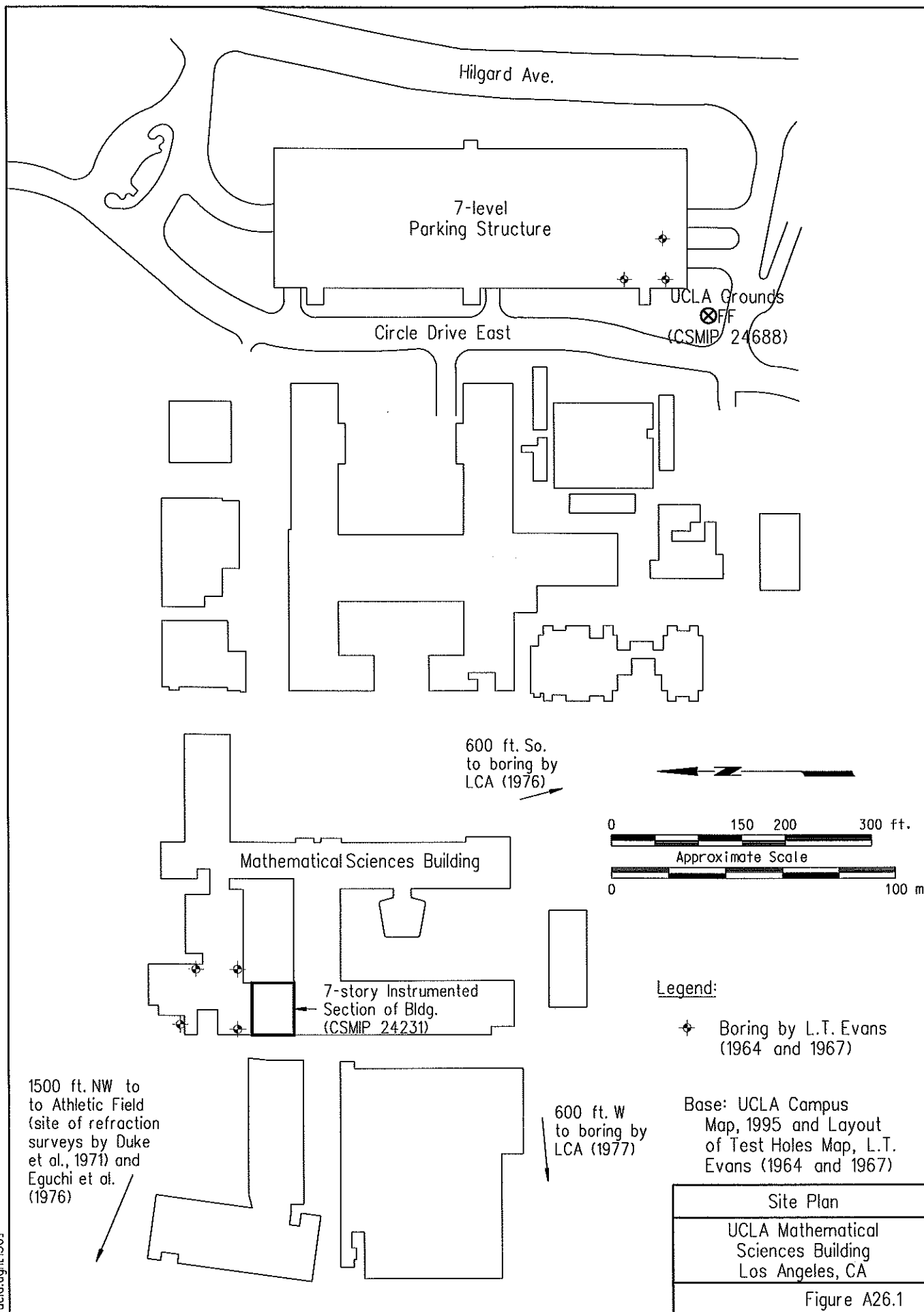
Table A26.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.10	-5.3	9.7	1.01	-6.0	7.3
	Pre.-Vel.	1.14	1.0	14.7	1.03	0.4	12.7
	Pre.-Bie.	1.17	4.5	17.0	1.06	1.5	13.0

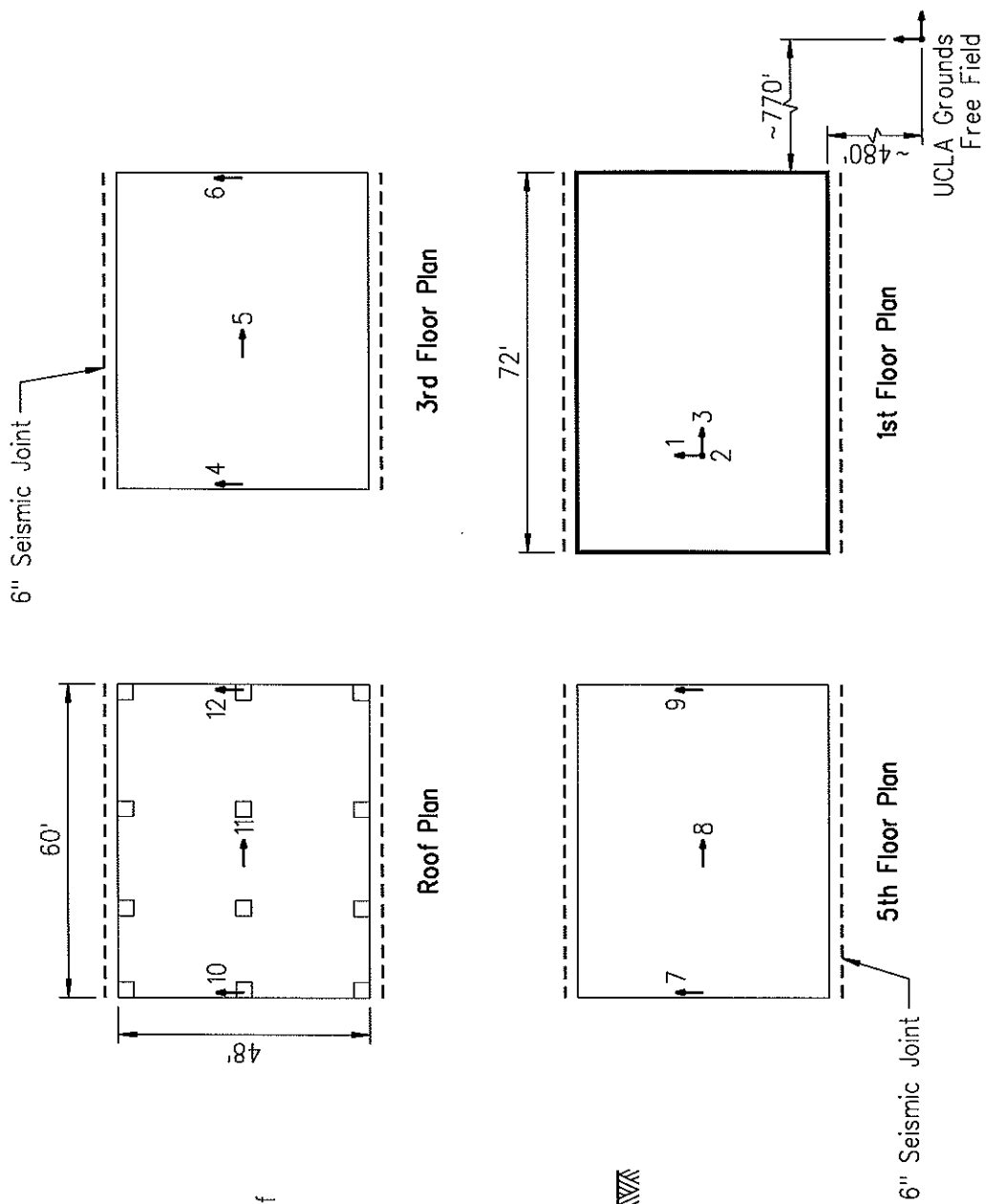
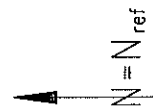
As discussed in Part (b), the negative $\tilde{\zeta}_0$ values indicate numerical errors in the system identification results. Nonetheless, it is interpreted from these results that $\tilde{\zeta}_0$ is small. The foundation damping and period lengthening were most accurately predicted by the “modified Veletsos” formulation.

Data Sources

- LeRoy Crandall and Assoc., LCA (1976 and 1977). unpublished foundation/geotechnical investigations, Nursing School at Tiverton and South Circle Drives and Parking Structure at Westwood Plaza and Strathmore Place, *Proj. Nos. AD-76124 and AD-77171*.
- Duke, C.M., Johnson, J.A., Kharraz, Y., Campbell, K.W., and Malpiede, N.A. (1971). “Subsurface site conditions and geology in the San Fernando Earthquake Area,” *Rpt. No. UCLA-ENG-7206*, School of Engineering and Applied Sciences, Univ. of California, Los Angeles.
- Eguchi, R.T., Campbell, K.W., Duke, C.M., Chow, A.W., and Paternina, J. (1976). “Shear velocities and near-surface geologies at accelerograph sites that recorded the San Fernando Earthquake,” *Rpt. No. UCLA-ENG-7253*, Univ. of California, Los Angeles.
- L.T. Evans, Inc. (1964 and 1967). unpublished foundation investigations, Mathematics-Science Bldg Addition and Parking Structure E, *Job. No. DU 9-1347 and DU 9-1347, DU 9-2775*.



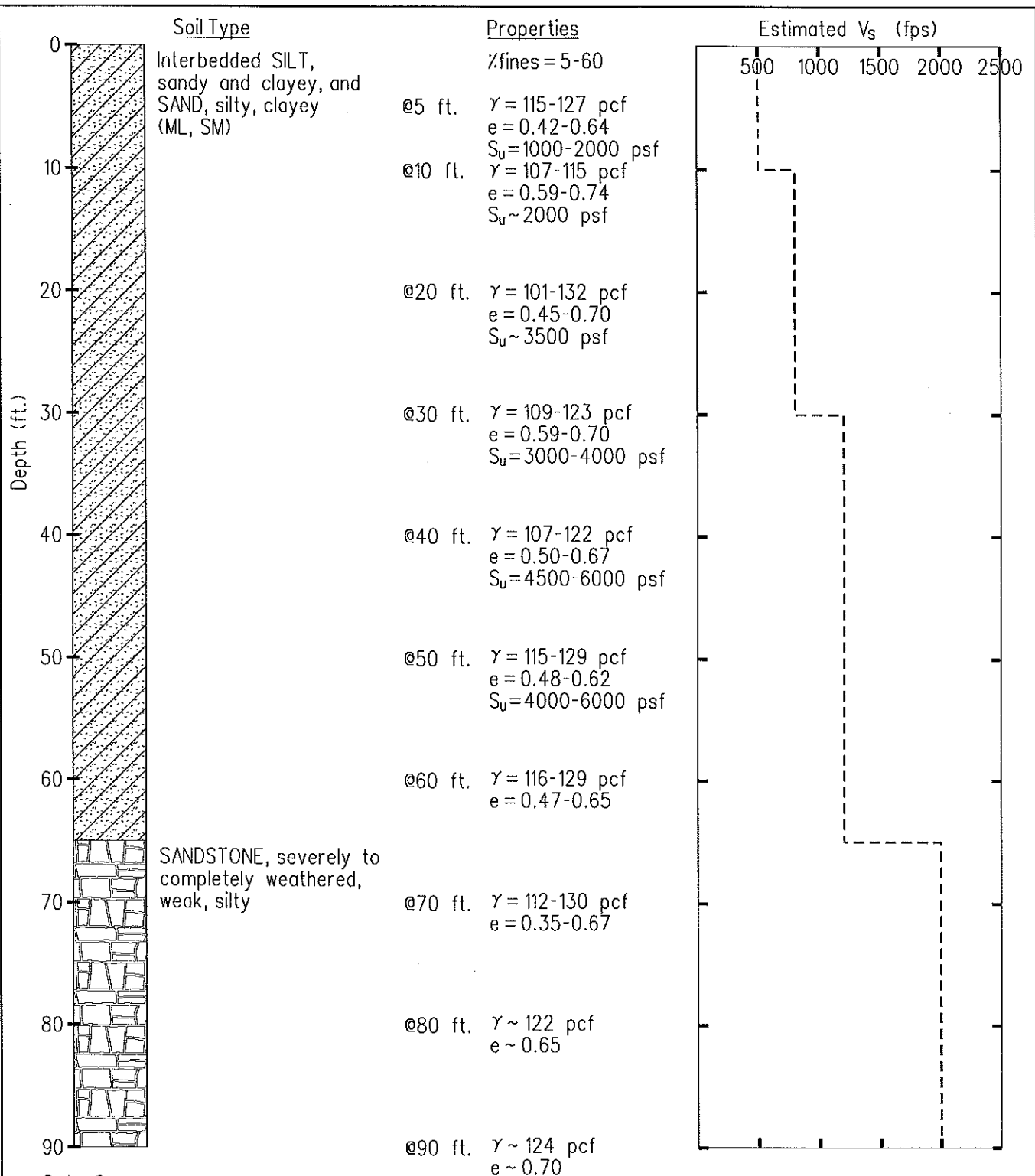
ucla.dgn[150]



Sensor Locations
UCLA Mathematical Sciences Building Los Angeles, CA
Figure A26.2

Note: Data provided by CSMIP

ucla.dgn



Data Source:
L.T. Evans (1964 & 1967)

Notes:

1. No groundwater encountered.
2. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
3. Shear wave velocities estimated based on in situ measurements in similar geologic materials at nearby sites (Pauley Pavillion, Duke et al., 1976 and Eguchi et al., 1976; Parking Str. 8, LCA, 1977; and Medical Center, LCA, 1976).

Generalized Soil Profile
UCLA Mathematical Sciences Building Los Angeles, CA
Figure A26.3

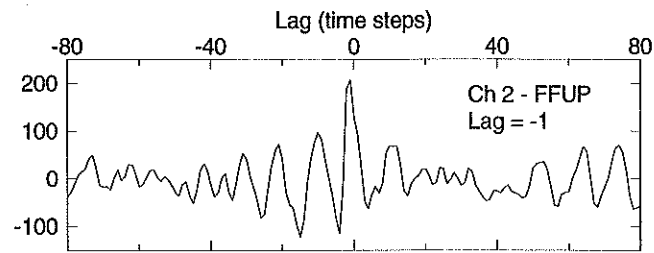


Fig. A26.NR0: Cross-correlation between vertical 1st floor and free-field accelerations

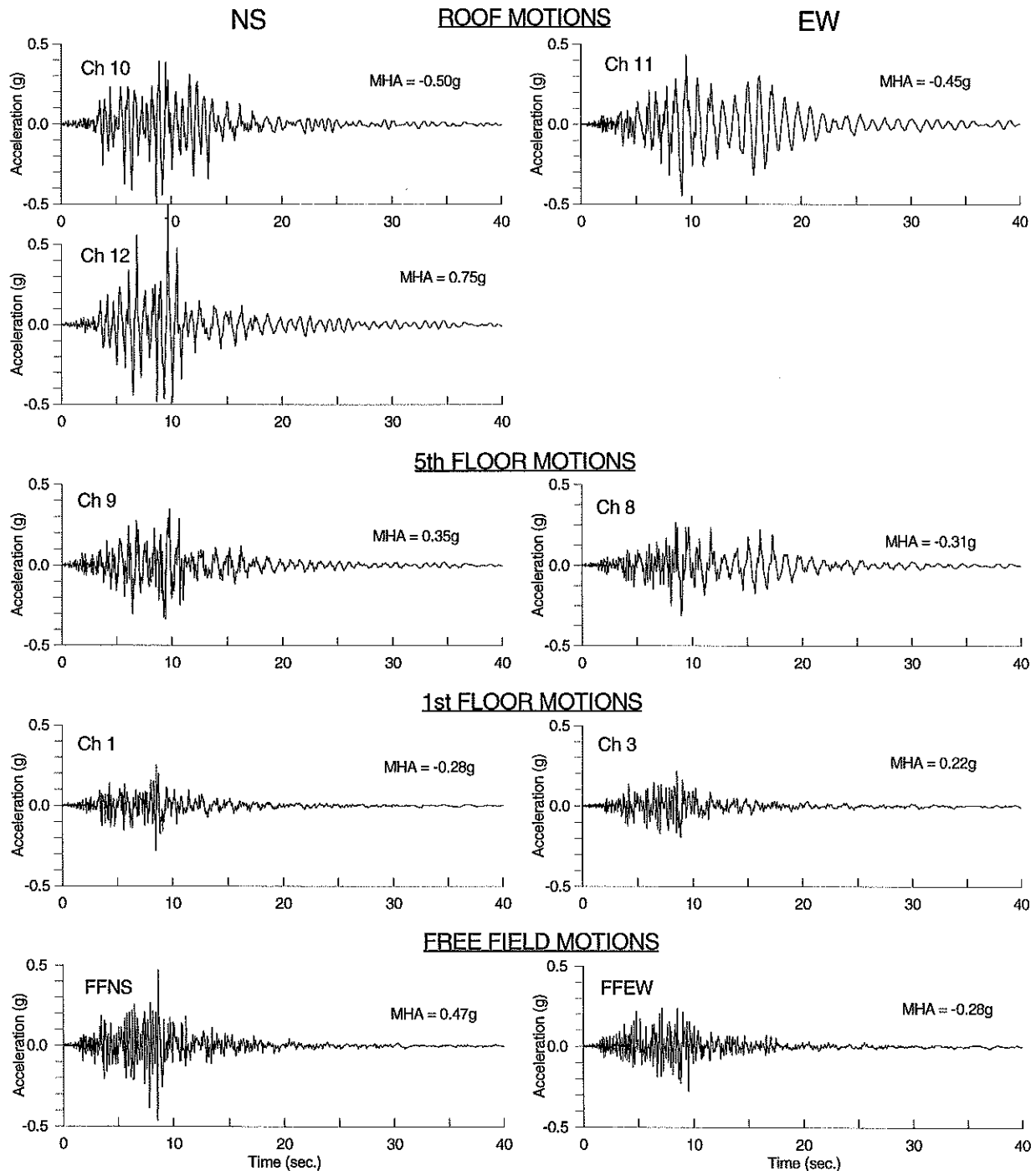


Fig. A26.NR1: LA 7 STORY UCLA MATH/SCIENCE BLDG
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

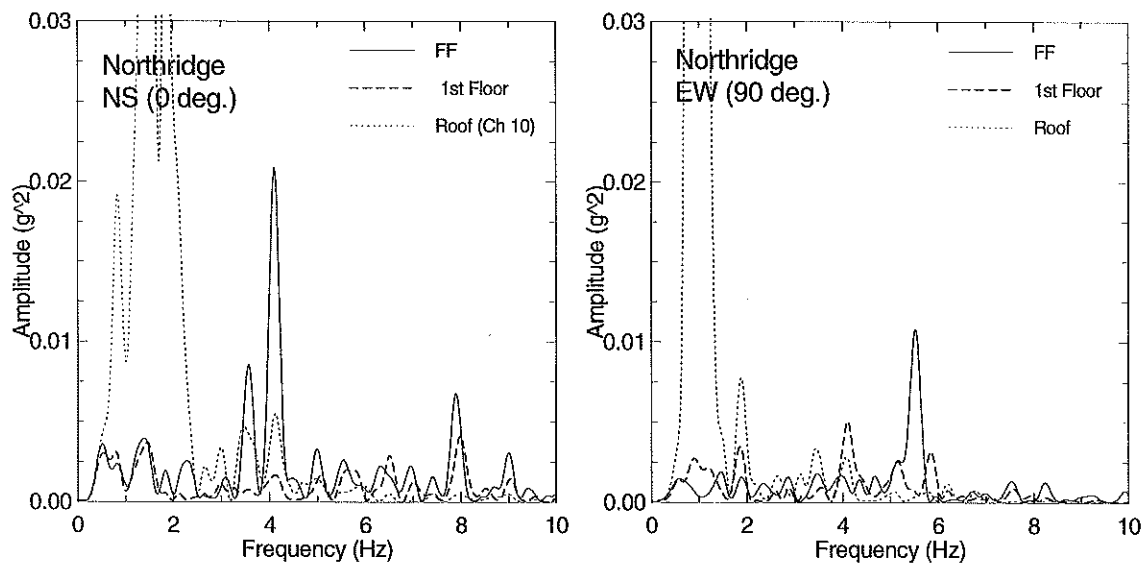


Fig. A26.NR2: LA 7 STORY UCLA MATH/SCIENCE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

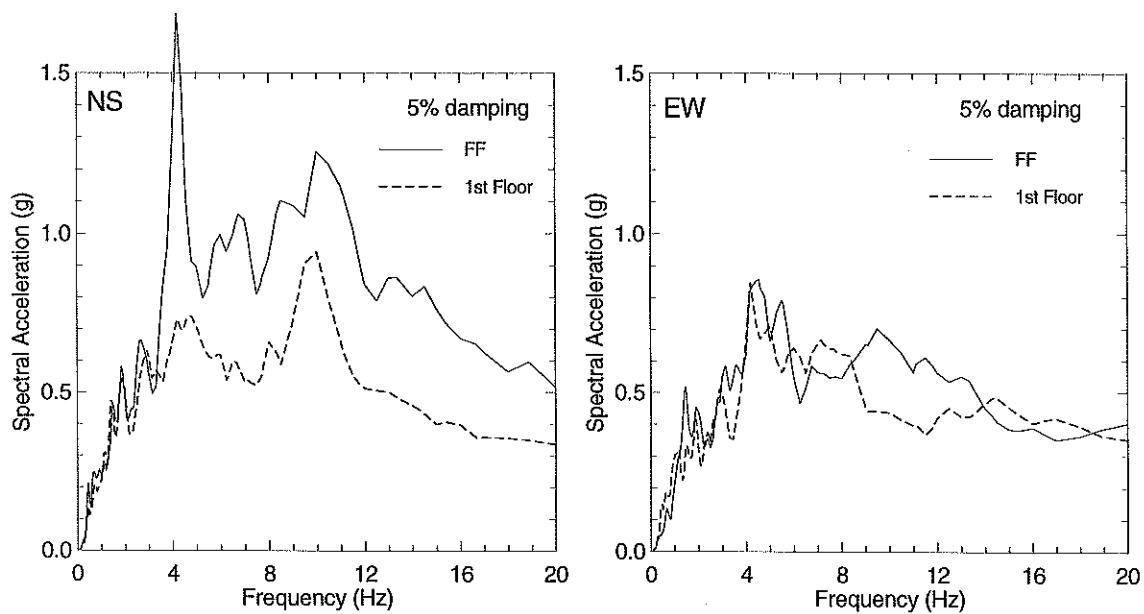


Fig. A26.NR3: LA 7 STORY UCLA MATH/SCIENCE BUILDING
Acceleration Response Spectra of 1st Floor and Free-Field Motions
1994 Northridge Earthquake

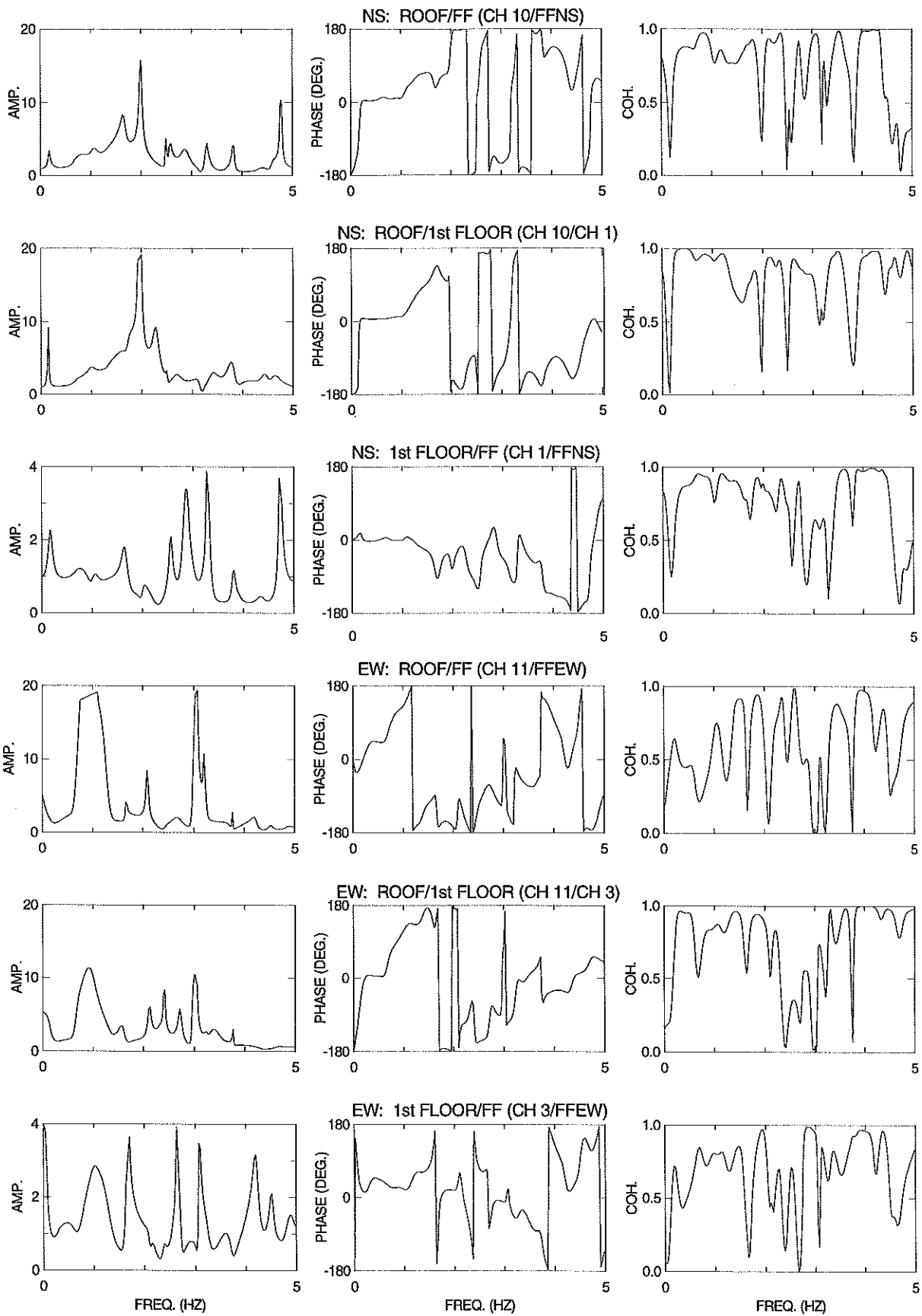


Fig. A26.NR4: LA 7 STORY UCLA MATH/SCIENCE BLDG
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A26-10

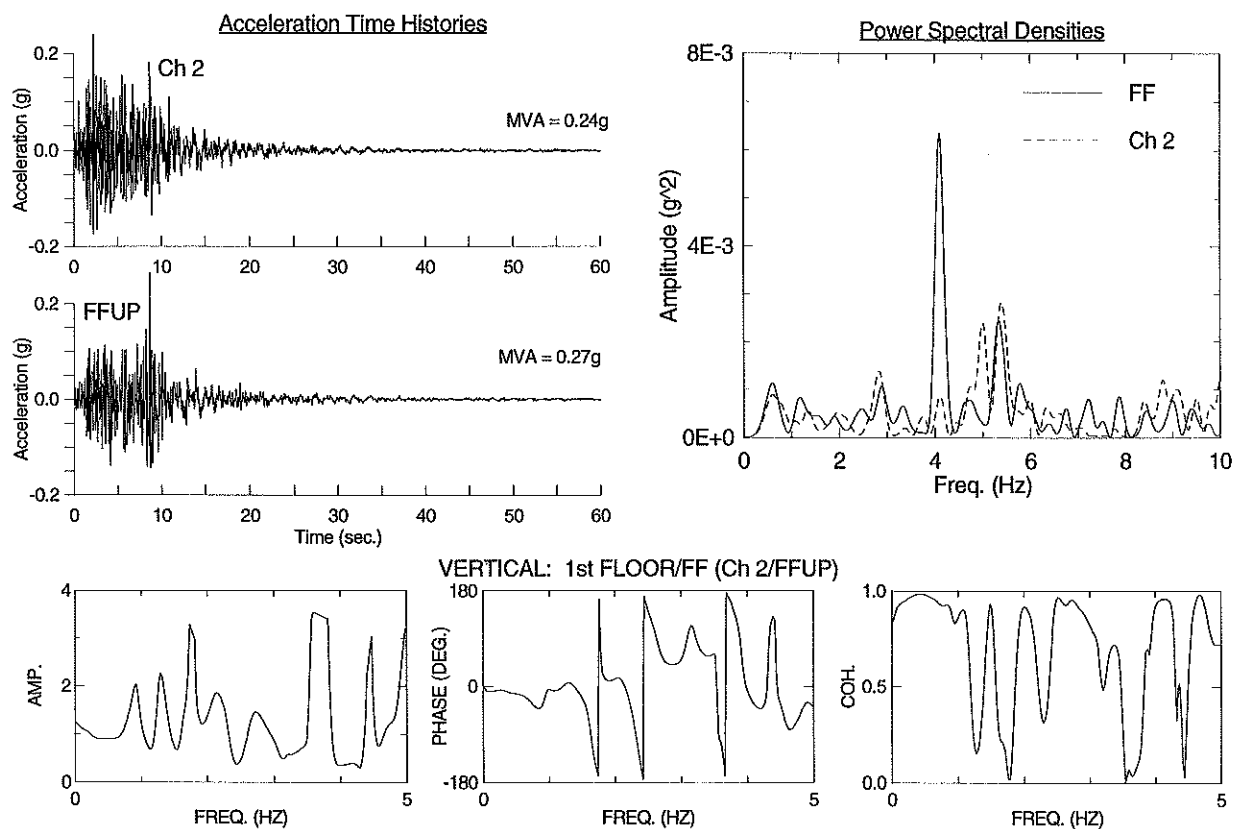


Fig. A26.NR5: LA 7 STORY UCLA MATH/SCIENCE BUILDING
Analysis of Vertical Motions
1994 Northridge Earthquake

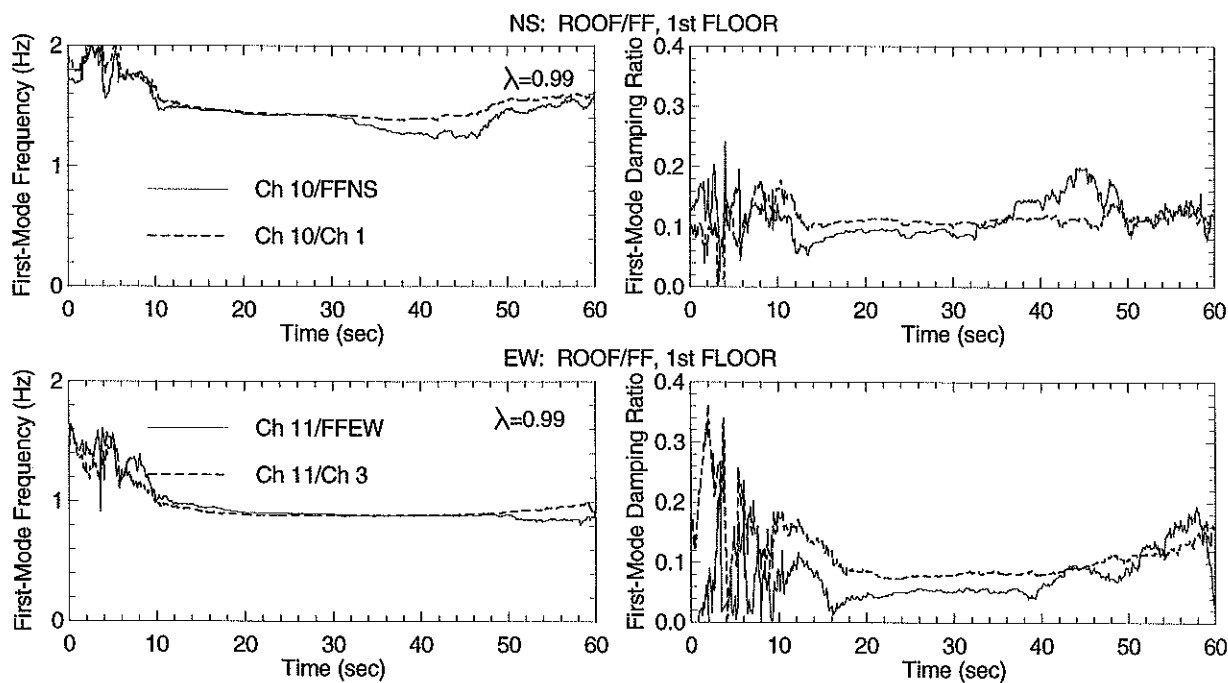


Fig. A26.NR6: LA 7 STORY UCLA MATH/SCIENCE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A27

Site Name: LA 15 Story DWP Office Building
Sensors: CSMIP 24611/24569

Lat: 34.058°
Long.: 118.249°

Floor Levels: 17 + penthouse
Bsmt. Levels: 0
Structure: Steel moment frames

Foundation: footings
Site Geology: Rock
Design Date: 1961

Recorded Peak Accelerations (g)

Earthquake	Free-Field			'B' Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers	0.028	0.038	0.020	0.032	0.034	0.015	0.060	0.077	--
1992 Big Bear*	0.03	0.02	0.01	0.03	0.02	0.01	0.06	0.04	--
1994 Northridge	0.19	0.17	0.097	0.20	0.13	0.066	0.23	0.19	--

-- = No instrument

Preprocessing

Landers (LD)

Preliminary: CSMIP; filtered, $f = 0.1\text{--}46\text{ Hz}$; $f_{Nyq}=50\text{ Hz}$; FF/str. data not synchronized; FF not aligned with str.

Final: FF data rotated 137° CCW to align with str.; decimated by 4 to $f_{Nyq}=12.5\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A27.LD0).

Northridge (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{Nyq}=50\text{ Hz}$; FF/str. data synchronized; FF not aligned with str.

Final: FF data rotated 137° CCW to align with str.; decimated by 4 to $f_{Nyq}=12.5\text{ Hz}$.

Figures

A27.1-A27.3: Site Plan, Sensor Locations, Generalized Soil Column
A27.LD0-5: LD Analysis Results
A27.NR1-6: NR Analysis Results



View of structure, looking north

DISCUSSION

Site Description

The building is 17-stories tall, with a large, bi-level base structure surrounding a rectangular tower with dimensions in plan of 158 by 344 feet. No structural separation is present between the tower perimeter and the adjacent base structure (A.C. Martin and Assoc., 1972). The lateral load resisting system consists of reinforced concrete shear walls between the 'B' level and 2nd floor, and moment resisting steel frames from the 2nd floor to the roof. The foundation consists of spread footings. As shown in Fig. A27.2, accelerographs are present on the 'B', 2nd, 8th, and roof levels of the structure. The free-field instrument is located about 750 feet northeast of the building in an open area.

The site is located on terrain having a mild slope down to the northeast (towards Highway 101). Subsurface exploration has been conducted at the site to a depth of 60 feet (LCA, 1960) and at a nearby site on Figueroa Street to a depth of 400 feet (SW/AA, 1980). Based on these sources, the geologic conditions at the site consist of: about 35 feet of highly weathered shale bedrock, 175 feet of weathered shale, and soft siltstone which extends to the maximum depth explored of 400 feet. The shear wave velocities indicated in Fig. A27.3 are based on in situ downhole measurements by SW/AA (1980).

Instrument Selection and Free-Field Conditions

Accelerographs about 60 feet inward from the south wall of the structure were used in the analyses, as indicated in Table A27.1. Acceleration time histories recorded by these and other sensors are shown in Fig. A27.**1.

Table A27.1: Sensors in structure used for analyses

'B' Level (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 2	Ch 3	Ch 1	Ch 11	Ch 12

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 'B' Level and Free-Field Motions

B/FF transmissibility functions (Fig. A27.**4) and the spectra of horizontal motions (Fig. A27.**2-3) indicate increasing de-amplification of B motions with frequency in both directions.

Transmissibility, coherency, and power spectral density functions for the FF and B vertical motions are presented in Fig. A27.**5. No significant amplification is evident near the first-mode frequencies for either direction, indicating that rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A27.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A27.2), and (3) time variation of first-mode parameters from the RPEM, NR only (Fig. A27.NR6).

Table A27.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW)		L (NS)		tr (EW)		L (NS)	
						LD: d=17, j=18 NR: d=16, j=22		d=17, j=20 d=16, j=22		d=18, j=18 d=18, j=22		d=12, j=20 d=14, j=22	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	--	--	0.32	1.9	0.31	3.4	0.32	2.0	0.32	4.8	0.32	2.4
	2					0.88	5.1	0.89	4.3	0.91	5.6	0.91	3.1
	3					1.63	6.9	1.48	6.3	1.54	10.0	1.41	4.1
NR	1	--	--	--	--	0.31	3.5	0.32	3.1	0.32	6.1	0.33	6.7
	2					0.86	4.4	0.86	4.6	0.88	0.74	0.86	7.6
	3					1.52	10.4	1.44	7.9	1.34	10.1	1.37	8.2

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in both directions.
- In most cases, fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical B/FF transmissibility functions, fixed-base parameters can be approximated by the pseudo flexible-base parameters. Where fixed-base parameters were estimated (longitudinal response, LD event), no significant differences between fixed- and pseudo flexible-base parameters were observed.
- The large number of modes used in the CEM analyses were necessary to develop adequate models of the structural behavior in the lower modes.

A.C. Martin and Associates (1972) performed dynamic response analyses with two- and three-dimensional models of the building framing using motions from the 1971 San Fernando Earthquake (which occurred before installation of the FF instrument). The stiffness of the models was adjusted to match calculated and recorded roof motions to obtain calibrated modal frequencies. The three lower-mode frequencies for the transverse direction were 0.31, 0.83, 1.41 Hz, which agrees well with the CEM results in Table A27.2. Similar agreement was obtained for the longitudinal direction.

The RPEM results (Figs. A27.NR6) indicate no significant time dependent changes in the first-mode parameters during the NR event, though the results are somewhat noisy over the first 25 seconds.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A27.3. The strain-dependent material properties were estimated from deconvolution analyses to a depth of 131 feet using the recorded free-field motions and the profile in Fig. A27.3. The instrumented structure was assumed to be independent of the surrounding parking garage for the calculation of foundation radii. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by a factor of 2.0 to account for shape effects.

Table A27.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1180$ fps	$h = 174$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 131$ ft		$r_2 = 110$ ft
LD	$\beta^* = 1.2\%$	$V_s^* = 1160$ fps	L: $r_1 = 131$ ft		$r_2 = 161$ ft
NR	$\beta^* = 2.0\%$	$V_s^* = 1120$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A27.3 with the fixed-base modal parameters in Table A27.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A27.4.

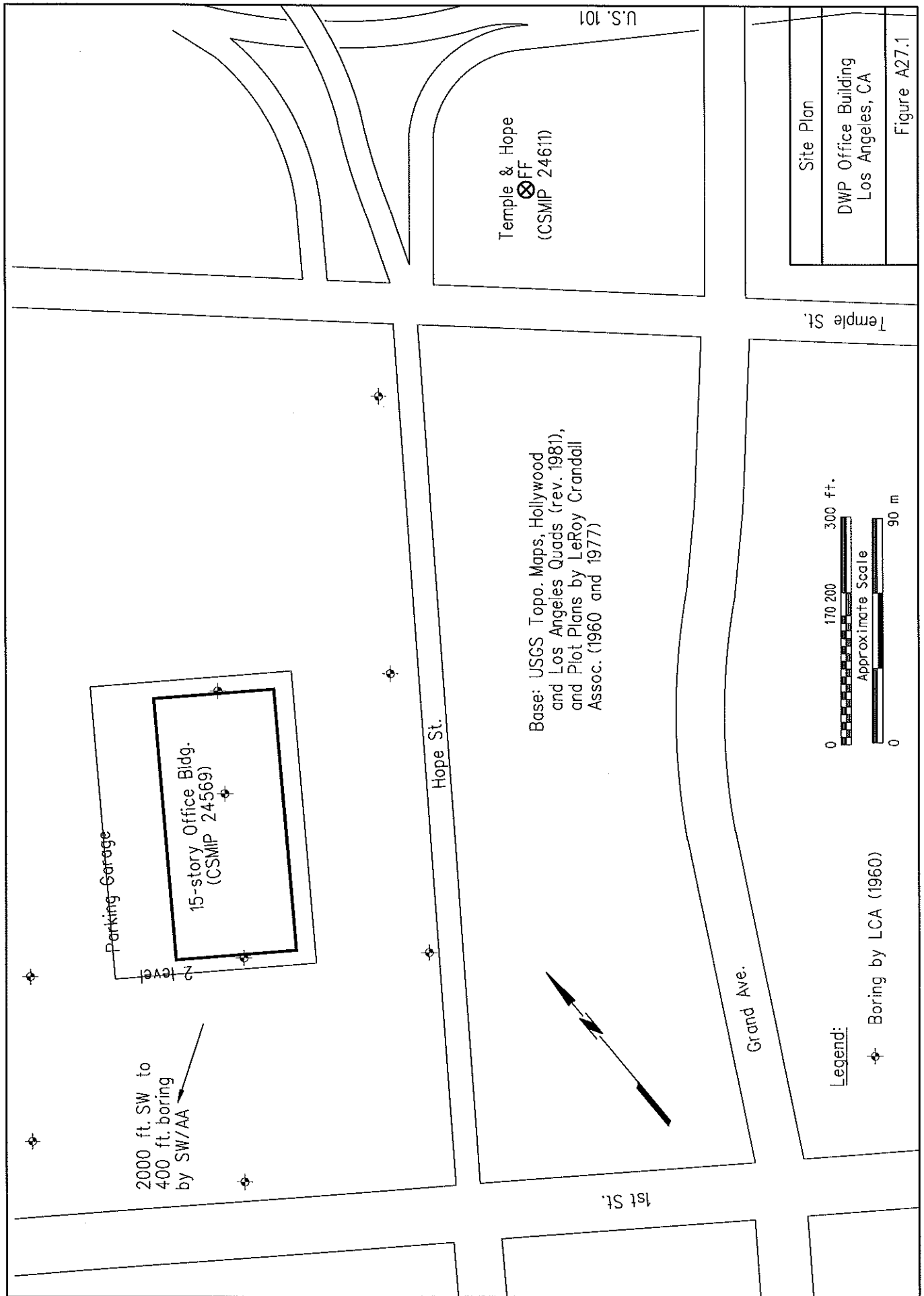
Table A27.4: Empirical and predicted inertial interaction effects

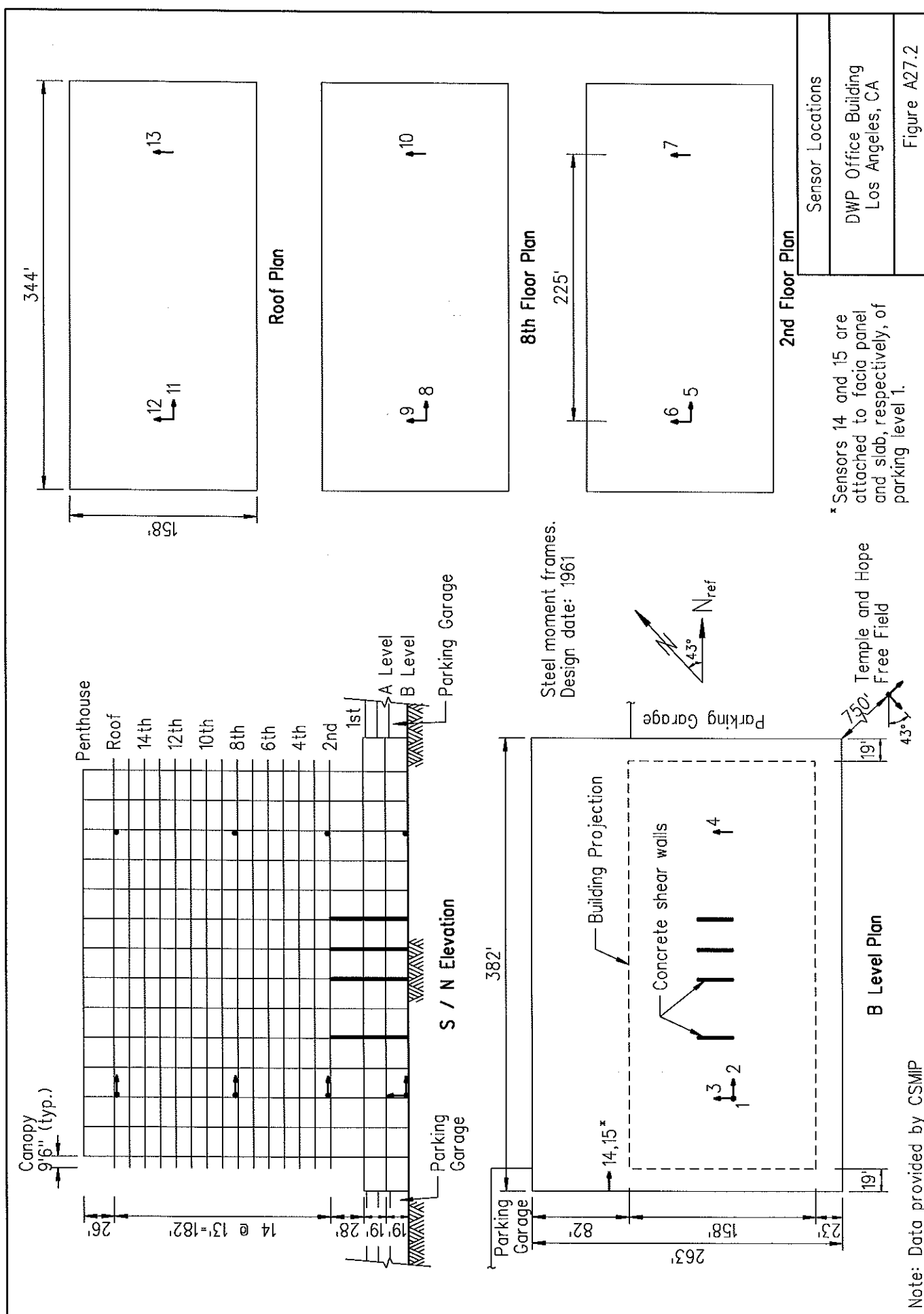
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LD	Sys. ID	0.99	1.3	4.8	1.00	0.5	2.4
	Prediction	1.02	0.1	3.3	1.01	0.1	1.9
NR	Sys. ID	0.98	2.6	6.1	0.98	3.6	6.7
	Prediction	1.02	0.1	3.4	1.01	0.1	3.1

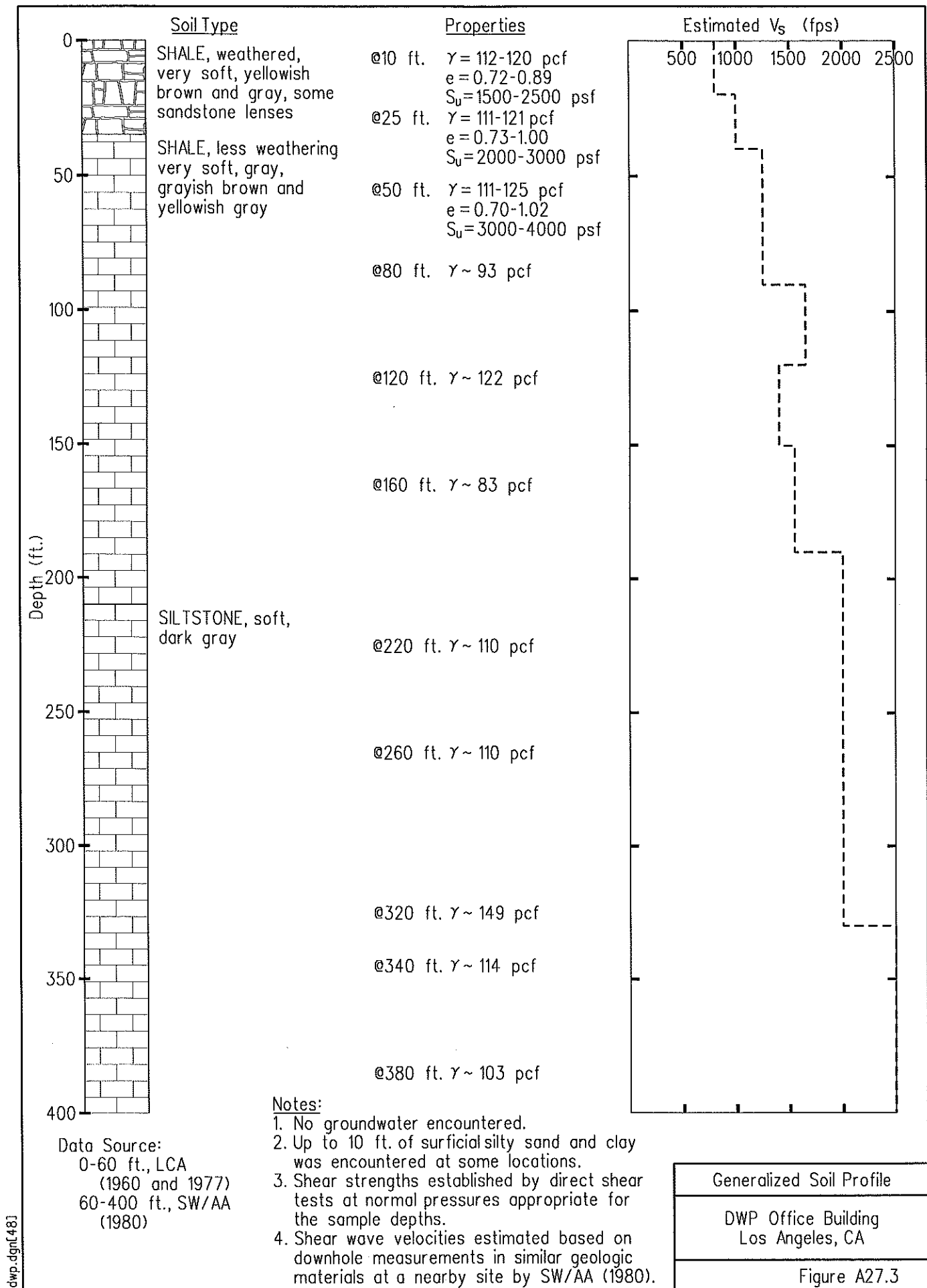
The predictions of period lengthening were reasonable, but the foundation damping was somewhat underpredicted. The underprediction of foundation damping may be associated with radiation damping from the pile foundations, as contributions from piles to foundation impedance are not included in the "modified Veletsos" formulation.

Data Sources

- LeRoy Crandall Assoc., LCA (1960). "Report of Foundation Investigation, Water and Power Administration Building, Los Angeles Civic Center, Los Angeles, CA," *Job. No. 60275*.
- A.C. Martin and Assoc. (1972). "Post-Earthquake Analysis, Department of Water and Power Headquarters Building, Los Angeles, CA," prepared for the Structural Engineers Association of Southern California.
- Shannon and Wilson/Agbabian Assoc., SW/AA (1980). "Verification of Subsurface Conditions at Selected "Rock" Accelerograph Stations in California," *Rpt. NUREG/CR-0055, Vol. 3*, prepared for U.S. Nuclear Regulatory Commission.







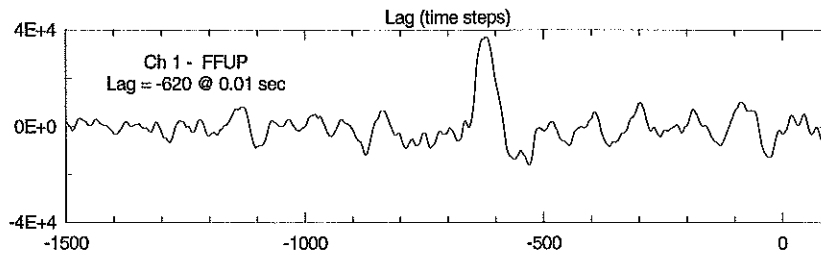


Fig. A27.LD0: Cross correlation function between vertical 'B' level and free-field accelerations

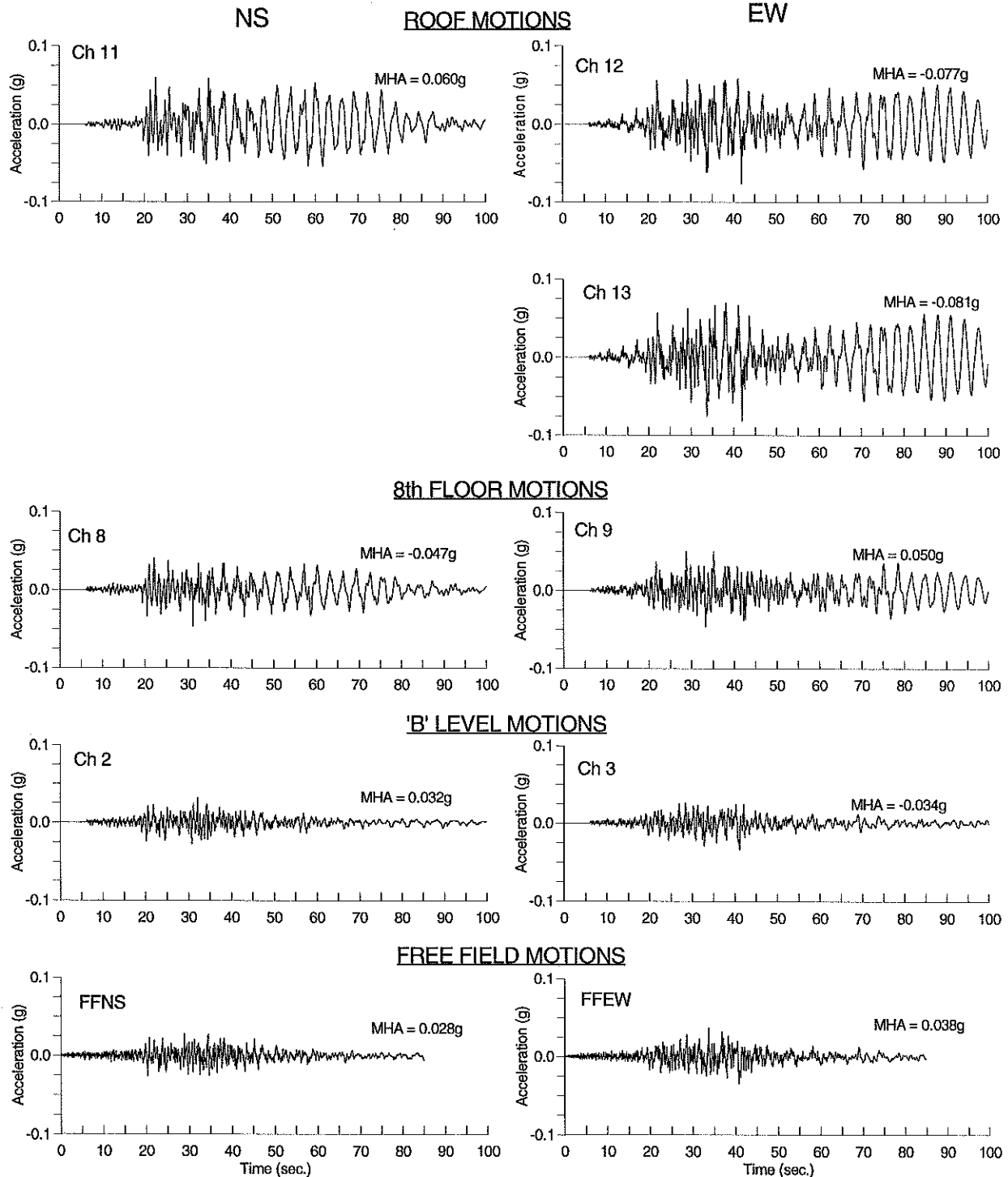


Fig. A27.LD1: LA 15 STORY DWP OFFICE BUILDING

Horizontal Acceleration Time Histories

1992 Landers Earthquake

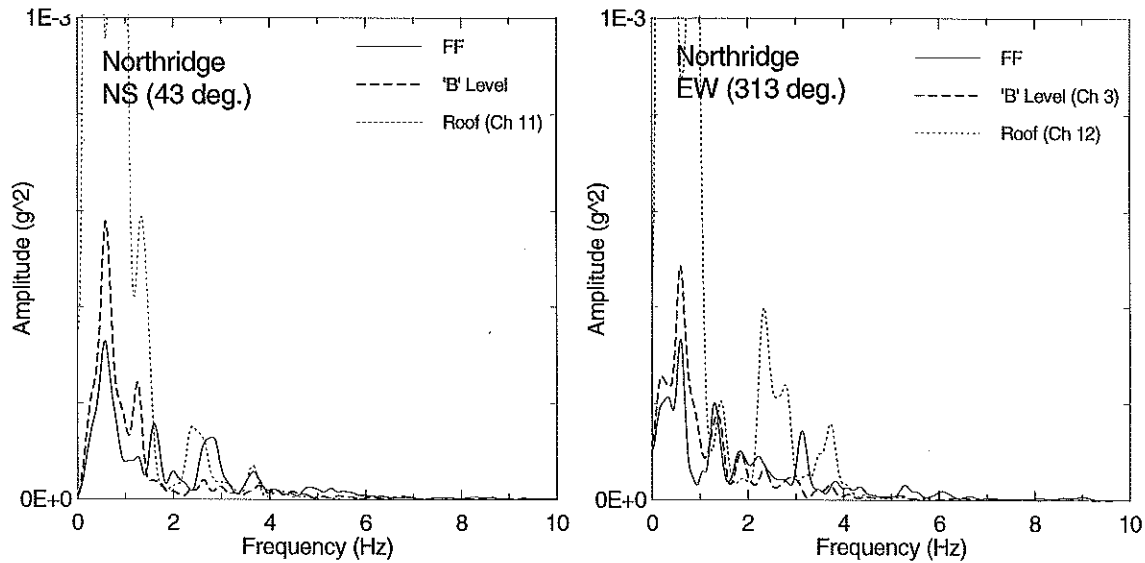


Fig. A27.LD2: LA 15 STORY DWP OFFICE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

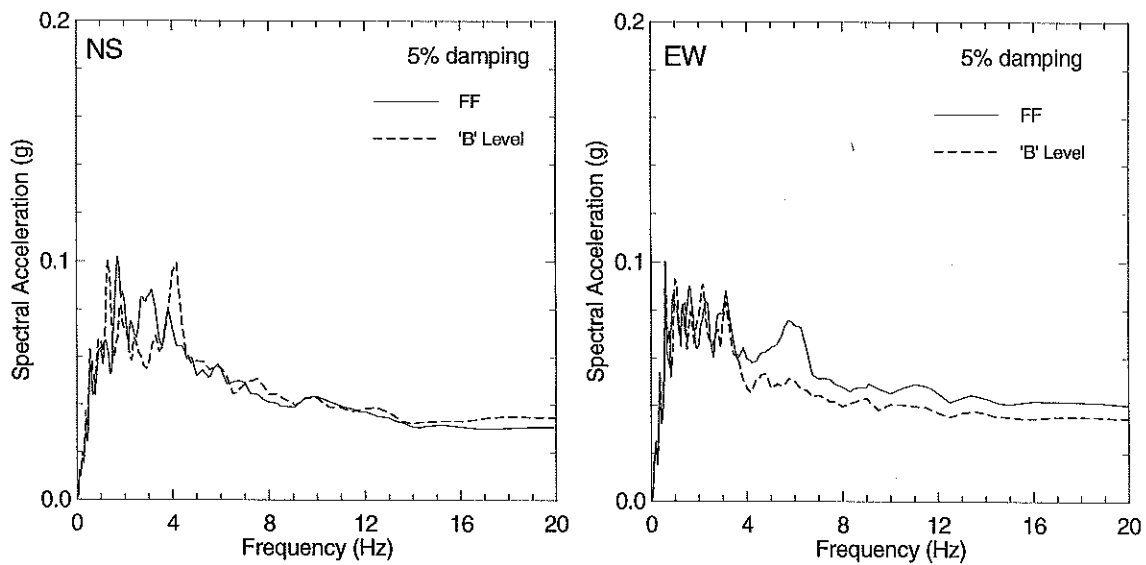


Fig. A27.LD3: LA 15 STORY DWP OFFICE BUILDING
Acceleration Response Spectra of 'B' Level and Free-Field Motions
1992 Landers Earthquake

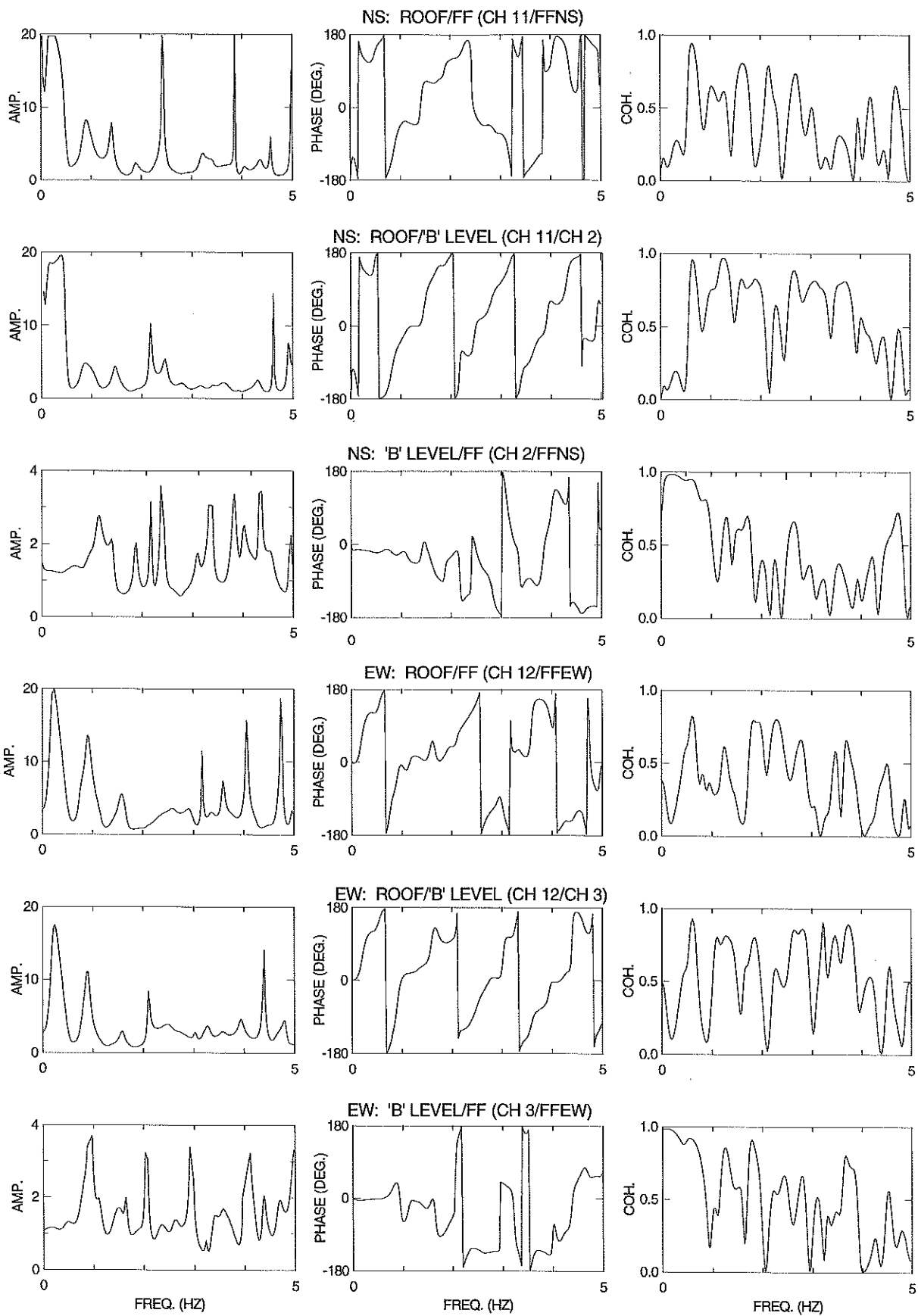


Fig.A27.LD4: LA 15 STORY DWP OFFICE BUILDING
Transmissibility and Coherency Functions
1992 Landers Earthquake
A27-10

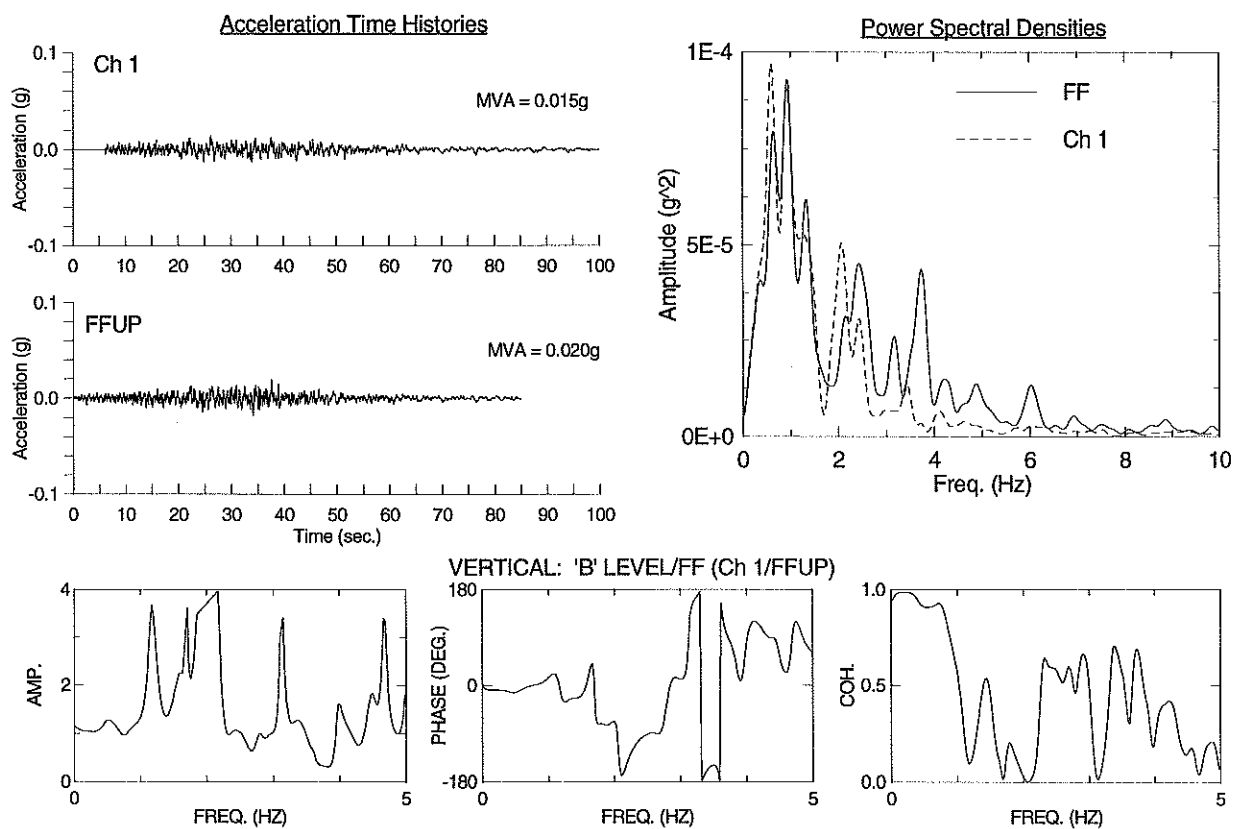


Fig. A27.LD5: LA 15 STORY DWP OFFICE BUILDING
Analysis of Vertical Motions
1992 Landers Earthquake

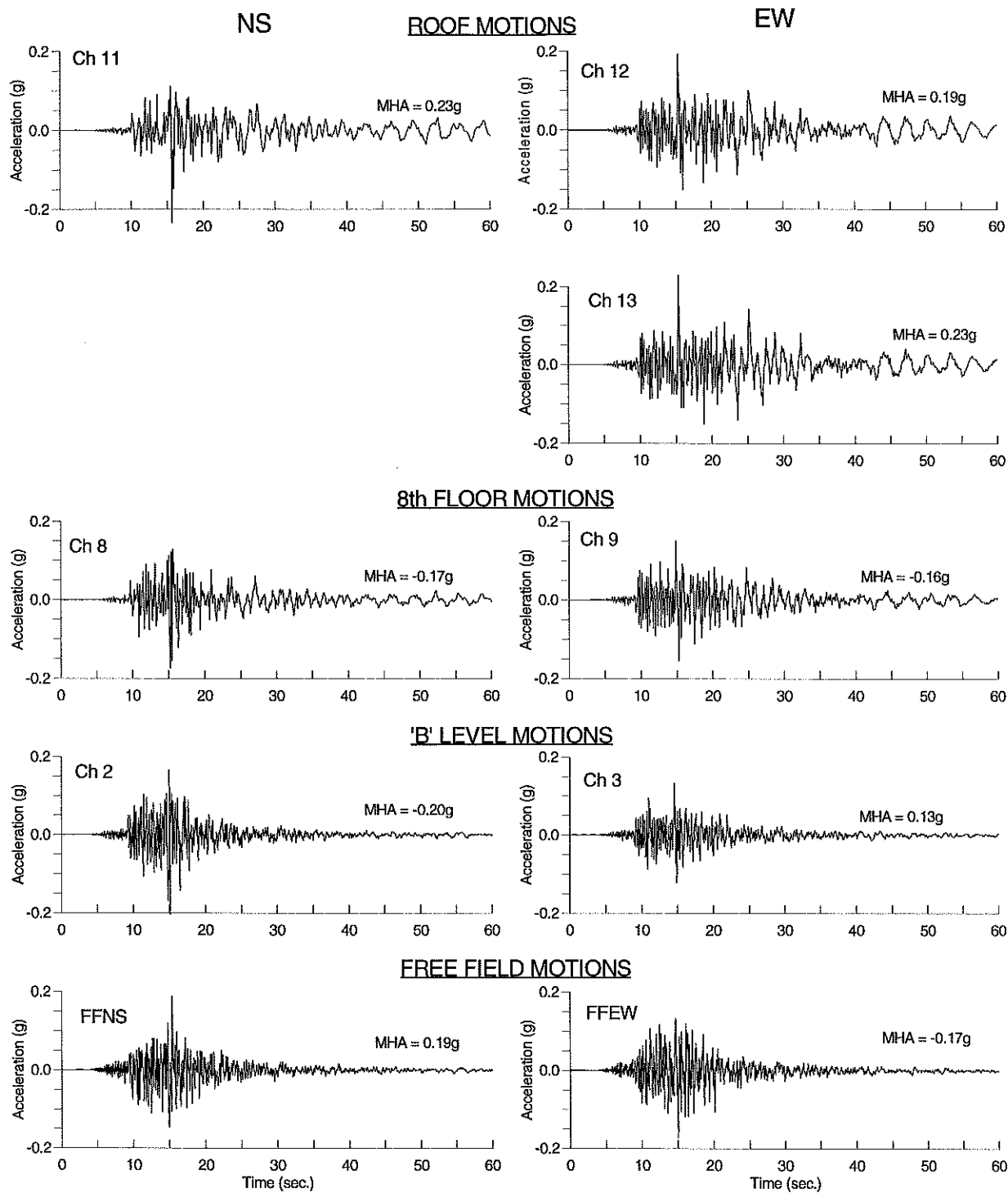


Fig. A27.NR1: LA 15 STORY DWP OFFICE BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

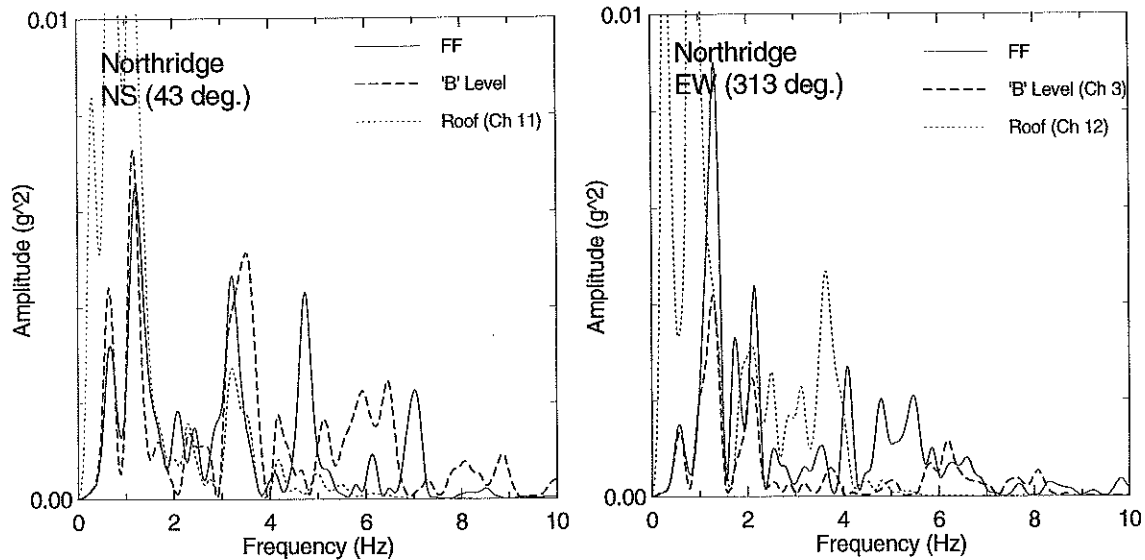


Fig. A27.NR2: LA 15 STORY DWP OFFICE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

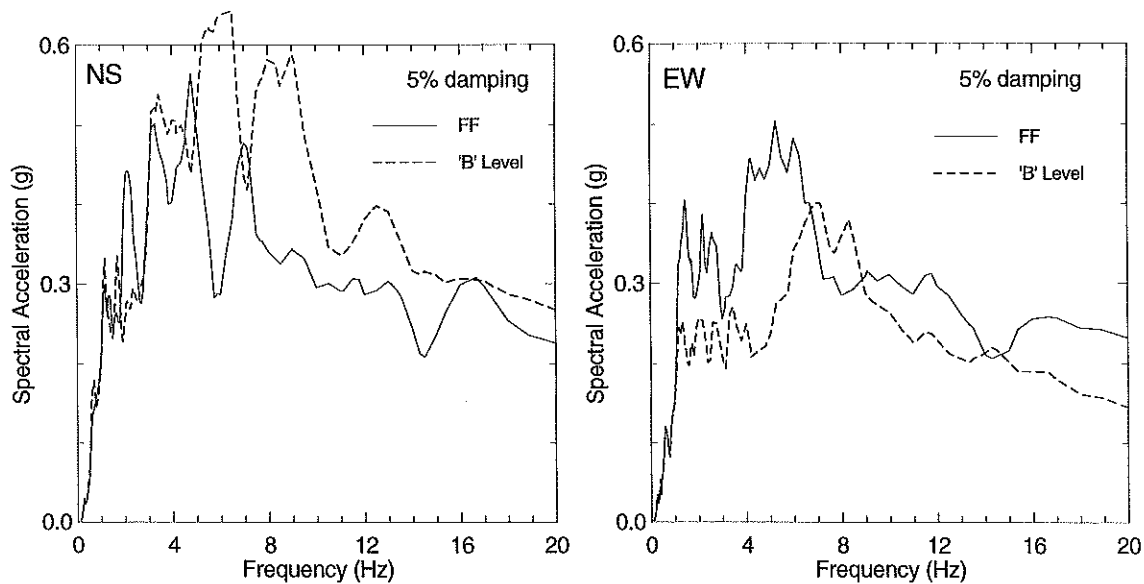


Fig. A27.NR3: LA 15 STORY DWP OFFICE BUILDING
Acceleration Response Spectra of 'B' Level and Free-Field Motions
1994 Northridge Earthquake

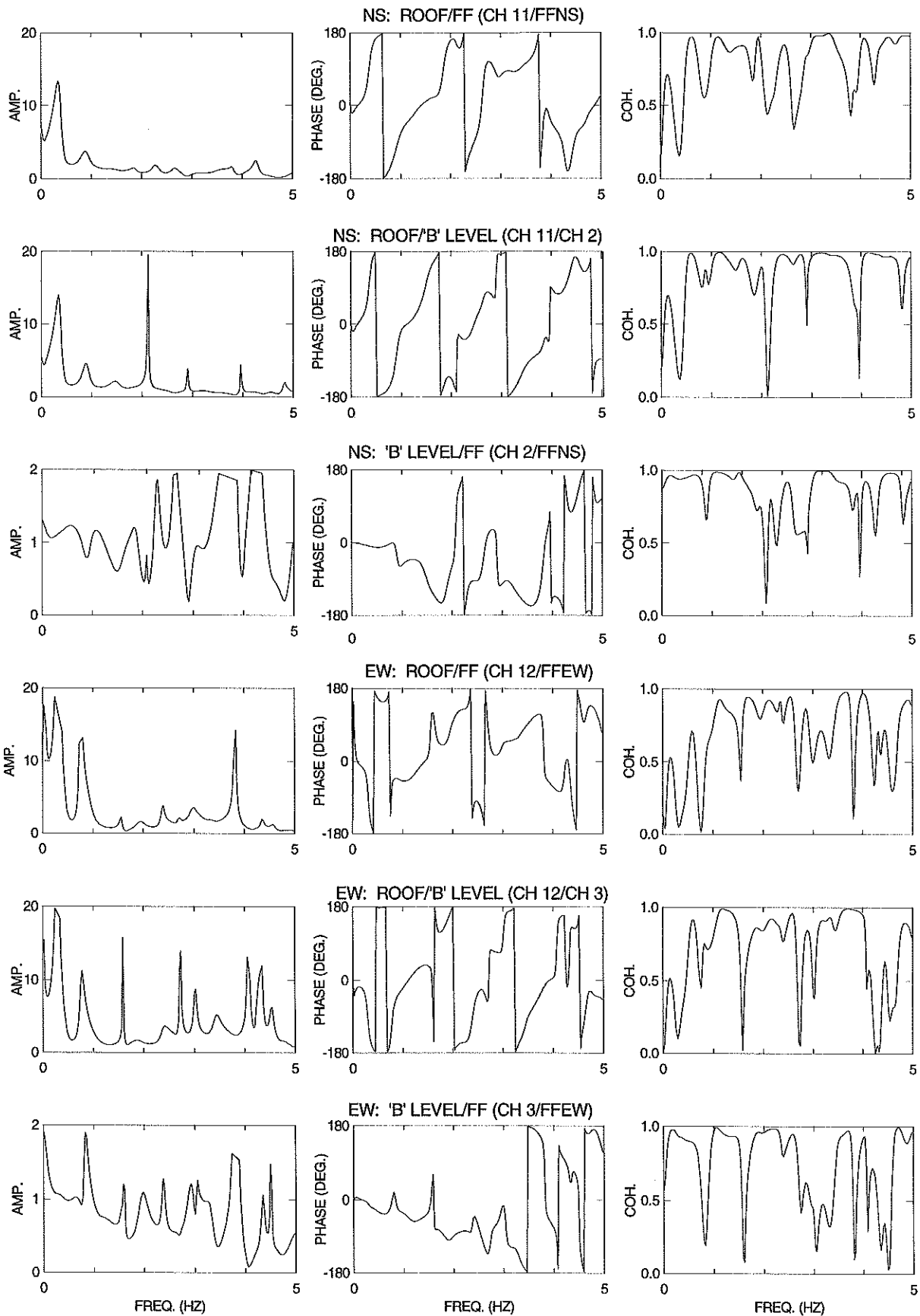


Fig.A27.NR4: LA 15 STORY DWP OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A27-14

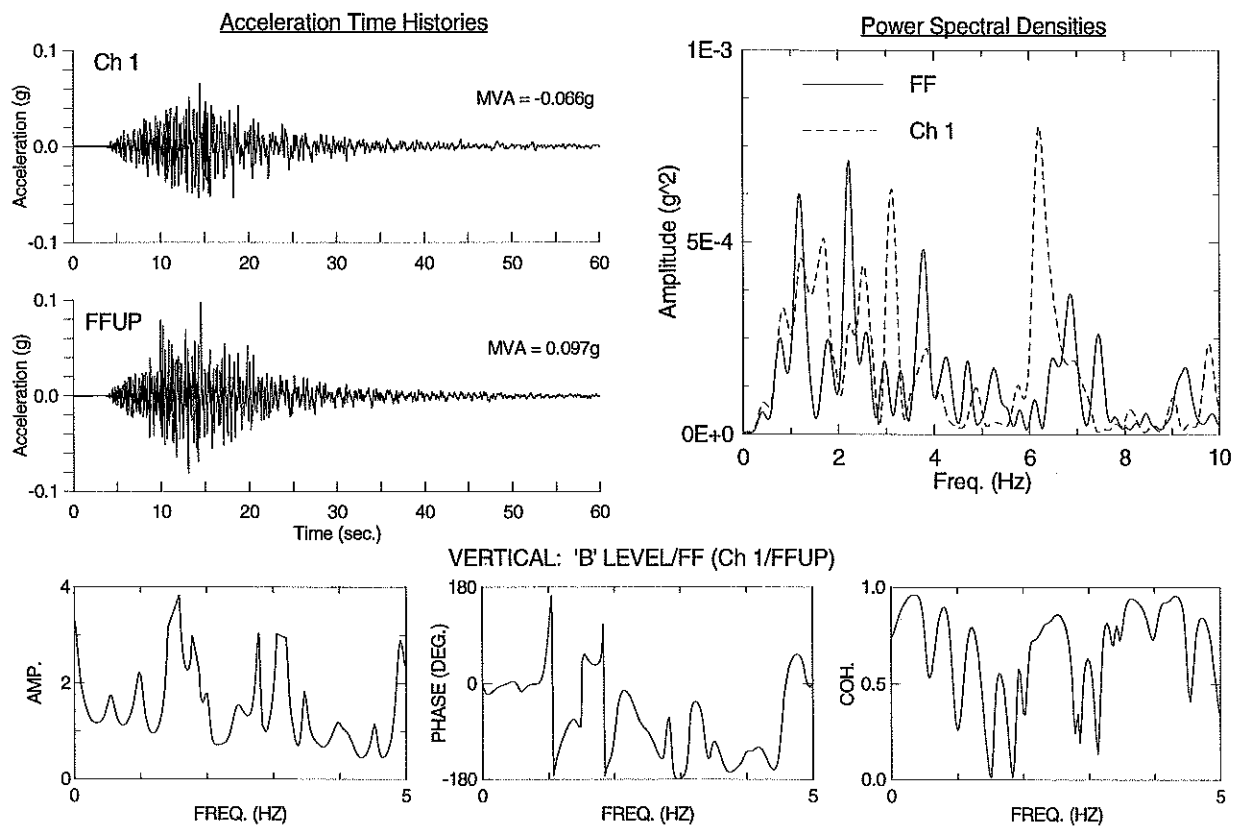


Fig. A27.NR5: LA 15 STORY DWP OFFICE BUILDING
Analysis of Vertical Motions
1994 Northridge Earthquake

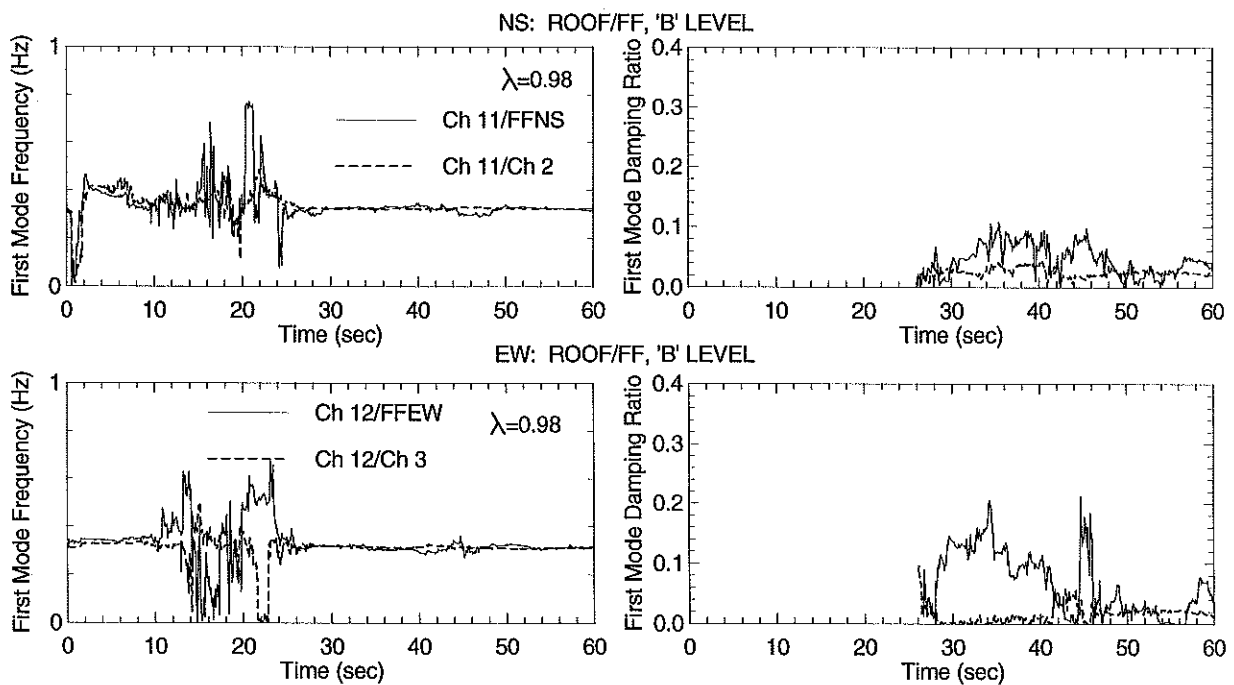


Fig. A27.NR6: LA 15 STORY DWP OFFICE BUILDING
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A28

Site Name: LA 19 Story Office Building
Sensors: CSMIP 24643/24390

Lat: 34.059°
Long.: 118.416°

Floor Levels: 23 + penthouse
Bsmnt. Levels: 4

Structure: Steel moment frames and braced frames

Foundation: piles
Site Geology: soil
Design Date: 1967

Recorded Peak Accelerations (g)

Earthquake	Free-Field			'D' Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.28	0.25	0.085	0.20	0.32	0.13	0.60	0.32	--

-- = No instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: *Str. Data*: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$; FF not aligned with str; FF/str. data not synchronized. *FF Data*: digitized and processed in this study, $f_{\text{HP}} = 0.13\text{--}0.20\text{ Hz}$, $f_{\text{LP}} = 12.5\text{--}25\text{ Hz}$, $f_{\text{Nyq}} = 100\text{ Hz}$.

Final: FF data rotated 46° CCW to align with str.; str. data decimated by 4, FF decimated by 8, to $f_{\text{Nyq}} = 12.5\text{ Hz}$; FF/str. data synchronized using cross-correlation of vertical velocities (Fig. A28.NR0).

Figures

A22.1, A22.3 Site Plan, Generalized Soil Column
A28.1 Sensor Locations
A28.NR0-6 NR Analysis Results

View of structure,
looking west



DISCUSSION

Site Description

The building is 19-stories tall above the ground surface with an enlarged basement which extends beyond the perimeter of the rectangular tower. The tower has plan dimensions of 110 by 240 feet. The structural system in the 4-level basement consists of reinforced concrete columns and slabs. From the 1st floor to the roof, X-braced steel frames are present in the transverse (NS) direction and moment resisting steel frames are present in the longitudinal (EW) direction. The foundation beneath the tower consists of 72-foot long steel I-beam piles installed in groups of 3 to 10 beneath each column. Pile caps vary in thickness from 3.7 to 5.7 feet, and the pile caps are connected by 2-foot square reinforced concrete tie beams (Hart, 1973). As shown in Fig. A28.1, accelerographs are present on the 'D', 1st, 2nd, 8th, and roof levels of the structure. The free-field instrument is located about 940 feet north of the building in a golf course.

Instrument Selection and Free-Field Conditions

Transverse response was evaluated using sensors along the west wall, and longitudinal response was evaluated from sensors near middle of the building (Table A28.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A28.NR1.

Table A28.1: Sensors in structure used for analyses

'D' Level (D)			Roof (R)	
NS	EW	V	NS	EW
Ch 2	Ch 4	Ch 1	Ch 13	Ch 15

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 'D' Level and Free-Field Motions

D/FF transmissibility functions (Fig. A28.NR4) and the spectra of horizontal motions (Fig. A28.NR2-3) indicate increasing de-amplification of 'D' level motions with frequency in both directions.

Transmissibility, coherency, and power spectral density functions for the FF and 'D' level vertical motions are presented in Fig. A28.NR5. No amplification is evident near the first-mode frequency, indicating that base rocking was unlikely to have significantly affected the structural response. The apparent amplification of 'D' level motions at higher frequencies (i.e. > about 2 Hz) is probably due to noise. Rocking at these high frequencies is unlikely.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A28.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A28.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A28.NR6).

Table A28.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (D-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=9, j=19		L (EW) d=20, j=22		tr (NS) d=21, j=19		L (EW) d=17, j=22	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	--	--	--	--	0.30	6.6	0.26	16.4	0.31	8.9	0.27	19.9
	2					1.20	4.3	0.72	5.2	1.23	2.2	0.75	5.4
	3					2.56	6.0	1.29	7.6	2.41	5.0	1.34	6.1
	4					4.39	4.6	2.38	5.3	4.21	4.23	2.41	3.1

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first four modes in both directions.
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical D/FF transmissibility function, fixed-base parameters can be approximated by the pseudo flexible-base parameters.
- The large number of modes used in the CEM analyses was necessary to develop adequate models of the structural behavior in the lower modes.
- CEM first-mode parameters for 1st floor-roof response were computed for comparison with D-R parameters. The frequencies identified from these analyses were essentially identical.

Using data from the 1971 San Fernando Earthquake, Hart (1973) analyzed Fourier amplitudes of roof records and D-R transfer functions to evaluate modal frequencies of 0.29, 1.23, and 2.5 Hz (transverse) and 0.29, 0.72, and 1.28 Hz (longitudinal). Using the halfpower bandwidth method, Hart also evaluated modal damping ratios of 3.1 to 3.4% in the first mode. The agreement between the modal frequencies identified by Hart and the CEM results in Table A28.2 is favorable, though damping from the NR data is significantly higher.

The RPEM results (Figs. A28.NR6) indicate no significant time dependent changes in the first-mode frequencies of the structure. The damping results are erratic, likely due to the high model order.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A28.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 130 feet using the recorded free-field motions and the profile in Fig. A22.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 1.5 and 1.7, respectively, to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A28.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
NR	Model = H	$V_s = 1160$ fps	$h = 220$ ft	emb. = 38 ft	$\gamma = 0.15$
	$\nu = 0.45$		tr: $r_1 = 92$ ft		$r_2 = 76$ ft
	$\beta^* = 5.9\%$	$V_s^* = 980$ fps	L: $r_1 = 92$ ft		$r_2 = 113$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A28.3 with the fixed-base modal parameters in Table A28.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A28.4.

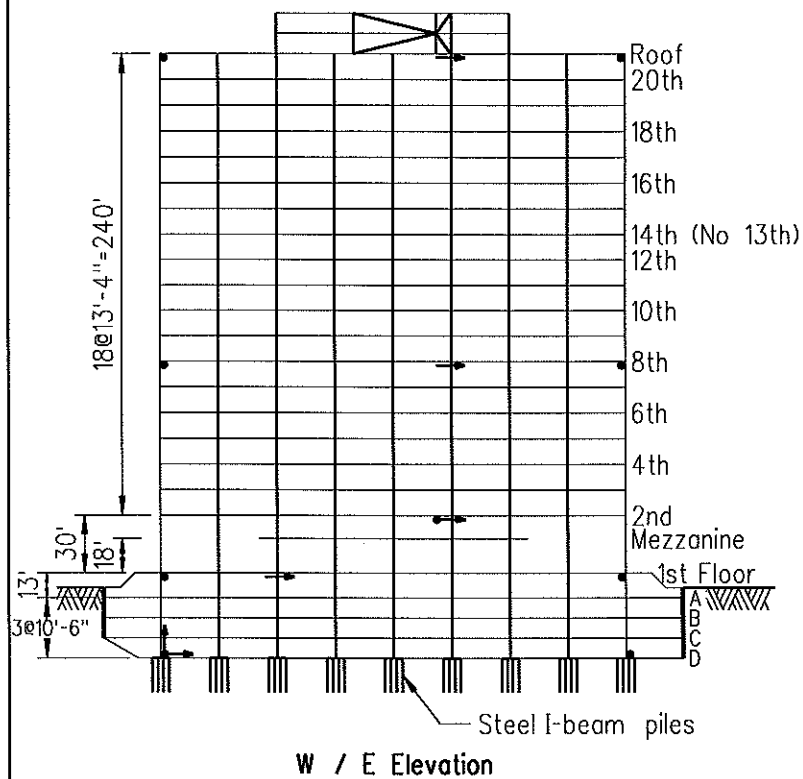
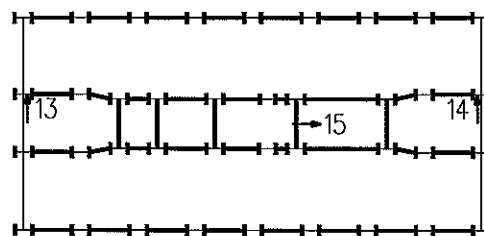
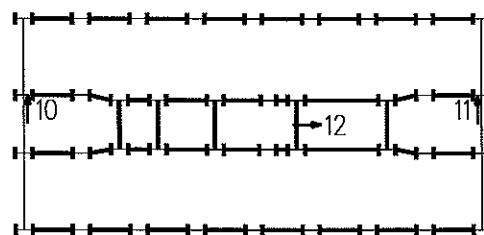
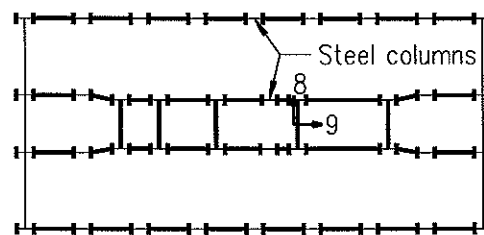
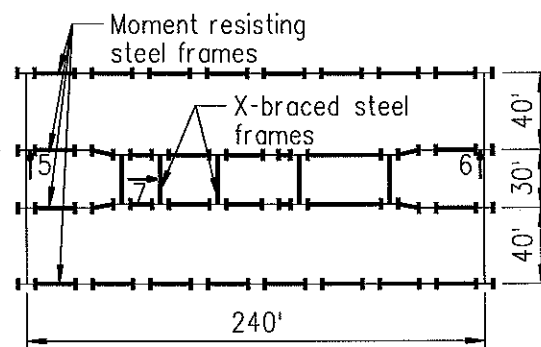
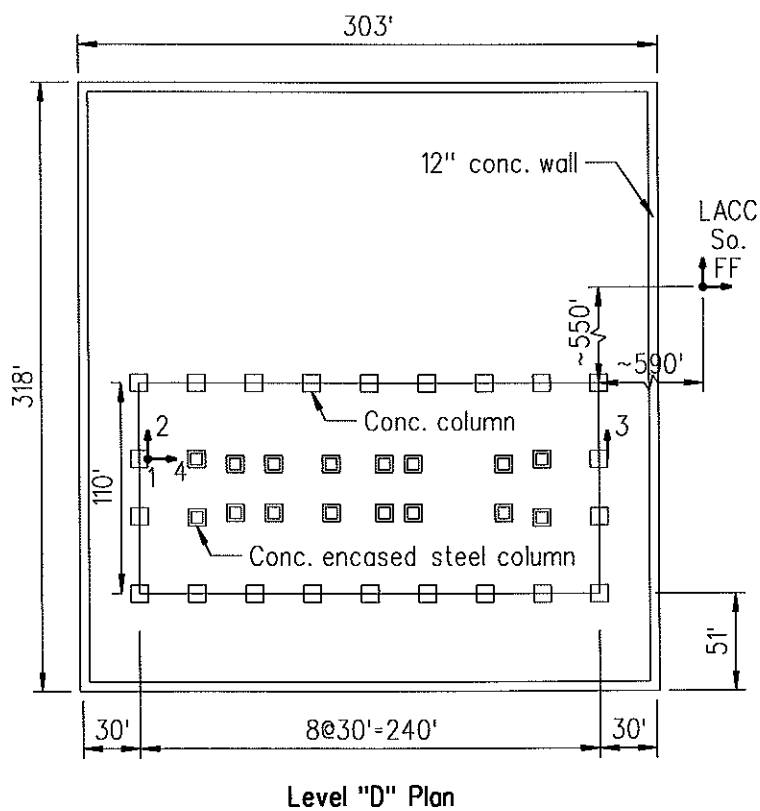
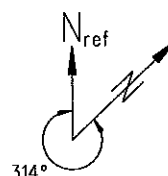
Table A28.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	0.96	2.3	8.9	0.96	3.5	19.9
	Pre.-Vel.	1.02	0.1	6.3	1.01	0.0	16.1
	Pre.-Bie.	1.02	0.4	6.5	1.02	0.2	15.9

The predictions of period lengthening were reasonable, but the foundation damping was underpredicted. The underprediction of foundation damping may be associated with radiation damping from the pile foundations, as contributions from piles to foundation impedance are not included in either the “modified Veletsos” or “modified Bielak” formulations.

Data Sources

Hart, G.C. (1973). “1901 Avenue of the Stars Building,” *San Fernando, California, Earthquake of February 9, 1971*, U.S. Dept. of Commerce, NOAA, Vol. 1B, 597-607.



Note: Data provided by CSMIP

Sensor Locations
19-story Building Century City Area Los Angeles, CA
Figure A28.1

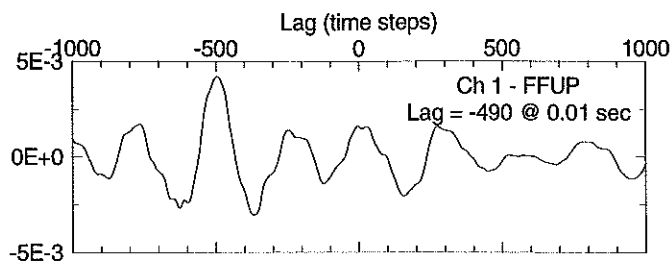


Fig. A28.NR0: Cross-correlation function between vertical "D" level and free-field velocities

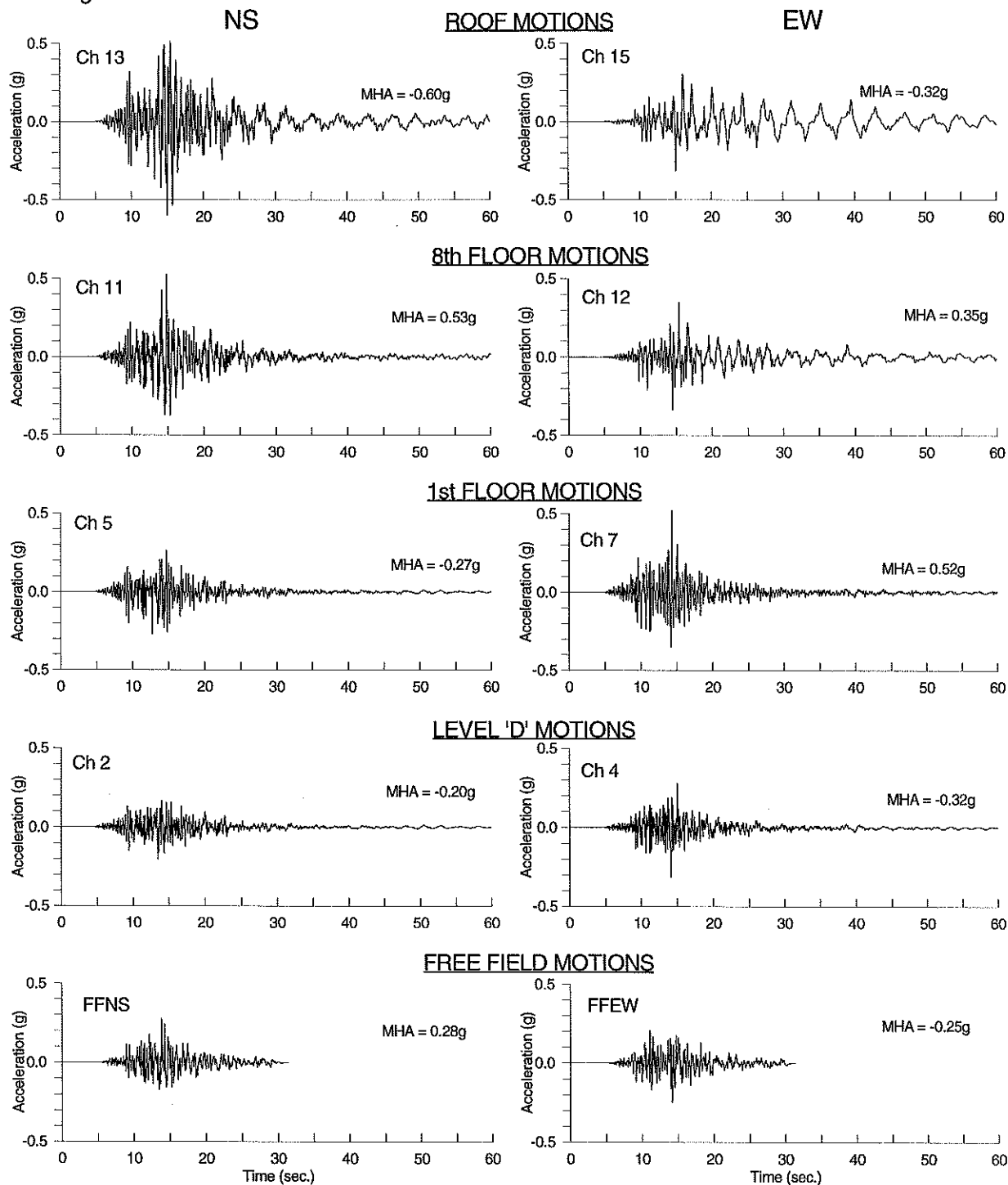


Fig. A28.NR1: LA 19 STORY OFFICE BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A28-6

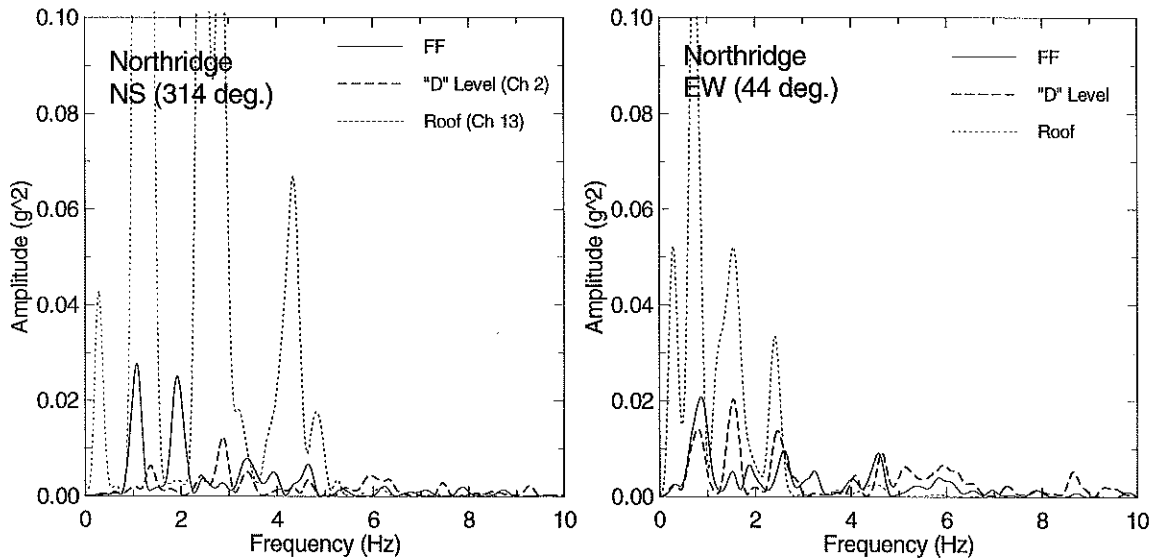


Fig. A28.NR2: LA 19 STORY OFFICE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

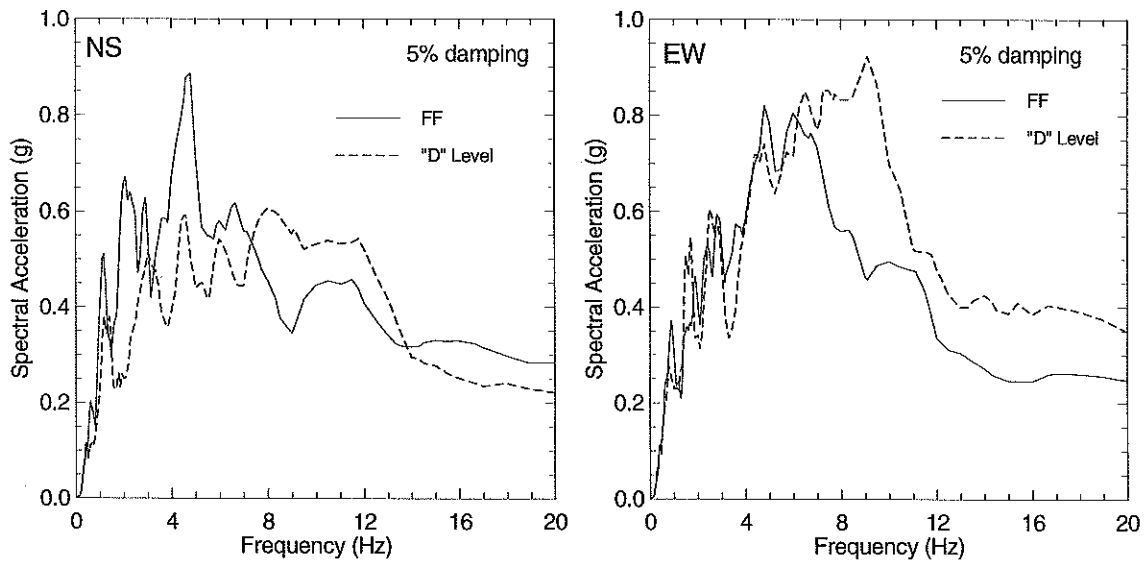


Fig. A28.NR3: LA 19 STORY OFFICE BUILDING
Acceleration Response Spectra of "D" Level and Free-Field Motions
1994 Northridge Earthquake

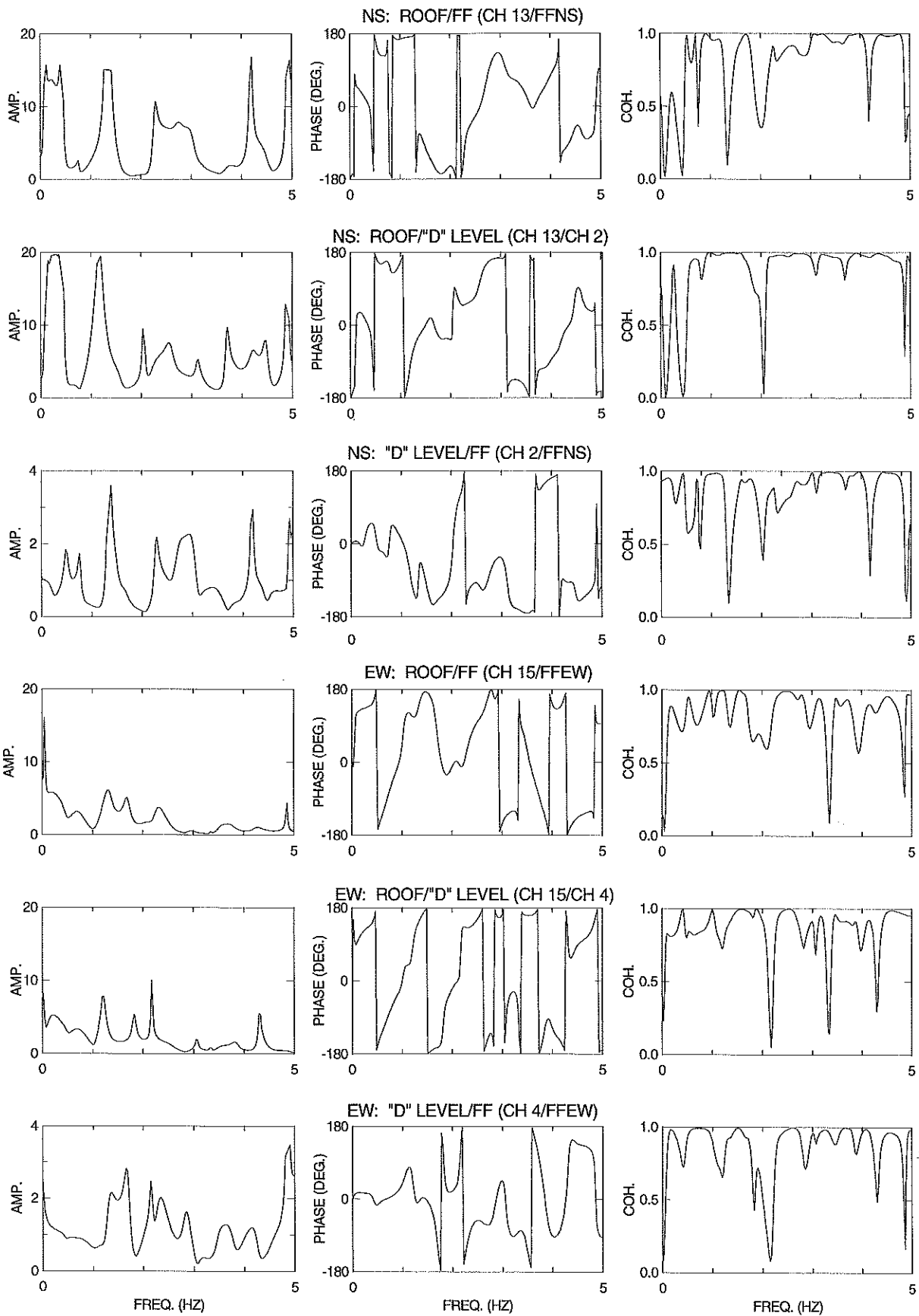


Fig.A28.NR4: LA 19 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A28-8

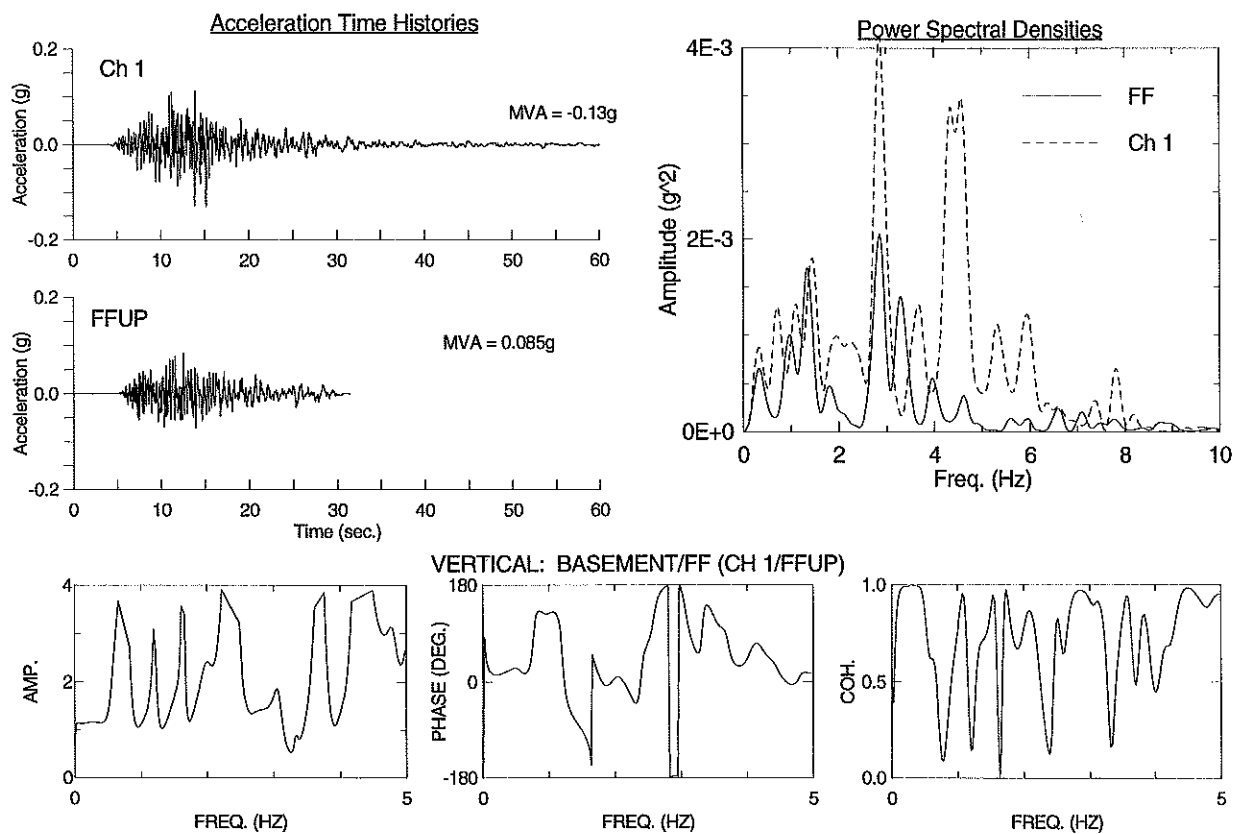


Fig. A28.NR5: LA 19 STORY OFFICE BUILDING
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

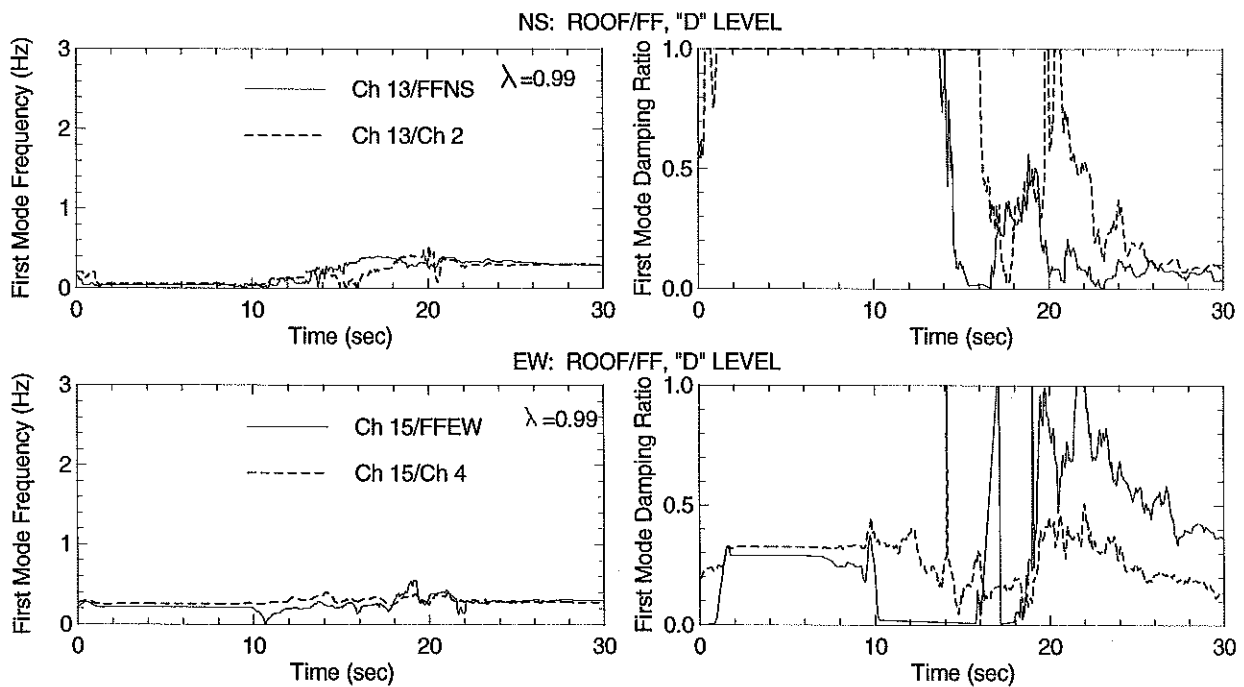


Fig. A28.NR6: LA 19 STORY OFFICE BUILDING
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A29

Site Name: LA Hollywood Storage Building
 Sensors: CSMIP 24236/24303
 Floor Levels: 15 + penthouse
 Bsmt. Levels: 1
 Structure: Concrete frames with infilled panels

Lat: 34.090°
 Long.: 118.338°
 Foundation: concrete piles
 Site Geology: Alluvium
 Design Date: 1925

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1952 Kern County	0.059	0.042	0.021	0.055	0.044	0.023	0.12	0.15	0.020
1971 San Fernando	0.17	0.21	0.088	0.11	0.15	0.050	--	--	--
1987 Whittier	0.21	0.11	0.067	0.11	0.058	0.038	0.20	0.19	--
1992 Landers*	--	--	--	0.03	0.04	0.01	0.12	0.10	--
1992 Big Bear*	0.03	0.03	0.01	0.03	0.03	0.02	0.09	0.08	--
1994 Northridge	0.39	0.23	0.23	0.28	0.21	0.086	0.49	--	--

*Not digitized

-- = Nonfunctional or no instrument

Preprocessing

Kern County Earthquake (KN)

Prelim.: Caltech.; filtered $f = 0.07\text{--}25\text{ Hz}$, $f_{Nyq}=25\text{ Hz}$, FF/Bsmt. synch., Bsmt./Roof not synch.

Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$, could not synchronize Roof and Bsmt. motions

San Fernando Earthquake (SF)

Preliminary: Caltech.; filtered $f = 0.07\text{--}25\text{ Hz}$; $f_{Nyq}=25\text{ Hz}$, FF/Bsmt. synch., no roof records

Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$.

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.4\text{--}23\text{ Hz}$; $f_{Nyq}=25\text{ Hz}$; FF/str. data synch.

Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$.

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}23\text{ Hz}$ (str.), $0.16\text{--}23\text{ Hz}$ (FF); $f_{Nyq}=25\text{ Hz}$; FF/str. synch.

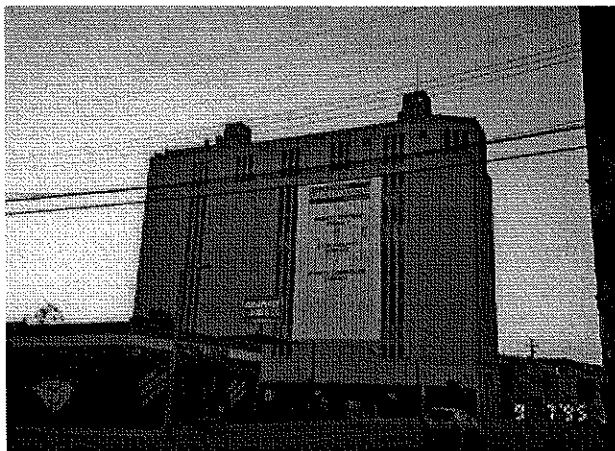
Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$.

Figures

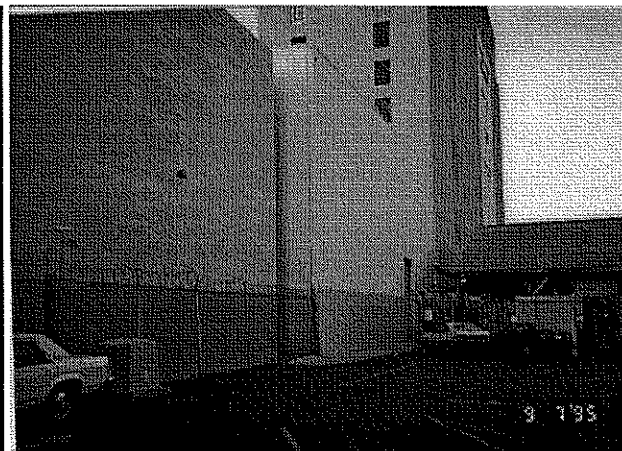
A29.1-A29.3 Site Plan, Sensor Locations, Generalized Soil Column

A29.KN/SF1-5 KN and SF Analysis Results

A29.WT/NR1-6 WT and NR Analysis Results



View of structure, looking north



View of west side of structure, FF in foreground
 (note proximity of warehouse to FF instrument)

DISCUSSION

Site Description

The building is 14-stories tall with a single level basement beneath the western side of the structure. The structure is rectangular, 51 by 217 feet in plan, and is connected on its west side to a 4-story warehouse. The lateral load resisting system consists of reinforced concrete frames in both directions. Longitudinal frames and transverse frames on the western side of the structure are infilled with 8-inch thick concrete panels (Fenves and Serino, 1992). The foundation consists of concrete piles varying in length from 12 feet at the edge of the building to 30 feet near the building center. As shown in Fig. A29.2, accelerographs are present in the basement, 8th, 12th and roof levels. The free-field instrument is located 139 feet west of the structure, but only about 6 feet south of another 4-story warehouse building constructed in 1986 (see photograph above).

The site is located near the north end of the Los Angeles Basin, about 8000 feet south of the Santa Monica Mountains. Based on a 360-foot boring by SW/AA (1980) and a 100-foot boring by Fumal et al. (1982), the generalized soil profile consists of (Fig. A29.3): about 90 feet of very stiff to hard sandy silty clays with occasional sand layers, about 60 feet of very dense silty sand with gravel zones, about 150 feet of hard sandy, clayey silt belonging to the Repetto Formation, and hard sandy silty clay which extends to the maximum depth explored of 360 feet. The Repetto Formation is often characterized by poorly indurated siltstone, sandstone, and shale, but these materials were classified as soil by SW/AA, as the engineering properties of these materials were more similar to a hard soil than a soft rock. The depth to firm bedrock has been estimated as about 350 to 400 feet by Duke and Leeds (1962). Using the subsurface data and power spectra of the free-field motions (Figs. A29.KN2, SF2, WT2, NR2), site frequencies are estimated as 1 Hz for the full soil profile and 2 Hz for the softer soils in the upper 150 feet of the profile. The significant energy in the NR and WT free-field motions at about 5 Hz (especially in the NS direction) is likely associated with vibrations of the 4-story warehouse located 6-feet north of the instrument. Significant energy at this frequency is not present in the SF or KN motions, as the 4-story warehouse had not yet been constructed.

Instrument Selection and Free-Field Conditions

For the KN and SF events, structural instrumentation was only present at the roof and basement levels, but the roof instrument was not triggered by SF. Fig. A29.2 shows the more extensive instrumentation present during the WT and NR events. Sensors near the building centroid were selected for analysis (Table A29.1). Some torsional response is likely due to the partial basement beneath the western side of the building. As shown in Figs. A29.WT/NR1, roof accelerations at the west end of the structure are less than those near the centroid. It should be noted that Ch 11 did not trigger during NR, so Ch 8 (a 12th-floor record) was used instead.

Table A29.1: Sensors in structure used for analyses (WT and NR events)

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 1	Ch 3	Ch 2	Ch 10	Ch 11 (8)

FF motions are affected by vibrations of a four-story warehouse 6 feet from the instrument. This warehouse was constructed in 1986, and significant FF energy at about 5 Hz (a potential vibration frequency of this structure) is evident in subsequently recorded WT and NR data, but not in earlier KN or SF data (Fig. A29.**2). The subject 14-story building may have also affected FF recordings, as notable FF energy at $\tilde{f}_1$ [≈ 0.5 Hz (tr), ≈ 1.3 -2 Hz (L)] is present. However, for the longitudinal direction (in which SSI effects are greatest), the site and structure frequencies are similar, so contamination of FF motions at this frequency is difficult to discern.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A29.**4) and the spectra for horizontal motions (Fig. A29.**2-3) generally indicate increasing de-amplification of basement motions with frequency.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Figs. A29.**5. No significant amplification of vertical basement motions was evident at the transverse first-mode frequency, suggesting that base rocking did not significantly influence the transverse structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Figs. A29.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures, WT and NR events (Table A29.2), and (3) time variation of first-mode parameters from the RPEM, WT and NR events (Figs. A29.WT/NR6). Parametric analyses were not performed with the KN data as the roof and basement motions could not be synchronized. For comparison, frequencies identified from forced vibration tests are presented in Table A29.3.

Table A29.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) WT: d=5, j=13 NR: d=6, j=13		L (EW) d=2, j=6 d=1, j=7		tr (NS) d=0, j=13 d=5, j=13		L (EW) d=--, j=-- d=0, j=7	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	--	--	na	na	0.54	7.1	1.60	9.44	0.56	9.1	na	na
	2					2.11	9.6	--	--	2.12	14.1	na	na
NR	1	0.47	14.6	1.34	8.8	0.46	15.4	1.29	8.1	0.46	16.5	1.25	8.1
	2					1.61	14.4	--	--	1.60	15.4	--	--

Table A29.3: Modal frequencies (in Hz) from forced vibration tests (Carder, 1964)

Mode	NS	EW
1	0.83	2.0
2	2.7	--

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the transverse and longitudinal structural responses were dominated by the lower two modes and the first mode, respectively.
- Fixed-base parameter estimation procedures could not be invoked for the transverse direction (WT event) because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical B/FF transmissibility functions, the fixed-base parameters can be approximated by the pseudo flexible-base parameters. Fixed-base parameters could be estimated for the NR event, and the variation between the fixed- and pseudo flexible-base parameters is small.
- Reliable CEM models for longitudinal response could not be developed from the WT data.
- The transverse first-mode frequency from CEM analyses (NR data) of 0.46 Hz differs from the apparent first-mode frequency in the transmissibility function (about 0.25 Hz). The nonparametric result may be unreliable as 0.25 Hz is near the high-pass corner frequency of 0.2 Hz, and the coherence at this frequency is low.

The above modal parameters from the WT data (B-R case) agree with those identified by Papageorgiou and Lin (1991), who employed both single-output/single-input and multiple channel parametric analyses. Fixed-base parameters were also estimated in their study using only pseudo flexible-base (B-R) parameters. This estimation was based on assumed values of soil shear wave velocity, foundation mass, and centroidal moment of inertia of the floors and foundation. First-mode fixed-base frequencies and damping ratios were estimated to be 1.62-1.78 Hz and 6.5-9.9% for the longitudinal direction and 0.55-0.57 Hz and 6.6-7.8% for the transverse direction. When compared to their B-R parameters of 1.56 Hz and 11.1% (EW) and 0.54 Hz and 7.7% (NS), these results indicate more significant SSI in the EW direction, a result consistent the findings of this study.

The RPEM results indicated no significant time-dependent changes in the first-mode parameters during the WT event. Results from NR data indicate a frequency reduction of about 0.25 Hz in the longitudinal (EW) direction for $t < 12$ seconds (the time period of highest amplitude shaking). No significant frequency reductions for the transverse (NS) direction were observed.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A29.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 59 feet using the recorded free-field motions and the profile in Fig. A29.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 3.0 and 2.0 to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Using the parameters in Table A29.4 with the fixed-base modal parameters in Table A29.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A29.5.

Table A29.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 955$ fps	$h = 96$ ft	emb. = 9 ft	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 59$ ft		$r_2 = 42$ ft
WT	$\beta^* = 2.8\%$	$V_s^* = 930$ fps	L: $r_1 = 59$ ft		$r_2 = 86$ ft
NR	$\beta^* = 4.4\%$	$V_s^* = 880$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

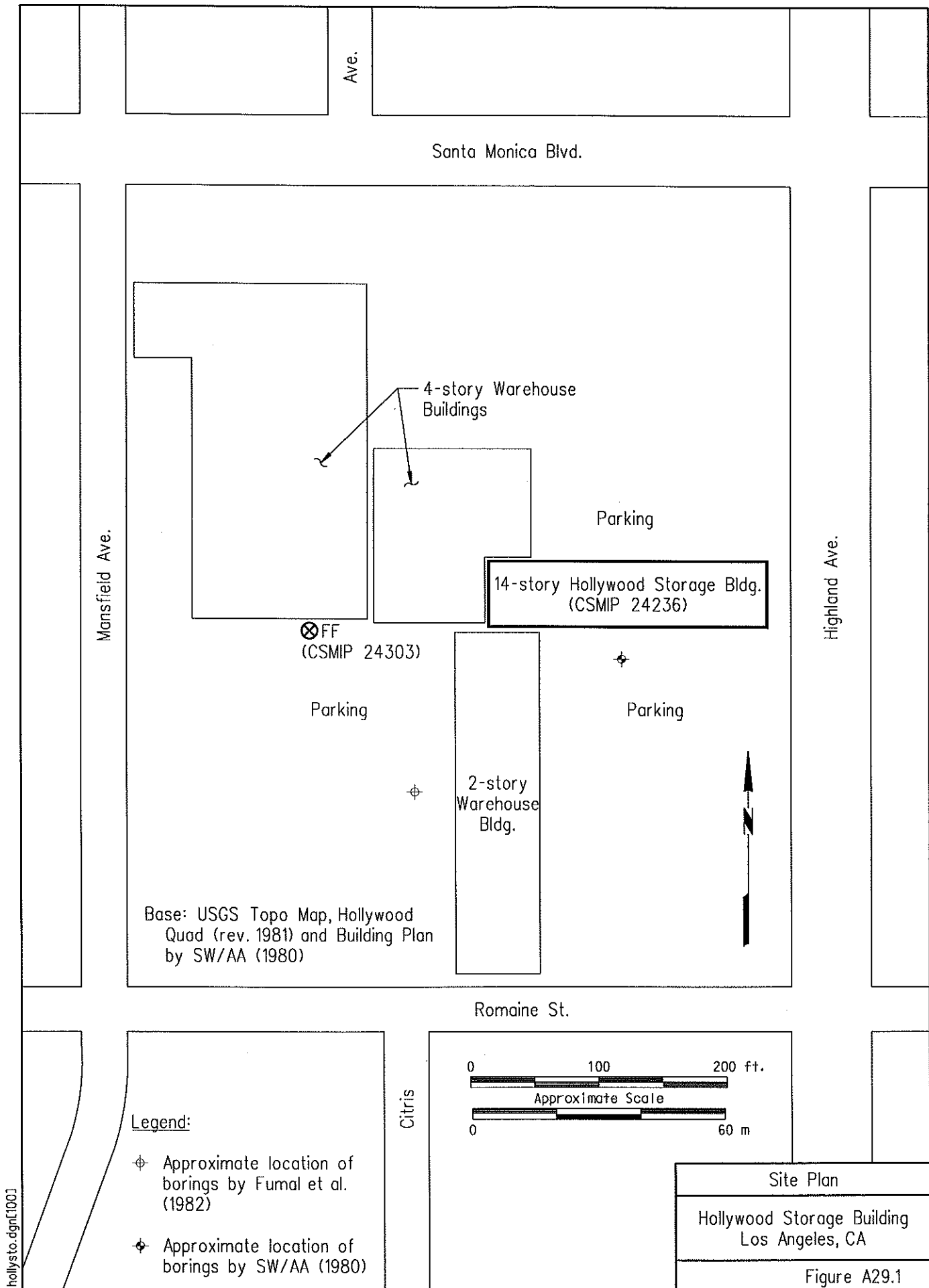
Table A29.5: Empirical and predicted inertial interaction effects

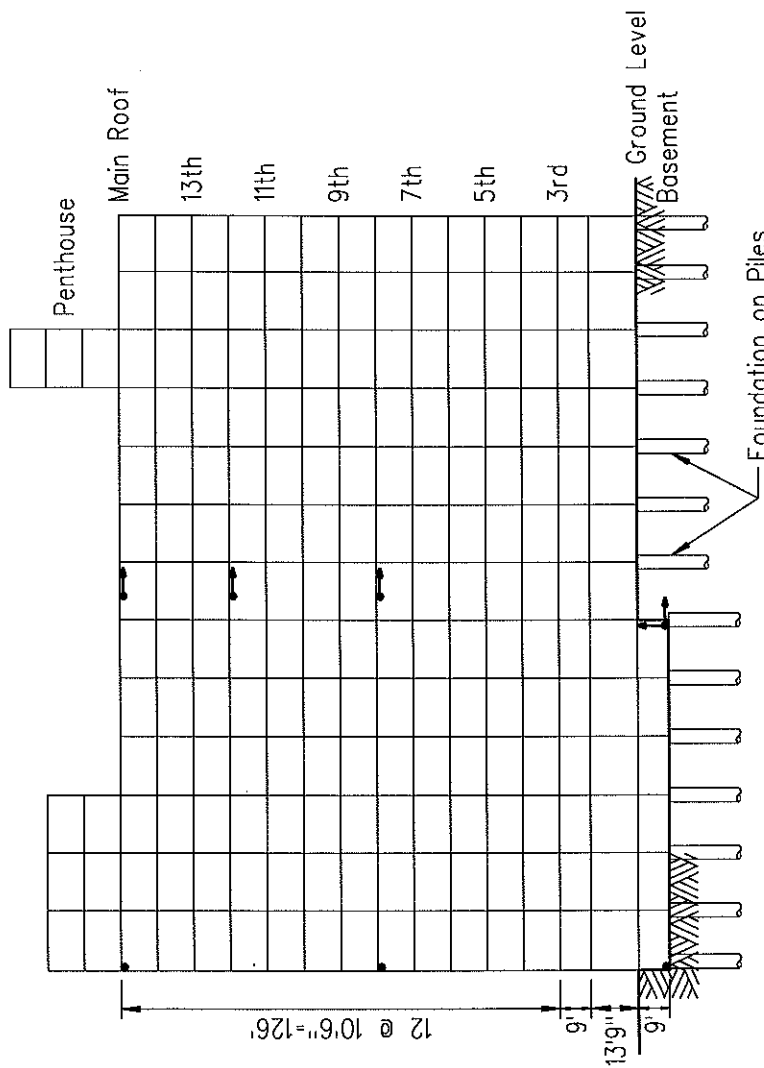
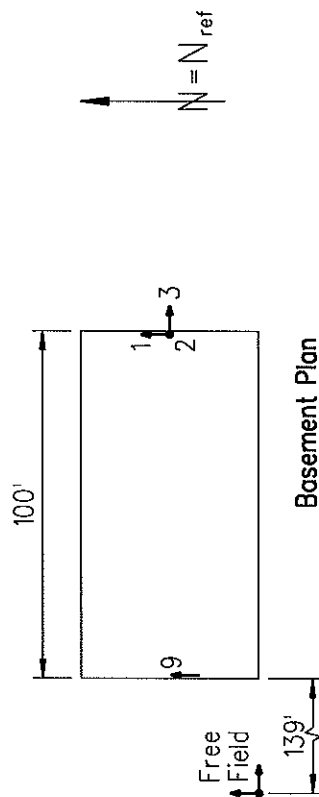
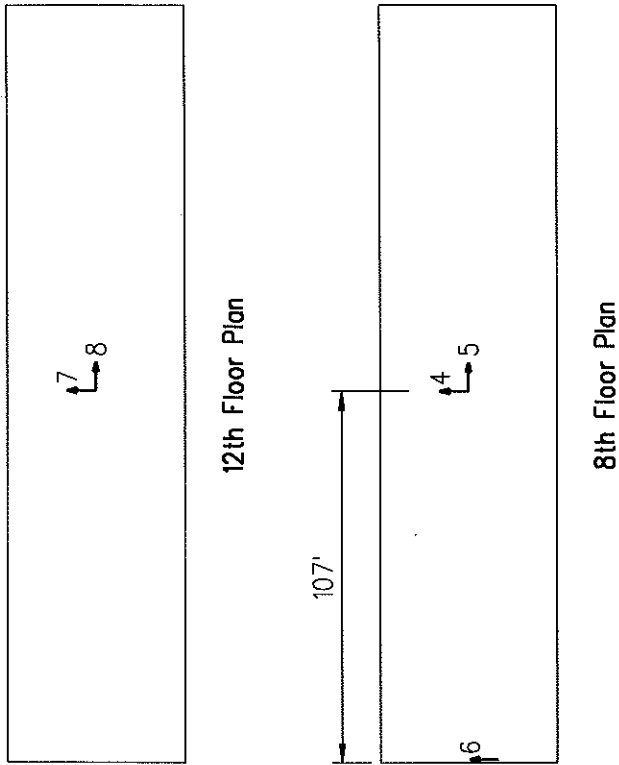
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
WT	Sys. ID	0.97	2.0	9.1	--	--	--
	Pre.-Vel.	1.02	0.1	6.7	--	--	--
	Pre.-Bie.	1.01	0.1	6.9	--	--	--
NR	Sys. ID	1.03	3.1	16.5	1.08	1.2	8.1
	Pre.-Vel.	1.02	0.1	13.8	1.05	1.2	8.9
	Pre.-Bie.	1.01	0.2	14.3	1.10	1.8	8.5

The period lengthening was reasonably well-predicted for both directions. Foundation damping was underpredicted for the transverse direction, but well-predicted for the longitudinal direction. Due to the low embedment ratio of $e/r = 9/59 = 0.15$, differences between the "modified Veletsos" and "modified Bielak" predictions are small.

Data Sources

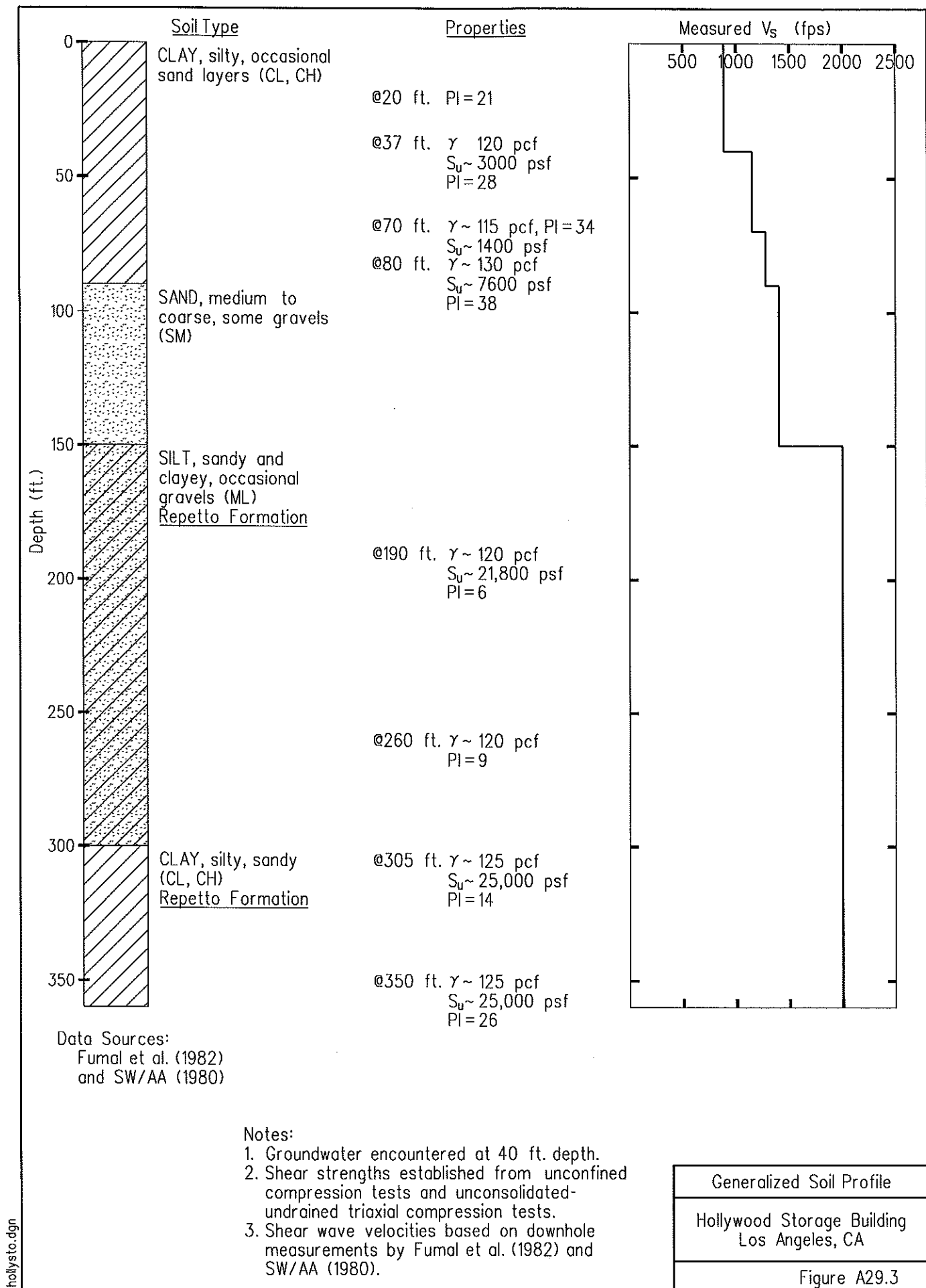
- Carder, D.S. (1964). "Earthquake Investigations in the Western United States, 1931-1964," *Publication No. 41-2*, Coast and Geodetic Survey, U.S. Department of Commerce, p. 165 and pp. 197-201.
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Sensor Locations
Hollywood Storage Building Los Angeles, CA
Figure A29.2

Note: Data provided by CSMIP
hollysto.dgn



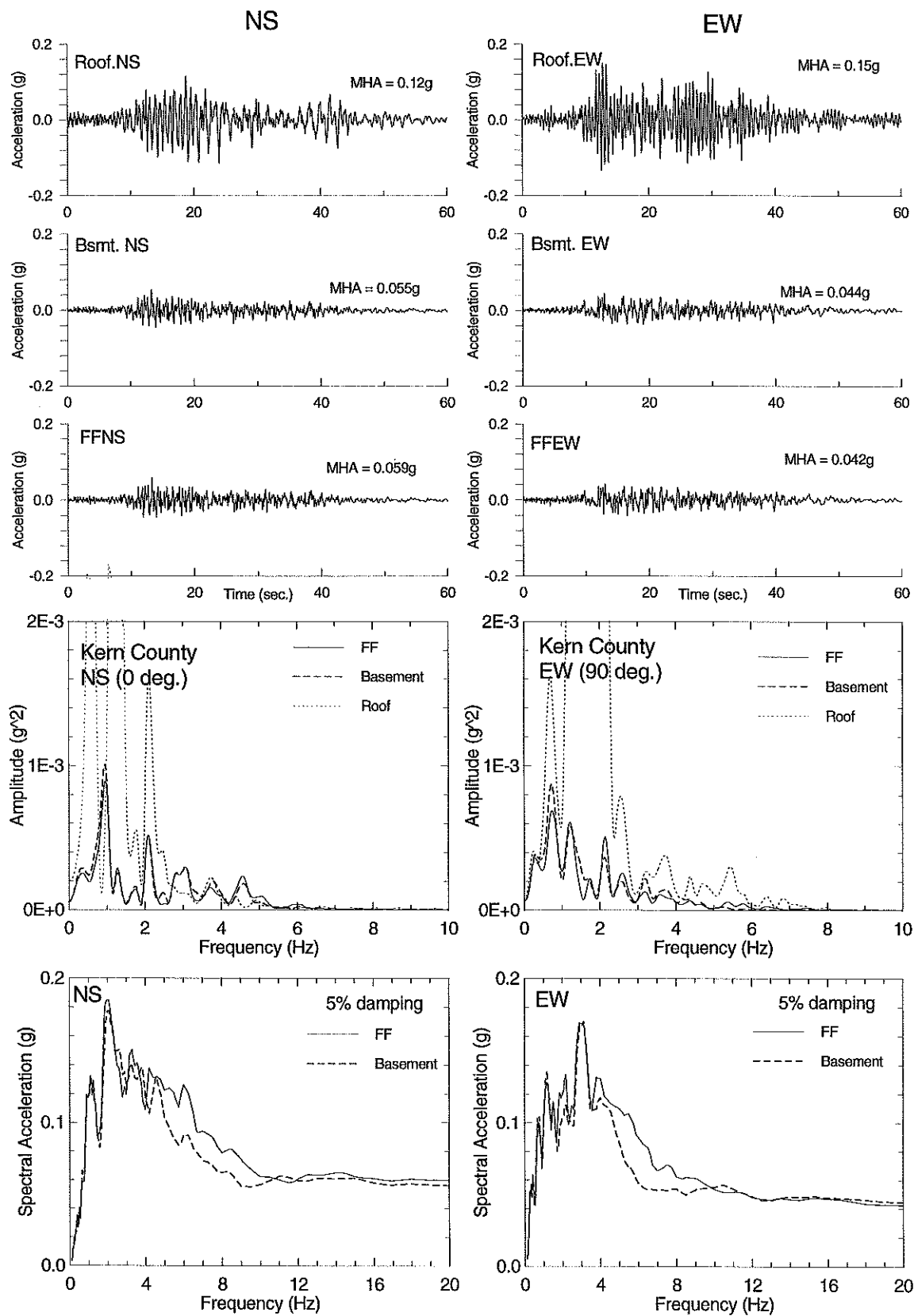


Fig. A29.KN 1 - 3: LA HOLLYWOOD STORAGE BUILDING
 (1) Horizontal Acceleration Time Histories,
 (2) Power Spectral Density of Selected Horizontal Accelerations, and
 (3) Acceleration Response Spectra of Horizontal Basement and Free-Field Accelerations
 1952 Kern County Earthquake

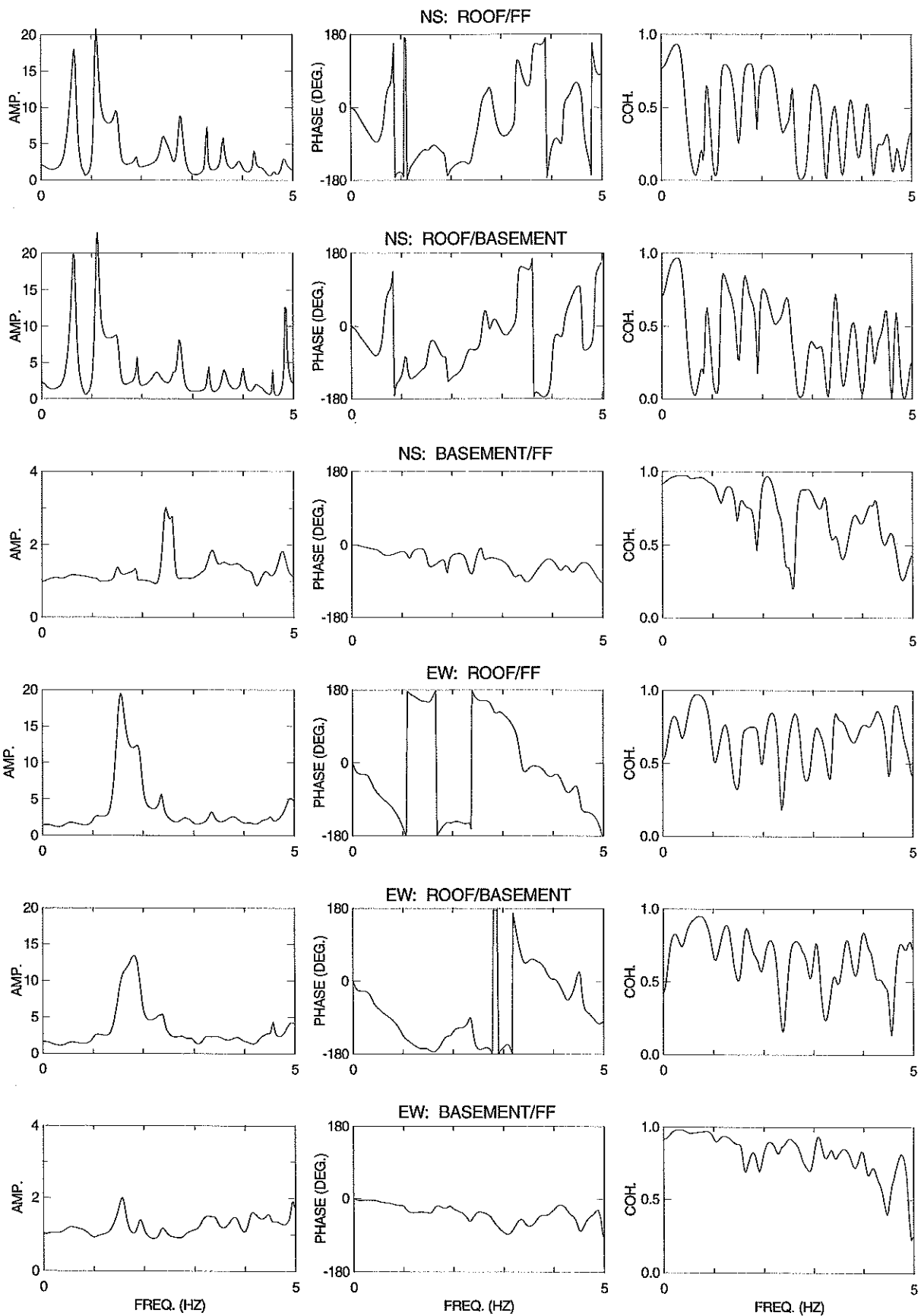


Fig.A29.KN4: LA HOLLYWOOD STORAGE BUILDING
Transmissibility and Coherency Functions
1952 Kern County Earthquake
A29-10

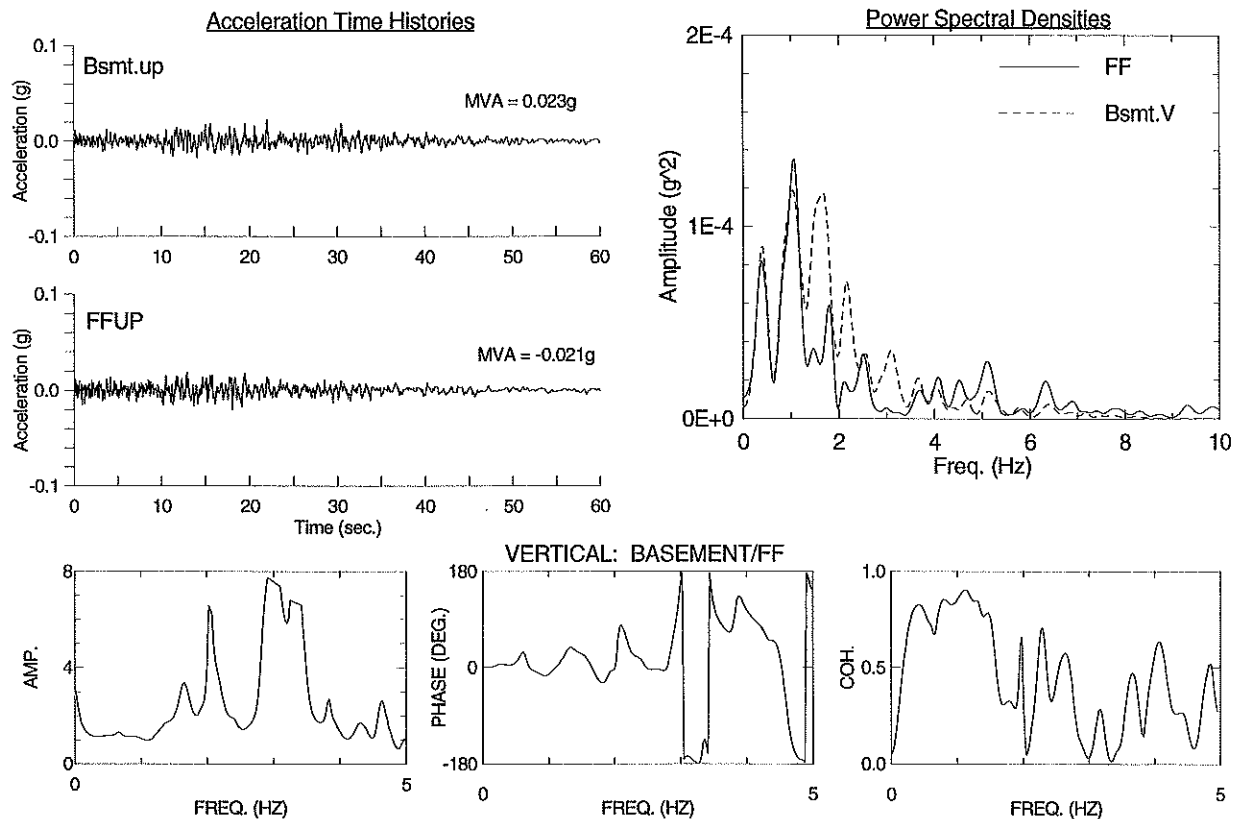


Fig. A29.KN5: LA HOLLYWOOD STORAGE BUILDING
Analysis of Vertical Motions
1952 Kern County Earthquake

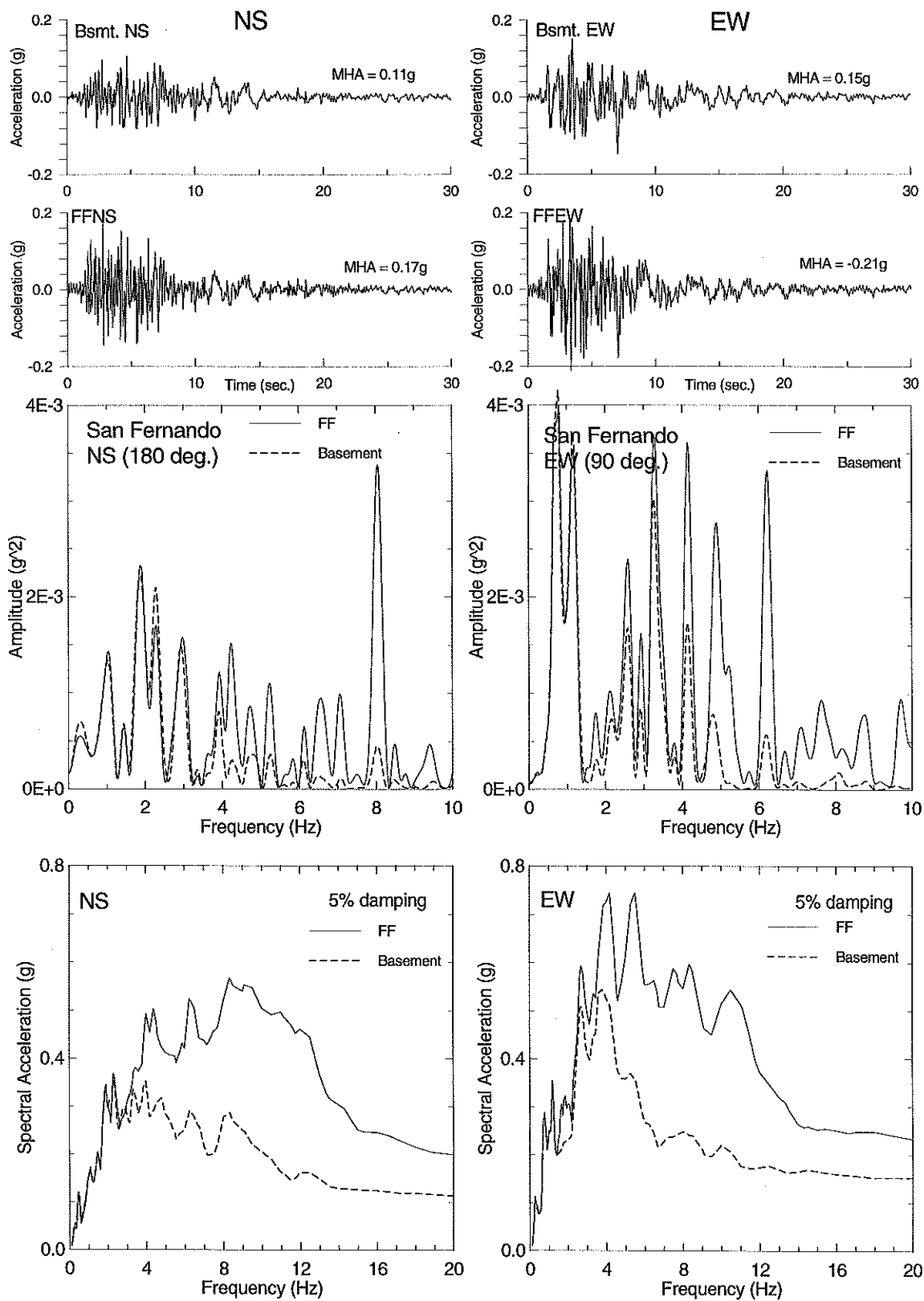


Fig. A29.SF 1 - 3: LA HOLLYWOOD STORAGE BUILDING
 (1) Horizontal Acceleration Time Histories,
 (2) Power Spectral Density of Selected Horizontal Accelerations, and
 (3) Acceleration Response Spectra of Horizontal Basement and Free-Field Accelerations
 1971 San Fernando Earthquake

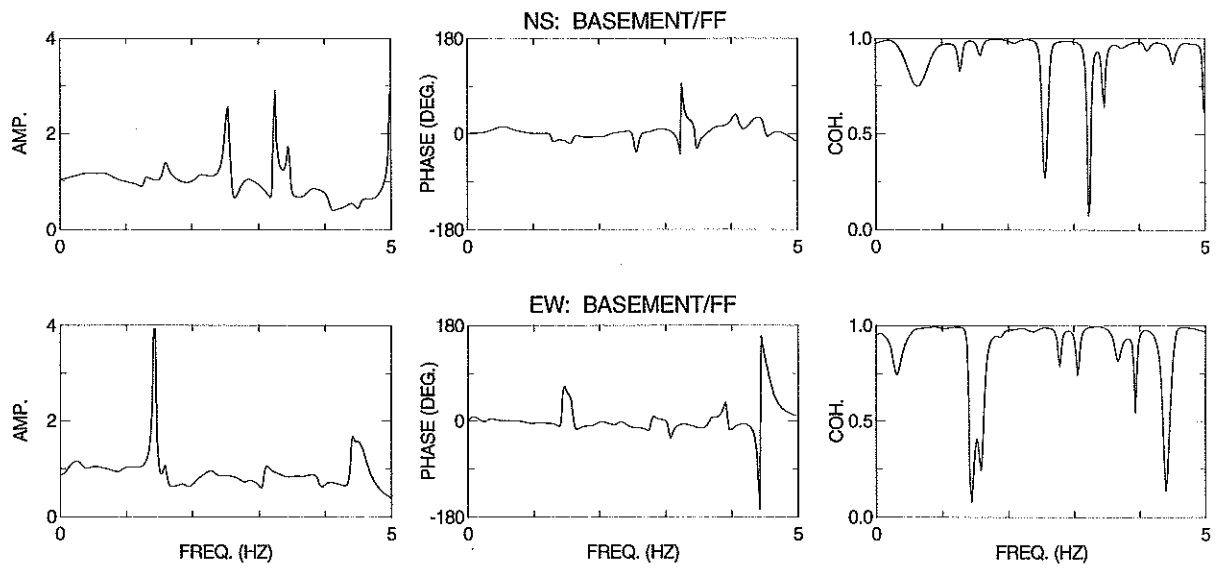


Fig. A29.SF4: LA HOLLYWOOD STORAGE BUILDING
Transmissibility and Coherency Functions
1971 San Fernando Earthquake

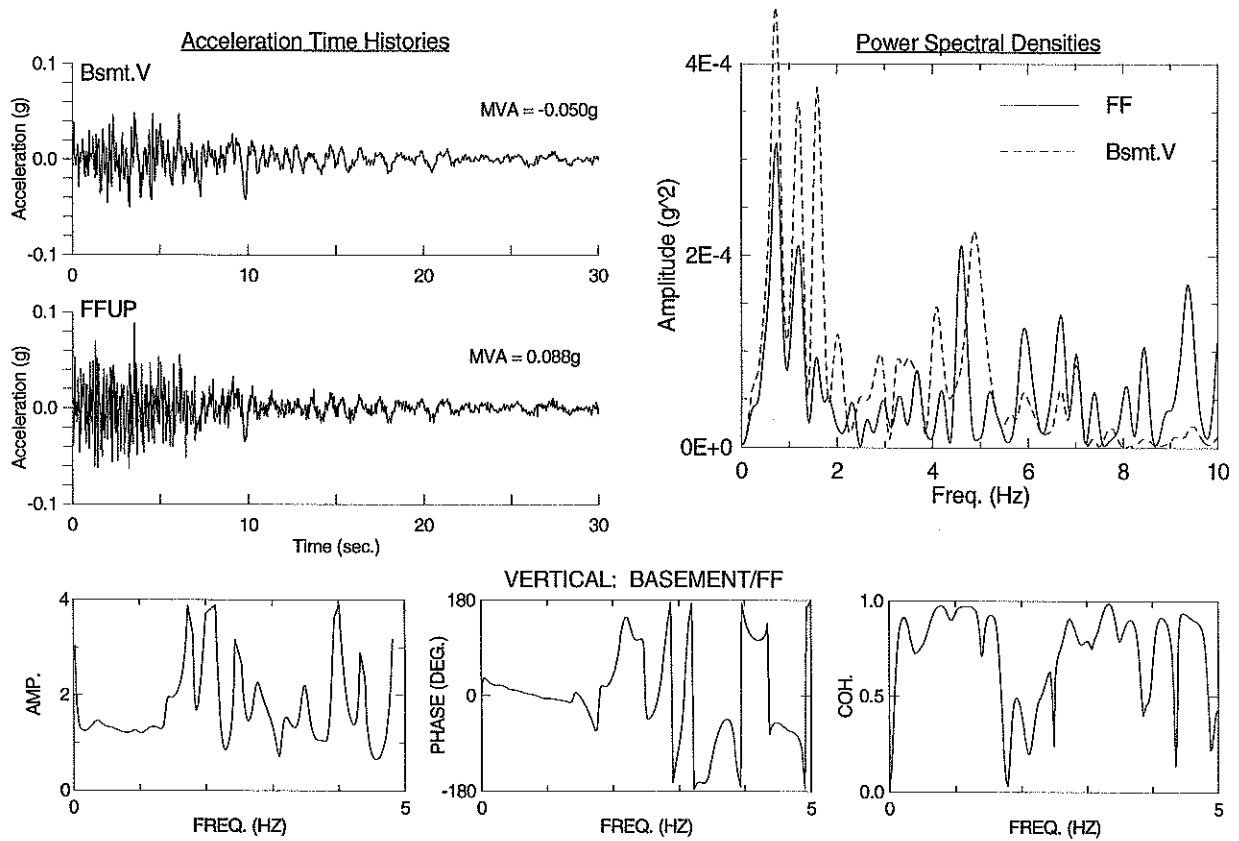


Fig. A29.SF5: LA HOLLYWOOD STORAGE BUILDING
Analysis of Vertical Motions
1971 San Fernando Earthquake

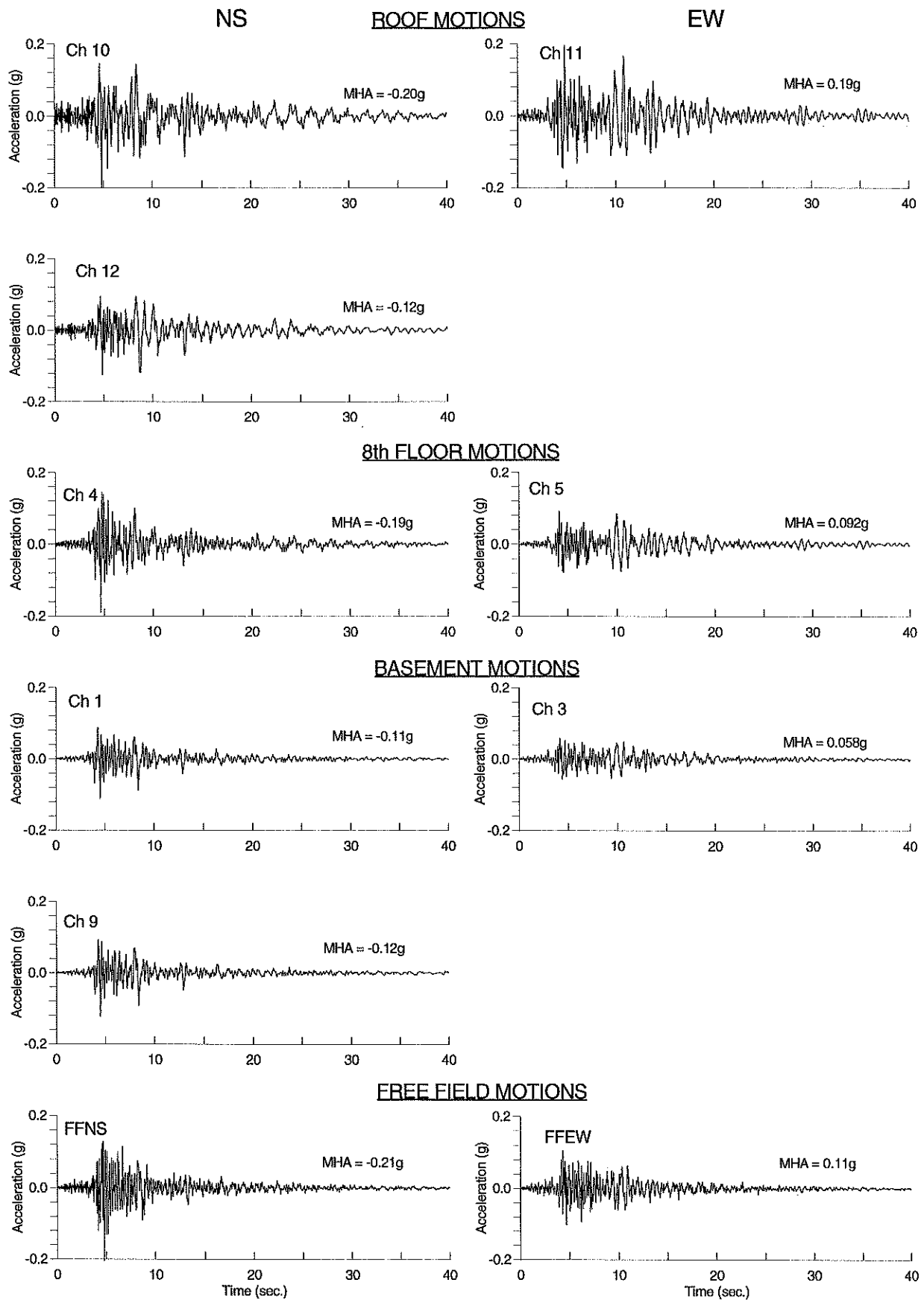


Fig. A29.WT1: LA HOLLYWOOD STORAGE BUILDING
Horizontal Acceleration Time Histories
1987 Whittier Earthquake
A29-14

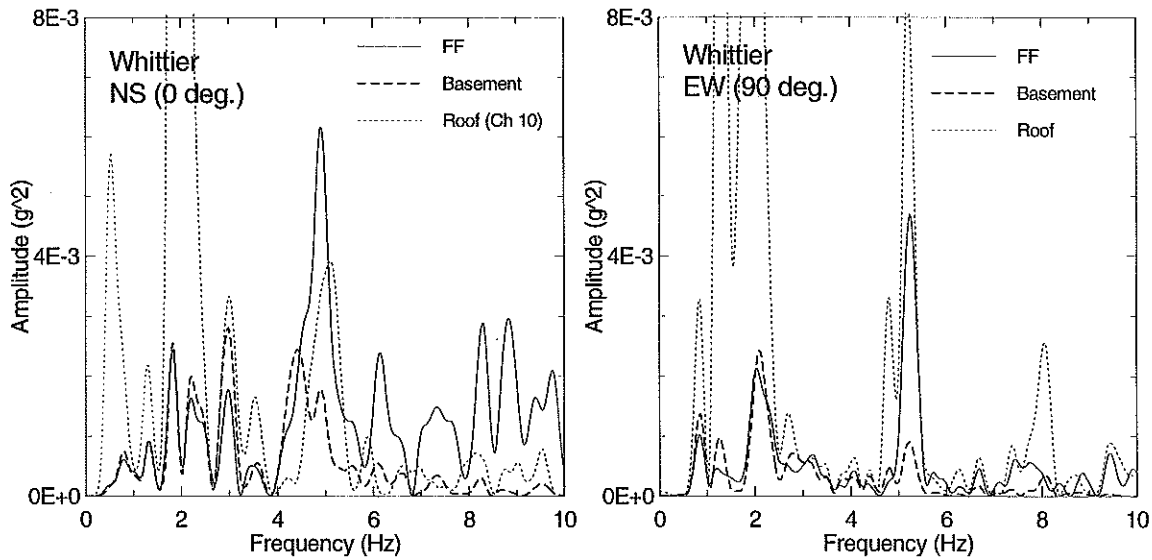


Fig. A29.WT2: LA HOLLYWOOD STORAGE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

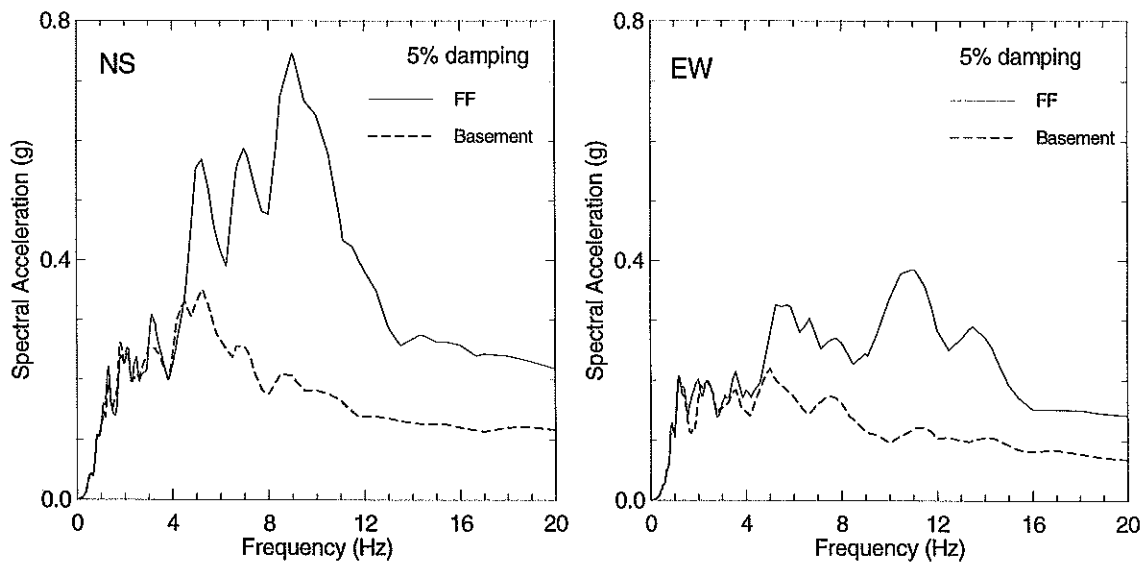


Fig. A29.WT3: LA HOLLYWOOD STORAGE BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1987 Whittier Earthquake

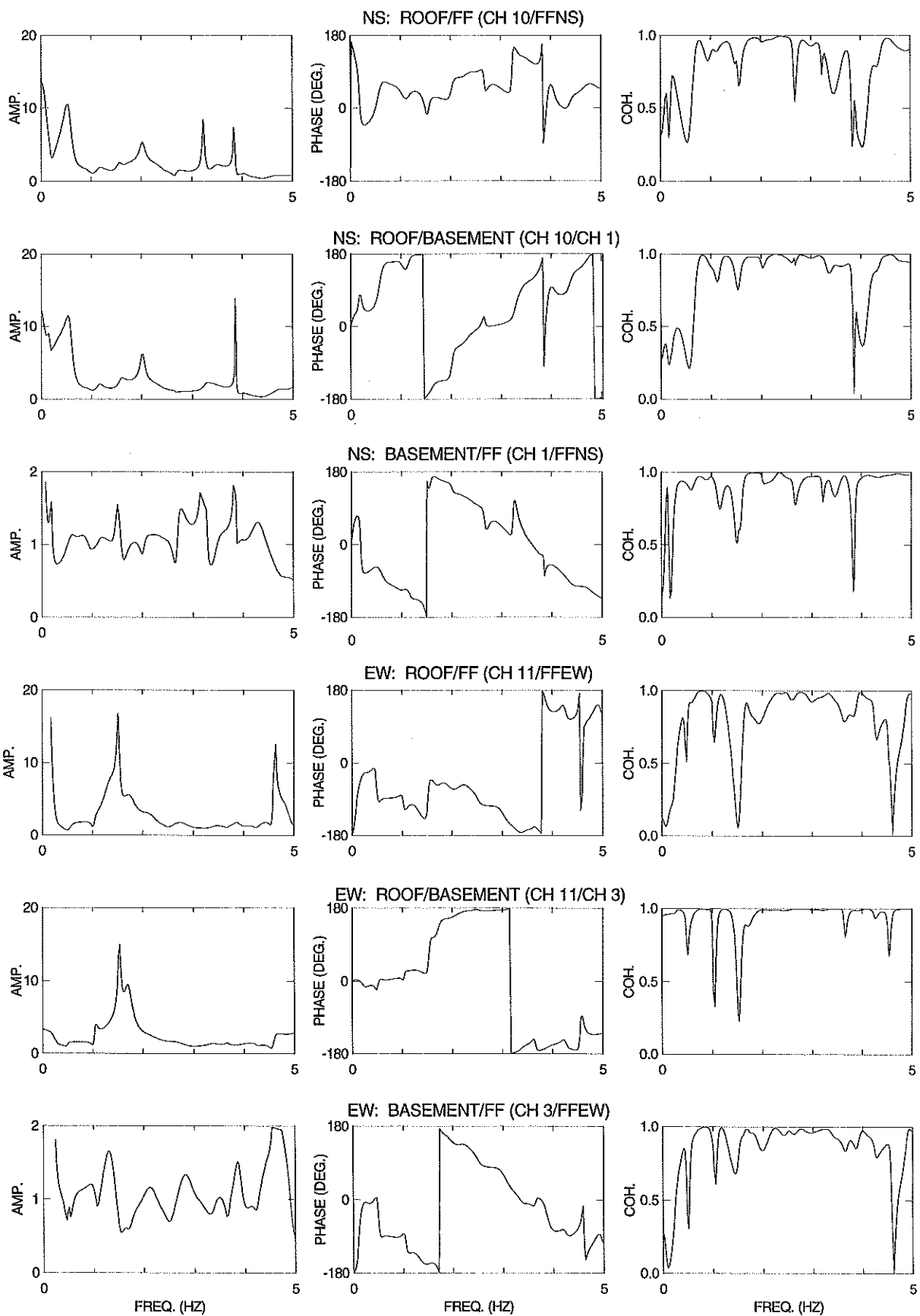


Fig. A29.WT4: LA HOLLYWOOD STORAGE BUILDING
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A29-16

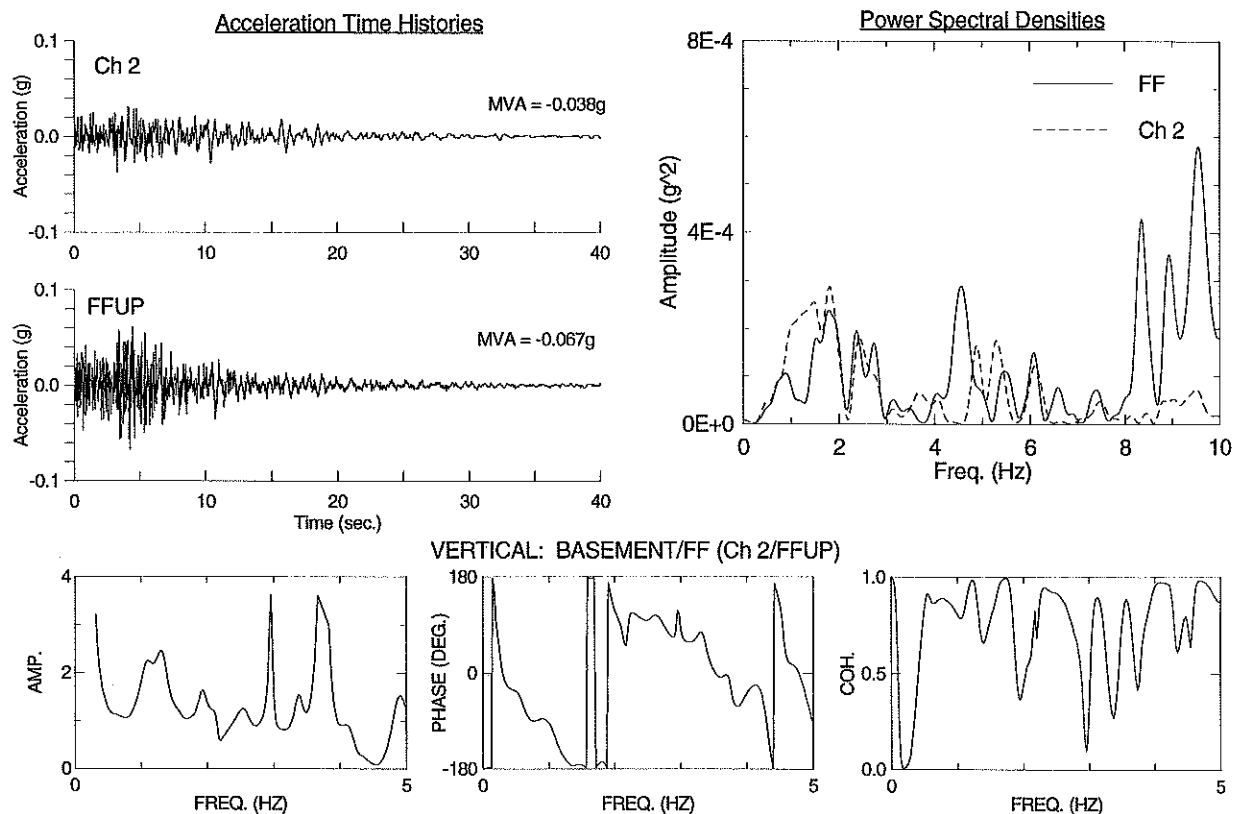


Fig. A29.WT5: LA HOLLYWOOD STORAGE BUILDING
Analysis of Vertical Motions
1987 Whittier Earthquake

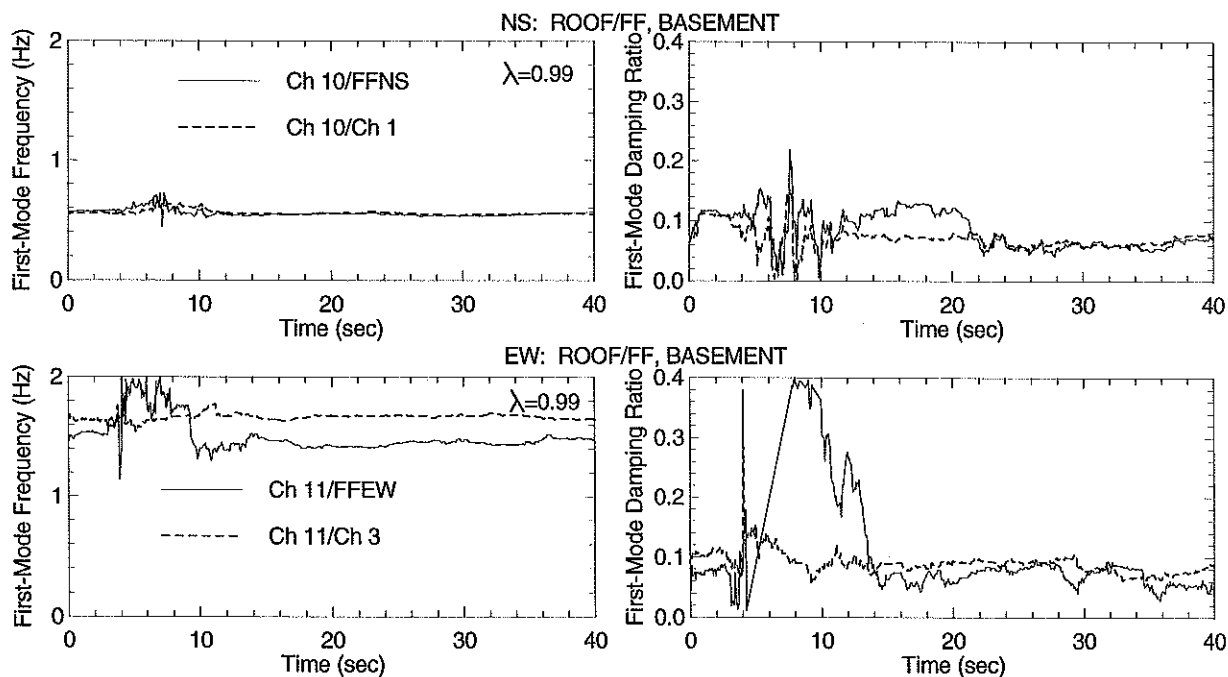


Fig. A29.WT6: LA HOLLYWOOD STORAGE BUILDING
Time Variation of First-Mode Parameters
1987 Whittier Earthquake

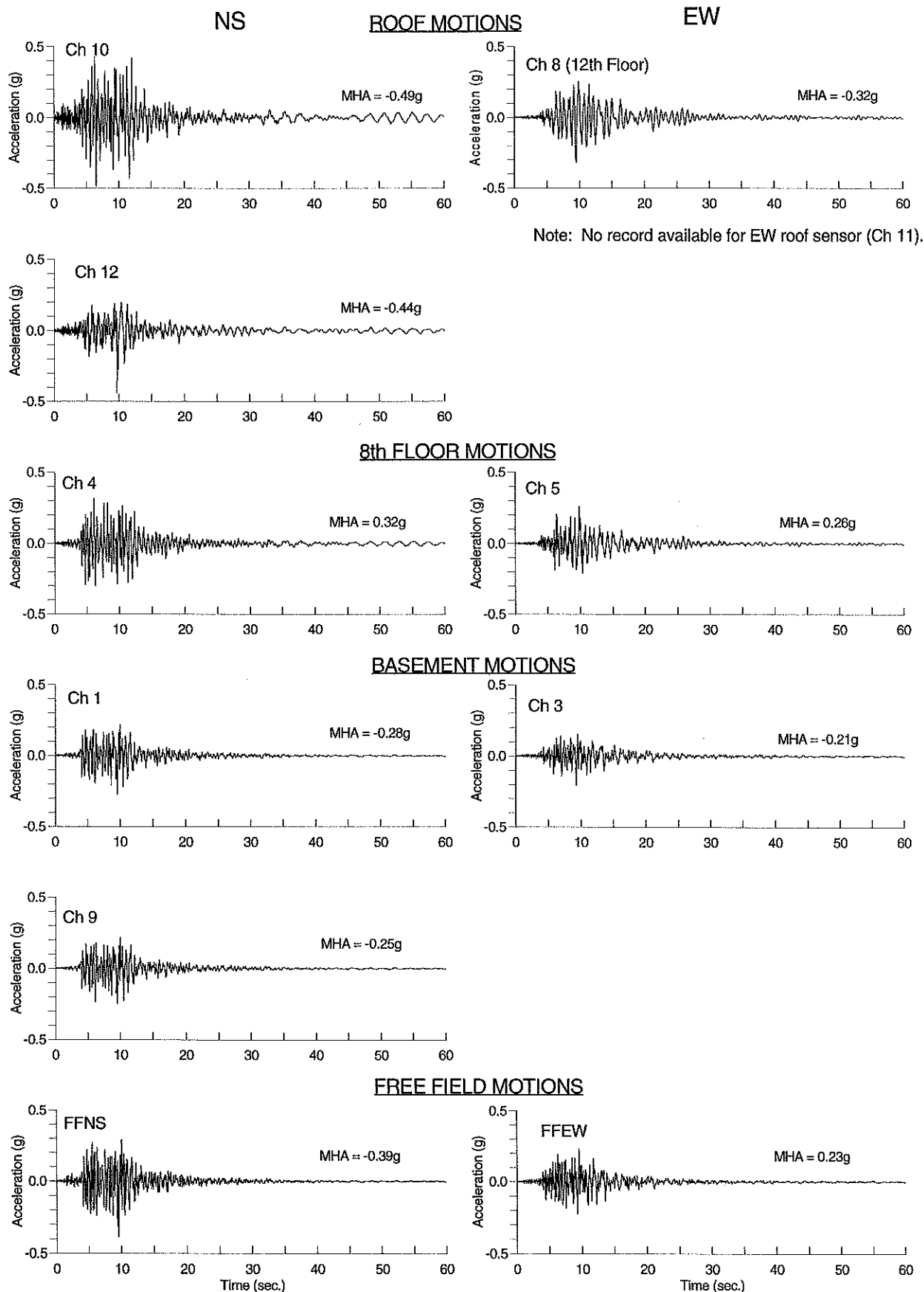


Fig. A29.NR1: LA HOLLYWOOD STORAGE BUILDING
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A29-18

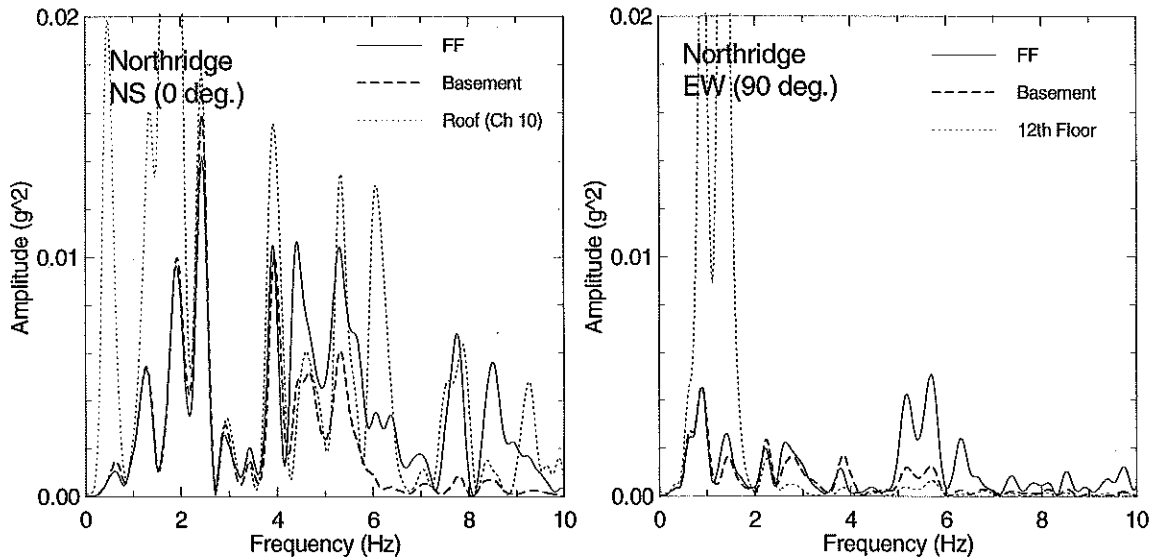


Fig. A29.NR2: LA HOLLYWOOD STORAGE BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

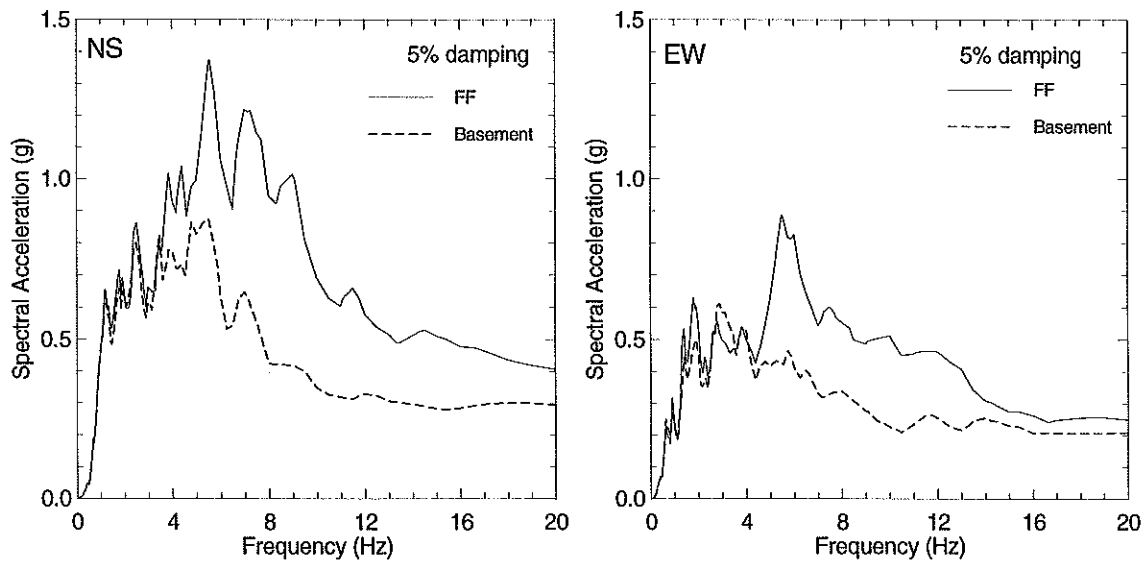


Fig. A29.NR3: LA HOLLYWOOD STORAGE BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

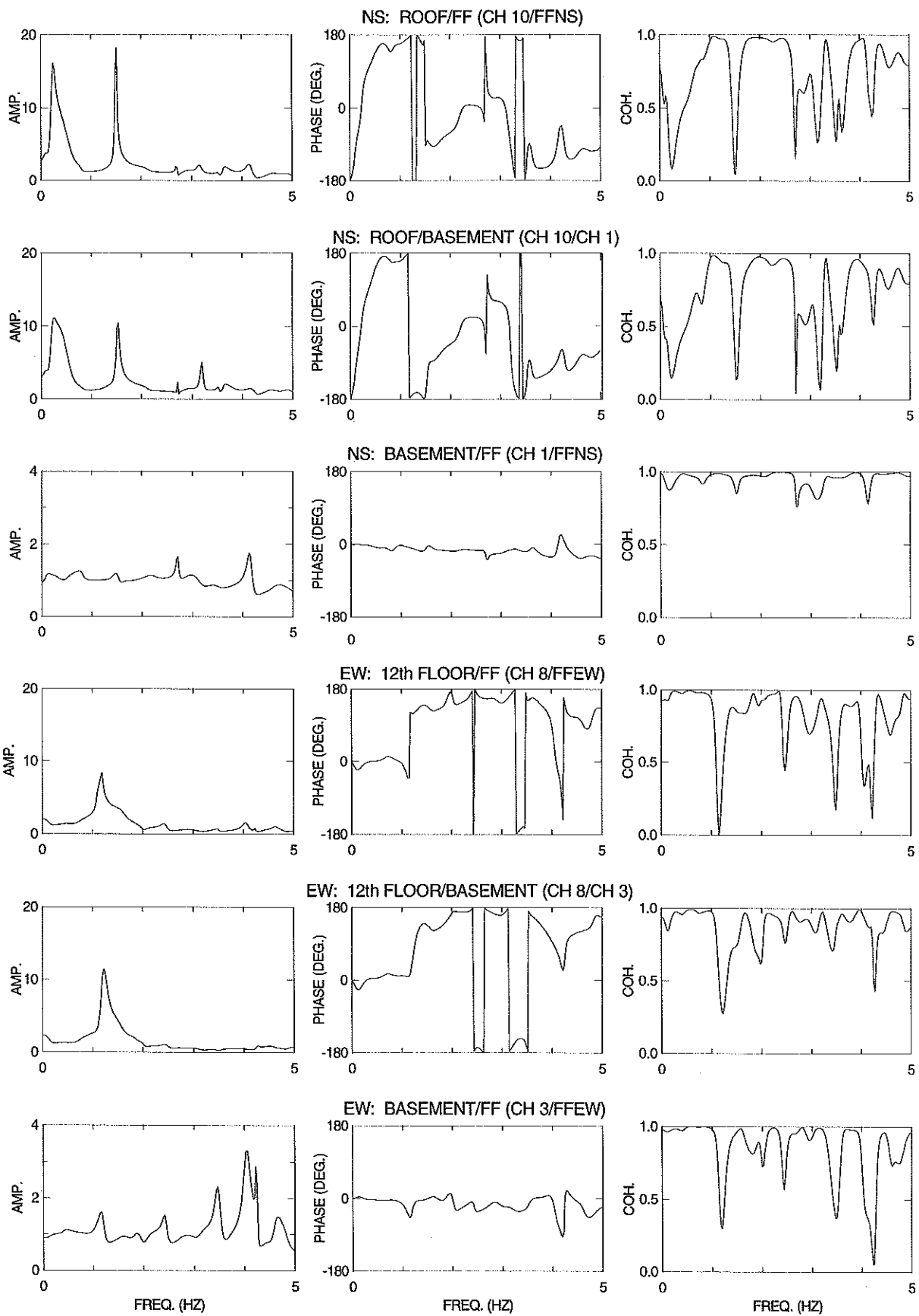


Fig. A29.NR4: LA HOLLYWOOD STORAGE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A29-20

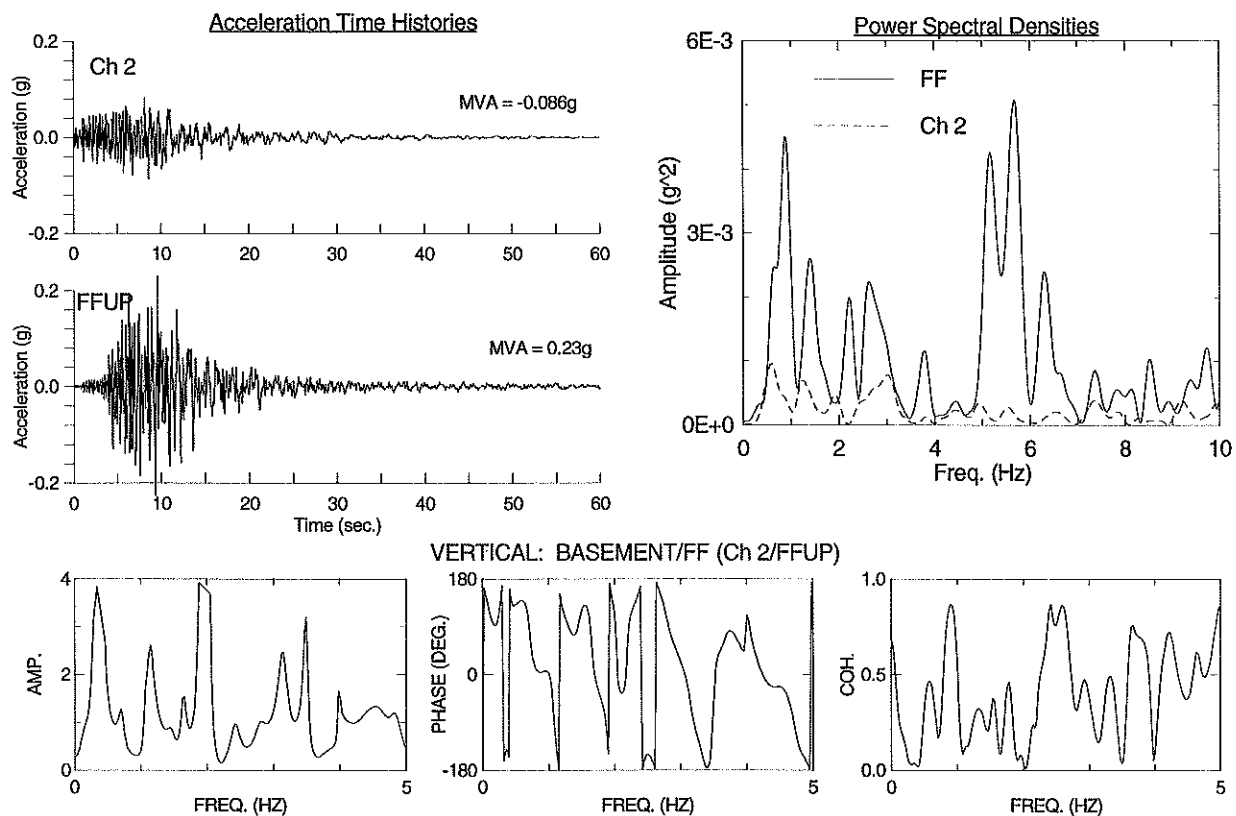


Fig. A29.NR5: LA HOLLYWOOD STORAGE BUILDING
Analysis of Vertical Motions
1994 Northridge Earthquake

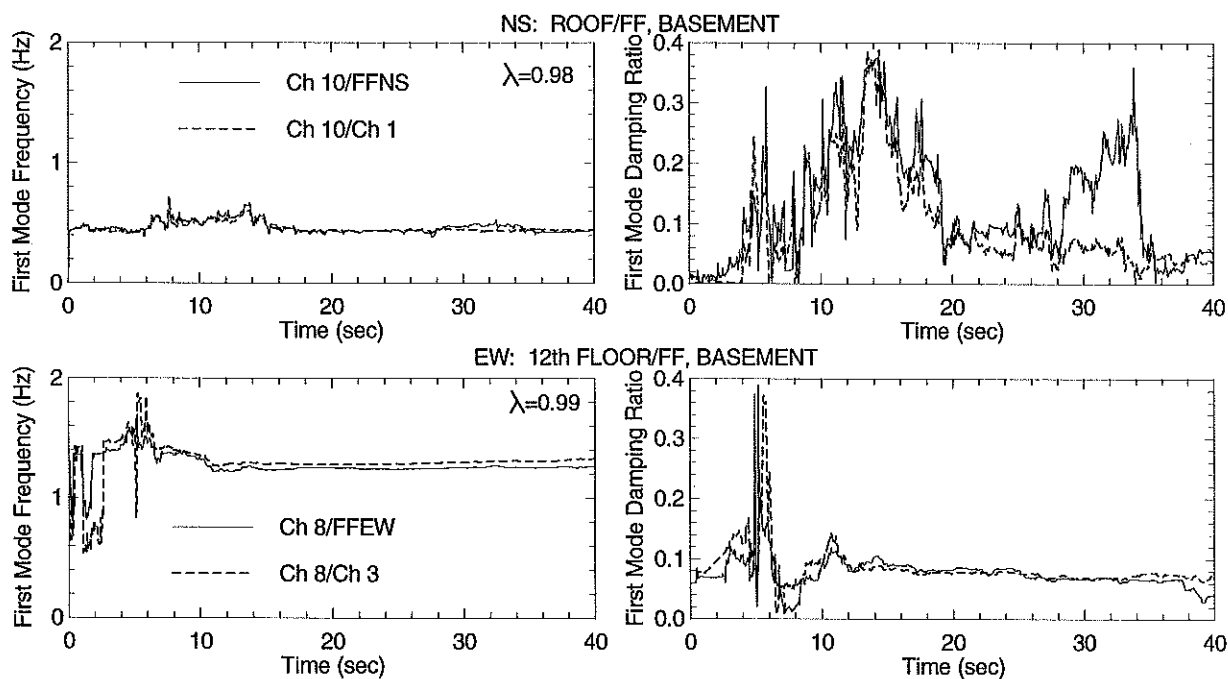


Fig. A29.NR6: LA HOLLYWOOD STORAGE BUILDING
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A30

Site Name: LA Wadsworth VA Hospital
Sensors: USGS 5082

Lat: 34.053°
Long.: 118.452°

Floor Levels: 8+ penthouse
Bsmt. Levels: 2
Structure: Steel moment frames

Foundation: concrete piles
Site Geology: soil
Design Date: 1974

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier*FF No.	0.07	0.08	0.03	0.07	0.07	0.04	0.13	0.15	--
FF So.	0.07	0.09	0.04						
1994 Northridge No.	0.25	0.25	0.15	0.17	0.27	0.091	0.42	0.36	--
FF So.	0.37	0.30	0.14						

* Not processed -- = no instrument

Preprocessing

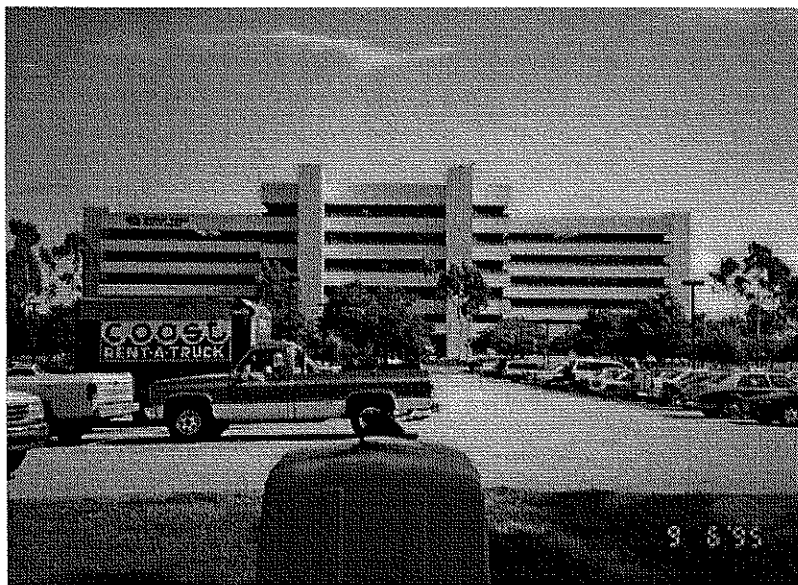
Northridge Earthquake (NR)

Preliminary: digitized at $f_{Nyq} = 100$ Hz; no processing, FF/str. data not synchronized

Final: filtered (this study), $f = 0.2$ -25 Hz; decimated by 4 to $f_{Nyq}=25$ Hz, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A30.NR0).

Figures

A30.1-A30.3: Site Plan, Sensor Locations, Generalized Soil Column
A30.NR0-6 NR Analysis Results



View of structure with FF No. in foreground

DISCUSSION

Site Description

The building is 6-stories tall above the ground surface and has a double-level partial basement. The structure has a square base (451 by 451 feet) at the first floor and a cross shaped tower for the upper 5 floors. No significant structures are located immediately adjacent to the building. The lateral load resisting system consists of four steel braced towers in each wing of the cross and a steel frame core. The foundation consists of 50-foot long reinforced concrete piles beneath the columns. A 4-inch thick slab is present across the base of the structure. As shown in Fig. A30.2, accelerographs are present on the basement and roof levels of the structure. The north and south free-field instruments are located in parking lots, about 470 feet northwest and 1320 feet southeast of the building, respectively.

The site is located on level terrain within the Los Angeles Basin. Available subsurface data for the area consists of several 100-foot borings at the site by L.T. Evans (1973), a 450-foot boring 2500 feet south of the site by Duke and Leeds (1962), and several 100 to 150-foot borings within about 3000 feet of the site by LCA (1981 and 1982). As shown in Fig. A30.3, soil conditions consist of: 10 feet of medium-stiff silts and clays, 50 feet of relatively dense sands and silts, about 240 feet of stiff clays and silts, and 100 feet of very dense sands and gravels which extend to shale bedrock at a depth of about 400 feet. Measurements by L.T. Evans indicate the depth-to-groundwater is about 50 to 60 feet. The shear wave velocities shown in Fig. A30.3 were estimated using downhole measurements from nearby sites having similar geologic conditions (Duke and Leeds, 1962; LCA, 1981 and 1982).

Instrument Selection and Free-Field Conditions

Accelerographs near the central core were used in the analyses, except for the roof NS sensor, for which only a wing instrument was available (Table A30.1). Acceleration time histories recorded by these sensors are shown in Fig. A30.NR1.

Table A30.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 7	Ch 8	Ch 9	Ch 6	Ch 3

There are no structures or significant slope breaks near the free-field instruments that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A30.NR4) and the spectra for horizontal motions (Fig. A30.NR2-3) generally indicate de-amplification of basement motions relative to the FF.

Coherencies obtained from FF south data are relatively low due to incoherence between the basement and FF motions.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Fig. A30.NR5. Amplification near the first-mode frequency for lateral response is not evident from the north B/FF pair, suggesting that base rocking was unlikely to have significantly affected the structural response. The low frequency amplification obtained from the south B/FF pair is less reliable due to incoherence effects.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A30.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A30.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A30.NR6).

Table A30.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF No/So.-R)			
		tr (EW) estimated		L (NS) estimated		tr (EW) d=9, j=10		L (NS) d=7, j=12		tr (EW) d=13/--, j=10/--		L (NS) d=10/12, j=12/12	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.10	9.1	na	na	1.09	9.0	1.05	5.1	1.06/--	9.7/--	1.09/1.11	4.8/4.7
	2					3.29	10.9	3.29	7.9	3.19/--	6.7/--	3.15/3.22	10.8/11.2
	3					5.61	7.3	4.82	7.5	5.40/--	10.2/--	4.93/4.83	11.1/12.2

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in both directions.
- For the transverse direction, the minor differences between the fixed- and pseudo flexible-base parameters are consistent with the lack of evidence for a rocking response in the vertical B/FF transmissibility function. For the longitudinal direction, fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$.
- Data from the two FF instruments gave similar transmissibility functions at low frequencies (i.e. $f < 1$ Hz), as well as similar modal parameters from CEM analyses.
- A reliable parametric model of FF So.-R response in the EW direction could not be developed.

The RPEM results (Fig. A30.NR6) indicate no significant time-dependent changes in the first-mode structural parameters. Data from the FF north instrument were used in these analyses.

(c) Comparison of Ground Floor and Free-Field Motions

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A30.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 189 feet using the recorded free-field motions and the profile in Fig. A30.3. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A30.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1130$ fps	$h = 78$ ft	emb. = 17 ft	$\gamma = 0.15$
NR	$\nu = 0.45$		tr:	$r_1 = 189$ ft	$r_2 = 199$ ft
	$\beta^* = 6.3\%$	$V_s^* = 980$ fps	L:	$r_1 = 189$ ft	$r_2 = 199$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A30.3 with the fixed-base modal parameters in Table A30.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A30.4.

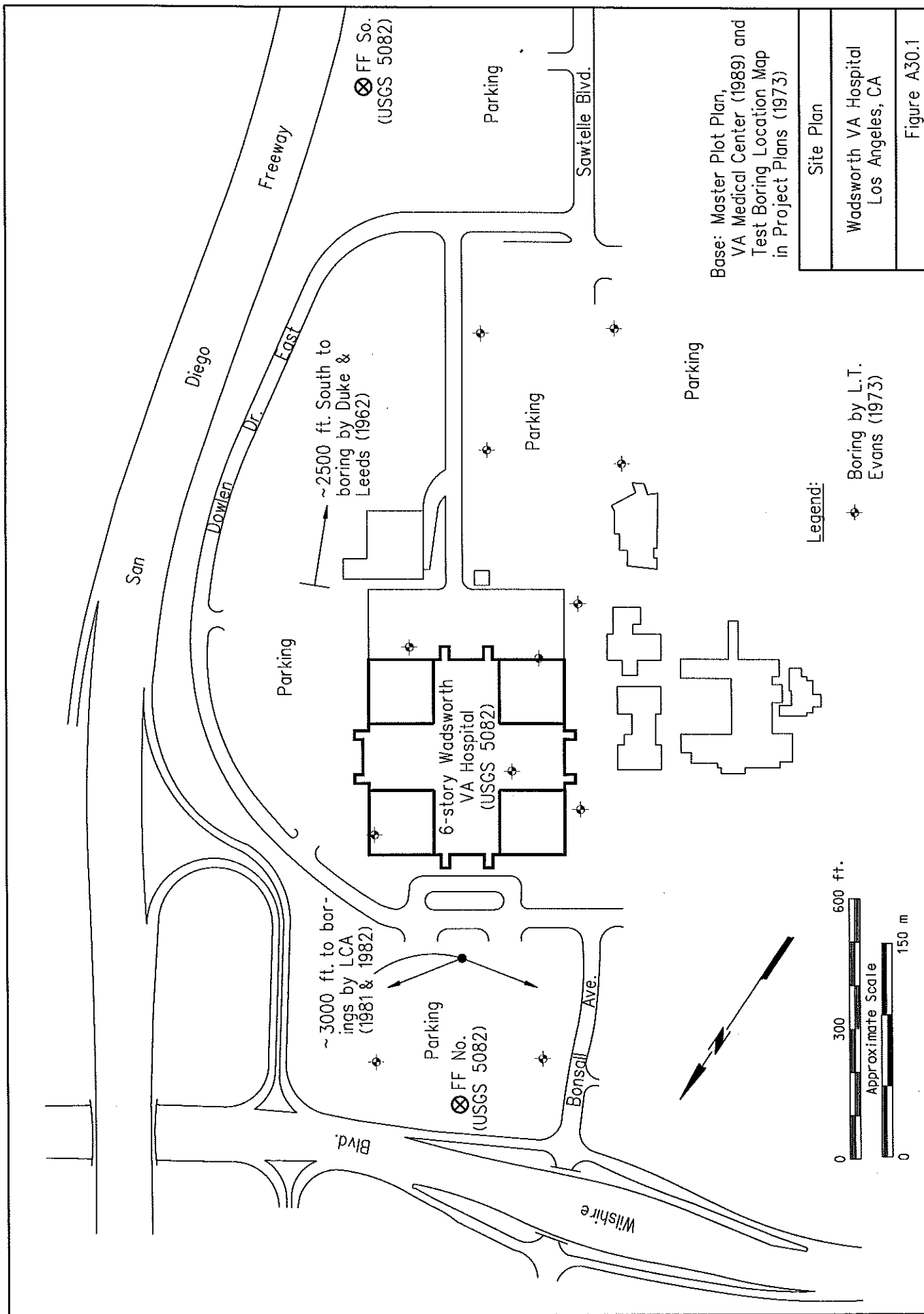
Table A30.4: Empirical and predicted inertial interaction effects

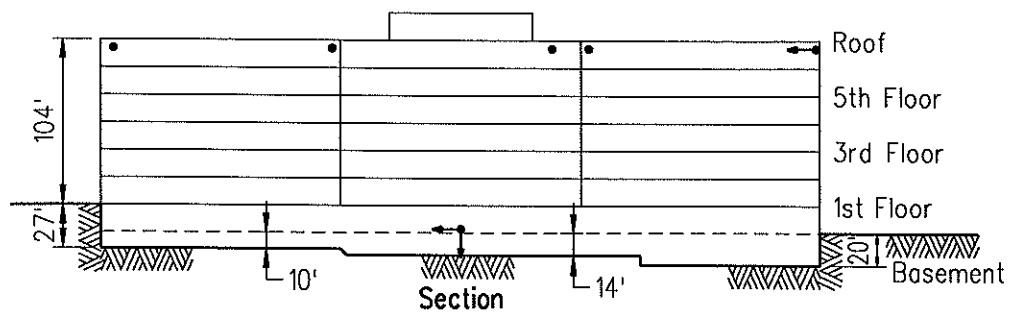
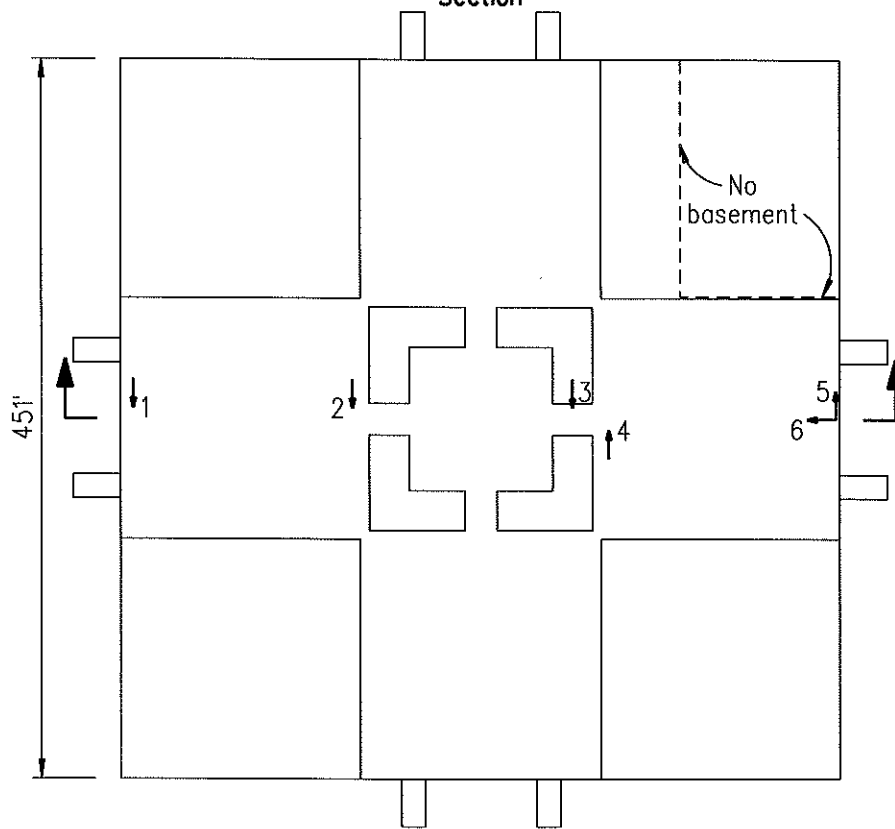
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.03	1.5	9.7	--	--	--
	Pre.-Vel.	1.04	2.0	10.1	--	--	--
	Pre.-Bie.	1.04	3.6	11.7	--	--	--

The period lengthening and foundation damping are satisfactorily predicted by the “modified Veletsos” formulation, and $\tilde{\zeta}_0$ was slightly overpredicted by the “modified Bielak” formulation.

Data Sources

- LeRoy Crandall and Assoc., LCA (1981). “Report of Geotechnical Investigation, Proposed One Westwood Office Building, Wilshire Blvd. and Veteran Ave., Westwood District, Los Angeles, CA,” *Proj. No. ADE-81162*.
- LeRoy Crandall and Assoc., LCA (1982). “Report of Geotechnical Investigation, Proposed Bren Westwood Gateway, Sepulveda and Santa Monica Blvds., Los Angeles, CA,” *Proj. No. ADE-82078*.
- Duke, C.M. and Leeds, D.J. (1962). “Site characteristics of Southern California strong motion earthquake stations,” *Rpt. No. 62-55*, Department of Engrg., Univ. of Calif., Los Angeles.
- Evans, L.T. (1973). Test borings in project plans.

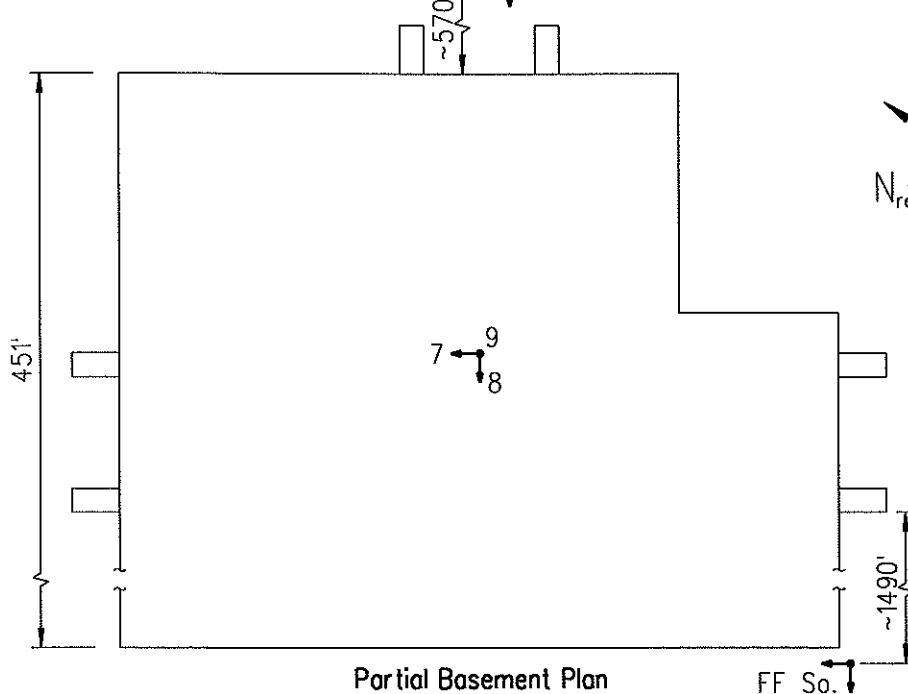
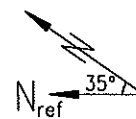




STRUCTURE:

Rectangular base, cross-shaped towers (5 stories)
Core: steel frame
Wings: steel braced towers
Foundation: R C piles

Roof Plan
FF No.



Note: Data provided by USGS

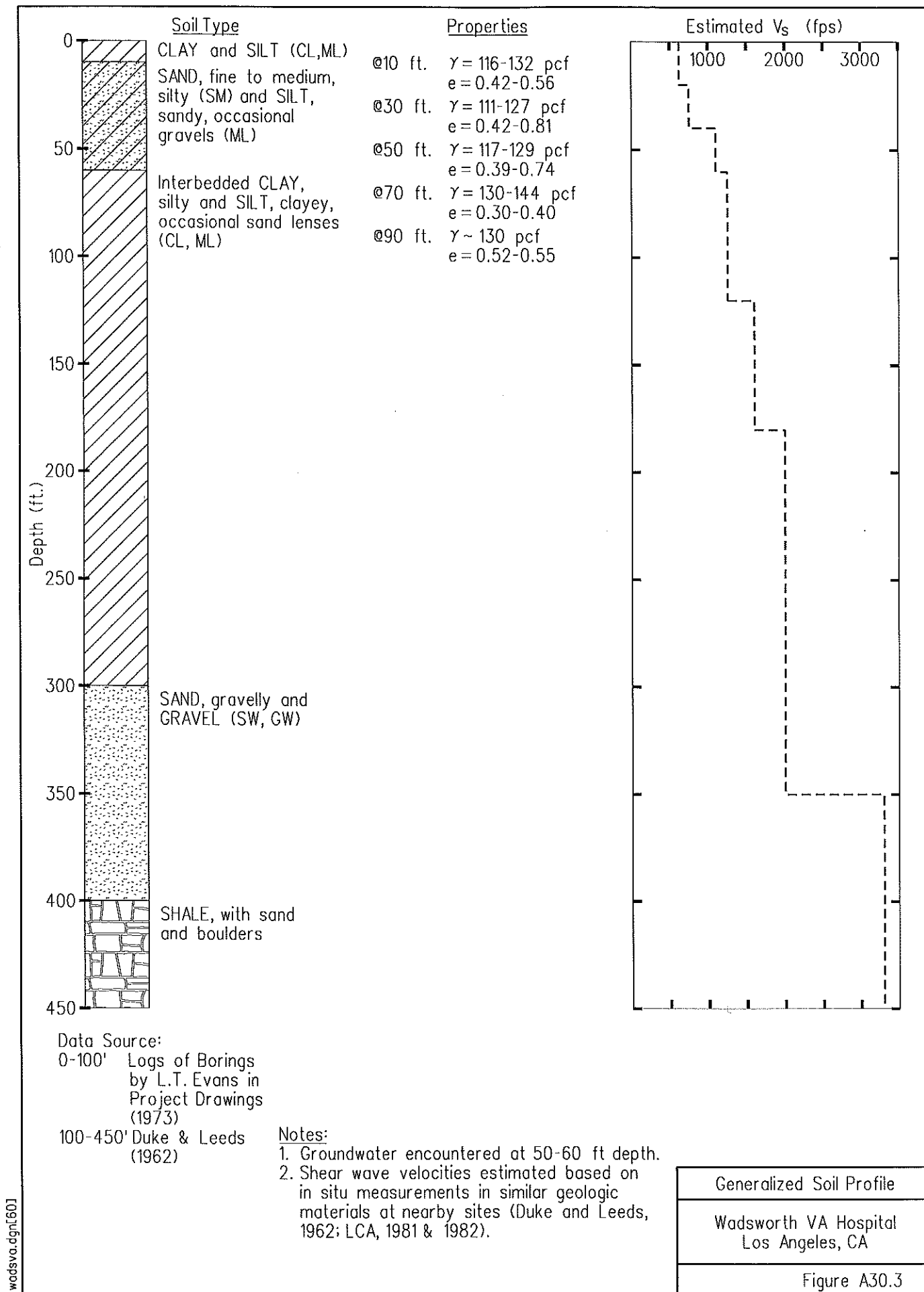
Sensor Locations

Wadsworth VA Hospital
Los Angeles, CA

Figure A30.2

Partial Basement Plan

FF So.



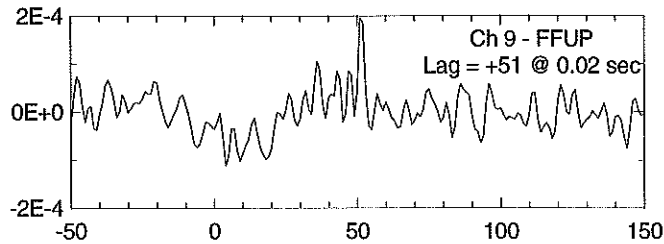


Fig. A30.NR0: Cross correlation function between vertical basement and free-field (north) accelerations

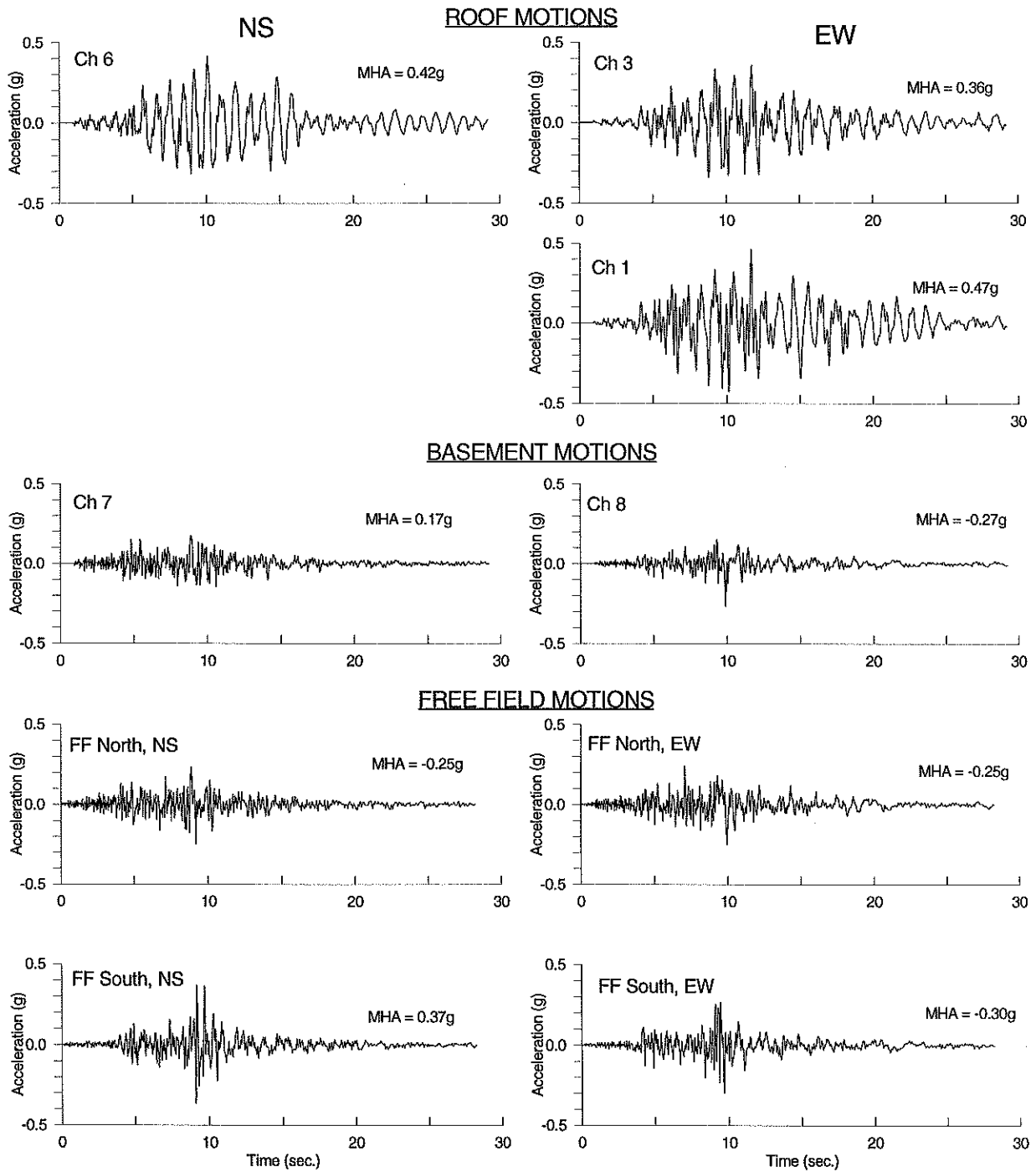


Fig. A30.NR1: LA WADSWORTH VA HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A30-8

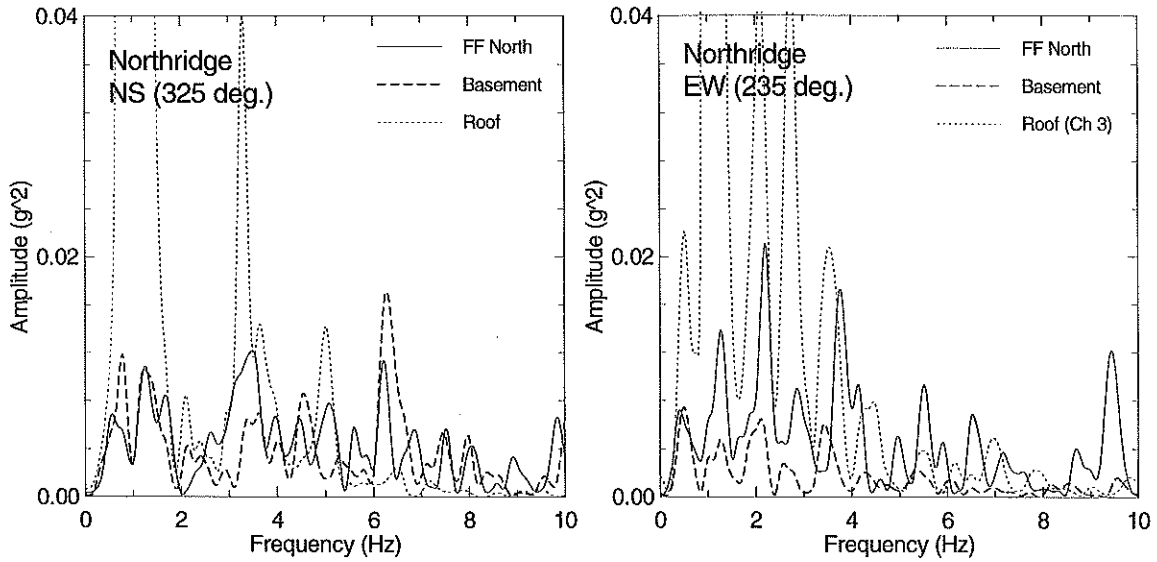


Fig. A30.NR2: LA WADSWORTH VA HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

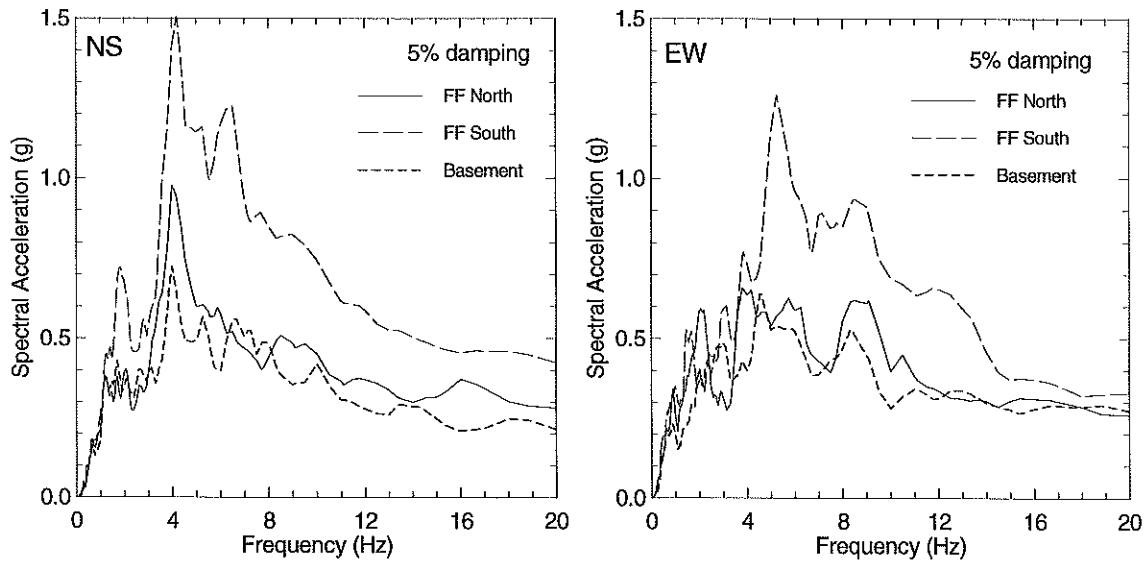


Fig. A30.NR3: LA WADSWORTH VA HOSPITAL
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

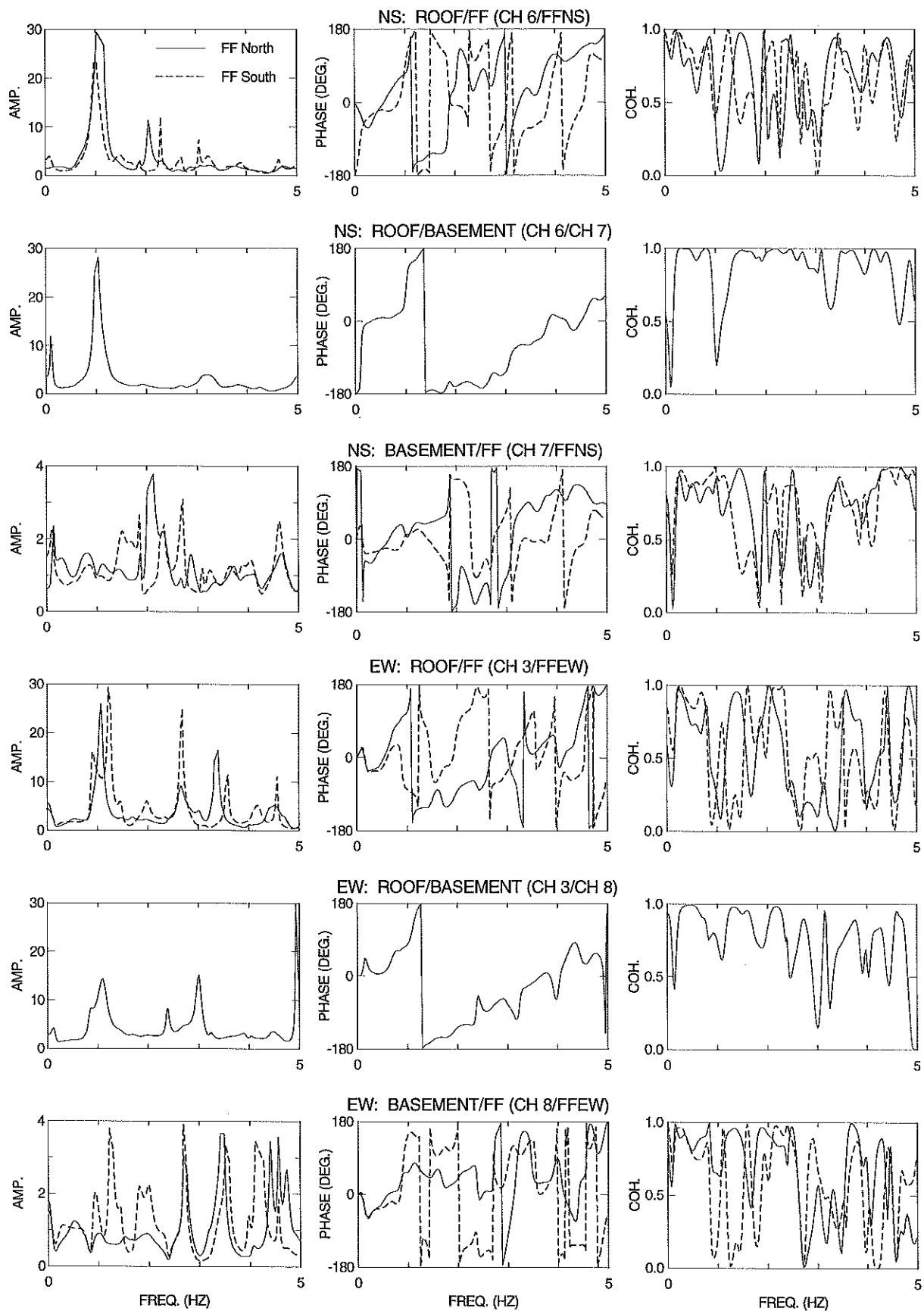


Fig. A30.NR4: LA WADSWORTH VA HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A30-10

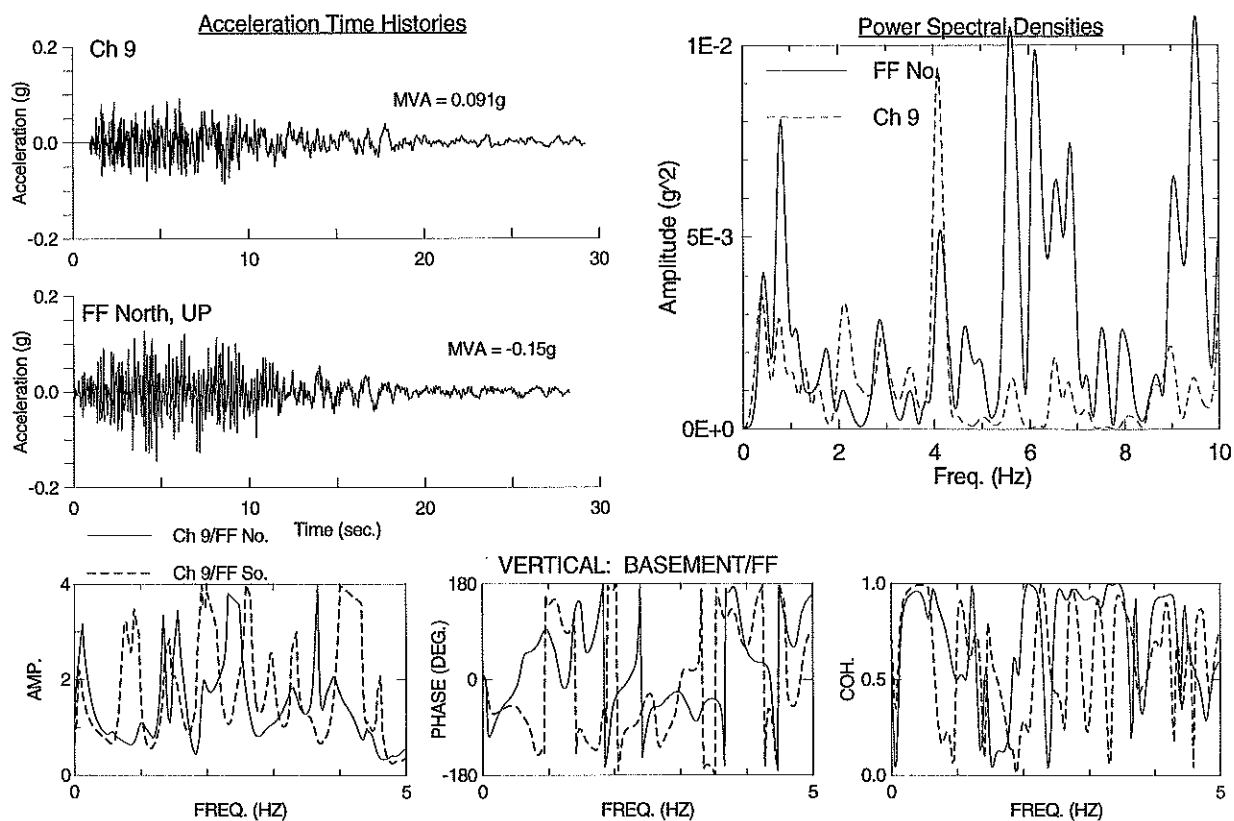


Fig. A30.NR5: LA WADSWORTH VA HOSPITAL
Analysis of Vertical Motions
1994 Northridge Earthquake

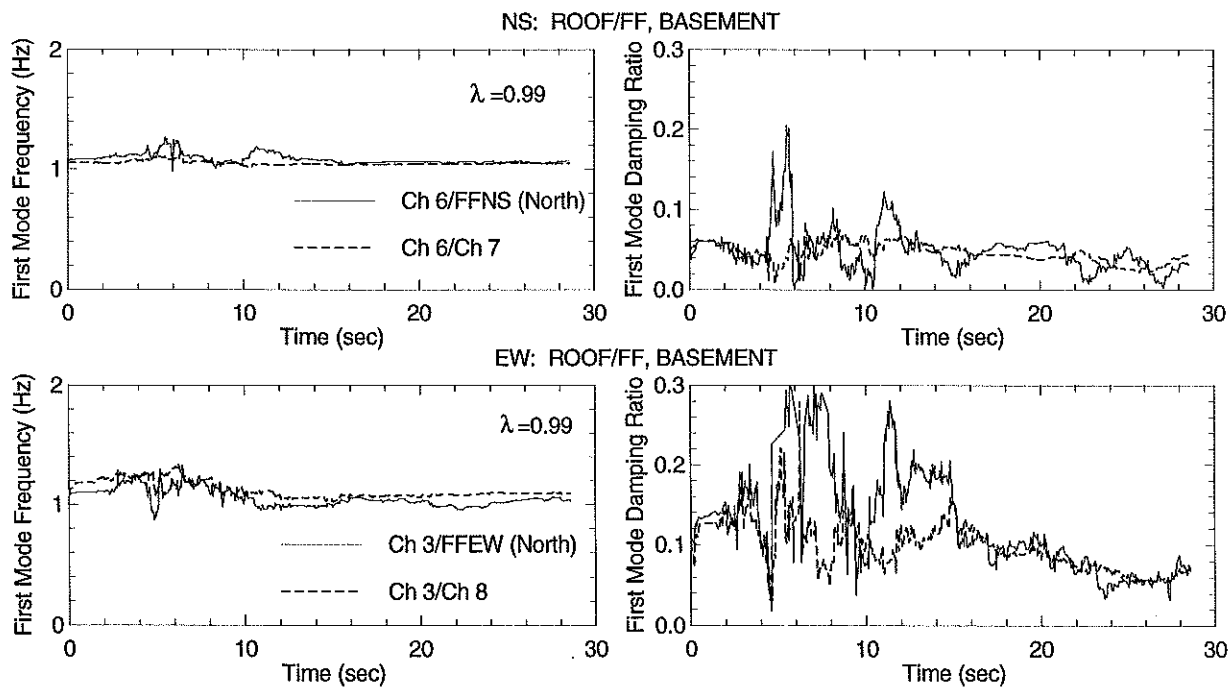


Fig. A30.NR6: LA WADSWORTH VA HOSPITAL
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A31

Site Name: Newport Beach Hoag Hospital
Sensors: CSMIP 13589/13610

Lat: 33.624°
Long.: 117.929°

Floor Levels: 11
Bsmt. Levels: 0
Structure: Perimeter shear walls

Foundation: footings
Site Geology: soil/rock
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Service Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers	--	--	--	0.047	0.043	0.024	0.22	0.18	--
1994 Northridge	0.094	0.10	0.019	0.053	0.076	0.033	0.19	0.19	--

-- = no instrument

Preprocessing

Landers Earthquake (LD)

Preliminary: CSMIP; filtered, $f=0.12-46$ Hz; $f_{Nyq}=50$ Hz

Final: data decimated by 4 to $f_{Nyq}=12.5$ Hz

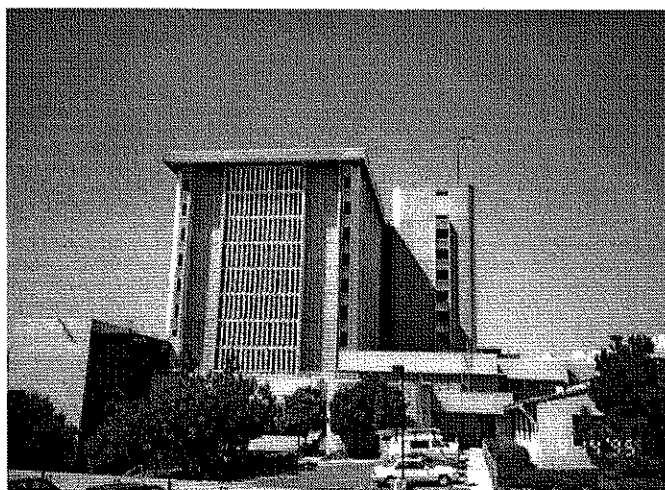
Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f=0.24-46$ Hz; $f_{Nyq}=50$ Hz; FF/str. data not synchronized; FF oriented 25° CW from str. reference south.

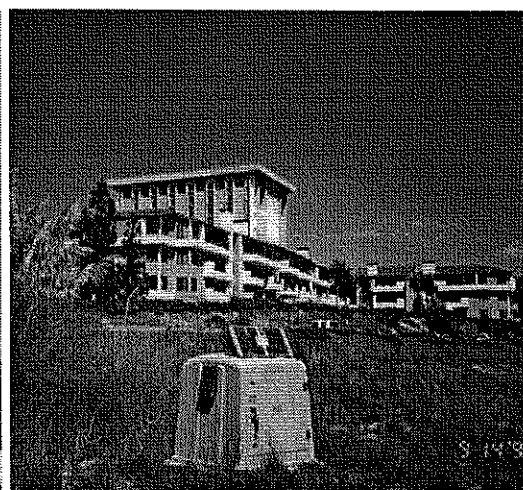
Final: FF data rotated 25° CCW; all data decimated by 4 to $f_{Nyq}=12.5$ Hz; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A31.NR0).

Figures

A31.1-A31.3: Site Plan, Sensor Locations, Generalized Soil Column
A1.LD1-3: LD Analysis Results
A1.NR0-6: NR Analysis Results



View of structure, looking north



View of structure, FF in foreground

DISCUSSION

Site Description

The structure is 11 stories tall above ground level with no basement, and is rectangular in plan view with dimensions 154 by 77 feet. Located adjacent to the structure on the northeast side is a 12-story elevator tower, and several large 2-story buildings are present along the entire north and east sides of the structure. The lateral load resisting system consists of perimeter concrete shear walls in both directions. A concrete frame in the building's interior was designed to only carry vertical loads. The foundation consists of shallow footings. As shown in Fig. A31.2, accelerographs are present at the service level, 3rd and 6th floors, and roof of the structure. The free-field instrument is located about 650 feet southwest of the structure in a landscaped area below a parking lot. This instrument was installed after the LD, but before the NR events.

The site is on the Newport Mesa near the Pacific Ocean. A south sloping coastal bluff up to about 50 feet high crosses the site below the hospital. The hospital structure is on relatively flat terrain, while the free-field instrument is on moderately sloping ground (1H:10V slopes) near the bluffs. The soil conditions shown in Fig. A31.3 are representative of the hospital building site above the bluffs, and consist of about 50 feet of sandy terrace deposits which were underlain by Monterey Formation siltstone and shale bedrock (LCA, 1971 and 1991). The free-field instrument is located near the geologic contact between the Terrace deposits and bedrock (Fig. A31.1) and is believed to be founded on rock (LCA, 1991). The shear wave velocities shown in Fig. A31.3 for the Terrace deposits were estimated using correlations with void ratio, whereas the rock velocities were assumed to be the same as those measured in Monterey Formation shale bedrock at the nearby Shellmaker Island site (Fumal et al., 1981).

Instrument Selection and Free-Field Conditions

Sensors near the south edge of the building were selected for the system identification analyses. The two vertical instruments located at the service level were used to evaluate rocking effects in the transverse (EW) direction. The specific sensors used are indicated in Table A31.1; acceleration time histories recorded by these sensors are shown in Fig. A31.**1.

Table A31.1: Sensors in structure used for analyses

Service Level (SL)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 4	Ch1,2	Ch 13	Ch 14

The following input was used in system identification analyses to evaluate fixed-base structural parameters for the EW direction,

$$\text{input} = \text{Ch4} + (\text{Ch2} - \text{Ch1}) \cdot 147/77 \quad (1)$$

The free-field instrument is located in an open area with a mild slope, and is unlikely to be affected by the any structural vibrations or topographic amplification along the bluffs. Free-field ground surface motions at the building site during the NR event were estimated using the recorded rock motions and 1-D wave propagation analyses (i.e. SHAKE). The modulus degradation and damping curves for sand discussed at the beginning of this report (Section 2.4) were used in these analyses. Both the calculated "soil" motions and the recorded rock motions were used in the system identification analyses.

Analysis Results

(a) Comparison of Service Level and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

SL/FF transmissibility functions (Fig. A31.NR4) and the spectra of horizontal motions (Fig. A31.NR2-3) indicate minor amplification of SL motions relative to the FF near structure's first-mode response frequency in the transverse direction. SL motions were generally de-amplified for frequencies greater than about 1.5 Hz.

SL/FF transmissibility functions for vertical motions (Fig. A31.NR5) indicate amplification near the transverse (EW) first-mode frequency, suggesting that rocking may have contributed to the structural response. A direct examination of rocking contributions to roof motion in the transverse direction is made in Fig. A31.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements and have about 20% of the roof displacement amplitude, indicating that base rocking significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A23.LD2/NR4), (2) modal parameters from CEM analyses (Table A31.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A23.LD3/NR6).

Table A31.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (SL-R)				Flex.-Base (FF-R)			
		tr (EW) LD: d=2, j=7 NR: d=1, j=7		L (NS) na estimated		tr (EW) d=3, j=7 d=2, j=7		L (NS) d=2, j=7 d=2, j=7		tr (EW) estimated d=2, j=7		L (NS) na d=0, j=7	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	1.40	3.9	na	na	1.23	4.5	1.40	3.2	1.20	8.5-4.5	na	na
	2	2.07	6.5			2.03	6.5	2.26	20.9				
	3	4.09	3.4			4.04	4.6	4.43	8.2				
NR	1	1.36	6.7	1.47	3.6	1.19	5.0	1.36	3.3	1.16	3.1	1.30	4.1
	2	2.10	7.2			2.06	7.6	2.48	17.3	2.14	13.3	2.45	30.9
	3	4.13	3.8			4.02	5.6	4.36	14.0	3.77	4.4	3.62	16.5

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The modest differences between the transverse fixed- and pseudo flexible-base parameters is consistent with the rocking effect evident in the vertical SL/FF transmissibility functions.
- The value of $\tilde{\zeta}$ for the LD event appears to have been overpredicted by the parameter estimation procedures, and is better estimated by $\tilde{\zeta}^*=4.5\%$. Estimated $\tilde{\zeta}$ values from NR data were found to be similarly overpredicted relative to "known" values (Stewart, 1996).
- FF-R CEM results from the NR data were essentially identical for both recorded FF "rock" motions and calculated "soil" motions (from SHAKE analyses, see above).

The RPEM results for the NR data (Fig. A31.NR6) indicate decreases of about 0.2 Hz in the first-mode frequencies for both directions at about 10 to 20 seconds. These times correspond to a period of gradually increasing shaking intensity. Results from LD data for the EW direction (Fig. A31.LD3) indicate similar reductions in first-mode frequency during the first 30 seconds.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A31.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 61 feet using the recorded free-field motions (NR data) and SL motions (LD data) and the profile in Fig. A31.3. The instrumented tower was assumed to be independent of the adjacent buildings for the calculation of foundation radii. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by 60% to account for shape effects.

Table A31.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1050$ fps	$h=90$ ft	emb.=0	$\gamma = 0.15$
	$\nu=0.4$		tr: $r_1 = 61$ ft		$r_2 = 52$ ft
LD	$\beta^*=2.3\%$	$V_s^*=1010$ fps	L: $r_1 = 61$ ft		$r_2 = 74$ ft
NR	$\beta^*=3.2\%$	$V_s^*=970$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A31.3 with the fixed-base modal parameters in Table A31.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A31.4.

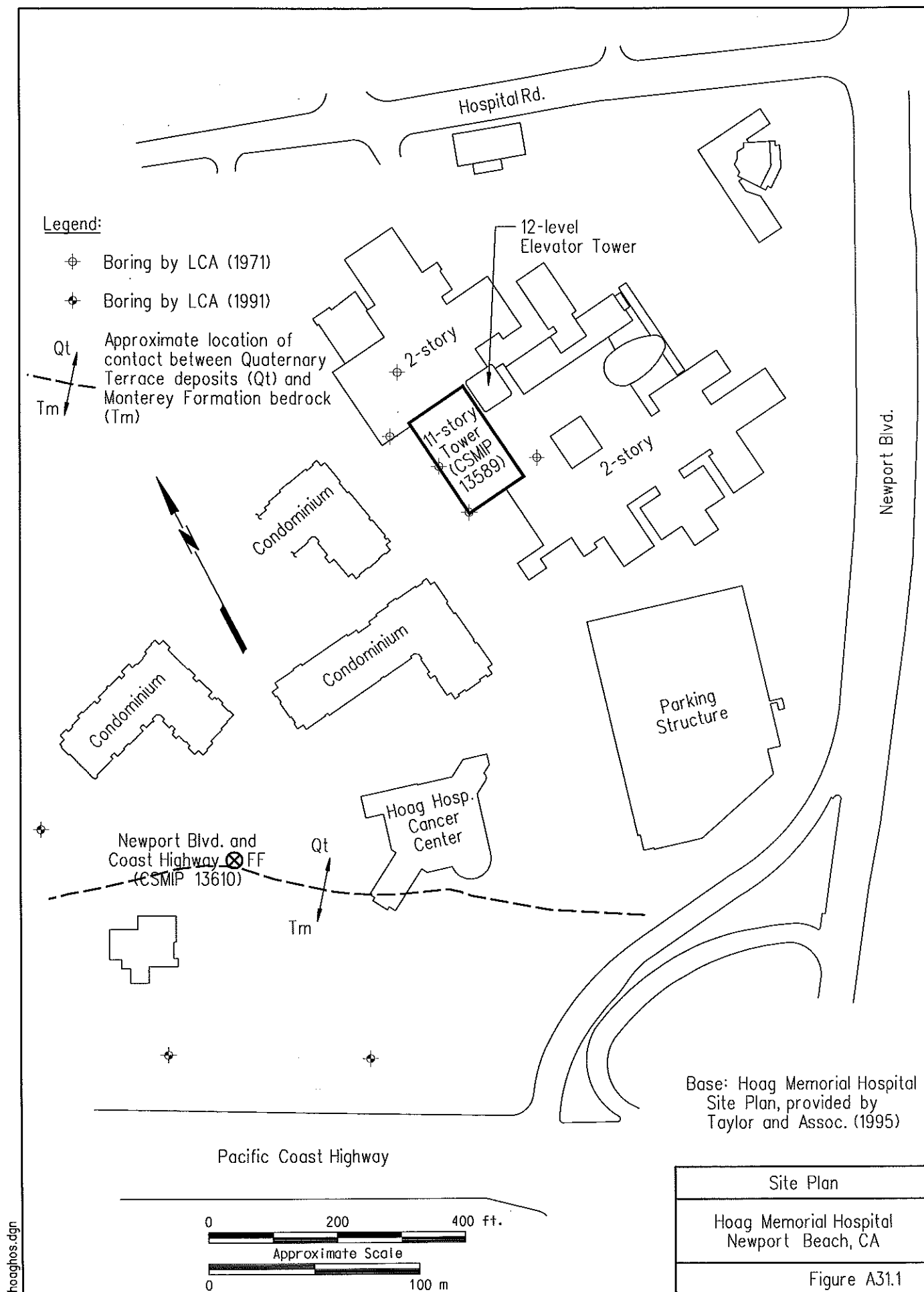
Table A31.4: Empirical and predicted inertial interaction effects

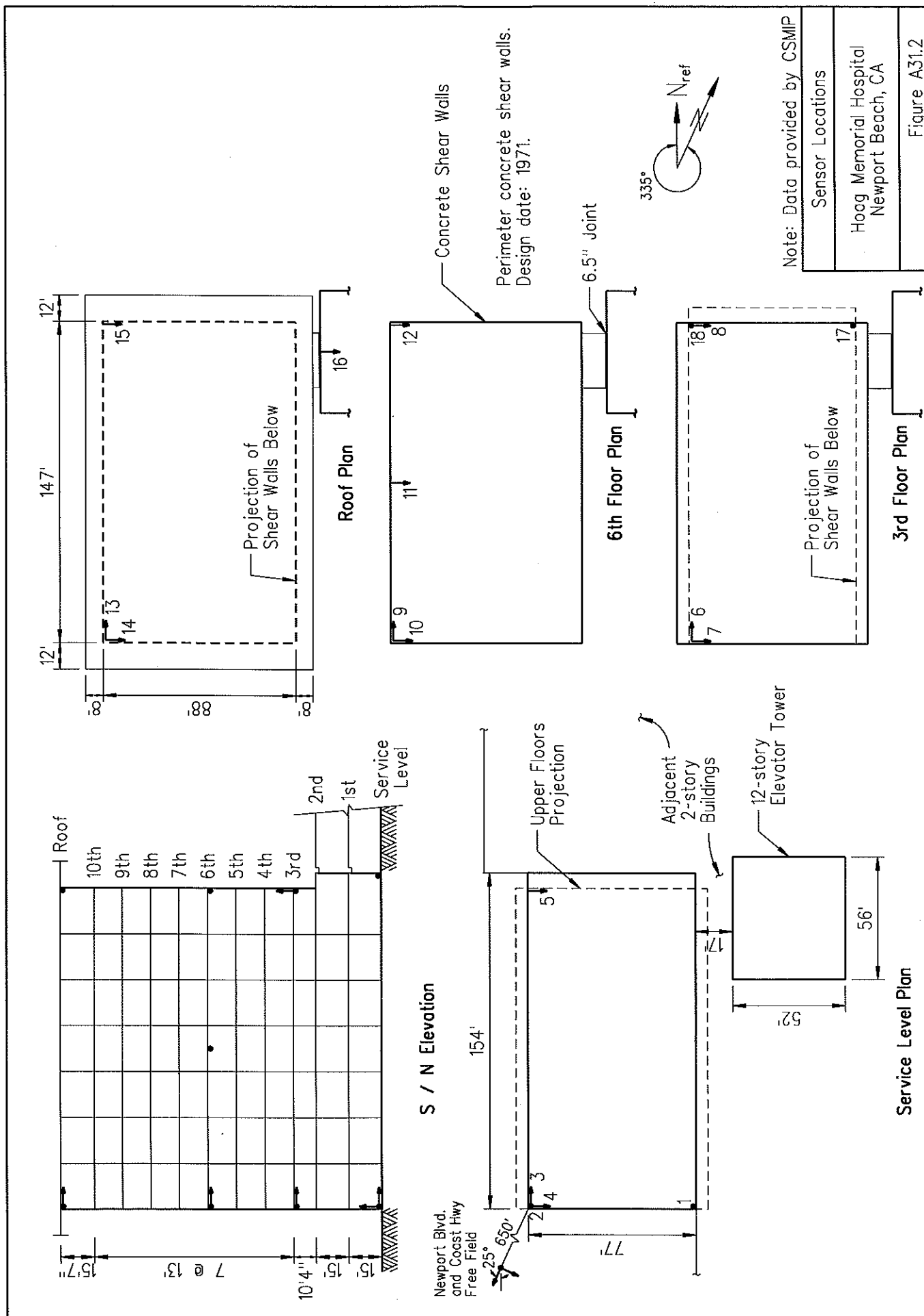
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
LD	Sys. ID	1.17	2.1	4.5	--	--	--
	Prediction	1.11	0.7	3.5	--	--	--
NR	Sys. ID	1.18	-1.0	3.1	1.14	1.6	4.1
	Prediction	1.11	0.7	5.6	1.06	1.1	4.2

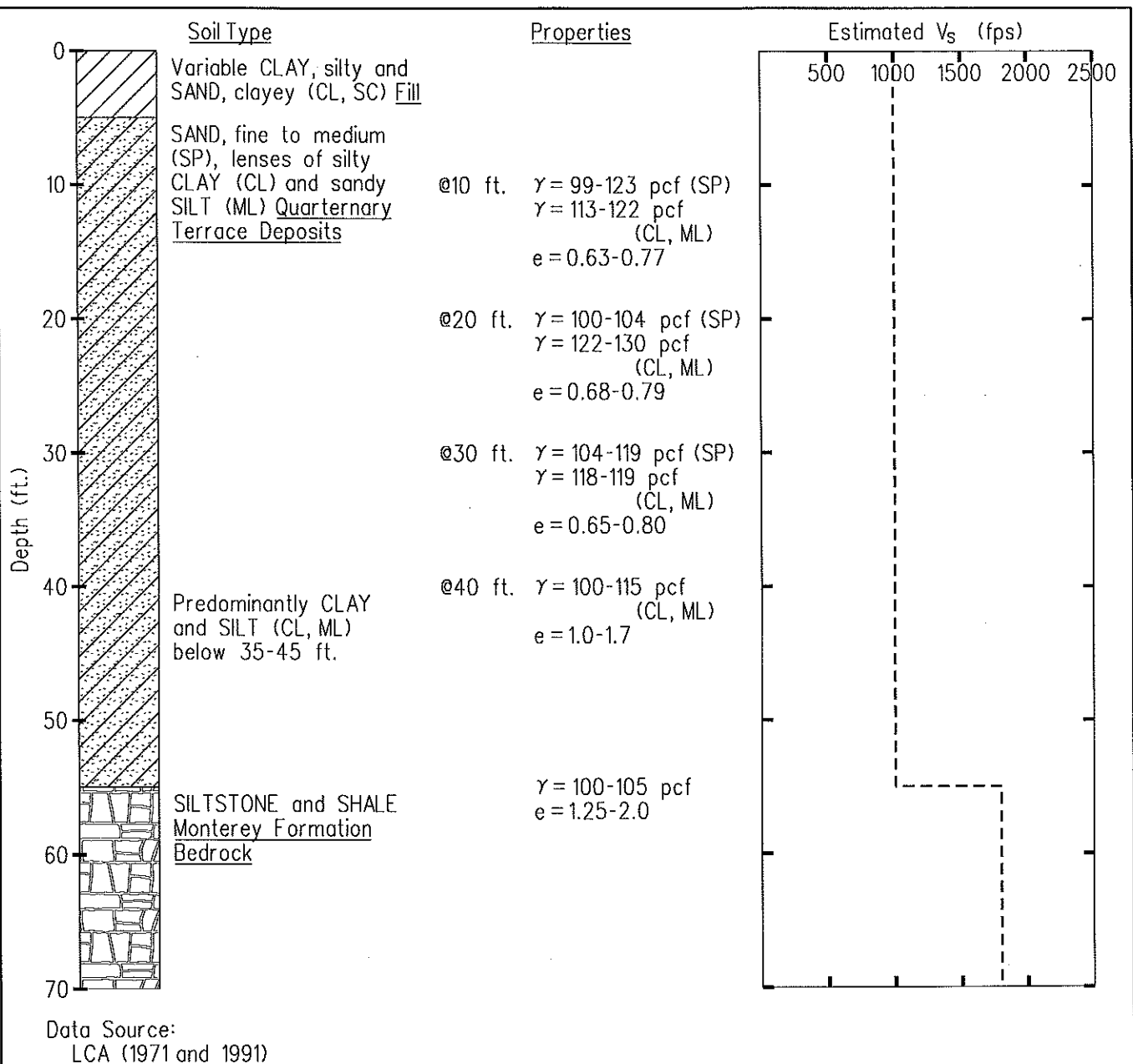
The period lengthening was underpredicted somewhat for both earthquakes, but is generally satisfactory given the uncertainty in the soil properties at the site. The negative foundation damping from the system identification analyses (NR data) indicates a numerical error in these results. However, the low foundation damping at this site was well predicted by the analyses.

Data Sources

Fumal, T.E., Gibbs, J.F., and Roth, E.F. (1981). "In situ measurements of seismic velocity at 19 locations in the Los Angeles, California region," *Open-File Rpt. 81-399*, U.S. Geol. Survey.
 LeRoy Crandall Assoc., LCA (1971 and 1991). unpublished foundation and geot. investigations, Hoag Memorial Hospital, Newport Beach, CA, *Job. No. A-69080 and O89034.AEO*.
 Stewart, J.P. (1996). "An empirical assessment of soil-structure interaction effects on the seismic response of structures," *Ph.D. Dissertation*, Univ. of California, Berkeley, Section 5.3.2.







Notes:

1. Groundwater encountered at 35-40 ft. depth.
2. Shear wave velocities in soil estimated based on correlations by Fumal & Tinsley (1985). Shear wave velocity in rock estimated based on in situ measurements in shale bedrock at the nearby Shellmaker Island site (Fumal et al., 1981).
3. The soil conditions illustrated here are representative of the hospital building site, but not of the free field site, where the thickness of the Terrace deposits is much smaller.

Generalized Soil Profile

Hoag Memorial Hospital
 Newport Beach, CA

Figure A31.3

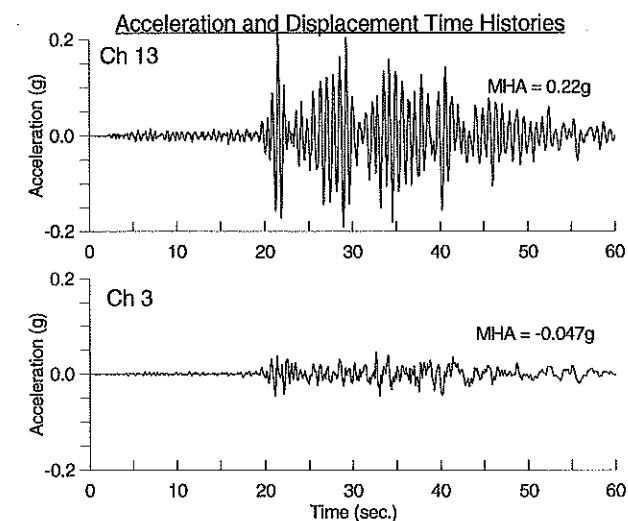
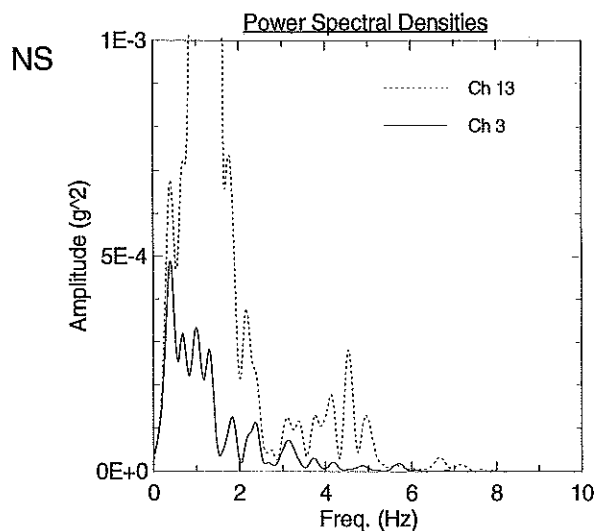
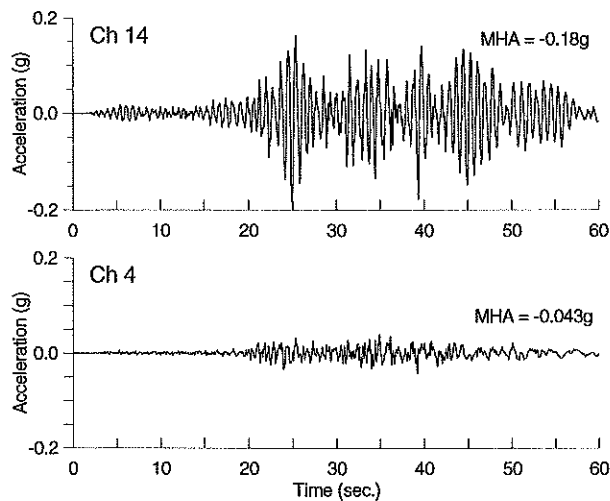
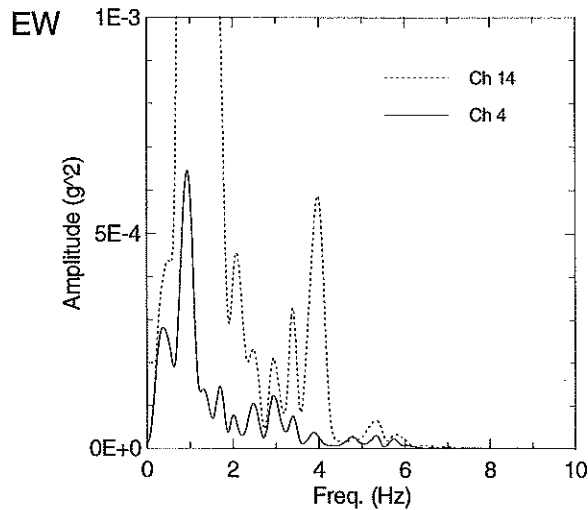
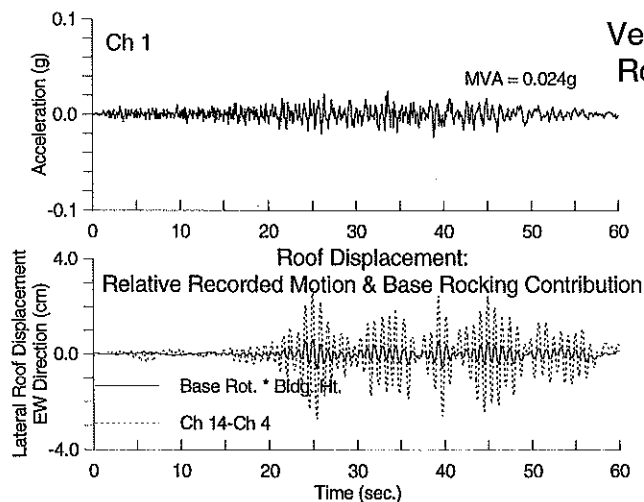
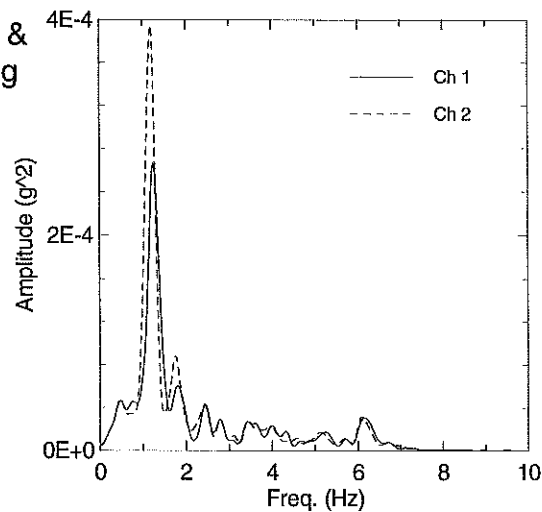


Fig. A31.LD1: NEWPORT BEACH HOAG HOSPITAL
Time Histories and Power Spectral Density Functions
1992 Landers Earthquake
A31-8

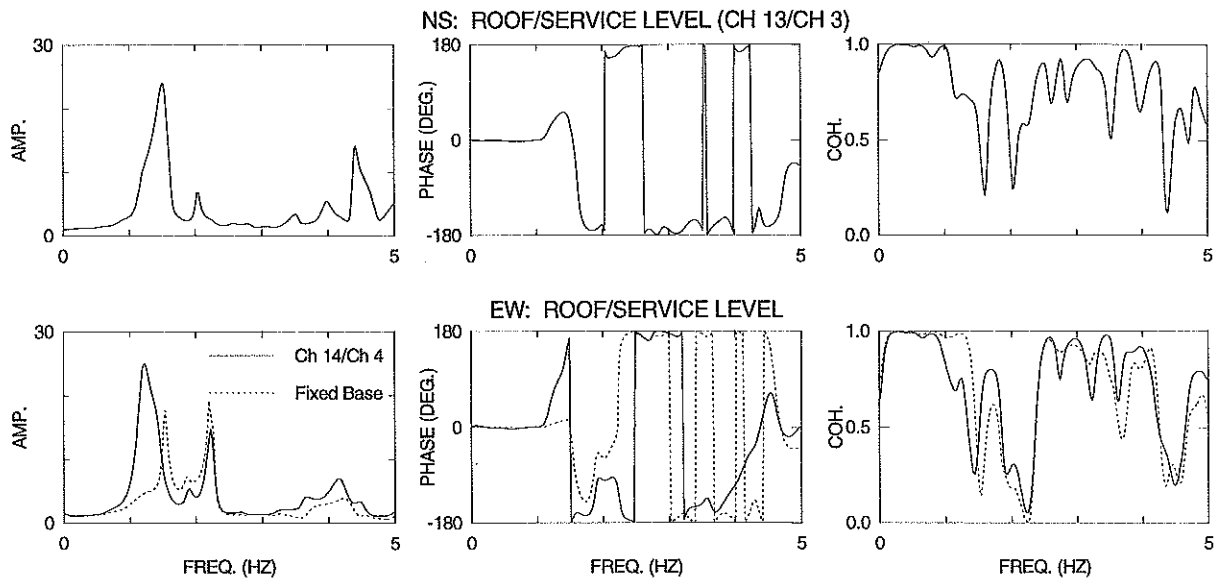


Fig. A31.LD2: NEWPORT BEACH HOAG HOSPITAL
Transmissibility and Coherency Functions
1992 Landers Earthquake

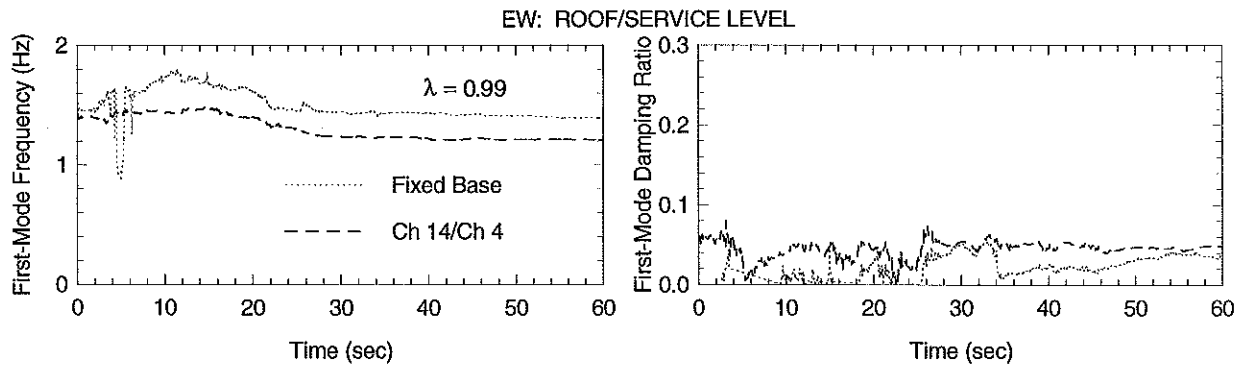


Fig. A31.LD3: NEWPORT BEACH HOAG HOSPITAL
Time Variation of First-Mode Parameters
1992 Landers Earthquake

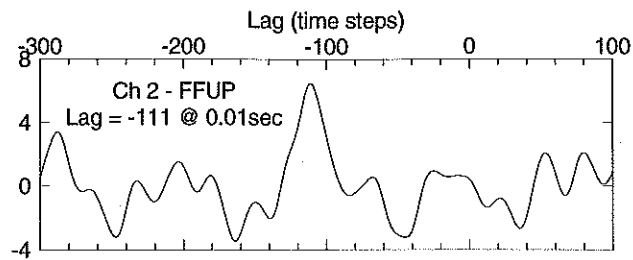


Fig. A31.NR0: Cross-correlation function between vertical service level and free-field accelerations

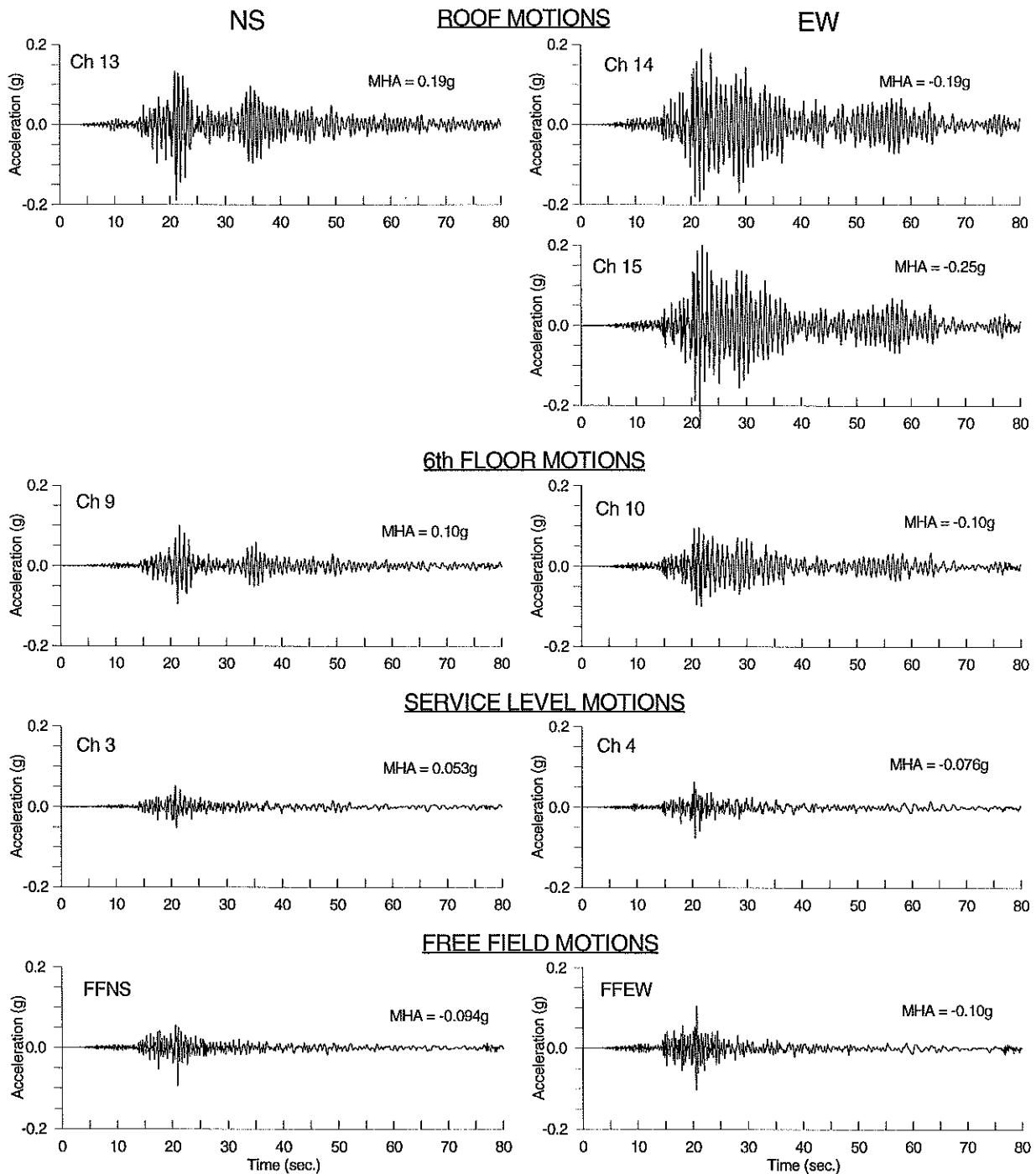


Fig. A31.NR1: NEWPORT BEACH HOAG HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

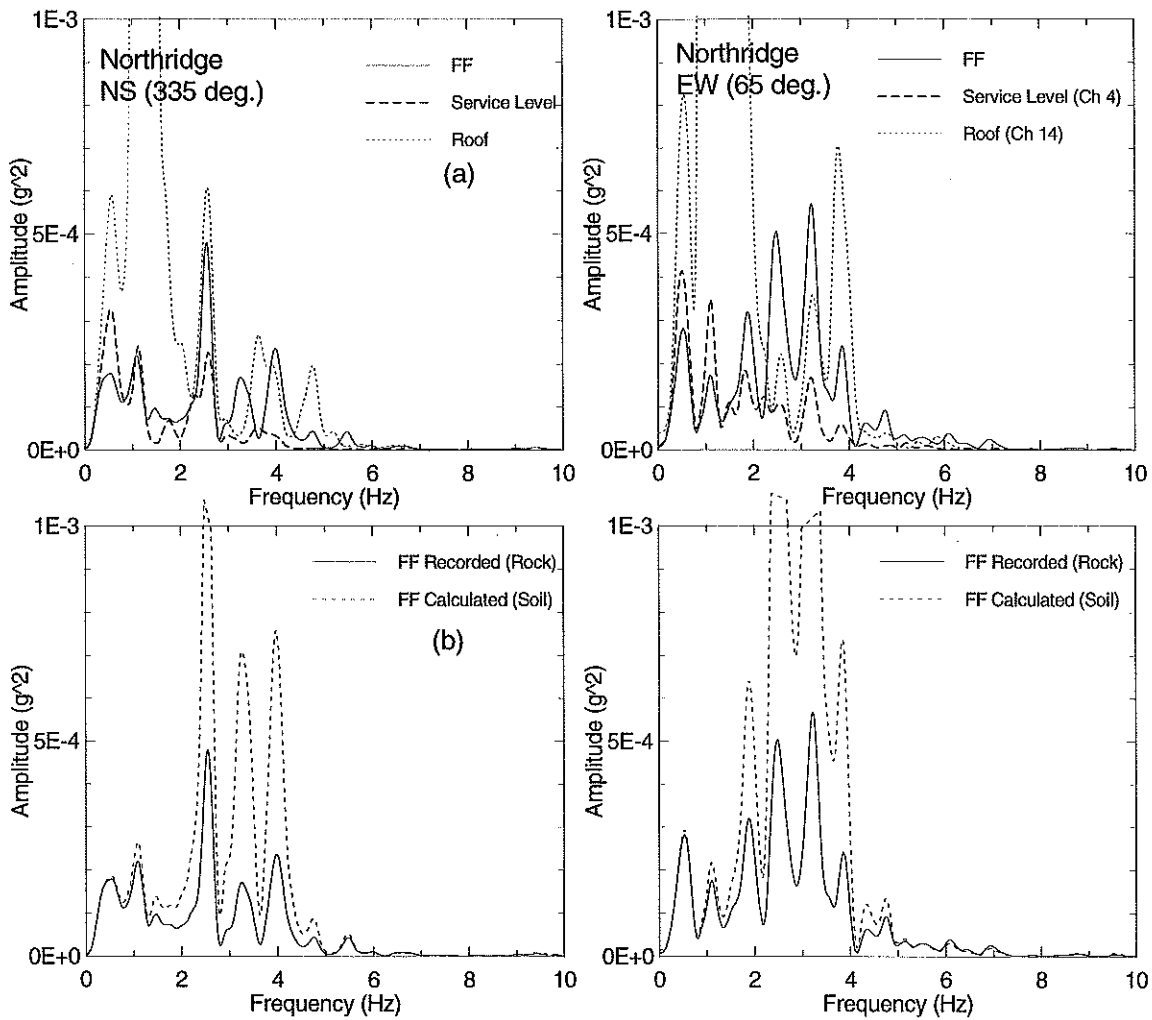


Fig. A31.NR2: NEWPORT BEACH HOAG HOSPITAL
 (a) Power Spectral Density of Selected Horizontal Accelerations
 (b) Spectral Comparison of Recorded Rock Motions and Calculated Soil Motions
 1994 Northridge Earthquake

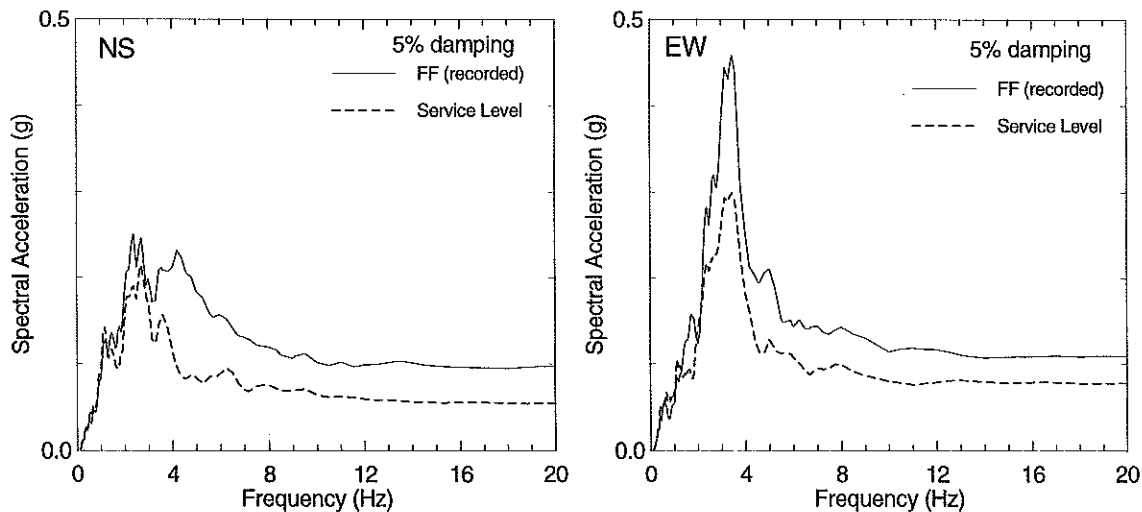


Fig. A31.NR3: NEWPORT BEACH HOAG HOSPITAL
 Acceleration Response Spectra of Service Level and Free-Field Motions
 1994 Northridge Earthquake

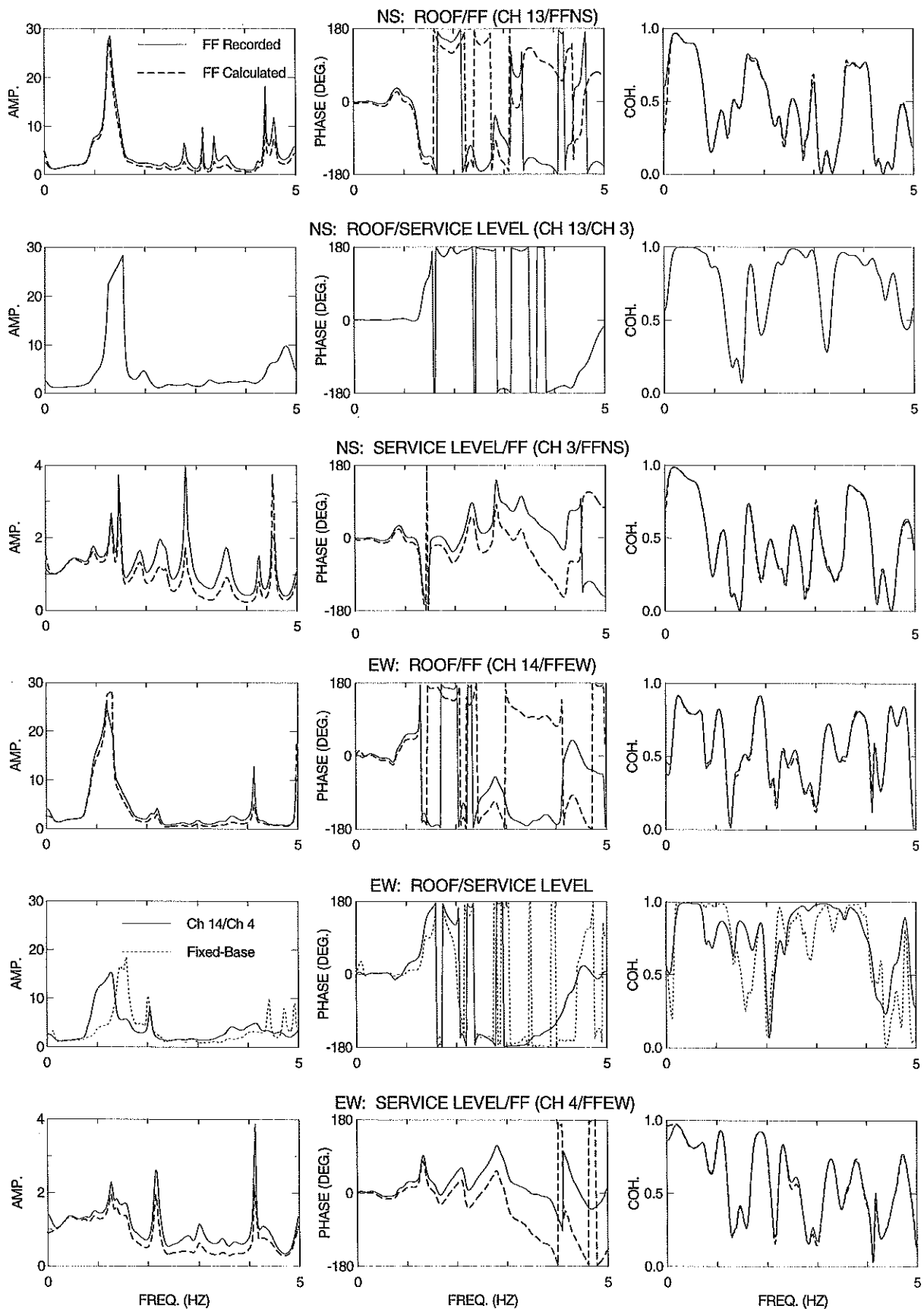


Fig. A31.NR4: NEWPORT BEACH HOAG HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake

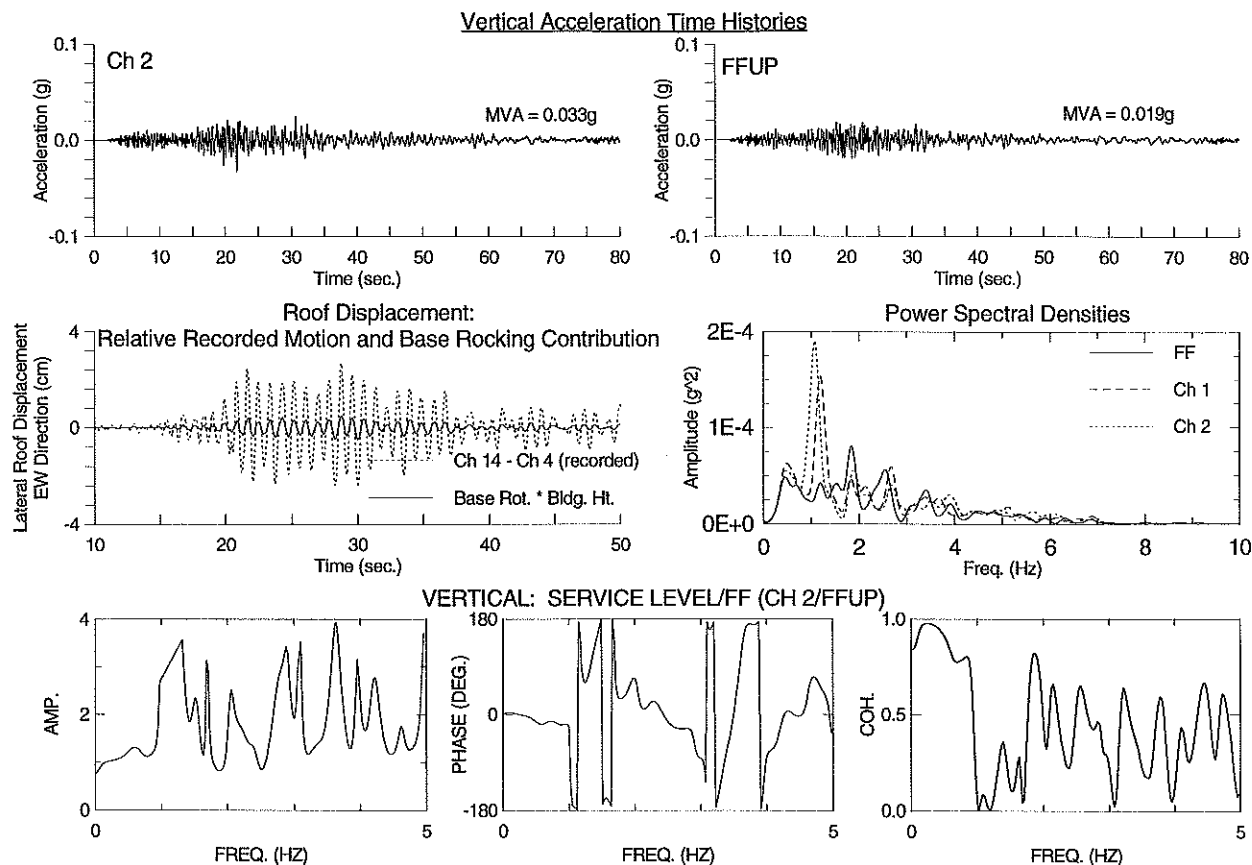


Fig. A31.NR5: NEWPORT BEACH HOAG HOSPITAL
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

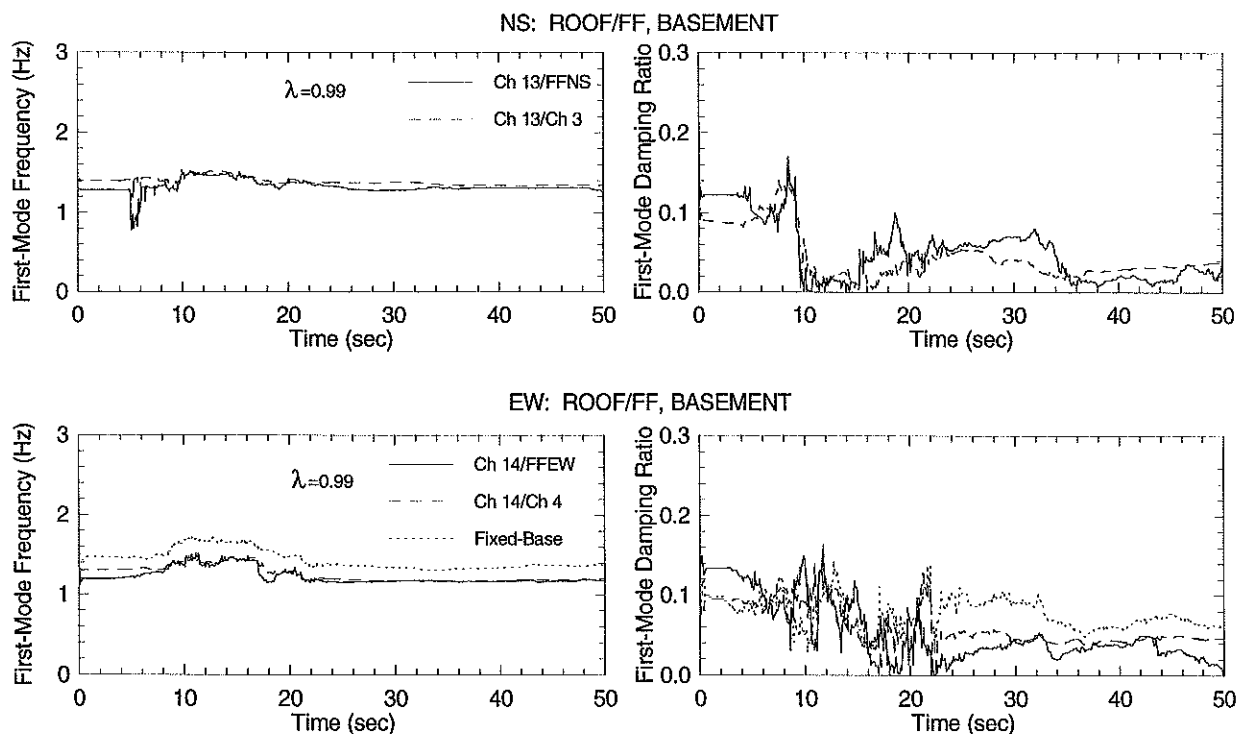


Fig. A31.NR6: NEWPORT BEACH HOAG HOSPITAL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A32

Site Name: Norwalk, 12400 Imperial Highway
Sensors: USGS 634 and USGS 5239

Lat: 33.916°
Long.: 118.065°

Floor Levels: 8
Bsmt. Levels: 1
Structure: exterior steel frame

Foundation: concrete piles
Site Geology: Alluvium
Design Date: early 1970s

Recorded Peak Accelerations (g)

Earthquake		Free-Field			Basement			Roof		
		NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	FF2	0.25	0.090	0.095	0.20	0.11	0.063	0.42	0.19	0.094
	FF3	0.22	0.13	0.13						
	FF4	0.20	0.088	0.091						

Preprocessing

Whittier Earthquake (WT)

Preliminary: by USGS; filtered, $f = 0.5\text{--}50\text{ Hz}$; $f_{Nyq} = 100\text{ Hz}$; FF/str. data not synchronized
Final: decimated by 8 to $f_{Nyq}=12.5\text{ Hz}$; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A32.WT0).

Figures

A32.1-A32.3: Site Plan, Sensor Locations, Generalized Soil Column
A32.WT0-6 WT Analysis Results



View of structure with FF2 in foreground

DISCUSSION

Site Description

The building is 7 stories tall above ground level with a single level basement, 130 by 210 feet in plan, and is located 53.5 feet from an adjacent 7-story building to the east. The lateral load resisting system consists of exterior steel frames, while the foundation consists of concrete piles varying in length from 25 to 35 feet (Celebi, 1993). As shown in Fig. A32.2, triaxial accelerographs were present in the basement, 5th, and 8th (roof) levels (the sensors were removed in 1992). Four free-field instruments (denoted FF1-4) are located 209, 502, 570, and 488 feet from the structure, respectively.

The site is located in the Los Angeles basin and is underlain by deep alluvium. Based on borings by R.T. Franklin and Assoc. (1981), near-surface soils consist of silty, clayey sands and silts which extend to the maximum depth explored of 60 feet (Fig. A32.3). Based on oil well logs for nearby areas, the near-surface Holocene sediments appear to be underlain at 75 feet depth by stiffer Pleistocene sediments with $V_s \approx 1200$ ft/sec which are underlain by sandstone bedrock at about 500 feet (Tinsley, 1996, Celebi, 1993).

Instrument Selection and Free-Field Conditions

The structural instrumentation consisted only of triaxial accelerographs near the centroid of the building. The sensors used for analysis are indicated in Table A32.1; acceleration time histories recorded by these sensors are shown in Fig. A32.WT1.

Table A32.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	UP	NS	EW
Ch 1	Ch 3	Ch 2	Ch 7	Ch 9

There are no structures or significant slope breaks near the free-field instruments that would be expected to compromise the validity of the recorded free-field ground motions. The FF1 instrument was not operational at the time of the WT event.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A32.WT4) and the spectra of horizontal motions (Fig. A32.WT2-3) generally indicate de-amplification of basement motions relative to the free-field for frequencies greater than about 2 Hz. The near zero amplitude of the transmissibility functions at frequencies less than about 0.5 Hz is unreliable as the data had been high-pass filtered during preprocessing at a corner frequency of 0.5 Hz.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Fig. A32.WT5. Amplification occurred near the transverse first-mode frequency, indicating a potential influence of rocking on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A32.WT4), (2) modal parameters from CEM analyses (Table A32.2), and (3) time variations of first-mode parameters from the RPEM (Fig. A32.WT6). For comparison, frequencies identified from ambient and forced vibration tests are presented in Table A32.3.

Table A32.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF 2/3/4 - R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=8, j=9		L (EW) d=7, j=9		tr (NS) d=7/8/6, j=9/9/9		L (EW) d=7/7/6, j=9/9/9	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	na	na	--	--	0.64	7.0	0.64	6.7	0.67/0.68/ 0.67	8.2/9.9/ 7.8	0.67/0.67/ 0.67	10.2/12.3/ 11.0 6.7
	2					1.99	9.1	1.99	8.5	2.05/1.97/ 2.02	9.2/8.9/ 8.6	2.14/2.14/ 2.12	9.1/5.9/ 11.0
	3					3.19	6.3	3.47	7.9	3.16/3.13/ 3.20	6.7/7.7/ 5.8	3.46/3.41/ 3.47	7.1/9.0/ 8.1

Table A32.3: Modal frequencies (in Hz) identified from ambient and forced vibration tests (Hadjian et al., 1977)

Mode	Ambient Vibration			Forced Vibration		
	NS	EW	Torsion	NS	EW	Torsion
1	1.03	1.07	1.35	0.92	0.93	1.20
2	3.20	3.20	4.00	3.10	3.00	--
3	5.10	5.22	7.18	4.60	5.00	6.80

Comments on the analytical results follow:

- As shown in Fig. A32.WT2, the roof motion for the transverse direction is dominated by first-mode structural response and excitation at the predominant frequency of the input motions ($f \approx 1.5$ to 2.0 Hz). In the EW direction, roof motions are dominated by structural response in the first and second modes
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. Given that a base-rocking response was evident from the vertical B/FF transmissibility function, transverse fixed-base parameters could not be estimated by pseudo flexible-base parameters. In the longitudinal direction, rocking is not likely to be significant, so fixed-base parameters can be approximated by pseudo flexible-base parameters.
- No significant differences are apparent in the spectra of the motions from the three FF instruments (Fig. A32.WT2), indicating that FF motions at the site were fairly coherent.
- The CEM results for B-R response are consistent the findings of Celebi (1993), who used similar identification techniques. The frequencies determined from low-amplitude forced and ambient vibration tests (Table A32.3) were significantly larger.

The RPEM results (Fig. A32.WT6) indicate no significant time-dependent changes in the first-mode parameters of the structure. Unlike the CEM analyses, RPEM analyses indicated FF-

R damping ratios which are essentially the same as those for the B-R case (i.e., as indicated in Table A32.2, a better estimate is $\tilde{\zeta} \approx 6.7$ Hz). The CEM and RPEM results were consistent with regard to the relative magnitude of the FF-R and B-R first-mode frequencies.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A32.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 93 feet using the recorded free-field motions and the profile in Fig. A32.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though the rocking radiation damping for the transverse direction was increased by 50% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A32.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
WT	Model = H	$V_s = 910$ fps	$h = 70$ ft	$emb. = 13.5$ ft	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 93$ ft		$r_2 = 83$ ft
	$\beta^* = 6.8\%$	$V_s^* = 730$ fps	L: $r_1 = 93$ ft		$r_2 = 106$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A32.4 with the fixed-base modal parameters discussed in Part (b), the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A32.5.

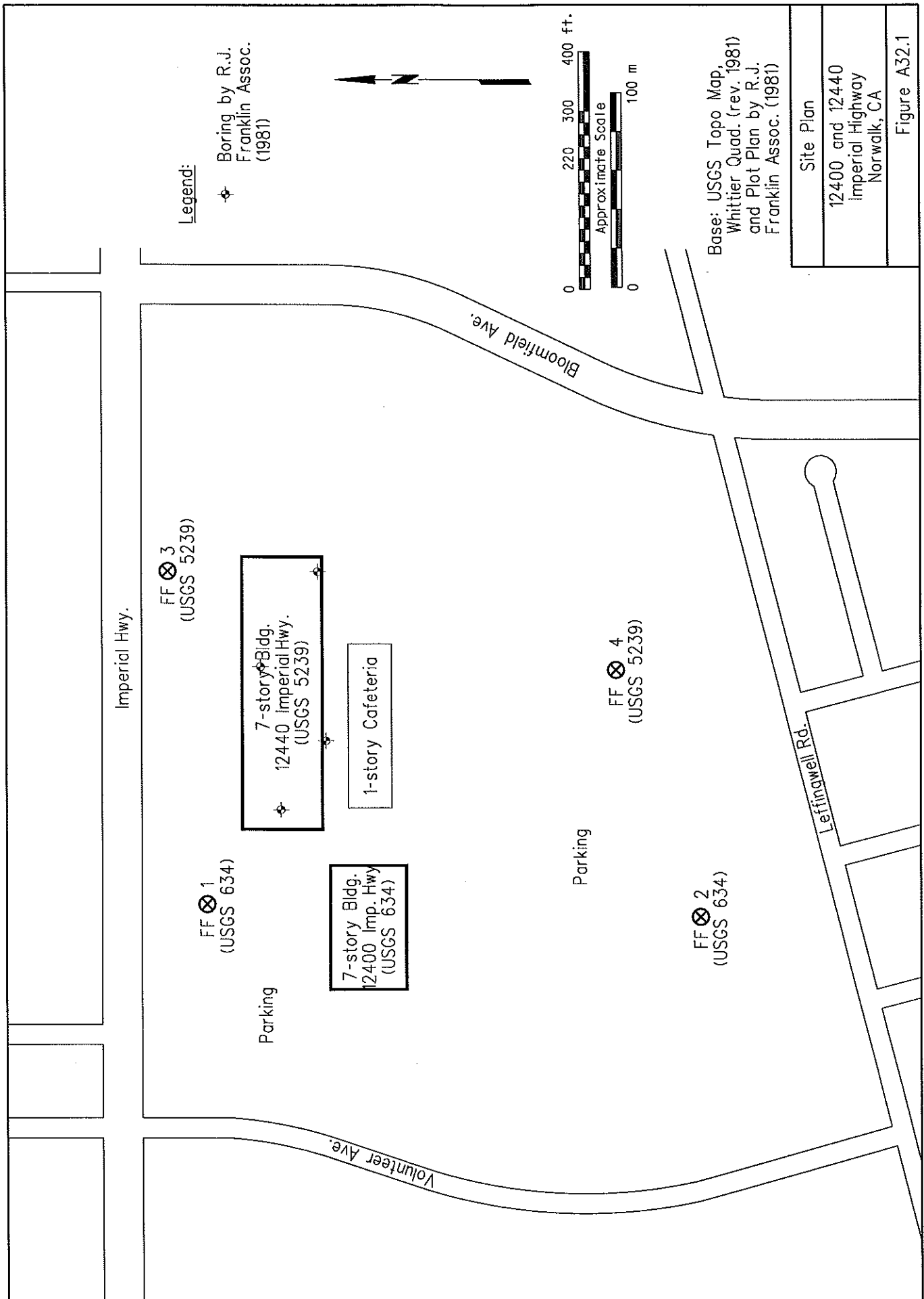
Table A32.5: Empirical and predicted inertial interaction effects

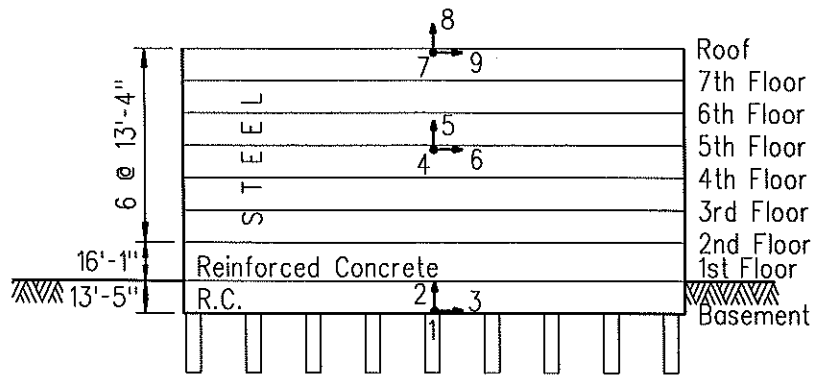
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
WT	Sys. ID	na	na	na	0.96	0	6.7
	Pre.-Vel.	na	na	na	1.01	0.3	6.8
	Pre.-Bie.	na	na	na	1.02	0.6	7.0

The predicted period lengthening by both formulations is satisfactory, given the lack of precision in the system identification results. The foundation damping factor $\tilde{\zeta}_0$ is slightly better predicted by the “modified Veletsos” formulation.

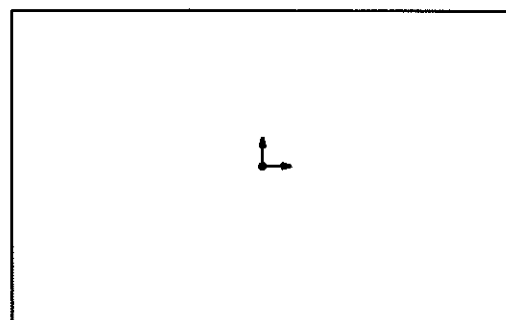
Data Sources

- Celebi, M. (1993). “Seismic response of two adjacent buildings. I: Data and analyses,” *J. Struct. Engrg.*, ASCE, 119(8), 2461-2476.
- Hadjian, A, Smith, C., Haldar, A., and Ibanez, P. (1977). “Variability in engineering aspects of structural modeling,” *Proc. 6th World Conf. on Earthquake Engrg.*, Vol. 3, 2729-2734.
- Franklin and Associates (1981), “Report of Foundation Investigation, Proposed Bechtel Building No. 43, 12400 Imperial Highway, Norwalk, CA,” *Job. No. 81-070-F*.
- Tinsley, J. (1996), pers. communication.



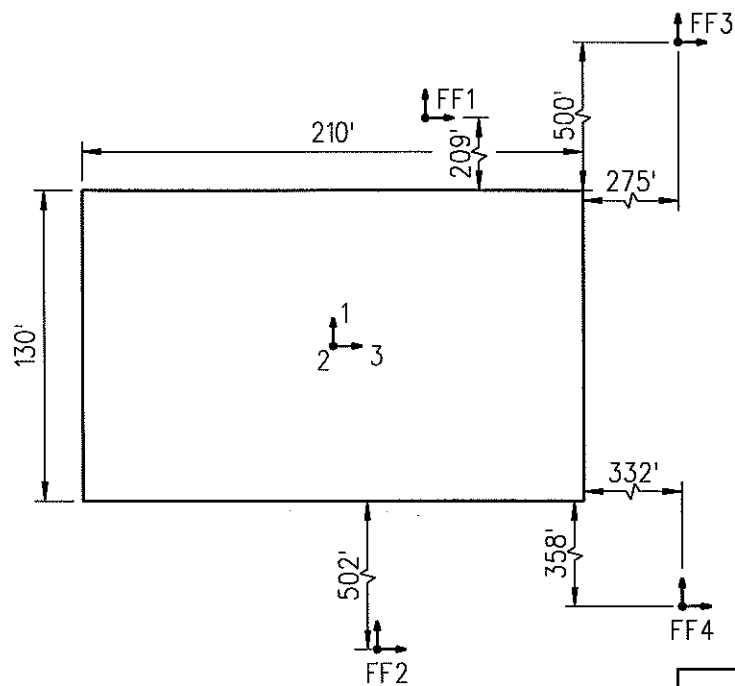


E / W Section



Plan View 5th & 7th Floors

$N = N_{ref}$

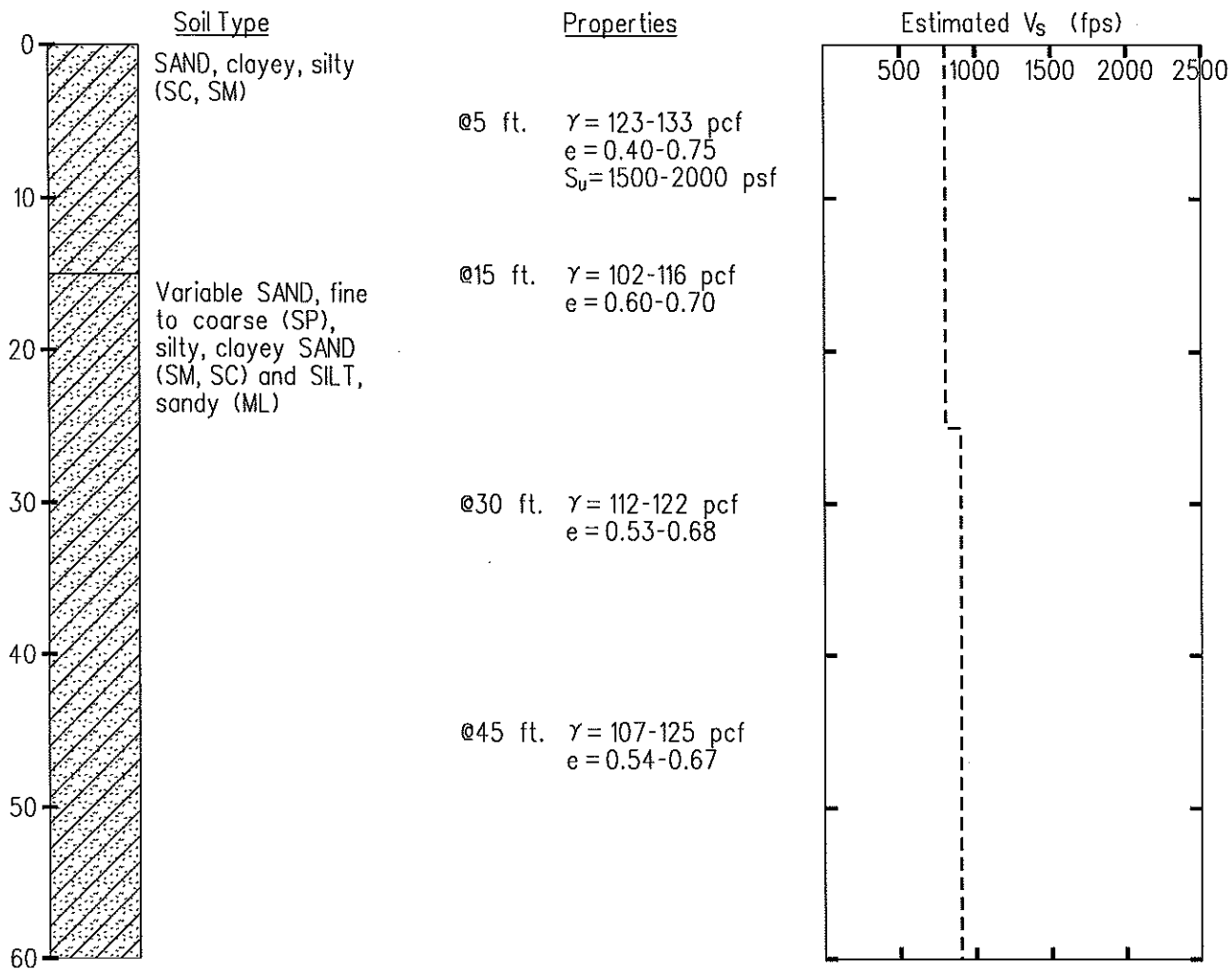


Basement Plan

Sensor Locations
12400 Imperial Highway Norwalk, CA
Figure A32.2

norwalk.dgn[80]

Note: Data provided by USGS



Data Source:
R.T. Franklin
Assoc. (1981)

Notes:

1. No groundwater encountered.
2. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile

12400 and 12440
Imperial Highway
Norwalk, CA

Figure A32.3

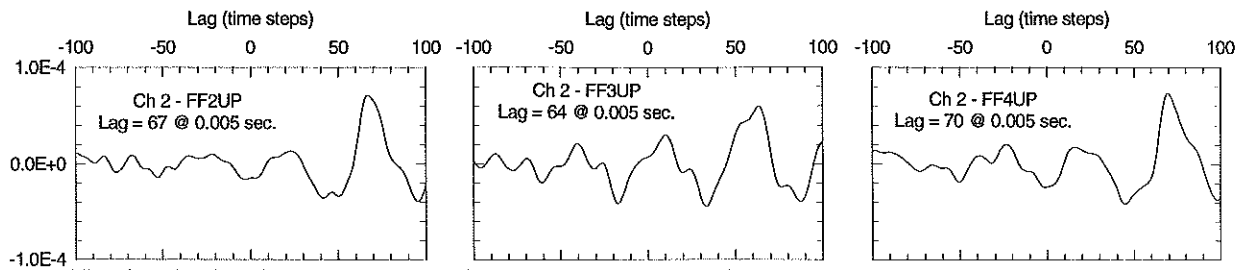


Fig. A32.WT0: Cross-correlation functions between vertical basement and free-field accelerations

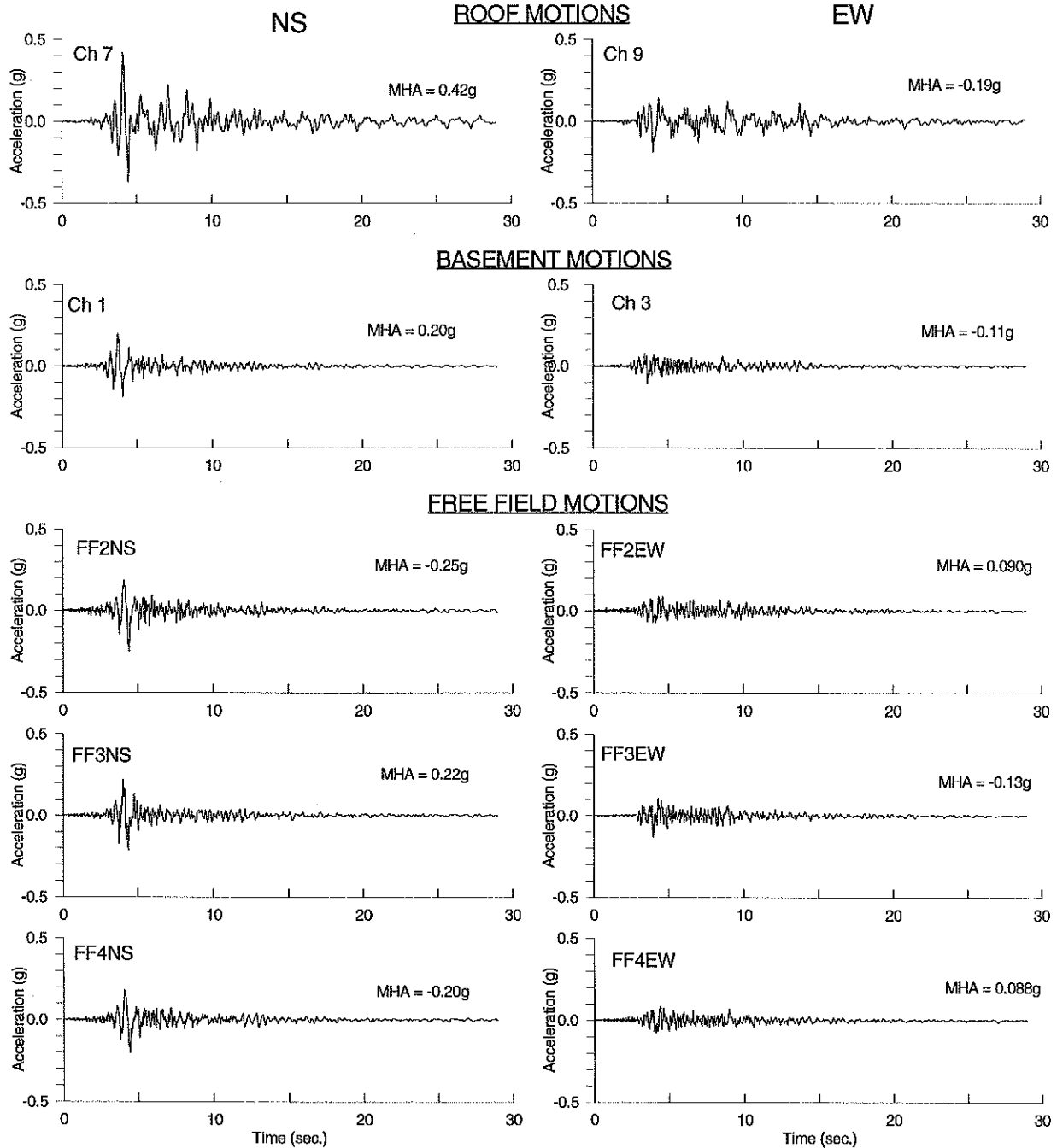


Fig. A32.WT1: NORWALK 12400 IMPERIAL HIGHWAY
Horizontal Acceleration Time Histories
1987 Whittier Earthquake

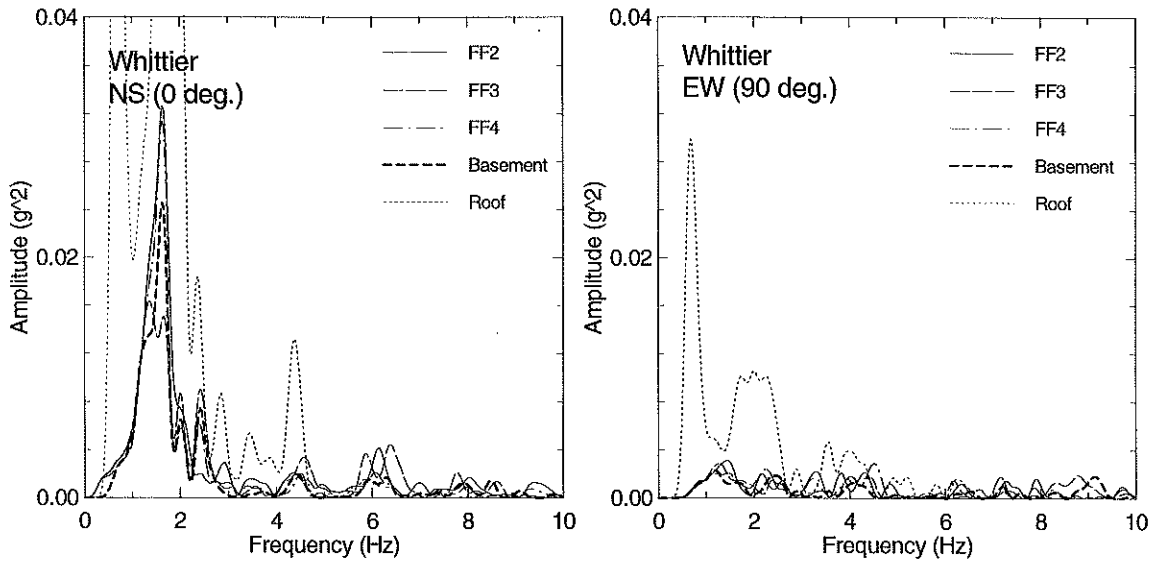


Fig. A32.WT2: NORWALK 12400 IMPERIAL HIGHWAY
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

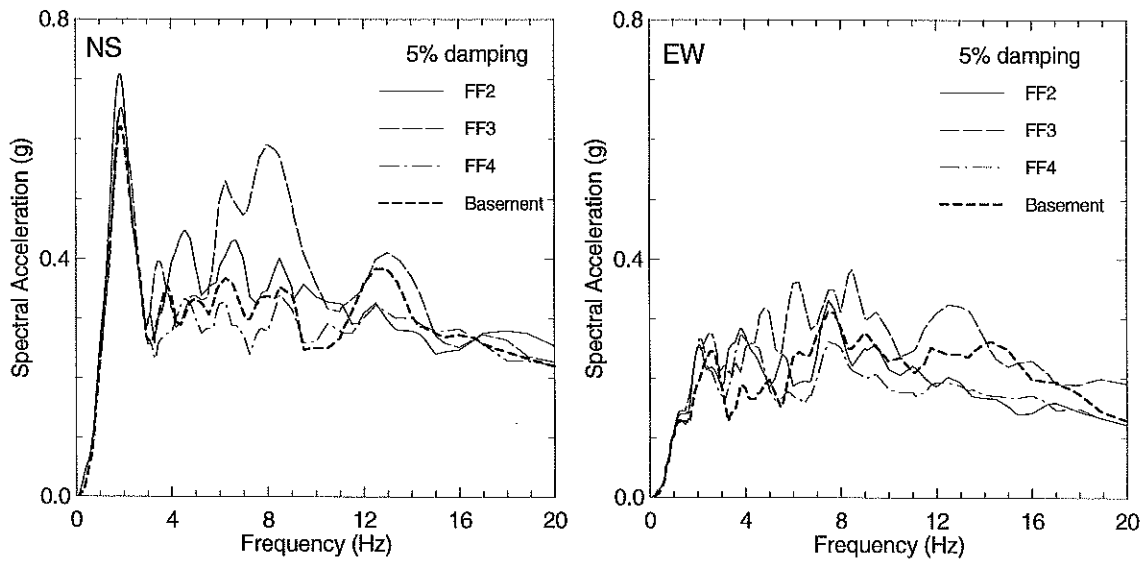


Fig. A32.WT3: NORWALK 12400 IMPERIAL HIGHWAY
Acceleration Response Spectra of Basement and Free-Field Motions
1987 Whittier Earthquake

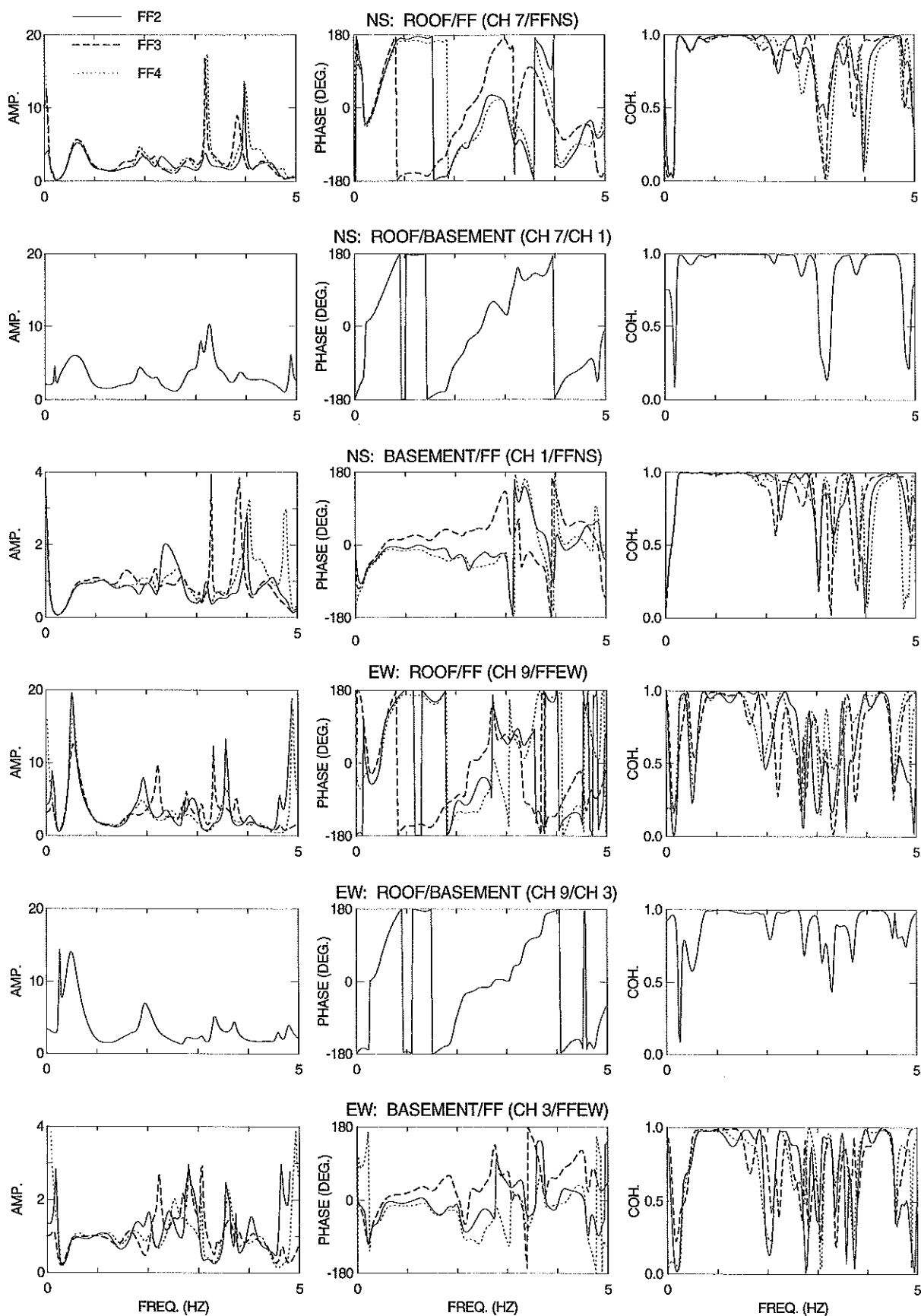


Fig. A32.WT4: NORWALK 12400 IMPERIAL HIGHWAY
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A32-10

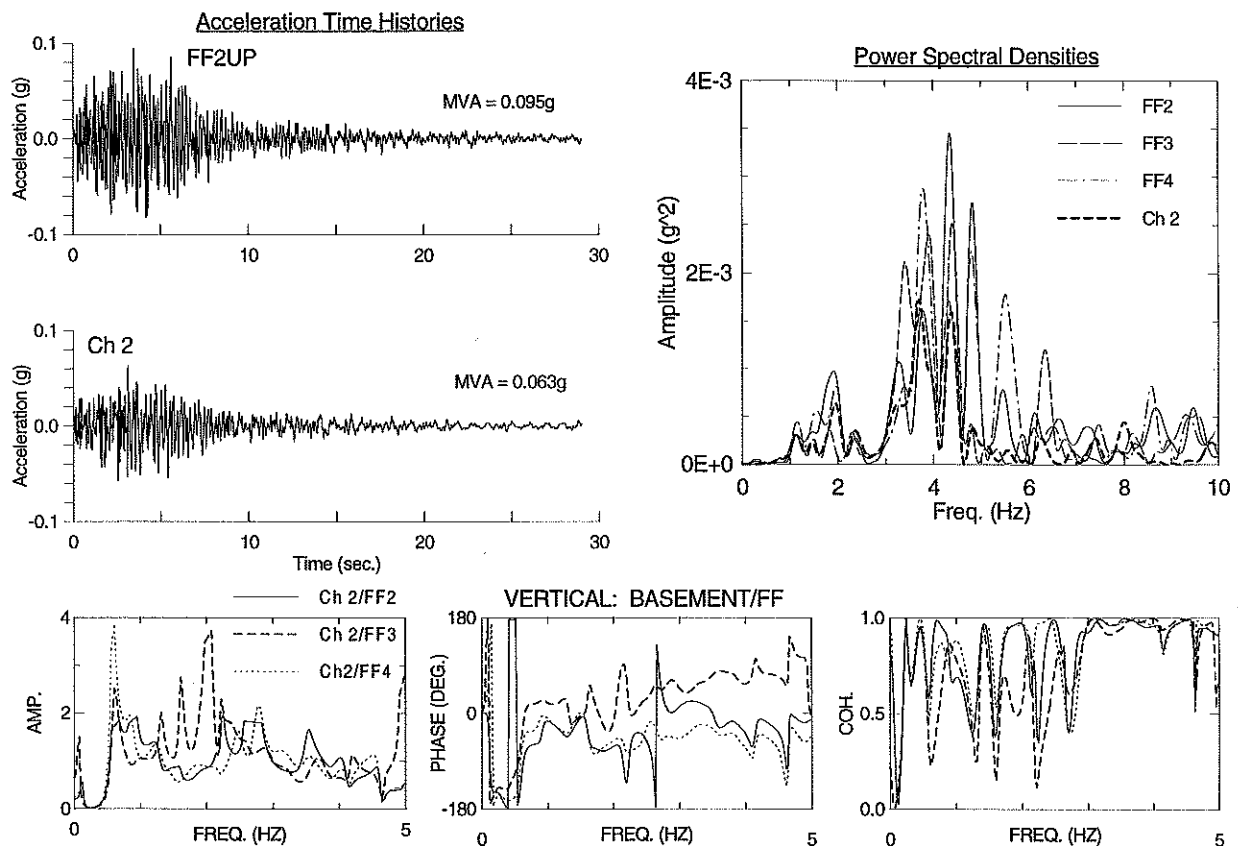


Fig. A 32.WT5: NORWALK 12400 IMPERIAL HIGHWAY
Analysis of Vertical Motions
1987 Whittier Earthquake

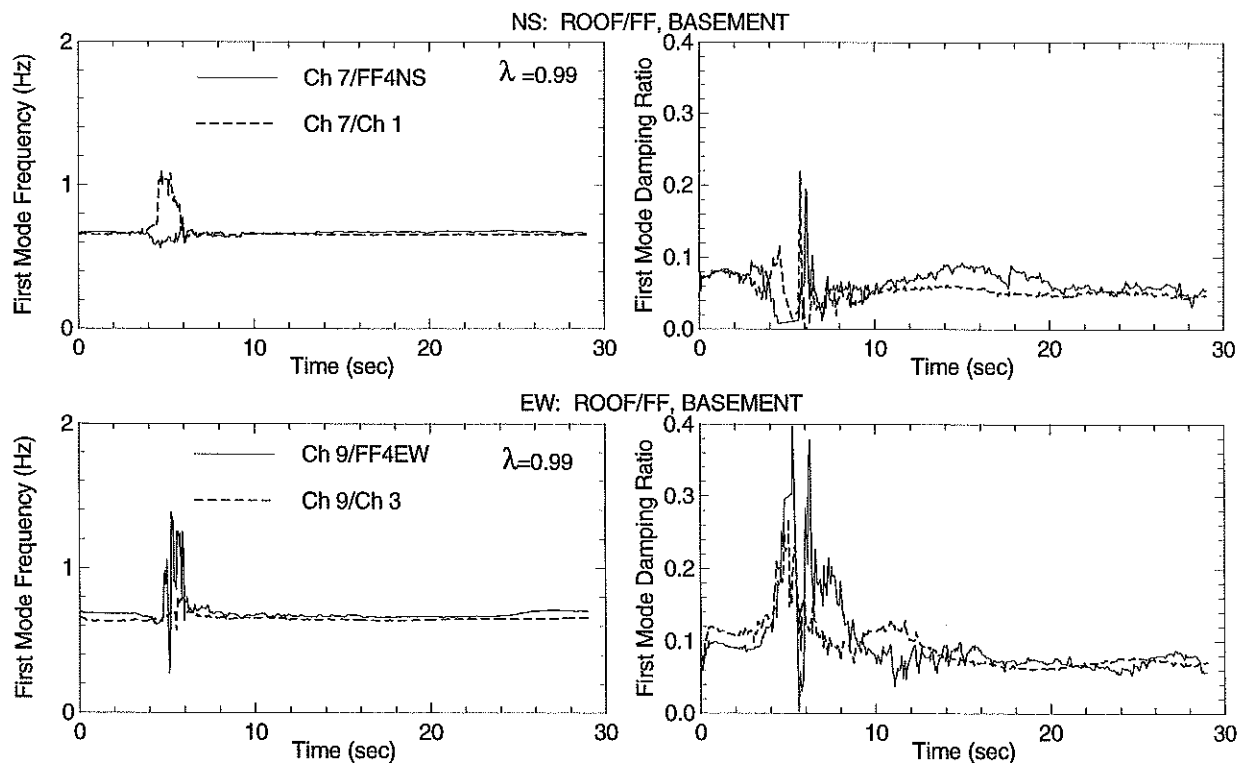


Fig. A32.WT6: NORWALK 12400 IMPERIAL HIGHWAY
Time Variation of First Mode Parameters
1987 Whittier Earthquake

SITE A33

Site Name: Norwalk, 12440 Imperial Highway
Sensors: USGS 634 and USGS 5239

Lat: 33.916°
Long.: 118.065°

Floor Levels: 8
Bsmt. Levels: 1
Structure: Moment-resisting steel frame

Foundation: drilled piers
Site Geology: Alluvium
Design Date: 1982

Recorded Peak Accelerations (g)

Earthquake		Free-Field			Basement			Roof		
		NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	FF2	0.25	0.090	0.095	0.19	0.11	0.054	0.41	0.17	--
	FF3	0.22	0.13	0.13						
	FF4	0.20	0.088	0.091						
1992 Landers*	FF2	0.05	0.06	0.05	0.06	0.05	0.02	0.24	0.12	--
	FF3	0.07	0.07	0.08						
1994 Northridge	FF1	0.071	0.074	0.051	0.058	0.048	0.029	0.14	0.14	--
	FF2	0.085	0.055	0.045						
	FF3	0.063	0.058	0.051						

*Not digitized

-- = No instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: by USGS; high-pass filtered @ $f = 0.17$ Hz; $f_{Nyq} = 100$ Hz; FF/str. data synchronized.

Final: decimated by 8 to $f_{Nyq} = 12.5$ Hz.

Northridge Earthquake (NR)

Preliminary: digitized at $f_{Nyq} = 100$ Hz; no processing, FF/str. data not synchronized

Final: filtered (this study), $f_{HP} = 0.13$ Hz; decimated by 8 to $f_{Nyq} = 12.5$ Hz, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A33.NR0).

Figures

A32.1, A32.3 Site Plan, Generalized Soil Column
A33.1 Sensor Locations
A33.WT1-6 WT Analysis Results
A33.NR0-6 NR Analysis Results

View of structure
with FF2 in foreground



DISCUSSION

Site Description

The building is a regular structure, 7 stories tall above ground level with a single level basement, 136 by 463 feet in plan, and is located 53.5 feet from an adjacent 7-story building to the west. The structure is a steel moment resisting ductile frame with a foundation consisting of 30-foot drilled caissons. As shown in Fig. A33.1, sensors are present at 30-foot depth at the base of a caisson, in the basement, and at the 1st, 2nd, 5th, and 8th (roof) levels. Four free-field instruments (denoted FF1-4) are located 141, 662, 125, and 502 feet from the structure, respectively. Soil conditions at the site are described in the discussion for site A32.

Instrument Selection and Free-Field Conditions

Sensors near the centroid of the structure were used for analysis, as indicated in Table A33.1. Acceleration time histories recorded by these and other sensors are shown in Figs. A33.**1. Ch 8 did not trigger in the NR event, so Ch 27 was used as the basement record for the longitudinal (EW) direction. The two vertical instruments located in the basement were used to evaluate rocking effects in the longitudinal direction.

Table A33.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 9	Ch 8, 27	Ch 7,26	Ch 17	Ch 1

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the longitudinal direction,

$$\begin{aligned}\text{WT:} \quad & \text{input} = \text{Ch8} + (\text{Ch26} - \text{Ch7}) \cdot 113/230 \\ \text{NR:} \quad & \text{input} = \text{Ch27} + (\text{Ch26} - \text{Ch7}) \cdot 113/230\end{aligned}\tag{1}$$

There are no structures or significant slope breaks near the FF instruments that would be expected to compromise the validity of the recorded free-field ground motions. FF1 was not operational at the time of the WT event, and FF4 was not operational during the NR event.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A33.**4) and the spectra for horizontal motions (Fig. A33.**2) generally indicate de-amplification of basement motions relative to the free-field at frequencies greater than about 1.5 Hz.

Transmissibility, coherency, and power spectral density functions for FF and B vertical motions are presented in Fig. A33.**5. No amplification occurred near the first-mode frequency, indicating that base-rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A33.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A33.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A33.**6).

Table A33.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base [B-R & (1F-R)*]				Flex.-Base [FF 2/3/4-R (WT) & FF 1/2/3-R (NR)]			
		tr (NS)		L (EW)		tr (NS)		L (EW)		tr (NS)		L (EW)	
		WT: estimated	NR: estimated	d=5, j=7	d=5, j=7	d=9(9), j=8(8)	d=9(8), j=8(8)	d=6(7), j=4(7)	d=6(6), j=7(7)	d=10/9/10, j=8/8/8	d=9/20/9, j=8/8/8	d=19/5/9, j=7/7/7	d=5/17/6, j=7/7/7
		f	ζ	f	ζ	f̃*	ζ̃*	f̃*	ζ̃*	f̃	ζ̃	f̃	ζ̃
WT	1	0.79	2.8	0.83	2.8	0.77 (0.78)	2.8 (2.7)	0.83 (0.82)	3.0 (2.9)	0.77/0.77/ 0.77	3.1/3.2/ 3.4	0.83/0.83/ 0.83	4.9/3.5/ 4.5
	2			2.34	4.8	1.95 (1.97)	5.6 (6.0)	2.34 (2.35)	4.4 (4.7)	2.00/1.95/ 1.97	4.7/6.3/ 4.5	2.38/2.32/ 2.39	1.8/3.4/ 4.7
	3			3.90	7.1	3.03 (3.06)	4.7 (5.7)	3.89 (3.98)	6.7 (7.1)	3.02/3.01/ 3.05	4.6/5.5/ 5.3	3.86/3.94/ 3.87	3.3/4.4/ 3.2
NR	1	0.79	3.4	0.82	2.9	0.77 (0.78)	3.4 (2.8)	0.82 (0.82)	2.6 (1.9)	0.77/0.78/ 0.78	2.5/4.1/ 4.7	0.82/0.82/ 0.82	1.2/4.6/ 2.9
	2			2.31	2.9	1.98 (2.01)	5.3 (5.2)	2.31 (2.32)	2.9 (3.5)	2.01/2.06/ 1.96	6.6/5.6/ 6.4	2.28/2.30/ 2.29	2.8/1.6/ 3.3
	3			3.91	4.9	3.04 (3.04)	3.3 (3.5)	3.90 (3.94)	5.2 (5.4)	3.07/3.09/ 3.05	2.5/3.0/ 3.8	3.82/3.67/ 3.86	4.8/5.7/ 4.7

* 1st Floor → Roof results shown in parentheses ()

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, structural responses in both directions are dominated by the lower two modes.
- For both directions, the similarity of the fixed- and pseudo flexible-base parameters suggests that rocking did not significantly influence the structural response, which is consistent with the lack of evidence for a rocking response in the vertical B/FF transmissibility function.
- No significant differences are apparent in the spectra of the motions from the three FF instruments (Fig. A33.**2), indicating that FF motions at the site were fairly coherent.
- Pseudo flexible-base parameters were computed using 1F-R pairs so that the difference from B-R parameters could be quantified. The deviation was about 0.01 Hz for first-mode frequency.
- The CEM results for B-R response are consistent with Celebi (1993), who used similar identification techniques in a study of the building's fixed-base seismic response.

No significant time-dependent changes in the first-mode properties of the soil-structure system were apparent from the RPEM analyses.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A33.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 142 feet using the recorded free-field motions and the profile in Fig. A32.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 2.0 and 1.7, respectively, to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A33.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_S=995$ fps	$h=72$ ft	emb.=15 ft	$\gamma = 0.15$
	$\nu=0.4$		tr: $r_1 = 142$ ft	$r_2 = 105$ ft	
WT	$\beta^*=7.3\%$	$V_S^*=795$ fps	L: $r_1 = 142$ ft	$r_2 = 195$ ft	
NR	$\beta^*=4.0\%$	$V_S^*=905$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A33.3 with the fixed-base modal parameters in Table A33.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A33.4.

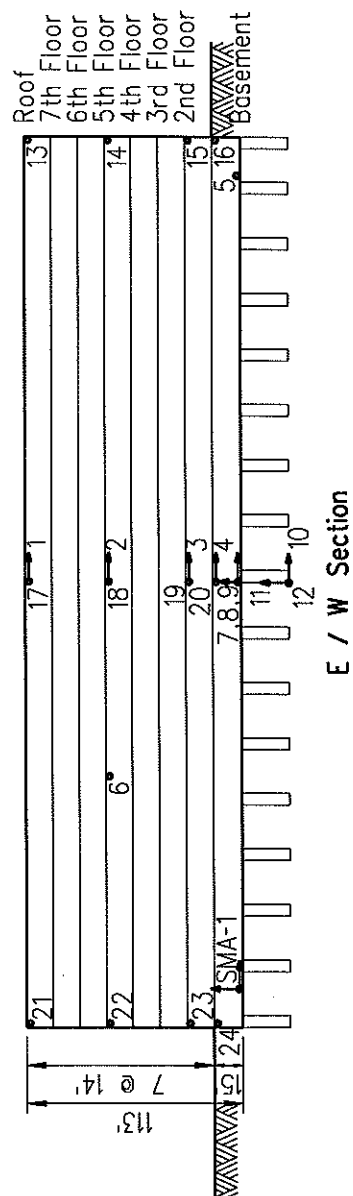
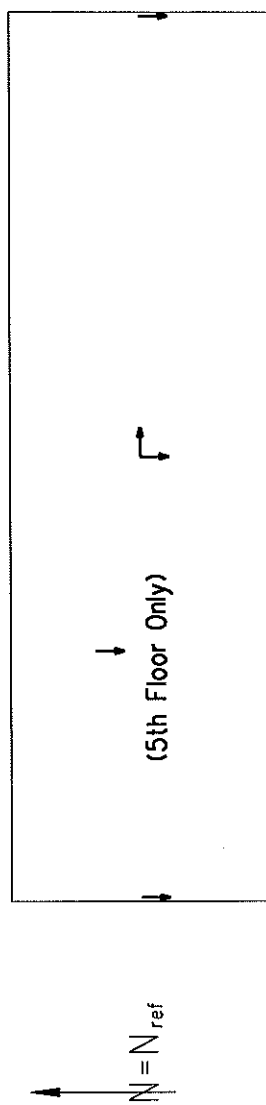
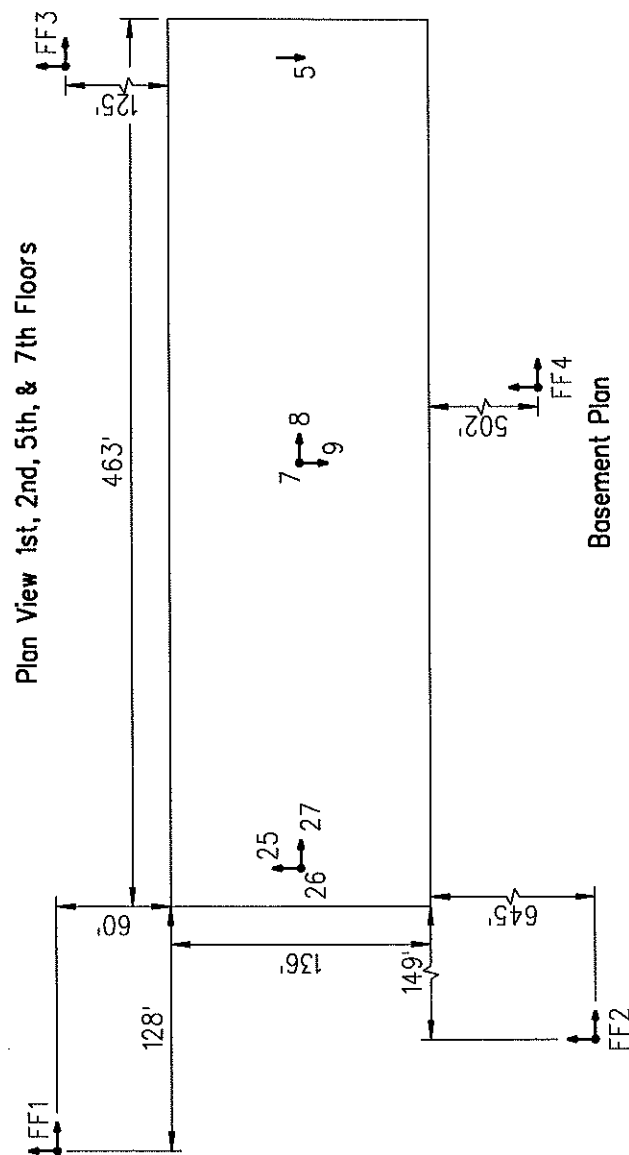
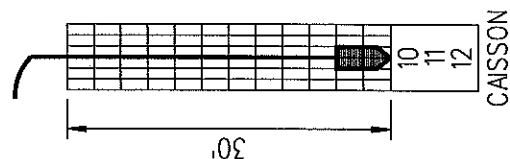
Table A33.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
WT	Sys. ID	1.03	0.6	3.2	0.996	1.5	4.3
	Pre.-Vel.	1.03	0.9	3.4	1.02	1.1	3.8
	Pre.-Bie.	1.02	1.6	4.1	1.03	2.1	4.7
NR	Sys. ID	1.01	0.5	3.8	1.004	0.0	2.9
	Pre.-Vel.	1.02	0.6	3.8	1.02	0.7	3.5
	Pre.-Bie.	1.02	0.9	4.1	1.02	1.3	4.0

The period lengthening and foundation damping were satisfactorily predicted by the “modified Veletsos” formulation. The “modified Bielak” formulation overpredicted the foundation damping.

Data Sources

Celebi, M. (1993). “Seismic response of two adjacent buildings. I: Data and analyses,” *J. Struct. Engrg.*, ASCE, 119(8), 2461-2476.



Note: Data provided by USGS

Sensor Locations

12440 Imperial Highway
Norwalk, CA

Figure A33.1

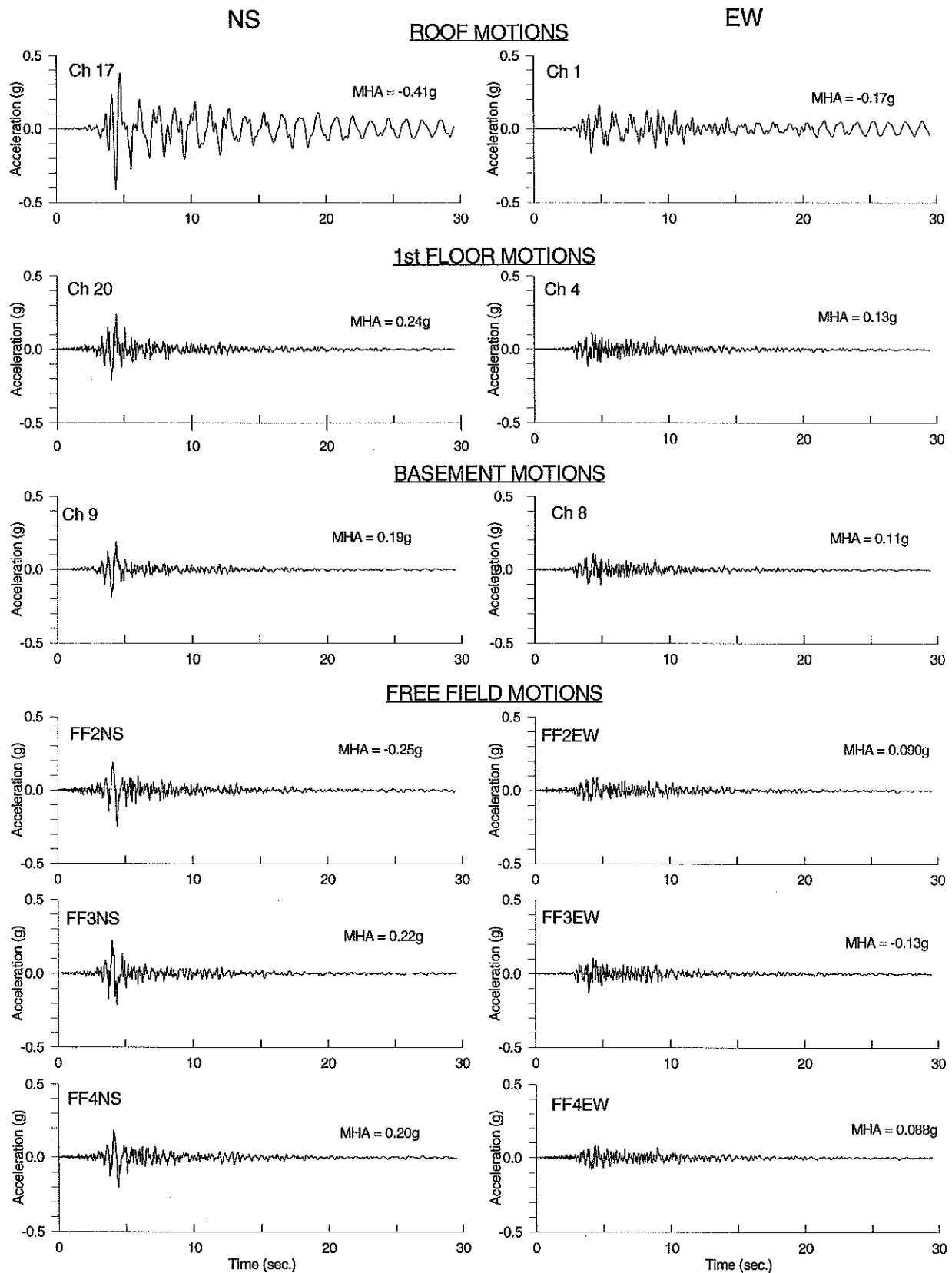


Fig. A33.WT1: NORWALK 12440 IMPERIAL HIGHWAY
Horizontal Acceleration Time Histories
1987 Whittier Earthquake

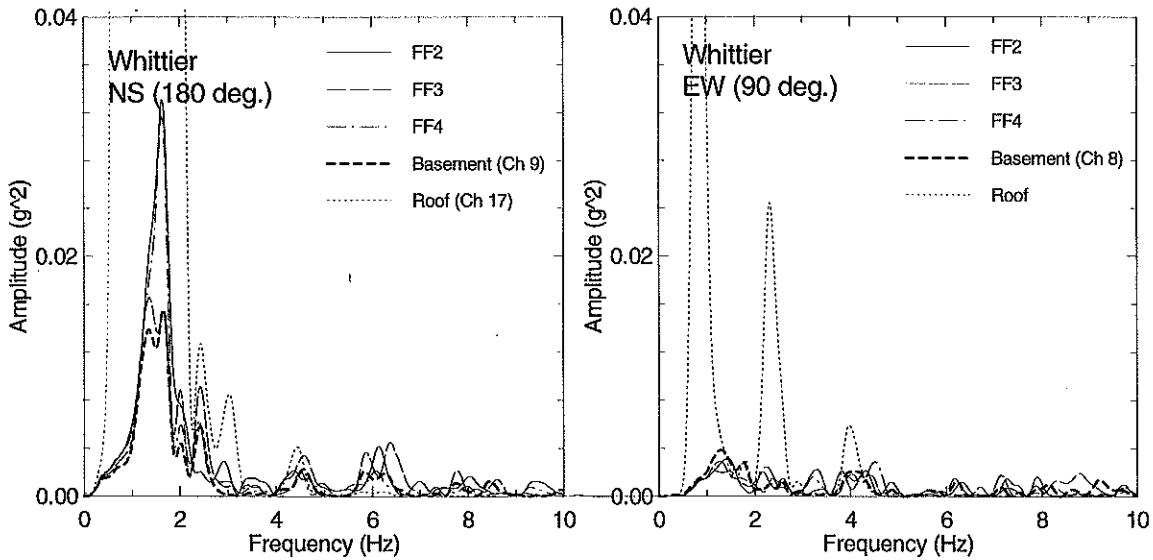


Fig. A33.WT2: NORWALK 12440 IMPERIAL HIGHWAY
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

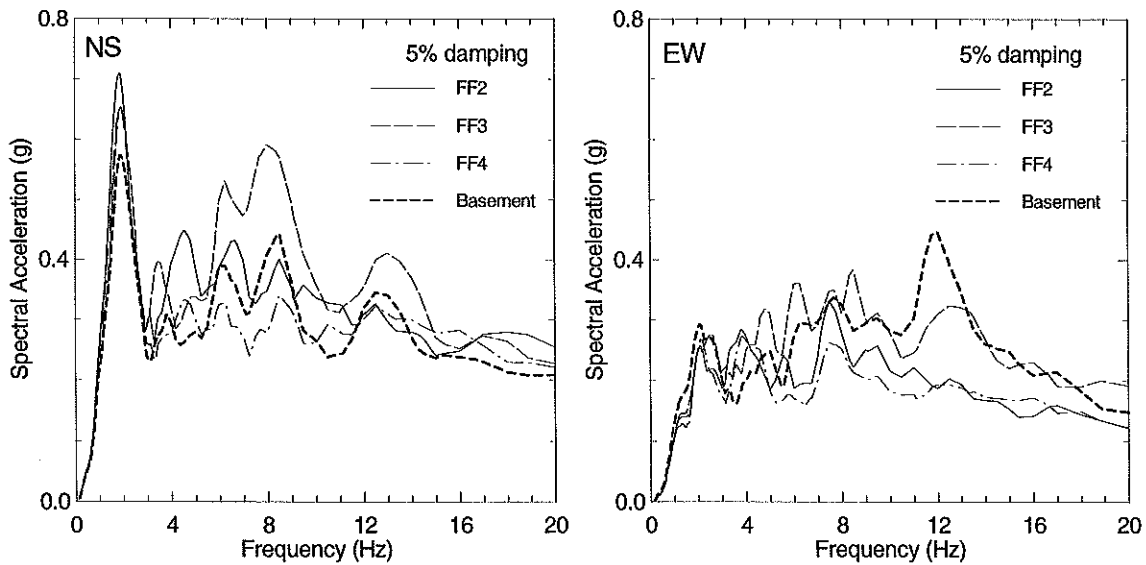


Fig. A33.WT3: NORWALK 12440 IMPERIAL HIGHWAY
Acceleration Response Spectra of Basement and Free-Field Motions
1987 Whittier Earthquake

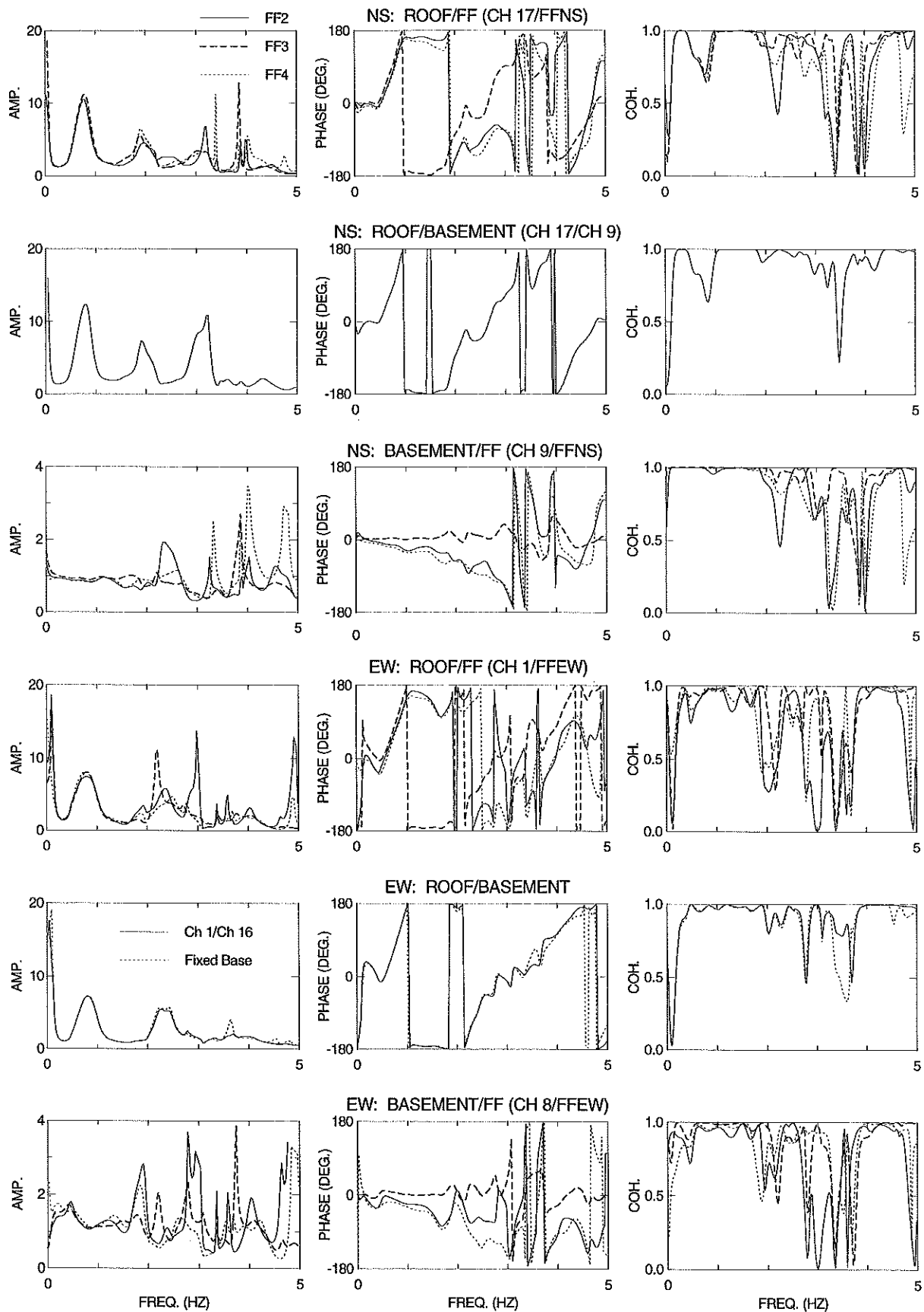


Fig. A33.WT4: NORWALK 12440 IMPERIAL HIGHWAY
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A33-8

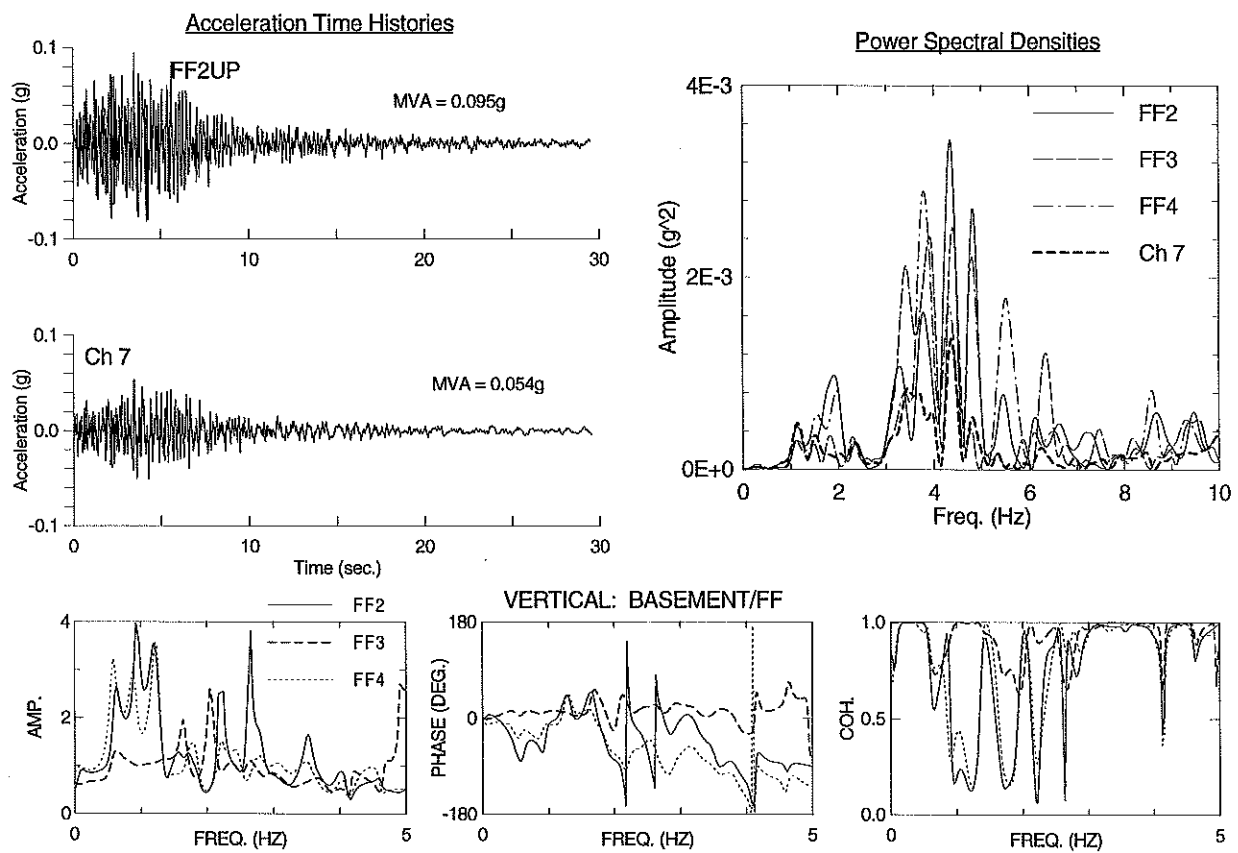


Fig. A33.WT5: NORWALK 12440 IMPERIAL HIGHWAY
Analysis of Vertical Motions
1987 Whittier Earthquake

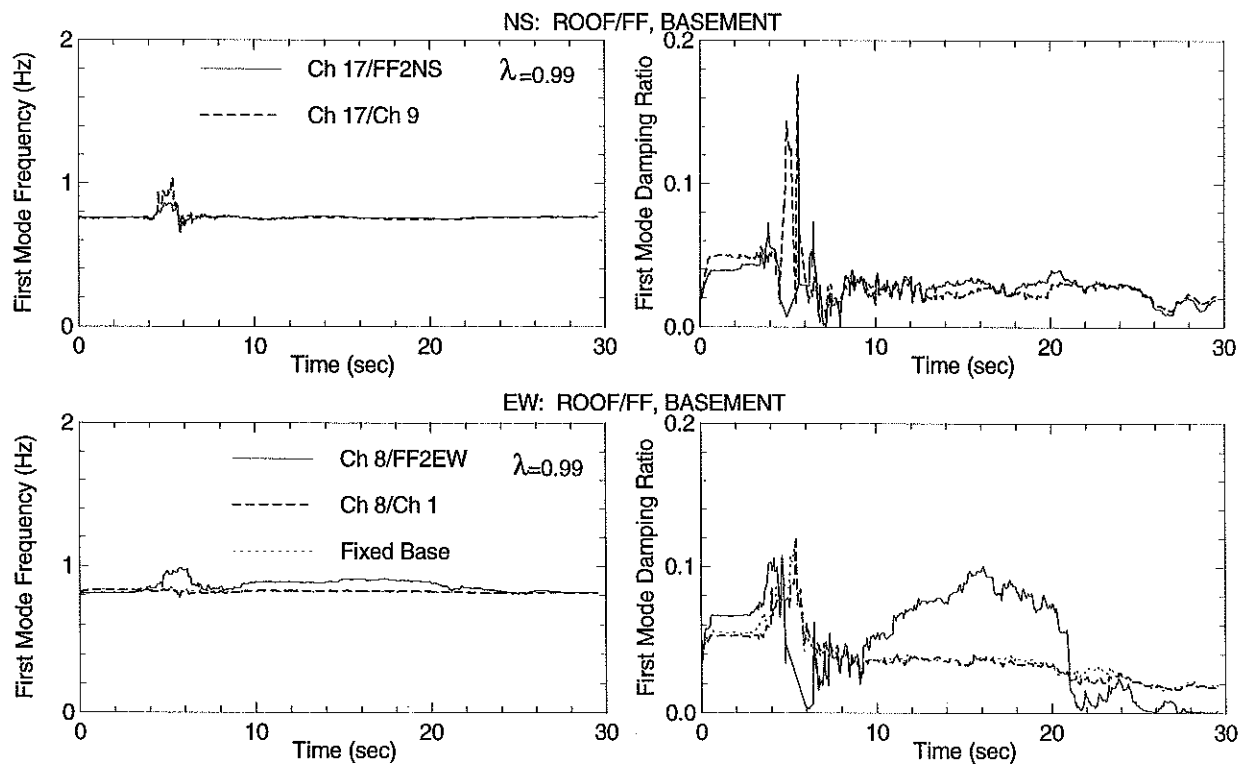


Fig. A33.WT6: NORWALK 12440 IMPERIAL HIGHWAY
Time Variation of First Mode Parameters
1987 Whittier Earthquake

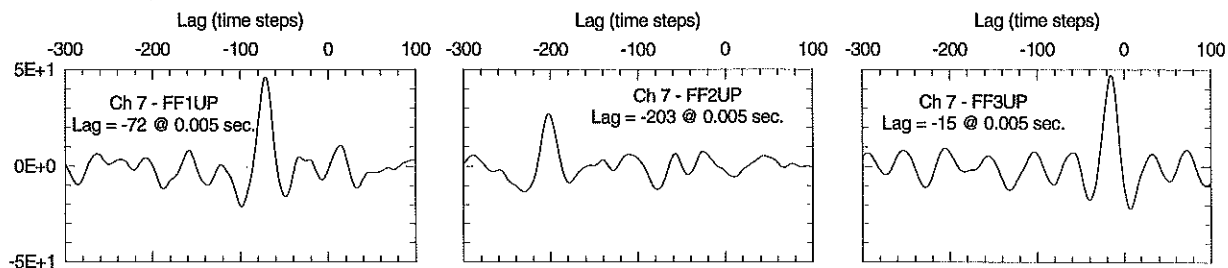


Fig. A33.NR0: Cross-correlation functions between vertical basement and free-field accelerations

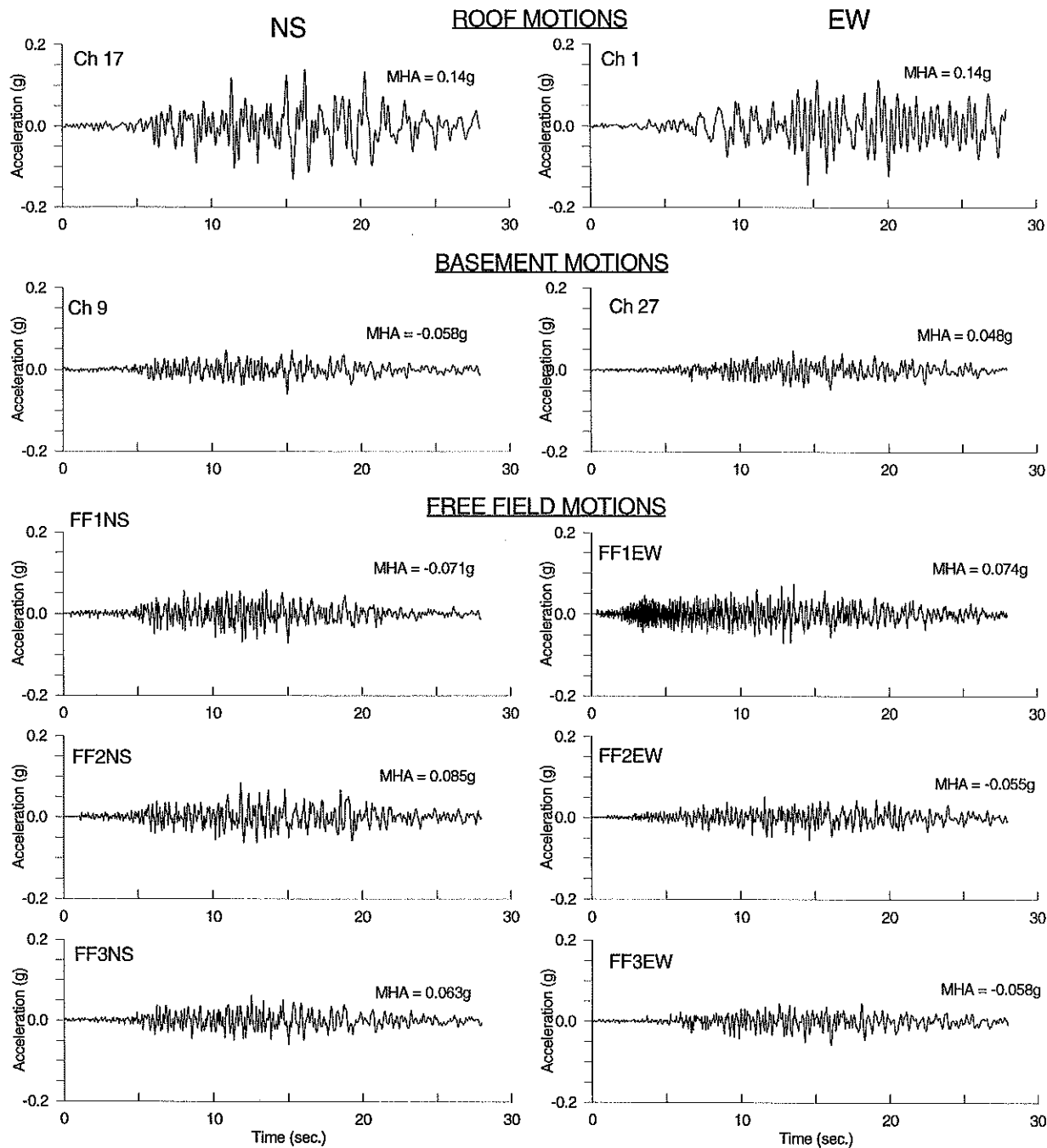


Fig. A33.NR1: NORWALK 12440 IMPERIAL HIGHWAY
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

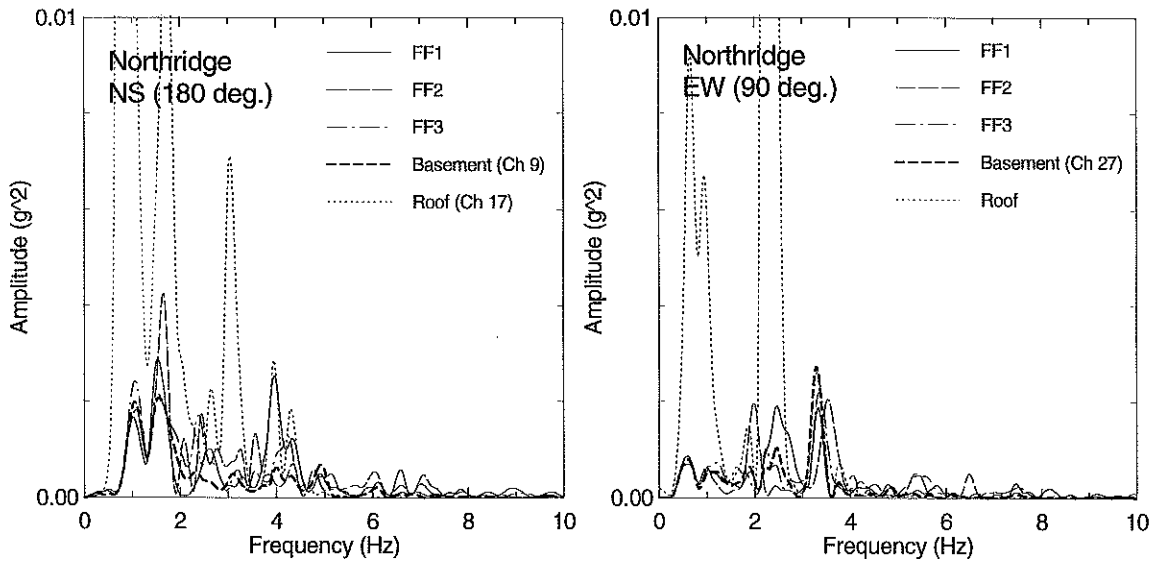


Fig. A33.NR2: NORWALK 12440 IMPERIAL HIGHWAY
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

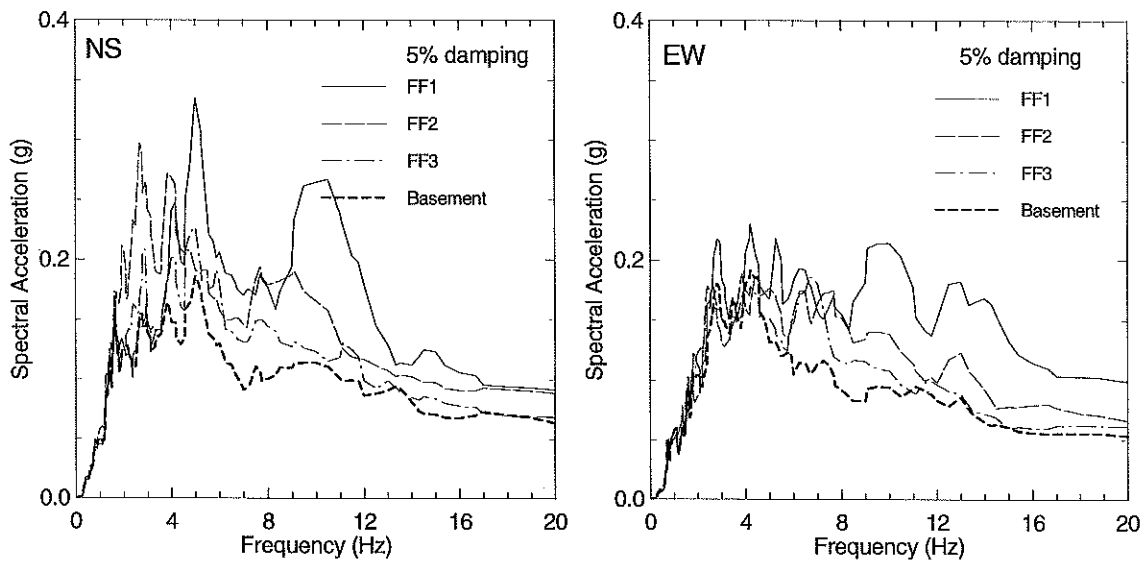


Fig. A33.NR3: NORWALK 12440 IMPERIAL HIGHWAY
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

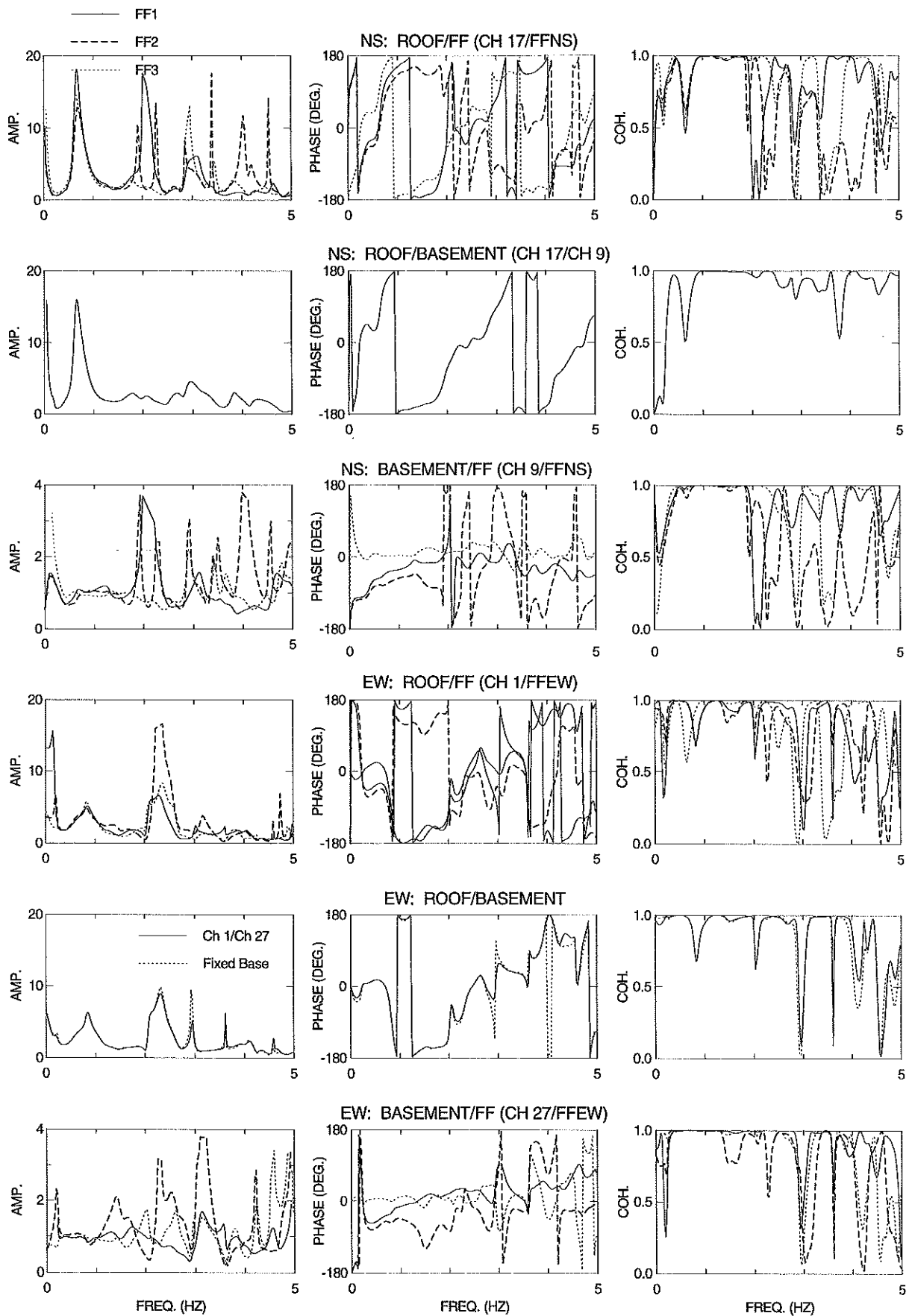


Fig. A33.NR4: NORWALK 12440 IMPERIAL HIGHWAY
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A33-12

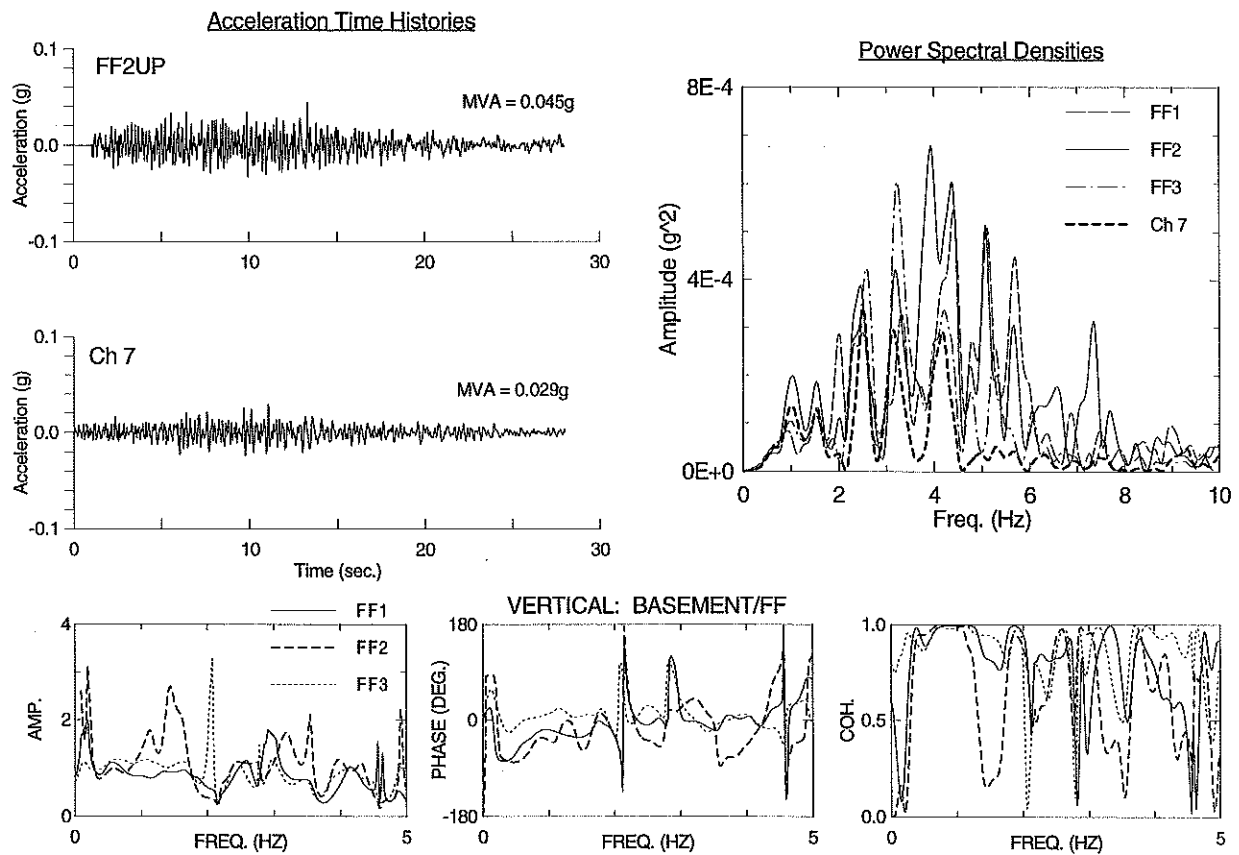


Fig. A33.NR5: NORWALK 12440 IMPERIAL HIGHWAY
Analysis of Vertical Motions
1994 Northridge Earthquake

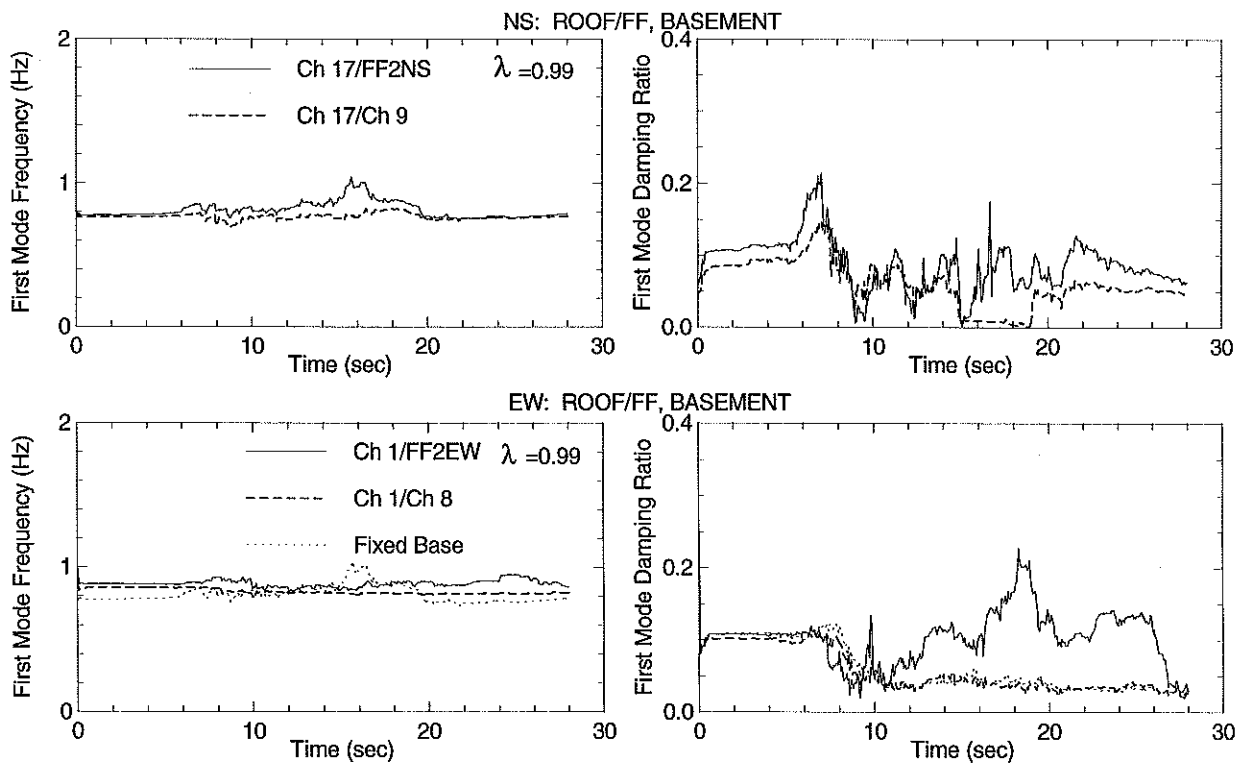


Fig. A33.NR6: NORWALK 12440 IMPERIAL HIGHWAY
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A34

Site Name: Palmdale 4 Story Hotel
Sensors: CSMIP 24232/24521

Lat: 34.581°
Long.: 118.134°

Floor Levels: 3
Bsmt. Levels: 0
Structure: concrete block shear walls

Foundation: footings
Site Geology: Alluvium
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			1st Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers*	0.05	0.05	0.04	0.06	0.06	0.03	0.09	0.13	0.05
1994 Northridge	0.080	0.065	0.040	0.085	0.057	0.057	0.12	0.11	0.050

*Not digitized

Preprocessing

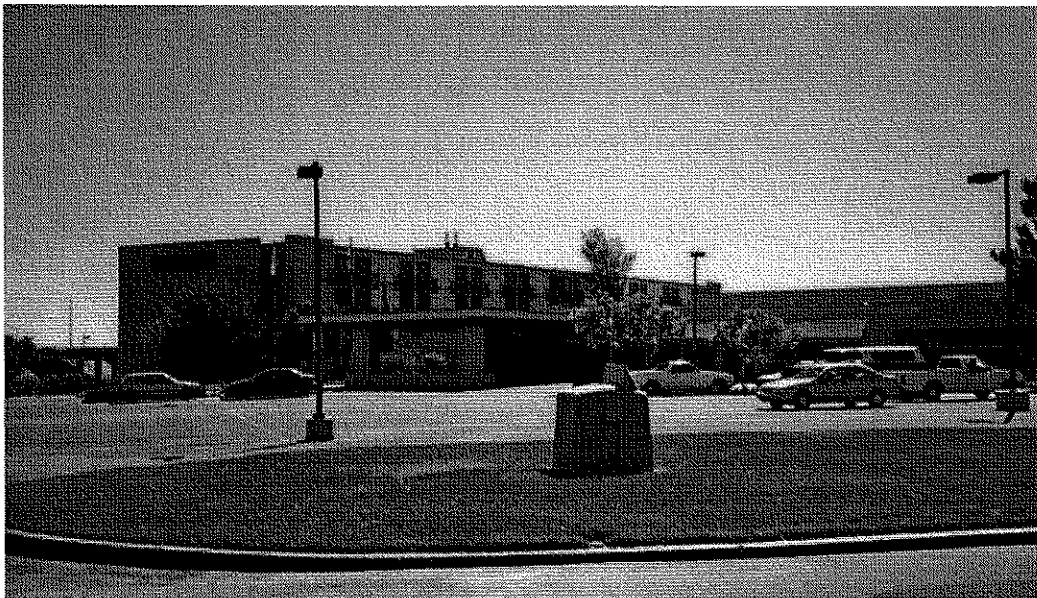
Northridge Earthquake (NR)

Preliminary: *Str. Data*: digitized and processed in this study, $f_{HP} = 0.23-0.40$ Hz, $f_{LP} = 20-30$ Hz, $f_{Nyq} = 100$ Hz. *FF Data*: CSMIP; filtered, $f = 0.2-46$ Hz; $f_{Nyq} = 50$ Hz; FF/str. data not synchronized, FF not aligned with str.

Final: Str. data decimated by 4, FF by 2, to $f_{Nyq} = 25$ Hz; FF data rotated 310° CCW to align with str.; FF/str. data synchronized using cross correlation of vertical accelerations (Fig. A34.NR0).

Figures

A34.1-A34.3: Site Plan, Sensor Locations, Generalized Soil Column
A34.NR0-6 NR Analysis Results



View of structure with FF in foreground

DISCUSSION

Site Description

The 3-level building is about 34.5 feet tall with no basement, and is rectangular in plan (60.7 by 248.7 feet). As shown in Fig. A34.2, lateral loads are resisted by numerous concrete block shear walls in the transverse (NS) direction. The walls are typically founded on 2'-4" deep footings. A 4-inch thick reinforced concrete slab-on-grade is present across the base of the building, and the floors of the building consist of 6-inch thick concrete slabs. The only structure near the hotel is a 1-story building about 25 to 45 feet to the south which is non-structurally connected to the subject building through a breezeway (Fig. A34.1). As shown in Fig. A34.2, accelerographs are present on the 1st, 3rd, and roof levels of the structure. The free-field instrument is located in a parking lot about 240 feet west of the structure.

The site is located on level terrain within the Antelope Valley. A boring drilled at the site by Fumal et al. (1982) encountered sands and silty sands which extended to the maximum depth explored of 100 feet. These soils gradually increase in density with depth, and are quite dense and stiff below about 50 feet. Soil conditions at greater depth and the depth to bedrock are poorly characterized in the area. The shear wave velocities in Fig. A34.3 were measured by Fumal et al. (1982) using downhole techniques.

Instrument Selection

The sensors used in the analyses are located near the building centroid (Table A34.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A34.NR1.

Table A34.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 1	Ch 3	Ch 2	Ch 8	Ch 7

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of 1st Floor and Free-Field Motions

1F/FF transmissibility functions (Fig. A34.NR4) and the spectra of the horizontal motions (Fig. A34.NR2-3) generally indicate amplification of 1F motions relative to the FF for frequencies less than about 5 to 6 Hz, and de-amplification at higher frequencies.

Transmissibility, coherency, and power spectral density functions for FF and 1F vertical motions are presented in Fig. A34.NR5. Amplification occurred at the first-mode frequency for the transverse direction, indicating the base rocking likely contributed to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A34.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A34.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A34.NR6).

Table A34.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=0, j=3		L (EW) d=1, j=3		tr (NS) d=0, j=3		L (EW) d=0, j=3	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	7.52	28.4	6.29	4.7	6.29	19.0	6.20	5.8	4.92	22.4	5.37	15.7
	2					16.3	11.7	18.2	5.1	15.2	9.7	16.8	8.0

Comments on the analytical results follow:

- The building is quite stiff in both directions, with frequencies of $\tilde{f}_1 \approx 4.9$ and 5.4 Hz in the transverse and longitudinal directions, respectively. Power spectra indicate the roof motions are dominated by first-mode structural response and the propagation of low-frequency energy (from the input motions) through the structure.
- As shown in Fig. A34.NR4, the first-mode is associated with low, broad transmissibility function amplitude peaks, meaning that the CEM models are fairly approximate (and sensitive to the value of j used).
- For the transverse direction, the large difference between the fixed- and pseudo flexible-base parameters is consistent with the rocking evident in the vertical 1F/FF transmissibility function. No significant rocking effect is evident in the longitudinal direction.

The RPEM results (Fig. A34.NR6) do not indicate any significant time variation in the first-mode parameters. The RPEM results generally confirm the CEM results with regard to the relative magnitude of the FF-R and 1F-R first-mode frequencies and damping ratios.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A34.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 69 feet using the recorded free-field motions and the profile in Fig. A34.3. The hotel structure was assumed to be independent of the neighboring one-story building for the calculation of foundation radii. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse direction were increased by 70% to account for shape effects.

Table A34.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1615$ fps	$h = 24$ ft	emb.=0	$\gamma = 0.15$
NR	$\nu = 0.33$		tr: $r_1 = 69$ ft		$r_2 = 49$ ft
	$\beta^* = 1.7\%$	$V_s^* = 1575$ fps	L: $r_1 = 69$ ft		$r_2 = 100$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A34.3 with the fixed-base modal parameters in Table A34.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A34.4.

Table A34.4: Empirical and predicted inertial interaction effects

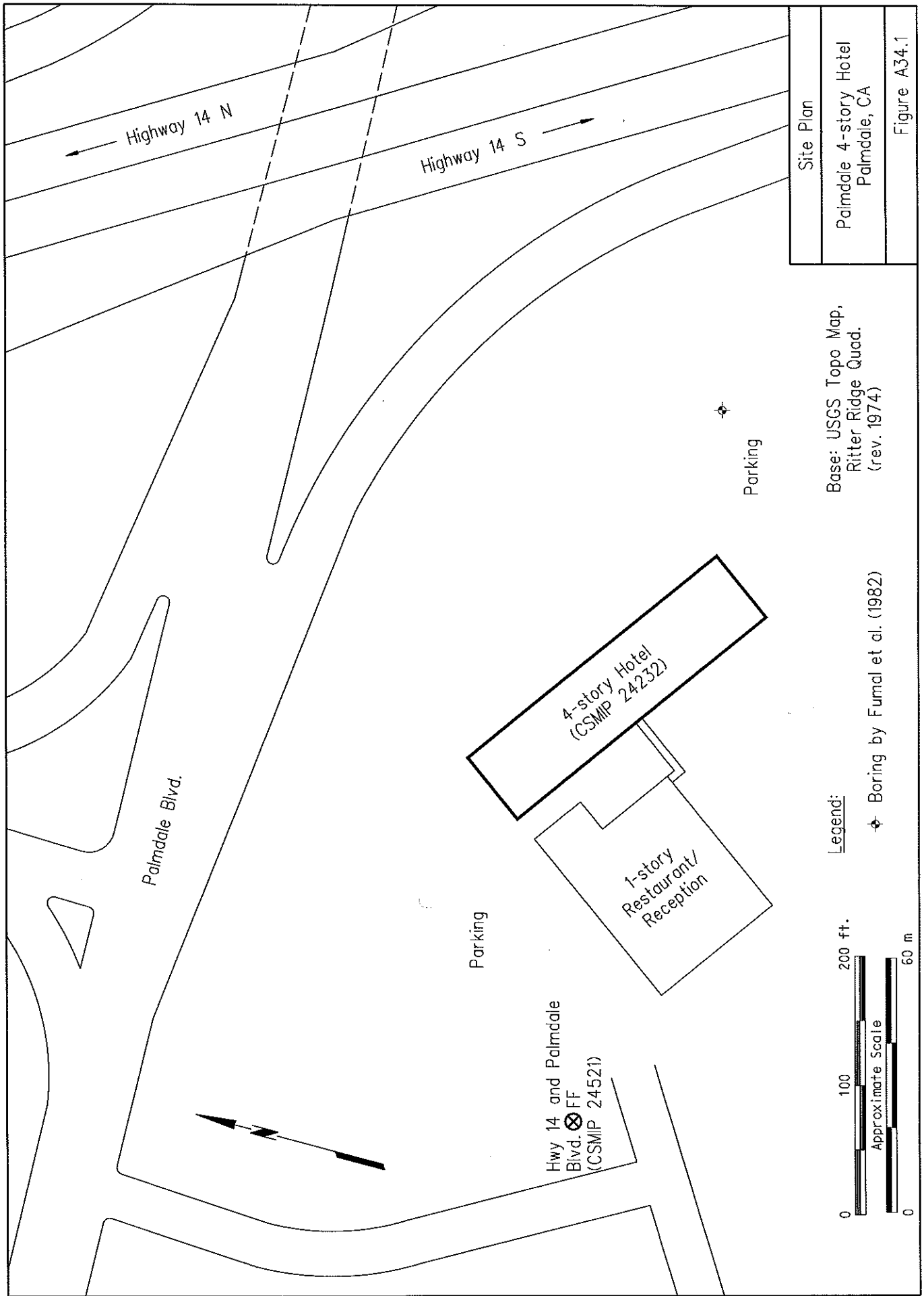
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.53	14.4	22.4	1.17	12.8	15.7
	Prediction	1.11	4.9	25.8	1.06	4.0	8.0

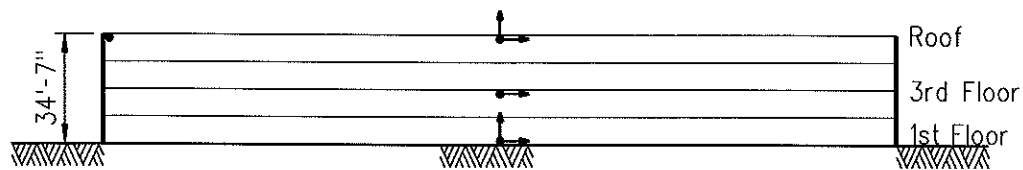
The period lengthening and foundation damping are significantly underpredicted for this structure. The most likely cause of this discrepancy is shear wave velocities which are too high. Generally, when both shear and compression wave velocities are measured, V_s is found to be about one-half of V_p in unsaturated cohesionless soils (implying that $\nu \approx 0.33$). For this site, however, V_s/V_p ratios were in the range of about 0.7 to 0.8, which is not possible, as this condition would require $\nu > 0.5$ for the sandy soils at the site. Hence, there is an error in the measured velocities, but it is not known if the error is in V_s or V_p (Gibbs, 1996). Thus, a conclusive determination as to the source of the unusual results at site A34 cannot be made. However, it is interesting to note that if V_s is lowered so that the $V_s/V_p = 0.5$, $1/\sigma$ increases to about 0.16, and $\tilde{T}/T$ and $\tilde{\zeta}_0$ predictions are increased such that the absolute differences with empirical values are about -0.3 and -5% in the transverse direction and -0.05 and -4% in the longitudinal direction. Such underpredictions would not be particularly significant relative to results for other sites.

Data Sources

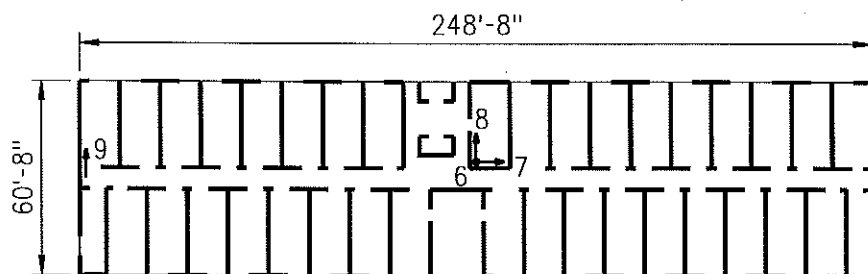
Fumal, T.E., Gibbs, J.F., and Roth, E.F. (1982). "In-situ measurements of seismic velocity at 22 locations in the Los Angeles, California Region," *Open File Rpt. 82-833*, U.S. Geological Survey.

Gibbs, J.F. (1996). pers. communication.

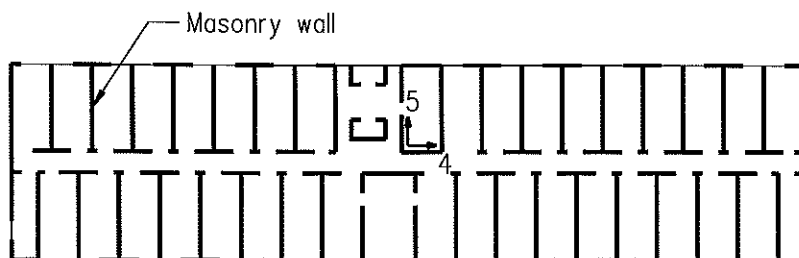




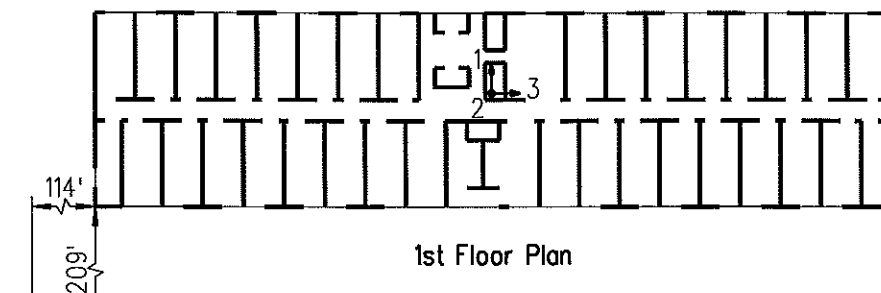
W / E Section



Roof Plan

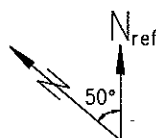


3rd Floor Plan



1st Floor Plan

Hwy 14 and
Palmdale Blvd.
Free Field

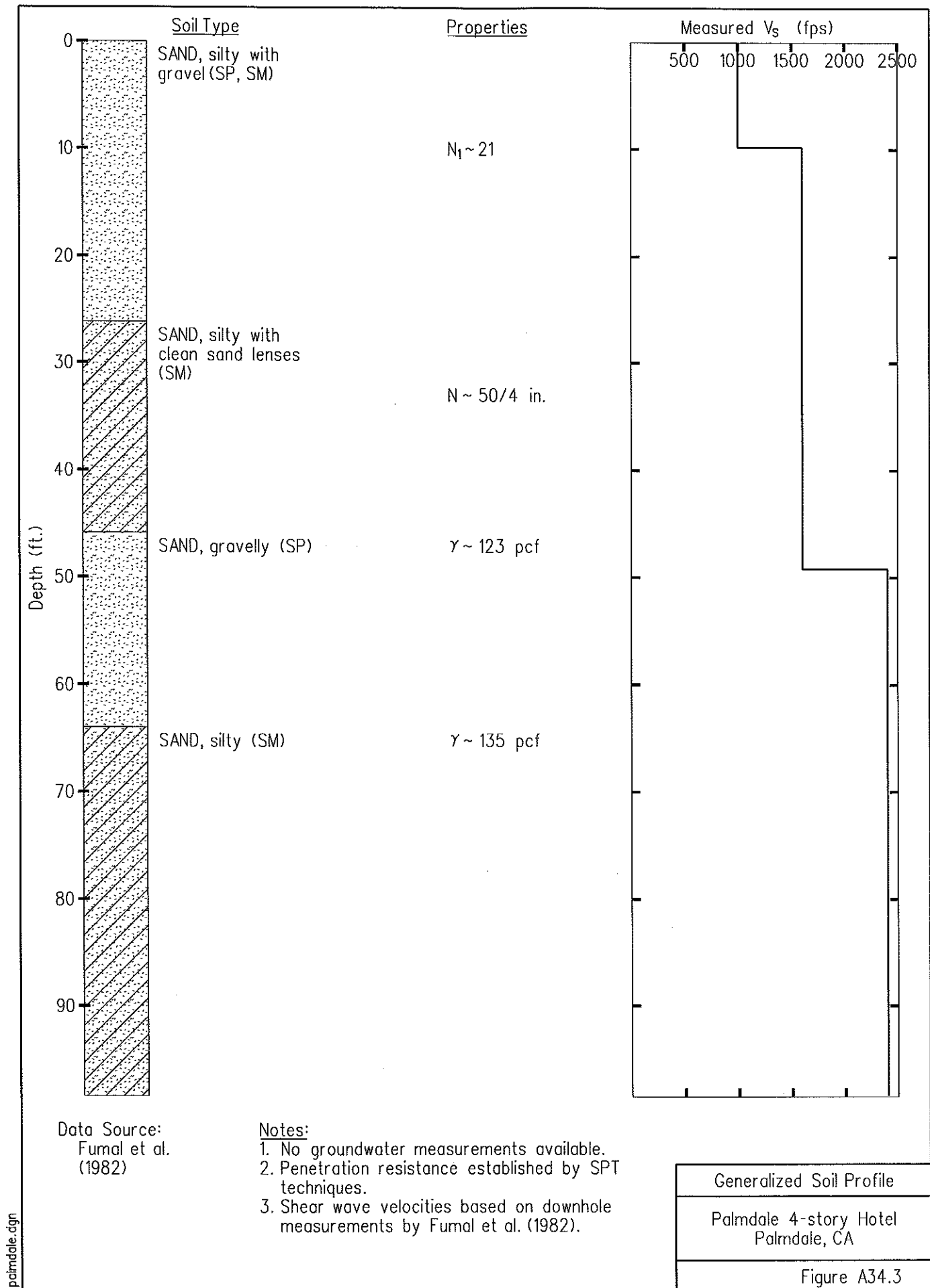


Note:

Accelerometers 1, 2 & 3
mounted on first floor slab.
Accelerometers 4 & 5
attached to underside of
3rd floor slab. Accelerometer
6 mounted on masonry wall
at roof level, directly above
accelerometer 2.
Accelerometers 7, 8 & 9
attached to underside of
roof slab.

Note: Data provided by CSMIP

Sensor Locations
Palmdale 4-story Hotel Palmdale, CA
Figure A34.2



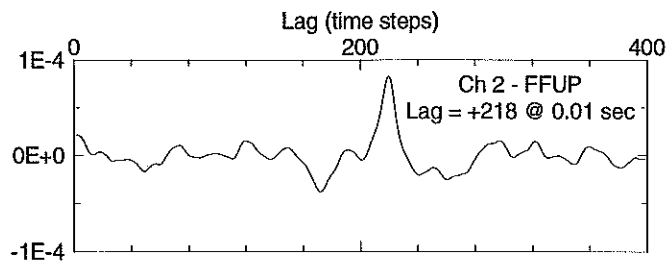


Fig. A34.NR0: Cross-correlation function between vertical 1st floor and free-field accelerations

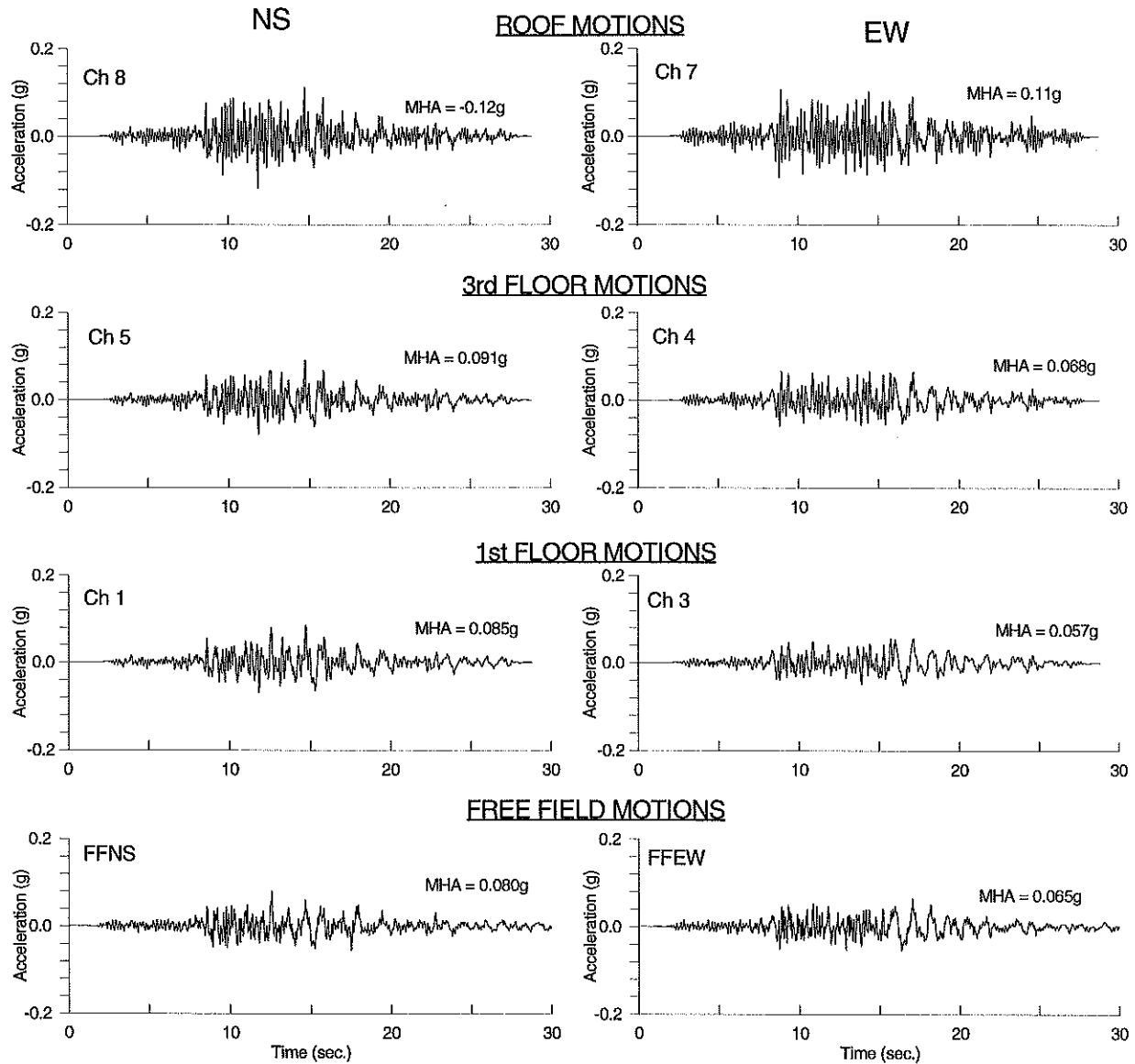


Fig. A34.NR1: PALMDALE 4 STORY HOTEL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

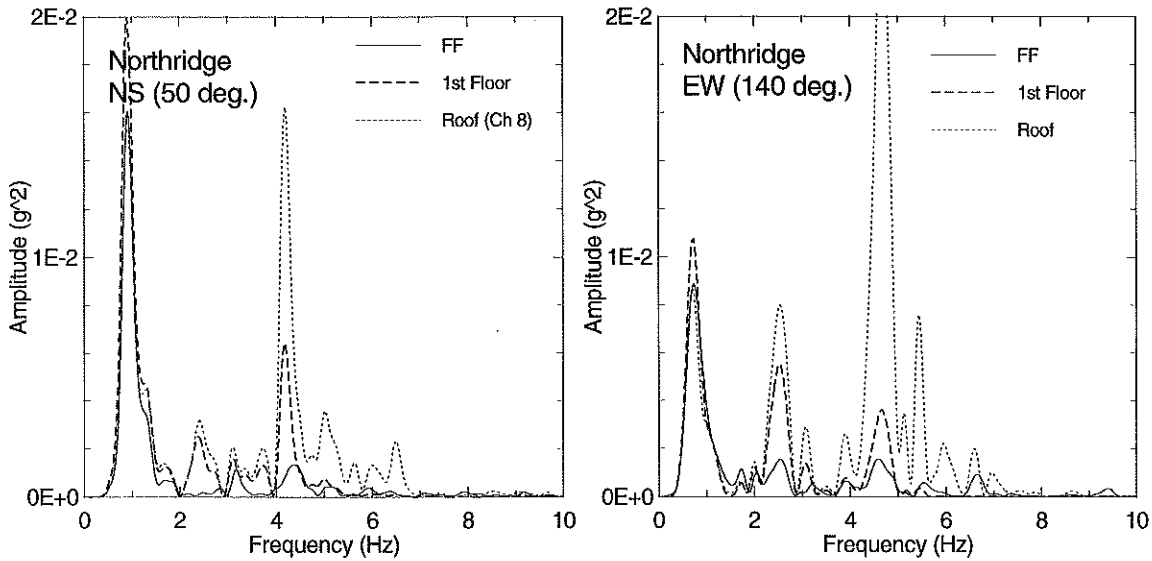


Fig. A34.NR2: PALMDALE 4 STORY HOTEL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

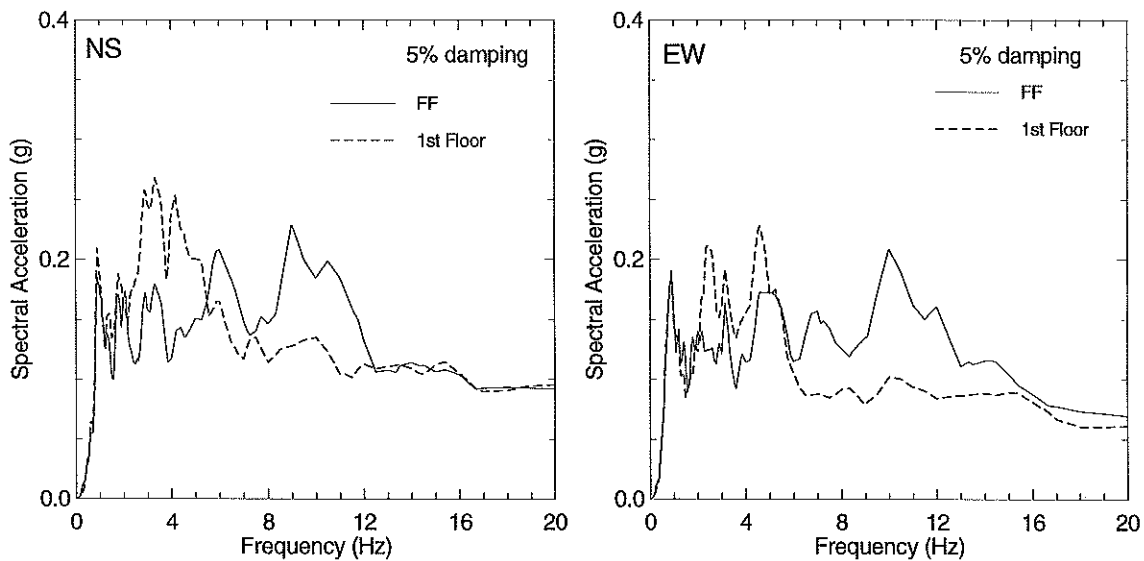


Fig. A34.NR3: PALMDALE 4 STORY HOTEL
Acceleration Response Spectra of 1st Floor and Free-Field Motions
1994 Northridge Earthquake

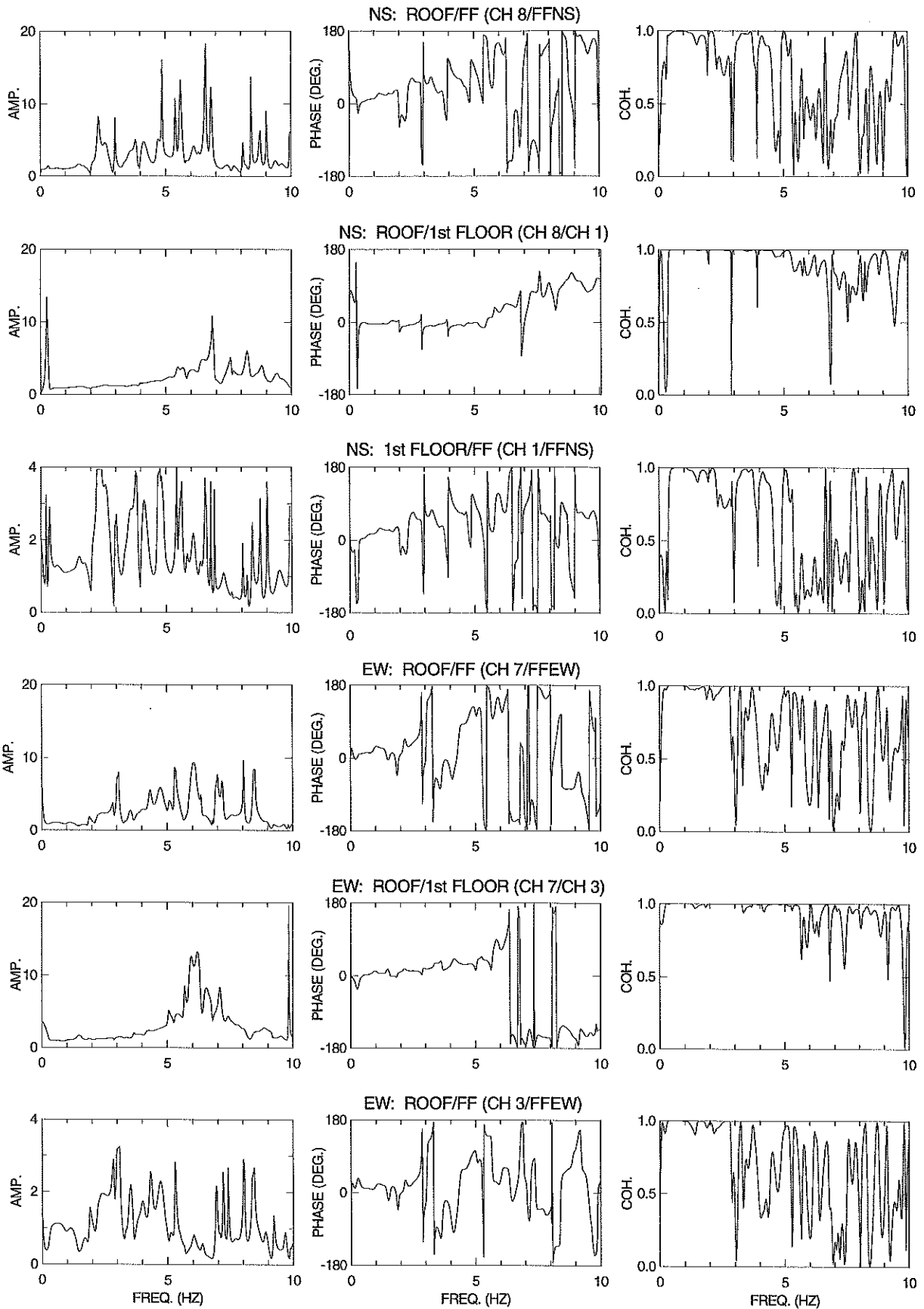


Fig. A34.NR4: PALMDALE 4 STORY HOTEL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A34-10

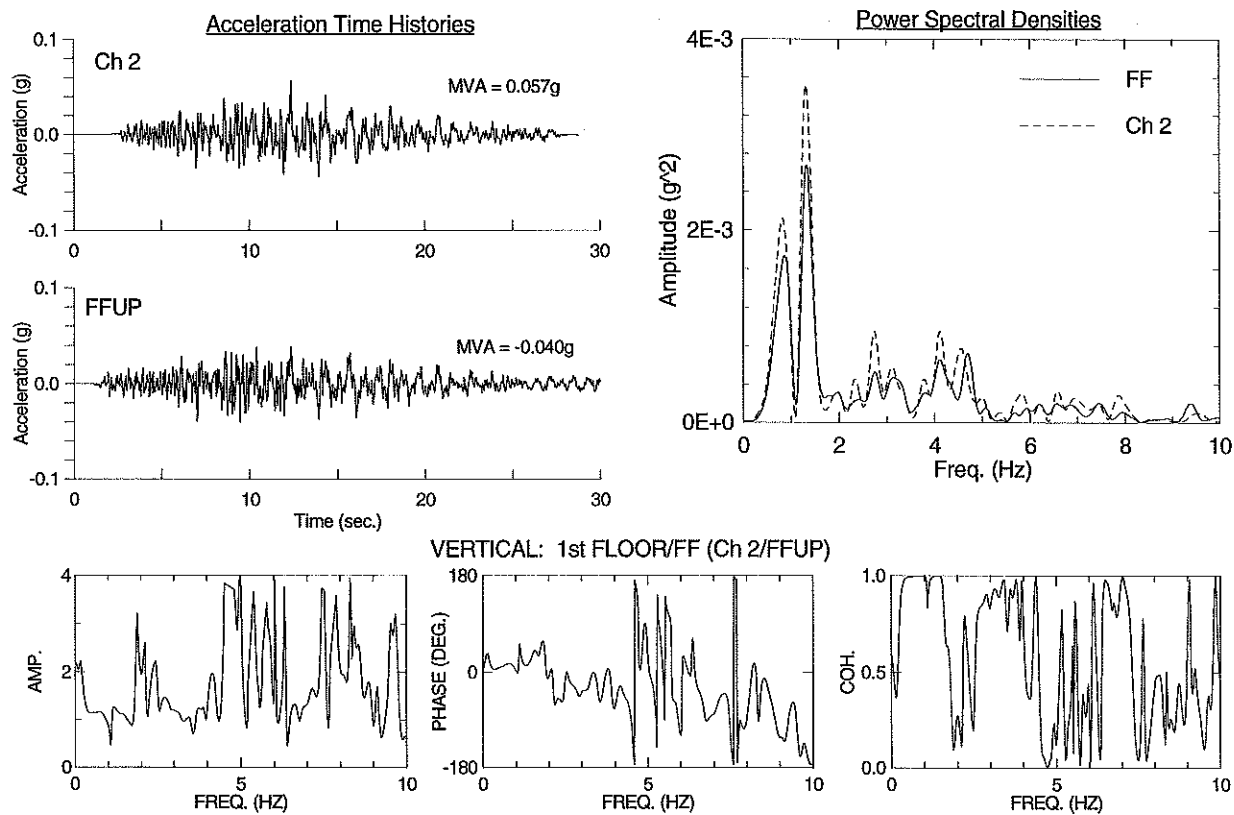


Fig. A34.NR5: PALMDALE 4 STORY HOTEL
Analysis of Vertical Motions
1994 Northridge Earthquake

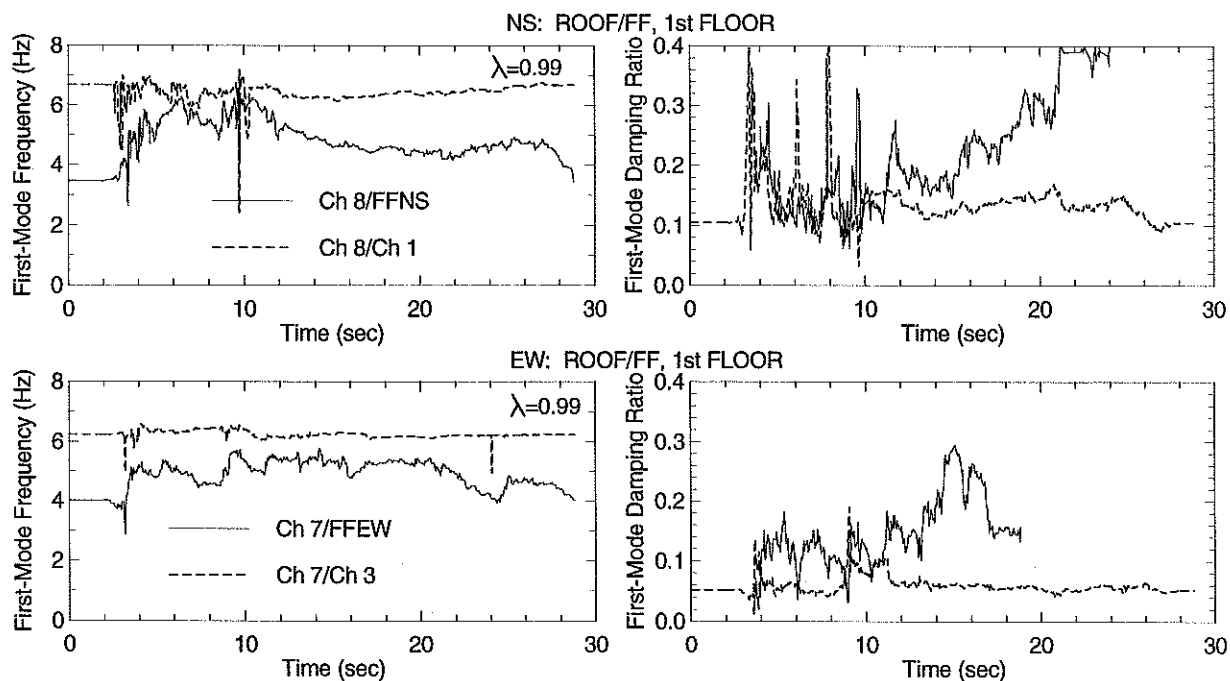


Fig. A34.NR6: PALMDALE 4 STORY HOTEL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A35

Site Name: Pomona 2 Story Building
Sensors: CSMIP 23511/23525

Lat: 34.054°
Long.: 117.749°

Floor Levels: 3
Bsmt. Levels: 1
Structure: Ductile concrete moment-resisting frames

Foundation: piles
Site Geology: Alluvium
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	0.064	0.057	0.041	0.050	0.046	0.024	0.15	0.10	--
1990 Upland	0.19	0.21	0.097	0.13	0.12	0.075	0.39	0.24	--
1992 Landers*	0.067	0.044	0.035	0.06	0.05	0.04	0.16	0.14	--
1992 Big Bear*	0.03	0.03	0.03	0.02	0.02	0.02	0.06	0.08	--
1994 Northridge*	0.05	0.05	0.04	0.05	0.05	0.04	0.15	0.22	--

*Not digitized

-- = No instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.8-23$ Hz; $f_{Nyq}=25$ Hz; FF/str. not synchronized; FF oriented 12° clockwise from str. reference north.

Final: decimated by 2 to $f_{Nyq}=12.5$ Hz, FF data rotated 12° CCW, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A35.WT0).

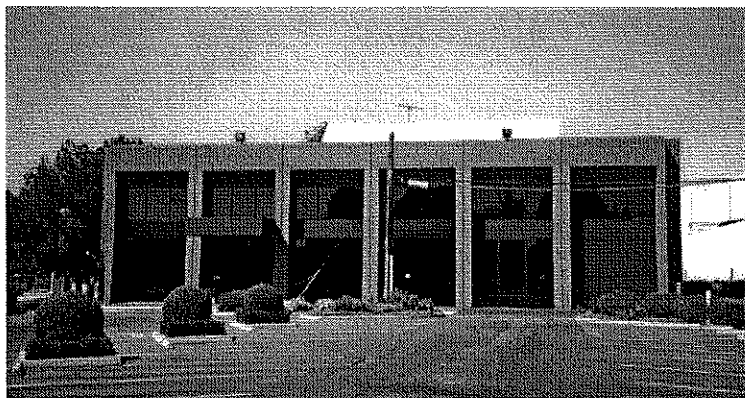
Upland Earthquake (UP)

Preliminary: CSMIP; filtered, $f = 0.5-23$ Hz; $f_{Nyq}=25$ Hz; FF/str. data synchronized.

Final: none

Figures

A35.1-A35.3: Site Plan, Sensor Locations, Generalized Soil Column
A35.WT0-6: WT Analysis Results
A35.UP1-6: UP Analysis Results



View of structure, looking west

DISCUSSION

Site Description

The building is 2-stories tall with a single level basement at its east end, and is rectangular in plan (96 by 115 feet). No large structures are located near the building or free-field instrument. As shown in Fig. A35.2, the lateral load resisting system consists of reinforced concrete moment frames in both directions. The building is supported on piles generally 24 inches in diameter, 28 to 36 feet deep, and installed in groups of 2 and 4. A 4-inch thick slab-on-grade is present across the base of the structure. As shown in Fig. A35.2, accelerographs are present in the basement, 2nd, and 3rd (roof) levels of the structure. The free-field instrument is located in a parking lot about 285 feet east of the east basement wall.

The site is located on an alluvial plain south of the San Bernardino mountains. Based on foundation investigations by LCA (1963, 1970), surficial soil conditions in the area generally consist of silty and gravely sands which extend to the maximum depth explored of 60 feet. The shear wave velocities shown in Fig. A35.3 were estimated to a depth of 60 feet using correlations with void ratio. Soil conditions at greater depth, and the depth to bedrock, are poorly characterized in the area.

Instrument Selection and Free-Field Conditions

Accelerographs at the south wall near the middle of the building in the EW direction were used in the analyses, as indicated in Table A35.1. Acceleration time histories recorded by these sensors are shown in Fig. A35.**1.

Table A35.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 10	Ch 6	Ch 1	Ch 8	Ch 2

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A35.**4) and the spectra for horizontal accelerations (Fig. A35.**2-3) generally indicate de-amplification of basement motions relative to the FF for frequencies greater than about 3 to 4 Hz. No significant amplification of basement motions occurred at smaller frequencies.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Fig. A35.**5. No significant amplification of vertical basement

accelerations occurred at the first-mode lateral response frequency, suggesting that base rocking was unlikely to have significantly affected the structural response. It should be noted, however, that the vertical basement instrument is not well positioned to detect motions from base rocking.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A35.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A35.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A35.**6).

Table A35.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (EW)		L (NS)		tr (EW)		L (NS)		tr (EW)		L (NS)	
		WT: estimated	UP: estimated	estimated	estimated	d=2, j=2	d=2, j=2	d=2, j=2	d=3, j=2	d=0, j=2	d=1, j=2	d=0, j=2	d=0, j=2
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	4.08	6.8	3.95	8.9	4.04	6.7	3.91	8.6	3.93	8.7	3.74	5.8
UP	1	3.50	6.7	3.49	11.7	3.46	6.7	3.44	11.3	3.33	9.3	3.24	11.4

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The small variation between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for a significant rocking response in the vertical B/FF transmissibility function.
- The structure had a stiffer response in WT than UP events, due to the higher amplitude shaking in the UP event.

No significant time-dependent changes in the first-mode parameters of the structure or soil-structure system were apparent from the RPEM analyses for either earthquake (Fig. A35.**6). The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and B-R first-mode frequencies and damping ratios.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A35.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 59 feet using the recorded free-field motions and the profile in Fig. A35.3. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A35.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1280$ fps	$h=28$ ft	$emb.=10.5$ ft	$\gamma = 0.15$
WT	$\nu=0.33$		tr:	$r_1 = 59$ ft	$r_2 = 57$ ft
	$\beta^*=1.6\%$	$V_s^*=1250$ fps	L:	$r_1 = 59$ ft	$r_2 = 63$ ft
	$\beta^*=3.2\%$	$V_s^*=1180$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A35.3 with the fixed-base modal parameters in Table A35.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A35.4.

Table A35.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
WT	Sys. ID	1.04	2.7	8.7	1.06	-1.8	5.8
	Pre.-Vel.	1.04	1.7	7.7	1.04	1.6	9.6
	Pre.-Bie.	1.04	3.4	9.4	1.04	3.4	11.3
UP	Sys. ID	1.05	3.5	9.3	1.08	2.1	11.4
	Pre.-Vel.	1.03	1.3	7.5	1.03	1.4	12.1
	Pre.-Bie.	1.04	2.7	8.8	1.04	2.9	13.5

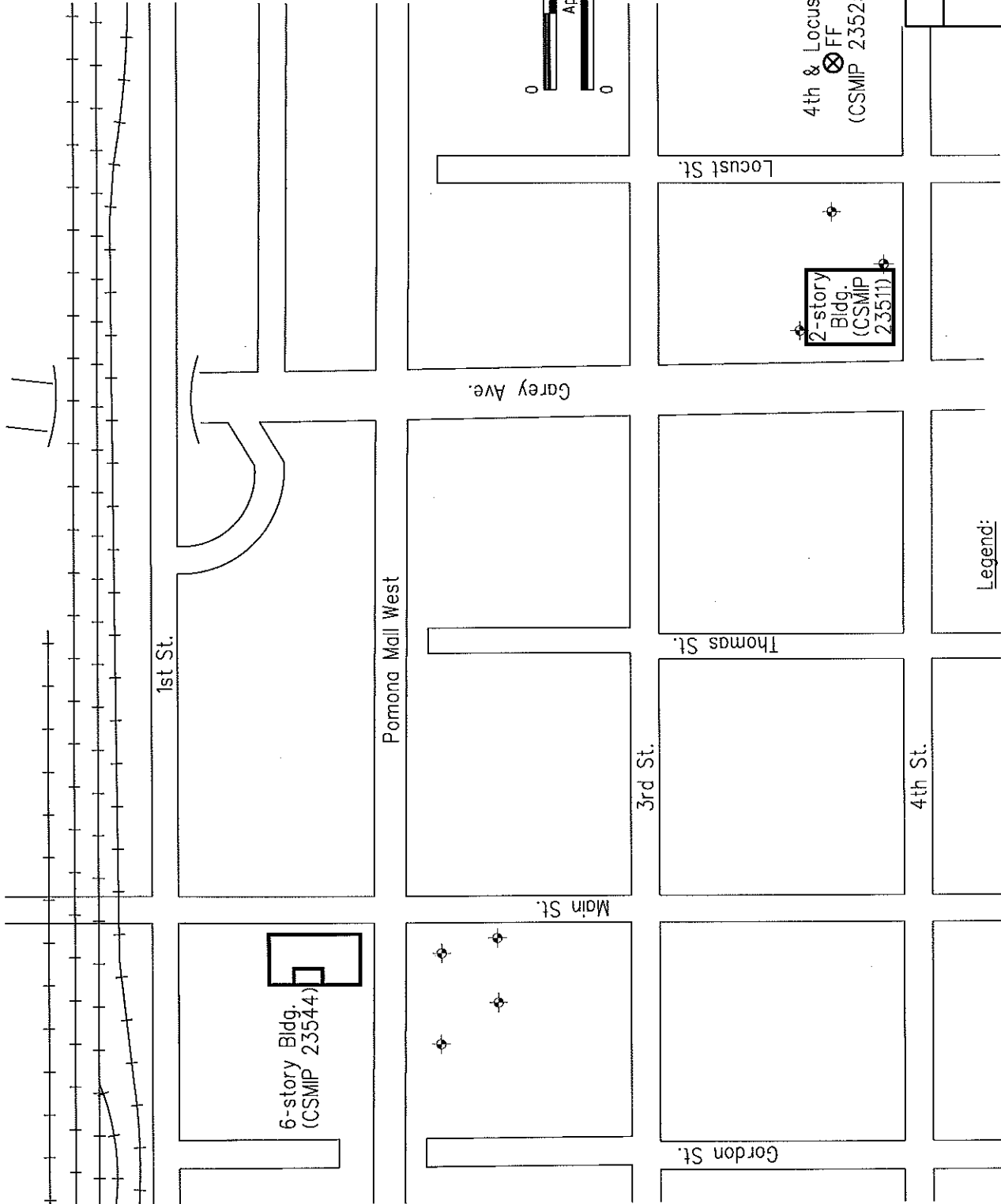
The period lengthening was satisfactorily predicted by both formulations. Transverse foundation damping was predicted better by the "modified Veletsos" formulation, whereas longitudinal foundation damping was better by the "modified Bielak" formulation. The negative foundation damping from the system identification analyses for the WT event in the longitudinal direction indicates a numerical error in these results.

Data Sources

LeRoy Crandall Assoc., LCA (1970). "Report of Foundation Investigation, Proposed Bank Building, Fourth Street and Garey Avenue, Pomona, CA," *Job. No. A-70018*.

LeRoy Crandall Assoc., LCA (1963). "Report of Foundation Investigation, Proposed Building, 300 Pomona Mall West, Pomona, CA," *Job. No. 63630*.

Base: USGS Topo Maps,
San Dimas Quad. (rev.
1981) and Ontario Quad.
(rev. 1981); Plot Plans
LCA (1963, 1970)

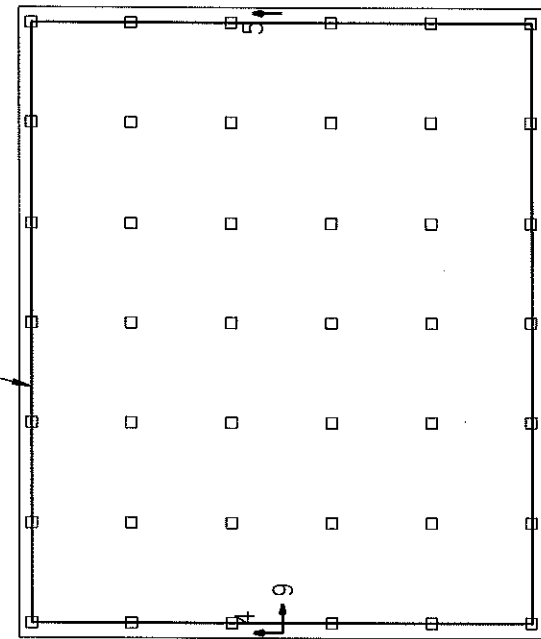


Site Plan

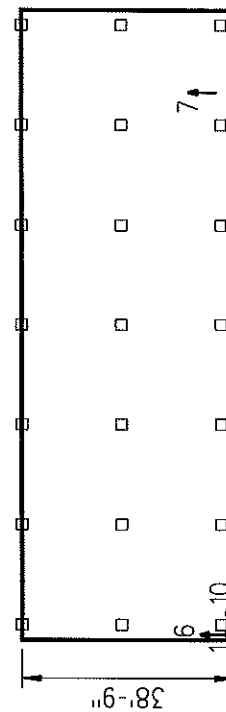
2- and 6-story Buildings
Pomona, CA

Figure A35.1

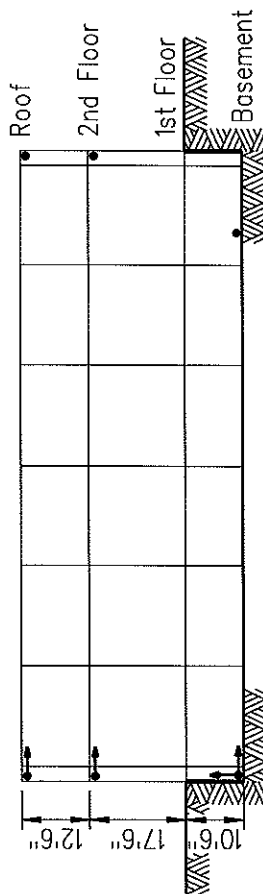
Ductile concrete moment frame



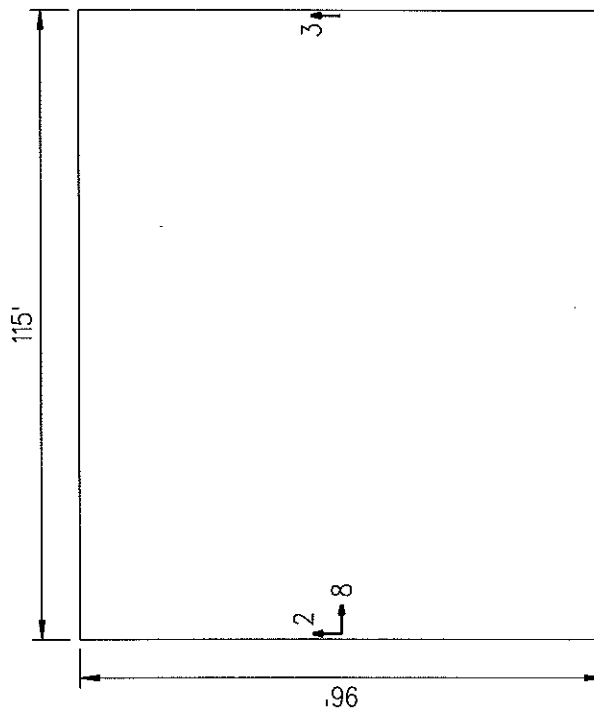
2nd Floor Plan



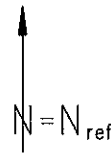
Basement Plan



S / N Elevation



Roof Plan



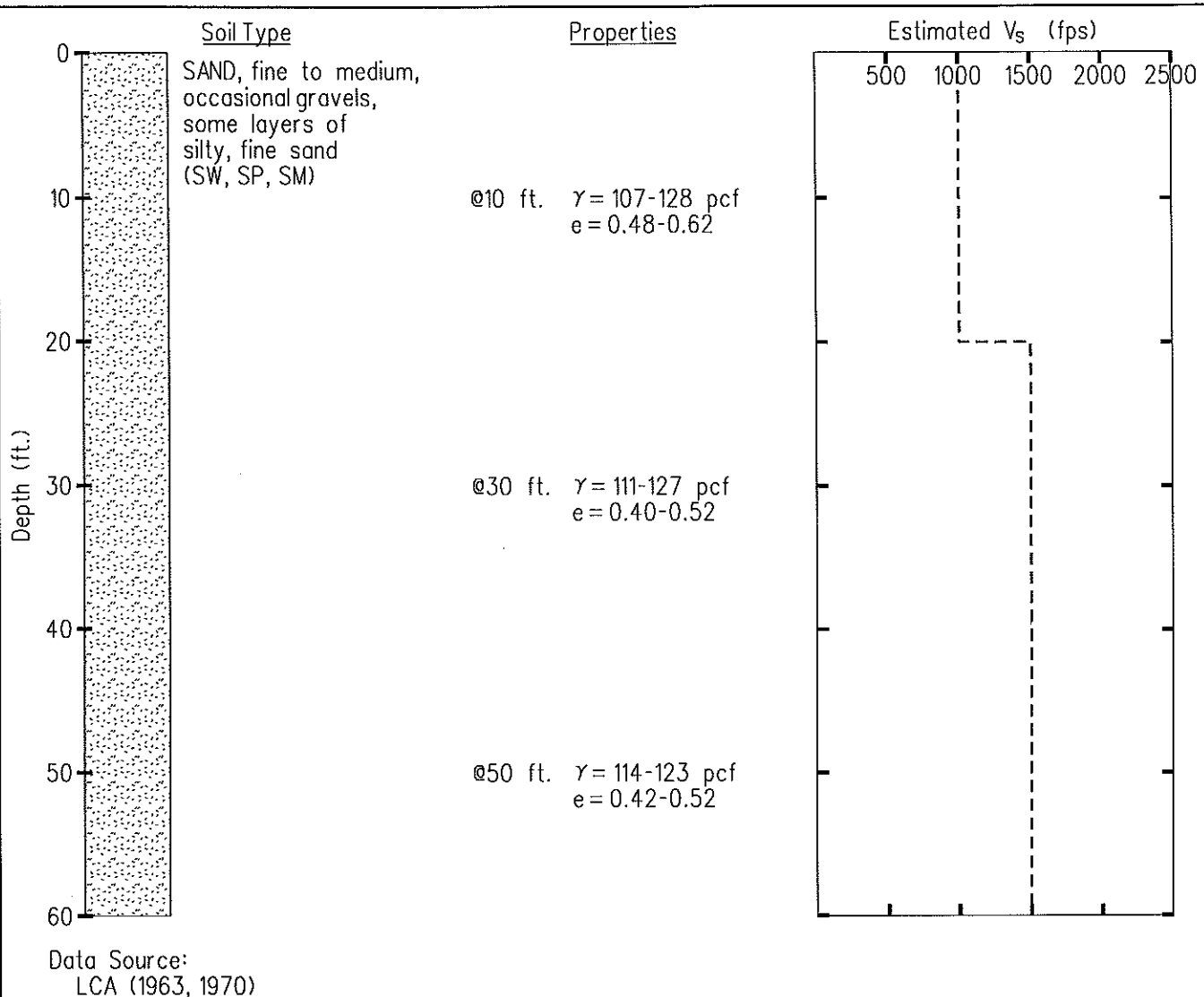
4th and Locust
Free Field

Sensor Locations
2-story Commercial Building Pomona, CA
Figure A35.2

Perimeter ductile concrete moment frames.
Design date: 1971

Note: Data provided by CSMIP

pomona.dgn



Notes:

1. No groundwater encountered.
2. Shear wave velocities estimated based on correlations by Fumai & Tinsley (1985).

Generalized Soil Profile
2- and 6-story Buildings Pomona, CA
Figure A35.3

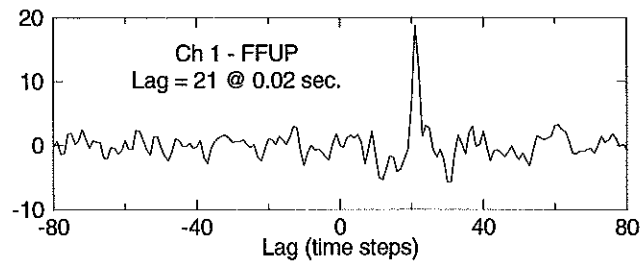


Fig. A35.WT0: Cross-correlation function between vertical basement and free-field accelerations

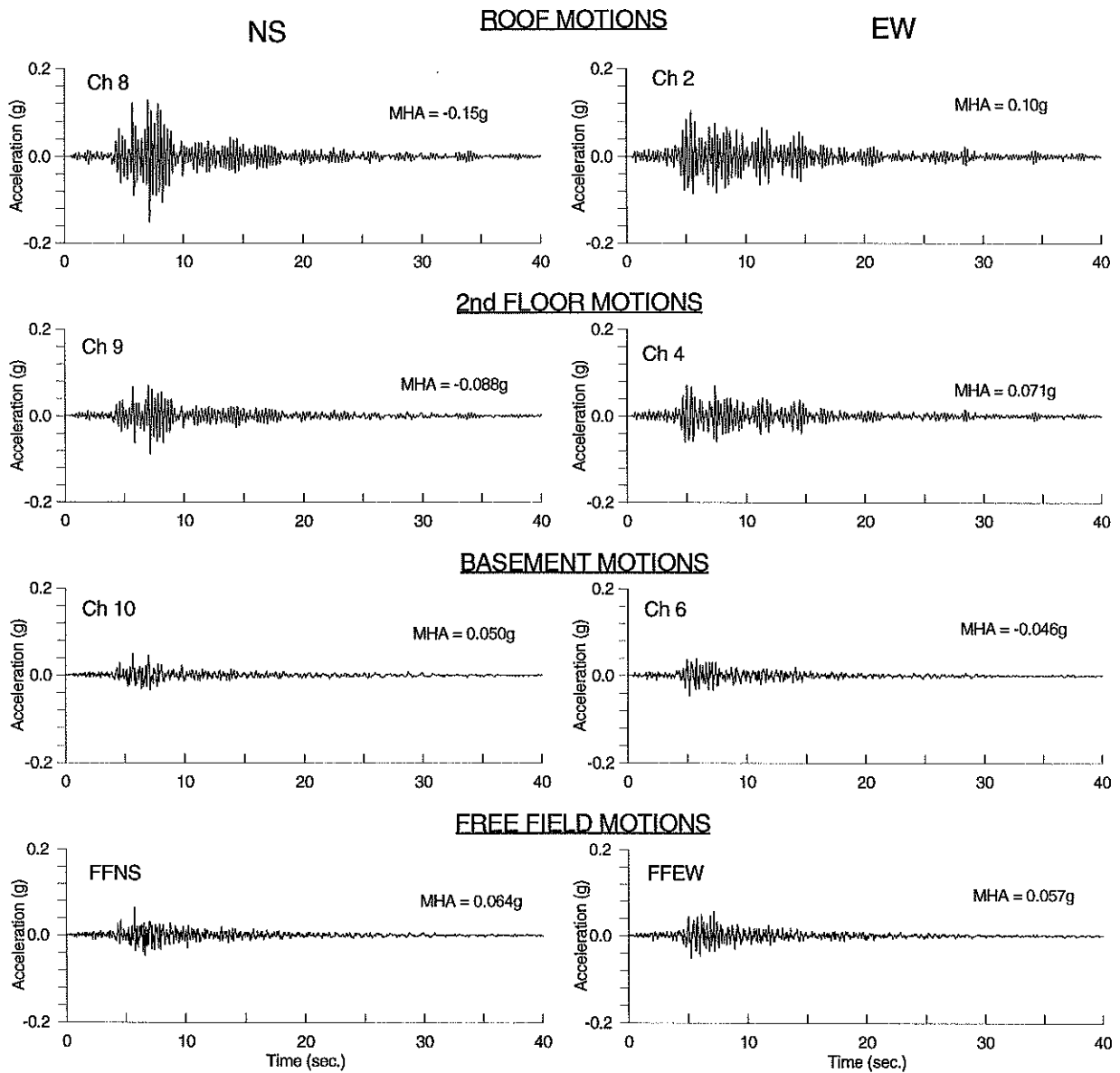


Fig. A35.WT1: POMONA 2 STORY BUILDING
Horizontal Acceleration Time Histories
1987 Whittier Earthquake

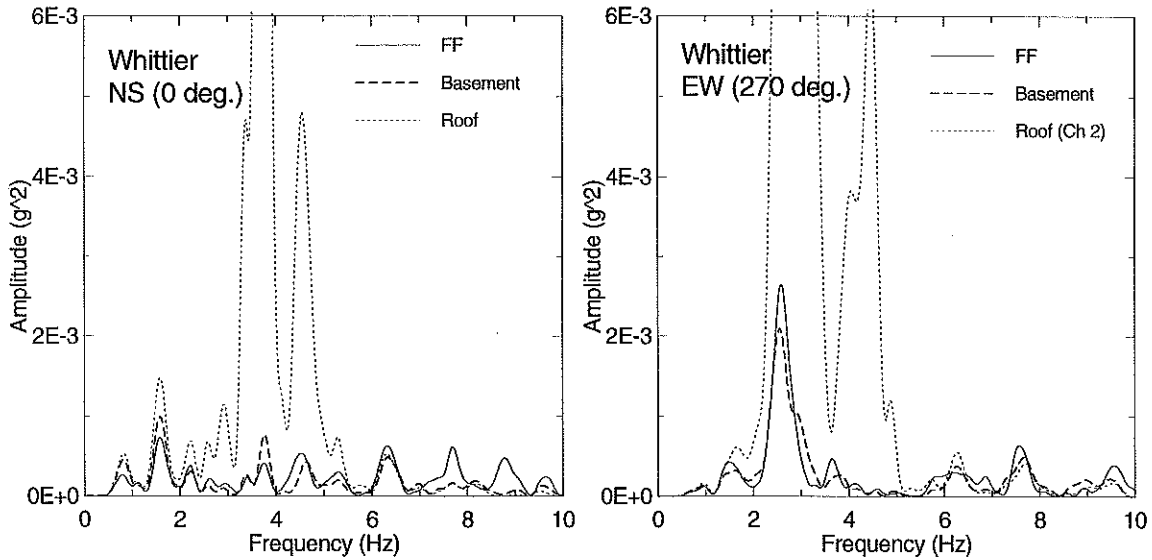


Fig. A35.WT2: POMONA 2 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

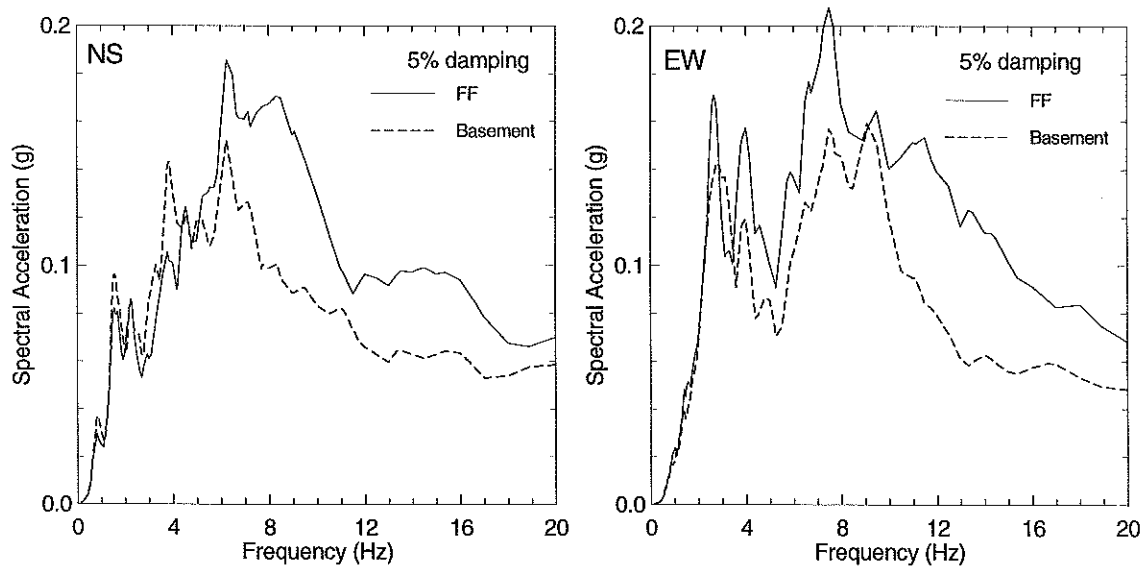


Fig. A35.WT3: POMONA 2 STORY BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1987 Whittier Earthquake

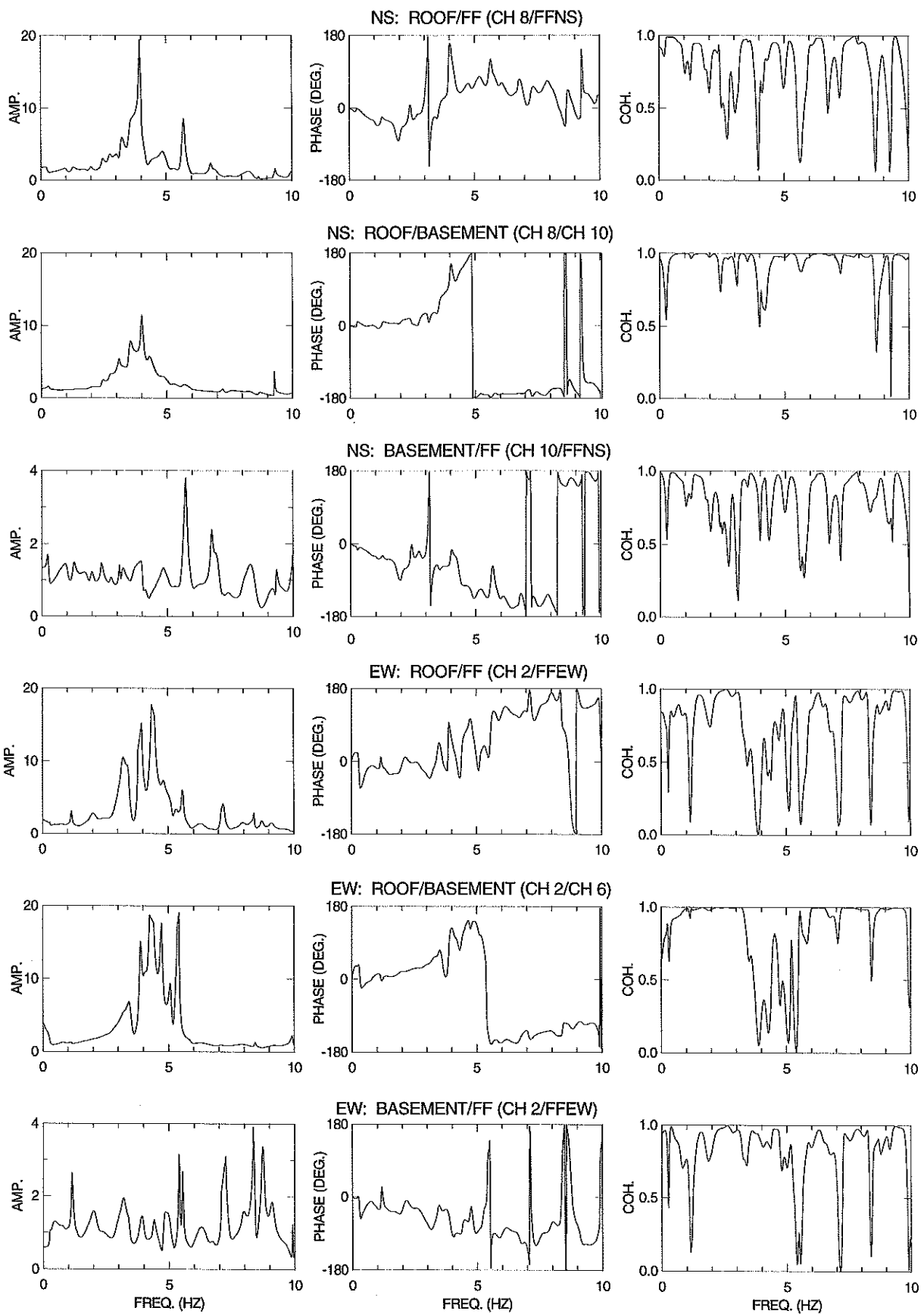


Fig. A35.WT5: POMONA 2 STORY BUILDING
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A35-10

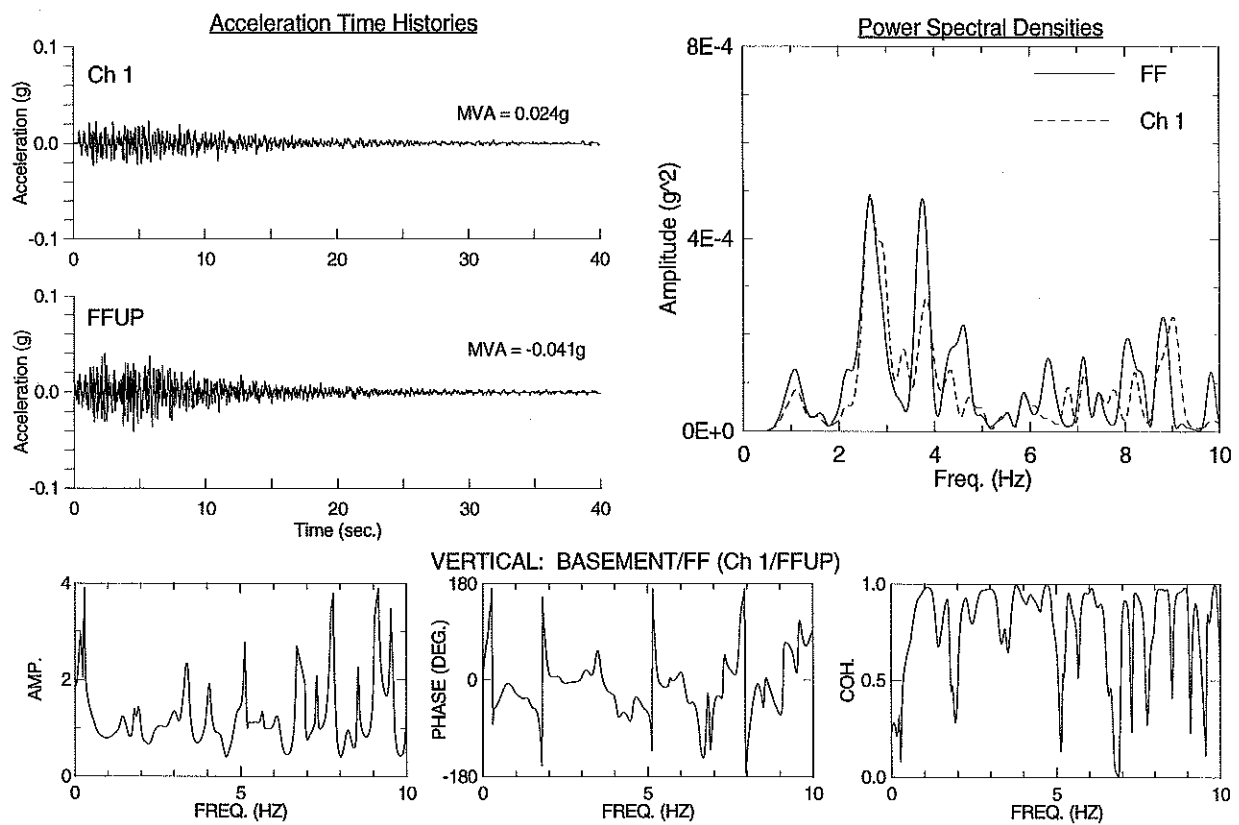


Fig. A35.WT5: POMONA 2 STORY BUILDING
Analysis of Vertical Motions
1987 Whittier Earthquake

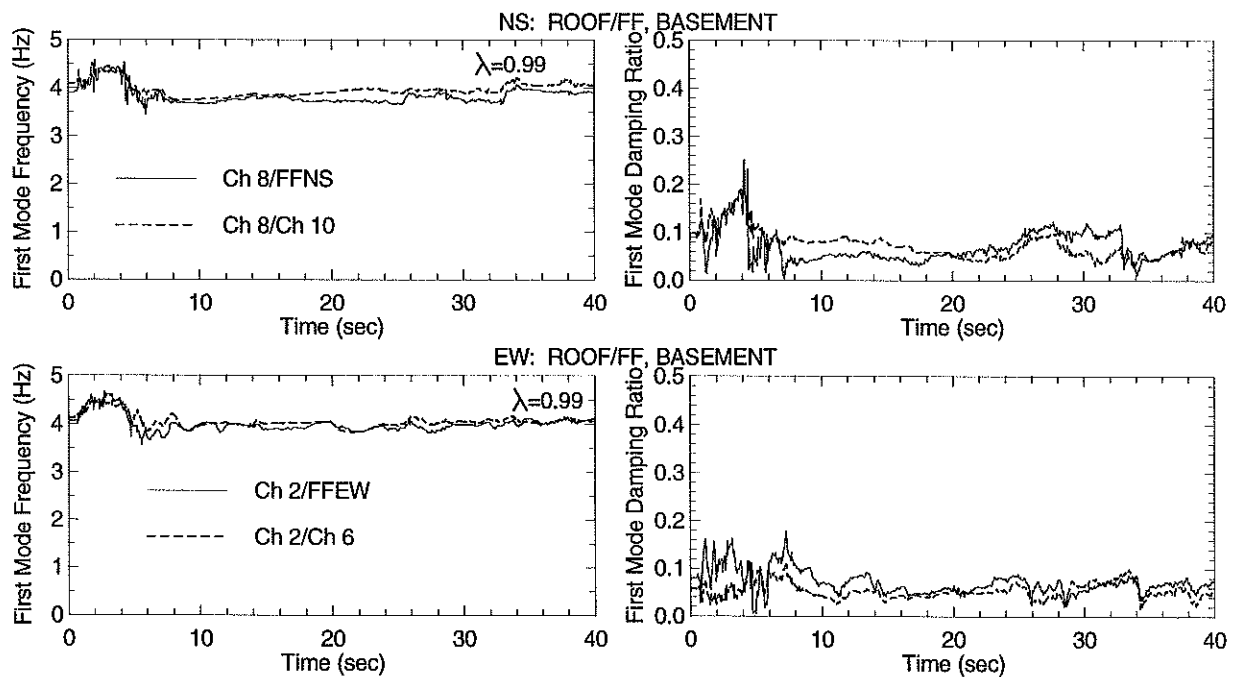


Fig. A35.WT6: POMONA 2 STORY BUILDING
Time Variation of First Mode Parameters
1987 Whittier Earthquake

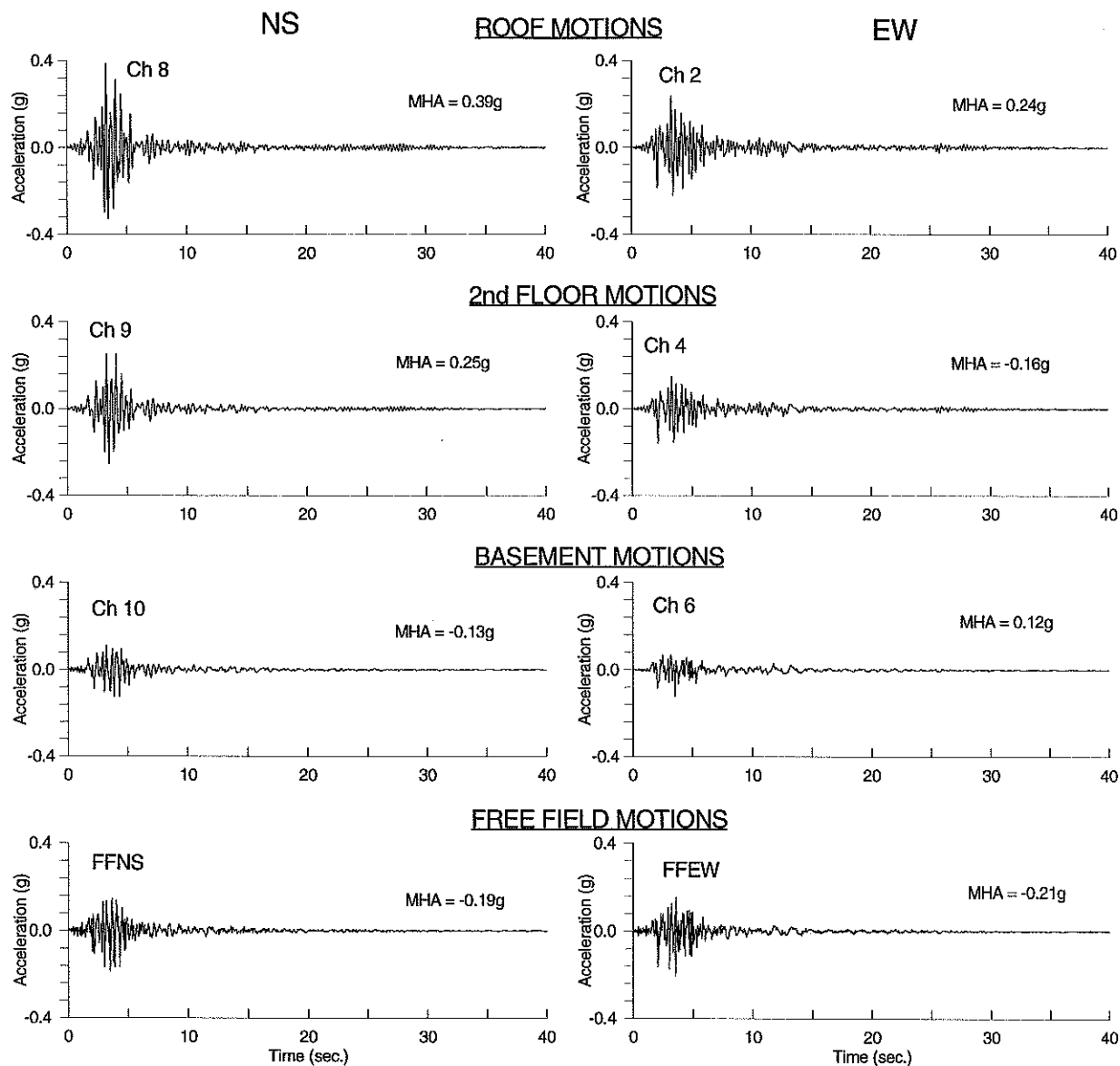


Fig. A35.UP1: POMONA 2 STORY BUILDING
Horizontal Acceleration Time Histories
1990 Upland Earthquake

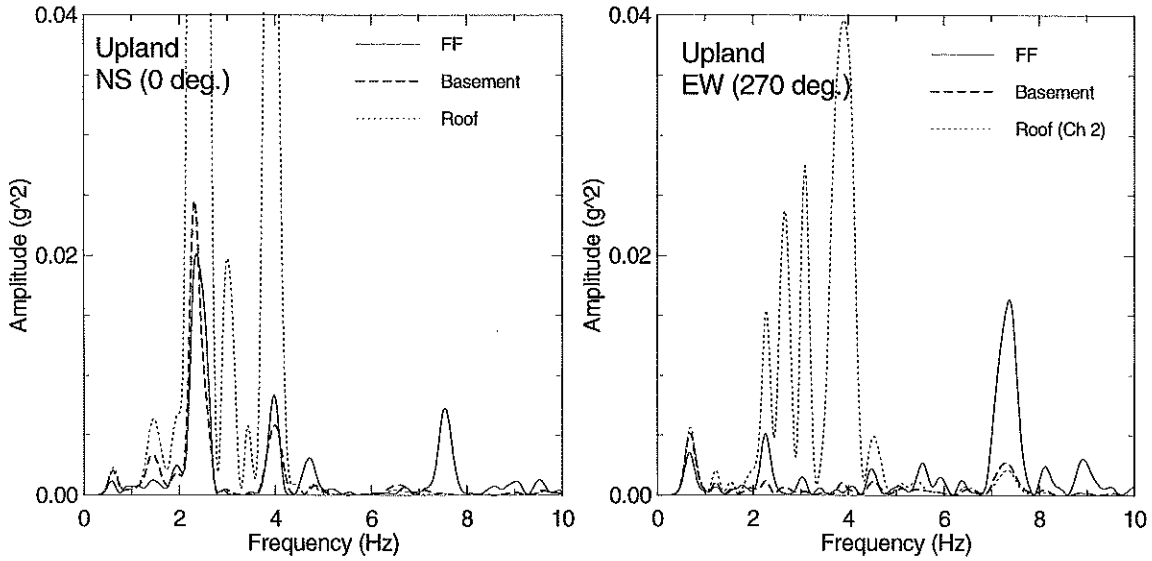


Fig. A35.UP2: POMONA 2 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1990 Upland Earthquake

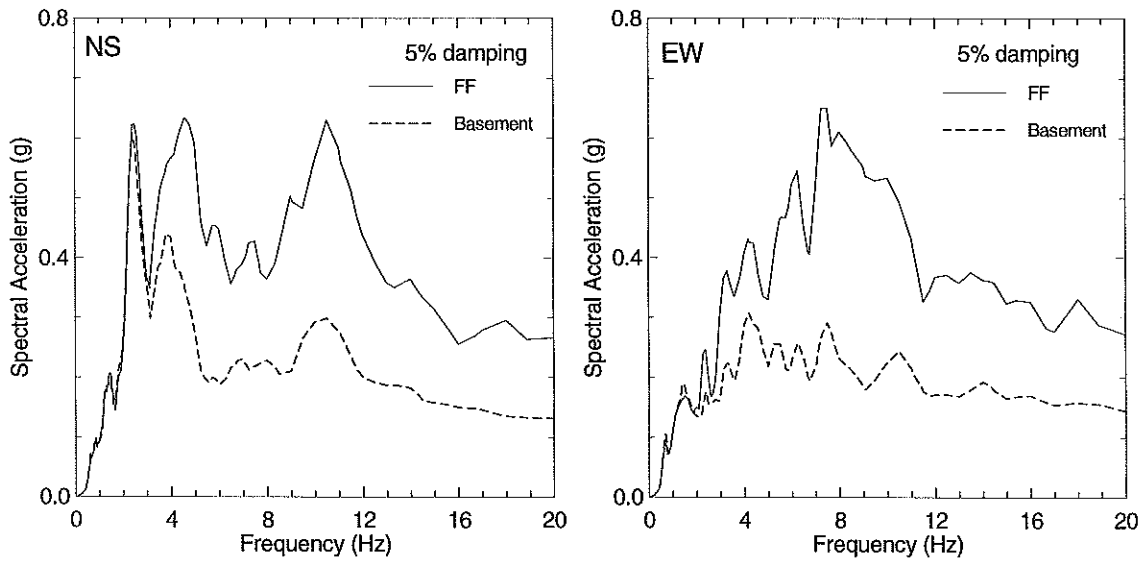


Fig. A35.UP3: POMONA 2 STORY BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1990 Upland Earthquake

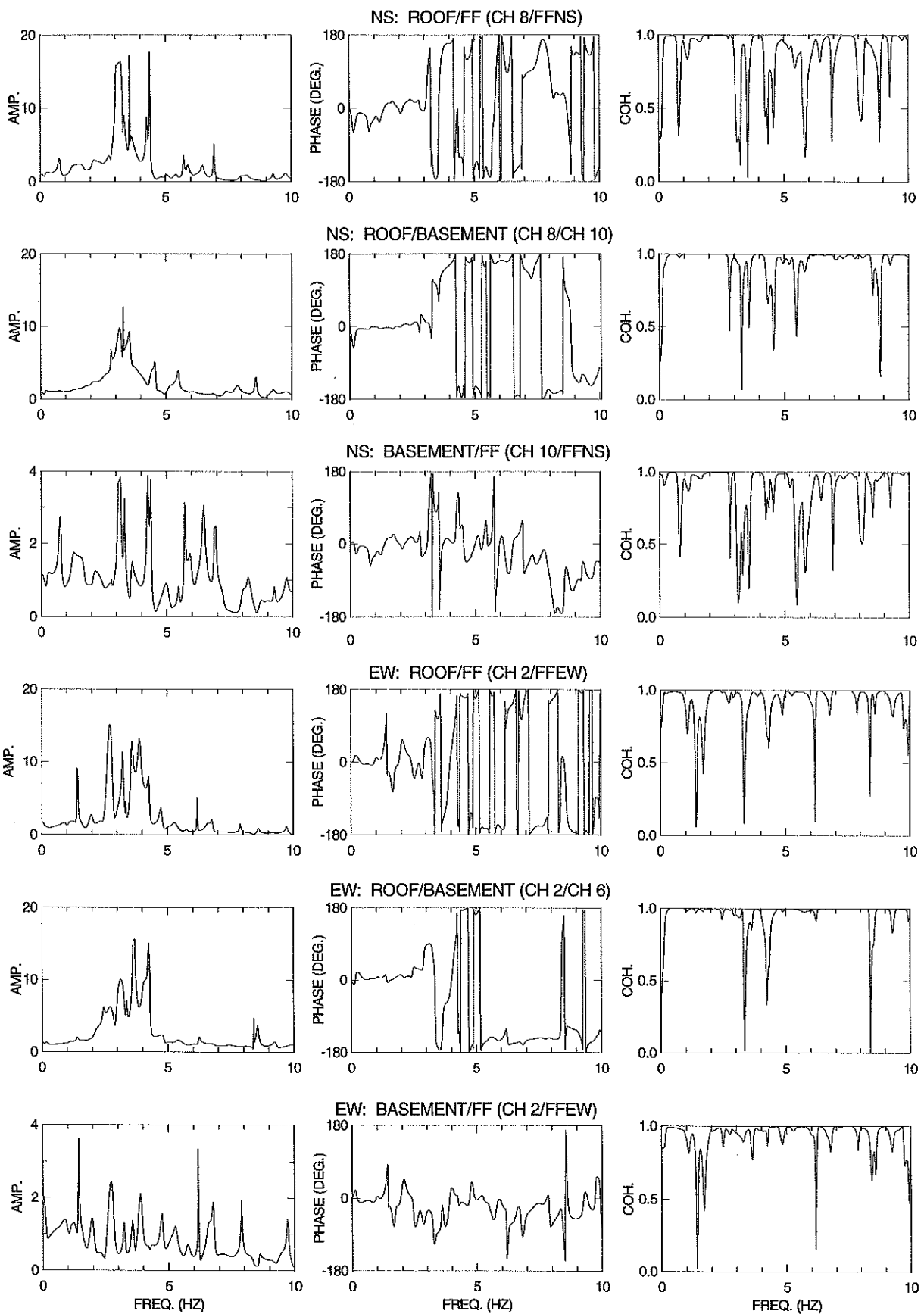


Fig. A35.UP4: POMONA 2 STORY BUILDING
Transmissibility and Coherency Functions
1990 Upland Earthquake
A35-14

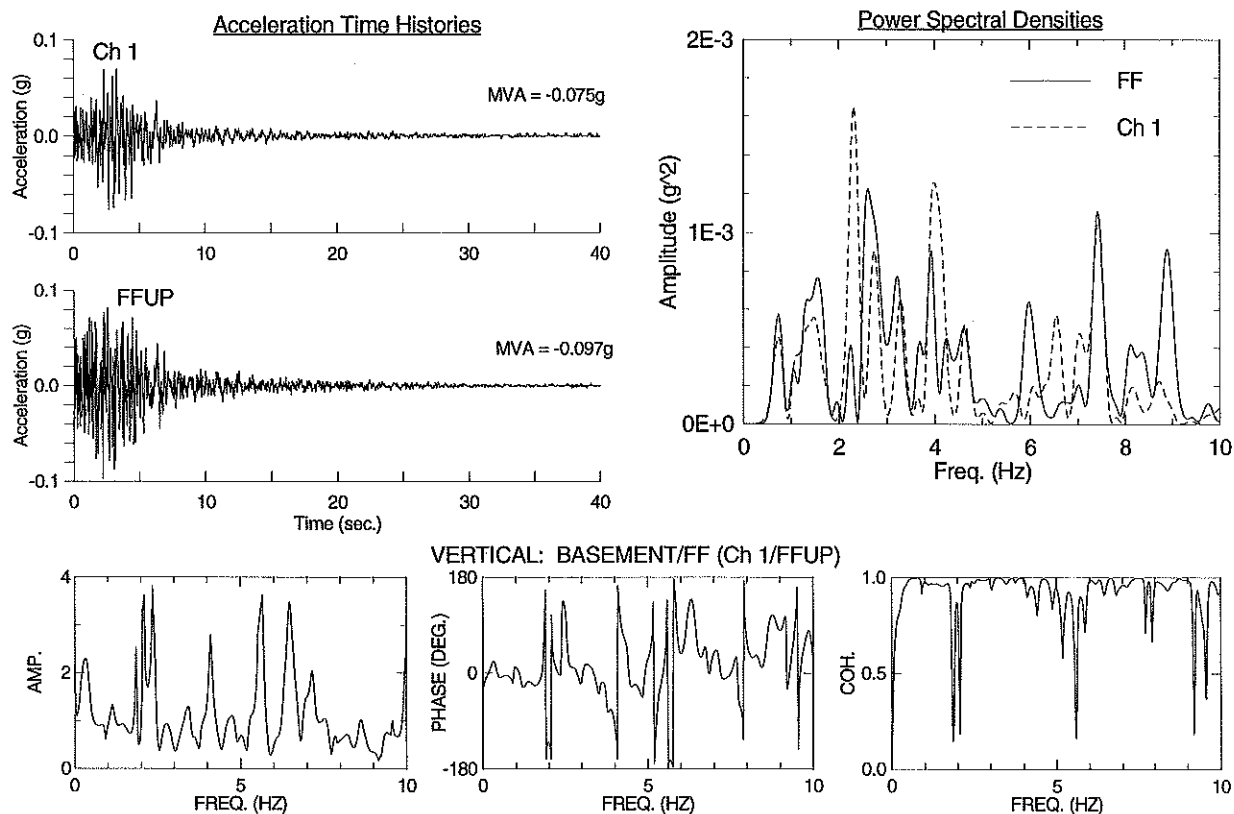


Fig. A35.UP5: POMONA 2 STORY BUILDING
Analysis of Vertical Motions
1990 Upland Earthquake

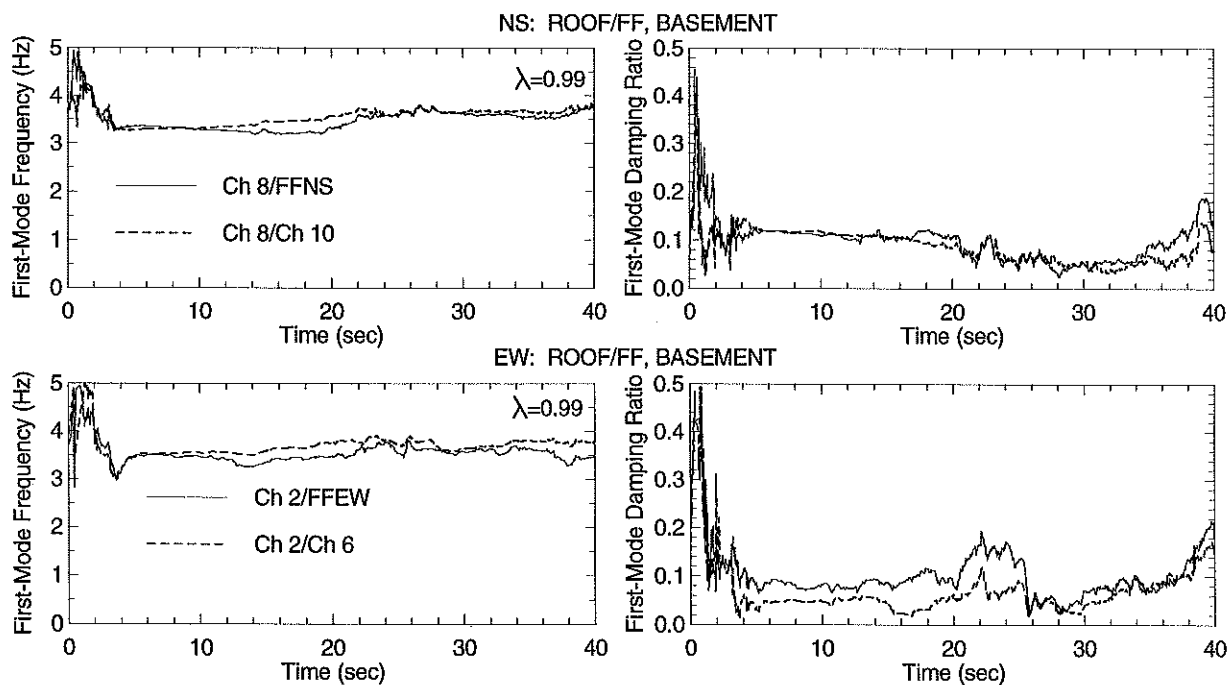


Fig. A35.UP6: POMONA 2 STORY BUILDING
Time Variation of First-Mode Parameters
1990 Upland Earthquake

SITE A36

Site Name: Pomona 6 Story Building
Sensors: CSMIP 23544/23525

Lat: 34.057°
Long.: 117.752°

Floor Levels: 7
Bsmt. Levels: 1
Structure: Concrete infilled frames

Foundation: footings
Site Geology: Alluvium
Design Date: 1923

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1990 Upland	0.19	0.21	0.097	0.13	0.087	0.063	0.30	0.28	--
1992 Landers	0.067	0.044	0.035	0.068	0.051	0.026	0.15	0.14	--
1992 Big Bear*	0.03	0.03	0.03	0.03	0.02	0.02	0.06	0.13	--
1994 Northridge*	0.05	0.05	0.04	0.04	0.06	0.03	0.08	0.16	--

*Not digitized

-- = No instrument

Preprocessing

Upland Earthquake (UP)

Preliminary: CSMIP; filtered, $f = 0.5\text{--}23\text{ Hz}$; $f_{Nyq}=25\text{ Hz}$; FF/str. data not synchronized

Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A36.UP0).

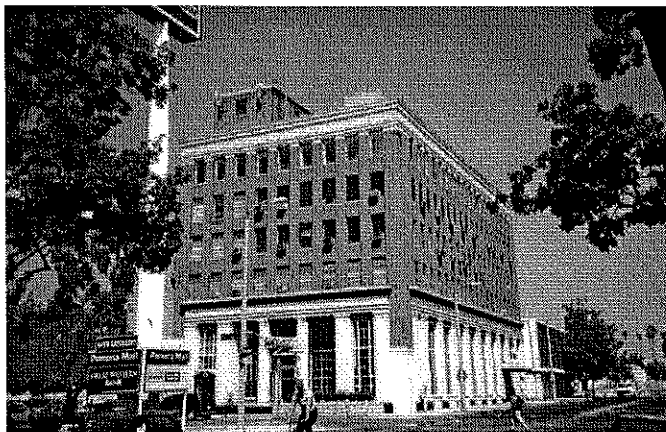
Landers Earthquake (LD)

Preliminary: CSMIP; filtered, $f = 0.4\text{--}23\text{ Hz}$ (str.), $0.12\text{--}23\text{ Hz}$ (FF); $f_{Nyq}=25\text{ Hz}$; FF/str. not synchronized.

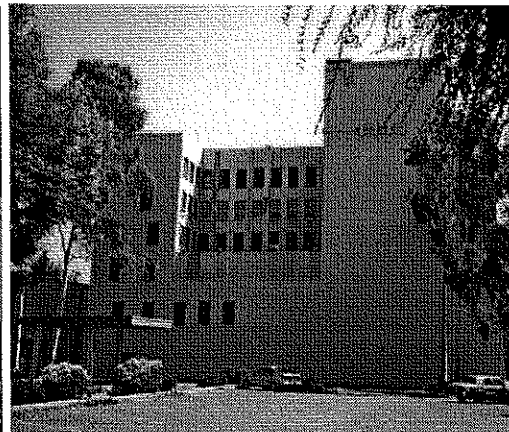
Final: decimated by 2 to $f_{Nyq}=12.5\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A36.LD0).

Figures

A35.1, A35.3 Site Plan, Generalized Soil Column
A36.1 Sensor Locations
A36.UP0-6 UP Analysis Results
A36.LD0-6 LD Analysis Results



View of structure, looking north-west



View of structure, looking east

DISCUSSION

Site Description

The building is 6-stories tall above the ground surface with a single level basement. The structure is rectangular (65 by 120 feet in plan) except for an opening above the 2nd floor on the west side of the building. No large structures are located immediately adjacent to the subject building or free-field instrument. The lateral load resisting system consists of reinforced concrete frames in both directions, with unreinforced masonry infill-walls with regular openings for windows. The foundation consists of footings. As shown in Fig. A36.1, accelerographs are present in the basement, 2nd floor and roof levels of the structure. The free-field instrument is located about 1270 feet southeast of the structure. Soil conditions at the site are described in the discussion for site A35.

Instrument Selection and Free-Field Conditions

The building is irregular due to the opening on the west side of the building above the second level, and consequently the transverse structural response is not uniform (i.e. there are torsional deformations). The primary analyses were performed using instrumentation along the south wall for transverse response and the north wall for longitudinal response. The transverse response was also evaluated from sensors on the north wall. The sensors used in the analyses are indicated in Table A36.1; acceleration time histories recorded by these sensors are shown in Fig. A36.**1.

Table A36.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	UP	NS	EW
Ch 6	Ch 11, 12	Ch 1	Ch 2	Ch 7, 8

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A36.**4) and the spectra for horizontal motions (Fig. A36.**2-3) generally indicate increasing de-amplification of basement motions with frequency. Coherencies for the B/FF transmissibility functions were generally fairly poor above 1 Hz, which can be attributed to the relatively large separation between the structure and FF instrument.

Transmissibility, coherency, and power spectral density functions for FF and basement vertical motions are presented in Fig. A36.**5. Minor amplification of basement vertical motions occurred at the first-mode transverse frequency, suggesting that base rocking may have modestly influenced the transverse structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A36.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A36.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A36.**6).

Table A36.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)*				Flex.-Base (FF-R)*			
		tr (EW)		L (NS)		tr (EW)		L (NS)		tr (EW)		L (NS)	
		UP: estimated	LD: estimated	estimated	estimated	d=4(4), j=8(8)	d=3(8), j=3(8)	d=5, j=8	d=2, j=8	d=6(14), j=8(8)	d=15(2), j=8(8)	d=0, j=8	d=-, j=-
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
UP	1	1.01	11.7	1.15	8.4	0.93 (1.36)	9.9 (14.6)	1.14	9.3	0.90 (1.43)	11.3 (15.6)	1.15	19.3
	2					2.00 (-)	11.1 (-)	1.87	7.1	2.02 (-)	13.1 (-)	2.02	20.7
	3					3.67 (-)	10.5 (-)	3.94	12.3	-- (-)	-- (-)	3.75	28.0
LD	1	0.87	7.9	--	--	0.82 (1.06)	6.6 (22.5)	1.01	10.3	0.79 (0.91)	5.9 (23.4)	--	--
	2					1.80 (1.78)	4.8 (8.3)	1.77	9.7	1.76 (1.71)	7.2 (8.1)	--	--
	3					3.50 (4.40)	8.9 (5.8)	--	--	3.46 (4.45)	7.9 (6.4)	--	--

*For the transverse direction, results for the south wall are shown without parenthesis, results for the north wall are shown with parenthesis.

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in both directions.
- The modest variations between the transverse fixed- and pseudo flexible-base parameters is consistent with the rocking effect evident from the vertical B/FF transmissibility function. No significant rocking appears to have occurred in the longitudinal direction.
- The south wall of the structure ($\tilde{f}_1=0.8$ to 0.9 Hz) is less stiff than the north wall ($\tilde{f}_1=0.9$ to 1.4 Hz), which likely causes significant torsional deformations. Such a torsional response may be responsible for the multiple spectral peaks in EW roof motions near 0.8 to 1.4 Hz.
- Based on a comparison of the nonparametric and parametric transmissibility functions for the longitudinal direction, the UP damping ratios may have been overestimated somewhat by the CEM.
- A reliable parametric model of longitudinal response could not be developed with the LD data.
- The structure had a stiffer response in the UP than LD events, despite the higher peak accelerations in UP. This likely resulted from the longer duration and larger low-frequency energy content of the LD data relative to the UP data.

The RPEM results (Figs. A36.**6) indicate no significant time-dependent changes in first-mode parameters during the UP event, but modest frequency shortening during the LD. First-mode frequencies for both directions were reduced by about 0.2 to 0.3 Hz over the time span of about 15 to 38 seconds.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A36.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 50 feet using the recorded free-field motions and the profile in Fig. A35.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though the rocking radiation damping for the longitudinal direction was increased by 60% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A36.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_S=1250$ fps	$h=53$ ft	$emb.=12.5$ ft	$\gamma = 0.15$
	$\nu=0.33$			tr: $r_1 = 50$ ft	$r_2 = 43$ ft
UP	$\beta^*=3.1\%$	$V_S^*=1150$ fps		L: $r_1 = 50$ ft	$r_2 = 59$ ft
LD	$\beta^*=2.2\%$	$V_S^*=1190$ fps			

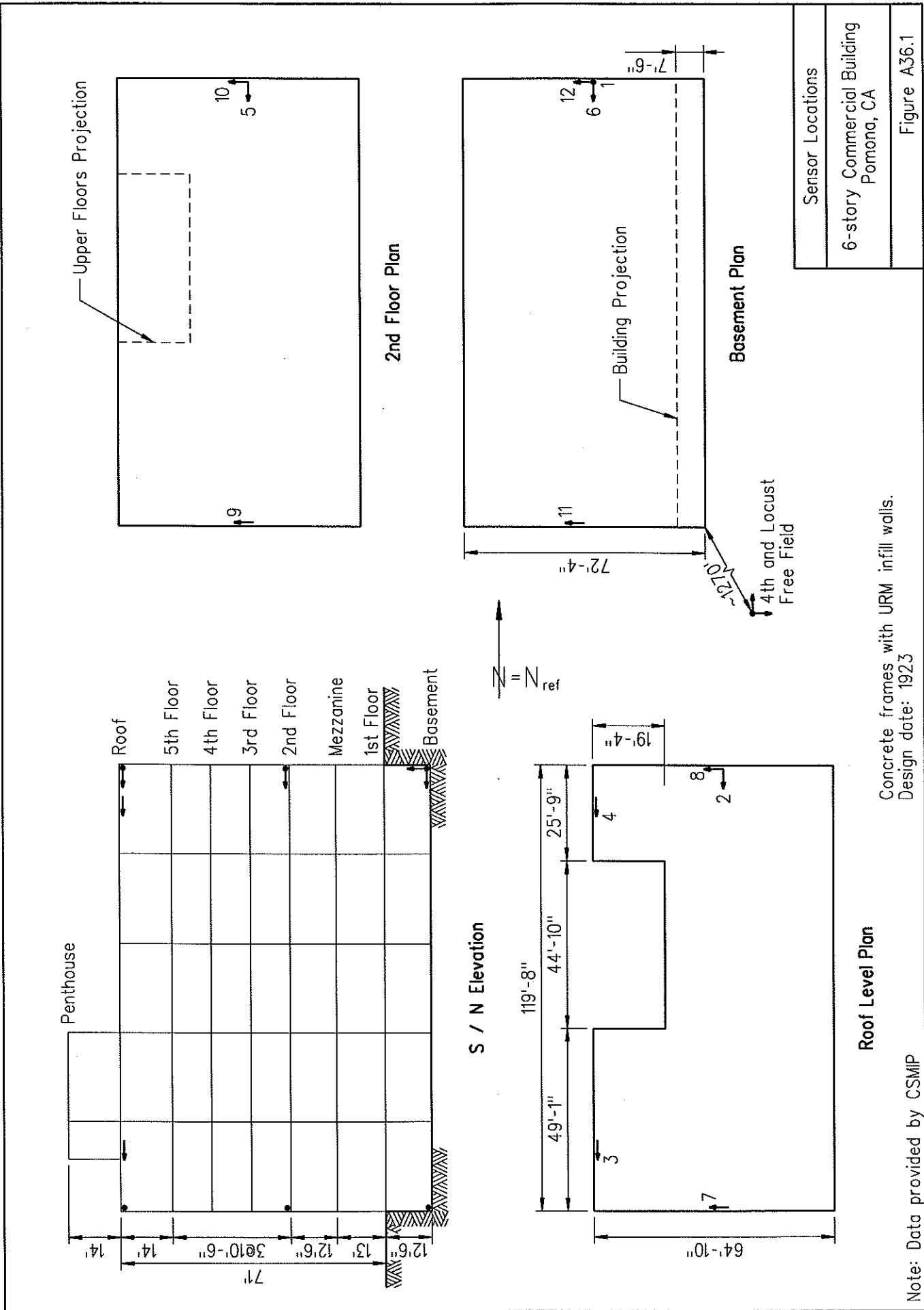
* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A36.3 with the fixed-base modal parameters in Table A36.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table A36.4.

Table A36.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
UP	Sys. ID	1.13	3.1	11.3	1.00	11.1	19.3
	Pre.-Vel.	1.01	0.1	11.5	1.01	0.1	8.3
	Pre.-Bie.	1.01	0.2	11.6	1.01	0.3	8.4
LD	Sys. ID	1.09	-0.2	5.9	--	--	--
	Pre.-Vel.	1.01	0.0	7.8	--	--	--
	Pre.-Bie.	1.01	0.1	7.8	--	--	--

The period lengthening was underpredicted by both formulations, while the foundation damping was well predicted for LD but underpredicted for UP. The two most likely sources of error are erroneous system identification results and incorrect shear wave velocities. As noted in Part (a), damping values for UP were likely overpredicted by the CEM, so the confidence level in the empirical $\tilde{\zeta}_0$ value is low. There is also uncertainty in the estimated shear wave velocities, as limited subsurface data is available in the vicinity of this structure. If the soil at the site is softer than is reflected in Fig. A35.3, the estimated $\tilde{T}/T$ and $\tilde{\zeta}_0$ values would be larger.



Sensor Locations
6-story Commercial Building Pomona, CA
Figure A36.1

Concrete frames with URM infill walls.
Design date: 1923

Note: Data provided by CSMIP
pomona.dgn(35)

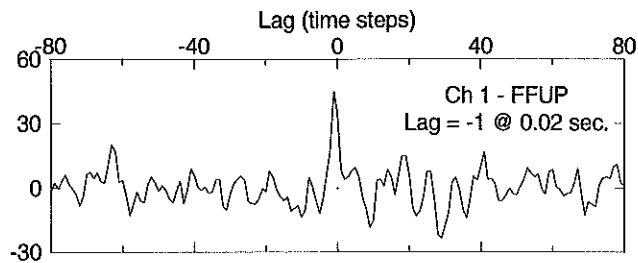


Fig. A36.UP0: Cross-correlation function between vertical basement and free-field accelerations

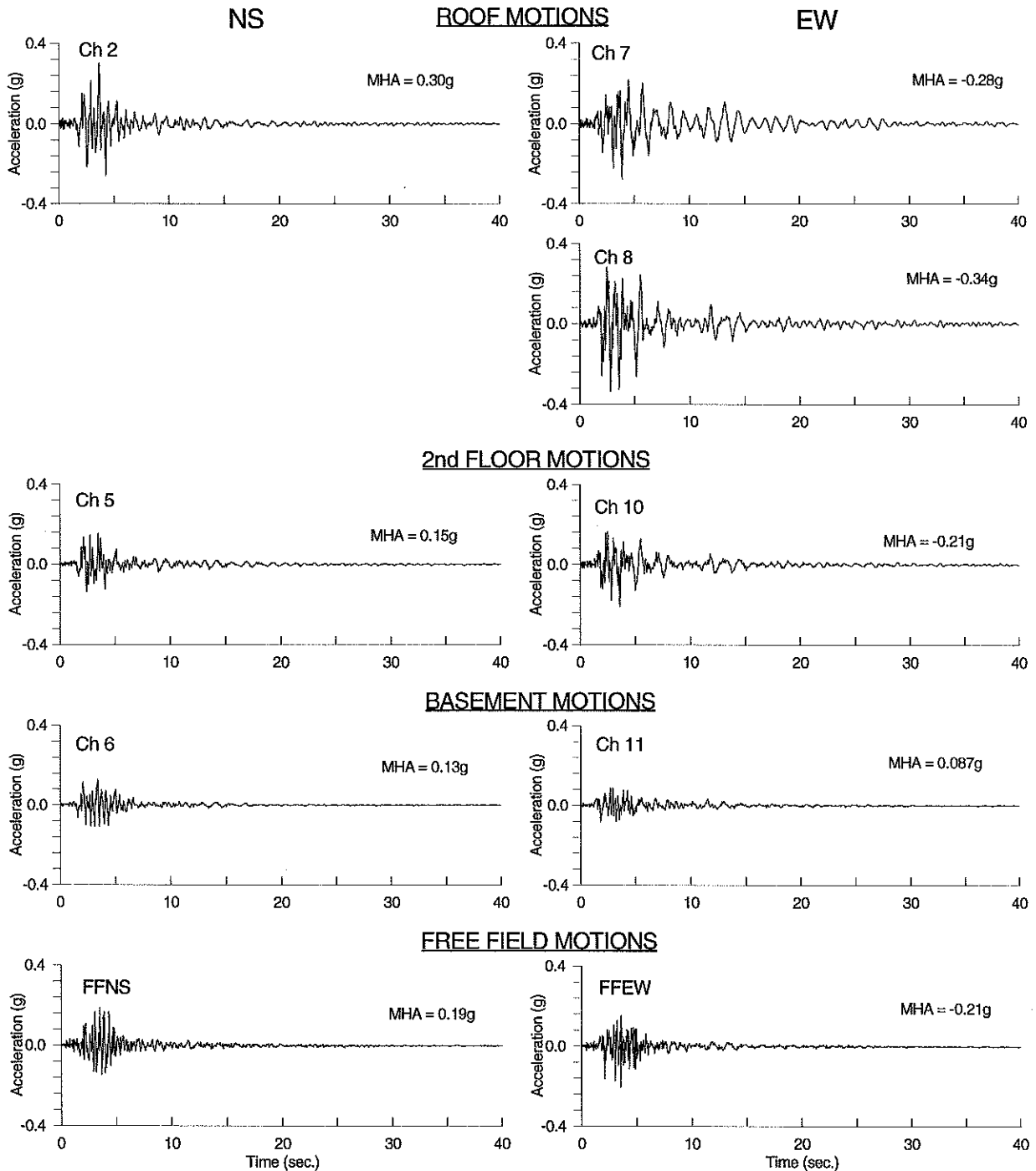


Fig. A36.UP1: POMONA 6 STORY BUILDING
Horizontal Acceleration Time Histories
1990 Upland Earthquake

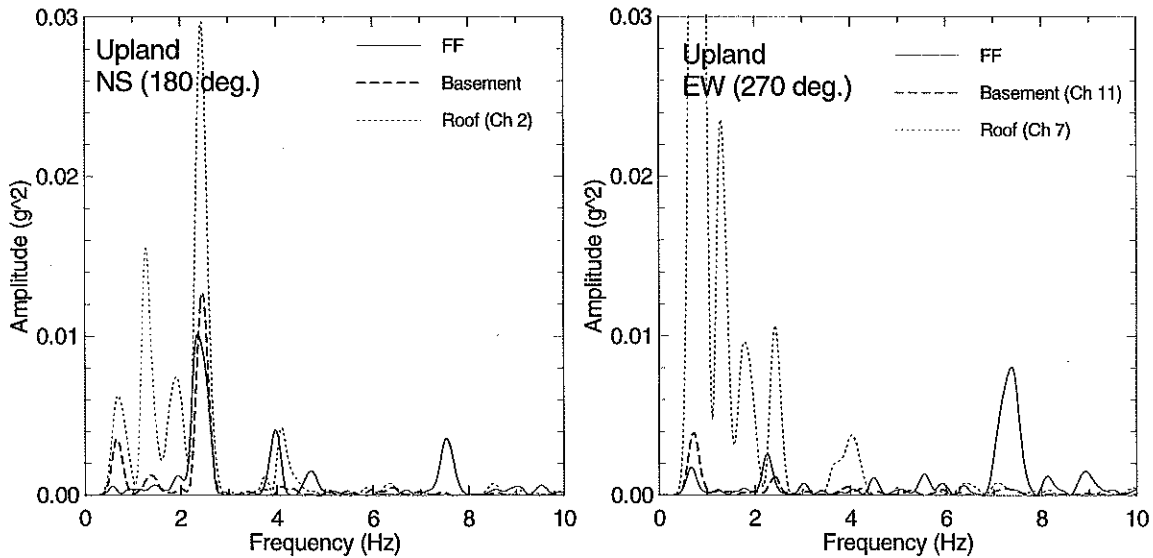


Fig. A36.UP2: POMONA 6 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1990 Upland Earthquake

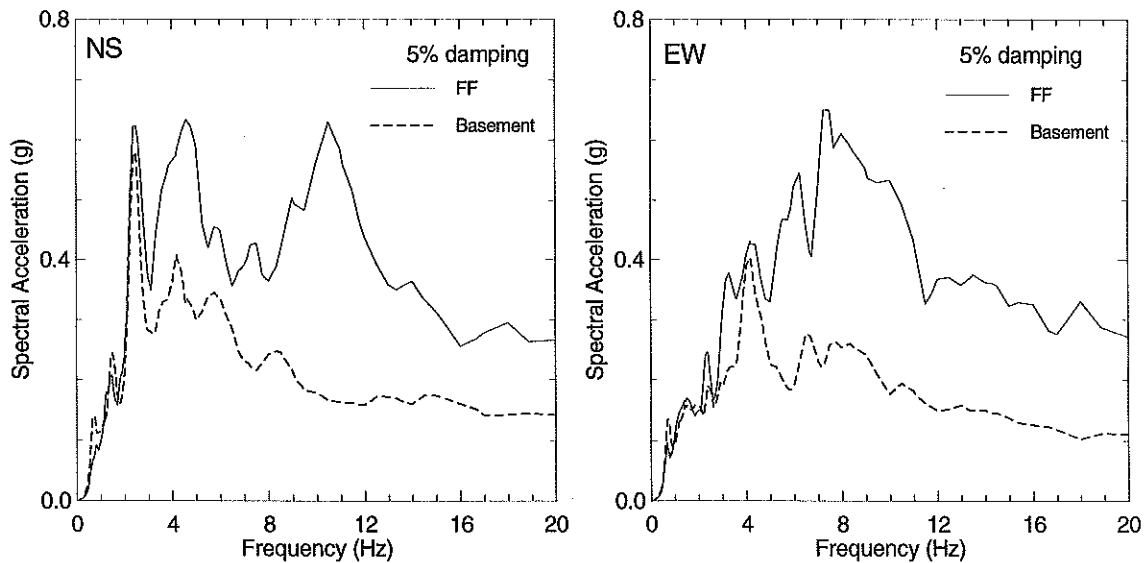


Fig. A36.UP3: POMONA 6 STORY BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1990 Upland Earthquake

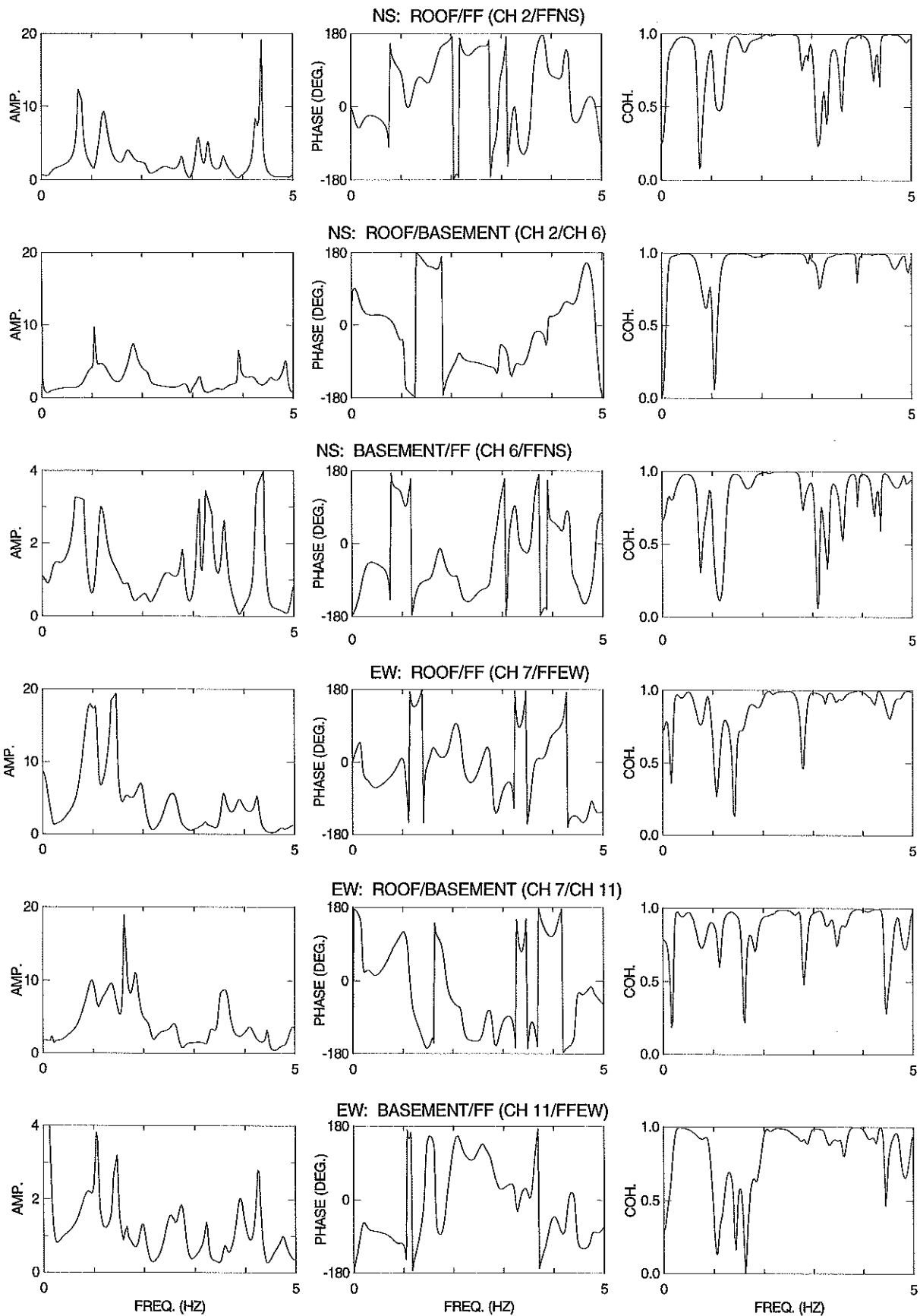


Fig. A36.UP4: POMONA 6 STORY BUILDING
Transmissibility and Coherency Functions
1990 Upland Earthquake
A36-8

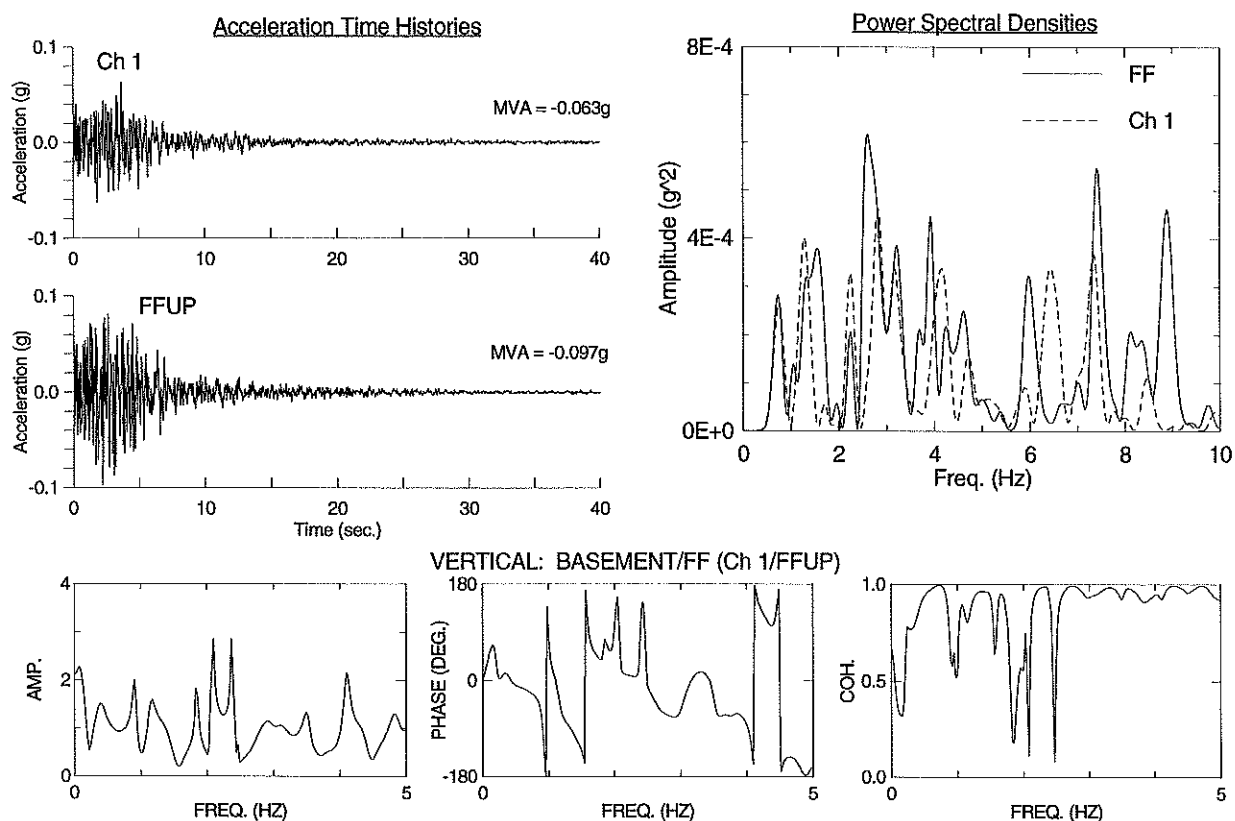


Fig. A36.UP5: POMONA 6 STORY BUILDING
Analysis of Vertical Motions
1990 Upland Earthquake

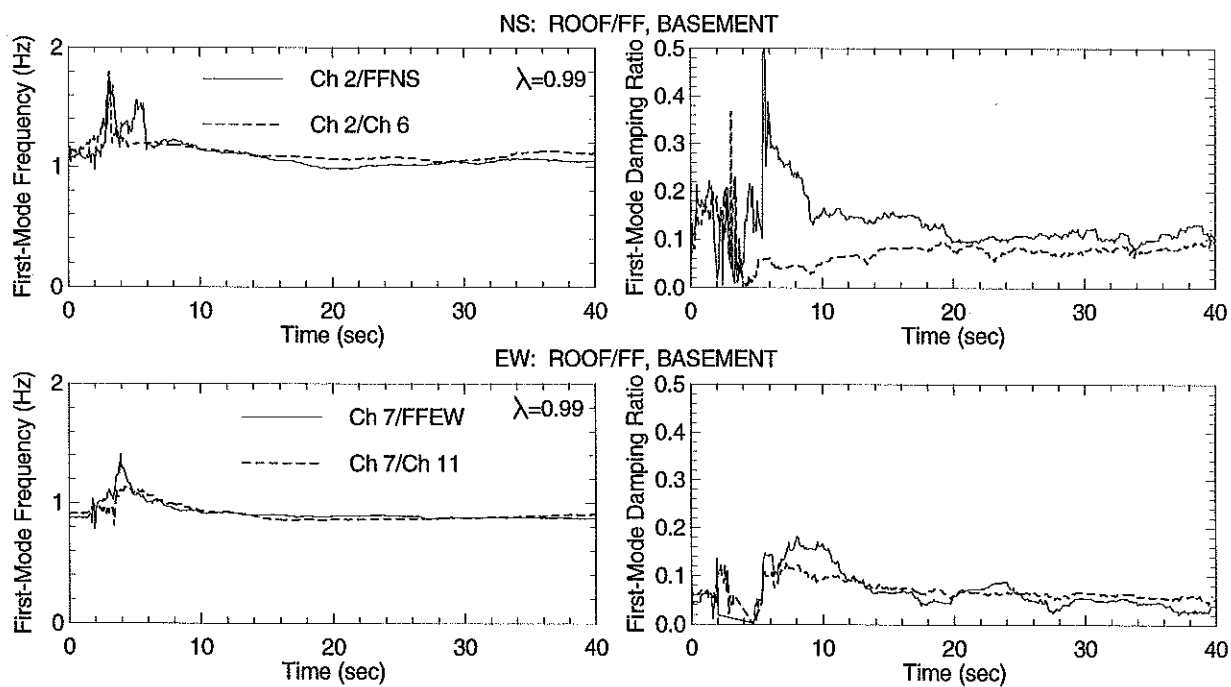


Fig. A36.UP6: POMONA 6 STORY BUILDING
Time Variation of First-Mode Parameters
1990 Upland Earthquake

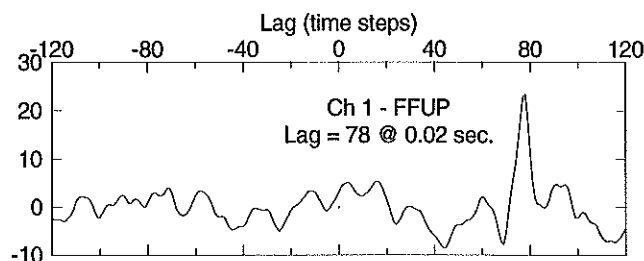


Fig. A36.LD0: Cross-correlation function between vertical basement and free-field accelerations

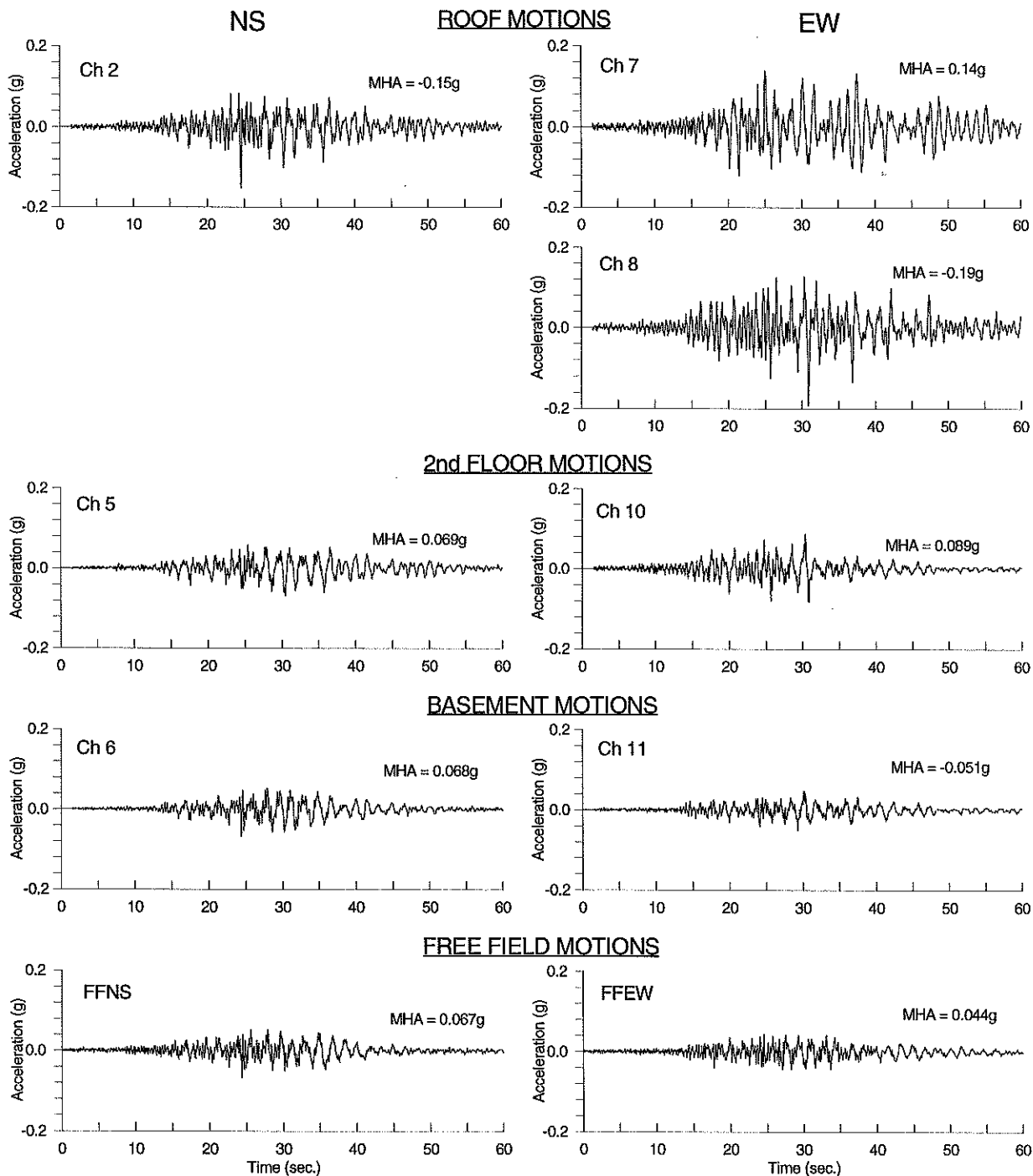


Fig. A36.LD1: POMONA 6 STORY BLDG
Horizontal Acceleration Time Histories
1992 Landers Earthquake

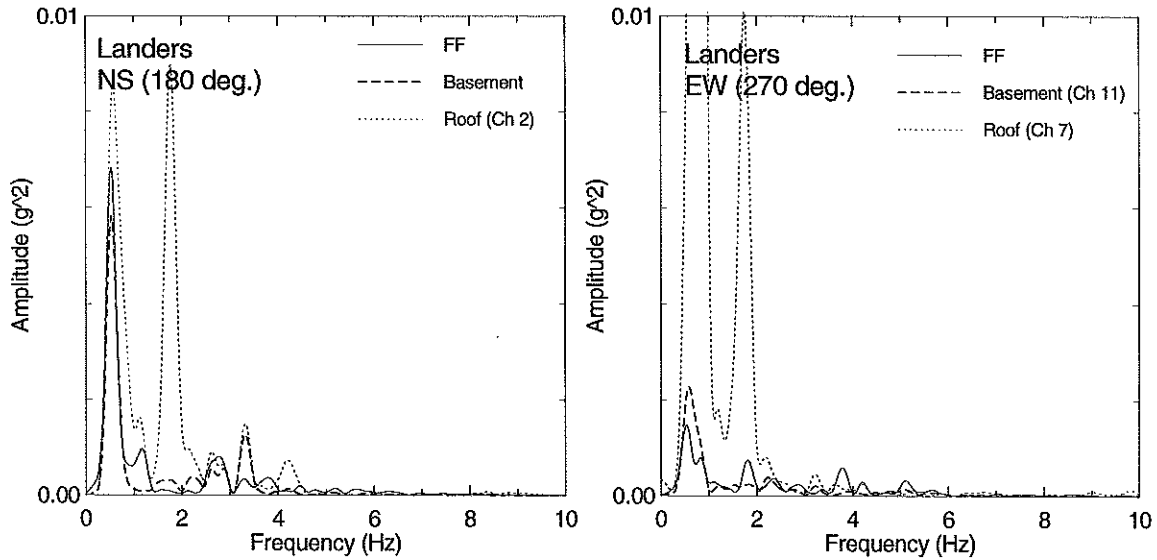


Fig. A36.LD2: POMONA 6 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

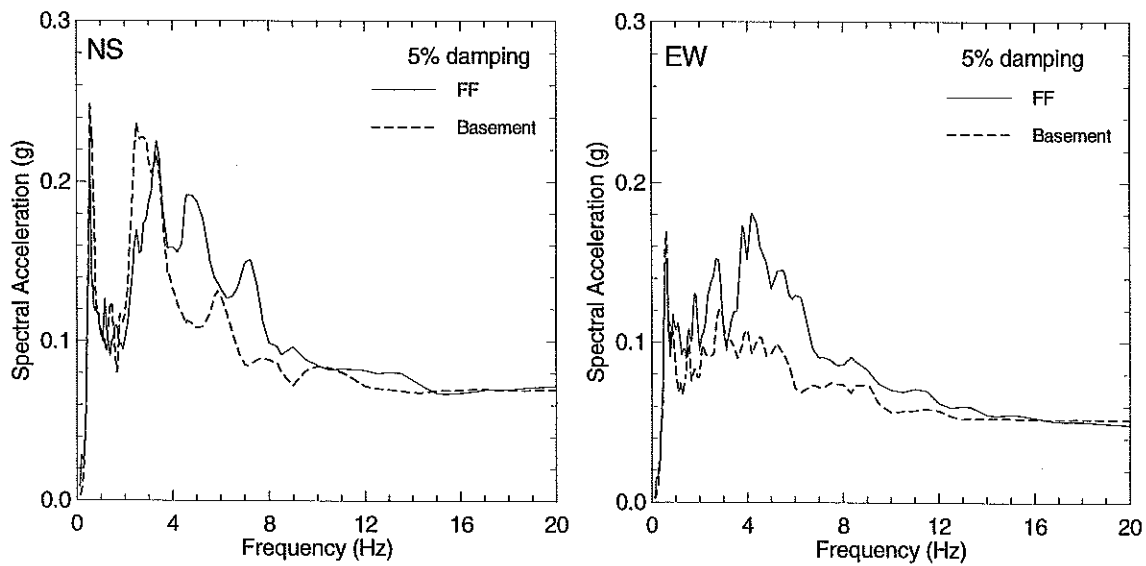


Fig. A36.LD3: POMONA 6 STORY BUILDING
Acceleration Response Spectra of Basement and Free-Field Motions
1992 Landers Earthquake

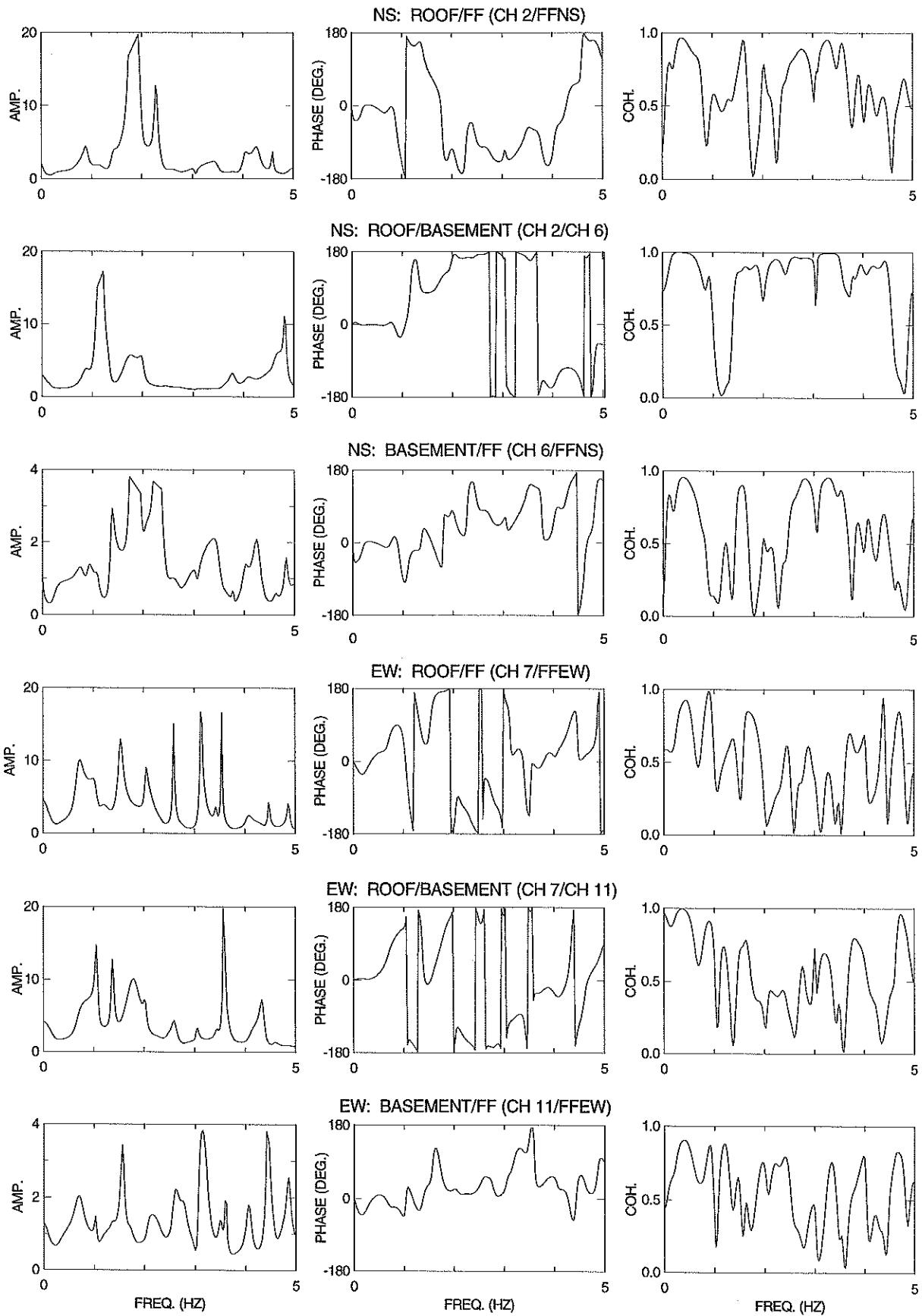


Fig. A36.LD4: POMONA 6 STORY BUILDING
Transmissibility and Coherency Functions
1992 Landers Earthquake
A36-12

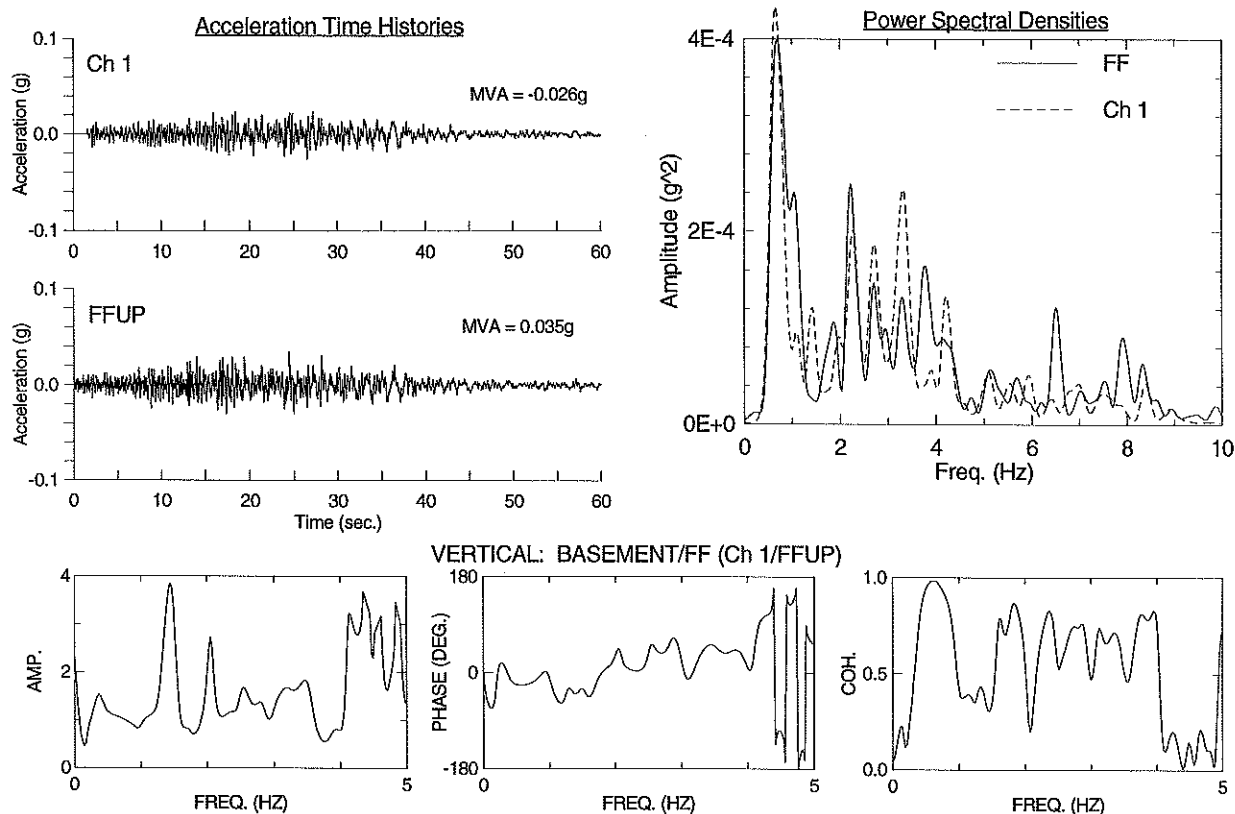


Fig. A36.LD5: POMONA 6 STORY BUILDING
Analysis of Vertical Motions
1992 Landers Earthquake

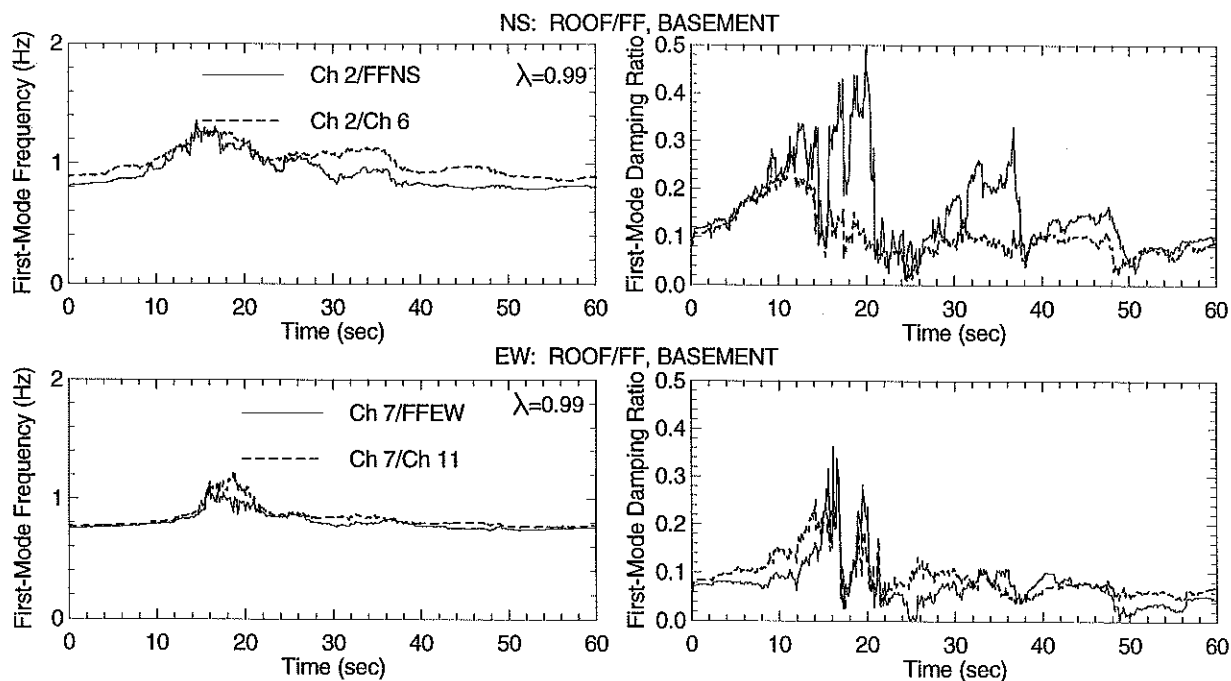


Fig. A36.LD6: POMONA 6 STORY BUILDING
Time Variation of First-Mode Parameters
1992 Landers Earthquake

SITE A37

Site Name:	Rancho Cucamonga Law & Justice Ctr.	Lat:	34.104°
Sensors:	CSMIP 23497	Long.:	117.574°
Floor Levels:	5	Foundation:	footings
Bsmt. Levels:	1	Site Geology:	Alluvium
Structure:	isolators, braced steel frames, shear walls	Design Date:	1983

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Foundation Level			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1985 Redlands	0.040	0.032	0.027	0.028	0.035	0.015	0.027	0.020	--
1987 Whittier	0.046	0.057	0.035	0.019	0.028	0.020	0.049	0.044	--
1990 Upland	0.24	0.23	0.16	0.14	0.11	0.084	0.16	0.087	--
1991 Sierra Madre*	0.04	0.04	0.03	0.02	0.03	0.02	0.08	0.08	--
1992 Landers	0.11	0.068	0.048	0.097	0.064	0.053	0.18	0.11	--
1992 Big Bear*	0.04	0.05	0.04	0.03	0.06	0.04	0.07	0.07	--
1994 Northridge	0.072	0.046	0.032	0.039	0.037	0.029	0.093	0.098	--

*Not digitized

-- = No instrument

Preprocessing

Redlands Earthquake (RD)

Preliminary: CSMIP; filtered, $f = 0.8\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized.

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A37.RD0).

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.8\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data synchronized.

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Upland Earthquake (UP)

Preliminary: CSMIP; filtered, $f = 0.6\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized.

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$; FF/str. data synchronized (Fig. A37.UP0).

Landers and Northridge Earthquakes (LD and NR)

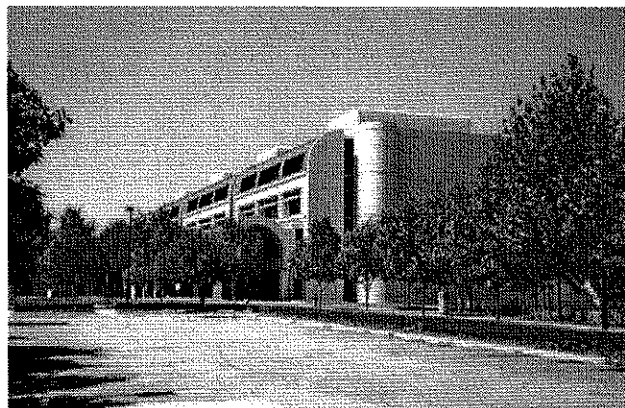
Preliminary: CSMIP; filtered, $f = 0.4\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data synchronized.

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Figures

A37.1-A37.3	Site Plan, Sensor Locations, Generalized Soil Column
A37.RD0-5	RD Analysis Results
A37.WT1-5	WT Analysis Results
A37.UP0-6	UP Analysis Results
A37.LD1-6	LD Analysis Results
A37.NR1-5	NR Analysis Results

View of structure,
looking north-west



DISCUSSION

Site Description

The building is 4-stories tall above ground level with a single level basement. The structure has a rectangular shape, with plan dimensions of 110 by 414 feet. No large buildings are located near the structure. The lateral load resisting system consists of braced steel frames in the upper 4 stories and concrete shear walls in the basement, as well as a base isolation system consisting of 98 elastomeric bearings at the base of columns below the basement level. The foundation consists of spread footings. As shown in Fig. A37.2, accelerographs are present at the foundation level (below the isolators), basement (above the isolators), 2nd Floor and roof. The free-field instrument is located in a parking lot about 330 feet southeast of the building.

The site is relatively level and is situated within a portion of the Upper Santa Ana River drainage area known as the Chino Basin. Based on soil borings to a depth of 40 feet by LCA (1982) and a general geologic description by Tarics et al. (1984), soil conditions at the site consist of about 3 to 10 feet of aeolian silty sand, 50 to 100 feet of Holocene alluvial fan deposits comprised of silty, gravely sand, and about 700 feet of Pleistocene coarse- to fine-grained fan deposits which extend to bedrock. Only the data from soil borings is presented in Fig. A37.3. The velocities of the near-surface soils were estimated using published correlations with SPT blow count and void ratio.

Instrument Selection and Free-Field Conditions

Instruments near the centroid of the structure were selected for analysis (Table A37.1). Acceleration time histories recorded by these sensors are shown in Fig. A37.**1. Two foundation level vertical sensors were used to evaluate transverse base rocking.

Table A37.1: Sensors in structure used for analyses

Foundation Level (FL)			Basement (B)		Roof (R)	
NS	EW	V	NS	EW	NS	EW
Ch 12	Ch 16	Ch 1, 2	Ch 9	Ch 15	Ch 6	Ch 14

The following input was used in system identification analyses to evaluate fixed-base structural parameters for the transverse direction,

$$\text{input} = \text{Ch12} + (\text{Ch1} - \text{Ch2}) \cdot 74/110 \quad (1)$$

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Foundation Level and Free-Field Motions

FL/FF transmissibility functions (Fig. A37.**4) and the spectra for horizontal motions (Fig. A37.**2-3) generally indicate de-amplification of FL motions relative to the FF for frequencies greater than about 2 Hz. The de-amplification was largest when the FF motions had significant

high-frequency energy content (e.g. WT, UP, NR). The FL and FF motions were similar for frequencies less than about 2 Hz.

Transmissibility, coherency, and power spectral density functions for FF and FL vertical motions are presented in Fig. A37.**5. No significant amplification is evident at the first-mode frequencies for lateral response, indicating that rocking did not significantly contribute to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A37.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A37.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A37.UP/NR6). Frequencies and damping ratios identified by Pardoan and Hart (1985) from ambient vibration tests are presented in Table A37.3.

Table A37.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (FL-R)				Flex.-Base (FF-R)			
		tr (NS) RD: d=2, j=6 WT: d=2, j=6 UP: d=3, j=6 LD: d=3, j=6 NR: d=3, j=6		L (EW) estimated		tr (NS) d=2, j=6 d=2, j=6 d=3, j=6 d=2, j=6 d=3, j=6		L (EW) d=3, j=5 d=3, j=5 d=3, j=5 d=3, j=6 d=4, j=5		tr (NS) d=3, j=6 d=2, j=6 d=3, j=6 d=2, j=6 d=2, j=6		L (EW) d=2, j=5 d=1, j=5 d=3, j=5 d=2, j=6 d=1, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
RD	1	1.70	3.7	1.67	4.2	1.68	4.1	1.67	4.3	1.65	3.3	1.66	5.6
	2	3.46	8.7			3.50	8.9	3.77	7.1	3.26	10.5	3.62	4.7
WT	1	1.59	5.4	1.54	6.1	1.57	5.2	1.54	6.3	1.55	4.1	1.52	8.4
	2	3.15	8.1			3.14	8.7	3.53	8.2	3.05	7.6	3.38	9.7
UP	1	1.31	7.6	--	--	1.30	7.8	1.29	6.7	1.31	8.3	1.33	6.0
	2	2.63	8.1			2.63	8.6	3.12	10.4	2.55	7.2	3.04	9.2
LD	1	1.20	13.9	1.15	17.4	1.18	13.7	1.15	17.4	1.16	14.0	1.12	17.2
	2	2.48	5.8			2.47	6.0	2.69	13.6	2.48	7.0	2.73	12.3
NR	1	1.34	6.1	1.26	8.9	1.32	5.8	1.26	8.8	1.32	4.6	1.23	7.5
	2	2.67	7.0			2.66	6.9	3.08	11.7	2.61	7.2	2.90	13.2

Table A37.3: First-mode frequencies (in Hz) identified from ambient vibration tests (Pardoan and Hart, 1985)

	NS	EW	Torsion
Frequency (Hz)	1.76	1.64	2.03
Damping (%)	2.3	2.4	2.1

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure generally responded strongly in its lower two modes. The first mode tended to dominate under relatively strong levels of shaking (i.e. UP and LD), while the second mode was also significant under lower

amplitude shaking in the other events. The base isolators caused large first-mode damping ratios of 14 to 17% (FL-R case) for strong events such as LD.

- The small variations between the fixed- and pseudo flexible-base parameters are consistent with the lack of evidence for a significant rocking response in the vertical FL/FF transmissibility function.
- The structural response above the isolators was stiffer and more lightly damped than is reflected by the modal parameters in Table A37.2 (which include the effects of the isolators). For comparison, first-mode parameters for B-R response in the LD event were 1.56 Hz and 4.1% damping (tr) and 1.49 Hz and 4.4% damping (L).
- The stiffness of the structural response was strongly dependent on the level of shaking, with long duration, high amplitude events such as LD having significantly “softer” response (as quantified by lower frequencies and higher damping) than smaller events such as RD.
- The frequencies and damping ratios from low-amplitude recordings (e.g. RD) are similar to those identified from ambient vibration tests (Table A37.3).

Previous studies of the building response using UP data (Maison and Ventura, 1992) found that torsional contributions to roof displacement were minor, and that rocking effects were insignificant. Further, the first-mode frequencies identified in the study were similar to those in Table A37.2. Studies by Papageorgiou and Lin (1989) employed different system identification techniques to those used here with RD data, and found first-mode parameters of $\tilde{f}^* \approx 2$ Hz and $\tilde{\zeta}^* \approx 2.5$ to 3%, which differs somewhat from the results of this study.

RPEM analyses for the UP and LD events indicated that first-mode frequencies for both directions decreased during periods of strong shaking (by as much as 50%, LD-tr), and subsequently increased gradually during lower amplitude shaking. These variations in first-mode frequency result primarily from stiffness degradation in the base isolation system during strong shaking. Similarly, damping values increased during strong shaking, and then gradually decreased as the shaking subsided.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A37.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 120 feet using the recorded FF motions and the profile in Fig. A37.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 2.0 and 1.7, respectively, to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Using the parameters in Table A37.4 with the fixed-base modal parameters in Table A37.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A37.5.

Table A37.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_S=1190$ fps	$h=56$ ft	emb.=14 ft	$\gamma = 0.15$
	$v=0.33$		tr:	$r_1 = 120$ ft	$r_2 = 87$ ft
RD	$\beta^*=1.4\%$	$V_S^*=1170$ fps	L:	$r_1 = 120$ ft	$r_2 = 170$ ft
WT	$\beta^*=1.7\%$	$V_S^*=1160$ fps	LD	$\beta^*=4.4\%$	$V_S^*=1040$ fps
UP	$\beta^*=4.5\%$	$V_S^*=1060$ fps	NR	$\beta^*=2.8\%$	$V_S^*=1110$ fps

* = strain-dependent soil properties determined from deconvolution analyses

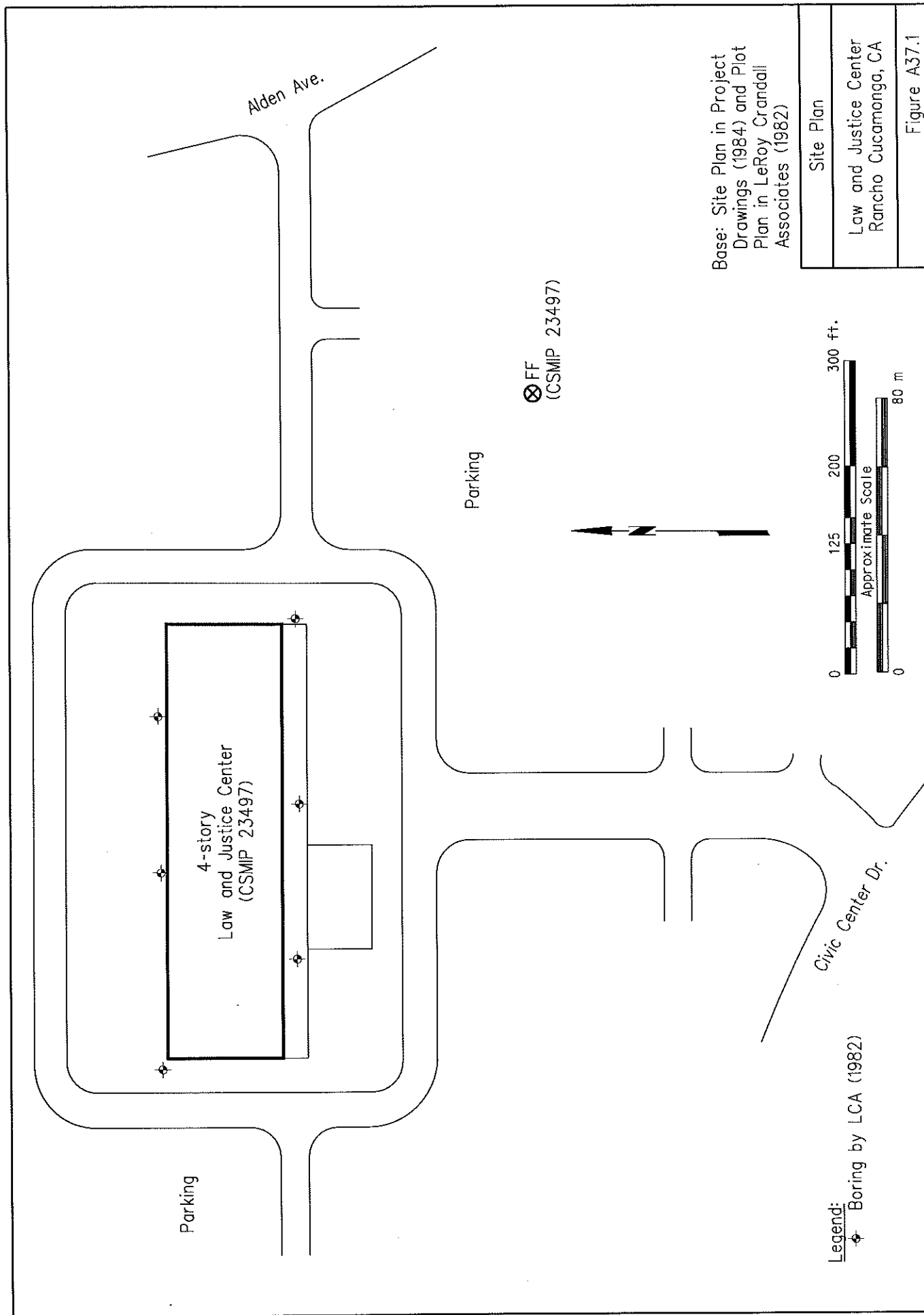
Table A37.5: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
RD	Sys. ID	1.03	0	3.3	1.00	1.4	5.6
	Pre.-Vel.	1.04	1.3	4.5	1.03	1.5	5.4
	Pre.-Bie.	1.03	2.3	5.6	1.04	2.6	6.4
WT	Sys. ID	1.03	-0.9	4.1	1.01	2.5	8.4
	Pre.-Vel.	1.04	1.1	6.0	1.02	1.3	7.0
	Pre.-Bie.	1.03	2.0	6.9	1.03	2.2	7.8
UP	Sys. ID	1.00	0.7	8.3	0.97	-0.7	6.0
	Pre.-Vel.	1.03	0.9	7.9	1.02	1.0	7.3
	Pre.-Bie.	1.02	1.6	8.7	1.03	1.9	8.1
LD	Sys. ID	1.03	1.4	14.0	1.02	0.9	17.2
	Pre.-Vel.	1.03	0.7	13.6	1.02	0.8	17.4
	Pre.-Bie.	1.02	1.3	14.4	1.02	1.4	17.8
NR	Sys. ID	1.01	-1.3	4.6	1.02	-0.9	7.5
	Pre.-Vel.	1.03	0.8	6.4	1.02	0.8	9.2
	Pre.-Bie.	1.02	1.4	7.1	1.02	1.4	9.8

The period lengthening and foundation damping were satisfactorily predicted by the "modified Veletsos" formulation. The "modified Bielak" formulation generally overpredicted foundation damping, but gave superior predictions for two cases (WT-L and LD-tr). Negative values of foundation damping from the system identification indicate numerical errors in these results.

Data Sources

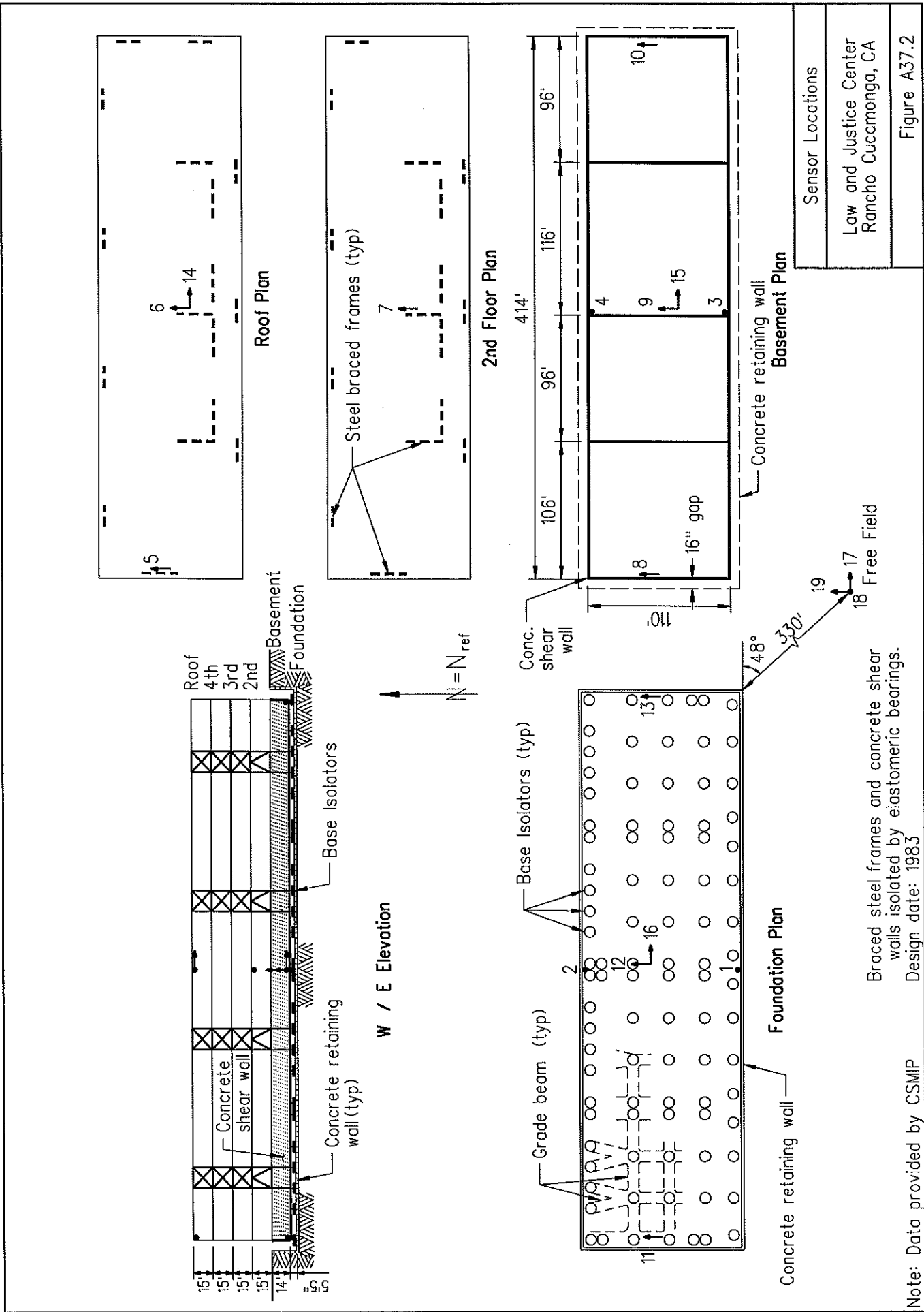
- LeRoy Crandall Assoc., LCA (1982). unpublished Geotechnical Investigation for West Valley Law and Justice Center, Rancho Cucamonga, CA," *Job. No. AE-82121*.
- Maison, B.F. and Ventura, C.E. (1992). "Seismic analysis of base-isolated San Bernardino County Building," *Earthquake Spectra*, EERI, 8(4), 605-631.
- Papageorgiou, A.A. and Lin, B-C. (1989). "Study of the earthquake response of the base-isolated Law and Justice Center in Rancho Cucamonga," *Earthquake Engrg. and Structural Dyn.*, Vol. 18, 1189-1200.
- Pardoan, G.C. and Hart, G.C. (1985). "Modal analysis of a base-isolated building," *Proc. 3rd Int. Modal Analysis Conf.*, Orlando, FL, 1312-1326.
- Tarics, A.G., Way, D, and Kelly, J.M. (1984). "The implementation of base isolation for the Foothill Communities Law and Justice Center, County of San Bernardino, CA," Report to the National Science Foundation and the County of San Bernardino.



Base: Site Plan in Project Drawings (1984) and Plot Plan in LeRoy Crandall Associates (1982)

Site Plan
Low and Justice Center Rancho Cucamonga, CA
Figure A37.1

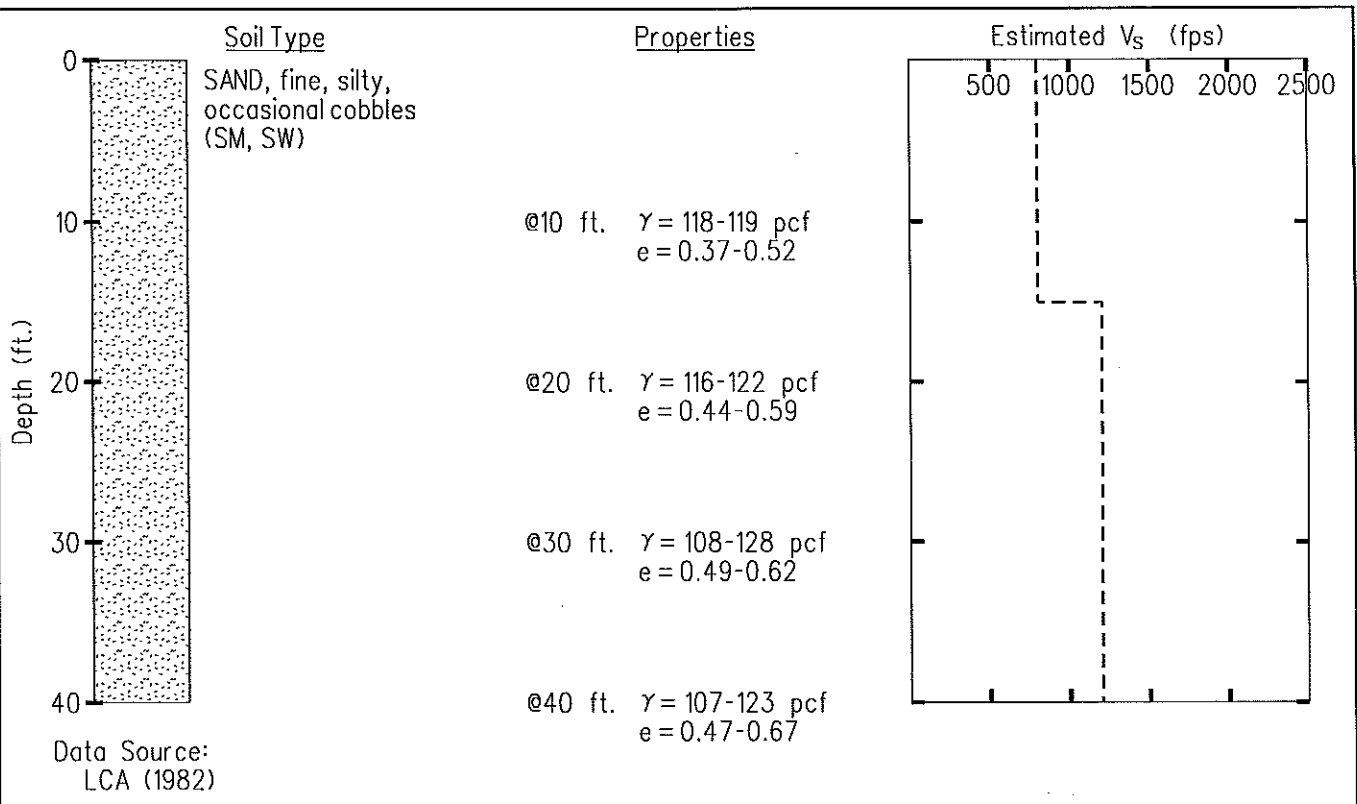
Legend:
 $\oplus$ Boring by LCA (1982)



Braced steel frames and concrete shear walls isolated by elastomeric bearings.
Design date: 1983

Note: Data provided by CSMIP

cucamon.dgn[100]



Notes:

1. Groundwater not encountered.
2. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile

Law and Justice Center
Rancho Cucamonga, CA

Figure A37.3

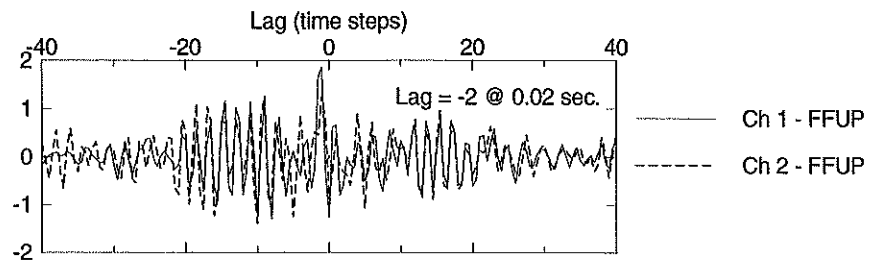


Fig. A37.RD0: Cross-correlation function between vertical foundation and free-field accelerations

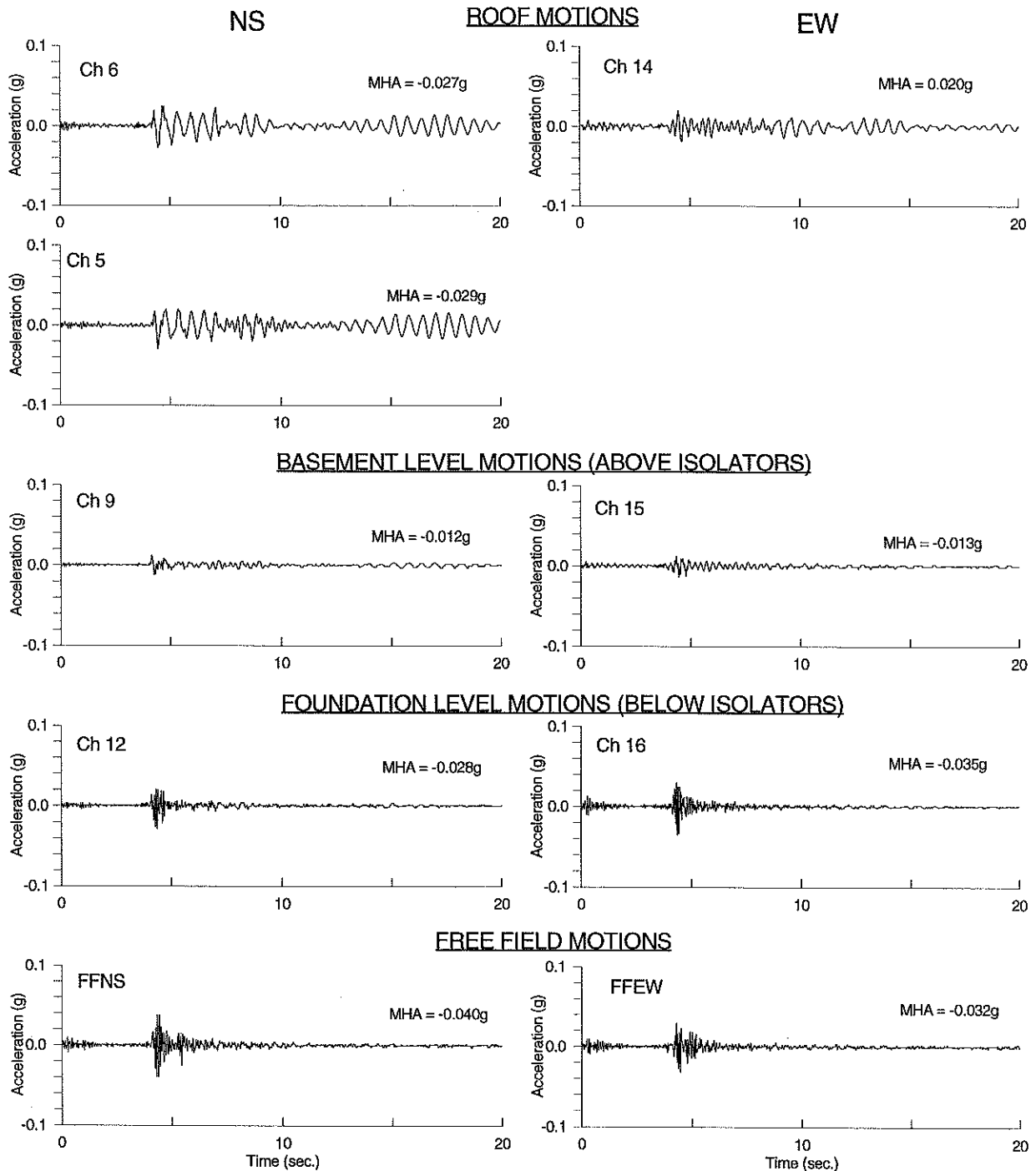


Fig. A37.RD1: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Horizontal Acceleration Time Histories
1985 Redlands Earthquake

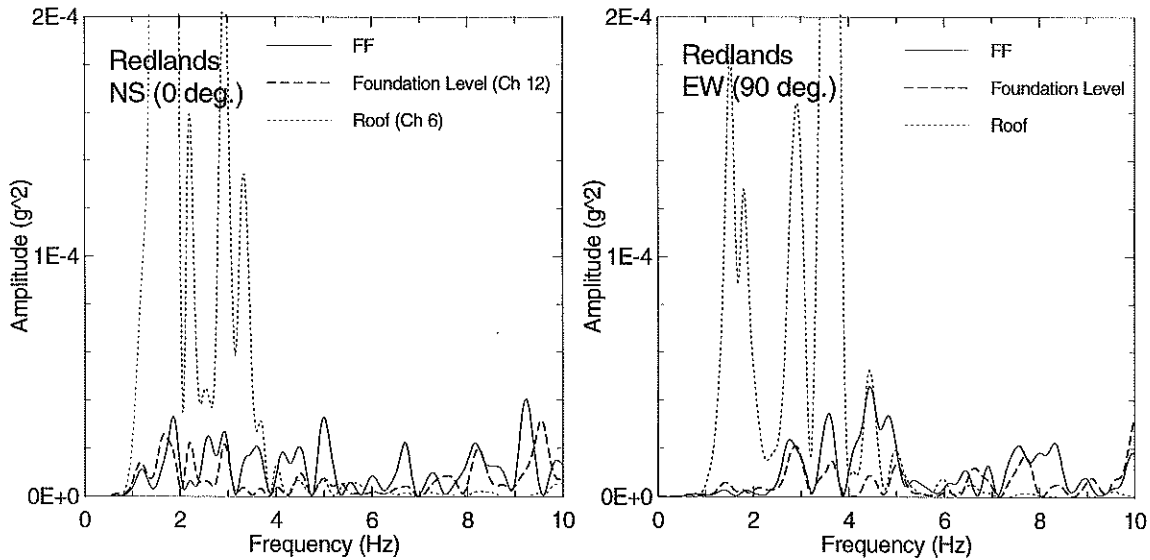


Fig. A37.RD2: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Power Spectral Density of Selected Horizontal Accelerations
1985 Redlands Earthquake

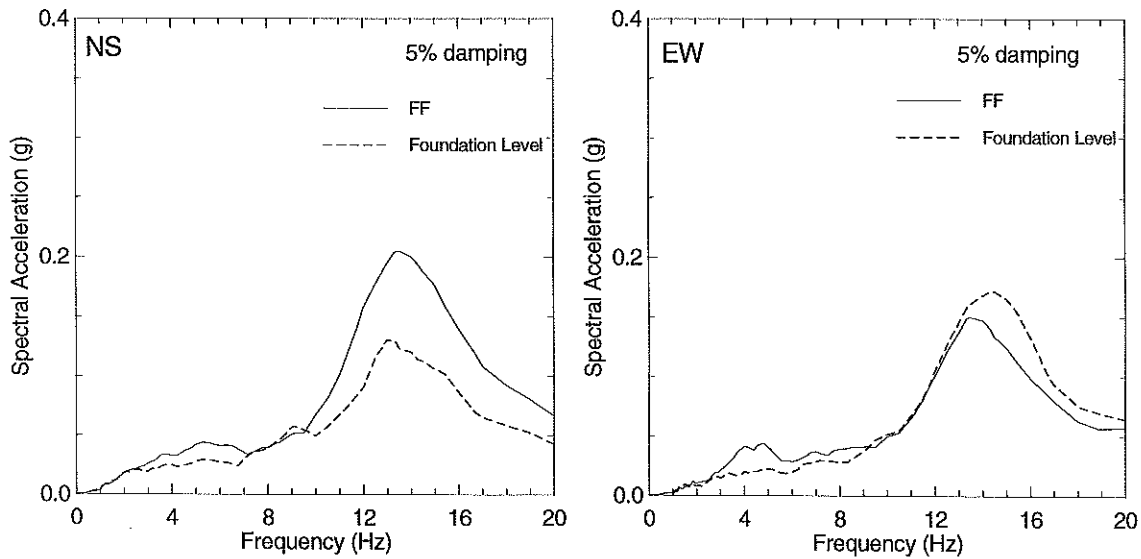


Fig. A37.RD3: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Acceleration Response Spectra of Foundation and Free-Field Motions
1985 Redlands Earthquake

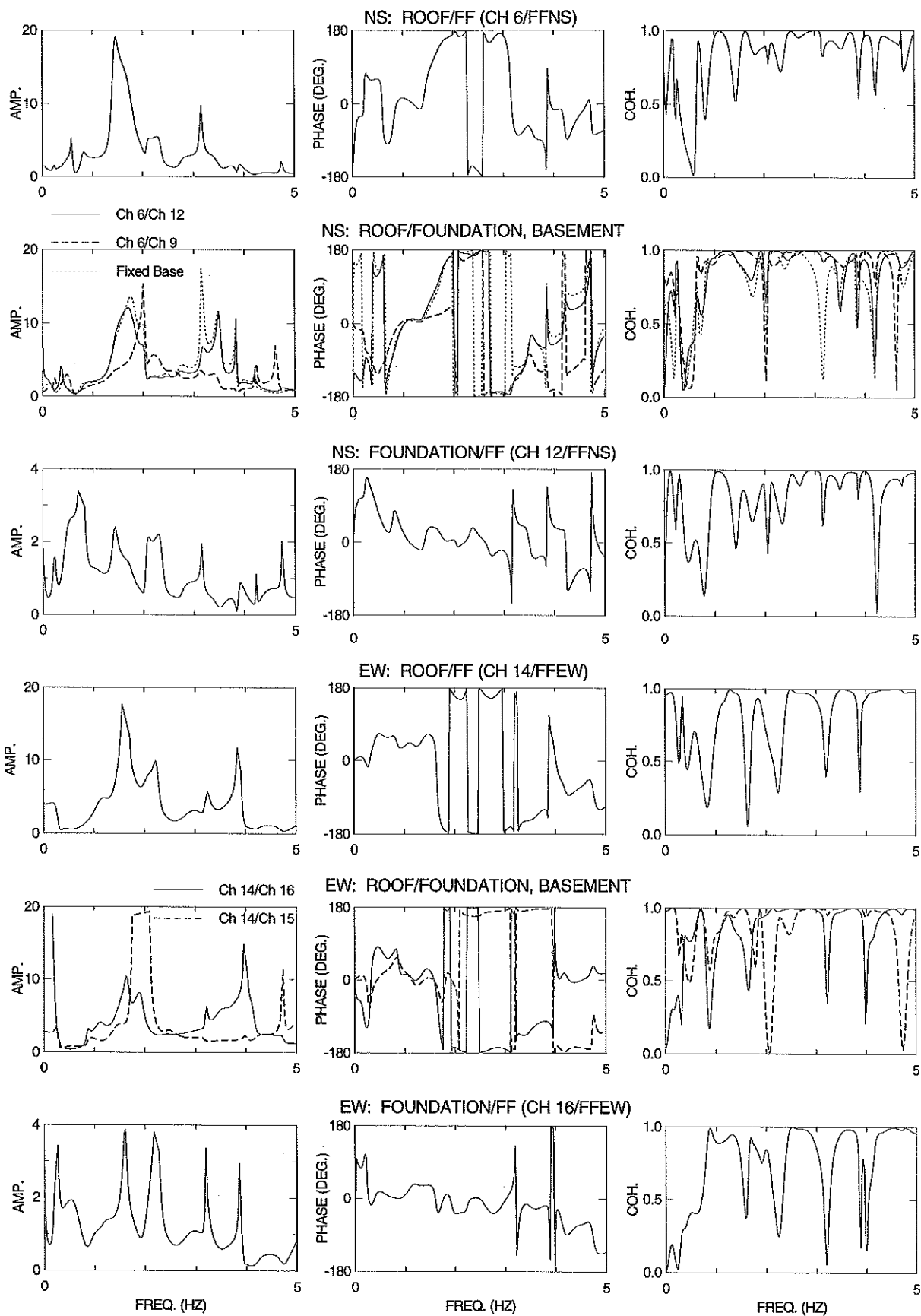


Fig. A37.RD4: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Transmissibility and Coherency Functions
1985 Redlands Earthquake
A37-11

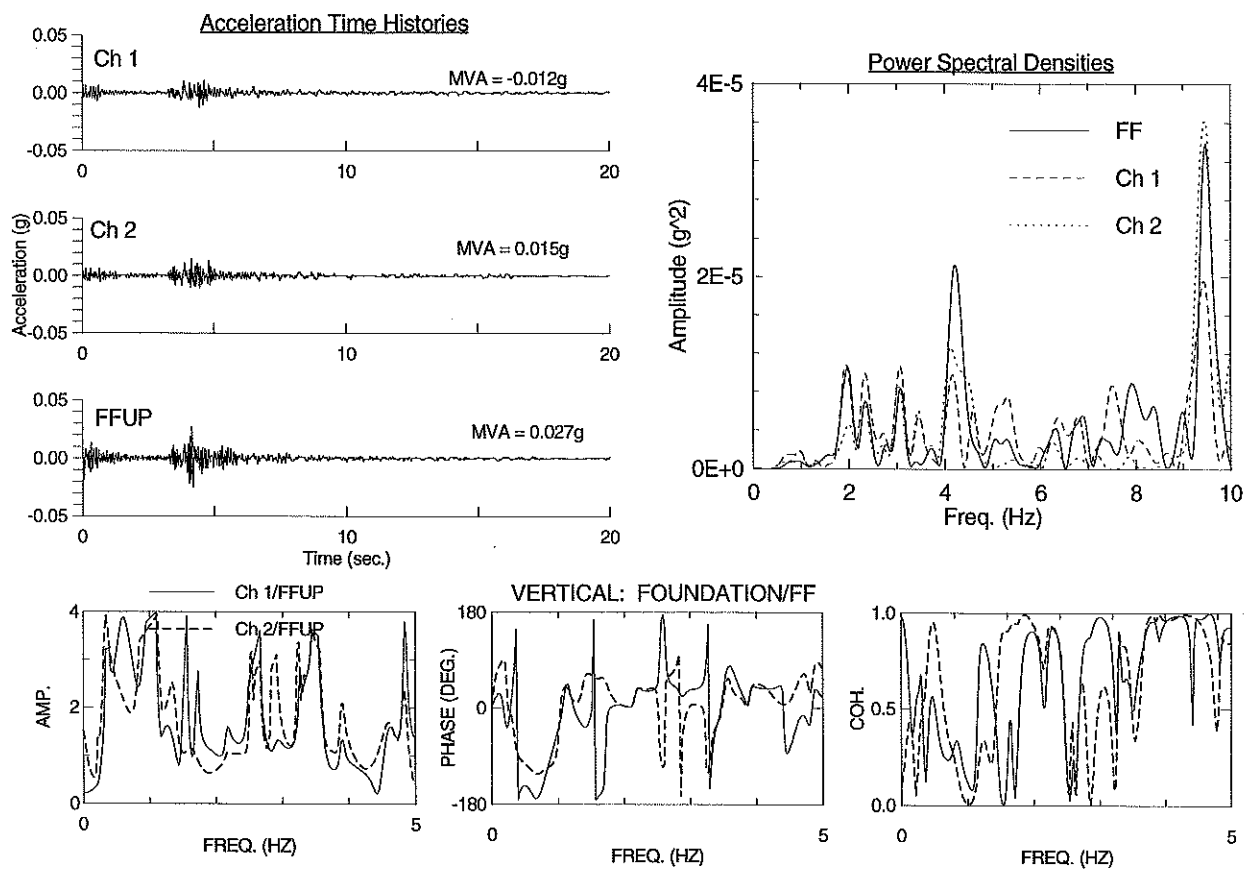


Fig. A37.RD5: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Analysis of Vertical Motions
1985 Redlands Earthquake

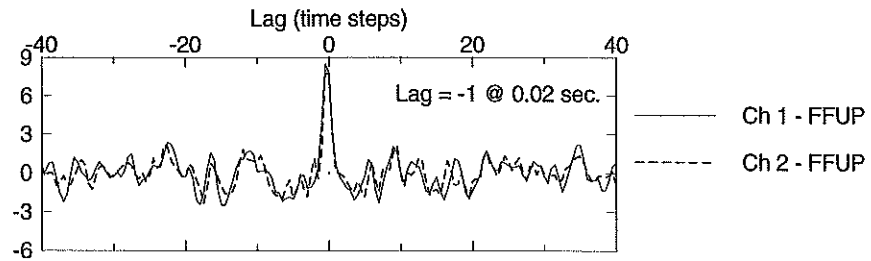


Fig. A37.WT0: Cross-correlation function between vertical foundation and free-field accelerations

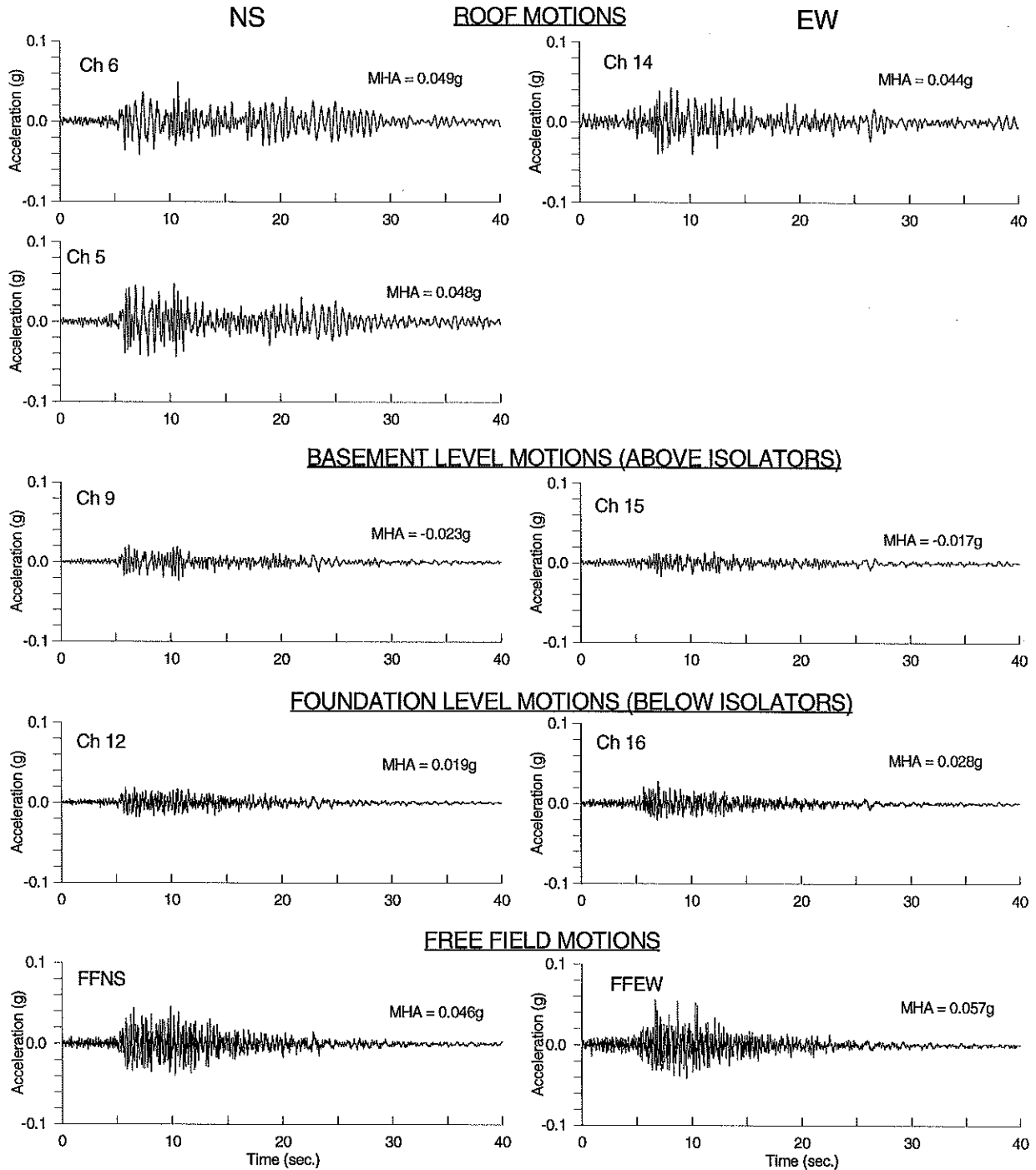


Fig. A37.WT1: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Horizontal Acceleration Time Histories
1987 Whittier Earthquake
A37-13

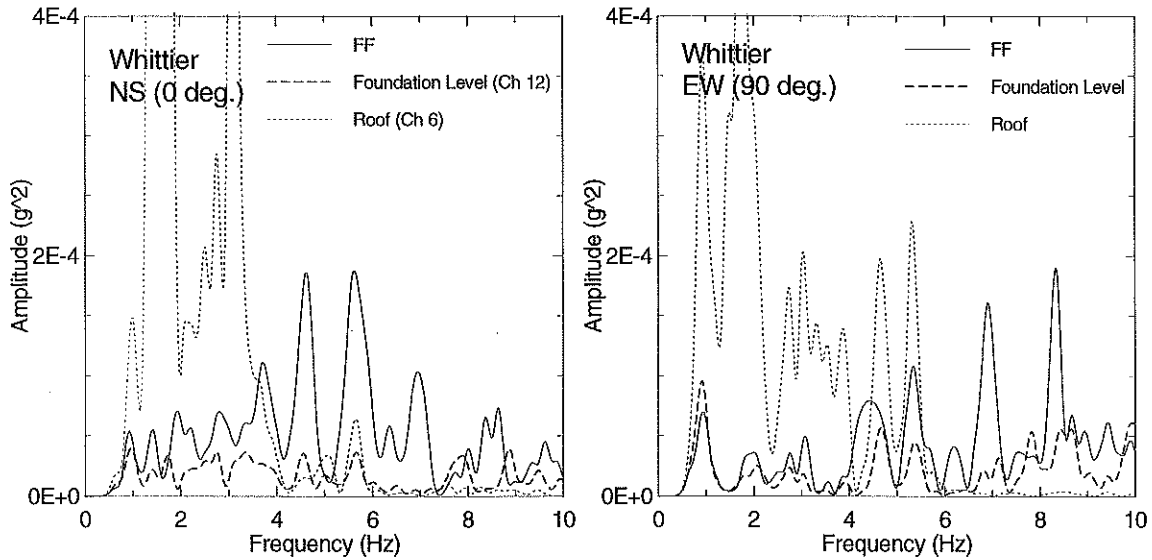


Fig. A37.WT2: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

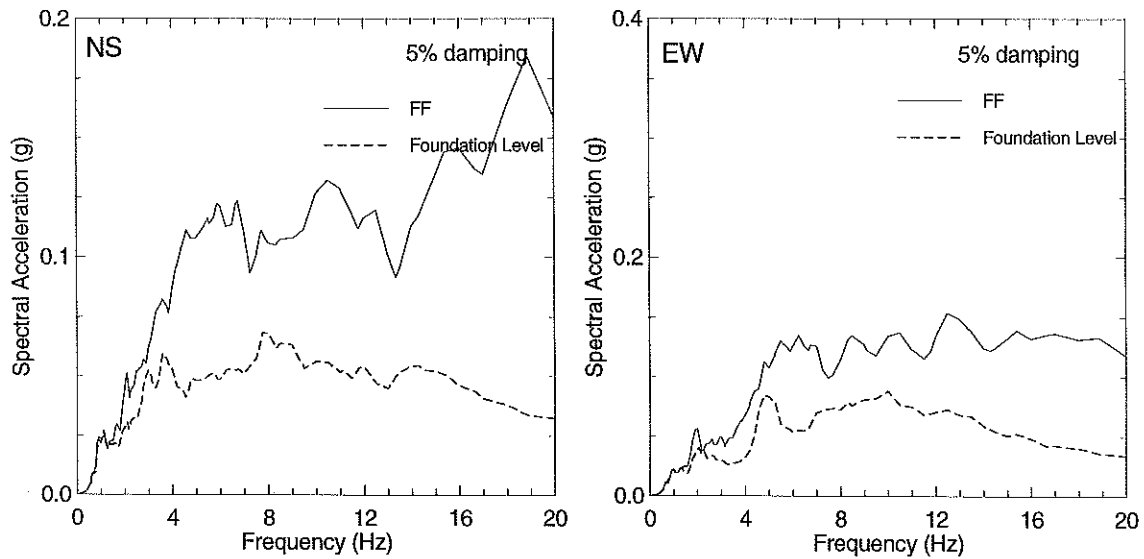


Fig. A37.WT3: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Acceleration Response Spectra of Foundation and Free-Field Motions
1987 Whittier Earthquake

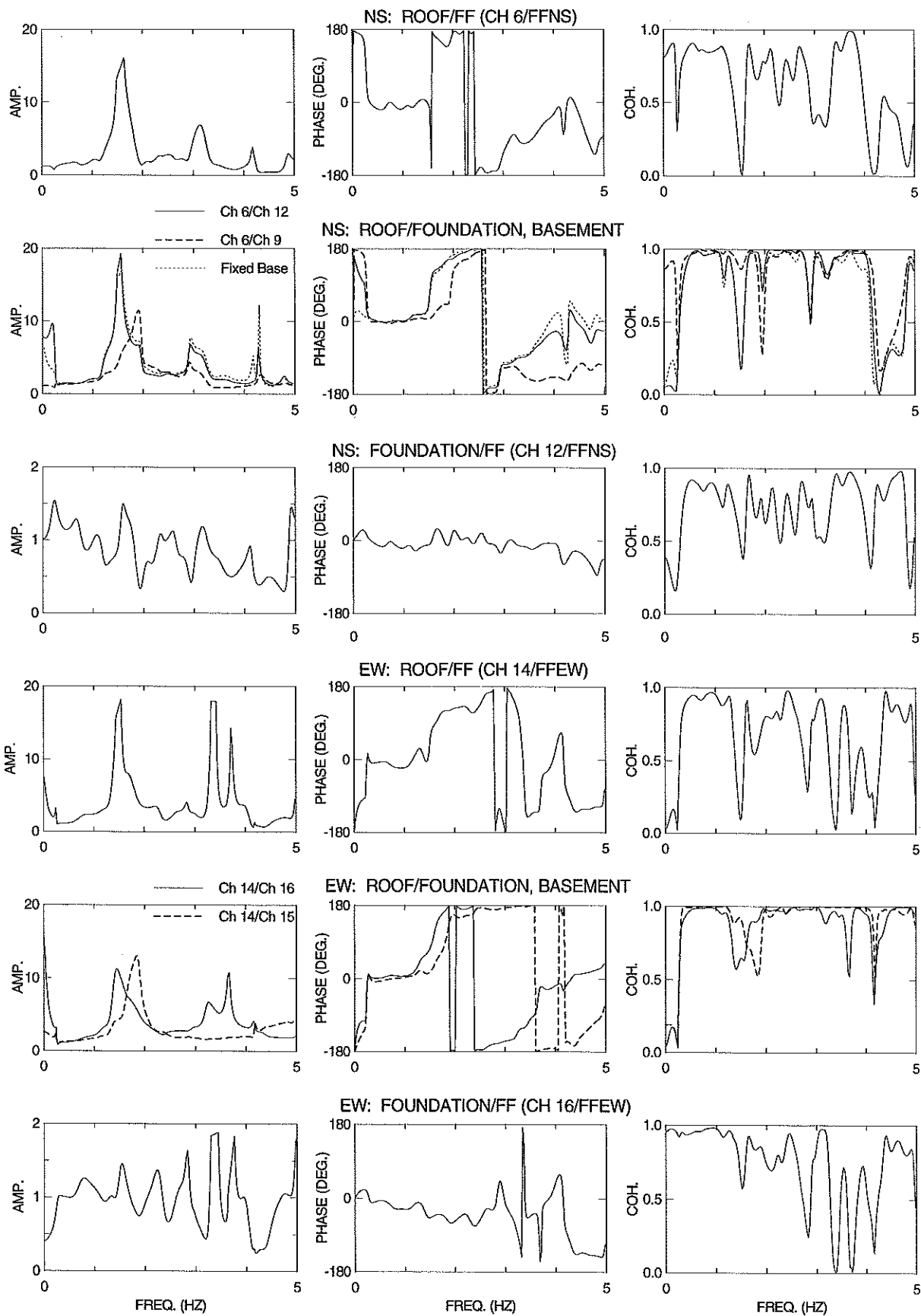


Fig. A37.WT4: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A37-15

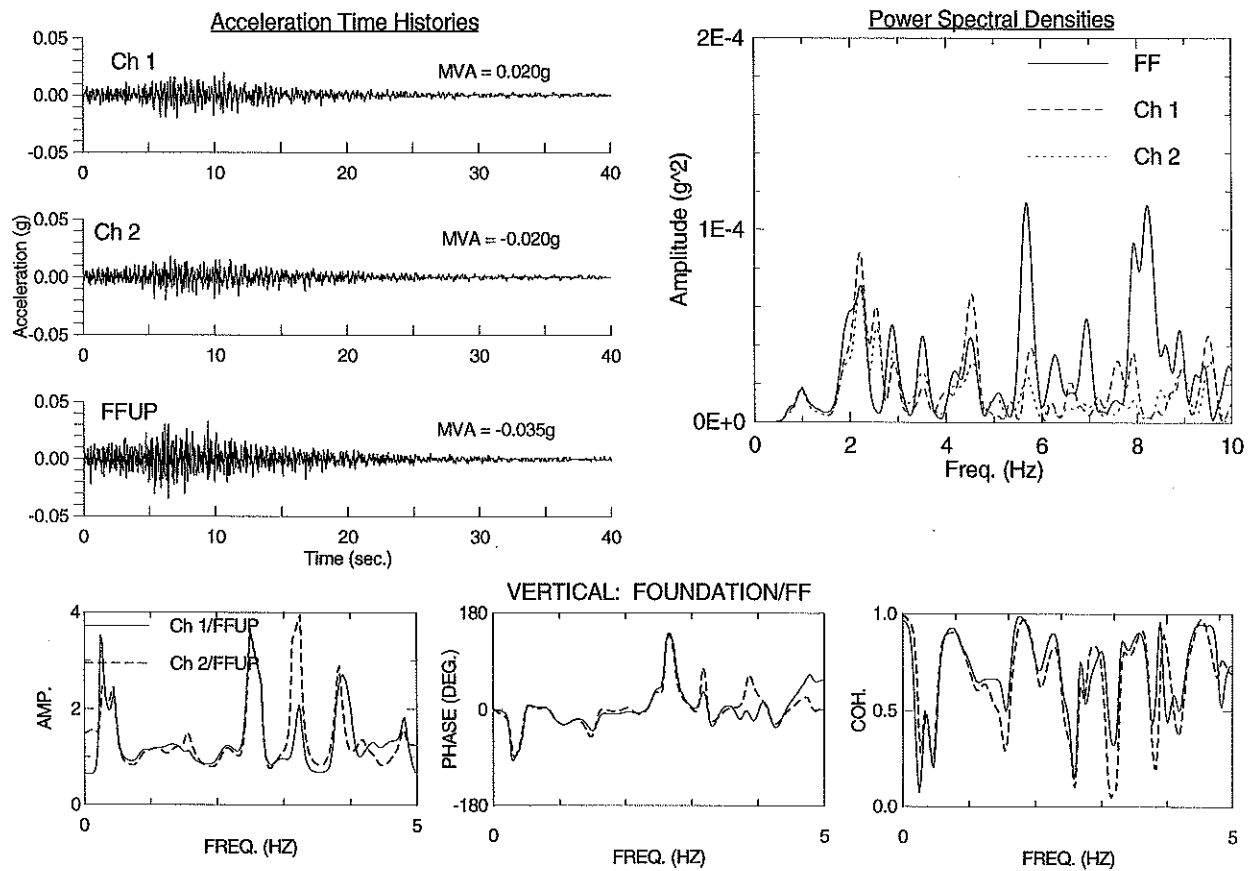


Fig. A37.WT5: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Analysis of Vertical Motions
1987 Whittier Earthquake

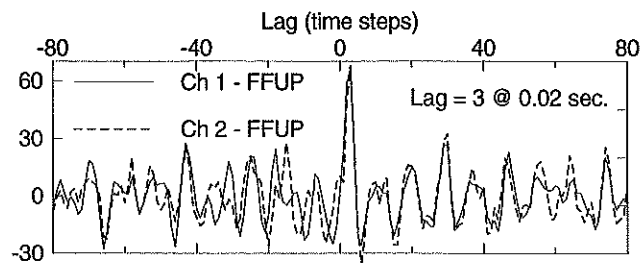


Fig. A37.UP0: Cross-correlation function between vertical foundation and free-field accelerations

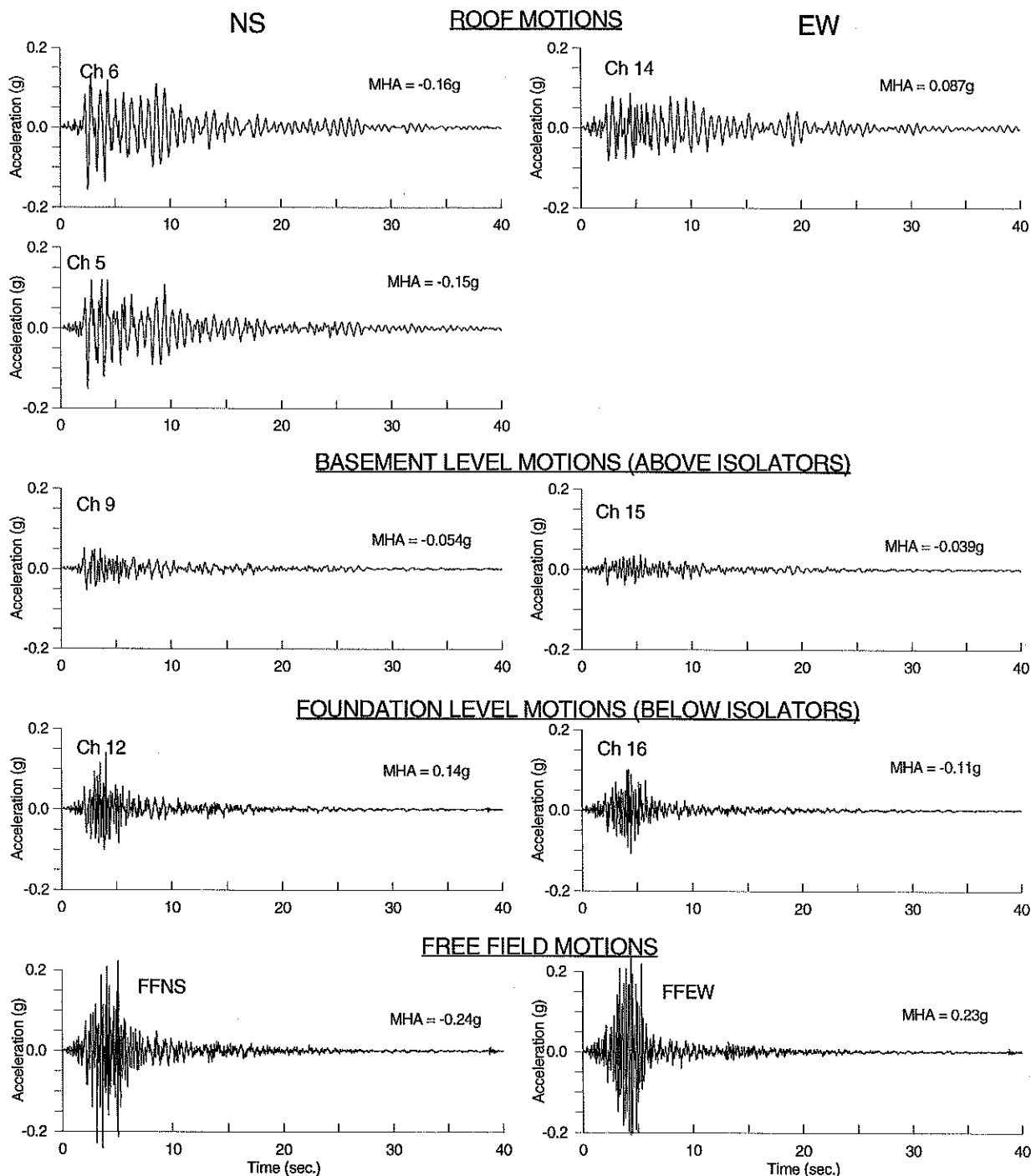


Fig. A37.UP1: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Horizontal Acceleration Time Histories
1990 Upland Earthquake

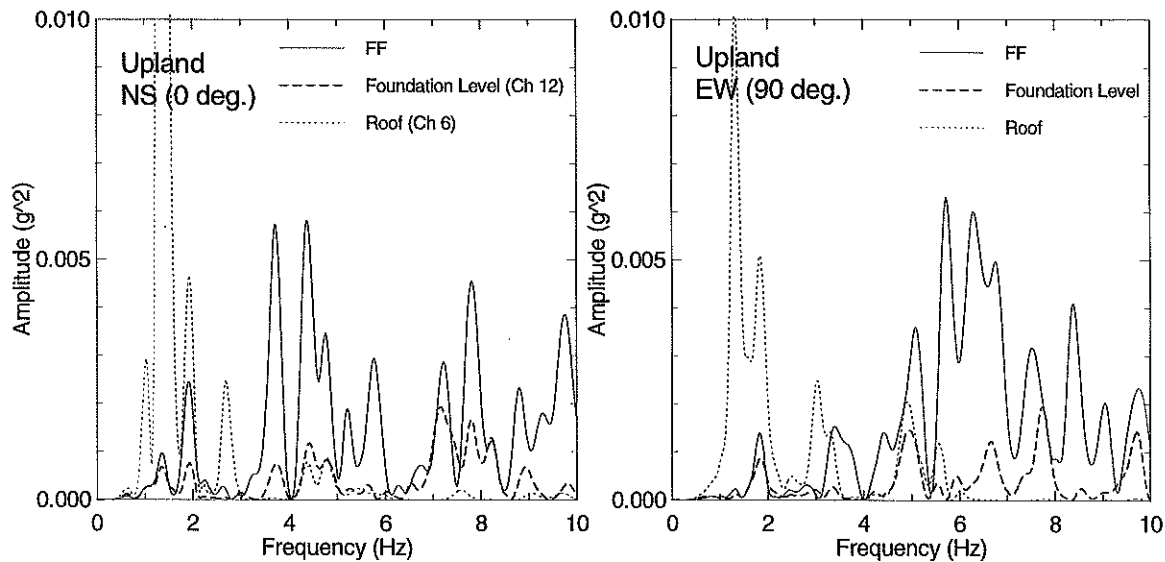


Fig. A37.UP2: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Power Spectral Density of Selected Horizontal Accelerations
1990 Upland Earthquake

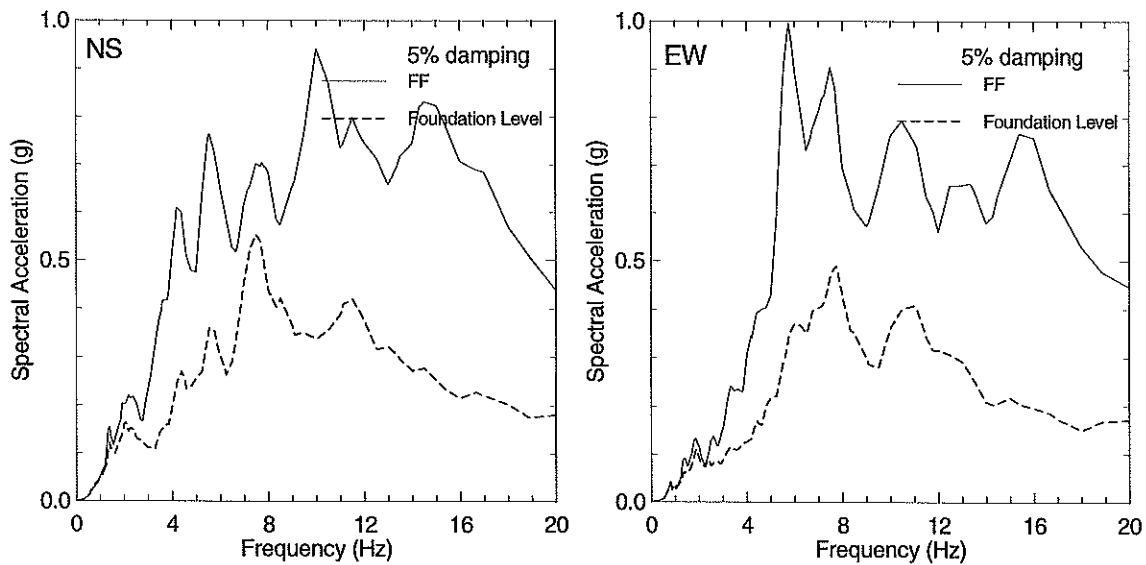


Fig. A37.UP3: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Acceleration Response Spectra of Foundation and Free-Field Motions
1990 Upland Earthquake

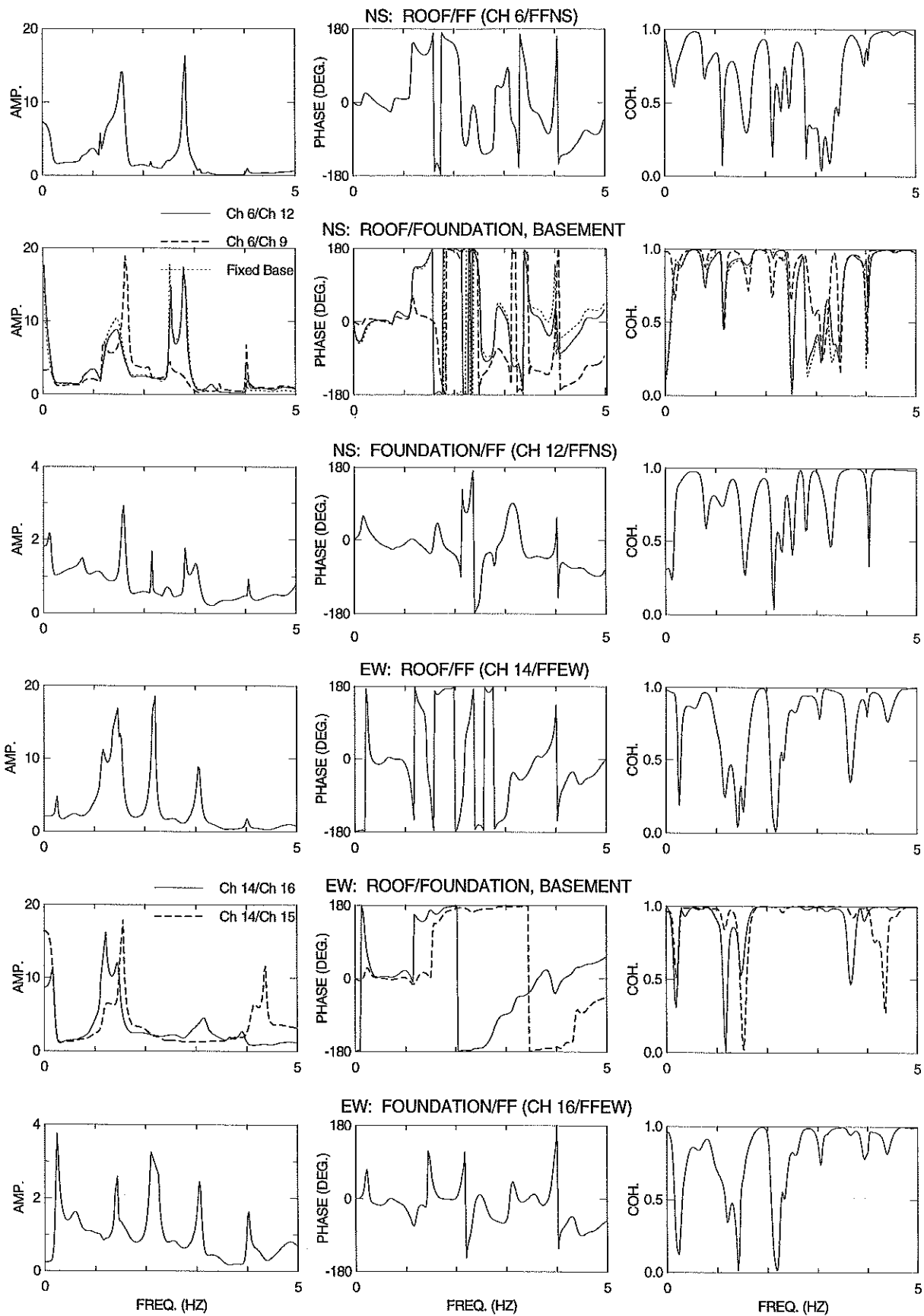


Fig. A37.UP4: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Transmissibility and Coherency Functions
1990 Upland Earthquake
A37-19

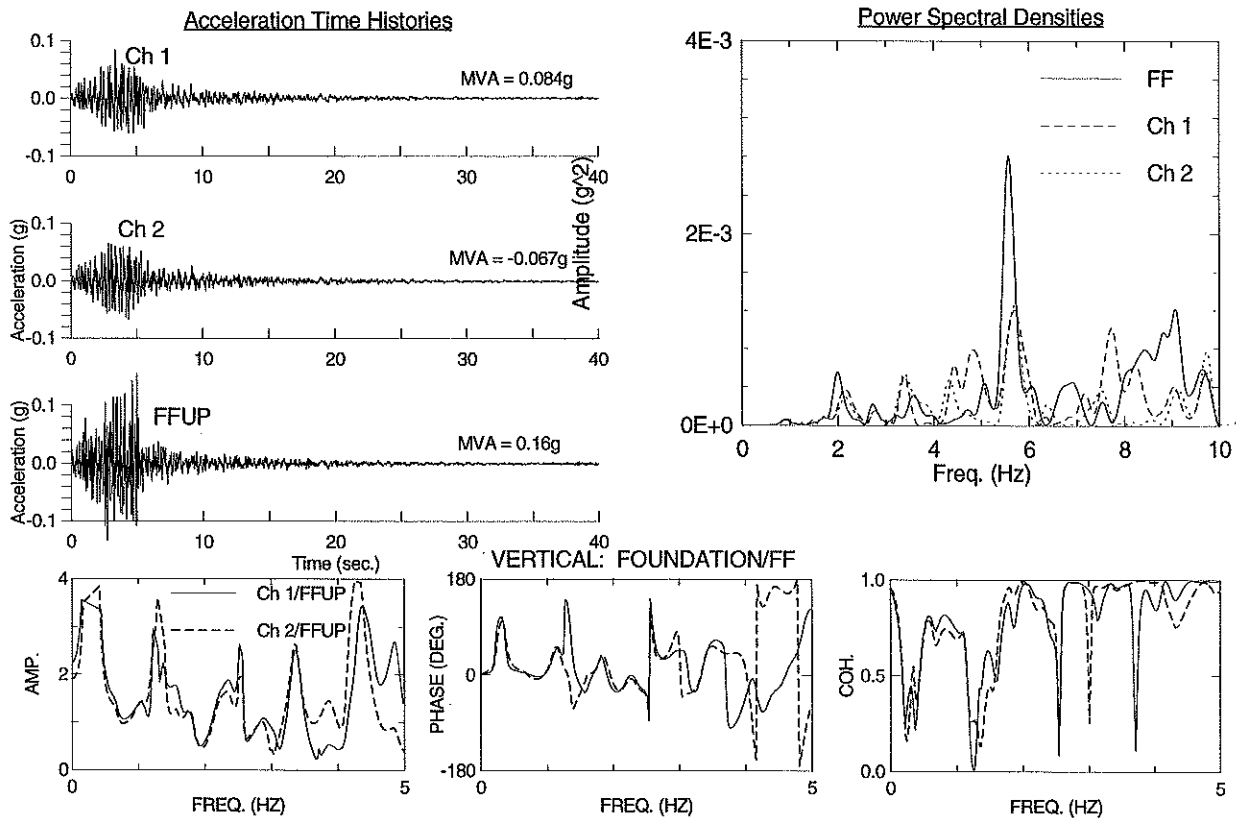


Fig. A37.UP5: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Analysis of Vertical Motions
1990 Upland Earthquake

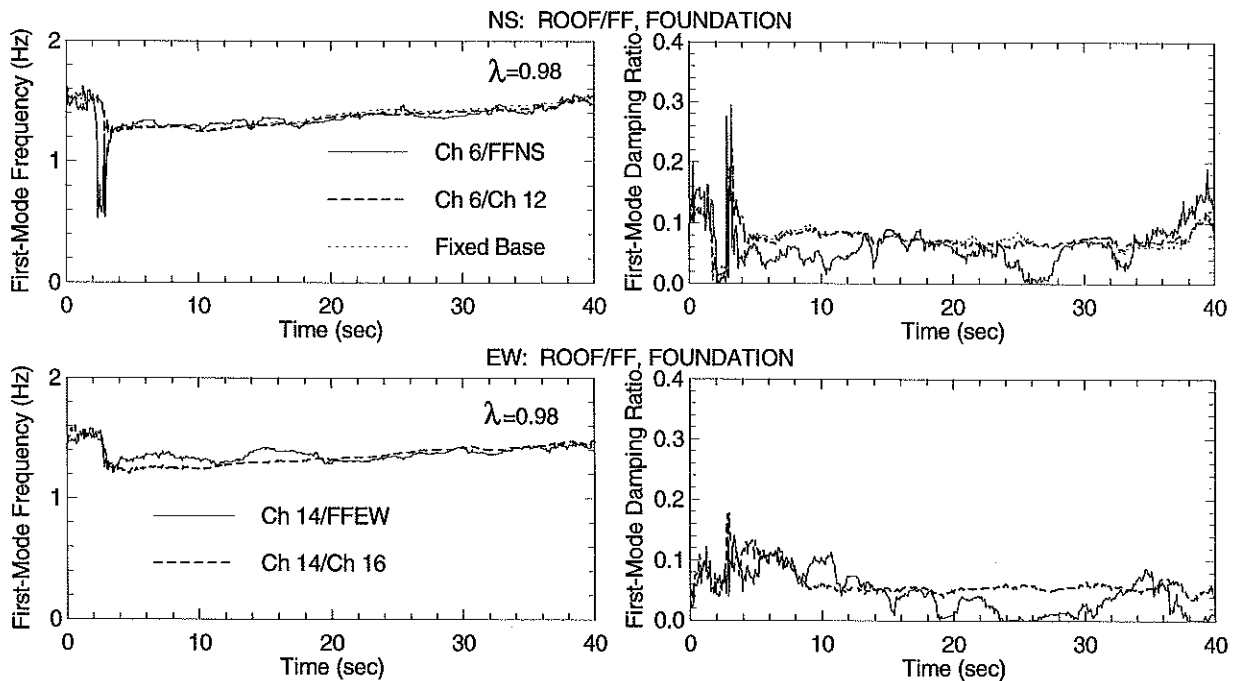


Fig. A37.UP6: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Time Variation of First-Mode Parameters
1990 Upland Earthquake

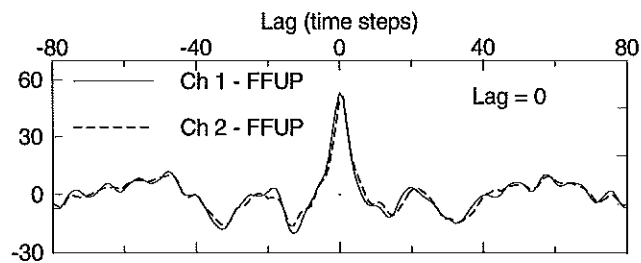


Fig. A37.LD0: Cross-correlation function between vertical foundation and free-field accelerations

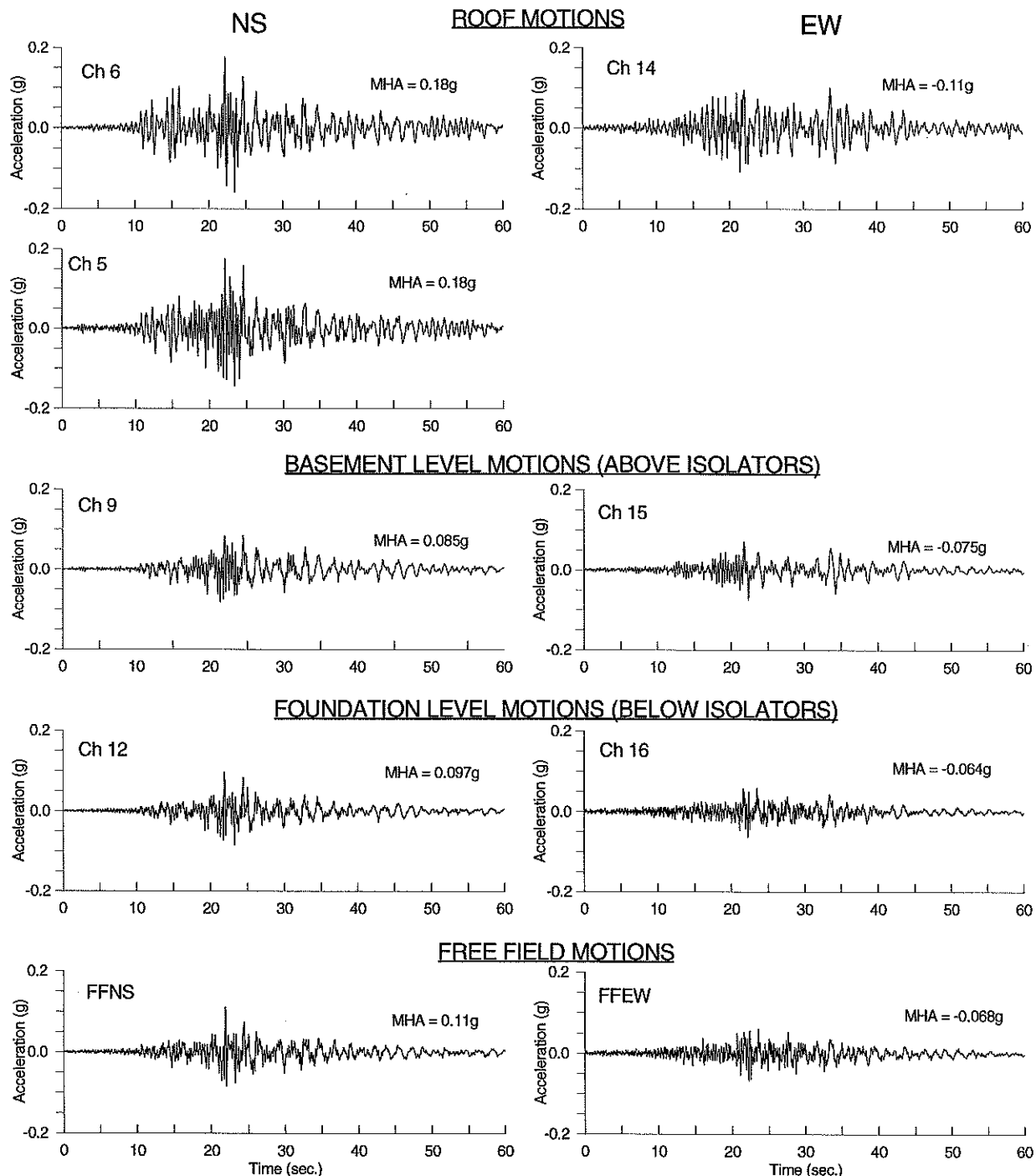


Fig. A37.LD1: RANCHO CUCAMONGA LAW AND JUSTICE CENTER

Horizontal Acceleration Time Histories

1992 Landers Earthquake

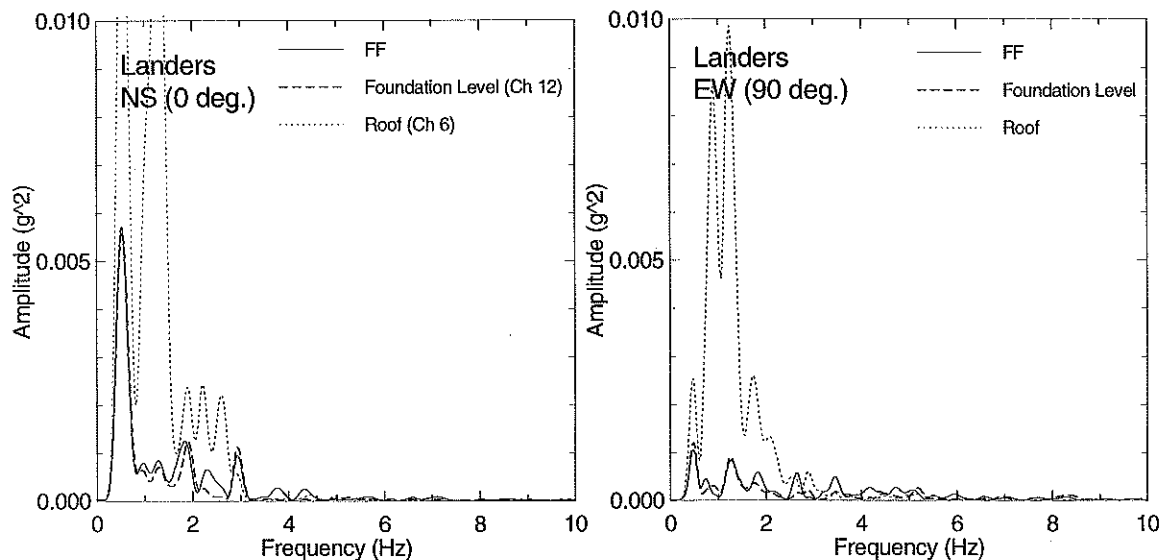


Fig. A37.LD2: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

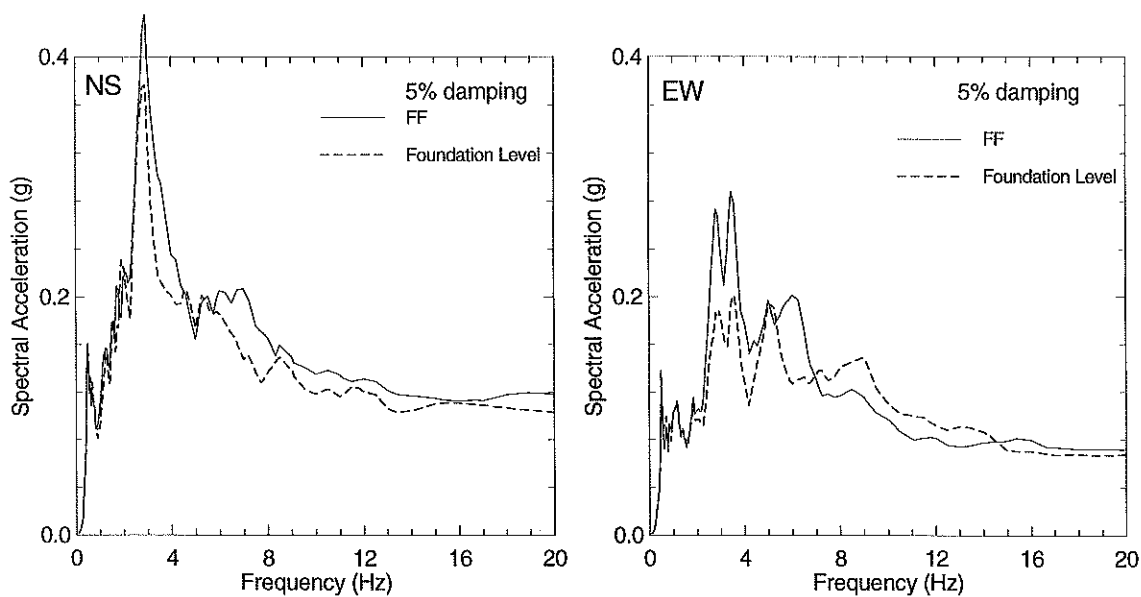


Fig. A37.LD3: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Acceleration Response Spectra of Foundation and Free-Field Motions
1992 Landers Earthquake

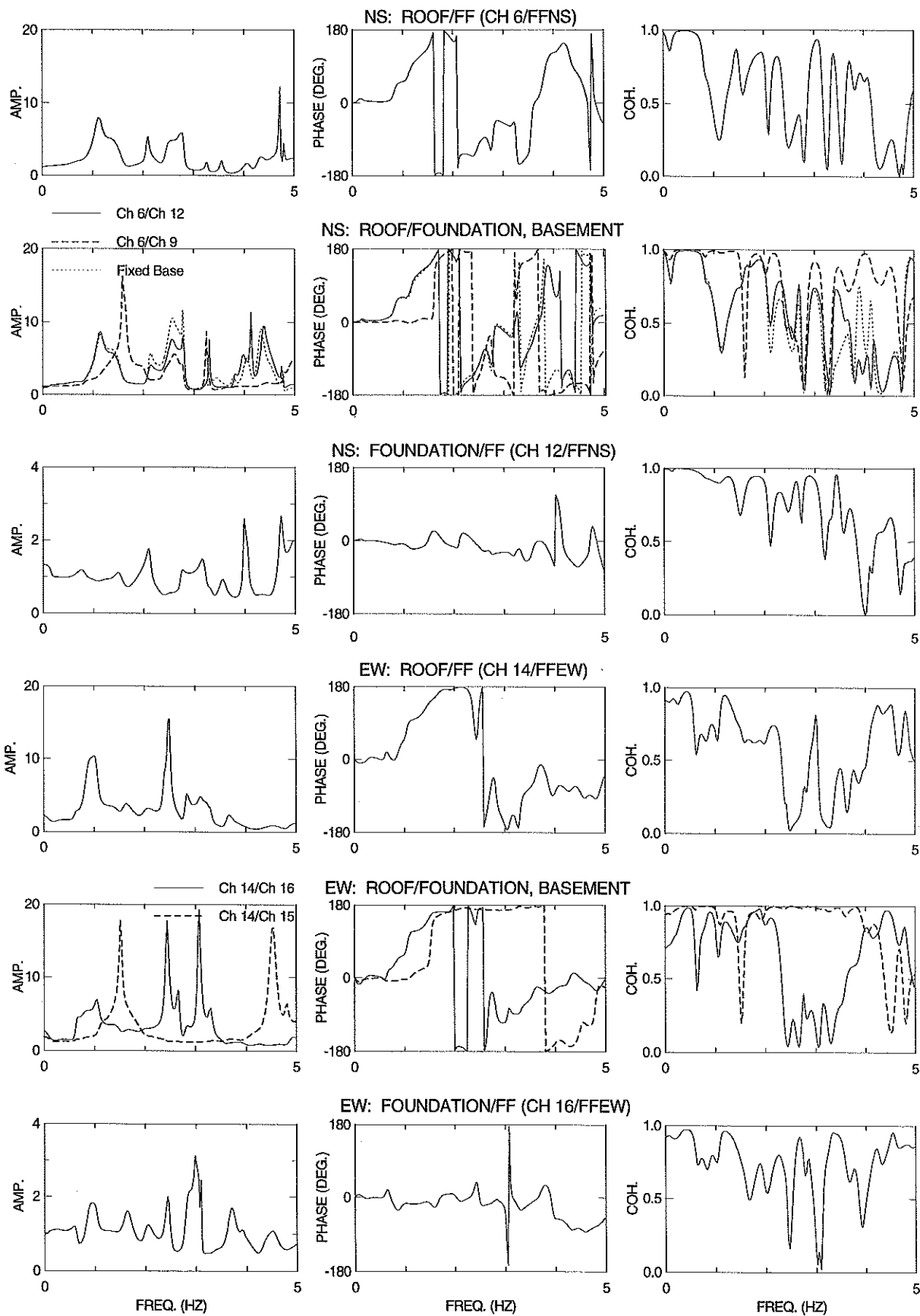


Fig. A37.LD4: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Transmissibility and Coherency Functions
1992 Landers Earthquake
A37-23

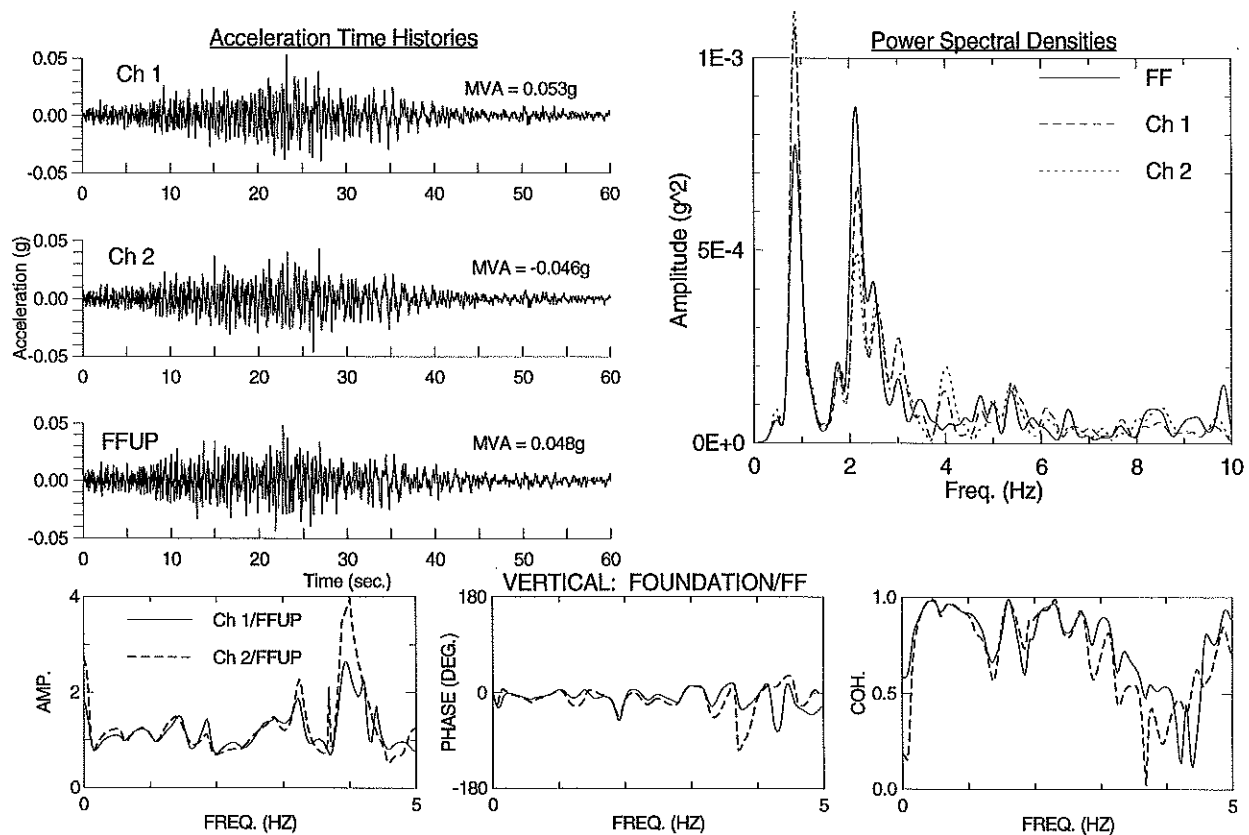


Fig. A37.LD5: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Analysis of Vertical Motions
1992 Landers Earthquake

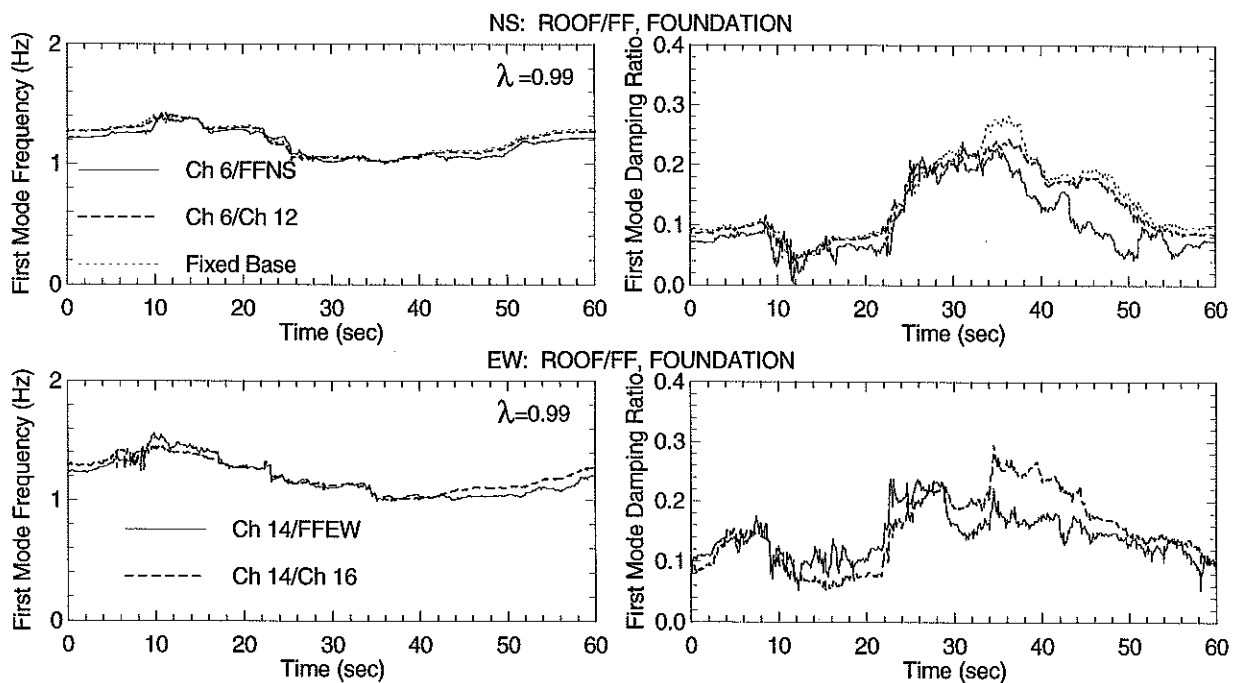


Fig. A37.LD6: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Time Variation of First Mode Parameters
1992 Landers Earthquake

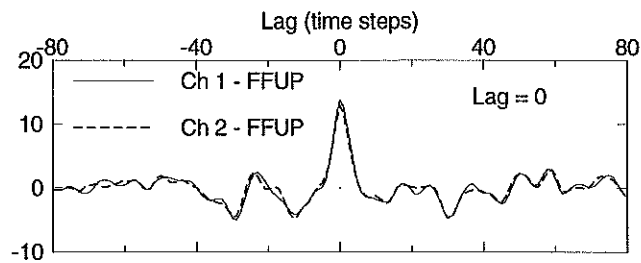


Fig. A37.NR0: Cross-correlation function between vertical foundation and free-field accelerations

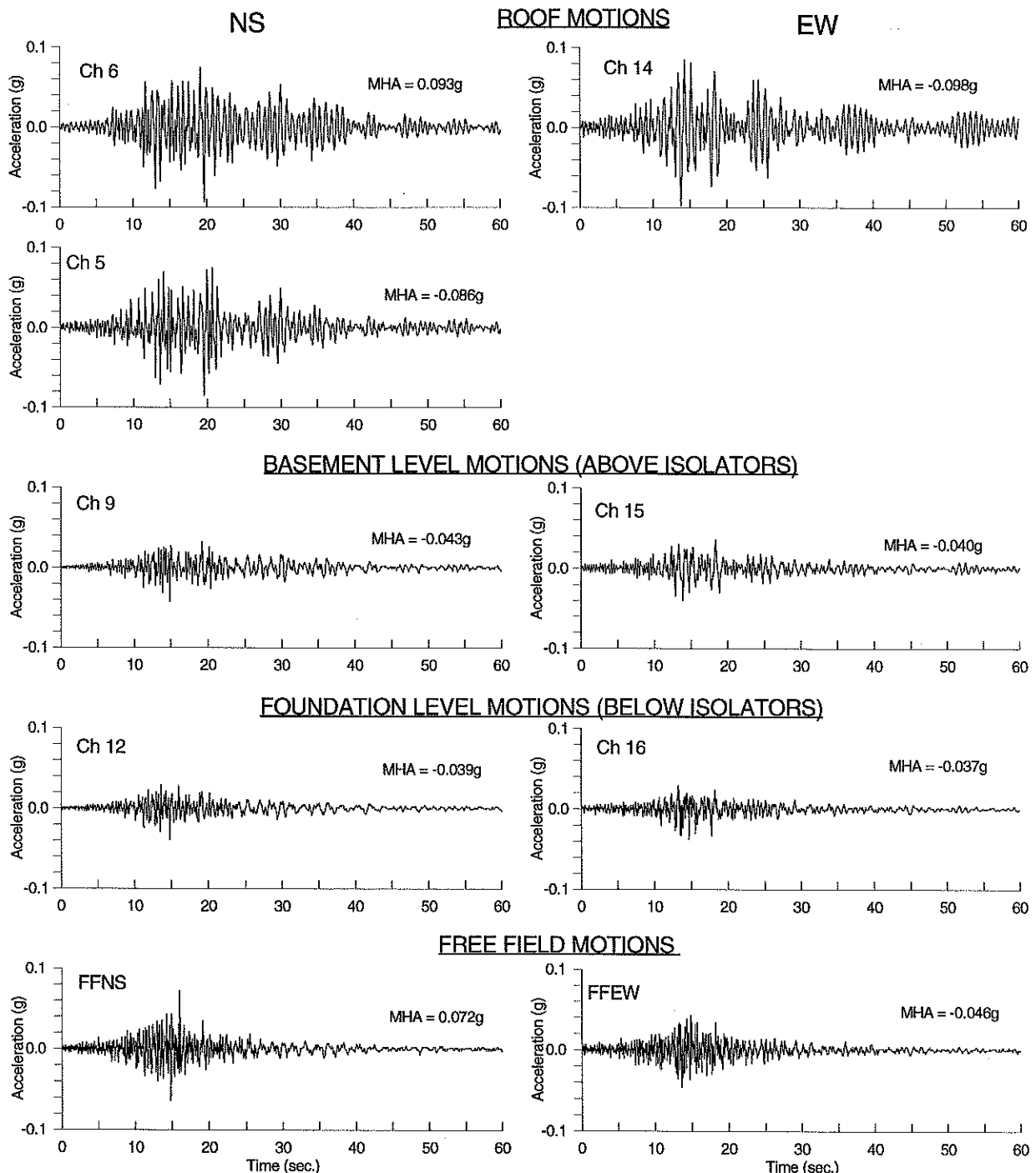


Fig. A37.NR1: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

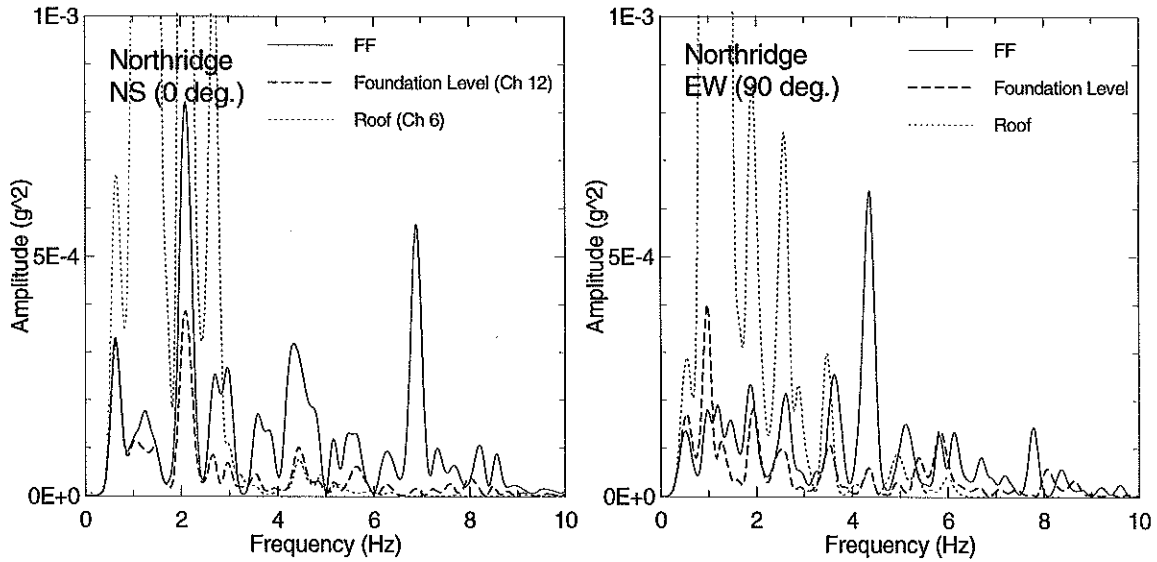


Fig. A37.NR2: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

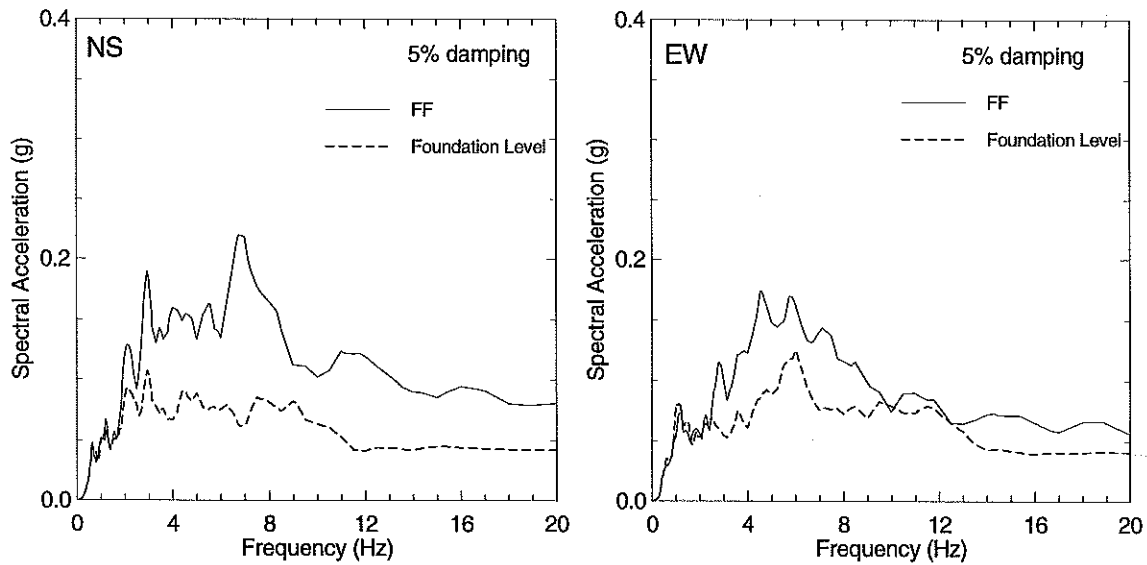


Fig. A37.NR3: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Acceleration Response Spectra of Foundation and Free-Field Motions
1994 Northridge Earthquake

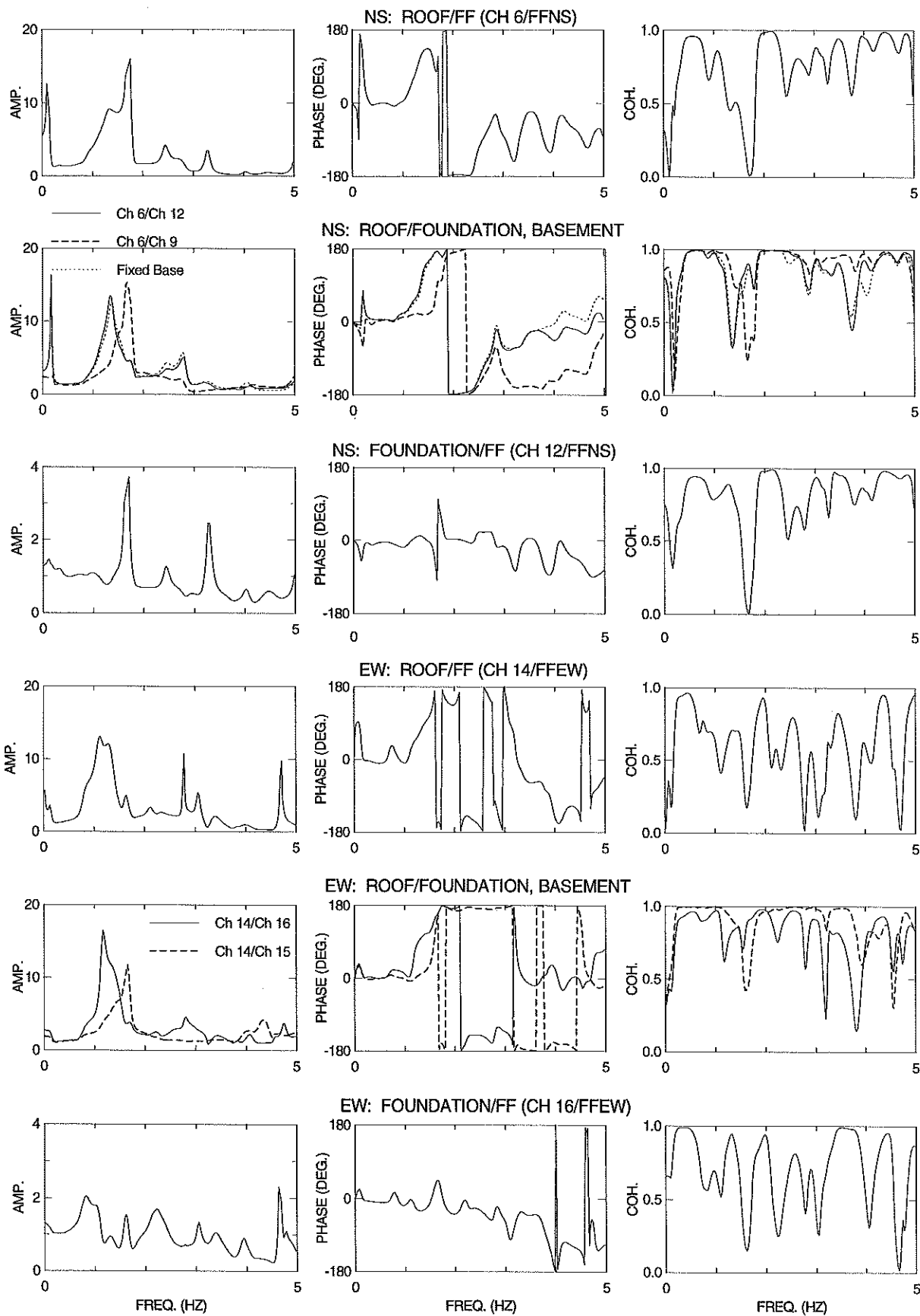


Fig. A37.NR4: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A37-27

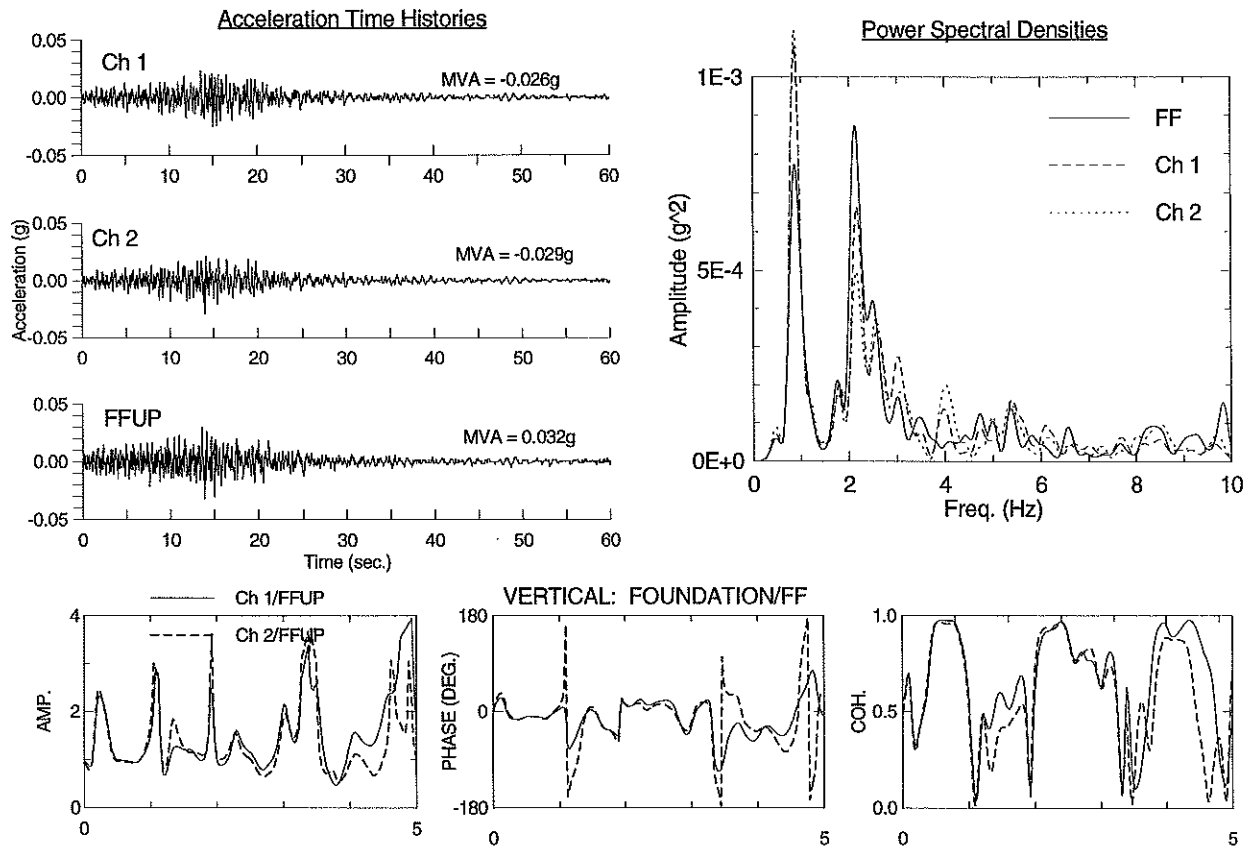


Fig. A37.NR5: RANCHO CUCAMONGA LAW AND JUSTICE CENTER
Analysis of Vertical Motions
1994 Northridge Earthquake

SITE A38

Site Name: San Bernardino 3 Story Building
Sensors: CSMIP 23516/23542

Lat: 34.065°
Long.: 117.289°

Floor Levels: 3
Bsmt. Levels: 0
Structure: Moment-resisting steel frames

Foundation: footings
Site Geology: soil
Design Date: 1983

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers	0.087	0.078	0.065	0.083	0.12	0.060	0.25	0.28	--
1992 Big Bear *	0.09	0.10	0.07	0.09	0.13	0.06	0.40	0.28	--
1994 Northridge *	0.10	0.08	0.04	0.06	0.08	0.03	0.14	0.20	--

* Not processed

-- = no instrument

Preprocessing

Landers Earthquake (LD)

Preliminary: *Str. Data*: digitized and processed in this study, filtered, $f_{HP} = 0.13-0.23$ Hz, $f_{LP} = 20-30$ Hz, $f_{Nyq} = 100$ Hz. *FF Data*: CSMIP, filtered, $f = 0.2-46$ Hz; $f_{Nyq} = 50$ Hz; FF/Str. data not synchronized.

Final: FF/Str. data synchronized using cross correlation of vertical accelerations (Fig. A38.LD0), FF data decimated by 4, str. data decimated by 2, to $f_{Nyq} = 25$ Hz.

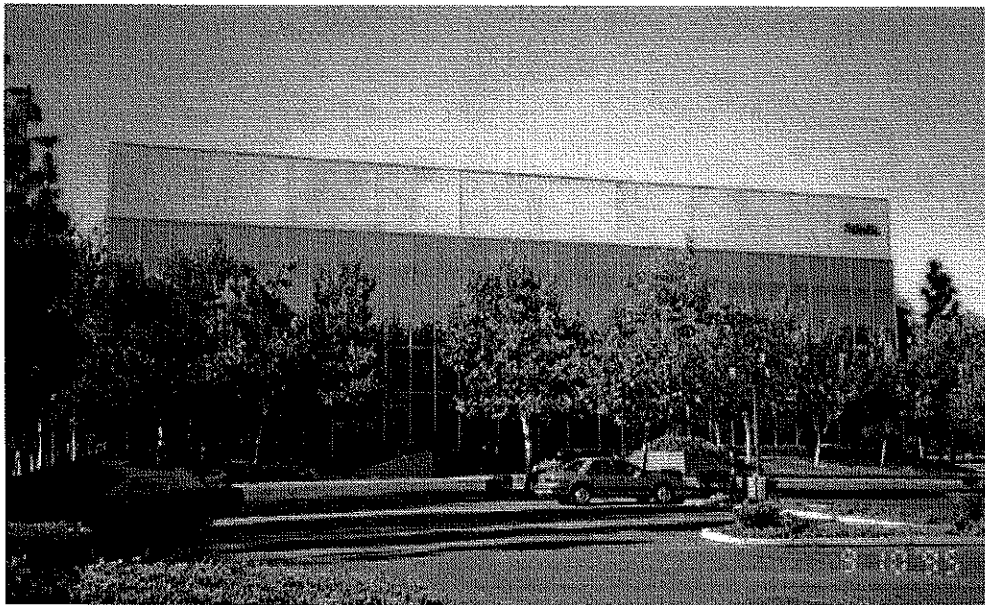
Figures

A38.1-3

Site Plan, Sensor Locations, Generalized Soil Column

A38.LD0-6

LD Analysis Results



View of structure, looking north

DISCUSSION

Site Description

The building is 3 stories tall above ground level with no basement, and is rectangular in plan (132 by 144 feet). There are no significant structures located near the building. The lateral load resisting system consists of perimeter moment resisting steel frames. The columns are supported by footings with lateral dimensions of 5.5 by 7.0 feet and founded 3 feet below the ground surface. The footings are not tied together with tie beams, though a 4-inch slab is present across the ground floor level. As shown in Fig. A38.2, accelerographs are present on the ground, 2nd, 3rd, and roof levels of the structure. The free-field instrument is located about 685 feet west of the structure in a parking lot.

The site is located on level terrain in the San Bernardino Valley, about 1200 feet south of the Santa Ana River. Near-surface soil conditions at the site have been investigated by Pioneer Consultants (1980). As shown in Fig. A38.3, borings drilled to a depth of 50 feet encountered 25 feet of interbedded silt, clay, and sand, and silty sand which extends to the maximum depth explored of 50 feet. These soils appear to be alluvial sediments derived from the nearby river. The shear wave velocities shown in Fig. A38.3 were estimated using correlations with void ratio and SPT blow count.

Instrument Selection and Free-Field Conditions

Roof sensors on the south moment frame were used in the analyses in combination with ground floor sensors in the middle of the building (Table A38.1). Acceleration time histories recorded by these sensors are shown in Fig. A38.LD1.

Table A38.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 13	Ch 9	Ch 1	Ch 10	Ch 2

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A38.LD4) and the spectra of the horizontal motions (Fig. A38.LD2-3) indicate generally increasing de-amplification of GF motions with frequency in both directions.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A38.LD5. No significant amplification was observed near the first-

mode lateral response frequencies, suggesting that rocking was unlikely to have significantly contributed to the structural response. It should be noted, however, that the vertical GF instrument is not well positioned to detect motions from base rocking.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A38.LD4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A38.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A38.LD6).

Table A38.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW)		L (NS)		tr (EW)		L (NS)		tr (EW)		L (NS)	
		LD: estimated		estimated		d=0, j=5		d=3, j=5		d=0, j=5		d=2, j=5	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	1.93	7.6	1.83	10.8	1.90	7.3	1.82	10.5	1.77	5.7	1.77	7.7
	2					5.59	10.4	6.07	13.8	--	--	--	--

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode. Higher modes could not be reliably identified for the FF-R pairs.
- The small differences between the fixed- and pseudo flexible-base parameters is consistent with the lack of evidence for a significant rocking response in the vertical GF/FF transmissibility function.

The RPEM results (Fig. A38.LD6) indicate gradual decreases in first-mode frequencies for both directions [i.e. about 2.3 to 1.8 Hz (tr) and 2.2 to 1.8 Hz (L)] during the build-up of ground shaking amplitude (i.e. the first 20 sec.). The RPEM results generally confirmed the CEM results with regard to the relative magnitudes of the FF-R and GF-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A38.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 78 feet using the recorded free-field motions and the profile in Fig. A38.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A38.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 980$ fps	$h = 29$ ft	emb.=0	$\gamma = 0.15$
LD	$\nu = 0.33$		tr: $r_1 = 78$ ft		$r_2 = 77$ ft
	$\beta^* = 3.8\%$	$V_s^* = 880$ fps	L: $r_1 = 78$ ft		$r_2 = 80$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A38.3 with the fixed-base modal parameters in Table A38.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A38.4.

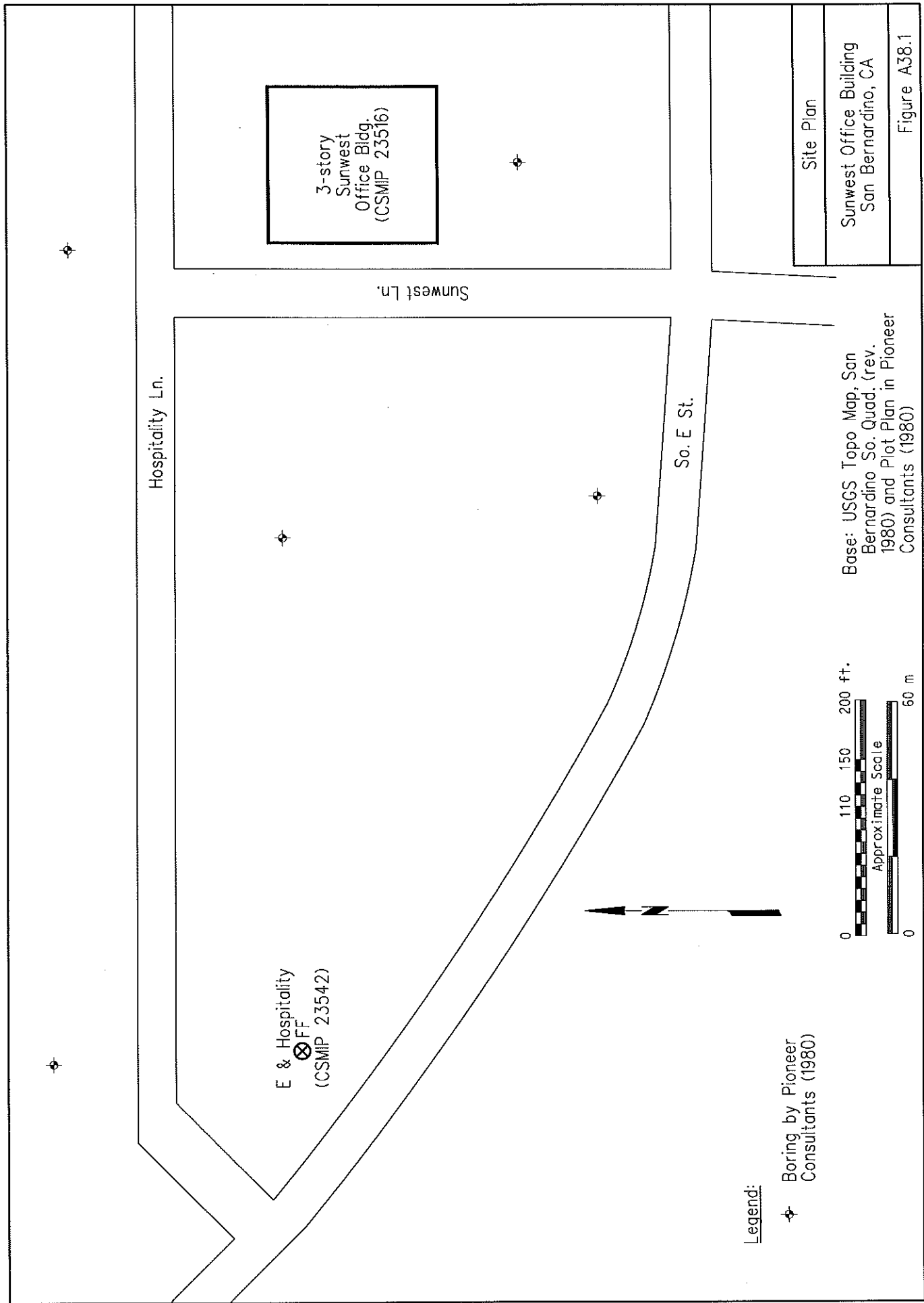
Table A38.4: Empirical and predicted inertial interaction effects

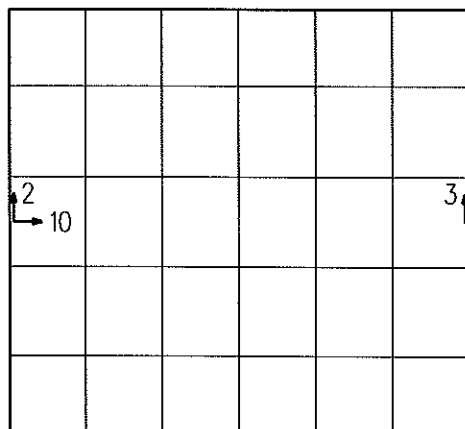
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
LD	Sys. ID	1.09	-0.1	5.7	1.03	-2.1	7.7
	Prediction	1.02	1.1	8.2	1.03	1.3	11.3

The period lengthening is well predicted for the longitudinal direction but underpredicted for the transverse direction. As the structural configuration is similar in both directions, the different period lengthenings computed for these directions by the system identification analyses may be erroneous. The negative foundation damping ratios from the system identification analyses indicate numerical errors in these results. However, the predictive analyses appear to have evaluated reasonably accurately the low foundation damping at this site.

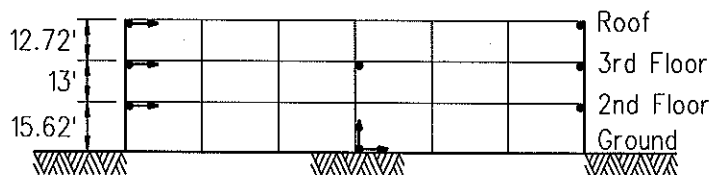
Data Sources

Pioneer Consultants (1980). "Soil and Foundation Investigation, Sunwest Development, San Bernardino, CA."

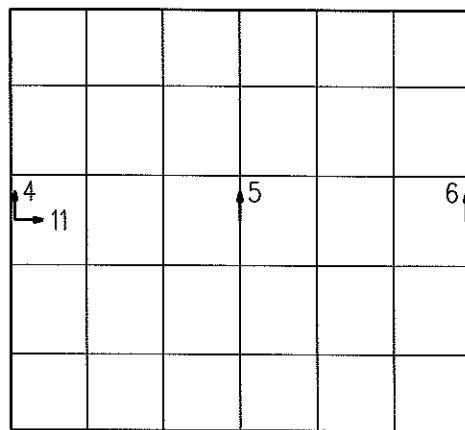




Roof Plan



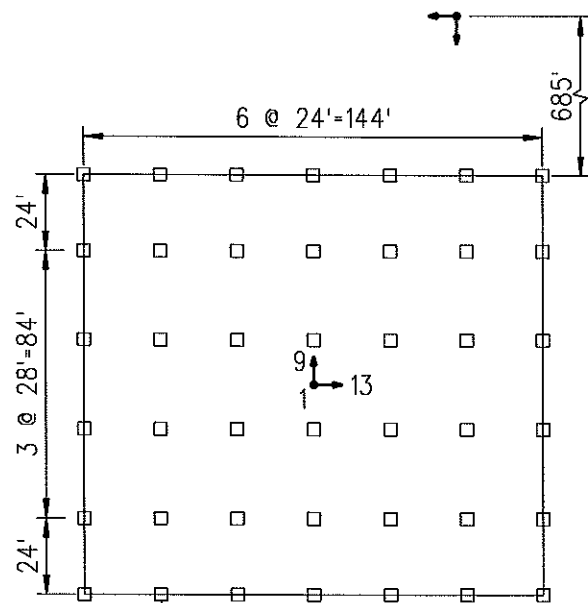
S / N Elevation



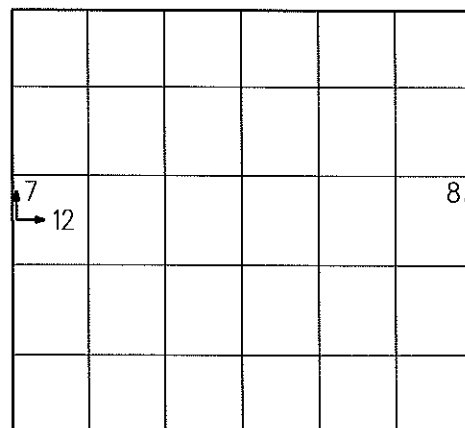
3rd Floor Plan

Perimeter moment resisting steel frames (typ)

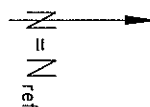
E & Hospitality Free Field



Ground Level Plan



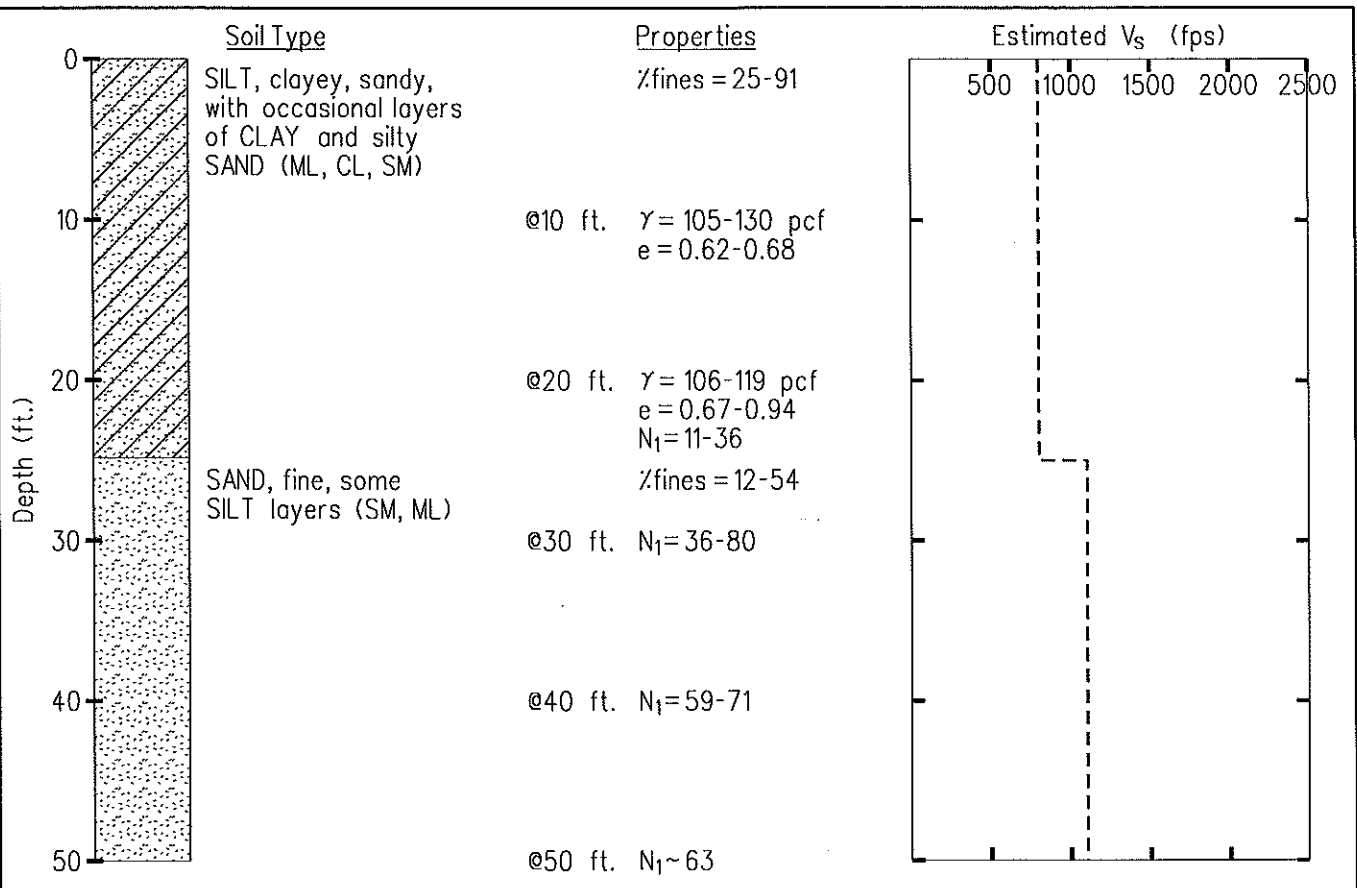
2nd Floor Plan



Perimeter steel moment frames.
Design date: 1983

Note: Data provided by CSMIP

Sensor Locations
Sunwest Office Building San Bernardino, CA
Figure A38.2



Data Source:
 Pioneer Consultants
 (1980)

- Notes:
1. No groundwater encountered.
 2. Penetration resistance established by SPT techniques.
 3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile
Sunwest Office Building San Bernardino, CA
Figure A38.3

sunwest.dgn

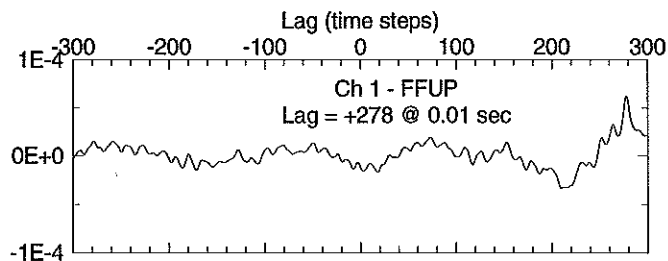


Fig. A38.LD0: Cross-correlation function between vertical ground floor and free-field accelerations

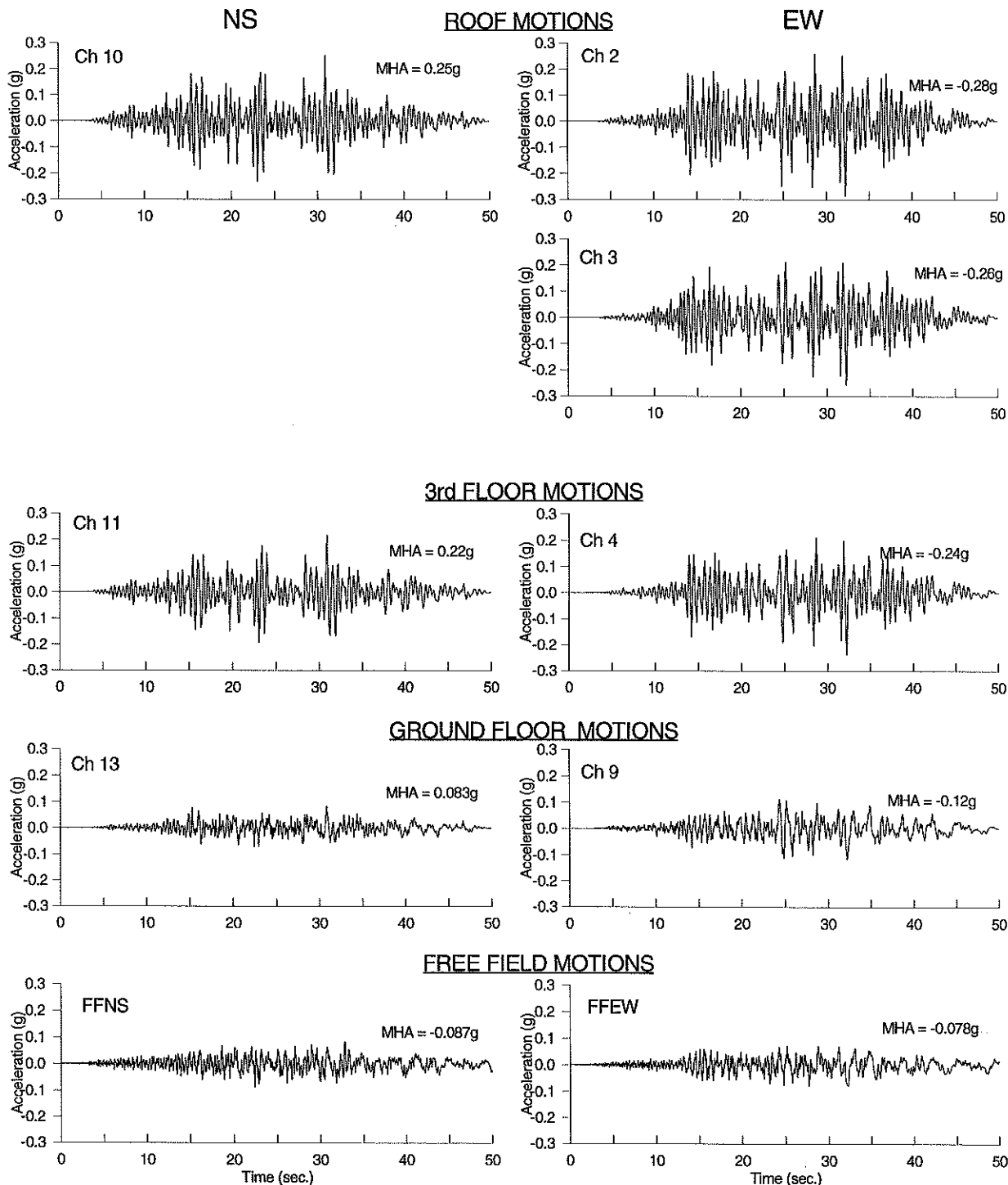


Fig. A38.LD1: SAN BERNARDINO 3 STORY BUILDING
Horizontal Acceleration Time Histories
1992 Landers Earthquake
A38-8

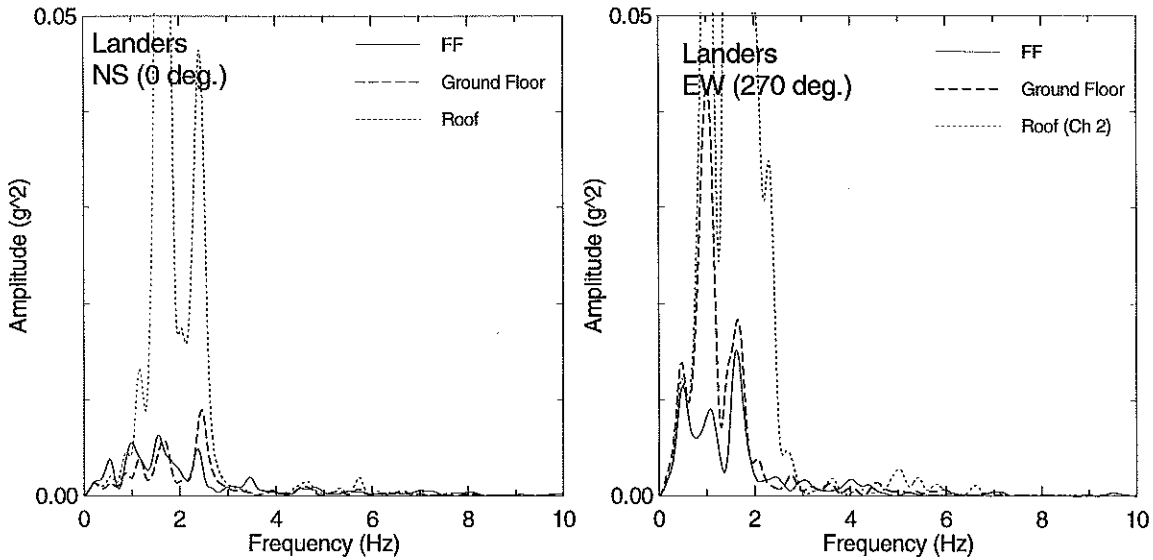


Fig. A38.LD2: SAN BERNARDINO 3 STORY BUILDING
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

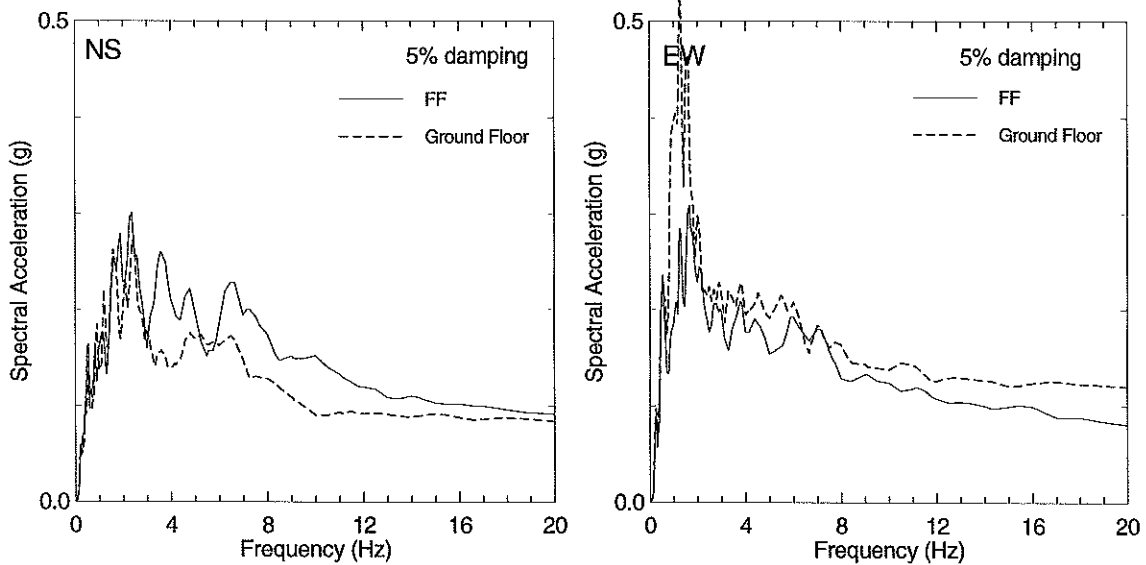


Fig. A38.LD3: SAN BERNARDINO 3 STORY BUILDING
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1992 Landers Earthquake

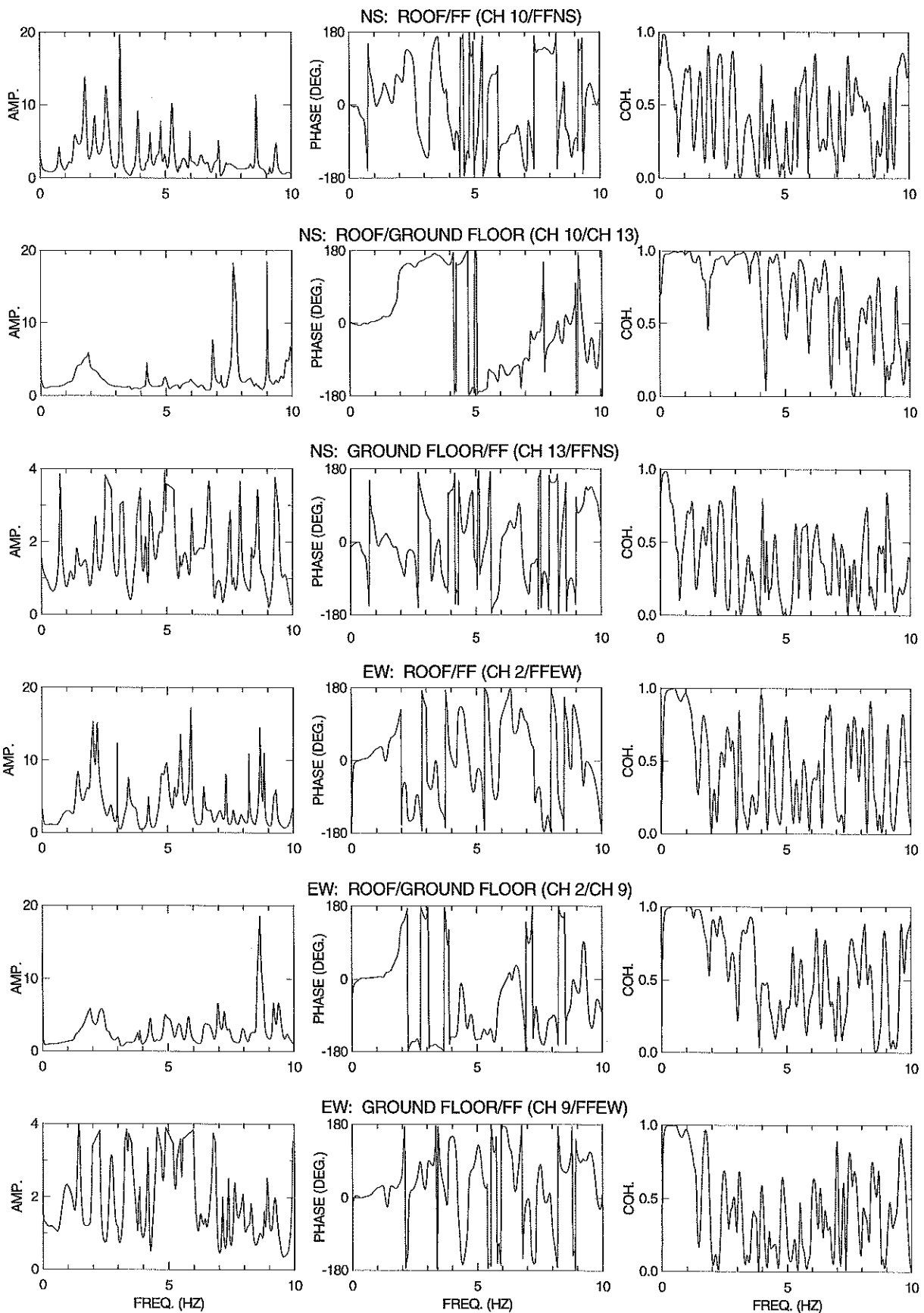


Fig. A38.LD4: SAN BERNARDINO 3 STORY BUILDING
Transmissibility and Coherency Functions
1992 Landers Earthquake
A38-10

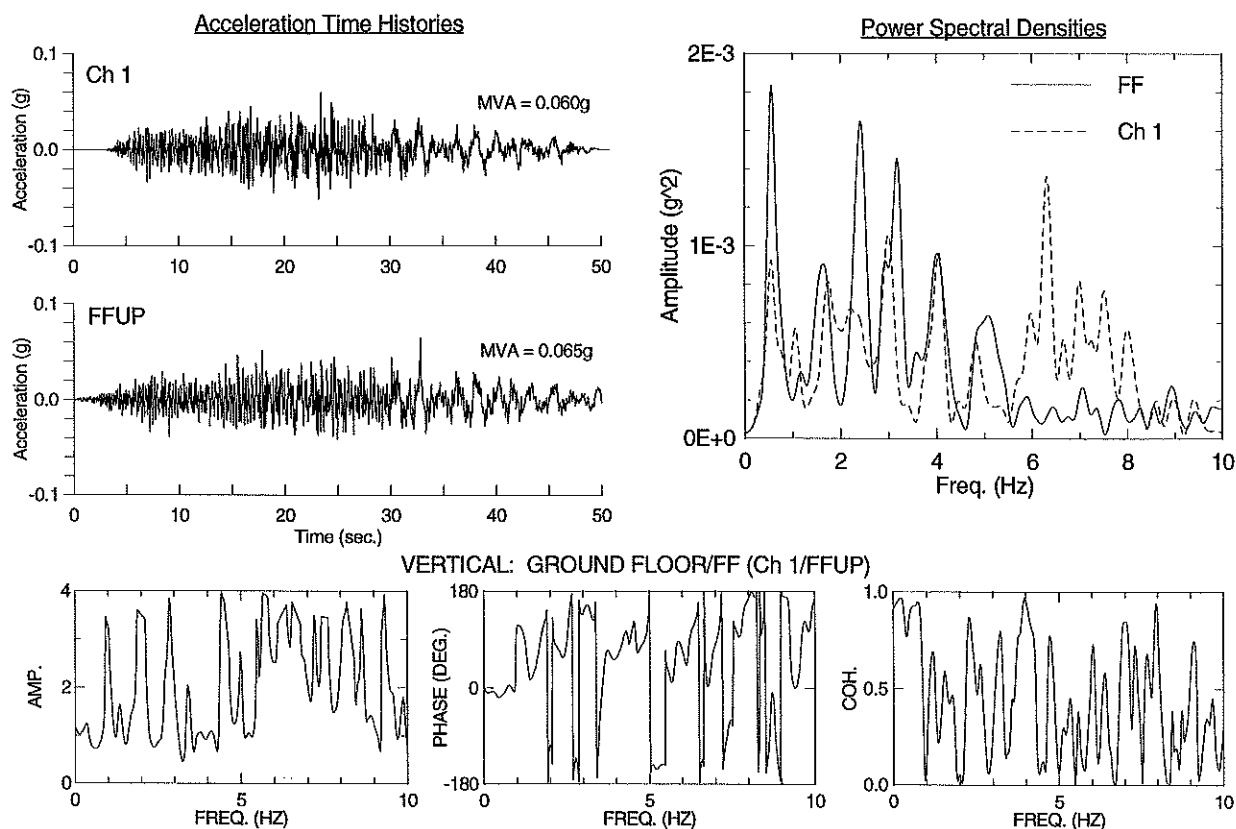


Fig. A38.LD5: SAN BERNARDINO 3 STORY BUILDING
Analysis of Vertical Motions
1992 Landers Earthquake

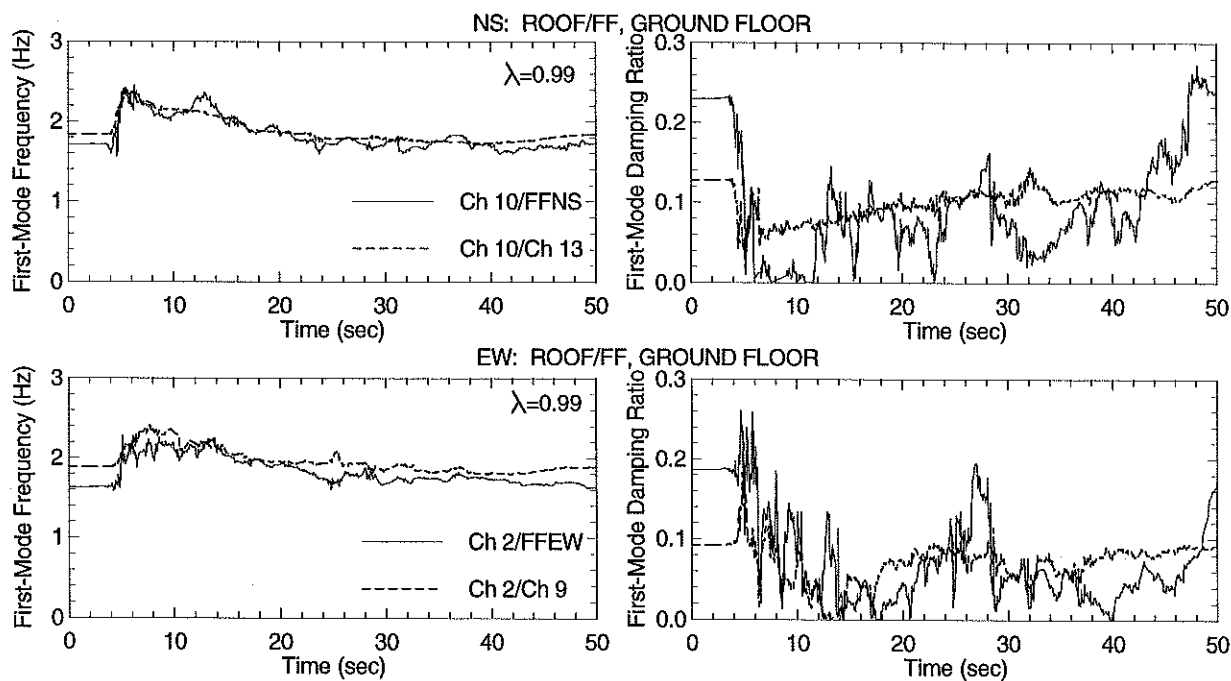


Fig. A38.LD6: SAN BERNARDINO 3 STORY BUILDING
Time Variation of First-Mode Parameters
1992 Landers Earthquake

SITE A39

Site Name: San Bernardino 5 Story CSU Library
Sensors: CSMIP 23285/23672

Lat: 34.183°
Long.: 117.323°

Floor Levels: 6
Bsmt. Levels: 1
Structure: Reinforced concrete shear walls

Foundation: footings
Site Geology: soil
Design Date: 1968

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.037	0.071	0.021	0.034	0.035	0.019	0.090	0.13	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: CSMIP, $f = 0.6\text{--}23\text{ Hz}$, $f_{Nyq} = 25\text{ Hz}$ (str.) 50 Hz (FF); FF/str. data not synchronized or aligned.

Final: FF data rotated 35° CW to align with str.; FF decimated by 2 to $f_{Nyq} = 25\text{ Hz}$, FF/str. data synchronized using cross-correlation of vertical displacements (Fig. A39.NR0).

Figures

A39.1-A39.3
A39.NR0-6

Site Plan, Sensor Locations, Generalized Soil Column
NR Analysis Results



View of structure, looking south-west

DISCUSSION

Site Description

The building is 5 stories tall above ground level with a single level basement, and is rectangular in plan (above-ground plan dimensions of 207.3 by 136.5 feet). A 5-story addition to the library is located immediately west of the structure. A seismic joint is present at the connection between the two buildings. The lateral load resisting system consists of reinforced concrete shear walls at all four corners of the building. The foundation consists of spread footings beneath the walls and columns, and a 4-inch thick concrete slab is present across the base of the structure. As shown in Fig. A39.2, accelerographs are present on the basement, 3rd, and roof levels of the structure. The free-field instrument is located in an open area near some temporary buildings, about 485 feet northeast of the structure.

The site is located on level terrain near the northern edge of the San Bernardino Valley near the San Andreas Fault. Based on data from soil borings by LCA (1966) and Geobase (1989), surficial soil conditions consist of gravely sands with occasional silt layers which extend to the maximum depth explored of 40 feet. The shear wave velocities shown in Fig. A39.3 were estimated using correlations with void ratio and SPT blow count.

Instrument Selection and Free-Field Conditions

The building response was evaluated using sensors near the building centroid (Table A39.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A39.NR1.

Table A39.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 1	Ch 2	Ch 5	Ch 6

There are no significant structures or slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions

B/FF transmissibility functions (Fig. A39.NR4) and the spectra of horizontal motions (Fig. A39.NR2-3) indicate increasing de-amplification of basement motions with frequency.

Transmissibility, coherency, and power spectral density functions for the FF and basement vertical motions are presented in Fig. A39.NR5. No amplification occurred at the first-mode frequencies, indicating that base rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A39.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A39.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A39.NR6).

Table A39.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R)			
		tr (EW) NR: estimated		L (NS) estimated		tr (EW) d=5, j=4		L (NS) d=3, j=3		tr (EW) d=3, j=4		L (NS) d=1, j=3	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	--	--	--	--	1.51	7.4	1.93	7.4	1.59	8.3	1.99	7.1
	2					6.12	11.8	8.14	12.9	5.91	10.6	7.29	13.7

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical B/FF transmissibility function, fixed-base parameters can be approximated by the pseudo flexible-base parameters.

The RPEM results (Fig. A39.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The relative magnitudes of the first-mode frequencies and damping ratios for the B-R and FF-R cases generally confirm the CEM results.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A39.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 95 feet using the recorded free-field motions and the profile in Fig. A39.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though the rocking radiation damping for the longitudinal direction was increased by 30% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A39.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1310$ fps	$h = 52$ ft	emb. = 13 ft	$\gamma = 0.15$
NR	$\nu = 0.33$		tr:	$r_1 = 95$ ft	$r_2 = 86$ ft
	$\beta^* = 2.6\%$	$V_s^* = 1235$ fps	L:	$r_1 = 95$ ft	$r_2 = 107$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A39.3 with the fixed-base modal parameters in Table A39.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A39.4.

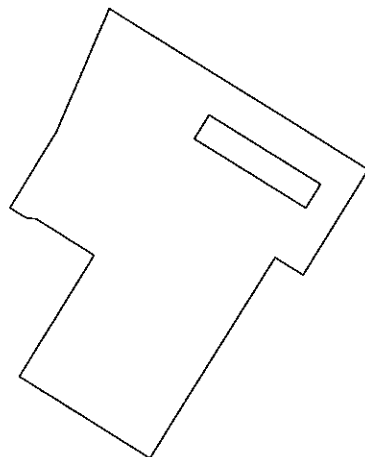
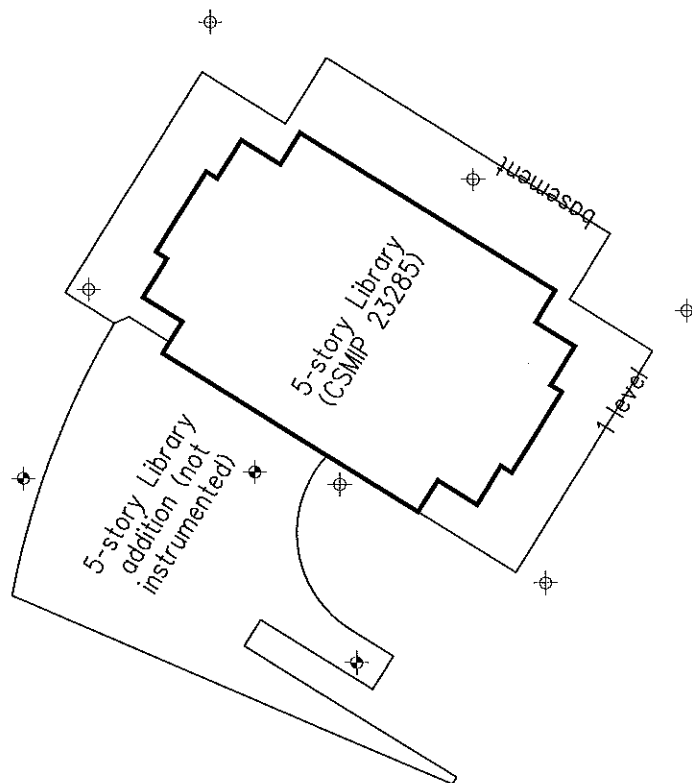
Table A39.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	0.95	0.9	8.3	0.97	-0.3	7.1
	Pre.-Vel.	1.02	0.5	7.5	1.03	1.1	7.9
	Pre.-Bie.	1.02	0.9	7.9	1.03	2.0	8.7

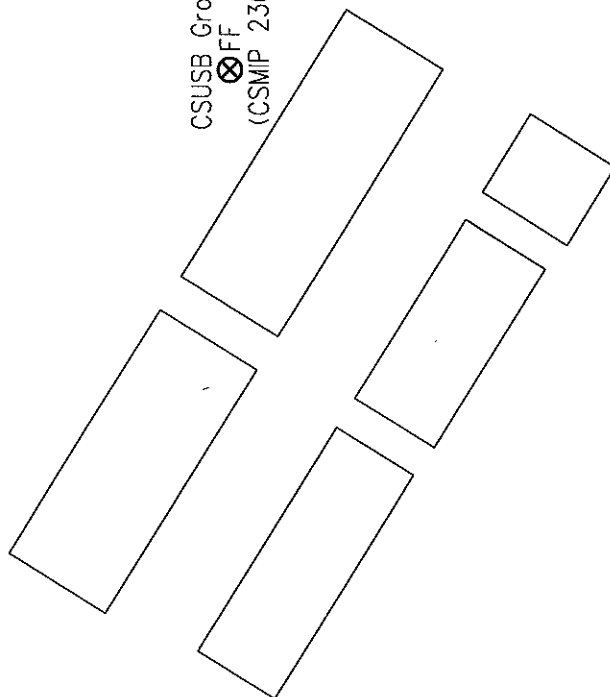
The predictions of period lengthening and foundation damping by both formulations are satisfactory. Although the $\tilde{T}/T < 1$ and $\tilde{\zeta}_0 < 0$ system identification results indicate numerical errors in these analyses, it is clear that the effects of SSI at this site are small.

Data Sources

- LeRoy Crandall Assoc., LCA (1966). “Report of Foundation Investigation, Proposed Library and Audio Visual Center, California State College at San Bernardino, San Bernardino County, CA,” *Job. No. A-66054*.
- Geobase (1989). “Preliminary Geotechnical Investigation for Library Addition, The California State University, San Bernardino, CA,” *Proj. No. P 172.01.01*.



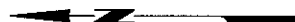
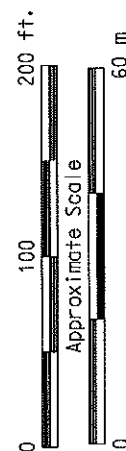
CSUSB Grounds
 (CSMIP 23672)



Legend:

- ⊕ Boring by Geobase (1989)
- ⊙ Boring by LCA (1966)

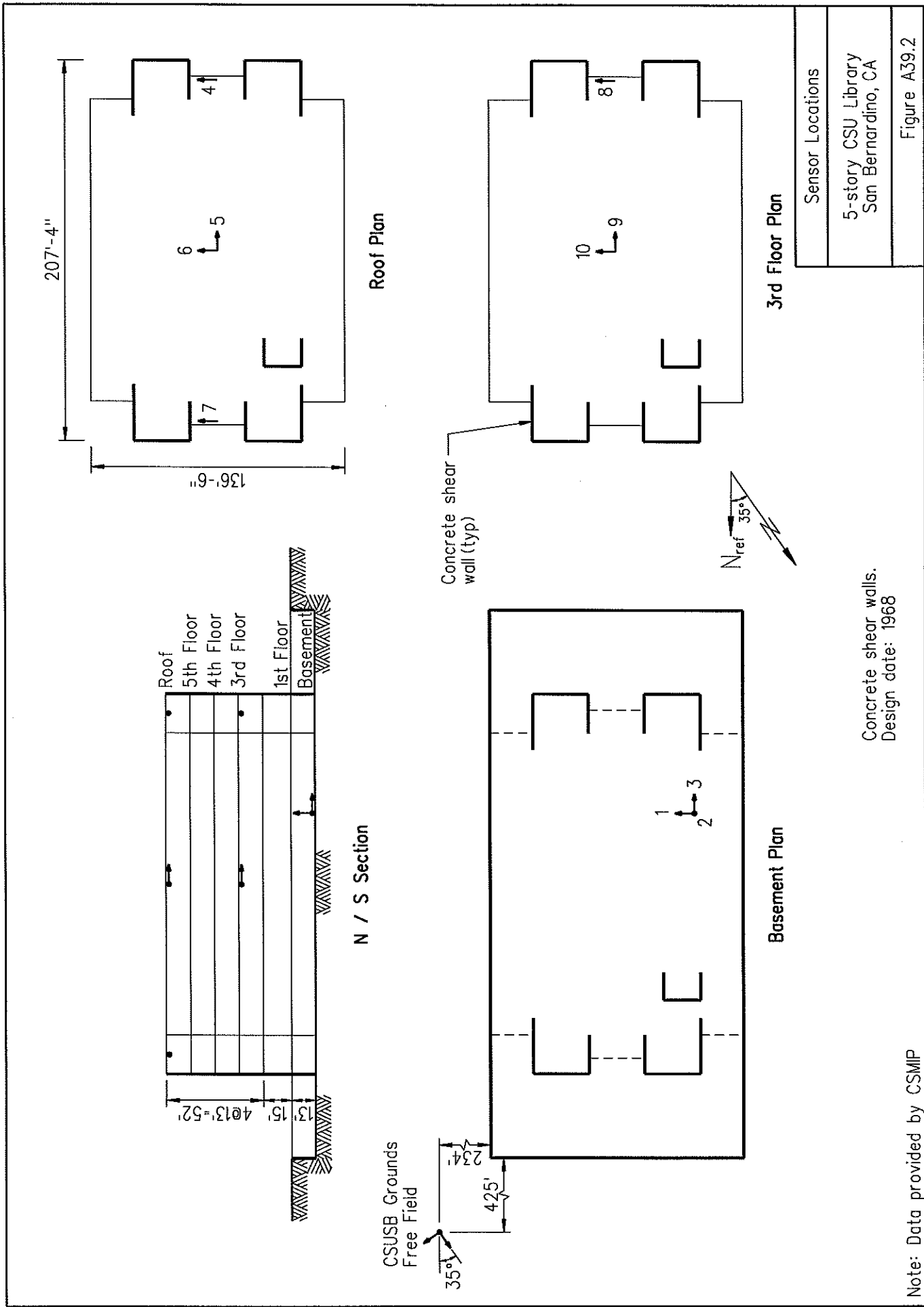
Base: CSUSB Campus Map,
 provided by CSUSB Physical
 Planning and Development
 Office, and Plot Plans by
 Geobase (1989) and LCA (1966)

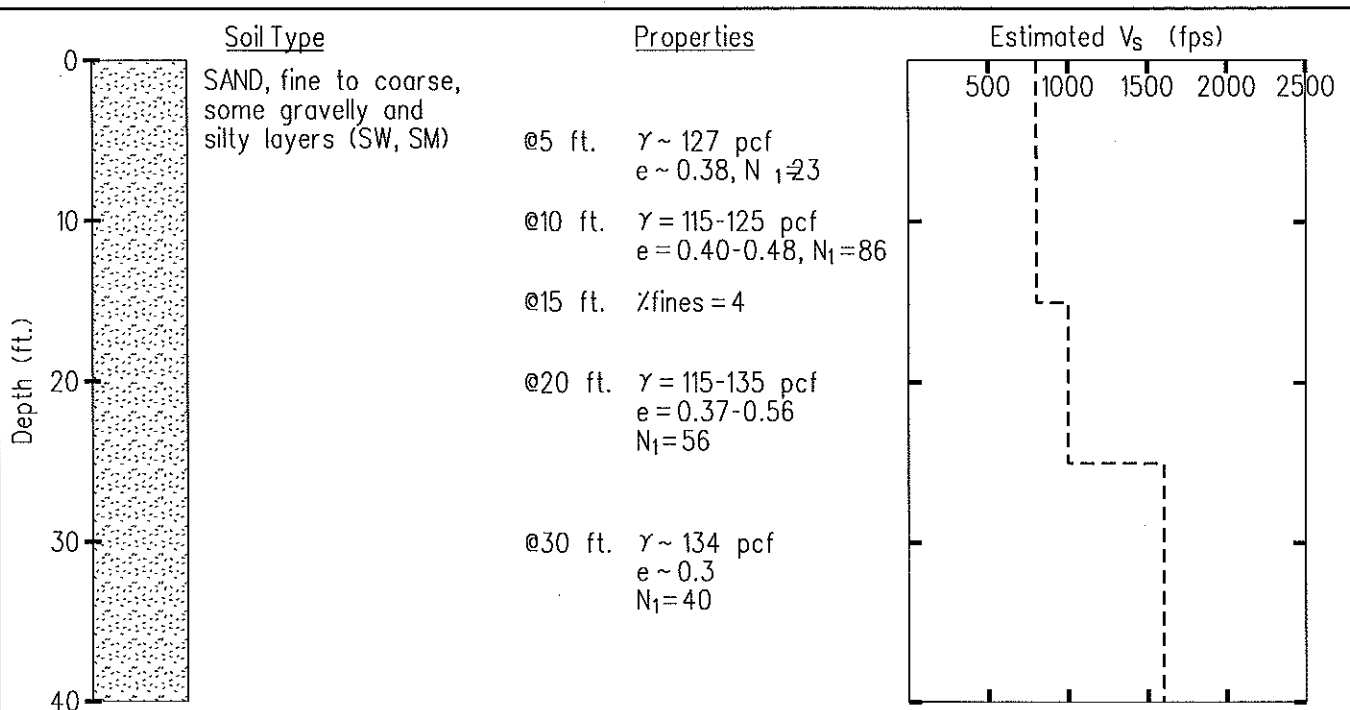


Site Plan

5-story CSU Library
 San Bernardino, CA

Figure A39.1





Data Source:
Geobase (1989) and
LCA (1966)

Notes:

1. No groundwater encountered.
2. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile

5-story CSU Library
San Bernadino, CA

Figure A39.3

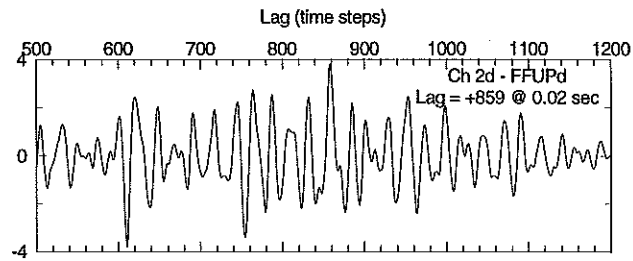
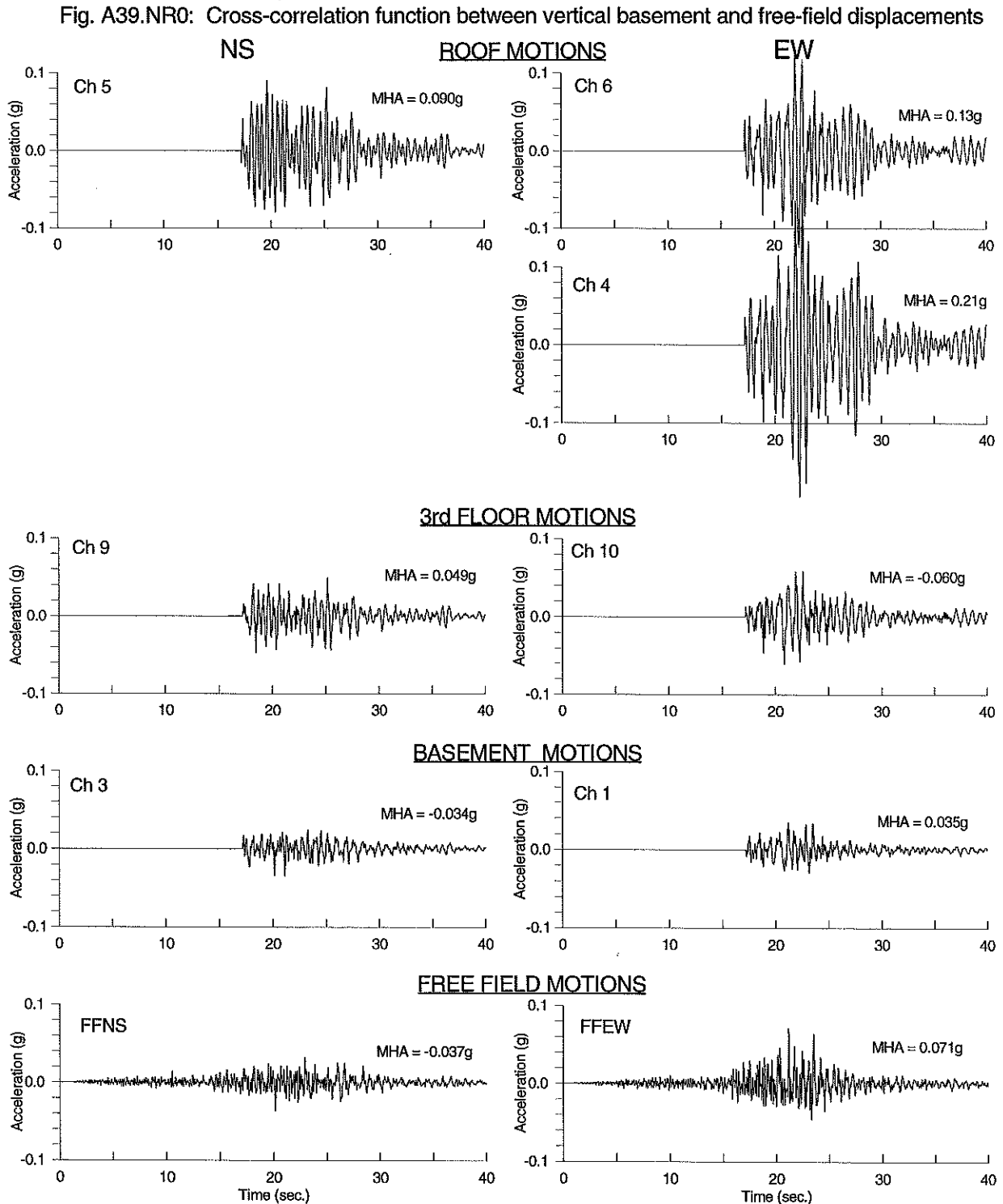


Fig. A39.NR1: SAN BERNARDINO 5 STORY CSU LIBRARY
Horizontal Acceleration Time Histories
1994 Northridge Earthquake
A39-8

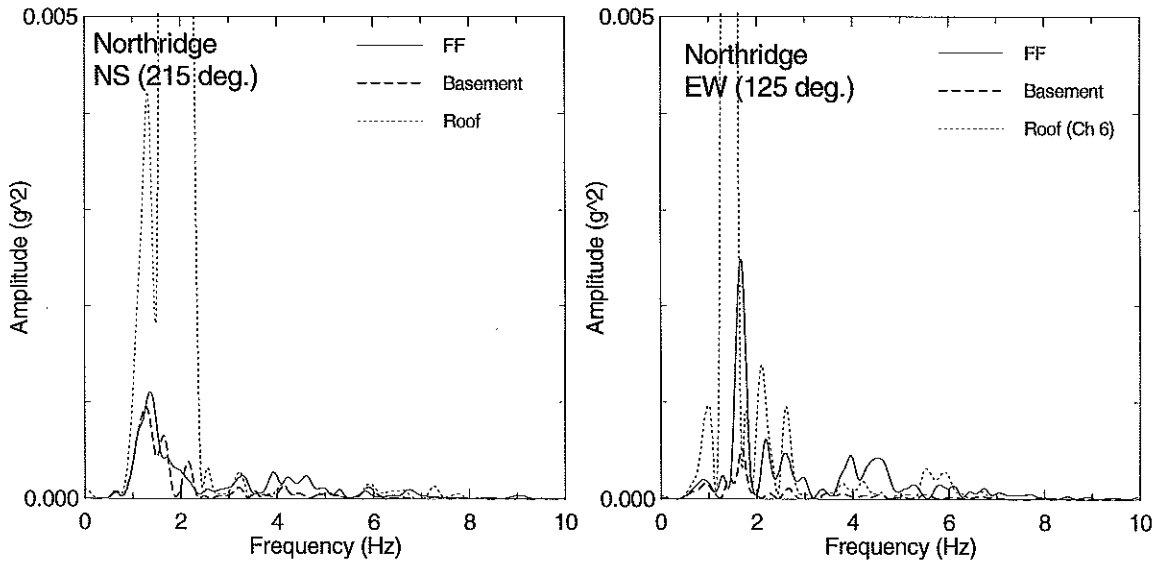


Fig. A39.NR2: SAN BERNARDINO 5 STORY CSU LIBRARY
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

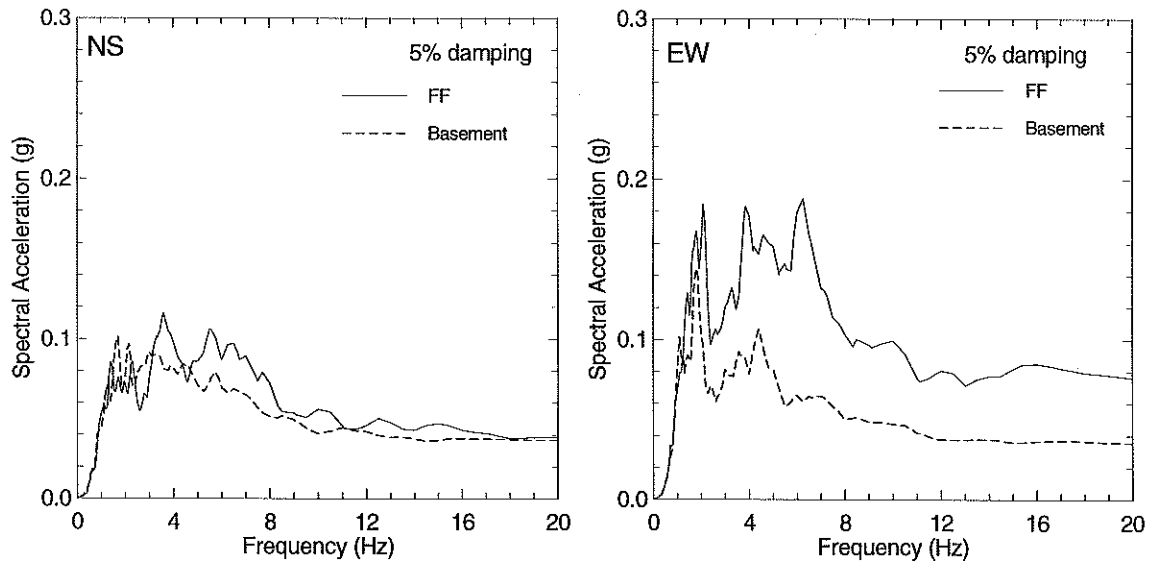


Fig. A39.NR3: SAN BERNARDINO 5 STORY CSU LIBRARY
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

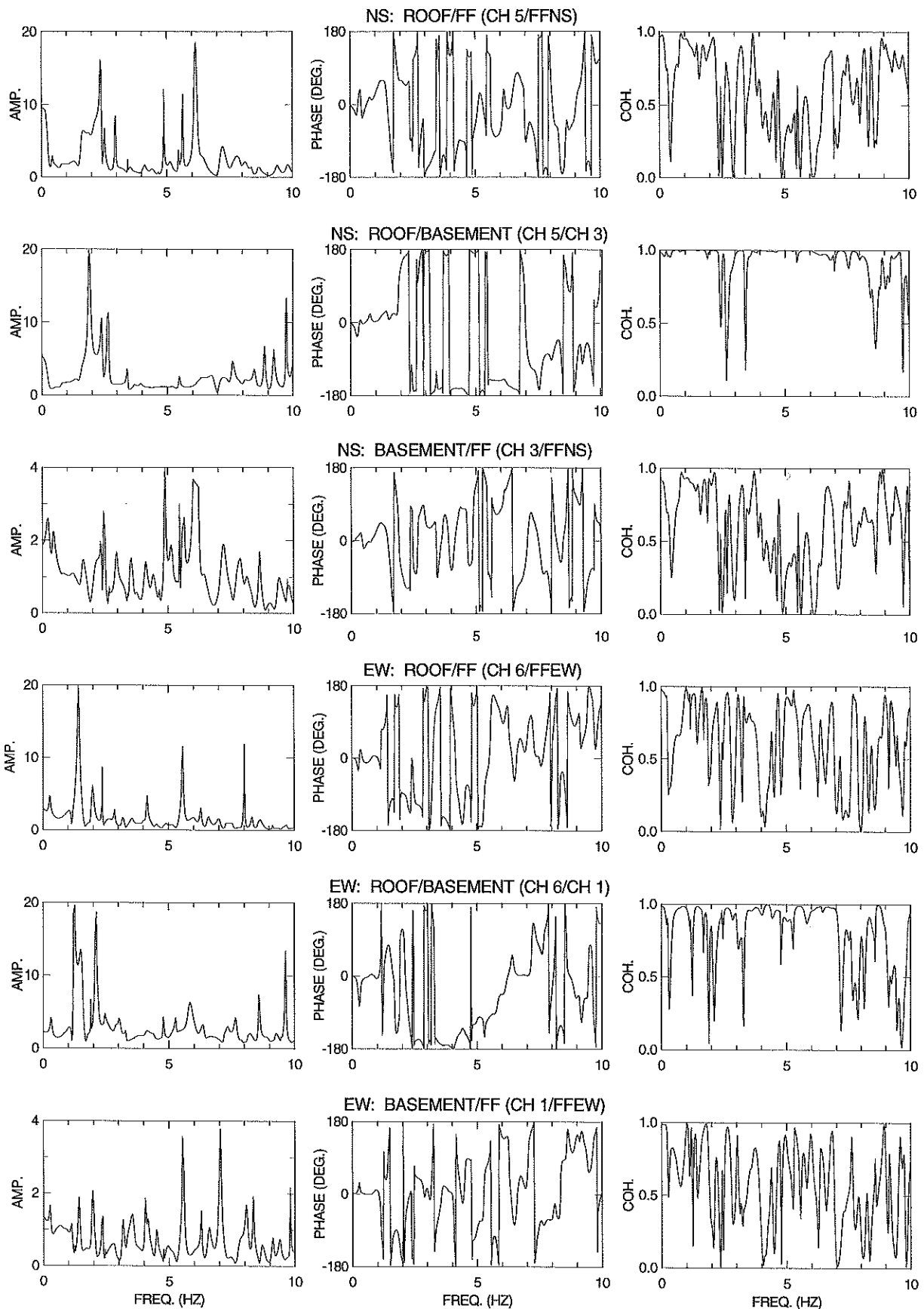


Fig. A39.NR4: SAN BERNARDINO 5 STORY CSU LIBRARY
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A39-10

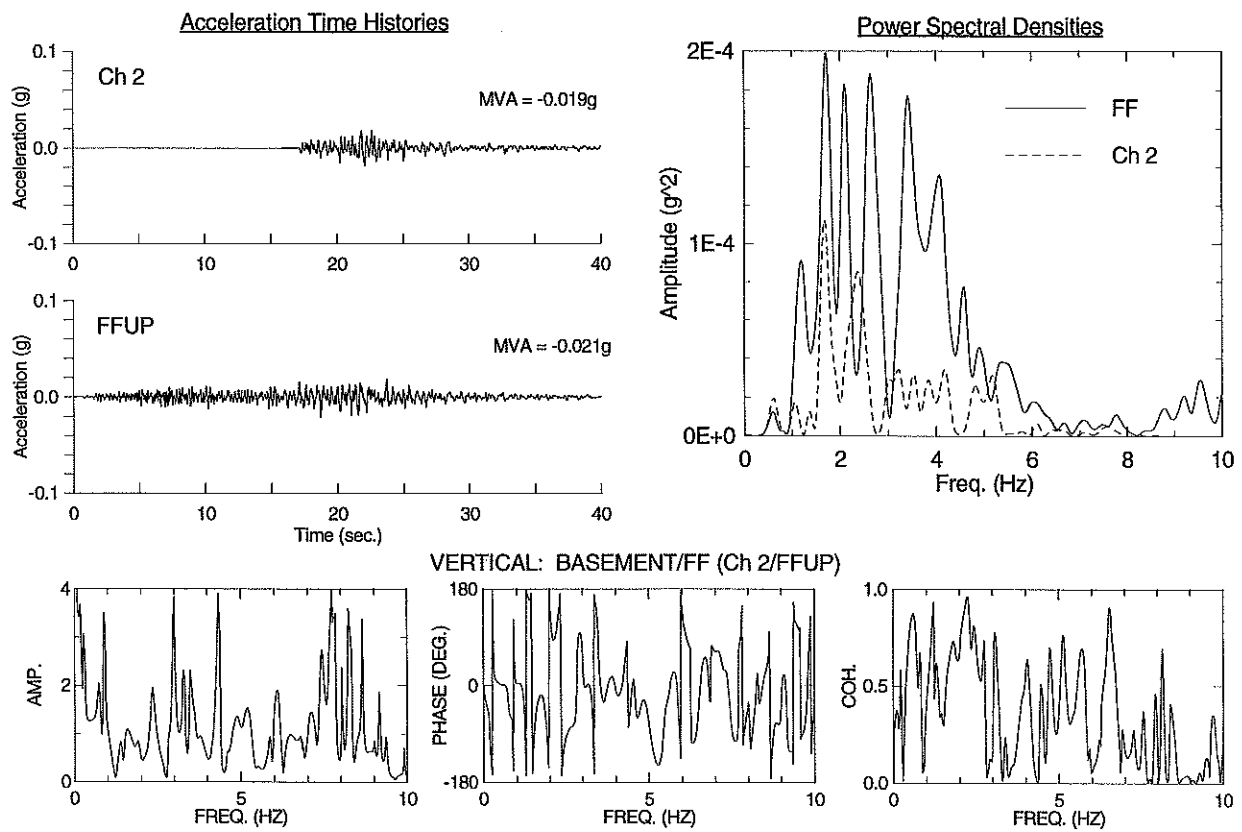


Fig. A39.NR5: SAN BERNARDINO 5 STORY CSU LIBRARY
Analysis of Vertical Motions
1994 Northridge Earthquake

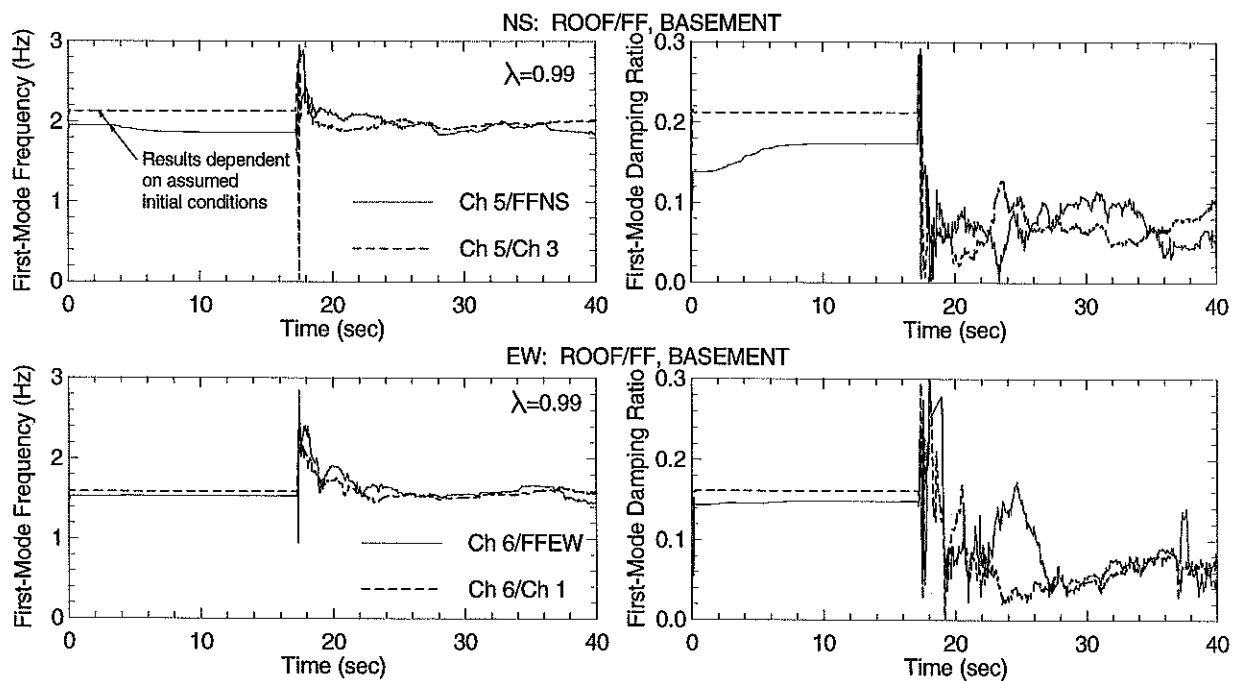


Fig. A39.NR6: SAN BERNARDINO 5 STORY CSU LIBRARY
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A40

Site Name: San Bernardino Vanir Tower
Sensors: CSMIP 23522/23515

Lat: 34.104°
Long.: 117.292°

Floor Levels: 9 + penthouse
Bsmt. Levels: 0
Structure: Moment-resisting steel frames

Foundation: concrete piers
Site Geology: soil
Design Date: 1974

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier*	0.02	0.04	0.02	0.03	0.03	0.02	0.02	0.02	--
1992 Landers	0.085	0.096	0.070	0.081	0.068	0.042	0.17	0.12	--
1992 Big Bear*	0.11	0.12	0.14	0.07	0.08	0.04	0.19	0.10	--
1994 Northridge*	0.04	0.04	0.02	0.04	0.05	0.02	0.06	0.07	--

* Not processed -- = no instrument

Preprocessing

Landers Earthquake (LD)

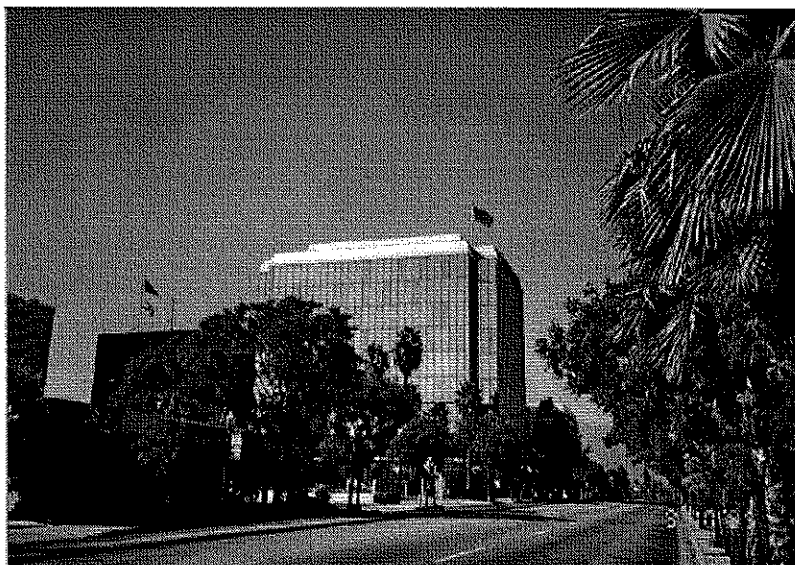
Preliminary: CSMIP; filtered, $f = 0.16\text{--}23\text{ Hz (str.)}$, $0.16\text{--}46\text{ Hz (FF)}$; $f_{Nyq} = 25\text{ Hz}$, 50 Hz (str.) , FF ; $FF/str.$ data not synchronized.

Final: FF data decimated by 4, $str.$ data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$, $FF/str.$ data synchronized using cross-correlation of vertical accelerations (Fig. A40.LD0).

Figures

A40.1-A40.3
A40.LD0-6

Site Plan, Sensor Locations, Generalized Soil Column
LD Analysis Results



View of structure, looking north

DISCUSSION

Site Description

The building is 9 stories tall above ground level with no basement, and is essentially rectangular in plan with dimensions 134 by 110 feet. Single-story extensions of the structure are present at the ground level north and south of the building. Several structures are located near the building, the most significant of which is a 6-story city hall building about 50 feet northwest of the structure. The lateral load resisting system consists of moment-resisting steel frames in both directions. Each column in the building is founded on a group of four 32- to 36-foot deep piles with a 3-foot thick pile cap. The pile caps are not tied together except for a tie beam around the building perimeter, though a 6-inch slab is present across the ground floor level. As shown in Fig. A40.2, accelerographs are present on the ground, 3rd, 7th, and roof levels of the structure. The free-field instrument is located about 910 feet southeast of the structure in a parking lot.

The site is located on level terrain within the San Bernardino Valley. Based on borings by LCA (1972), surficial soil conditions at the site consist of 20 feet of silts and clays underlain by sands and gravels which extend to the maximum depth explored of 60 feet. Based on water and oil well logs for the area, Duke and Leeds (1962) have estimated that the surficial young alluvial soils extend to about 150 feet, and that underlying older alluvium extends to early Pleistocene deposits at 900-foot depth. The shear wave velocities shown in Fig. A40.3 were estimated using correlations with void ratio and SPT blow count.

Instrument Selection and Free-Field Conditions

Sensors at the west end of the structure were used in the analyses (Table A40.1). Acceleration time histories recorded by these sensors are shown in Fig. A40.LD1.

Table A40.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 9	Ch 13	Ch 1	Ch 3	Ch 10

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A40.LD4) and the spectra for horizontal motions (Fig. A40.LD2-3) generally indicate de-amplification of GF motions for frequencies greater than about 2 to 3 Hz. No amplification is evident near the first-mode response frequencies.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A40.LD5. Amplification is not evident near the first-mode frequencies for either direction, indicating that rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A40.LD4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A40.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A40.LD6).

Table A40.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=8, j=8		L (EW) d=8, j=9		tr (NS) d=8, j=8		L (EW) d=7, j=9	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	--	--	--	--	0.50	5.9	0.48	5.6	0.50	4.7	0.50	7.9
	2					1.70	12.2	1.61	8.5	1.76	10.9	1.66	7.0
	3					3.03	7.9	2.80	7.7	3.02	6.9	2.75	7.2

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in both directions.
- Fixed-base parameter estimation procedures could not be invoked because $\tilde{f}_1 > \tilde{f}_1^*$. However, since no significant rocking effect was evident from the vertical GF/FF transmissibility function, fixed-base parameters can be approximated by the pseudo flexible-base parameters.

The RPEM results (Fig. A40.LD6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The relative magnitudes of the first-mode frequencies and damping ratios for the GF-R and FF-R cases generally confirm the CEM results.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A40.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 55 feet using the recorded free-field motions and the profile in Fig. A40.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A40.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 920$ fps	$h = 74$ ft	emb.=0	$\gamma = 0.15$
LD	$\nu = 0.33$		tr: $r_1 = 55$ ft		$r_2 = 52$ ft
	$\beta^* = 3.4\%$	$V_s^* = 850$ fps	L: $r_1 = 55$ ft		$r_2 = 59$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A40.3 with the fixed-base modal parameters in Table A40.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A40.4.

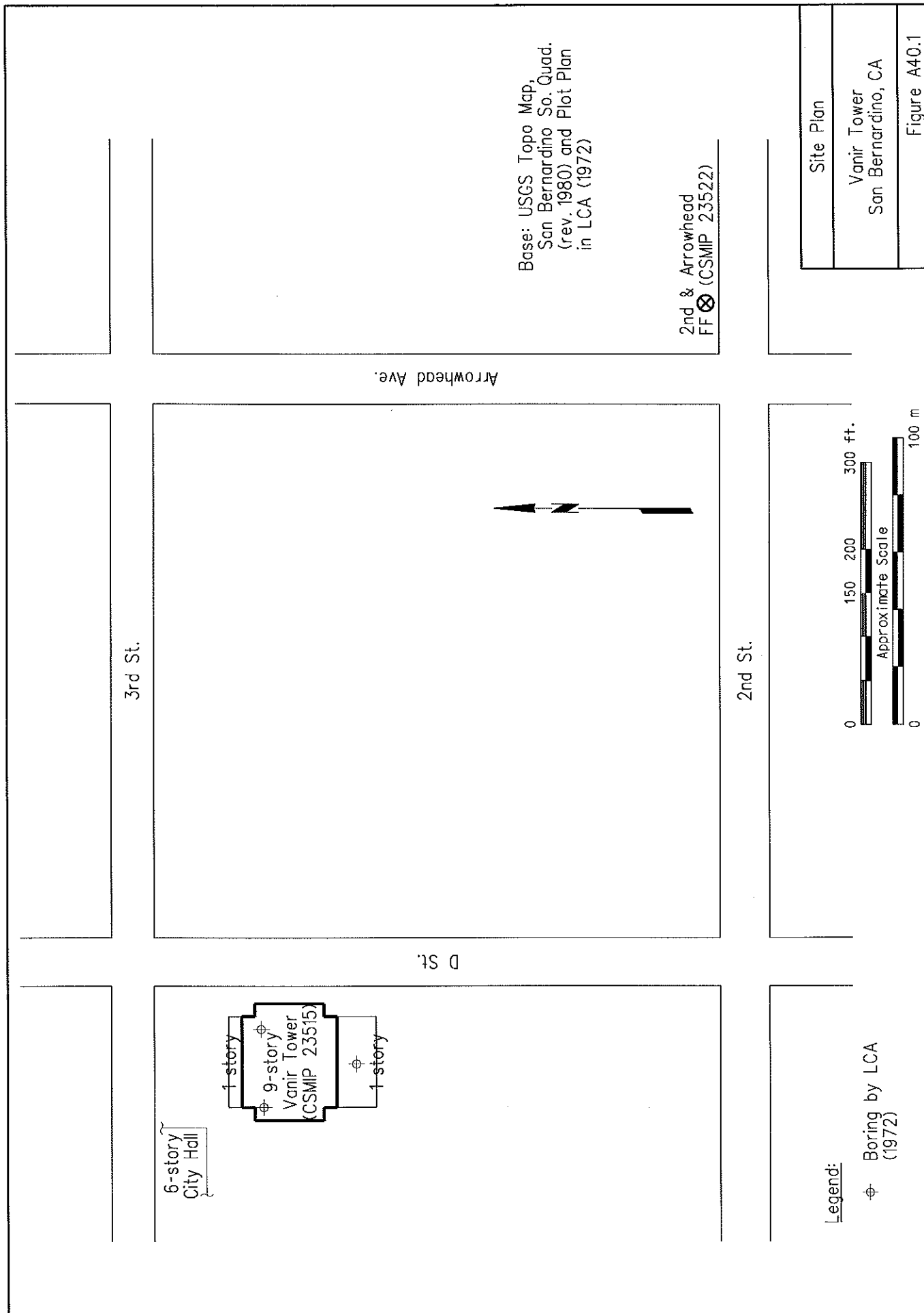
Table A40.4: Empirical and predicted inertial interaction effects

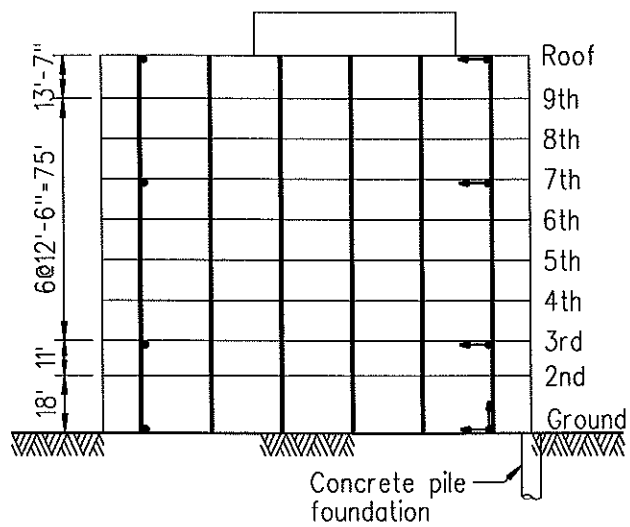
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LD	Sys. ID	1.00	-1.2	4.7	0.98	2.2	7.9
	Prediction	1.01	0.1	5.8	1.01	0.1	5.6

The predicted period lengthening and foundation damping is satisfactory. The negative $\tilde{\zeta}_0$ value from the system identification analysis indicates a minor numerical error in these results.

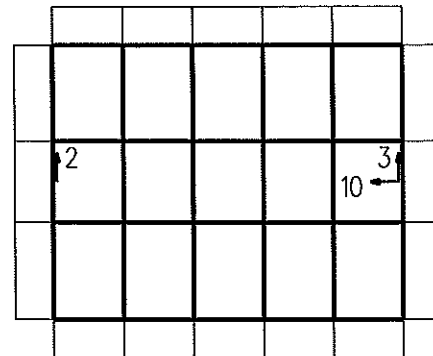
Data Sources

Duke, C.M. and Leeds, D.J. (1962). "Site characteristics of Southern California strong motion earthquake stations," *Rpt. No. 62-55*, Dept. of Engrg., Univ. of California, Los Angeles.
 LeRoy Crandall Assoc., LCA (1972). "Report of Foundation Investigation, Proposed Office Building, D Street near Court Street, San Bernardino, CA," *Job. No. A-72044*.

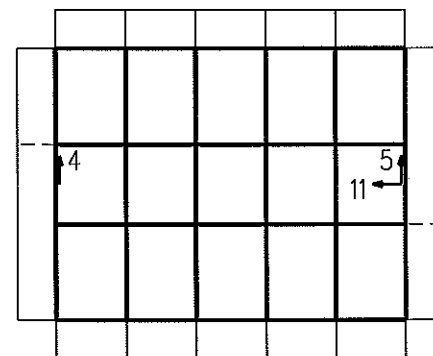




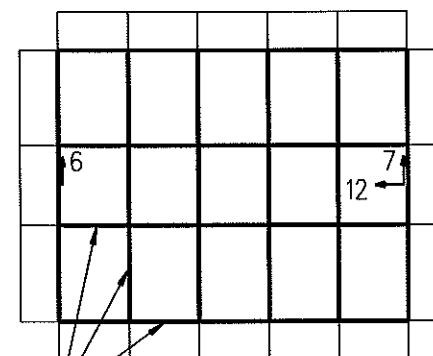
E / W Elevation



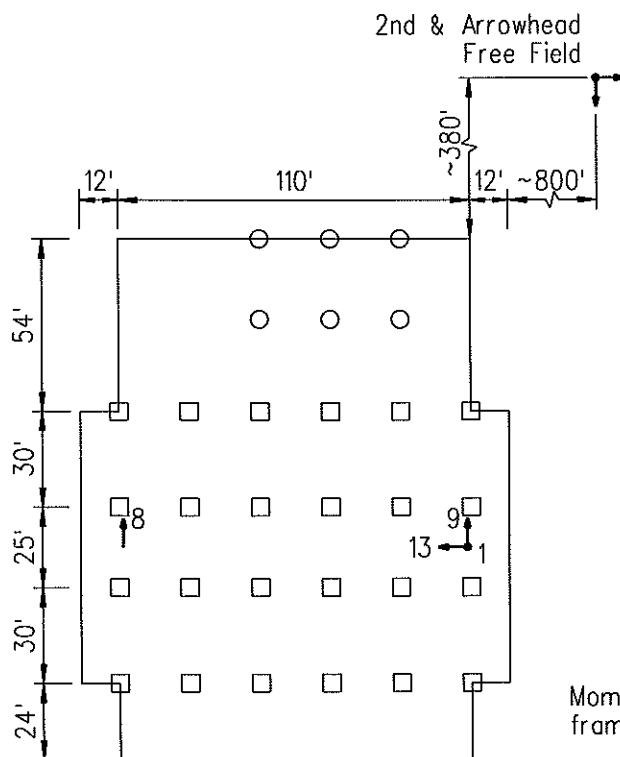
Roof Plan



7th Floor Plan



3rd Floor Plan

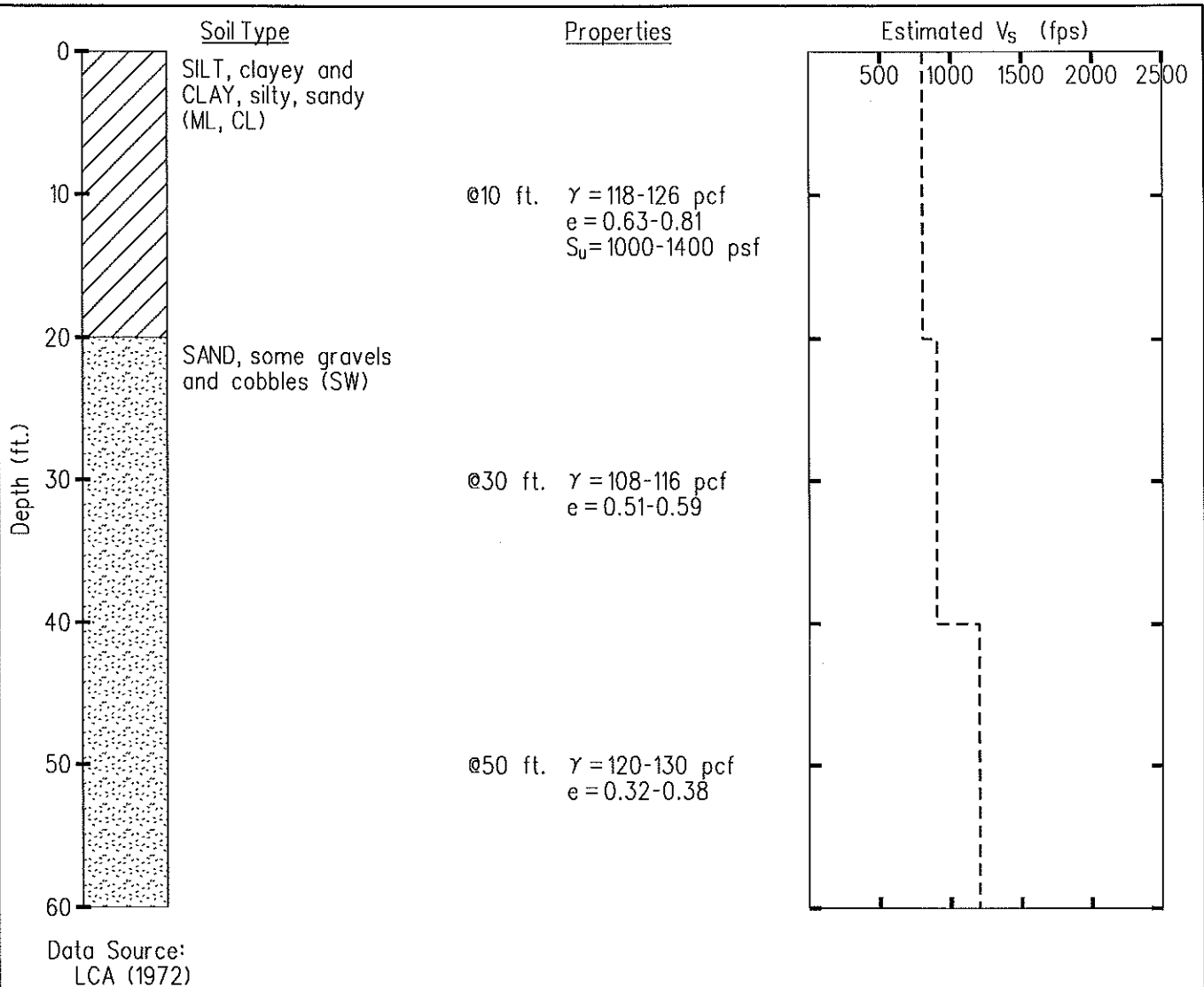


Ground Floor Plan

$N = N_{ref}$

Sensor Locations
Vanir Tower San Bernardino, CA
Figure A40.2

Note: Data provided by CSMIP



Notes:

1. No groundwater encountered.
2. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile

Vanir Tower
San Bernardino, CA

Figure A40.3

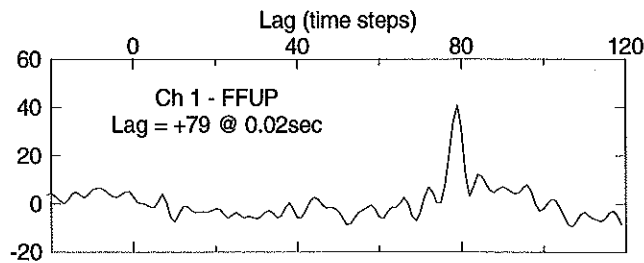


Fig. A40.LD0: Cross-correlation function between vertical ground floor and free-field accelerations

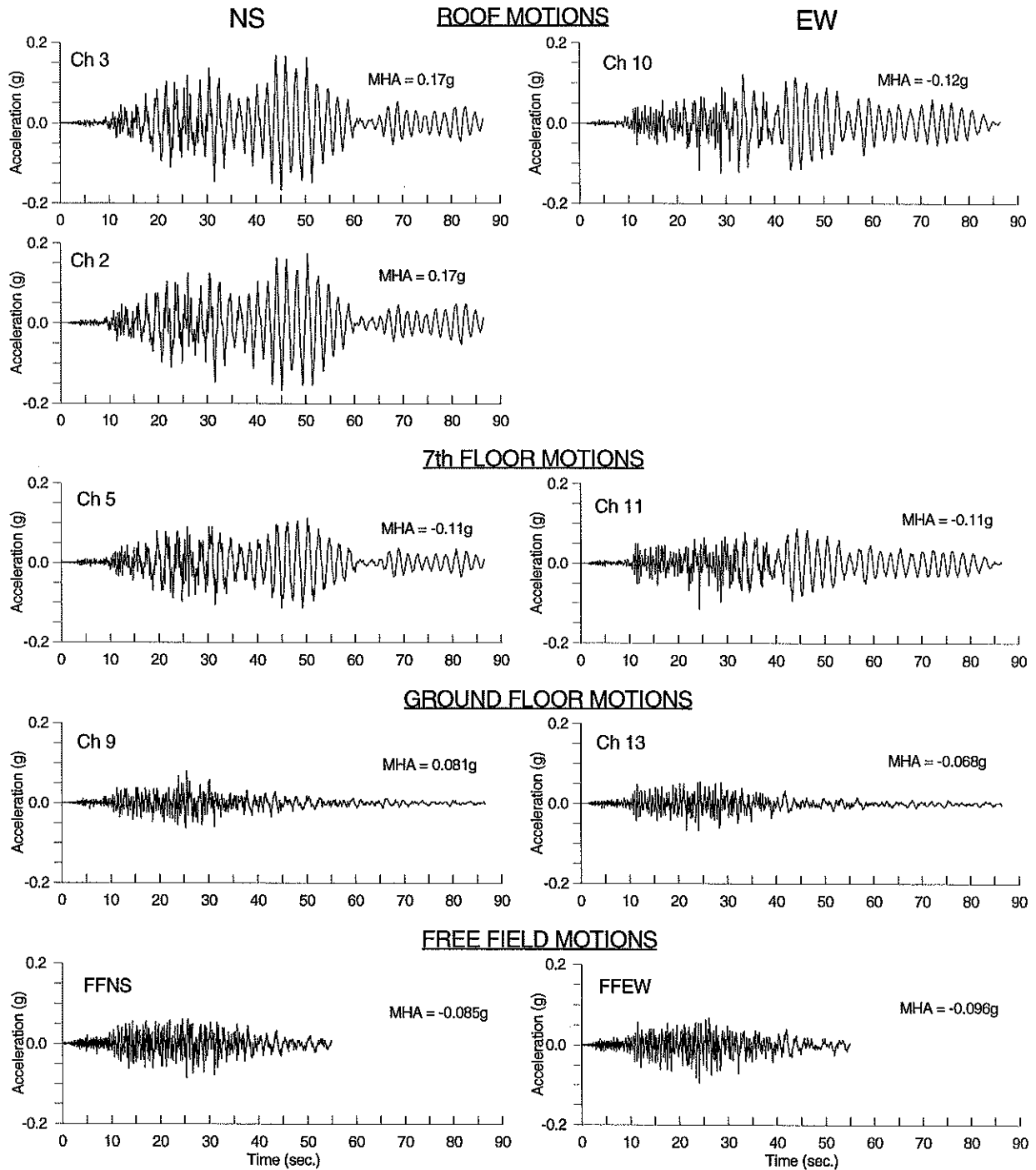


Fig. A40.LD1: SAN BERNARDINO VANIR TOWER
Horizontal Acceleration Time Histories
1992 Landers Earthquake

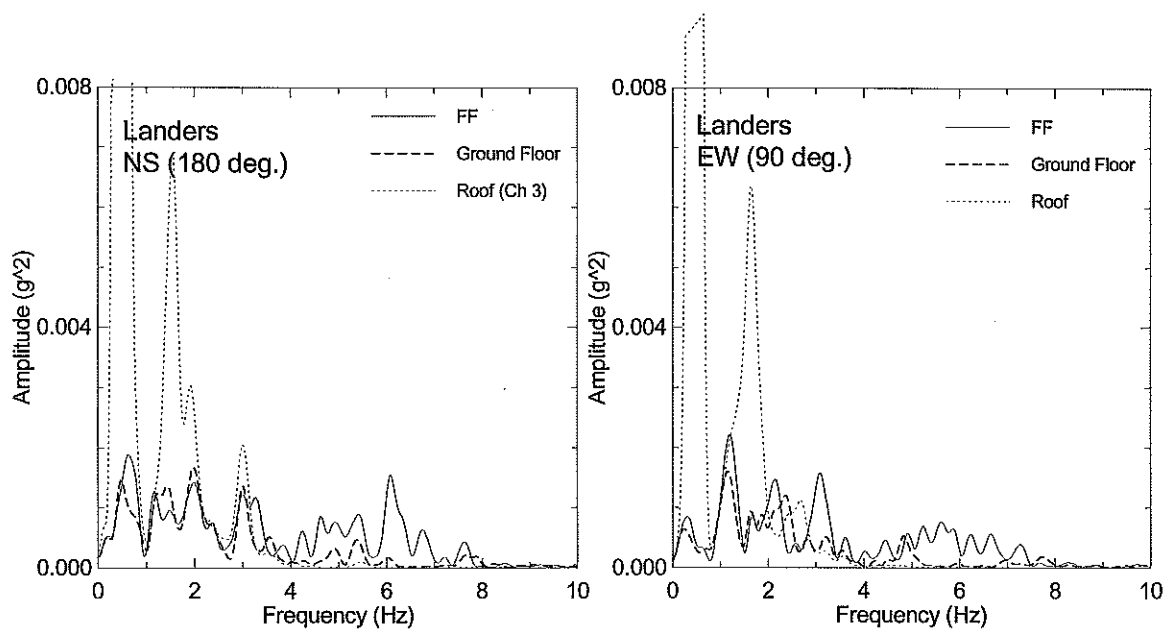


Fig. A40.LD2: SAN BERNARDINO VANIR TOWER
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

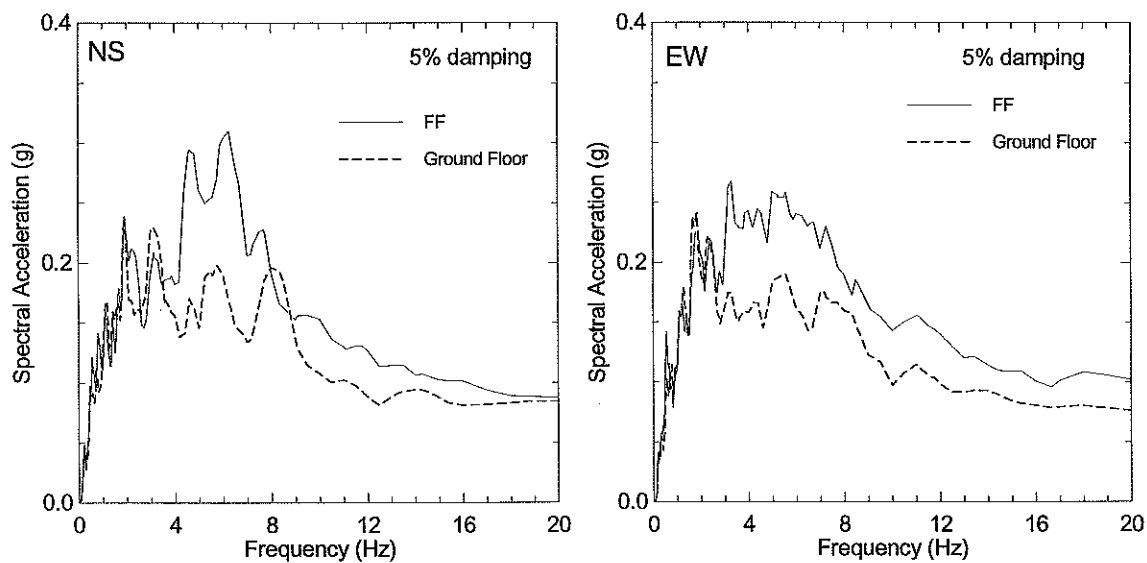


Fig. A40.LD3: SAN BERNARDINO VANIR TOWER
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1992 Landers Earthquake

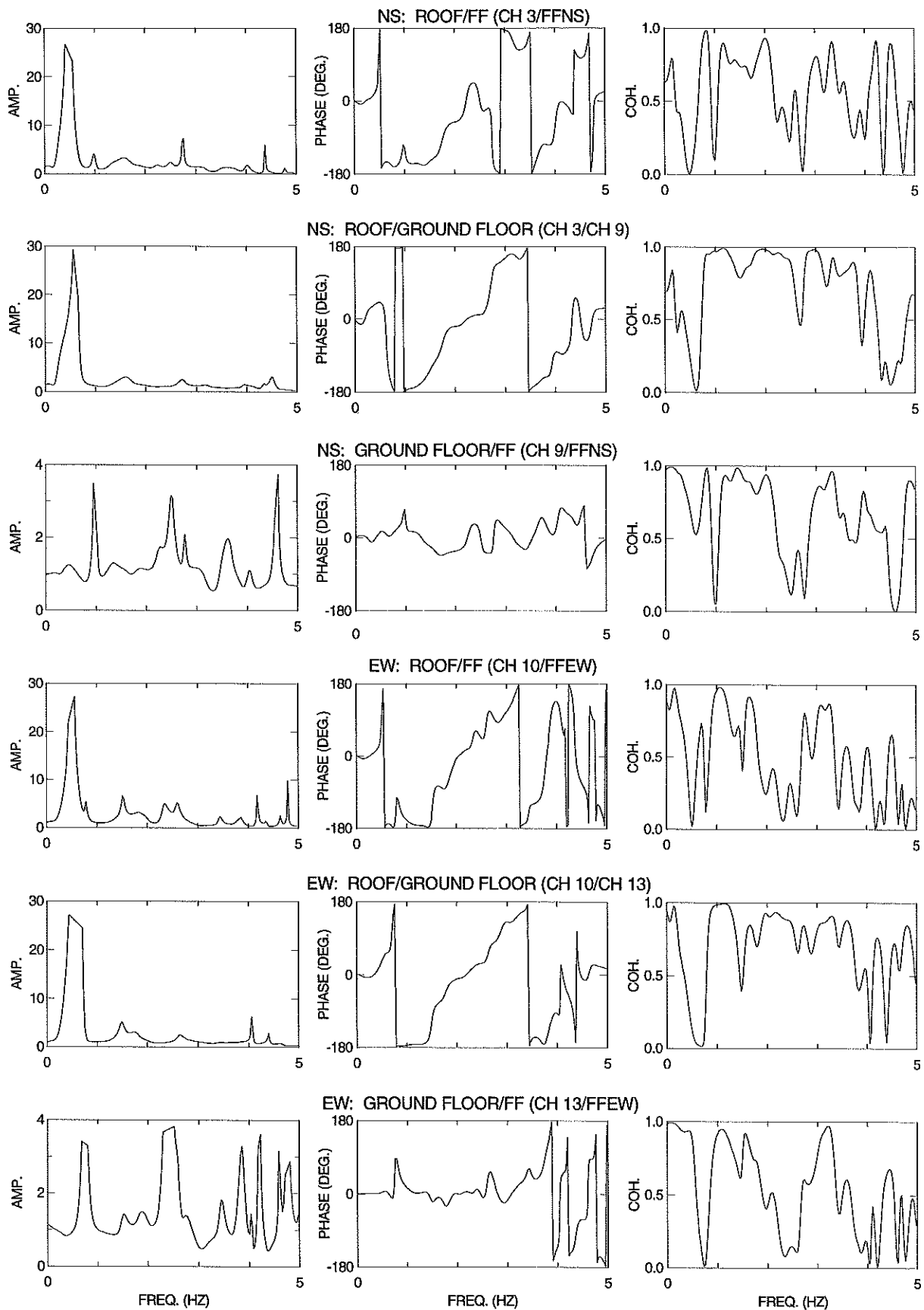


Fig. A40.LD4: SAN BERNARDINO VANIR TOWER
Transmissibility and Coherency Functions
1992 Landers Earthquake
A40-10

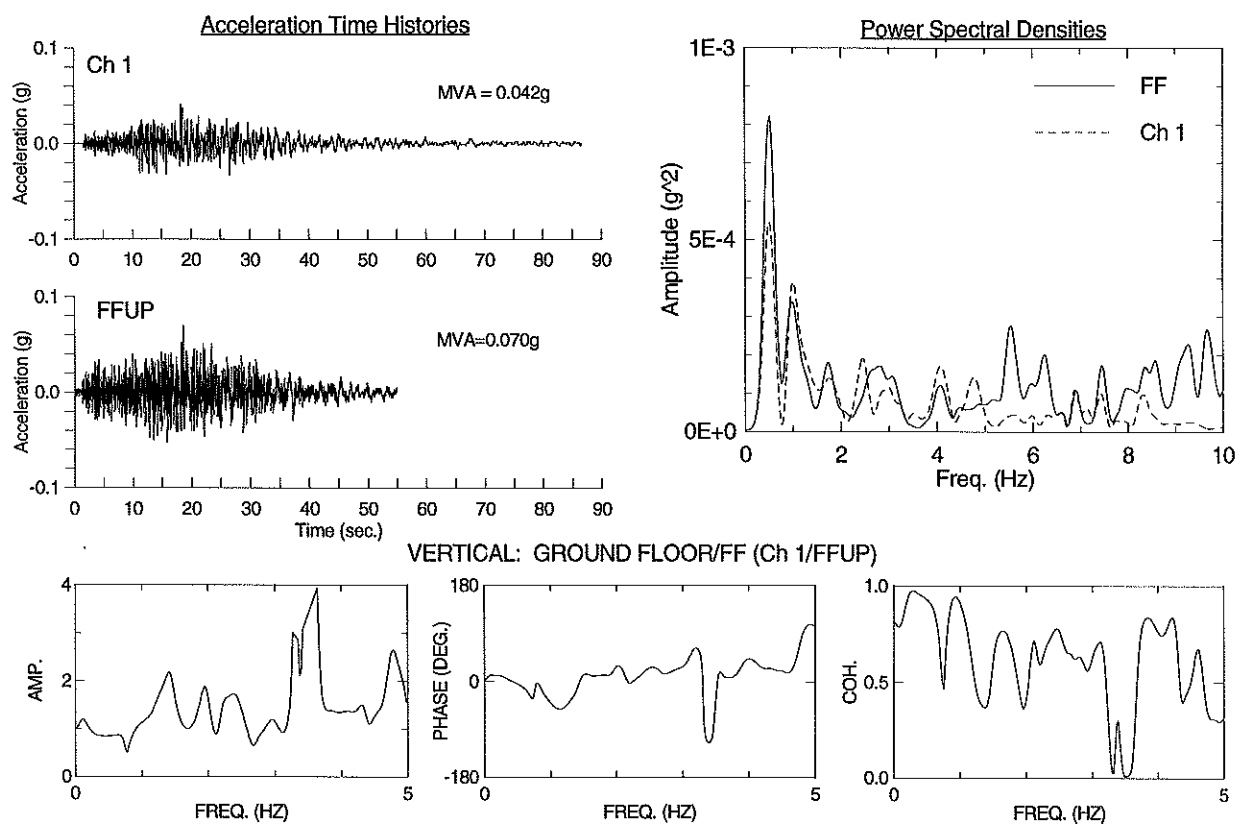


Fig. A40.LD5: SAN BERNARDINO VANIR TOWER
Analysis of Vertical Motions
1992 Landers Earthquake

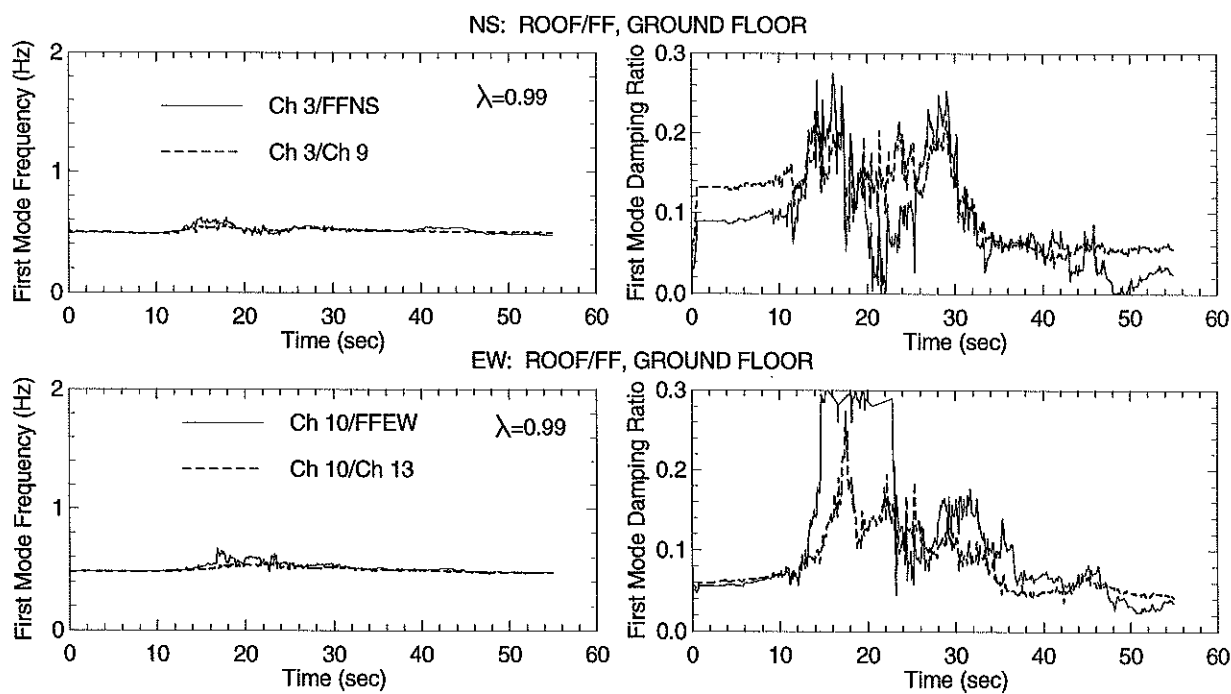


Fig. A40.LD6: SAN BERNARDINO VANIR TOWER
Time Variation of First Mode Parameters
1992 Landers Earthquake

SITE A41

Site Name: San Bernardino Co. Gov't Ctr.
Sensors: USGS 5245

Lat: 34.106°
Long.: 117.287°

Floor Levels: 5
Bsmt. Levels: 0
Structure: Braced steel frames

Foundation: concrete piers
Site Geology: soil
Design Date: 1983

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1992 Landers*	0.06	0.07	0.05	0.09	0.06	0.07	0.34	0.26	--
1994 Northridge	0.035	0.035	0.013	0.036	0.043	0.016	0.21	0.16	--

* Not processed -- = no instrument

Preprocessing

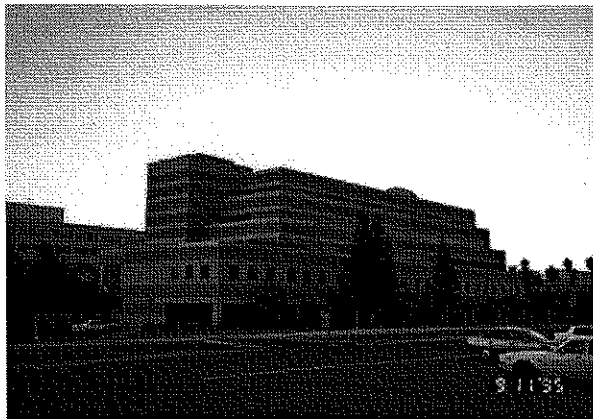
Northridge Earthquake (NR)

Preliminary: digitized at $f_{Nyq} = 100$ Hz; no processing, FF/str. data not synchronized.

Final: filtered (this study), $f = 0.4$ -12.5 Hz; decimated by 8 to $f_{Nyq}=12.5$ Hz, FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A41.NR0).

Figures

A41.1-A41.3 Site Plan, Sensor Locations, Generalized Soil Column
A41.NR0-6 NR Analysis Results



View of structure from FF (looking south-west)



View of structure, looking south-east

DISCUSSION

Site Description

The building is 5 stories tall above ground level with no basement, though the ground floor is stepped at its southern end (Fig. A41.2). The structure has an irregular shape in plan view, forming an "L" with ground-level dimensions of 255 by 260 feet. The structure is also irregular in profile, stepping inward at each floor level up from ground level. A small 2-story library is located 60 feet west of the building, and a 5-story court building is located as close as 60 feet from the south end of the structure. The lateral load resisting system consists of steel braced frames. Most columns are supported by 5-pile groups where each pile is 14-inch square and generally extends to 35-foot depth. The pile caps are interconnected by grade beams and a 5-inch thick slab is present across the base of the building. As shown in Fig. A41.2, accelerographs are present on the ground, 4th, and roof levels of the structure. The free-field instrument is located in a parking lot, about 210 feet northeast of the building.

The site is located on level terrain within the San Bernardino Valley. Based on borings by LCA (1982), surficial soils at the site consist of gravely sands and silts which extend to the maximum depth explored of 60 feet. Based on water and oil well logs for the area, Duke and Leeds (1962) have estimated that the surficial young alluvial soils extend to about 150 feet, and that underlying older alluvium extends to early Pleistocene deposits at 900-foot depth. Based on measurements by LCA, groundwater occurs at a depth of 25 feet at the site. The shear wave velocities shown in Fig. A41.3 were estimated using correlations with void ratio and SPT blow count.

Instrument Selection and Free-Field Conditions

Instrumentation is not available near the centroid of the building, so sensors from the broad west end of the structure were used in the analyses (Table A41.1). Acceleration time histories recorded by these sensors are shown in Fig. A41.NR1.

Table A41.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 13	Ch 15	Ch 14	Ch 8	Ch 9

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A41.NR4) and the spectra of horizontal motions (Fig. A41.NR2-3) indicate amplification of GF motions relative to the FF near the first-mode

frequencies for both directions. This amplification was most pronounced in the NS direction, which had the higher level of shaking during NR.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A41.NR5. Minor amplification occurred near the first-mode frequencies for both directions, indicating that rocking may have affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A41.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A41.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A41.NR6).

Table A41.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=5, j=3		L (EW) d=4, j=8		tr (NS) d=6, j=3		L (EW) d=5, j=8	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	1.96	3.8	1.10	3.8	1.95	3.7	1.10	3.7	1.94	2.4 3.8	1.07	4.1
	2					4.63	10.7	1.95	5.0	4.56	13.0	1.95	6.1
	3					--	--	3.26	11.6	--	--	3.21	9.8

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in the transverse direction, and in its first three modes in the longitudinal direction.
- As discussed below, the NS CEM $\tilde{\zeta}$ value of 2.4% was adjusted based on RPEM results.
- The small variation between the fixed- and pseudo flexible-base parameters suggests that rocking did not significantly influence the structural response.

The RPEM results (Fig. A41.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. Unlike the CEM analyses, RPEM analyses indicated FF-R damping ratios which slightly exceed those for the GF-R case during stable portions of the time history after 8 seconds. Based on these results, a better estimate of flexible-base damping is $\tilde{\zeta} \approx 8.2$ Hz, as indicated in Table A41.2. The CEM and RPEM results were consistent with regard to the relative magnitude of the FF-R and 1F-R first-mode frequencies.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A41.3. The strain-dependent soil properties were estimated from deconvolution analyses to a

depth of 114 feet using the recorded free-field motions and the profile in Fig. A41.3. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table A41.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1060$ fps	$h = 38$ ft	emb.=0	$\gamma = 0.15$
NR	$\nu = 0.45$		tr:	$r_1 = 114$ ft	$r_2 = 114$ ft
	$\beta^* = 2.4\%$	$V_s^* = 1010$ fps	L:	$r_1 = 114$ ft	$r_2 = 114$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A41.3 with the fixed-base modal parameters in Table A41.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A41.4.

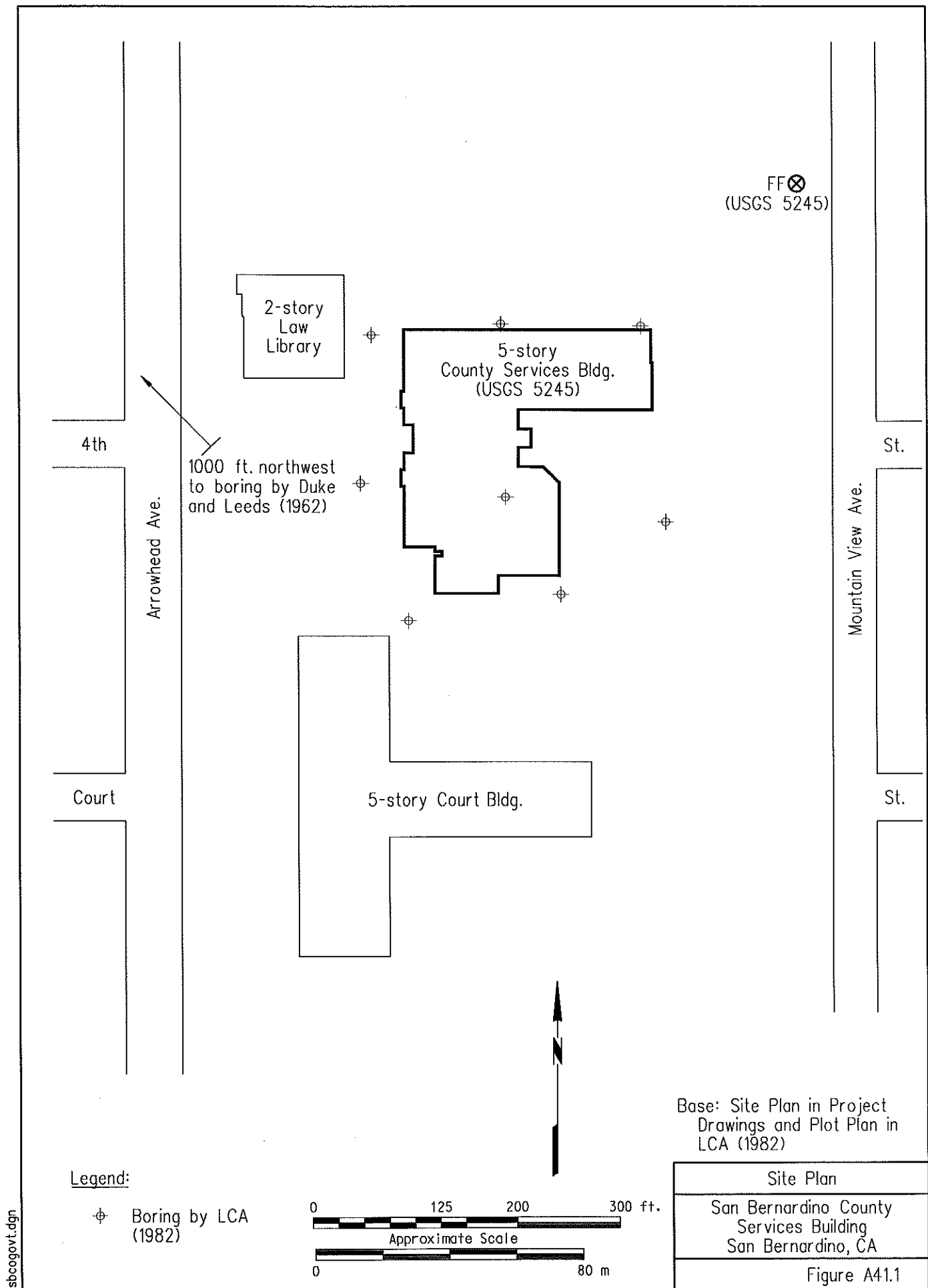
Table A41.4: Empirical and predicted inertial interaction effects

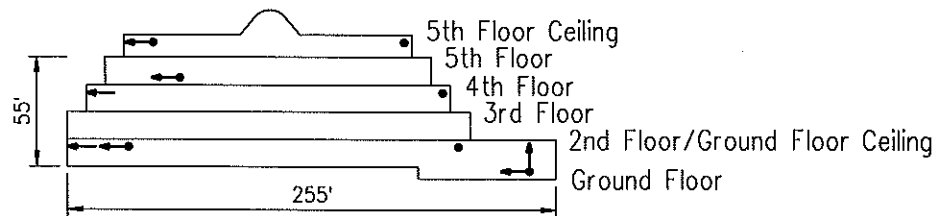
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.01	0.1	3.8	1.03	0.6	4.1
	Prediction	1.03	1.8	5.2	1.01	0.4	4.0

The period lengthening and foundation damping are reasonably well predicted, though the foundation damping is somewhat overpredicted for the NS direction.

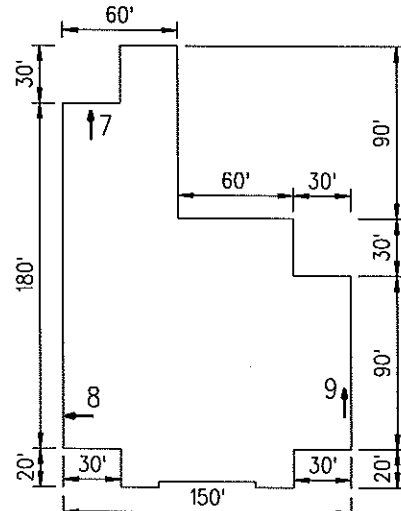
Data Sources

- Duke, C.M. and Leeds, D.J. (1962). "Site characteristics of Southern California strong motion earthquake stations," *Rpt. No. 62-55*, Dept. of Engrg., U.C. Los Angeles.
- LeRoy Crandall Assoc., LCA (1982). "Report of Foundation Investigation, Proposed San Bernardino County Center, Fourth Street Between Mountain View and Arrowhead Avenues, San Bernardino, CA," *Job. No. A-82018*.

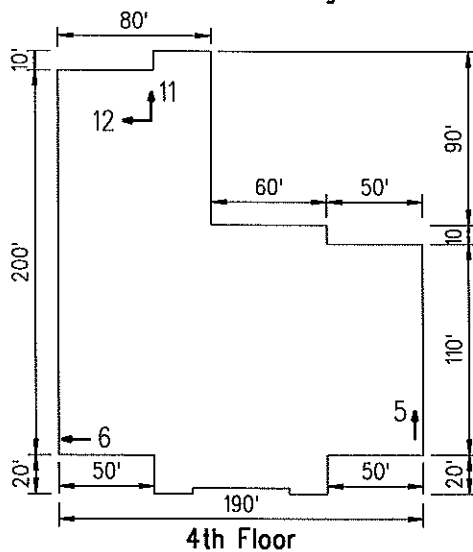




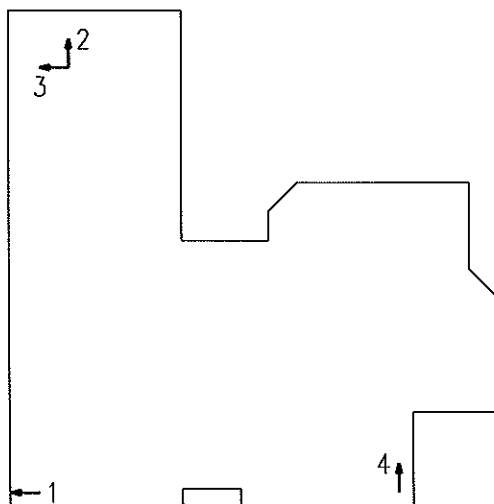
West Elevation



5th Floor Ceiling

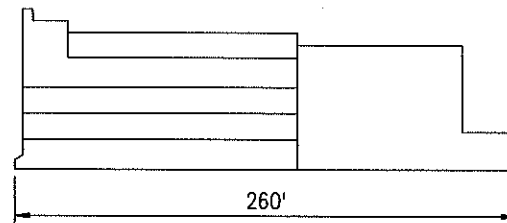


4th Floor

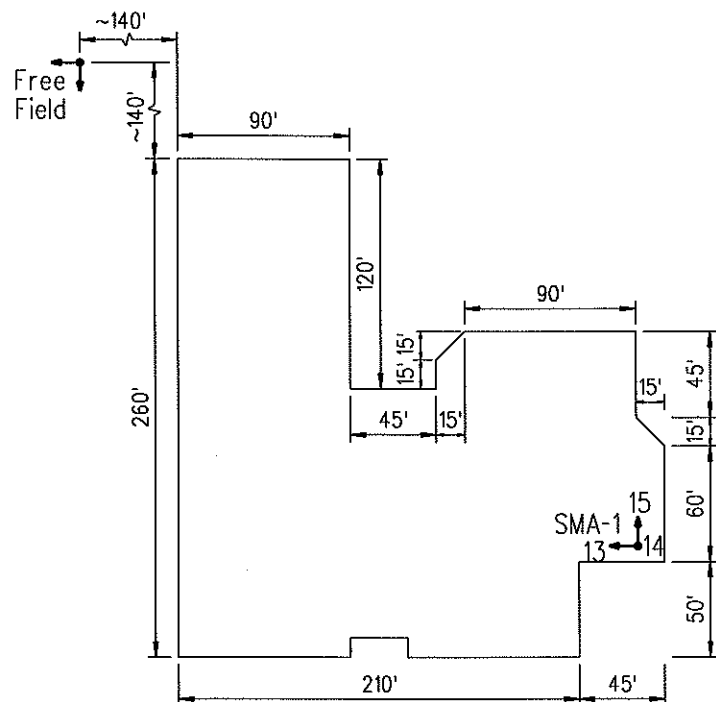


Ground Floor Ceiling

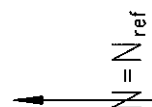
Braced steel frame.
Instruments installed 1987.



South Elevation



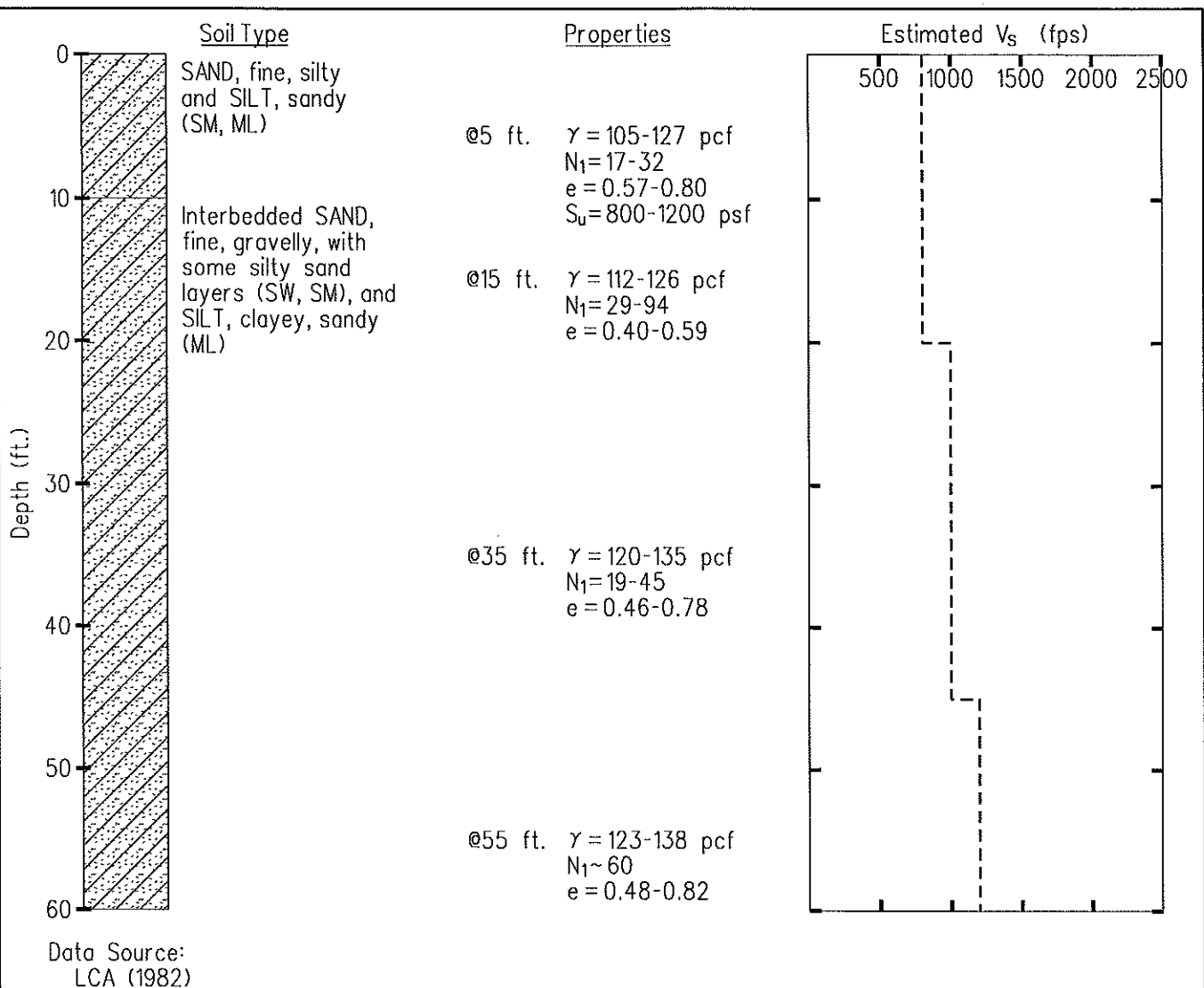
Ground Floor



Note: Data provided by USGS

Sensor Locations
San Bernardino County
Services Building
San Bernardino, CA

Figure A41.2



Notes:

1. Groundwater encountered at 25 ft. depth.
2. Penetration resistance established by SPT techniques.
3. Shear strengths established by direct shear tests at normal pressures appropriate for the sample depths.
4. Shear wave velocities estimated based on correlation by Fumal & Tinsley (1985) and Seed et al. (1984).

Generalized Soil Profile

San Bernardino County
Services Building
San Bernardino, CA

Figure A41.3

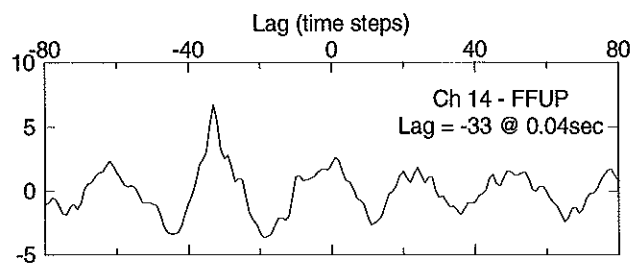


Fig. A41.NR0: Cross-correlation function between vertical ground floor and free-field accelerations

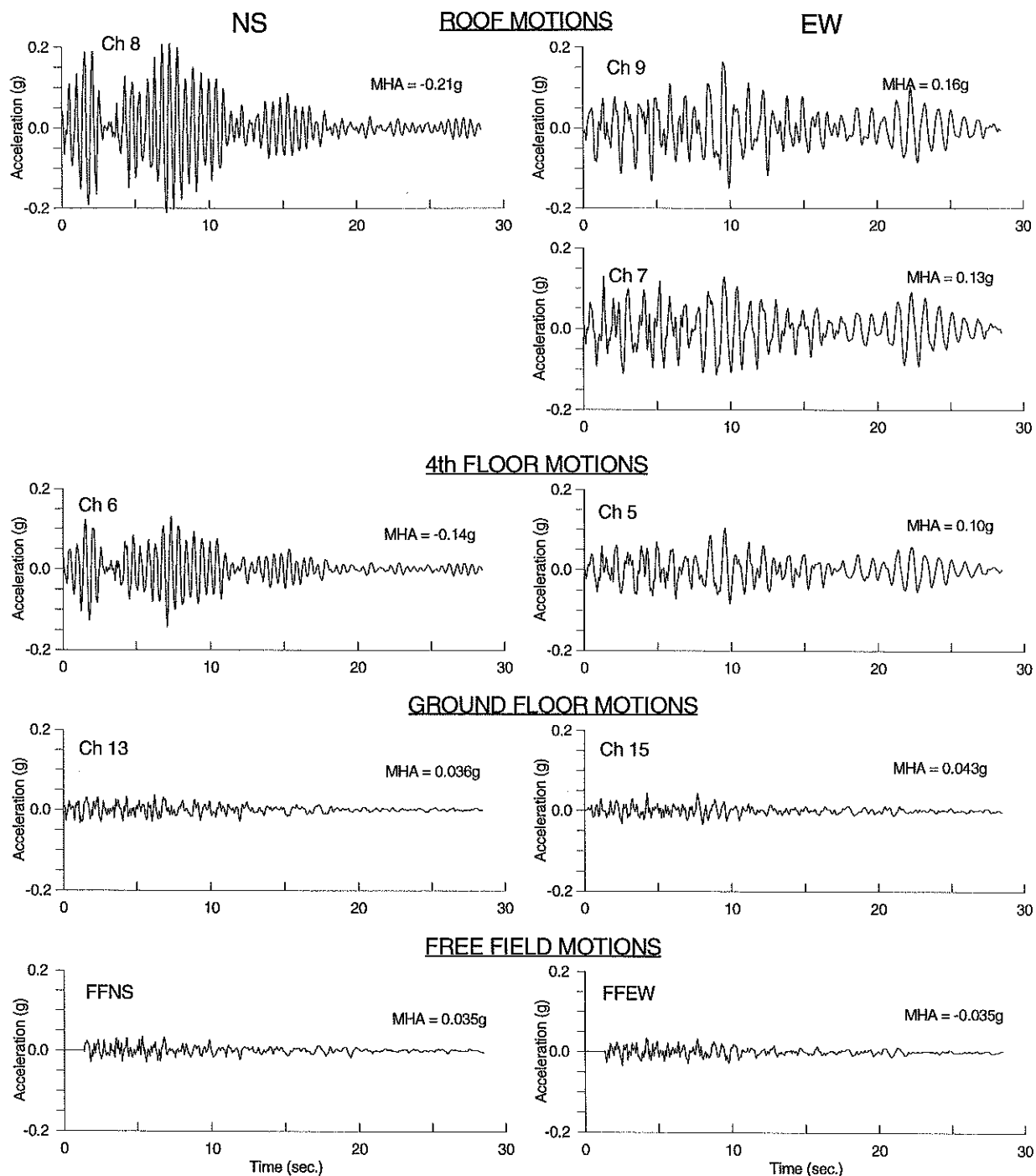


Fig. A41.NR1: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

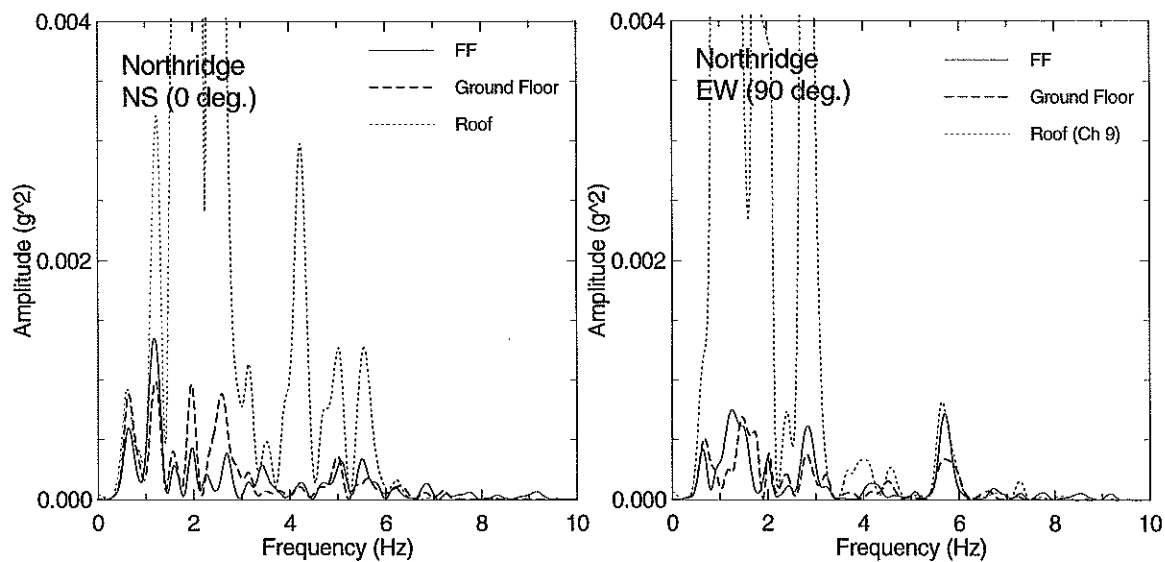


Fig. A41.NR2: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

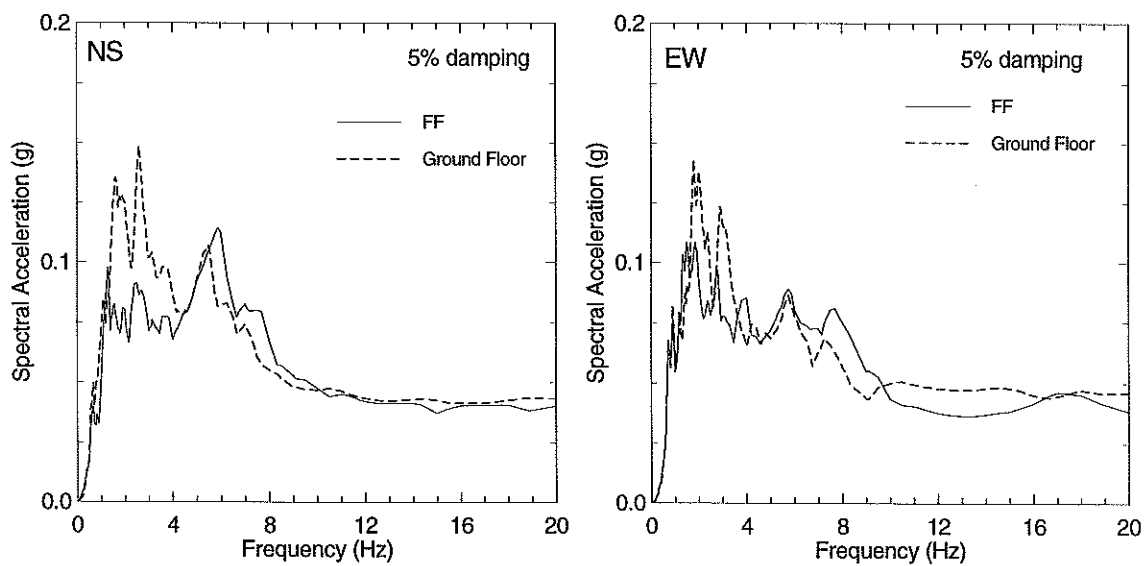


Fig. A41.NR3: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1994 Northridge Earthquake

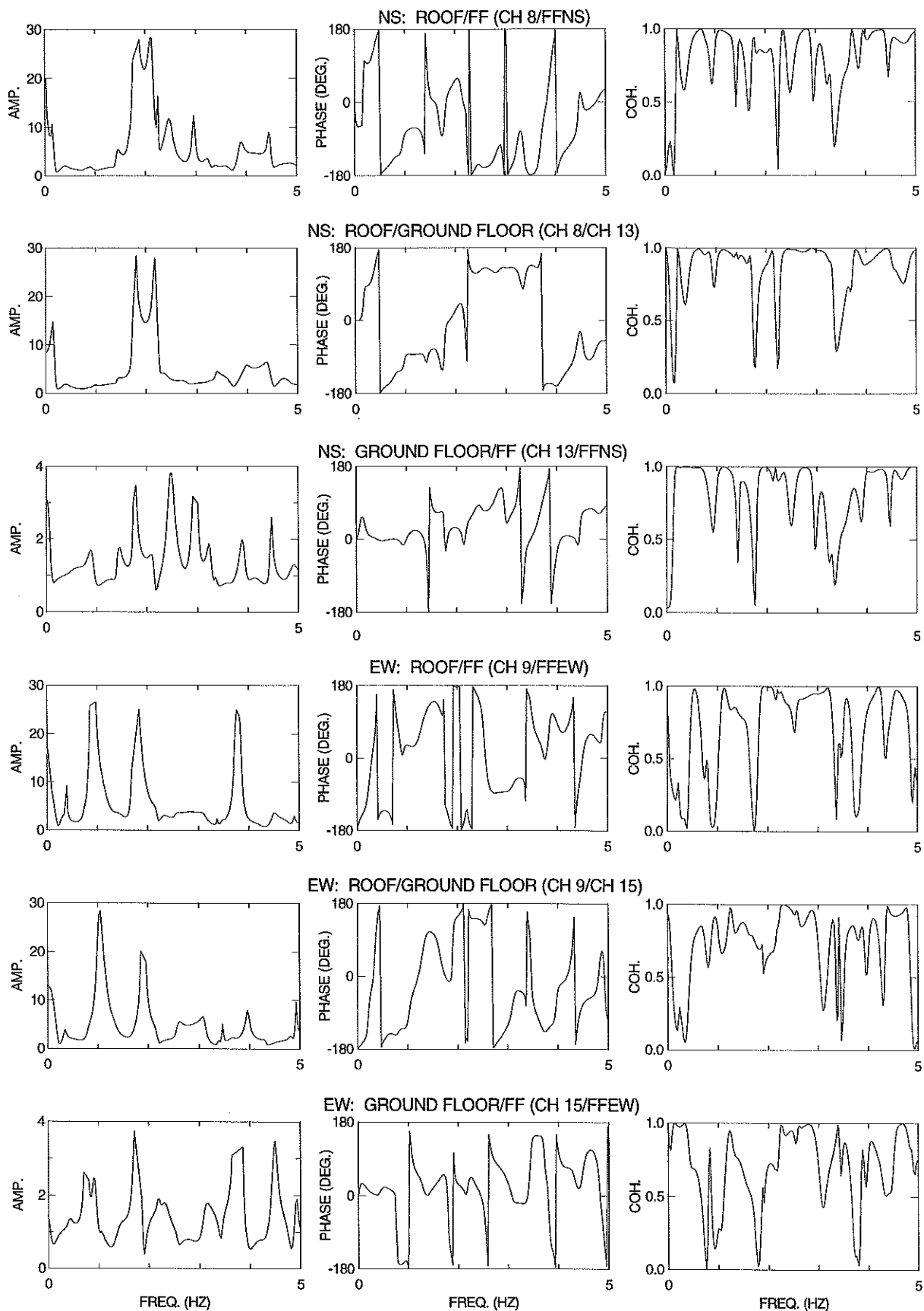


Fig. A41.NR4: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Transmissibility and Coherency Functions
1994 Northridge Earthquake

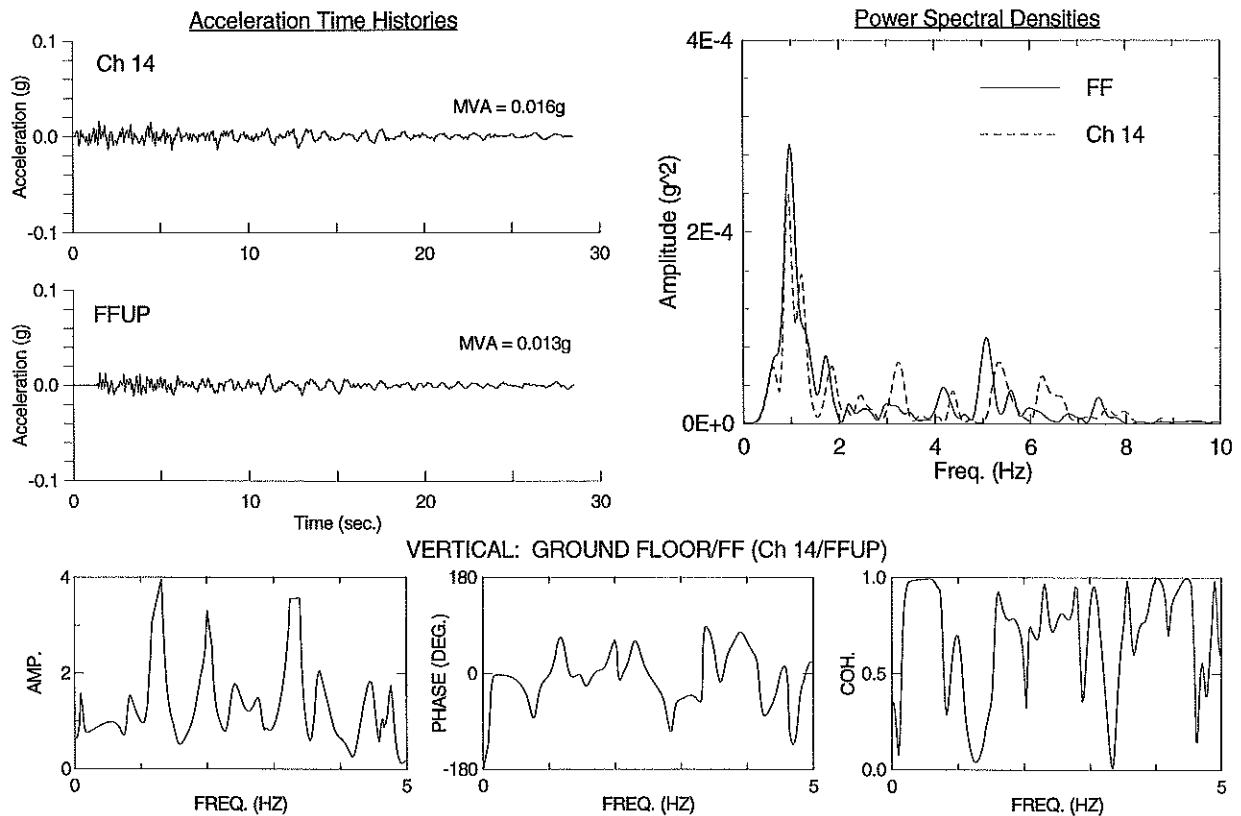


Fig. A41.NR5: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Analysis of Vertical Motions
1994 Northridge Earthquake

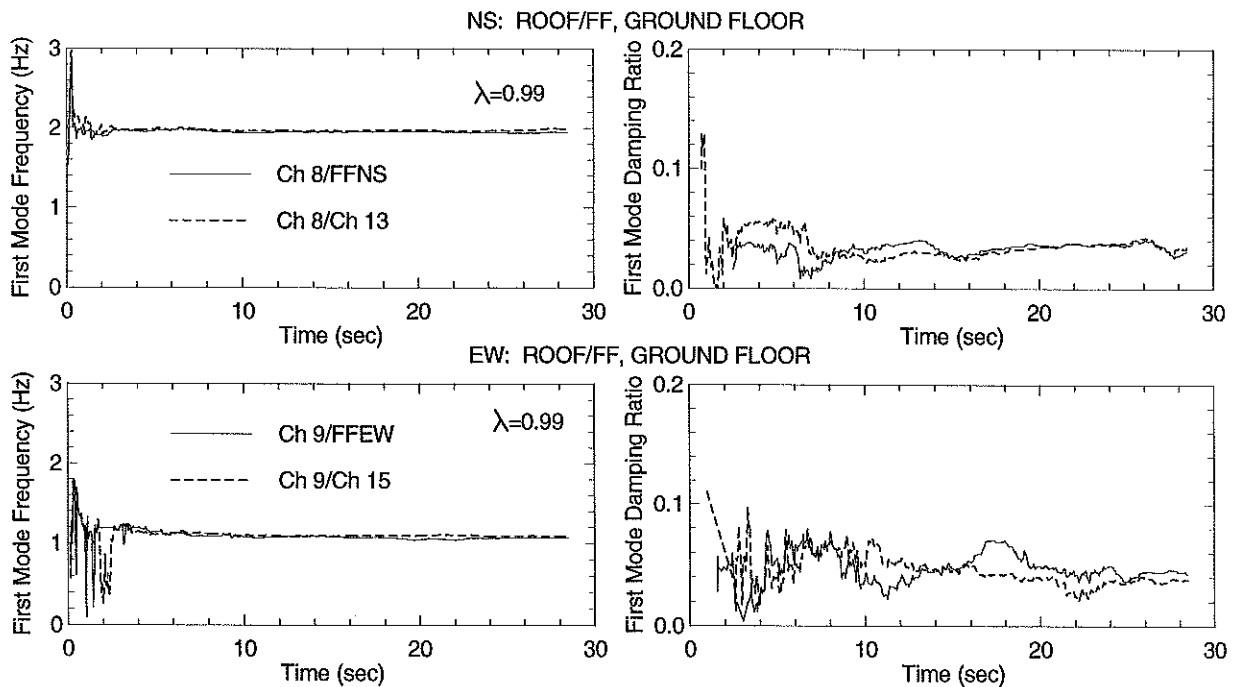


Fig. A41.NR4: SAN BERNARDINO COUNTY GOVERNMENT CENTER
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A42

Site Name: Santa Susana ETEC Building 462
Sensors: USGS 5108

Lat: 34.231°
Long.: 118.713°

Floor Levels: 8
Bsmt. Levels: 0
Structure: Steel braced frames

Foundation: footings
Site Geology: rock
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier *	0.03	0.03	0.01	0.02	0.02	0.01	0.07	0.08	0.04
1994 Northridge	0.26	0.28	0.17	0.33	0.25	0.21	0.61	0.41	0.63

* Not digitized

Preprocessing

Northridge Earthquake (NR)

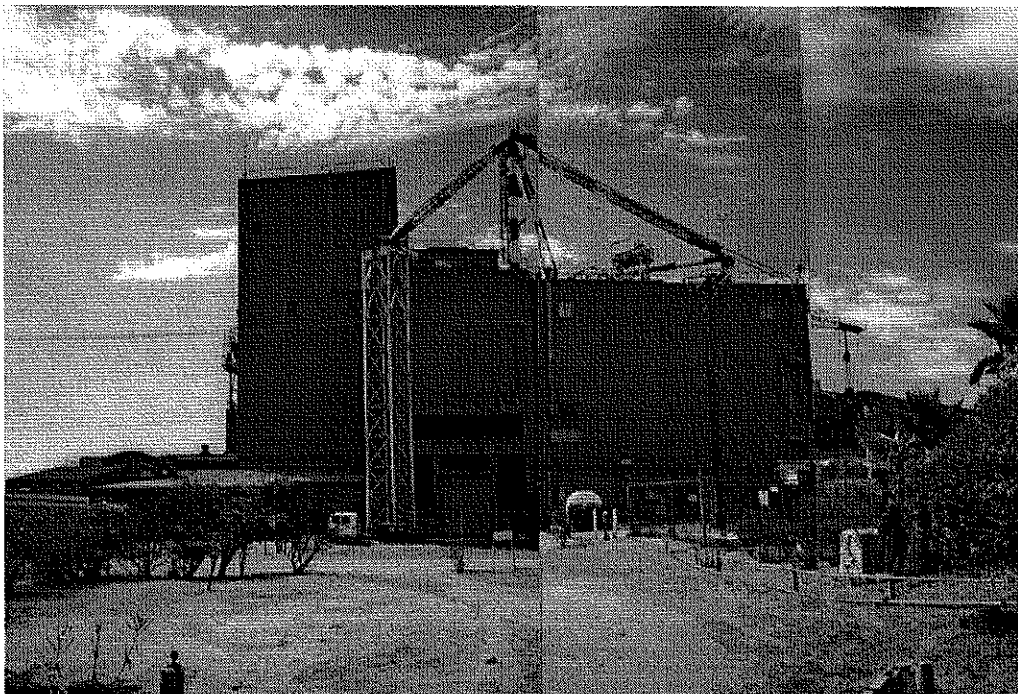
Preliminary: *Str. Data*: USGS, $f_{HP} = 0.1$ Hz; $f_{Nyq} = 100$ Hz. *FF Data*: Agbabian Assoc., not filtered, $f_{Nyq} = 100$ Hz; FF/str. data synchronized.

Final: FF filtered (this study), $f_{HP} = 0.1$ Hz; FF/str. data decimated by 4 to $f_{Nyq} = 25$ Hz.

Figures

A42.1-A42.3
A42.NR1-6

Site Plan, Sensor Locations, Generalized Soil Column
NR Analysis Results



View of structure with FF in foreground (instrumented structure is the tallest section of the building on the left side of the photograph)

DISCUSSION

Site Description

The instrumented section of Building 462 is an 8-level, 114-foot high rectangular structure with plan dimensions of 25 by 50 feet. Located immediately north and south of the 8-level tower are 1- and 4-level structures, respectively (Fig. A42.1). Building 026, located west of Building 462, is an open bay between Buildings 462 and 463. Foundations for Building 462 consist of 2-foot deep continuous footings which extend between columns in both directions, and an 8-inch floor slab is present across the base of the structure. The footings are not integrally connected to the foundation of the adjacent 1-story structure, but are connected through slab reinforcement to the top of a retaining wall at the north end of the adjacent 4-story structure. Mass in the structure is concentrated at the ground and 6th levels where liquid storage tanks and pumps are located. Maintenance staff report that only the ground floor liquid storage tanks were full at the time of the Northridge Earthquake. The 6th level has a concrete floor, while other floors consist of open-grid steel decking and have little permanent equipment. The lateral force resisting system consists of inverted chevron-type braced steel frames in both directions. As shown in Fig. A42.2, accelerographs are present on the ground and 6th levels, and the free-field instrument is located in an open area about 160 feet north of the structure. The ground floor instrument is located immediately north of a 6 to 8-foot deep pit for a liquid storage tank. A reinforced concrete retaining wall separates the ground floor area where the instrument is located from the pit.

The site is located on level terrain in the broad Burro Flats of the Simi Hills. Geologic conditions were investigated to a depth of 40 feet by LCA (1968), and consisted of very firm to hard sandstone bedrock beneath a shallow mantle of surficial soil. The bedrock in this area belongs to the Chatsworth Formation which dates to the Upper Cretaceous Period (Dibblee, 1992). No shear wave velocity measurements are available at the site, and no shear wave velocity data in this geologic formation was found in the literature at the time of this writing. Hence, the best available data appears to be downhole measurements in gneiss bedrock of similar age near Pacoima Dam (Roblee, 1996). These data are reported in Fig. A42.3.

Instrument Selection and Free-Field Conditions

The structural instrumentation consists of triaxial accelerographs on the ground and 6th levels. The sensor numbers used in the analyses are indicated in Table A42.1, and acceleration time histories recorded by these sensors are shown in Fig. A42.NR1.

Table A42.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 3	Ch 2	Ch 1	Ch 6	Ch 5

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A42.NR4) and the spectra for horizontal motions (Fig. A42.NR2-3) indicate amplification of NS GF motions and de-amplification of EW GF motions for frequencies greater than about 2 Hz. The high frequency amplification of NS GF motions may have resulted from topographic amplification associated with the 6 to 8-foot tall free-face just south of the GF instrument.

Transmissibility, coherency, and power spectral density functions for the FF and GF vertical motions are presented in Fig. A42.NR5. No significant amplification occurred near either of the first-mode frequencies, indicating that base rocking was unlikely to have significantly affected the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A42.NR4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A42.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A42.NR6).

Table A42.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS) estimated		L (EW) estimated		tr (NS) d=2, j=4		L (EW) d=2, j=4		tr (NS) d=0, j=4		L (EW) d=1, j=4	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	--	--	--	--	1.84	13.7	1.81	7.1	1.79	17.2	1.83	11.1
	2					4.42	17.9	4.59	11.6	4.42	22.2	4.58	15.3

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode.
- The large aspect ratio of the structure caused estimates of the fixed-base parameters to be highly sensitive to the CEM results and foundation dimensions. Given the uncertainty associated with the foundation radius (the structure's footing foundations are connected to foundations of neighboring structures) and the normal uncertainty in the CEM pseudo flexible- and flexible-base parameters, these estimates are unreliable and are not reported. However, as the structure is light above the ground floor (the 6th floor liquid storage tank was empty at the time of the earthquake), and no significant rocking response was evident from the vertical GF/FF transmissibility function, the fixed-base parameters can be reasonably approximated by the pseudo flexible-base parameters.

The RPEM results (Fig. A42.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The relative magnitudes of the first-mode frequencies and damping ratios for the GF-R and FF-R cases generally confirm the CEM results.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A42.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 24 feet using the recorded free-field motions and the profile in Fig. A42.3. The foundation of the building was assumed to be independent of the adjacent structures for the modeling of foundation impedance. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by 30% to account for shape effects.

Table A42.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 4500$ fps	$h = 91$ ft	emb.=0	$\gamma = 0.05$
NR	$\nu = 0.4$		tr:	$r_1 = 23.5$ ft	$r_2 = 20.5$ ft
	$\beta^* = 1.0\%$	$V_s^* = 4460$ fps	L:	$r_1 = 23.5$ ft	$r_2 = 27.6$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A42.3 with the fixed-base modal parameters in Table A42.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A42.4.

Table A42.4: Empirical and predicted inertial interaction effects

NR	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.03	4.6	17.2	0.99	4.0	11.1
	Prediction	1.01	0.0	13.5	1.00	0.0	7.1

Period lengthening ratio was well predicted by the analyses, while foundation damping was significantly underpredicted. The error in foundation damping may be associated with the effective area of the foundation system being larger than the footprint area of Building 462 (which was used for the predictions). Also, given the CEM FF-R and GF-R damping values reported in Table A42.2, if ζ had been computed by the parameter estimation procedures, it would have exceeded $\tilde{\zeta}^*$, which would yield a smaller $\tilde{\zeta}_0$ value.

Data Sources

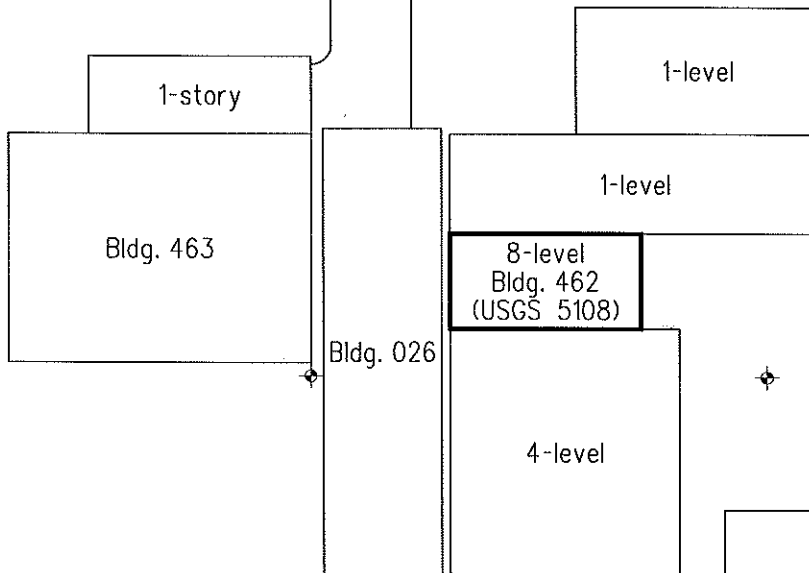
LeRoy Crandall Assoc., LCA (1968). "Report of Foundation Investigation, Proposed Sodium Pump Test Facility, Area bounded by 'F', 22nd, 'G', and 24th Streets, Santa Susana, CA," Job. No. A-68232.

Dibblee, T.W. (1992). "Geologic map of the Calabasas Quadrangle, Ventura and Los Angeles Counties, CA," Map DF-37, Dibblee Geological Foundation.

Robblee, C. (1996). pers. communication regarding downhole shear wave velocity measurements at Pacoima Dam downstream station.

ETEC Ground Site
 ⊗FF
 (USGS 5108)

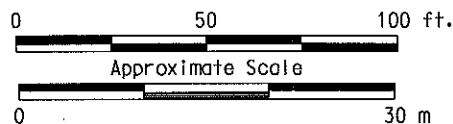
F St.



Base: Project Drawings for Sodium Pump Test Facility
 (1971), provided by Rockwell Energy Technology
 Engineering Center, and Plot Plan by LCA (1968)

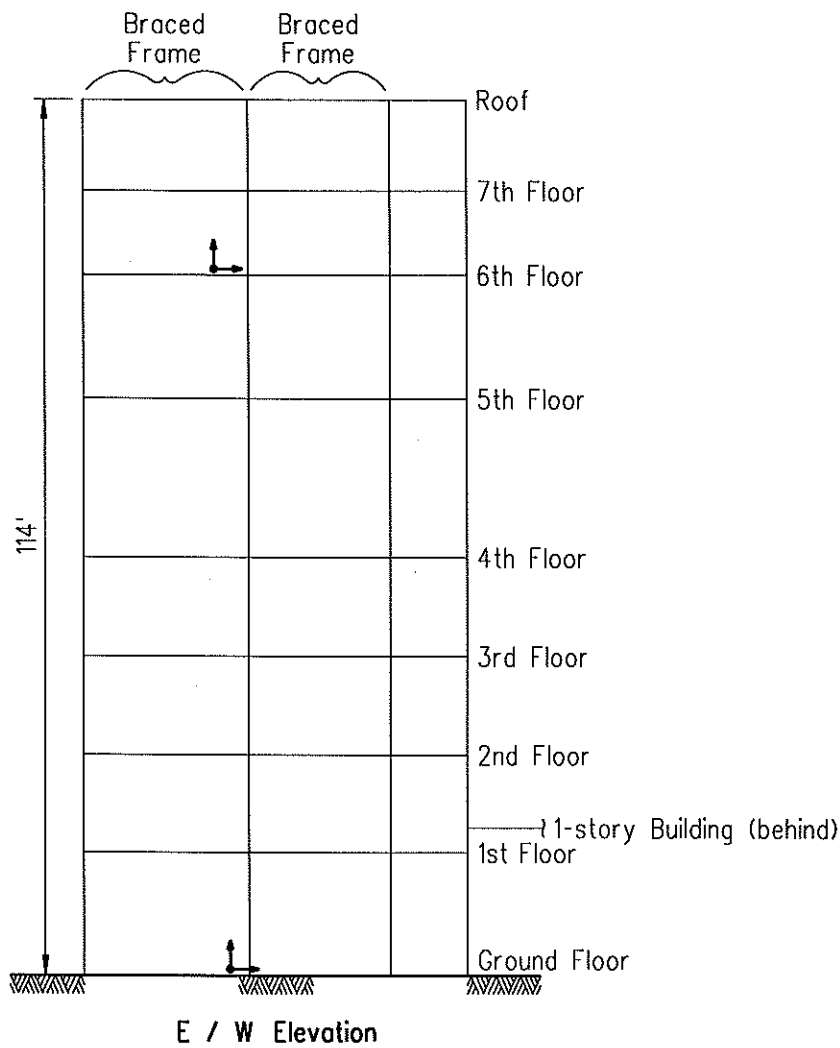
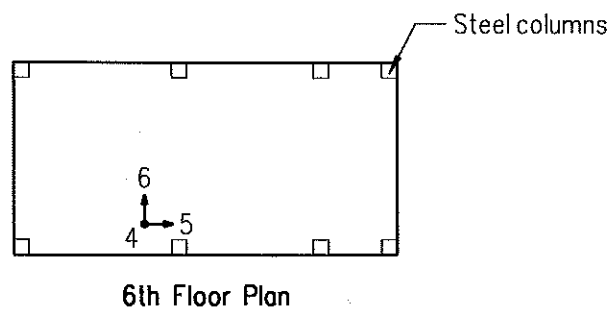
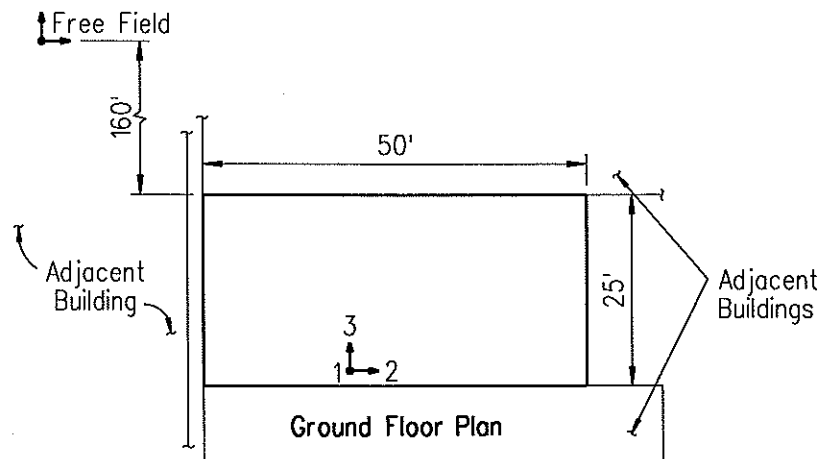
Legend:

⊗ Boring by LCA (1968)



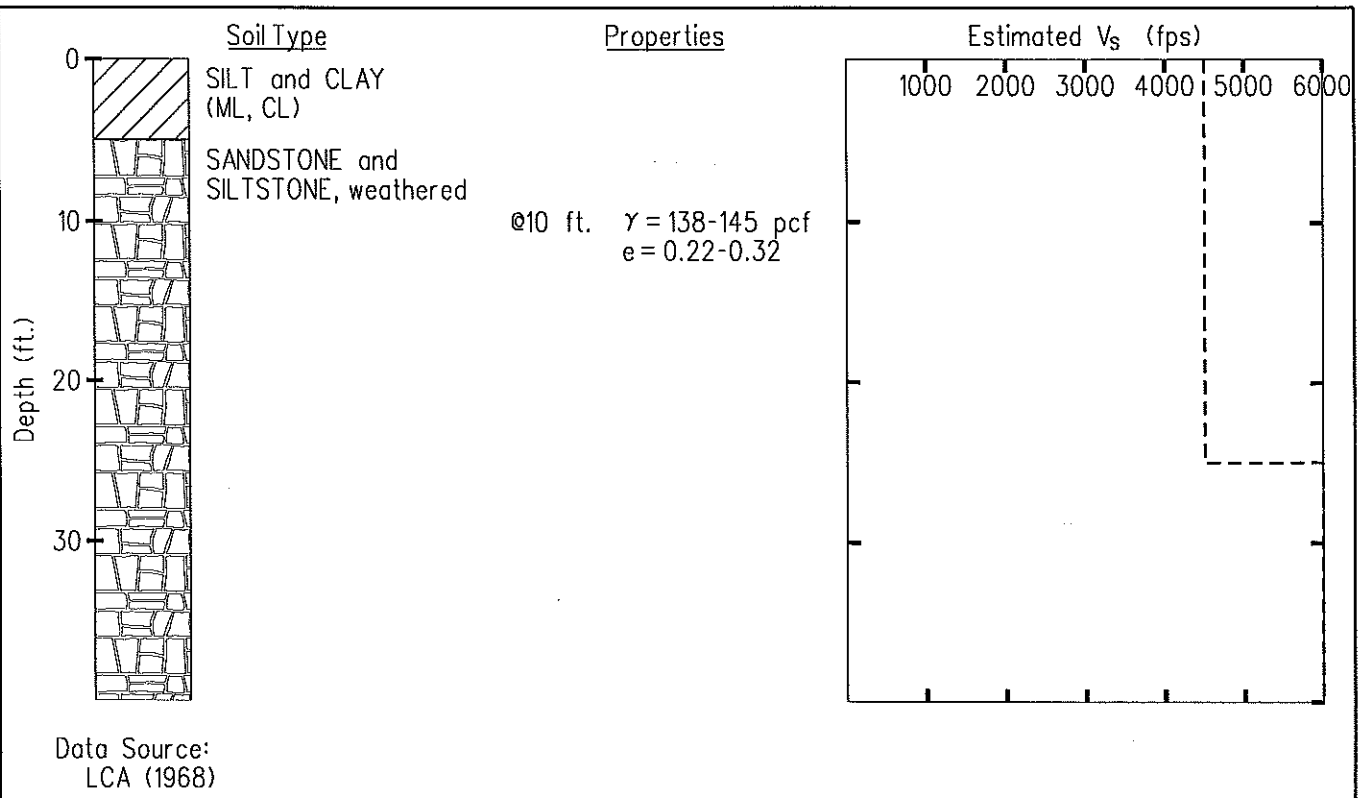
Site Plan
Energy Technology Engineering Center, Building 462 Santa Susana, CA
Figure A42.1

sanitasuz.dgn



Note: Drafted from structural drawings provided by ETEC staff

Sensor Locations
Energy Technology Engineering Center, Building 462 Santa Susana, CA
Figure A42.2



Notes:

1. No groundwater encountered.
2. Shear wave velocities estimated based on downhole measurements in geologic materials of similar age near Pacoima Dam (Robblee, 1996).

Generalized Soil Profile
Energy Technology Engineering Center, Building 462 Santa Susana, CA
Figure A42.3

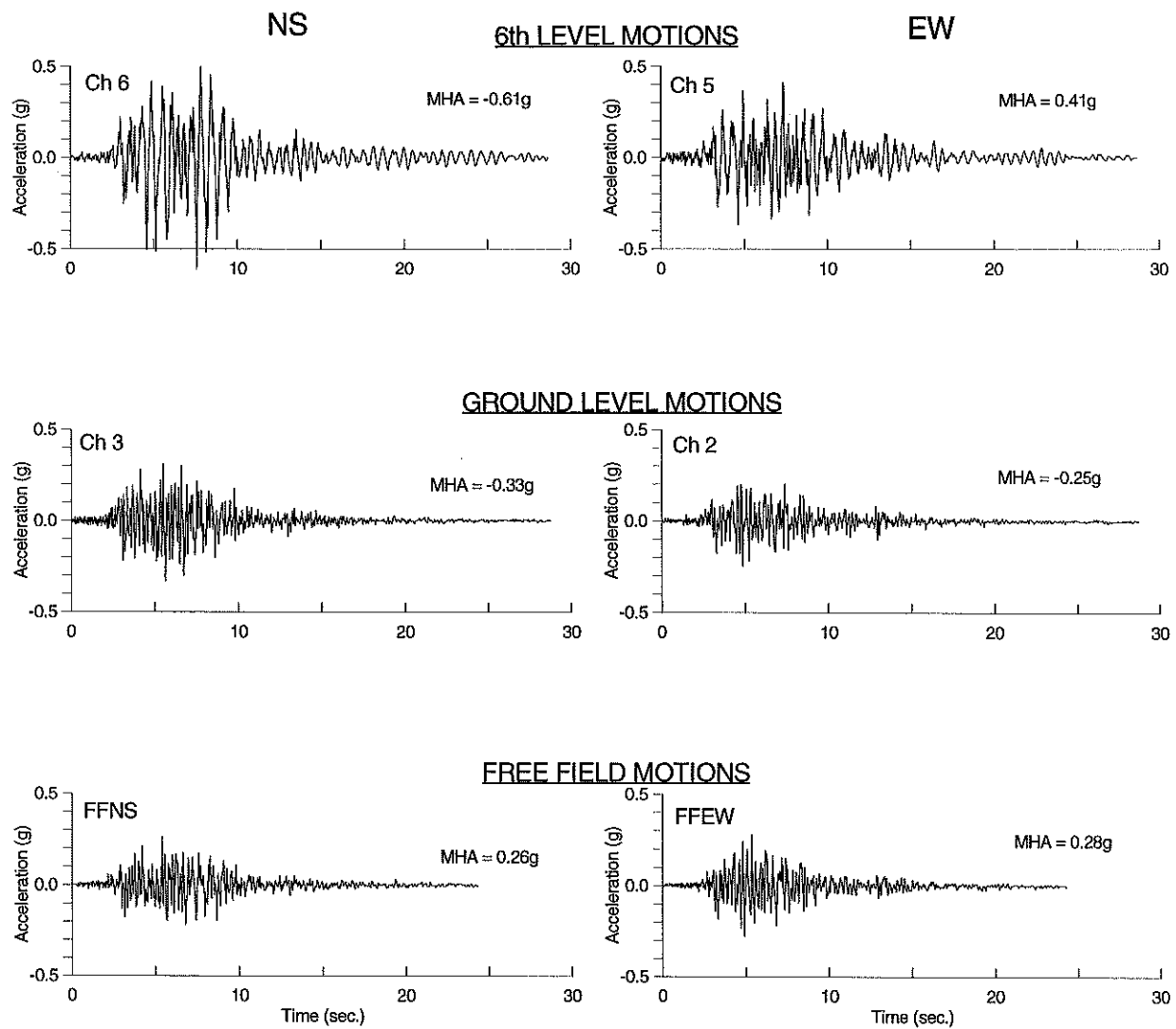


Fig. A42.NR1: SANTA SUSANA ETEC BUILDING 462
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

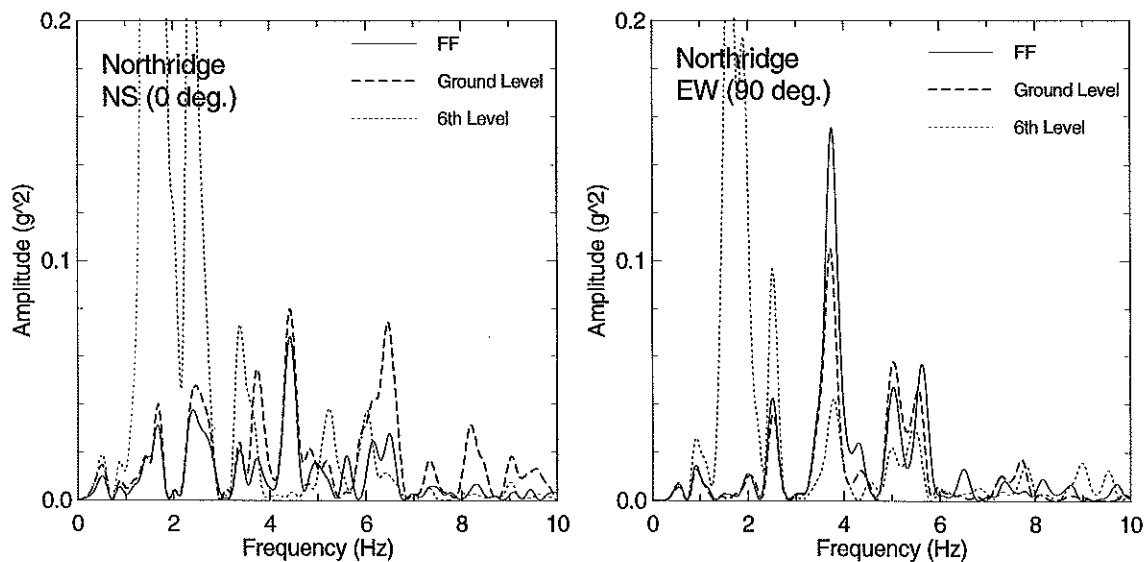


Fig. A42.NR2: SANTA SUSANA ETEC BUILDING 462
Power Spectral Density of Horizontal Accelerations
1994 Northridge Earthquake

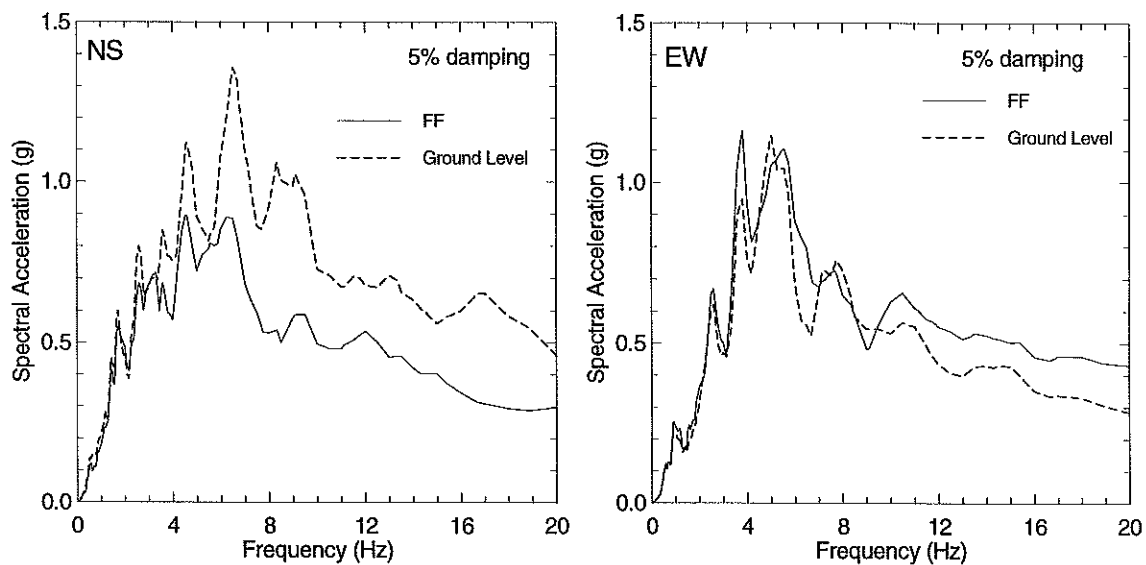


Fig. A42.NR3: SANTA SUSANA ETEC BUILDING 462
Acceleration Response Spectra of Ground Level and Free-Field Motions
1994 Northridge Earthquake

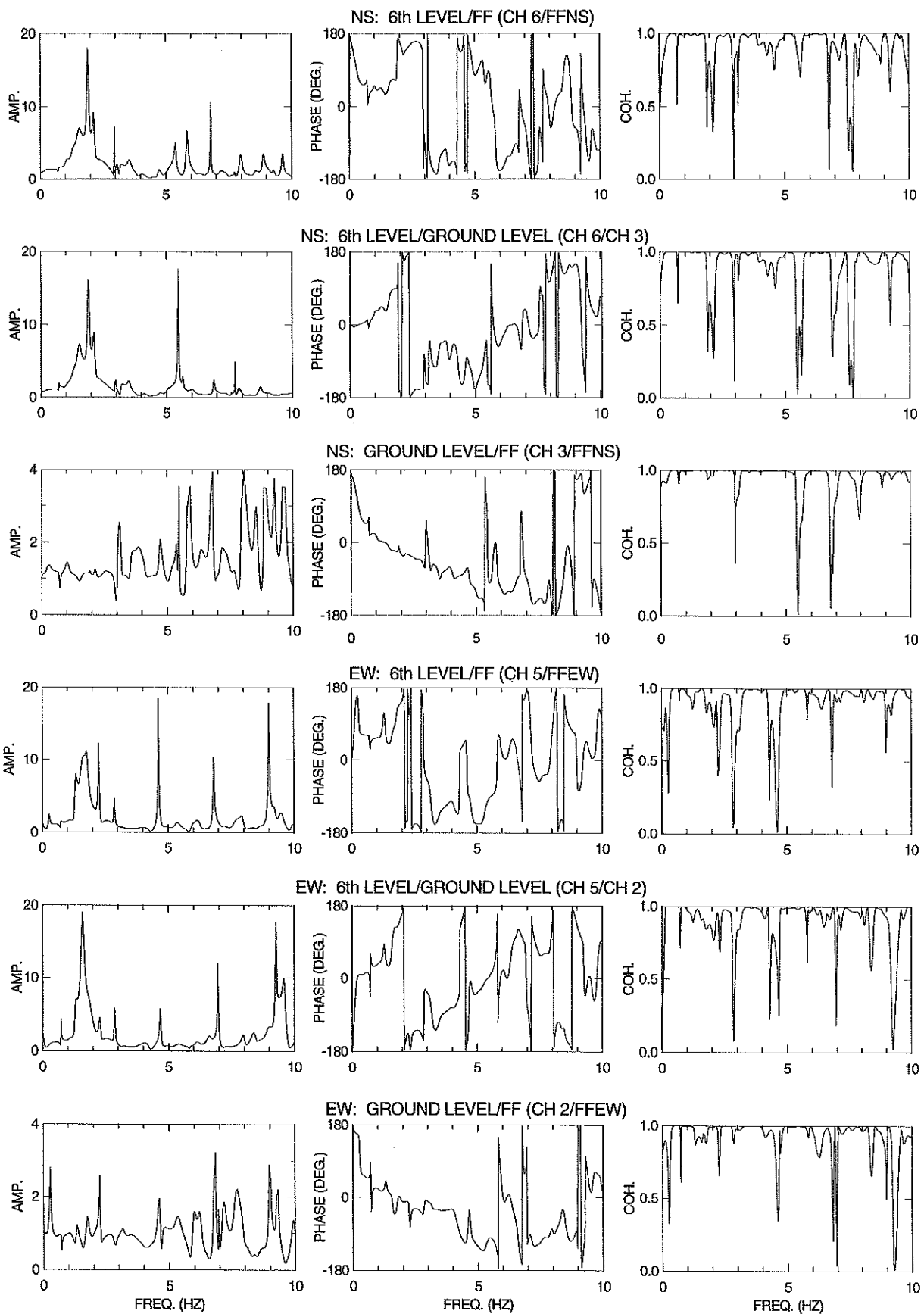


Fig.A42.NR4: SANTA SUSANA ETEC BUILDING 462
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A42-10

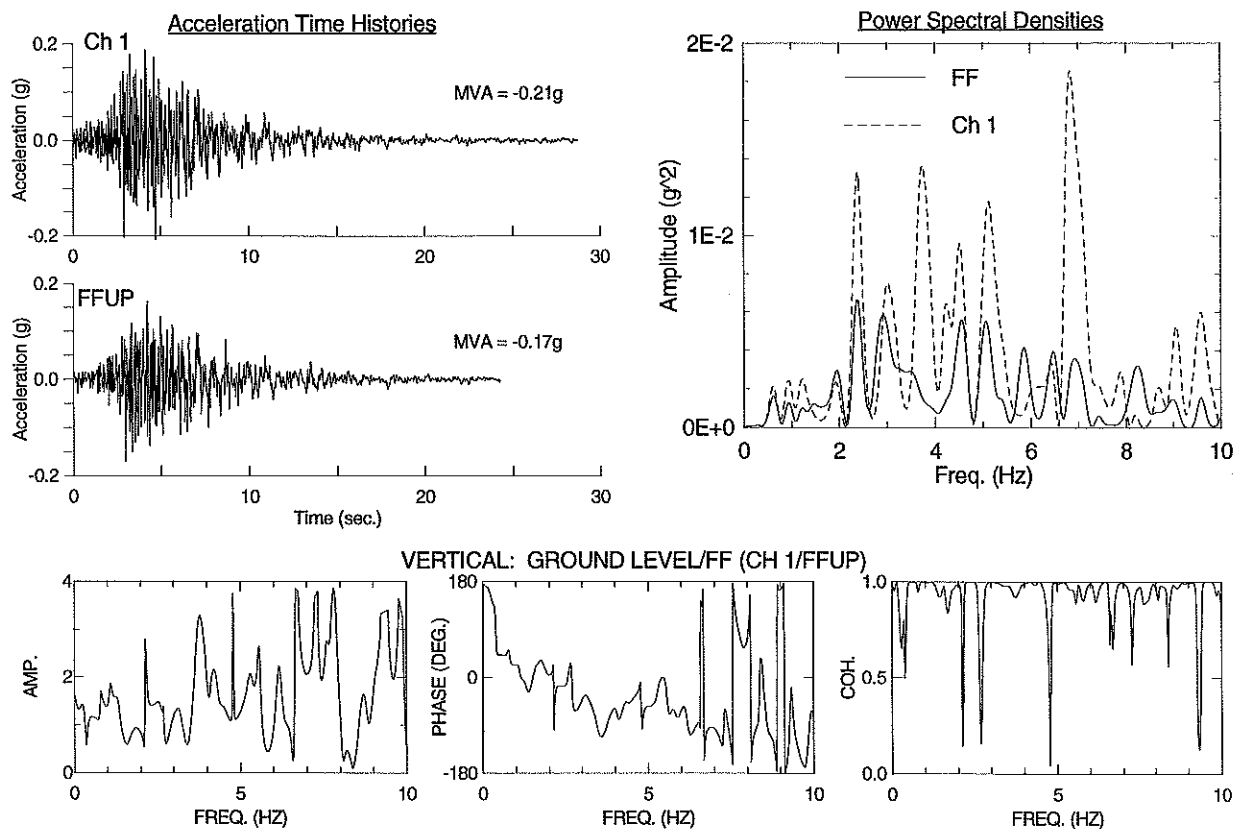


Fig. A42.NR5: SANTA SUSANA ETEC BUILDING 462
Analysis of Vertical Motions
1994 Northridge Earthquake

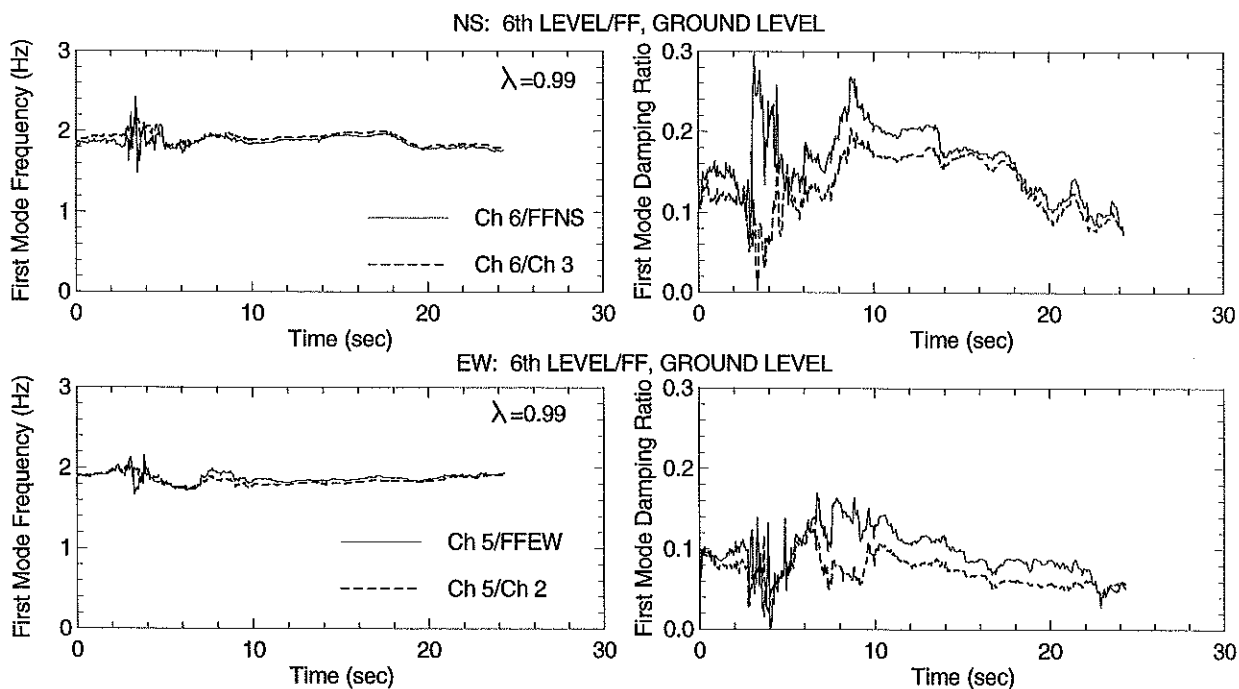


Fig. A42.NR6: SANTA SUSANA ETEC BUILDING 462
Time Variation of First Mode Parameters
1994 Northridge Earthquake

SITE A43

Site Name: Seal Beach Rockwell Building
Sensors: CSMIP 14578

Lat.: 33.757°
Long.: 118.084°

Floor Levels: 9
Bsm't. Levels: 1
Structure: Concrete frame, isolators between
1st and 2nd floors

Foundation: concrete piers
Site Geology: Soil
Design Date: 1967, retrofit
1990

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Basement			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1991 Sierra Madre*	0.02	0.02	0.01	0.02	0.02	0.01	0.03	0.03	0.05
1992 Landers	0.044	0.046	0.016	0.037	0.044	0.014	0.090	0.092	0.067
1992 Big Bear*	0.04	0.04	0.02	0.04	0.04	0.02	0.08	0.06	0.07
1994 Northridge	0.061	0.083	0.037	0.050	0.079	0.025	0.12	0.14	0.16

*Not digitized

Preprocessing

Landers Earthquake (LD)

Preliminary: CSMIP; filtered $f = 0.16\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data synchronized.

Final: data decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$.

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.3\text{--}46\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. synchronized.

Final: data decimated by 4 to $f_{Nyq} = 12.5\text{ Hz}$.

Figures

A43.1-A43.3

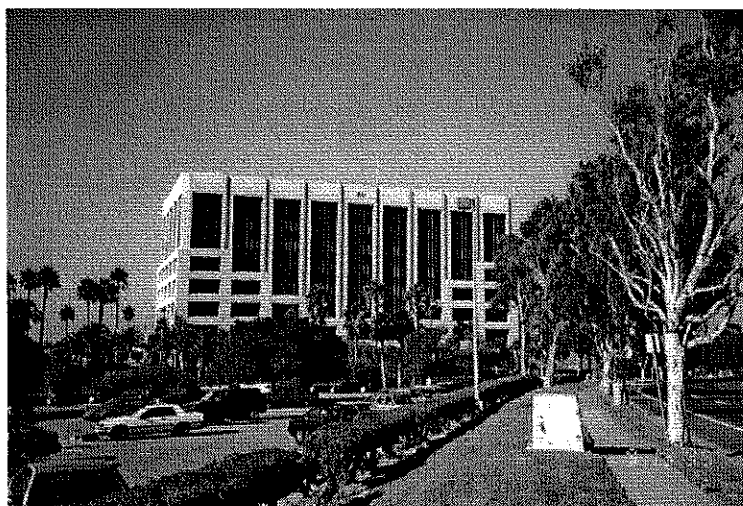
Site Plan, Sensor Locations, Generalized Soil Column

A43.LD1-6

LD Analysis Results

A43.NR1-6

NR Analysis Results



View of structure with FF in foreground

DISCUSSION

Site Description

The building is 8-stories tall with a single-level stepped basement, and is rectangular in plan with dimensions 244.5 by 136.5 feet. Originally designed in 1967, the structure was retrofitted in 1990 by installing isolators in the columns between the 1st and 2nd floors, and by stiffening the exterior concrete frames. The foundations consist of concrete piers which are typically 3 feet in diameter and 50 feet long. No structures of significant size are located immediately adjacent to the building. As shown in Fig. A43.2, accelerographs are present at the basement and roof levels, as well as the 1st, 2nd and 6th floors. The free-field instrument is located adjacent to a parking lot about 455 feet south of the building.

The site is located on level terrain, about 4000 feet east of the San Gabriel river and 8000 feet north of the Pacific coast. Soil conditions at the site were investigated by Dames and Moore (1988), and were found to consist of: 25 feet of stiff, silty clay, 10 feet of dense sand, 12 feet of very stiff to hard silty clay, and 53 feet of very dense sand which extended to the maximum depth explored of 100 feet. The depth to bedrock is poorly characterized in the area. Based on measurements by Dames and Moore, groundwater occurs at a depth of about 22 feet at the site. The shear wave velocities shown in Fig. A43.3 are based on in situ downhole measurements by Dames and Moore.

Instrument Selection and Free-Field Conditions

Instruments located near the centroid of the structure were selected for analysis (Table A43.1). Acceleration time histories recorded by these and other sensors are shown in Fig. A43.**1. The two vertical instruments located in the basement level were used to evaluate rocking effects in the transverse (NS) direction.

Table A43.1: Sensors in structure used for analyses

Basement (B)			1st Floor (1F)		2nd Floor (2F)		Roof (R)	
NS	EW	V	NS	EW	NS	EW	NS	EW
Ch 7	Ch 6	Ch4,5	Ch 10	Ch 8	Ch 19	Ch 25	Ch 16	Ch 14

The following input was used in system identification analyses to evaluate fixed-base structural parameters for the NS direction,

$$\text{input} = \text{Ch7} + (\text{Ch5} - \text{Ch4}) \cdot 132/136.5 \quad (1)$$

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Basement and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

B/FF transmissibility functions (Fig. A43.**4) and the spectra for horizontal motions (Fig. A43.**2-3) generally indicate de-amplification of basement motions for frequencies greater than about 4 to 6 Hz.

Transmissibility functions for B/FF pairs of vertical motions (Fig. A43.**5) indicate little or no amplification at the first-mode frequency for NS response, suggesting that base rocking did not significantly affect the structural response. A direct examination of rocking contributions to roof motion is made in Fig. A43.**5 by comparing relative lateral roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ values are negligibly small compared to the relative roof displacements, confirming that rocking did not significantly affect roof motions.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A43.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A43.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A43.**6). Frequencies identified by Dames and Moore (1988) prior to the building retrofit are presented in Table A43.3.

Table A43.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base [B-R & (1F-R)*]				Flex.-Base (FF-R)			
		tr (NS) LD: d=5, j=6 NR: d=3, j=6		L (EW) estimated		tr (NS) d=5(4), j=6(6) d=3(3), j=6(6)		L (EW) d=4(4), j=5(5) d=4(3), j=6(6)		tr (NS) d=4, j=6 d=4, j=8		L (EW) d=3, j=5 d=4, j=6	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	0.79	11.0	0.90	14.1	0.77 (0.79)	10.9 (10.7)	0.89 (0.90)	13.7 (13.5)	0.78	10.6	0.87	12.9
	2	2.49	5.8			2.52 (2.56)	6.3 (6.6)	2.86 (2.88)	5.1 (5.6)	2.49	6.3	2.88	5.7
NR	1	0.84	7.7	0.93	8.3	0.82 (0.83)	7.2 (7.0)	0.92 (0.93)	8.2 (8.2)	0.83	7.1	0.92	8.3
	2	2.40	5.7			2.43 (2.47)	6.0 (6.3)	2.81 (2.84)	6.2 (6.9)	2.38	4.0	2.80	6.0

* 1st Floor → Roof results shown in parentheses ()

Table A43.3: Measured response frequencies (in Hz) prior to building retrofit (Dames and Moore, 1988)

Mode	NS	EW
1	0.96	1.20
2	3.04	3.76

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure generally responded strongly in its lower two modes. The first mode tended to dominate during the long duration shaking of LD, while the second mode was also significant under the higher amplitude but lower duration shaking of NR. The base isolators caused large first-mode damping ratios (e.g. 11 to 14% for LD, B-R case).
- The small variations between the fixed- and pseudo flexible-base parameters are consistent with the lack of evidence for a significant rocking response noted in Part (a).
- The structural response above the isolators was stiffer and more lightly damped than is reflected by the modal parameters in Table A43.2 (which include the effects of the isolators). For comparison, first-mode parameters for 2nd Floor-Roof (2F-R) response in the NR event were 1.03 Hz and 4.4% damping (tr) and 1.26 Hz and 3.5% damping (L). These frequencies are similar to measured frequencies in the building prior to retrofit (Table A43.3).
- The structure had a stiffer response in NR than LD, despite the higher amplitude of shaking in NR. This likely resulted from the longer duration and larger low-frequency energy content of the LD data relative to the NR data.
- Pseudo flexible-base parameters were computed using 1F-R pairs so that the difference from B-R parameters could be quantified. The deviation was about 0.01-0.015 Hz for first-mode frequency.

The RPEM analyses for LD indicate that the first-mode frequency for both directions decreased during the build-up to the maximum amplitude shaking at 50 sec., and subsequently recovered as the shaking subsided. These variations in first-mode frequency are believed to result primarily from stiffness degradation in the base isolation system during strong shaking. Similarly, damping values appeared to decrease somewhat as the shaking subsided. Similar results were obtained from the NR data.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A43.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 103 feet using the recorded free-field motions and the profile in Fig. A43.3. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping values for the longitudinal direction were increased by 40% to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A43.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=975$ fps	$h=83$ ft	emb.=16 ft	$\gamma = 0.15$
	$\nu=0.45$		tr: $r_1 = 103$ ft	$r_2 = 90$ ft	
LD	$\beta^*=2.6\%$	$V_s^*=935$ fps	L: $r_1 = 103$ ft	$r_2 = 121$ ft	
NR	$\beta^*=3.3\%$	$V_s^*=910$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A43.4 with the fixed-base modal parameters in Table A43.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A43.5.

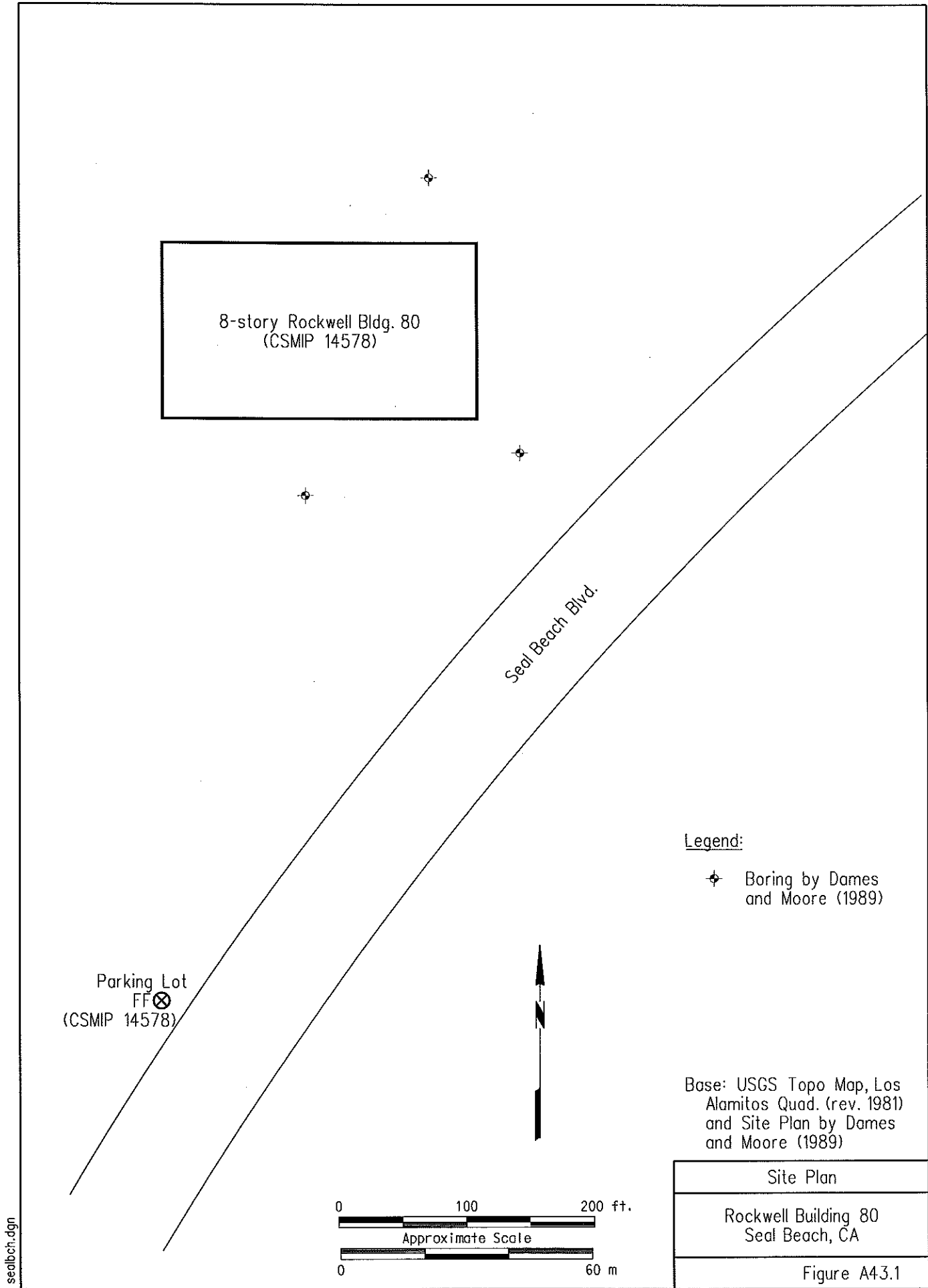
Table A43.5: Empirical and predicted inertial interaction effects

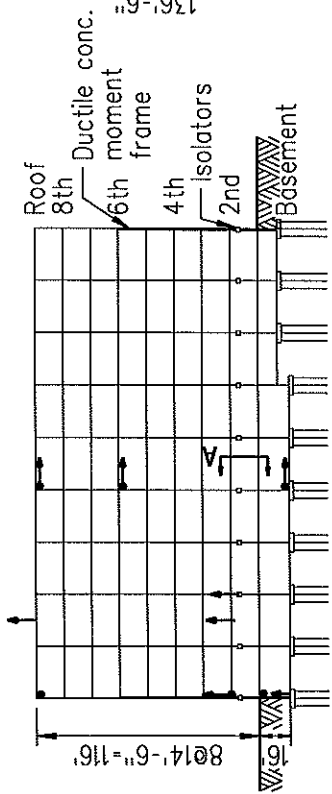
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LD	Sys. ID	1.01	-0.1	10.6	1.03	0.1	12.9
	Pre.-Vel.	1.02	0.3	10.7	1.02	0.4	13.8
	Pre.-Bie.	1.02	0.5	10.9	1.02	0.8	14.0
NR	Sys. ID	1.01	-0.3	7.1	1.01	0.3	8.3
	Pre.-Vel.	1.02	0.4	7.6	1.02	0.6	8.3
	Pre.-Bie.	1.02	0.7	8.0	1.03	1.0	8.7

The period lengthening and foundation damping were well predicted by the both formulations, though foundation damping values by the “modified Bielak” procedure were superior. The negative foundation damping values from the system identification analyses indicate numerical errors in these results.

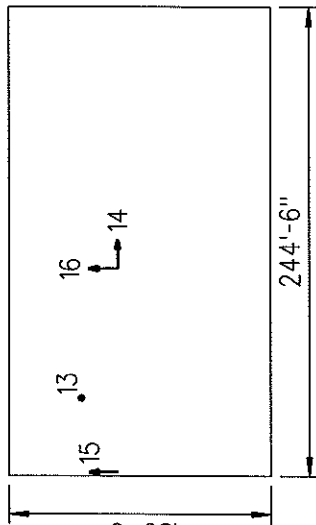
Data Sources

Dames and Moore (1988). “Report, Geotechnical Studies, Building 80 Base Isolation, for Rockwell International, Seal Beach, CA,” *Job No. 10839-030-15*.

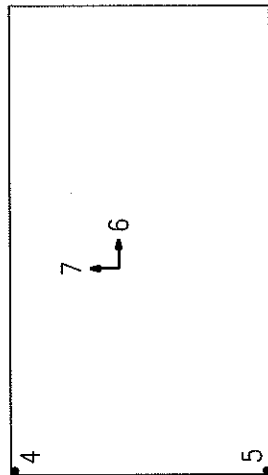




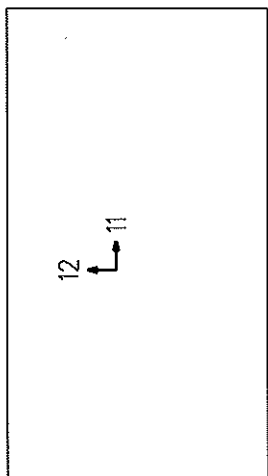
W / E Elevation



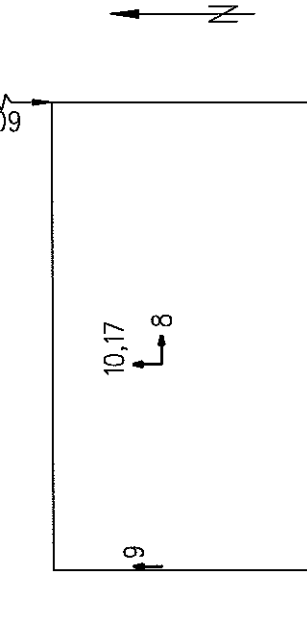
Roof Level Plan



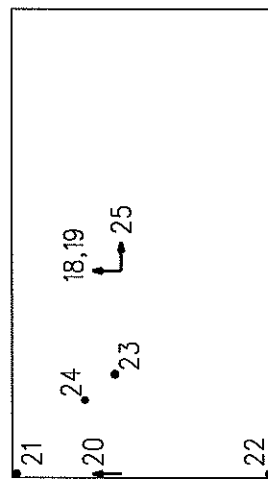
Basement Plan



6th Floor Plan

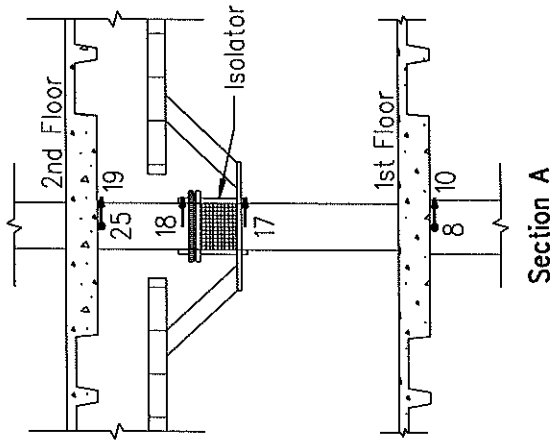


1st Floor Plan

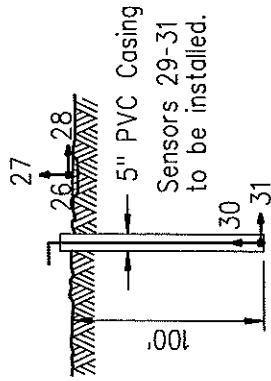


2nd Floor Plan

Concrete moment frames retrofitted with exterior ductile concrete moment frames and base isolation.
Construction date: 1967 and 1990



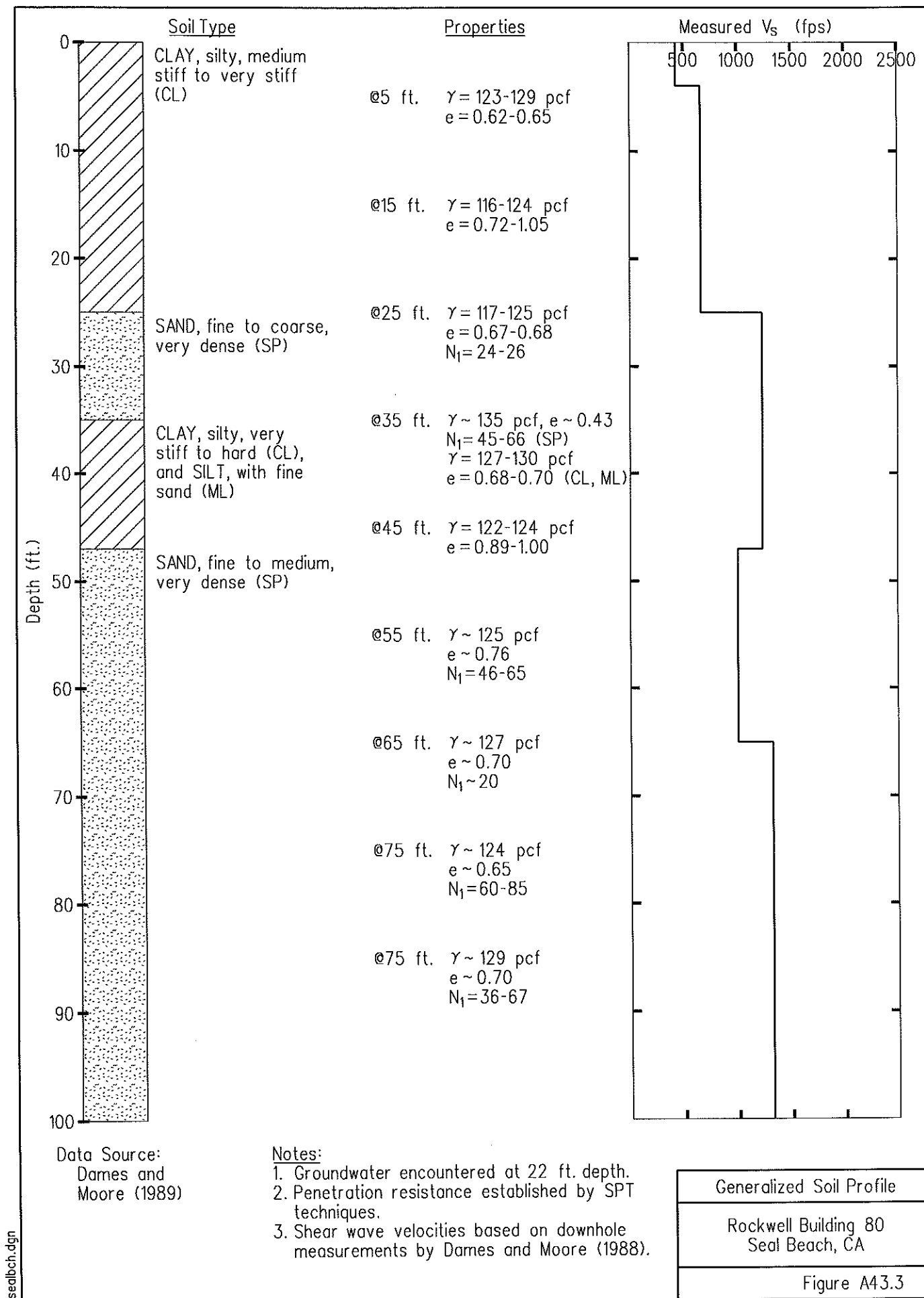
Section A



Downhole & Surface

Note: Data provided by CSMIP

Sensor Locations
Rockwell Building 80 Seal Beach, CA
Figure A43.2



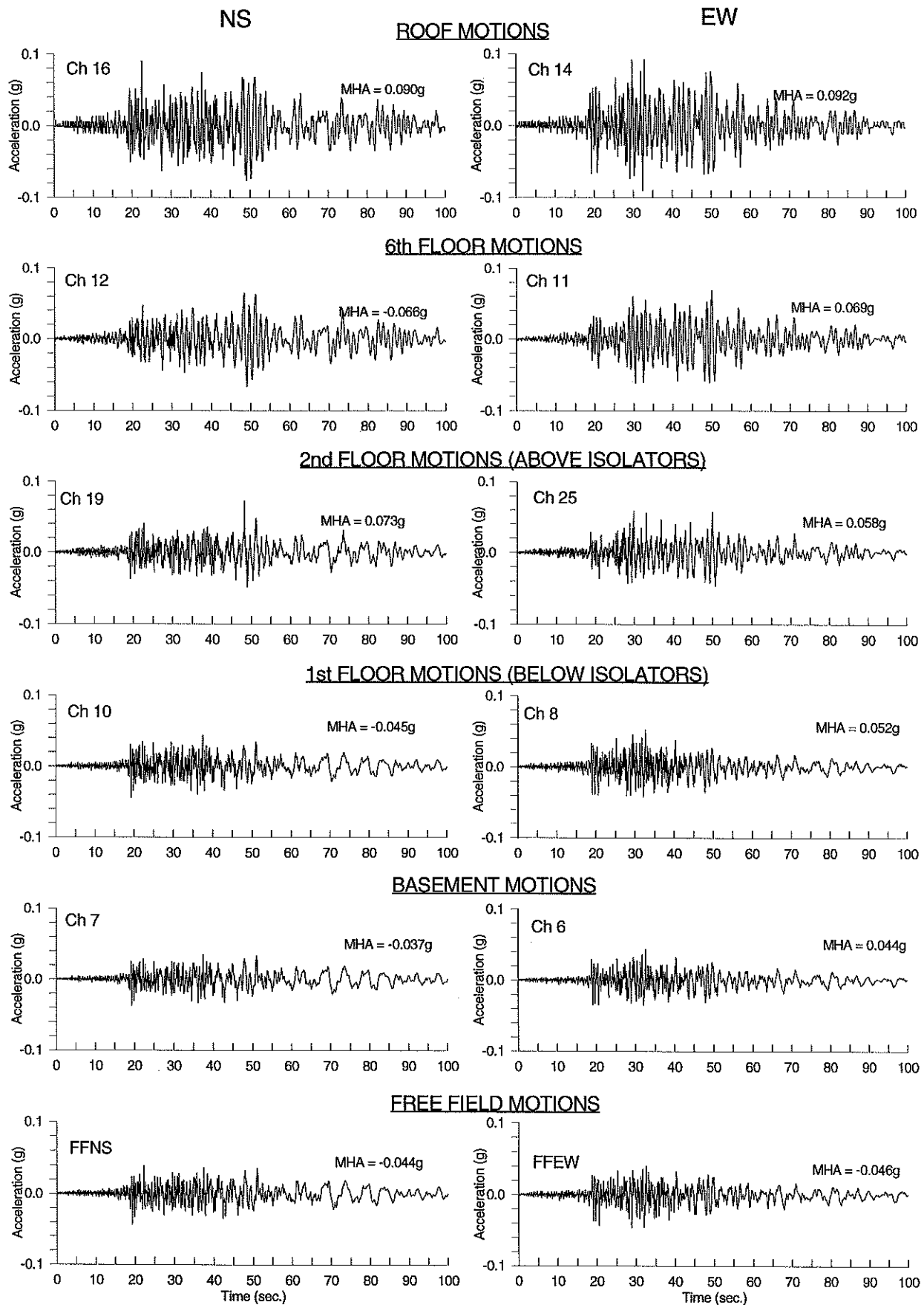


Fig. A43.LD1: SEAL BEACH ROCKWELL BUILDING 80

Horizontal Acceleration Time Histories

1992 Landers Earthquake

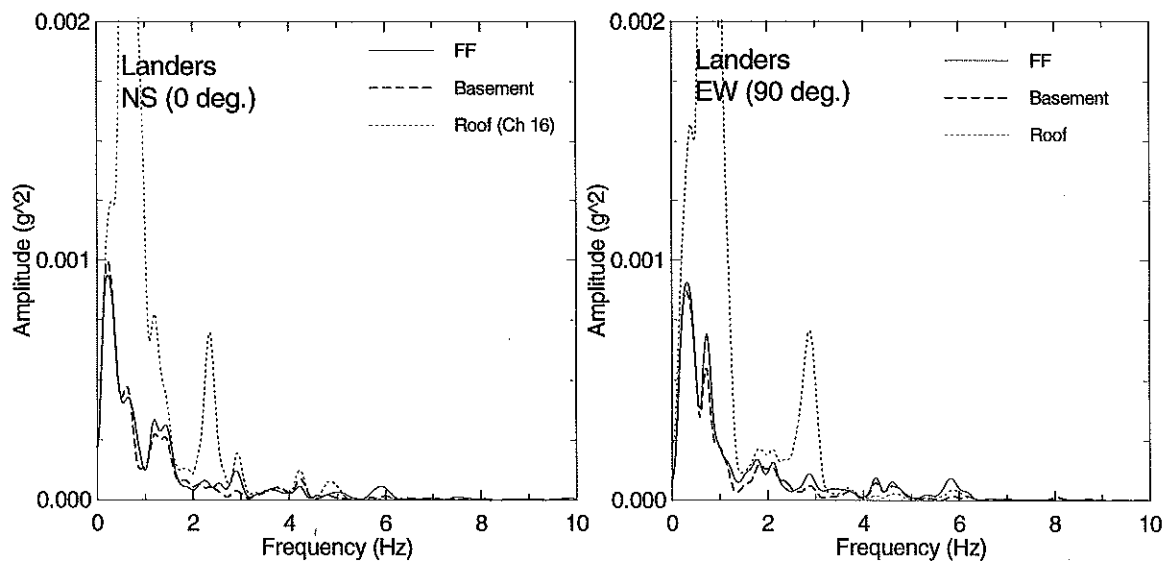


Fig. A43.LD2: SEAL BEACH ROCKWELL BUILDING 80
Power Spectral Density of Selected Horizontal Accelerations
1992 Landers Earthquake

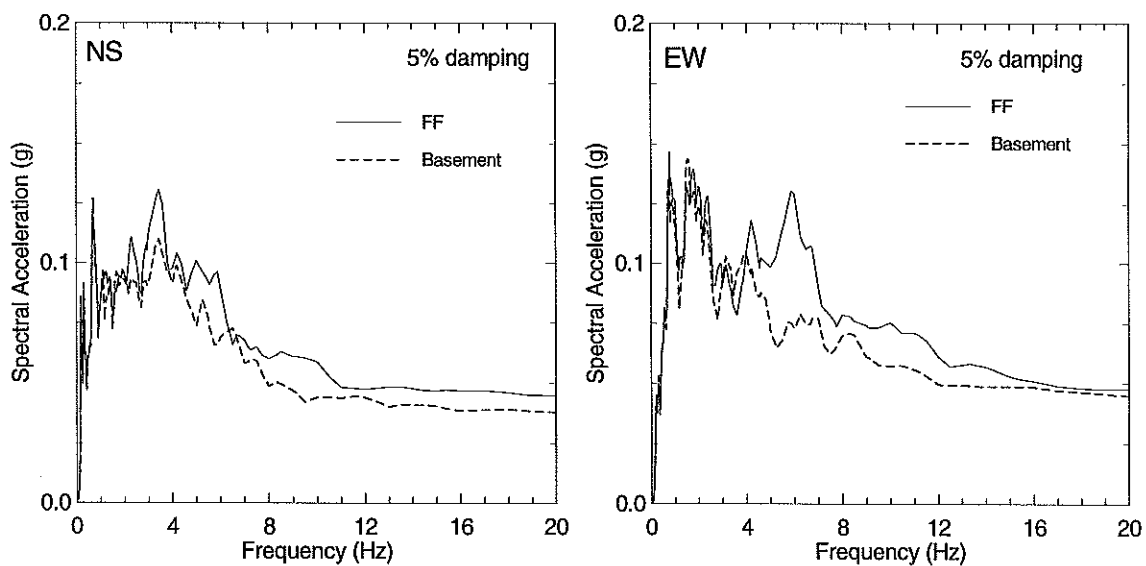


Fig. A43.LD3: SEAL BEACH ROCKWELL BUILDING 80
Acceleration Response Spectra of Basement and Free-Field Motions
1992 Landers Earthquake

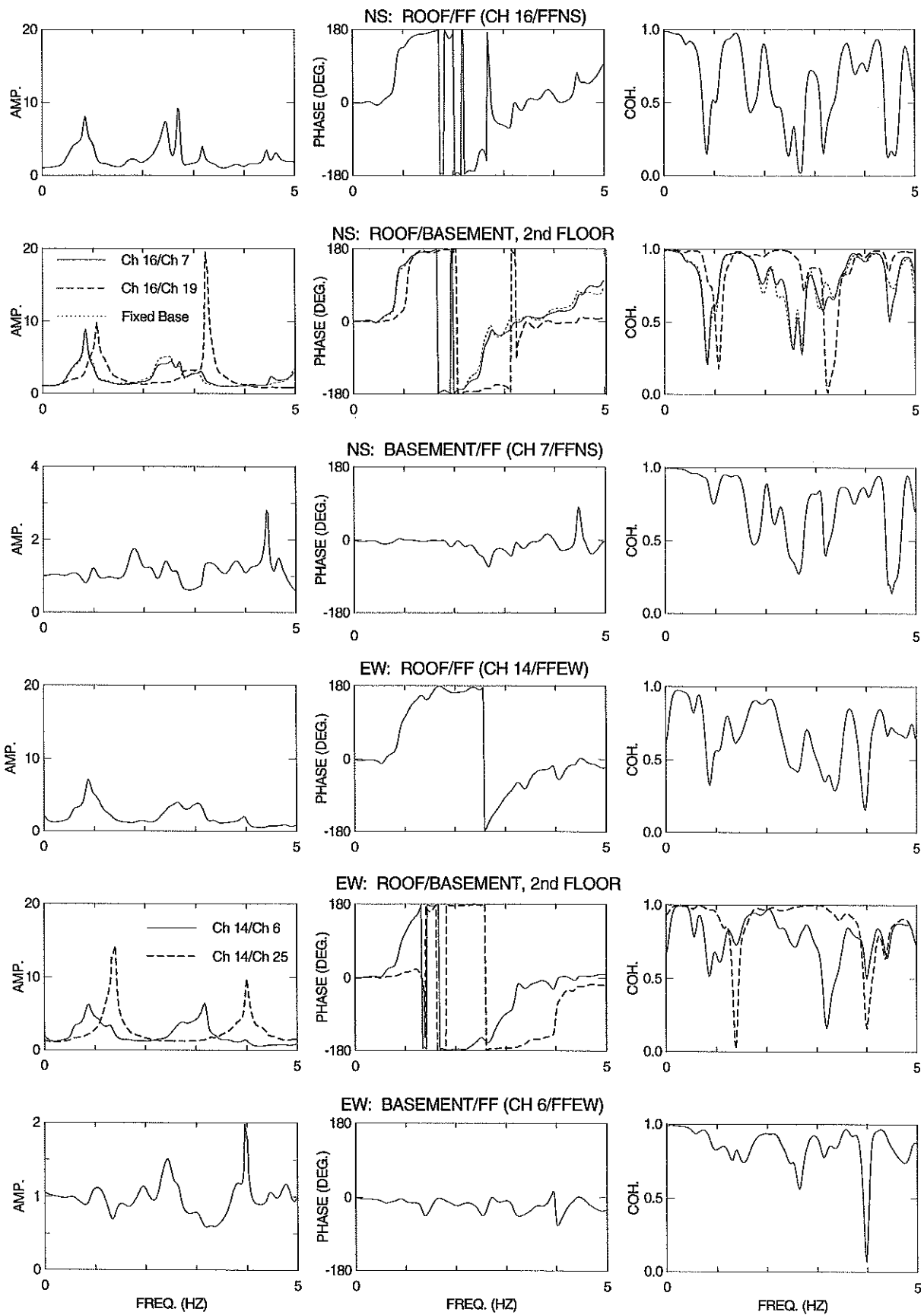


Fig. A43.LD4: SEAL BEACH ROCKWELL BUILDING 80
Transmissibility and Coherency Functions
1992 Landers Earthquake
A43-11

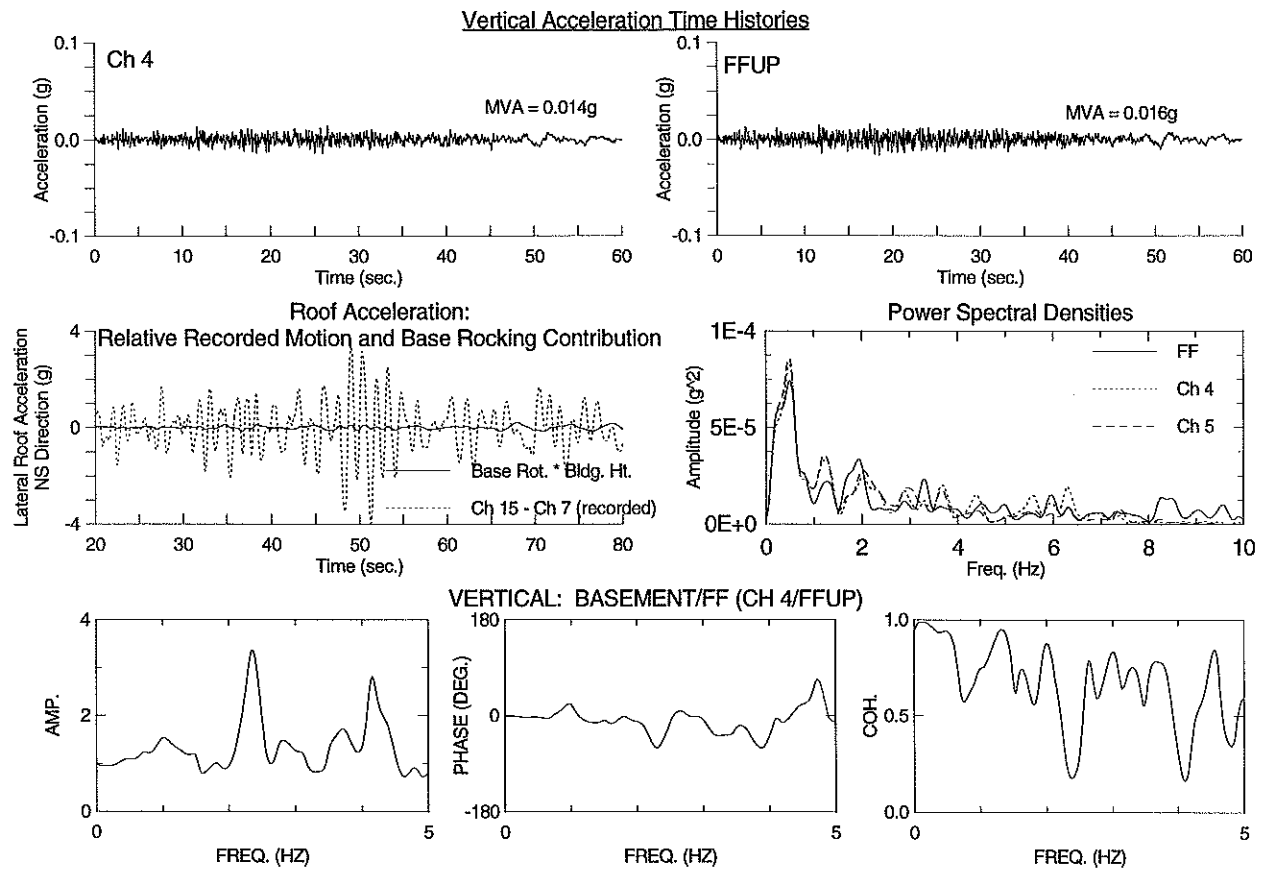


Fig. A43.LD5: SEAL BEACH ROCKWELL BUILDING 80
Analysis of Vertical Motions and Rocking Effects
1992 Landers Earthquake

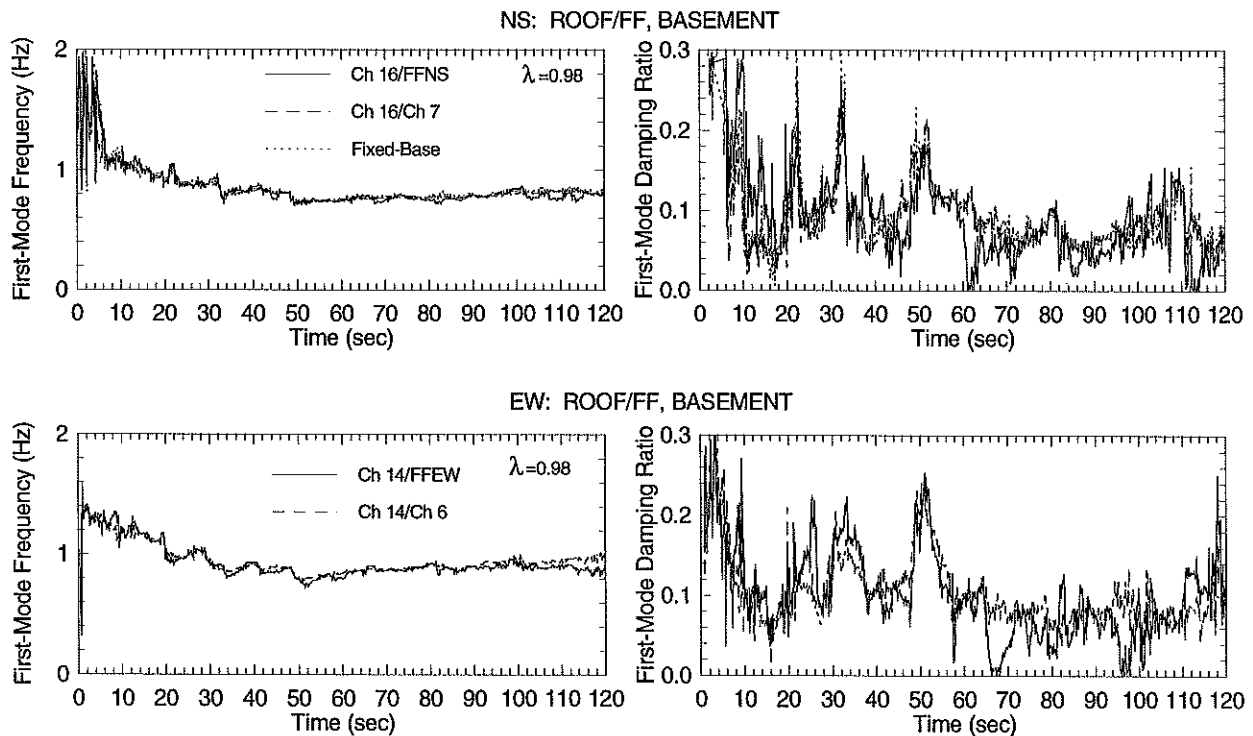


Fig. A43.LD6: SEAL BEACH ROCKWELL BUILDING 80
Time Variation of First-Mode Parameters
1992 Landers Earthquake
A43-12

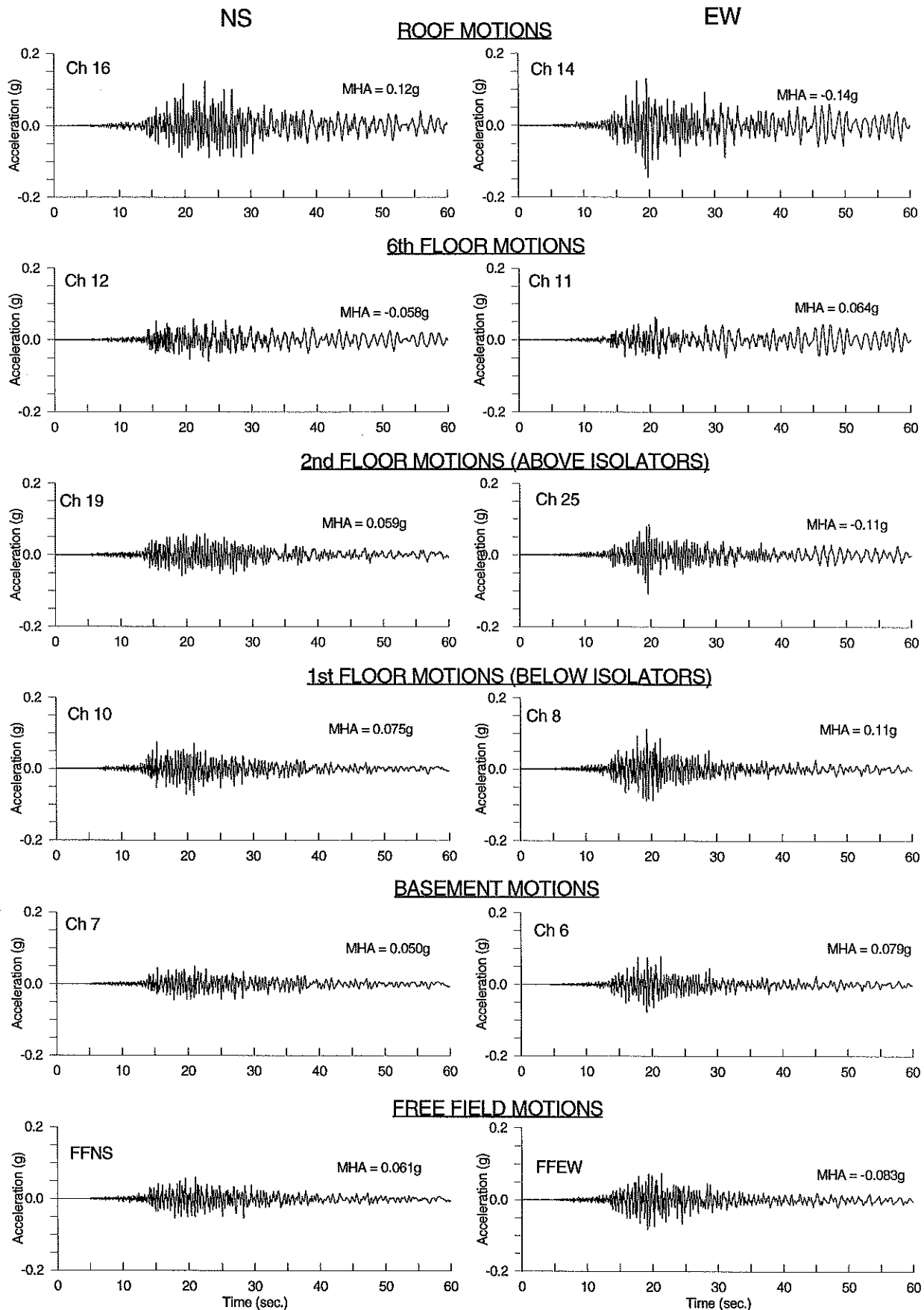


Fig. A43.NR1: SEAL BEACH ROCKWELL BUILDING 80
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

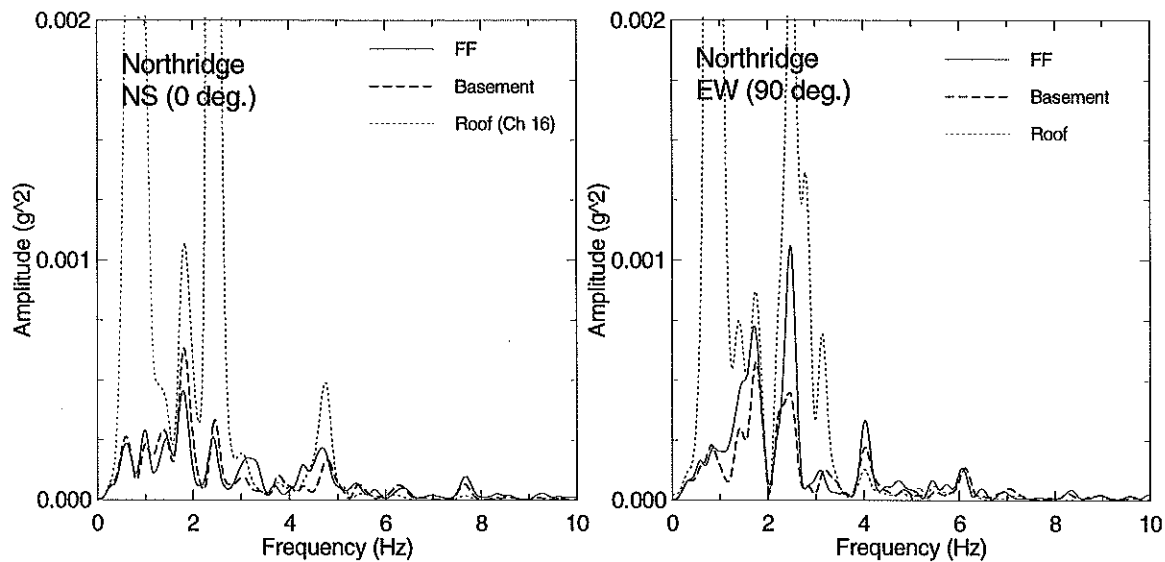


Fig. A43.NR2: SEAL BEACH ROCKWELL BUILDING 80
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

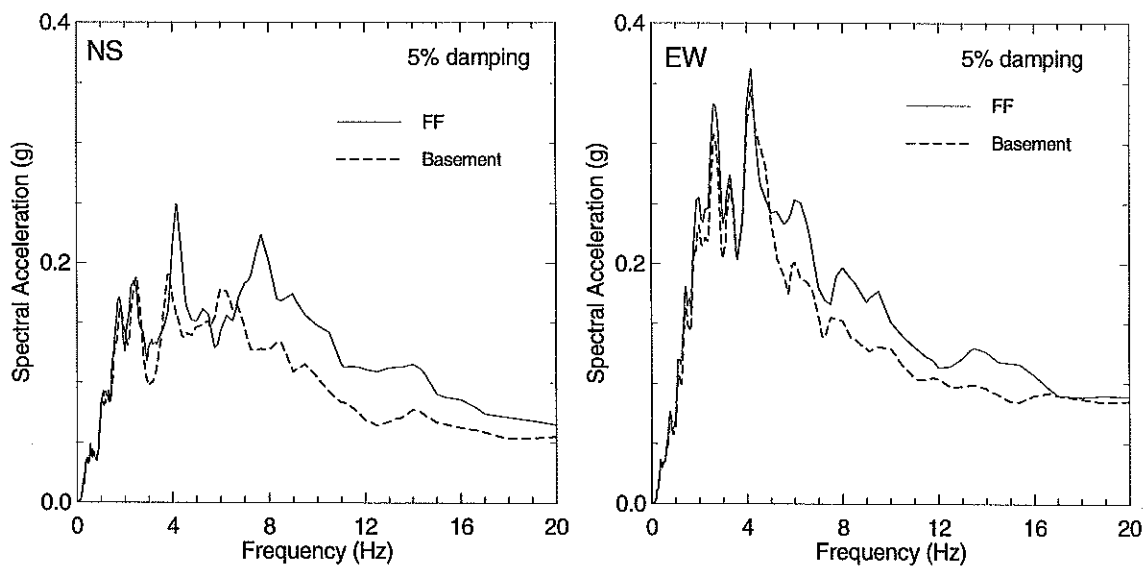


Fig. A43.NR3: SEAL BEACH ROCKWELL BUILDING 80
Acceleration Response Spectra of Basement and Free-Field Motions
1994 Northridge Earthquake

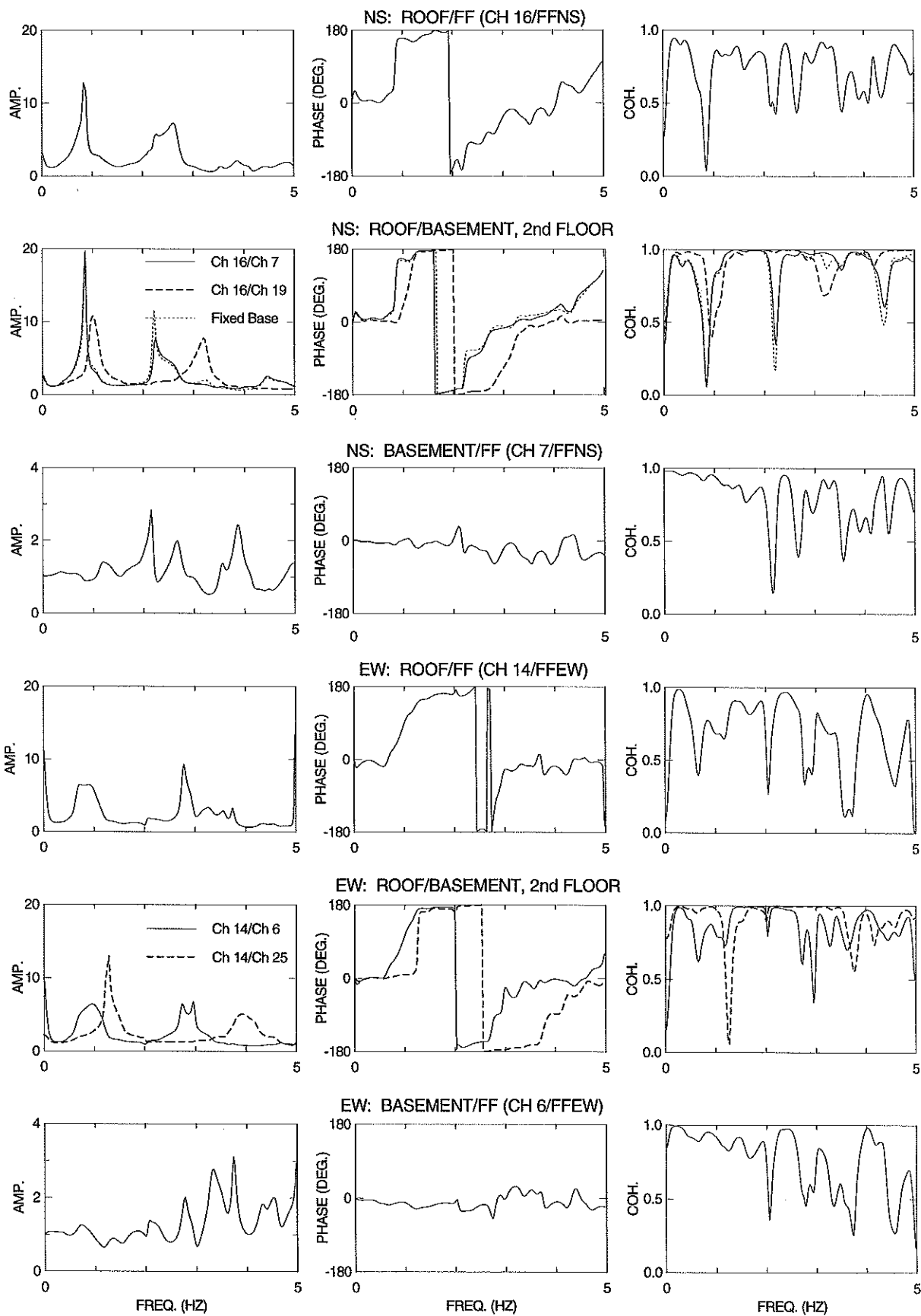


Fig. A43.NR4: SEAL BEACH ROCKWELL BUILDING 80
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A43-15

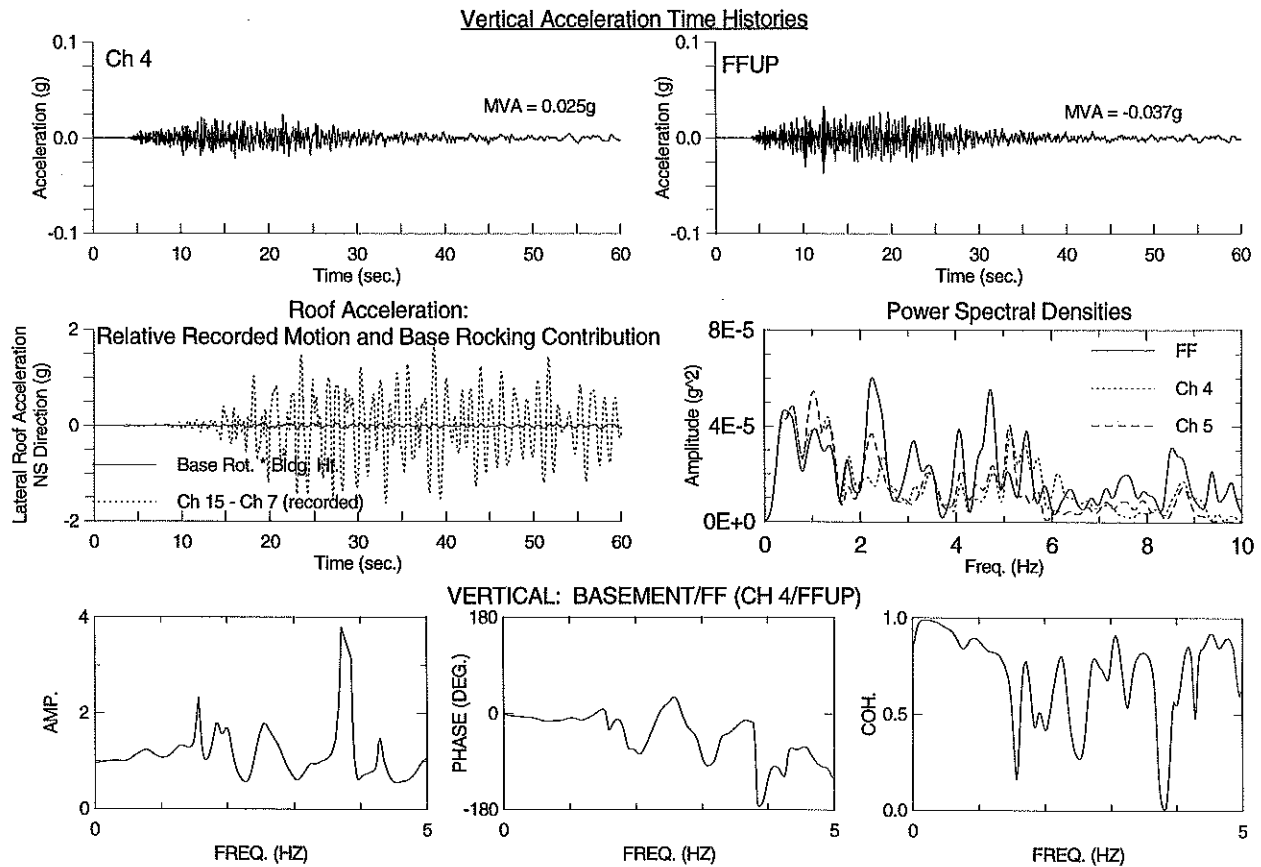


Fig. A43.NR5: SEAL BEACH ROCKWELL BUILDING 80
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

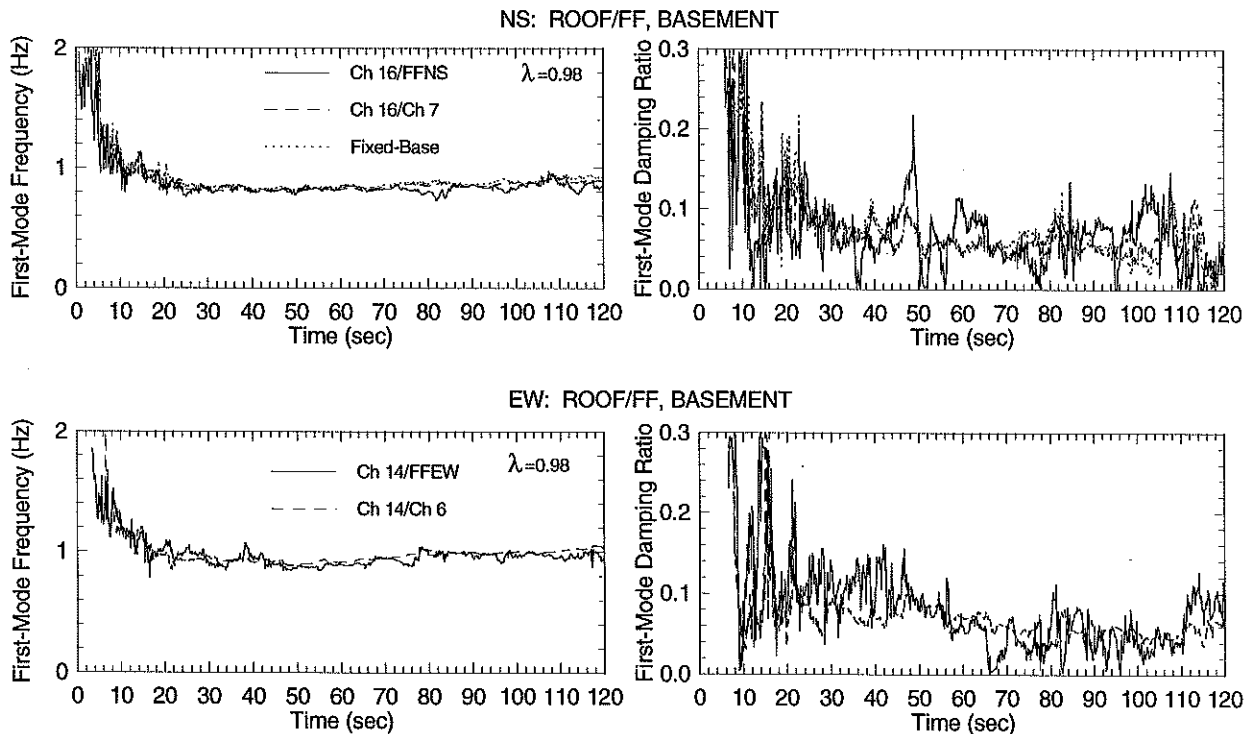


Fig. A43.NR6: SEAL BEACH ROCKWELL BUILDING 80
Time Variation of First-Mode Parameters
1994 Northridge Earthquake
A43-16

SITE A44

Site Name: Sylmar 6 Story County Hospital
Sensors: CSMIP 24514

Lat: 34.326°
Long.: 118.444°

Floor Levels: 6
Bsmt. Levels: 0
Structure: Concrete and steel shear walls

Foundation: footings
Site Geology: soil
Design Date: 1976

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1987 Whittier	0.058	0.052	0.041	0.051	0.057	0.046	0.20	0.15	--
1992 Landers*	0.04	0.06	0.01	0.06	0.03	0.02	0.08	0.05	--
1994 Northridge	0.84	0.60	0.54	0.80	0.38	0.33	1.50	0.75	--

* not digitized

-- = no instrument

Preprocessing

Whittier Earthquake (WT)

Preliminary: CSMIP; filtered, $f = 0.25\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$; FF/str. data not synchronized; FF not aligned with str.

Final: FF data rotated 4° CCW to align with str.; FF/str. data synchronized using cross-correlation of vertical accelerations.

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}23\text{ Hz}$ (str.), $0.12\text{--}23\text{ Hz}$ (FF); $f_{Nyq} = 25\text{ Hz}$; FF/str. data synchronized; FF not aligned with str.

Final: FF data rotated 4° CCW to align with str.

Figures

A44.1-A44.3

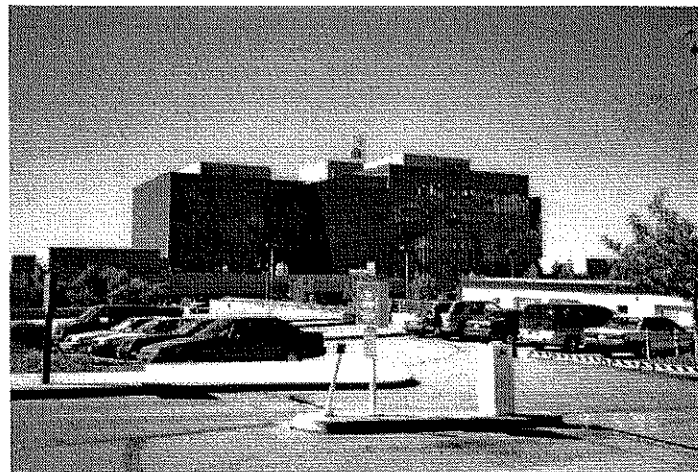
Site Plan, Sensor Locations, Generalized Soil Column

A44.WT1-6

WT Analysis Results

A44.NR1-6

NR Analysis Results



View of structure from FF location

DISCUSSION

Site Description

The building is 6-stories tall above the ground surface with no basement, and has a rectangular shape (302 by 452 feet in plan) from the base to 3rd level and a cruciform shape (101-foot wide wings) from the 3rd to roof levels. No significant structures are located near the building. As shown in Fig. A44.2, the lateral load resisting system consists of concrete shear walls from the ground to 3rd level, and steel shear walls from the 3rd to roof levels. The foundations consist of spread footings. Accelerographs are present on the ground, 3rd, 4th, and roof levels of the structure. The free-field instrument is located in an enclosed area adjacent to a parking lot, about 455 feet north of the building.

The site is located near the northern rim of the San Fernando Valley on terrain having a gradual slope down to the south. A number of subsurface investigations have been conducted at the site including a 100-foot boring by Fumal et al. (1982), seismic refraction surveys by Duke et al. (1971), and a 300-foot boring by Gibbs et al. (1996). As shown in Fig. A44.3, the soil profile consists of about 28 feet of Holocene sands and gravels, 14 feet of Pleistocene sand and gravel, 218 feet of Pleistocene silty clays with sand interbeds, and Saugus formation sandstone bedrock which extends to the maximum depth explored of 300 feet. Shear wave velocities at the site have been measured by SASW techniques (Stokoe, 1995), downhole measurements (Fumal et al., 1982 and Gibbs et al., 1996), and seismic refraction (Duke et al., 1971). Profiles of V_s from the SASW data and downhole measurements by Gibbs et al. are shown in Fig. A44.3. The downhole data was used for the analyses in this study.

Instrument Selection and Free-Field Conditions

The structure is instrumented in its west, east, and south wings. Given the instrument layout in the structure, EW response could only be assessed in the south wing, while NS response could be evaluated in the west or east wings. Recordings from the west wing were used for WT analyses and recordings from the east wing were used for NR analyses. For each earthquake, comparative analyses were also carried out using data for the other wing, and the results were found to be similar. The specific channels used in the primary analyses are indicated in Table A44.1; acceleration time histories recorded at these and other sensors are shown in Fig. A44.**1.

Table A44.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 8, 9	Ch 13	Ch 1	Ch 2, 3	Ch 10

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions

GF/FF transmissibility functions (Fig. A44.**4) and the spectra for horizontal motions (Fig. A44.**2-3) generally indicate increasing de-amplification of GF motions with frequency.

Transmissibility, coherency, and power spectral density functions for FF and GF vertical motions are presented in Fig. A44.**5. Pronounced amplification near the first-mode frequencies for lateral response is evident, indicating that rocking likely contributed to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A44.**4), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table A44.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A44.**6).

Table A44.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (NS)		L (EW)		tr (NS)		L (EW)		tr (NS)		L (EW)	
		WT: estimated	NR: estimated	estimated	estimated	d=0, j=3	d=1, j=3	d=0, j=3	d=0, j=3	d=0, j=3	d=0, j=3	d=0, j=3	d=0, j=3
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
WT	1	3.76	9.2	4.02	9.7	3.65	9.0	3.83	9.2	3.40	9.9	3.47	9.5*
	2					7.50	10.0	7.49	7.0	7.18	15.3	7.34	9.0*
NR	1	2.75	15.9	3.78	17.5	2.71	16.2	3.45	18.7	2.62	15.3 18.9	2.94	23.4*
	2					7.37	10.1	7.08	16.4	7.26	12.2	6.64	18.8*

* low confidence in these damping values (see below)

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode. During the NR event, a resonant condition between the dominant frequency of the input motion and structure occurred which led to very high amplitude shaking at the roof level (MHA = 1.5g).
- The moderate differences between the fixed- and pseudo flexible-base parameters are somewhat inconsistent with the significant rocking evident in the vertical GF/FF transmissibility function (i.e larger deviations are expected). The apparent rocking effect evident from the GF vertical data may be associated with deformations within an individual wing; such localized deformations were not accounted for in the calculation of the estimated first-mode parameters which used the entire footprint area of the cruciform-shaped structure. However, at present there is insufficient data to prove that localized base rocking occurred,

and such a determination could only be made with additional GF vertical sensors in the wings and core of the structure.

- As discussed below, the NS CEM $\tilde{\zeta}$ value of 15.3% from the NR data was adjusted based on RPEM results
- The R/FF transmissibility function amplitudes for both the NS and EW directions are quite spiky for both earthquakes. Of particular interest is several peaks at frequencies less than the first-mode response frequency. These peaks do not appear to be related to a structural response mode, as they are absent from the R/GF transmissibility functions. Rather, these low frequency peaks are associated with incoherent GF and FF motions (note the low coherencies in Figs. A44.**.4).
- R/GF and R/FF transmissibility functions were computed using recordings from both wings of the building in the NS direction, and were found to be similar.
- For FF-R response in the EW direction, the CEM models did not accurately reproduce the amplitude and width of the nonparametric transmissibility function peaks. Hence, the damping values from these analyses are approximate.
- The structural response during the high amplitude NR shaking was less stiff and more highly damped than the WT response, which was associated with relatively low amplitude shaking.
- The CEM results for GF-R response are consistent with Celebi (1996), who used similar identification techniques in a study of the building's fixed-base seismic response in the WT and NR events.

The RPEM results (Figs. A44.**6) indicate no significant time variation in first-mode parameters of the structure during WT (the variations of modal parameters in the first 6 seconds are likely unreliable due to the low amplitude shaking at these times). During NR, however, significant reductions in first-mode frequency occurred at 4 to 6 seconds (the time of the highest amplitude shaking). The relative magnitude of CEM parameters for the FF-R and GF-R cases was generally confirmed by the RPEM analyses, with the exception of damping in the NS direction from the NR event. During the most stable portions of the identification ($t > 16$ sec), $\tilde{\zeta}$ exceeds $\tilde{\zeta}^*$ by an absolute difference of about 2.7% (i.e. as indicated in Table A44.2, a better estimate is $\tilde{\zeta} \approx 18.9$ Hz).

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A44.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 126 feet using the recorded free-field motions and the profile in Fig. A44.3 (V_s values by Gibbs et al., 1996 were used). The cross-shaped hospital tower was assumed to be independent of the 2-story rectangular shaped portion of the building for the calculation of foundation radii. No adjustments to the impedance function were made for embedment, foundation non-rigidity or shape effects.

Table A44.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1530$ fps	$h = 63$ ft	emb.=0	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 126$ ft		$r_2 = 132$ ft
WT	$\beta^* = 1.9\%$	$V_s^* = 1505$ fps	L: $r_1 = 126$ ft		$r_2 = 132$ ft
NR	$\beta^* = 3.0\%$	$V_s^* = 1120$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A44.3 with the fixed-base modal parameters in Table A44.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A44.4.

Table A44.4: Empirical and predicted inertial interaction effects

		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
WT	Sys. ID	1.11	3.1	9.9	1.16	3.2	9.5
	Prediction	1.12	6.5	13.0	1.14	7.5	14.1
NR	Sys. ID	1.05	5.1	18.9	1.29	15.2	23.4
	Prediction	1.12	6.8	18.1	1.23	12.2	21.5

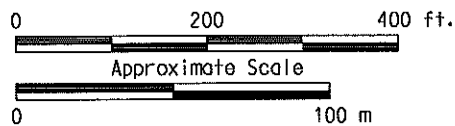
The period lengthening and foundation damping are reasonably well predicted for this structure.

Data Sources

- Celebi, M. (1996). "Seismic performance of the New Olive View Hospital during the Northridge and Whittier Earthquakes," submitted to *J. Structural Engrg.*, ASCE (review pending).
- Duke, C.M., Johnson, J.A., Kharraz, Y., Campbell, K.W., and Malpiede, N.A. (1971). "Subsurface site conditions and geology in the San Fernando Earthquake Area," *Rpt. No. UCLA-ENG-7206*, School of Engrg. and Applied Sciences, Univ. of Calif., Los Angeles.
- Gibbs, J.F., Tinsley, J.C. and Joyner, W.B. (1996). "Seismic velocities and geologic conditions at 12 sites subjected to strong ground motion in the 1994 Northridge, California Earthquake," *Open File Rpt. 96-740*, U.S. Geological Survey, 103 p.
- Fumal, T.E., Gibbs, J.F., and Roth, E.F. (1982). "In-situ measurements of seismic velocity at 22 locations in the Los Angeles, California Region," *Open File Rpt. 82-833*, U.S. Geological Survey.
- Stokoe, K. (1995). Unpublished shear wave velocity profiles determined by SASW testing for sites in the Los Angeles Basin.

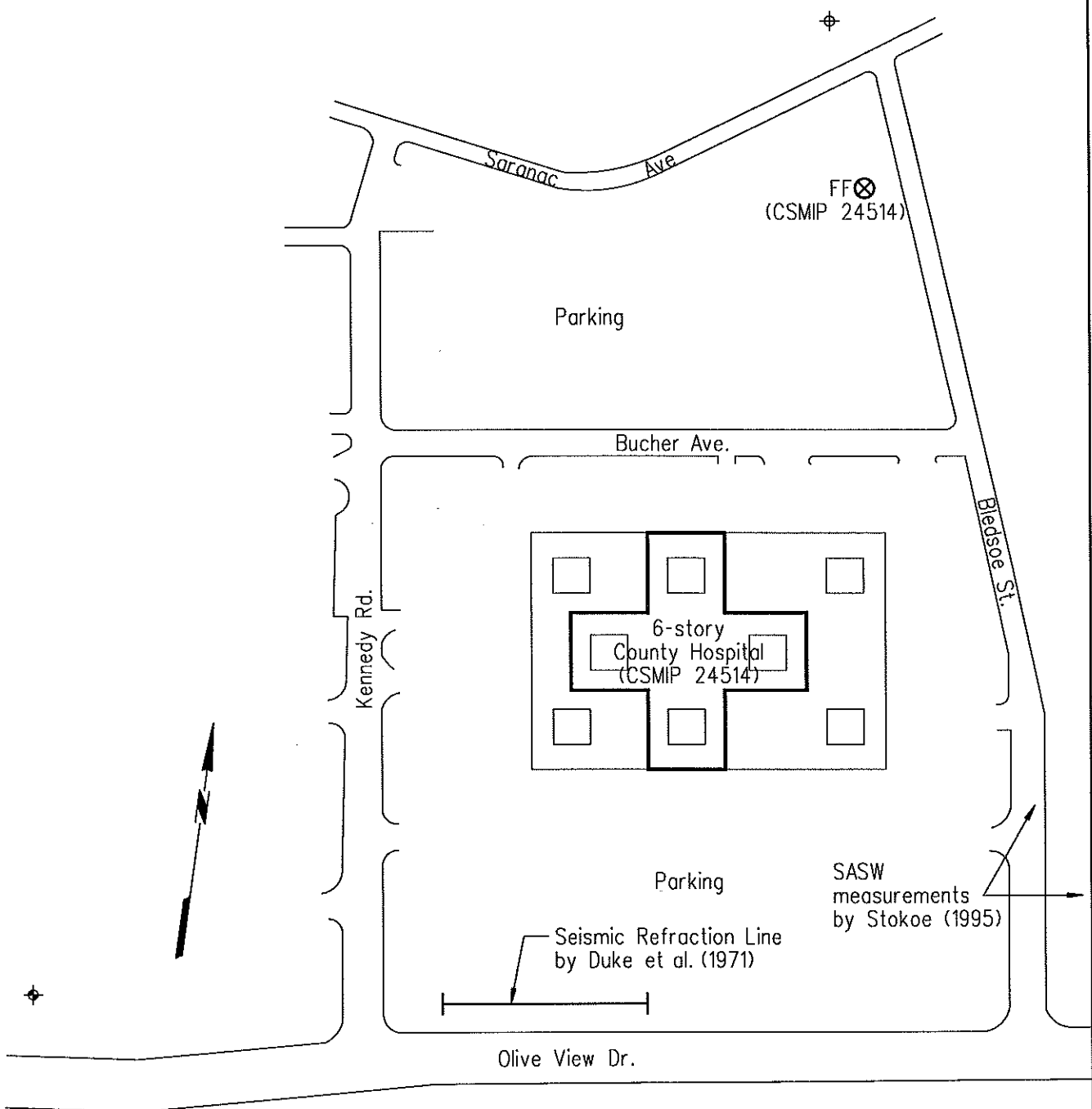
Legend:

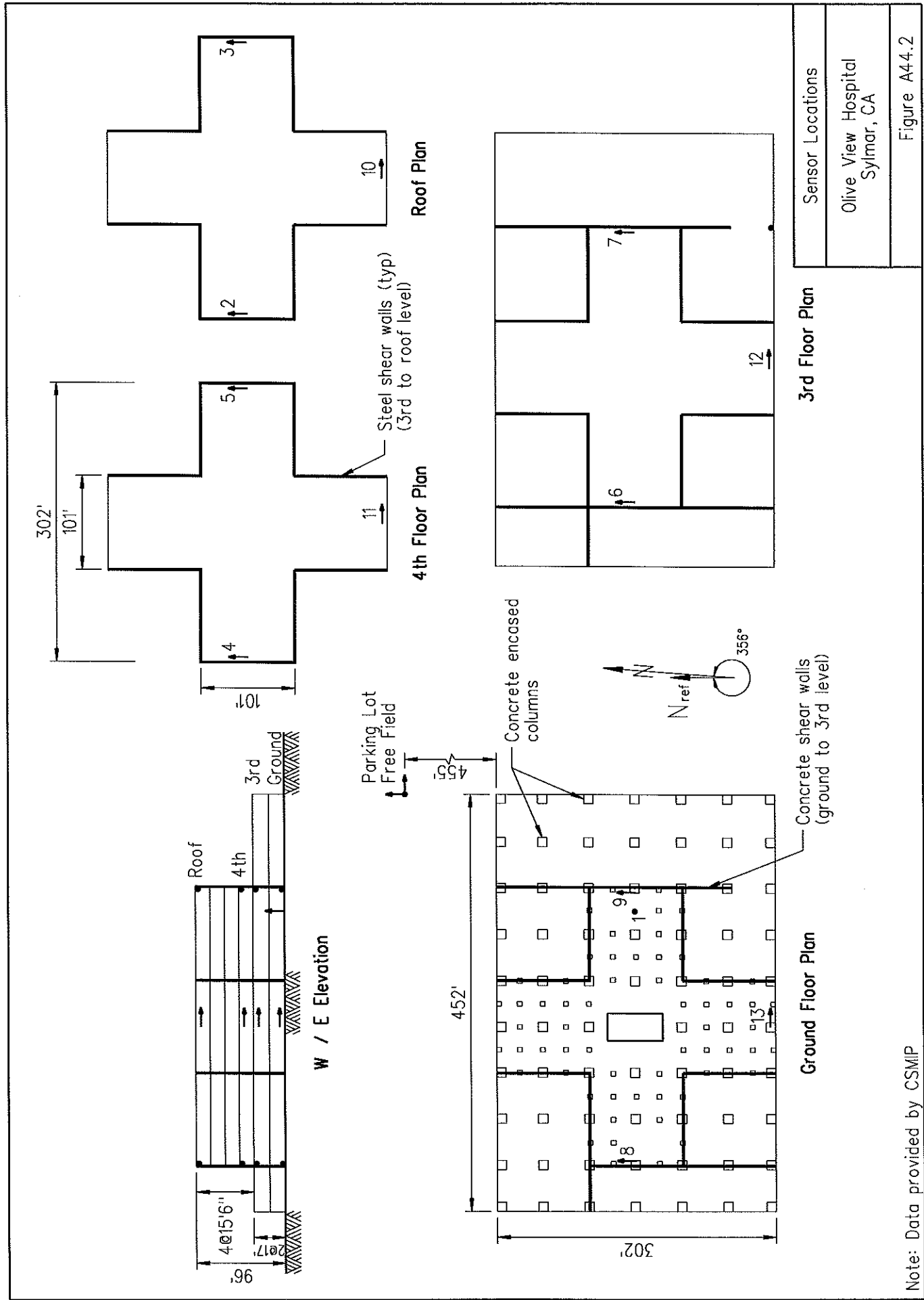
- ⊕ Boring by Fumal et al. (1982)
- ⊕ Boring by Gibbs et al. (1996)



Base: County Hospital Site Plan,
provided by Hospital Staff

Site Plan
Olive View Hospital Sylmar, CA
Figure A44.1

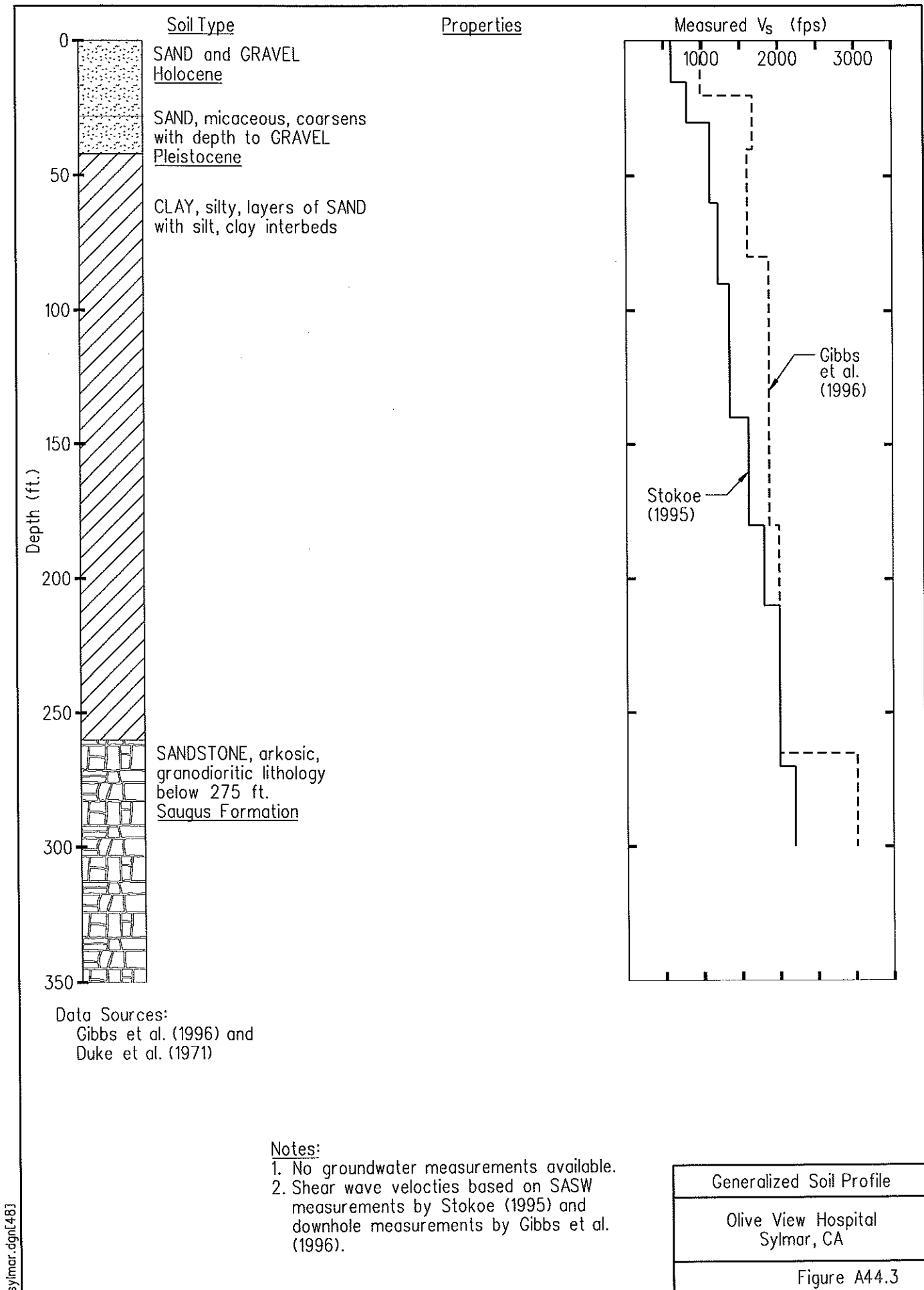




Sensor Locations
Olive View Hospital Sylmar, CA
Figure A44.2

Note: Data provided by CSMIP

sylmar.dgn



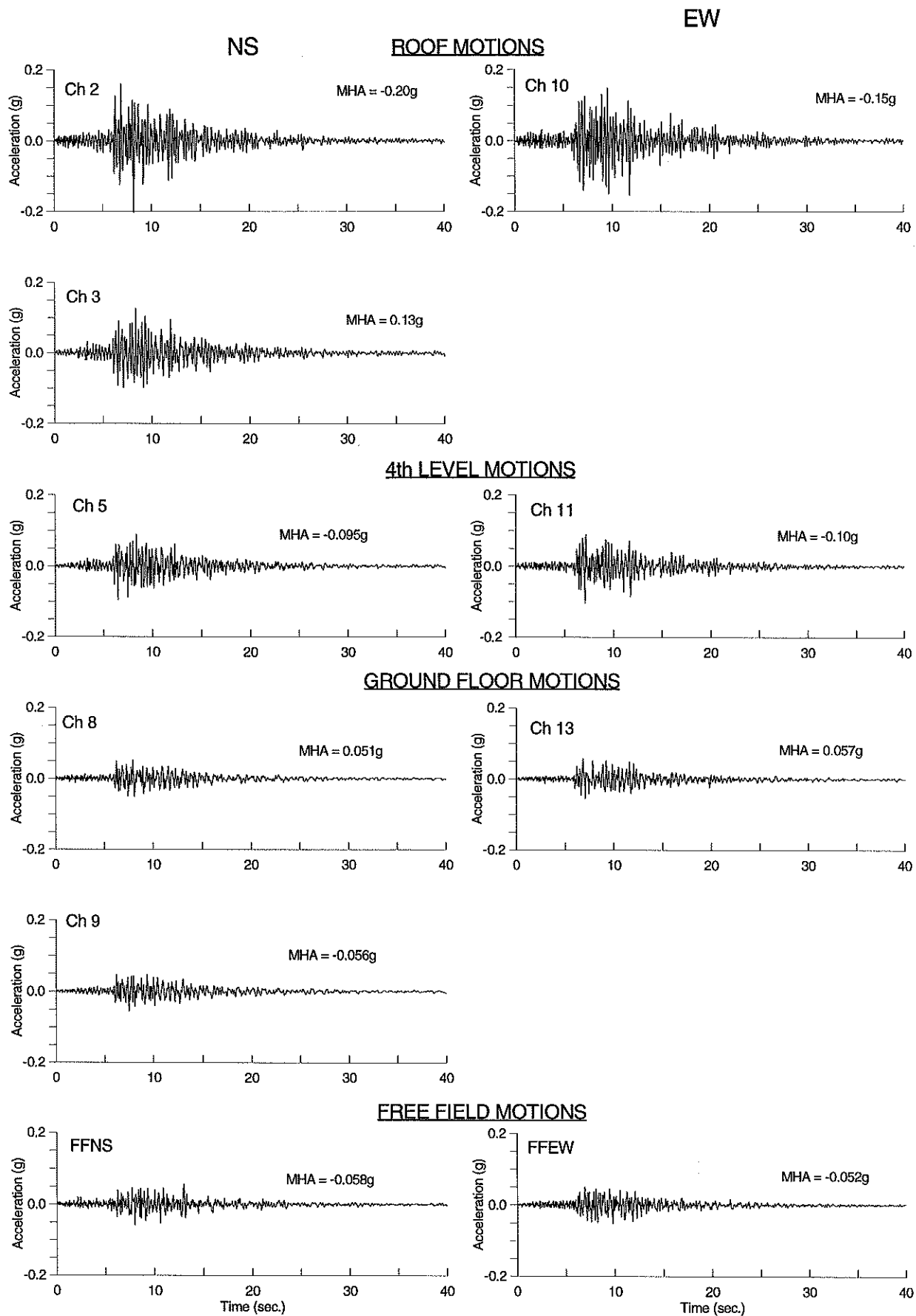


Fig. A44.WT1: SYLMAR 6 STORY COUNTY HOSPITAL
Horizontal Acceleration Time Histories
1987 Whittier Earthquake
A44-9

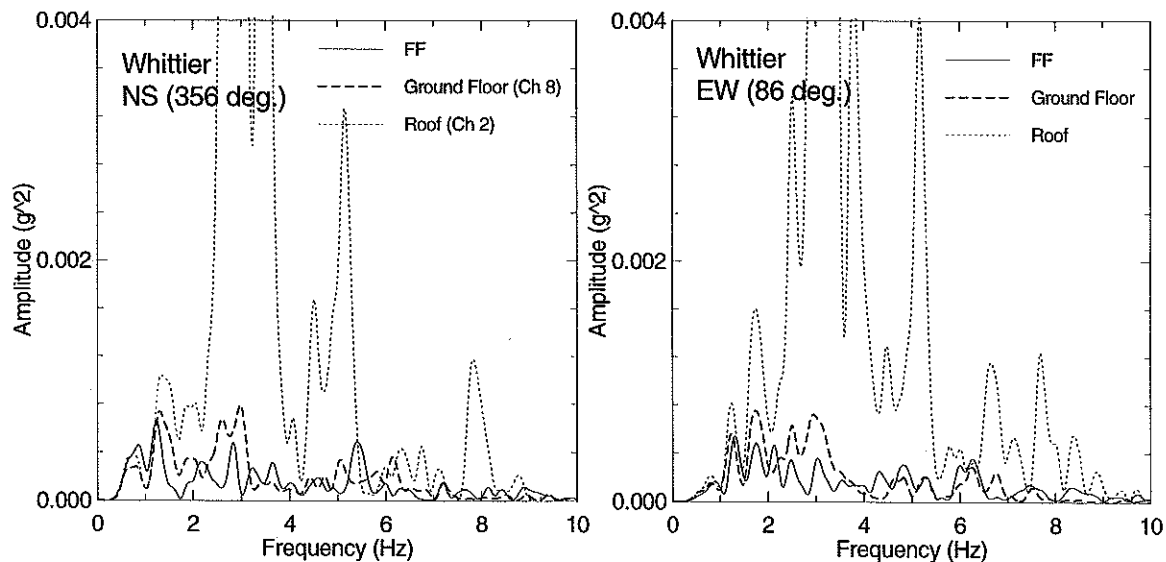


Fig. A44.WT2: SYLMAR 6 STORY COUNTY HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1987 Whittier Earthquake

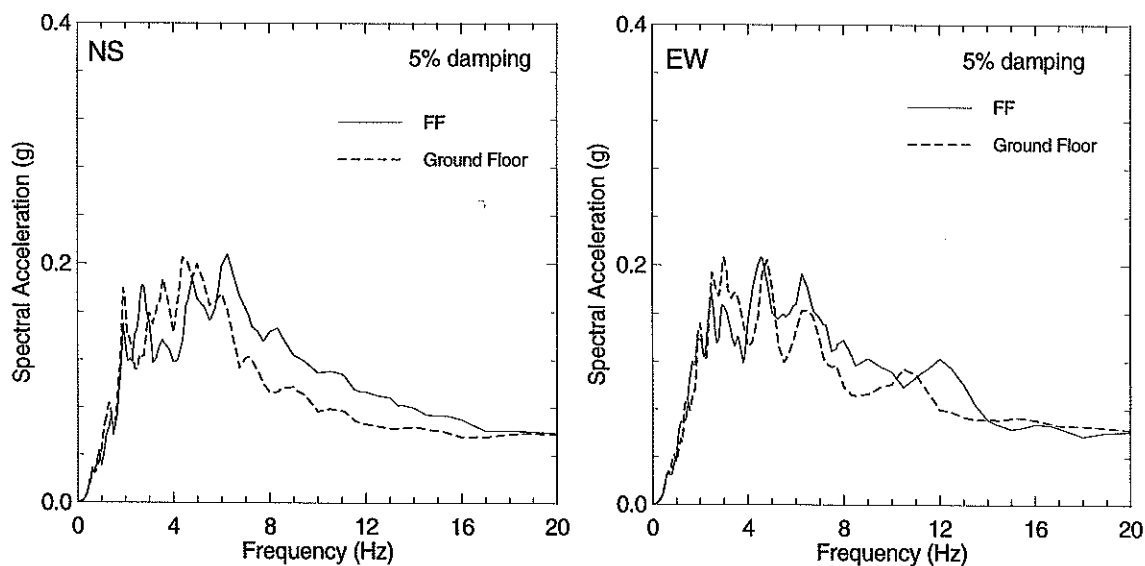


Fig. A44.WT3: SYLMAR 6 STORY COUNTY HOSPITAL
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1987 Whittier Earthquake

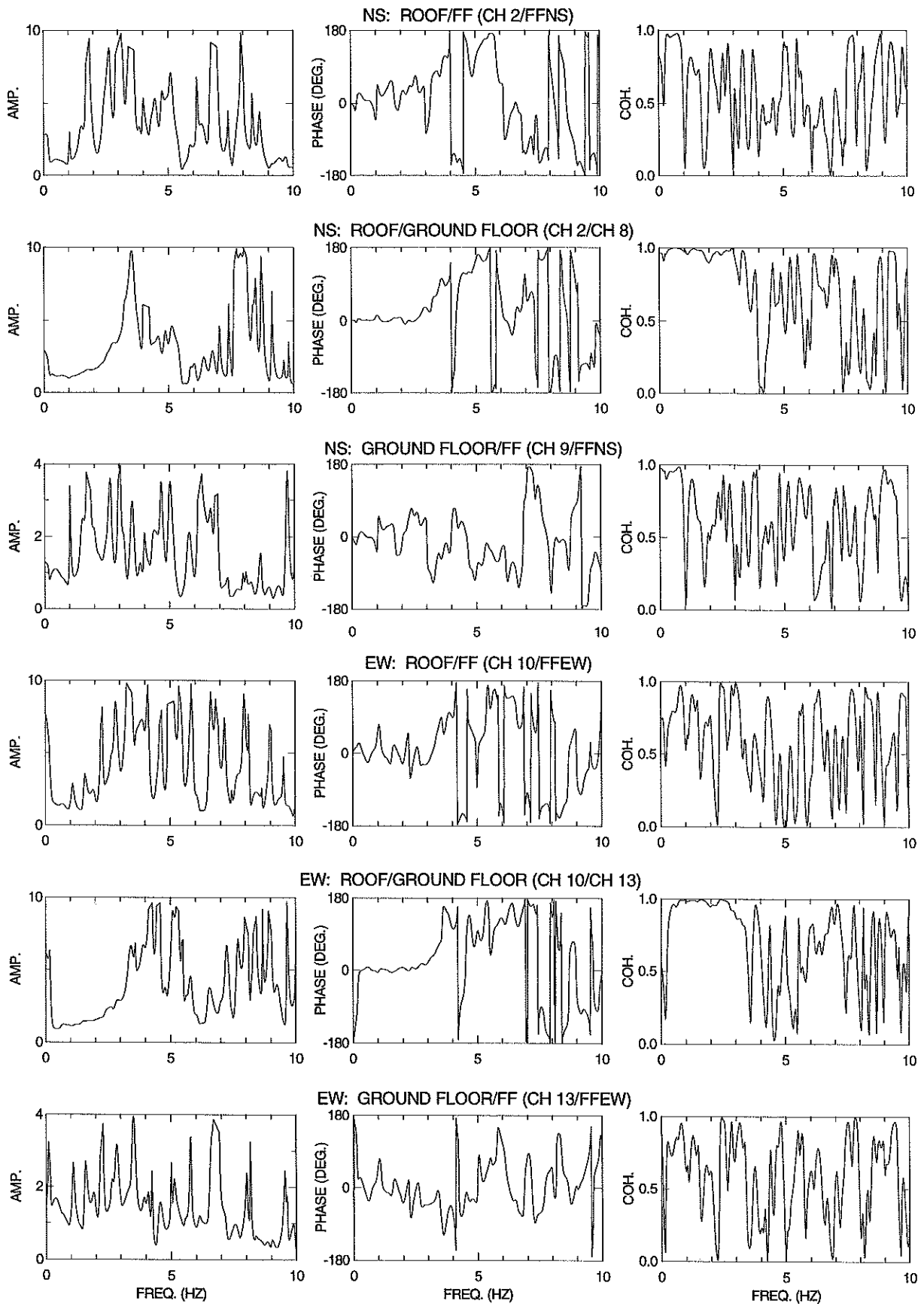


Fig. A44.WT4: SYLMAR 6 STORY COUNTY HOSPITAL
Transmissibility and Coherency Functions
1987 Whittier Earthquake
A44-11

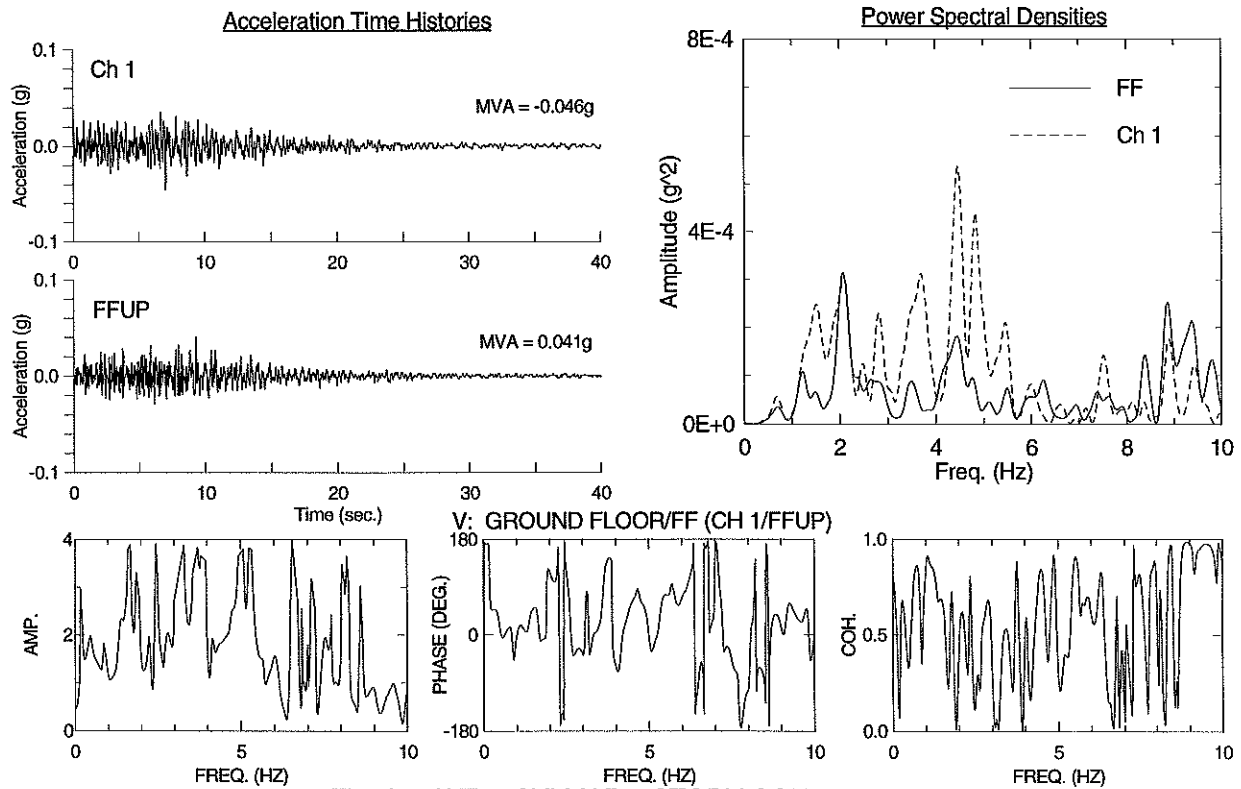


Fig. A44.WT5: SYLMAR 6 STORY COUNTY HOSPITAL
Analysis of Vertical Motions
1987 Whittier Earthquake

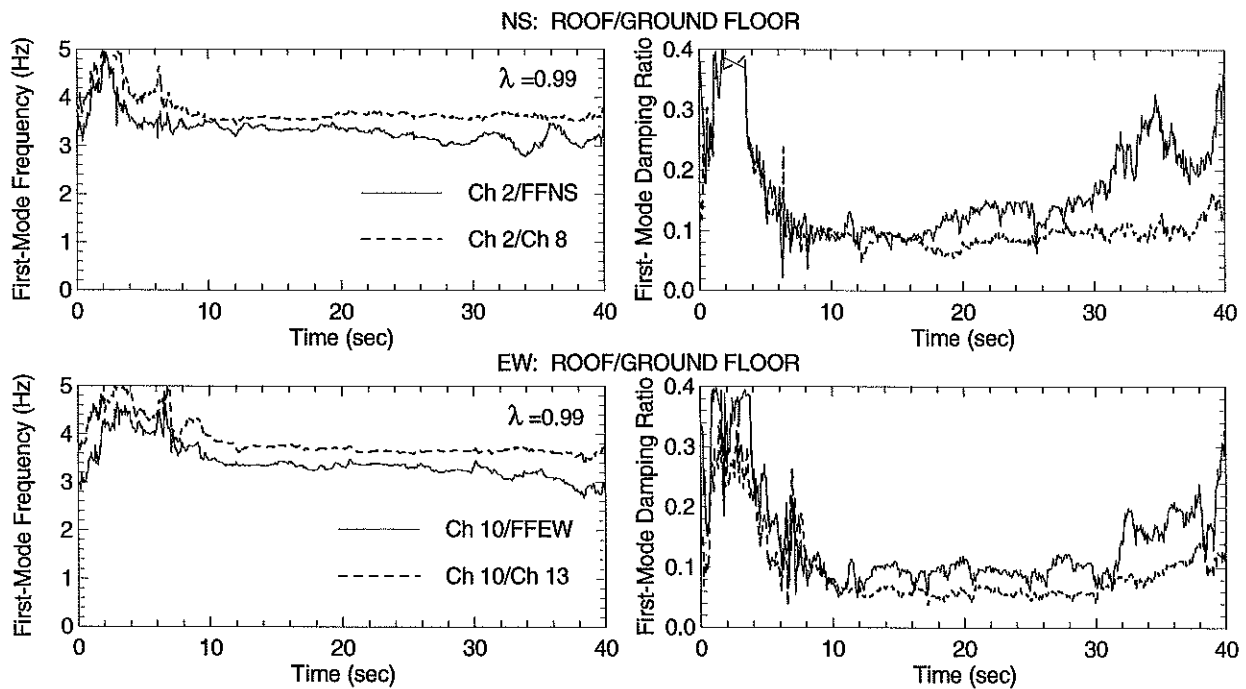


Fig. A44.WT6: SYLMAR 6 STORY COUNTY HOSPITAL
Time Variation of First-Mode Parameters
1987 Whittier Earthquake

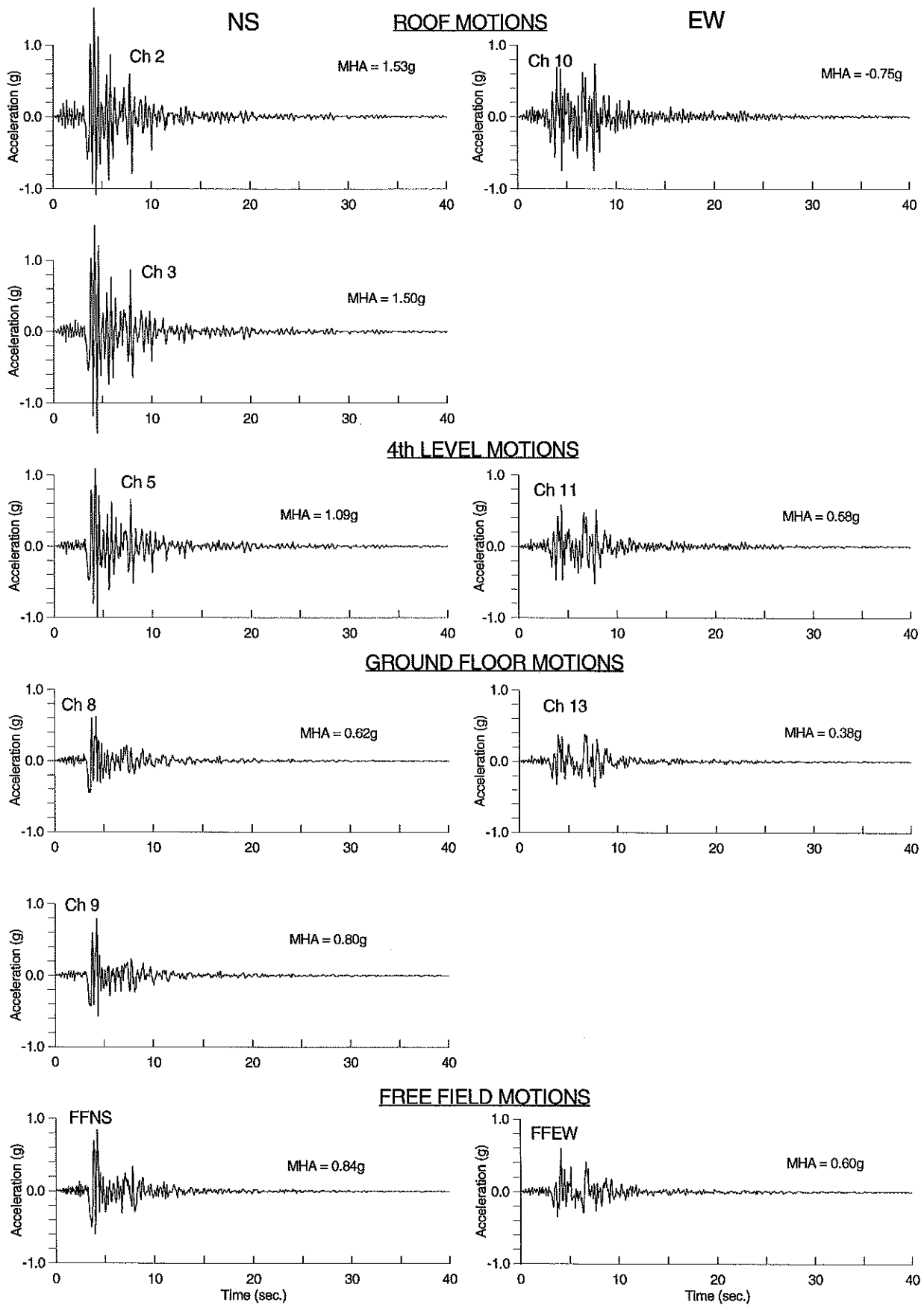


Fig. A44.NR1: SYLMAR 6 STORY COUNTY HOSPITAL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

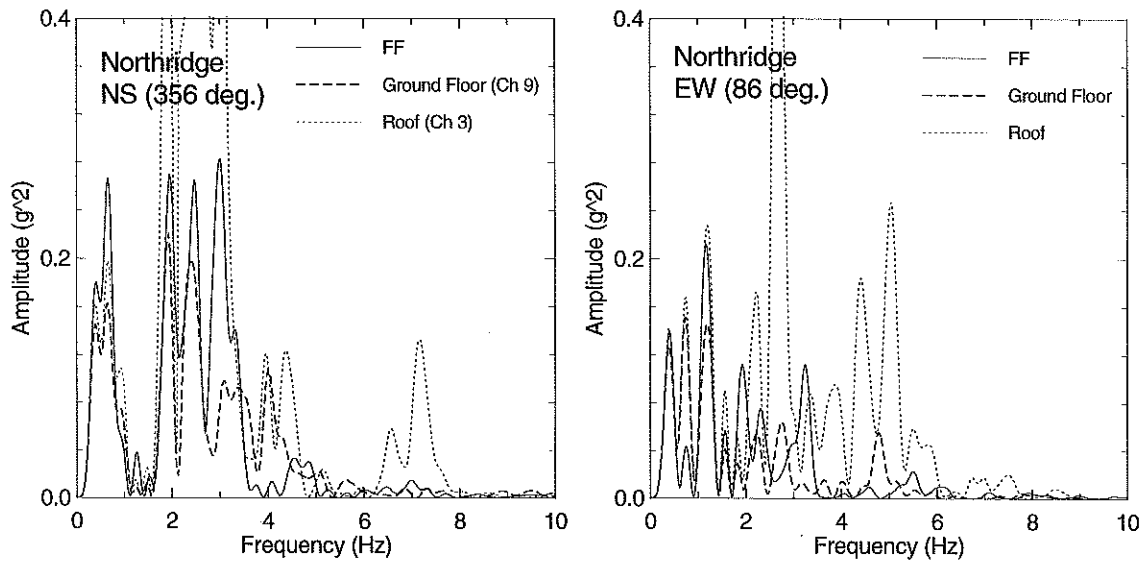


Fig. A44.NR2: SYLMAR 6 STORY COUNTY HOSPITAL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

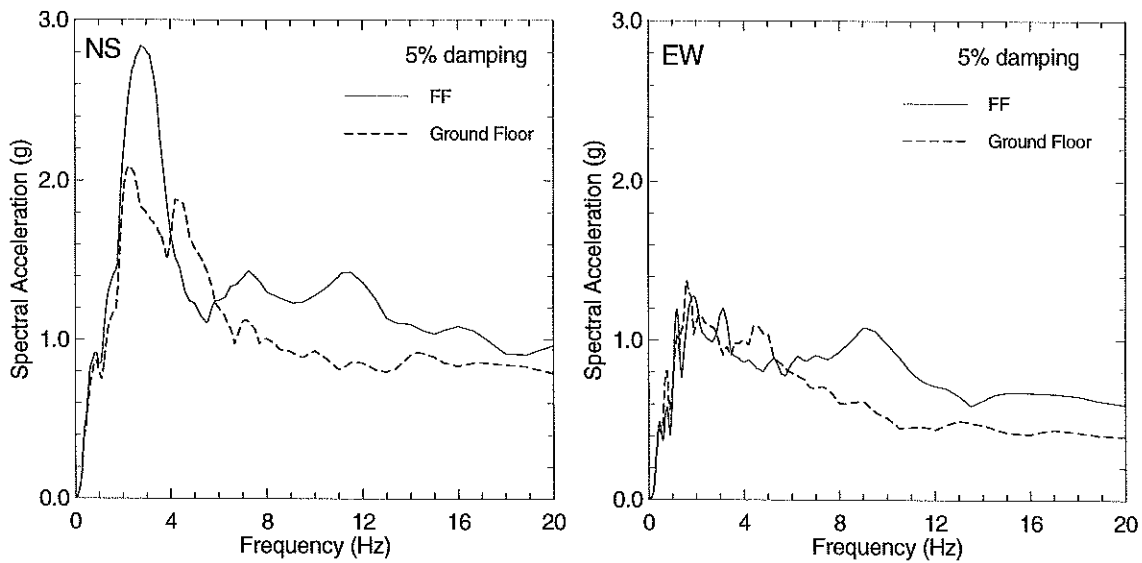


Fig. A44.NR3: SYLMAR 6 STORY COUNTY HOSPITAL
Acceleration Response Spectra of Ground Floor and Free-Field Motions
1994 Northridge Earthquake

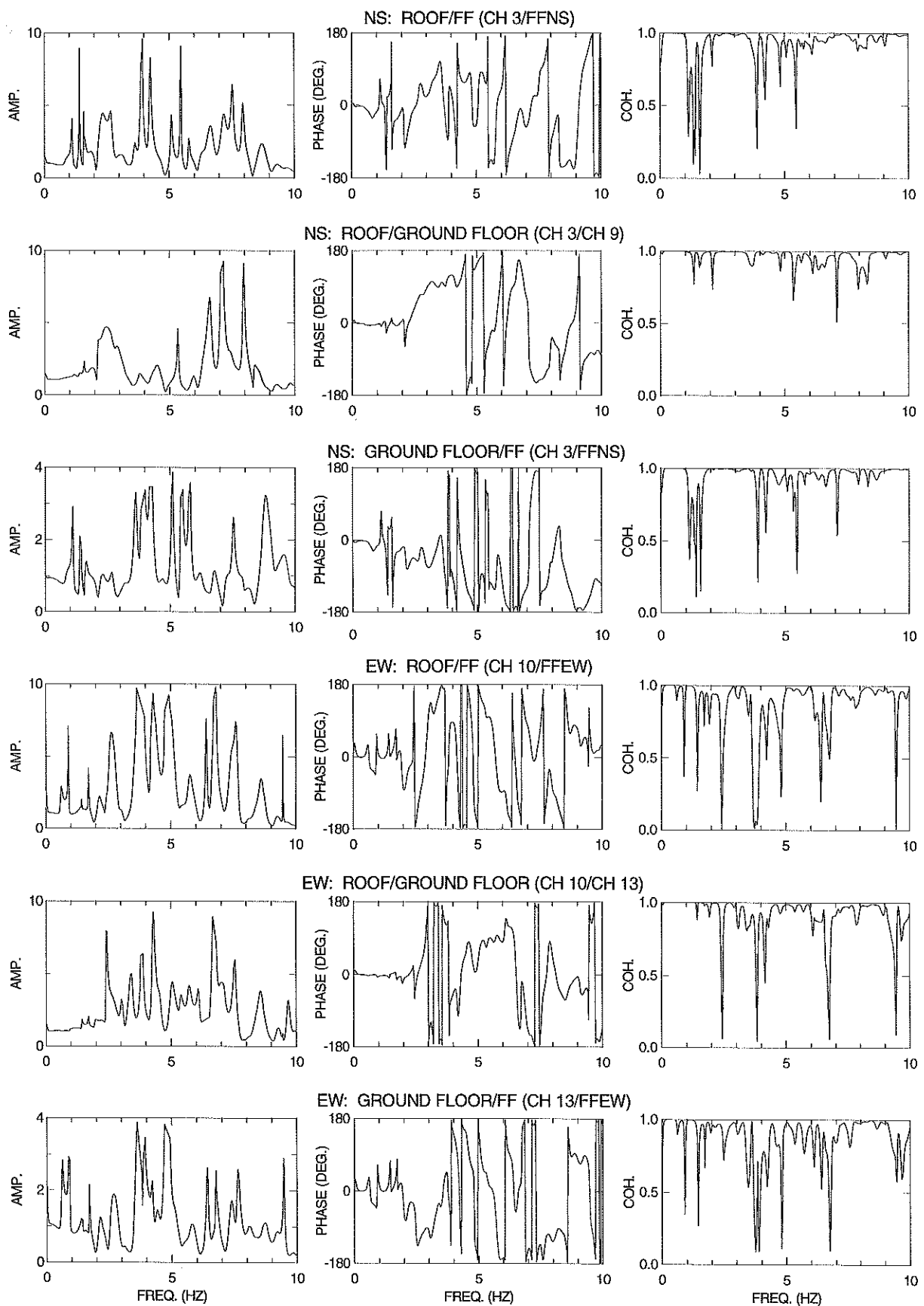


Fig. A44.NR4: SYLMAR 6 STORY COUNTY HOSPITAL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A44-15

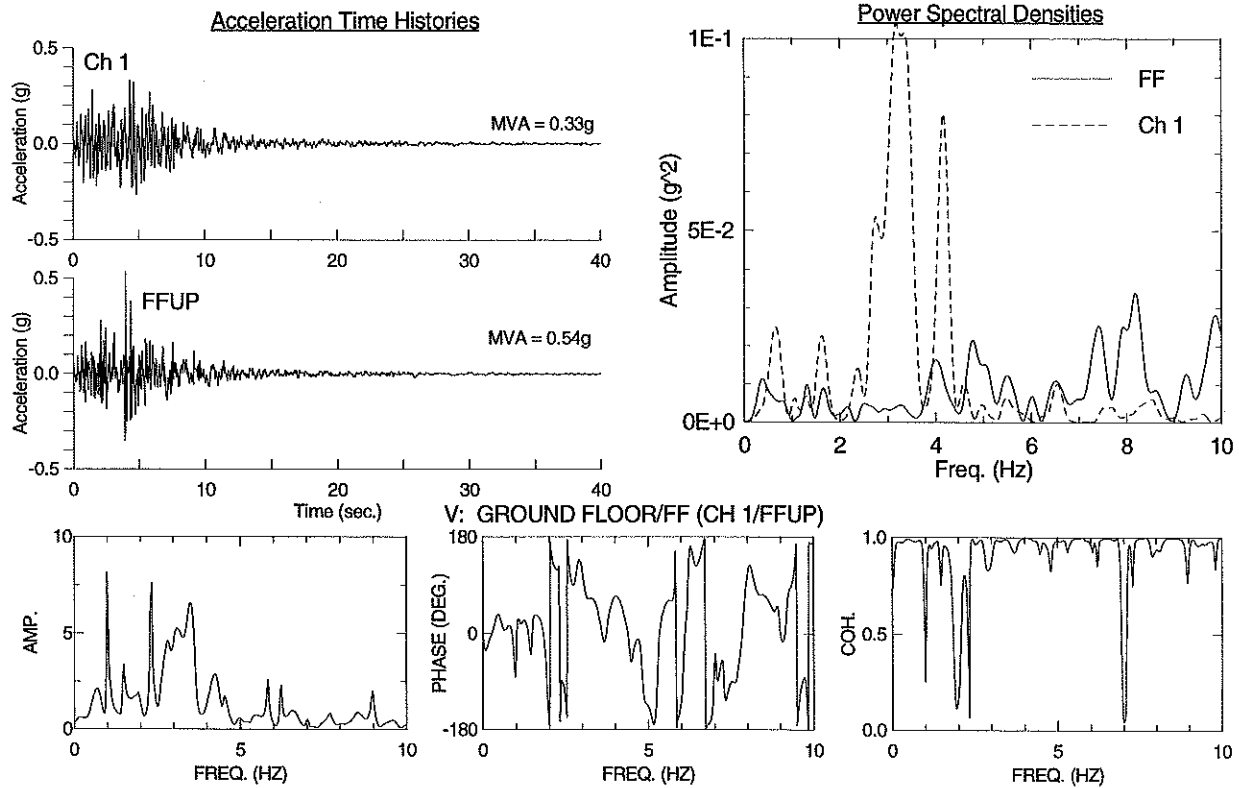


Fig. A44.NR5: SYLMAR 6 STORY COUNTY HOSPITAL
Analysis of Vertical Motions
1994 Northridge Earthquake

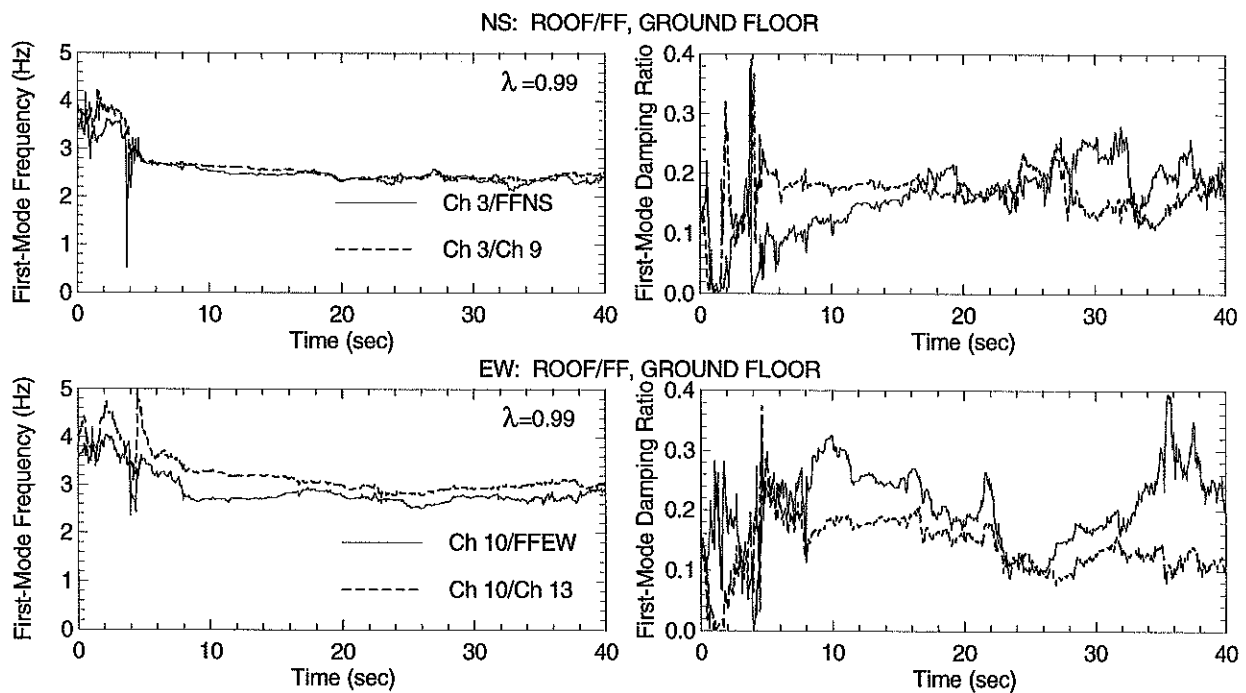


Fig. A44.NR6: SYLMAR 6 STORY COUNTY HOSPITAL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE A45

Site Name: Ventura 12 Story Hotel
Sensors: CSMIP 25339/25340

Lat: 34.276°
Long.: 119.292°

Floor Levels: 12
Bsmt. Levels: 0
Structure: tr: Shear walls, L: concrete frame

Foundation: piles
Site Geology: soil
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Ground Floor			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1994 Northridge	0.075	0.054	0.025	nr	0.071	0.036	0.29	0.25	--

nr = no record

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: CSMIP; (FF) filtered, $f=0.2-23$ Hz, $f_{Nyq}=25$ Hz; (Str. non-vertical channels) filtered, $f=0.24-23$ Hz, $f_{Nyq}=25$ Hz; (Str. Ch 1 & 14) filtered, $f=0.6-23$ Hz; $f_{Nyq}=25$ Hz; FF/str. data not synchronized.

Final: all data decimated by 2 to $f_{Nyq}=12.5$ Hz; FF/str. data synchronized using cross-correlation of vertical accelerations (Fig. A45.NR0).

Figures

A45.1-A45.3: Site Plan, Sensor Locations, Generalized Soil Column
A45.NR0-6 NR Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The structure is 12 stories tall above ground level with no basement, and is rectangular in plan (193 by 61.7 feet). As shown in Fig. A45.1, located adjacent to the structure on the east and west sides are single story buildings. The lateral load resisting system consists of concrete shear walls on the north and south ends of the building for transverse loading, and concrete moment frames coupled with a shear wall inside the building for longitudinal loading. The foundation consists of one-foot head diameter reinforced concrete Raymond tapered piles which extend to a depth of 61 to 66 feet. Beneath the north and south walls, the piles are installed in groups of 4 to 6 at a 3-foot center-to-center spacing, whereas piles in the building interior are installed in 2-pile groups. The heads of all piles are cast into 2.5-foot deep caps which are interconnected between columns by tie beams and a 4.5-inch thick slab-on-grade floor. As shown in Fig. A45.2, accelerographs are present at the ground, 4th, 8th, and roof levels. The free-field instrument is located about 370 feet west of the structure in a parking lot.

The site is located adjacent to a beach on Pierpont Bay and is essentially level. Soil conditions at the site have been investigated by Maurseth, Howe, Lockwood, and Assoc. (1971), and additional data for the Ventura area is available from Fumal et al. (1980). Borings at the building site encountered about 18 feet of clayey sandy silt with occasional sand lenses, 4 feet of fine silty sand, 28 feet of silty clay with thin sand lenses, and older (likely Pleistocene) interbedded clays and sands which extended to the maximum depth explored of 80 feet. Based on measurements by Maurseth, Howe, Lockwood, and Assoc., groundwater occurs at a depth of 20 feet at the site. The shear wave velocities for the upper 50 feet of soil in Fig. A45.3 (which are believed to be Holocene in age) were estimated based on correlations with void ratio and in situ downhole measurements in similar soils at the Ventura County General Hospital site (Fumal et al., 1980). Velocities in the underlying older soils were estimated from correlations with void ratio.

Instrument Selection and Free-Field Conditions

As shown in Fig. A45.2, the building is generally instrumented along its north and south shear walls. Due to an error in the film recordings, data from Channels 13 and 15 on the ground floor could not be processed. For this reason, the longitudinal response from ground floor-roof pairs could not be evaluated. It was also necessary to couple lateral ground floor motions from the north end of the building with base rocking motions from the south end for analysis of fixed-base response in the EW direction. The specific sensors used are indicated in Table A45.1; acceleration time histories recorded by these sensors are shown in Fig. A45.NR1.

Table A45.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 15	Ch 12	Ch1, 14	Ch 2	Ch 5

The following input was used in system identification analyses to evaluate fixed-base structural parameters for the EW direction,

$$\text{input} = \text{Ch12} + (\text{Ch1} - \text{Ch14}) \cdot 115.67/56 \quad (1)$$

There are no structures or significant slope breaks near the free-field instrument that would be expected to compromise the validity of the recorded free-field ground motions.

Analysis Results

(a) Comparison of Ground Floor and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

The GF/FF transmissibility function (Fig. A45.NR4) and the spectra of horizontal motions (Fig. A45.NR2-3) for the EW direction indicate amplification of GF motions relative to the FF near the first-mode frequency (about 0.5 to 2.5 Hz), and de-amplification at higher frequencies.

Transmissibility functions for the GF/FF vertical pair (Fig. A45.NR5) indicate significant amplification near the first-mode frequencies for lateral response, suggesting that rocking likely contributed to the first-mode structural response. A direct examination of rocking contributions to roof motions is made in Fig. A45.NR5 by comparing relative lateral roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements and have about 30 to 40% of the roof displacement amplitude, indicating that base rocking significantly influenced the roof recordings.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A45.NR4), (2) modal parameters from CEM analyses (Table A45.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A45.NR6).

Table A45.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flex.-Base (FF-R)			
		tr (EW) d=0, j=4		L (NS)		tr (EW) d=1, j=4		L (NS) d=--, j=--		tr (EW) d=1, j=4		L (NS) d=2, j=4	
		f	ζ	f	ζ	ƒ*	ζ*	ƒ*	ζ*	ƒ	ζ	ƒ	ζ
NR	1	1.88	5.7	na	na	1.49	3.7	na	na	1.41	4.0	0.82	9.4
	2	3.17	33.6			2.53	18.8			2.82	20.1	3.95	7.3
	3	6.40	6.7			6.80	7.7			6.44	9.0	6.82	6.5

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response in the transverse direction is dominated by the first mode.

- For the transverse direction, the significant difference between the fixed- and pseudo flexible-base parameters is consistent with the rocking effect noted in Part (a).

The RPEM results (Fig. A45.NR6) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the FF-R and GF-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A45.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 62 feet using the recorded free-field motions and the profile in Fig. A45.3. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values for the transverse direction were increased by a factor of 2.0 to account for shape effects.

Table A45.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=920$ fps	$h=69$ ft	emb.=0	$\gamma = 0.15$
NR	$\nu=0.45$		tr: $r_1 = 62$ ft		$r_2 = 47$ ft
	$\beta^*=2.8\%$	$V_s^*=890$ fps	L: $r_1 = 62$ ft		$r_2 = 83$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A45.3 with the fixed-base modal parameters in Table A45.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table A45.4.

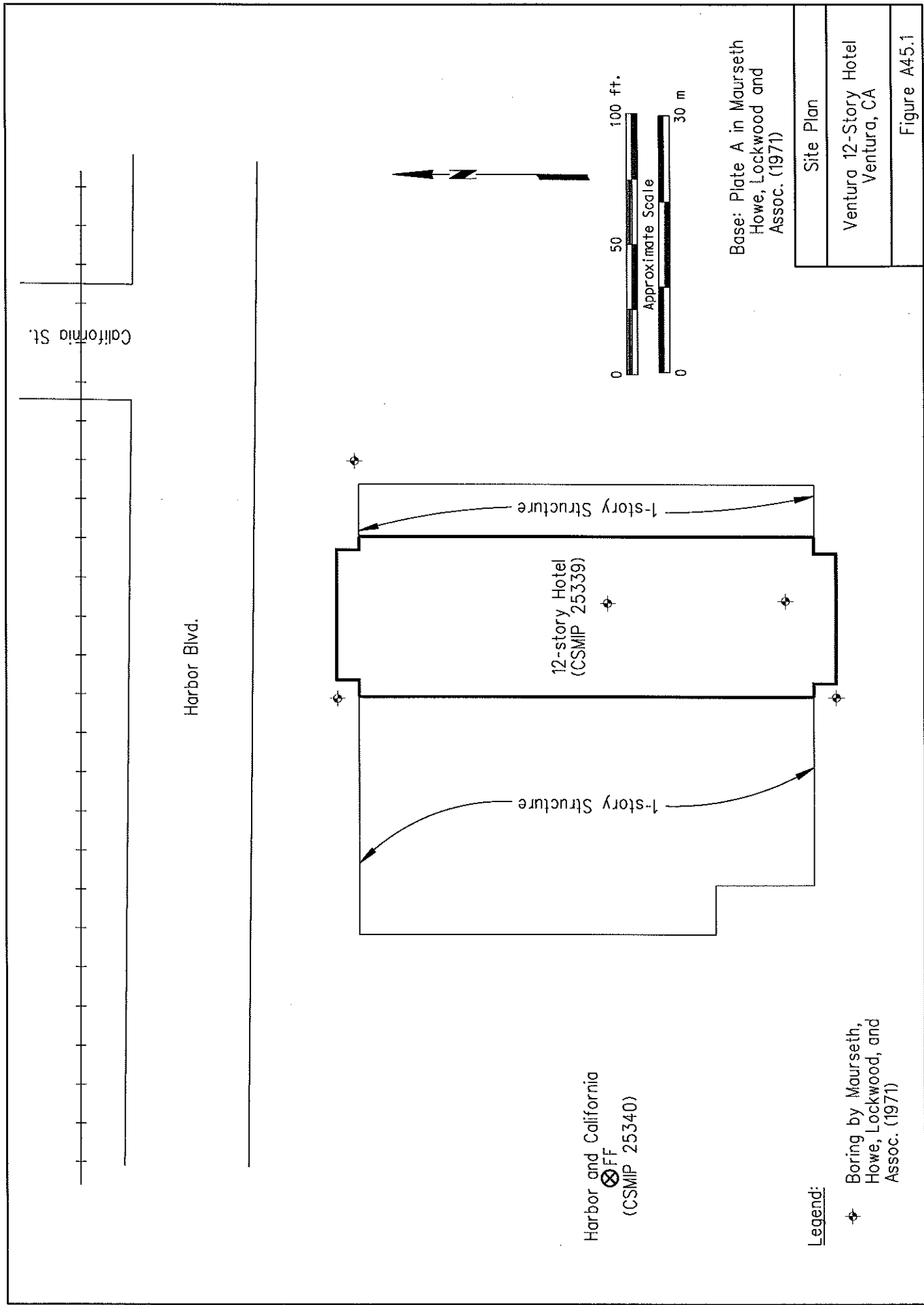
Table A45.4: Empirical and predicted inertial interaction effects

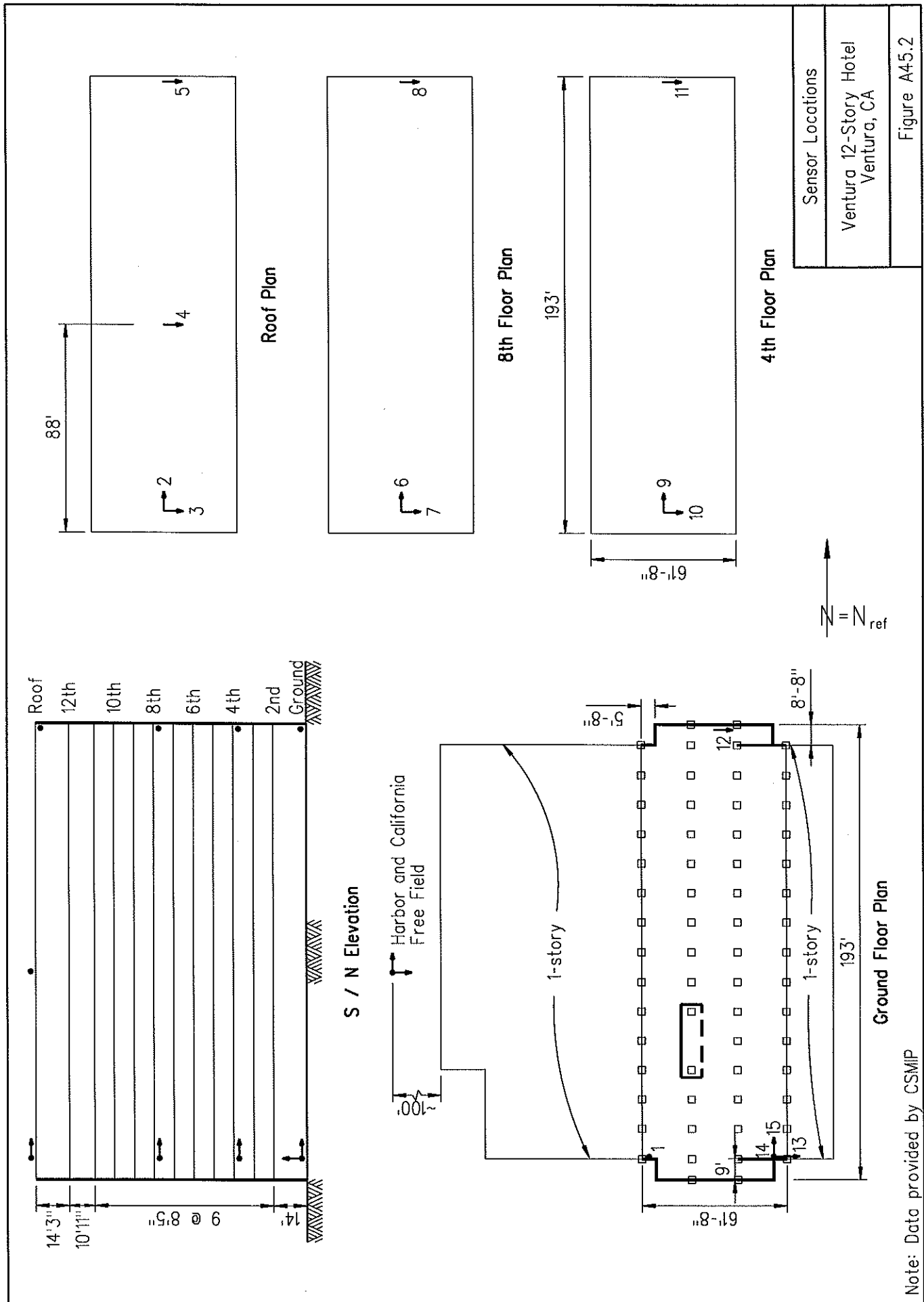
		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
NR	Sys. ID	1.34	1.6	4.0	--	--	--
	Prediction	1.14	1.4	5.3	--	--	--

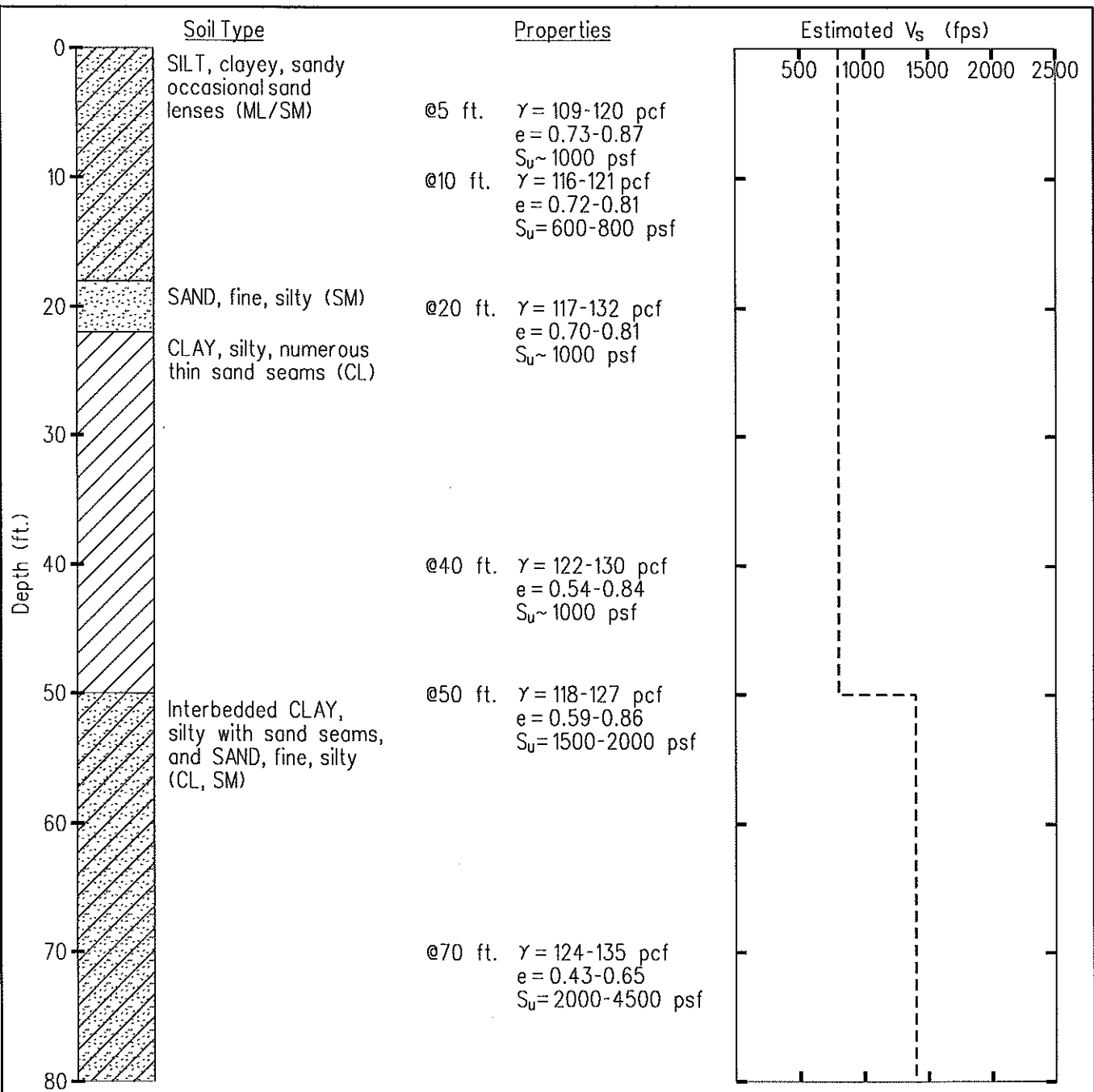
The period lengthening was underpredicted, while the foundation damping reasonably well predicted. Errors in the prediction of period lengthening were likely associated with incorrect characterization of the shear wave velocity profile at the site (the V_s values were estimated based on measurements at other sites and correlations with void ratio).

Data Sources

Gibbs, J.F., Fumal, T.E., and Roth, E.F. (1980). "In-situ measurements of seismic velocity at 27 locations in the Los Angeles, CA region," *Open File Rpt. 80-378*, U.S. Geological Survey.
Maurseth, Howe, Lockwood and Assoc. (1971). "Report, Foundation Investigation, Holiday Inn Hotel, Ventura, CA," *Proj. 5725-F*.







Data Source:
Maurseth, Howe,
Lockwood &
Assoc. (1971)

Notes:

1. Groundwater encountered at 17-20 ft. depth.
2. Shear strengths estimated by unconfined compressive strength tests.
3. Shear wave velocities estimated based on correlations by Fumal & Tinsley (1985) and in situ measurements in similar geologic materials at nearby sites (Ventura Co. General Hosp., Gibbs et al., 1980).

Generalized Soil Profile

Ventura 12-Story Hotel
Ventura, CA

Figure A45.3

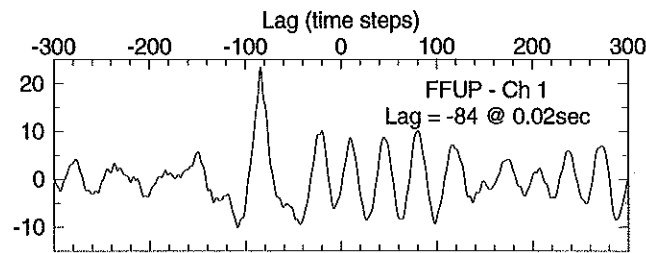


Fig. A45.NR0: Cross-correlation function between vertical ground floor and free-field accelerations

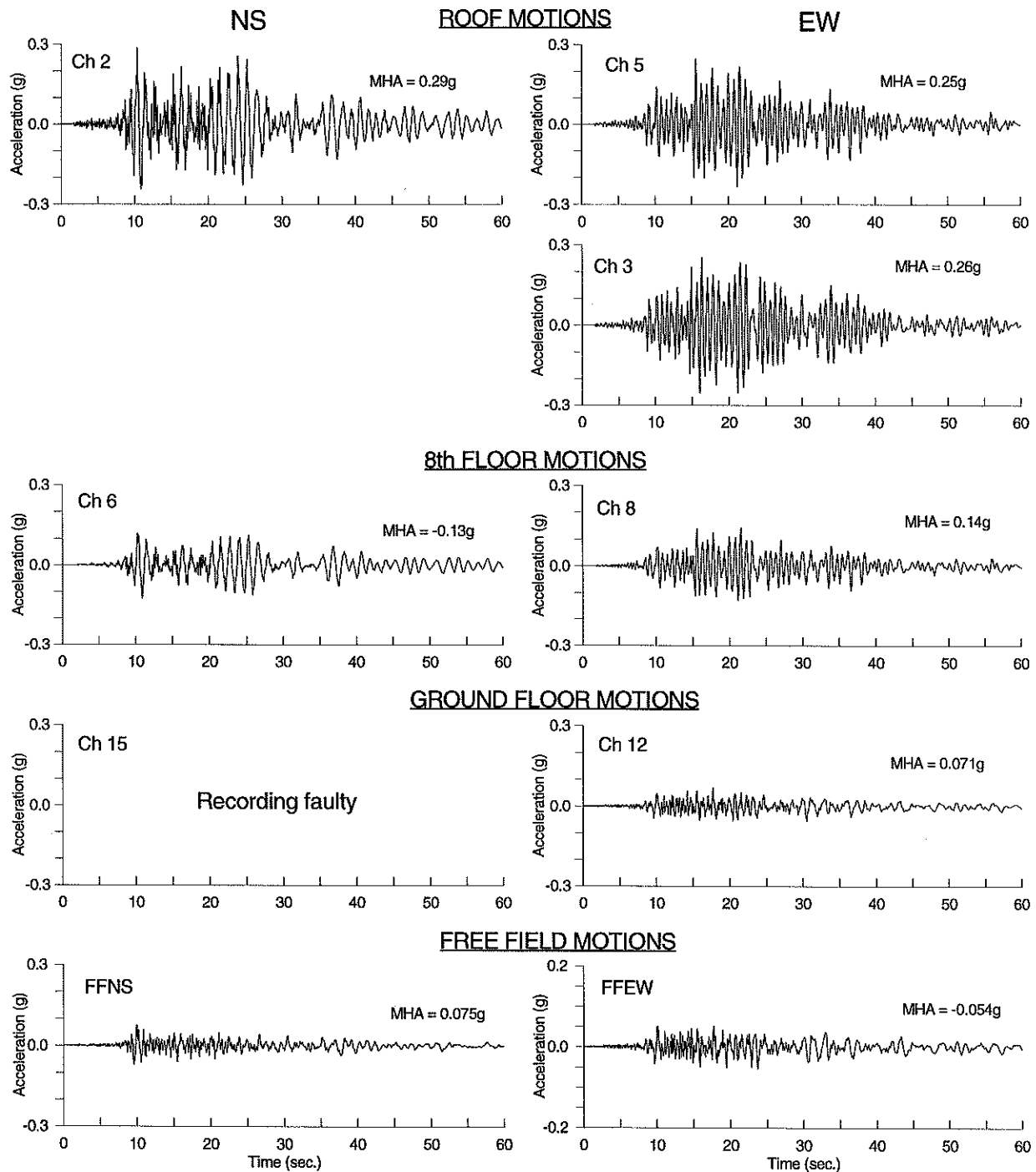


Fig. A45.NR1: VENTURA 12 STORY HOTEL
Horizontal Acceleration Time Histories
1994 Northridge Earthquake

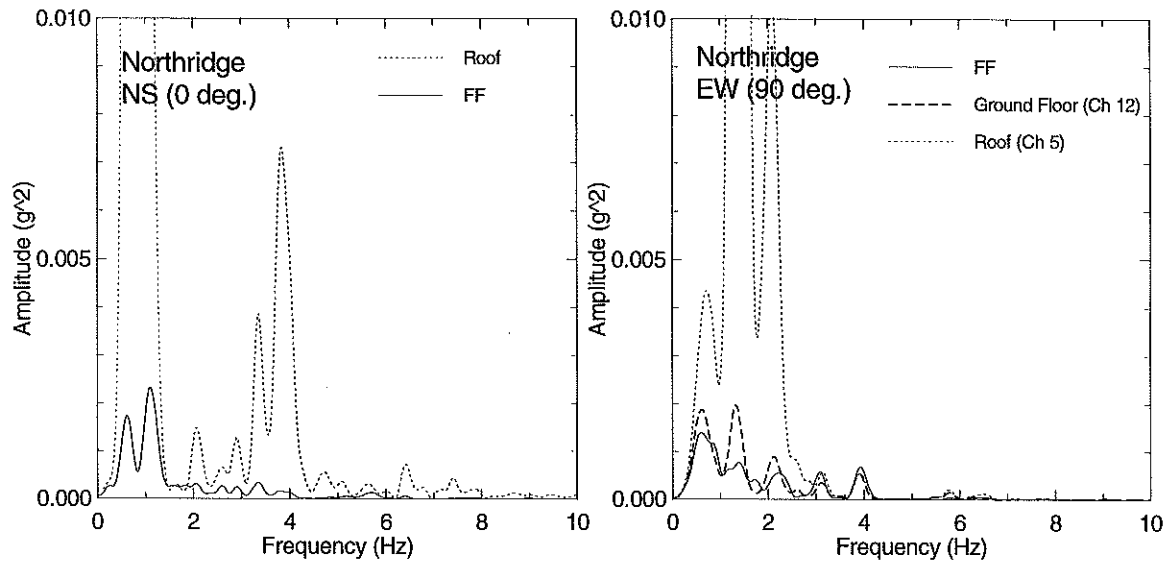


Fig. A45.NR2: VENTURA 12 STORY HOTEL
Power Spectral Density of Selected Horizontal Accelerations
1994 Northridge Earthquake

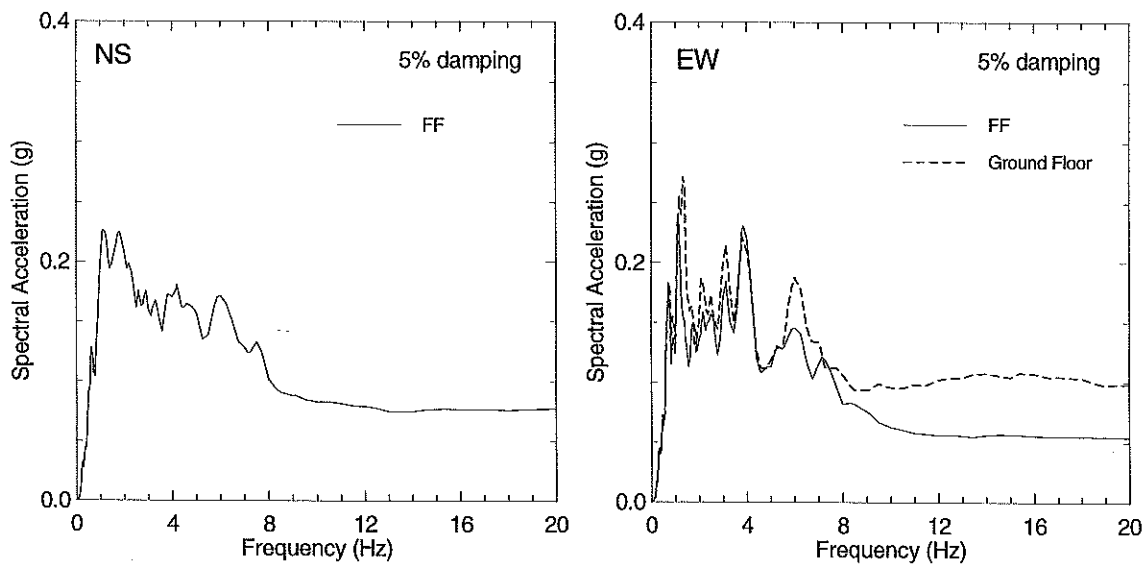


Fig. A45.NR3: VENTURA 12 STORY HOTEL
Acceleration Response Spectra of Horizontal Ground Floor and Free-Field Motions
1994 Northridge Earthquake

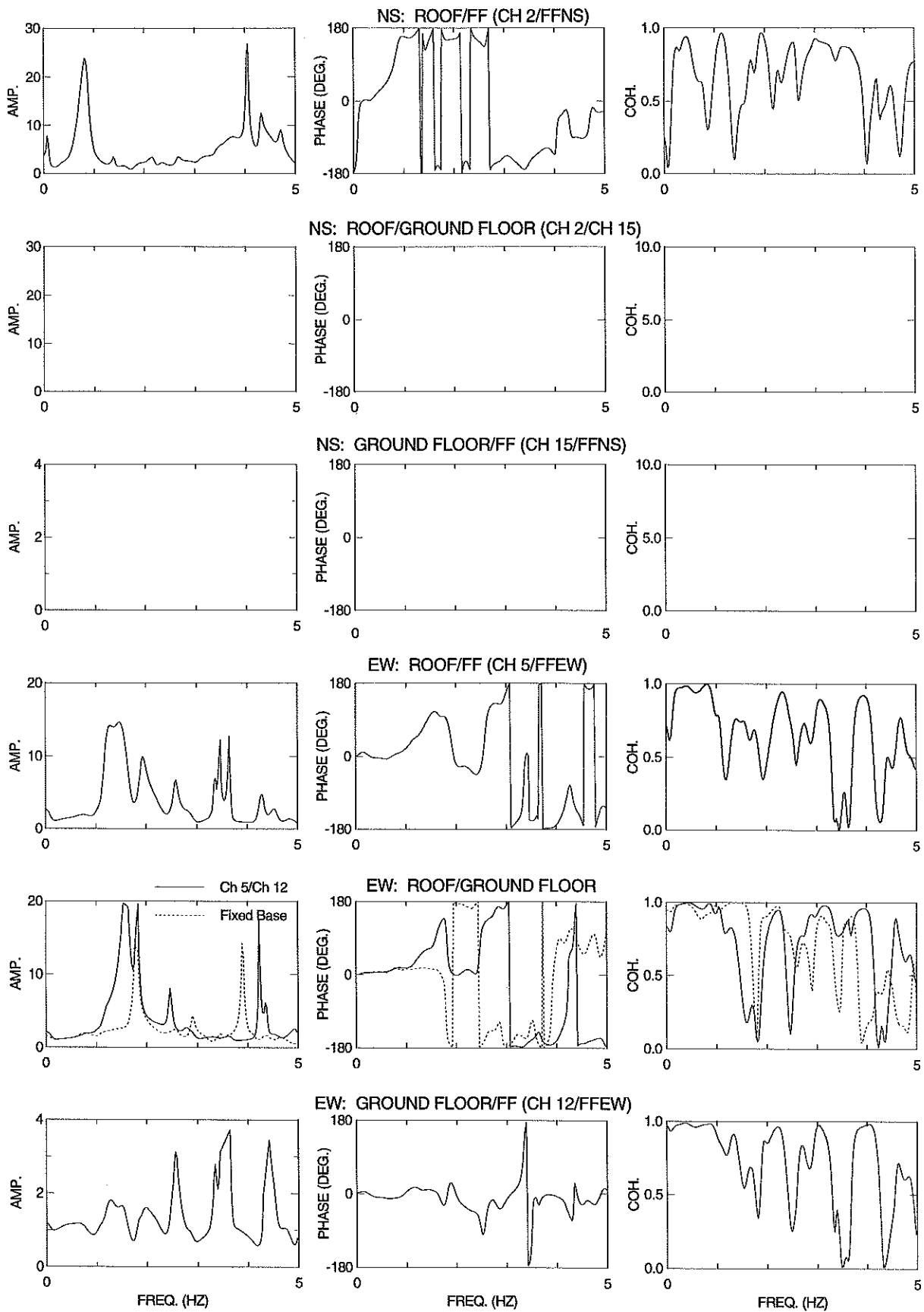


Fig. A45.NR4: VENTURA 12 STORY HOTEL
Transmissibility and Coherency Functions
1994 Northridge Earthquake
A45-10

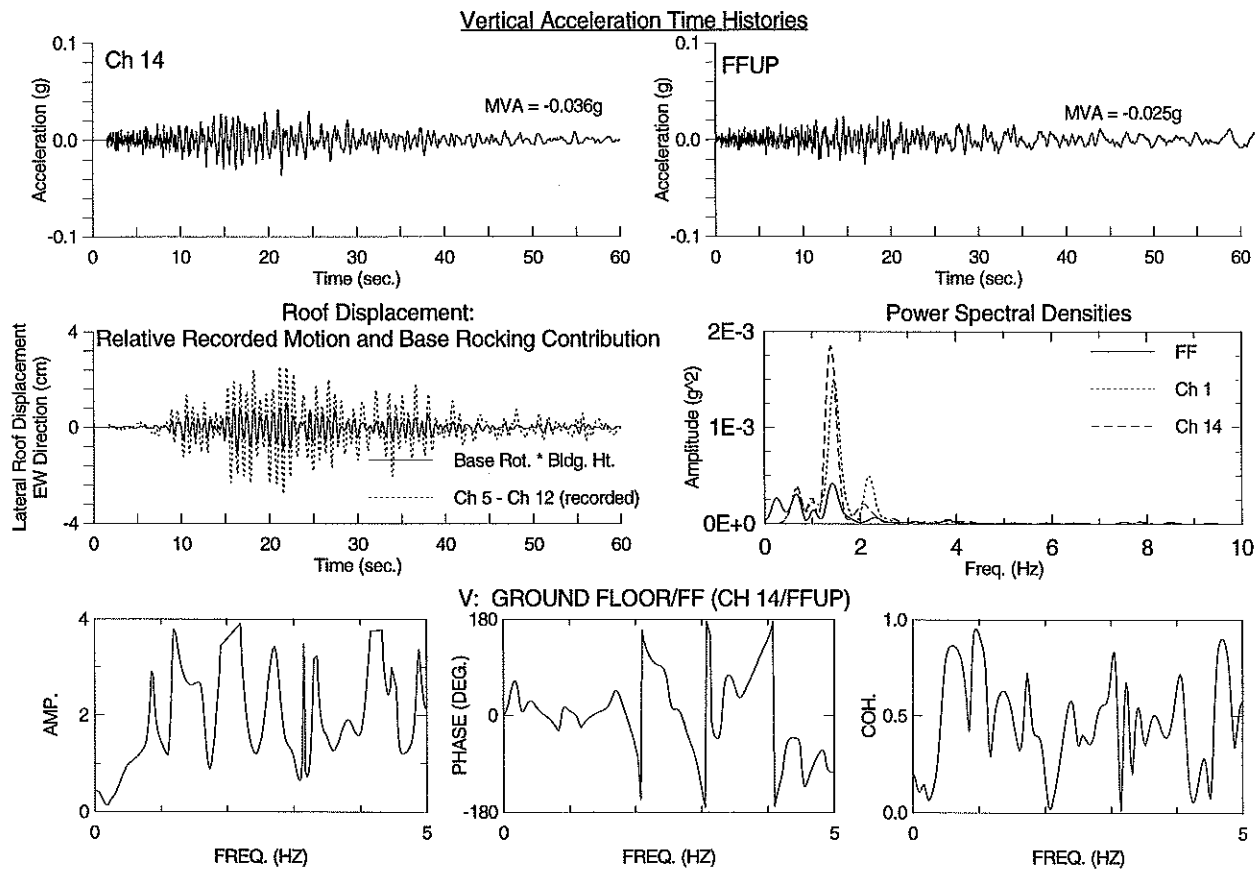


Fig. A45.NR5: VENTURA 12 STORY HOTEL
Analysis of Vertical Motions and Rocking Effects
1994 Northridge Earthquake

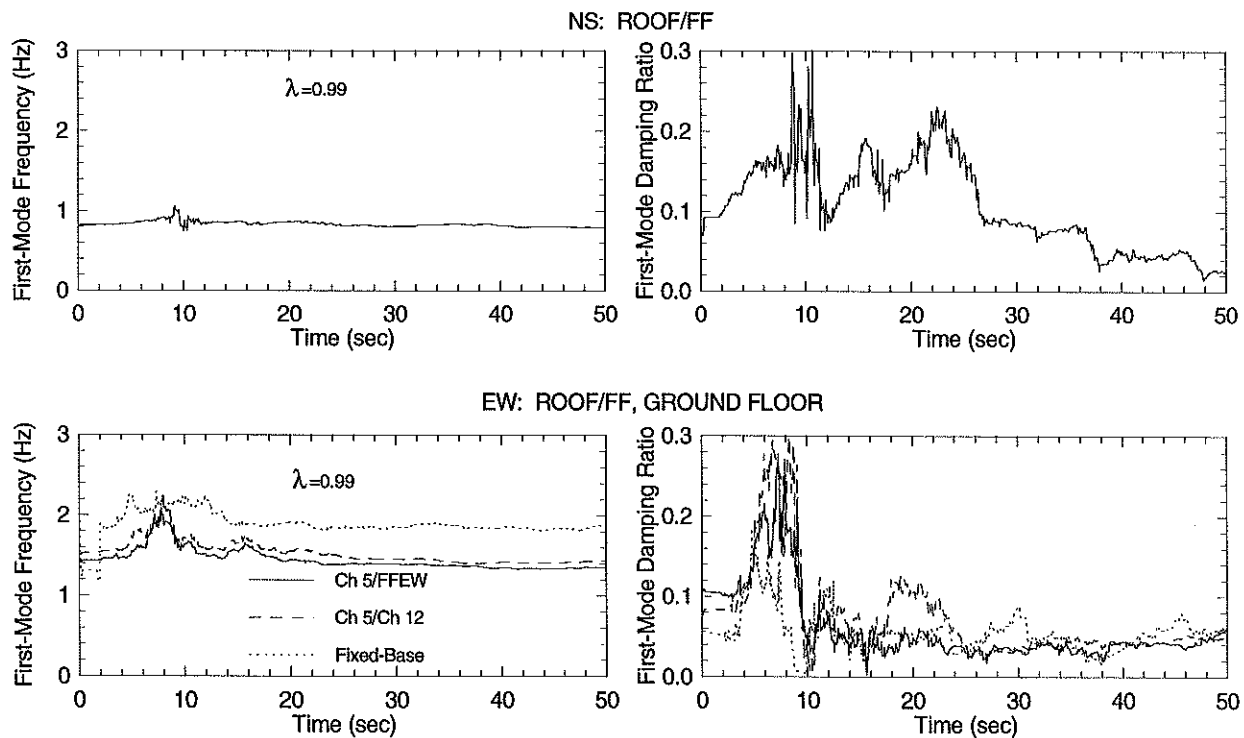


Fig. A45.NR6: VENTURA 12 STORY HOTEL
Time Variation of First-Mode Parameters
1994 Northridge Earthquake
A45-11

SITE A46

Site Name: Lotung, Taiwan 1/4-scale
Containment Model

Lat: 24.660°
Long.: 121.755°

Sensors: EPRI

Floor Levels: na
Bsmt. Levels: na
Structure: Cylindrical reinforced concrete

Foundation: mat
Site Geology: soil
Const. Date: 1985

Recorded Peak Accelerations (g)

Earthquake	Free-Field			Base			Roof		
	NS	EW	V	NS	EW	V	NS	EW	V
1986 LSST07	0.21	0.16	0.041	0.26	0.24	0.16	0.21	0.20	0.066
	0.19	0.16	0.043	0.13	0.15	0.070	0.22	0.20	0.071
	0.20	0.19	0.042	0.13	0.15	0.083	0.21	0.21	0.086
				0.13	0.15	0.041	0.22	0.21	0.042

list order: FA1-5, FA2-5, FA3-5 (FF); F4_N, F4_S, F4_W, F4_E (Str.)

Preprocessing

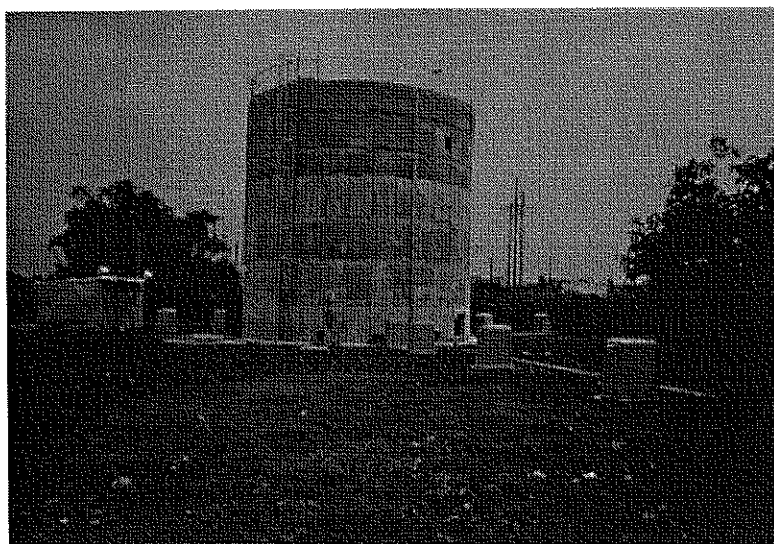
Event LSST07

Preliminary: EPRI $f_{HP} = 0.1$ Hz; $f_{Nyq} = 100$ Hz, FF/str. data synchronized; FF aligned with str.
Final: FF/str. data decimated by 4 to $f_{Nyq} = 25$ Hz.

Figures

A46.1-A1.3: Site Plan, Sensor Locations, Generalized Soil Column
A46.07-1 to 6 LSST07 Analysis Results

View of containment
structure and adjacent
free-field instruments
(photo courtesy of H.T.
Tang, Electrical Power
Research Institute)



DISCUSSION

Site Description

The structure is a reinforced concrete 1/4-scale model of a nuclear containment facility. As shown in Fig. A46.2, the cylindrical structure has a radius of 16.25 feet, an overall height of 50 feet, and is embedded 15 feet. The structure is founded on a 3-foot thick reinforced concrete mat foundation. The 1/4-scale structure is part of the Lotung large-scale seismic test (LSST) facility in Taiwan. This facility also has a 1/12 scale containment structure and a free-field array of 15 ground surface instruments spaced 5 to 100 feet from the 1/4-scale model and two downhole arrays to depths of about 150 feet. As shown in Fig. A46.2, accelerographs are present on four sides of the structure at the base level and about 8.5 feet below the roof level. Three free-field instruments were used in the analyses, each located 100 feet from the structure.

The site is located on level terrain near the southern end of the Lanyang River plain. Soil conditions encountered at the site (Fig. A46.3) consist of about 10 feet of silty sand and sandy silt with some gravels, about 100 feet of silty sand, sandy silt, and clayey sand, and about 90 feet of silty clay and clayey silt (Geomatrix, 1991). Based on depth-to-groundwater measurements, the soils at the site are assumed to be saturated below a depth of 3 feet. The shear wave velocities shown in Fig. A46.3 were measured by uphole and crosshole in situ testing (Geomatrix, 1991).

Instrument Selection and Free-Field Conditions

The configuration of the structure is regular, but a simulated steam generator is located 7.5 feet from the centerline. The lateral and vertical accelerographs used in the system identification analyses are presented in Table A46.1. Four vertical instruments located on the base level were used to evaluate rocking in both directions. Acceleration time histories recorded by these and other sensors are shown in Fig. A46.07-1.

Table A46.1: Sensors in structure used for analyses

Base (B)			Roof (R)	
NS	EW	V	NS	EW
F4LE	F4LS	all	F4UE	F4US

The following inputs were used in system identification analyses to evaluate "fixed-base" structural parameters for the NS and EW directions,

$$\text{NS: input} = F4LE_{ns} + (F4LS, V - F4LN, V) \cdot 38.5/32.5 \quad (1)$$

$$\text{EW: input} = F4LN_{ew} + (F4LW, V - F4LE, V) \cdot 38.5/32.5$$

There are no structures or significant slope breaks near free-field instruments FA1-5 and FA2-5 that would be expected to compromise the validity of the recorded free-field ground motions. FA3-5 is located on top of an approximately 3-foot high berm between two lower-level rice fields.

Analysis Results

(a) Comparison of Base and Free-Field Motions and Evaluation of Base Rocking Contributions to Roof Motion

B/FF transmissibility functions (Fig. A46.07-4) and the spectra for horizontal motions (Fig. A46.07-2 to 3) indicate de-amplification of base motions for frequencies greater than about 2 Hz.

Transmissibility functions for B/FF vertical pairs (Fig. A46.07-5) indicate significant amplification across a range of frequencies ranging from $f \approx 1$ to 3.5 Hz, suggesting that base rocking significantly contributed to the structural response. A direct examination of rocking contributions to roof motions is made in Fig. A46.07-5 by comparing relative lateral roof accelerations with the product of base rotation and building height ($R \times H$) over the time span of 5 to 25 seconds. The calculated $R \times H$ accelerations are generally in phase with the relative roof accelerations and have nearly 100% of the roof displacement amplitude, indicating that base rocking strongly influenced the lateral roof recordings. This is especially true in the EW direction where the $R \times H$ and roof accelerations are nearly indistinguishable.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. A46.07-4), (2) modal parameters from CEM analyses (Table A46.2), and (3) time variation of first-mode parameters from the RPEM (Fig. A46.07-6). First-mode parameters evaluated by Bechtel (1991) from forced vibration tests and four earthquakes are presented in Table A46.3 for comparison.

Table A46.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flex.-Base (FF-R) (FA1/2/3 - 5 $\rightarrow$ R)			
		tr (NS) see text		L (EW) see text		tr (NS) d=0, j=3		L (EW) d=0, j=3		tr (NS) d=0/0/1, j=3/3/3		L (EW) d=0/0/0, j=3/3/3	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	8.5	3	9.0	3	2.37	16.7	2.42	12.9	1.97/2.11/ 1.68	31.3/29.9/ 31.0	2.22/2.28/ 1.64	32.1/30.0/ 34.2

Table A46.3: Flexible-base first-mode parameters evaluated from locations of peaks in transfer function and half-power bandwidth method (Bechtel, 1991)

Excitation	NS	EW	NS		EW	
	MHA	MHA	f (Hz)	ξ (%)	f (Hz)	ξ (%)
FVT ¹	na	na	3.8	10	3.9	10
LSST06	0.03	0.04	3.6	13	3.3	13
LSST16	0.17	0.13	1.9	>25	2.2	>25
LSST12	0.19	0.16	2.1	>25	2.0	>25
LSST07	0.21	0.16	1.7	>25	2.2	>25

¹FVT = Forced Vibration Test

Comments on the analytical results follow:

- The power spectra (Fig. A46.07-2) indicate that the roof motion is dominated by excitation at the predominant frequency of the input motions ($f \approx 1.5$ to 2.0 Hz) and first-mode structural response.
- When base translation and rocking are removed by using Eq. 1 as input to the identification, the structure is found to be very stiff. This is illustrated in nonparametric analyses for the fixed-base case by transmissibility function amplitudes and coherencies close to unity and nearly zero phase shift from 0 to 10 Hz. Due to this stiff response (and a lack of significant seismic energy at the high structural frequencies), reliable parametric models for the fixed-base case could not be estimated. However, based on the nonparametric transmissibility functions, there appears to be some structural excitation and phase shift at about 8 to 9 Hz. This result is in general agreement with fixed-base frequencies of 10.8 to 11.4 Hz evaluated from finite element analyses using data from forced vibration tests (Bechtel, 1991). Based on these analyses, the fixed-base frequency during LSST07 is assumed to be 8.5 and 9.0 Hz in the NS and EW directions, respectively. Fixed-base damping ratios under seismic excitation of 1 to 4% have been estimated by various researchers (Bechtel, 1991), and were assumed to be 3% for this study.
- The large differences between the fixed- and pseudo flexible-base parameters is consistent with the large rocking effect noted in Part (a).
- The flexible-base modal parameters in Table A46.2 are similar to frequencies evaluated from the peaks of transfer functions and damping ratios determined by the half-power bandwidth method by Bechtel, 1991 (see Table A46.3).
- The reason for the significant difference between the flexible-base CEM results obtained using FA3-5 and FA1-5/FA2-5 is not certain, but may be related to topographic effects on the motions recorded at FA3-5. Previous researchers have noted differences between motions at FA3-5 and the other free field stations (Geomatrix, 1991). Based on these observations, it appears that the flexible-base results obtained from FA1-5 and FA2-5 are more reliable.
- As illustrated by a comparison of the first-mode parameters in Tables A46.2 and A46.3, the system response is much "softer" and more highly damped during strong earthquake shaking relative to forced vibration tests.

The RPEM results (Fig. A46.07-6) indicate significant reductions (about 1.3 Hz) in the first-mode frequencies for both directions coincident with the arrival of the first large amplitude shear wave at about 11 seconds. The cause of this reduction is not known, but is likely related to rapid degradation of the soil stiffness. As the amplitude of strong shaking subsided, the first-mode frequencies increased towards the initial value (indicating no permanent damage such as would result from damage to the structure). The CEM and RPEM results were consistent with regard to the relative magnitude of the FF-R and B-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table A46.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 31 feet using the recorded free-field motions and the profile in Fig. A46.2. No

adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table A46.4: Soil/structure parameters used in predictive analyses

	Geotechnical	Structural
LSST07	Model = H $V_s=460$ fps	$h=47$ ft emb.=15 ft $\gamma = 0.18$
	$\nu=0.45$	tr: $r_1=16.25$ ft $r_2=16.25$ ft
	$\beta^*=9.7\%$ $V_s^*=275$ fps	L: $r_1=16.25$ ft $r_2=16.25$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table A46.4 with the fixed-base modal parameters discussed in Part (b), the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table A46.5.

Table A46.5: Empirical and predicted inertial interaction effects

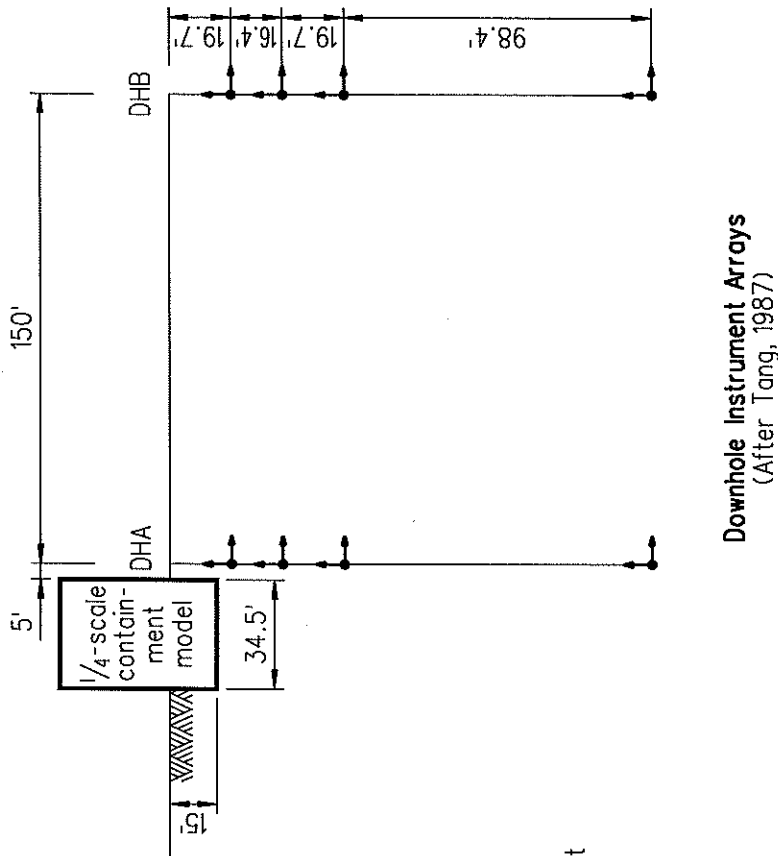
		tr (NS)			L (EW)		
		$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$	$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$
LSST07	Sys. ID	4.14	30.6	30.6	4.01	31.0	31.0
	Pre.-Vel.	3.76	3.3	3.4	3.97	3.3	3.4
	Pre.-Bie.	4.94	26.7	26.8	5.21	26.9	26.9

The period lengthening was reasonably well predicted by both formulations, though the foundation damping estimates by the “modified Bielak” formulation was much more accurate.

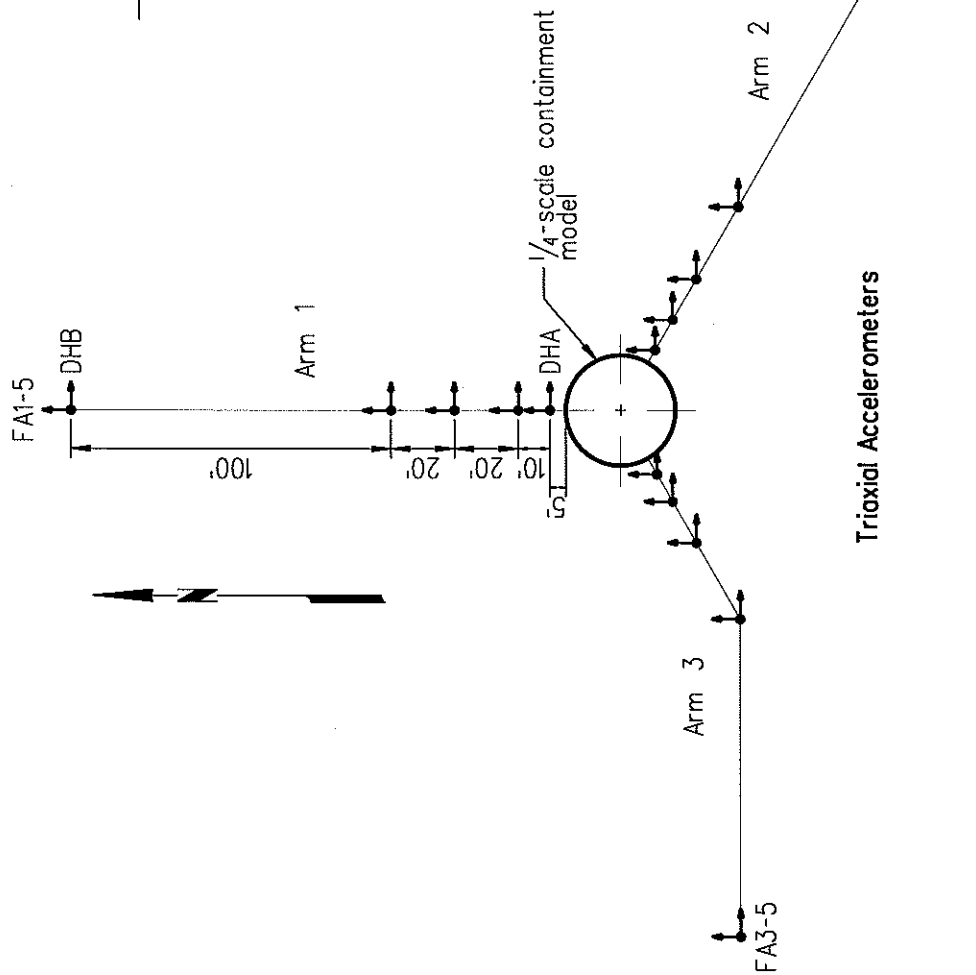
Data Sources

Bechtel Power Corporation (1991). “A synthesis of predictions and correlation studies of the Lotung soil-structure interaction experiment,” *Rpt. No. EPRI NP-7307-M*, Electrical Power Research Institute, October.

Geomatrix Consultants (1991). “Analysis of ground response data at Lotung large-scale soil-structure interaction experiment site,” *Rpt. No. EPRI NP-7306-M*, Electrical Power Research Institute, December.



Downhole Instrument Arrays
(After Tang, 1987)



Triaxial Accelerometers

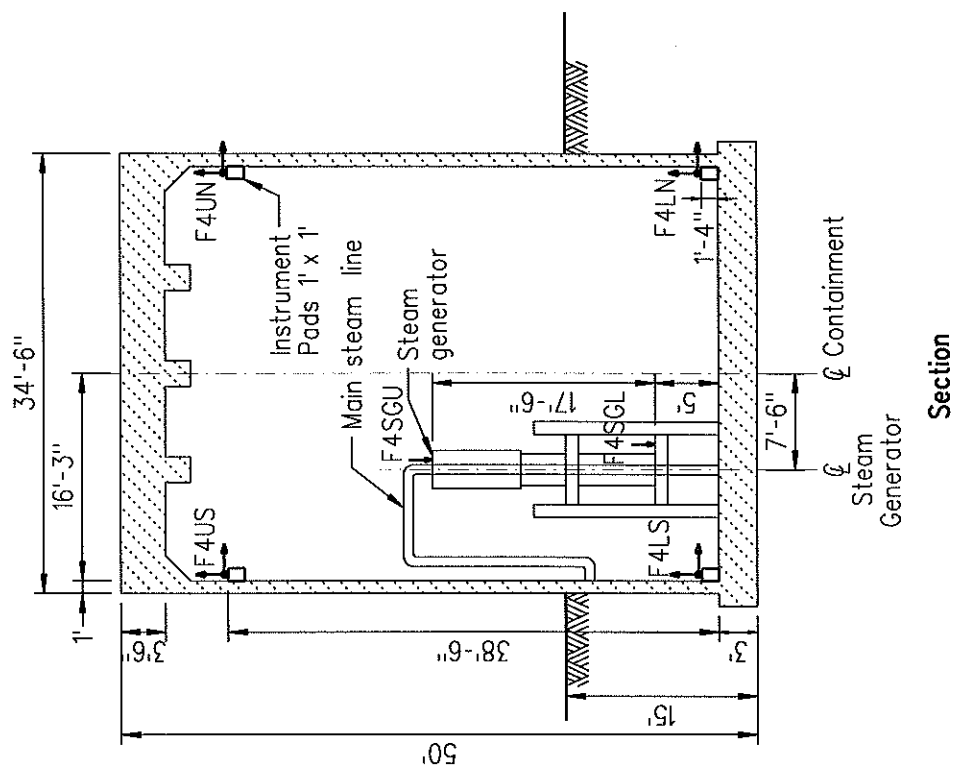
Surface and Downhole Instrumentation
Lotung Nuclear Containment Model
Lotung, Taiwan
Figure A46.1

Note: Data provided by Tang (1987)

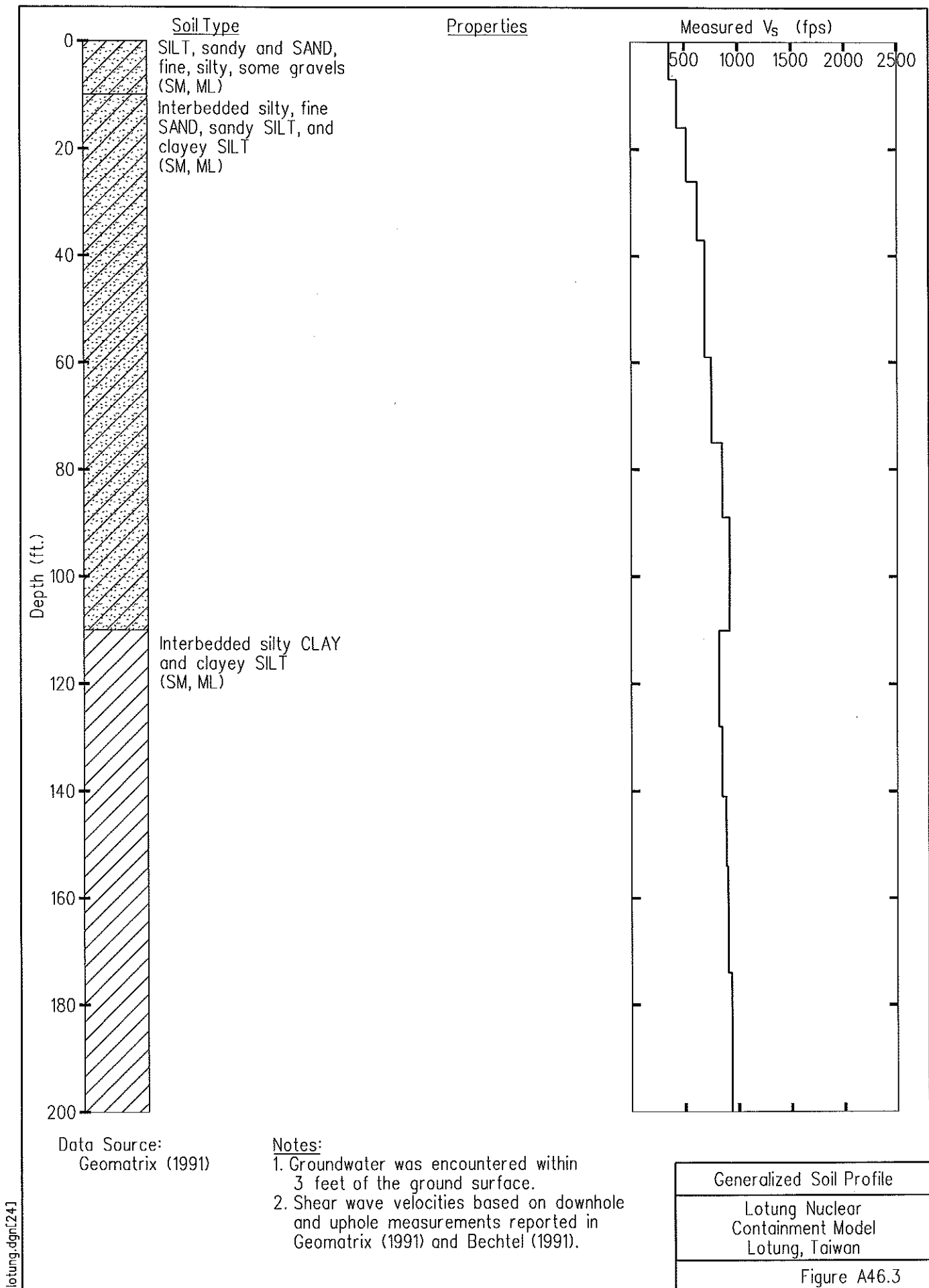
lotung.dgn(150)

Isometric Schematic

Isometric Schematic



Section



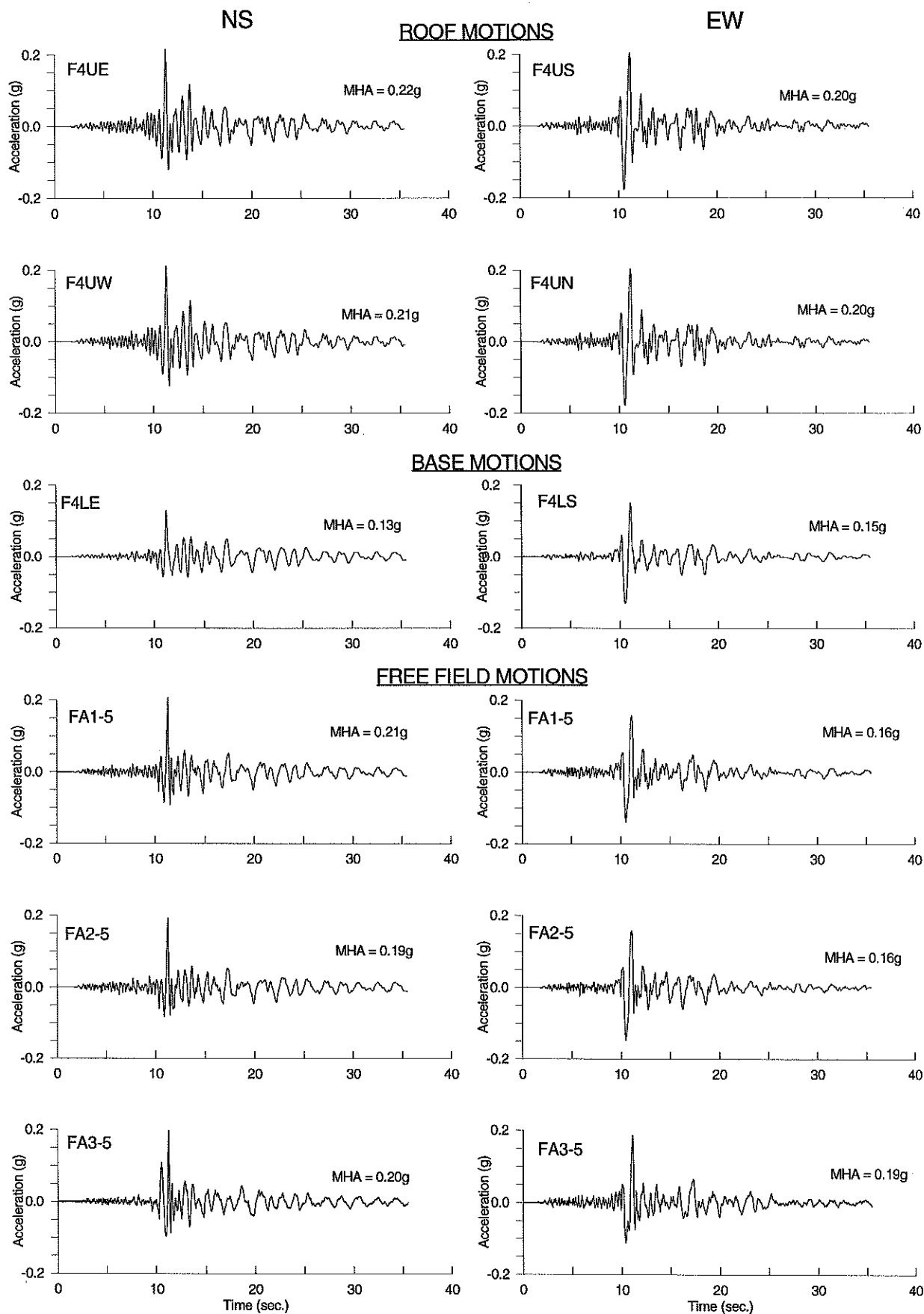


Fig. A46.07-1: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Horizontal Acceleration Time Histories
1986 Event LSST07
A46-9

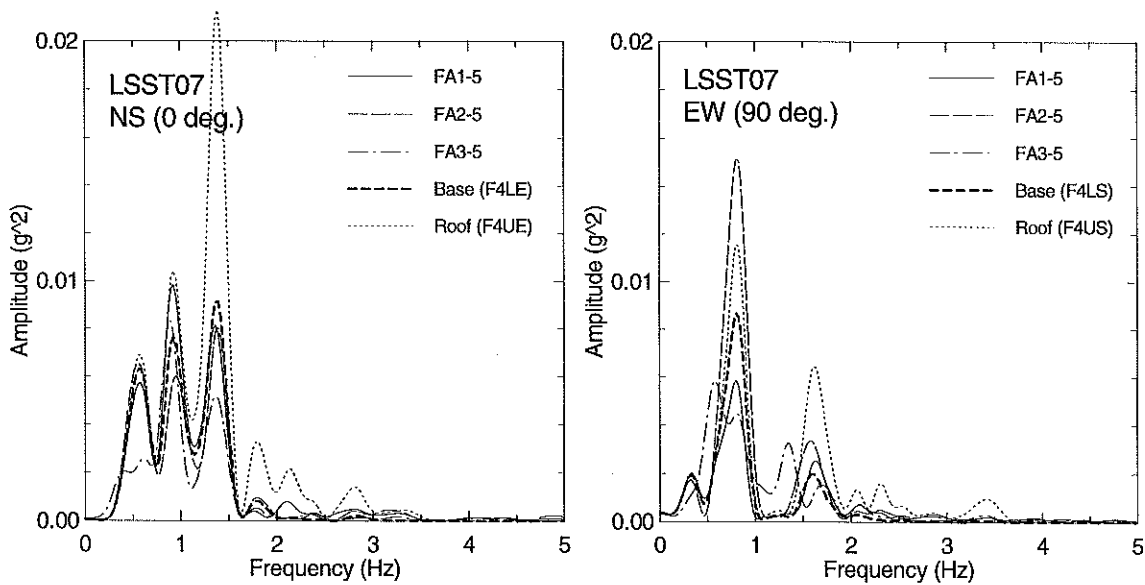


Fig. A46.07-2: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Power Spectral Density of Selected Horizontal Accelerations
1986 Event LSST07

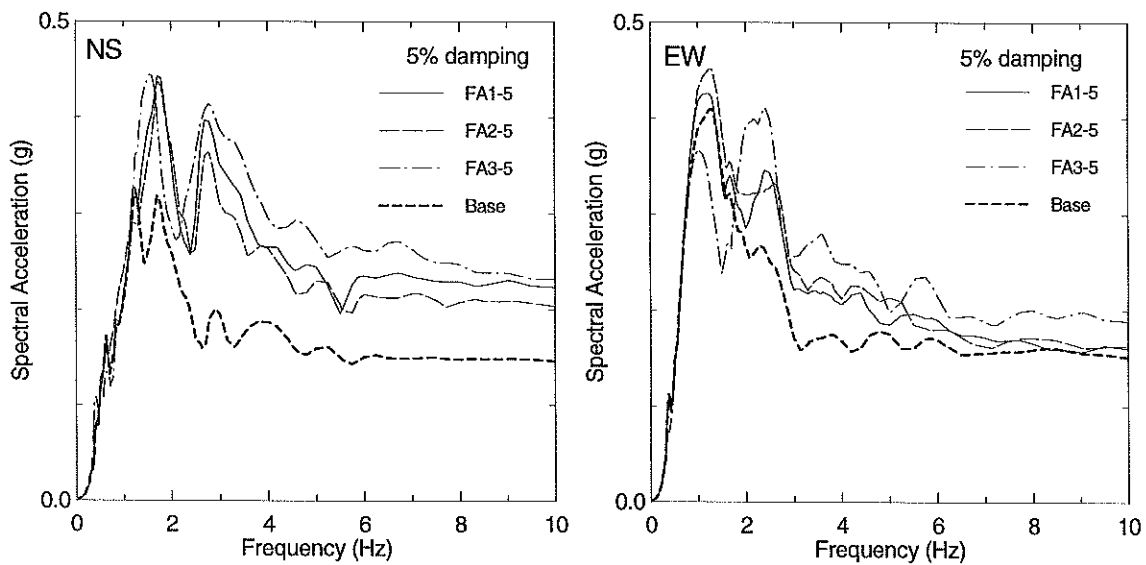


Fig. A46.07-3: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Acceleration Response Spectra of Basement and Free-Field Motions
1986 Event LSST07

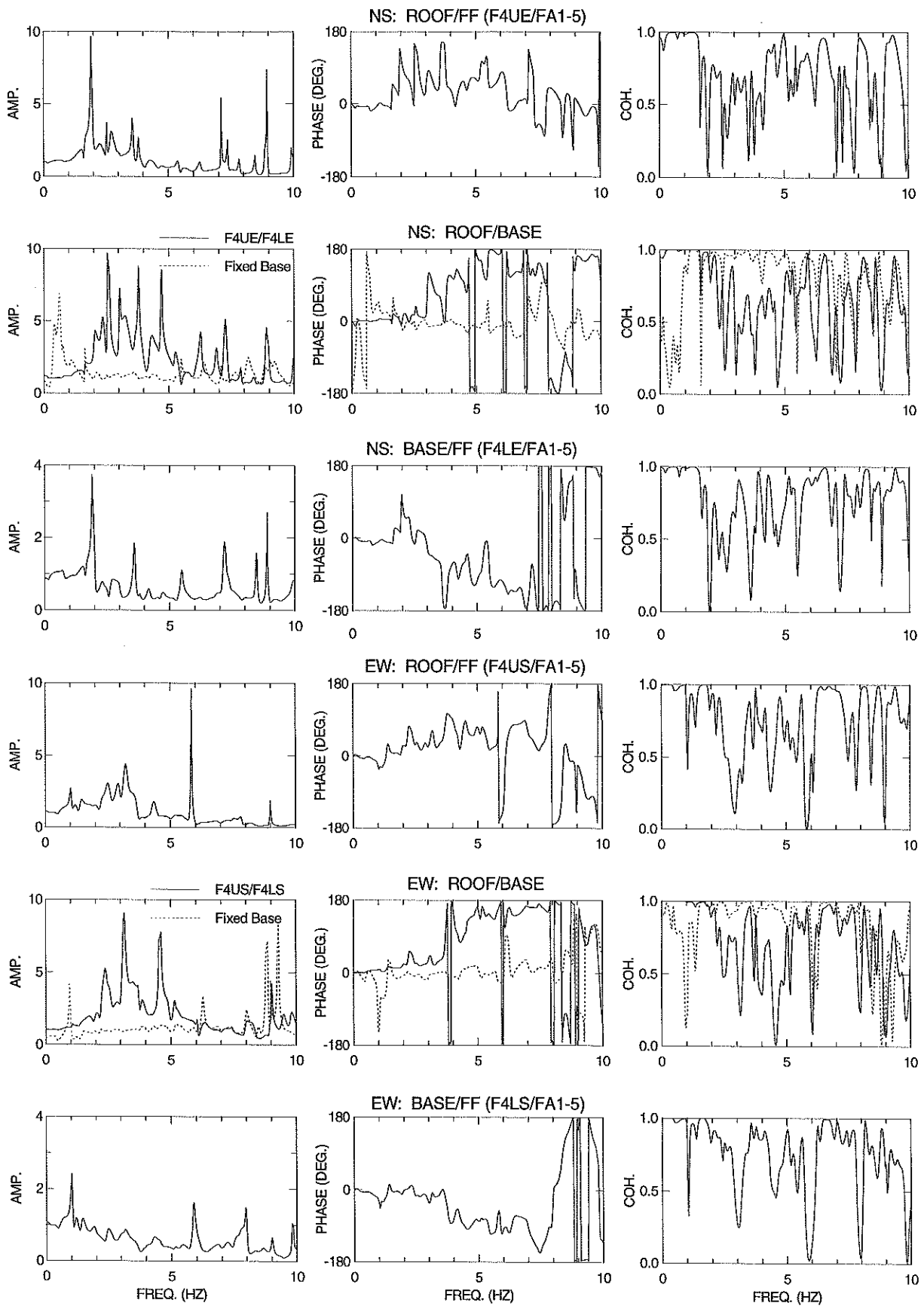


Fig. A46.07-4: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Transmissibility and Coherency Functions
1986 Event LSST07
A46-11

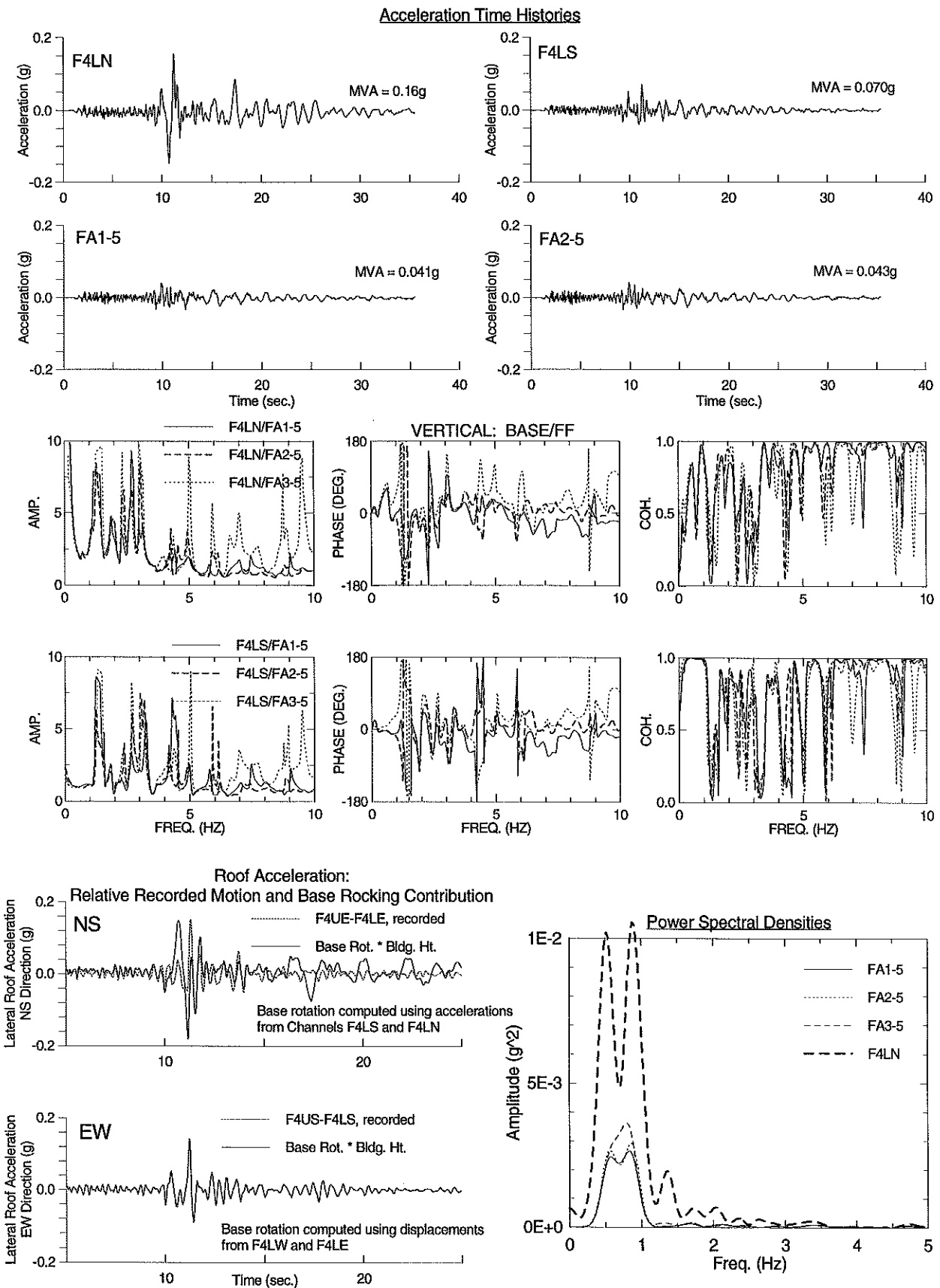


Fig. A46.07-5: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Analysis of Vertical Motions and Rocking Effects
1986 Event LSST07

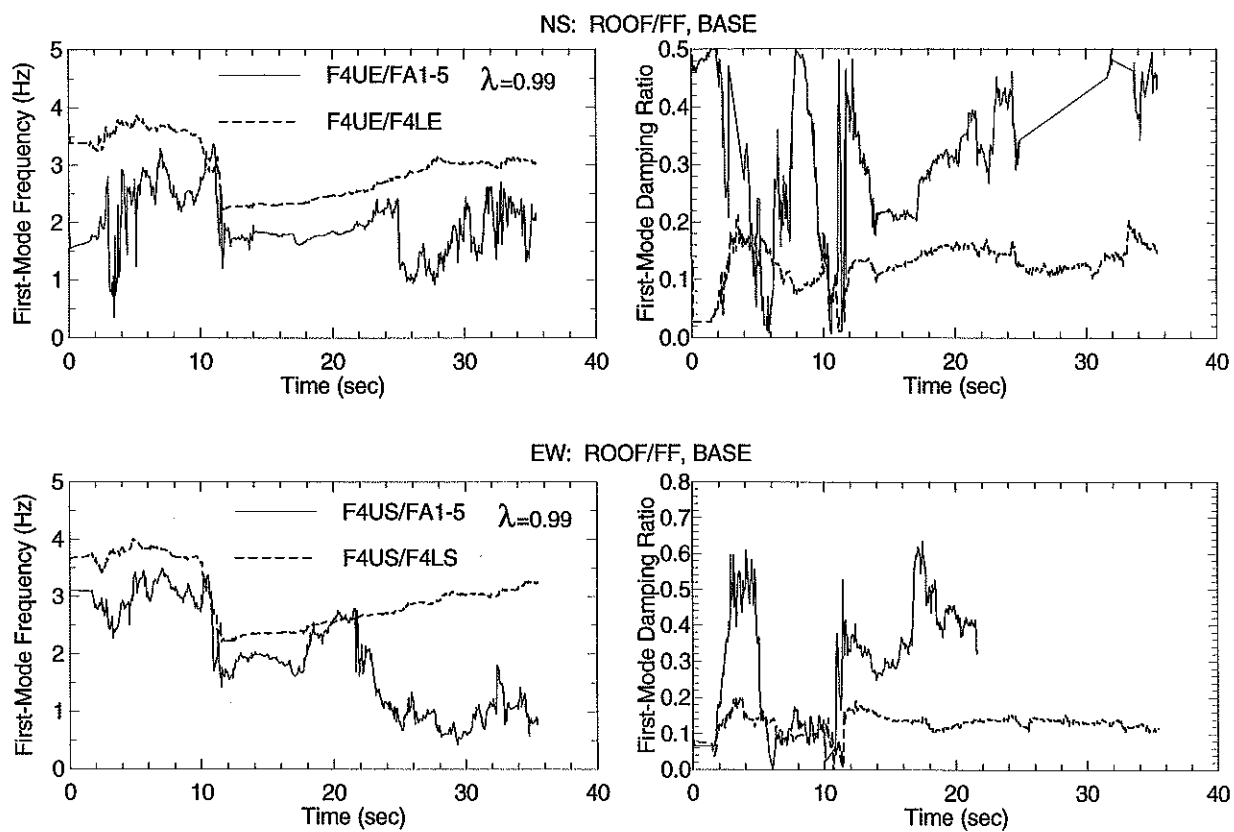


Fig. A46.07-6: LOTUNG 1/4-SCALE CONTAINMENT MODEL
Time Variation of First-Mode Parameters
1986 Event LSST07

SITE B1

Site Name: Milpitas 2 story Industrial Building
Sensors: CSMIP 57502

Lat: 37.430°
Long.: 121.897°

Floor Levels: 2
Bsmt. Levels: 0
Structure: perimeter concrete shear walls

Foundation: footings
Site Geology: soil
Design Date: 1984

Recorded Peak Accelerations (g)

Earthquake	Ground Floor			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.090	0.14	0.075	0.11	0.58	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.14\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$

Figures

B1.1-B1.2: Sensor Locations, Generalized Soil Column
B1.LP1-3 LP Analysis Results



View of structure, looking south-west

DISCUSSION

Site Description

The building is 2 stories tall above ground level with no basement and has a rectangular shape in plan view with dimensions 120 by 168 feet. As shown in Fig. B1.1, lateral loads are resisted by distributed 8-foot long reinforced concrete tilt-up shear walls along the building perimeter. Between the walls are gypsum board panels and windows. Interior 4-inch square columns only carry vertical loads. The 2nd floor is reinforced concrete and the roof is a plywood diaphragm. The foundation consists of 1.5-foot deep by 4-foot wide continuous spread footings around the building perimeter and 8-foot square by 2-foot deep spread footings beneath the interior columns. A 5-inch thick concrete slab is present across the ground floor. Most of the structural instrumentation is along the east and west sides, and ground floor vertical instruments are positioned to detect rocking at the base of a gypsum panel in the transverse (NS) direction.

Geotechnical exploration near the site by WCC (1987) revealed about 60 feet of stiff and plastic silty clays overlying dense sands and very stiff to hard silty clays which extend to the maximum depth explored of about 100 feet. Groundwater was encountered by WCC at a depth of about 6 feet. The shear wave velocities in Fig. B1.2 were estimated based on downhole measurements in similar geologic materials at a site about 7000 feet away by WCC (1994), and are in general agreement with velocity estimates for the site by Fumal (1991).

Instrument Selection and Free-Field Conditions

Instruments along the east side of the building were used in the analyses, as indicated by Table B1.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B1.LP1. The two vertical instruments located at the base of the panel on the ground floor level were used to evaluate rocking effects in the NS direction. It should be noted that the vertical instruments were positioned to measure base rocking of a gypsum panel, not a shear wall.

Table B1.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 9	Ch 13	Ch 1, 2	Ch 3	Ch 11

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS direction,

$$\text{input} = \text{Ch9} + (\text{Ch1} - \text{Ch2}) \cdot 31.5/16 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B1.LP1 by comparing relative roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are nearly in phase with the relative roof

displacements and contribute nearly 100% of the relative roof displacement amplitude, indicating that rocking significantly contributed to the roof motions.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B1.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B1.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B1.LP3).

Table B1.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flexible-Base			
		tr (NS) d=0, j=4		L (EW)		tr (NS) d=0, j=4		L (EW) d=2, j=4		tr (NS) estimated		L (EW)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	4.24	24.9	na	na	4.05	18.6	4.55	15.4	4.00	15.3 18.6	na	na
	2	8.03	29.6			8.22	24.3	8.08	14.2				

Comments on the analytical results follow:

- As indicated by the analyses for the NS direction, the structure is very stiff in the plane of the shear walls. This is illustrated in the R/GF transmissibility function (Ch3/Ch9) by amplitudes and coherencies close to unity and nearly zero phase shift from 0 to 7 Hz. The structural response was larger in the EW direction, as the roof sensor was oriented out of the plane of the shear walls.
- As indicated by the power spectral densities, the minor structural response that occurred in the transverse (NS) direction was dominated by the first mode. The transmissibility function amplitude peaks associated with this mode are broad, so the modal parameters in Table B1.2 are approximate.
- The minor difference between the flexible- and pseudo flexible-base frequencies indicates a minor SSI effect associated with base translation. Base rocking was more significant, as indicated by the modest difference between the first-mode frequencies for the fixed- and pseudo flexible-base cases. It should be noted, however, that these fixed-base parameters are for a gypsum panel, not a shear wall.
- The accuracy of the flexible-base damping estimate appears to be poor, as a number of 'A' sites with similar short, shear wall structures (e.g. A2, A7, A14, A17, and A34) have $\tilde{\zeta} - \tilde{\zeta}^*$ values of about +2 to 10 %. Since this difference was negative for this site, $\tilde{\zeta}$ was estimated by $\tilde{\zeta}^* = 18.6\%$.

The RPEM results (Fig. B1.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. For times between about 14 and 24 seconds, the GF-R and fixed-base first-mode frequency and damping were nearly identical, though differences similar to those in the CEM results are evident at earlier times.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B1.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 80 feet using the recorded ground floor motions from the east wall of the structure and the profile in Fig. B1.2. The 80-foot depth for deconvolution was based on the equivalent radius corresponding to the foundation's footprint area. The foundation radii used in the predictive analyses correspond to a 32-foot length of the footing beneath the instrumented gypsum panel and two adjacent wall panels. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping values were increased by a factor of 3.0 to account for shape effects.

Table B1.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=695$ fps	$h=26$ ft	emb.=0	$\gamma = 0.15$
LP	$v=0.45$		tr:	$r_1 = 6.4$ ft	$r_2 = 10.9$ ft
	$\beta^*=4.1\%$	$V_s^*=650$ fps	L:	$r_1 = --$	$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B1.3 with the fixed-base modal parameters in Table B1.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table B1.4.

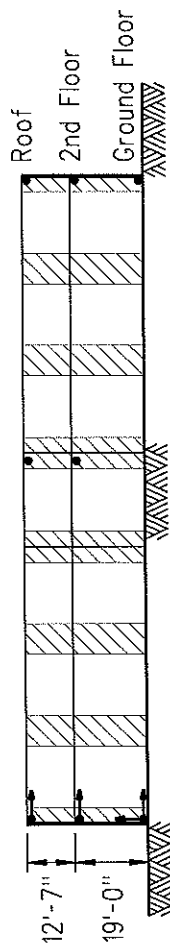
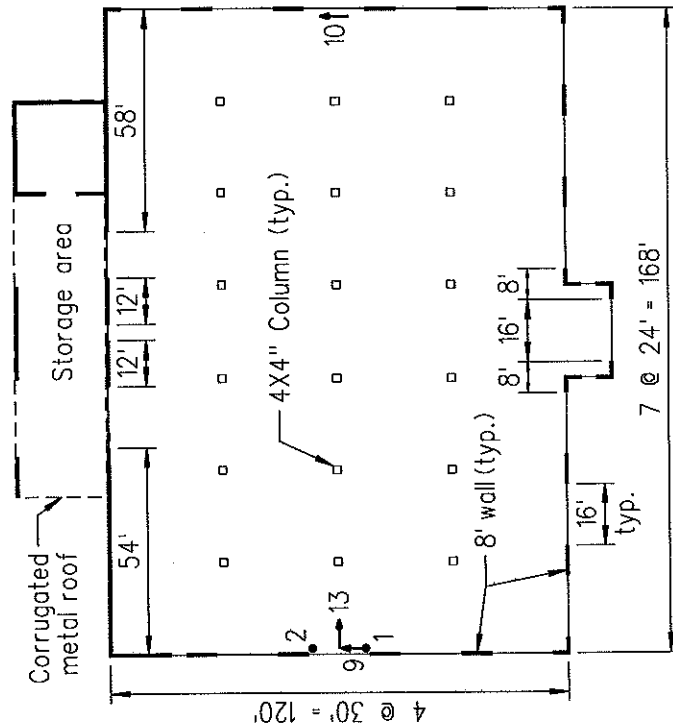
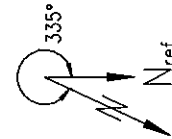
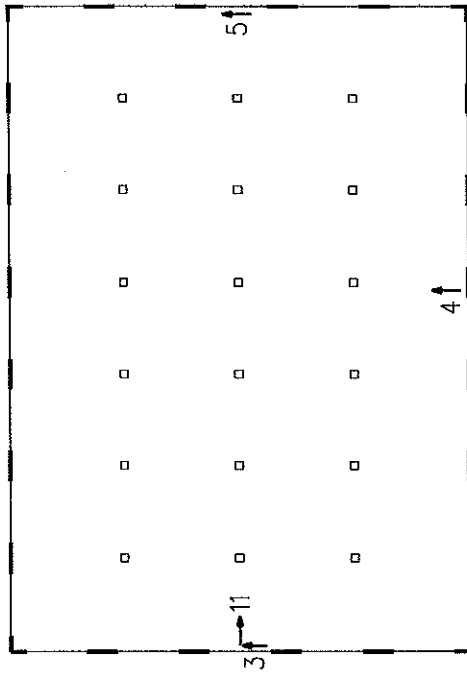
Table B1.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.06	-2.4	18.6	--	--	--
	Prediction	1.06	0.9	22.0	--	--	--

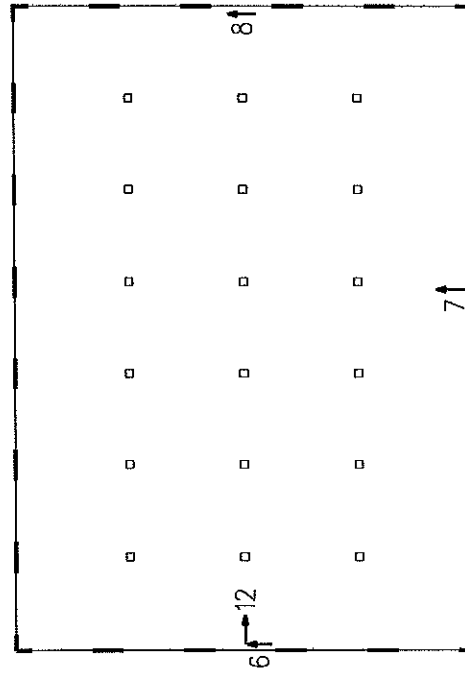
The negative foundation damping from the system identification analyses indicates a numerical error in these results. However, the apparently small foundation damping at the site was well predicted. The prediction of period lengthening was satisfactory.

Data Sources

- Fumal, T.E. (1991). "A compilation of the geology and measured and estimated shear-wave velocity profiles at strong motion stations that recorded the Loma Prieta, California, earthquake," *Rpt. No. OFR 91-311*, U.S. Geological Survey.
- Woodward-Clyde Consultants, WCC (1987). "Foundation investigation, Milpitas Water Tank, Milpitas, CA," *Proj. No. 16634-V*.
- Woodward-Clyde Consultants, WCC (1994). "Foundation investigation, Aerial Guideway Structure, Tasman Corridor LRT Project, Milpitas and San Jose, CA," Vol. 2, *Proj. No. 90C-0195V*.

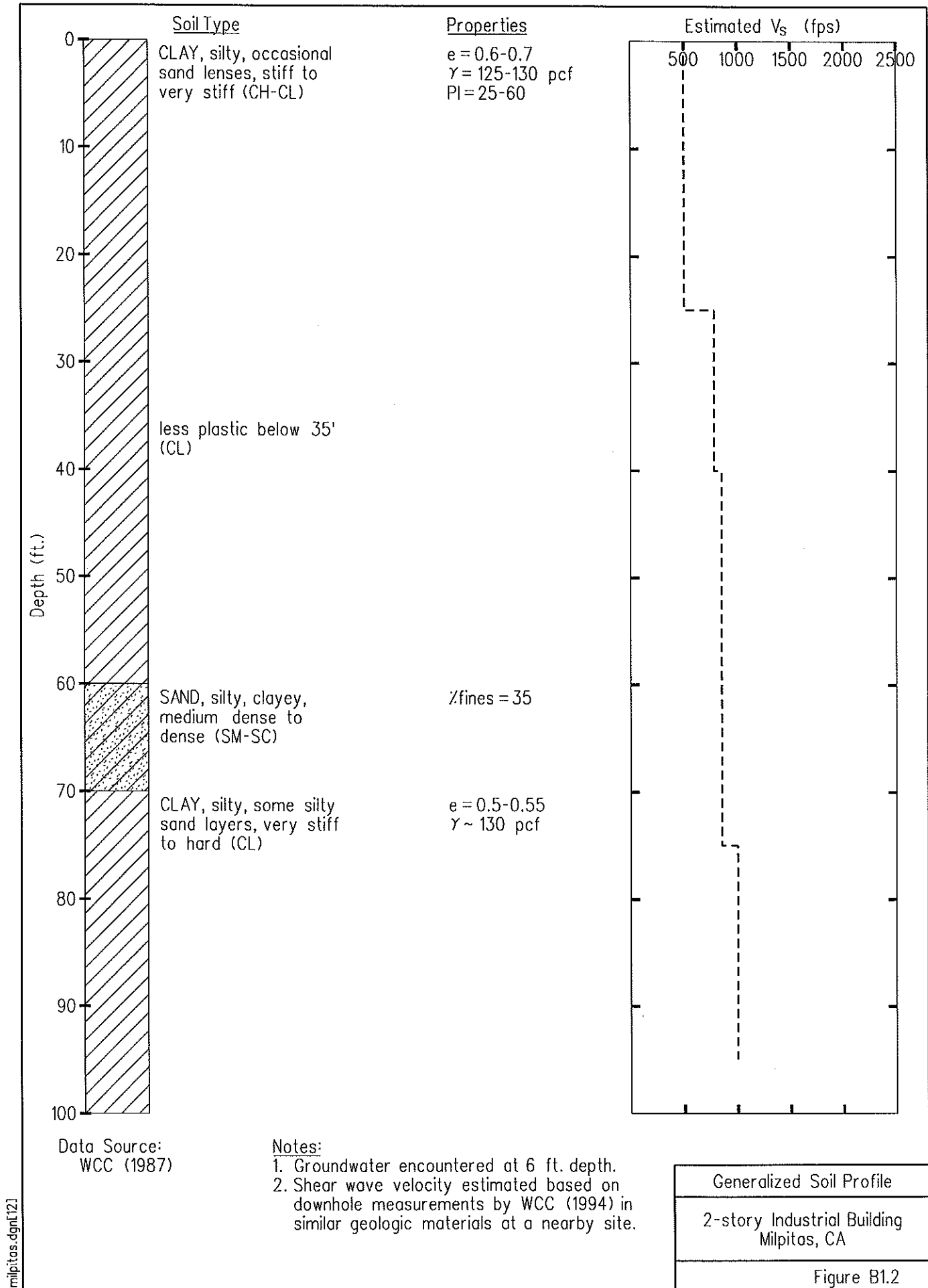


Roof Plan



2nd Floor Plan

Sensor Locations
2-Story Industrial Building Milpitas, CA
Figure B1.1



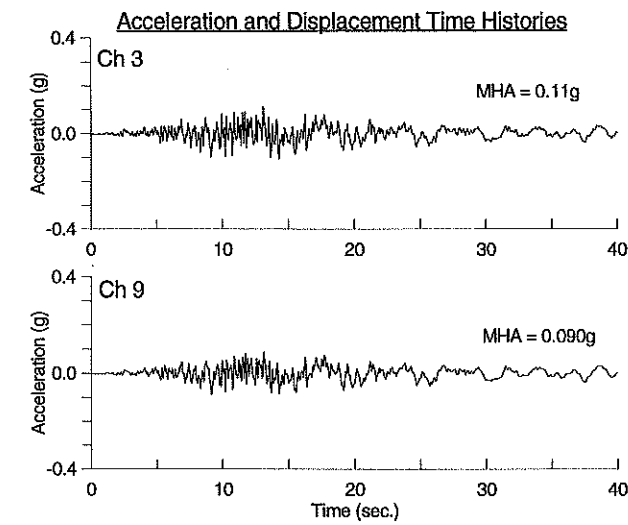
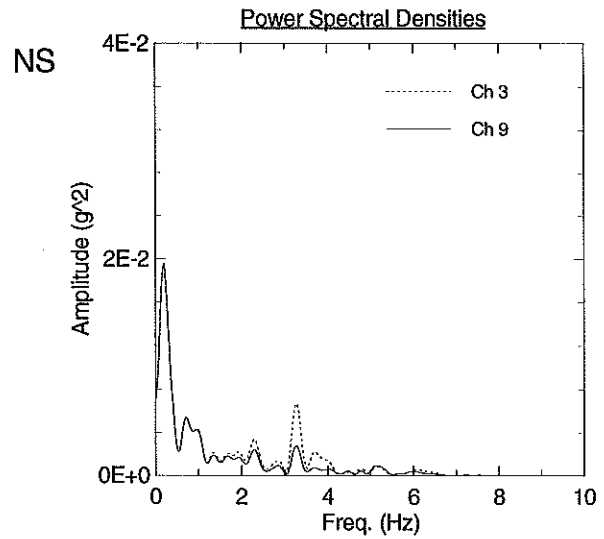
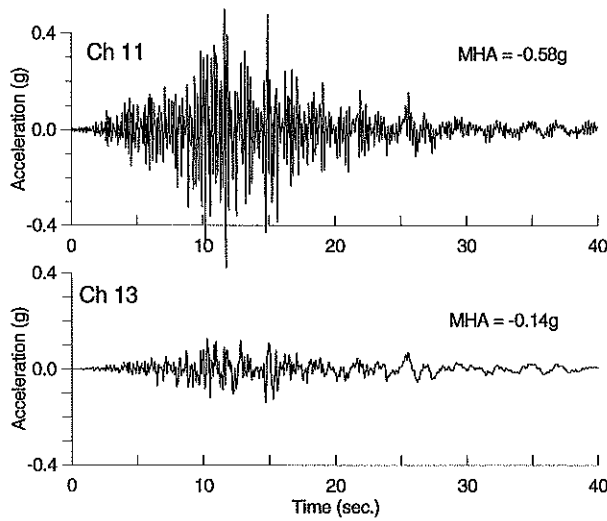
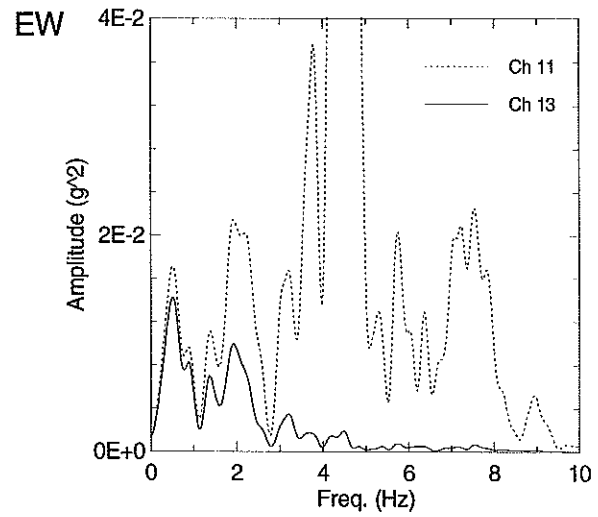
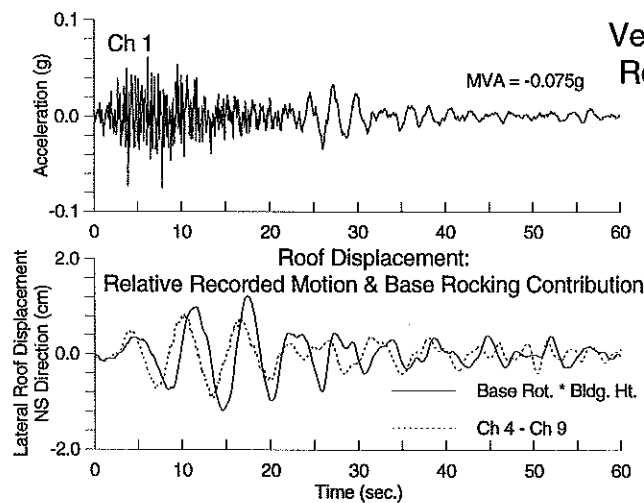
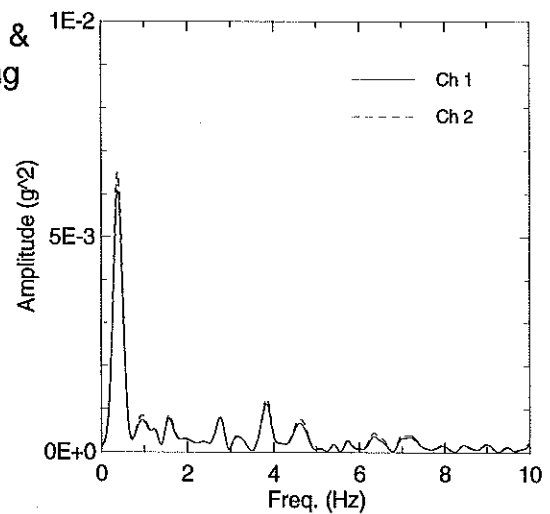


Fig. B1.LP1: MILPITAS 2 STORY INDUSTRIAL BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake
B1-7

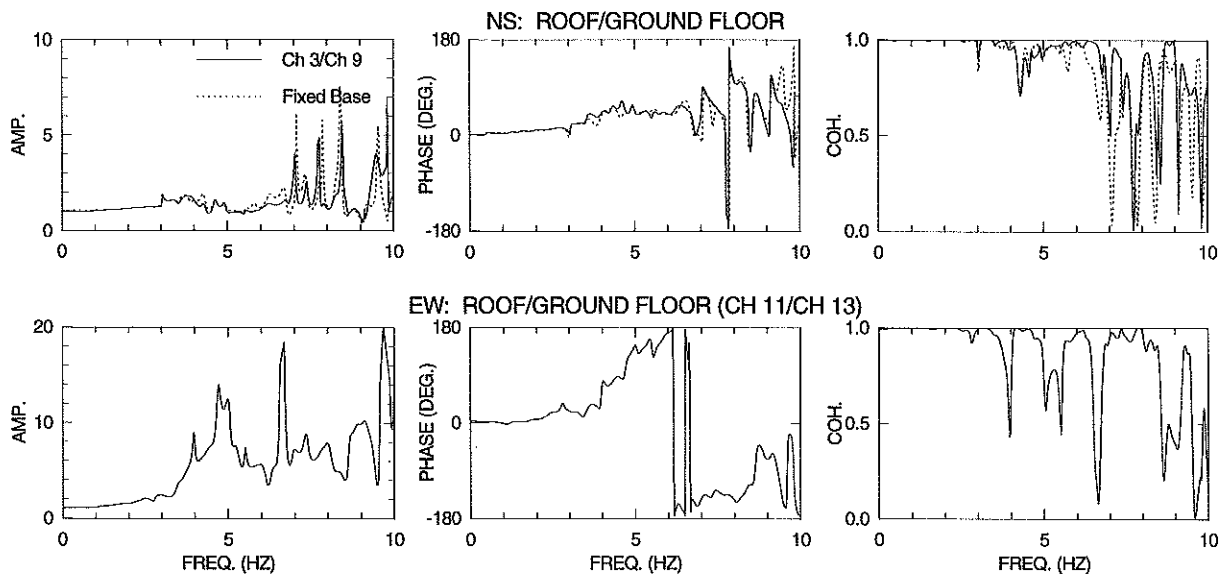


Fig. B1.LP2: MILPITAS 2 STORY INDUSTRIAL BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

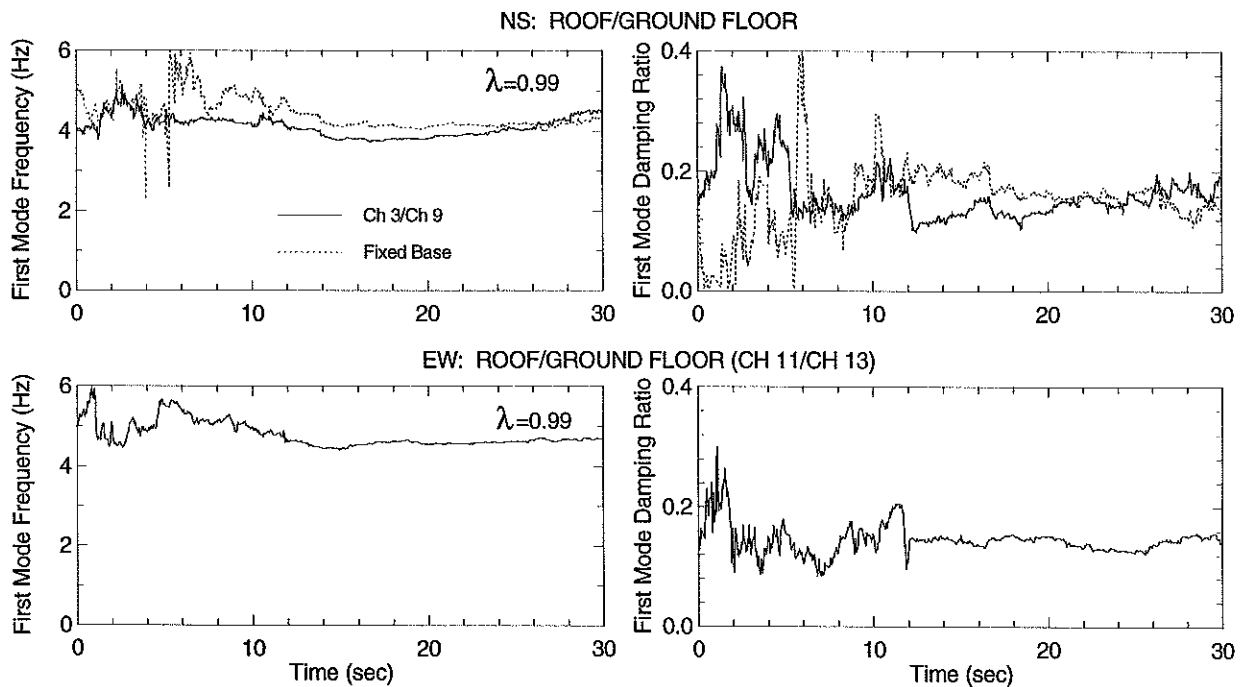


Fig. B1.LP3: MILPITAS 2 STORY INDUSTRIAL BUILDING
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE B2

Site Name: San Bruno 9 Story Gov't Offc. Bldg.
Sensors: CSMIP 58394

Lat: 37.627°
Long.: 122.424°

Floor Levels: 10
Bsmt. Levels: 0
Structure: center core concrete shear walls

Foundation: mat
Site Geology: soil
Design Date: 1972

Recorded Peak Accelerations (g)

Earthquake	Ground Floor			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.11	0.11	0.10	0.22	0.32	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.3\text{--}23\text{ Hz}$; $f_{\text{Nyq}} = 25\text{ Hz}$

Final: decimated by 2 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Figures

B2.1-B2.2: Sensor Locations, Generalized Soil Column
B2.LP1-3 LP Analysis Results



View of structure, looking in the reference south direction

DISCUSSION

Site Description

The building is 9 stories tall above ground level with no basement and has a rectangular shape in plan (84 by 192 feet). As shown in Fig. B2.1, lateral loads are resisted by reinforced concrete shear walls which form a central core. Columns outside of the central core are only intended to carry vertical loads. The foundation consists of a concrete mat which is 5 feet thick beneath the columns and walls and 2.5 feet thick in intermediate areas. Most of the accelerographs are located within the central core, and ground floor vertical instruments are positioned on either end of the west wall to detect rocking in the transverse (NS) direction.

Geologic exploration has been performed at the site by Evans (1971) and at a number of adjacent sites by other firms. As shown in Fig. B2.2, these studies encountered Colma Formation silty sands with occasional clayey and silty layers which extend to the maximum depth explored of about 60 feet. Shear wave velocities at the site were estimated based on downhole in situ measurements in Colma Formation soils at the San Francisco Airport (Gibbs et al., 1992) as well as correlations by Fumal (1978) and Seed et al. (1984).

Instrument Selection and Free-Field Conditions

Instruments in the central core of the building were used in the analyses, as indicated in Table B2.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B2.LP1. The two vertical instruments located at the ground floor level in the central core were used to evaluate rocking effects in the NS direction.

Table B2.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 9	Ch 14	Ch 1, 2	Ch 4	Ch 11

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS direction,

$$\text{input} = \text{Ch9} + (\text{Ch2} - \text{Ch1}) \cdot 104/18 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B2.LP1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are in phase with the relative roof displacements and

contribute significantly to the relative roof displacement amplitude, indicating that base rocking had a strong influence on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B2.LP2), (2) modal parameters from CEM analyses (Table B2.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B2.LP3).

In order to investigate the potential influence of foundation non-rigidity effects on estimated flexible-base parameters, first-mode parameter estimation procedures were invoked for four cases of foundation non-rigidity: (1) rigid foundation, (2) non-rigid foundation with corrections to foundation rocking impedance (both stiffness and damping), (3) non-rigid foundation with corrections to foundation rocking impedance for stiffness only, and (4) rigid foundation beneath core, but perfect flexibility outside of the core (i.e. only the core area is considered in calculating the foundation impedance). Flexible-base parameters for these various conditions are presented in Table B2.3.

Table B2.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flexible-Base			
		tr (NS) d=2, j=3		L (EW)		tr (NS) d=2, j=3		L (EW) d=2, j=5		tr (NS) estimated		L (EW)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	1.09	10.6	na	na	0.89	12.8	0.92	8.0	see	below	na	na
	2	3.93	10.5			3.96	10.5	3.4	13.1				
	3	7.74	9.1			8.22	13.5	4.7	13.5				

Table B2.3: Estimated flexible-base first-mode parameters

	$\tilde{f}$ (Hz)	$\tilde{\zeta}$ (%)
Rigid fndn.	0.82	32.2
Flexible fndn.	0.88	16.5
Mixed	0.88	12.7
Rigid core	0.89	13.3

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response for the transverse (NS) direction is dominated by the first mode.
- For the cases where foundation non-rigidity are accounted for, the difference between the flexible- and pseudo flexible-base frequencies are minor, indicating a only a minor SSI effect associated with base translation. Base rocking was more significant, as indicated by the relatively large difference between the first-mode frequencies for the fixed- and pseudo flexible-base cases and the results in Part (a).

The RPEM results (Fig. B2.LP3) indicate modest and gradual reductions (from about 1.3 to 0.9 Hz) in first-mode frequency and damping during roughly the first 20 seconds of strong shaking. The building is reported to have been damaged by the LP event, and was abandoned thereafter. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B2.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 72 feet using recorded ground floor motions from the central core of the structure and the profile in Fig. B2.2. No adjustments to the impedance function were made for embedment effects. Rocking radiation damping was increased by 60% to account for shape effects.

Table B2.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1005$ fps	$h=66$ ft	emb.=0	$\gamma = 0.15$
	$\nu=0.4$		tr (bldg)	$r_1 = 72$ ft	$r_2 = 59$ ft
			tr (core):	$r_1 = 21$ ft	$r_2 = 15$ ft
LP	$\beta^*=3.6\%$	$V_s^*=915$ fps	L:	$r_1 = --$	$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Due to the concentration of lateral force resisting elements within the central core of the structure, the effects of different assumptions of foundation rigidity were investigated. As discussed in Stewart (1996), the most critical case for foundation non-rigidity effects on foundation impedance is a rigid central core with a flexible slab extending beyond the core. This condition is not exactly duplicated in this structure due to the presence of vertical load bearing elements across the foundation. Hence, the suitability of theoretical corrections for foundation non-rigidity were investigated by repeating the predictive analyses for the four conditions listed in Part (b). Based on the theoretical solutions for non-rigid foundations by Iguchi and Luco (1982) discussed in Stewart (1996), the corrections for the stiffness and damping terms are 0.10 and 0.07, respectively (meaning that the non-rigid foundation has 10% of the rocking stiffness and 7% of the rocking damping of the rigid foundation).

Using the parameters in Table B2.4 with the fixed-base modal parameters in Table B2.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table B2.5.

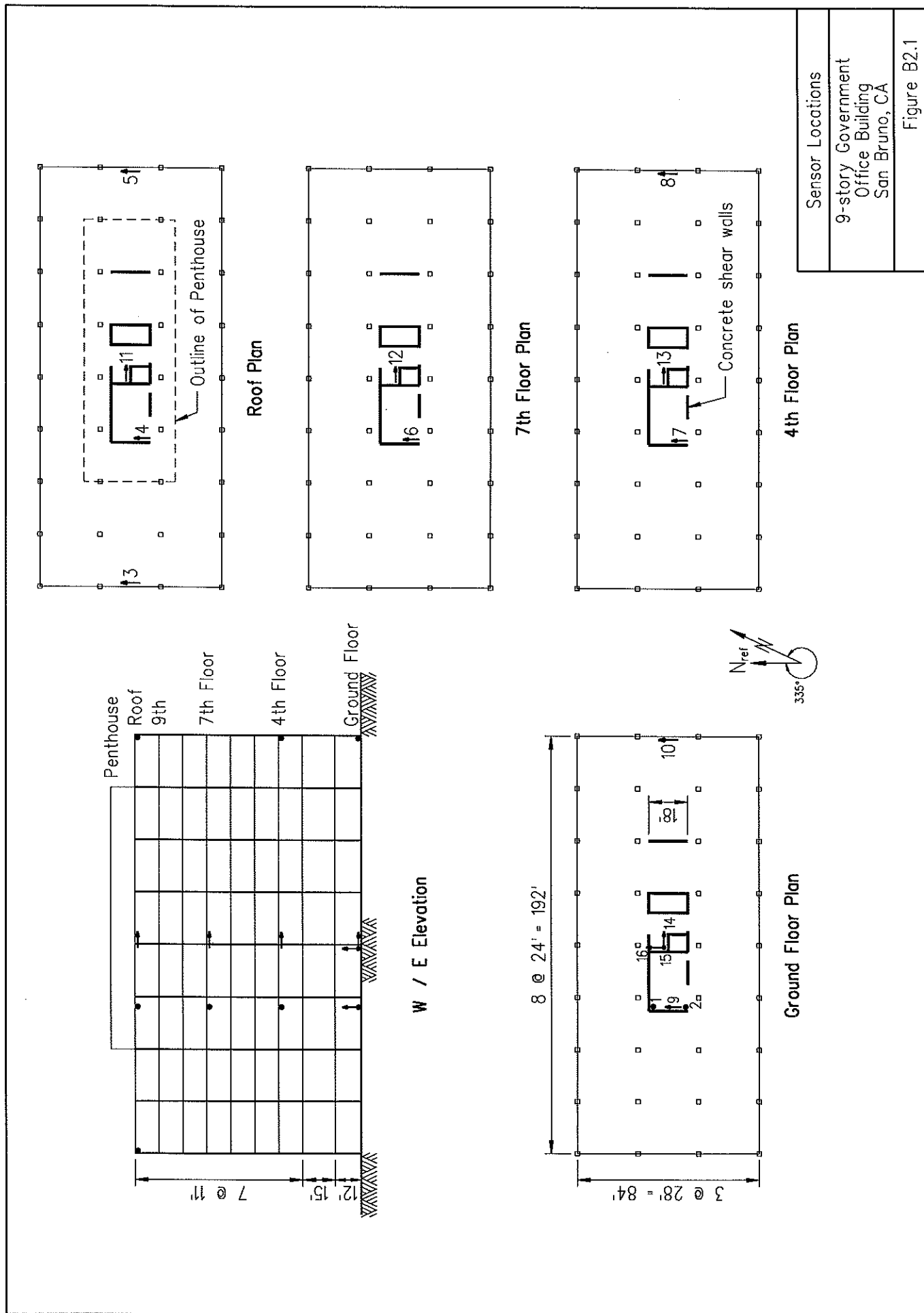
Table B2.5: Empirical and predicted inertial interaction effects

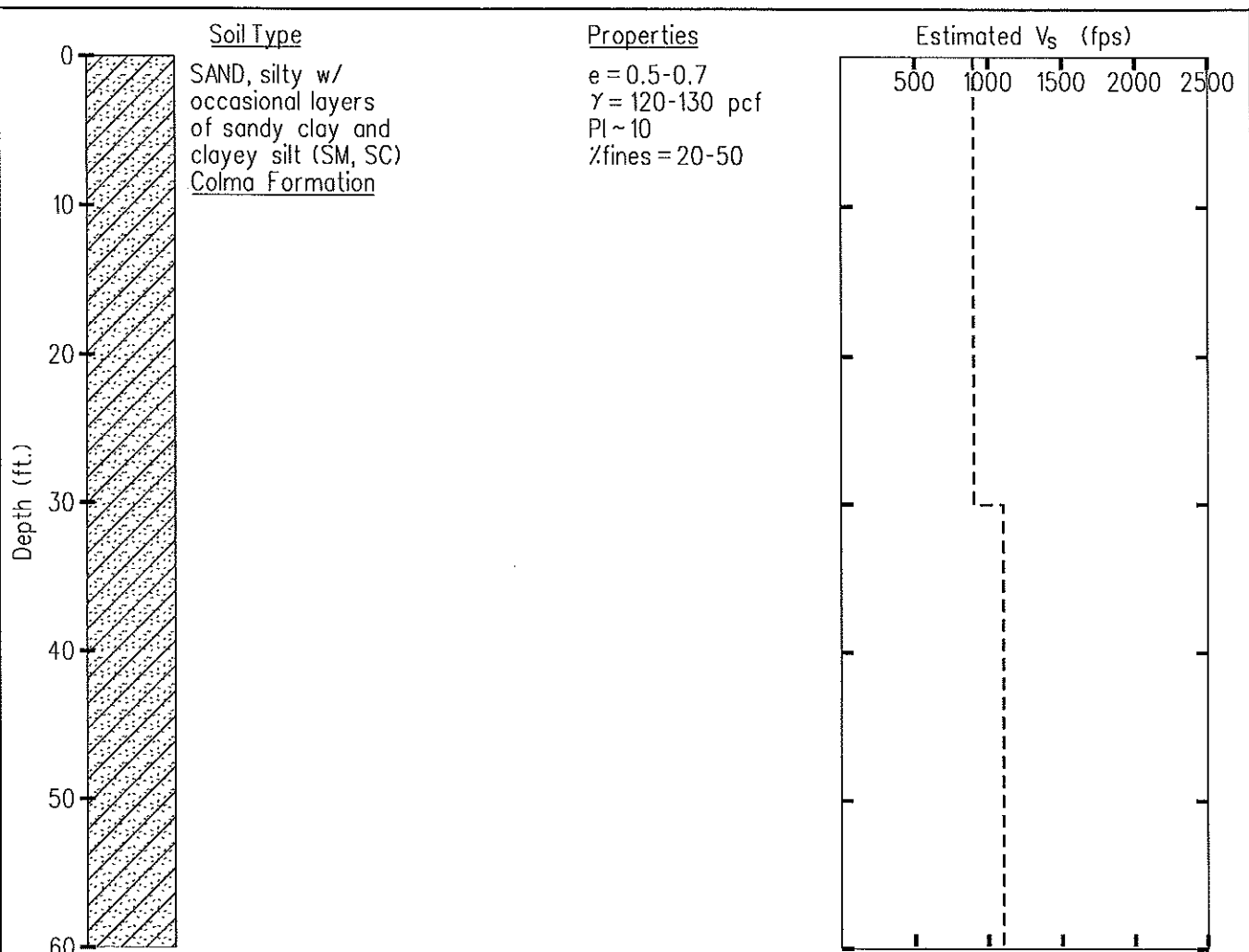
LP			tr (NS)		
			$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$
	1. Rigid fndn.	Sys. ID	1.34	18.8	32.2
		Prediction	1.04	0.5	10.0
	2. Flexible fndn.	Sys. ID	1.24	10.9	16.5
		Prediction	1.21	0.6	6.5
	3. Mixed	Sys. ID	1.24	7.1	12.7
		Prediction	1.21	4.2	10.1
	4. Rigid core	Sys. ID	1.23	7.7	13.3
		Prediction	1.11	0.2	7.9

It is clear from these results that the effects of foundation non-rigidity are significant for this structure, as the agreement between the empirical and predicted results are poor for the rigid foundation assumption (Condition 1) and for the case where infinitely flexible foundations outside of the core were assumed (Condition 4). When corrections for foundation non-rigidity are made to stiffness and damping components of rocking impedance (Condition 2), $\tilde{T} / T$ values are well predicted but $\tilde{\zeta}_0$ values are erroneous. The best results are achieved when corrections for foundation non-rigidity are only made for stiffness (Condition 3). This implies that the 2.5 to 5-foot thick foundation mat is unaffected by the vertical load bearing columns outside the core from the standpoint of rocking stiffness, but that the restraint on the foundation provided by these columns effectively eliminates any reduced damping effect that would result from foundation non-rigidity.

Data Sources

- Evans, L.T. (1971). unpublished boring logs
- Fumal, T.E. (1978). "Correlations between seismic wave velocities and physical properties of near-surface geologic materials in the Southern San Francisco Bay Region," *Open File Rpt. 78-1067*, U.S. Geological Survey.
- Gibbs, J.F. (1992). "Seismic velocities and geologic logs from borehole measurements at seven strong motion stations that recorded the 1989 Loma Prieta Earthquake," *Rpt. No. OFR 92-287*, U.S. Geological Survey.
- Iguchi, M. and Luco, J.E. (1982). "Vibration of flexible plate on viscoelastic medium," *J. Engrg. Mech.*, ASCE, 108(6), 1103-1120.
- Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K. (1984). "Moduli and damping factors for dynamic analyses of cohesionless soils," *Rpt. No. UCB/EERC-84/14*, Earthquake Engrg. Research Ctr., Univ. of California, Berkeley.
- Stewart, J.P. (1996). "An empirical assessment of soil-structure interaction effects on the seismic response of structures," *Ph.D. Dissertation*, University of California, Berkeley.





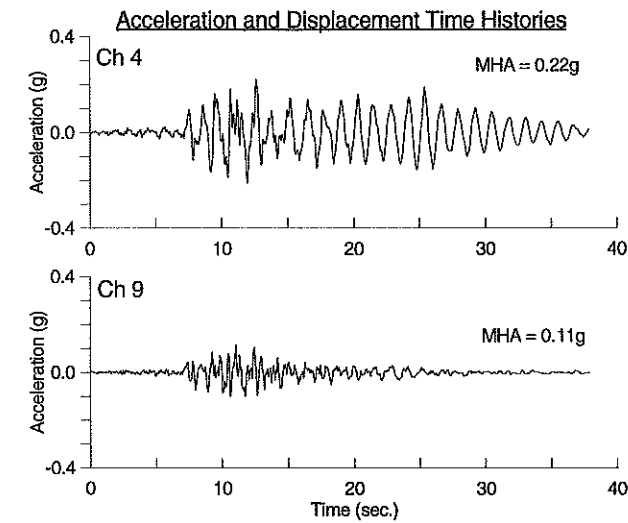
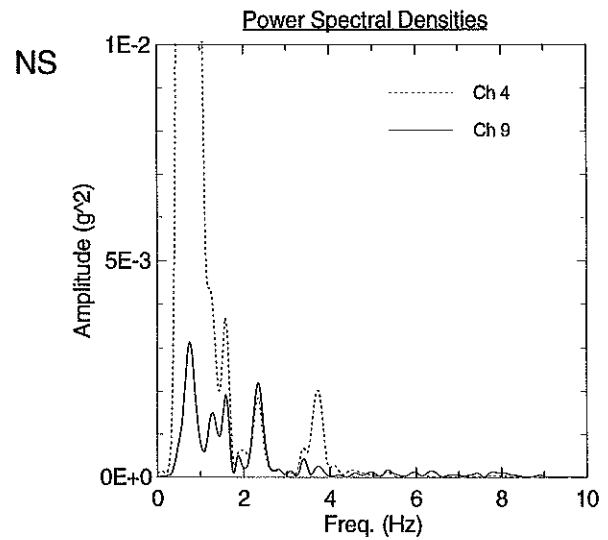
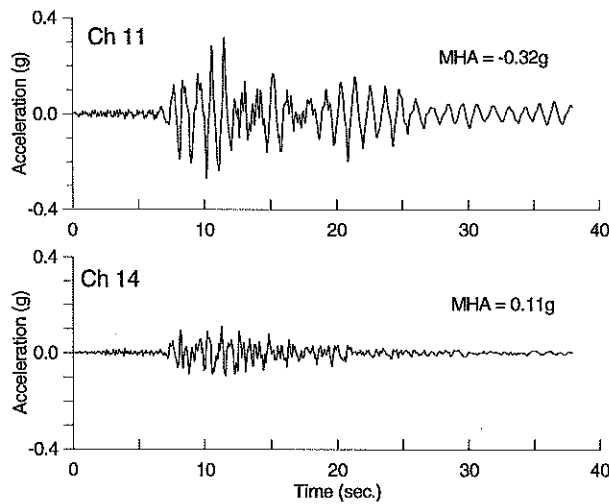
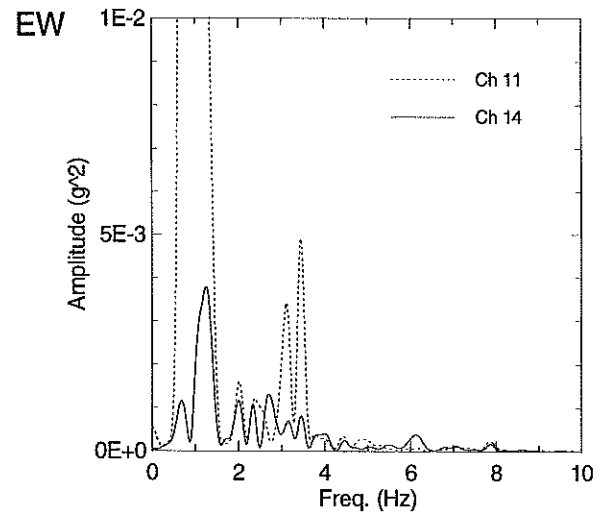
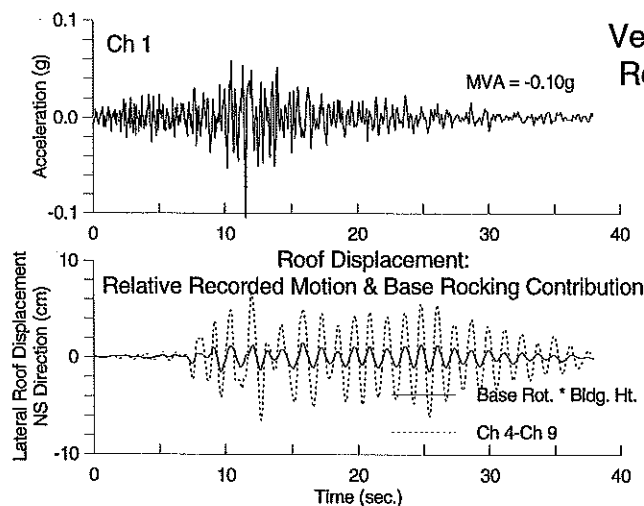
Data Source:
L.T. Evans (1971) and
numerous additional geotechnical
studies for sites in the vicinity

- Notes:**
1. Groundwater encountered at 50 ft. depth.
 2. Shear wave velocities based on downhole in situ measurements in similar geologic materials at a nearby site (SF Airport, Gibbs et al., 1992) and correlations by Fumal (1978) and Seed et al. (1984).

Generalized Soil Profile

9-story Government
Office Building
San Bruno, CA

Figure B2.2



Vertical & Rocking

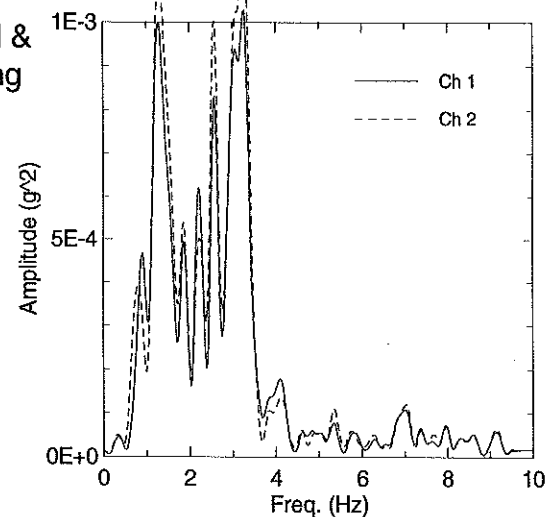


Fig. B2.LP1: SAN BRUNO 9 STORY GOVERNMENT OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake
B2-8

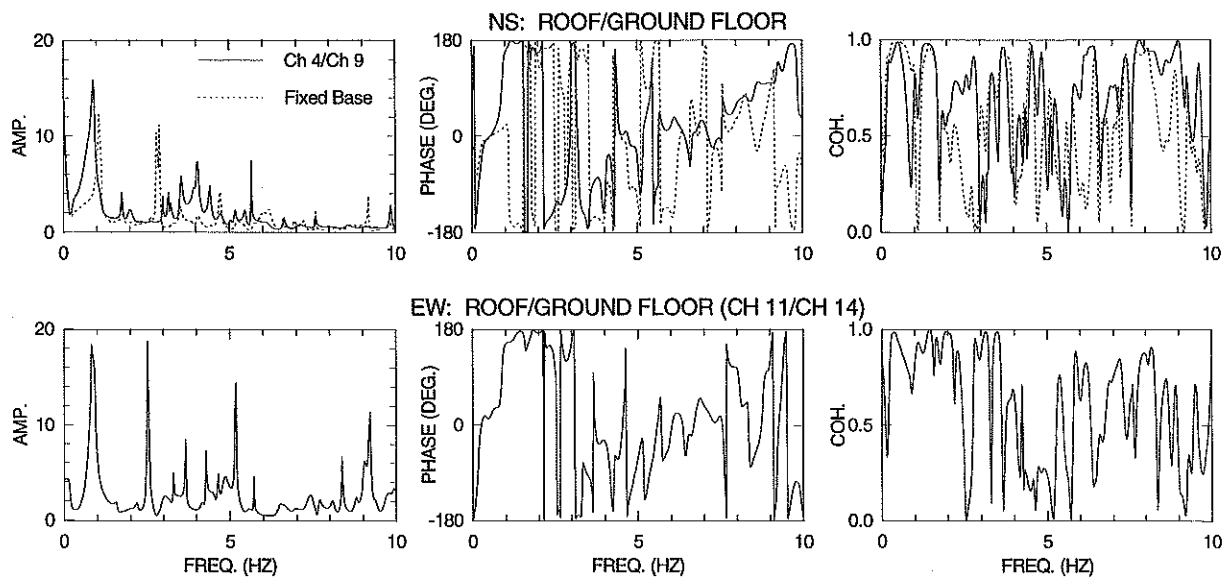


Fig. B2.LP2: SAN BRUNO 9 STORY GOVERNMENT OFFICE BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

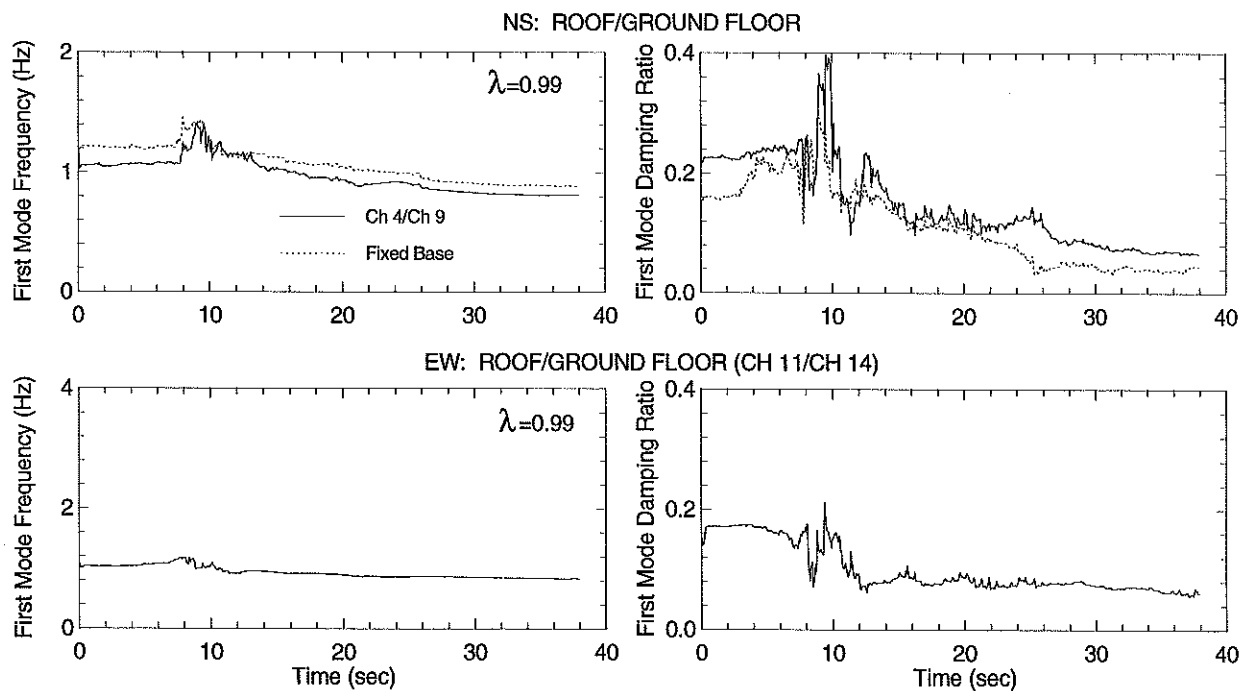


Fig. B2.LP3: SAN BRUNO 9 STORY GOVERNMENT OFFICE BUILDING
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE B3

Site Name: San Francisco 47 Story Offc. Bldg.
Sensors: CSMIP 58532

Lat: 37.796°
Long.: 122.396°

Floor Levels: 49
Bsmt. Levels: 2
Structure: NS: Steel braced frame
EW: Moment resisting frame

Foundation: mat/piles
Site Geology: soil
Design Date: 1978

Recorded Peak Accelerations (g)

Earthquake	'B' Level			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.11	0.16	0.04	0.47	0.40	--

-- = no instrument

Preprocessing

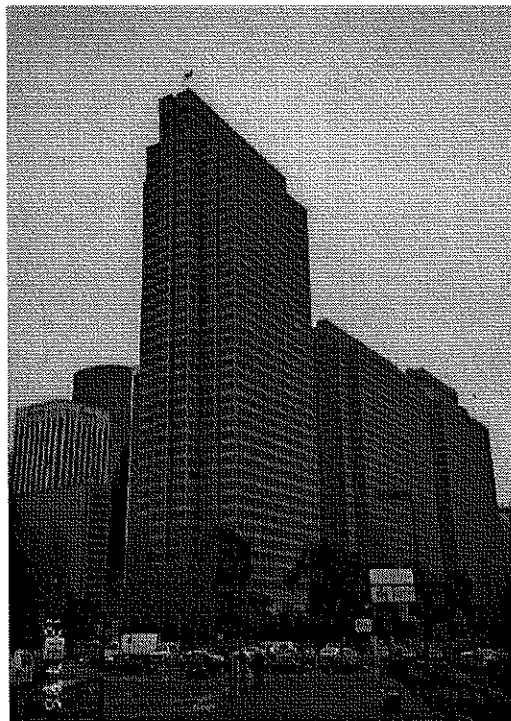
Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.14\text{--}23\text{ Hz}$; $f_{\text{Nyq}} = 25\text{ Hz}$

Final: decimated by 2 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Figures

B3.1-B3.2: Sensor Locations, Generalized Soil Column
B3.LP1-3 LP Analysis Results



View of structure, looking south-west

DISCUSSION

Site Description

The building is 47 stories tall above ground level with a 2-level basement and has a nearly rectangular shape in plan (122 by 180 feet). As shown in Fig. B3.1, lateral loads are resisted by steel eccentrically braced frames in the transverse (NS) direction and steel moment resisting frames in the longitudinal (EW) direction. The foundation consists of a 5-foot thick reinforced concrete mat supported by 150 steel pipe piles (200 feet long) infilled with concrete. Structural sensors are present on 6 levels of the structure. 'B' Level vertical instruments are positioned on the north and south sides of the building to detect rocking in the transverse (NS) direction.

As shown in Fig. B3.2, soil conditions at the site consist of about 28 feet of sandy fill, 83 feet of soft silty clay (Bay Mud), 36 feet of silty sand, and 67 feet of older (Pleistocene) bay clays which extend to graywacke and sandstone bedrock at a depth of about 214 feet (Gibbs et al., 1994). Based on the location of the site near the San Francisco Bay, the groundwater table is assumed to be located above the foundation level. The shear wave velocities shown in Fig. B3.2 are based on downhole measurements in the nearby Embarcadero Plaza by Gibbs et al. (1994).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B3.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B3.LP1. The two vertical instruments located at the 'B' level were used to evaluate rocking effects in the NS direction.

Table B3.1: Sensors in structure used for analyses

'B' Level (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 4	Ch 5	Ch 1, 2	Ch 17	Ch 16

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS direction,

$$\text{input} = \text{Ch4} + (\text{Ch2} - \text{Ch1}) \cdot 591/120 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B3.LP1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are in phase with the relative roof displacements but have a much smaller amplitude, indicating that base rocking had a small influence on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B3.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B3.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B3.LP3).

Table B3.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flexible-Base			
		tr (NS) d=17, j=37		L (EW)		tr (NS) d=28, j=37		L (EW) d=36, j=39		tr (NS) estimated		L (EW)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	0.20	4.3	na	na	0.19	5.7	0.16	4.1	0.19	6.1	na	na
	2	0.58	3.1			0.57	3.5	0.46	3.1				
	3	0.99	1.7			0.98	1.8	0.77	3.3				
	4	1.14	10.5			1.19	12.4	1.07	2.9				
	5	1.36	5.2			1.33	5.3	1.40	3.9				
	6	1.78	1.2			1.73	2.5	1.66	5.2				

Comments on the analytical results follow:

- The power spectra indicate that the structural response in both directions is dominated by energy at about $f \approx 0.8$ to 1 Hz. These frequencies are close to the third-mode frequency, but also to the predominant frequency of the base acceleration; hence, a resonant condition appears to have developed in the third-mode. Strong responses in the first-mode also occurred in both directions.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.
- The modal frequencies listed above for the pseudo flexible-base case are consistent with those identified by Chen et al. (1992).

The RPEM results (Fig. B3.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B3.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 122 feet using 'B' Level motions near the west wall of the structure and the profile in Fig. B3.2. The tower was assumed to be independent of the adjacent low-level structure for the calculation of foundation radii. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B3.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 570$ fps	$h = 414$ ft	emb. = 27 ft	$\gamma = 0.15$
LP	$\nu = 0.45$		tr:	$r_1 = 86$ ft	$r_2 = 80$ ft
	$\beta^* = 4.3\%$	$V_s^* = 480$ fps	L:	$r_1 = --$	$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B3.3 with the fixed-base modal parameters in Table B3.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table B3.4.

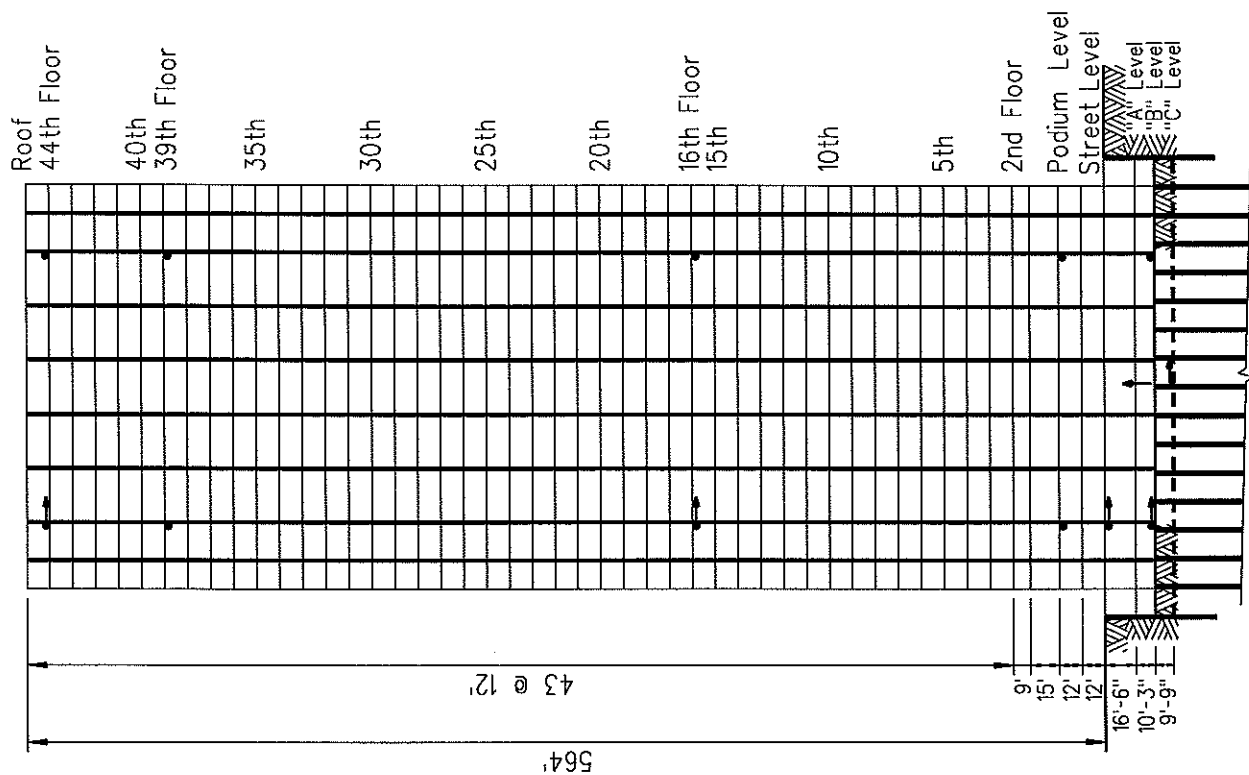
Table B3.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.03	2.3	6.1	--	--	--
	Pre.-Vel.	1.20	0.4	2.9	--	--	--
	Pre.-Bie.	1.24	1.7	3.9	--	--	--

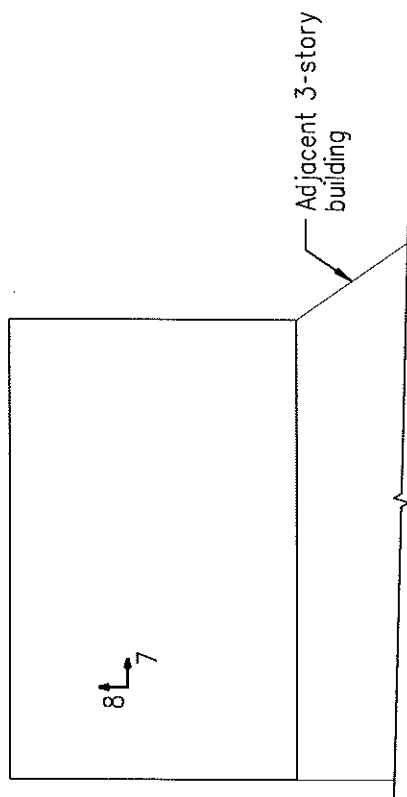
The period lengthening was significantly overpredicted and the foundation damping slightly underpredicted for this structure. This result may be associated in part with the influence of the pile foundations, which could increase the rocking stiffness and the radiation damping relative to the shallow foundation case assumed in the predictive analyses.

Data Sources

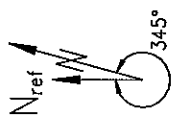
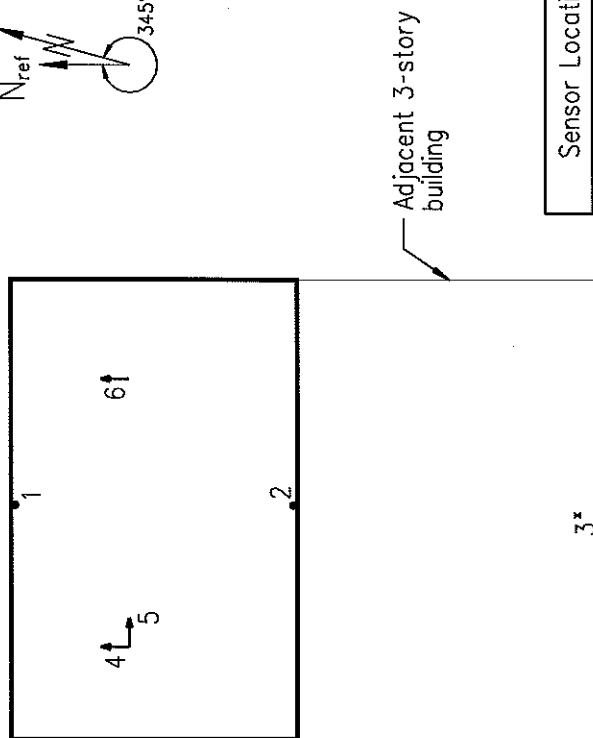
- Chen-Chih, C., Bonowitz, D., and Astaneh, A. (1992). “Studies of a 49 story instrumented steel structure shaken during the Loma Prieta Earthquake,” *Rpt. No. UCB/EERC-92/01*, Earthquake Engineering Research Center, University of California, May.
- Gibbs, J.F., Fumal, T.E., Borchardt, R.D., Warrick, R.E., Liu, H.-P., and Westerlund, R.E. (1994). “Seismic velocities and geologic logs from boreholes at three downhole arrays in San Francisco, California,” *Rpt. No. OFR 94-706*, U.S. Geological Survey.



W / E Elevation



Street Level Plan



"B" Level Plan

* (At "C" Level, below adjacent bldg.)

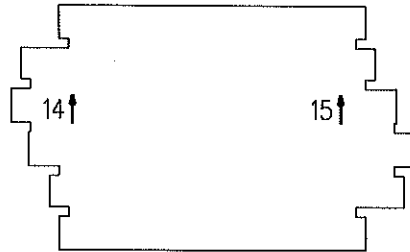
Sensor Locations (1/2)

47-story Office Building
San Francisco, CA

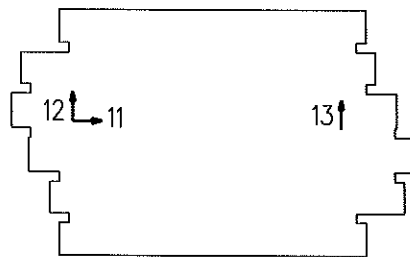
Figure B3.1(a)



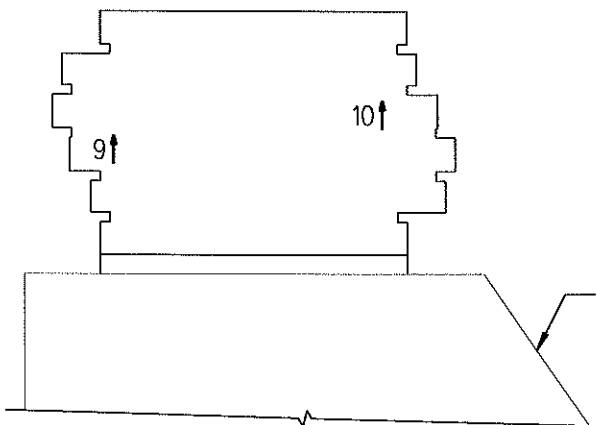
44th Floor Plan



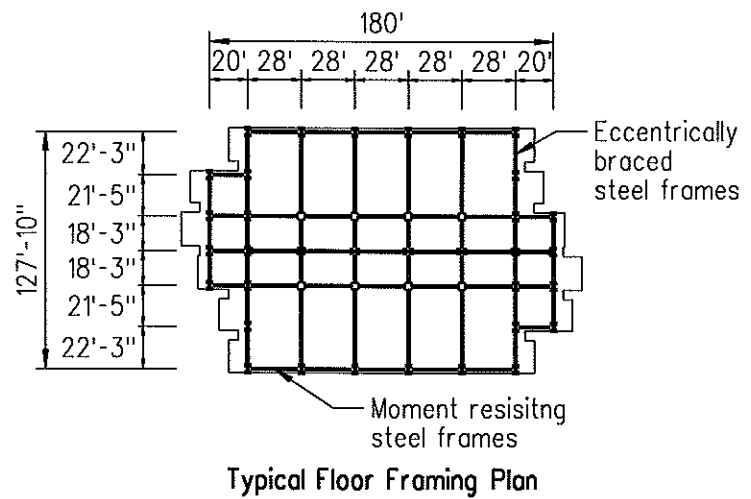
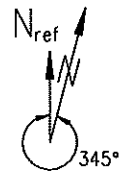
39th Floor Plan



16th Floor Plan



Podium Level Plan

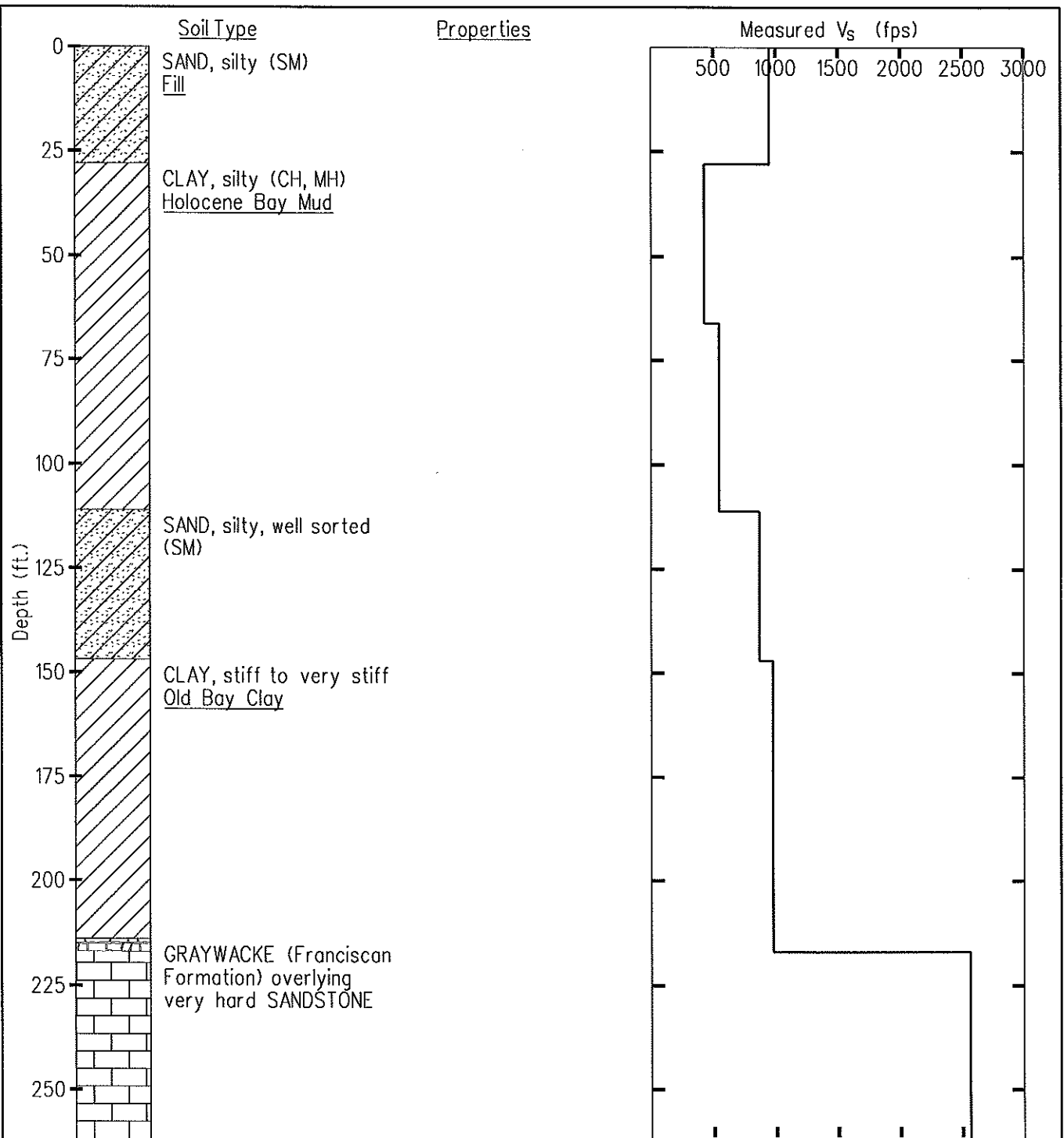


Adjacent 3-story building

Sensor Locations (2/2)

47-story Office Building
San Francisco, CA

Figure B3.1(b)



Data Source:
Gibbs et al. (1994)

Notes:

1. No groundwater data available.
2. Shear wave velocities based on downhole measurements by Gibbs et al. (1994).

Generalized Soil Profile

47-story Office Building
San Francisco, CA

Figure B3.2

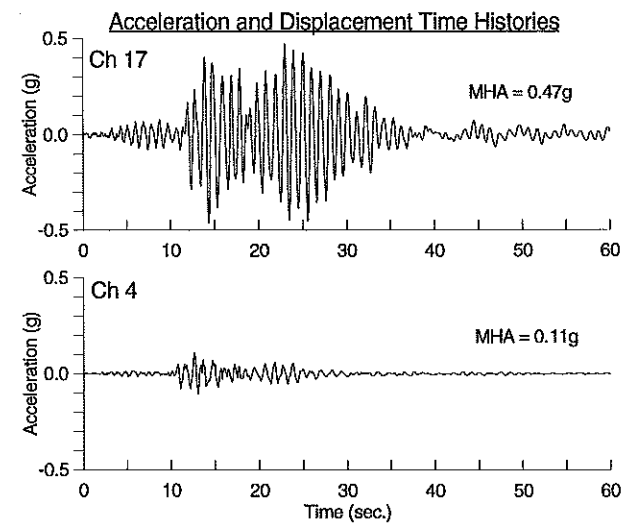
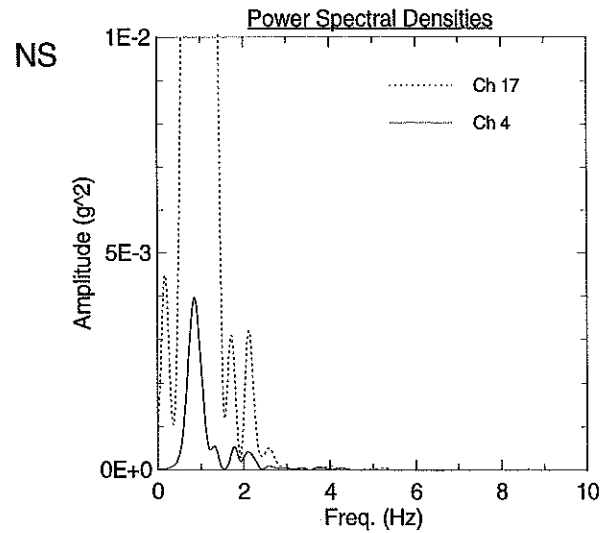
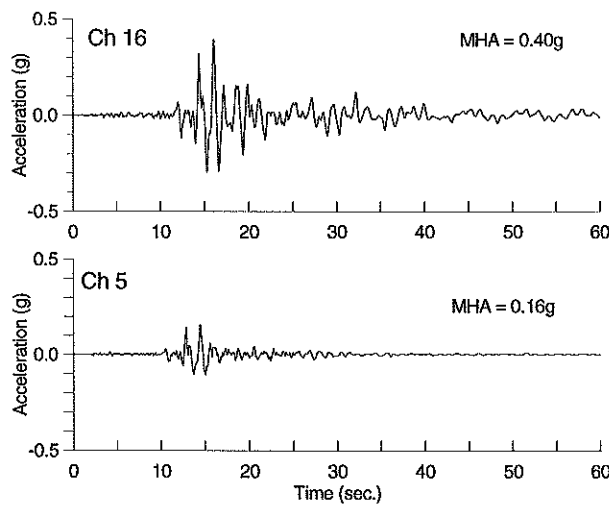
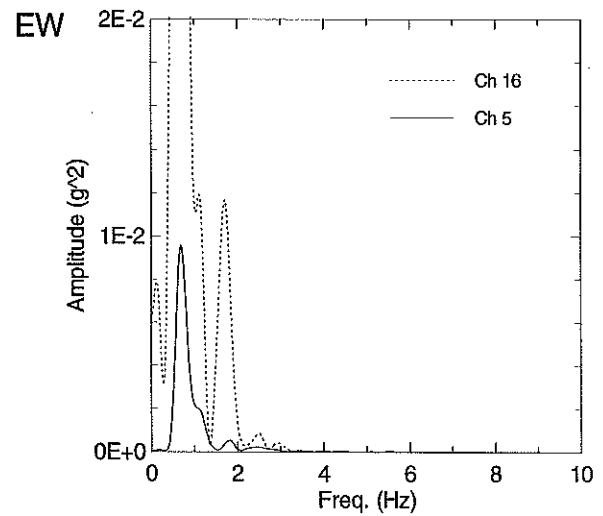
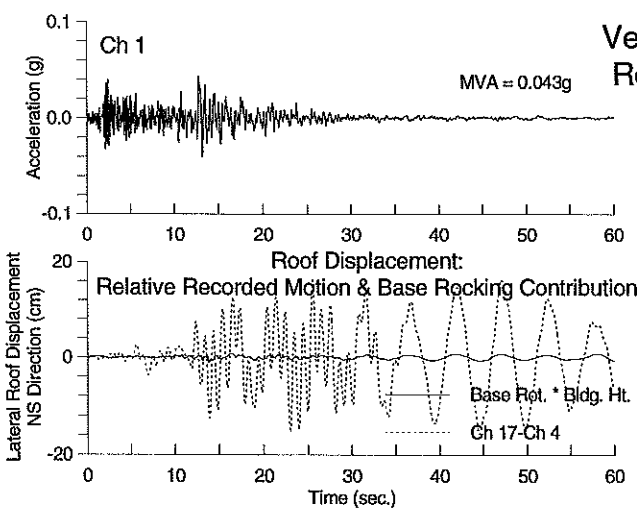
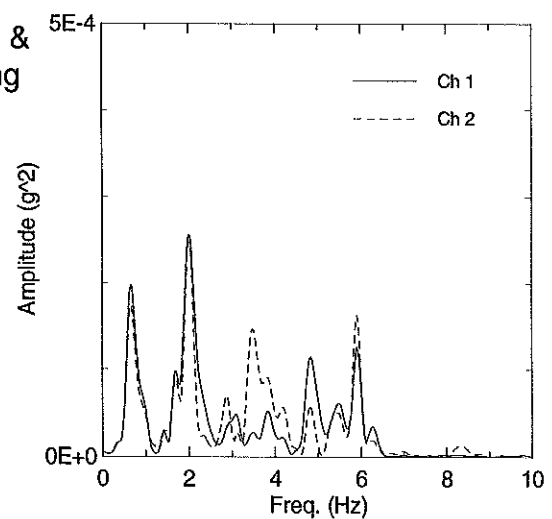


Fig. B3.LP1: SAN FRANCISCO 47 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake
B3-8

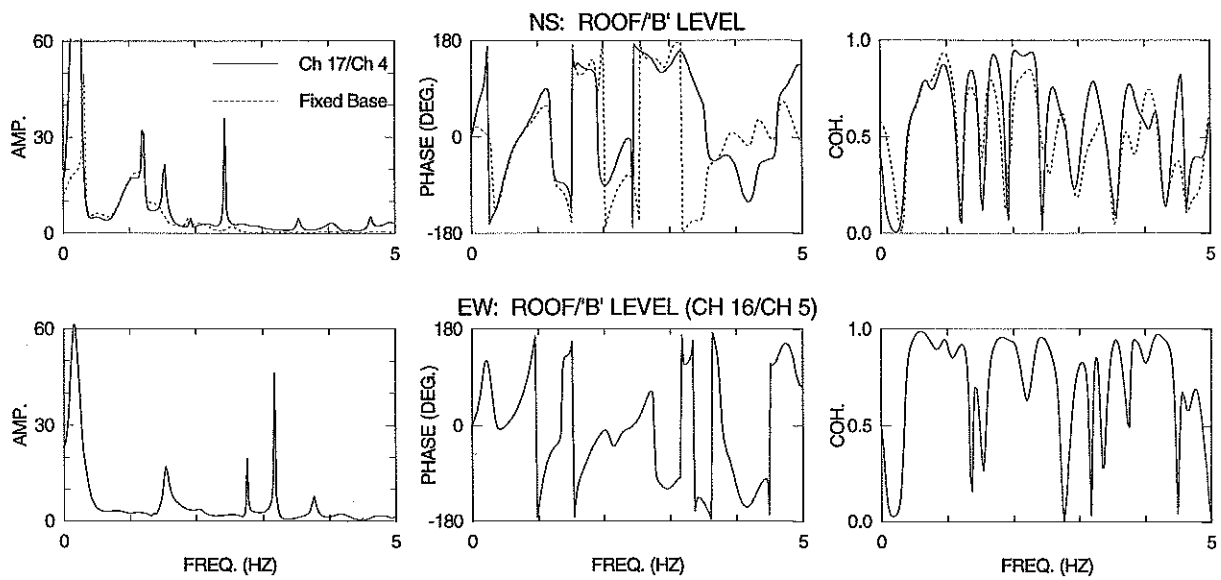


Fig. B3.LP2: SAN FRANCISCO 47 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

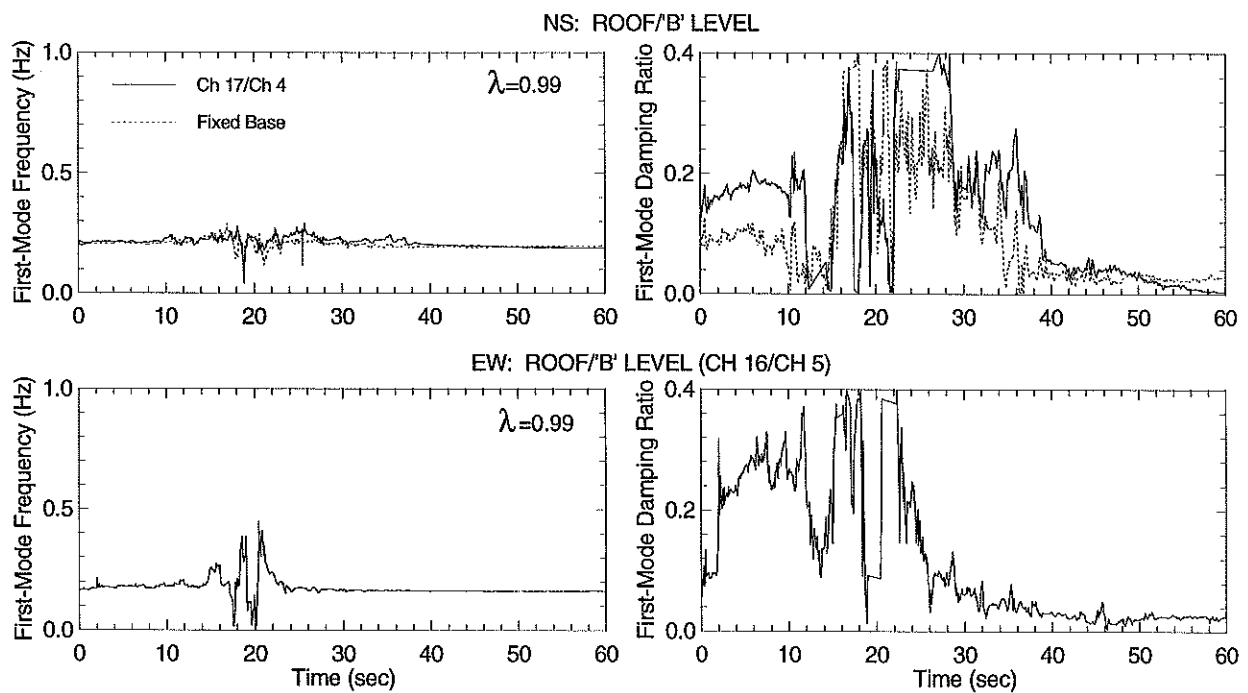


Fig. B3.LP3: SAN FRANCISCO 47 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE B4

Site No.: B4
Site Name: San Francisco Transamerica Tower
Str Sensors: USGS 1239

Lat: 37.80°
Long.: 122.40°

Floor Levels: 63
Bsmt. Levels: 3
Structure: steel frame

Foundation: mat
Site Geology: soil
Design Date: 1972

Recorded Peak Accelerations (g)

Earthquake	Foundation Level			49th Floor		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.11	0.12	0.07	0.30	0.31	0.13

Preprocessing

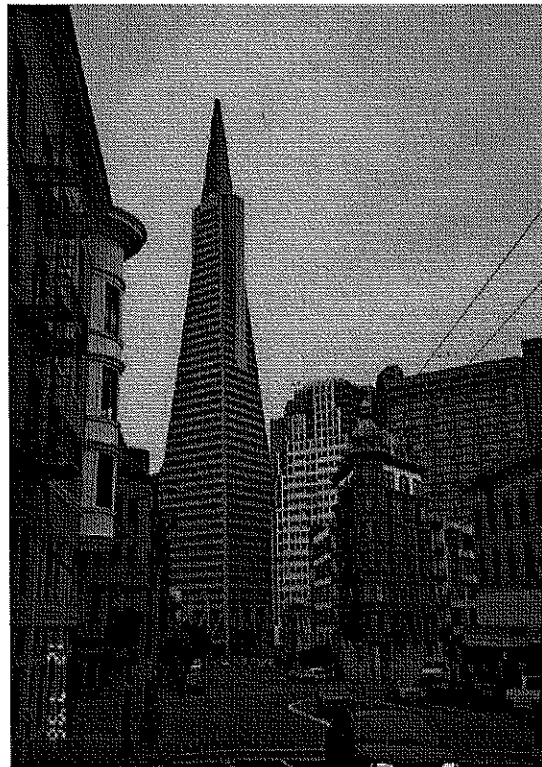
Loma Prieta Earthquake (LP)

Preliminary: USGS; $f_{HP} = 0.17$ Hz; $f_{Nyq} = 100$ Hz

Final: decimated by 8 to $f_{Nyq} = 12.5$ Hz

Figures

B4.1-B4.2: Sensor Locations, Generalized Soil Column
B4.LP1-3 LP Analysis Results



View of structure, looking south-east

DISCUSSION

Site Description

The building is 60 stories tall above ground level with a 3-level basement. The building is pyramid-shaped with ground level dimensions of 174x174 feet which reduce to 5th level dimensions of 145x145 feet. Above the 5th floor, the building has an exterior wall slope of about 1 to 11 upward. Lateral loads are resisted by steel frames in both directions, and the foundation consists of a 9-foot thick reinforced concrete mat with no piles. Accelerographs are present on 6 levels of the structure, with the highest sensors on the 49th floor. Foundation level vertical instruments are positioned on the east and west sides of the building to detect rocking in both the NS and EW directions.

Soil conditions at the site have been investigated to a depth of about 180 feet by Dames and Moore (1969). Based on these data, the generalized soil profile for the site consists of about 15 feet of sandy fill, 9 feet of recent bay deposits consisting of clayey silt and silty sand, 10 feet of sandy dune deposits, and about 135 feet of interbedded silt and clay layers and zones of fine cemented sand which extend to weathered shale bedrock at a depth of 170 feet. Based on measurements by Dames and Moore, the groundwater table is located at a depth of about 15 feet. The structure is founded about 51 feet below grade on relatively firm silts and clays. The shear wave velocities shown in Fig. B4.2 were estimated based on correlations by Fumal (1978) and Seed et al. (1984).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B4.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B4.LP1. The two vertical instruments at the south end of the foundation level were used to evaluate rocking effects in the EW direction. Recordings are not available for Channel 11, so rocking effects could not be evaluated in the NS direction.

Table B4.1: Sensors in structure used for analyses

Foundation Level (FL)			49th Floor (49F)	
NS	EW	V	NS	EW
Ch 21	Ch 22	Ch 7, 13	Ch 15	Ch 16

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch22} + (\text{Ch7} - \text{Ch13}) \cdot 700/170 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B4.LP1 by comparing relative 49th-level displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are much smaller than the 49th-level displacements, and the time histories do not appear to be in phase, indicating that base rocking did not significantly influence the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B4.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B4.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B4.LP3).

Table B4.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (FL-49F)				Flexible-Base			
		tr (EW) d=15, j=32		L (NS)		tr (EW) d=27, j=32		L (NS) d=29, j=23		tr (EW) estimated		L (NS)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	0.28	5.4	na	na	0.28	5.9	0.30	5.8	0.28	6.1	na	na
	2	0.54	7.4			0.54	7.9	0.92	11.9				
	3	0.99	20.0			0.97	19.1	1.35	4.9				
	4	1.23	8.1			1.22	10.4	1.74	13.1				
	5	1.55	6.2			1.57	6.0	2.23	6.6				
	6	1.94	2.1			1.99	3.2	2.77	7.4				

Comments on the analytical results follow:

- The power spectra indicate that the roof motions in both directions are dominated by energy at about $f \approx 0.3, 0.9\text{--}1.2$, and $1.9\text{--}2.2$ Hz. The first five to six modes of structural response are represented within these frequency ranges.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.
- The modal frequencies listed above are consistent with those identified by Safak and Celebi (1991), who used a single-output/multiple-input system identification analysis.

The RPEM results (Fig. B4.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The high damping early in the time history is likely associated with numerical instabilities resulting from the high model order and limited signal duration at these times. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B4.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 149 feet using foundation level motions recorded near the centroid of the structure and the profile in Fig. B4.2. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B4.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 960$ fps	$h = 475$ ft	emb. = 51 ft	$\gamma = 0.15$
LP	$v = 0.45$		tr:	$r_1 = 98$ ft	$r_2 = 99$ ft
	$\beta^* = 5.9\%$	$V_s^* = 800$ fps	L:	$r_1 = --$	$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B4.3 with the fixed-base modal parameters in Table B4.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table B4.4.

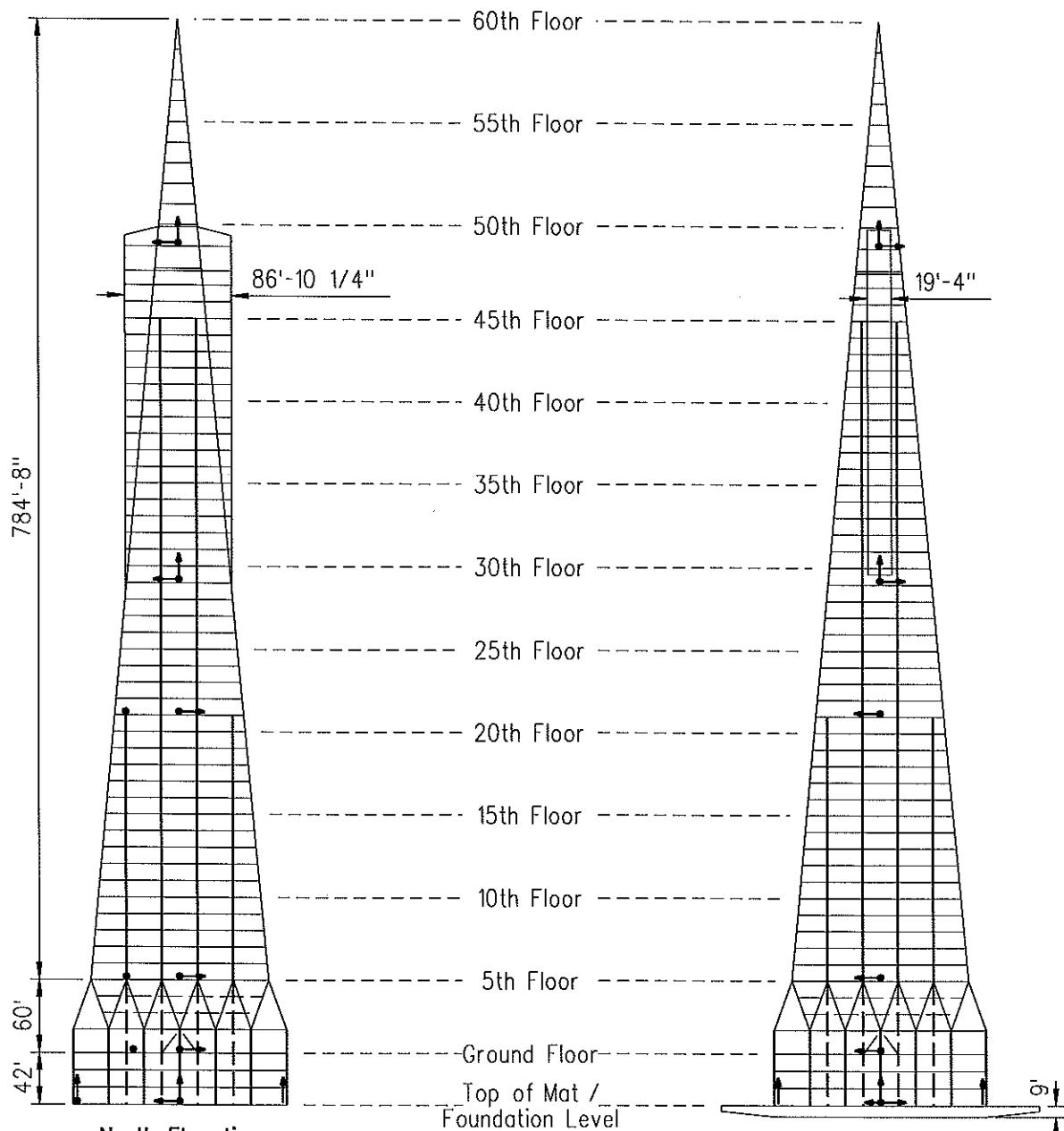
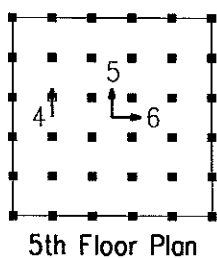
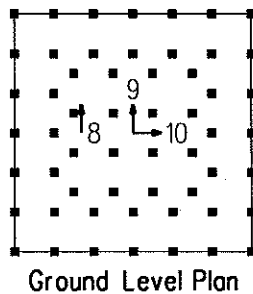
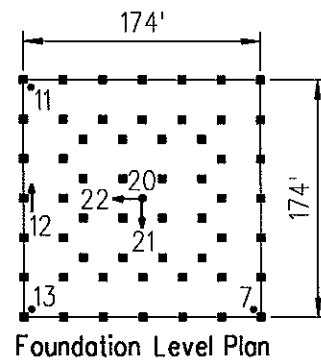
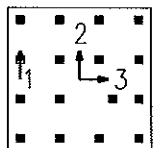
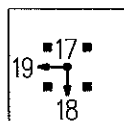
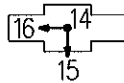
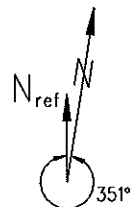
Table B4.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (EW)			L (NS)		
		$\tilde{T} / T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T} / T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.00	0.7	6.1	--	--	--
	Pre.-Vel.	1.12	0.4	4.2	--	--	--
	Pre.-Bie.	1.20	2.3	5.4	--	--	--

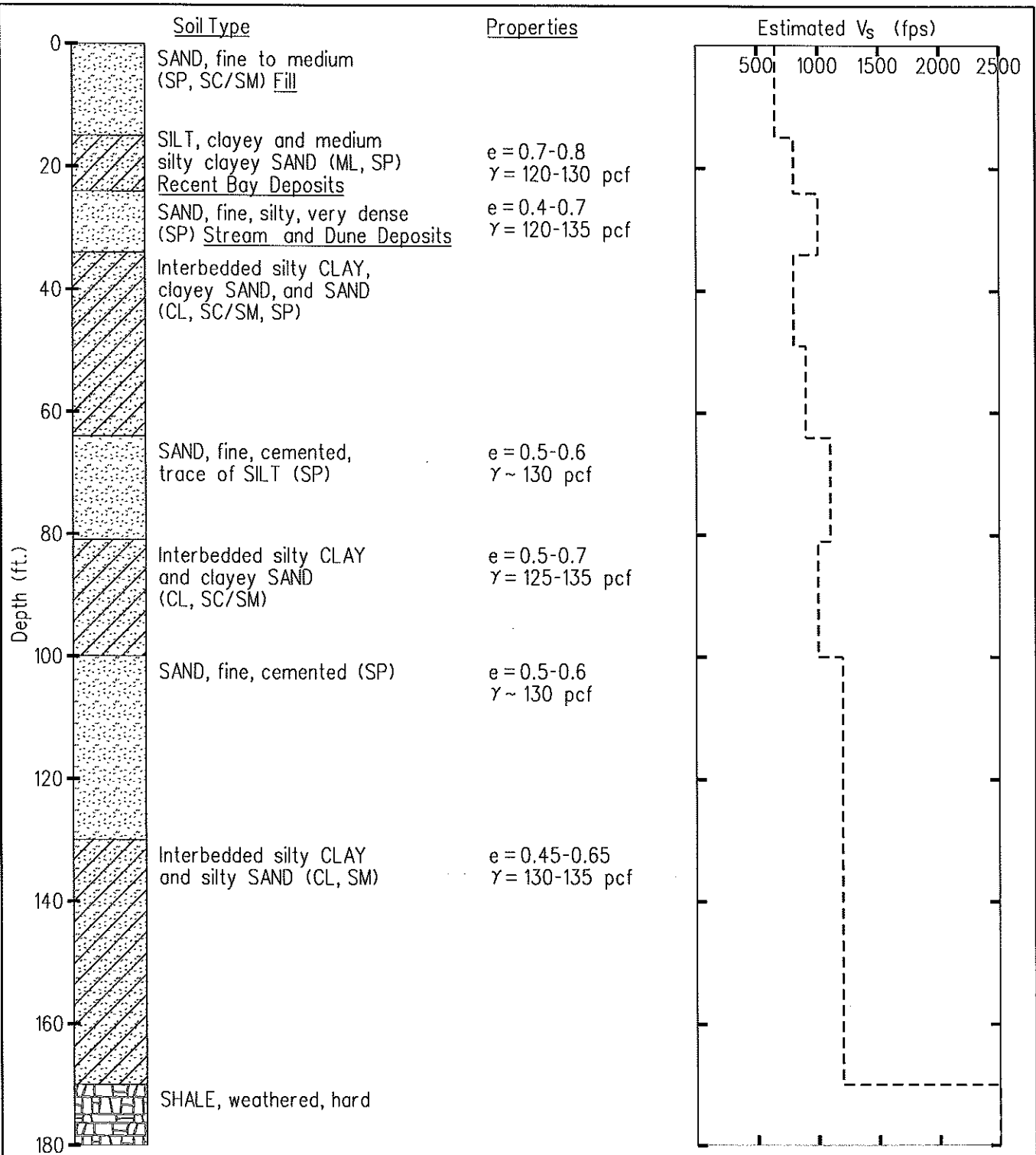
The period lengthening was significantly overpredicted by both formulations. The foundation damping is well predicted by the "modified Veletsos" formulation, but overpredicted by the "modified Bielak" formulation. Further research is needed to clarify the reasons for the overpredicted period lengthening at this and other sites with long period structures.

Data Sources

- Celebi, M. and Safak, E. (1991). "Seismic response of Transamerica Building. I: data and preliminary analysis," *J. Structural Engrg.*, ASCE, 117(8), 2389-2404.
- Dames and Moore. (1969). "Phase I Foundation Investigation, Proposed Transamerica Building, San Francisco, California," *Job No. 6093-001-03*.
- Fumal, T.E. (1978). "Correlations between seismic wave velocities and physical properties of near-surface geologic materials in the Southern San Francisco Bay Region," *Open File Rpt. 78-1067*, U.S. Geological Survey.
- Safak, E. and Celebi, M. (1991). "Seismic response of Transamerica Building. II: system identification," *J. Structural Engrg.*, ASCE, 117(8), 2405-2425.
- Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K. (1984). "Moduli and damping factors for dynamic analyses of cohesionless soils," *Rpt. No. UCB/EERC-84/14*, Earthquake Engrg. Research Ctr., Univ. of California, Berkeley.



Sensor Locations
Transamerica Building San Francisco, CA
Figure B4.1



Data Source:
Dames and Moore
(1969)

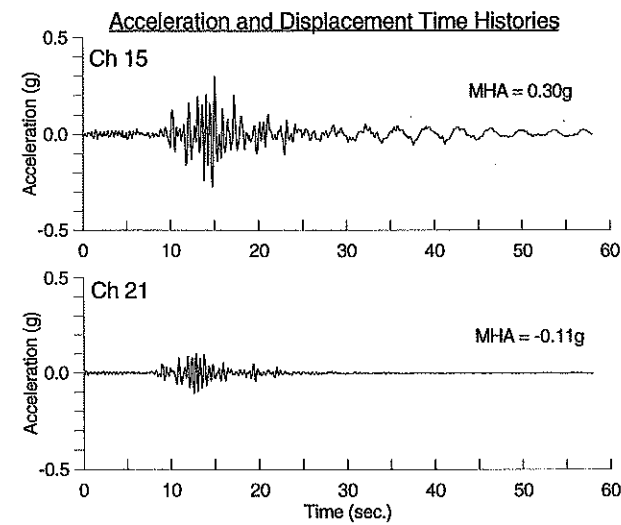
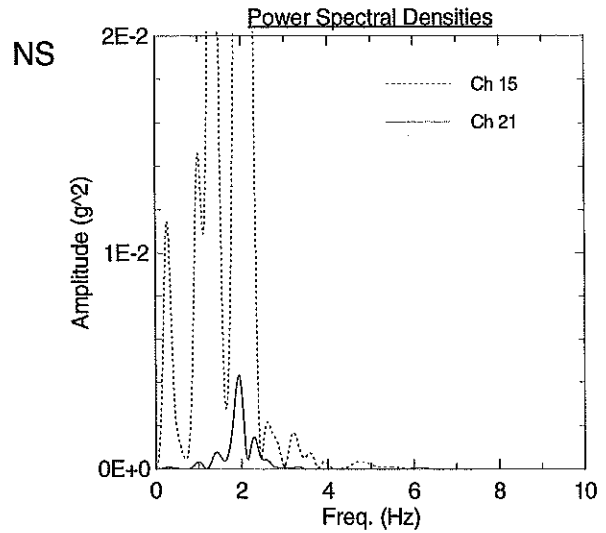
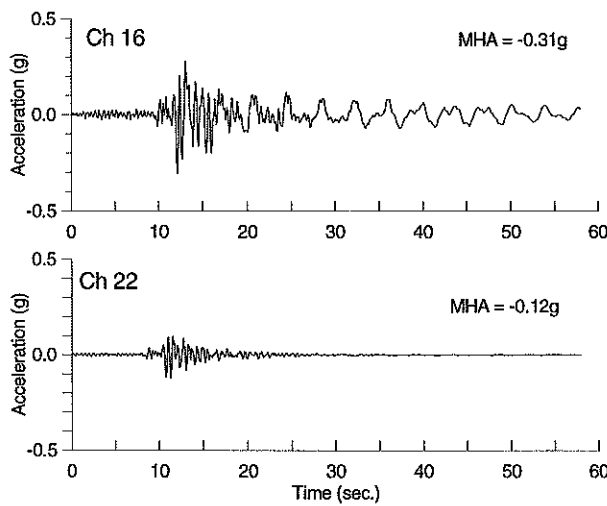
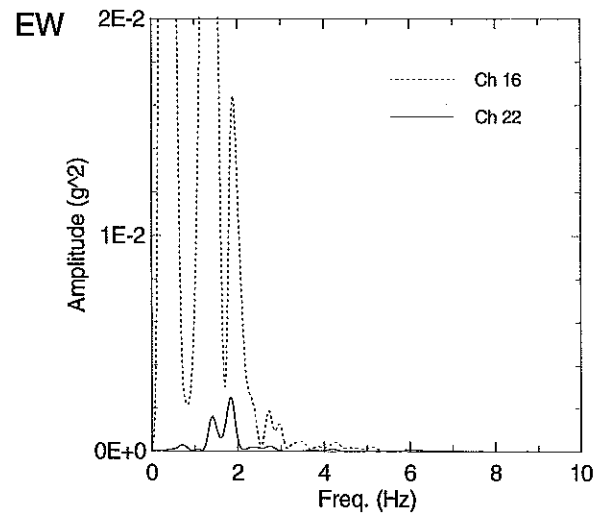
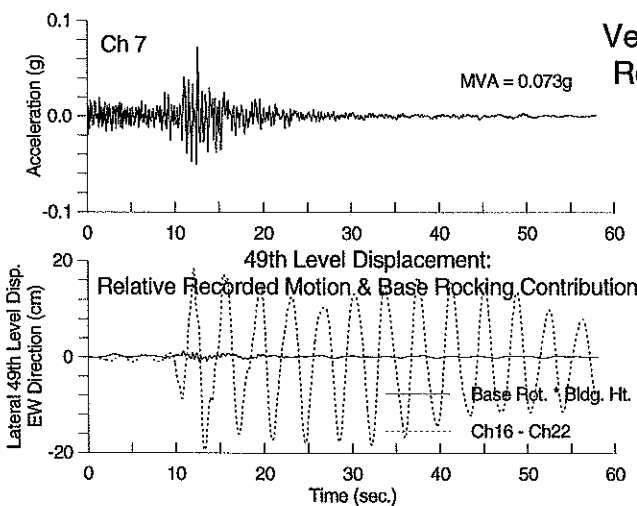
Notes:

1. Groundwater encountered at 15-ft. depth.
2. Shear wave velocities estimated based on correlations by Fumal (1978) and Seed et al. (1984).

Generalized Soil Profile

Transamerica Building
San Francisco, CA

Figure B4.2



Vertical & Rocking

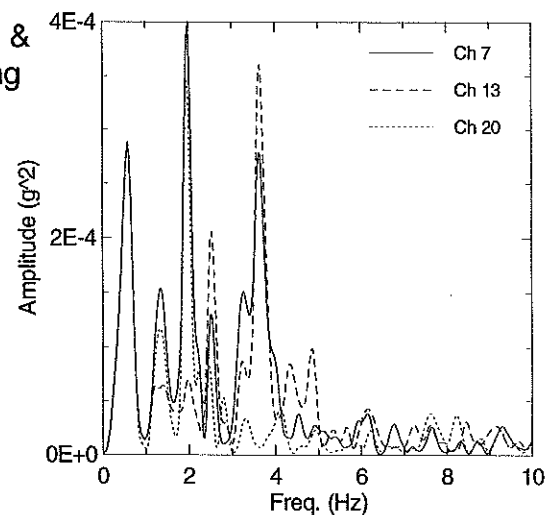


Fig. B4.LP1: SAN FRANCISCO TRANSAMERICA TOWER
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake

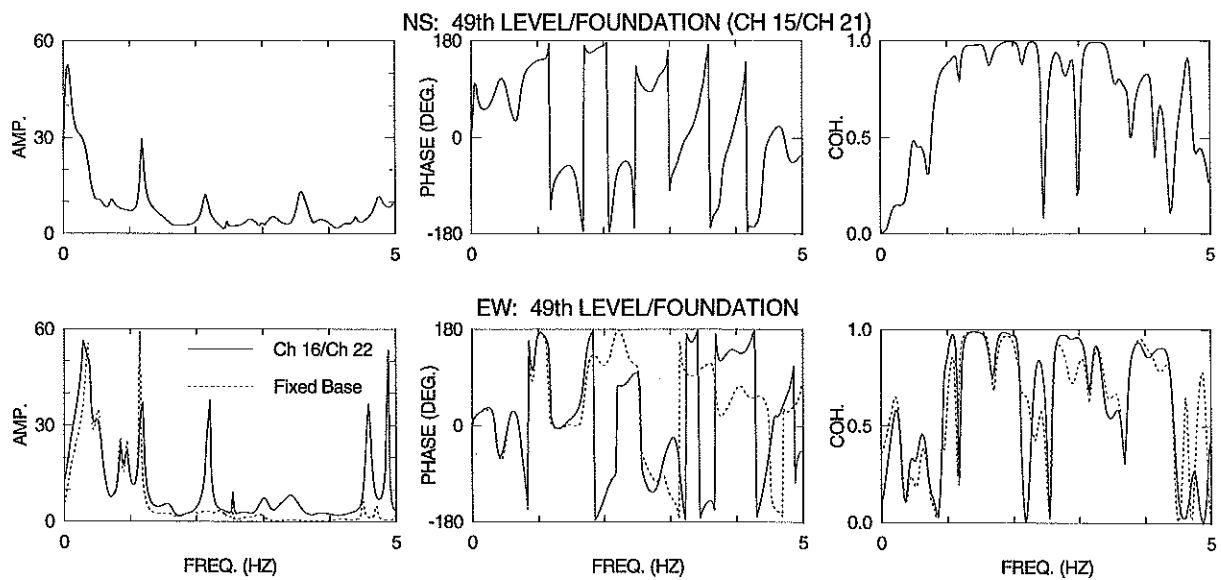


Fig. B4.LP2: SAN FRANCISCO TRANSAMERICA TOWER
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

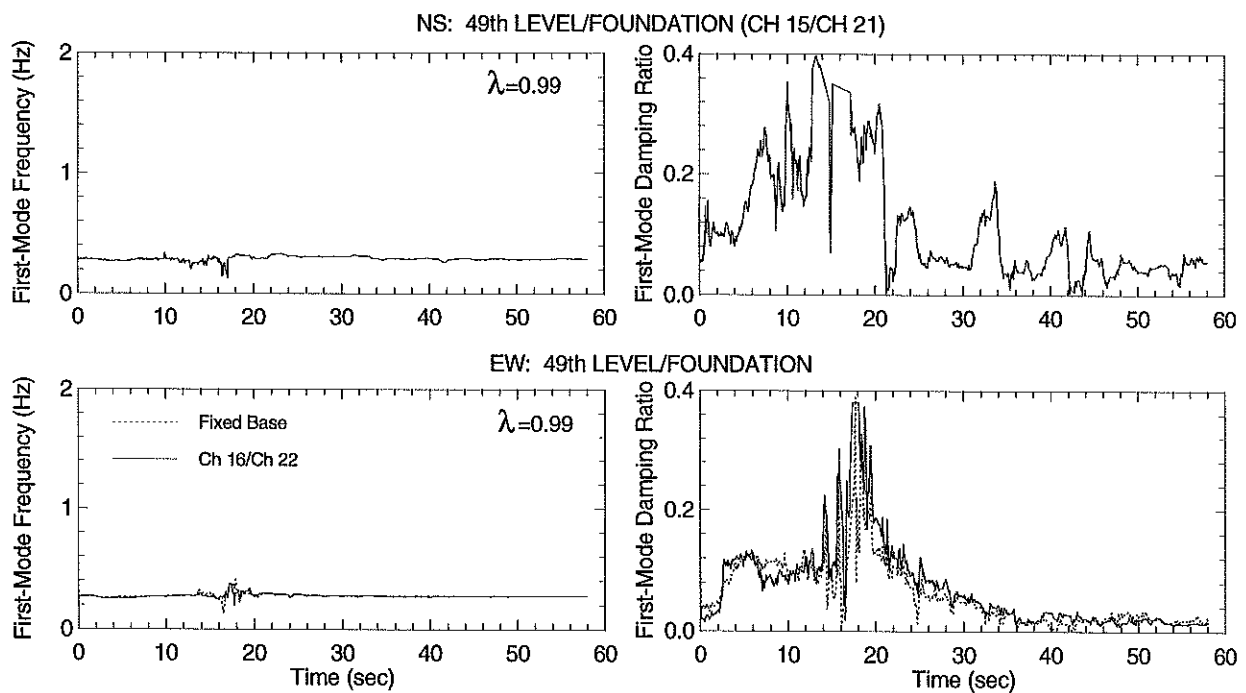


Fig. B4.LP3: SAN FRANCISCO TRANSAMERICA TOWER
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE B5

Site Name: San Jose 10 Story Resid. Bldg.
Sensors: CSMIP 57356

Lat: 37.338°
Long.: 121.888°

Floor Levels: 10
Bsmt. Levels: 0
Structure: Concrete shear walls

Foundation: piles
Site Geology: soil
Design Date: 1971

Recorded Peak Accelerations (g)

Earthquake	Ground Floor			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.093	0.12	0.072	0.36	0.23	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}23\text{ Hz}$; $f_{\text{Nyq}} = 25\text{ Hz}$

Final: no change

Figures

B5.1-B5.2: Sensor Locations, Generalized Soil Column
B5.LP1-3 LP Analysis Results



View of structure, looking south

DISCUSSION

Site Description

The building is 10 stories tall above ground level with no basement and has a rectangular shape in plan (63.5 by 210 feet). Lateral loads are resisted by distributed reinforced concrete shear walls in both directions. The foundations beneath the shear walls consist of 2-foot deep by 2.5 to 4.5-foot wide grade beams supported by 44-foot long, 12-inch square reinforced concrete piles. A 6-inch thick concrete slab-on-grade is present between the grade beams. Most of the accelerographs are located adjacent to either the north or south shear walls, and ground floor vertical instruments are positioned along the south shear wall to detect rocking in the transverse (EW) direction.

Geologic exploration has been conducted immediately adjacent to the site by Peter Kaldveer and Assoc. (1970) and at a number of other nearby sites by several consultants. As shown in Fig. B5.2, these studies encountered about 35 feet of firm to very stiff silts and clays, about 25 feet of very dense clayey sands and gravels, and additional silts and clays which extend to the maximum depth explored of 100 feet. The depth to bedrock has been interpreted in this area to be about 1500 feet (DWR, 1967). The depth-to-groundwater has been found to be about 15 feet in the area. Shear wave velocities in these materials were estimated based on correlations by Fumal (1978) and Fumal and Tinsley (1985).

Instrument Selection and Free-Field Conditions

Instruments along the south shear wall of the building were used in the analyses, as indicated by Table B5.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B5.LP1. Two vertical instruments located at the ground floor level along the shear wall were used to evaluate rocking effects in the EW direction. While both the Ch 2 and Ch 3 instruments could be used to evaluate rocking with Ch 1, the results obtained using Ch 2 were found to be more numerically stable.

Table B5.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 13	Ch 12	Ch 1, 2	Ch 7	Ch 6

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch12} + (\text{Ch2} - \text{Ch1}) \cdot 96/26 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B5.LP1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). Long period components are present in the base rotation trace which are not observed in the relative roof displacements. These long-period undulations may have resulted from errors in baseline correction of the seismograms. Nonetheless, it can be observed from the traces that near the period of the building, the calculated $R \times H$ displacements are in phase with the relative roof displacements and contribute a large fraction of the relative roof displacement amplitude, indicating that base rocking strongly influenced the EW structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B5.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B5.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B5.LP3).

Table B5.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flexible-Base			
		tr (EW) d=0, j=2		L (NS)		tr (EW) d=1, j=2		L (NS) d=4, j=5		tr (EW) estimated		L (NS)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	3.49	18.6	na	na	2.22	8.0	1.40	5.6	1.99	11.4	na	na
	2	11.1	23.2			12.8	15.8	10.3	9.9				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response for the transverse (EW) direction is dominated by the first mode.
- The modest difference between the flexible- and pseudo flexible-base frequencies indicates a moderate SSI effect associated with base translation. Base rocking was more significant, as indicated by the relatively large difference between the first-mode frequencies for the fixed- and pseudo flexible-base cases and the results in Part (a).

The RPEM results (Fig. B5.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B5.3. The strain-dependent soil properties were estimated from deconvolution analyses to a

depth of 65 feet using ground floor motions recorded near the south wall of the structure and the profile in Fig. B5.2. No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though the rocking radiation damping was increased by 60% to account for shape effects.

Table B5.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=840$ ft/s	$h=61$ ft	emb.=0	$\gamma = 0.15$
	$\nu=0.45$		tr: $r_1 = 65$ ft		$r_2 = 49$ ft
LP	$\beta^*=4.1\%$	$V_s^*=770$ ft/s	L: $r_1 = --$		$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B5.3 with the fixed-base modal parameters in Table B5.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table B5.4.

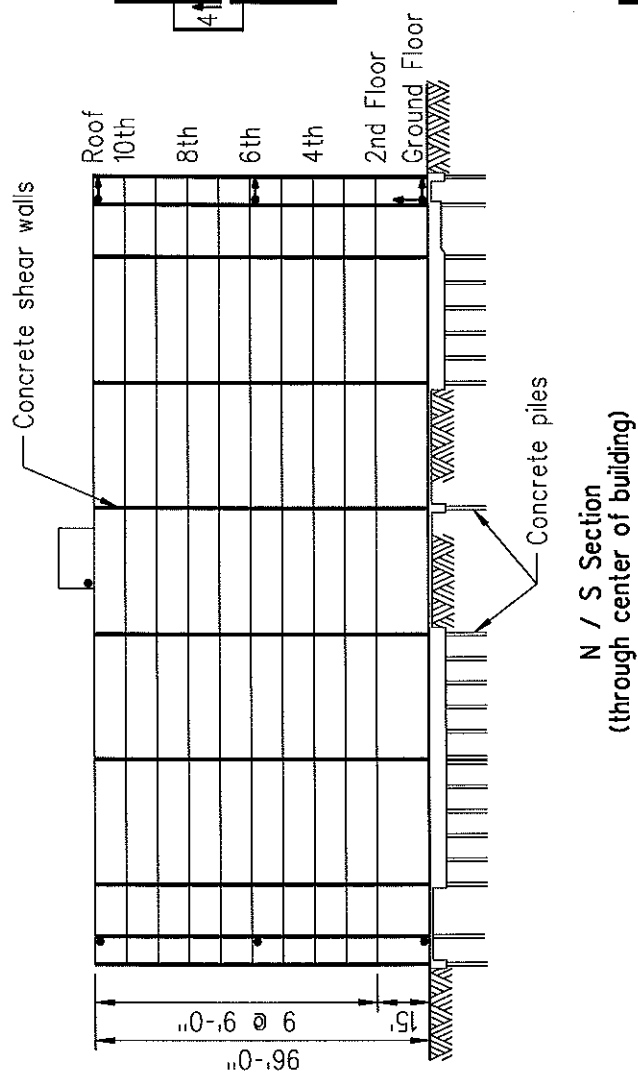
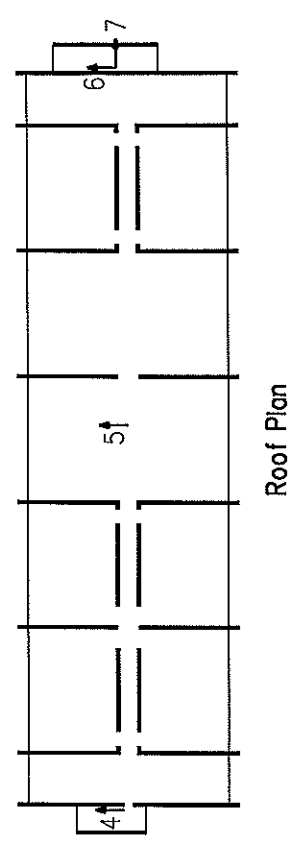
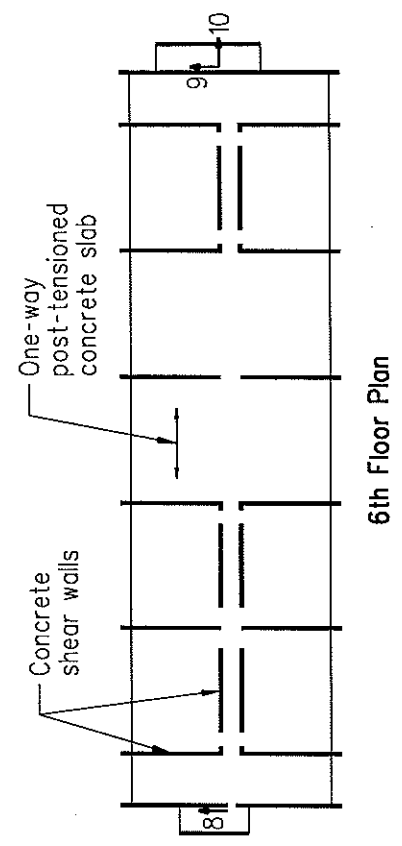
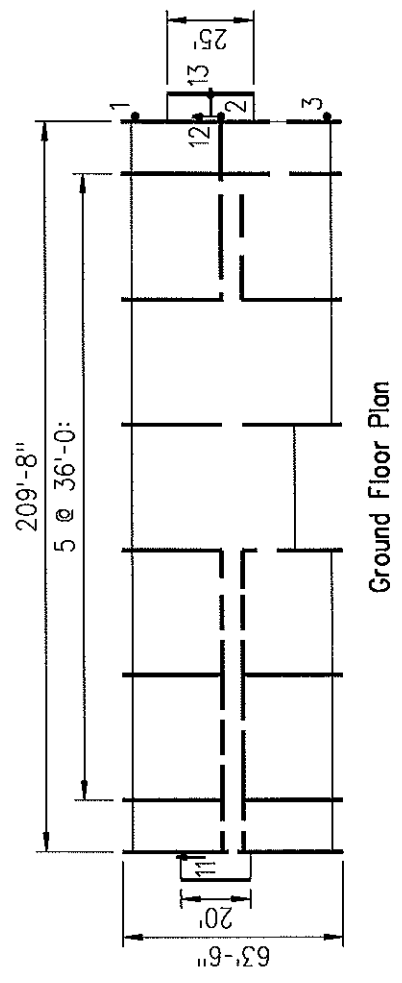
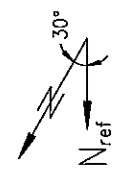
Table B5.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
LP	Sys. ID	1.75	7.9	11.4	--	--	--
	Prediction	1.44	5.2	11.5	--	--	--

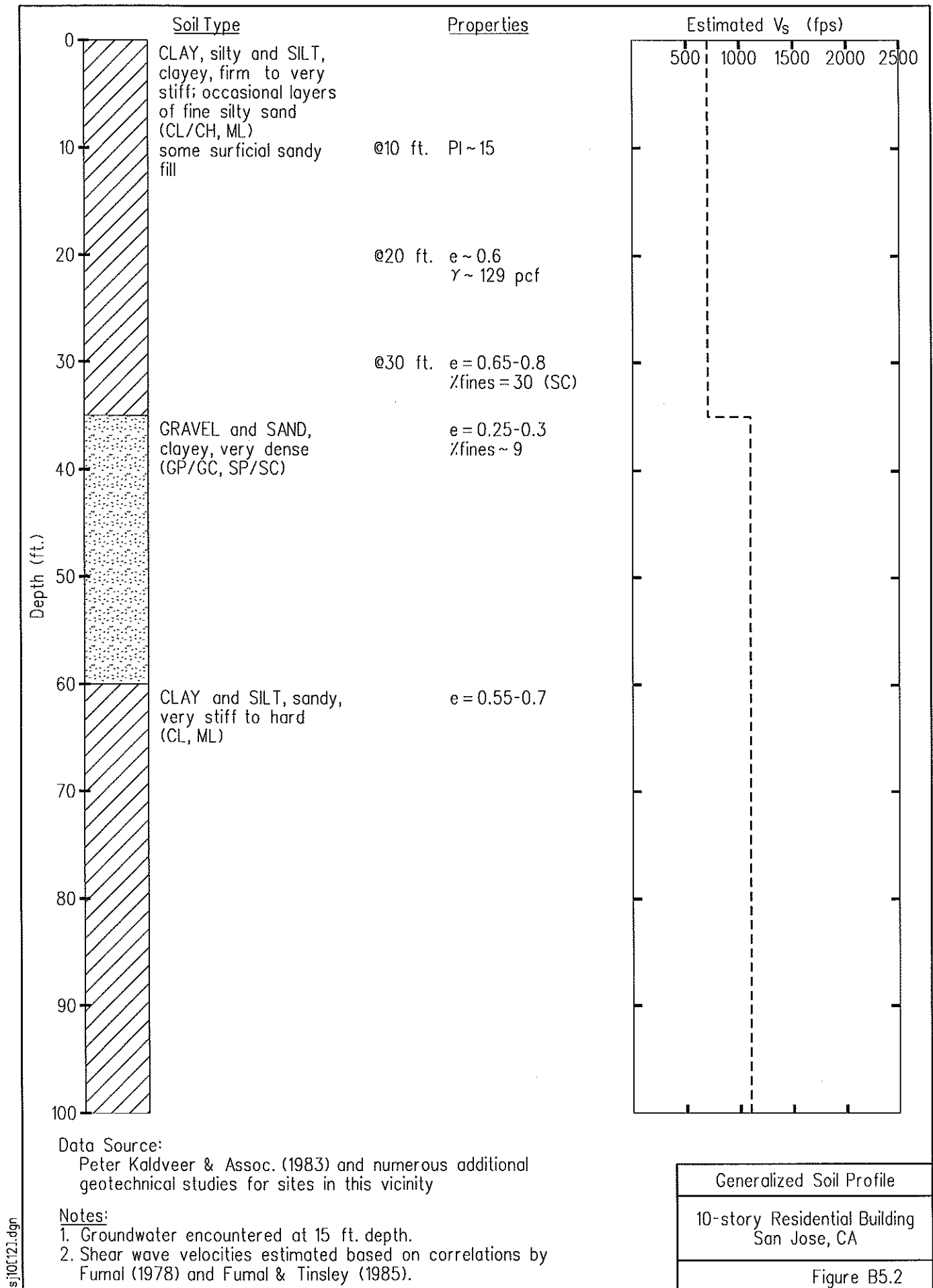
The period lengthening and foundation damping for this structure are somewhat underpredicted, but a significant SSI effects at the site is predicted. The cause of the discrepancies is likely incorrect shear wave velocities [the V_s profile for this site was estimated based on correlations by Fumal (1978) and Fumal and Tinsley (1985)].

Data Sources

- California Department of Water Resources, DWR (1967). "Evaluation of groundwater resources, South Bay," Appendix A; Geology, *Bull. No. 118-1*, The Resources Agency, Dept. of Water Resources.
- Fumal, T.E. (1978). "Correlations between seismic wave velocities and physical properties of near-surface geologic materials in the Southern San Francisco Bay Region," *Open File Rpt. 78-1067*, U.S. Geological Survey.
- Fumal, T.E. and Tinsley (1985). "Mapping shear-wave velocities of near-surface geologic materials," *Prof. Paper 1360*, U.S. Geological Survey.
- Peter Kaldveer and Associates (1983). "Foundation Investigation for Saint James Plaza Office Building, San Jose, CA," *Proj. No. K803-1*.



Sensor Locations
10-story Residential Building San Jose, CA
Figure B5.1



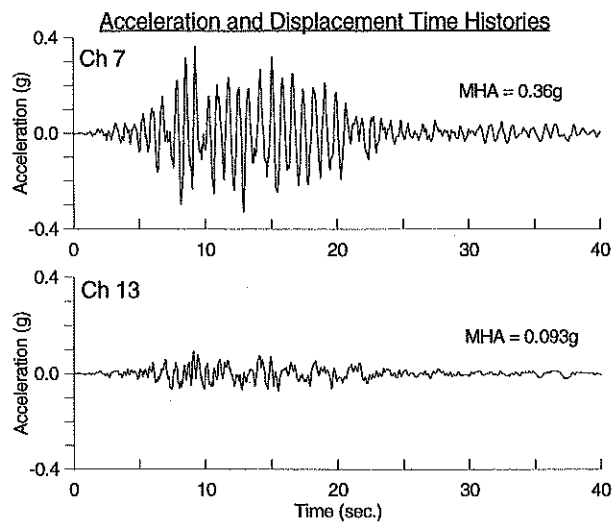
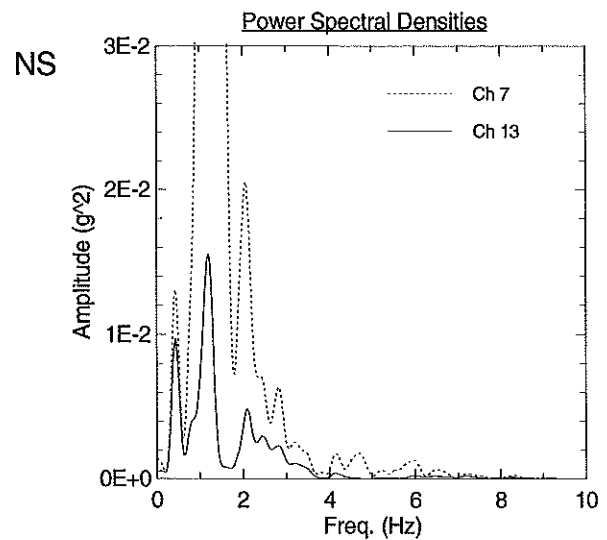
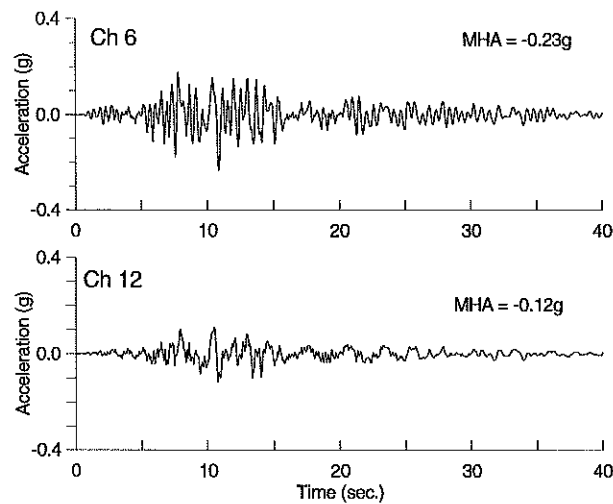
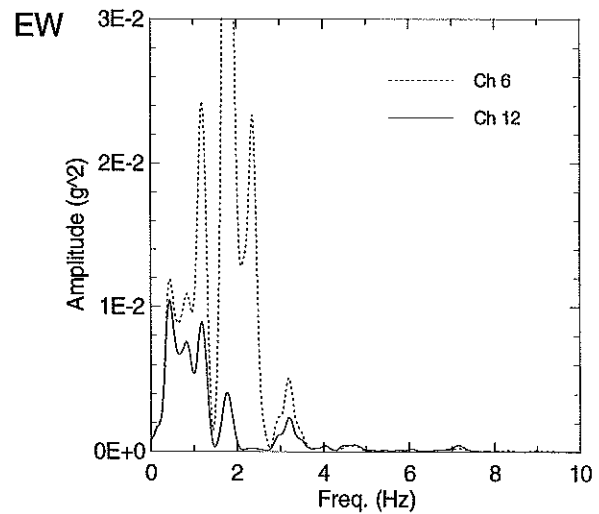
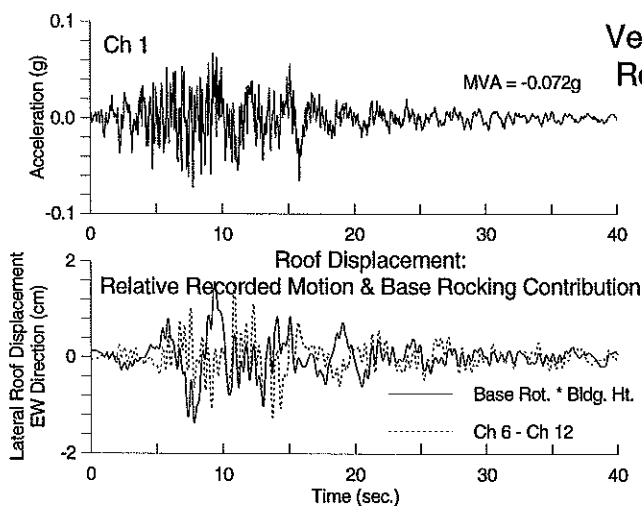
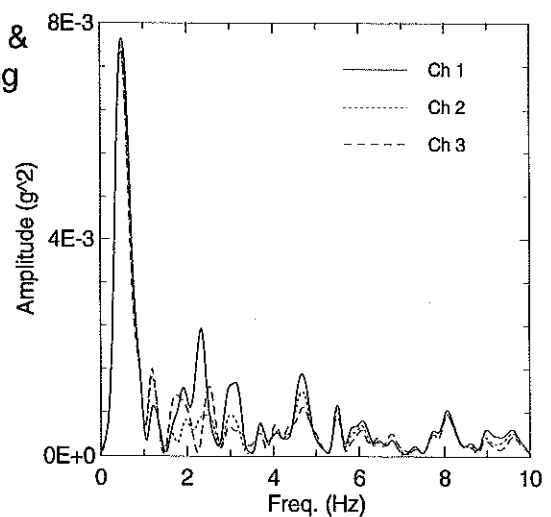


Fig. B5.LP1: SAN JOSE 10 STORY RESIDENTIAL BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake

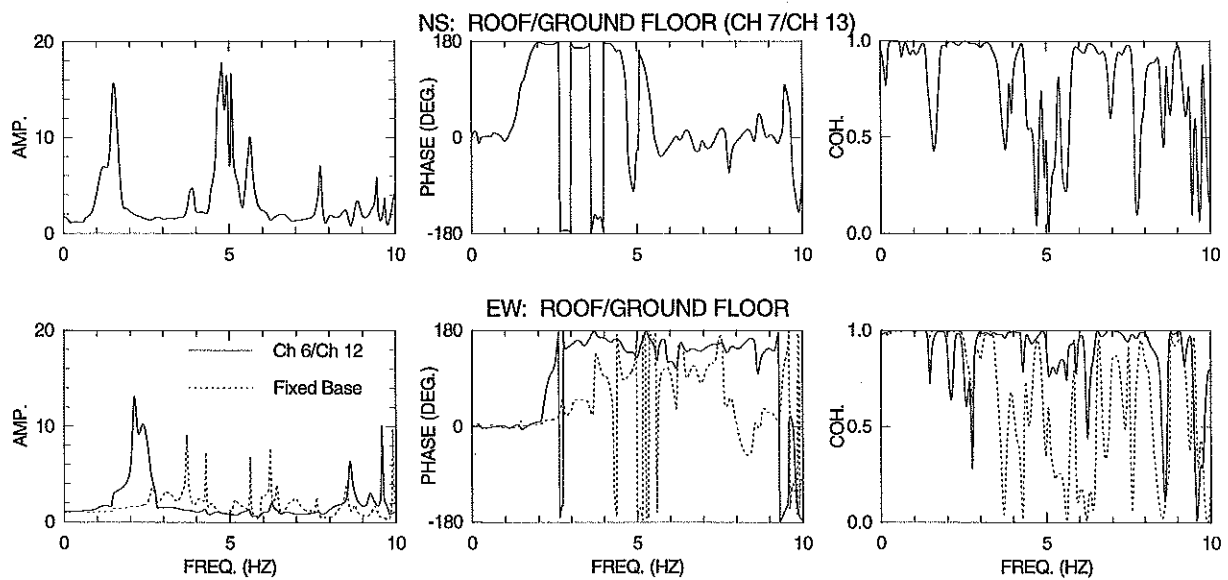


Fig. B5.LP2: SAN JOSE 10 STORY RESIDENTIAL BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

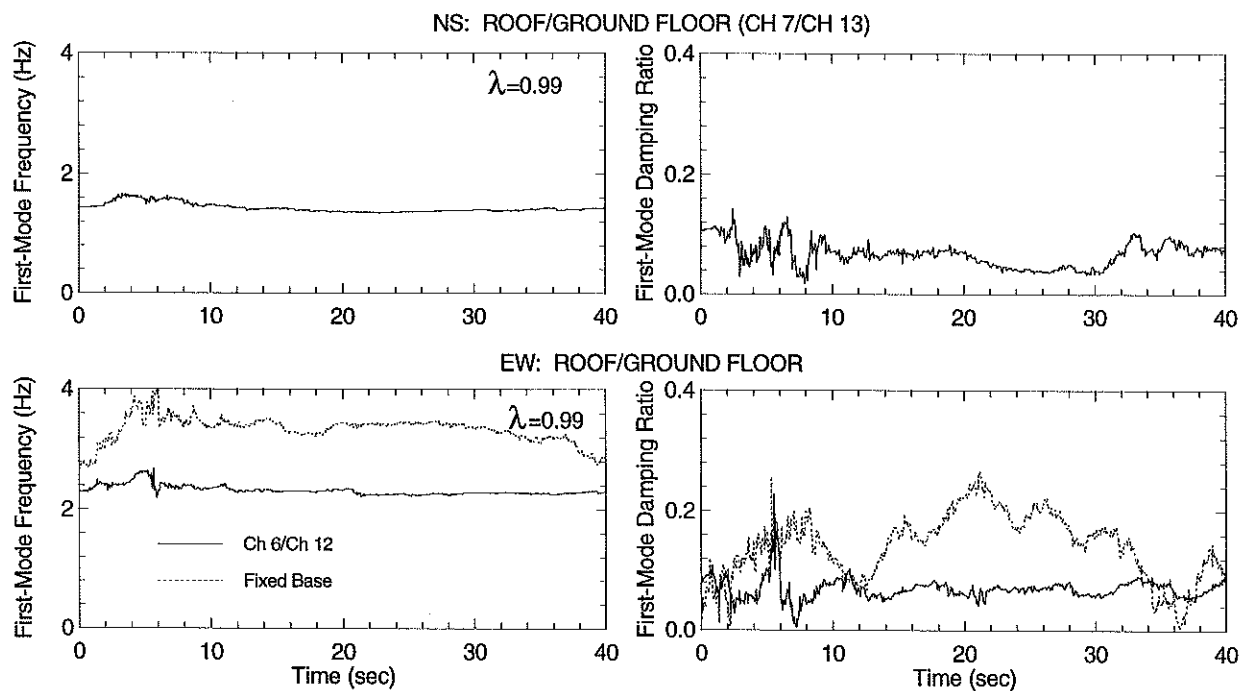


Fig. B5.LP3: SAN JOSE 10 STORY RESIDENTIAL BUILDING
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE B6

Site Name: San Jose 13 Story Government Bldg.
Sensors: CSMIP 57357

Lat: 37.353°
Long.: 121.903°

Floor Levels: 13
Bsmt. Levels: 1
Structure: moment resisting steel frame

Foundation: mat
Site Geology: soil
Design Date: 1972

Recorded Peak Accelerations (g)

Earthquake	Lower Level			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.098	0.084	0.09	0.34	0.31	--

-- = no instrument

Preprocessing

Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.24\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$

Figures

B6.1-B6.2: Sensor Locations, Generalized Soil Column
B6.LP1-3 LP Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The building is 13 stories tall above the lower level with a single level basement, and is roughly square in plan (147.5 x 147.5 feet). Lateral loads are resisted by steel moment frames in both directions, and the foundation consists of a reinforced concrete mat. Lateral accelerographs are present in the lower, 2nd, 7th, 12th, and roof levels, and vertical lower-level instruments are present in the west and south ends of the building to measure base rocking in the NS and EW directions.

Geologic exploration at the site by WCC (1958 and 1974) encountered about 60 feet of medium to very stiff silts and clays, about 20 feet of very dense sands and gravels, and additional silts and clays which extend to the maximum depth explored of 90 feet. The depth to bedrock has been interpreted in this area to be about 1500 feet (DWR, 1967). Measurements by WCC have indicated that the depth-to-groundwater is about 30 feet. Shear wave velocities in these materials were estimated based on correlations by Fumal (1978), Seed et al. (1984) and Fumal and Tinsley (1985).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B6.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B6.LP1. The three vertical instruments located in the lower level were used to evaluate rocking effects in the NS and EW directions.

Table B6.1: Sensors in structure used for analyses

Lower Level (LL)			Roof (R)	
NS	EW	V	NS	EW
Ch 22	Ch 21	Ch 1-3	Ch 6	Ch 5

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS and EW directions,

$$\begin{aligned}\text{input,ns} &= \text{Ch22} + (\text{Ch2} - \text{Ch1}) \cdot 187/117 \\ \text{input,ew} &= \text{Ch21} + (\text{Ch2} - \text{Ch3}) \cdot 187/130\end{aligned}\tag{1}$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B6.LP1 by comparing relative roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements but

have a much smaller amplitude, indicating a small rocking contribution to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B6.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B6.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B6.LP3).

Table B6.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (LL-R)				Flexible-Base			
		tr (NS) d=0, j=4		L (EW) d=0, j=4		tr (NS) d=0, j=4		L (EW) d=2, j=4		tr (NS) estimated		L (EW) estimated	
		f	ζ	f	ζ	$\bar{f}^*$	$\bar{\zeta}^*$	$\bar{f}^*$	$\bar{\zeta}^*$	$\bar{f}$	$\bar{\zeta}$	$\bar{f}$	$\bar{\zeta}$
LP	1	0.47	1.6	0.46	1.8	0.46	1.4	0.46	1.9	0.46	1.0	0.45	2.2
	2	1.38	4.1	1.36	5.1	1.39	3.4	1.37	5.6				
	3	2.33	6.4	2.36	4.5	2.31	6.9	2.33	5.8				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.

The RPEM results (Fig. B6.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B6.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 83 feet using lower level motions recorded near the southwest corner of the structure and the profile in Fig. B6.2. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B6.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 770$ fps	$h = 109$ ft	emb.=13.5 ft	$\gamma = 0.15$
LP	$\nu = 0.45$		tr: $r_1 = 83$ ft		$r_2 = 84$ ft
	$\beta^* = 3.5\%$	$V_s^* = 725$ fps	L: $r_1 = 83$ ft		$r_2 = 84$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B6.3 with the fixed-base modal parameters in Table B6.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table B6.4.

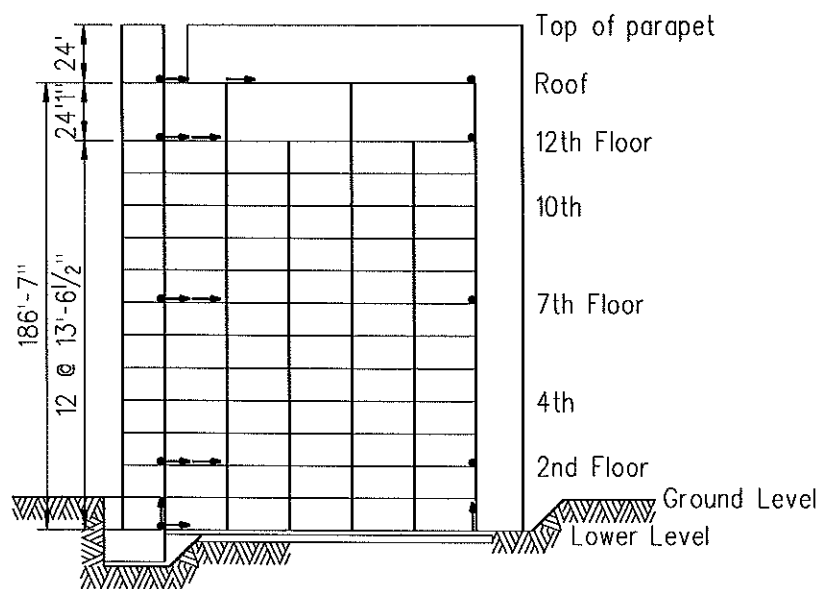
Table B6.4: Empirical and predicted inertial interaction effects

LP	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
		1.01	-0.5	1.0	1.02	0.5	2.2
	Pre.-Vel.	1.02	0.1	1.6	1.02	0.1	1.9
	Pre.-Bie.	1.02	0.3	1.8	1.02	0.3	2.0

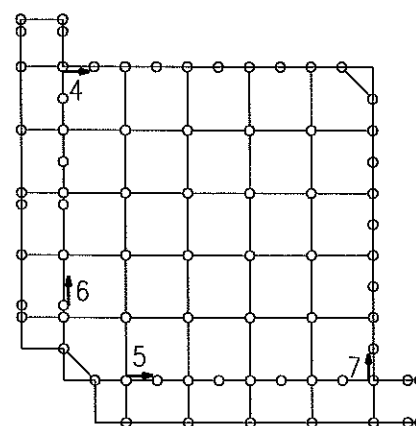
The period lengthening and foundation damping were satisfactorily predicted by both formulations. The negative foundation damping from the system identification analyses for the transverse direction indicates a minor numerical error in these results. Nonetheless, the predictive analyses evaluated reasonably accurately the low foundation damping at this site.

Data Sources

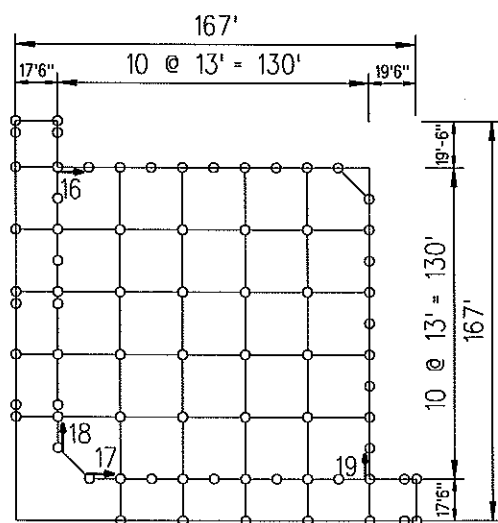
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- Woodward-Lundgren and Assoc., WCC (1974). “Soil investigation for the proposed City Hall Annex, San Jose, CA,” *Proj. No. JS12990*.



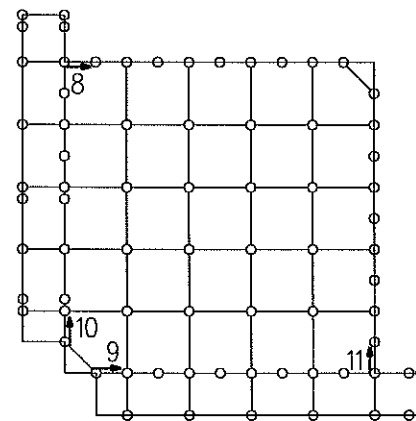
E / W Section



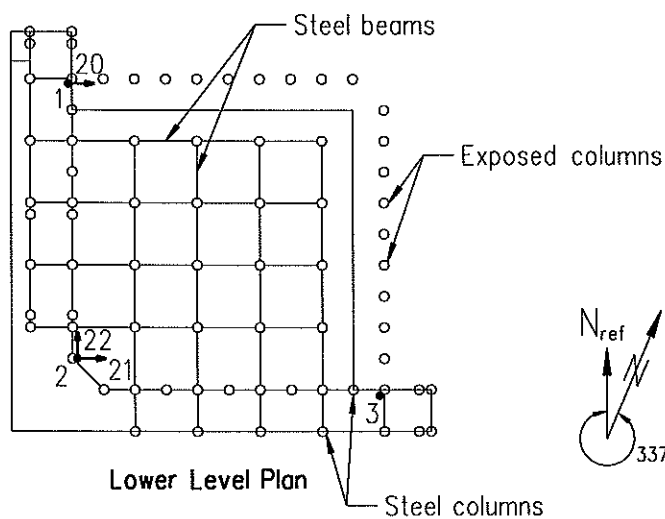
Roof Plan



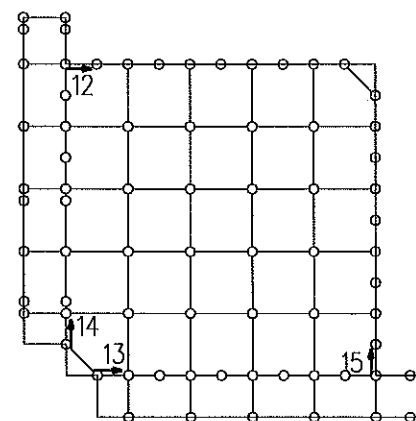
2nd Floor Plan



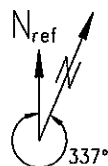
12th Floor Plan



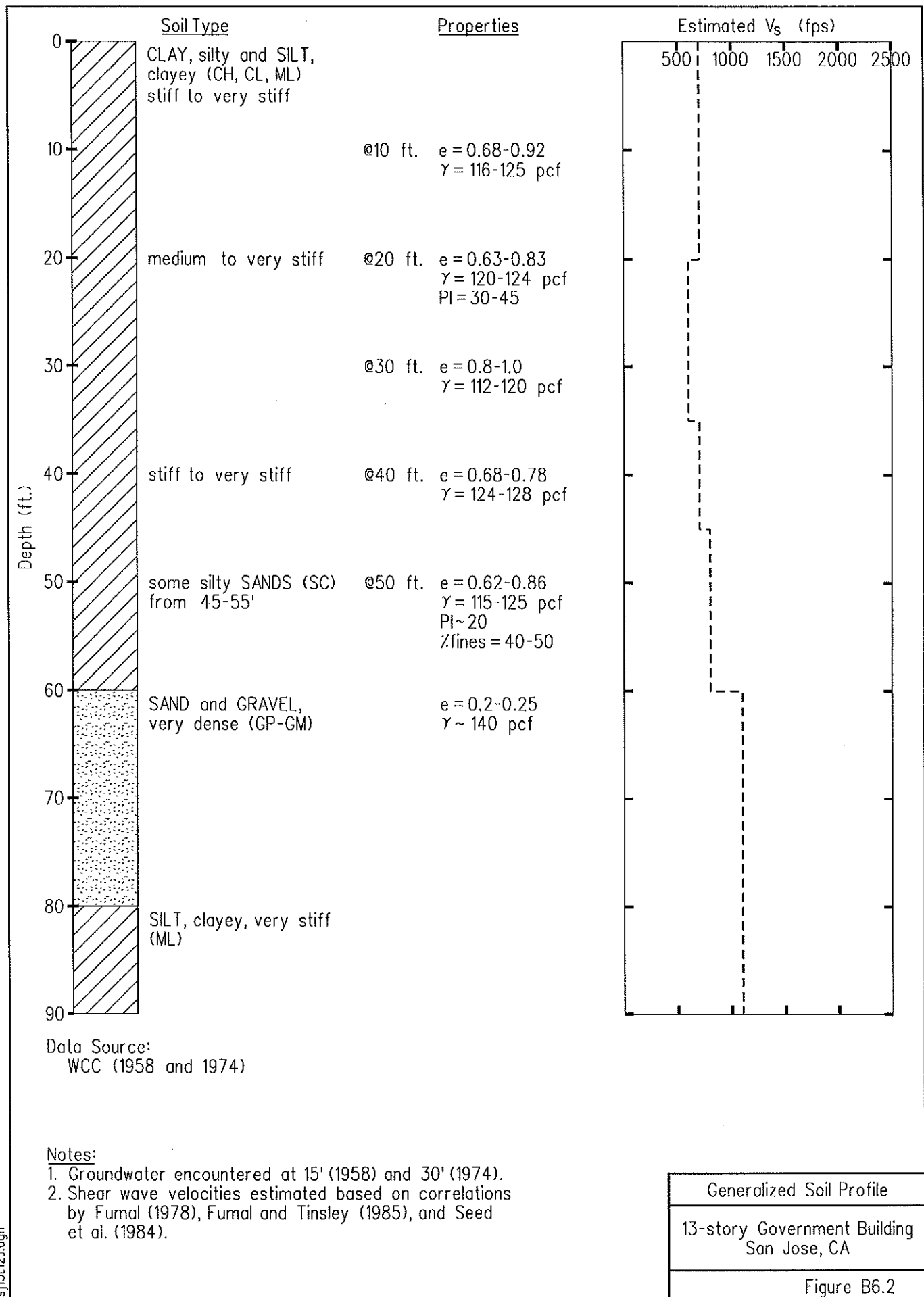
Lower Level Plan

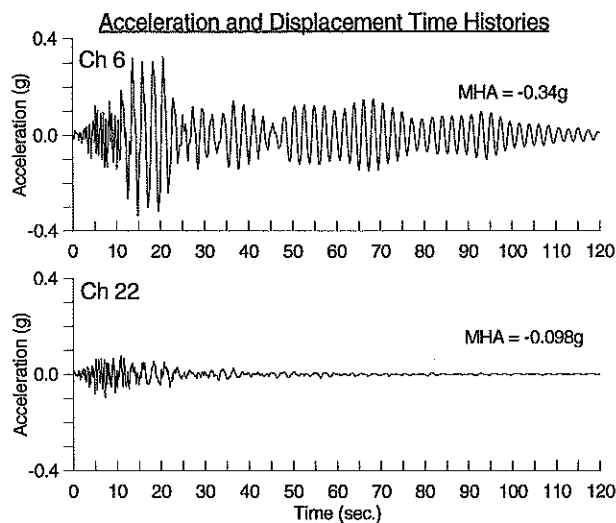
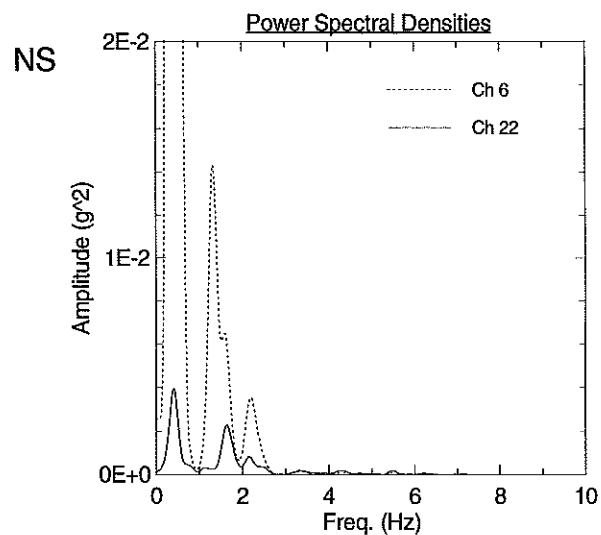
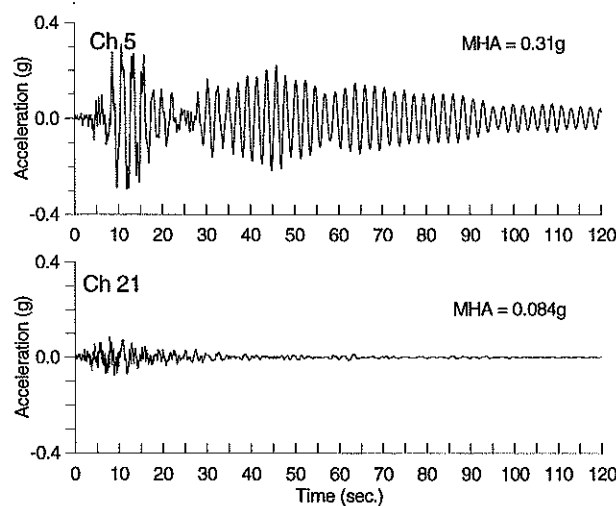
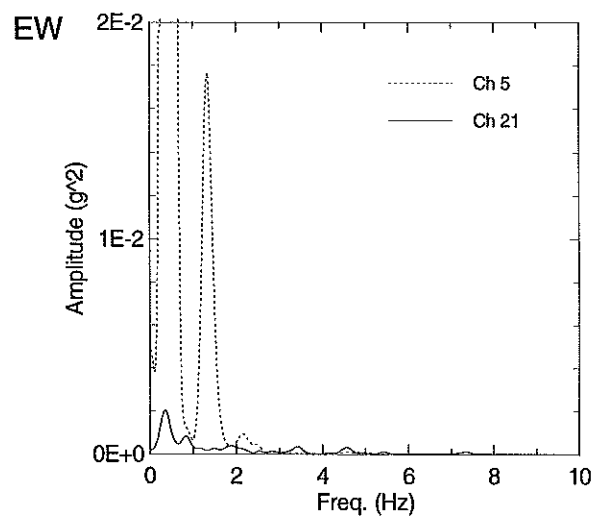
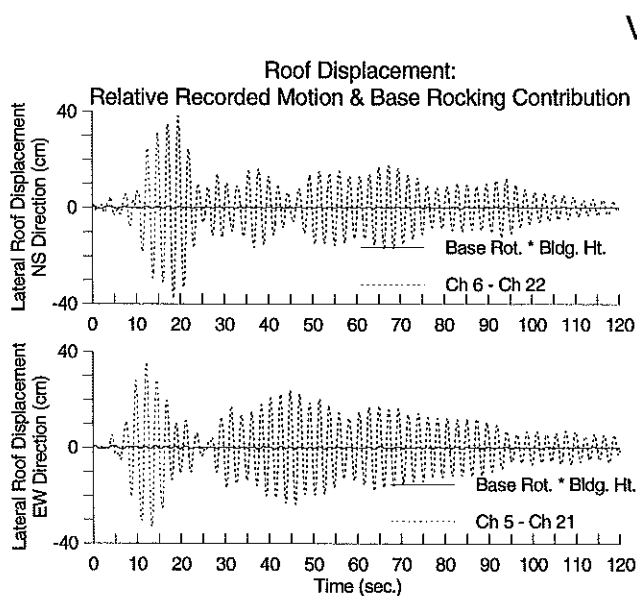


7th Floor Plan



Sensor Locations
13-story Government Building San Jose, CA
Figure B6.1





Vertical & Rocking

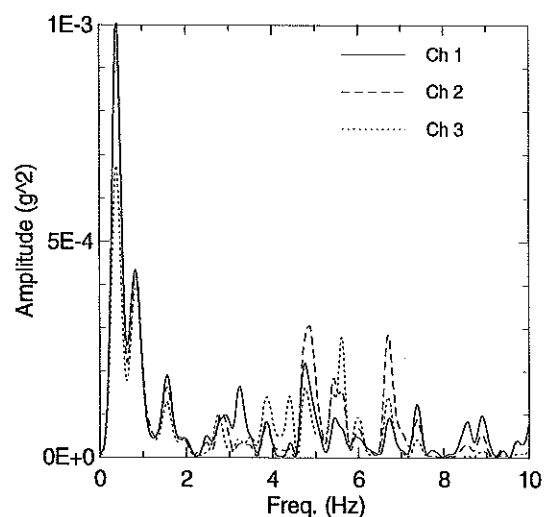


Fig. B6.LP1: SAN JOSE 13 STORY GOVERNMENT BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake

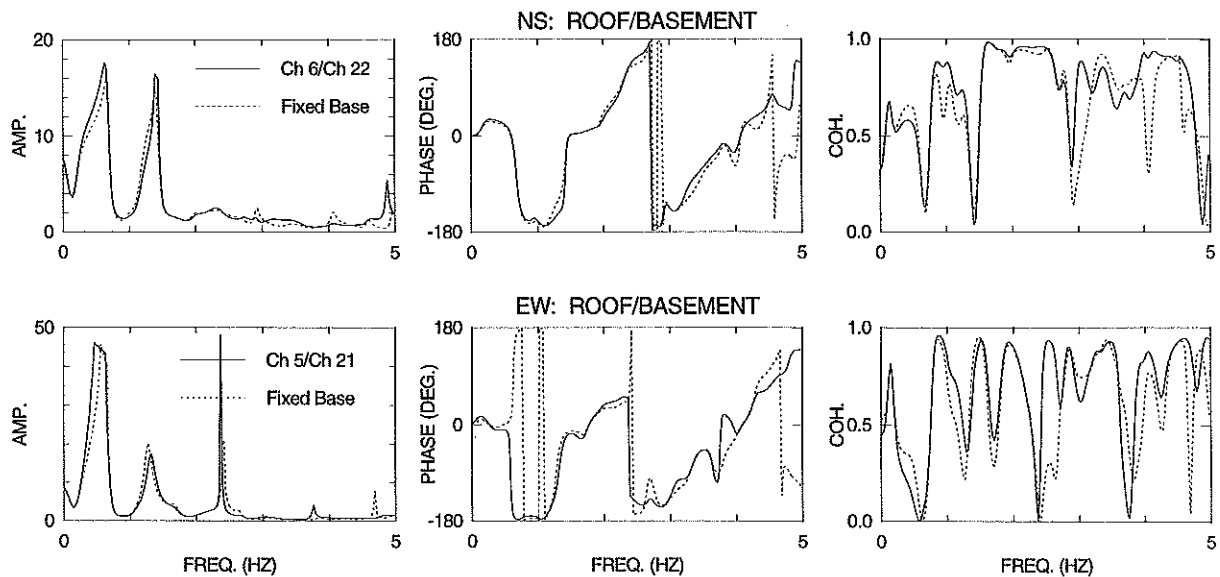


Fig. B6.LP2: SAN JOSE 13 STORY GOVERNMENT BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

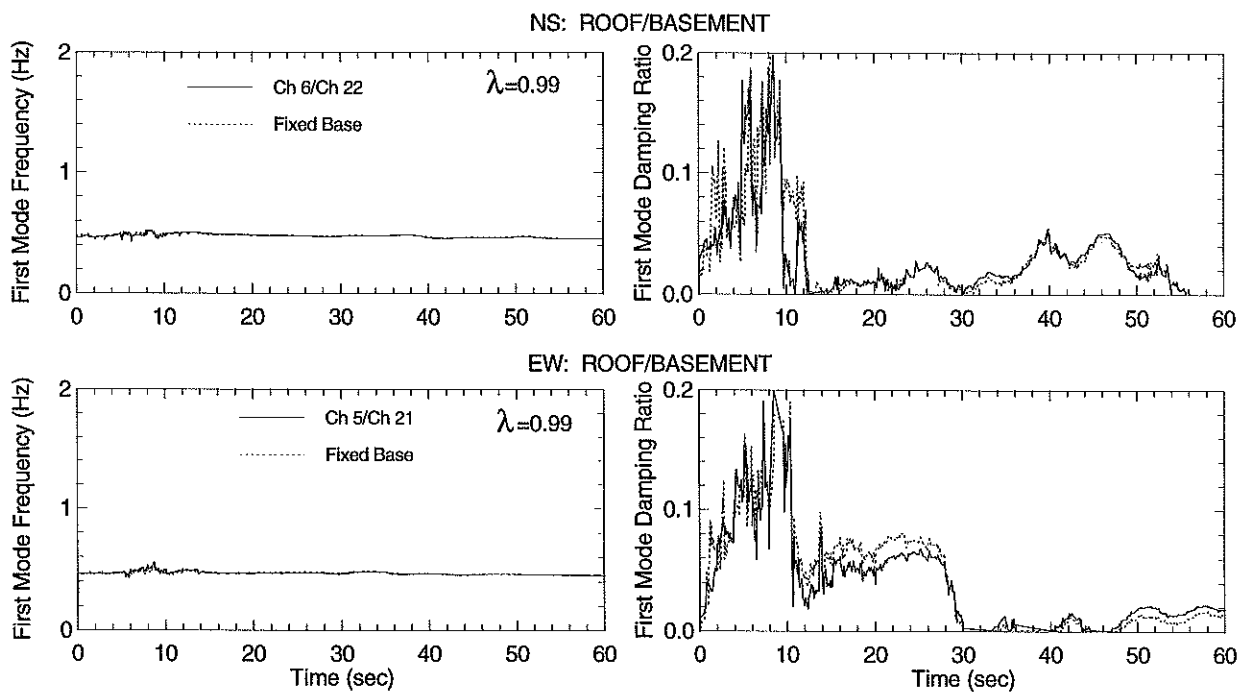


Fig. B6.LP3: SAN JOSE 13 STORY GOVERNMENT BUILDING
Time Variation of First Mode Parameters
1989 Loma Prieta Earthquake

SITE B7

Site Name: Walnut Creek 10 Story Comm. Bldg.
Sensors: CSMIP 58364

Lat: 37.907°
Long.: 122.065°

Floor Levels: 10
Bsmt. Levels: 0
Structure: Concrete shear walls and frames

Foundation: mat
Site Geology: rock
Design Date: 1970

Recorded Peak Accelerations (g)

Earthquake	Ground Floor			Roof		
	NS	EW	V	NS	EW	V
1989 Loma Prieta	0.098	0.044	0.052	0.21	0.17	--

-- = no instrument

Preprocessing

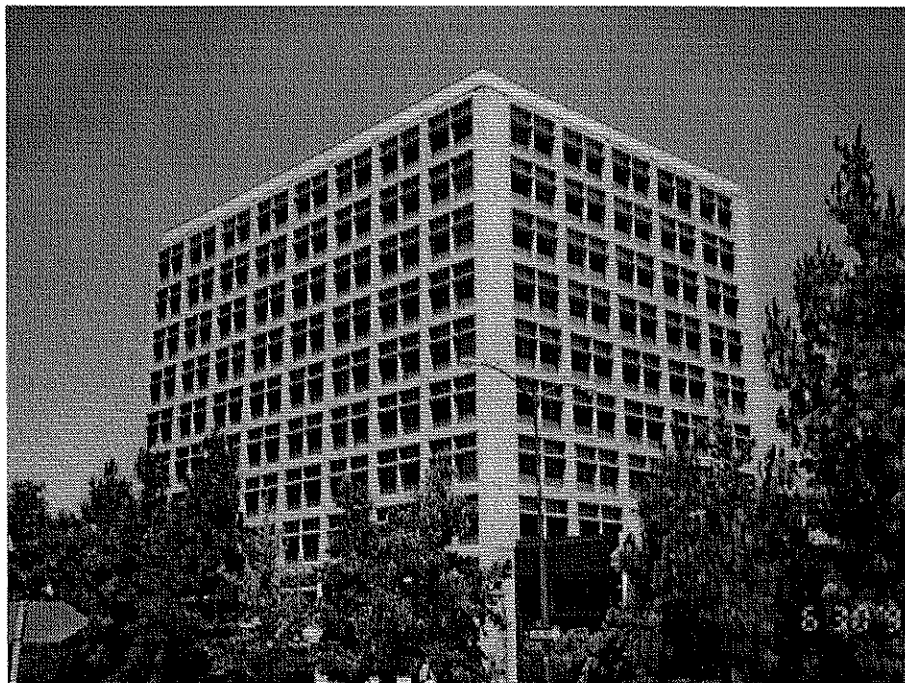
Loma Prieta Earthquake (LP)

Preliminary: CSMIP; filtered, $f = 0.3\text{--}23\text{ Hz}$; $f_{Nyq} = 25\text{ Hz}$

Final: decimated by 2 to $f_{Nyq} = 12.5\text{ Hz}$

Figures

B7.1-B7.2: Sensor Locations, Generalized Soil Column
B7.LP1-3 LP Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The building is 10 stories tall above ground level with no basement and has a rectangular shape in plan (104 by 148 feet). Lateral loads are resisted by a series of shear walls in the central core of the building, while columns around the building exterior carry vertical loads. The foundation for the central core consists of a 4-foot thick reinforced concrete mat, while the exterior columns are supported on belled concrete piers. Most of the accelerographs are located within the central core, though several lateral instruments are present at the north and south ends of the building. Ground floor vertical instrument are positioned within the central core to record rocking in the transverse (EW) direction.

Geologic exploration near the site has been conducted by Harding, Miller, Lawson and Assoc. (1970) and Raney Geotechnical (1983), and seismic refraction surveys were performed as part of the later study. Subsurface conditions near the central core of the building consist of about 10 feet of sandy and clayey soils overlying weathered siltstone and sandstone bedrock which extends to the maximum depth explored of 55 feet. Shear wave velocities in rock were estimated from the results of the geophysical surveys.

Instrument Selection and Free-Field Conditions

Instruments in the central core of the building were used in the analyses, as indicated by Table B7.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B7.LP1. The two vertical instruments located at the ground floor level in the central core were used to evaluate rocking effects in the EW direction.

Table B7.1: Sensors in structure used for analyses

Ground Floor (GF)			Roof (R)	
NS	EW	V	NS	EW
Ch 14	Ch 16	Ch 12,13	Ch 5	Ch 3

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch16} + (\text{Ch12} - \text{Ch13}) \cdot 128.5/24 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking effects is made in Fig. B7.LP1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are in phase with the relative roof displacements and contribute a large fraction of the relative roof displacement amplitude. Though the $R \times H$ displacements also contain relatively

long period energy, these results indicate that base rocking significantly influenced the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B7.LP2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B7.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B7.LP3).

Table B7.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (GF-R)				Flexible-Base			
		tr (EW) d=0, j=2		L (NS)		tr (EW) d=0, j=2		L (NS) d=0, j=3		tr (EW) estimated		L (NS)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LP	1	1.51	12.4	na	na	1.30	6.1	1.73	6.8	1.29	4.5	na	na
	2	5.89	19.4			5.44	10.2	5.30	17.3				
	3	--	--			--	--	7.01	16.0				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response for the transverse direction is dominated by the first mode.
- The minor difference between the flexible- and pseudo flexible-base frequencies indicates a minor SSI effect associated with base translation. Base rocking was more significant, as indicated by the modest difference between the first-mode frequencies for the fixed- and pseudo flexible-base cases and the results in Part (a).

The RPEM results (Fig. B7.LP3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B7.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 48 feet using ground floor motions recorded near the west wall of the central core and the profile in Fig. B7.2. Since the foundation for the central core of the structure is uncoupled from the remainder of the foundation and the instrumentation used in the system identification analyses are located in the central core, the footprint area for the mat foundation beneath the core was used for calculation of foundation radii. No adjustments to the impedance function were made for foundation non-rigidity effects, though rocking radiation damping was increased by a factor of 2.0 to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B7.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1425$ fps	$h=89$ ft	emb.=14 ft	$\gamma = 0.15$
LP	$\nu=0.4$		tr: $r_1 = 32$ ft		$r_2 = 26$ ft
	$\beta^*=1.2\%$	$V_s^*=1405$ fps	L: $r_1 = --$		$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B7.3 with the fixed-base modal parameters in Table B7.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table B7.4.

Table B7.4: Empirical and predicted inertial interaction effects

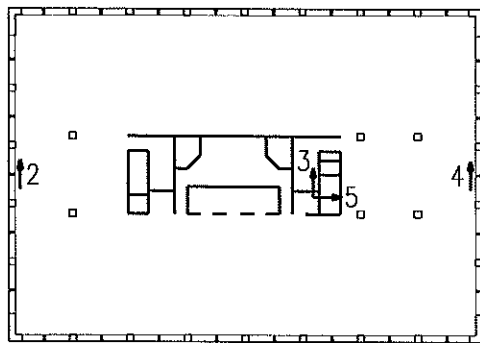
LP	Sys. ID	tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.17	-3.3	4.5	--	--	--
	Pre.-Vel.	1.05	0.1	10.8	--	--	--
	Pre.-Bie.	1.05	0.4	11.1	--	--	--

The period lengthening for this structure is significantly underpredicted by both formulations. The negative foundation damping result from the system identification analysis indicates a numerical error which is most likely associated with the estimated flexible-base damping. Nonetheless, both formulations predicted reasonably well the small foundation damping at this site.

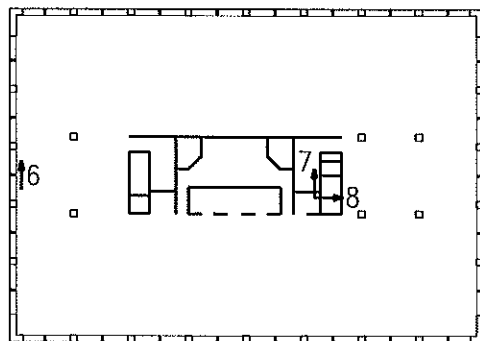
Data Sources

Harding, Miller, Lawson, and Associates (1970). "Foundation Investigation, Walnut Creek Office Building, for Dillingham Corp., Walnut Creek, CA," *Proj. No. 3008.002.04*.
 Raney Geotechnical (1983). "Foundation Investigation, North Main Centre, North Main Street and Pringle Avenue, Walnut Creek, CA," Job No. 029-005.

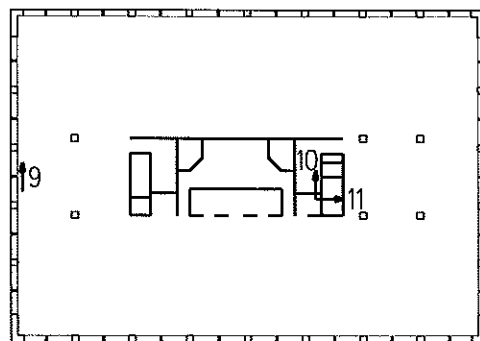
Interior core concrete shear walls and
 exterior concrete frame.
 Design Date: 1970



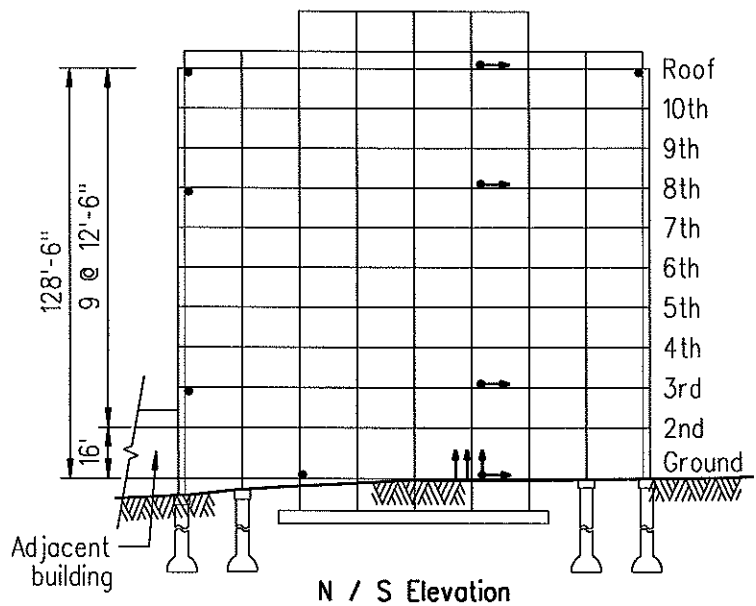
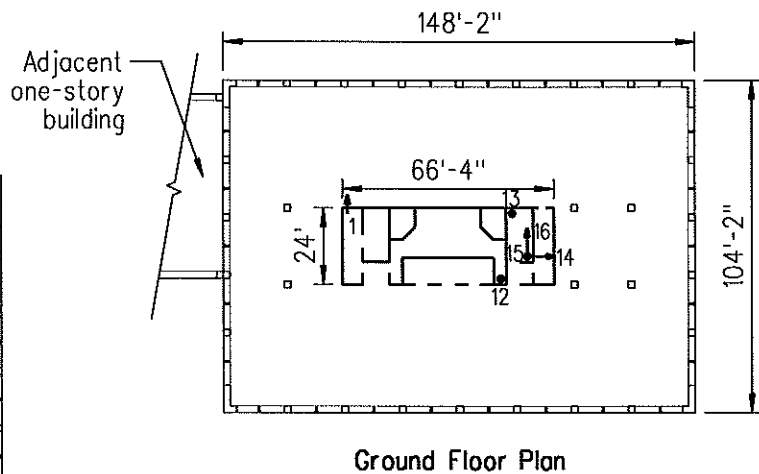
Roof Plan



8th Floor Plan



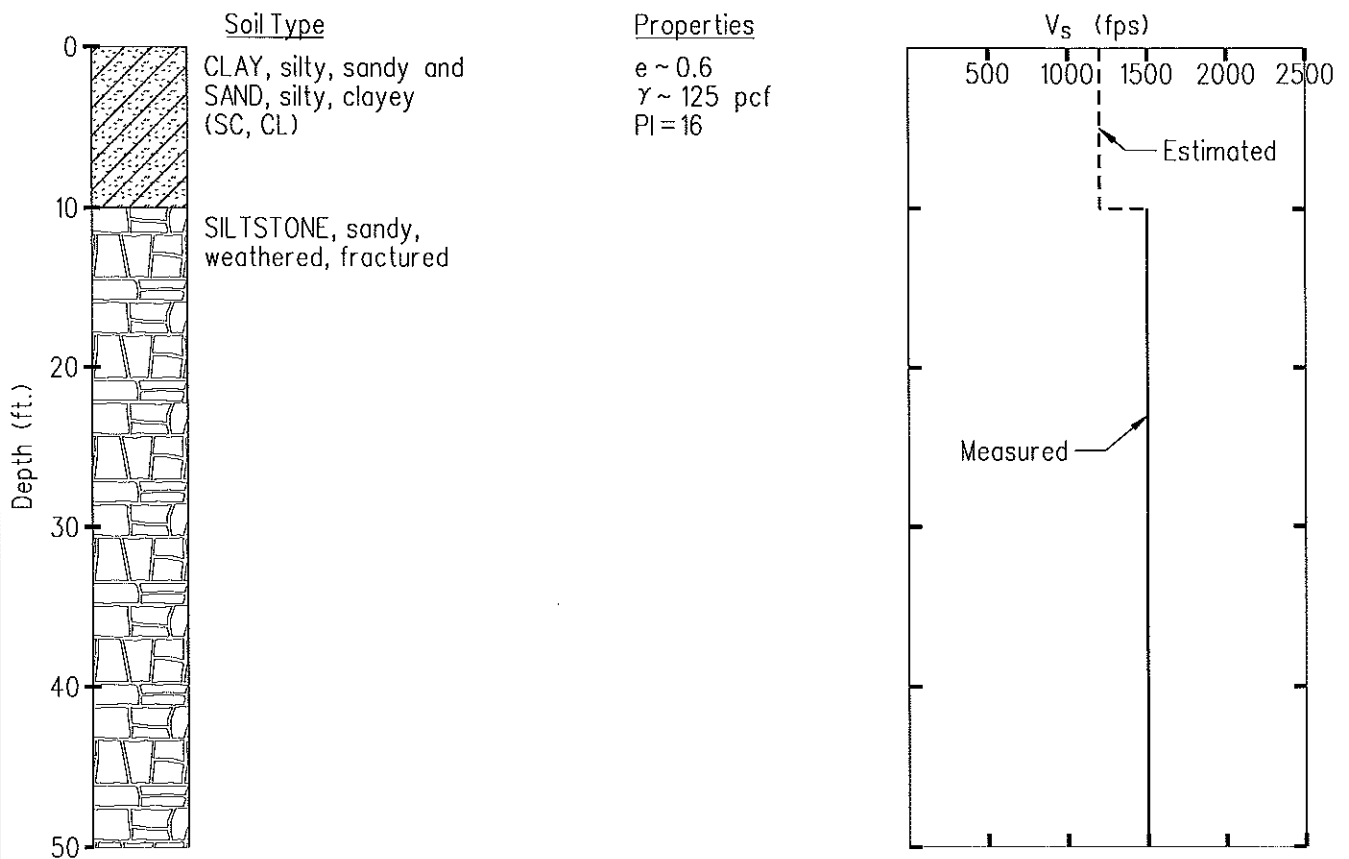
3rd Floor Plan



Ground Floor Plan



Sensor Locations
10-story Commercial Building Walnut Creek, CA
Figure B7.1



Data Source:
Harding, Miller, Lawson
& Assoc. (1970)

Notes:

1. Groundwater encountered at 20-40 ft. depth.
2. Shear wave velocities in soil estimated based on correlations by Fumal & Tinsley (1985).
Shear wave velocities in rock based on seismic refraction data by Raney Geotechnical (1983).

Generalized Soil Profile
10-story Commercial Building Walnut Creek, CA
Figure B7.2

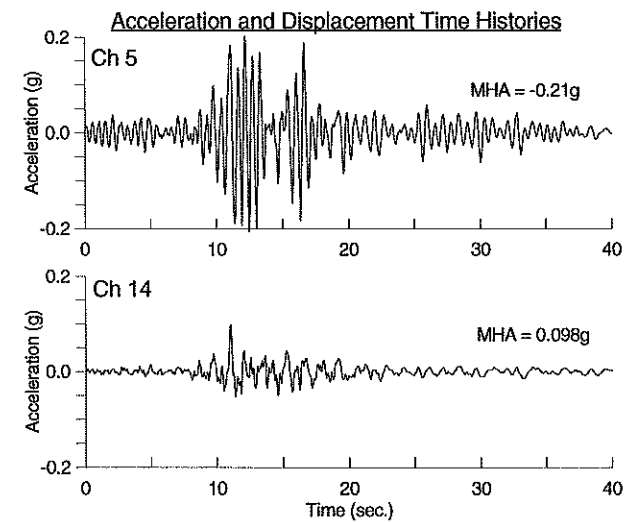
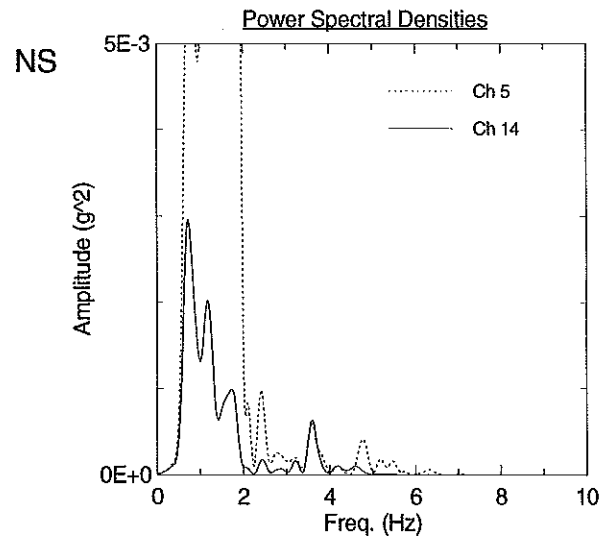
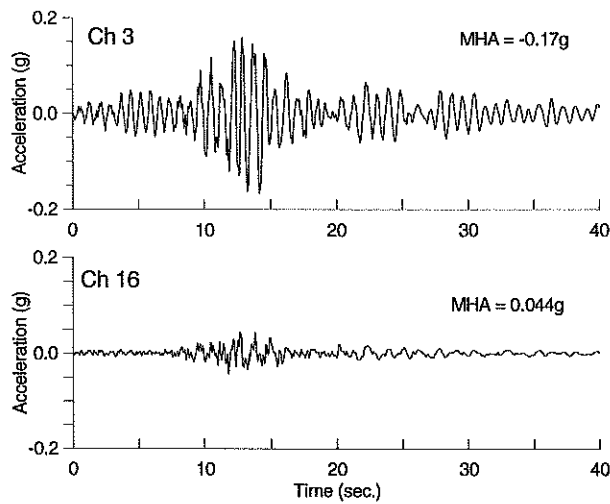
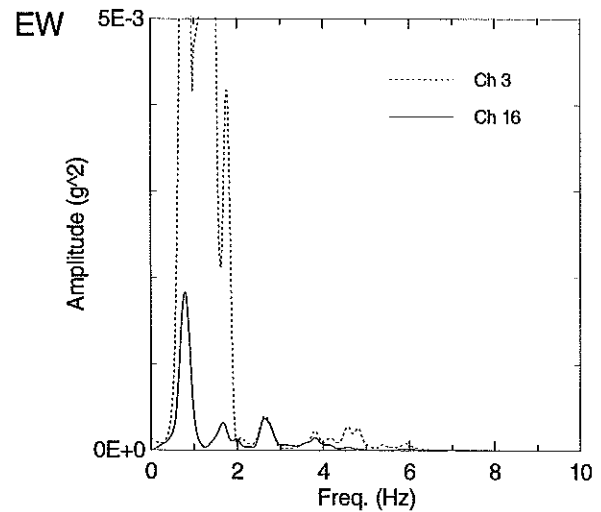
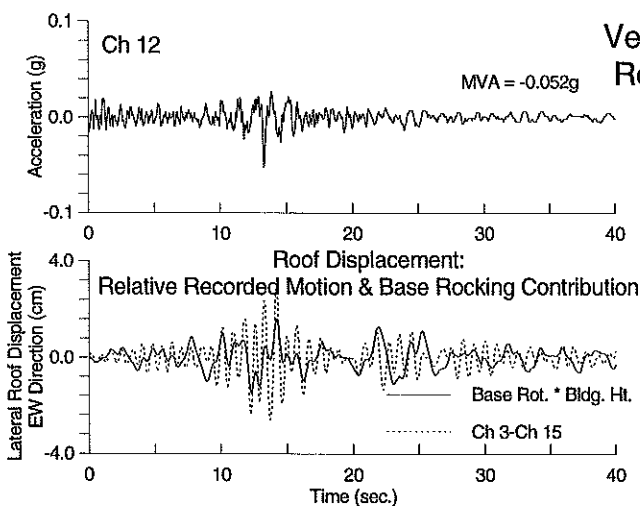
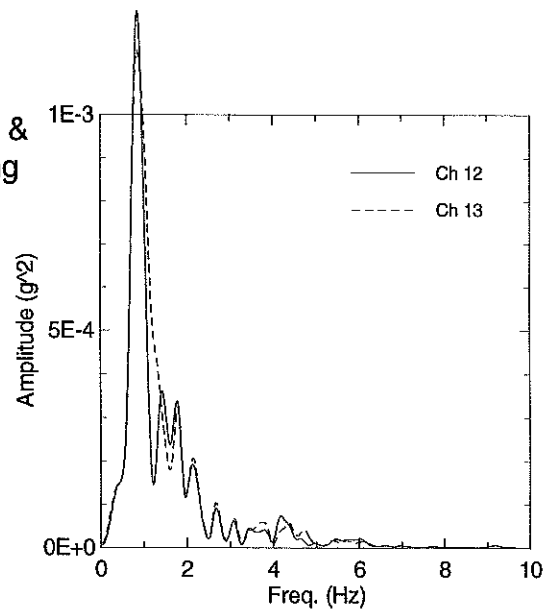


Fig. B7.LP1: WALNUT CREEK 10 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1989 Loma Prieta Earthquake

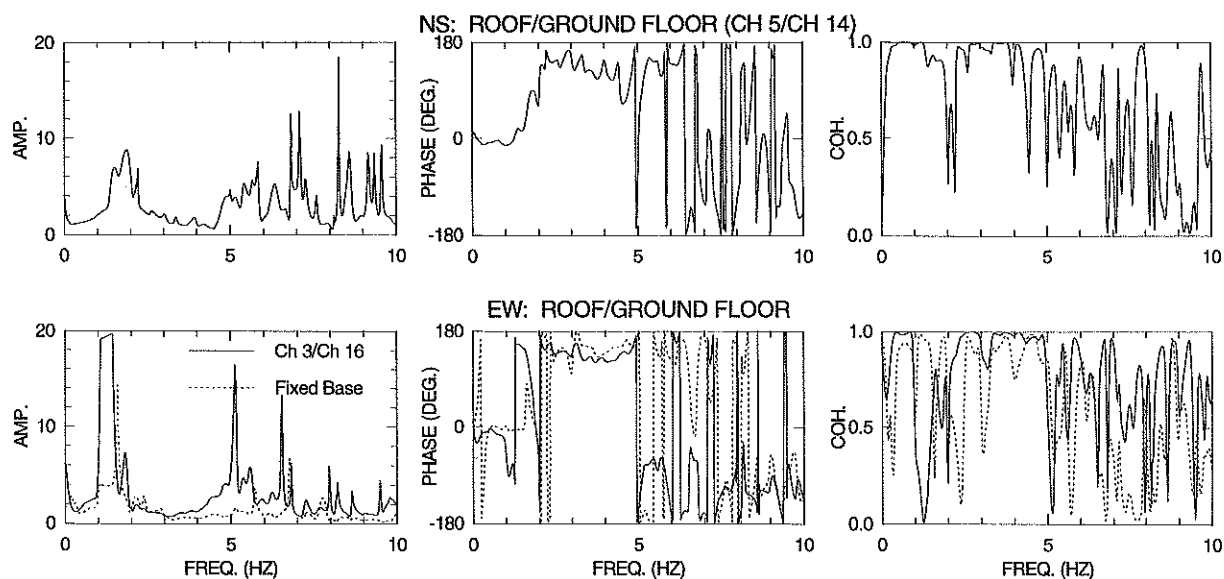


Fig. B7.LP2: WALNUT CREEK 10 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1989 Loma Prieta Earthquake

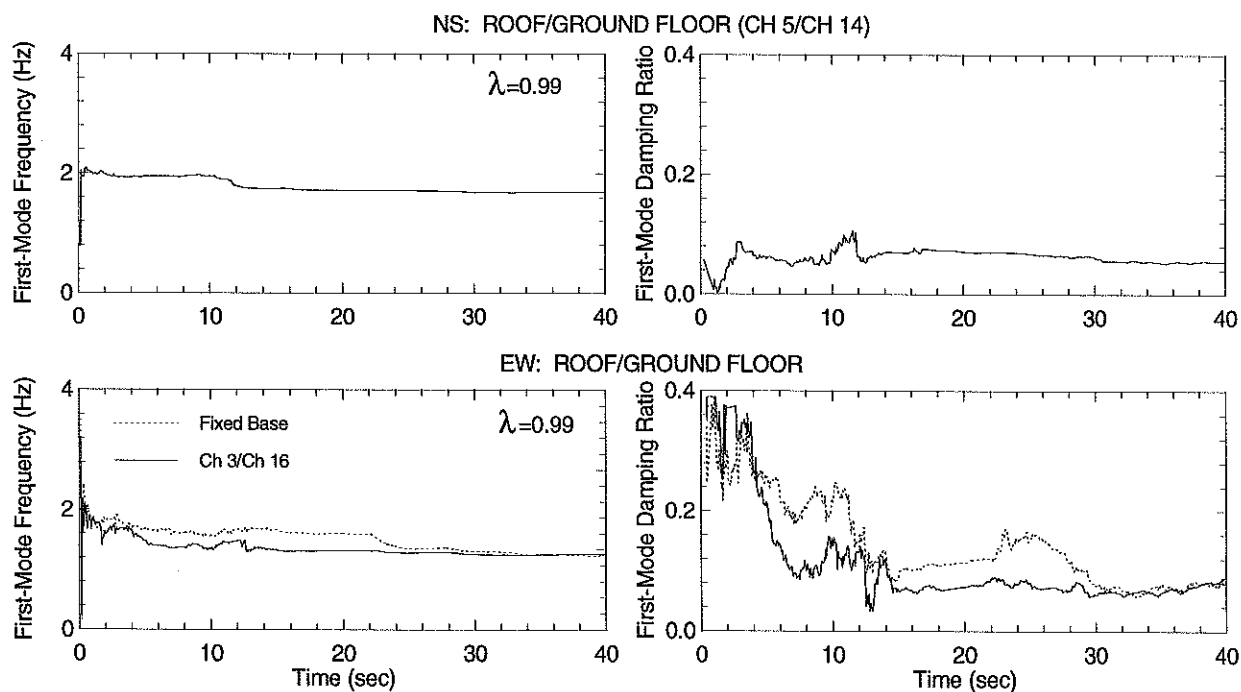


Fig. B7.LP3: WALNUT CREEK 10 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1989 Loma Prieta Earthquake

SITE B9

Site Name: El Segundo 14 Story Offc. Bldg.
Sensors: CSMIP 14654

Lat: 33.920°
Long.: 118.390°

Floor Levels: 14
Bsm't. Levels: 0
Structure: steel moment and braced frames

Foundation: grade beams
Site Geology: soil
Design Date: 1985

Recorded Peak Accelerations (g)

Earthquake	1st Floor			Roof		
	NS	EW	V	NS	EW	V
1994 Northridge	0.091	0.13	0.044	0.25	0.21	0.15

-- = no instrument

Preprocessing

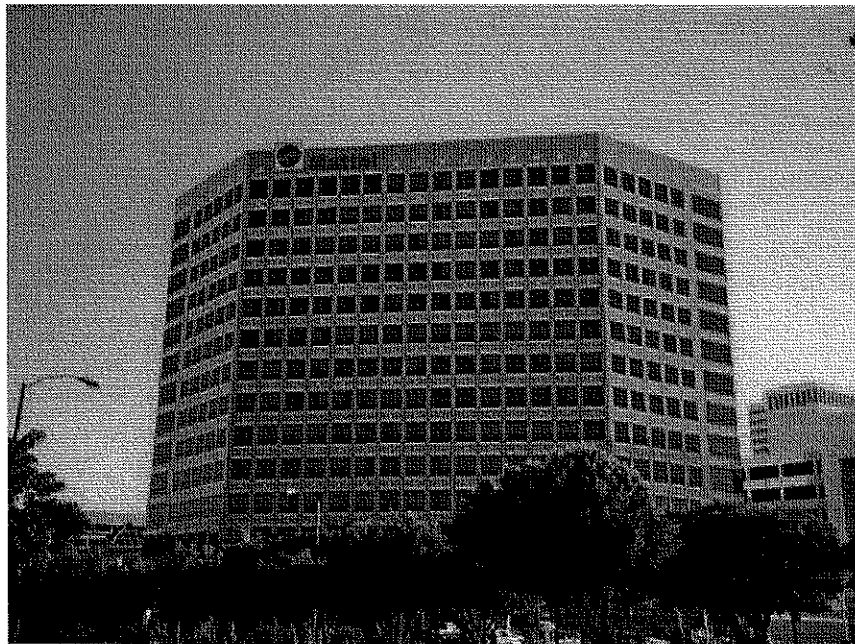
Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$

Final: decimated by 4 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Figures

B9.1-B9.2: Sensor Locations, Generalized Soil Column
B9.NR1-3 NR Analysis Results



View of structure, looking in the reference west direction

DISCUSSION

Site Description

The building is 14 stories tall above ground level with no basement, and has an irregular shape with maximum plan dimensions of about 177.5 x 177.5 feet. As shown in Fig. B9.1, lateral loads are resisted by a rectangular moment frame with dimensions 101 x 150 feet and two interior braced frames with dimensions of 27 x 30 feet. The foundation consists of a grid of 8.5-foot deep reinforced concrete grade beams beneath the columns in both directions. Lateral sensors are located on the 1st, 4th, 9th, and roof levels, and 1st floor vertical instruments are positioned on the east and west ends of the steel moment frame to detect rocking in the transverse (EW) direction.

Subsurface exploration by LCA (1983) encountered 10 feet of surficial silty sands overlying fine sands which extend to the maximum depth explored of 75 feet. The shear wave velocities shown in Fig. B9.2 are based on in situ downhole measurements by LCA (1983).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B9.1; acceleration time histories recorded by some of these sensors are shown in Fig. B9.NR1. The two vertical instruments located at the 1st Floor level were used to evaluate rocking effects in the EW direction.

Table B9.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 5	Ch 3	Ch 1, 2	Ch 13	Ch 11

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch3} + (\text{Ch2} - \text{Ch1}) \cdot 188/97 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B9.NR1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are generally in phase with the relative roof displacements but have much smaller amplitude, indicating that base rocking had a small influence on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B9.NR2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B9.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B9.NR3).

Table B9.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flexible-Base			
		tr (EW) d=11, j=9		L (NS)		tr (EW) d=11, j=9		L (NS) d=7, j=8		tr (EW) estimated		L (NS)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	0.49	3.0	na	na	0.48	2.9	0.59	2.9	0.48	2.9	na	na
	2	1.59	3.9			1.58	4.0	1.86	4.4				
	3	2.87	4.4			2.83	5.0	3.29	7.6				
	4	4.15	6.8			4.15	6.4	4.78	9.5				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structure responded strongly in its first two modes in both directions.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.

The RPEM results (Fig. B9.NR3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed-base and 1F-R first-mode parameters.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B9.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 69 feet using 1st floor motions recorded near the southwest corner of the structure and the profile in Fig. B9.2. Only the footprint area of the rectangular moment frame structure was used for the calculation of foundation radii. No adjustments to the impedance function were made for embedment, foundation non-rigidity, or shape effects.

Table B9.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1010$ fps	$h = 114$ ft	emb.=0	$\gamma = 0.15$
NR	$\nu = 0.33$		tr:	$r_1 = 69$ ft	$r_2 = 64$ ft
	$\beta^* = 4.1\%$	$V_s^* = 900$ fps	L:	$r_1 = --$	$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B9.3 with the fixed-base modal parameters in Table B9.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” formulation, with the results in Table B9.4.

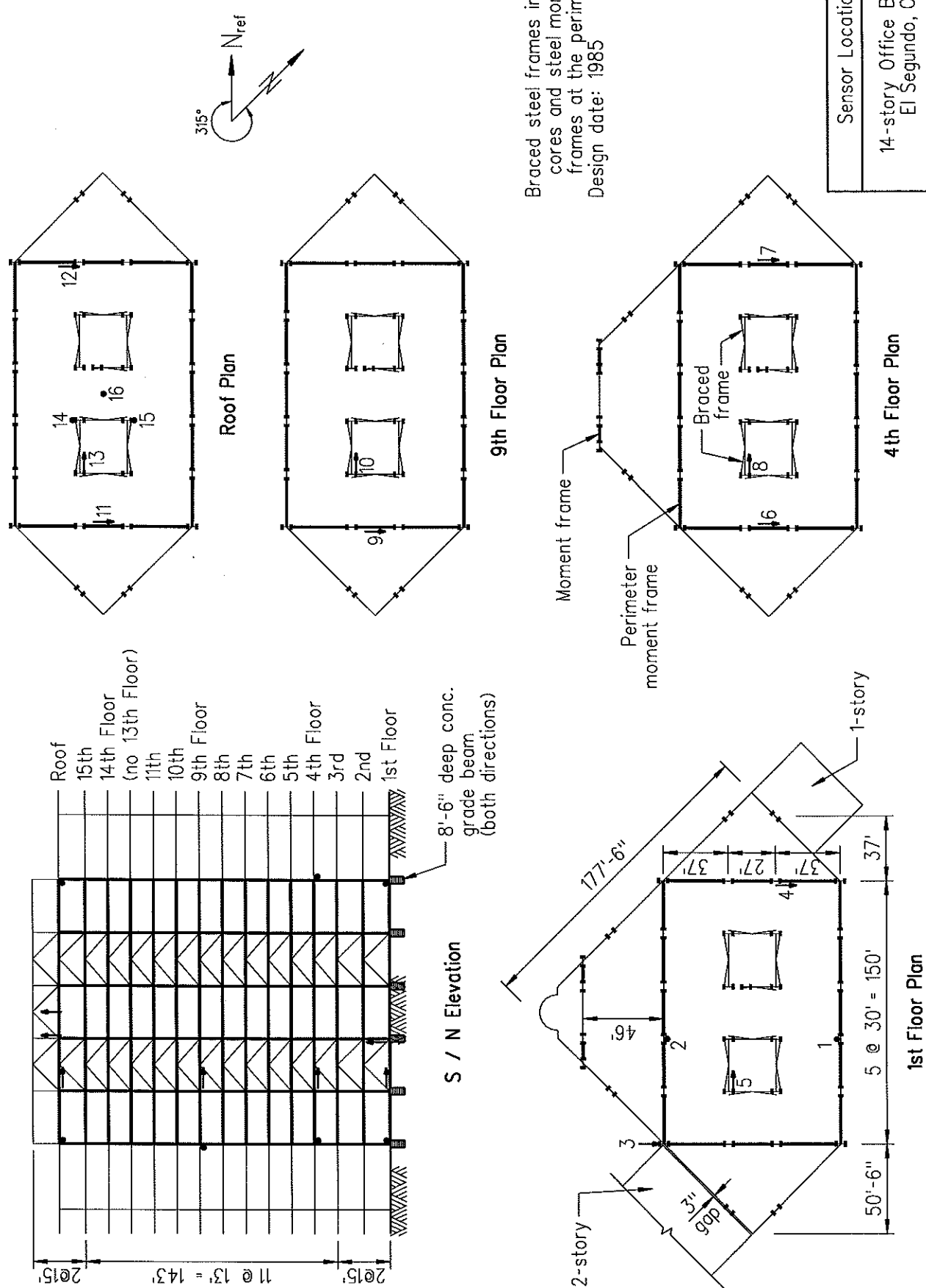
Table B9.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
NR	Sys. ID	1.02	0.0	2.9	--	--	--
	Prediction	1.02	0.1	3.0	--	--	--

The period lengthening and foundation damping were satisfactorily predicted for this site.

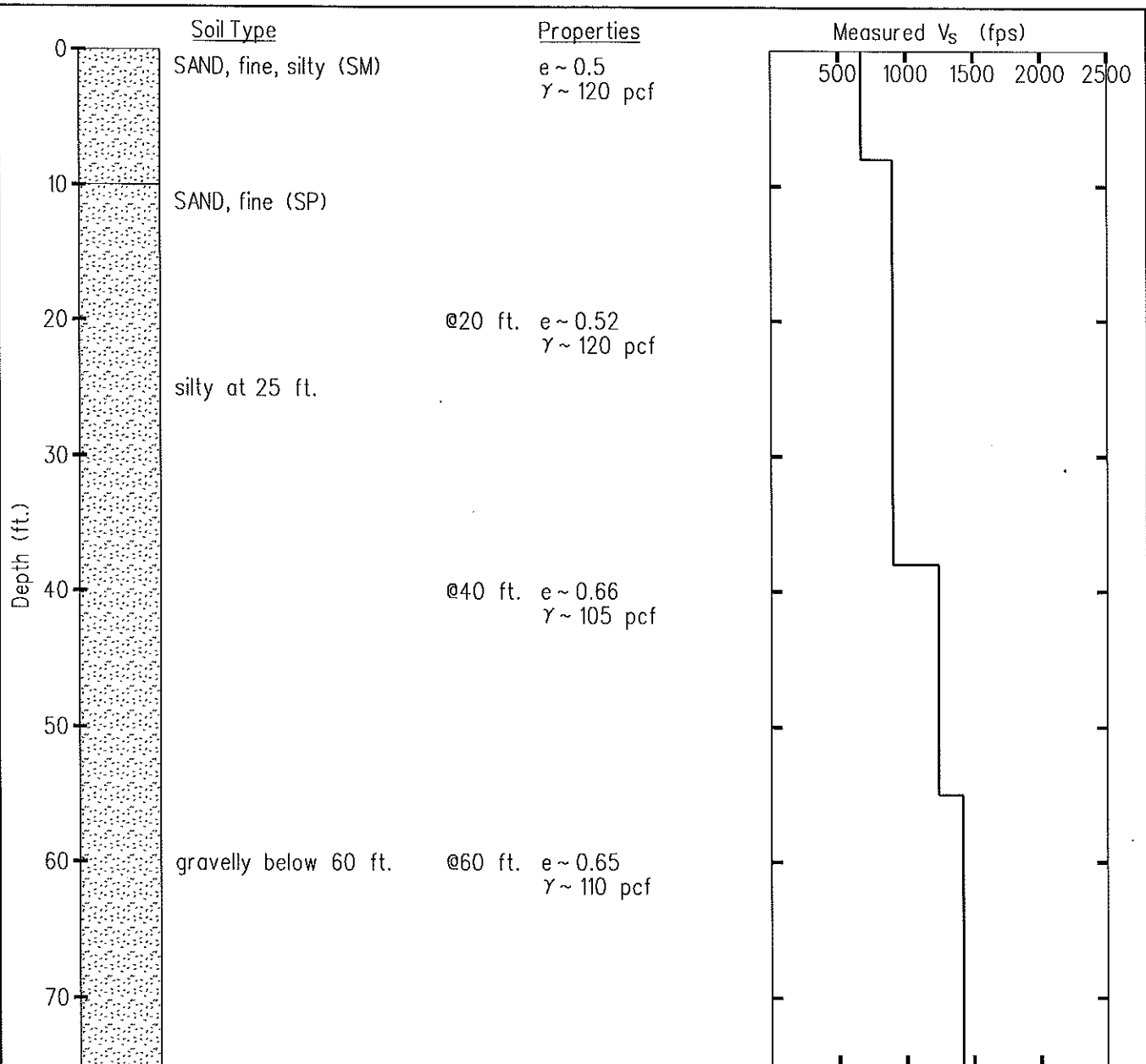
Data Sources

LeRoy Crandall and Assoc., LCA (1983). “Report of Geotechnical Investigation, Proposed ‘The Grand Way’ Development, Grand Avenue and Continental Blvd., El Segundo, CA,” *Job. No. AD-83307-B*.



Braced steel frames in the cores and steel moment frames at the perimeter.
Design date: 1985

Sensor Locations
14-story Office Building El Segundo, CA
Figure B9.1



Data Source:
LCA (1983)

Notes:

1. Groundwater not encountered.
2. Shear wave velocities based on downhole measurements by LCA (1983).

Generalized Soil Profile
14-story Office Building El Segundo, CA
Figure B9.2

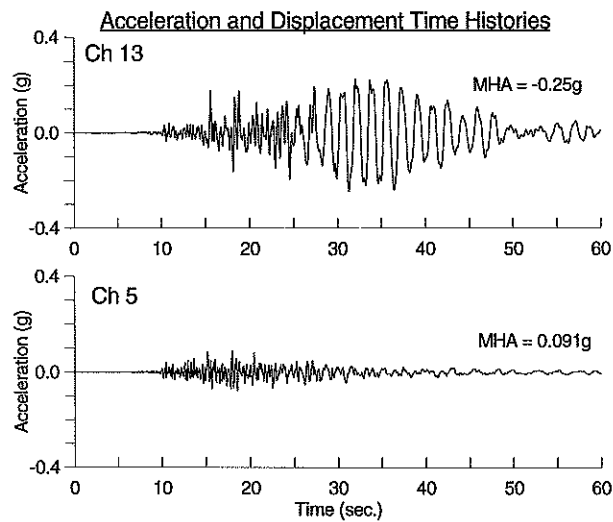
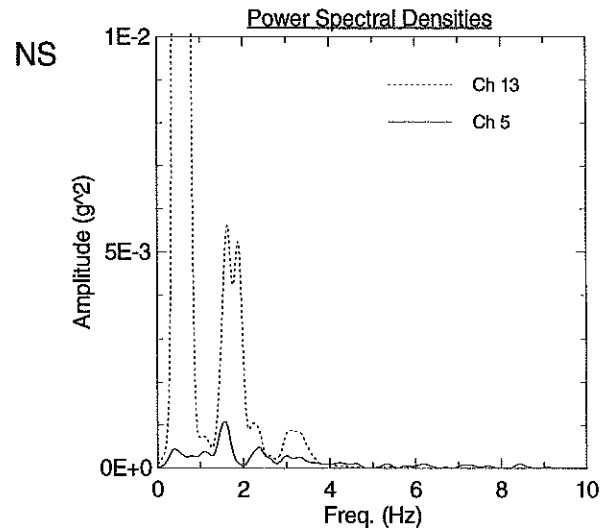
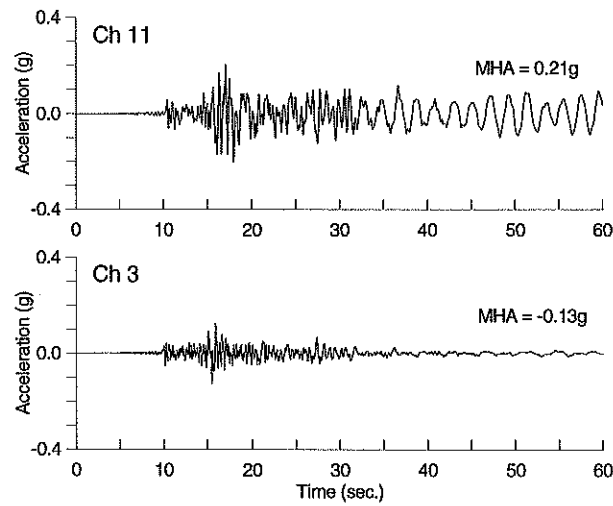
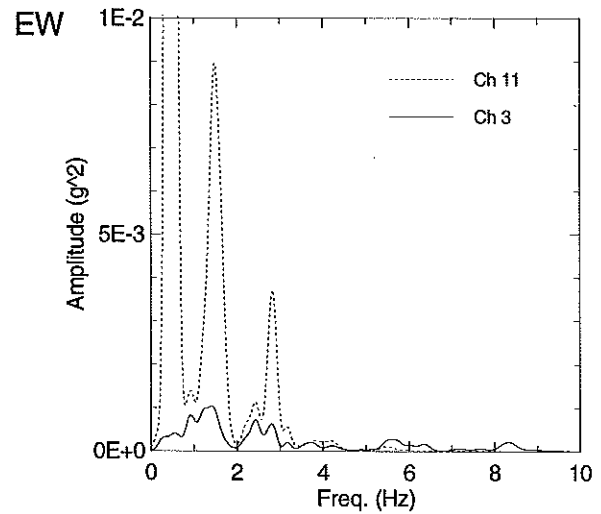
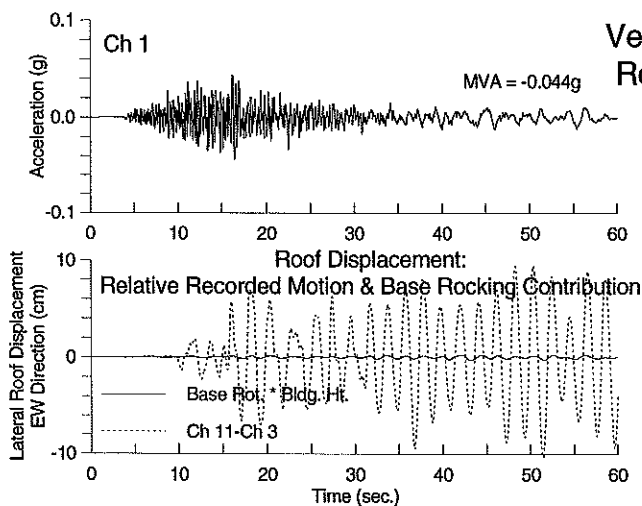
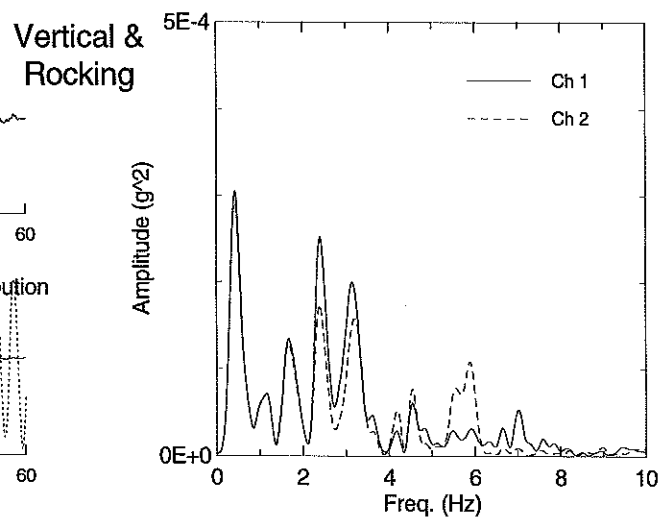


Fig. B9.NR1: EL SEGUNDO 14 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake

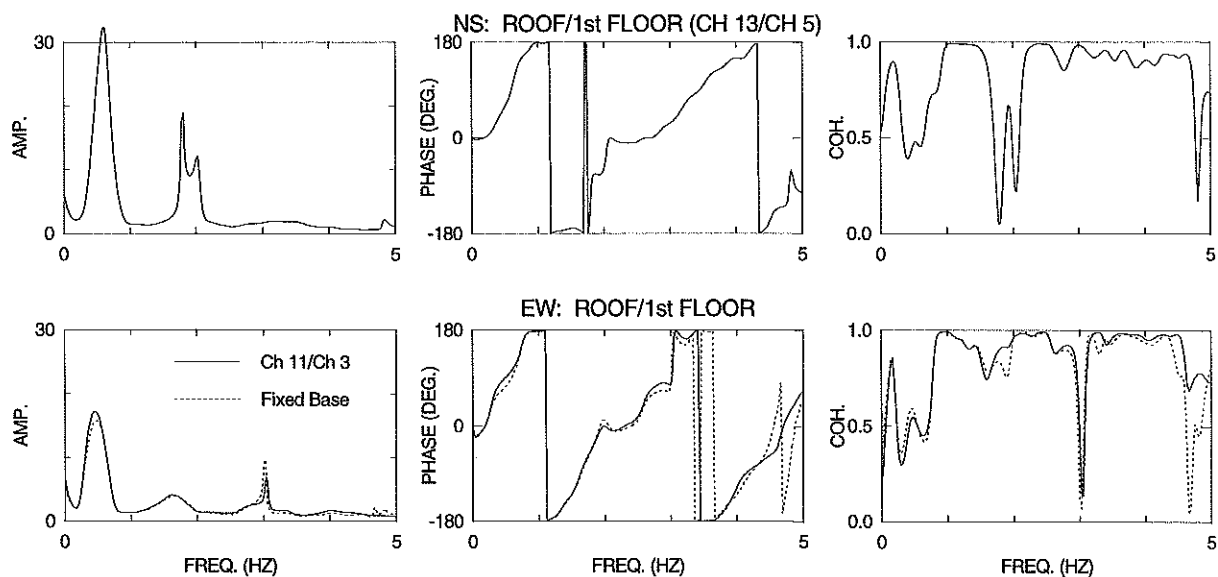


Fig. B9.NR2: EL SEGUNDO 14 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

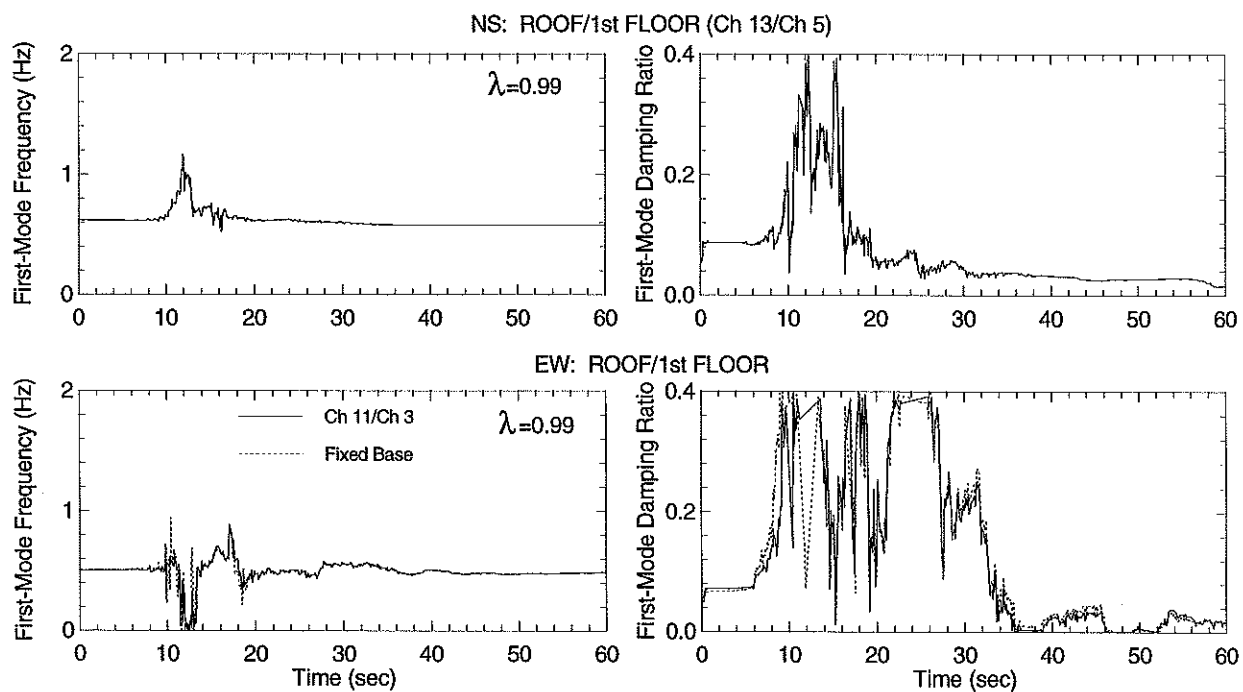


Fig. B9.NR3: EL SEGUNDO 14 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE B10

Site Name: LA 9 Story Offc. Bldg.
Sensors: CSMIP 24579

Lat: 34.044°
Long.: 118.261°

Floor Levels: 10

Bsmt. Levels: 1

Structure: concrete frames with perimeter masonry
infill walls

Foundation: footings

Site Geology: soil

Design Date: 1924

Recorded Peak Accelerations (g)

Earthquake	Basement			Roof		
	NS	EW	V	NS	EW	V
1994 Northridge	0.16	0.13	0.12	0.30	0.34	--

-- = no instrument

Preprocessing

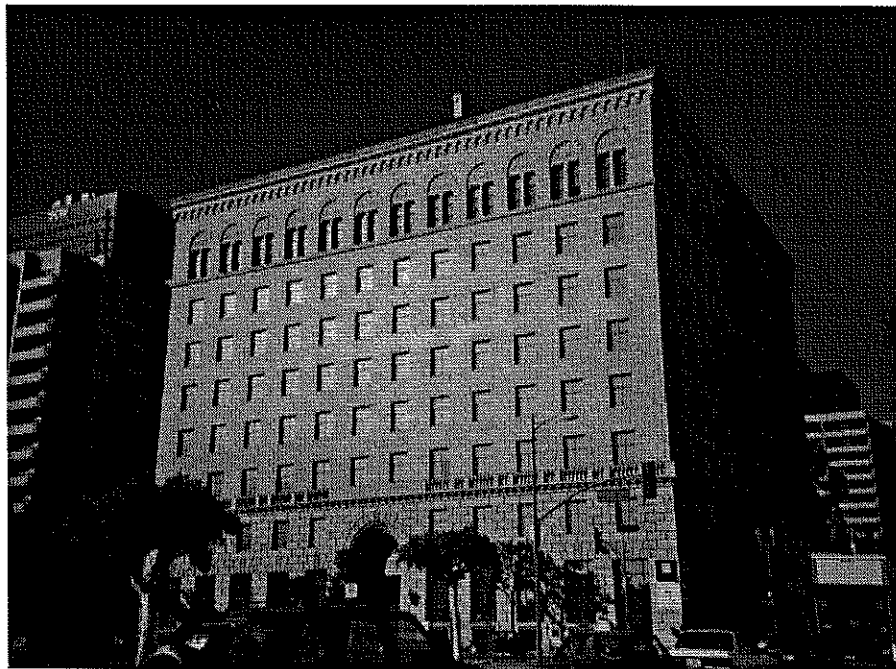
Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$

Final: decimated by 4 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Figures

B10.1-B10.2: Sensor Locations, Generalized Soil Column
B10.NR1-3 NR Analysis Results



View of structure, looking in the reference north direction

DISCUSSION

Site Description

The building is 9 stories tall above ground level with a single level basement, and has an L shape in plan, with each wing having side dimensions of 51 x 155 feet. Lateral loads are resisted by concrete frames with 13-inch wide perimeter masonry infill walls. Lateral sensors are present at the basement, 2nd, 5th, and roof levels, and vertical base-level instruments are present in the east wing of the structure to measure base rocking in the NS and EW directions.

Subsurface exploration by LCA (1981) at a nearby site encountered the following soil conditions: about 10 feet of silt and clay, 25 feet of gravely sand, 20 feet of interbedded silts and clays, and 20 feet of interbedded silts, clays and sands which extend to the maximum depth explored of 75 feet. The shear wave velocities shown in Fig. B10.2 are based on in situ downhole measurements by LCA (1981).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B10.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B10.NR1. The three vertical instruments located in the basement were used to evaluate rocking effects in the NS and EW directions.

Table B10.1: Sensors in structure used for analyses

Basement (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 6	Ch 4	Ch 1-3	Ch 17	Ch 14

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS and EW directions,

$$\begin{aligned}\text{input, ns} &= \text{Ch6} + (\text{Ch1} - \text{Ch2}) \cdot 141/155 \\ \text{input, ew} &= \text{Ch4} + (\text{Ch2} - \text{Ch3}) \cdot 141/51\end{aligned}\tag{1}$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B10.NR1 by comparing relative roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are in phase with the relative roof displacements but have a much smaller amplitude, indicating that base rocking had a small influence on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B10.NR2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B10.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B10.NR3).

Table B10.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flexible-Base			
		tr (EW) d=5, j=8		L (NS) d=2, j=5		tr (EW) d=6, j=8		L (NS) d=2, j=5		tr (EW) estimated		L (NS) estimated	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	0.78	14.2 11.5	1.01	13.4 14.4	0.77	12.8	0.99	14.1	0.77	15.1	0.98	15.2
	2	2.40	13.5	3.26	14.2	2.39	12.9	3.23	11.1				
	3	4.71	10.8	5.67	9.9	4.42	13.1	5.67	9.3				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode
- As discussed below, the EW and NS CEM ζ values of 14.2% and 13.4% were adjusted based on RPEM results.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.

The RPEM results (Fig. B10.NR3) indicate reductions in the transverse first-mode frequency from about 0.9 to 0.7 Hz during the first 30 seconds of strong shaking. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode frequencies. However, there was a discrepancy between the RPEM and CEM results with regard to the relative damping ratios. Based on the RPEM results during the most stable portion of the time history (20 to 60 seconds), ζ values deviate from $\tilde{\zeta}^*$ values by absolute differences of +0.3% and -1.4% for the NS and EW directions, respectively (i.e. as indicated in Table B10.2, better estimates are $\zeta=11.5\%$ and 14.4%).

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B10.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 50 feet using basement motions recorded near the southeast corner of the structure and the profile in Fig. B10.2. For the transverse (EW) direction, only the footprint area of the east wing of the structure (51 x 155 feet) was used for the calculation of foundation radii (since the instrumentation for EW response is located a significant distance from the south wing). For the longitudinal (NS) direction, the footprint area of the whole foundation was used to compute r_1 and only the east wing was used to compute r_2 . No adjustments to the impedance function were

made for foundation non-rigidity, though rocking radiation damping values for the transverse and longitudinal directions were increased by factors of 2.0 and 2.5 to account for shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B10.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 960$ fps	$h = 89$ ft	emb. = 13 ft	$\gamma = 0.15$
NR	$\nu = 0.4$		tr: $r_1 = 50$ ft		$r_2 = 38$ ft
	$\beta^* = 3.7\%$	$V_s^* = 880$ fps	L: $r_1 = 65$ ft		$r_2 = 67$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B10.3 with the fixed-base modal parameters in Table B10.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table B10.4.

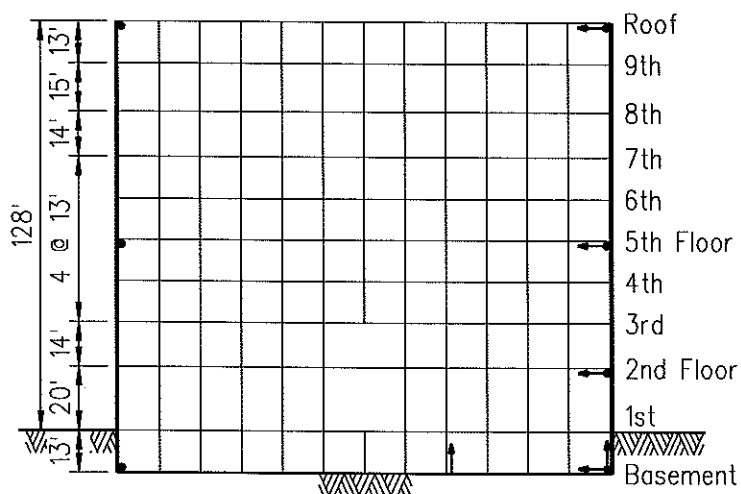
Table B10.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.02	4.2	15.1	1.03	2.0	15.2
	Pre.-Vel.	1.04	0.1	10.4	1.03	0.4	13.4
	Pre.-Bie.	1.03	0.4	11.0	1.04	0.9	13.6

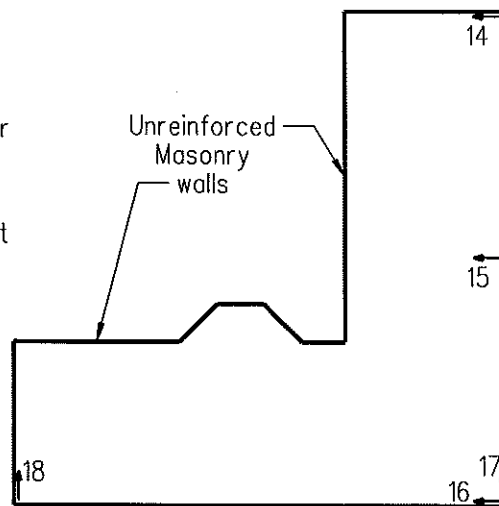
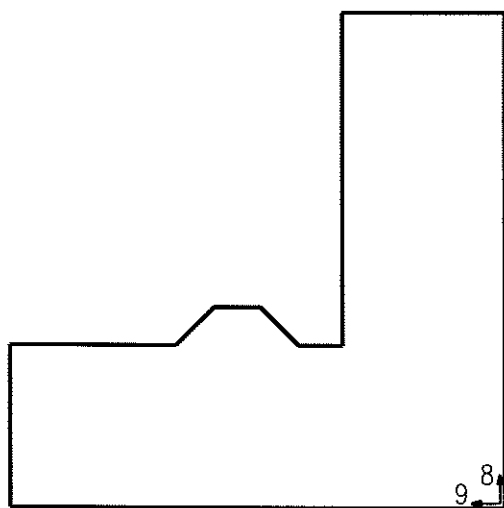
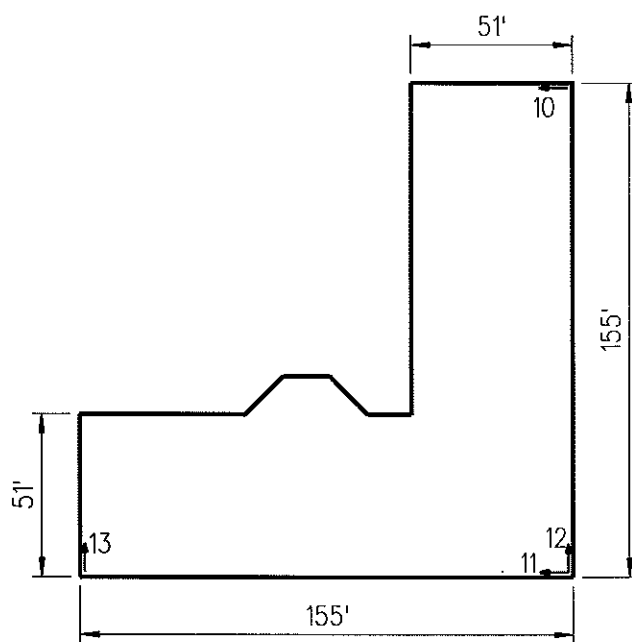
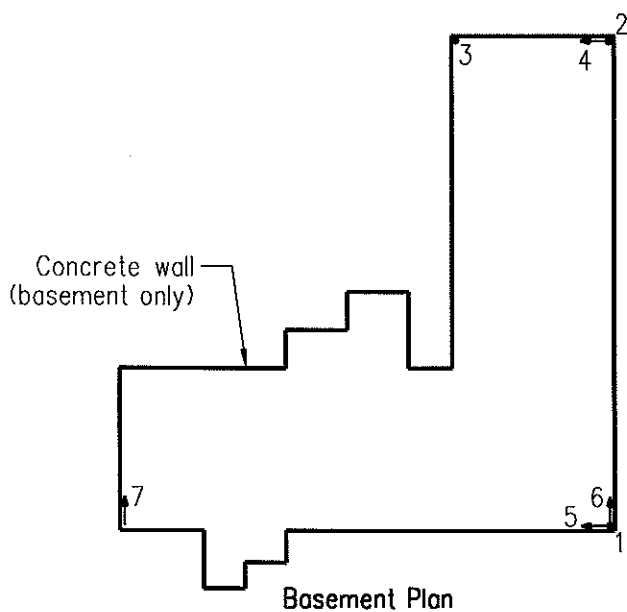
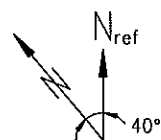
The period lengthening was satisfactorily predicted by both formulations. However, the foundation damping was underpredicted, with the “modified Bielak” results being the least erroneous. There are some uncertainties in the system identification results for foundation damping reported in Table B10.4 which are associated primarily with the previously noted discrepancies between the CEM and RPEM results and the characterization of the structure for the predictive analyses (e.g. the selection of r_1 and r_2 for the two directions). In consideration of these factors, the discrepancies between the damping results are not surprising.

Data Sources

LeRoy Crandall Assoc., LCA (1981). “Report of Geotechnical Investigation, Proposed PDC Office Building and Parking Structure, 8th Street and Grand Avenue, Los Angeles, CA,” *Job. No. ADE-81341*.

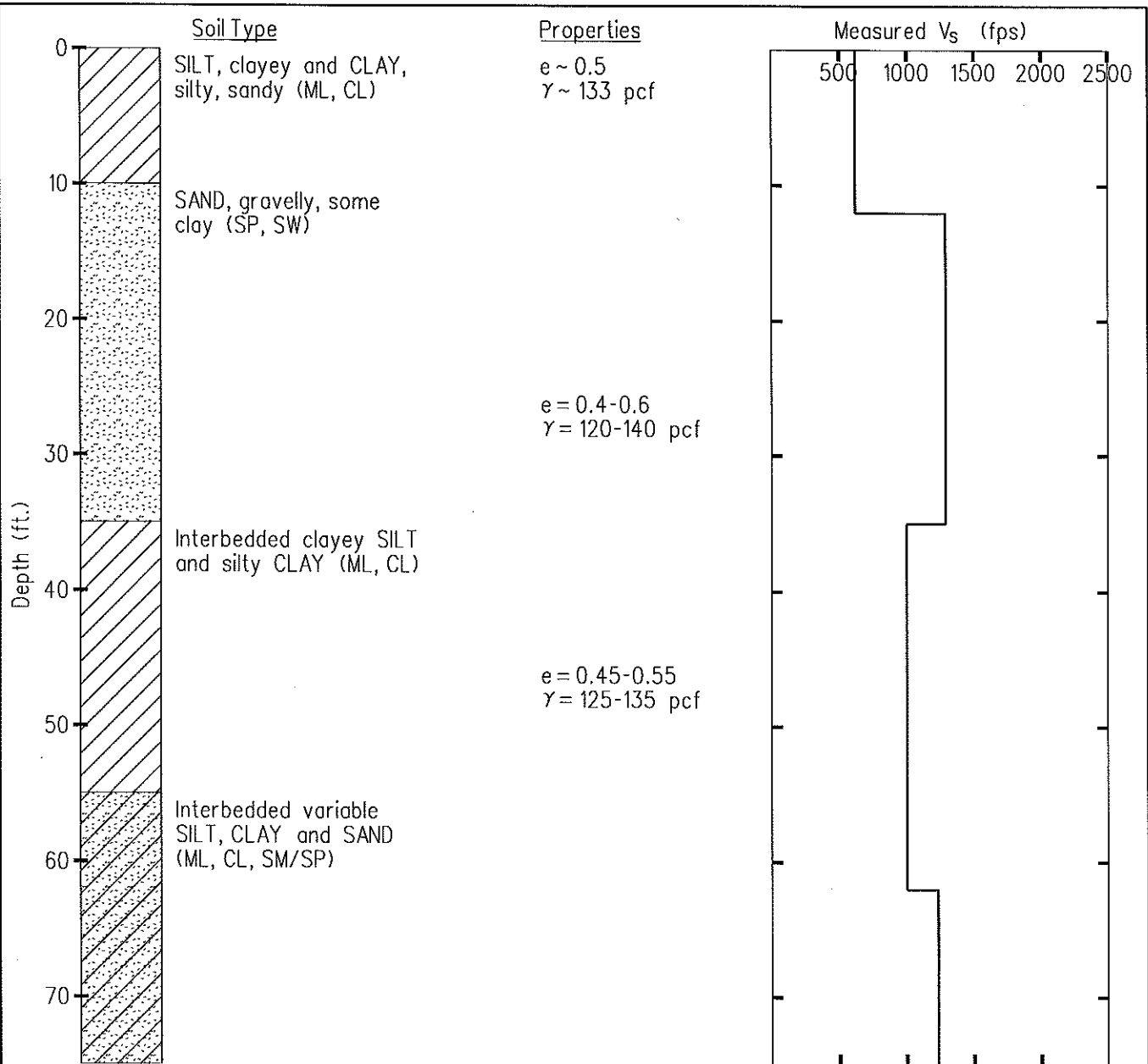


Concrete frames with URM infill walls.
Design date: 1923



Sensor Locations
9-story Office Building Los Angeles, CA
Figure B10.1

la9.dgn[60]

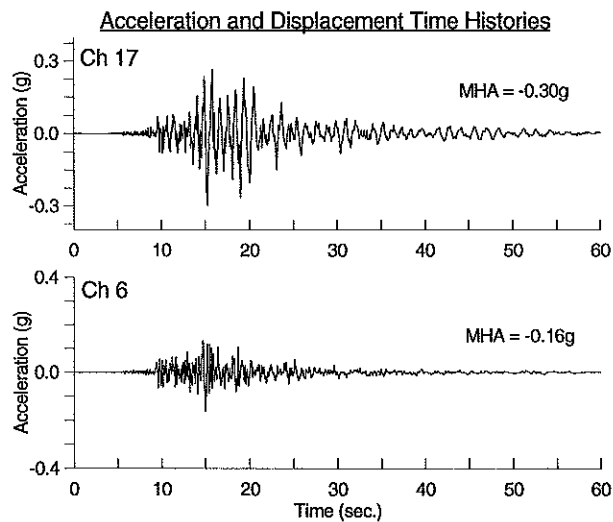
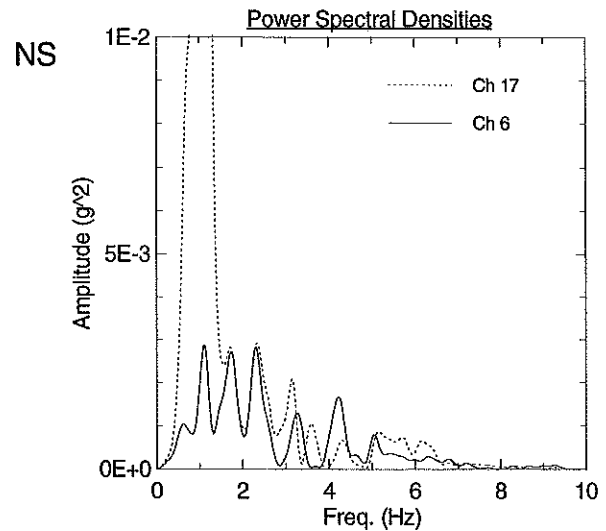
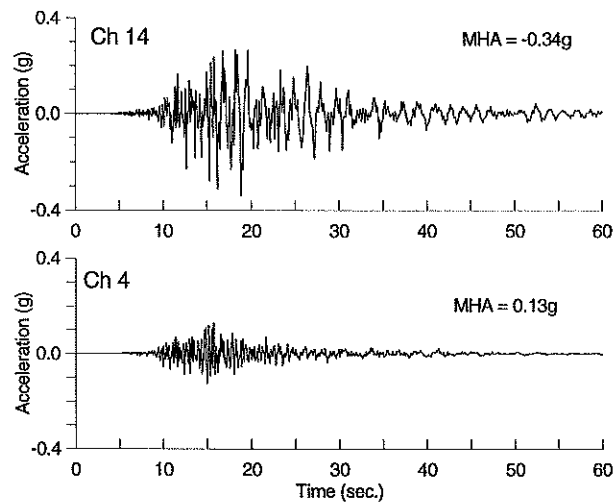
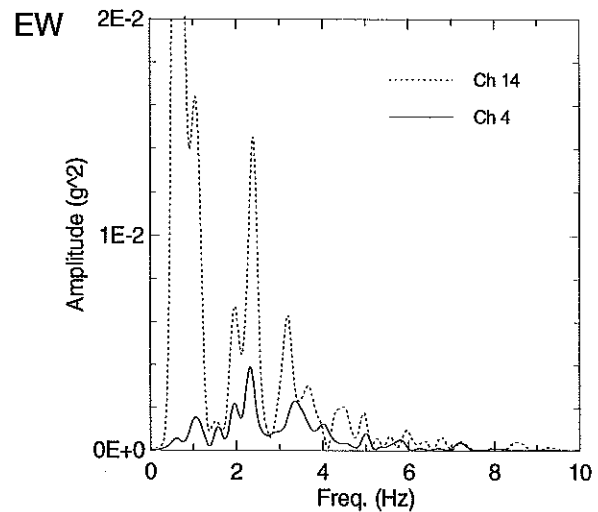
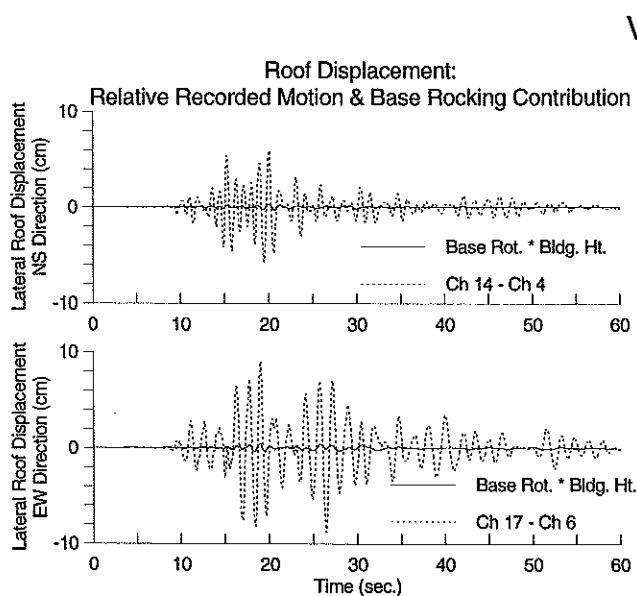


Data Source:
LCA (1981)

Notes:

1. Groundwater not encountered.
2. Shear wave velocities based on downhole measurements by LCA (1981).

Generalized Soil Profile
9-story Office Building Los Angeles, CA
Figure B10.2



Vertical & Rocking

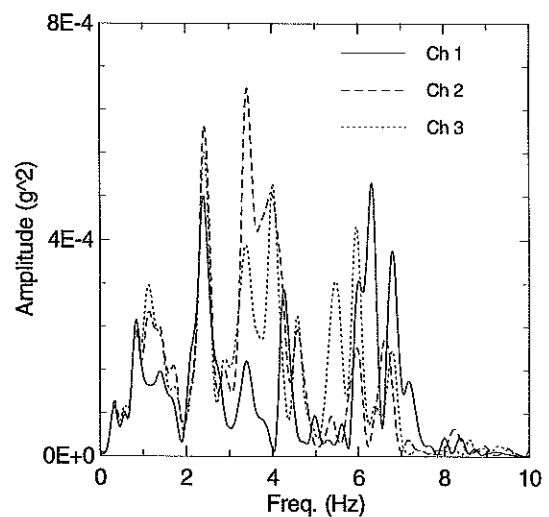


Fig. B10.NR1: LA 9 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake

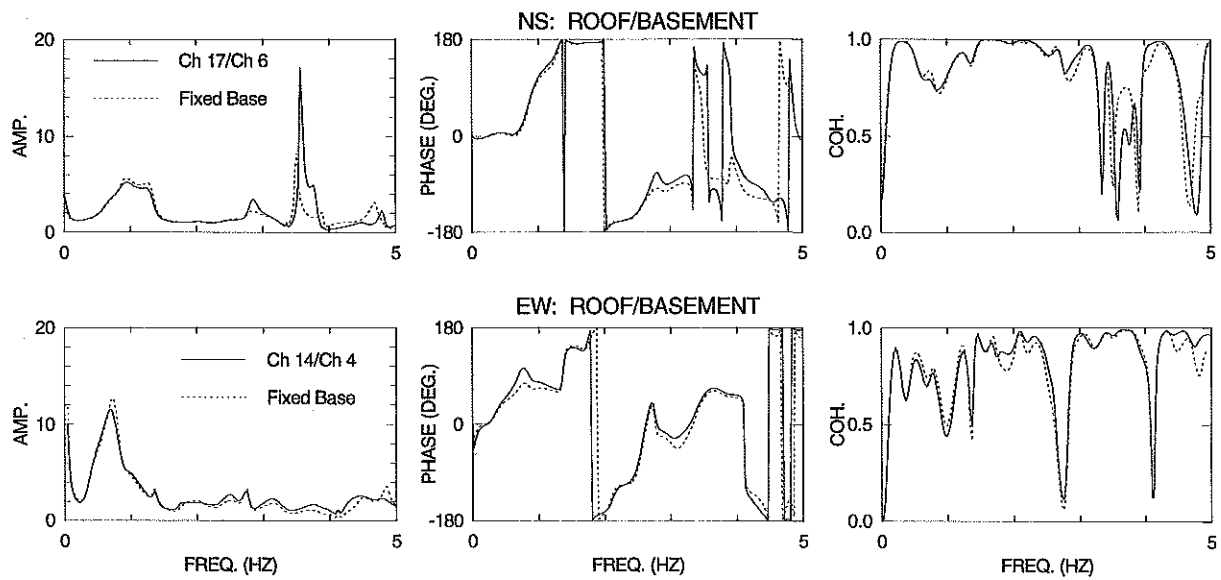


Fig. B10.NR2: LA 9 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

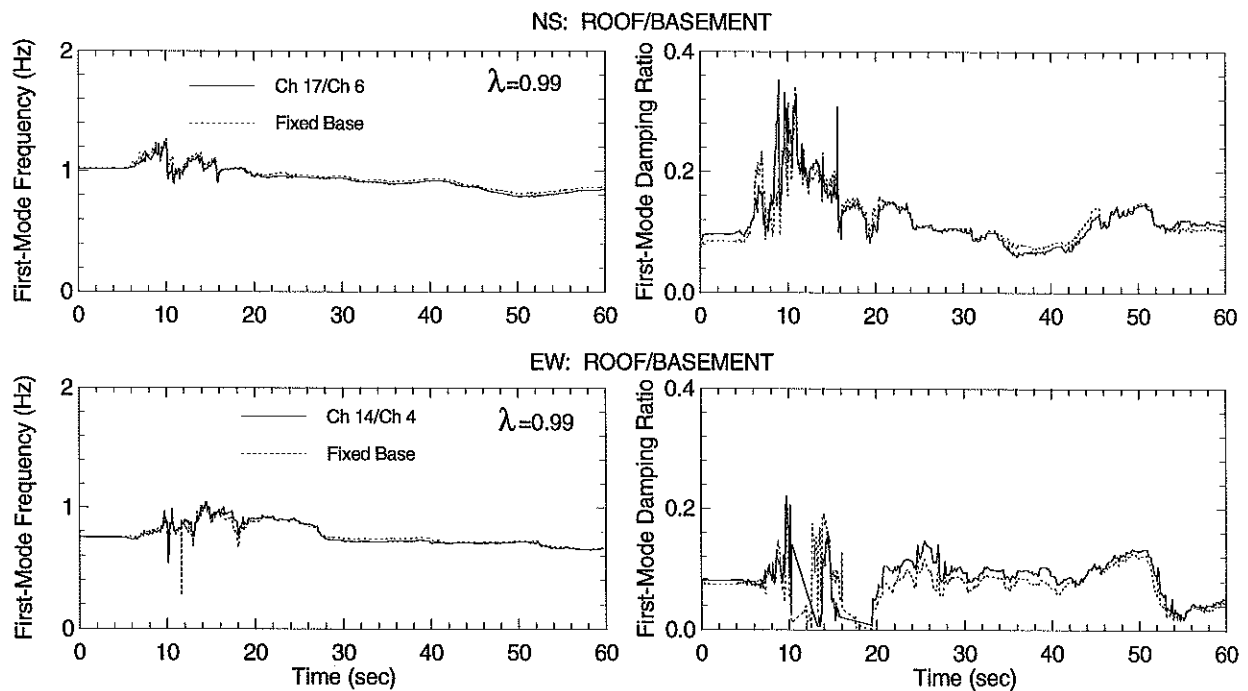


Fig. B10.NR3: LA 9 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE B11

Site Name: LA 17 Story Residential Bldg.
Sensors: CSMIP 24601

Lat: 34.053°
Long.: 118.248°

Floor Levels: 17
Bsmt. Levels: 0
Structure: concrete shear walls

Foundation: drilled piers
Site Geology: rock
Design Date: 1982

Recorded Peak Accelerations (g)

Earthquake	1st Floor			Roof		
	NS	EW	V	NS	EW	V
1992 Landers	0.043	0.045	0.019	0.21	0.16	--
1994 Northridge	0.26	0.19	0.08	0.46	0.56	--

-- = no instrument

Preprocessing

Landers Earthquake (LD)

Preliminary: CSMIP; filtered, $f = 0.1\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$

Final: decimated by 4 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$

Final: decimated by 4 to $f_{\text{Nyq}} = 12.5\text{ Hz}$

Figures

B11.1-B11.2: Sensor Locations, Generalized Soil Column
B11.LD1-3 LD Analysis Results
B11.NR1-3 NR Analysis Results



View of structure, looking north

DISCUSSION

Site Description

The building is 17 stories tall above ground level with no basement and has a roughly rectangular shape in plan (80 by 227 feet). Lateral loads are resisted by precast reinforced concrete shear walls throughout the building. The foundation consists of grade beams supported by 44-inch diameter piers of 54-foot length. Most of the sensors are located near the centroid and along a shear wall at the north end of the building, and 1st floor vertical instruments are positioned along the wall to detect rocking in the transverse (EW) direction.

Subsurface exploration by LCA (1977) encountered weathered siltstone bedrock which extends to the maximum depth explored of 50 feet. Shear wave velocities shown in Fig. B11.2 are based on in situ downhole measurements by LCA (1977).

Instrument Selection and Free-Field Conditions

Instruments near the north shear wall of the building were used in the analyses, as indicated by Table B11.1. Acceleration time histories recorded by some of these sensors are shown in Figs. B11.**1. The two vertical instruments located at the 1st Floor level next to the northernmost shear wall were used to evaluate rocking effects in the EW direction.

Table B11.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 5	Ch 3	Ch 1, 2	Ch 14	Ch 11

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the EW direction,

$$\text{input} = \text{Ch3} + (\text{Ch2} - \text{Ch1}) \cdot 150/31.5 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

Direct examinations of rocking contributions to roof motion are made in Figs. B11.**1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are in phase with the relative roof displacements and contribute significantly to the relative roof displacement amplitude, indicating a pronounced influence of base rocking on the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B11.**2),

(2) modal parameters from CEM analyses and appropriate estimation procedures (Table B11.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B11.**3).

Table B11.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flexible-Base			
		tr (EW)		L (NS)		tr (EW)		L (NS)		tr (EW)		L (NS)	
		LD: d=0, j=4 NR: d=0, j=4				d=2, j=4 d=2, j=4		d=3, j=4 d=3, j=4		estimated			
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
LD	1	1.18	3.4 4.5	na	na	1.04	3.1	0.94	3.1	1.03	2.9	na	na
	2	6.09	7.8			4.95	6.0	4.22	2.9				
	3	8.31	7.6			6.66	9.2	--	--				
NR	1	1.12	3.9 5.4	na	na	0.95	4.4	0.86	5.2	0.94	5.4	na	na
	2	5.27	9.1			4.78	7.7	4.00	4.3				
	3	6.62	14.5			6.57	7.3	6.69	15.5				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response for the transverse direction is dominated by the first mode.
- The minor differences between the flexible- and pseudo flexible-base frequencies indicate only a minor SSI effect associated with base translation. Base rocking was more significant, as indicated by the modest differences between the first-mode frequencies for the fixed- and pseudo flexible-base cases and the results in Part (a).
- As discussed below, the EW CEM ζ values of 3.4% (LD) and 3.9% (NR) were adjusted based on RPEM results.
- The structural response during the NR event was less stiff and more highly damped than the LD response, which was associated with relatively low amplitude shaking.

The RPEM results (Fig. B11.**3) indicate reductions in first-mode frequency of about 0.1 Hz during both the LD and NR events at the times of highest amplitude shaking (about 25 and 15 seconds, respectively). The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode frequencies. However, there was a discrepancy between the RPEM and CEM results with regard to the relative damping ratios. Based on the RPEM results during the most stable portion of the time history (LD: 10 to 100 sec., NR: 14 to 80 sec.), ζ exceeded $\tilde{\zeta}^*$ values by absolute differences of 1.5% and 1.0% for the LD and NR events, respectively [i.e., as indicated in Table B11.2, better estimates are $\zeta=4.5\%$ (LD) and $\zeta=5.4\%$ (NR)].

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B11.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 76 feet using 1st floor motions recorded near the centroid of the building and the profile

in Fig. B11.2. Since the shear walls along the east and west sides of the structure are only connected through relatively thin floor slabs and the foundation, it was assumed that the sets of walls behave independently for calculation of foundation radius r_2 (the structure's entire footprint area was used to calculate r_1). No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping was increased by a factor of 4.0 to account for shape effects.

Table B11.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=1210$ fps	$h=91$ ft	emb.=0	$\gamma = 0.15$
	$\nu=0.4$		tr: $r_1 = 76$ ft		$r_2 = 35$ ft
LD	$\beta^*=0.9\%$	$V_s^*=1190$ fps	L: $r_1 = --$		$r_2 = --$
NR	$\beta^*=1.4\%$	$V_s^*=1140$ fps			

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B11.3 with the fixed-base modal parameters in Table B11.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" formulation, with the results in Table B11.4.

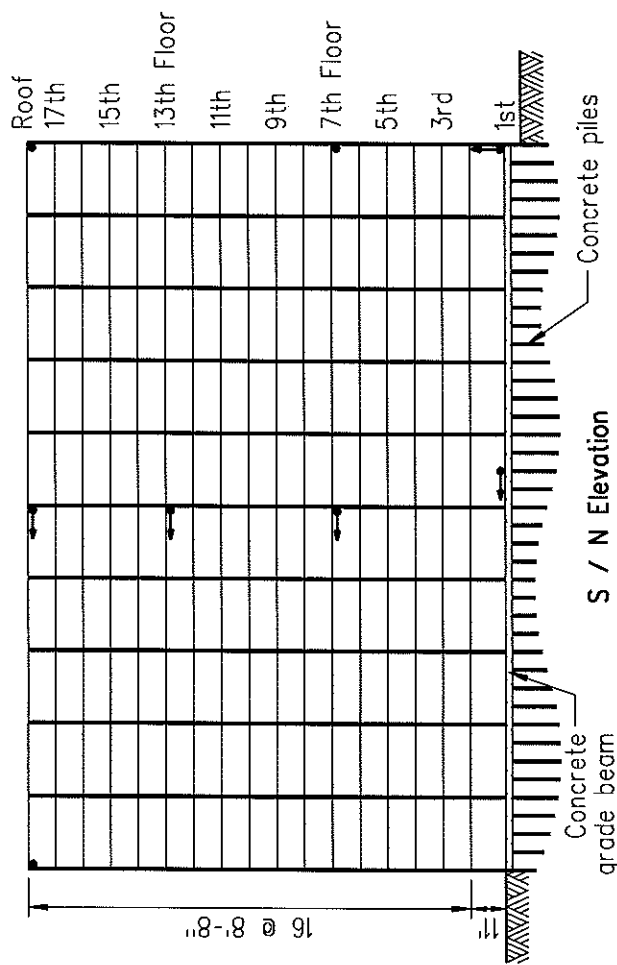
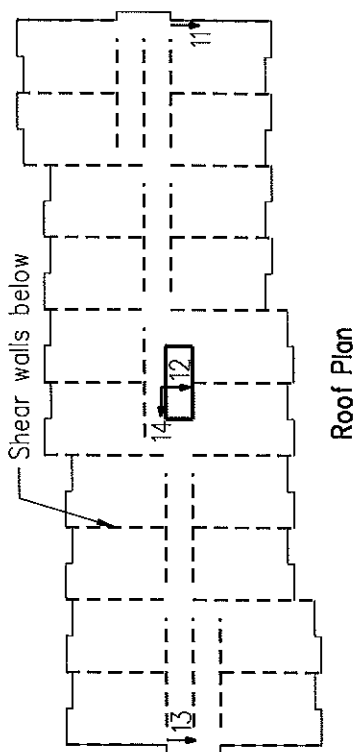
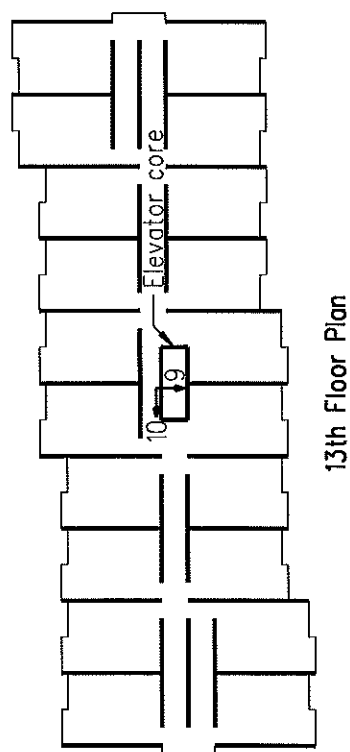
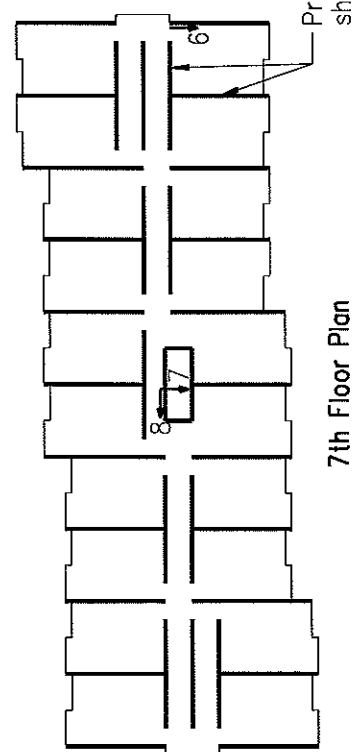
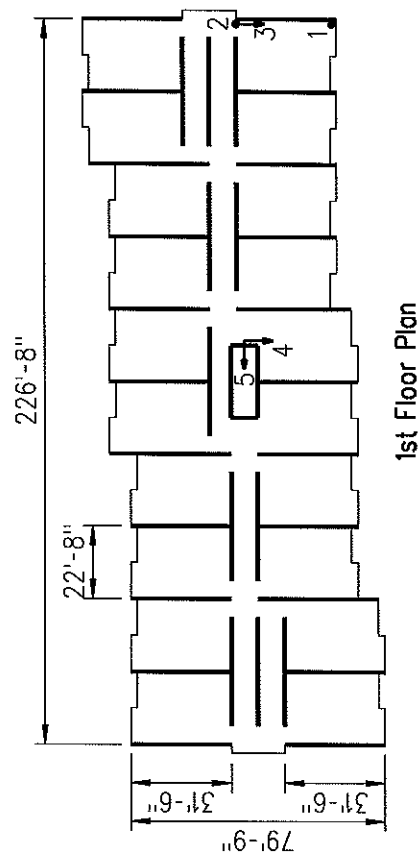
Table B11.4: Empirical and predicted inertial interaction effects

		tr (EW)			L (NS)		
		$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$	$\tilde{T}/T$	$\tilde{\zeta}_0(\%)$	$\tilde{\zeta}(\%)$
LD	Sys. ID	1.14	-0.2	2.9	--	--	--
	Prediction	1.20	0.2	2.9	--	--	--
NR	Sys. ID	1.19	2.1	5.4	--	--	--
	Prediction	1.19	0.3	3.4	--	--	--

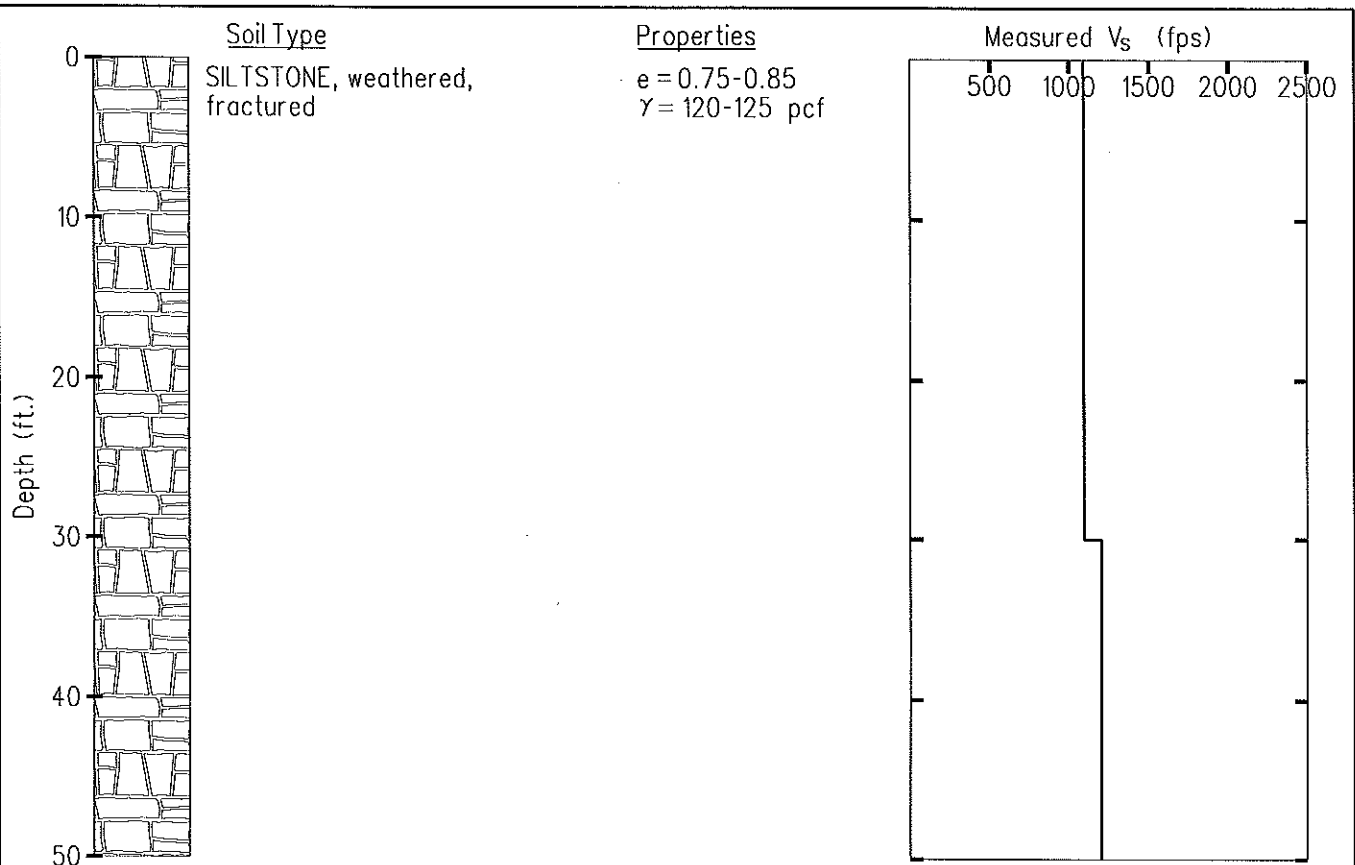
The period lengthening for this structure is satisfactorily predicted, particularly for the NR event. The overprediction of period lengthening for the LD event is likely a consequence of the east and west walls not deforming independently under the relatively low-level shaking from this event (hence, increasing the effective radii of the foundation relative to the assumed case of independent wall deformations). The foundation damping was generally satisfactorily predicted. The negative foundation damping from the system identification analyses for the LD event indicates a minor numerical error in these results.

Data Sources

LeRoy Crandall Associates, LCA (1977). "Report of Geotechnical Investigation, Proposed Bunker Hill Elderly Housing - Phase 1, Portions of Parcels X and Y, Bunker Hill Urban Renewal Area, between Olive and Hill Streets, South of Second Street, Los Angeles, California," *Job. No. AE-77245*.



Sensor Locations
17-story Residential Building Los Angeles, CA
Figure B11.1

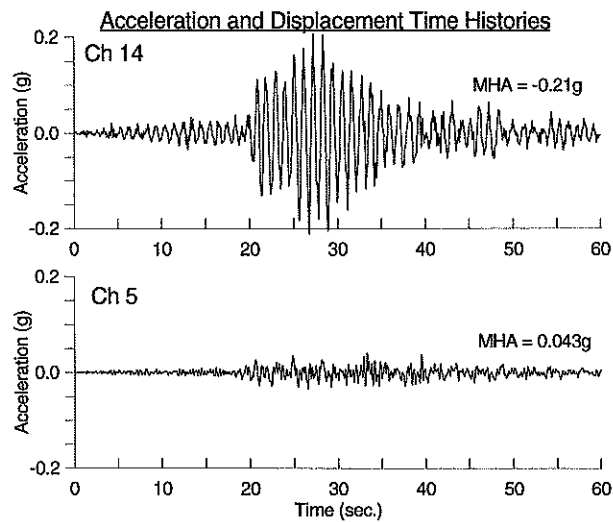
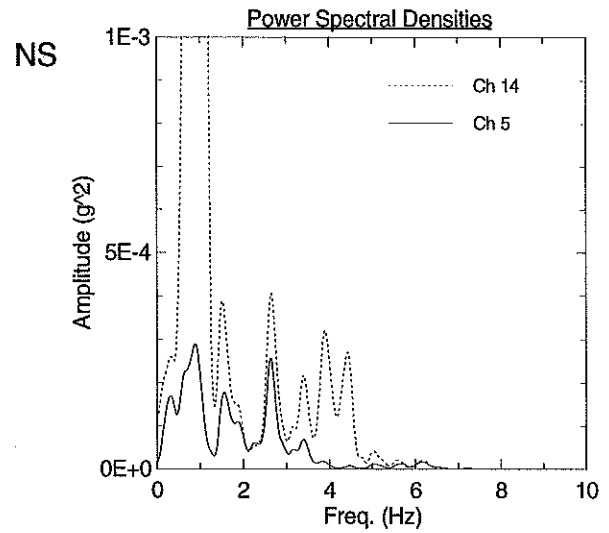
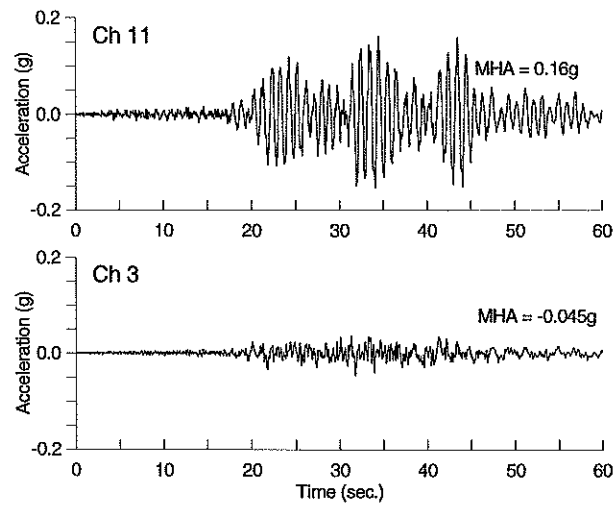
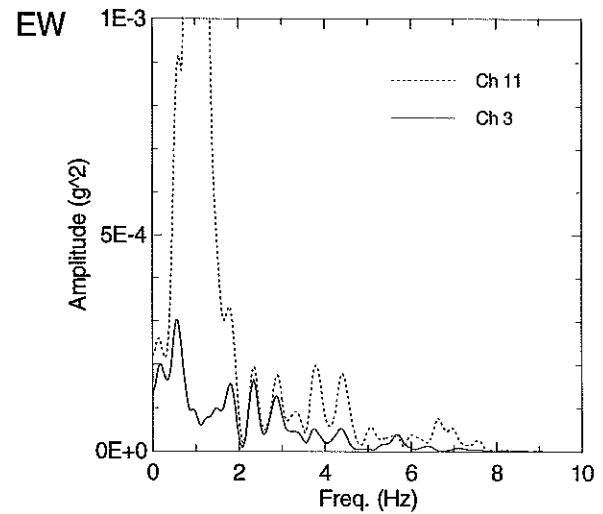
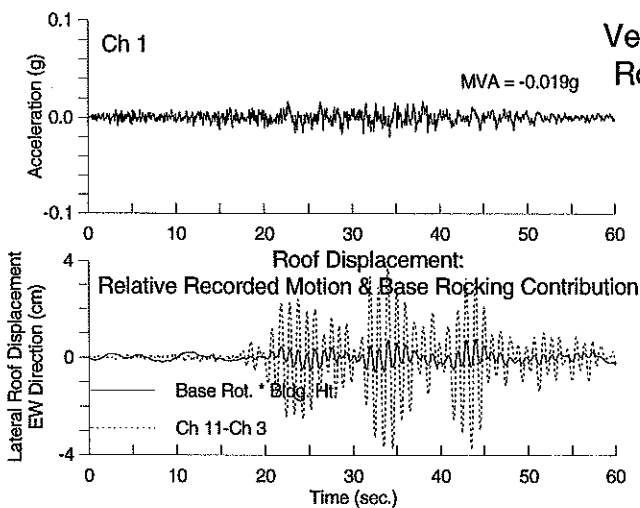


Data Source:
LCA (1977)

Notes:

1. Groundwater not encountered.
2. Shear wave velocities based on downhole measurements by LCA (1977).

Generalized Soil Profile
17-story Residential Building Los Angeles, CA
Figure B11.2



**Vertical &
Rocking**

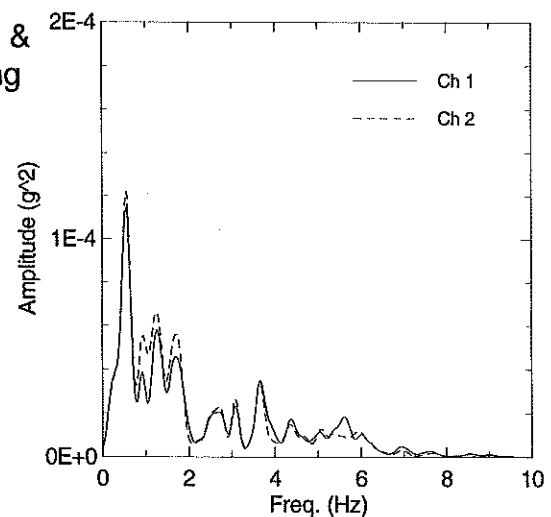


Fig. B11.LD1: LA 17 STORY RESIDENTIAL BUILDING
Time Histories and Power Spectral Density Functions
1992 Landers Earthquake
B11-7

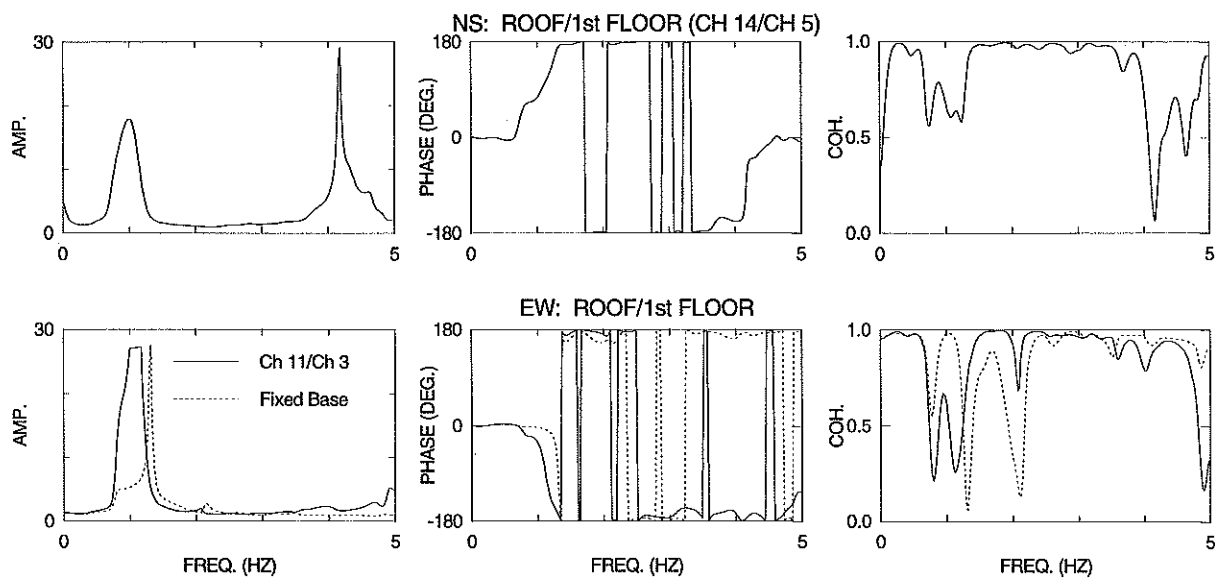


Fig. B11.LD2: LA 17 STORY RESIDENTIAL BUILDING
Transmissibility and Coherency Functions
1992 Landers Earthquake

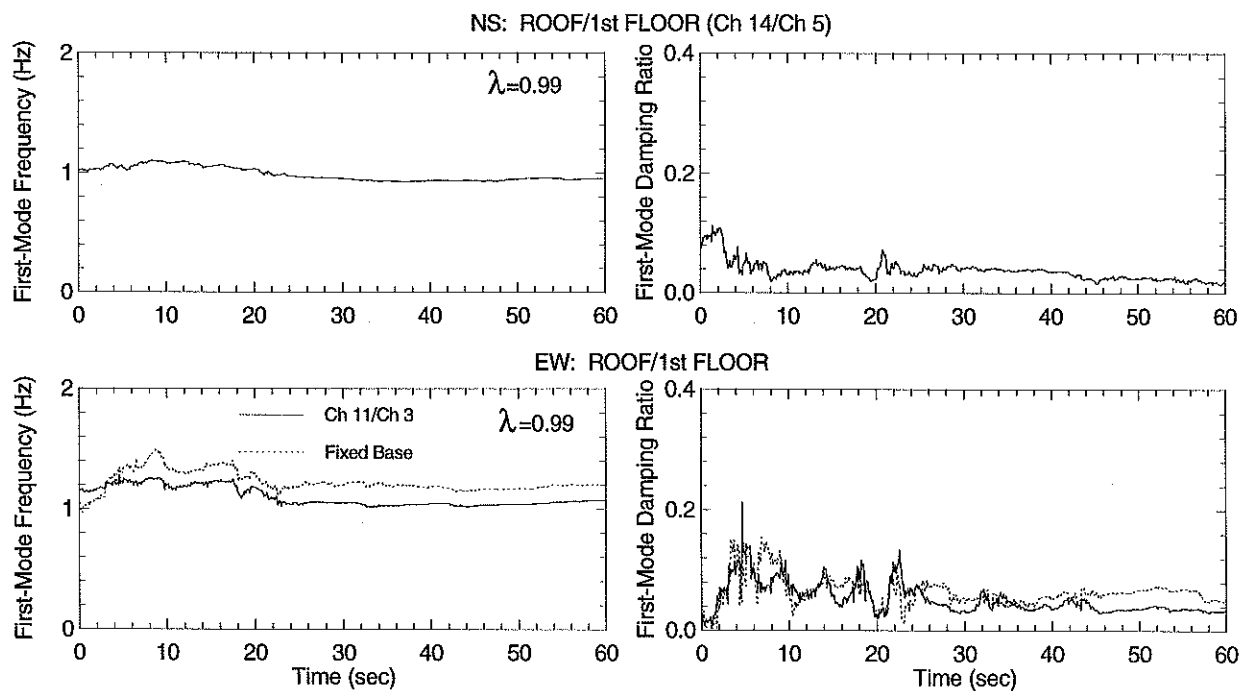


Fig. B11.LD3: LA 17 STORY RESIDENTIAL BUILDING
Time Variation of First-Mode Parameters
1992 Landers Earthquake

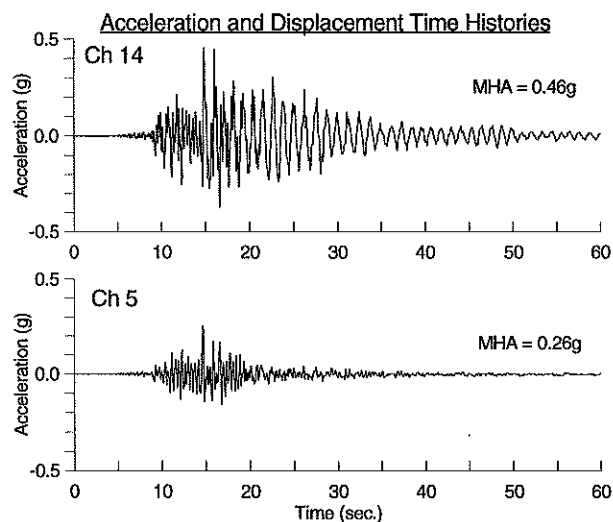
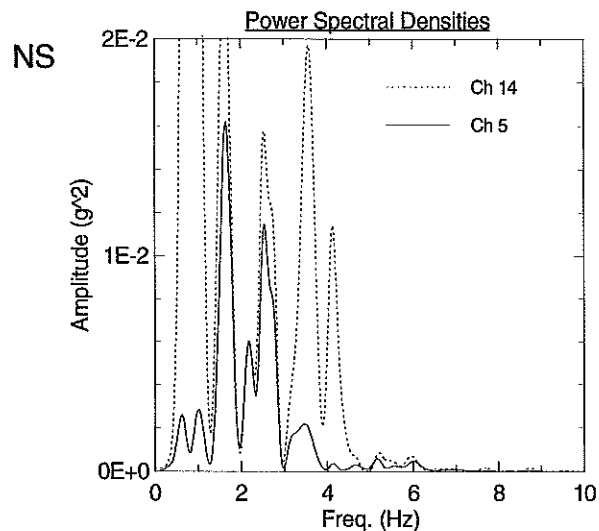
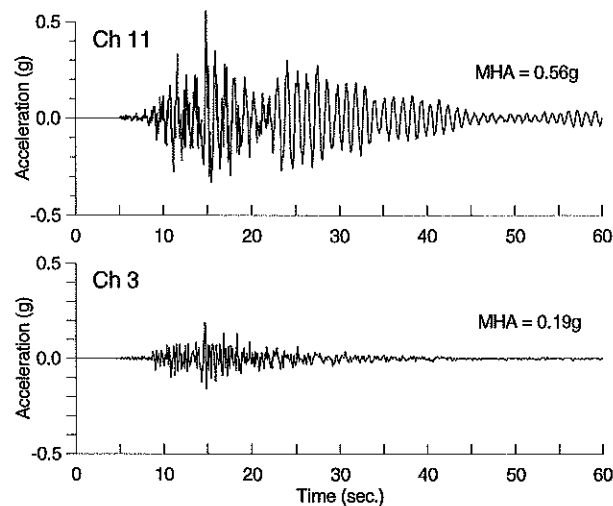
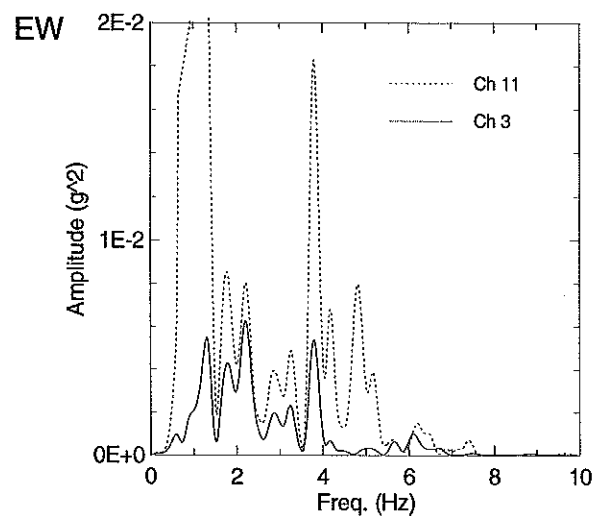
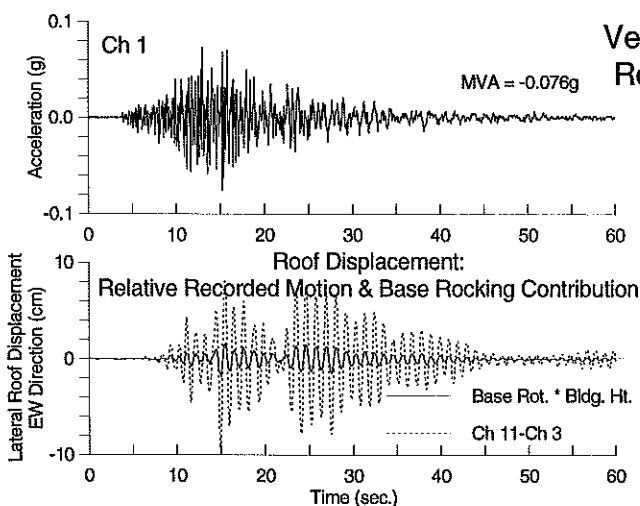
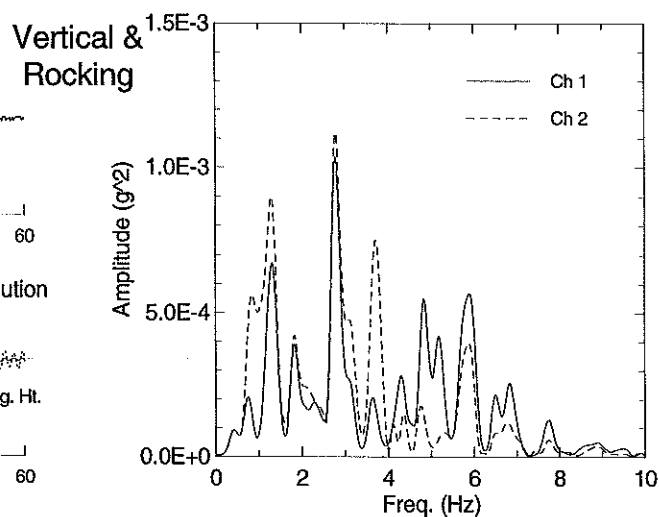


Fig. B11.NR1: LA 17 STORY RESIDENTIAL BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake
B11-9

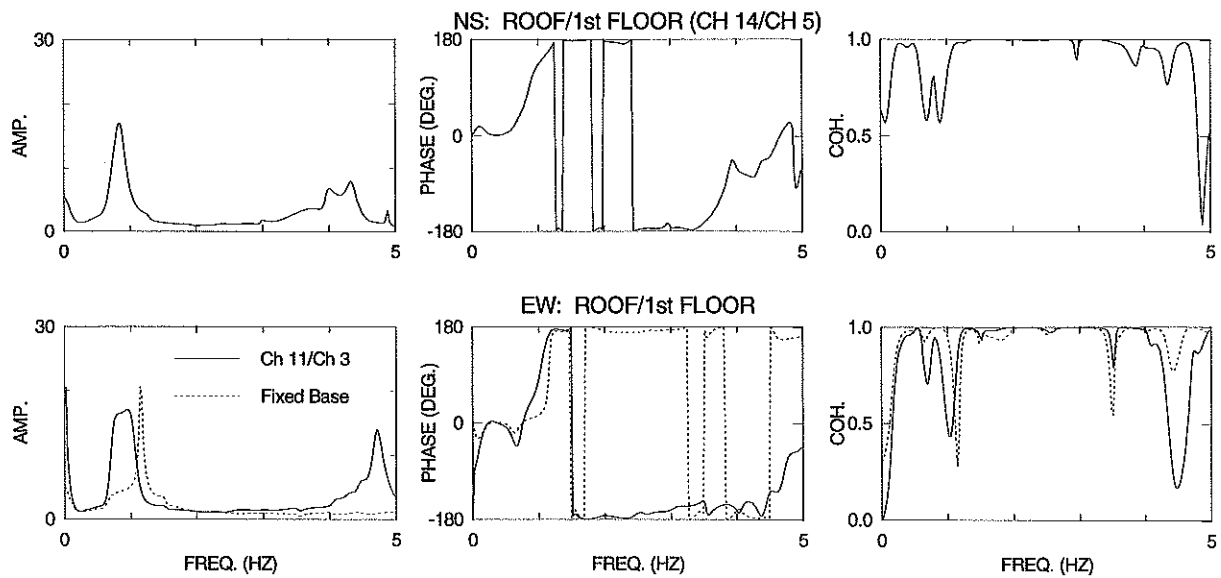


Fig. B11.NR2: LA 17 STORY RESIDENTIAL BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

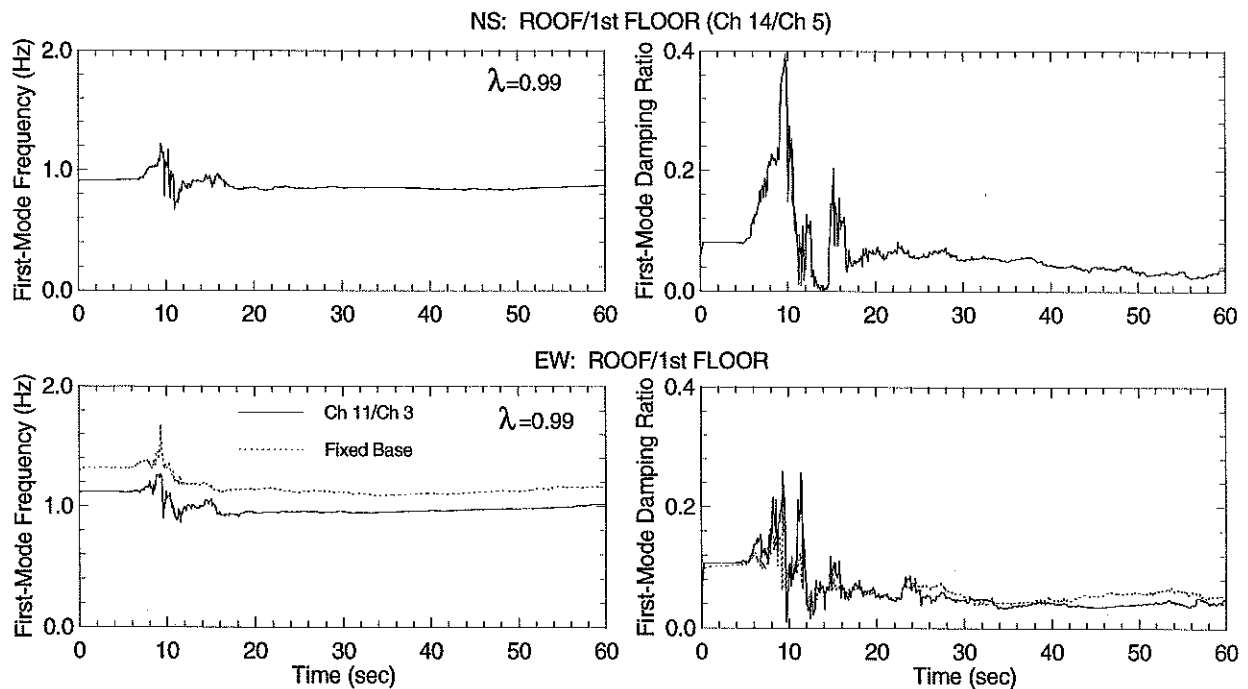


Fig. B11.NR3: LA 17 STORY RESIDENTIAL BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE B12

Site Name: LA 32 Story Offc. Bldg.
Sensors: USGS 5233

Lat: 34.052°
Long.: 118.263°

Floor Levels: 39
Bsmt. Levels: 4
Structure: Steel frame and concrete shear wall

Foundation: footings
Site Geology: rock
Design Date: 1983

Recorded Peak Accelerations (g)

Earthquake	Base			Roof		
	NS	EW	V	NS	EW	V
1994 Northridge	0.10	0.11	0.07	0.15	0.14	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

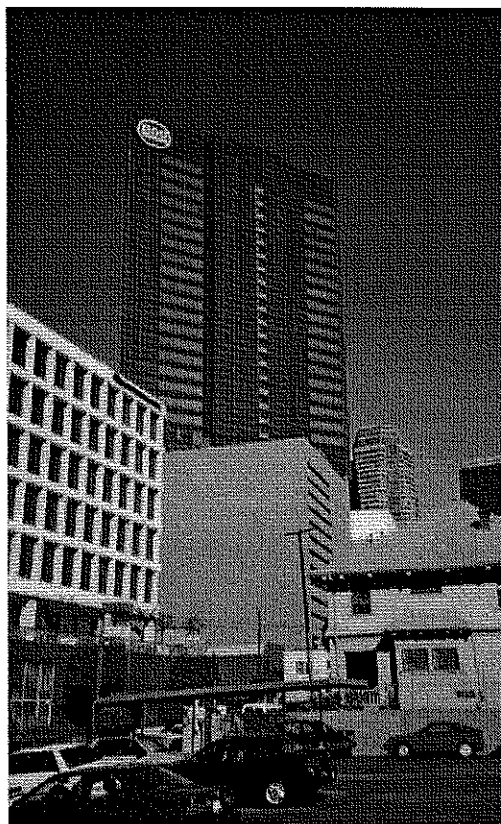
Preliminary: USGS; not filtered; $f_{Nyq} = 100$ Hz

Final: filtered (this study), $f_{HP} = 0.13$ Hz, decimated by 8 to $f_{Nyq} = 12.5$ Hz

Figures

B12.1-B12.2: Sensor Locations, Generalized Soil Column
B12.NR1-3 NR Analysis Results

View of structure,
looking east



DISCUSSION

Site Description

The building is 35 stories tall above ground level with a 4-level basement, and changes from a rectangular to a triangular shape in plan view above the 12th story (side dimensions 155 x 175 feet). Lateral loads are resisted by a dual steel frames/concrete shear wall system. The foundation consists of a 2-foot thick reinforced concrete footings around the building perimeter and spread footings beneath columns in the interior. A 5-inch thick slab-on-grade is present between the footings. Lateral accelerographs are present on five levels of the structure, with vertical base-level instruments capable of measuring rocking in the transverse (NS) and longitudinal (EW) directions.

Subsurface exploration by LCA (1982) encountered about 50 feet of weathered shale and siltstone bedrock overlying less weathered similar rock which extends to the maximum depth explored of 76 feet. Based on measurements by LCA, the groundwater table is located at a depth of 30 feet. The shear wave velocities shown in Fig. B12.2 are based on in situ downhole measurements by LCA (1982).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B12.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B12.NR1. The three vertical instruments located at the base level were used to evaluate rocking effects in the NS and EW directions.

Table B12.1: Sensors in structure used for analyses

Base Level (B)			Roof (R)	
NS	EW	V	NS	EW
Ch 21	Ch 14	Ch 13, 16, 19	Ch 9	Ch 7

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS and EW directions,

$$\begin{aligned} \text{input, ns} &= \text{Ch21} + (\text{Ch16} - \text{Ch19}) \cdot 484/150 \\ \text{input, ew} &= \text{Ch14} + (\text{Ch16} - \text{Ch13}) \cdot 484/170 \end{aligned} \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

Direct examinations of rocking contributions to roof motion are made in Fig. B12.NR1 by comparing relative roof velocities with the product of base rotation and building height (RxH). The calculated RxH velocities are generally not in phase with the relative roof velocities, and

have a much smaller amplitude. Hence, it appears from these results that base rocking did not significantly influence the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B12.NR2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B12.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B12.NR3).

Table B12.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (B-R)				Flexible-Base			
		tr (NS) d=11, j=10		L (EW) d=6, j=10		tr (NS) d=11, j=10		L (EW) d=10, j=10		tr (NS) estimated		L (EW) estimated	
		f	ζ	f	ζ	$\tilde{f}_1^*$	$\tilde{\zeta}_1^*$	$\tilde{f}_1^*$	$\tilde{\zeta}_1^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	0.55	6.6 7.1	0.53	12.2	0.53	7.4	0.54	11.9	0.53	9.3	--	--
	2	1.41	13.7	1.51	13.9	1.43	13.6	1.49	13.2				
	3	2.39	8.4	2.33	11.6	2.46	10.5	2.34	11.8				

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural responses for both directions are dominated by the first mode
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.
- As discussed below, the NS CEM ζ value of 6.6% was adjusted based on RPEM results.
- For the longitudinal (EW) direction, flexible-base parameter estimation procedures could not be invoked because $\tilde{f}_1^* > f$. There are several potential sources of this error, the most significant of which are the relatively crude processing of this strong motion data and the limited duration (about 30 seconds) of the time histories. This error precludes the use of EW data for the evaluation of SSI effects.
- A significant torsional response is expected in this unusually-shaped building, but these motions were not analyzed as part of this study.

The RPEM results (Fig. B12.NR3) indicate no significant time-dependent changes in the first-mode parameters of the structure. For the NS direction, the RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed-base and B-R first-mode frequencies. However, there was a discrepancy between the relative damping ratios from the RPEM and CEM analyses. Based on the RPEM results during the most stable portion of the time history (11 to 28.5 seconds), ζ is less than $\tilde{\zeta}_1^*$ by an absolute difference of about 0.3% (i.e., as indicated in Table B12.2, a better estimate is $\zeta = 7.1\%$).

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B12.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 142 feet using base motions recorded near the northwest corner of the structure and the profile in Fig. B12.2. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B12.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s = 1380$ fps	$h = 300$ ft	emb. = 54 ft	$\gamma = 0.15$
NR	$\nu = 0.45$		tr:	$r_1 = 93$ ft	$r_2 = 91$ ft
	$\beta^* = 1.2\%$	$V_s^* = 1340$ fps	L:	$r_1 = 93$ ft	$r_2 = 97$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B12.3 with the fixed-base modal parameters in Table B12.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” and “modified Bielak” formulations, with the results in Table B12.4.

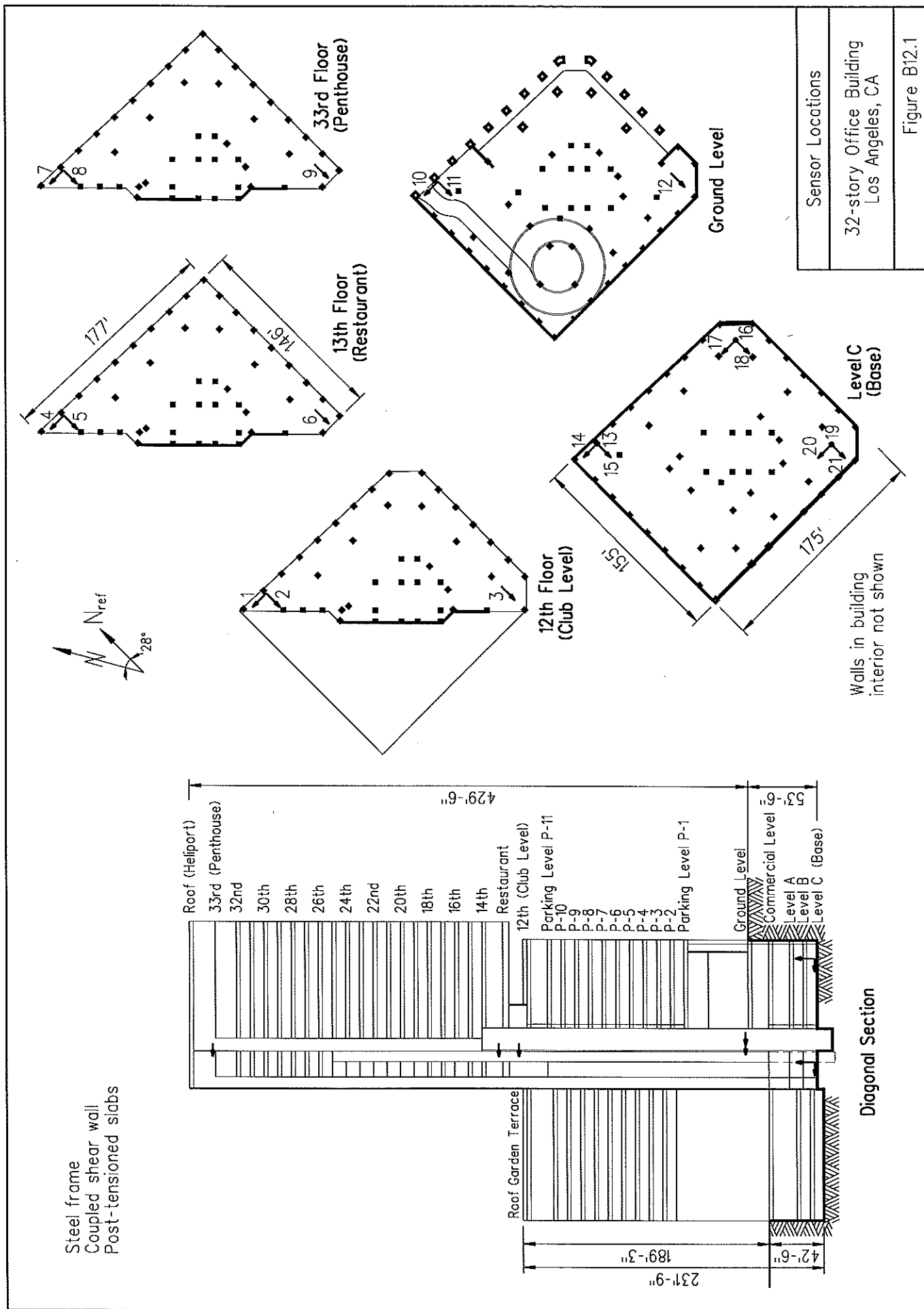
Table B12.4: Empirical and predicted inertial interaction effects

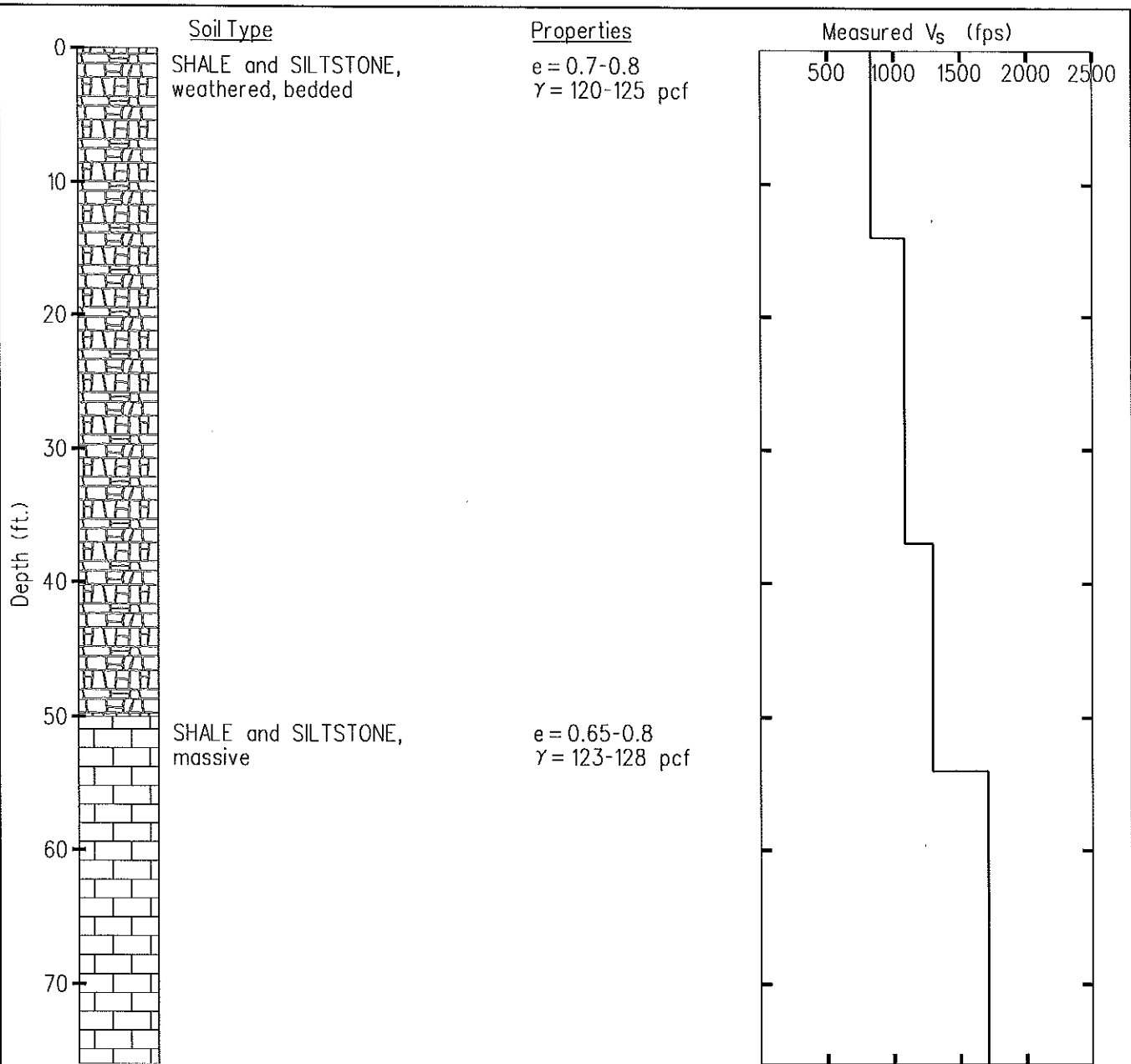
NR	Sys. ID	tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
	Sys. ID	1.03	2.9	9.3	--	--	--
	Pre.-Vel.	1.06	0.1	6.1	--	--	--
	Pre.-Bie.	1.10	1.0	6.3	--	--	--

The period lengthening was overpredicted and the foundation damping underpredicted by both formulations. However, there are some uncertainties associated with the system identification results due to the short duration of the seismograms and the associated difficulty in establishing stable parametric models. In consideration of these potential errors, these discrepancies between the empirical and predicted SSI effects are not surprising.

Data Sources

LeRoy Crandall Associates, LCA (1982). “Report of Geotechnical Investigation, Proposed Wilshire/Bixel Building, Wilshire Blvd. and Bixel Street, Los Angeles, California,” *Job. No. ADE-82247*.





Data Source:
LCA (1982)

Notes:

1. Groundwater encountered at about 30 ft. depth.
2. Shear wave velocities based on downhole measurements by LCA (1982).

Generalized Soil Profile
32-story Office Building Los Angeles, CA
Figure B12.2

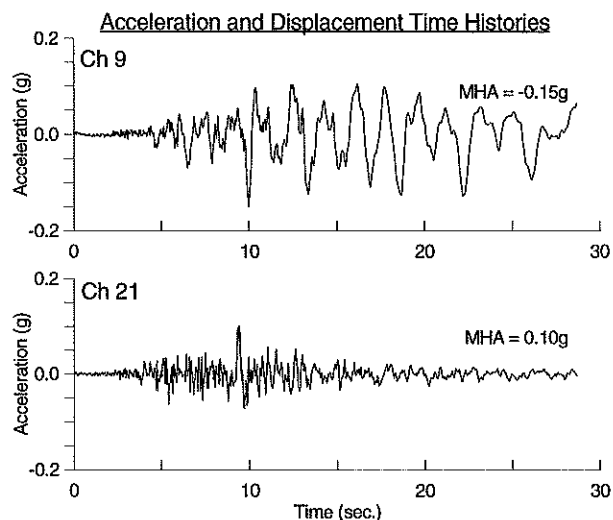
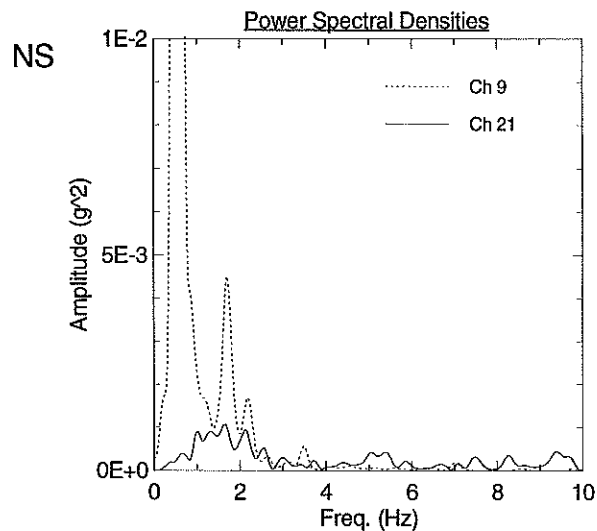
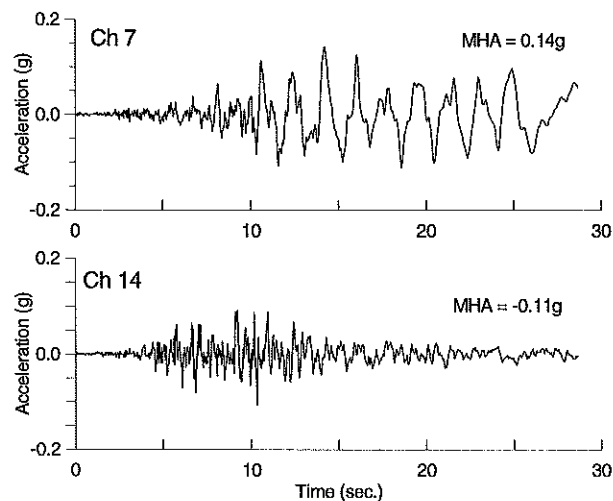
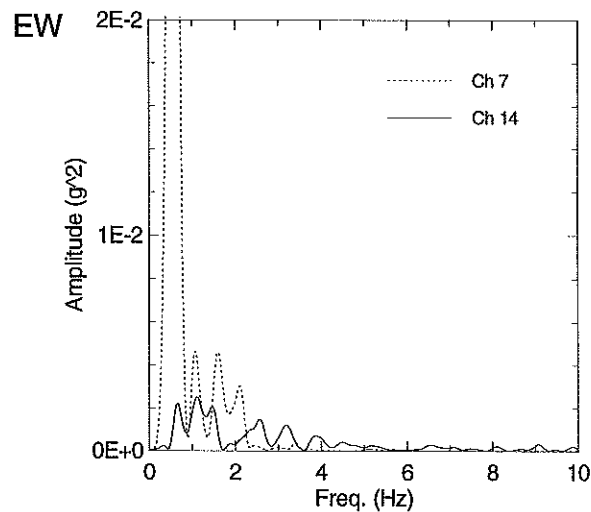
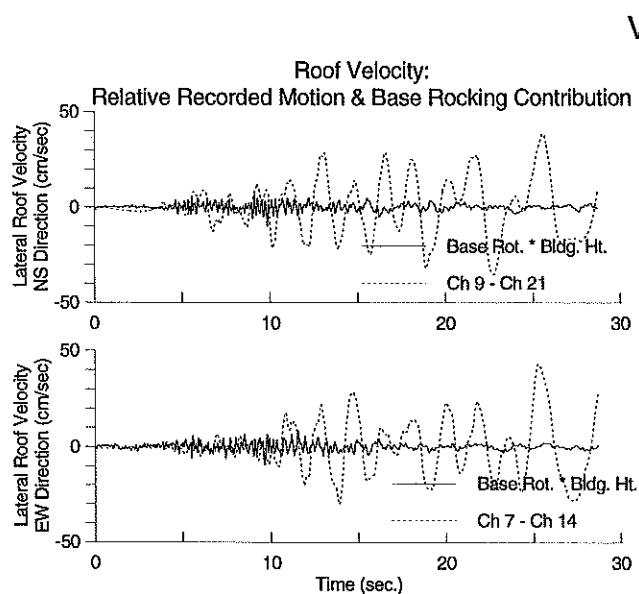
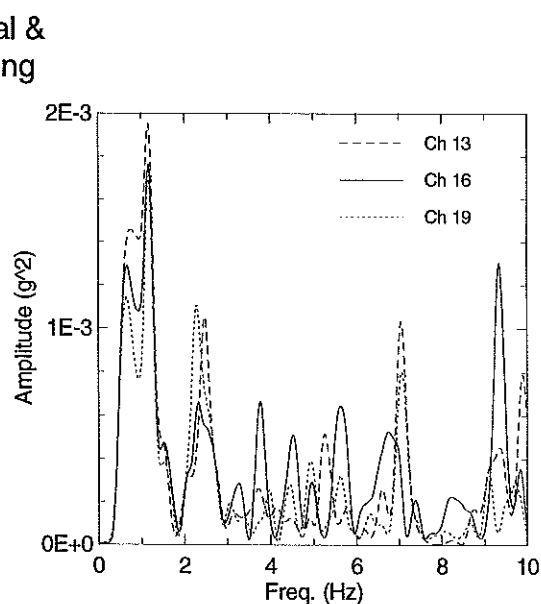


Fig. B12.NR1: LA 32 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake

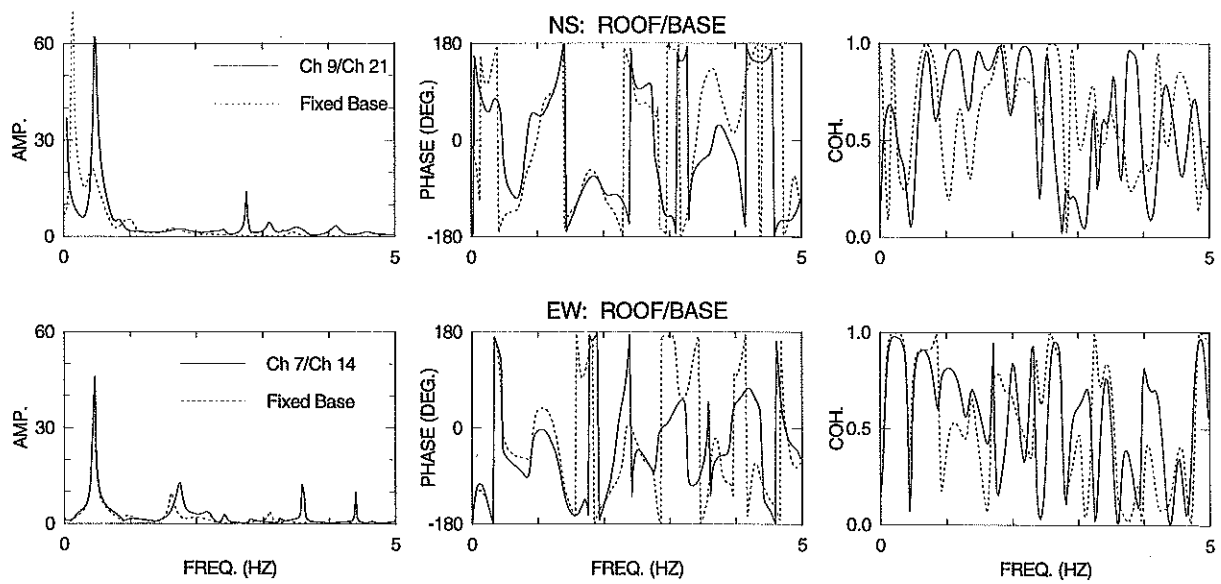


Fig. B12.NR2: LA 32 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

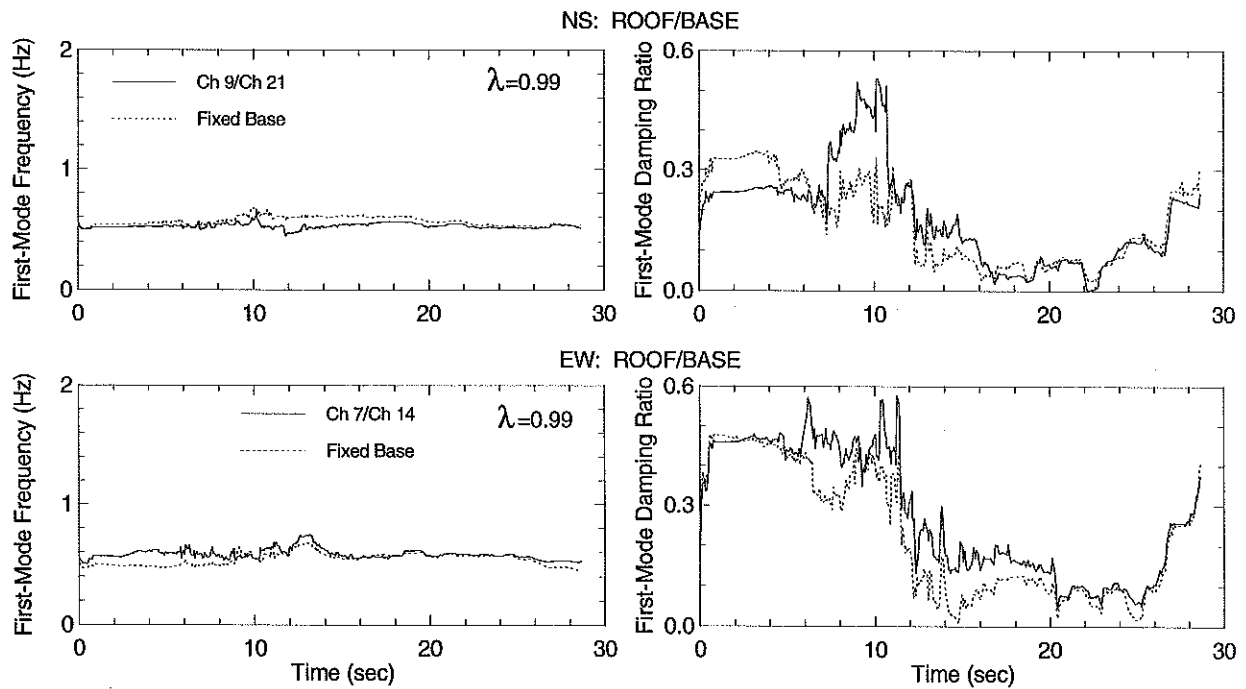


Fig. B12.NR3: LA 32 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE B13

Site Name: LA 54 Story Offc. Tower
Sensors: CSMIP 24629

Lat: 37.048°
Long.: 122.260°

Floor Levels: 56
Bsmt. Levels: 4
Structure: moment resisting steel frames

Foundation: mat
Site Geology: soil
Design Date: 1988

Recorded Peak Accelerations (g)

Earthquake	Foundation Level			54th Floor		
	NS	EW	V	NS	EW	V
1994 Northridge	0.14	0.13	0.07	0.19	0.14	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

Preliminary: CSMIP; filtered $f = 0.17\text{--}46$ Hz; $f_{Nyq} = 50$ Hz

Final: decimated by 8 to $f_{Nyq} = 12.5$ Hz

Figures

B13.1-B13.2: Sensor Locations, Generalized Soil Column
B13.NR1-3 NR Analysis Results

View of structure,
looking north



DISCUSSION

Site Description

The building is 52 stories tall above ground level with a 4-level basement. As shown in Fig. B13.1, the building is roughly rectangular in plan with base dimensions 136 x 212 feet and tapers inward at the 36th and 46th floors to 121 x 196 feet and 121 x 176 feet, respectively. Lateral loads are resisted by moment resisting steel perimeter frames in both directions, and the foundation consists of a reinforced concrete mat which is 9.5 feet thick in load bearing areas and 7 feet thick in intermediate areas. Accelerographs are present on six levels of the structure, with base (P4) level vertical instruments positioned on the north and south sides of the building to detect rocking in the transverse (NS) direction.

Subsurface exploration by LCA (1981) generally encountered 65 feet of sands with variable layers of silts and clays overlying siltstone and shale bedrock which extends to the maximum depth explored of 130 feet. Shear wave velocities shown in Fig. B13.2 are based on in situ downhole measurements by LCA (1981).

Instrument Selection and Free-Field Conditions

The instruments used in the analyses are indicated in Table B13.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B13.NR1. The two vertical instruments at the P4 level were used to evaluate rocking effects in the NS direction.

Table B13.1: Sensors in structure used for analyses

P4 Level (P4)			Roof (R)	
NS	EW	V	NS	EW
Ch 4	Ch 3	Ch 1, 2	Ch 19	Ch 18

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS direction,

$$\text{input} = \text{Ch4} + (\text{Ch1} - \text{Ch2}) \cdot 735.5/130 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B13.NR1 by comparing relative roof displacements with the product of base rotation and building height (RxH). The calculated RxH displacements are out of phase with the roof displacements and have a much smaller amplitude, indicating that base rocking did not significantly influence the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B13.NR2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B13.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B13.NR3).

Table B13.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (P4-R)				Flexible-Base			
		tr (NS) d=4, j=37		L (EW)		tr (NS) d=19, j=37		L (EW) d=29, j=35		tr (NS) estimated		L (EW)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	0.17	10.4 6.6	na	na	0.17	5.6	0.19	1.3	0.17	5.5	na	na
	2	0.50	1.7			0.50	1.6	0.53	1.4				
	3	0.84	2.9			0.83	2.4	0.83	2.4				
	4	1.17	0.3			1.16	2.5	1.19	2.3				
	5	1.50	1.9			1.50	3.1	1.51	3.0				
	6	1.81	6.1			1.86	5.5	1.99	3.1				

Comments on the analytical results follow:

- The power spectra indicate that the roof motions in both directions are dominated by energy at about $f \approx 0.2, 0.5, 0.8$, and 1.2 Hz, which correspond to the first four modal frequencies.
- The first-mode frequencies for all three conditions of base fixity are similar, indicating that neither base rocking nor base translation significantly affected the structural response.

The RPEM results (Fig. B13.NR3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode frequencies. However, there was a discrepancy between the relative damping ratios from the RPEM and CEM analyses. Based on the RPEM results during the most stable portion of the time history (34 to 90 seconds), ζ exceeds $\tilde{\zeta}^*$ by an absolute difference of about 0.3% (i.e., as indicated in Table B13.2, a better estimate is $\zeta = 6.6\%$).

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B13.3. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 165 feet using P4-level motions recorded near the west end of the building and the profile in Fig. B13.2. No adjustments to the impedance function were made for foundation non-rigidity or shape effects. Embedment effects were modeled with two separate analyses (see below).

Table B13.3: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
NR	Model = H	$V_s = 1430$ fps	$h = 414$ ft	emb.=57.5 ft	$\gamma = 0.15$
	$\nu = 0.4$		tr: $r_1 = 96$ ft		$r_2 = 87$ ft
	$\beta^* = 3.4\%$	$V_s^* = 1320$ fps	L: $r_1 = --$		$r_2 = --$

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B13.3 with the fixed-base modal parameters in Table B13.2, the first-mode period lengthening ratio and foundation damping were predicted using the "modified Veletsos" and "modified Bielak" formulations, with the results in Table B13.4.

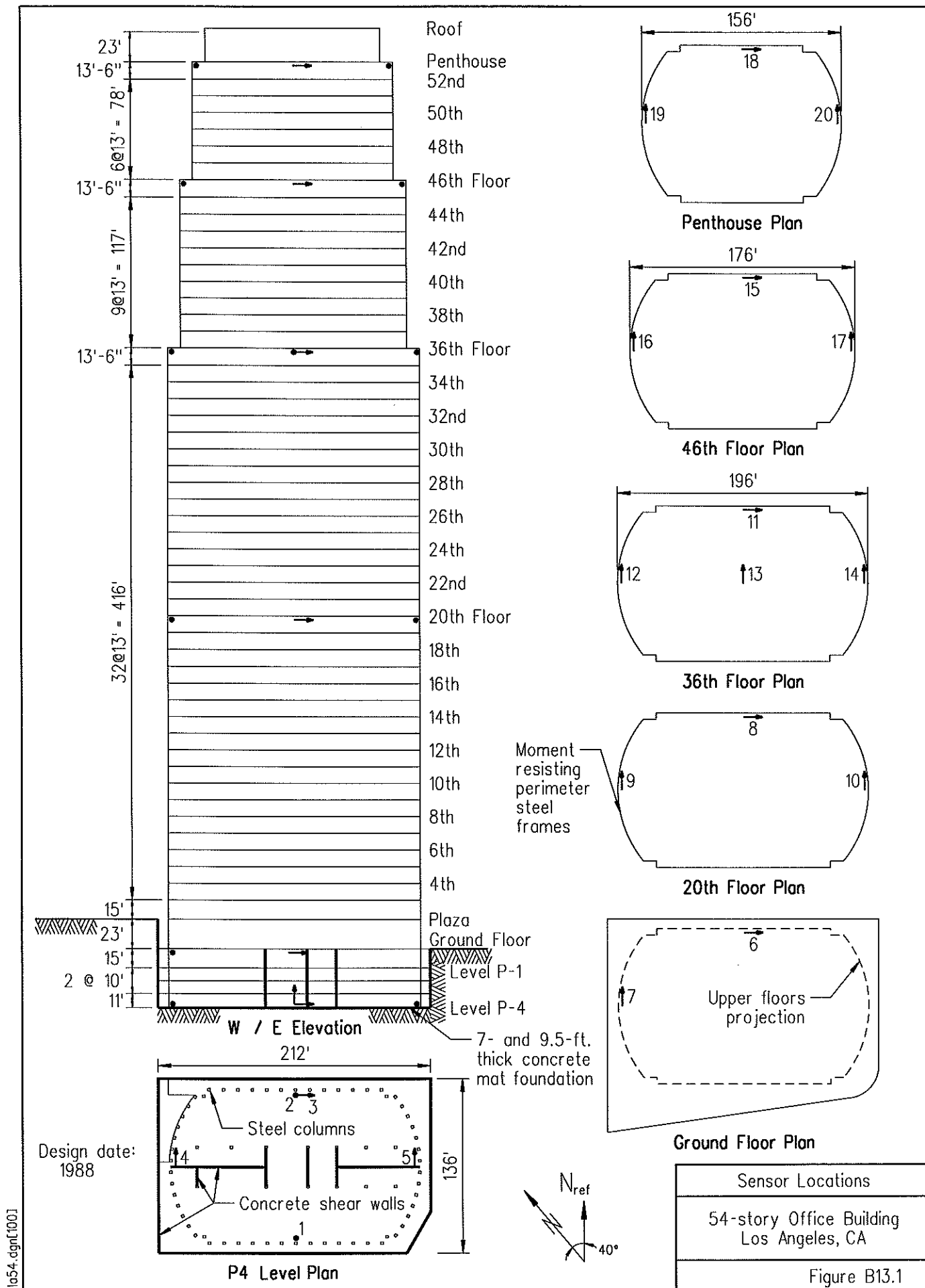
Table B13.4: Empirical and predicted inertial interaction effects

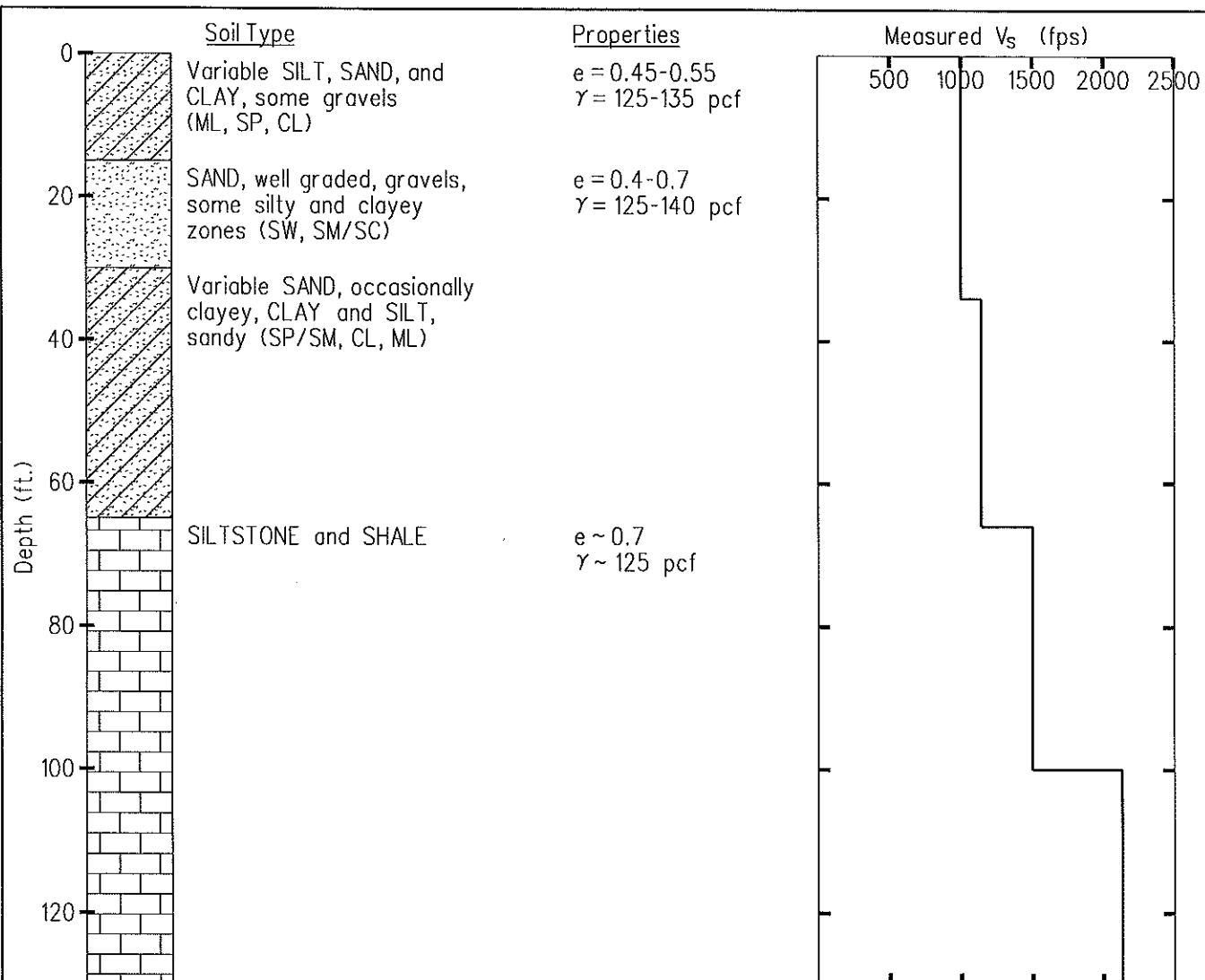
		tr (NS)			L (EW)		
		$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)	$\tilde{T}/T$	$\tilde{\zeta}_0$ (%)	$\tilde{\zeta}$ (%)
NR	Sys. ID	1.02	-0.7	5.5	--	--	--
	Pre.-Vel.	1.02	0.0	6.3	--	--	--
	Pre.-Bie.	1.02	0.2	6.4	--	--	--

The period lengthening and foundation damping were well predicted for this structure. The negative foundation damping from the system identification analyses indicates a minor numerical error in these results.

Data Sources

LeRoy Crandall Associates, LCA (1981). "Report of Geotechnical Investigation, Proposed High Rise Development, Bounded by Harbor Freeway and 7th, 8th, and Figueroa Streets, Los Angeles, California," *Job. No. ADE-81040*.





Data Source:
LCA (1981)

Notes:

1. Groundwater level not established.
2. Shear wave velocities based on downhole measurements by LCA (1981).

Generalized Soil Profile
54-story Office Building Los Angeles, CA
Figure B13.2

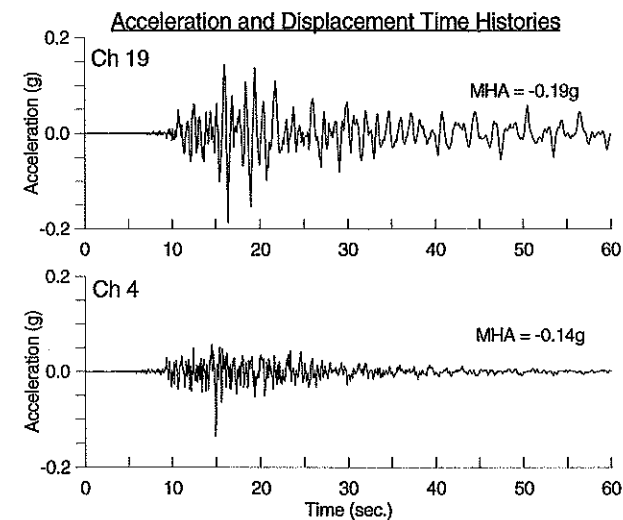
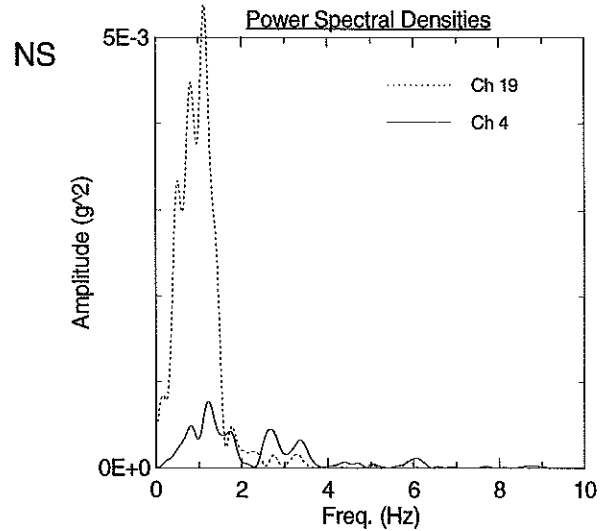
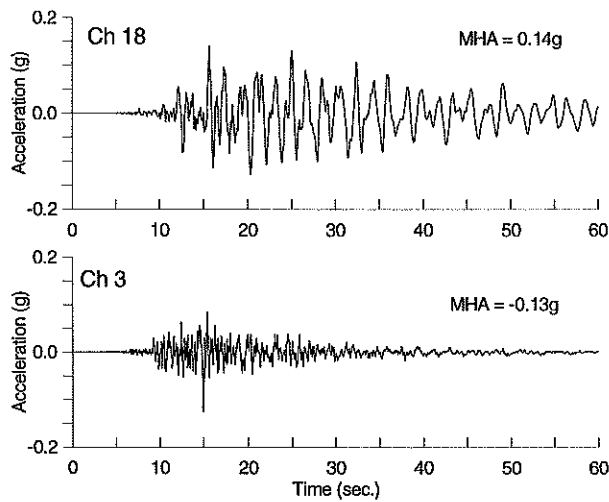
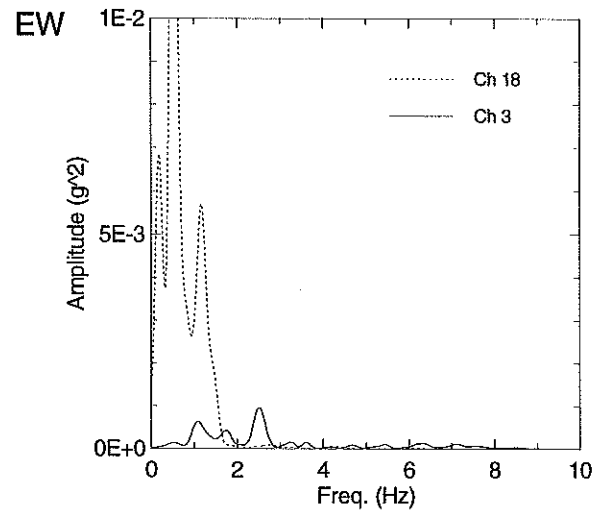
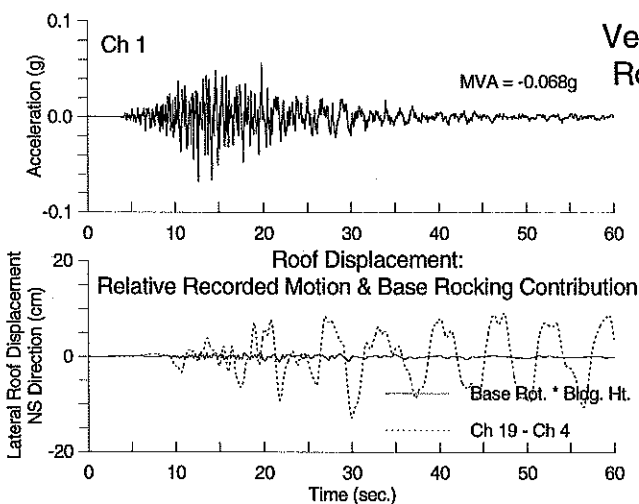
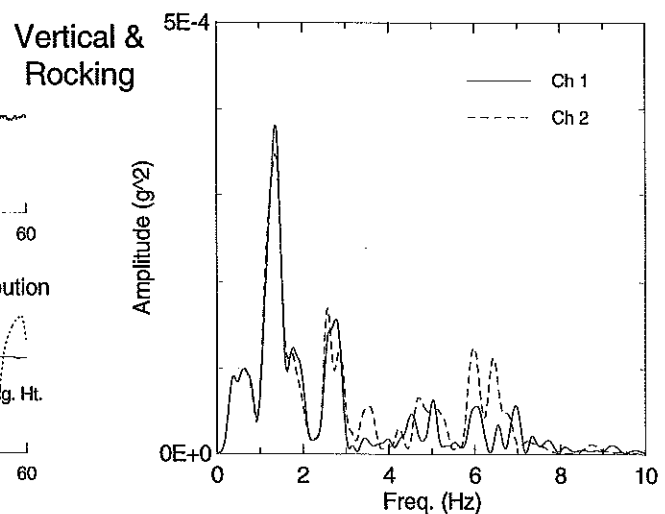


Fig. B13.NR1: LA 54 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake

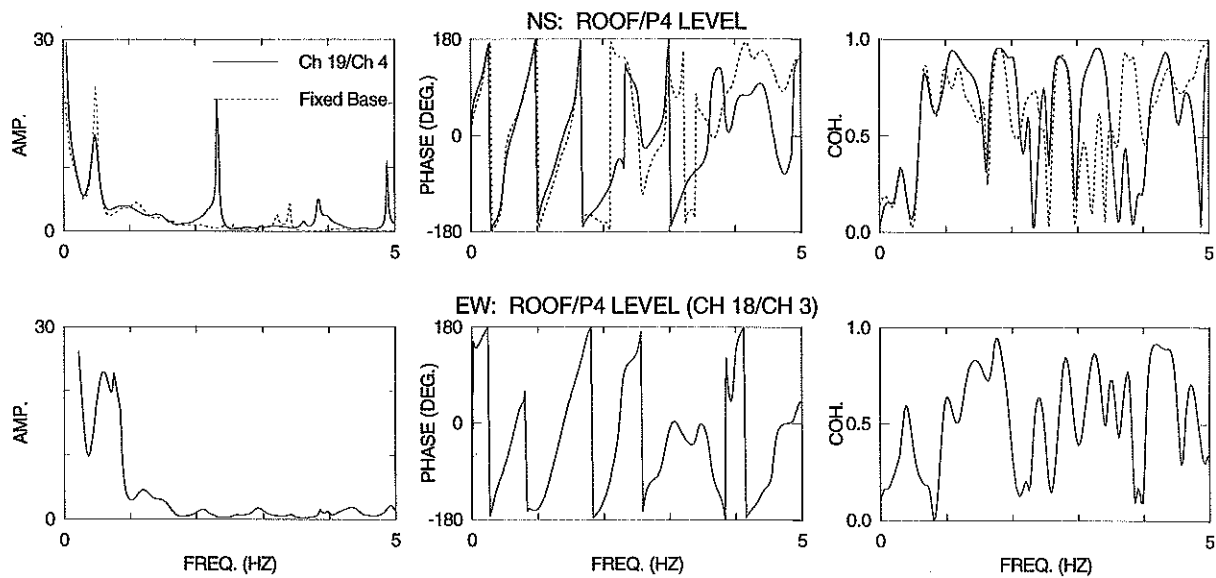


Fig. B13.NR2: LA 54 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

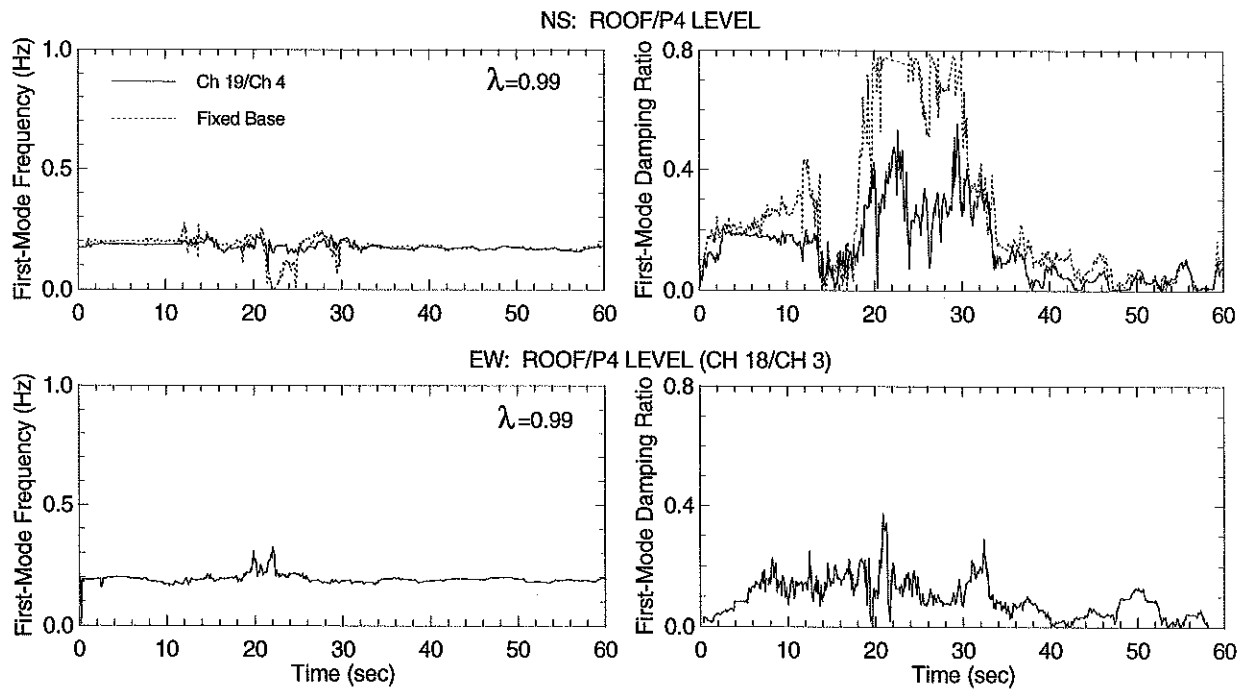


Fig. B13.NR3: LA 54 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake

SITE B14

Site Name: Whittier 8 Story Hotel

Sensors: CSMIP 14606

Lat: 33.975°

Long.: 118.036°

Floor Levels: 8

Bsmt. Levels: 0

Structure: block masonry shear walls

Foundation: footings

Site Geology: soil

Design Date: 1984

Recorded Peak Accelerations (g)

Earthquake	1st Floor			Roof		
	NS	EW	V	NS	EW	V
1994 Northridge	0.17	0.11	0.10	0.33	0.23	--

-- = no instrument

Preprocessing

Northridge Earthquake (NR)

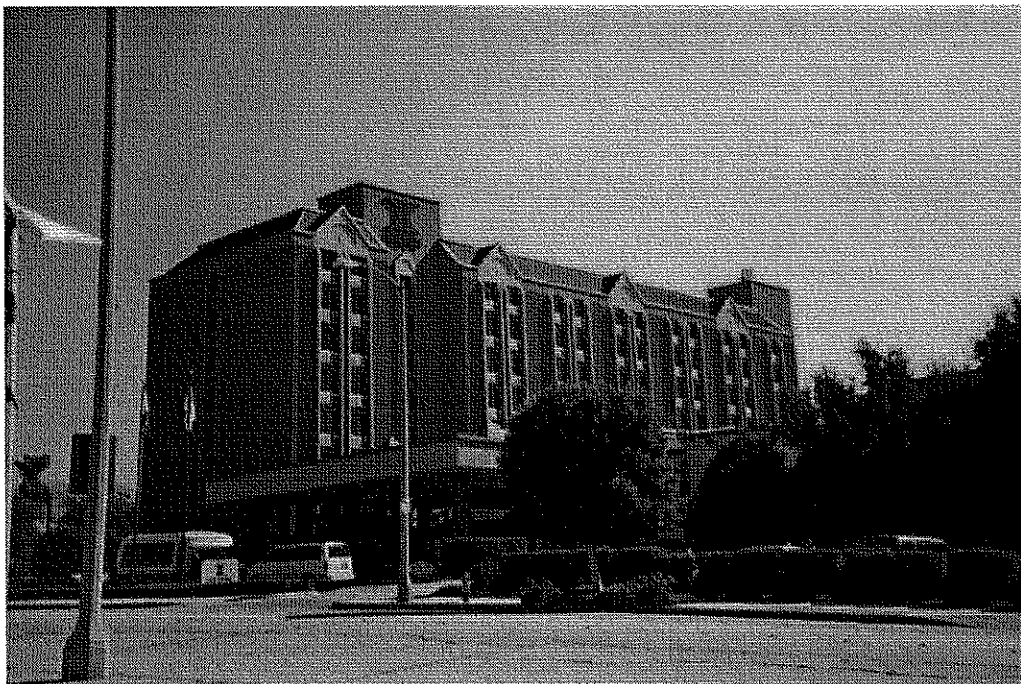
Preliminary: CSMIP; filtered, $f = 0.2\text{--}46\text{ Hz}$; $f_{\text{Nyq}} = 50\text{ Hz}$

Final: decimated by 2 to $f_{\text{Nyq}} = 25\text{ Hz}$

Figures

B14.1-B14.2: Sensor Locations, Generalized Soil Column

B14.NR1-3 NR Analysis Results



View of structure, looking north-east

DISCUSSION

Site Description

The building is 8 stories tall above ground level with no basement and has a rectangular shape in plan (61 by 215 feet). Lateral loads are resisted by 8-inch thick reinforced block masonry shear walls throughout the building. The foundation consists of 2.5-foot deep by 9.5 to 10-foot wide continuous footings along the base of the walls. A 4-inch thick concrete slab is present across the 1st floor level. Lateral accelerographs are located near the centroid and along the east and west shear walls, and 1st floor vertical instruments are positioned at the base of the northwest shear wall to detect rocking in the transverse (NS) direction.

Subsurface exploration near the site by LCA (1978) encountered sandy, clayey silts which extend to the maximum depth explored of 40 feet. Given the location of the site in a large basin, bedrock likely occurs at significantly greater depth. The shear wave velocities in Fig. B14.2 are based on SASW measurements by Rodriguez-Ordonez (1994) near a building about 600 feet from the site.

Instrument Selection and Free-Field Conditions

Instruments near the northwest shear wall of the building were generally used in the analyses, as indicated by Table B14.1. Acceleration time histories recorded by some of these sensors are shown in Fig. B14.NR1. The two vertical instruments located at the 1st floor level adjacent to the northwest shear wall were used to evaluate rocking effects in the transverse (NS) direction.

Table B14.1: Sensors in structure used for analyses

1st Floor (1F)			Roof (R)	
NS	EW	V	NS	EW
Ch 4	Ch 3	Ch 1, 2	Ch 10	Ch 9

The following input was used in system identification analyses to evaluate "fixed-base" structural parameters for the NS direction,

$$\text{input} = \text{Ch4} + (\text{Ch2} - \text{Ch1}) \cdot 76/22 \quad (1)$$

Analysis Results

(a) Evaluation of Base Rocking Effects

A direct examination of rocking contributions to roof motion is made in Fig. B14.NR1 by comparing relative roof displacements with the product of base rotation and building height ($R \times H$). The calculated $R \times H$ displacements are in phase with the relative roof displacements and contribute significantly to the relative roof displacement amplitude, indicating a pronounced contribution of base rocking to the structural response.

(b) Modal Frequency and Damping

Results of system identification analyses and first-mode parameter estimation procedures are presented as follow: (1) nonparametric transmissibility and coherency functions (Fig. B14.NR2), (2) modal parameters from CEM analyses and appropriate estimation procedures (Table B14.2), and (3) time variation of first-mode parameters from the RPEM (Fig. B14.NR3).

Due to uncertainty about the independence of the shear walls along the north and south sides of the structure (the wall are only connected through relatively thin floor slabs and the foundation), flexible-base parameters were estimated for two conditions: (1) "coupled walls," wherein the two sets of walls are assumed to have independent base rotation, and (2) "equivalent single wall," wherein the foundation beneath the walls is rigid in bending so that the walls have a common base rotation. As discussed in Part (c), the difference between these cases is manifest in the transverse foundation radius for rocking (r_2). Flexible-base parameters for these various conditions are presented in Table B14.3.

Table B14.2: Modal parameters (frequencies in Hz, damping in percent)

Eq.	Mode	Fixed-base				Ps. Flex.-Base (1F-R)				Flexible-Base			
		tr (NS) d=2, j=5		L (EW)		tr (NS) d=2, j=5		L (EW) d=4, j=5		tr (NS) estimated		L (EW)	
		f	ζ	f	ζ	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}^*$	$\tilde{\zeta}^*$	$\tilde{f}$	$\tilde{\zeta}$	$\tilde{f}$	$\tilde{\zeta}$
NR	1	2.04	13.7	na	na	1.78	12.0	1.46	7.2	see	below	na	na
	2	5.97	9.6			5.91	11.1	5.91	6.9				
	3	9.94	6.1			9.94	6.9	10.9	6.9				

Table B14.3: Estimated transverse flexible-base first-mode structural parameters

	$\tilde{f}$ (Hz)	$\tilde{\zeta}$ (%)
Coupled walls	1.75	15.6
Eq. single wall	1.66	20.4

Comments on the analytical results follow:

- As shown by power spectra of the roof accelerations, the structural response for the transverse direction is dominated by the first mode.
- For the coupled wall case, the difference between the flexible- and pseudo flexible-base frequencies are minor, indicating a only a minor SSI effect associated with base translation. Base rocking was more significant, as indicated by the relatively large difference between the first-mode frequencies for the fixed- and pseudo flexible-base cases and the results in Part(a).
- The accuracy of the flexible-base damping estimate appears to be poor, as a number of 'A' sites with similar shear wall structures (e.g. A31, A45) have small deviations between $\tilde{\zeta}$ and $\tilde{\zeta}^*$ values (i.e. absolute differences less than 2%). Hence, the flexible-base damping is likely better estimated by $\tilde{\zeta} = 12.0\%$.

The RPEM results (Fig. B14.NR3) indicate no significant time-dependent changes in the first-mode parameters of the structure. The RPEM results generally confirmed the CEM results

with regard to the relative magnitude of the fixed- and pseudo flexible-base first-mode frequencies and damping ratios.

(c) Comparison of Empirical and Predicted Inertial Interaction Effects

Geotechnical and structural parameters relevant to predictive analyses are presented in Table B14.4. The strain-dependent soil properties were estimated from deconvolution analyses to a depth of 64 feet using 1st floor motions recorded near the west wall of the building and the profile in Fig. B14.2. The analyses were performed for the “coupled wall” and “equivalent single wall” cases discussed in Part (b). The difference between these two cases is in the foundation dimensions used to calculate transverse foundation radius r_2 , as indicated in Table B14.4. In both cases, the structure’s entire footprint area was used to calculate r_1 . No adjustments to the impedance function were made for embedment or foundation non-rigidity effects, though rocking radiation damping was increased by a factor of 4.0 to account for shape effects for the coupled shear walls case and 2.0 for the equivalent single wall case.

Table B14.4: Soil/structure parameters used in predictive analyses

	Geotechnical		Structural		
	Model = H	$V_s=875$ fps	$h=48$ ft	$emb.=0$	$\gamma = 0.15$
NR	$\nu=0.4$		tr: $r_1 = 64$ ft		$r_{2-CW} = 29$ ft
	$\beta^*=3.2\%$	$V_s^*=840$ fps			$r_{2-ESW} = 48$ ft

* = strain-dependent soil properties determined from deconvolution analyses

Using the parameters in Table B14.4 with the fixed-base modal parameters in Table B14.2, the first-mode period lengthening ratio and foundation damping were predicted using the “modified Veletsos” formulation, with the results in Table B14.5.

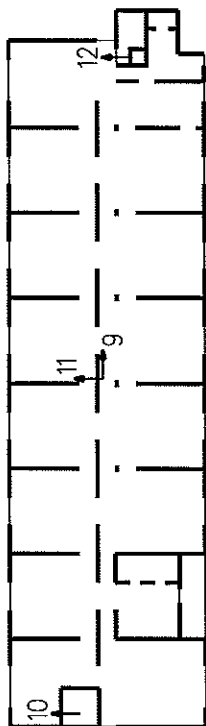
Table B14.5: Empirical and predicted inertial interaction effects

NR			tr (NS)		
			$\tilde{T} / T$	$\tilde{\zeta}_0 (\%)$	$\tilde{\zeta} (\%)$
	Coupled walls	Sys. ID	1.16	3.3	12.0
		Prediction	1.25	1.1	8.1
	Eq. single wall	Sys. ID	1.23	4.6	12.0
		Prediction	1.09	1.6	12.4

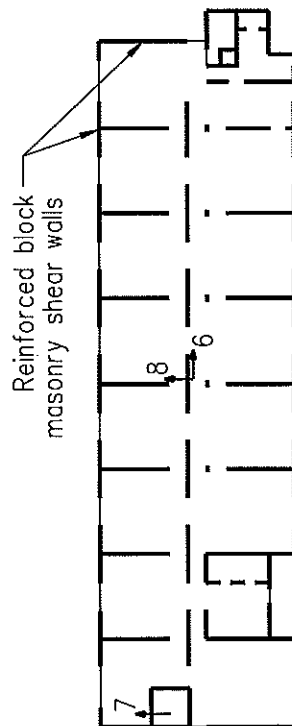
The period lengthening is overpredicted for the coupled shear wall case but significantly underpredicted for the equivalent single wall case. This indicates that the actual behavior of the structure is intermediate between these cases, but more closely approximated by the coupled walls model. The foundation damping was underpredicted for both cases.

Data Sources

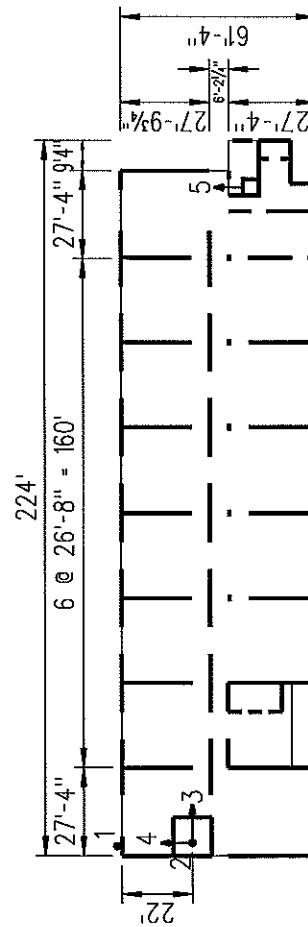
LeRoy Crandall Assoc., LCA (1978). “Report of Foundation Investigation, Proposed Building, Greenleaf Ave. between Mar Vista and Walnut Streets, Whittier, CA,” *Job. No. A-78333*.
Rodriguez-Ordóñez, J.A. (1994). “A new method for interpretation of surface wave measurements in soils,” *Ph.D. Dissertation*, North Carolina State University, Raleigh.



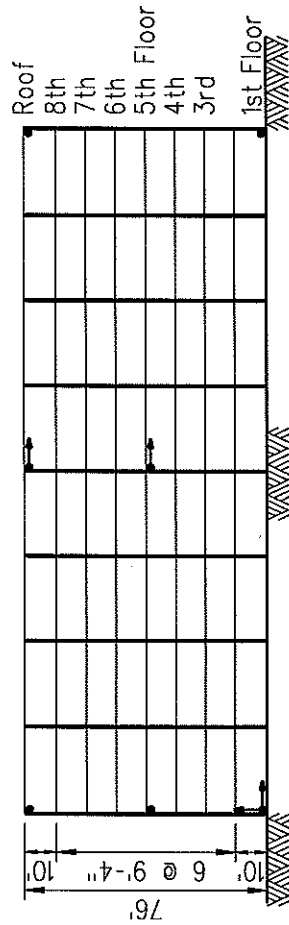
Roof Plan



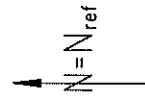
5th Floor Plan



1st Floor Plan

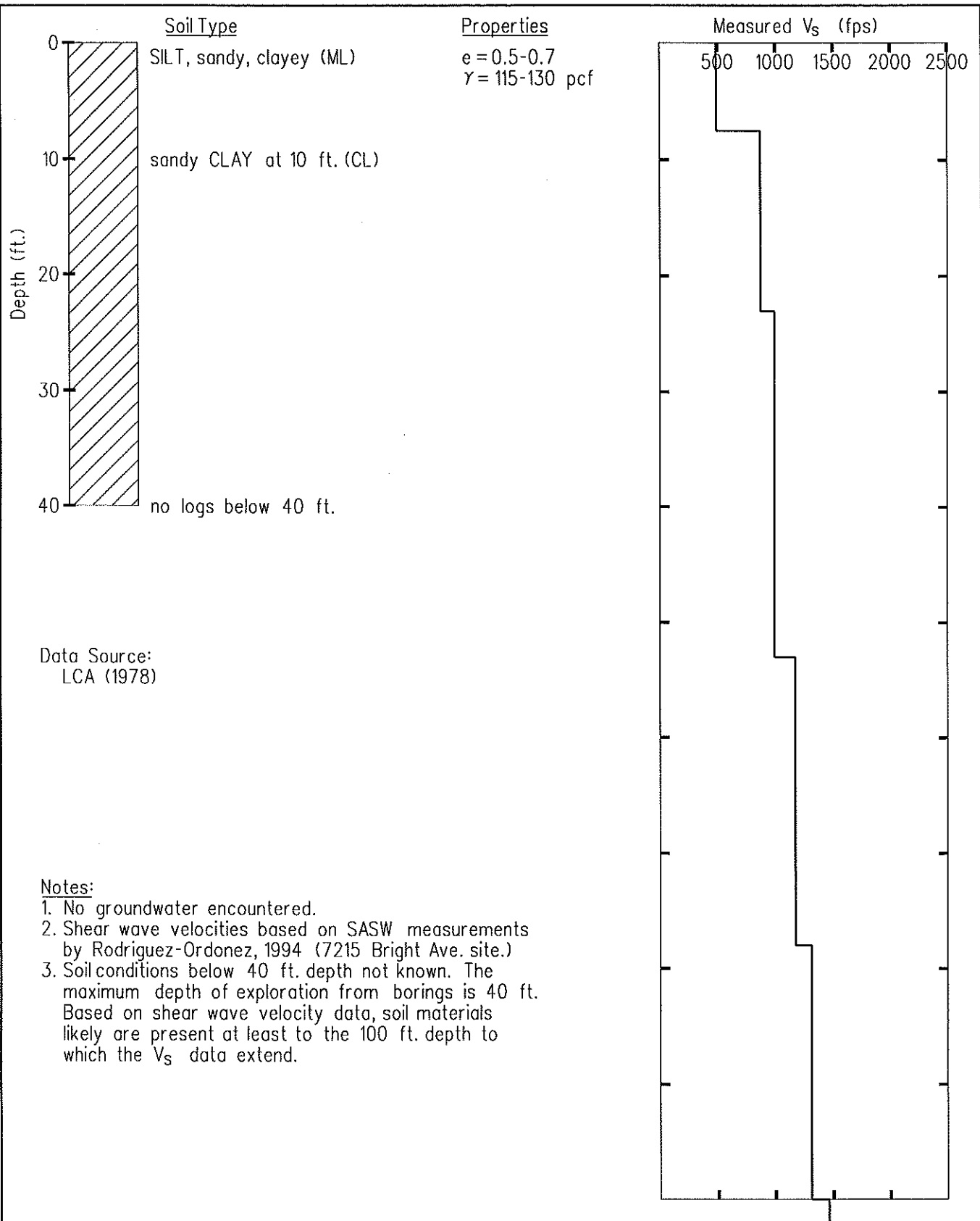


W / E Elevation

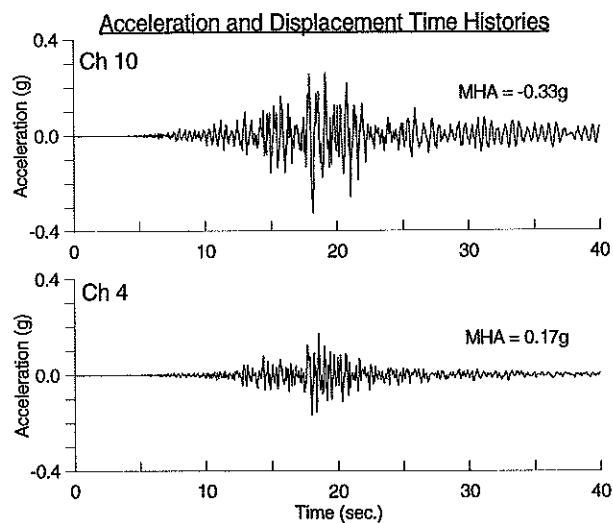
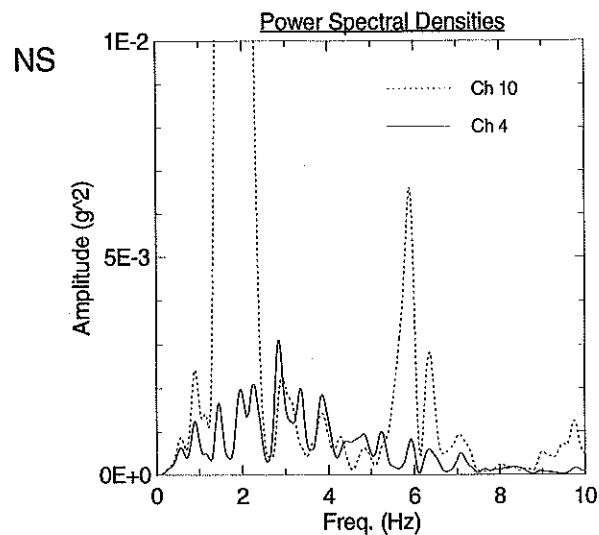
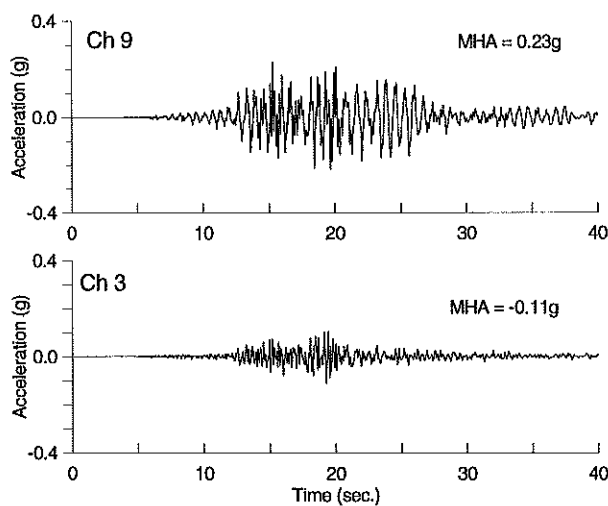
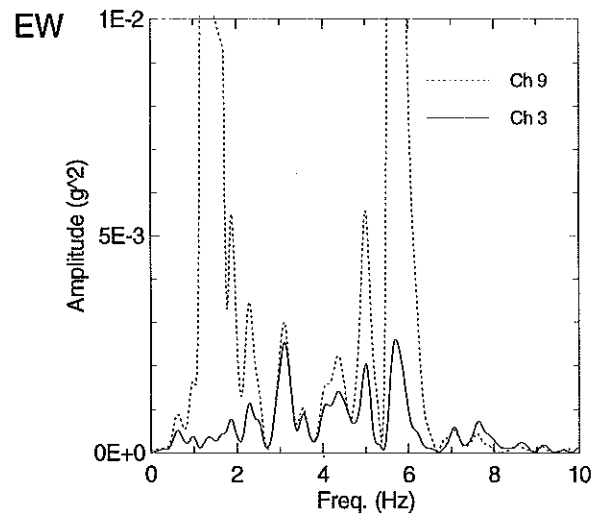
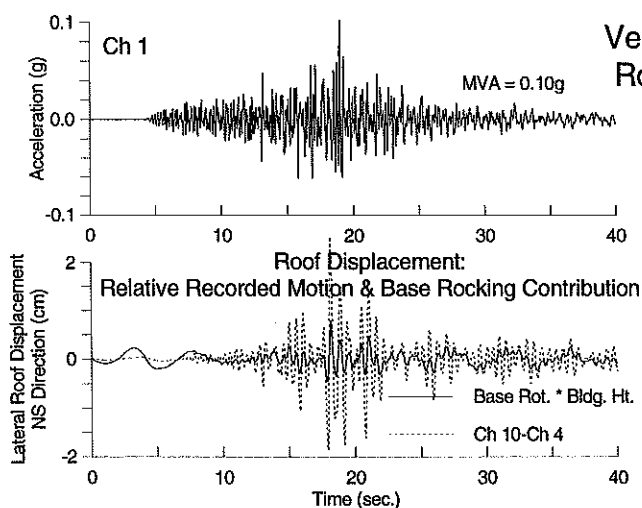


Sensor Locations
8-story Hotel Whittier, CA
Figure B14.1

Reinforced concrete block shear walls.
Design date: 1984



Generalized Soil Profile
8-story Hotel Whittier, CA
Figure B14.2



Vertical & Rocking

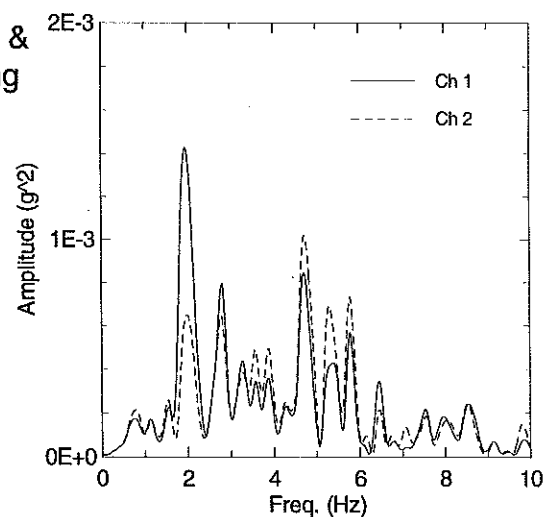


Fig. B14.NR1: WHITTIER 8 STORY OFFICE BUILDING
Time Histories and Power Spectral Density Functions
1994 Northridge Earthquake

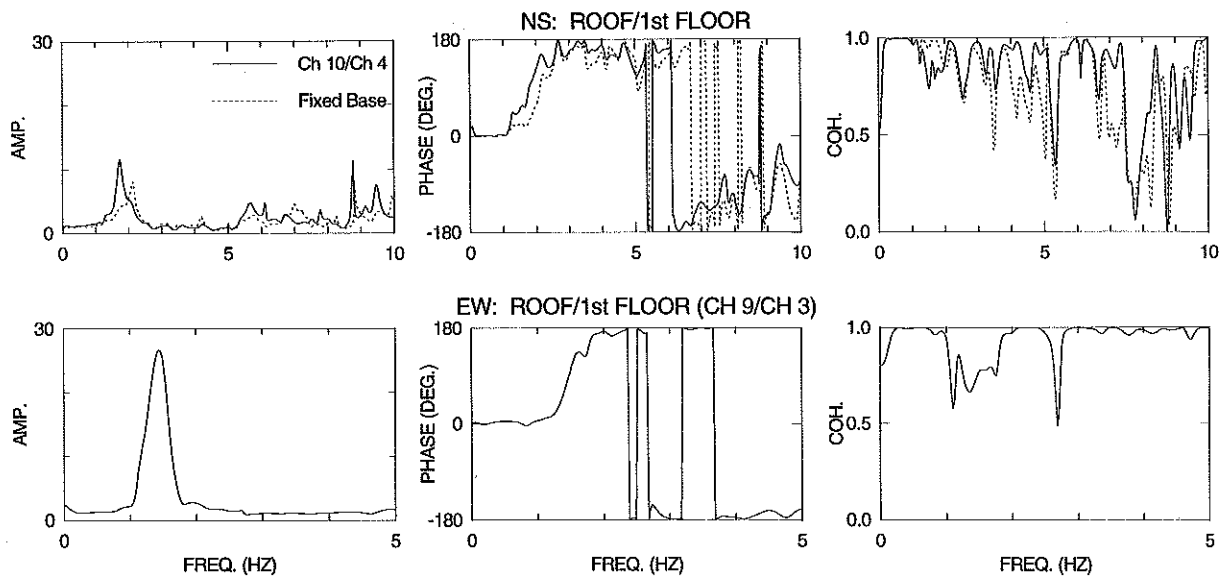


Fig. B14.NR2: WHITTIER 8 STORY OFFICE BUILDING
Transmissibility and Coherency Functions
1994 Northridge Earthquake

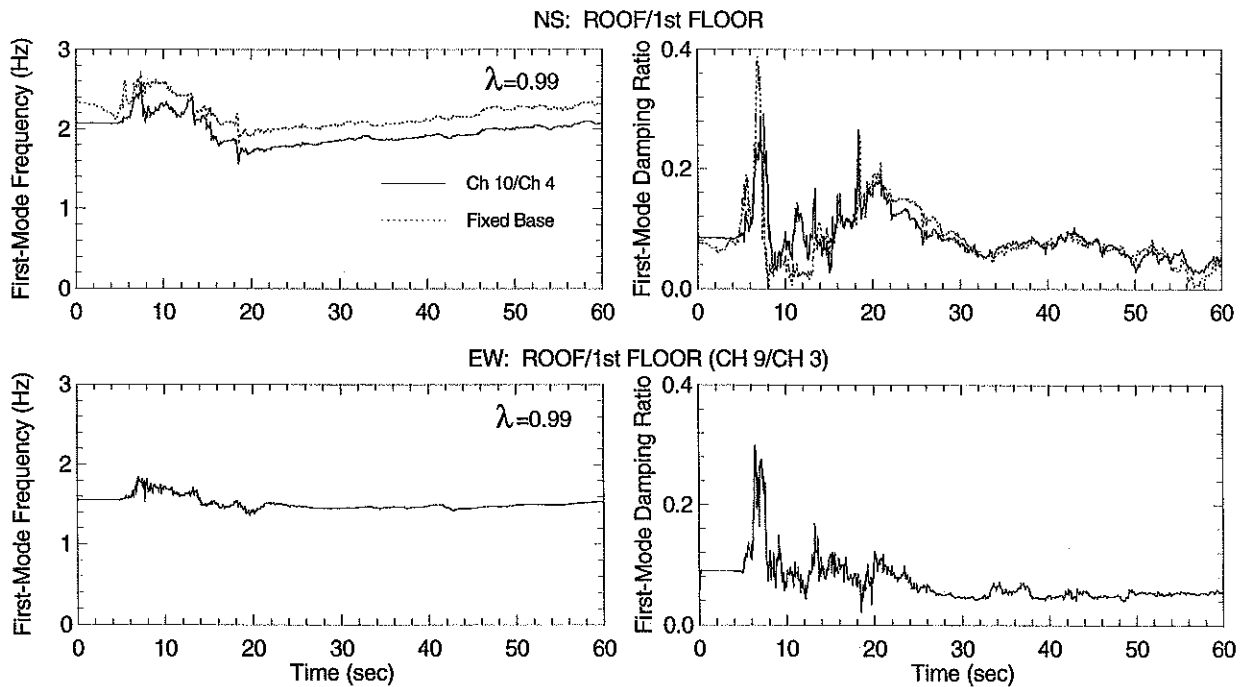


Fig. B14.NR3: WHITTIER 8 STORY OFFICE BUILDING
Time Variation of First-Mode Parameters
1994 Northridge Earthquake