TORSIONAL RESPONSE CHARACTERISTICS

OF REGULAR BUILDINGS UNDER

DIFFERENT SEISMIC EXCITATION LEVELS

by

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DISCLAIMER

PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of the this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Data Interpretation Project are to:

1.   Understand the spatial variation and magnitude dependence of earthquake strong ground motion.

2.   Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.

3.   Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.

4.   Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.

5.   Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is the sixth in a series of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Data Interpretation Research Project.

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ABSTRACT

Torsional response characteristics of three regular buildings in San Jose, and one in Watsonville, California, were studied by analyzing the strong motions recorded in these buildings during three recent earthquakes: 1989 Loma Prieta, 1986 Mt. Lewis, and 1984 Morgan Hill. The story shear forces, torsional moments and dynamic eccentricities in these buildings during the three earthquakes were obtained from an analysis of the recorded motions. The fundamental period of vibrations and damping ratios for these buildings were also estimated for the three earthquakes by using the Fourier Amplitude Spectra of the recorded motions. These results were then compared with the provisions of the 1988 Uniform Building Code. The results of our investigation indicate that the provisions of the 1988 UBC may sometimes not be adequate to realistically account for the torsional response of buildings during earthquakes, especially for steel moment frame buildings. The results of this investigation also indicate that the fundamental building period obtained using Method A in the 1988 UBC may be longer than the actual period of the building during earthquakes, especially for stiffer low to medium-rise buildings. This could result in unconservative estimate of earthquake design forces when using the static force procedure of the 1988 UBC.

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1.0 INTRODUCTION

The real response of buildings during earthquakes can, in general, be affected by the coupling of translational vibrations with rotational vibrations. If a three-dimensional dynamic analysis is performed as a part of the seismic design of the building, this coupled vibration, including dynamic amplification effects, can be estimated and used to guide the design. However, for a large class of buildings -- i.e., buildings that do not have plan or vertical irregularities and do not exceed certain height limits -- such a dynamic analysis is typically not required by most building codes [1]. Instead, an equivalent static force procedure is considered adequate for the seismic design of such buildings.

When using the equivalent static force procedure, most building codes prescribe simplified procedures for incorporating torsional effects into the seismic design of buildings. These provisions consider that the torsional moments at each story can be computed as the product of design story shear and a quantity named "design eccentricity". In most seismic codes, the design eccentricity consists of two components. The first component is a function of the distance between the center of mass and center of stiffness of each floor, called the static eccentricity. This part accounts for unsymmetrical distribution of mass and/or lateral load resisting elements in the plan of the structure. The second component of the design eccentricity, which is usually referred to as "accidental eccentricity", is to account for the effect of other factors; such as non-uniform ground motion along the foundation of the structure, torsional component of ground motions, and possible differences between actual

and computed eccentricities due to the uncertainties in workmanship and distribution of non-structural components.

The level of coupling between lateral and torsional responses in buildings during earthquakes can be larger than that implied by the Code equivalent static force procedure due to: (1) dynamic amplification effects; and (2) inelastic deformations which can, in some cases, lead to significantly higher lateral-torsional coupling than that predicted by elastic analysis.

The design eccentricity of most building codes can be written as:

$$e = \alpha e_s + \beta D \qquad (1\text{-}1)$$

where, α is a factor to account for dynamic amplification of static eccentricity e_s, and β is a factor to define accidental eccentricity in terms of a specified plan dimension, D, of the structure. The factors α and β are given different values in different building codes. For example,

α =   1.0 in 1988 Uniform Building Code [1]

=   1.5 in Mexican [2] and Canadian [3] Codes

β =   5% in UBC and Canadian Code

=   10% in Mexican Code

Thus, although an attempt has been made in the Mexican and Canadian Codes to allow for amplification in torsional moments by using an amplification factor, α, of 1.5, no such consideration is made in the 1988 UBC.

Examples of coupled lateral-torsional vibrations have been observed during past earthquakes, particularly during the September 1985 Mexico earthquake, where large torsion was reported to be one of the major factors responsible for the severe damage or collapse of several structures. Recent research [4,5,6] has confirmed that the building code provisions to account for torsional effects in regular buildings can, sometimes, be inadequate. Since majority of the past research has been of analytical nature on one-story idealized structures, the present investigation was undertaken to corroborate these findings by analyzing the measured response of actual buildings during past earthquakes.

The objective of this investigation was to analyze strong motions recorded in regular buildings during past earthquakes to study their torsional response characteristics. For three different levels of excitation represented by the 1989 Loma Prieta, 1986 Mt. Lewis, and the 1984 Morgan Hill earthquakes, the distribution of shear forces, torsional moments and dynamic eccentricities over the height of four buildings located in San Jose and Watsonville area were estimated. The fundamental periods of vibration and damping ratios for these buildings were also estimated from the recorded motions. These response quantities were then compared with the provisions of the 1988 Uniform Building Code [1] (UBC) to assess their adequacy.

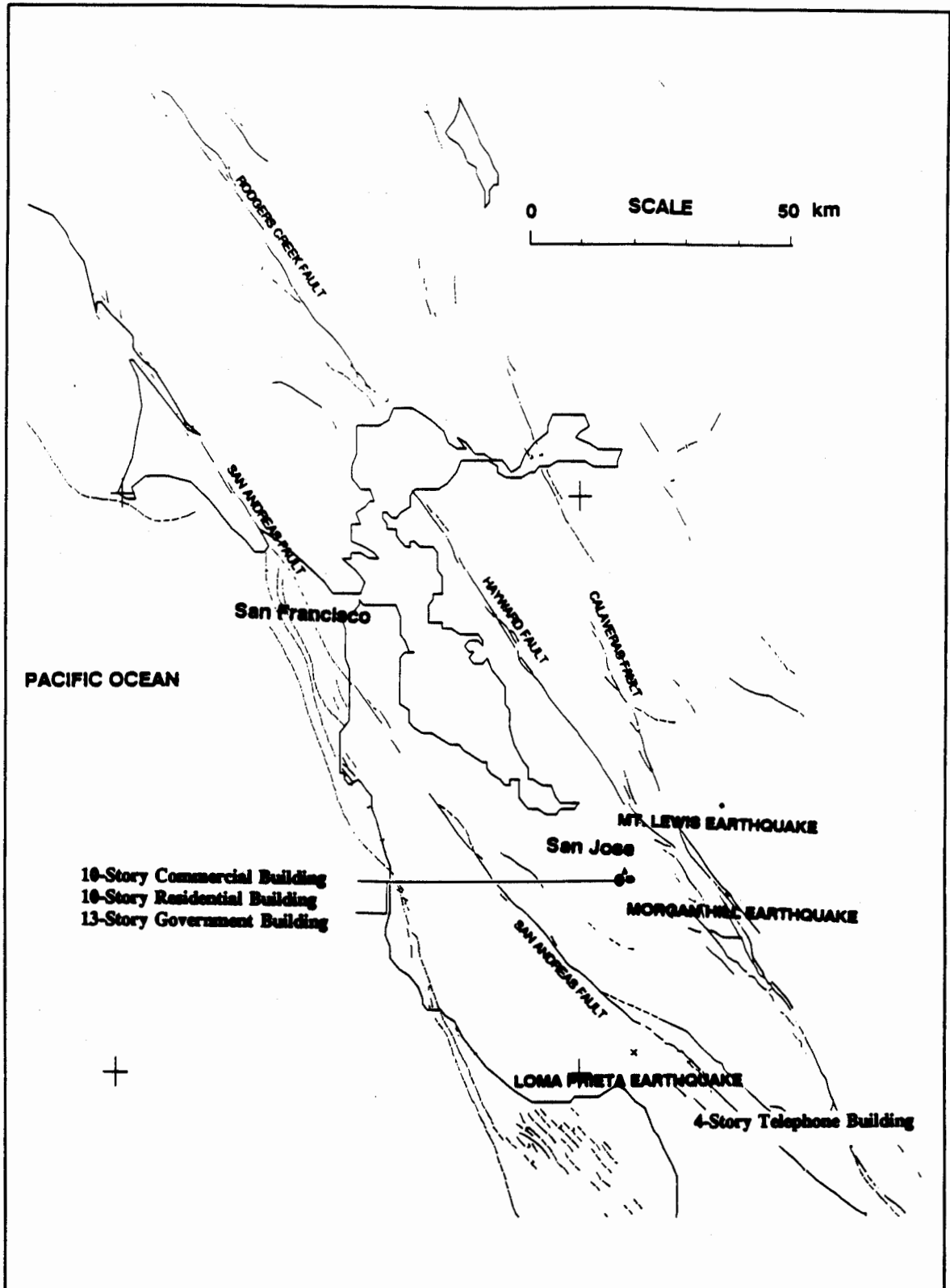## 2.0 SELECTED BUILDINGS AND RECORDED STRONG MOTIONS

Four buildings, three in San Jose and one in Watsonville, were selected for this study. These buildings are: 10-Story Residential, 10-Story Commercial, and 13-Story Government Buildings, all in San Jose; and a 4-Story Telephone Building in Watsonville. The basic structural features, and the motions recorded in these buildings during the 1989 Loma Prieta (magnitude 7.1), 1986 Mt. Lewis (magnitude 5.5), and 1984 Morgan Hill (magnitude 6.2) earthquakes are summarized in Table 2-1.

Figure 2-1 shows the location of the selected buildings relative to the epicenters of the three earthquakes and known active faults in the area. The Mt. Lewis earthquake was closest to the three buildings located in San Jose, and the Loma Prieta earthquake was closest to the building in Watsonville. Figures 2-2 to 2-5, reproduced from Reference 7, show the basic configuration and structural features, and also the location of the strong motion instrumentation for the four buildings.

The selected buildings represent different types of structural systems. The configurations of these buildings and the location of their lateral loads resisting elements is fairly regular, except for the 4-story building which has relatively large static eccentricity due to non-symmetric location of shear walls. The selected buildings also have relatively rigid in-plane floor diaphragms and all are relatively well instrumented so as to allow estimation of the torsional accelerations and their distribution over the height of the structure.

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The structural drawings for the buildings were provided by the California Division of Mines & Geology, Strong Motion Instrumentation Program (CSMIP). The corrected building motions, their response spectra, and Fourier Amplitude Spectra were also provided by the CSMIP on floppy disks.

TABLE 2-1 - SUMMARY OF RECORDED MOTIONS IN SELECTED BUILDINGS										
Bldg. Location (and type)	CSMIP Station Number	Stories	Lateral Force System	Peak Acceleration, g						
				Loma Prieta Earthquake (1989)		Mt. Lewis Earthquake (1986)		Morgan Hill Earthquake (1984)		
				Base	Roof	Base	Roof	Base	Roof
San Jose (Residential)	57356	10	SW	0.13	0.37	0.04	0.12	0.06	0.22
San Jose (Commercial)	57355	10	SW/MRCF	0.11	0.39	0.04	0.08	0.06	0.22
Watsonville (Telephone)	47459	4	SW	0.66	1.24	N.A.	N.A.	0.11	0.33
San Jose (Government)	57357	13	MRSF	0.11	0.36	0.04	0.32	0.04	0.17

Notes: SW   = Reinforced Concrete Shear Wall   MRSF   = Moment Resisting Steel Frame

MRCF   = Moment Resisting Concrete Frame   N.A.   = Not Available

PACIFIC OCEAN

ROGERS CREEK FAULT

SAN ANDREAS FAULT

HAYWARD FAULT

CALAVERAS FAULT

San Francisco

San Jose

MT. LEWIS EARTHQUAKE

MORGAN HILL EARTHQUAKE

SAN ANDREAS FAULT

LOMA PRIETA EARTHQUAKE

4-Story Telephone Building

10-Story Commercial Building
10-Story Residential Building
13-Story Government Building

0　　SCALE　　50 km

FIGURE 2-1:　LOCATION OF BUILDINGS AND EARTHQUAKE EPICENTERS
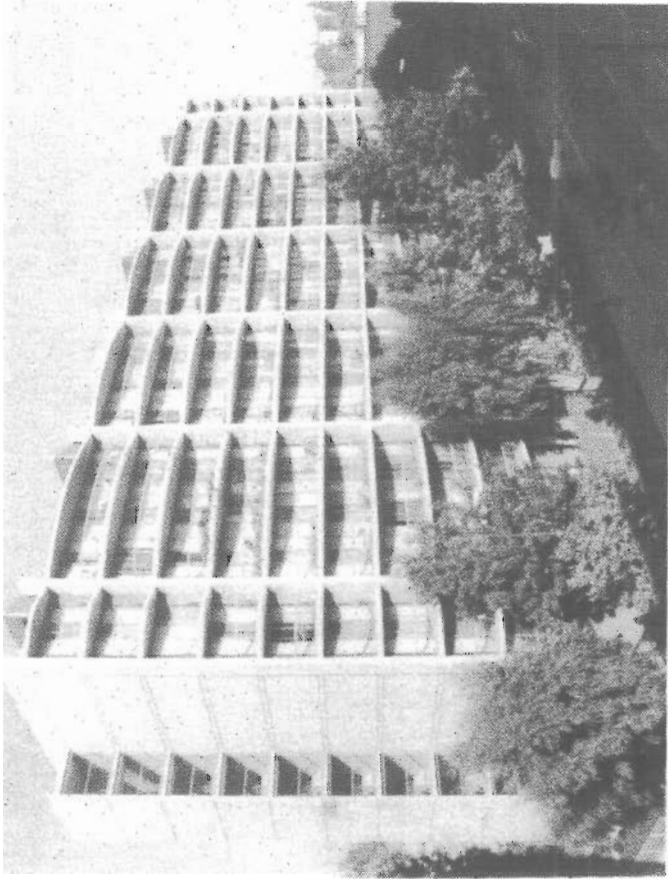
No. of Stories above/below ground: 10/0
Plan Shape: Rectangular
Base Dimensions: 210' x 64'
Typical Floor Dimensions: Same as base
Design Date: 1971
Construction Date: 1971-72

Vertical Load Carrying System:
  One-way post-tensioned flat slabs on reinforced concrete bearing walls.
Lateral Force Resisting System:
  Concrete shear walls at regular intervals in transverse direction, and along interior corridors in longitudinal direction.
Foundation Type:
  Precast-prestressed concrete piles under all walls.

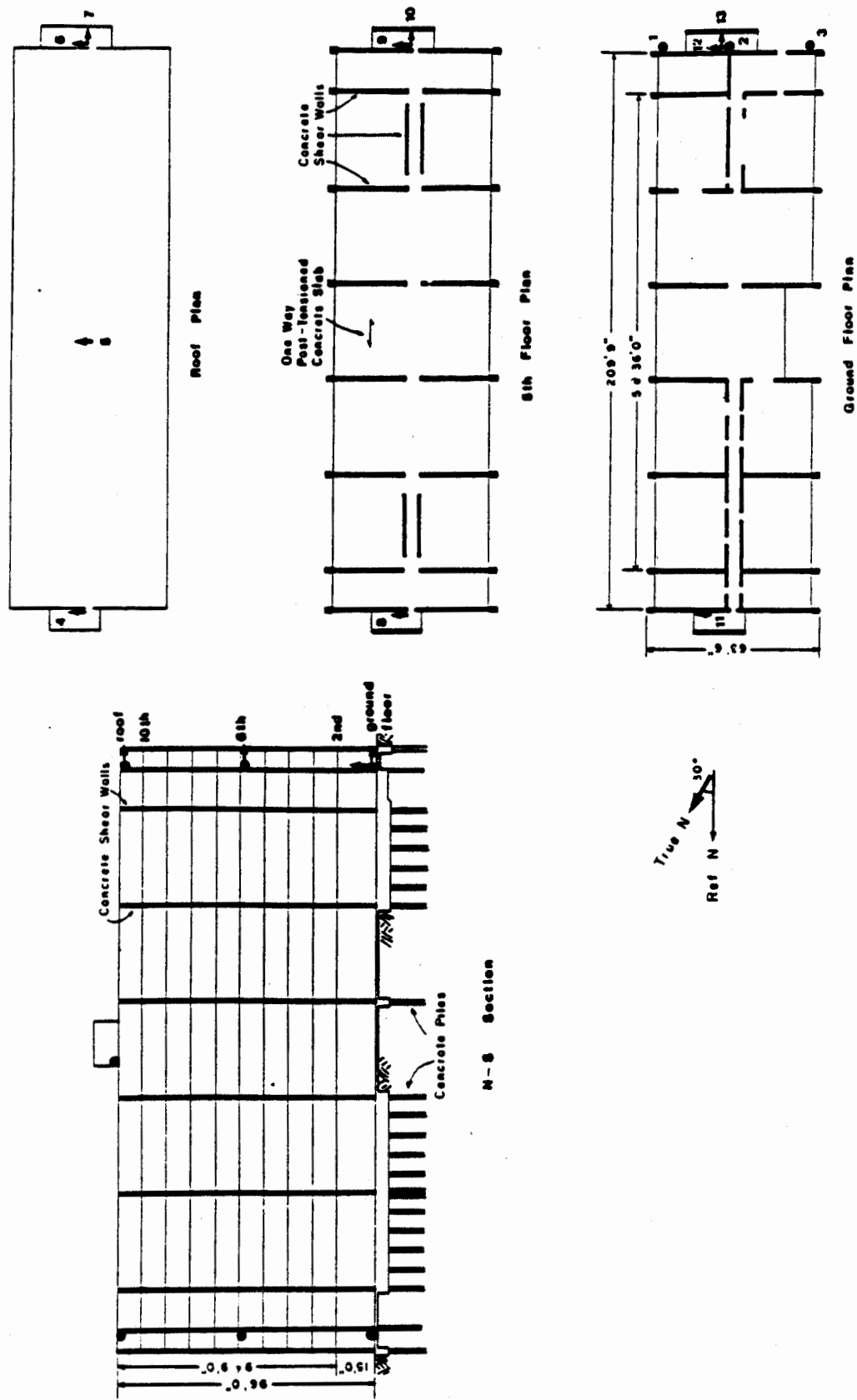**FIGURE 2-2a: 10-STORY RESIDENTIAL BUILDING, SAN JOSE [7]**

FIGURE 2-2b: 10-STORY RESIDENTIAL BUILDING, SAN JOSE - SENSOR LOCATIONS [7]

No. of Stories above/below ground: 10/1
Plan Shape: Rectangular
Base Dimensions: 190' x 96'
Typical Floor Dimensions: 190' X 82'
Design Date: 1964
Construction Date: 1967

Vertical Load Carrying System:
   Concrete floor slabs suported by concrete pan joists and concrete frames.
Lateral Force Resisting System:
   End concrete shear walls in transverse direction; moment-resistant concrete frame in longitudinal direction.
Foundation Type:
   Mat foundation.

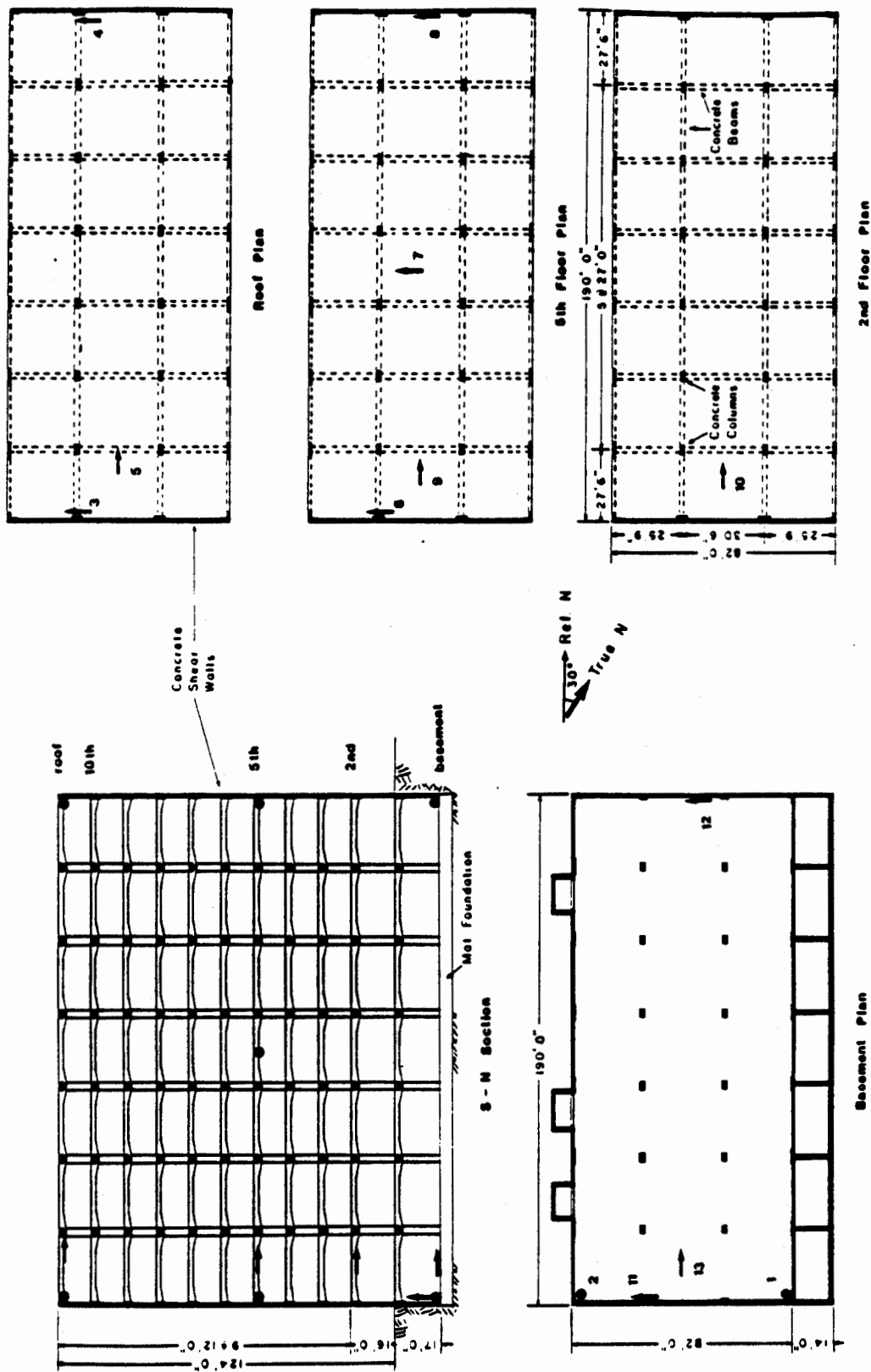**FIGURE 2-3a: 10-STORY COMMERCIAL BUILDING, SAN JOSE [7]**

FIGURE 2-3b: 10-STORY COMMERCIAL BUILDING, SAN JOSE - SENSOR LOCATIONS [7]

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No. of Storeis above/below ground: 13/0

Plan Shape: Rectangular

Base Dimensions: 173' x 167'

Typical Floor Dimensions: 167' x 167

Design Date: 1972

Construction Date: 1975-76

Vertical Load Carrying System:
3.5" concrete slab on metal deck; steel columns, beams and joists.

Lateral Force Resisting System:
Moment-resistant steel frame.

Foundation Type:
Mat foundation.

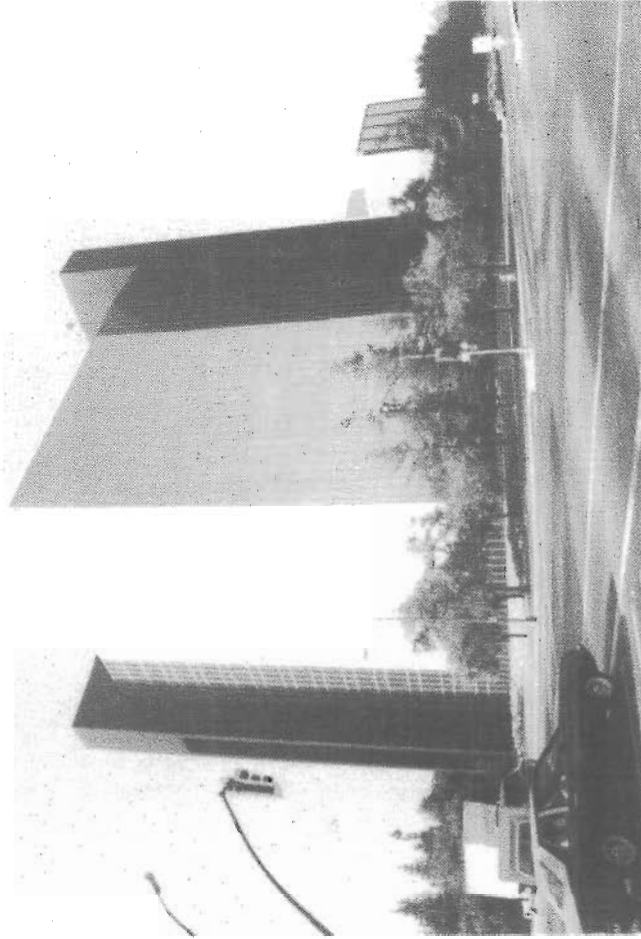**FIGURE 2-4a: 13-STORY GOVERNMENT BUILDING, SAN JOSE [7]**

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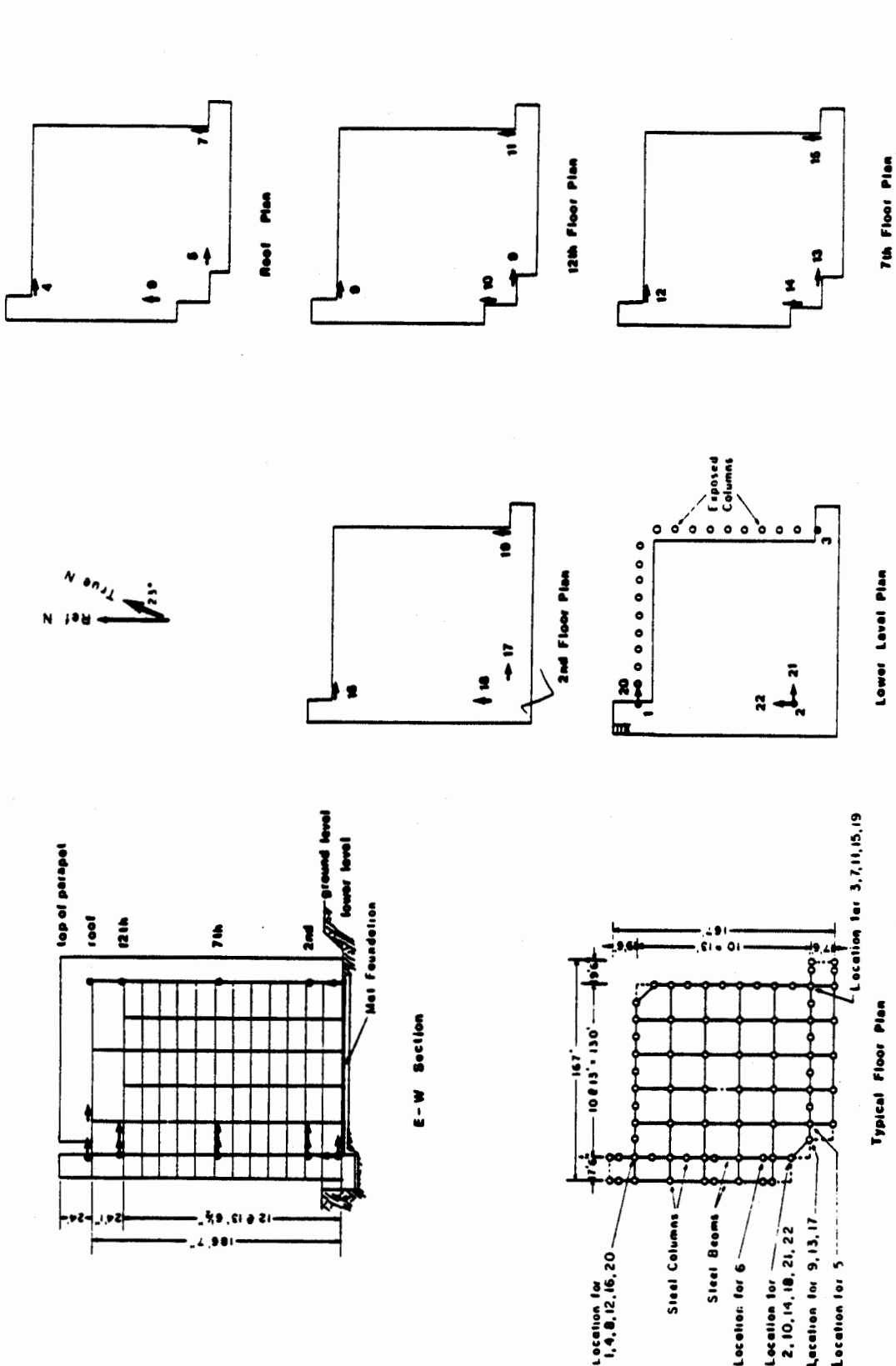
FIGURE 2-4b: 13-STORY GOVERNMENT BUILDING, SAN JOSE - SENSOR LOCATIONS [7]

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No. of Stories above/below ground: 4/0
Plan Shape: Rectangular
Base Dimensions: 75' x 71'
Typical Floor Dimensions: Same as base
Design Date: 1948 & 1955
Construction Date: 1948 & 1955
Note: 4th story was added in 1955

Vertical Load Carrying System:
  Composite concrete/steel beams, girders and
  columns; concrete slabs.
Lateral Force Resisting System:
  Concrete shear walls.
Foundation type:
  Spread footings.

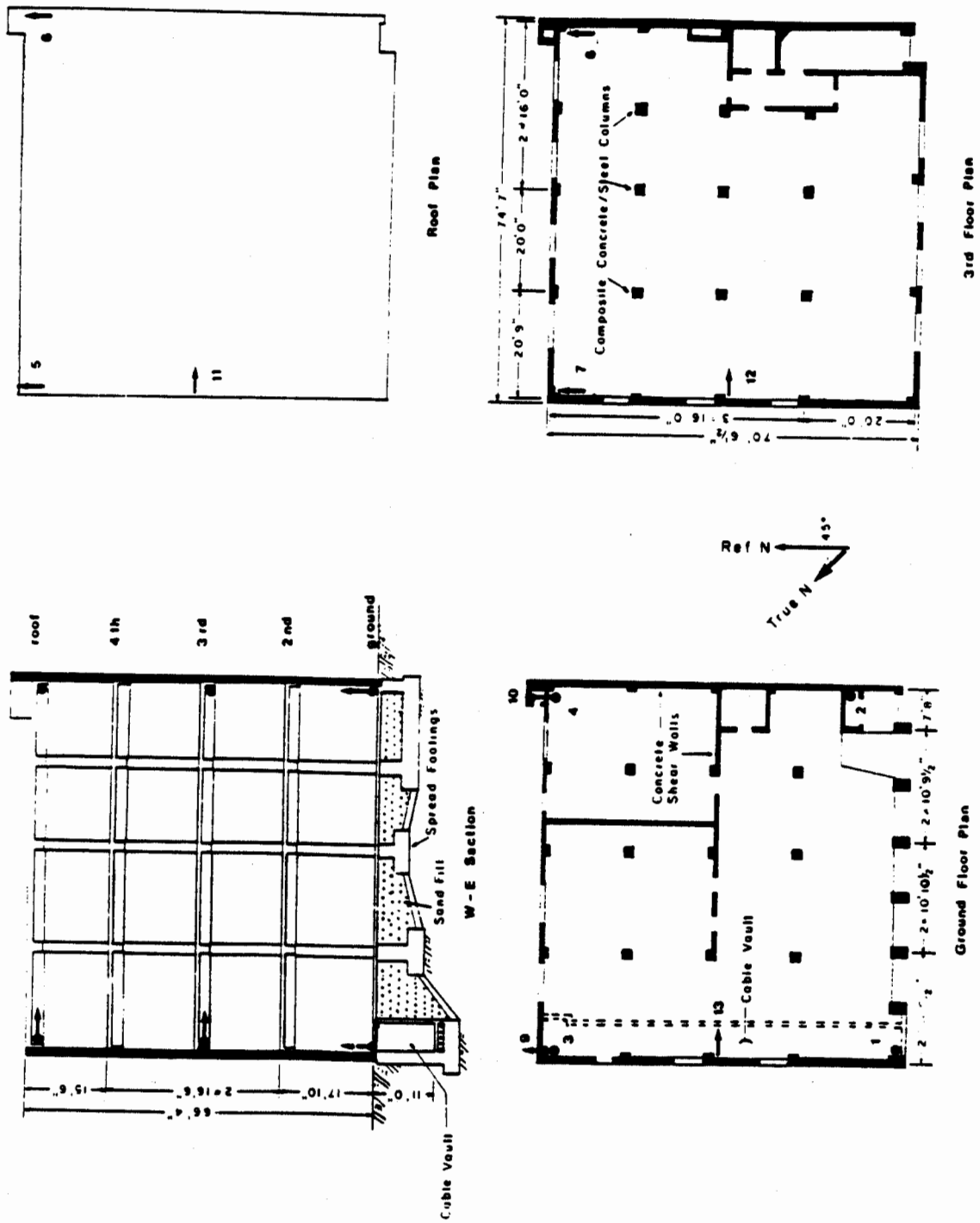**FIGURE 2-5a: 4-STORY TELEPHONE BUILDING, WATSONVILLE [7]**

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FIGURE 2-5b: 4-STORY TELEPHONE BUILDING, WATSONVILLE - SENSOR LOCATIONS [7]

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3.0 METHODOLOGY

The actual level of coupling between lateral and torsional responses of buildings in earthquakes can be represented through the use of a "dynamic eccentricity", e_d, computed as the ratio of the actual maximum dynamic torsional moment about the center of stiffness of each floor divided by the maximum dynamic shear force in a corresponding uncoupled system in which the centers of mass and stiffness are coincident [4,5,6]. In the present investigation, this dynamic eccentricity has been estimated by using the strong motions recorded in buildings.

The torsional rotation and lateral translation acceleration time-histories at the center of mass of each floor for a building with rigid floor diaphragms may be obtained by using the recorded responses at two points on that floor from the following relations:

$$\ddot{U}_\Theta(t) = (\ddot{U}_2(t) - \ddot{U}_1(t))/D$$
$$\ddot{U}_y(t) = \ddot{U}_1(t) - x\ddot{U}_\Theta(t)$$

(3-1)

where :

$\ddot{U}_\Theta(t)$ is the _computed_ rotational acceleration time-history (as a function of time t).

$\ddot{U}_y(t)$ is the _computed_ translational acceleration time-history (as a function of time t) at the center of mass of the floor.

$\ddot{U}_1(t)$ is the _recorded_ translational acceleration time-history at point 1 of the floor.

$\ddot{U}_2(t)$ is the _recorded_ translational acceleration time-history at point 2 of the floor.

D is the distance between points 1 and 2 for each floor.

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x *is the horizontal coordinate of point 1 with respect to the center of mass of the floor.*

The above parameters are all shown in Figure 3-1.

The torsional moment time-history about the center of mass of the *jth* floor may be calculated from:

$$T_{Mj}(t) = \sum_{j}^{n} m_j r_j^2 \, \ddot{U}_{\Theta j}(t) \tag{3-2}$$

and the coupled shear force time-history of the *jth* floor can be computed from:

$$V_j(t) = \sum_{j}^{n} m_j \ddot{U}_{yj}(t) \tag{3-3}$$

where:

     $T_{Mj}(t)$ *is the torsional moment time-history about the center of mass of the jth floor.*

     $V_j(t)$ *is the coupled shear force time-history at the jth floor.*

     r_j *is the radius of gyration of the jth floor about center of mass.*

     n *is the total number of stories in the building.*

The torsional moment time-history about the center of stiffness of the jth floor, $T_{sj}(t)$, is then:

$$T_{Sj}(t) = T_{Mj}(t) - e_j \, V_j(t) \tag{3-4}$$

where:

     e_j *is the eccentricity between centers of mass and stiffness of the jth floor.*
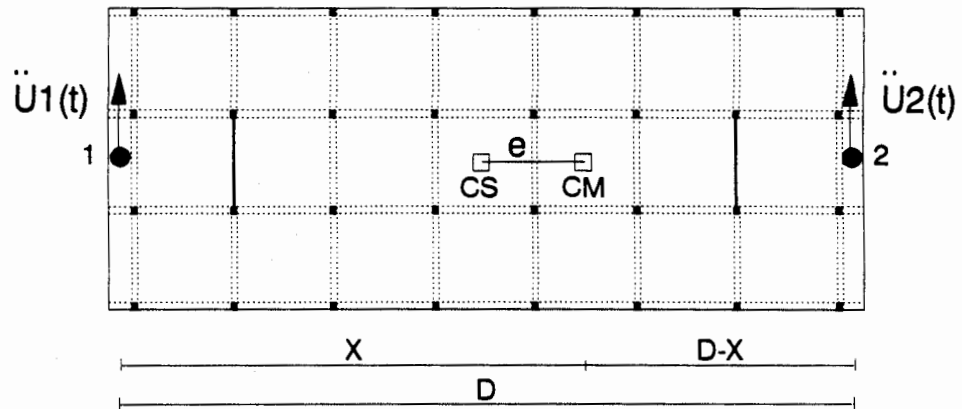
$\ddot{U}1(t)$ Recorded Acceleration Time History at Point 1

$\ddot{U}2(t)$ Recorded Acceleration Time-History at Point 2

CM   Center of Mass

CS   Center of Stiffness

e   Static Eccentricity

FIGURE 3-1: SCHEMATIC PLAN VIEW OF A TYPICAL BUILDING

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The maximum uncoupled story shear force at the jth floor can be approximated by:

$$V_{0j} = \sum_{i=j}^{n} f_{0i} \qquad (3\text{-}5)$$

and

$$f_{0i} = \frac{\sum m_j \phi_j}{\sum m_j \phi_j^2} \, (m_i \phi_i) \, S_a \qquad (3\text{-}6)$$

where:

V_{oj} *is the maximum <u>uncoupled</u> story shear at the jth floor.*

ϕ_i *is the ordinate of the fundamental mode shape at the ith floor.*

S_a *is the pseudo absolute acceleration of ground motion.*

Equations 3-5 and 3-6 are approximations of story shear and story lateral force because they are calculated based on the fundamental mode shape only. The value of dynamic eccentricity at each floor can be calculated from:

$$e_{dj} = (T_{Sj}(t))_{max} \, / \, V_{0j} \qquad (3\text{-}7)$$

where:

e_{dj} *is the dynamic eccentricity at the jth floor.*

$(T_{sj}(t))_{max}$ *is the maximum value of the torsional moment time history obtained by Equation 3-4.*

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Using the Equations 3-1 to 3-7, the following steps were then taken to achieve the objectives of this study:

Step 1: From the information shown on the structural drawings for the buildings provided by the CSMIP, the mass, location of the center of mass (CM), the radius of gyration, and the mass moment of inertia for each floor of each building were computed. The relative floor stiffness and the location of the center of stiffness (CS) for each floor were approximated by simplified hand calculations.

Step 2: The acceleration time-histories at those floors that were not instrumented were obtained by interpolation using 2nd order polynomial functions at each time step. From the translational accelerations at the two ends of the floor diaphragm, the rotational and translational acceleration time-histories in the transverse direction of the building were then calculated at the center of mass of each floor by assuming rigid floor diaphragm behavior (Equation 3-1). These were used to calculate the coupled shear force time-histories at the CM (Equation 3-2) and the torsional moment time-histories about the CS for each floor (Equation 3-4).

Step 3: In this step, the fundamental period of vibration, mode shape and the damping ratio for each building were estimated. The fundamental period and damping were estimated from the transfer functions of the Fourier Amplitude Spectrum (FAS) of the recorded motions provided by the CSMIP. The transfer functions were computed as the ratio of the FAS of

the recorded roof motions to the FAS of the corresponding motions at the ground level. For example, Figure 3-2 shows the FAS of the recorded motions at the roof and ground during Loma Prieta earthquake for the 10-Story Residential Building. Figure 3-3 shows the corresponding transfer functions for this building as well as for the other three buildings from the Loma Prieta earthquake motions. The FAS transfer functions exhibited well defined peaks. For the 4-Story Telephone and 10-Story Commercial Buildings, it was noted that the second mode had as much or more energy as exhibited by the peaks in the FAS transfer functions. The fundamental frequency of vibration in the transverse direction of the building was taken as the frequency at the location of the first peak [12, 13] and the damping ratio was estimated by applying the half-power method to that peak [12, 13]. The fundamental mode shape was taken as the deflected shape of the building at the instant of peak roof acceleration. This is a simple but commonly applied method to approximate the first mode of vibration of the building. This approximation will neglect the contribution of higher modes to the response of the building which in these cases does not appear to be important.

Step 4: The estimated fundamental periods, damping ratios, and mode shapes were used to obtain the spectral accelerations [7] and to compute uncoupled lateral forces by assuming the fundamental mode response as indicated by Equations 3-5 and 3-6. S_a in Equation 3-6 was obtained from the response spectra provided by CSMIP. Since, as discussed later, the modal damping ratios estimated under Step 3 using the half power method were shown in this report to be not very reliable, the spectral accelerations and uncoupled lateral forces were

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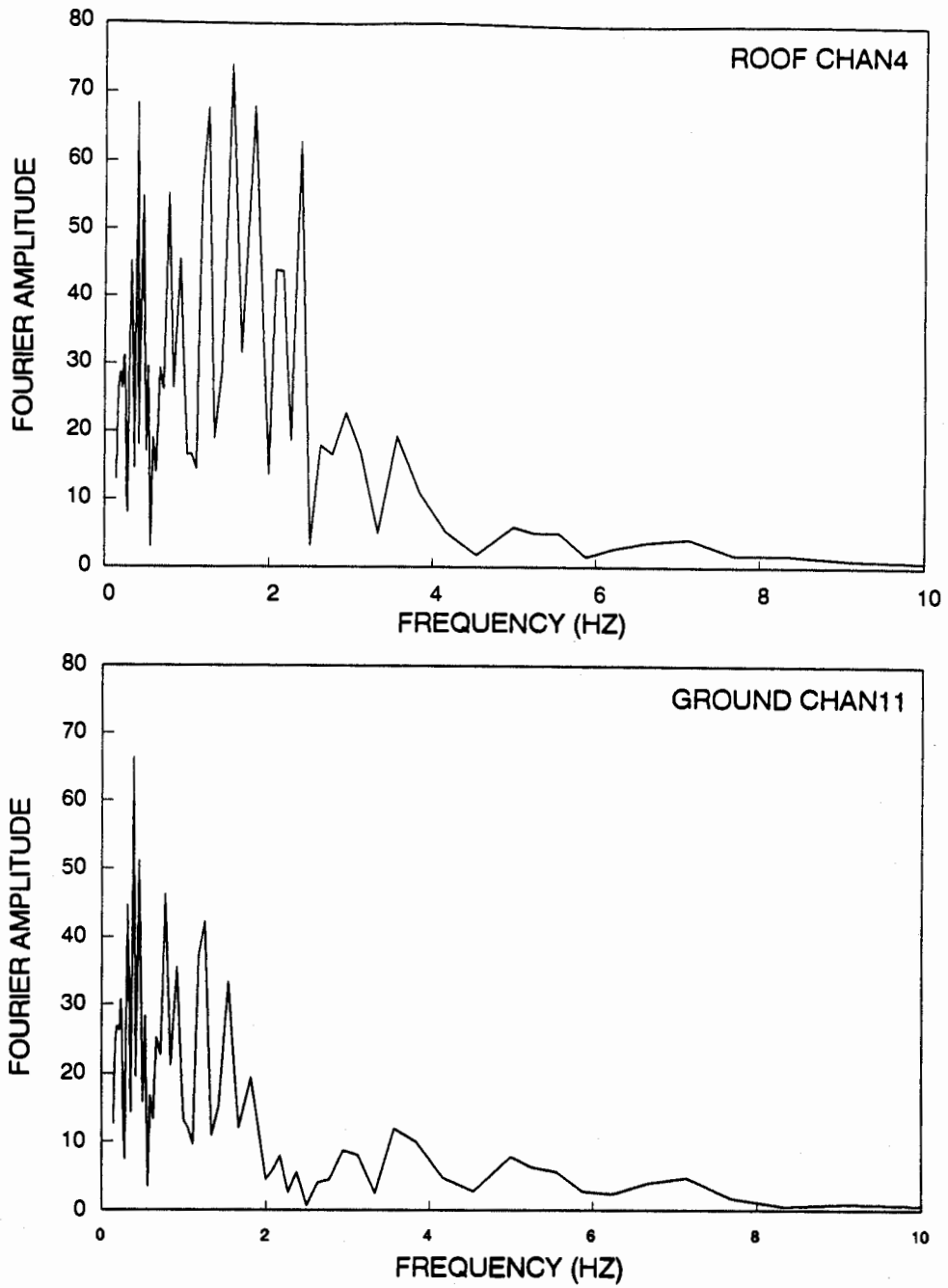
FIGURE 3-2: FOURIER AMPLITUDE SPECTRA, 10-STORY RESIDENTIAL BUILDING, LOMA PRIETA EARTHQUAKE

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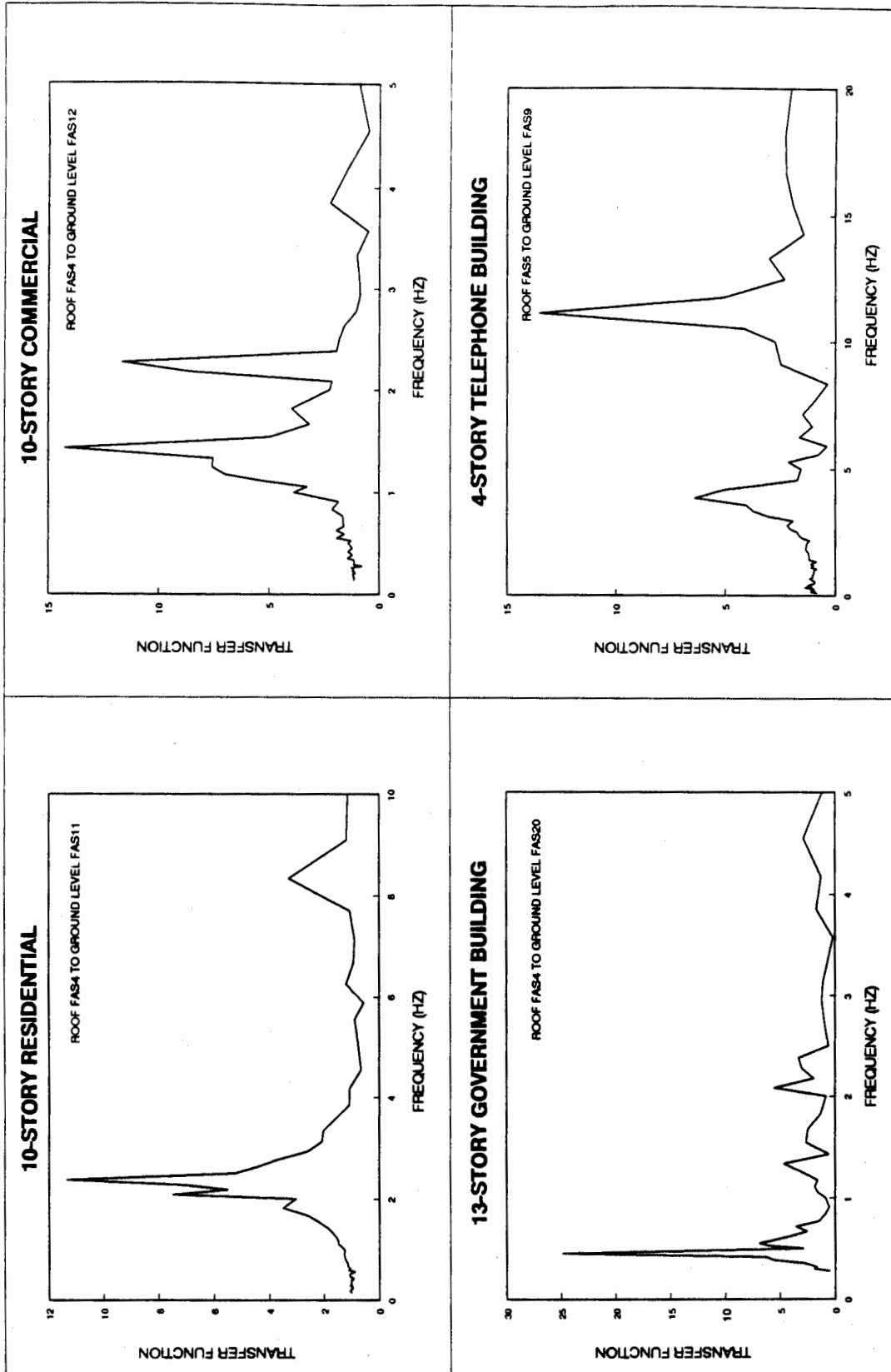
FIGURE 3-3: FOURIER AMPLITUDE SPECTRUM (FAS) TRANSFER FUNCTIONS

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instead estimated for damping ratios of 2%, 5%, and 10%, that were chosen as estimated bounds to the actual damping in the building. The total dynamic eccentricity at each floor was then obtained by dividing the maximum torsional moment about the CS calculated in Step 3 by the uncoupled story shear obtained from the uncoupled lateral forces.

Step 5: The design shear forces and the torsional moments were calculated using the provisions of the 1988 UBC. Since the buildings selected are fairly regular and under 240 feet in height, the static force procedure of the 1988 UBC was used for these computations. These values were then compared with those obtained for the three earthquakes from Step 1 to 4 above. The ratios of the total dynamic eccentricity as obtained in Step 4, to the total design eccentricity as prescribed by the 1988 UBC were also obtained. Since the actual measured motions are coupled, the methodology described above considers both the lateral and torsional motions. Also, since the buildings selected for this study have relatively rigid floor diaphragms, the recorded transverse motions completely define the torsional motions in the building.

4.0 DISCUSSION OF RESULTS

Fundamental Period of Vibration:

The fundamental periods of vibration for the four buildings as obtained from the FAS transfer functions are summarized in Table 4-1 for the three earthquakes. This table also shows the building periods as obtained from Method A of the 1988 UBC. As discussed earlier, only the transverse direction was considered in this study. These results indicate that, except for the 13-Story Government Building, the building periods predicted by 1988 UBC are 25% to 100% higher than the periods estimated from the recorded motions. This discrepancy was higher for the stiffer 4-story building. This suggests that the 1988 UBC equation for period calculation may sometimes be unconservative when used to calculate equivalent static earthquake design forces. For the 13-story building, the periods estimated from recorded motions were actually 100% higher than those given by 1988 UBC. These results also show lengthening of periods for the Loma Prieta earthquake, indicating that the buildings probably experienced some inelastic deformations during the Loma Prieta earthquake. The period of the 13-Story Government Building was not observed to lengthen during the Loma Prieta earthquake, probably because the amount of structural damage was not significant. However, the reason this building vibrated for such a long duration during this earthquake was most likely due to the coupling of lateral and torsional motions. This coupling can especially be very pronounced when the damping of the structure is low. The periods obtained here also compared well with those from previous studies [8], including those utilizing more sophisticated system identification methods [9].

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TABLE 4-1 - FUNDAMENTAL PERIOD OF BUILDINGS				
EARTHQUAKE	PERIOD (Seconds)			
	10-Story Residential Building (E.W.)	10-Story Commercial Building (E.W.)	4-Story Telephone Building (N.S.)	13-Story Government Building (E.W.)
Loma Prieta	0.45	0.7	0.22	2.2
Mt. Lewis	0.42	0.6	N.A.	2.2
Morgan Hill	0.42	0.6	0.21	2.2
UBC 88	0.61	0.74	0.46	1.01

Damping Ratios:

Table 4-2 summarizes the damping ratios obtained by applying the half-power bandwidth method to the FAS transfer functions. For each building, two damping ratios, corresponding to FAS transfer functions for the motions recorded at the two ends of the diaphragm, were obtained. A large variation in the damping ratio so obtained was observed. The damping ratios for the 10-Story Residential and Commercial Buildings also did not agree well with those obtained for these buildings from system identification methods [9]. This suggests some inherent limitations in the half-power method. The unreliability of half-power method for estimating damping may stem from several factors, such as: the presence of noise in the measured response; representation of complex energy dissipation phenomena with a simplified viscous damping ratio; the representation of actual nonlinear behavior with linear behavior; and, for some structures, closely spaced modes. Some of these difficulties

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in the use of half-power method to estimate damping ratios from earthquake motions have been pointed out by Beck and Beck [10].

| TABLE 4-2 - DAMPING RATIO FOR BUILDINGS				
EARTHQUAKE	DAMPING (%)			
	10-Story Residential Building (E.W.)	10-Story Commercial Building (E.W.)	4-Story Telephone Building (N.S.)	13-Story Government Building (E.W.)
Loma Prieta	3.7 to 9.8	4.1 to 4.6	7 to 11	3.2
Mt. Lewis	5.5 to 8.0	2.6 to 3.3	N.A.	3.4 to 4.7
Morgan Hill	2.1 to 2.8	2.4 to 2.6	5.1 to 13.9	4.3

Shear Forces:

The computed shear force time-histories due to Loma Prieta earthquake at the roof, the fifth floor and the first floor of the 10-Story Commercial Building are shown in Figure 4-1. The base shear time-history response of this building to the Loma Prieta, Mt. Lewis and Morgan Hill earthquakes are compared with the base shear presecribed by the 1988 UBC regulations in Figure 4-2. Similar time-histories were also obtained for the other buildings.

From the time-histories of the story shear forces the maximum shear forces over the height of the buildings were obtained. These are shown in Figure 4-3 and the ratio of the earthquake induced base shear forces to the 1988 UBC base shear forces are summarized in Table 4-3. It can be observed that the maximum base shear experienced by the 10-Story Residential Building during the Loma Prieta earthquake was about 15% larger than the 1988
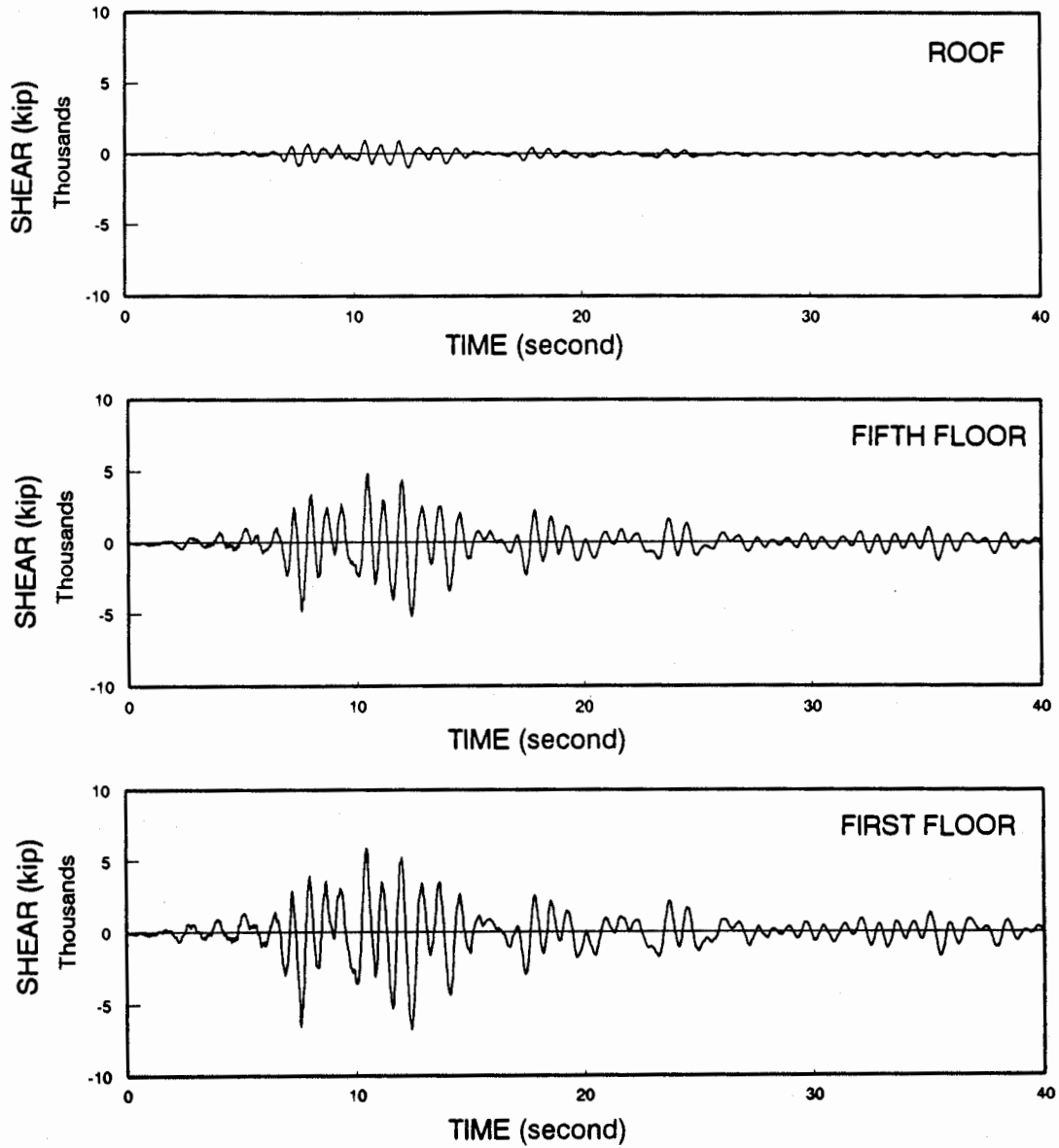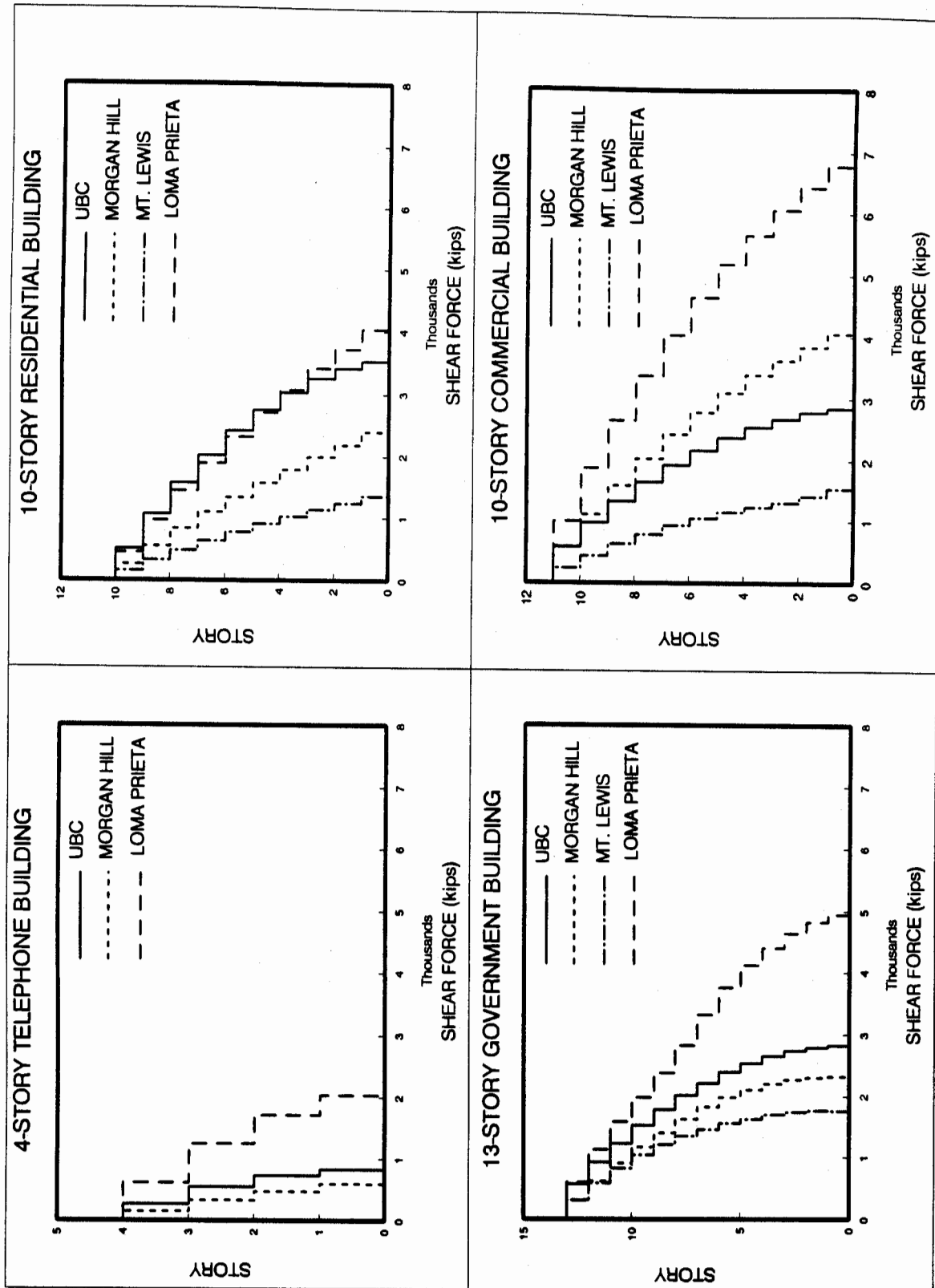
FIGURE 4-1: **VARIATION IN SHEAR FORCE TIME HISTORIES OVER THE HEIGHT OF THE 10-STORY COMMERCIAL BUILDING DURING THE LOMA PRIETA EARTHQUAKE**
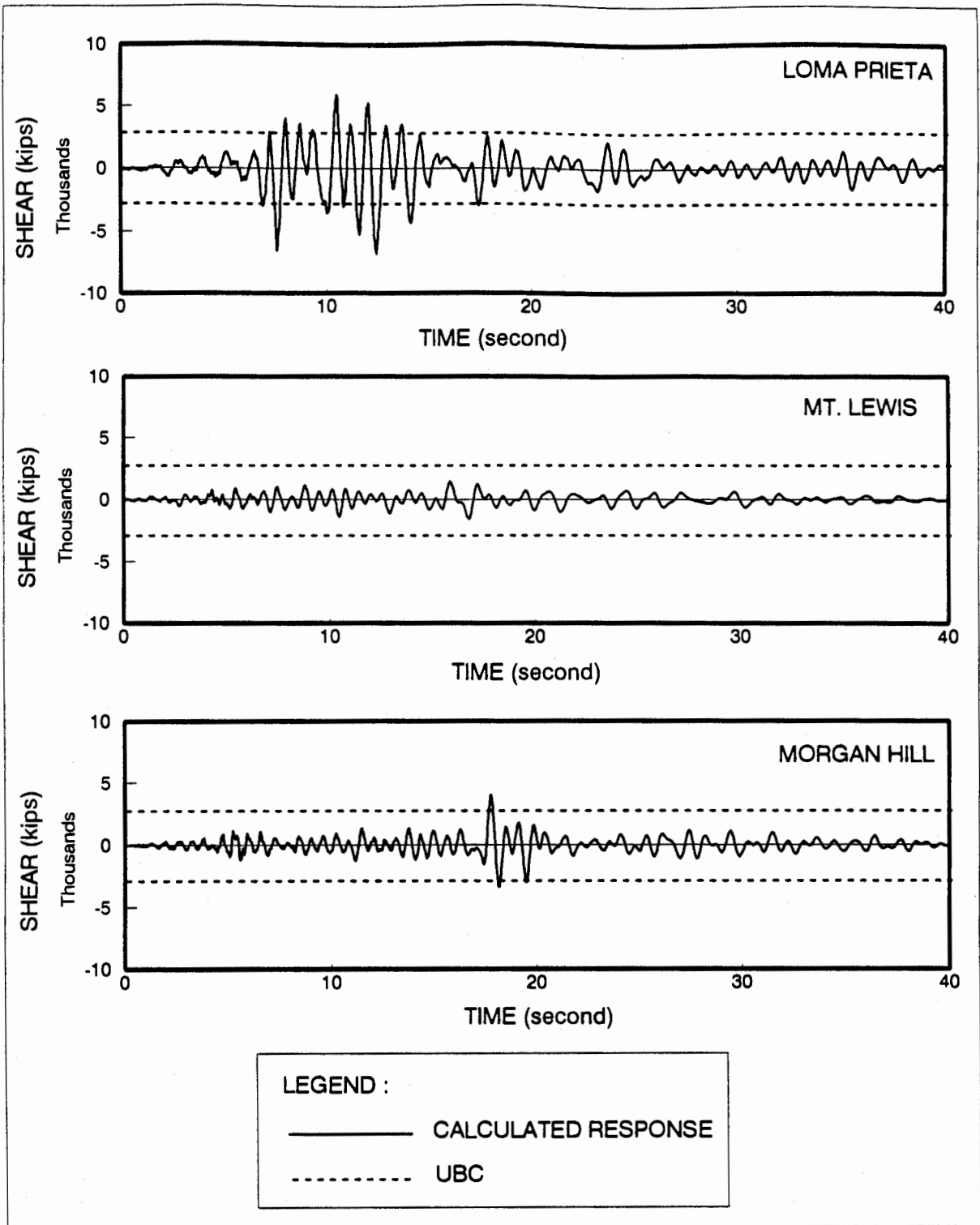
FIGURE 4-2: BASE SHEAR FORCE TIME-HISTORIES FOR THREE EARTHQUAKES, LOMA PRIETA, MT. LEWIS, MORGAN HILL FOR THE 10-STORY COMMERCIAL BUILDING

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FIGURE 4-3: VARIATION IN MAXIMUM SHEAR FORCES OVER THE HEIGHT OF BUILDINGS

UBC base shear, whereas, for the Mt. Lewis and Morgan Hill earthquakes, it was about 60% and 30% smaller than the 1988 UBC base shear. For the 10-Story Commercial Building, the total base shear experienced during the Loma Prieta earthquake was about 60% larger than the 1988 UBC base shear. For the Mt. Lewis earthquake the total base shear was about 60% smaller than the 1988 UBC base shear, while for the Morgan Hill earthquake it was approximately on the same order as the 1988 UBC base shear. For the 4-Story Telephone Building, the total base shear during the Loma Prieta earthquake was observed to be about 140% larger than the 1988 UBC base shear, while during the Morgan Hill earthquake the total base shear was about 30% smaller than the 1988 UBC base shear. For the 13-Story Government Building the total base shear forces during the Mt. Lewis and Morgan Hill earthquakes were 40% and 20%, respectively, smaller than the 1988 UBC base shear. For the Loma Prieta earthquake, the total base shear force was about 75% larger than the 1988 UBC base shear.

TABLE 4-3 - RATIO OF COMPUTED EARTHQUAKE TO 1988 UBC BASE SHEAR FORCES				
Building	10-Story Residential	10-Story Commercial	4-Story Telephone	13-Story Government
Loma Prieta	1.15	2.39	2.44	1.75
Mt. Lewis	0.38	0.54	--	0.62
Morgan Hill	0.68	1.43	0.71	0.82

Torsional Moments:

As an example, the time-histories of torsional moments for one of the buildings, the 10-Story Commercial Building, due to the Loma Prieta, Mt. Lewis, Morgan Hill earthquakes

are shown in Figures 4-4 and 4-5. The computed torsional moment time-histories due to Loma Prieta earthquake at the roof, the fifth floor and the first floor of the 10-Story Commercial Building are shown in Figure 4-4. The torsional moment time-history response of this building to the Loma Prieta, Mt. Lewis and Morgan Hill earthquakes are compared with the torsional moment prescribed by the 1988 UBC regulations in Figure 4-5. Similar time-histories were also obtained for other buildings.
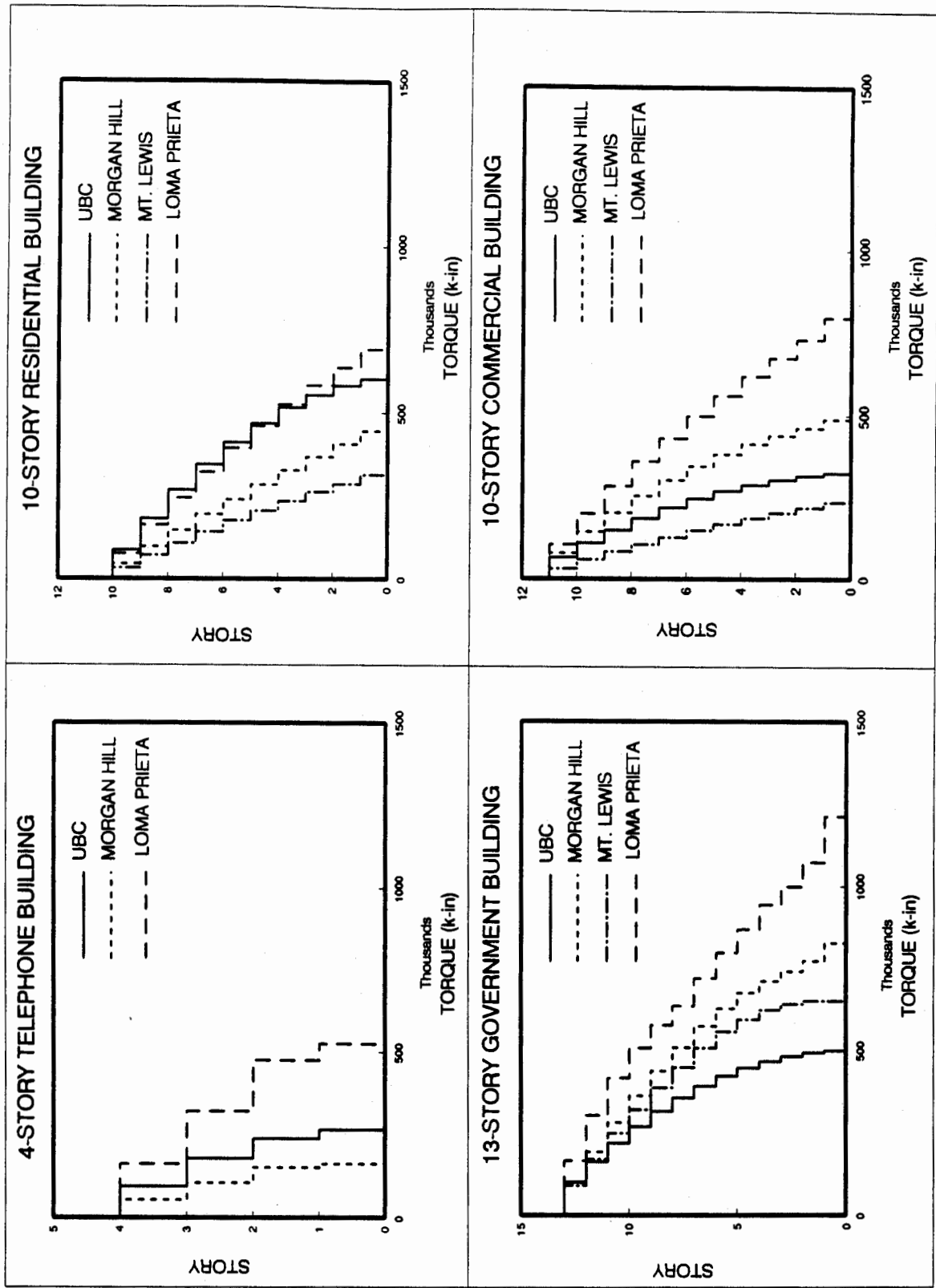
The variation of maximum torsional moments over the height of the three buildings as obtained from the time-histories for the torsional moments is shown in Figure 4-6. The ratio of the base torsional moments experienced by the buildings during the earthquakes to those obtained from 1988 UBC are summarized in the Table 4-4. It can be observed that for the 10-Story Residential and Commercial Buildings and the 4-Story Telephone Building, the torsional moments during Mt. Lewis and Morgan Hill earthquakes are smaller than those obtained by using 1988 UBC and are larger for the Loma Prieta earthquake. For the 13-Story Government Building, however, the maximum story torsional moments during the Mt. Lewis and Morgan Hill earthquakes were larger than the 1988 UBC torsional moments, even though the maximum story shears during these earthquakes were smaller than the 1988 UBC story shears. This interesting observation indicates that there is a large amplification of the eccentricity in this building and may be indicative of strong torsional coupling which may have contributed to the unusually long duration of response recorded in this building during the past three earthquakes.
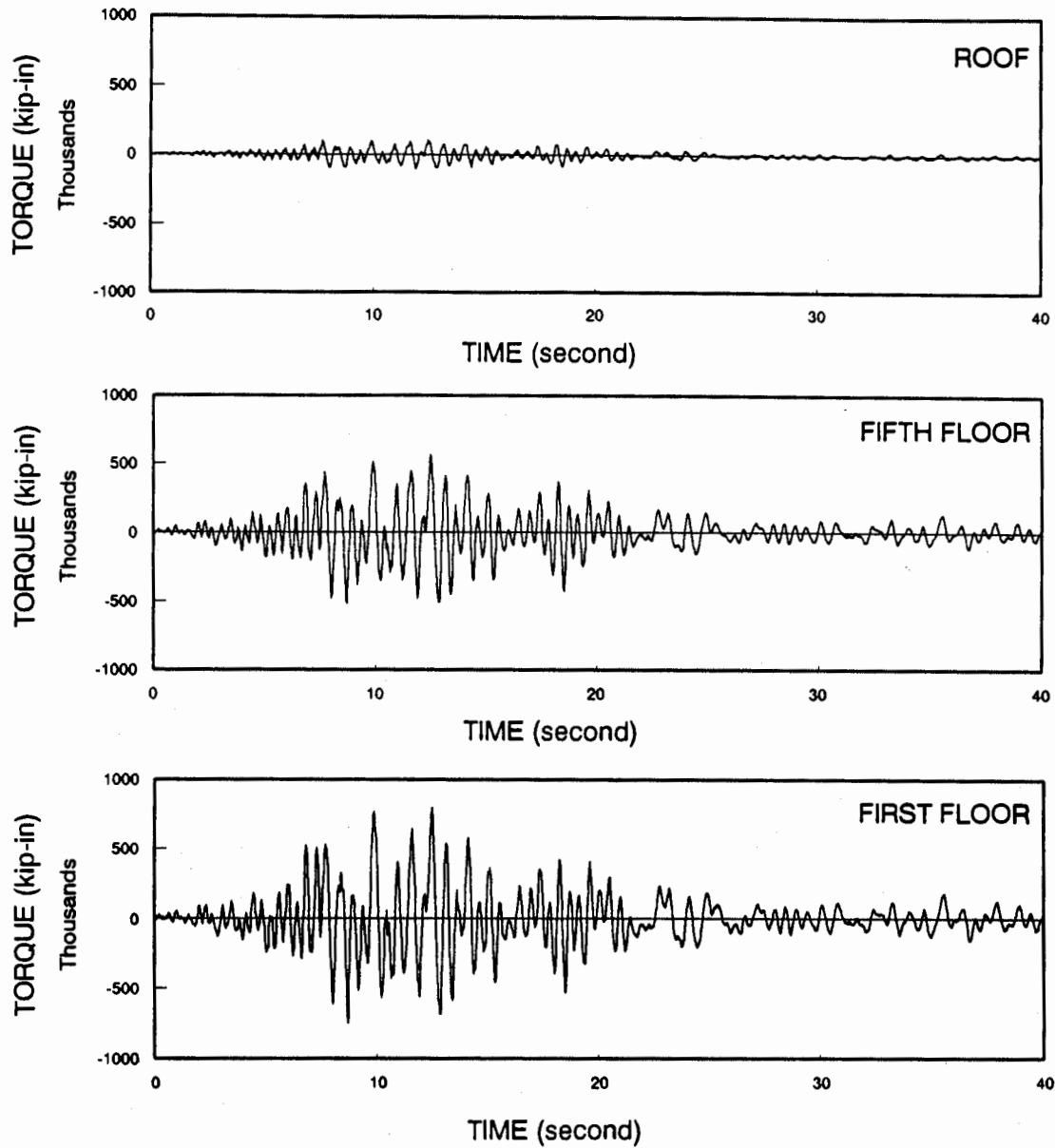
FIGURE 4-4: VARIATION IN TORSIONAL MOMENT TIME-HISTORIES OVER THE HEIGHT OF THE 10-STORY COMMERCIAL BUILDING DURING THE LOMA PRIETA EARTHQUAKE

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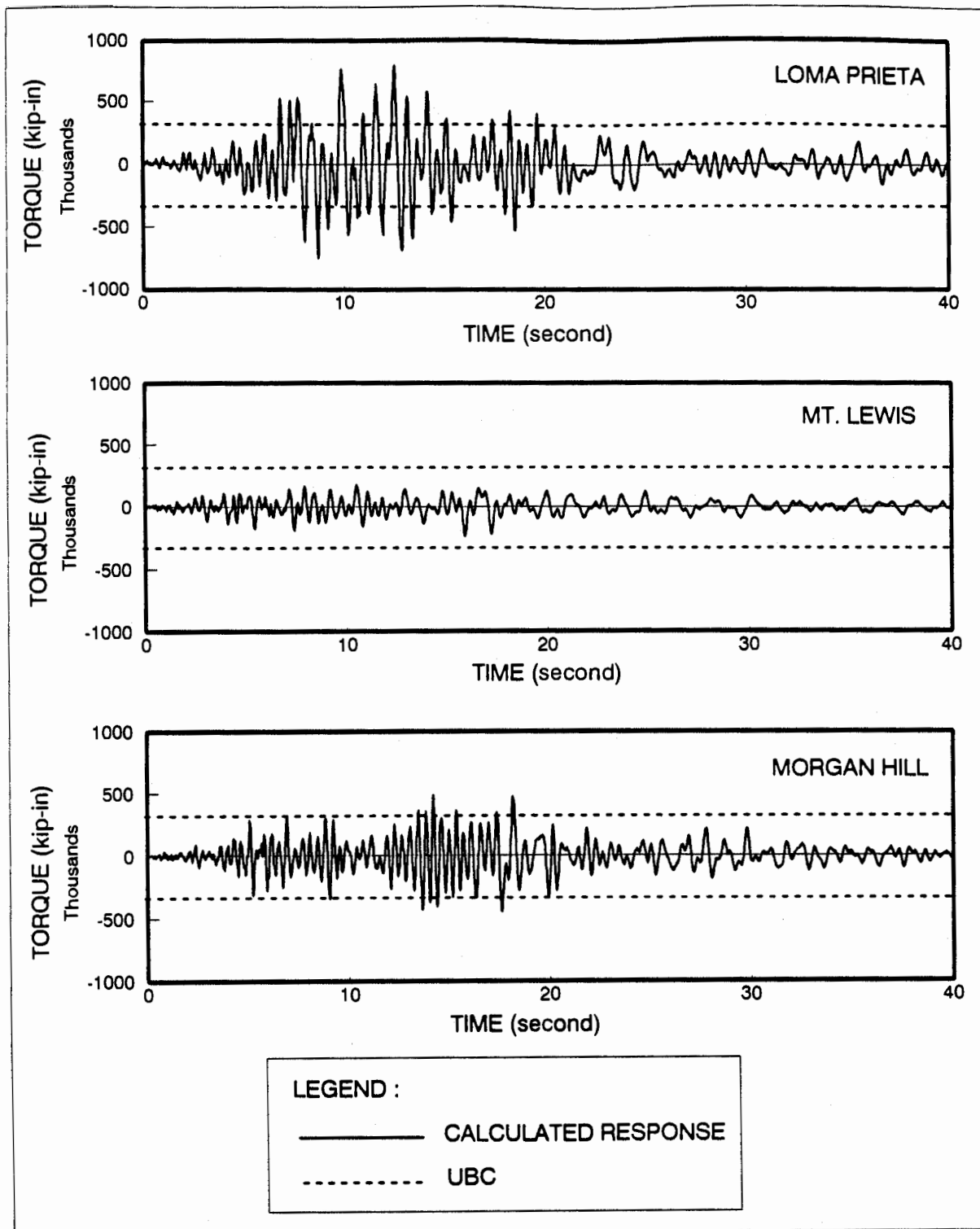
FIGURE 4-5: BASE TORSIONAL MOMENT TIME-HISTORIES FOR THREE EARTHQUAKES, LOMA PRIETA, MT. LEWIS, MORGAN HILL FOR THE 10-STORY COMMERCIAL BUILDING

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FIGURE 4-6: VARIATION IN MAXIMUM TORSIONAL MOMENT OVER THE HEIGHT OF BUILDINGS

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TABLE 4-4 - RATIO OF COMPUTED EARTHQUAKE TO 1988 UBC BASE TORSIONAL MOMENTS				
Building	10-Story Residential	10-Story Commercial	4-Story Telephone	13-Story Government
Loma Prieta	1.15	2.46	1.98	2.43
Mt. Lewis	0.52	0.74	---	1.30
Morgan Hill	0.74	1.5	0.61	1.65

Dynamic Eccentricities:

As explained earlier, dynamic eccentricity is defined as the ratio of maximum torsional moment to the corresponding uncoupled shear force in the building. The torsional moments in the building were obtained from actual recorded motions as described above and involve few assumptions. The uncoupled shear forces were, however, obtained using spectral response in the fundamental mode which requires an estimate of damping ratios along with period of vibration and mode shape. Since it was difficult to obtain the real damping ratios for the buildings, the uncoupled shear forces and the corresponding total dynamic eccentricities were calculated for a range of damping ratios: 2%, 5%, and 10%.

The total dynamic eccentricities, as obtained from the analysis of the recorded motions, include both the dynamic eccentricity and the accidental eccentricity. The total dynamic eccentricities obtained here were compared with the total 1988 UBC design eccentricities, which as defined by Equation 1-1 consist of static eccentricity and the 5% accidental eccentricity. The ratios of the total dynamic eccentricity for different damping

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ratios to the 1988 UBC design eccentricity were calculated over the height of each building for the three earthquakes and are shown in Figure 4-7.

From these figures it can be observed that the maximum amplification of eccentricity occurs in the first story. The ratios of total dynamic eccentricity to the total 1988 UBC design eccentricity in the first story of each building during the three earthquakes are summarized in Table 4-5.

TABLE 4-5 - RATIO OF FIRST STORY TOTAL DYNAMIC ECCENTRICITY TO THE 1988 UBC DESIGN ECCENTRICITY											
Building	10-Story Residential			10-Story Commercial			4-Story Telephone			13-Story Government		
Damping	2%	5%	10%	2%	5%	10%	2%	5%	10%	2%	5%	10%
Loma Prieta	0.98	1.07	1.18	0.92	1.06	1.14	0.53	0.71	0.95	1.82	2.22	2.80
Mt. Lewis	1.21	1.50	1.77	1.57	1.62	1.84	N.A.	N.A.	N.A.	2.14	2.57	3.39
Morgan Hill	1.13	1.25	1.41	0.99	1.07	1.16	0.53	0.66	0.79	1.50	2.28	3.54

The information presented in Figure 4-7 shows that the total dynamic eccentricity for the buildings during the earthquakes is generally larger than the total 1988 UBC design eccentricity, especially at the lower levels. Only for the 4-Story Telephone Building, which has a high static eccentricity, was the 1988 UBC prescribed total eccentricity observed to be larger than the total dynamic eccentricity. This observation is consistent with the results obtained from analytical studies of idealized one-story structures [5,6]. The amplification in the eccentricity was observed to be especially pronounced for the 13-Story Government Building which may indicate that this building experienced severe torsion during the past
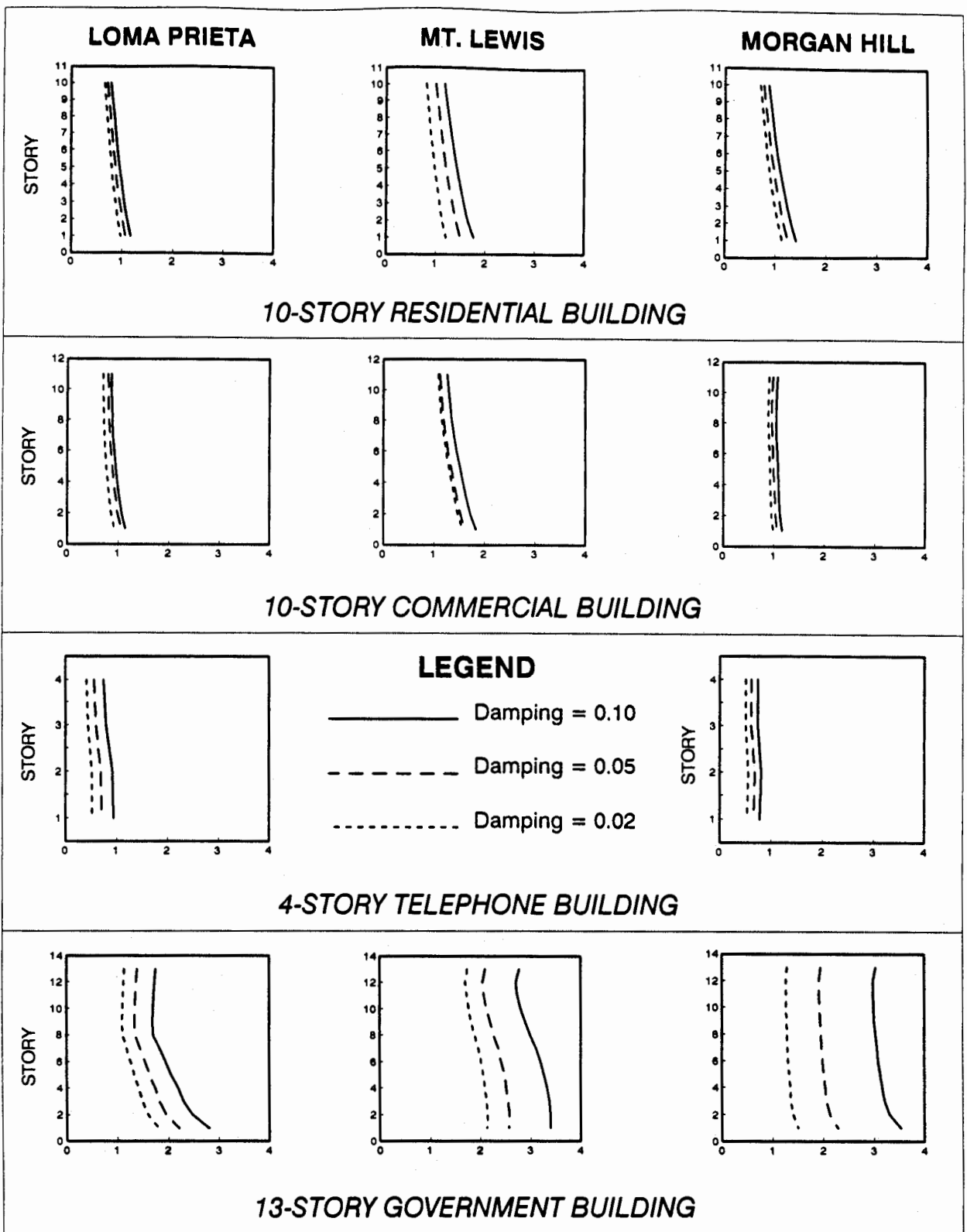
FIGURE 4-7: VARIATION OF TOTAL DYNAMIC ECCENTRICITY TO TOTAL DESIGN ECCENTRICITY OVER THE HEIGHT OF THE BUILDINGS

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earthquakes which could not have been realistically estimated by the 1988 UBC requirements.

It is also observed that the amplification effects for the 3 buildings in San Jose are greatest for the Mt. Lewis earthquake which was closest to these buildings. The sensitivity of the dynamic eccentricity ratio to damping was not observed to be very pronounced, except for the 13-Story Government Building.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The directed research project of the California Strong Motion Instrumentation Program (CSMIP) for the analysis of strong motions recorded in structures has provided a unique opportunity to investigate the actual behavior of buildings during earthquakes, and to assess the adequacy of current analytical methods and Code design provisions. The present investigation was undertaken to study the torsional response of four regular buildings by analyzing the strong motions recorded during the 1989 Loma Prieta, 1986 Mt. Lewis, and 1984 Morgan Hill earthquakes and to compare them with the provisions of 1988 UBC. The primary observations from this investigation are summarized below:

1. The transfer functions of the Fourier Amplitude Spectra of the recorded building motions can provide realistic estimates of fundamental building period. The theoretical background for this has also been shown previously [12, 13]. It is observed that the building periods obtained using Method A of the 1988 UBC may sometimes be unconservative, especially for stiffer buildings, when used to calculate earthquake design forces using static force procedure. The California Strong Motion Instrumentation Program has a significant data base of recorded motions in different types of building structures which may be used to make a more comprehensive assessment of the adequacy of the current provisions in the 1988 UBC for the estimation of fundamental building periods.

2. The real damping in structures subjected to earthquake motions cannot be accurately and reliably predicted using the half power method. This may be due to the representations of complex energy dissipation phenomenon with a simplified viscous damping ratio, nonlinear behavior, noise in the recorded motions, and, for some structures, closely spaced modes. Beck and Beck [10] have also discussed the limitations of using this method for estimating damping in structures.

3. The maximum shear forces estimated in the buildings for the Loma Prieta Earthquake were generally higher than the 1988 UBC prescribed shear forces. The maximum shear forces in the buildings during the Mt. Lewis and Morgan Hill earthquakes were generally smaller than the 1988 UBC shear forces.

4. The torsional moments during the Loma Prieta earthquake were higher than the 1988 UBC prescribed torsional moments for all buildings. For the Mt. Lewis and Morgan Hill earthquakes, the actual torsional moments in the 10-Story Residential, 10-Story Commercial, and 4-Story Telephone Buildings were smaller than the 1988 UBC. For the 13-Story Government Building, the moments during these were larger than those obtained from 1988 UBC even though the earthquake shear forces were smaller than the 1988 UBC shear forces.

5. The total dynamic eccentricities as obtained from the analysis of recorded motions were larger than the total design eccentricities given by the 1988 UBC for all buildings except

for the 4-Story Telephone Building. For 5% damping ratio, this increase in ranged from 5% to 60% for the 10-Story Residential and Commercial Buildings. The increase was especially pronounced for the 13-Story Government Building and was on the order of 150%. This amplification of eccentricities is due to lateral-torsional coupling and is consistent with the observations from previous analytical studies. Many building codes, such as Mexican and Canadian, have recognized this amplification of eccentricity by requiring that the computed static eccentricity be multiplied by a factor of 1.5. An amplification of static eccentricity by using the response spectrum amplification factors has also been suggested by Newmark and Hall [11]. In the light of these observations, the provisions of the 1988 UBC, which do not require an amplification in the static eccentricities, may require further evaluation by performing a more comprehensive study of the recorded building motions.

7.0 REFERENCES

1. Uniform Building Code, 1988 Edition

2. E. Rosenblueth, "Seismic Design Requirements in a Mexican 1976 Code", Earthquake Engineering and Structural Dynamics, Vol. 7, PP 49-61, 1979.

3. National Building Code of Canada, 1985, Subsection 4.1.9.1

4. H. Sedarat, V.V. Bertero, "Effects of Torsion on the Linear and Nonlinear Seismic Response of Structures," Report No. UCB/EERC-90/12, Earthquake Engineering Research Center, University of California, Berkeley, September 1990.

5. R. Hejal, A.K. Chopra, "Earthquake Response of Torsionally-Coupled Buildings," Report No. UCB/EERC-87/20, Earthquake Engineering Research Center, University of California, Berkeley, 1987.

6. W.K. Tso, K.M. Dempsey, "Seismic Torsional Provisions for Dynamic Eccentricity," Earthquake Engineering and Structural Dynamics, Vol. 8, pp. 275-289 (1980).

7. A.F. Shakal, et. al., "CSMIP Strong-Motion Records from the Santa Cruz Mountains (Loma Prieta), California Earthquake of 17 October 1989", Report OSMSF 89-06, California Department of Conservation, Division of Mines & Geology, 1989.

8. R.L. Boroschek, S.A. Mahin, C.A. Zeris, "Seismic Response and Analytical Modeling of Three Instrumented Buildings," Proceedings of Fourth U.S. National Conference on Earthquake Engineering, pp. 219-228, Volume 2, May 20-24, 1990, Palm Springs, California.

9. S.D. Werner, A. Nisar, and J.L. Beck, "Assessment of UBC Seismic Design Provisions Using Recorded Building Motions from Morgan Hill, Mount Lewis, and Loma Prieta Earthquakes", prepared for National Science Foundation, Washington, D.C., April 1992.

10. R.T. Beck, J.L. Beck, "Comparison Between Transfer Function and Modal Minimization Methods for System Identification," Report No. EERL 85-06, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California, November 1985.

11. N.M. Newmark and W.J. Hall, "Earthquake Spectra and Design," Earthquake Engineering Research Institute Monograph.

12. R.W. Clough, J. Penzien, "Dynamics of Structures", 1975, McGraw Hill, Inc.

13. A.K. Chopra, "Dynamics of Structues, A Primer", Earthquake Engineering Research Institute.