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Torsional response characteristics of regular buildings under different seismic excitation levels

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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 motion records from three recent earthquakes: 1989 Loma Prieta, 1986 Mt. Lewis, and 1984 Morgan Hill. The fundamental period of vibrations, damping ratios, story shear forces, torsional moments and dynamic eccentricities for these buildings during the three earthquakes were obtained from an analysis of the recorded motions. These results were then compared with the provisions of the 1988 Uniform Building Code (UBC). This comparison indicates that the provisions of the UBC sometimes may not realistically account for the torsional response of buildings during earthquakes.

INTRODUCTION

The real response of buildings to ground motion excitations can, in general, be affected by the coupling of translational vibrations with rotational vibrations. This coupling may occur because of several reasons, such as the presence of static eccentricity due to unsymmetrical distribution of mass and/or stiffness in the plan, and/or accidental eccentricity due to factors such as non-uniform ground motions across the foundation of the structure, torsional components in the ground motions, and detailing of non-structural components, etc. Recent research (Sedarat and Bertero, 1990; Hejal and Chopra, 1987; Tso, 1990) on the analytical response of one-story idealized structures indicates that the lateral-torsional response of a structure during earthquakes can be much larger than what can be estimated by linear elastic static analysis. This is mainly due to the dynamic amplification of eccentricity during strong ground shaking. This dynamic amplification can be more pronounced for structures with small static eccentricities, i.e., for regular structures. The provisions of 1988 UBC to account for torsional effects in regular buildings can, therefore, sometimes be unconservative.

The objective of this investigation is to analyze strong motions recorded in regular buildings during past earthquakes to study their torsional response characteristics. In this investigation, the distribution of shear forces, torsional moments and dynamic eccentricities over the height of four buildings in California has been estimated for three different levels of excitation during the 1989 Loma Prieta (M7.1), 1986 Mt. Lewis (M5.5), and the 1984 Morgan Hill (M6.2) earthquakes. The fundamental period of vibration and damping ratios are also estimated from the recorded motions. These response quantities are then compared with the provisions of the 1988 UBC.

DESCRIPTION OF BUILDINGS AND STRONG MOTIONS STUDIED

Four regular buildings in the San Jose and Watsonville area 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, California. The basic structural features, and the motions recorded in these buildings during the Loma Prieta, Mt. Lewis, and Morgan Hill earthquakes are summarized in Table 1.

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 significant 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 to allow estimation of the torsional accelerations and their distribution over the height of the structure.

METHODOLOGY

The following methodology was used for each building:

Step 1: From the information shown on the structural drawings for the buildings provided by the California Division of Mines & Geology (CDMG), 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. Relative floor stiffness and the location of the center of stiffness (CS) for each floor were obtained 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. 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. These were used to calculate the coupled shear force time-histories at the CM and the torsional moment time-histories about the CS for each floor.

Step 3: In this step, the fundamental period of vibrations, mode shapes and the damping ratios for the buildings were estimated. For this purpose, a transfer function was computed as the ratio of the Fourier amplitude spectrum (FAS) of the roof motions to the FAS of the corresponding ground motions. The FAS transfer functions exhibited well defined peaks. The fundamental frequency of vibration in the transverse direction of the building was taken as the frequency at the location of the first peak and the damping ratio was estimated by applying the half-power method to that peak. The fundamental mode shape was taken as the deflected shape of the building at the instant of peak roof acceleration.

Step 4: The estimated fundamental periods, damping ratios, and mode shapes were used to compute the spectral accelerations and uncoupled lateral forces by assuming a fundamental mode response. In this, the spectral accelerations and story shear forces at each floor are a function of the modal damping ratios estimated under Step 3 using the half power method which, as discussed later in this paper, are probably not fully reliable. For this reason, the spectral accelerations and lateral forces were estimated for a range of different damping ratios (2%, 5%, and 10%). The total dynamic eccentricity at each floor was then obtained by dividing the torsional moment about the CS calculated in Step 3 by the 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 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 UBC were also obtained.

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 2 for the three earthquakes. This table also shows the building periods as obtained from Method A in the 1988 UBC. These results indicate that, except for the 13-story government building, the building periods predicted by 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 UBC equation for period calculation may sometimes be unconservative when used to calculate earthquake design forces using the static force procedure. For the 13-story building, the periods estimated from recorded motions were actually 100% higher than those given by 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 periods obtained here also compared well with those from previous studies (Boroschek, et al, 1990), including those utilizing more sophisticated system identification methods (Werner, et al, 1992).

Damping Ratios: Table 2 also 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. 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 in the use of half-power method have been pointed out by Beck and Beck (1985).

Shear Forces: From the time-histories of the story shear forces, the maximum shear forces over the height of all four buildings were obtained. These are shown in Figure 1 and also summarized in Table 3. It can be observed that for the Loma Prieta earthquake the maximum base shear forces experienced by the four buildings are larger than those obtained from UBC, whereas for the Mr. Lewis and Morgan Hill earthquakes, they are smaller than the UBC base shears.

Torsional Moments: 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 2 and also summarized in Table 3. These results show 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 UBC torsional moments, even though the maximum story shears during these earthquakes were smaller than the UBC story shears. This interesting observation indicates that there is a large amplification in the eccentricity in this building and may be indicative of strong torsional coupling which probably led to the unusually long duration of response recorded in this building during the past three earthquakes.

Dynamic Eccentricities: Dynamic eccentricities are a function of the damping ratio. Since it was difficult to obtain the real damping ratios for the buildings, the 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 UBC design eccentricities, which in turn consist of static eccentricity and the 5% accidental eccentricity. The ratios of the total dynamic eccentricity for different damping ratios to the UBC design eccentricity were also calculated over the height of each building during the three ground motions and are shown in Figure 3. The information presented in Figure 3 shows that the total dynamic eccentricity for the buildings during the earthquakes is generally larger than the total design eccentricity from 1988 UBC.

CONCLUSIONS AND RECOMMENDATIONS

The present investigation was undertaken to investigate the actual behavior of buildings during earthquakes, and to assess the adequacy of current analytical methods, and Code design provisions 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. It is observed that the building periods obtained using Method A of the UBC may sometimes be unconservative, especially for stiffer buildings, when used to calculate earthquake design forces using static force procedure. CDMG's Strong Motion Instrumentation Program has resulted in 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 UBC for the estimation of fundamental building periods.

2. The real damping in structures 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, noise in the recorded motions, and, for some structures, closely spaced modes.

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

4. The torsional moments during the Loma Prieta earthquake were higher than the 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 UBC. For the 13-story government building, the moments during these two earthquakes were larger than those obtained from UBC even though the earthquake shear forces were smaller than the 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 UBC for all buildings except for the 4-story telephone building. For 5% damping ratio, this increase in eccentricity 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% and would not have been realistically estimated by UBC requirements. This amplification of eccentricities is most likely 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 (1982). In the light of these observations, the provisions of the current 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.

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Table 1 - Summary of recorded motions in selected buildings.

Building Location (and type)	CSMIP Station Number	Stories	Lateral Force System	Peak Acceleration, g: Loma Prieta Earthquake (1989) Base	Loma Prieta Roof	Mt. Lewis Earthquake (1986) Base	Mt. Lewis Roof	Morgan Hill Earthquake (1984) Base	Morgan Hill 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
MRCF = Moment Resisting Concrete Frame
MRSF = Moment Resisting Steel Frame
N.A. = Not Available

Table 2 - Fundamental period and damping ratios of buildings.

	10-Story Residential Building (E.W.)		10-Story Commercial Building (E.W.)		4-Story Telephone Building (N.S.)		13-Story Government Building (E.W.)	
EARTHQUAKE	Period (Sec.)	Damping (%)	Period (Sec.)	Damping (%)	Period (Sec.)	Damping (%)	Period (Sec.)	Damping (%)
Loma Prieta	0.45	3.7 to 9.8	0.70	4.1 to 4.6	0.22	7 to 11	2.20	3.2
Mt. Lewis	0.42	5.5 to 8.0	0.60	2.6 to 3.3	N.A.	N.A.	2.20	3.4 to 4.7
Morgan Hill	0.42	2.1 to 2.8	0.60	2.4 to 2.6	0.21	5.1 to 13.9	2.20	4.3
UBC 88	0.61		0.74		0.46		1.01	

Table 3 - Rate of computed response to UBC base shear and torsion.

Building	10-Story Residential		10-Story Commercial		4-Story Telephone		13-Story Government	
Earthquake	Shear	Torsion	Shear	Torsion	Shear	Torsion	Shear	Torsion
Loma Prieta	1.15	1.15	2.39	2.46	2.44	1.98	1.75	2.43
Mt. Lewis	0.38	0.52	0.54	0.74	--	--	0.62	1.30
Morgan Hill	0.68	0.74	1.43	1.50	0.71	0.61	0.82	1.65

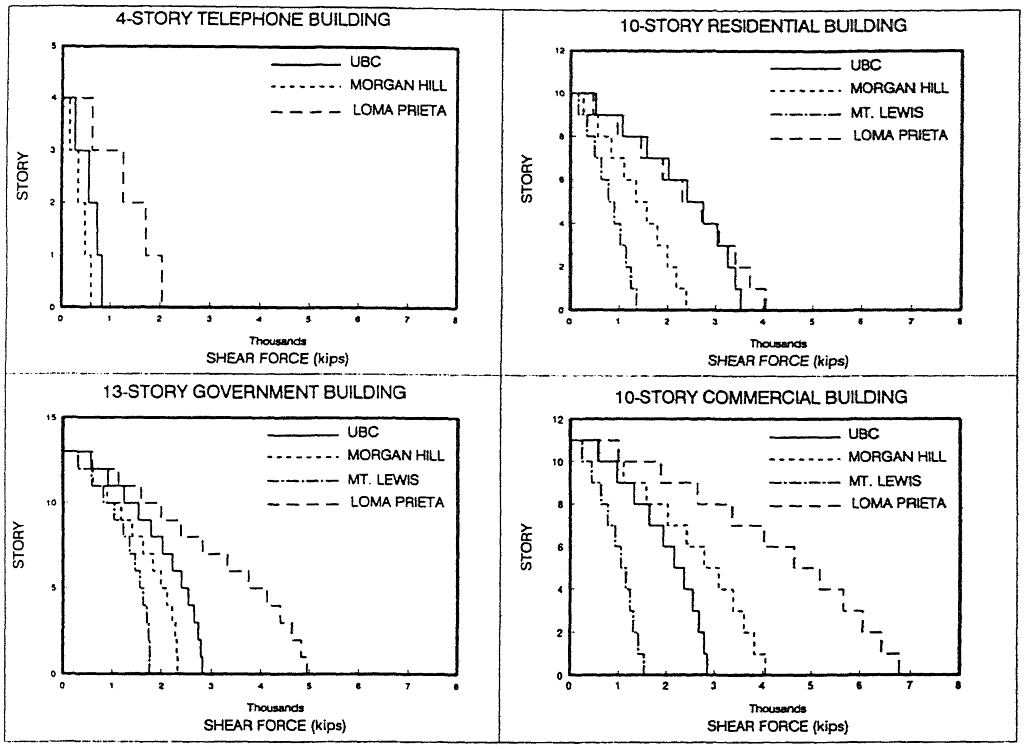

Figure 1. Variation of the maximum shear forces over the height of the buildings.

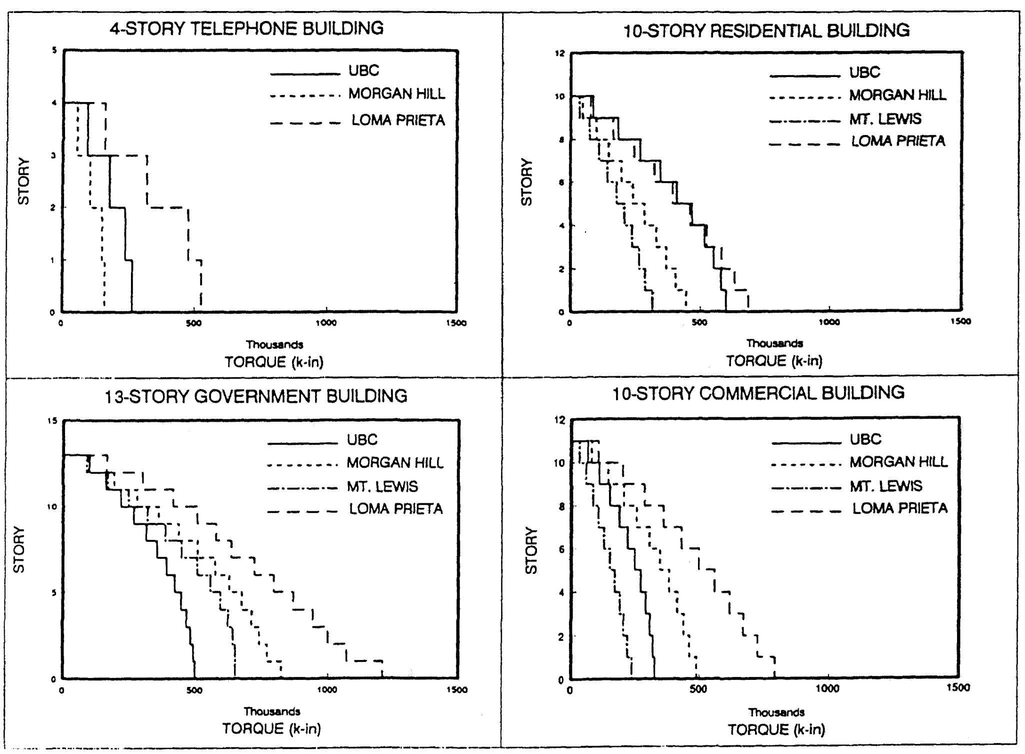

Figure 2. Variation of the maximum torsional moments over the height of the buildings.

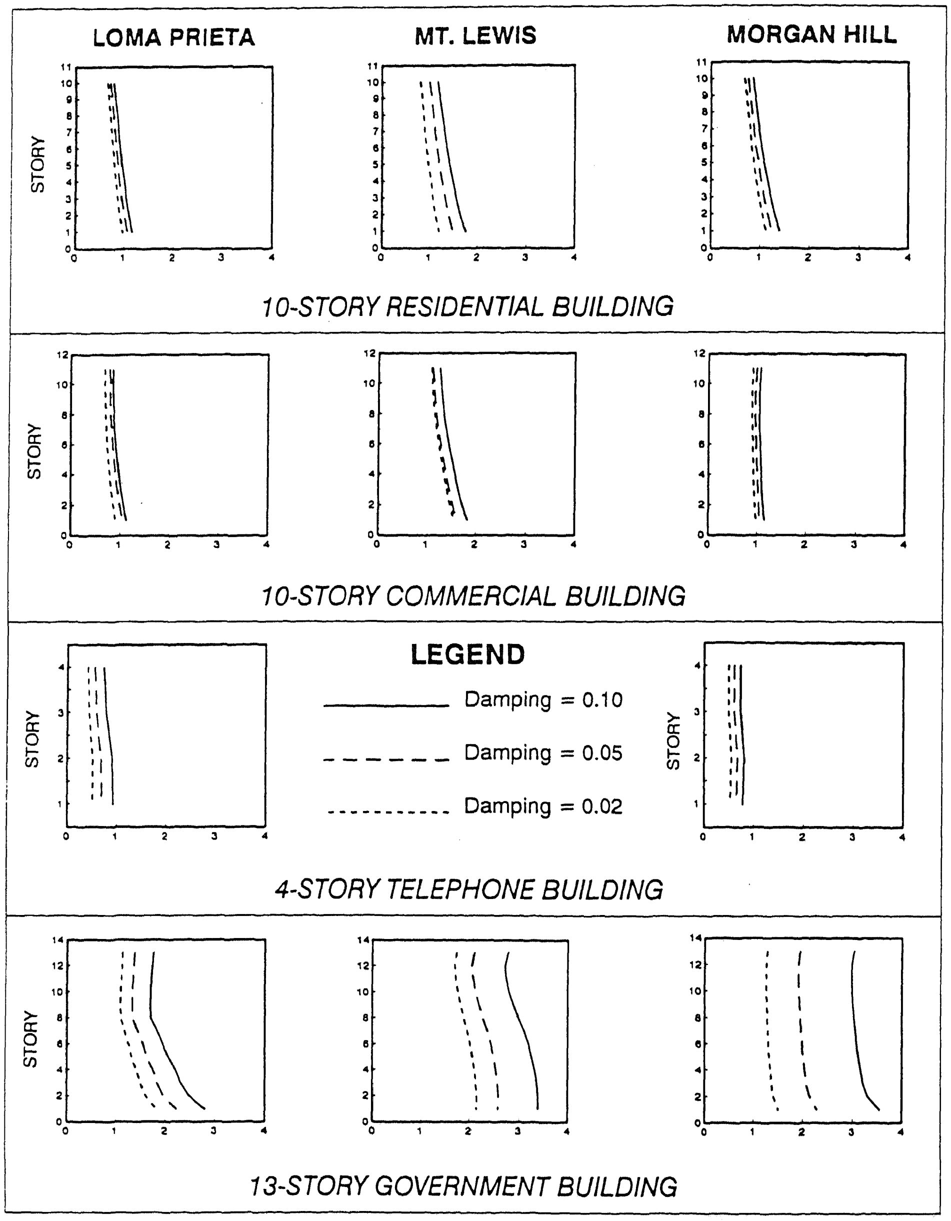

Figure 3. Variation of total dynamic to total design eccentricity over the height of the buildings.