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Highlights of Loma Prieta responses of four tall buildings

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ABSTRACT: Highlights of four tall instrumented buildings in San Francisco (and nearby Emeryville) that recorded the Loma Prieta (LPE) earthquake at approximately 100 km distance are presented. The buildings on soft sites had considerably amplified input motions that are characterized also by the dominant direction of their maximum energy being similar to the LPE rupture propogation direction. A significant effect of the dominant direction of the earthquake is that an unsymmetrical (three-winged) building exhibits disproportionate (as much as 3 times) response in one part of the building with respect to another. System identification techniques and spectral analyses are used in determining significant dynamic characteristics of the buildings showing some unique features that are highlighted.

1 INTRODUCTION

The purpose of this paper is to present significant highlights of response characteristics of four instrumented tall buildings (Transamerica Building [TRA], Pacific Park Plaza Building [PPP], Embarcadero Building [EMB] and 575 Market Street Building [CHE]) in the San Francisco Bay area, California, that recorded the (M_s = 7.1) October 17, 1989 Loma Prieta earthquake (LPE). Table 1 summarizes the descriptions of the buildings as well as peak accelerations and displacements at their top and ground levels. The vibrational characteristics of these buildings are determined using combinations of (a) spectral analyses and (b) system identification procedures. Finite-element analyses of the buildings are not within the scope of this paper.

Three-dimensional outlines and the instrumentation schemes of the four buildings are shown in Figure 1. The relative location of the buildings, all approximately 100 km from the epicenter of the earthquake, and their in-plan orientations and shapes are provided in Figure 2.

2 GENERAL ISSUES

A *significant aspect common to* all four buildings is the similarity of the dominant direction of their input motions (recorded at the basemats or basements) and their output motions (recorded at their top instrumented levels) to that of the strike (surface projection of the San Andreas fault rupture front) of the LPE determined as 128°(or 308°) clockwise from north by Kanamori and Satake (1990). The following relationship is used to determine the dominant direction of a pair of orthogonal records (Bendat and Piersol 1981):

$$\phi = 0.5 \times \tan^{-1}\left[\frac{2\times\sigma_{12}}{\sigma_1^2 - \sigma_2^2}\right]$$

where σ_1^2 and σ_2^2 are the variances of the recorded orthogonal motions (u_1 and u_2) at a location and σ_{12} is their cross-variance. The expression yields the angle ϕ by which u_1 and u_2 must be rotated in order to obtain the dominant orthogonal motions u_d and u_p. When the relationship is applied to the orthogonal pairs of motions recorded at input levels of the four buildings, the resulting dominant directions are within range of the 308°of the epicentral fault strike direction (Figure 2). The figure shows the major and minor axes and orientations (clockwise from north) of each building and the dominant direction (heavy solid) of input and output motions (heavy dashdot). For PPP, both are same.

Insignificant torsional motion is detected from the responses of the three symmetrical buildings. On the other hand, the dominant input direction (u_d) of PPP is approximately 290°which is the perpendicular direction to the long axis of the north wing (NW). The response records show the largest relative displacement (with respect to the ground floor) of 19.1 cm for this wing when compared with 6.7 cm for the west wing (WW). Therefore, for unsymmetrical structures or structures with wings, presence of a dominant direction of input motion results in a disproportionate response of one part of the structure with respect to another. This aspect of direction-

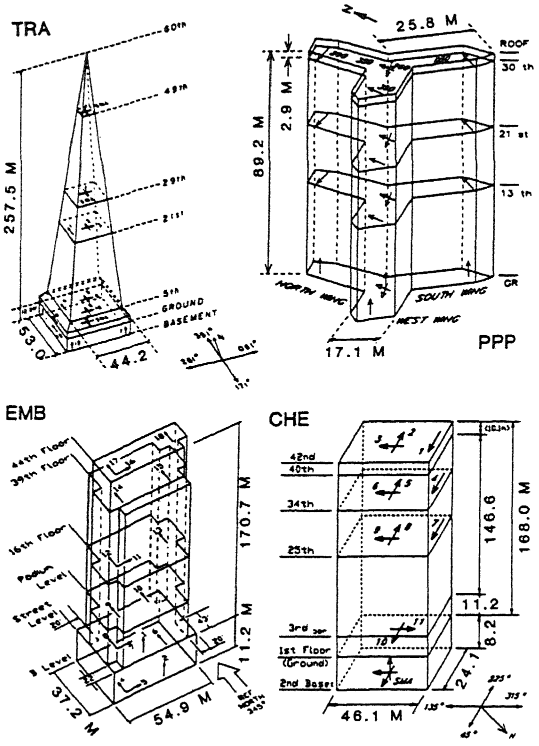

Figure 1. General views and instrumentation schemes of four tall buildings.

ality of the input motion is not addressed in building codes, zoning maps or in development of design response spectra in regions such as San Francisco Bay area, where there are at least two distinctive fault systems capable of generating large-magnitude earthquakes.

Also, all four buildings are founded on non-rock sites. The peak accelerations at the ground levels of the four buildings (Table 1) are higher (2–4 times) when compared with the 0.06 g peak acceleration at Yerba Buena Island (YBI), a rock site in San Francisco Bay (also 100 km from LPE epicenter). Response spectra (5% damping) shown in Figure 3 exhibits the degree of amplification at these sites.

The dynamic characteristics (frequency and damping) of the first three significant modes for the four buildings determined from routine application of system identification technique (Ljung 1987; Mathworks 1988) are summarized in Table 2.

3 TRANSAMERICA BUILDING [TRA]

The fundamental modal frequencies (periods) of TRA at 0.28 Hz (3.6 seconds) for both the NS and EW directions determined from LPE records do not compare well with the 0.35 Hz (2.86 sec) frequen-

Table 1. Descriptions and Peak Responses

Bldg.	H (m) h (m) N_A N_B		Peak A (g)	Peak D (cm)
TRA				
1) S/MR/F	257	T	.31	18.6
2) Pyramid	12.8			
3) 2.75 m RC/mat	60	G	.12	3.3
4) no piles	3			
PPP				
1) RC/MR/F	84	T	.38	21.9
2) Three Wings	0			
3) 1.5 m RC/mat	30	G	.21	9.2
4) friction piles	0			
EMB				
1) S/MR/F	172	T	.47	27.2
2) Rectangular	12			
3) 1.67 m RC/mat	47	G	.16	7.7
4) bearing piles (50 m)	2			
CHE				
1) S/MR/F	168	T	.22	25.9
2) Rectangular	8.2			
3) Pile Clusters	42	G	.11	6.6
4) precast (10 m)	2			

1) type (S–steel, RC–reinforced concrete, F–framed, MR–moment resisting); 2) shape; 3) foundation; 4) pile type (length); H (above)–h (below) ground height, A = acceleration (horizontal), D (displacement), T–top instrumented floor, G–ground/basement floor, N_A and N_B–floors above and below ground level.

Table 2. Dynamic Characteristics

Mode	TRA		PPP		EMB		CHE	
Axis	171	261	350	260	345	075	225	135
	Frequencies (f in Hz)							
1	.28	.28	.38	.38	.19	.16	.16	.21
2	.55	.52	0.95	0.95	.57	.46	.55	.61
3	.95	.86	1.95	1.95	.98	.80	.98	1.0
	Damping in (ξ %)							
1	2.1	1.6	11.6	15.5	2.5	3.7	4.1	5.1
2	3.3	2.0	7.7	3.4	2.2	2.8	4.5	3.4
3	2.1	3.2	6.5	4.4	1.4	3.6	3.6	9.2

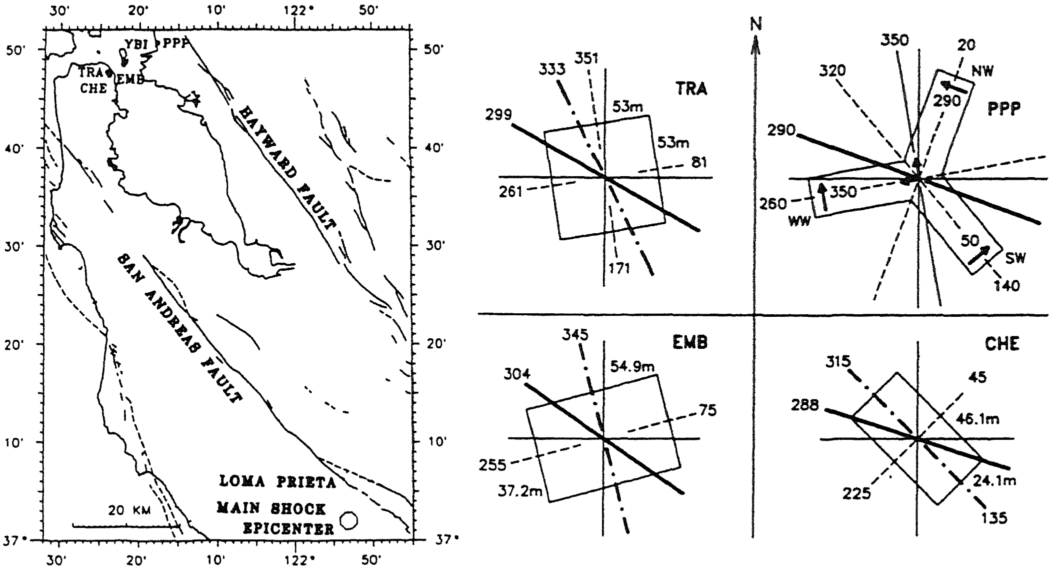

Figure 2. Location map of the four buildings, their plan views, orientations of major axes, and dominant direction of maximum energy (heavy dashed–input, heavy line–building).

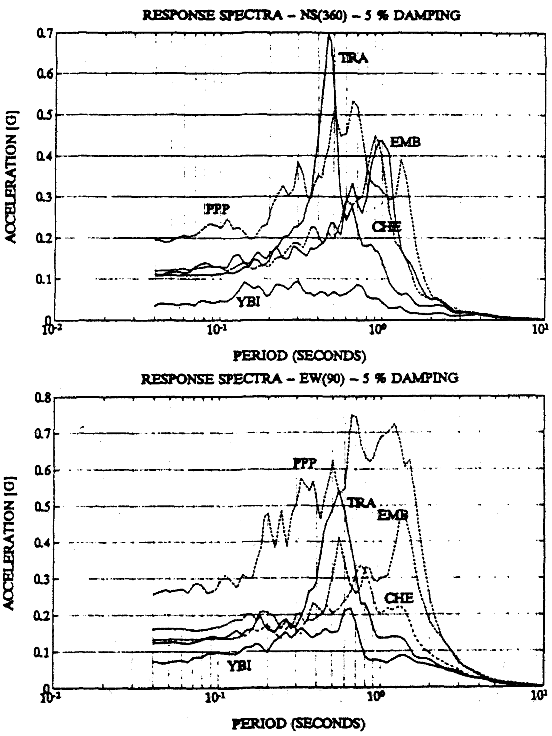

Figure 3. Response Spectra (5 % damping) of input motions of four buildings and the rock site (YBI)– all same distance from the epicenter.

cies (periods) determined from small amplitude tests (Çelebi, Phan and Marshall 1991). During the design process of TRA, the analytical models developed did not assume soil-structure interaction. Therefore, the need for consideration of soil-structure interaction during the design/analysis process is acute for realistic estimates of responses during strong-motion events (such as LPE).

The building experiences rocking at approximately 2 Hz (0.5 seconds) in both directions. Figure 4 shows (a) recorded 49th floor EW (261) acceleration, and (b) its Fourier amplitude spectrum, (c) rocking contribution of 49th floor EW acceleration, and (d) its Fourier amplitude spectrum, (e) 49th floor EW displacement, and (f) rocking contribution of 49th floor EW displacement. The contribution of rocking to motions at any level is calculated by taking two vertical components of motion at the basemat and multiplying it by the ratio of the height at which contribution is sought to the distance between the two vertical sensors at the basemat. Due to the relative high frequency (2 Hz) for rocking as compared to the fundamental translational frequency as low as 0.28 Hz, the acceleration contribution is comparatively large; however, the displacement contribution is small.

4 PACIFIC PARK PLAZA [PPP]

The fundamental mode at a frequency (period) of 0.38 Hz (2.63 seconds) and at other significant modes (at 0.95 Hz, 1.10 Hz and 1.95 Hz) of PPP sum-

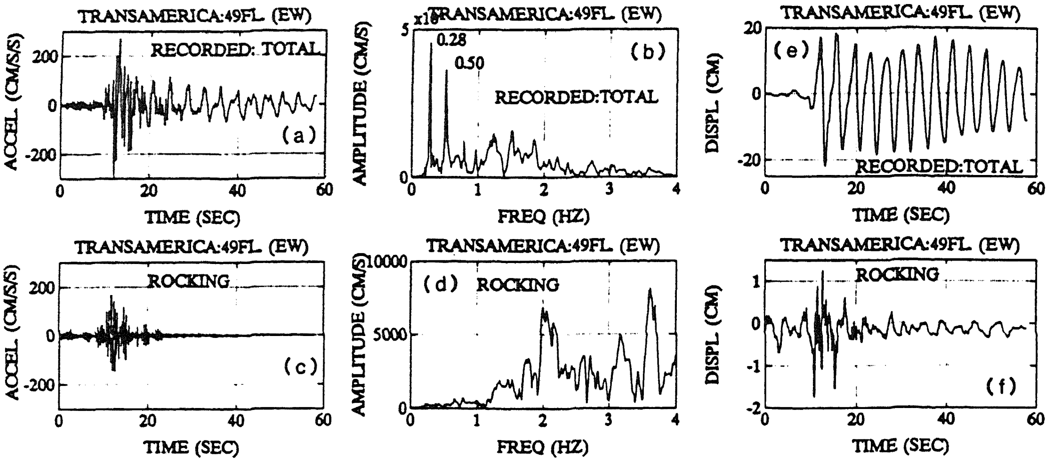

Figure 4. Contribution of Rocking Response at 49th Floor of Transamerica Building.

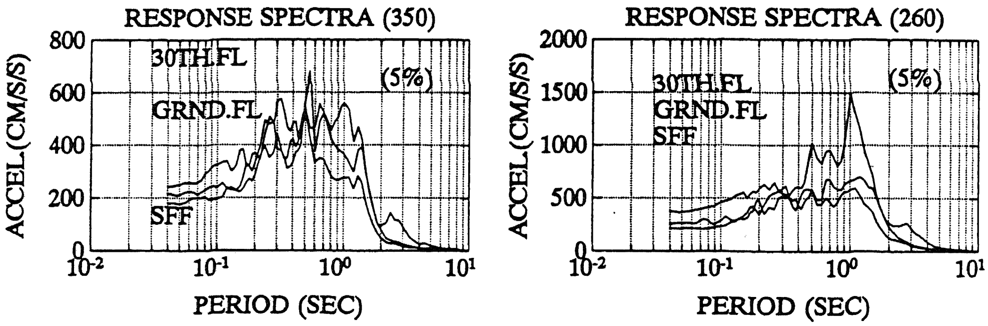

Figure 5. Response Spectra (5 % damping) of the 30th floor, ground floor and free-field (south) of Pacific Park Plaza Building.

marized in Table 1 are all coupled translational–torsional. An unusually high first modal damping (15.5%) is determined by system identification. This may be attributed to soil-structure interaction, which also alters the low-amplitude test fundamental frequency (period) at 0.48 Hz (2.08 seconds) (Çelebi, Phan, Marshall 1991).

The building instrumentation is complemented by two free-field sites (approximately at 25 and 125 m). The data from free-field sites contains frequencies that are structural (particularly 0.95 and 1.95 Hz). This is clearly depicted in the (5%) response spectra of the top floor, ground floor and south-free-field at 125 m (Figure 5). Therefore care is needed when using the free-field data that are in the vicinity of tall structures such as PPP.

5 EMBARCADERO BUILDING [EMB]

The 47-story building is moment-resisting steel-framed in both directions with four central NS bays (narrow direction) being eccentrically braced. The building sits on a 1.67-m-thick reinforced concrete mat supported by 50–67-m-long composite concrete and steel-bearing piles.

A sample system identification plot for this building (using only 80 out of the 120 second record) is shown in Figure 6 which exhibits the fundamental frequencies (periods) of the building at 0.19 Hz (5.26 sec) in the NS and 0.16 Hz (6.25 sec) in the EW direction. Corresponding modal damping is 2.55 and 3.7%, respectively. Second- and third-mode frequencies (periods) are 0.57 Hz (1.75 sec) and 0.98 Hz (1.02 sec) for NS, and 0.46 Hz (2.17 sec) and 0.8 Hz (1.25 sec) for EW, respectively. The periods summarized confirm the T, T/3, T/5 rule-of-thumb to be consistent with this building. The small damping percentages (Table 2) for the three significant modes explain why the response is longer than 120 seconds.

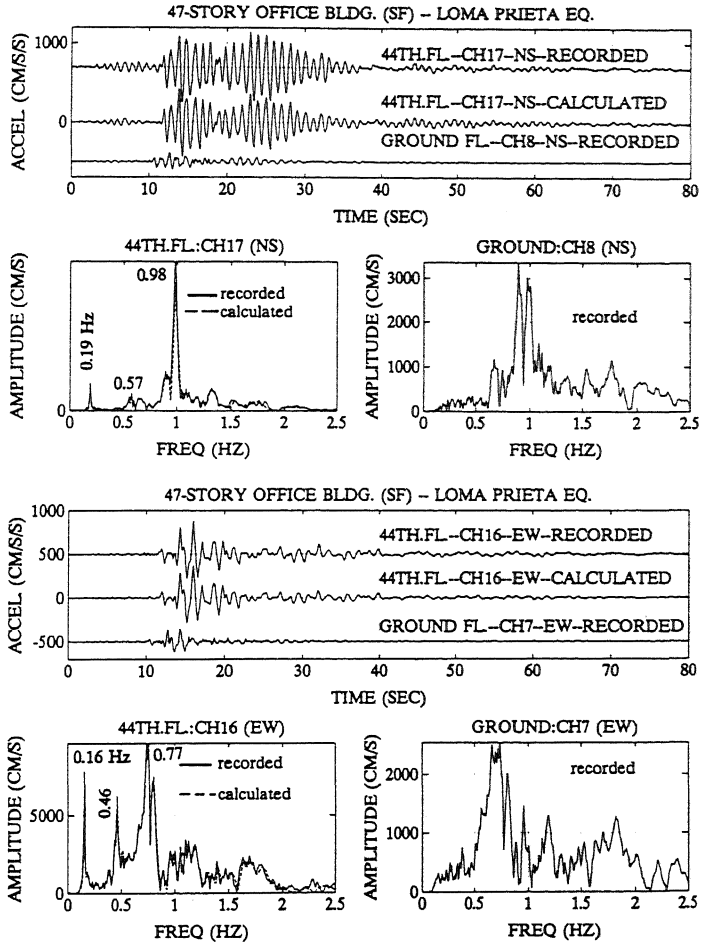

Figure 6. System Identification Application to NS and EW 44th floor (output) and basement (input) motions of Embarcadero Building.

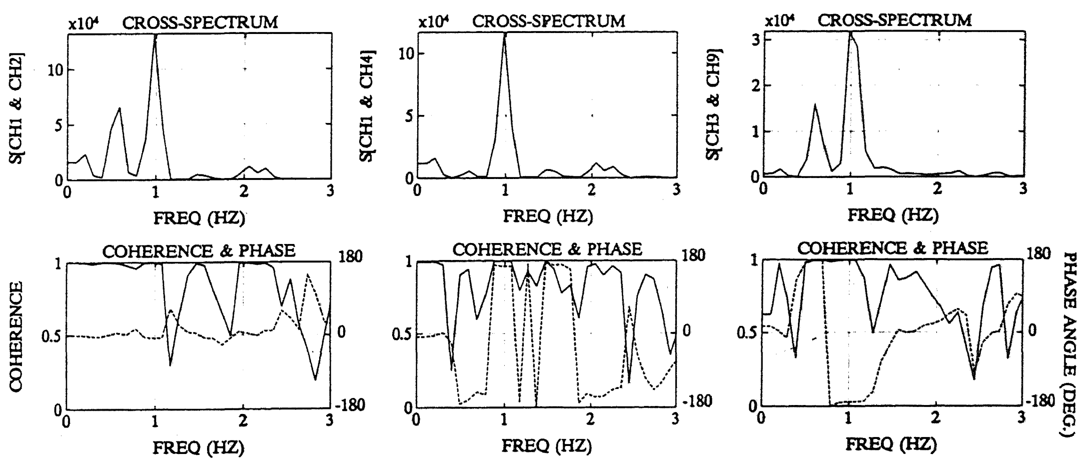

Figure 7. Coherence and Phase Angle Plots of 575 Market Street Building.

Further spectral analyses show that the mode at 0.98 Hz contains insignificant (NS) rocking components and insignificant torsional response occurs at 0.88 Hz.

6 MARKET ST. BUILDING [CHE]

Figure 7 shows coherency and phase-angle plots for (a) two parallel, 225°oriented motions (horizontal plane) at the roof, (b) two parallel 225°motions (in the vertical plane), and (c) two parallel 135°motions (also in the vertical plane) of this slender moment-resisting steel framed building. From these plots, we conclude that (a) there is no torsion since the motions in the horizontal plane at the roof are in phase and coherent, and (b) 0.55 Hz and 1.0 Hz in the 225°direction are the second and third modal frequencies since the phase angles are 180 and −180° out of phase, and (c) similarly for the 135° direction, 0.61 Hz and 1 Hz are the second and third modal frequencies.

7 CONCLUSIONS

The messages of this paper are: (a) Recorded strong-motion responses of tall buildings reveal significant information useful for comparison of real-life responses with those obtained through analytical models and/or low-level amplitude tests. Dynamic characteristics identified from LPE records are considerably different than those determined from low-level amplitude tests or computations. This may be attributable to nonlinear effects such as soil-structure interaction not accounted for in some analyses or extractable from low-level amplitude tests. (b) All four buildings on non-rock sites had significantly amplified (long distance effect) input motions. (c) Free-field motions are influenced by the presence of tall buildings. (d) Directionality of the earthquake motions does not cause significant torsion in symmetric buildings but can disproportionately affect the torsional response of non-symmetric buildings and buildings with wings. Current design processes do not consider dominant direction of earthquake motions in the development of site-specific design response spectra. It is shown herein that the dominant direction may be significant and should be at least considered in certain cases. Furthermore, this is important for regions such as the San Francisco Bay Area where two major faults almost parallel to one another are capable of generating large earthquakes.

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