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COMPARISON OF DYNAMIC CHARACTERISTICS OF TWO INSTRUMENTED TALL BUILDINGS

Carlos E. Ventura[1], Barry Laverick[2], Rune Brincker[3], and Palle Andersen[4]

Professor[1] and Research Assistant[2]
Department of Civil Engineering
The University of British Columbia
2324 Main Mall
Vancouver, B.C., Canada V6T 1Z4

Associate Professor[3]
Department of Building Technology
and Structural Engineering, Aalborg University
Sohngaardsholmsvej 57, 9000
Denmark

Managing Director[4]
Structural Vibration Solutions ApS
Niels Jernes Vej 10
9220 Aalborg East
Denmark

ABSTRACT

Comparison of recorded structural earthquake response and predicted response by dynamic analysis provides vital information to structural designers on the effectiveness of current methods of dynamic analysis. There have been a number of previous studies of this nature, but only a few have paid attention to the three-dimensional nonlinear dynamic behaviour of tall buildings, so there is a need for these types of studies. The purpose of this paper is to study the dynamic properties of two well-instrumented tall steel frame buildings in Los Angeles, California. These building are within a few blocks of each other and have been subjected to ground motions from several earthquakes, among which the most significant are those from the 1994 Northridge earthquake. The results of this study showed that although the buildings were subjected to similar level of ground shaking their different structural systems resulted in remarkably different building response. The differences and similarities of these responses are presented and discussed in this paper. Analyses of the recorded motions from these two buildings were conducted to determine the dynamic characteristics of each structure. Through the different analyses of the recorded a meaningful comparison of building behaviour could be made. The results of this study showed that very meaningful information can be extracted from recorded earthquake data, and that structural engineers can use this information to better understand the dynamic behaviour of very tall buildings. This information can also be used to gain confidence on finite element models used to predict the nonlinear response of buildings due to strong ground shaking.

1 INTRODUCTION

The purpose of this study is to investigate the response of two permanently instrumented buildings in Los Angeles, California. The buildings were instrumented by the California Division of Mines and Geology - Strong Motion Instrumentation Program (CSMIP) in the early 90's in order to obtain strong motion and building response data in the event of a significant ground motion episode. On January 17, 1994 many instruments installed by CSMIP throughout the Los Angeles region recorded valuable data during the Northridge earthquake.

Strong motion data for two buildings located in downtown Los Angles, the 777 Tower and the Figueroa at Wilshire Tower, were collected during the Northridge earthquake [1]. The 777 Tower is a 54-story building with a perimeter framed-tube structural system whereas the Figueroa at Wilshire Tower is a 52-story building with a spine structure and outrigger frames structural system. The 777 Tower (built in 1989) and the Figueroa at Wilshire Tower (built in 1990) have many similar characteristics and share many environmental conditions; therefore, an interpretation and comparison of the instrumentation data may be helpful in determining how each building type performed during the Northridge earthquake. An overview of the two buildings is shown in Fig. 1.

Fig. 1 Panoramic view of the buildings studied (Left: 52-storey; Right: 54-storey)

The study consisted of the following parts:

1) investigation of each building structural system;
2) comparison of base shaking experience by each building;
3) comparison of response (shock) spectra from the motions obtained at each building;
4) modal identification using the strong motion data collected at each building; and,
5) comparison of modal characteristics of each structural system.

2 DESCRIPTION OF THE BUILDINGS

In this section a general description of each of the buildings studied is presented. The structural system and geometry of each building is described as well. Details of the instrumentation and recorded motions during the 1994 Northridge earthquake at each building are also given in this section.

Figueroa at Wilshire Tower (LA52 Tower)

This 52-storey building (see Fig. 2) consists of a spine structure with braced frames and outrigger frames. The three main components are a braced core, twelve columns (eight on the perimeter and four in the core), and eight 91 cm (36 in) deep outrigger beams at each floor connecting the inner and outer columns. Generally, the footprint of this building is a 47.6 m (156 ft) squared and the overall elevation above ground level is 218 m (716 ft). There is a 5-storey parking underneath the building. Each of the main columns of the structure is supported on thick spread footings, and the site geology is alluvium over sedimentary rock. The Instrumentation of the building is at the E (lowest), A (ground), 14th, 22nd, 35th, 49th and roof levels, as shown in Fig 3.

As shown in Fig. 3, the outrigger frame is made up of outrigger beams that are connected to both the central core as well as the exterior columns. Therefore, as the central core (and the building for that matter) deflects due to lateral loads the exterior columns not only support gravity loads but also restrain the lateral movement of the building. The central core, restricted by the outrigger beams is not allowed to deflect as a cantilever and the outer columns are subjected to tensile and compressive forces depending on which side the building is deflecting to [2].

Fig. 2 LA52 Tower

777 Tower (LA54 Tower)

The 54-Story building (see Fig. 4) consists of a perimeter framed tube system with W24 perimeter columns spaced at about 3 m. Virendeel trusses were used for the vertical setbacks at the 36th and 47th floors. The footprint of this building is nearly rectangular; although, the walls in the north-south direction are rounded. The general dimensions of the footprint are 64.6 m (212 ft) by 41.5 m (136 ft) and the overall elevation above ground level is 218 m (715.5 ft). There is a 4-storey parking garage below grade. The foundation type is a reinforced concrete mat foundation and the site geology is alluvium over sedimentary rock (same as the other building). Instrumentation of the building is at the P4 (lowest), ground, 20th, 36th, 46th and penthouse levels (see Fig. 5).

Fig. 4 LA54 Tower

The tube system consists of a "rigid wall-like structure" around the exterior of the building. This structure is accomplished by the close spacing of columns along the outer edge of the building with the utilization of deep spandrel beams to connect the columns. Lateral loads are resisted through the overall bending of the tube. The columns on the side of the building undergoing an exterior force go into tension while the columns on the opposite side of the building go into compression.

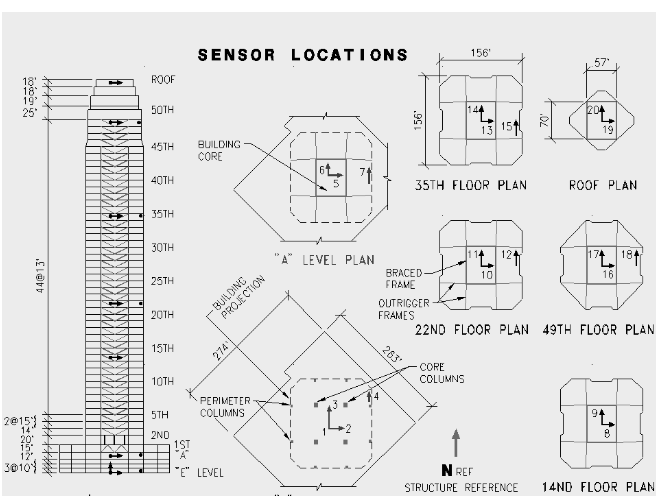


Fig. 3 Framing and Instrumentation of LA52 Tower

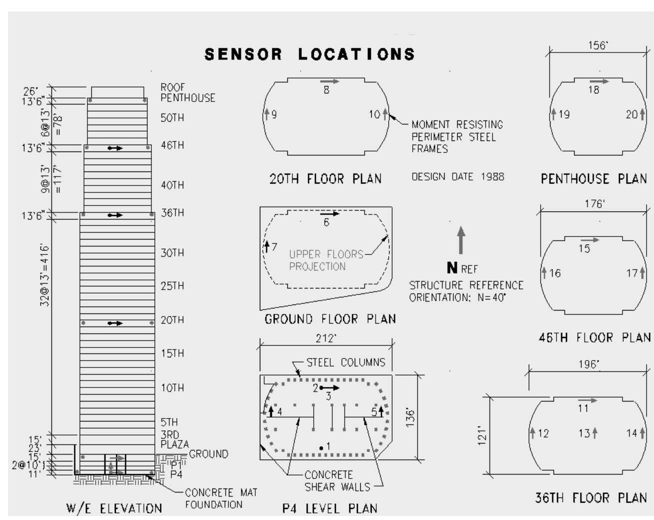


Fig. 5 Framing and Instrumentation of LA54 Tower

Recorded Motions

The epicentre of the Northridge earthquake was about 32 km northwest of the buildings and the 20 accelerometers installed in each building recorded the earthquake motions for about 180 seconds (at a sampling rate of 100 samples per second each). Tables 1 and 2 show the peak values of acceleration (A), velocity (V) and displacement (D) obtained from each of the records. Examples of the time histories of acceleration, velocity and displacement at the base and top levels of each of the buildings are presented in Figs. 6 through 13.

Table 1. Peak Values of Motions at the LA52 Tower

Level (Channel)	Location	Dir	A (g)	V (cm/s)	D (cm)
E (1)	CENTER	UP	0.11	5.33	2.02
E (2)	CENTER	E	0.11	7.97	4.04
E (3)	CENTER	N	0.14	10.99	1.97
E (4)	E. END	N	0.15	11.92	1.92
A (5)	CENTER	E	0.13	9.2	4.15
A (6)	CENTER	N	0.17	14.25	2.32
A (7)	E. END	N	0.18	14.67	2.39
14 (8)	CENTER	E	0.12	22.33	6.43
14 (9)	CENTER	N	0.19	19.76	5.18
22 (10)	CENTER	E	0.12	20.73	8.84
22 (11)	CENTER	N	0.19	22.68	6.06
22 (12)	E. WALL	N	0.19	28.48	8.28
35 (13)	CENTER	E	0.11	17.76	10.92
35 (14)	CENTER	N	0.22	20.38	7.41
35 (15)	E. WALL	N	0.23	21.64	7.35
49 (16)	CENTER	E	0.13	25.06	17.55
49 (17)	CENTER	N	0.13	21.16	10.95
49 (18)	E. WALL	N	0.15	28.28	10.48
ROOF (19)	CENTER	E	0.23	40.28	21.9
ROOF (20)	CENTER	N	0.41	39.03	13.77

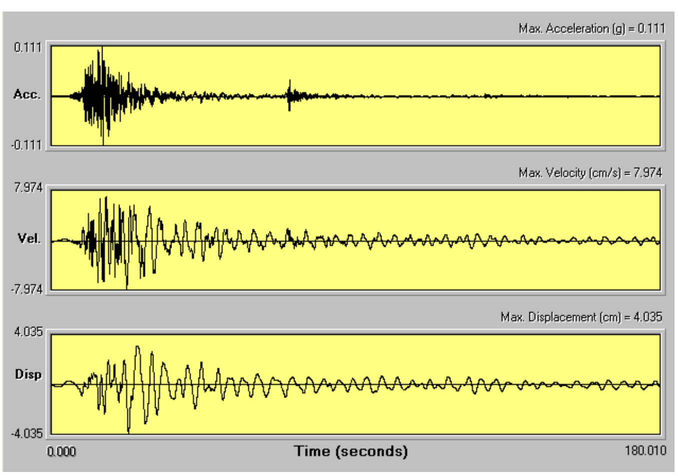


Fig. 6 Time histories of E/W motions at the base of LA52 Tower (channel 2)

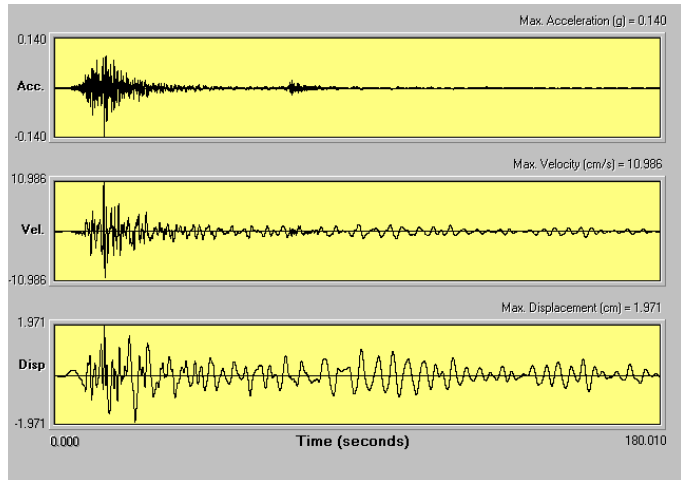


Fig. 7 Time histories of N/S motions at the base of LA52 Tower (channel 3)

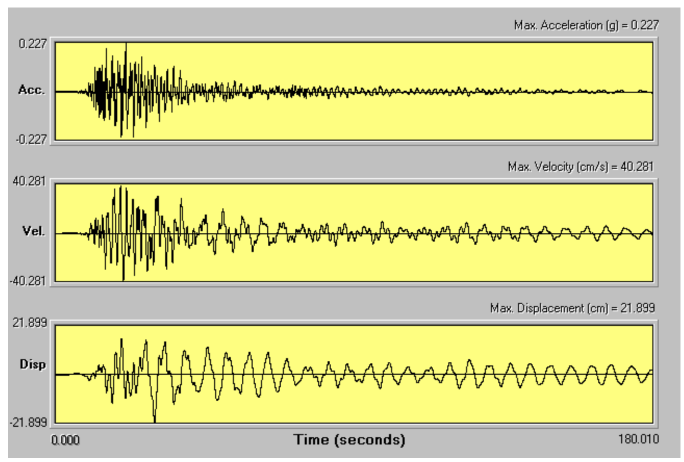


Fig. 8 Time histories of E/W motions at the top of LA52 Tower (channel 19)

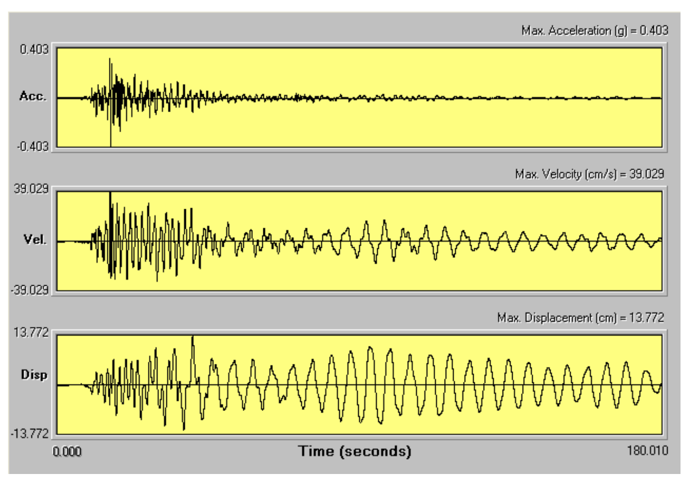


Fig. 9 Time histories of N/S motions at the top of LA52 Tower (channel 20)

Table 2. Peak Values of Motions at the LA54 Tower

Level (Channel)	Location	Dir	A (g)	V (cm/s)	D (cm)
P4 (1)	S. WALL	UP	0.07	5.22	1.89
P4 (2)	N. WALL	UP	0.08	5.33	1.93
P4 (3)	N. WALL	E	0.09	8.41	3.09
P4 (4)	W. WALL	N	0.14	9.77	2.88
P4 (5)	E. WALL	N	0.13	10.25	2.74
GROUND (6)	N. WALL	E	0.10	8.42	3.06
GROUND (7)	W. WALL	N	0.18	10.1	2.81
20 (8)	N. WALL	E	0.10	16.05	6.13
20 (9)	W. WALL	N	0.11	14.51	5.48
20 (10)	E. WALL	N	0.11	17.92	5.53
36 (11)	N. WALL	E	0.08	14.76	12.16
36 (12)	W. WALL	N	0.10	12.11	8.32
36 (13)	CENTER	N	0.12	11.45	7.93
36 (14)	E. WALL	N	0.12	10.58	8.46
46 (15)	N. WALL	E	0.14	19.12	15.15
46 (16)	W. WALL	N	0.10	15.86	9.16
46 (17)	E. WALL	N	0.09	15.12	10.4
PENTHOUSE (18)	N. WALL	E	0.14	33.54	16.76
PENTHOUSE (19)	W. WALL	N	0.19	24.42	11.34
PENTHOUSE (20)	E. WALL	N	0.14	21.57	11.69

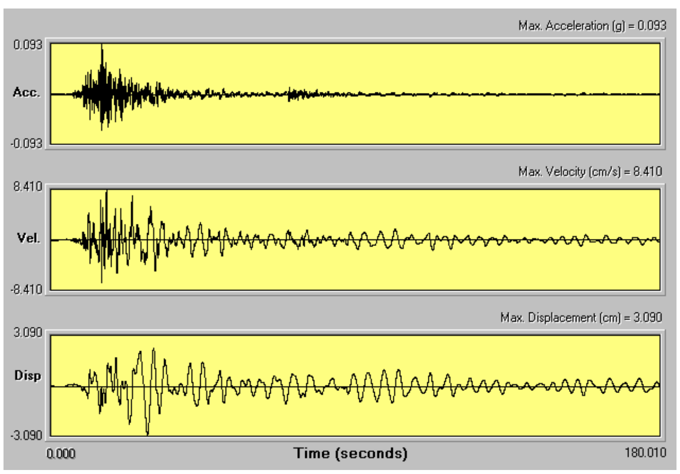


Fig. 10 Time histories of E/W motions at the base of LA54 Tower (channel 3)

As can be observed from these figures, the level of shaking at the base of each building is not that severe, but the motion at the top levels is more than twice the one at the base. Also, the duration of the strong shaking from the earthquake is only a few seconds, but the response of each building is longer than 3 minutes (180 seconds). This indicates that both buildings have very low damping and that their response after the strong ground shaking is mainly controlled by the fundamental mode of vibration. During the intensive part of the ground shaking it seems that the response is mainly dominated by the second and third modes of vibration (as evidence by the high frequencies in the early part of the responses shown in Figs. 8, 9, 12 and 13.

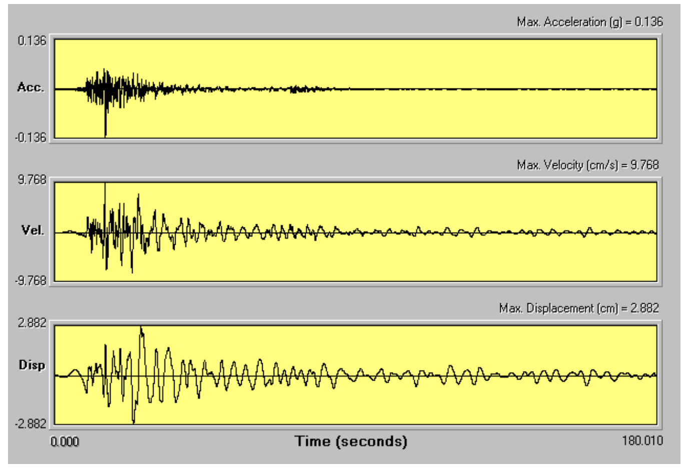


Fig. 11 Time histories of N/S motions at the base of LA54 Tower (channel 4)

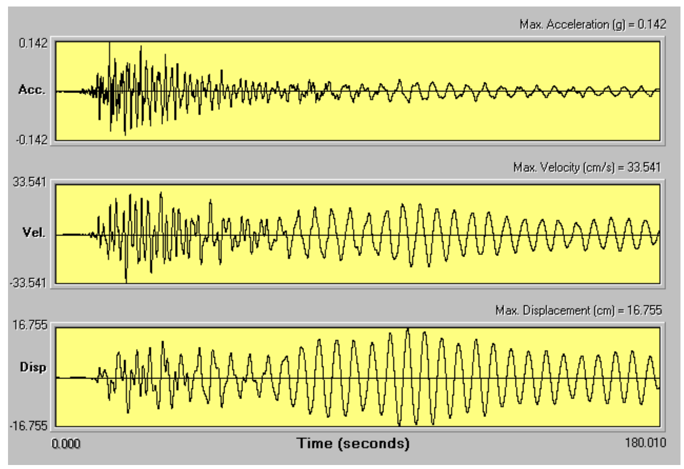


Fig. 12 Time histories of E/W motions at the top of LA54 Tower (channel 18)

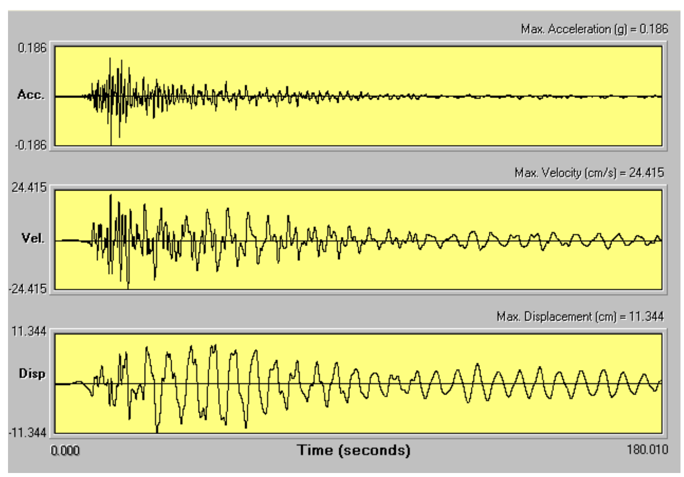
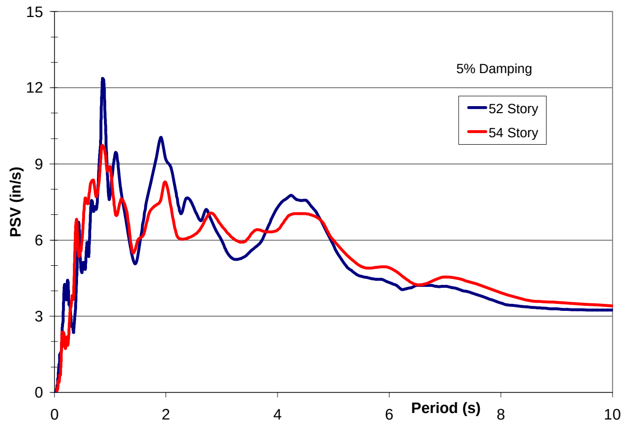
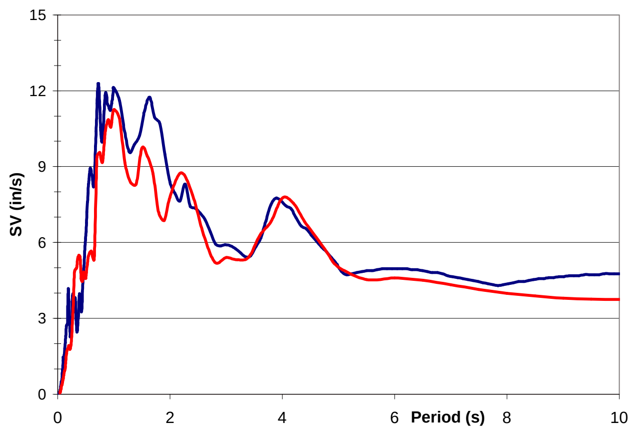


Fig. 13 Time histories of N/S motions at the top of LA54 Tower (channel 20)

Since the responses of the buildings are different, it is of interest to investigate if the reason for the difference is due to the ground shaking being different at the base of each building and the effect of the structural system used for each tower, or if the difference is only due to the latter. To this end, the motions at the base of each building needed to be rotated (Figs.3 and 5 give the building orientation angle) so that the components of base motion of the two towers were on the same direction, and thus a one-to-one comparison could be performed. A convenient way to compare the motions is by means of the response (shock) spectra for each of the motions investigated. This provides a clear picture of the similarities and differences of the motions to be compared. Figures 14 and 15 show the 5% damped Pseudo-Velocity Spectra (PSV) for the N/S and E/W components of ground shaking at the base of each building. The plots clearly show that the motions recorded at the base of each building have, in general, the same characteristics.



a) N/S component



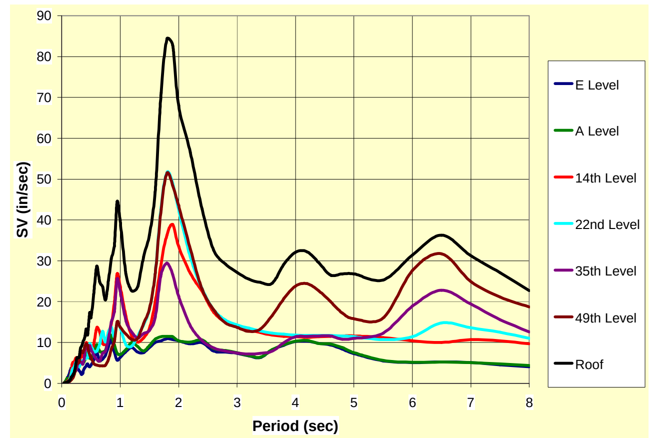
b) E/W component

Fig. 14 Pseudo-Velocity Response Spectra from motions recorded at the base of each tower

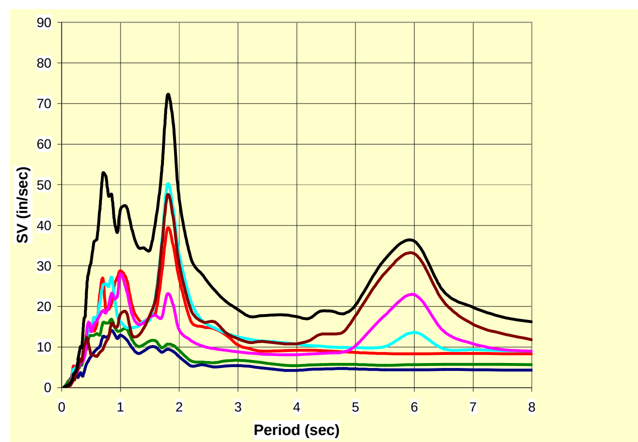
It is of interest to evaluate the response of the upper floors and compare these responses. Using the PSV spectrum as the tool for the comparison, the so-called floor-response spectra for 5% damping were computed for each of the motions recorded at each of the instrumented floors. Figures 15 and 16 show the results.

From both figures it can be seen that the larger spectral values occur at periods that may be associated with the modal periods of the building. For the period range shown in Fig. 15 it can be seen that there are five well defined periods at which significant peaks occur. The largest peak occurs at a period of about 1.8 sec, indicating that higher mode response was important for this building.

There is not much difference between the N/S and E/W components for periods below 3 sec and above 5 sec. But at around 4 sec. there is significant difference between the two components. This difference in the response spectra indicate that the motions in the E/W and N/S directions will exhibit different behaviour, as evidenced by a comparison of Figs. 8 and 9.



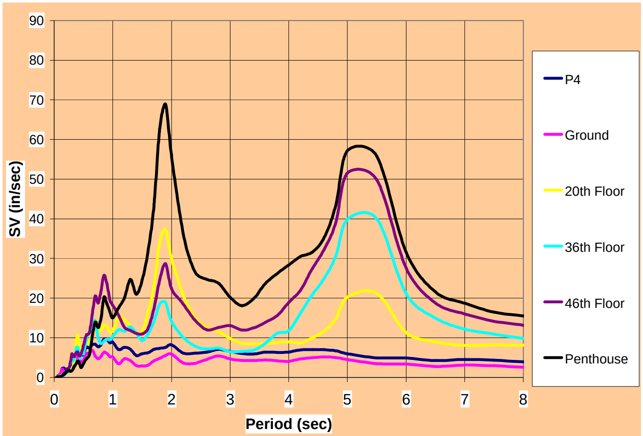
a) E/W components



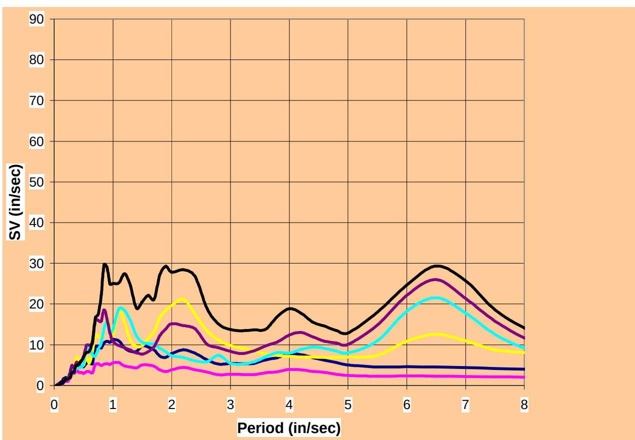
b) N/S components

Fig. 15 Pseudo-Velocity Response Spectra from motions recorded at instrumented floors of LA52 Tower

For the period range shown in Fig. 16 it can be seen that there are four well defined periods at which significant peaks occur. The largest peaks occur at periods of about 1.8 and 5.5 sec. In this case there is significant difference between the N/S and E/W components. This indicates that the building response was stronger along the E/W direction. This is also evident in Figs. 12 and 13.



a) E/W components



b) N/S components

Fig. 16 Pseudo-Velocity Response Spectra from motions recorded at instrumented floors of LA54 Tower

3 OUTPUT ONLY MODAL ANALYSIS

A possible way of determining the dynamic characteristics of these buildings from their recorded motions is to treat the recorded data as response-only data and ignore the source of the motion. Using computer program ARTeMIS [3], the Frequency Domain Decomposition technique was used to perform a modal analysis of each building. The dynamic characteristics of interest were the fundamental frequencies, damping and corresponding mode shapes of each building along its principal directions.

The ARTeMIS' FDD Peak Picking editor displays singular values of the spectral density matrices (see Fig. 17 and 18). The values of the first 6 structural mode peaks are shown in Table 3. These values represent the natural frequencies (periods) of each building. The FDD technique can also be used for modal damping estimation [4], and ARTeMIS has the capability of damping estimation using an Enhanced Frequency Domain Decomposition technique. As expected, the damping values listed in the table are all below 5%.

Even though the spatial distribution of the sensors is not enough to clearly define the mode shapes, a simple linear interpolation between instrumented floors helps provide a clear picture of the nature of each of the modes identified in the analysis. Figure 19 shows 3D wireframe views of the first six modes of vibration of each of the buildings. The modes are well defined in the N/S, E/W, and torsional directions.

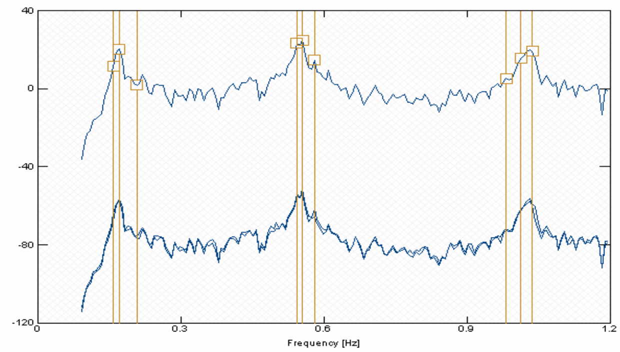


Fig. 17 LA52 Tower: singular values of the spectral density matrices

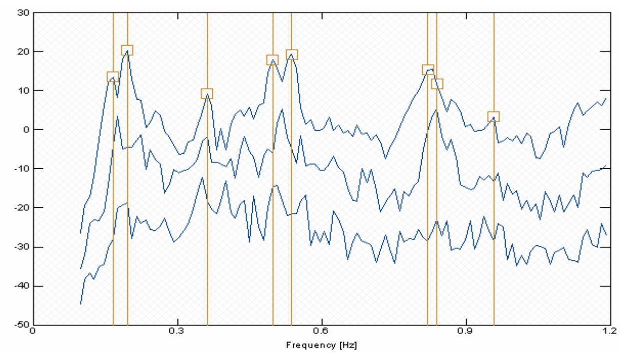


Fig. 18 LA54 Tower: singular values of the spectral density matrices

Table 3. First 6 natural frequencies and modal damping of the LA52 and LA54 Towers

Mode No.	*LA52 Tower*		
	Frequency (Hz)	***Period (sec)***	***Damping (%)***
1	0.159	6.30	1.3
2	0.171	5.85	2.7
3	0.207	4.82	1.8
4	0.543	1.84	1.5
5	0.555	1.80	1.3
6	0.983	1.73	-

Mode No.	*LA54 Tower*		
	Frequency (Hz)	***Period (sec)***	***Damping (%)***
1	0.166	6.02	4.8
2	0.195	5.12	3.0
3	0.361	2.77	1.9
4	0.498	2.01	1.9
5	0.537	1.86	1.4
6	0.820	1.22	1.0

4 CONCLUSIONS

Various time-domain and frequency-domain techniques have been used to extract useful information about the dynamic behaviour of two well instrumented buildings during a severe earthquake. It has been shown that similar ground shaking at the base of two nearby tall buildings of similar height and floor area can produce significantly different responses, depending on the type of lateral force resisting system. The study of each building's behaviour was significant for two reasons: 1) the structural differences between the buildings (while one building relies on a perimeter tube system the other building relies on a spine structure with outrigger frames); and, 2) the buildings shared similar environmental conditions. As a result, the investigation of how each building performed during the Northridge Earthquake has given good insight into how these buildings performed, and what can be expected during more severe earthquakes.

ACKNOWLEDGEMENTS

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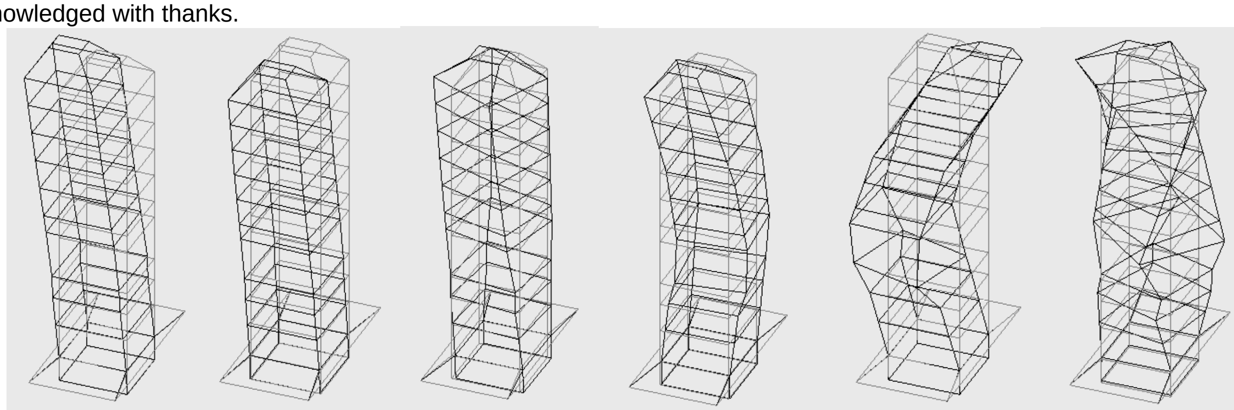
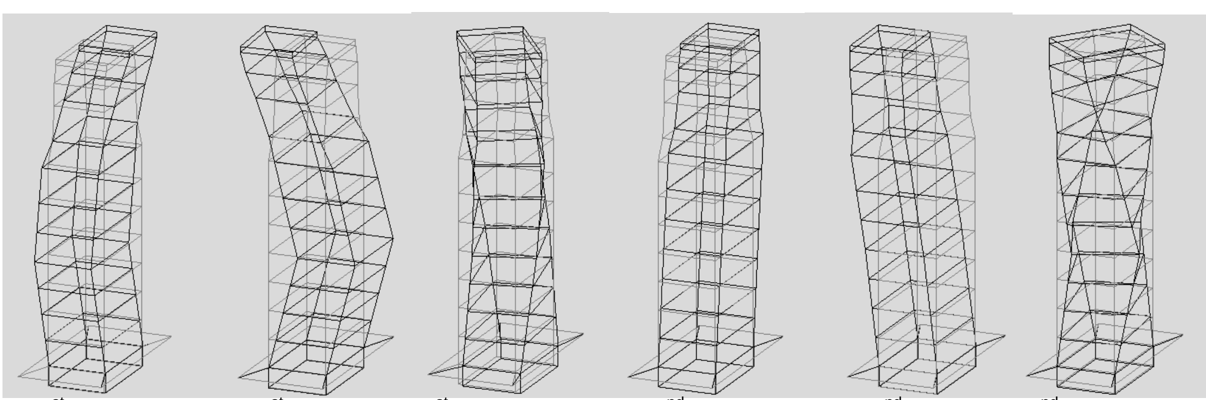


1st E/W (0.159 Hz) | 1st N/S (0.171Hz) | 1st Torsional (0.207 Hz) | 2nd E/W (0.543 Hz) | 2nd N/S (0.555 Hz) | 2nd Torsional (0.983 Hz)

a) Mode shapes of LA52 Tower



1st N/S (0.166 Hz) | 1st E/W (0.195Hz) | 1st Torsional (0.361 Hz) | 2nd N/S (0.498 Hz) | 2nd E/W (0.537 Hz) | 2nd Torsional (0.820 Hz)

b) Mode shapes of LA54 Tower

Fig. 19 First six mode shapes obtained from modal analysis