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COMPARISON OF NORTHRIDGE EARTHQUAKE RESPONSES OF A BASE-ISOLATED AND A CONVENTIONAL HOSPITAL BUILDING

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ABSTRACT

Recorded data during the 17 January 1994 Northridge, California earthquake (M_s=6.8) from the base-isolated University of Southern California (USC) and conventionally designed Olive View Hospital (OVH) buildings indicate that they were subjected to design level peak accelerations. The data reveals successes of the two different approaches of design. Both buildings performed well during the earthquake. The USC building is on rock and OVH is on alluvial deposit. The isolators of the USC building experienced non-linear displacements with peak at 3.5 cm that put them within 10 % shear strain and effectively dissipated the energy of incoming motions attaining equivalent viscous damping of approximately 10 % of the critical. The superstructure experienced reduced peak-accelerations as compared to the free-field or the foundation and did not suffer structural or contents damage. On the other hand, the conservatively designed OVH (with high lateral load resisting capability) experienced peak horizontal acceleration of 2.31 g at the roof compared to 0.82 g and 0.91 g, respectively, at the ground floor and free-field. The OVH suffered only limited non-structural damage. This stiff structure was not affected by the long duration pulses of the motions of this earthquake; however, it was affected by the resonating frequencies of the site.

KEYWORDS

base-isolation, viscous damping, hysteretic behavior, response spectra, shear wall, peak acceleration

INTRODUCTION

Studying the responses of structures during earthquakes constitutes an important element of earthquake hazard reduction programs in order to confirm or improve methodologies for design and analyses of structures. Response records retrieved by California Division of Mines and Geology (CDMG) from (1) the base isolated USC Hospital building in Los Angeles, at 36 km from the epicenter, and (2) conventionally designed OVH Building in Sylmar (California), at 16 km from the epicenter of the (M_s=6.8) Northridge (California) earthquake of January 17, 1994 are particularly significant (Shakal, et al. 1994) because both buildings were tested with input motions having peak acceleration levels equivalent to the postulated in their respective design criteria and long period pulses typically generated by near-fault ground motions or surface wave motions as in the Northridge earthquake. Long-period pulses result in large velocities and displacements. The effective fundamental period of an isolated structure is typically 3-5 times longer than an otherwise fixed base structure. On the other hand, the critical damping percentages of the isolators are typically greater than 10 %. Even then, how base-isolated systems respond to such long-period motions has been a cause for concern. The USC records are the first set of strong-motion records from a base-isolated structure where the displacements are large enough to show that the isolators experienced non-linear displacement excursions and effectively reduced the peak accelerations and relative displacements of the superstructure. The OVH response exhibits possibly the largest peak accelerations at the input (ground) level [0.91g] and the roof level [2.31g]. The OVH building was designed in 1976 to increased level of seismic forces as a reaction to the disastrous fate of its predecessor, the original OVH building that was severely damaged during the M_s=6.4 San Fernando (California) earthquake of February 9, 1971 and was later razed. Figure 1 shows the location of the USC hospital and the OVH building relative to the epicenter of the Northridge earthquake and significant peak accelerations recorded at select sites during that earthquake. The objective of this paper is to assess the performance of both hospital buildings during the Northridge earthquake. Response records of OVH building recorded at 45 km epicentral distance from the (M_s=5.9) Whittier earthquake of October 1, 1987 are also included in this study as they reveal significant response characteristics of this unique building (Huang, et al., 1989). The epicenter of that earthquake is also shown in Figure 1. Space limitations allow presentation of limited facts and references. Further details and references are provided in Çelebi (1996a and 1996b).

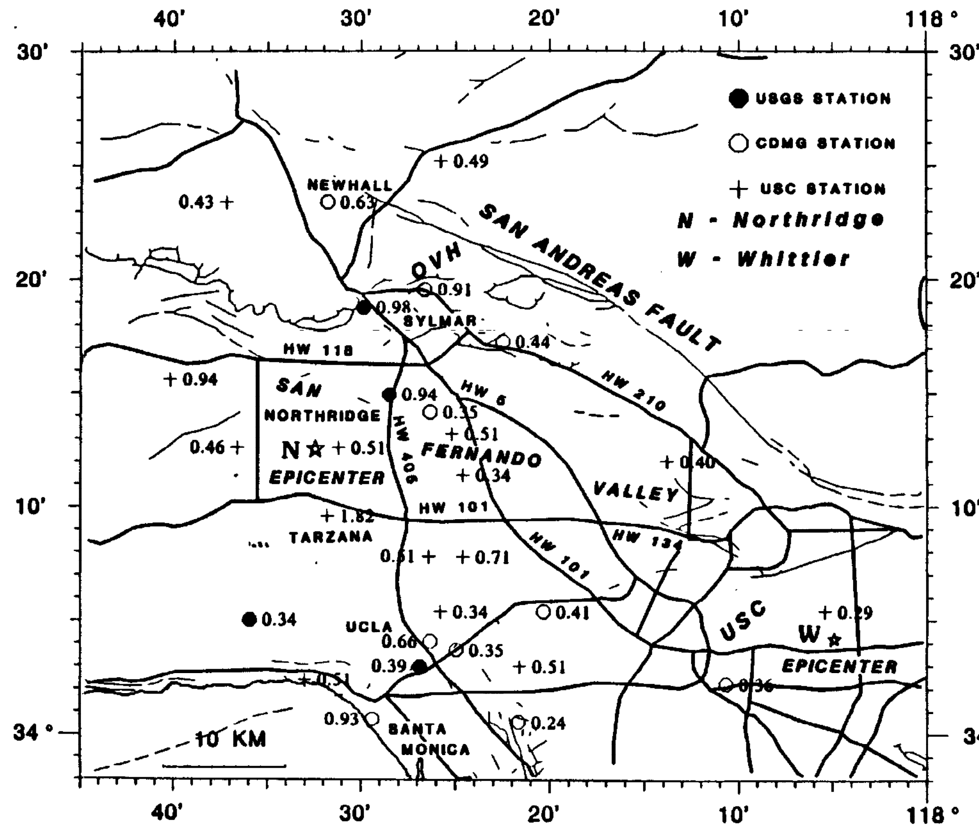

Fig. 1. Location of Sylmar relative to epicenters of the Northridge and Whittier earthquakes and significant peak accelerations recorded during the Northridge earthquake.

Table 1. Peak Accelerations and Dispacements of USC Hospital

Location	Max. Peak Accel. (g)			Max. Peak Displ. (cm)		
	NS	EW	UP	NS	EW	UP
Roof	.21	.19	-	3.9	5.1	-
6th Fl.	.11	.15	-	3.3	4.0	-
4th Fl.	.10	.16	-	3.1	3.3	-
Above Isolators	.13	.14	.10	2.8	3.0	1.3
Below Isolators	.37	.17	.09	1.7	2.3	1.4
Free-Field	.49	.22	.12	2.3	2.5	1.3

Three dimensional schematics of the eight story building and its instrumentation is seen in Figure 2a. Its steel superstructure is supported by 149 isolators (34.6 cm high) on continuous concrete spread footings. The diagonally braced perimeter frames supported by 67 lead-rubber isolators (55.9 cm square) are designed to carry the lateral loads. The internal vertical load carrying columns are supported by 82 elastomeric isolators (66.0 cm square). The building is on an outcrop within 15 km of the Newport-Inglewood fault. It is designed for a maximum relative isolator displacement of 26 cm and to ATC-6 [S1] (same as UBC 1988 and 1994) spectrum with ZPA equal to 0.4 g -- increased by 20 % to account for near field effects. The maximum peak accelerations and displacements extracted from the data, summarized in Table 1, indicate that the free-field peak in either direction is larger than that of (a) the design level, (b) above the isolators or the roof - clear evidence that the isolators were effective in dissipating the incoming vibrational energy. Response spectra (5 % damped) of horizontal free-field motions are compared in Figure 3a with the UBC (S1) design response spectra scaled to 0.48 g. The normalized free-field response spectra (5 % damped) are compared in Figure 3b with that of the shape of the design response for UBC (or ATC) S1 soil type. In general the shapes of the spectra of recorded components of motions appears to be well enveloped by the code spectrum except for some high frequency (>1Hz) bands for which it is exceeded. Figures 3c and d show comparison of 5 % damped response spectra for the locations at the roof, above and below isolators and free-field. It is seen that below 0.5 seconds, the free-field spectra exceeds those at the roof.

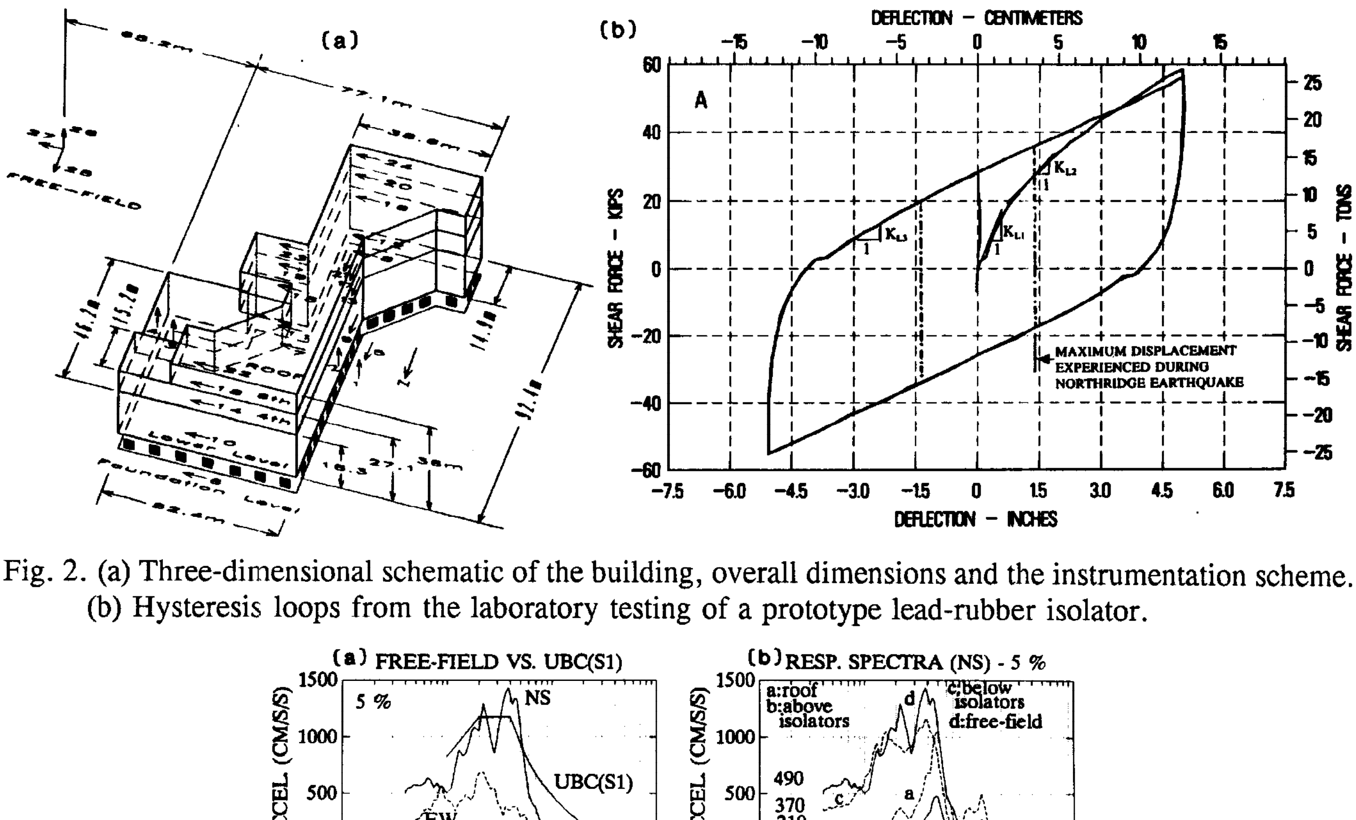

Fig. 2. (a) Three-dimensional schematic of the building, overall dimensions and the instrumentation scheme. (b) Hysteresis loops from the laboratory testing of a prototype lead-rubber isolator.

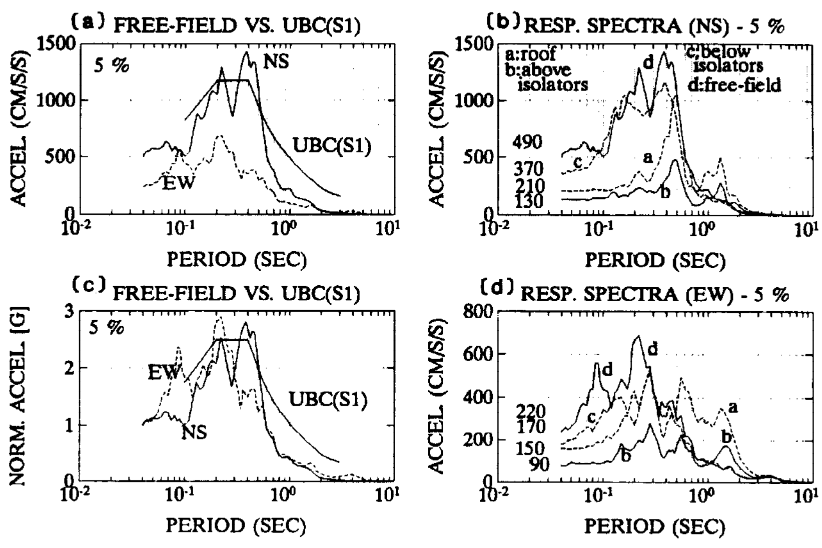

Fig. 3. Comparison of (a) free-field spectra with design response spectra, (b) normalized free-field and design response spectra and (c, d) spectra of motions at different levels.

Figure 4 shows accelerations at various levels of the building in both directions, their amplitude spectra and relative accelerations and displacements of the isolators. It should be noted that the roof response increases significantly as compared to the response of the 6th floor due to change in plan above that level. In the NS direction, the maximum relative displacement is about 3.5 cm while in the EW direction, it is about 3.0 cm (at the south end). The displacements and relative displacements correspond to (a) the largest drift ratio of approximately 10 % of the allowable, (b) maximum 10 % shear strain (maximum relative displacement divided by the height of the isolators), (c) stiffness K_2 of the actual hysteretic curve of the isolators (Figure 2b) and (d) to hysteretic damping of approximately 10 % calculated by the relationship $d=\Delta(w)/4\pi(W)$ where $\Delta(w)$ is the actual area of the hysteresis loop at a displacement level and W is the hypothetical elastic energy defined by the area formed by the line defined by the elastic slope at the same displacement. The level of damping, approximately 10 % and 15 %, for the 1st and 2nd modes, respectively are also extracted by system identification technique (Çelebi, 1986a). The first mode dominates the response of the building. Figure 5 shows cross-spectra, coherence function and phase angle plots of roof and 4th. floor, and relative torsional accelerations between the roof and the lower level. The family of (linear and non-linear frequencies attributed to the first and second modes seen in the cross-spectra are coherent and are in phase. The first mode frequency starts at 1 Hz and due to nonlinear behavior of the isolators shifts to 0.75 Hz in the NS direction and similarly starts with 1 Hz and shifts to 0.62 Hz in the EW direction. These clear peaks are indicative of the fact that distinctive instantaneous stiffnesses can be defined for the isolators as a function of the relative isolator displacements. Similarly, for the second mode, the motions are 180 deg out of phase at around 1.5 Hz and shift in the frequences are also clearly identifiable for that mode.

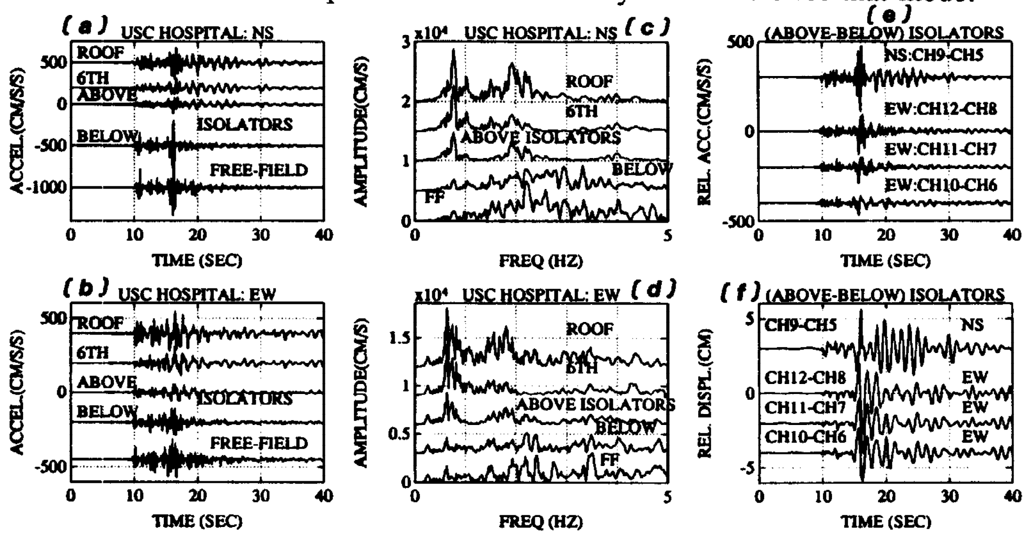

Fig. 4. (a) Accelerations and (b) displacements at various levels and (c, d) their amplitude spectra, (e, f) relative accelerations and displacement of the isolators.

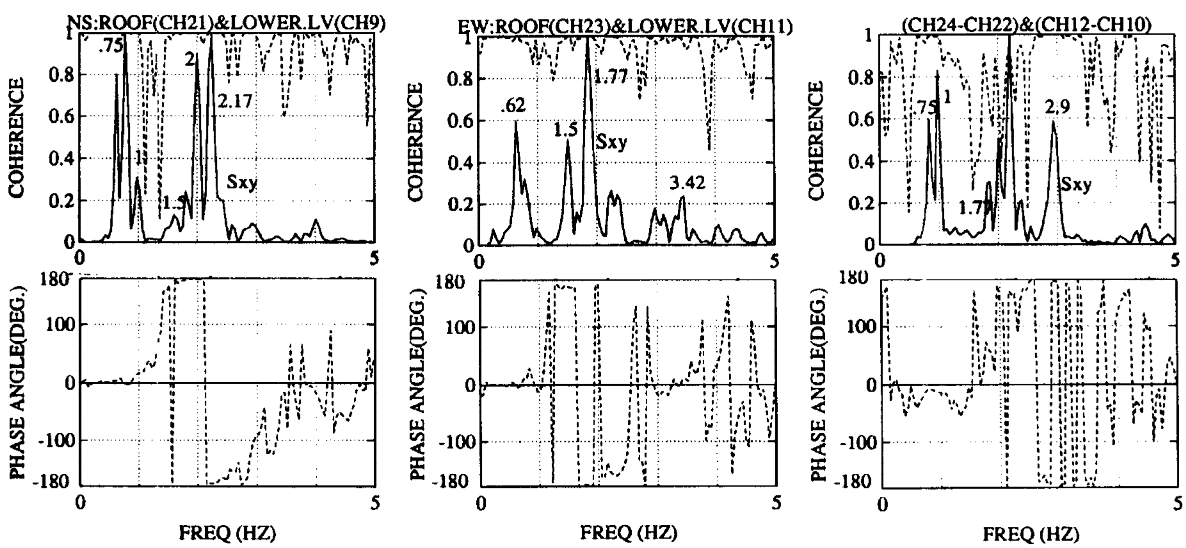

Fig. 5. Cross-spectra, coherence function and phase angle plots motions at roof and 4th floor and torsional motions at the roof and lower level.

Figure 6a shows vertical sections, major structural elements in the plan views and the instrumentation of the six story building and its associated free-field. The lateral force resisting system of the building is a mixed design of concrete and steel shear walls. The ground floor and second floor are typically 10 inches thick concrete shear walls that extend along several column lines. At the 3rd level, the plan of the building changes to a cross shape making a four story cruciform tower with steel shear walls surrounding the perimeter. The foundation of the building consists of spread footings and grade mat on the ground floor. The seismic design criteria of the building was based on two levels of performances defined by the design and survivability level earthquake spectra with ZPA of 0.52g and 0.69g respectively. The survivability earthquake is postulated for a Richter magnitude 8.5 earthquake. The two design response spectra (for 5% damping) are compared in Figure 7 with response spectra of the free-field at the OVH grounds. Also shown in the figure are the response spectra of the free-field (at epicentral distance of 45 km) motions recorded during the 1987 Whittier earthquake. Peak accelerations and absolute displacements for motions recorded at the roof, ground floor, and free-field, during both events are summarized in Table 2. The maximum drift reached in the Northridge event was less than 0.2%.

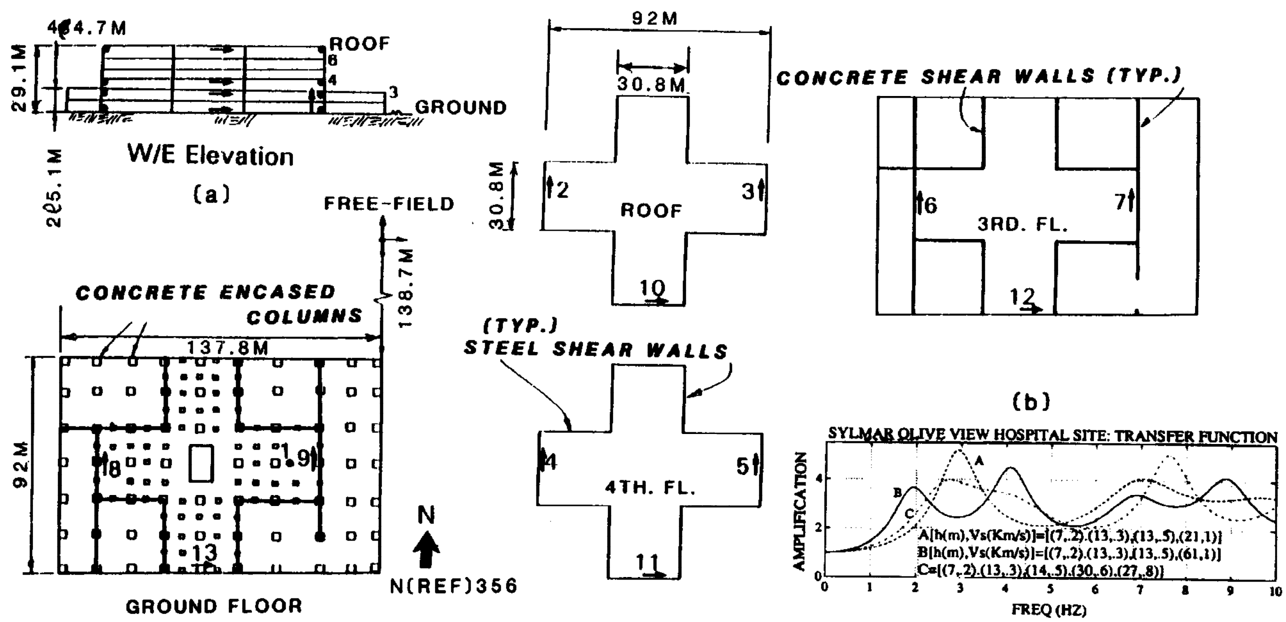

Fig. 6. (a) Vertical section, plan and seismic sensor layout of the OVMC Building, Sylmar, California, and (b) site transfer functions of OVMC building site.

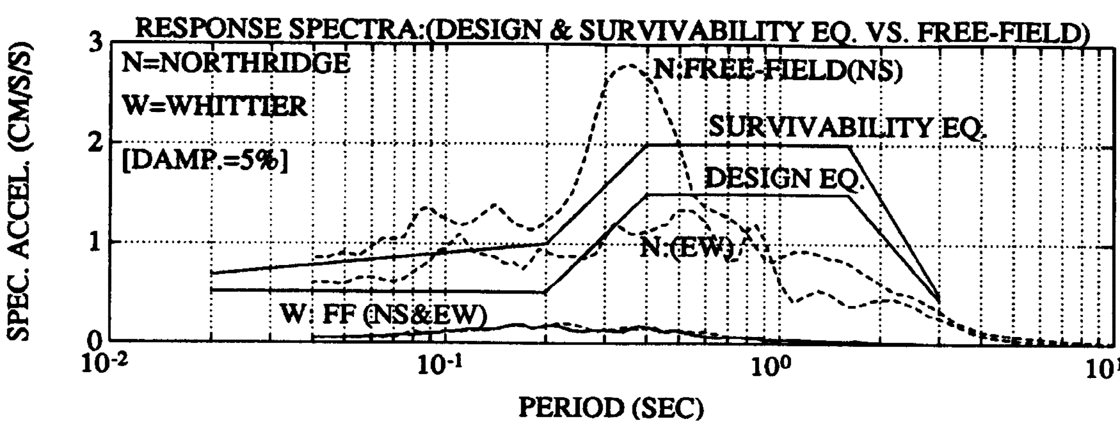

Fig. 7. Comparison of design response spectra with response spectra of free-field motions recorded at the OVMC site during the Northridge and Whittier earthquakes.

The building is located on an alluvial fan. Several borehole logs up to 100 m taken in the immediate vicinity of the building indicate that the site has approximately 3-5 m of dense to very dense silty sand and gravel, with underlying alluvial deposits of dense to very dense sand and silty sand with gravel and cobbles and that the depth to bedrock is between 52-92 m (see references in Çelebi [1986a]). The thickness of the alluvium decreases towards the hills (north of the hospital). Using three possible profiles, site transfer functions corresponding for two different depths and varying shear velocities of layered substrata are calculated based on shear wave propagation method resulting in the fundamental site frequency (period) of approximately 2-3 Hz (0.33-0.5 sec) [Figure 6b].

Table 2. Peak Acceleration and Displacements at OVH

Location	Northridge Eq.				Whittier Eq.			
	NS		EW		NS		EW	
	A(g)	D(cm)	A(g)	D(cm)	A(g)	D(cm)	A(g)	D(cm)
Roof	2.31	34.1	0.79	19.2	0.20	0.65	0.16	0.79
Ground Floor	0.82	28.3	0.42	18.4	0.06	0.53	0.06	0.56
Free-Field	0.91	32.6	0.61	15.2	0.06	0.55	0.05	0.51

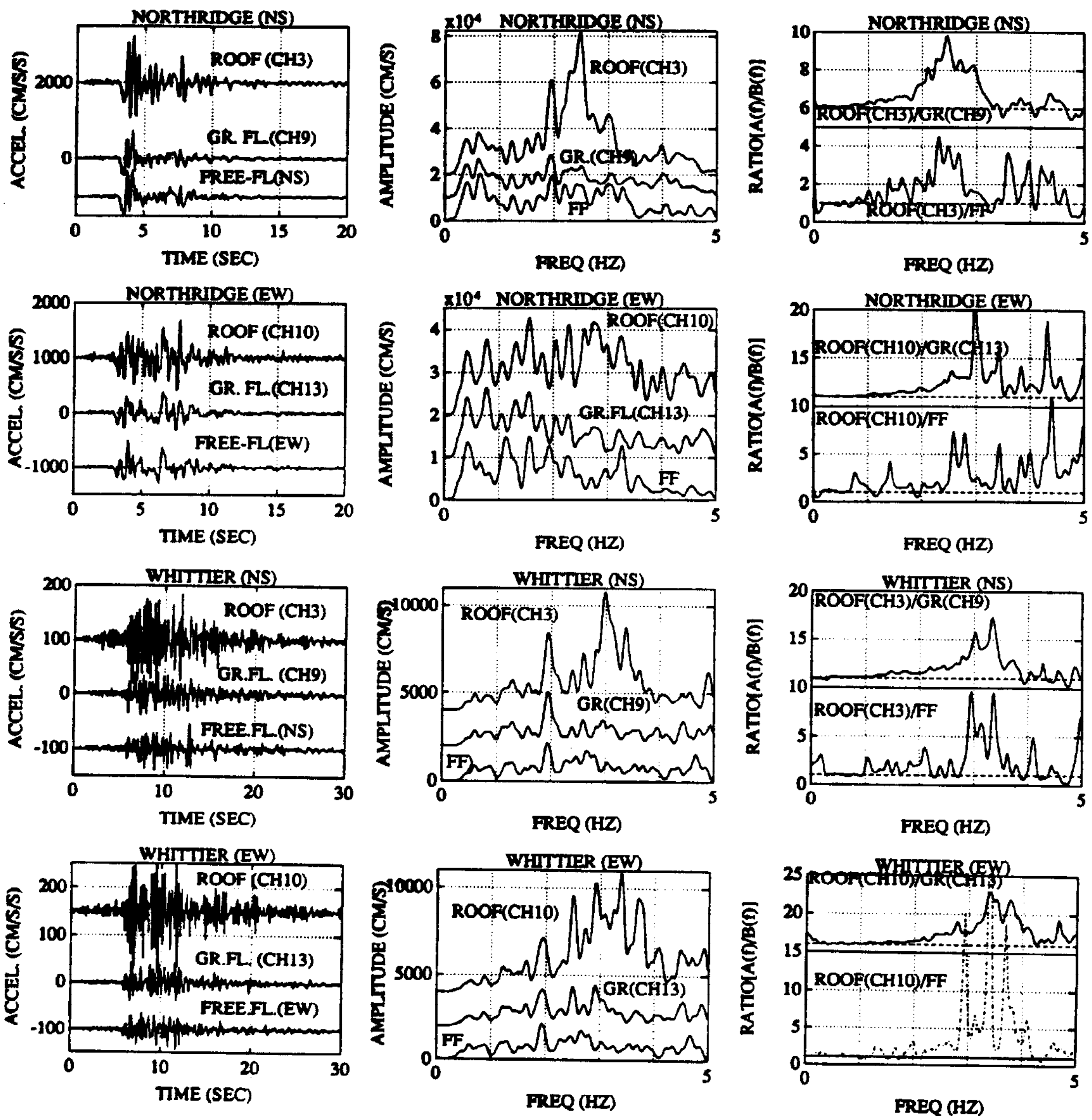

Fig. 8. Accelerations recorded during the Northridge and Whittier earthquakes at the roof, ground level and free-field and corresponding amplitude spectra and spectral ratios.

Figure 8 shows acceleration time histories for both the Northridge and Whittier events at the roof, ground floor and the free-field and corresponding amplitude spectra and spectral ratios. In the NS direction, for lower amplitude shaking during the Whittier earthquake, the structural frequencies are ≥ 3 Hz and for the stronger shaking during the Nothridge earthquake, the frequency cluster is between 2-3 Hz. In the EW direction, the lowest frequency is at 3 Hz (Northridge) and 3.3-3.6 Hz (Whittier). The 2 Hz (0.5 sec) is the fundamental site frequency (period) because the spectral ratio of roof/ground floor or roof/free-field motions is approximately 1 (the effect of the site frequency cancels out) or small. Therefore, two effects are occurring: (1) the frequencies (periods) at the roof are lower (higher) during the strong shaking and (2) the structural and free-field fundamental frequencies (periods) are very close to one another -- evidence that the structure was in resonance between 0.3-0.4 sec (2.5-3.3 Hz) which is also well within the site period (frequency) of 0.33-0.5 sec (2-3 Hz) calculated from the soil log. The building escapes most of the low frequency (<2 Hz) energy in both events.

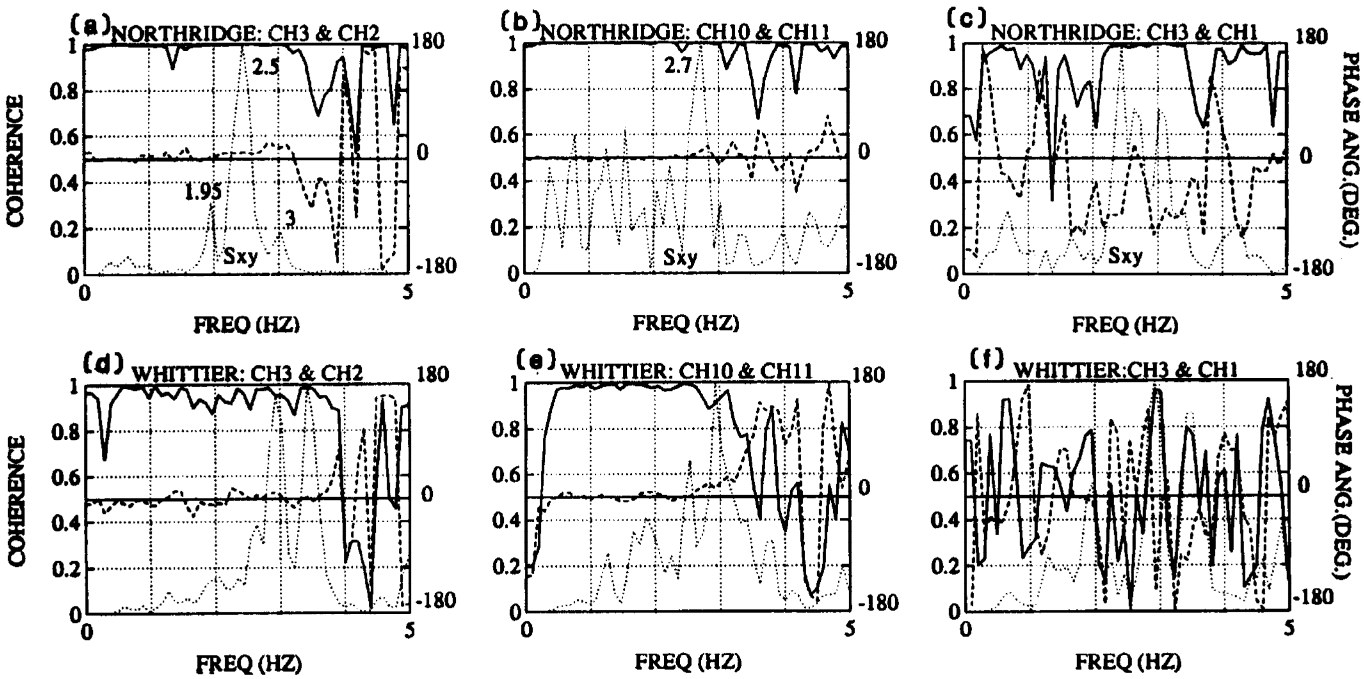

Fig. 9. Coherency, phase angle and cross-spectra plots of (a) parallel NS motions at the roof, (b) EW motions at the roof and 4th floor, and (c) horizontal motion at the roof and vertical motion at ground floor for the Northridge earthquake. d, e, and f are for the Whittier earthquake.

Figure 9a,b,d,e show the coherence, phase angle and cross-spectra (Sxy) plots of the two (NS) parallel motions at the roof and two parallel (EW) motions at the roof and 4th floor of the building for both events. The frequencies (*e.g.*, for Northridge, 3.0 and 2.5 Hz in the NS and 3.3 and 2.7 Hz in the EW direction) are close and are in phase. Therefore, the parallel motions at two wings of the cruciform tower are not torsional but can possibly be attributed to the fundamental mode of the complete structure (higher frequency) being different than that of the wing of the cruciform (lower frequency). To clarify this effect, an additional sensor should be deployed in the core of the cruciform. Figures 9c,f are prepared to investigate rocking as a form of SSI. While for the Northridge event, rocking may have taken place in the NS direction because of unity coherence and 0° degree phase angle at the fundamental mode frequency (2.5 Hz) that clearly peaks out in the cross-spectra, the same argument cannot be made for the EW direction motions or for the Whittier earthquake motions because they have low coherency and are not in phase at the appropriate frequencies. Therefore, more vertical sensors should be deployed on the ground floor to clarify this effect with certainty.

Damping ratios extracted from system identification procedures are 10-15 % (NS) and 5-10 % (EW) for Northridge and 1-4 % (NS) and 5-8 % (EW) for Whittier. These results are consistent with the direction and level of shaking during the Northridge or the Whittier earthquake. The high level of damping ratio assessed from stronger shaking during the Northridge earthquake may be attributed to radiation damping of the incoming motions. This is further explained elsewhere (Çelebi, 1986b).

CONCLUSIONS

Both the base-isolated USC hospital and conventionally and conservatively designed OVH building performed well during the Northridge earthquake of 17 January 1994. The USC data is the first set of data from any base-isolated building that exhibits excursions into the nonlinear range of the isolators. The drift ratios experienced by the superstructure is less than 10 % of the allowable which should explain that (a) there was no damage to the structure or its contents and (b) isolators performed well and effectively dissipated (reaching 10 % hysteretic damping) the energy of motions with acceleration levels equivalent to the design level accelerations. The effective performance of the isolators also reduced the drift ratios of the building.

The OVH Building data shows that with fundamental mode period (frequencies) between 0.3-0.4 sec (2.5-3.3 Hz) escapes the wrath of the long period pulses (> 1 sec) characteristic of the Northridge ground motions. The effective structural frequencies derived from the Northridge and Whittier data are different and exhibit variations attributable to non-linear effects: (a) soil-structure interaction which is more pronounced during the strong shaking of the Northridge earthquake. The building possibly experienced rocking at 2.5 Hz in the NS direction during the Northridge event and there is possibility that radiation damping at the foundation contributed to the response. (b) non-linear behavior due to minor structural damage during the Northridge earthquake. It is also likely that the cruciform wings responses with a different frequency than that of the overall building. Additional sensors are needed to distinguish the response of the wings and the core and to clearly verify if rocking occurs.

The designer of the building conceived a very strong and stiff structure (particularly in response to the disastrous performance of the original Olive View Hospital building during the 1971 San Fernando earthquake). However, site frequencies were not evaluated and therefore consequencies were not given proper attention in the development of the design response spectrum for the new OVMC building. The data shows that fundamental frequencies of the building are well within the influence of the site frequency (2-3 Hz) to cause resonance. This case study indicates that representation of site resonating frequencies need to be improved in development of design response spectra. Finally, although the performance of the OVMC may be considered to be satisfactory during the (M_s=6.8) Northridge event when the shaking was at the postulated peak design level accelerations, it is reasonable to ponder; however, what the realistic ground motions would be and how the building would perform during motions generated by the postulated M_s=8.5 design level earthquake as defined by the design criteria of this building.

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