Lessons Learned from Performance of Nonstructural Components During the January 17, 1994 Northridge Earthquake -- Case Studies of Six Instrumented Multistory Buildings --

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ABSTRACT: *As a part of a project sponsored by Strong Motion Instrumentation Program of state of California [4], twenty extensively instrumented buildings were inspected, their damage state documented, and the level of forces and deformations that they experienced were compared to design code levels at the time of design as well as more modern code provisions. Among these twenty buildings, there were several buildings from which significant amount of information, including photos and valuable response characteristics were gathered in relation to performance of nonstructural systems and components. Six of these buildings, as follows, are selected for presentation in this paper. 1. The Olive View Hospital Building in Sylmar, 2. A 13 story office building in Sherman Oaks, 3. A 10 story residential building in Burbank, 4. A 6 story commercial building in Burbank, 5. A 3 story department store in Century City, 6. A 20 story hotel in north Hollywood. The imposed seismic demands and the extent of nonstructural damage in each building is compared and contrasted with force demands as interpreted by model codes and guidelines such as UBC-97, NEHRP-97, and FEMA-273 documents.*

Keywords: Instrumented buildings; Extensive instrumentation; Seismic performance; Nonstructural systems; Performance of equipment; 1994 Northridge earthquake; Electronic information systems

1. INTRODUCTION

1.1. Description of Buildings

1.1.1. Burbank 10 Story Residential Building

This building was designed and constructed in 1974. Its vertical load carrying system consists of precast and poured-in-place concrete floor slabs supported by precast concrete bearing walls. The lateral load resisting system consists of precast concrete shear walls in both direction. The foundation system includes concrete caissons which are 7.6 to 10.6 meters deep. The largest peak horizontal accelerations recorded at the base (Channel 1, N-S) and at the roof (Channel 2, N-S) are 0.34g and 0.77g, respectively. The peak velocity at the roof is about 63cm/sec.

1.1.2. Burbank 6 Story Commercial Building

This steel moment frame building was designed in 1976 and constructed in 1977. The vertical load carrying system consists of 7.5cm concrete slab over metal deck supported by steel frames. The lateral load resisting moment frames are located at the perimeter of the building. The foundation system includes concrete caissons approximately 9.76 meters deep. The largest peak horizontal acceleration recorded at the base (Channel 9, E-W) and at the roof (Channel 3, E-W) were 0.36g and 0.47g, respectively. The peak velocity recorded at the roof was about 48cm/sec.

1.1.3. Los Angeles 3 Story Commercial Building

This department store building has three stories above and two parking levels below the ground. The building was designed in 1974 and constructed in 1975-76. The vertical load carrying system consists of 8.25cm of lightweight concrete slab over metal deck in upper three floors and 45.8cm thick waffle slabs in the basement floors. The lateral load resisting system is steel braced frames in the upper three stories and concrete shear walls in the parking floors. The foundation system consists of spread footings and drilled bell caissons. The largest peak horizontal accelerations recorded at the ground, Channel 9 was 0.36g. At the roof, (Channel 2, E-W) recorded a peak horizontal acceleration of 0.97g and a peak velocity of 57cm/sec. The overall drift index of more than 1% (see Table 1) is rather large for a braced frame system.

1.1.4. North Hollywood 20 Story Hotel

This hotel has 20 stories above and one level below the ground. It was designed in 1967 and constructed in 1968. The vertical load carrying system consists of 11.4 to 15.2 cm thick reinforced concrete slabs supported by concrete beams and columns. The lateral load resisting system consists of ductile moment resisting concrete frames in the upper stories and concrete shear walls in the basement. The exterior frames in the transverse direction are infilled between the second and the 19th floors. The building rests on spread footings. The largest peak horizontal accelerations recorded at the basement (Channel 1, N-S) and at the roof (Channel 2, N-S) were 0.33g and 0.66g, respectively. The largest velocity recorded at the roof was about 77cm/sec. The maximum overall drift index experienced by the building was moderate at 0.0036.

1.1.5. Sherman Oaks 13 Story Commercial Building

This office building has 13 stories above and two floors below the ground. It was designed in 1964. The vertical load carrying system consists of 11.4cm thick one-way concrete slabs supported by concrete beams, girders and columns. The lateral load resisting system consists of moment resisting reinforced concrete frames in the upper stories and reinforced concrete shear walls in the basements. The foundation system consists of concrete piles. The first floor spandrel girders were modified by post-tensioning after the 1971 San Fernando earthquake. The largest peak horizontal accelerations recorded at the basement (Channel 15, N-S) and at the roof (Channel 3, N-S) are 0.46g and 0.65g, respectively. The middle floors (see sensor data on the 2nd and 8th floors) experienced large acceleration in the neighborhood of 0.6g. The largest velocity recorded at the roof was about 68cm/sec. It is interesting to note that while the maximum base shear was experienced in the N-S direction, the maximum lateral displacement and an overall drift index of 0.0067 occurred in the E-W direction.

1.1.6. Sylmar 6 Story County Hospital

This hospital is a unique building built on the site of the old Olive View hospital building which suffered major and irreparable damage during the 1971 San Fernando earthquake. Designed with the explicit intention of resisting the most damaging earthquakes as perceived at the time, during the Northridge earthquake the structure passed the test of time with flying colors. What happened to the contents, however, as documented by dozens of photos contained in the SMIP information system [4] is an entirely another story. Damage to the contents of this building is discussed later in this paper. This six story cruciform shaped building has no basement. It was designed in 1976 and was constructed during the period of 1977 to 1986. Its vertical load carrying system consists of reinforced concrete slabs over metal deck supported by steel frames. The lateral load resisting system consists of reinforced concrete shear walls in lower two floors and steel shear walls encased in concrete at the perimeter of the upper four floors. The building rests on spread footings. The “free-field” station located at the parking lot adjacent to the building recorded 0.91g, 0.61g, and 0.60g in the N-S, E-W, and vertical directions, respectively. The largest peak horizontal accelerations recorded at the ground floor (Channel 9, N-S) and at the roof of the building (Channel 2, N-S) are unprecedented at 0.80g and 1.71g, respectively. The largest velocity recorded at the roof was as large as 140cm/sec The 0.97W maximum base

Tabel 1. Response summary for the selected instrumented buildings.

Response parameter	Dir.	Burbank 10 story	Burbank 6 story	Los Angeles 3 story	North Hollywood 20 story	Sherman Oaks 13 story	Sylmar County Hospital
Base shear (% total weight)	N-S	33.63	12.37	48.73	10.61	18.70	96.89
	E-W	20.39	22.07	42.62	5.77	7.57	53.76
Overturning moment (% total weight x feet)	N-S	1518	546	890	1320	1304	3646
	E-W	804	807	953	613	771	1786
Roof lateral displacement relative to the base (cm)*	N-S	6.19 (0.0023)	9.63 (0.0038)	5.65 (0.0111)	21.12 (0.0036)	24.10 (0.0048)	6.31 (0.0022)
	E-W	2.76 (0.0010)	9.68 (0.0039)	4.12 (0.0081)	10.63 (0.0018)	33.42 (0.0067)	2.12 (0.0007)

* Overal drift index values are shown in parantheses.
Note: 1 feet = 30.5cm

shear apparently experienced by the building in the N-S direction is several times larger than any value generally used in engineering practice. Considering the severity of the motion the building experienced the observed overall drift indices are surprisingly low. The maximum response times at which the forces and displacements occurred were relatively early compared to those for the other buildings, thus distinguishing the near-field effect or the "fling" of the ground motion at the site.

2. DEMAND INTERPRETATIONS OF CONTEMPORARY CODES AND GUIDELINES

The lateral force demand for mechanical, electrical, and plumbing equipment are evaluated using three different code specifications, namely UBC-97, NEHRP-97, and FEMA-273. The methods for calculation of these design forces are described below.

2.1. UBC-97

Section 1632 of the UBC-97 addresses the design force requirement for the "Lateral force on elements of structures, nonstructural components and equipment supported by structures" [3]. The total design lateral seismic force F_p is given by

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right) W_p \tag{1}$$

with a minimum F_p of 0.7 $C_a I_p W_p$ and need not be greater than $4 C_a I_p W_p$. Where h_x is the element or component attachment elevation with respect to grade, h_r is the structure roof elevation with respect to grade, W_p the weight of the equipment, C_a the seismic coefficient in Table 16-Q (UBC-97), and a_p and R_p factors given in Table 16-O (UBC-97).

The seismic design spectral acceleration for all buildings considered are based on a soil type of S_D, an I_p of 1.0, from Table 16-K (UBC-97), and seismic source type B with closest distance to known seismic source greater than 5km. Using these assumptions, the design spectral accelerations at the roof and the ground are

$$\text{Roof :}\quad \frac{F_p}{W_p} = 0.44\,(1)\,(1+3)\left(\frac{a_p}{R_p}\right) = 1.76\left(\frac{a_p}{R_p}\right) \tag{2}$$

$$\text{Ground :}\quad \frac{F_p}{W_p} = 0.44\,(1)\,(1+0)\left(\frac{a_p}{R_p}\right) = 0.44\left(\frac{a_p}{R_p}\right) \tag{3}$$

2.2. NEHRP-97

The total design lateral seismic force F_p is given by

$$F_p = \frac{0.4\, a_p\, S_{DS}\, W_p\, I_p}{R_p}\left(l + 2\frac{x}{h}\right) \tag{4}$$

with a minimum F_p of $0.3 S_{DS}\, I_p\, W_p$ and need not be more than $1.6 S_{DS}\, I_p\, W_p$ [2]. Where x is the height in structure of highest point of attachment of component, h is the average roof height of structure relative to grade elevation, S_{DS} is the design earthquake spectral response acceleration at short periods given by $S_{DS} = 2/3\, S_{MS}$. S_{MS} is the maximum considered earthquake spectral response acceleration for short periods given by

$$S_{MS} = F_a\, S_S \tag{5}$$

where S_S is the mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods given in Figure 1 and F_a obtained from Table 4.1.2.4a (NEHRP-97). For the buildings considered (Los Angeles) with approximate Latitude=34^0, and Longitude =118^0, S_S=2.4 giving an F_a equal to 1.0. The corresponding design spectral accelerations at the roof and the ground based on NEHRP-97 are

$$\text{Roof :}\quad \frac{F_p}{W_p} = 0.4(1.6)(1+2)\left(\frac{a_p}{R_p}\right) = 1.92\left(\frac{a_p}{R_p}\right) \tag{6}$$

$$\text{Ground :}\quad \frac{F_p}{W_p} = 0.4(1.6)(1+0)\left(\frac{a_p}{R_p}\right) = 0.64\left(\frac{a_p}{R_p}\right) \tag{7}$$

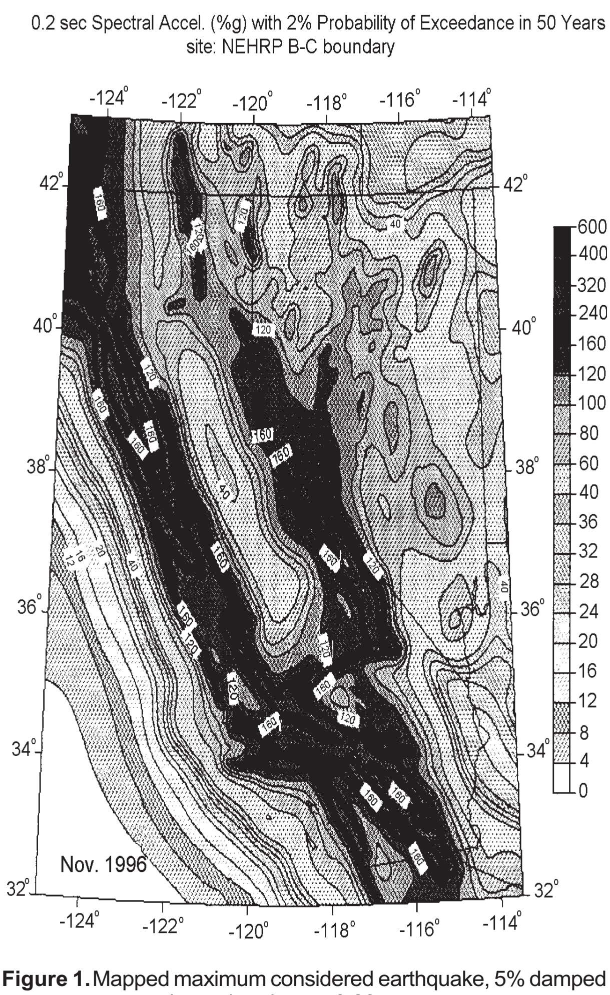

Figure 1. Mapped maximum considered earthquake, 5% damped spectral acceleration at 0.20sec.

with a minimum and maximum $F_p / W_p = 0.48$, and 2.56 respectively.

2.3. FEMA-273

The total design lateral seismic force F_p is given by

$$F_p = \frac{a_p C_a W_p}{R_p}\left(1 + 3\frac{x}{h}\right) \tag{8}$$

with a minimum F_p of $0.6\,C_a W_p$ and need not be more than $4\,C_a W_p$. Where x is the elevation of structure of component relative grade, h is the average roof elevation of structure relative to grade, C_a is the seismic coefficient at grade, equal to $\frac{S_{DS}}{2.5}$. S_{DS} is the Design (BSE-1 or BSE-2) spectral response acceleration at short periods for 5% damping, and is calculated as

$$S_{DS} = F_a S_S \tag{9}$$

where S_S is the spectral response acceleration at short periods, obtained from the response acceleration map given in Figure 2. From Table 2-13 (FEMA-273 for $S_S = 1.6$ (Los Angeles area Latitude $= 34^0$, Longitude $= 118^0$) the F_a in equation 9 is unity. The corresponding design spectral accelerations at the roof and ground are

$$\text{Roof}: \quad \frac{F_p}{W_p} = \frac{1.6}{2.5}(1+3)\left(\frac{a_p}{R_p}\right) = 2.56\left(\frac{a_p}{R_p}\right) \tag{10}$$

$$\text{Ground}: \quad \frac{F_p}{W_p} = \frac{1.6}{2.5}(1+0)\left(\frac{a_p}{R_p}\right) = 0.64\left(\frac{a_p}{R_p}\right) \tag{11}$$

with a minimum and maximum $F_p / W_p = 0.384$, and 2.56 respectively.

3. PERFORMANCE OF NONSTRUCTURAL SYSTEMS AND COMPONENTS

The damage to nonstructural components and equipment for the buildings considered in this study have been documented in the SMIP Interactive Information System [4]. Minor damage to the attachments of the equipment at the roof was observed for the Burbank 10 Story building (see Figure 3). The tearing of a small water pipe at the penthouse of the Burbank 6 story building resulted in flooding of the building and content damage (see Figure 4a). The anchorage of one roof equipment also failed in this building (see Figure 4b). No apparent sign of structural damage was observed at these three buildings.

At the Los Angeles 3 story commercial building, despite the large base shears and drifts, and a recorded roof acceleration of almost 100%g, no sign of structural damage was observed, and no damage was observed for the roof mounted equipment (see Figure 5). However, heavy content damage and some nonstructural damage to the hung ceilings, lights, and flooring were noticed.

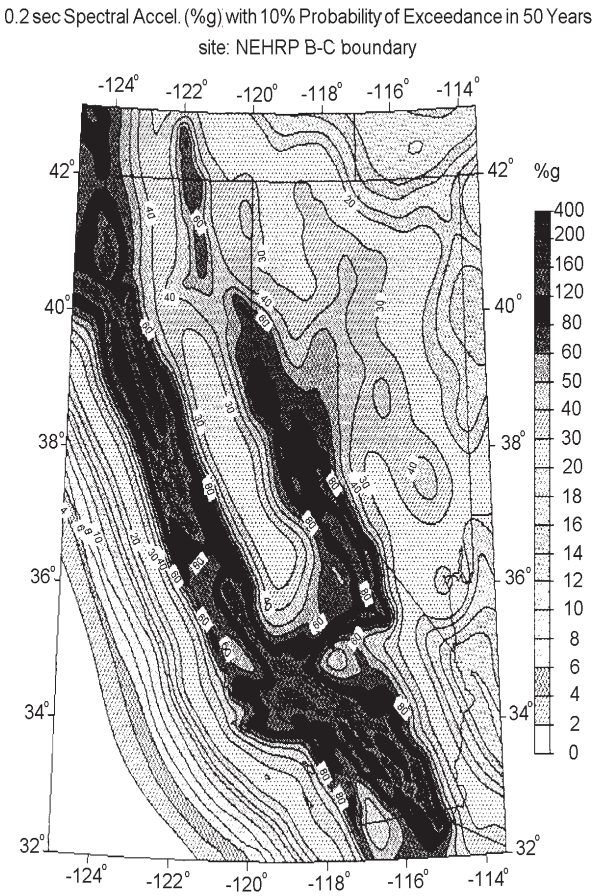

Figure 2. Mapped design basis earthquake, 5% damped spectral acceleration at 0.20sec.

Figure 3. Examples of nonstructural damage at the roof of 10 story Burbank Bldg.

Cont'd Figure 3.

The 20 story North Hollywood building experienced heavy nonstructural and content damage (see Figure 6). No sign of significant structural damage, however, was observed. Nonstructural damage varied from damage to partitions, doors, bathroom fixtures and tiles, to damage to chandeliers. Six to eight glass panels were broken. Cracks

(a) (b)

Figure 4. Examples of nonstructural damage at the roof of 6 story Burbank Bldg.

Figure 5. Examples of nonstructural component performance (LA 3 story Commercial Bldg.).

Figure 6. Examples of nonstructural component performance (North Hollywood 20 story Hotel).

were clearly visible on the sidewalk slabs on grade near the entrance of the building. Some oil spillage occurred at the basement equipment room. Aside from that, damage to mechanical equipment was minimal.

The 13 story Sherman Oaks building experienced noticeable but repairable structural damage in the form of cracks in the beams, slabs, girders, and walls (see Figure 7a). In contrast, no mechanical equipment damage was observed either at the roof or the basement (see Figures 7b, 7c, and 7d). This was due to proper mounting and anchorage details.

The structural system for 6 story Sylmar County Hospital experienced negligible damage, considering the magnitude of the spectral acceleration. Post earthquake survey of some of the steel plate welds showed signs of minor cracking. It is not clear, however, if these cracks were caused by the Northridge earthquake. The content damage was widespread and very significant as represented by photos of Figure 8.

A summary of the damage to the nonstructural components for the six buildings considered are given in Table 2. This table also gives the correspondings a_p and R_p factors for the considered codes and guidelines. The design values are compared to the observed spectral acceleration for these buildings in Figures 9 and 10 for the roof and ground levels, respectively.

4. CONCLUSION

Seismic force demands experienced at the roof and the base of several of the buildings considered in this paper exceeded the design force levels recommended by various

(a) (b)

Figure 7. Contrasting structural and nonstructural performance (13 story Sherman Oaks Bldg).

(c) (d)

Cont'd Figure 7.

Figure 8. Examples of content damage at the Sylmar County Hospital.

codes and guidelines (see Figures 9 and 10). This was most pronounced at vibration periods between 0.3 to 0.7 seconds where most of the ground motion input energy seems to have been concentrated.

Content damage seems to correlate with accelerations for the Sylmar hospital building as well as the three story commercial building. They do not, however, correlate very well with what was observed at the 20 story North Hollywood hotel. In this case, content damage was paramount while demand was not overwhelming.

Table 2. Summary of observed nonstructural damage and design coefficients.

Building	Component Description	Observed Damage	Design Coefficients		
			UBC-97	NEHRP-97	FEMA-273
			$\frac{a_p}{R_p}$	$\frac{a_p}{R_p}$	$\frac{a_p}{R_p}$
Burbank 10 Story Residential	Equipment vibration isolated at roof	Shearing of bolts	2.5/3	2.5/2.5	2.5/3
	Ceiling panels	Minor	1/3	1/2.5	1/1.5
	Equipment at ground	None	2.5/3	1/2.5	1/3
	Equipment non-vibration isolated	None	2.5/3	1/2.5	1/3
Burbank, 6 Story Commercial	Equipment vibration isolated	Failure of flexible mount	2.5/3	2.5/2.5	2.5/3
	Piping	Damaged	1/3	1/2.5	2.5/4
Los Angeles, 3 Story Commercial	Equipment vibration isolated	Twisting of bolts, concrete spalling below anchors	2.5/3	2.5/2.5	2.5/3
	Piping	Movement caused ceiling damage	1/3	1.0/2.5	2.5/4
North Hollywood, 20 Story Hotel	Chandeliers	Damaged in ballroom	1/3	1/2.5	1/1.5
	Sprinkles	Damage to hung ceiling	1/3	1/2.5	2.5/4
	Piping at ground	Ruptured	1/3	1/2.5	2.5/4
	False ceilings	Damaged	1/3	1/2.5	1/1.5
Sherman Oaks, 13 Story Commercial	Mechanical equipment at basement	No damage	2.5/3	1/2.5	1/3
	Piping	No damage	1/3	1/2.5	2.5/4
	Roof installed equipment	Little or no damage	2.5/3	1/2.5	1/3
Sylmar, 6 Story County Hospital	Ceiling	Damaged	1/3	1/2.5	1/1.5
	Storage racks	Damaged	2.5/4	1/2.5	2.5/4
	Book stacks	Damaged	1/3	1/2.5	1/3
	Ductwork	Damaged	1/3	1/2.5	1/3

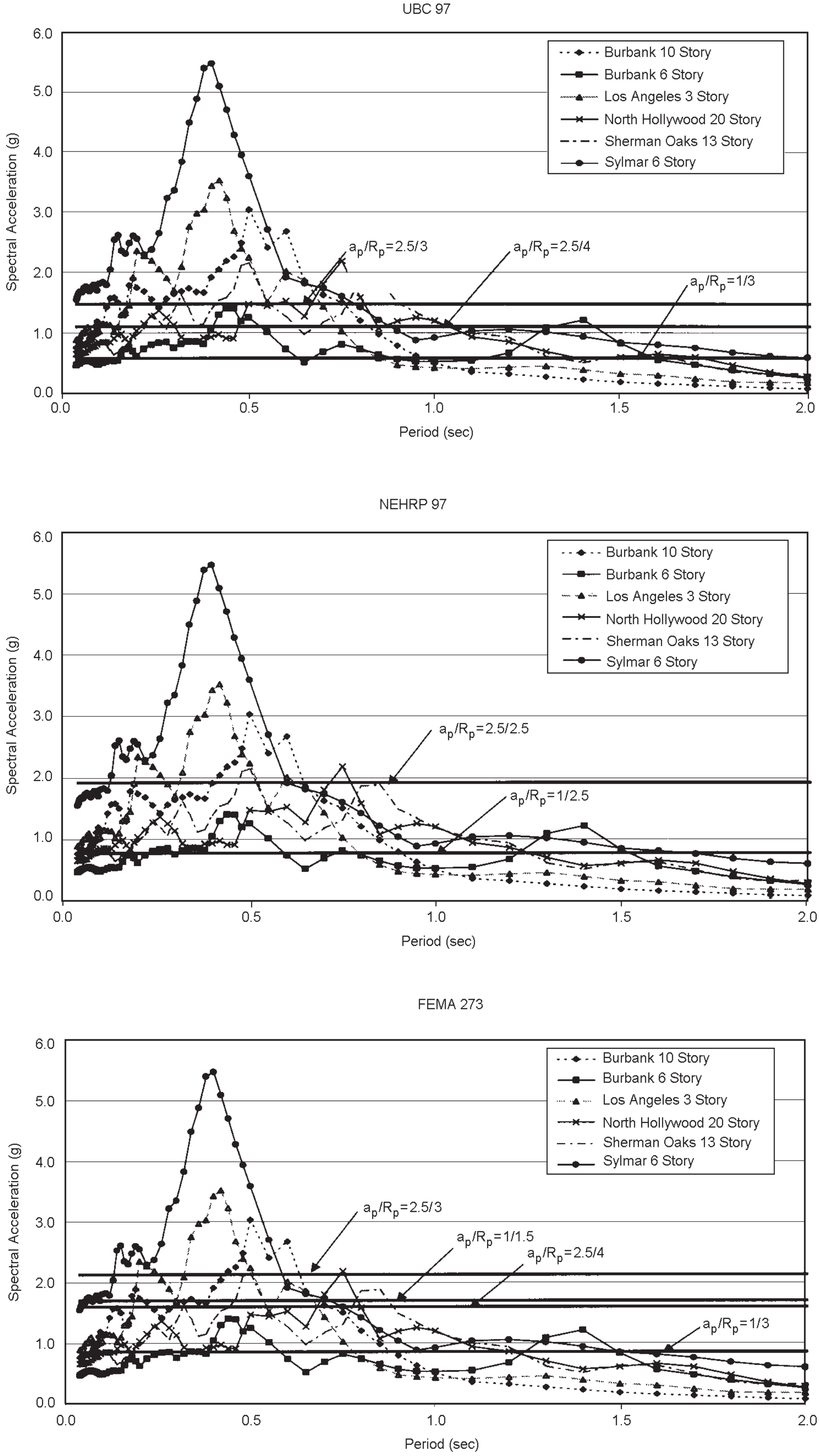

Figure 9. 5% damped spectral accelerations at roof.

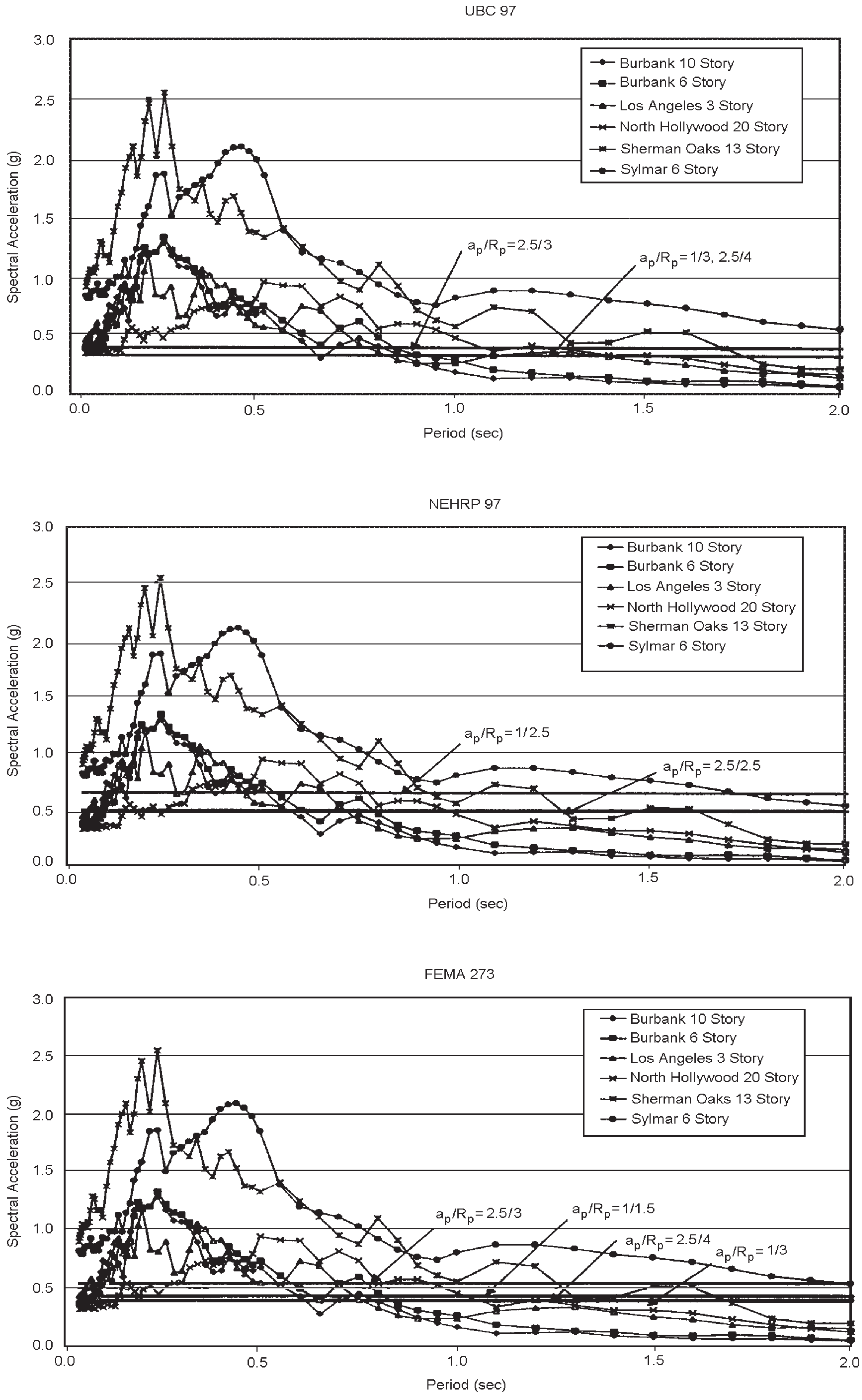

Figure 10. Ground level 5% damped spectral accelerations.

It is no surprise that minor equipment anchorage failure occurred at the roof of the 10 story Burbank building, nor is it a surprise that Sylmar hospital suffered major nonstructural damage. The surprise, however, is that the equipment supports for the Los Angeles 3 story building and the 13 story Sherman Oaks building did remarkably well given the large force demands they were subjected to.

It appears that more work is needed to establish a clear understanding of the relationship between demand and performance for nonstructural systems and components. As has been the case with the structural components, desired performance may have more to do with the attention to detail and workmanship than it has to do with the numerics of design.

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