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WHY PROPERLY CODE DESIGNED AND CONSTRUCTED BUILDINGS HAVE SURVIVED MAJOR EARTHQUAKES

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SUMMARY

During the past 5 years, major changes have been made to the earthquake provisions of building codes in the United States. It is the opinion of some that many of these changes, which tend to be more restrictive, were not necessary, and in fact, have led to unintended consequences. It has been observed that properly designed and detailed pre-1997 buildings have been able to perform without significant damage when subjected to earthquakes that generated ground motions that substantially exceeded those used as the design basis for the building code. Post-earthquake investigations tend to focus on the severely damaged buildings. This paper will focus on buildings that sustained little or no apparent damage and attempt to explain the reason for their good performance. The conclusions restate observations that have been made by others that buildings designed with attention to details and that have good quality of construction will generally do well. Making codes more restrictive and more complicated does not necessarily make for better buildings.

INTRODUCTION

Building Codes

Current seismic code provisions for the United States had their beginnings in the 1960s with the recommendations of the Structural Engineers Association of California (SEAOC) and their adoption into the Uniform Building Code (UBC). This work was supplemented with government sponsored development in the 1970s by the Applied Technology Council (ATC) and Federal Emergency Management Agency (FEMA-NEHRP). Prior to the 1960s, the seismic design provisions were primarily an appendix to the building code.

In the 1960s, major changes were made to the UBC [1] earthquake provisions on the basis of recommendations of SEAOC [2]. The lateral forces applied to a building were determined by the equation: $V = ZKCW$, where Z was the seismic zone factor ($Z = 1$ in the highest zone), K represented the structural system (ranged from 0.67 to 1.33), C was the seismic coefficient based on the fundamental period of the building, and W was the weight of the building. In the early 1970s, details in the seismic provisions were expanded, but no major changes were made to the lateral forces. In 1976, greatly influenced by the experience of the 1971 San Fernando earthquake in southern California, the UBC

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earthquake provisions were subjected to major revisions, and the number of seismic zones was increased from 3 to 4. The fourth zone identified areas within Zone 3 that were in proximity to certain major fault systems. A soil factor S was added to the lateral force coefficient to account for the effects of the soil profile on the seismicity of the site. S was calculated by an equation that was dependent on the ratio of the characteristic site period and the period of vibration of the building. An importance factor, I, was added to increase the forces for special occupancies. The lateral force equation expanded to V = ZIKCSW. Revisions were included to address ductility, drift and deformation compatibility. The 1988 edition of UBC again was subjected to major changes to the seismic provisions that included the seismic zonation, soil factors, the lateral force equations, irregularities, drift limitations and dynamic analysis. In 1997, soil classifications were further modified into categories S_A through S_F (i.e., classes A-F) and an additional factor N was added to account for near-field effects in zones of highest seismicity (i.e., seismic Zone 4), such as those in California. Additional revisions included controversial issues relating to drift limits, redundancy, overstrength factors and nonstructural provisions [3].

On the basis of the ATC 3-06 document developed in the 1970s [4], FEMA-NEHRP established guidelines for the development of new seismic provisions [5]. While some of the ATC 3-06 recommendations were incorporated into the UBC over the years, in the year 2000, the NEHRP provisions were adapted into a new building code called the IBC [6]. Although the IBC has been adopted by some states and regions, California has still maintained the 1997 UBC as its California Building Code (CBC).

As the seismic provisions have changed over the past 40 years, the design forces tend to increase and the detailed provisions tend to be more prescriptive. Although the new provisions may be economically feasible for new construction, they do tend to limit the structural engineers' flexibility of design and the ability to be innovative. But of greater importance are the effects of new provisions on existing buildings. Generally buildings have been considered safe and code compliant if they conformed to the code provisions being enforced at the time of construction. However, as codes become more severe and restrictive, there are some that believe that pre-existing buildings should be upgraded to meet the ever changing new standards.

Performance of Code Designed Buildings

This study is a follow-up to an earlier study [7] done by the author following the 1971 San Fernando earthquake [8]. In this current study, response spectra obtained from the 1994 Northridge earthquake recorded ground motions are compared with the capacities on code designed buildings. The procedure used to obtain correlation between performance of buildings and the demands earthquake ground motion is primarily the Capacity Spectrum Method (CSM) [9, 10] with back-up by inelastic nonlinear procedures.

THE 1994 NORTHRIDGE EARTHQUAKE AND THE UNIFORM BUILDING CODE

The Northridge Earthquake

The 1994 earthquake that affected the Los Angeles, California region was well recorded by strong motion instruments located in buildings as well as on the ground. Some of the recorded ground motions were unusually strong as can be illustrated by studying the resulting response spectra. For some locations spectral accelerations and displacements at 5% of critical damping were as high as four times those used as the basis for the Uniform Building Code (UBC). For this study, eight of the largest recorded ground motion stations were used to compare the provisions of the UBC.

The Uniform Building Code

In this paper, to illustrate the effects of building codes on how buildings perform, the seismic provisions of the 1973 through 1997 editions of UBC are used to establish equivalent strength design response spectra for seismic Zone 4 at a soil profile D site [1]. The response spectra were determined by adjusting the base shear coefficient, V/W, to represent a spectral acceleration at the design strength level for a strength reduction factor (ϕ) equal to unity. The structural system is representative of a basic system (i.e., K = 1 or R = 5.5). The response spectra, at an assumed damping of 5% of critical damping, representing pre-1976, 1976-1985, 1988-1994 and 1997 editions of the UBC are shown in Figure 1. They are plotted on an Earthquake Engineering Intensity Scale (EEIS) [11]. The UBC curves indicate that the design strengths are reached at roughly an EEIS VII for long period buildings and at an EEIS VI for moderate to short period buildings. These are based on pre-yield strength and do not represent ultimate capacities that would represent post yield excursions into inelastic response. In order to approximate performance for higher intensities, nonlinear inelastic procedures that take into account post yield capacities and actual earthquake demands are required.

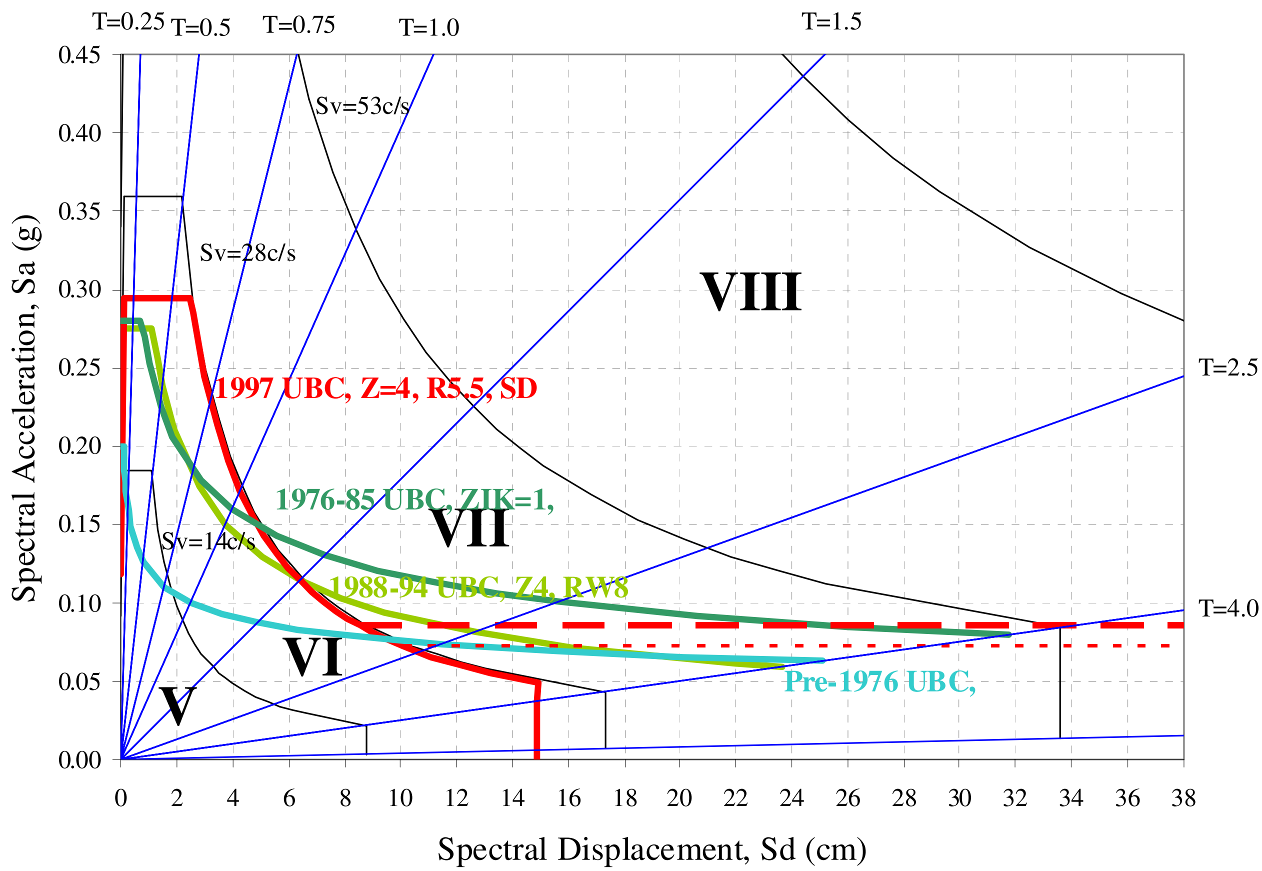

Figure 1. EEIS: UBC Zone 4, Soil D equivalents (design strength at ϕ = 1)

The Capacity Spectrum Method (CSM)

To approximate capacity curves for three sample buildings, adjustments are made to the design strengths discussed above. First, it is assumed that the effective yield strength of a building will be 1.25 times the design strength and that effective elastic period of the structure will be 1.3 times the design period. This accounts for some conservatism in the design code procedure. The inelastic portion of a bilinear curve is assumed to have a stiffness equal to 5-percent of the elastic stiffness. For this initial assumption for the capacity curve, a ductility of three is assumed (i.e. the inelastic excursion is twice the yield point displacement). The three sample buildings are assigned design periods of 0.3 sec, 0.8 sec and 1.7 sec, respectively. Thus, the effective elastic periods are 0.39 sec, 1.04 sec and 2.25 sec. The bilinear capacities are plotted in Figures 2 through 5 (green lines with open triangles). Figure 2 represents pre-1976 UBC editions (e.g. 1973 UBC), Figure 3 represents 1976 – 1985 editions, Figure 4 represents 1988 – 1994 editions, and Figure 5 represents the 1997 UBC. Seismic zone 4 and soil class D are assumed. The figures also show the unreduced 5% damped response spectrum for 1997 UBC Zone 4, soil class D (black line with open squares). In order to approximate reduced demands for inelastic response, higher damped response spectra are used to represent inelastic response spectra (e.g. 20% damping is used to estimate a displacement ductility of about 2.5 and 40% damping is used for a ductility of about 5) [9]. A target 20-percent damped demand spectrum (heavy black line with solid squares) is shown in Figures 2 through 5 to represent ductilities roughly equal to 2.5 to 3. In accordance with the CSM procedure, if the green bilinear capacities can cross the heavy black line, the structure will satisfy the initial assumptions for the sample buildings as stated above. From Figures 2 through 5, it can be seen that Sample #3 performs well for all four codes, and Sample #2 meets the criteria for all but the pre-1976. In all cases, the Sample #1 fails the test. However, this initial criterion is quite conservative, and it can generally be expected that a well designed, detailed, and constructed building will have better performance capabilities.

A revised criteria assumes the effective yield strength is 1.5 times the design strength and gives an outer limit ductility of six (i.e. $\mu = 6$). The revised capacity curves are summarized in Table 1 and shown in brown with solid markers designating ductilities 1 through 6. The observation that all the capacity curves, except the Sample #1 for pre-1976, cross the 20% damped curve indicate that the ductility demands for a zone 4, soil D, response spectrum are less than 6. In Figure 2, a 50% damped curve is shown to aid in understanding the CSM procedure. The 50% damped curve represents a ductility $\mu = 6$. The Sample #2 plot crosses the 50% curve at about $\mu = 2.5$ and crosses the 20% curve at about $\mu = 5$. The solution lies between 2.5 and 5, say at about $\mu = 3.5$. A damped curve at about 30 to 35% will cross between the ductility 3 and 4 markers. For Sample #1, the capacity curve barely meets the 50% damped curve, thus the structure would require a ductility equal to 6. Although the procedure is approximate, it does give a visual indication of how the structures might perform if subjected to an equivalent earthquake.

To summarize Figures 1 through 5, it appears that except for short period buildings designed before 1976, a well designed structure built in accordance with the UBC within the past 30 years should be able to perform satisfactorily (i.e. within reasonable ductility demands) for a design earthquake. This assumes there are no significant defects and that there is some built-in ductility and redundancy.

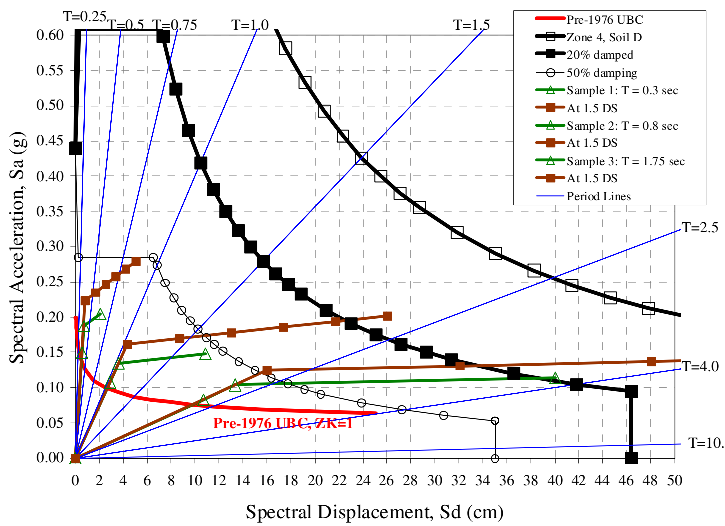

Figure 2. Pre-1976 UBC equivalent performance for Zone 4, Soil D

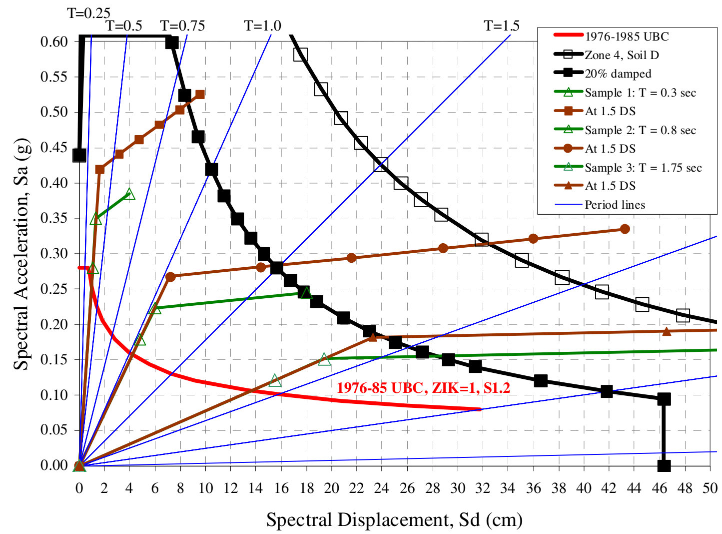

Figure 3. 1976-1985 UBC equivalent performance for Zone 4, Soil D

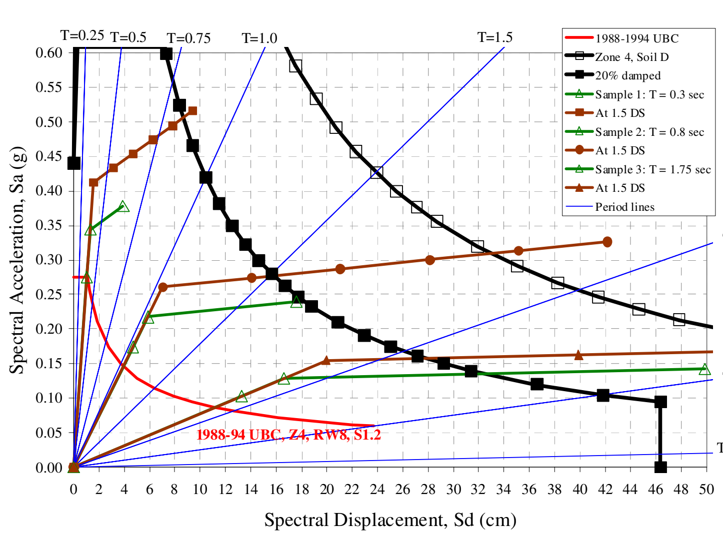

Figure 4. 1988-1994 UBC equivalent performance for Zone 4, Soil D

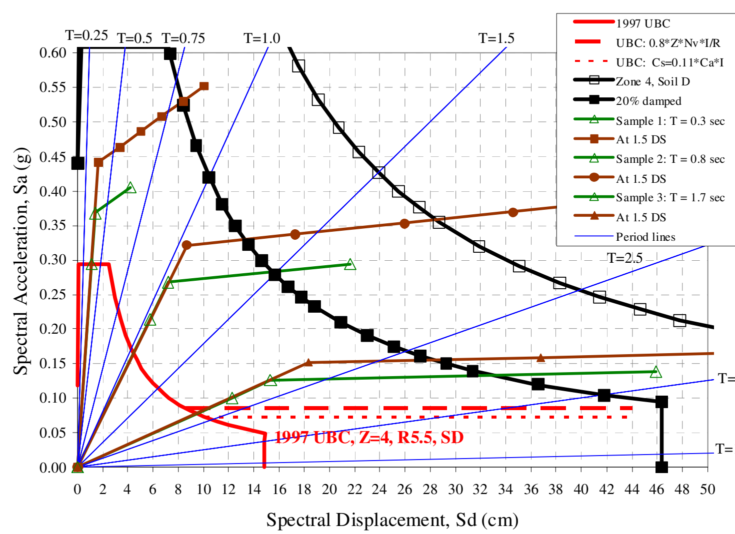

Figure 5. 1997 UBC equivalent performance for Zone 4, Soil D

Table 1. Sample bilinear capacities. UBC code design strength times 1.5 to equal yield, 5% damped, Tb = 1.3*Ta

	UBC Code	pre-1976	1976-1985	1988-1994	1997
Sample 1	Fy, kN	22.4	42.0	41.3	44.2
Ta=0.3 sec	Sdby-yield, cm	0.847	1.59	1.56	1.67
Tb = 0.390 sec	Saby, g	0.225	0.420	0.413	0.442
K1=26.4 kN/cm	Sdb6-mu=6, cm	5.08	9.53	9.36	10.03
K2=1.32 kN/cm	Sab6, g	0.280	0.525	0.516	0.552
Sample 2	Fy, kN	16.2	26.8	26.1	32.1
Ta=0.8	Sdby-yield, cm	4.35	7.2	7.03	8.65
Tb =1.04	Saby, g	0.162	0.268	0.261	0.321
K1=3.72	Sdb6-mu=6, cm	26.1	43.2	42.1	51.9
K2=0.186	Sab6, g	0.202	0.335	0.326	0.402
Sample 3	Fy, kN	12.4	18.1	15.5	15.1
Ta=1.75(1.70)	Sdby-yield, cm	16.03	23.3	19.9	18.4
Tb =2.28(2.21)	Saby, g	0.124	0.181	0.155	0.151
K1=0.777(.823)	Sdb6-mu=6, cm	96.2	140	120	110
K2=0.039(.041)	Sab6, g	0.156	0.226	0.194	0.189

W= 100kN

Northridge Earthquake Recordings

There is an abundance of ground motion recordings obtained during the 1994 Northridge earthquake. Eight locations, with some of the largest recorded ground motions, were selected for this study. The locations are summarized in Table 2 with approximate peak ground accelerations (PGA), velocities (PGV), and displacements (PGD) for each horizontal orthogonal direction. Figure 6a plots the 5% damped response spectra on the EEIS template [12]. The black lines represent the very high intensities (e.g. EEIS IX and X). The red lines represent high intensities (e.g. EEIS VIII plus) and green the moderate to high intensities (EEIS VII plus). The three brown lines radiating out from the origin represent the elastic periods of the sample designs. Figure 6b show the response spectra at 20% damping to represent a reduced demand for inelastic response. These spectra represent the demand for a target displacement ductility of 2.5 to 3 for bilinear capacity curves such as shown in Figures 2 through 5. To more clearly evaluate the response spectra in Figure 6a the ground motions have divided into three categories: very high intensities, high intensities, and moderate to high intensities (Figures 7, 8, and 9 respectively).

Figure 7 shows the 5% damped response spectra for Newhall, Rinaldi, Sylmar, and Tarzana station records. The heavy black lines delineate the EEIS zones. The blue line with open circles represents the unreduced response spectrum for UBC seismic zone 4 at a soil class D site (note that it is about coincident with the EEIS delineating VIII and IX). Twice the UBC spectrum is shown by a thin blue line with open squares (between IX and X). It can be seen that the Rinaldi (228) curve has a large bulge that exceeds twice UBC between periods of 0.75 sec and 1.5 sec. In the short period range, Tarzana (90) greatly exceeds twice UBC. On the basis of this alone, one might expect widespread damage at these period ranges. However, this does not appear to have occurred. Can we explain it? That is the question. The rest of the data points lie in EEIS IX, which exceed the UBC response spectrum. Much greater damage than appears to have been reported would be expected. This paper attempts to address these issues.

Table 2. List of 1994 Northridge earthquake ground motion records

EEIS	Location	Direction	PGA	PGV	PGD
		degrees	g	cm/sec	cm
Very High EEIS IX - X	**Newhall, Fire Station**	0	0.68	97	19.8
		90	0.66	81	35.6
	Rinaldi	228	0.53	99	18.5
		318	0.58	99	25.4
	Sylmar, Hospial	0	0.75	114	25.4
		90	0.38	71	19.1
	Tarzana, Cedar Hill Nursery	90	1.78	114	27.9
		0	0.99	79	33.0
High EEIS VIII plus	**Sherman Oaks**	0	0.87	60	20.1
		90	0.37	30	8.6
	Van Nuys, 7-Story Hotel	0	0.42	36	18.5
		270	0.45	51	20.3
Moderate to High EEIS VII plus	**Arleta**	90	0.34		
		0	0.31		
	Hollywood Storage Building	90	0.21	18	7.4
		0	0.39	22	9.4

Also shown on Figure 7 are three radial brown lines that represent the elastic periods of the three sample buildings discussed earlier (Table 1). It has generally been accepted (although there has been some debate on this) that a rough estimate of the inelastic displacement, on the average, can be approximated by the intersection of the elastic period with elastic response spectra. This idea, known as the equal displacement rule, is generally attributed to research by Nathan Newmark [13]. One rationale to explain this is as follows: inelastic action is generally expected to reduce the response due to energy absorption and the nonlinearity that interferes with resonance. One the other hand, stiffness reduction due to inelastic action tends increase the displacement. Thus, on average, the reduction may be offset by the increase. Using the equal displacement rule (ED), the intersection of the radial brown lines with the response spectra on Figures 7, 8, and 9 are shown on Tables 3a and 3b under the columns “ED”. Note the results are the same for pre-1976 and 1997 designs because the results are not dependent on the strength. These results are shown for comparisons with other methods as discussed later.

Figure 8 shows response spectra for Sherman Oaks and Van Nuys station records. Although spikes of Sherman Oaks (00) extend into EEIS IX, the spectra stay mostly in the EEIS range. Damage was observed at these locations (e.g. the Holiday Inn at Van Nuys) [14], but not as much as some might expect considering the size of the response spectra.

Figure 9 covers Arleta and the Hollywood Storage building stations. Except for excursions into EEIS VIII, these spectra generally lie in the EEIS VII range.

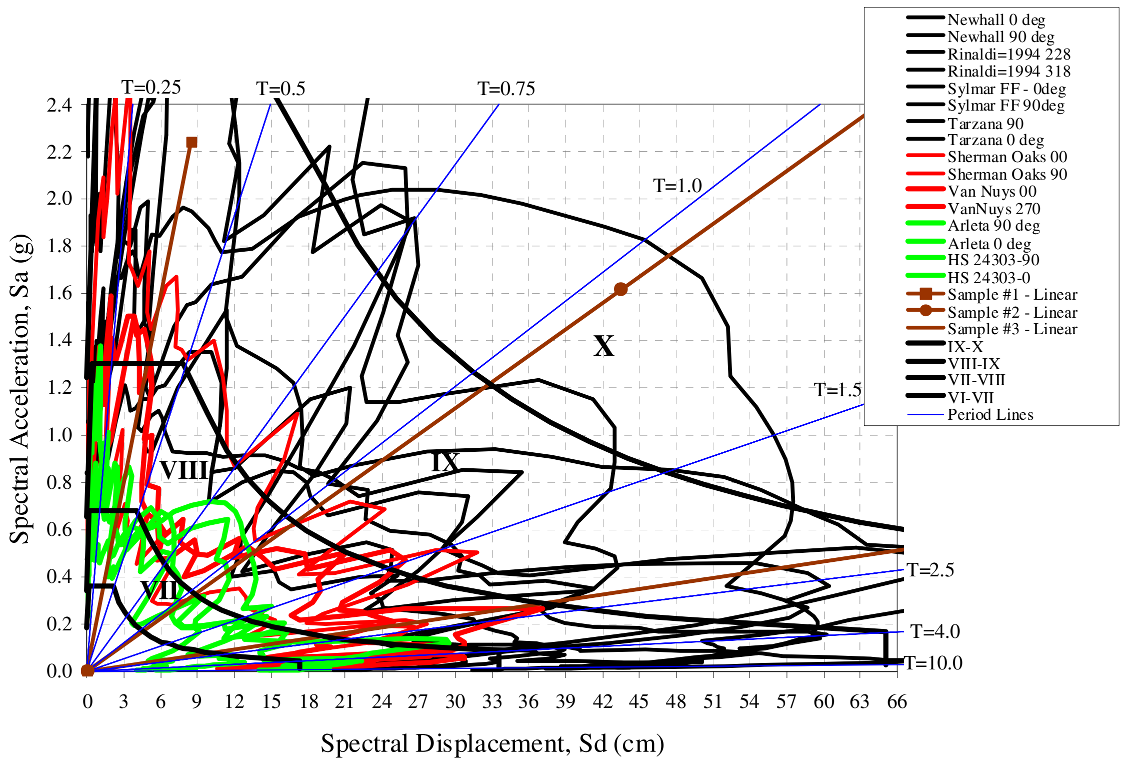

Figure 6a. EEIS & Northridge 1994 earthquake (5% damped)

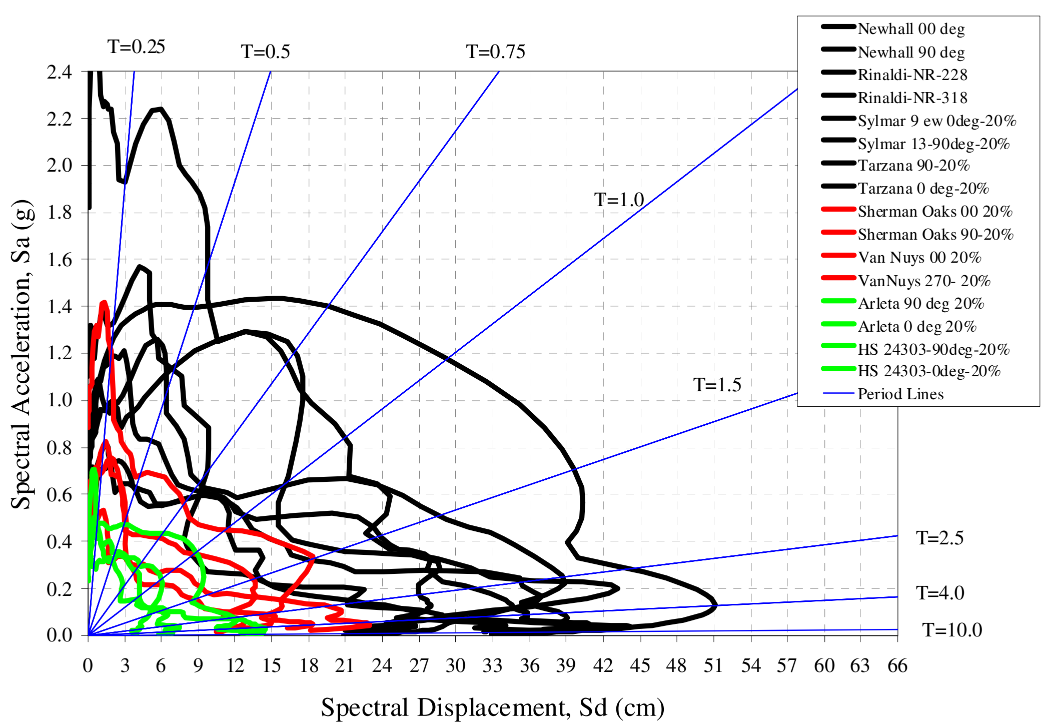

Figure 6b. Northridge 1994 earthquake (20% damped)

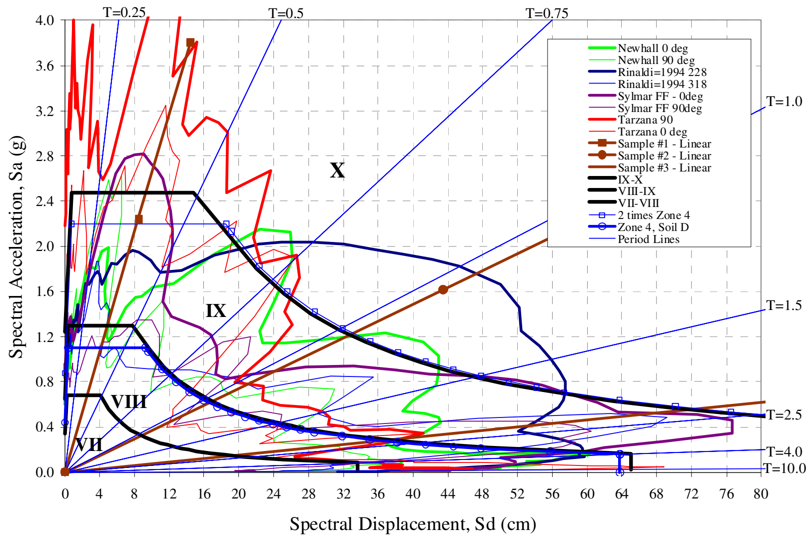

Figure 7. Very high intensity (IX & X), Northridge 1994, 5% damped

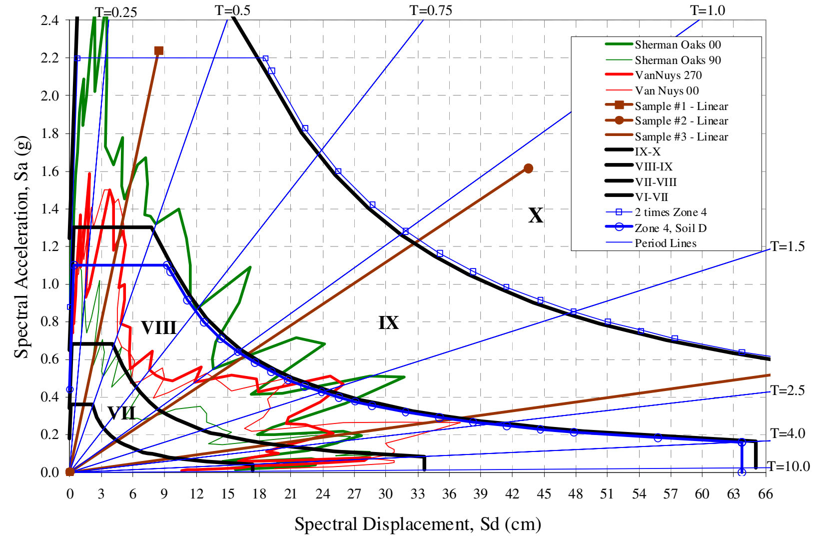

Figure 8. High intensity (VIII & IX), Northridge 1994, 5% damped

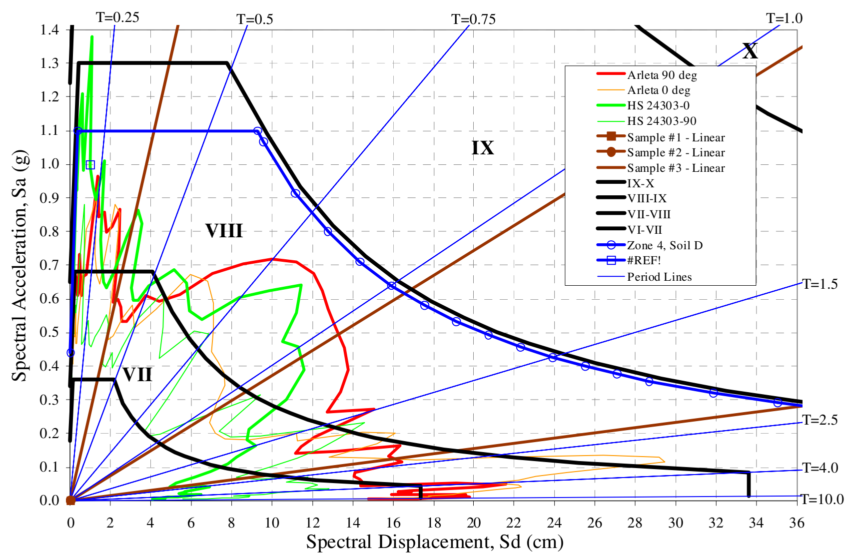

Figure 9. Moderate-high intensity (VII & VIII), Northridge 1994 EQ, 5% damped

Table 3a. Results for pre-1976 UBC design

	Pre-1976 Design												
			Sample #1				Sample #2				Sample #3		
Intensity	Location	Direction	ED	CSM	IRS	T-H	ED	CSM	IRS	T-H	ED	CSM	IRS
			cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm
Very High	**Newhall**	0	5	7		11	32	20		20	38		
		90	5	6		13	18	14		25	30		
	Rinaldi	228	8	>>6		16	47	>26		20	53		
		318	5	>6		12	21	23			65		
	Sylmar	0	10	8		9	24	23			65		
		90	4	6		9	14	23			56		
	Tarzana	90	13	9	18	13	21	21	25		38	27	32
		360	12	6	13	9	12	15	13		40	21	31
High	**Sherman Oaks**	0	6	5		7	17	12			26		
		90	2	3		2	9	9			24		
	Van Nuys	0	5	3	5	5	13	11	9		33	25	21
		270	5	5		5	13	10			23		
Moderate to High	**Arleta**	90	2	5			13	10			16		
		0	2	3			7	6			16		
	Hollywood Storage	0	4	3		2	7	8			10		
		90	2	2		2	11	5			6		
	CSM preliminary estimate		dy=0.85 cm				dy=4.35 cm				dy=16.0 cm		
			mu=3, d=2.5cm				mu=3, d=13cm				mu=3, d=48cm		
			mu=6, d=5.0cm				mu=6, d=26cm				mu=4, d=64cm		

Table 3b. Results for 1997 UBC design

	1997 Design												
			Sample #1				Sample #2				Sample #3		
Intensity	Location	Direction	ED	CSM	IRS	T-H	ED	CSM	IRS	T-H	ED	CSM	IRS
			cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm
Very High	**Newhall**	0	5				32				38		
		90	5				18				30		
	Rinaldi	228	8				47				53		
		318	5				21				65		
	Sylmar	0	10				24				65		
		90	4				14				56		
	Tarzana	90	13	10	16.0		21	18	20		38	28	28
		360	12	7	10.0		12	12	12		40	24	23
High	**Sherman Oaks**	0	6				17				26		
		90	2				9				24		
	Van Nuys	0	5	3	4.5		13	13	13		33	26	24
		270	5				13				23		
Moderate to High	**Arleta**	90	2				13				16		
		0	2				7				16		
	Hollywood Storage	90	3.5				7				10		
		0	2				11				6		
	CSM preliminary estimate		dy=1.67 cm				dy=8.65 cm				dy=18.4 cm		
			mu=3, d=5cm				mu=3, d=26cm				mu=3, d=55cm		
			mu=6, d=10.0cm				mu=6, d=52cm				mu=4, d=74cm		

INELASTIC ANALYSIS AND DUCTILITY DEMANDS

Inelastic Analysis Methods

As stated earlier, the capacities of three sample buildings were described by bilinear curves representing four UBC code editions. The CSM was used using two procedures: equating damped response spectra to ductility [9] and constructing constant ductility response spectra [15, 16].

Figures 10a and 10b show inelastic response spectra for Van Nuys (00) that were developed using the BISPEC program [16]. The family of curves includes ductilities 1 through 6 (ductility $\mu = 1$ is the elastic response spectra at 5% damping). On Figure 10a is superimposed the pre-1976 UBC capacities and on Figure 10b is superimposed the 1997 UBC capacities. Figures 10c and 10d parallel 10a and 10b, except damped response spectra are used in lieu of inelastic spectra. The 5% damped spectrum represents a ductility of 1; 10% approximates a ductility of 1.5; 20% is 2.5; 40% is 5; and 60% is roughly 7. The capacity curves have markers at ductilities 1 through 6. The performance is estimated by matching ductility demand with capacity. This can be done visually by using the graphics. As a check, a best-guess ductility demand curve can be plotted to see if it crosses the capacity curve at the same ductility (refer to earlier discussion). Results for the inelastic response spectra (IRS) for Van Nuys and Tarzana are shown in Tables 3a and 3b. Results for the damped spectra are shown under the columns labeled CSM. BISPEC was also used to determine maximum displacements by nonlinear time histories (T-H). The ductility demands can be calculated by dividing the maximum displacements by the yield displacements (dy).

Figure 11 shows inelastic spectra and damped spectra for Tarzana (90) and parallels those in Figure 10. In Figure 10, it can be seen that structures that have the capacity to behave in a ductile fashion can survive the demands of the Van Nuys ground motion, even those designed prior to 1976. The Holiday Inn 7-story building makes an interesting study [14]. However, for Tarzana the demands are quite excessive. Yet, reported damage does not appear to be excessive.

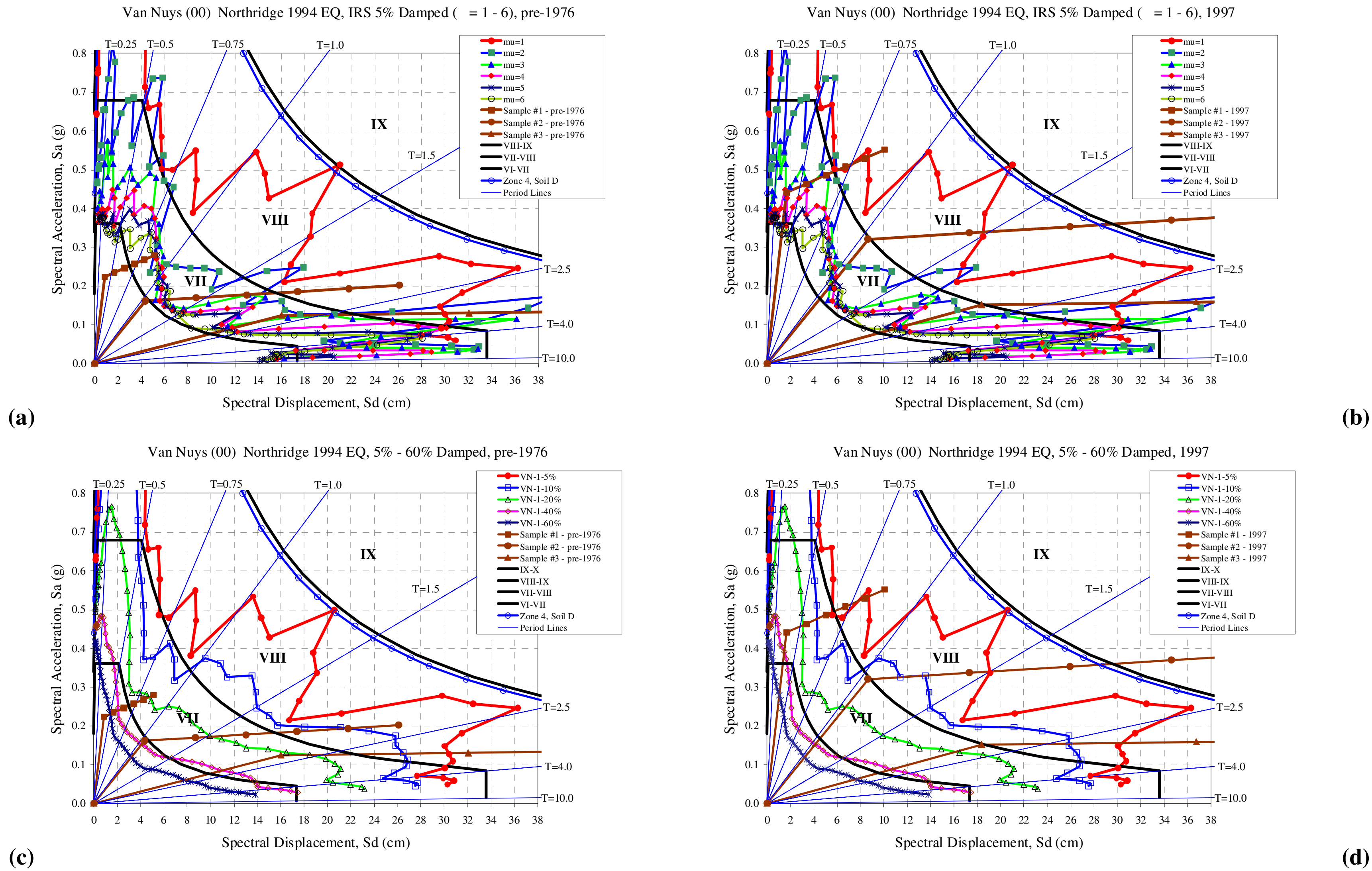

Figure 10. Van Nuys (00), Northridge 1994 EQ, IRS 5% damped (μ=1-6): (a) pre-1976 UBC; (b) 1997 UBC; Van Nuys (00), Northridge 1994 EQ, 5%-60% damped: (c) pre-1976 UBC; (d) 1997 UBC

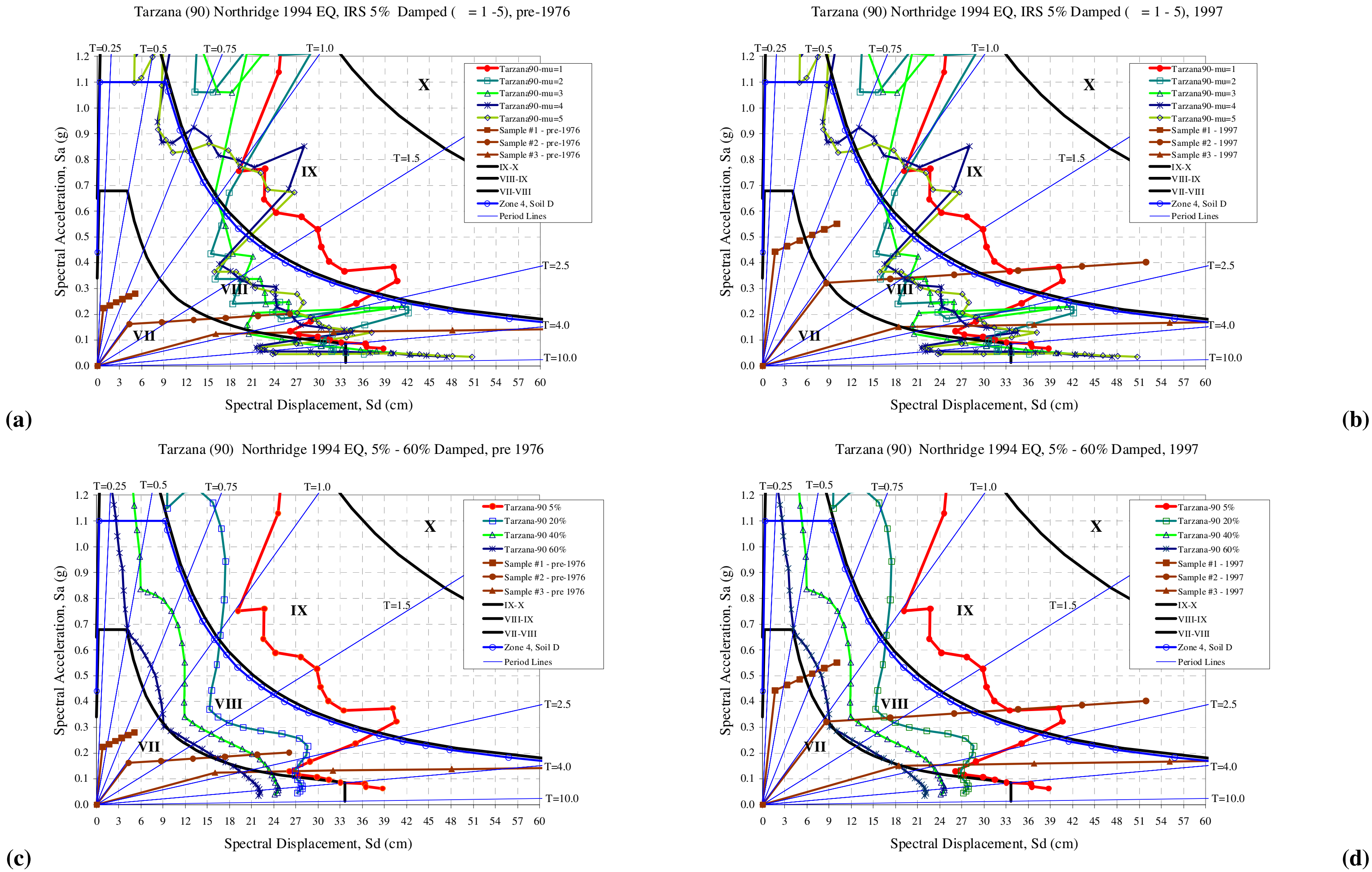

Figure 11. Tarzana (90), Northridge 1994 EQ, IRS 5% damped (μ=1-5): (a) pre-1976 UBC; (b) 1997 UBC; Tarzana (90), Northridge 1994 EQ, 5%-60% damped: (c) pre-1976 UBC; (d) 1997 UBC

CONCLUSIONS AND ACKNOWLEDGEMENTS

Conclusions

There are several explanations that one may explore to explain why so many buildings survived with relatively little or no damage. For example, buildings are better than we expect because of conservatism in design. Engineers often ignore structural elements not part of the designated lateral force system. Excess capacity in gravity load design is often available to resist lateral forces [17]. Also, an engineer with vision and experience can anticipate potential weak links and provide alternative load paths. Increasing forces prescribing strict limits can not substitute for good design practices.

Another explanation is that the recorded motions may not be really representative of the actual ground motion. After all, instruments are sitting on something, and maybe that something is responding in a manner different than the actual ground. The analytical methods used are generally precise, but are they accurate? Materials can have many variables that are not accounted for in the analytical models.

In this brief study the relationships between building response and earthquake recorded ground motions are compared. Although there are explanations why some buildings, even those built prior to 1976, perform much better than might be expected, there still appear to be some mysteries about buildings surviving very strong ground motions such as those recorded in Tarzana and Rinaldi during the 1994 Northridge earthquake. Continued studies and interaction between engineers, as well as with other design professionals, is required. Engineers need to think more about how buildings perform than how to satisfy building codes [18].

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REFERENCES

1. ICBO, et al. "Uniform Building Code (UBC)," by International Conference of Building Officials (ICBO), Whittier, California; 1961-1997.
2. SEAOC, et al. "Recommended Lateral Force Requirements and Commentary (Blue Book), Seismology Committee, Structural Engineers Association of California, Sacramento, California; 1958 through 1999.
3. Searer GR, Freeman SA. "Unintended consequences of code modification." Proceedings of the 7th U.S. National Conference on Earthquake Engineering, Boston, Massachusetts. Oakland, CA: Earthquake Engineering Research Institute, 2002.
4. ATC. "Tentative provisions for the development of seismic regulations for buildings." A Cooperative Effort with the Design Professions, Building Code Interests, and the Research Community, Prepared by Applied Technology Council associated with the Structural Engineers Association of California (ATC 3-06), National Bureau of Standards Special Publication 510. June 1978.
5. NEHRP, et al. "Recommended provisions for the development of seismic regulations for new buildings." 1985, 1991, 1994, and 1997 Editions, Part 1 Provisions, (FEMA 95, 222, 222A, and 302, respectively). Prepared by the Building Seismic Safety Council, for the Federal Emergency Management Agency, Washington, D.C.: 1985, 1991, 1994, 1997.
6. ICC, "International Building Code," Copyright© 2000 by International Code Council, Inc. (ICC), published in cooperation with Building Officials and Code Administrators International, Inc.

(BOCA), International Conference of Building Officials (ICBO), and Southern Building Code Congress International, Inc. (SBCCI). March 2000.

7. ATC. "An investigation of the correlation between earthquake ground motion and building performance - ATC 10." Applied Technology Council, CA: 1982.
8. Freeman SA. "On the correlation of code forces to earthquake demands." Proceedings of the 4th U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices, ATC 15-3, Applied Technology Council, Kailua-Kona, Hawaii, 1992.
9. Freeman SA. "Development and use of Capacity Spectrum Method". Proceedings of the 6th U.S. National Conference on Earthquake Engineering, Seattle, Washington. Oakland. Paper no. 269. Oakland, CA: Earthquake Engineering Research Institute, 1998.
10. Mahaney JA, Paret TF, Kehoe BE, Freeman SA. "The Capacity Spectrum Method for evaluating structural response during the Loma Prieta earthquake." Proceedings of the 1993 National Earthquake Conference Earthquake Hazard Reduction in the Central and Eastern United States: A Time for Examination and Action. Oakland, CA: Earthquake Engineering Research Institute, 1993: 501-510.
11. Freeman SA, Paret TF, Irfanoglu A. "Structural implications of the TriNet Instrumental Intensity Scale." Proceedings of the 7th U.S. National Conference on Earthquake Engineering, Boston, Massachusetts. Oakland, CA: Earthquake Engineering Research Institute, 2002.
12. Freeman SA, Irfanoglu A, Paret TF. "Earthquake Engineering Intensity Scale: A template with many uses." Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada. Paper no. 1667. 2004.
13. Newmark NM, Hall WJ. "Earthquake spectra and design." Oakland, CA: Earthquake Engineering Research Institute, 1982.
14. Gilmartin UM, Freeman SA, Rihal SS. "Using earthquake strong motion records to assess the structural and nonstructural response of the 7-Story Van Nuys Hotel to the Northridge earthquake of January 17, 1994." Proceedings of the 6th U.S. National Conference on Earthquake Engineering, Seattle, Washington. Oakland. Paper no. 268. Oakland, CA: Earthquake Engineering Research Institute, 1998.
15. Fajfar P. "Capacity Spectrum Method based on inelastic demand spectra." IKPIR Report EE - 3/98, Ljubljana, Slovenia; September 1998.
16. Hachem MM. "BISPEC: Interactive software for the computation of unidirectional and bidirectional nonlinear earthquake spectra." (Submitted for) Structures Congress 2004, SEI, American Society of Civil Engineers, Nashville, 2004.
17. Freeman SA. "Code designed steel frame performance characteristics." Proceedings of Dynamics of Structures, Structures Congress '87, Structural Division, American Society of Civil Engineers, Orlando, Florida. 1987: 383-396.
18. Freeman SA, Paret TF, Searer GR, Irfanoglu A. "Musings on recent developments in Performance-Based Seismic Engineering." (Submitted for) Structures Congress 2004, SEI, American Society of Civil Engineers, Nashville, 2004.