EVALUATATION OF RESPONSE OF A TALL CONCRETE FRAME BUILDING TO MULTIPLE EARTHQUAKES

G. Hart, M. Skokan, and A. Dumortier.
Department of Civil and Environmental Engineering
University of California, Los Angeles

And

S. Huang, Hart Consultant Group, Los Angeles

ABSTRACT

The response of an instrumented reinforced concrete moment-resisting frame (RCMRF) building, located in Southern California, was investigated to show how instrumented response can significantly improve the accuracy of performance based design. RCMRF buildings are particularly difficult to model and therefore this uncertainty reduction is very important. A model of the building using FEMA 273 recommended structural design variables is used as a baseline model. Performance based design estimates are made using this model and then compared with estimates made using improved models that benefit from the measured 1987 Whittier and the 1994 Northridge earthquake response of the building.

INTRODUCTION

The primary objective of the structural engineer for a new or seismic rehabilitation building is the development of a building design whose performance can be accurately estimated during a range of earthquakes. The goal of performance prediction can only be achieved when the design is based on a proper analytical model of the building system and the earthquake ground motion that the building can be expected to experience during its design life. It is obvious that an improved model of an existing building for PML studies can be done more accurately if the model incorporates measured building responses. This paper describes work in progress in this area. The reader is referred to paper approved for publication but in early under expansion with the same title in the Wiley journal entitled "Structural Design of Tall Buildings."

The SEAOC Vision 2000 report (OES, 1995) outlines a framework for implementing the performance base design concept. One of the first steps in performance based design is the selection of performance objectives, each of which requires the selection of a seismic hazard level and performance level. The seismic hazard level is defined by the selection of a return period for the earthquake motion and the performance level specifies a level of structural and non-structural damage by both qualitative and quantitative measures. For each of the selected performance objectives, an analysis of the building is performed using the specified level of seismic hazard and the performance of the building compared to the acceptance criteria for the specified performance level. It is during this phase of the performance base design procedure that the importance of a proper analytical model of the building is very critical.

In this research, the response of an instrumented RCMRF building, located in Southern California, was estimated and compared to the performance of linear elastic analytical models of the building developed with the benefit of measured building response. RCMRF buildings are particularly difficult to model when the objective is to predict the performance of the building. It is difficult to quantify the stiffness of the beams and columns in a linear elastic computer model primarily because the stiffness of each element is highly dependant on the level of strain induced by flexural and axial loads. The baseline model used in this research is based on FEMA 273 and represents a model that does not benefit from building instrumented response measurements. Improved building models are developed using the 1987 Whittier and 1994 Northridge earthquakes and these show the improved response. Furthermore, the contribution of the floor slab to the stiffness of the beams, the effect of confinement on the behavior of the columns, and the stiffness of the beam-column joints, further increases the complexity of the modeling decisions.

BUILDING DESCRIPTION

The focus of this study is a 20-story reinforced concrete frame hotel (Figure 1) located in North Hollywood, California, approximately 19 km from the epicenter of the 1994 Northridge Earthquake. Constructed in 1968, this building was the first to be designed using the 1966 Los Angeles building code that prescribed ductility requirements for reinforced concrete moment resisting frames (Wayman, 1968; Steinmann, 1998). As a result, the design features a strong column-weak beam concept, under-reinforced beams to assure steel yielding prior to concrete crushing, full hoop ties in the beam-column joints, continuous top and bottom beam bars through the joints, and column bar splices at the mid-height (Wayman, 1968).

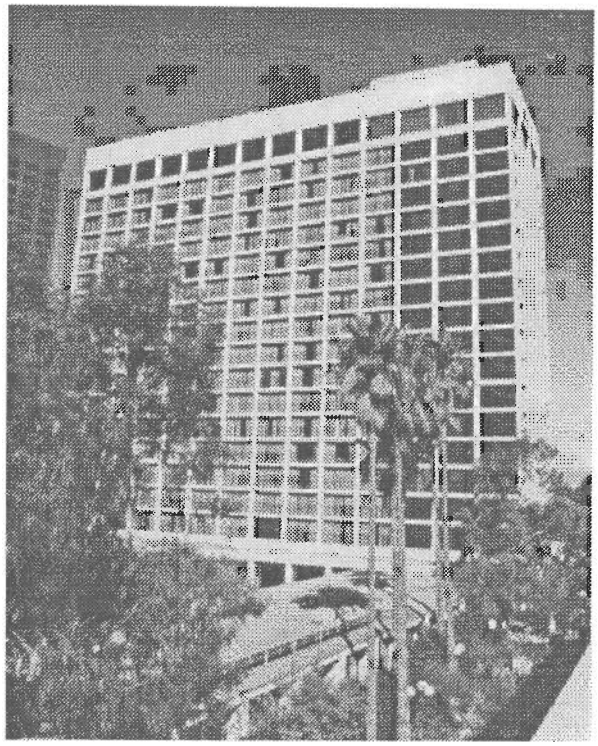


Figure 1 20-Story North Hollywood Building

All concrete is composed of lightweight aggregate with 3,000 psi and 4,000 psi compressive strength at 28 days. The reinforcing steel was specified to be both high-strength ASTM 432 grade with 60,000 psi yield strength and ASTM A-15 grade with 40,000 psi yield strength. The typical floor elevation is 8'-9" and a typical bedroom floor plan is shown in Figure 2. Below grade, perimeter concrete shear walls and spread footings support the 210-ft structure.

SEISMIC BUILDING RESPONSE

The case study building used in this research is a 20-story North Hollywood building that is instrumented with strong motion sensors by the California Strong Motion Instrumentation Program (CSMIP). Sixteen strong motion sensors, as shown in Figure 3, are located over the height of the building, with three sensors placed at each of four floors (3rd' 9th, 16th, and Roof) and four sensors located at the basement level. Strong motion records are available from five major earthquakes over the past 30 years including: 1971 San Fernando, 1987 Whittier, 1992 Landers, 1992 Big Bear, and 1994 Northridge.

Records from the 1987 Whittier and the 1994 Northridge earthquakes are addressed in this research. The ground acceleration time histories in the North-South and East-West directions are shown for the Whittier and Northridge earthquakes in Figures 4 and 5, respectively. Note that for both events, the East-West direction peak ground acceleration (PGA) is larger, and in the case Northridge, the East-West PGA is approximately three times larger. In terms of PGA, it is also observed that Northridge was clearly stronger than the Whittier event. A previous paper entitled "Response Evaluation of a 20-Story Concrete Frame Building to the Northridge and Other Earthquakes" presented at the 1998 SMIP annual meeting describes the building response in some details. Only the highlights are presented here.

Figure 6 shows the roof relative displacement ratio history at the center-of-mass location. The center-of-mass displacement history was calculated by transforming the displacement history from the three roof sensors using the methodology outlined by Naeim (1997). Figure 6 also indicates the displacement ratio suggested by the Vision 2000 report for the Fully Operational and Operational performance levels. In general, the displacement ratios fall within the definition of the Fully Operational performance level, however, one strong pulse in the North-South direction displaces the building to a 0.4% displacement ratio. This is near the displacement ratio used to define the Operational performance level. A goal of performance based design is to accurently calculate a response for the future earthquakes for evaluation of the type shown in Figure 6.

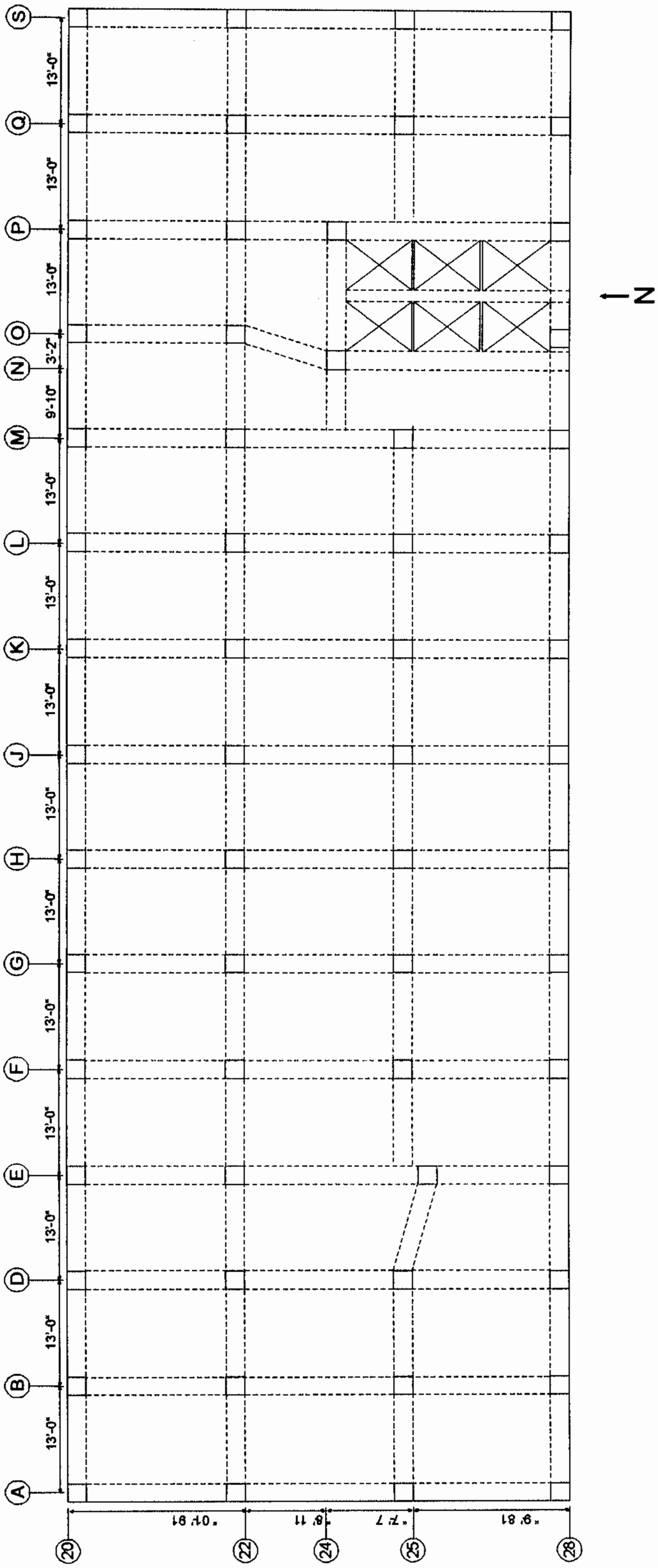


Figure 2 Typical Bedroom Floor Plan

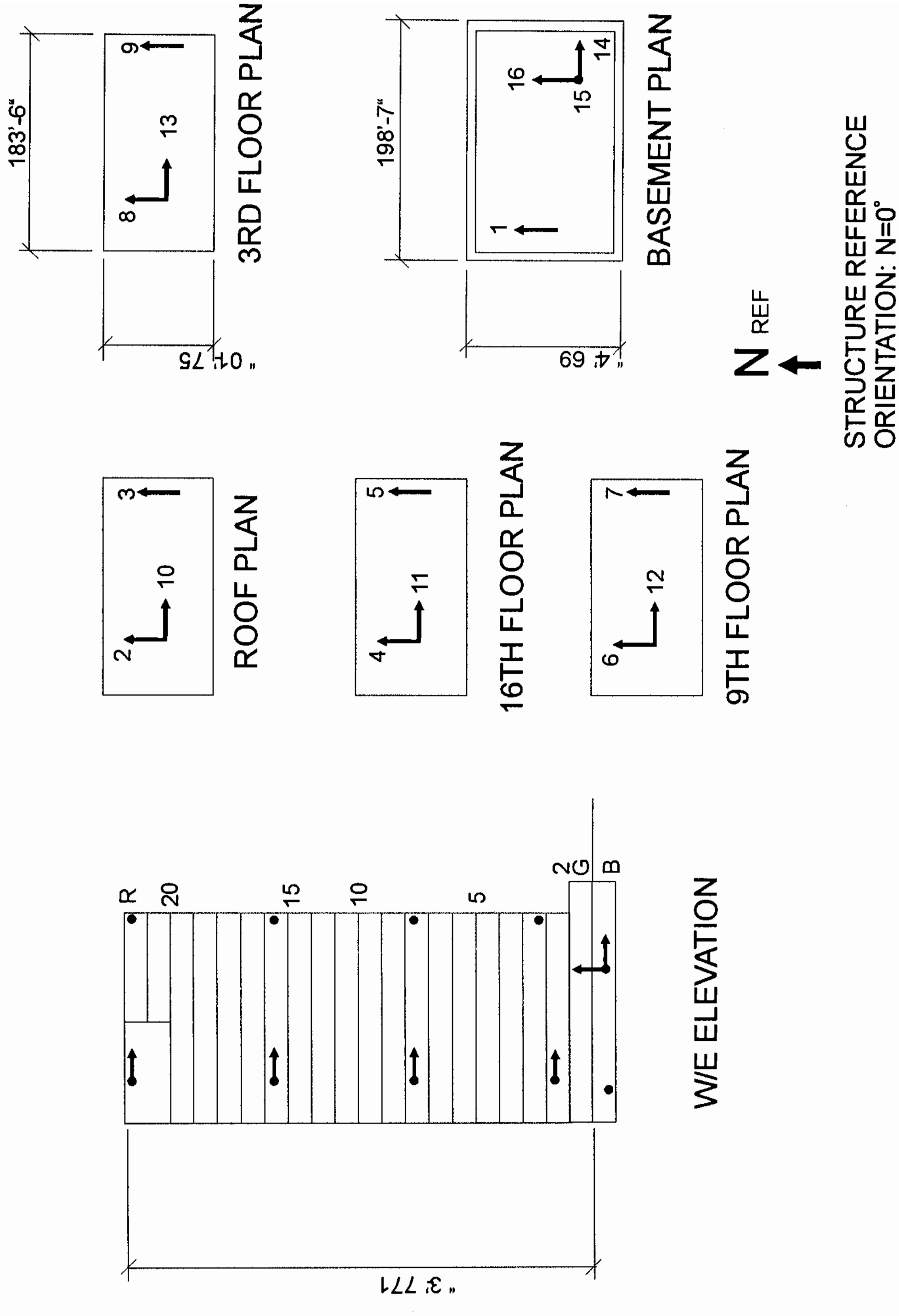
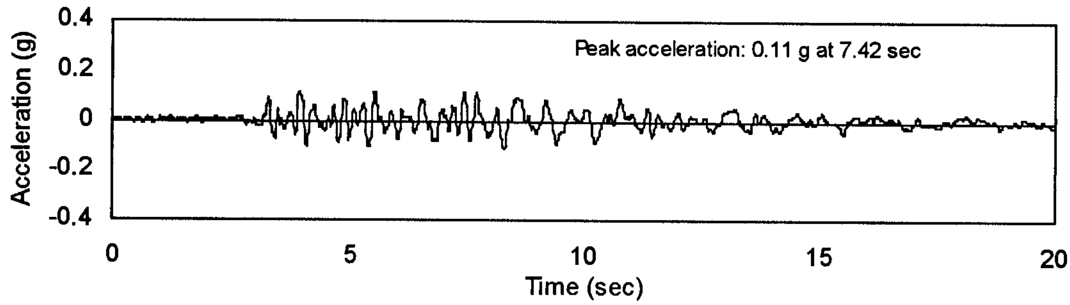
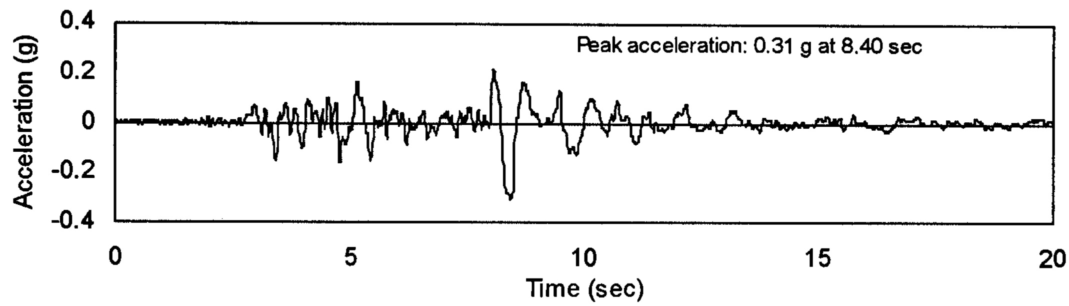


Figure 3 Sensor Locations

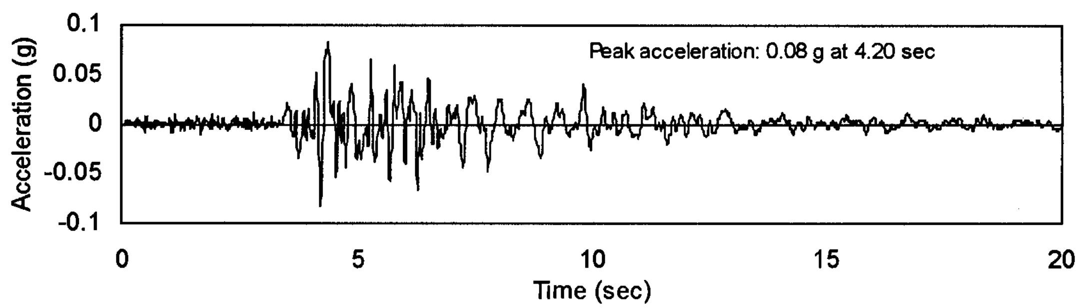


(a) North-South (Longitudinal) Direction (Channel 16)

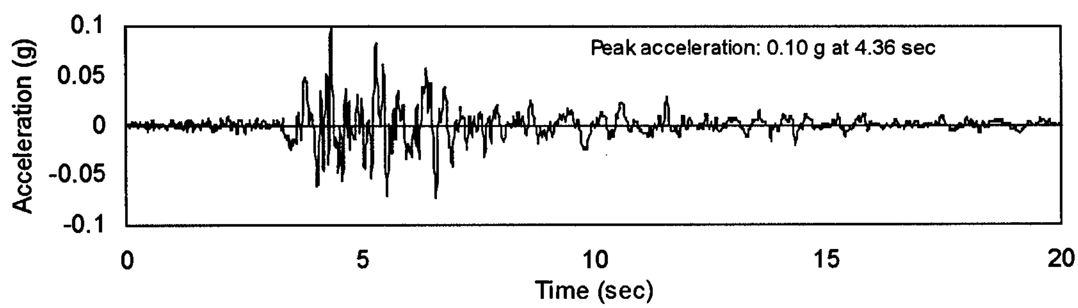


(b) East-West (Transverse) Direction (Channel 14)

Figure 4 1994 Northridge Earthquake Acceleration Time Histories at Ground Level



(a) North-South (Longitudinal) Direction (Channel 16)



(b) East-West (Transverse) Direction (Channel 14)

Figure 5 1987 Whittier Earthquake Acceleration Time Histories at Ground Level

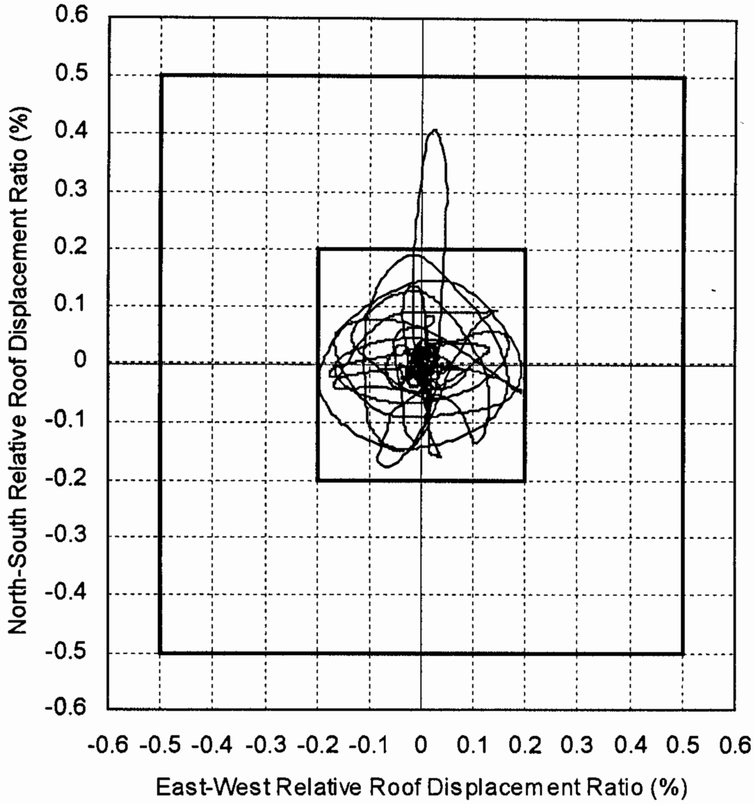


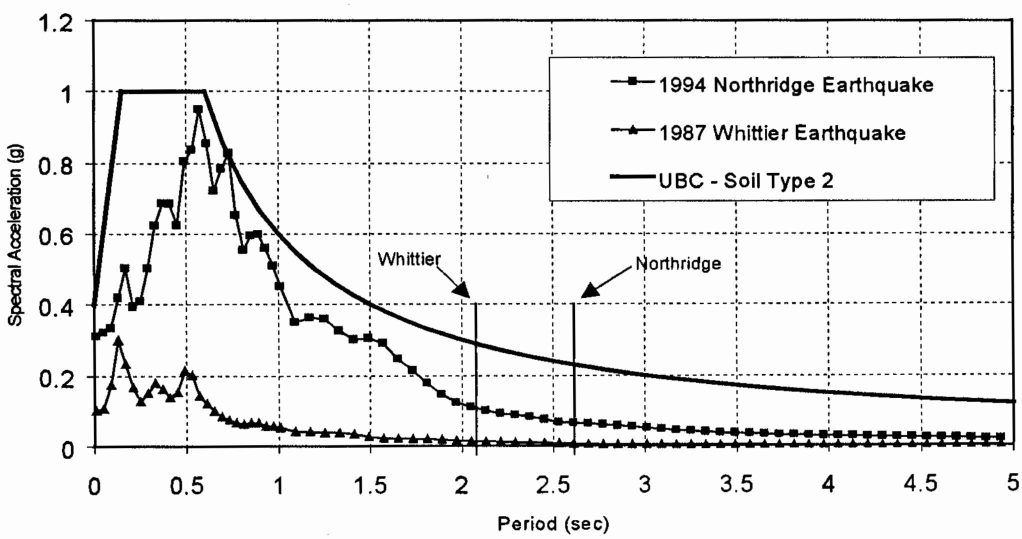
Figure 6 Northridge earthquake relative roof displacement ratio history with Vision 2000 (OES, 1995) performance level drift ratios

Studies of the instrumented response of the case study building (Goel, et. al, 1997; Naeim, 1997) have yielded estimates of the fundamental translational periods of vibration of the building. In addition, the studies by Goel, et. al. have resulted in estimates of the percentage of critical damping for these fundamental periods. The results of the work by Goel, et. al. are provided in Table 1. The results show that the periods of vibration in the transverse and longitudinal building directions are approximately equal. Note also that the fundamental periods estimated for the Northridge earthquake are approximately 18% larger than for the Whittier earthquake. This is most likely due to the fact that the displacement demands from the Northridge earthquake were significantly greater than during the Whittier earthquake.

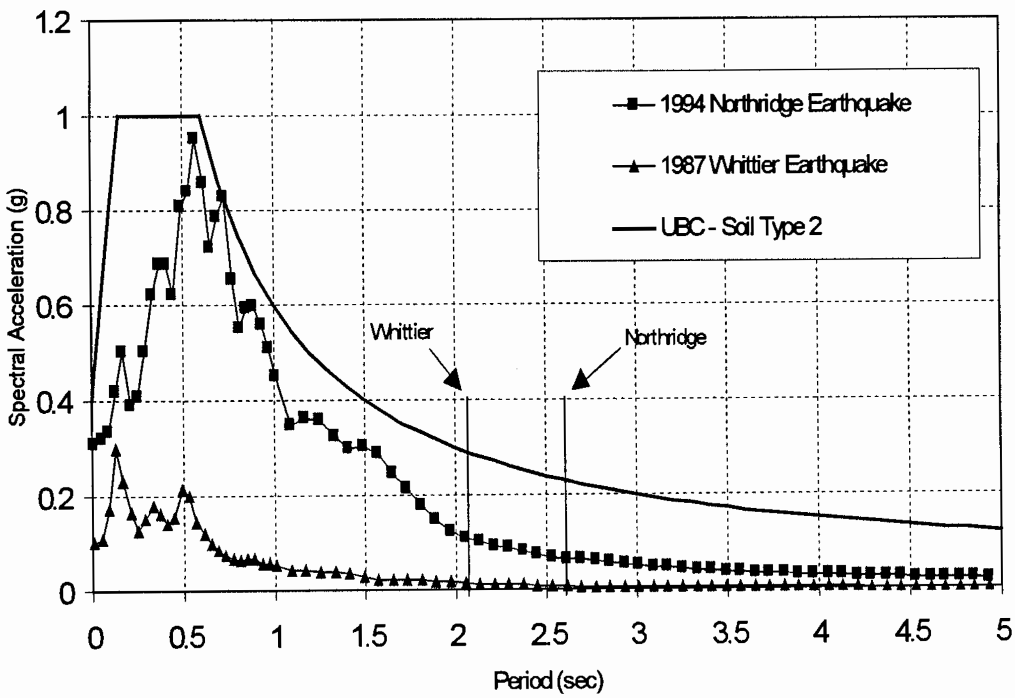
Table 1 Translational periods of vibration and percentage of critical damping (Goel, et. al, 1997)

Earthquake Record	North-South (Longitudinal)		East-West (Transverse)	
	Period (sec)	Damping (%)	Period (sec)	Damping (%)
1987 Whittier	2.15	---	2.21	---
1994 Northridge	2.60	5.9	2.62	6.5

Using the ground acceleration records for the Whittier and Northridge earthquakes, the 5% damped response spectra were calculated and are given in Figure 7 along with the UBC design spectrum for soil type 2. For the fundamental periods estimated by Goel, et. al., notice that the spectral accelerations are well below the values that given by the UBC design spectrum.



(a) North-South (Transverse) direction



(b) East-West (Longitudinal) direction

Figure 7 5% damped response spectra at ground level

BASELINE, PRE EARTHQUAKES, ANALYTICAL COMPUTER MODELS

The case study building was analyzed using the three-dimensional linear elastic computer program ETABS (CSI, 1997). All of the beams and columns of the moment resisting frames were included in the model along with the walls at the basement level.

The floor diaphragms were assumed to be rigid and assigned a mass, center-of-mass location, and mass moment of inertia based on detailed calculations assuming point masses at the column locations. Since the structural elements are composed of lightweight concrete, 105 pcf was assumed for the unit weight of all concrete. In addition, the weight of partitions, exterior cladding, and specific mechanical equipment was included along with 15 psf to account for mechanical, electrical, ceiling and floor finishes, and other miscellaneous items.

The *FEMA 273 Stiffness* model is considered to be the **Baseline** model. In the model, the beams and columns are assigned an effective moment of inertia based on a percentage of the gross moment of inertia suggested by the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings - FEMA 273* (ATC, 1997). FEMA 273 suggests the use of 50% of gross for beams, 70% of gross for columns in axial compression, and 50% of gross for columns in axial tension. Since the axial load in the columns will vary throughout the analysis, 60% of gross was assumed for the column stiffness in this model.

PREDICTED BUILDING RESPONSE

The Baseline model described in the previous section was used to calculate the natural frequencies and mode shapes for the building. Table 2 gives the periods of vibration and the percentage of participating mass in each direction for the first nine modes. They are also presented in Table 3 for a sensitivity study where key structural model variables were slightly changed from the FEMA values. The structural parameters considered were the rigid end zone (REZ), the inertia of the columns and the beams, and the modal damping. For each change of parameter, the maximum displacements predicted by the analyses were compared to the maximum recorded displacements at each instrumented floor. The results are displayed in Figure 8.

This led the writers to investigate the influence of each parameter on the structural response of the building in more details. The parameters were then varied from the FEMA model one at a time, from their minimum to their maximum values, and values of displacements were calculated and compared to the maximum recorded displacements. The stiffness of the beam-column joints was varied to reflect a range of behavior. This was accomplished by varying the length of the rigid end zone (REZ) at the beam-column intersections. In the *100% REZ* model, the entire beam-column intersection was assumed to rigid, the *50% REZ* model assumes that only one-half of the beam-column intersection is rigid, and the *Centerline* model assumes no rigidity of the beam-column intersection. As one would expect, as the stiffness of the beams, columns, and REZ's decreases, the periods of vibration tend to increase. Different reduction factors were also applied to the inertia of the columns and the beams, and the modal damping was also varied. The results plotted on Figure 9 through 14 show that the inertia of the columns and the beams have a greater influence on the structural response than the REZ and damping.

The results reported here are a portion of the ongoing research which is working towards improving the accuracy of performance based design. Further refinement is still underway and will be reported in future report.

Table 2 Periods of Vibration and Participating Mass from the Baseline Model

Mode #	FEMA			
	Period (Sec)	Participating Mass (%)		
		N-S	E-W	Rotation
1	3.03	36.5	1.1	30.4
2	2.86	24.1	28.5	17.8
3	2.82	9.8	40.6	18.9
4	1.03	9.0	0.0	3.0
5	0.95	1.1	6.0	3.3
6	0.93	0.9	4.6	5.5
7	0.61	4.1	0.0	0.3
8	0.57	0.1	3.3	1.4
9	0.57	0.2	1.8	3.3

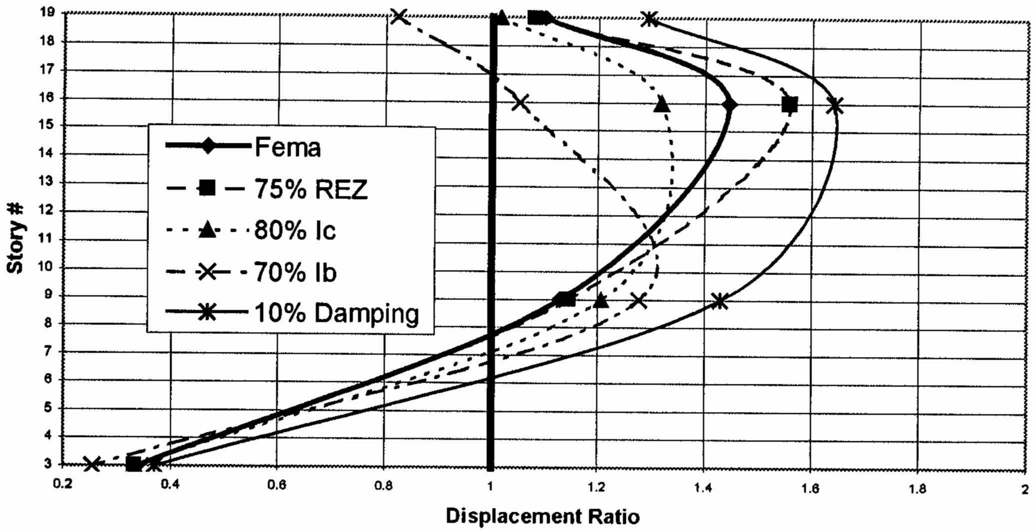
Table 3 Periods of Vibration and Participating Mass from the Sensitivity Study of the Baseline Model

Mode #	FEMA with 75% REZ			
	Period (Sec)	Participating Mass (%)		
		N-S	E-W	Rotation
1	3.14	34.7	1.6	31.7
2	2.98	14.1	51.1	5.4
3	2.93	21.6	17.6	30.0
4	1.07	8.8	0.1	3.1
5	1.00	0.8	7.5	2.0
6	0.97	1.2	3.0	6.7
7	0.64	4.1	0.0	0.3
8	0.60	0.1	4.0	0.7
9	0.59	0.2	1.0	3.9

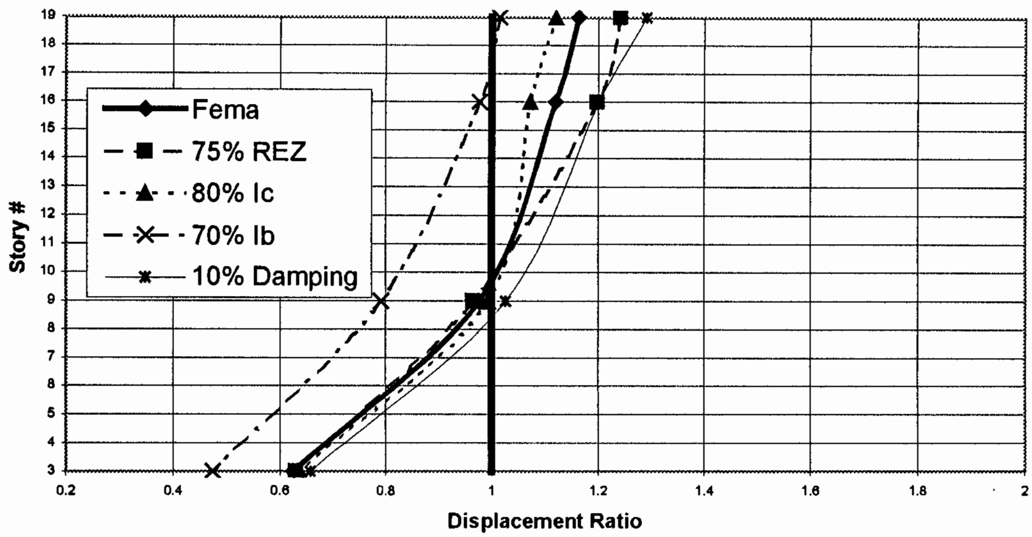
Mode #	FEMA with 80% I Column			
	Period (Sec)	Participating Mass (%)		
		N-S	E-W	Rotation
1	2.94	36.5	1.0	29.9
2	2.78	25.8	23.2	20.6
3	2.74	7.5	45.2	15.8
4	0.99	9.0	0.1	3.1
5	0.92	1.1	6.1	3.2
6	0.90	0.9	4.6	5.5
7	0.58	4.2	0.0	0.4
8	0.55	0.2	3.5	1.4
9	0.54	0.2	1.8	3.5

Mode #	FEMA with 70% I Beam			
	Period (Sec)	Participating Mass (%)		
		N-S	E-W	Rotation
1	2.70	40.4	0.9	27.4
2	2.55	25.9	18.2	26.8
3	2.51	4.6	52.1	13.5
4	0.93	9.5	0.0	2.6
5	0.86	1.1	4.8	4.6
6	0.84	0.6	5.8	4.7
7	0.55	4.1	0.0	0.2
8	0.52	0.1	2.3	2.0
9	0.51	0.1	2.4	2.6

Mode #	FEMA with 10% Damping			
	Period (Sec)	Participating Mass (%)		
		N-S	E-W	Rotation
1	3.03	36.5	1.1	30.4
2	2.86	24.1	28.5	17.8
3	2.82	9.8	40.6	18.9
4	1.03	9.0	0.0	3.0
5	0.95	1.1	6.0	3.3
6	0.93	0.9	4.6	5.5
7	0.61	4.1	0.0	0.3
8	0.57	0.1	3.3	1.4
9	0.57	0.2	1.8	3.3



(a) North-South Direction



(b) East West Direction

Figure 8 Maximum Recorded / Maximum Predicted Floor Displacement Ratios - Whittier EQ -

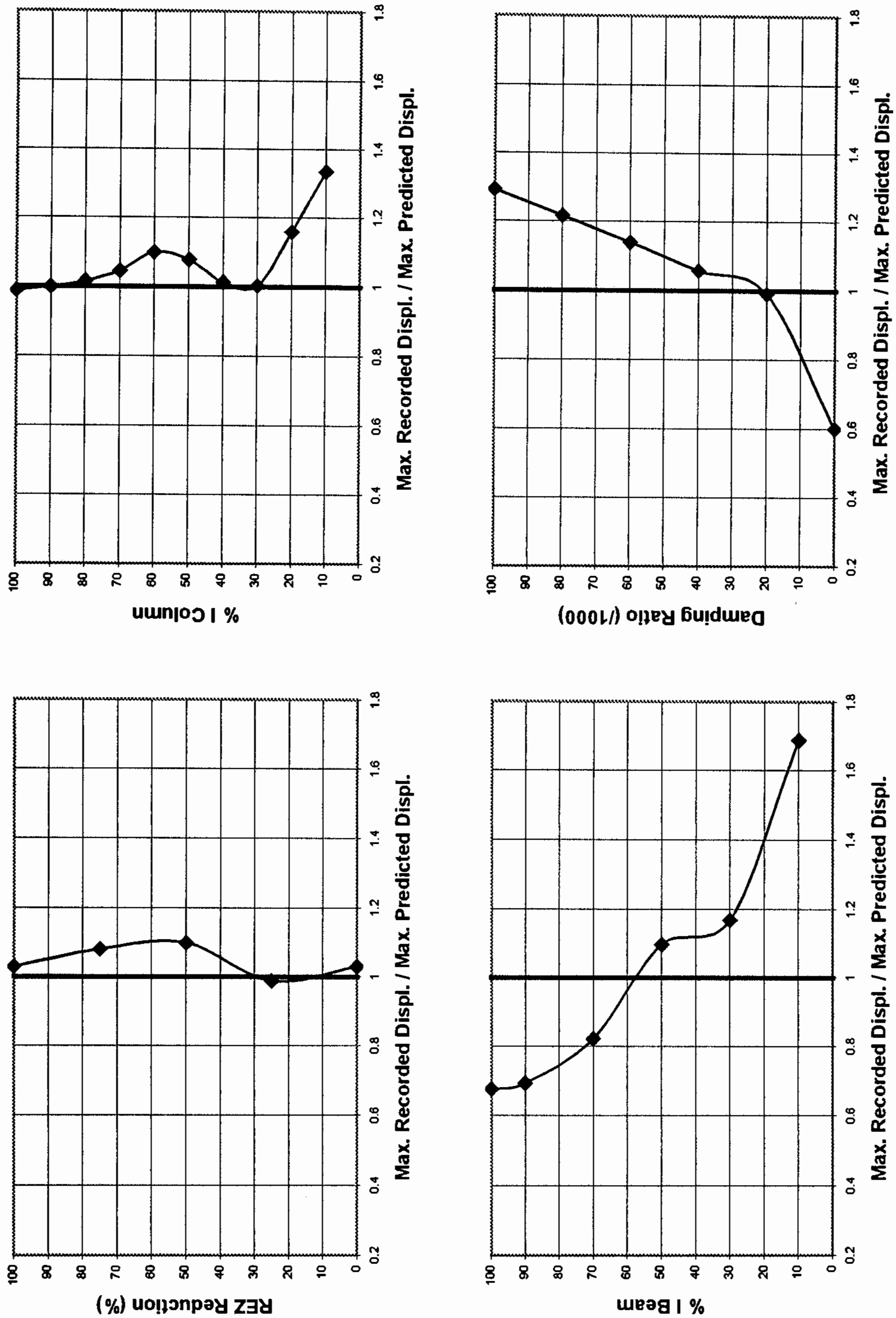


Figure 9 Roof Maximum Recorded / Maximum Predicted Displacement Ratios – Whittier EQ. – North-South Direction

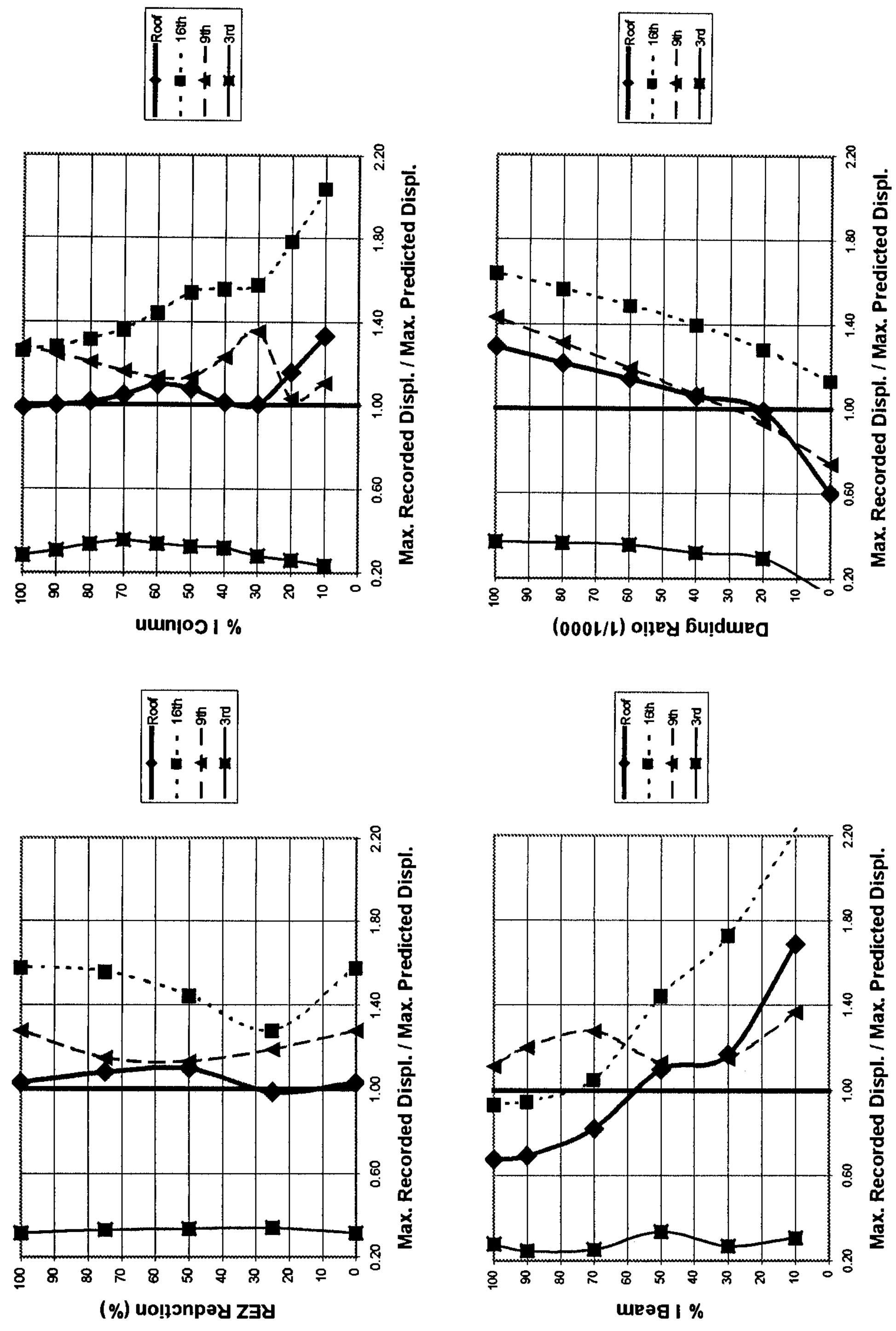


Figure 10 Story Maximum Recorded / Maximum Predicted Displacement Ratios – Whittier EQ. – North-South Direction

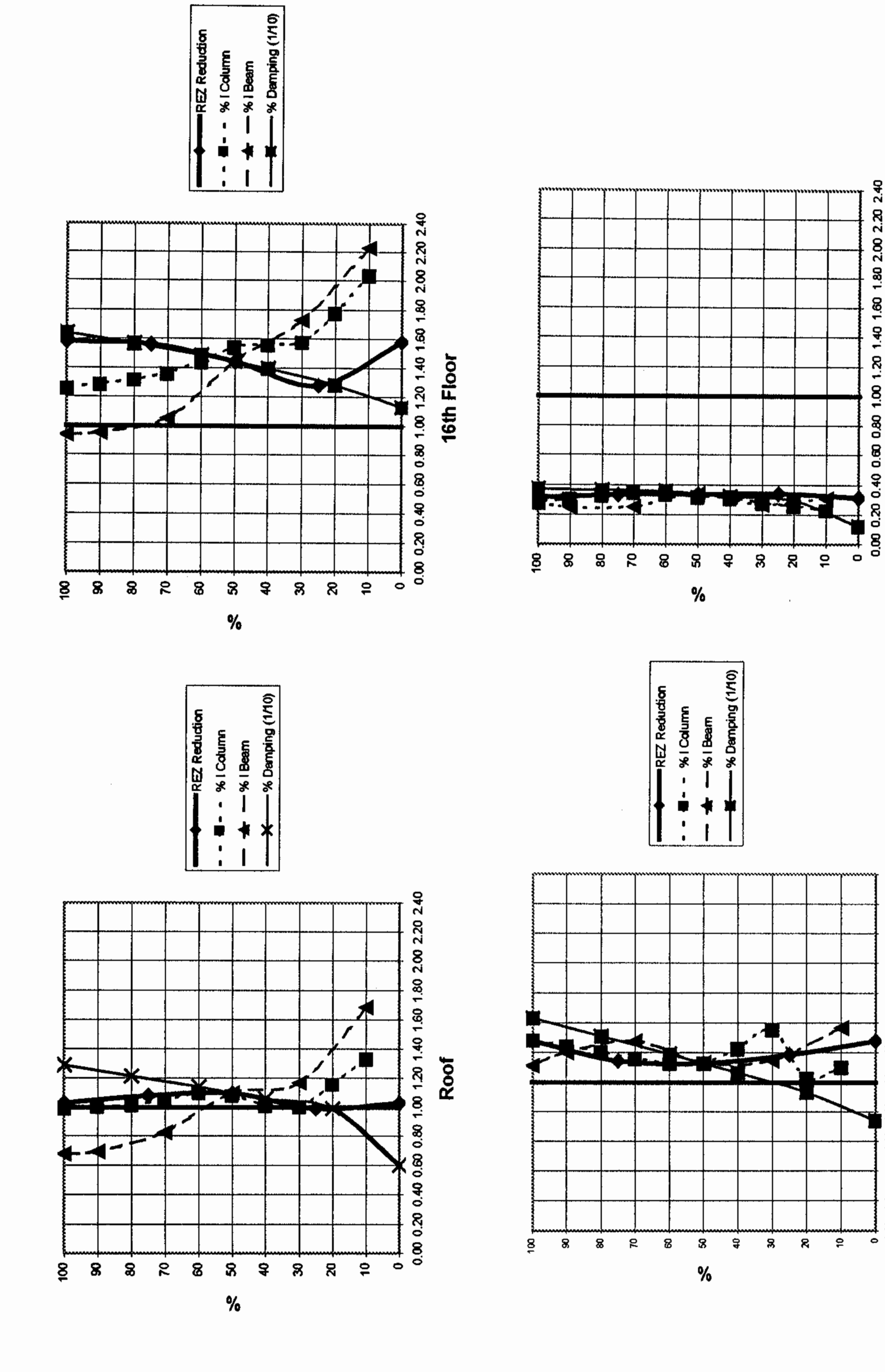


Figure 11 Maximum Recorded / Maximum Predicted Displacement Ratios – Whittier EQ. – North-South Direction

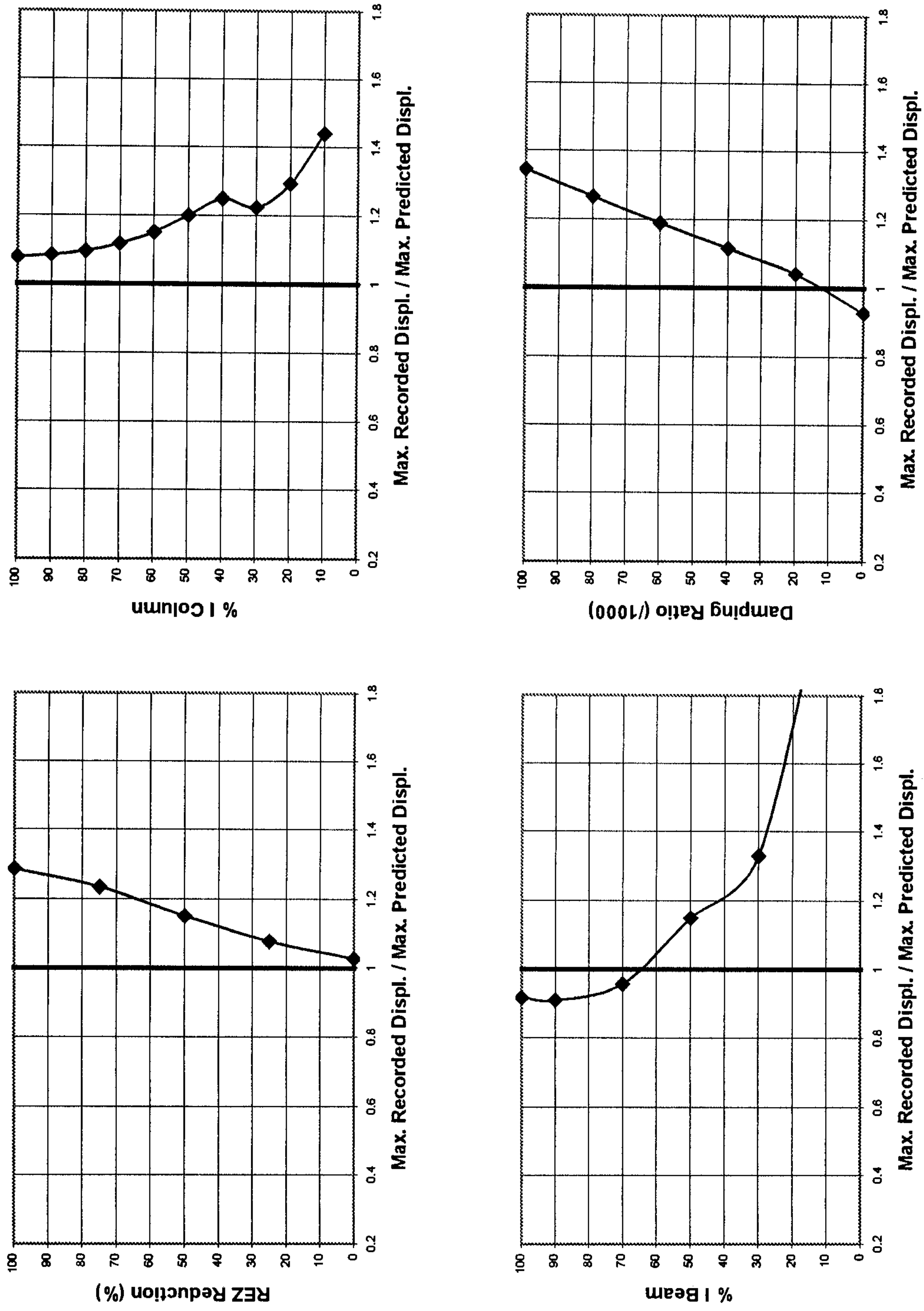


Figure 12 Roof Maximum Recorded / Maximum Predicted Displacement Ratios – Northridge EQ. – North-South Direction

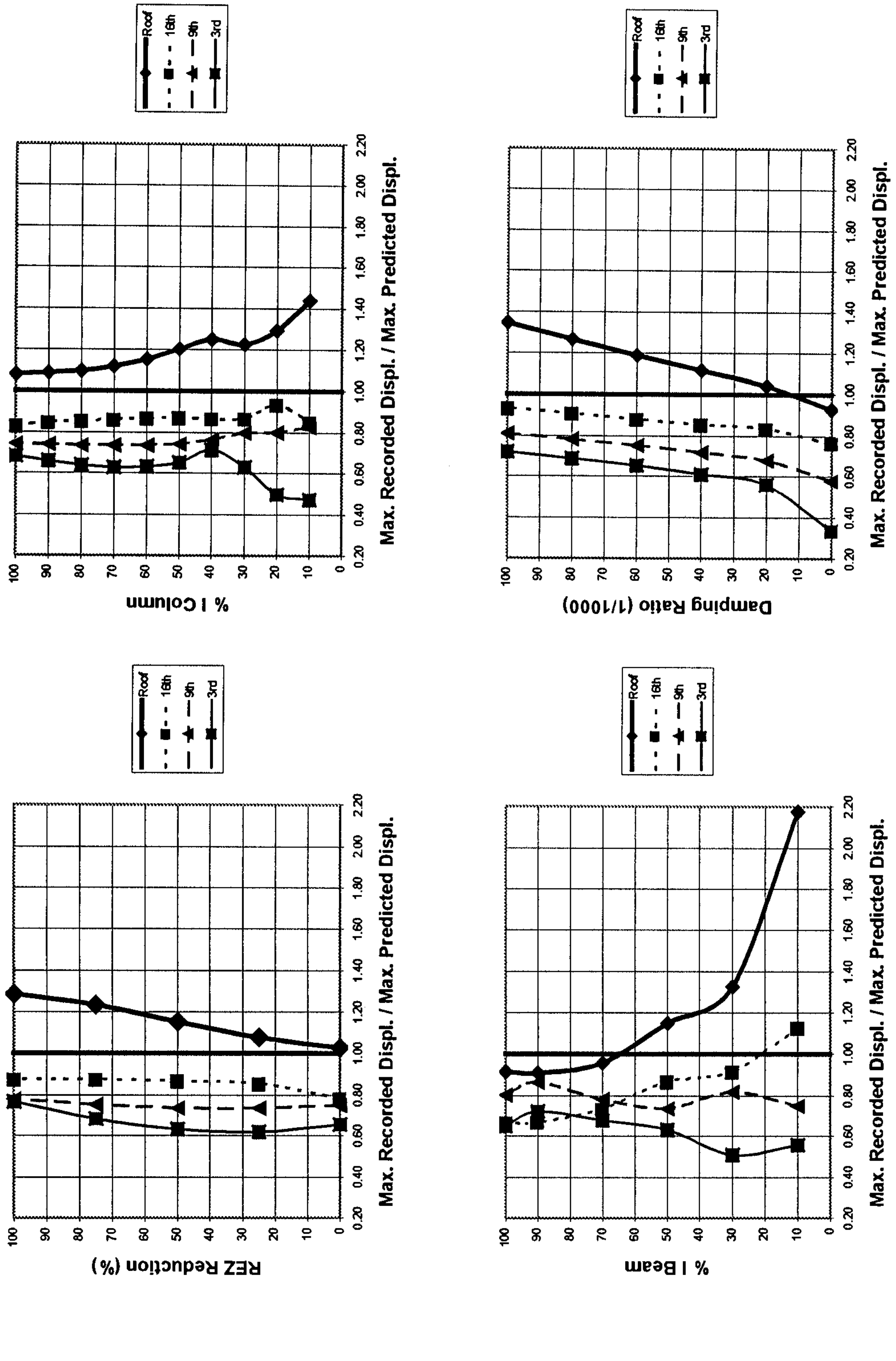


Figure 13 Story Maximum Recorded / Maximum Predicted Displacement Ratios – Northridge EQ. – North-South Direction

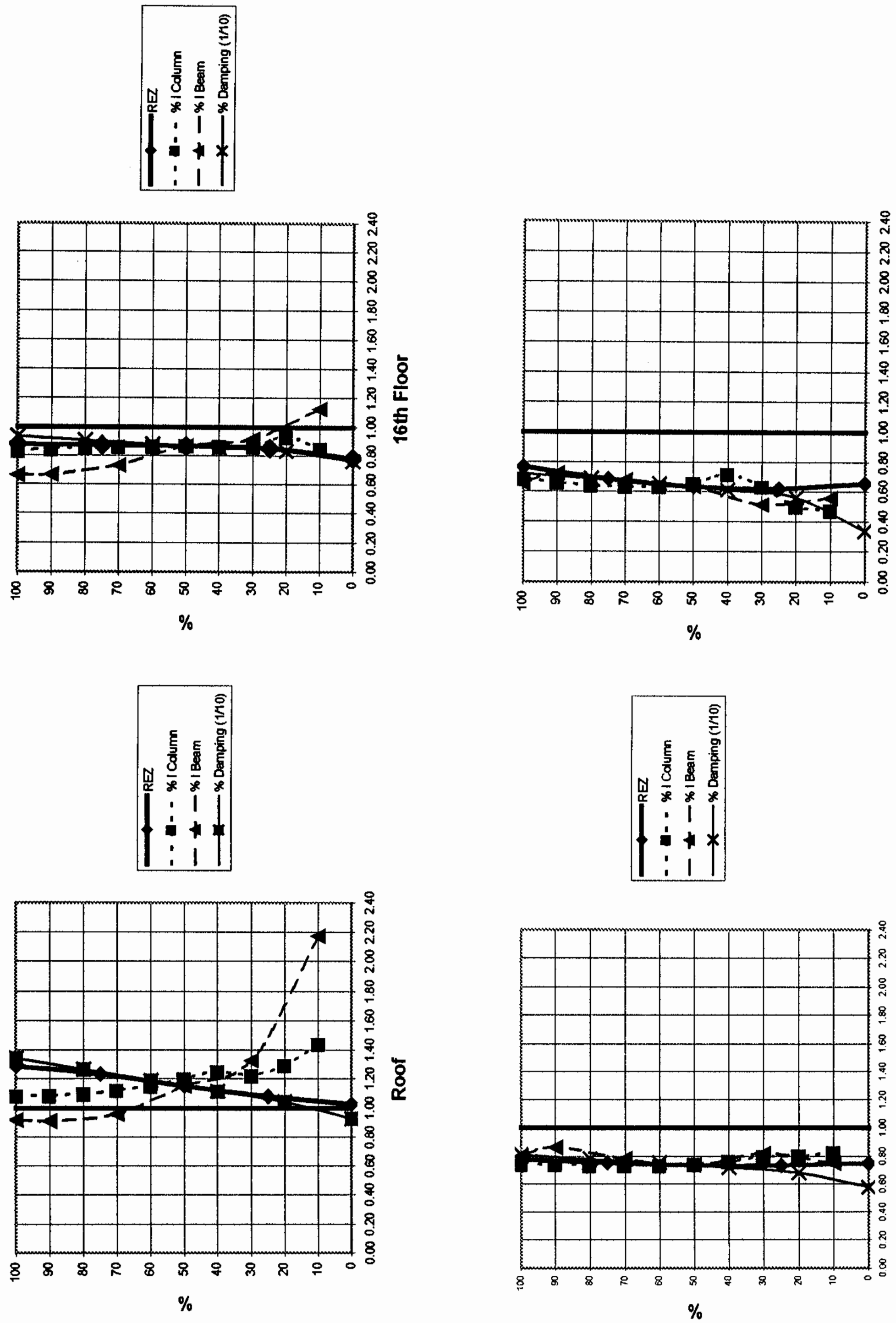


Figure 14 Maximum Recorded / Maximum Predicted Displacement Ratios – Northridge EQ. – North- South

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REFERENCES

ATC. (1997). *NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Volume I, Guidelines*. FEMA 273. Federal Emergency Management Agency.

CSI (1997) "ETABS – Extended Three Dimensional Analysis of Building Systems – Nonlinear Version 6.21," Computers & Structures, Inc.

Goel, R.K. and Chopra, A.K. (1997). "Vibration Properties of Buildings Determined From Recorded Earthquake Motions," *Report No. EERC/UCB-72/22*, Earthquake Engineering Research Center, University of California, Berkeley, California.

Naeim, F. (1997). "Instrumented Buildings Information System, January 14, 1994, Northridge, California Earthquake, Version 1.0," John A. Martin & Associates.

Paulay, T. and Priestley, M.J.N. (1992). *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley Interscience, 744 pp.

OES. (1995). California Office of Emergency Services, *Vision 2000: Performance Based Seismic Engineering of Buildings*. Prepared by Structural Engineers Association of California, Sacramento, California.

Saatcioglu, M. and Razvi, S.R. (1992). "Strength and Ductility of Reinforced Concrete," *Journal of Structural Engineering*, Vol. 118, No. 6, pp. 1590-1607.

Steinmann, H.G. (1998). Personal Communication.

Wallace, J.W. (1992). "BIAX: Revision 1 – A Computer Program for the Analysis of Reinforced Concrete and Reinforced Masonry Sections," *Report No. CU/CEE-92/4*, Clarkson University, Potsdam, New York.

Wing, W.C. (1968). "Hotel Pioneers Earthquake Code," *Engineering News Record*, Feb. 22.