RESPONSE EVALUATION OF A 20-STORY CONCRETE FRAME BUILDING TO THE NORTHRIDGE AND OTHER EARTHQUAKES

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ABSTRACT

The response of an instrumented reinforced concrete moment-resisting frame (RCMRF) building, located in Southern California, was investigated in this research and compared to the response of linear elastic analytical models of the building. RCMRF buildings are particularly difficult to model when the objective is to predict the performance of the building. Therefore, nine models of the case study building were created by making three assumptions for the stiffness of the beams and columns and three assumptions for the stiffness of the beam-column joints. Fundamental periods for the models were compared to the fundamental periods calculated directly from the building response recorded during the 1987 Whittier and 1994 Northridge earthquakes. In addition, the analytical models were subjected to the ground accelerations recorded during the Whittier and Northridge earthquakes in a time history analysis and the maximum floor displacements compared to the recorded floor displacements.

INTRODUCTION

The design goal of any structural engineer for a new building or seismic rehabilitation project must be the development of a building design whose performance during a range of earthquakes can be accurately estimated. This concept is known as Performance-Based Design (PBD) and is currently the subject of building code developments. 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.

The SEAOC Vision 2000 report (OES, 1995) outlines a framework for implementing the PBD concept. One of the first steps in PBD 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 seismic hazard and the building response is compared to the acceptance criteria for the specified performance level. It is during this phase of the PBD procedure that the importance of a proper analytical model of the building is realized.

In this research, the response of an instrumented RCMRF building, located in Southern California, was studied and compared to the response of linear elastic analytical models of the building. 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. 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).

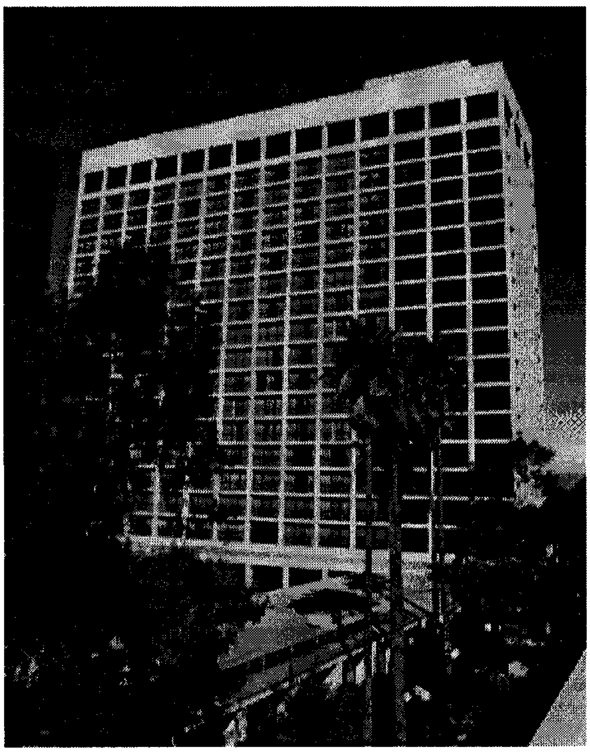


Figure 1 20-story North Hollywood building

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 20-story North Hollywood building 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.

Only the 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 Northridge and Whittier earthquakes in Figures 4 and 5, respectively. Note that for both events, the East-West direction peak ground acceleration (PGA) is the larger of the two components, 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.

During the Northridge earthquake, the case study building suffered heavy non-structural and content damage, with no signs of significant structural damage. The non-structural damage was limited to partitions, door openings, floor tiles, chandeliers, and broken glass. Sidewalk slabs on grade were cracked, some oil spillage occurred in the basement, and damage to mechanical equipment was minimal (Naeim, 1997).

In order to compare the observed damage and the performance level guidelines provided in the Vision 2000 report, the recorded displacement response from the Northridge earthquake was studied. 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. According to the Vision 2000 report, the damage expected at the Operational performance level consists of minor structural damage, light to moderate non-structural damage including broken glass, cracked partitions, minor damage to light fixtures, and minor content damage. In general, this is very consistent with the observed damage.

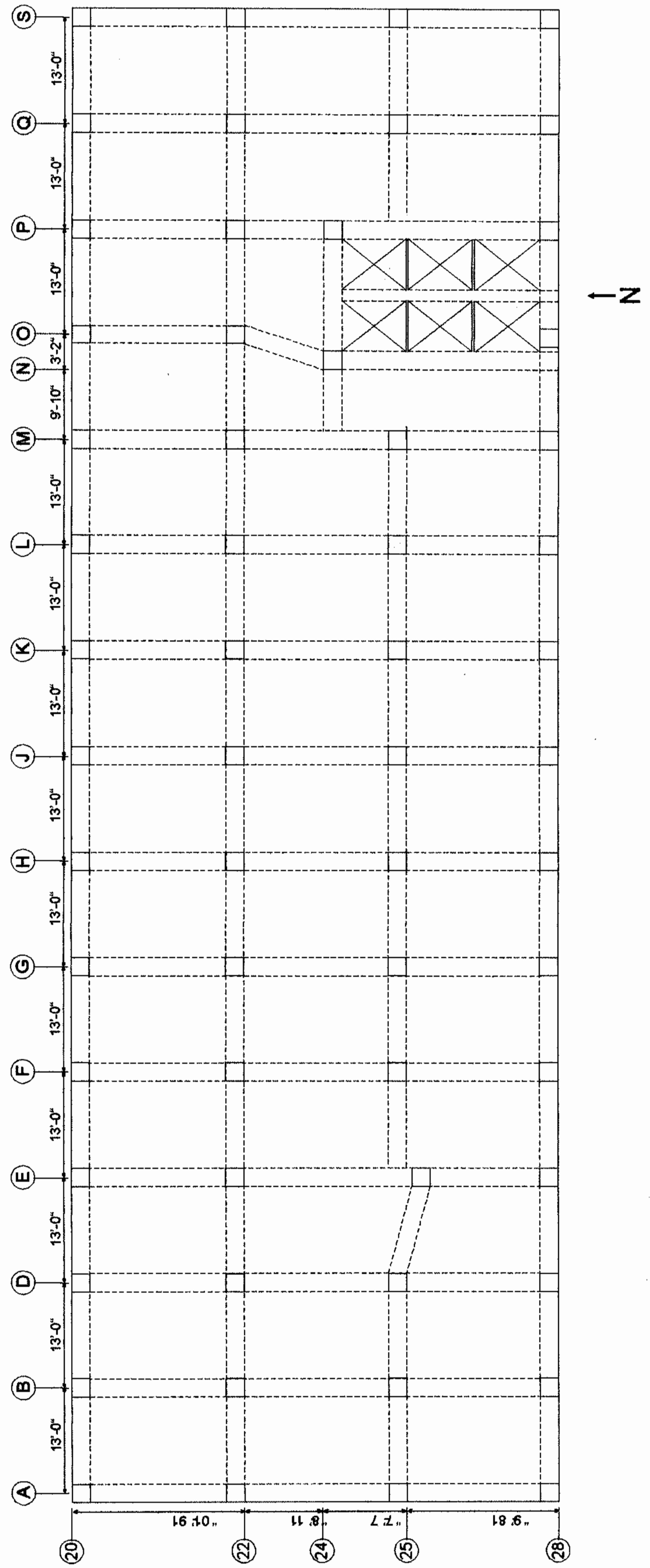


Figure 2 Typical bedroom floor plan

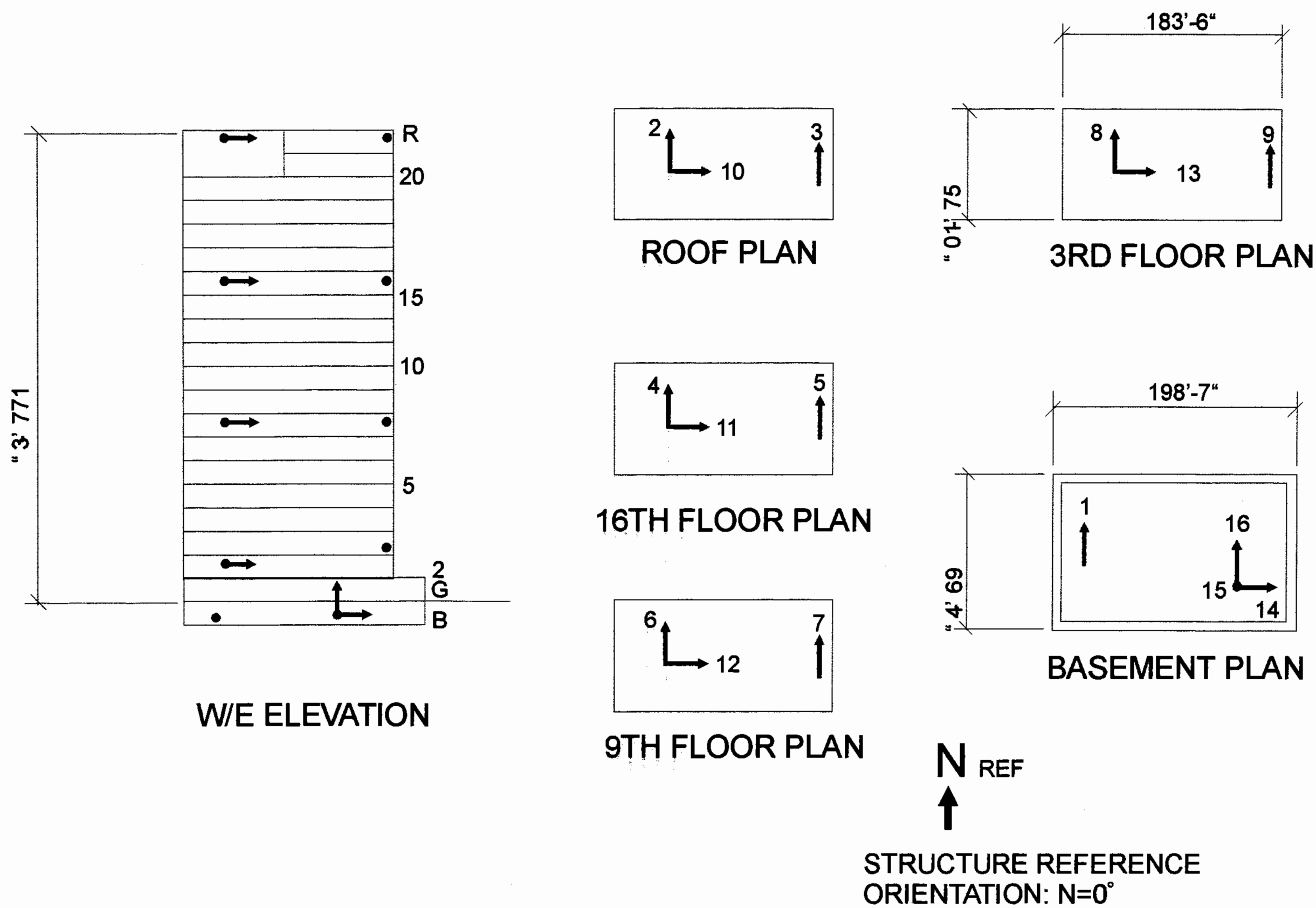
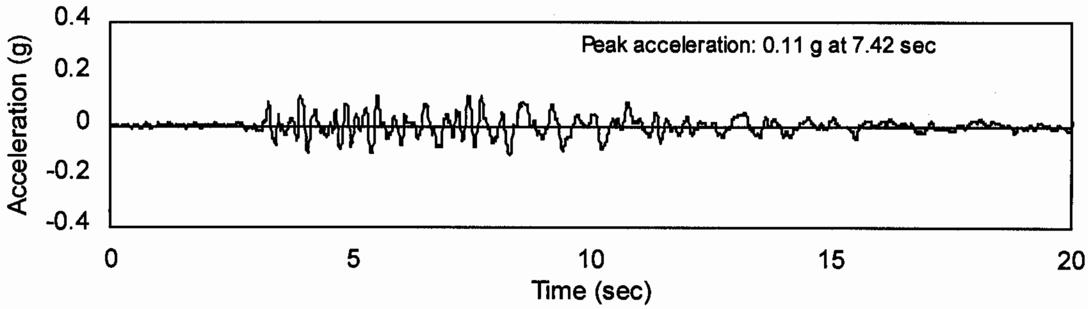
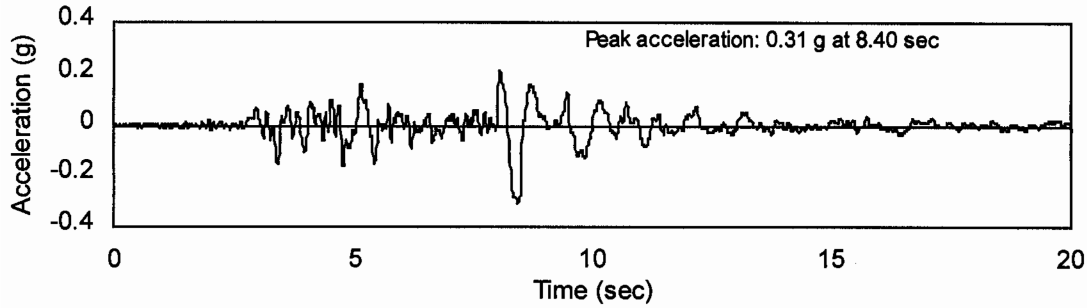


Figure 3 Sensor locations

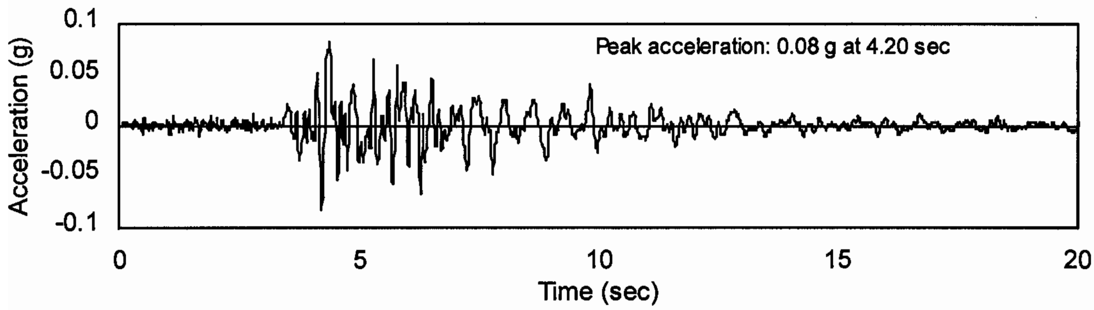


(a) North-South (Longitudinal) direction (Channel 16)

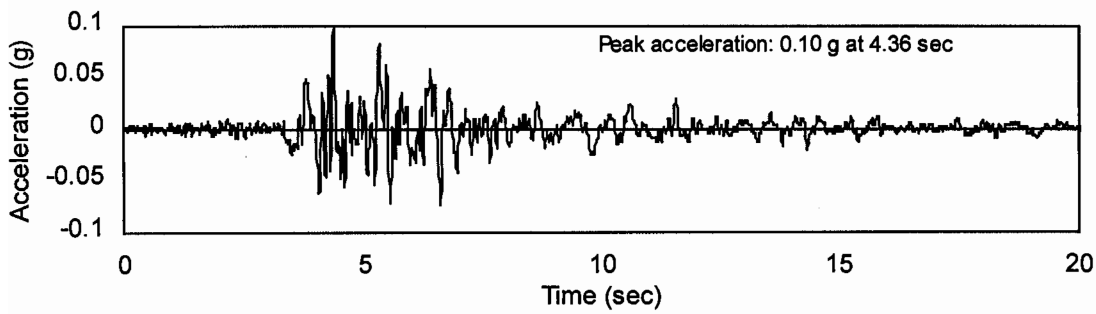


(b) East-West (Transverse) direction (Channel 14)

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

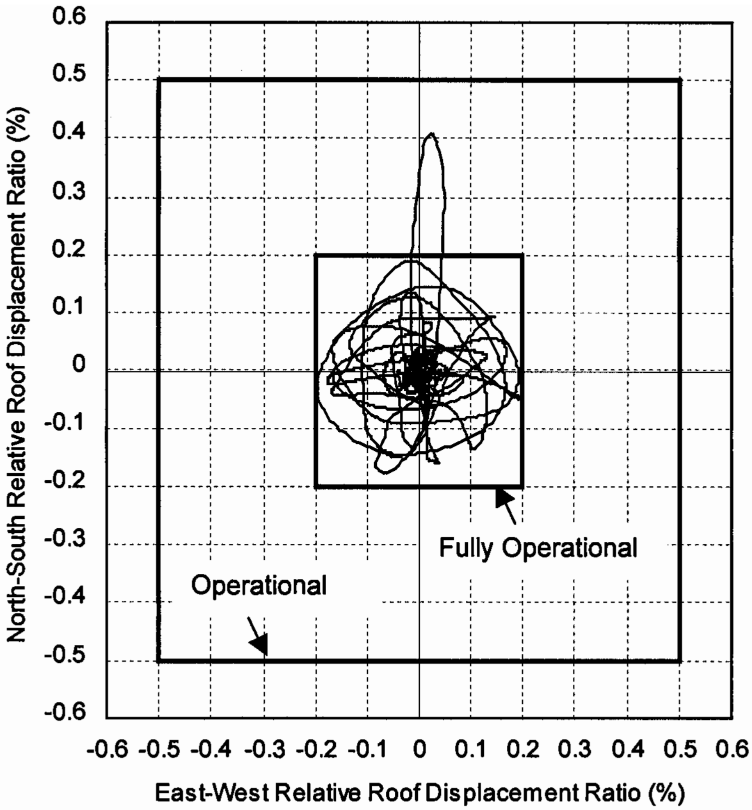


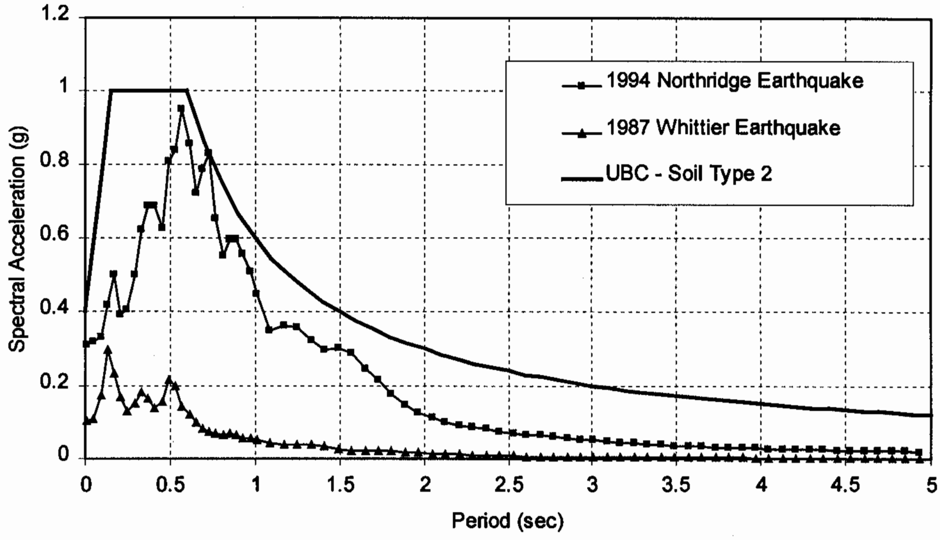
Figure 6 Northridge earthquake relative roof displacement ratio history with Vision 2000 (OES, 1995) performance level drift ratios

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 for 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.

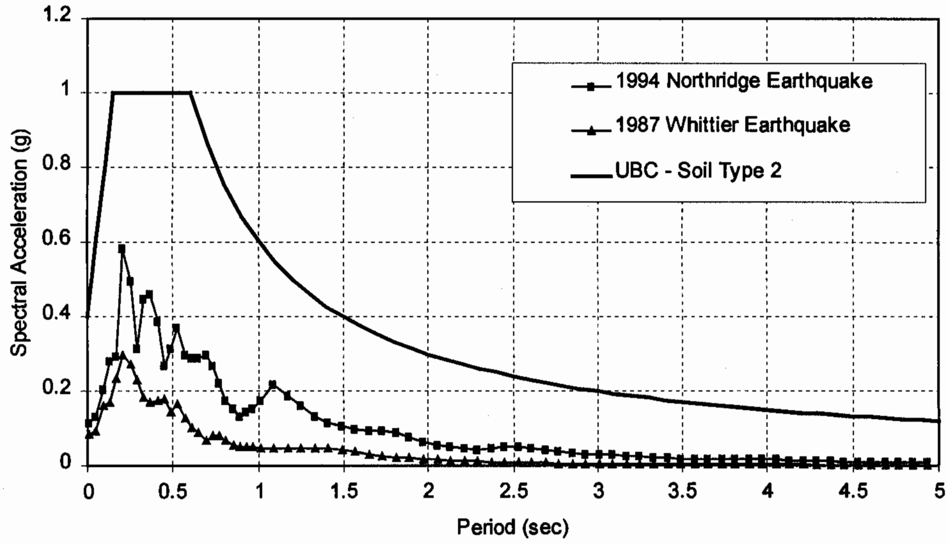
Table 1 Translational periods of vibration and percentage of critical damping (Goel, et. al, 1997)

Earthquake Record	North-South (Longitudinal)		East-West (Transverse)	
	Period (sec)	Damping (%)	Period (sec)	Damping (%)
1987 Whittier	2.15	---	2.21	---
1994 Northridge	2.6	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

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. A three-dimensional illustration of the computer model is provided in Figure 8.

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.

Nine models of the case study building were created by making three assumptions for the stiffness of the beams and columns and three assumptions for the stiffness of the beam-column joints.

For the *Gross Stiffness* model, all of the beams and columns were assigned a gross moment of inertia for flexure, representing the stiffness of the elements prior to cracking. This type of model would be applicable for buildings that are subjected to minor ground shaking inducing low levels of stresses in the structural elements.

In the *FEMA 273 Stiffness* model, the beams and columns were 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. These percentages are general guidelines intended to estimate the effective (or secant) stiffness of the elements at the first yield of the reinforcement.

The *Yield Stiffness* model is similar in concept to the FEMA 273 Stiffness model, except that the effective flexural stiffness at first yield of the reinforcement is determined from a moment-curvature analysis of the section. The reinforced concrete strength analysis computer program BIAX (Wallace, 1992) was used to calculate the moment-curvature relations. The analysis included the concrete stress-strain model developed by Saatcioglu, et. al. (1992) and strain hardening of reinforcing steel. Expected values were assumed for the concrete and steel strengths. For each beam, both a bare and flanged beam section were analyzed and the resulting stiffness values averaged based on the assumption that the beams will bend in double curvature. The effective slab width included in the flanged beam analyses was based on the recommendations by Paulay and Priestley (1992). For the columns, the axial load was varied in the analyses and the stiffness assigned to the column elements was based on an estimate of the axial load in the element due to gravity load. The results of the moment-curvature analyses showed that the effective stiffness typically ranges between 30-50% of gross for the beams and 30-60% of gross for the columns depending on the level of axial load.

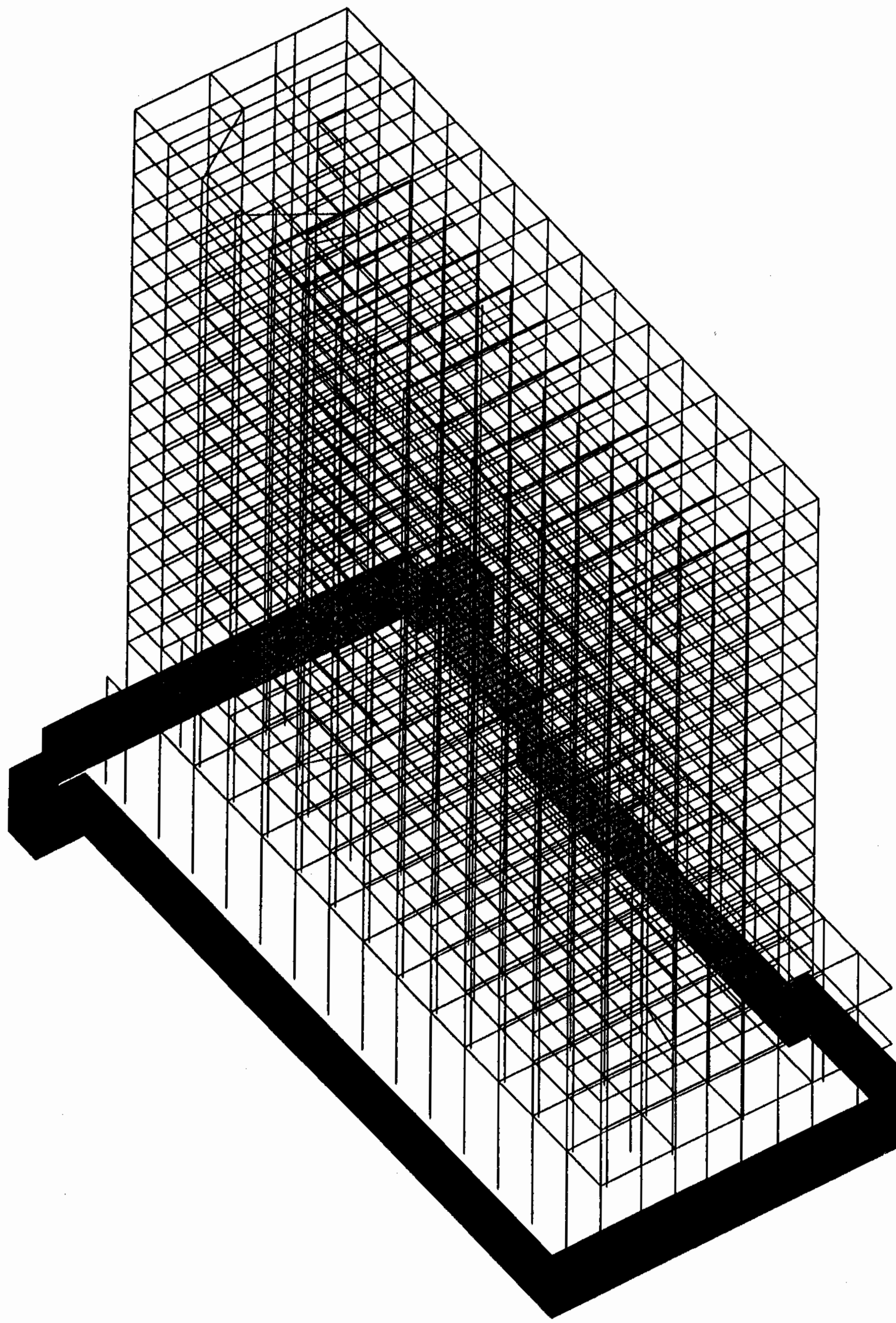


Figure 8 ETABS computer model

Both the FEMA 273 and Yield Stiffness models would be appropriate for a building subjected to a moderate level of ground shaking that induces sufficiently high stresses in the elements to cause cracking but not yield of the steel reinforcement.

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.

PREDICTED BUILDING RESPONSE

Using the nine ETABS models described in the previous section, eigenvalue analyses were performed to determine the natural frequencies and mode shapes for the models. Table 2 gives the periods of vibration and the percentage of participating mass in each direction for the first six modes of each model. As one would expect, as the stiffness of the beams, columns, and REZ's decrease, the periods of vibration tend to increase. Examination of these results also shows that there is a coupling between the fundamental North-South and Torsional modes of vibration. Figure 9 illustrates this coupling effect and the relatively uncoupled behavior of the East-West mode of vibration for the Gross Stiffness model with 50% REZ.

For the first three modes of vibration in the Gross Stiffness model with 50% REZ, the periods range between 2.27 and 2.08 seconds compared to the 2.15 and 2.21 second periods shown in Table 1 for the building response during the Whittier earthquake. Overall, the periods of vibration for the Gross Stiffness models fall within 15% of the periods shown in Table 1 for the Whittier earthquake. Comparison of the periods of vibration predicted by the analytical models and those given in Table 1 show that for the Northridge earthquake, the FEMA 273 Stiffness model with 100% REZ gives a good correlation, although the other FEMA 273 Stiffness models vary by only 25%. The Yield Stiffness models predict periods of vibration ranging between 25% and 40% of the values given in Table 1. Therefore, it can be said that the stiffness of the beams and columns during the Northridge earthquake is somewhere between the gross stiffness and the stiffness at first yield calculated from a moment-curvature analysis.

The nine ETABS models were subjected to the two components of ground acceleration (Channel 14 & 16) for the Northridge and Whittier earthquakes in a time history analysis. A sufficient number of modes were included in the analyses such that they account for at least 90% of the participating mass. As a benchmark consideration, 5% damping was assumed in the analyses for all modes, which is consistent with the damping estimated by Goel, et. al. for the fundamental modes of vibration during the Northridge earthquake.

Table 2 Periods of vibration and participating mass from ETABS models

(a) Gross Stiffness building model

Mode #	100% REZ Model				50% REZ Model				Centerline Model			
	Period	% Participating Mass			Period	% Participating Mass			Period	% Participating Mass		
		N-S	E-W	Rotation		N-S	E-W	Rotation		N-S	E-W	Rotation
1	2.11	48.6	0.3	19.3	2.27	44.7	0.6	22.9	2.43	40.9	1.2	26.0
2	1.98	21.3	2.4	45.4	2.13	24.3	7.5	37.8	2.30	19.8	32.8	18.0
3	1.90	0.1	67.8	2.3	2.08	1.1	62.6	6.4	2.26	9.4	36.9	23.1
4	0.71	10.5	0.0	2.2	0.77	10.1	0.0	2.4	0.83	9.7	0.0	2.6
5	0.65	1.2	1.6	8.2	0.71	1.1	4.0	5.5	0.77	0.8	7.2	2.5
6	0.63	0.1	9.2	1.8	0.69	0.4	6.7	4.1	0.75	0.9	3.5	6.8

* REZ = Rigid End Zones

(b) FEMA 273 Stiffness building model

Mode #	100% REZ Model				50% REZ Model				Centerline Model			
	Period	% Participating Mass			Period	% Participating Mass			Period	% Participating Mass		
		N-S	E-W	Rotation		N-S	E-W	Rotation		N-S	E-W	Rotation
1	2.81	40.3	0.6	27.2	3.03	36.5	1.1	30.4	3.25	33.0	2.3	32.7
2	2.64	29.4	4.5	35.8	2.86	24.1	28.5	17.8	3.11	8.9	60.6	1.0
3	2.57	0.7	64.9	3.9	2.82	9.8	40.6	18.9	3.04	28.5	7.5	33.3
4	0.95	9.3	0.0	2.7	1.03	9.0	0.0	3.0	1.11	8.7	0.1	3.2
5	0.87	1.5	2.4	6.6	0.95	1.1	6.0	3.3	1.04	0.7	8.5	1.2
6	0.85	0.3	8.3	2.4	0.93	0.9	4.6	5.5	1.01	1.4	2.0	7.3

* REZ = Rigid End Zones

(c) Yield Stiffness building model

Mode #	100% REZ Model				50% REZ Model				Centerline Model			
	Period	% Participating Mass			Period	% Participating Mass			Period	% Participating Mass		
		N-S	E-W	Rotation		N-S	E-W	Rotation		N-S	E-W	Rotation
1	3.12	28.7	4.8	35.5	3.38	14.1	23.1	31.8	3.67	2.4	55.5	11.4
2	3.04	34.1	26.9	10.1	3.33	19.0	43.2	7.1	3.60	22.5	13.1	33.0
3	3.01	9.2	37.4	22.7	3.27	38.8	2.9	29.6	3.52	46.9	0.7	24.0
4	1.10	9.9	0.1	1.3	1.19	9.6	0.2	1.5	1.29	8.3	1.3	1.6
5	1.06	0.3	8.8	1.9	1.17	0.4	10.1	0.6	1.28	1.5	9.4	0.0
6	1.03	0.4	2.5	8.5	1.13	0.5	1.1	9.6	1.23	0.5	0.6	9.8

* REZ = Rigid End Zones

The maximum displacement predicted by the analyses was compared to the maximum recorded displacements at each instrumented floor. The results of this comparison are shown in Table 3 and displayed graphically for the Northridge earthquake in Figure 10. The results from the FEMA 273 Stiffness model show that maximum displacement is predicted within 20% of the recorded displacement at each floor for the Northridge earthquake. Increased dispersion in the ratio between recorded and predicted maximum displacement is observed for the Gross Stiffness and Yield Stiffness models.

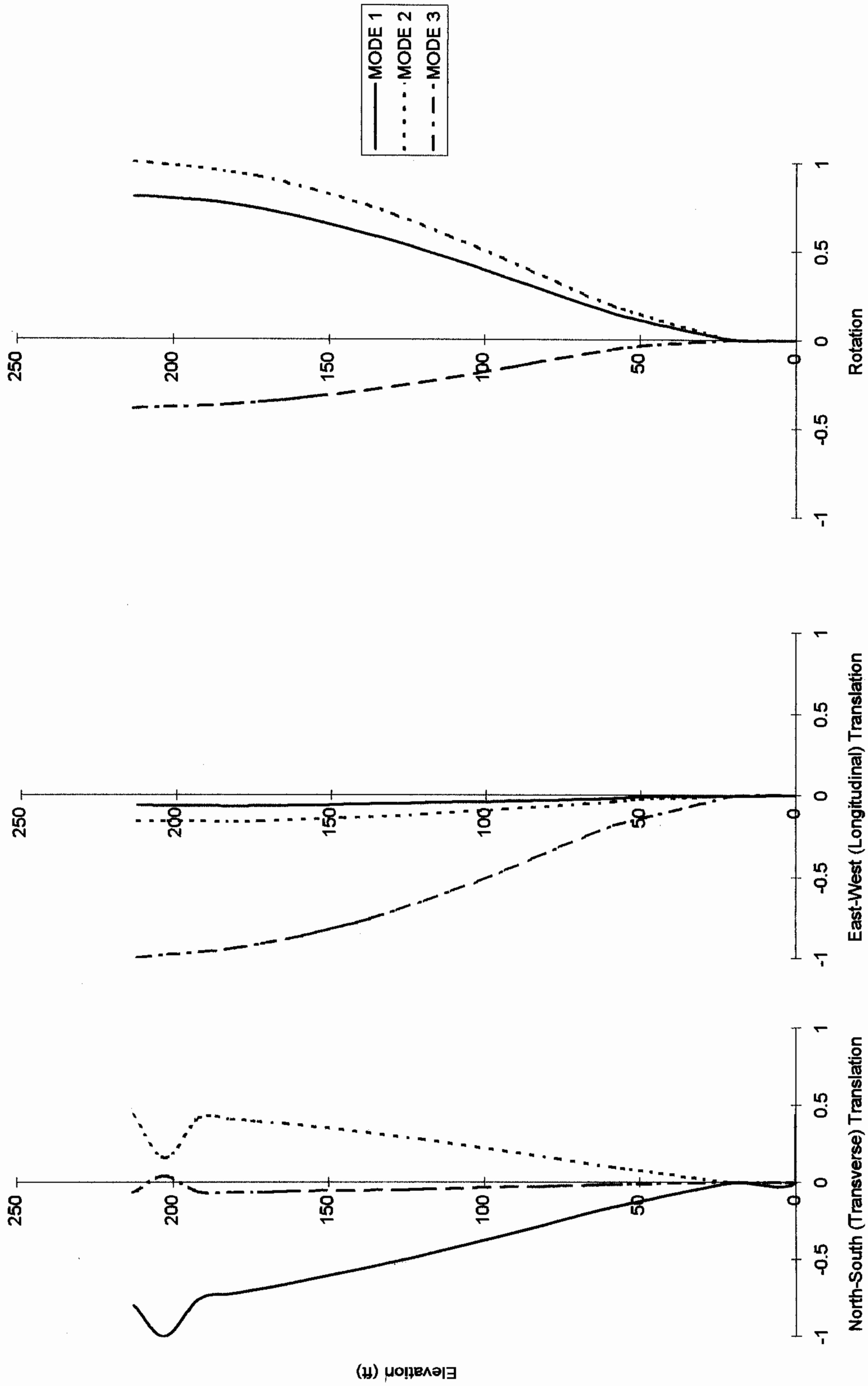


Figure 9 Normalized mode shapes for the Gross Stiffness model with 50% REZ

Table 3 Maximum recorded/maximum predicted floor displacement ratios

(a) 1994 Northridge earthquake North-South (Longitudinal) direction response

Floor	Recorded	Gross Stiffness Model			NEHRP Stiffness Model			Yield Stiffness Model		
	(in)	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline
Roof	8.67	1.00	0.99	0.94	1.03	1.15	1.29	1.25	1.20	1.33
16th	4.48	0.84	0.89	0.93	0.89	0.91	0.92	0.91	0.96	1.02
9th	3.13	0.74	0.79	0.90	0.85	0.82	0.91	0.89	0.92	0.86
3rd	1.52	0.92	1.01	1.12	1.04	1.01	1.10	1.06	0.72	0.68
Mean		0.87	0.92	0.97	0.95	0.97	1.05	1.03	0.95	0.97

* REZ = Rigid End Zones

(b) 1994 Northridge earthquake East-West (Transverse) direction response

Floor	Recorded	Gross Stiffness Model			NEHRP Stiffness Model			Yield Stiffness Model		
	(in)	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline
Roof	4.18	1.24	1.22	1.23	0.99	1.12	1.09	1.03	0.97	1.28
16th	3.35	1.20	1.26	1.39	0.93	1.09	1.08	1.16	1.13	1.42
9th	1.93	0.99	1.18	1.27	0.85	1.07	1.16	1.10	0.81	1.19
3rd	0.48	0.83	0.96	0.92	0.83	1.00	0.76	0.70	0.59	0.71
Mean		1.06	1.16	1.20	0.90	1.07	1.02	1.00	0.87	1.15

* REZ = Rigid End Zones

(c) 1987 Whittier earthquake North-South (Longitudinal) direction response

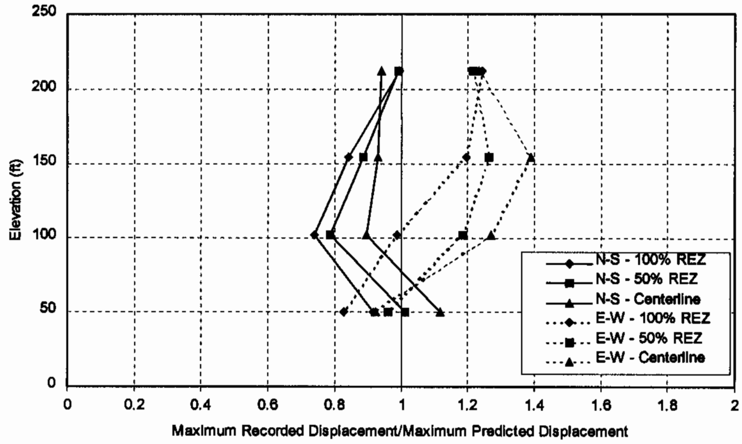
Floor	Recorded	Gross Stiffness Model			NEHRP Stiffness Model			Yield Stiffness Model		
	(in)	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline
Roof	0.65	0.78	0.84	0.71	0.92	1.10	1.03	0.98	1.03	1.18
16th	0.61	1.03	1.05	1.20	1.45	1.65	1.79	1.74	1.91	1.56
9th	0.46	1.00	1.07	1.35	1.39	1.21	1.39	1.35	1.18	1.00
3rd	0.08	0.57	0.50	0.47	0.38	0.53	0.44	0.42	0.38	0.40
Mean		0.85	0.87	0.93	1.04	1.12	1.17	1.13	1.12	1.04

* REZ = Rigid End Zones

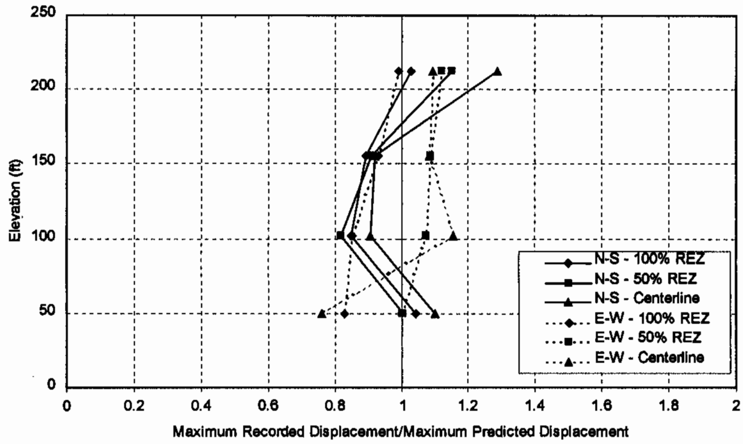
(d) 1987 Whittier earthquake East-West (Transverse) direction response

Floor	Recorded	Gross Stiffness Model			NEHRP Stiffness Model			Yield Stiffness Model		
	(in)	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline	100% REZ	50% REZ	Centerline
Roof	0.85	0.91	0.94	1.01	1.04	1.16	1.33	1.20	1.25	1.20
16th	0.64	0.83	0.82	0.86	1.23	1.39	1.45	1.39	1.49	1.60
9th	0.38	0.78	0.78	0.83	0.93	1.06	1.06	1.06	1.06	0.95
3rd	0.14	0.88	0.74	0.82	0.82	0.93	0.93	1.00	0.78	0.54
Mean		0.85	0.82	0.88	1.00	1.14	1.19	1.16	1.14	1.07

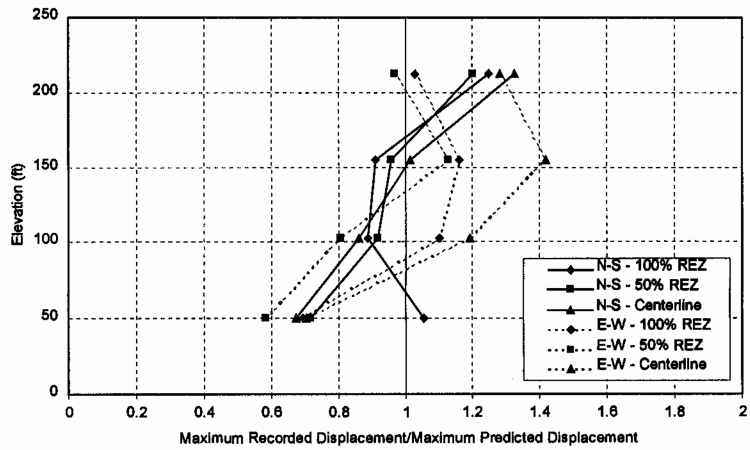
* REZ = Rigid End Zones



(a) Gross Stiffness building model



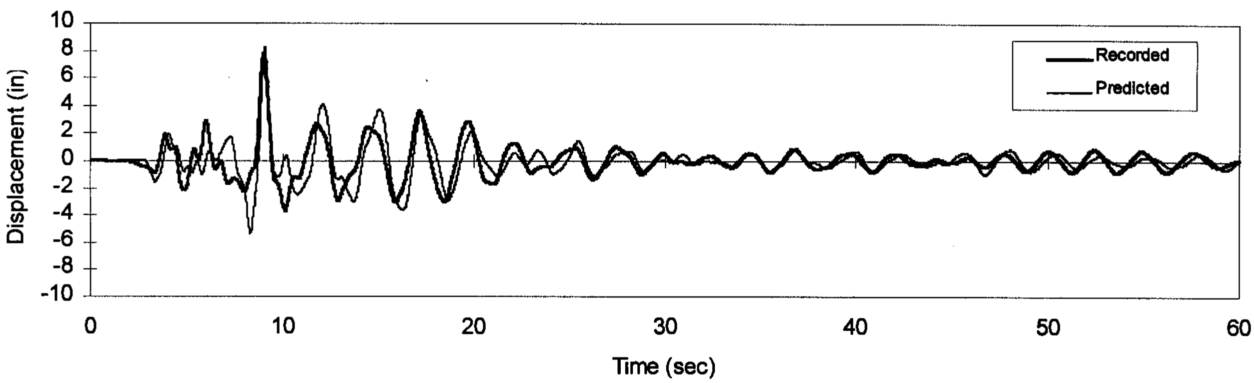
(b) FEMA 273 Stiffness building model



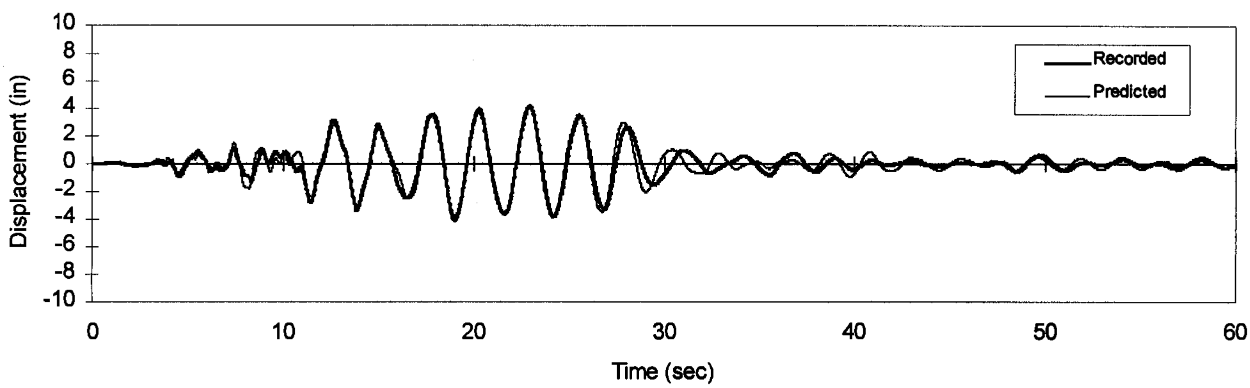
(c) Yield Stiffness building model

Figure 10 Maximum recorded/maximum predicted floor displacement ratios

The time history of relative roof displacement predicted by the FEMA 273 Stiffness model with 100% REZ is compared to the recorded relative displacement at the roof in Figure 11. Note that the peak displacement is accurately predicted and the predicted motion generally captures the frequency content of the recorded motion.



(a) North-South (Transverse)



(b) East-West (Longitudinal)

Figure 11 Relative roof displacement time history comparisons for the Northridge earthquake and the FEMA 273 Stiffness model with 100% REZ

CONCLUSIONS

The following conclusions can be drawn from the research reported in this paper:

1. The displacement response of the building and the damage observed following the Northridge earthquake is consistent with the Vision 2000 definition of the Operational performance level.

2. The FEMA 273 Stiffness model with 100% REZ gives a good correlation with the period of vibration and the maximum displacement response for the Northridge earthquake. In addition, the time history of displacement predicted by this model matches the peak displacement at the roof and the overall frequency characteristics of the motion.

3. The periods of vibration calculated using the Gross Stiffness model with 50% REZ are consistent with the building periods during the Whittier earthquake. The predicted maximum displacement response was within 5% of the recorded displacements at the 16th and 9th floor, but fails to accurately predict the displacement response at the Roof and the 3rd floor.

ACKNOWLEDGEMENTS

Funding for this research was provided by the State of California, Department of Conservation, Division of Mines and Geology, Strong Motion Instrumentation Program (SMIP). The authors wish to acknowledge the assistance and commitment of Dr. Marshall Lew, Dr. Farzad Naeim, Prof. Gerard Pardoen, Dr. Tom Sabol, Dr. Roger Li, Dr. Roy Lobo, Ms. Julia Li, Ms. Mary Elise Richter, and Mr. Insuck Song through the course of this research. Dr. Moh Huang of SMIP was essential in obtaining the recorded motions and the structural/architectural plans for this building. The additional efforts of Ms. Mary Elise Richter in the preparation of this paper are greatly appreciated.

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