SEISMIC PERFORMANCE OF FOUR INSTRUMENTED STEEL MOMENT RESISTING BUILDINGS DURING THE JANUARY 17, 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

This paper presents a summary of our comprehensive evaluation of the seismic performance of four instrumented steel moment resisting frame buildings during the 1994 Northridge earthquake. The buildings were inspected and repaired, where necessary, according to the requirements of the FEMA-267 Interim Guidelines [FEMA, 1995]. The basic premise of performance based seismic engineering is the ability to predict performance given the base earthquake ground motion and building characteristics. These four buildings provided a perfect vehicle to compare the current status of analytical abilities versus this basic requirement for achieving a meaningful performance based design. To this end, not only we developed and studied numerous linear and nonlinear, static and dynamic computer models of these buildings, but we evaluated the relevant provisions of the leading traditional as well as performance based codes and standards. As this paper indicates, we are not far from the ability to understand, model, and explain global performance of structures. However, we are farther away from accurate prediction of location and extent of local damages within a structure. It is hoped that future work by us and other researchers will remedy this shortcoming in the near future.

INTRODUCTION

Our purpose of this study was to evaluate seismic performance of these four instrumented buildings, compare the observed performance with those predicted by model codes and guidelines, and where applicable suggest modifications to the prevailing analysis and design techniques and code provisions. In this paper, we contrast the results obtained by evaluation of the actual performance of these buildings with the design requirements embodied in the UBC-97 Code [ICBO, 1997] and FEMA-273 Guidelines [FEMA, 1997].

The four instrumented Los Angeles SMRF office buildings evaluated in this study were chosen by CSMIP, they are: (1) a 20 story building located in Encino, (2) a 10 story building in Tarzana, (3) an eight story building in North Hollywood, and 4) a 16 story building located in Sherman Oaks. The buildings were inspected after the Northridge earthquake for potential damage using the procedures outlined by SAC Interim Guidelines [FEMA 267, 1995]. The structural engineers who inspected and/or repaired each building were consulted. Their observations were documented and the structural plans and their repair drawings and reports were acquired.

The Northridge earthquake strong-motion recordings, obtained from the sensors installed at the buildings, were obtained from CSMIP. The accelerations were recorded in by tri-channel accelerometers (two horizontal components and one vertical) located on the ground, at mid level and on the roof of each building. It is worthy of mentioning that the instruments at various floors of these buildings were not time-synchronized. That is, the zero time for the sensors located at various floors did not exactly coincide (because it takes time for the seismic waves to reach the upper floors from the ground level). Interestingly, however, this proved not to be a significant problem in engineering evaluation of the seismic response for these buildings.

ANALYTICAL MODELS

Numerous analytical models for each building were created for linear and nonlinear, static and dynamic analyses. The models were calibrated by comparing the recorded building response with the response obtained from analysis. The comparisons included, but were not limited to, building periods, drifts, higher-mode response, time histories and known location of damaged joints, if any. The observed building performances as evidenced by post earthquakes investigations and recorded by installed instruments were compared with analytical results and code provisions. The most calibrated model for each building was used for these comparisons. In comparisons with UBC-97, we used elastic demand capacity ratios, demand plastic moment ratios, inter-story drifts, redundancy requirements and other UBC-97 special provisions. In application of the FEMA-273 guidelines, the desired performance objective was assumed to be the Basic Safety Objective. We used the nonlinear static procedure for this evaluation. The buildings were pushed to the desired target displacement and the plastic rotations were compared with the acceptance criteria associated with that performance level.

The general procedure utilized for construction and calibration of various computer models of the buildings is shown in Figure 1. The primary lateral resisting system for all the buildings consists of Special Moment Resisting Frames (SMRF). One of the buildings had shear walls and cross braces at the lower levels which are also modeled as part of the lateral resisting system. We evaluated the contribution of the gravity framing to the lateral stiffness for one of the buildings (the North Hollywood building). The difference in the responses with and without the inclusion of the gravity framing was minimal (between 2% to 5%). Furthermore, the response of calibrated analytical models for other three buildings, without explicit inclusion of the gravity framing, matched the recorded response very well both in the frequency and time domains. Therefore, there was no need for explicit inclusion of the gravity framing in computer models of these buildings.

Initially, we created two distinct three-dimensional computer models of each building using the SAP2000 computer program [Computers and Structures, 1997]. The only difference between these two models was in the way the beam-column panel zone was defined. For the first model (Model 1) we assumed the beam-column panel zone to be fully rigid. In the second model (Model 2) we assumed no rigid end zones for beams (column-center to column-center length). Using the recorded ground motions from the Northridge earthquake, we performed a series of elastic time-history analyses and compared the analytical results with the recorded responses. Based on these comparisons, we constructed a third, best fit model (Model 3) using calibration techniques.

We investigated the influence of vertical ground motion on seismic response of the Tarzana building since this building was subjected to the most severe vertical ground motion in the group of buildings studied. Our analysis indicated the effects of vertical ground motion to be insignificant for this building in terms of increases in both stresses and displacements. Therefore, the effects of vertical ground motion were not included in performance analysis of other buildings.

For three of the four buildings (Tarzana, North Hollywood and Sherman Oaks), the precise locations of the seismographs/accelerometers were unavailable. Therefore, they were initially assumed at the center of mass of the respective floors. We evaluated the torsional components of the response obtained from our analytical models and found them to be insignificant. That is, there was no appreciable difference in the displacements in a particular direction measured at the corners of the building. The exact amount of torsional displacements experienced by these buildings during the earthquake could not be determined from the instrumented response because there was only one sensor located at a floor and the torsional component of the response was not explicitly recorded. The symmetrical configuration of all four buildings and the excellent agreement achieved between the recorded and analytical responses, however, strongly suggests that torsion was not a significant contributor to seismic response of these buildings during the 1994 Northridge earthquake.

We compared the acceleration, velocity and displacement responses for Model 1 and Model 2 with the recorded responses. If the predominant period of the building as interpreted from the recorded time-histories, throughout the duration of response, was bounded by the predominant periods of Models 1 and 2, the response was considered as essentially elastic. The reason is that in this case the predominant period of the building through the duration of response could be matched by adjusting the effective rigidity of the beam-column panel zones. The amplitudes then could be matched by adjusting modal damping values. This approach was used via a series of iterative schemes for development of a best fit model or Model 3. In contrast, if the recorded response indicated that the building was more flexible (even during just a portion of the response duration) than Model 2, a nonlinear model was required. This is due to the fact that in such cases the recorded response could not have been matched merely by adjusting the rigid end zones.

We examined the results from the time history analysis for Model 3 subjected to Northridge earthquake input ground motion with respect to the following parameters:

1. zones of high demand as suggested by analysis versus post earthquake inspection results.
2. Sensible estimates of effectiveness of the beam-column rigid zones based on connection details utilized in each building.
3. The values of equivalent modal damping needed to match analytical and recorded response amplitudes, and
4. Sensitivity of models to minor modeling variations.

The expected, rather than nominal, yield strengths were used for calculation of member capacities throughout. These are 1.5 times the yield strength for A36 steel and 1.15 times the yield strength for A50 steel [AISC 1997].

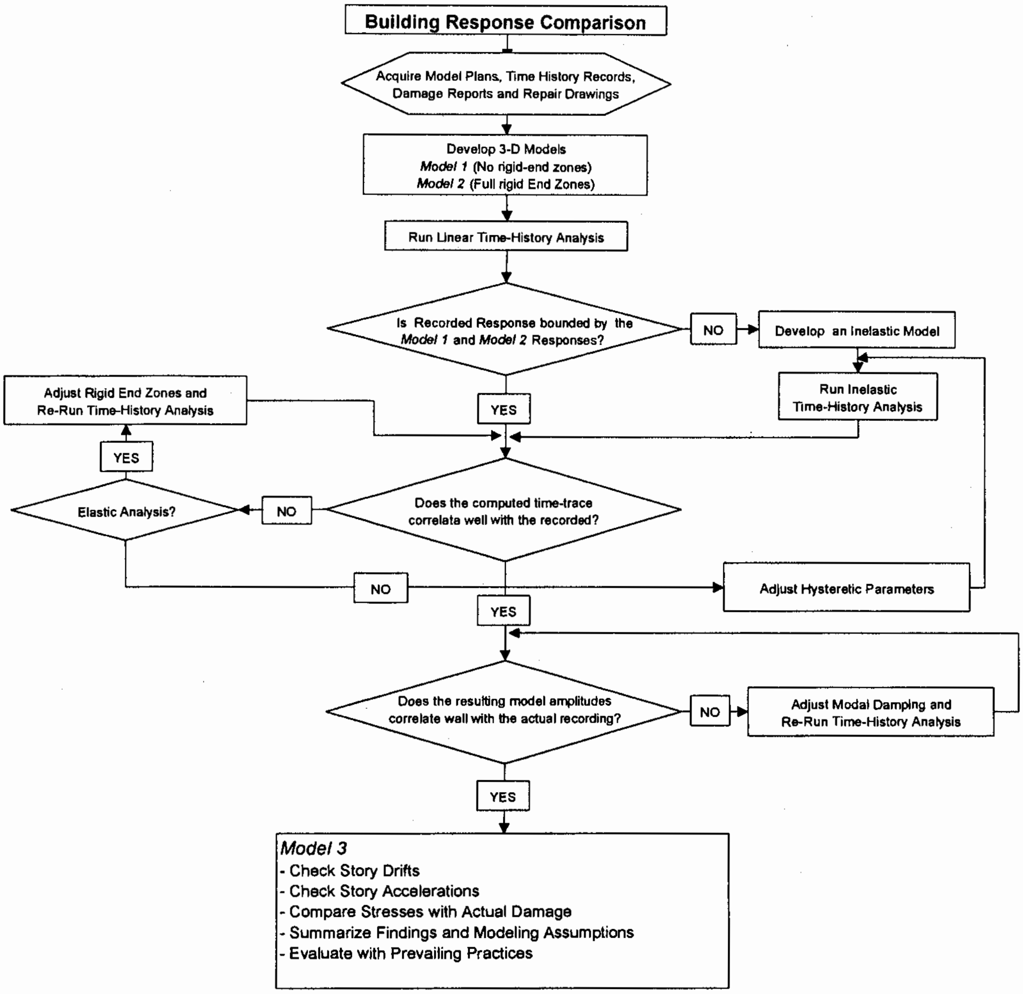


Figure 1. Procedure for Construction and Calibration of Analytical Models.

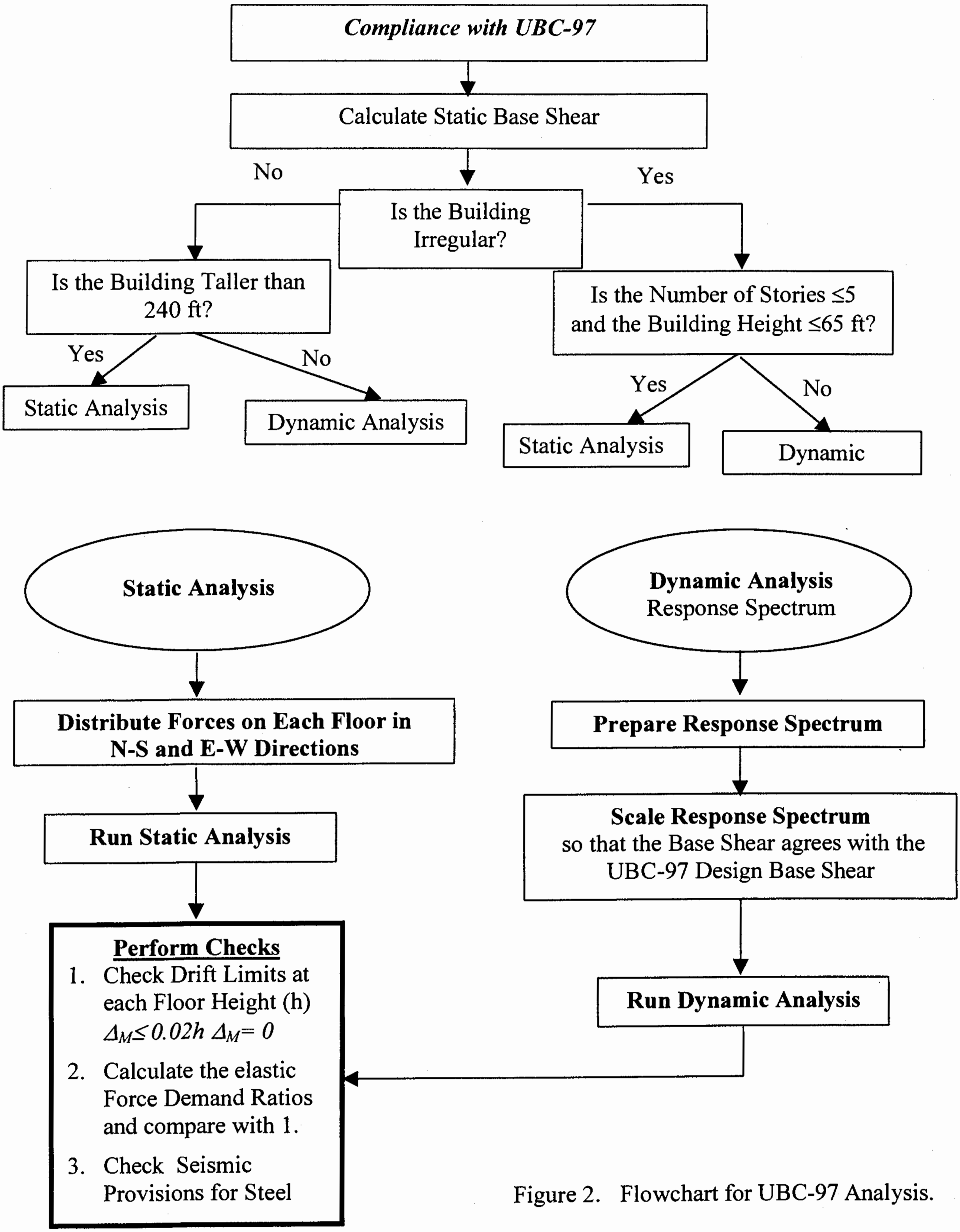


Figure 2. Flowchart for UBC-97 Analysis.

Design Risk/Performance Category

⇩

Analysis Options

a. Elastic-Static or Dynamic

→ **b. Non-Linear Static (Pushover)** ←

c. Non-Linear dynamic

⇩

Mathematical Model

- Modeled Lateral Resisting System of the Buildings (Model 3)

⇩

Lateral Load Patterns Applied

1. Uniform Distribution $F_X = \frac{m_X}{\sum m_i} V$
2. Linear Distribution $F_X = \frac{m_X h_X^k}{\sum m_i h_X^k} V$
3. Modal Distribution Response Spectra, Modal Analysis/Forces

⇩

Push to Target Displacement

$\delta_t = C_0 C_1 C_2 C_3 S_a (T_e^2 / 4\pi^2) g$ for BSE-1 and BSE-2

⇩

Acceptance Criteria

All Hinges must Meet BSO, Which is Life Safety for BSE-1 and Collapse Prevention

⇩ **YES** | **NO** ⇩

Building Meets FEMA-273 Criteria | **Building Does Not Meet FEMA-273 Criteria**

Figure 3. Flowchart for FEMA-273 Analysis

We analyzed and compared the status of all four buildings with respect to the UBC-97 and FEMA-273 requirements. This comparison can serve several purposes:

- Establishment of the relation between the demand imposed by the ground motion experienced during the 1994 Northridge earthquake to code level demands that buildings are to be designed for and withstand
- Identification of the relevance and relative importance of various codified detailed design procedures in seismic performance of existing structures
- Evaluation of reliability, or lack thereof, of codified procedure in identifying the weak links in the structure and the zones of high demand
- Comparing relative accuracy of specific, calibrated, analyses for actual earthquake ground motions to generic codified analyses in terms of ability to identify potential zones of damage
- Identification of those code provisions that are either over-conservative or under-conservative and development of alternate, more accurate, code provisions.
- Identification of areas that need improvement to accelerate realization of effective and reliable performance based design alternatives.

Flowcharts showing the general procedure of code comparisons are presented in Figures 2 and 3 for UBC-97 and FEMA-273 , respectively.

The idealized force deformation relation for each element defining its acceptance criteria is shown in Figure 4. All the elements in the model are primary members. The acceptance criteria for the beams and columns are taken from Table 5-4 of FEMA-273. The maximum plastic rotations corresponding to the Life-Safety (LS) and Collapse Prevention (CP) requirements as well as the plastic hinge properties are calculated as a function of the width-thickness ratio (b/t) of each section under the guideline specifications

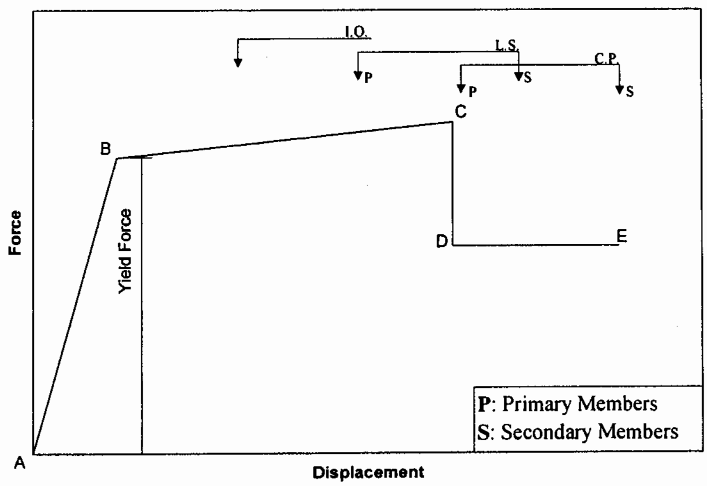


Figure 4. Component or Element Deformation Limits According to FEMA 273.

THE ENCINO BUILDING

This building consists of a twenty-story tower and an attached four-story parking garage. The structure of the tower is rectangular in plan and over 249 feet tall. The lateral resisting system of the building is provided by Steel Moment Resisting Frames (SMRF). The floor slabs consist of 5 in. thick light-weight concrete with no metal deck. The four story parking structure is offset to the east of the tower, but shares a continuous floor diaphragm with it. The parking structure has its own SMRF system. The parking structure has two four-bay moment frames in the North-South and East-West directions, respectively. The tower has four four-bay frames in the North-South direction and two seven-bay frames in the East-West direction over the first four floors. The frames are cutback by one bay on the upper floors in the North-South direction. There is a bank vault in the Southwest corner of the tower with four concrete shear walls at the first floor and two X-braces on the second floor.

A view of our three-dimensional model showing the lateral resisting system is presented in Figure 5. A plan view of the building showing the lateral frames, corresponding gridlines and column orientations may be seen in Figure 6. The beam-column connections are typical pre-Northridge SMRF connections. Continuity plates exist only on the upper five floors of the tower (floors 16 to 20). There are four non-prismatic girders per floor that are located on gridlines 4 and 7 and span from lines A to D and E to G. The rolled steel sections are Grade A572 steel and the plate girders, used for the non-prismatic members, are made of A36 steel. Tri-channel seismic sensors are located in the basement (arcade) level, 10th floor, and roof.

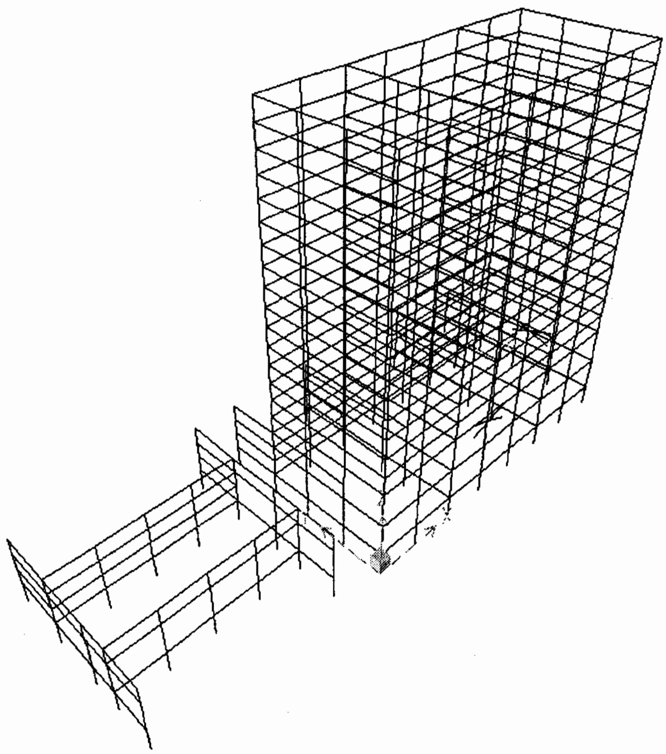


Figure 5. Three-dimensional Model of the Encino Building.

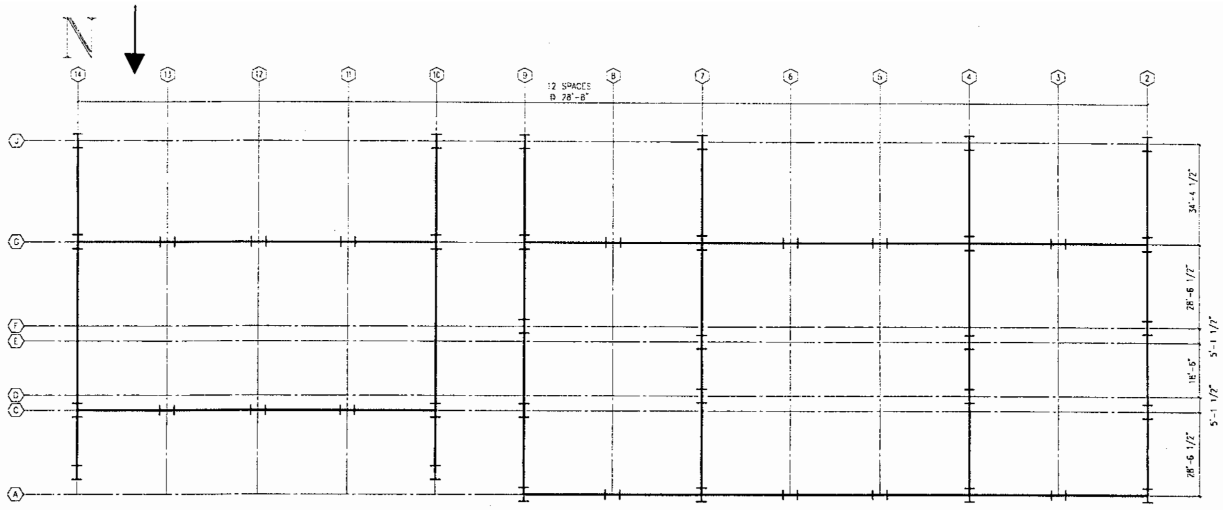


Figure 6. Plan View of Seismic Frames and Gridlines for the Encino Building.

The assumed damping levels and member end rigid zone factors used for various analytical models are shown in Table 1. Our best-fit model corresponded to 95% effective rigid end-zone for columns and 100% for beams.

Table 1. Rigid End Zone Factors and Modal Damping in the Models.

Model	Rigid End Zone Factors	Analysis	Modal Damping
Model 1	All Elements=100%	Elastic 3D	NS: 3% 1st ,3rd ,and 6th, all others 10% damped EW: 2.5% 1st and 5th, all others 10% damped
Model 2	All Elements=0%	Elastic 3D	
Model 3	<u>All Beams</u> 100 % <u>Columns</u> Moment connection in 2 directions 3 and 4 way=100% Corner=95% Moment connection in 1 direction With Continuity Plates=5% Without Continuity Plates=2.5%	Elastic 3D	

A comparison of the periods interpreted from recorded data by from the transfer function analysis and our best-fit model for the first three modes of vibration are given in Table 2. The largest percent difference is only 11.3%. This closeness indicated that the vibration periods of the computer model correlated well with the periods of the actual building.

Table 2. Comparison of Vibration Periods for *Model 3* and the Periods for the Recorded Response using the FFT Method.

North-South				East-West			
Mode	**Modal Periods (SAP2000)** *(sec)*	**Modal Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*	**Mode**	**Modal Periods (SAP2000)** *(sec)*	**Modal Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*
1	2.754	2.596	5.7	2	2.530	2.242	11.3
4	0.967	0.937	3.1	5	0.930	0.931	1.1
7	0.573	0.531	7.3	8	0.558	0.538	3.6

The recorded ground motions at the arcade level of this building are presented in Figure 7.

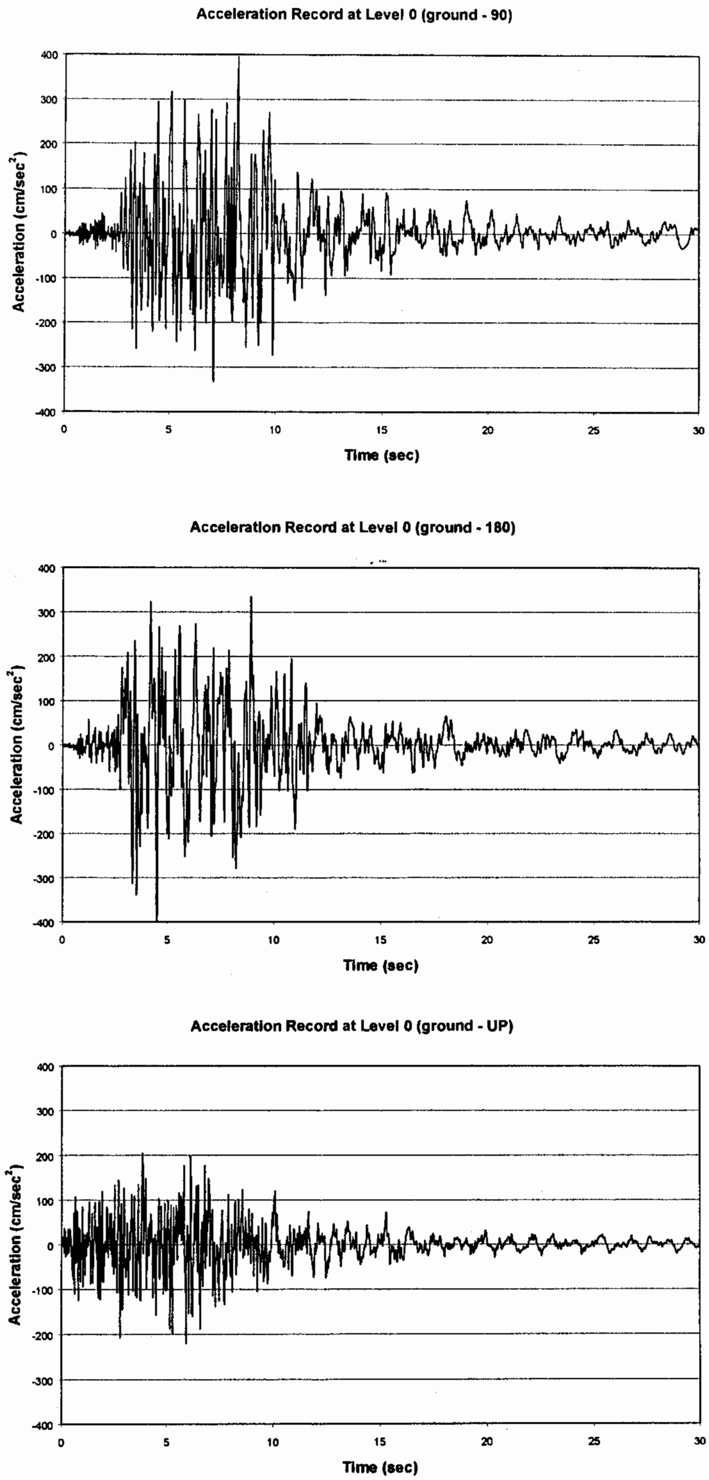


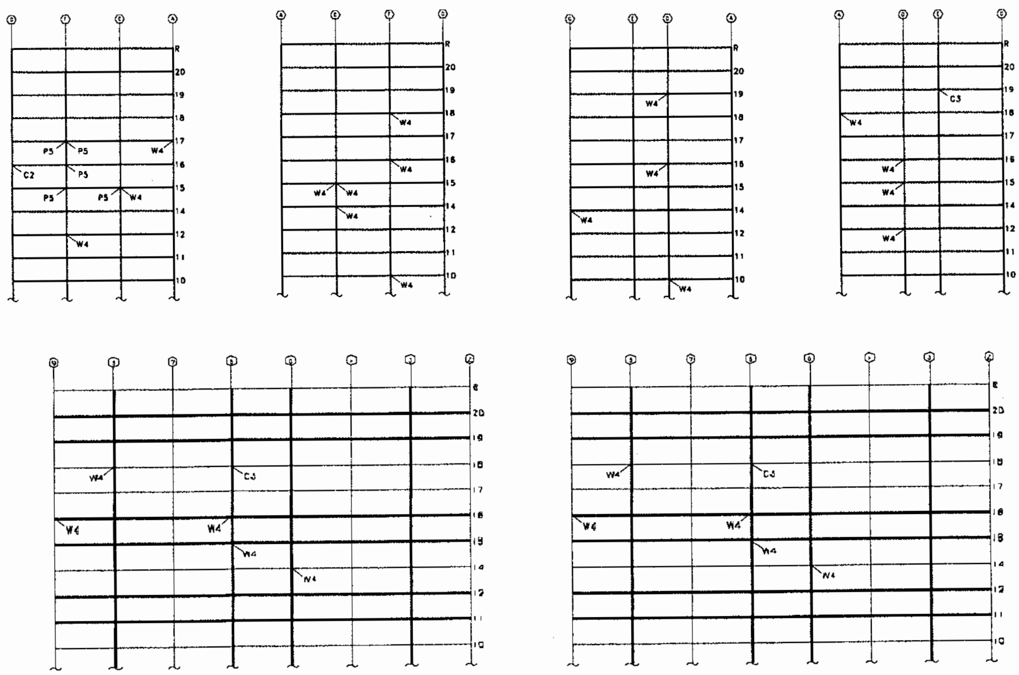
Figure 7. Recorded Northridge Ground Motions at the Arcade Level for the Encino Building.

After the Northridge earthquake, the building moment connections were inspected for damage. A summary of the different types of damage found and the corresponding (SAC) identifications are given in Table 3. The overall damage statistics is given in Table 4. The type 1D damage was small enough that it was not repaired. Therefore, the 1D type damage is excluded from any of the comparisons made in this study. Figures 8A through 8D show the different locations of observed damage on the moment resisting frames. Note that not all connections were inspected following the Northridge earthquake. The most severe damage was experienced in the columns on the North-South lateral frame along line 2, where the crack propagated all the way into the column web.

Table 3. Identification of Damage.

ID Name	SAC Identification Definitions
1D-W4	Light beam flange weld cracking
C2	Column flange damage: Complete flange tear out from beam flange weld
C3	Column flange damage: Partial cross-flange crack in HAZ
P5	Column Web Damage: Partial depth cracking originating from cracked col. Flange
W4	Beam Flange Damage: Crack at column interface (in weld)

(A) Damage on Gridlines 2 and 9 (B) Damage on Gridlines 4 and 7



(C) Damage on Gridlines A (D) Damage on Gridlines G

Figure 8. Damage revealed by inspection of the Encino Building.

Table 4. Summary of Damaged Connections.

East-West				North-South			
Member	Total Number of Connections	Number of Damaged Connections	Ratio (%)	Member	Total Number of Connections	Number of Damaged Connections	Ratio (%)
Columns	512	0	0	Columns	256	5	1.9
Beams	448	11	2.5	Beams	384	19	5

Responses computed by our best-fit model are contrasted with those experienced by the building during the Northridge earthquake in Figures 9 to 11. The accuracy of the analytical model in capturing the essence of building response is clear. There are some spikes in the analytical acceleration responses that are probably due to higher modes. Increasing the damping in the higher modes decreases the amplitudes of these spikes. These higher modes are probably highly damped in the actual structure as evident from of the recorded acceleration responses. The calculated displacement response amplitudes in the North-South direction show some departure from the recorded response from about 15 to 25 seconds. This is probably due to some inelastic response in this range. This, however, did not significantly affect the overall stiffness of the structure. Additional damping by yielding of some of the non-structural elements could have also resulted in the difference in the responses. Although the North-South direction experienced some inelastic behavior, the difference in the elastic response of the computer model to the inelastic behavior of the building is very small. In addition, there are no noticeable differences in the East-West direction response between the analytical and recorded response, although there was damage.

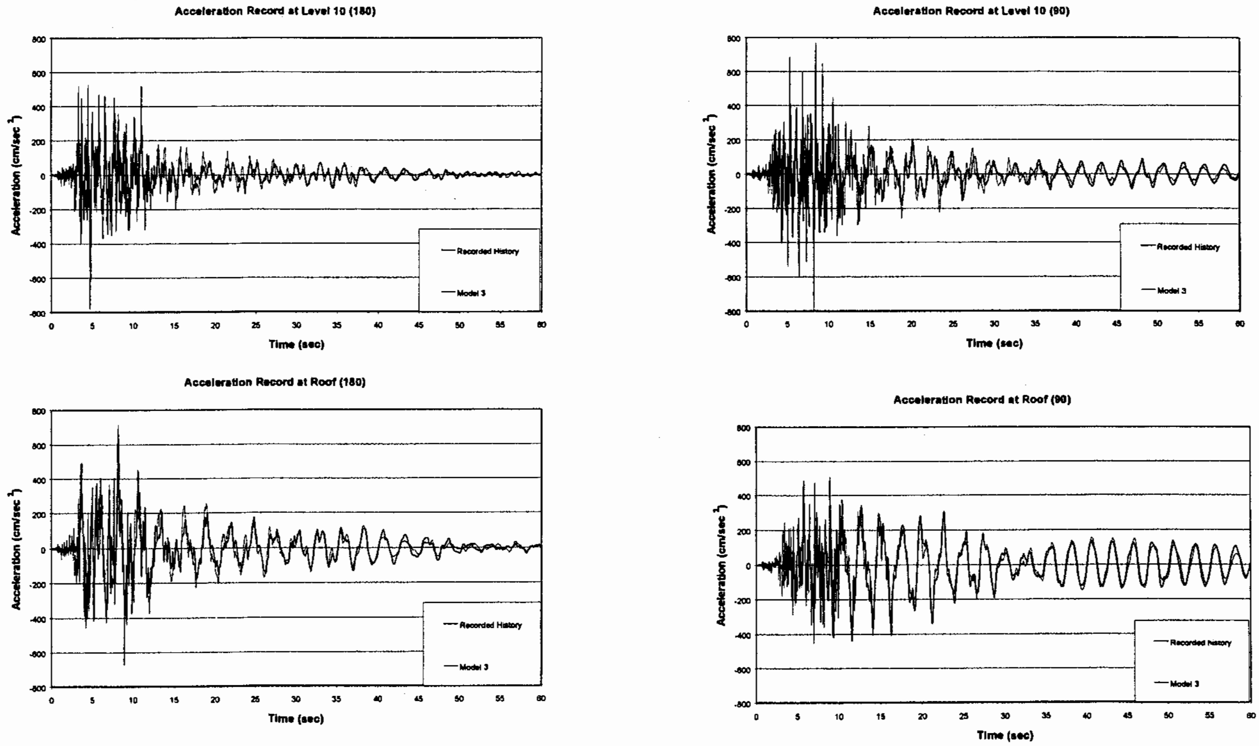


Figure 9. Acceleration Time Histories for the Encino Building.

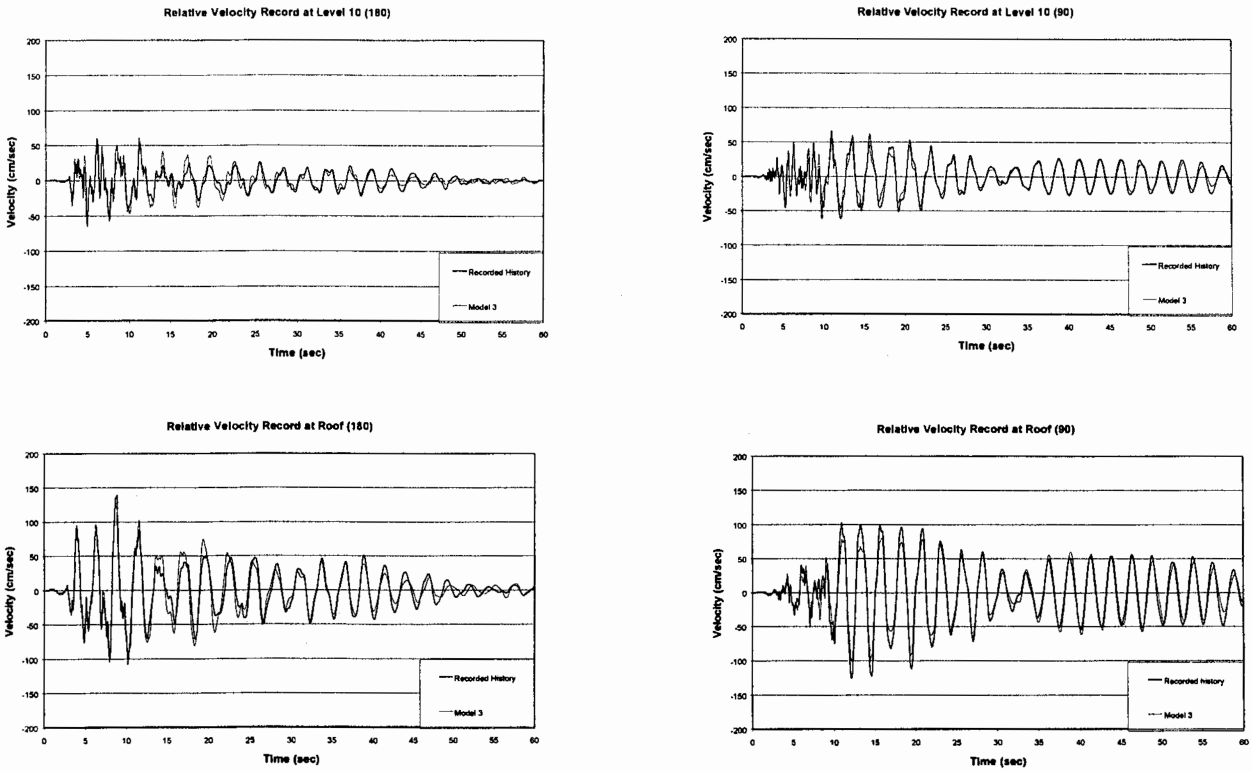


Figure 10. Velocity Time Histories for the Encino Building.

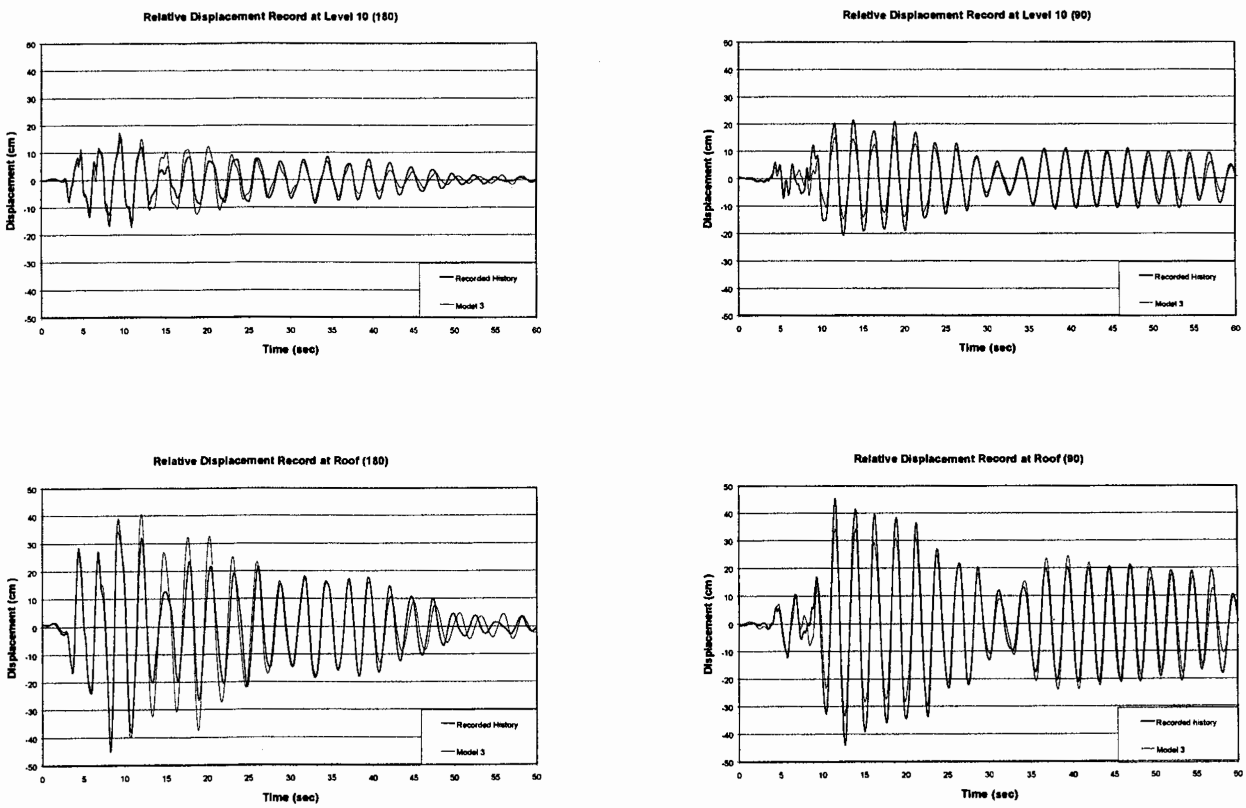


Figure 11. Displacement Time Histories for the Encino Building.

The Elastic Demand Ratios (EDR) are calculated from the Time History analyses in order to determine if the zones of excess correlate with the observed pattern of damage to the building. This is one way of establishing how effective, or ineffective, our analytical indicators of damage are for prediction of likely damages to this SMRF building. These EDR are calculated using the Load and Resistant Factor Design (LRFD) method with capacities for the EDR calculated using the expected yield strengths. Columns on lines 2, 4, 7, 9 which are common to seismic frames in each direction are the only members that have EDR greater than unity. No damage was observed in these columns. These columns had to resist biaxial moments, driving up the calculated demands. EDR calculations in time history analysis are inherently conservative. This is due to the fact that in using EDR formulas, the maximum actions (i.e., flexure or axial load) throughout the time history response are used while it is obvious that such maximums do not necessarily concurrent in time. The greater the number of actions combined to calculate EDR, the more exaggerated this conservativeness may become. That is why, calculated EDR for columns subjected to biaxial bending may be larger than other columns not because of higher actual demand but due to the bias in the conventional way EDR is calculated. For this reason, we also use a second damage indicator, Demand-Capacity Ratio (DCR). This second indicator, although is unconservative because it does not consider flexure-axial load interaction, does not suffer from the EDR bias mentioned above. When considered together, EDR and DCR can effectively bound the actual demand ratio imposed on a member.

The damaged members had ratios below unity and not appreciably higher than the undamaged members. For example, the EDR corresponding to the severely damaged column (F-2 on the 16th floor) is 0.67 and the undamaged column (F-9 on the 16th floor) on the opposite side of the building had a ratio of 0.64. Furthermore, both of these ratios are well below the values commonly attributed to onset of significant plastic deformations. For Example, the Tri-services Manual [Department of Defense; 1986] considers EDR below 2.0 to be an indicator of essentially elastic behavior. The EDR are unable to predict, with any degree of certainty, damage to the Encino building. This is primarily due to premature failure of structural members at seismic demands significantly lower than what was predicted by design assumptions. None of the demand capacity ratios exceeded unity, which is another indicator that if members had performed as expected no damages should have occurred. This, obviously, contradicts the actual state of the building after the Northridge earthquake.

UBC-97 analyses indicated that the building satisfied UBC-97 strength requirements but failed the code drift requirements at the upper 15 floors. The building also passed UBC-97 panel zone thickness requirements and continuity plate requirements. Strong column weak girder provision was satisfied everywhere except at the roof.

FEMA-273 push-over analysis predictions where in close agreement with the results of dynamic time history analyses. As with the time history analysis, this method was unable to pin-point the exact location of damaged joints. In our opinion, this is not a weakness of the method but the premature failure of the joints which was responsible for the lack of coherency. The FEMA-273 push-over curves identifying the demand and capacity curves, BSE-1 and BSE-2 target displacements, as well as the base shear and displacements observed during the Northridge earthquake are shown in Figures 12 and 13.

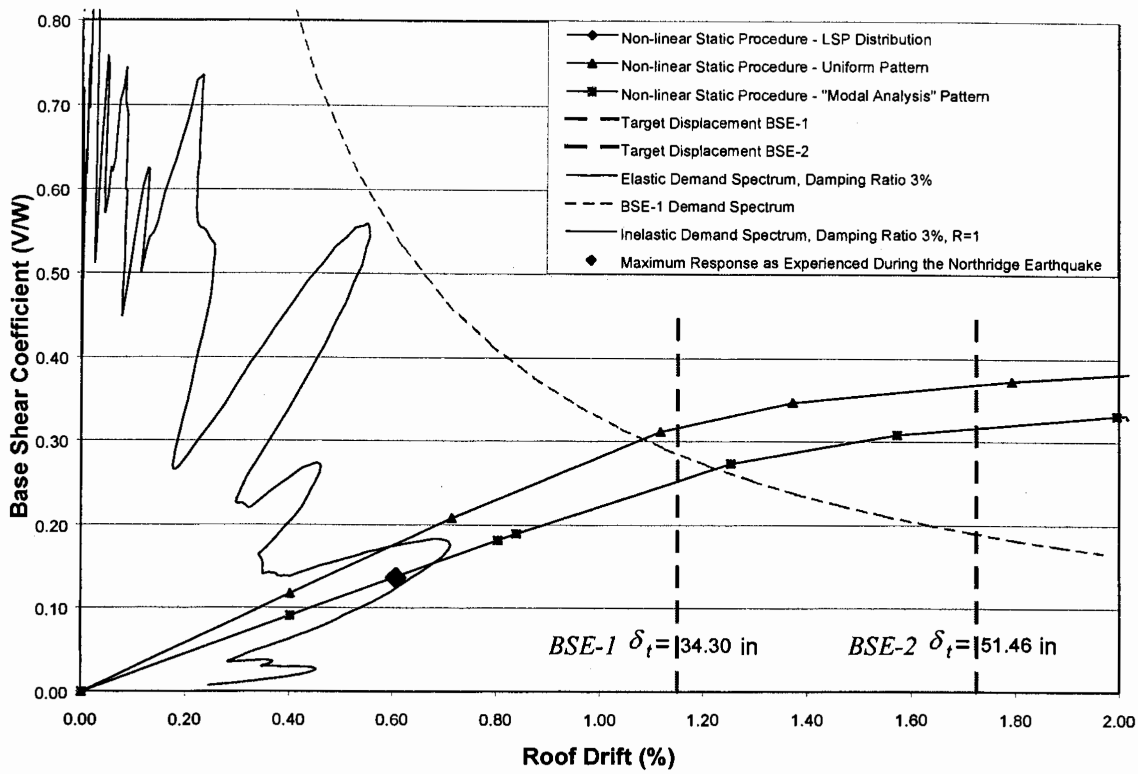


Figure 12. Demand-Capacity Spectrum for the North-South Direction of the Encino Building.

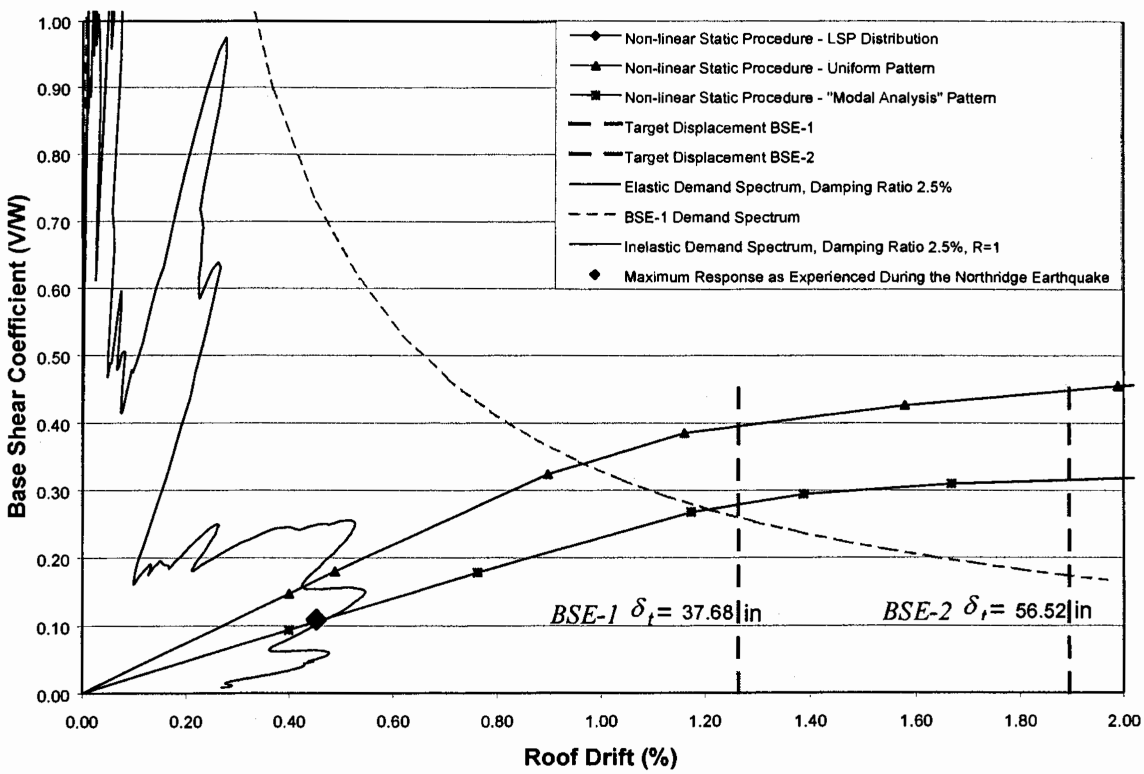


Figure 13. Demand-Capacity Spectrum for the East-West Direction of the Encino Building.

THE TARZANA BUILDING

The lateral resisting system of this ten-story Steel Moment Resisting Frame (SMRF) office building consists of three moment resisting frames in the North-South direction and six in the East-West direction. All beam-column connections in the building are moment resisting. The column spacings are 30 ft. on center. The floor plan of the lateral resisting system and the column orientations are shown in Figure 14. The first floor height is 16 ft. The remaining stories are 13 feet tall. The columns typically have continuity plates, but no doubler plates and use typical pre-Northridge rigid connections. The beams are made of A36 structural steel and columns are Grade A572 (Grade 50). The floor system consists of a 6¼" thick composite metal deck (a 3", 24-gauge metal deck, overlaid by 3¼" lightweight concrete).

The seismic sensors for this building are located at the base, the fifth floor, and the roof. The exact locations of the sensors were not identified. This did not prove to be a major issue in our investigation of this building. A sketch of the computer model of this building identifying the locations of discovered damages is shown in Figure 14.

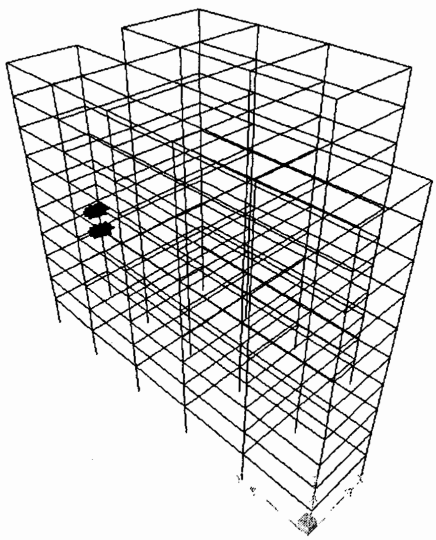


Figure 14. Computer Model of the Tarzana Building with Damaged Joints Identified.

The recorded response of this building indicated noticeable inelasticity. Therefore, our best-fit model for this building is a nonlinear model with hysteretic degradation characteristics. The hysteretic model chosen for this study is the multi-linear model, similar to that referred to in previous versions of IDARC2D as the three-parameter model [Kunnath et. al. 1992]. The member capacity is defined by a moment-curvature envelope with positive and negative moment capacities equal to the plastic moment of the section. We set the post yield stiffness at 3% of the riginal elastic stiffness. The stiffness degradation used is severe (see Table 5) with the value of parameter α (Figure 15) set to 1.7 in the East-West model and 3.0 in the North-South model. No strength degradation ($\beta=0$) or slip ($\gamma=1$) is considered for the hysteretic parameters, because this best simulates the observed behavior. The best results were obtained by assuming no end-rigid zones in the East-West direction and fully rigid end zones in the North-South Direction. Rayleigh damping was used and the best fit model reported 3% damping in the East-West direction and 7% in the North-South direction, respectively. The effective periods of vibration from the nonlinear analytical model are compared to those from data interpretations in Table 6. The recorded earthquake ground motions at the base of the building are shown in Figure 16. We created a separate computer model to investigate the effects of the vertical ground motion. In this model, we divided the floor slabs into individual panel elements with the masses assigned per unit volume. The individual floor slabs supported between frame grids were subdivided into a four by four grid system to ensure sufficient vertical degrees of freedom to capture the vertical effects of the ground motion. Our study showed that the change in the peak lateral displacements due to vertical effects to be insignificant. In fact, the time history responses using the vertical excitation were almost indistinguishable from the responses using only the horizontal excitations. There was a maximum increase of 6% in the Demand Capacity Ratio (DCR) of the beams at the mid-span, with an average difference of only 0.18%. At the beam-ends, there was only 2% difference in the DCR, with an average difference of 0.17%.

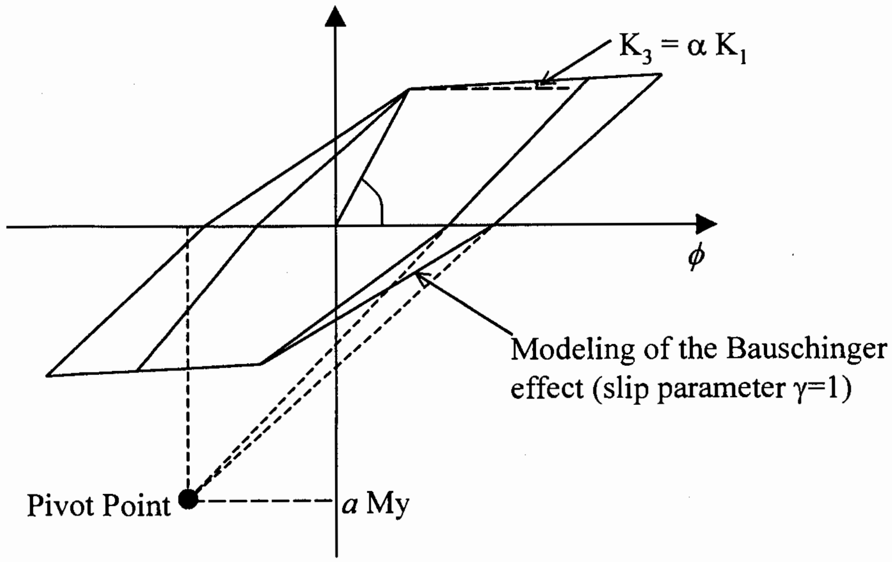


Figure 15. Hysteretic Model Used for this Study.

Table 5. Suggested Hysteretic Parameters for the Multi-linear Model.

PARAMETER	SLIGHT	MODERATE	SEVERE
Stiffness Degradation - α	2000	10	2
Strength Degradation - β	0.4	0.8	1.5
Slip - γ	$\cong$ 1	0.5	0.2

Table 6. Comparison of the Periods for Model 3 with the Periods from the Recorded Response for the Tarzana Building.

East-West				North-South			
Mode	**Vibration Periods (IDARC2D)** *(sec)*	**Vibration Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*	**Mode**	**Vibration Periods (IDARC2D)** *(sec)*	**Vibration Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*
1	2.349	2.272	3.24	1	2.153	2.222	3.11
2	0.796	0.833	4.48	2	0.721	0.75	3.87
3	0.458	0.602	1.07	3	0.412	Not identified	

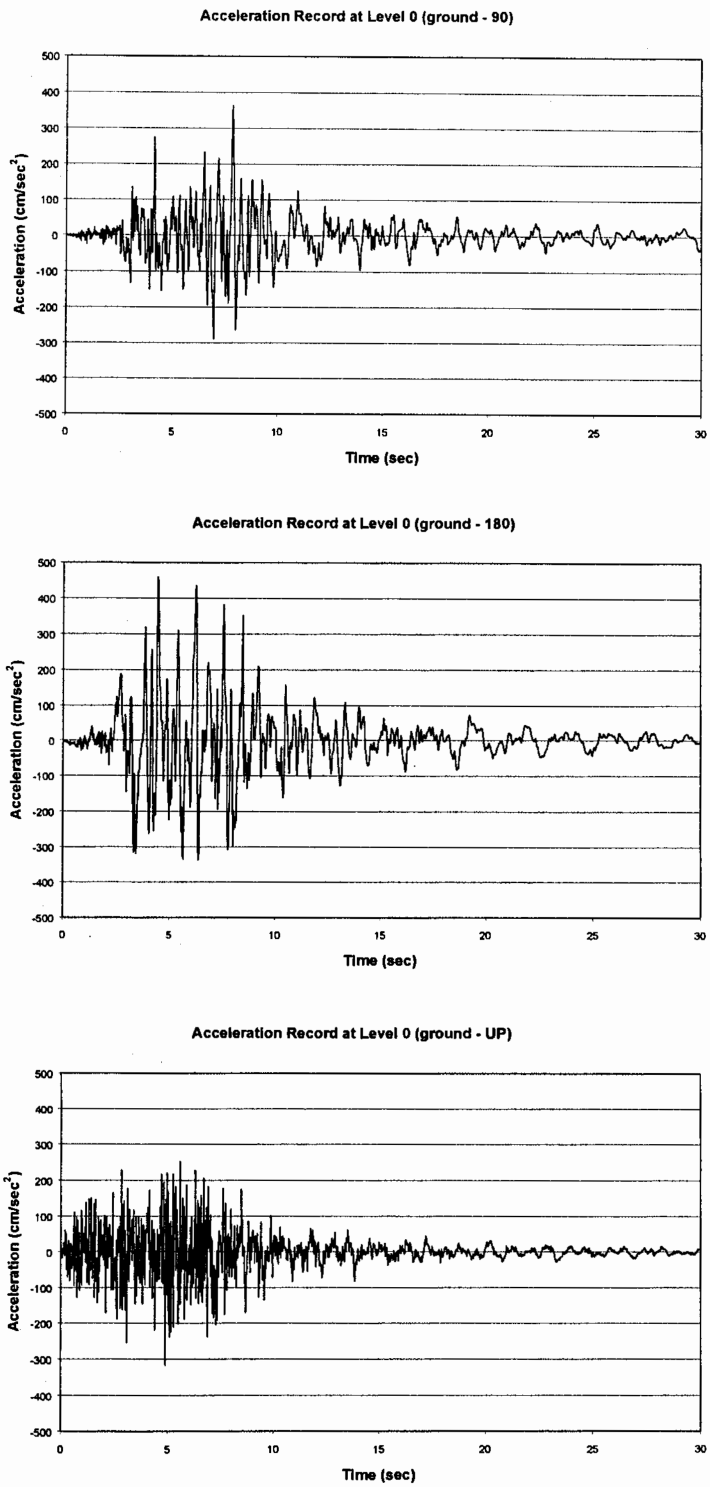


Figure 16. Recorded Northridge Ground Motions at the Base of the Tarzana Building.

The observations made with respect to reliability of EDR and DCR as well as UBC-97 analysis predictions apply equally as well to this building. All these procedures predict damage at locations where no damage occurred. Once again the FEMA-273 push-over analysis results provided a very accurate estimate of the overall response of the structure although the exact locations of the plastic hinges predicted by the push-over analyses did not correspond to the actual damage locations.

The nonlinear analytical response is compared to the observed response in Figure 17. The FEMA-273 push-over curves identifying the demand and capacity curves, BSE-1 and BSE-2 target displacements, as well as the base shear and displacements observed during the Northridge earthquake are shown in Figures 18 and 19.

The building passed the panel zone thickness, continuity plate, and strong column weak girder requirements of the UBC-97 code.

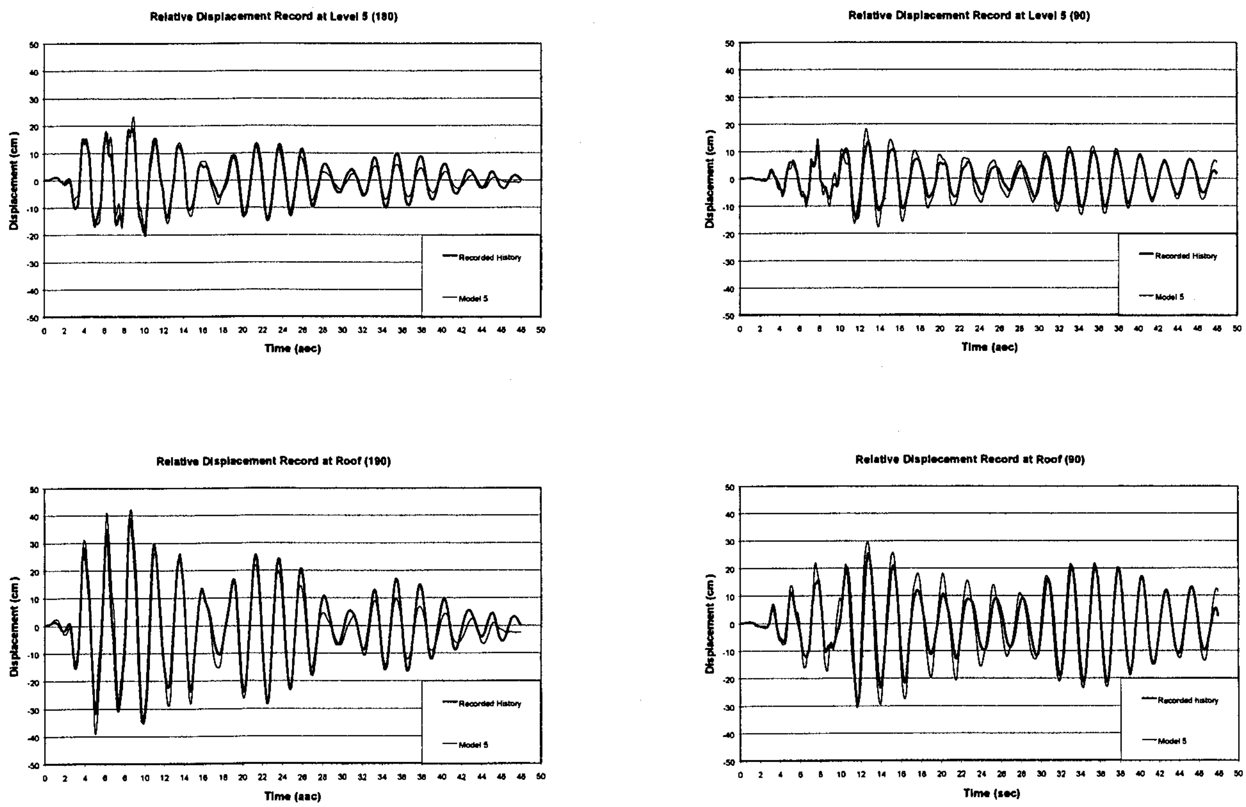


Figure 17. Displacement Time Histories from Northridge Earthquake for Model 3.

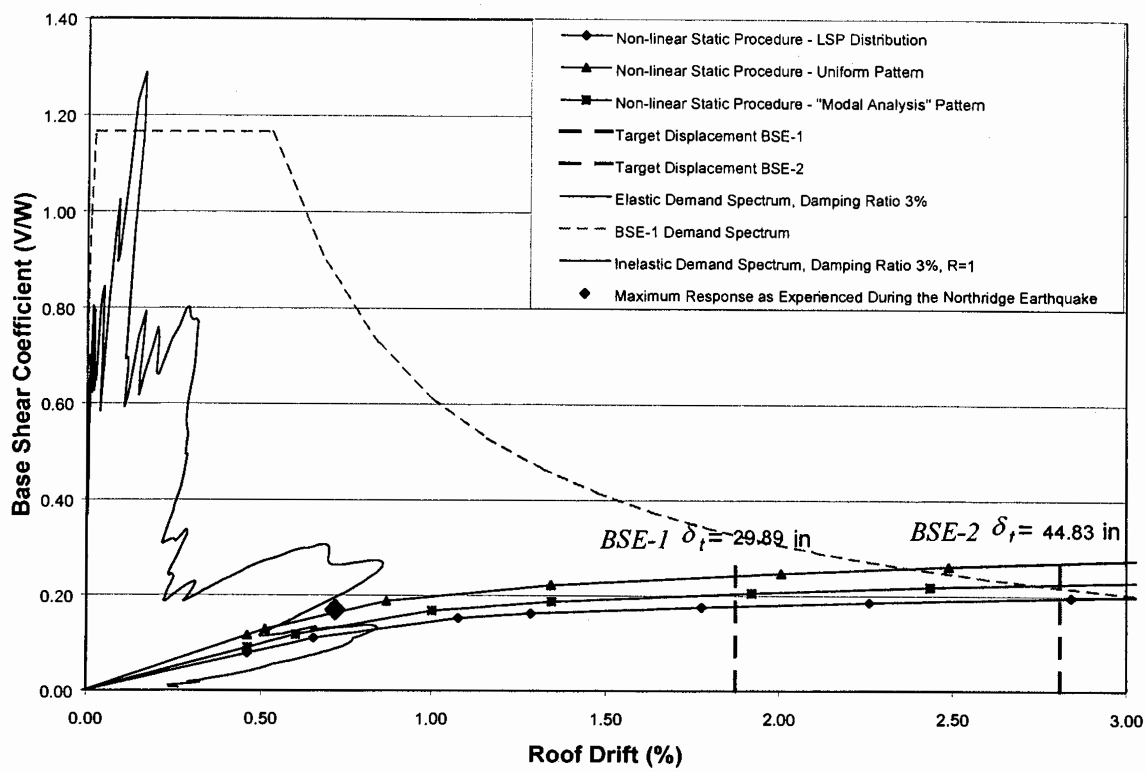


Figure 18. Demand-Capacity Spectra for the East-West Direction of the Tarzana Building.

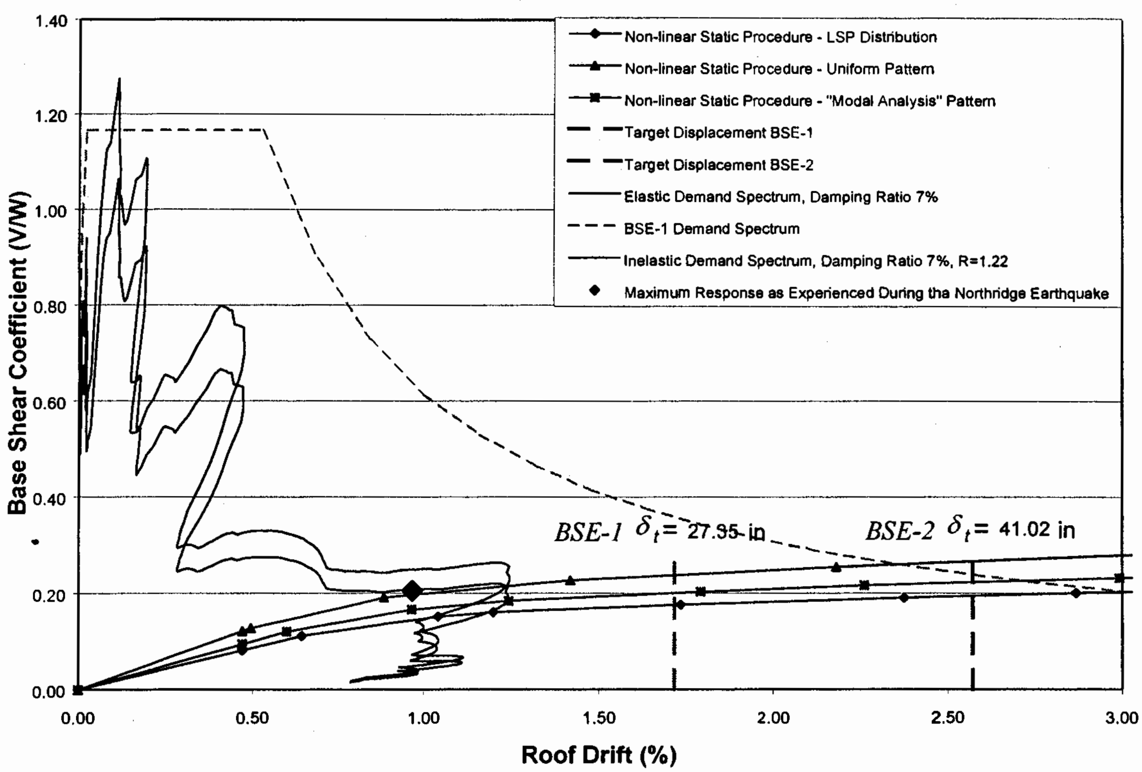


Figure 19. Demand-Capacity Spectra for the North-South Direction of the Tarzana Building.

THE NORTH HOLLYWOOD BUILDING

The lateral resisting system of this eight-story Steel Moment Resisting Frame (SMRF) office building consists of three moment resisting frames in the North-South direction and six in the East-West direction. It is rectangular in plan, with approximate plan dimensions of 71 ft. by 192 ft. The lateral resistance in the North-South direction is provided by four single bay moment resisting frames along the centerline. In the East-West direction, two bay moment resisting frames at the North and South edges of the building provide the lateral resistance.

The beam column connections are typical pre-Northridge SMRF connections. The structural steel used is either Grade A36 or Grade A572 (Grade 50) as specified on the structural plans. The floor system at all floors except the roof is composed of QL-99-20 steel deck overlaid with 3¼" lightweight concrete. The roof is a combination of a QL-99-20 steel deck overlaid with 3¼" lightweight concrete and a TUFCOR 24 GA. metal deck with 2¼'' zonolite. Seismic sensors are located at the base, the fifth floor and the roof. The exact locations of the sensors were not identified in the documentation we were able to obtain, but this proved not to be of a practical concern for this building.

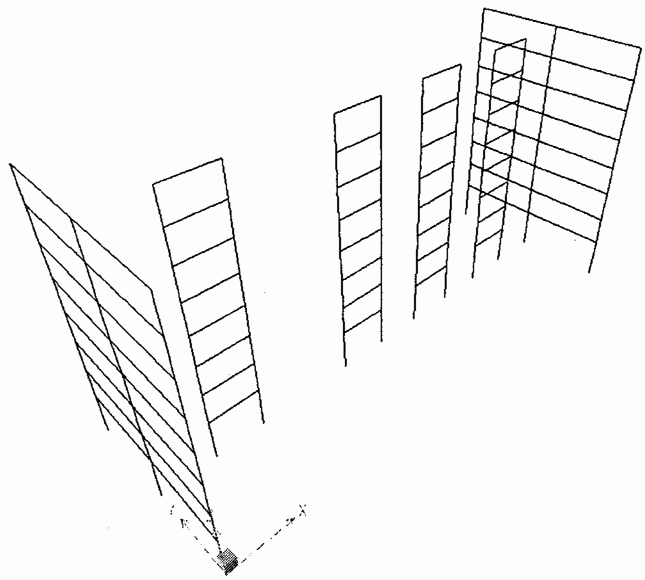


Figure 20. Three Dimensional Model of the North Hollywood Building.

The effectiveness of the beam-column rigid end zones for our best-fit model (*Model 3*) is calibrated at 80% of the full rigid zone length for the East-West direction frames and 85% for the North-South direction frames. The damping ratios used for the fundamental periods of *Model 3* were set at 5% for the East-West direction and 4% for the North-South direction. All higher modes are damped at 10% so that the contribution of the high frequency response in the acceleration time histories is not overstated. A summary of the modeling assumptions is presented in Table 7.

Table 7. Modeling Differences Between the Various Models for the North Hollywood Building.

Model	Rigid Zones	Analysis	Yield Stress (*ksi*)	Modal Damping
Model 1	All Elements	Elastic 3D	-	EW: 5% 1st, all others 10% damped NS: 4% 1st, all others 10% damped
Model 2	None	Elastic 3D	-	
Model 3	80 % EW 85% NS	Elastic 3D	-	

Model 3, has a fundamental period of 2.57 seconds in the East-West and 2.19 in the North-South direction. The natural periods of the building were also interpreted from the recorded response to the Northridge earthquake using the transfer functions of the story accelerations with respect to input base motion in the frequency domain . The periods calculated using this method match well with the periods obtained from the modal analysis for *Model 3* (see Table 8).

Table 8. Comparison of the Vibration Periods for Model 3 and the Periods Obtained from the Recorded Response Using the Fast Fourier Transform (FFT) Method.

East-West				North-South			
Mode	Vibration Periods (SAP2000) (*sec*)	Vibration Periods (FFT Analysis) (*sec*)	Diff. (%)	Mode	Vibration Periods (SAP2000) (*sec*)	Vibration Periods (FFT Analysis) (*sec*)	Diff. (%)
1	2.565	2.6	1.3	2	2.189	2.111	3.6
4	1.028	1.039	1.1	5	0.746	0.771	3.2
7	0.583	0.540	7.3	8	0.412	Not Identified	

The earthquake ground motions recorded at the base of the building during the 1994 Northridge earthquake are shown in Figure 21. The results for the best-fit model (*Model 3*) are compared to the recorded response if Figures 22 and 23. The time history signatures closely follow the recorded responses. Therefore, its appears that there was little inelasticity in the North Hollywood building and its behavior was essentially elastic during the Northridge earthquake.

There are 92 moment resisting frame connections in the North Hollywood building, 64 of which are in the North-South and 28 in the East-West direction. This building was inspected for damage after the earthquake with 11 connections tested in the North-South direction and 6 in the East-West direction using visual and ultrasonic examination. The inspection results showed no detectable defects or damage caused by the earthquake. Essentially similar conclusion may be drawn from the displacement responses obtained by our best-fit elastic model which coincides with the real recordings. The observations made with respect to reliability of EDR and DCR as well as UBC-97 analysis predictions apply equally as well to this building. All these procedures predict damage at locations where no damage occurred. Once again the FEMA-273 push-over analysis results provided a very accurate estimate of the overall response of the structure although the exact locations of the plastic hinges predicted by the push-over analyses did not correspond to the actual damage locations. The FEMA-273 push-over curves identifying the demand and capacity curves, BSE-1 and BSE-2 target displacements, as well as the base shear and displacements observed during the Northridge earthquake are shown in Figures 24 and 25.

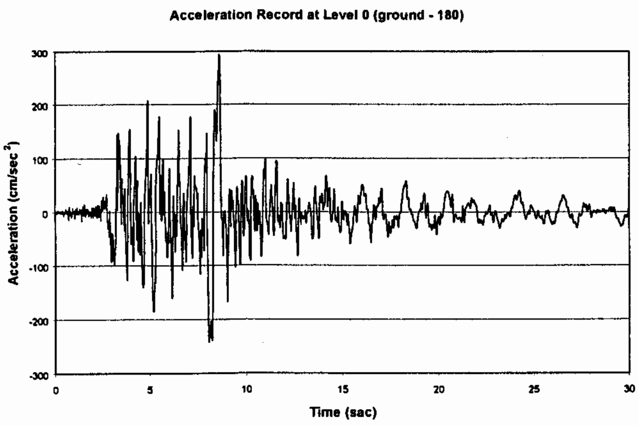
A summary of analytical indicators of this building's response is presented in Table 9.

Table 9. Summary of the North Hollywood Building Analysis.

	Northridge Earthquake		
	Observed Damage?	**Elastic Demand Ratios (Model 3)**	**Design/Capacity Ratios (Model 3)**
Remarks	No –Elastic Response--	Ratios >1 in Beams and Columns	Ratios >1 in Beam and Column Structural Elements

	UBC-97					
				Special Provisions		
	EDR	**Drift Limits**	**Redundancy Factors**	**Panel zones**	**Continuity Plates**	**Column-Beam Moment Ratios**
Compliance	No EDR>1 in Several Elements	No Limit Exceeded at all Levels	**>1.25** Exceed Code Limitations	**OK**	**OK** Provided where needed	No Failed to pass the test on the top 2 floors

	FEMA-273		
	Life Safety-BSE-1	**Collapse Prevention-BSE-2**	**Demand-Capacity Spectra**
Compliance	**OK**	Failed in East West Direction	Elastic Behavior.



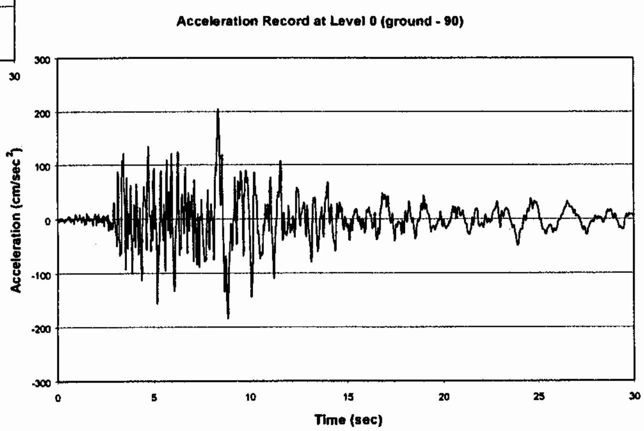


Figure 21. Recorded Northridge Ground Motions at the Base of the North Hollywood Building.

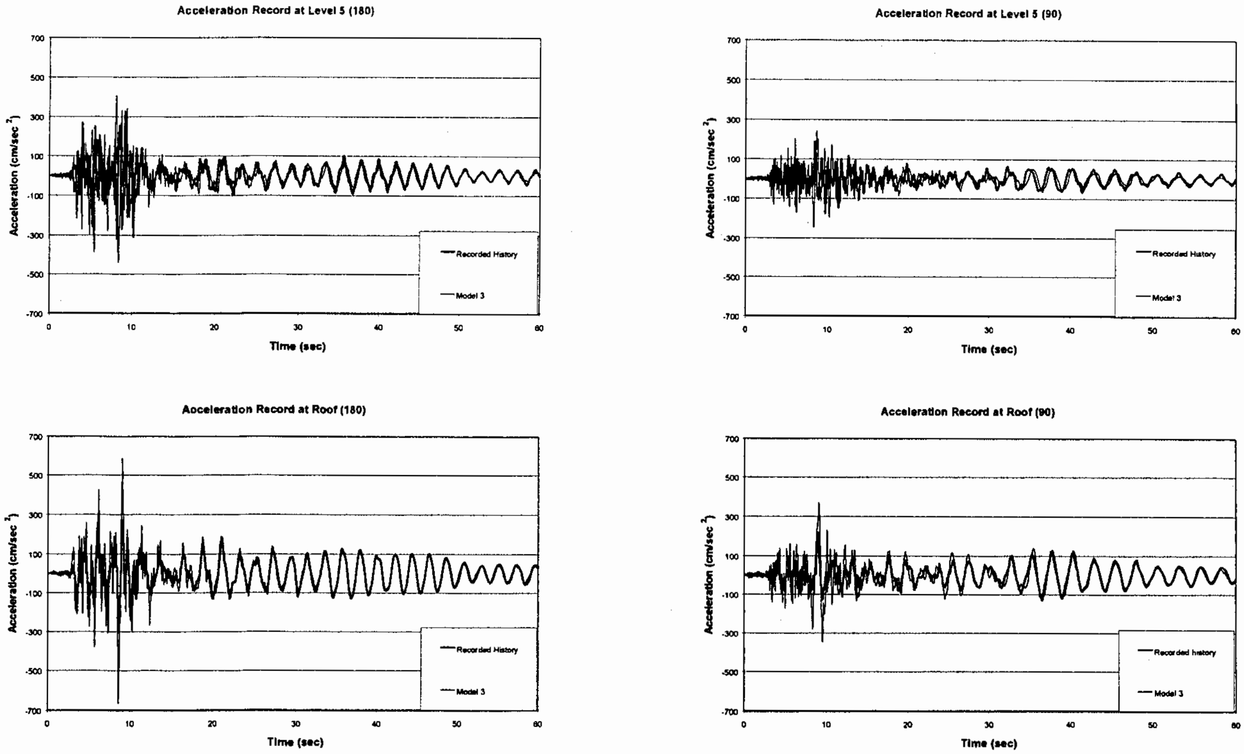


Figure 22. Acceleration Time Histories from Northridge Earthquake for Model 3.

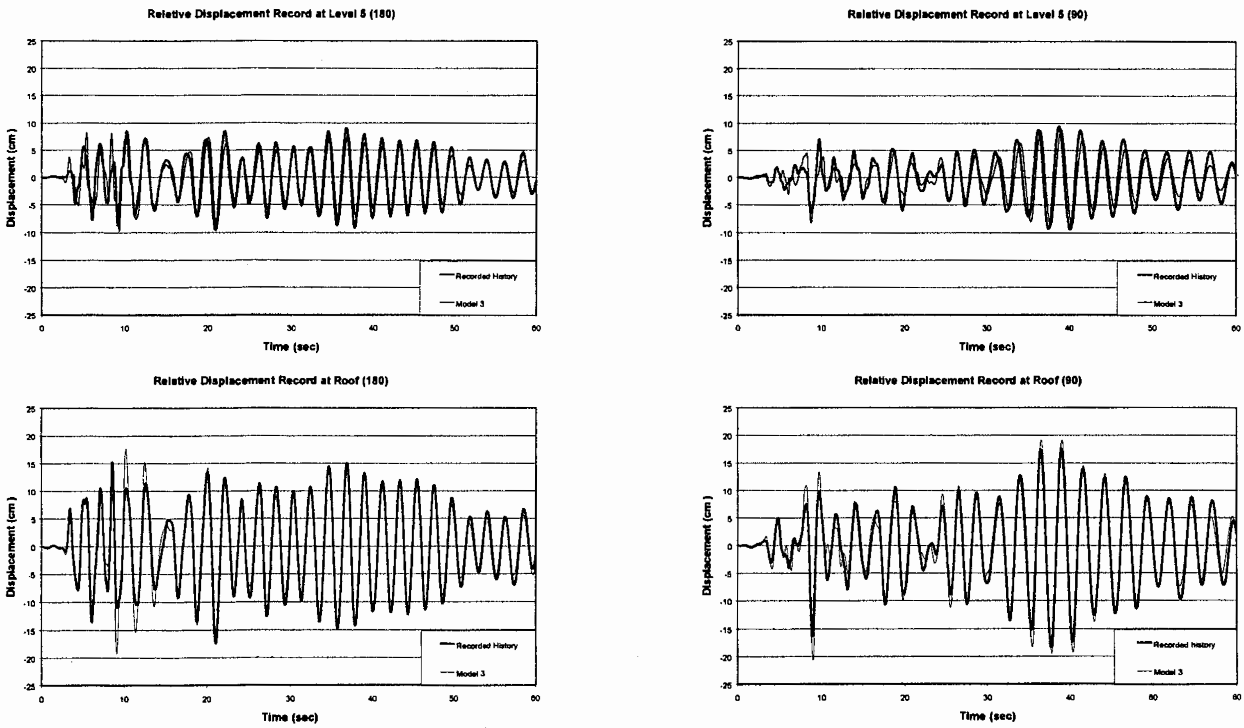


Figure 23. Displacement Time Histories from Northridge Earthquake for Model 3.

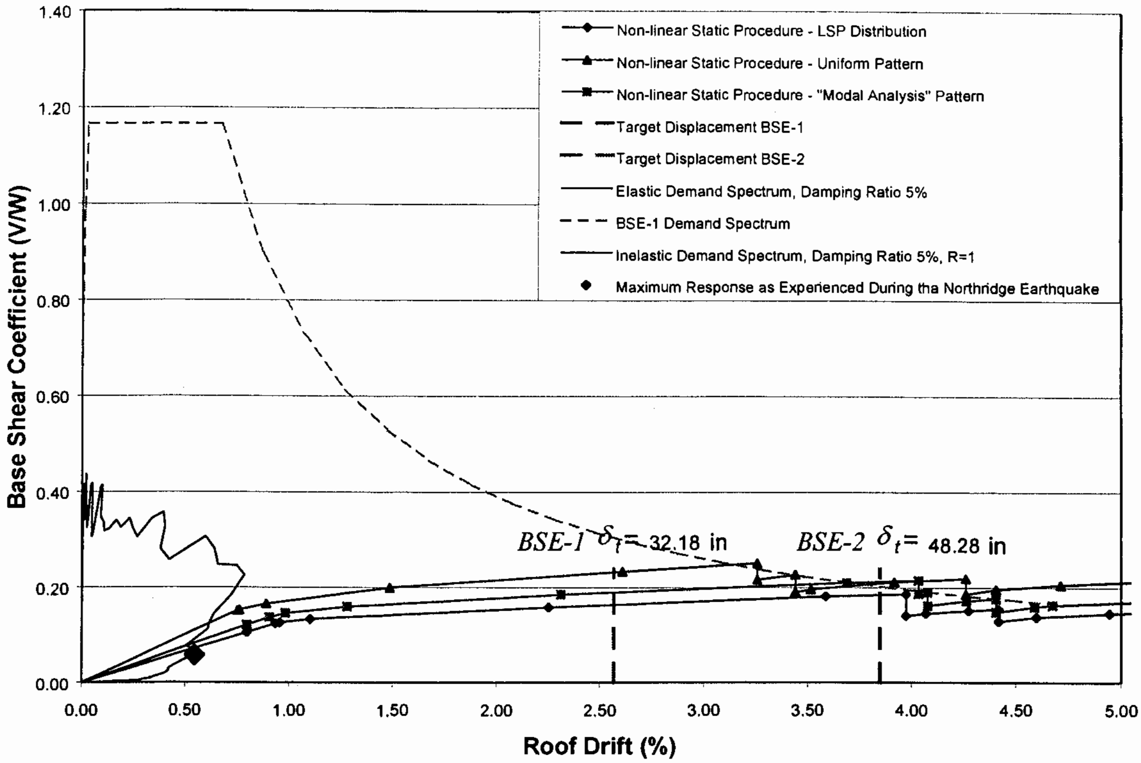


Figure 24. Demand-Capacity Spectra for the East-West Direction.

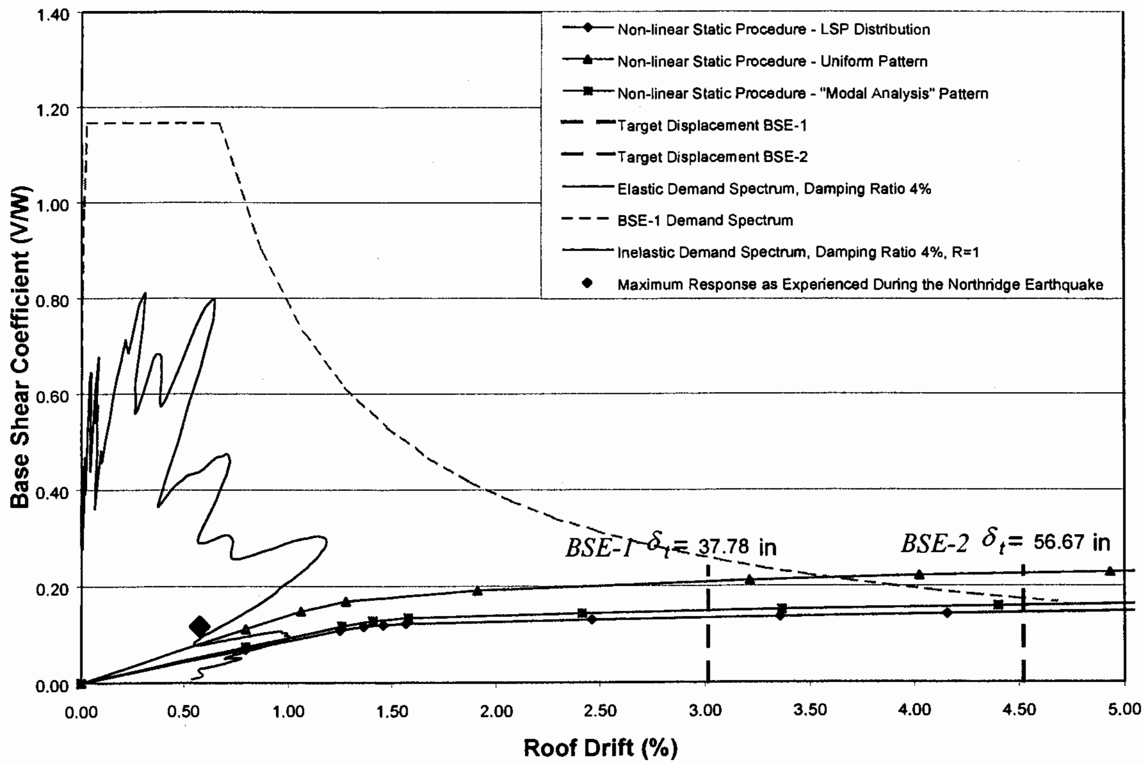


Figure 25. Demand-Capacity Spectra for the North-South Direction.

THE SHERMAN OAKS BUILDING

This is a sixteen-story Steel Moment Resisting Frame (SMRF) office building. It is rectangular in plan, with approximate plan dimensions of 129 ft. by 152 ft. The lateral resisting system of the structure consists of two pairs of identical multiple bay moment resisting frames along the perimeter of the building, in the North-South and the East-West directions with typical pre-Northridge rigid connections. The structural steel is either Grade A36 or Grade A572 (Grade 50) as specified on the construction drawings. The floor system at all floors except the roof is composed of QL-3-20 GA 1½" steel deck overlaid with 2½" lightweight concrete. For the roof, a QL-3-18 GA 1½" steel deck overlaid with 4½" lightweight concrete was used. Seismic sensors are located at the base, the eighth floor and the roof. The exact locations of the sensors were not identified, but surprisingly this proved not to be a practical concern for this building. A view of the computer model for this building is shown in Figure 26.

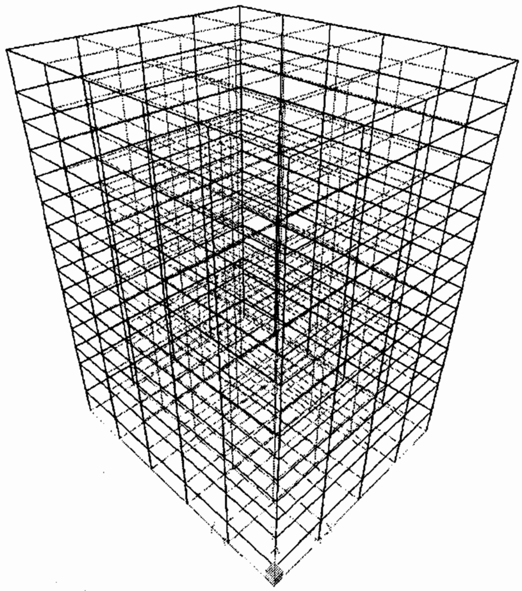


Figure 26. Three Dimensional Model of the Sherman Oaks Building.

The effectiveness of the rigid zones for the best fit model (*Model 3*) is 80% of the full rigid zone length for the East-West direction frames and 34% for the North-South direction frames. The damping ratios used for the first two modes of *Model 3* in the East-West direction were set at 2% and 3% and in the North-South direction at 1% and 3%. All higher modes were damped at 6%, so that the contribution of the high frequency response in the acceleration time histories is not exaggerated. A summary of the modeling differences between the models is presented in Table 10.

Table 10. Modeling Differences Between the Various Models.

Model	Rigid Zones	Analysis	Yield Stress (*ksi*)	Modal Damping
Model 1	All Elements	Elastic 3D	-	1%EW, 2% NS
Model 2	None	Elastic 3D	-	1%EW, 2% NS
Model 3	80 % EW 34% NS	Elastic 3D	-	EW: 2% 1st, 3% 2nd, 6% all others NS: 1% 1st, 3% 2nd, 6% all others

The vibration periods obtained from data interpretation are compared to those obtained by analysis in Table 11. The earthquake ground motions recorded at the base of the building are presented in Figure 27.

Table 11. Comparison of the Vibration Periods for Model 3 and the Periods Obtained from the Recorded Response Using the Fast Fourier Transform (FFT) Method.

North-South				East-West			
Mode	**Vibration Periods (SAP2000)** *(sec)*	**Vibration Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*	**Mode**	**Vibration Periods (SAP2000)** *(sec)*	**Vibration Periods (FFT Analysis)** *(sec)*	**Diff.** *(%)*
1	3.610	3.55	1.8	2	3.267	3.0	9.0
4	1.234	1.13	9.2	5	1.104	1.083	1.9
7	0.689	0.66	4.2	8	0.629	0.66	5.1

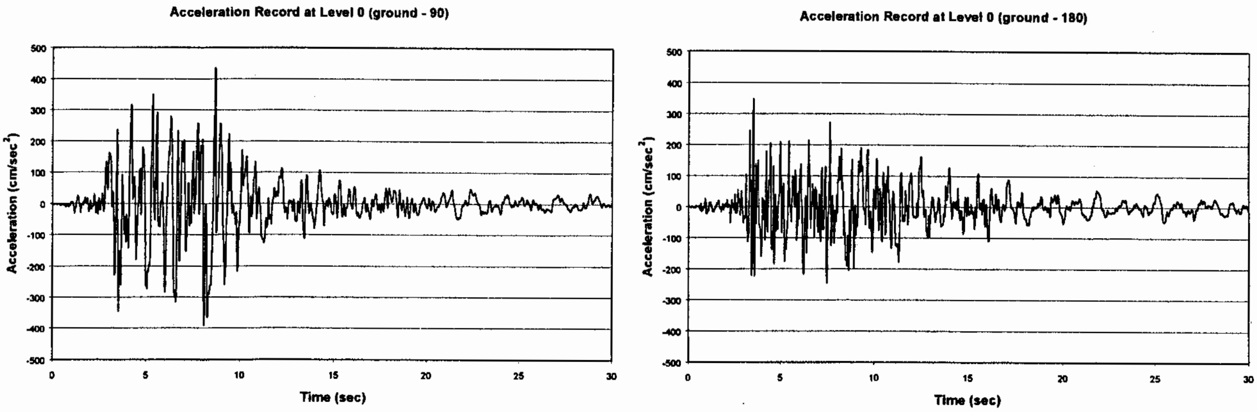


Figure 27. Recorded Northridge Ground Motions at the Base of the Sherman Oaks Building.

Details of performance analyses of this building may be found in our report to CSMIP [Naeim and others, 1999]. In this paper, however, we concentrate on what seems to be a clear case of resonant building response and possibly the first such response ever reported for a building in southern California.

The only difference between our various elastic analytical models for this building was the percentage of effectiveness assigned to beam-column end zones. In the case of this building, such an adjustment not only made the anticipated differences in the vibration periods, but made a huge difference in the amplitude of the observed response.

The response for Model 1 (Figure 28) shows that from the initial portion of the response the actual structure is more flexible than that predicted by this model. The response at the latter half does not come anywhere close to the actual response either because of resonant response not being captured. The response for *Model 2* (see Figure 29) shows that this model is unable to capture the response in the latter portion of the analysis, even though the initial portion seems to follow closely the initial response. The response for Model 3 (Figure 30), however, clearly matches the observed response showing that this building behaved in an essentially elastic manner during the Northridge earthquake.

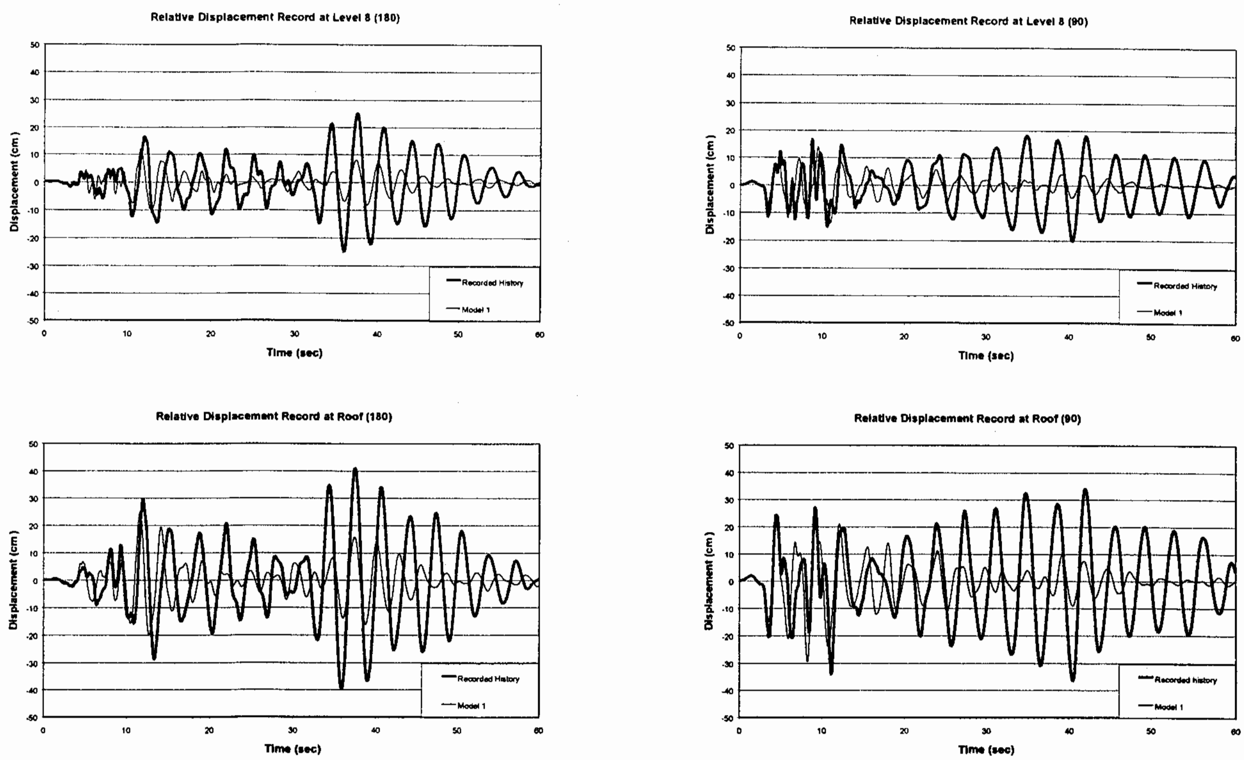


Figure 28. Displacement Time Histories from Northridge Earthquake for Model 1.

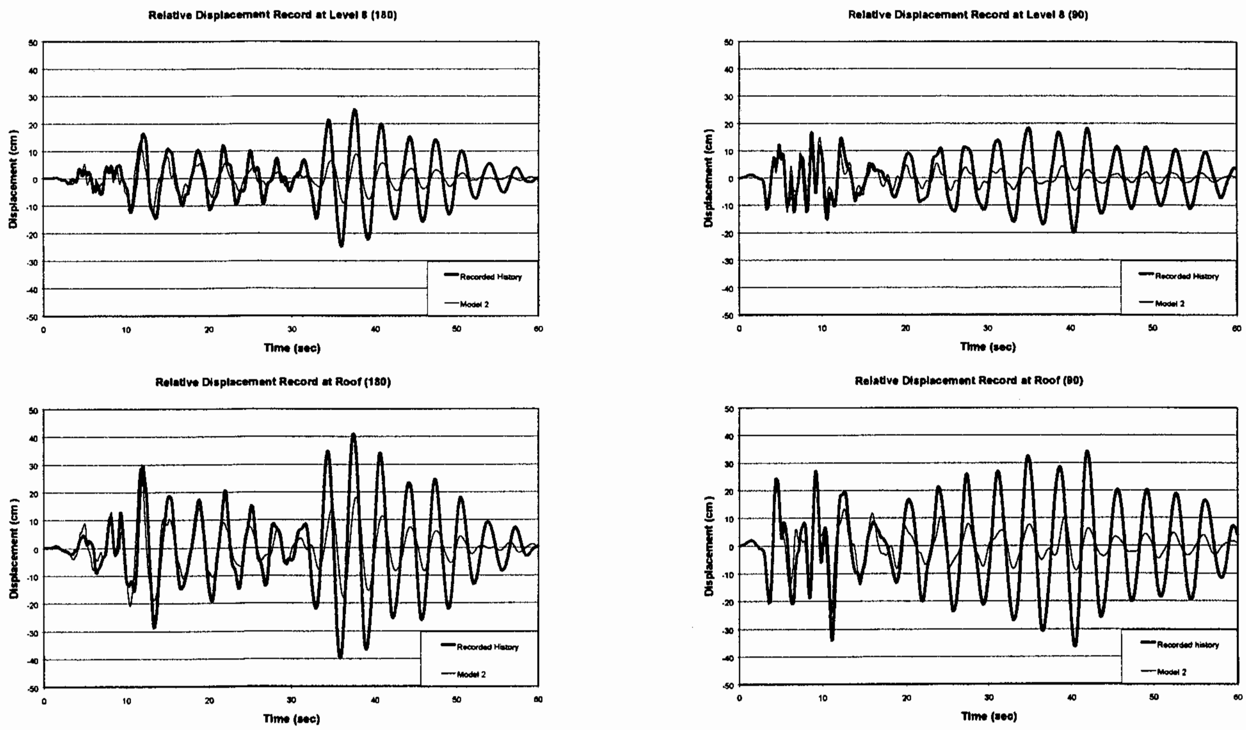


Figure 29. Displacement Time Histories from Northridge Earthquake for Model 2.

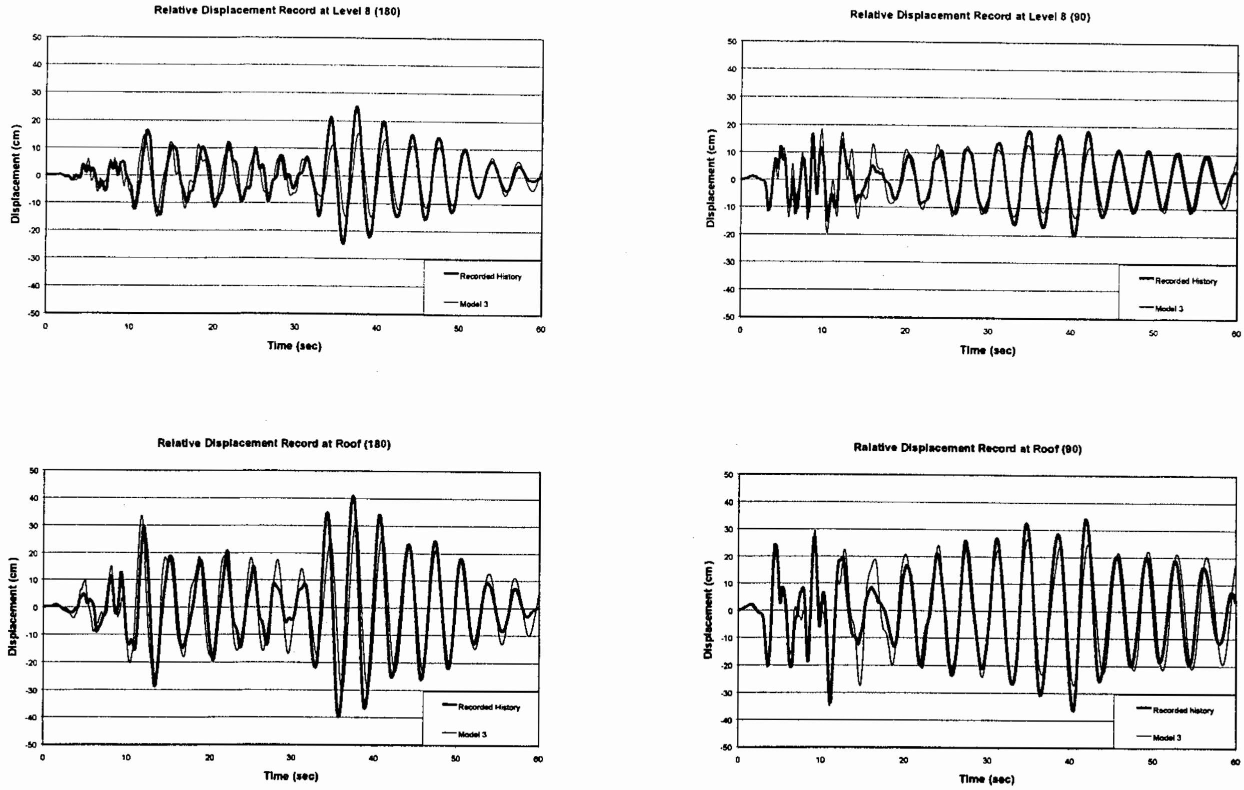


Figure 30. Displacement Time Histories from Northridge Earthquake for Model 3.

CONCLUSIONS

We have studied the seismic performance of four instrumented steel moment resisting buildings during the 1994 Northridge earthquake. These commercial buildings ranged from 8 stories to 20 stories in height and all had rectangular and regular plans, although two had elevation irregularities. The buildings were instrumented at the ground floor, a floor near the mid-height, and at the roof. For three of the four buildings the precise location of the instruments in plan was not known. However, due to insignificant contribution of torsion to seismic response of these buildings, that did not prevent us from being able to effectively and accurately match the observed response by analytical means. Furthermore, the sensors at various floors were not linked to a common timer and therefore the timing of their recordings were not synchronized. Although it was initially expected that the phase lag present between the roof records and the base may present some problems in matching the analytical and recorded responses, lack of synchronization proved not to be a significant issue.

Among the four buildings evaluated in this study, the Tarzana building experienced the most severe horizontal and vertical ground motion at the base. Therefore, the effects of vertical accelerations on seismic performance of this building were evaluated and proved to be insignificant.

Gravity framing has been known to have some participation in the seismic response. For the buildings investigated in this report, however, the observed response could be matched analytically both in terms of frequency content and response amplitude without the need for explicit modeling of the gravity frames. This was achieved simply by adjusting the effective width of the beam and column panel zones and using appropriate modal damping ratios.

One of the buildings located in Sherman Oaks exhibited strong evidence of resonant behavior as a result of the long-period content of the base motion which was close to the fundamental period of the building. To our knowledge, this is the first time that resonant behavior of a building in response to a southern California earthquake has been documented and reported.

We performed linear and nonlinear, static and dynamic analyses of these buildings and compared the design details and performance with those suggested or anticipated by the provisions of the UBC-97 code and FEMA-273 guidelines. Generally speaking, the nonlinear static procedure as suggested by the FEMA-273 guidelines provided the most accurate reflection of the observed response. This is true even for the 20 story building which had significant higher mode participation. Due to the conservative nature of the prevailing methods of calculating elastic demand ratios (EDR) in time history analysis, as explained previously in this report, time history analysis overestimated the seismic demand imposed on beams and columns and was not as good as the pushover-method in predicting the state of individual elements. This is true in spite of the fact that the time history displacement response of our calibrated models matched the observed response very closely.

It is fair to say that none of the methods utilized were accurate enough in pin-pointing the exact locations of the observed damage. More research is needed to overcome this shortcoming which has been reported by many investigators [see Naeim and others 1995, 1997].

The UBC-97 methodology consistently underestimated the force demand and significantly over-estimated the drift demands. Underestimating the force demand by UBC-97 is understandable in the light of the code reduction factor and expectation of nonlinear behavior where most of these buildings remained essentially elastic in spite of damage to some joints. UBC's overestimating of the demand may require a revision of the code drift demand provisions since they do not seem realistic.

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