RESPONSE EVALUATION OF THREE CONCRETE FRAME BUILDINGS TO THE JANUARY 17, 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

As a part of a project sponsored by the Strong Motion Instrumentation Program of state of California (SIMP), responses of three concrete frame buildings during the 1994 Northridge earthquake are being evaluated. This investigation is at various stages of progress for the three buildings. For the 13 Story Sherman Oaks Building both linear and nonlinear analysis have been performed and nonstructural components response has been evaluated. For the 32 Story Burbank Building, linear analysis had been completed and nonlinear analysis is in progress. For the 7 Story Van Nuys Hotel construction of computer models are in progress. This paper serves as a status report on what has been learned so far from these investigations. Further details and refinements to observation presented in this paper will be forthcoming in a report to the SMIP.

13 STORY SHERMAN OAKS BUILDING

Description of the Building

A photo of the building retrieved from the SMIP Information System (Naeim,1997) is shown in Figure 1. This office building has 13 stories above and two floors below the ground. It was designed in 1964. The vertical load carrying system consists of 4.5 inches thick one-way concrete slabs supported by concrete beams, girders and columns. The lateral load resisting system consists of moment resisting concrete frames in the upper stories and concrete shear walls in the basements. The foundation system consists of concrete piles. The first floor spandrel girders were modified by post-tensioning after the 1971 San Fernando earthquake. Sketches of plan and elevation of the building showing the location of sensors are presented in Figure 2.
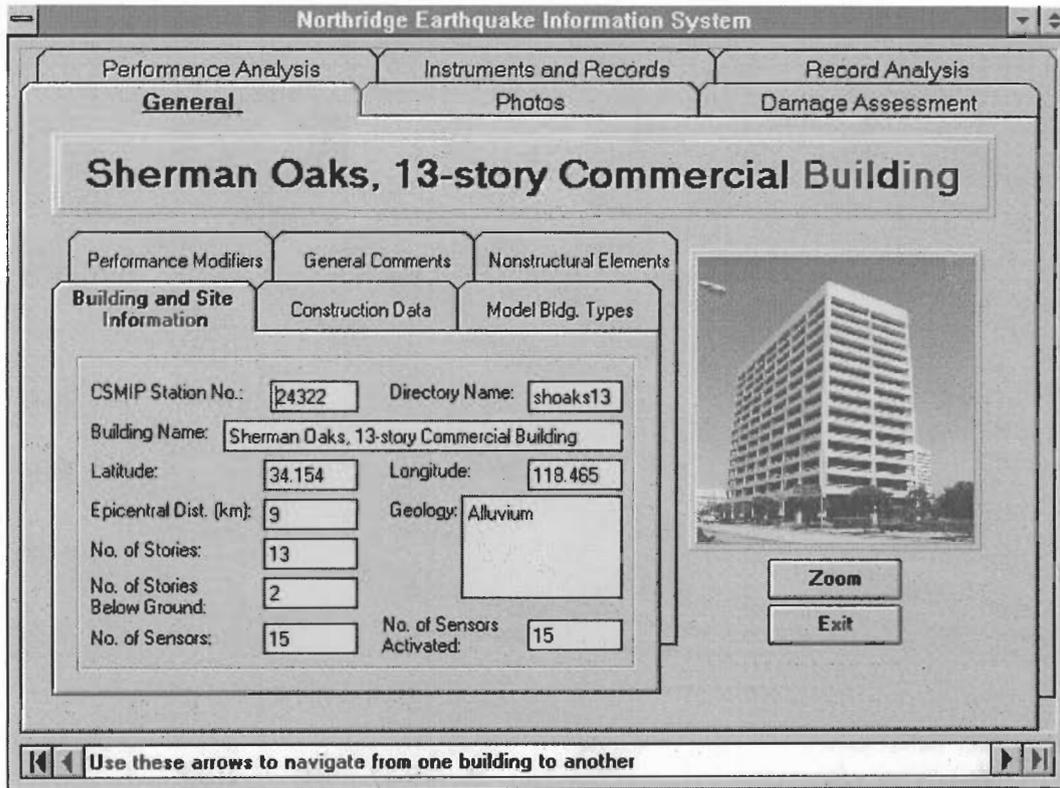
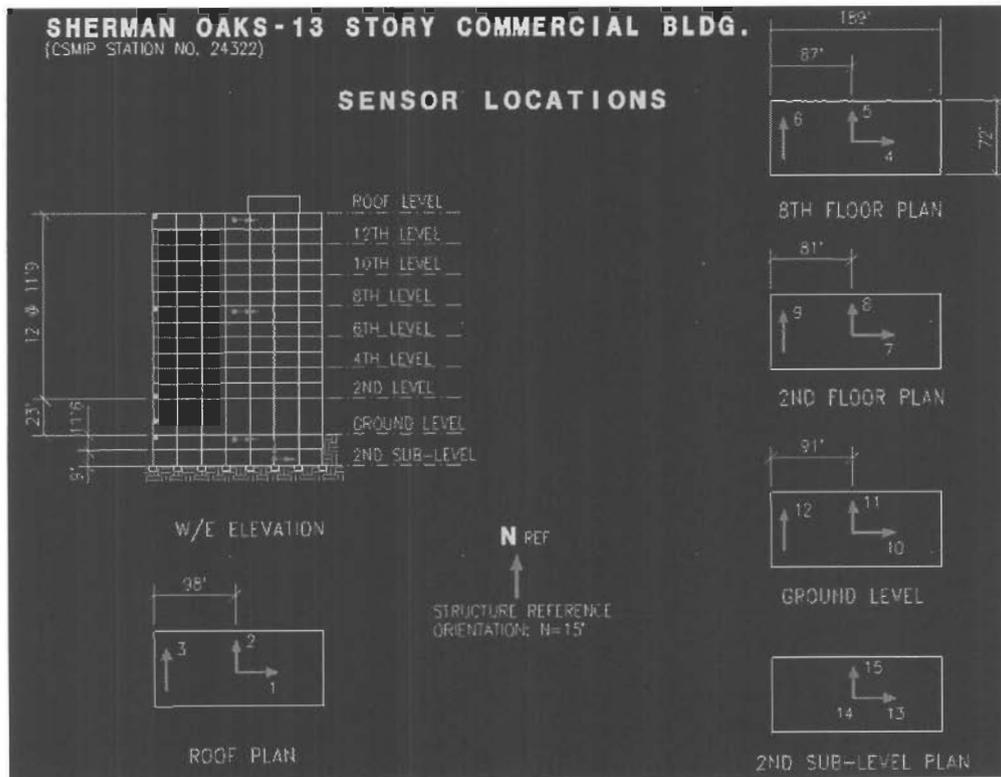Figure 1. Three Dimensional View of the Building (from Naeim, 1997)

Figure 2. Location of Instruments (from Naeim, 1997)

Building Performance

The largest peak horizontal accelerations recorded at the basement (Channel 15, N-S) and at the roof (Channel 3, N-S) are 0.46g and 0.65g, respectively. The middle experienced large acceleration in the neighborhood of 0.6g. The largest velocity recorded at the roof is about 68 cm/sec.

Performance analysis calculations for this building perfromed by Naeim (1997) are summarized in Table 1. The 0.19W maximum base shear apparently experienced by the building in the N-S direction is significantly larger than UBC strength design base shear of about 1.4x0.04W= 0.06W for a ductile moment resisting frame with this configuration. Notice that while the maximum base shear is experienced in the N-S direction, the maximum lateral displacement and an overall drift index of 0.0067 occurs in the E-W direction.

TABLE 1. Response Summary for Sherman Oaks 13-Story Office Building (from Naeim, 1997).

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	5.14	18.70
	E-W	12.72	7.57
	DIFF	3.24	6.69
Overturning Moment (% Total Weight x feet)	N-S	3.22	1304
	E-W	11.52	771
	DIFF	3.22	615
Roof Lateral Displacement Relative to the Base (cm)	N-S	10.86	24.10 (0.0048)*
	E-W	37.98	33.42 (0.0067)*
	DIFF	11.00	4.30 (0.0009)*

* Overall drift index values are shown in brackets.

As documented by the SMIP Information System photos the building experienced noticeable but repairable structural damage in the form of cracks in the beams, slabs, girders, and walls. According to one source the repair costs exceeded several million dollars. In contrast, no mechanical equipment damage was observed either at the roof or the basement. Thanks to proper mounting and anchorage details.

As can be seen in Figure 3, participation of higher modes were particularly significant in the response of this building to Northridge earthquake. The N-S period of about 2.6 seconds is significantly larger than code estimated periods of 1.27 per UBC-67 and 1.60 seconds per UBC-94. In the E-W direction, a fundamental period of about 2.9 seconds is implied by FFT analysis (Figure 4). Our moving windows FFT analysis points to a softening of the structure which may be attributed to the concrete cracking (Figure 5).
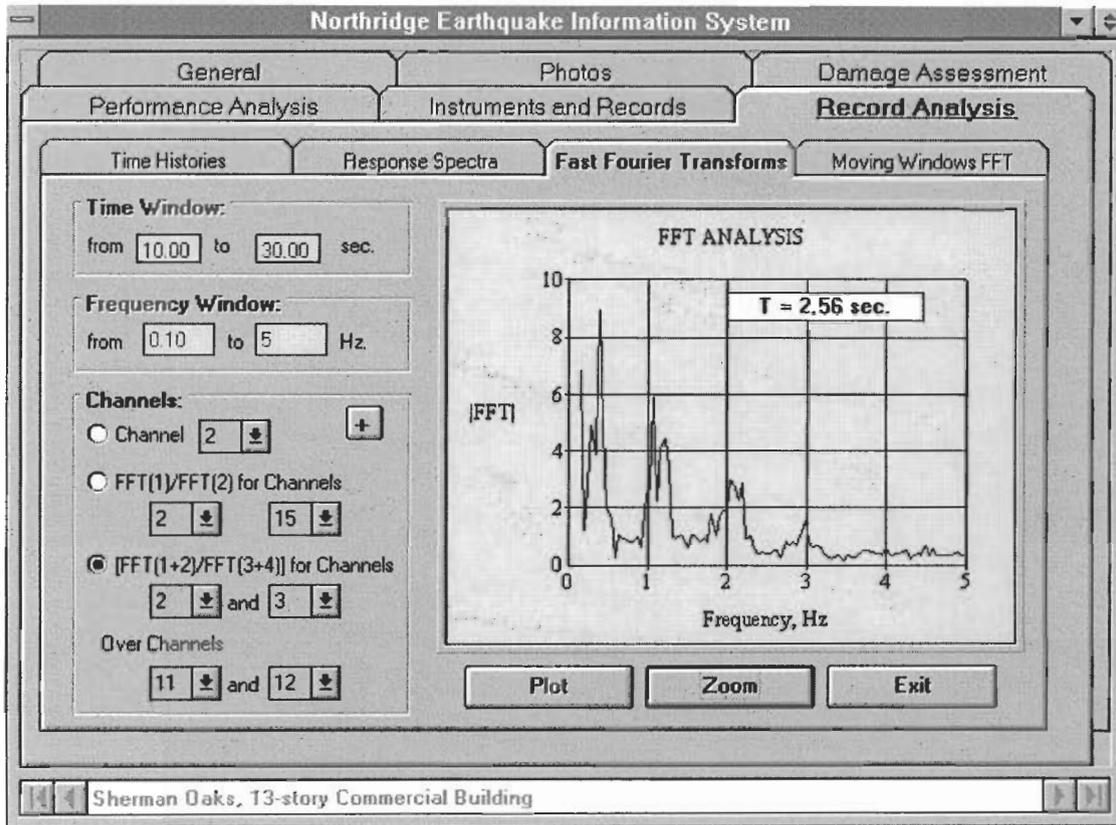
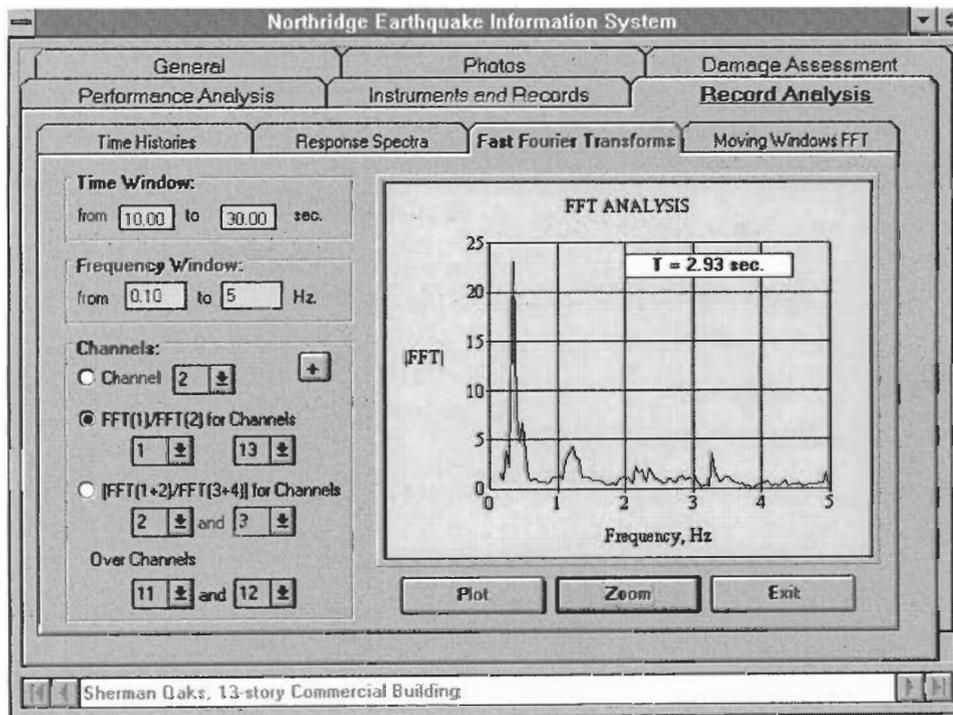Figure 3. An FFT analysis for the N-S response (from Naeim, 1997).

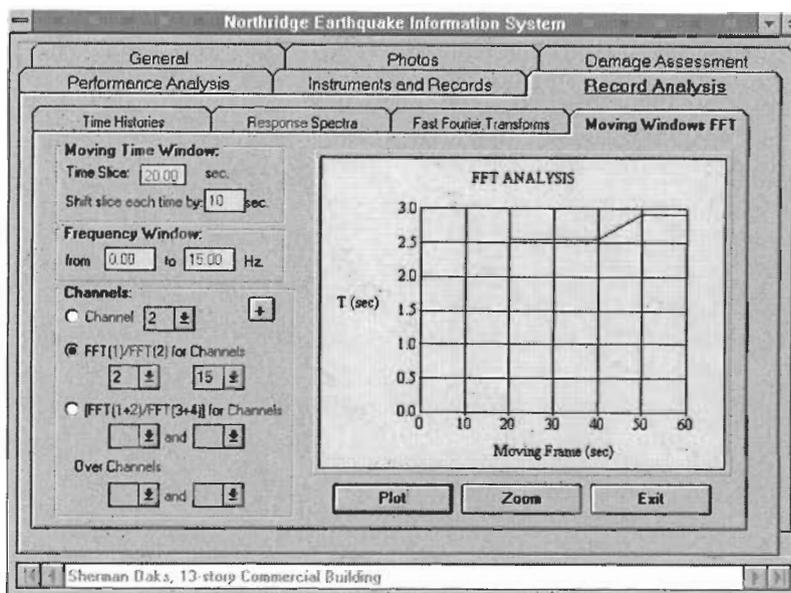Figure 4. An FFT analysis for the E-W response (from Naeim, 1997).

Figure 5. Moving windows FFT analysis indicates period elongation (from Naeim, 1997).

Performance of Non-Structural Systems and Components

The damage to non structural components and equipment for the building have been documented in the SMIP Interactive Information System (Naeim,1997). This building experienced noticeable but repairable structural damage in the form of cracks in the beams, slabs, girders, and walls (Figure 6a). In contrast, no mechanical equipment damage was observed either at the roof or the basement (Figures 6b, 6c, and 6d). This was due to proper mounting and anchorage details.

A summary of the damage to the nonstructural components are given in Table 2. This table also gives the corresponding a_p and R_p factors for the considered codes and guidelines.

TABLE 2. Summary of observed nonstructural damage and design coefficients

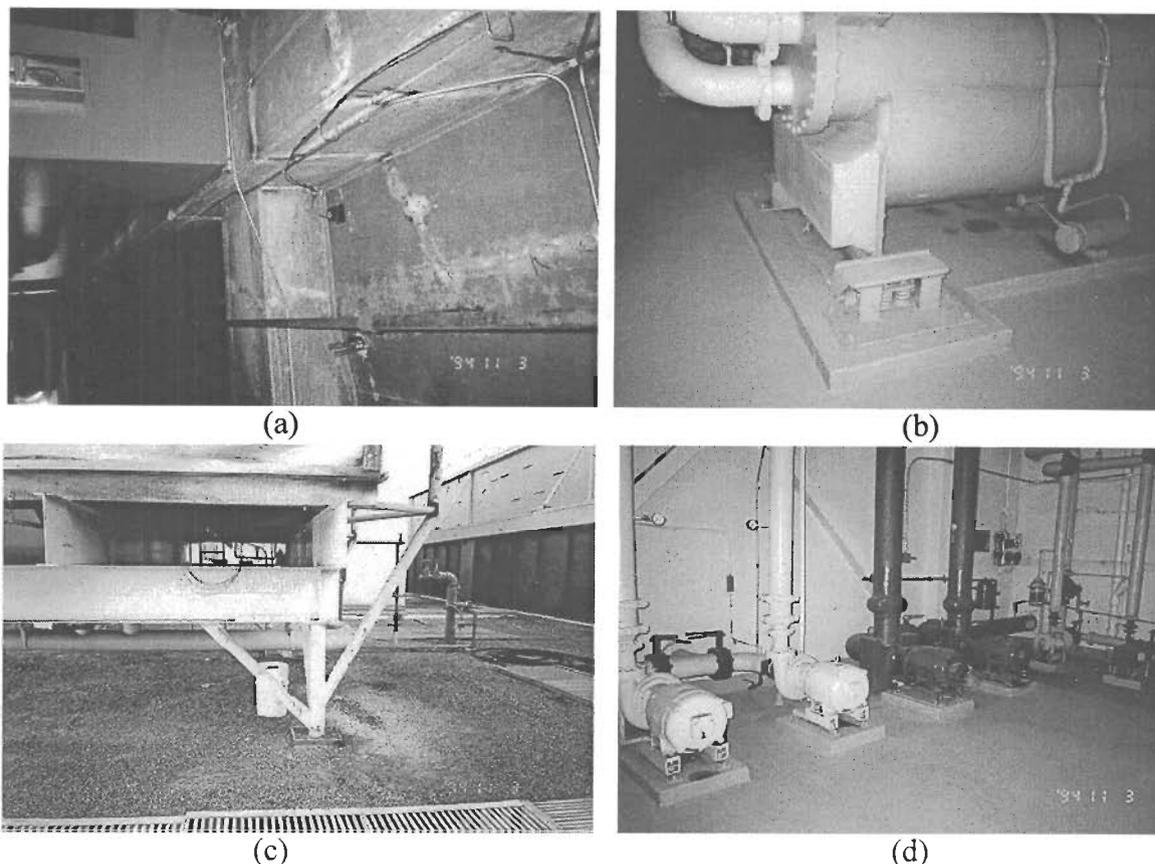| Building	Component Description	Observed Damage	Design Coefficients		
			UBC-97	NEHRP-97	FEMA-273
			$\dfrac{a_p}{R_p}$	$\dfrac{a_p}{R_p}$	$\dfrac{a_p}{R_p}$
Sherman Oaks, 13 Story Commercial	Mechanical Equipment at basement	No damage	2.5/3	1/2.5	1/3
	Piping	No damage	1/3	1/2.5	2.5/4
	Roof installed equipment	Little or no damage	2.5/3	1/2.5	1/3

Figure 6. Contrasting Structural and nonstructural performance (13 Story Sherman Oaks Bldg).

Three different computer models were developed to analyze the 13 Story Sherman Oaks Building. The first is a three-dimensional linear elastic SAP-2000 model referred to as the *Elastic* model. The second and third are two-dimensional IDARC inelastic models referred to as the *Inelastic-EW* model and the *Inelastic-NS* model, respectively. The two sublevels below the ground floor were considered restrained because of the presence of perimeter shear walls limiting the motion at the basement levels. These levels were therefore ignored in the analysis, and the columns were considered fixed at the ground level.

For the *Elastic* model, the section properties are based on the gross section dimensions of the elements with a reduced modulus of elasticity. The modulus of elasticity for the beams was taken as $0.4E_g$, and for the columns as $0.5E_g$. The disadvantage of this model is that it not only reduces the flexural stiffness, but also the axial stiffness of the columns. The floor slabs are modeled as rigid diaphragms, therefore the axial stiffness of the beams do not play a part in the performance of the structure. The other obvious disadvantage of this model is that it does not capture any inelastic behavior that could occur.

The choice of the computational platforms available to analyze the *Inelastic-EW* and *Inelastic-NS* models were carefully selected in view of the following considerations. Firstly the very fact that these models are inelastic in nature, require additional data to be processed before the building can be analyzed. This includes the development of the moment curvature envelopes for each

member type. Column elements having different axial loads have to be treated as separate element types, even though they may have the same dimensions and reinforcement. Beam elements require a moment curvature analysis to be performed on the top and bottom face at each end of the beam. Additionally due consideration needs to be given to the development length of the reinforcement. While full capacity may not be developed with inadequate development lengths, there is definitely some contribution to the flexural capacity, at least for short-term dynamic loading. This could also include increased capacity by dynamic strain rate effects on the properties of the concrete. Gravity loading also needs to be considered to account for the initial state of stress in beams and columns (see Naeim et. al., 1998).

In light of the above considerations, the computer program IDARC4.0 was chosen as the computational platform to perform the analysis. This program was modified from its original form to include the effects of shear deformation. As this computational platform can only analyze buildings in two dimensions, the individual frames were modeled as duplicate frames in parallel to include the entire stiffness of the structure.

The dynamic characteristics of the building for the three models are compared with the interpreted characteristics in Table 3.

TABLE 3. Response Summary.

Response parameter	Dir.	Observed	Elastic	Elastic - E=0.7Egross	Inelastic-NS	Inelastic-EW
Period	N-S	2.56	2.74	2.15	2.03	
	E-W	2.93	2.40	1.91		1.84
Roof Lateral Displacement	N-S	9.26	11.86	13.40	8.67	
Relative to the Ground	E-W	13.16	13.27	6.86		10.91

Time history analyses were performed on the three models using the actual recorded response at the ground floor. Figures 7 through 14 show the comparison of the responses for the various models, at the levels at which the instruments were located. The comparison of displacements at the second floor for both the Inelastic models causes a bit of concern. Probably indicating that the assumption that the boundary conditions are fully restrained is not entirely valid. In addition, there is an additional low roof structure at this level which is not modeled. The forces from this structure could be responsible for the differences between analysis and recorded responses. If the differences at this floor are reconciled, the comparison of responses for other floors become much better. We need to re-evaluate further our modeling assumptions with respect to the first floor low-rise area and boundary conditions.
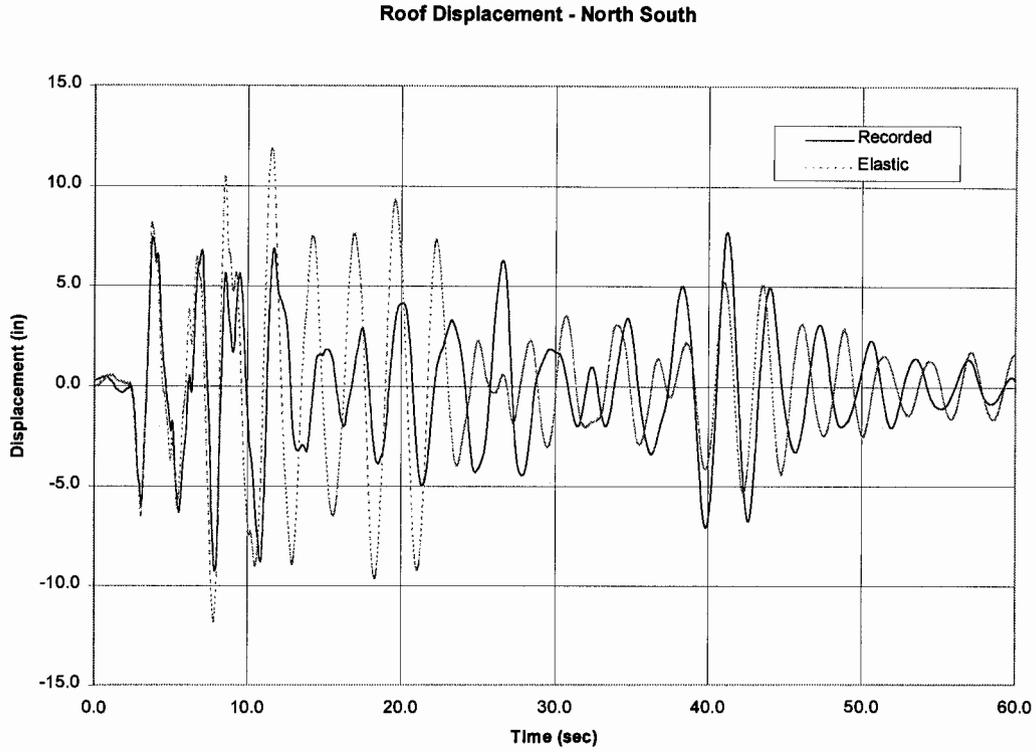
Roof Displacement - North South

Figure 7. Comparison of Recorded Vs Elastic in North-South direction at Roof.

Roof Displacement - North South
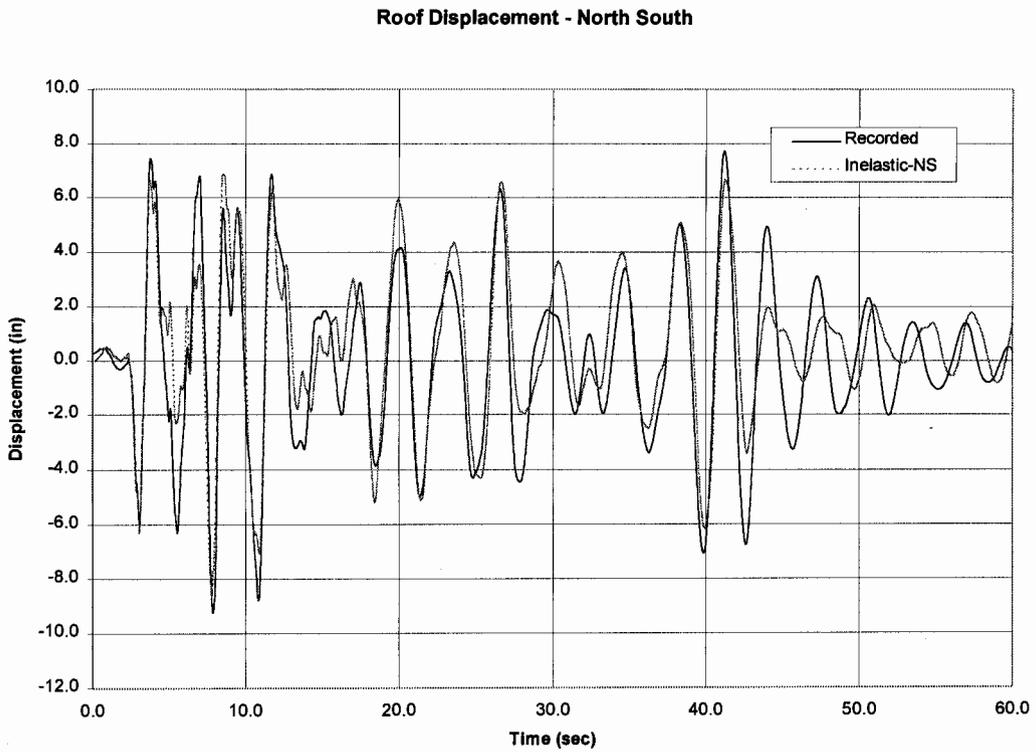
Figure 8. Comparison of Recorded Vs Inelastic-NS in North-South direction at Roof.

Roof Displacement - East West
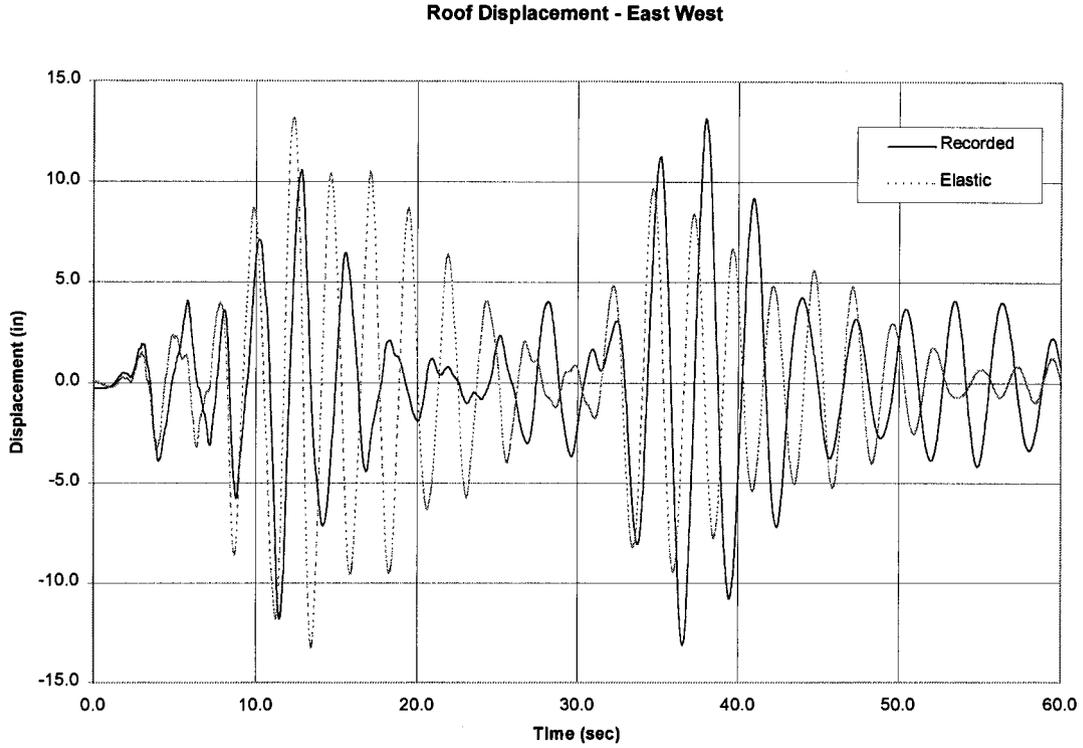
Figure 9. Comparison of Recorded Vs Elastic in East-West direction at Roof.

Roof Displacement - East West
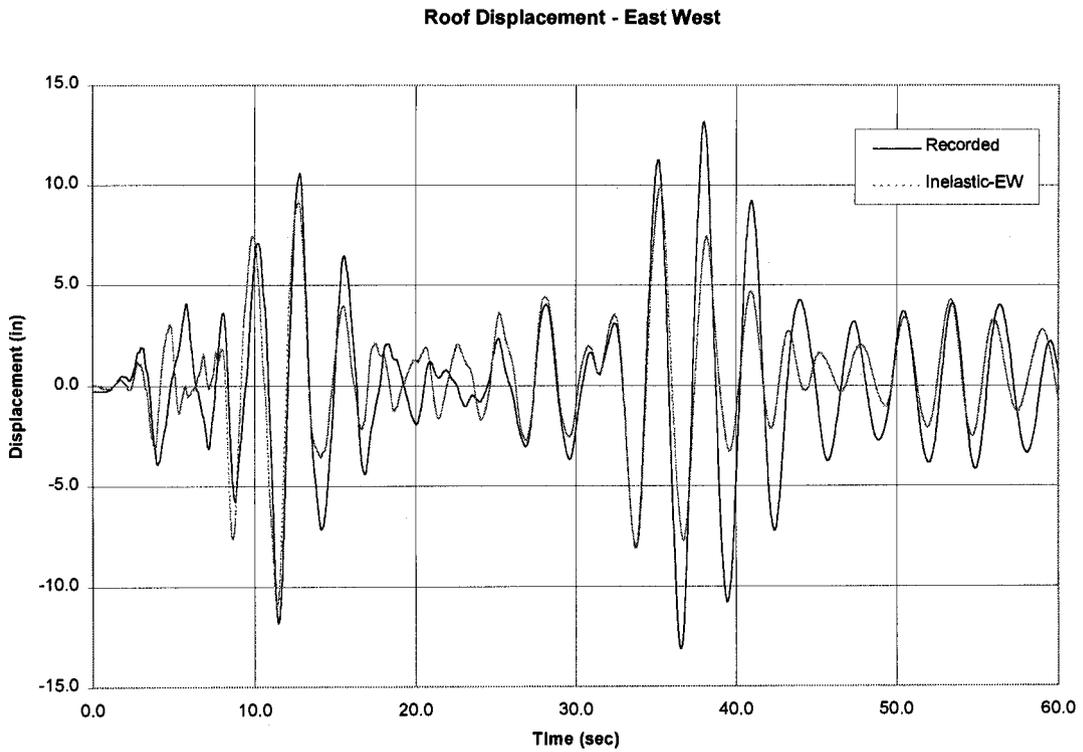
Figure 10. Comparison of Recorded Vs Inelastic-EW in East-West direction at Roof.

Second Story Displacement - North South
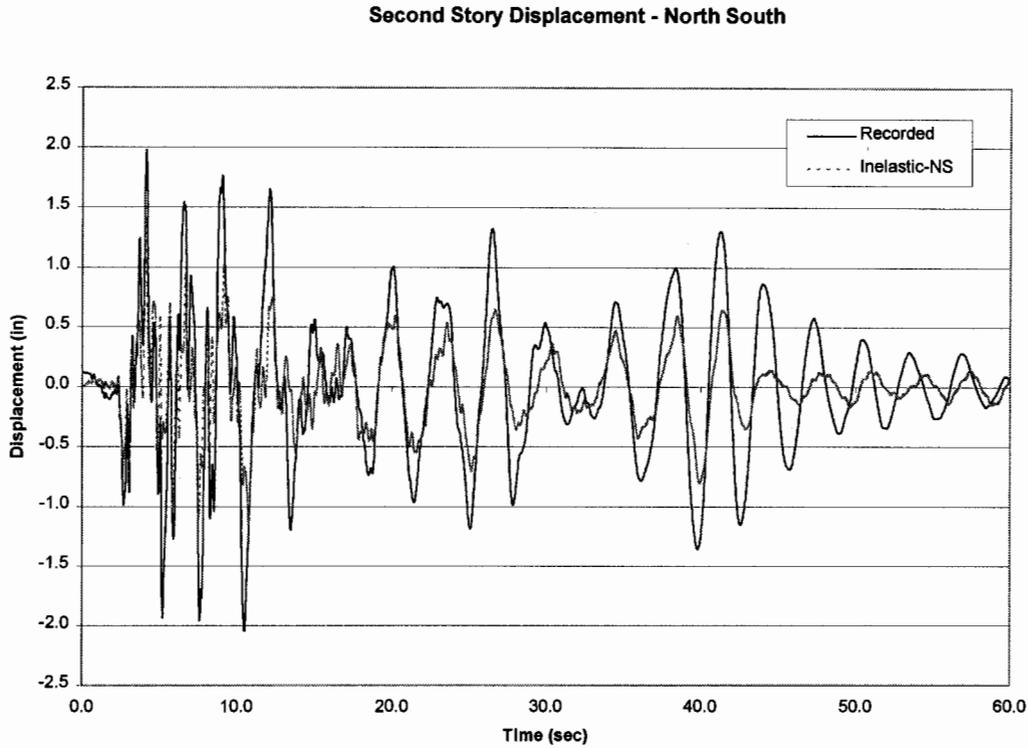
Figure 11. Comparison of Recorded Vs Inelastic-EW in North-South direction at 2nd.
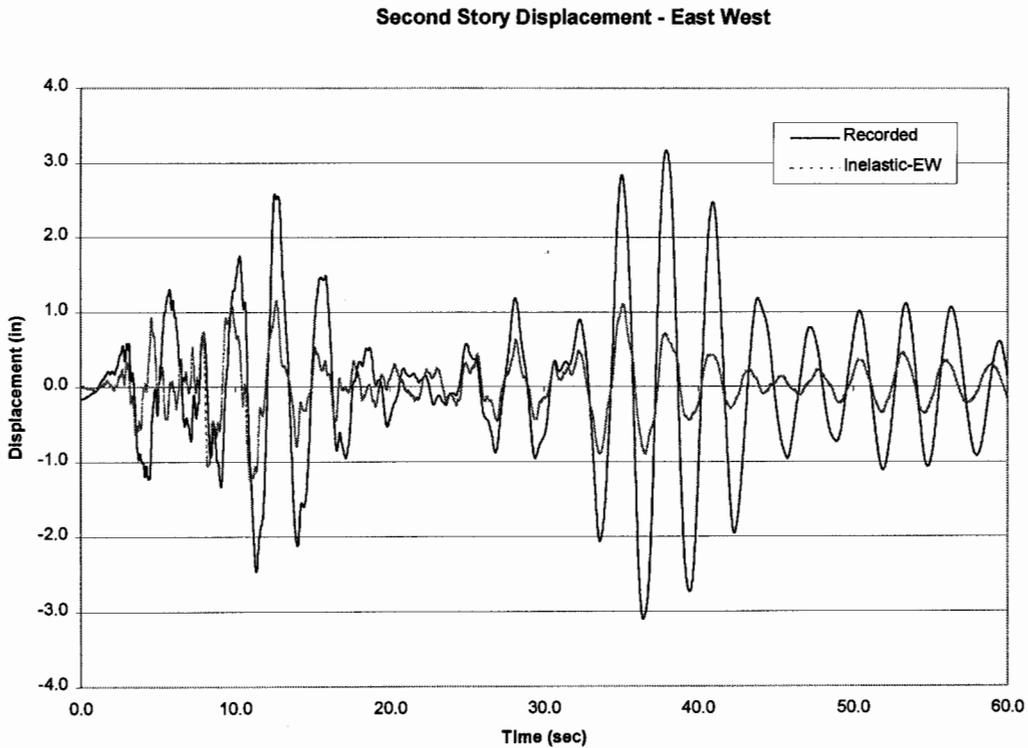
Second Story Displacement - East West

Figure 12. Comparison of Recorded Vs Inelastic-EW in East-West direction at 2nd.

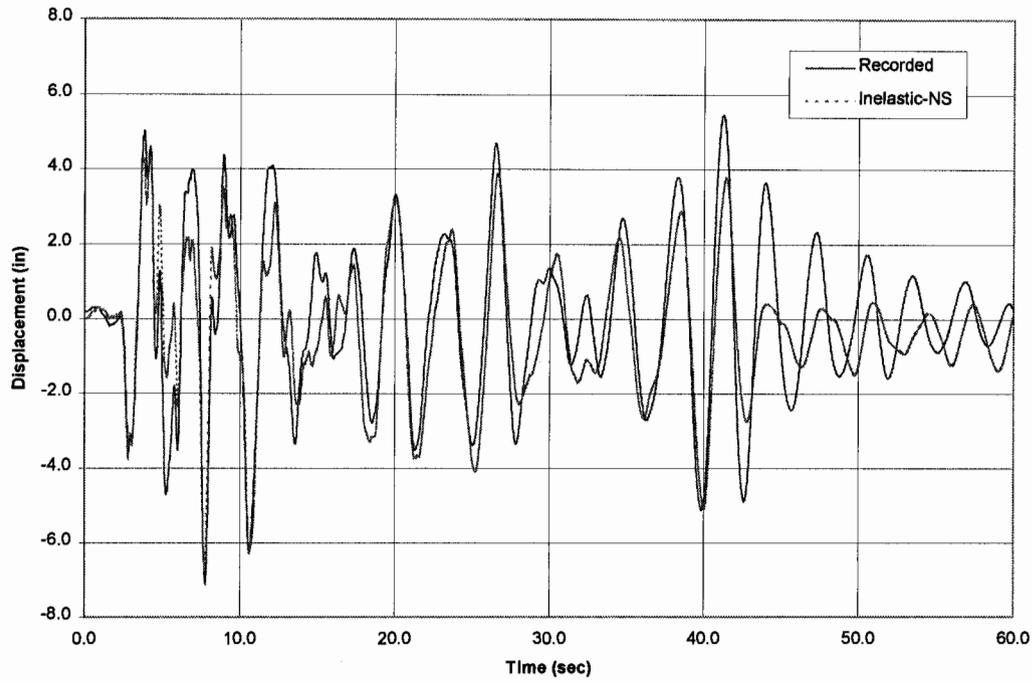**Eight Story Displacement - North South**

Figure 13. Comparison of Recorded Vs Inelastic-NS in North-South direction at 8th.
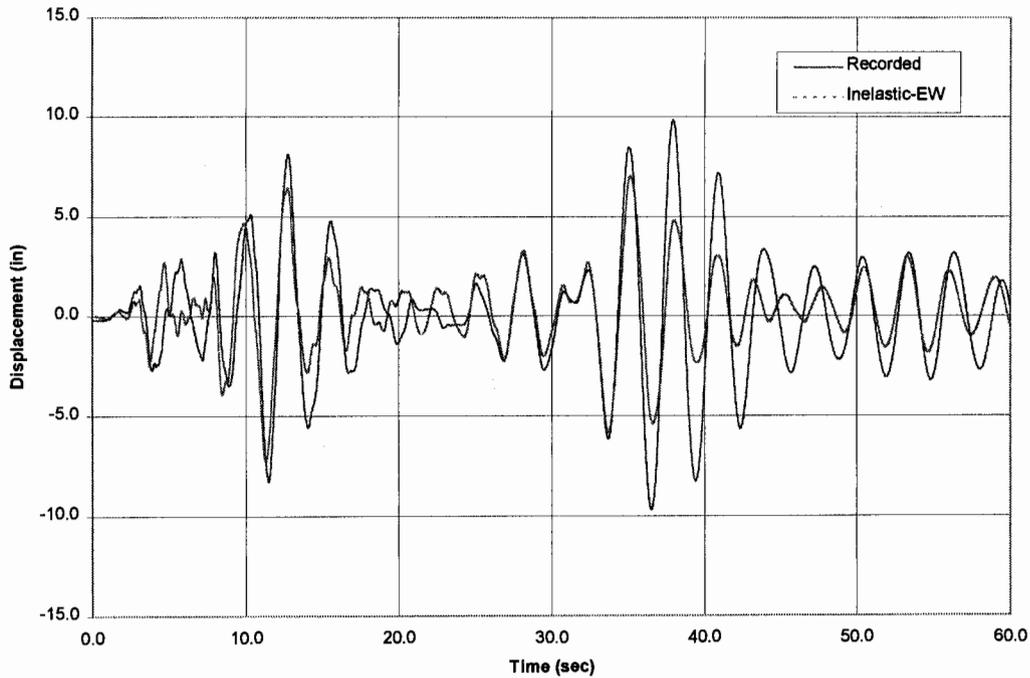
Eighth Story Displacement - East West

Figure 14. Comparison of Recorded Vs Inelastic-EW in East-West direction at 8th.

The *Elastic* model does not produce very different results in terms of peak displacements, but provides a very different time history response than the one actually observed. The lengthening of the period as the earthquake progressed is fairly well captured, however, with the inelastic models. An important point to note is that the initial gravity loading plays a significant role in the response of the inelastic models. A comparison was performed where the gravity loads were not included and the response was very different from the recorded. This is because the bottom reinforcement at the supports is much less than the top reinforcement and causes early cracking in positive bending, significantly altering the response.

32 STORY BURBANK BUILDING

Building Description

This building consists of a thirty-two story office building (referred to here as the "Tower Building"), and an eight-story parking structure (referred to here as the "Low-Rise Building"). Figure 15 shows a southeast elevation of the building. It was constructed in 1987 with a completed cost of $42 million dollars.

The Tower Building has typical plan dimensions of approximately 269 feet in the northwest-southeast (longitudinal) direction and 81 feet in the southwest-northeast (transverse) direction. As shown in Figure 16, columns are typically placed at 26.87 feet on center in both longitudinal and transverse directions. The typical story height for the Tower Building is 12 feet. The Low-Rise parking structure has an eight story parking garage with four levels below grade and four levels above grade. The story height for the parking structure is typically 10 feet with an exception of 14 feet for the eighth level. According to the structural drawings, the reinforced concrete column strength varies from 4 ksi to 6 ksi. The reinforced concrete beam strength is 5 ksi. The floor system consists of 7 inch post-tensioned concrete flat plate slabs with a strength of 5 ksi. Reinforcement in frame ductile members complies with then UBC Standard No. 26-4 for low alloy A706.

The lateral resisting system of the Tower Building is comprised of reinforced concrete moment frames in both longitudinal and transverse directions. Frame column dimensions vary from 18 inches wide by 36 inches deep at higher levels to 48 inches wide by 44 inches deep at lower levels. The typical frame column size is 36 inches by 36 inches. Frame beam dimensions in the longitudinal direction vary from 18 inches wide by 38 inches deep to 20 inches wide by 38 inches deep. In the transverse direction, frame beams have dimensions of 30 inches wide by 33 inches deep to 36 inches wide by 33 inches deep. Ties for the frame column are typically #6 or #5 spaced at 3 or 3.5 inches on center throughout the full height of the column. Stirrups for the frame beams in the longitudinal direction typically consist of D31 wire fabric at 4 inches on center and in the transverse direction #5 rebars at 6 inches on center. The Low-Rise Building has four perimeter reinforced concrete shear walls up to 18 inches thick as its lateral resisting system.

Instrumentation Information

The Tower Building is instrumented at the roof, 16th, and basement levels. Instrumentation data were retrieved and processed after the 1994 Northridge earthquake. Figures 17 through 25 illustrate the acceleration, velocity and displacement time histories at the roof, 16th, and basement levels in the longitudinal, transverse, and vertical directions. The epicenter of the Northridge earthquake is located about 12 miles northwest of the building. No damage to the building was reported after the earthquake. The ground shaking lasted about 20 seconds. The peak ground accelerations are estimated as 72.6 in/s/s (0.188g) in the longitudinal direction, 80.4 in/s/s (0.208g) in the transverse direction, and 78.8 in/s/s (0.204g) in the vertical direction. Estimated fundamental building periods from these time histories are 1.48 seconds for the longitudinal direction and 3.43 seconds for the transverse direction.

Objectives of Evaluation

The objective of this study is to evaluate the response of the Tower Building to the 1994 Northridge earthquake and compare the recorded data with the results from elastic structural analysis (using ETABS) and inelastic structural analysis (using DRAIN2DX).

Elastic Analysis

An elastic analysis was conducted using ETABS to obtain the building's mode shapes and natural periods and compare the natural periods with the estimated periods from the recorded building response data during the 1994 Northridge earthquake. A three-dimensional modal was established. The grid line of the ETABS model is shown in Figure 12 and a three-dimensional view of the model is shown in Figure 13. Only the Tower Building is included in the model. The shear walls in the Low-Rise Building are modeled as external spring restraints to the Tower Building. The model of the building is based on the existing structural plan. The post-tensioned concrete slabs are converted to equivalent concrete beams. The assumptions used in the model primarily follow the recommendations from ACI 318-95 (ACI, 1995) and ATC 40 (ATC, 1996). The following summarizes the most significant modeling assumptions used for the analysis.

1. Gross Section Stiffness Reduction Factors for Beams and Columns:
    Concrete cracks when stressed beyond its tensile strength. Effective stiffness of a member represents an average stiffness after concrete cracks. The common approach for estimating an effective stiffness is to use gross section stiffness times a reduction factor. ACI 318-95 recommends using 0.35Ig as effective stiffness for beams, 0.70Ig for columns and 0.35Ig for cracked walls. ATC-40 (1996) uses 0.5Ig as effective stiffness for non-prestressed beams, 1.0Ig for prestressed beams, 0.7Ig for columns in compression and 0.5Ig for cracked walls. Due to the moderate nature of the ground shaking that the building experienced during the 1994 Northridge earthquake, the stiffness reduction factors are assumed to be 0.6 for beams, 0.8 for columns, and 1.0 for equivalent post-tensioned slabs.

2. Rigid End Zone Reduction:

    The rigid zone reduction was assumed to be 0 (no rigid end zone reduction). Since beam-to-column joints are well-confined, they are estimated to remain un-cracked under such a moderate ground shaking.

3. P-Delta Effect:

    P-Delta effect is included in the analysis.

4. Effective Width of Post-Tensioned Slab as Equivalent Beam:

    According to ATC-40, Hwang and Moehle (1993) recommend an effective beam width model having an effective width for interior framing lines equal to $\beta(5c_1+0.25L_1)$, where β represents cracking effects and ranges typically from one-third to one-half, c_1 = column dimension in the direction of framing, and L_1 = center-to-center span in the direction of framing. For exterior frame lines, use half this width. Use gross-section flexural stiffness properties for the effective width. In the case of prestressed slabs, β is taken as 1.0, since post-tensioned slabs are likely to have less cracking.

Analytical Results

The first three mode shapes from the ETABS analysis are shown in Figures 26 through 29. The first, second, and third modes are in the transverse, longitudinal, and rotational directions, respectively. The corresponding natural periods are 3.27 seconds, 2.23 seconds and 1.71 seconds, respectively.

Nonlinear Analysis

The nonlinear analysis of the building using DRAIN2DX is still on-going. A two-dimensional model will be established in the transverse direction to conduct a time history analysis using the recorded ground motion obtained at the basement of the building. The response of the building from nonlinear analysis will be compared with the recorded data at the roof and 16th floor. Conclusions regarding validity of modeling assumptions and nonlinear analysis will be drawn from this analysis.

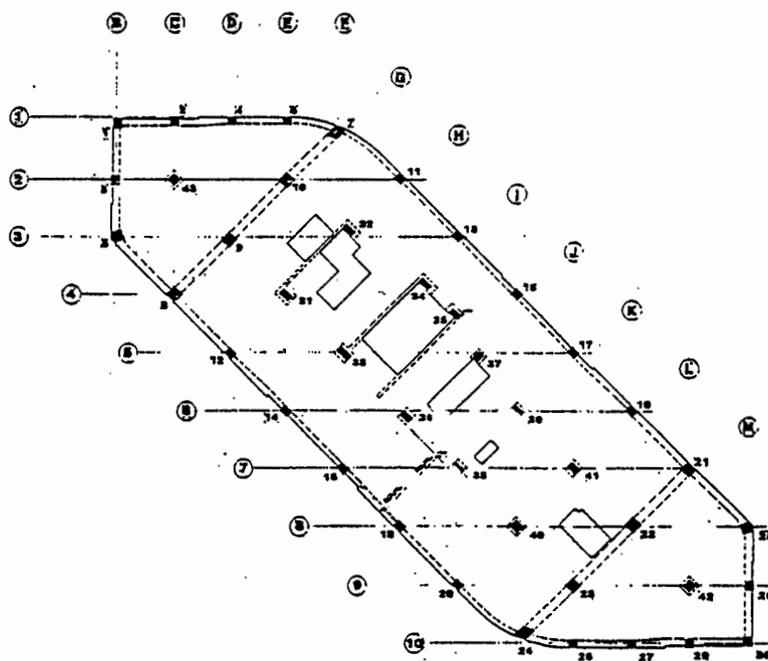Figure 15. Southeast elevation
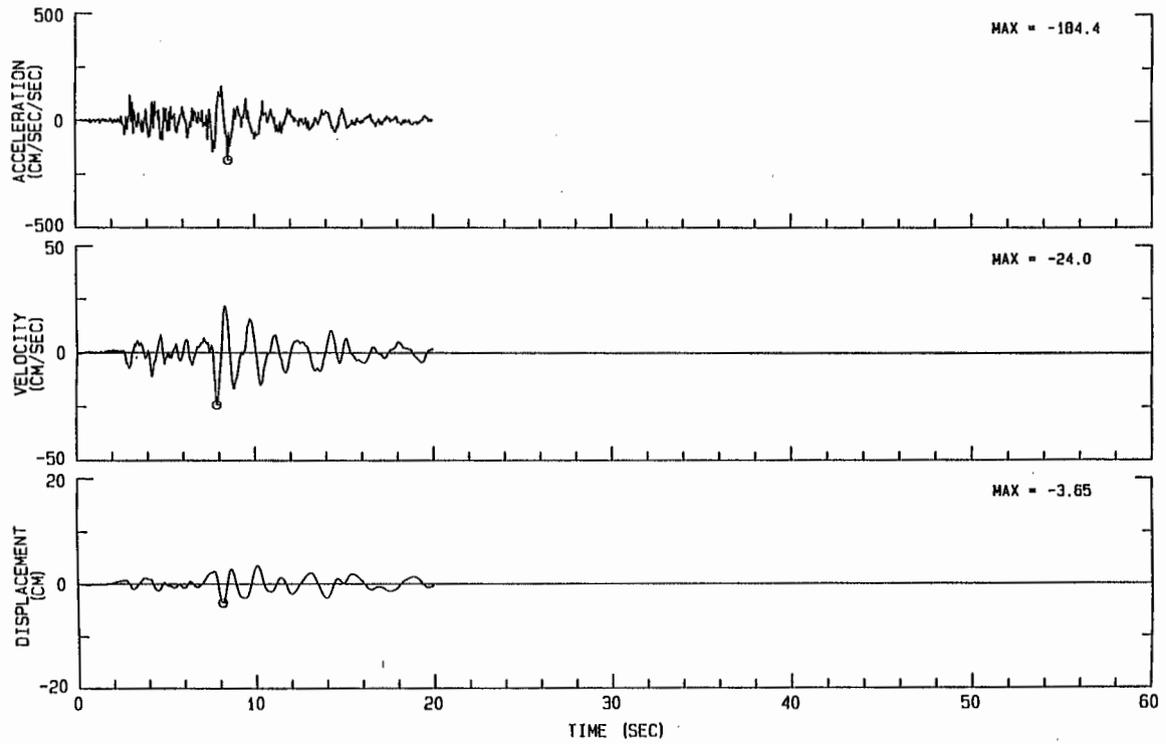

Figure 16. Typical framing plan of Tower Building

Figure 17. Response time histories in the longitudinal direction at the basement level. 1994 Northridge earthquake
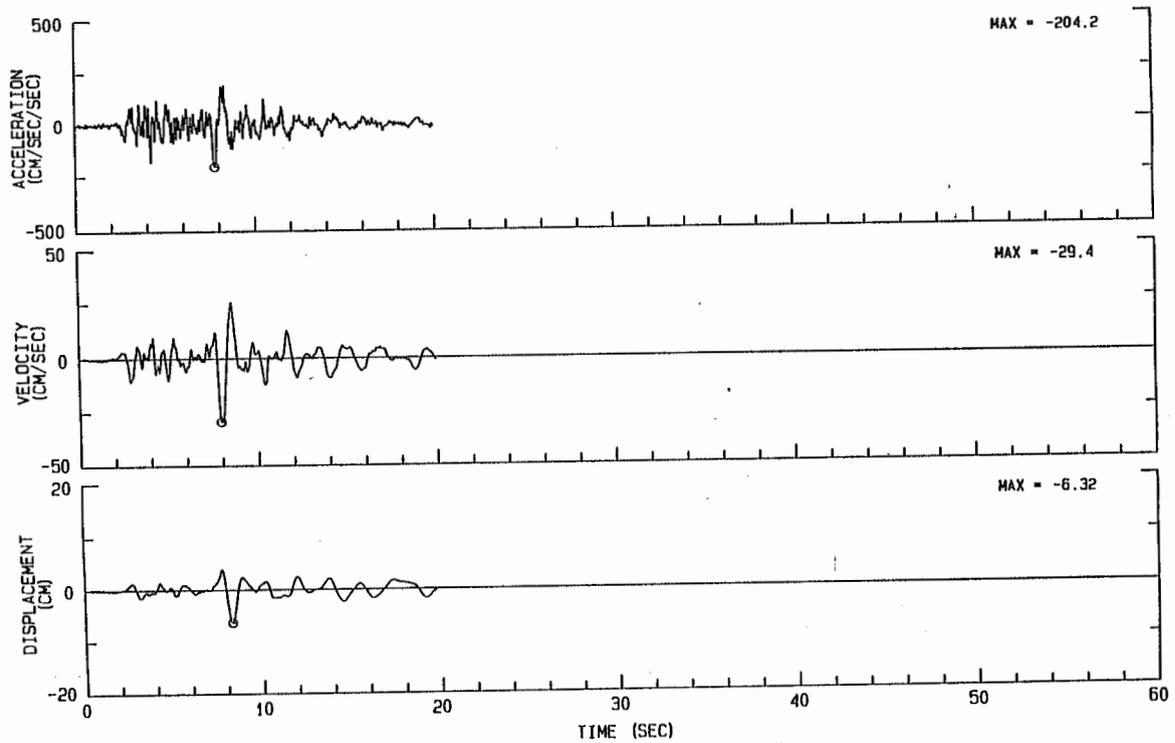
Figure 18. Response time histories in the transverse direction at the basement level. 1994 Northridge earthquake
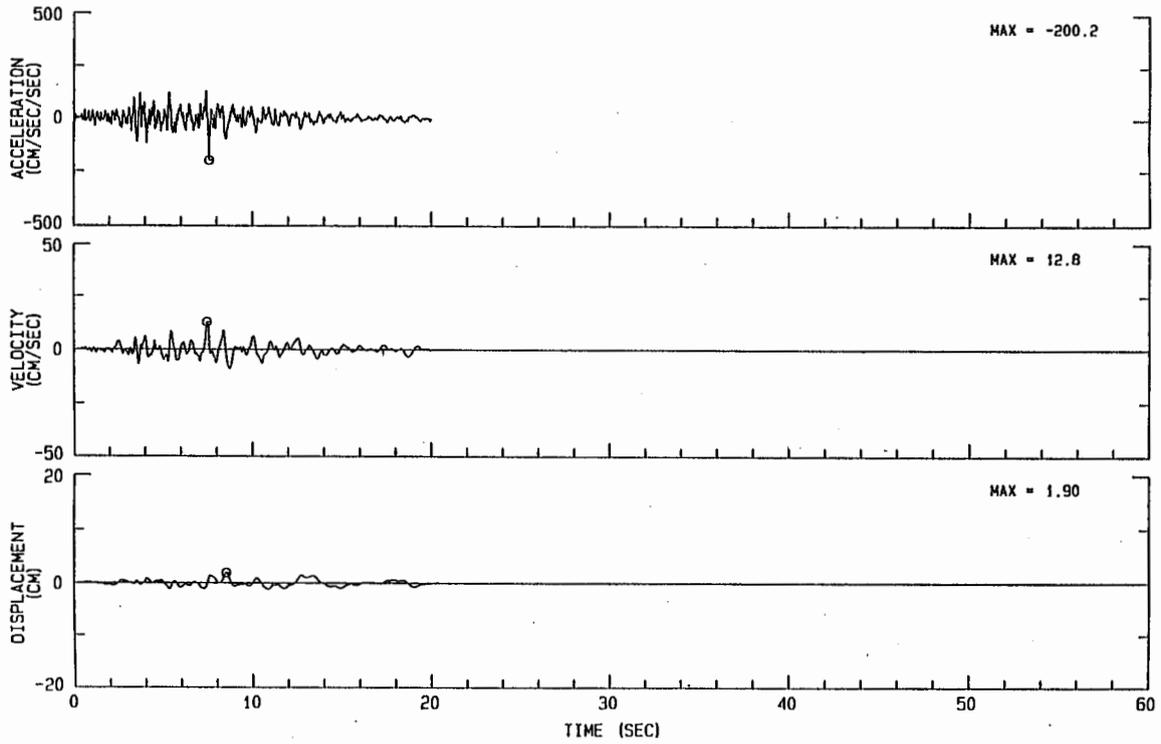
106

Figure 19. Response time histories in the vertical direction at the basement level. 1994 Northridge earthquake
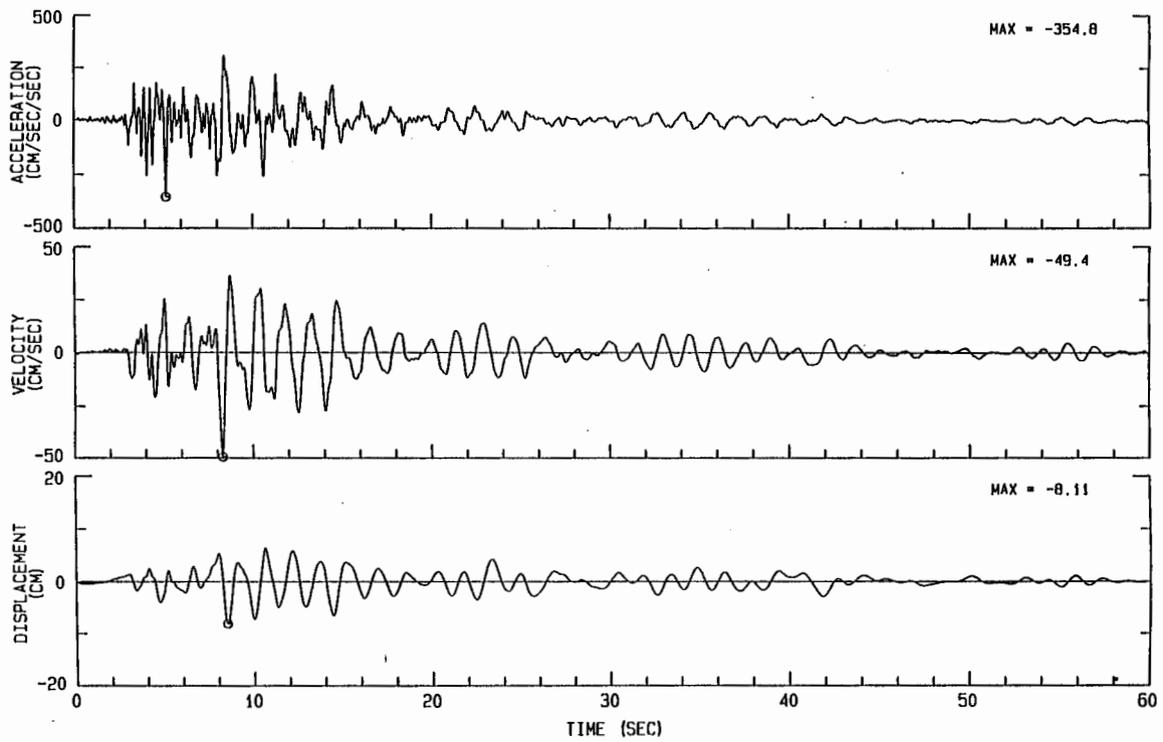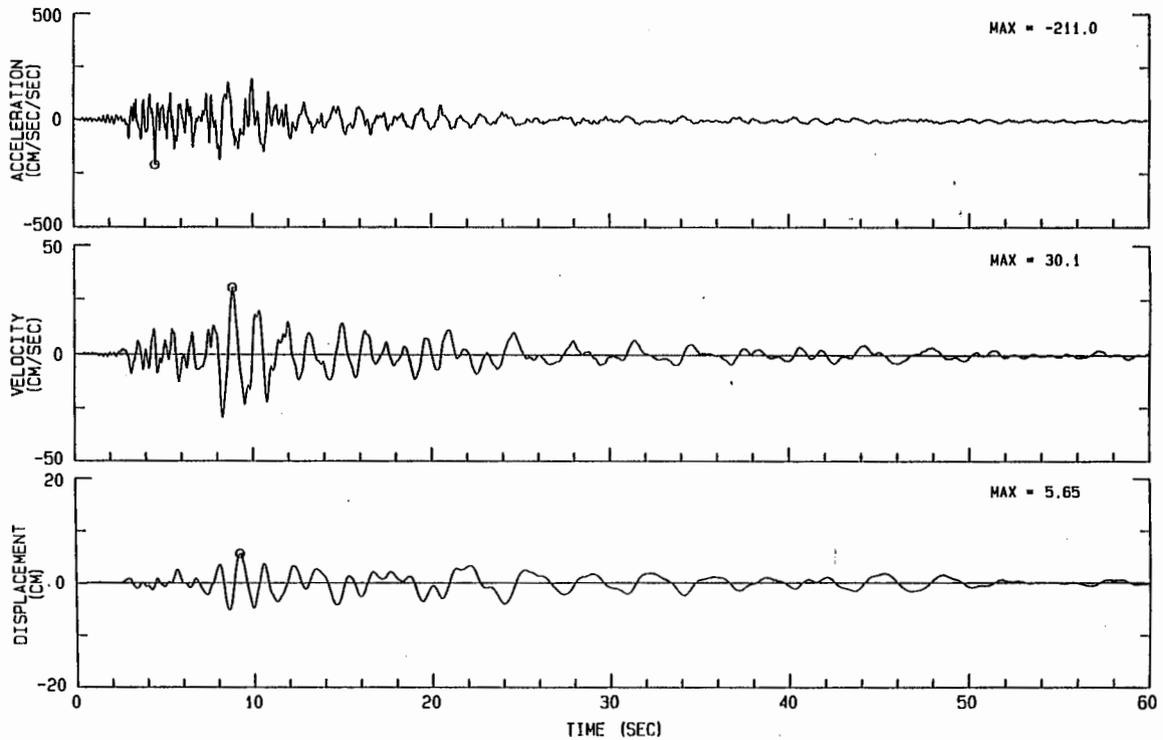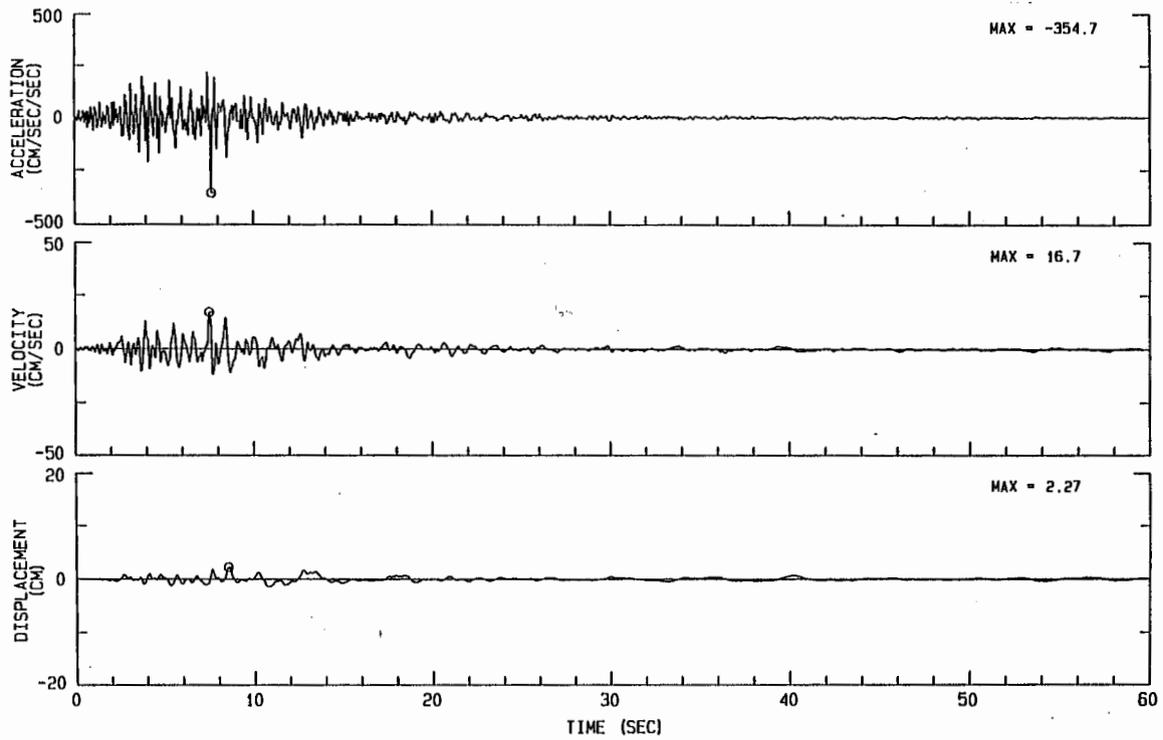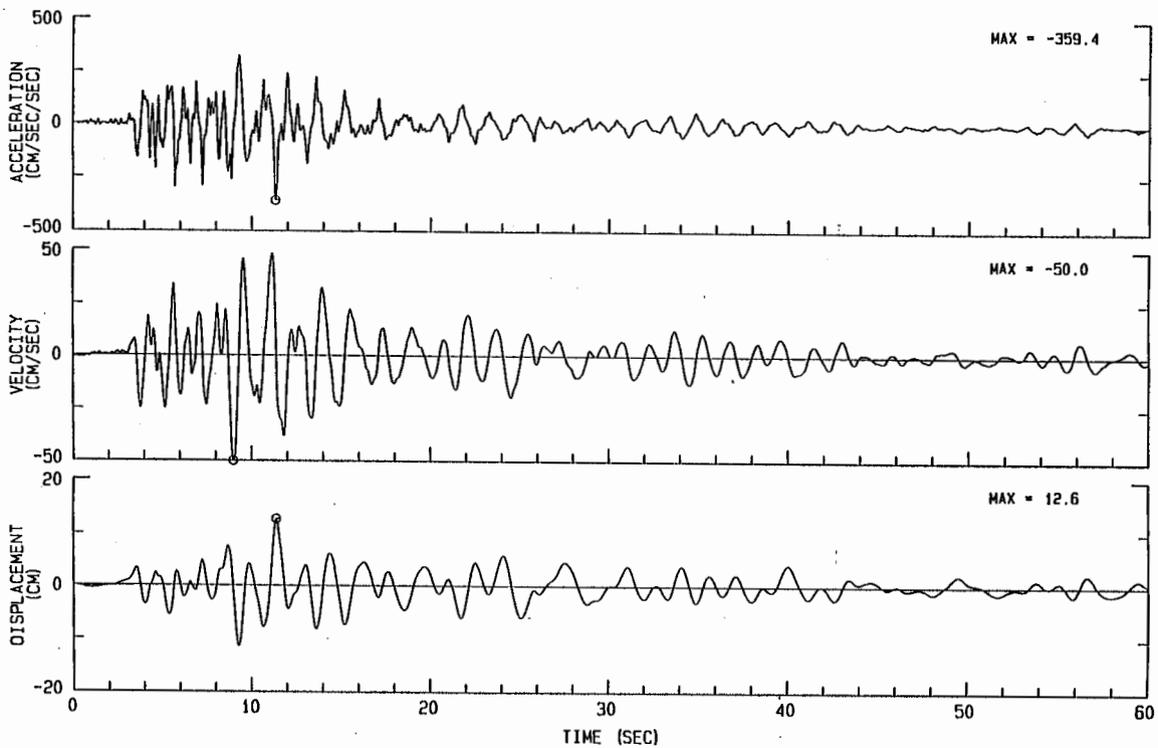
Figure 20. Response time histories in the longitudinal direction at the 16th level. 1994 Northridge earthquake
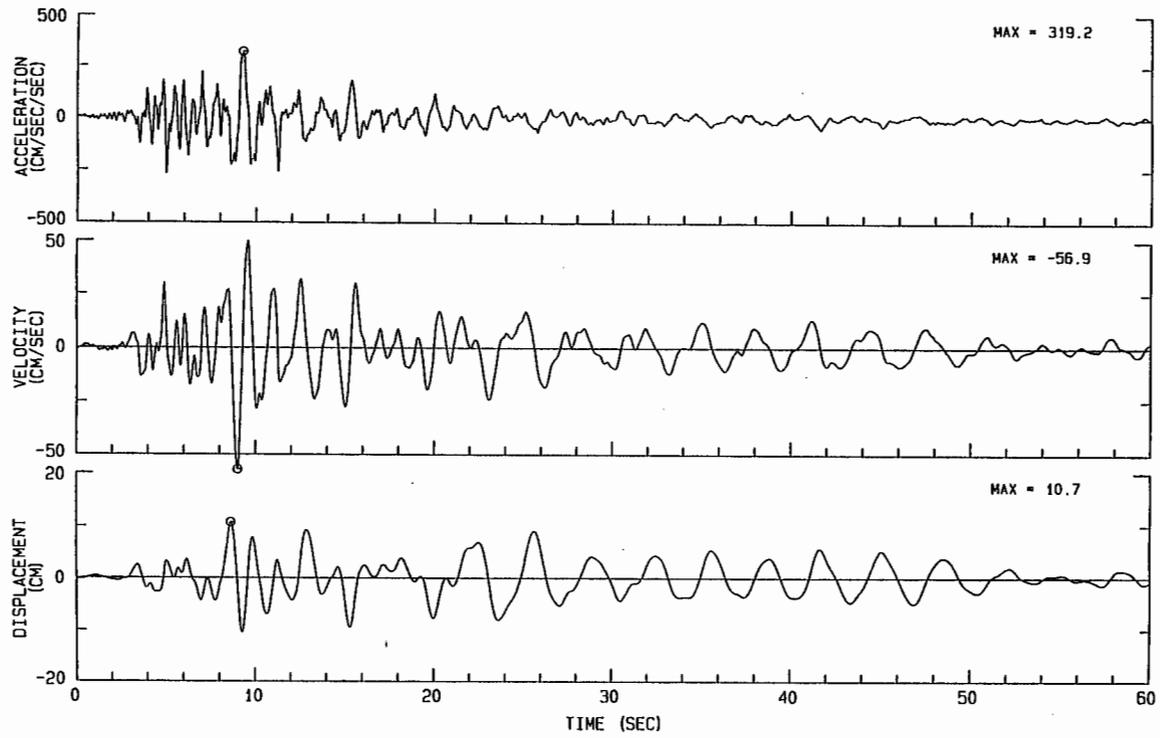

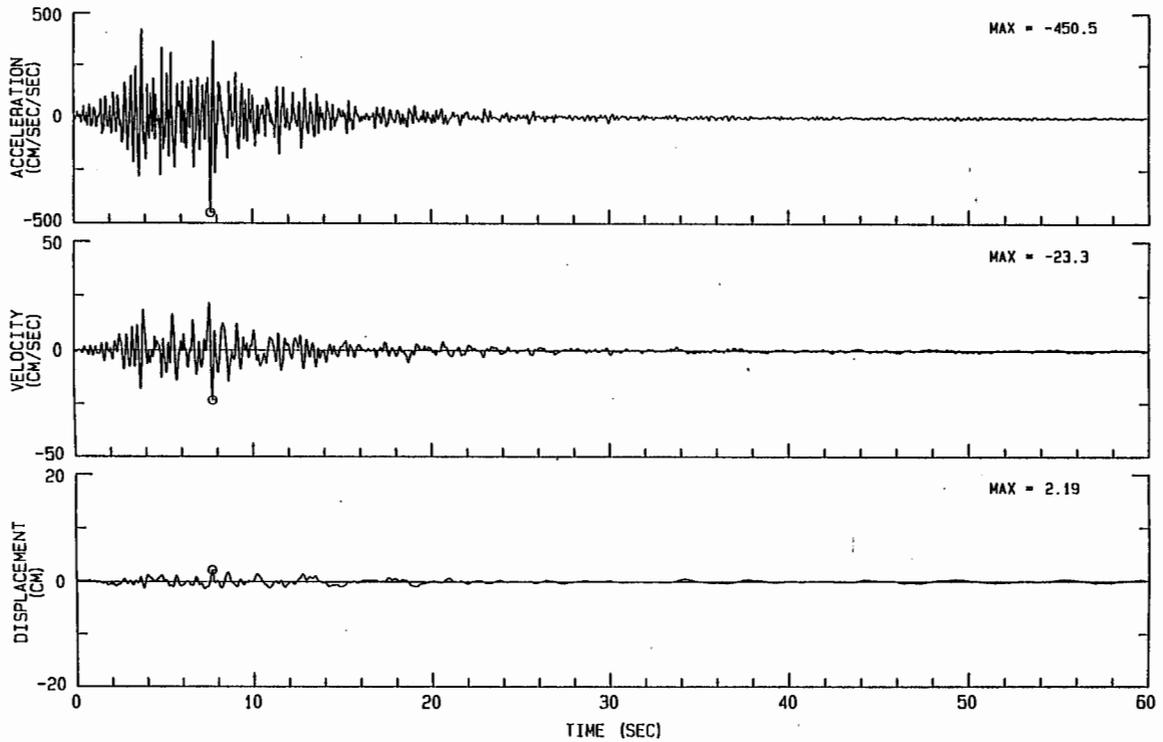Figure 21. Response time histories in the transverse direction at the 16th level. 1994 Northridge earthquake

Figure 22. Response time histories in the vertical direction at the 16th level. 1994 Northridge earthquake

Figure 23. Response time histories in the longitudinal direction at the roof level. 1994 Northridge earthquake

109

Figure 24. Response time histories in the transverse direction at the roof level. 1994 Northridge earthquake

Figure 25. Response time histories in the vertical direction at the roof level. 1994 Northridge earthquake
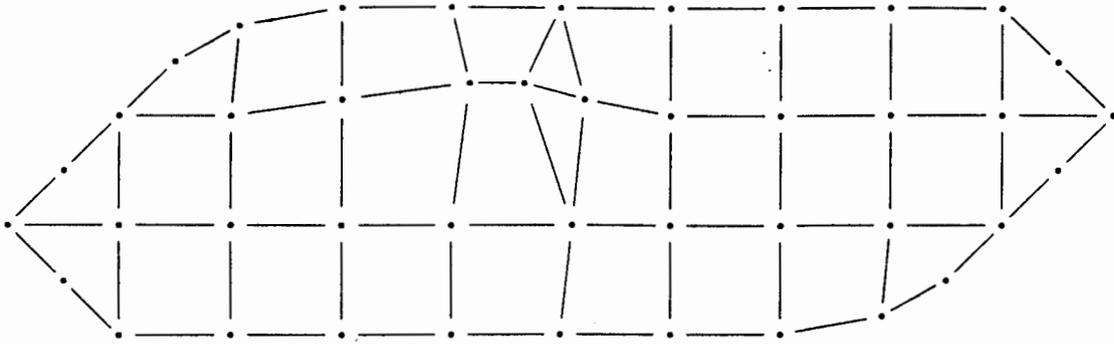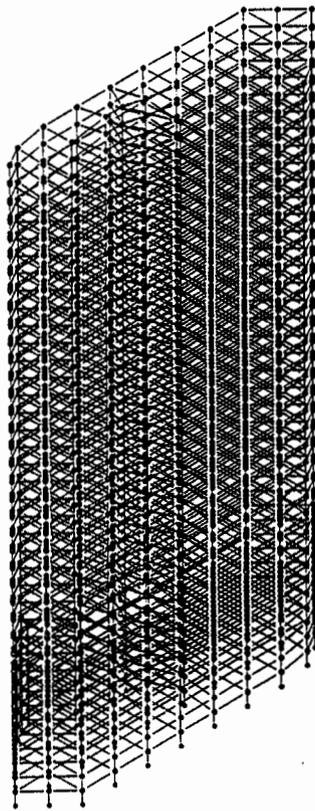
Figure 26. Frame grid line in the ETABS model

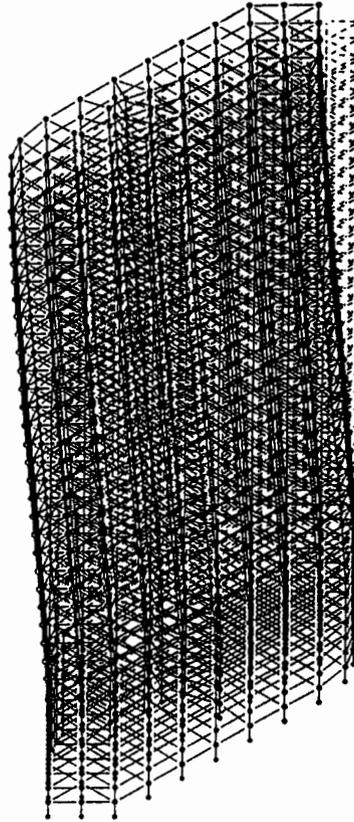Figure 27. Three dimensional view of the ETABS model

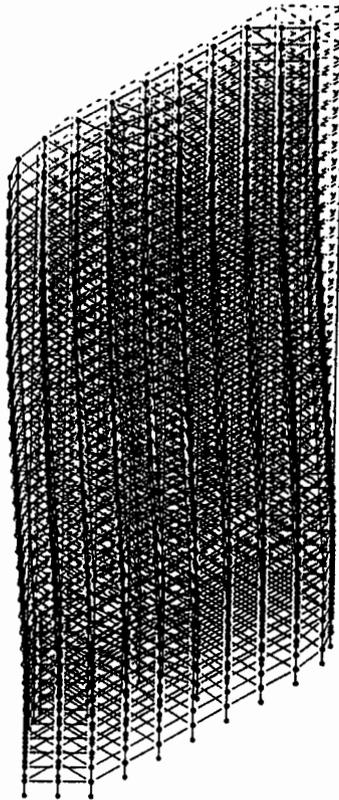Figure 28. First mode shape from the ETABS analysis

Figure 29. Second mode shape from the ETABS analysis

VAN NUYS, 7 STORY HOTEL

Evaluation of this building as a part of this project is in progress. The following information is based on the SMIP Information System (Naeim, 1997).

This 7 story reinforced concrete structure with no basements was designed in 1965 and constructed in 1966. Its vertical load carrying system consists of 8 in. and 10 in. concrete slabs supported by concrete columns, and spandrel beams at the perimeter. The lateral load resisting system consists of interior column-slab frames and exterior column-spandrel beam frames. The foundations consist of 38 inch deep pile caps, supported by groups of two to four poured-in-place 24 inch diameter reinforced concrete friction piles. Sketches of plan and elevation of the building showing the location of sensors are presented in Figure 31.
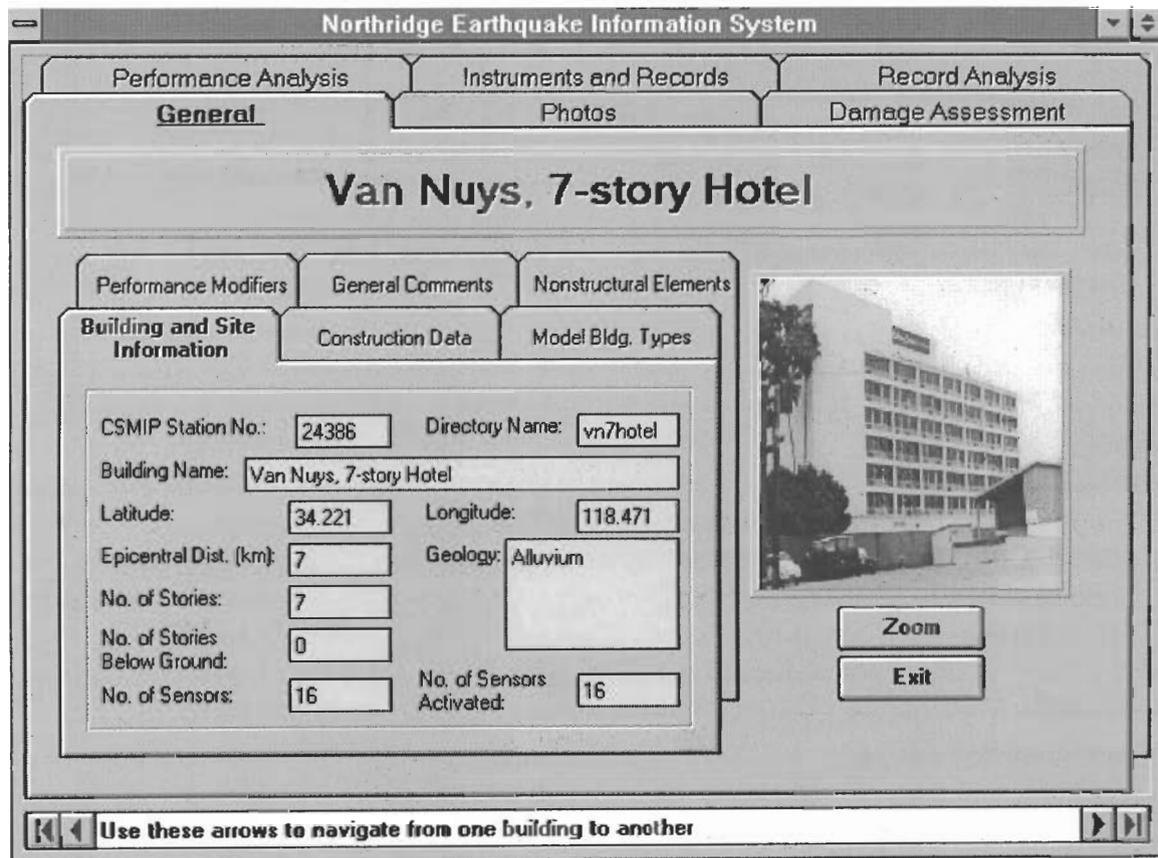

Figure 30. A view of the Van Nuys 7 Story Building (from Naeim, 1997)

The largest peak horizontal accelerations recorded at the basement (Channel 16, E-W) and at the roof (channels in both directions) are 0.45g and 0.58g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

Performance analysis calculations for this building are summarized in Table 4 where the significance of torsion (or differential response) to overall seismic behavior may be clearly seen.

The 0.33W maximum base shear apparently experienced by the building in the E-W direction is significantly larger than the 1964 UBC strength design base shear of about 1.4x0.05W= 0.07W and somewhat larger than the UBC-94 value of about 1.4x0.15= 0.21W for a non-ductile moment resisting frame system (see Appendix B for backup calculations). Notice that the building experienced significant deformation particularly in the E-W direction with an overall drift index exceeding one percent of the height.
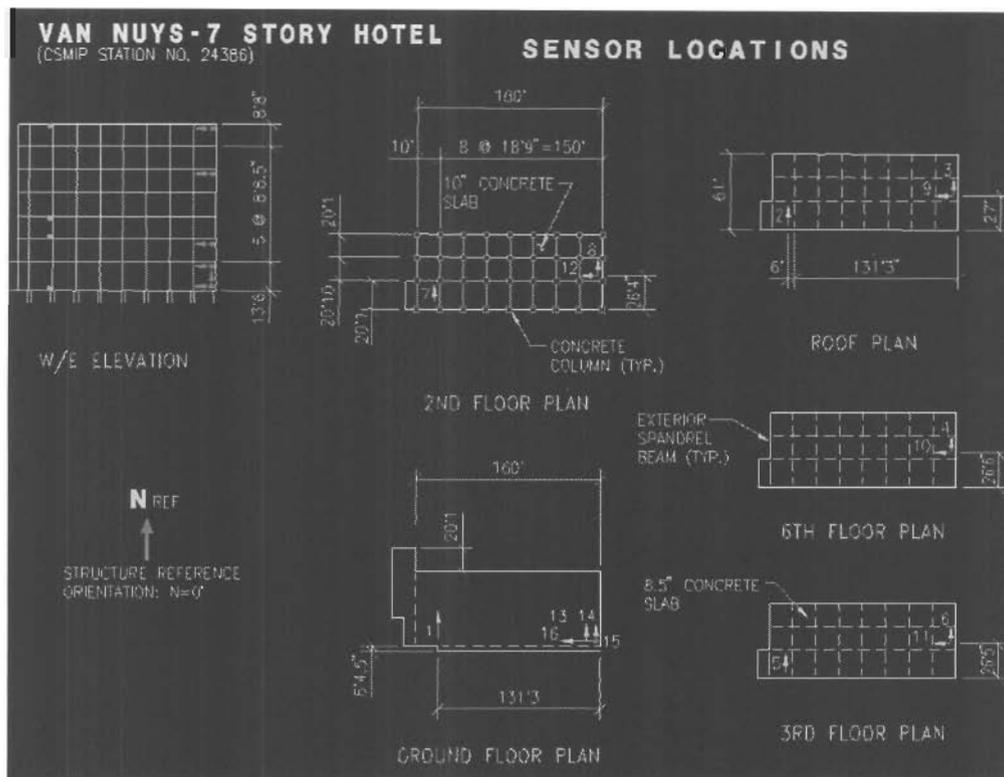

Figure 31. Instrumentation map for the Van Nuys 7 Story Building (from Naeim, 1997)

TABLE 4. Response Summary for Van Nuys 7-Story Hotel (from Naeim, 1997).

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	8.38	27.68
	E-W	9.24	33.30
	DIFF	8.56	40.46
Overturning Moment (% Total Weight x feet)	N-S	8.38	830
	E-W	9.24	1058
	DIFF	4.56	1070
Roof Lateral Displacement Relative to the Base (cm)	N-S	10.68	19.82 (0.0099)*
	E-W	9.36	23.36 (0.0117)*
	DIFF	8.74	13.91 (0.0069)*

* Overall drift index values are shown in brackets.

The building had suffered minor structural damage and extensive nonstructural damage during the 1971 San Fernando earthquake and was subsequently repaired. The building experienced heavy damage during the Northridge earthquake where the South side exterior columns at fourth floor failed in shear (Figure 32). The building was repaired following the Northridge earthquake and the structural system in the E-W direction was changed to a shear-wall frame interaction system. The content damage was also heavy as documented in the information system. Surprisingly however, the mechanical equipment installed at the roof did not suffer any noticeable damage. The racking of the fourth floor was so significant that some of the hotel room doors needed to be opened using sledge hammers to get the occupants out (Figure 33).

A series of moving windows FFT analysis by Naeim, 1997 indicated an initial E-W fundamental period of about 1.4 seconds which elongates to about 2.2 seconds towards the end of the strong motion (Figure 34). Similar analysis shows a more moderate period elongation in the N-S direction from about 1.3 to 1.8 seconds, except when the kick from the E-W failure is captured (Figure 35). These periods are more than twice the code estimated periods of about 0.7 seconds.
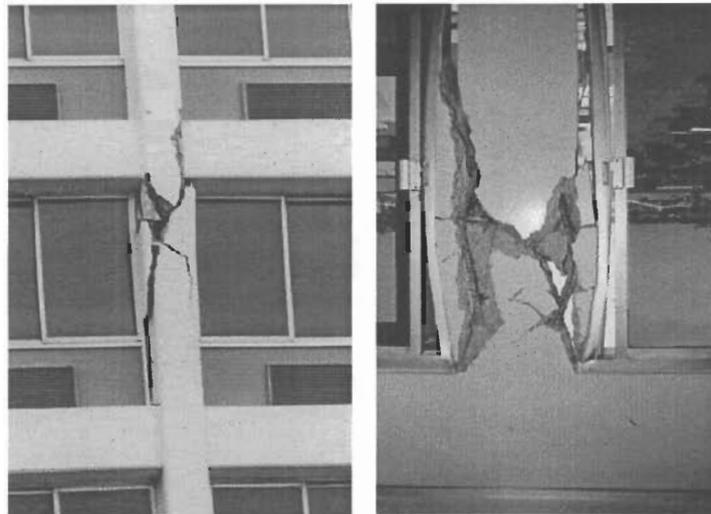

Figure 32. Shear failure of column at the fourth floor (Naeim, 1997).

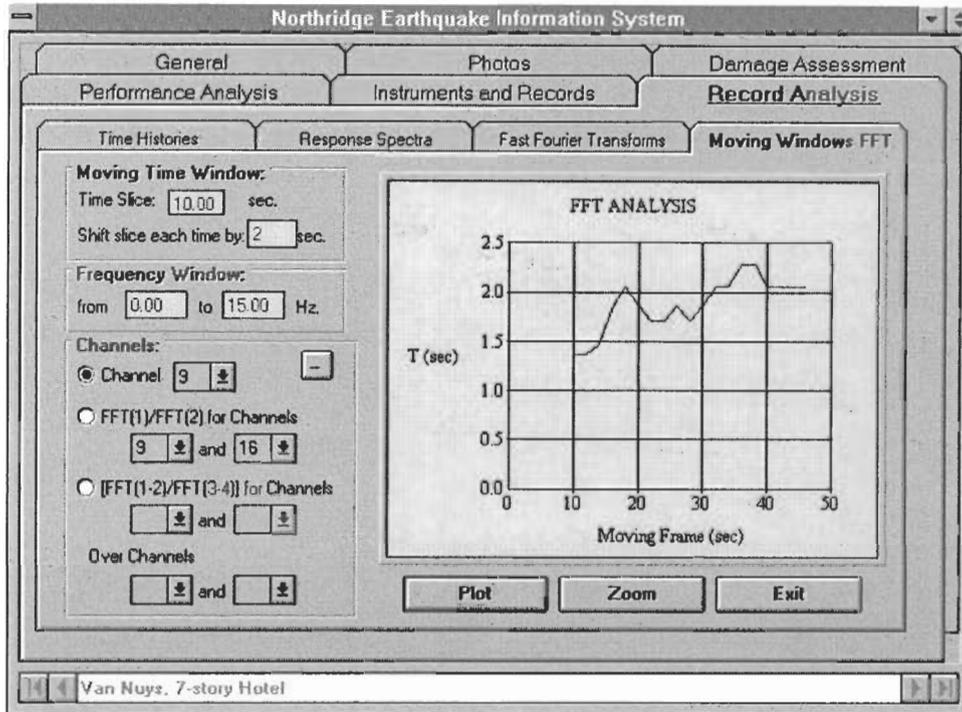Figure 33. Doors had to be opened with sledge hammers (Naeim, 1997).

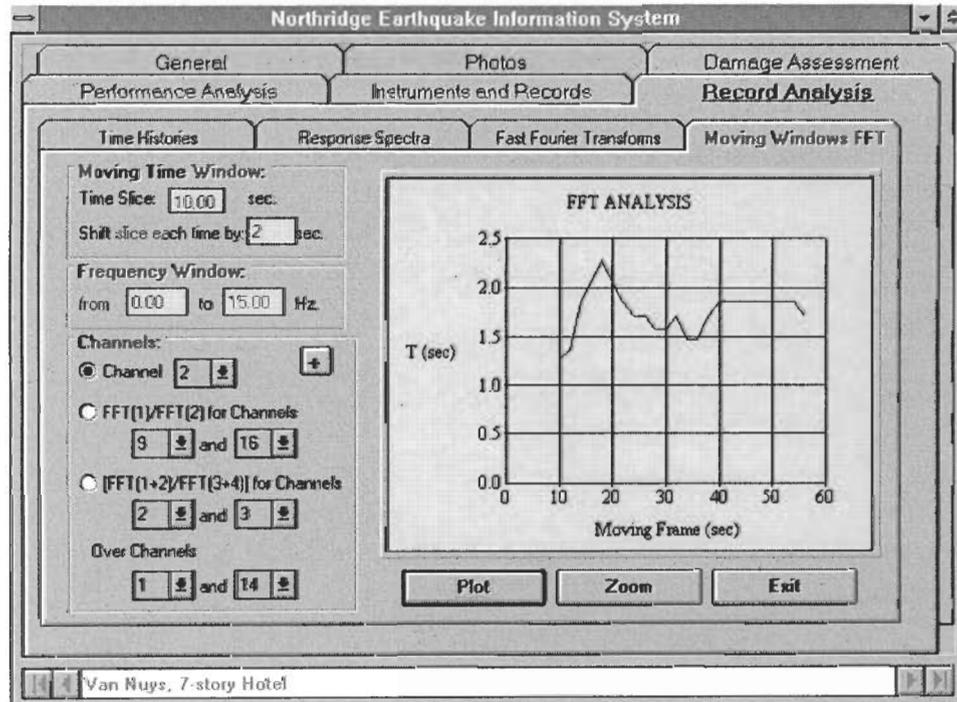Figure 34. Moving windows FFT analysis for E-W response (Naeim, 1997).

Figure 35. Moving windows FFT analysis for E-W response (Naeim, 1997).

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