EVALUATING DESIGN PROVISIONS AND ACTUAL PERFORMANCE OF A MODERN HIGH-RISE STEEL STRUCTURE

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ABSTRACT

The objectives of this project were to study CSMIP records obtained during the October 17, 1989 Loma Prieta earthquake at a 49-story steel structure and to conduct an investigation of current structural engineering design procedures related to the response. The recorded data indicated that the top 6 stories of the building have experienced much greater drift than lower floors due to discontinuity of mass and stiffness. The results of elastic and inelastic dynamic analyses compared to CSMIP records confirmed validity of many design assumption currently used while resulting in better understanding of actual behavior of these modern structures and possible refinement of design procedures.

INTRODUCTION

The California Strong Motion Instrumentation Program (SMIP) of Division of Mines and Geology of California has many strong motion recording stations throughout the greater San Francisco Bay Area. When Loma Prieta earthquake of 1989 occurred, a 49 story instrumented steel high rise in San Francisco was shaken among many other structures. The building had 18 accelographs and all instruments recorded more than 120 seconds of valuable acceleration response of the building. This paper summarizes important aspects of a study of the response of this building during the Loma Prieta earthquake and lessons learned.

The building is located in downtown San Francisco and was designed in 1977-78 and its construction was completed in 1979. A view of the building is shown in Figure 1. The seismic design was according to UBC-76 and included modal analysis and response spectra analyses (3). The floor system consists of 2.5 inch concrete over a 3 inch metal deck connected to steel framing by shear studs and puddle welds. The structural framing system consists of special moment resisting space frames in both East-West and North-South directions. However, for extra lateral stiffness, moment frames in narrow direction (N-S) have two bays braced using eccentric braces. The length of shear links in eccentric braces is about 4.5 feet. The building is supported by a 5 feet thick mat foundation which in turn is supported by 150-200 feet deep composite steel/concrete piles. Figure 2 shows details of a typical floor and the East-West and North-South frames.

The building is instrumented by the Strong Motion Instrumentation Program of California Department of Mines and Geology of Department of Conservation and has 18 accelerographs installed at various levels and directions as shown in Figure 3. During the Loma Prieta earthquake, all strong motion instruments were activated and all have collected reliable data for more than 120 seconds.

RESEARCH METHODOLOGY

Major phases of the study were:

1. Collect data on geometry, material, non-structural elements, equipment, dead and live load and any damage.

2. Obtain, process and analyze data recorded by CSMIP.

3. Construct realistic elastic and inelastic computer models of the structure above the top of foundations.

4. Subject the elastic computer model of the structure to base excitations recorded during the Loma Prieta earthquake using ETABS computer program and study the response and predictions of the dynamic analysis. The ETABS software represents typical dynamic analysis software used in design offices today.

5. Subject the inelastic 2-dimensional computer model of the structure to base excitations recorded during the Loma Prieta earthquake as well as to scaled-up base excitations. The objective here was to obtain insight to the inelastic behavior of the structure during future strong earthquakes.

6. Study the code provisions and seismic design practice used in design of the building and investigate the adequacy and accuracy of the current seismic design practice.

7. Formulate recommendations with regard to refining the existing instrumentation installed in the building.

8. Formulate recommendations that can be used to improve seismic design practice and code provisions.

RECORDED RESPONSE OF THE BUILDING TO LOMA PRIETA EARTHQUAKE

A. Accelerations

Figure 4 shows time histories of the E-W and N-S components of acceleration recorded at the 44th floor and at the Basement "B" level (4). The ratios of maximum peak acceleration of 44th floor to basement B were 2.53 in E-W direction and 4.73 in the N-S direction.

SIGNIFICANT RESULTS

A report on the study (1) is in preparation which provides detailed information on the study. Due to space limitations, some significant results available at the time of this writing are provided here.

A. Period

Table 1 shows selected periods of vibration of the structure obtained from the recorded data as well as from various analyses.

Table 1. Selected Modal Periods of Vibration

Mode Number	CSMIP Records	TABS-4 (Ref.3)	ETABS 3-Dim.	ETABS 2-Dim.	N/10 Rules	UBC 1976	UBC 1991
1	6.5X	6.41X	6.54X	6.62X	4.90X	4.70X	4.05X
2	--	5.13Z	5.09Z	--	--	--	--
3	5.0Y	5.00Y	4.70Y	5.14Y	4.90Y	4.00Y	4.05Y
4	2.0X	2.34X	2.35X	2.36X	1.63X	--	--
5	--	1.80Z	1.72Z	--	--	--	--
6	1.8Y	1.70Y	1.69Y	1.72Y	1.63Y	--	--
7	1.3X	1.41X	1.39X	1.39X	.98X	--	--
8	--	1.05Z	1.01Z	--	--	--	--
9	1.0X	1.03X	1.00X	1.00X	.98Y	--	--
10	--	.98Y	.98Y	.99Y	--	--	--

NOTES: X and Y indicate modes in E-W and N-S directions respectively.
T indicates torsional mode.

B. Damping Ratio

A preliminary analysis of the CSMIP data indicated that the damping during the earthquake was about 1.7 for the N-S braced frame direction and about 2.0-2.6 percent for the E-W moment frame direction. However, the results of elastic analyses matched the recorded data better when a constant critical damping ratio of 3% was used. In the inelastic analyses, a critical damping ratio of 2.75 was used.

C. Deflected Shape of the Structure

Figure 5 shows the animated plots of the CSMIP recorded displacements in the N-S and E-W direction. The motion of the structure in the N-S direction was dominated by the higher modes during the first 30 seconds of the motion while ground motion was being applied. After the first 30 seconds, the vibration of the structure was dominated by the first mode. In the E-W direction the motion was dominated by the first mode throughout the recorded motion. Figure 6 shows drift ratios for N-S and E-W directions. As figure indicates, drift ratios for top six stories in the N-S direction were relatively high compared to the rest of the structure.

Maximum displacement in the N-S direction occurred after 47 seconds of motion and was equal to 6.45 inches at 44th floor. The maximum displacement in the E-W dirction occurred during first 30 seconds and was equal to 10.67 inches at 44th level.

Figure 7 shows a comparison of the displacement histories of 44th floors in the E-W direction obtained from elastic analysis and CSMIP records. The elastic analysis was done using ETABS and the structure was modeled as a modern structure would be modeled in a design office without very refined research oriented modeling. The comparison shows that predictions of the current dynamic analyses in design offices for this case were very good compared to the recorded response.

In order to obtain an understanding of inelastic response of the structure, two inelastic 2-dimensional time history analyses were conducted. In one analyses, acceleration time history recorded at the basement of the building was considered to be a representative of magnitude 7 earthquake and was used as base excitation. In the second inelastic analysis, to cause severe yielding, the amplitude of the Loma Prieta acceleration time history was multiplied by 2.75 to obtain a base excitation history that will represent a magnitude 8.3 severe earthquake.

In seismic design of the structure (3) two response spectra with maximum peak accelerations of .84g and 1.16g were developed to represent magnitudes 7 and 8.3 earthquakes respectively. These spectra are shown in Figure 8 along with response spectra of Loma Prieta record at the building basement level as well as a spectra corresponding to 2.75 times Loma Prieta record. The design spectra which were based on the assumption of the rupture of nearby faults and on the assumption of rock support shows peaks over the short periods of abouts 0.4 seconds whereas the Loma Prieta spectra show larger amplifications for longer periods of about 1.5 seconds.

Figure 9 shows a comparison of the displacement time histories for 16th and 44th floors due to the Loma Prieta record, 2.75 time the Loma Prieta and the CSMIP recorded response. The analyses indicated that the E-W frames remained elastic during the Loma Prieta earthquake and experienced inelasticity and plastic hinge formations when subjected to 2.75 times Loma Prieta acceleration records. Figure 10 shows plastic hinges that formed during the initial 60 seconds of the motion when the frame on line "E" was subjected to 2.75 times the Loma Prieta record of the Basement B.

CONCLUSIONS AND RECOMMENDATIONS

The comparative study of the response data recorded by the CSMIP and the results of elastic and inelastic dynamic analyses indicated that:

1. The currently available design office computer programs used for 3-D dynamic analyses, adequately predicted the response of the building to Loma Prieta.
2. The stiffness and mass discontinuities of the top 6 stories affected the behavior of the whole structure significantly.
3. It appears that the structure will experience hinge formations and inelasticity during a magnitude 8.3 earthquake, however, maximum displacement of 44th floor in the E-W direction will be in the order of 27 inches.
4. To capture a obtain a cimplete set of strong motion data, it is recommended that at least 10 more instruments be added to this structure to capture E-W response and vertical response of cantilevers more accurately.

ACKNOWLEDGMENTS

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REFERENCES

1. Astaneh, A., Bonowitz, D. and Chen, C. "Studies of a 49-Story Instrumented Steel Structure Shaken During the 1989 Loma Prieta Earthquake", EERC Research Report, in preparation, Earthquake Engineering Research Center, University of California at Berkeley, to be released in 1991.

2. "CSMIP Strong Motion Records from the Santa Cruz Mountains (Loma Prieta) California Earthquake of October 17, 1989), 1989

3. Merovich, A.T., Nicoletti, J.P., and Hartle, E., "Eccentric Bracing in Tall Buildings", Journal of Structures Division,, ASCE, Vol 108, NO. ST9, September 1982, pp. 2066-2081.

4. "Plots of the Processed Data for San Francisco 47-Story Office Building from the Santa Cruz Mountains (Loma Prieta) Earthquake of October 17, 1989", CSMIP, 1990.

5. "Uniform Building Code", 1976 & 1991 Editions, International Conference of Building Officials, Whittier, CA. 1976, 1991.

Figure 1. A View of the Structure

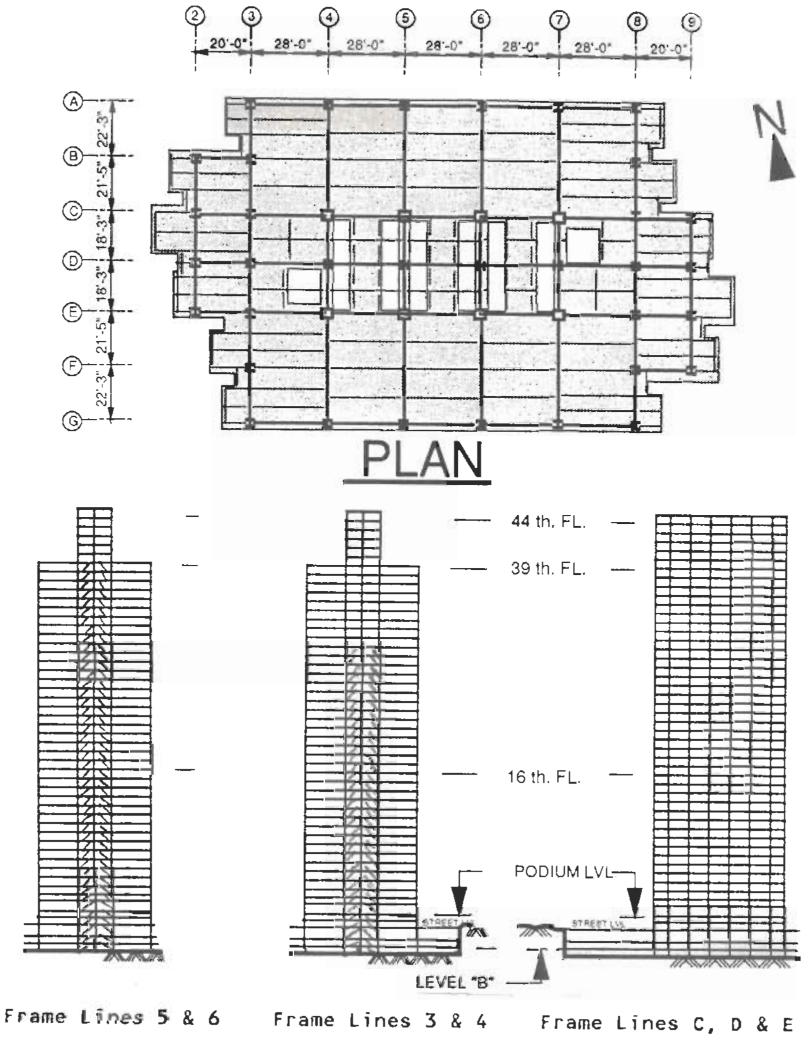


Figure 2. Typical Framing Plan, East-West and North-South Frames

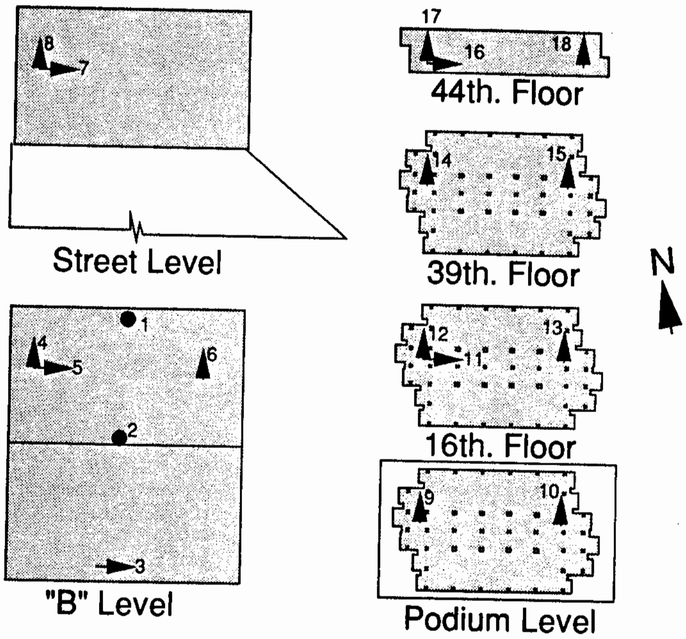


Figure 3. Plan of SMIP Instrumentation

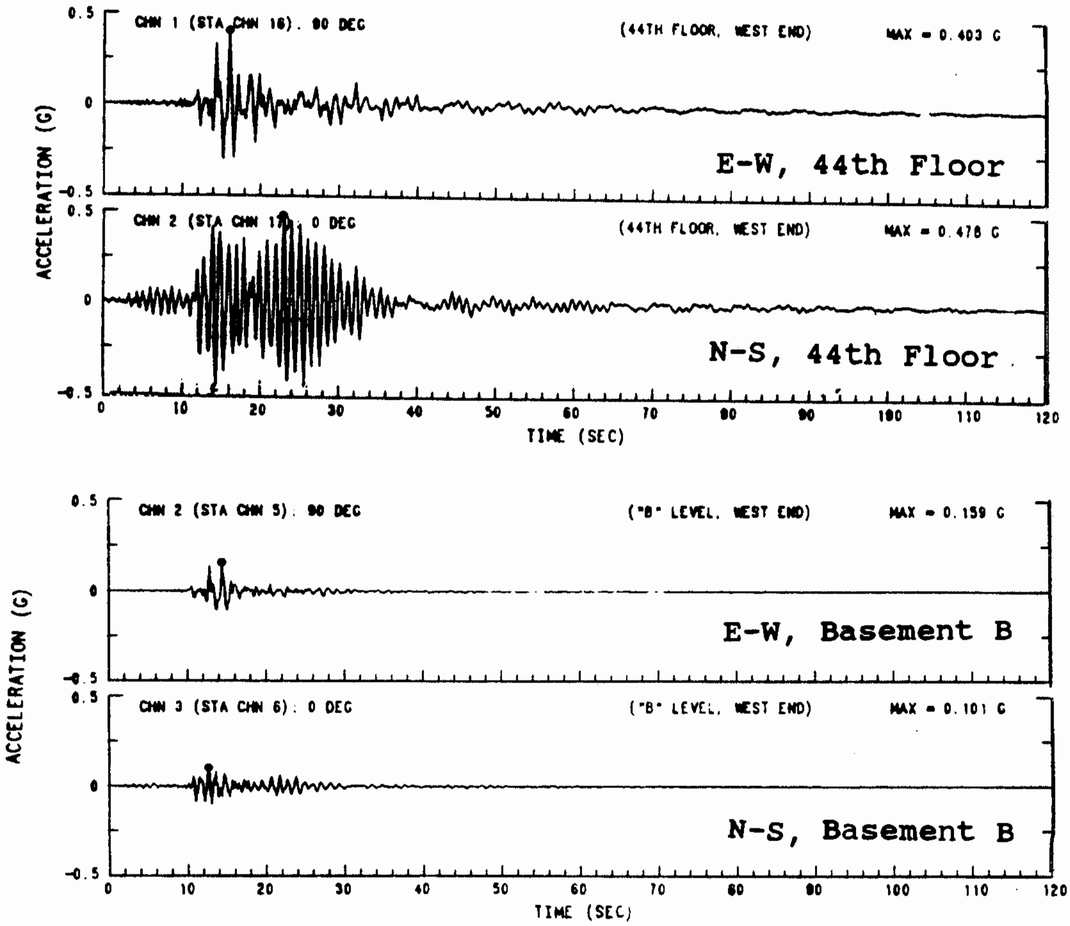
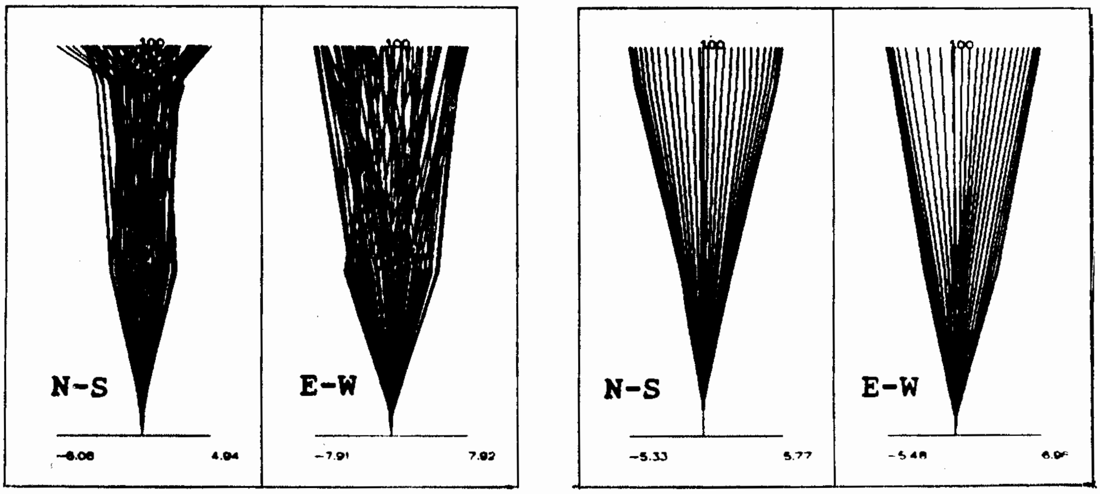


FIgure 4. Acceleration Time Histories for 44th Fl. and Basement



(a) (b)

Figure 5. Animated Deflected Shape of the Structure
(a) During Ground Shaking
(b) After Ground Shaking Subsided

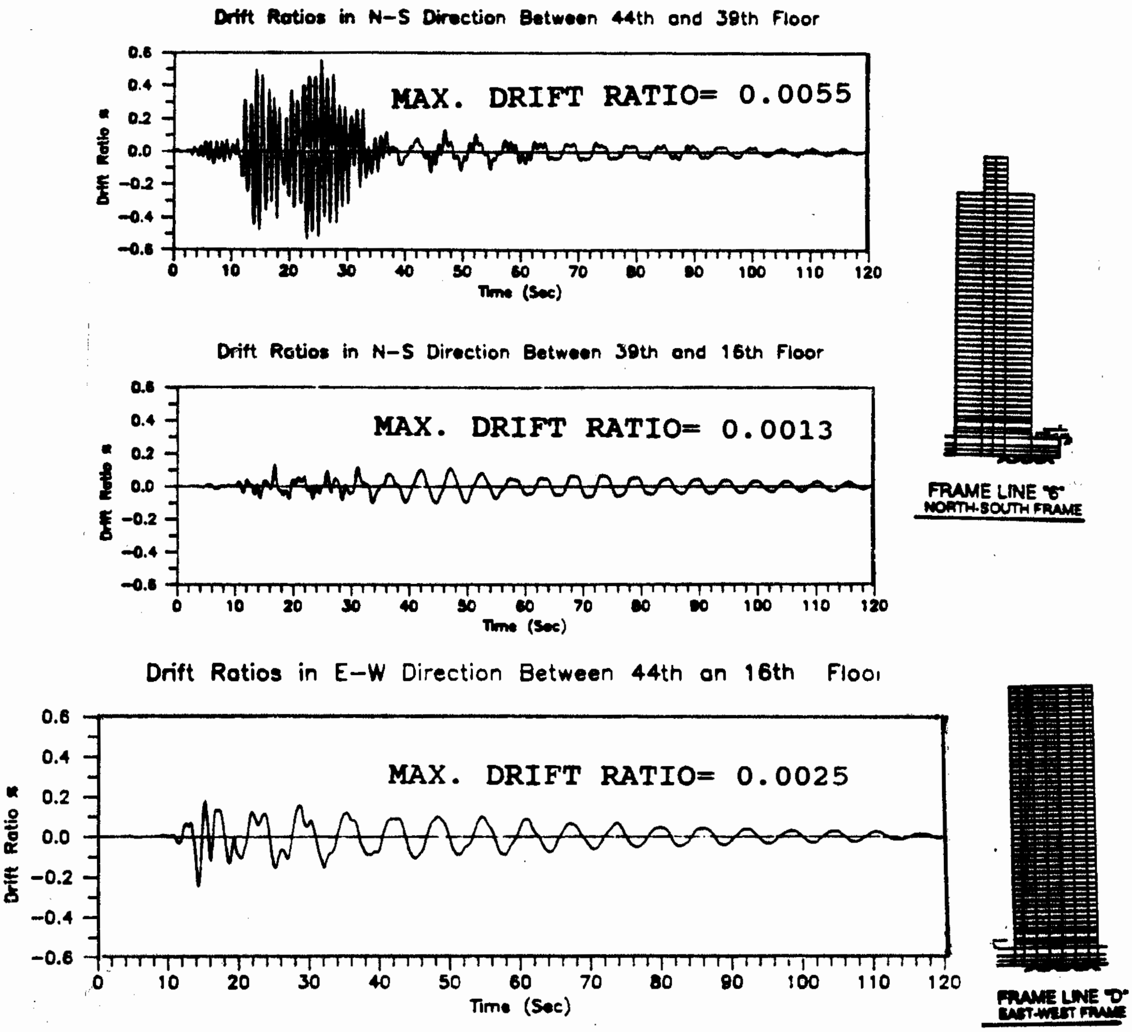


Figure 6. Drift Time Histories

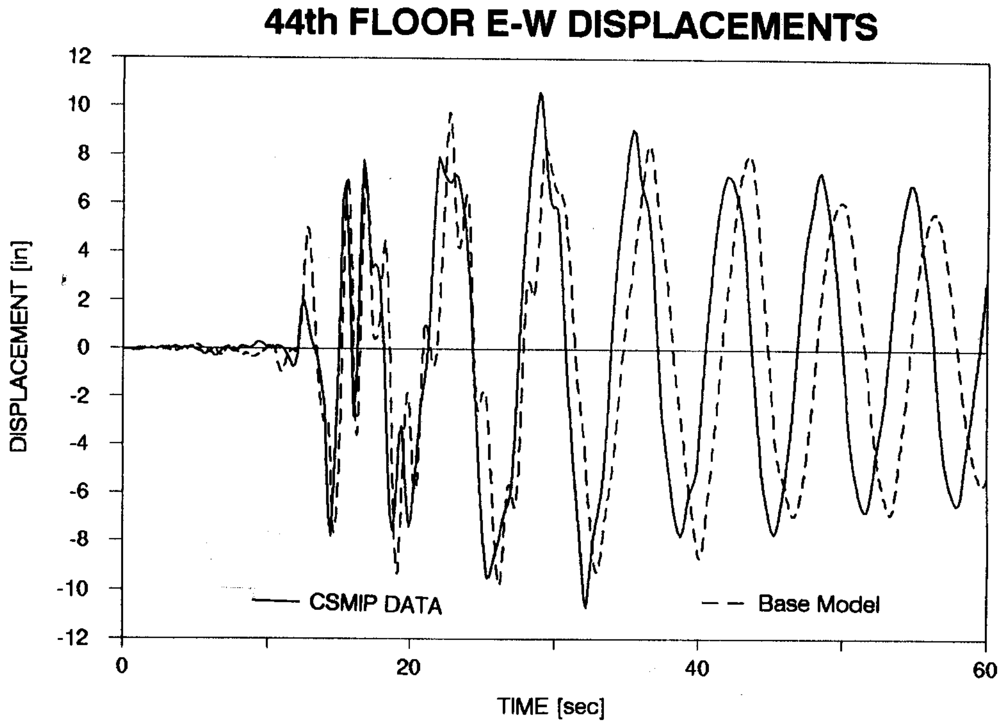


Figure 7. Comparison of the Displacement Time Histories

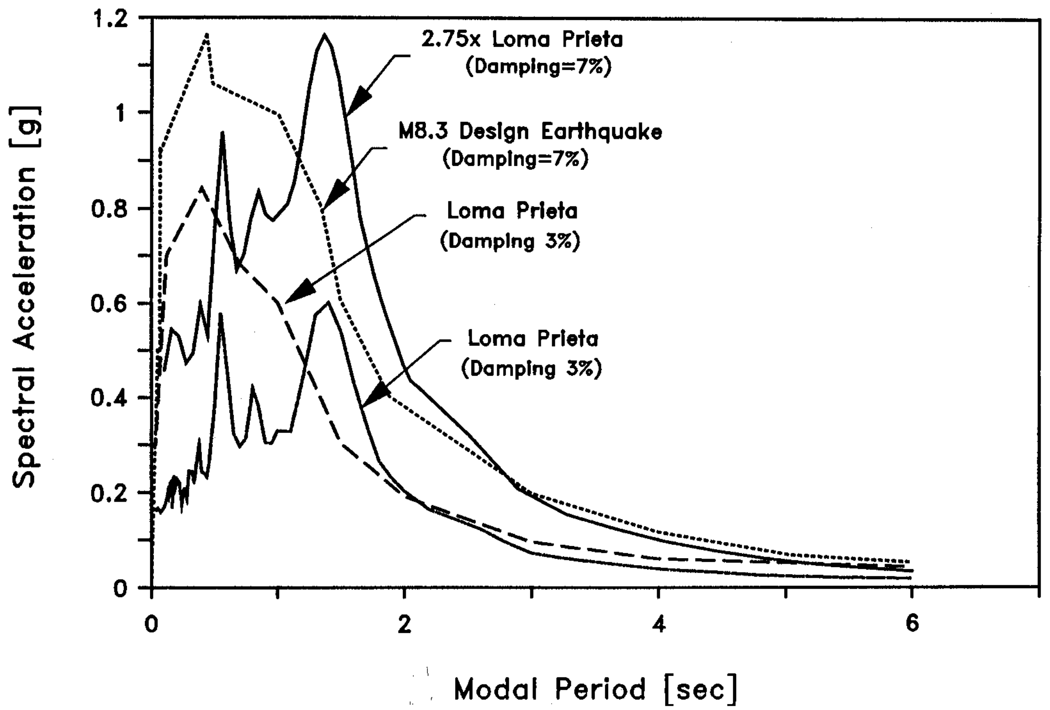


Figure 8. Response Spectra Used in Design and Obtained from Loma Prieta

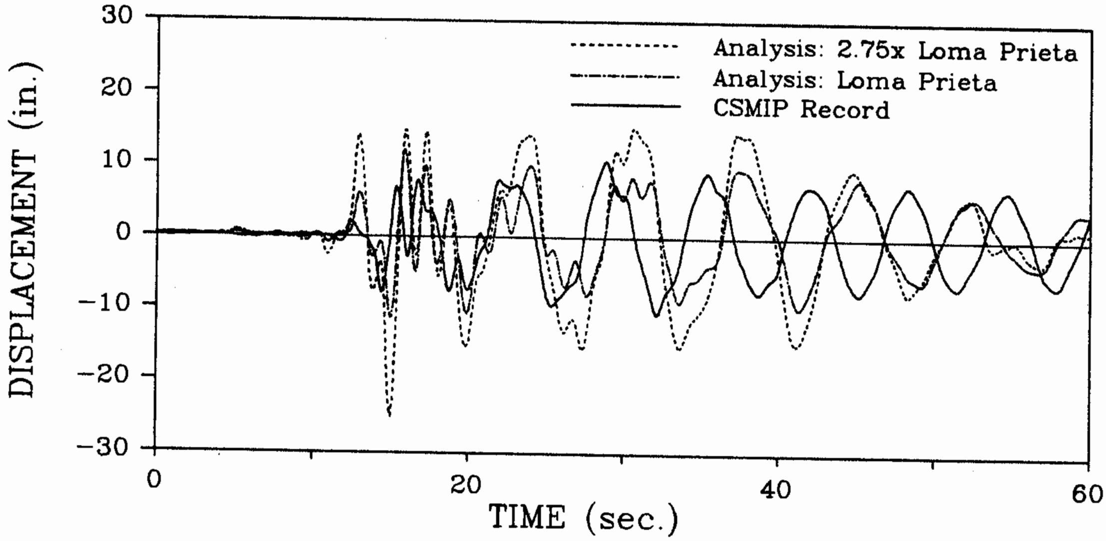


Figure 9. Displacement Time Histories for the E-W Direction Obtained from Inelastic Analyses

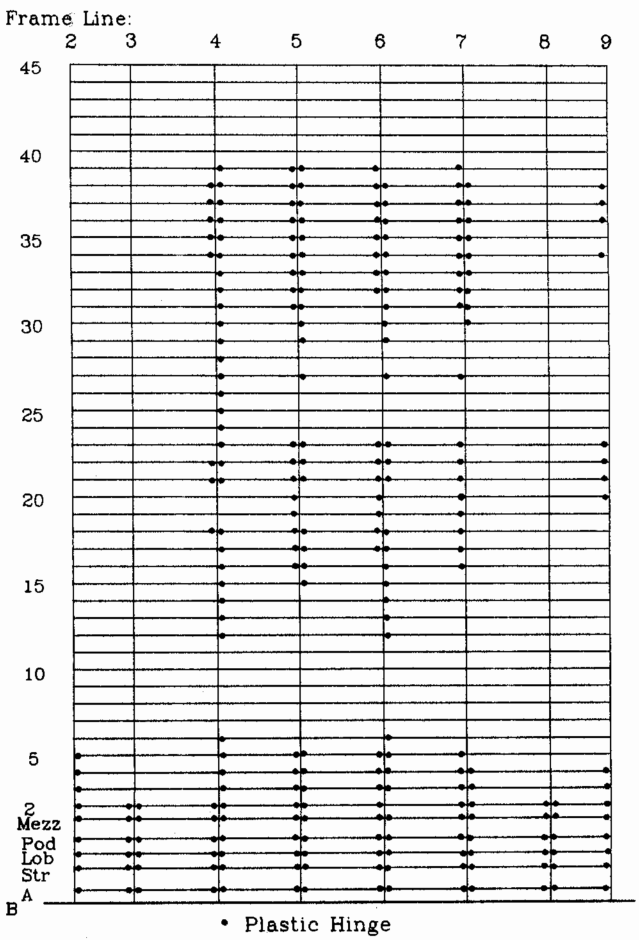


Figure 10. Plastic Hinges Formed in the E-W Frame