ANALYSIS OF STRONG MOTION RECORDS FROM NON-DUCTILE CONCRETE MOMENT FRAME BUILDINGS

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Abstract

The California Strong Motion Instrumentation Program has obtained significant records of earthquake motions of non-ductile concrete moment frames in Southern California. This research was performed to verify or develop methods for better understanding and prediction of the seismic performance of these buildings.

Three dimensional models of non-ductile reinforced concrete buildings were developed to test a variety of analytic techniques and materials assumptions against the recorded data. Linear elastic models which explicitly account for the stiffness contribution of diaphragms, in addition to the building frames, provided fairly accurate prediction for the low to moderate levels of earthquake motions.

Introduction

This study is part of the Data Interpretation Project of the California Strong Motion Instrumentation Program (CSMIP) in the Department of Conservation, Division of Mines and Geology. The purpose of this study is to use strong motion records from non-ductile concrete moment frame buildings to verify or develop methods for better understanding and prediction of the seismic performance of these buildings.

The three subject buildings for this study are:

Van Nuys Bldg.	Pasadena Bldg.	Sherman Oaks Bldg.
CSMIP Sta. #24386	CSMIP Sta. #24571	CSMIP Sta. #24322

All three buildings are fairly regular in plan and represent a good range of typical mid-height non-ductile concrete moment frame buildings, thus allowing some generalizations from this research for non-ductile concrete frame buildings as a class. See Table 1 and Figures 1,2, and 3 for building descriptions.

The strong-motion records available for this research represent low to moderate input earthquake motions. The input base accelerations ranged from a maximum of 0.25g to a low of 0.024g, and the majority of the records show peak input base accelerations of less than 0.10g (see Table 2). None of the subject buildings exhibited residual displacements from these earthquakes, or indicated significant nonlinear behavior.

Two types of dynamic analysis were tested to verify their ability to predict the building response: Three Dimensional Linear Analysis and Three Dimensional Nonlinear Analysis. The linear analysis was further subdivided into two types, according to modeling assumptions: Rigid Diaphragms and Flexible Diaphragms.

Analytical Techniques

The initial models used the simplest and most common assumptions used to analyze concrete frames. These models were developed using the computer program "ETABS"[1]. The ETABS models assumed rigid diaphragms so that the story stiffnesses were condensed into three degrees of freedom per floor: two translational and one rotational. Floor diaphragm rotational stiffness was

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neglected. In addition, section properties were based on I_{gross}, and damping was assumed to be 5% of critical for all earthquakes. Our next refinement of the models used the linear computer program "SAP90"[2]. Here the diaphragms were explicitly modeled as shell elements to include shear and bending deformations, and $I_{effective}$ and damping were varied. Lastly, a highly refined, three dimensional nonlinear analysis using the computer program "DRAIN-3DX"[3] was performed for the only structure with a building response above 0.3g (Pasadena Bldg.).

Linear Analysis with Rigid Diaphragms - "ETABS Analysis"

<u>Structural Models</u>
All necessary structural information was obtained from the structural drawings provided by CSMIP. The member sizes for beams and columns were obtained from the design drawings. The floor slab bending stiffness was ignored. Shear deformations in beams and columns were included, and the rigid joint offsets were modeled.

The masses of the buildings were computed including the floor slabs, columns, beams, and exterior skin. Penthouse and mechanical room masses were lumped to the roof. Young's modulus was calculated using the ACI section 8.5.1. For the time history analysis, a standard 5% damping ratio was assumed. Soil-structure interaction effects were ignored.

<u>Analysis Results</u>

A single earthquake record was selected for the linear time history analysis of each building. The calculated and recorded values for peak accelerations and time history displacements were compared. As shown in Tables 3,4 and 5, the peak accelerations results differed by as much as 60%. Comparisons of recorded and calculated displacement time histories (not included in this paper) show that phase and amplitudes were not in agreement, indicating that the model stiffnesses and dampings were incorrect.

Linear Analysis with Flexible Diaphragms - "SAP90 Analysis"

Because of the poor match between the recorded and calculated peak acceleration and displacement responses, further refinement was required to accurately predict the response of the buildings. The models were refined by including the flexible floor diaphragms and by calibrating the structural periods and damping.

<u>Structural Model Refinements</u>
Starting with the previously developed linear models, the flexible diaphragms were added by modeling the roof/slabs with shell elements. The gross sections of the slabs were used. The model stiffness was "calibrated" using the building responses from one arbitrarily chosen earthquake. To perform this calibration, we carefully examined the time histories, Fourier spectra, and response spectra provided by CSMIP to determine the fundamental periods of vibration of the buildings. The model response periods were then calibrated to match the recorded periods by adjusting the frame stiffnesses and roof/slab stiffnesses.

After calibrating the structural periods, the damping values were adjusted by comparing the calculated and recorded peak responses (acceleration and displacement), using the same earthquake that was used for the period calibration. Effective damping values were determined in the long and short directions of buildings. Only one effective damping per direction was assumed, and the selected damping values remained constant for all other earthquake records.

<u>Van Nuys - 7-story Hotel</u>
From the Big Bear records, the fundamental periods of 1.2 seconds (long) and 1.3 seconds (short) were estimated. Effective damping values of 9% and 15%

were determined in the long and short directions, respectively. Both damping values seem quite high for the low amplitude of vibration. Possible reasons for the high values include soil-structure interaction and participation of the nonstructural elements (partitions, etc.).

The slab bending stiffness significantly affected the transverse mode. By including 62% of the slab shell in the short direction, the transverse mode period decreased from 1.7 seconds to 1.3 seconds. Although there are no interior beams in the transverse direction, the thick slabs contribute partial frame action with the columns. In the longitudinal direction, slabs provided only a small increase in the longitudinal stiffness.

<u>Time History Analysis Results</u>
The building was subjected to three recorded base accelerations: The Big Bear Earthquake, The Landers Earthquake, and The Whittier Earthquake. (See Table 2.) Comparisons of calculated and recorded peak accelerations in the long direction of the building show an agreement within 10% (see Table 6). In the short direction, the maximum accelerations generally agree within 25%. The Whittier results cannot be directly compared, for only the base input in the short direction was used. (The base sensors failed in the long direction during the earthquake.)

Displacement time history plots provide a more complete basis for comparing the performance of the model than peak accelerations. Comparisons of displacement time histories also show very close agreement; the displacement response frequencies and amplitudes matched quite well between the recorded and calculated values for all earthquake record (see Figures 4 and 5).

<u>Pasadena - 9-story Commercial Building</u>
The Sierra Madre Earthquake was used to determine the fundamental periods of vibration of 1.2 seconds (long dir.) and 2.0 seconds (short dir.). Effective damping values of 3% and 4% were selected for the long and short directions.

The modal analysis results indicated that the bending stiffness of the slab does not affect the fundamental modes, since the building has complete frames in both directions. The stiffness calibration required a stiffness reduction of 25% in frames in the longitudinal direction.

<u>Time History Analysis Results</u>
The building was subjected to three recorded base accelerations: the Big Bear Earthquake, the Landers Earthquake, and the Sierra Madre Earthquake. Comparisons of calculated and recorded peak accelerations in the longitudinal direction are within 20% (see Table 6). In the short direction, the maximum accelerations generally agree within 20%.

Comparisons of displacement responses during the Sierra Madre Earthquake, which produced the highest recorded accelerations in the building, yield interesting observations: The calculated and recorded roof displacements in the long direction of the building match well for the first 6 seconds. Then, the recorded data shows an increase in the structural period between 6 and 30 seconds of the strong motion. In the last 10 seconds of the earthquake,the two responses match again. This observation suggests that the structure softened due inelastic behavior. (This was verified by the nonlinear analysis of the building.) However, the degree of nonlinearity was not severe enough to cause a significant deviation between the recorded and calculated displacements (see Figure 5).

Overall, the calculated and recorded displacement responses matched fairly well (see Figures 5 and 6).

<u>Sherman Oaks - 13-story Commercial Building</u>
The Whittier Earthquake records indicate the fundamental periods of vibration of 2.2 seconds (long dir.) and 2.4 seconds (short dir.). The modal analysis

results indicated that the bending stiffness of the slab did not affect the fundamental modes, since the building has complete frames in both directions, and the slab is relatively thin. The period calibration required a frame stiffness reduction of about 15% in both directions.

Effective damping values of 5% were determined in both the long and short directions. Although the selected damping value matched the displacements, the peak accelerations could not be matched as well. Time history acceleration and displacement responses show that the peak acceleration occurs at the beginning of the strong motion, and appears to be associated with high frequency motions. [4]

Time History Analysis Results

The building was subjected to two recorded base accelerations: the Whittier Earthquake and the Landers Earthquake. Comparisons of displacements during the Whittier Earthquake show good agreement in frequency and amplitude.

Results from the Landers Earthquake are quite interesting. Time histories, Fourier spectra, and response spectra of the recorded motions indicate that the building's fundamental periods are 2.8 seconds in both directions. This is significantly different from the periods 2.2 seconds (long) and 2.4 seconds (short) during the Whittier Earthquake. Since the model was "calibrated" to the Whittier Earthquake, calculated and recorded responses do not match well for the Landers Earthquake. Figure 7 shows that the calculated and recorded roof displacements do not agree in frequency and amplitude. The change in structural periods, perhaps, may be due to soil-structure interaction. The soil-structure interaction can change the fundamental structural periods. [5]

Non-Linear Analysis - "DRAIN-3DX Analysis"

As a final phase of the research, the importance of the material nonlinearity was explored. The Pasadena 9-story Commercial Building and the Sierra Madre Earthquake records were selected for detailed nonlinear analysis. The choices were based on the peak building acceleration responses. During the Sierra Madre Earthquake, the Pasadena building experienced the maximum base acceleration of 0.23g, and the maximum structural response acceleration of 0.43g. This was the highest recorded acceleration of all three selected buildings in this research.

The three dimensional nonlinear finite element model used fiber beam-column elements to model member nonlinearities. Each fiber beam-column element consisted of an elastic bar element in the middle, and nonlinear fiber hinges at each end. The assembly of fiber beam-column elements allowed nonlinear actions at every joint in the model.

The non-linear moment curvature relationship for each fiber hinge was developed using the program "BIAX" [6]. The non-ductile concrete stress strain relationship was developed using the Shiehk-Uzumeri relation [6]. The column moment curvature relationships were developed using the unfactored dead load as the axial load to capture the P-M interaction. The developed nonlinear moment curvature relationships were, then, idealized as bilinear stress-strain relations for the fiber hinges. In addition to the material nonlinearity, the geometric nonlinearity due to the P-delta effect was included in the time history analysis. The nonlinear time history analyses were performed using the Rayleigh damping equivalent to the 5% viscous damping.

Two models were tested for the nonlinear dynamic analysis, with the elastic portion of the fiber beam-column elements varied in each model. The moment of inertia for the elastic portion of the beams and columns was varied from $1.0xI_{gross}$ to $0.5xI_{gross}$ in two separate models. The use of $I=0.5xI_{gross}$ provided a fairly accurate match of displacements to the recorded responses in the longitudinal direction. (See Figure 8.)

Time history analysis results in the long direction, the predominant earthquake input direction, show that only a small number of beam/column joints experienced minor incursions into the inelastic region. From this observation we concluded that the material nonlinearity was not a significant factor for predicting the responses of the buildings in this study.

Nonlinear (DRAIN-3DX) and SAP90 Compared
Figure 8 shows the roof displacement time histories in the long direction of the Pasadena Building during the Sierra Madre Earthquake. The DRAIN-3DX calculated displacements match the recorded displacements fairly well. SAP90 calculated displacements, on the other hand, match the beginning portion of the time history and "misses" the stiffness change in the structure. Thus the linear model, which was "calibrated" to the initial elastic stiffness of the structure, remained elastic while DRAIN-3DX model updated the minor stiffness degradation experienced by the structure.

With the exception of the frequency of motion, the linear analysis results produced satisfactory results. Since the nonlinearities experienced by the structure were minor due to the relative low base input accelerations, the deviations between the two analyses results were not significant. However, if the earthquake motions were stronger, we would expect that only nonlinear analysis will accurately predict the behavior of the structure. (Only the nonlinear analysis is capable of updating the stiffness changes taking place, and of predicting the structural capacity limit during strong earthquake motions, i.e., the Northridge Earthquake of January 17, 1994.)

Conclusions

Important findings of this research are:

- Linear analysis can be "calibrated" to accurately predict motions of nonductile reinforced concrete moment frame structure during small to moderate earthquake strong motions.
- Two parameters available for "calibrating" the linear model are the frame stiffness and damping ratio.
- In-situ concrete frame member stiffnesses varies between 75% to 85% of the I_{gross}, and the viscous damping varied between 3% to 15% of critical.
- Floor slab bending stiffness should be included in the model.

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Table No. 1: Description of Building

Name SMIP Station	Date of Construct.	Plan Dimensions	Stories Above Ground/ Below Ground	Foundation System
Pasadena Bldg. Sta. #24571	1964	213'x86'	9/1	Spread Ftgs.
Sherman Oaks Bldg. Sta. #24322	1965	193'x75'	13/2	Concrete Piles
Van Nuys Bldg. Sta. #24386	1966	151'x63'	7/0	Concrete Pile

Table 2: Earthquake Records used in Research

Building	Record Name	Richter Magnitude	Peak Base Accel.	Peak Building Accel.
Pasadena Building Sta. #24571	Landers EQ of 28 June, 1992	7.4	.047g	.23g
	Big Bear EQ of 28 June, 1992	6.4	.039g	.09g
	Sierra Madre EQ of 28 June, 1991	5.8	.23g	.43g
Sherman Oaks Bldg. Sta. #24322	Landers EQ of 28 June, 1992	7.4	.045g	.11g
	Whittier EQ of 1 October, 1987	5.9	.25g	.21g
Van Nuys Bldg. Sta. #24386	Landers EQ of 28 June, 1992	7.4	.042g	.19g
	Big Bear EQ of 28 June, 1992	6.4	.024g	.06g
	Whittier EQ[1] of 1 October, 1987	5.9	.16g	.20g

[1] This input earthquake record was incomplete in one direction.

Table 3: Pasadena Bldg. ETABS Peak Accelerations for Sierra Madre E.Q.

FLOOR	RESPONSE (PREDICTED/RECORDED absolute g)			% DIFFERENCE
2nd	Channel	6	0.300/0.344	-13
		8	0.112/0.092	+22
5th	Channel	9	0.234/0.291	-20
		10	0.078/0.067	+17
		11	0.086/0.091	-10
Roof	Channel	12	0.361/0.425	-15
		13	0.086/0.091	-6
		14	0.113/0.100	+12

Table 4: Sherman Oaks Bldg. ETABS Peak Accelerations for Whittier E.Q.

FLOOR	RESPONSE (PREDICTED/RECORDED absolute g)			% DIFFERENCE
2nd	Channel	1	0.1268/0.14	-9
		2	0.1034/0.14	-26
		3	0.1128/0.197	-43
8th	Channel	4	0.0758/0.107	-29
		5	0.062/0.114	-46
Roof	Channel	7	0.0718/0.109	-34
		8	0.0789/0.171	-54

Table 5: Van Nuys Bldg. ETABS Peak Accelerations for Big Bear E.Q.

FLOOR	RESPONSE (PREDICTED/RECORDED absolute g)			% DIFFERENCE
2nd	Channel	7	.0227/0.03	-24
		12	0.037/.03	+23
3rd	Channel	5	0.0244/0.046	-39
		11	0.0492/0.03	+64
6th	Channel	4	0.039/0.04	-3
		10	0.0586/0.05	+1
Roof	Channel	3	0.0614/0.04	+54
		9	0.084/0.06	+40

Table 6: SAP90 Peak Accelerations

Building	Earthquake Record Name	Floor Level	Recorded/ Predicted Accel. Long Dir.	Recorded/ Predicted Accel. Short Dir.
Pasadena Building Sta. #24571 Model Damping: Long=3% Short=4%	Landers EQ of 28 June, 1992	Roof	0.23/0.21	0.07/0.08
		5th	0.19/0.17	0.05/0.07
		2nd	0.13/0.11	0.04/0.04
	Big Bear EQ of 28 June, 1992	Roof	0.09/0.10	0.04/0.04
		5th	0.06/0.06	0.03/0.03
		2nd	0.04/0.05	0.03/0.03
	Sierra Madre EQ of 28 June, 1991	Roof	0.41/0.43	0.09/0.11
		5th	0.29/0.26	0.07/0.07
		2nd	0.34/0.32	0.09/0.10
Sherman Oaks Bldg. Sta. #24322 Model Damping: Long=5 % Short= 5%	Landers EQ of 28 June, 1992	Roof	0.09/0.06	0.09/0.06
		8th	0.06/0.05	0.06/0.04
		2nd	0.04/0.03	0.04/0.03
	Whittier EQ of 1 October, 1987	Roof	0.14/0.11	0.14/0.09
		8th	0.11/0.06	0.11/0.06
		2nd	0.10/0.07	0.17/0.09
Van Nuys Bldg. Sta. #24386 Model Damping: Long=9% Short=15%	Landers EQ of 28 June, 1992	Roof	0.13/0.12	0.11/0.08
		6th	0.09/0.10	0.09/0.06
		2nd	0.05/0.05	0.05/0.04
	Big Bear EQ of 28 June, 1992	Roof	0.06/0.05	0.04/0.05
		6th	0.05/0.05	0.04/0.04
		2nd	0.03/0.03	0.03/0.03
	Whittier EQ[1] of 1 October, 1987	Roof	N.A.	0.15/0.10
		6th	N.A.	0.08/0.06
		2nd	N.A.	0.14/0.10

[1] This input earthquake record was incomplete in one direction.

Van Nuys - 7-story Hotel
(CSMIP Station No. 24386)

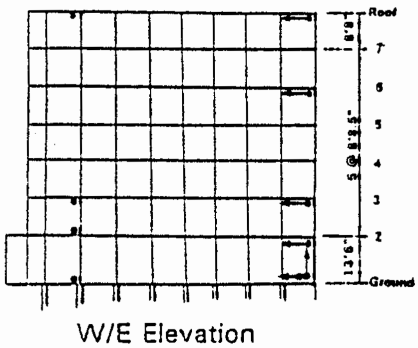

Figure 1: Van Nuys - 7 Story Hotel

Pasadena - 9-story Commercial Bldg.
(CSMIP Station No. 24571)

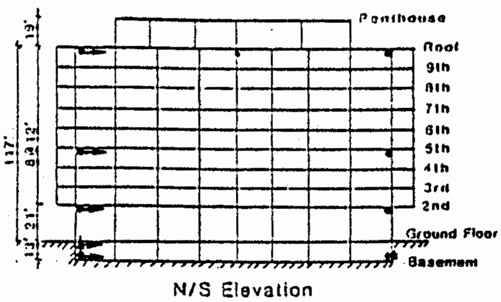

Figure 2: Pasadena - 9-story Commercial Bldg.

Sherman Oaks - 13-story Commercial Bldg.
(CSMIP Station No. 24322)

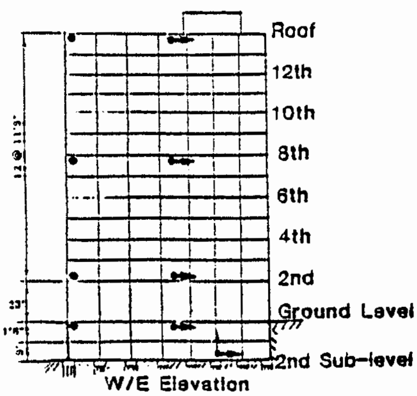

Figure 3: Sherman Oaks - 13-story Commercial Bldg.

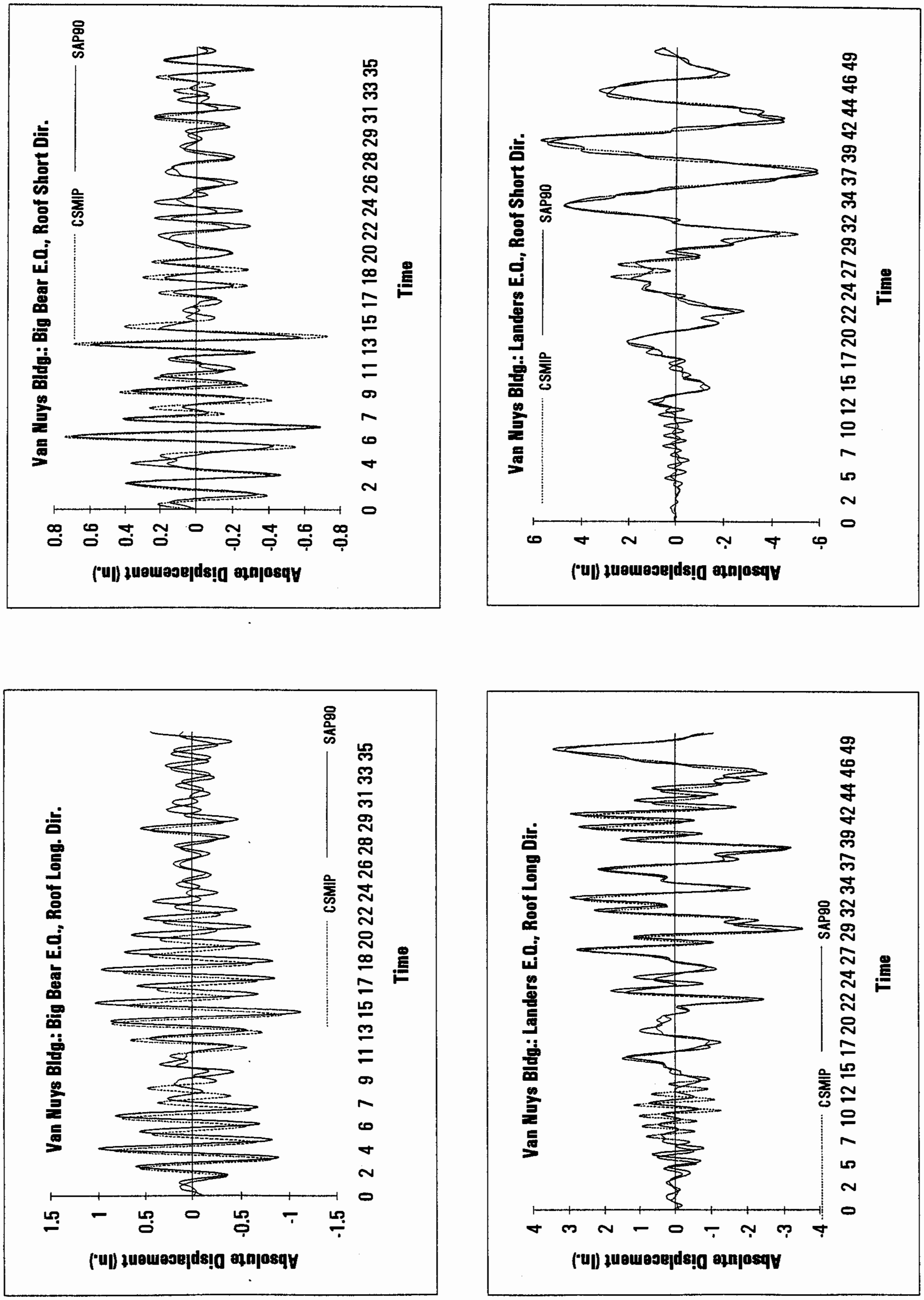

Figure 4: SAP90 Time History Displacement Plots

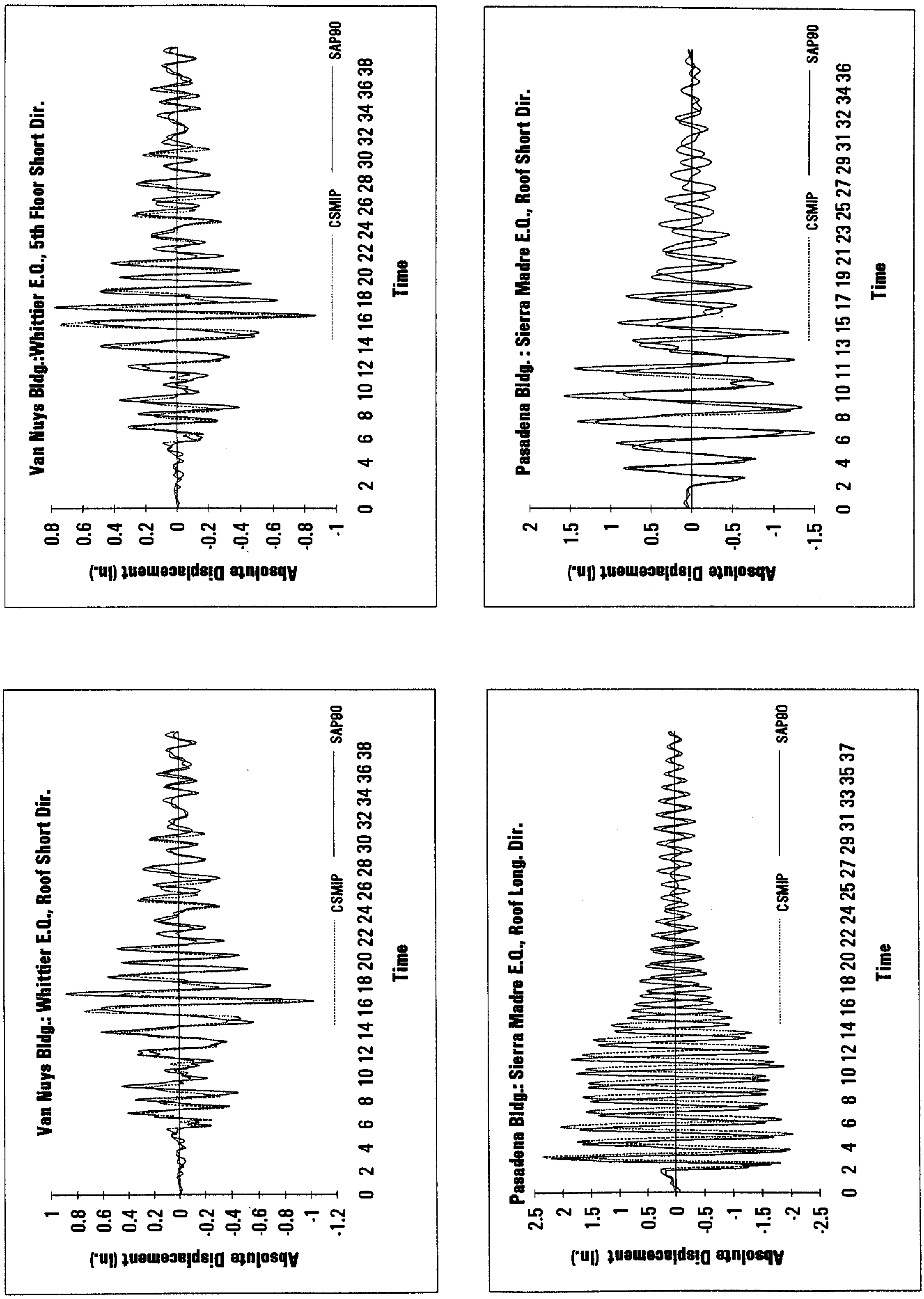

Figure 5: SAP90 Time History Displacement Plots

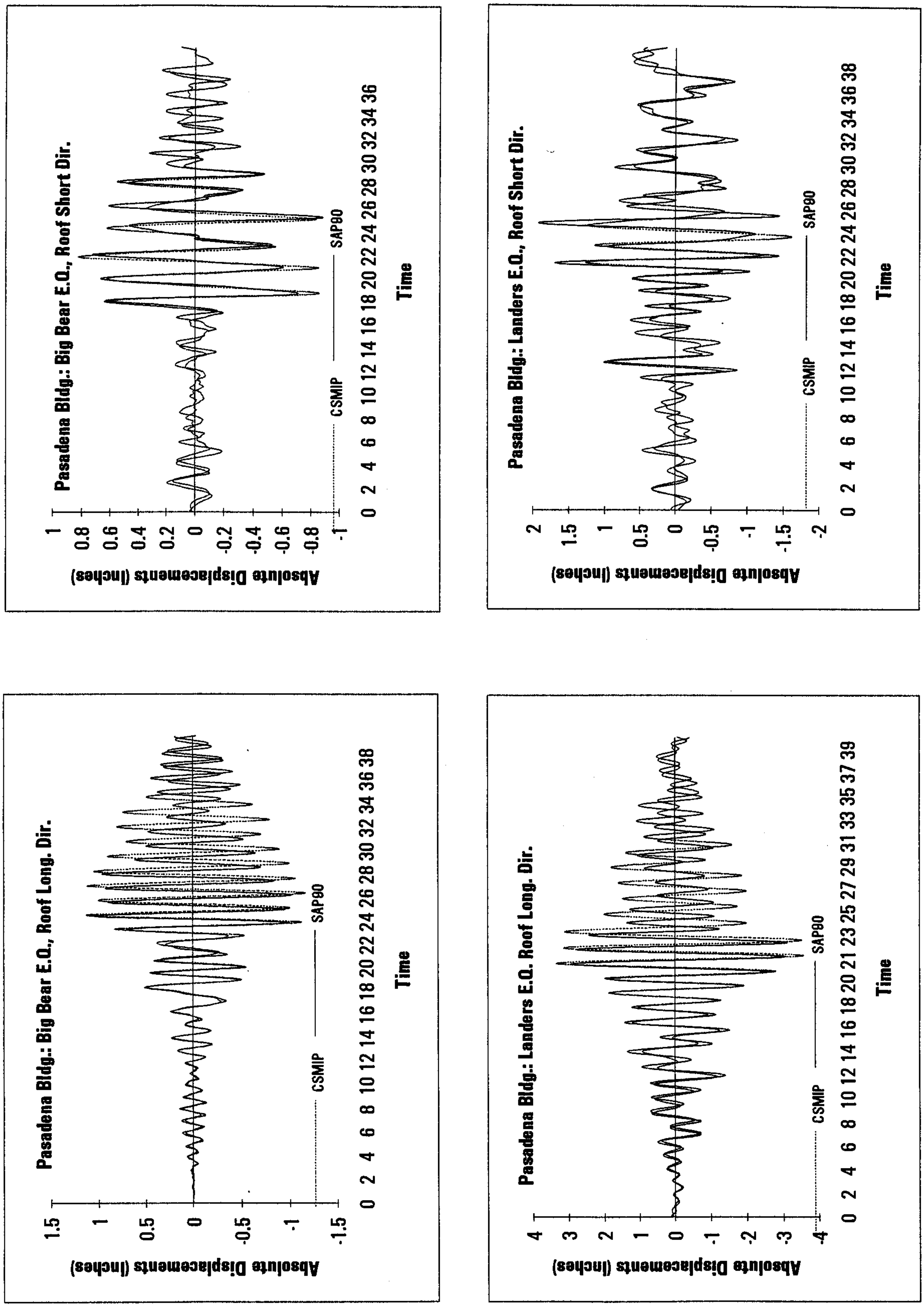

Figure 6: SAP90 Time History Displacement Plots

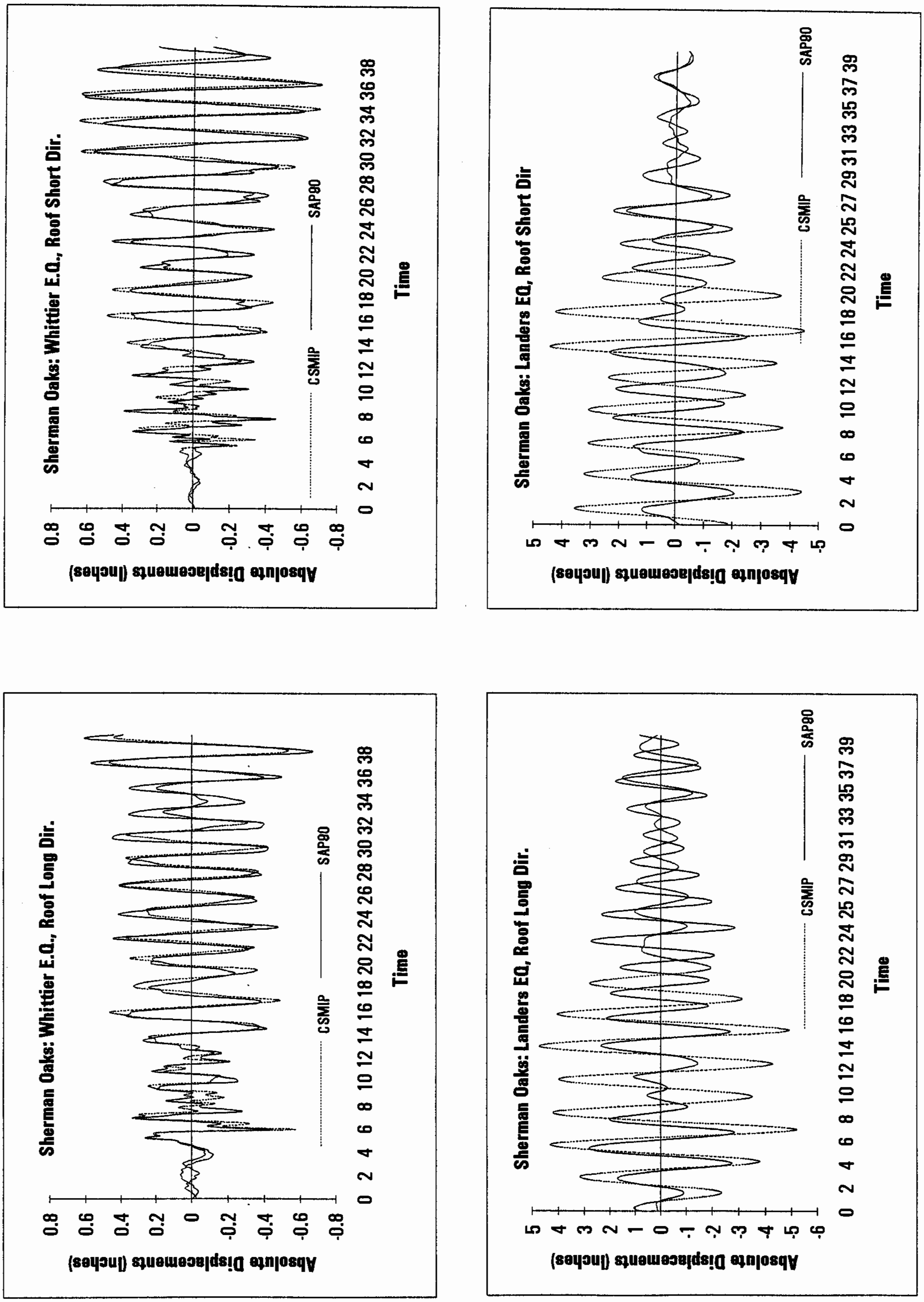

Figure 7: SAP90 Time History Displacement Plots

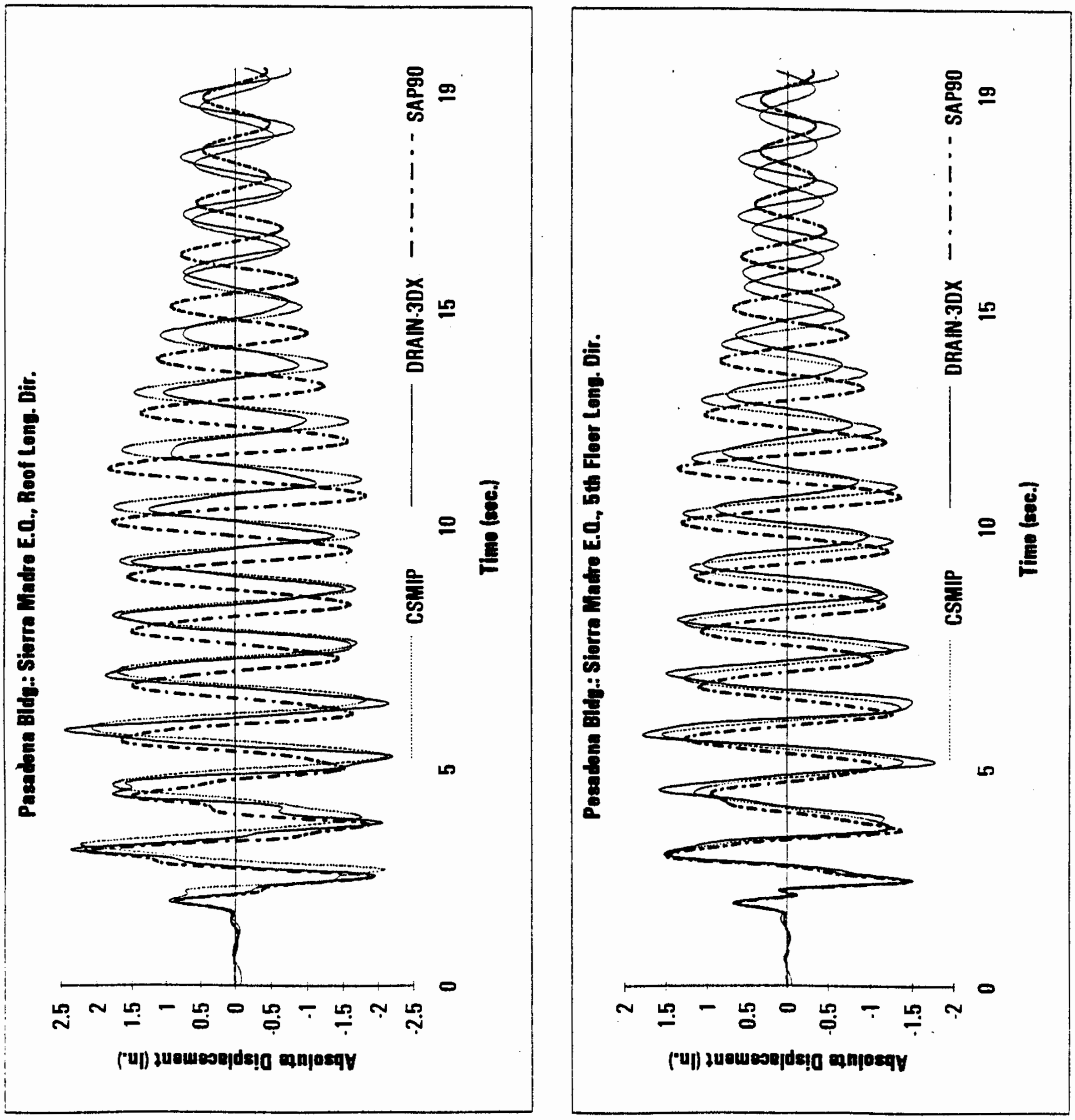

Figure 8: DRAIN-3DX & SAP90 Time History Displacement Plots