APPLICATION OF RECORDED MOTION TO POST-NORTHRIDGE EVALUATION OF STEEL FRAME BUILDINGS

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ABSTRACT

Contradictory recommendations have been made in the Interim Guidelines FEMA 267/ Aug 1995, and the SAC Technical Report. 95-04, as to the value of analysis in post-earthquake evaluation of WSMF buildings.

Analytical studies of the response of welded steel moment frames (WSMF) using data recorded at the base and up the height of the building have been shown to be very useful in defining the zone of the building where earthquake damage is most likely. When damage is found in the zone where analysis would have predicted, this zone must be extensively sampled. Effort expended in a random survey for earthquake-caused damage is not earthquake response related and is not cost-effective.

A random testing procedure is useful to determine the effectiveness of the original quality control procedure. An analysis based damage survey is more productive in finding if the site shaking caused earthquake damage.

INTRODUCTION

The 1994 Northridge earthquake caused unexpected damage to the joints of welded steel moment frame (WSMF) buildings. The damage was concealed in many buildings by fire protection and ceilings. An external survey of the WSMF buildings did not find interstory drifts that are generally associated with structural damage. Investigations directed by engineers uncovered a level of damage that caused industry, academia and professional societies to convene committees that would advise the engineering profession, the building owners, the building officials and the construction industry on how to conduct an investigation, interpret the results and plan a repair program.

The SAC Joint Venture Partnership convened groups to prepare advisory documents. The group charged with consideration of the possible problems of existing buildings discussed how should WSMF buildings be evaluated, what constitutes a minimum inspection program, and how can earthquake damage be distinguished from quality control deficiencies.

General agreement was that a simplified analysis to identify locations of high stress was useful. The concept was that the significant earthquake damage would logically be in these "high stress" areas. The SAC Joint Venture funded studies of WSMF buildings to find if structural analysis could be utilized to reduce the numbers of joints investigated and/or provide data that could limit the investigation to locations in the building shown to be critical by analysis.

INTERIM GUIDELINES

The Interim Guidelines, FEMA 267/August 1995, were developed from the SAC Advisories. The Interim Guidelines in Chapter 4, Post-Earthquake Evaluation states that "damaged connections tend to be widely distributed throughout the building frames, often at locations analysis would not predict. This approximates a random distribution". This statement contradicts the general observations in the SAC Technical Report 95-04, December 1995. This states; "All the analytical procedures were able, in at least a limited fashion, to provide an indication of locations within buildings where connection damage was most probable. That is, analytical indicators could be identified in all cases that would provide a better indicator of damage location than random sampling". Not all of the buildings analyzed in these SAC studies had an instrumental record of the shaking at the base of the building. The key to the success of analysis in prediction of the zone of probable earthquake damage is the availability of the base motion record.

The SAC studies also concluded that if incipient cracks were included in the report as earthquake damage the "damage zone" became more random in nature. The reverse of the opinion is: the existence of partial cracks in welds is not earthquake damage and this is supported by the prediction of the zone of column damage by analysis. Column damage is less likely to be related to flaws in the welding quality control program.

The repair of welded steel moment frames has commonly included repair of fractures in columns. Analysis have been very effective in predicting these zones of earthquake damage in columns when recorded data is used for loading of the WSMF structures.

MODELING OF THE WSMF

The quality of the results of an analysis is highly dependent on the quality of the modeling. The analysts in the SAC research were required to model the building as commonly designed. That is, only the WSMF is used to resist the earthquake loading. However, in reality the beams and girders are composite with the concrete floor slabs. Research by Roberto Leon and others has shown that even welding of steel decking to beams and girders mobilizes the concrete slab placed on the decking as a part of the composite beam. Figure 1 shows the difference in the response to the recorded motion of a so-called "bare frame" from the response of the "probable" model. The difference in the curvature demand on the columns is not explicitly shown in this Figure. The increase in stiffness and strength in the girders causes the interstory displacement to be a "shear type" strain distribution. The curvature demand at each joint is concentrated in the columns.

The analysts were required to have a common method of computing a "demand/capacity" ratio. This was use of a "bare frame" model and response spectrum analysis. The response spectrum used in the analysis was a five percent damped spectrum prepared from the recorded motions . The demand/capacity ratio for the columns of the N-S frames were less than or equal to 1.0 except at the fourteenth floor level. The maximum at this level was 1.23 for an interior column and 1.10 for the corner column. The demand/capacity ratio for the girder at the fifteenth floor level was 1.19 and 1.08 at the interior and exterior bays respectively. If the expected stiffness of the girders has been used in the analysis, the curvature demand in the columns would be increased.

INFLUENCE OF THE ANALYSIS METHOD

Figure 2 shows the difference between the results of a response spectrum analysis and an elastic time-history analysis. The data shown as story drift is the maximum value for the duration of the time-history analysis. These interstory drifts do not necessarily occur at the same time. If the analysis had been made using the "probable" model, even the lesser interstory drifts would significantly increase the curvature demand in the columns. A review of Figure 1 shows that the curvature demand on the column increases from the eighth floor level to a maximum at the fourteenth floor level.

A two-dimensional nonlinear analysis of the N-S frames resulted in the formation of the yield sequences shown in Figure 3. These calculations used two and one-half percent strain hardening up to 9.06 seconds of the base motion record. At the time the analysis stopped due to a negative tangent stiffness. This negative stiffness was caused by the effects of axial loads in the columns causing secondary moments ($P\Delta$ effects). The strain hardening effects were assumed to be ten percent and the nonlinear program became stable. Three snapshots of the displacement of the building relative to its base are shown in Figure 4.

The damage found by an investigation of all beam-column joints is shown in Figure 5 and 6. The masonry shear wall shown in Figure 6 is supposedly isolated from the frame by slotted holes in the connection angles. It is highly likely that this shear wall modified the probable response of Line 20. There was evidence that Line 5 impacted on the reinforced concrete shear wall parking structure. The effects of this impact on the observed damage cannot be estimated.

These studies clearly show the advantage of analysis using a recorded base motion for prediction of zones of probable earthquake damage. The anomaly at floor levels 7 and 9 on Line 5 may be related to the collision with the parking structure. The column damage at these levels is related to axial tensile stresses in the columns. This axial tensile stress is due to minimal dead load on the corner columns and to overturning moment effects.

INFLUENCE OF THE CHARACTER OF THE GROUND MOTION

The recorded total energy at any site by the Northridge earthquake when calculated by methods, such Housner Intensity, was significantly less than that associated with a design level spectrum. This should not be considered unusual as the Northridge earthquake is only one of the family of earthquakes that populate a design level spectrum.

Figure 7 compares the five percent damped spectra for four scenarios. These are:

- 1994 UBC, S_2 type soils
- UCBC - Division 95, City of Los Angeles
- Recorded N-S component at the site.
- Uniform risk-10% probability of exceedence in 50 year time period-for the Olive View site.

The effects of each of these loadings on the N-S frames of the building is shown in the plot of story displacements. These displacements were calculated by response spectrum analysis using a “bare frame” model. This analysis method is shown by Figures 1 and 2 to be relatively a poor procedure for prediction of possible earthquake damage, but shows the influence of the energy contained in each spectrum.

The Uniform Risk 10-50 spectrum developed for the Olive View site has a predicted ground acceleration of 0.7 g. The effects of the N-S component recorded at the Olive View site on this steel frame building is shown on Figure 3. The nonlinear analysis becomes mathematically unstable at 9.22 seconds. The instability is due to secondary effects caused by axial loading of the columns.

The energy content of a earthquake record was previously related to “Housner Intensity”. This energy measure is the integral of the spectral velocity between defined frequencies such as from 0.1 seconds to 5.0 seconds. The principal difference between a spectrum for a 6 3/4 Richter magnitude event (and the Division 95, soil type 2 spectrum) and the UBC design level spectrum is that these use average spectral amplification values in lieu of mean plus one sigma spectral amplification values. Figure 7 clearly shows the difference in spectral acceleration values for periods longer than 2.5 seconds, and that there is a substantial difference of mean and mean plus one sigma spectral accelerations for longer periods.

The character of the Northridge earthquake was such that the excitation of primary mode of tall WSMF’s was subdued. However, the energy available for excitation of second, third and fourth modes was nearly equal to a design level earthquake. This information alone indicates that the surveys for detection of earthquake damage should not be randomly distributed over the height of the building.

A very similar building about two blocks east on Ventura Boulevard had all instruments required by the Los Angeles City Building Code function. The copy of the recording tape, Figure 8, can be considered as a record of a forced vibration to about 11 seconds and a free vibration for the remainder of the record. The record shown as longitudinal is in the N-S direction. The fundamental period is estimated as about 2.5+ seconds in the N-S direction and about 2.0+ seconds in the E-W direction. The tenth story record generally shows that response effects in the fundamental mode in the longitudinal direction were not significant. The transverse (E-W) effects of fundamental response mode are more obvious. The record at the arcade (basement) level does not have any visible evidence of soils period effects.

The records of earthquake loading and structural response of a 24 story WSMF building on Wilshire Boulevard near the 405 Freeway have been digitized by CSMIP and spectra has been prepared. The roof record in the narrow direction of the building clearly indicates the free vibration state, Figure 9. The absolute acceleration vs. period plot has peaks at 1.4 and 4.0 seconds. The record perpendicular to Wilshire has similar response but with significantly less peak accelerations. The records at the 12th floor are nearly identical except for reduced accelerations.

These records can be interpreted as response in the fundamental and second mode. The second mode is quickly damped and does not appear in the free vibration response. The survey should be uniformly distributed over the height of the moment frames that are parallel to Wilshire. An investigation of the moment frames perpendicular to Wilshire will be a verification of the quality control program.

CONCLUSION

If the purpose of the evaluation of welded steel moment frame buildings is to determine if the intensity of ground motion at that site caused damage that warrants repair, and this is the principal purpose of this evaluation, and recorded motions are available, the investigation should be concentrated in zones predicted to have the highest probability of damage by analysis of the building and the records. If damage is found in the zone with the highest probability of damage then the investigation is carried outward from this zone. Determination of the effectiveness of the original quality control program should not be a priority unless so determined by the building owner.

The records obtained by the UBC mandated program are of value to estimate the fundamental modes of the building. Knowing this and the character (Richter or moment magnitude) of the earthquake it can be determined whether fundamental or higher mode response was critical for possible damage. If the records have been digitized and integrated the interpretation of the records is simplified. It is probable that analysis of the records alone is adequate to plan an evaluation program to detect earthquake damage.

If dynamic analysis of the building is believed to be cost-effective, the building should be modeled with its expected stiffness. Our studies have found the elastic or nonlinear time-history studies are most reliable for prediction of possible damage zones. Nonlinear analysis requires significantly more effort and did not contradict the results of the elastic time-history studies. It is likely that elastic time-history studies can adequately locate the zones when the evaluation of the WSMF should begin.

STORY DISPLACEMENTS & INTERSTORY DRIFT
BARE MODEL VS. PROBABLE MODEL
TIME-HISTORY ANALYSIS USING NORTH-SOUTH COMPONENT OF BASE RECORD

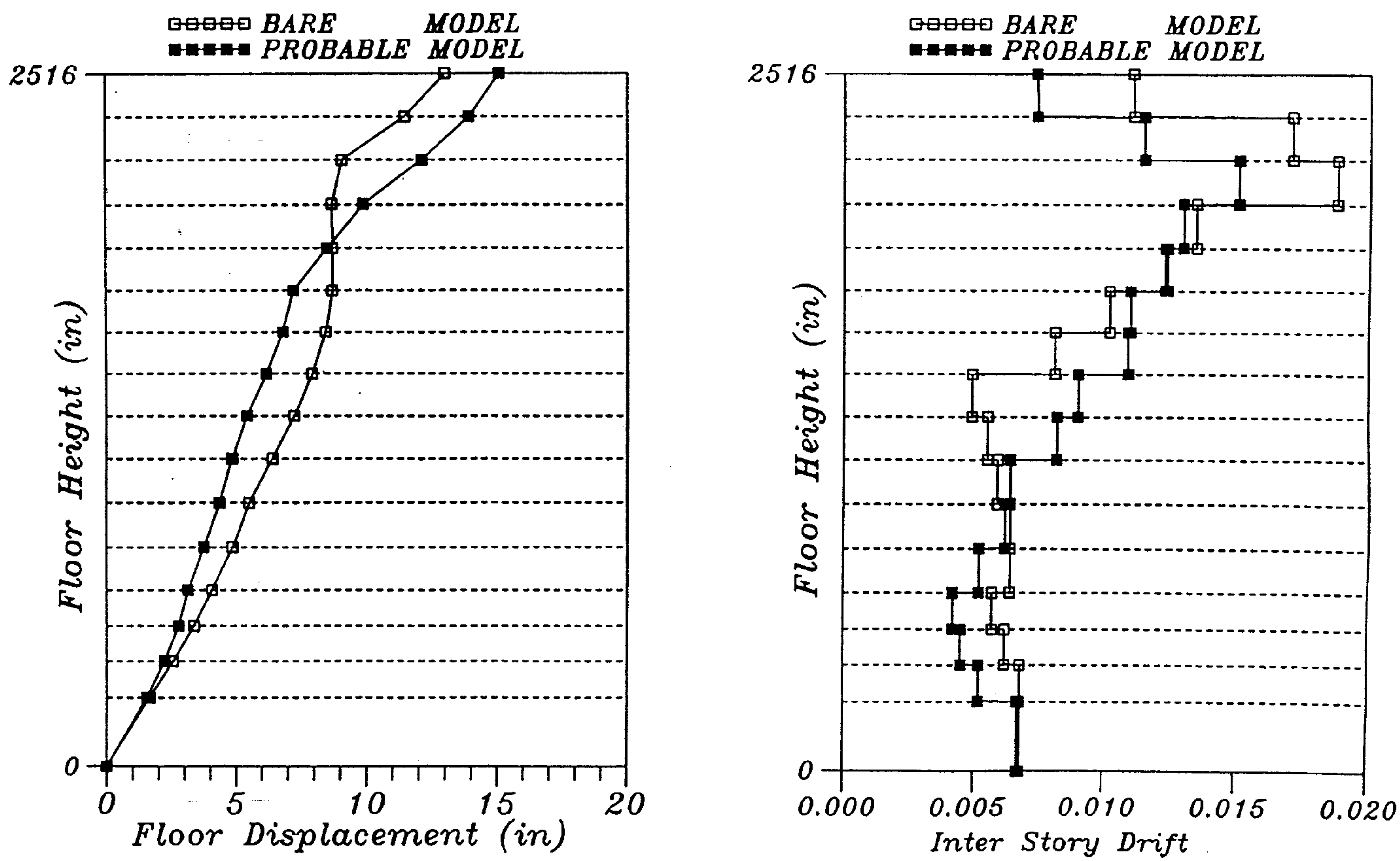


FIGURE 1

STORY DISPLACEMENT & INTERSTORY DRIFT
RESPONSE SPECTRUM VS. TIME-HISTORY ANALYSIS
NORTH-SOUTH COMPONENT OF BASE RECORD - BARE FRAME

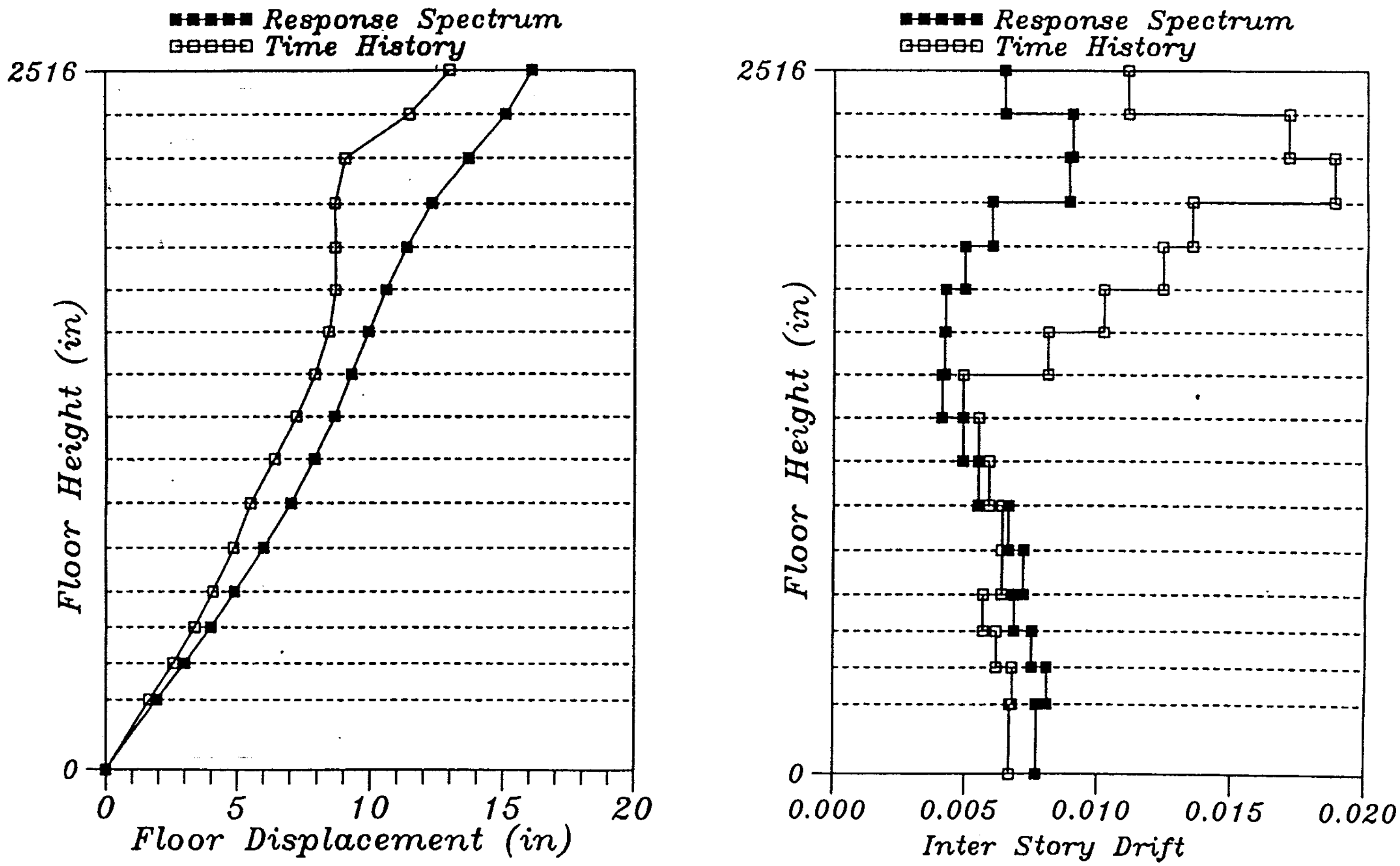


FIGURE 2

SEQUENCE OF FORMATION OF PLASTIC HINGES IN NORTH-SOUTH FRAMES ON LINES 5 & 20
(a)RECORDED BASE MOTION - 2.5% STRAIN HARDENING TO 9.06 SEC.
(b)SYLMAR RECORD - 10% STRAIN HARDENING

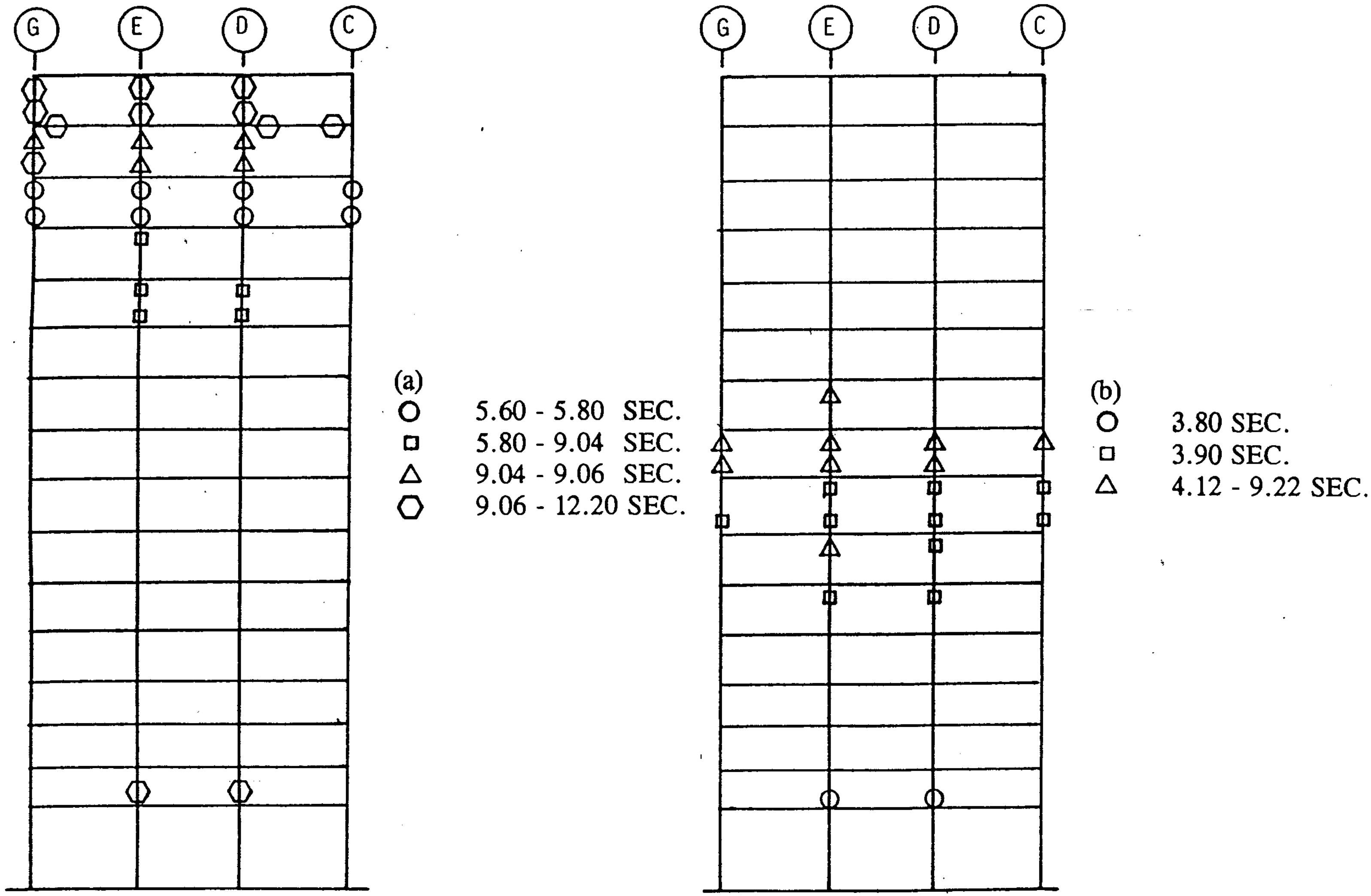
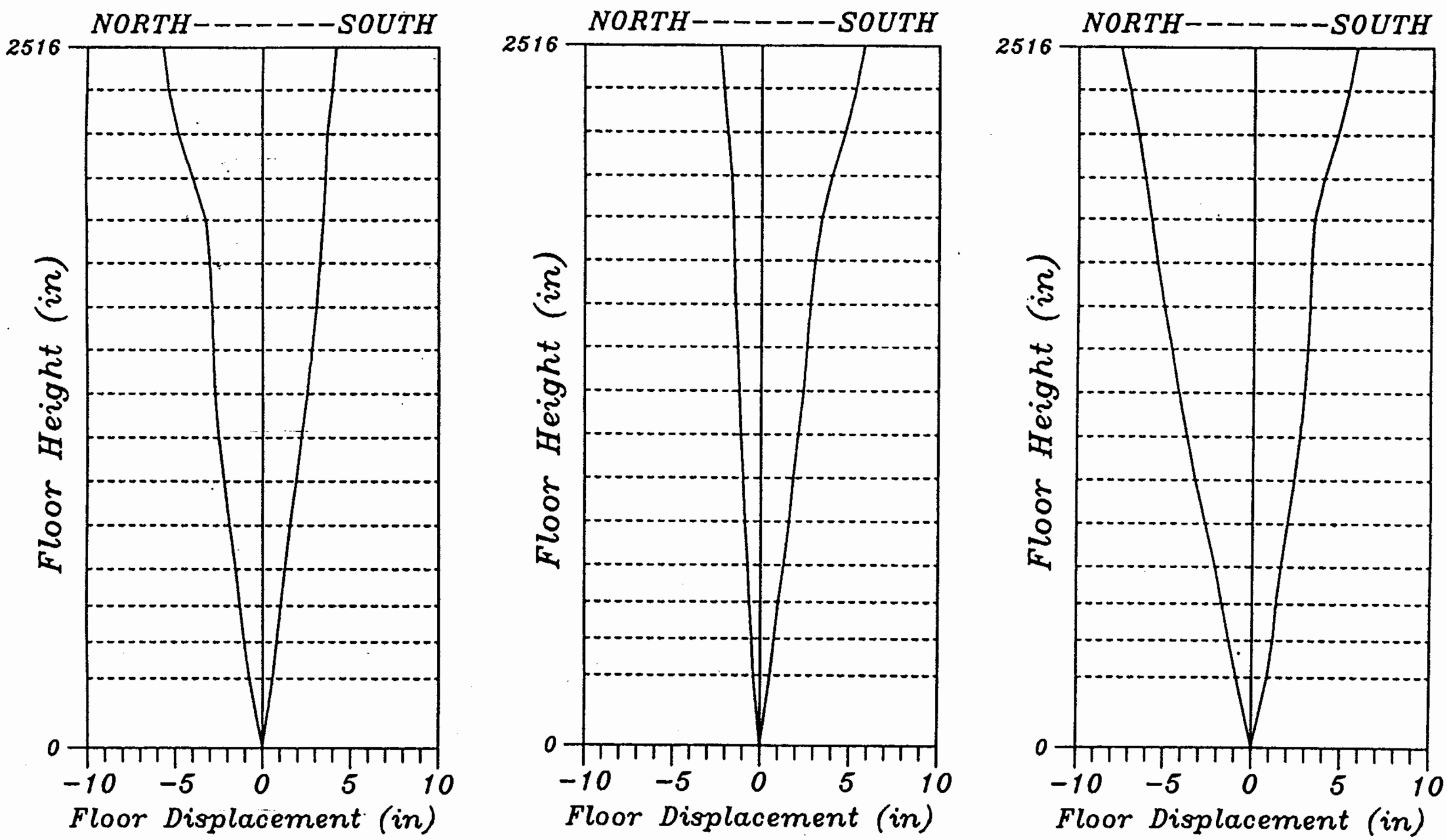


FIGURE 3

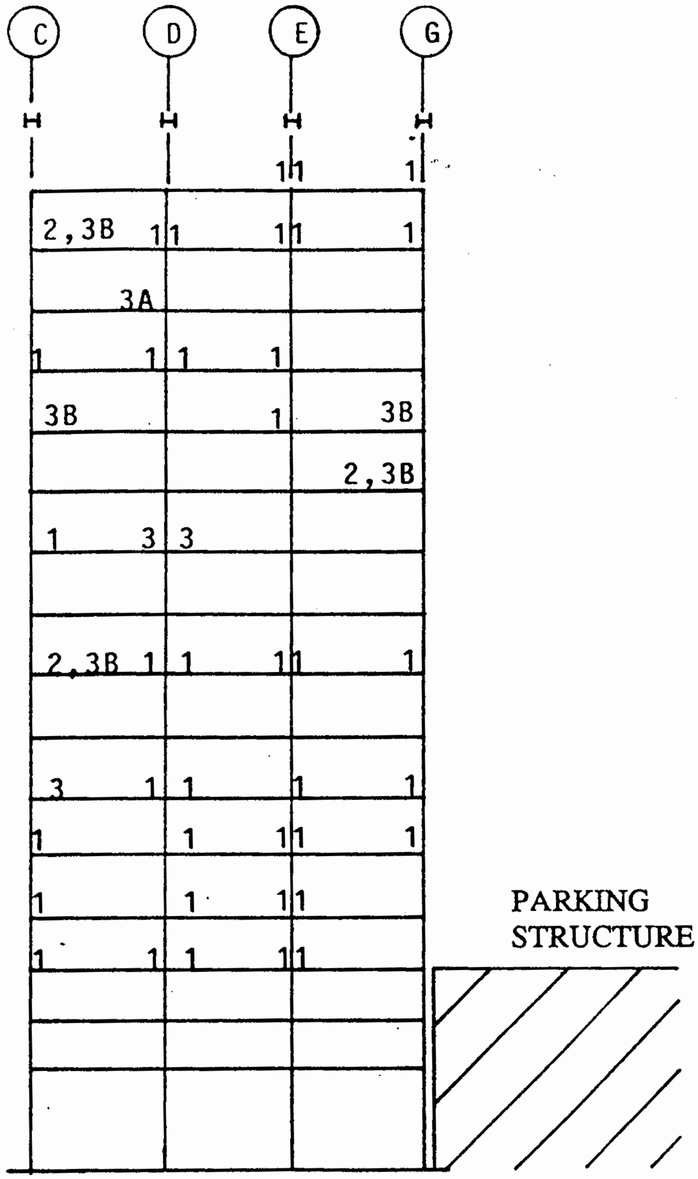
RESULTS OF THE NONLINEAR ANALYSES IN THE NORTH-SOUTH DIRECTION
(a) MAXIMUM STORY DISPLACEMENTS AT 9.0 SEC. OF THE BASE RECORD - 2.5% STRAIN HARDENING
(b) MAXIMUM STORY DISPLACEMENTS AT 9.0 SEC. OF THE BASE RECORD - 10% STRAIN HARDENING
(c) MAXIMUM STORY DISPLACEMENTS AT 15.0 SEC. OF THE BASE RECORD - 10% STRAIN HARDENING



(a) (b) (c)

FIGURE 4

EAST ELEVATION - LINE 5



LEGEND

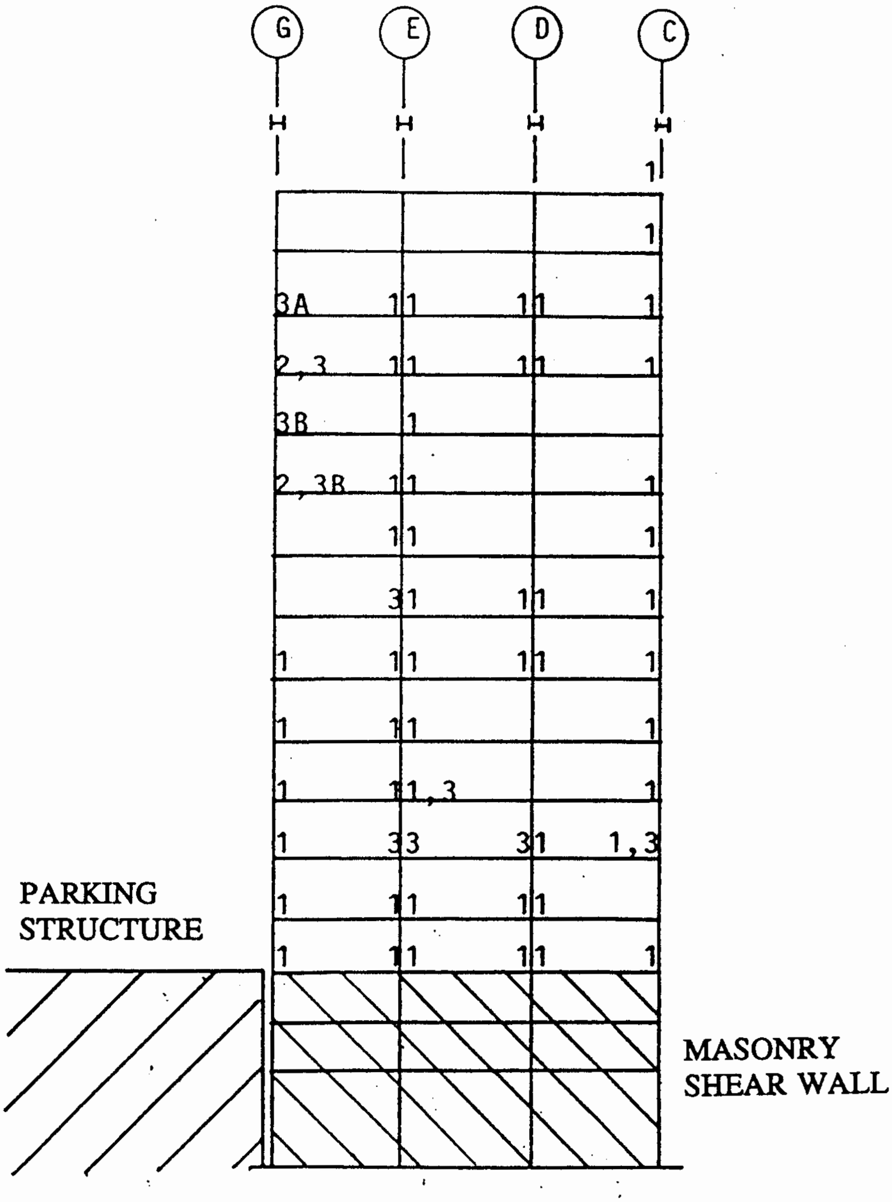
1. PARTIAL CRACK IN WELD OF BOTTOM FLANGE OF BEAM
2. COMPLETE CRACK THROUGH WELD OF BOTTOM FLANGE OF BEAM
3. PARTIAL CRACK INTO COLUMN FLANGE

3A. COMPLETE CRACK THROUGH COLUMN FLANGE

3B. COMPLETE CRACK THROUGH COLUMN FLANGE PLUS CRACK INTO COLUMN WEB

FIGURE 5

WEST ELEVATION - LINE 20



LEGEND
1. PARTIAL CRACK IN WELD OF BOTTOM FLANGE OF BEAM
2. COMPLETE CRACK THROUGH WELD OF BOTTOM FLANGE OF BEAM
3. PARTIAL CRACK INTO COLUMN FLANGE
3A. COMPLETE CRACK THROUGH COLUMN FLANGE
3B. COMPLETE CRACK THROUGH COLUMN FLANGE PLUS CRACK INTO COLUMN WEB

FIGURE 6

COMPARISON OF RESPONSE OF BARE FRAME TO FOUR SPECTRA
NORTH-SOUTH DIRECTION

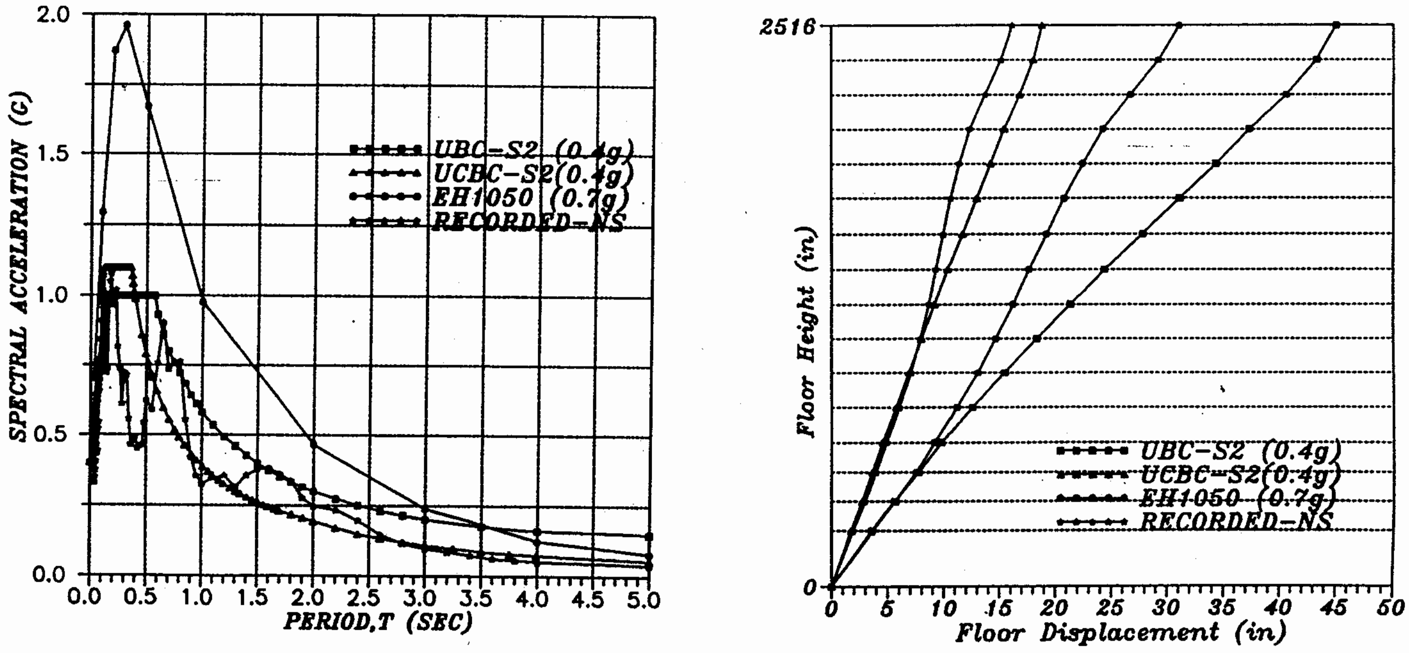
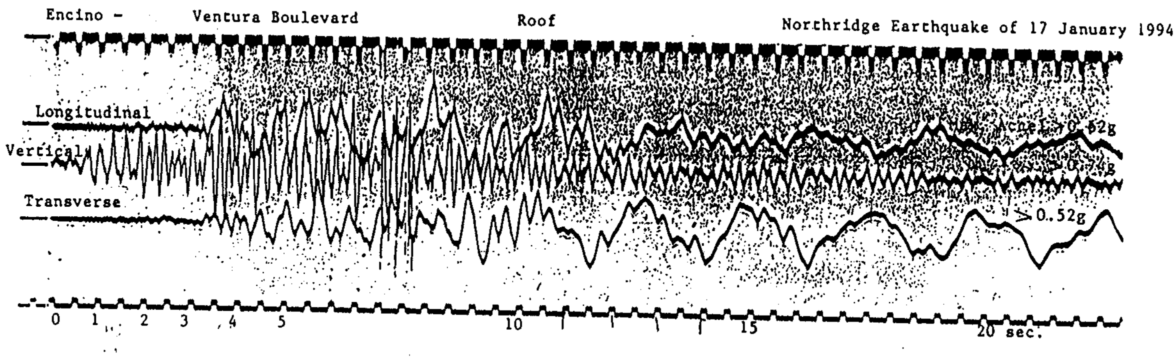
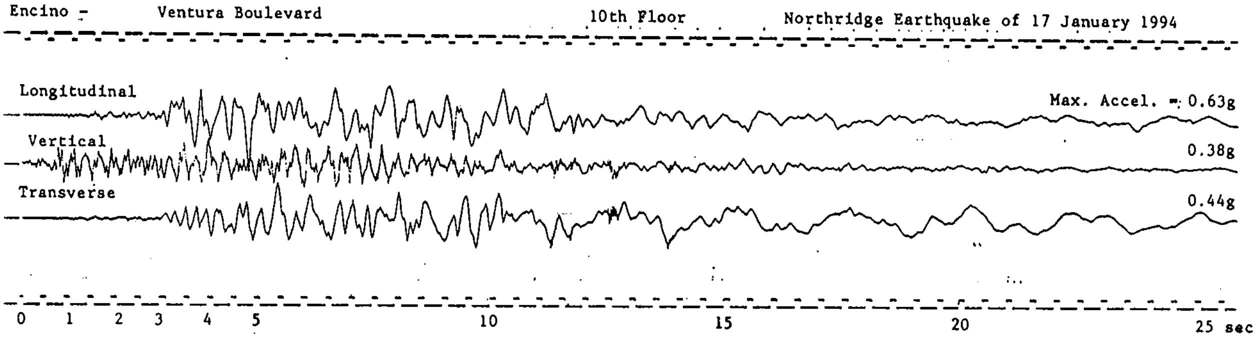


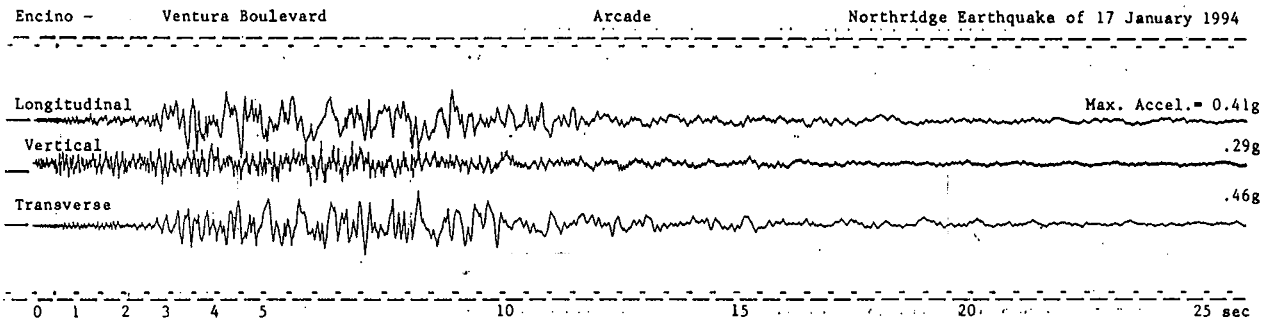
FIGURE 7



Note: Longitudinal direction is perpendicular to Ventura Boulevard.

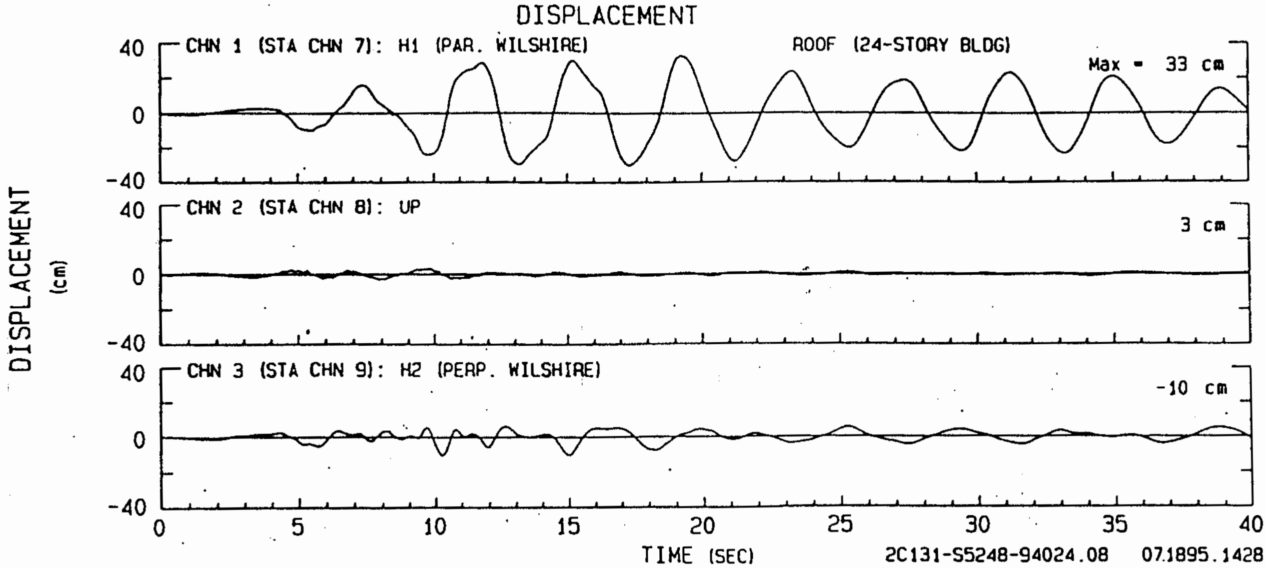


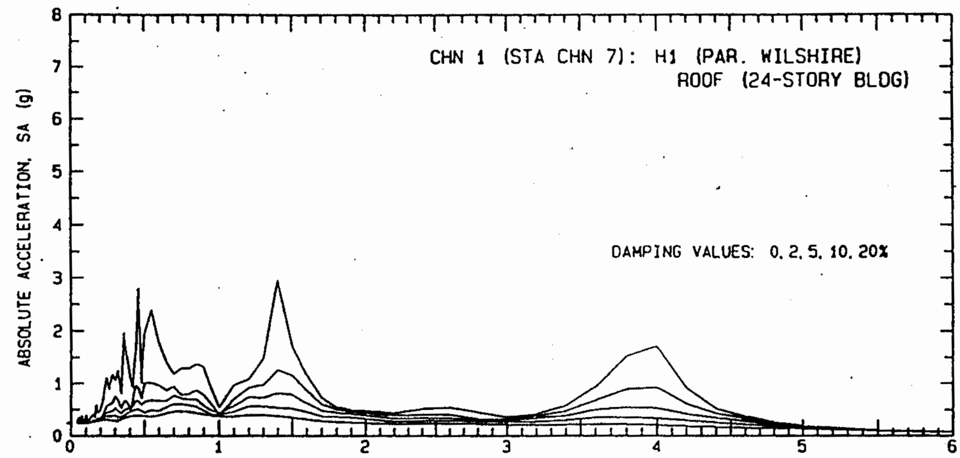
Note: Longitudinal direction is perpendicular to Ventura Boulevard



Note: Longitudinal direction is perpendicular to Ventura Boulevard.

FIGURE 8


DISPLACEMENT
CHN 1 (STA CHN 7): H1 (PAR. WILSHIRE)
ROOF (24-STORY BLDG)
Max = 33 cm
CHN 2 (STA CHN 8): UP
3 cm
CHN 3 (STA CHN 9): H2 (PERP. WILSHIRE)
-10 cm
DISPLACEMENT (cm)
TIME (SEC)
2C131-S5248-94024.08
07.1895.1428


CHN 1 (STA CHN 7): H1 (PAR. WILSHIRE)
ROOF (24-STORY BLDG)
DAMPING VALUES: 0, 2, 5, 10, 20%
ABSOLUTE ACCELERATION, SA (g)

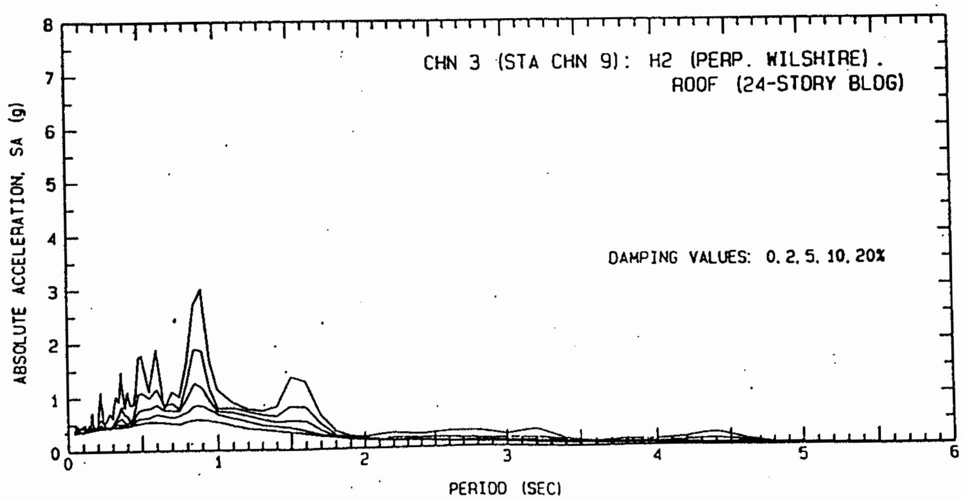

CHN 3 (STA CHN 9): H2 (PERP. WILSHIRE).
ROOF (24-STORY BLDG)
DAMPING VALUES: 0, 2, 5, 10, 20%
ABSOLUTE ACCELERATION, SA (g)
PERIOD (SEC)

FIGURE 9