Degradation of Plywood Roof Diaphragms under Multiple Earthquake Loading

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ABSTRACT

This paper summarizes the interim findings of research examining the recorded response of three buildings with concrete walls and plywood roof diaphragms to repeated strong motion events. Observed stiffness characteristics of the diaphragms are compared for each successive event and with that predicted by design formula and available data from static tests. Recorded response of the diaphragms indicates an initial dynamic stiffness substantially in excess of that predicted by static tests and design formulae. Damping for these diaphragms is determined to be low, on the order of 5% or less. Degradation of dynamic stiffness, of highly stressed diaphragms with large aspect ratios is apparent. However, the observed degraded stiffness of these diaphragms is still in excess of that predicted by conventional design formulae. Research was performed under a grant from the California Division of Mines and Geology.

INTRODUCTION

The use of long span plywood roof diaphragms, with large aspect ratios, has been common practice in low rise commercial and industrial construction throughout California and other western states. Commonly used in combination with tiltup concrete and reinforced masonry walls, these diaphragms are often heavily loaded and are expected to experience large deformations under seismic loads. The performance of early structures of this type in strong ground motion has been poor. Failures occurred in the 1964 Alaska earthquake[1], the 1971 San Fernando Earthquake[2] and the 1987 Whittier earthquake[3]. These failures could be attributed to two principal failure modes: 1- cross grain tension or flexural failure of the wood framing at plywood margins; or 2- pull-out failure of the nails through the edges of the plywood.

Subsequently, the Structural Engineers Association of California recommended[4] and the Uniform Building Code[5] adopted detailing provisions intended to prevent such failures. These provisions included prohibition of the use of wood framing in cross-grain tension or flexure and requirements for providing continuous ties across the width of the diaphragms to prevent tensile failure. The performance of buildings designed to these more recent provisions has been substantially improved, however, evidence of secondary modes of diaphragm degradation have been reported[3]. Observed damage has included substantially weakened nailing of the plywood to framing members, including nail withdrawals as well as edge failures resulting from nails pulling through the edges of plywood sheets.

Concern has been expressed that typical post-earthquake damage inspections of these structures may not indicate the presence of such damage unless it is extreme, and degraded plywood diaphragms may never be restored to their original condition. This presents a significant potential problem in zones of high seismicity, where structures may experience several strong ground motion events in the course of their useful lives, with continual degradation of their capacity to resist such motions. Buildings in the San Jose, California region for example have been subjected to strong ground motion several times over the last 10 years. Events have included the 1979 Halls Valley (M6.0), 1984 Morgan Hill (M6.2), 1988 Alum Rock (M5.5) and 1989 Loma Prieta (M7.1) earthquakes. Although only the latter event could be considered a major earthquake, it should be remembered that buildings of this type have seen significant damage in low magnitude events such as Whittier Narrows (M5.9).

A primary objective of this research is to determine if recordings of ground motion and structural response for three concrete tiltup buildings with plywood roof diaphragms, in successive earthquake excitations, indicate any significant degradation in structural rigidity, as evidenced by their response. Secondary objectives for this research are to determine if conventional design assumptions on the stiffness and loadings assigned to these structures are realistic in light of observed response.

SUBJECT BUILDINGS

The three buildings investigated in this research are a single story warehouse in Hollister (CSMIP Station No. 47391), a single story gymnasium structure at the West Valley College in Saratoga (CSMIP Station No. 58235) and a two story office structure in Milpitas (CSMIP Station No. 57502). Table 1 summarizes the ground motion records reviewed for each building under this project. The performance of the West Valley College Gymnasium building, during the 1984 Morgan Hill Earthquake has previously been evaluated by other researchers[6]. Due to a delay in obtaining data on the buildings and recorded ground motions, analysis of all three buildings is not complete as of this writing. The findings on the studies of the West Valley College Gymnasium, a single story structure in Saratoga, and the two story Milpitas Office building will be presented in an additional paper.

The single story Hollister warehouse has overall dimensions of 100 feet east to west by 300 feet north to south. It is constructed of 6" thick precast concrete wall panels, with heights of 30 feet and widths varying from 18 to 22 feet. Panel joints consist of cast-in-place pilasters, in which horizontal panel steel is embedded. The roof is a panelized plywood system consisting of glulam beams at 18 feet, spanning north to south between the pilasters and a single row of columns; 4 x 14 sawn timber purlins spanning east to west at 8 feet; 2 x 4 sub purlins; and plywood sheathing. Plywood at the ends of the structure is 3/4 inch thick with 10d nails at 1-3/4" for boundaries, 3" for discontinuous edges and 12" in the field. The balance of the plywood sheathing is 1/2 inch thick with nail spacing varied as required for shear. No interior partitions are present. Figure 1 is a photograph of the building while Figure 2 indicates the basic construction and instrument locations.

Anchorage of the precast concrete walls to the roof diaphragm is accomplished with a double row of nails from the edge plywood into a 3x nailer along the top of the wall. Nails straddle the line of bolts anchoring the nailer to the wall and avoid placing the nailer into cross grain tension. Diaphragm cross ties are provided by Simpson MST hardware across purlin lines and by bolted splice plates across glulam connections.

ANALYSIS

A code analysis of the Hollister structure was performed to determine the diaphragm capacity. Based upon the design provisions of the 1988 *Uniform Building Code* (UBC)[7], the roof diaphragm has a sufficient capacity to resist a ZPA earthquake of 0.406g. This equates to an equivalent static lateral design force of 0.186g. The design lateral capacity of the diaphragm is limited by the shear in the plywood.

In the Morgan Hill and Hollister earthquakes, the Hollister warehouse was subject to moderate ground shaking, with PGA's of 0.08g and 0.11g, respectively in the direction of interest, which is east to west. The east-west PGA at the site in the Loma Prieta Earthquake was 0.25g. Peak roof accelerations at the center of the diaphragm, the base of the north wall, the top of the north wall, and demand-capacity ratios of the roof diaphragm are presented in Table 2. The Loma Prieta earthquake, with a PGA of 0.25 g, produced peak horizontal accelerations at the center of the roof (D/C_a in Table 2) over 4 times the code level static design load. Very little amplification of motion between the ground and the top of the tilt-up walls (channels 3 and 8) was observed, indicating that the walls are behaving as rigid bodies. The response of the structure is dominated by the dynamic properties of the roof diaphragm.

The above behavior is contrary to the typical model assumed by designers of these structures. UBC design procedures assume that the entire diaphragm responds at the modified spectral acceleration, taken as ZC/R_W, or 0.458Z, where Z is the peak ground acceleration, C is the spectral amplification taken as 2.75 and R_W is a response modification coefficient taken as 6. The observed behavior indicates a variation in accelerations along the diaphragm length, starting at nearly Z adjacent to the end walls and peaking at approximately 3Z at the diaphragm center. This would result in an average effective spectral acceleration, over the length of the diaphragm, of approximately 2Z. The column of D/C_V values in Table 2 expresses the relationship between the average spectral acceleration calculated for the diaphragm in each event to that implied by code, and is a measure of overstrength shear demand on the diaphragm.

Data published by the APA indicates that working stress values for plywood diaphragms incorporate a factor of safety slightly in excess of 4 against the ultimate strength condition. The code level diaphragm shear demand of 0.458Z with an APA factor of safety of about 4.2, yields an ultimate diaphragm capacity of about 1.9Z, which compares favorably with the observed average response of about 2Z. This indicates that current code design strength levels for these structures are appropriate.

Plywood diaphragms exhibit highly non-linear behavior[8], and therefore they do not possess a single fundamental frequency, except under low levels of excitation. However, for a given input motion, a predominant frequency range can be obtained. A Fast Fourier Transform (FFT) was applied to selected pairs of acceleration, velocity, and displacement records, from which the predominant structural frequencies of the roof diaphragm were extracted. Figures 3, 4, and 5 show representative plots of the frequency versus acceleration transfer function magnitude for roof and ground records in the Morgan Hill, Hollister, and Loma Prieta earthquakes, respectively.

Comparing the plots of the acceleration FFT's for the Morgan Hill and Hollister earthquakes with the plot for the Loma Prieta earthquake, a shift in the predominant frequency of the roof diaphragm is noted. This indicates a softening of the diaphragm stiffness. The magnitude of change in predominant frequency and the associated changes in the relative diaphragm stiffness between the three earthquakes is presented in Table 3. This degradation or softening of plywood diaphragms under high loads has been previously noted in static tests[8]. Non-cyclical tests of plywood diaphragms have shown them to be highly non-linear. A substantial portion of this non-linearity can be attributed to nail slip, a progressive and degenerative process. A representative load-deflection curve of test on a full size 1/2 inch plywood diaphragm, showing degradation of diaphragm stiffness with increasing load is shown in Figure 6.

Plots of acceleration versus displacement at the center of the roof (channel 4) for the Hollister and Loma Prieta earthquakes are shown in Figures 7 and 8. The acceleration versus displacement plot for the Hollister earthquake (Figure 7) is generally linear, indicating that the diaphragm remained essentially elastic and suffered no apparent degradation in stiffness due to the earthquake. This correlates well with the computed D/C_V of 0.93. The plot for the Loma Prieta earthquake (Figure 8) shows considerable softening of the diaphragm, as evidenced by the trend towards decreasing slope in the acceleration versus displacement plot. The magnitude of the change in diaphragm stiffness is in good agreement with the change in stiffness predicted by the period shifts observed using the FFT.

In order to compare the computed diaphragm stiffness to the stiffness actually observed, and to evaluate the accuracy of current methods for predicting wood diaphragm displacements, a simple linear-elastic finite element model of the roof diaphragm was constructed. The model properties were tuned to produce deflections under static lateral load equal to those obtained using the deflection formula in the 1988 UBC Standard 25-9[9]. For the purposes of this model, flexural properties were calculated as those produced by chords consisting of a one-half height strip of the side walls. The fundamental frequency of this "code" model was evaluated, and found to be 0.83 hertz, less than one half the predominant frequencies observed in both the Morgan Hill and Hollister events. By increasing the model diaphragm shear stiffness to 4.5 times the shear stiffness computed per UBC Standard 25-9, a fundamental frequency comparable to the predominant frequency observed in the Hollister and Morgan Hill events was obtained. To match the model fundamental frequency to the predominant frequency observed in the Loma Prieta earthquake, it was necessary to increase the diaphragm shear stiffness to 2.3 times the UBC stiffness.

There are several possible sources for the wide difference between the observed diaphragm stiffness and that computed by conventional methods. Principal components of the UBC stiffness calculation are the flexural contribution of the chords, slip in chord connections (neglected in this case), elastic properties of the wood membrane, and slip of the nails. In the current model, flexural effects account for approximately half of the diaphragm flexibility. In a building with relatively solid and rigid side walls, such as the Hollister warehouse, conventional simple span assumptions may be highly inaccurate. Further, elastic properties of the wood membrane and nail slip values are based upon moderate duration, statically applied loads. Under short duration dynamic loading, it would be reasonable to expect stiffer response.

Using the roof diaphragm frequencies obtained from the FFT's and the recorded peak roof accelerations, the damping of the roof system was estimated using the ground response spectra prepared by CDMG for each event. In all cases, damping was found to be substantially less than 5% of critical. This is confirmed by the relatively closed loops observed on the time history plots of acceleration versus displacement, indicating little hysteretic behavior.

CONCLUSIONS

A study of the earthquake records of the Hollister warehouse for three successive events has been completed, and is underway for two other structures. The last event, the Loma Prieta earthquake, produced peak accelerations in the center of the diaphragm over 4 times the psuedo-static design acceleration. Based upon the available data, the following observations were made:

- In the Loma Prieta earthquake, the roof diaphragm of the Hollister warehouse showed a marked decrease in stiffness, when compared to its performance in the more moderate Morgan Hill and Hollister events. However, the degraded stiffness of the diaphragm was still several times greater than that predicted by conventional design models.

- Peak ground accelerations experienced by the Hollister warehouse in the Loma Prieta earthquake were approximately 65% of the nominal design basis of 0.4g. Diaphragm performance at this level was acceptable.

- Conventional design models for structures of this type assume dynamic amplification in the shear walls and a uniform acceleration of the diaphragm. Observed response indicates negligible amplification in the walls and significant diaphragm response and amplification. Regardless, conventional design ppprocedures and force levels appear to provide adequate strength for the criteria earthquake.

- o The method presented in UBC Standard 25-9 for computing displacement of plywood diaphragms is a poor predictor of the dynamic stiffness of these structures at working stress levels. It also appears that actual dynamic displacements during strong ground motion are substantially over-estimated. Additional data would be required to evaluate the dynamic displacements of the diaphragms at ultimate load.

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7- International Conference of Building Officials. 1988. "Uniform Building Code." Whittier, California

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9- International Conference of Building Officials. 1988. "Uniform Building Code Standards." Whittier, California

TABLE 1

SUMMARY OF DATA INVESTIGATED

Building	Station	Constructed	Earthquake	PGA
Hollister Warehouse	47391	1979	1984 Morgan Hill	0.06g
			1986 Hollister	0.13g
			1989 Loma Prieta	0.35g
West Valley College	58235	1971	1984 Morgan Hill	0.1g
			1989 Loma Prieta	0.53g
Milpitas 2 story	57502	1984	1988 Alum Rock	
			1989 Loma Prieta	0.14g

TABLE 2

BUILDING RESPONSE - TRANSVERSE (EAST-WEST) DIRECTION

Event	Recorded Peak Accelerations (g) Channel 8	Channel 3	Channel 4	D/C_a Ratio	D/C_v Ratio
1984 Morgan Hill	0.08	0.09	0.25	1.37	0.93
1986 Hollister	0.11	0.13	0.29	1.58	1.15
1989 Loma Prieta	0.25	0.25	0.79	4.32	2.84

Channel 8 - at grade, North wall
Channel 3 - at roof, North wall
Channel 4 - at center of roof

Code Static Design Force C = 0.183 g
Demand D = peak acceleration, Channel 4
D/C_a = the ratio of peak diaphragm acceleration to code design acceleration
D/C_v = the ratio of average peak diaphragm acceleration to code design acceleration

TABLE 3

OBSERVED CHANGE IN DIAPHRAGM STIFFNESS

Event	Predominant Frequency (hz)	Relative Stiffness
1984 Morgan Hill	1.77	1.00
1986 Hollister	1.72	0.94
1989 Loma Prieta	1.23	0.48

Relative Stiffness normalized to that observed in the 1984 Morgan Hill earthquake.

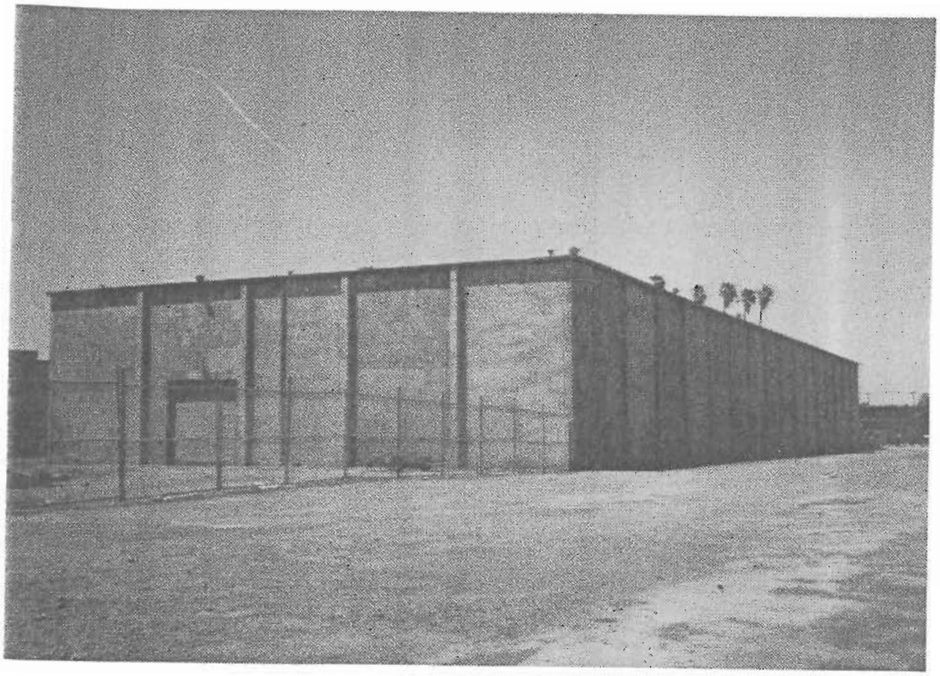
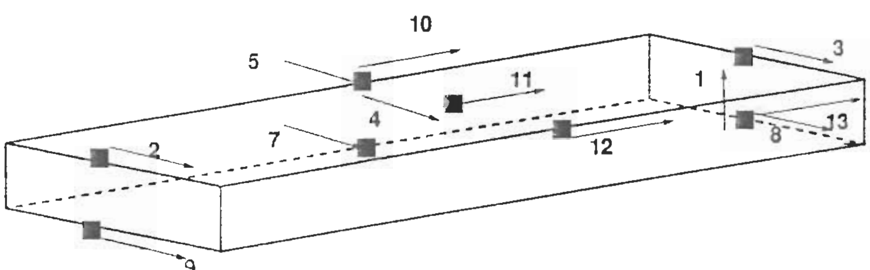


Figure 1. Hollister Warehouse



Isometric View of Instrument Channels

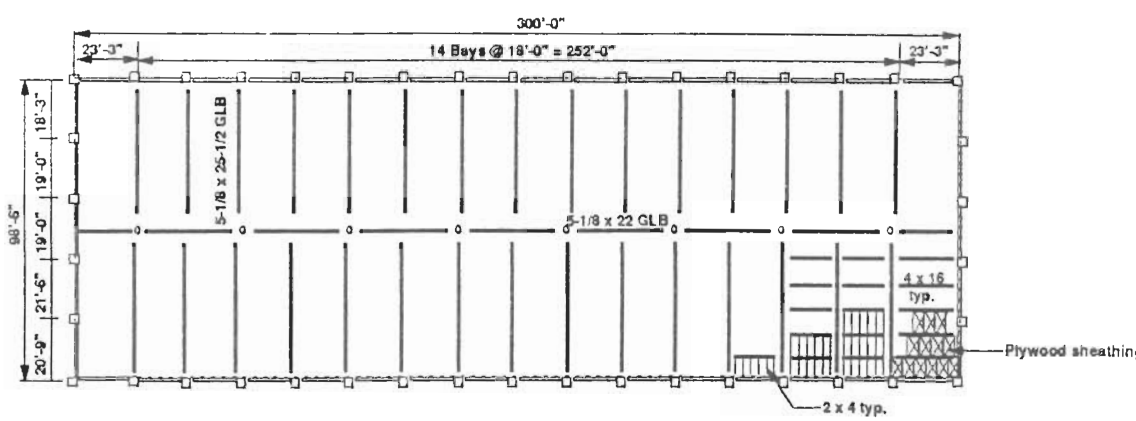


Figure 2. Hollister Warehouse, Roof Plan
and Instrument Layout

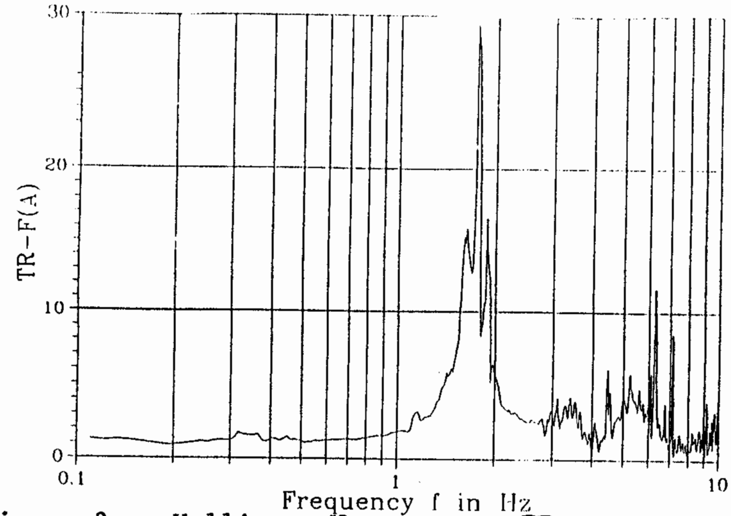


Figure 3. Hollister Warehouse - FFT(A) vs. Frequency
Morgan Hill earthquake, April 24, 1984.

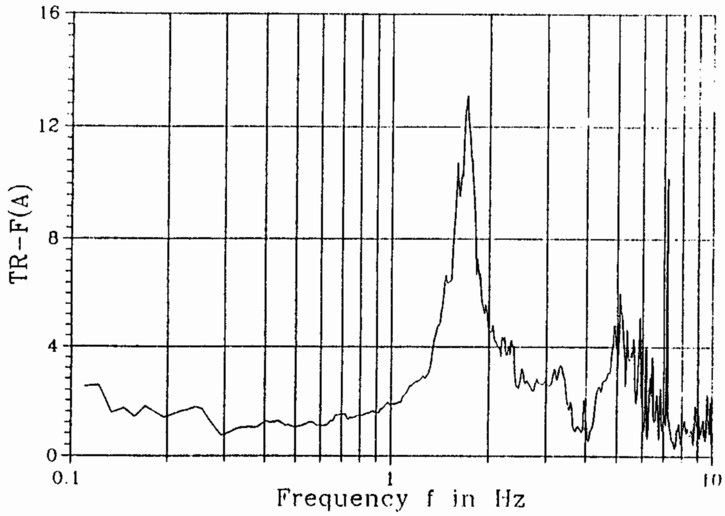


Figure 4. Hollister Warehouse - FFT(A) vs. Frequency
Hollister earthquake, January 26, 1986.

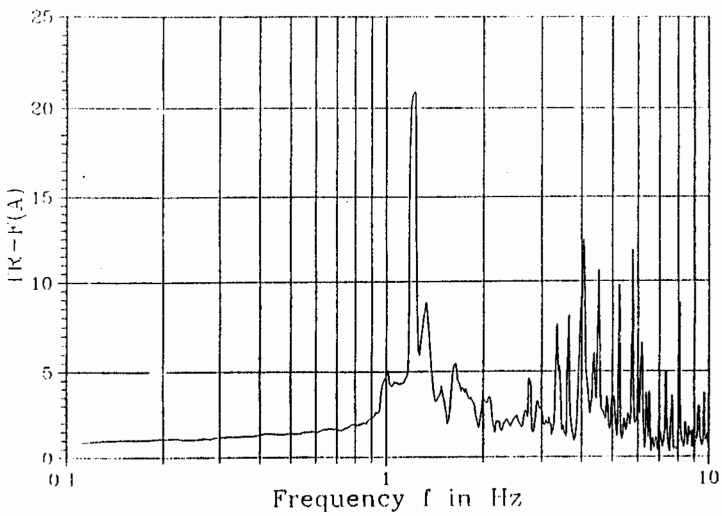


Figure 5. Hollister Warehouse - FFT(A) vs. Frequency
Loma Prieta earthquake, October 17, 1989.

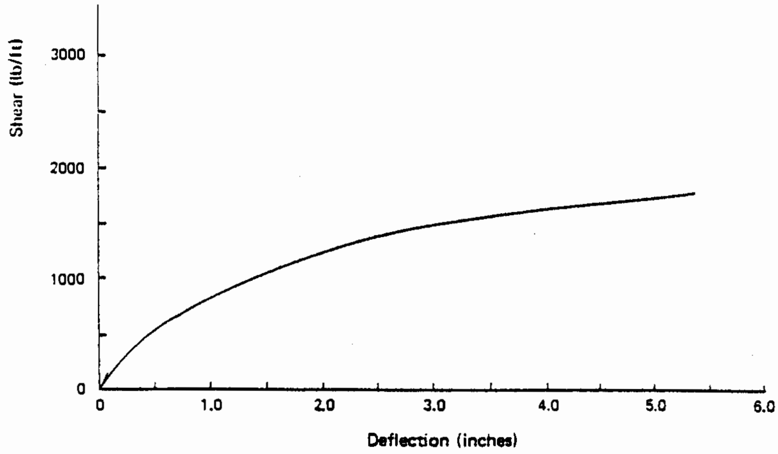


Figure 6. Shear-Deflection Plot for 1/2 inch Plywood Diaphragm. (APA)

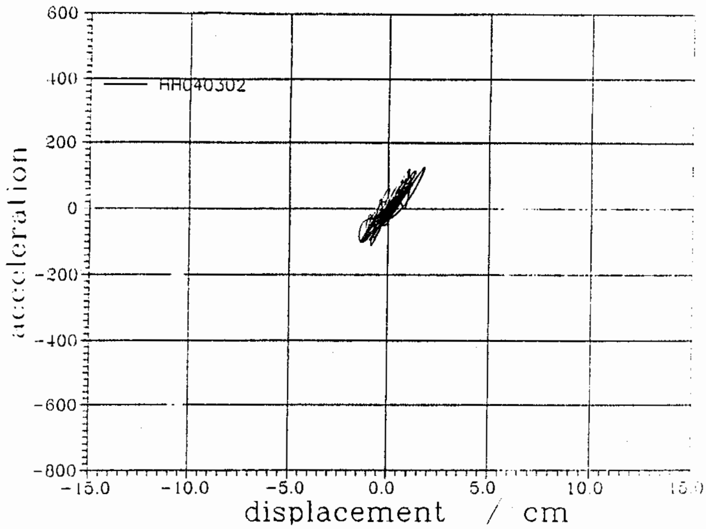


Figure 7. Acceleration vs. Displacement
Hollister earthquake, January 26, 1986.

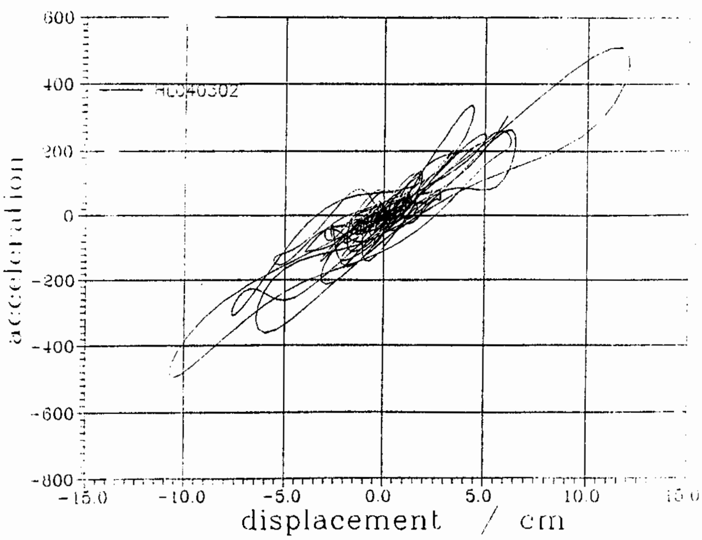


Figure 8. Acceleration vs. Displacement
Loma Prieta earthquake, October 17, 1989