DEGRADATION OF PLYWOOD ROOF DIAPHRAGMS UNDER MULTIPLE EARTHQUAKE LOADING

by

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Data Utilization Report CSMIP/94-02

California Strong Motion Instrumentation Program

February 1994

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This study was conducted at the EQE International in San Francisco and was supported by the Department of Conservation under Contract No. 1090-503.

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PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of the this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Data Interpretation Project are to:

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.

2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.

3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.

4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.

5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is the seventh in a series of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Data Interpretation Research Project.

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ABSTRACT

This report summarizes the findings of a study 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 generally be low, on the order of 5% or less. Degradation of dynamic stiffness of highly stressed diaphragms is apparent. However, the observed degraded stiffness of these diaphragms is still in excess of that predicted by conventional design formulae. It was determined that the design of roof diaphragms for a response modification factor (Rw) of 6 is appropriate regardless of the building Rw. The building period increases after repeated earthquakes due in part to the degradation of the plywood diaphragm. Following a strong earthquake, (a design level earthquake), the roofing should be removed to inspect the nailing and renail as appropriate in order to restore the stiffness of the diaphragm.

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INTRODUCTION

The use of long span plywood roof diaphragms, some 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 tilt-up concrete, cast-in-place 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 (NRC), the 1971 San Fernando earthquake (U.S.D.C.) and the 1987 Whittier earthquake (Hamburger). 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 (SEAOC, 1973) and the Uniform Building Code (ICBO, 1973) 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. 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 moderate 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 shear wall 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 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 Saratoga gymnasium building, during the 1984 Morgan Hill earthquake has previously been evaluated by other researchers (Celebi, et al).

Table 1
SUMMARY OF DATA INVESTIGATED

Building	Station	Earthquake	PGA N-S	PGA E-W
Hollister Warehouse	47391	1984 Morgan Hill	0.06g	0.08g
		1986 Hollister	0.13g	0.11g
		1989 Loma Prieta	0.35g	0.25g
Saratoga Gymnasium	58235	1984 Morgan Hill	0.04g	0.04g
		1989 Loma Prieta	0.35g	0.24g
Milpitas 2-Story Bldg	57502	1988 Alum Rock	0.07g	0.07g
		1989 Loma Prieta	0.09g	0.14g

The Hollister One-Story Warehouse
(CSMIP Station No. 47391)

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 horizontal panel reinforcement are diagonally lapped across the thickness of the pilaster and assumed to be developed and adequate to transfer chord forces. The roof is a panelized plywood system consisting of glulam beams at 18 feet, spanning east to west 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 and continuous edges, 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 dimensions and sensor locations.

Figure 1. Hollister Warehouse

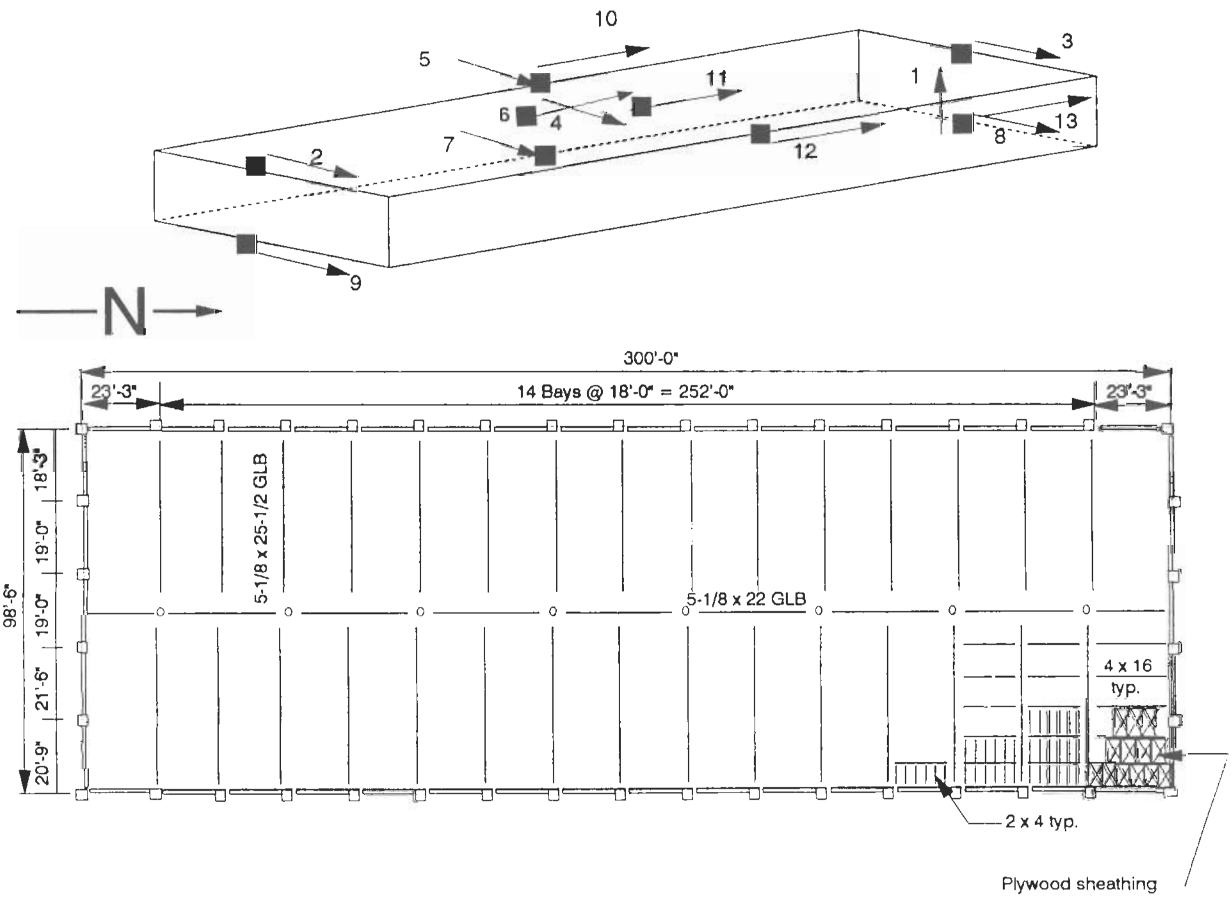

Figure 2. Hollister Warehouse, Roof Plan and Sensor Layout

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, in an attempt to 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.

The Saratoga One-Story Gymnasium

(CSMIP Station No. 58235)

The single-story gymnasium in Saratoga has overall dimensions of 144 feet north to south by 112 feet east to west. The roof of this single story structure is located 34 feet above grade. The roof construction consists of 3/8" plywood over 3x6 Tongue and Groove (T&G) straight sheathing, supported by W8x10 steel purlins at 8 feet spacing. The purlins span 16 feet to a series of long span Vierendeel trusses which span transversely (east to west) across the gymnasium. The trusses are fabricated from 12 inch channels, placed back to back, and have a total depth of 6 feet. They are supported by a series of cast-in-place concrete buttresses, 5 feet long and 18 inches wide, located around the perimeter of the building. The walls and buttresses are supported on 5 foot wide by 5 foot deep perimeter grade beams. Lateral load resistance is provided by a series of infill concrete shear panels, located between selected buttresses. Figure 3 is a photograph of this building and Figure 4 shows its basic dimensions and the locations of sensors.

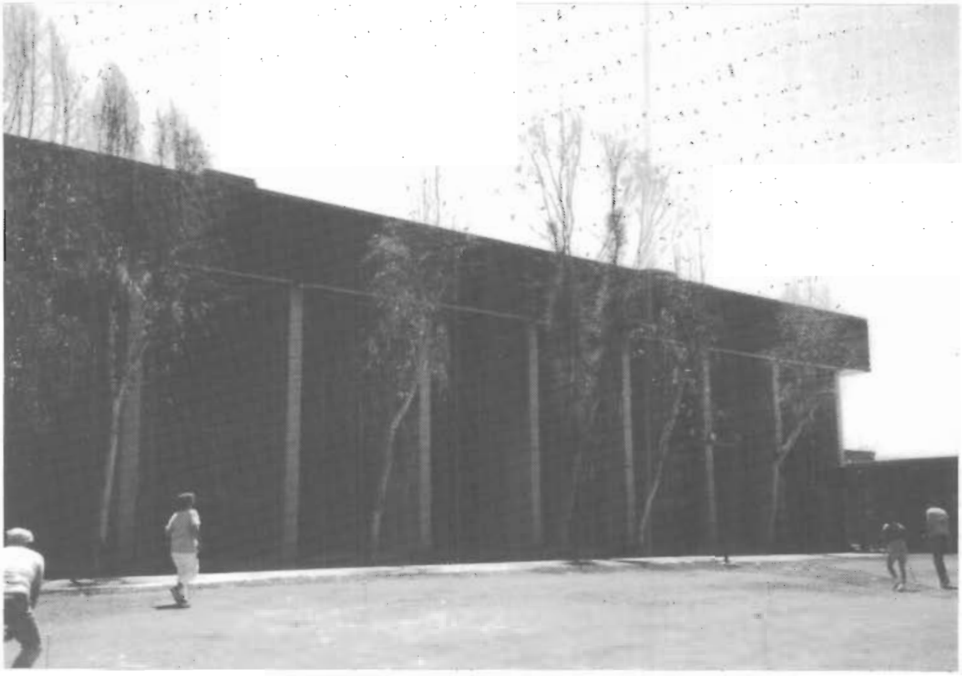

Figure 3. West Valley College Gymnasium

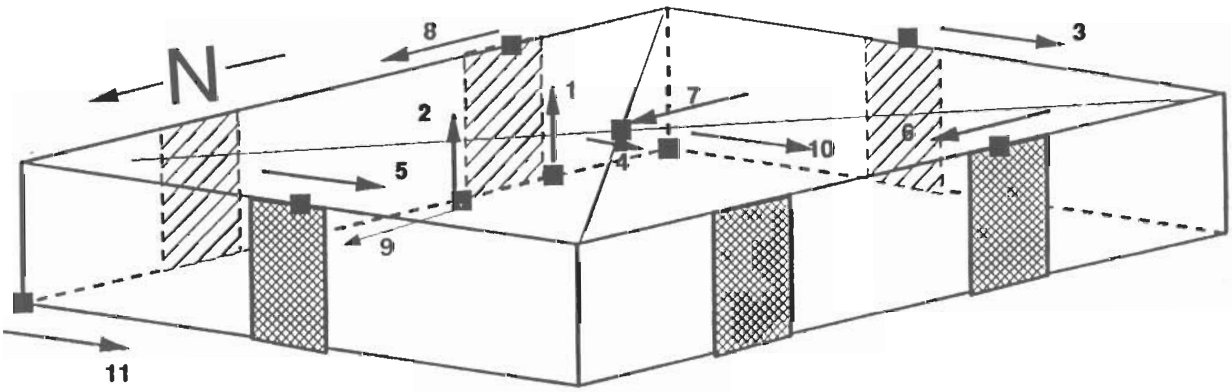

Figure 4. West Valley College Gymnasium, Roof Plan and Instrument Layout

Anchorage of the roof structure to the perimeter walls and buttresses is provided by connection of steel roof framing to embedded plates. The steel roof framing members also form a system of continuous crossties. The 3/8 inch plywood sheathing is applied over the 3x6 tongue and groove straight sheathing, which in turn spans to the steel support structure. The plywood is nailed to the T&G sheathing with 10 penny nails at 3 inch spacing at panel edges and 10 inch spacing in the field. Diaphragm chord members are S12x50 structural steel shapes spliced with welded connections.

The Milpitas Two-Story Industrial Building

(CSMIP Station No. 57502)

The two-story Milpitas Office building has overall dimensions of 120 feet north to south by 168 feet east to west. A 24 feet wide by 96 feet long, single story extension is present at the center of the south side and a 12 by 24 feet entrance alcove is present at the north side. Exterior walls are 8 inch thick precast concrete panels, two stories tall. The wall panels are pierced for large window openings, and the vertical edges of the walls are thickened to 16 inches.

The roof is a panelized plywood system consisting of 1/2 inch plywood over 2x4 sub-purlins, supported by 4x16 sawn purlins at 8 feet and 5-1/8" glulam beams at 24 feet. Spacing of the 10d nailing of the diaphragm varies between 2 and 6 inches along panel edges to match the shear demand on the diaphragm. Nails in the field are spaced at 12 inches. Wall anchorage is accomplished by hold-down hardware attached directly to the sides of roof framing. Continuous diaphragm cross ties are provided by Simpson MST hardware across purlin joints and by steel plates across glulam connections. A false mansard roof is present around the perimeter of the building.

The second floor is constructed of a 2-1/2 inch thick concrete slab on metal deck, supported by open web steel joists. Joists span north to south and are supported by open web joist girders, spaced 30 feet apart and spanning 24 feet between exterior walls and interior columns. Figure 5 is a photograph of the building and Figure 6 shows the basic dimensions and locations of sensors.

Figure 5. Milpitas Two-Story Industrial Building

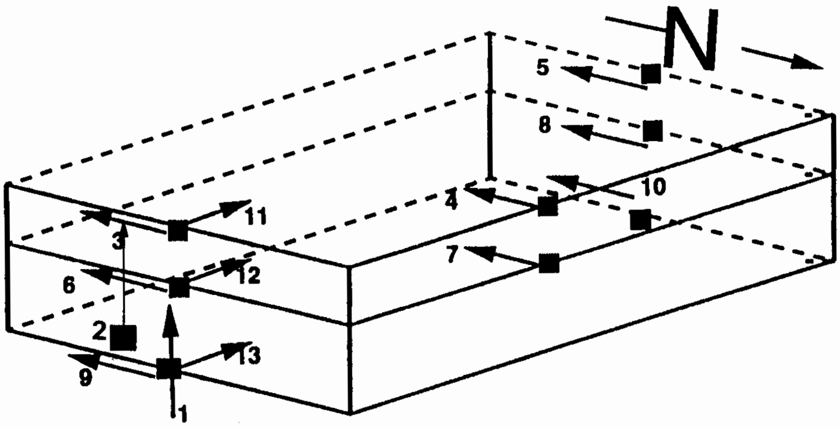

Figure 6. Milpitas Two-Story Industrial Building
Building Instrument Layout

ANALYSIS

For each of the three buildings, an analysis of the roof diaphragms was performed, based upon the provisions of the 1988 *Uniform Building Code.* Plywood diaphragms are considered to behave as horizontal beams which uniformly distribute load. The plywood acts as the web of the beam, transferring shear. The members at the periphery of the roof act as the flanges of the beam. The force on these chord elements is calculated as the moment on the diaphragm (assuming a simple span condition) divided by the depth of the diaphragm. The purpose of this analysis was to determine the design lateral capacity of the diaphragm. In addition, expected diaphragm deflections were obtained, using the deflection formula found in 1988 UBC Standard 25-9, equation 1.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188\,\mathrm{Le_n} + \frac{\Sigma(\Delta_c X)}{2b}$$

Plywood diaphragms exhibit highly non-linear behavior (Elliott), 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. For each of the buildings, 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. A FFT is a method of representing ground motion as a sum of a series of sinusoidal functions of varying frequencies. The frequency of sinusoidal functions having the greatest participation of wave form indicates the predominate response frequency. The FFT process was repeated for each of the earthquake records available. Observed shifts in the predominant frequency of the diaphragm were related to changes in the overall diaphragm stiffness.

Plots of acceleration versus relative displacement at the center of the roof were made, which might provide further indication of possible softening of the diaphragms when subjected to multiple earthquakes. Finally, the damping of each roof system was estimated using the recorded peak roof accelerations and roof diaphragm frequencies obtained from the FFT's.

The first structure examined in this study was the Hollister Warehouse. In light of the results obtained in the Hollister Warehouse investigation, the West Valley College Gymnasium and the Milpitas 2-Story Building were examined, to establish whether these two buildings are exhibiting similar behavior.

The Hollister Warehouse

The code analysis of the Hollister structure, based upon the design provisions of the 1988 *Uniform Building Code* (UBC), indicates that the roof diaphragm has a sufficient capacity to resist an earthquake with an effective peak ground acceleration of 0.406g (i.e., the value of Z as defined for formula 12-1 in the 1988 UBC, (EQN 2), could be 0.406, based upon the capacity of the diaphragm).

$$V = \frac{Z\,I\,C}{R_W}\,W \qquad (12\text{-}1)$$

This equates to an equivalent static lateral design force of 0.186g per equation 12-1. The design lateral capacity of the diaphragm is limited by 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.

Table 2
HOLLISTER WAREHOUSE
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	1.34	0.91
1986 Hollister	0.11	0.13	0.29	1.56	1.13
1989 Loma Prieta	0.25	0.25	0.79	4.25	2.79

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

Code Static Design Force C = 0.186 g
Demand D = peak acceleration, Channel 4
D/C_a = the ratio of peak diaphragm acceleration to code design acceleration (0.186g)
D/C_V = the ratio of average peak diaphragm acceleration* to code design acceleration
* the average peak diaphragm acceleration is the average of the maximum diaphragm acceleration and the average of the diaphragm accelerations at the two end walls.

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 (D) to that implied by code (C_V), and is a measure of shear demand versus capacity for the diaphragm.

Data published by the American Plywood Association (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, and that the use of larger response modification factors (R_W) would be unconservative.

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 7, 8, and 9 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.

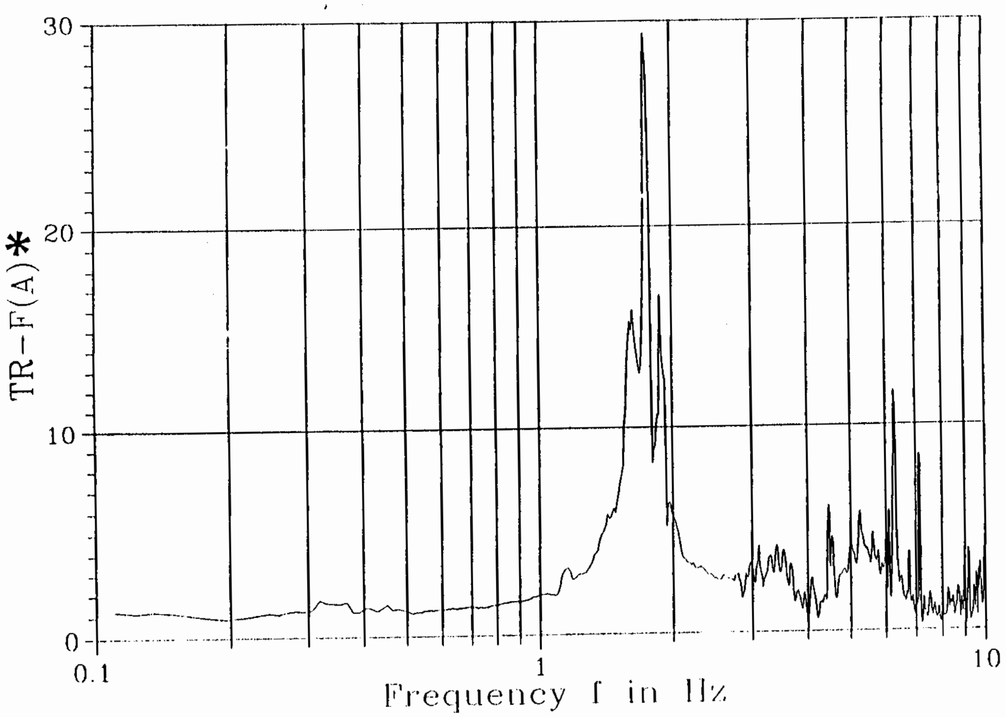

Figure 7. Hollister Warehouse - FFT(A) vs. Frequency
Morgan Hill earthquake, April 24, 1984.
* = Transfer function between roof acceleration and ground acceleration

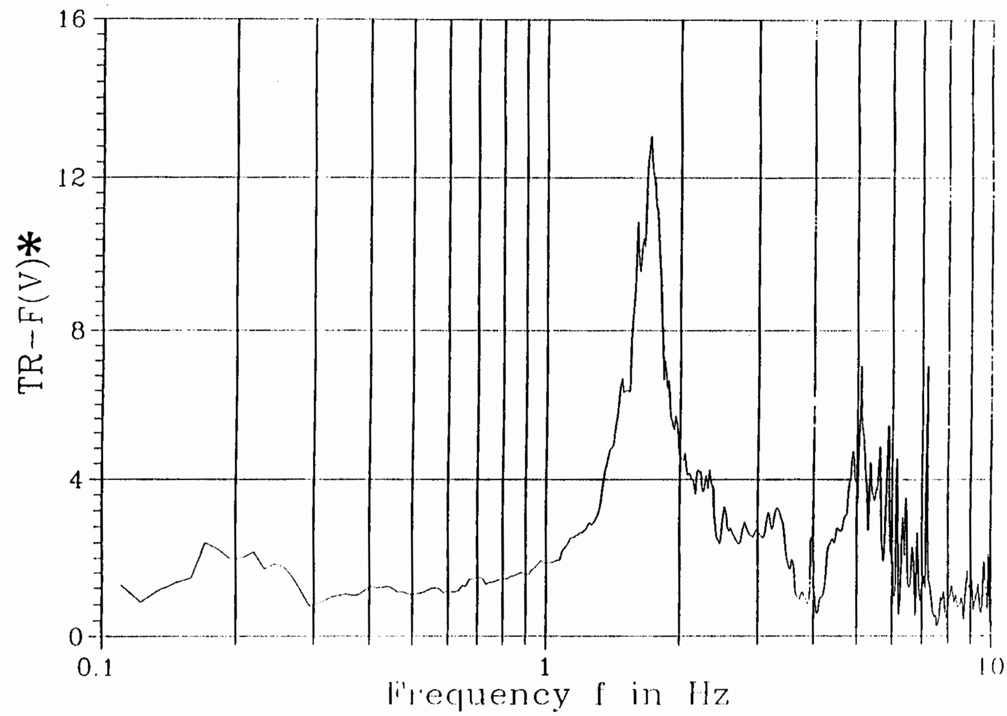

Figure 8. Hollister Warehouse - FFT(A) vs. Frequency
Hollister earthquake, January 26, 1986.
* = Transfer function between roof velocity
and ground velocity

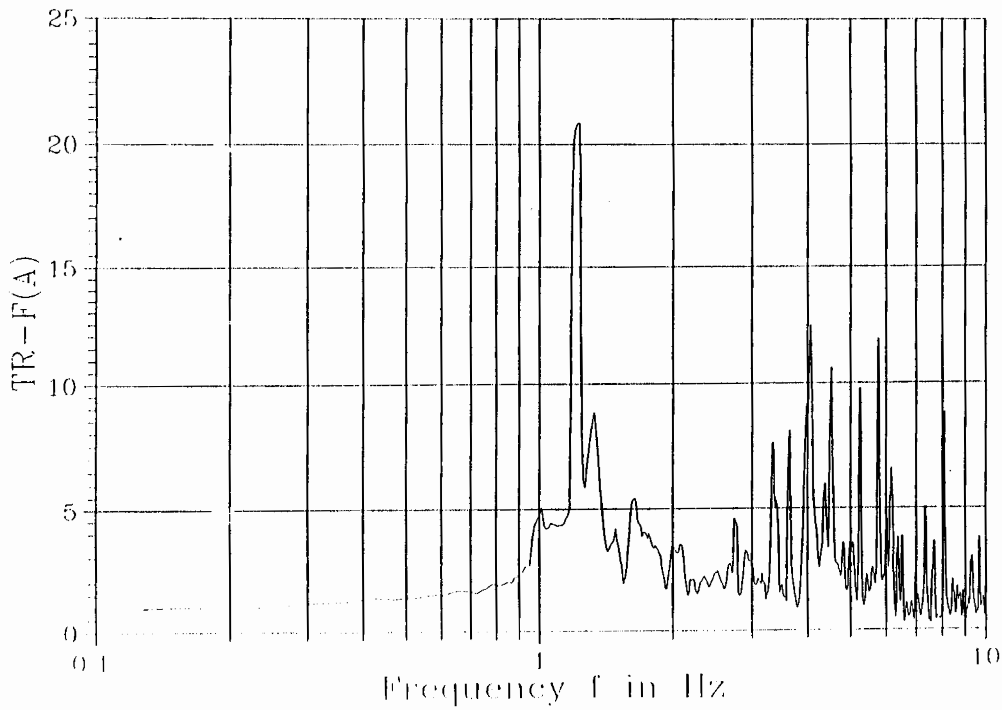

Figure 9. Hollister Warehouse - FFT(A) vs. Frequency
Loma Prieta earthquake, October 17, 1989.

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. 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 tests on a full size 1/2 inch plywood diaphragm, showing degradation of diaphragm stiffness with increasing load is shown in Figure 10.

Table 3
HOLLISTER WAREHOUSE
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 is normalized to that observed in the 1984 Morgan Hill earthquake.

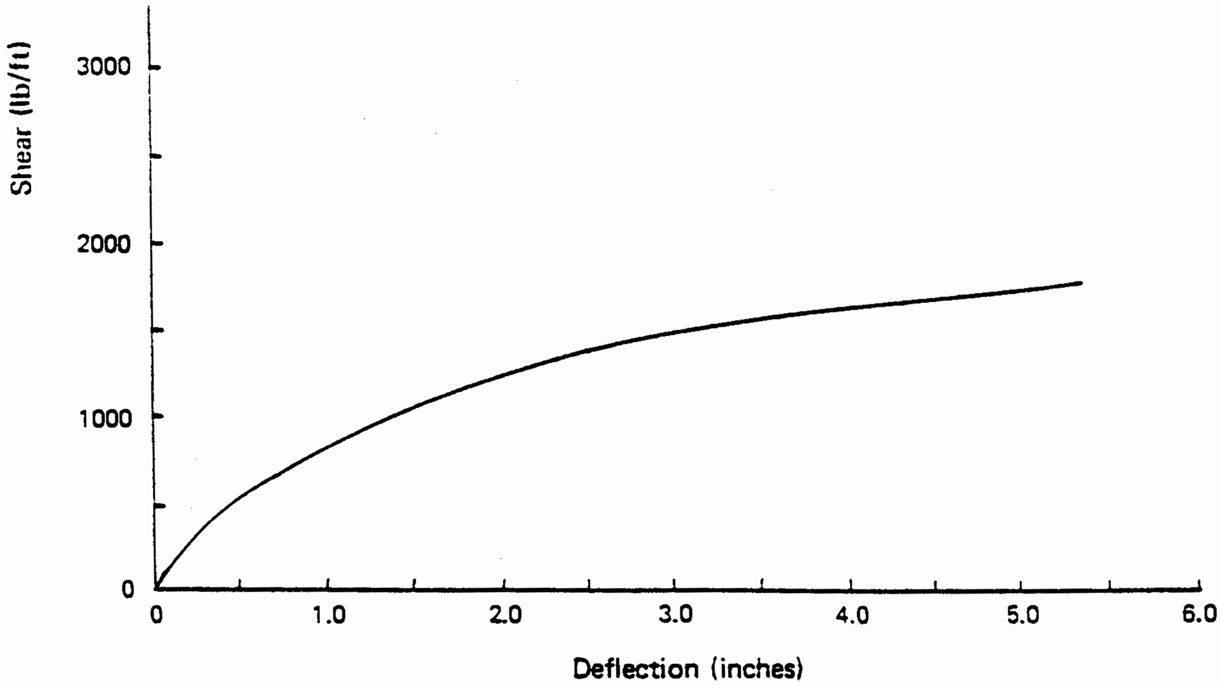

Figure 10. Shear-Deflection Plot for 1/2 inch Plywood Diaphragm. (APA)

Plots of acceleration versus relative displacement at the center of the roof (channel 4) for the Hollister and Loma Prieta earthquakes are shown in Figures 11 and 12. The acceleration versus displacement plot for the Hollister earthquake (Figure 11) 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 0.91. The plot for the Loma Prieta earthquake (Figure 12) 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 computed by the period shifts observed using the FFT.

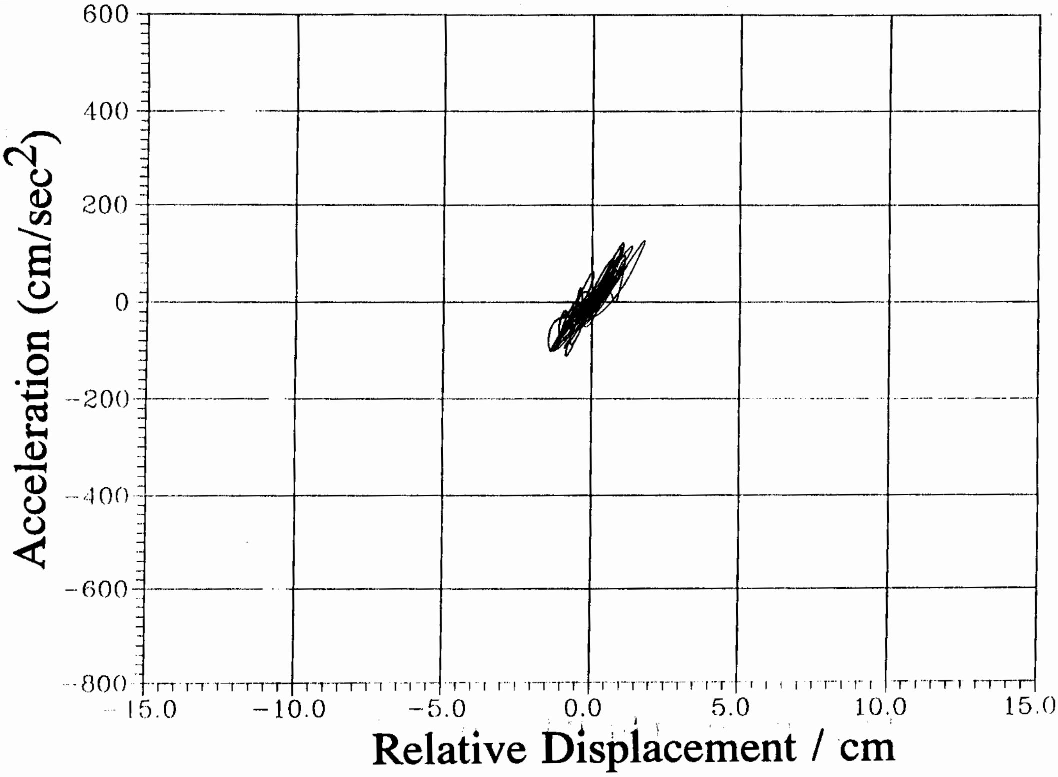

Figure 11. Hollister Warehouse Acceleration vs. Displacement
Hollister earthquake, January 26, 1986.

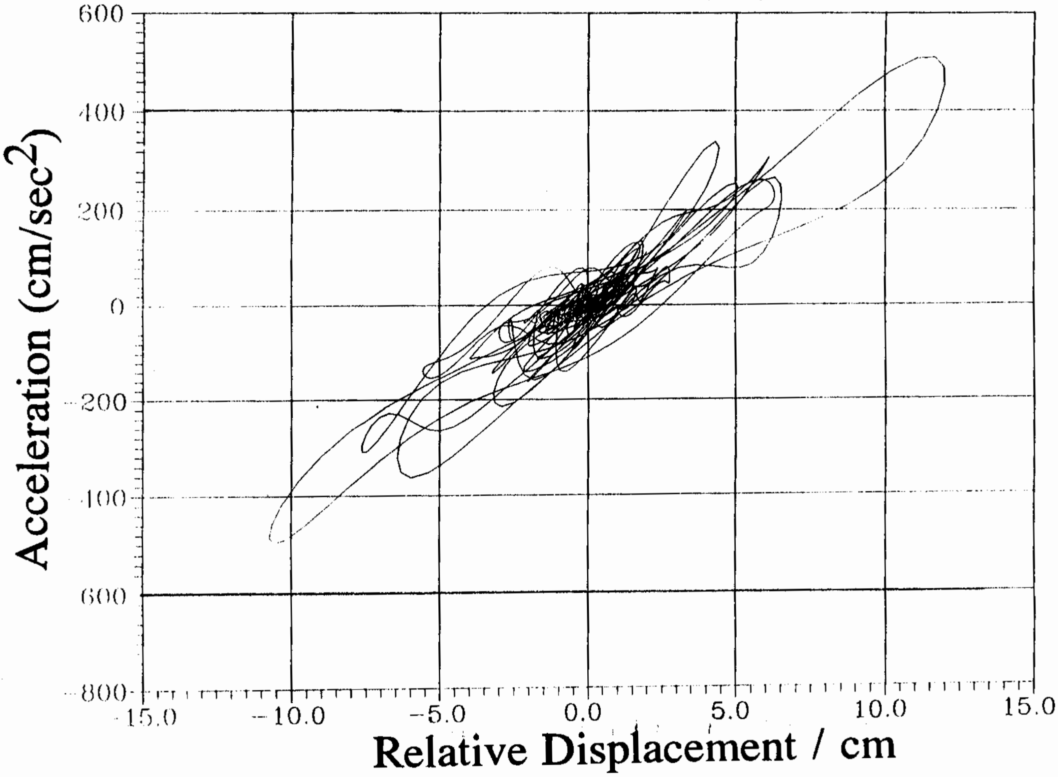

Figure 12. Hollister Warehouse Acceleration vs. Displacement
Loma Prieta earthquake, October 17, 1989

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. 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 solid and rigid side walls, such as the Hollister warehouse, conventional simple span assumptions may be highly inaccurate, particularly given a building with cast-in-place pilasters where return walls at corners provide significant rigidities. 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 about 2%. This is confirmed by the relatively closed loops observed on the time history plots of acceleration versus displacement, indicating little hysteretic behavior.

The Saratoga One-Story Gymnasium

The analysis of the Saratoga One-story gymnasium is complicated somewhat by the presence of the 3x6 T&G straight sheathing beneath the plywood diaphragm. However, this type of roof construction is somewhat similar to that found in many unreinforced masonry (URM) buildings with retrofitted plywood diaphragms, and therefore is of significant interest.

Compared to the plywood sheathing, the 3x6 T&G straight sheathing provides negligible lateral shear capacity (about 3% of the total diaphragm capacity, based upon the tabulated shear capacities of the plywood and computed shear capacity for the straight sheathing). The low shear capacity of the sheathing can be attributed to the long span (8 feet) between sheathing supports. It is therefore likely that the straight sheathing contributes little to the overall rigidity of the system.

The cast-in-place concrete buttresses around the perimeter of the building appear to have little impact on the response of the roof diaphragm. The lateral stiffness of the buttresses is small compared to that of the perimeter shear walls, especially if torsional rotation of the grade beams is considered. As discussed in the following paragraphs, a comparison of the peak accelerations at the center and ends of the roof diaphragm indicates that the roof diaphragm is behaving predominantly as if it were supported at the ends only, with no intermediate supports present.

The code analysis of the gymnasium indicates that for transverse (east-west) seismic forces the roof diaphragm has sufficient capacity to resist an earthquake with an effective peak ground acceleration (ZPA) of 0.522g, which equates to an equivalent static lateral design force of 0.239g. The design lateral capacity of the diaphragm is limited by the shear in the plywood. In the longitudinal (north-south) direction, the roof diaphragm has a sufficient design capacity to resist an earthquake with a peak ground acceleration (ZPA) of 0.437g, which equates to an equivalent static lateral design force of 0.200g. The design lateral capacity of the diaphragm in the longitudinal direction is also limited by the shear in the plywood.

In the Morgan Hill earthquake, the gymnasium was subject to moderately low intensity ground shaking, with PGA's of 0.04g and 0.10g in the transverse and longitudinal directions, respectively. The transverse and longitudinal PGA's at the site in the Loma Prieta earthquake were 0.35g and 0.24g, respectively. Peak roof accelerations in the longitudinal direction at the center of the diaphragm, the base of the east wall, the top of the east wall, and demand-capacity ratios of the roof diaphragm are presented in Table 4. Peak roof accelerations in the transverse direction at the center of the diaphragm, the base of the south wall, the top of the south wall, and demand-capacity ratios of the roof diaphragm are presented in Table 5.

Table 4
WEST VALLEY COLLEGE GYMNASIUM
BUILDING RESPONSE - TRANSVERSE (EAST-WEST) DIRECTION

Event	Recorded Peak Accelerations (g) Channel 10	Channel 3	Channel 4	D/C_a Ratio	D/C_v Ratio
1984 Morgan Hill	0.043	0.061	0.204	1.02	0.66
1989 Loma Prieta	0.354	0.448	0.875	4.38	3.31

Channel 10 - at grade, south wall
Channel 3 - at roof, south wall
Channel 4 - at center of roof

Code Static Design Force C = 0.200 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 5
WEST VALLEY COLLEGE GYMNASIUM
BUILDING RESPONSE - LONGITUDINAL (NORTH-SOUTH) DIRECTION

Event	Recorded Peak Accelerations (g) Channel 9	Channel 8	Channel 7	D/C_a Ratio	D/C_v Ratio
1984 Morgan Hill	0.100	0.136	0.408	1.71	1.14
1989 Loma Prieta	0.236	0.344	0.714	2.99	2.21

Channel 9 - at grade, East wall
Channel 8 - at roof, East wall
Channel 7 - at center of roof

Code Static Design Force C = 0.239 g
Demand D = peak acceleration, Channel 7
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

A comparison of the data in Tables 4 and 5 reveals similarities in the response of the structure in the longitudinal and transverse directions. In both directions, ground motion was amplified between the base of the wall and the roof level by a factor ranging from 1.3 and 1.45. The motion between the center of the diaphragm and the top of the wall in both directions was amplified further by a factor of approximately 2 in the Loma Prieta earthquake. In the lower intensity Morgan Hill event, the amplification between the top of wall and the center of the diaphragm was higher, with factors of 3.3 and 3.0 for the transverse and longitudinal directions, respectively. The lower amplification observed in the Loma Prieta earthquake may be attributed to the frequency content of the ground motion at the site. Spectral accelerations in the predominant frequency range of the diaphragm were lower than those for stiffer or more flexible structures, resulting in reduced response.

In the transverse direction, ground motion was amplified by the response of the diaphragm to a peak level of 2.5Z in the Loma Prieta earthquake. The average spectral acceleration over the length of the entire diaphragm was 1.9Z in the Loma Prieta earthquake, which compares favorably with the results of the Hollister Warehouse study. The correlation between the PGA and the average spectral acceleration over the length of the diaphragm is poorer for the Morgan Hill earthquake. The ground motion was amplified to a level of 4.7Z. However, ground shaking intensity was very low, with a recorded peak of 0.043g. At this shaking level, little inelastic action would be expected and the building responded without damping.

The response of the roof diaphragm to strong ground motion in the longitudinal direction was similar to that observed in the transverse direction. In the Loma Prieta earthquake, peak spectral acceleration in the longitudinal direction was about 3.0Z, and average spectral acceleration was 2.2Z, compared to 4.1Z and 2.7Z, respectively, observed in the Morgan Hill earthquake. The D/C_V ratio of 1.14 in the Morgan Hill earthquake indicates that the roof diaphragm experienced little inelastic behavior in that event, while the D/C_V ratio of 2.21 observed in the Loma Prieta earthquake implies considerable inelastic action occurred.

The predominant period of the roof structure was extracted using FFT's on select pairs of instrument records. Figures 13 and 14 are representative plots for the Morgan Hill and Loma Prieta earthquakes in the transverse direction. As with the Hollister

Warehouse, a shift in the predominant frequency of the roof diaphragm was observed, indicating a reduction of stiffness of the diaphragm. This behavior is expected, given the amount of inelastic behavior suggested by the D/C_V ratio of 3.31 observed in the Loma Prieta earthquake. The magnitude of change in the predominate frequency and diaphragm stiffness are summarized in Table 6. Plots of acceleration versus displacement at the center of the roof in the transverse direction for the Morgan Hill and Loma Prieta earthquakes are shown in Figures 15 and 16. The decreasing slope of plot for the Loma Prieta earthquake confirms that softening of the diaphragm is occurring. The magnitude of the change in diaphragm stiffness indicated by the decreasing slope of the acceleration versus displacement plots correlates well with the observed period shifts obtained using the FFT's. The damping ratio for the roof diaphragm in the transverse direction, obtained using the frequencies obtained with the FFT's and the recorded peak roof accelerations, was found to be in the range of 5%. A review of the FFT's and acceleration versus displacement plots indicate no substantial softening of the diaphragm occurred in the longitudinal direction, despite the D/C_V ration of 2.21 observed in the Loma Prieta event. Damping in the longitudinal direction was estimated at about 5% of critical.

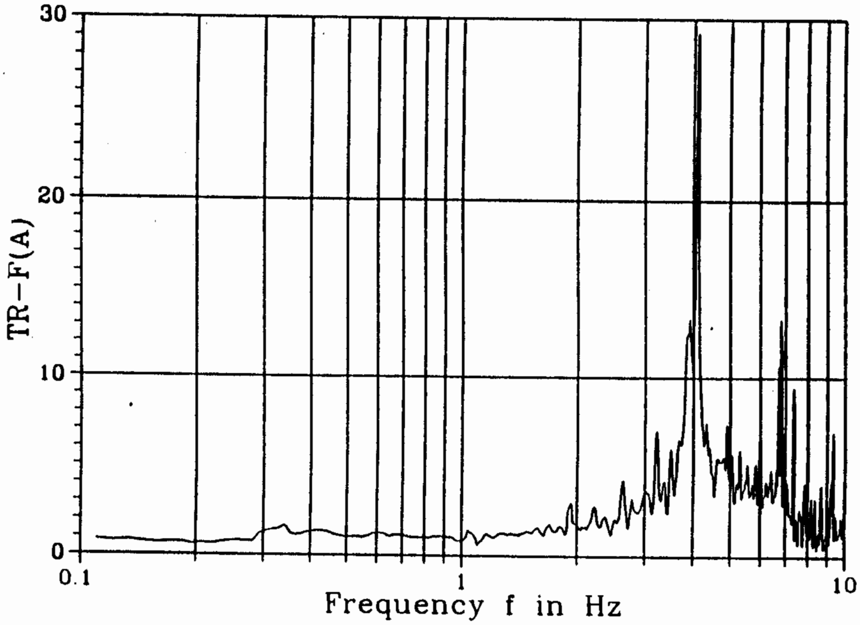

Figure 13. Saratoga Gym - FFT(A) vs. Frequency
Morgan Hill earthquake, April 24, 1984.

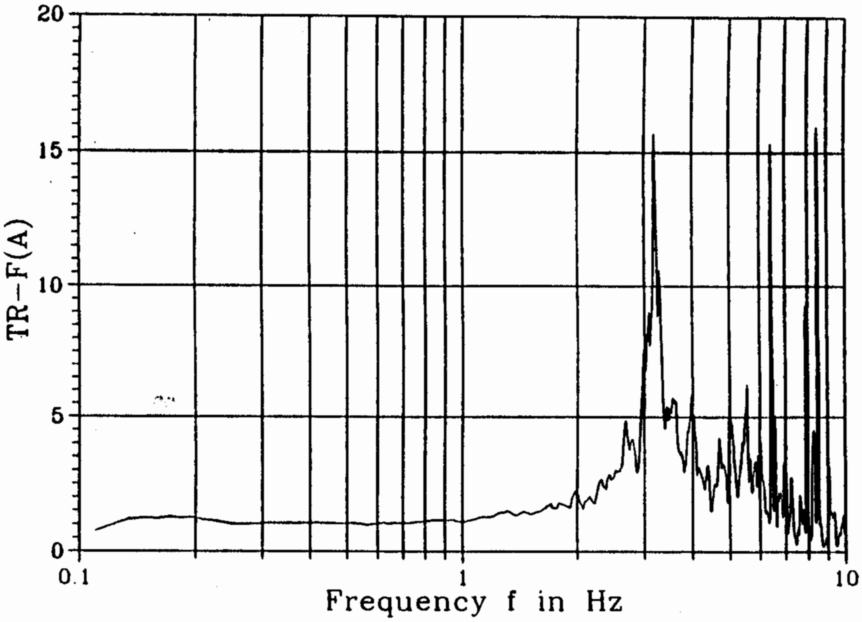

Figure 14. Saratoga Gym - FFT(A) vs. Frequency
Loma Prieta earthquake, October 17, 1989.

Table 6
WEST VALLEY COLLEGE GYMNASIUM
OBSERVED CHANGE IN DIAPHRAGM STIFFNESS - TRANSVERSE DIRECTION

Event	Predominant Frequency (hz)	Relative Stiffness*
1984 Morgan Hill	4.0	1.00
1989 Loma Prieta	3.1	0.61

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

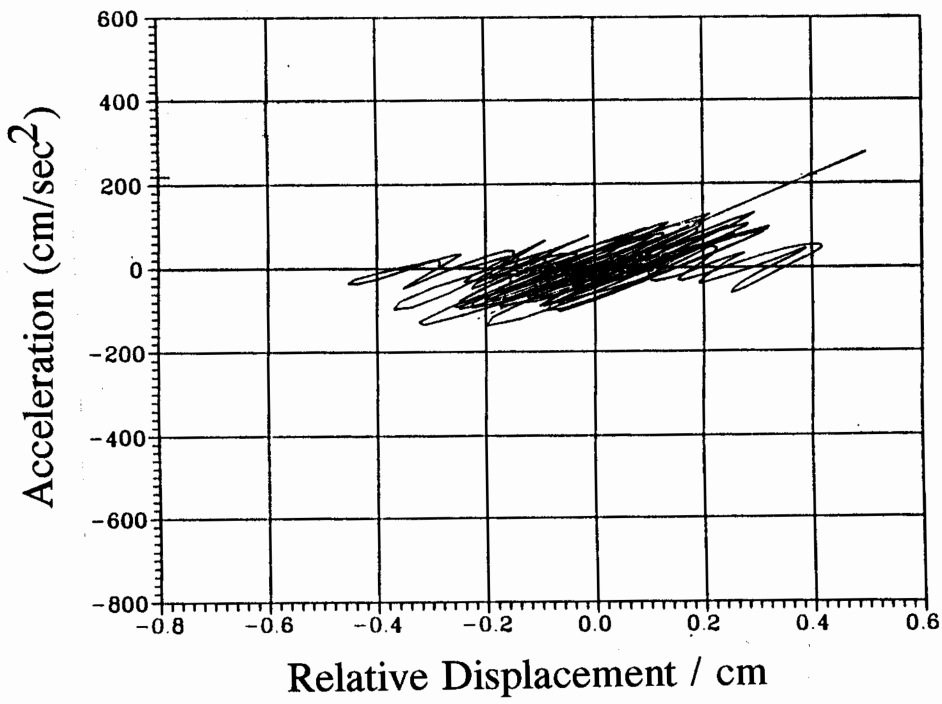

Figure 15. West Valley College Gym Acceleration vs. Displacement Morgan Hill earthquake, April 24, 1984.

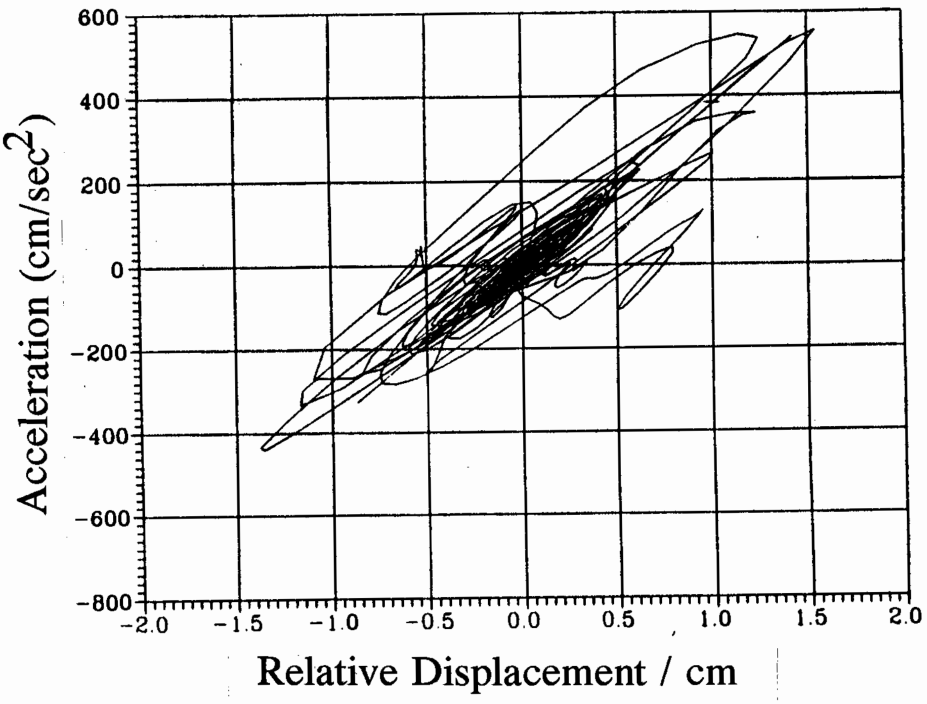

Figure 16. West Valley College Gym Acceleration vs. Displacement
Loma Prieta earthquake, October 17, 1989

The Milpitas Two-Story Industrial Building

The behavior of the Milpitas Two-Story Industrial Building in the Alum Rock and Loma Prieta earthquakes was similar to that observed in the Hollister Warehouse. In both earthquakes, the building experienced strongest shaking in the east-west direction.

A code analysis of the Milpitas building shows that for east-west seismic forces the roof diaphragm has a sufficient capacity to resist an earthquake with a peak ground acceleration (ZPA) of 0.4g, which equates to an equivalent static lateral design force of 0.183g. The design lateral capacity of the diaphragm is limited by the shear in the plywood.

In the Alum Rock earthquake, the building was subject to moderately low intensity ground shaking, with a PGA of 0.076g in the long direction. The east-west PGA in the Loma Prieta earthquake was 0.138g. Peak roof accelerations at the center of the diaphragm, the base of the north wall, and demand-capacity rations for the two earthquakes are summarized in Table 7. Based upon response data gathered from instruments in the north-south direction (Table 8), it appears that amplification of motion from ground to the roof level was about 30%.

Table 7
MILPITAS TWO-STORY INDUSTRIAL BUILDING
BUILDING RESPONSE - (EAST-WEST) DIRECTION

Event	Recorded Peak Accelerations (g) Channel 13	Channel 11	D/C_a Ratio
1988 Alum Rock	0.076	0.414	2.26
1989 Loma Prieta	0.138	0.578	3.15

Channel 13 - at grade, East Wall
Channel 11 - at center of roof, East wall

Code Static Design Force C = 0.183 g
Demand D = peak acceleration, Channel 11
D/C_a = the ratio of peak diaphragm acceleration to code design acceleration

Table 8
MILPITAS TWO-STORY INDUSTRIAL BUILDING
BUILDING RESPONSE - (NORTH-SOUTH) DIRECTION

Event	Recorded Peak Accelerations (g) Channel 9	Channel 3	Channel 4	D/C_a Ratio	D/C_v Ratio
1988 Alum Rock	0.067	0.073	0.161	0.88	0.64
1989 Loma Prieta	0.090	0.113	0.311	1.70	1.16

Channel 9 - at grade, East wall
Channel 3 - at roof, East wall
Channel 4 - at center of roof, North wall

Code Static Design Force C = 0.183 g
Demand D = peak acceleration, Channel 7
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

In the Loma Prieta earthquake, the east-west PGA of 0.138g produced peak horizontal accelerations at the center of the roof (D/C_a in Table 7) over 3 times the code level static design load. As with the Hollister Warehouse, the low degree of amplification between the ground and the top of the tilt-up walls (north-south, channels 3 and 9) indicate that the response of the structure is dominated by the dynamic properties of the roof diaphragm. In the Loma Prieta earthquake, the peak acceleration at the center of the roof diaphragm was over 4Z in the east-west direction and about 3.5Z in the north-south direction. Average spectral acceleration over the length of the diaphragm was 2.4Z in the north-south direction. This behavior is in general agreement with that observed in the other buildings in this study. However, the average spectral acceleration of 2.4Z in the north-south direction compares less favorably with the previously estimated ultimate diaphragm capacity of 1.9Z. Since the ZPA at the site was substantially lower than the ultimate design load, no failure was observed to occur.

The predominant period of the roof structure was extracted using FFT's on select pairs of instrument records in both the north-south and east-west directions. A shift in the predominant frequency of the roof diaphragm was observed in the east-west direction, but no appreciable change was noted in the north-south direction. This indicates a reduction of stiffness of the diaphragm in the east-west direction, but no reduction in the north-south direction. This behavior is expected, given the amount of inelastic behavior suggested by the peak roof acceleration of 0.578g in the east-west direction observed in the Loma Prieta earthquake. The magnitude of change in the predominate frequency and diaphragm stiffness in the east-west direction is summarized in Table 9. The damping ratio for the roof diaphragm, obtained using the frequencies obtained with the FFT's and the recorded peak roof accelerations, was found to be less than 5%.

Table 9
MILPITAS TWO-STORY INDUSTRIAL BUILDING
OBSERVED CHANGE IN DIAPHRAGM STIFFNESS - EAST-WEST DIRECTION

Event	Predominant Frequency (hz)	Relative Stiffness*
1988 Alum Rock	5.1	1.00
1989 Loma Prieta	4.2	0.68

* Relative Stiffness normalized to that observed in the 1988 Alum Rock earthquake.

CONCLUSIONS

A study of the earthquake records of the Hollister warehouse for three successive events, and for the West Valley College Gym and the Milpitas Two-Story Industrial building for two earthquakes, has been completed. The last event, the Loma Prieta earthquake, produced peak accelerations in the center of the diaphragms 2 to 4 times the psuedo-static design accelerations. Based upon the available data, the following observations were made:

- o In the Loma Prieta earthquake, the roof diaphragms of the Hollister warehouse, Saratoga One-story gymnasium, and Milpitas Two-Story building showed marked decreases in stiffness, when compared to their performance in the earlier and more moderate Morgan Hill, Alum Rock, and Hollister events.

- o Despite significant observed degradation in plywood diaphragm stiffness, structural damping remains at a low level, estimated at less than 5%. This implies that the mechanisms associated with the stiffness degradation do not effectively dissipate energy from the system.

- An employee working in the Hollister Warehouse when interviewed saw no damage to the roof diaphragm.

- Degraded stiffness of the Hollister warehouse diaphragm was still several times greater than that predicted by conventional design models.

- Peak ground accelerations experienced by the Hollister warehouse, West Valley College gym, and the Milpitas Two-Story building in the Loma Prieta earthquake were approximately 65%, 89%, and 35%, respectively, 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 procedures and force levels appear to provide adequate strength and stiffness for the criteria earthquake.

- Design force levels in the current code (UBC) are controlled by response modification factors (R_W). It appears that R_W values of 6 are appropriate for this construction while higher values may be unconservative.

- 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.

o A free field ground motion sensor at each of the sites would have enabled a study of the soil-structure interaction (SSI) effects.

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