SIMULATION OF THE RECORDED RESPONSE OF UNREINFORCED (URM) INFILL BUILDINGS

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ABSTRACT

The Strong Motion Instrumentation Program of the California Department of Mines and Geology (CSMIP) has obtained records of the response of four buildings with unreinforced masonry (URM) infills. The response was to the Landers, Upland and Sierra Madre earthquakes. The objective of this research was to replicate by computer analysis the CSMIP records.

Three dimensional elastic computer models were prepared from data obtained from the original construction documents. The URM infills were modeled as diagonal braces in the frame. The stiffness properties of the infills were determined by a nonlinear finite element analysis.

INTRODUCTION

The Strong Instrumentation Program of the California Department of Mines and Geology (CSMIP) has instrumented buildings with unreinforced masonry infills. Four of these buildings were shaken by the Landers earthquake. Two of these buildings had been shaken by near small magnitude earthquakes, the 1990 Upland and the 1991 Sierra Madre earthquakes. These buildings are:

- A six-story commercial building in Pasadena (CSMIP Station No. 24541) that was constructed in 1906. It has a steel frame infilled with unreinforced brick masonry. The maximum acceleration at the basement level was 0.195g during the Sierra Madre earthquake and 0.04 g during the Landers earthquake.
- A six-story commercial building in Pomona (CSMIP Station No. 23544) that was constructed in 1923. It has a reinforced concrete frame with unreinforced brick masonry infills. The maximum acceleration at the basement level was 0.13g for the 1990 Upland earthquake and 0.07g for the 1992 Landers earthquake.
- A nine-story office building in Los Angeles (CSMIP Station No. 24579) that is L-shaped in plan. It was constructed in 1923 and has a reinforced concrete frame with unreinforced masonry infills. The maximum acceleration at the basement level was 0.05g during the Landers earthquake.
- A twelve-story commercial/office building in Los Angeles (CSMIP Station No. 24581) that was constructed in 1925. It has a concrete encased steel frame and unreinforced brick masonry infills. The maximum acceleration at the basement floor level was 0.04 g during the Landers earthquake.

STATEMENT OF THE PROBLEM

The data recorded by CSMIP was the response of buildings that have very significant vertical and plan irregularities. The lateral resistance was provided by the frames and the unreinforced masonry that is infilled into the frame. The masonry is multi-wythe brick laid in lime, Portland cement, and mortar. Cast stone, terra cotta and brick veneer wythes are a part of the masonry infills. The material properties of the masonry were estimated by comparison with masonry that had been tested by the flat jack method.

The problem is to simulate the recorded response of these buildings to the motions recorded at the lowest level. The existing building is a complex assembly of materials with nonlinear behavior. The mechanical properties of the structural materials must be estimated and effects of systems such as stairs that are continuous between floors and interior partitioning, cannot be quantified. The problem is to reduce these complex buildings to a simple linear elastic model that has similar stiffness and damping characteristics.

GOAL OF THE RESEARCH

The goal of this research is to provide information for the development of standards and ordinances for earthquake hazard reduction in this class of building. The research will provide information of how to model the frame, how to include the effect of the infill on the frame and how to account for stiffness degradation the frame-infill system. Development of a procedure for conversion of the infill, in any configuration or shape, into an equivalent diagonal brace is the goal. Without procedures for the estimation of effective stiffness of these structural systems, prescription of drift limits and calculation of drift is not possible.

RESEARCH PLAN

The existing structural systems, the mass of the building and the geometry of the system was determined by review of the existing drawings. The weight and center of gravity of each story level above the base of the building was estimated. Elevations of each column-beam line and sketches of the infilled bays were prepared. The size and location of all openings within the infilled bays were noted on the elevations.

This data was developed for each of the four buildings. Concurrently, the recorded data for each building was examined and analyzed. The time-displacement histories obtained from the CSMIP records were differenced to determine the average interstory deformation caused by the ground shaking. This interstory displacement was used in the development of the equivalent strut. The records of instruments located on a common floor level that recorded parallel motions were differenced. This was converted to rotation by dividing the difference by the distance between instruments. This data was used to isolate rotational modes and to confirm that the floor is rigid in its plane. The frequency content of instrumental records was analyzed by preparation of damped spectra and by Fourier analysis. After this raw data was accumulated and analyzed, the buildings were modeled by the SAP 90 linear-elastic three-dimensional program.

The exterior elevations showing openings in the infills of the buildings were used to determine "typical" infill patterns. The parameters for establishing "typical" infills were:

- Moment of inertia and area of the confining frame members.
- Story height and length of the infilled bay.
- Location of the openings relative to the frame and number and size of the openings.

The initial compressive modulus of elasticity, the tensile cracking stress, the strain associated with peak compressive stress and the peak compressive stress were chosen by experience and/or visual evaluation of the exposed masonry. The force-displacement relationship for each of the "typical" infill panels was calculated by use of a nonlinear finite element program developed by Robert D. Ewing, Ahmad El-Mustapha and John Kariotis (FEM Version 1.08). An effective stiffness of a pair of diagonal braces within the bay of the infilled frame was substituted for the unreinforced masonry. This effective stiffness was determined by the following process:

- For each typical infill bay configuration, the confining frame and the masonry was analyzed by the nonlinear FEM.
- The force-displacement relationship of the frame and its infill was determined by incrementally displacing the assembly. This analysis determines the stiffness degradation of the system due to cracking and strain in the frame and infill.
- The confining frame was analyzed without any infill.
- The force-displacement relationships of the infilled frame and the frame alone was differenced.
- The area and modulus of elasticity of the equivalent diagonal braces was calculated to provide an effective system stiffness at the story displacement as determined by evaluation of the CSMIP displacement data.

The process of obtaining a best-fit computer replication was an iterative process. The viscous damping used in the linear-elastic model was established using the best available data. The computed periods of the linear-elastic model were compared to estimated periods extracted from the CSMIP data. Rotational periods for the SAP model and for the CSMIP data were compared. The parameters that were modified to improve the fit were the effective stiffness of the frame members, the effective stiffness of the diagonal struts that represent the infills and the percent of critical damping. These parameters are variables as the materials properties of the concrete frames, the stiffness of the beam-column connections of the steel frames and the material properties of infills are estimated, not quantified by physical testing.

ANALYSIS

The data available to the researchers consisted of the building plans, plans and elevations showing the location of all instruments, and the processed records of each of the instruments. There were conflicts between the existing construction of CSMIP Station No. 23544 in Pomona as shown on the original construction documents and observations of the exterior walls. The light well on the west begins at the second floor level rather than at the mezzanine level as shown on the drawings. There is a conflict as to the materials of the frame that extends from the main floor to the second floor level at the south end. The original drawings show that these columns are reinforced concrete. A supplemental drawing shows a structural steel girder at the second floor level supported by steel columns encased in concrete. Additions have been made to CSMIP Station No. 24541 in Pasadena. These additions tie the two wings of the U-shape together at all level.

The CSMIP Station Nos. 24579 and 23544 have reinforced concrete frames. Station No. 23455 has a severe plan irregularity below the second floor level and a lesser degree of plan irregularity from the second floor to the roof level. A mass irregularity is at the roof level. The lateral resistance at the east and south is provided by the concrete frame and minimal infills. The percentage of the gross moment of inertia of the columns at this level that should be used as effective stiffness was investigated. Station No. 24579 is an L-shaped building that has a single story garage structure constructed in the portion of the property not occupied by the nine-story building. Reinforced concrete walls separate the occupancies. These reinforced concrete infills were analyzed by methods identical to those used for unreinforced masonry infills. The effect of changing the modulus of elasticity of the concrete frame independent from changing the effective area of the masonry strut was investigated.

The CSMIP Station No. 24541 and 24581 have structural steel frames and multi-wythe brick masonry infills. Station No. 24541 has a severe plan and stiffness irregularity below the second floor. The south and east street fronts have only frames to resist lateral displacements. The west wall below the second floor is infilled with a small window in each bay. The north end is highly perforated with openings. Above the second floor the infilled walls at the perimeter of the light well add stiffness, especially in the north-south direction. The exterior walls have more symmetry in plan above the second floor except that the east and south walls are thicker. This moves the probable rotational center of the building above the second floor in the opposite direction from the probable location below the second floor. Station No. 24581 is nearly symmetrical in plan in the north-south direction. A plan irregularity exists in the east-west direction. The floor beams are encased in concrete. The columns of both buildings are encased in brick or clay tile. The floor beams in Station No. 24541 support a clay tile arch system topped with an unreinforced concrete slab.

The infill within the steel or reinforced concrete frame resists shear distortion of the frame. Experimental testing of solid infills have shown that the behavior of the infill can be represented by a compression-only strut extending from the upper to lower corners of the bay of the frame. Experimental testing of infills with openings has shown that the presence of openings changes the effective stiffness of the infill. The effect of the infill with openings was represented by pinned-end struts placed diagonally in the frame for all opening configurations. The area in this diagonal was determined by the nonlinear finite element analysis. The nonlinear finite element model must be programmed with materials behavior and this materials behavior should be determined by physical testing. The materials properties needed for the nonlinear analysis of an infill are:

- Tensile cracking strain. This property is assumed to be isotropic.
- Initial modulus of compression.
- Strain at peak compressive stress. This should be the strain caused by cyclic loading in compression.
- Peak compressive stress.
- Mechanical and physical properties of the confining frame if structural steel.
- Properties of the concrete such as described for the masonry if the confining frame is reinforced concrete.
- Assumption of a tension stiffening model for the reinforced concrete elements.

The choice of element size used in the nonlinear analysis is critical. Small elements must be used in critical stress and strain zones adjacent to the confining frame. The reinforcement in a reinforced concrete frame may be a smeared model, that is the quantity of reinforcement is uniformly distributed over the gross area. The steel member may be represented by flange and web or by an appropriately sized rectangle. The nonlinear analysis of infilled frames is a two-part analysis. The frame is first analyzed without infills. The second analysis is of the frame and the masonry infill. The force-displacement plot of the monotonic loading is differenced and used as the effective stiffness of the diagonal members that represent the infill. In these analyses, the relative displacement at each story level has been estimated by use of the CSMIP displacement data. This story displacement is used in conjunction with the FEM analysis to determine a secant stiffness of the system. This stiffness is assigned to a pair of struts of elastic material that are identical to that material used for the beams and columns. These analyses initially did not analyze the steel frames without infill. The area of the diagonal members was determined directly from the nonlinear analysis of the masonry and the confining steel frame. However, the dynamic analysis of CSMIP Station No. 24581 found that the stiffness of the steel frame must be deducted from the results of the nonlinear FEM analysis.

All beams that frame into the building columns were included in the model. All beam-column joints were considered fixed. This assumption was used for the structural steel systems regardless of the detailed connection. The analyses of CSMIP Station No. 24581 found that the stiffness of the steel beams in the frame must be adjusted to less than 100% to account for the flexibility of the beam-column connection. The diagonal members were given pinned-ends to eliminate any contribution to flexural stiffness. Eighty percent of the stiffness determined from the FEM analysis was used as the initial elastic stiffness. This was chosen to estimate the stiffness on reloading to a stabilized force-displacement envelope. The base of the building was taken as the top of the first floor. This assumption was made as reinforced concrete perimeter walls are below this level. All columns were considered fixed at this level. This assumption and the assumption of a fixed base building, that is no rotation of the building on the supporting soils, increased the effective stiffness of the computer model of the building over that of the existing building. There are three critical unknowns as to the dynamic response of these buildings. These are:

- Translational stiffness on the x and y axes.
- Rotational stiffness at levels of plan irregularity.
- Damping that occurred during the recorded time.

Matching of the CSMIP time-displacement records would require that all three of these critical unknowns be calculable. The translation and torsional stiffness was calculated for the computer model using "typical" infilled bays. The damping force used in the linear-elastic model was a viscous damper that functions full time during the time-history analysis. The percentage of critical damping is calculated for the structural stiffness of each mode. The dynamic damping force is related to the response velocity. The actual damping is hysteretic and does not have a damping force acting opposite to the loading force on a loading cycle. The real damping is due to nonlinear cyclic distortion of the masonry infill. The damping ratio used in these analyses was limited to five percent of critical damping.

The data recorded in the building was the response of a building founded on soils at a story height below the base elevation that was used in the linear-elastic model. The added story height and flexibility of the soils increased the recorded building period over that calculated by the linear-elastic model. It is probable that the top displacement may be unchanged by the increase in period. The basement spectra at the Pasadena and Pomona sites, as shown on tripartite plots, has a near constant displacement (SD) branch for periods greater than about 1.5 seconds . The frequency of the rotational modes should be less affected by the added story height and soil flexibility than translational modes. All parameters that affect displacement and modal frequencies were subject to modification. However, the frequencies calculated by the SAP model should be less than that deduced from the CSMIP data.

RESULTS OF THE ELASTIC ANALYSES
CSMIP STATION NO. 23544, LANDERS EARTHQUAKE

A damping ratio of 2% of critical was used. The effective stiffness of the diagonal members was 100% of that calculated by the FEM analysis. The effective stiffness of the beams was taken as 70% of that calculated using the concrete section. Sixty percent of the stiffness of the concrete columns above the second floor and 35% of the stiffness of the concrete columns below the second floor was used to estimate the reduction in stiffness due to cracking of the concrete. A comparison of the relative displacements recorded and calculated in given in Table 4.1. The values from the CSMIP data and calculated by SAP have very good correlation in peak value. The comparison is plotted in time in Figures 1 and 2. The channels that recorded translational and rotational modes show that the SAP model over predicts the displacement in the beginning of the shaking but has better correlation from 25 seconds to 45 seconds.

CSMIP STATION NO. 23544, UPLAND EARTHQUAKE

The Upland earthquake preceded the Landers earthquake. The ground motion recorded at the base of the building during the Upland earthquake was used to excite the SAP model correlated to the Landers data. The north wall in the mezzanine floor level was damaged by the Upland earthquake. A comparison of the relative displacements recorded and calculated is given in Table 4-2. A better correlation is made with peak values than with the plots of displacement-time shown in Figures 3 and 4.

CSMIP STATION NO. 24541, LANDERS EQ.

This building has one significant translational line of resistance below the second floor. All modes with significant mass coupling are torsional. The torsional stiffness above the second floor greatly exceeds the torsional stiffness below the second floor. The stiffness of the infill panels was taken directly from the FEM analyses. No reduction in stiffness of the infill due to cyclic loading was taken. The stiffness of the structural steel frame was not deducted from the infilled system stiffness. The material properties used to model the masonry were identical to that used for the other three buildings. It is possible that the estimated materials properties exceeds those that would be determined by testing. The SAP model generally over estimated the dynamic displacement at the second floor level and under estimated the displacements at the roof. Five percent damping was used for all nodes. Six modes of response were used in the SAP model. The relative displacements shown in Table No. 4-3 have a reasonable agreement. The displacement-time record shown in Figures 5 and 6 are out of phase. The difference appears to be related to the frequency of rotational modes.

CSMIP STATION NO. 24541, SIERRA MADRE EQ.

The comparison of measured and calculated displacements is shown in Table No. 4-4. The stiffness model used for these predictions is the same as used for predicting the displacements caused by the Landers earthquake. The quality of the predictions when plotted in time vs. displacement, Figure 7 and 8, are better in phase relationship.

CSMIP STATION NO. 24579, LANDERS EQ.

The comparison of measured and calculated displacements is shown in Table No. 5. The time-history analyses used six modes with 3% of critical damping for the first 3 modes and 5% of critical damping for the nest 3 modes. The stiffness used for diagonals was 70% of the FEM results and the stiffness of the concrete frame was taken as 85% of the gross section stiffness. The plots of the time-displacements shown in Figures 9 and 10 show that the torsional response recorded in Figure 10 is out-of-phase with the calculated response at the roof level. The torsional stiffness is provided by tall slender frames at the north and west. The response shown in Figure 9 has less coupling with torsion.

CSMIP STATION NO. 24581, LANDERS EQ.

The comparison of measured and calculated displacements is shown in Table 6. The time-history analyses used six modes with 3% of critical damping for the first 3 modes and 5% for the remainder. The preliminary analyses found that the stiffness of the steel frame must be deducted from the stiffness of the infilled system. Figure 11 shows the response of the building in the longitudinal direction. The recorded response shows that the higher modes are not included in the SAP model. The response at the 12th floor level is a better correlation of recorded an calculated response. Figure No. 12 is the response of the building at an end wall in the transverse direction. The comparison at the 12th floor level show the difference in frequency between the building and the SAP model.

CONCLUSION

The elastic three-dimensional analyses successfully predicted the maximum values os the relative displacement of four buildings with URM infills. The comparative time-histories shows that the technical limitations of the elastic model to replicate nonlinear behavior limits the matching of displacement records to a small segment of time. Variables used to improve the fit of the calculated data to the recorded data included damping, reduction of the stiffness of concrete frames from uncracked stiffness, reduction of the stiffness of the equivalent diagonal braces from that determined by the nonlinear finite element analysis and reduction of the stiffness of beams in a steel frame due to flexibility of the beam-column connection. There is technical substantiation for the values used in these studies. Additional research is needed to establish most probable values of element stiffness but the methodology used in this research has been shown to be adequate.

TABLE NO. 1
COMPARISON OF DISPLACEMENTS FOR STATION NO. 23544, LANDERS EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	Mid 2nd Fl.	5	0.36	0.36
E-W	S. 2nd Fl.	9	1.33	1.38
E-W	N. 2nd Fl.	10	0.76	0.68
N-S	Mid Roof	2	0.59	0.61
E-W	S. Roof	7	1.86	2.00
E-W	N. Roof	8	1.47	1.32
N-S	W. Roof	3	0.37	0.42
N-S	W. Roof	4	0.36	0.42

TABLE NO. 2
COMPARISON OF DISPLACEMENTS FOR STATION NO. 23544, UPLAND EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	Mid 2nd Fl.	5	0.63	0.56
E-W	S. 2nd Fl.	9	1.28	1.52
E-W	N. 2nd Fl.	10	0.95	1.17
N-S	Mid Roof	2	1.09	1.05
E-W	S. Roof	7	1.83	2.05
E-W	N. Roof	8	1.90	2.08
N-S	W. Roof	3	0.68	0.71
N-S	W. Roof	4	0.70	0.71

TABLE NO. 3
COMPARISON OF DISPLACEMENTS FOR STATION NO. 24541, LANDERS EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	W. 2nd Fl.	1	0.24	0.26
N-S	E. 2nd Fl.	2	0.90	1.34
E-W	N. 2nd Fl.	11	0.80	1.05
E-W	S. 2nd Fl.	12	0.75	0.85
N-S	W. Roof	3	1.40	1.06
N-S	E. Roof	4	2.00	1.97
E-W	N.W. Roof	5	2.50	2.00
E-W	N.E. Roof	6	2.50	2.00
E-W	Mid Roof	7	2.02	1.57
E-W	S. Roof	8	1.35	1.35

TABLE NO. 4
COMPARISON OF DISPLACEMENTS FOR STATION NO. 24541, SIERRA MADRE EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	W. 2nd Fl.	1	0.20	0.20
N-S	E. 2nd Fl.	2	0.90	0.86
E-W	N. 2nd Fl.	4	0.50	0.75
E-W	S. 2nd Fl.	12	0.50	0.60
N-S	W. Roof	3	1.60	0.84
N-S	E. Roof	4	1.60	1.15
E-W	N.W. Roof	5	1.50	1.52
E-W	N.E. Roof	6	1.50	1.52
E-W	Mid Roof	7	0.98	1.02
E-W	S. Roof	8	0.80	0.91

TABLE NO. 5
COMPARISON OF DISPLACEMENTS FOR STATION NO. 24579, LANDERS EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	S.W. 2nd Fl.	8	0.18	0.16
E-W	S.W. 2nd Fl.	9	0.16	0.17
E-W	N. 5th Fl.	10	1.15	1.13
E-W	S.W. 5th Fl.	11	0.60	0.54
N-S	S.W. 5th Fl.	12	0.55	0.49
N-S	W. 5th Fl.	13	0.91	0.94
E-W	N. Roof	14	2.22	2.06
E-W	S.W. Roof	16	0.96	1.01
N-S	S.W. Roof	17	0.87	0.88
N-S	W. Roof	18	1.56	1.77

TABLE NO. 6
COMPARISON OF DISPLACEMENTS FOR STATION NO. 24581, LANDERS EQ.

DIRECTION	FLOOR & LOCATION	CHANNEL	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	W. Mezzanine	5	0.37	0.32
N-S	Mid Mezzanine	6	0.19	0.25
N-S	E. Mezzanine	7	0.38	0.35
E-W	Mid Mezzanine	8	0.11	0.11
N-S	Mid 4th Fl.	9	0.57	0.80
E-W	Mid 4th Fl.	10	0.60	0.42
N-S	W. 12th Fl.	11	2.56	2.25
N-S	Mid 12th Fl.	12	2.28	2.31
N-S	E. 12th Fl.	13	2.10	2.12
E-W	Mid 12th Fl.	14	2.48	2.57
N-S	Mid Roof	15	2.51	2.38
E-W	Mid Roof	16	2.57	2.63

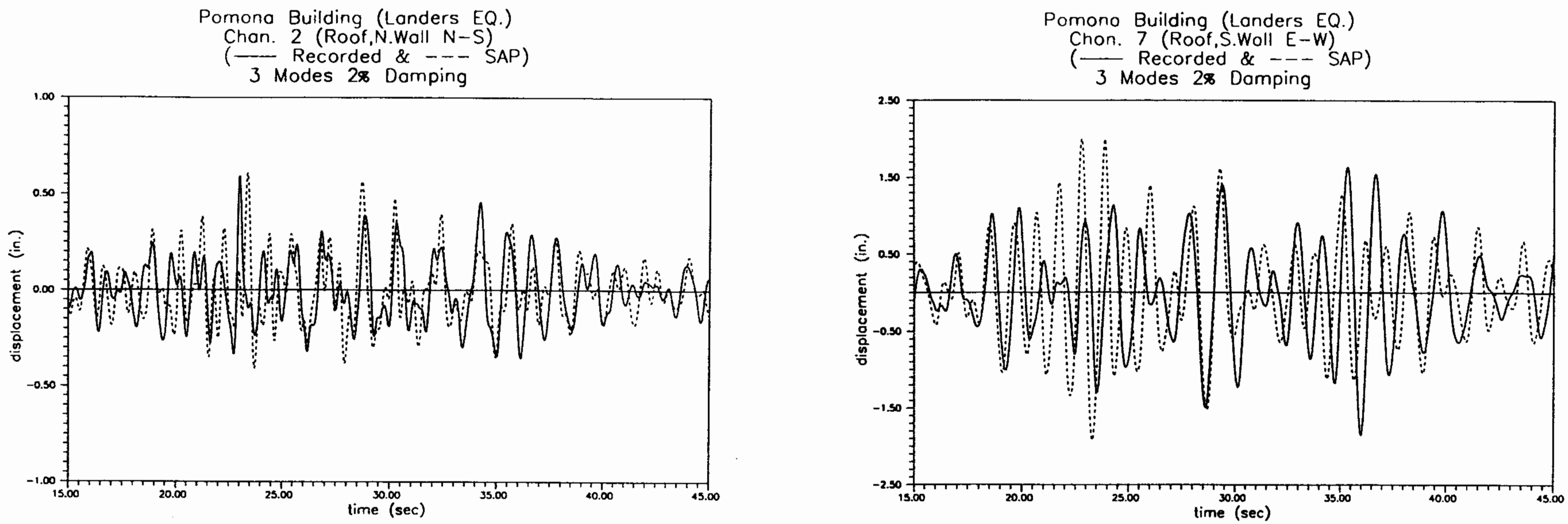


FIGURE NO. 1 - RESPONSE OF CSMIP STATION NO. 23544 AT ROOF, LANDERS EQ.

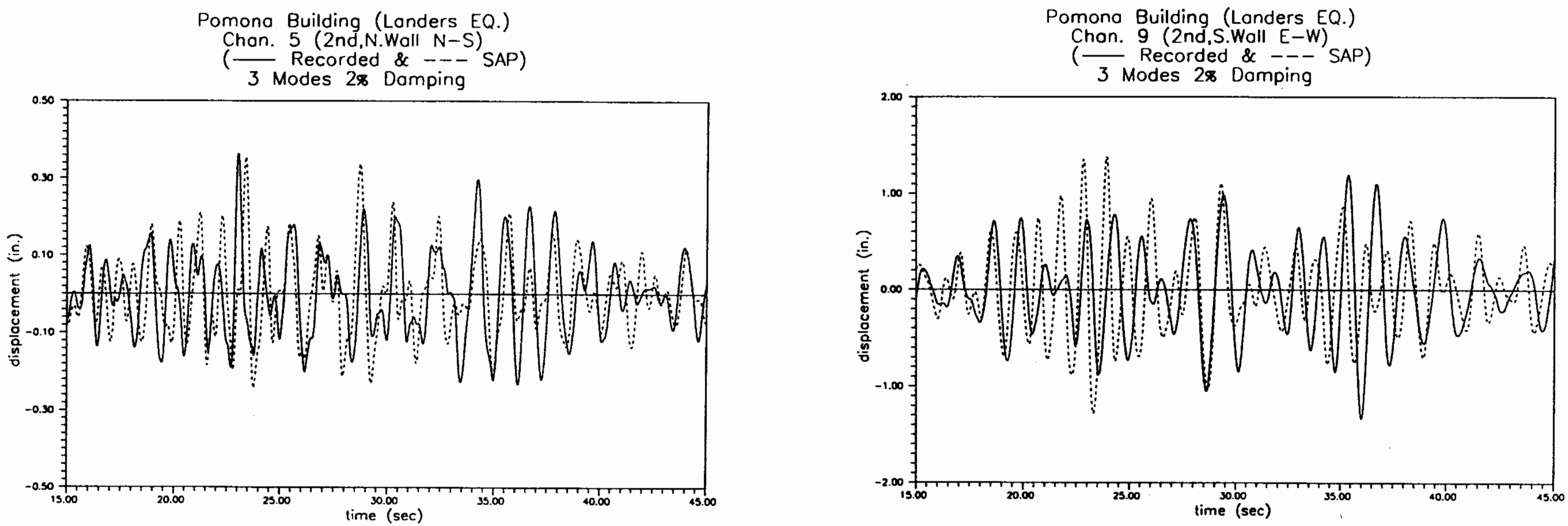


FIGURE NO. 2 - RESPONSE OF CSMIP STATION NO. 23544 AT 2nd FLOOR, LANDERS EQ.

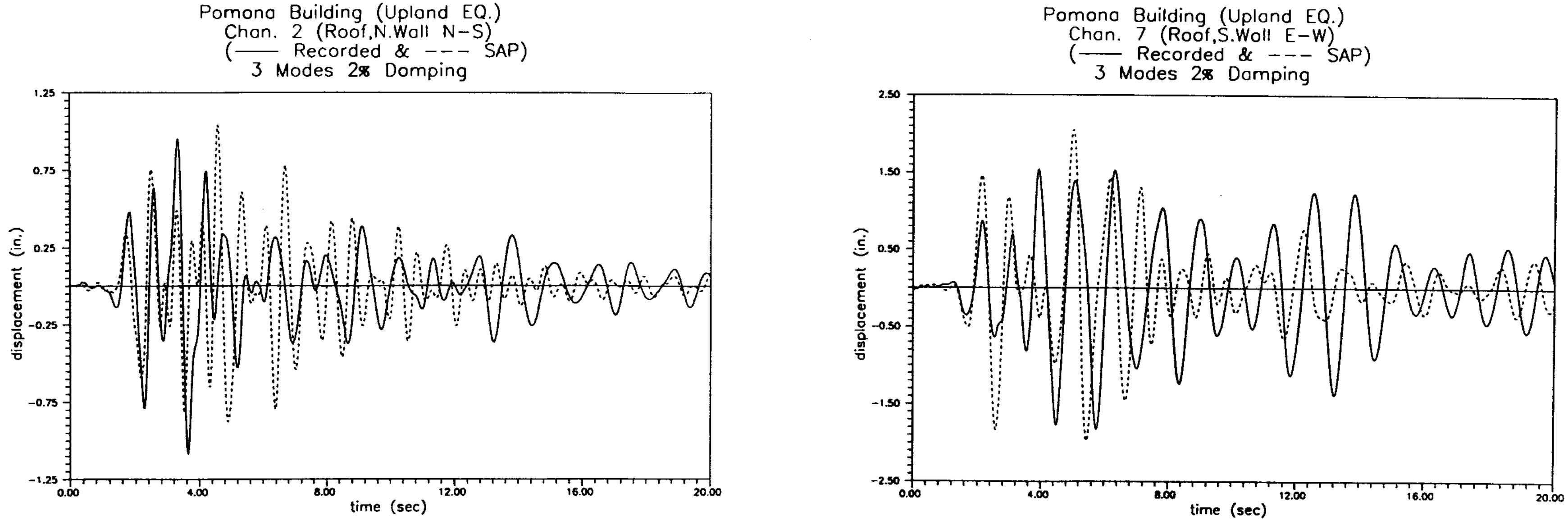


FIGURE NO. 3 - RESPONSE OF CSMIP STATION NO. 23544 AT ROOF, UPLAND EQ.

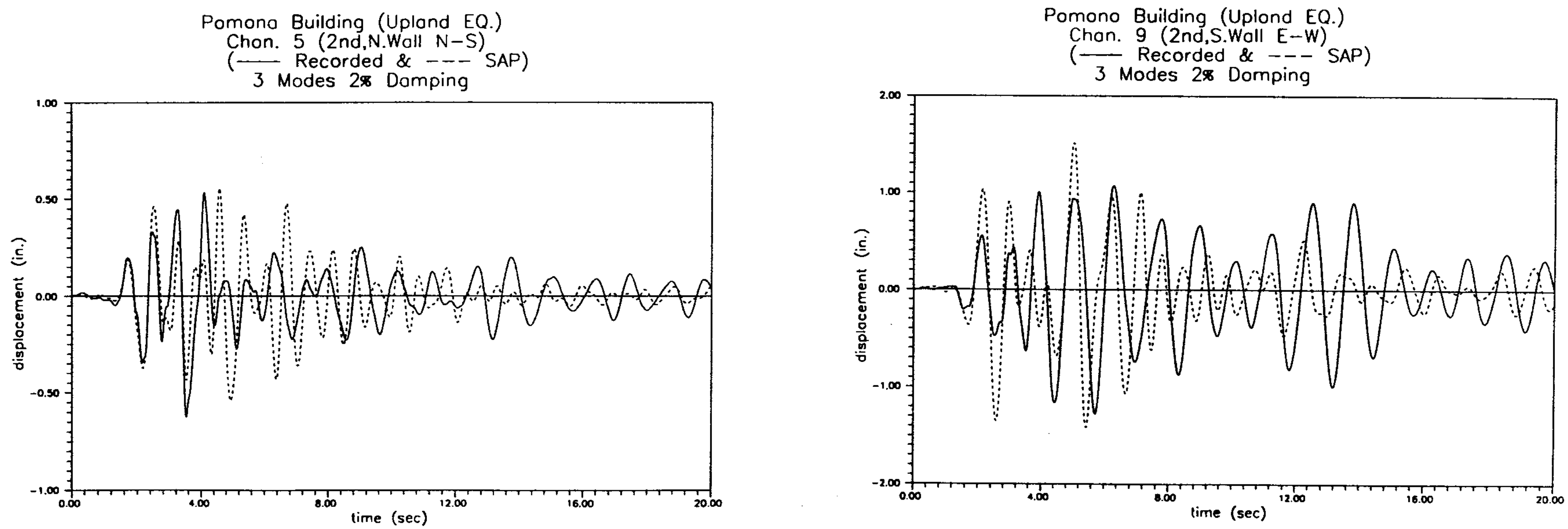
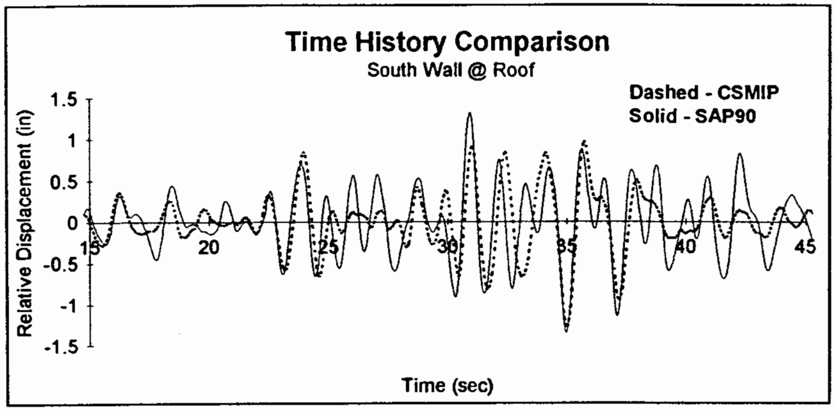


FIGURE NO. 4 - RESPONSE OF CSMIP STATION NO. 23544 AT 2nd FLOOR, UPLAND EQ.



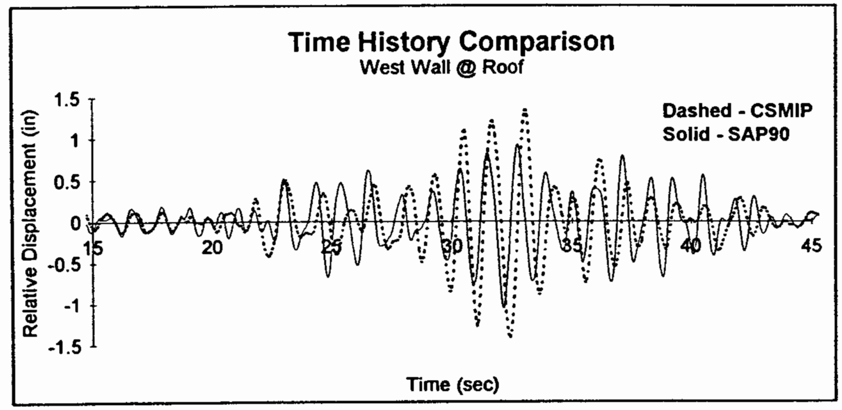
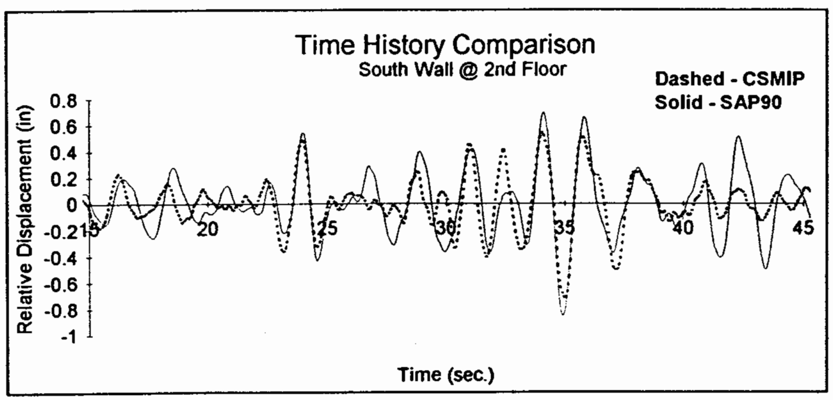


FIGURE NO. 5 - RESPONSE OF CSMIP STATION NO. 24541 AT ROOF, LANDERS EQ.



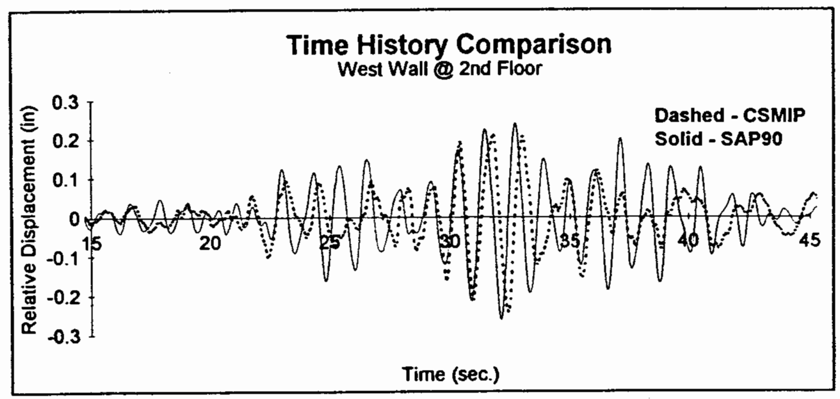
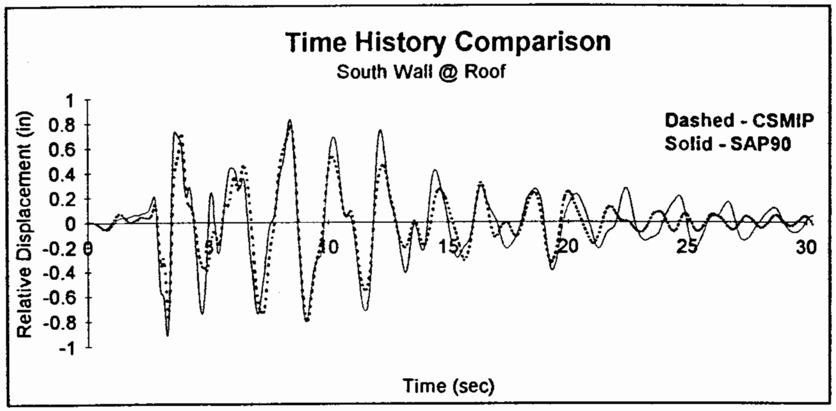


FIGURE NO. 6 - RESPONSE OF CSMIP STATION NO. 24541 AT 2nd FLOOR, LANDERS EQ.



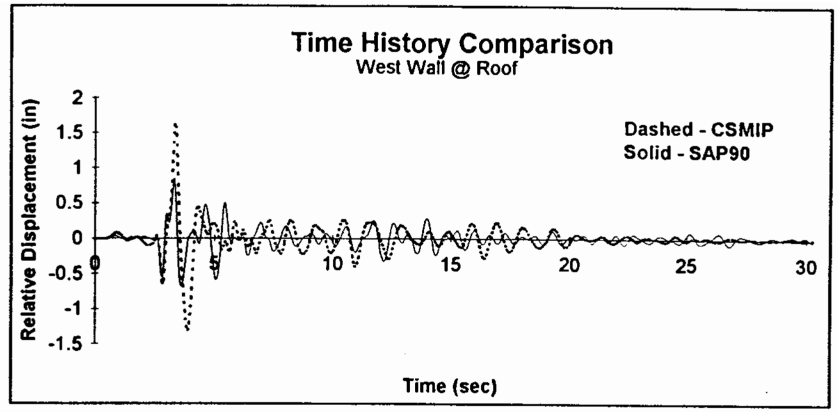
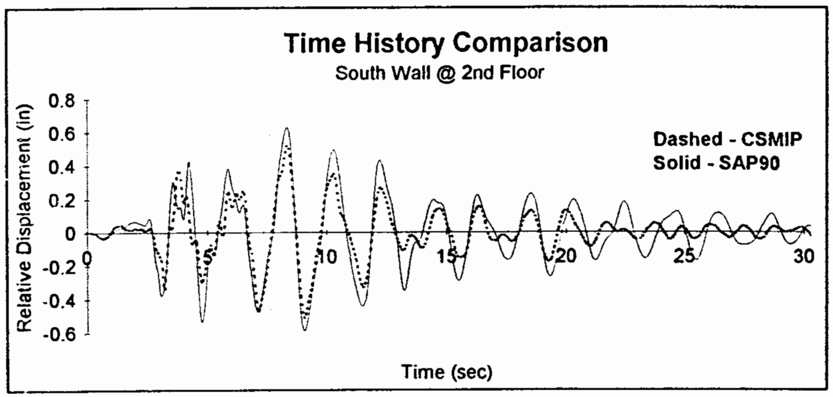


FIGURE NO. 7 - RESPONSE OF CSMIP STATION NO. 24541 AT ROOF, SIERRA MADRE EQ.



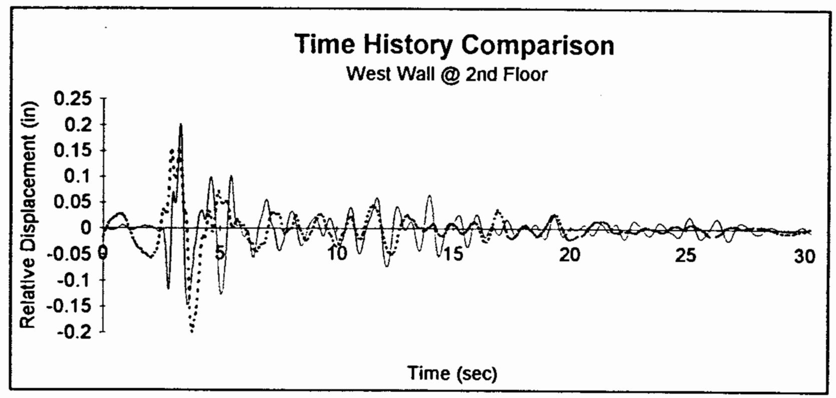


FIGURE NO. 8 - RESPONSE OF CSMIP STATION NO. 24541 AT 2nd FLOOR, SIERRA MADRE EQ.

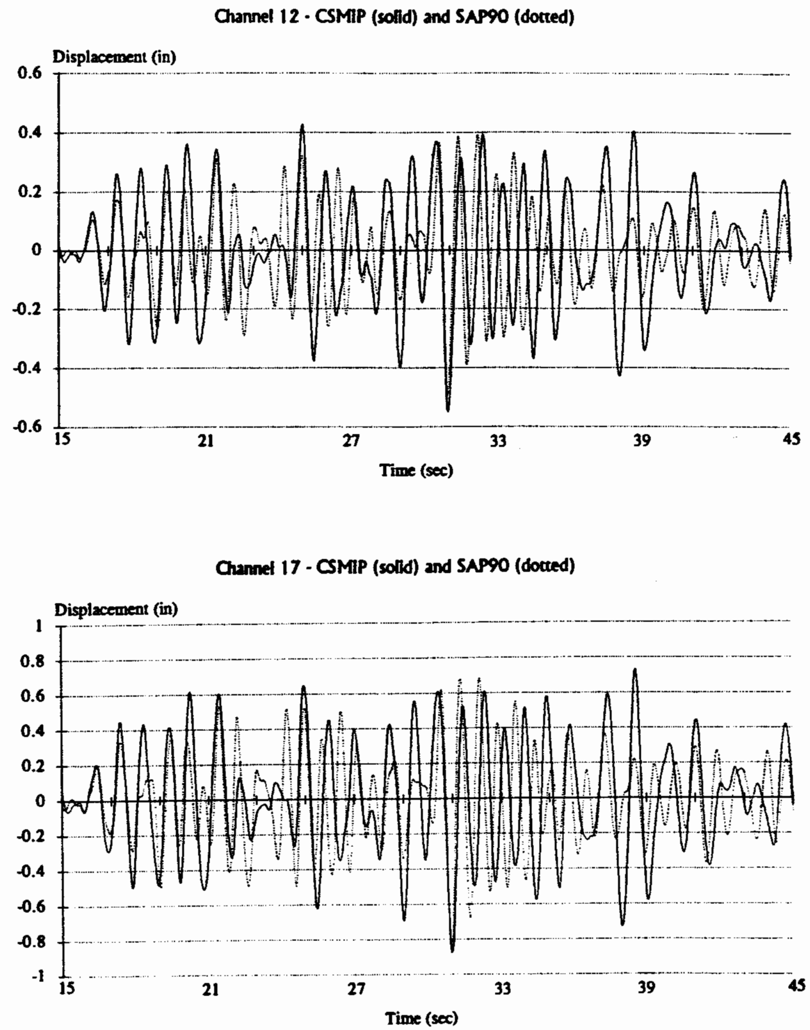


FIGURE NO. 9 - RESPONSE OF CSMIP STATION NO. 24579 AT 5th FLOOR (12) AND ROOF (17), LANDERS EQ.

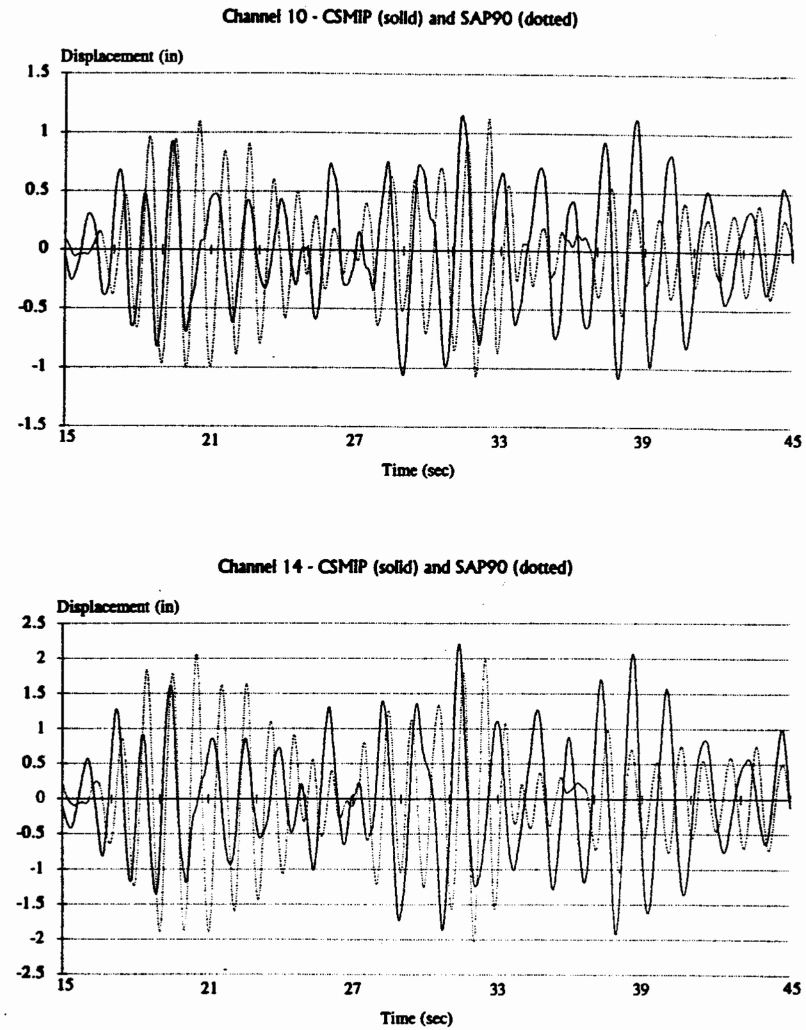


FIGURE NO. 10 - RESPONSE OF CSMIP STATION NO. 24579 AT 5th FLOOR (10) AND ROOF (14), LANDERS EQ.

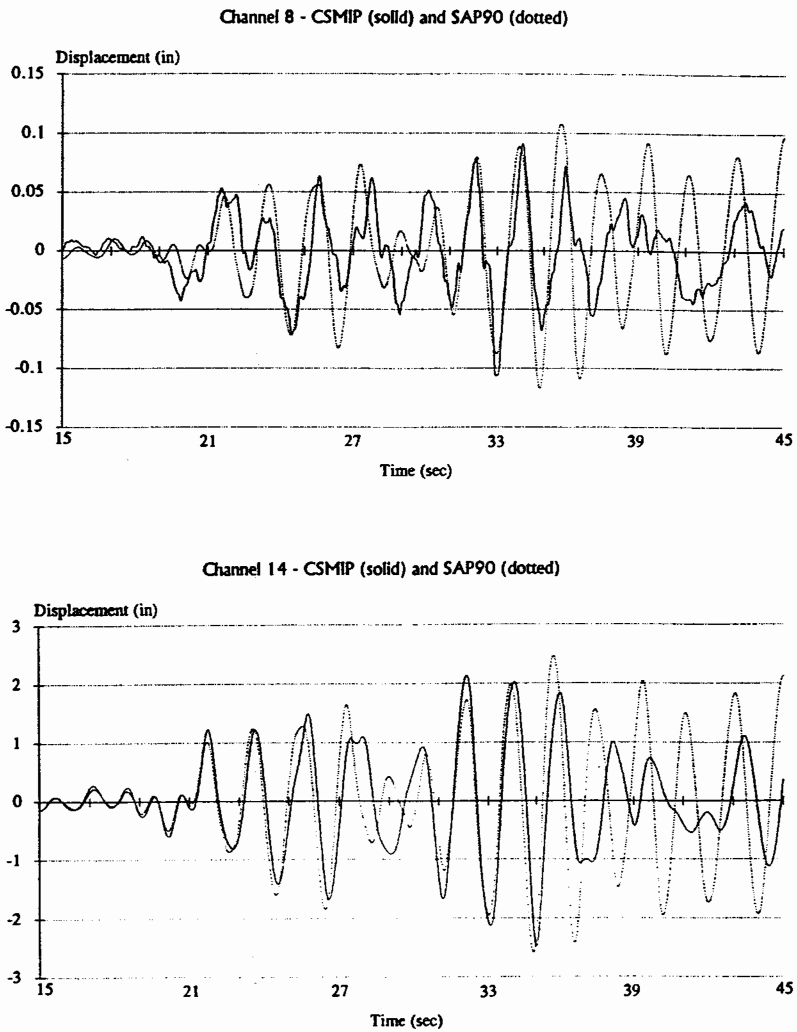


FIGURE NO. 11 - RESPONSE OF CSMIP STATION NO. 24581 AT 2nd FLOOR (8) AND 12th FLOOR (14), LANDERS EQ.

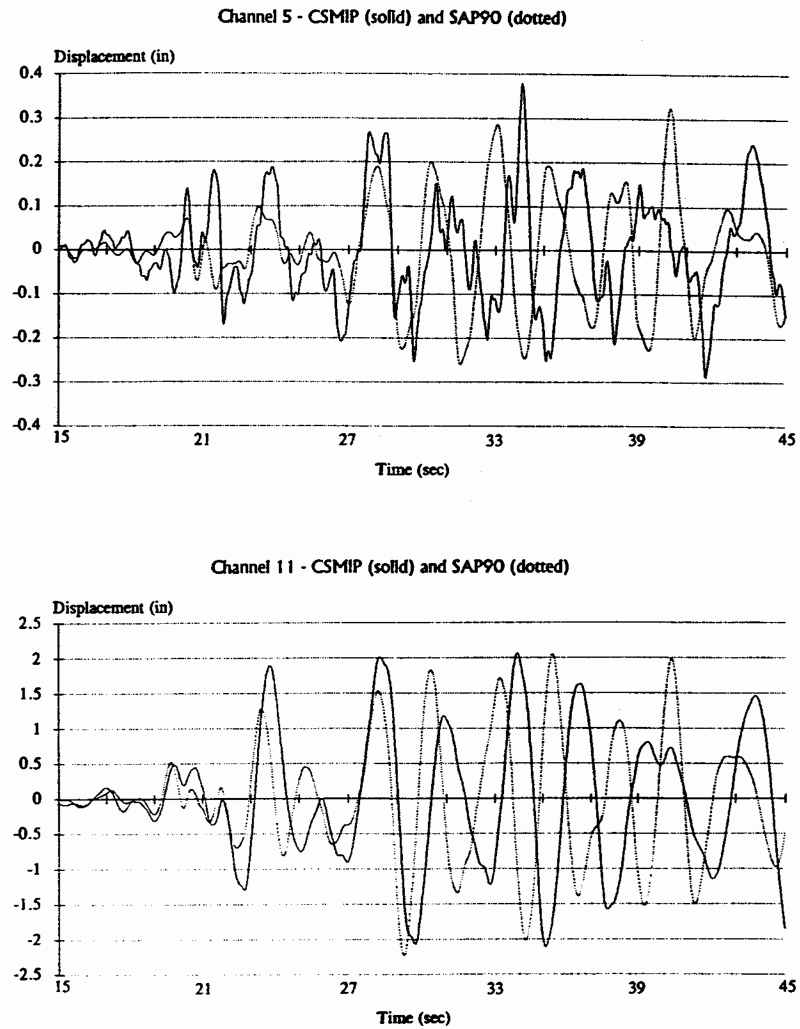


FIGURE NO. 12 - RESPONSE OF CSMIP STATION NO. 24581 AT 2nd FLOOR (5) AND 12th FLOOR (11), LANDERS EQ.