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A TIMBER DIAPHRAGM : ANALYSIS AND EARTHQUAKE RECORDS

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SUMMARY

In this paper are analyzed the records obtained from a building instrumented with strong motion accelerographs during a M_s=6.1 earthquake. The results of the direct analysis of the records (natural frequencies and relative displacements) are matched to those from mathematical analysis through a finite element model of the structure and from system identification technique. Comparison with current design formulas are performed both for the natural periods and for the maximum displacements of the building.

INTRODUCTION

A complete set of acceleration records were obtained from the West Valley College gymnasium in Saratoga, California, during the M_s=6.1 Morgan Hill earthquake of 24 April 1984. The gymnasium was instrumented by the California division of Mines and Geology (CDMG) as part of its Strong Motion Instrumentation Program (SMIP). In a previous paper the performance of the roof diaphragm was studied (Ref. 1) using the raw and processed records made public by CDMG (Refs. 2). The results of that paper were based purely on studying the relative displacements and Fourier spectra of the record obtained at the roof diaphragm level.

The roof diaphragm of the 34.14 m x 43.90 rectangular,symmetric gymnasium consists of 0.95 cm plywood over tongue-and-groove sheating attached to steel trusses supported by reinforced concrete columns and walls. A three-dimensional schematic view of the gymnasium and the locations of the eleven channels of force-balance accelerometers (connected to a central recording unit) and associated directions are provided in Fig. 1. There were no free-field instrument at the site.

Recorded acceleration time histories are shown in Fig. 2. Peak accelerations experienced at key locations of the gymnasium are summarized in Table 1.

TABLE 1

Summary of Peak accelerations

	Peak Accelerations (g)		
Location	Vertical	N-S	E-W
Roof Edges	-	0.13, 0.15	0.06, 0.07
Roof Center	-	0.42	0.20
Ground	-	0.10	0.04

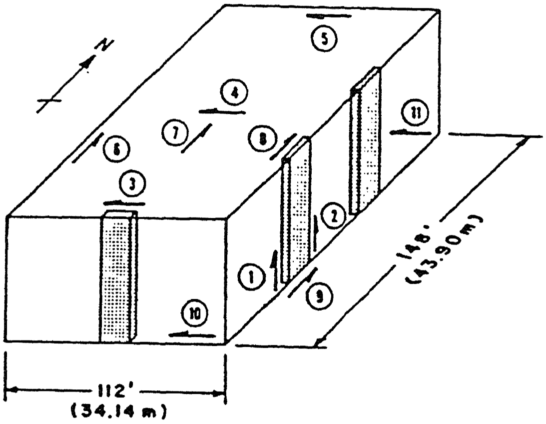

Fig. 1

The frequencies of the N-S and E-W directions are identified as 3.8 and 3.9 Hz from the Fourier amplitude spectra of the recorded accelerations.

The displacements integrated from the recorded accelerations show the general similarity at long periods between the motions of the roof and the floor in both the main directions. In order to investigate more closely the behavior of the roof diaphragm the frequencies below 1.0 Hz have been removed. The peak displacements are provided in Table 2. By averaging several successive cycles of the displacement plots at the center of the roof, the dominating frequencies were evaluated as 4.0 and 4.2 Hz, respectivelY.

TABLE 2

Peak Displacements of the Diaphragm
(From Recorded Motions)

	Peak Displacements (cm)	
Location	N-S	E-W
Roof Edges	0.31, 0.36	0.12
Roof Center	0.78	0.39
Ground	0.28	0.09
Relative Displacement (Center to Edges)	0.47	0.28

MATHEMATICAL MODEL AND ANALYSES

A three-dimensional finite-element model was developed representing only one quarter of the symmetric gymnasium. The simplified model utilizes only beam elements for the shear walls and columns and in-plane stress elements for the diaphragm. The properties of the diaphragm were selected from standard references (Refs. 4,5,6).

In essence, the analyses performed were two-dimensional in that only single horizontal component input was applied at a time to the fixed base nodes of the model, restraining the tranlational degree of freedom in transverse direction for the nodes on the line of symmetry in the direction of application of the input acceleration. In Fig. 3 is shown the N-S case.

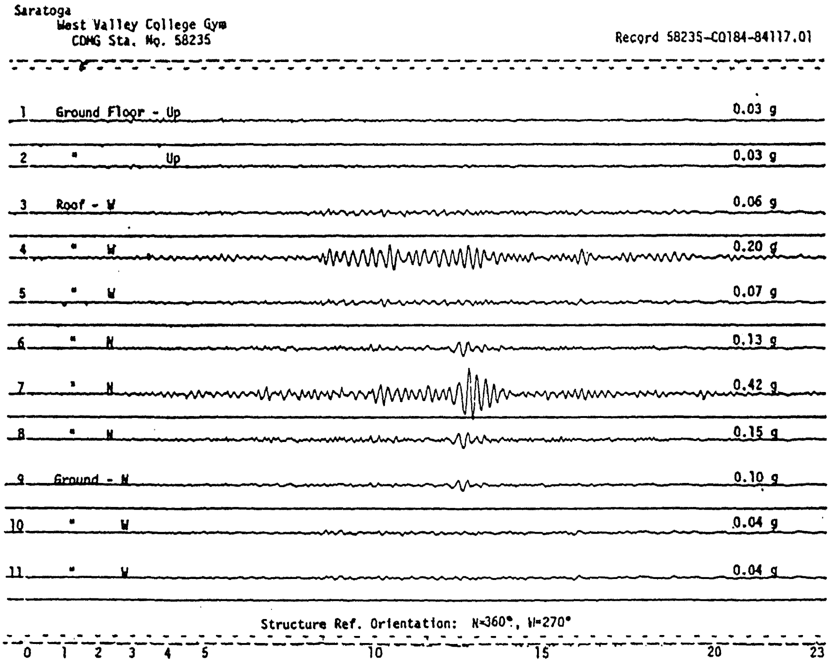

Fig. 2

The frequencies obtained from modal analyses are 4.0 Hz and 3.9 Hz in the N-S and E-W directions. Three time history analyses were performed for each of the thre critical damping percentages-10%, 8% and 6%. The best agreement between the displacements of the history analyses and those obtained from the integration of the recorded motions of the roof diaphragm corresponds to 6% damping. The results of the three history analyses are provided in Table 3.

TABLE 3
Peak Displacements from Time History Analyses
(cm)

Location	N-S x-Direction Damping (%)			E-W y-Direction Damping (%)		
(Node, Description)	6	8	10	6	8	10
Node 27 (Center-Diaphragm)	0.7	0.64	0.59	0.35	0.31	0.24
Node 24 (Edge-Diaphragm)	0.16	0.15	0.15			
Node 11 (Edge-Diaphragm)				0.01	0.01	0.01

DISPLACEMENTS AND RELATIVE DISPLACEMENTS

In Fig.4 the displacement time-histories of the center of the diaphragm from both analysis and double integration of recorded motions are provided for the N-S direction. Besides the peak values of Table 5, the comparison of the shapes of these time-histories sems satisfactory.

Fourier spectra using the calculated center-roof displacements give 4.0 Hz as dominant frequency for both directions.

The calculated relative displacement between the center and the edge of the diaphragm for the N-S direction shows good agreement with that from records. The same does not happen for the E-W direction, possibly due to the not exact coincidence of the physical location of the sensors with the geometric representation in the mathematical model. A better agreement is obtained considering the relative displacement of the center of the roof with respect to node 15. This strengthens the hypothesis that the main response of the building to the earthquake is by the roof diaphragm.

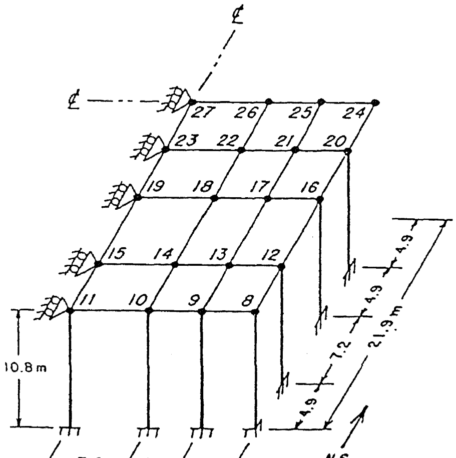

Fig. 3

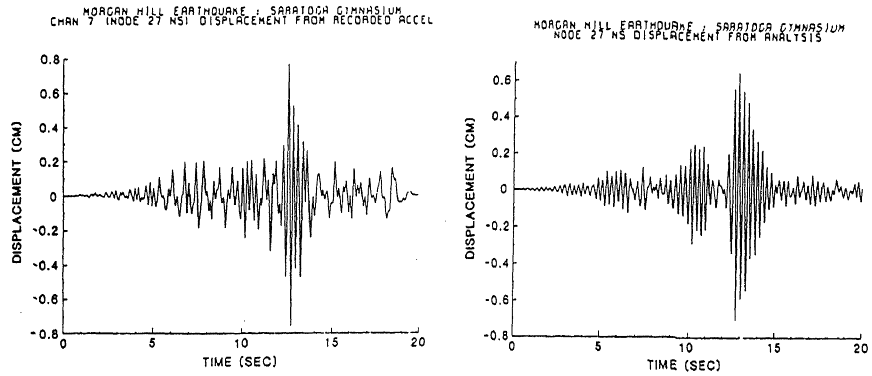

Fig. 4

SYSTEM IDENTIFICATION

In order to further substantiate the findings of the analyses, a system identification routine was used (Ref. 6). The most interesting result was the closeness of the damping values (5.4% and 6.2% in the two directions) determined by this technique and the 6% damping used in the time-history analyses. The results of the system identification approach are summarized in Table 4.

TABLE 4
Dynamic Characteristics Determined by System Identification Technique

Dynamic Characteristic	N-S	E-W
Frequency (Hz)	4.1	3.7
Damping (%)	5.4	6.2

CODE CONSIDERATIONS

Ultimately, since the study of recorded motions is intended to compare code design and analysis procedures with those of recorded responses, we make the following comparisons:

1. The code formulas used to calculate the natural period of vibration of structures $T= 0.05*h/D^{1/2}$ yelds 0.144 and 0.165 seconds respectively for the two directions. These are lower than those derived from recorded motions (approx. 0.25 s).

2. Using code base shear formulas and applying the base shear force statically to the center of the diaphragm (with W=216 tons, Z=1.0, K=1.0, C=0.12, S=1.5, CS=0.14 (maximum according to the code) and therefor V=30.24 tons) the calculated static displacements are 0.45 cm and 1.68 cm for the N-S and E-W directions. These do not compare well with the peak displacements calculated from recorded motions (0.78 and 0.39 cm) .

3. On the other hand, using ATC-7 (1981) recommended equations, a total displacement (including bending and shear deformations) of the diaphragm only of 1.79 cm is calculated corresponding to equivalent static force using the peak diaphragm acceleration.

CONCLUSIONS

The study presented illustrates some unique aspects of diaphragm behavior as well as those those recorded response motions during the 24 April 1984 Morgan Hill Earthquake (M_s=6.1):

1. The recorded peak accelerations in the N-S and E-W directions of the ground floor as well as the diaphragm differ considerably. The recorded peak ground floor acceleration in the N-S direction (perpendicular to the narrower dimension) is 0.10 g while in the E-W direction it is 0.04 g. Consequently the displacements in the stronger (N-S) direction are larger than those in the weaker (E-W) direction. Such differences in ground level accelerations are not accounted for in design considerations.

2. A very simple finite element model is successfull in matching the fundamental frequencies and the displacements (damping 6%) from recorded motions, within acceptable tolerances.

5. The calculated or the recorded displacements do not compare well with those from either the ATC-7 recommended formulas or by calculations based on UBC seismic forces.

6. In future design of instrumentation for recording of diaphragm motions, it is advisable to have sensors to record vertical motions at the center and other locations of the diaphragm. For those structures already instrumented and yet lacking such sensors it is envisaged that this recommendation would not require extensive alteration in the existing scheme.

REFERENCES

1. Celebi, M., Brady, G., Safak,E. and Converse, A., Performance of an Earthquake Excited Roof Diaphragm, Proc. ASCE Conference on Dynamic Response of Structures, U.C., Los Angeles, March 31-April 2, 1986 (1986)
2. Shakal, A. F., Sherburne, R. W. and Parke, D. L., CDMG Strong Motions Records from the Morgan Hill, California, Earthquake of 24 April 1984, Office of Strong Motion Studies Report N_o OSMS 84-7, CDMG Sacramento, CA (1984)
3. ASCE, Wood Structures-A Design Guide and Commentary (1975)
4. ATC-7, Guidelines for the Design of Horizontal Wood Diaphragms, Applied Technology Council, Palo Alto, CA (1981)
5. Baird, S. A. and Ozelton, E. C.,Timber Designer's Manual, Granada Publishing Co. (1984
6. Safak, E.,USGS Open-File Report (in preparation) (1987)