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NONSTATIONARY SEISMIC RESPONSE OF AN INSTRUMENTED FIVE-STORY PRECAST REINFORCED CONCRETE BUILDING

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ABSTRACT

The results of study aimed at the analysis of the response of an instrumented five-story reinforced concrete building in Mexico City are presented. The building is located in the so-called transition zone of Mexico City. The structural system of the building in both directions is formed by entirely-precast moment-resistant reinforced concrete frames. Non-structural elements consist of precast light-weight concrete elements in all facades of the building with interior partitions located at beam lines. The building forms part of the strong-motion accelerograph network of the Mexican Center for Disaster Preventions, CENAPRED. A brief description of the building and its instrumentation is presented. The objectives of this paper are first, to describe analytical studies conducted to obtain the dynamic characteristics of the building from the records obtained during earthquakes using system identifications techniques; and second, to describe the main features of the seismic response of the structure. The analysis of the recorded response shows that there in an important change in the lateral stiffness of the building during each earthquake, particularly in the longitudinal direction. Without experiencing structural or non structural damage variations of nearly 50 percent in the fundamental period were identified from the earthquake record.

KEYWORDS

Precast building; System identification; Recorded response; Instrumented building; Non-stationarity.

DESCRIPTION OF THE STRUCTURE

The building studied here is a four-story precast reinforced concrete building whose structural system in both directions is formed by moment-resistant frames. The building is located in the north of Mexico City intermediate soil conditions, in the so-called transition zone (Fig. 1). In the longitudinal direction the building has 20 bays of 3.3 m each and in the transverse direction has only one bay of 14.7 m. The building rests on a two-meters thick box foundation. Figure 2 shows plan and elevations of the building. Non-structural elements consist of precast light-weight concrete elements in all facades of the building with mansory interior partitions located at beam lines.

The building forms part of the strong-motion accelerograph network of the Mexican Center for Disaster Prevention, CENAPRED. The building was instrumented in 1990 with three SMAC-MD digital accelerographs. These instruments are located as follow: one in the roof of the building; one in the basement; and the other in the free field. As shown in Fig. 2 instruments 2 and 3 are located in the middle of the building.

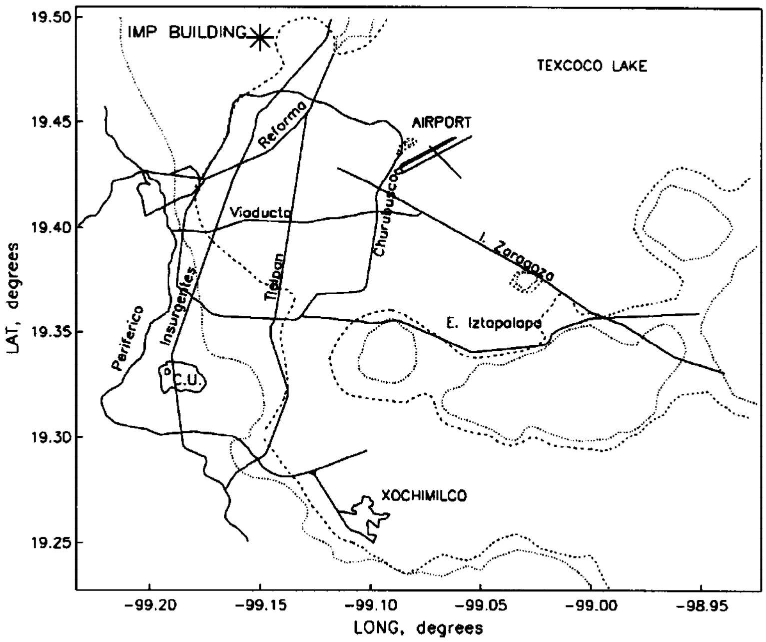

Fig. 1. Map of Mexico City showing the location of the building.

MEASURED RESPONSE

In the five years in which the building has been instrumented, it has been subjected to several earthquake ground motions. The magnitudes (Ms) of the earthquakes that have triggered the instruments in the building are between 5.8 and 7.6. These earthquakes are listed in Table 1. Also listed in this table are the maximum accelerations recorded in each instrument in each direction during each earthquake. It can be seen that the largest magnitude earthquake that has shaken the building is the october 9, 1995 Manzanillo earthquake (Ms=7.6), however the epicenter was located more than 500 km away from the building, so the maximum accelerations are smaller than those recorded as a result of smaller-magnitude events. The maximum ground acceleration that has experienced in the building is 28.11 cm/s^2 which corresponds to the september 14, 1995 Copala earthquake. During this same earthquake the maximum acceleration recorded in the roof was 75.4 cm/s^2.

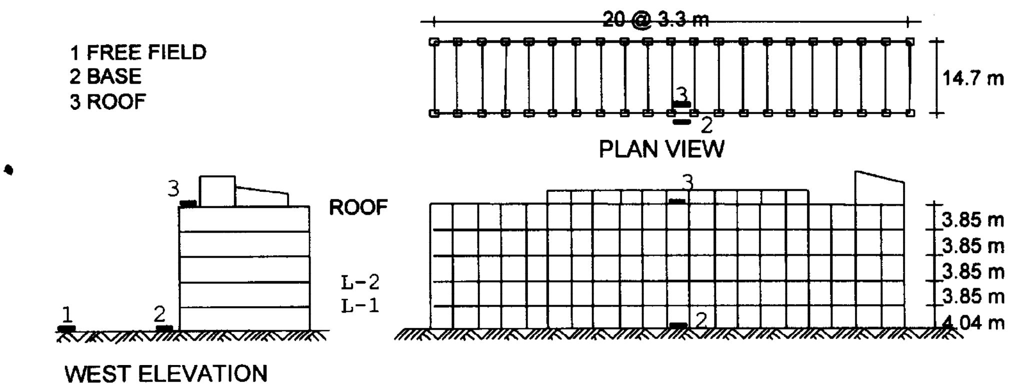

Fig 2. Plan and elevation of the building showing its instrumentation.

Table 1. Main characteristics of the events recorded in the building.

DATE	M_s	EPICENTRAL DISTANCE (Km)	INSTRUMENT	MAXIMUM ACCELERATION (gals) T	L	V
MAY 31, 1990	5.8	305.42	FREE FIELD BASE ROOF	-10.16 -6.84 17.03	-10.77 -7.57 22.58	2.87 -1.50 1.98
MAY 15, 1993	5.9	339.55	FREE FIELD BASE ROOF	-6.76 4.51 10.24	-8.06 6.80 27.84	1.90 -1.63 -1.97
OCTOBER 24, 1993	6.6	328.70	FREE FIELD BASE ROOF	14.65 -9.40 19.29	12.08 -7.42 -44.71	-2.72 1.89 2.53
MAY 23, 1994	6.2	216.32	FREE FIELD BASE ROOF	-12.76 -7.84 -18.95	-9.19 -7.35 -19.99	-4.97 -4.39 -5.86
DECEMBER 10, 1994	6.4	296.34	FREE FIELD BASE ROOF	9.00 7.20 -14.04	-14.25 8.42 -44.16	-2.72 -2.50 -2.78
SEPTEMBER 14, 1995	7.2	309.03	FREE FIELD BASE ROOF	28.11 -22.89 -50.69	24.78 19.32 -75.41	5.92 5.55 6.44
OCTOBER 9, 1995	7.6	517.32	FREE FIELD BASE ROOF	6.74 -6.20 -8.48	4.61 -3.69 12.79	1.28 -1.40 -1.77

DYNAMIC CHARACTERISTICS OF THE BUILDING

When a building is instrumented the occurrence of an earthquake can be viewed as a full-scale, large amplitude experiment on structure that offers the opportunity to make a quantitative study of the response of buildings when subjected to earthquake ground motions. In particular it is important to assess the modeling and analysis techniques commonly used in practice to evaluate the force and deformations introduced by earthquakes. In this study three system identification techniques were used to obtain the dynamic characteristics. Response analysis was only done for three recorded events: may 31, 1990, october 24, 1993 and december 10, 1994.

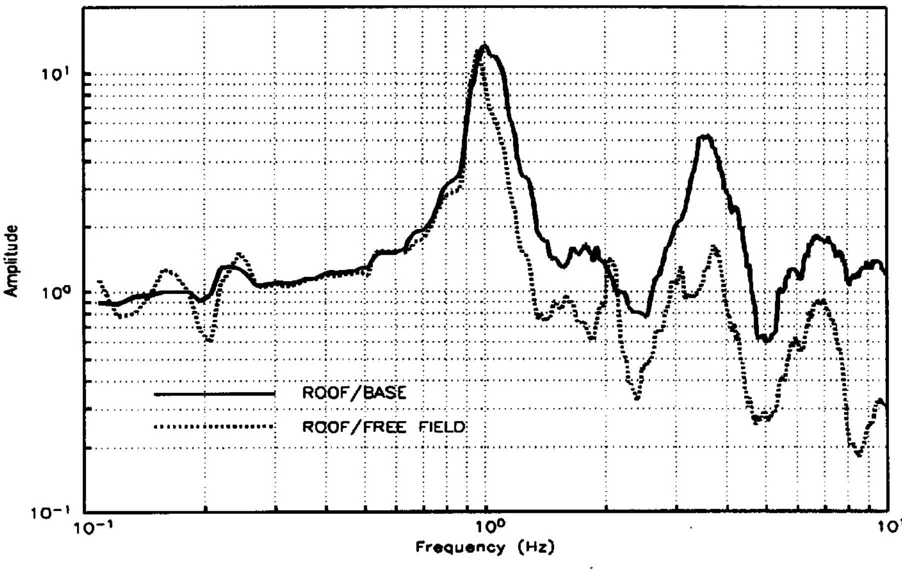

Fig 3. Transfer functions of the longitudinal direction for the december 10, 1994 event.

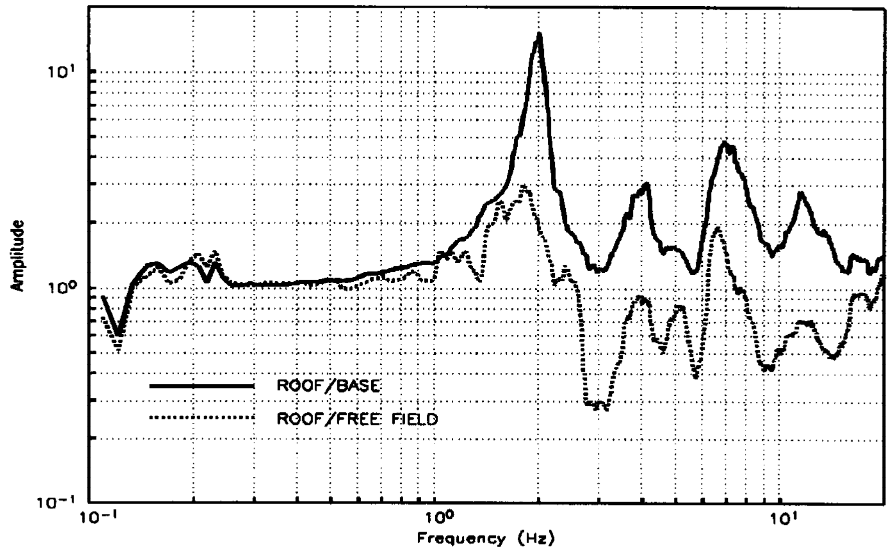

Fig. 4. Transfer functions of the transverse direction for the december 10, 1994 event.

The Fig. 3 shows the transfer functions for the longitudinal direction for the earthquake of december 10, 1994. It can be seen that very large amplifications occur around 1 Hz (T=1 s) and around 3.6 Hz (T=0.28 s) which correspond to the first and the second modes of vibrations of the building. The Fig. 4 shows the transfer function for the transverse direction for the earthquake of december 10, 1995. Peaks at 2 Hz and 7 Hz correspond to the first and second translational modes in this direction, whereas the peak around 4 Hz corresponds to the first torsional mode. As expected, in the high frequency range the roof/base transfer function is larger than the roof/free-field transfer function because of kinematic interaction effects.

By comparing the transfer function between roof and the base and between the roof and the free field it can be seen that soil-structure interaction does not have strong influence in the first mode response of the longitudinal direction, however for the transverse direction the first mode is significantly affected by soil-structure interaction, as it can see in the Figure 4, where is very clear that there is a shift of the fundamental frequency and that equivalent damping in the structure is increase from the roof/base transfer function to the roof/free-field transfer function. Table 2 shows fixed-base periods, T_s, inferred from earthquake records for each direction and for each event. Also shown in the table is the period on flexible foundation, T_i, and the ratio between T_i and T_s. This latter parameter is a measure of the influence of soil-structure interaction. It can be seen that, on average, there is in much larger influence in the transverse direction, where the slenderness ratio of the building (height to base width) is much larger (4.5 times larger) than for the longitudinal direction.

Table 2. Fixed-base periods, T_s, and period on flexible foundation, T_i, identified from records.

EVENT	LONGITUDINAL			TRANSVERSAL		
	T_s	T_i	T_i/T_s	T_s	T_i	T_i/T_s
MAY 31, 1990	1.02	1.05	1.03	0.52	0.58	1.12
MAY 15, 1993	0.98	1.06	1.09	0.51	0.57	1.12
OCT 24, 1993	0.95	1.00	1.05	0.51	0.57	1.12
DEC 10, 1994	1.00	1.04	1.04	0.50	0.55	1.10
SEP 14, 1995	0.93	1.08	1.16	0.51	0.58	1.13

Table 3. Fixed-base periods, T_s, for the three first modes in each direction, identified from records.

EVENT	LONGITUDINAL			TRANSVERSAL		
	T_1	T_2	T_3	T_1	T_2	T_3
MAY 31, 1990	1.02	0.32	0.16	0.52	0.14	0.09
MAY 15, 1993	0.98	0.28	0.15	0.51	0.14	*
OCT 24, 1993	0.95	0.28	0.14	0.51	0.14	0.08
DEC 10, 1994	1.00	0.28	0.15	0.50	0.14	0.09
SEP 14, 1995	0.93	0.28	0.15	0.51	0.14	0.09

* was not possible to identify

Table 3 shows the fixed-base periods for the first three modes of vibration of the structure in each direction that were inferred form the earthquake records corresponding to each event. It can be seen that the inferred periods, in general, are very similar for all events. There are larger differences for the fundamental period in the longitudinal direction. This is partly caused by the fact that the period does not remain constant during the earthquake. Periods shown in this table correspond to periods inferred from time-invariant system identification techniques. The ratio between the first three modes in the longitudinal direction is practically the same as the one expected for a pure shear-beam building, suggesting that the building lateral deformation profile is dominated by shear deformations. On the other hand, for the transvers direction the ratio between the first three modes suggest a mix of shear and flexural lateral deformations.

Figure 5 shows the transfer functions for the longitudinal direction between the roof and the free field for three different events: may 31, 1990, october 24, 1993 and december 10, 1994. The maximum acceleration recorded in the first event is about half of that recorded during the other two events. However, it can be seen that the three transfer function are, in general, very similar, with only a slightly larger amplification for the first mode in the october 24, 1993 event.

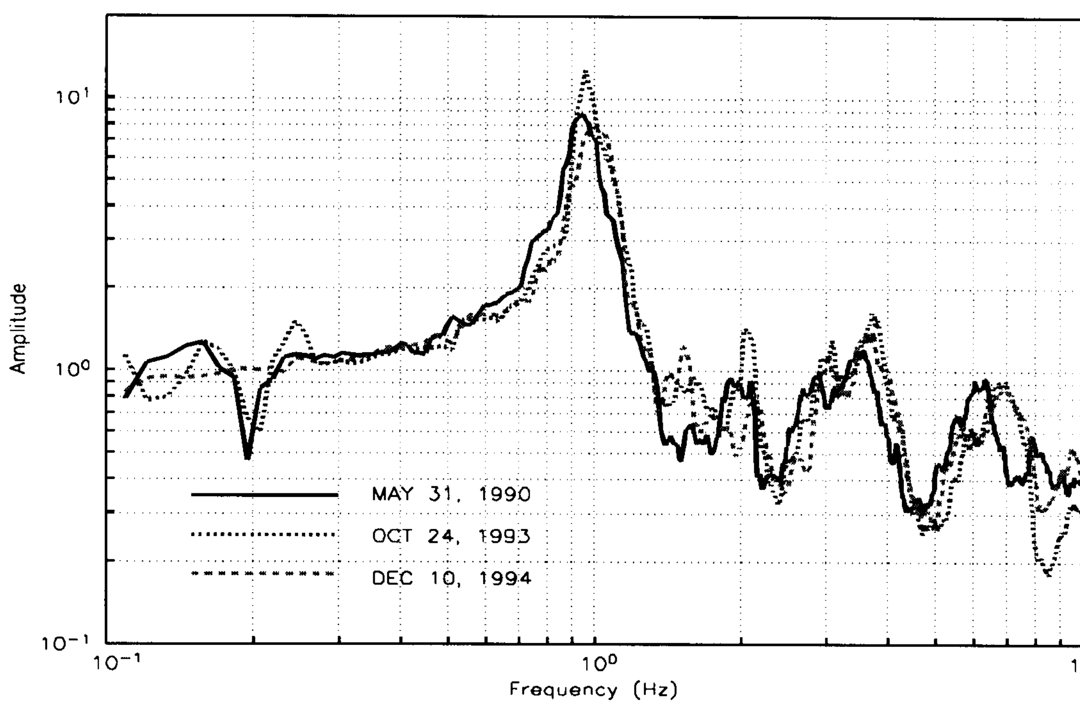

Fig. 5. Transfer functions for the longitudinal direction for three different events.

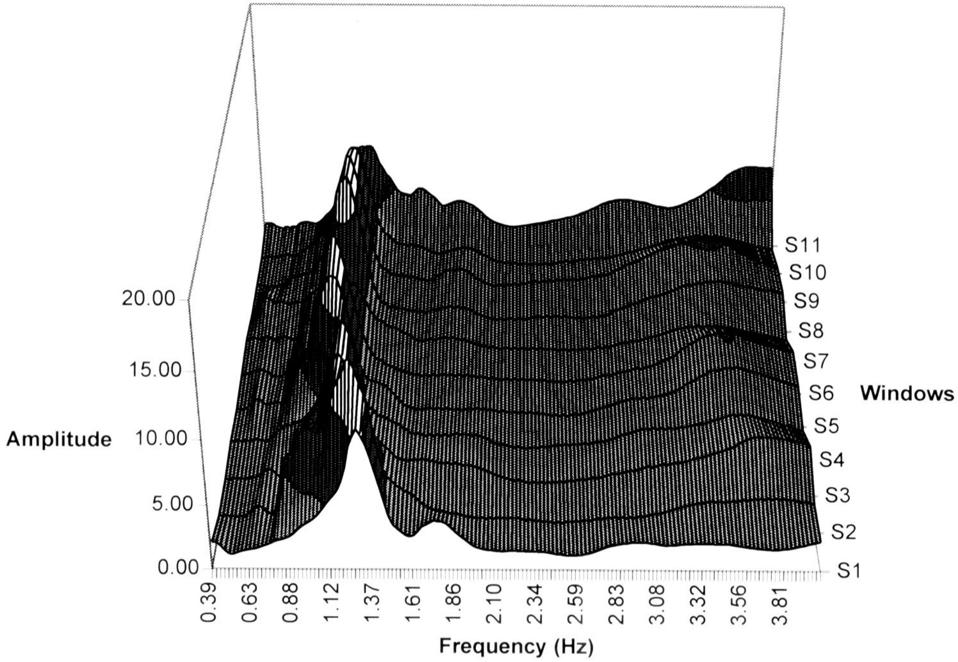

Figure 6. Evolutionary transfer function for december 10, 1994 event.

NONSTATIONARY SEISMIC RESPONSE OF THE BUILDING

As mentioned before in the longitudinal direction changes in the fundamental period were observed during the earthquake ground motion. In order to quantify the changes in stiffness during the earthquake time-variant system identification techniques were used. Figure 6 shows a evolutionary plot of the transfer function for frequencies between 0.4 and 4 Hertz. The data plotted in this figure corresponds to 25 second windows of the december 10, 1994 earthquake. In this tridimentional plot the first window is shown in the front and time advances toward the back of the plot. It can be seen that as intensity increases there is a decrease of the fundamental frequency. This change in frequency can be better observed in Figure 7 which shows a equal-amplification contours of the evolutionary transfer function. As shown in this figure at the beginning of the earthquake the building has a fundamental frequency of about 1.3 Hz which slowly decreases to a minimum of about 1 Hz at the 85 s mark.

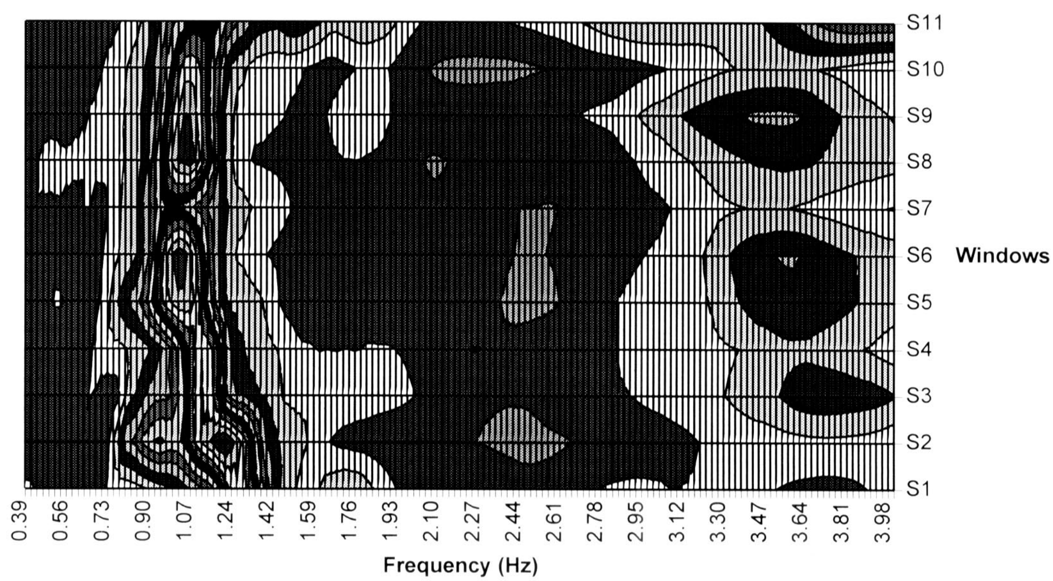

Figure 7. Equal-amplification contours of the evolutionary trasfer function for december 10, 1994 event.

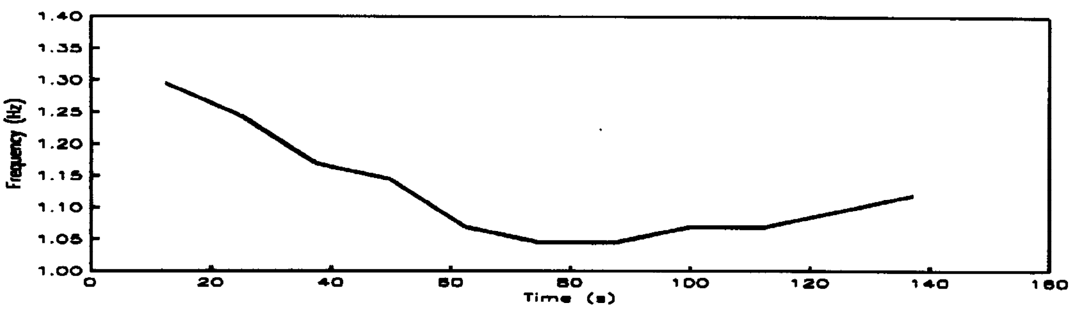

Figure 8. Fundamental frequency time history inferred from the december 10, 1994 event.

As shown in Figure 8 when the intensity of the earthquake begins to diminish there is a recovery of lateral stiffness which produces a increase in fundamental frequency, however not full recovery is attained. Analysis of the next recorded ground motion shows that further recovery to practically the initial frequency occurs between earthquakes.

An attempt was made to try to correlate the increase in fundamental period and the corresponding decrease in lateral stiffness with the level of lateral deformation in the structure. For this purpose the root mean square relative displacement (roof relative to the ground) was computed for each time window and plotted against the ratio of the period inferred in each window to the period at the beginning of the earthquake. The results for the october 24, 1993 and the december 10, 1994 earthquakes are shown in Figure 9. It can be seen that there is an increase in period with an increase in relative displacements between the roof and the base, and that the trend is very similar for both earthquakes. In none of these earthquakes damage has been observed either structural or not structural. However changes about to 50 percent in the fundamental period of the structure have been observed. The change in lateral stiffness is characterized by a rapid loss of stiffness for small levels of deformation which points out that periods of vibration inferred from ambient vibrations may not be representative of the period of vibration during earthquake ground motions and that strong changes in the fundamental period of vibration do not necessarily imply damage in the structure, which is a conclusion that was previously drawn by other studies (Anderson, et al., 1991; Murià, et al., 1996; Celbi, et al., 1193). The points

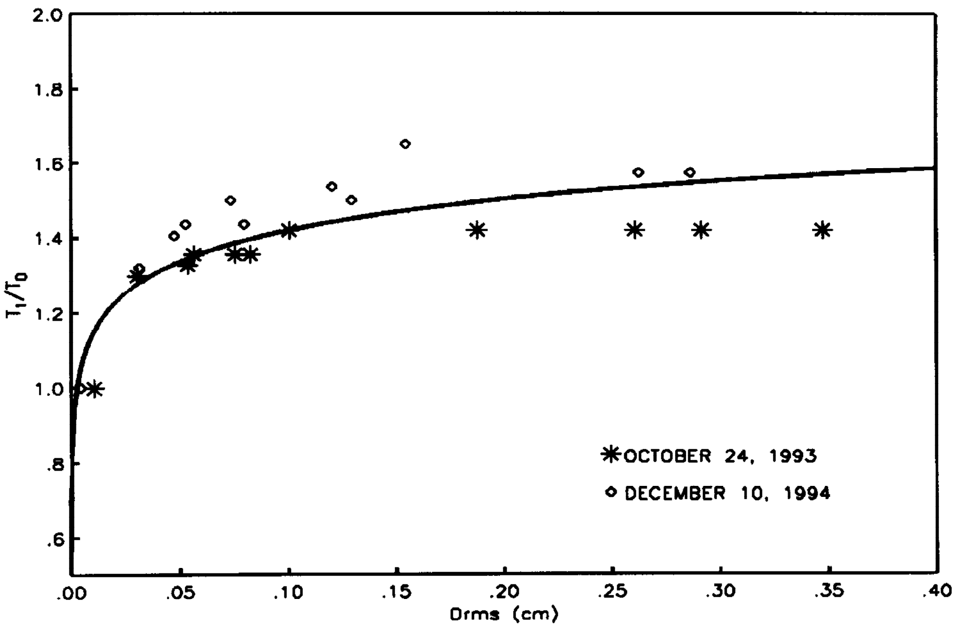

Figure 9. Variation of the fundamental period with changes of the root mean square of roof relative displacements.

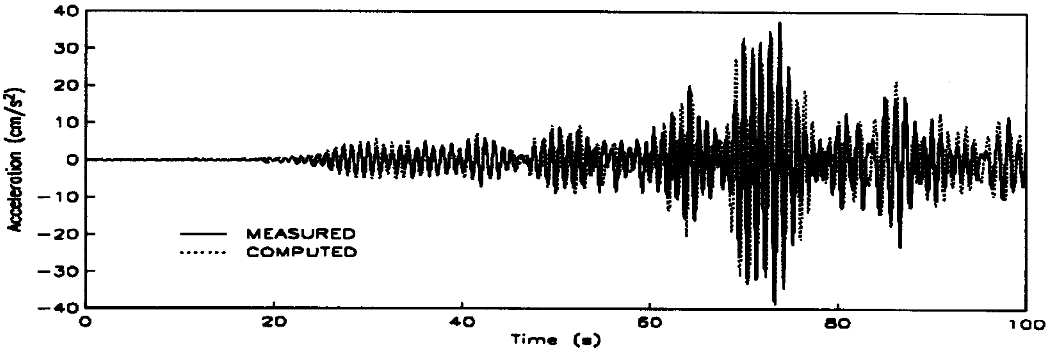

Figure 10. Comparison of measured and computed response for december 10, 1994 event.

shown in Figure 9 were fitted to a logarithmic curve through regression analysis. The fitted curve is as follows:

$$\frac{T_1}{T_0} = 1.6978 + 0.12061 Ln(D_{rms}) \quad (1)$$

whre T_0 is the fundamental period corresponding to the first time window (at the begining of the earthquake); T_1 is the "instantaneaus" fundamental period and D_{rms} is the root mean square relative roof displacement.

Using a simplified three-degrees-of-freedom model which takes into account soil-structure interaction and equation 1 the response of the building was computed. In the model , the first degree of freedom (DOF) corresponds to the total displacement of the mass; the second DOF corresponds to the lateral displacement of the base and the third DOF corresponds to the rocking of the base. A comparison between the computed and the measured absolute acceleration time history at the roof for the december 10, 1994 earthquake is shown in Figure 10. It can be seen that there is a very good agreament between the computed response and the measured respponse.

CONCLUSIONS

During the time that the building has been instrumented, it has been subjected to 7 earthquakes larger than 5.8. No damege has been observed in any of these earthquakes. Changes in fundamental periods of nearly 50% have been identified in the building which implies that the lateral stiffness during the most intense part of the earthquake can be less than 50% of that inferred from low amplitude shaking at the begining of the earthquake. This loss of lateral stiffness is partially gained as intensity of the earthquake is reduced towards the end of the earthquake.

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