

## **SEISMIC RESPONSE OF A TWICE-RETROFITTED BUILDING**

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### **ABSTRACT**

Well-known methods of spectral analyses and system identification are used to analyze a set of 11 acceleration records obtained between November 1992 to December 1999 from a 14-storey reinforced concrete building founded on soft soil in Mexico City. The building has been repaired twice during its lifetime. The first retrofit was in 1986 due to the structural damage produced by effects of the  $M_s = 8.1$ , 1985 Michoacán earthquake in Mexico City. Because of the deficient behaviour of the structure observed as a consequence of moderate intensity earthquakes between 1986 and 1995, it was then upgraded again.

The purpose of this paper is to study the main features of the seismic response of the building and its interaction with the soil when subjected to several small and moderate size earthquakes in a seven-year period. From the analysis of the seismic records, variations of natural frequencies and damping of the soil-structure system were observed. It was found that the dynamic properties of the system are very sensitive to the intensity of the ground shaking. Significant stiffness degradation of the building was found as well. The variation of structural parameters of the system can be attributed mainly to different non-linearity sources of the structure.

**KEYWORDS:** Earthquake Records, Structural Damage, Structural Rehabilitation, Soil-Structure Interaction, Instrumented Buildings

### **INTRODUCTION**

Currently, there are up to 20 seismically-instrumented buildings in Mexico City, but only five of them have enough instruments that may help visualize a "detailed" global response of the building during an earthquake. One of these five buildings, known as Jal building (Meli et al., 1998) is the subject of this paper. The study of Jal building is part of a joint project between Instituto de Ingeniería, UNAM (Mexico), and Politecnico di Milano (Italy).

Jal building is located in a central district of Mexico City and it is founded on soft soils with dominant site frequency of about 0.5 Hz. Typical ground motion records for the site are characterized by long durations and quasi-monochromatic frequency contents. Jal building is one of the several buildings that were damaged during the September 19, 1985 Michoacán earthquake and that were later retrofitted. Different retrofit strategies were used in Mexico City at the time to rehabilitate buildings, knowing the global benefits of many of them, as summarized elsewhere (for example, Tena-Colunga et al., 1996). However, there are still many questions to answer about the actual behaviour of a given retrofitted building, among other reasons because of the variations and adaptations of the different rehabilitation techniques for specific buildings, the limited number of experimental data available for some specific retrofit strategies, and in light of the knowledge gained during the last 15 years on the rehabilitation of buildings.

The seismic instrumentation of Jal building started in 1992. During the first seven years of continuous monitoring of this building, a large number of good-quality seismic records have been acquired (Table 1). Most of the recorded events were small and moderate-size earthquakes; nevertheless, structural and non-structural damage was observed during many of these events. Given the poor structural performance of the original retrofit for Jal building, the owners decided to improve the existing retrofit between 1996 and 1997. However, during the 1999 seismic events (similar in intensity to some moderate-size earthquakes previously recorded), the study of the recorded response revealed that the structural system significantly deteriorated again.

The acceleration records of this building are particularly interesting to study because they are associated with seismic events of different intensities. The interpretation of these records revealed that the structural

behaviour of the building is sensitive to most events and cumulative damage that is associated with a substantial stiffness degradation for the structure.

**Table 1: Main Characteristics of Recorded Earthquakes**

Event	Location of epicentre	Date	$M_w$	Epicentral distance (km)	$I_{Arias}$ (cm/s)	PGA (cm/s <sup>2</sup> )
93-3	Guerrero	15/05/93	6.0	318	0.3	4
93-4	Guerrero	15/05/93	6.1	315	1.2	11
93-11	Guerrero	24/10/93	6.7	303	2.8	13
94-1	Guerrero	23/05/94	6.3	206	0.6	7
94-3	Guerrero	10/12/94	6.5	280	5.4	17
95-1	Guerrero	14/09/95	7.5	298	19.5	37
95-2	Colima	09/10/95	7.9	532	4.3	12
96-1**	Guerrero	15/07/96	6.6	291	0.7	9
97-1**	Michoacán	11/01/97	7.1	427	5.5	16
97-2*	Guerrero	22/05/97	6.5	285	0.5	5
98-1*	Oaxaca	03/02/98	6.3	515	0.2	4
99-1*	Puebla	15/06/99	6.9	222	13.7	25
99-2*	Guerrero	21/06/99	6.2	285	0.4	5
99-3*	Oaxaca	30/09/99	7.4	455	19.7	34

\* Second rehabilitation completed      \*\* Second rehabilitation in process

The main purpose of this work is to evaluate the seismic response of the building, based on the identified structural parameters and soil-structure interaction (SSI) effects derived from the analyses of the seismic records obtained for the building. This paper shows the observed variations on the system vibration frequencies and damping, estimates the stiffness degradation in terms of the changes observed on the structural frequencies for the different recorded seismic events, defining two types of stiffness degradation: permanent and apparent, and highlights the importance of SSI effects on the fluctuations observed on the structural frequencies for the building.

## THE BUILDING UNDER STUDY

Since it is located in the Mexico City area where the greatest damage was observed during the 1985 earthquake, and because large vibrations are expected due to its flexibility, the Jal building was selected among 50 candidates for study. This was built in 1981 using the Mexico's Federal District Constructions Code (RCDF, 1976). It is a 14-storey reinforced concrete structure occupied by offices, except for the first four stories, which are used for parking. The building has a small three-level adjoining structure, separated by 150 mm from the main structure and attached at the base. Originally the structure consisted of a waffle flat-plate (450 mm thick) on rectangular columns with a box-type masonry wall around the staircases, a small core of concrete shear walls around the elevator shaft, and masonry infill walls in the end frames of the longitudinal direction (L). The detailing of the reinforcement does not comply with present requirements for ductile structures, especially regarding the transverse reinforcement in columns and in slab-column joints. Figure 1 shows a schematic view of the structure.

The foundation consists of a concrete slab on friction piles, driven according to a rather peculiar solution, as shown in Figure 2. The so called A-piles are fixed to the foundation slab and driven into the clay soil without reaching the firm layers. B-piles are driven down to the firm layer but are separated from the foundation. The weight of the building is assumed to be supported by the lateral friction of the piles and partially transferred from the A-piles to the B-piles, thus avoiding a foundation failure while at the same time allowing the building to sink following the areal settlement of the ground.

In-situ measurements of the mechanical properties of the soil including dynamic characteristics were carried out in 1992. Cone penetration testing (CPT) was performed under the building and in a parking lot in

front of it, where a field (surface) station was placed. Also, at the field site, a selective sampling and seismic cone testing (SCT) was executed. Figure 3 shows the soil profiles at the building site. In general terms, under a crust of about 5 m, the upper soft clay deposits extend to a depth of 29.5 m, followed by a 3 m thick intermediate firm layer. The lower clay layer extends from 32.5 to 38.5 m depth, followed by the so-called deep firm deposits. The shear wave velocity profile, determined from the SCT tests, is also shown in this figure.

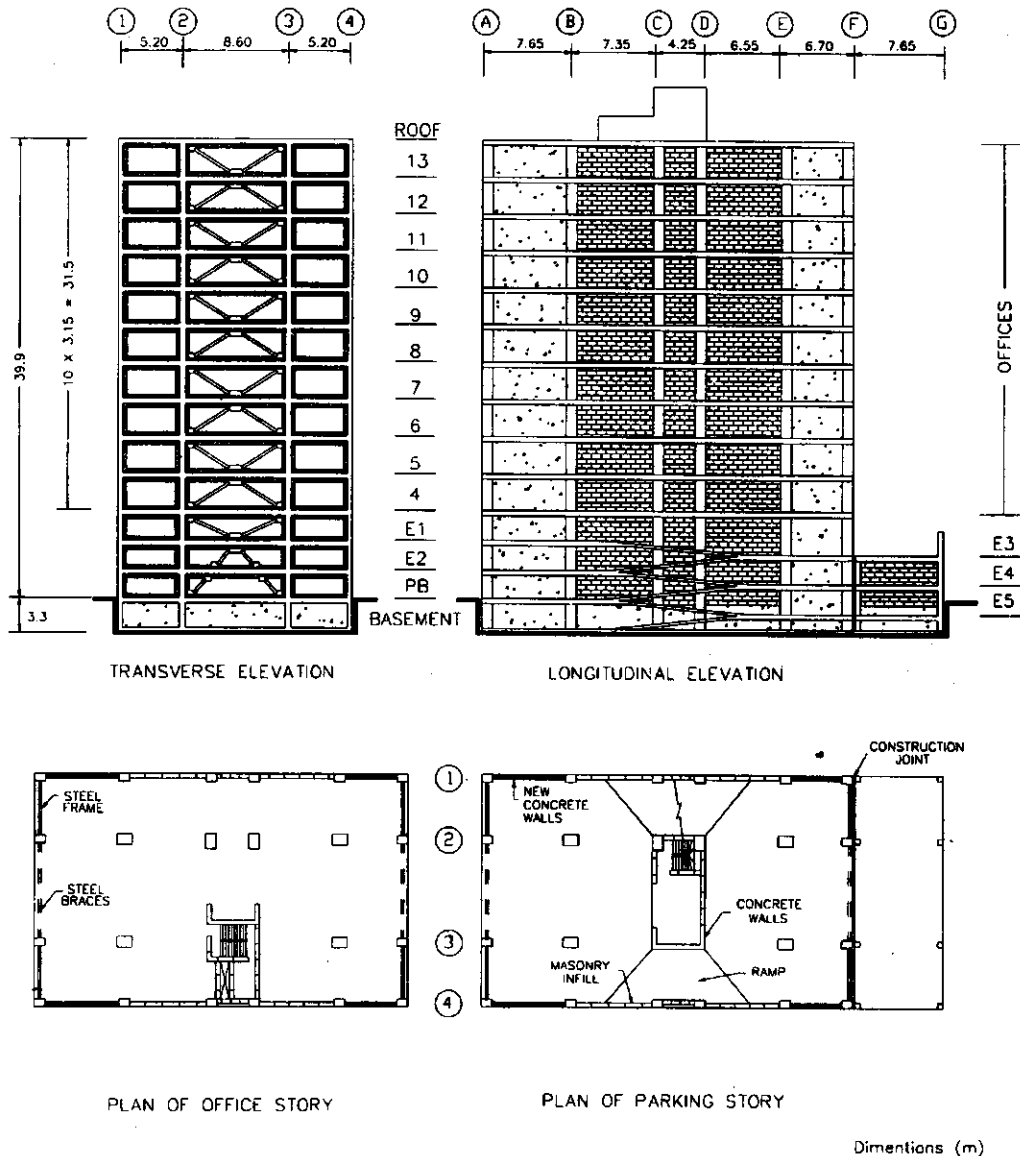


Fig. 1 Elevation and plans of building

Properties of the building's concrete were determined in 1993 by coring and by indirect methods (rebound hammer and ultrasonic tests). Quality was fairly uniform throughout the structure. A change in concrete type was detected above the 7th storey. Below this storey, the average compressive strength was 28 MPa, and the modulus of elasticity was 19,000 MPa. Above the 7th storey, the compressive strength was 25 MPa and the modulus of elasticity was equal to 18,000 MPa.

### 1. First Rehabilitation

In the 1985 earthquake, the JAL building suffered moderate structural damage which consisted of the bending and shear cracking of columns and severe shear cracking of masonry infill walls, especially above the seventh storey. The widths were smaller than 1 mm except in a few columns whose widths were greater

than 1 mm. Widespread cracking of the ribs of the waffle slab was also evident, revealing positive and negative yield lines. No appreciable settlement of the foundation was observed. It is convenient to point out that we did not inspect the building.

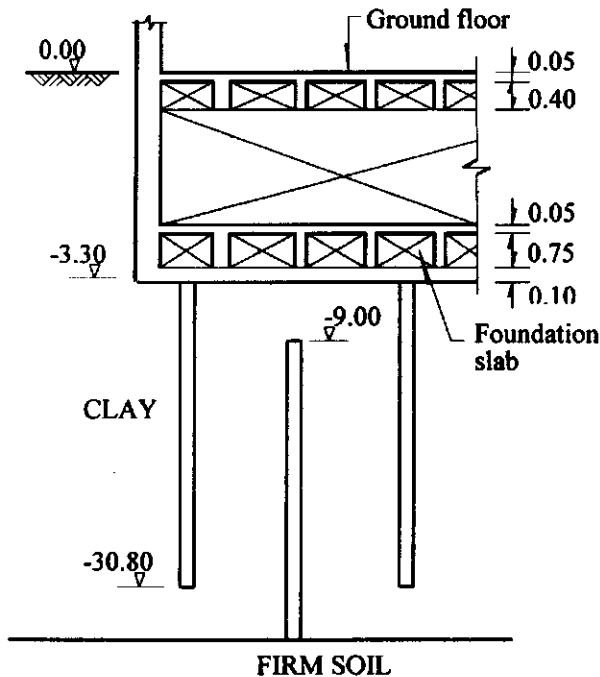


Fig. 2 Vertical cross-section of the building foundation

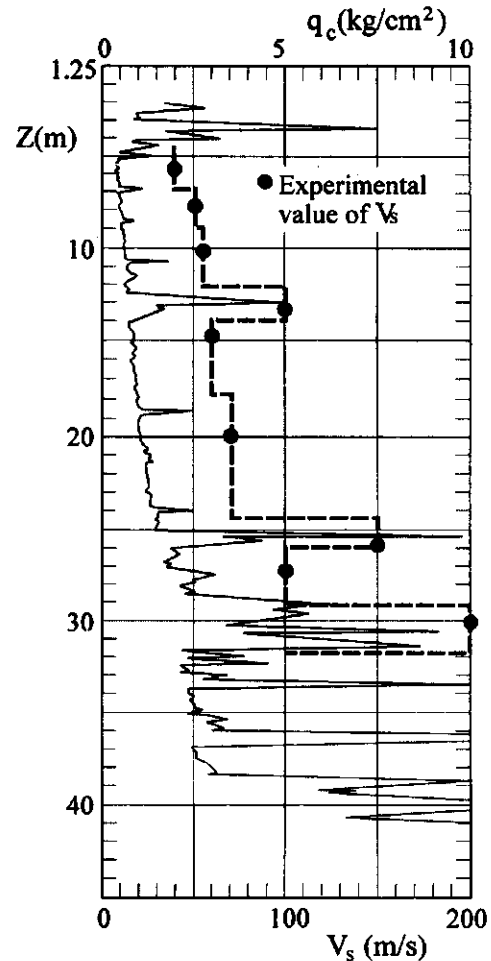


Fig. 3 Shear wave velocity ( $V_s$ ) profile and cone penetration resistance ( $q_c$ ) at the building site

The building was then strengthened in 1986 by adding infill concrete walls in the L direction connected to the original structure only through the upper slab. The sections of the interior columns were jacketed with new reinforced concrete without any modification to the foundation (Figure 1). The masonry walls were merely covered with plaster, and the cracks of the slabs were not repaired.

## 2. Second Rehabilitation

The December 10, 1994 earthquake (event 94-3) produced moderate damages in non-structural elements (cracking in masonry walls, as well as cracking and detachment of finished plaster and ceilings), and light structural damages (bending cracking and the perimeter sliding in concrete walls). These damages were attributable to the accumulation of the effects of previous earthquakes (mainly due to the September 1985 and April 1989 earthquakes). A light shear cracking also appeared in some columns of the parking area due to differential settlements. After the September 14, 1995 earthquake (event 95-1), more cracks in walls appeared. The bigger cracks were covered with plaster. In addition, some suspended ceiling elements fell down. The magnitude of previous settlements increased and the width of cracks in columns went up to 0.85 mm.

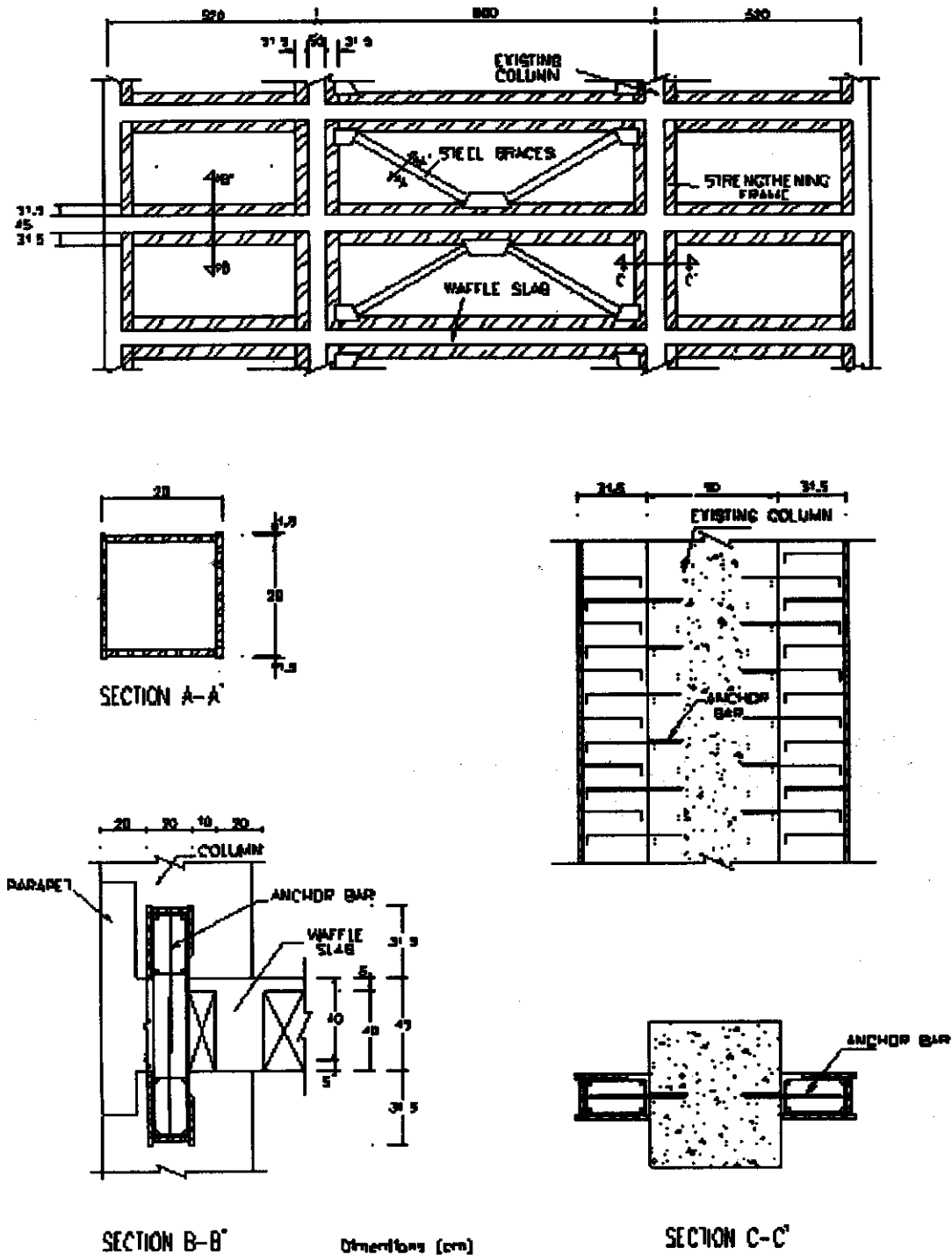


Fig. 4 Details of second rehabilitation

The deficient behaviour of the structure observed as a consequence of the moderately intense earthquakes between 1986 and 1995 led the owners to make a second rehabilitation. This time, the building was strengthened in the transverse direction (T) by steel bracing which was attached in the central bay of the exterior transversal frame through an additional steel and reinforced concrete frame. These coupling frames were put on all bays of the transversal façades. The vertical members of these frames were attached by using steel bars welded to the steel plates and glued to the concrete columns with epoxy resin through perforations. The bracing elements were square box-cross sections formed with steel plates welded to the

new coupling frames. The bracing system consisted of continuous X-bracing across two stories (Figures 1, 2 and 4).

The widths of the cracks in columns greater than 0.5 mm were injected with epoxy resin. The masonry walls with major damage were partially or totally replaced; the larger cracks were repaired with the injection of epoxy resin and then a mortar or plaster cover was applied. Moreover, an emergency steel staircase at the southwest corner of the building was built. At one end, it was supported by the building at each one of its storeys and by two steel columns at the other extreme. These steel columns were connected to two new reinforced concrete columns built over the full height of the adjoining structure.

### 3. Damages Observed after the Second Rehabilitation

The June 15 and September 30, 1999 earthquakes, events 99-1 and 99-3 respectively, produced diverse damages in the building. On several stories, detachment of finished plaster and ceilings was observed and some suspended ceiling elements fell down. Some parapets in contact with the columns at the last garage storey showed some damage, except those with a flexible material joint. All the masonry walls were damaged (Figures 5 and 6).

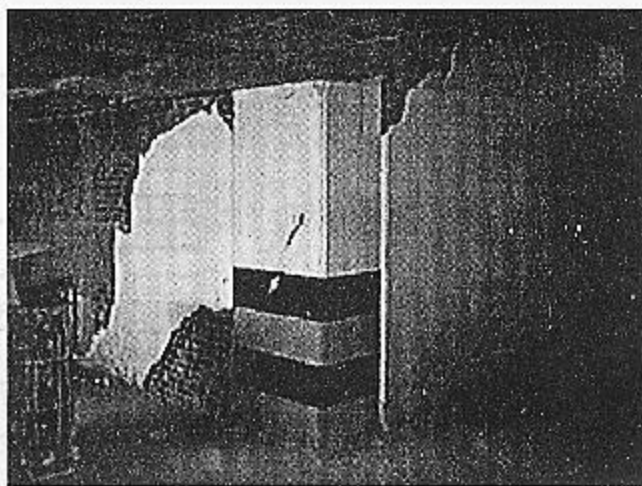


Fig. 5 Reopening of cracking at basement column 2D, parking view

At the concrete shear walls around the elevator shaft and in several columns of the main structure (B4, C4, D4, D2, F1, F2, F3, F4) and of the three-level small adjoining structure (F1, F2, F3, F4), some cracks previously filled with cement were opened (Figures 5 and 6). In addition, new cracking bordering old cracking injected with epoxy resin in the previous rehabilitation was detected. Their widths were less than 1 mm.

In the T-direction of the building, a parapet restricting the C4 column displacement produced a short column problem at the last parking storey; values up to 0.7 mm width in reopened old cracks and in new cracks were observed at this column.

In two of the central columns of the basement and in the concrete wall of the foundation box corresponding to the main façade, new cracks were observed. Particularly shear cracks up to 0.6 mm width were observed at the north and south faces of columns A2 and A3. Apparently, these cracks were due to the eccentricity between the plane where the braced frames were built and the wall of the foundation box.

Some cracking at the perimeter of the concrete-steel and concrete-concrete interfaces between the old and new elements was detected (Figure 7). In addition, at the parking stories, light cracking, perpendicular or inclined with respect to the metallic elements, was perceived. Effects of the 99-3 event increased the number of cracks, and their widths were less than 0.2 mm.

After the 1999 earthquakes, the building tilt was measured. Greatest value found in the L-direction was 0.29% and in the T-direction was 0.25%. These values are almost the same as those obtained in March 1997 (Muria-Vila and Rodríguez, 2001).

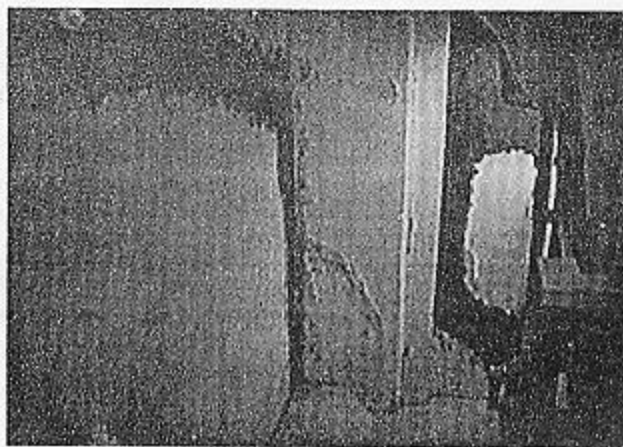


Fig. 6 Reopening of cracking at basement column 2D, stair-case view

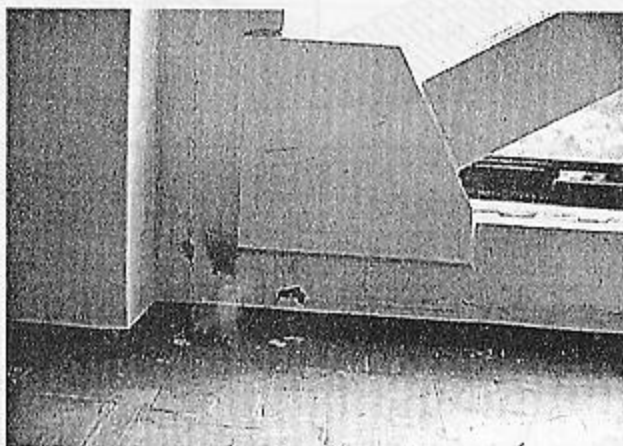


Fig. 7 Perimeter cracking at concrete-steel and concrete-concrete interfaces between new and old structural elements

### SEISMIC INSTRUMENTATION

The building was instrumented with a 14-accelerograph network. Eleven accelerographs were placed into the building, whereas the remainder were located on the ground. The field station instrument (G) was located about 50 m away from the building (Figure 8).

All instruments are tridirectional solid state, digital accelerographs (Terratech DCA-333R), interconnected in a master-slave configuration to allow for a simultaneous trigger when the threshold of the master instrument is reached. Also, all instruments have a common time reference. To optimise the quality and resolution of the records, each instrument was factory-set up with a particular full-scale range in accordance with the amplitude of the vibration expected at its location within the structure.

This structure, one of the best-instrumented buildings in Mexico, was designed to provide: a detailed record of the building vibration during earthquakes, a specific appraisal of the soil-structure interaction, and a picture of the wave amplification from the deepest layers to the surface.

Since the beginning of the operation of the array in November 1992, several earthquakes have been recorded. The more significant earthquakes that have been studied, the epicentre location and the general characteristic of these events are shown in Table 1 and in Figure 9.

The 95-1, 99-1 and 99-3 events were those of higher magnitude and horizontal intensity (Arias, 1970), 19.5, 13.7 and 19.7 cm/s, respectively. These values greatly contrast with the 376 cm/s calculated for the SCT records of the September 1985 earthquake.

Events 94-3, 95-1, 95-2, 99-1 and 99-3 produced visible structural and non-structural damages to the building. Event 97-1, having magnitude similar to the event 94-3, did not produce any visible damages to the building, but some light cracking on plaster cover at the perimeter of the masonry walls was detected. At this time, the second rehabilitation was almost concluded.

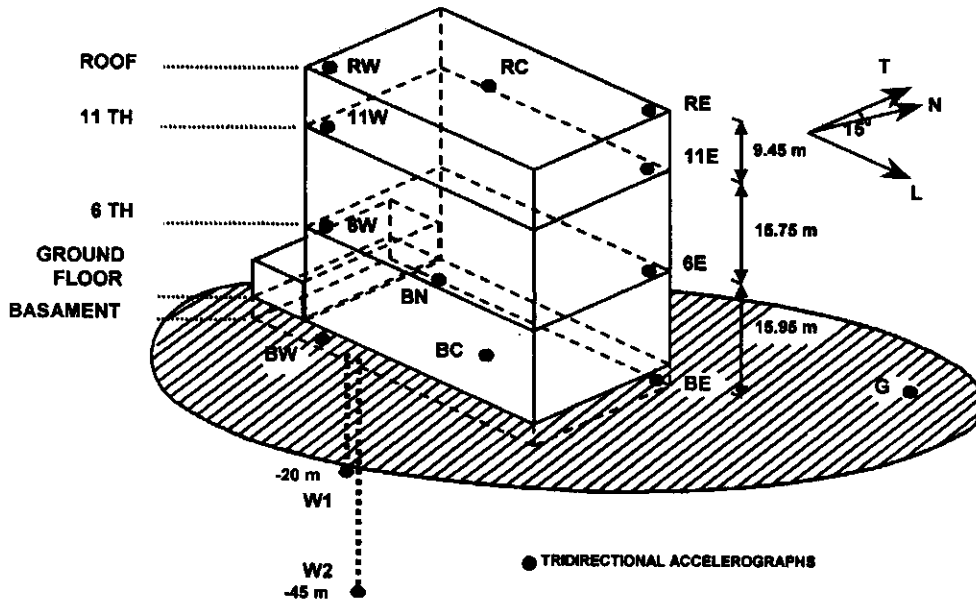


Fig. 8 Overview of the building instrumentation

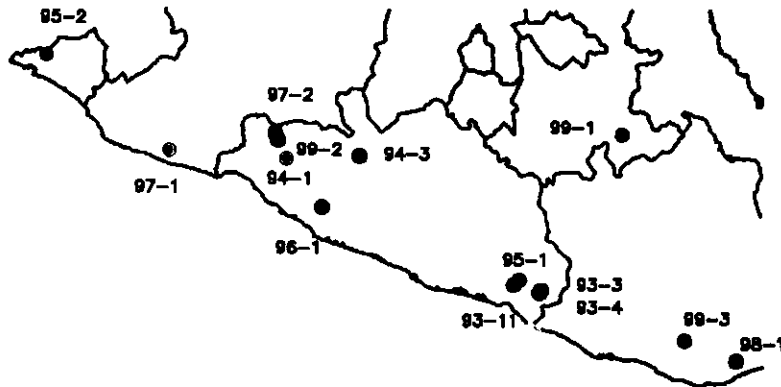


Fig. 9 Location of epicentres

### AMBIENT VIBRATION TESTS

In order to determine the main changes in the dynamic properties of the building under low vibration intensity, ambient vibration (AV) was measured. A set of eight uniaxial force balance accelerometers of high resolution was used in several set-up positions, each one of which was designed to identify a particular vibration mode or characteristic of the vibration of soil-structure system. Procedures based on random vibration theory (Bendat and Piersol, 1986) were used.

Seven rounds of AV measurements were performed, one before the instrumentation installation (October 1991), another at the completion of the instrumentation (September 1992) and the others after the



significant earthquakes of November 1994, February 1995, September 1995, February 1997 and July 1999. Table 2 shows the first frequency values of longitudinal (L), transversal (T) and torsion modes.

**Table 2: Dynamic Properties of Building from Ambient Vibration Tests**

Event	Date	Frequency (Hz)			Critical damping ratio (%)	
		T	L	Torsion	T	L
1AV	23/10/91	0.45	0.73	0.85	3.0	2.4
2AV	23/09/92	0.44	0.73	0.83	3.4	2.2
3AV	25/11/93	0.44	0.70	0.68	1.6	2.3
4AV	04/02/95	0.44	0.64	0.65	1.9	5.1
5AV	23/09/95	0.43	0.65	0.64	3.5	2.6
6AV	25/02/97	0.68	0.78	1.47	3.6	3.2
7AV	06/07/99	0.64	0.78	1.37	2.0	2.8

T – Transversal direction      L – Longitudinal direction

The first two AV tests revealed that the vibration frequency values did not change. After the 1993 earthquakes, the frequencies of the translation components remained practically unchanged, while the torsion frequencies of the system decreased by 20%. In the following three tests, November 1994, February 1995 and September 1995, changes of frequency up to 4, 11 and 23% in T, L and torsion, respectively, were observed.

The sixth test took place when the second rehabilitation was almost concluded. The construction of the emergency stairway, the replastering of walls and the placement of some windowpanes were still pending. A significant increment of the vibration frequencies was obtained in torsion and T-direction.

After the 1999 earthquakes, the frequencies obtained from the last test (7AV) were slightly higher than those obtained from the previous test. Nevertheless, it is necessary to point out that when the test was carried out, the walls had already been repaired with mortar or plaster cover. This accounts for the fact that the frequency in L-direction was the same as the sixth AV test and shows the contribution of these non-structural elements in the vibration frequencies measured.

**Table 3: Peak Accelerations and Displacements**

Event	Acceleration (cm/s <sup>2</sup> )						Displacement (cm)					
	Direction T		Direction L		Direction V		Direction T		Direction L		Direction V	
	Soil	Building	Soil	Building	Soil	Building	Soil	Building	Soil	Building	Soil	Building
93-3	4	11	4	10	3	3	0.26	1.42	0.31	0.99	0.07	0.15
93-4	8	38	8	27	—	6	0.84	3.17	0.62	2.56	---	0.31
93-11	13	35	11	56	6	11	1.05	4.86	0.97	5.49	0.19	0.72
94-1	7	19	5	19	6	6	0.33	1.36	0.36	1.85	0.11	0.86
94-3	13	49	17	124	8	23	1.43	6.45	1.59	12.93	0.2	1.76
95-1	35	116	37	130	16	33	3.72	16.99	2.43	13.75	0.95	2.31
95-2	12	32	8	72	3	14	1.42	5.63	1.05	8.54	0.28	1.05
96-1**	8	22	9	35	9	8	0.41	2.68	0.41	3.00	0.15	0.35
97-1**	16	54	15	49	7	11	1.86	5.02	2.11	5.24	0.54	0.85
97-2*	5	20	6	16	4	6	0.43	1.29	0.38	1.03	0.08	0.24
98-1*	4	20	4	12	2	3	0.24	1.01	0.24	0.60	0.06	0.19
99-1*	25	225	20	86	19	57	2.25	19.57	1.77	7.7	0.41	1.95
99-2*	5	20	5	24	4	4	0.32	1.96	0.38	1.69	0.08	0.26
99-3*	34	304	22	116	14	64	3.68	28.1	2.58	10.19	0.57	3.83

\* Second rehabilitation completed

\*\* Second rehabilitation in process

## EARTHQUAKE RECORDS

## 1. Characteristics of Accelerograms

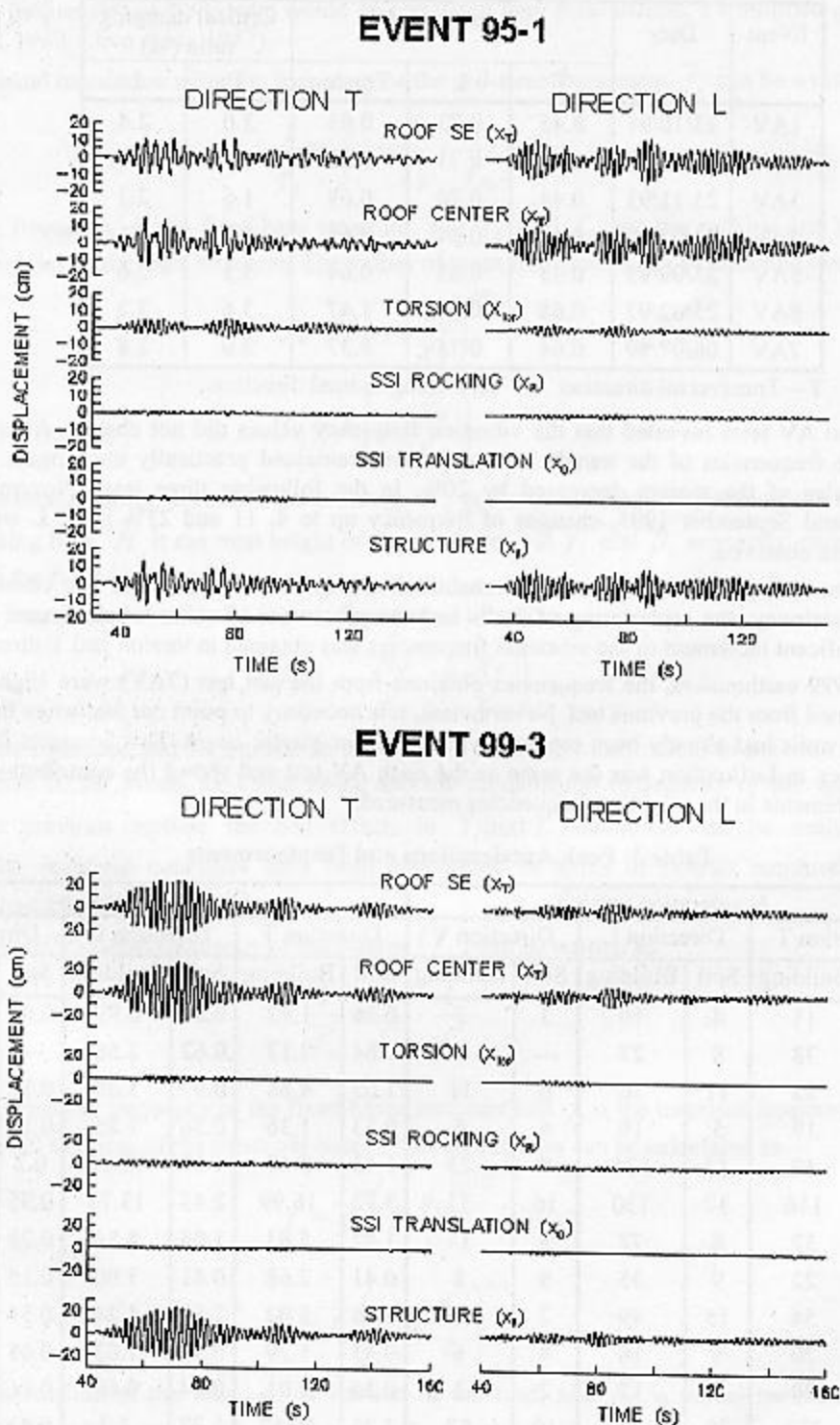


Fig. 10 Time-histories of displacements at roof level, with contributions of different sources deformation, calculated for events 95-1 and 99-3

The maximum accelerations and displacements in the L-direction were  $130 \text{ cm/s}^2$  and  $13.75 \text{ cm}$ , and in T-direction,  $304 \text{ cm/s}^2$  and  $28.1 \text{ cm}$  during the events 95-1 and 99-3, respectively. Table 3 summarizes the peak accelerations and displacements of the earthquake records.

The horizontal motions recorded were a compound effect of the contribution of the soil, the translation and the rocking of the base (SSI effect), the relative deformation of structure and its torsion. The array of the recording instruments within the building and soil allowed the decomposition of different sources of displacements that contributed to the building's vibration.

Figure 10 shows the case of the 95-1 and 93-3 events. In this figure, time-histories of the total displacements ( $x_T$ ) at the centre and at one corner of the roof level in the T-direction are shown, along with the contributions to these displacements, due to rocking and translation at the base plus the torsion. The torsional motion ( $x_{tor}$ ) was derived from the differences between the horizontal movements at roof corners. The rocking motion ( $x_R$ ) was calculated by the product of building height and angle of base rotation, assuming that the base is rigid for out-of-plane bending. The angle was derived from the vertical motions recorded in the corners of the basement level. Base translation displacement ( $x_0$ ) was computed by subtracting that at the field site instrument (G) from that at the basement (BC). Finally, displacements due to the flexibility of the structure ( $x_F$ ) were obtained by subtracting from the motion at the roof (RC) those obtained from base rocking and translation.

Though the torsional vibration was negligible in the 99-3 event, it was appreciable in the 95-1 event. Differences must have been due essentially to the structural characteristics of the building. Records of the 95-1 event show beatings that can be attributed to the similarity of the prevailing frequencies of the soil and the frequencies of the structure for torsional and longitudinal vibrations.

## 2. Mean Inter-story Drifts

A drift index was calculated by dividing the relative displacements by the vertical distance between instrumented stories (roof, 11th, 6th and basement). The contribution of SSI effects was not considered for the calculation of this index, for it was assumed that it did not contribute to structural damage. The drift indices between instrumented stories, calculated from the records of the 14 events, are shown in Table 4.

**Table 4: Building Interstory Drifts, in Percentage**

Event	Direction T			Direction L		
	RC/L11	L11/L6	L6/BC	RC/L11	L11/L6	L6/BC
93-3	0.024	0.041	0.025	0.011	0.017	0.009
93-4	0.063	0.086	0.055	0.034	0.050	0.028
93-11	0.091	0.121	0.056	0.090	0.112	0.067
94-1	0.027	0.040	0.024	0.028	0.036	0.023
94-3	0.130	0.217	0.106	0.223	0.287	0.181
95-1	0.267	0.434	0.259	0.246	0.303	0.199
95-2	0.129	0.172	0.091	0.175	0.214	0.129
96-1**	0.050	0.071	0.039	0.057	0.080	0.043
97-1**	0.063	0.066	0.050	0.080	0.088	0.060
97-2*	0.021	0.020	0.016	0.021	0.029	0.013
98-1*	0.020	0.015	0.012	0.007	0.009	0.006
99-1*	0.312	0.323	0.251	0.141	0.135	0.105
99-2*	0.039	0.028	0.030	0.039	0.035	0.021
99-3*	0.415	0.536	0.315	0.288	0.161	0.128

\* Second rehabilitation completed    \*\* Second rehabilitation in process

The allowable drifts according to the Mexico's Federal District Construction Code are 0.6% if there are walls or non-structural elements which are not properly separated from the structural system (RCDF-93, 1993). None of the drift values computed exceeded this limit. However, the maximum values of these

parameters in several events became greater than 0.15%, which corresponds to the formation of diagonal cracks in masonry walls (Alcocer et al., 1999). Furthermore, the 94-3, 95-1, 99-1 and 99-3 events caused the values to exceed the implicit maximum drift of 0.3% in the complementary technical procedures of masonry structures of this code.

The first event that caused visible damage on structural and non-structural elements was denoted as 94-3, with a peak acceleration of  $124 \text{ cm/s}^2$  and a maximum inter-story drift value of 0.29%. Measured vibration frequencies, from ambient vibration test after the event and from seismic records of this event, in L-direction and torsion diminished. Similar results were observed due to the event 95-1, whose peak acceleration was  $130 \text{ cm/s}^2$ , and whose maximum inter-story drift values were 0.30 and 0.43% for the L and T directions, respectively. Since a small increment of the damage was observed, the vibration frequencies decreased slightly.

Through an increment of the vibration frequencies and a reduction in the inter-story drift values, the 97-1 event showed the repair effect of the stiffness of the building caused by the construction of steel braces.

It is interesting to compare the drift indexes in Table 4 due to earthquakes of the same intensity, which occurred before and after the second rehabilitation. Drifts of 97-1 event are more than two times less than those of 94-3. On the other hand, compared with those of events 95-1 and 99-3, whose intensities were similar and the greatest registered, the drifts for the 99-3 event grew to 0.54% in the T-direction and similarly in the L-direction. This behaviour is explained by the second rehabilitation of the building that produced changes in its dynamic properties, particularly the vibration frequencies. Furthermore, the fact that the peak inter-story drift was higher after the second rehabilitation, indicates that the building with the event 99-3 suffered damages that affected its stiffness and consequently reduced its vibration frequency in such a way that it practically coincided with the frequency of the soil, the one which varies between 0.48 and 0.54 Hz.

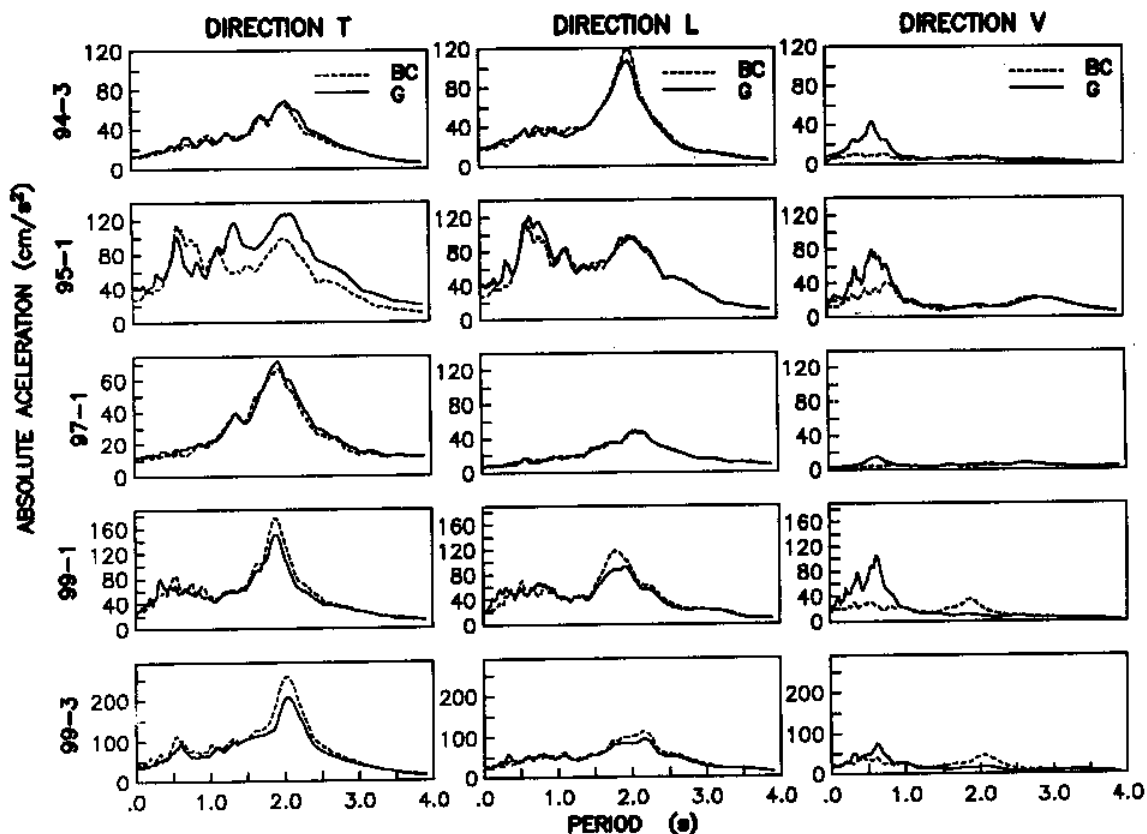


Fig. 11 Response spectra from the basement centre (BC) and from field site (G)

### 3. Response Spectrum

Response spectra, for 5% of critical damping, were calculated with the records of the field site and the centre of foundation in site BC. In Figure 11, for the greatest intensity events, the spectra of the three components (T, L and V) of these two measurement points are compared.

Except that of the 95-1 event, which contrasts with respect to the others by the fact that significant spectral amplitudes occur in the period range of 0.5-2.1 s, the response for the horizontal components have certain similarity, although the movements originated from different epicentral regions of the country, with maximum ordinates in the range of 1.9-2.1 s.

Moreover, the acceleration responses of the base of the building and of the field site are similar. On the other hand, in the vertical component there is observed at the base of the building a significant attenuation of the movements of the ground for periods lower than 1 s.

The envelope of the responses makes it clear that the significant motions of the horizontal components are located between 0.4 and 2.3 s, and of the vertical between 0.4 and 0.9 s.

### ANALYSIS OF THE RECORDS

The dynamic properties of the soil-structure system were calculated through an analysis of the earthquake records by using non-parametric and parametric estimation techniques.

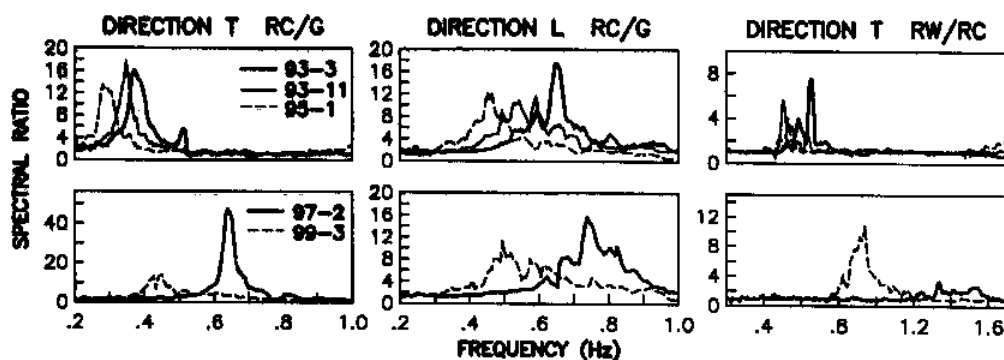


Fig. 12 Estimated transfer functions between the motions at roof centre and at field site (RC/G), and between the corner and the roof centre (RW/RC)

#### 1. Non-parametric Technique

In first instance, in spite of the fact that the spectral analyses based on conventional non-parametric method (Bendat and Piersol, 1986) are not very reliable for transient signals, especially for damping estimation (Beck and Beck, 1985), those were used. The Fourier spectra of the records and a selective transfer function (amplitudes and phase angles) and coherence functions between two records were obtained. The most relevant results were those obtained for the motions of roof, both transverse and longitudinal, and the field site motion, and for the motions at a corner and at the centre of the roof, while allowing for the determination of the torsional vibration (Muria-Vila and Rodríguez, 2001).

Figure 12 shows some estimated transfer functions obtained to illustrate the variations of the frequencies of three principal modes. The shapes of the curves and the intervals of maximum ordinates vary from one event to another. Furthermore, for some events, more than one significant peak appeared around a modal frequency. Therefore, the system exhibited non-linear behaviour even during seismic intensities considered as low or moderate.

An important aspect of the input ground motions is that the most of its energy is located essentially between 0.2 and 2.0 Hz. Thus, given the system characteristics and those of the soil, the three first modes were identified with an adequate resolution. On the other hand, the higher modes could not always be identified with an adequate resolution. Table 5 shows the identified frequencies and the damping of the system. In the cases where several significant peaks appeared, that of the smaller frequency was chosen. These frequency intervals presented in several cases are attributed to non-linear behaviour of the system

and to the quality of signals. The vibration frequencies derived from the seismic records were significantly smaller than those obtained from the ambient vibration tests.

The critical damping ratios for the fundamental vibration modes were estimated from transfer functions between the roof and field-surface. Values between 1 to 5% were obtained.

## 2. Parametric Technique

In order to understand the time variations of the dynamic parameters, an alternative description of linear systems was selected with the computer code MIMO (Li and Mau, 1990). This program is used for system identification with multiple input and output signals. It is based on the modal superposition method proposed by Beck and Jennings (1980) for which a structural system, represented as a coupled second order equations system, is transformed into a system of uncoupled equations, each equation having one time-dependent variable. Parameters of the system are determined by using the least squares method. The criterion function is minimized for the time window considered in the parametric analysis for which the system behaviour is assumed as linear. The parameters identified were the modal frequencies, damping ratios, modal participation factors and mode shapes.

**Table 5: The Lowest Frequencies and Damping Estimated with the Non-parametric (NPT) and Parametric (PT) Techniques**

Event	Mode	Frequencies (Hz)						Critical damping ratio (%)			
		Direction T		Direction L		Torsion		Direction T		Direction L	
		NPT	PT	NPT	PT	NPT	PT	NPT	PT	NPT	PT
93-3	1	0.37	0.39	0.65	0.63	0.60	0.54	3.1	3.3	2.9	4.2
	2	1.34	1.35	2.58	2.52	2.16	2.00		3.4		3.3
93-4	1	0.35	0.36	0.61	0.57	0.56	0.51	2.6	3.4	3.7	5.9
	2	1.29	1.27	2.66	2.34	1.89	1.77		4.0		4.0
93-11	1	0.35	0.34	0.57	0.51	0.53	0.44	2.8	2.8	4.3	5.3
	2	1.26	1.19	2.47	2.21	1.86	1.82		4.2		3.2
94-1	1	0.37	0.36	0.58	0.58	0.51	0.53	2.0	3.6	2.2	4.8
	2	1.31	1.27	2.22	2.27	1.94	1.94		3.3		3.1
94-3	1	0.29	0.31	0.45	0.46	0.43	0.44	3.4	3.4	4.5	6.9
	2	1.22	1.15	1.57	1.49	1.87	1.74		3.3		6.8
95-1	1	0.28	0.30	0.45	0.44	0.43	0.42	3.9	5.3	4.1	5.4
	2	1.16	1.03	1.94	1.29	1.77	1.50		3.8		3.4
95-2	1	0.29	0.30	0.45	0.44	0.46	0.42	4.5	4.1	4.2	5.0
	2	1.14	1.02	1.88	1.80	1.66	1.48		3.5		4.0
96-1**	1	0.34	0.33	0.48	0.51	0.48	0.49	4.2	4.5	3.2	5.2
	2	1.18	1.18	1.92	2.05	2.10	2.14		3.8		3.7
97-1**	1	0.60	0.61	0.56	0.59	1.15	1.07	3.2	2.9	4.2	5.0
	2	2.20	2.39	2.30	2.33	3.50	-		2.2		3.6
97-2*	1	0.64	0.66	0.74	0.73	1.25	1.11	1.1	2.4	3.2	4.7
	2	2.52	2.44	2.94	2.95	4.19	4.20		1.4		4.2
98-1*	1	0.63	0.66	0.74	0.77	1.29	1.25	1.5	1.8	2.6	2.9
	2	2.51	2.43	2.94	2.95	4.17	3.69		1.2		1.7
99-1*	1	0.54	0.48	0.52	0.53	0.99	0.87	4.0	4.1	5.2	5.8
	2	1.50	1.35	2.13	2.17	3.24	-		3.8		4.8
99-2*	1	0.54	0.53	0.59	0.60	1.01	0.98	3.3	3.7	4.4	5.1
	2	1.93	1.96	2.16	2.43	2.94	-		2.4		3.6
99-3*	1	0.45	0.43	0.52	0.49	0.92	0.77	3.6	7.2	4.4	7.1
	2	1.37	1.23	1.84	1.91	2.68	-		3.3		3.7

Second rehabilitation completed \*\* Second rehabilitation in process

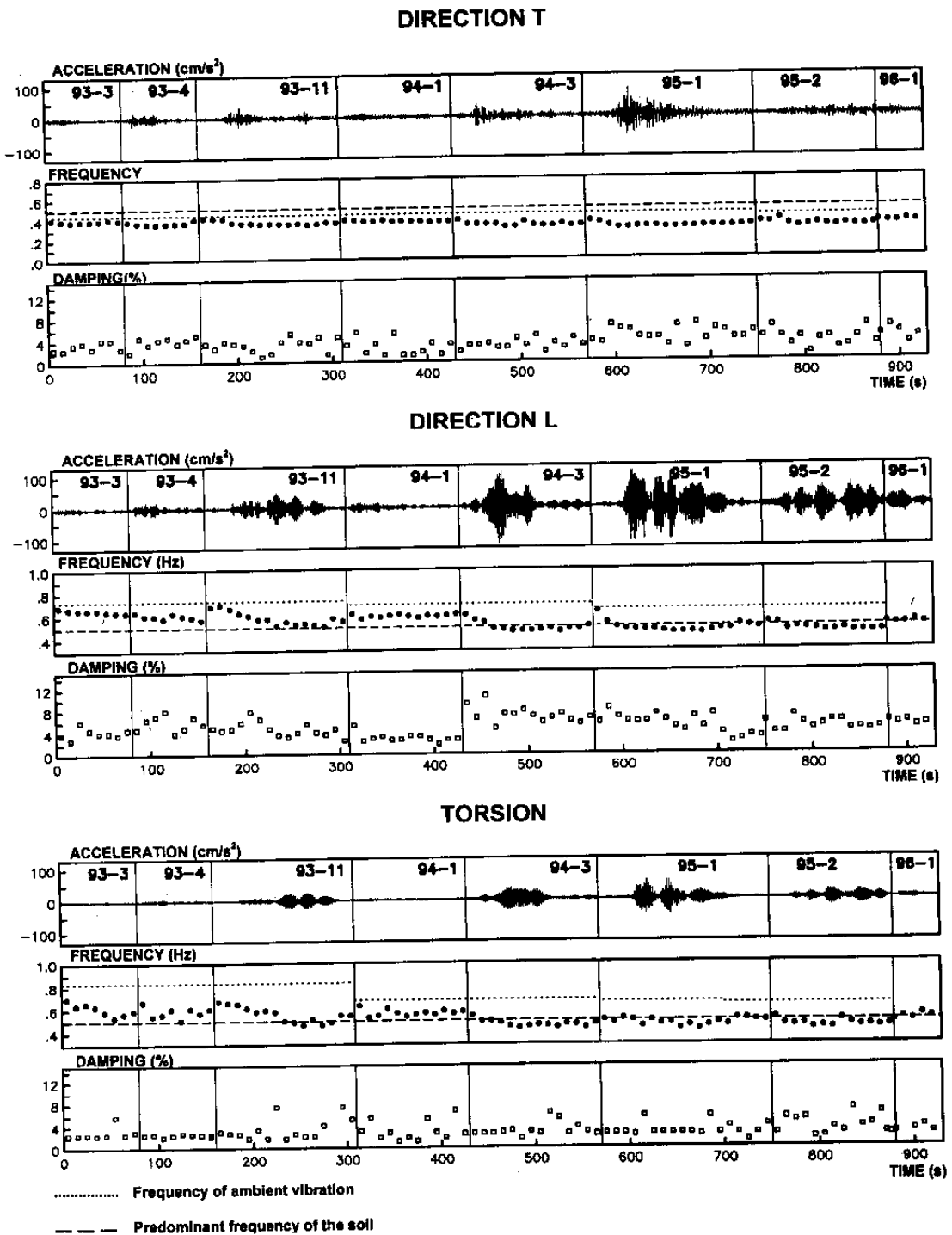


Fig. 13 Acceleration time-history recorded at RC site and time-variation of the dynamic properties of the events before the second rehabilitation

The estimation of structural parameters from seismic records was based on a simplified model (Durrani et al., 1994). The output signals used were the histories of translational and torsional acceleration responses recorded at three levels of the structure; the input signals were the field station translational acceleration time-histories. By using this model, the dynamic properties of L and T translational modes and the torsional modes of the structure can be computed.

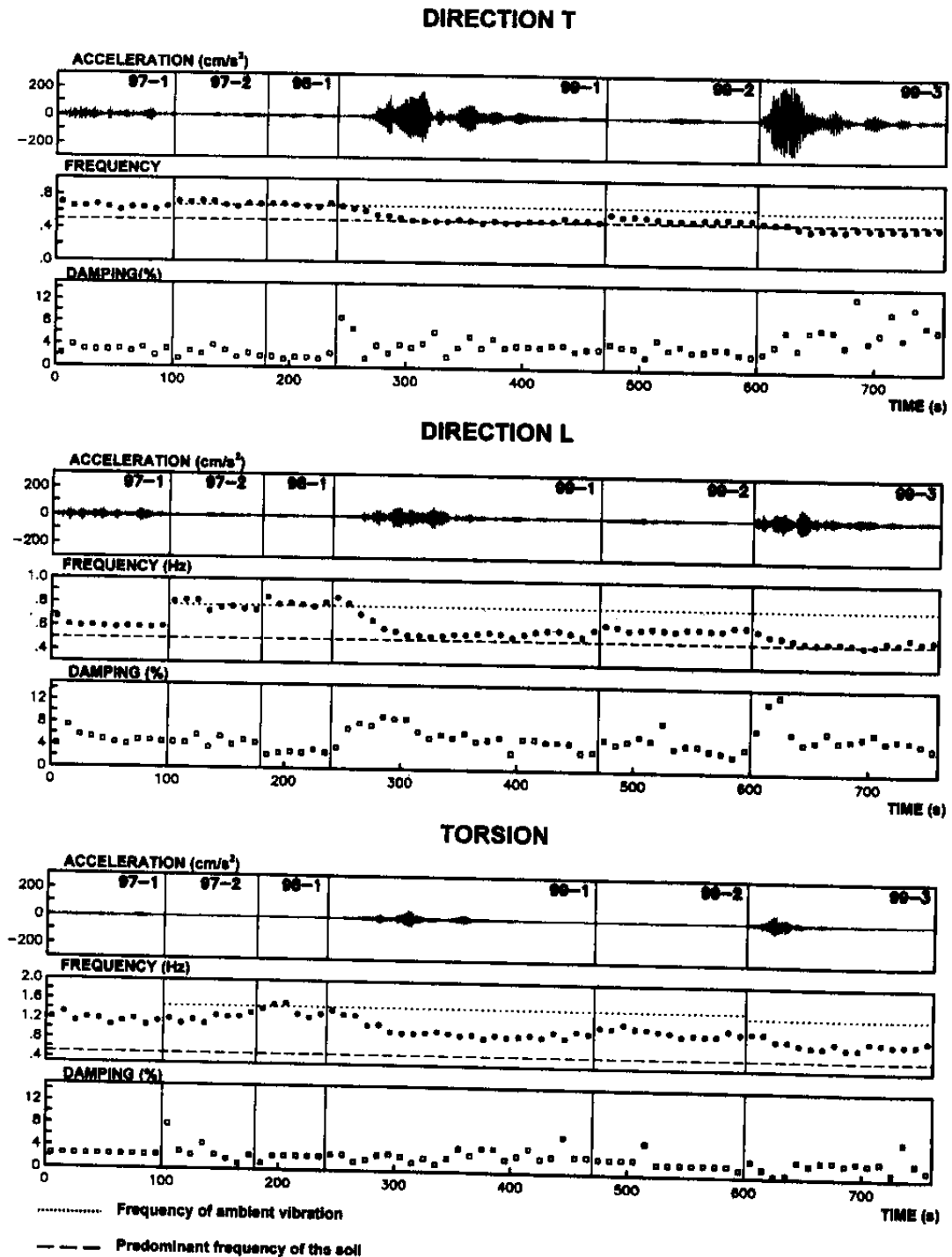


Fig. 14 Acceleration time-history recorded at RC site and time-variation of the dynamic properties of the events after the second rehabilitation

Based on these results, variations of the building dynamic properties under the records studied were assessed by dividing the records in 10 s segments and by applying the above-mentioned identification technique. For the first window, initial values for the optimisation in the estimation of the parameters were equal to those obtained from a spectral analysis. For the following segments, the estimate from the previous window was used.



For all cases, the identification of two modes was enough to reproduce the observed responses. The modal participation factors of these modes show that at the beginning of the intense phase of each event, the fundamental vibration mode participates more than 90%. Therefore, the computed responses are practically equal to the recorded ones. The reason for this is that the excitation-frequency content is around the dominant frequency of soil, and close to the first frequencies of the building.

Figures 13 and 14 show accelerograms recorded at a point located on the roof. The first modal frequency (dot) and damping (rectangle) are plotted versus the time corresponding to the middle of each window. The horizontal line represents the AV frequencies and the predominant frequency of the soil. It can be seen that values along the duration of each earthquake have an amplitude-dependence on the input motion. In the intense phase of the event, the major reduction of frequencies occurred, and after this phase of the most significant events, a partial recovery of the values of the frequencies was observed.

From the analysis of the events occurring in the period of 1993-1995, before the second strengthening of the building, the T-direction frequency varied from 0.41 to 0.30 Hz, which is considered very low for a building of this height, showing a high flexibility in this direction. These values were lower than the dominant frequency of the soil. The initial frequencies for the L-direction (0.69 Hz) and for torsion (0.70 Hz) varied to values close to the prevailing frequency of the soil. The earthquakes that produced significant decreases of the frequencies of the system (events 93-11, 94-3, 95-1 and 95-2) show, in the acceleration records of the building, a beating effect due to the proximity of these frequencies to the dominant frequency of the soil.

At the end of 1995, the second rehabilitation of the building was started. During the repair work, events 96-1 and 97-1 were recorded. The first one occurred when only the construction of the structural braces was completed at level denoted as E3, and the second occurred when almost all the braces were installed, practically at the end of the rehabilitation tasks. The estimated frequencies from event 96-1 already reflected the contribution of the brace stiffening. Once the rehabilitation was finished, values of vibration frequency significantly increased, especially in torsion and T-direction. The frequencies of sixth AV test (6AV) increased 1.6, 1.2 and 2.3 times compared with 5AV test for components T, L and torsion, respectively.

The system also showed that the SSI effects and the non-linear response of the building under records of the 99-1 and 99-3 events reduced the frequency values and gave rise to resonance problems for the transversal and longitudinal modes. This fact led in T-direction to more important amplifications of the input vibration than those before the strengthening.

The lowest values of frequencies of each event identified from the non-parametric (NPT) and parametric (PT) techniques are compared in Table 5. The estimated results are quite similar. After each rehabilitation of the building, a clear process of stiffness degradation of the system was observed. This fact is attributed to cumulative effects of several earthquake motions during the two periods of observation. The vibration frequency tends to decrease for increasing intensities of the motions or drift indexes, as is shown in Figure 15. In this figure, after the second rehabilitation, the influence of the braces in the T-direction frequency is evident.

Before the second rehabilitation, the maximum reductions in the fundamental frequency of lowest PT values between the first event (93-3) and the 95-2 event were 23, 30 and 22% in T, L and torsion, respectively. However, 5 to 11% of these reductions have been recovered at the end of the 95-2 event. When the second rehabilitation was completed, the reductions between events 97-2 and 99-3 were 35, 33 and 31%, respectively. In this observation period, 13 to 47% of reductions have been recovered at the end of the event 99-3.

These decreases of frequencies suggest apparent losses of stiffness of the system. If it is assumed that the masses have not changed, these losses can be considered from the variations of the fundamental frequencies between two events. In the first case, for those changes that occurred previous to the second rehabilitation, the smaller frequencies, estimated by using the analysis by windows, of the events 93-11 (not-visible damage), 94-3 (visible damage), 95-1 (high intensity record) and 95-2 (visible damage), were compared with respect to the first registered event (93-3). In the second case, for events that occurred after the second rehabilitation i.e. 99-1 (visible damage), 99-2 (not-additional damage) and 99-3 (high intensity record), results are compared with respect to the event 97-2. The results presented in Table 6 show large apparent losses of the stiffness. In the first case, the highest value corresponds to L-direction (51%) and in the second case, to the T-direction (58%).

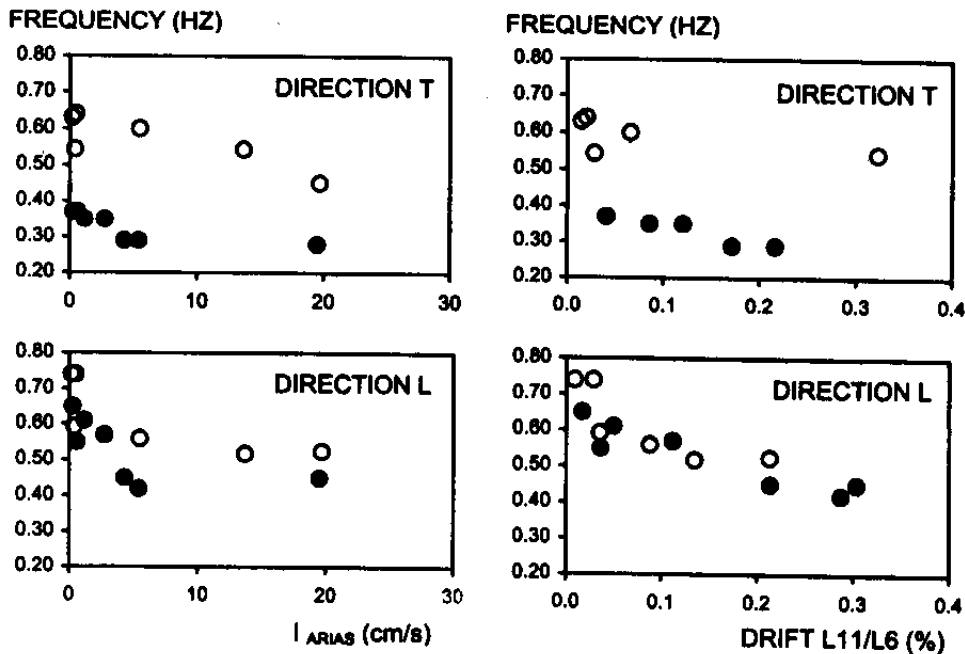


Fig. 15 Arias intensity ( $I_{Arias}$ ) and inter-story drift variation of soil-structure system frequencies (• before the second rehabilitation, ○ after the second rehabilitation)

Table 6: SSI Effect Analysis Assessed with Simplified Model

Event	Direction T						Direction L						Torsion			
	$X_0$	$H\phi$	$f_1$	$f_i$	$f_R$	$f_H$	$X_0$	$H\phi$	$f_1$	$f_i$	$f_R$	$f_H$	$\frac{\phi_{1t}}{\phi_1}$	$\bar{f}_{1t}$	$f_{1t}$	$f_t$
	$X_T$	$X_T$	(Hz)	(Hz)	(Hz)	(Hz)	$X_T$	$X_T$	(Hz)	(Hz)	(Hz)	(Hz)	(%)	(Hz)	(Hz)	(Hz)
2AV	5	8	0.44	0.47	1.61	1.76	6	24	0.73	0.88	1.48	2.53	4	0.83	0.85	3.76
93-3	1	9	0.37	0.39	1.26	2.69	3	21	0.65	0.75	1.40	3.07	5	0.60	0.62	2.31
93-4	2	8	0.35	0.37	1.31	2.41	4	19	0.61	0.70	1.39	2.74	4	0.56	0.58	2.32
93-11	2	7	0.35	0.37	1.33	2.16	5	17	0.57	0.65	1.38	2.14	5	0.53	0.55	1.97
3AV	3	15	0.44	0.49	1.18	2.14	5	18	0.70	0.81	1.65	2.61	3	0.68	0.70	3.30
94-1	2	8	0.37	0.39	1.32	2.59	3	15	0.58	0.65	1.51	3.06	5	0.51	0.52	1.95
94-3	2	6	0.29	0.30	1.19	1.72	3	11	0.45	0.49	1.37	2.06	2	0.43	0.44	2.48
4AV	5	18	0.44	0.50	1.08	1.75	8	19	0.64	0.68	1.47	1.87	3	0.65	0.67	2.91
95-1	2	6	0.28	0.29	1.18	1.72	2	13	0.45	0.49	1.26	2.57	3	0.43	0.43	2.16
95-2	2	6	0.29	0.30	1.21	1.78	1	15	0.45	0.49	1.33	3.40	1	0.46	0.46	3.36
96-1**	2	7	0.34	0.37	1.36	2.11	2	8	0.48	0.51	1.66	3.14	2	0.48	0.48	3.03
97-1**	6	26	0.60	0.73	1.20	2.06	3	12	0.56	0.61	1.62	2.77	6	1.15	1.18	3.91
6AV*	6	42	0.68	0.95	1.09	2.35	7	29	0.78	1.00	1.45	2.44	5	1.47	1.52	5.52
97-2*	5	29	0.64	0.79	1.22	2.43	3	20	0.74	0.85	1.65	3.44	9	1.25	1.34	3.43
98-1*	6	28	0.63	0.78	1.22	2.15	3	21	0.74	0.86	1.61	3.38	11	1.29	1.41	3.20
99-1*	2	25	0.54	0.63	1.13	3.15	5	14	0.52	0.58	1.38	1.93	4	0.99	1.03	4.12
99-2*	4	22	0.54	0.63	1.20	2.43	3	17	0.59	0.67	1.44	2.82	14	1.01	1.14	2.24
7AV*	5	38	0.64	0.83	1.07	2.50	6	29	0.78	1.00	1.43	2.59	11	1.37	1.49	3.48
99-3*	4	17	0.45	0.51	1.12	2.00	3	18	0.52	0.59	1.23	2.70	17	0.92	1.05	1.86

\* Second rehabilitation completed

\*\* Second rehabilitation in process

Since structural response is sensitive to the input amplitude, in order to approximately assess the permanent losses of the structure's stiffness during the main events of Table 6, the fundamental frequency changes were derived in three ways: (a) from the first seismic event of the two periods of observation (93-3 or 97-2) and the frequencies of the last windows of the more significant events, (b) from those

seismic events of small and similar intensity among which one of moderate intensity occurred, and (c) from the AV tests between two measurement tests.

The calculated values of the permanent losses of stiffness are shown in Table 7. Only for the events 93-11 and 99-1, there is sufficient information to estimate the losses in the three ways. It can be seen that the first two ways of estimation provide consistent results. However, the comparisons of these losses with the estimations from the data of the AV tests, show in the T and L directions that AV losses were much less, and in the torsion, for the events before the second rehabilitation, these were much more than those obtained from the other ways.

On comparing the torsion frequencies from the AV tests or the seismic events that happened before the second rehabilitation, a great sensitivity to the small excitation amplitudes is seen. This fact is attributed essentially to the masonry walls. It should be remembered that these structural elements were only replastered before the damages suffered by the large earthquake of September 1985 ( $M_s = 8.1$  and  $I_{Arias} = 376$  cm/s). After the second rehabilitation, this variability in the torsional response of the building diminished considerably due to considerable contribution of the braces to the rotational stiffness of the building.

**Table 7: Apparent Loss of Stiffness between Two Seismic Events, in Percentage**

Event	Between events	Direction T	Direction L	Torsion
Before 2 <sup>o</sup> rehabilitation				
93-11	93-3 and 93-11	24	34	34
94-3	93-3 and 94-3	37	47	34
95-1	93-3 and 95-1	41	51	37
95-2	93-3 and 95-2	41	51	40
After 2 <sup>o</sup> rehabilitation				
99-1	97-2 and 99-1	47	47	39
99-3	97-2 and 99-3	58	55	52

The damages produced by the event 93-11 were reflected as 15 and 21% of stiffness reductions in the L-direction and torsion, respectively; even though no visible damage was detected from subsequent inspection. The damages produced by the events 93-11 and 94-3 on the masonry walls located in the L-direction were so great that when the event 95-1 occurred, with an intensity 3.6 times greater than the event 94-3, the losses increased slightly.

The losses assessed after the 99-1 and 99-3 events showed an important reduction in stiffness. For the T-direction, light damage was observed from the visual inspection. On the other hand, for the L-direction, the assessed losses were consistent with the losses of the first period of observation.

In all cases presented in Table 7, two facts stand out. First fact is that although there is a recovery of loss of stiffness at the end of each one of the seismic events, the approximate permanent losses accumulate and reach values of 37 to 49% and of 30 to 51% before and after the second rehabilitation, respectively. The second fact is that the increments of the losses from the AV tests are smaller than those estimated from the seismic records.

The computed damping varied in L-direction from 1 to 14%. The damping values show approximately an amplitude-dependency with respect to the input motions. This trend can be explained by the deterioration of the masonry and concrete elements. In the process of crack formation and while the old fracture planes are activated, the damping values are high. When the cracks opened up and their faces slipped relative to one another, the damping values decreased. The total process depends on the friction between crack faces, the irregularities of fracture plane and the fluctuations of motion amplitude. In T- direction, where there was a minor density of masonry walls, the damping was smaller than in the other direction. The damping ratio average in PT case turned out to be bigger than that obtained in the NPT case, particularly in L-direction (Table 5).

## SOIL-STRUCTURE INTERACTION EFFECTS

With the purpose of having an approximate idea of the contribution of the SSI effects on the response of the building, identifying the vibration frequencies of the fixed-base building, and of exploring if the variations of the frequencies of the system would be correlated with these effects, a simplified model has been used (Luco, 1980; Luco et al., 1987).

The fundamental translation vibration frequency of the soil-structure system  $\bar{f}_1$  can be written as

$$\frac{1}{\bar{f}_1^2} = \frac{1}{f_1^2} + \frac{1}{f_R^2} + \frac{1}{f_H^2} \quad (1)$$

where  $f_1$  is the frequency of the fixed-base structure, and  $f_R$  and  $f_H$  are the rocking and horizontal translation frequencies of the rigid structure. The values of these frequencies can be obtained from

$$f_R = \bar{f}_1 \left( \gamma_1 \frac{H\phi}{X_T} \right)^{-1/2} \quad (2)$$

$$f_H = \bar{f}_1 \left( \beta_1 \frac{X_0}{X_T} \right)^{-1/2} \quad (3)$$

where  $X_T$  is the total response at roof of the building,  $X_0$  is the translational response of the base,  $\phi$  is the angle of rocking base,  $H$  is the total height of the structure, and  $\gamma_1$  and  $\beta_1$  are modal parameters of the first mode of the fixed-base structure.

Substitution from Equations (2) and (3) into Equation (1) leads to

$$f_1 = \bar{f}_1 \left( 1 - \gamma_1 \frac{H\phi}{X_T} - \beta_1 \frac{X_0}{X_T} \right)^{-1/2} \quad (4)$$

By means of these equations and the translation and rocking response contributions in the total record (as is shown in Figure 10 for events 95-1 and 99-3), and the fundamental frequencies of the system ( $\bar{f}_1$ ) obtained in the previous section, the SSI effects in T and L-directions can be analysed. The  $X_T$ ,  $X_0$  and  $H\phi$  response quantities have been established in terms of Fourier amplitudes at the predominant frequencies of each event.

The fundamental torsional frequency of the system ( $\bar{f}_{11}$ ) can be written as

$$\frac{1}{\bar{f}_{11}^2} = \frac{1}{f_{11}^2} + \frac{1}{f_t^2} \quad (5)$$

where  $f_{11}$  is the torsional frequency of the fixed-based structure and  $f_t$  is the torsional frequency defined in terms of torsional stiffness of the structural base. These frequencies can be calculated as

$$f_t = \bar{f}_{11} \left( \beta_1 \frac{\phi_{bt}}{\phi_t} \right)^{-1/2} \quad (6)$$

$$f_{11} = \bar{f}_{11} \left( 1 - \beta_1 \frac{\phi_{bt}}{\phi_t} \right)^{-1/2} \quad (7)$$

where  $\phi_{bt}$  is the rotation of the base,  $\phi_t$  is rotation of the roof, and  $\beta_1$  is modal parameter of first torsional mode of fixed-base structure.

The rotation responses  $\phi_{bt}$  and  $\phi_t$  are obtained from the difference of the records obtained at the east and west corners of the basement and the top level in T-direction and divided by the distance between these points.

In Table 8, the results obtained for each seismic event are shown. The results of the events occurring before and after the second rehabilitation, as well as those in which the structure suffered visible damages (events 94-3, 95-1, 95-2, 99-1 and 99-3), are distinguished.

**Table 8: Approximate Estimation of Loss of Stiffness between Two Seismic Events or Ambient Vibration Tests, in Percentage**

Event	Between events	Direction T	Direction L	Torsion
<b>Before 2° rehabilitation</b>				
93-11	93-3 and 93-11	15	21	24
	2AV and 3AV	0	8	33
	93-3 and 94-1	15	15	4
94-3	93-3 and 94-3	28	37	24
	2AV and 4AV	0	23	39
95-1	93-3 and 95-1	37	40	16
	2AV and 5AV	4	21	41
95-2	93-3 and 95-2	37	49	37
<b>After 2° rehabilitation</b>				
99-1	97-2 and 99-1	40	32	19
	6AV and 7AV	13	0	14
	97-2 and 99-2	36	32	22
99-3	97-2 and 99-3	51	45	30

The contributions of the translation and rocking of the base in the total translation response on the roof of the structure can be assessed approximately by using the quotients  $X_0/X_T$  and  $H\phi/X_T$ , respectively. The sum of these two quotients represents the total contribution of the SSI effects on the translation response of the structure. The average total contribution in L-direction was 20% and shows no difference before and after the rehabilitation. On the other hand, the total contribution for T-direction was 9 and 28% before and after rehabilitation, respectively. These values reflect the changes suffered by the structure in the T-direction. In all cases, the rocking effects represent an average of 80% of the total contribution ( $X_0/X_T + H\phi/X_T$ ). The contributions of the movements assessed from the ambient vibration records were greater than those in case of the seismic records. For the events before the second rehabilitation, it can be perceived that the total contributions diminished with respect to the two initial events of small intensity from 22 to 15%, in L-direction. This reduction is attributed to the damage of the masonry walls, which when losing lateral stiffness, produced greater structural deformations.

With the three most intense events before rehabilitation, a decrease of the frequency of the structure ( $f_1$ ) was detected, as justified by the damages caused. This took place mainly during the event 94-3 and in the L-direction where the masonry walls are located.

The relationship between the frequencies of the structure and those obtained for the system ( $f_1/\bar{f}_1$ ) is another indication of the SSI effects. In the T-direction, the mean ratio value obtained previous to the rehabilitation was 1.06 before the damage and 1.04 after the damage produced by the event 94-3. For the rehabilitated structure, this value changed from 1.24 to 1.15, while the first damage event was 99-1. In the L-direction, this value changed from 1.14 to 1.09, and for rehabilitated structure, from 1.16 to 1.13. The SSI effects, which were more meaningful in the response of the rehabilitated building, were more evident in the T-direction.

**Table 9: Variation of Vibration Frequencies of the System between Two Seismic Events or Ambient Vibration Tests, in Percentage**

Event	Between events	Direction T				Direction L				Torsion		
		$\overline{\Delta f}_1$	$\Delta f_1$	$\Delta f_R$	$\Delta f_H$	$\overline{\Delta f}_1$	$\Delta f_1$	$\Delta f_R$	$\Delta f_H$	$\overline{\Delta f}_{1t}$	$\Delta f_{1t}$	$\Delta f_t$
Before 2 <sup>o</sup> rehabilitation												
93-11	93-3 and 93-11	-5	-5	6	-20	-12	-13	-1	-30	-12	-11	-15
	93-3 and 94-1	0	0	5	-4	-11	-13	8	0	-15	-16	-16
94-3	93-3 and 94-3	-22	-23	-6	-36	-31	-35	-2	-33	-28	-29	7
95-1	93-3 and 95-1	-24	-26	-6	-36	-31	-35	-10	-16	-28	-31	-6
95-2	93-3 and 95-2	-22	-23	-4	-34	-31	-35	-5	11	-23	-26	45
After 2 <sup>o</sup> rehabilitation												
99-1	97-2 and 99-1	-16	-20	-7	30	-30	-32	-16	-44	-21	-23	20
	97-2 and 99-2	-16	-20	-2	0	-24	-33	-1	16	-19	-15	-35
99-3	97-2 and 99-3	-30	-35	-8	-18	-30	-31	-25	-22	-26	-22	-46

- $\overline{\Delta f}_1$  variation of translation frequency of the system  
 $\Delta f_1$  variation of translation frequency of the structure  
 $\Delta f_R$  variation of rocking frequency of the base  
 $\Delta f_H$  variation of translation frequency of the base  
 $\overline{\Delta f}_{1t}$  variation of torsion frequency of the system  
 $\Delta f_{1t}$  variation of torsion frequency of the structure  
 $\Delta f_t$  variation of torsion frequency of the base

The above-mentioned data shows great sensitivity of the SSI effects to the intensity of the input motion and to the changes in the structural characteristics following both deterioration and rehabilitation work. In order to analyse this behaviour for the four most intense events, the variations of the fundamental frequencies of the system and of the structure with the Fourier analysis of moving windows (Udwadia and Trifunac, 1974) have been estimated. The length of the time window is taken as 10 s. Figure 16 presents the frequencies  $\overline{f}_1$ ,  $f_1$  and  $f_R$  obtained from the moving window analysis during these events. The  $\overline{f}_1$  and  $f_1$  values show the amplitude-dependence. Furthermore, the relationship  $f_1/\overline{f}_1$  also shows the amplitude-dependence during an event; this is very clear for the event 99-1. The values of  $f_H$  (not given here) and  $f_R$  do not have a clear tendency, and their frequencies ranges are 0.90-3.70 Hz and 1.00-1.70 Hz, respectively.

The characteristics of the SSI effects of torsion of the building are also presented in Table 8. The analysis of this data leads to the following observations. The contribution to the torsion response, as associated with the SSI effects, can be estimated approximately with the help of  $\phi_w/\phi_t$  quotient. Before the rehabilitation, the contribution of the SSI effects to the total torsion response is lower than 5%. Relationship of the torsional frequencies of the structure and those of the system before the rehabilitation, obtained from ambient vibration as well as from seismic records, varied from 1.00 to 1.03. This means that the structure behaved as if it was practically fixed at the base. After the rehabilitation, the contributions of the total mean values to the response were 10% approximately, and the relationship values of the torsional frequency of the structure and that of the system were from 1.08 to 1.10.

With this adopted simplified procedure, the reductions of the frequencies of vibration of the system ( $\overline{\Delta f}_1$ ), of rocking ( $\Delta f_R$ ), of translation ( $\Delta f_H$ ), and of the structure ( $\Delta f_1$ ) during the most intense events with regard to the first registered event can be analysed approximately (Table 9). When comparing the reductions of  $\overline{\Delta f}_1$  and  $\Delta f_1$ , it is observed that those are similar. This indicates that the variations of the

frequencies of rocking and translation have a little influence on the variation of the frequency of the system; this suggests that it is feasible to estimate the losses of stiffness from the variations of the frequencies of the system. This also holds for the variations in the frequencies of torsion of the system ( $\Delta f_{tr}$ ) and of the structure ( $\Delta f_{st}$ ).

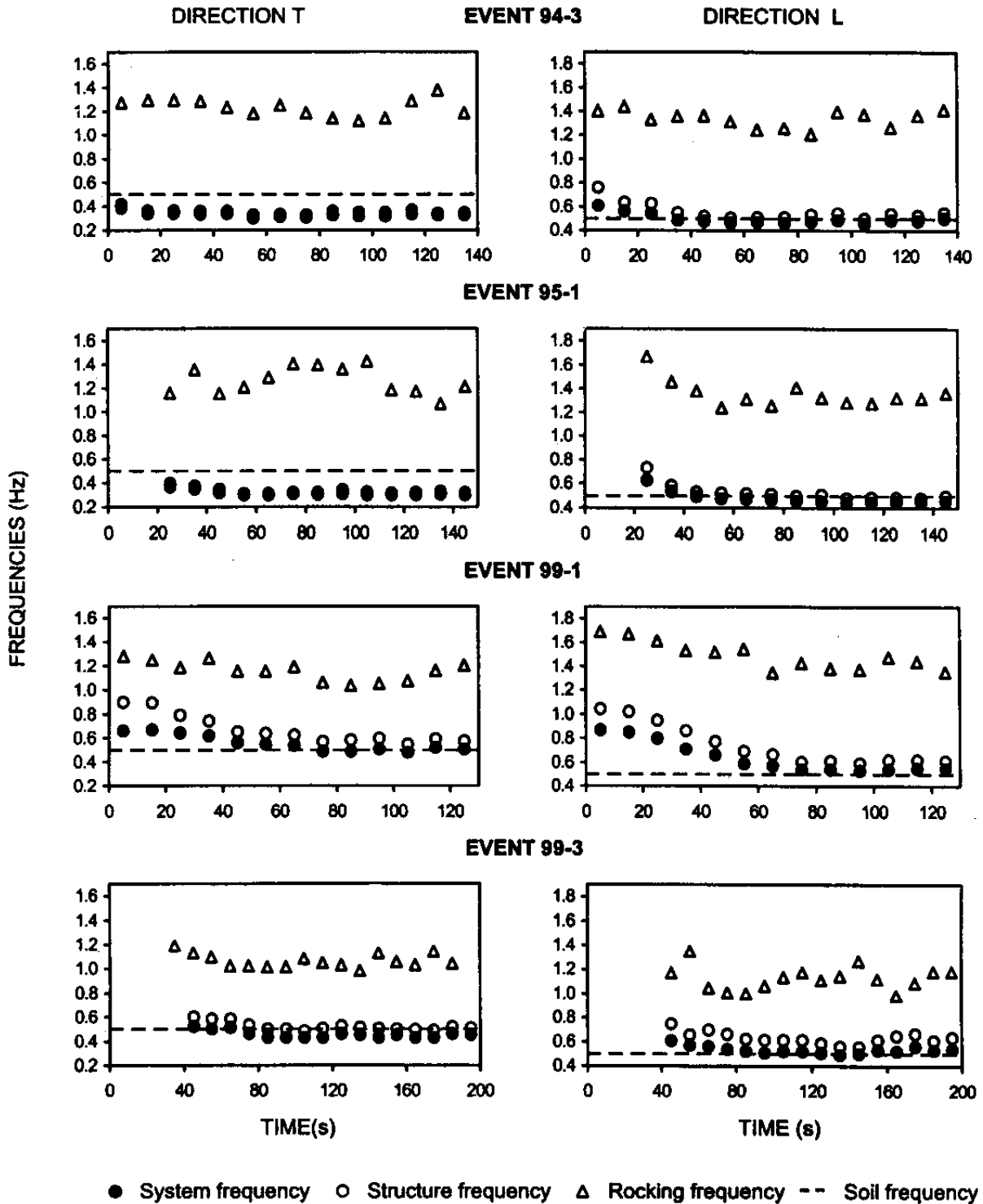


Fig. 16 Time variations of frequencies, before second rehabilitation (94-3 and 95-1 events) and after second rehabilitation (99-1 and 99-3 events)

## DISCUSSION AND CONCLUSIONS

The analysis of the building response in a seven-year period leads to the following observations and conclusions.

The values of the lowest modal frequencies of the soil-structure system identified for each event from the non-parametric and parametric techniques were approximately similar. The average damping ratio of parametric technique turned out to be larger than that obtained from the non-parametric technique.

It can be seen that the frequency and damping values along the duration of each earthquake are amplitude-dependent. After the intense phase of the most significant events, a partial recovery of frequencies is observed. Before the second rehabilitation, the maximum reductions in the fundamental frequency values, after considering the recovery, were 21, 29 and 20% in T, L and torsion components, respectively. When the second rehabilitation was completed, the total accumulated reductions were 30, 26 and 16%, respectively. The damping values show a certain amplitude-dependency but without a clear tendency. The identified damping varied in L-direction from 1 to 14%. In T-direction, where there is a minor density of masonry walls, the damping was smaller than the L-direction.

In all cases, it was possible to reproduce the recorded response with the parametric estimation technique by including only the first two vibration modes. The fundamental mode participation is more than 90%. This is because the energy content of the input ground motion is located essentially around the dominant frequencies of the soil (0.48-0.54 Hz) which are close to the fundamental frequencies of vibration of the building.

The SSI effects show a sensitivity to the intensity of the input ground motions and to the variations in the structural characteristics from deterioration and from the increments of stiffness contributed by the new structural elements. In particular, this is very substantial in the T-direction. The rocking and translation of the base of the building founded on soft clay have moderate effects on the overall displacements, before the second rehabilitation. However, after the rehabilitation, the contribution became significant in T-direction (about 28% of the roof response). In all the cases, the contributions of the rocking are bigger than those of the translation.

At this stage of the analysis, it is difficult to assess whether the foundation-soil interface contributes to the non-linear behaviour of the system. Its participation is not ruled out, but for the observed involvement of the movement of the base in the displacement response, it is anticipated that this is small. Thus, these non-linear effects were probably not relevant. With respect to the dynamic properties of the soil, it is known that those do not change with the amplitude of motion, due to the very extended range of linear behaviour which characterizes the Mexico City clays.

It was found that the dynamic response of the structure is very sensitive to the amplitude of the imposed ground motion. The main factor affecting the vibration frequencies was the non-linear behaviour of the building, even for rather small levels of excitation. Stresses induced during the ambient vibration tests were much smaller than those generated by earthquakes, and therefore, corresponded to greater stiffness of the structure. The non-linear behaviour of the building can be attributed essentially to stiffness changes due to different deterioration magnitudes of structural and non-structural elements, and the rehabilitation of the building. The most clear evidence of non-linear behaviour is obtained by computing the variations of the natural frequencies of the building during the duration of a specific event. Moreover, the successive low and moderate intensity earthquakes produced continuous reduction of vibration frequencies.

The variations of the structural parameters of the building can be attributed to different sources of non-linearity. In the first case, before the second rehabilitation, it was due mainly to the reduction in the participation of the masonry infills in the lateral stiffness of the building and partly to the reduction in the slope of the stress-strain curve of concrete with increasing stress levels, some sliding between the interface of concrete infill walls and frames, cracking of the floor system, and degradation effects of the structural and non-structural elements under several seismic motions. Thus the reduced contribution of the masonry and concrete infill walls of the longitudinal façade produced a decrease in the longitudinal and torsional stiffness of the building.

After the second rehabilitation, the deterioration in the interfaces between the new and old structural elements of the transversal façades caused a considerable observed reduction of lateral stiffness. This source of non-linear behaviour and those mentioned before contributed to the appreciable decrease of the vibration frequencies.



The ambient vibration tests and the analysis of the seismic records of the building show the influence of mortar and plaster cover in columns and walls on the frequencies values. It is the usual practice to make this kind of repair immediately after earthquake damages are detected. This is one of the aspects that do not allow making a reliable estimate of the loss of effective stiffness between two ambient vibration tests.

The drift indices between instrumented stories, as calculated from the records of the low and moderate intensity events, show that they were smaller than those commonly associated with structural damage. This suggests that the damage is related with the damage accumulated in the elements of concrete and masonry under the effects of several seismic events. Before the second rehabilitation, the event 95-1 produced a peak inter-story drift of 0.30 and 0.43% for the L and T-directions, respectively. Compared with those, during event 99-3 after the second rehabilitation, whose intensities were analogous and the greatest registered, the drift grew to 0.54% in the T-direction and similarly in the L-direction. The fact that the peak inter-story drift was higher after the second rehabilitation under the same severity of input ground motion, indicates that during the event 99-3, the building suffered damages that affected its stiffness and consequently reduced its vibration frequency in such a way that it practically coincided with the frequency of the soil.

After the second rehabilitation, the approximate losses of stiffness from earthquakes were bigger than 30%. However, from the visual inspection in T-direction, only light damages were detected. On the other hand, for the L-direction, the assessed losses were consistent with the losses observed in the building prior to the second rehabilitation.

The use of braces to provide stiffness and strength to structures has demonstrated in laboratory tests and several practical cases its feasibility in the rehabilitation of flexible structures. However, for the case of the studied building, a significant deterioration in stiffness was observed that has taken the building to a state of quasi-resonance with the predominant frequencies of the soil. In consequence, the response of the building during earthquakes of moderate intensity has shown a faulty behaviour again.

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