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SEISMIC RESPONSE OF A TALL BUILDING IN MEXICO CITY

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ABSTRACT

The seismic behavior of an instrumented tall building founded on soft soil in Mexico City to 6 earthquakes is presented. Estimations of the structural parameters were carried out based on the time histories of the ground motion and on the structural response. The results have shown that natural frequencies change with the intensity of the motion. For the first 4 events considered this fact can not be attributed to visible structural damage, but to an early non-linear behavior of the building, affected by loss of the contribution of non-structural components to the lateral stiffness. In the last 2 events a light structural damage was found. The accuracy of different analytical models of the structure and of the soil-structure interaction are discussed.

INTRODUCTION

This work is part of the Institute of Engineering of the National University of Mexico and the Technical University of Milan joint research program. It started in 1991 (Meli et al., 1996) and the aim of the project is to record and study the seismic response of an instrumented building and its subsoil.

This study presents, in the first part, a synthesis of the analysis of the building records obtained from 1993 to 1995. The dynamic properties of the building obtained from the these records, and those obtained from 6 ambient vibration tests are compared. In the second part, the analyses of the linear analytical models, that reproduce with high accuracy the seismic structural response of the earthquake records, are discussed.

THE BUILDING

The building studied is a 14-story reinforced concrete structure quite regular in plan and elevation. It has a waffle slab floor system. It suffered moderate structural damage during the 1985 earthquake and it was later strengthened by adding infill reinforced concrete walls in the longitudinal direction and jacketing the columns. There was also evident cracking in the slabs revealing positive and negative yield lines. No appreciable settlement of the foundation was observed. The structure was then strengthened as previously described, without any modification to the foundation. The building has a basement and a box-type foundation on 54 friction piles. Geotechnical investigations were performed which showed that the soil profile at the building site is typical of the lake-bed of Mexico City. The depth of firm soil is 38.5 m and the soft soil strata is 31 m. A full description of the building is presented by Meli et al. (1996).

AMBIENT VIBRATION TESTS

In order to determine the main changes of the dynamic characteristics of the building, ambient vibrations were measured.

A set of eight one-directional accelerometers of high resolution was used in eleven different setups. Each one was designed to identify a particular vibrational mode or characteristics of the vibration of soil-structure system. Acceleration time histories were recorded for 13.65 min during each test. The records were digitized and filtered out with a cut frequency of 30 Hz. Using well established procedures based on random vibration theory (Bendat and Piersol, 1986), power spectra averages, transfer functions and coherence functions were calculated.

Ambient vibration tests in the building were carried out in 6 stages. The first 3 stages (June 1990, October 1991 and September 1992) showed that the dynamic properties did not change. From the spectral analysis, natural frequencies and modal shapes were derived, as shown in Table 1 and Fig. 1, respectively.

After the small and moderate intensity earthquakes changes in the structural properties of the building were produced (Table 1). The natural frequencies, after the 1993 earthquakes, of the translation components remained practically unchanged, while the rotational component decreased 22 percent; but the last two earthquakes (events 94-3 and 95-1) changed frequencies in as much as 55 percent.

A modal coupling between the fundamental torsional vibration and longitudinal translation modes was detected. The first mode equivalent damping ratios of the building varied between 0.02 to 0.04.

Table 1. Experimentally identified parameters

Event	Date	Mode	Frequencies (Hz)			A_{max} (cm/s^2)		γ^L_{max} (x10^{-3})	γ^T_{max} (x10^{-3})
			L	T	R	SOIL	ROOF		
		1	0.73	0.44	0.83				
AV3	23/Sep/92	2	3.12	1.60	2.54	<0.1	<0.1	--	--
		3	5.95	2.93	4.30				
		1	0.65	0.37	0.57-0.67				
93-3	15/May/93	2	2.58	1.34	2.16-2.28	4	11	0.21	0.46
		3	4.81	2.47	3.50-3.80				
		1	0.61	0.35	0.51-0.60				
93-4	15/May/93	2	2.66	1.29	1.89-1.97	11	28	0.76	0.94
		3	4.72	2.25	3.30-3.48				
		1	0.57	0.35	0.50-0.55				
93-11	24/Oct/93	2	2.47	1.26	1.86-1.92	13	56	1.42	1.32
		3	4.67	2.20	3.02-3.20				
		1	0.70	0.44	0.68				
AV4	25/Nov/93	2	2.64	1.45	2.31	<0.1	<0.1	--	--
		3	4.91	2.56	3.81				
		1	0.55	0.37	0.51-0.59				
94-1	23/May/94	2	2.22	1.31	1.94	7	19	0.44	0.45
		3	4.52	2.30	3.28				
		1	0.42-0.53	0.29-0.31	0.43-0.51				
94-3	10/Dec/94	2	1.57-2.32	1.22	1.87-2.45	17	124	3.45	2.40
		3	3.29-4.65	2.12-2.19	3.25-3.39				
		1	0.64-0.66	0.44	0.65-0.68				
AV5	4/Feb/95	2	2.50-2.54	1.43-1.47	2.25-2.30	<0.1	<0.1	--	--
		3	4.83	2.59	3.64-3.86				
		1	0.45	0.28	0.43				
95-1	14/Sep/95	2	1.94	1.16	1.77	37	130	3.76	4.71
		3	3.82	1.67	3.31				
		1	0.66	0.42-0.44	0.64-0.66				
AV6	23/Sep/95	2	2.51	1.42-1.47	2.20-2.25	<0.1	<0.1	--	--
		3	4.64	2.34-2.54	3.74-3.77				

L - Longitudinal direction, T - Transversal direction, R - Torsion, A_{max} - Maximum acceleration
$\gamma^L_{máx}$ and γ^T_{max} - Maximum interstory drift in directions L and T, AV - Ambient vibration test

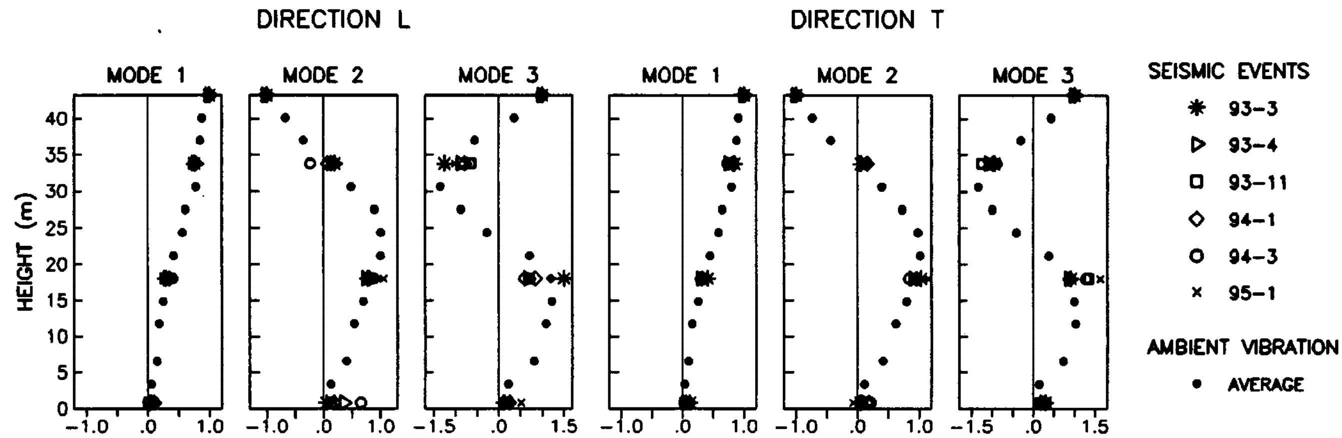

Fig. 1. Modal shapes derived from the ambient vibration and seismic records.

SEISMIC RECORDS

To record the structural response, the building was instrumented with 11 three-directional solid-state digital accelerographs. Additionally, two instruments were installed at 20 and 45 m depth under the building, and one on the ground (Fig. 2). For more details see the parallel paper by Meli et al. (1996).

The strongest earthquakes that hit the Mexico City from 1993 and 1995 were recorded in all the instruments of the building network. The only exception is the instrument located at 20 m depth which was out of place by maintenance during one of the events. In all cases, the epicentral distance was 300 km approximately. The September 14, 1995 earthquake produced the highest accelerations and displacements in the building: the maximum at the roof of the building were 130 gal and 17 cm, and in the ground surface 37 gal and 3.72 cm, respectively.

Fig. 3 shows some records of each event. The torsional vibration is relatively small for the low intensity events (93-3, 93-4 and 94-1), but it becomes higher for the moderate intensity events (93-11, 94-3 y 95-1). This difference could be caused by the characteristic of the motion imposed at the base of the building.

Estimation of structural parameters

In order to study the seismic records in the building, the time and frequency domain data were analyzed. Two parametric and non-parametric estimation techniques were used to analyze the earthquake records. One of the estimation technique utilized is based on a conventional spectral analysis (Bendat y Piersol, 1986). The other one, using a simplified soil-structure mathematical models, defines the parameters with an optimum correlation between the computed and measured structural response (González, 1996).

Non-parametric estimation. The vibration frequencies derived from the seismic records, shown in Table 1, are significantly smaller than those obtained from the ambient vibration tests; they also show a trend to decrease for higher intensities of the ground motion. The largest reduction was for the torsional mode of vibration.

As it can be appreciated from Table 1, the natural frequencies obtained from the ambient vibration tests are greater than those derived from the seismic records; these values diminish after each moderate intensity earthquake. Therefore, the shortening of the vibration frequencies can be attributed to early non-linear behavior of the building rather than to permanent structural damage. Nevertheless, for the torsional mode, the final frequencies are significantly smaller than the initial ones, indicating that some permanent reduction in stiffness has occurred. This reduction can be attributed to some decrease in the contribution of the masonry infills to the building stiffness.

In spite of frequency variations, the mode shapes associated with the first three vibration modes were similar during all stages of measurements (Fig. 3). Damping ratios for the fundamental modes of vibration were derived from the transfer functions between the roof (point RC) and ground surface (point G). Ratios damping between 0.02 and 0.08 were obtained.

Parametric estimation. To analyze the variation of the dynamic characteristics during the seismic events studied, the building was idealized with a 3D linear mathematical model, using a time window analysis. This

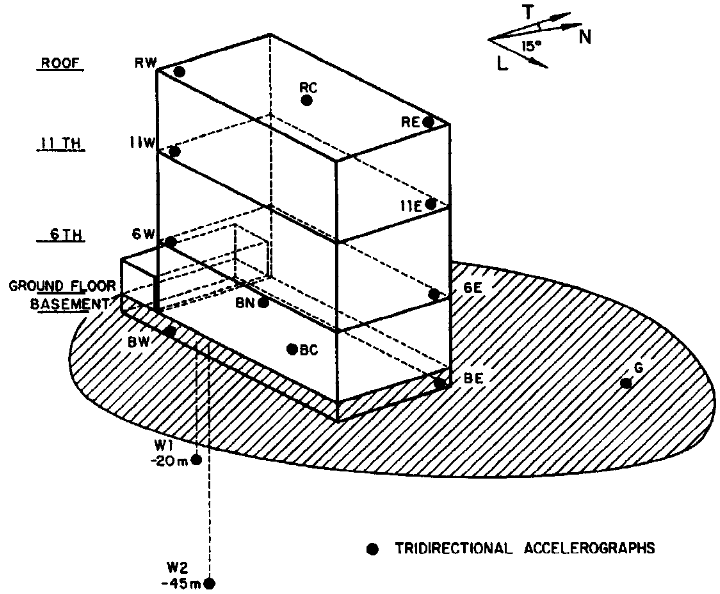

Fig. 2. Overview of the building instrumentation.

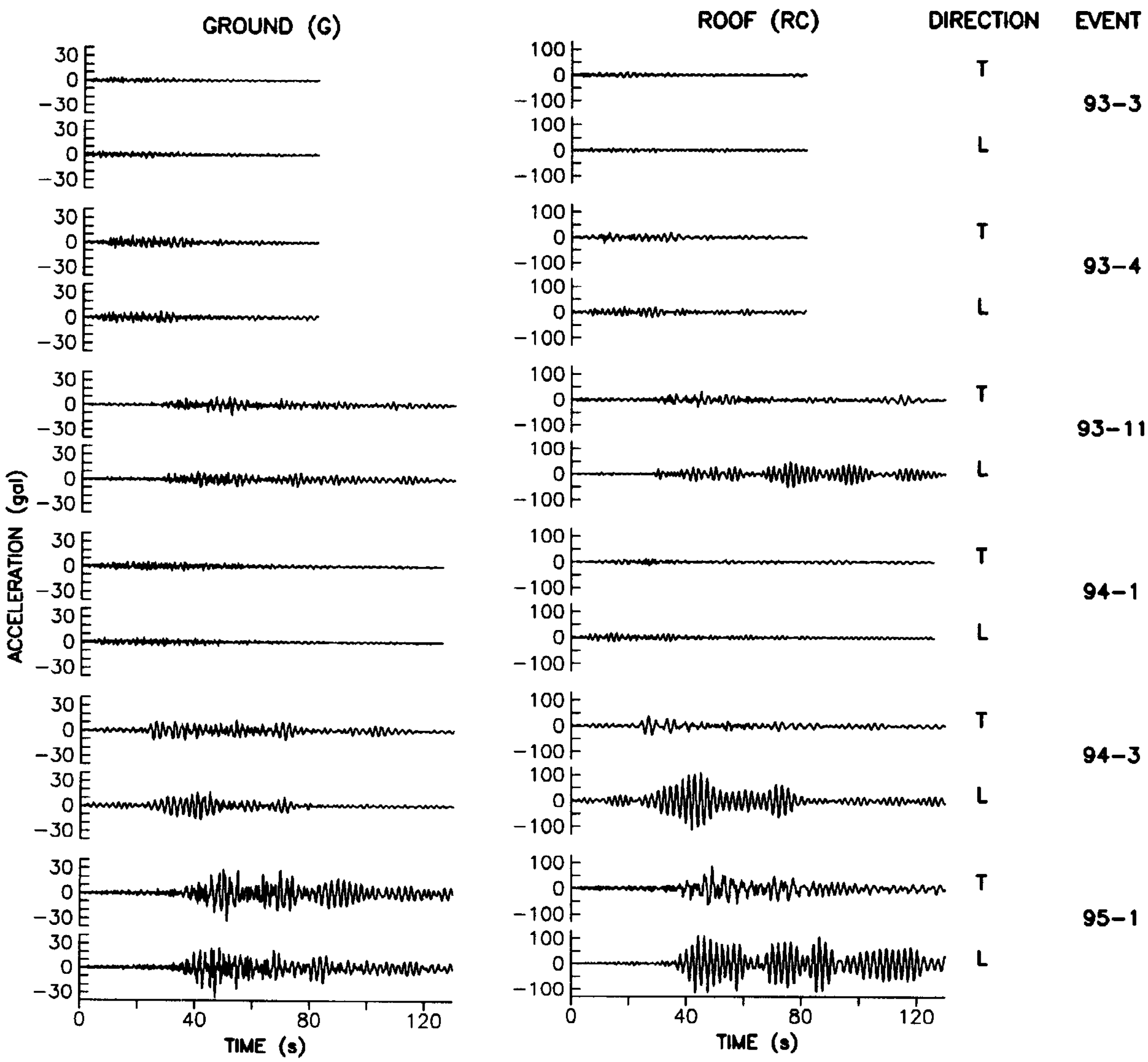

Fig. 3. Some records for each event in directions L and T.

model considers two degrees of freedom: one associated to translation movements and the other to torsional movements (González, 1996).

The variation of the translational natural frequencies is shown in the Fig. 4. These frequencies are compared with those obtained from the ambient vibration tests (dotted line) and with the site dominant frequency (dashed line). From this figure it can be seen that the system had a non-linear dynamic behavior. It was detected that during the 94-3 and 95-1 events, the site dominant and the L-direction system frequencies were very similar.

Records of the moderate intensity events show a beating phenomenon. These beatings are attributed to the similarity of the dominant frequencies of the soil and the structure for torsional and longitudinal vibrations. The same phenomenon has been detected in other instrumented buildings (Boroschek and Mahin, 1991).

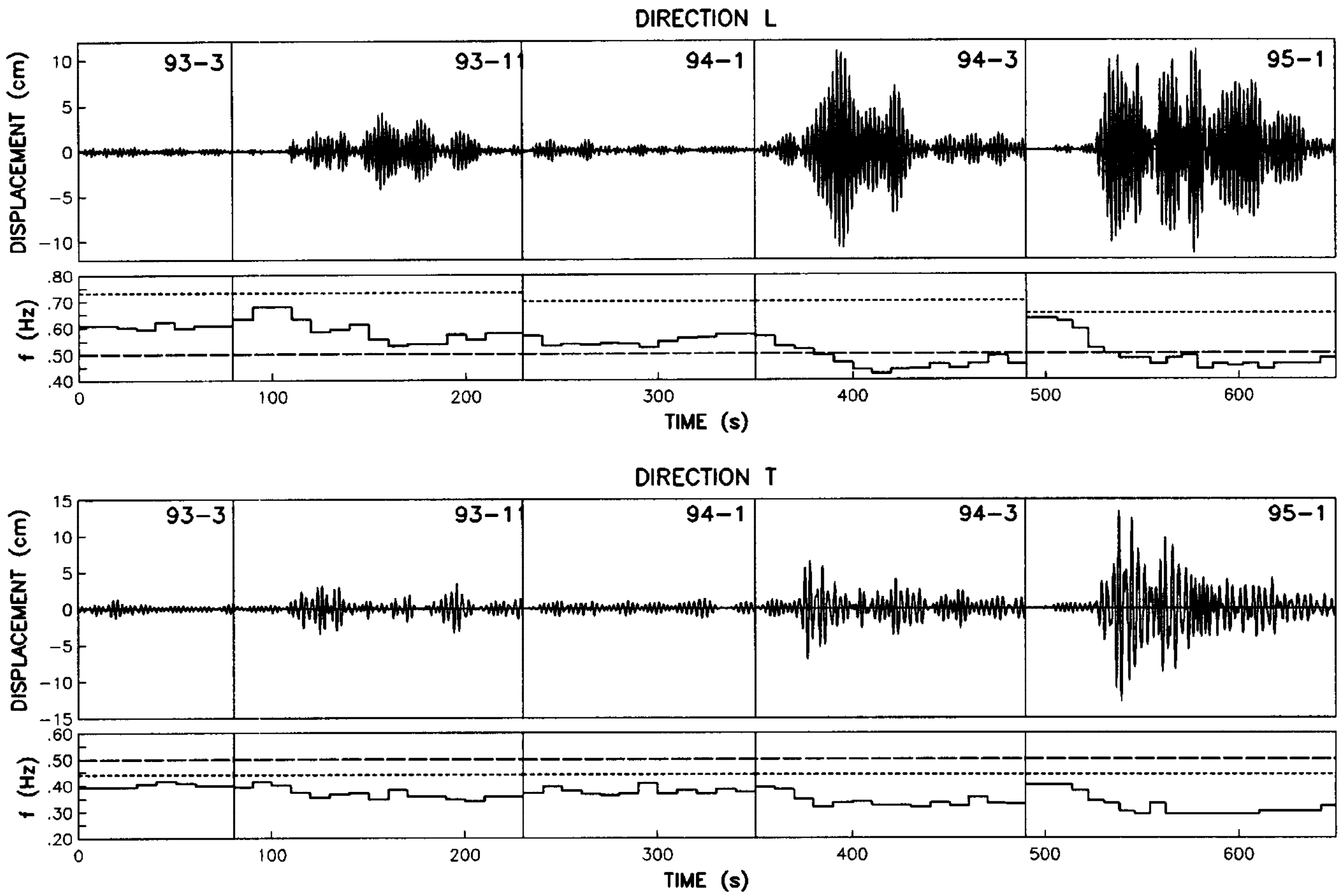

Fig. 4. Acceleration time history and variation of the natural frequencies (f).

ANALYSIS OF STRUCTURAL RESPONSE

Only the studies oriented to evaluate the present practice for seismic analysis of the building will be presented here. Several mathematical models were developed to determine the dynamic properties and the structural response. The contribution of the structural and non-structural elements to the building stiffness and the effect of soil-structure interaction were considered.

A 3D linear mathematical model was built following the assumptions commonly made in design practice. Floors systems were considered to be rigid. Moments of inertia were based on gross sections for columns and on cracked sections for flexural members. The flexural coupling produced by the flat plate spanning between columns and walls, was modeled through equivalent beams with the moment of inertia of the slab in a column strip. Two classes of structural models were considered: the first with fixed base, and the second taking into account soil-structure interaction (SSI) effects. The flexibility of the soil was incorporated in the 3D model by five additional degrees of freedom corresponding to base translation (2), rocking (2) and torsional base rotation. The SSI parameters of case D2 are described by Paolucci (1993).

Table 2. Natural frequencies (Hz) calculated from models

Model	Mode	Flexible base L	Flexible base T	Flexible base R	Rigid base L	Rigid base T	Rigid base R
1 including masonry infills	1	0.68	0.40	0.80	0.89	0.47	0.84
	2	2.51	1.35	2.62	3.25	1.41	2.90
	3	4.60	2.46	4.23	6.83	2.61	5.54
2 without masonry infills	1	0.55	0.37	0.66	0.64	0.41	0.70
	2	2.22	1.21	2.37	2.89	1.24	2.57
	3	4.32	2.24	3.98	6.46	2.34	5.41
3 cracked section in columns and concrete walls	1	0.50	0.36	0.60	0.60	0.41	0.66
	2	1.97	1.20	2.14	2.50	1.24	2.21
	3	3.96	2.23	3.57	5.41	2.35	4.32

Table 3. Comparison of experimental and computed interstory drift

Event	Direction T ($x 10^{-3}$)						Direction L ($x 10^{-3}$)					
	RC/L11		L11/L6		L6/BC		RC/L11		L11/L6		L6/BC	
	E	C	E	C	E	C	E	C	E	C	E	C
93-3	0.30	0.27	0.46	0.31	0.30	0.22	0.17	0.17	0.21	0.17	0.13	0.10
93-4	1.19	0.69	0.94	1.01	0.56	0.64	0.49	0.36	0.72	0.35	0.40	0.18
93-11	1.05	0.73	1.32	0.98	0.70	0.84	1.26	0.75	1.42	0.74	0.97	0.40
94-1	0.31	0.28	0.45	0.36	0.28	0.28	0.34	0.28	0.44	0.27	0.27	0.14

E-Experimental C-Calculated
L11 - 11th story L6 - 6th story RC - Roof Level, Center BC - Basement Center

From the above described models, only three were selected and the results are shown in Table 2. The fundamental frequencies obtained from the first model were reasonably in agreement with those obtained from the ambient vibration tests, but significantly greater than those derived from the seismic records. The differences are attributed to two main reasons: the reduction of the slope of stress-strain curve of concrete with the level of stresses, and the reduction in the participation of the masonry infills in the lateral stiffness of the building. These are probably badly cracked since the 1985 earthquake and have not been properly repaired. They maintain a certain continuity with the structure through the renewed plaster, but a small shaking is enough to separate the walls from the main structure and to eliminate their contribution to the lateral stiffness.

A second analytical model was studied in which all the masonry infills were suppressed. The modal frequencies are now much closer to those of the 4 first seismic records, thus supporting the previous explanation.

Due that in the December 1994 earthquake, some cracks in columns and walls were detected in the structure, for the third structural model a cracked section was assumed to compute the geometrical properties of concrete walls and columns. In spite of this assumption, it was not enough to get a good correlation with the measured dynamical characteristics.

A step-by-step analysis of the structural response of the second model using the 4 recorded motions was carried out. Computed time-histories of accelerations at the center of the roof levels in both directions are compared with instrumental records in Fig 5. A rather good agreement can be appreciated.

Transfer functions between the roof and the ground, obtained from the corrected analytical model are compared in Fig. 6 with those derived from the records of the two events. Although, in general terms, the agreement is acceptable, some significant differences can be appreciated.

A more detailed comparison of the measured and computed response can be made through the relative displacements between stories where the instruments were placed (Table 3). A drift index is derived dividing the relative displacements by the vertical distance between instrumented stories (Roof, 11th, 6th and Basement).

The average drifts between instrumented stories are much lower than those commonly associated to structural damage. This could be expected considering that the ground motion imposed has been very low. Comparison of computed and measured drifts shows a good agreement for the transverse direction, whereas for the longitudinal one computed values are definitely smaller than those recorded, especially for the lower stories. Similar results were found by Durrani et al. (1994). Further refinement of the analytical model is required in order to obtain a closer representation of the observed response.

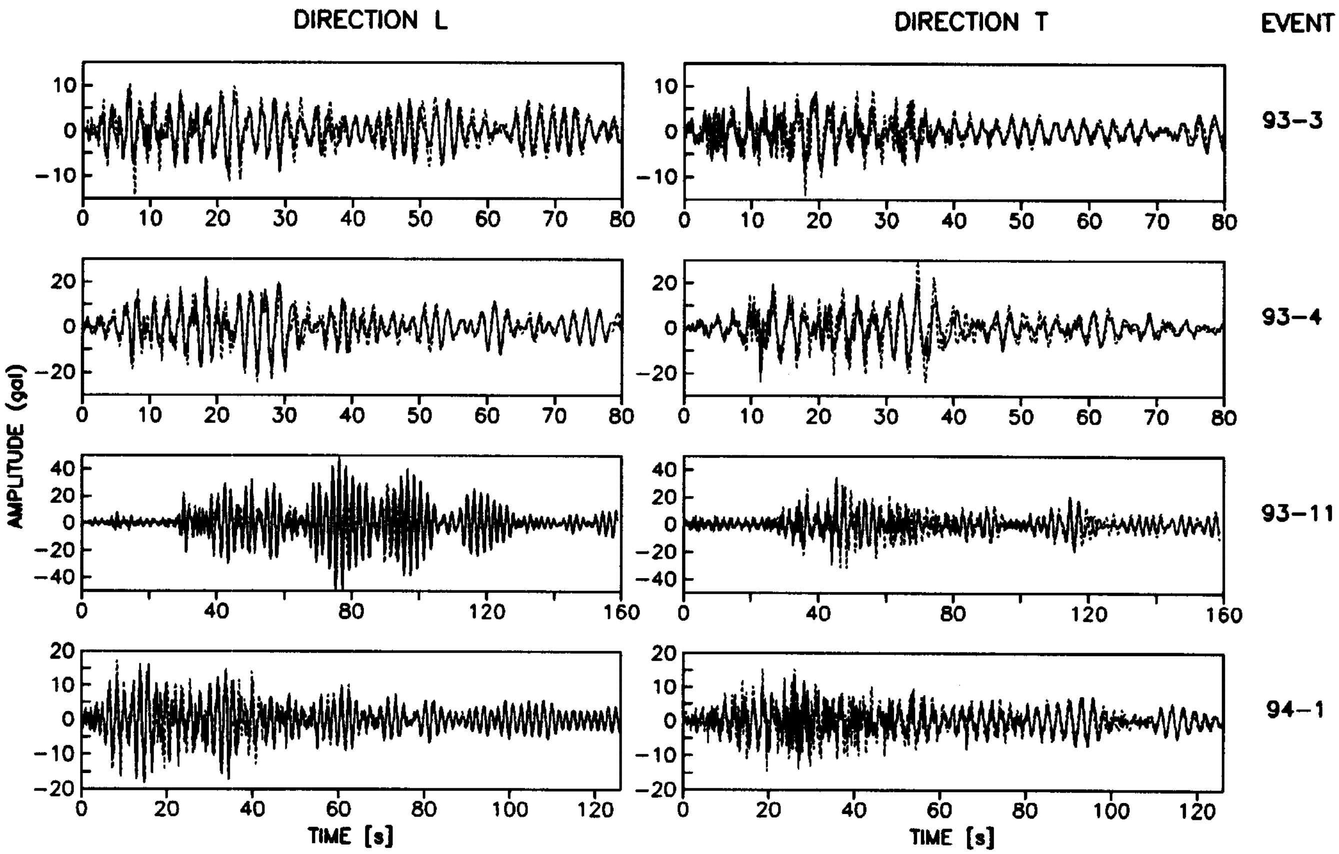

Fig. 5. Measured and calculated acceleration time-histories at roof center.

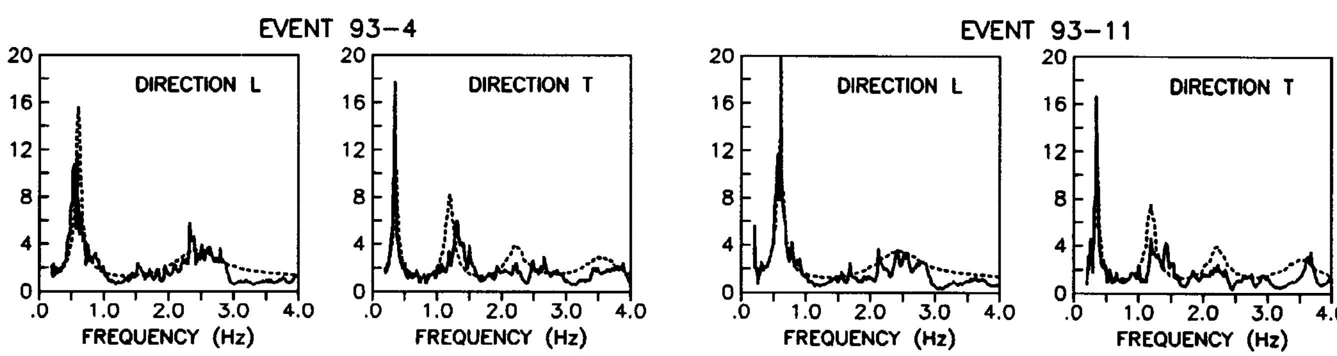

Fig. 6. Transfer functions between the motion at the roof center and at the ground, obtained from the records and from the analytical model.

FINAL REMARKS

The ambient vibration tests provided a very complete information on the dynamic properties of the building, allowing to estimate not only its modal frequencies and shapes but also the transfer functions between different points. Nevertheless, the results are restricted to the behavior under very low levels of stresses. The dynamic response of the structure is very sensitive to the amplitude of the imposed ground motion. Early non-linear behavior produced continuous reduction of the lateral stiffness with increasing level of stresses.

It was possible to evaluate the variation of the dynamic characteristics of the system, using non-parametric and parametric system identification techniques. A non-linear behavior of the building was detected during the seismic events studied.

As a consequence of the similarity of the values of the soil dominant frequency and the longitudinal translational frequencies, a beating effect for the moderate intensity earthquakes was detected.

For the first 4 seismic events the analytical models currently used for the calculation of the building seismic response produce large differences in the results depending on the assumptions regarding the properties that govern the stiffness of the structural members. When these properties are accurately chosen to represent the behavior under the particular amplitude of motion, reasonable results can be obtained concerning the overall response of the building, although significant differences can be expected at a local level response.

The variation of the building structural parameters can be attributed to different sources of non-linearity in the structure, and in the soil-structure interaction. For the former case they can be due to: the change in the elasticity module of concrete, the discontinuity of the concrete panels and the structural elements, the damage level of the masonry walls and cracking of the floor system, and the degradation effects of the structural and non-structural elements under several seismic motions. For the latter, a discontinuity between the foundation-soil interface.

The effects of moderate earthquakes on the building suggest that high intensity earthquakes will produce a high damage level. Nevertheless, it must be remembered that before being strengthened the building withstood the great earthquake of 1985 without major damage.

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