Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VII)

12-4-12

FORCED VIBRATION STUDIES OF AN RC BUILDING RETROFIT WITH STEEL BRACING

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SUMMARY

A ten-story reinforced concrete building located in the soft soil zone of Mexico City was severely damaged during the March 14, 1979 Mexico earthquake. Following that earthquake, the building was repaired and strengthened. It sustained minor structural damage during the September 19, 1985 Mexico earthquake even though the intensity of shaking was much greater than in 1979. Detailed analytical models in conjunction with results from forced vibration studies are being used to interpret and evaluate the response of the building. Results indicate that the steel bracing scheme used to repair this building was an important factor in its structural behavior during the 1985 earthquake. The repaired building had greater strength and stiffness than the original structure. Lateral displacements were controlled and pounding against adjacent buildings was reduced.

INTRODUCTION

On September 19, 1985, a major earthquake ($M_S = 8.1$) occurred in Mexico near the coast of Michoacan. Although it caused only moderate damage to engineered structures in the epicentral region, it caused extensive damage to many structures in downtown Mexico City: 486 buildings experienced either total or partial collapse and another 271 were severely damaged (Ref. 2). Further examination of these buildings revealed that pounding against adjacent buildings occurred in over 40% of the cases, and that for at least 15% of these, it was the primary cause of collapse. Many of these had been designed using modern seismic building codes but, because of the unexpected severity of ground motion, the minimum code requirements regarding building separation were insufficient to prevent pounding (Ref. 1).

The Park España building discussed in this paper is of interest because it incurred only minor structural damage during the 1985 earthquake even though it has characteristics similar to those buildings which experienced greatest occurrence of collapse (Ref. 2). Some pounding was experienced between this building and an adjacent building. The building is a ten-story, flat slab, asymmetric reinforced concrete structure located within the most heavily damaged region of Mexico City. The structure is of particular interest because it was repaired and strengthened following the 1979 earthquake via the addition of structural steel and reinforced concrete elements. A description of the building, its underlying soil conditions, and its retrofit system are presented below. The results of forced vibration tests and analytical studies are used to suggest reasons for the good performance of the building and to evaluate the role of the seismic strengthening technique that was used to repair the building in 1979.

DESCRIPTION OF BUILDING

Building Description. The building is a ten-story reinforced concrete condominium apartment building that stands 28.2 *m* above the foundation level, penthouse included. The building has an irregular floor plan which measures roughly 10.8 *m* by 17.45 *m*. Typical floor and foundation plans are shown in Fig. 1. The floor system is a reticular waffle slab 5 *cm* thick with 35 *cm* deep ribs. Girders are formed by providing longitudinal reinforcement in the ribs of the slab along the column lines. Moment frames in each direction resist lateral forces. The infill walls of the longitudinal exterior frames and the interior partition walls are constructed of siporex blocks. Because siporex is a very soft material (Young's modulus is less than 1% of that for concrete), it provides minimal stiffness and strength. Therefore, neither the exterior wall infills nor the partition walls were considered as structural elements in the original analysis of the building. No gaps exist between the siporex blocks and the building frame.

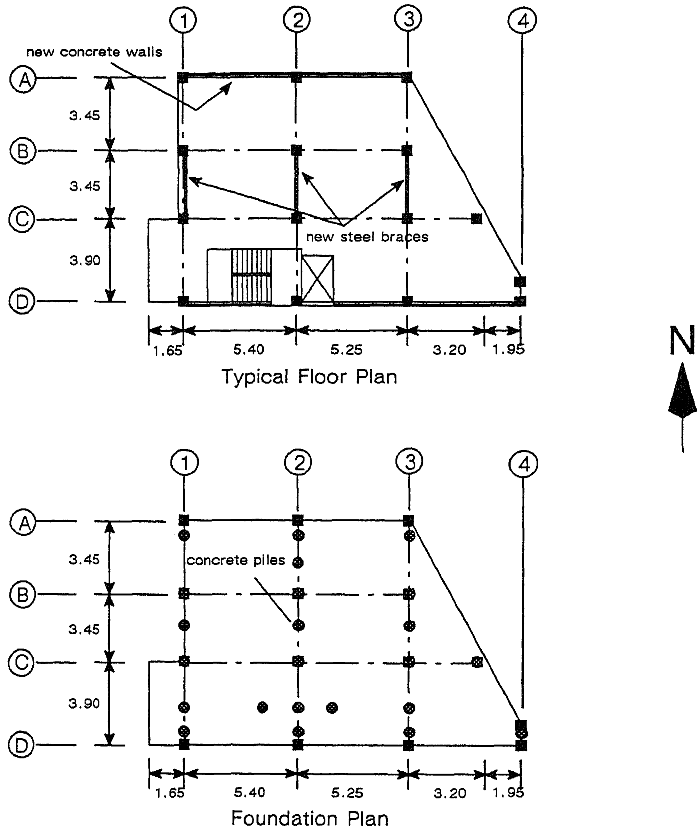

Fig. 1 Typical Floor and Foundation Plans

The foundation for the building is a solid 15 *cm* thick mat underlain by deep, slender stiffening beams (140 *cm* x 40 *cm* N-S and 140 *cm* x 30 *cm* E-W) located along the column lines. The stiffening beams are supported on friction piles. A geotechnical investigation revealed that the building rests on approximately 32 *m* of soft, saturated clay. Blow counts of 1 to 2 and water contents of 150% to 300% are typical throughout the clay layer.

Repair and Strengthening. The building was severely damaged during the 14 March 1979 Mexico earthquake ($M_S = 7.6$). One of the fourth story columns failed as a result of pounding against an adjacent four-story building located approximately 5 *cm* north of the Park España building. Because the floor systems of the two buildings were at a different levels, the pounding effects were aggravated and greater damage was incurred by the Park España building. The Park España building also experienced large interstory deformations of its frame, reflecting the very flexible nature of its structural system. Because no gaps were provided between the exterior wall infills and the building frames, these deformations resulted in damage to the exterior walls (both longitudinal and transverse). In addition, the longitudinal and transverse partition walls were badly cracked at several levels. No indications of foundation failure were observed.

After the 1979 earthquake, diagonal steel cross-bracing was added to the central bay of frames 1, 2 and 3 to increase the strength and stiffness in the transverse direction (Figs. 1 and 2). The cross-braces were

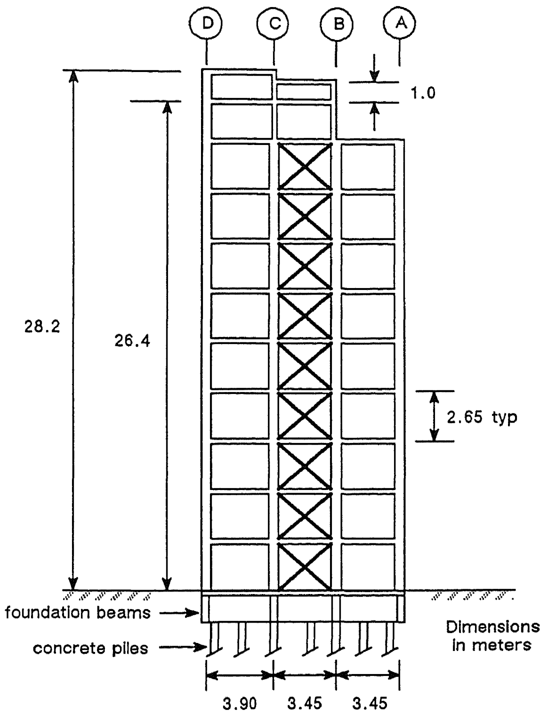

Fig. 2 N-S Elevation of Building

fabricated by continuously welding two angles together toe-to-toe to form a structural box. The columns of the three braced bays were encased in a steel lattice composed of angles at the corners and diagonal flat plates. This encasement provided the additional strength necessary to carry the increased axial forces anticipated in the columns of the braced bays. These forces result from the additional overturning moment attracted to the braced bays. Special steel collars were fabricated and placed at the top and bottom of each column to facilitate the attachment of the steel cross-braces. These collars were grouted and bolted to both the original concrete column and the adjoining slab to smooth out the transfer of forces between stories.

To increase the strength in the longitudinal direction, new reinforced concrete infill walls 4 *cm* thick were added to all bays of the exterior longitudinal frames. The reinforcement ratio of these walls was about 0.64% in both the horizontal and vertical directions. The design details included driving nails into the existing siporex walls, attaching welded wire reinforcing mats to the nails and trowelling new concrete into place (Ref. 3).

Story weights before and after the structural repair are listed in Table 1. The steel braces and reinforced concrete walls increased the weight of the building approximately 3%. No additional piles or other foundation modifications were required largely because the weight of the structure was essentially unchanged. Besides, failure of the foundation was not observed following the 1979 earthquake.

Table 1

Story Weights
Before and After Retrofit

Level	Before Weight *(tons)*	After Weight *(tons)*
P.H.	42.2	42.2
9	163.3	166.0
8	158.5	163.8
7	158.6	164.0
6	159.1	164.7
5	160.5	166.2
4	162.1	167.8
3	163.1	168.9
2	164.6	170.5
1	176.7	182.5
P.B.	192.2	195.0
Total	1700.9	1751.6

1 ton = 1000 kg

Table 2

Measured Dynamic Properties

Mode	Frequency *(Hz)*	Damping *(% critical)*
1 NS	0.90	3
1 EW	1.10	7
1 T	1.50	-
2 NS	3.50	-
2 EW	4.25	-
2 T	> 5.00	-

RESULTS OF VIBRATION TESTS AND PRELIMINARY STUDIES

Results of Vibration Tests. Forced vibration tests were conducted in January 1987. Two eccentric mass vibration generators were used to induce steady-state sinusoidal motion at the roof level, as shown in Fig. 3.

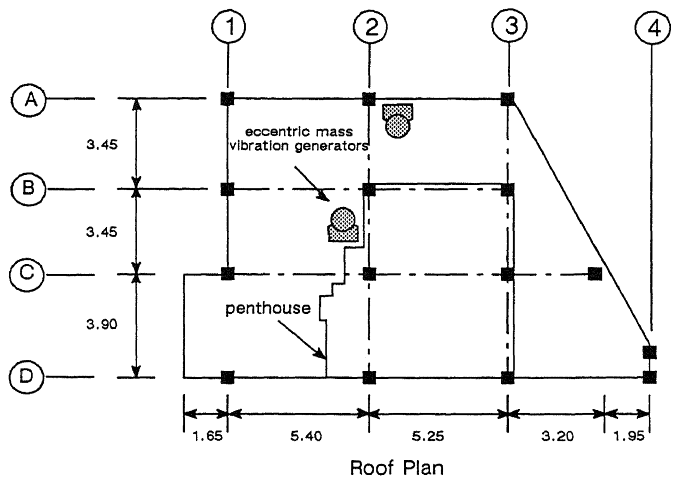

Fig. 3 Roof Plan of Building Showing Location of Vibration Generators

Building response was measured using one Model SS-1 Ranger seismometer and five FBA-11 force balance accelerometers. These measurements were recorded on two media: an eight-channel HP Model 7418A thermal tip recorder and a four-channel HP 3960 analog tape recorder. The data were digitized and filtered prior to analysis. Typically, for a given mode, measurements were taken at columns A-2 and D-2 and at the middle of the steel braces of frames 1 and 3 for each floor level (see Fig. 1). From these measurements, the presence of modal coupling was observed. Excitation of the building in the N-S direction resulted primarily in N-S translational response, but response in the E-W direction was also detected. Hence, a rotational component was present in the motion as well. The presence of modal coupling was expected because the floor plan was asymmetrical and the centers of mass and rigidity did not coincide. The first five measured natural frequencies and the first two damping ratios of the building are given in Table 2. The first two mode shapes are shown in Fig. 4. This figure shows the contribution of roof displacement from base translation, foundation rocking, and structure distortion. The large influence of foundation flexibility is evident from these results.

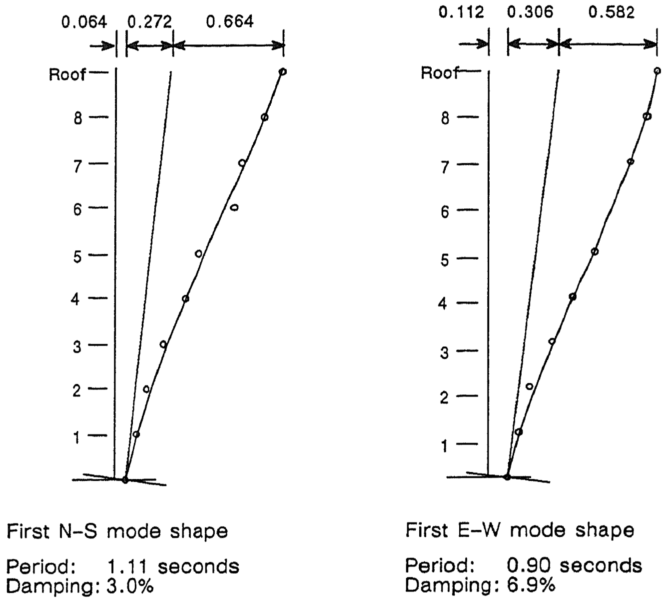

Fig. 4 Measured Mode Shapes of the Building

Results indicate coupled translational and rotational response due to the asymmetry of the structural configuration. The close spacing of the two lowest vibration frequencies reflects the modal interference found to exist for the structure.

Both prior to and immediately following the reparation, the frequency in the N–S direction was determined from ambient vibrations using a portable seismometer. These values were 0.95 Hz (before) and 1.25 Hz (after), revealing that the steel cross–bracing increased the average stiffness in the N–S direction by approximately 50%. Comparison of the 0.90 Hz frequency determined by the forced vibration tests for the first N–S mode with the 1.25 Hz measured immediately following repair reflects the fact that some structural damage did occur in the building during the 1985 earthquake. The damage sustained, however, was small compared with that experienced during the 1979 earthquake. Minor pounding with the adjacent four–story building caused loosening of the brace attachments, superficial spalling of column plaster and cracking of transverse partition walls at the fourth floor level. Some cracking of partition walls at other levels also occurred. Had the building not been repaired, it is likely that it would have collapsed as a result of pounding with the adjacent building, precipitating from fourth story damage. Thus, the repair and strengthening scheme used for the building was successful. The tenants were able to remain in their apartments during the repairs following the 1985 earthquake.

One surprising result of the vibration tests is the large amount of foundation compliance observed in the building. Figure 4 shows that over 34% of the roof motion in the N–S direction and 42% of the roof motion in the E–W direction was the result of translation and rotation of the base. The large foundation contribution to motion results from the combination of very soft soil and a stiff building.

<u>Results of Preliminary Studies</u>. A response spectrum analysis of the building was performed using the 1985 SCT N–S accelerogram and the 3% damping factor determined from the forced vibration tests. Only the first

mode was considered in the analysis. The maximum base shear determined for this motion for the retrofit structure was 0.14W. It is estimated that this motion induced a maximum compressive stress of 976 kg/cm^2 (13.9 *ksi*) in the first story braces. A maximum base shear of about 0.28W would have been required to buckle these braces (according to the AISC specification). Of course, because of the pounding which occurred with the adjacent building, it is difficult to determine exactly what the base shear might have been.

If the 1979 SCT N-S accelerogram is used instead of the 1985, the maximum base shear determined for the retrofit structure is approximately 0.07W, or half of the 0.14W calculated for the 1985 earthquake. Despite the stiffening of the structure, pounding still occurred during the 1985 earthquake. While the strengthened structure was nearly 1.5 times as stiff as the original structure, the forces experienced during the 1985 earthquake were far greater than those sustained by the original structure. The increase in the transverse stiffness via the steel cross-braces was not adequate to completely prevent pounding. It did, however, localize damage to the fourth floor, keeping damage to the other floors to an acceptable minimum.

CONCLUDING REMARKS

It is well known that inadequate separation between adjacent buildings is the cause of structural pounding and that structural pounding is detrimental to the seismic performance of buildings. Yet, inadequate separation between buildings does exist and this problem requires attention. The stiffening technique described above performed well since it minimized the damage experienced by the structure in which it was implemented, despite the unexpectedly large ground motions of the 1985 Mexico earthquake.

The great success of stiffening the building was largely due to the shape of the response spectrum. The peak value of the spectrum (*e.g.* SCT) was about 0.5 *Hz* and the building frequencies were greater than 0.5 *Hz*. In the case of a structure located in the lake bed region of Mexico City with a fundamental frequency less than 0.5 *Hz*, or for a structure in California on stiff soil, the retrofit solution presented here might not be as efficient. In these cases the response would increase, rather than decrease, as the structure is stiffened and would require additional strengthening.

ACKNOWLEDGMENTS

This research was partially funded through grant ECE 86-10785 from the National Science Foundation. This support is gratefully acknowledged. The findings, opinions and conclusions expressed in this paper are solely those of the authors and do not necessarily represent those of the sponsor.

REFERENCES

1. Bertero, V.V. "Observations on Structural Pounding," *The Mexico City Earthquakes – 1985, Factors Involved and Lessons Learned*, ASCE, 1987, pp. 264–278.

2. Borja, G., Diaz, M., Vazquez, A., Del Valle, E. "Damage Statistics of the September 19, 1985 Earthquake," *The Mexico City Earthquakes – 1985, Factors Involved and Lessons Learned*, ASCE, 1987, pp.70–77.

3. Del Valle, E. "Some Lessons From the March 14, 1979 Earthquake in Mexico City," *Proceedings of 7WCEE*, Vol.4, Part 1, Istanbul, Turkey, September, 1980, pp. 545–552.