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SEISMIC RETROFIT OF A RC BUILDING: A CASE STUDY

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SUMMARY

A twelve-story reinforced concrete frame building located in Mexico City was repaired and strengthened after suffering extensive damage during a moderate earthquake that shook the city in 1979. The building suffered no damage during the great Michoacan Earthquake that rocked Mexico City on 19 September 1985, even though the shaking was much greater than that in 1979. Forced vibration tests and analytical studies were conducted on the building to determine why the building behaved so well during this earthquake. Results indicate that the steel braced frames that were attached to the building strengthened it, and they stiffened the structure, moving its natural period away from the predominant ground period of 2.0 sec.

INTRODUCTION

In the aftermath of a great earthquake such as the 19 September 1985 Michoacan Earthquake that rocked Mexico City, it is vital that as many lessons as possible be learned from the tragedy, so that new buildings might be designed and constructed better, or existing structures strengthened against future shocks. While many important lessons may be learned from studying buildings that have been badly damaged or collapsed, buildings that performed well during strong shaking may also provide important insights to structural engineers and architects. The object of this study is a twelve-story reinforced concrete building that was repaired after the 1979 earthquake and strengthened prior to the great earthquake of 19 September 1985. The building was strengthened using structural steel frames in one direction and new infill reinforced concrete walls in the other (Ref. 1) The building suffered no damage during the 1985 earthquake, even though it was located in the vicinity of several collapsed buildings (Ref. 2) and was located in the part of the city that experienced the strongest ground shaking. Extensive experimental and analytical investigations of the building are being undertaken to evaluate the retrofit scheme that was used. The reasons for the excellent performance of this building will be ascertained and recommendations for similar schemes will be proposed and evaluated.

DESCRIPTION OF BUILDING AND RETROFIT SYSTEM

Building Description The Durango building is a twelve-story reinforced concrete building that measures 11.9 x 21.3m in plan and stands 36.4m above the foundation level, excluding the penthouse. A typical floor plan is shown in Fig. 1. Lateral forces in the original structure were resisted by Frames 1-5 in the E-W direction and by frames A and C in the N-S direction. Frames 2 through 4 are identical and are characterized by haunched beams as shown in Fig. 2. Frame 1 and Frame 5 have only two columns each, and are identical except that the first story beam was omitted from Frame 1 to provide access for a driveway. However, deep spandrel beams were provided in Frames 1 and 5 as part of the facade. An elevation view of Frame 5 is shown in Fig. 3. This figure also depicts the location where damage occurred during the 1979 earthquake. Notice that extreme columns of Frames 1-5 are tapered continuously from bottom to top due to architectural reasons; central columns of Frames 2-4 are reduced in steps in several floors in order to save material. Frames A and C were identical, but the floor system was not connected to the columns on Frame B to form a seismic frame. Masonry infill walls were provided in Frames A and C to enclose the building and to provide minimum insulation. These infill walls were "non-structural" and were not assumed to participate with the frames in resisting seismic forces. A gap filled with celotex was provided to isolate them from the structure, as the masonry material is very stiff but weak.

The foundation is a rigid, voided mat 2.4m thick on concrete friction piles. A geotechnical investigation conducted as part of this study revealed that the building rests on about 40m of soft, saturated clay that is characterized by blow counts of one or less over much of the depth and by water content that exceeds 460% in one region.

Repair and Strengthening A moderate earthquake shook Mexico City in 1979 and caused considerable damage to the Durango building as a result of shaking in the E-W (transverse) direction. The damage was restricted to the first four stories as shown in Fig. 3. The spandrel beams and columns in Frames 1 and 5 experienced diagonal cracking over much of their length in the first floors. In addition, the beam-column joints of these frames suffered severe cracking and spalling. The middle column in the fourth story of Frame 3 suffered cracking and crushing.

The cracked beams and columns were repaired with epoxy injection. The columns of Frames 1 and 5 were encased in steel through the fourth story level. Braced steel frames were attached to Frames 1 and 5 on the outside of the building to increase the strength and stiffness of the structure in the E-W direction. An elevation view showing the braced frames is shown in Fig. 4. The columns of the frames and diagonal bracing at the first level were fabricated steel boxes. The other bracing members were made from 2 channels placed toe-to-toe with gusset plates between them. New footings and piles were placed under the columns of the new frames and were attached to the original foundation to ensure monolithic action. A 1.3m wide section of the floor slab was strengthened at each location where the new steel frame was attached to existing structure.

New reinforced concrete infill walls were provided to strengthen the building in the N-S direction. The walls were placed in the 1-2 and 4-5 bays of Frames A and C for the full height of the building. Nails were inserted into the existing masonry walls. Reinforcing mats were attached to the nails, and the concrete was placed by trowels; the celotex plates that filled the gap between walls and structure were removed and the gap was filled with mortar. All of the new strengthening elements are shown in the shaded areas in Fig. 1.

The weight of each floor before and after strengthening is given in Table 1, showing an average increase of only 6.5%.

RESULTS OF EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS

Results of Vibration Testing of the Durango Building Forced vibration tests of the building were conducted during January 1987. The building was shaken using two eccentric mass vibration generators, owned by the California Institute of Technology, that were mounted on the roof of the structure. The measured natural frequencies and damping values for the first five modes of the vibration are listed in Table 2. The mode shapes for the first E-W mode and first N-S mode are shown in Figs. 5 and 6, respectively. Prior to the addition of the new walls and frames, the period of the building in the E-W direction was measured to be about 1.85 sec. for ambient levels of vibration. Thus, the steel frames increased the stiffness of the building considerably as indicated by the current period of 1.26 sec. The low damping in the E-W direction is also an indication that the steel frame dominates the response in this direction. Since the period of the site is thought to be about 2.0 sec., the stiffening effect of the steel frame which shifted the period of the building away from the danger region was a major cause of the excellent behavior during the 1985 earthquake. One surprising result of the test was the relatively large amount of foundation compliance present for the building in both directions of vibration. The translation and rotation of the base contributed over 50% of the roof motion. This is the result of having a very stiff building founded on very soft soil.

Results of Analytical Studies A three-dimensional analytical model of the building was developed. All of the reinforced concrete beams and columns of the original building as well as the new steel frames and infill walls were included in the model. Linear horizontal and vertical springs were placed at the base of the model under each column to simulate the flexible soil condition at the site. The properties of the foundation springs were adjusted until the analytical predictions of the first two mode shapes and frequencies of vibration for the model matched those measured during the forced vibration tests. The analytical and measured results for both directions are shown in Figs. 5 and 6.

A response spectrum analysis of the structure was conducted using the ground motion measured at the SCT location in Mexico City. The soil borings from the site indicated that the soil was very nearly the same as that at the SCT location, so the ground motions should be expected to be very similar. The calculated base shear for the structure was about 27% of the total weight of the building for E-W motion. The stresses in the steel members at this level of shaking were slightly smaller than the allowable stresses specified in the AISC specifications. Thus, the strengthening effect of the new steel frames was important, and was more than adequate. The strengthening system was designed in accordance with 1976 Mexico City Code, but with the ductility factor of 2, rather than 4 which would have been allowed. The analytical studies indicated that the steel frames carry about 80% of the total seismic force in the E-W direction. The analysis also indicated that large axial forces (including tension) developed in the RC frame that was attached to the steel frame due to compatibility requirements. Also, the eccentricity between the steel and RC frames caused large transverse moments to develop in the columns of the RC frame at points of attachment to the steel frame.

An estimate of the effects of the foundation flexibility on the dynamic properties of the structure may be obtained by making the foundation springs very stiff. The results indicate that the foundation flexibility increases the period of the structure from about 0.9 sec. to 1.26 sec. in the E-W direction.

Thus, the steel frames increase the stiffness of the building by over a factor of 4, but almost 50% of this stiffness is lost due to foundation compliance.

A limit analysis for the original structure for response in the E-W direction was conducted to determine the base shear capacity of the building. In the analysis, it was assumed that the theoretical ultimate capacity of the beams and the columns could be developed, and the effects of axial forces on the moment capacity of the columns were included. The forces were assumed to be distributed through the height of the structure in accordance with response in the first E-W mode of vibration. The haunched beams of Frames 2, 3 and 4 complicated the analysis, since hinges could be assumed to form either at the beam-column interface or at the beginning of the haunch in the beam. The virtual work method was used for the analysis, and 28 possible failure mechanisms were assumed. The failure mechanism giving the smallest capacity includes hinges at the base of the first story columns, at the top of the sixth story columns, and in the beams of the first five stories. The hinging pattern for the beams is mixed, with the hinges forming at the column face for the positive moment region and forming at the beginning of the haunch in the negative moment region. The results of the analysis indicate that the base shear capacity of the building was only about 0.13W before retrofit. A response spectrum analysis of the original building was conducted using the E-W accelerogram measured at SCT during the 1979 earthquake. The results indicated that the maximum base shear reached for E-W response was probably about 0.09W. The results seem to be consistent with the observed damage. No member hinging occurs at this level of response, but the shear stresses that develop in the connection region at the beam-column joints exceed the design capacity at the first four levels of the building. A response spectrum analysis of the building was also conducted for the accelerogram measured during the 1985 earthquake. For the building to remain elastic during this earthquake, the base shear capacity would have had to be over 0.8W. Since very few ties were provided at the base of the first story columns (#3 bars at 30 cm.), it is quite likely that the building would have collapsed had the retrofit not been undertaken.

CONCLUDING REMARKS AND DESIGN IMPLICATIONS

Because of the life threatening situations that old, under-designed buildings represent, strengthening of existing buildings for seismic forces is a subject that has received considerable attention from researchers during the past few years. Many buildings in Mexico City, in California, and in other parts of the world are being strengthened. It is important that lessons learned from this research and from the actual case histories be shared so that they may be incorporated into design and construction practices around the world.

ACKNOWLEDGEMENTS

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Table 1 Story Weights Before and After Retrofit

Level	Before Weight (kips)	After Weight (kips)
P.H.	135.4	135.4
Roof	406.3	425.1
10	425.6	455.3
9	425.6	455.3
8	425.6	455.3
7	425.6	455.3
6	425.6	455.3
5	425.6	455.3
4	425.6	455.3
3	425.6	455.3
2	425.6	455.3
1	467.2	503.0
P.B.	393.9	412.0
Base	1136.6	1356.9
Total	6370.1	6930.1

Table 2 Measured Dynamic Properties

Mode	Period (seconds)	Damping (% critical)
1 EW	1.26	3
2 EW	0.28	5
1 NS	1.00	5
2 NS	0.21	9
1 Torsion	0.48	6

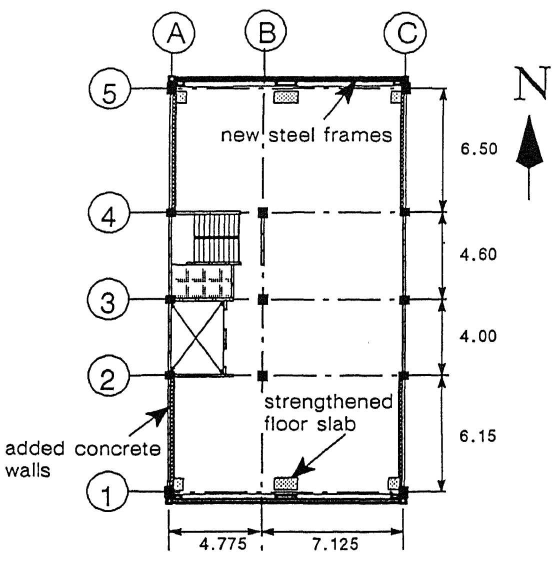

Fig. 1 Typical Floor Plan

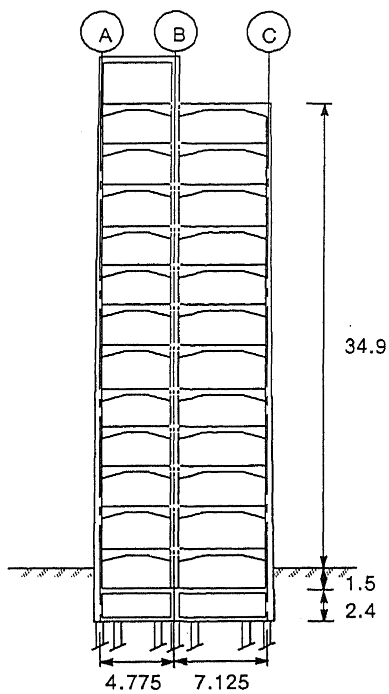

Fig. 2 Elevation View of Frames, 2, 3 and 4

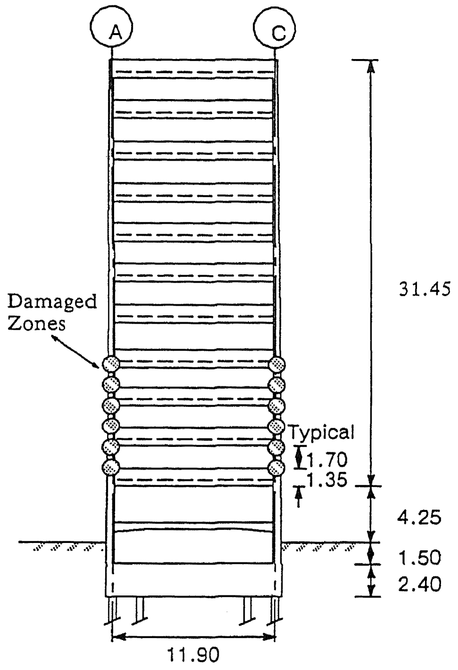

Fig. 3 Elevation View of Frame 5 Showing Damage that Occurred During the 1979 Earthquake

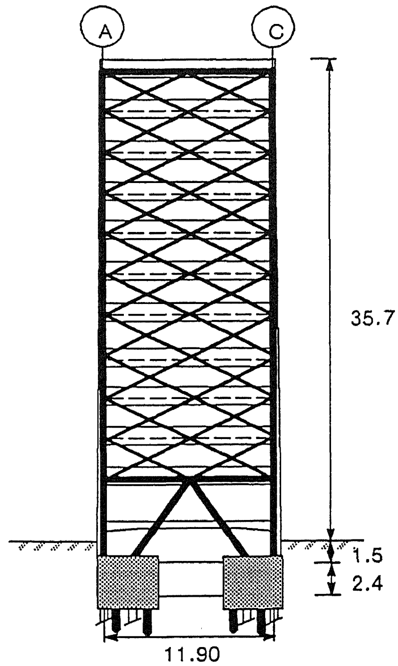

Fig. 4 Elevation View of Frame 4 Showing New Steel Frame

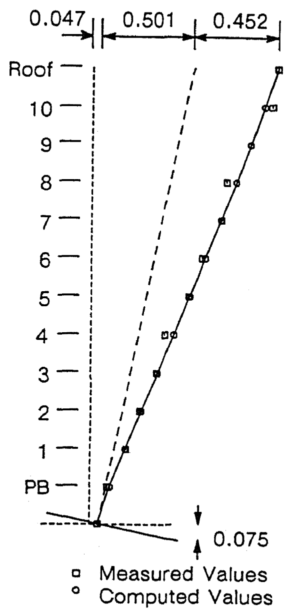

Fig. 5 Measured and Computed E-W Mode Shape

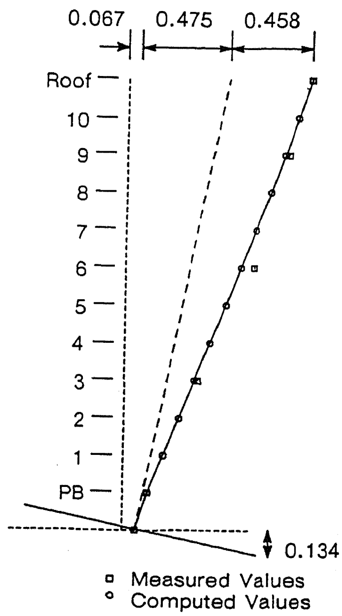

Fig. 6 Measured and Computed N-S Mode Shape