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Collapse assessment of a tall steel building damaged by 1985 Mexico earthquake

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ABSTRACT: The collapse assessment of a 22-story steel building during the September 19, 1985 Mexico earthquake is presented. The assessment is mainly based on a series of inelastic analyses, with laboratory experiments and field observations, using multicomponent seismic input of actual Mexico City earthquake records. In order to perform inelastic analyses of this building, the hysteresis models for columns, girders, and bracings are first constructed according to experimental as well as analytical results of those structural members. The analyses show that the structural response exceeds the original design ductility of this building because most girders in the building have suffered large inelastic deformations. Ductile failures of girders combined with the local bucklings of columns on floors 2, 3, and 4 of the building appear to have resulted in significant story drift, building tilt, P-Δ effect, and the failure mechanism.

1. INTRODUCTION

On September 19, 1985, a major earthquake of 8.1 Richter magnitude occurred; its epicenter was about 250 miles from Mexico City. During this earthquake, the Pino Suarez complex located in Mexico City suffered severe damages. The complex, shown in Fig. 1, comprised five high-rise moment-resisting steel buildings: building B, C, and D are identical 22-story structures and buildings A and E are identical 15-story structures. During the Mexico earthquake, building D collapsed onto building E, and building B and C, with severe damage, came close to collapse.

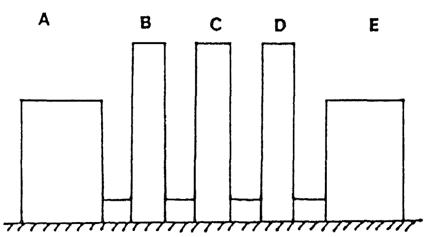

Figure 1. Pino Suarez Complex

This 22-story building is shown in Fig. 2. It contains four bays in the building's long direction (E-W) and two bays in its short direction (N-S). A bracing system is located near the service core, which is composed of K-bracings in the long direction and X-bracings in the short direction. Since the bracing system is only on one side of the building, the configuration is unsymmetric.

Girders in the structure's long direction are box-shape open-web joists. The typical long-direction girder is shown in Fig. 3(a). Top chords, bottom chords, and web members are angle sections. There are cover plates located near the two ends of the girder. Girders in the structure's short direction are I-shape open-web joists as shown in Fig. 3(b). Top chords and bottom chords are angle sections. Web member is a box section made by stitch welding two equal-leg angles together. All the columns are box sections made by continuous fillet welding four equal thickness plates together. The bracing members are build-up sections with three plates welded together into an H shape. The material yielding stress is 36 ksi for all the structural components.

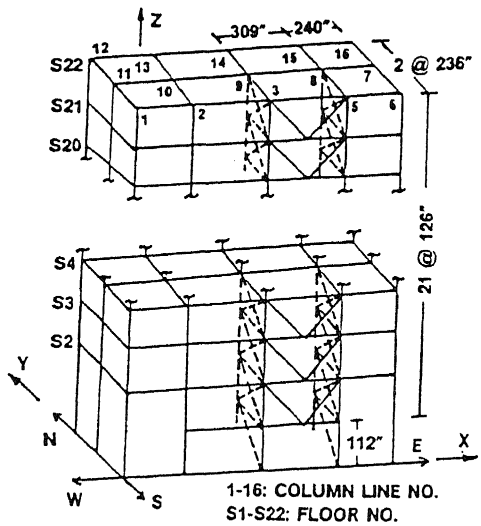

Figure 2. Configuration of Pino Suarez building

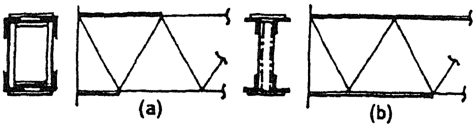

Figure 3. Typical girders, (a) long direction, (b) short direction

From field observation, several local buckled columns occurred in building C. These columns are located on the fourth story at column lines 3, 5, and 6 (see Fig. 2). Local buckling was near the end of the column and the column plates were no longer connected to each other. As a result, significant reductions of axial and moment capacities can be expected. Most long-direction and short-direction girders failed locally. In short-direction girders, many web members had local buckling at both ends of the members. In long-direction girders, web members were too weak to resist the shear force, leading to overall buckling. Bracing members also exhibited overall buckling.

from the above observation, it is apparent that many constituent members in this extensively damaged building behaved inelastically during the earthquake. The inelastic behavior of Pino Suarez building has been studied by several investigators. Most of the building's structural properties used in their studies are only estimated and not based on actual design information. The hysteretic behavior of constituent members is based on existing conventional models and is not derived from structural properties of building. Osteraas and Krawinkler (1989) use a two-dimensional structural model and E-W component earthquake records to study the performance of the building. Results indicate that structural response is highly dependent on column post-buckling behavior and that the effect of post-buckling loss of strength can produce large drifting in structural response.

The objective of this research is three-fold: 1) to study the hysteretic behavior of open-web girders, bracing members, and box columns of Pino Suarez building; 2) to develop nonlinear hysteresis models of these constituent members; and 3) to use them to investigate the building's performance during the 1985 Mexico earthquake. Here, a three-dimensional structural model is used in analysis and Mexico earthquake E-W, N-S, and vertical components are used as input. The structural properties of Pino Suarez building in this study are based on actual engineering design data.

2 HYSTERESIS MODEL OF INDIVIDUAL COMPONENTS

2.1 Long-direction girders

The hysteretic behavior of long-direction girders is investigated by analytical approach. This approach is based on finite-segment technique (Chen and Sugimoto, 1987; Cheng and Ger, 1990) for angle web members with load eccentricities and initial imperfections. Using this technique, each web member is divided into several segments in its longitudinal direction and the cross-section of each segment is divided into many small elements. When loads are applied to the member, each segment is deformed and may become partially plastic. The plastification of the cross-section can be detected by the material stress-strain relationship. Fig. 4 shows axial load versus deformation at mid-height of a single angle L2x2x1/4 subjected to an eccentric load with eccentricities $e_x = -0.41$ in. and $e_y = 0.804$ in. The member is divided into 16 segments, restrained about X-axis at the support, with no initial imperfection. Results calculated by this technique are very close to experimental results of Usami and Galambos (1971). By applying rotational increments to the two ends of a girder, hysteresis loops of a girder can be generated theoretically. Based on these loops, the hysteresis model of long-direction girders is developed as shown in Fig. 5. Using this model, the stiffness matrix of a girder can be determined according to the rules of the hysteresis model (Ger, 1990; Cheng and Ger, 1990).

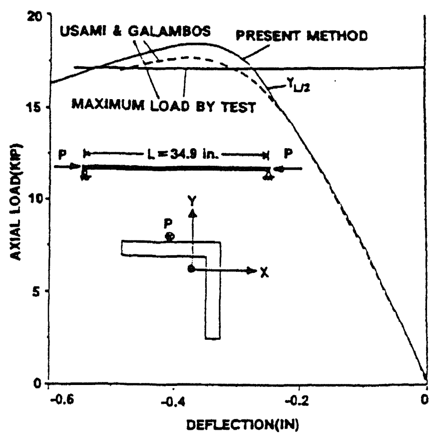

Figure 4. Displacement of mid-height of the member

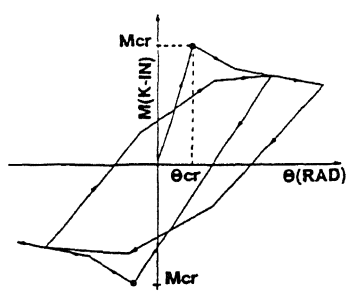

Figure 5. Hysteresis model for long-direction girders

2.2 Short-direction girders

In order to investigate the hysteretic behavior of short-direction girders, the hysteresis model (Ger, 1990) of a typical web member is first established based on experimental results (Chen, 1991). The experimental specimen is a box section made by stitch welding two equal-leg angles 2L-2.5x2.5x3/16 together as shown in Fig. 6. The member is pinned-end and subjected to axial load only. The material yield stress is 52.7 ksi and elastic modulus is 30542 ksi. Test results show that the hysteretic behavior is dominated by local buckling near the web's end. Local buckling occurs shortly after the axial compression load reaches the axial yielding load. Fig. 6 also shows hysteresis loops of solid lines generated by hysteresis model and those of dashed lines directly from the test. The envelope generated by the hysteresis model is close to test results. Using this hysteresis model, the hysteretic behavior of the girder is investigated by applying rotational increments to both ends of the girder. It shows that the hysteresis loops of short-direction girders are quite stable and close to bilinear behavior. Thus the hysteresis model for short-direction girders is considered a bilinear shape as shown in Fig. 7. It is interested to note that, instead of using stitch weld, Chen (1990) also tested a boxed angle member made by solid welding two angles. Test results indicate that local buckling was not developed until significant overall buckling occurred and that solid weld boxed members provide better ductility than stitch weld members do.

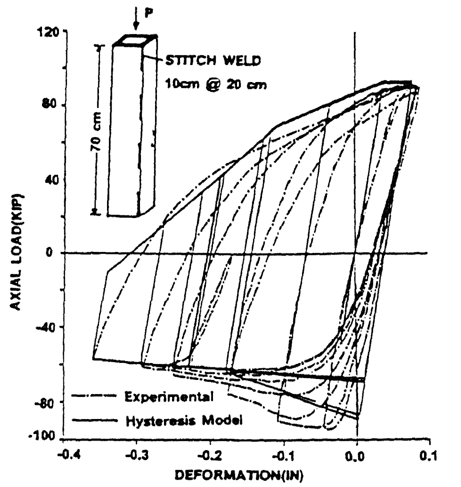

Figure 6. Web member hysteresis loops

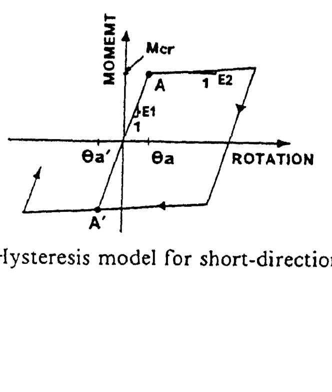

Figure 7. Hysteresis model for short-direction girders

2.3 Box columns

Observation of damage to building C in the field shows three local buckled columns located on the fourth story at column lines 3, 5, and 6. However, according to engineering data, column at line 3 is a compact section and columns at lines 5 and 6 are non-compact sections. Apparently, these columns underwent significant yielding before plates buckled. Bending moment combined with axial load and shear appear to cause severe inelastic deformations in plates of box columns. Since significant yielding precedes local buckling, the hysteresis model for box columns is assumed that local buckling will occur when the cross-section of a column reaches its fully plastic condition. The fully plastic condition of a box column can be detected by the interaction equation for box columns (Zhou and Chen, 1985). The axial hysteresis model and bending hysteresis model for box columns are shown in Figs. 8(a) and 8(b), respectively. In Fig. 8(a), P_{cr} is critical axial load. In Fig. 8(b), $(M_y)_{cr}$ and $(M_z)_{cr}$ are critical moments in the direction along the strong and weak axes, respectively. P_{cr}, $(M_y)_{cr}$, and $(M_z)_{cr}$ can be determined by the interaction equation for box columns. Since experimental results (Liew et al.,1989) show that axial load capacity drops significantly in the post-buckling region, it is assumed in the axial hysteresis model that axial load will drop from P_{cr} to $\beta' P_{cr}$ with zero rigidity in the post-buckling region as shown in Fig. 8(a). When axial force is in tension, the member resists tension yield load, P_y. The four plates separated due to failed welding and thereby caused significant reduction of bending strength. This behavior is confirmed by laboratory experiments with axial load columns (Chen, 1991). Therefore, it is assumed that flexural rigidity in the post-buckling region is nearly equal to zero and the column is still resistant to residual bending strength, expressed as βM_{cr} in Fig. 8(b).

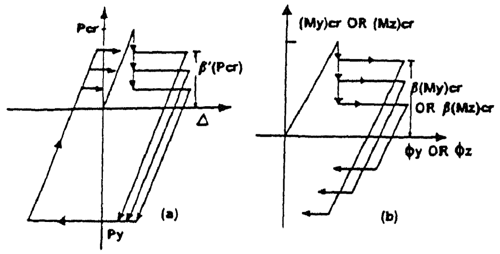

Figure 8. Hysteresis model for local-buckling columns, (a) axial load, (b) bending moment

2.4 Bracing members

The hysteresis model for bracing members of H-shape cross-section adopts the hysteresis model of Jain-Goel-Hanson's model (1980). However, several control points in Jain-Goel-Hanson's model are modified to fit the experimental results for wide flange section conducted by Black, Wenger, and Popov (1980). Hysteresis loops based on the adjusted model are then compared with the experimental loops. Using this adjusted model, the hysteresis loops for two braces, W6x20 and W6x16, with respective slenderness ratios 80 and 120 are calculated and compared with experimental loops. This comparison shows that the analytical loops are in favorable agreement with the experimental loops. The loops based on experimental and analytical approaches for a member with slenderness ratio equal to 80 are shown in Fig. 9. Since the slenderness ratios of bracing members in this building are close to 100 which is between the experimental range of 80 to 120, this model serves to estimate hysteretic behavior of the building's bracing members with consideration of actual slenderness ratios.

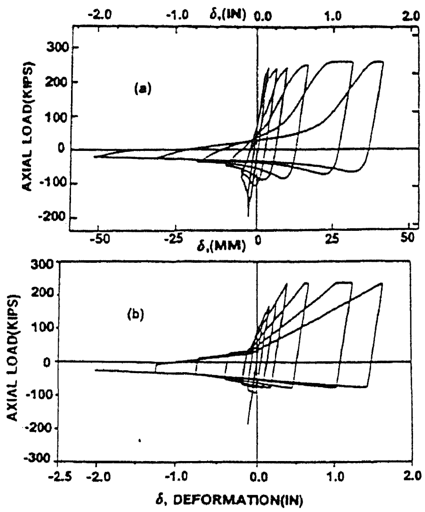

Figure 9. Hysteresis loops, (a) experimental results (Black et al., 1980), (b) analytical results

3 INELASTIC RESPONSE BEHAVIOR OF PINO SUAREZ BUILDING

Based on aforementioned hysteresis models, the three-dimensional inelastic analyses of this building are carried out by consideration of three-dimensional interacting ground motions, P-Δ effect, and the effects of failure ductility of girders and local buckling of columns. Here, the failure ductility of a girder is defined as $\mu_d = \delta_{max}/\delta_y$ where δ_{max} represents the maximum end rotation of a girder; δ_y is the end rotation of a the girder when its end moment reaches the critical moment, M_{cr}. In a given structure, the definition of system ductility is similar to the failure ductility of a girder. For system ductility, δ_y is the structural response when the first element of the structure reaches its critical load -- i.e., critical moment, plastic moment, or buckling load -- and δ_{max} represents maximum structural response. Results from inelastic analyses are described in the three cases of numerical studies discussed below.

3.1 Without consideration of failure ductility of girders and local buckling of columns

In this case, most long-direction girders and short-direction girders behave inelastically but not many columns yield. Yielding columns are located primarily on floors 2, 3, and 4 and near the service core. Structural response is quite stable and the building does not collapse. System ductility at each floor is calculated and compared with the building's original design ductility of 4. This comparison shows that system ductilities from floors 4 to 22 are greater than 4. Average ductilities of long-direction girders from floors 4 to 10 are greater than 4. Average ductility of long-direction girders on a given floor is obtained by averaging ductilities of all long-direction girders with ductilities greater than or equal to one on that floor. For short-direction girders, average ductilities from floor 3 through 22 are greater than 1.5 but less than 4.

3.2 With consideration of failure ductility of girders and without local buckling of columns

Two cases are investigated: 1) failure ductilities for long-direction and short-direction girders with a value of 4, and 2) failure ductilities for long-direction and short-direction girders with a value of 3. Results show that structural responses in case 2 are much larger than those in case 1. The smaller girders' failure ductilities underlie the larger structural responses. When girders reach their failure ductilities, their internal forces are released and redistributed to adjacent columns and bracings. Thus, column ends may yield due to redistributed forces from ductile-failed girders as well as additional story shear from buckled bracing members. As a result, P-Δ effect and story drift are increased. Responses at the top floor mass center in E-W direction for case 2 with and without consideration of P-Δ effect are compared in Fig. 10. As shown in this figure, building collapse is hidden unless P-Δ effect is considered. Since several local buckled columns have been observed in the remaining building C, it is believed that structural collapse is attributed to not only ductile-failed girders but also local buckled columns. Therefore, structural responses for case 2 cannot explain the actual collapse behavior of the building.

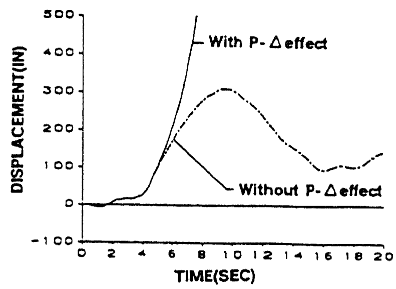

Figure 10. Comparison of structural responses at top floor mass center

3.3 With consideration of failure ductility of girders and local buckling of columns

Failure ductilities for long-direction and short-direction girders are assumed to be 4; $\beta = \beta' = 0.5$ are assigned to columns with consideration of local buckling. Results shown in Figs. 11(a), 11(b), and 11(c) correspond to translational responses in the top floor mass center's X and Y directions and torsional response in the Z direction, respectively. Displacement in the postive X direction increases dramatically from 10 to 12 seconds and in the negative Y direction from 13 to 15 seconds. Significant rotation develops in the negative Z direction from 16 to 20 seconds with a maximum value of about 14.5 degrees. The torsional effect on the collapse of this building is significant. The first local buckled column is on floor 4 at column line 5 at time equal to 4.9 seconds, consistent with field observations at building C. Due to load redistribution effects from the failed girders and columns, local bucklings occur at adjacent columns. At time equal to 15 seconds, most columns on floors 2, 3, and 4 lose their load carry capacities, which cause the building to tilt in the positive X direction and rotate in the negative Z direction.

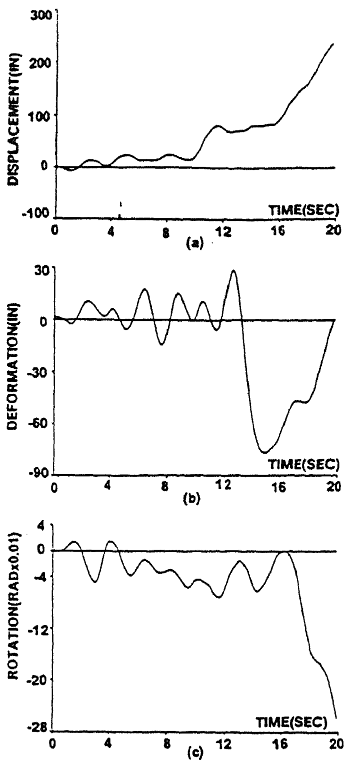

Figure 11. Responses at top floor's mass center (a) in X direction, (b) in Y direction, and (c) in Z direction

4 CONCLUSIONS

With laboratory tests and field observations, hysteresis models for truss-type girders, bracing members, and box columns with consideration of local buckling are developed to investigate Pino Suarez building's performance during the 1985 Mexico earthquake. It was found that most long-direction and short-direction girders exhibit severe inelastic behavior, and cannot provide enough ductilities to resist earthquake excitation. Also, system ductilities from floor 4 through 22 are greater than the building's design ductility of 4. It is believed that ductile breakdown of girders combined with local buckling of columns in the lower part of the building create a failure mechanism which causes significant story drift, building tilt, increasing P-Δ effect, and structural collapse. It should be point out that actual seismic load is much larger than design load. Adequate ductility factor of girders, continuous welding of web members , and symmetrical structural plans could improve structural performance.

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