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INELASTIC BEHAVIOR OF FULL-SCALE CONCENTRICALLY K-BRACED STEEL BUILDING

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SUMMARY

Reported in this paper are seismic test results on a full-scale six-story, 2 by 2 bay, steel building with concentric-K braces forming a "dual system". The full-scale seismic tests were run as a six-degree-of-freedom pseudo-dynamic system (on-line computer-actuator testing system). The test results showed not only a lot of capacity of the pseudo-dynamic testing technique but also the significant interaction between the post-buckling behavior of the bracing members and the inelastic behavior of the moment-resisting frames. In addition, it was recognized that early fracture of the cold-formed bracing members placed a limit upon the seismic capability of the test structure.

INTRODUCTION

As part of the U.S.-Japan Cooperative Research Program Utilizing Large-Scale Testing Facilities, a six-story, 2 by 2 bay, concentrically K-braced steel building was constructed in full-scale, and subjected to the Miyagi-Ken-Oki Earthquake with different intensities by using the pseudo-dynamic (PSD) test facilities. To simulate working load conditions, the earthquake motion was scaled to 65 gal (the Elastic test); for a moderate earthquake the peak intensity was set at 250 gal; (the Moderate test) and the maximum earthquake simulation test was run with 500 gal peak input level (the Final test).

The test building was designed to satisfy the requirements of both the 1976 Uniform Building Code and the 1981 Seismic Design Code of Japan. The plan and elevation of the structure are shown in Fig. 1. This building structure consisted of two unbraced moment-resisting frames (Frames A and C) and a concentrically K-braced frame (Frame B). The actual dead load of the whole building was 523.6 tons (1154 kips) while the design dead load was 634.7 tons (1399 kips). Member sizes of the test structure are summarized in Table 1. Principal design criteria and details were: 1) The design base shear coefficient should be 0.197; 2) Live load and exterior wall weight should not be included in the design earthquake lateral forces; 3) Columns and girders should be wide flange shapes of ASTM A36 structural steel, and braces be cold-formed square tubing section of ASTM A500 Grade B structural steel; 4) Girders and floor beams should act compositely with the floor using shear studs; 5) Braces should resist both tension and compression; 6) Girder-to-column connections should be moment connections in loading direction; 7) Column bases should be fixed; 8) Braces should be directly welded to surrounding frames without gusset plates.

OVERALL RESPONSE AND GROSS BEHAVIOR OF TEST STRUCTURE

In the Final test, at the 1st through 5th stories, the braces buckled in-plane and/or out-of-plane of the frame. In particular, the braces at the 2nd and 3rd stories buckled laterally 21 to 24 cm (8.27 to 9.54 in.) with local buckling and tearing at the mid-span and both ends of the braces. In this Final test, it should be noted that the maximum roof displacement reached 22 cm (8.66 in.) and the maximum base shear was 331 tons (729 kips) at an interstory displacement of 2.3 cm (0.91 in.) in the 1st story. Fig. 2 shows the relationship of story shear force vs. interstory displacement in the Final test (solid lines) of the 2nd story. Finally, the test structure sustained an interstory drift angle of 1/57 at the 2nd story, and the north side brace at the 3rd story ruptured completely at 11.37 sec. into the earthquake record.

In Fig. 2, analytical results are also shown with dotted lines. The DRAIN-2D computer program developed by Kanaan and Powell (Ref. 1) was used for the analysis. In this analysis, a degrading hysteresis model was applied to the post-buckling and cyclic behavior of braces. This model was developed by modifying the Jain-Goel-Hanson hysteresis model of braces (Ref. 2). The comparison indicates that the analytical model gives good prediction on the overall behavior of the test building.

STORY SHEAR FORCE CARRIED BY BRACES AND BY MOMENT FRAMES

The ratio of story shear forces carried by seismic resistant braces and by moment-resisting frames is a basic parameter not only to design such a dual system as the test structure but also to evaluate the actual seismic performance, in particular, after buckling of the braces. Here, the properties of the braces at each story are listed in Table 2.

Fig. 3 gives time histories on the ratios of story shear forces carried by the braces and by the moment frames to those induced by the actuator forces respectively, for the 2nd story in the Final test (solid lines). It can be seen in this figure that the line on the braces turns down, and that on the moment frames turns up, having an intersection at the time of 7.25 sec.. Around this time the north brace began to develop severe buckling with the compressive strength much decreased. This means that the deterioration of the brace capacity forced the moment frames into inelastic activity. On the contrary in the Elastic test, even in the Moderate test, the ratios were very stable. Table 3 gives the ratios (in average) for the two tests. These values were close to those expected in the design process. For the Final test, the ratios were also fairly well predicted by the analysis as shown in Fig. 8 (dashed lines). However, this prediction was considerably disturbed by the local failure of the braces such as local buckling, kinks and cracking.

LOCAL BEHAVIOR IN THE FINAL TEST

Behavior of Braces The axial force vs. axial displacement relationships for the pair of the braces in the 2nd story are shown in Fig. 4. As seen from Fig. 4, both the braces did not show tensile yielding at all. This is because the concentrated downward force acting at the mid-span of the 3rd floor girder, which was produced by the difference of the axial forces between the compressive and tensile braces, induced downward displacement at the mid-span. This displacement made the tension side brace relax, resulting in no tensile yielding. This peculiarity is very significant to constitute a structural model for such a K-braced system (Ref. 3).

Fig. 5 shows the time history of each axial force of the 2nd story braces, that of the vertical displacement at the brace junction at the 3rd floor and that of the story displacement in the 2nd story. As seen from these time histories, the increase of the vertical displacement at the brace junction took place when the decrease of the compressive strength of the buckled brace started. Then, it continued till the reversal of the sign of the story displacement.

At the termination of the Final test, the north brace at the 3rd story ruptured completely, and also both the braces at the 2nd story had severe cracks; 25 to 75 percent of sectional area was teared at the both ends and mid-spans of them. By sight observations, the growth of those cracks was found to be very rapid; after the local-buckling occurrence, a few cycles of inelastic axial deformation sufficiently developed such fatal cracks. The fragility of cold-formed square tubular sections against local buckling and cracking should be marked pronouncedly, because the early fracture of braces would cause dominant interstory drift into the story concerned or into vicinity stories (Ref. 4).

Behavior of Girder-to-Column Panels The girder-to-column connection panels of the test structure did not need special strengthening in the working stress design. However, this does not always assure the later yielding of the panel-zones than that of the adjacent members in the ultimate state of the structure. The shear deformation vs. panel moment relationships of the interior joint panels at the 4th floor are shown in Fig. 6. The panel moment was estimated as the sum of face moments of the upper and lower columns connected to the panel-zone. The calculated yield panel moments by the AIJ Steel Design Standard (Ref. 5) are also shown with broken lines in the figure. As seen in the figure, the joint panels showed stable hysteresis loops with large shear deformations. Further, the plastic shear deformation in the panel of the braced-bay column showed a drift into one side because of the axial thrust in the column.

Behavior of Columns Axial force vs. axial deformation relationships are shown in Fig. 7 for the 3rd story interior braced-bay columns (B2-columns). Obviously it showed likely behavior of yielding. However, the calculated axial yield strength (Ny) is about two times as high as the force level corresponding to the outward yielding. Further, the moment vs. axial force correlation diagrams of both the ends of the column indicates that the column would not have yielded as shown in Fig. 8. This contradictory appearance is believed to be resolved by the fact that a column adjacent to a much yielded joint panel yield under a relatively small amount of bending moment because of early local yielding in the section of the column close to the panel. However, this interpretation is not yet confirmed stringently in a quantitative manner.

Behavior of Girders Remarkable yielding was not observed in the girders. Fig 9 shows the moment vs. curvature relationships measured at two sections of the 2nd floor girder in Frame A. Positive and negative stiffnesses calculated by the AIJ recommendations and also the stiffness of the bare steel girder are shown in the figure. The test results show fairly good agreement with the calculated stiffnesses.

Also in Fig. 9, apparent yielding under the positive bending and stiffness reduction under the negative bending can be found. The theoretical full plastic moment of the composite girder is far larger as shown in the figure. Slippage between the steel girder and the concrete slab can explain this apparent yielding. Namely, it can be considered that the composite girder comes to be unable to sustain the increase of the flexural moment when the slippage occurs, and then the moment vs. curvature relationship shows such apparent yielding. After the above stiffness reduction under the negative bending, the stiffness well fell on the stiffness estimated for the bare steel girder (Fig. 9), so that the slippage between the steel girder and concrete slab is the most possible reason for the stiffness reduction.

CONCLUSIONS

1) The overall behavior as well as the local behavior of a full-scale K-braced steel structure were obtained by a lot of capacity of the pseudo-dynamic testing technique that simulated realistic earthquake responses.
2) The ratio between the story shear forces carried by the K-braces and by the moment-resisting frames were stable and predictable before the local failure of the braces. However, after the failure, the ratios changed so largely that further studies are needed to estimate exactly the overall response on the basis of predicting such failure. Alternatively, to prevent the early failure of cold-formed braces, some counterplans should be immediately considered in practical design aspects.
3) Data on the members and sections in the full-scale structure were verified to be reliable and useful for attaining knowledge on the behavior of individual members in actual structures. However, some local phenomena in the members have not yet been explained quantitatively.

REFERENCES

(1) Kanaan, A. E. and Powell, G. H. (1973). "General purpose computer program for inelastic dynamic response of plane structure." Report No. UCB/EERC 73-6, Uni. of California, Berkeley, April.
(2) Jain, A. K., Goel, S. C., and Hanson, R. D. (1978). "Hysteresis Behavior of bracing members and seismic response of braced frames with different proportions." Report No. UMEE 78R3, University of Michigan, Ann Arbor, July.
(3) Yamanouchi, H., Fukuta, T., Yasuda, S., and Kato, B. (1986). "Seismic performance of K-braced steel structure." Proceedings of Pacific Structural Steel Conference, Auckland, New Zealand, August 4-8, vol. 1, 97-116.
(4) Tang, X and Goel, S. C. (1987). "Seismic analysis and design considerations of braced steel structures." Research Report UMCE 87-4, University of Michigan, Ann Arbor, April.
(5) AIJ. (1970)."Design standard for steel structures." the Architectural Institute of Japan, Tokyo.

Table 1. Member Size of Test Structure

(a) Column Schedule

STORY \ MARK	C1	C2	C3	C4	C5
6-5	W 10 x 33	W 10 x 33	W 10 x 49	W 10 x 33	W 12 x 40
4-3	W 10 x 39	W 12 x 53	W 12 x 65	W 10 x 60	W 12 x 72
2	W 12 x 50	W 12 x 65	W 12 x 79	W 12 x 79	W 12 x 106
1	W 12 x 65	W 12 x 87	W 12 x 87	W 12 x 106	W 12 x 136

(b) Girder Schedule

FLOOR \ MARK	G1	G2	G3	G4
RFL-6FL	W 16 x 31	W 16 x 31	W 18 x 35	W 21 x 50
5FL	W 16 x 31	W 18 x 35	W 18 x 35	W 21 x 50
4FL	W 18 x 35	W 18 x 35	W 18 x 35	W 21 x 50
3FL	W 18 x 35	W 18 x 40	W 18 x 35	W 21 x 50
2FL	W 18 x 40	W 18 x 40	W 18 x 35	W 21 x 50

(c) Brace Schedule

STORY \ MARK	BR1
6	ST 4x4x1/5.56
5	ST 5x5x1/5.56
4	ST 5x5x1/4
3-2	ST 6x6x1/4
1	ST 6x6x1/2

Table 2. Brace Properties

Story \ Item	A (cm²)	r (cm)	L (cm)	L/r	KL/r (K=0.7)	P_{yn} (ton)	Pyp (A F) (ton)
6 S N	17.21	3.94	442.3	112.3	78.6	40.6	74.5
5 S N	. 21.85	4.98	441.0	88.6	62.0	59.5	84.8
4 S N	29.61	4.88	435.7	89.3	62.5	81.1	117.0
3 S N	36.06	5.92	434.0	73.3	51.3	109.5 113.8	140.3 147.8
2 S N	36.06	5.92	432.9	73.1	51.2	109.6 111.0	140.3 142.8
1 S N	66.84	5.61	513.4 512.4	91.5 91.3	64.1 63.9	192.1 191.3	292.8 290.1

A=Sectional Area, r=radius of gyration,
L=Clear Length of Braces,
Pyn=Buckling Load; $Pyn=Pyp\{1-0.545(\lambda-0.3)\}$
Pyp=Axial yield strength, $\lambda = \frac{1}{\pi}\sqrt{F/E}\cdot KL/r$, F=yield stress
(by AIJ Plastic Design Recommendation for Steel Structure, 1975)

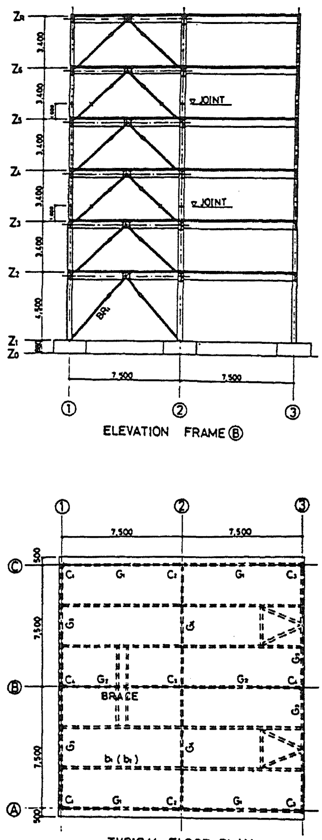

Fig. 1 Plan and Elevation of Test Structure

Table 3. Share Ratio of Braces in Story Shear Forces

STORY	ELASTIC	MODERATE	FINAL	STATIC ANALYSIS ELASTIC	STATIC ANALYSIS ULTIMATE
6	0.60	0.61	0.60	0.74	0.76
5	0.68	0.69	0.68	0.78	0.49
4	0.71	0.72	0.75	0.78	0.51
3	0.75	0.76	0.51*	0.82	0.56
2	0.71	0.71	0.22*	0.78	0.48
1	0.83	0.65*	0.64*	0.85	0.58

* values at the final stage of test

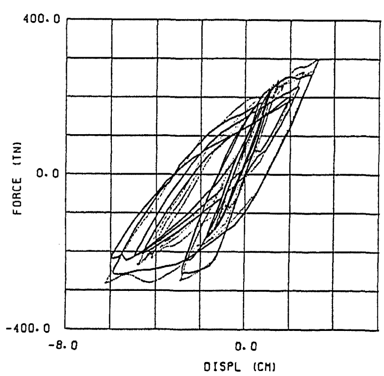

Fig. 2 Story Shear Forces vs. Interstory Displacements

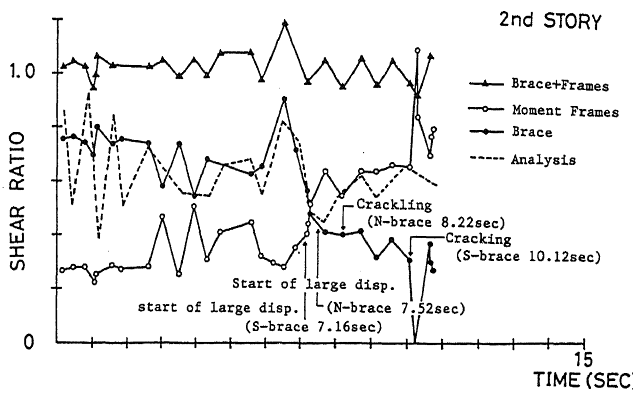

Fig. 3 Ratios of Story Shear Forces Carried by Braces and by Moment Frames to Those Induced by Actuator Forces

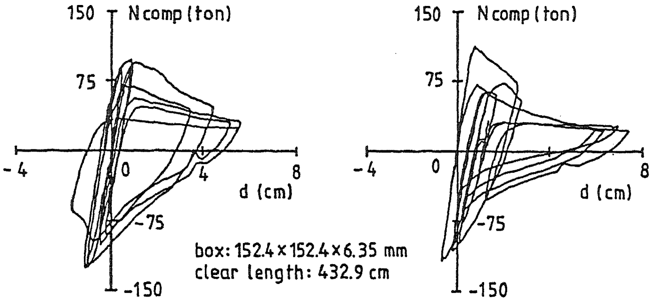

Fig. 4 Axial Force vs. Axial Displacement Relationships of K-Braces in the 2nd Story

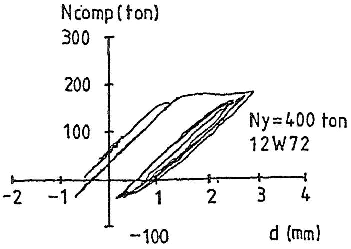

Fig. 7 Axial Force vs. Axial Deformation relationship (3rd Story B2-Column)

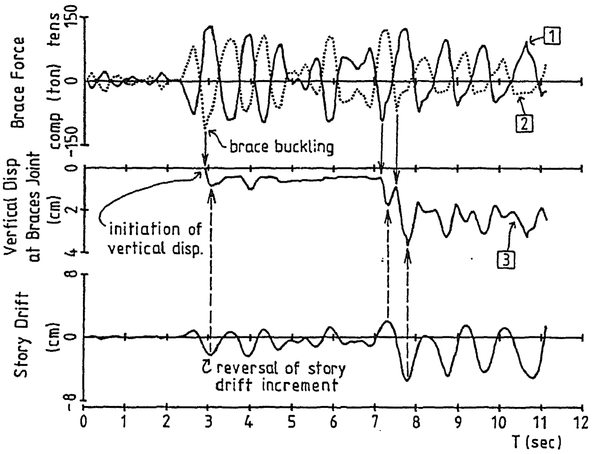

Fig. 5 Time Histories of Brace Axial Force, Vertical Disp. at Brace Junction and Story Displacement

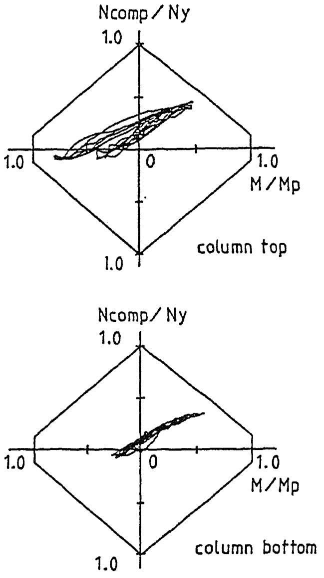

Fig. 8 M-N Diagram (3rd Story B2-Column)

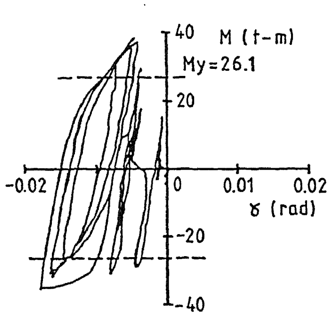

Fig. 6 Shear Deformation vs. Panel Moment Relationships of the Girder-to-Column Joint Panels at the 4th Floor

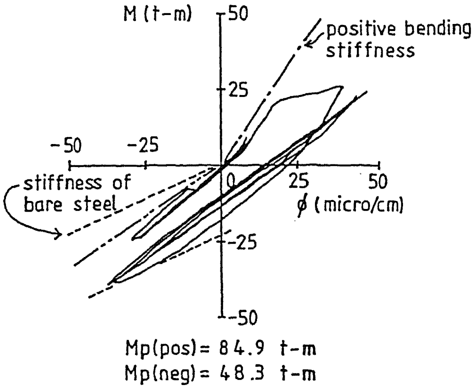

Fig. 9 Moment vs. Curvature Relationships of the 2nd Story Girder