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

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SUMMARY

Seismic tests on a six-story, 2 by 2 bay, full-scale steel building with eccentric K-braces were run as a six degree-of-freedom pseudo-dynamic system; which followed the similar tests on the building with concentric K-braces. The input excitation was the 1952 Taft (NS) Earthquake scaled to its maximum intensity at two levels; 65gal to simulate working load conditions and 500gal to simulate maximum earthquakes. A sinusoidal excitation input test followed them to find out the strength, deformability and failure mechanism in the ultimate stage. The results validated the design philosophy of the eccentrically K-braced system which showed remarkable energy absorption capacity in the shear links.

INTRODUCTION

As part of the U.S.-Japan Joint Research Program (Ref.1), full scale seismic tests on a steel office building were carried out at the Building Research Institute (Tsukuba, Japan) for comparing actual full-scale building behavior with small-scale model or member behavior and for assessing the damage and safety levels after an earthquake of buildings designed to satisfy the requirements of current seismic design codes.

In this paper, presented are the results of the seismic tests on a six-story, 2 by 2 bay, full-scale steel building with eccentric K-braces which followed the similar tests on the building with concentric K-braces (Ref.2). All the concentric K-braces, some of which had buckled in the preceding tests, had been cut out and the cracked R/C slabs were repaired by epoxy injection. The eccentric K-braces were newly installed and welded to the existing frames by field welding.

All the seismic tests were run as a six degree-of-freedom pseudo-dynamic (PSD) system. The input excitation was the 1952 Taft Earthquake, and the maximum intensity of it was set at two levels. In order to find out the strength, deformability and failure mechanism of the eccentric K-bracing system in the ultimate stage, a sinusoidal excitation input test followed these seismic response tests. In addition to these seismic tests, supplementary elastic tests were carried out to find out the dynamic characteristics of the test building.

ECCENTRIC K-BRACES AND SHEAR LINKS (TEST BUILDING)

The plan and elevation of the eccentrically K-braced building are shown in Fig.1. The typical shear link details are shown in Fig.2. The sectional shapes of the braces were rectangular tubes and they were arranged so that their weak axes coincided with the plane of the braced frame. The braces were so selected that they would not buckle until the shear links in the girders yield due to the shear force produced by the brace axial force. The member sizes of the braces are listed in Table 1. Those of the columns and girders are shown in Ref.2. This structural system (eccentric K-bracing system) is aimed to make the shear links yield prior to the buckling or yielding of the eccentric K-braces, and to absorb the earthquake input energy in these shear links thanks to their stable restoring force characteristics. As shown in Fig.2, three vertical stiffeners (PL12) were welded on one side of each of the shear links to prevent them from shear buckling in the early stage of loading. Sub beams in the transverse direction were installed to restrain the distortion of the girders at both ends of the shear links on one side of the girders (Fig.1(a)).

NATURAL PERIODS AND DAMPING RATIOS

Table 2 shows the dynamic characteristics of the test building, which were calculated by frame analysis (Ref.3) and observed in the supplementary elastic tests; the Free and Forced Vibration Tests, the Each Floor Level Loading (FLL) Tests, the PSD Free Vibration Tests and the PSD Pulse Response Tests.

Natural periods of each of the modes obtained by the additional tests conducted prior to the PSD inelastic seismic response tests (the Inelastic PSD Final Test and the Inelastic PSD Sine Wave Input Test) were well coincided with those calculated by frame analysis. The fundamental natural period after the seismic response tests (0.679-0.686 sec.) was almost 1.2 times longer than that measured before the tests.

The damping ratios of 1st and 2nd modes obtained by the Forced Vibration Test conducted before the seismic response tests were 0.35% and 0.31%, respectively. On the basis of these results, Rayleigh type damping was used in the PSD seismic tests; both the 1st and 2nd damping ratios were set at 0.35%. However, the apparent 1st damping ratios observed in the PSD Free Vibration Test and the PSD Pulse Response Tests were 1.17-5.90% and they were larger by 0.82-5.55% than those (0.35%) used in the PSD seismic tests. This difference seems to come from the additional damping produced by the testing system.

PSEUDO-DYNAMIC SEISMIC RESPONSE TESTS AND SINE WAVE INPUT TESTS

The PSD seismic response tests and the sine wave input tests were carried out by using a six degree-of-freedom pseudo-dynamic testing technique. In the PSD seismic response tests, the input excitation was the NS component of the 1952 Taft Earthquake accelerogram and the maximum intensity of it was set on two levels, that is, 65gal to simulate the working load conditions (the Elastic PSD Test) and 500gal as the maximum earthquake (the Inelastic PSD Final Test). In the sine wave input tests, the maximum acceleration was set at three levels; 92.6-100gal, 270gal and 320gal. These sine wave input tests were carried out in place of the static cyclic loading tests. In these tests, the periods of the input sine waves were so selected that they coincided with those of the 1st mode of the test building in the stationary response conditions under the different excitation levels. Then, the tests was continued for about 0.5-1.0 cycle on each of the excitation levels. The maximum responses obtained from these tests are tabulated in Table 3.

<u>Elastic PSD Test</u> The test was continued up to 17.92 sec.. The maximum roof horizontal displacement was 1.41 cm at 12.39 sec., the maximum story drift angle was 1/1339 rad. in the 3rd story and the maximum shear force in the 1st story was

59.6 tonf which corresponds to 0.114 of the base shear coefficient (ratio between the shear force in the 1st story and the total weight of the building - only the actual weight of the test building as built is used in this estimation).

Inelastic PSD Final Test After the Elastic PSD Test, the 1952 Taft Earthquake scaled to 500 gal was input and the test was continued for 17.10 second. The time history of the horizontal displacement at the roof floor level is shown in Fig.3 and the relationships of story shear force vs. story drift in the 1st and 2nd stories are shown in Fig.4. In Fig.5, shown are the shear force vs. shear deformation relationships in the shear links of the web plates of the girders where the eccentric K-braces are connected. The shear force is estimated from the vertical components of the axial forces of a pair of braces.

The roof horizontal displacement reached its maximum value of 8.8 cm, at 14.505 sec.. As shown in Fig.4, the relationships of story shear force vs. story drift obtained in the 1st and 2nd stories showed stable hysteresis loops without deterioration. As shown in Table 3, the maximum responses of the story drift angle are larger in lower stories, that is, 1/150 rad. in the 1st story, 1/156 rad. in the 2nd story and so forth. The maximum shear force in the 1st story was 312.8 tonf which corresponds to 0.597 of the base shear coefficient. This maximum shear force corresponds to 0.383 of the base shear coefficient if live loads for seismic design and exterior wall weight besides actual dead weight of the test building (816.5 tonf) are considered.

The shear deformations in the shear links where several yieldings were observed were relatively large in the lower three stories, and the shear deformation angle reached 0.047 rad., 0.029 rad. and 0.015 rad. in each of the stories from the lowest story around 14.5 sec.. Although these shear links yielded so much, they kept relatively large stiffness in the plastic range. Besides these yieldings in the shear links, some other yieldings were observed in the columns and in the gusset plates connectting braces and girder. However, the apparent damage is not severe. In Fig.6, the damage patterns of the braced frame after the Inelastic PSD Final Test are shown.

Inelastic PSD Sine Wave Input Test In order to find out the strength, deformability and failure mechanism of the eccentrically K-braced structural system in the ultimate stage, three levels of sine wave input tests were conducted following the Inelastic PSD Final Test. The time history of the horizontal displacement at the roof floor level is shown in Fig.7 and the relationships of story shear force vs. story drift in the lowest two stories are shown in Fig.8. In these figures, the test results obtained in the three independent tests are simply combined and shown. Therefore, the figures do not mean continuous responses.

In Figs.7 and 8, the origins of the figures coincided with the residual displacements at the beginning of the sine wave input tests. Therefore, the maximum displacements in these figures are smaller than those shown in Table 3 owing to the residual displacements after the Inelastic PSD Final Test.

As shown in Fig.8, the story shear force vs. story drift relationships showed slight degradation successively in the last one and a half cycle. This degradation was due to the local deformation in the girder web and the out-of-plane deformation of the gusset plate near the joint of the braces in the 1st story and the girder of the 2nd floor level. Because of the out-of-plane deformation of the gusset plate, lateral-torsional deformation occurred in the girder and the braces in the 1st story initiated to buckle. Therefore, the test was stopped when the roof horizontal displacement reached its maximum value of 23.2 cm. The story drift angles in the lowest three stories finally reached 1/46.4 rad. in the 1st story, 1/52.7 rad. in the 2nd story and 1/96.7 rad. in the 3rd story as shown in Table 3.

The maximum story shear force in the 1st story was 359.9 tonf which corresponds to 0.687 of the base shear coefficient, and at this stage the story drift angle in the 1st story was 1/102 rad.. In this estimation, only the actual weight of the test structure was considered as the building weight. If live loads for seismic design and exterior wall weight besides actual weight of the test structure are considered, the base shear coefficient changes to 0.441.

CONCLUSION

In the Inelastic PSD Final Test, the maximum base shear coefficient was 0.597, the roof horizontal displacement reached its maximum 8.8 cm and the maximum story drift angle reached 1/150 rad. in the 1st story. At the same time, large amount of shear yielding was observed in the lowest three stories and the shear deformation angle attained 0.047 rad. in the shear link of the second floor level. In spite of much yielding in the shear links, their hysteresis loops were quite stable and the observed damage of the test building was little.

In the PSD Sine Wave Input Test, the maximum base shear coefficient was 0.687, the maximum roof horizontal displacement was 23.2 cm and the maximum story drift angle was 1/46.4 rad. in the 1st story. The strength of the test building gradually decreased and the test building almost reached its ultimate stage. Finally, local deformation in the girder web next to the shear link and severe out-of-plane buckling of the gusset plate at the brace-girder junction were observed. Also the lateral-torsional deformation of the girder occurred being associated with the out-of-plane buckling of the brace.

The results of the seismic response tests on the full-scale six-story eccentric K-braced steel building using the multi-degree-of-freedom pseudo-dynamic testing system at the Building Research Institute validated the design philosophy of the eccentric K-bracing system which showed remarkable capacity of energy absorption.

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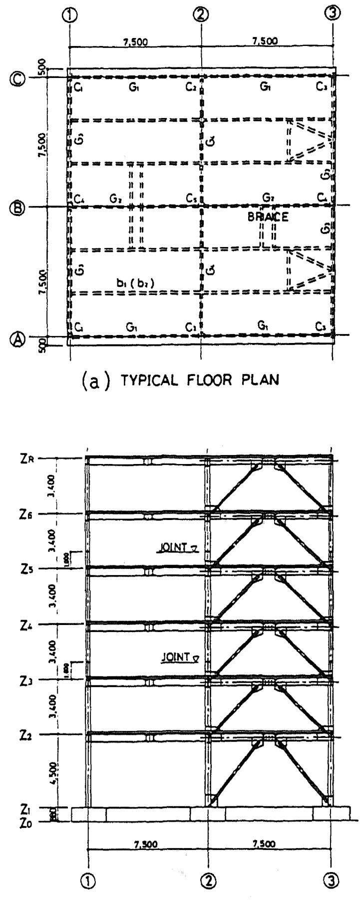

(a) TYPICAL FLOOR PLAN

(b) ELEVATION FRAME Ⓑ

Fig.1 Test Building

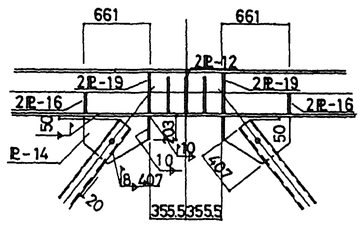

Fig.2 Shear Link Details

Table 1 Brace Sizes

STORY	BRACE
6 - 5	Tube 8x6x0.313 Box-203.2x152.4x7.95
4 - 1	Tube 8x6x0.375 Box-203.2x152.4x9.53

Note: ASTM A500 GRADE B structural steel

Table 2 Natural Periods and Damping Ratios

TEST	NATURAL PERIOD(sec) 1st	2nd	3rd	DAMPING RATIO(%) 1st	2nd	3rd
Analysis	0.595	0.214	0.119	—	—	—
Free VT Test #3	0.565~0.567			0.35~0.37(0.6~2.2ton)*1		
Forced VT Test #3	0.568	0.201	—	0.35	0.31	
FLL Test #4	0.545	0.193	0.106	—	—	—
Elastic PSD Test - 65gal peak input						
PSD F-VT Test #4	0.553	—	—	1.25(6~10mm)*2		
PSD Pulse Test #1	0.550	—	—	3.10(1.4~4mm)*2		
PSD Pulse Test #2	0.554	—	—	2.68(2.5~4.5mm)*2 1.17(4.5~12mm)*2		
PSD Pulse Test #3	0.550	—	—	—		
PSD Pulse Test #4	0.553	—	—	5.90(2.3~5mm)*2 3.40(5~12mm)*2		
Inelastic PSD Final Test - 500gal peak input Inelastic PSD Sine Wave Input Test - 92.6~320gal peak input						
Free VT Test #4	0.679~0.686			0.50~0.55(0.5~1.7ton)*1		
Forced VT Test #4	0.680	0.229	—	0.48	0.34	—

Note: *1) Pulling force *2) Roof displacement amplitude

Table 3 Maximum Responses

TEST	NOMINAL DAMPING COMBINATION	DISPLACEMENT AT FLOOR LEVELS	(cm)	(sec)	INTERSTORY DISPLACEMENT	(cm)	(sec)	DRIFT ANGLE (radian)	BASE SHEAR (ton)
ELASTIC		RF	-1.405	12.39	6	-0.226	12.39	1/1504	
PSD TEST	0.35 0.35	6F	-1.204	12.38	5	-0.240	12.40	1/1417	+59.6
	2 90	5F	-0.988	12.37	4	-0.239	12.36	1/1423	12.66sec
	90 90%	4F	-0.749	12.37	3	-0.254	12.36	1/1339	
65gal	(1st-6th)	3F	+0.512	12.65	2	-0.228	12.37	1/1491	-59.2
peak input		2F	+0.305	12.66	1	+0.305	12.66	1/1475	12.93sec
INELASTIC		RF	-8.841	14.505	6	-0.935	7.275	1/364	
PSD FINAL	0.35 0.35	6F	-8.337	14.510	5	-1.236	7.285	1/275	+293.6
TEST	5 90	5F	-7.560	14.515	4	-1.279	7.295	1/266	14.175sec
	90 90%	4F	-6.586	14.515	3	+1.512	7.040	1/225	
500gal	(1st-6th)	3F	-5.170	14.520	2	-2.173	14.525	1/156	-312.8
peak input		2F	-3.004	14.520	1	-3.004	14.520	1/150	14.515sec
INELASTIC		RF	-23.239	1.75	6	+1.090	1.32	1/312	
PSD SINE WAVE	0.35 0.35	6F	-22.454	1.75	5	+1.428	1.36	1/238	+359.2
INPUT TEST	5 90	5F	-21.308	1.75	4	+1.832	1.25	1/186	0.85sec
	90 90%	4F	-19.671	1.75	3	-3.517	1.75	1/96.7	
92.6-320gal	(1st-6th)	3F	-16.154	1.75	2	-6.457	1.75	1/52.7	-359.9
peak input		2F	-9.697	1.75	1	-9.697	1.75	1/46.4	1.03sec

Note: 1) Displacement and shear force to the left in Fig. 1 are positive.
2) The origins of displacements at floor levels in Inelastic PSD Sine Wave Input Test are the same as those in Inelastic PSD Final Test.

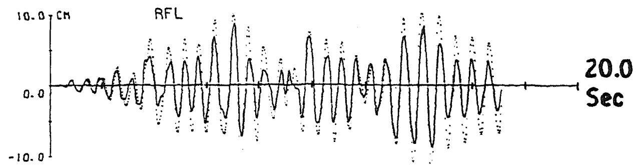

Fig.3 Time History of Hori. Displacement (Final)

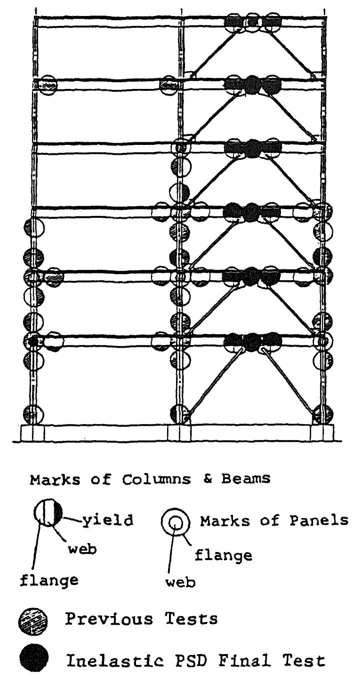

Fig.6 Damage Pattern of Braced Frame (Final)

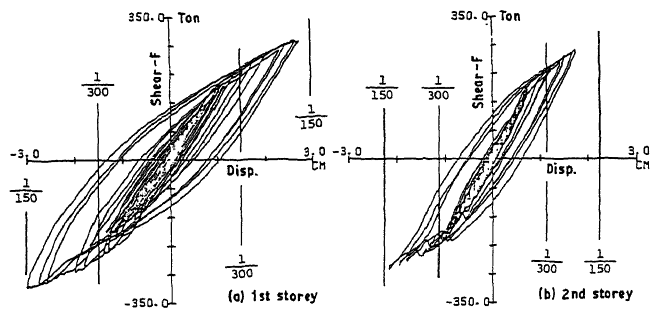

Fig.4 Inter-Story Behavior (Final)

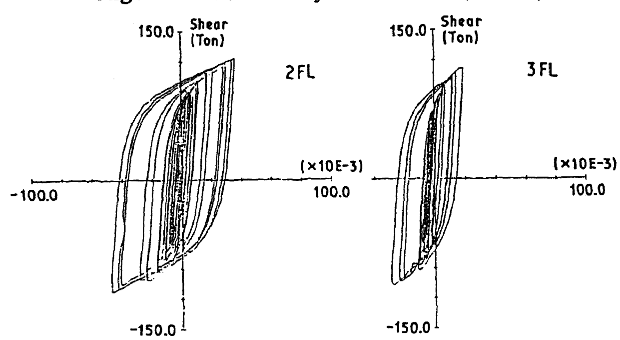

Fig.5 Shear Link Behavior (Final)

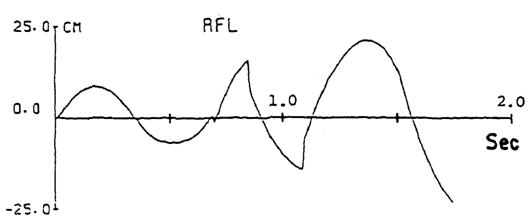

Fig.7 Time History of Hori. Displacement (Sine Wave Input)

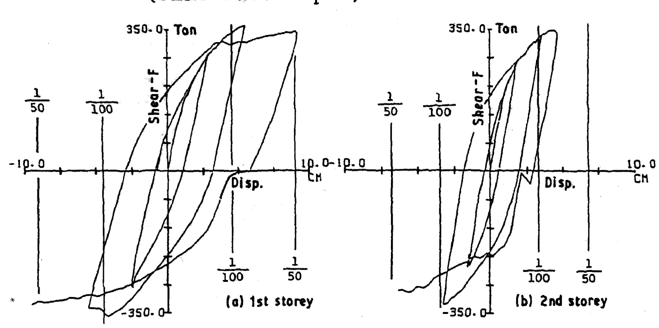

Fig.8 Inter-Story Behavior (Sine Wave Input)

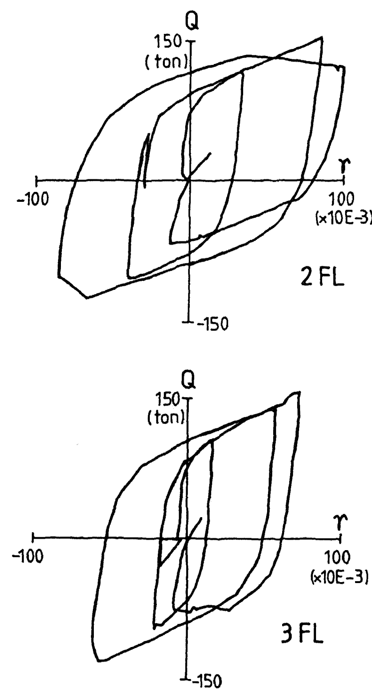

Fig.9 Shear Link Behavior (Sine Wave Input)