

An Experiment on Steel Chevron Braced Moment Resisting Frames

1 層 1 スパン K 形ブレース付ラーメン架構の実験

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Abstract

An experimental study was initiated to study the behavior of a Steel Chevron-Braced, Moment Resisting Frames (CB-MRFs) subjected to cyclic loads. Four specimens that represent Japanese practice but cover a range of possible member proportions were designed. One of the specimens has been tested. This specimen was provided with a rather small section for the beam intersected by braces. After the braces buckled, this beam developed severe vertical deflection, associated with plastic hinging and distortion at midspan of beam. Consequently, the braces deformed primarily in contraction and did not develop their yield strength in tension. The measured lateral strength was smaller than the predicted value. Nonetheless, the specimen exhibited stable behavior to a large story drift of plus-and-minus 0.05 rad.

Keywords: Concentrically Braced Moment Resisting Frame, Cyclic loading, Plastic Mechanism.

1. Introduction

Steel braced frames in Japan typically comprise chevron braces placed in a moment resisting frame. Such systems may be referred to as Chevron Braced Moment Resisting Frames (CB-MRFs). Despite their wide use in office buildings, shopping centers, parking buildings, etc., there is limited guidance on the design of concentrically braced moment resisting frames: The member proportion rules and connections and details rely very much on the judgment of each structural engineer. In contrast, AISC341 [1] in the US and EC8 [2] in Europe provide extensive provisions for steel concentrically braced frames.

Therefore, an experimental program was initiated with the objective to develop design recommendations for CB-MRFs. Four large-scale specimens were designed with different sections for beams, columns and braces to cover a wide range of design proportions. For far, one specimen has been tested.

2. Literature Review

Seki et al. [3] conducted six specimens scaled to 60% of a typical building to find that CB-MRFs and their bracing connections can achieve ductile behavior to a story drift ratio of 0.04 rad. These specimens featured a weak beam whose yielding prevented the braces to reach the tensile yield strength.

Okazaki et al. [4], conducted a dynamic test of a single-story, single-bay specimen whose bracing connection adopted the AISC suggestion to secure an

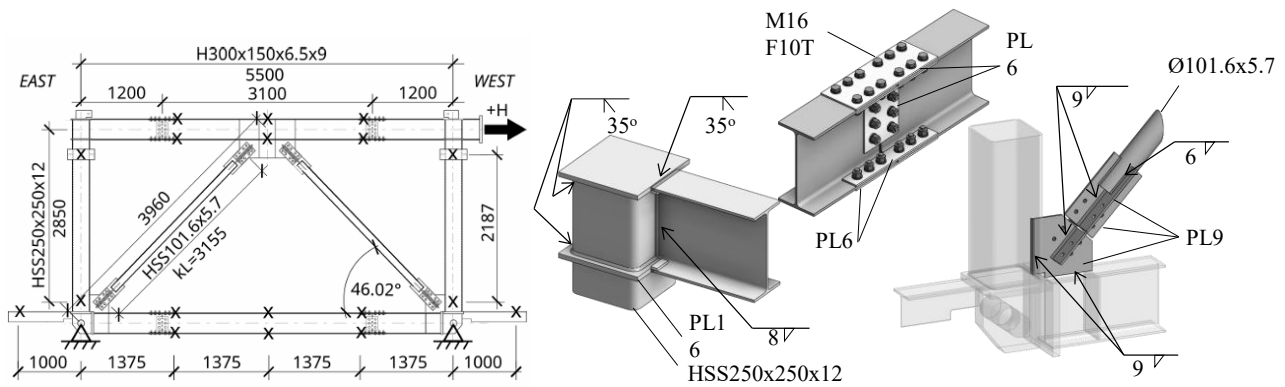
elliptical fold line in the gusset plate. The braced fractures at a story-drift ratio of 0.03 rad, but no discernible damage was observed in the bracing connection. Elastic deformation of the beam prevented the braces from reaching the tensile yield strength.

From the proportioning of the specimens, section and connections of the reviewed literature it can be noticed that no standard method was applied to design the specimens.

3. Test plan

Fig. 1 shows Specimen 1 of the test program along with the load, support conditions and locations of lateral restraints. The specimen represented 80% of a typical building. The figure highlights key connections, namely, rigid beam-to-column connections, slip-critical, bolted beam splices and bracing connections. At the end of the brace, the circular tube section was welded to a cruciform steel section, which in turn was welded to a gusset plate stiffened by fin plates. Table 1 lists the member sections of the Specimens 1 to 4, ductility class per the US provisions [1] and Building Standard Law rank [5], and whether the specimens have a concrete slab or not.

Table 2 summarizes the expected strengths of the specimens, H , computed as the sum of the strength attributed to the moment-resisting frame, H_f , and the the braces H_b . per Fukuta et al [6].



As in Seki et al [3], the strength was evaluated at two stages, at the onset of brace buckling, and the state when an energy-dissipating mechanism is formed, represented by subscript 1 and 2, respectively. As done by Asada [7], the column panel and segments stiffened by gusset plates were assumed rigid for increased accuracy. The highlighted column on the table lists the ratio of the unbalance force over the plastic strength of the beam, which, if greater than 2.0 indicates the beam would likely yield before the braces develop their tensile yield strength, and if less than unity indicates the beams will not yield and the braces will develop their tensile yield strength.

Loaded will be applied from the West side following the loading protocol by AISC341 up to a story drift of 0.05 rad. The beams were braced against lateral deflection and torsion at three intermediate points. The unbraced length of beams fulfill the requirement for Special Concentrically Braced Frames per AISC [1]. The columns were restrained at the top and bottom for lateral motion and at the bottom for torsion.

4. Test results

Figure 2 shows the global response of Specimen 1 plotting the story shear force versus story drift ratio. Cyclic loading was applied starting from negative drift, which elongated the West brace and contracted the East

brace, followed by positive drift. During the first positive and negative excursions of ± 0.002 rad, the braces buckled out-of-plane in opposite directions. The specimen afterwards lost the initial stiffness but exhibited stable and symmetric behavior up to ± 0.02 rad. Noticeable strength degradation occurred beyond ± 0.03 rad in negative excursion only. Loading was continued to ± 0.05 rad, or more precisely, $+0.053$ rad to -0.043 rad and the test was terminated due to

Table 2. Strength of specimens

	H ₁	H _{2b} + H _{2f}	H ₂	$\frac{(N_y - 0.3N_{cr})\sin\alpha}{(4M_p/l)}$	$\frac{H_{2b}}{H_2}$
ID	[kN]	[kN]			
1	515	238 + 334	572	3.31	0.58
2	898	461 + 627	1088	1.48	0.58
3	578	461 + 531	992	1.36	0.54
4	527	301 + 448	749	2.07	0.60

reaching the capacity of the lateral bracing system.

Figure 3 shows the relationship between (a) brace elongation, (b) beam deflection (upwards positive), and (c) local beam rotation versus the story drift ratio. The braces deformed primarily in contraction. Both braces exhibited similar deformation until ± 0.02 rad. From ± 0.03 rad, the West brace did not restore its original length during the tension excursion, while the East brace did. The maximum contraction was 27 to 30

Fig. 1. Specimen sections and check of ductility

ID	Columns		Beam				Brace		Ductility	Slab
	Section		Upper		Lower		Section	kL/r		
	(BCR295)	Ductility	(SS400)	Ductility	(SS400)	Ductility	(STK400)			
1	□250x250x12	MD/FA	H300x150x6.5x9	MD/FA	H300x150x6.5x9	MD/FA	Ø101.6x5.7	98	HD/FA	No
2	□300x300x12	MD/FA	H400x200x8x13	MD/FA	H300x200x8x12	MD/FA	Ø139.8x4.5	66	MD/FA	No
3		MD/FA		MD/FA	H300x200x8x12	MD/FA	Ø101.6x5.7	94	HD/FA	No
4	□250x250x12	MD/FA	H300x150x6.5x9	MD/FA	H300x150x6.5x9	MD/FA	Ø101.6x5.7	98	HD/FA	Yes

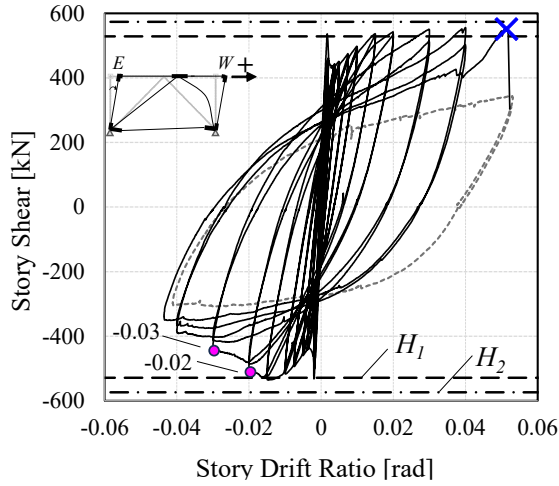


Fig 2. Global hysteresis of specimen 1

times the theoretical yield limit, Δy somewhat larger in the East brace. The beam deflected downward as much as 180 mm prior to the fracture of the East brace. Both beam ends rotated in one direction only (either positive or negative), and the measured rotation was twice the story drift ratio, indicating that the plastic mechanism involved the beam resisting the unbalanced force between the braces.

At the end of the test, the East brace was fractured, the West brace developed cupping deformation at the plastic hinge, and the beam intersecting the braces was severely distorted, especially at the West side of the midspan gusset plate connection. Fig. 4 presents visual documentation of the observed damage, Fig. 4b and 4c show different view angles of the upper beam intersected by braces; Distortion extended to the panel stiffened by gusset plate and stiffeners, but minimal distortion was observed at the East side of this beam. The four beam ends developed severe distortion; Despite the very large, experienced story drift, no cracks were noted at the typical CJP groove welds in any of the beams, and no distortion of the section was observed within the stiffened panels of the lower beams.

The bracing connections deformed out-of-plane in the gusset plates and thereby nicely accommodated to the buckling deformation of the braces. Cracks were developed in the gusset plates at the termination of fin plates and at the free edge connecting to the beam.

5. Discussion

The lateral strength at imminent brace buckling was +536 kN and -532 kN. The maximum lateral strength after the braces had buckled was +556 kN in positive excursion, recorded at -0.04 rad, and -536 kN in negative excursion, recorded at -0.015 rad. The

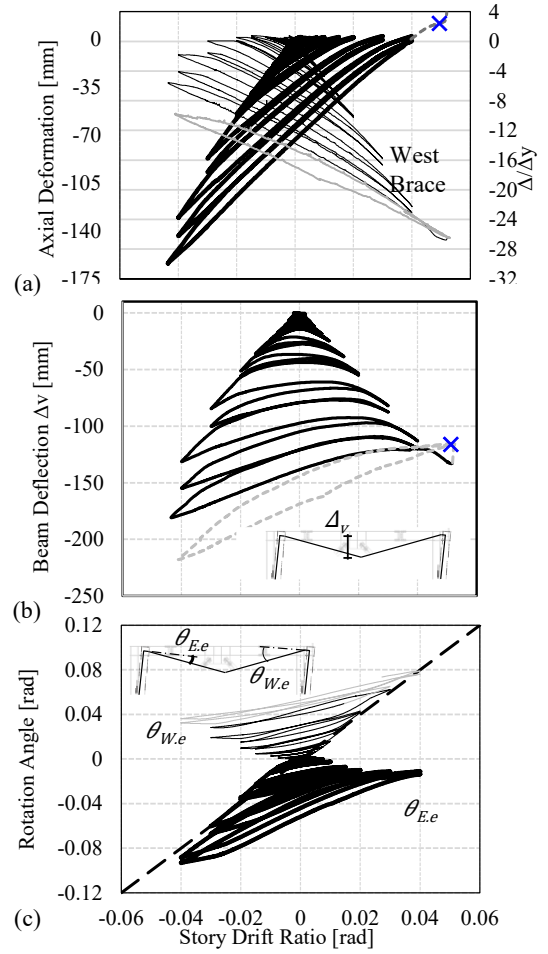


Fig 3. Measured results

corresponding design strengths, H_1 and H_2 listed in Table 2, are indicated in Figure 1. The strength at first brace buckling was close to H_1 , but the maximum strength after brace buckling was smaller than H_2 . Most notably, the specimen developed a little overstrength beyond the plastic strength.

Fig. 5 shows the hysteresis decomposed into the story shear force resisted by the frame and braces. The former was computed based on strain measured from the columns, and the latter was computed by subtracting the former from the applied load. The force resisted by the braces at first brace buckling was close to the estimate of H_{1b} . The force resisted by the frame and braces at the post buckling state, H_{2f} and H_{2b} , respectively, are shown in the figure. The measured strength of the frame and braces were close to the predicted strength values. The frame developed little overstrength beyond H_{2f} , while the strength of the tensile and compressive braces combined gradually decreased below H_{2b} with story drift ratio. The strength of the braces degraded more substantially in negative loading, which extended the West brace and contracted the East brace, than in positive loading. The strength

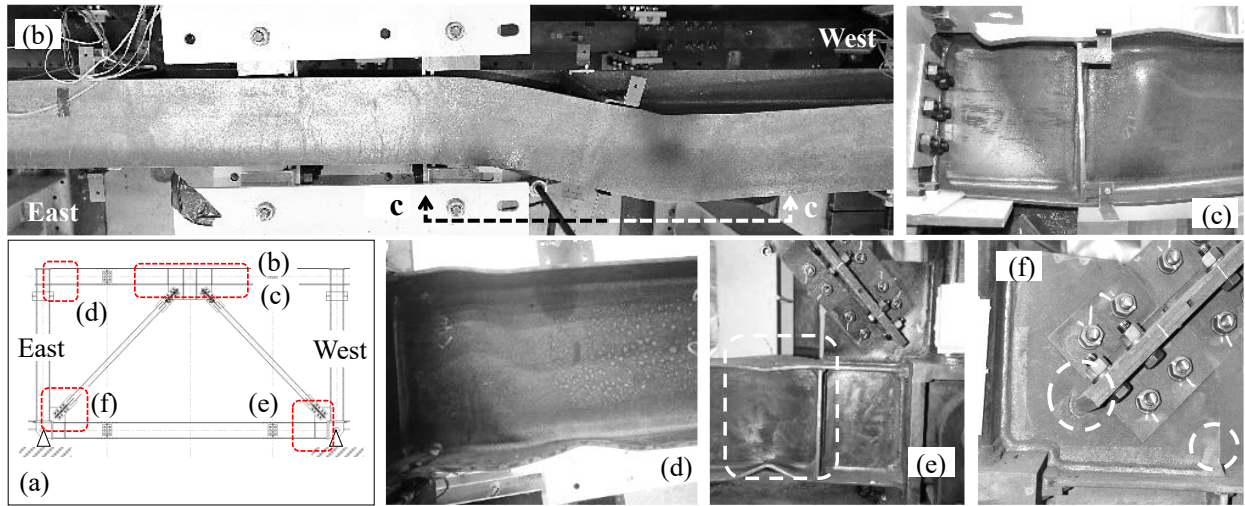


Fig. 4. Observed damage

degradation is attributed to the beam distortion shown in Figure 4b and 4c which prevented from stretching of the West brace.

6. Summary and conclusions

The first of four large specimen was conducted under cyclic loads.

The proportion of the beam with respect to the braces was not able to resist the unbalanced force resulting after the buckling. As a consequence, the midspan beam deformed substantially downwards and the braces behaved primarily in compression with little incursion in elongation.

The East brace fractured at 0.05rad and the west experienced large compressive deformation. The cruciform gusset plate bracing connection survived with cracks at the termination of the fin plane and crack initiation at the free edge of the gusset plate.

The provided lateral bracing system for the upper beam fulfilling the American provisions kept the beam in plan until 0.02rad. Beyond that level of story drift ratio large out-of-plane deformation happened. This study suggests that as the upper beam only qualifies a moderate ductile, fulfilling the spacing is not enough to

prevent put of plane deformation. Additionally, large-out-of plane deformation and beam deflection exceeded the capacity of the lateral bracing system, which led to the premature termination of the test.

Despite severe distortion was experienced at four beam ends and at the midspan of the upper beam only in the latter case the distortion extended to the stiffened panel and no CJP weld groove resulted damaged.

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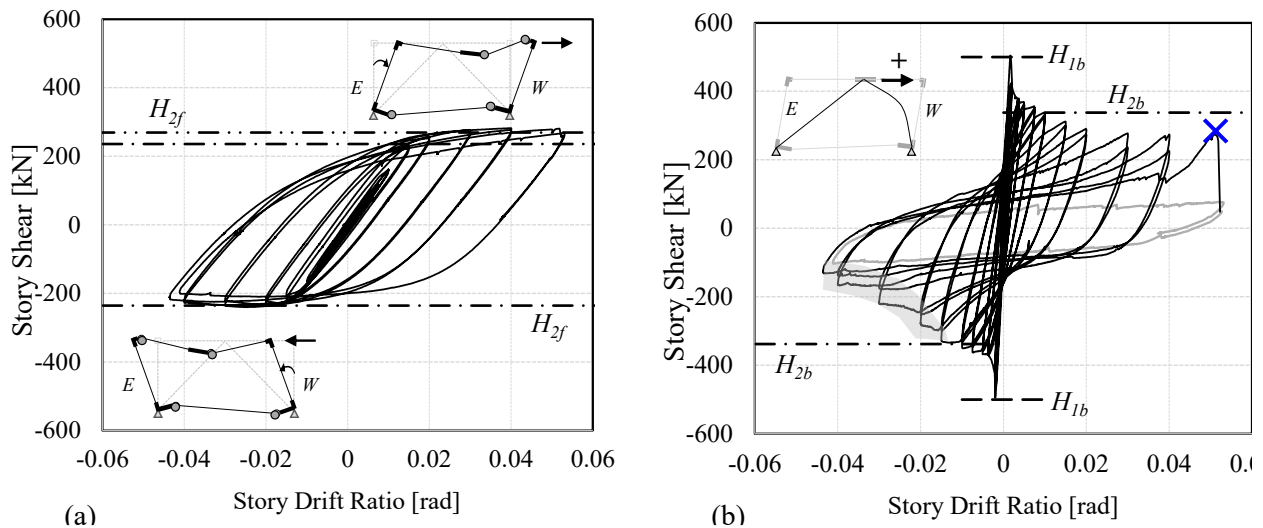


Fig. 5. Response of the decomposed portions