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Tests of chevron panel buckling-restrained braced steel frames

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ABSTRACT

Two specimens were tested to investigate the effects of constructional details, including panel buckling-restrained braces (panel BRBs), connections, strengthening measures, etc., on the hysteretic behavior of two-story chevron-shaped panel buckling-restrained braced frame (PBRBF). Each panel BRB is an unbonded steel plate brace encased in either an assembled steel panel or a steel-concrete composite panel. Tests revealed that obvious yielding of panel BRBs and framing members occurred after an average inter-story drift of approximately 1/375 and 1/75 respectively. Generally, both specimens from the capacity design exhibited stable hysteretic behavior within story drifts of 1/50, while large local bucking, low cyclic fatigue cracks and tension fracture of steel components, which result in eventual failure of the PBRBFs, occurred at the drifts far larger than 1/50. The PBRBF1 with moment-resisting beam-to-column connections has trilinear skeleton curves within story drifts of 1/30. The PBRBF2 with non-moment-resisting beam-to-column connections, along with the panel BRBs, has bilinear skeleton curves prior to failure of panel. Although local failure of a composite panel occurred at story drifts of 1/68 in the first test, the PBRBF2 still exhibited stable hysteretic behavior in the second test and finally failed at story drifts of 1/33. The constructional details are generally acceptable to ensure good performance of the PBRBFs. Furthermore, if 1/200 is considered as a permissible residual drift level, the residual drifts would challenge function of use when the PBRBFs underwent peak inter-story drifts larger than approximately 1/100.

1. Introduction

Compared with conventional steel braces, the buckling-restrained braces (BRBs) [1–4] would be a better choice to restrain buckling of braces in axial compression and to improve ductility and energy dissipation capacity of braces under seismic actions. These advantages facilitate the application of buckling-restrained braced frames (BRBFs) in recent years [1,2,4–6]. The panel buckling-restrained brace (panel BRB), usually consisting of one or two unbonded core steel plate braces and one encasing panel [1,2,4,7–11], is a typical form of buckling-restrained braces (BRBs) [1,2,4,12]. Besides, the encasing panels can act as partition walls in buildings, such as apartments, hotels, etc. [1,2,12]. For a panel BRB subjected to large cyclic loads, besides yielding of the core plate brace under axial tension, the core brace under axial compression can also yield if lateral restraint of the encasing panel to the core brace is adequate [9,10]. Since unbonded layers or air gaps are reserved between the brace and the panel (see Fig. 1), the axial strength of a panel BRB is intended to be totally provided by the core plate brace.

Constructional details of panel BRBs directly affect hysteretic behavior of steel frames with the panel BRBs (PBRBFs). Two tentative tests on steel frames with moment-resisting beam-to-column connections and with the reinforced concrete panel BRBs revealed that punching shear failure of the reinforced concrete panel deteriorates the ductility of both panel BRBs and PBRBFs [7,8]. Recently, to avoid punching shear failure of reinforced concrete panel BRBs [1,2,7–10], two novel type panel BRBs were proposed and tested by Ding [13,14]. One is a plate brace encased in a light-weight assembled steel panel [13]. The other is an unbonded plate brace encased in a steel-concrete composite panel [14]. Hysteretic behavior of the panel BRBs is stable prior to eventual tension fracture of encased braces due to low cyclic fatigue [13,14]. Compared with the reinforced concrete panel BRBs [7–10], punching shear failure of panel was avoided for either the assembled steel panel or the steel-concrete composite panel and ductility of the proposed panel BRBs was actually improved [13,14], revealing that the two type panel BRBs are available to the application of PBRBFs in future. Therefore, it is worthwhile to use them to improve the performance of PBRBFs and to investigate appropriate constructional details to ensure that the two type panel BRBs installed in steel frames can also have satisfied hysteretic behavior in the component tests [13,14].

Besides, exploring appropriate configurations of PBRBFs, especially beam-to-column and brace-to-frame connection details, is also needed

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to improve hysteretic behavior of the PBRBFs. As a type of seismic force resisting system, BRBFs can be usually constructed with either moment-resisting or non-moment-resisting beam-to-column connections and a BRB member connects to framing members by gusset plates [3,6,15–23]. Some researchers pointed out that gusset plates used to connect BRBs with beam-to-column connections are inclined to increase stiffness of both beam-to-column connections and frame (these gusset plate connections will transfer in-plane bending moments to the BRBs and therefore decease the effectiveness of the BRBs, such as cumulative inelastic axial deformation capacity). Certain undesirable performances, such as buckling of gusset plates, fracture of welds between the gusset plates and the framing members, out-of-plane buckling of BRBs, etc., would occur [15–23]. The study [17] further found that moment-resisting beam-to-column connections would be more likely to fail at smaller drift levels than non-moment-resisting connections. It indicated that non-moment-resisting beam-to-column connections can be used to improve the performance of BRBFs. In addition, compact connections between BRBs, which are intended to be used as axially loaded members, and framing members will decrease bending moments transferred to the BRBs and increase both length of yielding segments and low cyclic fatigue life of the BRBs.

In view of the above considerations, it is worthwhile to explore configurations to further improve working behavior of the PBRBFs. Therefore, hysteretic behavior of two PBRBF specimens with chevron-shaped panel buckling-restrained braces was tested and the objective is to examine effects of constructional details, such as connections, configurations of panel BRBs, reinforcing measures, etc., on hysteretic behavior of the PBRBF with either moment-resisting or non-moment-resisting beam-to-column connections. To realize the objective, some methods are as follows. (1) to examine the impacts of the configurations of the panel BRBs, both the assembled steel panel BRBs and the steel-concrete composite panel BRBs [13,14] were used in each PBRBF specimen; (2) in each specimen, each end of the encased brace was directly welded to the flange plate of a framing member to replace large gusset plate connections and to mitigate the impact of in-plane bending moments on the brace. Besides, to ensure stable behavior of panel BRBs, segments of framing members attached to encased braces were strengthened to prevent out-of-plane failure of the panel BRBs, which would be induced by inelastic deformations of framing members; (3) a capacity design was used to confine most inelastic behavior of the PBRBFs in the panel BRBs while preventing the framing members, especially the columns, from large inelastic deformations within the inter-story drift level of 1/50, and therefore the framing members and connections were designed and strengthened based on the adjusted strength of the panel BRBs; and (4) for each panel, the gaps between the panel and the encased brace, as well as those between the panel and the framing members, were determined based on the maximum inter-story drift level of 1/30 to further inspect the behavior of the PBRBFs.

2. Arrangements of tests

2.1. Specimens

2.1.1. Outline of specimens

Two 2-story planar specimens are steel braced frames which are labeled PBRBF1 and PBRBF2, shown in Figs. 1-3. All dimensions are given in millimeters. Framing members in the PBRBF1 were directly welded together to form the moment connections and those in the PBRBF2 were connected together by double angles to form non-moment connections (Figs. 2 and 3). For each steel brace, the lower end was directly welded to a steel column and the upper end was welded to a steel beam, ensuring that the brace was generally concentrically connected to framing members while reducing eccentricities. Moreover, steel-concrete composite panels and assembled steel panels were used in the first and the second story, respectively (Fig. 1(c) and (d)). Some reinforcements, such as vertical or horizontal steel plates (or stiffeners), were used near the brace-to-frame and beam-to-column connections in each PBRBF to avoid unexpected failure of these connections (Figs. 2 and 3). During testing, two bases of column in each PBRBF were fixed on a foundation beam by high-strength bolts (Fig. 4).

2.1.2. Design of specimens

The procedure including capacity design in the study [11] was carried out to confine the inelastic behavior mainly in panel BRBs while keeping the framing members, especially columns, in elastic in general within the inter-story drift of 1/50 for each PBRBF. Strength of framing members and connections was checked by taking into account the anticipated maximum axial strength [11] (also referred to as adjusted axial strength in the Seismic Provisions [6]) of panel BRBs to consider the overstrength of panel BRBs. Here, the adjusted axial strength is N_{cmax} (or N_{tmax}) for a panel BRB in axial compression (or tension). Based on the Provisions [6], there is $N_{\mathrm{cmax}} = \beta\omega N_y$, where $N_{\mathrm{y}} = A_c f_y$ is actual axial yield strength of core brace and A_{c} and f_{y} are actual cross section area of core brace and yield stress from coupon tensile tests, respectively. The compression strength adjustment factor (β) is used to reflect the compression overstrength due to frictional actions between panel and encased brace, etc. and the tension strength adjustment factor (ω) is used to reflect the effect of strain hardening at a specific story drift. When a panel BRB is in tension, there is $N_{\mathrm{tmax}} = \omega N_{\mathrm{y}}$. The values of the two parameters are depended on constructional details of panel BRBs and demands of lateral drift levels. As constructional details of panel BRBs in these tests are generally similar to those in the tests [13,14], the values of β and ω under a specific lateral drift level can be got based on test results on the panel BRBs [13,14], which were assigned 1.20 and 1.35, respectively, at the story drift of 1/50.

As mentioned above, the specimens were tested up to the maximum inter-story drift of 1/30. Therefore, further checking for the strength of framing members and connections, such as high-strength bolt connections between bases of column and the foundation beam and those between the south column and the actuator (Fig. 4(c)), etc., was conducted to ensure that the tests of the PBRBFs can be conducted up to the drift of 1/30. The values of β and ω were assigned 1.27 and 1.50, respectively, at the story drift of 1/30 based on the tests [13,14]. It should be noted that it does not guarantee that framing members can also avoid large inelastic behavior after the drift of 1/50, such as the drifts near 1/30, because it is nearly inevitable for framing members developing some inelastic behavior under these large drifts. Especially, some connections or members will be prone to low cyclic fatigue and even fracture, which would trigger large inelastic behavior development in other members under large reversal loading.

2.1.3. Configuration details of steel frames

For each planar PBRBF, framing members have hot rolled H-shaped sections (Figs. 2–4) and the H194 × 150 × 6 × 9 (all dimensions are in millimeters) was chosen as the cross section of beams in the initial design without out-of-plane issues. Further checking revealed that the beam at the second floor should be enlarged to guarantee that horizontal loads can be transferred from the actuator to chevron-shaped panel BRBs without potential out-of-plane instability of the beam. Besides, to simplify available sizes of cross sections for framing members, a checking showed that the cross section of beam at the second floor with H200 × 200 × 8 × 12 nearly has no effect on the initial design results. Therefore, cross sections of both the beam at the second floor and columns are H200 × 200 × 8 × 12 and those of the beam at the first floor are H194 × 150 × 6 × 9. Moreover, to ensure reliable ductile behavior of the members, width-to-thickness ratios for flange and web plates of framing members (see Table 1) in these 2-story structures were chosen to satisfy the requirements of seismic grade III according to the Code for Seismic Design of Buildings [24] for the braced steel frames (the site where these structures locate was assumed to have 8° seismic precautionary intensity). Regarding these cross sections, further