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Seismic performance of prefabricated steel beam-to-column connections

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ABSTRACT

Three types of prefabricated steel beam-to-column connection different from common welded unreinforced flange-bolted web (WUF-B) connections are examined in this paper. Full-scale specimens with specific joint configurations were prepared, so that the effects of joint detail on the failure mode, ultimate capacity and ductility could be identified. They were tested under cyclic loading to further investigate their seismic performance. Experimental results showed that the measured moment capacities of these connections at the face of a column flange reached 120% to 140% of the beam's full plastic moment. The maximum plastic rotations of all connection types were greater than 0.025 rad and the cumulative plastic rotations were 20 to 30 times the maximum plastic rotation. In this way, connection Types I and III were capable of accommodating a story drift angle of 0.04 rad. These results demonstrated excellent joint strength, and showed potential opportunities for connection Types I and III to be used in special moment frames (SMFs) in AISC Seismic Provisions, and for connection Type II to be used in intermediate moment frames (IMFs). Finite-element analysis was performed and showed good agreement with the experimental results. It was found that rigid end-plate assumption is not appropriate for connection Type I, and higher likelihood of fracture was detected in the cover plates than in beam flanges for connection Types II and III.

1. Introduction

Steel moment frames are currently widely used in multi-story buildings as lateral force resisting systems due to their superior ductility and energy dissipation capacity. The use of steel moment frames for seismic design dates back to the 1960s to the 1970s when welding began to be adopted. A few buildings at that time were constructed with unreinforced flange-welded web (WUF-W) connections. However, the practice quickly evolved to unreinforced flange-bolted web (WUF-B) connections because of greater economy and also because of the satisfactory energy dissipation capacity of WUF-B connections under cyclic loading, as evidenced by research [1,2]. Generally, seismic provisions require that energy dissipation should take place mainly at the beam ends, and eventually at the base section of the columns. As a result, these connections should possess sufficient strength and rotational stiffness to permit the development of yielding and strain hardening in the zones that dissipate energy (such as the beam end, panel zone, and base section of column) before their final fracture. In the 1994 Northridge and the 1995 Kobe earthquakes, however, unexpected brittle failures were observed between the weld metal and base metal at the toe of the weld access holes within steel moment frames [3–5]. After those earthquakes, experimental and analytical studies were conducted to investigate the causes of such failures and to provide reliable and economical solutions to avoid them. The efforts have led to improvements of ANSI/AISC 341 [6] and development of new seismic design criteria for beam-to-column connections, such as FEMA-355D [7].

Improvements in traditional WUF-B connections were achieved in two main ways. One was to reduce welding defects and the associated stress concentration in the vicinity of weld access holes through improved welding techniques or modification of local details of connections [8–10]. The other was to move the location of the plastic hinge away from column-to-beam flange groove welds by allowing yielding of the corresponding beam. This concept was achieved by reinforcing the beam flanges as in haunched, cover-plate and flange-plate connections [11], or by weakening the beam flanges as in reduced-beam-section (RBS) connections [12]. Research outcomes have shown that these methods are effective in providing satisfactory performance of connections in steel moment frames and that brittle failure of connections may be avoided.

Connections developed previously were often made on-site. Investigated in this paper are three types of connection extracted from the prototype frame as shown three-dimensionally in Fig. 1(a); subassemblies containing those connections are prefabricated in the shop and then assembled on-site into the frame. As illustrated in Fig. 1(b), the connection Type I is similar to the traditional WUF-W connection in that the beam flanges (BFX) are connected to the column flange (CF) through complete joint penetration welds. The differences are that 1) the beam web (BWX) is stiffened with a shear plate (SP) and angle (AG) on each side of connection Type I whereas only one side of the WUF-W is

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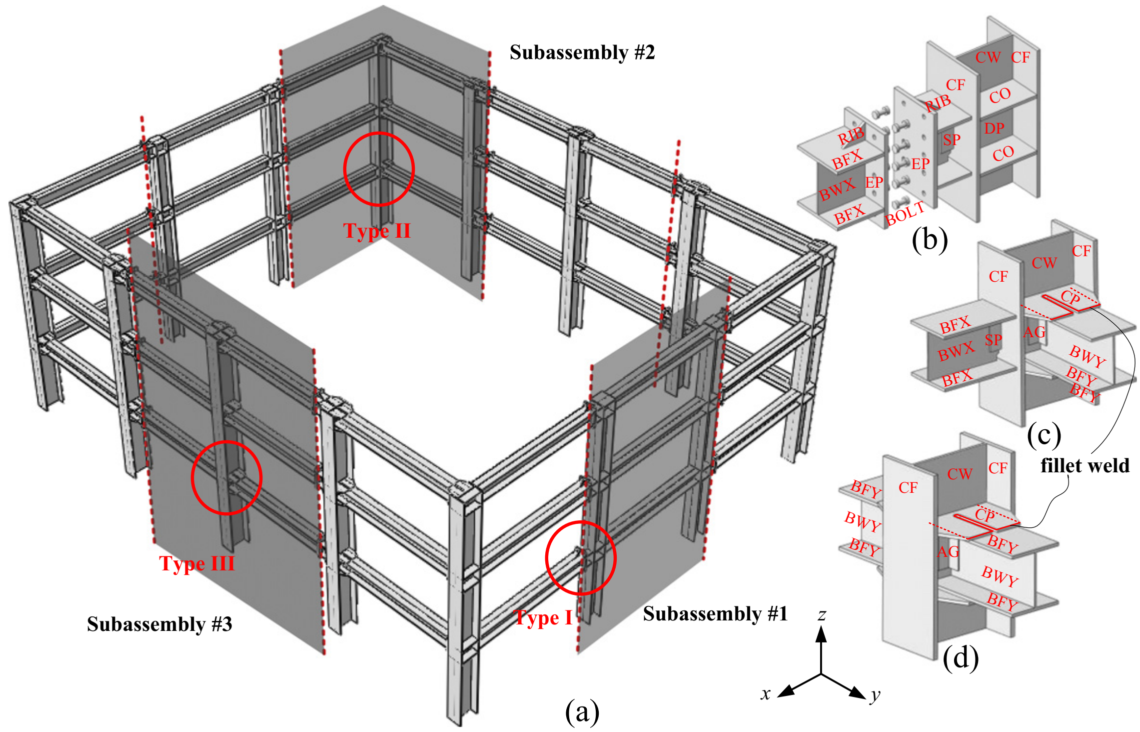

Fig. 1. Configuration and components of proposed connections: (a) prototype frame; (b) connection Type I; (c) connection Type II; and (d) connection Type III.

connected to the shear plate; and 2) a bolted end-plate connection (EP, BOLT, RIB) lies near the face of the column flange in connection Type I. This end-plate connection is used for assembly. Connection Type II is shown in Fig. 1(c). The strong-axis (x axis) configuration is the same as in Type I but without the end-plate connection, and in the weak-axis (y axis) the beam flanges (BFY) are connected to novel cover plates (CP) using fillet welds along the long slot, nose and inner sides of cover plates (see Fig. 1(c)). The connection of the beam web (BWY) to the column web (CW) is stiffened with angles (AG) on both sides. This is quite different from the traditional weak-axis moment connections in which beam flanges are connected to continuity plates with complete joint penetration welds and the beam web is connected to the shear plate with fillet welds [12]. Unlike connection Type II, connection Type III is cruciform, as shown in Fig. 1(d). It consists of two beams attached to the column with the same weak-axis configuration as in connection Type II. These types of joint configuration differ from commonly-used connections prequalified in ANSI/AISC 358 [13], and allow prefabrication and on-site installation. Their mechanical performance has not been well studied previously.

In order to investigate their seismic behavior and failure mechanisms, three full-scale beam-to-column specimens were fabricated using corresponding connection types, and examined under cyclic load. The load carrying capacity and rotational capacity, as well as ductility and energy dissipation capacity are discussed in detail. Furthermore, finite-element analysis was conducted for comparison with the experimental results and to study the local stress distribution and likelihood of fracture of several key components in the connections. This work aims to provide valuable experimental evidence and to develop reliable modeling approaches for the evaluation and design of these specific types of prefabricated beam-to-column connections for engineering applications.

2. Experimental program

2.1. Materials

Hot-rolled sections and several plates of low carbon steel Q235B were used to fabricate the specimens. Tensile tests were carried out using three identical coupon specimens for each kind of section or thickness of the plates to obtain averaged results. The results are summarized in Table 1, where ε_y is the yield strain, ε_{st} is the strain at the end of yielding plateau, and f_u and ε_u are the ultimate strength and strain respectively. Note that measured yield strength (f_y) exceeds the nominal yield strength of 235 MPa.

2.2. Specimens

Three specimens were prepared using the corresponding connection types introduced above as shown in Fig. 2. The beam in Specimen I was connected to the prefabricated column-tree joint using ten M30 bolts of class 10.9 (see Fig. 2(a)), while Specimens II and III were both prefabricated with beams and column as a whole (see Fig. 2(b) and (c)). The beam components used in all specimens were hot-rolled sections of H400 × 400 × 13 × 21 and the column component was a welded section of H500 × 500 × 16 × 25 in Specimen I and H700 × 500 × 20 × 35 in the other two specimens. The column height was chosen as 2430 mm for all specimens, and the resulting distance between the top and bottom boundary constraints was 2210 mm. The length between the loading point at the end of the beam and the column axis was chosen as 2650 mm for Specimen I, 2750 mm for the strong-axis

Table 1
Results of tensile coupon test.

Material		f_y/MPa	ε_y/%	ε_{st}/%	f_u/MPa	ε_u/%
Hot-rolled section	H400 × 400 × 13 × 21	292.3	0.147	2.013	449.0	20.173
	L100 × 10	306.4	0.151	1.621	464.7	14.798
Plate in welded section	12 mm thick	305.4	0.151	1.895	453.4	16.822
	16 mm thick	289.4	0.142	1.895	450.4	19.270
	20 mm thick	273.4	0.134	1.449	439.2	19.103
	25 mm thick	244.8	0.117	1.499	440.4	19.413
	30 mm thick	269.9	0.120	1.245	444.1	18.656
	35 mm thick	349.7	0.168	0.534	541.1	12.727

Note: H400 × 400 × 13 × 21 represents H-shaped section with flange width and sectional height equal to 400 mm, web and flange thickness equal to 13 mm and 21 mm respectively; L100 × 10 represents equal-leg angle section with leg length and thickness equal to 100 mm and 10 mm respectively.