Experimental and Numerical Analyses of Blind Bolted Moment-resisting Composite Joints to CFST Columns with RC Slab

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Abstract

In this paper, experimental and numerical analyses were conducted to investigate the structural properties and failure modes of the blind bolted flush or extended end plate joints between concrete-filled steel tubular (CFST) columns and composite beams. The experimental program and the main results of the tests were elaborated. Four full-scale subassemblages beam-to-column exterior joints to hot-rolled and cold-formed CFST columns were tested. The failure modes and moment-rotation relation curves of the joints were evaluated. The test results showed that all specimens exhibited favorable strength and stiffness, and larger deformation capacities. Meanwhile, finite element analysis (FEA) modeling of the specimens was built and analyzed. The results obtained from the FEA modeling were satisfactory agreement with the experimental tests results. Then, an extensive parametric study was conducted to investigate the influence of various parameters on the moment capacity and rotation stiffness of composite joints under positive or negative moment. The experimental studies and numerical analyses can improve the design of the typed joints to be used in moment-resisting composite frames.

Keywords: concrete-filled steel tube (CFST), composite joint, finite element analysis (FEA) model, end plate joint, blind bolt

1. Introduction

The CFST composite frame structure is composed of CFST column and composite beam. It has been used in the multi-story and high-rise buildings owing to its smaller component sectional and lighter weight. It also exhibits excellent static and earthquake-resistant properties, such as high strength and stiffness, better ductility and larger energy absorption capacity. The beam-to-column connection is one of the complicated components of a CFST frame structure, which plays an important role in the seismic resistance and construction procedure. The traditional joint to CFST columns is complicated and it involves welding, such as the use of internal (Elremaily et al., 2001) and external diaphragm plates (Kang et al., 2001), or embedded steel bars (Schneider et al., 1998) etc. The most common beam-to-column connection method used in practice is welding, due to the ability to use different types of connections and resist different types of loads. However, the use of welded connections has not always been convenient in construction practice. Considerable work in erection is required while extension welding and high tolerances required in detailing at the meantime. Concerns are also raised in relation to the exorbitant costs involved with some of these methods.

The blind bolted end plate connections have been suggested by contemporary scholars to improve the above shortcomings of welded connections. Murray and Meng (1995) proposed the extended end plate as the alternative design to eliminate beam flange-to-column welded connection. The blind bolt overcomes the inconvenience of welding and allows the bolt installation from one side. This type of connection is favored since it involves shop welding and on-site bolting instead of site welding. Murray and Borgsmlleer (1995) provide a design method for the multiple row extended 1/3 unstiffened moment end-plate connection.

Due to those advantages, some researches were conducted on the static behavior of HSS or CFST column connections with blind fasteners, such as France et al. (1999), Lee et al. (2011), Yao et al. (2014), and Wang et al. (2016). Wang et al. (2009, 2012 and 2012) carried out a series of static tests on the blind bolted end plate joints, and summarized the structural performance and failure modes of the blind bolted connection consisting of H-shaped steel beams and CFST columns. In addition, some studies conducted by Elghazoui et al. (2009), Wang et al.
(2009), and Tizani et al. (2013) were focused on the hysteretic behavior of the blind bolted connections to HSS and CFST columns. Moreover, several numerical analyses were conducted, such as Hu et al. (2003), Elllobody et al. (2006). But most of these studies were focused on the full-rigid joints. It needs to pay more attention to the research of semi-rigid joints.

For another, the reinforced concrete (RC) slab is poured on the steel beam flange in the reality, and then the composite beam is formed by setting up a sufficient number of shear connectors between steel beam and RC slab. Research results revealed that the RC slabs in the composite joints could prevent the lateral torsion buckling and raise the moment capacity of the steel beams (Xiao et al., 1994). Some researchers conducted on the seismic performance of CFST column to steel beam joint with RC slab, such as Han and Li (2010), Cheng (2007), Leon RT et al. (1998), Gracia et al. (2010), Mirza and Uy (2011). It highlights that an extensive further research need to be focused on seismic performance of the semi-rigid blind bolted end plate joints to CFST column and composite beam with RC slab.

In this paper, experimental tests of four specimens were carried out and the corresponding FEA models were built. The tested and predicted results of the blind bolted end plate composite joints to CFST columns were verified. The mechanical properties and failure mode of semi-rigid joints were conducted and compared. A large number of parametric analyses were carried out and the major parameters influencing of the ultimate moment capacity and the initial stiffness were summarized under positive or negative moment. The research showed that the blind bolted end plate joints between CFST column and composite beam with RC slab have the advantage of enhancing the ductility and energy dissipation. And the analysis results indicate that the type of joints is worth to promote in the practical engineering.

2. Experimental Program

2.1. Test specimens description

Four full-scale blind bolted end plate joints between square CFST columns and steel beams with RC slabs were designed and made. The beam-to-column joints are divided into two forms, flush end plate for specimen FSD1 and TFD1, extended end plate for specimen FSD2 and TED1. All specimens were subjected to a cyclic loading to simulate seismic loading conditions. The details of the end plate connection specimens to CFST columns are illustrated in Fig. 1. The geometry dimensions of each specimen are illustrated in Table 1. The cross section of columns for specimen FSD1 and FSD2 is 200×200×10 mm while for specimen TFD1 and TED1, the cross section is 200×200×10 mm. The heights all columns were designed to 3400 mm. The steel tubes for specimen FSD1 and FSD2 were made by hot-rolled steel sheet with thickness of 10 mm. For specimen TFD1 and TED1, the steel tubes were made by four welded lipped angle with 3 mm thickness cold-formed thin steel sheet. The subassemblages beam-to-column joints (Fig. 2) was chosen as the typical element from conventional frame structure. The storey height of the test frame was set as 3400 mm. The inflection points of columns of the frame structure correspond to the top and bottom of column. To match the test device determined by the laboratory conditions, the beam length in specimens was set as 1700 mm.

Four joints were made up of H-shape steel beam with a cross-section of H300×150×6×10 mm and length of 1700 mm. For specimen FSD1 and FSD2, a piece of 18 mm thick end plate was welded to the beam end by fillet welds and connected to the column by blind bolts. The end plate thickness was changed to 12 mm for specimen TFD1 and TED1. The width and thickness of cast-in-situ slabs were 1200 and 120 mm, respectively. The RC slab and steel beam were connected by a single row of steel shear studs with the diameter of 19 mm and height of 90 mm. The shear studs were welded to the top flange of the steel beam in order to meet the full shear connection and negative bending shear requirements. Numbers and layout of shear studs were designed under the full shear connection criteria recommended in GB50010-2010 (2010).

The specimens were fabricated and erected at laboratory. Reinforcement bars were prepared and assembled manually in the laboratory after the erection completed. Wood molds were applied for slab concrete works by after. Then normal concrete was poured into the formworks after the columns were filled with self-consolidating concrete (SCC) mix.

Figure 3 illustrated the blind bolts with extensions into concrete core which were used to fasten the end plate to square steel tube. The high strength blind bolt is Grade 10.9 M20. The exterior diameter of the bolt is 20 mm and the ultimate strength is 1000 MPa. The ratio of yielding strength to ultimate strength of the bolts is set as 0.9. All the blind bolts were tightened to a torque of 442 N.m to stand by specification GB50017-2003 (2003).

2.2. Material properties

Three tensile coupons were cut from the two different steel tubes and sheets. All the coupons were tested to find out main properties of steel such as the yield stress ($f_y$), ultimate stress ($f_u$), young’s modules ($E$) and elongation at fracture ($\delta$). The results of the material tests were summarized in Table 2. Besides, the nominal yield stress and ultimate stress of the Grade 10.9 M20 blind bolts were 942 MPa and 1048 MPa, respectively.

The self-consolidating concrete (SCC) was poured into CFST columns while the normal commercial concrete was applied for slabs. Each column and slab in the test was filled with SCC or normal commercial concrete from the same batch. Two sizes of concrete cubes were manufactured for material properties test during the process of concrete