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Behaviour of bolted end-plate connections to concrete-filled steel columns

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ABSTRACT

This research aims to investigate the cyclic behaviour of composite joints consisting of concrete-filled steel tubular (CFST) columns, steel beams, and through-bolt connections. A total of 10 specimens, including 5 specimens with square CFST columns and 5 specimens with circular CFST columns, were tested under lateral cyclic loading with horizontal displacements imposed at the top of the column. The main experimental parameters were the cross-sectional type of the column, the axial load level in the column and the cross-sectional configuration of the beam. The experimental results are analysed in this paper to evaluate the performance of the through-bolt connections. Based on numerical analysis, the performance of the bolted joint is further compared with that of the counterpart with external diaphragm. In general, the initial stiffness of the bolted end-plate CFST connections investigated is greater than $8EI_b/L_b$, so a fixed connection can be assumed in modelling non-sway frames. However, the connection rotation in unbraced frames may need to be considered to minimise the modelling error since the initial connection stiffness is usually smaller than $25EI_b/L_b$.

1. Introduction

Concrete-filled steel tubular (CFST) columns have excellent load-carrying capacity under various loading conditions. Besides, the constructability of the CFST columns is also prominent, for a lot of formwork and construction time can be saved [1–2]. Meanwhile, suitable protections against fire and corrosion may be required for the CFST columns since the steel tubes are exposed. In buildings, CFST columns are often connected to steel/composite beams, and various connection details have been invented and studied [3–7]. The most widely used beam-column connections are diaphragm connections and through-beam connections [8]. These connection details have favorable earthquake-resistant properties and result in “rigid” connections, i.e., the connections can transfer axial load, bending moment and shear force while the relative rotation between the beam and column is negligible [9–11]. However, most rigid connections require considerable in-situ welding or embedded components to achieve sufficient moment capacity. Moreover, the welded beam-to-column connections were found to have brittle fractures in structures during the post-earthquake survey [10].

Some improved connections, such as the connection using high-strength bolts, have been used in CFST structures as alternative solutions. The bolted connection with end-plates avoids the potential failure associated with weld fracture, and requires only shop welding and on-site bolting works. It is also a good choice for prefabricated buildings when rapid construction is required. Some research work has been conducted on CFST column to steel beam joints using bolted end-plates (or similar connection details). Beutel et al. [12] performed six cyclic tests on steel beam to CFST column joints, one of which had through-bars welded to the flange of the beam and fixed on the opposite side of the column. It was found that the bars were effective to transfer both compressive and tensile loads into the column, and it was adequate to classify the connection as a “rigid” one according to the stiffness and strength. Ricles et al. [13] tested 10 full-scale moment resisting connections connecting wide flange beams to CFST columns, where two of the specimens had split-tee moment connection details. The split-tee connections were used to activate a diagonal concrete compression strut within the panel zone under the action of moment, while the weld of the web was omitted. The test results showed that this kind of connection achieved good deformation ability, while the inelastic storey drift exceeded the expected maximum inelastic drift in the design basis earthquake. Wu et al. [14] established a mechanical model to predict the shear behaviour of connections, and conducted cyclic tests on bidirectional bolted beam-to-column joints to CFST columns. The experimental results showed that the bolted CFST joints had excellent earthquake resistance. Wang et al. [15] investigated flush end-plate CFST connections using blind bolts. It was shown that all tested specimens displayed large rotation ductility. The connections

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could be classified as full strength semi-rigid connections based on the Eurocode 3 classification method [16]. Li et al. [17] conducted cyclic tests on bolted extended end-plate CFST connections, where high-strength bolts were used and transverse ribs were welded to provide a plane for the bolts. The effects of reduced beam section (RBS) and the presence of the reinforced concrete (RC) slab were investigated. The test results showed that the connections exhibited good ductility and energy-dissipation capability. The RC slab could contribute to the capacities under both sagging and hogging moments. Sheet et al. [18] tested four CFST composite connections, among which two specimens were designed with bolted extended end-plates. For the column with circular cross section, the threaded rods passed through the section in an "X" shape. Ataei et al. [19] tested beam-to-column deconstructable composite connections with high strength steel S690 flush end plates. The results showed that the rod in a pass-through design worked effectively, and the end-plate connections to both circular and rectangular columns showed similar performance.

Although CFST joints with end-plates and bolts have been investigated as described in the above literature review, some issues still need to be further explored. For instance, there is still limited information on the behaviour of composite joints under various column axial load levels, which may affect the joint behaviour. Meanwhile, numerical models should be established to analyse the joint behaviour, for limited investigation has been carried out on the full-range analysis of this type of joint. The possible failure modes of the joints should be clarified and cataloged. The influences of various design parameters should also be investigated, such as the thickness of the end-plate and the pretension of the bolts. Moreover, the classification of the joints using through bolts should be analysed in order to provide useful information for the design.

This paper presents an experimental investigation regarding the steel beam to CFST column joints using high-strength through-column bolts and extended end-plates. Ten interior joint specimens were designed and the parameters included the column type, the axial load level in the column and the beam configuration. The experimental results are analysed to evaluate the performance of the through-bolt connections. A finite element (FE) model is established and verified to fully study the influences of different design parameters on the joint performance. A comparison with the commonly used diaphragm connections is made in terms of the moment versus rotation curve. The main purposes of this investigation are:

1) to provide test data for the bolted end-plate CFST joints;
2) to propose a verified FE model which can reflect the performance of the composite joints; and.
3) to understand the influences of different design parameters on the bolted end-plate CFST joints.

2. Experimental investigation

2.1. Test program

A total of 10 cruciform joint specimens were designed, and the specimens were approximately one-third scale of the joints in a typical 10-storey building. All specimens were designed following a strong column-weak beam philosophy. The main parameters chosen were as follows.

- Column type: The cross-section of the columns was either circular or square. The overall diameter (D) of the circular columns and the overall width (B) of the square columns had a same value of 140 mm.
- Axial load level (n): Three axial load levels of 0.05, 0.3 and 0.6 were chosen for the columns, where the definition of the axial load level is expressed as:

$$n = \frac{N}{N_u} \tag{1}$$

in which N is the axial compressive load applied to the column, and N_u is the ultimate load-carrying capacity of the CFST column predicted by Eurocode 4 [20].

- Beam cross-sectional configuration: Different beam sections were obtained by varying the depth of the steel beam. The beam to column stiffness ratio and the beam to column strength ratio were changed accordingly. The beam to column stiffness ratio (η_{stiff}) is calculated as follows:

$$\eta_{stiff} = \frac{(EI)_b H}{(EI)_c L} \tag{2}$$

where $(EI)_b$ and $(EI)_c$ are the flexural stiffness of the beam and column, respectively. The stiffness $(EI)_b$ and $(EI)_c$ are estimated in accordance with Eurocode 3 [21] and Eurocode 4 [20], respectively. H and L are the height of the column and the span of the beam, respectively.

The beam to column strength ratio ($\eta_{strength}$) is expressed as follows:

$$\eta_{strength} = \frac{\sum M_b}{\sum M_c} \tag{3}$$

where ΣM_b and ΣM_c are the summations of the flexural capacities of the beam and column, respectively. Once again, ΣM_b and ΣM_c are estimated in accordance with Eurocode 3 [21] and Eurocode 4 [20], respectively.

Fig. 1(a) depicts the specimen configuration. The cross sections of the circular and square columns were ○-140 × 3 mm and □-140 × 3 mm, respectively. Three types of beams were used in the joint specimens and the dimensions are presented in Table 1, where h, b_f, t_w, and t_f are the overall height, overall width, web thickness and flange thickness of the I-beam, respectively. Curved and flat extended end-plates with a thickness of 10 mm were used for joints with circular and square CFST columns, respectively. The beam ends were welded to the end-plates, and circular holes with a diameter of 13.5 mm were drilled in order to connect the beams to the columns using class 10.9 high strength bolts. The diameter of the bolts was 12 mm. Custom-made curved washers were used for joints with circular CFST columns to install the high-strength bolts. Fig. 1(b) shows the configurations of the end-plates and the curved washer. It should be noted that the end-plates and bolts were tentatively designed in accordance with Eurocode 3 [16] for bolted end-plate connections. In the design, the end-plates were expected to reach 85–90% of their design resistance to bending, while the maximum tensile force in a bolt generated by the applied load could reach about 60% of the applied bolt pretension force (50 kN). Therefore, the design of the end-plates and bolts was relatively safe.

Detailed information of the joint specimens is summarised in Table 1, where the measured material properties were used in the calculation of n, η_{stiff} and $\eta_{strength}$. Since the joint design follows the "strong-column-weak-beam" criterion, $\eta_{strength}$-value of a joint ranges from 0.297 to 0.671, denoting that the beam is much weaker than the column. Furthermore, the ultimate shear strengths of the joints with circular and square CFST columns are calculated according to [10,14], respectively. It is found that the concrete contributes approximately 49% to the ultimate shear strength of joints with circular columns, while the corresponding contribution is about 57% for the joints with square columns. In the design, the ultimate shear strength is 1.5–1.9 times the maximum shear force within the panel zone for the joints with circular columns. Similarly, for the joints with square columns, the ultimate shear strength is 1.8–2.6 times the maximum shear force within the panel zone. Clearly, no shear failure of the panel zone would be expected to occur in any joint specimen and the expected failure mode is flexural failure of the beam.

The beams and square tubes in the experiment were manufactured from steel sheet, whereas commercially available cold-formed tubes