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Experimental study on blind bolted connections to concrete-filled stainless steel columns

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A B S T R A C T

This paper studies the performance of blind bolted connections to concrete-filled stainless steel tubular columns. An experimental investigation was conducted on seven full-scale joints to study the effects of the following parameters: (a) presence of the floor slab or not; (b) with or without binding bars in the connection region; (c) type of steel used for the column (stainless or carbon steel); and (d) loading type (monotonic or cyclic). The experimental results demonstrated the good performance of the blind bolts in this type of connection. No bolt shank fracture was observed in any of the test specimens. The presence of the binding bars slightly enhanced the performance of the blind bolted connections, whereas the presence of composite slabs significantly affected the failure mode, initial stiffness, flexural resistance, and rotation capacity of the joints. Owing to the use of flush end plates, the joint without floor slab can be classified as nominally pinned joint according to EC3. But in the presence of the floor slab, the joint nearly reached its full strength and the stiffness was also significantly increased close to the limit for rigid sway frames. In addition, the test results indicated that the material type of the steel tube had no obvious influence on the joint performance. In contrast, the cyclic loading led to slightly decreased joint strength, but had more obvious detrimental influence on the joint stiffness.

1. Introduction

In the last few decades, concrete-filled steel tubular (CFST) columns have been widely used in engineering practice because of their excellent structural performance. Meanwhile, the understanding of structural performance of stainless steel has also significantly advanced [1]. Therefore, there is an increasing interest in investigating concrete-filled stainless steel tubular (CFSST) columns, aiming to further increase the performance of conventional carbon steel CFST columns to achieve improved corrosion resistance, aesthetic appearance, ductility, fire resistance, and impact resistance [2–6].

Beam-column connections are critical elements for transferring floor and beam loads to columns and ensuring structural safety. In the past, many different types of connections have been developed to connect steel beams to conventional CFST columns, from simple fin-plate connections to rigid connections with internal or external diaphragms [7]. In regions of high seismicity, such as China and Japan, rigid connections are widely used since they are effective in transferring bending moments between beams and columns. However, rigid connections are expensive to make due to the inclusion of many stiffening elements and the need of heavy welding [8,9].

To avoid the need of in situ welding, bolted connections are preferred over welded connections in many countries, such as Australia and the UK [10,11]. Due to the difficulty of gaining access to the inside of hollow sections, through-plate and fin-plate connections have been developed accordingly. In these types of connections, through-plates or fin-plates need to be welded in the workshop to the steel tube, to which the supported beam web is bolted on site [12]. These types of connections, however, are normally classified as simple pinned connections [7], and the connected beams are not assumed to be able to transfer bending moments through the connection, resulting in loss of structural efficiency of the beams. In return, this could lead to increased construction costs and floor-to-floor heights, as well as reduced structural robustness [9]. Obviously, there is a need to develop new bolted connections to CFST columns for transferring bending moments.

In recent years, various blind fasteners, such as the Lindapter Hollo-Bolt and the Ajax ONESIDE bolt, have been developed, allowing the installation from the outside of the tube. The availability of these novel blind bolts on market facilitates the development of blind bolted connections to CFST columns, and some studies have been conducted accordingly [9,13–21]. These studies confirm that properly designed blind bolted connections can normally be considered as semi-rigid connections with relatively high initial stiffness, flexural resistance, and ductility.

Since CFSST columns made from stainless steel are still relatively new, little attention has been paid to develop reliable beam-column

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connections for the application of CFSST columns. Considering the fact that stainless steel is more expensive than carbon steel, it is logical to utilise stainless steel to enhance the durability of critical elements in a structure, such as the columns. For economical reasons, the authors proposed to use carbon steel beams to connect with CFSST columns. To develop this type of hybrid stainless-carbon steel joint, the adoption of blind bolting technology can eliminate the dissimilar welds between stainless steel and carbon steel, which have increased potential risk of cracking and brittle fracture [22].

This paper investigates the feasibility of connecting carbon steel beams to CFSST columns using blind bolts. Tests were conducted on a total of seven full-scale joints under either monotonic or cyclic loading. Effects of various parameters on the joint failure mode, initial stiffness, flexural resistance, and rotation capacity are discussed.

2. Experimental program

2.1. Specimen preparation

The seven cruciform joints were designed according to Eurocode 3 [23] to study the effects of the following parameters: (a) presence of the floor slab or not; (b) with or without binding bars in the connection region; (c) type of steel used for the column (stainless or carbon steel); and (d) loading type (monotonic or cyclic). The investigated variables are summarised in Table 1. Square columns were used in four of the joint specimens (SB1-0 to SB1-3), whereas circular columns were adopted for the rest three specimens (CB2-1 to CB2-3). No slab was provided for the specimen SB1-0, and all other specimens had steel-concrete composite slabs, as shown in Fig. 1. Due to the pull-out force from the blind bolts, it has been reported in [16,17] that the corner of the square steel tube might crack in the connection region and the tube wall could develop severe local outward deformation. In contrast, only slight deformation of the tube wall was observed in circular CFST columns. To improve the joint performance, binding bars were used in three joint specimens with square columns, i.e., SB1-0, SB1-2 and SB1-3, to tie the opposite surfaces of the steel tube together, as shown in Fig. 2. A total of eight binding bars with a diameter of 20 mm were provided for each of these specimens in the connection region. It should be noted that the location of these binding bars may affect their effectiveness to transfer the tensile force. In this paper, they were tentatively installed near the top flange of the steel beam. Further research is required to optimise the position of the binding bars. No binding bars were used in the reference specimen SB1-1 and other specimens with circular columns. To investigate the influence of material type of the column, conventional carbon steel CFST column was used in the specimen CB2-3, whereas CFSST columns were used in the rest of the specimens. Meanwhile, carbon steel was used to fabricate all other steel components, such as the steel beams, end plates and profiled steel sheeting. Most joint specimens were subjected to monotonic loading condition, and cyclic loads were applied to the tips of the two beam segments of specimens SB1-3 and CB2-2.

The geometric details of the six joint specimens with composite slabs are shown in Fig. 1, and those of the reference specimen SB1-0 are similar except for the absence of the composite slab. All steel tubes had a width or diameter (D) of 360 mm and a thickness (t) of 6 mm. The total height of a CFST column with two 20 mm thick end plates was 2200 mm. A primary beam and a secondary beam were provided for each joint specimen. Universal beam sections 310UB40.4 with cross-sectional dimensions of 304 × 165 × 6.1 × 10.2 mm were adopted for both types of beams. The total length of the primary beam was 3500 mm, whereas that of the secondary beam was 900 mm. Flush end plates with a thickness of 10 mm were provided for these beams. Rectangular plane end plates as shown in Fig. 2(a) and curved end plates as shown in Fig. 3(a) were used for square columns and circular columns, respectively.

The top view of the 120 mm thick composite slab is illustrated in Fig. 1(b), which had a width of 900 mm and a height of 3500 mm. It should be noted that the 900 mm width was chosen because of the limitation of the test setup. According to the Australian standard AS 2327.1 [24], the effective width of the slab was determined as 2086 mm. However, the laboratory testing jig only permitted a width of 900 mm. Thus, the whole slab section could be considered effective in the joint design. Further research is required to study the influence of slab width on the joint behaviour. The composite slab was cast on Bondek steel profile with a thickness of 1.0 mm, where the ribs of the profiled sheeting were parallel to the axis of the primary beam. To reinforce the composite slab, six deformed reinforcing bars (12 mm in diameter) at 160 mm interval and 18 distribution bars (10 mm in diameter) at 200 mm interval were used. It should be noted that two longitudinal bars terminated near the column face to avoid the difficulty in installation. Therefore, only four reinforcing bars were effective and the reinforcement ratio was 0.76% of the effective cross-sectional area [slab width × (slab depth-rib height of the profile sheet)] of the slab. The clear cover to the longitudinal reinforcement was 20 mm. For each slab, 16 M19 stud shear connectors in the longitudinal direction and four in the transverse direction were welded on the top flange of the steel beam to provide full composite action between the steel beam and slab. The arrangement of the shear connectors is shown in Fig. 1(b).

In specimen preparation, various steel components were fabricated first. The steel tubes and beams were cut to the required lengths. To allow the installation of the binding bars, holes were drilled on the corresponding square steel tubes. Then the binding bars were passed through the holes and welded to the steel tube by plug welds followed by machining. After that, two square steel plates were welded to the ends of each tube. The top plate had a circular hole with a diameter of 200 mm used for pouring concrete. A flush endplate was welded to one end of each steel beam segment by full penetration butt welds. After receiving the steel components, the beam segments were connected to the columns in the Structures Laboratory at Western Sydney University. The details of connections to square and circular columns are shown in Figs. 2 and 3, respectively. Six grade 8.8 M20 blind bolts (Lindapter HB20-1 Hollo-Bolts) were used to connect each primary beam segment to the steel tube. Meanwhile, four HB20-1 blind bolts were adopted to connect each secondary beam segment to the steel tube. A recommended torque of 300 N m was used to tighten all the blind bolts. After that, the profiled sheeting was installed followed by the welding of the stud shear connectors onto the steel beam. Then the formwork for the slab was assembled and the slab reinforcement was placed. Finally, ready mixed concrete with a target compressive strength of 32 MPa at 28 days was used to cast the concrete slabs and fill the steel tubes.

Table 1
Details of joint specimens.

Specimen label	Column section type	Column steel type	Binding bars	f_c' (MPa)	N_0 (kN)	N_u (kN)	n	Slab	Test type
SB1-0	Square	Stainless	8Ø20	32.8	3822.3	6037.5	0.63	No	Monotonic
SB1-1		Stainless	–	33.6	3873.4	6122.8	0.63	Yes	Monotonic
SB1-2		Stainless	8Ø20	35.7	3811.9	6346.6	0.60	Yes	Monotonic
SB1-3		Stainless	8Ø20	39.8	3938.7	6783.5	0.58	Yes	Cyclic
CB2-1	Circular	Stainless	–	42.3	3833.5	5661.1	0.68	Yes	Monotonic
CB2-2		Stainless	–	41.2	3921.5	5573.2	0.70	Yes	Cyclic
CB2-3		Carbon	–	43.8	3825.6	5756.4	0.66	Yes	Monotonic