SUSTAINABLE HIGH STRENGTH STEEL FLUSH END PLATE BEAM-TO-COLUMN COMPOSITE JOINTS WITH DECONSTRUCTABLE BOLTED SHEAR CONNECTORS

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ABSTRACT

The design of engineering structures for deconstructability can reduce the energy and cost required for their demolition and the disposal of their construction waste, and it also enhances the sustainability of a building by allowing for easy dismantling and the reuse or recycling of structural components and construction materials at the end of the service life of the building. In addition, using high performance materials such as High Strength Steel (HSS) can improve the sustainability of a structure by providing for higher design stresses and accordingly reducing the self-weight of the structure. This paper describes the results of four full-scale beam-to-column deconstructable composite joints with HSS S690 flush end plates. The structural behaviour of the new system in conjunction with application of post-installed friction-grip bolted shear connectors for developing deconstructable composite floors is investigated. The test results show that the proposed composite beam-to-column joints can provide the required strength and ductility according to EC3 and EC4 specifications, and that the system can be easily deconstructed at the end of the service life of the structure as a proof of concept.

KEYWORDS

Composite joint, bolted shear connectors, blind bolting, deconstructability, high strength steel, sustainability.

INTRODUCTION

Among different construction materials, steel has a great potential to significantly improve the sustainability of construction industry; steel structures have high strength to weight ratios, they can be erected rapidly and their construction and demolition waste can be minimised by employing prefabricated and deconstructable systems. Moreover, using prefabrication and deconstruction in conjunction with steel frames can drastically facilitate the full recycling and reuse of the construction materials and structural components. Accordingly, over the past decade several attempts have been made to enhance the sustainability of steel structures by either using high-strength durable steels or developing prefabricated demountable steel framing systems (Gogue 2012); however, the application of HSS in conjunction with deconstructable frames remains unexplored and this is the main focus of the present study.

The use of HSS has recently gained popularity in the construction industry owing to its higher yield strength, greater corrosion resistance and higher toughness compared with mild steel. In HSS construction, design stresses can be increased and thickness of plates may be reduced that, in turn, can save on the costs of labour, welding, transportation, erection and fabrication. The cost of the foundation may be reduced owing to lower self-weight of HSS structures compared to mild steel structures and significant cost benefits and reduced construction time can be achieved by increasing design stresses and reducing the thickness of plates (Mursi and Uy 2004). However, the efficient use of HSS in structural members has been hampered by problems associated with its lower ductility, weldability, toughness and fatigue resistance. In particular, the lower ductility of the HSS can potentially affect the structural performance of beam-to-column connections where the steel plates can experience large strains well-beyond the yield strain (Giaro Coelho and Bijlaard 2007; Girao Coelho *et al*. 2010). Girao Coelho and Bijlaard (2007) carried out an experimental investigation of moment connections with end plates made from high strength steel of grades S460, S690 and S960 to provide insight into the nonlinear behaviour of these joints and it was concluded that the extrapolation of the design philosophy in the current EC3 provisions, based on the semi-continuous/partially-restrained concept, can provide accurate strength predictions. In addition, it was shown that the HSS end plate connections can provide the rotation demands required for beam-to-column connections of rigid/semi-rigid moment resisting frames.

Apart from its attributes of high-strength and high-performance, design for deconstruction in conjunction with the use of recycled steel can significantly enhance the sustainability of steel structures. In a fully deconstructable steel frame, the beam-to-column connections as well as the floor slab to steel beam connections should have the potential to be easily dismantled. Bolted beam-to-column connections with flush- or extended end plates can partly provide the ease required for dismantling steel frames, but existing composite steel-concrete floor systems typically take advantage of monolithic construction to ensure adequate performance (*i.e*. near full composite action) and hence they cannot be easily disassembled and reused at the end of the service life of the structure. Furthermore, the demolition of monolithic concrete-steel composite floors in which the shear studs have been permanently buried in cast *in situ* concrete (or pockets filled with grout), requires much energy and leads to large amounts of construction waste. Accordingly, there is a need to develop deconstructable steel-concrete composite floors that can be easily dismantled at the end of a structure's service life.

Post-installed Friction-grip Bolted Shear Connectors (PFBSCs) installed through bolt holes placed in precast slabs and pre-drilled in the top flange of the steel beams is a novel method for developing composite action between precast concrete slabs and steel girders. The composite floors employing PFBSCs can be easily dismantled at the end of their service life, and this in turn can minimise the construction waste associated with the demolition of composite floors and can maximise the possibility for future reuse of the structural components (Marshall *et al*. 1971; Dallam 1968; Dallam and Harpster 1968; Kwon *et al*. 2010, 2011; Bradford and Pi 2012a, 2012b; Rowe and Bradford 2013; Lee and Bradford 2013; Ataei and Bradford 2013; Ataei *et al*. 2014, 2015). Furthermore, demountable composite floors with precast slabs and prefabricated steel girders can increase the speed, accuracy and quality of construction and reduce the time and environmental impact (*viz*. noise, disruption to traffic and pollution) of the construction.

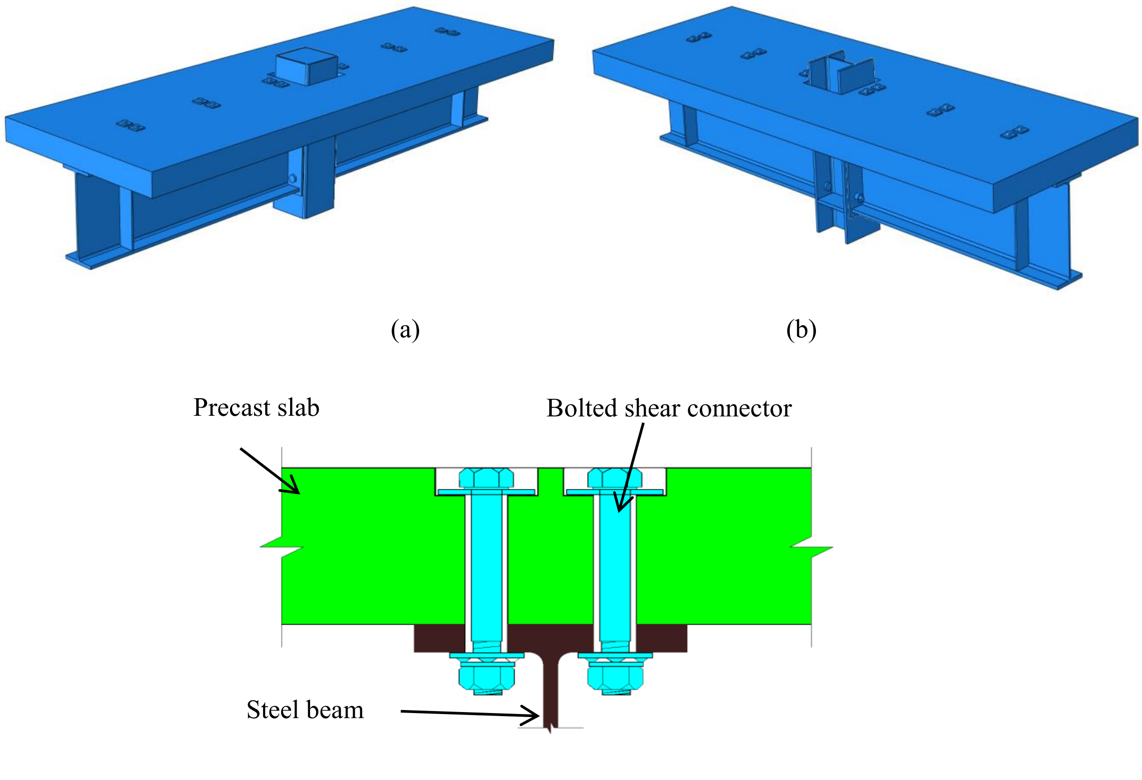
The first tests on bolted shear connectors date back to the late 60s (Dallam 1968; Dallam and Harpster 1968), but surprisingly limited studies have been conducted on the behaviour and application of bolted shear connectors since then (Marshall *et al*. 1971; Kwon *et al*. 2010, 2011; Bradford and Pi 2012a, 2012b; Row and Bradford 2013; Lee and Bradford 2013; Ataei and Bradford 2013; Ataei *et al*. 2014, 2015), and most of these studies are related to bolted shear connectors permanently buried in concrete or grout-filled pockets (Dallam 1968; Dallam and Harpster 1968; Kwon *et al*. 2010, 2011) with less attention being paid to the potential application of PFBSCs for developing deconstructable steel-concrete composite floors (Bradford and Pi 2012a, 2012b; Rowe and Bradford 2013; Lee and Bradford 2013; Ataei and Bradford 2013; Ataei *et al*. 2014, 2015). In general, the available test results show that bolted shear connectors exhibit higher load capacity and significantly higher fatigue strength than those of stud shear connectors (Dallam 1968; Dallam and Harpster 1968; Kwon *et al*. 2010, 2011). Moreover, limited experimental studies on bridge decks have demonstrated the adequacy of PFBSCs for strengthening non-composite bridge girders by increasing their stiffness, load carrying capacity and fatigue strength (Kwon *et al*. 2010, 2011).

This paper presents the results of static tests conducted on four full-scale Flush End Plate Semi Rigid (FEPSR) beam-to-column joints made up of grade S690 HSS in a steel-concrete composite frame that takes advantage of deconstructable PFBSCs and precast "Green Concrete" (GC) slabs associated with reduced cement content (Boral Australia 2013). The main objective is to determine the failure mode and characterise the moment and rotation capacity, moment-rotation relationship and ductility of this new sustainable composite system with high strength steel FEPSR beam-to-column joints. Moreover, the provisions of EC3 (2005) and EC4 (2006) are employed to assess the structural performance of the HSS FEPSR joints with deconstructable composite beams and the influence of the type of bolted shear connectors, degree of shear connection and type of columns (open sections and concrete filled steel tubes) on the structural behaviour of the proposed composite joints are investigated.

TEST SPECIMENS

Specimen design

Four full-scale cruciform beam-to-column joints with flush end plates were designed and constructed according to the provisions of EC3 (2005) and EC4 (2006) to evaluate the stiffness, ductility, bending moment and rotation capacity of the proposed deconstructable composite joints with HSS flush end plates. The beam-to-column assemblages were symmetric to simulate behaviour of an internal joint in a semi-rigid frame. The specimens were tested under a displacement-controlled vertical load applied at the tip of the beam. The geometry, dimensions and details of all specimens are illustrated in Figures 1 to 3 and the details of composite beams and post-tensioned PFBSCs are given in Table 1.



(c)

Figure 1 Schematic outline of deconstructable composite beam-to-column joint with flush end plate connection: (a) pictorial view of CJ1 and CJ2; (b) pictorial view of CJ3 and CJ4; (c) friction-grip bolted shear connection.

Table 1 Test specimens.

Specimen	Column type	Steel beam	Shear connector	Degree of shear connection (%)	Hole diameter in slab (mm)	Hole diameter in steel beam (mm)	Applied bolt pretension (kN)
CJ1	250×250×12.5	460UB82.1	6M20	195	24	22	145
CJ2	250×250×12.5	460UB82.1	6M16	124	20	18	95
CJ3	250UC89.5	460UB82.1	4M16	82	20	18	95
CJ4	250UC89.5	460UB82.1	4M20	130	24	22	145

All four cruciform joints (*viz*. CJ1 to CJ4) consisted of 460UB82.1 steel beam sections. For specimens CJ1 and CJ2, the columns were a concrete-filled tubular steel 250×250×12.5 mm columns and for specimens CJ3 and CJ4, a 250UC89.5 I-section was used. Composite action between the precast concrete slabs and steel beams was provided by the bolted shear connectors installed in pairs as shown in Figure 1(c). Grade 8.8 M20 or M16 high strength bolts were used to attach the precast concrete slab to the top flange of the steel beam. In order to confirm the minimum post-tensioning forces of 95 kN and 145 kN respectively induced in the M16 and M20 bolts, an electric control torque wrench with Squirter Direct Tension Indicating washers were used. The outline and general configuration of the cruciform joints before installation of the precast concrete slabs is shown in Figure 4(a), and the precast concrete slabs after de-moulding are shown in Figure 4(b).

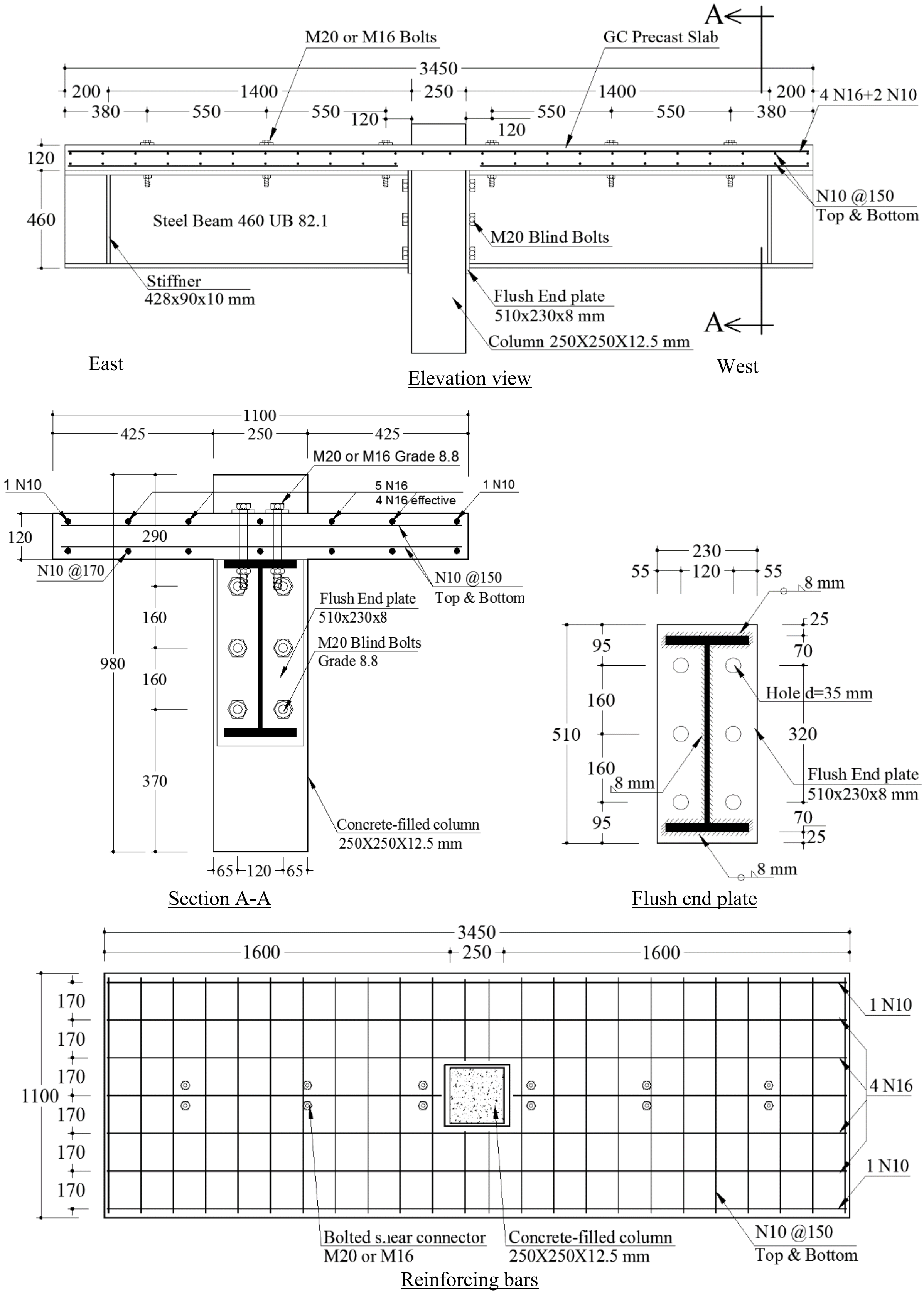


Figure 2 Geometry and details of joints CJ1 and CJ2 (unit: mm).

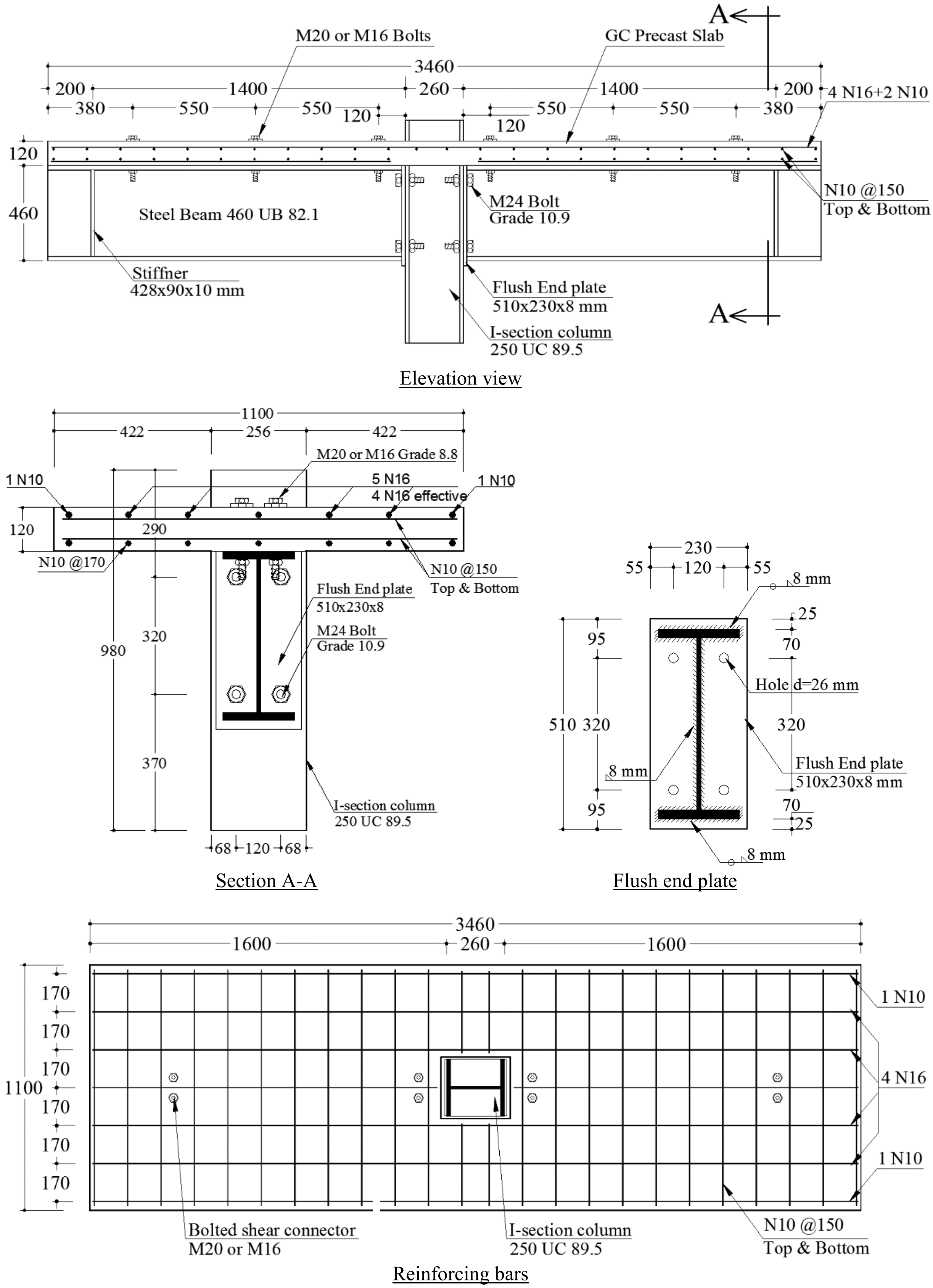


Figure 3 Geometry and details of joints CJ3 and CJ4 (unit: mm).

According to the provisions of EC3 and EC4, in order to prevent non-ductile failure of connections made up of mild steel grades, the thickness of the end plate should be limited to 60% of the bolt diameter (*e.g*. 12 mm thick plate for M20 bolts and 15 mm thick plate for M24 bolts). However, in a study conducted by Ataei *et al*. (2015), it was shown that the end plate thickness recommended in EC3/EC4 cannot sufficiently prevent the non-ductile mode of failure associated with rupture of bolts in FEPSR beam-to-column composite joints with deconstructable PFBSCs, mainly because of the stiffening effect of reinforced concrete slabs that has not been considered in the EC3/EC4 recommended end plate thickness. Accordingly, in this study, an 8 mm thick flush end plate (40% of the M20 bolt diameter for concrete-filled column and about 35% of the M24 bolt diameter for I-section column) made of S690 grade steel was welded to the end of steel girders to ensure ductile mode of failure. Six M20 grade 8.8 Hollow-bolt (blind bolts) (Lindapter International 2004; Standards Australia 1996; Loh *et al*. 2006) and four M24 grade 12.9 bolts were used for connecting the steel girders to CFST columns and I-section columns, respectively.

(a) (b) (c)

Figure 4 (a) Assembled cruciform steel joints before installation of precast concrete slabs; (b) precast concrete slabs after de-molding; (c) specimens CJ1 and CJ2 ready to be tested.

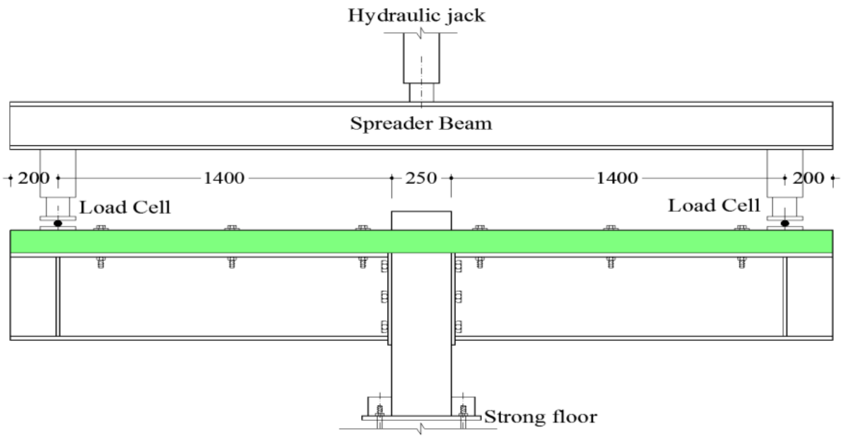
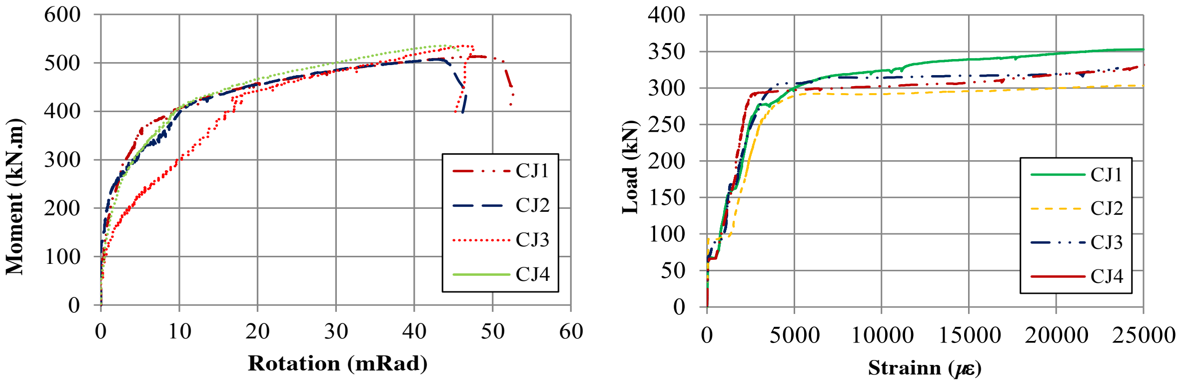


Figure 5 Test set up for the joint tests.

In all tested joints, the precast GC concrete slabs attached to the top flange of the steel girders were continuous over the column as shown in Figure 4(c). A longitudinal reinforcing ratio of 0.73% was provided for the slabs with the reinforcing bar configurations shown in Figures 2 and 3. It is noteworthy that the N10 bottom bars in the slabs were terminated near the column face to prevent their contributing to the bending moment resistance of the joints. Moreover, two layers of N10 bars were used in the transverse direction (see Figures 2 and 3) to prevent the longitudinal splitting of the precast slabs. The configuration of specimens CJ1 and CJ2 after assembling the continuous precast concrete slab are shown in Figure 4(c).

Experimental setup and loading procedure

The test setup and loading procedure for the composite beam-to-column joints are shown in Figure 5.Vertical displacement-controlled loading was applied to the both ends of the composite beams by a 5000 kN capacity actuator and a spreader beam. In order to verify the test set-up and performance of the components and instrumentation, before conducting each test, a small load about 10% of the predicted ultimate load carrying capacity of the specimens was applied to the specimens, and the specimens were then unloaded. The specimens were reloaded until no further load could be sustained by the specimen. Three displacement rates of 0.3 mm/min for the linear elastic range and 0.6 mm/min and 1.2 mm/min for the nonlinear plastic parts were used sequentially.



(a) (b)

Figure 6 (a) Moment-rotation response of the specimens and (b) tensile strain in longitudinal reinforcing steel bars at the mid-span.

EXPERIMENTAL RESULTS

Moment-rotation response

To establish the moment-rotation response of the joints, the moment acting on the connection was obtained by multiplying the load applied at the end of the composite beam by the lever arm (*i.e*. distance between the centre of the loading on composite beam and the column face). Regarding the rotation of the joints, two different methods were adopted; in the first method, the difference between the rotation of the column and the steel beam measured by inclinometers was considered as the rotation of the connection, whereas in the second method, the rotation of the joint was obtained by subtracting the displacements measured by bottom LSCT from that measured by top LSCT and dividing the result by the distance between these two LSCTs. The moment-rotation responses of all four specimens are shown in Figure 6(a). Moment capacities of 513.0, 506.8, 534.8 and 535.4 kNm with rotation capacities of 48.6, 43.3, 46.8 and 44.1 mrad were recorded for specimens CJ1, CJ2, CJ3 and CJ4 respectively, and all specimens exhibited significant non-linearity with very satisfactory moment-rotation behaviour. Figures 7 (a) and (b) illustrate the state of specimens CJ2 and CJ4 after the test. The typical failure mode of the specimens is also shown in Figure 7(c).

(a) (b) (c)

Figure 7 (a) Specimen CJ2 after the test; (b) specimen CJ4 after the test; (c) typical failure mode of the specimens.

According to the EC3 and EC4 provisions, among three possible modes of failure, including yielding of the end plate or column flange, bolt failure combined with column flange or end plate yielding and bolt failure, only the first failure mode associated with complete yielding of the end plate or column flange can be considered as ductile and the third mode of failure in which only bolt rupture occurs should be considered as brittle (non-ductile). In the present experimental study, yielding and plastic deformation of the HSS grade S690 flush end plate in the beam-to-column joints with deconstructable PFBSCs took place with no fracture of the bolts located in the tension zone of the connection, which is characteristic of the ductile failure mode specified in EC3/EC4.

Moreover, in accordance with EC3 and EC4, in order to allow for plastic analysis and design, the rotation capacity of the joint must be greater than 30 mrad and as can be seen in Table 2, all specimens had a rotation capacity higher than that specified in the EC3/EC4 design codes. Accordingly, it can be concluded that despite using HSS flush end plates, the proposed deconstructable connections have sufficient rotation capacity and can be considered as ductile.

Table 2 Test result of composite and non-composite FEPSR beam-to-CFST column joints.

Specimen	Moment capacity (kNm)	Rotation capacity (mrad)	Mode of failure
CJ1	512.3	48.6	Bar fracture
CJ2	506.8	43.3	Bar fracture
CJ3	534.8	46.8	Bar fracture
CJ4	535.4	44.1	Bar fracture

Strain in longitudinal reinforcing bars

The load versus tensile strain in the longitudinal reinforcing bars at mid span of all specimens is shown in Figure 6(b). The sudden increase in tensile strain of steel bars at a load of about 80 kN can be attributed to the development of a transverse crack in the precast concrete slab and at sections adjacent to column. Moreover, it is observed that the average longitudinal strain in the reinforcing bar at the mid span for all composite specimens including the ones with weak partial shear interaction (*i.e*. specimen CJ3) exceeded the yield strain at the ultimate load.

CONCLUSIONS

In this paper, the behaviour of four full-scale sustainable high strength steel Grade S690 FEPSR beam-to-column composite joints with deconstructable PPBSCs was investigated. A novel steel-concrete composite floor with precast GC concrete slabs and deconstructable PFBSCs was adapted to improve the sustainability of the composite floor and enhance possibility for recycling and reuse of construction materials and subsequently reduce the carbon footprint of the construction industry. Type of bolted shear connectors, degree of shear connection and column type were the main variables in the experimental programme. Based on the experimental results, the structural behaviour of this novel composite system that take advantage of precast slabs and PFBSCs in conjunction with high-strength steel flush end plates were investigated. From the experimental results the following conclusions can be drawn.

- The test results show that a HSS FEPSR composite joint with PFBSCs can provide a higher rotation capacity (43 mrad and above) than that specified in the EC3/EC4 codes.
- A ductile mode of failure (as specified in the EC3/EC4 provisions) for beam-to-column joints with deconstructable PFBSCs and HSS flush end plates can be achieved, provided the end plat thickness is limited to 35-40% of the bolt size.
- The friction-grip mechanism mobilised by the pre-tensioning force in the bolted shear connectors, can effectively transmit shear between the precast concrete slabs and top flange of the steel beam at the initial stage of loading.
- Decreasing the degree of shear connection leads to a decrease in the initial strength of composite joint that in turn can increase the deflection of the composite beams under service-load condition.
- First slip with stronger bolted shear connectors occurs at higher levels of load as expected and partial shear connection leads to more ductile behaviour for the composite joint.
- Yielding and plastic deformation of the flush end plate made up of HSS S690 occur without fracture in blots located in the tension zone of the connection, provided the thickness of end plate is limited to 35-40% of the bolt size.

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