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Sustainable Design of Deconstructable Steel-Concrete Composite Structures

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Abstract

As attention is being focused increasingly towards minimising carbon emissions and enhancing the possibly of material recycling in the construction industry, traditional composite systems are recognised as being problematic on several counts. Composite action between the conventional concrete slab and steel beam is provided typically by stud shear connectors welded to the top flange of the steel beam, and the demolition of such members requires a considerable amount of time and energy, as well as being environmentally intrusive and creating much waste. In addition, existing composite systems mostly utilise conventional concrete made from ordinary Portland cement whose production is attributed to a large portion of carbon emissions worldwide. As an alternative, it is proposed that precast concrete slabs be attached to a steel frame with semi-rigid bolted connections using high-strength friction grip bolts as the elements to provide the shear connection. Moreover, the use of geopolymer concrete in the casting of the slabs eliminates the use of ordinary Portland cement entirely. The paper reports tests undertaken on full-scale beams and on full-scale joints in this sustainable and deconstructable system. This study shows that both the joints and beams demonstrate very significant ductility, with large rotations, deformations and interface slips being developed and sustained during the testing.

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Keywords: Composite structures, Bolted shear connectors, Deconstructable, Sustainable design, Geopolymer concrete.

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1. Introduction

Composite steel-concrete structural systems have been widely used for many years throughout the world for medium-sized office buildings, owing to their excellent structural performance in terms of stiffness and strength, their relative ease of construction and the significant economic benefits that accrue to this structural form. Composite steel-concrete beams have higher stiffnesses and strengths, reduced defections and higher span to depth ratios than traditional bare steel or concrete beams, because they take advantage of the favourable compressive strength of the concrete slab and the high tensile strength of the steel joist in a symbiotic configuration. In addition, the rigidity and ductility of composite beam-to-column connections allows for adequate moment redistribution in steel frames subjected to overload and extreme loading scenarios. Composite connections have higher initial stiffnesses and moment capacities as well as greater rotational ductilities compared to bare steel connections, owing to the beneficial effect of the steel reinforcing bars placed in the slab. However, the composite action between the steel beams and concrete slabs is achieved almost universally by using headed stud shear connectors that are welded to the top flange of the steel beam and which are embedded in the cast *in-situ* concrete slab; this makes deconstruction and reuse of any of the concrete and steel components virtually impossible and the associated demolition process is wasteful, energy-intensive and environmentally-intrusive.

Post-installed Friction-grip Bolted Shear Connectors (PFBSCs) can be installed through bolt holes cast in precast slabs and pre-drilled in the flange of the steel beam. These bolted shear connectors can provide efficient composite action between the precast slab and the steel beam by friction-grip and bearing mechanisms. Furthermore, composite floors that take advantage of PFBSCs can be easily deconstructed at the end of the service life of structure and this, in turn, can minimise wastage of the construction materials (associated with demolition of the structure) and maximise the possibility for future recycling of the structural components. Another advantage of this novel composite construction is that the slabs can be cast from geopolymer concrete (GPC) as a replacement for concrete using ordinary Portland cement and with the the steel frames being fabricated off-site, the construction time is reduced and the accuracy and quality of the construction are improved.

Research on the use of bolts as shear connectors in composite construction dates back to the late 1960s. Twelve push tests on high-strength bolts as shear connectors were carried out and reported by Dallam [1]. Dallam [1] noted that high-strength bolted connectors had a higher load capacity (about twice) of that of stud shear connectors. Subsequently, six full-scale simply supported composite beams with high-strength bolted shear connectors were tested by Dallam and Harpster [2], with the bolts being embedded in the concrete slab in the same way as for the push tests [1]. It was concluded that the high-strength bolted shear connectors provide a very rigid connection between the steel beam and concrete slab under service loads, and that a reserve capacity sufficient for the development of the ultimate moment of the fully composite section was also attainable. Marshall et al. [3] appear to be the first investigators to report the use of bolted shear connectors through pre-drilled holes, but the context of the application is not entirely clear in their study. Three decades later, a series of individual shear connector tests was conducted on three types of post-installed shear connectors under static and fatigue loading by Kwon et al. [4]. The results showed that bolted shear connectors exhibit significantly higher fatigue strengths than those of stud shear connectors. Kwon et al. [5] also tested five full-scale beams, in order to investigate the feasibility of using bolted shear connectors for retrofitting non-composite bridge girders. It was found that the strength and stiffness of a non-composite bridge girder could be significantly improved by using post-installed bolted connectors. Recently, a number of numerical and experimental studies, which are parts of this research, have been conducted at UNSW Australia on the use of high-strength friction-grip bolts as shear connectors [6-14].

This paper presents a comprehensive experimental study of the behaviour of composite beams and joints with PFBSCs and precast GPC slabs, as a replacement for traditional composite systems. The detailed results of quasi-static tests on three full-scale composite beams and on four full-scale joints are reported. The structural responses of the composite beams and joints are assessed under a monotonically increasing static load.

2. Composite beam testing

2.1. Test setup

The test specimens comprised of three full-scale composite beams with PFBSCs. The schematic outline of the deconstructable composite beam and the cross-sectional configurations of the PFBSCs are shown in Figs. 1 and 2 respectively, and details of the specimens are summarised in Table 1. The specimens were simply supported with the total length of 7.3 m with a spacing of between the supports of 7 m. All composite beams had a 460UB67.1 steel beam section and the reinforced square GPC slabs were 1000 mm in width with a thickness of 150 mm.

Table 1 Summary of the composite beam specimens.

Specimen	Steel beam	Precast concrete panel (mm)	No. of PFBSCs per panel	Total number of bolts	Degree of shear connection
CB1	460 UB 67.1	1000x1000x150	8	56	>100
CB2		1000x1000x150	4	28	97
CB3		1000x1000x150	2	18	55

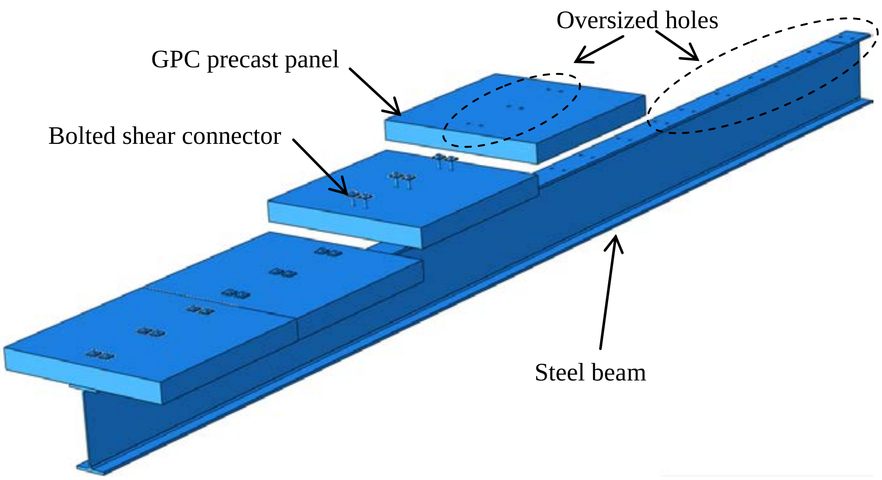

Fig. 1. Schematic outline of sustainable composite beam.

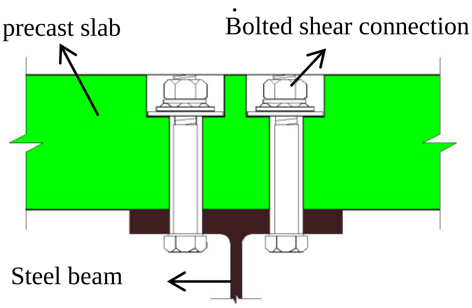

Fig. 2. Cross-sectional configuration of PFBSCs.

Composite beams were designed with the same configuration, consisting of PFBSCs and precast GPC panels. The difference between them was the number of PFBSCs distributed along the beam. Specimen CB1 was designed with full shear connection. A total of 56 post-installed bolted shear connectors of Grade 8.8 M20 bolts (8 bolts per concrete panel) were used in pairs (Fig. 3). Specimens CB2 and CB3 were designed with 97% (14 connectors in each shear span) and 55% (9 connectors in each shear span) shear connection respectively. One additional pair of bolts was used in CB3 near each end of the beam to ensure that a concrete panel would not drop from the beam in the laboratory as a result of bolt fracture. In order to confirm the minimum post-tensioning force of 145 kN induced in the M20 bolts, an electric control torque wrench with squirter direct tension indicating washers was used. The concrete slab for all specimens was assembled from seven juxtaposed precast GPC panels installed on the top flange of the steel beam.

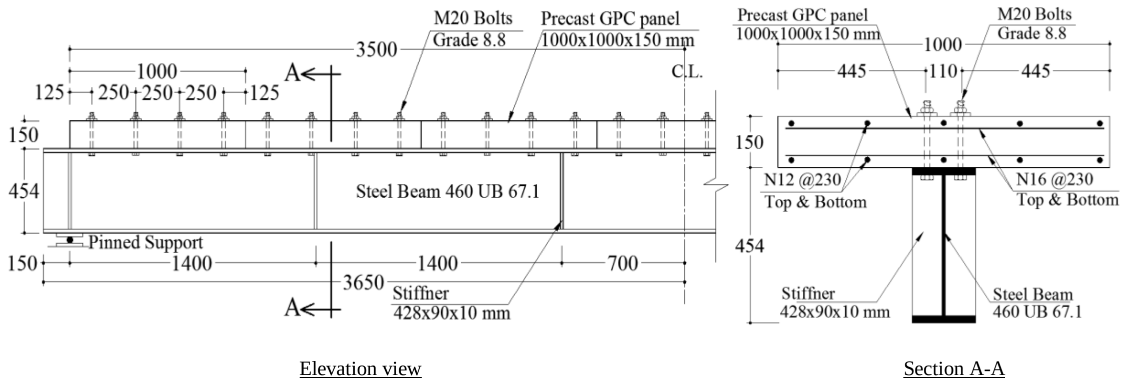

Fig. 3. Geometry and details of composite beam CB2 (unit: mm).

2.2. Experimental results

All composite beams were tested under a monotonically-increasing displacement-controlled quasi-static loading regime and the results are summarised in Table 2.The load versus mid-span deflection curves for all the specimens are compared in Fig. 4. Table 2 indicates that the initial stiffnesses of the composite beams were slightly different, due to the differences in the levels of shear connection. Values of the initial stiffnesses of 23.7 kN/mm, 21.5 kN/mm and 22.1 kN/mm were obtained for specimens CB1, CB2 and CB3 respectively. Specimen CB1 with the highest degree of shear connection achieved 110% and 107% of the initial stiffness of specimens CB2 and CB3, respectively. Nevertheless, it can be concluded that by using tensioned bolted shear connectors, composite beams with partial shear connection can achieve sufficient initial stiffness.

Table 2 and Fig. 4 also show that the specimens with post-installed bolted shear connectors demonstrate very significant ductility, with large deformation and interface slips being developed and sustained during the testing. The Australian composite structures standard AS2327.1 [15] requires that the mid-span deflection of a composite beam under service load conditions does not exceed 1/250 of the span of the composite beam. It can be seen in Fig. 23 that the load corresponding to this deflection (*viz.* 7000/250 = 28 mm) is around 530 kN for all beams, which corresponds to 55%, 57% and 58% of the ultimate loads of CB1 to CB3 respectively. Typically for secondary beams spaced 3 m apart, this corresponds to 530/ (7 × 3) = 25 kN/m^2, which is many times the service load levels encountered in buildings. It may thus be concluded that all the deconstructable beams considered in this study are very stiff in regard to the serviceability limit state, owing to the frictional resistance at the interface.

Table 2 Composite beam test results.

Specimen	Initial stiffness kN/mm	Load Ult. kN	Load Final kN	Deflection Ult. mm	Deflection Final mm	Moment Ult. kN.m	Moment Final kN.m	Maximum end slip West mm	Maximum end slip East mm	Maximum end slip Ave. mm	Failure mode
CB1	23.7	973	887	208	377	1022	932	7.5	5.0	6.3	CC&FLB
CB2	21.5	932	855	278	355	979	898	7.8	6.8	7.3	CC&FLB
CB3	22.1	909	851	248	367	954	894	9.7	9.8	9.7	CC&FLB

Notes: CC= Concrete Crushing; FLB= Flange Local Buckling.

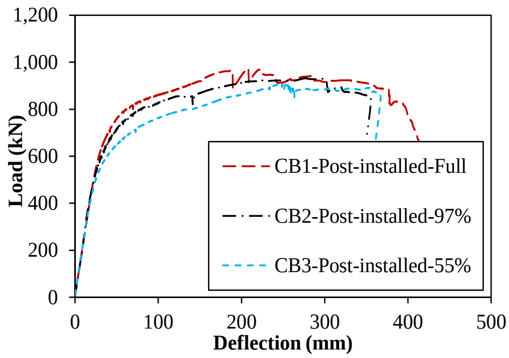

Fig. 4. Load-deflection response of all specimens.

3. Beam-to-column joint testing

3.1. Test setup

Four full-scale beam-to-column joints were designed and constructed in a cruciform arrangement to simulate the internal joint in a semi-rigid frame, and to evaluate the stiffness, moment capacity and rotation capacity of these prototype joints. The sematic outline of the deconstructable composite joint is shown in Fig. 5 and the details of the specimens are summarised in Table 3. Joints CJ2, CJ3 and CJ4 were made composite by bolting precast concrete slabs to the steel beams using PFBSCs that were pre-tensioned, while SJ1 was a plain steel joint, *viz.* without a concrete slab. All specimens consisted of a 460UB82.1 steel beam and a 250UC89.2 steel column. A 12 mm thick flush end plate welded at the end of the steel beam was connected to the flange of the column using four M24 Grade 8.8 bolts. Shear connection was provided by M20 Grade 8.8 bolts placed in pairs. The geometric and design details of specimen CJ1 are illustrated in Fig. 6. Specimen CJ4 used an alternative configuration for the slab. For this, two precast slabs were used, enabling their positioning around a pre-existing column of any height. These two slabs were connected to the steel beams on each side of the column using PFBSCs in the same way as specimen CJ2, but N16 reinforcing bars were passed through preformed ducts cast into the slabs using plastic tubing.

3.2. Experimental results and discussion

The test results for the joints are summarised in Table 4. These results show that the composite joints have credible rotation and moment capacities according to EC4 [16], and that fracture of the joints occurs after a substantial rotational deformation had been reached.

Table 3 Summary of the joint specimens.

Specimen	Steel column	Steel beam	Slab depth (mm)	Rein. ratio	End plate (mm)	Bolt	PFBSCs per beam
SJ1	250 UC 89.5	460 UB 82.1	N.A.	0.00	12	M24	N.A.
CJ2			120	0.90	12	M24	6M20
CJ3			120	0.73	10	M24	4M16
CJ4			120	0.90	12	M24	6M20

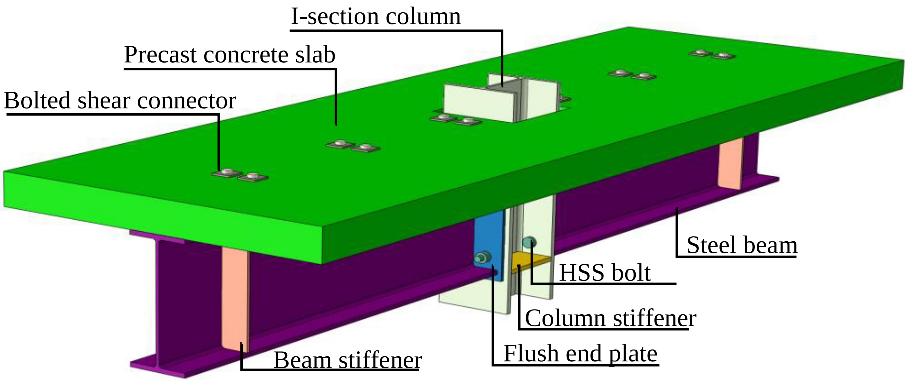

Fig. 5: A typical deconstructable flush end plate composite joint.

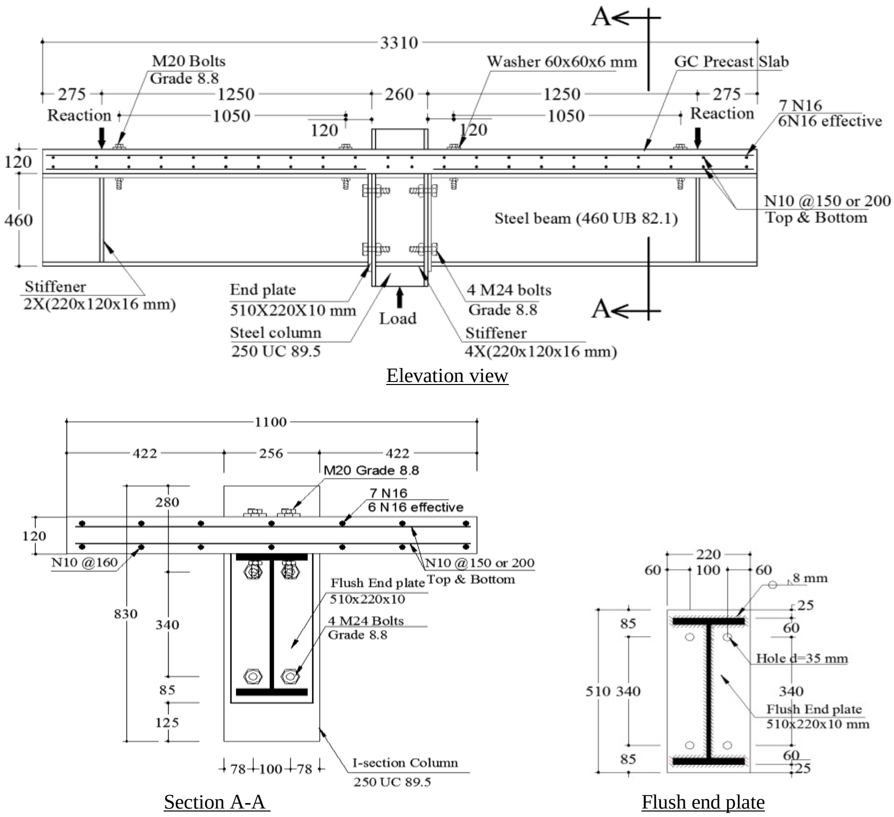

Fig. 6: Details of joints CJ1 (units: mm).

Fig. 7 shows the evolution of the moment versus rotation curves for the three joints; the applied moment being determined by applying the load at the support by the distance from the centre of the support to the column face. The moment capacities for SJ1, CJ2, CJ3 and CJ4 were 256 kNm, 630 kNm, 535 kNm and 622 kNm respectively, and the counterpart rotation capacities were 42 mrad, 44 mrad, 55 mrad and 42 mrad respectively. The rotations were determined as the difference between those of the steel column and steel beam as measured by inclinometers. In terms of strength, the presence of the precast slabs is significant, with the capacities of the composite joints being nearly 2.5 that of the steel joint. Importantly, although the capacity of CJ2 and CJ4 is similar, their structural behaviour is quite different. Almost similar initial stiffnesses were observed in both joints, but the secant stiffness of CJ4 is smaller than that of CJ2. For more detail refer to [14].

Table 4: Joint test results.

Specimen	Initial stiffness (kN.m/mrad)	Moment capacity (kN.m)	Rotation capacity (mrad)	Maximum slip (mm)	Mode of failure
SJ1	42	256	42	N.A.	Bolt fracture
CJ2	139	630	44	4.0	Bolt fracture
CJ3	86	535	55	4.3	Bar fracture
CJ4	122	622	42	0.2	Bolt fracture

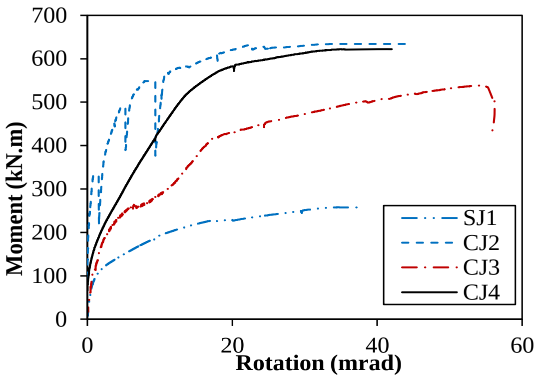

Fig. 7 Moment-Rotation response of specimens.

4. Conclusions and summary

This paper has reported the testing of three full-scale composite beams and four beam-to-column joint tests with PFBSCs. These structural systems possess the novelty of (i) precast GPC slabs associated with low CO2 emissions during their manufacture and (ii) deconstruction and reuse.

Several structural aspects can be concluded from the composite beam tests in which utilize bolts for the shear connection:

- The use of tensioned friction-grip bolts as shear connectors in composite beams with partial shear connection enables initial stiffnesses close to those with full shear connection to be obtained.
- Composite beams with tensioned friction-grip bolted shear connectors are very ductile when compared with conventional composite beams with headed stud shear connectors.
- The interface friction between the slab and steel beam induced by tensioning friction-grip bolted shear connectors resists the shear flow force at that location throughout the early stages of loading, allowing for near to full shear interaction.
- Eventual crushing and top flange local buckling characterises the failure of composite beams having precast panels and tensioned friction-grip bolted shear connectors.
- Composite beams with precast geopolymer concrete panels for their slabs and attached to the steel beam with PFBSCs in clearance holes can be deconstructed successfully when loaded into the service load range, and the components are able to be reused.

Additionally, several pertinent conclusions can also be drawn in regard to the joints which utilise PFBSCs:

- EC4 requires a rotation capacity in excess of 30 mrad for plastic analysis and design, and the system provides a rotation of about 1.5 times this.
- Despite being designed to EC3 and EC4 for complete yield, bolt fracture determined the ultimate strength of the joint, but the joint nevertheless behaved in a ductile fashion.
- Precast slabs improved the moment capacities of the joint significantly, being about 2.5 higher than that for bare steel.
- The behaviour of a monolithic slab and two joined slabs was completely different, with the former being associated with significant slip and the latter with very little slip.

Acknowledgements

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