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Composite Construction of Cold-Formed Steel (CFS) Section with High Strength Bolted Shear Connector

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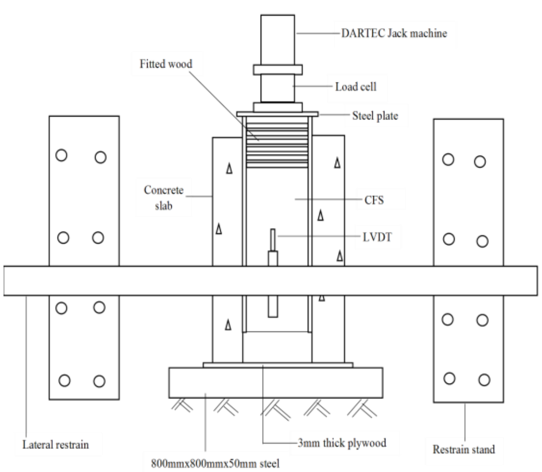
Graphical abstract



Abstract

Composite construction is well known to be effectively used in the construction of buildings and bridges using hot rolled steel (HRS) and headed studs connectors. The shear connectors provides the enhancement as established in current design codes. In this paper, the shear connection mechanism was proposed using high strength bolted shear connector to provide composite action between concrete and the steel section. The conventional headed stud shear connector was eliminated since Cold-formed steel (CFS) was used and welding was not practically possible due to thinness nature of the CFS section. Therefore, in this study investigation was carried out on the strength capacity, ductility and the ultimate flexural capacity of the proposed high strength bolted shear connector and the composite beam specimens respectively. Four push-out and two full-scale composite beam specimens were fabricated using high strength M16 bolted shear connector of Grade 8.8 connected to the top flanges of the CFS I-section and tested to failure using push-out and four-point bending tests respectively. The results show that ultimate load and ultimate moment capacities of the proposed system were significantly improved by using the proposed connectors. The experimental results were compared with theoretical results based on the provision of Eurocode 4, and good agreement between the results was observed. In conclusion, compared results proved that the ultimate moment capacity of the proposed composite beams can be estimated efficiently by using the constitutive laws as prescribed by Eurocodes and British standards.

Keywords: Composite construction, cold-formed steel section, flexural capacity, ultimate load capacity, high strength bolted shear connector, self-compacting concrete

1.0 INTRODUCTION

Cold-formed steel (CFS) sections are lightweight materials that are produced by bending of a flat steel sheet at room temperature [1-3] into a desired shape that can withstand more load than the flat sheet itself; and are suitable for building construction owing to their high structural performance [4]. The most common sections of CFS are channel lipped C and Z sections, and the typical thickness ranging from 1.2 to 6.4 mm with a depth range of 51 mm to 305 mm [5]. In steel construction industries, hot-rolled steel (HRS) and CFS are two distinguished known material types that are popular and frequently been used. However, among the two steel materials, HRS is more familiar among the building contractors and engineers. The steel sections are usually incorporated with concrete to be use in composite construction. Composite construction is achieved when two or more materials are put together to act as a single entity [6-9]. The application of CFS sections started in United States of America (USA) and Great Britain for a number of decades, mainly for non-structural purposes only. Though, the use of CFS is expanding in the present era of building constructions [10, 11]. However, in the mid- 20th century, the structural use of CFS sections has become popular especially for commercial and industrial building constructions [1, 12]. To date, the use of CFS sections as an alternative material for roof and other structural member in buildings is increasing due to the quality assurance of steel products [4]. Significant number of research studies [13-21] demonstrated the advantages of using CFS section in composite construction and bolted shear connectors and substantial findings were recorded. For instance, an extensive study was conducted by Hanaor [13] on the use of CFS section as a composite beam in cast-in situ and precast concrete slabs. Tests conducted consisted of push-out and full-scale flexural tests. Varied in the push-out test using the in situ concrete were the shear enhancements consisted of screwed channel (CS), welded channel (CW) and screwed deck (DS) connectors of the same CFS section embedded in the concrete slab. For the precast specimens, screwed bolt connections and through bolted connections were used. The results showed that the connection methods were effective for attaining the desired capacity and the CFS composite slabs responses were ductile. Irwan *et al.* [15, 17] studied shear transfer enhancement in steel and concrete composite beam system using CFS section. In the study, shear transfer enhancement of bent-up triangular tab shear transfer (BTTST) were created on the flanges of the CFS section to provide the shear connection mechanism. Varied in the study and their influence investigated were dimension and angle of BTTST, concrete compressive strength and CFS section thickness. Push-out and full-scale specimens were fabricated and tested to establish the strength and ductility of the shear transfer enhancement and flexural capacity of the composite beam specimens. The results showed that the ultimate capacity of specimens employed with BTTST significantly increased with an increase in the angle and dimension of the BTTST, thickness of the CFS beam and concrete compressive strength. They concluded that better performance in the ultimate resistance was provided by BTTST shear enhancement. In this paper, the use of CFS section is revealed together with high strength bolts as shear connectors, which could provide suitable means of shear connection in composite construction. In other words, HRS with headed studs shear connectors in the construction of small and medium size buildings will be eliminated. The study aimed in establishing a new concept of composite construction based on the observation that limited technical information is available on composite system that incorporates the use of light gauge steel sections (i.e. CFS section), despite the demonstrated advantages for the system in residential constructions.

2.0 METHODOLOGY

2.1 Material Property Test

CFS channel lipped section with web depth of 250 mm, width of 75 mm and lipped depth of 18 mm and a thickness of 2.3 mm; high strength bolted shear connector of M16 grade 8.8; welded wire fabric mesh A142 of 6 mm thick spaced 200 mm x 200 mm; and self-compacting concrete (SCC) of grade 40 N/mm^2 respectively are the materials used in this study. The materials were tested to obtain their actual strength by tensile, compression and modulus of elasticity tests respectively. The results are shown in Tables 1 and 2.

Table 1 Material property test result

Materials	Yield strength *fy*, (N/mm^2)	Ultimate strength *fu*, (N/mm^2)
CFS	487.43	523.85
Bolt connector M16	768	897
Wire mesh fabric A142	502	595

Table 2 SCC property test result

	Push-out test Concrete sample	Full-scale test Concrete sample
Fresh property		
Slump flow (mm)	610	625
T_{500} (sec)	2.0	2.0
Hardened property		
Compressive strength (N/mm^2) @ 28 days	41.0	40.7
Modulus of elasticity (kN/mm^2) @ 28 days	30.0	35.4

2.2 Push-out Test Specimens

Four push-out test specimens comprised of high strength M16 bolted shear connector of grade 8.8, longitudinally spaced at 250 mm and 300 mm and laterally at 75 mm were fabricated, cast and tested until failure. The I-section beam was made by orienting the CFS back-to-back using a self-drilling screws of 5.8 mm diameter. Bolt holes of 17 mm diameter were drilled on the top flange of the CFS then the M16 bolted shear connectors were installed with single nut and washer at top and beneath the CFS flange through the bolt hole with height of the bolt connector put at 60 mm above the upper flange of the CFS beam section. The push-out test specimens were 800 mm x 600 mm x 75 mm. Welded wire fabric was installed to prevent creeping and shrinkage of the concrete. Figure 1 shows the test specimen preparation.

2.3 Full-scale Composite Beam Specimens

The composite beam specimens were 4500 mm length, effectively spanned at 4200 mm between supports. The effective width of the slab was 1500 mm with a depth of 75 mm. The CFS section was oriented back-to-back to form an I-section beam using a self-drilling screws of 5.8 mm diameter. Bolt holes of 17 mm diameter were drilled on the upper flanges of the CFS beam section. For the shear connection to be provided between the concrete slab and the CFS section, bolted shear connectors of M16 were installed as the same manner in the push-out test program. The fabric wire mesh was installed to prevent creeping and shrinkage of the concrete. Figure 2 shows the preparation of the test specimens.

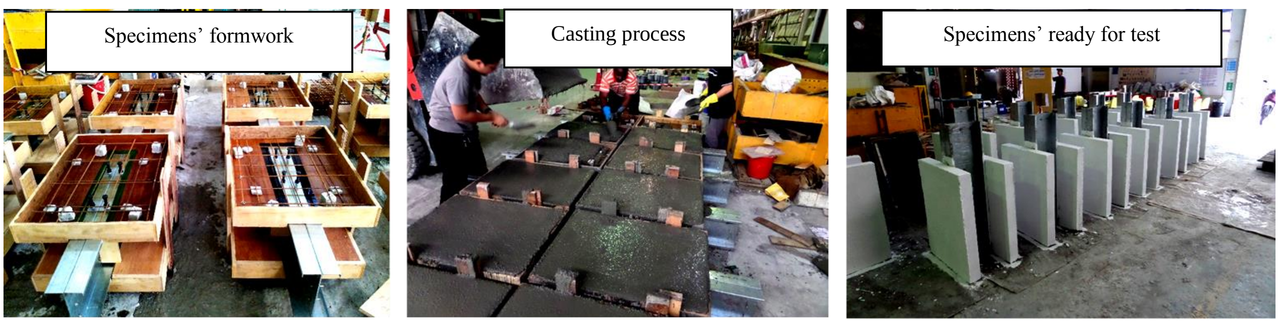


Figure 1 Push-out test specimens preparation

Figure 2 Full-scale test specimens preparation

2.4 Test Set-up and Procedure

2.4.1 Push-out Test

The push-out test set-up is shown in Figure 3, and all specimens were tested using the same procedure. Each specimen was placed on 3mm thick plywood and on a steel section (800mm x 800mm x 50mm thick) to properly lay the concrete slabs. A restraint of an angle steel section was provided to hinge the movement of the test sample of 2000mm length and 10mm thick when the load was applied from the universal testing machine. The capacity of the load cell was 2000 kN and it was applied on the upper vertical side of the CFS beam section. Each specimen was equipped with two linear variable displacement transducers (LVDT's) on the sides of the CFS beam section to monitor the vertical slip between the concrete and the CFS section. The load cell and the LVDT's were connected to a data logger for data collection and subsequent analysis. The load was applied at a constant rate of 0.2 kN/s up to 40% of the predicted failure load. The loading was cycled three times (loading and unloading) between 5% and 40% of the expected failure load. After the cyclic loading, the load was then applied until failure. The loading was stopped when there is a drop of 20% from the maximum load or the specimen fail to resist any additional load.

2.4.2 Full-scale Flexural Test

All the composite beam specimens were tested in the same procedure using DARTEC universal testing machine with a load cell capacity of 2000 kN. Test specimen was subjected to four point bending test, where the load from the hydraulic jack was applied on a distribution beam that rested on two line loads at 1050 mm (shear span) from the supports. The specimen was placed as simply supported beam as shown in Figure 4. Deflections of the specimens were monitored at the mid-span and at the quarter spans underneath the bottom flanges of the CFS section using linear variable displacement transducers (LVDT's). Strains in the specimens were monitored on the concrete slab surface and at the bottom flanges of the CFS section using strain gauges. All LVDT's and strain gauges were connected to the data logger. Due to high concentration of stresses at the supports, premature failure of the CFS may occur. Therefore it was prevented by fitting the supports with a CFS section of dimensions 150 mm x 65 mm x18 mm of thickness 2.3 mm (see Figure 4). Load from the jack machine was applied on the specimen at a constant rate of 0.2 kN/s through the distribution beam which transfer it to the concrete slab through the line load beams. The line load beams were rested on a steel spreader plates of 200 mm x 150 mm x 12 mm thick, to spread the load as a point load on the concrete slab. The specimen was loaded up to 15% of its predicted failure capacity and then initiated to zero. This was to ensure that the instrumentation process was okay and that the specimen was in equilibrium state prior to the proper testing. The specimen was loaded again this time not to 15% of its predicted capacity. Load was further increased until failure of the specimen occurred. The failure of the specimen was considered when there was a significant drop in the applied load or when a large deformation of the test specimen was observed. Lateral restrains were provided during the test to prevent specimen from having premature failure due to lateral torsional buckling.

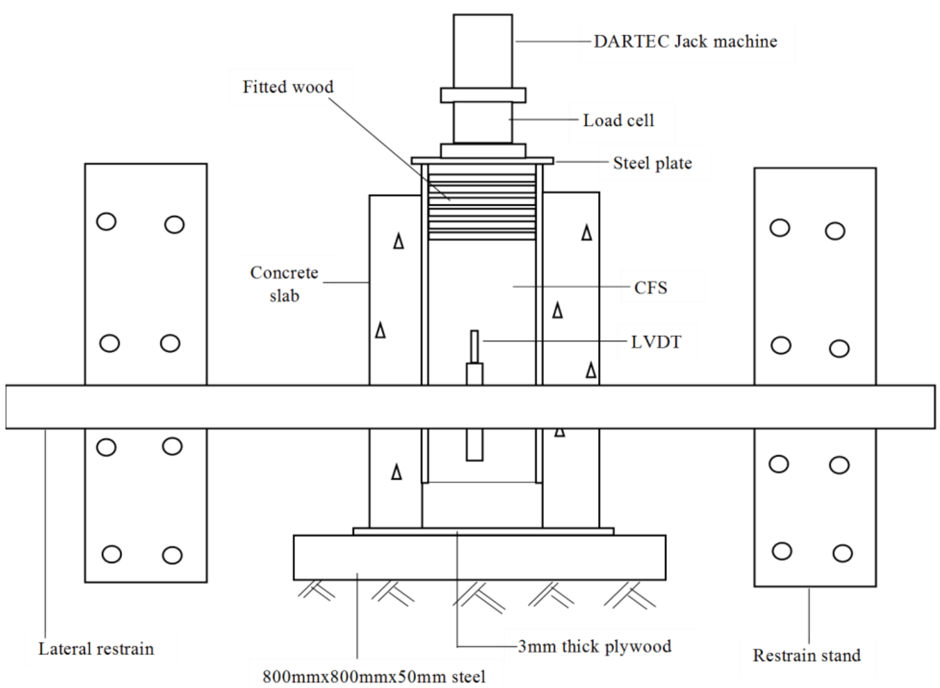


Figure 3 Push-out test set up

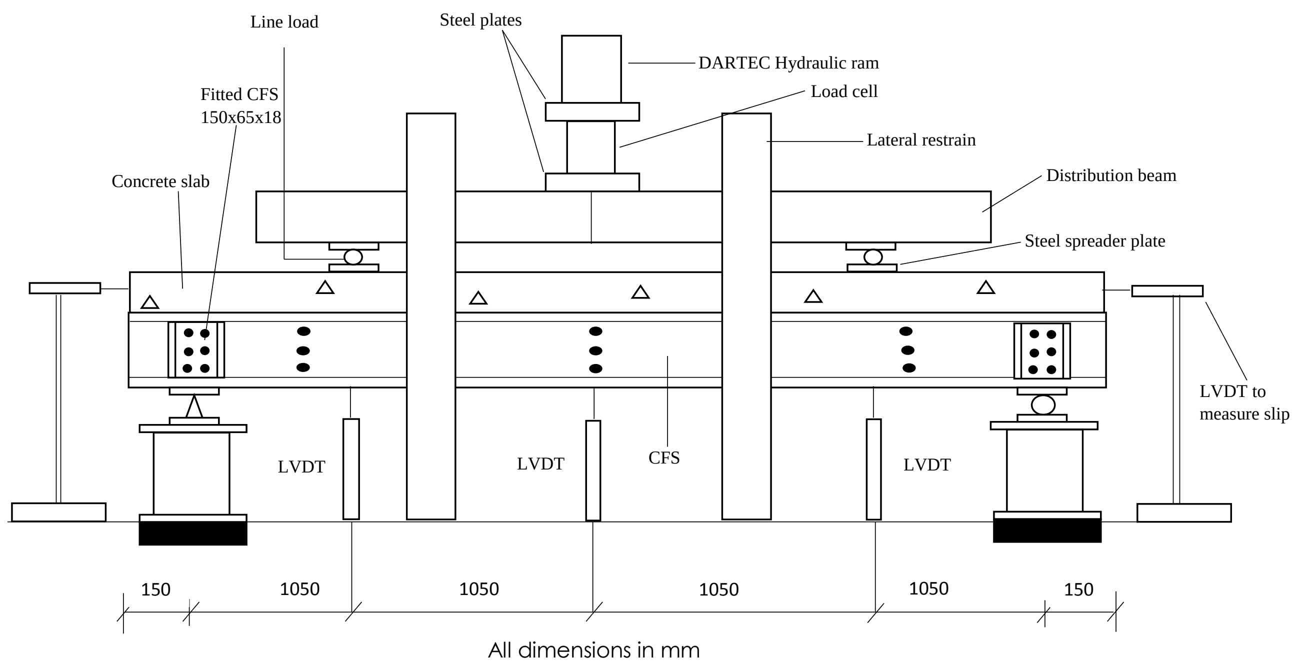


Figure 4 Full-scale composite beam specimens test set up

3.0 RESULTS AND DISCUSSIONS

3.1 Push-out Test

The push-out test result is presented in Table 3. Figure 5 shows the load-slip relationships of tested specimens. Referring to Figure 5, it can be observed that the specimens showed a good resistance capability from the applied load. The specimens started having cracks at load levels above 250 kN before attaining their ultimate load levels above 450 kN as shown. The failure mode obtained from the tested specimen can be categorized as:

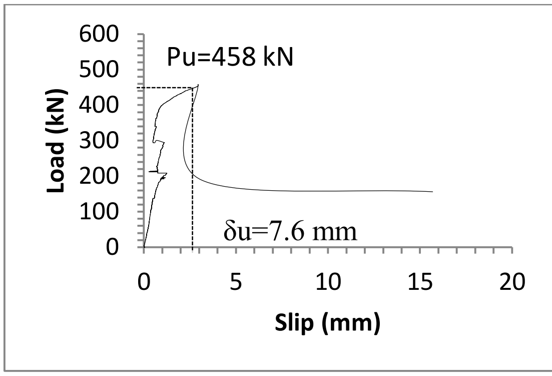
i. Concrete cracking and crushing
ii. Steel flange buckling

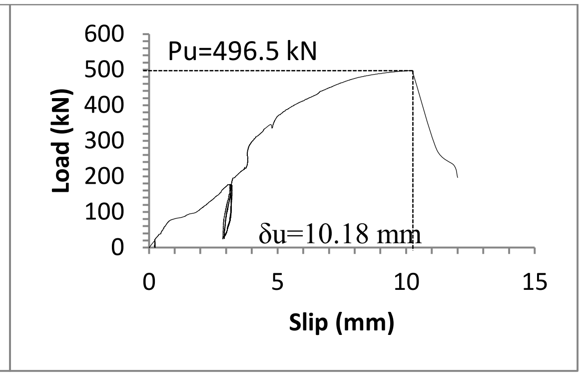
The failure mode of the test specimens could be attributed to the concrete cracking on the surface of the slab both longitudinally and transversely and beneath the concrete slab. Concrete crushing beneath the slab was observed as shown in Fig. 6 (a-c). Failure of the steel section due to flange buckling was observed at the ultimate load level. The steel buckling failure happened at the top shear connectors position of the specimens that was near the load application position. It then extended upwards from the initial position where it had occurred to the part where the CFS was not covered by the concrete slab as shown in Figure 6(d).

Table 3 Push-out test result

Specimen ID	P_u per connector (kN)	$P_{u\,pre.}$ per connector (kN)	$P_{u\,exp.}/P_{u\,pre.}$	δ_u (mm)	δ_{uk} (mm)	Failure mode
PS250-16-1	57.25	64.01	0.89	7.6	6.8	Concrete cracks + crushing+ Steel buckling
PS250-16-2	62.06	64.01	0.97	10.2	9.2	
PS300-16-1	59.35	59.41	1.00	13.6	12.2	
PS300-16-2	62.14	59.41	1.05	7.9	7.1	
Mean	60.20	61.71	0.98	9.83	8.80	
Standard deviation	2.40	2.70	0.07	2.80	2.50	

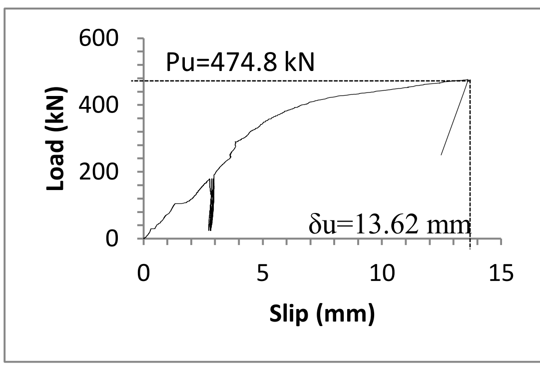
PS300: push specimen@300 mm spacing; Pu: ultimate load; Pu pre: predicted load; δu: slip at ultimate load; δuk: characteristic slip capacity

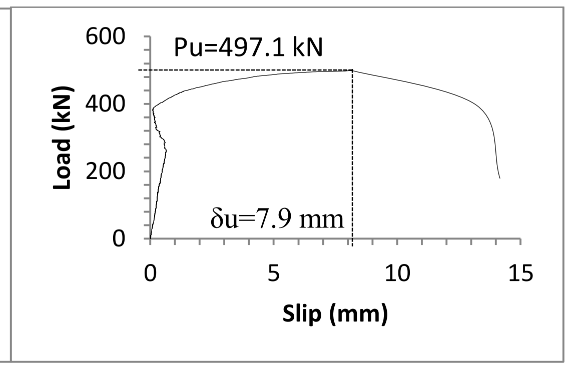




(a)PS250-16-1

(b) PS250-16-2





(c) PS300-16-1

(d) PS300-16-2

Figure 5 Load-slip responses of push-out test specimens

(a) Longitudinal and transverse cracks on slab surface

(b) Transverse cracks beneath the concrete slab

(c) Concrete crushing beneath the slab

(d) CFS flange buckling

Figure 6 Failure modes of push-out test specimens

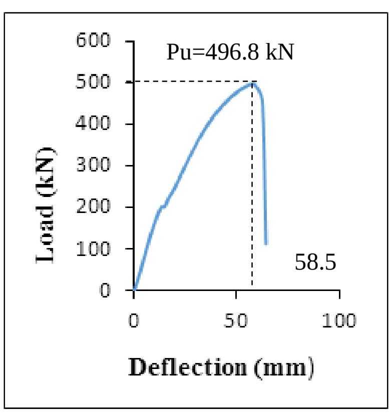
3.2 Full-scale Composite Beam Test

The experimental test result of the composite beam specimens is presented in Table 4. Figure 7 shows the load against mid-span deflections of the composite beam specimens. From Figure 7 and Table 4, the ultimate loads ($P_{u, exp.}$) attained for specimens FS250-16 and FS300-16 were 496.8 kN and 499.6 kN with an initial crack observed at loads level of 228 kN and 230 kN respectively. Mean and standard deviation of 498.2 kN and 1.98 were obtained. Mid-span deflections at ultimate loads level were recorded as 58.5 mm and 66.6 mm for FS250-16 and FS300-16 specimens respectively. Mean and standard deviation of 62.55 mm and 5.73 were also obtained. The specimens exhibited the same failure modes by flexure which resulted to concrete crushing and transverse cracks underneath the concrete slab. CFS Web buckling was noted at the point load position due to excessive stress that was developed when the specimen's ultimate load was reached. From the results, specimen with M16 bolted shear connector at 300 mm attained an ultimate moment capacity of 0.6% higher than that at 250 mm longitudinal spacing. However, this shows that, the moment carrying capacity between the specimens does not differ much. Therefore, this is an indication that the specimens could provide the required composite action, considering the load resisted by the specimens (see Table 4). The ultimate moment has a mean and standard deviation of 261.55 kNm and 1.06 (see Table 4). It can be clearly observed that, as the shear connector longitudinal spacing was increased from 250 mm to 300 mm, the ultimate moment capacity attained also increased. This shows that, the shear connector longitudinal spacing influenced the load and moment carrying capacity of the specimens. The influence of the longitudinal spacing of the shear connector on the ultimate moment capacity agrees well with investigation carried out by Lakkavalli and Liu [14]. Figure 8 (a-b) shows the failure modes of the test specimens, and Figure 8(c) shows the shear connector status after test.

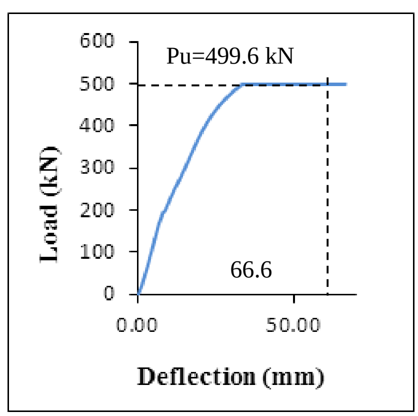
Table 4 Flexural test result of composite beam specimens

Specimen ID	f_{ck} at test day (N/mm^2)	Ultimate load, $P_{u, exp.}$ (kN)	Deflection at $P_{u, exp.}$ $\delta_{u, exp.}$ (mm)	Ultimate moment, $M_{u, exp.}$ (kNm)
FS250-16	30.0	496.8	58.5	260.8
FS300-16	32.0	499.6	66.6	262.3
Mean	31	498.20	62.55	261.55
Standard deviation	1.41	1.98	5.73	1.06

FS250-16: Full-specimen @ 250 mm spacing center-to-center with M16 bolt diameter



(a) FS250-16



(b) FS300-16

Figure 7 Load-mid-span deflection of composite beam specimens

(a) (b) (c)

Figure 8 Failure modes of composite beam specimens [(a) Concrete crushing on slab surface, (b) Transverse cracks beneath the slab and web buckling, (c) Non-deformed shear connector]

3.3 Comparison between Experimental and Theoretical Results

The experimental results were compared with theoretical results. The theoretical calculation was based on the well-known rigid plastic analysis. From the results of the analysis, it can clearly be seen that, the experimental results values agrees well with the theoretical values. The experimental shears (V_{exp}) were determined by dividing the ultimate load by 2. Whilst, the experimental moments (M_{exp}) were evaluated by multiplying the experimental shears by shear span distance that is provided in the test set-up (i.e. 1050 mm = 1.05 m). The results of the comparison is presented in Table 5. Referring to Table 5, it can be observed that, the ratio of experimental shears to that of theoretical ranges from 1.02 to 1.07 with mean and standard deviation values of 1.00 and 0.04 respectively. Moreover, in terms of moment capacity, the ratios ranges from 1.06 to 1.11 with mean and standard deviation values of 1.09 and 0.04 respectively. This shows that, close agreement between the compared results is well achieved.

Table 5 Results of comparison between experimental and theoretical values

Specimen ID	Experimental results		Theoretical results			
			Interpolation method			
	$V_{exp.}$ (kN)	$M_{exp.}$ (kNm)	V_{theory} (kN)	$V_{exp.}/V_{theory}$	M_{theory} (kNm)	$M_{exp.}/M_{theory}$
FS250-16	248.4	260.8	242.5	1.02	247.1	1.06
FS300-16	249.8	262.3	232.6	1.07	236.0	1.11
Mean				1.00		1.09
Std. deviation				0.04		0.04

4.0 CONCLUSION

From the results of the experimental tests, the following conclusions can be drawn:

1. The shear connector used can be classified as ductile connector as it achieved an average characteristic slip capacity of 8.8 mm more than 6 mm as recommended by Eurocode 4.
2. The shear connector used demonstrated a good strength capacity.
3. It can be concluded that, concept of composite construction using light gauge steel section (CFS) is demonstrated. This is proved by the compatibility of bolted shear connection system with CFS section.
4. Close agreement is observed between experimental and theoretical shears of the push-out specimens with a standard deviation value of 0.07.
5. Experimental shear and moment capacities of the full-scale specimens are in good agreement with the theoretical values with standard deviation of 0.04.
6. Strength capacity of the composite beam specimens increases with an increased in the spacing of the shear connector.
7. The shear connector showed to have the capacity of providing good composite action between concrete slab and the steel section as no deformation is observed on the shear connector.
8. The results compared proved that the plastic analysis for the ultimate moment capacity of the composite beams can be estimated efficiently by using the constitutive laws as prescribed by Eurocodes and British standards.

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