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FEASIBILITY OF USING BOLTED SHEAR CONNECTOR WITH COLD-FORMED STEEL IN COMPOSITE CONSTRUCTION

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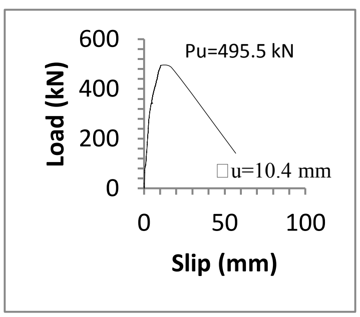
Graphical abstract



PS300-14 Specimen

Abstract

In conventional composite construction for hot-rolled steel (HRS) section, the composite action is usually achieved by using headed studs shear connectors. But, for cold-formed steel (CFS) section, the use of headed studs is not feasible as the section is very thin and difficult to be weld. Therefore, an innovative way of shear connection mechanism of using bolt and nut is suggested in this study. This paper presents the feasibility of using bolt as shear connector by presenting experimental test results so as to explore more on their capability to be used as shear connectors. The study investigated the structural capability of the proposed bolted shear connector when used in concrete known as Self-compacting concrete (SCC) integrated with CFS to provide the required composite action. Push out test specimens with bolted shear connector of grade 8.8 at designated intervals longitudinally spaced were fabricated, cast and tested to failure. The results showed that the proposed shear connector was structurally capable and also an appreciable strength resistance was achieved.

Keywords: Cold-formed steel, Composite construction, Composite Beam, Self-compacting concrete, Bolted shear connector, Push-out test

1.0 INTRODUCTION

The application of cold-formed steel (CFS) in light steel framing structures could be one of the Industrialized Building System (IBS) that can optimizes economical and sustainable construction [1]. In composite construction for buildings and bridges, the use of Hot rolled steel (HRS) with headed studs shear connectors is the most widely used [2] than with the CFS. Therefore, the use of bolts as shear connectors with CFS can eliminate the dependency on using HRS with headed studs shear connectors in the construction of small and medium size buildings. Many research studies were conducted [3-6] on push-out test to study the strength capability and ductility of different types of shear connectors using various types of steel sections. For instance, a study by Xu *et al*. [6] was conducted using headed stud shear connector with HRS. Another study conducted by Baran and Topkaya [7] used channel as a perfobond shear connector with HRS. Bolted shear connectors with HRS were used by Pablovic *et al*. [8]

and good performance was recorded. But, few works have been reported on using CFS with bolted shear connectors. Alenezi *et al.* [9] studied the performance capability of M10 bolt as a shear connector when used with CFS by push-out test. Bamaga [10] investigated the possibility of using CFS with headed stud shear connectors by push-out test. Recently, Saggaff *et al.* [11] investigated the use of CFS with different types of bolted shear connectors and ferrocement; good performance was recorded. Another study was conducted by Alenezi *et al.* [12] in which ferrocement with bolted shear connectors were used in composite construction and good performance was reported to be demonstrated. From the reported literatures, none has reported on the use of bolted shear connectors with Self-Compacting Concrete (SCC) integrated with CFS section. Therefore, this paper reports on the feasibility of using bolted shear connector integrated with CFS and SCC in composite construction as it's yet to be established.

2.0 MATERIALS AND METHODS

2.1 Materials

CFS lipped channel section, high tensile hexagon head bolt of grade 8.8, welded wire fabric and a ready mix SCC were the materials used in this study. The steel yield strength was 450 N/mm^2 minimum as specified by the manufacturer with a web depth of 250 mm, flange width of 75 mm, lipped depth of 18 mm and a thickness of 2.3 mm. The high tensile hexagon head bolt was of M14 with length of 75 mm and a specified ultimate strength of 800 N/mm^2. The wire mesh was A142 of 5.8 mm thick. SSC was of 40 N/mm^2 at 28 days. Tensile test was conducted on the CFS, bolt connector and the wire mesh to determine the actual strengths of the materials.

2.2 Material Properties

2.2.1 Cold-Formed Steel Section (CFS)

Properties of CFS section were evaluated by coupon tensile test. The size of coupon and the procedure of the tests were conducted based on BS EN10002-11 [13]. Three specimens each from web and flange of the CFS were taken and were designated A1, A2, A3 and B1, B2, B3 respectively. The test was conducted such that the yield and the ultimate stresses of the CFS section can be obtained using a Universal Testing Machine INSTRON 600DX with a capacity of 600 kN. After the specimen was placed in the machine's gripped ends, the test was conducted under stress-strain mode with a speed of 5 mm/ min. Load was increased until failure had occurred i.e. when the ultimate tensile stress was reached. The results are presented in Table 1.

Table 1 Results of CFS coupon tensile test

	Web			Flange			Avrg	fu/fy	CFS Thickness, t (mm)
	A1	A2	A3	B1	B2	B3			
fy	490.88	478.33	480.62	488.93	490.91	494.93	487.43		
fu	526.84	513.01	516.09	526.61	528.49	532.04	523.85	1.07	2.3
Es	212.88	209.65	214.59	227.09	199.37	204.98	211.43		

fy: yield stress (N/ mm^2); fu: ultimate stress (N/ mm^2); Es: elasticity modulus (kN/ mm^2)

2.2.2 Bolted Shear Connector

The type of shear connector used in this study was high tensile hexagon head bolt of grade 8.8 of size M14 x 75 mm. Properties of the bolted shear connector was obtained by tensile test in accordance with BS EN ISO898-1 [14]. The test was conducted using a Universal Testing Machine INSTRON 600DX with capacity of 600 kN such that the yield strength, ultimate strength and the maximum failure load can be obtained. After the bolt specimen was gripped in the machine, the test was conducted under stress-strain mode with a speed of 5 mm/ min. The results of the bolt tensile test are presented in Table 2.

Table 2 Results of bolts tensile test

Bolt size (mm)	Maximum load (kN)	Average Max. load (kN)	Yield stress, fy (N/mm^2)	Average fy (N/mm^2)	Ultimate stress, fu (N/mm^2)	Average fu (N/mm^2)	fu/fy	Average fu/fy (N/mm^2)
M14-1	95	96	715	720	813	807	1.14	1.12
M14-2	97		725		800		1.10	

2.2.3 Welded Wire Fabric Mesh

The fabric wire mesh used in this study was of 5.8 mm thick actual diameter and spaced 200 mm x 200 mm centers. The steel bar of the welded wire fabric was of grade 460 N/ mm^2 and was tested for tensile to obtained the yield and the ultimate strengths in accordance with BS4449 [15]. Three sample specimens were randomly selected for the test, and it was conducted using a Universal Testing Machine INSTRON 8801 with capacity of 100 kN. The test results are presented in Table 3.

Table 3 Results of welded wire fabric reinforcement tensile test

	Specimen			Average	fu/fy	Diameter (mm)
	C1	C2	C3			
Yield strength, fy (N/mm^2)	512.46	495.19	499.66	502.44	1.18	5.8
Ultimate strength, fu (N/mm^2)	603.87	578.44	602.48	594.93		

2.2.4 Self-Compacting Concrete (SCC)

The concrete used was a ready mixed SCC designed for 40 N/mm^2 cube compressive strength at 28 days and a slump flow of 550 mm-650 mm. Fresh properties of the SCC were evaluated in accordance with standards [16, 17]; while the hardened properties of the SCC, i.e. cube compressive strength (fcu) and elastic modulus (Ec) were obtained by cube compression and cylinder tests respectively. The compression test was conducted in accordance with BS EN12390-3 [18] and the cylinder test was conducted in accordance with BS1881-121 [19]. Both fresh and hardened properties test results are presented in Table 4.

Table 4 Results of SCC fresh and hardened properties

Fresh				
Slump flow (mm)	T_{500} (sec)	J-ring slump (mm)	T_{500j} (sec)	V-funnel time (sec)
590	2.0	650	2.0	4.0
610	1.5	635	2.0	4.0
630	1.5	620	1.5	3.0
Average 610	2.0	635	2.0	4.0

Hardened			
Cube compressive strength, fcu (N/mm^2)	Cylinder compressive strength, fck (N/mm^2)	Splitting tensile stress, fct,sp (N/mm^2)	Elastic Modulus, Ec (kN/mm^2)
40.25	32.20	3.0	28.68
42.21	33.77	3.5	29.23
42.11	33.69	2.9	29.04
Average 41.52	33.22	3.1	28.98

2.3 Method

2.3.1 Push-out Test

The push-out test specimens were fabricated by orienting two lipped channel CFS back-to-back using a self-drilling screw of 5.8 mm diameter to form an I-beam section. Bolt holes of 15 mm were drilled on the upper flanges of the CFS section and the bolted shear connectors of M14x75 mm were installed with single nut and washer above and beneath the CFS top flange. Fabric wire mesh was installed to prevent shrinkage, creeping and cracks of the concrete slab. The concrete slabs were cast and designated as slab A and B which were of 800mm x 600mm x 75mm (i.e. height, width and thickness) respectively. Height of the bolted shear connectors embedded in the concrete were kept 60 mm and spaced laterally at 75 mm and longitudinally at 150 mm, 250 mm and 300 mm centers. A recess of 80 mm was provided to allow for slip during testing. A total of three tests were conducted using DARTEC universal testing machine with a load cell capacity of 2000 kN. Each push-out specimen was equipped with two displacement transducers (DT) on either side of the steel beam to measure vertical slip during testing between steel beam and the concrete slabs. The two DT's and the load cell were all connected to a data logger. The test set-up and the data acquisition are shown in Figure 1. The loading was applied at a constant rate of 0.2 kN/s up to 40% of the expected 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 loading was then initialized to zero. The specimen was further loaded again until failure occurred. Loading was stopped when the specimen failed to resist any additional load, or a drop of at least 20% from the maximum load occurred as stated in Eurocode 4 [20].

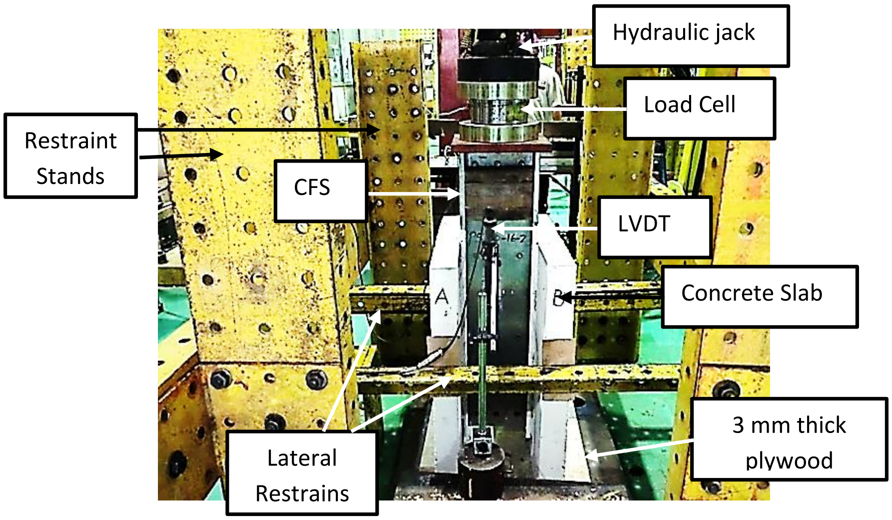


Figure 1 Push-out test set-up and data acquisition

3.0 RESULTS AND DISCUSSIONS

The results of the experimental test are presented in Table 5. Figure 2 (a-c) shows the plots of the load-slip relationships of tested specimens PS300-14, PS250-14 and PS150-15 respectively. It can be observed that a linear relationship between the load and the slip was noticed up to a load levels of about 400 kN, 350 kN and 300 kN (see Figure 2 (a-c)) for specimens PS300-14, PS250-14 and PS150-14 respectively. Then non-linear behaviour started until when the ultimate load was reached. The failure modes of the test specimens with M14 bolted shear connector could be ascribed to the cracks developed on the surface and underneath the concrete slabs. The cracks became moderately larger at underneath the concrete slabs as the applied load was increased which resulted to crushing of concrete slab (Figure 3 (a-c)). But, as the ultimate load was reached, the CFS section experienced a failure by flange buckling. Perhaps high resistance of the applied load by the concrete slabs and bolted shear connector could be liable for it. The steel buckling failure occurred at the top shear connectors' level of the specimens which were close to the load application position. The failure then extended upwards from the initial position where it had occurred to the part where the CFS was not covered by the concrete slab; and it also resulted in the web buckling (Figure 3 (d)). A remarkable shear resistance and slip at ultimate load were attained by the specimens with the bolted shear connector of M14 (see Figure 2 (a-c)). From Table 5 and Figure 2 (a-c), it can be observed that the M14 bolted shear connectors developed a ductile behaviour with an average characteristic slip capacity of 9.7 mm which is higher than 6 mm recommended by Eurocode 4 [20]. Therefore, the M14 bolted shear connector used can be classified as ductile connector. Comparison between experimental results with that of theoretical results showed that there was an increased in shear strength capacity of 32% higher than that of the theoretical values for specimen that had 300 mm spacing. Again, for specimen with 250 mm spacing, the increase observed was 16% between experimental value and the theoretical value. An increased in shear strength of 14% was also manifested between experimental and the theoretical results for specimen with 150 mm spacing. The predicted ultimate load values were determined based on the equation given in Eurocode 4 [20] as shown in Eqn. (1). Therefore, from this study, it showed that the specimens demonstrated good shear carrying capacity. Furthermore, it can be observed that ratio of experimental ultimate load to that of predicted ultimate ranged from 1.14 to 1.32, with mean and standard deviation of 1.21 and 0.10. This is an indication that good strength capability was established by the test specimens with incorporation of M14 bolted shear connector.

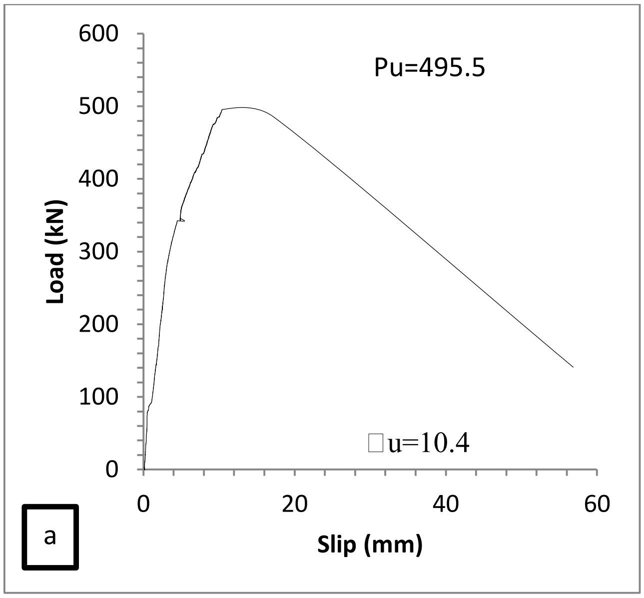
$$P_{Rd} = \frac{0.29\alpha d^2\sqrt{f_{ck}E_{cm}}}{\gamma_v} \qquad (1)$$

Where;

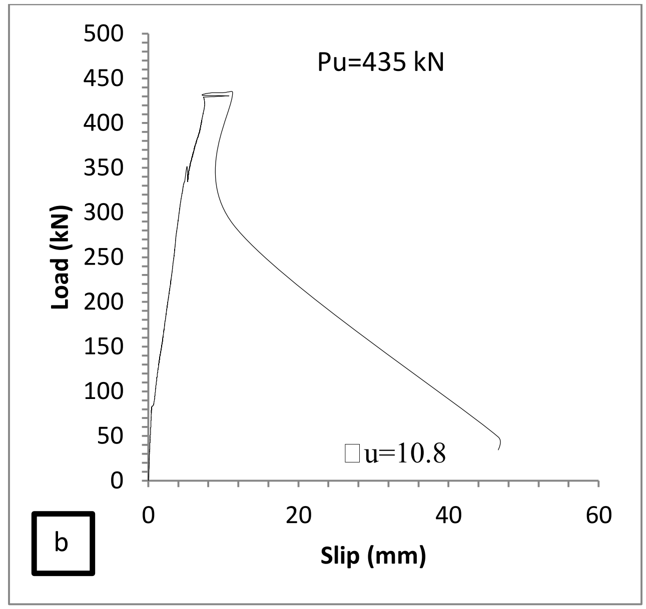
P_{Rd}: Shear connector design resistance

α: Dimensional coefficient

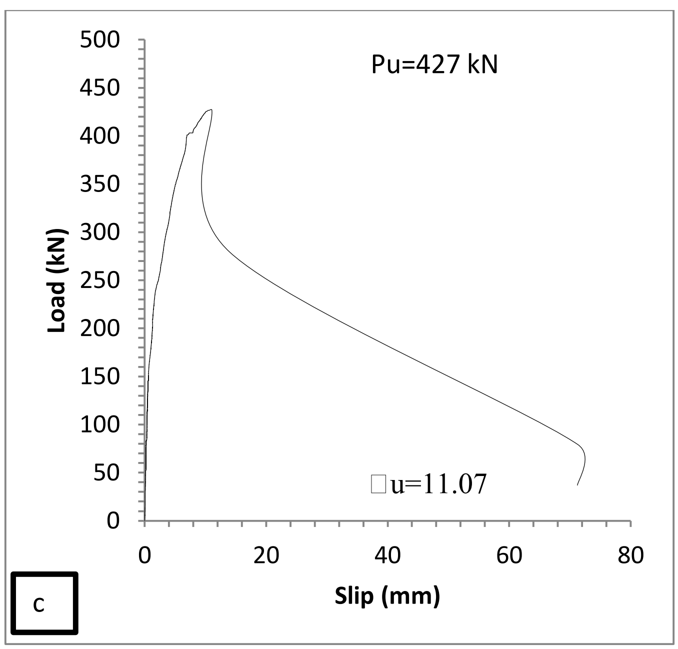
d: Diameter of the stud (bolt)
f_{ck}: Cylinder compressive strength
E_{cm}: Concrete modulus of elasticity
γ_v: Partial factor of safety (taken as 1.25)



PS300-14 Specimen

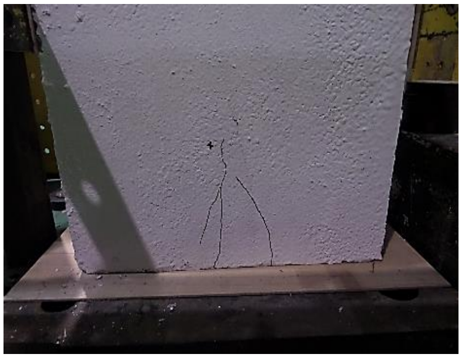


PS250-14 Specimen



PS150-14 Specimen

Figure 2 Load-slip curves of tested specimens



(a) Cracks on the slab surface

(b) Cracks underneath the slab

(c) Concrete crushing

(d) CFS flange failure

Figure 3 Failure modes of tested specimens

Table 5 Experimental test results

Specimen ID	Pu per connector (kN)	Pu pre. per connector (kN)	Pu exp/Pu pre	δuk (mm)	Failure mode
PS300-14	61.94	47.00	1.32	9.4	Concrete
PS250-14	54.38	46.80	1.16	9.7	crushing +
PS150-14	53.38	46.90	1.14	10.0	Steel
					buckling
Mean	56.57	46.90	1.21	9.70	
Standard deviation	4.68	0.10	0.10	0.30	

PS300-14: push-out specimen@300 mm longitudinal spacing with M14 bolt connector; Pu: ultimate load; Pu pre: predicted load; δuk: characteristic slip capacity

4.0 CONCLUSION

Laboratory investigations on push-out test for three specimens have been successfully carried out. All specimens failed due to concrete cracking, crushing and CFS flange buckling, which also leads to web buckling. It was noted that by increasing the longitudinal interval of the bolted shear connector from 150 mm to 250 mm and to 300 mm, the shear carrying capacity increased in the range of 1.14 to 1.32 (see Table 5). This work is limited only to one single type shear connector (i.e. M14 bolt). It is recommended that more investigations should be carried out on different types of bolted shear connectors below and above the size considered in this study; so that different failure modes could be identified and also to establish their feasibility as shear connectors in composite construction.

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