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Bolted Shear Connectors Performance in Self-Compacting Concrete Integrated with Cold-Formed Steel Section

Abstract

In conventional composite construction involving hot-rolled steel section, the composite action is usually achieved by using conventional headed shear studs connectors. However, use of headed studs is not practical for cold-formed steel (CFS) section due to its very thin form that is difficult to weld. A suitable bolt and nut type shear connector was proposed; experimental tests were carried out to evaluate its performance. This study also investigated the use of Self-Compacting Concrete (SCC) integrated with CFS by means of bolted shear connectors to develop composite action. Therefore, the objective of this paper was to investigate the performance of bolted shear connectors in SCC integrated with CFS section as composite beam system. Eighteen push-out test specimens comprising of M12, M14 and M16 sized bolted shear connector of grade 8.8 with longitudinal spacings of 150 mm, 250 mm and 300 mm centers were fabricated, cast and tested until failure. The experimental results showed that all bolted shear connectors possessed a good shear resistance capacity. Not much difference in strength capacity of specimens with M14 and M16 bolted shear connectors was found, perhaps due to CFS failure. Varying the size and longitudinal spacing of the bolted shear connectors revealed that the connectors' strength capacity changed slightly between M14 and M16 bolts specimens but, significantly for M12 bolt specimens.

Keywords

Bolted Shear Connectors, Self-Compacting Concrete, Cold-Formed Steel, Composite Beam, Longitudinal Spacing, Push-out Test.

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1 INTRODUCTION

Steel-concrete composite beam has been in use as a structural member in buildings and bridges for decades (Pavlovic et al., 2014). In composite beam, the shear connection between steel and concrete

slab is inherently significant, because it resists separation between the two components and it also enhances longitudinal shear transmission (Kim et al., 2011). The most common form of shear connection in composite beams system is the use of mechanical devices referred as shear connectors (Parakash et al., 2012). Conventional shear connection mechanism with headed studs shear connectors is the most widely used system. The stud connectors are welded to a steel flange and encased in concrete to provide the composite action (Pallares and Hajjar, 2010). The strength and ductility of shear connectors can be examined through push-out test, in which two concrete composite slabs are connected to both flanges of the steel beam.

A significant number of research works on push-out test using different types of studs with different types of steel sections have been published. For instance, a push-out test conducted by Xu, et al. (2012) used headed studs shear connectors on six specimens which were tested to failure. Smith and Couchman (2010) conducted push-out test on nine test specimens. They concluded that sufficient ductility of the shear connectors was achieved. Tahir, et al. (2009) also developed a pinned-connected shear stud system in composite beam using a hot rolled steel section which was subjected to push-out test. They concluded that the strength and ductility of the pinned-connected stud system could be improved by increasing the penetration depth of the pins and the material of the base plate used in which the studs were installed. Strength of stud connectors in lightweight and normal weight concrete was investigated by Ollgaard et al. (1971). The authors concluded that the stud connectors embedded in the normal-weight and light-weight concrete, were influenced by the concrete strength. Lam (2007) investigated the capacities of headed stud connectors in composite steel beams with hollowcore concrete slabs. He concluded that good shear resistance capacity was achieved in the hollowcore concrete slabs. Bamaga (2013) studied the possibility of using Cold-Formed Steel (CFS) with headed stud shear connectors welded on the flanges of CFS by push-out test. In the study, 16 mm diameter headed stud shear connector was used and the results attested better performance in strength and ductility when compared with other shear connectors used in the study. Push-out test using CFS in composite beam was also conducted by Bamaga and Tahir (2013). They used a variety of shear connection mechanisms (single bracket CFS shear connector, double bracket CFS shear connector and hot rolled steel (HRS) plate shear connector). The shear connectors showed high strength and ductility. A study on push-out test using CFS was investigated by Alenezi et al. (2013) and shear connectors used includes: bar angle bolt (10 mm diameter), self-drilling screws (6.3 mm diameter) and bolts (10 mm diameter). They found that the bolt (M10) showed a better performance as compared with other connectors used.

In composite construction bolted shear connectors are rarely used. However, the use of bolted shear connectors has a tremendous advantage because of their ease in dismantling and demountable processes (Pavlović, et al., 2013; Moynihan and Allwood, 2014), if reinstatement of the concrete slab is required for structural purpose. Pavlovic et al. (2013) conducted push-out test on four specimens with bolted shear connectors. Shear connectors comprised of M16 x 140 mm sized of grade 8.8 were used with HEB260 (S235) steel section. The bolt spacings were laterally and longitudinally kept at 100 mm in both directions. The failure observed was shear of the bolts connectors' shank. The authors concluded that bolted shear connectors achieved a good shear strength resistance. Post-installed behavior of bolted shear connectors was investigated by Kwon, et al. (2010). Their study focused on strengthening non-composite floor systems in existing bridge girders. They inferred that same techniques can

be used for strengthening non-composite floor systems in existing buildings with flat soffit solid slabs. The authors concluded that post-installed bolted shear connectors were feasible for the strengthening of the bridge girders. The performance and viability of bolted shear connectors as demountable connectors in composite system was investigated by Moynihan and Allwood (2014). Their findings pointed out that the bolted shear connectors possess good strength capacities.

It is evident in practical situations that there were important applications for concrete where compaction becomes problem or physically not feasible (Bartos, 2013). For instance, such applications could be found in construction works underwater, foundation works consisting of large diameter concrete piles, very deep concrete diaphragm walling etc. Therefore, a concrete mix required for such kind of applications had to be self-compacting (De Schutter *et al.,* 2008). Therefore, this study aimed to investigates the use of Self-Compacting Concrete (SCC) instead of conventional Normal Vibrated Concrete (NVC) as lack of inadequate compaction on the latter lead to creation of air voids. Thus, leading to the weakening of the concrete mass which could result to structural failure. But, the SCC does not require any external vibration for its compaction to be achieved, because that is achieved by its own self weight and homogeneity is maintained even in the presence of congested reinforcement.

A review of previous studies on bolted shear connectors is clearly indicative that the concept of using such type of connectors with CFS section and concrete is getting popular and its advantages have been proven by many researchers (Hanaor, 2000; Alenezi et al., 2013; Saggaff et al., 2015). However, the use of CFS with SCC and bolted shear connectors in a composite beam system is yet to be reported. Previous research reports delineated the following points.

i. None of the research works presented earlier reported the use of CFS with SCC and bolted shear connectors integrated as composite beam system.
ii. None of the studies show the performance of CFS-SCC and bolted shear connectors in composite construction in full scale testing.

Hence, this research study was undertaken to assess the shear capacity performance of bolted shear connectors used with CFS and SCC for composite beam system by means of push-out tests.

2 MATERIALS AND METHODS

2.1 Materials

The materials comprised of CFS lipped channel section of 250 mm x 75 mm x 18 mm and thickness of 2.3 mm; High tensile bolts of M12 x 75 mm, M14 x 75 mm and M16 x 75 mm of grade 8.8; welded wire fabric mesh A 142 of 6 mm diameter spaced 200 mm x 200 mm and ready-mixed SCC of 40 N/mm^2 with a slump flow ranged between 550 mm – 650 mm. All materials were tested in the Structures and Materials Laboratory of Universiti Teknologi Malaysia to establish the properties.

2.2 Material Properties

The CFS used was of 990 mm2 gross cross-sectional area, the bolted shear connectors consisted of M12, M14 and M16 with cross-sectional areas of 113.1 mm^2, 153.9 mm^2 and 201.1 mm^2 respectively. Whereas area of the fabric wire mesh was 28.3 mm^2.

2.2.1 Cold-formed Steel Section (CFS)

Properties of the CFS section were evaluated by coupon tensile test. The size of coupon and the procedure of the tests were conducted based on BS EN10002-1 (2001). The test was conducted so that the yield and the ultimate stresses of the steel section could be obtained, on a Universal Testing Machine INSTRON 600DX. Three specimens, each from the web and flange of the steel were taken and denoted as W1, W2, W3 and F1, F2, F3 respectively. The results are presented in Table 1. Figure 1 shows the test set-up and the tested specimens.

	Web			Flange			Avrg	f_u/f_y	CFS Thickness, t (mm)
	W1	W2	W3	F1	F2	F3			
f_y	490.88	478.33	480.62	488.93	490.91	494.93	487.43		
f_u	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).

Table 1: Result of CFS coupon tensile test.

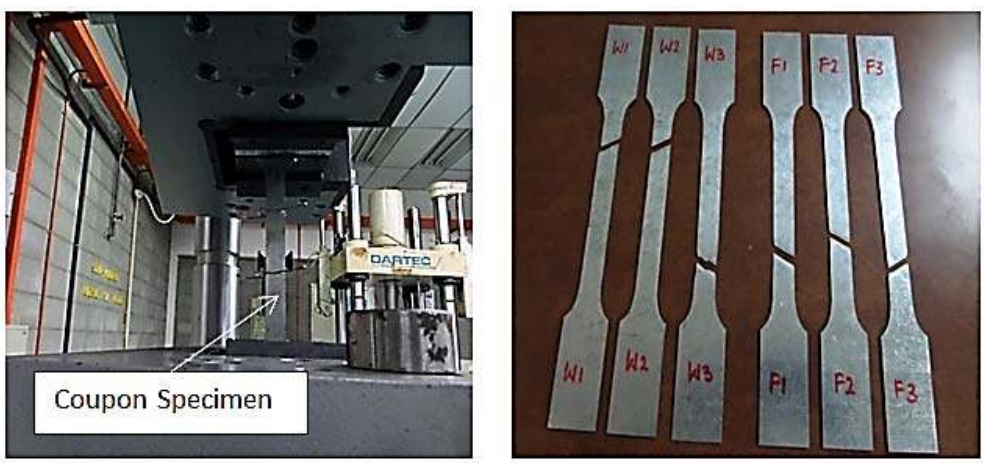


Figure 1: Test set-up and tested coupon specimens.

2.2.2 Bolted Shear Connectors

The types of shear connectors used in this study are high tensile bolts of grade 8.8 with diameters M12 x 75 mm, M14 x 75 mm and M16 x 75 mm with corresponding cross-sectional areas as provided in section 2.2. Properties of the bolted shear connectors were obtained by tensile test in accordance with BS EN ISO898-1 (1999). The test was conducted using a Universal Testing Machine INSTRON 600DX, in such a way that the yield strength, ultimate strength and the maximum failure load could be obtained. The result of the bolts tensile test is shown in Table 2. Figure 2 shows the test set-up and the tested specimens.

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)
M12-1	72	71	767	761	852	843	1.11	1.11
M12-2	70		755		834		1.10	
M14-1	95	96	715	720	813	807	1.14	1.12
M14-2	97		725		800		1.10	
M16-1	144	143	780	768	897	897	1.15	1.17
M16-2	141		755		896		1.19	

Table 2: Result of bolts tensile test.



Figure 2: Test set-up and tested bolt specimen.

2.2.3 Welded Wire Fabric Reinforcement

The wire mesh fabric used in this study was of 5.8 mm thick in actual diameter and spaced 200 mm x- 200 mm centers. The steel bar of the welded wire fabric was of grade 460 N/ mm2 and was tested for tensile in order to obtain the yield and the ultimate strengths in accordance with BS4449 (1997). Three sample specimens were randomly selected for the test, and the test was conducted using a Universal Testing Machine INSTRON 8801. Result is shown in Table 3. Figure 3 shows the test set-up and the tested specimens.

	Specimen			Average	fu/fy	Diameter (mm)
	B1	B2	B3			
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		

Table 3: Result of welded wire fabric reinforcement tensile test.



Figure 3: Test set-up and tested mesh specimen.

2.2.4 Self-Compacting Concrete (SCC)

The concrete used was a ready mixed Self-Compacting Concrete (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 EFNARC (2005) and BS EN206-9 (2010); while the hardened properties of the SCC, i.e. compressive strength (f_{cu}) and elastic modulus (E_c) were obtained by cube compression and cylinder tests respectively. The compression test was conducted in accordance with BS12390-3 (2009) and the cylinder test was conducted in accordance with BS1881-121 (1983). Both fresh and hardened properties test results are presented in Table 4. Figure 4 shows the test set-up and the tested specimens.

Fresh				
Slump flow (mm)	T500 (sec)	J-ring slump (mm)	T500j (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, fcc (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	

Table 4: Result of SCC fresh and hardened properties.



Figure 4: Cube and Cylinder test of concrete.

2.3 Push-out test

2.3.1 Test Specimens

Eighteen push-out test specimens comprised of M12, M14 and M16 sized bolted shear connectors of grade 8.8 with longitudinal spacing of 150 mm, 250 mm and 300 mm centers were fabricated, cast and tested until failure. The test specimens were made by orienting the CFS back-to-back using self-drilling screws of 5.8 mm diameter. Bolt holes of 13 mm, 15 mm and 17 mm diameters were drilled for M12 x 75 mm, M14 x 75 mm and M16 x 75 mm bolts respectively. The bolts with single nut and washer were installed through the bolt holes drilled on the top flanges and beneath the CFS and were arranged at designated longitudinal intervals of 150 mm, 250 mm and 300 mm center-to-center. Sixty millimeters height of the bolts were kept above the upper flanges of the CFS and spaced 75 mm centers laterally which were embedded in the concrete slabs as shown in Figure 5. The height, width and thickness of the concrete slabs were kept identical (800 mm x 600 mm x 75 mm) with a double layer of fabric wire mesh A142 of 6 mm diameter spaced at 200 mm centers installed to prevent creeping and shrinkage. Bonding at the interface between steel beam flanges, concrete and the formwork was prevented by lubrication as recommended by Eurocode 4 (BS EN1994-1-1, 2004). For each size of bolt connector, a push-out test specimen was cast for 150 mm, 250 mm and 300 mm longitudinal spacings. Each specimen had two slabs; namely slab A, and slab B. Slab A was cast three days before slab B where the concrete cover was maintained at 15 mm thickness. The slabs were cast in horizontal fashion to simulate the actual casting condition of composite beam in practice. For all the push-out specimens, recess of 80 mm was provided between the lower end of the steel beam and the concrete slab to allow for slip during testing. It is worth mentioning that no external vibration was applied to remove any air voids around the bolt connectors. This was achieved by the concrete's own self weight being a self-compacting. Three cubes of 150 mm were cast from each concrete batch to determine the cube compressive strength on the test day of the specimens. The push-out test specimen configuration is shown in Figure 6. Figure 7 shows the specimen before and after casting.

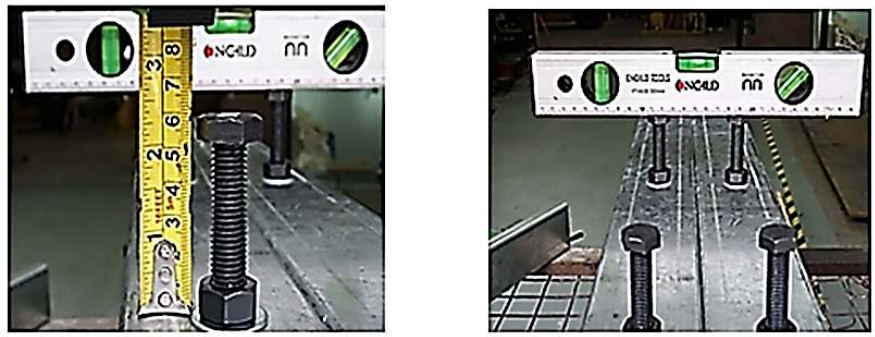


Figure 5: Installed bolt connectors for push-out test specimen.

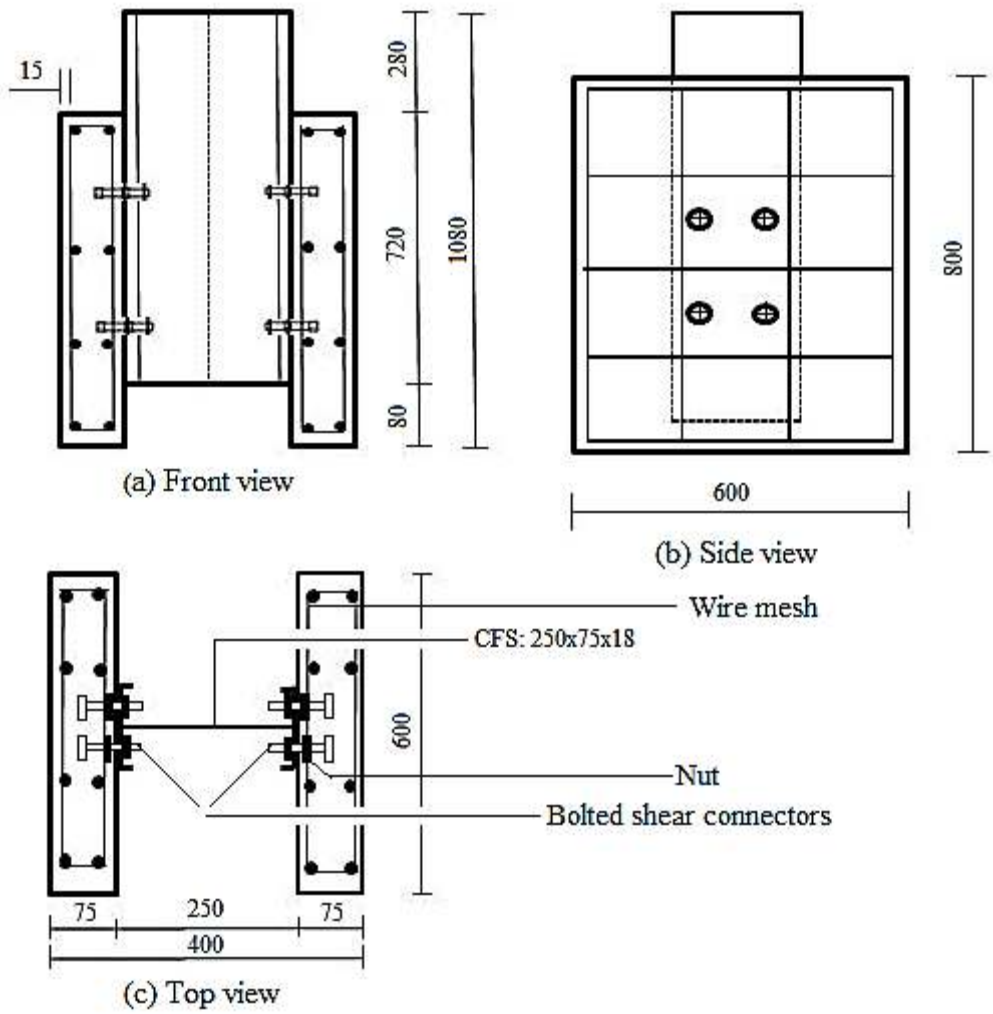


Figure 6: Push-out test configuration (all dimensions in mm).



Figure 7: Push-out test specimens before and after casting.

2.3.2. Test Set-up and Instrumentation

The test set-up is shown in Figure 8. Each push-out specimen was placed on 3 mm thick plywood and on 800 mm x 800 mm x 50 mm thick steel section, in order to properly place the concrete slabs. A restrain using steel angle section of 2000 mm length and 10 mm thickness was provided to hinge the movement of the test specimen while load was applied. Load using DARTEC Jack machine was applied on the upper part of the steel beam which has a load cell capacity of 2000 kN. Each push-out specimen was equipped with two linear variable displacement transducer (LVDT), each on either side of the steel beam web to measure slip in the vertical direction between the concrete slab and the steel beam. The LVDT's and the load cell were all connected to a data logger. Average reading of the LVDTs was used for the result 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 a drop of 20% from the maximum load of the specimen occurred or the specimen failed to resist any additional load as stated in Eurocode 4.

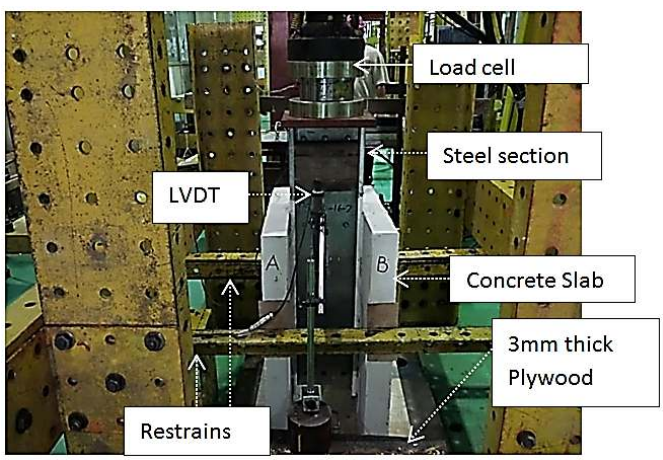


Figure 8: Push-out test arrangement.

3 EXPERIMENTAL RESULTS AND DISCUSSIONS

The push-out test result is presented in Table 5. Figure 9 shows the load-slip curves of tested specimens. From Table 5, the predicted values were obtained based on Equations (1) and (2) whichever is smaller as provided in Eurocode 4.

$$P_{Rd} = \frac{0.8 f_u \pi \frac{d^2}{4}}{\gamma_v} \tag{1}$$

Or

$$P_{Rd} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v} \tag{2}$$

Whichever is smaller from the equations;

With:

$$\alpha = 0.2\left(\frac{h_{sc}}{d} + 1\right) for 3 \le \frac{h_{sc}}{d} \le 4 \tag{3}$$

$$\alpha = 1.0 for \frac{h_{sc}}{d} \tag{4}$$

Where:

P_{Rd}: Design resistance of shear connector

f_u: Ultimate tensile strength of the stud (bolt) taken as the maximum value in this study

d : Shank diameter of the stud (bolt)

γ_v: Safety factor (taken as 1.25)

f_{ck}: Characteristic cylinder compressive strength

α: Dimensional coefficient

E_{cm}: Concrete modulus

h_{sc}: Height of shear connector

3.1 Failure Modes

The failure modes of the tested specimens can be categorized as;

(i) Shear connector failure either by deformation, pull-out and/or shear-off.
(ii) Concrete failure by crushing.
(iii) Steel failure by buckling of either the flange or web or both.
(iv) Bearing failure of steel section.

From the test conducted on all the push-out specimens, the failure modes exhibited by each category of specimens are discussed. For specimens with M12 bolt connectors, the failure modes were similar. The failure was connectors sheared–off as shown in Figure 9a & b. The test results

indicated that the load slip responses for specimens with M12 bolts seemed to exhibit a similar pattern as shown in Figure 10a (i) through (vi). It can be observed that a remarkable shear resistance and slip at ultimate loads were attained by the specimens. Specimens with M14 bolt manifested good shear resistance and good slip at ultimate loads, as shown in Figure 10b (i) through (vi). Same explanation is extended to the specimens with M16 bolted shear connectors. They also showed a remarkable increase in the shear resistance values as well as slip at ultimate loads (see Figure 10c (i) through (vi)). The failure mode of the test specimens with bolt diameters of M14 and M16 could be attributed to the failure of the steel section due to flange buckling. Perhaps high resistance to the applied load by the concrete slabs and bolted shear connectors could be responsible for it. The failure due to steel buckling occurred at the top shear connector of the specimens that was near the position of the load application. 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 which also resulted in the web buckling (see Figure 11a & b). Considering the load-slip curves, it can be seen that loading and unloading on some test specimens were done. This was to ensure that equilibrium was maintained from the loading set-up before the other specimen was tested in the category. From the load-slip curves of specimens with M14 bolt connector, it can be observed that a non-linear behavior was demonstrated by the specimens with exception of PS150-14-1 specimen. The specimen attained a slip at ultimate load of 5.56 mm, and it resulted from failure of CFS section that occurred which prevented further resistance of the applied load by the specimen. However, the specimen demonstrated good resistance capability of an ultimate load of 447.5 kN despite the CFS section failure. For specimens with M16 bolt connector, similar observation was made. The specimens' load-slip curves behaved non-linearly with good ultimate load resistance and large slip at ultimate load was demonstrated. The non-linearity resulted due to the failure of CFS that started at the advanced stage of loading before reaching the ultimate failure load. The same scenario was observed for PS250-16-1 specimen though, it showed very low slip due to the resistance of the load by the concrete and the shear connector up to the ultimate load level. No significant difference in the shear resistance capacities of M14 and M16 bolts specimens were observed due to the failure of the CFS. But, the shear resistance capacity was seen to be slightly increased. It is evident that the maximum design capacity of the specimens with M14 and M16 bolts connectors is based on the tested value obtained which failed due to CFS failure. However, the value obtained for M16 bolt specimen at 300 mm spacing had an increment of 17% shear capacity over a conventional headed shear studs size of 16 mm at 325 mm spacing used by Bamaga (2013). This could be attributed to the welding of the stud connector on the CFS. However, it shows that the shear connection mechanism employed in this study has the ability to increase the shear capacity of shear connectors. Again, referring to Table 5, it can be observed that the bolted shear connectors used attained an average characteristic slip capacity (δ_{uk}) of more than 6 mm recommended by Eurocode 4. A shear connector is classified as ductile connector when it achieved a characteristic slip capacity of 6 mm as per Eurocode 4 recommendations, otherwise it is classified as brittle connector. Shear connectors in this study of M12, M14 and M16 achieved a value of 7.5 mm, 8.4 mm and 8.6 mm respectively.

Specimen ID	f_{cc} at test date (N/mm^2)	Pu per connector (kN)	Pu Pre. Per connector (kN)	Pu exp./ Pu Pre. Per connector	□u (mm)	□uk (mm)	Avrg. □uk (mm)	Failure mode
PS300-12-1	36.7	47.38	35.20	1.35	11.40	10.26	7.5	Connector sheared-off
PS300-12-2	36.7	44.06	35.20	1.25	6.28	6.00		
PS250-12-1	37.8	44.13	35.90	1.23	9.03	8.13		
PS250-12-2	37.8	43.38	35.90	1.21	10.70	9.63		
PS150-12-1	37.0	43.94	35.40	1.24	7.16	6.44		
PS150-12-2	37.0	43.19	35.40	1.22	4.73	4.26		
PS300-14-1	35.7	59.19	47.00	1.26	8.29	7.46	8.4	Steel buck-ling
PS300-14-2	35.7	61.94	47.00	1.32	10.40	9.36		
PS250-14-1	35.5	57.13	46.80	1.22	9.62	8.66		
PS250-14-2	35.5	54.38	46.80	1.16	10.80	9.72		
PS150-14-1	35.6	55.94	46.90	1.19	5.56	5.00		
PS150-14-2	35.6	53.38	46.90	1.14	11.07	9.96		
PS300-16-1	33.0	59.35	55.20	1.08	13.62	12.26	8.6	Steel buck-ling
PS300-16-2	33.0	62.14	55.20	1.13	7.90	7.11		
PS250-16-1	37.2	57.25	59.90	0.96	2.95	2.66		
PS250-16-2	37.2	62.06	59.90	1.04	10.18	9.16		
PS150-16-1	37.4	56.94	60.20	0.95	11.99	10.79		
PS150-16-2	37.4	54.88	60.20	0.91	10.95	9.86		

PS300-16-1: Push-out specimen at 300 mm longitudinal spacing with M16 bolt connector number one; Pu: ultimate load; Pu Pre.: Predicted ultimate load; □uk: characteristics slip capacity.

Table 5: Push-out test result.



(a) Concrete surface (b) Steel surface

Figure 9: Failure mode of specimens with M12 bolts.

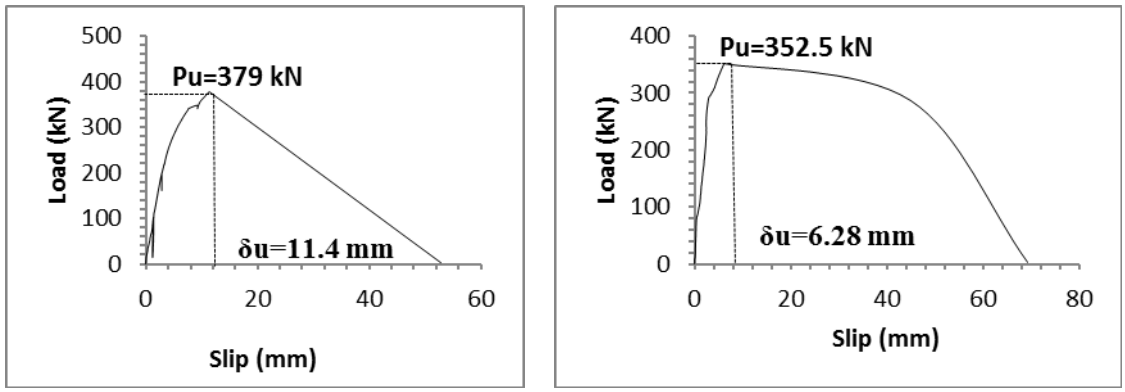


Figure 10a: (i) PS300-12-1 (ii) PS300-12-2.

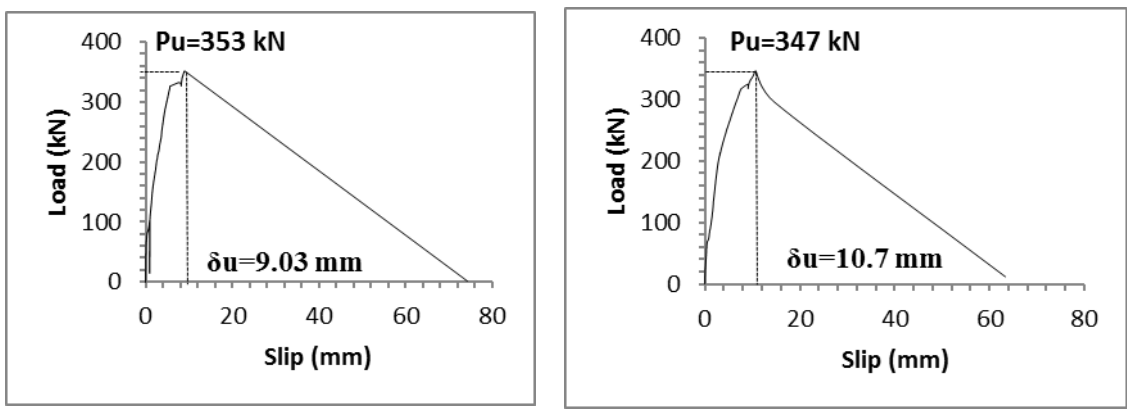


Figure 10a: (iii) PS250-12-1 (iv) PS250-12-2.

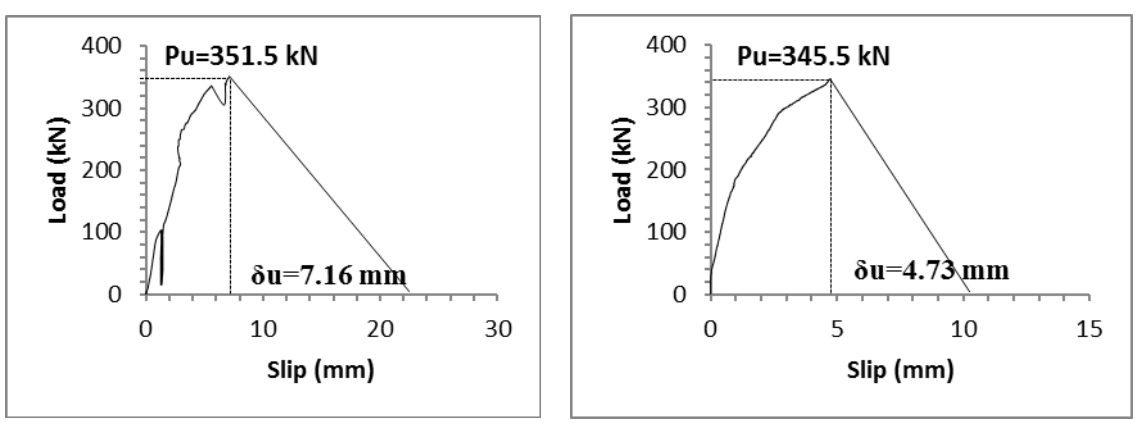


Figure 10a: (v) PS150-12-1 (vi) PS150-12-2.

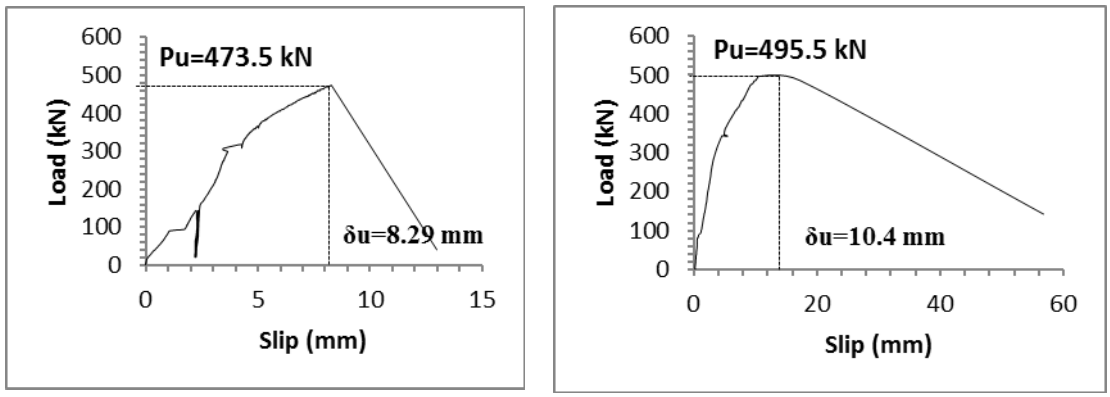


Figure 10b: (i) PS300-14-1 (ii) PS300-14-2.

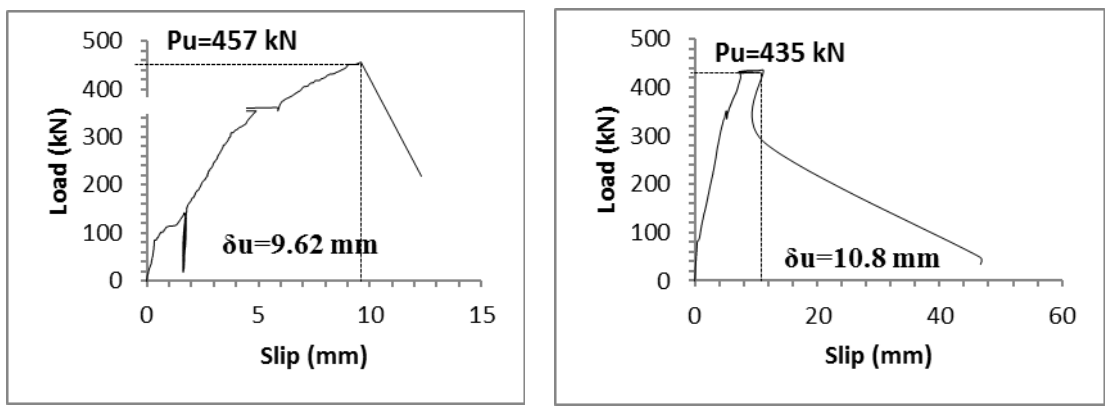


Figure 10b: (iii) PS250-14-1 (iv) PS250-14-2.

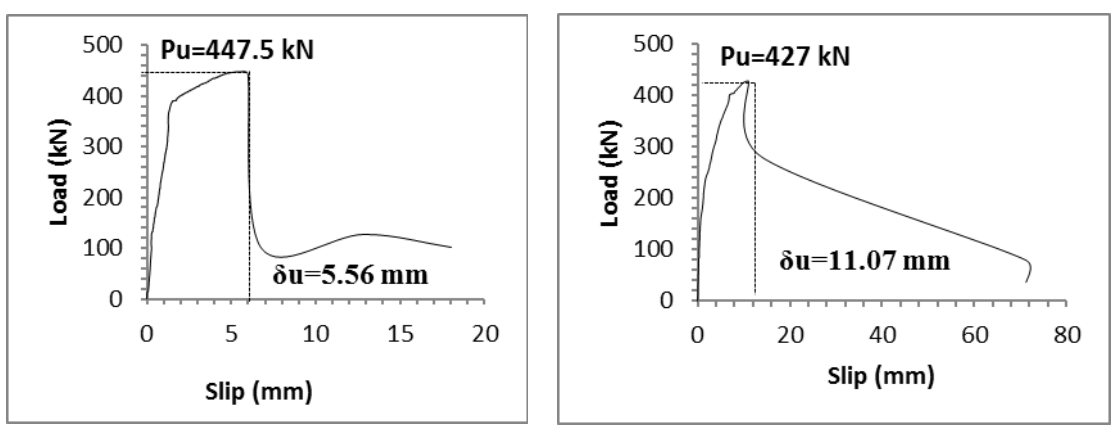


Figure 10b: (v) PS150-14-1 (vi) PS150-14-2.

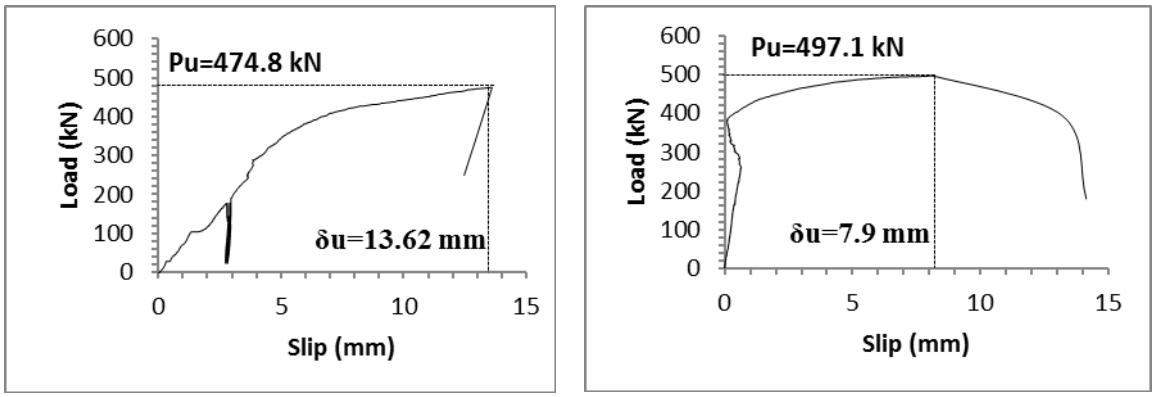


Figure 10c: (i) PS300-16-1 (ii) PS300-16-2.

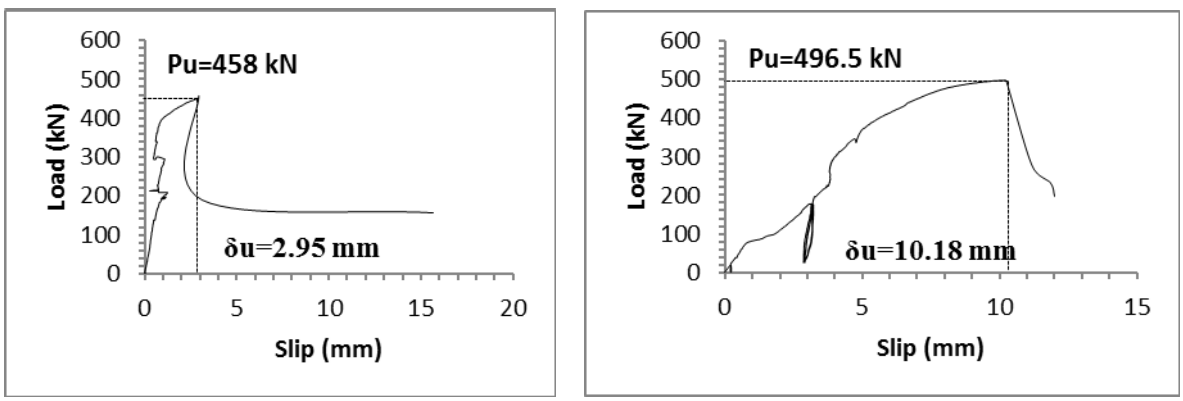


Figure 10c: (iii) PS250-16-1 (iv) PS250-16-2.

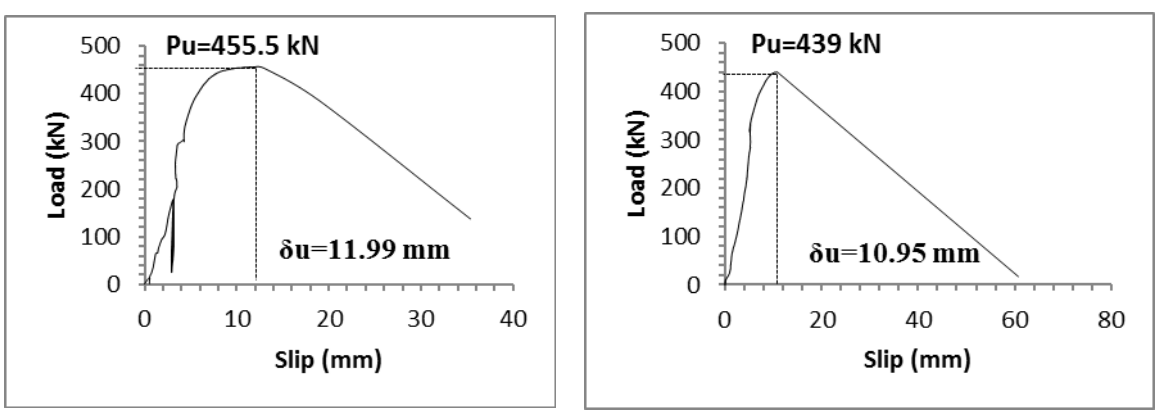
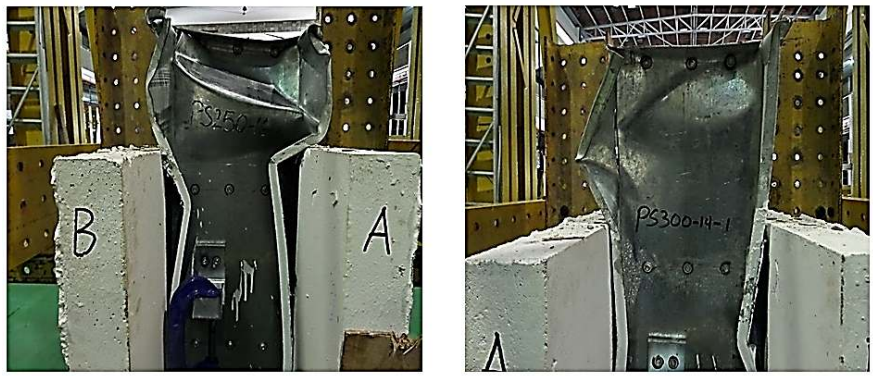


Figure 10c: (v) PS150-16-1 (vi) PS150-16-2.

Figure 10: Load-slip relationship of test specimens.



(a) Specimen with M16 bolt (b) Specimen with M14 bolt

Figure 11: Failure mode of specimens with M16 and M14 bolts connectors.

3.2 Effect of Shear Connector Size and Longitudinal Spacing on the Ultimate Load (Pu)

The effect of shear connector size and longitudinal spacing was investigated in order to establish their influence on shear strength capacity of the shear connectors. It was observed that an increase in shear strength capacity of 32.4% between M12 and M14; 32.9% between M12 and M16; 0.3% between M14 and M16 bolted shear connectors when used at a spacing of 300 mm. An increase in the strength capacity is observed between M12 and M14 of 27.4%; between M12 and M16 of 36.4%; between M14 and M16 of 7.0% bolts connectors respectively, when used at a spacing of 250 mm. Moreover again, an increase of 25.5% was also observed between M12 and M14; 28.4% between M12 and M16; 2.3% between M14 and M16 bolts respectively for a longitudinal spacing of 150 mm. The increase in the strength capacities observed with respect to the shear connector size is in agreement with the findings of (Hanaor, 2000; Irwan et al., 2011; Alenezi et al., 2013). The increase in shear strength capacity observed with respect to the longitudinal spacing (see Figure 12) shown to have influenced the ultimate load capacity. It also agreed well with the observation made by Bonilla Rocha et al. (2012). Effect of shear connector size on ultimate load capacity is presented in Table 6.

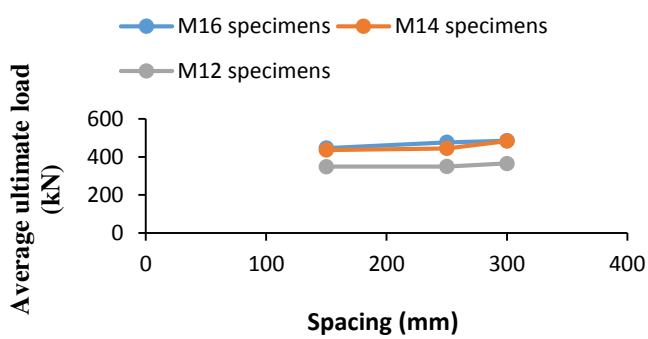


Figure 12: Effect of longitudinal spacing on ultimate load.

Specimen ID	Average Pu, exp. (kN)	Increase of Pu (%)	
PS300-12	365.8	-	-
PS300-14	484.5	PS12vsPS14 32.4	-
PS300-16	486	PS12vsPS16 32.9	PS14vsPS16 0.3
PS250-12	350	-	-
PS250-14	446	PS12vsPS14 27.4	-
PS250-16	477.3	PS12vsPS16 36.4	PS14vsPS16 7.0
PS150-12	348.5	-	-
PS150-14	437.3	PS12vsPS14 25.5	-
PS150-16	447.3	PS12vsPS16 28.4	PS14vsPS16 2.3

Table 6: Effect of shear connector size on ultimate load capacity.

4 CONCLUSIONS

Push-out test for specimens with M12, M14 and M16 bolts of grade 8.8 used as shear connectors were conducted. The effect of shear connectors' size and longitudinal spacings were investigated in order to establish their influence on the ultimate shear carrying capacity of the shear connectors. Following conclusions can be drawn:

- Among the proposed bolted shear connectors, M12 bolted shear connector performed well with the CFS of 2.3 mm thick.
- Not much difference in strength capacity was noted between M14 and M16 bolted shear connectors as the failure of the specimens occurred at the CFS.
- The shear connectors used can be classified as ductile connectors as they achieved an average characteristic slip of more than 6 mm as recommended by Eurocode 4.
- Size of shear connector had shown to have influenced the strength capacity of the shear connectors on the ultimate load capacity as it increased from M12 to M14 and to M16.
- Longitudinal spacing of shear connector also had shown to have influenced the ultimate load capacity of the specimens as it increased from 150mm to 250mm and to 300mm.
- The same configurations of test specimens considered in this study should be investigated with inclusion of stiffeners in the CFS web to strengthen the steel section for specimens with M14 and M16 bolt connectors.
- Due to the failure of the CFS for specimens with M14 and M16 shear connectors, it is recommended that a steel section with thickness above 2.3 mm should be used for further study.

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