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Influence of Shear Connector Size on Ultimate Strength in Composite Construction with Cold-Formed Steel Channel Lipped Section

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Abstract

In conventional composite construction for buildings and bridges, Hot-Rolled Steel (HRS) section is well known to be used. The composite action is usually achieved by using conventional headed stud shear connectors. However, for Cold-Formed Steel (CFS) section, the use of headed studs shear connectors is not feasible as the section is very thin and incapable to be weld. Therefore, a suitable shear connection system of bolt and nut is proposed in this study by varying the size of the bolted shear connectors. This paper presents the prospect of using a bolt and nut as shear connector that could be well-suited with CFS section when integrated in Self-Compacting Concrete (SCC). The experimental test comprised of push-out and was conducted to determine the strength and ductility of the proposed bolted shear connectors as used in composite construction. Eight push-out test specimens of bolted shear connector size consist of M12, M14 and M16 of grade 8.8 were fabricated and tested to failure. The experimental results show that the bolted shear connectors used possessed good shear resistance capacity. Influence of varying the size of the bolted shear connectors was investigated. The results show that the size of bolted shear connectors influenced the ultimate strength capacity of the shear connectors significantly.

Keywords: Shear connector size, Cold-formed steel lipped section, Composite construction, Strength capacity, Push-out test.

1. Introduction

The application of Cold-Formed Steel (CFS) sections started in the United States of America (USA) and Great Britain for quite decades, mainly for non-structural purposes. Although, the use of CFS is expanding in the present era of building constructions (Kibert, 2012). However, in the mid- 20th century, the structural use of CFS sections began especially for commercial and industrial building constructions (Hancock *et al.*, 2001; Riley and Cotgrave, 2014). The use of CFS sections as an alternative material for roof structure keep increasing due to the quality assurance of steel structures (Yu, 2000). Composite construction using CFS section and concrete began in Europe in the mid-1940s for used as floor system (Allen, 2006). The composite action is categorized by an interactive behaviour between structural steel and the concrete designed to utilize the best load resistance capability. For the concrete and steel to act compositely, a mechanical means of shear connection must be provided (Prakash *et al.*, 2012) and the most widely used shear connection system is welding of conventional headed studs on the flanges of the steel section to resist the longitudinal shear that will be transferred between concrete slab and the steel section (Kim *et al.*, 2001).

CFS sections are used with concrete as composite structural component as applied to Hot Rolled Steel (HRS) section, and the resulting performances were found to be encouraging. CFS structural members have several benefits over their conventional counterpart HRS, such as lightness, reduced thickness, high strength and stiffness, accurate detailing, non-shrinking and non-creeping at ambient temperature, non-combustibility, fast and easy erection, ease of fabrication and mass production and easy to install (Yu and LaBoube, 2010). However, with the advantages demonstrated by CFS for use in composite construction, welding of the conventional headed shear studs is not feasible (Hanaor, 2000). Therefore, the development of feasible shear connector to be compatible with the CFS section and the concrete as a composite entity is of paramount significance. The use of CFS sections in composite with concrete is still very few reported. However, a number of studies reported on the use of CFS section with concrete and different types of shear connectors in composite construction.

Irwan *et al.* (2008;2009;2011) 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.

Alenezi *et al.* (2015) conducted push-out test on 8 specimens that consisted of CFS lipped channel section assembled with ferrocment jacket. The study was conducted to improve the compressive capability of a CFS composite column by means of shear connection mechanism comprised of bolts (10 mm and 12 mm diameters), self-drilling screw (6.3 mm diameter x 12 mm long) and a bar angle bolt (10 mm diameter). The layers of wire mesh was varied in the study. The results showed that load carrying capacity of the composite column section increased as the size of the shear connector and number of the wire mesh layer were increased from 2 to 4 and to 6; with better results obtained with 12 mm diameter bolt connector. From their findings it can be categorically observed that the size and number of mesh layers influenced the ultimate load carrying capacity of the composite section. They concluded that experimental results agreed well when compared with simulated results by finite element modelling. In this paper, the use of CFS channel lipped section is reported with variety of bolted shear connectors to investigate the influence of shear connector size on ultimate strength capacity. Therefore, this paper highlighted on the use of high strength bolted shear connectors in composite construction with CFS, specifically the M14 bolt connector as it was not established.

2. Methodology

2.1. Push-Out Test

Eight push-out specimens were fabricated by orienting the CFS back-to-back to form an I-section beam, using self-drilling screws of 5.8 mm diameter. Bolted shear connectors of M12x75 mm, M14x75 mm and M16x75 mm of grade 8.8 were installed with nut and washer through bolt-holes of 13 mm, 15 mm and 17 mm diameters respectively. Bolt holes were drilled to accommodate the bolted shear connectors. The bolts were embedded in the concrete slabs made of Self-Compacting Concrete (SCC) placed at 60 mm height above the CFS flanges and were spaced laterally at 75 mm. Among the test specimens, six are consisted of 8 number bolted connectors and the other two comprised of 4 number bolted connectors. The slabs were cast in horizontal fashion as recommended by Eurocode 4 in order to simulate the actual casting situation of a composite beam in practice. The concrete slabs had a recess of 80 mm between bottom of the concrete slab and lower end of the steel beam to allow for slip during testing. The push-out test specimens were 800 mm x 600 mm x 75 mm. Fig. 1 shows the test specimen preparation.

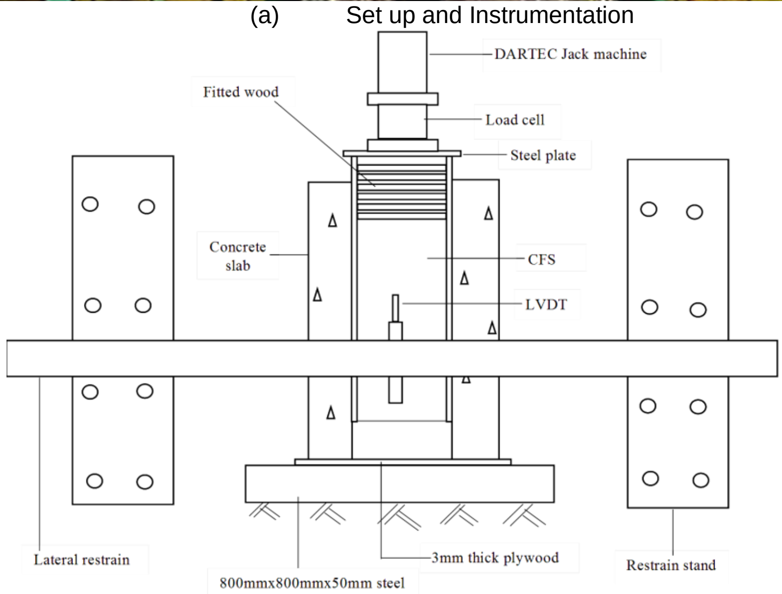
(i) (ii) (iii)

Figure-1. Preparation of test specimens [(i) Formwork, (ii) Casting, (iii) Finished samples

2.2. Test Set Up, Instrumentation and Procedure

The push-out test set-up is shown in Fig. 2, and all specimens were tested in the same manner. Each specimen was placed on 3mm thick plywood and on a steel section (800mm x 800mm x 50mm thick) to properly bed the concrete slabs. A restrain of an angle steel iron of 2000mm length and 10mm thick was provided to hinge the movement of the test sample when the load was applied from the Jack machine. The capacity of the Jack machine 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 occurred. 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.

(a) Set up and Instrumentation



(b) Schematic diagram

Figure-2. Push-out test arrangement

3. Results and Discussions

The push-out test result is presented in Table 1. Fig. 3 (a-h) shows the load-slip relationships of tested specimens. The failure modes experienced by the tested specimen can be categorized as:

i. Concrete cracking and crushing
ii. Steel flange buckling
iii. Connector sheared-off

The failure modes of specimens with 8 numbers M16 bolted shear connectors could be attributed to 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 the crushing of concrete and subsequent flange buckling of CFS section (Fig. 4 (a-d). The flange buckling of CFS occurred due to high stress from the applied load at the ultimate load level. Failure loads recorded per shear connector were 62.14 kN and 62.06 kN, for specimens PS8-30-16 and PS8-25-16 respectively. For specimen with 4 number M16 bolted shear connector (S4M16) the failure modes initiated with observance of minor cracks on the slab surface. At the ultimate load level, crushing of concrete underneath the slab was observed (Fig. 5 (a-c)). The CFS flange buckling failure was not observed. The recorded load per shear connector was 91.27 kN.

For specimens with 8 numbers M14 bolted shear connector, the failure observed in this category of test specimens were almost similar to that suffered by the specimens with M16 bolted shear connector. A part from cracks on the concrete slab surface, crushing of concrete was also noticed in the specimens. Failure due to CFS flange buckling was observed at the ultimate load level (Fig. 6 (a-d)). The recorded load at failure of the test specimens were 61.94 kN and 54.38 kN for specimens PS8-30-14 and PS8-25-14 respectively. But, for specimen with 4 numbers M14 bolted shear connector (S4M14), the failure observed was crushing of concrete slab with a recorded failure load at ultimate load level of 71.52 kN (Fig. 7). Failure modes of specimens with M12 bolted shear connector was attributed to connector sheared-off (Fig. 8). However, the specimens demonstrated a remarkable resistance capacity of 44.06 kN and 43.38 kN per shear connector for specimens PS8-30-12 and PS8-25-12 respectively. Neither cracks nor crushing of concrete slab were observed on the test specimens in this category. Influence of shear connector size was investigated and the results are shown in Table 2. From Table 2, it could be observed that PS8-30-16 is 0.3% and 41% higher in strength than PS8-30-14 and

PS8-30-12, respectively. Again, PS8-25-16 is 14% and 43% higher than PS8-25-14 and PS8-25-12 respectively. Between PS8-30-14 and PS8-30-12 an increase of 41% is observed, and between PS8-25-14 and PS8-25-12 an increase of 25% is noted. But, S4M16 specimen is 47% higher than PS8-30-16 and PS8-25-16 in strength capacity respectively. Furthermore, it is also 47% and 68% higher than PS8-30-14 and PS8-25-14 respectively. Also an increase of 107% is observed between S4M16 and PS8-30-12, and of 110% is observed between S4M16 and PS8-25-12 respectively. But, an increase of 28% is registered between S4M16 and S4M14 specimen.

An increase of 41% and 25% is observed between PS8-30-14 and PS8-30-12, and between PS8-25-14 and PS8-25-12 respectively. However, an increase in ultimate capacity of 15%, 15% and 62% is noted between S4M14 and PS30-16, S4M14 and PS8-30-14, and S4M14 and PS8-30-12 respectively. Again, an increase of 15%, 32% and 65% is registered between S4M14 and PS8-25-16, S4M14 and PS8-25-14, and S4M14 and PS8-25-12 respectively. Although, not much difference is observed between specimens with 8 numbers shear connectors of M16 and M14 in terms of the ultimate capacity. This is due to the buckling failure that occurred on the CFS flange. However, experimental results were compared with theoretical results based on Eurocode 4, and good agreement was achieved as shown in Table 1. Therefore, it could be deduced from the various increase in ultimate strength that shear connector size has significantly influenced the ultimate strength capacity of the specimens.

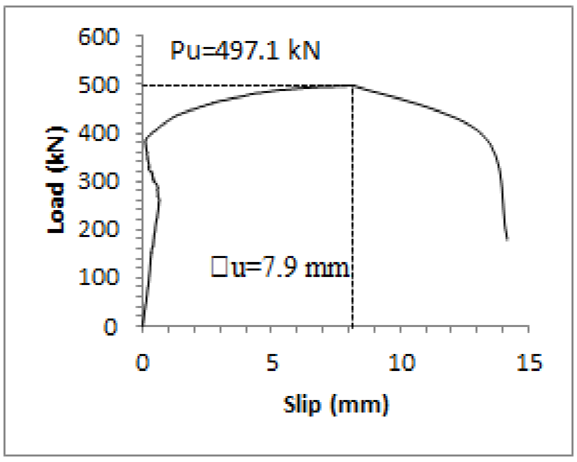
Table-1. Push-out test result

Specimen ID	$P_{u, exp.}$ per connector (kN)	$P_{u\ pre.}$ per connector (kN)	$P_{u\ exp.}/P_{u\ pre}$	δ_u (mm)	δ_{uk} (mm)
PS8-30-16	62.14	59.41	1.05	7.9	7.1
PS8-25-16	62.06	64.01	0.97	10.2	9.2
PS8-30-14	61.94	50.24	1.23	10.4	9.4
PS8-25-14	54.38	50.10	1.09	10.8	9.7
PS8-30-12	44.06	37.59	1.17	6.3	6.0
PS8-25-12	43.38	38.26	1.13	10.7	9.6
S4M16	91.27	64.18	1.42	24.0	21.6
S4M14	71.52	51.73	1.38	18.5	16.7

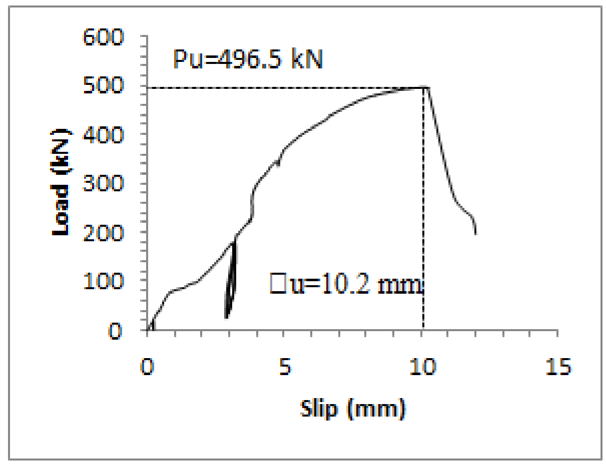
PS8-30-16: push-out specimen with number shear connector @ 300 mm spacing with M16 bolt; S4M14: specimen with 4 number of M14 bolt; Pu, exp.: ultimate load experimental; Pu, pre.: predicted load; δ_u: slip at ultimate load; δ_{uk}: characteristic slip capacity

Table-2. Influence of shear connector size on ultimate load capacity

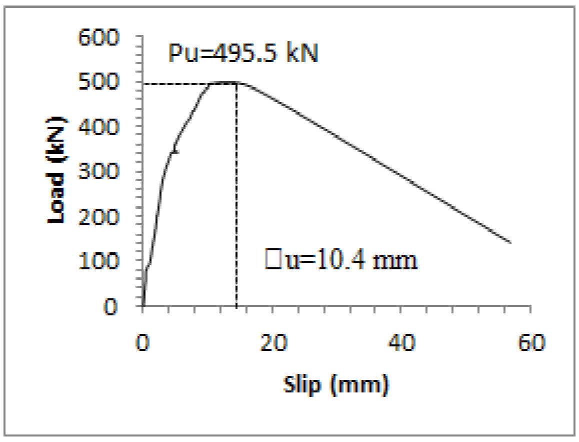
Specimen ID	Type of shear connector	Pu, exp. Per connector (kN)	Increase of Pu (%)	
PS8-30-12	M12	44.06	-	-
PS8-30-14	M14	61.94	PS8-30-12vsPS8-30-14 41.0	-
PS8-30-16	M16	62.14	PS8-30-12vsPS8-30-16 41.0	PS8-30-14vsPS8-30-16 0.3
PS8-25-12	M12	43.38	-	-
PS8-25-14	M14	54.38	PS8-25-12vsPS8-25-14 25.0	
PS8-25-16	M16	62.06	PS8-25-12vsPS8-25-16 43.0	PS8-25-14vsPS8-25-16 14.0
PS8-30-12	M12	44.06	-	-
PS8-30-14	M14	61.94	-	-
PS8-30-16	M16	62.14	-	-
S4M16	M16	91.27	PS8-30-12vsS4M16 107.0	-
-	-	-	PS8-30-14vsS4M16 47.0	PS8-30-16vsS4M16 47.0
PS8-25-12	M12	43.38	-	-
PS8-25-14	M14	54.38	-	-
PS8-25-16	M16	62.06	-	-
S4M16	M16	91.27	PS8-25-12vsS4M16 110.0	-
-	-	-	PS8-25-14vsS4M16 68.0	PS8-25-16vsS4M16 47.0
PS8-30-12	M12	44.06	-	-
PS8-30-14	M14	61.94	-	-
PS8-30-16	M16	62.14	-	-
S4M14	M14	71.52	PS8-30-12vsS4M14 62.0	-
-	-	-	PS8-30-14vsS4M14 15.0	PS8-30-16vsS4M14 15.0
PS8-25-12	M12	43.38	-	-
PS8-25-14	M14	54.38	-	-
PS8-25-16	M16	62.06	-	-
S4M14	M14	71.52	PS8-25-12vsS4M14 65.0	-
-	-	-	PS8-25-14vsS4M14 32.0	PS8-25-16vsS4M14 15.0
S4M14	M14	71.52	-	-
S4M16	M16	91.27	S4M14vsS4M16 28.0	-



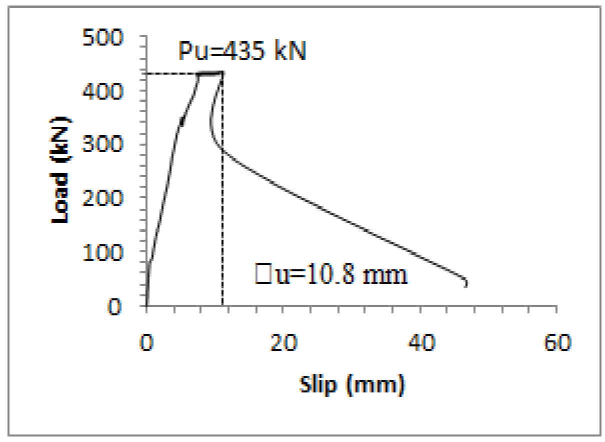
(a) PS8-30-16



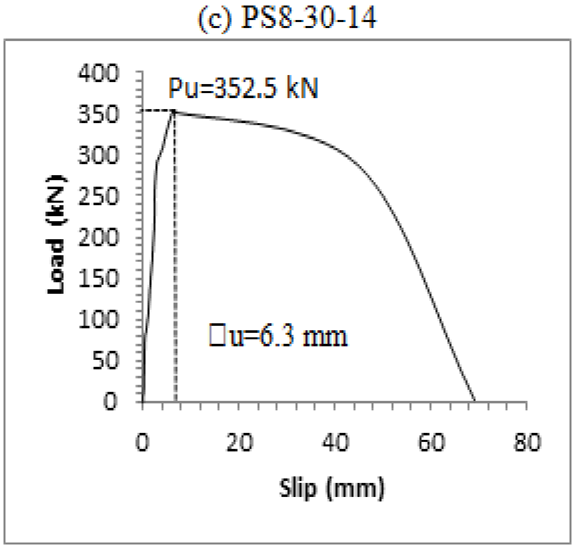
(b) PS8-25-16



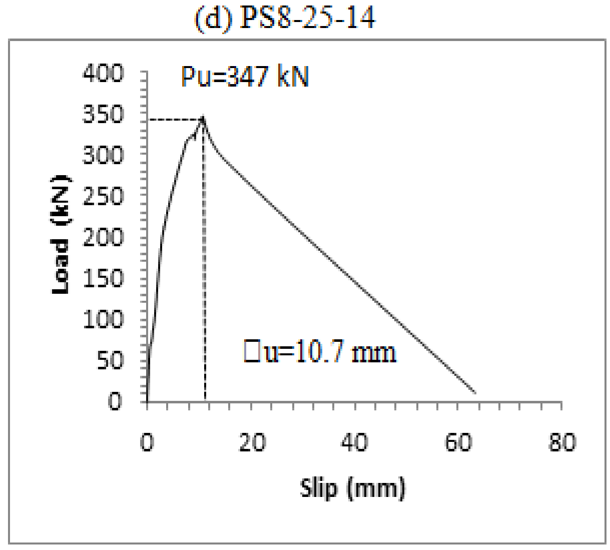
(c) PS8-30-14



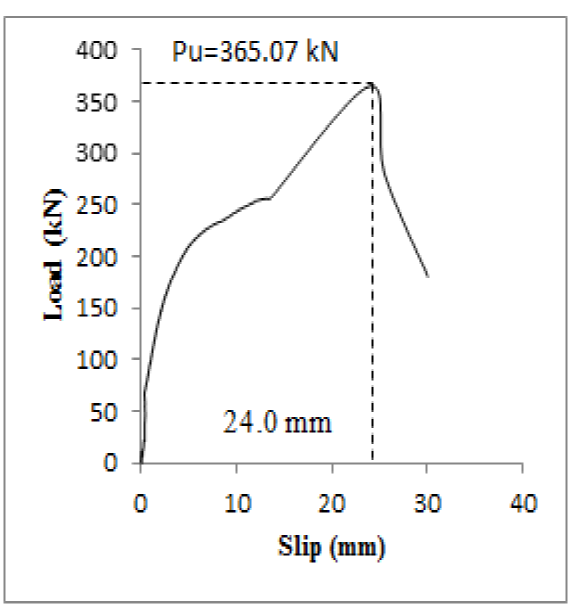
(d) PS8-25-14



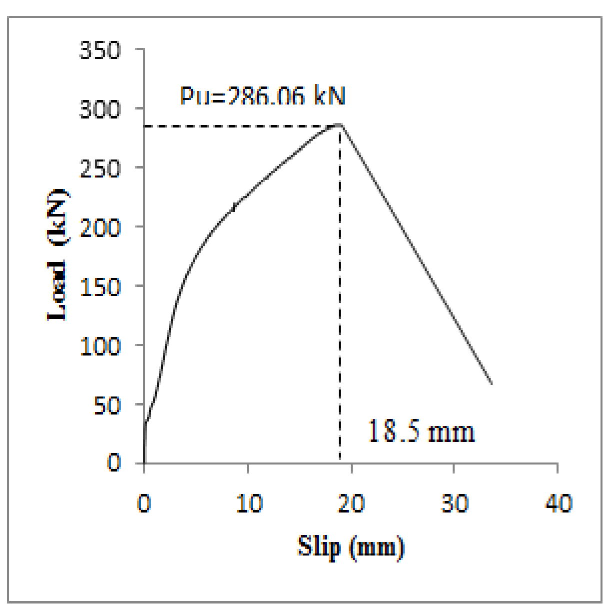
(e) PS8-30-12



(f) PS8-25-12



(g) S4M16



(h) S4M14

Figure-3. Load-Slip responses of push-out test specimens

(a) Surface cracks on slab

(b) Cracks underneath slab

(c) Crushing of concrete

(d) CFS buckling

Figure-4. Failure modes of specimens with 8 numbers M16 bolt

(a)

(b)

(c)

Figure-5. Failure modes of S4M16 specimen [(a) Minor cracks, (b) Concrete crushing, (c) Intact CFS section]

(a) Cracks on slab surface

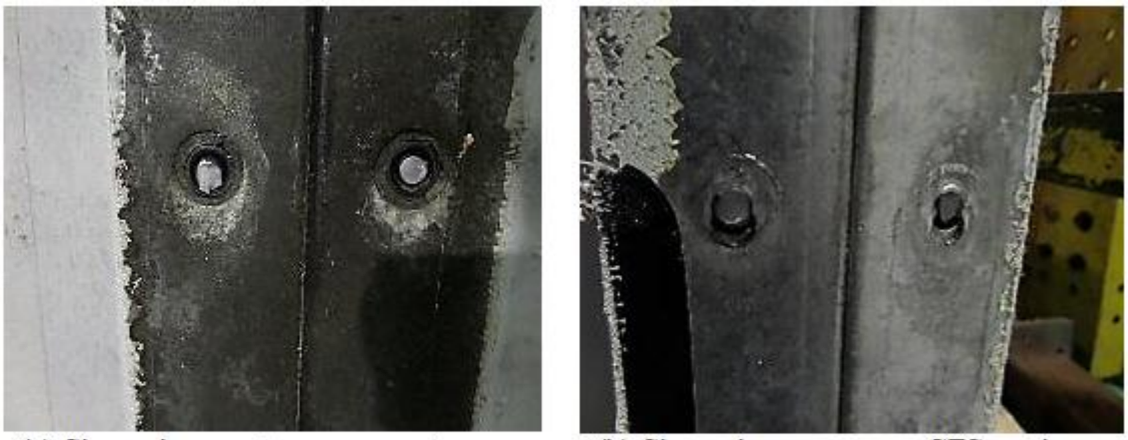
(b) Cracks underneath slab

(c) Crushing of concrete

(d) CFS flange buckling

Figure-6. Failure modes of specimens with 8 numbers M14 bolt

Figure-7. Failure mode of S4M14 test specimen



(a) Sheared connector on concrete (b) Sheared connector on CFS section

Figure-8. Failure mode of specimens with M12 bolt

4. Conclusion

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

1. Failure modes of test specimens with 8 number shear connectors were attributed to concrete cracks on slabs, crushing of concrete slabs underneath and then tailed by CFS flange buckling at ultimate load level.
2. Failure modes of specimens with 4 number shear connectors were crushing of concrete slabs from underneath, and no CFS flange buckling failure was observed.
3. Mode of failure exhibited by specimens with M12 bolted shear connectors was sheared-off of connector at the ultimate load level, neither concrete cracks nor crushing were observed.
4. All bolted shear connectors used demonstrated good ultimate strength resistance capacity.
5. All bolted shear connectors used could be classified as ductile connectors as they attained a characteristics slip capacity of more than the recommended value of 6 mm by Eurocode 4, with the exception of that of specimen PS8-30-12 which attained 6 mm as the recommended value.
6. Good agreement was achieved between experimental and theoretical results of specimens with 8 number shear connectors with a ratio range of 0.97 to 1.42.
7. It was observed that shear connector size remarkably influenced the ultimate load carrying capacity of the specimens.

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