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TECHNICAL PAPER

Shear connector systems for composite beam with cold-formed steel of lipped C-channel section

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Abstract
Headed stud is well-known shear connector, which is usually welded to the steel beam of the composite beams to connect the steel beam to the concrete slab to achieve the required composite action and limits the interface slip. Despite the efficiency of using the headed studs for Hot-Rolled Steel beams and steel plate girders, the application of headed studs may not be practical for very thin beam section such as cold-formed steel (CFS) sections due to the limitation of welding of the thin-walled steel sections. Thus, this study proposes innovative shear connector systems for composite cold-formed steel–concrete beams. The behaviour of the shear connector systems, including embedded CFS beam with/without holes, U-shaped steel bar embedded from top, embedded CFS beam and steel bar at the sides, embedded CFS beam with Bent-up tabs, and brackets at the top and bottom, were experimentally investigated through push-out tests in accordance with Eurocode 4 using 8 specimens of composite beam using different orientation. Experimental tests were aimed to determining the maximum resistance of the proposed shear connectors. The results indicated that the strength of a shear connector was very reliant on its deformation and the crushing of concrete. The ductile behaviour of most of the tested shear connectors satisfied the ductility requirements of Eurocode 4. In this study, it was concluded that the proposed shear connectors are suitable for the construction of composite beams with CFS.

Keywords Shear connector · Push-out test · Cold-formed steel · Composite beam

Introduction

Cold-formed steel (CFS) sections are lightweight materials that are manufactured by bending a flat steel sheet into a designated shape at room temperature [1–3]. They can sustain higher loads than the flat sheet itself and are useful in construction due to their superior structural performance [3, 4]. Channels of lipped C and Z sections are the most common CFS sections, with thicknesses ranging from 1.2 to 6.4 mm and depths ranging from 51 to 305 mm [3, 5]. Hot-rolled steel (HRS) and cold-formed steel (CFS) are two well-known and widely used material types in the steel building industry. However, HRS is more well-known as it is a preferred construction material for heavy structural uses in building construction, among contractors and engineers. In light-weight construction, cold-formed steel sections provide more advantages over hot-rolled steel sections, such as ease of handling, quick manufacture and transportation, high strength-to-weight ratio, cost-effective material, and 100% recyclable material [5, 6]. Because of these qualities, cold-formed steel is a cost-effective option for residential

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and commercial construction. However, because of their thin sections and, consequently, insufficient strength and stiffness, beams manufactured from CFS sections are still contentious when utilised as key structural components in construction. One solution is to create composite sections made up of CFS beams and concrete, and this has been a topic of research in recent years [7, 8]. When two or more materials are combined to behave as a singular body, composite construction is achieved [3, 9–11]. The shear connection between the steel section and the concrete slab is a critical component of a composite beam's structural integrity. Mechanical shear connectors are used to make this connection, which allows forces to be transferred from the concrete to the steel and vice versa, while also resisting vertical uplift forces at the steel–concrete contact [12, 13].

Shear connectors are typically welded to the top flange of a steel beam and embedded in the concrete slab to provide shear resistance. These connectors enable the two different construction materials that make up the composite part to function as if they are one solid piece of material. Shear connectors, such as headed studs, are commonly used for hot-rolled beams but are not suited to cold-formed steel beams because the steel sections are of a light gauge, making welding of the shear studs impractical. Although other types of connectors, such as self-drilling screws, are viable alternatives, the implementation of these connectors may be prohibitively expensive [6] and so, in order to accomplish this, additional research into novel types of shear connectors for CFS composite construction is required. As a result, researchers are currently creating and investigating various types of shear connectors for CFS composite sections [6, 7, 14–17]. As a result, the design and execution of composite beams with CFS sections could provide such devices that can produce the composite action, enhance the strength capacity, and meet the ductility demands of shear connection while maintaining the composite action.

Several academics have investigated the use of cold-formed steel in composite sections with concrete and their findings have been published. According to Nguyen [5, 18], the cost of manufacturing concrete beams can be reduced by replacing traditional reinforcing bars with a cold-formed steel lipped channel section of equal cross-sectional area. Nguyen revealed that composite sections are used as formwork to support fresh concrete, as well as reinforcements in composite-reinforced concrete beams.

Hossain investigated the behaviour of a concrete-filled cold-formed steel box [5, 19], after demonstrating that the buckling capacity of a steel plate restricts the strength capacity of beams with such sections. The investigation of the behaviour of a thin-walled composite-filled beam utilised four strength-enhancement devices. These were: a Cold-formed steel open box section (OS), an Open box section made of cold-formed steel with a welded extension (WE), a Cold-formed steel open box section with Welded Extension and Rod (WER) (in the same manner as Welded Extension (WE) but with additional rods of 6 and 10 mm diameter and 100 and 200 mm spacing), and a Cold-formed steel open box section with reinforced concrete (RC).

Four-point load tests revealed that Open-Section (OS) beams failed due to the lateral buckling of loose open-top flanges, separating steel from concrete. The use of strength enhancement devices in WE and WER beams increased the lateral buckling capacity of the beams, by strengthening the top flange of the steel section and preventing the separation of steel from concrete. It has been discovered that, by increasing the depth of the welded extension plate, the strength of WE beams increases. However, WE beams have more strength and ductility than OS beams. WER and RC beams had better strength capacity but lower ductility than WE beams. The reinforced concrete in the RC beams outperformed the infill concrete in the OS and WE beams. The welded rod in the WER beams added rigidity to the extension plate and top flange of the steel section, as well as extra concrete confinement. The generated analytical model matched the experimental results within the range of 1.02–1.15.

Hanaor investigated the composite action of a concrete slab and an I-beam (built from two steel pieces of cold-formed lipped channels) [7, 15] using two types of shear connectors. The first type consisted of a small length of a channel with the same dimensions as the beam section, with its web linked to the top flange of the beam in two distinct ways (self-drilling screws and welding) parallel to the beam length. The second form of shear connector was thought to be appropriate for some composite deck slabs. The deck was made up of large channel sections with holes in their webs that allowed the concrete to pass through. The specimen's collapse began with buckling of the steel section, followed by concrete crushing and welding failure. The screws themselves could be determined as the weak points in fastened shear connectors. For the second type of shear connections, when the applied load increases, the screws tend to either tilt and shear off at ultimate load or pull-out with deck buckling.

Lakkavalli and Liu [6] investigated the behaviour of cold-formed steel C-sections and concrete composite slab joists. They tested four shear transfer mechanisms: surface bond, bent-up tabs, pre-drilled holes, and self-drilling screws. On the surface of the concrete flange, four shear connector mechanisms were used. The first mechanism was the surface bond where shear and friction resistance between the concrete and flange surface of steel beams were transferred as a longitudinal shear force. The authors attribute the drop in ultimate strength to the overlapping of longitudinal shear stress fields formed by the knife action of bent-up tabs and screws, as the spacing reduced from 200 to 150 mm, resulting in weakening of the concrete between the connectors. A longitudinal crack in the concrete around the lower end of the concrete column caused failure at ultimate load in most push test specimens. With the crushing of the concrete in the bearing area and excessive yielding of the steel beam bottom flange, the failure occurred in full-scale beam tests. The self-drilling screws rotated significantly around their base, resulting in lower strength capability, compared to prefabricated bent-up tabs and pre-drilled holes. Self-drilled screws can bend more than prefabricated bent-up tabs and pre-drilled holes. However, none of the tested specimens satisfied Eurocode 4's ductility standards (at least 6 mm).

Irwan et al. [20–22] investigated the shear connection between CFS sections and concrete, using a shear transfer enhancement known as bent-up triangular tab shear transfer (BTTST), which involves forming a small triangular tab in the top flange of a cold-formed section and then bending it up to the necessary angle. Concrete's compressive strength and modulus of elasticity, as well as the strength of cold-formed sections, the size and angle of BTTST, and the thickness of cold-formed sections, were investigated. Specimens using the shear connection proposed by Lakkavalli and Liu [6] were made for comparison. BTTST resists longitudinal shear by its end bearing and shear area (which is the contact area between BTTST and the top flange). The capacity of BTTST rises with increasing bearing or shear area. As the thickness of the steel beam, angle, and dimensions of the BTTST increase, so does capacity. The failure mode of the BTTST shear connector push test specimens began by diagonal cracking in the concrete slab borders, followed by concrete crushing; the diagonal cracking was attributed to the BTTST resistance due to transverse orientation. In full-scale beam tests, specimens with thicker steel beams failed owing to concrete crushing, whereas specimens with thinner steel beams failed due to steel beam fracture. Push-out tests revealed that, as the concrete compressive strength increased, the strength capability of the BTTST shear connector increased.

Lawson et al. [14] used two cold-formed steel C-sections instead of hot-rolled steel I-beams to explore the behaviour of composite light steel beams. A new shear connector (a profiled strip shear connector) was designed, and powder-actuated pins were used to attach it to the flanges of the C-sections and the rib deck. The shear connectors were sufficiently ductile and failed when the pins were rotated and then pulled out.

Lawan [3] also researched high-strength bolted shear connections for cold-formed steel (CFS) and concrete composite beams. Four push-out specimens were made and tested for strength and ductility. Using self-drilling screws, the I-section beam was created. On the top flange of the CFS, 17 mm bolt holes were drilled, and M16 bolted shear connectors were inserted with a single nut and washer at the top. The dimensions of the push-out test specimens were 800 mm × 600 mm × 75 mm. Welded wire fabric was used to keep the concrete from creeping and shrinking. The results showed that the high-strength bolts can be used as ductile shear connectors to improve the ultimate load and moment capacity of composite beams, and that the shear connector can provide good composite action between the concrete slab and the steel section because no deformation was observed on the shear connector.

In another study, Saggaf proposed three different forms of shear connectors (bolt, bar angle, and self-drilling screw) for CFS composite beams. Using 10 push-out specimens, the proposed stud strength and deformation capacities were determined. The failure mode was due to crushing in the slab and the bolt with two nuts was found to have the best shear capacity [23].

Wehbe et al. carried out a study to determine the feasibility of attaching a concrete slab to a cold-formed steel (CFS) track to work compositely in a light-gauge steel construction. This concept significantly reduced the cost of a light-gauge steel construction by avoiding the use of heavy hot-rolled steel angles and hollow section steel tubes, which are typically welded to the top of CFS load-bearing walls to serve as load distribution members in the construction of CFS structures. The CFS channel section was used as tension reinforcement in this study and the composite action was given by a stand-off screw system [24]. The stiffness and strength capacity of CFS tracks increase when track thickness increases. In the elastic stage, the thicker CFS track showed better stiffness than the others. However, it produced unanticipated results in the nonlinear stage, with the composite section's ultimate strength capacity being lower than portions with thinner CFS tracks. This is because the thickest CFS track has a lower tensile strength than the other CFS tracks.

Specimens description and specifications

The use of shear connectors in the C-channel section is usually by bolt and nut or by a reinforcement bar embedded in the beam. The installation of these shear connectors is complex, time-consuming, and costly. Therefore, this study proposed an alternative method of providing shearing resistance by embedding a top flange of the steel beam of the C-channel in the concrete slab. The proposed embedded top flange, as a shear connector, could eliminate the typical shear connector and reduce the time and cost of construction. Eight experimental push tests were carried out to study the behaviour of several innovative shear connectors, including:

- An embedded CFS beam shear connector where the beam was oriented back-to-back without holes

- An embedded CFS beam with holes shear connector where the beam was oriented back-to-back
- An embedded CFS beam shear connector where the beam was oriented toe-to-toe
- An embedded CFS beam with holes shear connector where the beam was oriented toe-to-toe
- An embedded CFS beam with a bent-up tabs
- CFS beam with hot-rolled steel rebar at the top
- An embedded CFS beam with hot-rolled steel rebar at the sides
- Brackets at the top, fixed by screws

All of the specimens were prepared in accordance with Eurocode 4 [25]. The strength capacity, ductility, and failure mode of each specimen were determined and discussed. The proposed composite beam comprises two cold-formed lipped C-sections, a metal deck profiled concrete slab was used. A portion of the top beam embedded in the slab was used as a shear connector in five specimens and the other two specimens were not embedded but fabricated using deformed rebar with steel grade S500 and used as a shear connector with brackets at the top, fixed by self-drilling screws, respectively. The specimens were made with and without pre-drilled holes, to investigate the effectiveness and economy of circular holes. Two parallel circular-cut flanges with a dimension of Φ 30 mm were made, with a transverse distance of 100 mm and longitudinal spacing of 330 mm. The portion from the top beam was embedded in the slab, with a height of 70 mm through the profiled slab. The specimen with a bent-up tabs shear connector was made by making a rectangular cut in the top of a beam flange, with a height of 30 mm and width of 25 mm, then bent-up at 90°. Also, the embedded portion of the shear connector of the hot-rolled steel rebar in the slab was 70 mm and 130 embedded inside a CFS beam. The dimensions of the brackets shear connector were 50 mm in height and 50 mm long. The length, width and thickness of the concrete slab were kept uniform (660 mm × 800 mm × 125 mm) with a BRC wire-mesh A142 fabricated installation space of 200 mm × 200 mm, installed to prevent creep and shrinkage.

The dimensions and details of typical test specimens are presented in Figs. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11.

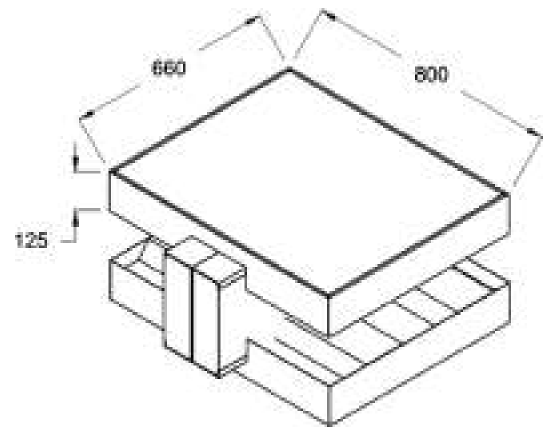


Fig. 1 Typical dimensions of push test specimens

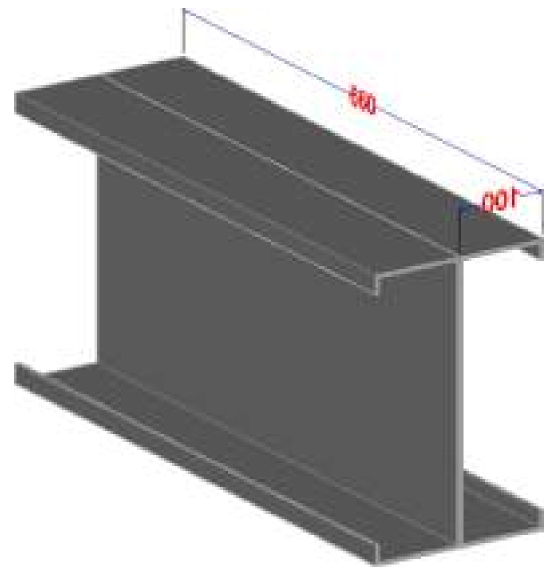


Fig. 2 Cold-formed steel lipped C channel beam fabrication for specimen back-to-back without holes

Push-out test

Description of the push-out test specimens

A total of eight push test specimens were fabricated based on the dimensions recommended by Eurocode 4 [25]. The top flange beam embedded in the slab was used as shear connectors in some of the samples and other types of shear connectors were also used. All of the push test specimens were labelled from 1 to 8 (see Table 1). The specimens were fabricated using cold-formed lipped channel steel sections, metal decks and concrete slabs. Two cold-formed steel lipped C channel Sections (660 mm in length) were oriented back-to-back, to form an I-section steel beam, and toe-to-toe, to form a hollow box section. Specimens numbered 2, 4, 5 and 6 were prefabricated with 30 mm diameter holes, acting as a shear connector. For specimens numbered 2 and 4, a steel rebar of grade 500 N/mm^2 was embedded inside a CFS beam. Four shear connectors were used in each slab in specimens 5, 6 and 8. This arrangement is recommended by Eurocode 4, to allow for redistribution of the load. Two concrete slabs (660 mm in length, 800 mm in width and 125 mm in thickness) were used with a perpendicular metal deck.

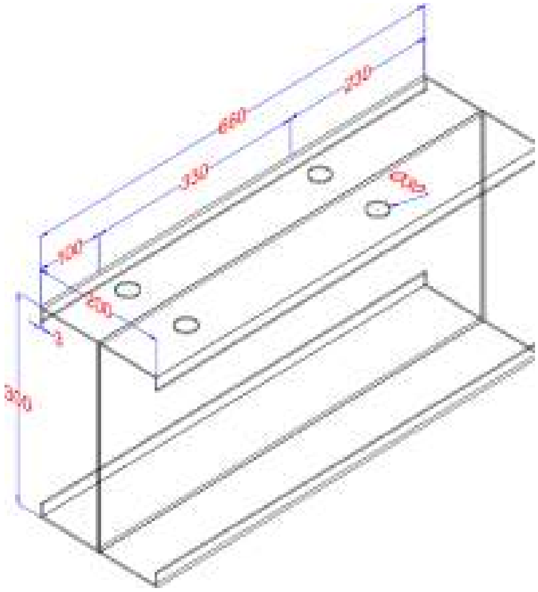


Fig. 3 Cold-formed steel lipped C channel beam fabrication for specimen back-to-back with holes

Fig. 4 Specimen No. 1: Beam oriented back-to-back without holes

One layer of prefabricated reinforcement mesh (A142) with a 5.7 mm diameter and 200 mm spacing (in both directions), was installed with a nominal concrete cover of 25 mm to prevent concrete surface cracks and longitudinal shear. The concrete slabs of the push test specimens were cast horizontally, as recommended by the standards (EN1994-1-1, 2004). Hence, the first concrete slabs (labelled 'A') was cast earlier than the concrete slabs labelled 'B'. For the first phase, concrete slab A (with eight push test specimens) was cast on the first day. Concrete slab B was then cast the next day for the

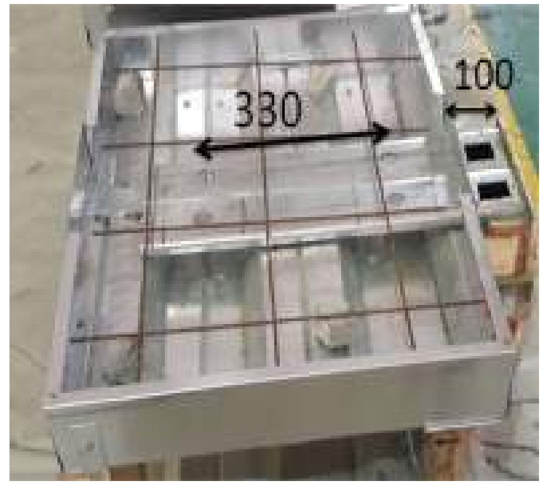


Fig. 5 Specimen No. 2: Beam oriented back-to-back with holes

Fig. 6 Specimen No. 3: Beam is oriented toe-to-toe without holes

Fig. 7 Specimen No. 4: Beam is oriented toe-to-toe with holes

Fig. 8 Specimen No. 5: Steel rebar embedded from the top

Fig. 9 Specimen No. 6: Steel rebar at the sides

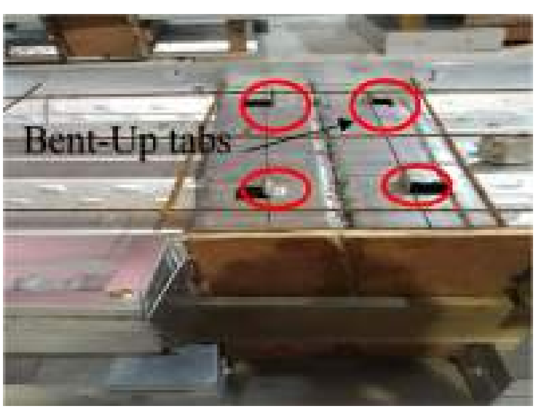


Fig. 10 Specimen No. 7: Bent-up tabs

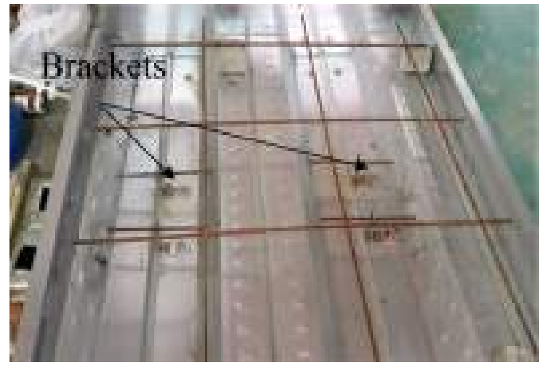


Fig. 11 Specimen No. 8: using brackets at the top

Table 1 Labelling of push test specimens

Specimen no.	The orientation of the CFS beam	Shear connector type
1	Back-to-back	Embedded CFS beam without holes
2	Back-to-back	Prefabricated holes
3	Toe-to-toe	Embedded CFS beam without holes
4	Toe-to-toe	Prefabricated holes
5	Toe-to-toe	Steel rebar embedded from the top
6	Toe-to-toe	Steel rebar at the sides
7	Toe-to-toe	Bent-up tabs
8	Back-to-back	Brackets at the top

same specimens. Six cubes and six cylinders were used to determine the compressive strength of concrete on the test date of the push test specimen and 28 days after. For easy visual inspection of the cracks, the push test specimens were painted white. A 100 mm recess between the bottom of the concrete slab and the lower end of the cold-formed steel beam was provided in the push test specimens to allow for slip during testing.

The longitudinal distance between the connectors (either by pre-drilled holes, bent-up tabs, steel rebar or brackets) was kept uniform at 330 mm and the transversal distance was kept at 100 mm. Four shear connectors were used in each slab in specimens 5, 6, 7 and 8.

Figure 12 shows the "embedded back-to-back without holes" specimen which used the embedded portion of the top flange of the steel beam as shear connection system. This system comes with two different configurations, the first configuration is 'without pre-drilled holes' and the second one is 'with pre-drilled holes. The specimen is oriented 'back-to-back' to form an I-beam and the top part of the steel beam was embedded in the concrete slab, which allows full contact along the whole width of the specimen. Figures 13, 14 show the second configuration which is "with pre-drilled holes".

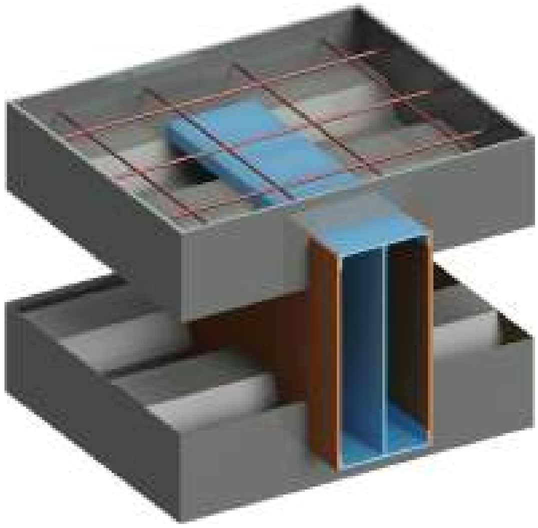


Fig. 12 Back-to-Back without holes

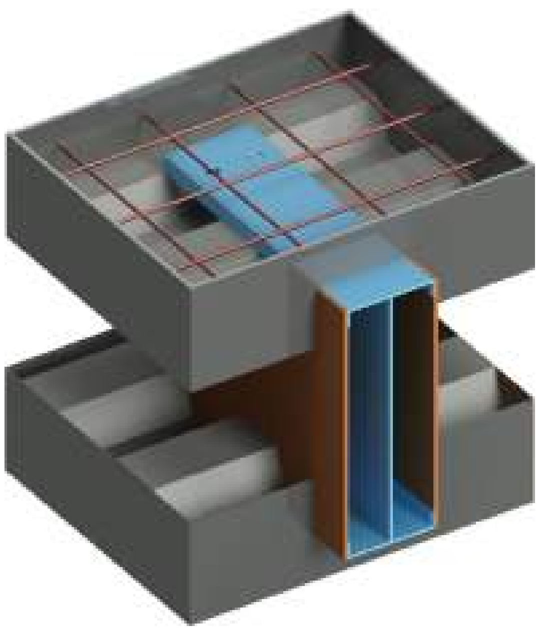


Fig. 13 Full view of embedded back-to-back with holes system

Figure 15 below shows the "embedded toe-to-toe without holes" specimen before and after concrete pouring, while Fig. 16 shows the "embedded toe-to-toe with holes" which used as shear connection mechanism.

In the "toe-to-toe with bent-up tabs" shear connection system, a small tab is formed in the top flange of the cold-formed section and then bent up to an angle of 90° to act as shear connector as shown in Fig. 17.

Figure 18 shows the "embedded toe-to-toe with steel rebar at the sides" system, in this shear mechanism, a

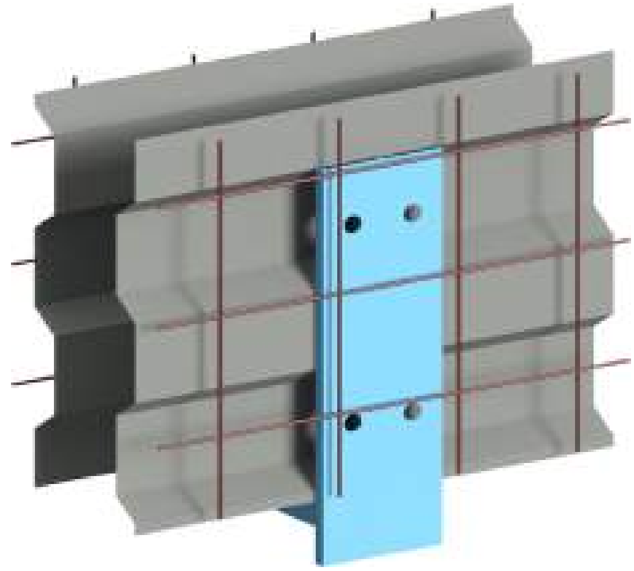


Fig. 14 Upside-down view of embedded back-to-back with holes system

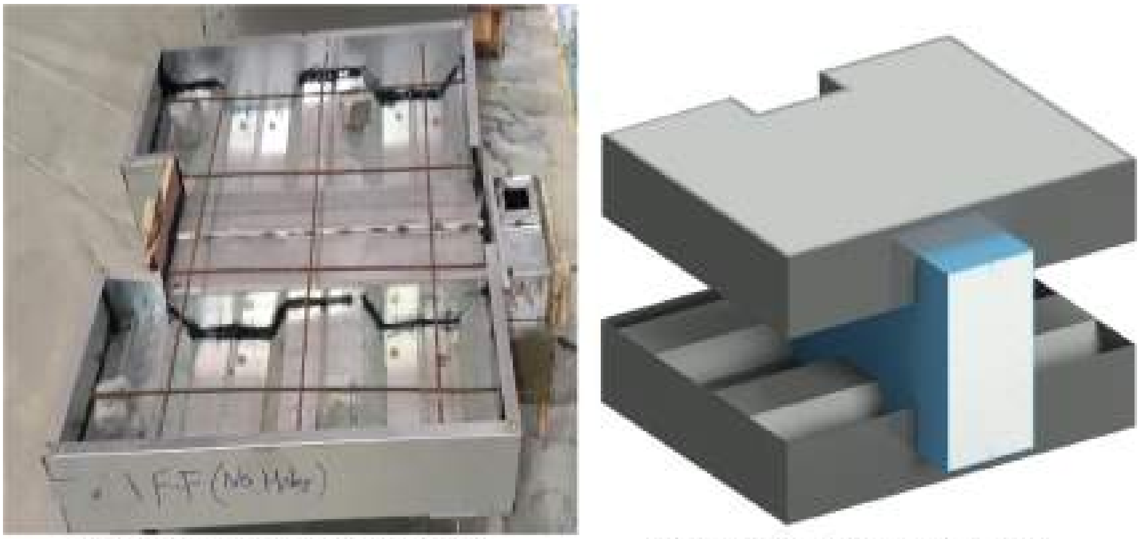
traditional steel rebar of 16 mm diameter was embedded in the concrete slab to work as a shear connector, Fig. 19 shows the type of the shear connector used in this system. The embedded length was 80 mm in the concrete slab, which was then bent at 90° and 50 mm height. Figure 20 shows the pre-drilled holes made in the steel beam to allow access of the steel rebar inside of the beam.

Figure 21 shows the "steel rebar embedded from the top" shear connection system. This shear mechanism resembles the previous system of steel rebar at the side of CFS, the only difference being that shear connectors are drilled and embedded from the top, Fig. 22 shows the steel rebar dimensions which act as shear connector in this system. The dimensions of the connectors are steel rebars of 16 mm diameter and the embedded length in the concrete slab is 70 mm and embedded 130 mm inside the CFS beam.

The last system investigated in this study was "Brackets fixed by screw" shear connection system, 50 × 50 × 2.6 mm bracket was used as a shear connector, Fig. 23 shows the brackets used in this shear system. The brackets were connected to the cold-formed steel beam by using screws of 30 mm height and 3 mm diameter, as shown in Fig. 24.

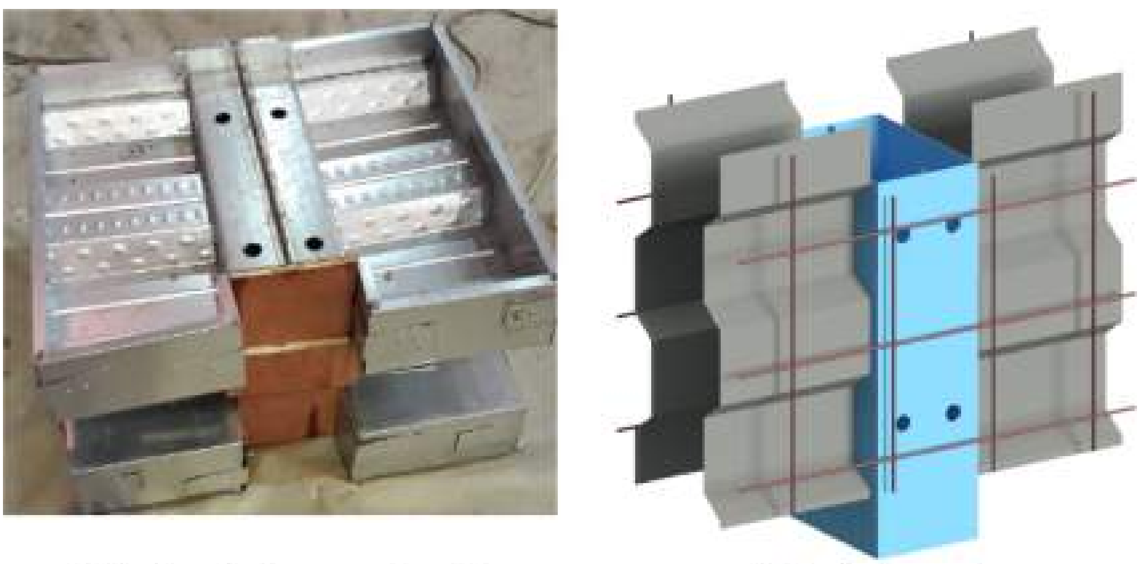
Test set-up and instrumentation

The test set-up is shown in Fig. 25. Each push-out specimen was placed on an 800 mm × 800 mm × 100 mm thick steel section, to properly place the concrete slabs. A DARTEC Jack machine was used to provide load to the bottom half of the steel beam, which had a load cell capacity of 2000 kN. Each push-out specimen was



a) Specimen before concrete casting b) Specimen after concrete casting

Fig. 15 Embedded toe-to-toe without holes as shear connection system.



a) Specimen before concrete casting b) Upside-down view

Fig. 16 "Toe-to-toe" with holes system.

equipped with four linear variable displacement transducers (LVDT), two on either side of the steel beam web and the other two on the upper surface of the concrete slabs, to measure slip in the vertical direction between the concrete slab and the steel beam. The LVDTs and the load cell were all connected to a data logger for data collection; the load was recorded in kN and displacement in mm. The testing procedure was carried out in accordance with Eurocode 4. The load was applied at a constant rate of up to 40% of the specimen's predicted failure load. The loading was cycled 25 times (loading and unloading) between 5 and 40% of the expected failure load, to ensure that the specimen was in an equilibrium stage. After the cyclic loading sequence, the load was then applied at a constant rate 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. Each specimen took about 45 min until it completely failed. At each load increment, values of the slip between the steel beam and the

Fig. 17 Bent-up tabs shear connector

Fig. 18 Embedded toe-to-toe with steel rebar at the sides

concrete were recorded. The average displacement value was plotted against the load (Figs. 26, 27, 28, 29, 30).

Material properties

The materials used in this study were:

- Cold-formed lipped channel steel sections with a depth of 300 mm, width of 100 mm, lipped depth of 20 mm and thickness of 3 mm;
- An "AJIYA PEVA 50" metal deck of profiled steel sheet, with 1 mm nominal thickness and minimum yield strength of 550 MPa;

Fig. 19 Steel rebar used as shear connector

Fig. 20 Pre-drilled holes at sides to allow for shear connector

- Steel rebar shear connector of 500 N/mm^2 grade;
- Welded wire fabric mesh A142, 5.7 mm thick and spaced 200 mm × 200 mm; and
- Concrete grade of 25 N/mm^2.

The materials were tested to obtain their actual strength by tensile, compression and modulus of elasticity tests. The results of the concrete testing are shown in Table 2. After the concrete was cast for push-out specimens, concrete cylinders (100 mm diameter × 200 mm length) were prepared during each pouring. These concrete cylinders were tested for compressive strength on the same day as the push-out tests. Table 2 shows the results of the compressive strength tests. The tensile strength and the elastic modulus of the concrete were obtained according to the procedure proposed

Fig. 21 Steel rebar embedded from the top

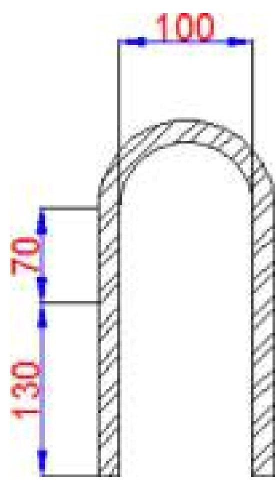


Fig. 22 Shear connector used in the system

by Eurocode 2. The properties of the steel materials used in this testing program were obtained from the tensile coupon test, in accordance with BS EN ISO 6892-1:2009 (2009).

Results

The push-out test results and the failure modes are presented in Table 3. Figures 31, 32, 33, 34, 35, 36, 37, 38 show the load-slip relationships of the tested specimens; the specimens showed good resistance capability based on the applied load. The specimens started showing cracks at fluctuating levels, depending on the type of shear connector, before attaining their ultimate load levels of about 490 kN (specimen No. 2). The failure mode obtained from all of the push test specimens developed as sheared-off failure or bracket-buckling, which could be attributed to the concrete cracking, followed by longitudinal concrete

Fig. 23 Brackets used as shear connectors

Fig. 24 Brackets “fixed by screw” shear connection system

crushing along the whole length of the specimen. The cracks were observed in slab B, followed by another crack in slab A. This is due to the lower compressive strength of concrete slab B, which was cast after concrete slab A. In the advanced stage of loading, longitudinal cracks were observed along the centre of the slabs. The reason for these longitudinal cracks could be due to the longitudinal shear force transferring from the concrete slab to the steel

Fig. 25 Set-up of push-out test

Fig. 26 Cold-formed lipped channel steel sections with depth of 300 mm

beam through shear connectors. The transverse separation between concrete slabs and steel beams was also observed in some specimens, such as those using steel rebar embedded from the top shear connectors. It is worth mentioning that no sign of buckling was observed on the flange or web of steel beam during the test. The specimens were dismantled to investigate and visualise the cold-formed steel and shear connector status and it was found that no deformation on the part of CFS and shear connector embedded in the concrete slab was observed.

Fig. 27 Metal deck used in the study of 1 mm thick

Fig. 28 Wire fabric mesh A142

Discussion

Effects of embedded 'back-to-back' as a shear connector system

When cold-formed steel is embedded and oriented 'back-to-back' in the concrete slab, it's allowing full contact along the whole width of the specimen, which provides a very strong bonding between the concrete slab and CFS. Thus, the embedded portion of the CFS resisted the applied load along the whole length of the specimen, resulting in high loading capacity compared to other configurations. At the advanced stage of the loading, the longitudinal and

a) b)

Fig. 29 Specimens after casting

a) b)

Fig. 30 Specimens curing

Table 2 Concrete properties

Curing time	Cylinder compressive strength, f_{ck} MPa	Cube compressive strength, f_{cu} MPa	Splitting tensile stress, f_{ct} MPa	Modulus of elasticity, E MPa
Concrete @ 28 days	22.00	28.70	2.20	27230
Concrete @ test day	22.13	27.10	2.30	27230

Table 3 Push-out test results

Specimen no.	Total P_u (kN)	P_u per connector (kN)	Δu (mm)	First crack @ (kN)	Failure mode
1	412.60	No shear connectors used	5.02	125.00	Transverse and longitudinal cracks followed by concrete crushing
2	486.60	60.83	9.10	185.00	Transverse and longitudinal cracks followed by concrete crushing
3	56.00	No shear connectors used	9.78	14.00	Separation of the slab from the beam due to bracket–buckling beneath the slab (non-composite construction behaviour)
4	113.00	14.13	6.20	67.00	Bracket–buckling beneath the slab then longitudinal cracks in the slab followed by concrete crushing
5	213.00	26.7.00	6.30	125.00	Transverse and longitudinal cracks and concrete crushing
6	256.00	32.00	2.20	155.00	Transverse and longitudinal cracks followed by concrete crushing
7	160.00	20.00	6.40	110.00	Transverse and longitudinal cracks followed by concrete crushing
8	64.50	8.07	3.21	–	Screws shearing-off

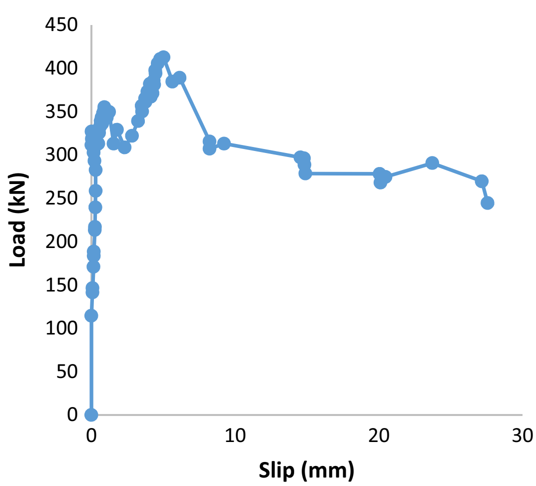


Fig. 31 Load-Slip curve for specimen 1

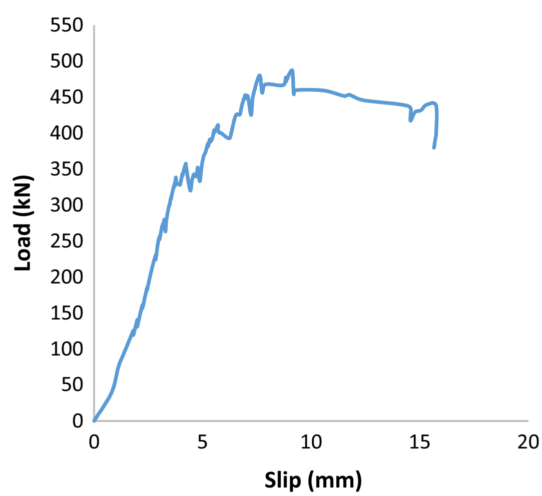


Fig. 32 Load-Slip curve for specimen 2

transverse cracks were developed as shown in Fig. 39, followed by concrete crushing. No deformation developed in the CFS section. However, when the concrete was crushed, the bonding between the concrete slab and CFS was lost, which shows that the higher concrete grade has resulted in higher load capacity. The concrete grade in this configuration was found to govern the design load, not the CFS deformation. The embedded back-to-back CFS beam with holes achieved the highest loading capacity (approximately 490 kN) compared to other specimen configurations, followed by the embedded back-to-back CFS specimen without holes (which achieved 412.6 kN). This could be due to the surface area of CFS exposed to a concrete slab larger than back-to-back CFS beam without holes, which shows that the pre-drilled holes effectively work as a shear connector. The pre-drilled holes with the embedded back-to-back beam could provide an alternative shear connector, which is easy to fabricate, time-saving and easy to install, compared to a traditional shear connector.

Effects of embedded 'toe-to-toe without holes' as a shear connector system

Specimens of CFS embedded 'toe-to-toe' without holes in the concrete slab, allow full contact along the whole width of the specimens. The composite interaction between the CFS beam and profiled metal decking slab is developed due to

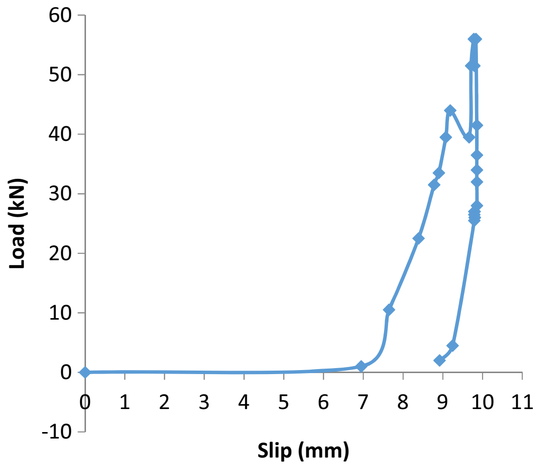


Fig. 33 Load-Slip curve for specimen 3

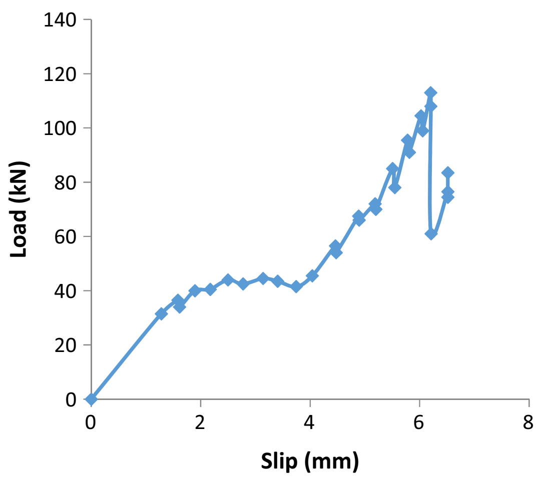


Fig. 34 Load-Slip curve for specimen 4

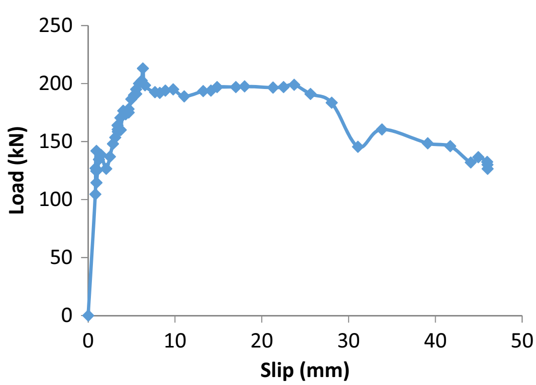


Fig. 35 Load-Slip curve for specimen 5

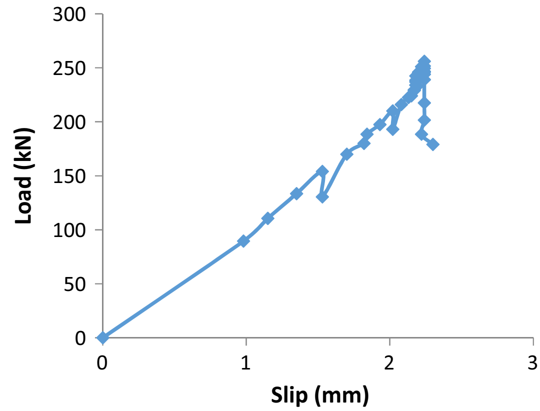


Fig. 36 Load-Slip curve for specimen 6

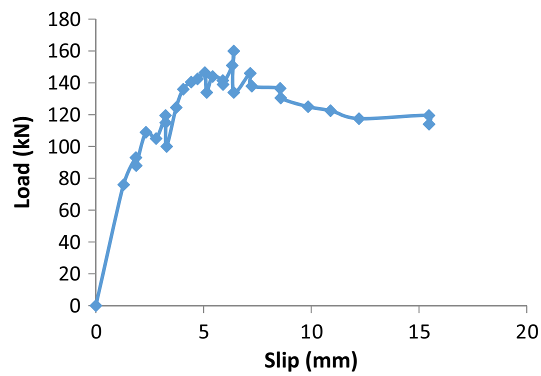


Fig. 37 Load-Slip curve for specimen 7

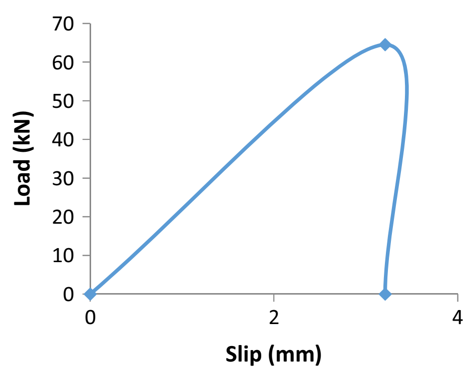


Fig. 38 Load-Slip curve for specimen 8

the abrasive surface of the beam and the corrugated profile of the decking. There is no contact between the concrete inside the CFS section, the concrete from the slab and the embedded portion of the CFS section; it did not contribute to the resistance of the applied load along the whole length

a) b)

Fig. 39 Longitudinal and transverse cracks of B-B specimens

Fig. 40 Buckling of the brackets holding the concrete slab

of the specimen. The brackets that hold the slab resisted the applied load and resulted in the buckling of the bracket beneath the slab as shown in Fig. 40. At an advanced stage of loading, separation of the slab from the beam is noted, without any development of longitudinal or transverse cracks as shown in Fig. 41. This shows that there is no composite action between the CFS and the slab because there is no interaction between them.

Fig. 41 Failure mode of the toe-to-toe without holes system

Effects of embedded 'toe-to-toe with holes' as a shear connector system

When cold-formed steel beam embedded and oriented as 'toe-to-toe' with pre-drilled holes in the concrete slab allow full contact along the whole width of the CFS section. During the test, the specimen showed interaction between the concrete inside the CFS section and the concrete from the slab because of the pre-drilled holes. The embedded portion

a) b)

Fig. 42 Longitudinal cracks along the slab

a) b)

Fig. 43 Failure mode of toe-to-toe with holes system

of the CFS beam resisted the applied load along the whole length of the beam section, which proves that the pre-drilled holes work effectively as a shear connector. The brackets that hold the slab resisted the applied load at the initial stage of loading where the brackets beneath the slab buckled. At the advanced stage of the loading, longitudinal cracks along the slab were developed as shown in Fig. 42a, b followed by crushing of the concrete slab. This failure could be due to composite interaction between the slab and the beam due to pre-drilled holes, as shown in Fig. 43a, b.

a) b)

Fig. 44 Crack pattern of bent-up tabs

a) Transverse separation b) Transverse slip

Fig. 45 Transverse separation after test of bent-up tabs specimen. **a** Transverse separation **b** Transverse slip

Effects of bent-up tabs as a shear connector system

The prefabricated bent-up tabs shear connectors developed good performance until failure at 160 kN. The prefabricated bent-up tabs resisted the longitudinal shear force in its embedded portion in the concrete slab as well as the contact area between the concrete slab and the concrete inside the CFS beam. The failure mode of the push test specimen with this system of shear connector was initiated by transverse and longitudinal cracking at the concrete slab surface, followed by concrete crushing, as shown in

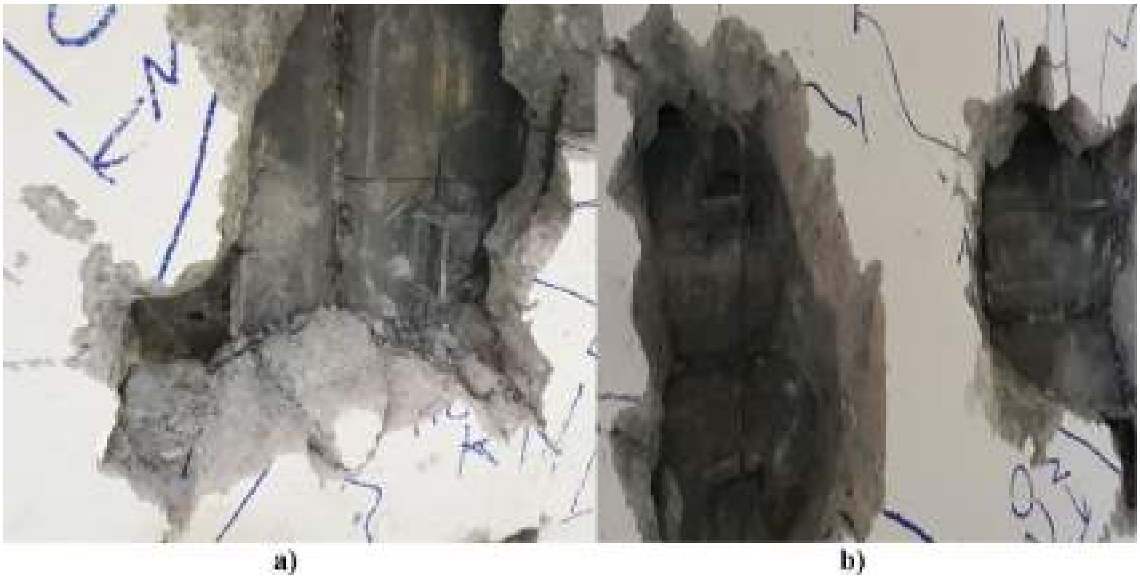


Fig. 46 Bent-up tabs status after the test

Fig. 47 Failure mode of using steel rebar at the sides system

Fig. 44a, b. A transverse slip was noted at the maximum strength capacity of the specimen, as shown in Fig. 45a, b. Concrete crushing governed the fracture and failure of this system and when the compressive strength of concrete increased, the shear capacity of this type of system also increased. The specimen was dismantled to investigate and visualise the failure of the CFS and the shear connector status after the test. The bent-up tabs shear connectors remained in good condition after the test, as shown in Fig. 46a, b and they could still carry more applied load.

Effect "steel rebar at the sides" as a shear connector system

This type of shear connector system performed well, until its failure at 256 kN. Shear connectors resisted the longitudinal shear force by their embedded portion in the concrete slab and by the contact area between the concrete slab and the concrete inside the CFS beam. The failure mode of the push test specimen with this system of shear connector is initiated by transverse and longitudinal cracking at the concrete slab surface, followed by concrete crushing, as shown in Fig. 47. The concrete crushing governed the fracture and failure of this system; if the compressive strength of concrete increased, the shear capacity of this type of system also increased. The specimen was dismantled to investigate and visualise the CFS and shear connector status after the test; it was found that no deformation on the part of the CFS and shear connector embedded in the concrete slab was observed, as shown in Fig. 48a, b.

Effect of "steel rebar embedded from the top" as a shear connector

This type of shear connector system has shown very good performance, where the first crack occurred at 125 kN and further developed until it failed at 213 kN, as shown in Fig. 49a, b. Just before reaching the final failure at 210 kN, an internal crushing sound was heard, and many tiny cracks

Fig. 48 Shear connector status after the test

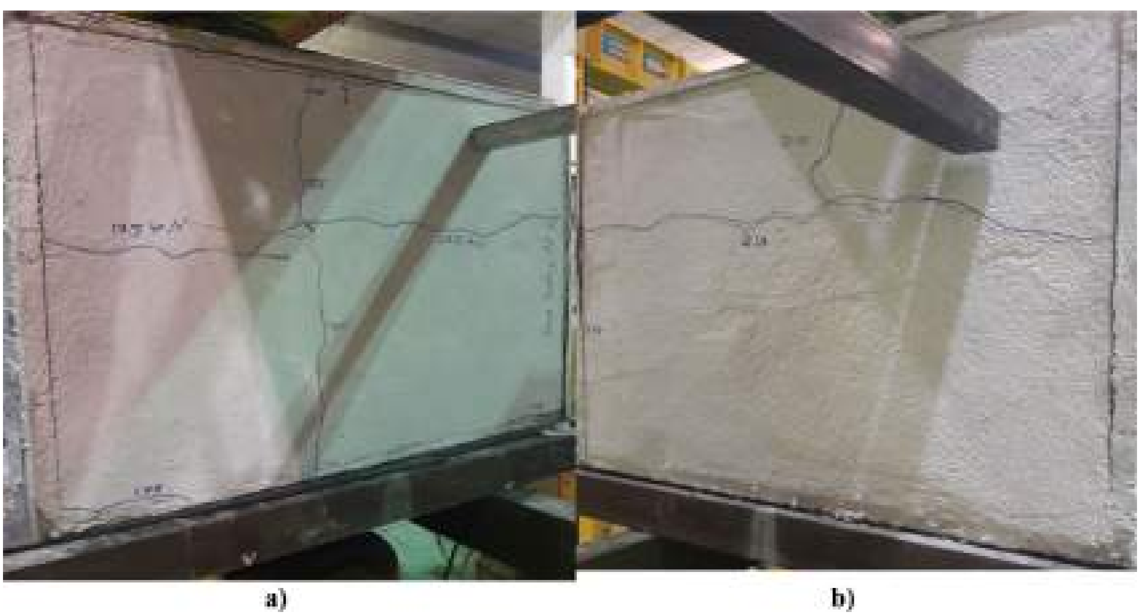


Fig. 49 Crack pattern of the specimen

developed in both slabs as a result of the friction and interaction between the concrete slabs and shear connectors. A significant transverse slip was noticed at the maximum strength capacity of the specimen, as shown in Fig. 50. Shear connectors resisted the longitudinal shear force by its embedded portion in the concrete slab and by the contact area between the concrete slab and the concrete inside the CFS section. The failure mode of the push test specimen with this system of shear connector is initiated by transverse and longitudinal cracking at the concrete slab surface, followed by concrete crushing. The concrete crushing governed the fracture and failure of this system, shear connectors remained in good condition after the test, as shown in Fig. 51a, b and can still carry more applied load (Fig. 52).

Effect of using brackets as a shear connector system

These shear connectors failed at 64.5 kN. The screws showed shearing-off behaviour as shown in Fig. 53 and then the slab separated from the steel beam. The failure mode of the push test specimen with this system of shear connector was due

Fig. 50 Transverse slip at the maximum strength capacity of the specimen

to the screws shearing-off which contributed to the failure of this system.

Conclusion

Several push-out tests were conducted to investigate the strength capacity and ductility of various shear connector types with different CFS orientations. All of the push-out specimens were developed from the same cold-formed steel materials, while the shear connectors were formed either as cold-formed steel or steel rebar from hot-rolled steel.

Shear connector systems of back-to-back without holes and back-to-back with holes showed a plastic deformation and maintained an adequate strength capacity, achieving the highest strength capacity compared to the other shear connector systems before reaching their ultimate failure load (at which the push test specimens failed because of concrete crushing). This shows that using pre-drilled holes and back-to-back orientation of CFS is an effective and sustainable shear connector system that does not require time or materials. The behaviour of specimens ‘using steel rebar embedded from the top’, ‘embedded CFS beam and steel rebar at the sides’, and ‘embedded CFS beam with bent-up tabs’ showed plastic deformation while maintaining an adequate strength capacity before reaching the ultimate failure load, at which the push test specimens failed because of concrete crushing. The ‘toe-to-toe embedded CFS beam with holes’ system showed an acceptable strength capacity as the crushing of the concrete slab is governed the loading design.

Pre-drilled holes showed that there is a composite interaction between the concrete slab and the steel beam, as the strength capacity increased by 100%, when the pre-drilled holes were used, compared to ‘toe-to-toe embedded CFS beam without holes. The experimental results revealed acceptable compatibility with the theoretical results. The proposed shear connectors and beam configurations showed very good efficiency, in terms of strength and ductility, when compared with the connectors tested by other researchers and compared to Eurocode 4.

The results revealed that using holes and embedding the top flange of the steel beam as shear connection system can be the better method of providing the shear connection due to its simplicity instead of using the conventional way of using the bolts or nuts which is time-consuming and complex.

a) b)

Fig. 51 Shear connector status after the test

Fig. 52 Specimen before the test starting

Fig. 53 Shearing-off behaviour of the specimen

However, the test results are based on the push-out test of the specimens, further studies are needed based on three-point or four-point bending moment test for the full-scale composite beam to examine bearing capacity of the investigated shear connection systems.

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Declarations

Conflict of interest The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Ahmed Almassri reports financial support was provided by University of Technology Malaysia. Ahmed Almassri reports a relationship with University of Technology Malaysia that includes: funding grants.

Ethical statement This material is the authors' own original work, which has not been previously published elsewhere.

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