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Bending experiment on a novel configuration of composite system using rebar as shear connectors with partially encased cold-formed steel built-up beams

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A B S T R A C T

In order to achieve greater ductility and strength, as well as to produce a more economical design, a novel composite beam and floor system have been developed to achieve higher strength and ductility, as well as to yield a more economical design purpose. This paper has put focus on this newly developed composite beam system which consists of a profiled metal decking slab made with self-compacting concrete (SCC). It has been joined cold-formed steel (CFS) built-up beams. These beams have been infilled with SCC by means of U-shaped rebar used as shear connectors. The researcher, in order to construct an open section, put together two CFS C-lipped channel sections in a back-to-back formation, and to construct a closed section, the formation was made to be toe-to-toe. The flexural behaviour of the partly encased composite beam was evaluated through experimentation by the researchers. So that the researchers could observe the failure modes and flexural capacity of the construction, a four-point bending test procedure was performed on two samples taking into consideration both closed and open built-up beam sections. The results of the test demonstrated that the open sections were able to exhibit a 24 percent higher ultimate moment capacity as well as greater stiffness and higher vertical deflection. As can be seen from the results of the experimental bending test, the built-up design had a great impact on the capacity and deflection of the section, and the section that was encased was able to reach the ultimate strength when the proposed shear connector was placed in the composite action. In order to validate the present test results, the design and analysis of the new composite beams have been evaluated.

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1. Introduction

The construction industry has been attempting to raise the section capacity in order to lower costs for several decades. There are lots of solutions for accomplishing this task, and one of those solutions is to manifest the benefits of the materials with none of their inherent disadvantages. The main idea behind this concept was to join two or more materials together so that they act as one material with the combined properties of both, which is called composite action. The typical technique that the construction industry follows is by using a mechanical part to improve the composite action.

Composite construction has been well-established for quite some time as a construction technique; however, the process has traditionally utilized hot-rolled steel sections instead of light steel (cold-formed) sections. The key elements of the traditional style of composite construction have been frameworks made of hot-rolled steel, shear connectors, in-situ concrete with mesh reinforcing steel, and steel decking. There are several benefits of using this method of construction and a few of the most imperative of them include the economy in the use of the materials, speed of the

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construction because of the quick erection of the steel framework, robustness to damage, and good service performance. To these, other advantages can be added if light steel sections are employed instead of the conventional hot-rolled type. There are essentially two preferred viewpoints regarding the use of light steel. The first is that steel which is cold-formed is not as expensive as hot-rolled steel and the second is that, it is also not as heavyweight [1].

Members which are made from cold-formed steel (CFS) are being used more and more as primary structural components in buildings because of the thin, high strength steel is readily available and due to the advanced technologies for cold-forming. These buildings are made using thinner steel created by a different kind of forming; moreover, their cross-sections are commonly unsymmetrical or mono symmetrical. Therefore, the behaviour of the lateral-torsional buckling is rather more complicated than the behaviour of a doubly symmetric hot-rolled beam. The key benefit of the CFS beams compared to the hot-rolled beams is because of how thin the material is that is used to form the sections. Some of the common failure modes of thin-walled sections include web crippling, and flexural buckling, local, and torsional buckling. It is able to resist bending in addition to membrane forces although the structure is small in comparison to its other dimensions for thin-walled structures. Thin-walled sections often buckle under lower stress and thus, they have to be stiffened to avoid this situation. Conservative design rules are used to determine their ultimate moment capacities [2].

A CFS composite beam is made up of two cold-formed steel sections which are partly encased in self-compacting concrete (SCC) and used to replace the reinforced concrete or hot-rolled steel, shear connectors, and concrete slabs. The concrete slab behaves as an integral part of the composite beam which is unlike the beams with no composite action. A composite beam gains benefits from its various components. First, it can take advantage of the high tensile strength of the steel and second, it benefits from the high compressive strength of the concrete, both of these lead to the significant enhancement of the flexural strength of the beam. The cold-formed in-filled CFS-SCC composite beam is a feasible component for use in engineering applications. These beams possess the following benefits: (1) The encased concrete is an effective structural support for the external CFS section, which is thin-walled so that the local buckling of the steel can be impeded for a while or possibly even completely prevented. This is in contrast to the H-section of the open steel found in the traditional steel–concrete composite beams for which the width to thickness ratio has to be constrained; (2) Together with the encased concrete, the external CFS is able to obtain a greater shear capacity which helps it avoid brittle shear failures; (3) The sections have no need for any temporary formwork for the in-filled concrete; this is because the steel plays the role of a formwork during the construction stage and as a reinforcement during the service stage [3–7].

Short channels or headed shear studs are commonly used in the traditional composite beams to transfer the shear force between the concrete slabs and the beams as well as to prevent the separation of the slabs from the supporting beams. Recently, researchers in the related field have been focusing on creating novel forms of composite beams that can aid in the reduction of the costs, prevent unfavourable behaviours related to the steel beam or get rid of the construction difficulties attributed to the shear connectors [8,9]. Amongst the many novel forms of composite beams, a popular one consists of a Cold-formed steel built-up section with bent-up rebar used as a shear connector. This type of composite beam is presented schematically in Fig. 1. As can be observed, the shear connector is encased in the composite specimen, with part of it in the in-filled CFS beam and the other part in the concrete slab. In comparison with the headed shear studs, the bent-up rebar has no need for any special equipment or laborers for their installation and, correspondingly, they are less costly. Besides that, because of the concrete in-fill, the CFC section is better able to resist the torsional and local buckling when compared to the sections that are open. Furthermore, the in-filled concrete also has the potential to be able to delay the rise in the temperature of the steel girder and thereby enhance the fire resistance of the composite beams, the same as with other composite members [4].

This paper has presented a new design for a composite beam and floor system consisting of an innovative system that has been patented, as seen in Fig. 1, to develop the composite beams and floor system which are easy to construct, ductile, and strong. The beams and floor system using materials that are economical and readily available to builders, do not need any heavy tools, are lightweight and easy to handle and transport, as well as being easy to assemble, and can be prefabricated in a factory or cast on site. The composite beam and floor system proposed in this work employ a unique type of shear connector which is made of a bent-up, u-shaped, structural, ribbed rebar. This shear connector is not only lightweight, but it also has an easy installation process with an easy to handle set of equipment. The following features are a part of the CFS section: (1) The slab is made integral with the encased concrete in the CFS composite beam by simultaneous casting; (2) The cold-formed composite beam of light dead weight, and it requiring no propping; (3) The bent-up rebar is able to at least slow down if not completely prevent not only slip but also uplift effects at the interface between the lower part and the concrete slab.

2. Material and methods

2.1. Material

The materials utilized in this present study were the CFS of a lipped C-channel section with a lip depth of 20 mm and thickness of 2.4 mm. The width of the section was 150 mm and the depth were 250 mm. Tests of all the materials utilized when constructing the proposed composite specimens (shear connectors, reinforcement mesh, and CFS channel sections) were performed in order to determine the properties of the materials. Coupons were cut out of the transverse and longitudinal directions of the CFS channel sections. They were then tested in accordance with the BS EN 10002–1[10], using the INSTRON 600 DX as can be seen in Fig. 2a. The ultimate strength, yield strength, and modulus of elasticity of the steel components have been denoted as $f_y f_u$, andE_s, respectively, as presented in Table 1.

The SCC came as a ready-mix from the LAFARGE company. The casting was carried out in the laboratory and there was no need for any extra formwork. The modulus of elasticity, as well as the compressive strength of the concrete employed in this study, were calculated from the test of the compressive strength of the cylinders with the dimensions of 150 mm × 300 mm and the standard concrete cubes with the dimensions of 150 mm × 150 mm × 150 mm. Before being tested until the point of failure, the cylinders and cubes were prepared and cured for 28 days. The performances of the compression tests were in accordance with the recommendations provided in the BS EN 12504–1[11]. The lists of the compressive strength values obtained for the concrete cylinders f_{cd} and the concrete cubes f_{cu} can be seen in Table 2. The lists of values for the modulus of elasticity E_c of the concrete employed in this study are also presented in Table 2. All the values for the mechanical properties of the tested specimens can be seen in Fig. 2b.

The yield strength of the profiled steel deck was 350 MPa with a nominal thickness of 1.0 mm as well as a profiled sheeting rib height of 50 mm. The dimensions the composite sample have been presented in Table 3.

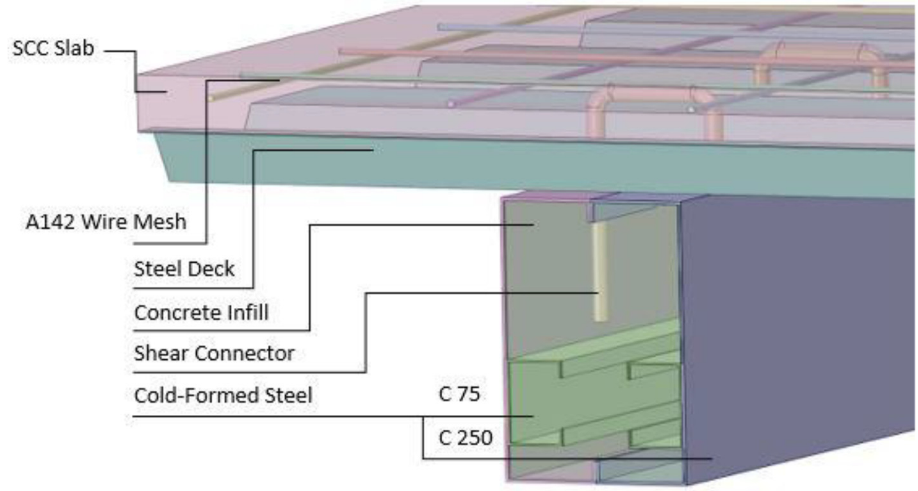


Fig. 1. Details of the composite beam and floor system.

a) Coupon sample for CFS properties b) Cubes and cylinder for hardened properties

Fig. 2. Tested material.

Table 1
Properties of steel.

Material	Diameter/Thickness mm	Yield Stress f_y (N/mm^2)	Ultimate Stress f_u (N/mm^2)	Elastic Modulus E_s (N/mm^2)	$\frac{f_u}{f_y}$
CFS	2.4	524.064	592.182	237547.0	1.130
Wire Mesh	Ø6	712.666	725.333	149000.0	1.017
Reinforcement	Ø12	656.666	746.666	213333.3	1.137

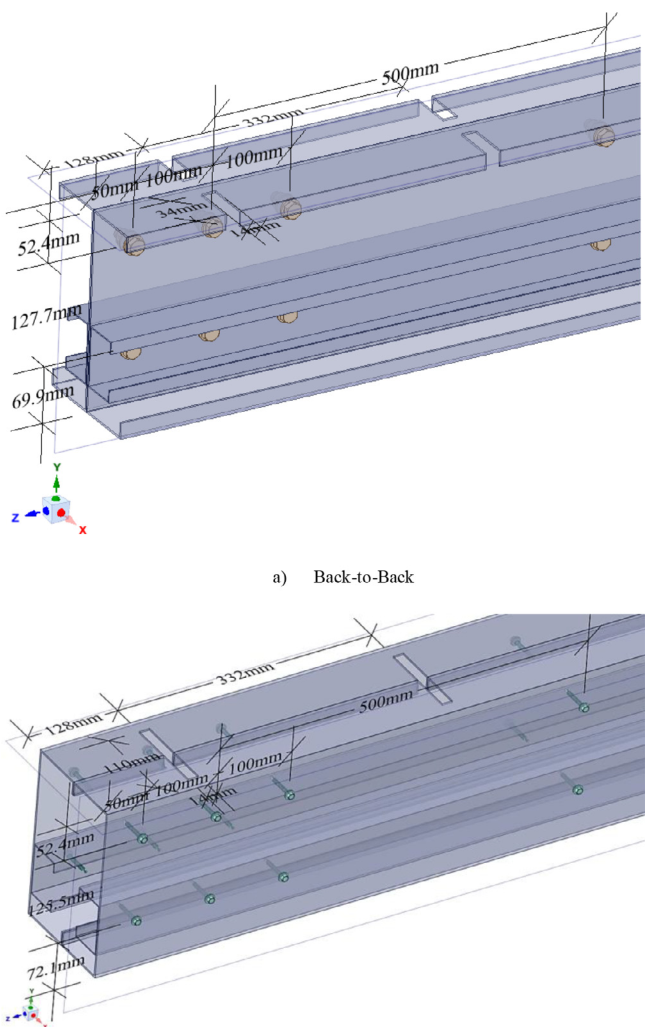
Table 2
Properties of the hardened concrete.

Properties	C1	C2	C3	Average
Cube Compressive Strength f_{cu}(N/mm^2)	60.7	56.9	63.9	60.5
Cylinder Compressive Strength f_{cd}(N/mm^2)	48.6	45.5	51.2	48.4
Modulus of Elasticity E_c(N/mm^2)	37797.6	37072.1	38389.7	37753.1

Table 3
Composite specimen dimensions.

Specimen ID	Length	Slab		CFS Beam			Shear Connectors		
		Width (mm)	Depth (mm)	Thickness (mm)	Depth (mm)	Transverse Spacing (mm)	Number	Size (mm)	Height (mm)
BB-250–12-12	4000	1500	100	2.4	250	750	13	12	75
TT-250–12-12	4000	1500	100	2.4	250	750	13	12	75

All the test specimens consisted of two parallel composite beams with a spacing of 750 mm from the centres, as well as a composite slab. These beams were fabricated through the orientations of the two CFS channels' lipped sections in a back-to-back manner to form an I-section or in a toe-to-toe manner to form a Box-section as in Fig. 3. This type of orientation suppressed both the lateral-distortional and lateral-torsional buckling [12]. In the case of an I-section, as in Fig. 3.a, there was a total of two rows consisting of 13 pairs of M12 $\times$ 45 mm hexagon bolts with a grade 8.8 and two washers with a single nut to act as fasteners in the beam. Installation of the bolts was performed through bolt holes that had been drilled on the web of the CFS, and they were arranged in designated longitudinal intervals of 500 mm centre-to-centre in the intermediate region and in intervals of 100 mm at the end region.



a) Back-to-Back

b) Toe-to-Toe

Fig. 3. CFS beam fabrication orientation.

As presented in Fig. 3.b, the Box-sections were fabricated by positioning the CFS channel's lipped sections in a toe-to-toe manner and welding the flanges from the top and bottom with an arc welder and having a spacing of 300 mm [13]. Additionally, two rows of self-drilling screws with a 5.8 mm diameter were installed at a spacing of 500 mm in the intermediate region and of 100 mm at the end region; this was for the enhancement of the CFS-SCC contact behaviour.

The shear connector proposed in this work was made from a deformed bent-up rebar 12 mm in diameter and a steel grade of S460. The shear connectors were embedded in the SCC by way of a CFS flange that had been cut for the installation of the shear connectors. The properties of the reinforcement mesh included a BRC welded wire mesh A142 with a 6 mm diameter and a 200 mm spacing in both directions. As well, the SCC of grade 40 N/mm2 at 28 days was also one of the materials used in this present study.

2.2. Test setup loading protocol and instrumentation

A hydraulic jack machine with a maximum capacity of 1000 kN was employed for the testing of all of the full-scale composite beam specimens. Testing of the specimens was achieved by subjecting them to the four-point bending load test, with the load being applied at 1000 mm away from the supports. A load cell was placed between the jack and the spreader beam so that the applied load could be measured when the load cell was connected to the data logger. When the load had been applied to the specimen, it was received through the spreader beam from the jack and then the load was transferred to the concrete slab. A rubber gasket strip was put under the point loads and used for the absorption of any imperfections in the surface of the concrete slab and so that it would be a uniform contact surface between the point load and the concrete slab.

The specimen was fixed in a simply supported position. As presented in Fig. 4, for the measurement of the value of the vertical deflection at the mid-span of the beam specimen, six linear variable displacement transducers (LVDTs) were used to measure the deflections of the specimen. The LVDTs were situated at the critical zones of the loading points and the mid-span of the test specimen. Slips could also be noticed when two LVDTs were affixed at the centreline of the concrete slab faces at the sides of the support sides, i.e., the roller and the pin. The data logger was connected to each of the LVDTs and the displacements were measured in "mm". Strain gauges were installed in the critical zones at the bottom flange of the steel beam as well as the top surface of the concrete slab at the maximum shear and bending areas.

After setting-up and instrumenting the test specimen, the specimen was loaded up to 15% of the expected ultimate capacity, then there was a pause of 5 min. The load was then removed to ensure that the specimen and instrumentation were settled. Each reading of the load, LVDTs, and strain gauges was set to zero so that equilibrium could be assured. After that, loading increments of 15 kN, with a pause of 5 min between each increment, were used to apply the load to the specimen. This mode was used to allow for enough time to observe and visualize the specimen and crack pattern until the point of failure. Increases were made to the load until the ulti-

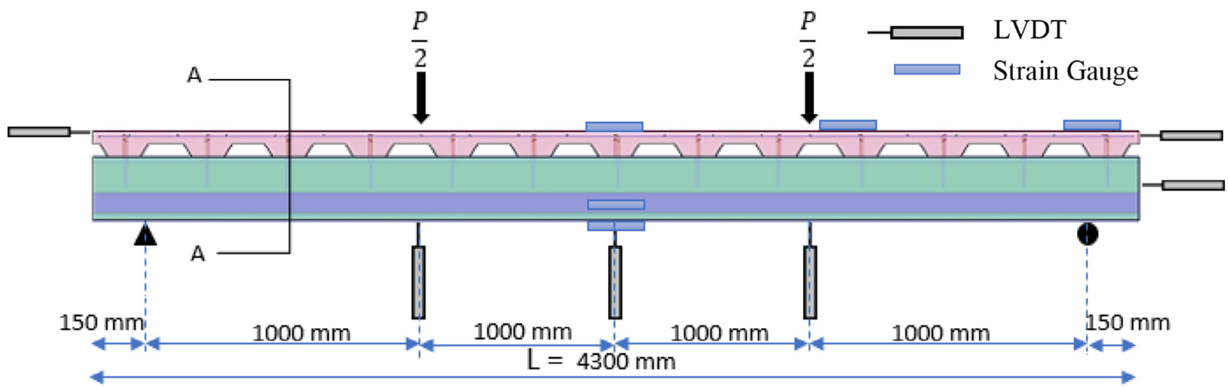


Fig. 4. Geometries and instrumentation used in the experimental program.

mate capacity of the specimen was reached. The specimen was considered to have failed if there was a substantial amount of deformation in the test specimen noticed or there was a large drop in the applied load.

3. Experimental results and validation

As presented in Fig. 5, for the tested specimens, the load-carrying capacity or the Load against the mid-span deflection curves had similar patterns and similar propagation shapes with only one difference of the ultimate load capacity. When the load reached around 50% of the ultimate load, the first crack appeared as a transverse crack at the top surface of the slab close to the loading point. As the load further increased. More and more transverse cracks became noticeable with the introduction of longitudinal cracks along the line of the shear connectors which were forming between the support zones and leading to the loading points. This can also be attributed to the force going upward form the reaction and force going downward from loading point causing vertical shearing effect and hence transverse cracks. With further increases in the load, the shear forces in the shear span increases resulting in more transverse cracks to form between the loading point and the support zones and the stiffness of the composite beams further decreased, consequently, the beam deflections rapidly increased up to a failure point for the specimens.The stiffness of the specimen was lowered further with the further addition of the amount of the applied load, and more cracks formed followed by the crushing of the concrete and local buckling of the CFS at the loading area because of the combination of the maximum moment and maximum shear, respectively. The load was increased, monotonically, until the failure finally occurred.

The theoretical ultimate moment capacity values which were predicted following the EC3 and EC4 guidelines [13,14,15] were compared to the ultimate moment capacity values which were obtained in the experimentation stage, and the results are presented in Table 4. The values of the ultimate loads (Pu, exp.) for the specimens were 713.6 kN and 886.2 kN with 45 mm and 62.8 mm mid-span deflections recorded at the ultimate load levels for the Box-section and I-section beams, respectively. It is quite noticeable that the variation incapacity of the section between the two specimens was 24%; as well, the I-section beam was able to bear increased loads with a greater value of the mid-span deflection as presented in Table 4 and Fig. 5. this could be attributed to the fact that the I-sections are restrained from both sides while box-sections are restrained from one side only which means higher combined thickness in the web for I-sections resulting into more effective section compared to the box-section leading to more rigid specimens. Fig. 6 presents the modes of failure of the specimens which were the local buckling of the CFS and cracks in the concrete slab.

4. Conclusions

The following significant conclusions have been drawn based on the findings from the analytical and experimental work which was completed in this study:

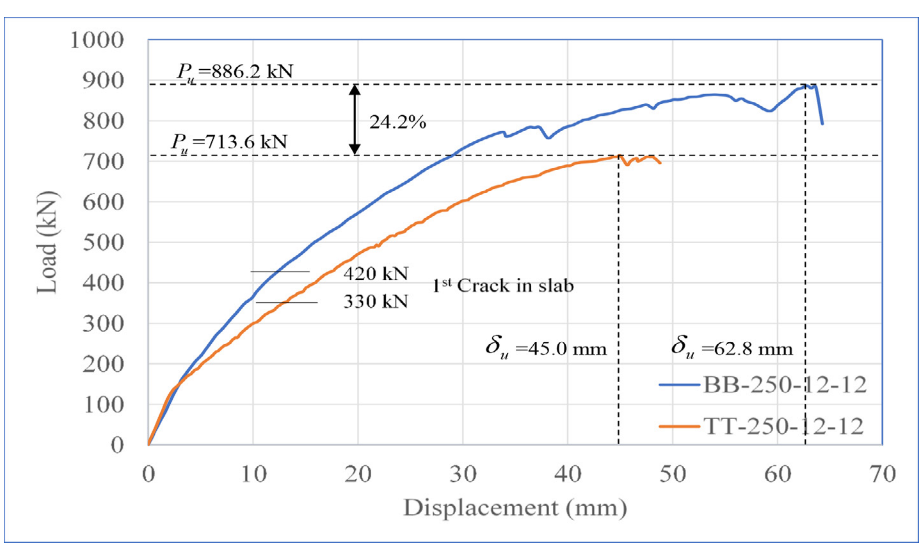
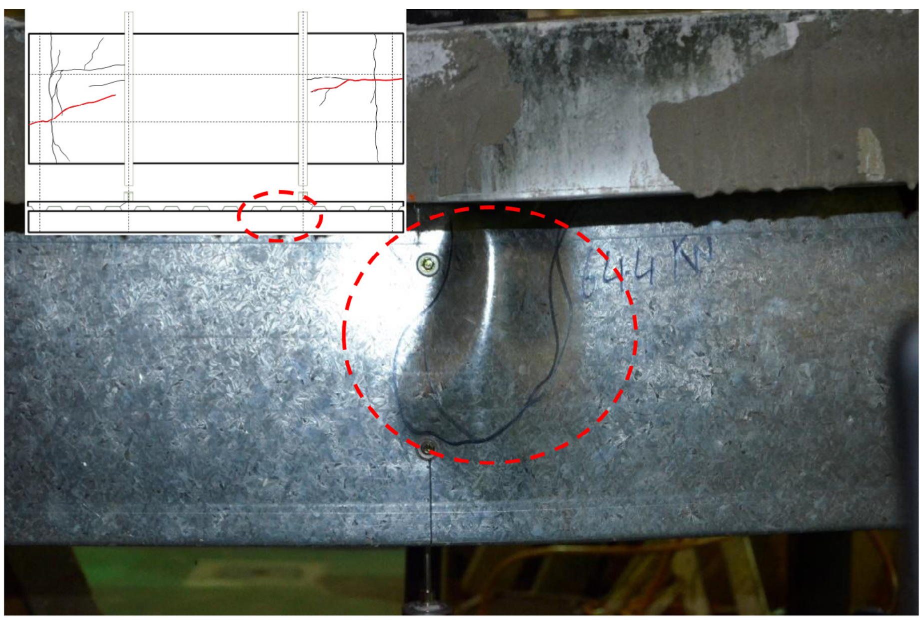


Fig. 5. load-Deflection curves.

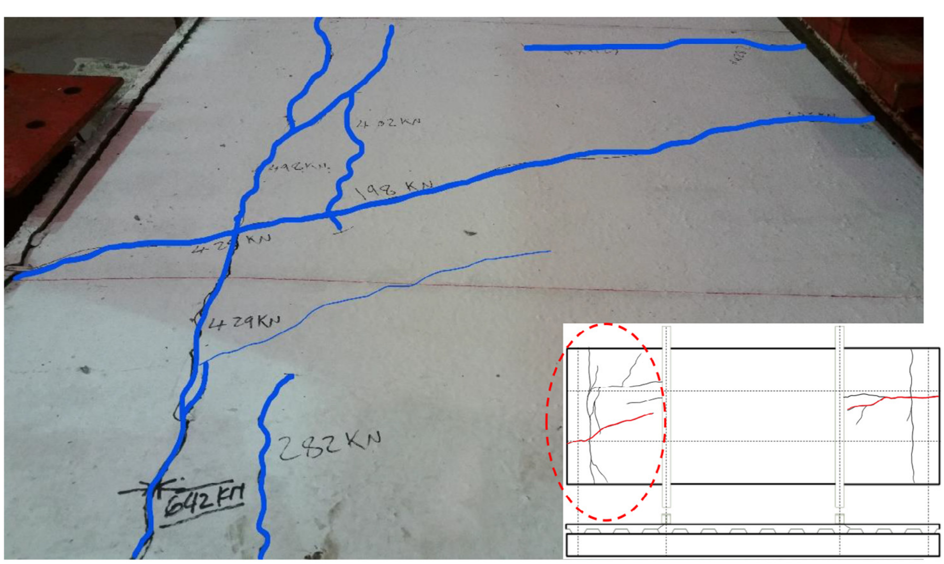
Table 4
Experimental and theoretical capacities of composite specimen.

Specimen ID	*BB-250–12-12*	*TT-250–12-12*
f_{ck}at test day (N/mm^2)	55.6	50.8
Ultimate load $P_{u,\exp}$ (kN)	886.2	713.6
Mid-Span Deflection at $P_{u,\exp}$ (mm)	62.8	45.0
Ultimate Moment $M_{u,\exp}$ (kNm)	443.1	356.8
Theoretical moment $M_{u,theory}$ (kNm)	433.7	372.6
$M_{u,\exp}/M_{u,theroy}$	1.02	0.96

1 Although the material properties and volume remained the same in both samples, the I-section provided a 24.2% greater flexural capacity and an increased deflection in comparison to the Box-section. This led to the conclusion that the I-section was better able to exhibit the material efficiently because of its better utilization of the material, causing a reduction in the buckling area, which resulted in an increase in the effective area and hence increases in ultimate moment capacity.
2 The partial encasement of CFS beams achieved its purpose of enhancing the interaction between the concrete and the CFS resulting in attaining higher moment capacity for CFS before failure.
3 The modes of failure for the Box-section and the I-section were consistent in regard to their crack propagations, locations, and patterns.
4 "Utilisation of the bent-up rebar to act as a shear connector provided an increased level of the composite action ratio. This meant that a greater yield strength was achieved by the CFS in the composite section. This was due to the continuous shear connector having aided in the movement of the neutral axis of the composite section upwards, thus preventing the flange at the top of the CFS from buckling under the compression.



b) CFS local buckling under loading point



a) Concrete crack pattern at maximum bending shear region

Fig. 6. Failure modes of CFS and concrete slab.

5 The experimental specimens of the partly encased composite beams were consistent in demonstrating their ductile flexural behaviors. The greater ductility and flexural strength which was seen in each specimen suggested that the partly encased composite beam has the potential to be employed with future practice.

CRediT authorship contribution statement

Musab N.A. Salih: Conceptualization, Investigation, Methodology, Validation, Formal analysis, Resources, Data curation, Writing - original draft. **Mahmood Md Tahir:** Supervision, Funding acquisition, Writing - review & editing. **S. Mohammad:** Supervision. **Y. Ahmad:** Supervision, Writing - review & editing. **P.N. Shek:** Project administration, Resources. **Achmad Abraham:** Investigation, Methodology, Data curation. **Muhammad Firdaus:** Investigation, Methodology, Data curation. **Khadavi:** Investigation, Methodology, Data curation.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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