Strength Capacity and Failure Mode of Shear Connectors Suitable for Composite Cold Formed Steel Beams: Numerical Study

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Abstract: In this paper, the findings of numerical modeling of the composite action between normal concrete and Cold-Formed Steel (CFS) beams are presented. To obtain comprehensive structural behavior, the numerical model was designed using 3-D brick components. The simulation results were correlated to the experimental results of eight push tests, using three types of innovative shear connectors in addition to standard headed stud shear connectors, with two different thicknesses of a CFS channel beam. The proposed numerical model was found to be capable of simulating the failure mode of the push test as well as the behavior of shear connectors in order to provide composite action between the cold-formed steel beam and concrete using the concrete damaged plasticity model.

Keywords: composite beam; CFS section; push test; shear connector; finite element modeling

1. Introduction

Currently, there is extensive use for Cold-Formed Steel (CFS) sections in the construction industry. They are used as lightweight load-bearing frames [1]. In the context of its widespread use, the first standard code for CFS construction was published in 1946 [2]. To improve CFS sections’ strength and toughness and prevent instability due to buckling, stiffening CFS sections by edge stiffeners was introduced [3,4]. In residential building construction, the use of CFS sections has become increasingly popular, due to their appreciable efficiency [5]. It is a relatively new concept to develop composite action between CFS sections and concrete to form composite beams in small and medium buildings [6–9]. The difficulty of introducing conventional shear connectors to CFS sections due to a lack of specifications may be the reason behind the limitations of its usefulness in composite applications.

However, researchers were excited to conduct investigations on composite CFS concrete members. For example, an innovative lightweight composite bridge girder was investigated [10]. A composite concrete CFS slab connected using a stand-off screw was studied by [11]. Composite concrete slabs with CFS I-sections and new CFS shear connectors connected by self-drilling screws and welding were investigated by [12]. Composite concrete CFS beams and slab joists with different bent-up, pre-drilled holes, self-drilling screws, a CFS channel, and angle shear connectors were experimentally studied by Irwan et al. [13–15]. Lakkavalli and Liu [16], and Malite et al. [17]. In composite concrete steel slabs, parallel or perpendicular metal decking is usually used. Therefore, developing shear connectors suitable for composite metal decking concrete CFS floors is recommended [18]. The important part of such composite concrete CFS beams is the shear connectors that are essentially responsible for an efficient composite system [19]. Experimental investigations were conducted on composite metal decking concrete beam systems with CFS sections [20,21]. However, so far, this new topic needs more investigation...
to fill a knowledge gap. Therefore, more experimental and numerical works are highly recommended [22]. Due to the higher cost of experimental works that make it difficult to provide sufficient information, numerical modeling has become an alternative reliable approach. Several researchers have used this approach [19,23,24].

2. The Proposed Shear Connectors and Push Test Setup

Innovative composite concrete CFS beams were studied. Two CFS sections with different thicknesses (i.e., 2.00 mm and 2.3 mm) were used. Three innovative shear connectors made of CFS sections were investigated. They are namely SBSC (Figure 1a), DBSC (Figure 1b), and HPSC (Figure 1c). The details of the proposed composite beams and shear connectors are presented in [21]. Moreover, the conventional headed stud shear connectors, namely HSSC, were also investigated. A Headed stud shear connector (HSSC) of 16 mm in diameter, a 76 mm height, and with the yield strength of 450 MPa was used to investigate the feasibility and behavior of welded studs to a thin flange of a CFS section. The stud was welded to the top flange of the steel beam. The welding was formed from the top and bottom faces of the steel flange. The top flange of the steel beam was pre-holed using a 16 mm diameter to allow for such a welding method. Gas metal arc welding with electricity conditions of 20 A and 68 V was used to perform the welding around the stud connector from both sides. Figure 2 depicts the arrangement of the proposed shear connectors in push test specimens.

![Figure 1. Types of shear connectors: (a) SBSC shear connector, (b) DBSC shear connector, (c) HPSC shear connector [21].](image1)

![Figure 2. Cont.](image2)
To allow for slippage during the test, all specimens were constructed with an 0.080 m reset between the ends of the concrete slab and the steel beam. The schematic push test specimen is shown in Figure 3. The push test setup is shown in Figure 4. The test was stopped when the specimen could no longer handle the additional load, and the overall load was decreased by 20%.

Figure 2. Shear connectors’ arrangement [21]. (a) SBSC shear connector. (b) DBSC shear connector. (c) HPSC shear connector. (d) HSSC shear connector.
Figure 3. Test configuration for a DBSC250 specimen [21].

Figure 4. Push test set up [21].

3. Description of the FE Model

We conducted 3-D nonlinear FE simulations using ABAQUS software [25], while considering the material and geometric nonlinearities. The geometric nonlinearity was considered by activating “NLGEOM” in ABAQUS/Standard. This was done to account for significant deformation and local instability results. Figure 5 shows the finite element model for the structural arrangement of the push test for the proposed shear connectors to be simulated in this research.

The simulation methodology can be summarized as follows:

- Three-dimensional solid elements (C3D8) were used to model the key structural members to obtain detailed structural behavior (CFS beam, bolts, concrete slab, and shear connector components).
- Mechanical properties of the steel beams and shear connectors [20,21] were obtained in compliance with BS EN ISO 6892-1:2009. These properties were used to develop the stress-strain constitutive relationships used in the FE models. Table 1 shows the yield strength, ultimate strength, and elastic modulus.
Figure 5. Cont.
For reinforced concrete elements, the ABAQUS program’s concrete damaged plasticity model for reinforced concrete elements was used, which can reflect the complete inelastic behavior of concrete in stress and compression, including damage characteristics. The concrete damaged plasticity model, which takes into account isotropic elastic damage and plastic behavior of materials, can simulate tensile cracking and compressive crushing of concrete materials [26,27].

For bolts and nuts, the mechanical properties were assumed to be elastic-perfectly plastic. For Grade 8.8 bolts, the yield strength was 640 MPa and the elastic modulus was 210,000 MPa.

The welding between headed stud shear connectors and CFS flange were modeled using the “tie” type constraint in ABAQUS.

To prevent the FE model to move or twist, it was restrained from both sides to simulate the actual boundary conditions, as shown in Figures 3 and 4.

The ABAQUS contact function was used to model the interaction between components, including the interface between the two channels, between the concrete slab and the beam, between the shear connectors and concrete slab, between the bolt shanks and the web of the beam, between the bolt heads and the web of the beam, and between the web of the channel and the shear connectors.

A contact was simulated as a surface-to-surface contact with a small sliding choice. To avoid mutual penetration of the steel and concrete in the normal direction, a “hard contact” was assumed for the normal contact behavior, while a friction contact with a coefficient of $m = 0.3$ was applied tangentially to the surface of the contact pairs.

The loads were applied to the beam at one-point loads (as shown in Figures 3 and 4) and were increased gradually until failure. In addition, normal forces were sustained for both concrete slabs using a yoke assembly operated by a hand hydraulic pump to replicate the actual situation of the composite beam in structures and prevent the
rotation of the last studded rib at the top of the push test specimen. The magnitude of the normal force is kept at about 0.1 of the vertical applied loads.

- A basic Rankine criterion was used to detect crack initiation for crack detection [28]. It was also assumed that cracking occurs when the tensile equivalent plastic strain exceeds zero and the maximum principal plastic strain also exceeds zero [29].
- The values of the damage parameters used in the model are as follows: dilation angle $\alpha = 30$, eccentricity $e = 0.1$, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress was $f_{0b} / f_{0c} = 1.16$, the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian $K_c = 0.6667$, viscosity parameter $= 0.0001$.

4. Results and Discussion

The experimental push tests conducted by [20,21] were simulated by the authors using the ABAQUS software package. In this investigation, the FE results are presented in main two groups: deformed shapes of each component and the modes of failure.

4.1. Bracket Shear Connectors

The observed failure modes of SBSC, DBSC, and HPSC push test specimens are similar. These types of shear connectors failed by concrete crushing proved by cracks observed along the slabs located in the center of slabs, which is captured by the finite element results as shown in Figure 6, which represent the pattern of the typical cracks after failure for the SBSC, DBSC, and HPSC. As can be seen in Figure 6, an excessive plastic strain which is represented by a transverse crack occurred underneath the concrete rib of the top and bottom level of shear connectors. The reason for these longitudinal cracks is due to the fact that as applied load increases, the longitudinal shear force transferring from concrete slab to steel beam through shear connectors tends to split the concrete slab longitudinally.

In the test, it was found that SBSC, DBSC, and HPSC shear connectors were rotated (a very small rotation). As a consequence of the initial rotation of SBSC and DBSC shear connectors, the shear connector was deformed, as can be seen from Figure 7a,b. For HPSC shear connectors, no deformation on the part of the shear connector embedded in the concrete slab was observed, as can be seen from Figure 7c.

![Figure 6. Cont.](image-url)
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**Figure 6.** Failure modes of push test specimens compared to proposed FE model. (a) Failure mode of SBSC specimen. (b) Failure mode of DBSC specimen. (c) Failure mode of HPSC specimen.

**Figure 7.** Cont.
Figure 7. Comparison of deformed shear connectors from the FE model and push test specimens. (a) SBSC Shear Connector. (b) DBSC Shear Connector. (c) HPSC Shear Connector.

Figure 7 shows a concentration of stresses around the bolt hole near the CFS beam flange, indicating a strong bearing around the bolt of shear connector near the concrete slab in the test results for SBSC and HPSC specimens, and a low degree of bearing for DBSC specimens. Since the shear connector was able to move around the bolt (i.e., far from the concrete slab) after the concrete was crushed, the bolt near the concrete slab was the only one that could resist the applied load, causing bolt bearing failure.

In DBSC specimens, Figure 8 indicates an excessive separation between both the concrete slab and the metal deck. This is due to the DBSC shear connector’s wide flange surface (double of the SBSC), which results in a weak connection between the concrete’s upper and lower surfaces.

4.2. Standard Headed Stud Shear Connector (HSSC)

For push test specimens with headed stud shear connectors (HSSC), the observed failure mode was caused by deformation of the shank of the stud followed by pulling-out from the thinner flange of steel beam, as shown in Figure 9. This mode of failure was captured by the finite element by the excessive plastic strain in weld connected the stud to the thinner flange of steel beam, as shown in Figure 10. No cracks at the concrete slabs, as well as no separation between the concrete slab and metal deck, were observed.
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Figure 8. Failure mode of the DBSC test specimen compared to the proposed FE model.

Figure 9. Failure mode of the HSSC test specimen compared to the FE model.

Figure 10. Plastic strain distribution from the FE model and pulling-out of the stud from test specimen. (a) Push test specimen. (b) Plastic strain in the FE model.

5. Validation of the Proposed FE Model

Table 2 shows the strength capacity of the shear connectors based on experimental and finite element results. For the SBSC, DBSC, HPSC, and HSSC shear connectors, a very good agreement between experimental and finite element results was observed, ranging from 0.94 to 1.04. With such a clear correlation, it was possible to conclude that the FE model accurately depicts the behavior and strength capacity of the proposed composite beams with the CFS section.

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| Specimen  | Avg. $P_{u, per \ connector}$ (kN) | Predicted Value by Finite Element Results | Averg. Exp./pre. Ratio |
|-----------|----------------------------------|------------------------------------------|-----------------------|
|           | $P_{pre. Per \ connector}$ (kN)              |                                          |                       |
| SBSC250-20| 43.03                                           | 43.63                                    | 0.99                  |
| DBSC250-20| 52.15                                           | 55.34                                    | 0.94                  |
| HPSC250-20| 55.45                                           | 55.27                                    | 1.00                  |
| HSSC250-20| 54.93                                           | 53.80                                    | 1.02                  |
| SBSC250-23| 44.58                                           | 43.00                                    | 1.04                  |
| DBSC250-23| 53.60                                           | 55.79                                    | 0.96                  |
| HPSC250-23| 55.15                                           | 55.45                                    | 0.99                  |
| HSSC250-23| 53.97                                           | 53.80                                    | 0.94                  |

6. Conclusions

This study was focused on developing an FE model to simulate the composite CFS beam behavior. A total of eight push tests were modeled using non-linear material properties of concrete and steel and concrete damaged plasticity model in ABAQUS. Four types of shear connectors were modeled (namely SBSC, DBSC, HPSC, and HSSC shear connectors) with two different thicknesses of CFS channel beam. The following conclusions may be drawn:

- The developed finite element models are in good agreement with the experimental results of the push tests and observations. The deformed shapes and the relevant failure modes were accurately captured by the model for all types of shear connectors. For example, the concrete crushing and shear connector rotation in SBSC, DBSC, and HPSC push test specimens were clearly captured by the FE model.
- The concrete damaged plasticity model in ABAQUS can accurately capture crushing and the longitudinal crack of a concrete slab, which is the common failure mode for SBSC, DBSC, and HPSC.
• The developed finite element models accurately model the interaction between the metal decking and concrete slab. For DSBC, the separation between the concrete slab and metal deck occurred is clearly captured by the proposed finite element models. This weak connection is due to the DBSC shear connector’s wide flange surface (double of the SBSC) that leads to such separation.

• The bearing around the bolt of shear connector that is near to the concrete slab (which was the only bolt resisting the rotation after concrete slab crushing) in the test results of SBSC, DBSC, and HPSC specimens is accurately predicted by the model by observed stress concentration around the bolt hole connecting the shear studs to the web of the cold formed beam.

• The proposed finite element models have clearly captured the failure modes of HSSC specimens being the deformation of the shank and the pulling-out of the stud from the thinner flange of steel beam for HSSC specimens, while no crack was observed in the concrete slab.

• The finite element results of ultimate loads were found to be in very acceptable agreement with experimental data, ranging between 0.94 and 1.04. However, the results of the section analysis are generally conservative compared to experiments.

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References
1. Schafer, B.W. Cold-formed steel structures around the world: A review of recent advances in applications, analysis and design. Steel Constr. 2011, 4, 141–149. [CrossRef]
2. Macdonald, M.; Heiyantuduwa, M.A. Rhodes J. Recent developments in the design of cold-formed steel members and structures. Thin-Walled Struct. 2008, 46, 1047–1053. [CrossRef]
3. Yu, W.W. Cold-Formed Steel Design, 3rd ed.; John Wiley and Sons Inc.: Bridgewater, NJ, USA, 2000.
4. Hancock, G.J.; Murray, T.; Ellifrit, D.S. Cold-Formed Steel Structures to the AISI Specification; CRC Press: Boca Raton, FL, USA, 2001. [CrossRef]
5. Allen, D. Mid-rise construction detailing issues with cold-formed steel and compatible construction materials. In Proceedings of the Structures Congress and Exposition, St. Louis, MO, USA, 18–21 May 2006. [CrossRef]
6. Nguyen, R.P. Thin-Walled, Cold-Formed Steel Composite Beams. J. Struct. Eng. 1991, 117, 2936–2952. [CrossRef]
7. Hossain, K.M.A. Designing thin-walled composite-filled beams. Proc. Inst. Civ. Eng. Struct. Build. 2005, 158, 267–278. [CrossRef]
8. Hossain, K.M.A. Experimental & theoretical behavior of thin walled composite filled beams. Electron. J. Struct. Eng. 2003, 3, 117–139.
9. Abdullah, R.; Tahir, M.; Osman, M. Performance of cold-formed steel of box-section as composite beam. In Proceedings of the 6th International Conference on Steel and Space Structures, Singapore, 1–3 September 1999.
10. Nakamura, S. Bending Behavior of Composite Girders with Cold Formed Steel U Section. J. Struct. Eng. 2002, 128, 1169–1176. [CrossRef]
11. Webbe, N.; Wehbe, A.; Dayton, L.; Sigl, A. Development of concrete/cold formed steel composite flexural members. In Proceedings of the Structures Congress 2011, Las Vegas, NV, USA, 14–16 April 2011; pp. 3099–3109. [CrossRef]
12. Hanor, A. Tests of composite beams with cold-formed sections. J. Constr. Steel Res. 2000, 54, 245–264. [CrossRef]
13. Irwan, J.M.; Hanizah, A.H.; Azmi, I.; Bambang, P.; Koh, H.B. Shear Transfer Enhancement In Precast Cold-Formed Steel-Concrete Composite Beams: Effect of Bent-Up Tabs Types and Angles. In Proceedings of the Technology and Innovation for Sustainable Development Conference (TISD2008), Khon Kaen, Thailand, 28–29 January 2008.
14. Irwan, J.M.; Hanizah, A.H.; Azmi, I.; Koh, H.B. Large-scale test of symmetric cold-formed steel (CFS)concrete composite beams with BTTST enhancement. J. Constr. Steel Res. 2011, 67, 720–726. [CrossRef]
15. Irwan, J.M.; Hanizah, A.H.; Azmi, I. Test of shear transfer enhancement in symmetric cold-formed steel-concrete composite beams. *J. Constr. Steel Res.* **2009**, *65*, 2087–2098. [CrossRef]

16. Lakkavalli, B.S.; Liu, Y. Experimental study of composite cold-formed steel C-section floor joists. *J. Constr. Steel Res.* **2006**, *62*, 995–1006. [CrossRef]

17. Malite, M.; Nimir, W.A.; de Sales, J.J.; Gonçalves, R.M. On the structural behavior of composite beams using cold-formed shapes. In Proceedings of the International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, USA, 19–20 October 2000.

18. Bamaga, S.O.; Tahir, M.M.; Tan, T.C.; Mohammad, S.; Yahya, N.; Saleh, A.L.; Rahman, A.B.A. Feasibility of developing composite action between concrete and cold-formed steel beam. *J. Cent. South Univ.* **2013**, *20*, 3689–3696. [CrossRef]

19. Hsu, C.T.T.; Munoz, P.R.; Punurai, S.; Majdi, Y.; Punurai, W. Behavior of composite beams with cold-formed steel joists and concrete slab. In Proceedings of the 21st International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, USA, 24–25 October 2012.

20. Bamaga, S.O. Structural Behavior of Composite Beams with Cold Formed Steel Sections. Ph.D. Thesis, Universiti Teknologi Malaysia, Skudai, Johor, Malaysia, July 2013.

21. Bamaga, S.O.; Tahir, M.M.; Tan, C.S.; Shek, P.N.; Aghlara, R. Push-out tests on three innovative shear connectors for composite cold-formed steel concrete beams. *Constr. Build. Mater.* **2019**, *223*, 288–298. [CrossRef]

22. Amsyar, F.; Tan, C.S.; Ma, C.K.; Sulaiman, A. Review on Composite Joints for Cold-Formed Steel Structures. *E3S Web Conf.* **2018**, *65*, 08006. [CrossRef]

23. Queiroz, F.D.; Vellasco, P.C.G.S.; Nethercot, D.A. Finite element modeling of composite beams with full and partial shear connection. *J. Constr. Steel Res.* **2007**, *63*, 505–521. [CrossRef]

24. Firdaus, M.; Saggaff, A.; Tahir, M.M. Finite element analysis of composite beam-to-column connection with cold-formed steel section. *AIP Conf. Proc.* **2017**, *1903*, 020024. [CrossRef]

25. SIMULIA. ABAQUS Analysis User’s Manual. Available online: [http://193.136.142.5/v6.11/pdf_books/SCRIPT_USER.pdf](http://193.136.142.5/v6.11/pdf_books/SCRIPT_USER.pdf) (accessed on 25 June 2021).

26. Lubliner, J.; Oliver, J.; Oller, S.; Oñate, E. A plastic-damage model for concrete. *Int. J. Solids Struct.* **1989**, *25*, 299–326. [CrossRef]

27. Lee, J.; Fenves, G.L. Plastic-Damage Model for Cyclic Loading of Concrete Structures. *J. Eng. Mech.* **1998**, *124*, 892–900. [CrossRef]

28. Genikomsou, A.S.; Polak, M.A. Damaged plasticity modeling of concrete in finite element analysis of reinforced concrete slabs. In Proceedings of the 9th International Conference on Fracture Mechanics of Concrete and Concrete Structures, Berkeley, CA, USA, 28 May–1 June 2016. [CrossRef]

29. Demir, W.; Fukang, H. Investigation for plastic damage constitutive models of the concrete material. *Procedia Eng.* **2017**, *210*, 71–78. [CrossRef]