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Experimental Evaluation of Composite Beam-to-Column Joint Using Cold-Formed Steel Sections

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ABSTRACT

This paper is focusing on the experimental investigation of composite beam-to-column joint formed by a composite concrete slab and two lipped cold-formed steel C-sections placed back-to-back as beam and column. As the bare joint of cold-formed sections might not have higher moment resistant, a composite action from the composite concrete slab is utilized and expected to enhance the load carrying capacity of the joint. Two specimens namely the composite joint and non-composite joint are constructed and tested until failure under a point load at a certain eccentricity from the face of the column to induce moment to the joint. All components which are the decking, hot rolled plate, bolts and reinforcement bar used were the same for both specimens except the arrangement of the longitudinal bars. The relationship between the load and deflection, and subsequently, the moment and rotation of the joints are obtained specifically, the ultimate load and the respective moment resistance. It is found out that the composite joint yields higher ultimate load and moment resistance compare to the non-composite joint. The results also indicate that a composite joint has credible rotation and moment resistance, and the fracture of the joint occurs after substantial rotational deformation has been achieved.

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1. INTRODUCTION

Cold-formed steel sections have been used as members in various applications either structurally or non-structurally. In buildings, cold-formed steel members have been used since 1850. From a structural viewpoint, these types of steel members can be classified as (1) individual structural framing members, and (2) decks and panels. The thicknesses of the section most frequently used for this type of structural members range from 0.4mm (0.015in.) to about 6.4mm (0.25in.). Today, thicknesses of up to 12mm (1/2 in.) can be cold-formed to sections (Yu, 1991).

In building construction, cold-formed steel section can be used as primary and secondary framing in low-rise buildings usually up to eight stories in height. For high-rise steel buildings, the main frame is typically composed of heavy, hot-rolled structural steel shapes. Cold-formed steel members, such as roof and floor decks, steel joists, wall panels, sandwich panels, partitions, door and window frames, duct systems, and entrance structures, have been successfully used for buildings in supplementary and complementary manner. Steel roof and floor deck are used in either composite or non-composite system. From the viewpoint of the designer, steel decks not only provide structural strength to carry normal loads, they also provide surfaces for roofs, floors or concrete fill. In addition, such structural elements can provide spaces for electrical conduits or can be perforated and combined with sound-absorption material to form an acoustically conditioned

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ceiling. The cells of cellular panels are also often used as ducts for heating and air-conditioning (Yu, 1993).

Recently, cold formed steel members are extensively used in construction industry. Builders, contractors and companies have realized the efficiency of using cold formed steel sections as primary framing system in low-rise and mid-rise construction; and as secondary framing system in high-rise

or long-span construction (Allen, 2006; Ziemian, 2010). However, the efficiency of using such structural members with concrete to form composite beam members is not properly investigated. Exploitation of composite principles seems appropriate for promoting the use of cold formed steel sections to a wider range of applications (Juki et al., 2008). Cold-formed steel structural member can become even more effective when used in conjunctions with other materials but the main problem in implementation of the designs is ensuring adequate shear transfer between slab and the coldformed section. This behaviour need to be studied to increase the reliability of the use of cold-formed steel beam as a structure component. For example, composite beam with cold-formed steel section could be an alternative economic beam for reinforced concrete and hot rolled steel beams in medium and small size buildings (Hossien, 2005).

Also, in cold formed steel housing and residential buildings, designing beams as composite beams rather than the current practice where the beams designed as non-composite beams could improve their strength capacity and stiffness, and in some cases provide simple fabrication and installation without the requirements of specially trained labour (Fox et al., 2008). However, the thinness of cold formed steel section materials is a big challenge faces the designers and researchers. Introducing composite action between cold formed steel beam and concrete slab should be based on their strength and ductility requirements (Hossien, 2005).

The following objectives are aimed to be achieved in this research:

- a) To obtain the characteristics of the joint through the load-deflection and moment-rotation behaviour
- b) To observe the mode of failures of the joint
- c) To compare the behaviour of the composite beam-to-column joint with different arrangement of reinforcements

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This study is mainly an experimental investigation of the composite beam-to-column joint formed by a concrete slab and two cold-formed lipped C-channel sections bolted back to back (as beam and column element) with two different arrangement of reinforcements. Figure 1 (a) and (b) show the composite joint Specimen 1 and Specimen 2 respectively. The effect of the arrangement of reinforcements on the behaviour of the joint will be the main interest. The cross-section and length of the beam and column, and the length and width of the concrete slab are kept constant.

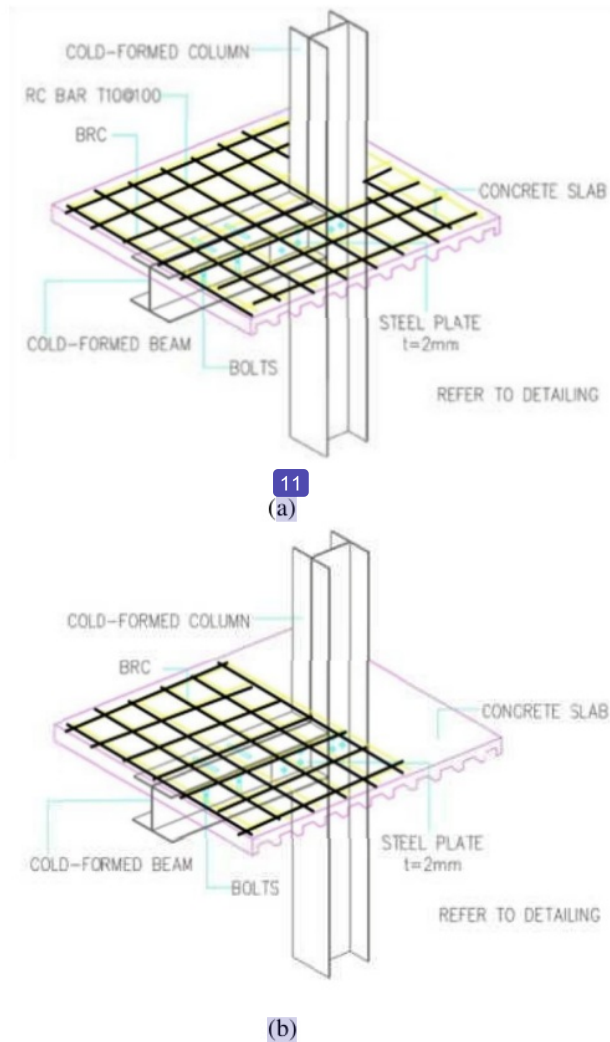


Figure. 1. a) Specimen 1 and b) Specimen 2

The findings from this study are important in verifying the moment-rotation and load-deflection of composite joints using cold-formed steel as a structural component. The results from this study will also display the extent to which the strength can be enhanced by using composite joints column-to-beam with cold-formed steel. From this study, a better understanding of the structural behaviour for steel-concrete composite beam-to-columns in determining the strength and stiffness of the connection can be achieved.

2. SPECIMEN DETAILS

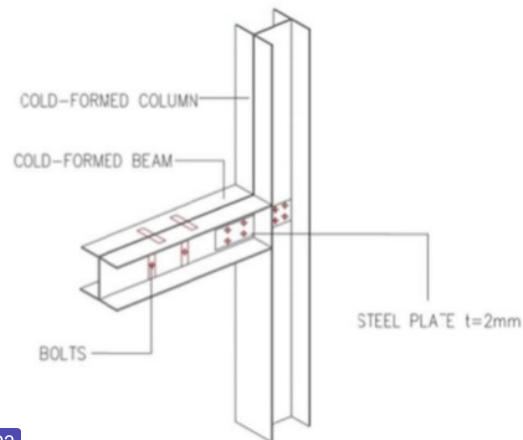
2.1 Cold-Formed Steel Section

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The two cold-formed lipped C-channel sections were assembled back-to-back using bolts to form an I-beam as shown in Figure 2. The channels were fixed together with 13mm diameter bolts at their web to prevent premature failure caused by individual section deformation.



Figure 2. Cold-formed lipped C-channel sections



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Figure 3. Sections are bolted back to back to form the beam and column

As for the plate connection at the beam and column interface, the bolts were installed at 50 mm from both ends and 100 mm spacing as shown in Figure 3. Figure 4, on the other hand, shows the actual fixture of the plate connection and the profiled steel decking.



Figure 4. Bolt arrangement and profiled steel decking

2.2 Slab Deck for Concrete Slab

The slab component was constructed using cold-formed profiled steel decking with dimensions of 1050mm x 500mm x 100mm. Reinforcement bars of 10mm diameter were laid at 100 mm spacing. During this process, it was ensured that the position of the reinforcement bars do not coincide with the position of the shear studs. This was done to ensure that the shear studs can penetrate better into the concrete component during the casting later on. The purpose of tying the BRC to reinforcement bars is to ensure that the BRC would not bulge when the concentrated load is exerted on the specimen during testing. The bulging of the BRC may cause internal cracks and some expansion. This could result in the premature failure of the concrete slab component before reaching its ultimate failure.

2.3 Preparation of Beam-to-Column Joint

The beam-to-column joint was prepared into two (2) specimens that have an end appearance as shown in Figure 5. Four numbers of T10 reinforcement bar were placed in the formwork before concrete was poured in. The concrete mix was poured into the formwork in stages and then vibrated using the vibrator to mitigate the formation of honeycombs and voids. Excess concrete was removed and the concrete surface was smoothed using a trowel. The curing method utilized was wet curing. Wet gunny sacks were used to cover the specimen in order to minimize the loss of moisture due to evaporation. The specimen was left to cure for 28 days. After 28 days, the wooden formwork was removed. The gunny sacks were also ensured to be constantly wet during the curing period. Prior to testing, a few thin layers of white paint were applied on the concrete to make it easier to identify the cracks and fractures during the test.



Figure 5. Reinforcement bars and BRC inside the slab deck for Specimen 1

2.4 Concrete Mix

The specimen was cast with concrete mix of grade C25. The concrete mix was prepared using Ordinary Portland Cement (OPC), fine aggregate passing 5mm sieve and clean tap water. The water-cement ratio was kept at a value of 0.53 to ensure that there is enough moisture to aid in the hydration process of cement. A lower water-cement ratio would lead to a higher strength and durability, but could lower the workability. On the other hand, a mix with too much water would have more shrinkage as the excess water evaporates, which could result in internal cracks and visible fractures.

This too could cause the ultimate strength to reduce.

Concrete mix cubes (150mm x 150mm x 150mm) were also prepared for obtaining the actual compressive strength. After 24 hours of casting, the cubes were then cured in water for 28 days. Compressive test was conducted for each and its corresponding compressive strength was recorded.

2.5 Instrumentation Setup

The instrumentation was setup as shown in Figure 6 (a) and (b). The surface was wiped cleaned using brush to remove any impurities on the surface since it could affect the readings and weaken the adhesion of the inclinometers on the beam and column.

Four (4) Linear Variable Displacement Transducers (LVDTs) were utilized to record the deflection and translation in the vertical direction of the beam and horizontal direction of the column respectively. LVDT1 and LVDT2 were located at the beam's bottom flange for measuring vertical deflection. Whilst LVDT3 and LVDT4 were located at the column's flange and concrete slab respectively for measuring horizontal translation. In addition, two (2) inclinometers were placed at the suitable location for measuring rotation. One was attached at the centre of the connection on beam while the other one was attached at the centre of the connection on column. For data collection, a load cell with 70kN capacity, hydraulic jack and data logger were utilized.

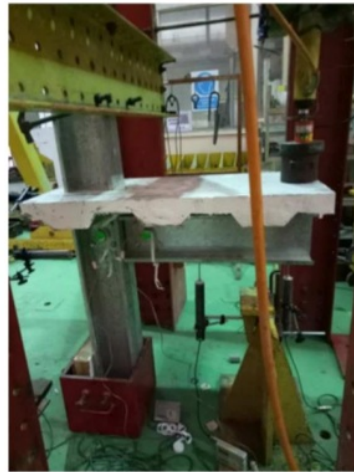
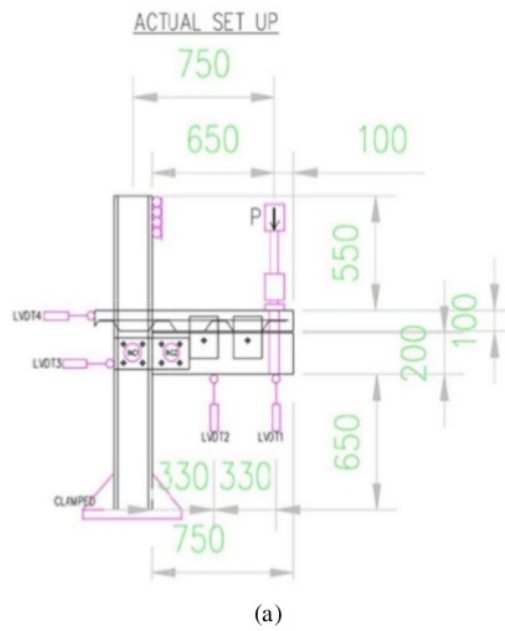


Figure 6. Specimen ready for testing: a) schematic, and b) actual

3. RESULTS AND DISCUSSIONS

3.1 Isolated Joint Test

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For each test, the rotations and deflections of members were recorded. Rotations were taken from inclinometer readings in unit of degree and converted from the unit of degree to unit of radian, whilst deflections were measured directly using the LVDTs. The load was applied gradually at a distance of 650mm from the centre of the plate connection (face of the column) until failure. The moments at the joint were then calculated by multiplying the applied loads with the above-mentioned distance.

Subsequently, the relationship between the applied load and deflection was obtained. Moreover, from the same data, the relationship between the moment and rotation was also obtained.

3.2 Failure modes

Both specimens were tested until failure. However, as the test progressed, the deformations and crack propagations were observed as well. Several deformations were identified and can be listed out as follows:

- i. Bearing in the web section around bolt hole
- 21 Deformation of the beam flange
- iii. Deformation of the column flange
- iv. Crushing of the column web
- v. Deformation of the connection's plate

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During both tests, no vertical slip was observed between the connection's plate, beam and column. This was primarily due to the fact that the tightness of the bolts was considered during installation and setting up. Figure 7 shows the condition of Specimen 1 at ultimate failure.



Figure 7. Condition of Specimen 1 at ultimate failure

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At ultimate failure, the web of the beam buckled underneath the point loads due to the combination of maximum bending and shear. The failure load was recorded as 14.0 kN for Specimen 1 and 3.60 kN for Specimen 2. From the visual inspection during the test, it was found

that the concrete slab of Specimen 1 started to crack at 1.8kN in the tension area near to the column with some transverse cracks on the concrete slab. It is worth noted that the metal decking has not deformed when the ultimate load had been reached. Typical crack pattern of a specimen is shown in Figure 8.

At 5.49kN, a cracking sound was heard and some transverse cracks were beginning to form. When the load was increased to 11.04kN, a pop sound appeared but no visual failure observed. The pop sound might have come from reinforcement bar slippage. At 12.5 kN, the beam and column started to deform especially around the column web and flange due to the compression action from the bottom flange of the beam. The ultimate load was reached at 14.0kN with a crushing of the column web and beam underneath the applied load, and wider transverse cracks in the concrete slab.



Figure 8. Transverse cracks on concrete slab



Figure 9. Cracking near to the column in tension zone

For the Specimen 2, as the load increases, ²⁸ initial transverse crack around the column was observed at 2.0kN. This crack was propagated across the slab and become wider at 2.27kN. The load was increased gradually until the ultimate load was reached at 3.60kN. As the load increased, transverse cracks were observed to form substantially at the column area of the specimen at 3.40kN. However, at 3.42kN, the column web and flange was started to buckle and crush.



Figure 10. Cracks of Specimen 2

3.3 Load-Deflection Behaviour

Load-deflection curves for Specimen 1 and 2 are presented in Figure 11. Tabulated in Table 1 are the ultimate load and the maximum deflection associated with both specimens. By referring to Figure 11, it could be interpreted that the load carrying capacity of both joints differ substantially. It is clearly seen that the joint of Specimen 1 can no longer sustain the after at 14.0kN. Hence 14.0kN is taken as the ultimate load with the corresponding deflection of 46.6mm.

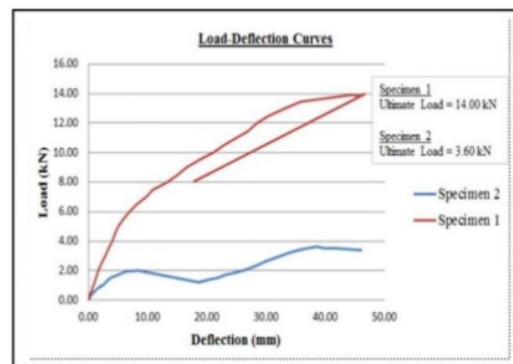


Figure 11. Load-deflection curves for Specimen 1 and 2

Table 1. Ultimate load and maximum deflection

Specimen	Ultimate Load (kN)	Maximum Deflection (mm)
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Specimen 1	14.00	46.60
Specimen 2	3.60	38.50

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For Specimen 2, it can be seen that the ultimate load of the joint is 3.60kN with a corresponding deflection of 38.5mm. The first observed failure pattern was the cracking across the direction of column on the top of the concrete surface. As the load increased, the crack propagated further through the whole width of the slab. Finally, the joint reached the ultimate load and failed by buckling at the web and flange of the column due to the compression action from the bottom flange of the beam.

3.4 Moment-Rotation Behaviour

The most significant characteristic that describes the overall behaviour of the joint is the moment-rotation curve. Moment-rotation curves for Specimen 1 and 2 are presented in Figure 12. Tabulated in Table 2 are the ultimate moment and the maximum rotation associated with both specimens.

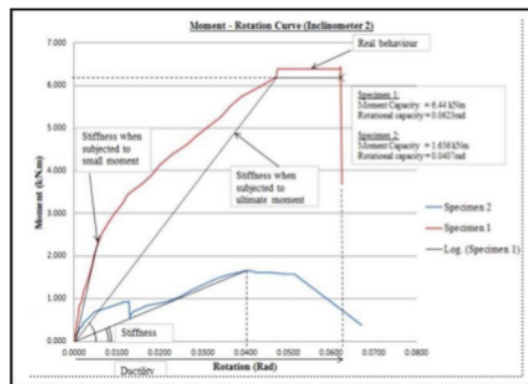


Figure 12. Moment-rotation curves for Specimen 1 and 2

Table 2. Ultimate moment and maximum rotation

Specimen	Maximum moment (kNm)	Rotation (rad)	Initial Stiffness, $S_{j,ini}$ (kNm/rad)
Specimen 1	6.44	0.0623	103.4
Specimen 2	1.66	0.0407	40.68

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Specimen 3 attained its ultimate moment resistance of 6.44 kNm at a rotation of 3.57° or 0.0623rad. The initial rotation 3 stiffness of the test specimen is calculated to be 103.4kNm/rad. Specimen 2, on the other 3 and, attained its ultimate moment resistance of 1.66kNm at a rotation of 2.33° or 0.0407rad. The initial rotational stiffness of the test specimen is calculated to be

40.68kNm/rad.

Both specimens developed high ductility performance as sudden failure was not observed, and the rotations of both joints were more than 0.03 rad. The bilinear concept was considered in determining the initial stiffness from the moment-rotation curve. The results also show that a higher initial stiffness associates with a higher moment resistance. This might be due to the addition of the T10 reinforcement bars and the BRC, and their placement continuously around the column (as in Specimen 1).

4. CONCLUSIONS

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Basically, this study investigates the behaviour of composite beam-to-column joints using coldformed steel sections by conducting isolated tests. From the M-θ curves, the behavioural characteristics of the joints can be determined based on the moment resistance (strength), rotational stiffness (rigidity) and rotational capacity (ductility).

Consequently, the bolted connection is shown to be effective in transmitting moment between the connected sections, enabling effective moment framing among cold formed steel structures. In addition, members formed by connecting two single C-channel sections back to back is also simple to fabricate and suitable for structural application.

From the results, it can be concluded that the beam-to-column joint of Specimen 1 performed better and exhibits the behaviour of a composite joint. On the other hand, the beam-to-column joint of Specimen 2 can be considered as a non-composite joint.

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In all, the study has also proved that the combination of concrete and cold formed steel section as a composite joint can enhance the load carrying capacity of the typically low strength cold formed steel sections.

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