Experimental Study Of Composite Connections For Cold-Formed Steel Using Isolated Joint Test

Muhammad Firdaus, Anis Saggaff, Mahmood Md Tahir, Shek Poi Ngian, KM Aminuddin, Saloma

Abstract:— Cold-formed research that has been studied, which still focused on the capacity of beam and column components. Nonetheless, there is still a lack of data and knowledge on the composite construction behavior, and the performance of CFS Composite beam advantages such as strength and cheapness have led to the dominance of composite beam in the commercial building. The purpose of this study is to investigate the failure modes that occur and present the results of the strength and stiffness of the gusset plate connection of the composite structure. This study uses one type of connection made in the form of cantilever beams whose floor plates use concrete cast with metal decking. The shear connector used uses M12 grade 8.8 bolts with the same tensile stress as the other bolt connections. Beams and columns use cold-formed steel with a thickness of 2.4 mm with a DLC200 size for the beam and a DLC300 size for the columns. The connection uses a hot-rolled steel type gusset plate with a thickness of 6 mm. This failure mode can be indicated that there is an influence of the sliding bolt resistance and bolt support on the joint resistance. Deformation also occurs on a concrete plate that occurs at a distance of 100 mm from the face of the column flange The recording results show the ultimate load achieved is 31.6 kN with 62.26 mm deflection. The ultimate moment occurs is 31.6 kNm with a rotation of 0.038 rad, and the stiffness experiment is 730 kNm/rad. The ratio of the moment experimental with analytical calculations showing a good agreement of 1.41. The ratio between other study has 0.50 ratio of moment resistance and 0.46 for stiffness

Index Terms:— Cold-formed steel, composite beam, gusset plate, moment resistance

1 INTRODUCTION

Cold-formed steel thin-walled members deliver many economic and efficient benefits, including a high strength for a lightweight, relatively straightforward production process and convenient in transportation [1]. The use of cold-formed sections (CFS) has rapidly increased in the construction industry. It has been recognized as a critical contributor to the development of sustainable structures for low-rise residential and commercial buildings in developed countries [2]. Recently, the research of CFS was still focused on the capacity of beam and column components [2-4]. Nevertheless, some scientists have focused on cold-formed connections. Nonetheless, bolts or gusset plates with full-scale experiments [5-10]. Aminuddin [6, 7] was conducted a gusset plate connection study by applying a CFS lipped C-section with depth of C200 for beam and C300 for column, and it was found the lateral-torsional buckling failure at the beam end. The thinness of section makes the local buckling, torsional, and lateral-torsional was happened earlier due to the slenderness section. Therefore, a reasonable solution is by combining CFS section and concrete slab as a composite beam. Nonetheless, there is still a lack of knowledge of the composite connection behavior with CFS [11]. The advantages of composite construction, such as strength and less weight consumption have led to the dominance of the composite beam in the commercial building. The benefits of the composite beam further could be extended by applying with lightweight steel section and metal decking floor to reduce the use of formwork.

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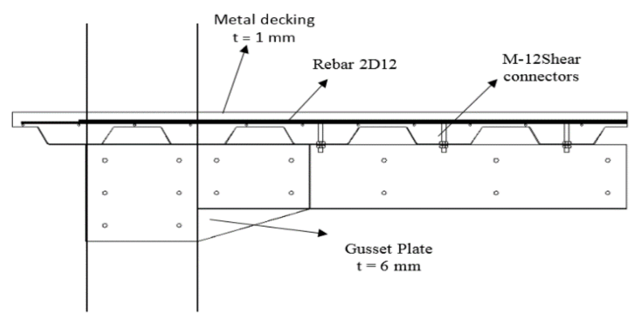
Based on the description above, the experiment was conducted on investigating the behavior of composite connections with cold-formed steel sections. The purpose of this study is to investigate the failure modes and present the experimental results of the strength and stiffness of the gusset plate composite connection.

2 EXPERIMENTAL PROCEDURES

2.1 Parameter of the specimens

The parametric drawing of the proposed connection is presented in Figure 1. The gusset plate of hot-rolled steel was installed as connected plate between the beam and column section (Figure 2**Error! Reference source not found.**). The thickness was 6 mm and design strength of Fy = 320 MPa. Beam and column used cold-formed steel with a thickness of 2.4 mm with a depth of DLC200 for the beam and DLC300 size for the column as presented in Figure 3 and Table I, the design strength Fy=350 Mpa, Fu=420 Mpa. The thickness of concrete slab was 100 mm and 750 mm width. The design strength of concrete was fc '= 30 MPa and cast on metal decking. As shown Figure 4, the 12 mm of reinforcement bars was placed close around the column so that excessive slip does not occur in the concrete slab, the design strength was 250 MPa The M12 bolts grade 8.8 was used as fastening and shear connector with



tensile stress refer to EC3 with Fy = 640 MPa and Fu=800 Mpa [12].

Figure 1 *Connection Model*

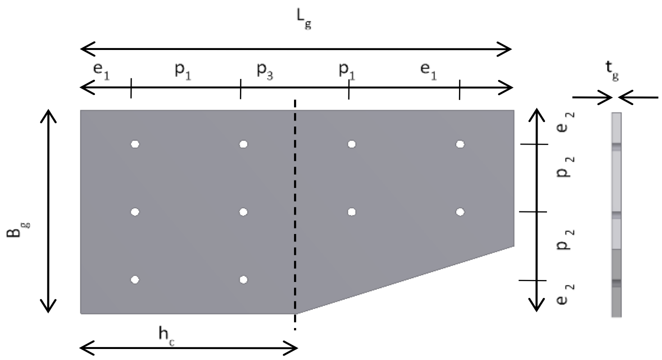


Figure 2 *Gusset plate dimension*

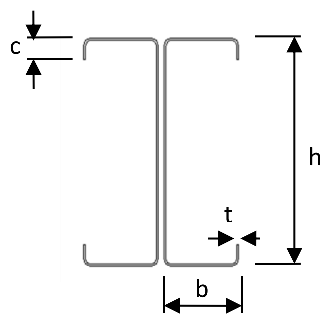


Figure 3 *Double lipped channel profile*

TABLE I*. THE DIMENSION OF DOUBLE LIPPED CHANNEL PROFILE*

Dimension	*h*	*b*	*c*	*t*
	mm	*mm*	*mm*	*mm*
DC200	*200*	*75*	*16*	*2.4*
DC300	*300*	*100*	*25*	*2.4*

TABLE II*. BOLT DISTANCE*

No	*Bg*	*Lg*	*e1*	*e2*	*p1*	*p2*	*p3*
	mm	*mm*	*mm*	*mm*	*mm*	*mm*	*mm*
IJT-01	*300*	*600*	*75*	*50*	*150*	*100*	*150*

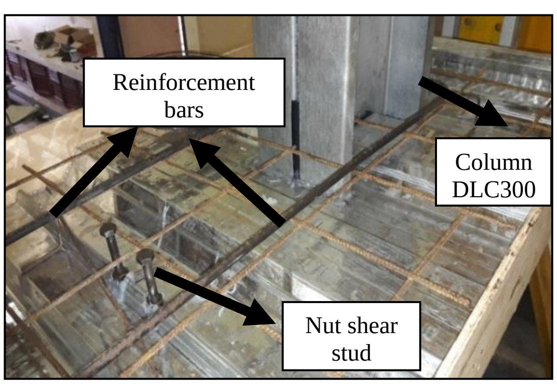


Figure 4 *The rebar and shear connector*

2.2 Test Set-up and Procedures

The isolated joint test was conducted so that the moment-rotation of the connection could be determined. The specimens were installed in frame rig with cantilever beam configuration, and the load cell was placed 1000 mm from the column flange to obtain the applied moment quickly. Figure 5 shows the schematic of full-scale test. The beam was 1100 mm, and the column of 3000 mm was used to represent sub portal frame. Metal decking was permanently attached at the top of the beam flange by M12 bolts with a length of 75 mm. The two inclinometers were installed on the column (Inc 1) and the beam (Inc 2) to find out the rotational data. Four Linear Variable Differential Transformers (LVDT) was installed, therefore the displacement according to instrument orientation could be obtained. The point load was provided by hydraulic jack at the beam end position. The specimen loading was carried out at intervals of 0.5 kN to 25% of the ultimate load. Inclinometer and LVDT readings were recorded to the data logger and save in computer for further analysis. Figure shows the specimen after installed in the frame rig.

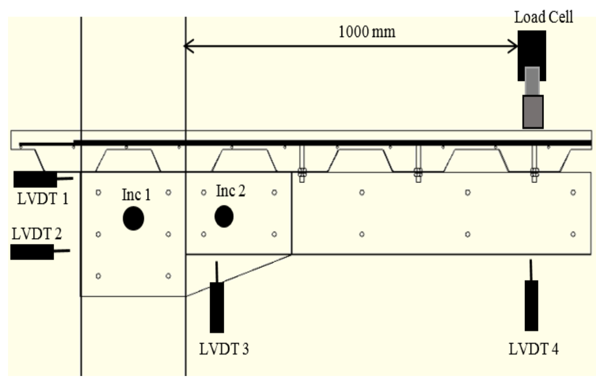


Figure 5 *The Schematic of isolated joint test*

Figure 6 *Actual view of the material*

3 RESULTS AND DISCUSSION

3.1 Load versus deflection

The load versus deflection data was obtained from LVDT-4 and presented in Figure 6 below. The specimen starts to deform after the applied load was more than 5 kN. The deflection was increased linearly until the ultimate load was reached. This is the indication that the specimen was still in the elastic limit behavior, and becomes non-linear at final load was achieved. The deflection of the beam was 61.22 mm, with the final load was 31.60 kN.

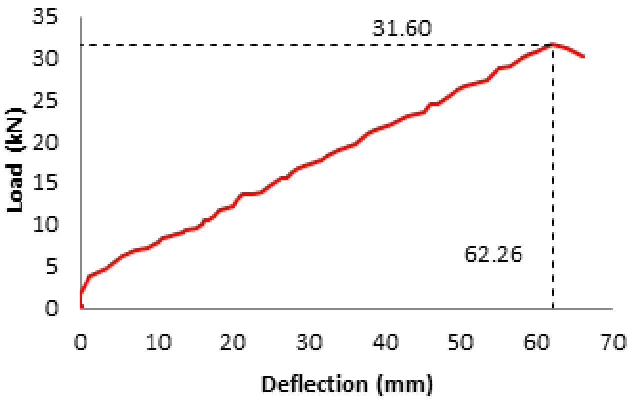


Figure 6 *Load versus deflection*

3.2 Failure Modes

Figure 7 shows the deformation of the beam and concrete slab (red dash line). It was detected the torsional at the end of the beam due to thin-wall behavior as demonstrated in Figure 8. At the maximum rotation of the connection, the lower part of the beam flange was push the the column flange (Figure 9). Furthermore, the local buckling was detected at the column flange because of height compression However, there was no failure mode happen on gusset plates. It was concluded that the deformation could be due to bearing failure at the bolt hole.

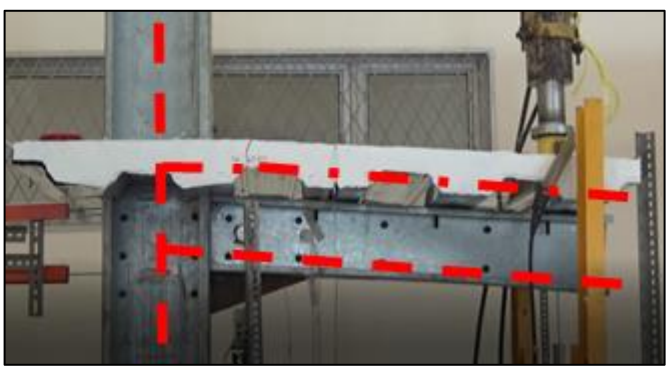


Figure 7 *Deformation of the specimen*

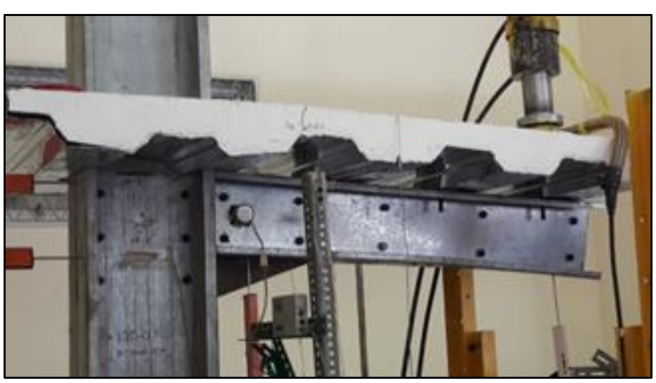


Figure 8 *Torsional of the beam*

Figure 9 *Local buckling of column flange*

3.3. Crack pattern of concrete slab

The initial crack of concrete plate was started at of 6.2 kN The distance of transverse crack about 110 mm from the face of column flange, and it precisely at the top of the thinnest concrete slab (Figure 10). After the load was increased with 8.5 kN , the crack pattern was shift started from the front of column flange towards the edge of concrete slab. Until final load, the crack of concrete slab was not happen because of the torsional deformation of the beam occurs first.

Figure 10 *Crack pattern of the concrete slab*

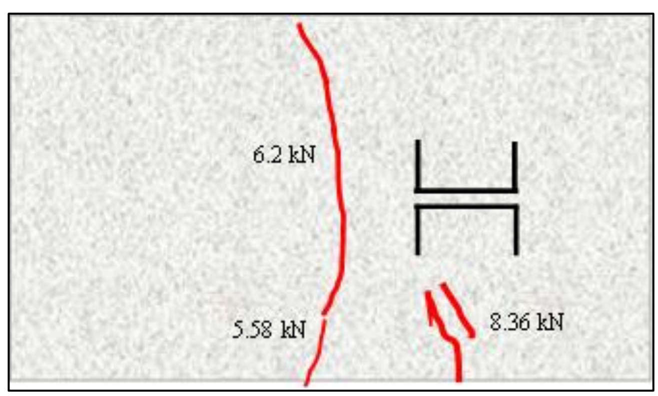


Figure 11 *Crack pattern of the concrete slab*

3.4 Moment vs. Rotation

The moment vs. rotation curve was obtained from the inclinometer data and moment of connection that calculated from point load multiply by lever arm of 1000 mm as Figure . Figure 12 presents the moment-rotation from experimental results. The curve of moment-rotation was mostly identical compare with the load-deflection curve. The high slope was observed at load below 5 kN, and this could be the bolts that were fully tightened. The curve starts to decrease afterload achieved more than 5 kN. This happens because of the crack of the concrete slab was taking place. The ultimate moment of the joint was reached at 31.6 kNm, and the rotation was 0.038 rad, so the connection could be classified as ductile because the ultimate rotation was more than 0.03 rad.The stiffness of connection of 730 kNm/rad was obtained from the linear part of the curve as shown Figure 12. From investigation, the gusset plate also rotates, but it was not accompanied by buckling on the plate and M12 bolts. It could be the bearing failure around the bolt holes was happen due to the thinness plate of the beam.

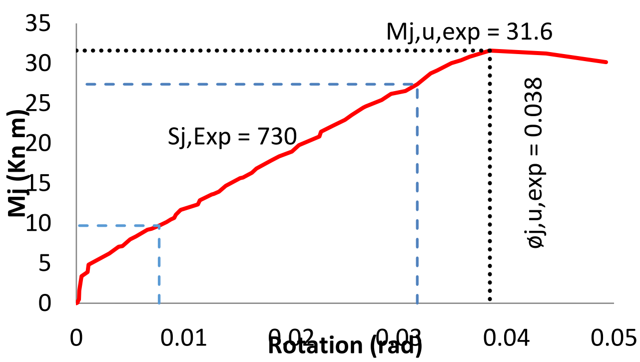


***Figure 12** Moment rotation of the connection*

3.5 Analytical Study

Based on the observation of the failure mode from the experiment, it was indicated that there was a failure of the bolt hole and bending on the flange under the column. As a result of these, it can be calculated the capacity moment connection (Mj) which is illustrated in Figure 12. F_{bolt} is the resistance of bolt group, F_{rebar} is a resistance of the main reinforcement bars of the floor plate, and F_{cwc} is a web column resistance when compression occurs.

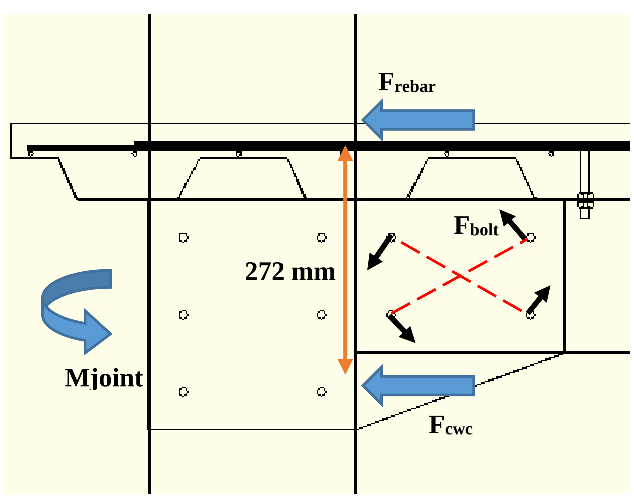


***Figure 13** Analytical of moment capacity of joint*

The resistance of the bolt group consists of shear and bearing that refer to BS EN 1993-1-3:2006[13]

Shear Resistance:

$$F_{V,Rd} = \frac{\alpha_V f_{ub} A_s}{\gamma_{M2}} = \frac{0.6\ x\ 800\ MPa\ x\ 84.3\ mm^2}{1.25} = 32.37 \text{ kN}$$

Where: α_V= 0.6 for bolt grade 8.8; f_{ub}= ultimate strength of bolt; A_s= stress area; γ_{M2}= 1.25

Bearing Resistance:

$$F_{b,Rd} = \frac{2.5\ \alpha_b k_t f_{u,beam} d_{bolt} t_{beam}}{\gamma_{M2}}$$

$$F_{b,Rd} = \frac{2.5\ x\ 1\ x\ 1\ x\ 420\ MPa\ x\ 12\ mm\ x\ 2.4\ mm}{1.25} = 24.19 \text{kN}$$

Where: α_b= 1; k_t= 1; $f_{u,beam}$= ultimate strength; d_{bolt}= the diameter of bolt; t_{beam}= thickness of beam; γ_{M2}= 1.25

Thus the bolt resistance (F_{bolt}) was taken as the minimum of $F_{V,Rd}$ and $F_{b,Rd}$ = 24.19 kN

The moment capacity of group bolt (M_{bolt})
M_{bolt} = 4 x (F_{bolt} x center of beam group bolt)
M_{bolt} = 4 x 24.19 kN x 112.48 mm = 10.88 kNm

The resistance of reinforcement bars (F_{rebar}):

$$F_{rebar} = f_y x\ A_{rebar} = 250\ MPa\ x \frac{1}{4} x\ 3.14\ x 12^2 = 28.26\ kN$$

The resistance of the column web (single section) was taken from the minimum of column web in crushing and buckling.

Crushing of column web:

$$F_{c,wc,Rd,c,1} = \frac{\omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M0}}$$

Where: $b_{eff,c,wc}$= effective width of web in compression; t_{wc}= the column web thickness; A_{vc}= the shear area of the column (single section); ω = reduction factor; $f_{y,wc}$= yield stress of web

$$b_{eff,c,wc} = t_{fb} + 2a_p\sqrt{2} + 5\left(t_{fc} + s\right) + s_p$$

$$b_{eff,c,wc} = 2.36 + 2(0)\sqrt{2} + 5(2.36 + 5) + 4.72 = 43.88\ mm$$

$$\omega_1 = \frac{1}{\sqrt{(1+1.3(b_{eff,c,wc}\ t_{wc}\ /A_{vc})^2}}$$

$$\omega_1 = \frac{1}{\sqrt{(1 + 1.3(43.88\ x\ \ 4.72\ /842.37)^2}} = 0.963$$

Refer to SCI P398, k_{wc} could be taken as 0.7

$$F_{c,wc,Rd,c,1} = \frac{0.963\ x\ 0.7\ x\ 43.88\ x\ 4.72\ x\ 350}{1} = 48.859 \text{ kN}$$

Buckling of column web :

$$F_{c,wc,Rd,b,1} = \frac{\omega\ \rho\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M1}}$$

$$\lambda_p = 0.932\sqrt{\frac{b_{eff,c,wc}\ d_{wc}\ f_{y,wc}}{E\ t_{wc}^2}}$$

$$\lambda_p = 0.932\sqrt{\frac{43.88\ x\ \ 285.28\ x\ 350}{210000\ \ x\ 4.72^2}} = 0.9019$$

ρ is the reduction factor for plate buckling and depend on plate slenderness factor λ_p

$\rho = \frac{\lambda_p - 0.2}{\lambda_p^2}$ if $\lambda_p > 0.72$

$$\rho = \frac{\lambda_p - 0.2}{\lambda_p^2} = \frac{0.9019 - 0.2}{0.9019^2} = 0.863$$

$$F_{c,wc,Rd,b,1} = \frac{0.963\ x\ 0.863\ x\ 0.7\ x\ 43.88\ x\ 4.72\ x\ 350}{1} = 42.160 \text{ kN}$$

So : $F_{c,wc,Rd,1}$= min of $F_{c,wc,Rd,c,1}$ and $F_{c,wc,Rd,b,1}$ = 42.160 kN

The moment capacity of rebar
M_{rebar} = 2 x F_{rebar} x 272 mm = 2 x 28.26 kN x 272 mm = 15.37 kNm

Moment capacity of column web in compression (Mcwc) = Fcwc x (272 mm) = 42.160 kN x 272 mm = 11.47 kNm

Moment capacity of joint (Mj) = Mbolt + (min of Mrebar or Mcwc) = 10.88 + 11.47 = 22.35 kNm

Table 3 presented the result of the analytical calculation was 22.35 kNm that influenced the resistance of bolt, rebar and column web. Table 3 shows the ratio of the moment experimental with analytical calculations showing a good agreement of 1.42.

TABLE III. THE RATIO OF MOMENT

Moment experiment	Moment analytical	Ratio
31.6 kNm	22.35 kNm	1.41

The comparison of test results with other studies is shown in Table 4. The sample used has the same column size. The beam size used is the same but varied in thickness. The results of the current study's moment experiment were more excellent than Aminuddin's research with a ratio of 0.49, which indicates that the composite structure can resist heavy loads due to the function of reinforcement bars. In the current study, the value of the stiffness experiment is higher than the study of Aminuddin with a ratio of 0.33, which means that the composite structure is rigid than the non-composite structure.

TABLE IV. THE COMPARISON WITH OTHER STUDY

Comparison	Current study	Aminuddin's study [6]
Beam section	C20024	C20019
Column section	C30024	C30024
Moment experiment (kNm)	31.6	15.68
Ratio of moment experiment	1.00	0.50
Stiffness experiment (kNm/rad)	730	333
Ratio of Stiffness experiment	1.00	0.46
Type of structure	Composite	Non composite

4 CONCLUSION

The experiment of the isolated joint test was conducted successfully. From this study, make us obtain the following **conclusions:**

1. This failure mode can be indicated that there is an influence of the sliding bolt resistance and bolt support on the joint resistance
2. The recording results show the ultimate load achieved is 31.6 kN with 62.26 mm deflection
3. The ultimate moment occurs is 31.6 kNm with a rotation of 0.038 rad and 730 kNm/rad of stiffness
4. The ratio was of the moment experimental with analytical calculations showing a good agreement of 1.41. The ratio between other studies has 0.50 ratio of moment resistance and 0.46 for stiffness.

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