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

by Anis Saggaf

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

Abstract:— Connections on the cold-formed steel (CFS) section become an interesting topic concerning its failure mode that the thin-walled behaviour controls it. CFS weakness is usually caused by early local buckling and local instability. The bolt connections and the gusset plate are the easy way to install a joint of steel structures on CFS application. This study is aimed to investigate the behaviour of the CFS connection of double lipped channel C-sections of C20019 for the beam and C30024 for the back-to-back column with a slip-in haunched gusset plate. The haunched gusset plate is S275 steel grade, 6 mm thickness connected with M12 bolt. The experiment was carried out by isolated joint test to acquire moment capacity and rotation of the connection. It is observed that the connection failure modes were on the beam bolt and the local buckling on the column flange. The existence of the slip-in gusset plate reduces the buckling effect induced by the behaviour of the thin plate. The moment of joint (Mj) obtained from the experiment was 20 kNm, the stiffness (Sj) was 250 kNm/rad, and the rotation (Π) at the final load was 0.079 radian. The comparison moment of joint by analytical and experimental was 0.78, while the ratio of stiffness was 2.04. The analytical and experimental comparison between moment of joint and moment resistance of beam was 0.55. It concluded that the connection was categorized as partial strength and ductile connection for cold-formed steel section application.

Index Terms:— Cold-formed steel; gusset plate; connection; partial strength; moment of joint

1 INTRODUCTION

In recent centruies, the cold-formed steel (CFS) section was used as a secondary structure, including rails, roof purlins, wall cladding and even residential buildings at the middle level [1], [2]. Connections between cold-formed steel members are generally considered to be shear connections with two bolts per member [3] while some can be created of hot-rolled steel combined with bolts and angle cleats [4]. However, some of the studies have proven that early local buckling and instability mostly happen at the CFS [5], [6], because of the thin-walled behaviour. In recent years, the experiments of light steel joints combine with gusset plates and bolts have been carried out intensively by several researchers, due to the limited design guidelines in Eurocode 3 for this type of connections. The choice of bolts, besides being compatible with gusset plates, it is also relatively easier and requires no special skills to install compared to welding joints [7]. Tan in [8] and Aminuddin in [9] have experimented the slip-in gusset plate in rectangular shape with 6 mm of thickness for cold-formed steel connections using back-to-back double lipped channel C-sections. The same beam size of 200 mm was used for both experiments, while the column depth was different with a depth of 250 mm and 300 mm respectively. From the study, the moment of joints was almost the same with 14.27 kNm and 15.68 kNm. Bucmys [7] also investigated the cold-formed connection using T-shape gusset plate 12 mm of thickness. The joint capacity was 25 kNm. The study as in [7] – [9] show that the slip-in gusset plates can increase cold-formed steel joint capacity to achieve the behaviour of partial strength connection. The CFS joints with haunched gusset plate have been studied by Bucmys as in [10] with the C-profile section, and by Sabbagh with the curve flange profile section [11]. The two researchers only investigated the behaviour of the connection based on a numerical study using finite element analysis (FEA). Based on the experimental and numerical studies, it can be said that the shape and the thickness of the gusset plate could influence the moment of joint. The presence of a slip-in gusset plate could reduce the buckling effect of the cold-formed cross-section caused by the behaviour of thin plates. This paper covers the experimental investigation of slip-in haunched gusset plate connection for cold-formed steel sections on beam and column joint, where not many researchers have studied in-depth. The experiment was done on isolated joint, with the purpose to obtain the moment-rotation and stiffness of the connections and validated with other researchers. The proposed connections were expected to classify as partial strength connections.

2 THE MATERIAL AND METHODS

The experiment on the specimen was carried out on isolated joint test where the cantilever beam has lateral restraint, and the end of the beam has loaded by a load cell. The specimen was installed by fabricating 3 meters length column with 1.1 meters length beam. The joint between beam and column used hot-rolled haunched gusset plate in steel grade S275 and the thickness of 6 mm. The nominal yield stress (fy) and the ultimate tension stress (fu) were 275 MPa and 430 MPa, respectively. The isolated joint was executed to obtain the characteristic of the connection, i.e. moment of joint (Mj) and stiffness (Sj) [12]. This study was held in Steel Structure Laboratory of Civil Engineering Department, Universitas Sriwijaya. The preparation of the specimen is presented in Fig.1. The cold-formed steel G450 was used with yield stress

• *Muhammad Firdaus is PhD. student of Civil Engineering, Faculty of Engineering, Sriwijaya University, Indonesia. Corresponding Email: firdaus@univpgri-palembang.ac.id*
• *Anis Saggaff, KM Aminuddin, and Saloma Civil Engineering Department, Faculty of Engineering, Sriwijaya University, Indonesia.*
• *Mahmood Md Tahir, and Shek Poi Ngian, Construction Research Centre (CRC), Faculty of Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor Bahru, Malaysia*
• *Fathoni Usman, Institute of Energy Infrastructure, Universiti Tenaga Nasional, 43000, Kajang, Selangor, Malaysia*

(fy) 450 MPa and ultimate tension stress (fu) = 480 MPa. The section of double lipped channel was installed back-to-back to increase the moment inertia and to delay the local buckling [6]. The section of the beam is C20019 with 200 mm of depth, 76 mm of width, 1.9 mm of thickness and 1.1 m of length, while the column section was C30024 with 300 mm of depth, 96 mm of width, 2.4 mm of thickness and 3 m of length. All of the bolts grade 8.8 M12 were installed with yield stress (fy) = 640 MPa and ultimate tension stress (fu) = 800 MPa, refers to a study in [13]. Two inclinometers were located at the beam (Inc-beam) and column (Inc-col) to measure and record the rotation of the connection. Four LVDT were positioned at the beam (LVDT-2 and LVDT-1) and column (LVDT-3 and LVDT-4) to obtain the horizontal and vertical displacement of specimens. The configuration of the haunched gusset plate is presented in Fig. 2. Table I shows the dimension of connections and the position of the bolt. The distance between bolts was arranged according to the spacing requirements by EC3 [14].

TABLE 1

THE DISTANCE OF CONNECTIONS (MM)

Bg	*Lg*	*e1*	*e2*	*p1*	*p2*	*tg*
409	*600*	*50*	*50*	*200*	*200*	*6*

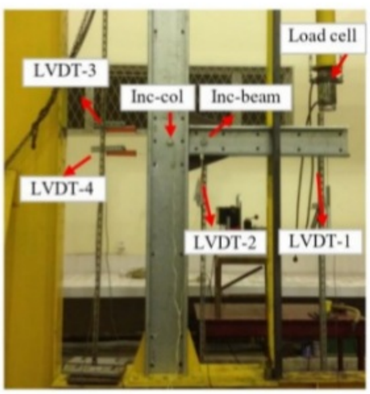


Fig. 1. *Isolated joint test configuration*

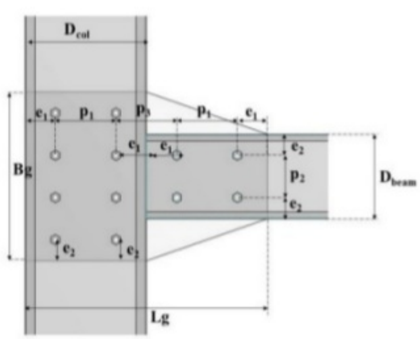
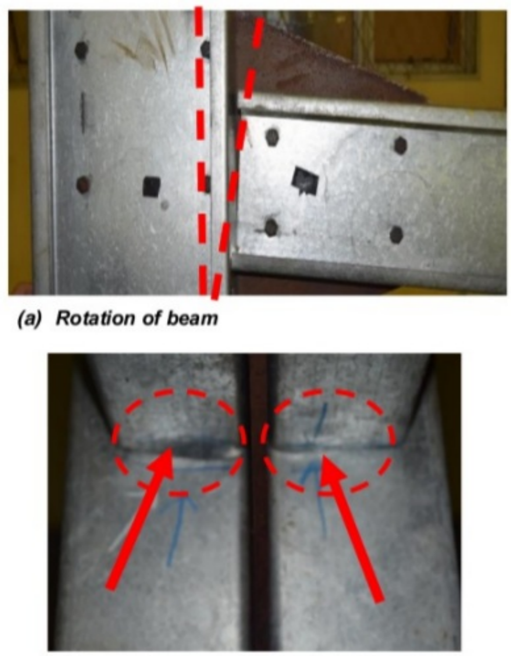


Fig. 2. *The dimension of connections*

3 Results and Discussion

3.1 Failure modes

The test carried out by applying a point load that gradually increased by a hydraulic jack. Visual observation was carried out throughout the test. Fig. 3 (a) shows the beam rotated after the ultimate load reached 20 kN. The beam rotates, causing the bolt group to react so that the elongation occurs around the bolthole. Besides, the gusset plate does not deform significantly when compared to the beam. The beam rotation caused the column flange to be compressed at the bottom beam flange. The height compression stress was developed at the compression zone caused local buckling at column flange (Fig. 3 (b)). Fig. 4 shows the failure mode of the beam bolthole. Elongation of boltholes (i.e. red arrow in Fig. 4) indicates the bearing failure of the bolt due to the thickness of CFS that only 2 mm.



(a) Rotation of beam

(b) The compression at the bottom flange column

Fig.3 *Deformation of the specimen*

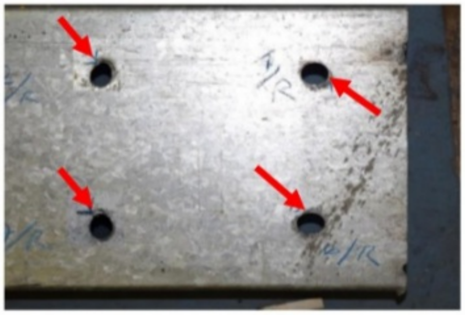


Fig. 4. The elongation of beam bolt hole

3.2 Load vs Deflection Curve

Load vs. deflection curves was obtained from LVDT-1 and load cell data that directly recorded by the data logger. Fig. 5 shows a height slope at a range of 1 – 4 kN. It could be due to bolts were fully tightened.

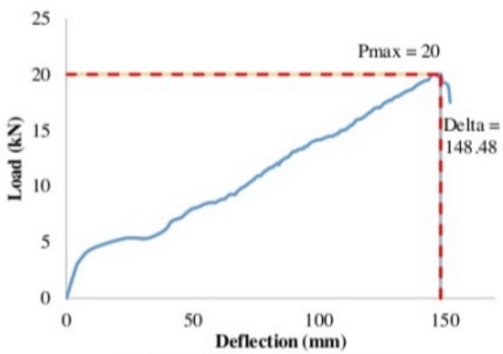


Fig. 5. Load vs deflection curve

The slope started to decrease after the load increased until the final load of 20 kN was reached. It was shown that the relationship between load-deflection is non-linear. The maximum deflection before the graph become plateau or decrease is 148.48 mm.

3.3 Moment vs rotation curve

Fig. 6 presents the moment-rotation curves of the connection. The moment was obtained by multiplying the load (P) with a distance of about 1 m from the load cell to the face of the column flange. The rotational data were taken from the difference of beam rotation and column rotation in radians. The graph shows that the ultimate moment of the joint was 20 kNm and the rotation of 0.079 rad. The stiffness was taken from the slope of the line with a result of 250 kNm/rad. The test object was categorized as ductile as the elastic rotation is more than 0.03 rad.

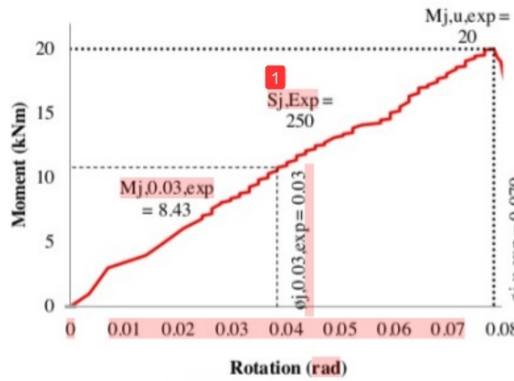


Fig. 6. Moment vs rotation curve

3.4 Analytical calculation

Based on observations of the failure modes, the elongation of beam bolt holes was clearly visible. The failure implies that the minimum resistance of the joint occurs at the group of bolts on the beam. Therefore, the analysis can be simplified by determining the moment resistance induced by the group of bolts on the beam as shown in Fig. 7. According to Fig. 7, the joint resistance is limited by shear and bearing resistance of the bolts [15]. The shear resistance of bolt can be calculated as:

$$F_{V,Rd} = \frac{\alpha_V f_{ub} A_s}{\gamma_{M2}} \quad (1)$$

$$\boldsymbol{F_{V,Rd}} = \frac{\mathbf{0.6\,x800\,MPa\,x\,84.3\,mm^2}}{\mathbf{1.25}} = 33.17\ \text{kN}$$

Where: $\boldsymbol{\alpha_V}$= 0.6 for bolt grade 8.8; $\boldsymbol{f_{ub}}$= ultimate strength of bolt; $\boldsymbol{A_s}$= stress area; $\boldsymbol{\gamma_{M2}}$= 1.25

The bearing resistance depends on the thickness connected part with the equation:

$$F_{b,Rd} = \frac{2.5\,\alpha_b k_t f_{u,beam} d_{bolt} t_{beam}}{\gamma_{M2}} \quad (2)$$

$$\boldsymbol{F_{b,Rd}} = \frac{\mathbf{2.5\,x\,1\,x\,1\,x\,480\,MPa\,x\,12\,mm\,x\,2.4\,mm}}{\mathbf{1.25}} = 27.65\text{kN}$$

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

Thus, the resistance of bolt (F_{bolt}) is the minimum of $F_{b,Rd}$ and $F_{V,Rd}$.

F_{bolt} = min (33.17, 27.56) = 27.56 kN

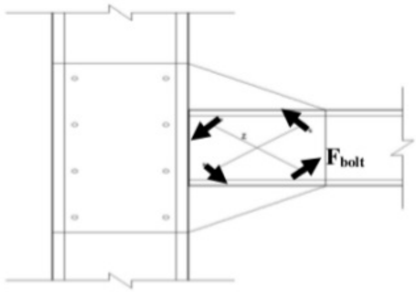


Fig. 7. Moment of joint

The resistance of joint representing by moment-resistance is:
Moment joint (Mj) = number of bolt x $F_{b,Rd}$ x lever arm (3)

Moment joint (Mj) = 4 x 27.56 x 141.42 = 15.64 kNm.

The calculation of joint stiffness (Sj) for the haunched gusset plate was referred to Klich in [16]. The method of calculation was divided into three groups:

a. The stiffness of column bolt group (Sc,bg,ini)
b. The stiffness of beam bolt group (Sb,bg,ini)
c. The rotational stiffness of the gusset plate (Sgp,ini)
d. The stiffness of column bolt group (Sc,bg,ini)

The spring models were presented with built from three component groups: i) bolt in shear (k_{11}); ii) bolt-in bearing of CFS (k_{12b}); iii) bolt in bearing for gusset plate ($k_{12,g}$) [14]. Fig. 8 shows the component method of the stiffness column bolt group.

The stiffness coefficient of bolt-in shear as in Eq. (4):

$$k_{11,c} = \frac{16\, c_s\, n_b\, d^2\, f_{ub}}{E\, d_{M16}} \qquad (4)$$

Where : c_s= number of shear plane; n_b= 0.5 for single bolt; d = diameter of bolt; f_{ub}= ultimate tensile strength; E = the elastic modulus; d_{M16}= nominal diameter of M16 bolt

$$k_{11,c} = \frac{16\ x\ 2\ x\ 0.5\ x\ (12\ mm)^2\ x\ 800\ MPa}{210000\ MPa\ x\ 16\ mm} = 0.549$$

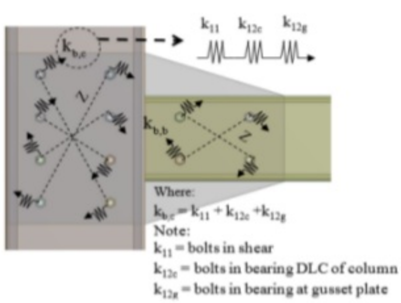


Fig. 8. Mechanical models of the stiffness of column bolt group

The stiffness coefficient corresponds to the bolts in bearing for column stiffness is calculated as follows:

$$k_{12,c} = \frac{24\, n_b\, k_b\, k_t\, d\, f_u}{E} \qquad (5)$$

$$k_t = \frac{1.5\, t_j}{d_{M16}} \leq 2.5 \qquad (6)$$

Where: n_b = 0.5 for single bolt; k_b = 1.25; k_t=1.5x2x1.9/16≤2.5=0.356 ; d= diameter of bolt; f_u= ultimate tensile strength of steel.

$$k_{12,c} = \frac{24\ x\ 0.5\ x\ 1.25\ x\ 0.356\ x\ 12\ mm\ x\ 450\ MPa}{210000\ MPa} = 0.137$$

Bolt-in bearing for gusset plate:

$$k_{12,gc} = \frac{24\, n_b\, k_b\, k_t\, d\, f_u}{E} \qquad (7)$$

Where: n_b= number of bolt (i.e. 0.5 for one bolt); k_b= 1.25; k_t = 1.5x6/16 < 2.5 = 0.563; d = diameter of bolt; f_u= ultimate tensile strength of gusset plate

$$k_{12,gc} = \frac{24\ x\ 0.5\ x\ 1.25\ x\ 0.563x\ 12\ mm\ x\ 430\ MPa}{210000\ MPa} = 0.208$$

The equivalent of the stiffness coefficient:

$$\sum \frac{1}{k} = \frac{1}{0.549} + \frac{1}{0.137} + \frac{1}{0.208} = 13.928$$

The equation of stiffness of the column bolt group is shown in Eq. (8):

$$S_{c,bg,ini} = \frac{E\, z^2}{\sum \frac{1}{k}} \qquad (8)$$

Where: E = the elastic modulus; z = lever arm of the bolt

The outcome of the column bolt group's stiffness used Eq. 8 and presented in Table 2 below.

TABLE 2
THE STIFFNESS CALCULATION OF COLUMN BOLT GROUP

Bolts	*E (MPa)*	*Z (mm)*	$\frac{1}{\sum k}$	$S_{C,bg,ini}$ *(kNm/rad)*
1	*210000*	*167.705*	*13.928*	*424.055*
2	*210000*	*167.705*	*13.928*	*424.055*
3	*210000*	*90.139*	*13.928*	*122.506*
4	*210000*	*90.139*	*13.928*	*122.506*
5	*210000*	*90.139*	*13.928*	*122.506*
6	*210000*	*90.139*	*13.928*	*122.506*
7	*210000*	*167.705*	*13.928*	*424.055*
8	*210000*	*167.705*	*13.928*	*424.055*
		Total		*2186.244*

Hence, the stiffness of the group of column bolts ($S_{C,bg,ini}$) was 2186.244 kNm/rad.The calculation of Sb,bg,ini is similar to the Sc,bg,ini. Fig. 9 presented the component method of the stiffness of the column bolt group.

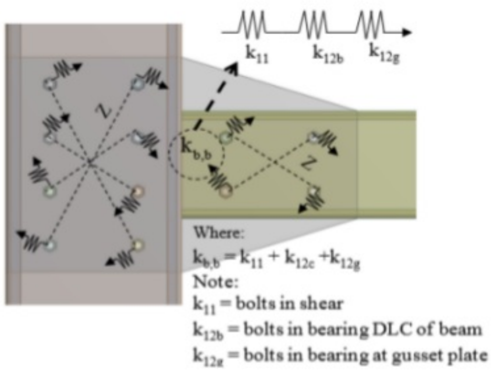


Fig. 9. *Mechanical models of the stiffness of beam bolt group*

Bolt-in shear as follows in Eq. (4).

$$k_{11,b} = \frac{16\ x\ 2\ x\ 0.5\ x\ (12\ mm)^2\ 800\ MPa}{210000\ MPa\ x\ 16\ mm} = 0.549$$

Bolt-in bearing for beam stiffness as follows in Eq. (5) and Eq. (6).

$$k_{12,b} = \frac{24\ x\ 0.5\ x\ 1.25\ x\ 0.356\ x\ 12\ mm\ x\ 450\ MPa}{210000\ MPa} = 0.137$$

Bolt-in bearing for haunched gusset plate as follows in Eq. (7).

$$k_{12,gb} = \frac{24\ x\ 0.5\ x\ 1.25\ x\ 0.563\ x\ 12\ mm\ x\ 430\ MPa}{210000\ MPa} = 0.208$$

The equivalent of the stiffness coefficient:

$$\Sigma\frac{1}{k} = \frac{1}{0.549} + \frac{1}{0.137} + \frac{1}{0.208} = 13.928$$

The equation of stiffness of the beam bolt group as follows:

$$S_{b,bg,ini} = \frac{E\,z^2}{\Sigma\frac{1}{ki}} \qquad (9)$$

The outcome of the beam bolt group's stiffness was shown in Table 3 below.

TABLE 3

THE STIFFNESS CALCULATION OF BEAM BOLT GROUP

Bolts	*E (MPa)*	*z (mm)*	$\frac{1}{\sum k}$	$S_{b,bg,ini}$ *(kNm/rad)*
1	*210000*	*90.139*	*13.928*	*122.506*
2	*210000*	*90.139*	*13.928*	*122.506*
3	*210000*	*90.139*	*13.928*	*122.506*
4	*210000*	*90.139*	*13.928*	*122.506*
		Total		*490.022*

Hence, the stiffness of the group of beam bolts ($S_{b,bg,ini}$) was 490.022 kNm/rad. The analysis of the rotational stiffness of the gusset plate has not been found in EC3. The calculation referred to Bucmys work in [7]. Fig. 10 shows haunched gusset plate rotation induced by point load (P) and moment (M). Furthermore, ϕ_1 is rotation due to the beam's vertical load (P), ϕ_2 is rotation due to moment on the beam (Mb) and ϕ_3 is rotation due to moment on the column (Mc). The rotation of the gusset plate is a total of ϕ_1, ϕ_2 and ϕ_3 as in Eq. (14).

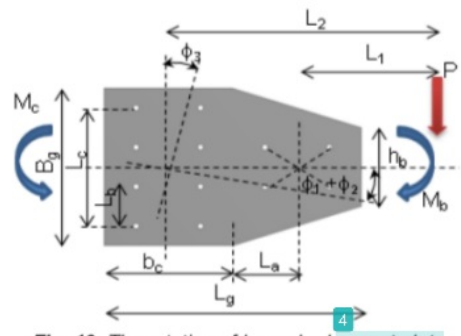


Fig. 10. *The rotation of haunched gusset plate*

Therefore, the rotation of the gusset plate as follows:

$$M_b = PL_1 = 20\ kN\ x\ 850\ mm = 17\ kN.m$$
$$M_c = PL_2 = 20\ kN\ x\ 1150\ mm = 23\ kN.m$$

Where: P was taken from the ultimate load; L_1 and L_2 = the distance, as shown in Fig. 10.

$$\phi_1 = \frac{PL_a^2}{E\,I_b} (10)$$

$$\phi_1 = \frac{20kN.(150\ mm)^2}{210000\ MPa\ .9.87x10^6\ mm^4} = 0.000213\ Rad$$

$$\phi_2 = \frac{M_b L_a^2}{E\,I_b} \qquad (11)$$

$$\phi_2 = \frac{17\ kNm\ .(150\ mm)^2}{210000\ MPa\ .\ 9865749.419\ mm^4} = 0.00109\ Rad$$

$$\phi_3 = \frac{M_c L_b^2}{L_c E\,I_c} \qquad (12)$$

$$\phi_3 = \frac{23\ kNm\ .(50\ mm)^2}{300\ mm\ .210000\ MPa\ .\ 2.67x10^7\ mm^4} = 3.42\ x\ 10^{-5}\ Rad$$

Where: L_b and L_c = the distance on Fig. 10; I_b= moment inertia of beam; I_c= moment inertia of column; E = the elastic modulus. Thus, the total rotation of the gusset plate:

$$\phi_g = \phi_1 + \phi_2 + \phi_3 \qquad (13)$$

$$\phi_g = 0.000213\ Rad + 0.00109\ Rad + 0.0000342\ Rad$$
$$= 0.00134\ Rad$$

The rotational stiffness of the gusset plate as follows equation:

$$S_{gp,ini} = \frac{M_c}{\phi} = \frac{M_c}{\phi_1+\phi_2+\phi_3} \quad (14)$$

$$S_{gp,ini} = \frac{23\ kN.m}{0.00134\ Rad} = 17164.178\ kNm/Rad$$

The stiffness joint of haunched gusset plate was calculated using:

$$S_{j,ini} = \frac{1}{\frac{1}{S_{b,bg,ini}}+\frac{1}{S_{c,bg,ini}}+\frac{1}{S_{gp,ini}}} \quad (15)$$

$$\boldsymbol{S_{j,ini}} = \frac{\mathbf{1}}{\frac{\mathbf{1}}{\mathbf{2186.244}}+\frac{\mathbf{1}}{\mathbf{490.022}}+\frac{\mathbf{1}}{\mathbf{17164.178}}}$$

$$\boldsymbol{S_{j,ini} = 391.183\ kNm/Rad}$$

3.4 Discussion

The result of the experiment was compared with other researcher findings and presented in Table 4 and Table 5. Table IV presents the information about the shape and thickness of gusset plates that have been carried out by previous research and the current study. As shown in Table V, Aminuddin in [9] has experimented using a rectangular gusset plate; the ratio with the current study were 1.28 for moment joint and 0.12 for stiffness. The comparison with Tan in [8] presented a ratio of 1.40 for moment joint and 1.02 for stiffness. Tan used a rectangular gusset plate for connection beam-to-column but have different depth of column. A similar failure mode with the current study is found as well in [8] and in [9]. It is concluded that the local buckling at the column flange leads the need of the reinforcement at compression zone. Besides, the result in [9] also presented the bearing failure of beam boltholes. Based on the comparison as Table 5, it is found that the shape of the gusset plate could influence the joint performance as the study result in [17] and in [10].

TABLE 4

COMPARISON MATERIAL WITH OTHER STUDIES

Comparison	Type of connection	The gusset plate's thickness (mm)	Depth of beam (mm)	Depth of column (mm)
Aminuddin (2017) Result	Rectangular gusset plate	6 mm	200	300
Tan (2011) Result	Rectangular gusset plate	6 mm	200	250
Current Study	Haunched gusset plate	6 mm	200	300

TABLE 5

COMPARISON RESULT WITH OTHER STUDIES

Comparison	Mj, exp (kNm)	Ratio Mj,exp	Sj, exp (kNm/rad)	Ratio Sj,exp
Aminuddin (2017) Result	15.68	1.28	1948.06	0.12
Tan (2011) Result	14.27	1.40	245	1.02
Current Study	20	1.00	250	1.00

To verify the experiment results, it is necessary to compare with analytical results based on failure modes observations. Table 6 and Table 7 presented the comparison of the moment of joint from the experiment with analytical calculation and bending capacity of the beam. The ratio of Mj between analytic calculation and experimental results shows a good agreement with ratio value 0.78 while the ratio of stiffness joint (Sj) between analytical calculation and experiment was 1.56. Accordance with EC3, the partial strength classification was ranged by 25% Mcx < Mj < Mcx, where Mcx is the moment resistance of beam. The ratio for moment of joint experiment (Mj) and the moment resistance of beam (Mcx) was 0.55, which is more than 25% of the beam capacity. The connection is validated as partial strength.

TABLE 6

COMPARISON WITH ANALYTICAL RESULT AND MOMENT RESISTANCE OF THE BEAM

Moment of Joint (Mj) kNm		Ratio Mj,the/Mj,exp	Moment Resistance of Beam (Mcx) kNm	Ratio Mj,exp/Mcx
Experiment Mj,exp.	Analytical Mj,the.			
20	15.64	0.78	36,611	0,55

TABLE 7

COMPARISON WITH ANALYTICAL RESULT

Stiffness Joint (Sj) kNm		Ratio Sj,the/Sj,exp
Experiment Sj,exp	Analytical Sj,the	
250	391.183	1.56

4 CONCLUSION

The experiment of a slip-in haunched gusset plate connection on CFS section beam to column joint has successfully done. It can be concluded that:

1. The connection failure modes are located on the beam bolthole and the local buckling at the bottom of the column flange.
2. The ratio moment of joint (Mj) between the other studies have ranged from 1.28 to 1.40 while the ratio of stiffness is ranged between 0.12 to 1.02.
3. The comparison of analytic and experimental for moment resistance of joint was 0.78, and the ratio of stiffness was 1.56
4. The elastic rotation was found beyond of 0.003 radians. The ratio for moment resistance of joint (Mj) to moment resistance of beam (Mcx) was 0.55, therefore the slip-in haunched gusset plate with cold-formed steel section beam was classified as ductile and partial strength connections.

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