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Experimental Behaviour of Beam-Column Connection using Cold-Formed Steel Sections with Rectangular Gusset-Plate

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Abstract. Beam-to-column connections setting up as isolated joint of cold-formed steel sections were tested up to failure. This experiment was conducted to observe the behaviour of connection in term of strength, stiffness and ductility. The type of connection used was rectangular gusset plate which stiffen the beam-to-column connection. The behaviour of the proposed connection was expressed with Moment-Rotation curves plotted from the experiment test results. The capacity of connections on this research were done in two ways: theoretical calculation by adopting Eurocode 3 BS EN 1993-1-8:2005 and experimental test results. The theoretical calculation of the moment capacity of the proposed connection has found (Mj) to be 10.78 kNm with joint stiffness (Sj) amount to 458.53 kNm/rad. The experimental test results has recorded that the Moment capacity (Mj) of 15.68 kNm with joint stiffness (Sj) of 1948.06 kNm/rad. The moment ratio of theoretical to experimental amount to 0.69. The joint stiffness ratio of theoretical to experimental amount to 0.24.

INTRODUCTION

Gusset-plate connections are the easiest connections used for beam-to-column connection [1]. Gusset-plate connections have advantages than other connections that are easy to install and maintain. Gusset-plate connections contain various shape such as haunch and rectangular. Rectangular gusset-plate connection is compared to haunch gusset-plate has advantage that is due to its shape adjusted with beam and column height so there is no excess of gusset-plate. If it is applied to building system for wall's erection method, rectangular gusset-plate connection is easier to install compare to haunch gusset-plate connection.

Tan was conducted the research of double lipped C-Channel (DLC) with non-composite connection by applying gusset plate [2] and flange cleats [3]. The dimension of the beam varies, ie DLC150, 200, and 250, while the column dimension remains the same i.e. DLC250. From the research result it can be concluded that the gusset plate connection has better connection capacity compared to flange cleat. This means by using the same beam, the gusset plate connection configuration can be more applied to larger loads compare to the flange cleats. The use of cold-formed steel as part of the main construction can provide advantages due to having a highest strength-to-weight ratios. Nevertheless, the innovations concerning of cold-formed steel is remain. such as the research conducted by Sabbagh ([4]; [5]) proposing the use of a curved flange section of cold-formed steel, with the aim of increasing the moment resistant with the same weight profile.

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This paper presents the behaviour of a rectangular slip-in gusset plate connection using DLC200 for beam and DLC300 for columns. The connection behaviour expressed by the moment of rotation will be discussed in this publication.

METHODOLOGY

The capacity of connections is compared using two calculation methods that are theoretical calculation and experimental test results. Eurocode 3 BS EN 1993-1-8:2005 [4] was used in theoretical calculation to obtain the capacity of connections and joint stiffness. Isolated joint test is used in experimental test to obtain the capacity of connections and joint stiffness. Moment and joint stiffness of theoretical calculation are compared to experimental test result so it is seen ratio of moment and joint stiffness between all these calculation methods. Furthermore, it is seen moment – rotation curve comparison between these calculation methods.

Cold-formed steel sections (CFS) with G450 of yield strength (f_y = 450 MPa) and tensile ultimate strength (f_u = 480 MPa) were used for beam and column section. Column of size C30024 cold-formed steel section where height 300 mm, width 96 mm and thickness 2.4 mm. The beam of size C20019 cold-formed steel section with height 200 mm, width 76 mm and thickness 1.9 mm. Hot-rolled material grade S275 was used for gusset-plate with yield strength (f_y = 275 MPa) and tensile ultimate strength (f_u = 430 MPa), while grade 8.8 material used for M12 bolt and nut with yield strength (f_y = 640 MPa) and tensile ultimate strength (f_u = 800 MPa). Figure 1 shows the specimen connection configuration, the dimension and connection configuration of rectangular gusset-plate connection is shown in Table 1.

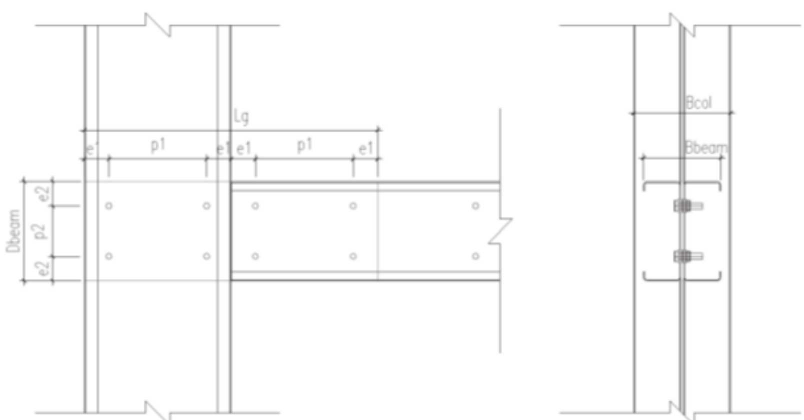


FIGURE 1. The specimen connection configuration

TABLE 1. Dimension and connection configuration of rectangular gusset-plate connection (mm)

D_b	B_b	D_c	B_c	L_g	t_g	e_1	p_1	e_2	p_2
203	152	300	192	600	6	50	200	50	103

RESULTS

Theoretical results

The theoretical moment capacity of connections and joint stiffness established from Eurocode 3 BS EN 1993-1-8:2005 are discussed as follows. Shear capacity was calculated with Equation 1.

$$F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A_{sb}}{\gamma_{M2}} \tag{1}$$

The values of α_v and γ_{M2} based on BS EN 1993-1-8:2005. Moment capacity was taken from the smallest value of Equation 2 and 7. In equation 2, χ_{LT} is reduction factor for lateral-torsional buckling, W_y is elastic section modulus of effective cross section, f_{yb} is yield strength of cold-formed steel and γ_{M0} is *partial factor*. The value of γ_{M0} based on BS EN 1993-1-1:2005. The values of χ_{LT} and W_y are calculated with Equation 3 and 4.

$$M_{b,Rd} = \frac{\chi_{LT} \cdot W_y \cdot f_{yb}}{\gamma_{M0}} \tag{2}$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{LT}^2}} \leq 1 \tag{3}$$

$$Wy = \frac{I_{eff}}{(0.5 \cdot h_w)} \tag{4}$$

Φ_{LT} is the value to calculate reduction factor for lateral-torsional buckling. λ_{LT} is non-dimensional slenderness factor for lateral-torsional buckling. I_{eff} is the second moment of area of effective cross section. The values of Φ_{LT} and λ_{LT} are calculated with Equation 5 and 6. α_{LT} is imperfection factor for lateral-torsional buckling with the value based on BS EN 1993-1-8 :2005.

$$\Phi_{LT} = 0.5 \cdot \left(1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2\right) \tag{5}$$

$$\lambda_{LT} = \sqrt{\frac{W_y \cdot F_y}{M_{cr}}} \tag{6}$$

$$M_g = \sum F_i \cdot L_i \tag{7}$$

Equation 2 is used to calculate the elastic critical moment capacity for lateral-torsional buckling and Equation 7 is used to calculate moment capacity due to capacities of bolts group. The capacity of connections was calculated using Equation 1 through 7. As a result, the shear and moment capacity of the proposed connection is shown in Table 2. The joint stiffness of rectangular gusset-plate connection is obtained from relation between moment and rotation can be seen in Equation 8. Table 3 show the joint stiffness of theoretical calculation result. The moment–rotation curve for theoretical calculation result is shown in Figure 2.

TABLE 2. The capacity of rectangular gusset-plate connection of theoretical calculation result

Column section	Beam section	Shear resistance, F_v (kN)	Moment resistance, M_j (kNm)
C30024	C20019	129.49	10.78

$$S_j = \frac{S_{j,ini}}{\eta} \leq 1 \quad and \quad \theta = \frac{M_j}{S_j} \leq 1 \tag{8}$$

TABLE 3. The joint stiffness of rectangular gusset-plate connection of theoretical calculation result

Column section	Beam section	Load, P (kN)	Stiffness, $S_{j,ini}$ (kNm/Rad)	Rotation, θ (mRad)
C30024	C20019	10	458.53	47.00

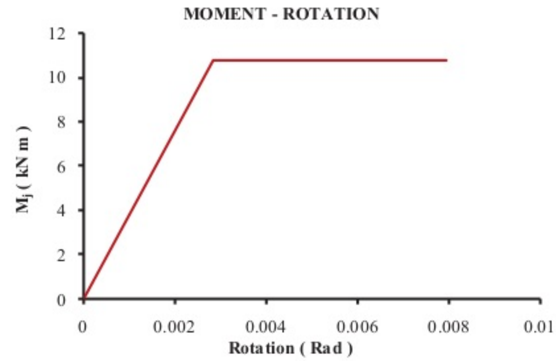
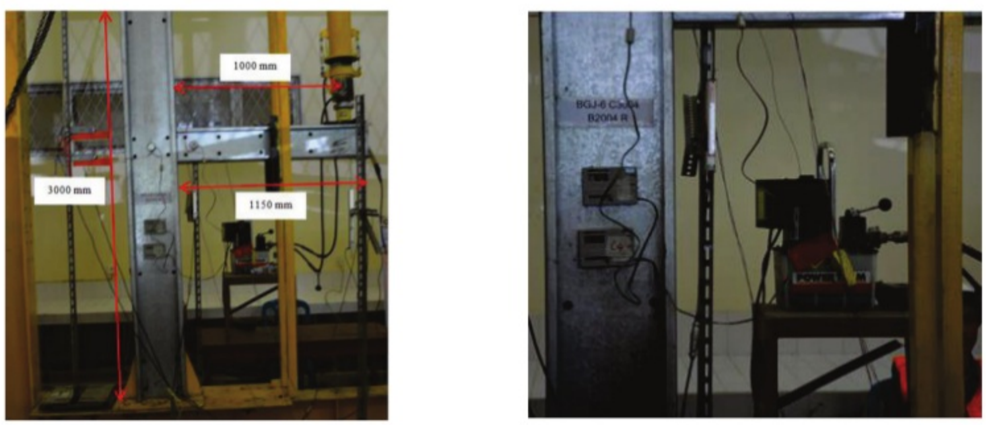


FIGURE 2. Moment – rotation relationship curve of theoretical calculation result

Experimental Test

Full-scale of isolated joint test was conducted to establish moment capacity and initial stiffness of the proposed connection. The configuration of specimen of experimental test is shown in Figure 3a. The configuration of full-scale isolated joint test was recorded as 3000 mm for column length, 1150 mm for beam length, and the load cell which was placed on beam with 1000 mm distance from column face. Four LVDT was used to observe deformation of specimen, two LVDT placed on column and two LVDT placed on beam. The specimen loaded with one-third loading of theoretical analysis, then unloaded to see residual deformation which occurs on the specimen. Figure 3b shows the initial condition of specimen with all of LVDT and inclinometers. In initial condition, inclinometers show 0.23 degree on column and 0.27 degree on beam, so it obtains rotation of connection in initial condition amount to 0.04 degree (0.698 mRad).



a) b)

FIGURE 3. The initial condition of specimen

Figure 4a shows the condition on one-third loading amount to 3.04 kN. In inclinometers show 0.26 degree on column and 1.02 degree on beam, so it obtains rotation of connection on one-third loading amount to 0.76 degree (13.265 mRad). Figure 4b shows the condition after unloading. In inclinometers show 0.24 degree on column and 0.94 degree on beam, so it obtains rotation of connection after unloading amount to 0.70 degree (12.217 mRad).

a) The condition on one-third loading b) The condition after unloading

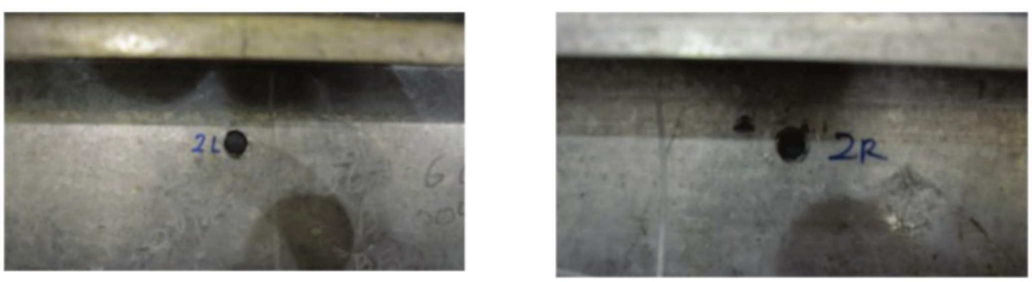
FIGURE 4. The condition after loading and unloading

Loads are applied slowly and steadily increasing until structural failure occurs. In this experimental test, the failure mode is torsional buckling which is shown in Figure 5a. In inclinometers shows the rotation about 0.56 degree on the column and 3.38 degree on the beam, so it obtains rotation of connection on failure mode amount to 2.82 degree (49.218 mRad). Furthermore, Figure 5b shows deformation mode in gusset-plate which shows that the stress has exceeded yield strength of gusset-plate.

a) Failure mode on structure (torsional buckling) b) Deformation mode that occur in gusset-plate

FIGURE 5. Failure mode

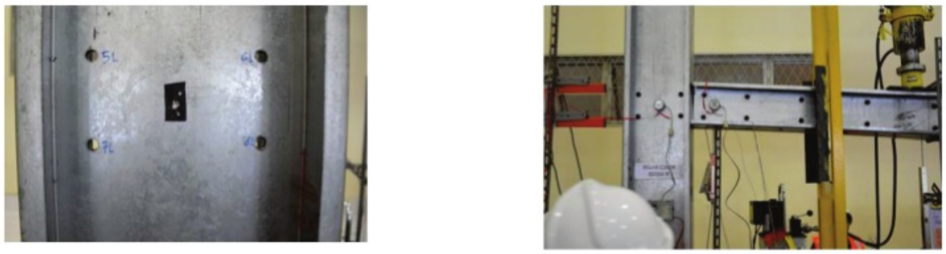
The deformation not only occurs in gusset-plate but also occur in bolt holes in beam and column. This is because the bolt holes in cold-formed steel section (CFS) experience bearing failure. Figure 6 and Figure 7 shows deformation mode in the bolt holes in beam and column. As shown in Figure 6a, the bolt hole in beam is pulled upward resulting deformation of bolt hole to be oval in shape. Figure 6b, the bolt hole in beam experience deformation due to bearing and shear.



a) Deformation mode that occur in beam (2L) b) Deformation mode that occur in beam (2R)

FIGURE 6. Deformation mode that occur in beam

Figure 7a indicated the 5L bolt holes in column experience deformation due to tension and bearing as a result of yield strength of beam is higher compare to the yield strength of column. Figure 7b shows beam undergoes a vertical displacement, the plastic deformation will increase as the load increase until failure mode occurs which is indicated by deformed shape of the bolt hole and gusset plate.



a) Deformation mode that occur in column (5L through 8L) b) Deformed shape of structure (front view)

FIGURE 7. Deformed shape of connection

The capacity of connections and joint stiffness are shown in Table 4. The initial stiffness is obtained from the slope of the straight line, and the ultimate load is obtained while the point loads was decrease or suddenly drop. Table 4 shows the ultimate load in experimental test amount to 15.68 kN and stiffness of joint is 1948.06 kN m/rad. From visual observations, the structure failure due to torsional buckling can be caused by various factors such as the placement of the load is eccentric due to center of gravity of the beam or the beam property is too slender.

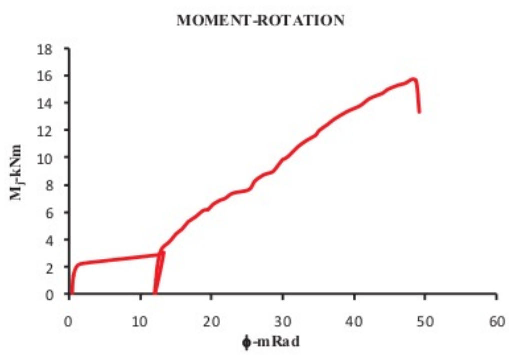


FIGURE 8. Moment – rotation relationship curve of experimental test result

Table 4. The capacity and joint stiffness of rectangular gusset-plate connection of experimental test result

Column section	Beam section	Load, P (kN)	Moment, M_j (kNm)	Stiffness, $S_{j,ini}$ (kNm/Rad)
C30024	C20019	15.68	15.68	1948.06

Comparison

To verify the results, the theoretical calculation and experimental test results are compared. Experimental test results are used as reference to verify the theoretical calculation result. The moment capacity and joint stiffness ratio of theoretical calculation result and experimental test result are shown in Table 5. The comparison has shown the moment capacity ratio amount to 0.69, it shows that there is difference of the moment capacity from experimental test but has close result. The joint stiffness ratio amount to 0.24, it shows that there is difference of the joint stiffness from experimental test. Both of ratio value above shows that the moment capacity and the joint stiffness from theoretical calculation result still different to experimental test result. The moment – rotation curve for these two methods;

theoretical calculation and experimental test are compared (Fig. 9). It is seen the initial stiffness from these two methods are same. However, as the load increase, the specimen continues to undergo plastic deformation with the difference of stiffness values of both methods.

Table 5. The moment capacity and joint stiffness ratio experimental – theoretical results

Moment capacity, M_j (kNm)			Joint stiffness, S_j (kNm/Rad)		
Exp	Theo.	Ratio	Exp	Theo.	Ratio
15.68	10.78	0.69	1948.06	458.53	0.24

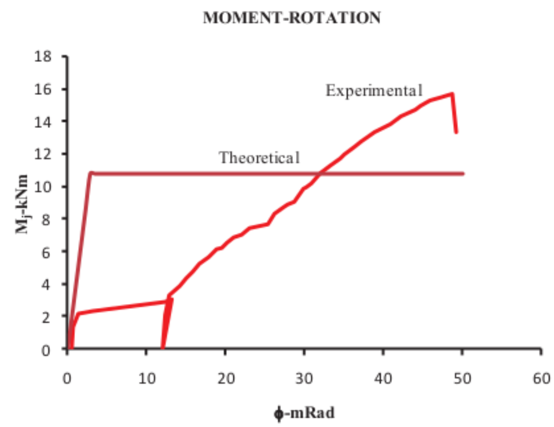


FIGURE 9. Comparison of moment – rotation relationship curve between these two methods

CONCLUSIONS

It can be concluded that the moment versus rotation of the connection developed from the experimental test results has proved that the connection is very ductile and can be categorised as pin connection. The gusset plate has not been able to significantly stiffen the connection. Comparison of moment capacity between theoretical and experimental test results obtains the ratio amount to 0.69 and the joint stiffness obtains the ratio amount to 0.24.

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