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Loading Capacity Calculation of Integrated Precast Slab and Column Panel Using Cold-formed Steel

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Loading Capacity Calculation of Integrated Precast Slab and Column Panel Using Cold-formed Steel

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Abstract. In the current study, the precast panel using a cold-formed steel section integrated with a cold-formed steel section integrated with self-compacting concrete was connected to the precast column panel. A T-shaped plate was used as a joint connector. Point loading applied onto the free-side of the slab panel. The material used to form a composite slab panel was C12524-type of cold-formed steel section as the reinforcements and it was integrated with self-compacting concrete. The connection in this research was divided into two-part. It was the side part and the middle part. The quality of cold-formed steel was fy = 530 MPa and fu = 590 MPa, the quality of the T-shaped plate connector grade was S355. The bolt diameter was variated with 10 mm, 12 mm, 14 mm, and 16 mm. The bolt quality was grade 8.8 (fy = 800 MPa). The calculation was the moment joint capacity of the connection and the stiffness. The moment joint capacity was increased within the bolt diameter increased. The side part of the specimen had the highest stiffness value; the bolts that could be used were M10, M12, and M14. To use the M16 bolt, configure the bolt spacing to be compatible with the standard BS EN 1-8:2005 [1].

1. Introduction

Composite construction was the combination of steel and concrete to form a single unit. It began to be used in about 1926 [2]. However, the use of steel structures especially the non-composite CFS sections leads to a buckling problem which reduces the maximum load, especially when used as compression members. Therefore, steel beams without lateral restrain are subjected to lateral-torsional buckling and twisting. However, agile development in technology leads to the use of CFS in the Industrialization of Building Systems (IBS) and it became more popular and well-accepted in developed and developing countries in the globe respectively [3]. In the steel construction industry, the hot-rolled steel (HRS) section and cold-formed steel (CFS) section are two distinguished steel sections that are used. However, among the two steel sections, HRS is the most familiar among building contractors and engineers [4].

There was an analysis study on the composite connection of cold-formed steel [5, 6]. The composite is between cold-formed steel and normal concrete slab (fc'= 30 MPa until fc'= 43 MPa). Another research about cold-formed steel connection is about the bolt connection with a various gusset plates [7, 8]. On the composite [5, 6] research, there was a result of moment-rotation, failure mode, and the gusset plate was a rectangular and haunched gusset plate. On the steel connection [7, 8] research, there was a moment rotation, failure mode, and the gusset plate was rectangular and haunched.

Based on recent research, there was no research of a composite cold-formed connection with a self-compacting concrete as the slab part. In this study, the precast panel slab with a size of 1 x 1 meter using a cold-formed steel section will be connected by T-shape connection to precast column panel with the size of 1 x 3 meter using cold-formed steel section integrated with self-compacting concrete. There was a comparison of joint capacity based on various bolt diameter.

2. Methodology

Parameter of Specimen

The specimen was shown isometrically in Figure 1. The specimen was built by cold-formed steel. This specimen was built within bolt connection, shear studs, and slab components without the SCC.

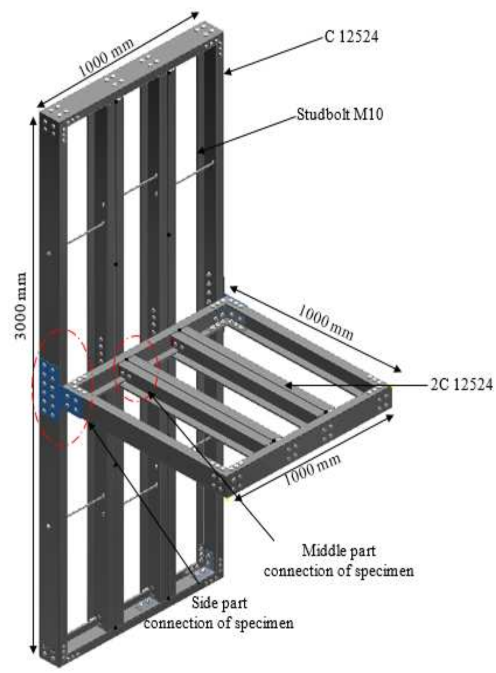

Figure 1 Isometric view of the specimen

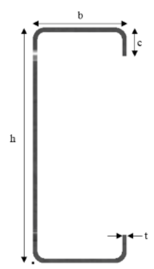

Figure 2 Cold-Formed Steel Cross Section

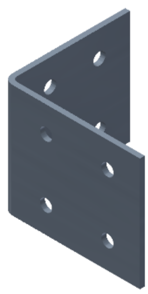

Figure 3 Angle Clamp

From Figure 1. The height of the column was 3,000 mm and width 1,000 mm. The column was made by CFS with C 12524 profile. The slab part had length 1000 mm and width 1000 mm. There was 2C 12524 CFS profile in the middle of the column and the middle of the slab. For CFS profile design strength was Fy = 530 MPa, Fu = 590 MPa.

There was a bolt connection for slab and column. The bolts were designed based on BS EN 1-8:2005 [1]. All bolts grade were 8.8. where fu bolt was 800 MPa. The bolts stress area were 58.0 for M10, 84.3 for M12, 115 for M14, and 157 for M16. There was a t-shaped plate between the bolt and the channel lip. The t-shaped plate had a thickness of 6 mm [5]. The t-shaped connection was used for the side part connection and middle part connection. T-shaped plate grade was S355, Fy = 355 MPa, Fu= 510 MPa based on BS EN 1-1: 2005[10]. The side part connection and the middle part connection could be seen in Figure 4. The specimen used a channel lips profile as the mainframe of the specimen. The channel lips section could be seen in Figure 2. The channel lips profile used in this research was C

12524. The detail were thickness 2.4 mm, height 125 mm, broad 50 mm, and lips 15 mm. There were an angle clamp with grade S355, fy = 355 MPa, fu = 510 MPa. The angle clamp thickness was 4 mm. The figure of an angle clamp was shown in Figure 3.

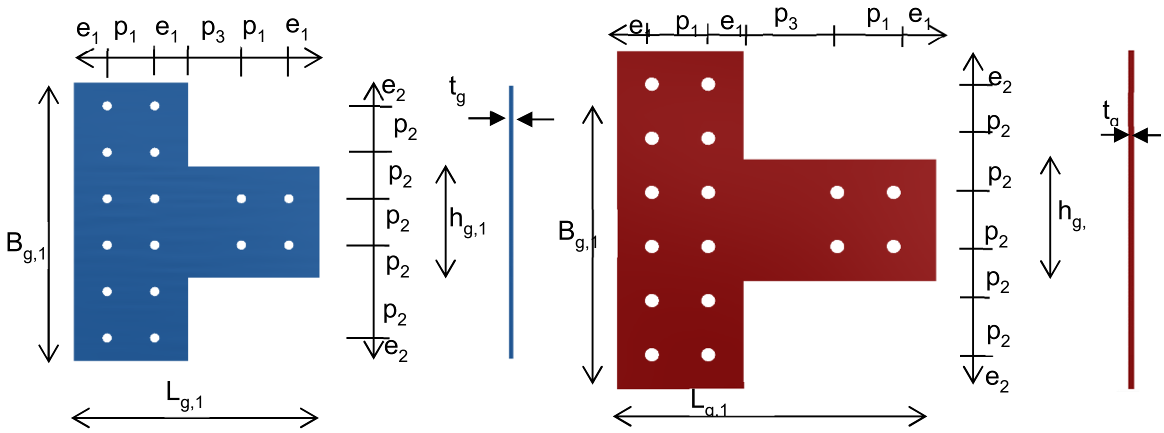

Figure 4. Bolt Space Dimension and Plate Size of Side Part (Blue) and Middle Part (Red)

In Figure 4. there were a bolt spacing configuration. The bolt spacing configuration was based on BS EN 1-8:2005 [1], there were edge distance between the bolt and the edge of plate. The steel specimen was exposed to the weather and other corrosive influences. From BS EN 1-8:2005 [1] M10 bolts the hole diameter was 12 mm, M12 had a hole diameter 13 mm, M14 had a hole diameter of 15 mm, and M16 had a hole diameter 18 mm. Based on BS EN 1-8:2005 [1], the outer thickness of the thinner outer connection was 4 mm because the plate is on the outer side. So, for minimum edge distance for all bolts diameter were.

Minimum $e_1 = 1.2d_0$ .... (1)

Maximum $e_1 = 4(t)+40$ mm........ (2)

d_0 was the hole diameter and t was the thickness of the thinner outer connected part.

The minimum horizontal spacing between the bolt (p1) had to be calculated from (**Figure 4** and **5**) based on BS EN 1-8:2005 [1].

Minimum $p_1 = 2{,}2d_0$........ (3)

Maximum $p_1 = 14t$ .... (4)

Another bolt configuration on the vertical direction (p_2) for Figure 4 and 5.

Minimum $p_2 = 2{,}4d_0$........ (5)

Maximum $p_2 = 14t$ .... (6)

The Maximum Joint Capacity

The bolt resistance were consist of shear resistance and bearing resistance based on BS EN 1-3:2006 [11].

Shear resistance (F_v, R_d) = $\frac{0{,}6\, f_{ub} A_s}{\gamma_{M2}}$ x c_s........ (7)

Based on BS EN 1-3:2006 [11] f_{ub} = ultimate strength of the bolt, A_s = area of the bolt (mm^2), and γ_{M2}= partial factor resistance of cross-sections in tension to fracture (1,25). C_s was contact shear number. For bolt it was 2. The bolt were M10, M12, M14, M16 and have a 8.8 grade. The bearing resistance calculation was based on BS EN 1-3:2006 [11].

Bearing resistance for CFS (F_b,$R_{d,cfs}$) = $\frac{2.5 a_b k_t f_u d t}{\gamma_{M2}}\ x\ cbp_{cfs}$........ (8)

Bearing resistance for gusset plate ($F_b,R_{d,gp}$) = $\frac{2.5 a_b k_t f_{ug} d t_g}{\gamma_{M2}}\ x\ cbp_{gp}$ .... (9)

Bearing resistance for angle clamp ($F_b,R_{d,ac}$) = $\frac{2.5 a_b k_t f_{ac} d t_{ac}}{\gamma_{M2}}\ x\ cbp_{ac}$ .... (10)

Based on BS EN 1-3:2006 [11], k_1 = 2,5 because t > 1,25 mm; a_b= 1.0 or e_1/3d, in this research could use 1.0; fu = ultimate strength of channel lips section; d = diameter of bolt; t = thickness channel lips profile, and γ_{M2}= partial factor resistance of cross-sections in tension to fracture (1,25). There are three type of bearing resistance. Bearing resistance were caused by cold-formed steel, caused by angle clamp, and caused by gusset plate. Cbp was number of bearing plate contact and cs was number of shear contact. For single section channel lips Cbp was 1 and 2 for double section. For gusset plate and angle cleat had 1 number of Cbp.

Based on BS EN 1-8:2005 [1], the bolt resistance is taken from the minimum value of F_v, R_d and F_b,R_d. To find the moment capacity of bolt (M_{bolt})

M_{bolt} = 4 x F_{bolt} x lever arm of beam bolt group. .... (11)

M_{total} = M_{bolt1} + M_{bolt2} .... (12)

3. Result and Discussion

Range of Validity

Minimum and maximum edge distance (**Figure 4** and **5**) for M10 bolt were shown in calculation below. The M10 d_0 was 11 mm. For other bolts minimum and maximum edge distance was shown in (**Table 3**).

Minimum e_1 = 1.2d_0 = 1.2 (11) = 13.2 mm

Maximum e_1 = 4(t)+40 mm = 4(4) + 40 = 56 mm

The minimum and maximum horizontal spacing between the bolt (p_1) had to be calculated from Figure 4 and 5 based on BS EN 1-8:2005 [1]. For M10 bolt minimum and maximum horizontal spacing were shown in calculation below. For other bolts minimum and maximum horizontal spacing was shown in Table 4.

Minimum p_1 = 2.2d_0 = 2.2(11) = 24.2 mm

Maximum p_1 = 14t = 14(4) = 56 mm

Another bolt configuration on the vertical direction (p_2) for Figure 4 and 5. For M10 bolt minimum and maximum vertical spacing between bolts were shown in calculation below. For other bolts minimum and maximum vertical spacing was shown in Table 5.

Minimum p_2 = 2,4d_0 = 2.4(11) = 26.4 mm

Maximum p_2 = 14t = 14(4) = 56 mm

Table 1. Range Validity of Edge Spacing

Bolt	Min. Edge Spacing (mm)	Edge Spacing (mm)				Max. Edge Spacing (mm)	Status
		Side		Middle			
M10	13.2	35	25	25	25	56	Ok
M12	15.6	35	25	25	25	56	Ok
M14	18.0	35	25	25	25	56	Ok
M16	21.6	35	25	25	25	56	Ok

Table 2. Range Validity of Horizontal Spacing

Bolt	Min. Horizontal Spacing (mm)	Horizontal Spacing (mm)		Max. Horizontal Spacing (mm)	Status
		Side	Middle		

M10	24.2	50	40	56	Ok
M12	28.6	50	40	56	Ok
M14	33.0	50	40	56	Ok
M16	39.6	50	40	56	Ok

Table 3. Range Validity of Vertical Spacing

Bolt	Min. Vertical Spacing (mm)	Vertical Spacing (mm)		Max. Vertical Spacing (mm)	Status
		Side	Middle		
M10	26.4	50	40	56	Ok
M12	31.2	50	40	56	Ok
M14	36.0	50	40	56	Ok
M16	43.2	50	40	56	No

From Table 1. to Table 3. the range validity of each bolt was summarized. The edge spacing at Table 3 for each bolt was suitable with BS EN 1-8:2005 [1]. The horizontal spacing between bolts were compatible at Table 4. The horizontal spacing for all various bolts was appropriate with BS EN 1-8:2005 [1]. The vertical spacing between bolts had a problem in M16.

In Table 3, M16 bolts had a bigger value of minimum vertical spacing. The designed vertical spacing did not suited with the minimum and maximum range. M16 was not calculated for the next step, because the design did not met the range validity.

Moment Joint Capacity

To calculate moment joint capacity, first, identify the minimum value between shear resistance and bearing resistance. Then choose the minimum value.

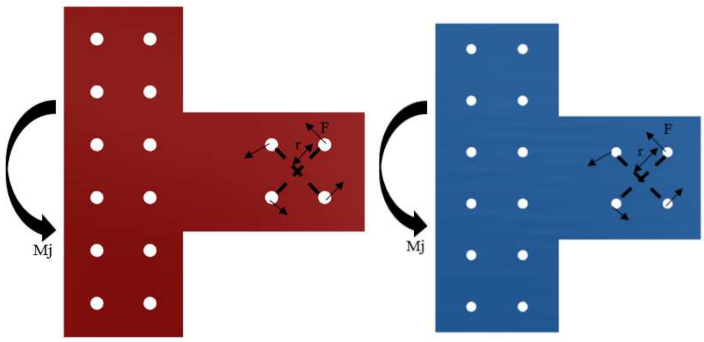

Figure 5. Moment Joint Capacity of The Middle Part (Red) and The Side Part (Blue)

Find the value of the lever arm of the group bolt (r). Figure 5. shown the illustration of the moment in the middle and side part of the bolt. For the bolt resistance, there were shear resistance ($F_{v,Rd}$) and bearing resistance ($F_{b,Rd}$). For the shear resistance of the bolt based on BS EN 1-3:2006 [11] from equation (7) in this paper. For side part and middle part, shear resistance was similar:

Shear resistance (F_v, R_{dM10}) for side and middle part.

$$(F_v, R_{dM10}) = \frac{0{,}6\, f_{ub} A_s}{\gamma_{M2}} \text{ x } c_s = \frac{0{,}6 \, x \, 800 \, x \, 58}{1.25} \text{ x } 2 \; = 22.27 \text{ kN x } 2 = 44.54 \text{ kN}$$

The bearing resistance calculation was based on BS EN 1-3:2006 [11]. The equation were used equation (8) to (10) in this paper. The bearing resistance had to be divided between the side part and middle part, because the side part and middle part had different condition of CFS and angle clamp.

Bearing resistance (F_b,R_{dM10}) for side part

Bearing resistance for CFS ($F_b,R_{d,cfs}$) $= \frac{2.5 a_b k_t f_u d t}{\gamma_{M2}}\ x\ cbp_{cfs} = \frac{2.5\ x\ 1\ x\ 590\ \ x 10\ x\ 2.4}{1.25}$ x 1

= 27.848 kN

Bearing resistance (F_b,R_{dM10}) for middle part. In the middle part there was no bearing resistance for angle clamp, because angle clamp was used on the side part.

Bearing resistance for CFS ($F_b,R_{d,cfs}$) $= \frac{2.5 a_b k_t f_u d t}{\gamma_{M2}}\ x\ cbp_{cfs} = \frac{2.5\ x\ 1\ x\ 590\ \ x 10\ x\ 2.4}{1.25}$ x 2

= 55.7 kN

So, the minimum value between shear resistance and bearing resistance was the bearing resistance in CFS for the side part. For the middle part the minimum resistance was bearing resistance for gusset plate. Then, there was a calculation of the moment capacity of the joint. Where $M_{j,side}$ for the side part and $M_{j,middle}$ for the middle part,

$M10_{j,side}$ = 4 x $F_{b,Rd,CFS}$ x lever arm of beam bolt group
= 4 x 27.848 x 35.36 mm = 3.94 kNm

$M10_{j,middle}$ = 4 x $F_{b,Rd,gp}$ x lever arm of beam bolt group
= 4 x 40.800 kN x 28.28 mm = 4.62 kNm

$M10_{total}$ = $2M10_{j,side} + 2M10_{j,\ middle}$ = 17.11 kNm

In Table 4 there was a comparison table between shear resistance and bearing resistance of each bolt diameter. From Table 4 was shown that the minimum value of middle bearing resistance had a high value than the side part. It caused by the double section was applied in the middle part. All bearing resistance in Table 4 had a minimum value than the shear resistance. The bearing resistance determine the moment joint capacity from side part and the middle part.

Table 4. Comparison Between Shear Resistance and Bearing Resistance

No	Bolt Diameter	Shear Resistance	Bearing Resistance		Moment Joint Capacity
			side	middle	
		(kN)	(kN)		kNm
1	M10	44.54	27.848	40.80	17.11
2	M12	64.74	33.42	48.96	20.53
3	M14	88.32	38.99	57.12	23.95

4. Conclusion

Based on this research, it could be concluded that.

1. M10 bolt and M12 bolt was very applicable to be used, because it had safe in range validity.
2. The middle part of the specimen had a low moment capacity, it caused by the lever arm of the bolt was shorter than the side part.
3. To use the M16 bolt, there was needed for spacing configuration to compatible with the standard BS EN 1-8:2005 [1].

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