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Finite Element Analysis of Composite Beam-to-Column Connection with Cold-Formed Steel Section

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Abstract. Cold-formed steel (CFS) sections are well known due to its lightweight and high structural performance which is very popular for building construction. Conventionally, they are used as purlins and side rails in the building envelopes of the industrial buildings. Recent research development on cold-formed steel has shown that the usage is expanded to the use in composite construction. This paper presents the modelling of the proposed composite connection of beam-to-column connection where cold-formed steel of lipped steel section is positioned back-to-back to perform as beam. Reinforcement bars is used to perform the composite action anchoring to the column and part of it is embedded into a slab. The results of the finite element and numerical analysis has showed good agreement. The results show that the proposed composite connection contributes to significant increase to the moment capacity.

INTRODUCTION

The use of construction materials, especially for middle rise building leads to more environment friendly material, easy to transport, fast in installation and anti-termite. The main criteria that must be met among others, highest strength-to-weight ratios, consistency in size and dimension, and it is not from organic material. Cold formed steel (CFS) is one of the most developed and reliable as an alternative material such as hot-rolled steel and wood. The thickness between 1.9-3.2 mm gives a large contribution to the strength-to-weight ratios. The yield strength about 350-450, even for some cases 550 MPa [1]. CFS can be categorized as slim profile or slender section, generally has only one axis of symmetry. This feature is prone to premature failure, local buckling, and torsional buckling [2]. In addition, as long as the CFS has economic potential and generates better construction, the use of CFS could be more popular and convincing than the hot rolled section in composite connection. However, the use of CFS still limited to low-risk building with short span. Overall, CFS only used as an alternative for timber material that commonly used for wall partition, purlin and roof truss. The connection is relatively simple and only to resist the compressive and tensile forces.

Connections are usually designed as pinned (moments= 0) which associated with simple construction or rigid (rotation= 0) which is associated with continuous construction. However, the actual behaviour falls between these two extreme cases. The use of partial strength or semi-rigid connections (moment and rotation ≠ 0) has been encouraged by codes and studies on the matter known as semi-continuous construction have proven that substantial savings in steel weight of the overall construction. Composite connection is introduced in this paper to improve the performance of partial strength connection. The objective of this paper is to propose and model an innovative connection for cold-formed beam-column connection designed as composite connection.

Classification of the moment and strength of the connection is very much depends on whether the connection is simple, semi-rigid, or rigid (Fig.1). The initial stiffness of the connection is obtained from the slope of the straight

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line of the moment-rotation curve. This curve is obtained from a full-scale experimental test. However, to establish the behaviour and performance of all connections with by experimental tests could be very expensive and time consuming.

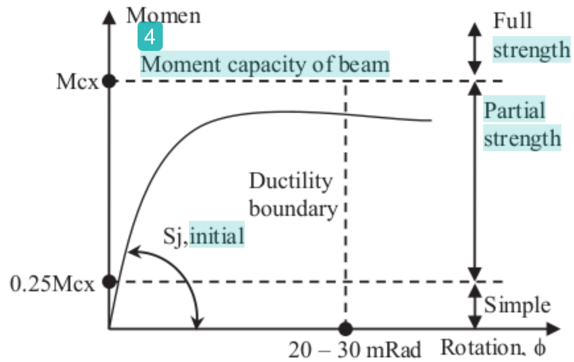


FIGURE 1. Moment-rotation curve

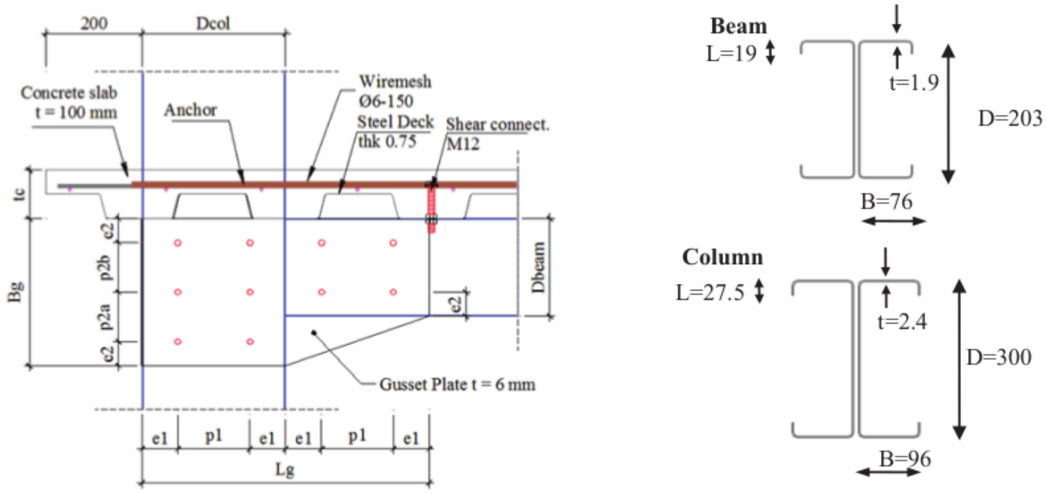
Alternatively, finite element method (FEA) can be used. Some of the previous studies discussing semi-rigid connections through full-scale and FEA tests ([3]; [4]; [5]; [6]), it has been proven that FEA is reliable as a coMParator and predictor for a full-scale test. In addition, EC3 [7] provides guidance for design semi-rigid connection with component method. It is a mechanical approach by separating the connection into individual components based on stiffness and internal force (shear, tensile or compressive forces). However, EC3 does not provide the information procedure to calculate the stiffness of gusset plate connection [8].

From the research of the non-composite CFS connection, Tan has concluded that gusset plate connection provides the highest moment capacity coMPared to flange cleat and web cleat connection [9]. Bucmis and Daniunas [8] has stated that one convenient way to connect between beams and columns is gusset plate. In this paper, the connection media between columns to beam is hot rolled steel gusset plate, the moment rotation performance of the connection under monotonic loading is studied.

MATERIAL AND METHODE

The proposed CFS beam to column composite connections (Fig. 2a) is chosen by considering its function as a part of medium-rise buildings. To obtain the moment rotation curve, the analysis method was used are FEA and component method adopted from EC3 (EN 1993-1-1-8 2005) and will be explained in the next chapter. The section of double lipped channel back to back with yield strength 450 MPa is used for beams and columns with dimensions as shown in Fig. 2b. The gusset plate thickness 6 mm with grade S275. M12 bolts with G8.8 used as fasteners between gusset plate and beam column, and placed in accordance with the minimum and maximum distance allowed by EC3. The distance of bolts is e1 = 50 mm, e2 = 50 mm, p1 = 200 mm, p2 = 103 mm, Bg = 306 mm and Lg = 600 mm.

The dimension of concrete slab is 100 mm thickness and 750 mm width and compressive strength fc = 30 MPa. Wire mesh ϕ6-150 and ϕ12 anchor reinforcement embedded with concrete cover of 30 mm, both of yield strength material used are 240 MPa. Metal decking thickness 0.75 mm with rib spacing of 350 mm is used as a permanent formwork. Shear connector M12 G8.8 is placed on the top flange of the beam and the distance between the shear connector according to a rib of metal decking.



(a) Cold-formed composite connection configuration (b) Double lipped channel profile

FIGURE 2. Proposed cold-formed composite connection

Steel is a ductile material, when applied load under yield stress (σy), elastic deformation could occur and the elastic modulus can be obtained from the slope of the straight line on the stress-strain curve. If the yield stress is exceeded, plastic deformation could occur and the yield stress increases until the ultimate tensile stress limit (σu) is reached. In this paper, the steel grade materials used as shown in Table 1.

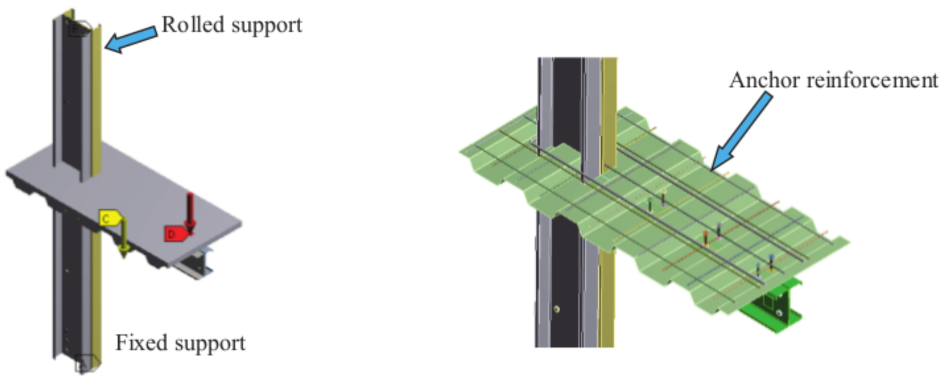
TABLE 1. Steel material properties

Elements	σy MPa	σu MPa	Ei MPa	Et MPa
Beam, column	450	480	2.05+E5	205
Gusset plate	275	430	2.05+E5	205
Wiremesh	240	400	2.05+E5	205
Rebar	240	400	2.05+E5	205
Bolt,shear connector	640	800	2.05+E5	205
Metal decking	405	480	2.05+E5	205

Finite Element Modelling

Ansys program was used to simulate non-linear responses of structures. As shown Fig.3a, the gravity loading due to self-weight of structure is applied, the incremental loading is given at 1 meter from the face of the column. Unrestrained condition is applied at the end of the beam as well as cantilever beams, the main purpose is to obtain the structural responses in the form of moment-rotation curves. The column height is 3 meters and fixed supported at its base. To resist lateral movement, at the upper end of the front flange column is restrained by roller support.

The frictional contact is applied for beams, columns, bolt shanks and gusset plate. The contact for material embedded in the concrete (wire mesh, anchor reinforcement, shear connector) and contact area between metal decking to the top of beam flange are used and is assumed not to slip. Therefore, bonded contact is selected in order to represent the composite behaviour. The U shape of anchor reinforcement (Fig.3b) serves as reinforcement in tension zone and placed as close as possible around the column, in order to enable the transfer of load into the column without excessive slip, deformations and cracking [10].

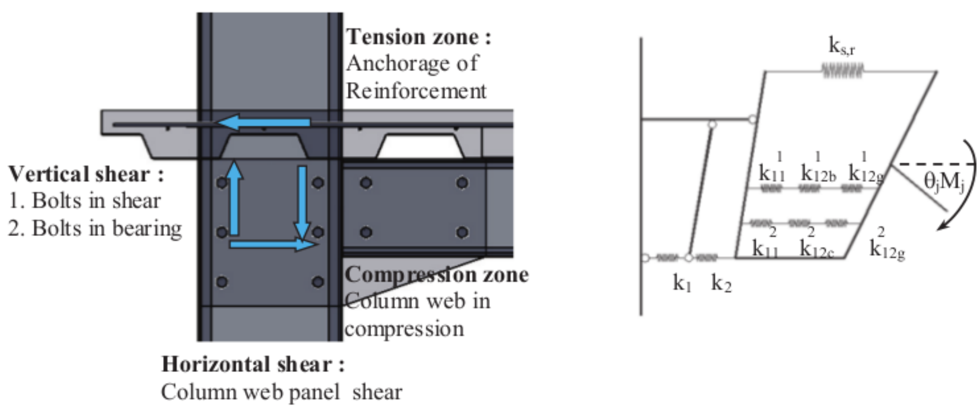


(a) Load and boundary condition (b) Anchor reinforcement position

FIGURE. 3. Isometric view of finite element models

Parametric Analysis

The capacity of the proposed connections configuration (Fig. 4a) is calculated according to EC3-1-8 [7] and EC4-1-1 . In this research, each connected component contributes according to its function are: 1) The bolt capacity (bearing capacity and shear capacity) of the gusset plate; 2) The bolt capacity (bearing capacity and shear capacity) of CFS; 3) The moment resistance of the gusset plate, which is the minimum value of the moment resistance buckling of gusset plate and bolt group; 4) The moment resistance of anchor reinforcement; 5) The total moment resistance, which is the sum of the moment capacity of gusset plate and the anchor reinforcement. The classification of the connection is determined by the initial rotational stiffness of the connection (Sj, ini). In this case, this value depends on the component involved in the joint connection: 1) The components of the tensile reinforcement in the concrete slab; 2) The shear strength and bearing strength of the bolt; 3) The column web in the compression; 4) The column web panel in shear. For composite connections, following components were identified as shown in Fig. 4b, the values of k can be referred in EC3.

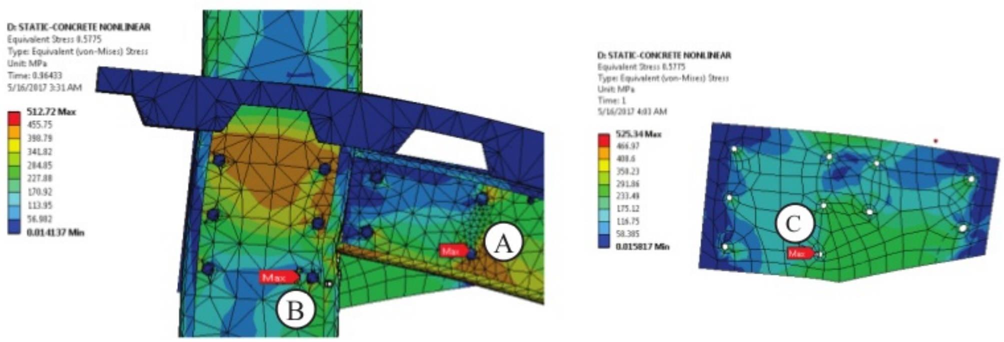


(a) Component of connection (b) Spring models of connection

FIGURE 4. Element stiffness connection contribution

RESULT AND DISCUSSION

For the first method, the finite element non-linear analysis was conducted by involving non-linear material properties and the structural responses were obtained by applying the "Large deflection" feature in Ansys. Von mises yield criteria was used to find out the stress distribution and concentration in the elements. The stress distribution of von mises of the joint is shown in Fig. 5a. It's clearly visible the maximum stress concentration occurs in the bolt hole of the beam (notation A) with a value of 503 MPa. At the column position the maximum stress occurring in the bolt hole located near the compression zone (notation B), the maximum stress is 490 MPa. Both values exceeded the yield stress of cold formed steel 450 MPa. The maximum stress of the bolt occurs in a position corresponding to the maximum stress in the bolt hole of the beam. The von mises stress is 525 MPa means the bolt is still in an elastic condition (less than 640 MPa), it can be concluded the cold form steel yielded first. The maximum stress distribution of gusset plate generally occurs in the compression zone as indicated in Fig.5b. At the upper position of gusset plate, most of the stress still in the range of elastic limit due to the contribution of the concrete slab. The maximum stress is 288 MPa (notation C) exceeds the yield stress of the gusset plate (275 MPa).



(a) Von mises of a solved connection

(b) Von mises of gusset plate

FIGURE 5. Finite element analysis result

Concrete material is strong in compression but not able to resist direct tension. In tension zone, even if the concrete is cracks, the connection resistance is provided by anchor reinforcement located at the upper part of the steel connection. Fig. 6 shows the connection with the hidden mode of the concrete slab. The finite element result shows that the reinforcement is still in elastic condition with tension stress 130 MPa. According to EC3, tensile forces will be balanced by compressive action of the beam and column flange at the compression zone. At this location, the flange beam receives a maximum compression of 453.28 MPa and column flange of 414.74 MPa. Those values indicate the compression zone is critical compared to the tension zone.

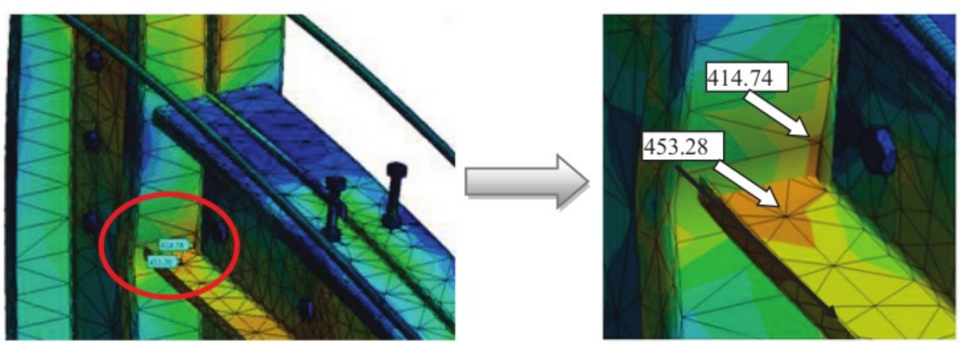


FIGURE 6. Deformed shape of von mises stress at compression zone

For the second method, the component method was used to obtain the moment capacity of the connection. The plate bending moment resistance is calculated by Eq.(1) which refers to BSEN 1993-1-1 [12]. The parameters that is used are the section modulus (W_y), yield stress (f_y) and the partial factor (γ_{M1}). The slenderness effect of the cross-section is expressed by buckling reduction factor χ_{LT} (Eq.2), this value is obtained by the slenderness factor λ_{LT} and value to determine the reduction factor Φ_{LT} (Eq.3). The imperfection factor α_{LT} in Eq.(3) is determined from Eurocode 3. In this case, because of the lipped channel profile is defined as "other cross sections", then $\alpha_{LT} = 0.76$ is selected.

$$M_{b,Rd} = \frac{\chi_{LT} \cdot W_y \cdot f_y}{\gamma_{M1}} \tag{1}$$

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

$$\Phi_{LT} = 0.5 \cdot \left(1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2\right) \tag{3}$$

For non-uniform panel, conservative approach can be applied with condition, if the angle (α) of the trapezoid plate exceeds 10 degrees, then the non-uniform plate can be approximately by a rectangular plate [13]. In this paper, however, the geometry of the gusset plate is a combination of rectangular and trapezoidal shape. For a more precise calculation, a method to predict the moment resistance for the non-uniform plate is proposed. The plates is divided into sub-sections and iterations need to be done for each sub-sections (Fig. 7). The final iteration produces the moment resistance of the gusset plate M_g as shown in Eq. (4). Tension resistance (P_{reinf}) of anchor reinforcement is obtained by multiplying the yield reinforcement ($f_{y,\ reinf}$) with the total area of steel bars, in addition the tensile strength of the concrete is ignored. These values are used to calculate the moment resistance (M_{reinf}) of anchor reinforcement as shown in Eq. (5). From the procedure above, the following values have been obtained for moment capacity, Mg = 19.053 kN m, M_{reinf}=10.816 kN m, Mj=29.87 kN m, respectively.

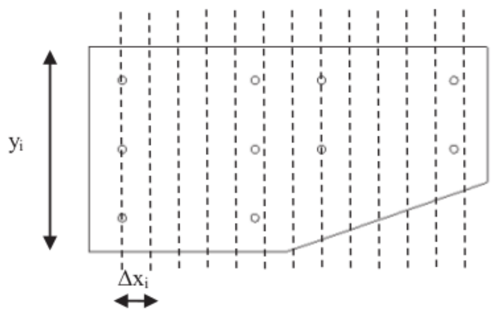


FIGURE 7. Sub-section of gusset plate

$$M_g = M_{b,Rd} = \sum_{i=1}^{n} \frac{\chi_{LT,i} \cdot W_{y,i} \cdot f_y}{\gamma_{M1}} \tag{4}$$

$$M_{reinf} = P_{reinf} \cdot y_{reinf} \tag{5}$$

The calculation of the rotational stiffness connection capacity using by component method is carried out by the procedure described in EC3. The final result is a moment rotational curve representing the non-linear characteristics of the joint. For practical purposes BS EN 1993-1:8 [12] provide a simplified guide where the behaviour of non-linear connections can be simplified into bilinear curves (Fig. 8).

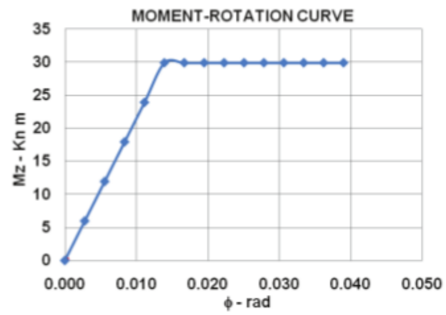


FIGURE 8. The moment rotation curve of component method

The summary results of FEA and EC3 are shown in Table 2 and Fig. 9, overall EC3 prediction gives under estimate result coMPared to FEA. It can be concluded that FEA gives more rigid results and is a logical consequence because the contribution of concrete plates is negligible on component methods. The ratio between FEA and EC3 as shown in Table 3 revealed a need to improve the calculation of both methods. Furthermore, the full scale test should be done as the basis of the correction.

TABLE 2. Summary results

Methode	Momen Mj Kn m	Stiffness Sj,ini Kn m/m Rad
Finite element	137.5	16032
EC3	30	2143
Ratio	4.6	7.5

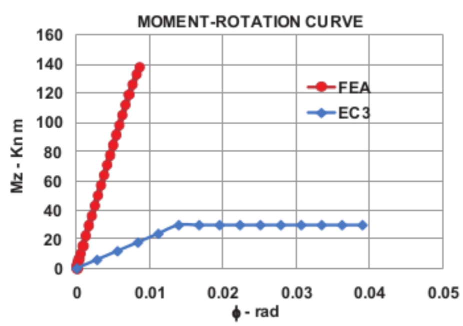


FIGURE 9. The moment rotation curve of analysis component results and finite elements

CONCLUSIONS

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The main purpose of this research is to study the behaviour of CFS composite joints with a gusset plate as connection media between beam and column. The following conclusions can be listed as:

- The predicted failure mode is in the bolt hole of the beam and gusset plate.
- The beam flanges and columns in the compression zone play an important role to provide sufficient stiffness of the connection.
- The results of the FEA is quite high as coMPared to the component method developed from EC3. This could be due to the effect of including the reinforcement bars in the composite connection. The rigid behavior of the re-bars could contribute to the stiffening of the connection.

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