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Experimental and analytical study on composite connection with cold-formed steel of double channel sections

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Abstract. The opportunity of Cold-Formed Steel (CFS) as an alternative construction material is still worth to be considered. The amounts of research concerning using this material is the evidence that the use of CFS is getting more acceptable. However, the studies of CFS as part of composite joint is yet to be established. This paper presents the use of 10mm thick hot-rolled gusset plate slipped between double lipped C-channel arranged back-to-back to form the proposed composite connection. Two specimens with and without seat angle were tested until failure. The weakness of thin plate behavior at the joint area was reduced by means of angle stiffener placed at the column web. The calculation procedure was developed due to limited design rule in Eurocode for this connection type, and compare with the experimental results to obtain the relationship between these two methods. It can be concluded that the use of seated angle has contributed to an increase in connection capacity.

1. Introduction

The uses of cold-formed steel (CFS) is gaining popularity for residential construction. This cannot be separated from the fact that this material has a high strength to weight ratio rather than hot-rolled steel. Due to its lightness, the tools required for construction is relatively easy to use and inexpensive. CFS is fabricated at low temperatures, therefore unusual cross-sectional forms can be produced [1]. One of the popular sections is the "C" channel, commonly used for purlin and roof truss.

The production of CFS with depth of more than 200 mm gives an opportunity to be able to use as a main structure. The current design procedures have proven some of the benefits of composite construction. The advantages of composite beam can be improved by considering the combination CFS with concrete slab. Some of the advantages can be listed as: fast installation, rust-resistance, good performance in service, reduce the weight and dimension of the material.

In building construction, the advantages of composite joints could only work if it supported by a good connection. The connection at the joint should be simple, fast and easy to install. Bucmys and Danihnas [2] stated that the use of gusset plate is one of the connection types that can meet these criteria. Siang [3] concluded that the gusset plate gave the best results in terms of strength and ductility, the bolted connection was used because it is easier as compared to the welded connection which is not recommended because of thin plate [4].

The use of gusset plate in beam-to-column connection with re-bars installed around the column and embedded in concerete slab could categorised the connection between two ideal conditions (pin or rigid), resulting in a semi-rigid joint with nonlinear behavior. The contribution of the connection is calculated by means of equivalent spring constant as an input data. The use of semi-rigid connections

leads to economy in member designs, however, the study in depth is required because this connection makes the analysis somewhat difficult.

Eurocode 3 provides guidance on connection design with component methods, but there is no design information for gusset plate connection [2].The study of CFS with gusset plates has been done by several researchers ([2, 5-12]), mostly discussing the behavior and classification of gusset plate connection. Some calculation procedures were discussed due to limited guidance in Eurocode 3.

Research on the potential of CFS as a composite beam has been done for several years [13-19]. Some innovations in the use of shear connectors or concrete mixtures were conducted with the aim of improving the performance of CFS and reducing dependence on HRS at once. However, most of the research does not discuss the behavior of the beam-column composite connection.

This paper presents the results of an isolated joint test of a gusset plate composite joint combined with CFS. The characteristics of connection are predicted using component methods, referring to Eurocode 3 [20] and Eurocode 4 [21]. The results of both methods are presented in the form of moment-rotation curves and show satisfactory results, proving that the Eurocode procedure can also be applied to materials of less than 4 mm thick.

2. Material and method

2.1. Test specimen and procedures

The proposed connection consisted of two specimens with label IJT-01 (**Figure 1.a**) and IJT-04 (**Figure 1**.b). Each specimen consisted of double lipped CFS where coupon tests are prepared for tensile strength test and the results are shown in **Table 1**. The size of beam is 200 mm deep and the size of column is 300 mm deep. Both sections are 2.4 mm thick. For gusset plate a steel with grade S275 was used with 10 mm thick. The specimens were connected using 10M12 bolts Grade 8.8., not beyond the validity range according to Eurocode rules.

**(a)** Angle stiffner   **(b)** Seat angle
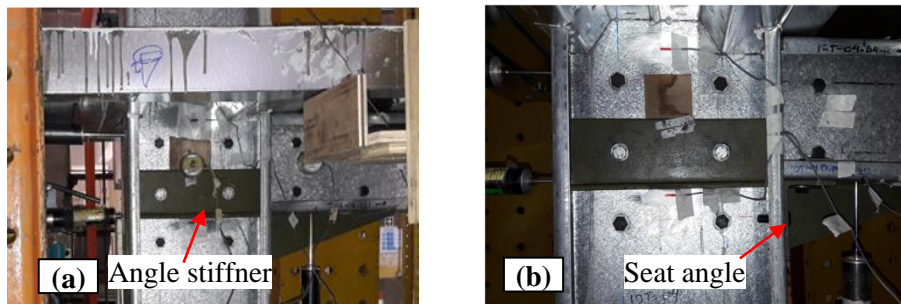
Figure 1. Isolated joint test configuration. (a) Specimens IJT-01 (b) Specimens IJT-04

M12 bolt was also used for shear connector to avoid welding work (**Figure 2**), placed in the strong side i.e. in the portion of the deck rib closest to the beam end [22].The tensile strength of M12 bolt adopted from Muhammad Lawan experiment [23] with a yield strength of 800 Mpa (**Figure 3**).The angle stiffener was made of steel grades S275, placed on the web column and parallel to the bottom beam flange, with the aim of avoiding premature buckling at the column flange. For IJT-04, two seat angles were placed at the bottom of the beam flange (**Figure 1**.b.).

As shown in **Figure 2**, the U shape anchor reinforcement size D12, fy = 460 Mpa was placed as closely as possible around the column to allow the load transfer without deformation, cracks and excessive slip as suggested by Anderson [24]. The actual size of the wire mesh is 4.8 mm and 200 mm distance, the yield strength from the manufacturer is 460 MPa. The concrete slab width 750 mm and

thickness 100 mm was used and placed on metal decking, the compressive design strength of the self-compacted concrete is 40 MPa on the 28th day.
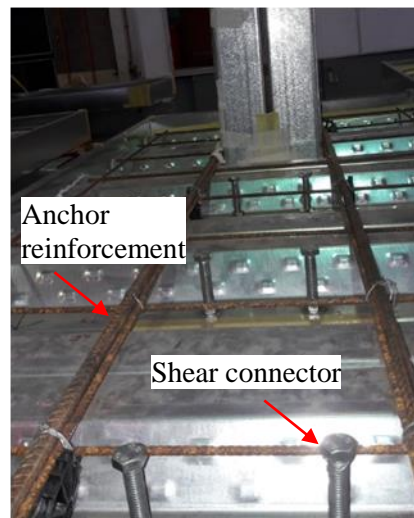
Figure 2. The concrete slab of the specimens

Table 1. Tension test

|	Component	Yield strength Mpa	Ultimate strength Mpa	Modulus young Kn/mm2	Strain/elongation %
CFS	Sample 1	493	623	179729	3.54
	Sample 2	561	636	151070	2.5
	Sample 3	568	639	158333	3.22
	Sample 4	558	649	187500	3.55
	Average	545	637	169158	3
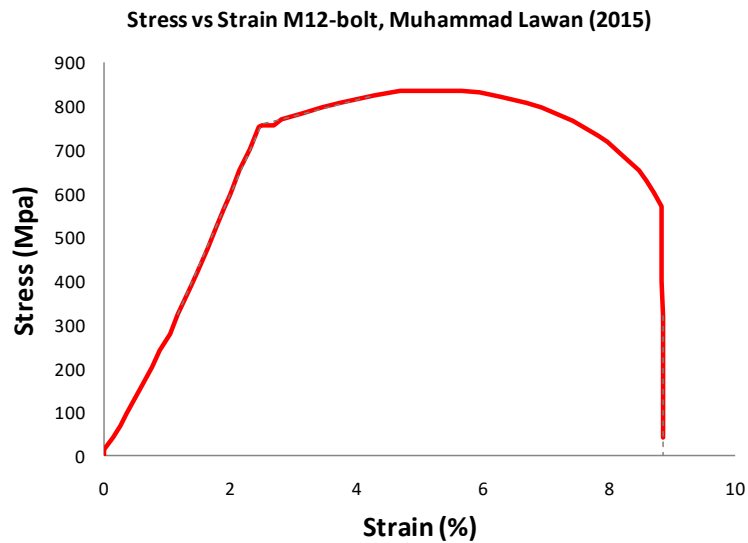
Figure 3. M12 bolt stress-strain curve Muhammad Lawan [23]

2.2. Test procedures

To obtain the moment-rotation data by means of isolated joint test procedure, the point load was applied at a distance of 1 meter from the face of the column, the incremental loading with ranged between 0.2-0.5 was chosen because it can be done manually via hydraulic jack (**Figure 4**.a,).To find out the initial stiffness, the unloading procedure was performed, at this stage, the bolt usually moves close to or touch the bolt hole. Next step, the loading procedure is repeated until the collapse pattern is detected.

The moment of the joint was obtained from the maximum load multiplied by the lever arm distance measured from the loading point to the surface of the column. Two inclinometers (INC1 and INC2) were used to find out the rotation data of the connection. INC1 was placed on the centre line of the beam at a distance of 100 mm from the column face. INC2 was placed on a web column and also an intersection of the centreline of the beam and column. Four Linear Variable Differential Transformers (LVDT) was used to obtain the displacement of the specimen. LVDT 1 was placed just below the load cell to attain the P-Delta data, whereas LVDT 2,3,4, and 5 were used to obtain the displacement in accordance with the orientation of LVDT position. Two strain gauges were placed on the top and bottom flanges of the beams (SG1 and SG2), while SG3 and SG4 were positioned on the column web and also parallel to the beam flanges. Finally, one strain gauge (SG5) was attached on reinforcing bars embedded in a concrete slab.

All instrument data were recorded automatically by a data logger except inclinometer. During the experiment, the angular rotation data of the inclinometer were recorded manually along with the incremental load. Visual inspection is conducted to observe the deformation of the structure as a whole. If necessary, the experiment will be paused while keeping the load value constant. Any information obtained were recorded in the Logbook so that it can be used for evaluation. The experiment is discontinued if: a) Sudden drop of the load was observed b) Large deflection without increase applied load. The specimen was connected to the frame rig with the base plate. The roller support was placed at the top of the front of the beam and serve when the load is applied to the specimen. The lateral bracing was applied to avoid torsion of the slab and beam (**Figure 4**.b.).
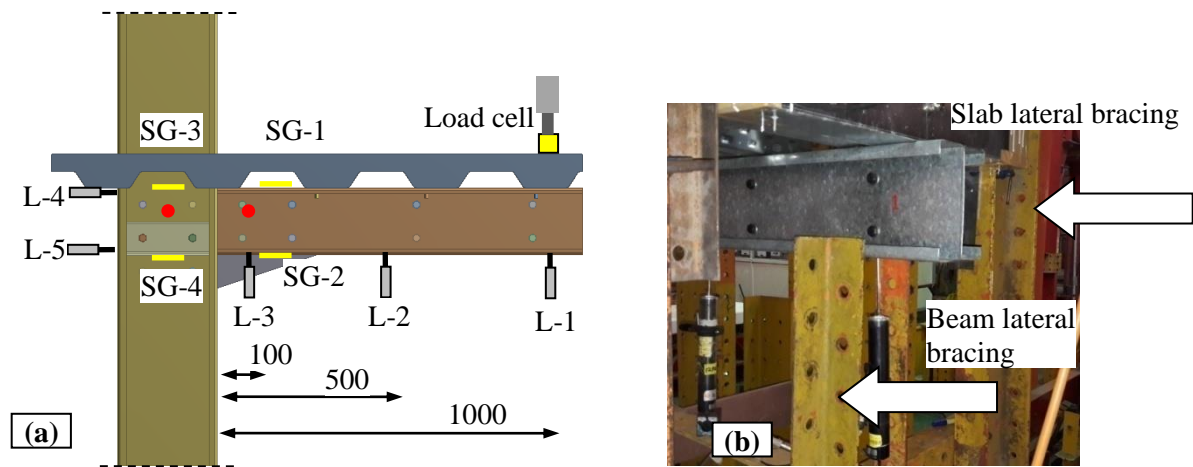
Figure 4. Isolated joint test configuration (a) Data acquisition layout (b) Lateral Bracing

2.3. Parametric analysis

Generally, the moment capacity of the connection is calculated by considering the limited force in the compression zone. However, the use of angle stiffener and seat angle at the column web has improved the moment resistance of the connection as these angles have contributed to premature failure in compression zones due to buckling. As shown **Figure 5**.a, the tensile force in the reinforcement ($F_{t, reinf}$) must be less than that of the column web resistance (Fc). Therefore the moment resistance of the anchor reinforcement ($M_{j, reinf}$) with lever arm h_{reinf} can be calculated with the formula:

$$M_{reinf} = F_{t,reinf} \cdot h_{reinf} \qquad (1)$$
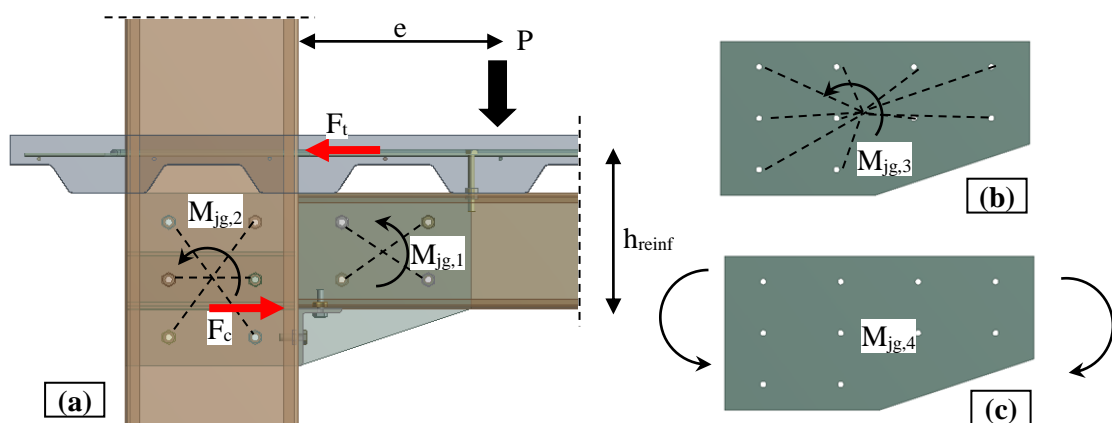
Figure 5. Parametric symbol (a) Connection (b) The resistance of bolt group (c) The resistance of gusset plate

The moment resistance of the gusset plate (M_{jg}) is determined from the weakest component, i.e. beam bolt group $M_{jg,1}$ or the column bolt group $M_{jg,2}$ (**Figure 5**.a.), the bolt group on the gusset plate $M_{jg,3}$ (**Figure 5**.b.) and buckling moment resistance of gusset plate $M_{jg,4}$ (**Figure 5**.c.), as equation (2).

$$M_{jg} = min(M_{jg,1}, M_{jg,2}, M_{jg,3}, M_{jg,4}) \tag{2}$$

The design resistance moment ($M_{j,Rd}$) is derived from the contribution of the anchor reinforcement and the gusset plate can be expressed as:

$$M_{j,Rd} = M_{jg} + M_{reinf} \tag{3}$$

To evaluate the joint stiffness, each part of the connection was modeled by means of equivalent linear spring, elastic behavior assumed to be relevant for small displacements and deformations. The active components which must be considered are the stiffness of the anchor rod (k_{sr}), the stiffness of the bolt group in the column and the beam due to shear (k_{11}) and bearing (k_{12}), the stiffness of the gusset plate (k_{gp}), the column web stiffness (k_2) and the column web panel shear (k_1). The component stiffness and the lever arm are used to obtain the equivalent stiffness (k_{eq}) and the lever arm equivalent (z_{eq}). Furthermore, the stiffness of the individual components can be transformed into a single initial stiffness ($S_{j, ini}$) by using the following equation (4).

$$S_{j,ini} = \frac{E \cdot z_{eq}^{2}}{\sum 1 / k_{eq}} \tag{4}$$

In case of global elastic analysis, the stiffness of the connection should be able to distribute the internal force between the component of the connection, therefore EC3 EN 1993-1-8: 2005 (E) article 5.1.2 (4) regulates the use of the stiffness reduction factor (η) based on the actual bending moment in connection ($M_{j, Ed}$) to the design moment resistance ($M_{j, Rd}$) as shown in Fig. 5.

$$S_j = S_{j,ini} \text{ if } M_{j,Ed} \le \frac{2}{3} \cdot M_{j,Rd} \text{ or } S_j = S_{j,ini} / \eta \text{ if } M_{j,Ed} \le M_{j,Rd} \tag{5}$$

3. Result and discussion

3.1. Parametric vs experiment test for specimen IJT-01

The moment-rotation from experimental and parametric for each specimen are presented in **Figure 6** and **Figure 8** respectively. For IJT-01 (**Figure 6**), start with a linear curve due to friction between the components of the connection. At rotation of more than 0.005 radians, the decrease of stiffness could cause by the initial crack of the concrete slab, the tensile stress transferred to the anchor reinforcement gradually. In addition, the possibility of slippage in the pretension bolt holes where the curve becomes horizontal or reduced [25]. At rotation more than 0.015 radians, the stiffness increases and the deformation continues until its collapses. This may be due to contact between bolt and bolt hole, or lower beam flange to the column flange reinforced by angle stiffener. Visually, the slope of the line is in the range 0.02 - 0.03 radians tend to be constant; therefore the rotational stiffness can be determined (1590 Knm/rad). Theoretically, the hogging moment capacity of the composite beam can be calculated referring to the publication of Chen [22] gives 41.194 KNm, smaller than experimental (55.8 Knm), can be classified as full-strength. The rotation at the moment of ultimate is 0.035 radians exceeding 0.03 radians, categorized as ductile.
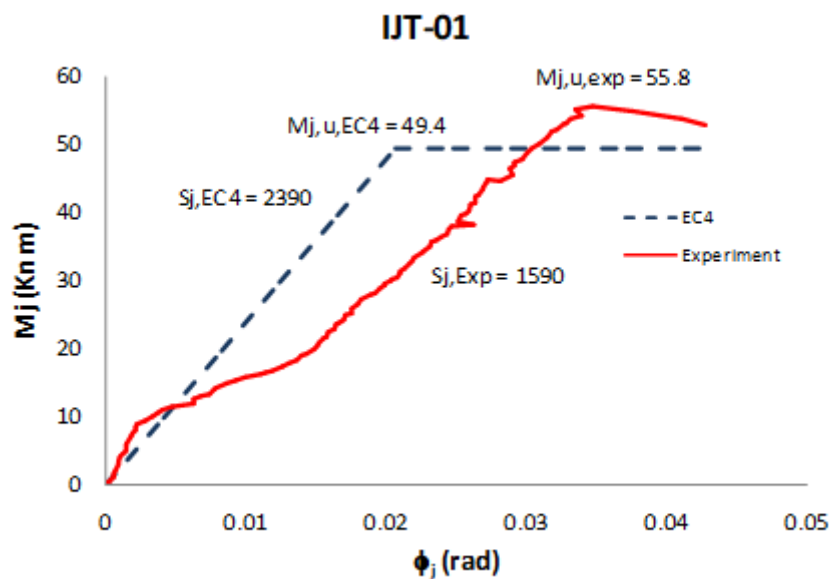
Figure 6. Moment-rotation experiment vs EC4 for specimen IJT-01

In the compression zone, there is no visible buckling in the column flange due to the pressure from the beam flange (**Figure 7**.a.). There was no failure mode recorded on gusset plate or angle stiffener even the bolt itself, however, observation showed that at rotation of 2.26 degrees or 0.038 radians (see **Figure 7**.b.), bearing failure occured at the bolt hole.
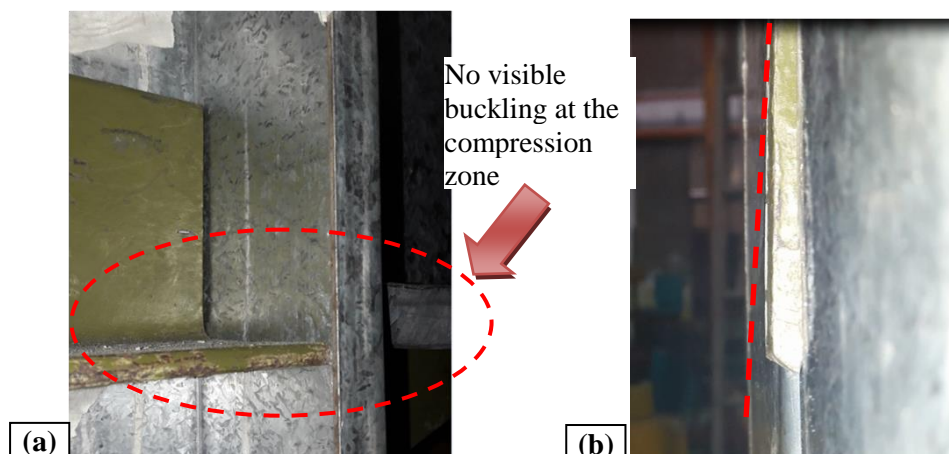

Figure 7. Joint rotation for IJT-01 (a) Angle stiffener (b) Rotation of gusset plate

As shown in **Figure 8**, the initial stiffness for IJT-04 was obtained at range of 0.01-0.03 radians which is 1790 Knm/rad, stiffer than IJT-01. The concrete slab started to crack at Mj greater than 14 KNm, initiated by a slope decrease at that region. The ultimate loading was reached at 0.04 radians greater than 0.03 radians which is able for the connection to be classified as ductile. The ultimate moment was recorded as 65.4 Knm greater than previous connection. It was proven that the addition

of seat angle can improve the connection capacity about 1.17%. Automatically, the category of this connection is a full strength.
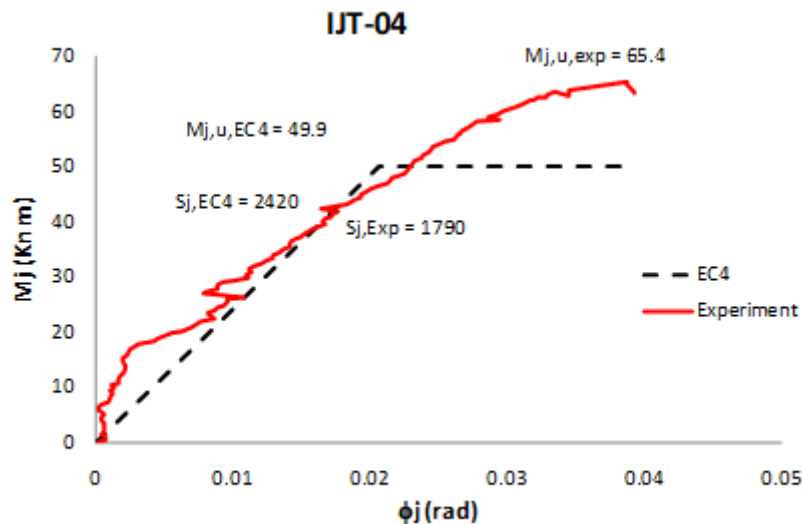
Figure 8. Momen-rotation experiment vs EC4 for specimen IJT-04

The rotation on the gusset plate and beam were shown in **Figure 9**.a. However, seat angle remains in the same condition at initial stage, no visible bending on the angle section or even the bolts pulled out from the column flange (see **Figure 9**.b). There is also no buckling mode in the contact area between the column flange and the angle stiffener. From the above description, it is concluded that the gusset plate, seat angle and angle stiffener were strong enough to avoid any crushing failure. The rotation of the connection probably due to bearing failure at the bolt hole which is the weakest connection component.
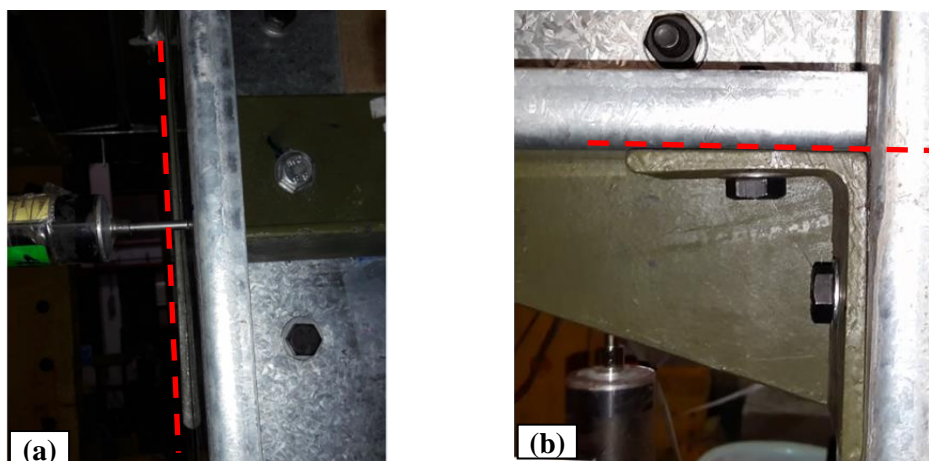
Figure 9. Joint rotation for IJT-04 (a) Rotation of gusset plate (b) Deformation of seat angle

3.2. Parametric vs experiment test for specimen IJT-04
The comparison between experimental and parametric is presented in **Table 2**. The ultimate moment ratio shows the parametric results close to the experimental results. The stiffness ratio exhibits an overestimate value with the ratio between 0.66 - 0.74. Overall, specimen IJT-04 has performance

greater than IJT-01, which proved that the contribution of seat angle could improve the connection performance.

Table 2. Results of parametric and experimental

Specimen	$S_{j,EC4}$ Knm/rad	$S_{j,Exp}$ Knm/rad	$M_{j,u,EC4}$ Knm	$M_{j,Exp}$ Knm	$\dfrac{S_{j,Exp}}{S_{j,EC4}}$	$\dfrac{M_{j,u,Exp}}{M_{j,EC4}}$
IJT-01	2410	1590	49.4	55.8	0.66	1.13
IJT-04	2430	1790	49.9	65.4	0.74	1.31

4. Conclusions

The composite connection of CFS with slip in gusset plate was predicted by parametric analysis and compared with experimental results. Two specimens with and without seat angle were utilized and the test results exhibit a nonlinear behavior before collapse. The prediction of the object was developed by means of the component method; the moment capacity results were shown lower than full-scale test and shown satisfactory results with the ratio 1.3. However, the stiffer connection was produced compared with the experimental; the ratio is between 0.6-0.8. The use of seat angle is one of the important elements that influence to the performance of the connection. The increase of the moment resistance and joint stiffness from both two methods of angle stiffeners can be used as a reinforcing element in composite connection where no welding work is needed in the compression zone.

Acknowledgment

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